Chapter A-7

ARMOURDALE UNIT PUMP STATION ANALYSIS

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CHAPTER A-7
ARMOURDALE UNIT
PUMP STATION ANALYSIS

A-7.1 SOURCES OF INFORMATION

Corps staff visually inspected every pump station studied, including outlet works when accessible. Printed information such as pump curves or as-built drawings was unavailable or limited for several of the facilities.

TABLE A-7.1
Summary of Pump Station Available Information

Name	Station	As-Builts?	Pump Curves?
Armourdale Unit Pump Stations:			
Osage (KCK #14)	76+83	Yes	Yes
12th St (KCK #13)	129+20	Partial	No
Mill St (KCK #12)	156+75	Yes	No
5th St (KCK #11)	185+70	Yes	No
MW Cold Storage	194+60	No	No
Shawnee Ave (KCK #10)	230+78	Yes	No
KCS RR	276+79	Yes	Yes
PBI Gordon	286+59	No	Yes
Nat'l Beef (KCK #8)	295+52	No	No
Central Ave	299+21	Yes	Yes

A-7.2 SUPPORTING INFORMATION AND CALCULATIONS

Supporting information for the pump station analysis is presented following the pump station write-up:

Exhibit 1 – Photographs
Exhibit 2 – Hydraulic Calculations
Exhibit 3 – Geotechnical Data
Exhibit 4 – Structural Calculations

A-7.3 METHODS OF ANALYSIS

Pump stations were evaluated both for structural reliability and, in the case of stations handling seepage flow from relief wells discharging below grade, hydraulic capacity. Detailed analyses were conducted for existing conditions in an effort to define the risk of failure under current project conditions. Detailed analyses were also conducted for the n500+3 design loading to determine the required modifications to

accommodate the higher loading associated with a prospective levee or floodwall raise, consistent with the recommended levee raise across the Kansas River on the Armourdale Unit (Phase 1). Two other design conditions, n500+0 and n500+5 were also considered, but primarily by engineering judgment rather than detailed analysis. The purposed of the two additional design points was to better define where the various design conditions fall on the net benefits curve, relative to the NED plan.

Structural analysis considered both strength and uplift (also called "stability" or "flotation" throughout the report) with water to the top of the existing line of protection. These analyses were applied to the pump station structures as well as outlet works (pipes, RCBs and gatewells). In some cases, as-built drawings and specifications were available, few assumptions had to be made, and the analysis was rather clear. In other cases, however, details of the original construction were not available and assumptions had to be made based on standard practice at the time of construction (sometimes as early as pre-1914) and previous findings on similar structures. Conclusions were drawn based on those assumptions, which were in some cases tagged for verification during PED. Initial strength and uplift analyses were based upon geotechnical parameters applied across the levee unit and hydraulic grade lines (HGLs) with water to the top of the existing line of protection. Initial calculations neglected the affects of skin friction, water that would be present in the pump station under normal operating conditions, and the affect of existing relief wells (where present) to lower HGLs. When such analysis showed "marginal" results, the analysis was refined, considering all of the available forces (i.e. skin friction, wells, water weight) to resist failure. In those cases where pump stations have separate gatewells within the line of protection, the gatewell analysis is presented in the general structural portion of the feasibility report (along with drainage structures not associated with pump plants).

Strength analysis entailed examination of walls and slabs for shear and bending moment capacity. If the resulting FS for flexure and shear was 1.5 or greater, the member was considered to perform to 99.8% reliability when exposed to the existing condition event loading. If the resulting FS was less than 1.5, a reliability analysis was conducted.

Uplift analysis entailed examination of structures for flotation with water to the top of the existing line of protection. Several stations had subcomponents such as gatewells and discharge chambers that were constructed at different times. In those cases, the uplift analysis considered the subcomponents separately, and the recommended modification considered the ability of the subcomponents to be tied together to transfer forces. If the resulting FS was 1.1 or greater, the structure was considered to perform reliably.

Screening level hydrologic analyses were conducted at the beginning of the feasibility phase for interior drainage. Records and drainage studies by others were reviewed, and local Sponsors consulted. Since the focus of this study is an existing system and there were no interior drainage problems identified or reported by Sponsors, more detailed interior flood hydrology studies were not undertaken. It is also worth

noting that there are several interior storm drainage systems that handle upland and lowland flow, ultimately draining to pump stations. In some cases, the total flow generated by the storm drainage system exceeds the capacity of the lowland system, including the pump station. In other words, if the drainage systems were improved such that the flow could travel unimpeded, the pump stations would be overloaded. Research and discussion with Sponsors indicates that this situation has existed for many years, and no improvements are planned. Therefore, no pump station improvements were considered based solely upon interior flood hydrology.

Several pump stations in the system, however, handle seepage flow from well systems (relief wells / header systems discharging below grade). If this flow is not pumped, the factor of safety of the line of protection will be compromised. In these cases, existing condition analysis verified the relief well flow/coincident storm and pump station capacity to handle the flow, with the required redundancy. Alternatives analysis was tied closely to geotechnical requirements for underseepage control (i.e. berms, cutoff trench, or relief wells). Care was taken to ensure that alternatives calling for additional relief wells considered all resultant costs, including pumping costs.

A-7.4 FINDINGS AND RECOMMENDATIONS

The analysis of the pump stations along the Armourdale line of protection resulted in the findings and recommendations below. Existing condition results and n500+3 design condition recommendations are also presented in the exhibits following paragraph A-7.6 and are also summarized in the tables in paragraph A-7.5. Required modifications to Osage, 12th St Sewer, Mill St, 5th St, KCS RR, National Beef and Central Ave are cost shared as part of the proposed Federal project. Required modifications to private pump stations with no formal easement (Midwest Cold Storage, PBI Gordon) are the responsibility of the owners.

A-7.4.1 Osage Pump Station (KCK #14)

The Osage Pump Station handles interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade). The Sponsor reported no problems with the hydraulic capacity of this pump plant. Hydraulic analyses were conducted for the purposes of evaluating alternatives involving additional relief well flow discharging below grade. These alternatives included a replacement pump station, adding a new pump station, and including a slurry cutoff wall. The hydraulic calculations are included for all these alternatives in Exhibit 2, Part 2, for reference and comparison. The final recommended levee section in this area is the slurry cutoff wall, negating the need for relief wells and consideration of handling additional flows at this via a replacement or additional pump station.

The structural evaluation indicates that the Osage pump station is marginally acceptable for flotation with water to the top of the existing line of protection. For any line of protection raise, addition of weight or control of subsurface hydrostatic pressure is required to alleviate flotation issues. A slurry cutoff wall is proposed in this area as part

of the recommended n500+3 design and provides sufficient reduction in hydrostatic pressure for the pump station to meet flotation criteria. Strength of the pump station is adequate, both for existing conditions and for levee raises up to and including n500+3, given the planned slurry cutoff wall.

The existing discharge configuration is a chamber within the pump station that receives pumped flow, pushing it through the gravity discharge line and to the river (i.e. the gatewell is within the pump station; not a separate structure in the levee crest). Strength of the discharge pipe was found to be marginal under existing conditions, and therefore inadequate for additional loading. N500+3 design conditions require minor work to address erosion in the invert, plus a structural liner (i.e. CIPP or steel slip liner). For any line of protection raise, the top of the discharge chamber will need to be raised to coincide with the new line of protection. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem.

The pump station receives inflow from a brick sewer. There is no information regarding the age of the brick sewer, construction documents, or inspection documents. A flotation or strength analysis was not performed on the brick sewer. Inspection of the sewer by local entities to determine the condition of the sewer is recommended.

The Osage pump station was designed and constructed as part of the original Corps of Engineers flood control project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required. The recommended section in this area is a flood wall. There are no earth fill impacts on the pump station structure (i.e. no additional earth fill against the pump station).

N500+0 design conditions would include a cutoff wall to address underseepage concerns in the area, alleviating any flotation issues. No structural modifications of the pump plant structure would be required. Strength of the discharge pipe would be inadequate and still require a structural liner. The top of the discharge chamber would need to be raised to coincide with the new n500+0 level of protection.

N500+5 design conditions would include a cutoff wall to address underseepage concerns in the area, alleviating any flotation issues. No structural modifications of the pump plant structure would be required. Strength of the discharge pipe would be inadequate and still require a structural liner. The top of the discharge chamber would need to be raised to coincide with the new n500+0 level of protection.

A-7.4.2 12th Street Pump Station

The 12th St Pump Station handles interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade). Structural evaluations indicate that the 12th St Pump Station is unacceptable for both flotation and strength with water to the top of the existing line of protection. For any line of protection

raise, addition of weight or control of subsurface pressure is required to alleviate flotation issues. Structural modifications are required to meet strength criteria. Calculations show that this pump station may be strengthened with a brace/strut system, and additional weight added at key locations to meet criteria for strength and flotation. To ensure adequate transfer of uplift forces, the gatewell may require anchoring to the pump station.

The existing pump discharge configuration is twin ductile iron pump discharge lines "up and over" the levee, discharging into a gatewell on the riverside levee crest. Strength of the pump discharge lines was not evaluated due to the "up and over" configuration. The existing gravity discharge is a reinforced concrete box. Strength of the RCB is adequate, both for existing conditions and for the n500+3 design condition. Strength and stability of the gatewell is adequate, both for existing conditions and for the n500+3 design condition. For any line of protection raise, the top of the gatewell will need to be raised to coincide with the new line of protection. Raising the gatewell will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of a new stem. The twin "up and over" discharge lines will need to be relocated up and over the new, higher level of protection.

The pump station receives inflow from a brick sewer. There is no information regarding the age of the brick sewer, construction documents, or inspection documents. A flotation or strength analysis was not performed on the brick sewer. Inspection of the sewer by local entities to determine the condition of the sewer is recommended.

The 12th Street pump station was designed and constructed prior to Corps of Engineers involvement in the flood control project. Though drawings were limited, the recommended remedy entails extensive structural re-work, including extensive brace and strut systems and addition of weight to achieve an adequate factor of safety against uplift forces. While such testing as concrete coring is not required during the next project phase, further examination will be required during PED to assess the transfer of relative uplift forces acting on the pump station structural components.

The recommended section in this area is a T-wall on existing levee, resulting in 3-5 feet of additional feel against the existing pump station structure. Preliminary consideration indicates that the pump station can safely withstand the increased lateral earth load. A short retaining wall may be considered during PED if required.

N500+0 design conditions would result in loadings exceeding the existing condition. Since strength and uplift were unacceptable for existing conditions, major modifications similar to those required for a n500+3 raise would also be required for the n500+0 condition. Strength of the existing RCB is acceptable; no modifications would be required. Strength and uplift for the gatewell are acceptable for the n500+0 condition, however the top of the gatewell would need to be raised to coincide with the new n500+0 level of protection. The twin "up and over" discharge lines would need to be relocated up and over the new, higher level of protection.

N500+5 design conditions would result in loadings in excess of what the existing structure and the separate gatewell could withstand with any level of modification.

Complete replacement of the pump station and gatewell would be required. Though the strength of the existing RCB is acceptable at the n500+5 condition, the required new pump station would likely be reconfigured in conformance with current recommended design practices (i.e. integral with the line of protection), and with a completely different discharge configuration. Therefore, it is likely that the existing RCB would also be replaced.

A-7.4.3 Mill Street Pump Station (KCK #12)

The Mill St Pump Station handles interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade). Structural evaluations indicate that the Mill St Pump Station is unacceptable for both flotation and strength with water to the top of the existing line of protection. For any line of protection raise, addition of weight or control of subsurface pressure is required to alleviate flotation issues. Structural modifications are required to meet strength criteria. Calculations show that this pump station may be strengthened with a brace / strut system, and additional weight added at key locations to meet criteria for strength and flotation. To ensure adequate transfer of uplift forces, the gatewell may require anchoring to the pump station.

The existing pump discharge configuration is twin ductile iron pump discharge lines “up and over” the levee, discharging into a gatewell on the riverside levee crest. Strength of the pump discharge lines was not evaluated due to the “up and over” configuration. The existing gravity discharge is a reinforced concrete box. Strength of the RCB is adequate, both for existing conditions and for the n500+3 design condition. Strength and stability of the gatewell is adequate, both for existing conditions and for the n500+3 design condition. For any line of protection raise, the top of the gatewell will need to be raised to coincide with the new line of protection. Raising the gatewell will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem. One pump will also need to be upgraded to maintain original design discharges, but at the new, higher head that will result from any proposed levee raise. The twin “up and over” discharge lines will need to be relocated up and over the new, higher level of protection.

The pump station receives inflow from a brick sewer. There is no information regarding the age of the brick sewer, construction documents, or inspection documents. A flotation or strength analysis was not performed on the brick sewer. Inspection of the sewer by local entities to determine the condition of the sewer is recommended.

The Mill Street pump station was designed and constructed prior to Corps of Engineers involvement in the flood control project. Though drawings were limited, the recommended remedy entails extensive structural re-work, including extensive brace and strut systems and addition of weight to achieve an adequate factor of safety against uplift forces. While such testing as concrete coring is not required during the next project phase, further examination will be required during PED to assess the transfer of relative uplift forces acting on the pump station structural components.

The recommended section in this area is a T-wall on existing levee, resulting in 3-5 feet of additional feel against the existing pump station structure. Preliminary consideration indicates that the pump station can safely withstand the increased lateral earth load. A short retaining wall may be considered during PED if required.

N500+0 design conditions would result in loadings exceeding the existing condition. Since strength and uplift were unacceptable for existing conditions, major modifications similar to those required for a n500+3 raise would also be required for the n500+0 condition. Strength of the existing RCB is acceptable; no modifications would be required. Strength and uplift for the gatewell are acceptable for the n500+0 condition, however the top of the gatewell would need to be raised to coincide with the new n500+0 level of protection. The twin "up and over" discharge lines will need to be relocated up and over the new, higher level of protection.

N500+5 design conditions would result in loadings in excess of what the existing structure and the separate gatewell could withstand with any level of modification. Complete replacement of the pump station and gatewell would be required. Though the strength of the existing RCB is acceptable at the n500+5 condition, the required new pump station would likely be reconfigured in conformance with current recommended design practices (i.e. integral with the line of protection), and with a completely different discharge configuration. Therefore, it is likely that the existing RCB would also be replaced.

A-7.4.4 5th Street Pump Station

The 5th St Pump Station handles interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade). Structural evaluations indicate that the 5th Street Pump Station is unacceptable for flotation, both for water to the top of the existing levee and for the n500+3 design condition. For a line of protection raise, control of or resistance to subsurface hydrostatic pressure is required to alleviate flotation issues. Features such as a slurry cutoff wall or relief wells may be used, or structural modifications such as base slab modifications or work anchorage may be considered. Rock anchorage is considered to be a more feasible alternative, given the large base slab extensions that would be required to stabilize the structure against uplift forces. The rock anchorage alternative may require micropiles if the rock anchors prompt settlement below the structure. Strength of the pump station is acceptable for existing conditions. Minor modifications to the base slab are required for the n500+3 design condition.

The existing discharge configuration is a chamber within the pump station that receives pumped flow, pushing it through the gravity discharge line and to the river (i.e. the gatewell is within the pump station; not a separate structure in the levee crest). The discharge line was not accessible for inspection. As-built drawings show the discharge line to be a reinforced concrete pressure pipe, however the type, class, bedding and associated load bearing capacity are unknown. Based on information supplied by Price Brothers, it is assumed that the pressure pipe is AWWA C302. This assumption is based

upon records of pipe shipments to the area during construction of the 5th St. pump station. For purposes of adequately capturing potential costs, a structural liner is considered for feasibility-level analysis. Further investigations will be conducted during PED to more accurately determine the load carrying capacity of the pipe. Further inspection consists of performing an annual visual inspection of the pipe. This annual inspection should occur over a consecutive ten year period. This is necessary to monitor how the pipe behaves after addition of the levee fill material. If pipe distress is found, a structural liner should be installed. Any line of protection raise will require raising the top of the discharge chamber to coincide with the new line of protection. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem.

The as-built drawings show the pump station inlet pipe as being constructed of the same material as the aforementioned outlet pipe. The recommendations made for the outlet pipe also apply to the inlet pipe.

The 5th Street pump station was designed and constructed by local interests to Corps criteria. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not recommended, except as noted above for the outlet and inlet pipes.

The recommended section in this area is a flood wall. There are no earth fill impacts on the pump station structure (i.e. no additional earth fill against the pump station).

N500+0 design conditions would result in loadings exceeding the existing condition. Since uplift was unacceptable for existing conditions, modifications similar to those required for a n500+3 raise would also be required for the n500+0 condition. Strength of the existing outlet pipe may be acceptable, but would need to be verified during PED since engineers were unable to determine the exact type of pipe used. The top of the discharge chamber would need to be raised to coincide with the new n500+0 level of protection.

N500+5 design conditions would also result in loadings exceeding the existing condition. Since uplift was unacceptable for existing conditions, modifications similar to those required for a n500+3 raise but designed for higher loading would also be required for the n500+5 condition. Strength of the structure was unacceptable for the floor slab under n500+3 conditions, requiring minor modification, and will be unacceptable under the higher loading condition of n500+5. Wall strengths were marginally acceptable for n500+3. Strength of the existing outlet pipe may be acceptable, but would need to be verified during PED since engineers were unable to determine the exact type of pipe used. The top of the discharge chamber would need to be raised to coincide with the new n500+5 level of protection. Since strength of the main pump station structure under less stringent n500+3 conditions is either inadequate (base slab) or marginal (walls), since the uplift problem is severe even under existing conditions, and since solving uplift problems requires anchorage to the structure (intimately related to strength), one may conclude that

this pump plant and related outlet works would require complete replacement for n500+5 loading.

A-7.4.5 Midwest Cold Storage

The Midwest Storage Pump Station is a small privately owned sump and abandoned mechanical equipment located inside the Midwest Cold Storage warehouse, adjacent to the levee. The pump station was previously used to pump out condensate which accumulated as a result of the refrigeration equipment. The refrigeration equipment was upgraded some time ago, and the pump station was not used since. Preliminary plans considered the abandonment of the sump in place; however the proposed floodwall section in this area (which includes a line of relief wells) addresses uplift concerns at this location, negating the need for further analysis.

Both n500+0 and n500+5 design conditions would likely result in a floodwall section with wells in this area, and similar recommendations regarding Midwest Cold Storage (i.e. no modifications required – proposed section addresses potential uplift concerns).

A-7.4.6 Shawnee Avenue Pump Station (KCK #10)

The Shawnee Avenue Pump Station handles both interior drainage and relief well flows discharging below grade, through a collector system to the pump plant. ***Under current conditions, pumping of the relief well flow is required during high river stage events to maintain the integrity of the line of protection along the relief well system***.

Hydraulic analysis for the pump station under existing conditions with water to the top of the existing line of protection (i.e. maximum relief well flow of 35cfs with 15.5cfs small coincident storm) shows excess capacity.

Under higher head conditions of n500+3, the existing line of wells will produce approximately 47cfs. Additional surface discharging relief wells producing a total of 40cfs are also recommended for the n500+3 design condition. Of this 40cfs which would be generated at maximum flow conditions (water to top of n500+3 levee), roughly half of the flow, or 20cfs, would be discharged within the service area of the Shawnee Avenue Pump Plant. The coincident storm produces 28cfs, so the total flow that must be pumped in order to maintain the below-grade discharge relief wells (thereby maintaining the factor of safety) is 47+20+28=95cfs. The pump station capacity at maximum n500+3 river stage ranges from 95 to 55 cfs, depending on water surface elevation at the intake, and is therefore inadequate. There is ample room within the existing pump station structure to install larger pumps to accommodate the n500+3 design flows.

The structural evaluation indicates that the Shawnee Avenue pump station is adequate for strength and flotation with water to the top of the existing line of protection, as well as for levee raises up to and including n500+3.

The existing discharge configuration is a chamber within the pump station that receives pumped flow, pushing it through the gravity discharge line and to the river (i.e. the gatewell is within the pump station; not a separate structure in the levee crest). Strength of the discharge RCB was found to be marginal under existing conditions, and therefore inadequate for additional loading. N500+3 design conditions require a structural liner. For any line of protection raise, the top of the discharge chamber will need to be raised to coincide with the new line of protection. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem.

The inlet pipe consists of a 96" diameter RCP. The inlet pipe, upstream junction box and pump station design are shown on the same record drawings. The class of RCP used is not specified in the drawings. Given that pipe bedding conditions tend to improve over time, the project team believes this segment of pipe is adequate for the n500+3 raise. It is recommended, however, that local interests perform a visual inspection of the pipe prior to construction and annually for the following ten years upon completion of the project.

The Shawnee Avenue pump station was designed and constructed by local interests to Corps criteria. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not recommended.

The recommended section in this area is a T-wall on existing levee. Since the Shawnee Avenue pump plant is essentially constructed integral to the line of protection, the T-Wall would be constructed around the pump plant.

The pump station structure is adequate for both n500+0 and n500+5 design conditions. No structural modifications of the pump plant structure would be required. Strength of the discharge RCB would be inadequate for either condition would require a structural liner. The top of the discharge chamber would need to be raised to coincide with the new n500+0 or n500+5 level of protection.

A-7.4.7 KC Southern Railroad Pump Station

The KCSouthern RR Pump Station handles both interior drainage and relief well flow which discharges at grade and flows overland to the pump station. Under existing conditions, the pump station may not be operated (i.e. ponded water from the relief well discharge may not be pumped) when the river is rising and passes a certain stage in an effort to add weight, thus maintaining the factor of safety for the line of protection in this area.

The n500+3 recommendation for this area entails filling of the "slot", the low lying area to which the relief wells flow, and abandonment of the wells. Although the pump station capacity is very low and will no longer be required for seepage flows, the pump station is recommended to remain in place to handle the remaining interior drainage flow.

The structural evaluation indicates that the KCS RR pump station is adequate for strength and flotation with water to the top of the existing line of protection, as well as for levee raises up to and including n500+3.

The existing discharge configuration is a chamber within the pump station that receives pumped flow, pushing it through the gravity discharge line and to the river (i.e. the gatewell is within the pump station; not a separate structure in the levee crest). The discharge line is cast iron, reportedly with leadite joints. Although detailed inspections of the discharge line were not possible, leadite has a history of poor performance, and unprotected ferrous pipe would likely experience corrosion. For these reasons, the discharge line replacement of the discharge line is recommended for any raise. For any line of protection raise, the top of the discharge chamber will need to be raised to coincide with the new line of protection. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem.

According to available drawings, the pump station receives inflow from a 24" diameter corrugated metal pipe (CMP). A flotation or strength analysis was not performed on the CMP. Inspection by local entities to determine the condition of the CMP is recommended.

The KCS RR pump station was designed and constructed by the Corps as part of the original Federal project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not recommended.

The recommended section in this area is a flood wall. There are no earth fill impacts on the pump station structure (i.e. no additional earth fill against the pump station).

The pump station structure is adequate for both n500+0 and n500+5 design conditions. No structural modifications of the pump plant structure would be required. The discharge pipe would be inadequate for either condition would require replacement. The top of the discharge chamber would need to be raised to coincide with the new n500+0 or n500+5 level of protection.

A-7.4.8 PBI Gordon

PBI Gordon is a private business that routes all of its parking lot runoff to a small pump station, which then pumps directly to a gravity drainage pipe riverward of a gatewell structure, where it drains to the river. The sluice gate in this gatewell is permanently closed, allowing all runoff to be routed to the sump and sampled prior to pumping to the river.

The pump station is inadequate for flotation with water to the top of the existing line of protection, as well as for any raise. Based on the available information, the PBI

Gordon pump plant can remain in place and perform reliably for the n500+3 event by adding weight inside the structure. To ensure adequate transfer of the uplift loads, anchoring of the gatewell to the pump pit may be necessary. Strength evaluation of the structure was not possible since drawings were not available. However addition of fill material will inherently strengthen the structure, and bracing / struts may be added as necessary to achieve the required strength.

The gatewell and gravity drainage pipe is recommended for abandonment, since its condition is unknown and all flow is pumped. The pump station discharge line will be routed up and over the new line of protection.

The pump station receives inflow from a 24" diameter vitrified clay pipe (VCP – file A-10-1994). There is no information regarding the material, age of the pipe, construction documents, or inspection documents. A flotation or strength analysis was not performed on the pipe. Inspection of the sewer by local entities to determine the condition of the pipe is recommended.

The age of the pump station is unknown. Though drawings were unavailable, the recommended remedy entails addition of significant amounts of weight. While such testing as concrete coring is not required during the next project phase, further examination will be required during PED to assess the transfer of uplift load between the adjacent gatewell (to be abandoned) and the pump station structure.

The recommended section in this area is a flood wall. There are no earth fill impacts on the pump station structure (i.e. no additional earth fill against the pump station).

The pump station structure could be modified for either n500+0 or n500+5 design conditions by adjusting the weight and bracing commensurate with the design loading. The discharge pipe would be routed "up & over" the new line of protection.

A-7.4.9 National Beef (KCK #8)

The National Beef pump station was originally designed to handle interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade) from a small drainage area. The businesses for which the pump station was needed have since disappeared. While the pump does run and the station is officially considered to be still in service, preliminary investigations show that it is no longer needed, and that the pump station may be abandoned in place for the n500+3 raise, as well as for n500+0 or 500+5.

A-7.4.10 Central Avenue Pump Station.

The Central Avenue pump station handles only relief well flows discharging below grade, through a collector system to the pump plant. ***Under current conditions,***

pumping of the relief well flow is required during high river stage events to maintain the integrity of the line of protection along the relief well system.

Structural evaluations indicate that the Central Avenue pump station is acceptable for flotation with water to the top of the existing line of protection. For any line of protection raise requiring continued service of the pump plant, control of subsurface hydrostatic pressure is required to alleviate flotation issues. Features such as a slurry cutoff wall or relief wells may be used, or alternately a structural modification such as base slab extensions or micropiles may me utilized to meet flotation criteria. Strength of the pump station is adequate for existing conditions, though modifications are required for levee raises up to and including n500+3.

This station services an existing relief well system (no interior drainage) and has no excess capacity. Geotechnical investigations in the area show that, under n500+3 design conditions, underseepage may be controlled with relief wells discharging to the surface (i.e. relief well flow would *not* need to be pumped) and draining to a nearby low area, as shown in Exhibit 2 . Portable pumping could be provided at the discretion of the Sponsor, and the Central Avenue pump plant could then be abandoned. The proposed ponding area is currently occupied by a local business and used as a storage area. Real estate considerations are currently underway, however it is anticipated that the business could continue to use the ponding area under non-flooding conditions. Under flooding conditions, the wells would start flowing at a very low rate once the river stage reached the elevation of the well discharges (several feet above bank full). As river stages reach the top of the line of protection, the wells would reach a maximum combined flow rate of roughly 22 cfs.

N500+0 or n500+5 design conditions would be accommodated by a similar section and line of new surface-discharging relief wells / ponding area. Therefore, the pump station would not be required under either design condition.

A-7.5 SUMMARY OF RESULTS

See the following pages for tabulated summary of results.

Table A-7.5-1 Summary of Existing Conditions Results

Revision Date: 6/24/2013

EXISTING CONDITIONS CID PUMP STATION ANALYSIS RESULTS

Pump Station	Sta.	Minimum Shear Wall F.S.(W1,W2)	Minimum Wall Moment F.S.(W1/W2)	Minimum Base Slab Shear F.S	Minimum Base Slab Moment F.S	Stability (Flotation)	Outlet Pipe Reliability	Comments
Osage (KCK #14)	76+83	ok	ok	ok	ok	marginal	marginal	
12th St (KCK #13)	129+20	0.54	0.36	1.29	0.60	1.07	RCB is reliable	
Mill St (KCK #12)	156+75	0.71	0.43	1.4	0.63	1.1	RCB is reliable	
5th St (KCK #11)	185+70	1.5	1.5	2.17	1.5	0.88	marginal	
MW Cold Storage	194+60	privately owned sump and abandoned mech equipment, see write-up						
Shawnee Ave (KCK #10)	230+78	ok	ok	ok	ok	ok	marginal	
KCS RR	276+79	ok	ok	ok	ok	ok	no good	
PBI Gordon	286+59	no drawings available - no analysis performed				0.60	no good	
Nat'l Beef (KCK #8)	295+52	can be abandoned, see write-up						
Central Ave	299+21	ok	ok	ok	ok	ok	ok	

Strength: 1.5 FS required per established criteria
Stability: 1.1 FS required per established criteria

Green - FS equal to or greater than 1.5 for strength and greater than or equal to 1.10 for stability
Yellow - FS between 1.3 and 1.5 for strength and/or between 0.96 and 1.09 for stability
Red - FS <1.3 and/or FS < 0.95 for stability

Table A-7.5-2 Summary of N500+3 Conditions Results and Recommendations

Revision Date: 6/24/2013

FUTURE CONDITIONS (N500+3) CID PUMP STATION ANALYSIS RESULTS

Pump Station	Sta.	Minimum Shear Wall F.S.(W1,W2)	Minimum Wall Moment F.S.(W1/W2)	Minimum Base Slab Shear F.S	Minimum Base Slab Moment F.S	Stability (Flotation)	Outlet Pipe Reliability	Comments
Osage (KCK #14)	76+83	1.15	1.27	1.63	2.74	0.84	likely to require rehab due to loading	structural modifications necessary for uplift - cut off wall, raise discharge chamber
12th St (KCK #13)	129+20	0.5	0.34	1.18	0.55	1.07	RCB is reliable	Major structural modifications. Brace and shore for strength, add weight for stability
Mill St (KCK #12)	156+75	0.67	0.41	1.29	0.58	1.1	RCB is reliable	Major structural modifications. Brace and shore for strength, add weight for stability
5th St (KCK #11)	185+70	1.49	1.5	1.99	1.4	0.74	RCP will require rehab	Minor strength mod required, major stability mod required
MW Cold Storage	194+60	privately owned sump and abandoned mech equipment, remains and no need for structural mod						
Shawnee Ave (KCK #10)	230+78	1.5	1.5	1.44	1.5	1.08	RCB undergo rehab prior to placing additional fill over it	No major structural mod required.
KCS RR	276+79	1.54	1.52	4.68	2.01	1.208	42-inch CIP will likely require rehab.	No major structural mod required.
PBI Gordon	286+59	no drawings available - no analysis performed				0.67	all flow is pumped over line of protection- recommend abaondonment	To remain. Bracing/struts may need to be added to achieve required strength
Nat'l Beef (KCK #8)	295+52	Abandon in place, allow ponding, temporary pumping						
Central Ave	299+21	1.61	1.5	1.5	1.34	0.908		Abandon in place, install new line of relief wells, allow ponding, temporary pumping

Red=Rehabilitation Required
Green - FS equal to or greater than 1.5 for strength and greater than or equal to 1.10 for stability
Yellow - FS between 1.3 and 1.5 for strength and/or between 0.96 and 1.09 for stability
Strength: 1.5 FS required per established criteria for shear and moment
Stability: 1.1 FS required per established criteria

FC=n500+3 Future Condition Event
FS=Factor of Safety, n500+3 event

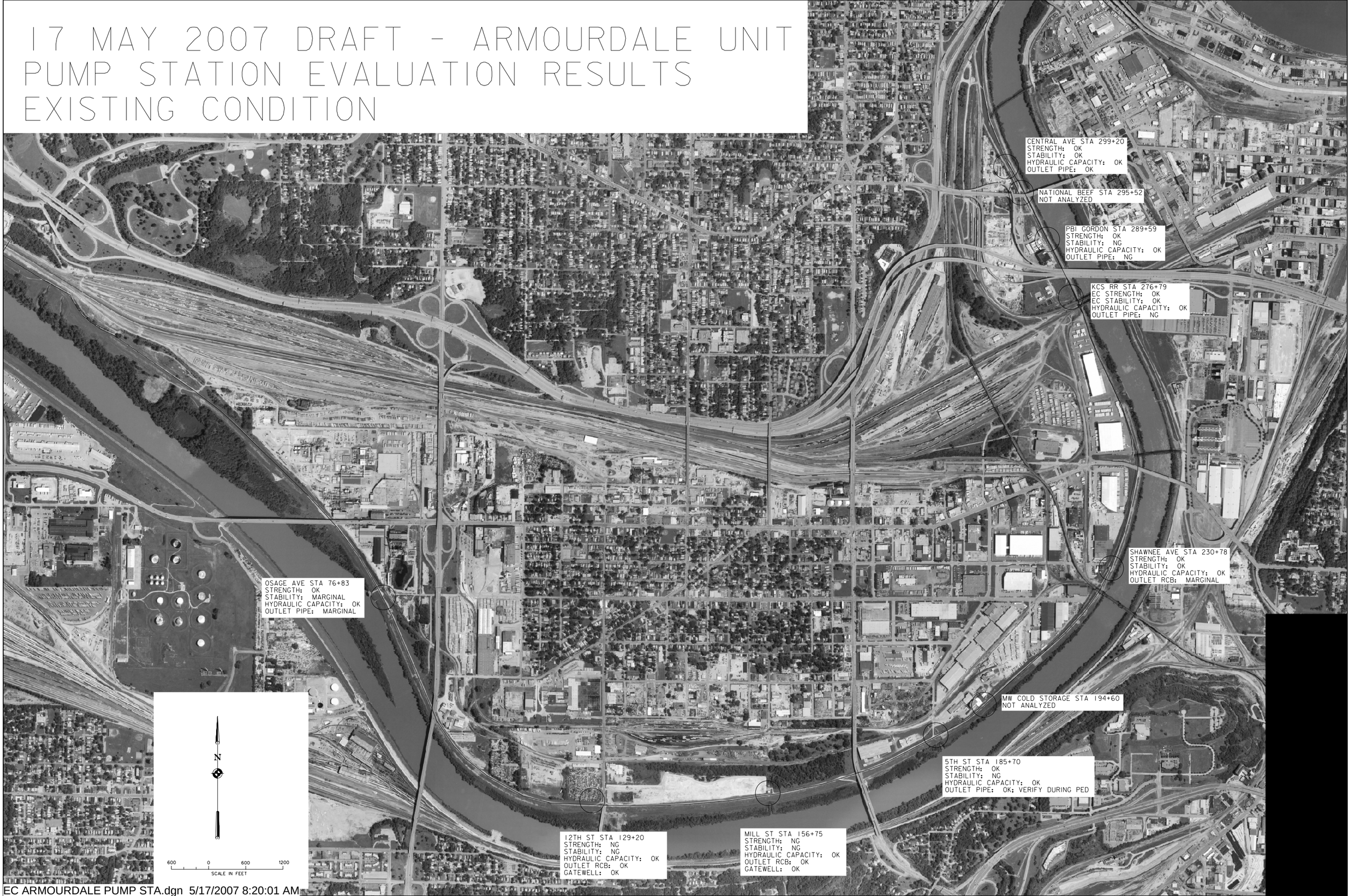
A-7.6 RELIABILITIES

Probability of failure curves for 12th Street, Mill Street and 5th Street pump stations were used in the economic model. See Figure A-7.4.

Calculations for the 12th Street and Mill Street pump stations revealed a 100% probability of failure with water at top of levee, driven primarily by inadequate strength of key structural members. Though both pump stations also show a very high probability of failure with water at much lower levels, they performed adequately during the 1993 flood event. Therefore, a 25% probability of failure is considered reasonable for the 1993 flood elevations at these two pump stations.

Uplift calculations of the 5th Street pump station reveal a 100% probability of failure with water several feet below the existing top of levee. Probabilities of failure for lower water levels were calculated, and defined the curve which was used in the economic model.

17 MAY 2007 DRAFT - ARMOURDALE UNIT PUMP STATION EVALUATION RESULTS EXISTING CONDITION



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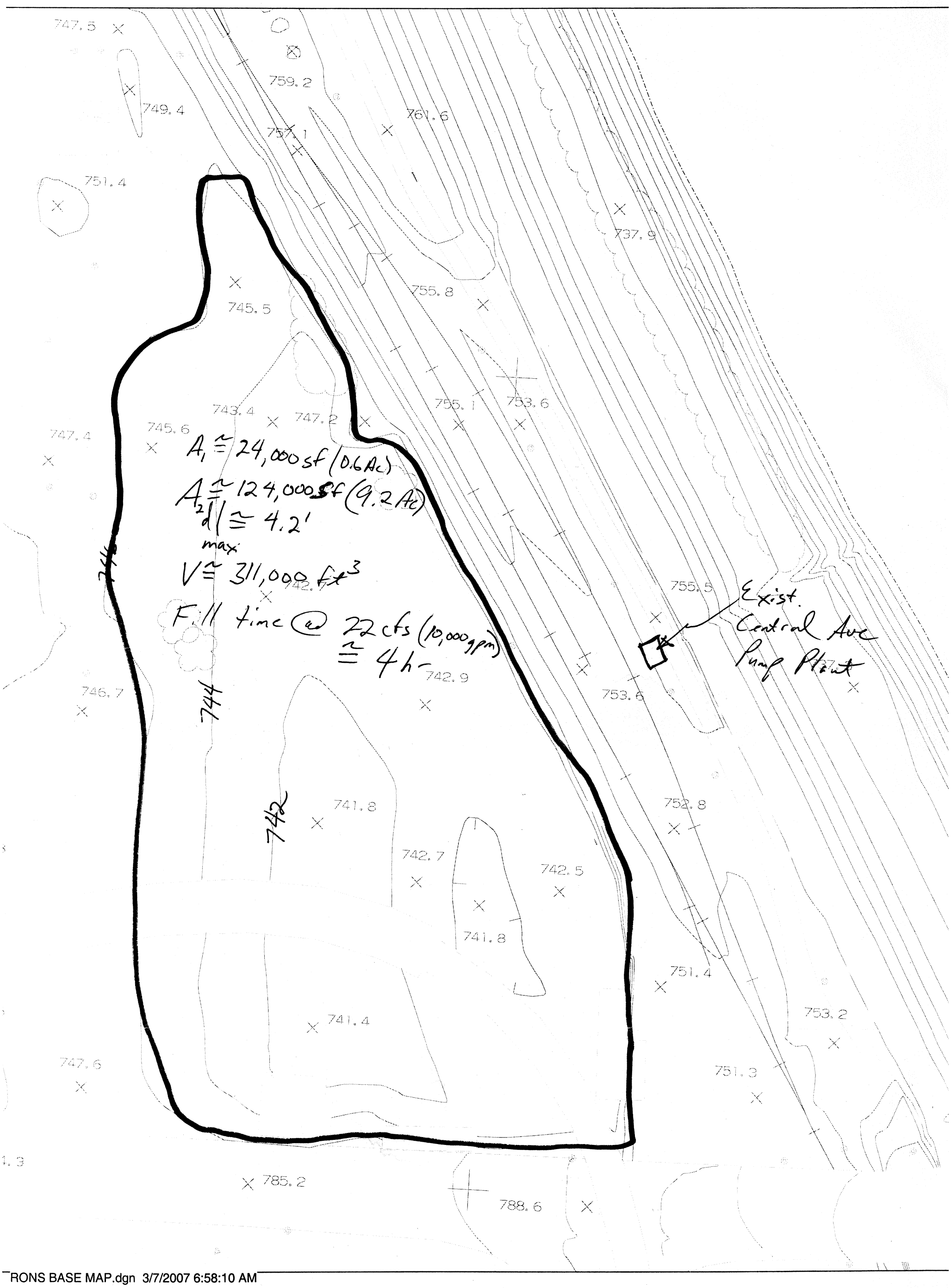
17 MAY 2007 DRAFT - ARMOURDALE UNIT PUMP STATION RECOMMENDED MODIFICATIONS N500+3 DESIGN CONDITION

N500+3 ARMOURDALE PUMP STA.dgn 5/17/2007 12:38:56 PM



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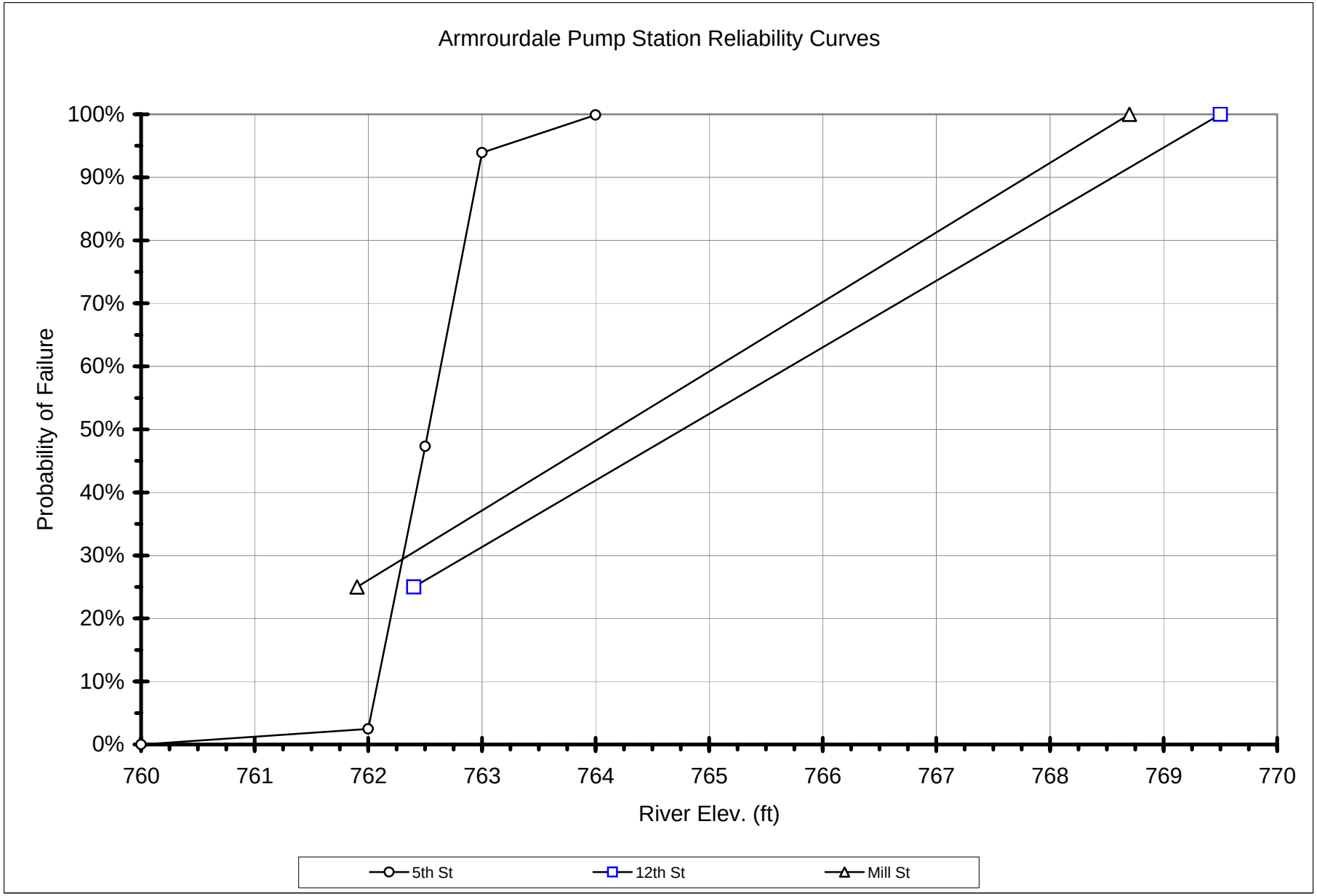

Armrourdale Pump Station Reliability Curves
Probability of Failure
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
760
761
762
763
764
765
766
767
768
769
770
River Elev. (ft)
5th St
12th St
Mill St

Osage Ave (KCK #14)

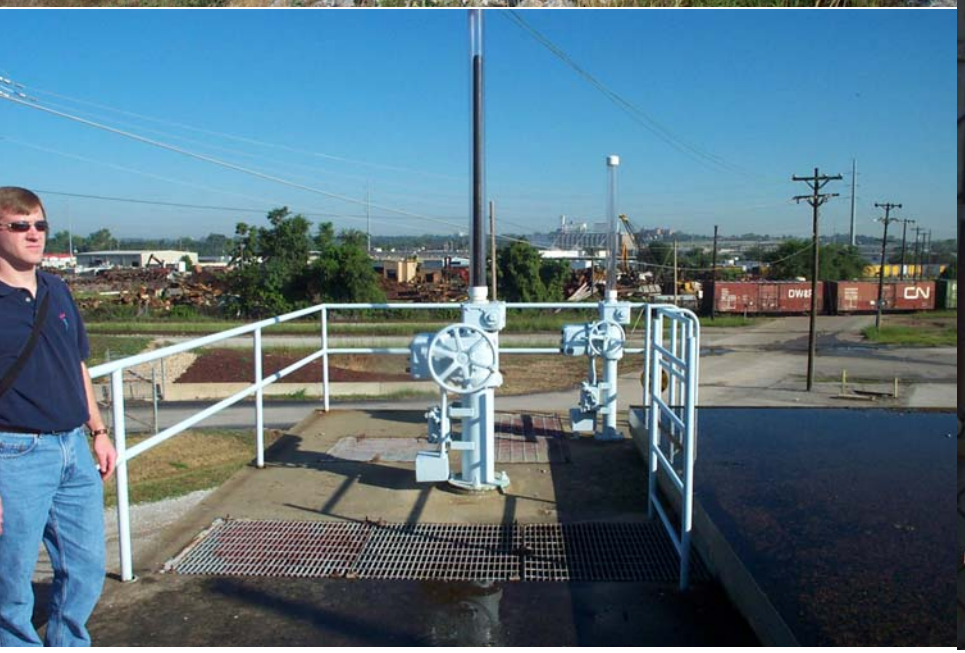
Osage Ave (KCK #14)

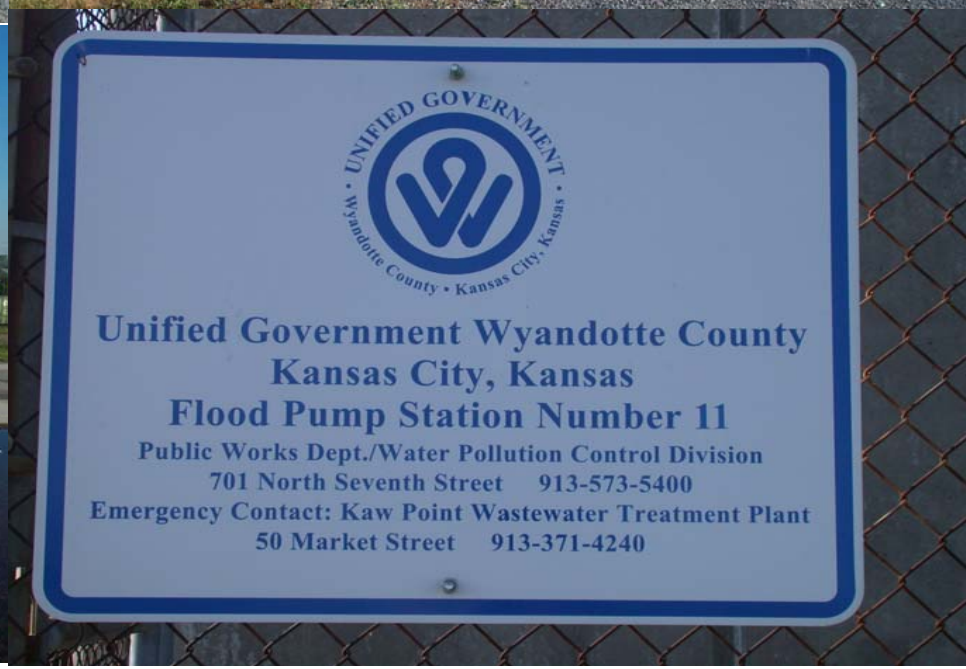
12th St (KCK #13)

12th St (KCK #13)

Mill St (KCK #12)

Mill St (KCK #12)




UNIFIED GOVERNMENT
Wyandotte County • Kansas City, Kansas
Unified Government Wyandotte County
Kansas City, Kansas
Flood Pump Station Number 11
Public Works Dept./Water Pollution Control Division
701 North Seventh Street 913-573-5400
Emergency Contact: Kaw Point Wastewater Treatment Plant
50 Market Street 913-371-4240

5th St (KCK #11)

5th St (KCK #11)

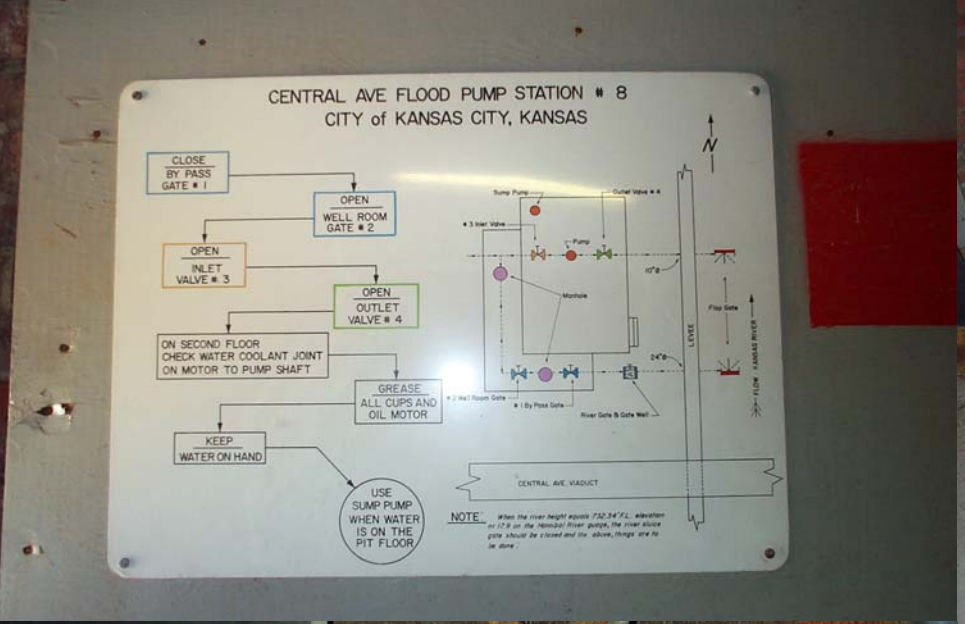
Shawnee Ave (KCK #10)

Shawnee Ave (KCK #10)

KCSouthern RR

KCSouthern RR

National Beef (KCK #8)



National Beef (KCK #8)

Central Avenue

Central Avenue

EXHIBIT 2, PART 1 - ARMOURDALE PUMP STATION INFORMATION

1. Osage (KCK FPS 14)

a. Year Constructed: 1953[12]
b. Owner/Operator: KCK
c. Levee Stationing: 76+83[1]
d. Distance From Flood Protection Centerline: 10ft[7]
e. Station Capacity: 11,650gpm[4]
f. Pumps:
   i. 16" 4,900gpm @40hp and 590 RPM[4]
   ii. 16" 4,900gpm @40hp and 590 RPM[4]
   iii. 6" 1,850gpm@15hp and 704 RPM[4]
g. Contributing Flows:
   i. Storm Sewer Flow from Prospect Park and the West Armourdale Sewer System, 344ac
h. River Discharge: 72" RCP
i. Discharge Arrangement: Gatewell at Pumping Plant & pressurized pipe under levee[7]
j. Drawing Availability:
   i. Date Unknown – Gatewell drawing [5]
   ii. Dec 1979 – General Plan & Profile [7]
   iii. Aug 1954 – Record Drawings (18 sheets) [7]
k. General
   i. Manhole #42 (see 1948 "Supplement on Interior Drainage", plate 14) separates the lowlands from the uplands. [3]
   ii. The excess runoff that bypasses the upland sewer system is stored in a large depression south of Muncie Boulevard. [3]
   iii. The flowline out at MH #42 is two feet higher than the critically low spots at the intersections of 18th Street and West Kansas Avenue and at 18th Street and Osage Avenue. Because of the 2 ft. of available head, the trunk sewer can discharge the hillside runoff through the 30-inch lateral sewers onto the intersections mentioned above. [3]
   iv. From field observations performed in 1948, flooding seemed to be more extreme at 18th Street and West Kansas Avenue. [3]
   v. Development has occurred in lower areas adjacent to the levee, increasing percent impervious area. This, in turn, has decreased the ponding area for seepage to collect. While the seepage flow was not designed to be collected and pumped, it may reach the pumping plant and decrease its pumping capacity. [3]
   vi. A pump was originally proposed to limit ponding in the area south of MH #42 so that, at a stage of 26.2 ft., the ponding will not be in excess of 0.158 in. This is the same ponding experienced at a stage of 14.0 ft. under gravity conditions. [3]
   vii. Pump plant located on/in earthen levee[4]
   viii. River Discharge shows degradation (scour hole) at conduit outlet. Hole approximately 1ft in depth and 1ft in width at largest extents. [9]

2. 12th Street Sewer (KCK FPS 13)

a. Year Constructed: N/A
b. Owner/Operator: KCK
c. Levee Stationing: 129+20[1]
d. Distance From Flood Protection Centerline: 80ft[5]
e. Station Capacity: 16,000gpm[1]
f. Pumps:
   i. 18" 8,000gpm @ 39.5TDH 125hp[1] 1185rpm[9]
   ii. 18" 8,000gpm @ 39.5TDH 125hp[1] 1185rpm[9]
   iii. Sump Pump[4]
g. Contributing Flows:
   i. Storm Sewer Flow 298ac of drainage[1]
   ii. Seepage Flow from Sta. 97+00 to Sta.129+20[1]
h. River Discharge: 5'x6' RCB[3]
i. Discharge Arrangement: Pressure pipe passing through levee (centerline @ 767.24) discharging to gatewell riverward side of levee.[5]
j. Drawing Availability:
   i. Jul 1995 – Mechanical Renovation [5]
   ii. Dec 1979 – General Plan & Profile [7]
k. General
   i. Seepage will collect in the low areas between the levee and the railroad track between stations 103+001 and 135+001, covering about 10 acres. Seepage will begin at about stage 32.8 and will reach a max flow of 3.2 cfs at design-flood stage (40.8).[3]
   ii. The ponding that is caused by the river tailwater which limits the capacity of the outfall is less then the ponding due to the insufficient capacity of the sewer during low river stages. In other words, the temporary ponding due to the higher river stages between 14 ft and 21.9 ft is less than the unavoidable ponding that occurs during lower stages from 14 ft and below. [3]
   iii. Pumping is started at stage 21.9', which corresponds to the ground elev. of the lowest manhole in the system. It is shown that the gravity flow capacity of the system is greater then the pump capacity at stage 21.9', but due to the high river stage resulting in backwater requires the gate to be close which effectively makes the sewer capacity zero. [3]
   iv. The COE, in the 1948 "Supplement on Interior Drainage", proposed no additional pumping capacity because the ponding due to the lack of pumping capacity is less then the ponding due to the sewer capacity. [3]
   v. The new pumps were installed in 1996. These two pumps were cited to have a capacity of 8000 gpm @ TDH 39.5 each. This capacity was compared to the original pumps at the same head and HNTB found that the original pump only had 6000 gpm @ TDH 39.5. This shows that the pump plant capacity has increased at the stages stated in the table. However, it is unknown how much the capacity has increased. [3]
   vi. Areas were originally determined to be undrained have been developed, thereby increasing the percent impervious. This increase could cause the peak flow to make the design pump capacity insufficient. [3]
   vii. Pump Station in close proximity to 12th street bridge, private auto salvage yard and overhead lines. [9]

3. Mill Street (KCK FPS 12)

a. Year Constructed: <1952[3]
b. Owner/Operator: KCK
c. Levee Stationing: 156+75[9]
d. Distance From Flood Protection Centerline: 30ft[7]
e. Station Capacity: 16,800gpm[4]
f. Pumps:
   i. 8,000gpm @ 39.5ft TDH[3]; 125hp @ 1185rpm[9]
   ii. 8,000gpm @ 39.5ft TDH[3]; 125hp @ 1185rpm[9]
   iii. Sump Pump[4]
g. Contributing Flows:
   i. Storm Sewer Flow 580ac[1]
   ii. Seepage Flow from Sta.129+20 to Sta. 173+00[1]
h. River Discharge: 6'x8' RCB[3]
i. Discharge Arrangement: Pressure pipe passing through levee (centerline @ 766.66) discharging to gatewell riverward side of levee.[5]
j. Drawing Availability:
   i. Jul 1995 – Mechanical Renovation [5]
   ii. Dec 1979 – General Plan & Profile [7]
   iii. Mar 1940 – Repair of pump house and floodgate [5]
k. General
   i. Seepage between stations 135+401 and 180+001 will begin at a stage of 32.8 ft. and reach a maximum flow of 4.5 cfs at the design flood stage (40.8 ft.). Seepage will pond in the depressed areas between the levee and the railroad tracks to the north. This area is drained by evaporation and infiltration. [3]
   ii. The present pump starting stage as indicated by the City of Kansas City, Kansas in 1948, is 14.9 ft. on the Hannibal Bridge gage. The pump operator actually uses that gage as a warning and the pumps are started when backflow in the sewer is about to pond in the intersection of Mill Street and Shawnee Avenue, which is equivalent to a stage of 19.4 ft. on the Hannibal Bridge gage. At this stage the sewer capacity and pump capacity are equal, therefore no additional pumping capacity was proposed. [3]
   iii. The Design Memorandum No. 3 written by the COE for the 1962 Modification states that the existing pump plant is outdated and is scheduled to be replaced by the city of Kansas City, Kansas. [3]
   iv. Overland flow cannot supplement sewer delivery to this outlet, due to a recently placed landfill adjacent to the levee. The landfill encroaches heavily on former ponding areas.[3]
   v. Areas that were originally undrained are now serviced by the pump plant due to the development in the area. The outlet now drains more then the original 304 acres, possibly the total 580 acres, and has an increased percent impervious. [3]
   vi. Two new pumps were installed in 1996. These pumps were cited to have a capacity of 8000 gpm @ TDH 39.5. This capacity was compared to the original pumps at the same head and HNTB found that the original configuration had one pump with 6000 gpm @ TDH 39.5 and another pump with 5750 gpm @ TDH 39.5. This shows that the pump plant capacity has increased at the stages stated in the table. However, it is unknown how much the capacity has increased.[3]
   vii. Pump station located under large overhead transmission lines.[9]

4. 5th Street (KCK FPS 11 / AID)

a. Year Constructed: 1963[12]
b. Owner/Operator: KCK
c. Levee Stationing: 185+70[1]
d. Distance From Flood Protection Centerline: 35ft[7]
e. Station Capacity: 39,000gpm[3]
f. Pumps:
   i. Fairbanks Morse 20" 13,000gpm @75hp and 580 RPM[4]
   ii. Fairbanks Morse 20" 13,000gpm @75hp and 580 RPM[4]
   iii. Fairbanks Morse 20" 13,000gpm @75hp and 580 RPM[4]
   iv. Sump Pump[4]
g. Contributing Flows:
   i. Storm Sewer Flow ~80ac[1]
   ii. Seepage Flow from Sta. 173+00 to Sta. 190+75 [1]
h. River Discharge: 72" RCP[3]
i. Discharge Arrangement: Gatewell & pressurized pipe under levee[7]
j. Drawing Availability:
   i. Dec 1979 – General Plan & Profile [7]
k. General
   i. It is believed that when the packing industry dissolved and left this area, it was decided by either the KVDD or other local interests to improve the drainage system so that the land would be more appealing for development. For this reason, the 5th Street Pump Plant was installed. This changed the drainage characteristics that existed during the composition of the 1948 "Supplement on Interior Drainage".[3]
   ii. Originally, water that was collected in the area of the 5th Street pump plant services was allowed to pond and would have been removed by infiltration and evaporation. It is the area described in the 1948 "Supplement on Interior Drainage", paragraph 24g, as the non-draining area. This non-draining area would have ponded water and then overflowed to the Shawnee Avenue Sewer System if the ponded water reached a certain control elevation. Now the 5th Street Pump Plant removes the water from this area. [3]
   iii. The typical design condition variables do not apply, as it appears that the pump plant was just designed to handle certain flow rates at certain stages. No problems were reported with this pump plant. [3]

5. MidWest Cold Storage

a. Year Constructed: N/A
b. Owner/Operator: Mid West Cold Storage
c. Levee Stationing: 194+60 [1]
d. Distance From Flood Protection Centerline:20ft
e. Station Capacity: 1,800gpm
f. Pumps: N/A
g. Contributing Flows:
   i. Storm Sewer Flow [1]
h. River Discharge: 24" CIP[3]
i. Discharge Arrangement: Gatewell & pressurized pipe under levee[7]
j. Drawing Availability:
k. General
   i. The 1948 "Supplement on Interior Drainage" explains that the pump was for the Midwest Cold Storage property. The outfall for the pump was a 6" removable pipe and pumped over the levee. [3]
   ii. It is believed that the Armourdale Pump Plant has made the function of this pump plant unnecessary. The KVDD is unaware of its use. However, the status has not been confirmed and the pump plant may be used periodically for the Midwest Cold Storage property. [3]
   iii. Pump Station has been abandoned. Discharge pipe passing through the levee has been filled with concrete. Gatewell on outside of building has been filled with rock and pump pit has not been filled.[10]

6. Shawnee Ave. (KCK FPS 10)

a. Year Constructed: 1954[3]
b. Owner/Operator: KCK
c. Levee Stationing: 230+77[1]
d. Distance From Flood Protection Centerline: 10ft[5]
e. Station Capacity: 63,000gpm[1]
f. Pumps:
   i. 30" 125hp[1] 710rpm[9]
   ii. 30" 125hp[1] 710rpm[9]
   iii. 30" 125hp[1] 710rpm[9]
   iv. 20" 100hp[1] 880rpm[9]
g. Contributing Flows:
   i. Storm Sewer Flow from Shawnee Sewer System, 140ac[1]
   ii. Seepage Flow from Sta. 190+75 to Sta. 246+35[1]
      1. Headerwell System 1 (1 to 24)[8]
h. River Discharge: 7.5'x7.5' RCB[3]
i. Discharge Arrangement:
j. Drawing Availability:
   i. Aug 1996 – Automatic lubrication system [5]
   ii. Dec 1979 – General Plan & Profile [7]
   iii. Sep 1969 – Record Drawings [5]
k. General
   i. The average elevation of the low lying contributing area is 14 feet below the design flood elevation. [3]
   ii. It was shown that this pump plant was replaced with a new pump plant in the plans for the 1962 revision. It was necessary to increase pump capacity because of the ponding that occurred during design flood stage. [3]
   iii. Ponding will occur under gravity flow due to inadequate sewer capacity. [3]
   iv. The main purpose of this plant is to discharge seepage from the Header Well System I. The current design needed to handle more seepage because the Design Flood elevation increased from original pump plant construction. [3]
   v. The recent (late 1970s) storm sewer system along Shawnee Ave., in conjunction with urban renewal, increased the delivery of storm water.[3]
   vi. Because of the seepage at stage 40.81, 88 acres of once pervious area is now computed to be impervious which increases runoff. Note that the area between 180+001 and 235+001 is storage. [3]
   vii. Railroad fill across Shawnee Avenue near the levee denies overland flow to the outlet. [3]
   viii. River discharge at outfall has been recently replaced.[9]

7. East Outfall Sewer Pump Plant

a. Year Constructed: N/A
b. Owner/Operator: KCK
c. Levee Stationing: 246+53[3]
d. Station Capacity: >12,000 gpm
e. Pumps: N/A
f. Contributing Flows:
   i. Storm Sewer System Flow (water from the railyards area)
g. River Discharge: 30” DIP[3]
h. Discharge Arrangement:
i. Drawing Availability:
j. General
   i. This pump is located in the railyards 650 feet west of 4th Street and Berger Avenue.[3]
   ii. A 24" DIP sewer line intercepts drainage from a 33" RCP culvert located under the tracks just west of the pump plant.[3]
   iii. The plant pumps water through the 24" sewer line to the 30" DIP outfall at station 246+53.[3]
   iv. The static head was determined as the difference between the centerline of the pipe at the sluice gate for the 30" DIP outfall and the minimum water level in the station. The elevation at the sluice gate is 748.1 (approx. stage 29.0 ft.). The impact of higher river stages on the static head was disregarded since the probability of simultaneous occurrence with the peak discharge of the pump plant was very remote. [3]
   v. Conditions in the area contributing to the pump plant have not changed significantly since the design of the plant. [3]
   vi. Pump plant is located approximately ¼ mile from flood protection and not station strength or floatation not considered to be an issue.[10]
   vii. Outlet conduit will be considered as utility. [10]

8. KCS RailRoad

a. Year Constructed: 1949[12]
b. Owner/Operator: KVDD
c. Levee Stationing: 276+79[1]
d. Station Capacity: N/A
e. Distance From Flood Protection Centerline: 40ft[7]
f. Pumps:
   i. 14" Fairbanks Morse submersible, 703RPM 37.9HP[11]
g. Contributing Flows:
   i. Seepage Flow from Sta. 268+59 to Sta. 282+29[1]
      1. Headerwell System 2 (1 to 14) [8]
h. River Discharge: 42"CIP[1]
i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under levee[7]
j. Drawing Availability:
   i. Dec 1979 – General Plan & Profile [7]
   ii. Apr 1951 – Structural Details [7]
k. General
   i. No performance curves were found for the pump, so the capacity was assumed to be 2.23 cfs, which is the pump rating. [3]
   ii. The pump outfall is through a 6 inch pipe 8 inches below the top of wall. Gravity flow is through a 10" CIP. Because of the low capacity of the pump, it was proposed (in the "Supplement on Interior Drainage") to construct a pump plant with a capacity of 13.5 cfs. This did not happen. [3]
   iii. Wells #1 through #6 no longer discharge to the KCS Pump because of a stability berm that now covers the discharge openings at the top of the structure.[3]
   iv. This pump is to be operated only when the Kansas River is below elevation 747.4 and falling. If the Kansas River is rising, the plant sluice gate must be closed which causes ponding in the railroad area. [3]
   v. An open ditch conveys seepage water to the plant. Due to the open ditch, there is a runoff flow in an uncontrolled condition. The worst case is at stage 17.7 when the runoff is 4.6 cfs.[3]
   vi. A new pump was added in 1995. [3]
   vii. Pump plant is located approximately 40ft from floodwall.
   viii. Some drawings found in Vol 1 O&M manual on the E:drive.
   ix. The main purpose of this pump is to discharge seepage from the Header Well System II. [1]
   x. Wells, #1 through #6, no longer discharge to the KCS Pump because of a stability berm that now covers the discharge openings at the top of the structure. However, the pump capacity is likely still inadequate. [1]
   xi. Old vertical pump was removes and replaced with submersible pump in 1995. Pump information including curves was supplied by KVDD.[10]

9. PBI Gordon

a. Year Constructed:
b. Owner/Operator: PBI Gordon
c. Levee Stationing: 286+59[6]
d. Distance From Flood Protection Centerline: 20ft
e. Station Capacity: 600gpm
f. Pumps:
   i. Aurora Pump 4x5x9B 600gpm at 36ft of head, 1750rpm, powered by a 7.5hp motor
g. Contributing Flows:
   i. Storm Runoff[6] Pumps drainage from employee parking[10]
h. River Discharge:
i. Discharge Arrangement: Gatewell at pump station and pressurized pipe through levee[10]
j. Drawing Availability:
k. General: Aurora pump noted above was new and not yet installed at Aug2005 site visit. PBI contact said that the new pump was approx. the same size / capacity as old one. Normally, gate is closed (i.e. ALL drainage flow from small parking area is pumped). This is to ensure that any chemical leaks are contained.

10. National Beef (KCK FPS 8)

a. Year Constructed: 1928[5]
b. Owner/Operator: KCK
c. Levee Stationing: 295+52 [1]
d. Distance From Flood Protection Centerline: 40ft[7]
e. Station Capacity: 3,000gpm [5]
f. Pumps:
   i. Vertical Pump 12" suction and 10" discharge 3,000gpm[5]
   ii. Sump Pump 3" discharge
g. Contributing Flows:
   i. Storm Sewer System Flow from the Riverside Sewer System, 8 acres. [1]
h. River Discharge: 10"CIP[5]
i. Discharge Arrangement: Pressurized pipe exiting at floodwall.[5]
j. Drawing Availability:
   i. Dec 1979 – General Plan & Profile [7]
   ii. March 1928 – General layout, plans & elevations, and structural steel [5]
   iii. March 1918 – Proposed Riverside Pump Station [5]
k. General
   i. The plant was originally built to service the National Beef Packing Plant, which included surface runoff, seepage, and negligible sanitary flow on the National Beef property. This industry has dissolved, but the drainage characteristics have not changed. Therefore, the pump is still in service. [3]
   ii. Seepage flow has been removed because the Header Well System III was built and now collects the seepage flow. [3]
   iii. The KVDD refers to this pump plant as the Central Avenue pump plant, yet the operational drawings label this pump plant as the National Beef pump plant.[3]
   iv. The pump capacity is remains inadequate at the Design River Stage, but ponding should not cause damage.[3]

11. Central Avenue

a. Year Constructed: 1976[3]
b. Owner/Operator: KVDD
c. Levee Stationing: 299+20[1]
d. Distance From Flood Protection Centerline: 16ft[7]
e. Station Capacity: N/A
f. Pumps:
   i. 3200gpm at 32.5ft of head at 1760rpm powered by a 40hp motor[10]
   ii. 3200gpm at 32.5ft of head at 1760rpm powered by a 40hp motor[10]
   iii. Sump Pump[9]
g. Contributing Flows:
   i. Seepage Flow from Sta. 296+23 to Sta. 302+40[1]
      1. Headerwell System 3 (1 to 7a) [8]
h. River Discharge: 2 – 14" CIP, 3" CIP
i. Discharge Arrangement: Pressurized pipes, above levee, terminating riverward of floodwall[7]
j. Drawing Availability:
   i. Dec 1979 – General Plan & Profile [7]
   ii. Sep 1978 – Structural Plans [7]
k. General
   i. The well system consists of seven artesian wells.[1]
   ii. Conditions have not changed since the pump plant design.[1]
   iii. Since its construction it has operated in 1993 only.[10]

References:

1. HNTB Drainage Feature Inventory 2001 with updates
2. HNTB Ownership Listing
3. HNTB Latest
4. KCK O&M Manuals from KCK Maintenance Library (Cary Houchins)
5. KCK As-builts from KCK Sewer Department (Larry Henak)
6. Information provided verbally by Ernie Quinto
7. O&M Drawings located at E:\sec\LocalProtectionProjects\KansasCity
8. Per Scott Loehr, EC-GD
9. Per 25Jul05 site visit
10. Site visit with KVDD (Ernie Quinto & Larry Brennen)
11. Asbuilts obtained from KVDD
12. 1962 Mod Design Memorandum #3

EXHIBIT 2, PART 2: OSAGE AVE PUMP PLANT ANALYSIS

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station
file=Osage Avenue EC-FC Hyd.xls

Existing conditions calculations for Osage Avenue Pump Station.

Step 1 - Determine pump operating capacity at the following conditions

Pump 1

River Elevations - 753.00 and 772.37
Sump Elevations - 747.75 and 753.75

Static Heads

	River Elevation	Sump Elevation	Static Head
1	753 ft	753.75 ft	-0.75 ft
2	753 ft	747.75 ft	5.25 ft
3	772.37 ft	753.75 ft	18.62 ft
4	772.37 ft	747.75 ft	24.62 ft

System loss curves for both pumps were developed and included in the design plans. This system curve is attached to the calculations.

Osage Avenue Pump Station Head Losses Pump 1					
Flow	System Losses	Static Head			
GPM	ft	-0.75 ft	5.25 ft	18.62 ft	24.62 ft
1,000	0.20	-0.55 ft	5.45 ft	18.82 ft	24.82 ft
2,000	0.40	-0.35 ft	5.65 ft	19.02 ft	25.02 ft
3,000	0.80	0.05 ft	6.05 ft	19.42 ft	25.42 ft
4,000	1.50	0.75 ft	6.75 ft	20.12 ft	26.12 ft
5,000	2.20	1.45 ft	7.45 ft	20.82 ft	26.82 ft
6,000	3.20	2.45 ft	8.45 ft	21.82 ft	27.82 ft
7,000	4.25	3.50 ft	9.50 ft	22.87 ft	28.87 ft
8,000	5.60	4.85 ft	10.85 ft	24.22 ft	30.22 ft

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Existing conditions calculations for Osage Avenue Pump Station.

Step 1 (cont.) - Determine pump operating capacity at the following conditions

Pump 2

River Elevations -	753.00	and	772.37
Sump Elevations -	747.00	and	753.00

Static Heads

	River Elevation	Sump Elevation	Static Head
1	753.00 ft	753.00 ft	0.00 ft
2	753.00 ft	747.00 ft	6.00 ft
3	772.37 ft	753.00 ft	19.37 ft
4	772.37 ft	747.00 ft	25.37 ft

System loss curves for both pumps were developed and included in the design plans. This system curve is attached to the calculations.

Osage Avenue Pump Station Head Losses Pump 2					
Flow	System Losses	Static Head			
GPM	ft	0.00 ft	6.00 ft	19.37 ft	25.37 ft
1,000	0.20	0.20 ft	6.20 ft	19.57 ft	25.57 ft
2,000	0.40	0.40 ft	6.40 ft	19.77 ft	25.77 ft
3,000	0.80	0.80 ft	6.80 ft	20.17 ft	26.17 ft
4,000	1.50	1.50 ft	7.50 ft	20.87 ft	26.87 ft
5,000	2.20	2.20 ft	8.20 ft	21.57 ft	27.57 ft
6,000	3.20	3.20 ft	9.20 ft	22.57 ft	28.57 ft
7,000	4.25	4.25 ft	10.25 ft	23.62 ft	29.62 ft
8,000	5.60	5.60 ft	11.60 ft	24.97 ft	30.97 ft

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Existing conditions calculations for Osage Avenue Pump Station.

Step 2 - Determine pump station operating capacity at the following conditions

Pump Station Capacity

Case 1

River Elevation -	753.00		
Sump Elevations -	747.75		
Pump 1	7,250	gpm	@ 9.9ft head
Pump 2	7,250	gpm	@ 9.9ft head
	14,500	gpm	
	32.3	cfs	

Case 2

River Elevation -	772.37		
Sump Elevations -	747.75		
Pump 1	4,700	gpm	@ 26.32ft head
Pump 2	4,700	gpm	@ 26.32ft head
	9,400	gpm	
	20.9	cfs	

Case 3

River Elevation -	753.00		
Sump Elevations -	753.00		
Pump 1	7,700	gpm	@ 5ft head (slightly off chart)
Pump 2	7,700	gpm	@ 5ft head (slightly off chart)
	15,400	gpm	
	34.3	cfs	

Case 4

River Elevation -	772.37		
Sump Elevations -	753.00		
Pump 1	5,550	gpm	@ 22ft head
Pump 2	5,550	gpm	@ 22ft head
	11,100	gpm	
	24.7	cfs	

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Future Conditions Hydraulics

Future conditions calculations for Osage Avenue Pump Station.

Step 1 - Determine pump operating capacity at the following conditions

Pump 1

River Elevations -	753.00	and	774.00
Sump Elevations -	747.75	and	753.75

Static Heads

	River Elevation	Sump Elevation	Static Head
1	753 ft	753.75 ft	-0.75 ft
2	753 ft	747.75 ft	5.25 ft
3	774.00 ft	753.75 ft	20.25 ft
4	774.00 ft	747.75 ft	26.25 ft

System loss curves for both pumps were developed and included in the design plans. This system curve is attached to the calculations.

Osage Avanue Pump Station Head Losses Pump 1					
Flow	System Losses	Static Head			
GPM	ft	-0.75 ft	5.25 ft	20.25 ft	26.25 ft
1,000	0.20	-0.55 ft	5.45 ft	20.45 ft	26.45 ft
2,000	0.40	-0.35 ft	5.65 ft	20.65 ft	26.65 ft
3,000	0.80	0.05 ft	6.05 ft	21.05 ft	27.05 ft
4,000	1.50	0.75 ft	6.75 ft	21.75 ft	27.75 ft
5,000	2.20	1.45 ft	7.45 ft	22.45 ft	28.45 ft
6,000	3.20	2.45 ft	8.45 ft	23.45 ft	29.45 ft
7,000	4.25	3.50 ft	9.50 ft	24.50 ft	30.50 ft
8,000	5.60	4.85 ft	10.85 ft	25.85 ft	31.85 ft

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Future Conditions Hydraulics

Existing conditions calculations for Osage Avenue Pump Station.

Future conditions calculations for Osage Avenue Pump Station.

Pump 2

River Elevations -	753.00	and	774.00
Sump Elevations -	747.00	and	753.00

Static Heads

	River Elevation	Sump Elevation	Static Head
1	753.00 ft	753.00 ft	0.00 ft
2	753.00 ft	747.00 ft	6.00 ft
3	774.00 ft	753.00 ft	21.00 ft
4	774.00 ft	747.00 ft	27.00 ft

System loss curves for both pumps were developed and included in the design plans. This system curve is attached to the calculations.

Osage Avanue Pump Station Head Losses Pump 2					
Flow	System Losses	Static Head			
GPM	ft	0.00 ft	6.00 ft	21.00 ft	27.00 ft
1,000	0.20	0.20 ft	6.20 ft	21.20 ft	27.20 ft
2,000	0.40	0.40 ft	6.40 ft	21.40 ft	27.40 ft
3,000	0.80	0.80 ft	6.80 ft	21.80 ft	27.80 ft
4,000	1.50	1.50 ft	7.50 ft	22.50 ft	28.50 ft
5,000	2.20	2.20 ft	8.20 ft	23.20 ft	29.20 ft
6,000	3.20	3.20 ft	9.20 ft	24.20 ft	30.20 ft
7,000	4.25	4.25 ft	10.25 ft	25.25 ft	31.25 ft
8,000	5.60	5.60 ft	11.60 ft	26.60 ft	32.60 ft

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Future Conditions Hydraulics

Existing conditions calculations for Osage Avenue Pump Station.

Future conditions calculations for Osage Avenue Pump Station.

Pump Station Capacity

Case 1

River Elevation -	753.00		
Sump Elevations -	747.75		
Pump 1	7,250	gpm	@ 9.9ft head
Pump 2	7,250	gpm	@ 9.9ft head
	14,500	gpm	
	32.3	cfs	

Case 2

River Elevation -	774.00		
Sump Elevations -	747.75		
Pump 1	4,350	gpm	@ 28 ft head
Pump 2	4,350	gpm	@ 28 ft head
	8,700	gpm	
	19.4	cfs	

Case 3

River Elevation -	753.00		
Sump Elevations -	753.00		
Pump 1	7,700	gpm	@ 5ft head (slightly off chart)
Pump 2	7,700	gpm	@ 5ft head (slightly off chart)
	15,400	gpm	
	34.3	cfs	

Case 4

River Elevation -	774.00		
Sump Elevations -	753.00		
Pump 1	5,250	gpm	@ 23.7 ft head
Pump 2	5,250	gpm	@ 23.7 ft head
	10,500	gpm	
	23.4	cfs	

8000 PROPELLER PUMPS
PUMP PERFORMANCE

398

16″
8312

705 RPM

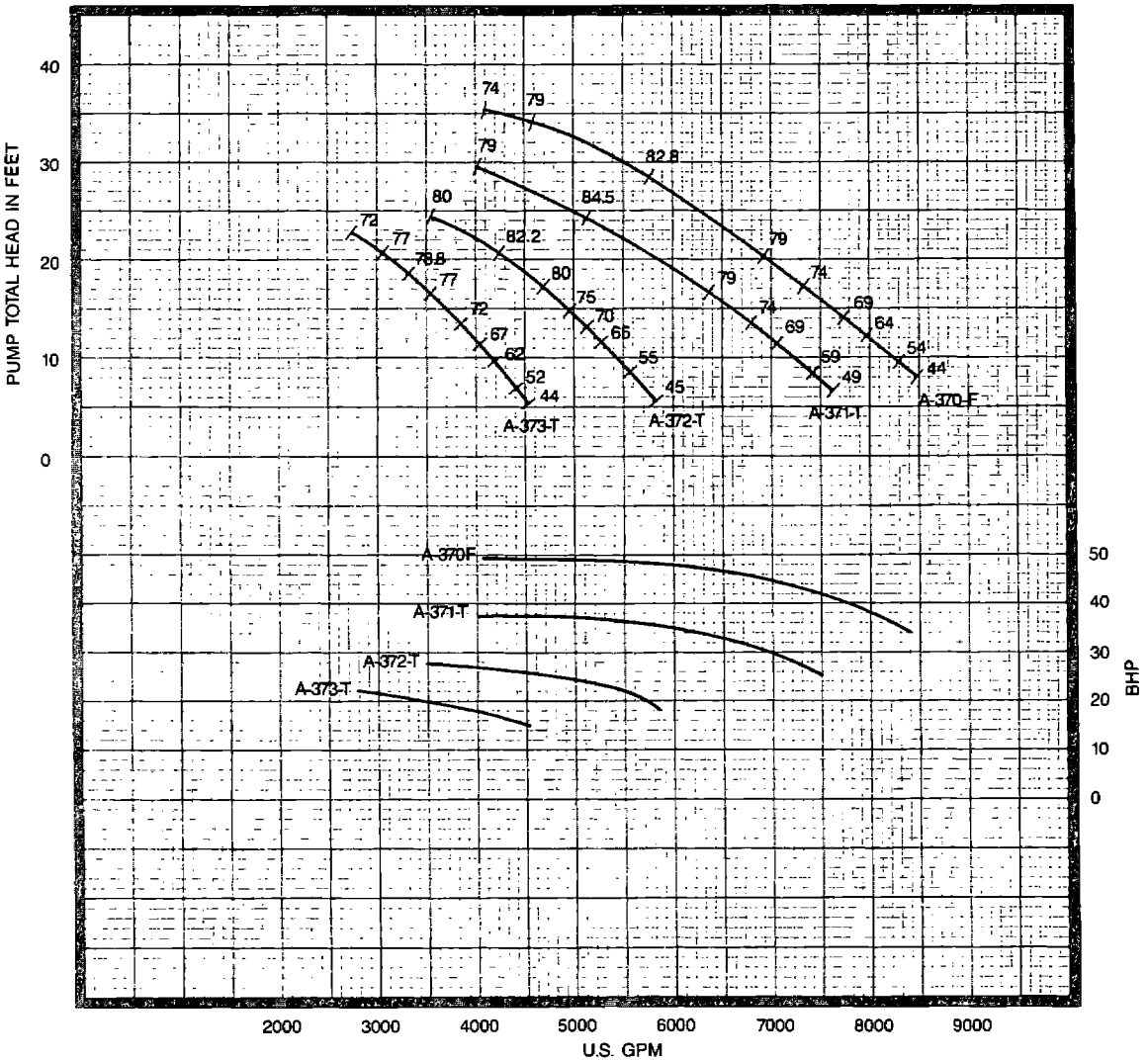
2 STAGE

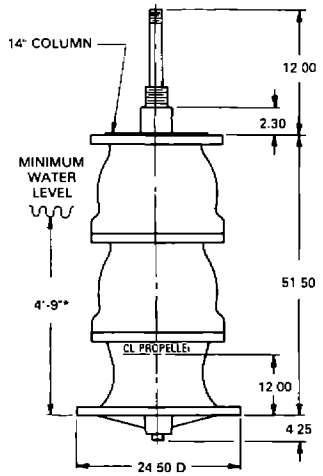
16″ COLUMN

16″ FABRICATED STEEL ELBOW

1-11/16″ LINESHAFT

2-1/2″ ENCLOSING TUBE





*This value is the minimum submergence required to prevent vortexing only. This value may need to be increased to provide adequate NPSHA.

DATA	VALUE	
PUMP SHAFT DIAMETER	1.6875 IN.	
MAXIMUM SPHERE SIZE	2.63 IN.	
Kt (THRUST FACTOR)	62 LBS./FT.	
Ka (TOTAL ROTOR WEIGHT)	132 LBS.	
Ks (SETTING CONSTANT)	7.6 LBS./FT.	
WK^2	28 LBS.-FT.^{2}	
BOWL ASSEMBLY WEIGHT	1025 LBS.	
EYE AREA: PROPELLER NO. A-372-T	124.0 SQ. IN.	3 VANE
PROPELLER NO. A-373-T	124.0 SQ. IN.	3 VANE
PROPELLER NO. A-370-F	124.0 SQ. IN.	4 VANE
PROPELLER NO. A-371-T	124.0 SQ. IN.	3 VANE
PROPELLER NO.		
PROPELLER NO.		

HYDRAULIC PERFORMANCE IS CONTINGENT ON FURNISHING THE PUMP WITH SPECIFIED AMOUNT OF CLEAR, FRESH, NON-AERATED WATER NOT TO EXCEED 85° F.

PUMP PERFORMANCE SHOWN IS BOWL ASSEMBLY WITH 10 FEET OF COLUMN INCLUDING A STANDARD ABOVE GROUND DISCHARGE ELBOW. ADDITIONAL COLUMN LOSSES SHOULD BE ADDED WHEN SETTINGS ARE DEEPER THAN 10 FEET AND/OR FOR OTHER DISCHARGE ARRANGEMENTS.

8000 PROPELLER PUMPS
PUMP PERFORMANCE

20″
8312

705 RPM

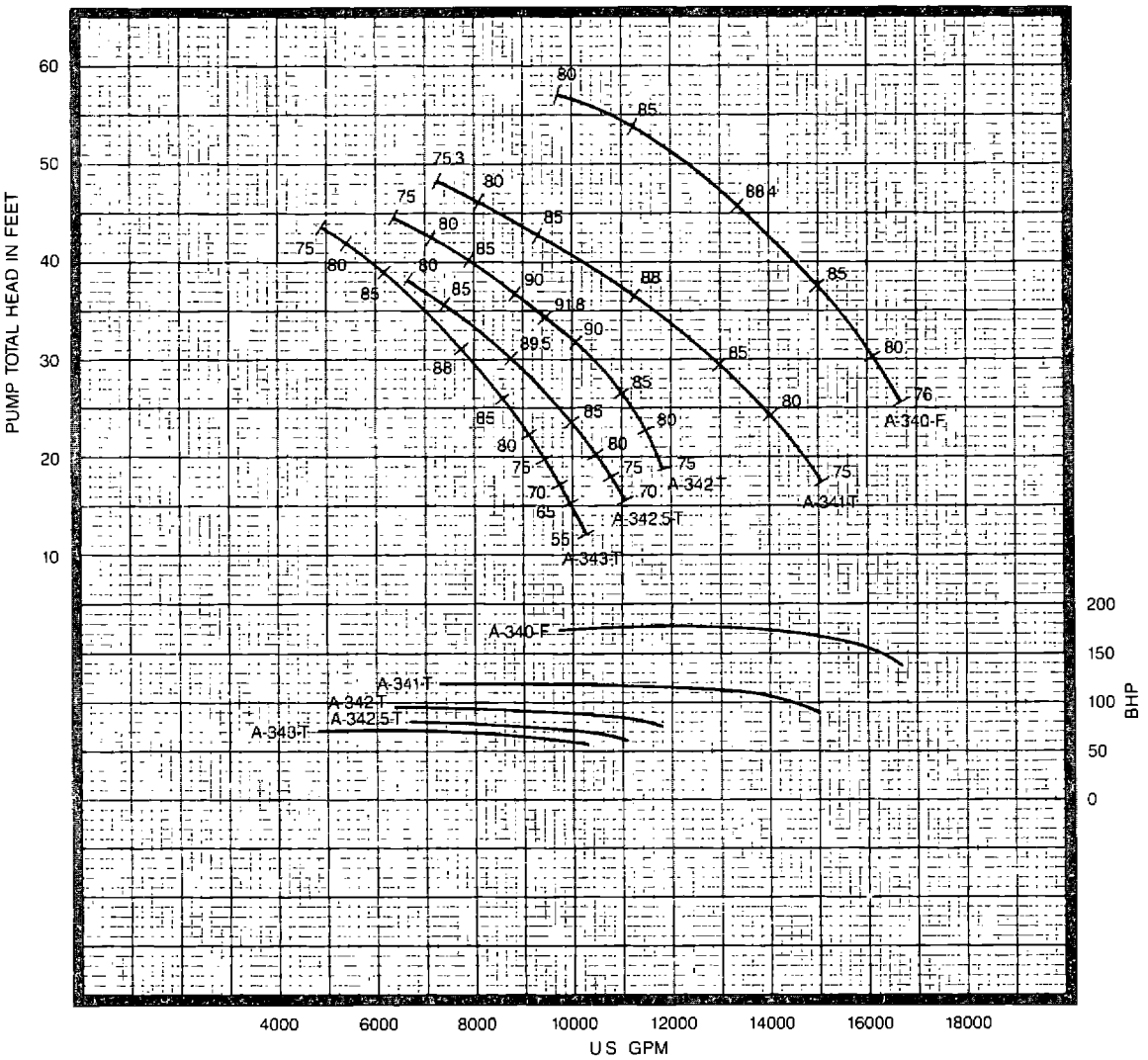
2 STAGE

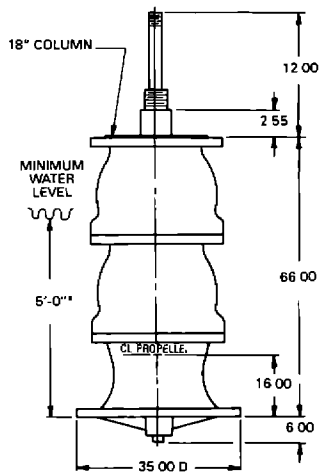
24″ COLUMN

24″ FABRICATED STEEL ELBOW

1-15/16″ LINESHAFT

3″ ENCLOSING TUBE





*This value is the minimum submergence required to prevent vortexing only. This value may need to be increased to provide adequate NPSHA.

DATA	VALUE	
PUMP SHAFT DIAMETER	1.9375 IN.	
MAXIMUM SPHERE SIZE	3.38 IN.	
Kt (THRUST FACTOR)	101 LBS./FT.	
Ka (TOTAL ROTOR WEIGHT)	250 LBS.	
Ks (SETTING CONSTANT)	10.0 LBS./FT.	
WK^2	86 LBS.-FT.^{2}	
BOWL ASSEMBLY WEIGHT	2150 LBS.	
EYE AREA: PROPELLER NO. A-340-F	203.0 SQ. IN.	4 VANE
PROPELLER NO. A-341-T	203.0 SQ. IN.	3 VANE
PROPELLER NO. A-342-T	203.0 SQ. IN.	3 VANE
PROPELLER NO. A-342.5-T	203.0 SQ. IN.	3 VANE
PROPELLER NO. A-343-T	203.0 SQ. IN.	3 VANE
PROPELLER NO.		

HYDRAULIC PERFORMANCE IS CONTINGENT ON FURNISHING THE PUMP WITH SPECIFIED AMOUNT OF CLEAR, FRESH, NON-AERATED WATER NOT TO EXCEED 85° F.

PUMP PERFORMANCE SHOWN IS BOWL ASSEMBLY WITH 10 FEET OF COLUMN INCLUDING A STANDARD ABOVE GROUND DISCHARGE ELBOW. ADDITIONAL COLUMN LOSSES SHOULD BE ADDED WHEN SETTINGS ARE DEEPER THAN 10 FEET AND/OR FOR OTHER DISCHARGE ARRANGEMENTS.

8000 PROPELLER PUMPS
PUMP PERFORMANCE

418

30″
8312

580 RPM

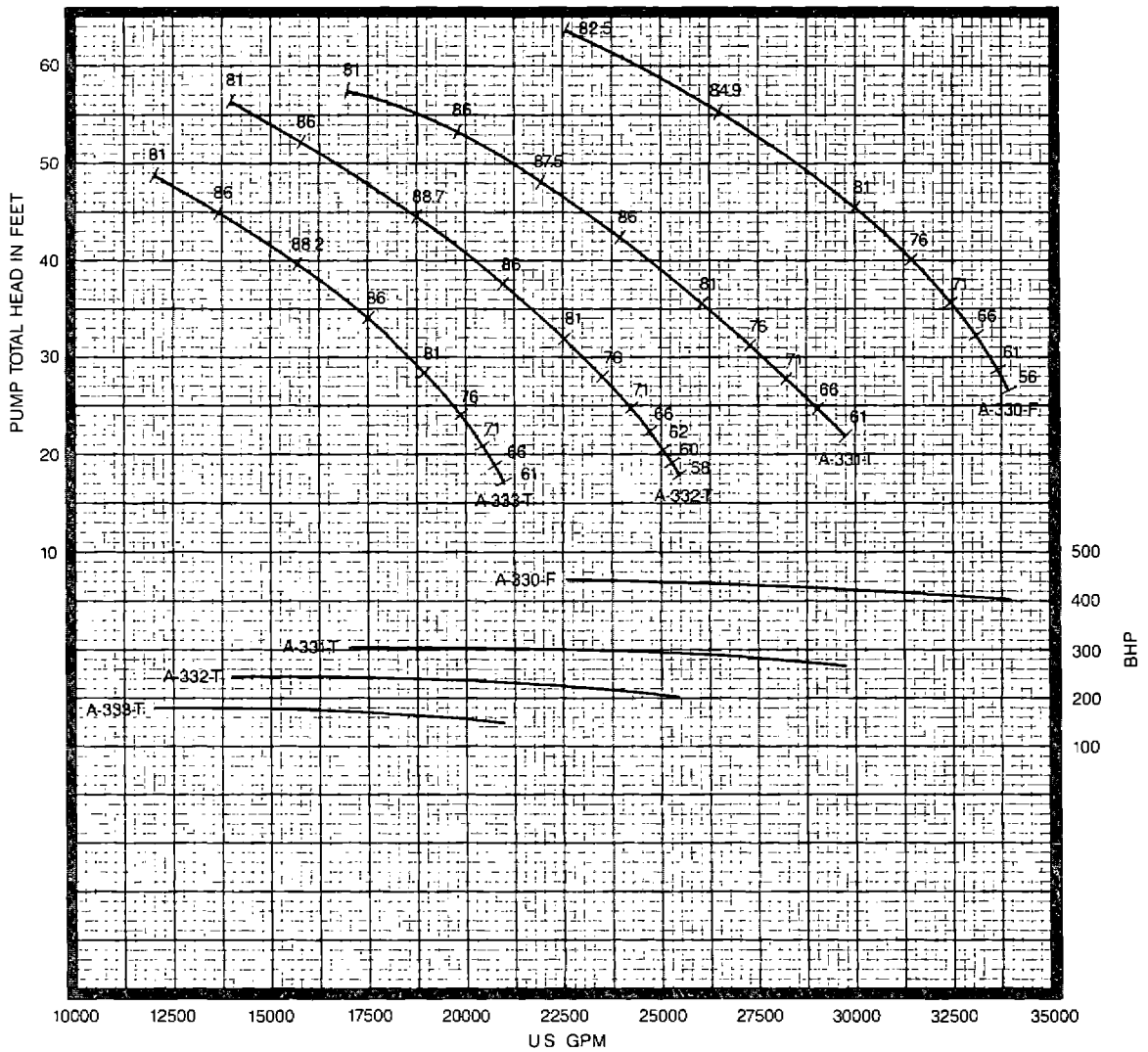
2 STAGE

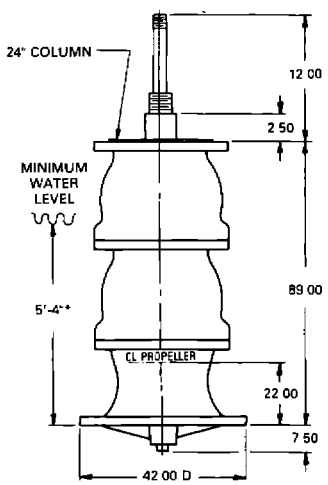
30″ COLUMN

30″ FABRICATED STEEL ELBOW

2-15/16″ LINESHAFT

5″ ENCLOSING TUBE





*This value is the minimum submergence required to prevent vortexing only. This value may need to be increased to provide adequate NPSHA.

DATA	VALUE	
PUMP SHAFT DIAMETER	2.9375 IN.	
MAXIMUM SPHERE SIZE	4.50 IN.	
K_t (THRUST FACTOR)	185 LBS./FT.	
K_a (TOTAL ROTOR WEIGHT)	620 LBS.	
K_s (SETTING CONSTANT)	23.0 LBS./FT.	
WK^2	390 LBS.-FT.^{2}	
BOWL ASSEMBLY WEIGHT	4300 LBS.	
EYE AREA: PROPELLER NO. A-330-F	367.8 SQ. IN.	4 VANE
PROPELLER NO. A-331-T	367.8 SQ. IN.	3 VANE
PROPELLER NO. A-332-T	367.8 SQ. IN.	3 VANE
PROPELLER NO. A-333-T	367.8 SQ. IN.	3 VANE
PROPELLER NO.		
PROPELLER NO.		

HYDRAULIC PERFORMANCE IS CONTINGENT ON FURNISHING THE PUMP WITH SPECIFIED AMOUNT OF CLEAR, FRESH, NON-AERATED WATER NOT TO EXCEED 85° F.

PUMP PERFORMANCE SHOWN IS BOWL ASSEMBLY WITH 10 FEET OF COLUMN INCLUDING A STANDARD ABOVE GROUND DISCHARGE ELBOW. ADDITIONAL COLUMN LOSSES SHOULD BE ADDED WHEN SETTINGS ARE DEEPER THAN 10 FEET AND/OR FOR OTHER DISCHARGE ARRANGEMENTS.

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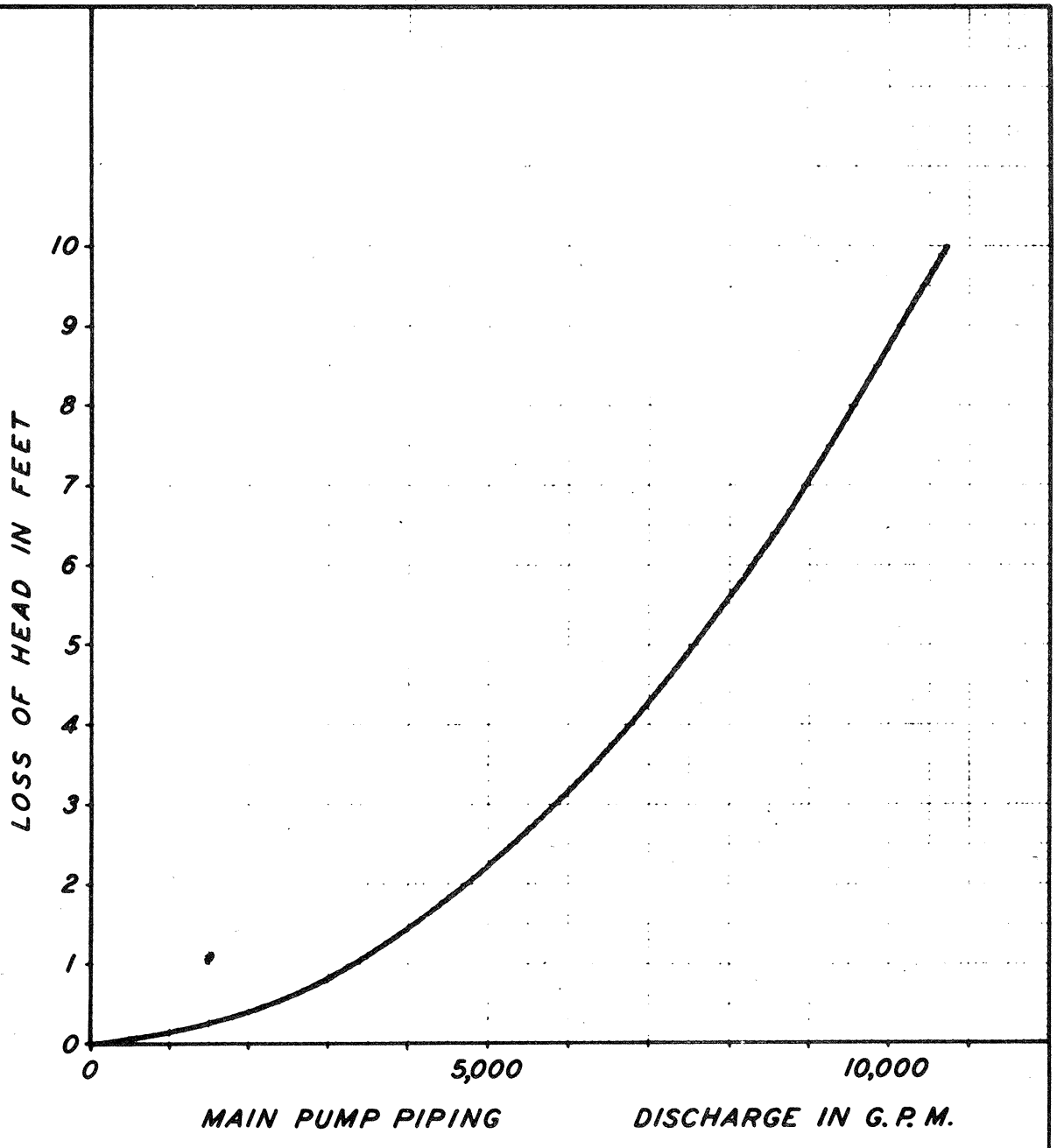

KANSAS CITYS FLOOD CONTROL PROJECT

ARMOURDALE UNIT

OSAGE AVENUE PUMPING PLANT

LOSS OF HEAD CURVE
MAIN PUMP PIPING

OFFICE, DISTRICT ENGINEER KANSAS CITY. MO.
KANSAS CITY DISTRICT APRIL 1952
FILE NO. A-10-3131

To accompany specifications dated: Sept. 1952

DRAFT **DRAFT**

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station
file=Replacement Pump Station.xls

Relief Wells and Collector Pipe

WELLS

The following information was supplied by EC-GD regarding relief wells

Flow from each well	1.25	cfs
Number of wells	26	
Total system flow	32.5	cfs
Discharge depth below grade	1.5	ft

HEADER

Assumptions/Notes

Pump station is located at one end of the area of concern. Therforeone header systems will be installed.
Per UHC 3-240-07FA header pipe is designed to flow at 80% depth of pipe diameter.

Header Pipe Diameter	36	in
Flow Depth (Q_{max})	28.8	in
FL EL in	-1.5	ft
FL EL out	-6	ft
Length	1800	ft
Slope	0.0025	
Manning's "N" (RCP)	0.012	
Flow Area	873.0	in^2
Wetted Perimeter	79.7	in
Flow	35.31	cfs
Velocity	5.83	ft/s

MANHOLES

Assumptions/Notes
In order to account of entrance and exit losses through manholes each invert out will be set 3" lower than inlet invert.

El drop at each manhole	3	inches
Number of manholes	26	
Elevation Drop Through Manholes	6.5	ft

ELEVATION OF HEADER PIPE INVERT INTO PUMP STATION

Elevation below grade = (Pipe Length * Slope) + El drop through Manholes + Discharge depth + Pipe Diameter

Elevation below grade = 15.5 ft below grade

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

General information regarding project design.

Length of discharge line	150	ft
Design Low Water	740	ft (msl)
Highest Point in System	776	ft (msl)
Static Head	36	ft
Runoff Flow	16	cfs
Seepage Flow	32.5	cfs
Desired System Output	21,768	gal/min
Desired Pump Output	10,884	gal/min

Friction loss in discharge pipe can be found with the Hazen Williams Formula.

$h_f = (4.72*Q^{1.852}*L)/(C^{1.852}*D^{4.87})$ (AWWA M11 Eq. 3-2)

h_f	Headloss due to friction		
Q	Flow	24.25	ft^3/s
L	Length of discharge pipe	150	ft
C	Hazen-Williams Coefficient	100	
D	Inside pipe diameter		ft

Velocity head loss can be figured by the following.

$h_v = V^2/2g$ (AWWA M11 Eq. 3-8)

h_v	Headloss in feet		
V	Velocity		
g	acceleration due to gravity	32.2	ft/s^2

Minor losses were then found using the following assumptions. From AWWA M 11 Figure 3.5

Description	K	Number	$K_{subtotal}$
60 degree miter bend	0.6	4	2.4

$h_m = K(V^2/2g)$

The following table presents head required for various pipe sizes

Diameter	Velocity	Head Loss (ft)				
in	*ft/s*	Static	Friction	Velocity	Minor	Total
20	11.12	36	4.27	1.92	4.60	46.8
24	7.72	36	1.76	0.93	2.22	40.9
30	4.94	36	0.59	0.38	0.91	37.9
36	3.43	36	0.24	0.18	0.44	36.9
	Used in Calculations for system curve					

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Kansas City District

PROJECT Kansas City Seven Levees

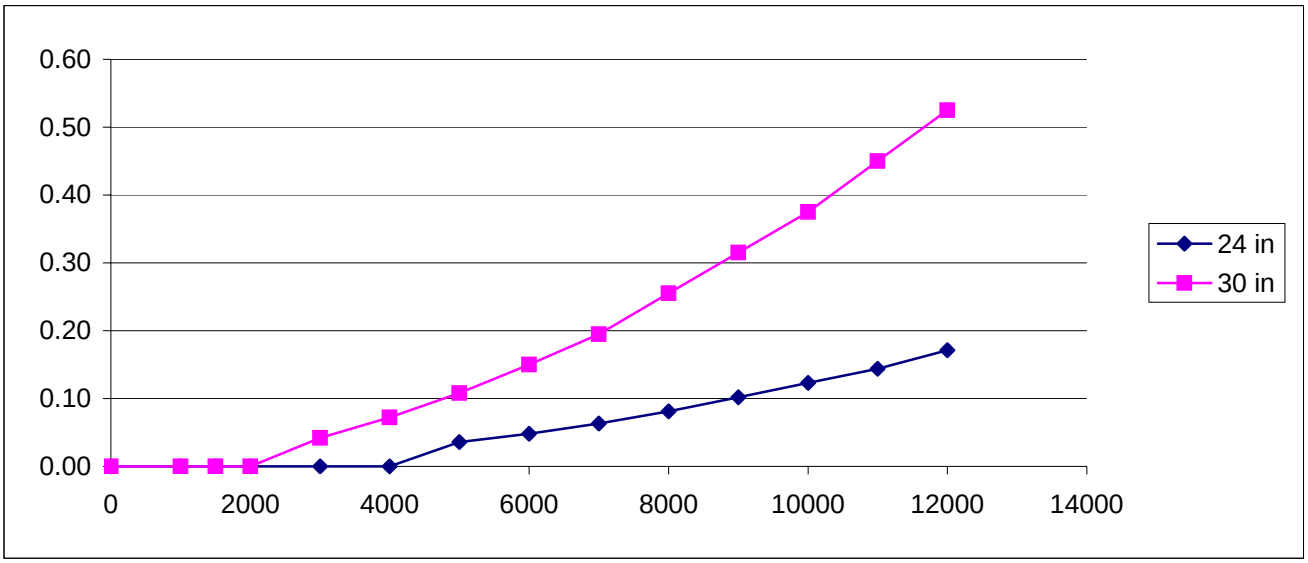
TASK **Osage Avenue Pump Station**
Replacement Pump Station

Column losses for this pump were determined using pg 276 of the Fairbanks Morse 8000 Propeller Pumps Application & Reference Data

The following table presents column losses in ft/100 of column for 20 and 24 inch columns.

	24 in		30 in	
GPM	per 100ft	**Total**	per 100ft	**Total**
0	0.00	**0.00**	0.00	**0.00**
1,000	0.00	**0.00**	0.00	**0.00**
1,500	0.00	**0.00**	0.00	**0.00**
2,000	0.00	**0.00**	0.00	**0.00**
3,000	0.14	**0.04**	0.00	**0.00**
4,000	0.24	**0.07**	0.00	**0.00**
5,000	0.36	**0.11**	0.12	**0.04**
6,000	0.50	**0.15**	0.16	**0.05**
7,000	0.65	**0.20**	0.21	**0.06**
8,000	0.85	**0.26**	0.27	**0.08**
9,000	1.05	**0.32**	0.34	**0.10**
10,000	1.25	**0.38**	0.41	**0.12**
11,000	1.50	**0.45**	0.48	**0.14**
12,000	1.75	**0.53**	0.57	**0.17**

* 30 feet of Column



US Army Corps of Engineers
Kansas City District

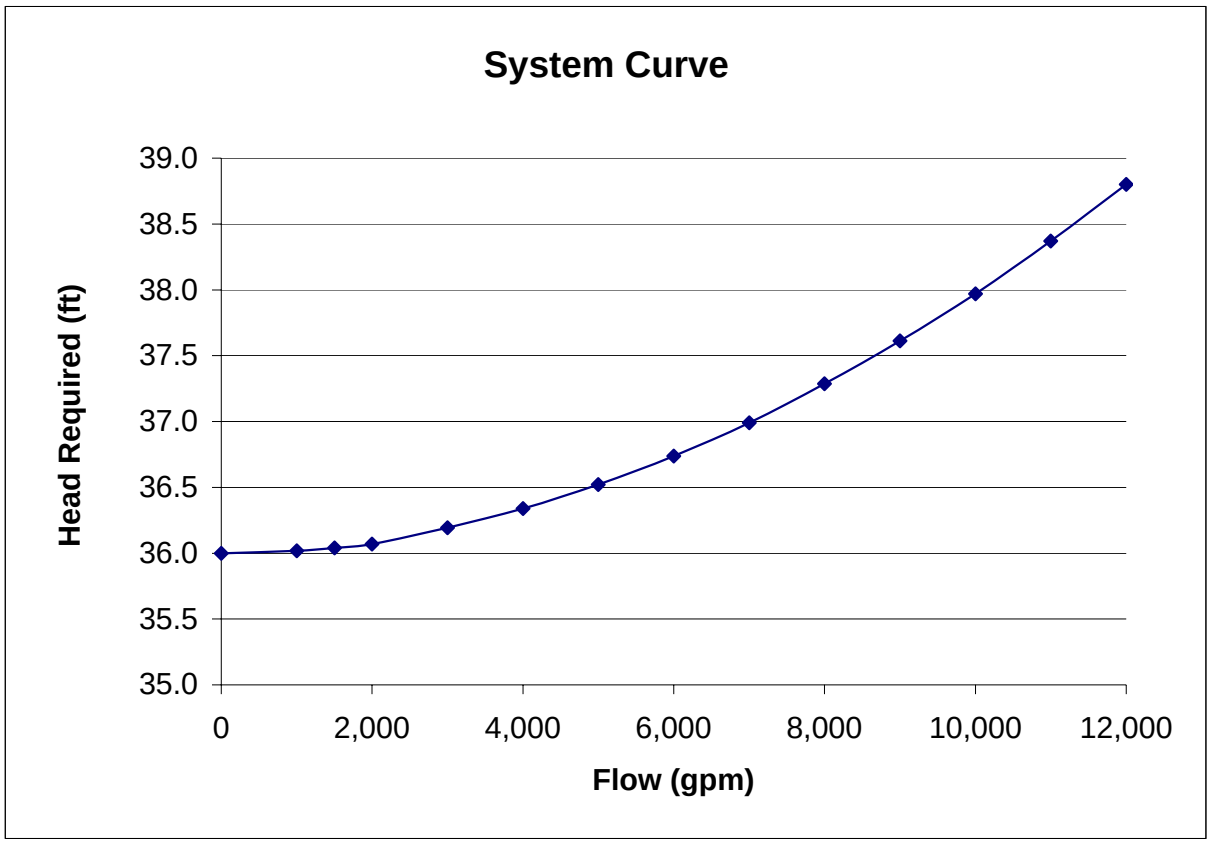
PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

PRELIMINARY SYSTEM CURVE

Flow		Head			
GPM	ft^3/s	Static	Dynamic	Column	Total
0	0.00	36.00	0.00	0.00	36.00
1,000	2.23	36.00	0.02	0.00	36.02
1,500	3.34	36.00	0.04	0.00	36.04
2,000	4.46	36.00	0.07	0.00	36.07
3,000	6.68	36.00	0.15	0.04	36.19
4,000	8.91	36.00	0.27	0.07	36.34
5,000	11.14	36.00	0.41	0.11	36.52
6,000	13.37	36.00	0.59	0.15	36.74
7,000	15.60	36.00	0.79	0.20	36.99
8,000	17.82	36.00	1.03	0.26	37.29
9,000	20.05	36.00	1.30	0.32	37.61
10,000	22.28	36.00	1.59	0.38	37.97
11,000	24.51	36.00	1.92	0.45	38.37
12,000	26.74	36.00	2.28	0.53	38.80

*System Curve, given: 30 in discharge pipe



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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

Horsepower Required at top of shaft.

Brake Horsepower = [Total head * Flow (GPM)]/[3960 * Pump efficiency]

Total Head	38.37	ft
Flow	10,884	gpm
Pump Eff.	0.82	

Brake Horsepower = 129 hp

From pg 277 of Fairbanks Morse Pumps line shaft horse power loss per 100ft of 2-7/16" shaft is 0.72 at 580 RPM

Hp line shaft loss = (loss/100ft)*setting depth/100

Loss/100ft	0.96	hp
setting depth	30	ft

Hp line shaft loss = 0.29 hp

Pump thrust = head * Kt + Ka + Setting in ft * Ks

Total Head	38.37	ft
Kt	127	lb/ft
Ka	350	lb
setting depth	30	
Ks	15.9	lb/ft

Pump thrust = 5,700 lb

Loss through thrust bearing = .0075hp per 1,000lb at 100rpm

RPM	580	
Thrust	5,700	lbs

Loss through thrust bearing = 0.25 hp

Motor HP = (HP_{brake} + $HP_{lineloss}$ + $HP_{lossthrustbearing}$)

Motor HP = 129

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PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

Pump station discharge when inflow equal to 1/2 the flow rate of first pump.

Pump Discharge	24.25	cfs
	10,884	gpm
Max starts per hr	2	
V=tQ/4	10,912	ft^3

Pump Station Discharge inflow equal to 1/2 the difference of flow rate of first pump and second pump.

Pump Discharge	24.25	cfs
	10,884	gpm
Max starts per hr	2	
V=tQ/4	10,912	ft^3

In order to obtain required storage volume the following storage dimensions were used.

Storage Configuration

Width	34	ft
Length	34	ft
Depth	10.0	ft
Volume	11,560	ft^3

Sump Depth

Sump Depth = Storage Depth + Pump Submergence + Pump Hieght off Floor + Inflow Pipe Invert

Storage Depth =	10.0	ft
Pump Submergence =	5	ft taken from pump literature
Pump Hieght off Floor =	20	in assumed to be 1/2 bell diameter
Inflow Pipe Invert =	15.50	ft below grade
Sump Depth =	32.2	ft below grade
Assumed grade elevation =	766.5	ft
Sump Floor Elevation =	734.3	ft

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

EXCAVATION

Assumptions

Mudsill will be placed to work from.
Walls will be 1in thick for every foot of depth
Heel extensions will be provided to prevent uplift.
Floor thickness will be assumed

Calculations

Heel Extension Width	1.5	ft
Wall Thickness	33	in
Max Sump Depth	32	ft
Sump Floor Thickness	3	ft
Mudsill Thickness	1	ft
Max Excavation Depth	36	ft

Excavation Length 43 ft
Sump Length + 2*Wall Thickness + 2*Heel Width

Excavation Width 43 ft
Sump Width + 2*Wall Thickness + 2*Heel Width

Assume Excavation Slope of 1.5 (H):1(V)

Excavation Area at max depth 1,806 ft^2

Excavation Length at grade 139 ft
Excavation Width at grade 139 ft

Excavation Area at grade 19,321 ft^2

Excavation Volume 382,051 ft^3
14,150 yd^3
(Area at depth + Area at Grade)/Depth

Volume of pump station/Spoil 2,152 yd^3
(Area at Depth * Depth)

Volume to be Compacted 11,998 yd^3
(Excavation - Pump station Vol)

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PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station
file=New Pump Station.xls

Relief Wells and Collector Pipe

WELLS

The following information was supplied by EC-GD regarding relief wells

Flow from each well	1.25	cfs
Number of wells	26	
Total system flow	32.5	cfs
Discharge depth below grade	1.5	ft

HEADER

Assumptions/Notes

Pump station will be located in center of area of concern. Therfore two header systems will be installed. Each system will handle 1/2 of the flow.
Per UHC 3-240-07FA header pipe is designed to flow at 80% depth of pipe diameter.

Header Pipe Diameter	24	in
Flow Depth (Q_{max})	19.2	in
FL EL in	-1.5	ft
FL EL out	-6	ft
Length	900	ft
Slope	0.005	
Manning's "N" (RCP)	0.012	
Flow Area	388.0	in^2
Wetted Perimeter	53.1	in
Flow	16.94	cfs
Velocity	6.29	ft/s

MANHOLES

Assumptions/Notes
In order to account of entrance and exit losses through manholes each invert out will be set 3" lower than inlet invert.

El drop at each manhole	3	inches
Number of manholes	13	
Elevation Drop Through Manholes	3.25	ft

ELEVATION OF HEADER PIPE INVERT INTO PUMP STATION

Elevation below grade = (Pipe Length * Slope) + El drop through Manholes + Discharge depth + Pipe Diameter

Elevation below grade = 11.25 ft below grade

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PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

General information regarding project design.

Length of discharge line	150	ft
Design Low Water	745	ft (msl)
Highest Point in System	776	ft (msl)
Static Head	31	ft
Desired System Output	14,585	gal/min
Desired Pump Output	7292.5	gal/min

Friction loss in discharge pipe can be found with the Hazen Williams Formula.

$h_f = (4.72*Q^{1.852}*L)/(C^{1.852}*D^{4.87})$ (AWWA M11 Eq. 3-2)

h_f	Headloss due to friction		
Q	Flow	16.25	ft^3/s
L	Length of discharge pipe	150	ft
C	Hazen-Williams Coefficient	100	
D	Inside pipe diameter		ft

Velocity head loss can be figured by the following.

$h_v = V^2/2g$ (AWWA M11 Eq. 3-8)

h_v	Headloss in feet		
V	Velocity		
g	acceleration due to gravity	32.2	ft/s^2

Minor losses were then found using the following assumptions. From AWWA M 11 Figure 3.5

Description	K	Number	$K_{subtotal}$
60 degree miter bend	0.6	4	2.4

$h_m = K(V^2/2g)$

The following table presents head required for various pipe sizes

Diameter	Velocity	Head Loss (ft)				
in	*ft/s*	Static	Friction	Velocity	Minor	Total
20	7.45	31	2.03	0.86	2.07	36.0
24	5.17	31	0.84	0.42	1.00	33.2
30	3.31	31	0.28	0.17	0.41	31.9
36	2.30	31	0.12	0.08	0.20	31.4
	Used in Calculations for system curve					

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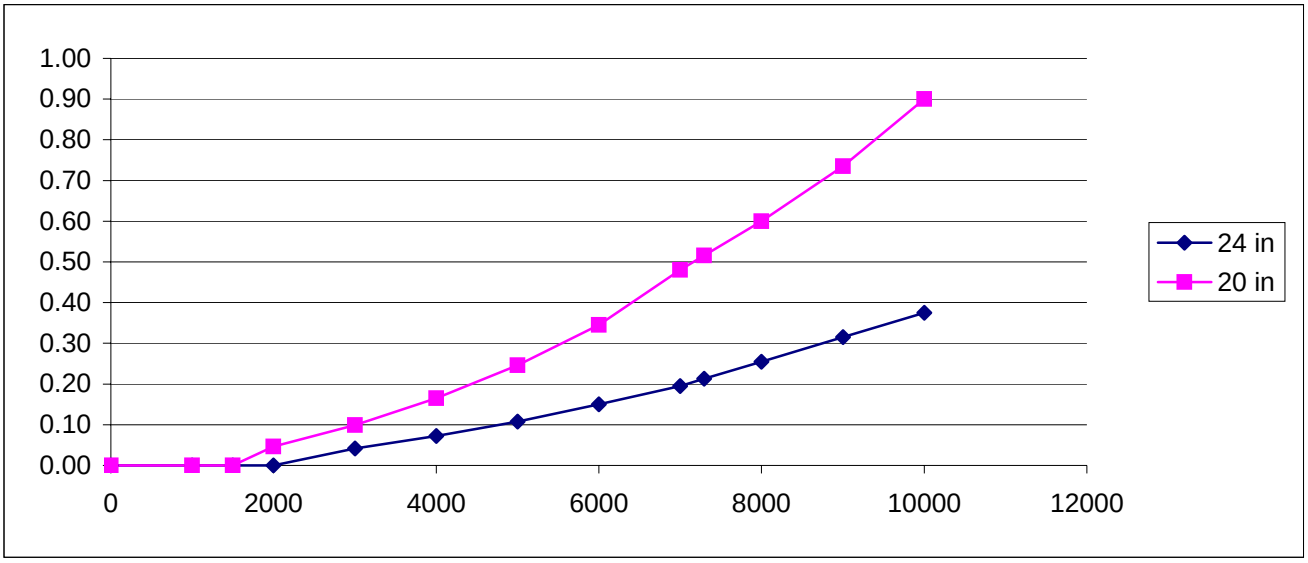
PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Column losses for this pump were determined using pg 276 of the Fairbanks Morse 8000 Propeller Pumps Application & Reference Data

The following table presents column losses in ft/100 of column for 20 and 24 inch columns.

	20 in		24 in	
GPM	per 100ft	**Total**	per 100ft	**Total**
0	0.00	**0.00**	0.00	**0.00**
1,000	0.00	**0.00**	0.00	**0.00**
1,500	0.00	**0.00**	0.00	**0.00**
2,000	0.16	**0.05**	0.00	**0.00**
3,000	0.33	**0.10**	0.14	**0.04**
4,000	0.55	**0.17**	0.24	**0.07**
5,000	0.82	**0.25**	0.36	**0.11**
6,000	1.15	**0.35**	0.50	**0.15**
7,000	1.60	**0.48**	0.65	**0.20**
7,293	1.72	**0.52**	0.71	**0.21**
8,000	2.00	**0.60**	0.85	**0.26**
9,000	2.45	**0.74**	1.05	**0.32**
10,000	3.00	**0.90**	1.25	**0.38**
11,000	0.00	**0.00**	0.00	**0.00**
12,000	0.00	**0.00**	0.00	**0.00**
*	30 feet of Column			



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Kansas City District

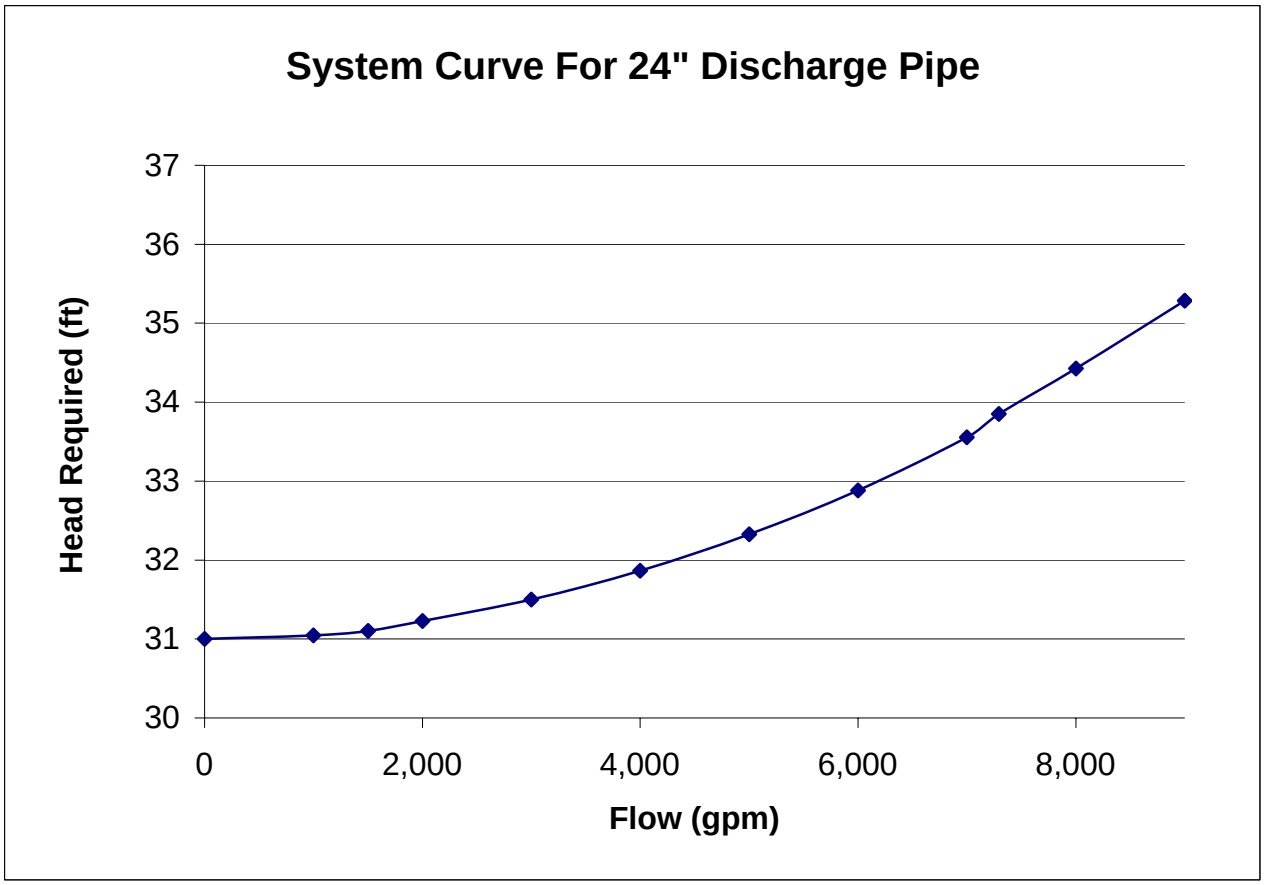
PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

PRELIMINARY SYSTEM CURVE

Flow		Head			
GPM	ft^3/s	Static	Dynamic	Column	Total
0	0.00	31.00	0.00	0.00	31.00
1,000	2.23	31.00	0.05	0.00	31.05
1,500	3.34	31.00	0.10	0.00	31.10
2,000	4.46	31.00	0.18	0.05	31.23
3,000	6.68	31.00	0.40	0.10	31.50
4,000	8.91	31.00	0.70	0.17	31.86
5,000	11.14	31.00	1.08	0.25	32.33
6,000	13.37	31.00	1.54	0.35	32.88
7,000	15.60	31.00	2.08	0.48	33.56
7,293	16.25	31.00	2.25	0.60	33.85
8,000	17.82	31.00	2.69	0.74	34.43
9,000	20.05	31.00	3.39	0.90	35.29

*System Curve, given: 24 in discharge pipe



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PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Horsepower Required at top of shaft.

Brake Horsepower = [Total head * Flow (GPM)]/[3960 * Pump efficiency]

Total Head	33.85	ft
Flow	7,293	gpm
Pump Eff.	0.80	

Brake Horsepower = 77.9 hp

From pg 277 of Fairbanks Morse Pumps line shaft horse power loss per 100ft of 1-15/16" shaft is 0.72 at 720 RPM

Hp line shaft loss = (loss/100ft)*setting depth/100

Loss/100ft	0.72	hp
setting depth	30	ft

Hp line shaft loss = 0.22 hp

Pump thrust = head * Kt + Ka + Setting in ft * Ks

Total Head	33.85	
Kt	101	lb/ft
Ka	250	lb
setting depth	30	
Ks	10	lb/ft

Pump thrust = 3,969 lb

Loss through thrust bearing = .0075hp per 1,000lb at 100rpm

RPM	705	
Thrust	3,969	lbs

Loss through thrust bearing = 0.21 hp

Motor HP = (HP_{brake} + $HP_{lineloss}$ + $HP_{lossthrustbearing}$)

Motor HP = 78.3

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PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Pump station discharge when inflow equal to 1/2 the flow rate of first pump.

Pump Discharge	16.25	cfs
	7,293	gpm
Max starts per hr	3	
V=tQ/4	4,874	ft^3

Pump Station Discharge inflow equal to 1/2 the difference of flow rate of first pump and second pump.

Pump Discharge	16.25	cfs
	7,293	gpm
Max starts per hr	3	
V=tQ/4	4,874	ft^3

In order to obtain required storage volume the following storage dimensions were used.

Storage Configuration

Width	25	ft
Length	25	ft
Depth	8.0	ft
Volume	5,000	ft^3

Sump Depth

Sump Depth = Storage Depth + Pump Submergence + Pump Hieght off Floor + Inflow Pipe Invert

Storage Depth =	8.0	ft
Pump Submergence =	5	ft taken from pump literature
Pump Hieght off Floor =	20	in assumed to be 1/2 bell diameter
Inflow Pipe Invert =	11.25	ft below grade
Sump Depth =	25.9	ft below grade
Assumed grade elevation =	766.5	ft
Sump Floor Elevation =	740.6	ft

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PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

EXCAVATION

Assumptions

Mudsill will be placed to work from.
Walls will be 1in thick for every foot of depth
Heel extensions will be provided to prevent uplift.
Floor thickness will be assumed

Calculations

Heel Extension Width	1.5	ft
Wall Thickness	26	in
Max Sump Depth	26	ft
Sump Floor Thickness	3	ft
Mudsill Thickness	1	ft
Max Excavation Depth	30	ft

Excavation Length 32 ft
Sump Length + 2*Wall Thickness + 2*Heel Width

Excavation Width 32 ft
Sump Width + 2*Wall Thickness + 2*Heel Width

Assume Excavation Slope of 1.5 (H):1(V)

Excavation Area at max depth 1,045 ft^2

Excavation Length at grade 110 ft
Excavation Width at grade 110 ft

Excavation Area at grade 12,118 ft^2

Excavation Volume 196,908 ft^3
7,293 yd^3
(Area at depth + Area at Grade)/Depth

Volume of pump station/Spoil 1,003 yd^3
(Area at Depth * Depth)

Volume to be Compacted 6,289 yd^3
(Excavation - Pump station Vol)

Exhibit 2, Part 3: Shawnee Avenue Pump Plant Analysis

TABLE 12.--SHAWNEE AVENUE SEWER SYSTEM

Summary of run-offs at various stages

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Stage			Rainfall		Unit run-off		Peak run-off						PONDAGE[a]			
													Without pumping		With pumping present facilities	
Argentine Bridge gage	Kansas City gage	Equiv. water surface elevation	60-min. i.p.h.	16-min. i.p.h.	Imp. i.p.h.	Perv. i.p.h.	c.f.s.	i.p.h.	Avg. supply rate i.p.h.	Mass run-off inches	Max. cap. sewer (gravity flow) c.f.s.	Pump cap. c.f.s.	Avg. depth inches	Duration min.	Avg. depth inches	Duration min.
7.2	12.2	732.6(b)	2.50	4.98	4.42	1.50	399.0	2.84	3.52	0.94	145	-	0.39	55	-	-
12.5	16.0(c)	737.2	2.26	4.54	3.98	1.05	336.0	2.40	3.09	0.82	111	27.4	0.38	63	0.70	272
15.0	18.0	739.6	2.01	4.00	3.44	0.73	277.0	1.98	2.55	0.68	88	27.0	0.32	66	0.56	228
17.4	20.0	741.8	1.52	3.02	2.46	0.18	172.0	1.23	1.57	0.42	62	26.4	0.18	59	0.31	144
18.5	21.0	743.0	1.43	2.84	2.28	0.11	155.0	1.12	1.39	0.37	41	26.2	0.21	79	0.26	128
19.6	22.0	744.0	1.36	2.71	2.15	0.07	144.0	1.03	1.26	0.34	0	26.0	0.34		0.23	119
22.4	25.0	747.1	1.07	2.13	1.57	0	101.0	0.72	0.72	0.19	0	25.4	0.19		0.11	68
23.2	26.8	748.8	0.78	1.52	0.96	0	61.9	0.44	0.44	0.12	0	25.2	0.12		0.05	43
26.3	30.0	751.3	0.50	1.00	0.44	0	28.4	0.20	0.20	0.05	0	24.7	0.05		Negligible	18
27.6	32.0	752.8	0.40	0.80	0.24	0	15.5	0.11	0.11	0.03	0	24.2	0.03		0.0	0
31.8	40.8(d)	758.8	0.40	0.80	0.24	0	39.4	0.17	0.17	0.05	0	22.0	0.05		0.02	41

Total tributary area is 140.03 acres of which 64.44 acres are impervious.

(a) Pondages were computed by reducing the pump and sewer capacities by the amount of sanitary sewage (1.9 c.f.s.)
(b) Intrados of the 57-inch outlet at the pumping plant
(c) Present pump starting stage
(d) Refer to paragraph 24g

NOTE: There is no longer sanitary flow, and the seepage flow is intercepted at the levee toe by relief well system. Extent of drainage basin is essentially unchanged from 1947. Therefore, use coincident storm flow of 15.5 cfs.

f. Shawnee Avenue Pumping Plant.

(1) The Shawnee Avenue Pumping Plant at station 231+39 is deteriorated and obsolete. A new outlet and pumping plant just upstream to this location is being built by Kansas City, Kansas, and engineered by Schlup, Becker & Brennan, Consulting Engineers, at an estimated cost of $650,000. Construction is expected during 1971. The proposed outlet plans indicate a 7-1/2- by 7-1/2-foot RCB with integral sluice gate and pumping plant within the levee fill. A newly proposed storm sewer system along Shawnee Avenue, in conjunction with urban renewal, will increase the delivery of stormflow to the outlet. A railroad fill across Shawnee Avenue near the levee denies overland flow to the outlet structure. The outlet will discharge a relatively high frequency which cannot be pinpointed due to lack of data. Four vertical, mixed flow propeller pumps; three 30-inch pumps driven by 200 hp. motors; and one 20-inch pump driven by a 100 hp. motor will be installed. Using the proposed pumps, the plant will have a capacity of 141 c.f.s. when pumping with river at top of levee elevation 765.2 feet from a pit level of 734.0 feet, m.s.l., the average of pump starting elevations. The pumps will discharge 185 c.f.s. with river stage at elevation 740. The elevation 740 becomes a significant river stage because at that stage the pumping plant capability is equal to the gravity outlet capability. Therefore, with the river above this stage more stormflow can be discharged by pumping, and below this stage more can be discharged by gravity (assuming the temporary ponding within the protected area is comparable). The maximum estimated pressure relief well seepage of 35 c.f.s. with river stage at 765.2 will decrease to zero at stage 745. The proposed capability of the storm pumping plant when gaged by the 1948 Supplement approaches a flood recurrence interval of 100 years.

(2) The existing Shawnee Avenue Pumping Plant is located landward of the levee at station 231+39. The pumping plant is equipped with two pumps, one 16-inch vertical centrifugal sewage pump driven by a 100 hp. electric motor, and one 10-inch horizontal sewage pump driven by a 40 hp. motor. Both pumps are of the dry pit type. The plant has a capacity of 21.5 c.f.s. with river at top of levee elevation 765.2 feet, m.s.l., when pumping from the pump starting elevation of 737.2 feet, m.s.l. At a river stage of 747.2 feet, m.s.l., and the above pit elevation, the pumping capacity is 26.2 c.f.s.

NOTE: The pump curves provided indicate slightly less capacity than indicated here. Need to verify exactly which pumps we have.....OK for existing condition either way, but if the pumps are as described on this page, may be adequate capacity for n500+3 design condition. As analyzed (i.e. with the smaller pumps, per pump curves provided), there is insufficient capacity at n500+3 max river stage condition requiring pump upgrades.

Cook Industries

Fairbanks Morse
Pump Division

Shawnee Ave Pumps

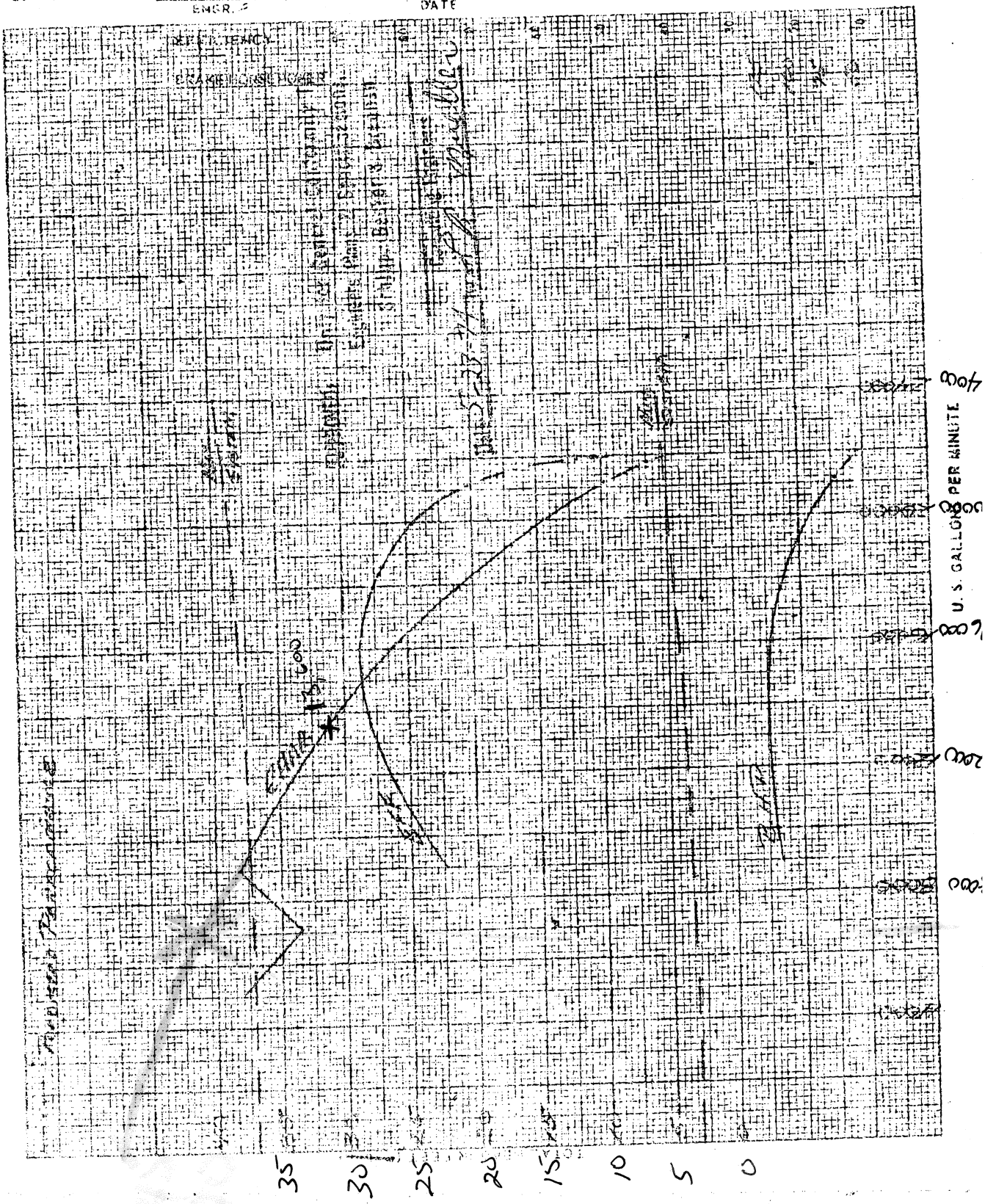
CERTIFIED CURVE

PUMP NO. [illegible] STAGES 1 SIZE-FIGURE #20-6260

TESTED 5/14/24 DRIVER Test Mtr. IMPELLER [illegible]

IMP. DIA. 22 3/32"

Certified Correct by [illegible] ENGR. 5/14 DATE R.P.M. 705



Colt Industries Fairbanks Morse Pump Division

CERTIFIED CURVE

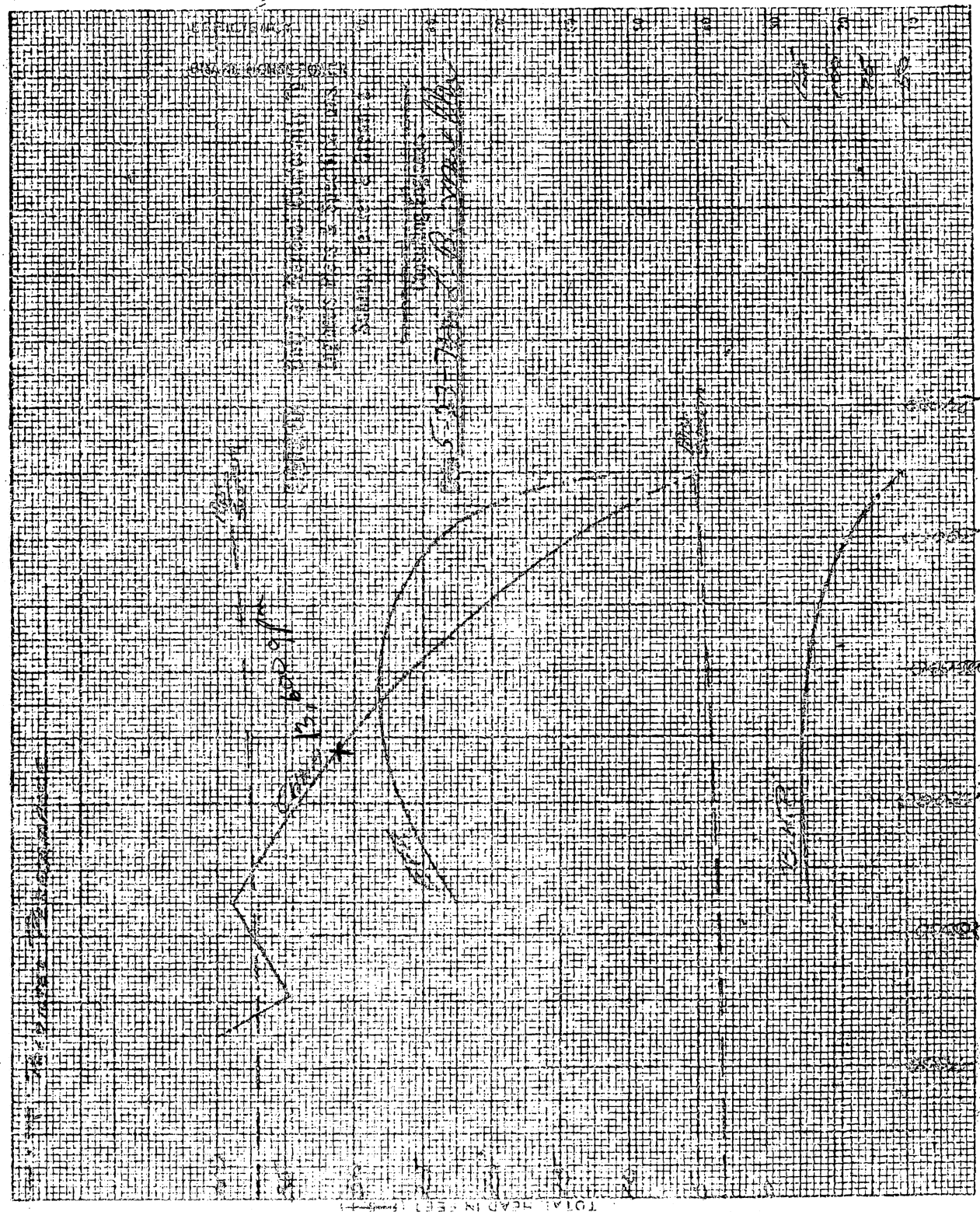
PUMP NO. 42V2-071099-0 STAGES 1 SIZE-FIGURE #20-6360

TESTED 5/13/74 DRIVER Test Arr. IMPELLER F2F9W

IMP. DIA. 22 3/32

Certified Correct by [illegible] 5/74 R.P.M. 705

ENGR. DATE



Colt Industries — Fairbanks Morse Pump Division

CERTIFIED CURVE

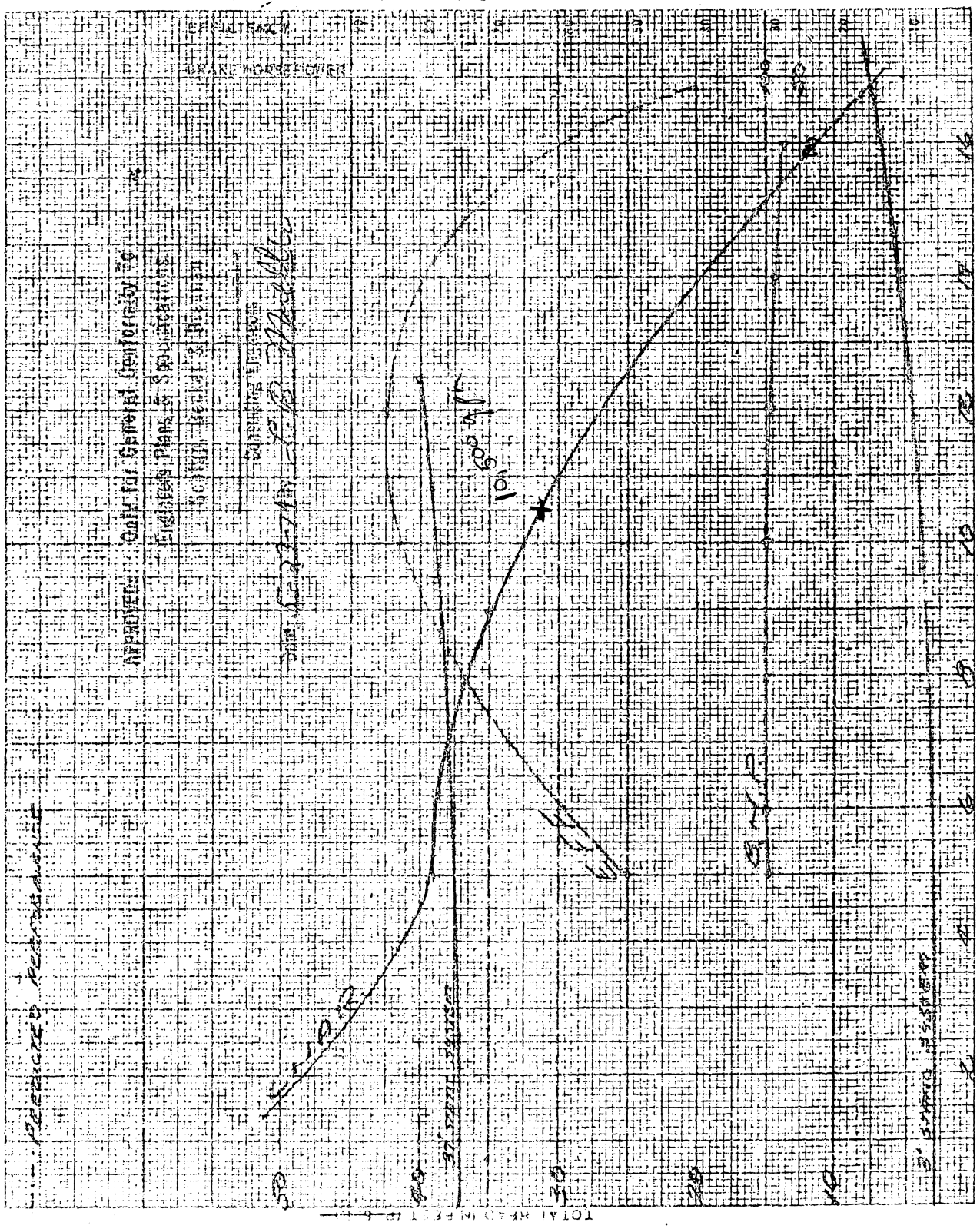
PUMP NO. K2VR-071170 STAGES 1 SIZE-FIGURE F18 - 6360

TESTED 5/16/74 DRIVER TMC - 450 IMPELLER F2E90

IMP. DIA. 19 1/2

Certified Correct by [signature] 5/20/74 R.P.M. 880

ENGR. DATE



Colt Industries — Fairbanks Morse Pump Division

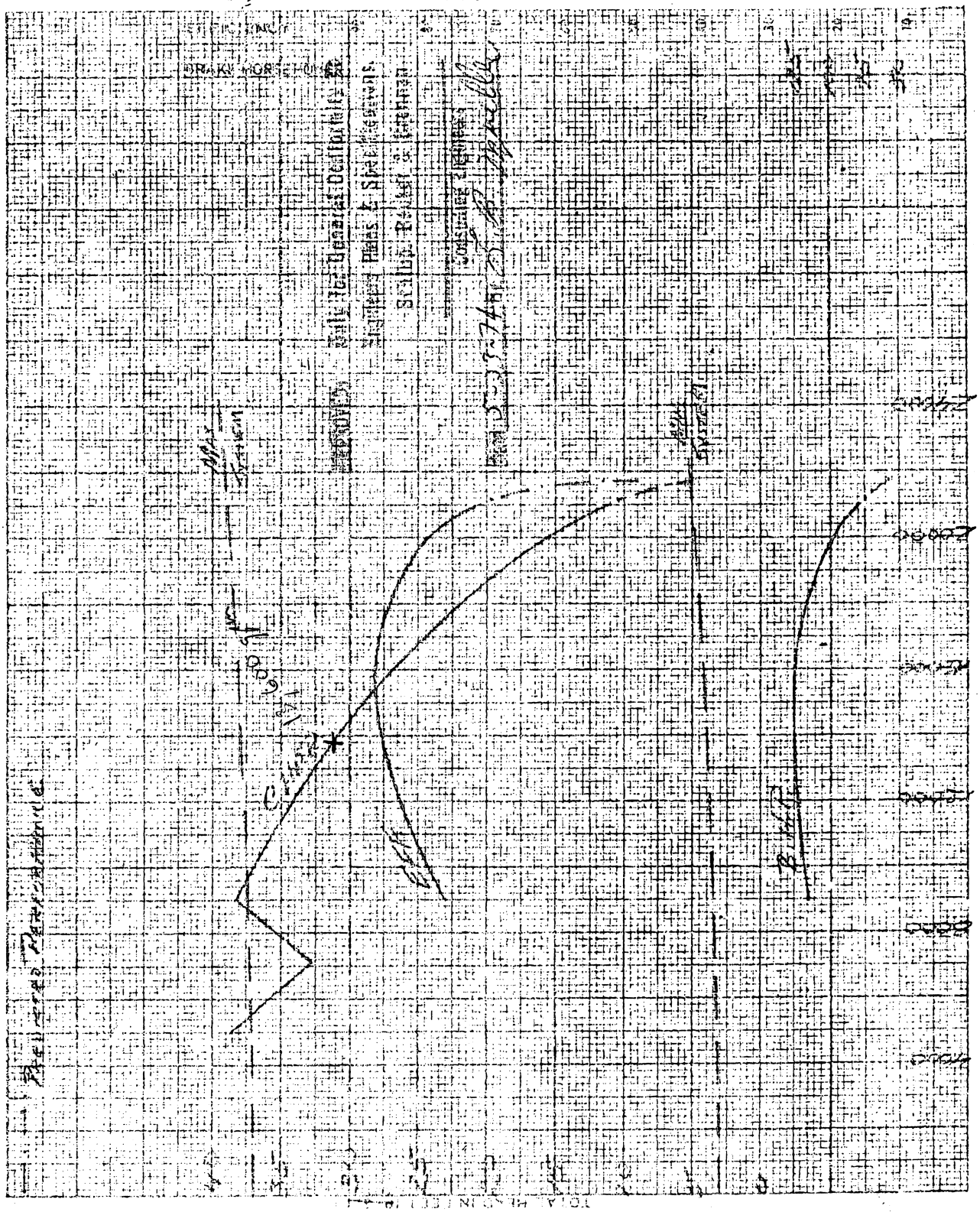
CERTIFIED CURVE

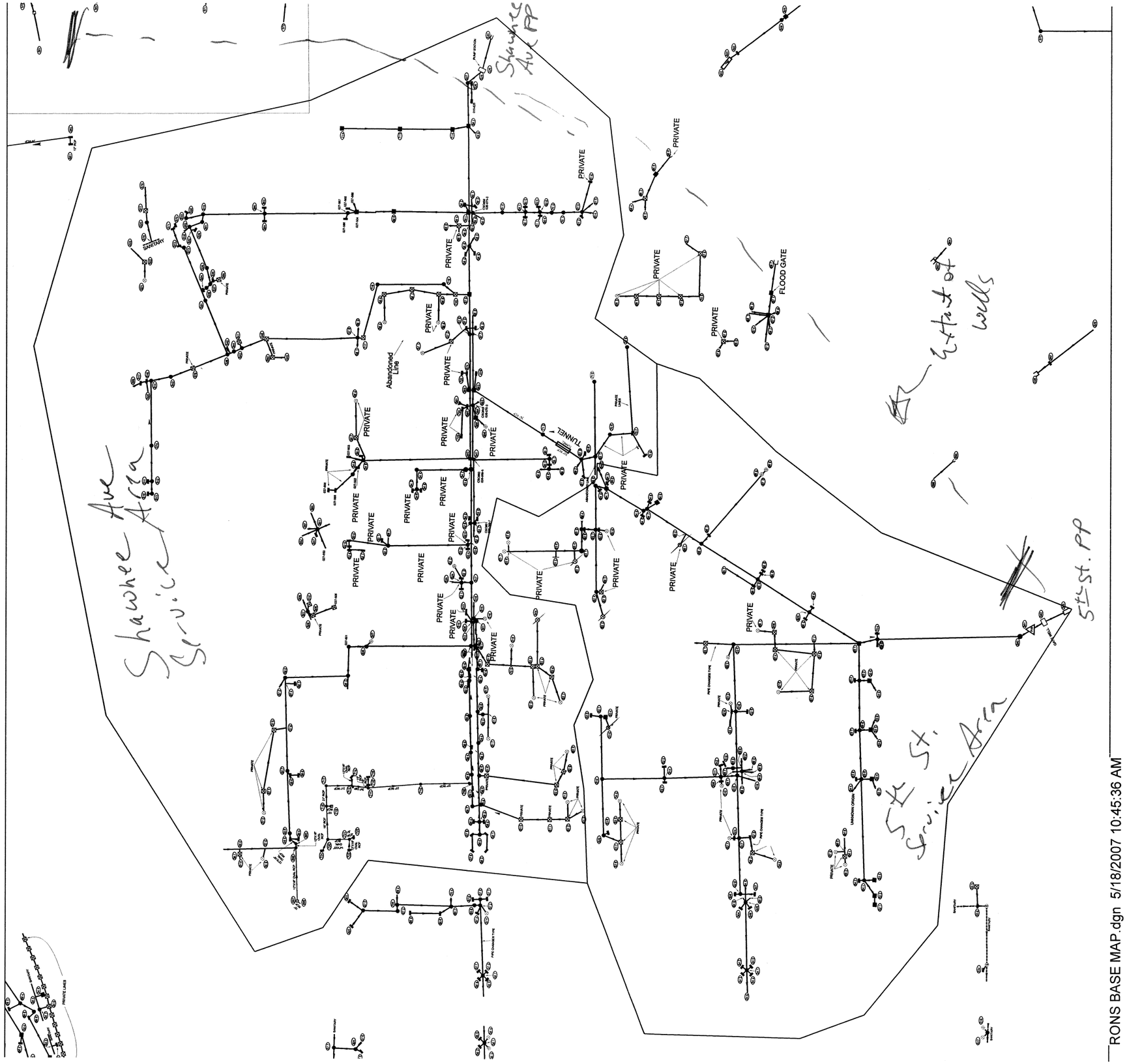
PUMP NO. [illegible] STAGES 1 SIZE-FIGURE 40-6360

TESTED [illegible] DRIVER [illegible] IMPELLER EZ FPN

IMP. DIA. 22 3/32

Certified Correct by [illegible] ENGR. [illegible] DATE R.P.M. 765





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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Existing Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Pump Curve Table
Pump Number 1
(Test Pump # K2V2-071100)

Head	Capacity gpm
6.5	17,100
10.0	16,300
12.0	15,800
20.0	
25.0	
30.0	11,000
35.0	8,800
37.0	8,000
40.0	
42.0	3,700
50.0	

Pump Curve Table
Pump Number 2-4

Head	K2V2-071099-0 Capacity gpm	K2V2-071099-1 Capacity gpm	K2V2-071099-2 Capacity gpm	Average Capacity gpm
5.0	21,000	21,000	20,700	20,900
10.0	1			1
12.0	20,600	20,600	20,800	20,667
20.0	1			1
25.0	1			1
30.0	14,200	14,200	14,400	14,267
35.0	11,400	11,200	11,400	11,333
37.0	10,200	9,800	9,900	9,967
40.0	1			1
45.0	1			1
50.0	1			1

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PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
New Pump Station

Relief Wells and Collector Pipe

WELLS

The following information was supplied by EC-GD regarding relief wells
This is the n500+3 design condition for the EXISTING line of relief wells

Flow from each well	0.75	cfs
Number of wells	47	
Total system flow	35	cfs

HEADER

Assumptions/Notes

Pump station is located within the of area of concern. Therfore two header systems service the pump station. Each system will handle a portion of the flow.
Per UHC 3-240-07FA header pipe is designed to flow at 80% depth of pipe diameter.

Reach Description	Number of wells	Flow	
From sta 190+75 to pump station	27	20.1	cfs
From pump station to sta 246+35	20	14.9	cfs

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PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Existing Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Step 1 - Determine pump operating Ranges

Pump Number	Start Elevation	Stop Elevation	Description
1	732	724	$19^{1}/_{2}$" Impeller @ 880 RPM
2	733	725	$22^{3}/_{32}$" Impeller @ 705 RPM
3	734	726	$22^{3}/_{32}$" Impeller @ 705 RPM
4	735	727	$22^{3}/_{32}$" Impeller @ 705 RPM

Assumptions

Since pump outlet piping is short (<10') dynamic heads were assumed to be negligible
Average pump curve developed for pumps 2-4 based on supplied pump curves.

Step 2 - Determine head pumps will operate at

Case 1A - Low river stage with large interior event (max capacity)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
740	735	5 *	17,100	20,900	79,800	178

* Off chart for pump 1 used 6.5' head

Case 1B - Low river stage with large interior event (min capacity with all pumps running)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
740	728	12	15,800	20,650	77,750	173

Case 2A - High river stage (max capacity)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
765.2	735	30.2	11,000	14,250	53,750	120

Case 2B - High river stage (min capacity with all pumps running)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
765.2	728	37.2	8,000	9,950	37,850	84

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Existing Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Step 3 - Determine pumping capacity for interior Drainage

Case Number	Total Flow cfs	Seepage Flow cfs	Pumped Interior Drainage cfs
1	178	0	178
2	173	0	173
3	120	35 *	85
4	84	35 *	49

Step 4 - Determine if sufficient redundancy for seepage flow exists.

Redundancy requirements state 100% capacity is required in the event of any one pump failure. Critical pump failure would be pump 2, 3 or 4.

Interior drainage flow for case 4	15.5	cfs*
Total inflow to station for case4	50.5	cfs
Pump station capacity at case 4	84	cfs
Capcity of pump 2 at case 4	22	cfs
Pump station capacity with pump 2 failure	62	cfs

REDUNDANCY REQUIREMENT MET

Assumptions
* 15.5cfs value obtained from 1949 Supplement on Interior Drainage, and considers coincident storm only. Any seepage flow from original design is now intercepted by exist. wells.

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
New Pump Station

Relief Wells and Collector Pipe

WELLS

The following information was supplied by EC-GD regarding EXISTING relief wells
but now flowing under higher head associated with n500+3

Flow from each well	1	cfs
Number of wells	47	
Total system flow	47	cfs

THERE IS ALSO a proposed line of surface discharging wells flowing at a total of 40cfs.
Roughly HALF of this flow will be in the basin flowing back to the Shawnee Ave pump plant.

HEADER

Assumptions/Notes

Pump station is located within the of area of concern. Therfore two header systems service the pump station. Each system will handle a portion of the flow.
Per UHC 3-240-07FA header pipe is designed to flow at 80% depth of pipe diameter.

Reach Description	Number of wells	Flow	
From sta 190+75 to pump station	27	27	cfs
From pump station to sta 246+35	20	20	cfs

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PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Future Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Step 1 - Determine pump operating Ranges

Pump Number	Start Elevation	Stop Elevation	Description
1	732	724	$19^{1}/_{2}$" Impeller @ 880 RPM
2	733	725	$22^{3}/_{32}$" Impeller @ 705 RPM
3	734	726	$22^{3}/_{32}$" Impeller @ 705 RPM
4	735	727	$22^{3}/_{32}$" Impeller @ 705 RPM

Assumptions

Since pump outlet piping is short (<10') dynamic heads were assumed to be negligible
Average pump curve developed for pumps 2-4 based on supplied pump curves.

Step 2 - Determine head pumps will operate at

Case 3A - Low river stage with large interior event (max capacity)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
740	735	5 *	17,100	20,900	79,800	178

* Off chart for pump 1 used 6.5' head

Case 3B - Low river stage with large interior event (min capacity with all pumps running)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
740	728	12	15,800	20,650	77,750	173

Case 4A - High river stage (max capacity)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
770.0	735	35	8,800	11,300	42,700	95

Case 4B - High river stage (min capacity with all pumps running)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
770.0	728	42	3,700	7,000	24,700	55

off chart - values are extrapolated
NOTE that this is less than optimum efficiency, and outside of desired range of pumps

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Future Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Step 3 - Determine pumping capacity for interior Drainage

Case Number	Total Cap'y cfs	Below-Grade Seepage Flow cfs	Above-grade Seepage Flow cfs	Capacity for Interior Drainage Flow cfs
1	178	0	0	178
2	173	0	0	173
3	95	47	20	28
4	55	47	20	-12

Step 4 - Determine if sufficient redundancy for seepage flow exists.

Redundancy requirements state 100% capacity is required in the event of any one pump failure. Critical pump failure would be pump 2, 3 or 4.

Interior drainage flow for case 4	15.5	cfs*
Surface discharge relief well flow	20	cfs
Total inflow to station for case4	83	cfs
Pump station capacity at case 4	55	cfs
Capcity of pump 2 at case 4	22	cfs
Pump station capacity with pump 2 failure	33	cfs

REDUNDANCY REQUIREMENT NOT MET

Step 5 - Select new pumps to meet redundancy requirements.

Pump 1 - remain unchanged
Pump 2 - Select FM 30" 580RPM 2 stage with A-331-T impellers.
Pump 3 - Select FM 30" 580RPM 2 stage with A-331-T impellers.
Pump 4 - Select FM 30" 580RPM 2 stage with A-331-T impellers.

Check for clear space between pump bowls. 1 bowl diameter required.
Pump centerline spacing 96" is adquate for spacing of 42" bowls.

Check for sumbergance 5'-4" required plus 1/2 bowl diameter off floor

Floor El	716	ft
Distance from floor	40	in
Submergance depth	64	in
Submergance EL	724.7	ft

Borderline Pump 2 stop at 725ft
Consider raising operational levels

Assumptions
* 15.5cfs value obtained from 1949 Supplement on Interior Drainage, and considers coincident storm only. Any seepage flow from original design is now intercepted by exist. wells.

Exhibit 3 Geotechnical Data

Hydraulic Grade Lines for Pump Plants

In addition to the overall underseepage analysis performed for the Armourdale Unit, the underseepage conditions were also evaluated specifically for all pump plants using the underseepage criteria discussed previously. The underseepage was evaluated for existing conditions, the n500+0, n500+3 and n500+5 river levels. For feasibility level detail, the cross section of the protection was not changed to reflect changes required for higher raises. The results of this analysis should still reasonably reflect the underseepage conditions that should be expected for river levels higher than existing conditions, and should be conservative.

The purpose of the analysis was to provide the Structural Engineers with an excess head value at the base of the natural blanket for the different river levels at each pump plant location. To accomplish this, hydraulic grade lines were computed at each pump plant location for each river level.

Pump Plants Analyzed for Underseepage

Pump Plant	Approximate Station
Osage	76+80
12th Street	129+20
Mill Street	156+75
5th Street	185+70
Midwest Cold Storage	194+50
Shawnee	230+80
Kansas City Southern	276+80
PBI Gordon	289+60
National Beef	295+80
Central Avenue	299+20

Armourdale Kansas - Hydraulic Grade Line for the Osage Pump Station Sta 76+80

ARM Osage PP 76+80.xls

NOMENCLATURE USAGE PUMP PLANT

FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD

Date : 2/11/2007

Checked by : Bellew EC-GD

Date : 02/26/2007 GMB

NOTES:
1. Replace floodwall with floodwall proposed
2. L_2 not revised for minor change due to floodwall raise
3. Design includes fully penetrating cutoff wall from station 62+00 to 83+00

Elevation Used

Top of Existing Levee:	772.4	(From O&M Manual)
500 year + 0':	774.0	
500 year + 3':	777.0	
500 year + 5':	779.0	
Ground Surface Land Side:	760.0	
Ground Surface River Side:	750.0	(Average blanket thickness over slope)
Bottom of Blanket:	736.0	
Top of Bedrock:	668.0	

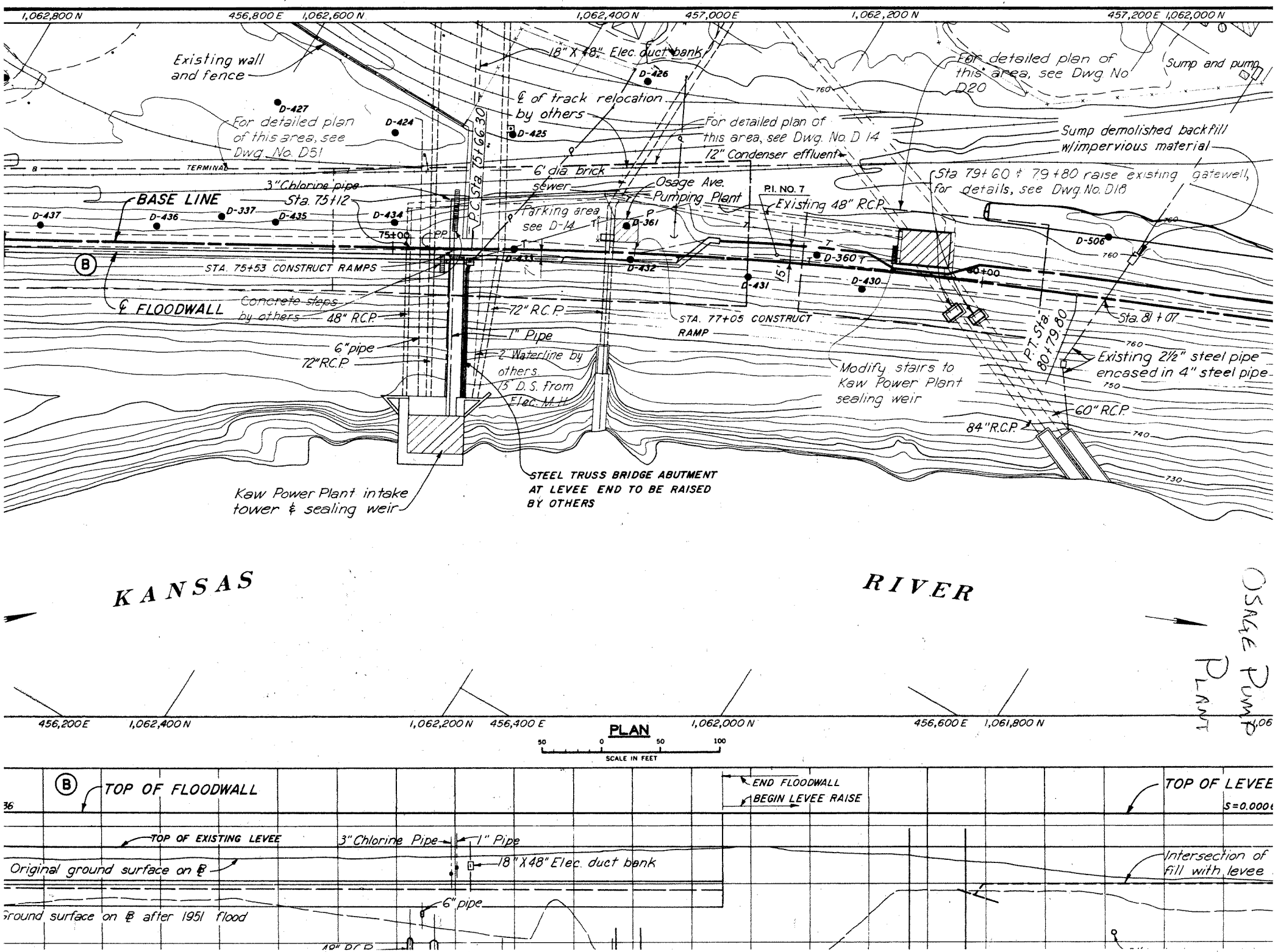
Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

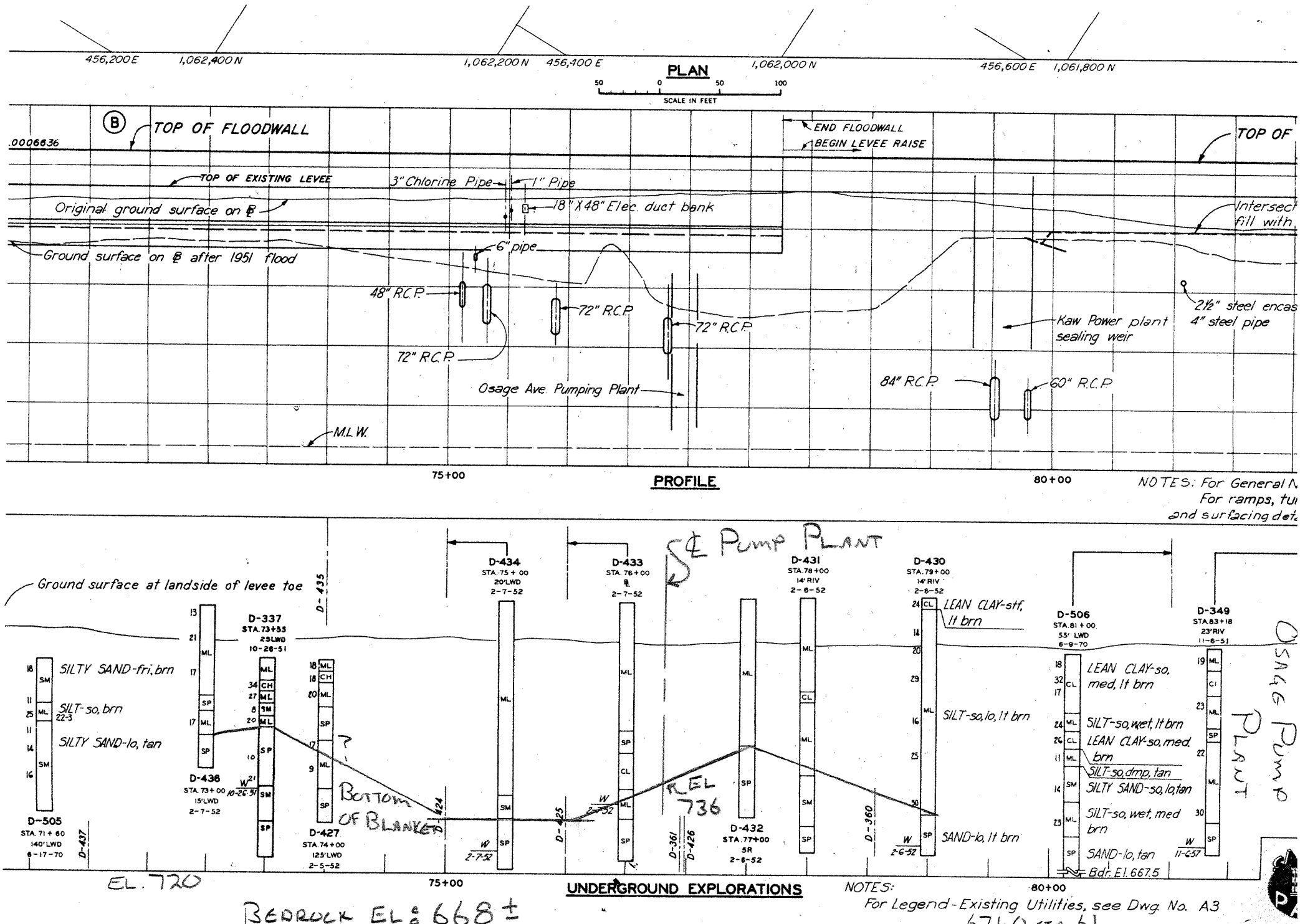
Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2L/C-1)}) / (e^{(2L/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_1	= Total Effective Length + 1/2 of Berm

Station			Input Parameters								
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks
76+80	Existing Conditions		300	300	24.0	19.0	14.0	68.0	100	12.4	EC
76+80	n500 + 0		300	300	24.0	19.0	14.0	68.0	100	14.0	n500
76+80	n500 +3		300	300	24.0	19.0	14.0	68.0	100	17.0	n500+3
76+80	n500 + 5		300	300	24.0	19.0	14.0	68.0	100	19.0	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
534	700	99	50	700	849	10.22	0.43	0.84	1.97		
534	700	99	680	700	1479	6.63	0.28	0.84	3.04		
534	700	99	680	700	1479	8.05	0.34	0.84	2.51		
534	700	99	680	700	1479	8.99	0.37	0.84	2.24		

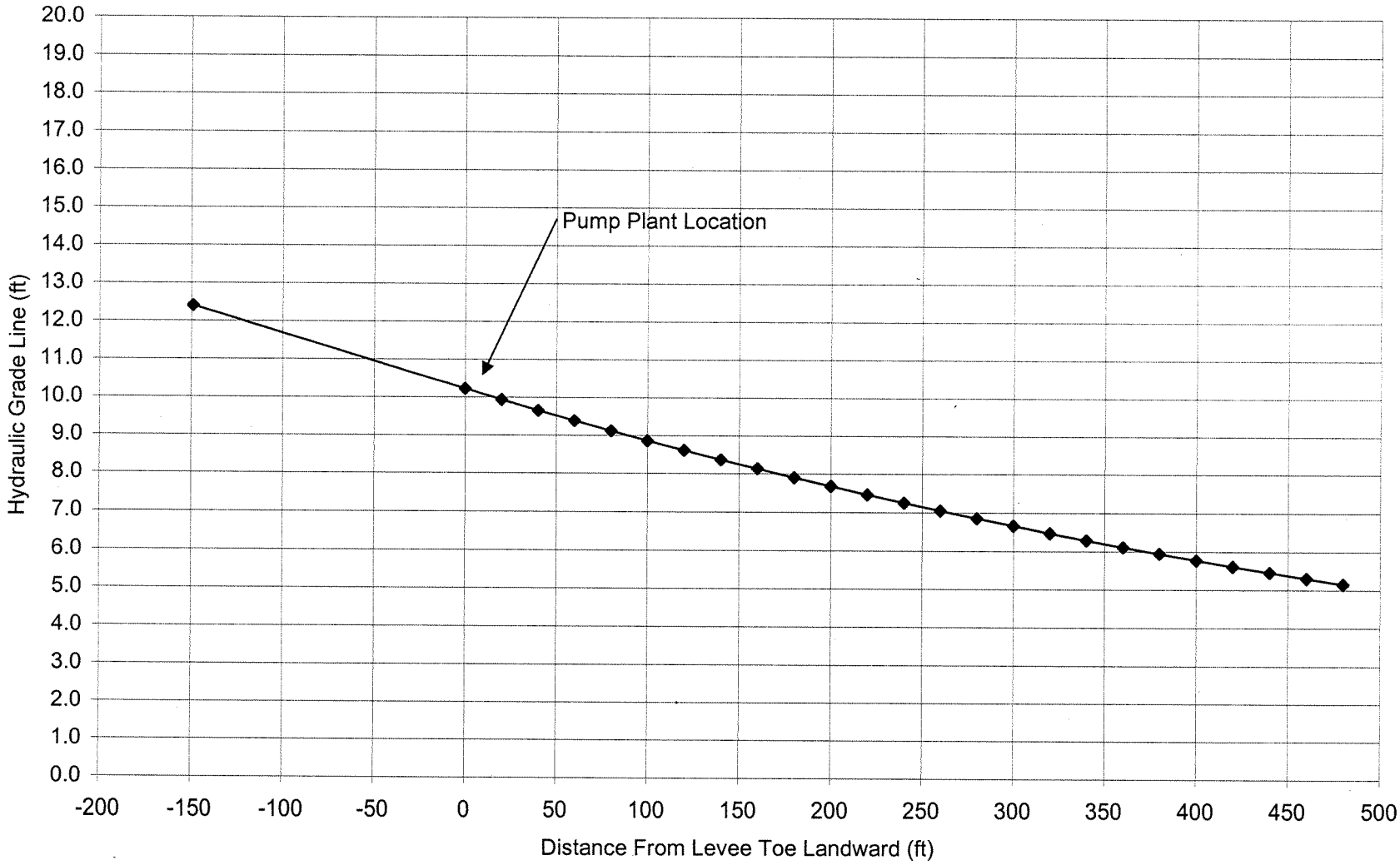

1,062,800 N
456,800 E
1,062,600 N
1,062,400 N
457,000 E
1,062,200 N
457,200 E
1,062,000 N
Existing wall and fence
18" X 48" Elec. duct bank
D-426
For detailed plan of this area, see Dwg No D20
Sump and pump
D-427
D-424
D-425
℄ of track relocation by others
For detailed plan of this area, see Dwg. No D 51
For detailed plan of this area, see Dwg. No. D 14
12" Condenser effluent
Sump demolished backfill w/impervious material
P.C. Sta. 75+66.30
TERMINAL
6' dia. brick sewer
Osage Ave. Pumping Plant
P.I. NO. 7
Existing 48" R.C.P.
Sta 79+60 & 79+80 raise existing gatewell, for details, see Dwg. No. D18
3" Chlorine pipe
Sta. 75+12
BASE LINE
D-437
D-436
D-337
D-435
D-434
Parking area see D-14
D-361
75+00
PP
D-506
D-433
D-432
D-360
D-431
D-430
80+00
STA. 75+53 CONSTRUCT RAMPS
℄ FLOODWALL
Concrete steps by others
48" R.C.P.
72" R.C.P.
1" Pipe
6" pipe
72" R.C.P.
2" Waterline by others. 15' D.S. from Elec. M.H.
STA. 77+05 CONSTRUCT RAMP
Modify stairs to Kaw Power Plant sealing weir
P.T. Sta. 80+79.80
Sta. 81+07
Existing 2½" steel pipe encased in 4" steel pipe
60" R.C.P.
84" R.C.P.
STEEL TRUSS BRIDGE ABUTMENT AT LEVEE END TO BE RAISED BY OTHERS
Kaw Power Plant intake tower & sealing weir
KANSAS
RIVER
Osage Pump Plant
456,200 E
1,062,400 N
1,062,200 N
456,400 E
1,062,000 N
456,600 E
1,061,800 N
PLAN
SCALE IN FEET
TOP OF FLOODWALL
END FLOODWALL
BEGIN LEVEE RAISE
TOP OF LEVEE
TOP OF EXISTING LEVEE
3" Chlorine Pipe
1" Pipe
Original ground surface on ℄
18" X 48" Elec. duct bank
Intersection of fill with levee
6" pipe
Ground surface on ℄ after 1951 flood


456,200 E
1,062,400 N
1,062,200 N
456,400 E
1,062,000 N
456,600 E
1,061,800 N
PLAN
SCALE IN FEET
TOP OF FLOODWALL
END FLOODWALL
BEGIN LEVEE RAISE
TOP OF
TOP OF EXISTING LEVEE
3" Chlorine Pipe
1" Pipe
Original ground surface on ℄
18" X 48" Elec. duct bank
Intersect
fill with
Ground surface on ℄ after 1951 flood
6" pipe
48" R.C.P.
72" R.C.P.
72" R.C.P.
72" R.C.P.
Kaw Power plant
sealing weir
2½" steel encas
4" steel pipe
Osage Ave. Pumping Plant
84" R.C.P.
60" R.C.P.
M.L.W.
75+00
PROFILE
80+00
NOTES: For General N
For ramps, tu
and surfacing det
Ground surface at landside of levee toe
D-505
STA. 71+60
140' LWD
6-17-70
SILTY SAND-fri, brn
SILT-so, brn
SILTY SAND-lo, tan
D-436
STA. 73+00
15' LWD
2-7-52
D-337
STA. 73+55
25 LWD
10-26-51
D-427
STA. 74+00
12.5' LWD
2-5-52
Bottom of Blanket
D-434
STA. 75+00
20' LWD
2-7-52
D-433
STA. 76+00
2-7-52
℄ Pump Plant
REL 736
D-432
STA. 77+00
5R
2-6-52
D-431
STA. 78+00
14' RIV
2-6-52
D-430
STA. 79+00
14' RIV
2-6-52
LEAN CLAY-stf, lt brn
SILT-so, lo, lt brn
SAND-lo, lt brn
D-506
STA. 81+00
55' LWD
6-9-70
LEAN CLAY-so, med, lt brn
SILT-so, wet, lt brn
LEAN CLAY-so, med. brn
SILT-so, dmp, tan
SILTY SAND-so, lo, tan
SILT-so, wet, med brn
SAND-lo, tan
Bdr. El. 667.5
D-349
STA. 83+18
23' RIV
11-6-51
Osage Pump Plant
D-435
D-437
D-424
D-425
D-361
D-426
D-360
EL. 720
75+00
UNDERGROUND EXPLORATIONS
80+00
NOTES:
For Legend-Existing Utilities, see Dwg. No. A3
BEDROCK EL: 668±
671.0 STA 61

Hydraulic Grade Line for the Osage Pump Plant
Existing Conditions

Elevation Top of levee (MSL)	772.4	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	736		
Driving Head (ft)	12.4		
L1 (ft)	99		
L2 (ft)	50		
Le (ft)	700		
ho	10.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-149	12.40	772.40
Pump Plant Location	0	10.22	770.22
	20	9.94	769.94
	40	9.66	769.66
	60	9.38	769.38
	80	9.12	769.12
	100	8.86	768.86
	120	8.61	768.61
	140	8.37	768.37
	160	8.13	768.13
	180	7.91	767.91
	200	7.68	767.68
	220	7.47	767.47
	240	7.26	767.26
	260	7.05	767.05
	280	6.85	766.85
	300	6.66	766.66
	320	6.47	766.47
	340	6.29	766.29
	360	6.11	766.11
	380	5.94	765.94
	400	5.77	765.77
	420	5.61	765.61
	440	5.45	765.45
	460	5.30	765.30
	480	5.15	765.15

Existing Conditions Hydraulic Grade Line for the
Osage Pump Plant Sta 76+80
Armourdale Levee Unit

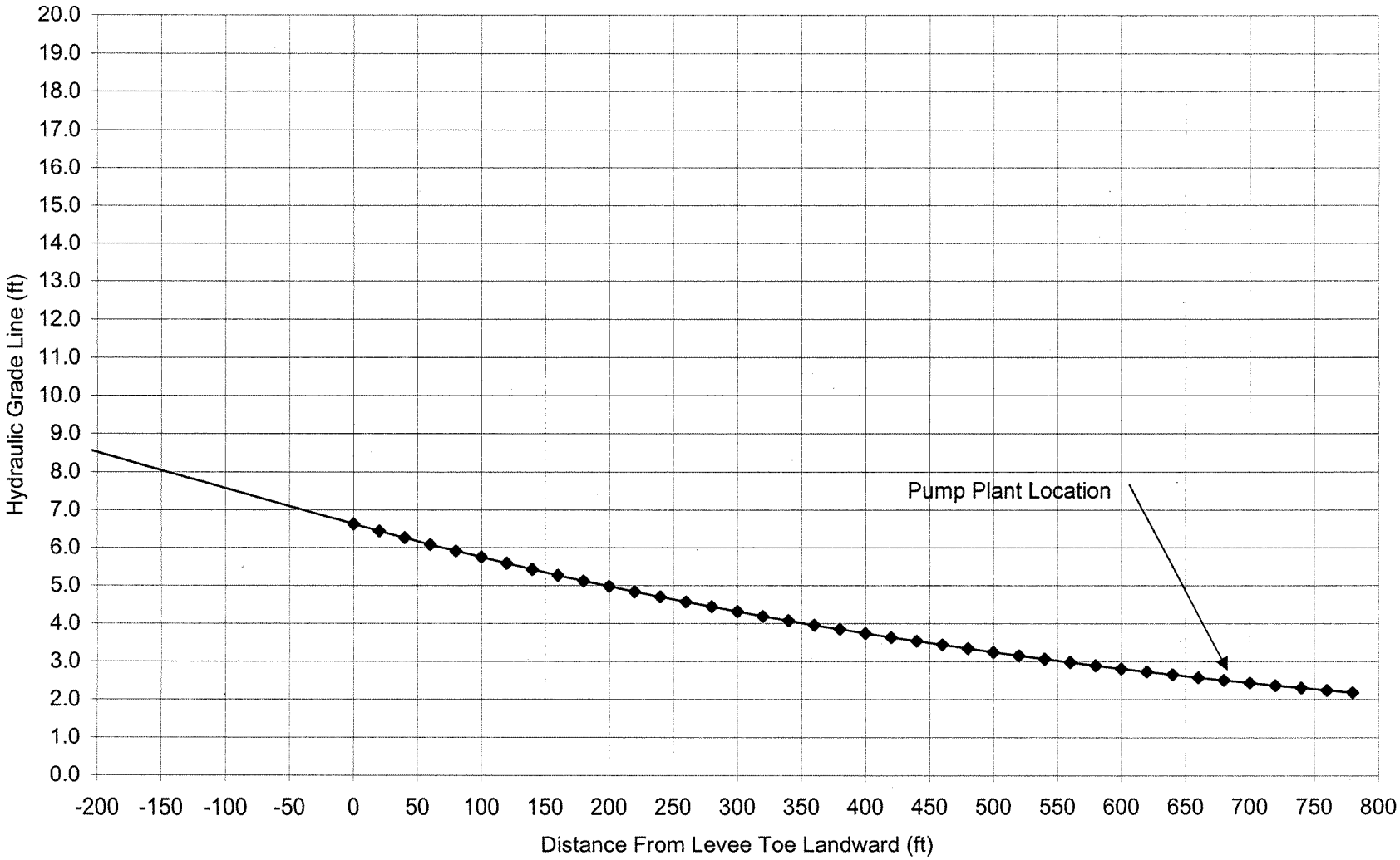


Hydraulic Grade Line for the Osage Pump Plant
Design Conditions n500+0 with cutoff wall

Elevation Top of levee (MSL)	774	n500+0
Elevation at Landside Levee Toe (MSL)	760	
Elevation Bottom of Blanket (MSL)	736	
Driving Head (ft)	14	
L1 (ft)	99	
L2 (ft)	680	
Le (ft)	700	
ho	6.6	

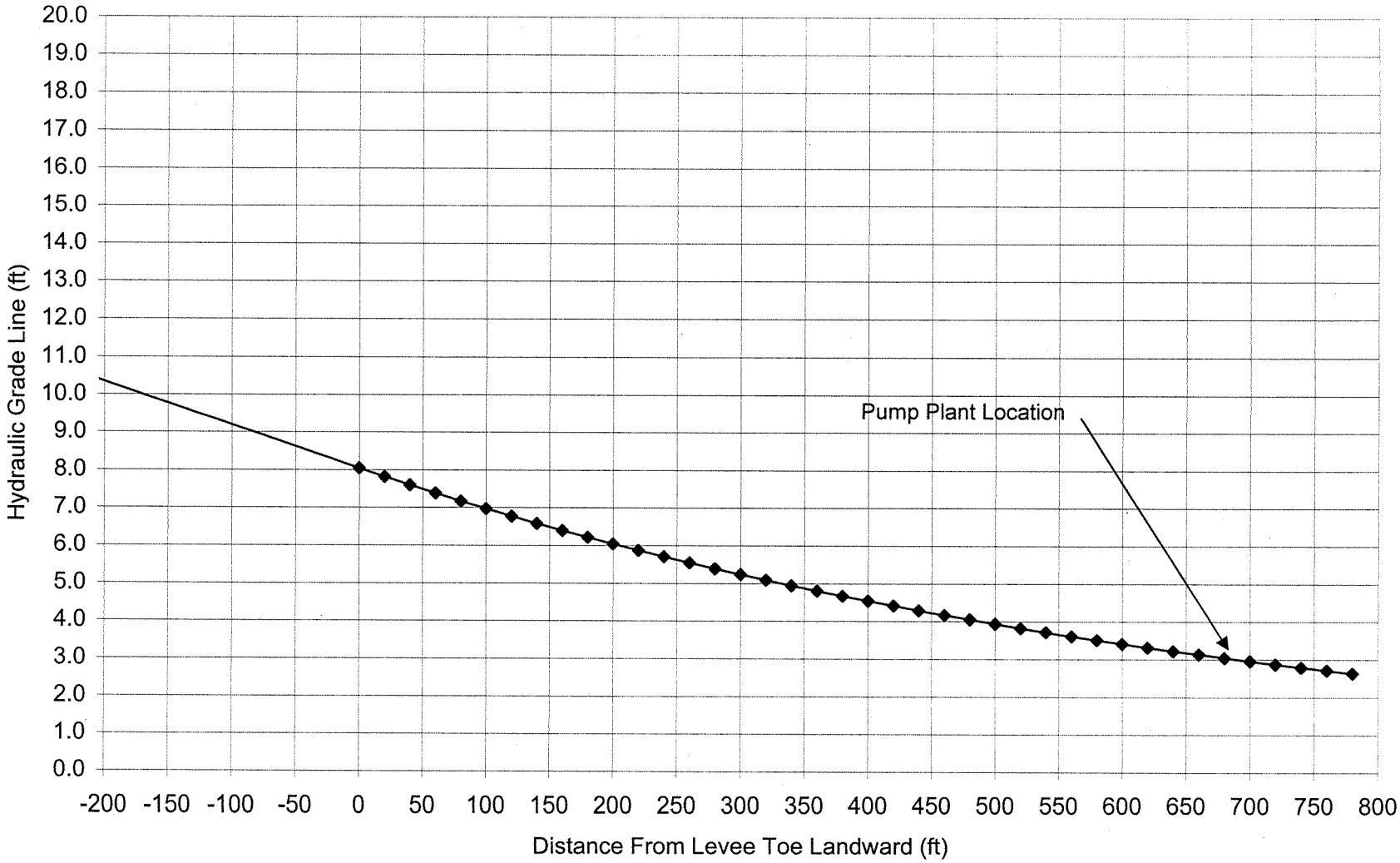
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-779	14.00	774.00
	0	6.63	766.63
	20	6.44	766.44
	40	6.26	766.26
	60	6.08	766.08
	80	5.91	765.91
	100	5.74	765.74
	120	5.58	765.58
	140	5.42	765.42
	160	5.27	765.27
	180	5.12	765.12
	200	4.98	764.98
	220	4.84	764.84
	240	4.70	764.70
	260	4.57	764.57
	280	4.44	764.44
	300	4.32	764.32
	320	4.19	764.19
	340	4.08	764.08
	360	3.96	763.96
	380	3.85	763.85
	400	3.74	763.74
	420	3.64	763.64
	440	3.53	763.53
	460	3.43	763.43
	480	3.34	763.34
	500	3.24	763.24
	520	3.15	763.15
	540	3.06	763.06
	560	2.98	762.98
	580	2.89	762.89
	600	2.81	762.81
	620	2.73	762.73
	640	2.65	762.65
	660	2.58	762.58
Pump Plant Location	680	2.51	762.51
	700	2.44	762.44
	720	2.37	762.37
	740	2.30	762.30
	760	2.24	762.24
	780	2.17	762.17

Design Condition n500+0 Hydraulic Grade Line for the
Osage Pump Plant Sta 76+80
Armourdale Levee Unit



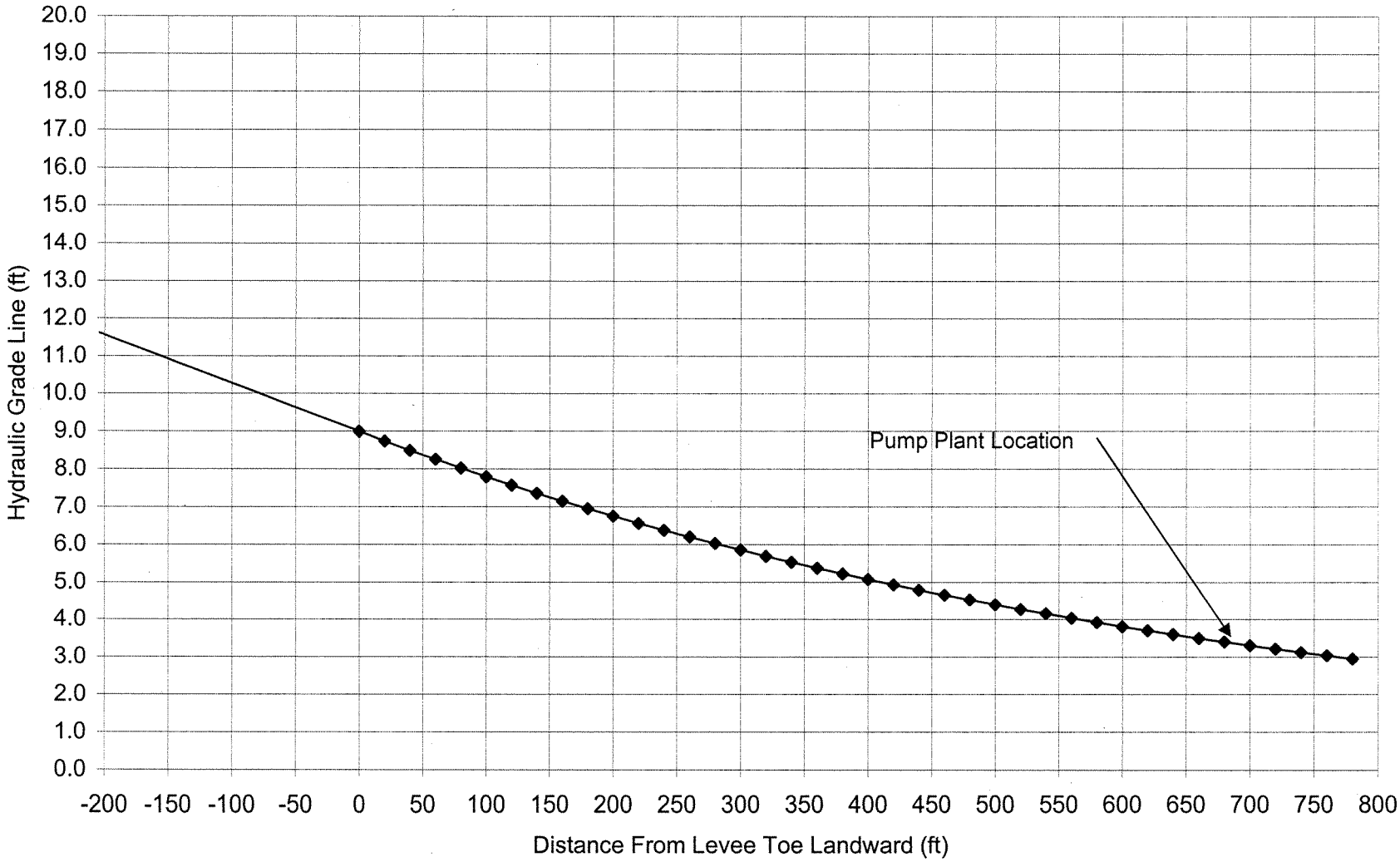
Hydraulic Grade Line for the Osage Pump Plant Design Conditions n500+3 with cutoff wall			
Elevation New Top of levee (MSL)	777	n500 +3	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	736		
Driving Head (ft)	17		
L1 (ft)	99		
L2 (ft)	680		
Le (ft)	700		
ho	8.05		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-779	17.00	777.00
	0	8.05	768.05
	20	7.82	767.82
	40	7.60	767.60
	60	7.38	767.38
	80	7.18	767.18
	100	6.97	766.97
	120	6.78	766.78
	140	6.59	766.59
	160	6.40	766.40
	180	6.22	766.22
	200	6.04	766.04
	220	5.87	765.87
	240	5.71	765.71
	260	5.55	765.55
	280	5.39	765.39
	300	5.24	765.24
	320	5.09	765.09
	340	4.95	764.95
	360	4.81	764.81
	380	4.67	764.67
	400	4.54	764.54
	420	4.41	764.41
	440	4.29	764.29
	460	4.17	764.17
	480	4.05	764.05
	500	3.94	763.94
	520	3.83	763.83
	540	3.72	763.72
	560	3.61	763.61
	580	3.51	763.51
	600	3.41	763.41
	620	3.32	763.32
	640	3.22	763.22
	660	3.13	763.13
Pump Plant Location	680	3.04	763.04
	700	2.96	762.96
	720	2.88	762.88
	740	2.79	762.79
	760	2.72	762.72
	780	2.64	762.64

Design Condition n500+3 Hydraulic Grade Line for the Osage Pump Plant Sta 76+80 Armourdale Levee Unit



Hydraulic Grade Line for the Osage Pump Plant Design Conditions n500+5 with cutoff wall			
Elevation New Top of levee (MSL)	779	n500 +5	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	736		
Driving Head (ft)	19		
L1 (ft)	99		
L2 (ft)	680		
Le (ft)	700		
ho	8.99		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-779	19.00	779.00
	0	8.99	768.99
	20	8.74	768.74
	40	8.49	768.49
	60	8.25	768.25
	80	8.02	768.02
	100	7.79	767.79
	120	7.57	767.57
	140	7.36	767.36
	160	7.15	767.15
	180	6.95	766.95
	200	6.76	766.76
	220	6.57	766.57
	240	6.38	766.38
	260	6.20	766.20
	280	6.03	766.03
	300	5.86	765.86
	320	5.69	765.69
	340	5.53	765.53
	360	5.38	765.38
	380	5.22	765.22
	400	5.08	765.08
	420	4.93	764.93
	440	4.79	764.79
	460	4.66	764.66
	480	4.53	764.53
	500	4.40	764.40
	520	4.28	764.28
	540	4.16	764.16
	560	4.04	764.04
	580	3.93	763.93
	600	3.81	763.81
	620	3.71	763.71
	640	3.60	763.60
	660	3.50	763.50
Pump Plant Location	680	3.40	763.40
	700	3.31	763.31
	720	3.21	763.21
	740	3.12	763.12
	760	3.03	763.03
	780	2.95	762.95

Design Condition n500+5 Hydraulic Grade Line for the Osage Pump Plant Sta 76+80 Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the 12th Street Pump Station Sta 129+20

ARM 12th Street PP 129+20.xls

NOMENCLATURE

12TH ST. PUMP PLANT

FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD

Date : 2/11/2007

Checked by : Bellew EC-GD GMB

Date : 02/12/07

NOTES:

1. Landside levee raise proposed
2. L_2 not revised for minor change due to levee raise

Elevation Used

Top of Existing Levee:	769.5 (From O&M Manual)
500 year + 0':	771.4
500 year + 3':	774.4
500 year + 5':	776.4
Ground Surface Land Side:	760.0
Ground Surface River Side:	742.0
Bottom of Blanket:	734.0
Top of Bedrock:	671.5

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

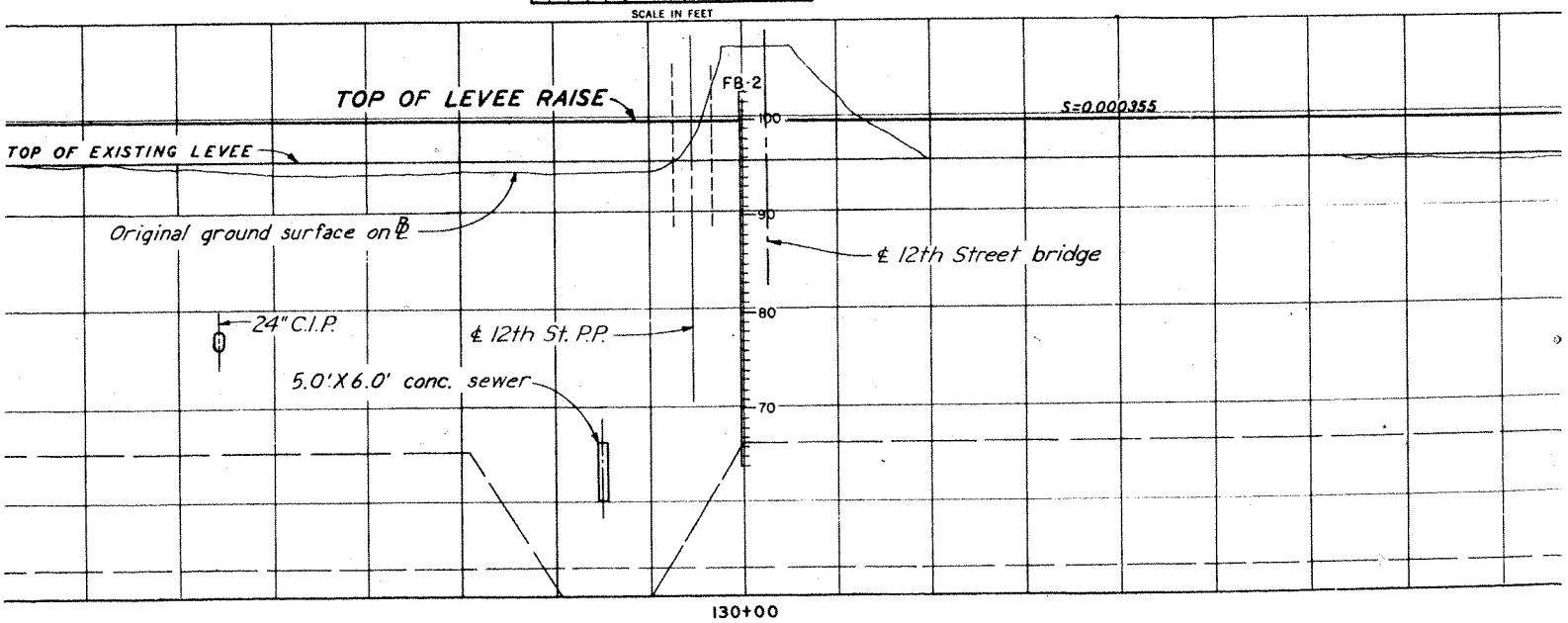
Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundati	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

			Input Parameters								
Station			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks
129+20	Existing Conditions		300	300	26.0	17.0	8.0	62.5	20	9.5	EC
129+20	n500 + 0		300	300	26.0	17.0	8.0	62.5	20	11.4	n500
129+20	n500 +3		300	300	26.0	17.0	8.0	62.5	20	14.4	n500+3
129+20	n500 + 5		300	300	26.0	17.0	8.0	62.5	20	16.4	n500+5

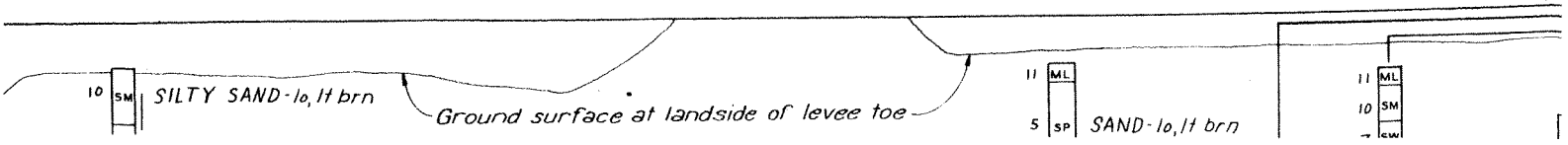
Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
387	698	20	120	698	838	7.91	0.30	0.84	2.76		
387	698	20	120	698	838	9.50	0.37	0.84	2.30		
387	698	20	120	698	838	12.00	0.46	0.84	1.82		
387	698	20	120	698	838	13.66	0.53	0.84	1.60		

PLAN

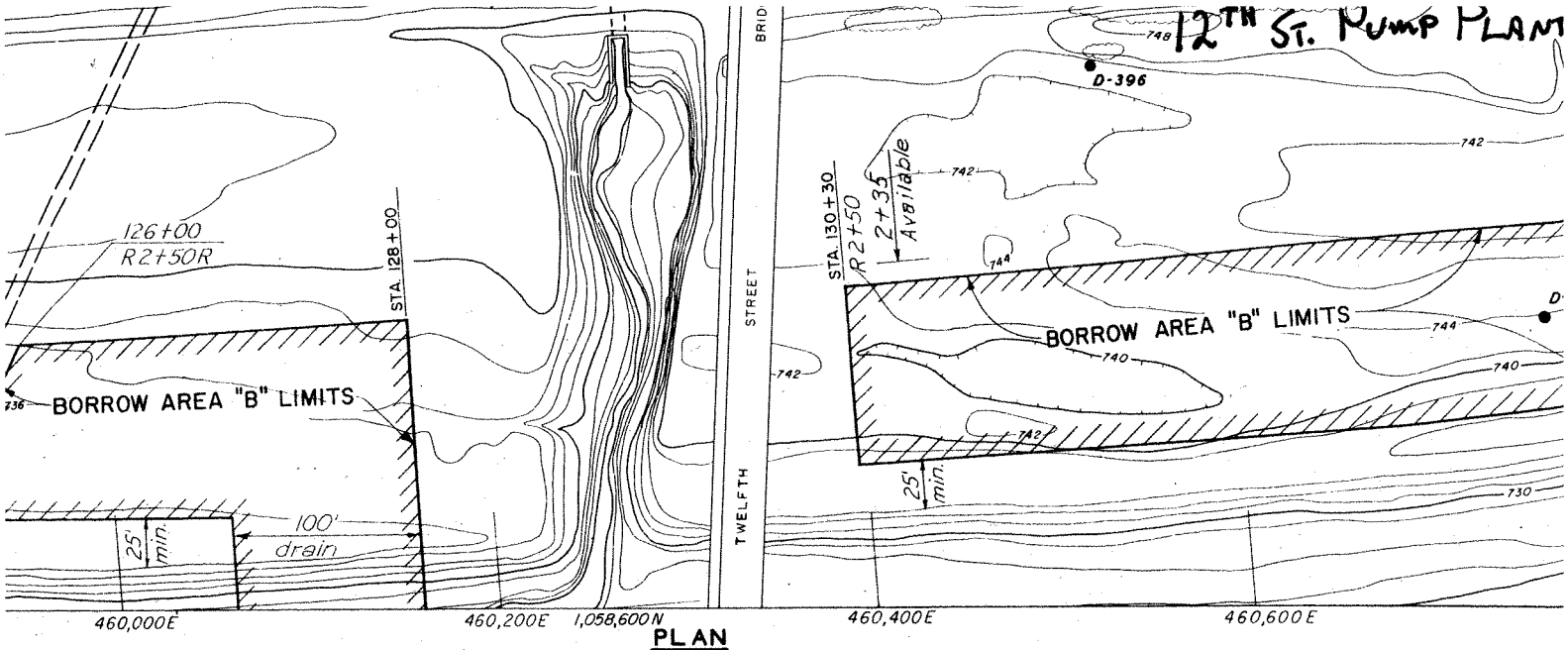
SCALE IN FEET



PROFILE

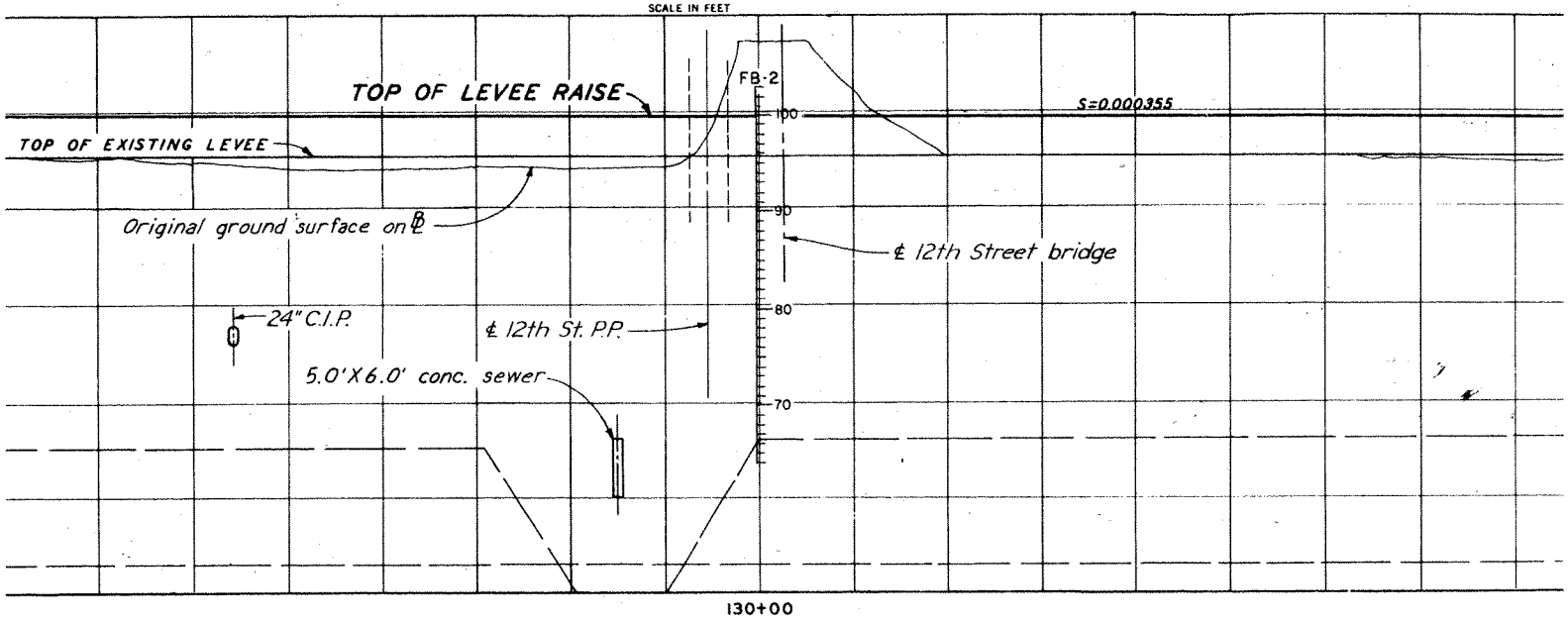


12TH ST. PUMP PLANT

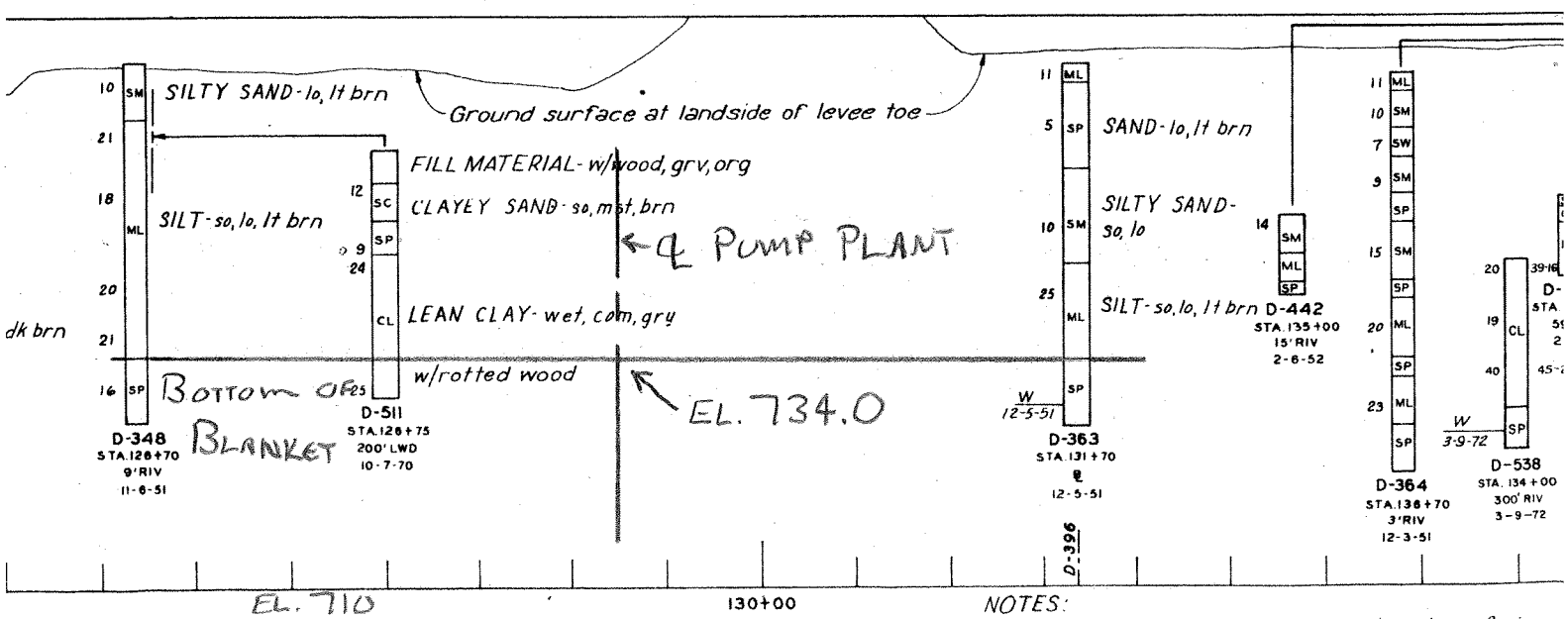


PLAN

SCALE IN FEET 50 0 50 100



PROFILE



UNDERGROUND EXPLORATIONS

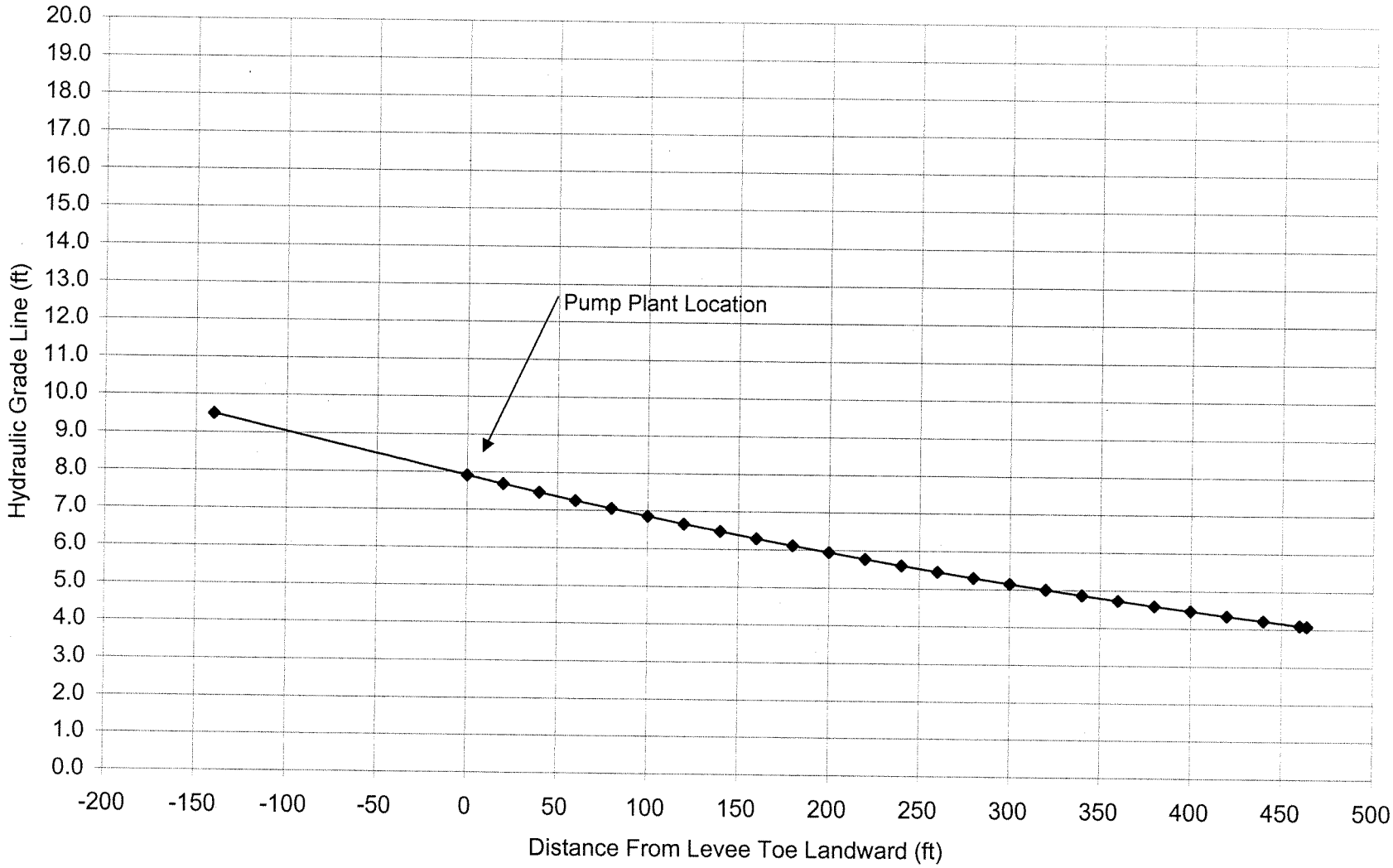
NOTES:
For ramps, turnouts, turnaround and surfacing details see Dwg. No C7

TOP OF BEDROCK: 671.5 ± 667.5 STA 80 672.6 STA 140

Hydraulic Grade Line for the 12th Street Pump Plant
Existing Conditions

Elevation Top of levee (MSL)	769.5	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	734		
Driving Head (ft)	9.5		
L1 (ft)	20		
L2 (ft)	120		
Le (ft)	698		
ho	7.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-140	9.50	769.50
Pump Plant Location	0	7.91	767.91
	20	7.69	767.69
	40	7.47	767.47
	60	7.26	767.26
	80	7.06	767.06
	100	6.86	766.86
	120	6.66	766.66
	140	6.48	766.48
	160	6.29	766.29
	180	6.12	766.12
	200	5.94	765.94
	220	5.77	765.77
	240	5.61	765.61
	260	5.45	765.45
	280	5.30	765.30
	300	5.15	765.15
	320	5.00	765.00
	340	4.86	764.86
	360	4.73	764.73
	380	4.59	764.59
	400	4.46	764.46
	420	4.34	764.34
	440	4.21	764.21
	460	4.09	764.09
	464	4.07	764.07

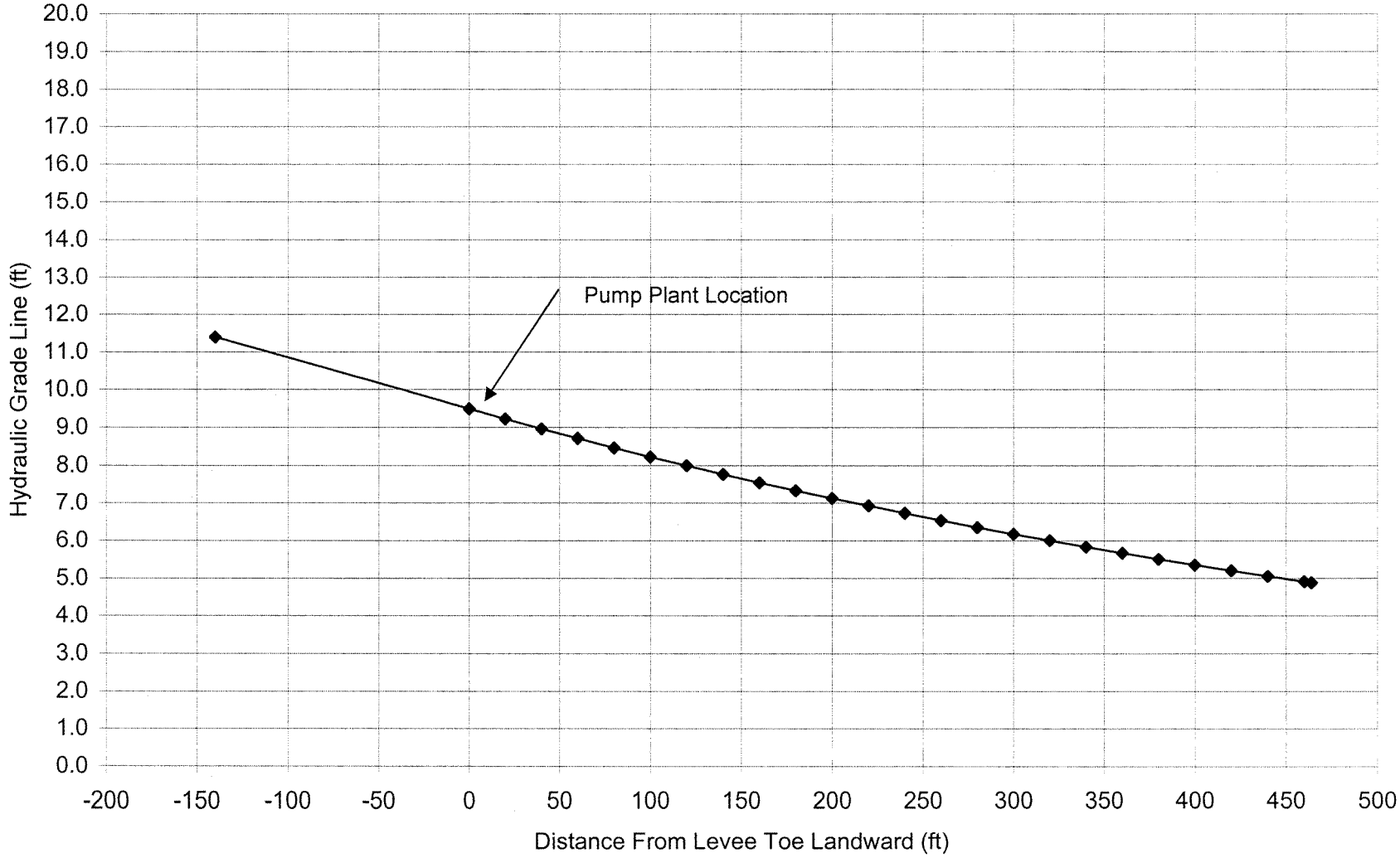
Existing Conditions Hydraulic Grade Line for the
12th Street Pump Plant Sta 129+20
Armourdale Levee Unit



Hydraulic Grade Line for the 12th Street Pump Plant
Design Conditions n500+0

Elevation Top of levee (MSL)	771.35	n500+0	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	734		
Driving Head (ft)	11.4		
L1 (ft)	20		
L2 (ft)	120		
Le (ft)	698		
ho	9.5		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-140	11.40	771.40
Pump Plant Location	0	9.50	769.50
	20	9.23	769.23
	40	8.97	768.97
	60	8.71	768.71
	80	8.47	768.47
	100	8.23	768.23
	120	8.00	768.00
	140	7.77	767.77
	160	7.55	767.55
	180	7.34	767.34
	200	7.13	767.13
	220	6.93	766.93
	240	6.73	766.73
	260	6.54	766.54
	280	6.36	766.36
	300	6.18	766.18
	320	6.00	766.00
	340	5.84	765.84
	360	5.67	765.67
	380	5.51	765.51
	400	5.35	765.35
	420	5.20	765.20
	440	5.06	765.06
	460	4.91	764.91
	464	4.89	764.89

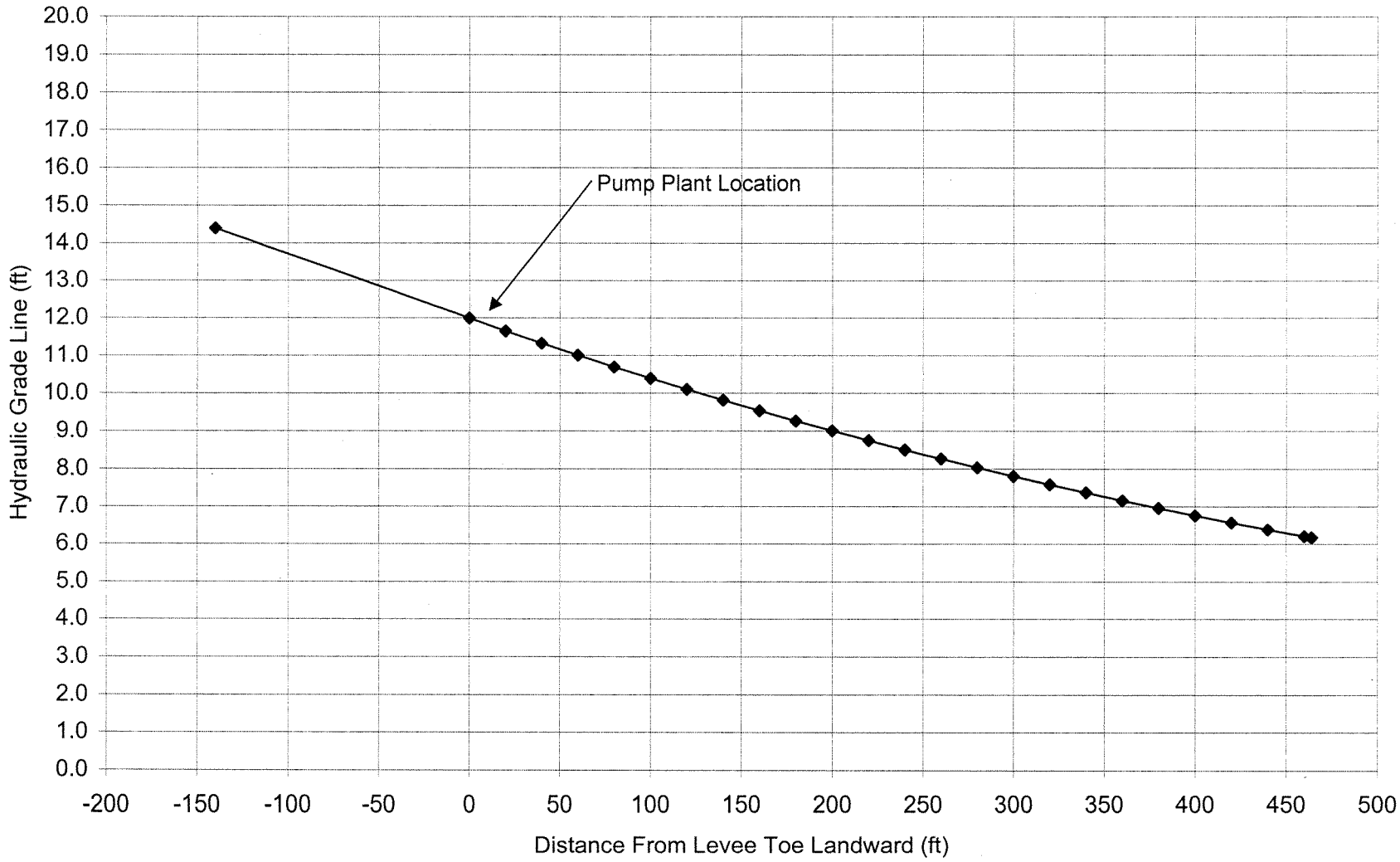
Design Condition n500+0 Hydraulic Grade Line for the
12th Street Pump Plant Sta 129+20
Armourdale Levee Unit



Hydraulic Grade Line for the 12th Street Pump Plant
Design Conditions n500+3

Elevation New Top of levee (MSL)	774.35	n500 +3	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	734		
Driving Head (ft)	14.4		
L1 (ft)	20		
L2 (ft)	120		
Le (ft)	698		
ho	12.00		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
	-140	14.40	774.40
	0	12.00	772.00
	20	11.66	771.66
	40	11.33	771.33
	60	11.01	771.01
	80	10.70	770.70
	100	10.39	770.39
	120	10.10	770.10
	140	9.82	769.82
	160	9.54	769.54
	180	9.27	769.27
	200	9.01	769.01
	220	8.75	768.75
	240	8.51	768.51
	260	8.27	768.27
	280	8.03	768.03
	300	7.81	767.81
	320	7.59	767.59
	340	7.37	767.37
	360	7.16	767.16
	380	6.96	766.96
	400	6.76	766.76
	420	6.57	766.57
	440	6.39	766.39
	460	6.21	766.21
	464	6.17	766.17

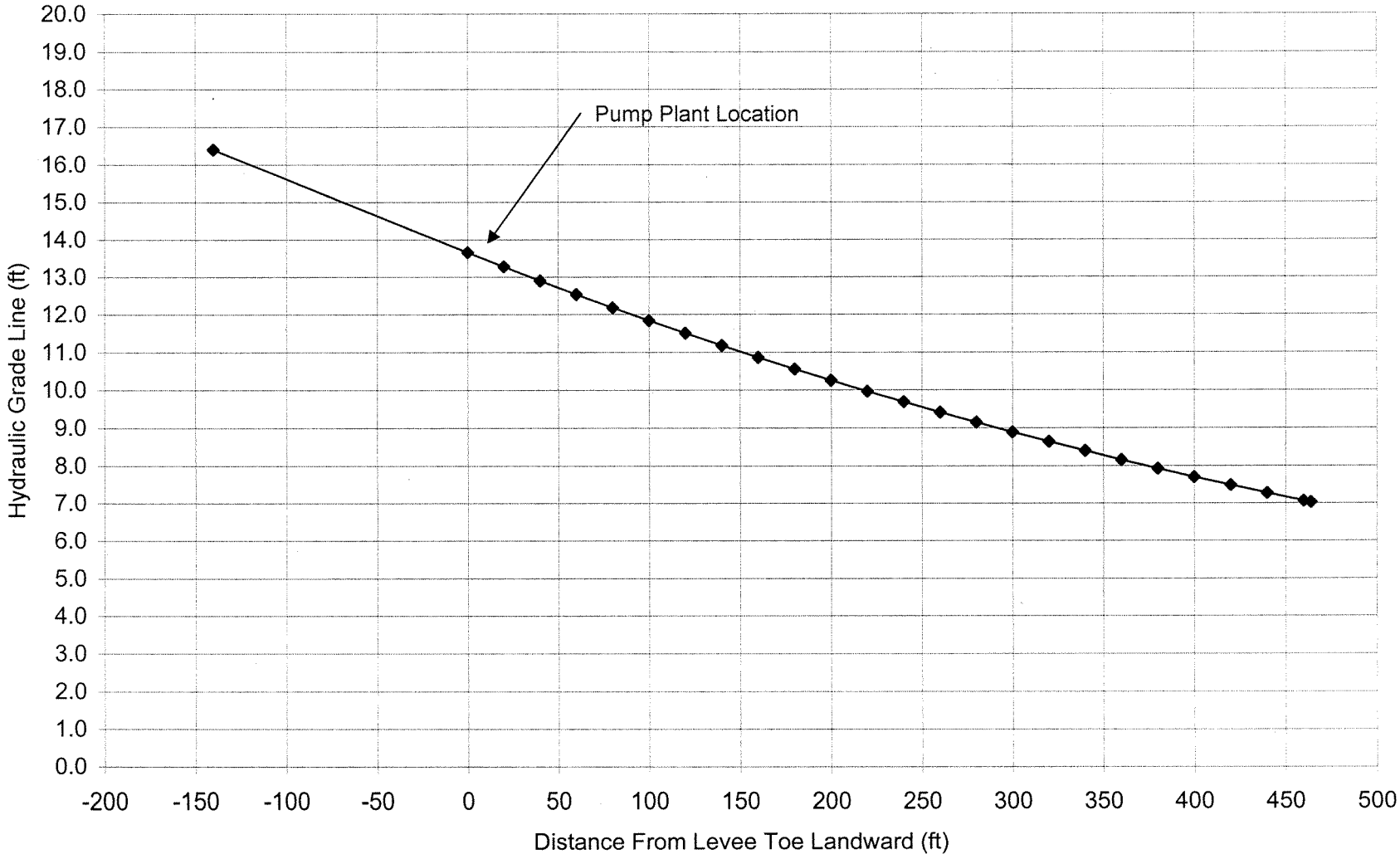
Design Condition n500+3 Hydraulic Grade Line for the
12th Street Pump Plant Sta 129+20
Armourdale Levee Unit



Hydraulic Grade Line for the 12th Street Pump Plant
Design Conditions n500+5

Elevation New Top of levee (MSL)	776.35	n500 +5	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	734		
Driving Head (ft)	16.4		
L1 (ft)	20		
L2 (ft)	120		
Le (ft)	698		
ho	13.66		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-140	16.40	776.40
Pump Plant Location	0	13.66	773.66
	20	13.28	773.28
	40	12.90	772.90
	60	12.54	772.54
	80	12.18	772.18
	100	11.84	771.84
	120	11.50	771.50
	140	11.18	771.18
	160	10.86	770.86
	180	10.56	770.56
	200	10.26	770.26
	220	9.97	769.97
	240	9.69	769.69
	260	9.41	769.41
	280	9.15	769.15
	300	8.89	768.89
	320	8.64	768.64
	340	8.39	768.39
	360	8.16	768.16
	380	7.93	767.93
	400	7.70	767.70
	420	7.49	767.49
	440	7.27	767.27
	460	7.07	767.07
	464	7.03	767.03

Design Condition n500+5 Hydraulic Grade Line for the
12th Street Pump Plant Sta 129+20
Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the Mill Street Pump Station Sta 156+75

ARM Mill Street PP 156+75.xls

NOMENCLATURE MILL ST PUMP PLANT FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD GMB
Date : 02/12/07

NOTES:
1. T-wall on existing levee raise proposed

Elevation Used

Top of Existing Levee:	768.7 (From O&M Manual)
500 year + 0':	770.3
500 year + 3':	773.3
500 year + 5':	775.3
Ground Surface Land Side:	760.0
Ground Surface River Side:	740.0
Bottom of Blanket:	725.0
Top of Bedrock:	667.0

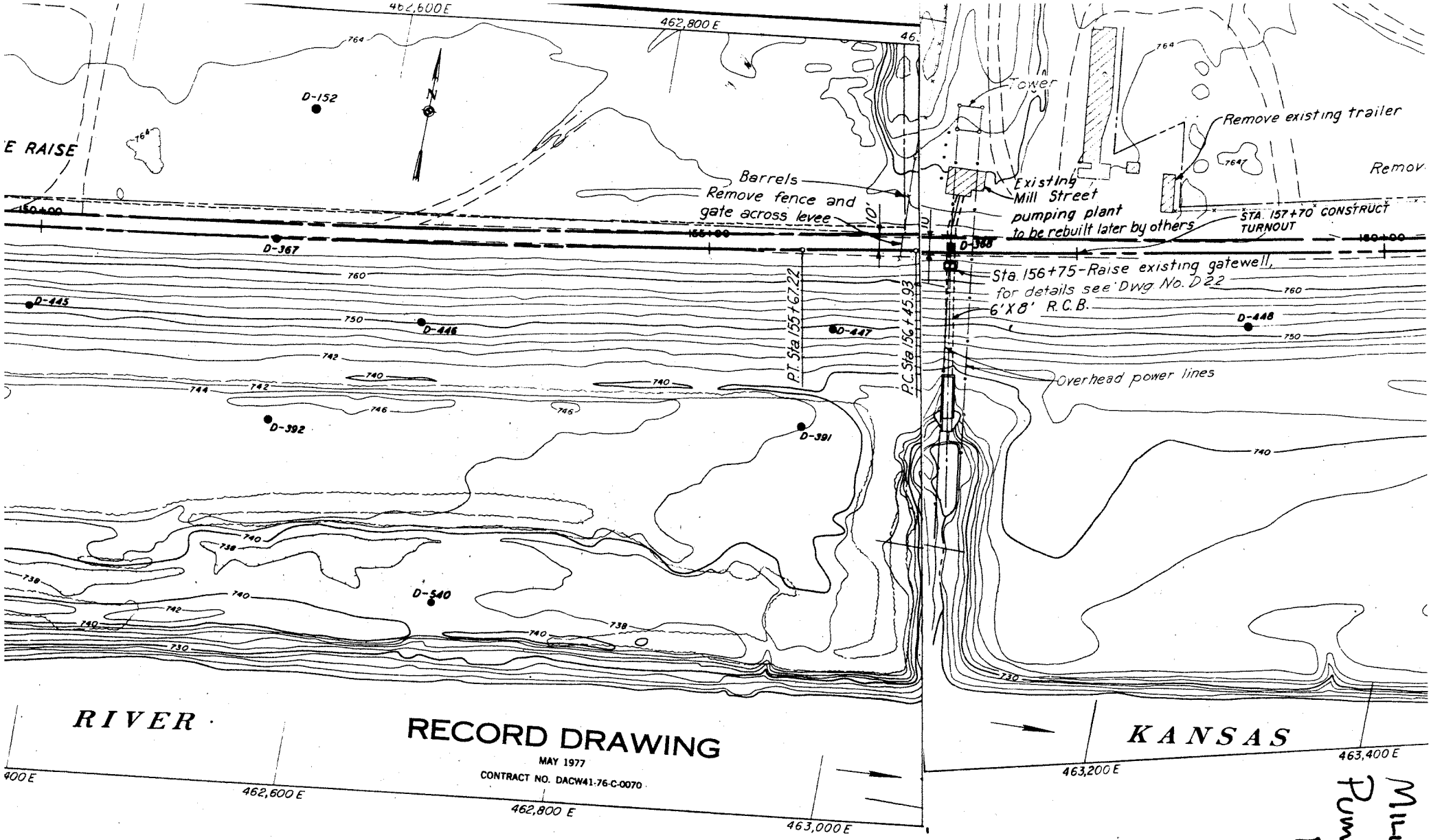
Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

Station			Input Parameters								Remarks
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	
156+75	Existing Conditions		300	300	35.0	25.0	15.0	58.0	75	8.7	EC
156+75	n500 + 0		300	300	35.0	25.0	15.0	58.0	75	10.3	n500
156+75	n500 +3		300	300	35.0	25.0	15.0	58.0	75	13.3	n500+3
156+75	n500 + 5		300	300	35.0	25.0	15.0	58.0	75	15.3	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
511	780	74	100	780	955	7.11	0.20	0.84	4.13		
511	780	74	100	780	955	8.42	0.24	0.84	3.49		
511	780	74	100	780	955	10.87	0.31	0.84	2.70		
511	780	74	100	780	955	12.50	0.36	0.84	2.35		



MILL STREET
PUMP PLANT
FEB 2007

Sta 156+50
El. 768.66

Sta. 156+50
El. 768.66

S=0.00

Original ground surface on ℄

6.0'X8.0' R.C.B.

MILL STREET PP
FEB 2007

155+00

160+00

℄ PUMP PLANT

NOTES:
For General Notes, see Dwg. No. A3
For Legend-Existing Utilities, see Dwg. No. A3

155+00

750
740
730
720

SYM.	DESCRIPTION	
	Revised for "As Built" conditions	9/
	REVISIONS	

KANSAS RIVER, KANSAS

KANSAS CITY, KANSAS - 1962 I
ARMOURDALE UNIT

PLAN, ~~PROFILE AND~~
UNDERGROUND EXPLORATIONS
STA. 138+50 TO STA. 156+50

In 109 sheets
Sheet No. 11
Dwg. No. A11

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

Submitted: A. L. Schuster
CHIEF, LOCAL PROTECTION SECTION

Recommended: R. D. Willey
CHIEF, FOUNDATIONS & MAT'LS BR.

Approved:
CHIEF, ENG

COMPILED BY: T.A.K.-J.A.H.
DRAWN BY: J.M.M.
CHECKED BY: R.L.B.

SAFETY PAYS

D-540
STA. 153+00
280' RIV
3-9-72

D-391
STA. 155 + 67
137' RIV
12-18-51

D-447
STA. 155+91
61' RIV
2-6-52

D-368
STA. 156+75
2' RIV
12-4-57

D-448
STA. 158+97
57' RIV
2-6-52

D-369
STA. 161+73
2' RIV
12-4-51

D-402

Sand-lo, brn

Silt-lo, sl pl, brn

EL. 725

BOTTOM OF BLANKET

BEDROCK EL: 667±

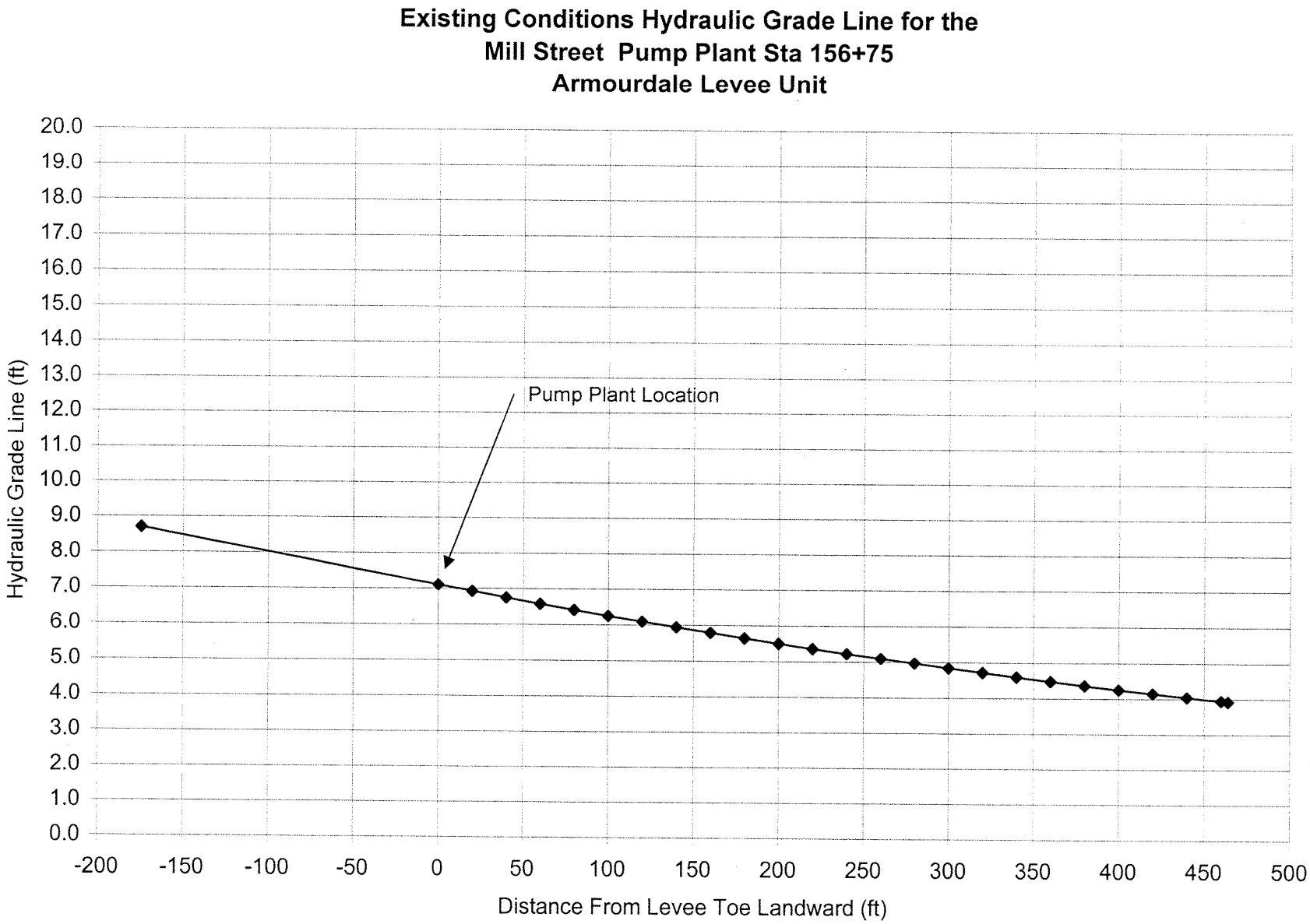
EL 672.6 STA 139
EL 655.0 STA 190

O & M M P I

Hydraulic Grade Line for the Mill Street Pump Plant
Existing Conditions

Elevation Top of levee (MSL)	768.65	Existing Conditions
Elevation at Landside Levee Toe (MSL)	760	
Elevation Bottom of Blanket (MSL)	725	
Driving Head (ft)	8.7	
L1 (ft)	74	
L2 (ft)	100	
Le (ft)	780	
ho	7.1	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-174	8.70	768.70
Pump Plant Location	0	7.11	767.11
	20	6.93	766.93
	40	6.76	766.76
	60	6.58	766.58
	80	6.42	766.42
	100	6.26	766.26
	120	6.10	766.10
	140	5.94	765.94
	160	5.79	765.79
	180	5.65	765.65
	200	5.50	765.50
	220	5.36	765.36
	240	5.23	765.23
	260	5.10	765.10
	280	4.97	764.97
	300	4.84	764.84
	320	4.72	764.72
	340	4.60	764.60
	360	4.48	764.48
	380	4.37	764.37
	400	4.26	764.26
	420	4.15	764.15
	440	4.05	764.05
	460	3.94	763.94
	464	3.92	763.92

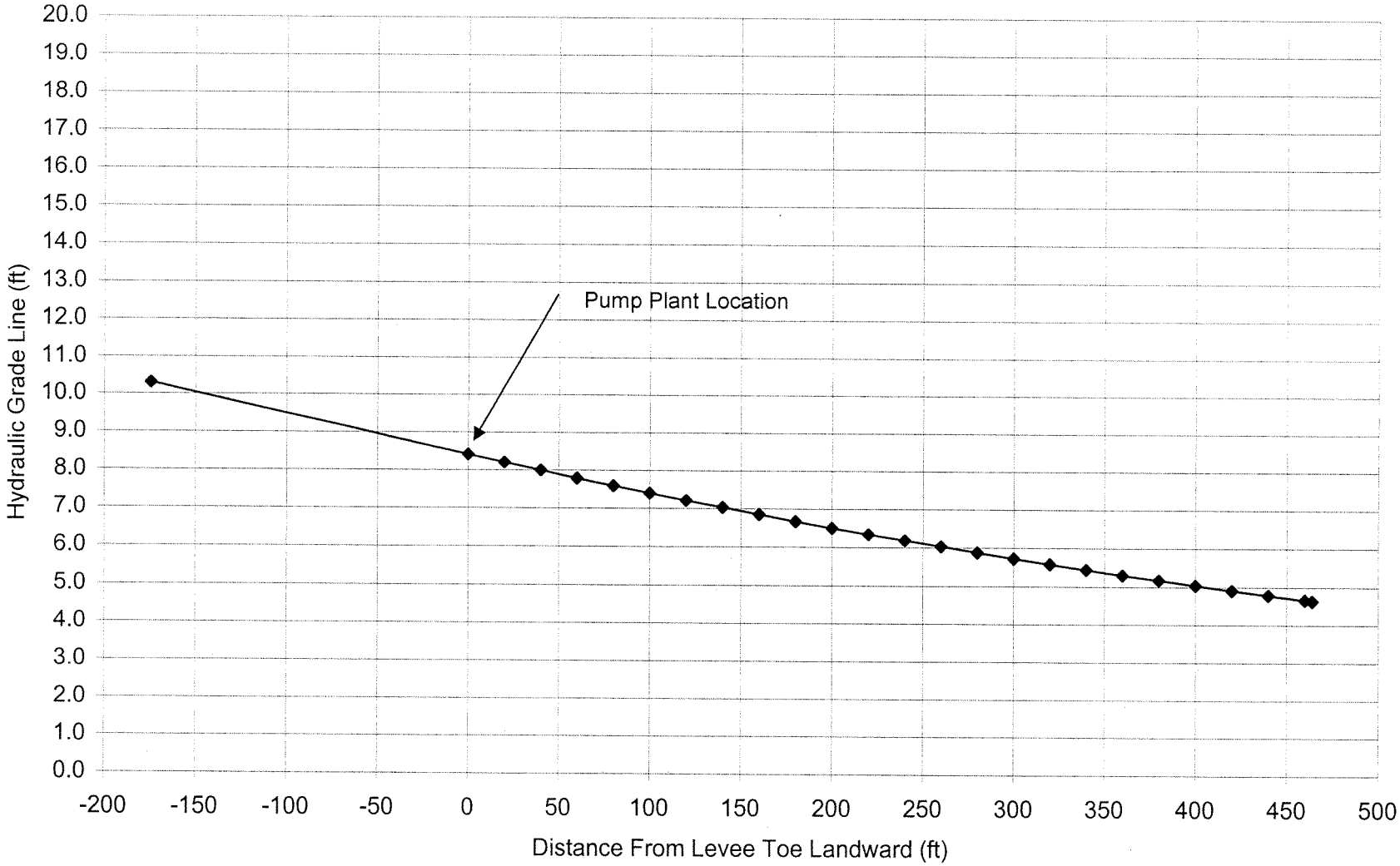

Existing Conditions Hydraulic Grade Line for the
Mill Street Pump Plant Sta 156+75
Armourdale Levee Unit
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Pump Plant Location
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the Mill Street Pump Plant
Design Conditions n500+0

Elevation Top of levee (MSL)	770.3	n500+0
Elevation at Landside Levee Toe (MSL)	760	
Elevation Bottom of Blanket (MSL)	725	
Driving Head (ft)	10.3	
L1 (ft)	74	
L2 (ft)	100	
Le (ft)	780	
ho	8.4	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-174	10.30	770.30
Pump Plant Location	0	8.42	768.42
	20	8.21	768.21
	40	8.00	768.00
	60	7.80	767.80
	80	7.60	767.60
	100	7.41	767.41
	120	7.22	767.22
	140	7.04	767.04
	160	6.86	766.86
	180	6.68	766.68
	200	6.51	766.51
	220	6.35	766.35
	240	6.19	766.19
	260	6.03	766.03
	280	5.88	765.88
	300	5.73	765.73
	320	5.59	765.59
	340	5.44	765.44
	360	5.31	765.31
	380	5.17	765.17
	400	5.04	765.04
	420	4.91	764.91
	440	4.79	764.79
	460	4.67	764.67
	464	4.65	764.65

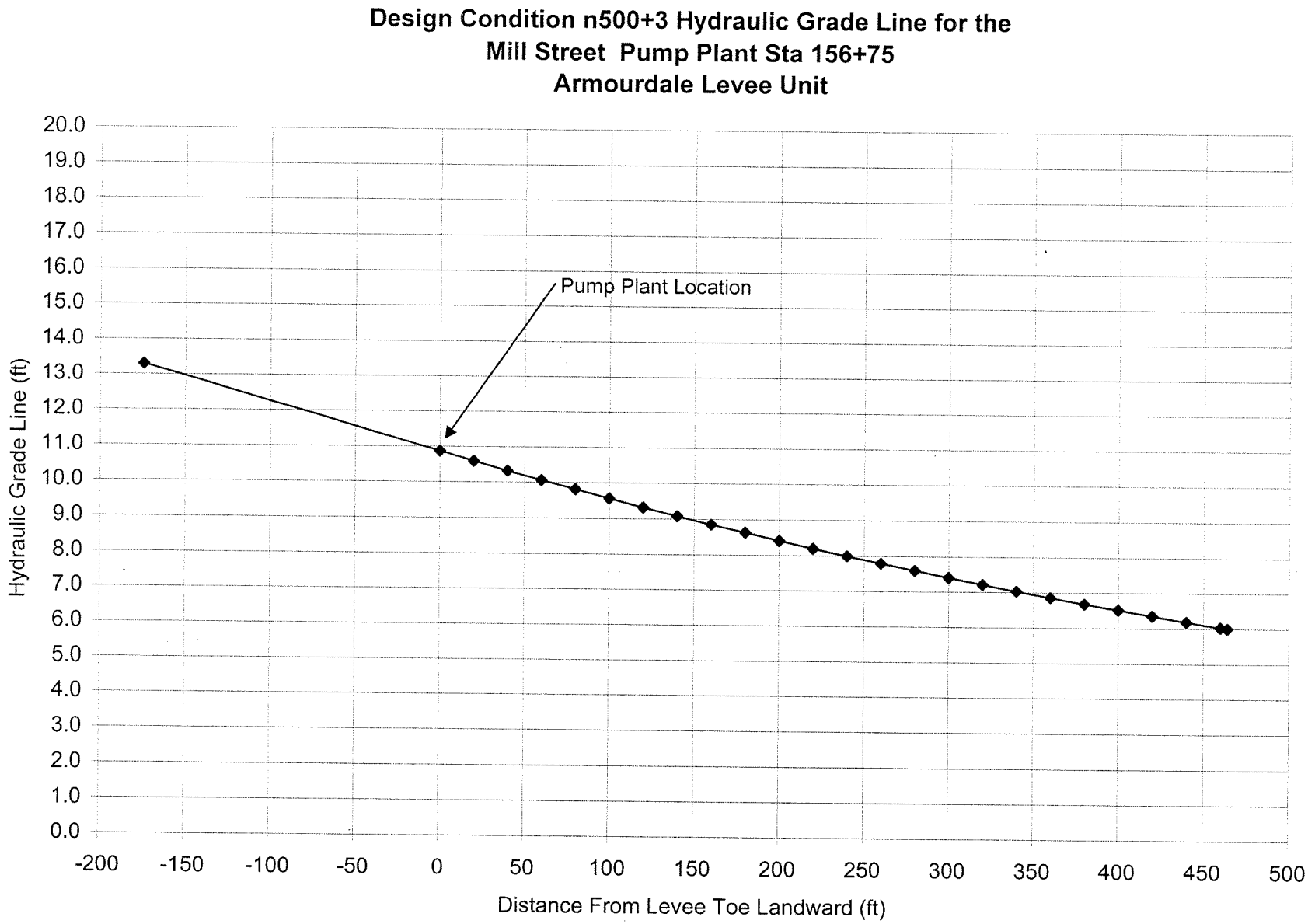
Design Condition n500+0 Hydraulic Grade Line for the
Mill Street Pump Plant Sta 156+75
Armourdale Levee Unit



Hydraulic Grade Line for the Mill Street Pump Plant
Design Conditions n500+3

Elevation New Top of levee (MSL)	773.3	n500 +3
Elevation at Landside Levee Toe (MSL)	760	
Elevation Bottom of Blanket (MSL)	725	
Driving Head (ft)	13.3	
L1 (ft)	74	
L2 (ft)	100	
Le (ft)	780	
ho	10.87	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
	-174	13.30	773.30
	0	10.87	770.87
	20	10.59	770.59
	40	10.33	770.33
	60	10.07	770.07
	80	9.81	769.81
	100	9.56	769.56
	120	9.32	769.32
	140	9.08	769.08
	160	8.85	768.85
	180	8.63	768.63
	200	8.41	768.41
	220	8.20	768.20
	240	7.99	767.99
	260	7.79	767.79
	280	7.59	767.59
	300	7.40	767.40
	320	7.21	767.21
	340	7.03	767.03
	360	6.85	766.85
	380	6.68	766.68
	400	6.51	766.51
	420	6.35	766.35
	440	6.19	766.19
	460	6.03	766.03
	464	6.00	766.00

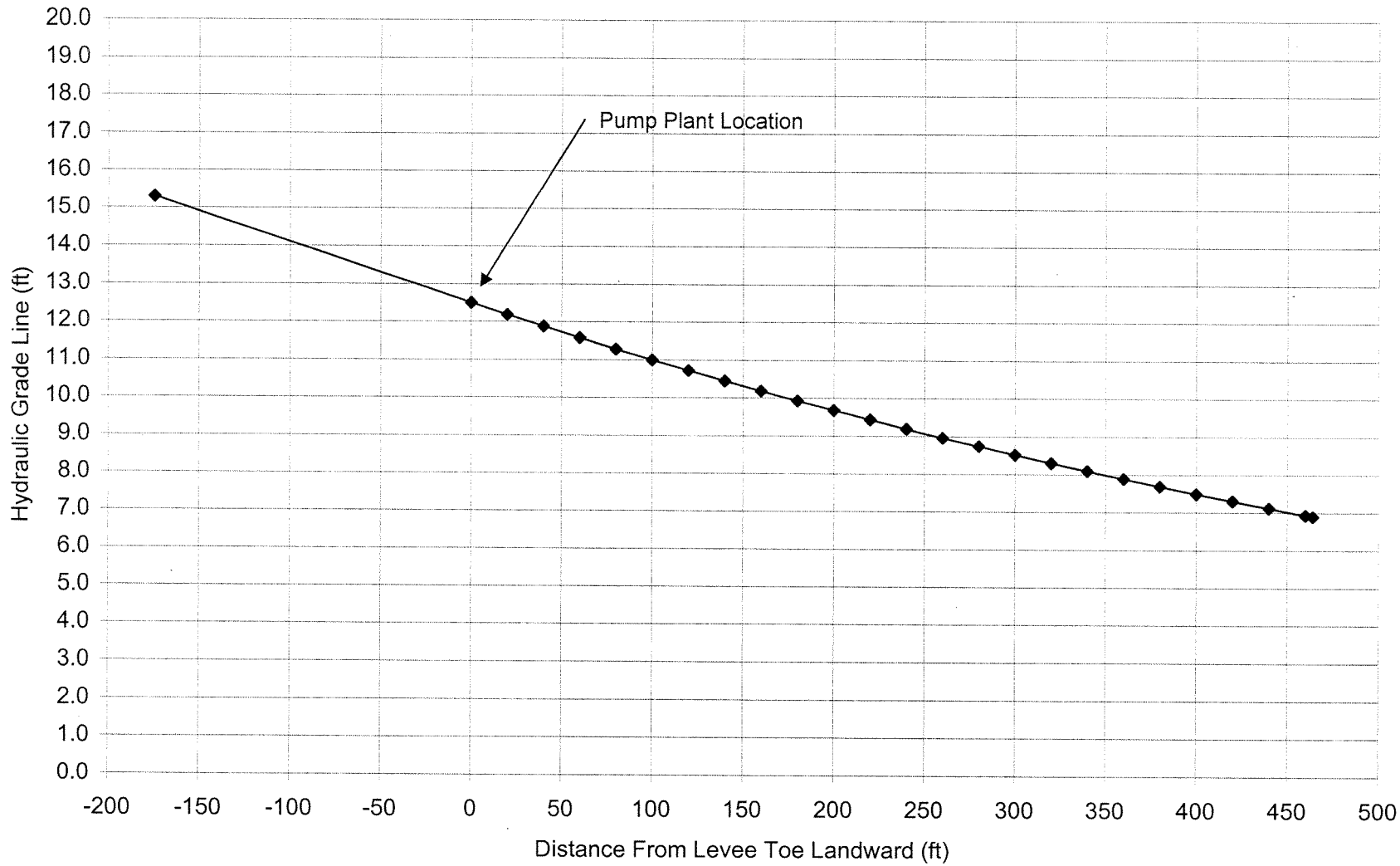

Design Condition n500+3 Hydraulic Grade Line for the
Mill Street Pump Plant Sta 156+75
Armourdale Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the Mill Street Pump Plant
Design Conditions n500+5

Elevation New Top of levee (MSL)	775.3	n500 +5
Elevation at Landside Levee Toe (MSL)	760	
Elevation Bottom of Blanket (MSL)	725	
Driving Head (ft)	15.3	
L1 (ft)	74	
L2 (ft)	100	
Le (ft)	780	
ho	12.50	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-174	15.30	775.30
Pump Plant Location	0	12.50	772.50
	20	12.19	772.19
	40	11.88	771.88
	60	11.58	771.58
	80	11.29	771.29
	100	11.00	771.00
	120	10.72	770.72
	140	10.45	770.45
	160	10.19	770.19
	180	9.93	769.93
	200	9.68	769.68
	220	9.43	769.43
	240	9.19	769.19
	260	8.96	768.96
	280	8.73	768.73
	300	8.51	768.51
	320	8.30	768.30
	340	8.09	768.09
	360	7.88	767.88
	380	7.68	767.68
	400	7.49	767.49
	420	7.30	767.30
	440	7.12	767.12
	460	6.94	766.94
	464	6.90	766.90

Design Condition n500+5 Hydraulic Grade Line for the
Mill Street Pump Plant Sta 156+75
Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the 5th Street Pump Station Sta 185+70

ARM 5th Street 185+70.xls

5TH ST. PUMP PLANT
FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/10/2007

Checked by : Bellew EC-GD
Date : 2/26/2007

NOTES:
1. Landside levee raise proposed
2. L_2 not revised for minor change due to levee raise

Elevation Used

Top of Existing Levee:	767.6 (From O&M Manual)
500 year + 0':	769.0
500 year + 3':	772.0
500 year + 5':	775.0
Ground Surface Land Side:	755.0
Ground Surface River Side:	742.0
Bottom of Blanket:	720.0
Top of Bedrock:	655.0

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

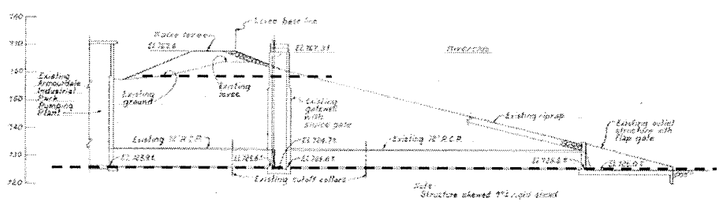
Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

NOMENCLATURE

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

			Input Parameters								
Station			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks
185+70	Existing Conditions		300	300	35.0	30.0	22.0	65.0	45	12.6	EC
185+70	n500 + 0		300	300	35.0	30.0	22.0	65.0	45	14.0	n500
185+70	n500 +3		300	300	35.0	30.0	22.0	65.0	45	17.0	n500+3
185+70	n500 + 5		300	300	35.0	30.0	22.0	65.0	45	19.0	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
655	826	45	100	826	971	10.72	0.31	0.84	2.74		
655	826	45	100	826	971	11.91	0.34	0.84	2.47		
655	826	45	100	826	971	14.46	0.41	0.84	2.03		
655	826	45	100	826	971	16.16	0.54	0.84	1.56		



5TH ST PUMP PLANT ~~OSAGE~~ EGB 2007 RSK

465,400 E
1,060,400 N
465,600 E
465,800 E
1,060,600 N
466,000 E

FIFTH STREET

Existing Armourdale Industrial Park Pumping Plant

Anchor wire

Existing P.Z. (to be raised by others)

Power lines to be relocated by others

℄ HEADER WELL SYSTEM I

D-515

Concrete remain Existing undisturb

WELL

℄ New ditch

D-388

STA 184+70 CONSTRUCT TYPE "B" RAMP

Anchors

Remove fence and gate across levee

Sta. 185+70 For detail, see Dwg. No. D22

M.H.

30"

54"

72" R.C.P.

D-377

190+00

P.C. Sta. 188+19.00

Remove steel drop inlet and 18" steel pipe

Greystone Heights sewer, constructed by others

D-407

RIVER

1,059,800 N
465,800 E
466,000 E
1,060,000 N
466,200 E

PLAN

0 50 100
SCALE IN FEET

TOP OF EXISTING LEVEE

S = 0.000429

inal ground surface on base line

tom of existing rock toe trench

72" R.C.P.

1.L.W.

PROFILE

185+00

190+00

RECORD DRAWING

MAY 1977

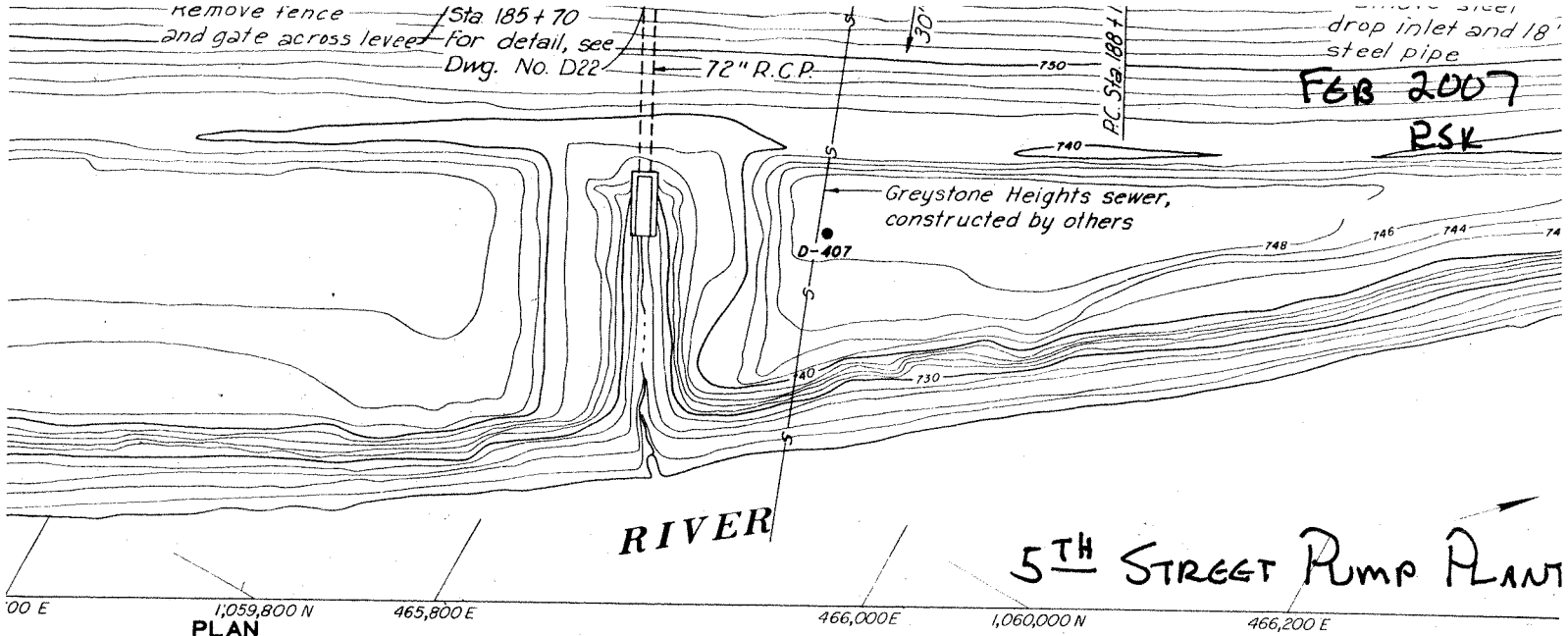
CONTRACT NO. DACW41-76-C-0070

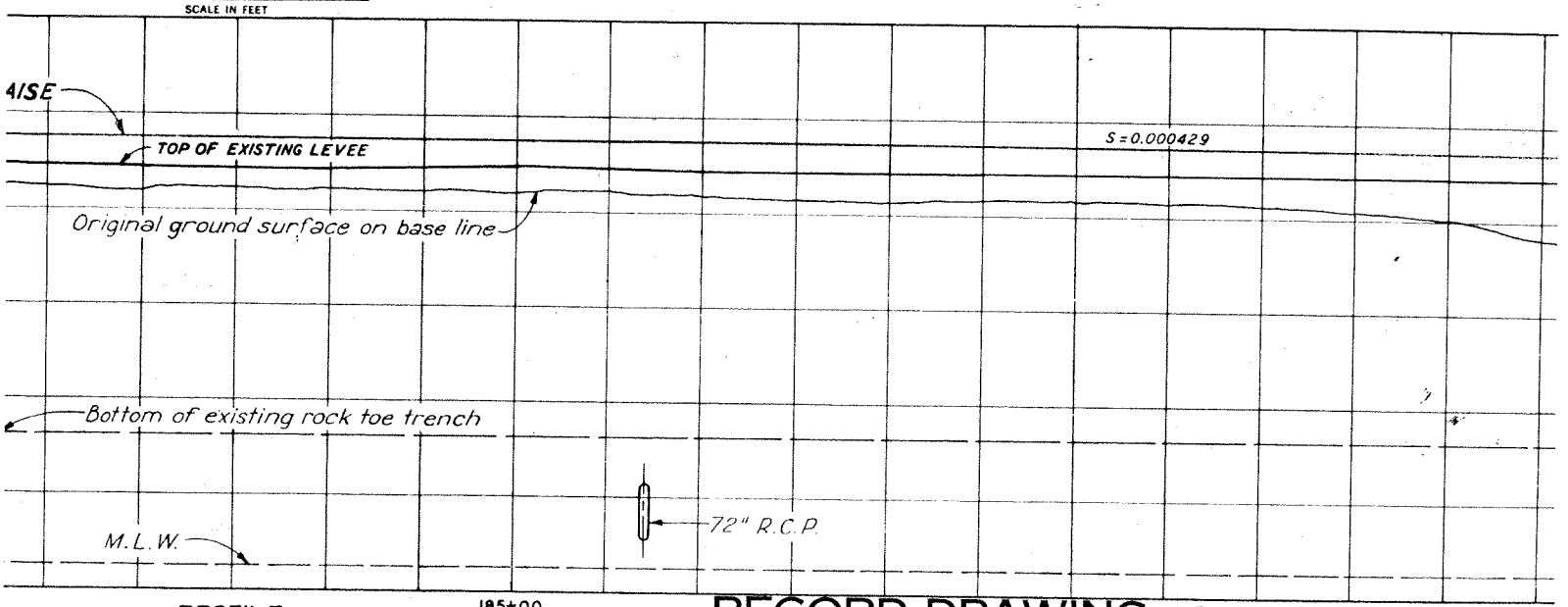
NOTES:

For General Notes, see Dwg. No. A3

For ramps, turnouts, turnaround, & surfacin

Ground surface landward of levee



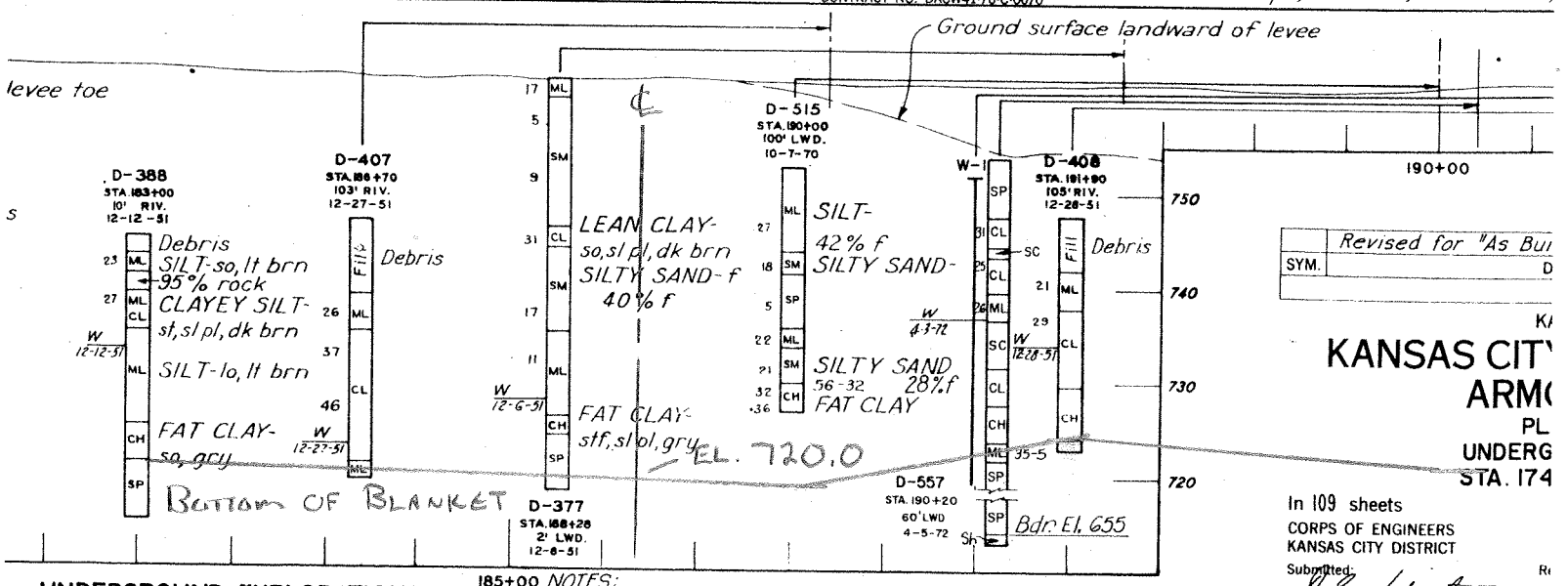


RECORD DRAWING

MAY 1977

CONTRACT NO. DACW41-76-C-0070

NOTES:
For General Notes, see Dwg. No. A
For ramps, turnouts, turnaround,



UNDERGROUND EXPLORATIONS

NOTES:
For details of relief wells, manholes & piezometer; see Dwg. Nos. C2 & C3
For Legend-Existing Utilities, see Dwg. No. A3

SYM.	Revised for "As Bu
	D

KANSAS CITY ARMY
PL
UNDERG
STA. 174

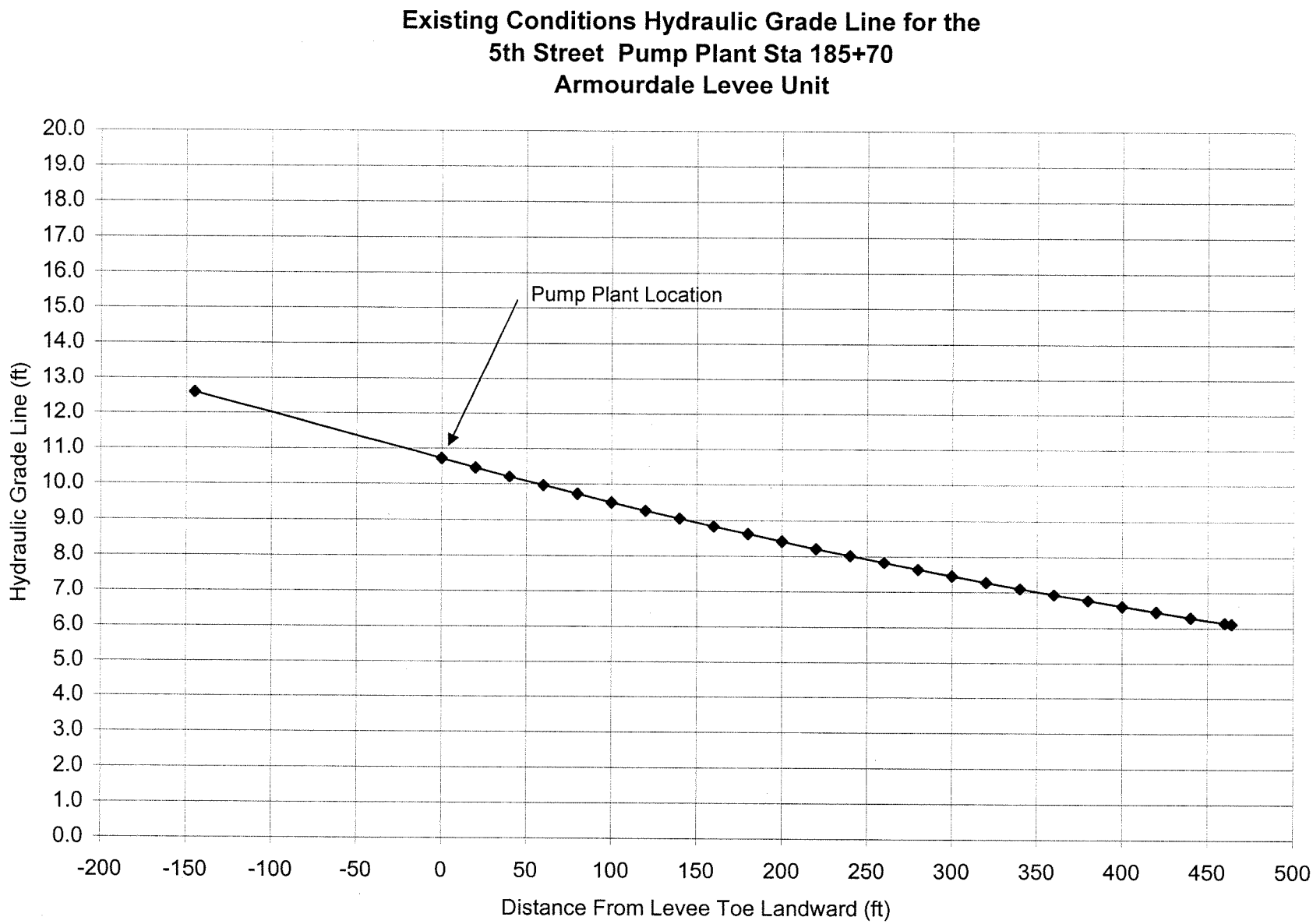
In 109 sheets
CORPS OF ENGINEERS
KANSAS CITY DISTRICT
Submitted:
CHIEF, LOCAL PROTECTION SECTION
COMPILED BY: T.A.K.-J.A.H.
DRAWN BY: J.M.M.

TOP OF ROCK EL. 655.0

Hydraulic Grade Line for the 5th Street Pump Plant
Existing Conditions

Elevation Top of levee (MSL)	767.6	Existing Conditions
Elevation at Landside Levee Toe (MSL)	755	
Elevation Bottom of Blanket (MSL)	720	
Driving Head (ft)	12.6	
L1 (ft)	45	
L2 (ft)	100	
Le (ft)	826	
ho	10.7	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-145	12.60	767.60
Pump Plant Location	0	10.72	765.72
	20	10.46	765.46
	40	10.21	765.21
	60	9.97	764.97
	80	9.73	764.73
	100	9.50	764.50
	120	9.27	764.27
	140	9.05	764.05
	160	8.83	763.83
	180	8.62	763.62
	200	8.41	763.41
	220	8.21	763.21
	240	8.02	763.02
	260	7.83	762.83
	280	7.64	762.64
	300	7.46	762.46
	320	7.28	762.28
	340	7.10	762.10
	360	6.93	761.93
	380	6.77	761.77
	400	6.61	761.61
	420	6.45	761.45
	440	6.29	761.29
	460	6.14	761.14
	464	6.11	761.11

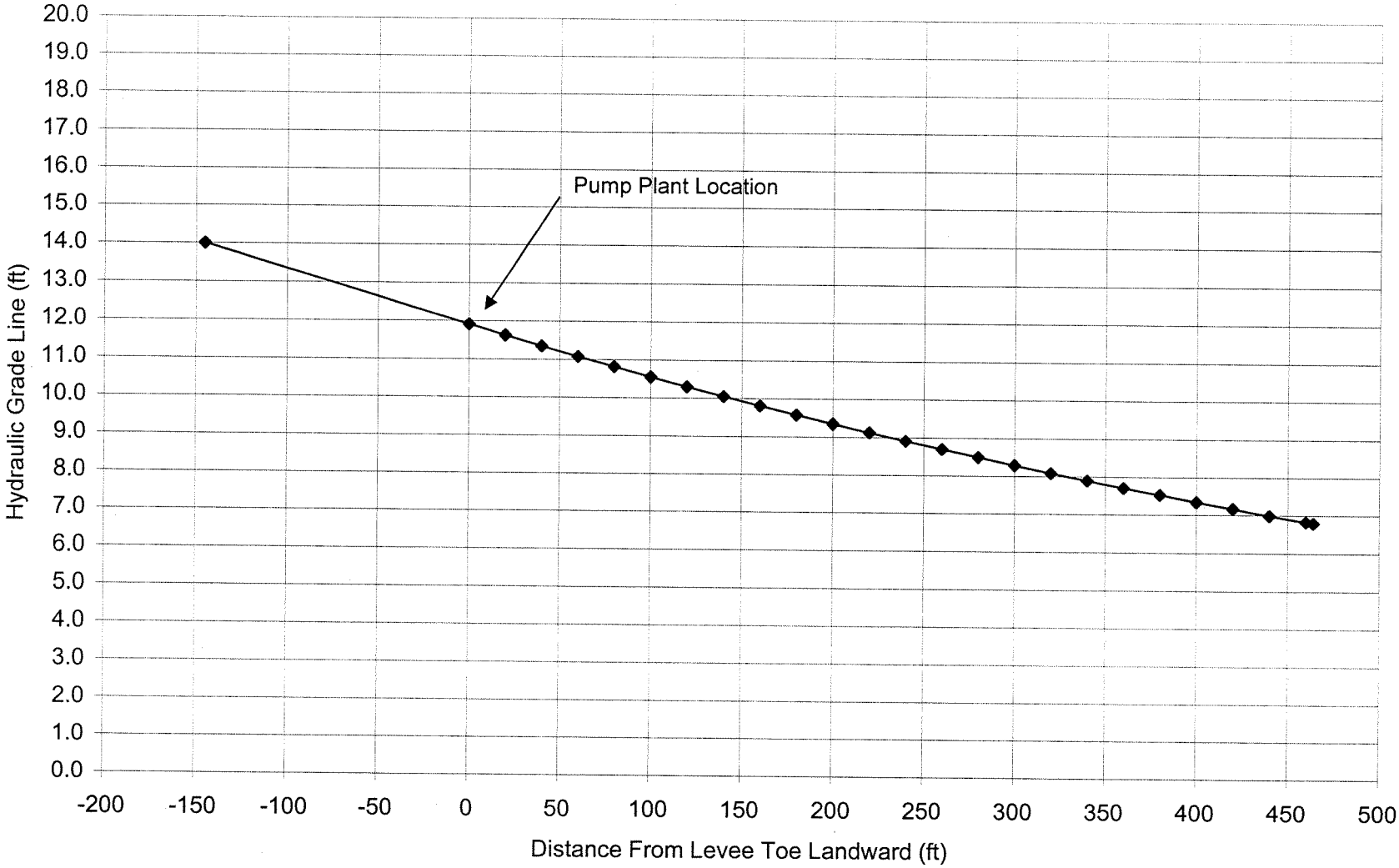

Existing Conditions Hydraulic Grade Line for the
5th Street Pump Plant Sta 185+70
Armourdale Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the 5th Street Pump Plant
Design Conditions n500+0

Elevation Top of levee (MSL)	769	n500+0
Elevation at Landside Levee Toe (MSL)	755	
Elevation Bottom of Blanket (MSL)	720	
Driving Head (ft)	14	
L1 (ft)	45	
L2 (ft)	100	
Le (ft)	826	
ho	11.9	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-145	14.00	769.00
Pump Plant Location	0	11.91	766.91
	20	11.63	766.63
	40	11.35	766.35
	60	11.08	766.08
	80	10.81	765.81
	100	10.55	765.55
	120	10.30	765.30
	140	10.05	765.05
	160	9.81	764.81
	180	9.58	764.58
	200	9.35	764.35
	220	9.13	764.13
	240	8.91	763.91
	260	8.69	763.69
	280	8.49	763.49
	300	8.28	763.28
	320	8.09	763.09
	340	7.89	762.89
	360	7.70	762.70
	380	7.52	762.52
	400	7.34	762.34
	420	7.16	762.16
	440	6.99	761.99
	460	6.83	761.83
	464	6.79	761.79

Design Condition n500+0 Hydraulic Grade Line for the
5th Street Pump Plant Sta 185+70
Armourdale Levee Unit

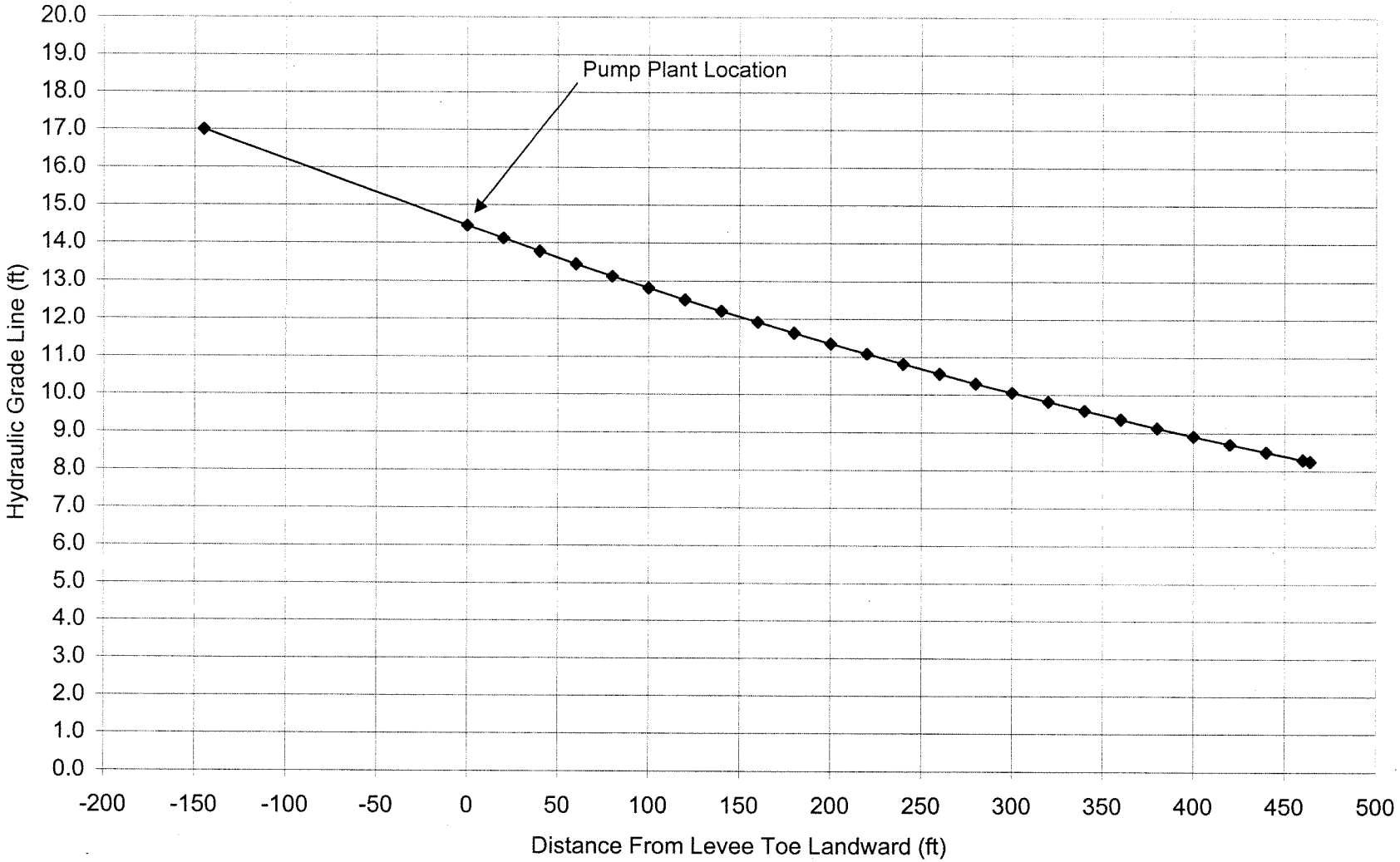


Hydraulic Grade Line for the 5th Street Pump Plant
Design Conditions n500+3

Elevation New Top of levee (MSL)	772	n500 +3
Elevation at Landside Levee Toe (MSL)	755	
Elevation Bottom of Blanket (MSL)	720	
Driving Head (ft)	17	
L1 (ft)	45	
L2 (ft)	100	
Le (ft)	826	
ho	14.46	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-145	17.00	772.00
Pump Plant Location	0	14.46	769.46
	20	14.12	769.12
	40	13.78	768.78
	60	13.45	768.45
	80	13.13	768.13
	100	12.81	767.81
	120	12.51	767.51
	140	12.21	767.21
	160	11.92	766.92
	180	11.63	766.63
	200	11.35	766.35
	220	11.08	766.08
	240	10.82	765.82
	260	10.56	765.56
	280	10.31	765.31
	300	10.06	765.06
	320	9.82	764.82
	340	9.58	764.58
	360	9.35	764.35
	380	9.13	764.13
	400	8.91	763.91
	420	8.70	763.70
	440	8.49	763.49
	460	8.29	763.29
	464	8.25	763.25

Design Condition n500+3 Hydraulic Grade Line for the
5th Street Pump Plant Sta 185+70
Armourdale Levee Unit

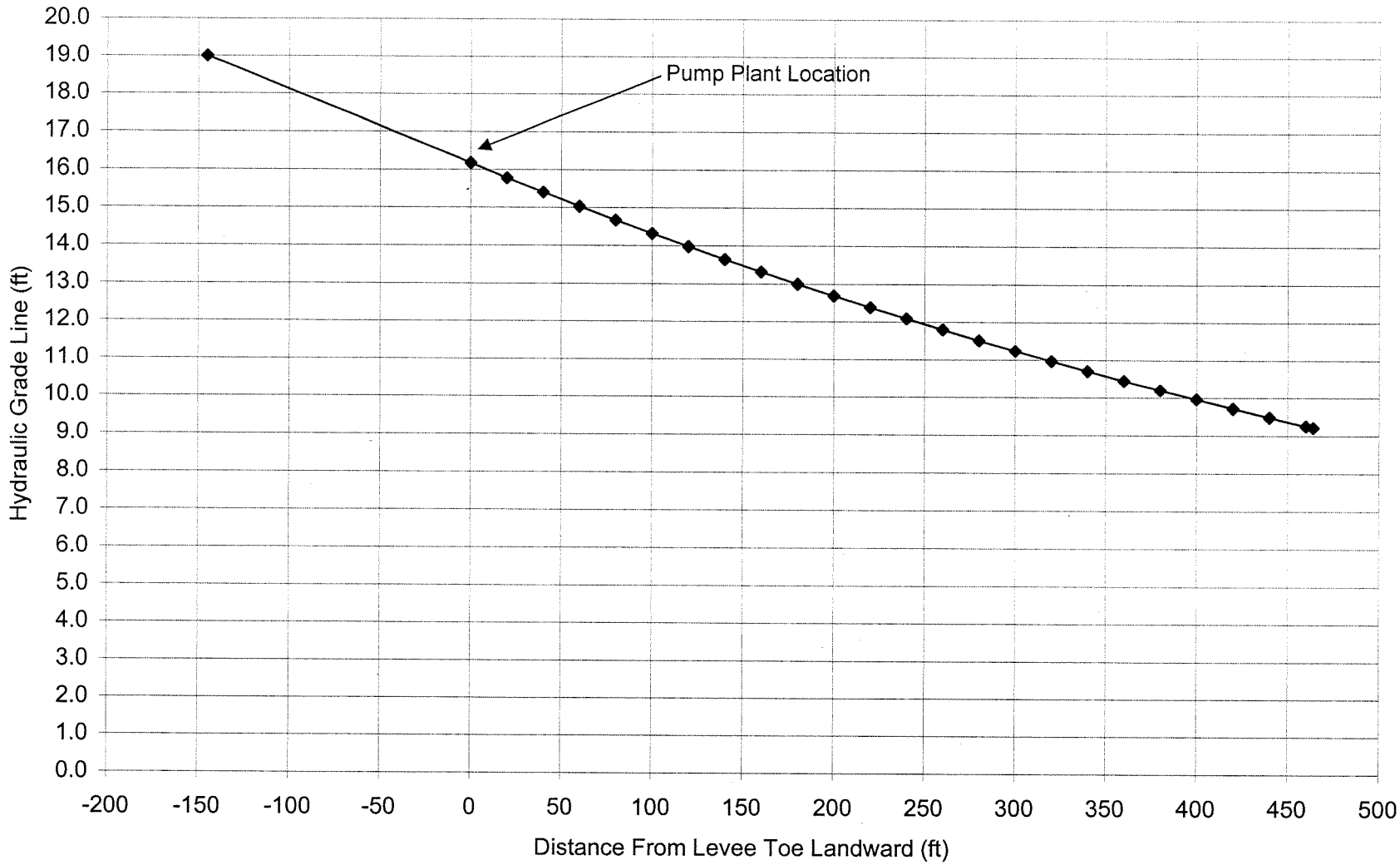


Hydraulic Grade Line for the 5th Street Pump Plant
Design Conditions n500+5

Elevation New Top of levee (MSL)	775	n500 +5
Elevation at Landside Levee Toe (MSL)	755	
Elevation Bottom of Blanket (MSL)	720	
Driving Head (ft)	19	
L1 (ft)	45	
L2 (ft)	100	
Le (ft)	826	
ho	16.16	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-145	19.00	774.00
Pump Plant Location	0	16.16	771.16
	20	15.78	770.78
	40	15.40	770.40
	60	15.03	770.03
	80	14.67	769.67
	100	14.32	769.32
	120	13.98	768.98
	140	13.64	768.64
	160	13.32	768.32
	180	13.00	768.00
	200	12.69	767.69
	220	12.39	767.39
	240	12.09	767.09
	260	11.80	766.80
	280	11.52	766.52
	300	11.24	766.24
	320	10.97	765.97
	340	10.71	765.71
	360	10.45	765.45
	380	10.20	765.20
	400	9.96	764.96
	420	9.72	764.72
	440	9.49	764.49
	460	9.26	764.26
	464	9.22	764.22

Design Condition n500+5 Hydraulic Grade Line for the
5th Street Pump Plant Sta 185+70
Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the Midwest Cold Storage Pump Station 194+60

ARM Midwest Cold Storage 194+60.xls

NOMENCLATURE MIDWEST COLD STORAGE FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/10/2007

Checked by : Bellew EC-GD
Date : 2/12/2007

NOTES:
1. T-wall on levee raise proposed
2. L_2 not revised, levee footprint won't change

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundati	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

Elevation Used

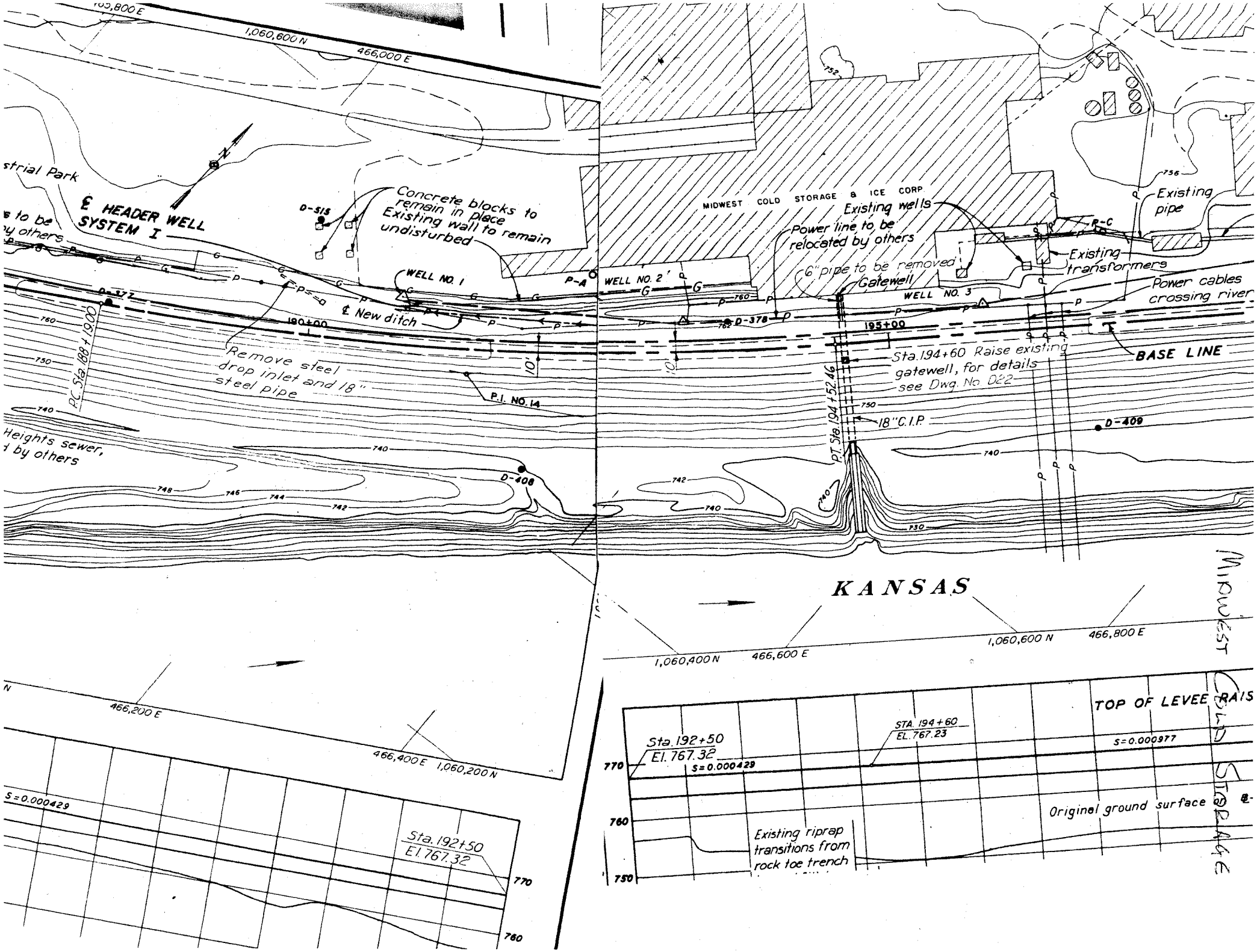
Top of Existing Levee:	767.2	(From O&M Manual)
500 year + 0':	768.4	
500 year + 3':	771.4	
500 year + 5':	773.4	
Ground Surface Land Side:	754.0	(Basement Elevation)
Ground Surface River Side:	740.0	
Bottom of Blanket:	726.0	
Top of Bedrock:	659.0	

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

			Input Parameters								
Station			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks
194+60	Existing Conditions		300	300	28.0	21.0	14.0	67.0	80	13.2	EC
194+60	n500 + 0		300	300	28.0	21.0	14.0	67.0	80	14.4	n500
194+60	n500 +3		300	300	28.0	21.0	14.0	67.0	80	17.4	n500+3
194+60	n500 + 5		300	300	28.0	21.0	14.0	67.0	80	19.4	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
530	750	79	100	750	930	10.65	0.38	0.84	2.21		
530	750	79	100	750	930	11.62	0.42	0.84	2.02		
530	750	79	100	750	930	14.04	0.50	0.84	1.67		
530	750	79	100	750	930	15.66	0.75	0.84	1.13		


1,060,600 N
466,000 E
strial Park
℄ HEADER WELL SYSTEM I
D-515
Concrete blocks to remain in place
Existing wall to remain undisturbed
WELL NO. 1
P-A
WELL NO. 2
MIDWEST COLD STORAGE & ICE CORP.
Existing wells
Power line to be relocated by others
6" pipe to be removed
Gatewell
Existing transformers
Existing pipe
P-C
Power cables crossing river
WELL NO. 3
D-378
℄ New ditch
190+00
195+00
BASE LINE
Remove steel drop inlet and 18" steel pipe
P.I. NO. 14
P.C. Sta 188+19.00
Sta. 194+60 Raise existing gatewell, for details see Dwg. No. D22
P.T. Sta 194+52.46
18" C.I.P.
D-409
D-408
Heights sewer, by others
KANSAS
1,060,400 N
466,600 E
1,060,600 N
466,800 E
466,200 E
466,400 E
1,060,200 N
S=0.000429
Sta. 192+50
El. 767.32
770
760
750
TOP OF LEVEE RAISED
STA. 194+60
EL. 767.23
S=0.000977
Original ground surface
Existing riprap transitions from rock toe trench
MIDWEST COLD STORAGE

KANSAS

1,060,000 N | 466,200 E | 466,400 E | 1,060,200 N | 1,060,400 N | 466,600 E | 1,060,600 N | 466,800 E

Sta. 192+50
El. 767.32

S = 0.000429

760

750

740

730

190+00

Sta 192+50
El. 767.32

S = 0.000429

STA. 194+60
EL. 767.23

TOP OF LEVEE RAI.

S = 0.000977

Existing riprap transitions from rock toe trench to rockfill toe

Original ground surface on ℄

18" C.I.P.

Sta 193+50

El. 736.0

Sta 194+40

Bottom of existing rockfill toe El. 717.4

195+00

DRAWING NOTES:

For General Notes, see Dwg. No. A3

For ramps, turnouts, turnaround, & surfacing details, see Dwg. No.

1977

DACW41-76-C-0070

Ground surface landward of levee

W-1

D-408
STA. 181+90
105' RIV.
12-28-51

SP
CL
SC
CL
ML
SC
CL
CH
ML
SP
SP

31
25
26
21
29
35-5

Fill
Debris
ML
CL
CH

W
12-28-51

750
740
730
720

Bdn. El. 655

190+00

Revised for "As Built" conditions | 9/20/
SYM. | DESCRIPTION | DATE
REVISIONS

KANSAS RIVER, KANSAS

KANSAS CITY, KANSAS - 1962 MC
ARMOURDALE UNIT

PLAN, PROFILE AND
UNDERGROUND EXPLORATIONS
STA. 174+50 TO STA. 192+50

In 109 sheets
CORPS OF ENGINEERS
KANSAS CITY DISTRICT

Sheet No. 13
Dwg. No. A13

Scale:

Submitted: CHIEF, LOCAL PROTECTION SECTION
Recommended: CHIEF, FOUNDATIONS & MATLS. BR.
Approved: CHIEF, ENGINEERING

COMPILED BY: T.A.K.-J.A.H.
DRAWN BY: J.M.M.
CHECKED BY: R.L.B.

A-

O & M M PLATE

& piezometer, see Dwg Nos. C2 & C3
Dwg. No. A3

Ground surface landward of levee

Ground surface at landside of levee toe

℄ OF PUMP PLANT

W-2

W
12-6-51

ML
SM
CL
ML
CH
SP

SILTY SAND- lo, lt. brn.

SILT-so, dmp, dk brn.

SILT-so, lo, gry

EL 726

FAT CLAY-so, sl, pl, brn.

D-378
STA. 193+66
27' LWD
12-8-51

BOTTOM OF BLANKET

W-3

24
32

W
12-28-51

CL
CH

D-409
STA. 198+75
81' RIV.
12-28-51

31
2
19

SM
SP
ML
SM
ML
SP

W
12-6-51

SAN
SILT
lt br

D-3
STA. 198
27 L
12-6-

195+00

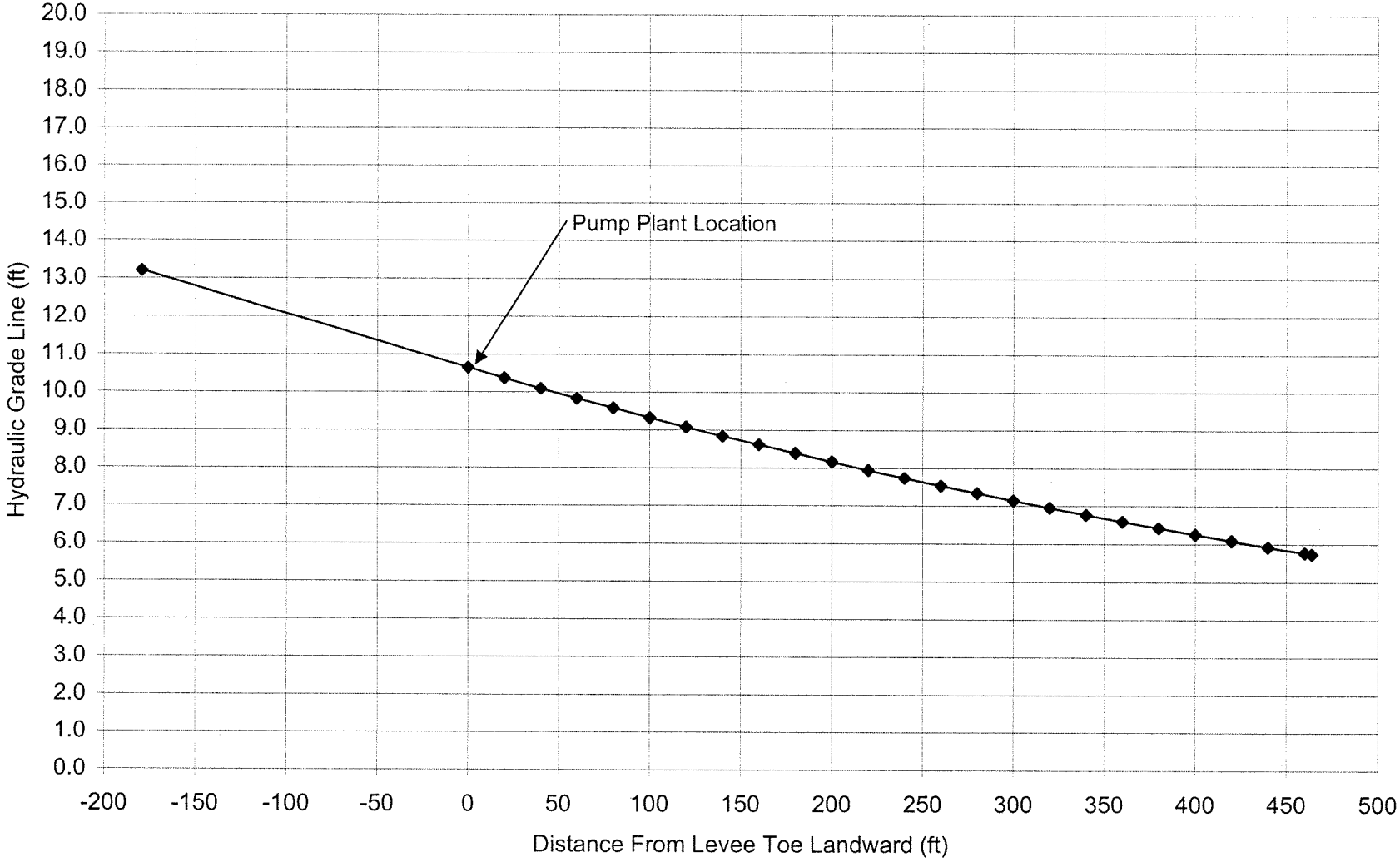
TOP OF BEDROCK: 659±

STA 190+20 EL 655

Hydraulic Grade Line for the Midwest Cold Storage Pump Plant
Existing Conditions - Assume No Wells

Elevation Top of levee (MSL)	767.2	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	13.2		
L1 (ft)	79		
L2 (ft)	100		
Le (ft)	750		
ho	10.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-179	13.20	767.20
Pump Plant Location	0	10.65	764.65
	20	10.37	764.37
	40	10.10	764.10
	60	9.83	763.83
	80	9.58	763.58
	100	9.32	763.32
	120	9.08	763.08
	140	8.84	762.84
	160	8.61	762.61
	180	8.38	762.38
	200	8.16	762.16
	220	7.95	761.95
	240	7.74	761.74
	260	7.53	761.53
	280	7.33	761.33
	300	7.14	761.14
	320	6.95	760.95
	340	6.77	760.77
	360	6.59	760.59
	380	6.42	760.42
	400	6.25	760.25
	420	6.09	760.09
	440	5.93	759.93
	460	5.77	759.77
	464	5.74	759.74

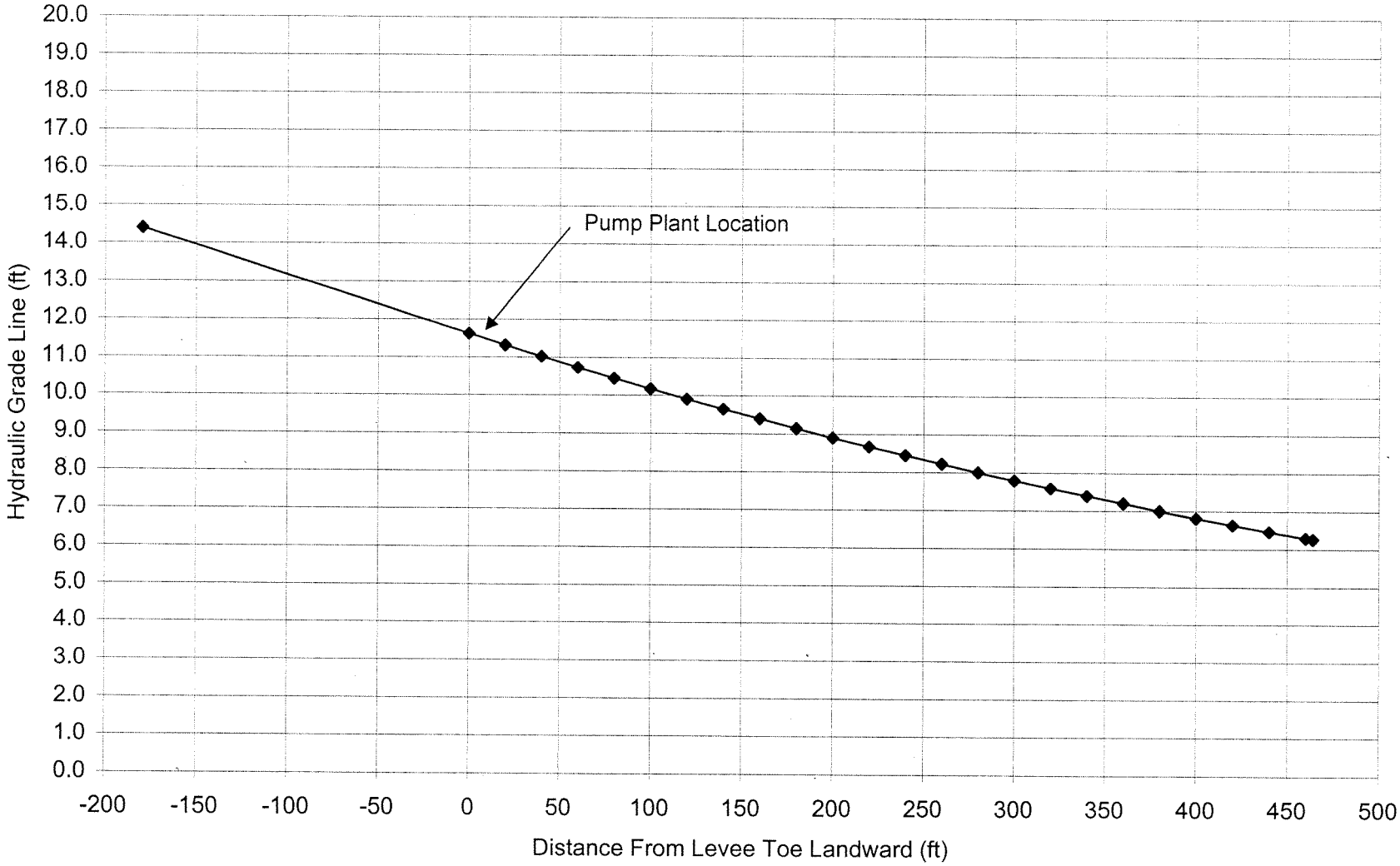
Existing Conditions Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Sta 194+60 Armourdale Levee Unit



Hydraulic Grade Line for the Midwest Cold Storage Pump Plant
Design Conditions n500+0 - Assumes no Wells

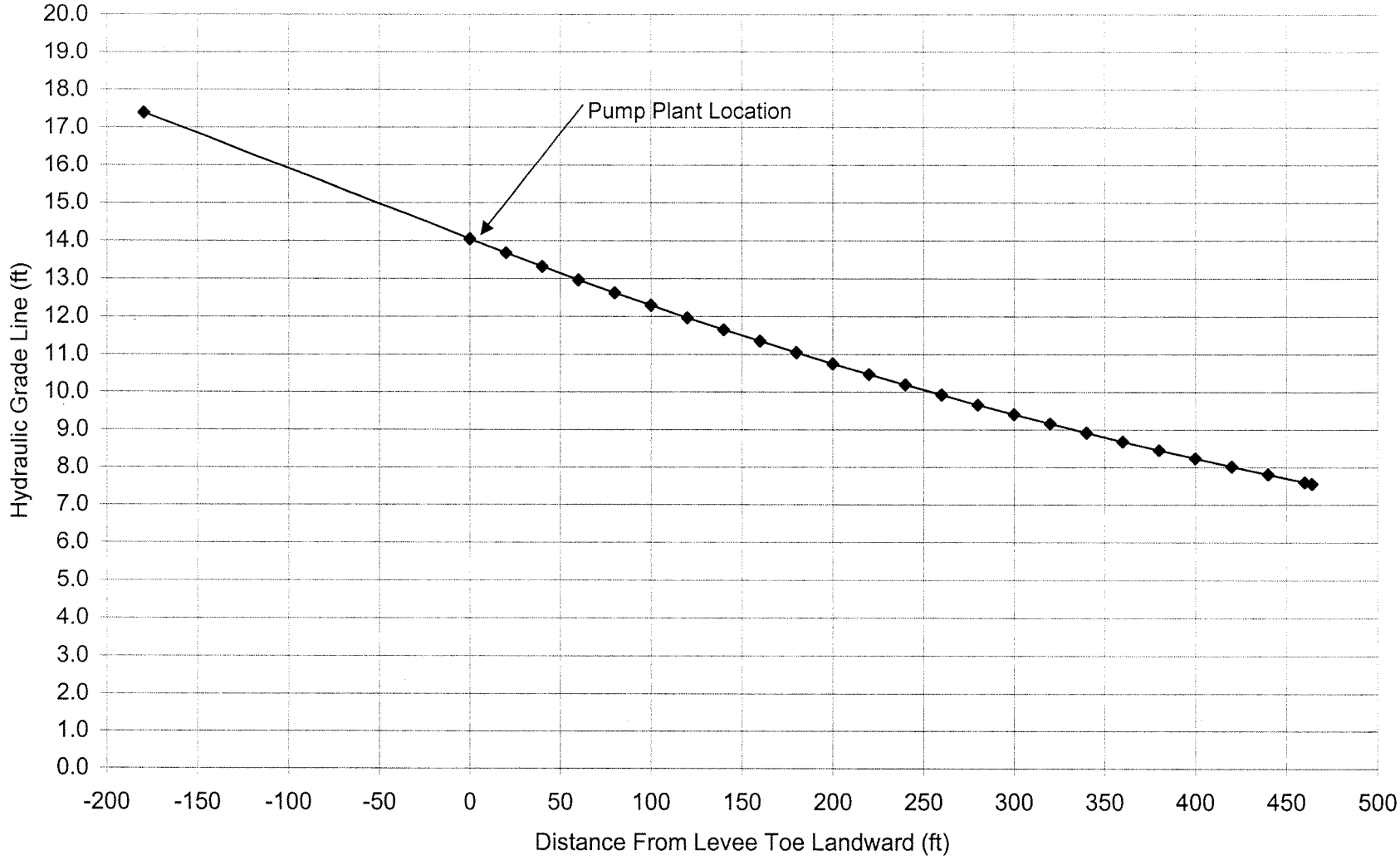
Elevation Top of levee (MSL)	768.4	n500+0	
Elevation at Landside Levee Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	14.4		
L1 (ft)	79		
L2 (ft)	100		
Le (ft)	750		
ho	11.6		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-179	14.40	768.40
Pump Plant Location	0	11.62	765.62
	20	11.32	765.32
	40	11.02	765.02
	60	10.73	764.73
	80	10.45	764.45
	100	10.17	764.17
	120	9.90	763.90
	140	9.64	763.64
	160	9.39	763.39
	180	9.14	763.14
	200	8.90	762.90
	220	8.67	762.67
	240	8.44	762.44
	260	8.22	762.22
	280	8.00	762.00
	300	7.79	761.79
	320	7.59	761.59
	340	7.39	761.39
	360	7.19	761.19
	380	7.00	761.00
	400	6.82	760.82
	420	6.64	760.64
	440	6.46	760.46
	460	6.29	760.29
	464	6.26	760.26

Design Condition n500+0 Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Sta 194+60 Armourdale Levee Unit



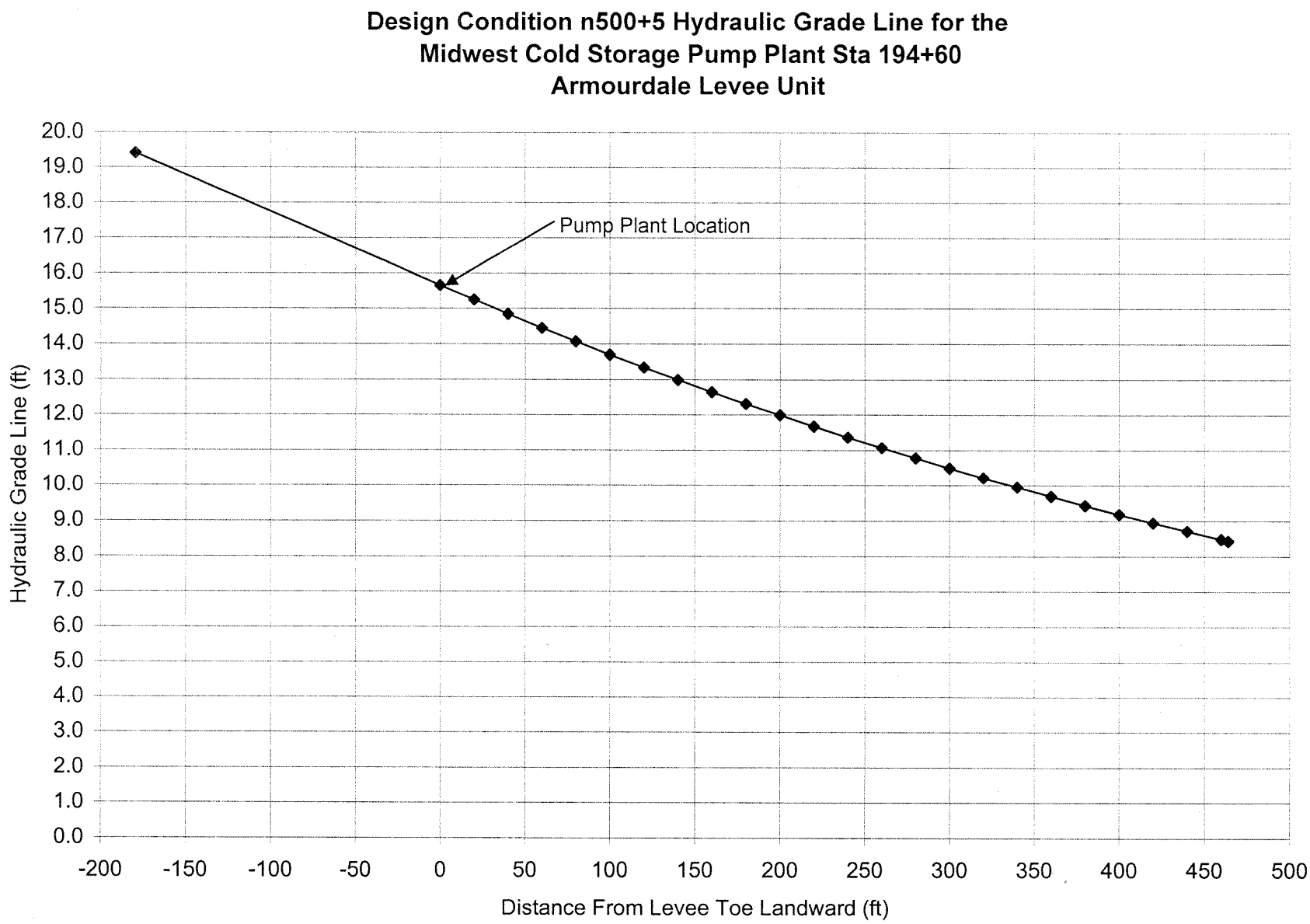
Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Design Conditions n500+3 - Assumes no Wells			
Elevation New Top of levee (MSL)	771.4	n500 +3	
Elevation at Landside Levee Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	17.4		
L1 (ft)	79		
L2 (ft)	100		
Le (ft)	750		
ho	14.04		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-179	17.40	771.40
Pump Plant Location	0	14.04	768.04
	20	13.67	767.67
	40	13.31	767.31
	60	12.96	766.96
	80	12.62	766.62
	100	12.29	766.29
	120	11.97	765.97
	140	11.65	765.65
	160	11.35	765.35
	180	11.05	765.05
	200	10.76	764.76
	220	10.47	764.47
	240	10.20	764.20
	260	9.93	763.93
	280	9.67	763.67
	300	9.41	763.41
	320	9.17	763.17
	340	8.92	762.92
	360	8.69	762.69
	380	8.46	762.46
	400	8.24	762.24
	420	8.02	762.02
	440	7.81	761.81
	460	7.61	761.61
	464	7.57	761.57

Design Condition n500+3 Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Sta 194+60 Armourdale Levee Unit



Hydraulic Grade Line for the Midwest Cold Storage Pump Plant
Design Conditions n500+5 - Assumes no Wells

Elevation New Top of levee (MSL)	773.4	n500 +5	
Elevation at Landside Levee Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	19.4		
L1 (ft)	79		
L2 (ft)	100		
Le (ft)	750		
ho	15.66		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-179	19.40	773.40
Pump Plant Location	0	15.66	769.66
	20	15.24	769.24
	40	14.84	768.84
	60	14.45	768.45
	80	14.07	768.07
	100	13.70	767.70
	120	13.34	767.34
	140	12.99	766.99
	160	12.65	766.65
	180	12.32	766.32
	200	11.99	765.99
	220	11.68	765.68
	240	11.37	765.37
	260	11.07	765.07
	280	10.78	764.78
	300	10.50	764.50
	320	10.22	764.22
	340	9.95	763.95
	360	9.69	763.69
	380	9.43	763.43
	400	9.19	763.19
	420	8.94	762.94
	440	8.71	762.71
	460	8.48	762.48
	464	8.43	762.43


Design Condition n500+5 Hydraulic Grade Line for the
Midwest Cold Storage Pump Plant Sta 194+60
Armourdale Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Armourdale Kansas - Hydraulic Grade Line for the Shawnee Pump Station Sta 230+80 Assumes no Wells

ARM Shawnee PP 230+80 No Wells.xls

NOMENCLATURE

SHAWNEE PUMP PLANT
FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/10/2007

Checked by : Bellew EC-GD GMB
Date : 02/12/07

NOTES:
1. T-wall on existing levee raise proposed
2. L_2 not revised for minor change due to raise

Elevation Used

Top of Existing Levee:	765.3 (From O&M Manual)
500 year + 0':	766.4
500 year + 3':	769.4
500 year + 5':	771.4
Ground Surface Land Side:	750.0
Ground Surface River Side:	738.0 (Average blanket thickness over slope)
Bottom of Blanket:	725.0
Top of Bedrock:	658.0

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

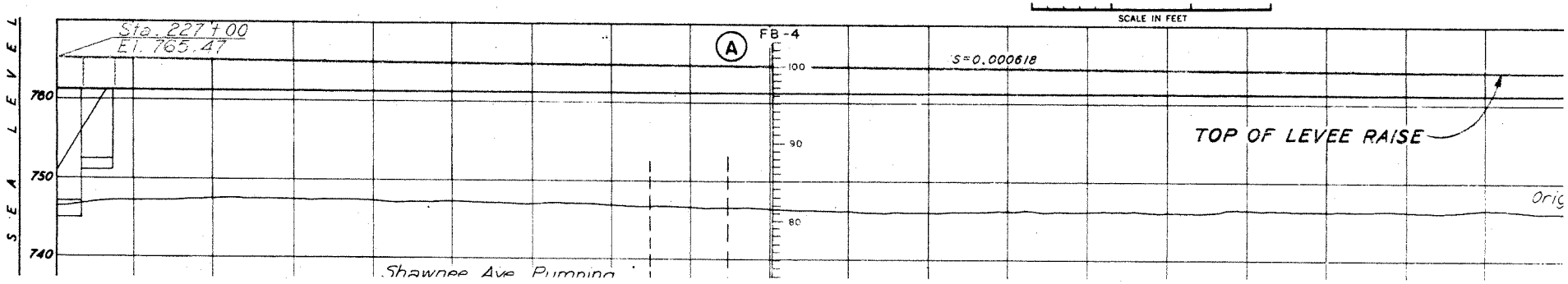
Station			Input Parameters								
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks
230+80	Existing Conditions		400	400	25.0	19.0	13.0	67.0	120	15.3	EC
230+80	n500 + 0		400	400	25.0	19.0	13.0	67.0	120	16.4	n500
230+80	n500 +3		400	400	25.0	19.0	13.0	67.0	120	19.4	n500+3
230+80	n500 + 5		400	400	25.0	19.0	13.0	67.0	120	21.4	n500+5

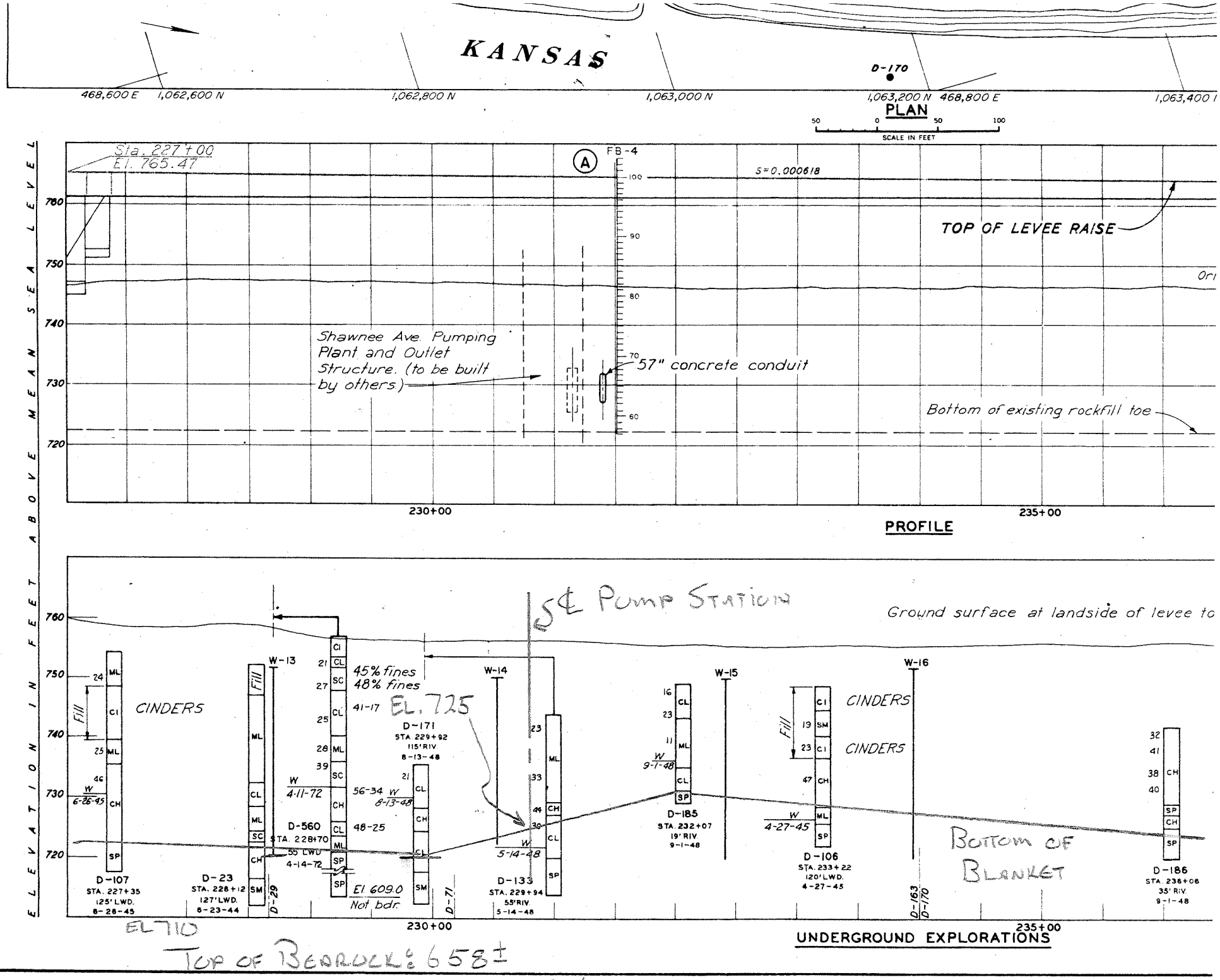
Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
590	819	118	100	819	1037	12.08	0.48	0.84	1.74		
590	819	118	100	819	1037	12.95	0.52	0.84	1.62		
590	819	118	100	819	1037	15.31	0.61	0.84	1.37		
590	819	118	100	819	1037	16.89	0.89	0.84	0.94		

1,062,800 N
1,063,000 N
1,063,200 N
468,200 E
1,063,400 N
468,200 E 1,062,600 N
468,400 E
Match Line Dwg. No. A15
PILLSBURY COMPANY
SAMBOL PACKING CO.
D-107
D-23
P-H
WELL NO. 13
D-560
REMOVE EXISTING RETAINING WALL
WELL NO. 14
Existing Shawnee Ave. Pumping Plant to be removed by others
96" conduit
M.H.-3
WELL NO. 15A
D-106
Remove existing bldg and plug existing water well
WELL NO. 16
℄ HEA
Fence
D-10
P.I. NO. 20
P.C. STA. 227+75.14
230+00
D-29
℄ Ramp
KANSAS
Brace pole (To be relocated by others)
REPAIR EXISTING RETAINING WALL FOR LIMITS SEE DWG. NO. D28
CITY
℄ LEVE
Shawnee Ave Pumping Plant (built by others)
D-71
FB-4
D-133
D-185
235+00
STA. 231+00 - CONSTRUCT SPECIAL RAMP
Sta. 231+38.91
P.I. NO. 21
BASE LINE
7.5' x 7.5' conduit
D-186
D-171
Protective Berm for Outlet, see Dwg. No. B-4
57" conduit to be plugged by others
D-163
KANSAS
D-170
468,600 E
1,062,600 N
1,062,800 N
1,063,000 N
1,063,200 N
468,800 E
1,063,400 N

PLAN

50 0 50 100
SCALE IN FEET




KANSAS
D-170
468,600 E
1,062,600 N
1,062,800 N
1,063,000 N
1,063,200 N
468,800 E
1,063,400 N
PLAN
SCALE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
Sta. 227+00
El. 765.47
FB-4
S=0.000618
TOP OF LEVEE RAISE
Shawnee Ave. Pumping Plant and Outlet Structure. (to be built by others)
57" concrete conduit
Bottom of existing rockfill toe
230+00
235+00
PROFILE
℄ Pump Station
Ground surface at landside of levee to
CINDERS
Fill
45% fines
48% fines
41-17
EL. 725
D-107
STA. 227+35
125' LWD.
8-28-45
D-23
STA. 228+12
127' LWD.
6-23-44
D-560
STA. 228+70
55 LWD
4-14-72
D-171
STA. 229+92
115' RIV.
8-13-48
El 609.0
Not bdr.
D-133
STA. 229+94
55' RIV.
5-14-48
D-185
STA. 232+07
19' RIV.
9-1-48
D-106
STA. 233+22
120' LWD.
4-27-45
D-186
STA. 236+06
35' RIV.
9-1-48
Bottom of Blanket
UNDERGROUND EXPLORATIONS
EL 710
Top of Bedrock: 658±
EL 666 STA 222+0
EL 652 STA 229+50

Hydraulic Grade Line for the Shawnee Pump Plant
Existing Conditions - Assumes No Wells

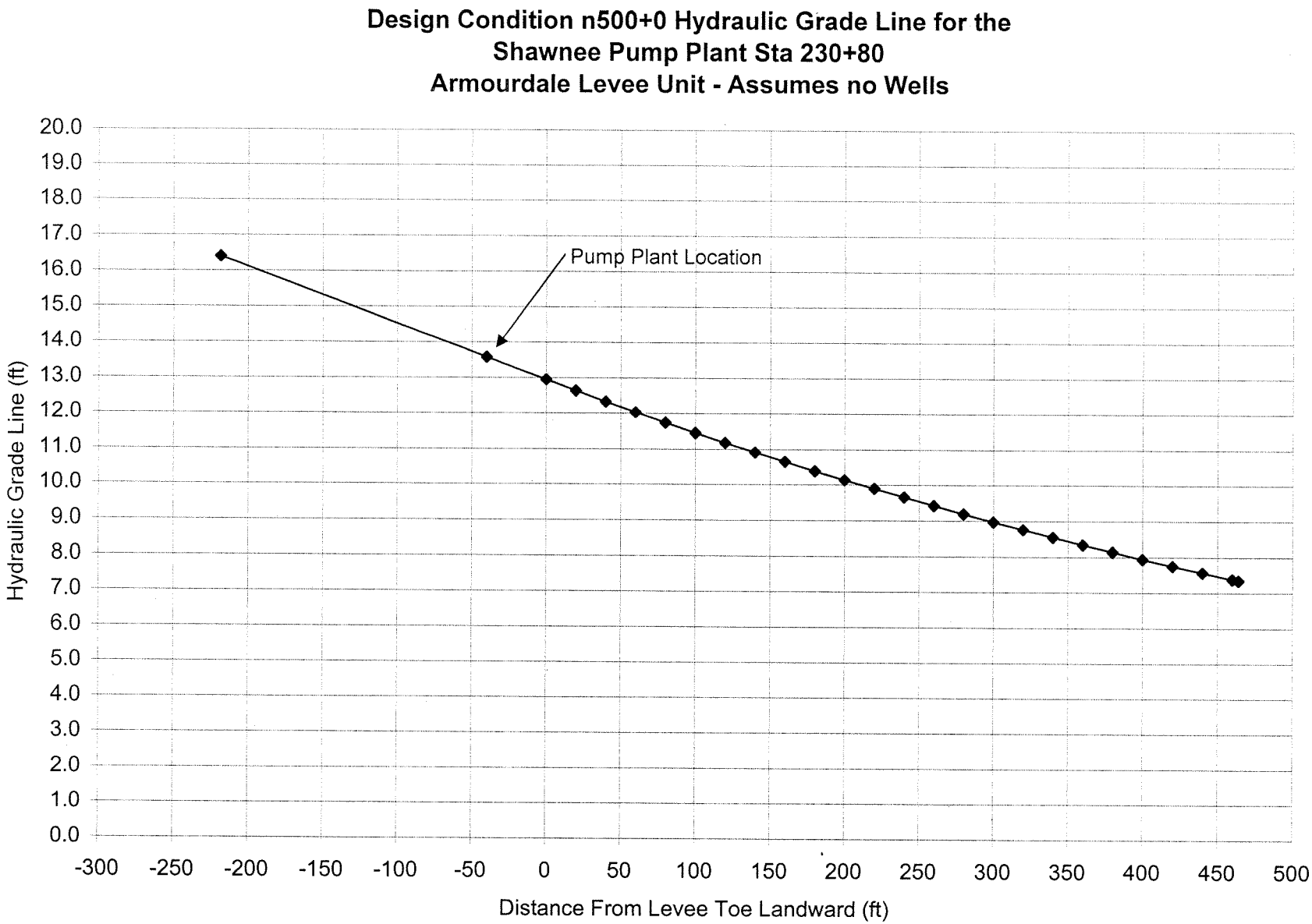
Elevation Top of levee (MSL)	765.3	Existing Conditions
Elevation at Landside Levee Toe (MSL)	750	
Elevation Bottom of Blanket (MSL)	725	
Driving Head (ft)	15.3	
L1 (ft)	118	
L2 (ft)	100	
Le (ft)	819	
ho	12.1	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-218	15.30	765.30
Pump Plant Location	-40	12.67	762.67
	0	12.08	762.08
	20	11.79	761.79
	40	11.50	761.50
	60	11.22	761.22
	80	10.95	760.95
	100	10.69	760.69
	120	10.43	760.43
	140	10.18	760.18
	160	9.93	759.93
	180	9.69	759.69
	200	9.46	759.46
	220	9.23	759.23
	240	9.01	759.01
	260	8.79	758.79
	280	8.58	758.58
	300	8.37	758.37
	320	8.17	758.17
	340	7.97	757.97
	360	7.78	757.78
	380	7.59	757.59
	400	7.41	757.41
	420	7.23	757.23
	440	7.06	757.06
	460	6.89	756.89
	464	6.85	756.85

Hydraulic Grade Line for the Shawnee Pump Plant
Design Conditions n500+0 - Assumes no Wells

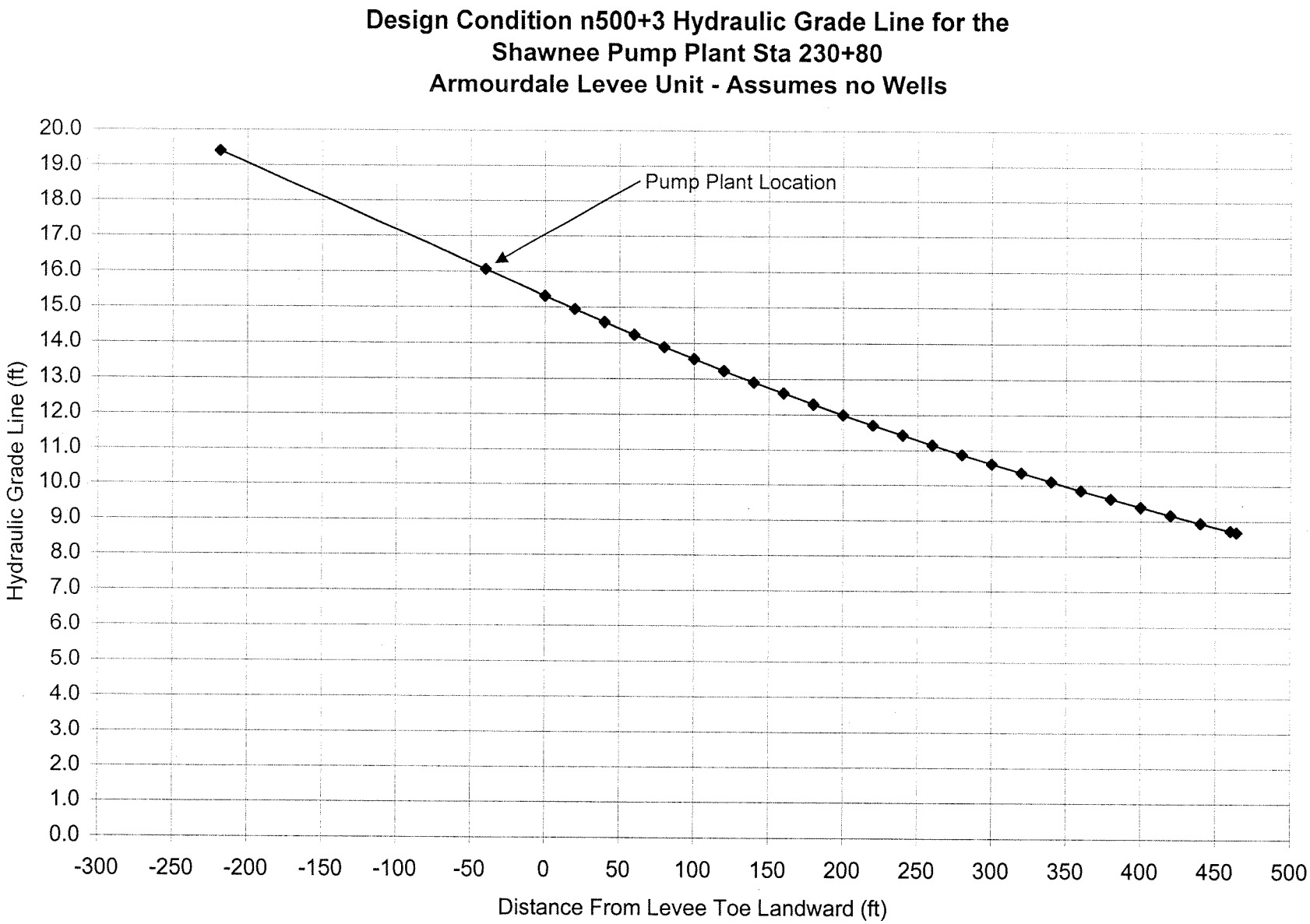
Elevation Top of levee (MSL)	766.4	n500+0
Elevation at Landside Levee Toe (MSL)	750	
Elevation Bottom of Blanket (MSL)	725	
Driving Head (ft)	16.4	
L1 (ft)	118	
L2 (ft)	100	
Le (ft)	819	
ho	12.9	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-218	16.40	766.40
Pump Plant Location	-40	13.58	763.58
	0	12.95	762.95
	20	12.63	762.63
	40	12.33	762.33
	60	12.03	762.03
	80	11.74	761.74
	100	11.46	761.46
	120	11.18	761.18
	140	10.91	760.91
	160	10.65	760.65
	180	10.39	760.39
	200	10.14	760.14
	220	9.89	759.89
	240	9.66	759.66
	260	9.42	759.42
	280	9.20	759.20
	300	8.97	758.97
	320	8.76	758.76
	340	8.55	758.55
	360	8.34	758.34
	380	8.14	758.14
	400	7.94	757.94
	420	7.75	757.75
	440	7.56	757.56
	460	7.38	757.38
	464	7.34	757.34


Design Condition n500+0 Hydraulic Grade Line for the
Shawnee Pump Plant Sta 230+80
Armourdale Levee Unit - Assumes no Wells
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

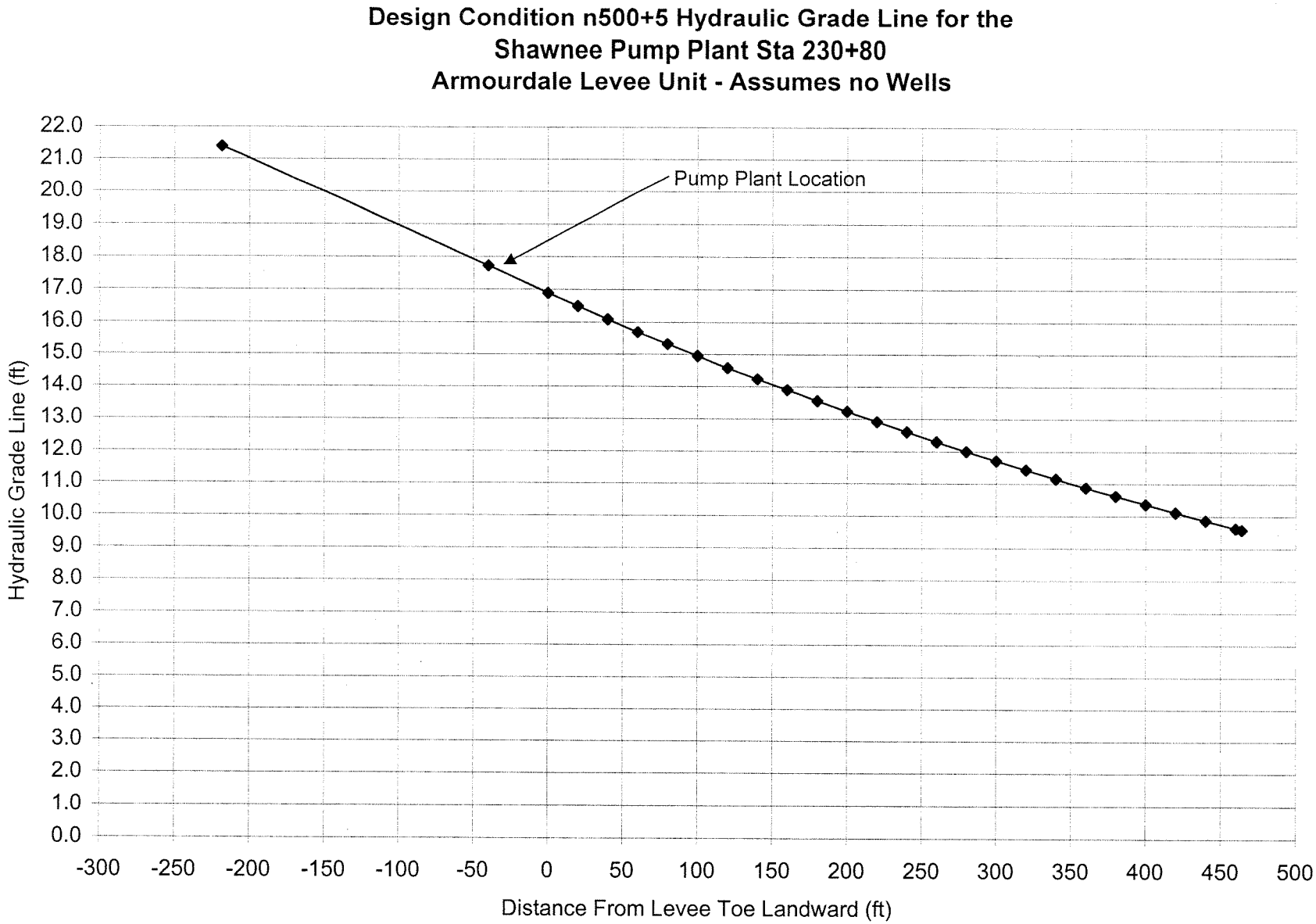
Hydraulic Grade Line for the Shawnee Pump Plant
Design Conditions n500+3 - Assumes no Wells

Elevation New Top of levee (MSL)	769.4	n500 +3	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	19.4		
L1 (ft)	118		
L2 (ft)	100		
Le (ft)	819		
ho	15.31		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-218	19.40	769.40
Pump Plant Location	-40	16.06	766.06
	0	15.31	765.31
	20	14.94	764.94
	40	14.58	764.58
	60	14.23	764.23
	80	13.89	763.89
	100	13.55	763.55
	120	13.23	763.23
	140	12.91	762.91
	160	12.60	762.60
	180	12.29	762.29
	200	11.99	761.99
	220	11.71	761.71
	240	11.42	761.42
	260	11.15	761.15
	280	10.88	760.88
	300	10.62	760.62
	320	10.36	760.36
	340	10.11	760.11
	360	9.86	759.86
	380	9.63	759.63
	400	9.39	759.39
	420	9.17	759.17
	440	8.95	758.95
	460	8.73	758.73
	464	8.69	758.69


Design Condition n500+3 Hydraulic Grade Line for the
Shawnee Pump Plant Sta 230+80
Armourdale Levee Unit - Assumes no Wells
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the Shawnee Pump Plant
Design Conditions n500+5 - Assumes on Wells

Elevation New Top of levee (MSL)	771.4	n500 +5	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	21.4		
L1 (ft)	118		
L2 (ft)	100		
Le (ft)	819		
ho	16.89		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-218	21.40	771.40
Pump Plant Location	-40	17.72	767.72
~~Pump Plant Location~~	0	16.89	766.89
	20	16.49	766.49
	40	16.09	766.09
	60	15.70	765.70
	80	15.32	765.32
	100	14.95	764.95
	120	14.59	764.59
	140	14.24	764.24
	160	13.89	763.89
	180	13.56	763.56
	200	13.23	763.23
	220	12.91	762.91
	240	12.60	762.60
	260	12.30	762.30
	280	12.00	762.00
	300	11.71	761.71
	320	11.43	761.43
	340	11.15	761.15
	360	10.88	760.88
	380	10.62	760.62
	400	10.36	760.36
	420	10.11	760.11
	440	9.87	759.87
	460	9.63	759.63
	464	9.58	759.58


Design Condition n500+5 Hydraulic Grade Line for the
Shawnee Pump Plant Sta 230+80
Armourdale Levee Unit - Assumes no Wells
Pump Plant Location
Hydraulic Grade Line (ft)
22.0
21.0
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Armourdale Kansas - Hydraulic Grade Line for the KCS Railroad Pump Station Sta 276+80
Assume No Wells and Ignore the Slot

ARM KCS RR PP 276+80.xls

NOMENCLATURE KCS RAILROAD PUMP PLANT
FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD GMB
Date : 02/12/07

NOTES:
1. New floodwall raise proposed
2. L_2 not revised for minor change due to floodwall raise

Elevation Used

Top of Existing Levee:	762.5 (From O&M Manual)
500 year + 0':	762.4 (Equal to Existing Conditions)
500 year + 3':	765.4
500 year + 5':	767.4
Ground Surface Land Side:	750.0 (Ignore the slot)
Ground Surface River Side:	737.0 (For average blanket thickness)
Bottom of Blanket:	722.0
Top of Bedrock:	672.0

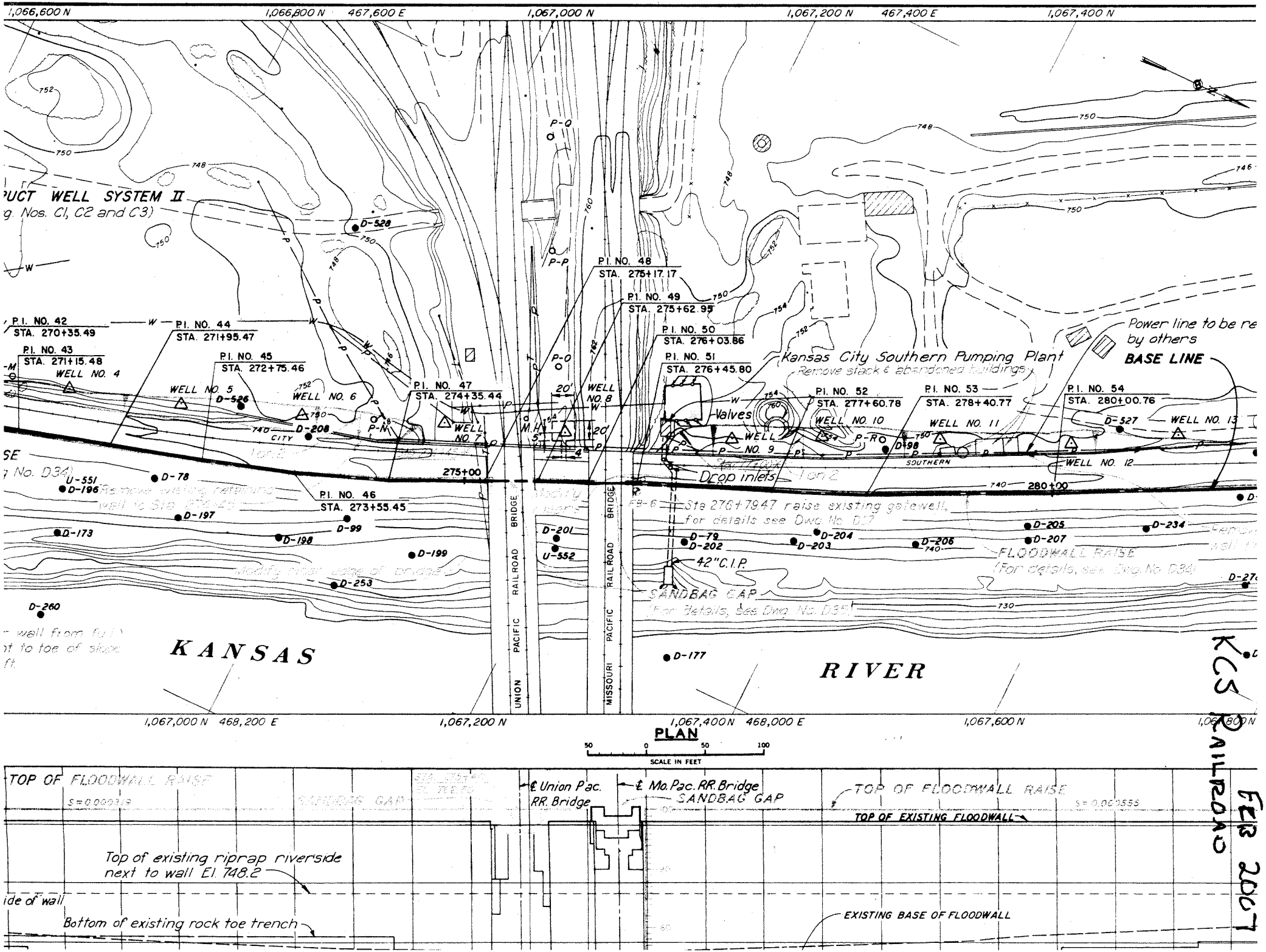
Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2L_R/C-1)}) / (e^{(2L_R/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

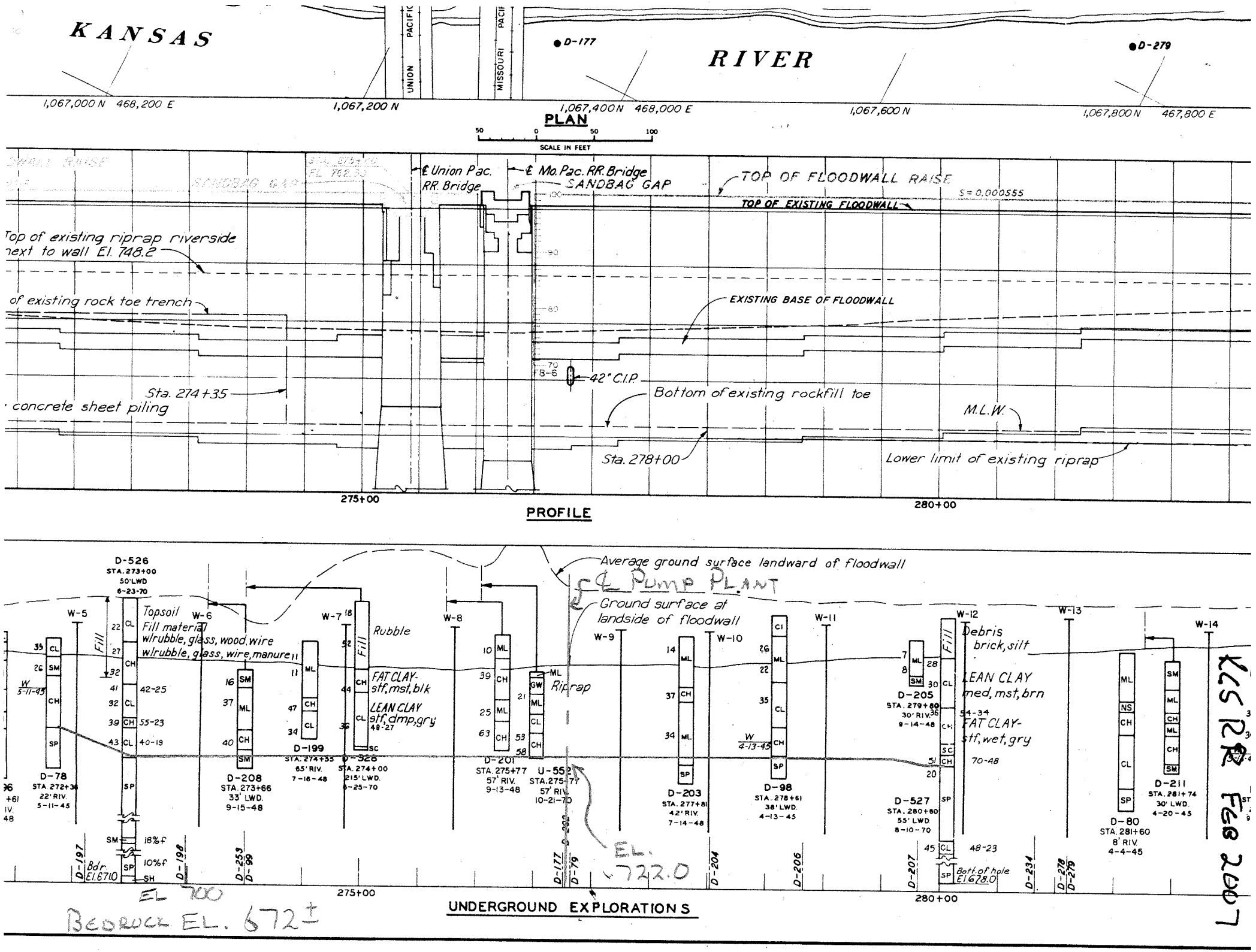
Station			Input Parameters								Remarks
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	
276+80	Existing Conditions		400	400	28.0	22.0	15.0	50.0	120	12.5	EC
276+80	n500 + 0		400	400	28.0	22.0	15.0	50.0	120	12.4	n500
276+80	n500 +3		400	400	28.0	22.0	15.0	50.0	120	15.4	n500+3
276+80	n500 + 5		400	400	28.0	22.0	15.0	50.0	120	17.4	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
548	748	118	20	748	886	10.55	0.38	0.84	2.23		
548	748	118	20	748	886	10.47	0.37	0.84	2.25		
548	748	118	20	748	886	13.00	0.46	0.84	1.81		
548	748	118	20	748	886	14.69	0.52	0.84	1.60		



KCS RAILROAD

FEB 2007


KANSAS
RIVER
UNION PACIFIC
MISSOURI PACIFIC
D-177
D-279
1,067,000 N 468,200 E
1,067,200 N
1,067,400 N 468,000 E
1,067,600 N
1,067,800 N 467,800 E
PLAN
SCALE IN FEET
℄ Union Pac. RR. Bridge
℄ Mo. Pac. RR. Bridge
SANDBAG GAP
TOP OF FLOODWALL RAISE
S = 0.000555
TOP OF EXISTING FLOODWALL
Top of existing riprap riverside next to wall El. 748.2
of existing rock toe trench
EXISTING BASE OF FLOODWALL
42" C.I.P.
Sta. 274+35
concrete sheet piling
Bottom of existing rockfill toe
M.L.W.
Sta. 278+00
Lower limit of existing riprap
275+00
280+00
PROFILE
Average ground surface landward of floodwall
℄ PUMP PLANT
Ground surface at landside of floodwall
D-526
STA. 273+00
50' LWD
6-23-70
Topsoil
Fill material
w/rubble, glass, wood, wire
w/rubble, glass, wire, manure
Rubble
FAT CLAY-
stf, mst, blk
LEAN CLAY
stf, dmp, gry
Riprap
Debris
brick, silt
LEAN CLAY
med, mst, brn
FAT CLAY-
stf, wet, gry
D-78
STA. 272+36
22' RIV.
5-11-45
D-208
STA. 273+66
33' LWD.
9-15-48
D-199
STA. 274+53
65' RIV.
7-16-48
D-328
STA. 274+00
215' LWD.
6-25-70
D-201
STA. 275+77
57' RIV.
9-13-48
U-552
STA. 275+77
57' RIV.
10-21-70
D-203
STA. 277+81
42' RIV.
7-14-48
D-98
STA. 278+61
38' LWD.
4-13-45
D-205
STA. 279+80
30' RIV.
9-14-48
D-527
STA. 280+60
55' LWD.
8-10-70
D-80
STA. 281+60
8' RIV.
4-4-45
D-211
STA. 281+74
30' LWD.
4-20-45
W-5
W-6
W-7
W-8
W-9
W-10
W-11
W-12
W-13
W-14
Bdr. El. 671.0
Bott. of hole El. 678.0
EL. 722.0
EL 700
BEDROCK EL. 672±
275+00
280+00
UNDERGROUND EXPLORATIONS
KCS RR Feb 2007

Hydraulic Grade Line for the KCS Railroad Pump Plant
Existing Conditions - Assumes No Wells and No Slot

Elevation Top of levee (MSL)	762.5	Existing Conditions
Elevation at Landside Levee Toe (MSL)	750	
Elevation Bottom of Blanket (MSL)	722	
Driving Head (ft)	12.5	
L1 (ft)	118	
L2 (ft)	20	
Le (ft)	748	
ho	10.6	

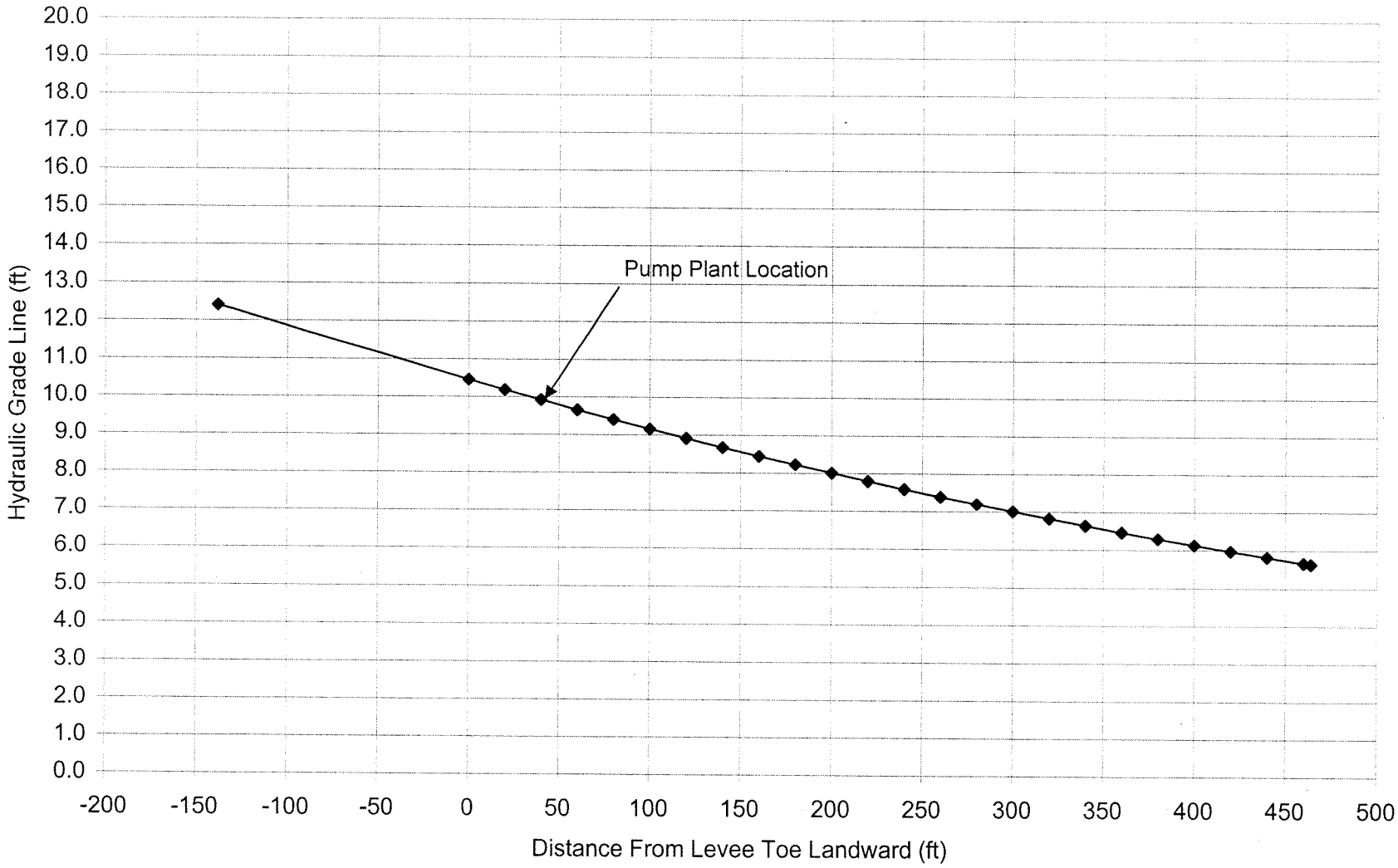
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-138	12.50	762.50
	0	10.55	760.55
	20	10.27	760.27
Pump Plant Location	40	10.00	760.00
	60	9.74	759.74
	80	9.48	759.48
	100	9.23	759.23
	120	8.99	758.99
	140	8.75	758.75
	160	8.52	758.52
	180	8.30	758.30
	200	8.08	758.08
	220	7.86	757.86
	240	7.66	757.66
	260	7.46	757.46
	280	7.26	757.26
	300	7.07	757.07
	320	6.88	756.88
	340	6.70	756.70
	360	6.52	756.52
	380	6.35	756.35
	400	6.18	756.18
	420	6.02	756.02
	440	5.86	755.86
	460	5.71	755.71
	464	5.68	755.68

Hydraulic Grade Line for the KCS Railroad Pump Plant
Design Conditions n500+0 - Assumes No Wells and No Slot

Elevation Top of levee (MSL)	762.4	n500+0
Elevation at Landside Levee Toe (MSL)	750	
Elevation Bottom of Blanket (MSL)	722	
Driving Head (ft)	12.4	
L1 (ft)	118	
L2 (ft)	20	
Le (ft)	748	
ho	10.5	

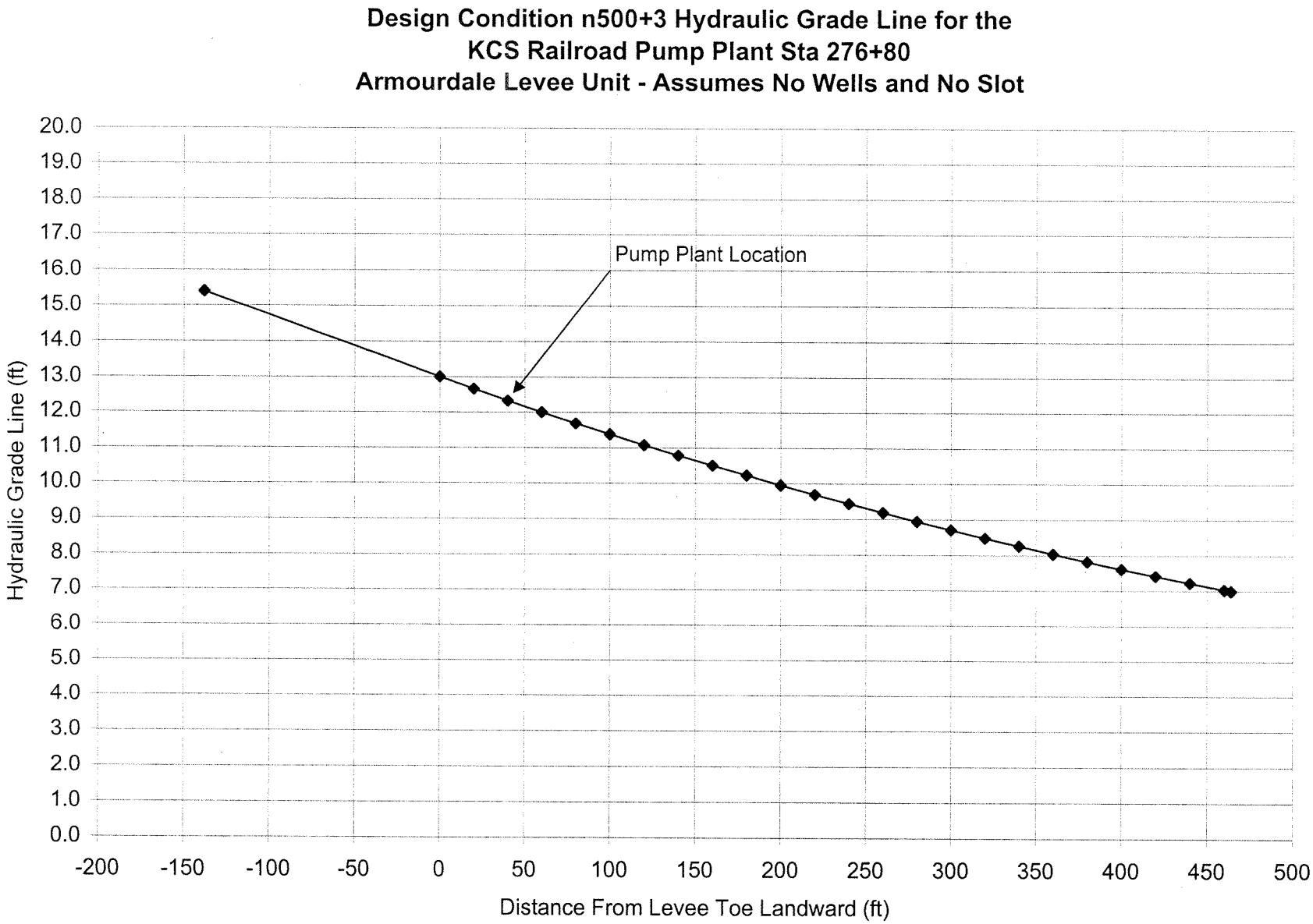
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-138	12.40	762.40
	0	10.47	760.47
	20	10.19	760.19
Pump Plant Location	40	9.92	759.92
	60	9.66	759.66
	80	9.41	759.41
	100	9.16	759.16
	120	8.92	758.92
	140	8.68	758.68
	160	8.45	758.45
	180	8.23	758.23
	200	8.01	758.01
	220	7.80	757.80
	240	7.60	757.60
	260	7.40	757.40
	280	7.20	757.20
	300	7.01	757.01
	320	6.83	756.83
	340	6.65	756.65
	360	6.47	756.47
	380	6.30	756.30
	400	6.13	756.13
	420	5.97	755.97
	440	5.81	755.81
	460	5.66	755.66
	464	5.63	755.63

Design Condition n500+0 Hydraulic Grade Line for the KCS Railroad Pump Plant Sta 276+80 Armourdale Levee Unit - Assumes No Wells and No Slot



Hydraulic Grade Line for the KCS Railroad Pump Plant
Design Conditions n500+3 - Assumes No Wells and No Slot

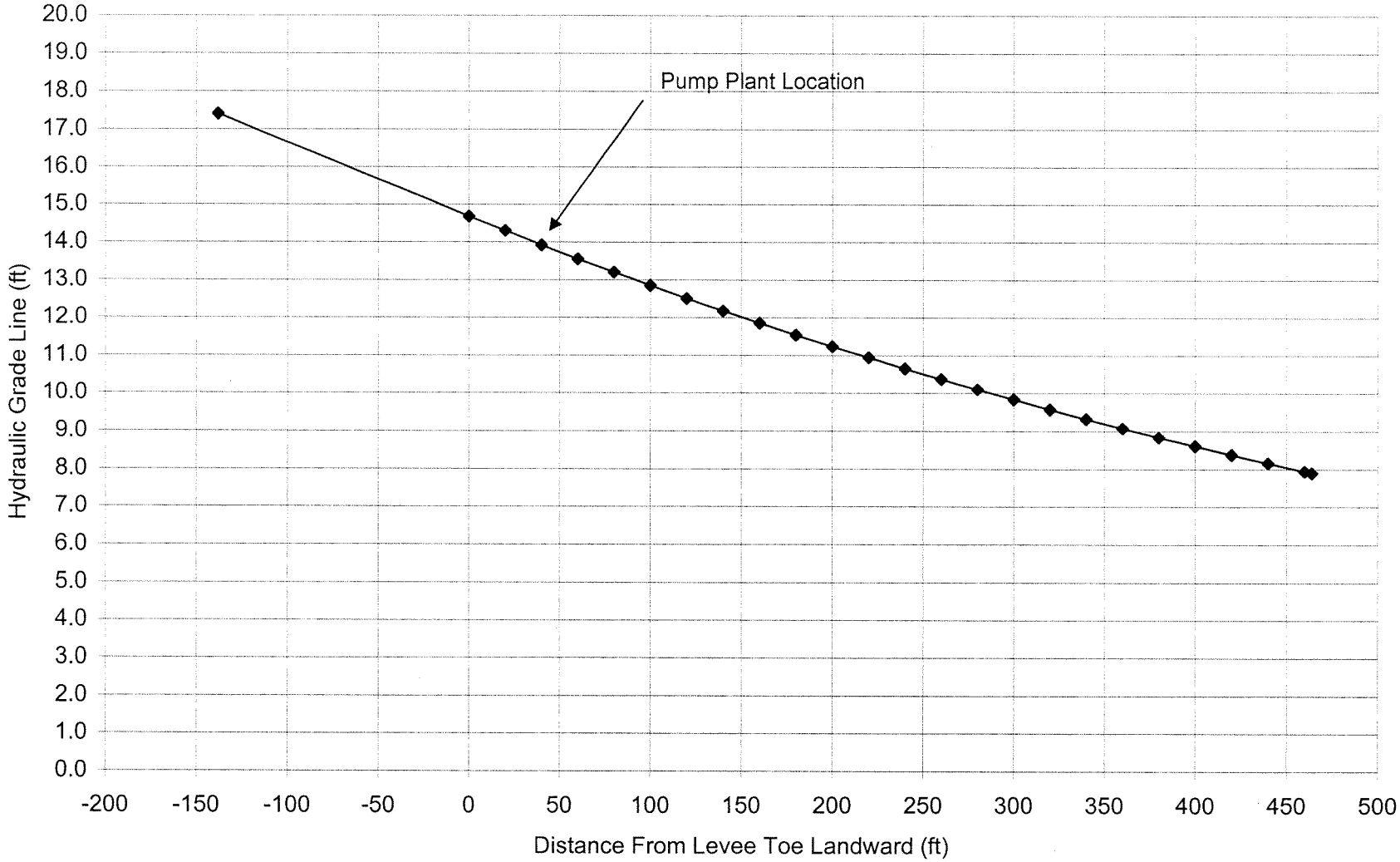
Elevation New Top of levee (MSL)	765.4	n500 +3	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.4		
L1 (ft)	118		
L2 (ft)	20		
Le (ft)	748		
ho	13.00		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-138	15.40	765.40
	0	13.00	763.00
	20	12.66	762.66
Pump Plant Location	40	12.32	762.32
	60	12.00	762.00
	80	11.68	761.68
	100	11.37	761.37
	120	11.07	761.07
	140	10.78	760.78
	160	10.50	760.50
	180	10.22	760.22
	200	9.95	759.95
	220	9.69	759.69
	240	9.43	759.43
	260	9.18	759.18
	280	8.94	758.94
	300	8.71	758.71
	320	8.48	758.48
	340	8.25	758.25
	360	8.04	758.04
	380	7.82	757.82
	400	7.62	757.62
	420	7.42	757.42
	440	7.22	757.22
	460	7.03	757.03
	464	6.99	756.99


Design Condition n500+3 Hydraulic Grade Line for the
KCS Railroad Pump Plant Sta 276+80
Armourdale Levee Unit - Assumes No Wells and No Slot
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the KCS Railroad Pump Plant
Design Conditions n500+5 - Assumes No Wells and No Slot

Elevation New Top of levee (MSL)	767.4	n500 +5	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	17.4		
L1 (ft)	118		
L2 (ft)	20		
Le (ft)	748		
ho	14.69		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-138	17.40	767.40
	0	14.69	764.69
	20	14.30	764.30
Pump Plant Location	40	13.92	763.92
	60	13.56	763.56
	80	13.20	763.20
	100	12.85	762.85
	120	12.51	762.51
	140	12.18	762.18
	160	11.86	761.86
	180	11.55	761.55
	200	11.24	761.24
	220	10.95	760.95
	240	10.66	760.66
	260	10.38	760.38
	280	10.10	760.10
	300	9.84	759.84
	320	9.58	759.58
	340	9.33	759.33
	360	9.08	759.08
	380	8.84	758.84
	400	8.61	758.61
	420	8.38	758.38
	440	8.16	758.16
	460	7.94	757.94
	464	7.90	757.90

Design Condition n500+5 Hydraulic Grade Line for the KCS Railroad Pump Plant Sta 276+80 Armourdale Levee Unit - Assumes No Wells and No Slot



Armourdale Kansas - Hydraulic Grade Line for the PBI Gordon Pump Station Sta 289+60

ARM PBI Gordon PP 289+60.xls

NOMENCLATURE

PBI GORDON PUMP PLANT FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS Name : Kuzniakowski EC-GD Checked by : Bellew EC-GD

Date : 2/11/2007 Date : 07/28/07 GMB

NOTES:

1. Modify floodwall proposed

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundati	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

Elevation Used

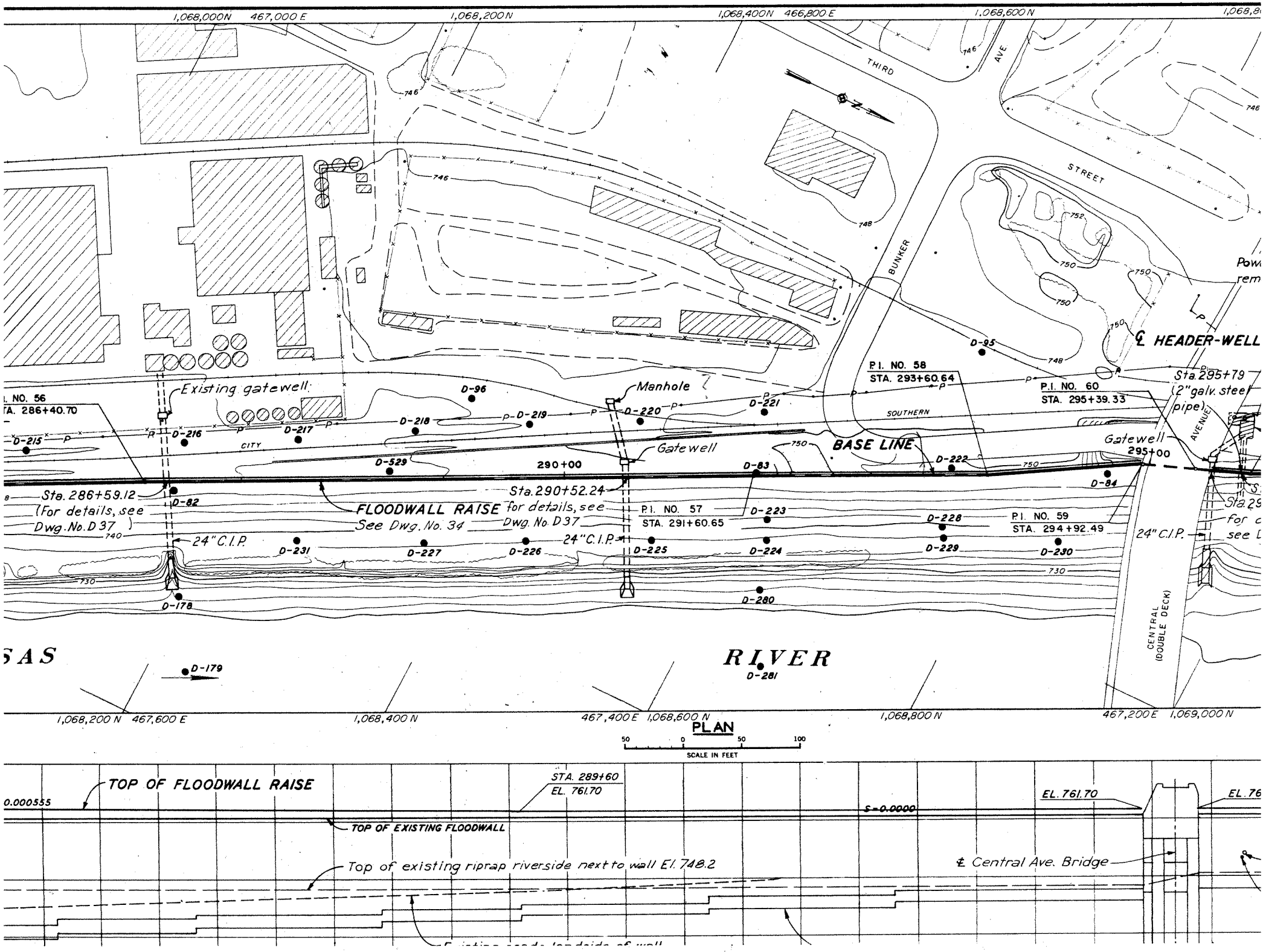
Top of Existing Levee:	761.7 (From O&M Manual)
500 year + 0':	761.0 (Lower than existing conditions)
500 year + 3':	764.0
500 year + 5':	766.0
Ground Surface Land Side:	747.0
Ground Surface River Side:	732.0 (To obtain average thickness of blanket due to slope)
Bottom of Blanket:	720.0
Top of Bedrock:	678.0

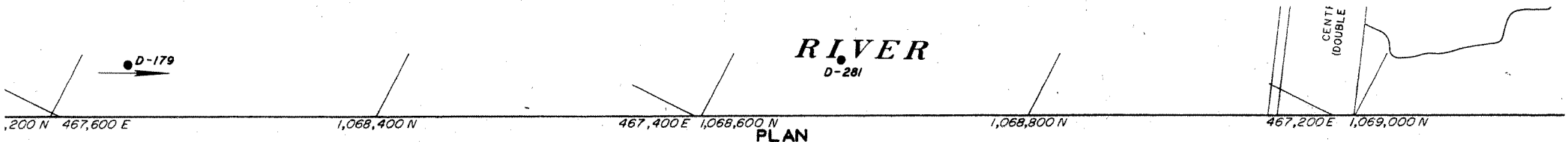
Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Station			Input Parameters								Remarks
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	
289+60	Existing Conditions		400	400	27.0	20.0	12.0	42.0	80	14.7	EC
289+60	n500 + 0		400	400	27.0	20.0	12.0	42.0	80	13.4	n500
289+60	n500 +3		400	400	27.0	20.0	12.0	42.0	80	16.4	n500+3
289+60	n500 + 5		400	400	27.0	20.0	12.0	42.0	80	18.4	n500+5

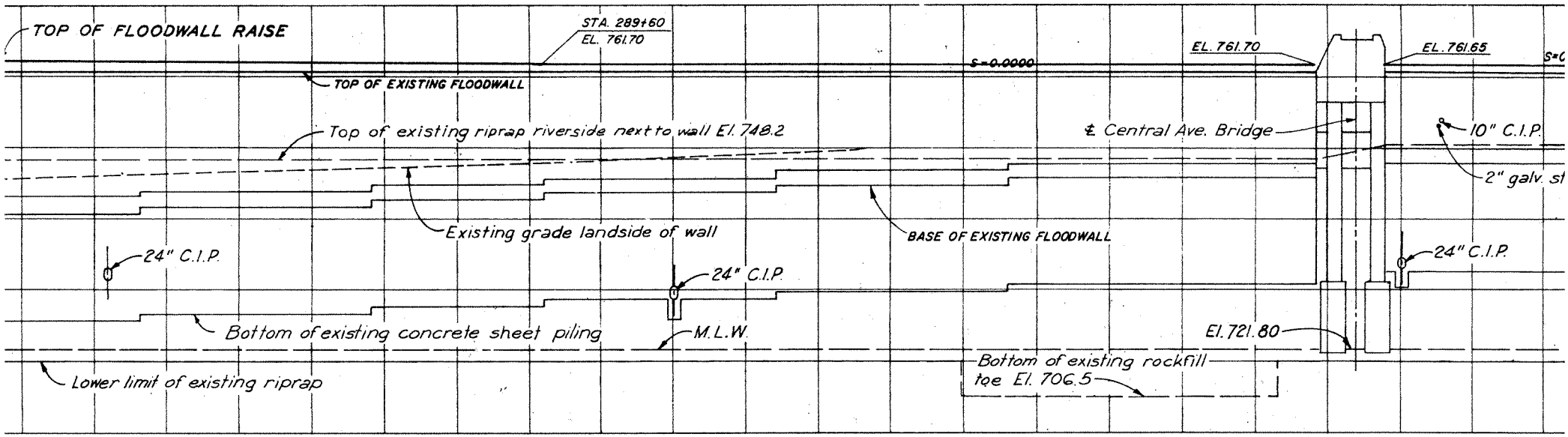
Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
449	673	79	50	673	803	12.33	0.46	0.84	1.84		
449	673	79	50	673	803	11.24	0.42	0.84	2.02		
449	673	79	50	673	803	13.76	0.51	0.84	1.65		
449	673	79	50	673	803	15.44	0.57	0.84	1.47		


1,068,000N 467,000 E
1,068,200N
1,068,400N 466,800 E
1,068,600N
THIRD
AVE
STREET
BUNKER
SOUTHERN
CITY
AVENUE
℄ HEADER-WELL
P.I. NO. 58
STA. 293+60.64
P.I. NO. 60
STA. 295+39.33
Sta. 295+79
(2" galv. steel pipe)
Existing gatewell
Manhole
Gatewell
BASE LINE
Gatewell
295+00
290+00
Sta. 286+59.12
(For details, see Dwg. No. D 37)
Sta. 290+52.24
FLOODWALL RAISE for details, see Dwg. No. D37
See Dwg. No. 34
P.I. NO. 57
STA. 291+60.65
P.I. NO. 59
STA. 294+92.49
24" C.I.P.
CENTRAL (DOUBLE DECK)
D-95
D-96
D-215
D-216
D-217
D-218
D-219
D-220
D-221
D-222
D-223
D-224
D-225
D-226
D-227
D-228
D-229
D-230
D-231
D-529
D-83
D-84
D-82
D-178
D-179
D-280
D-281
RIVER
1,068,200 N 467,600 E
1,068,400 N
467,400 E 1,068,600 N
1,068,800 N
467,200 E 1,069,000 N
PLAN
SCALE IN FEET
TOP OF FLOODWALL RAISE
STA. 289+60
EL. 761.70
EL. 761.70
S=0.0000
TOP OF EXISTING FLOODWALL
Top of existing riprap riverside next to wall El. 748.2
℄ Central Ave. Bridge

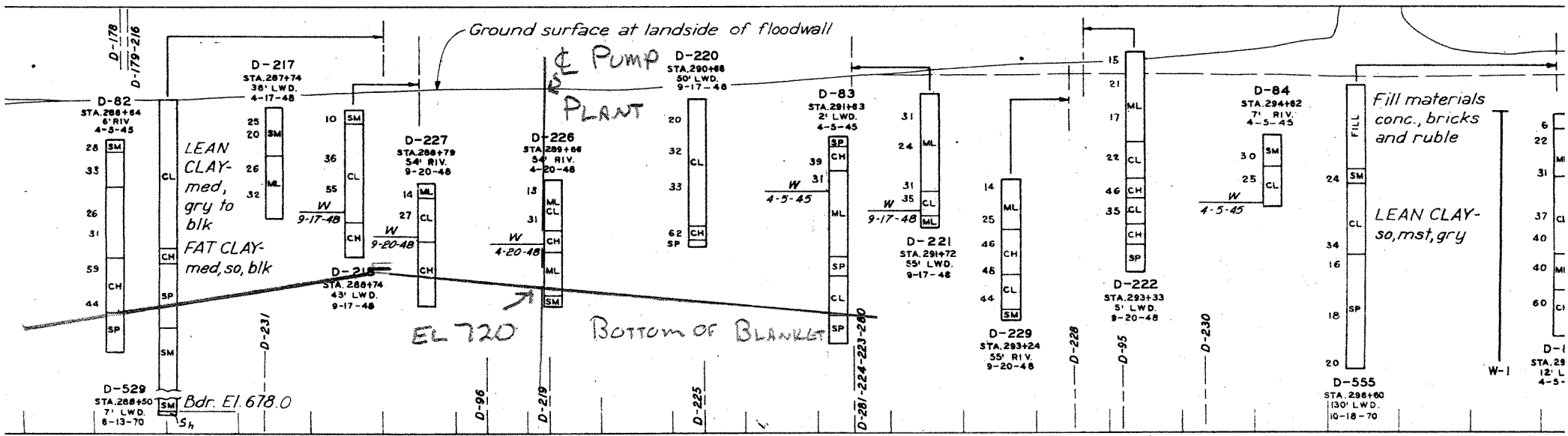


PLAN

SCALE IN FEET 50 0 50 100



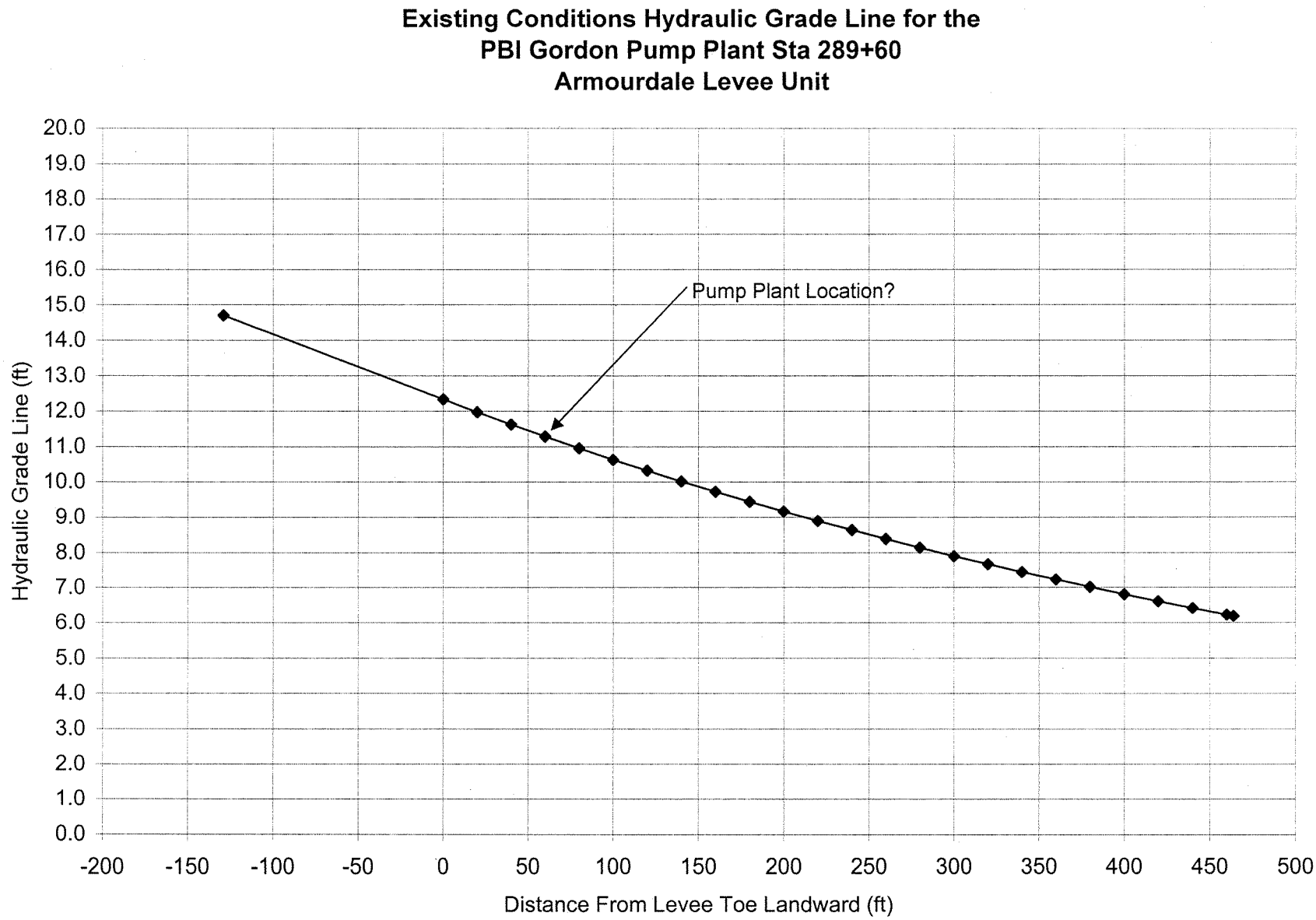
PROFILE



UNDERGROUND EXPLORATIONS

Hydraulic Grade Line for the PBI Gordon Pump Plant
Existing Conditions

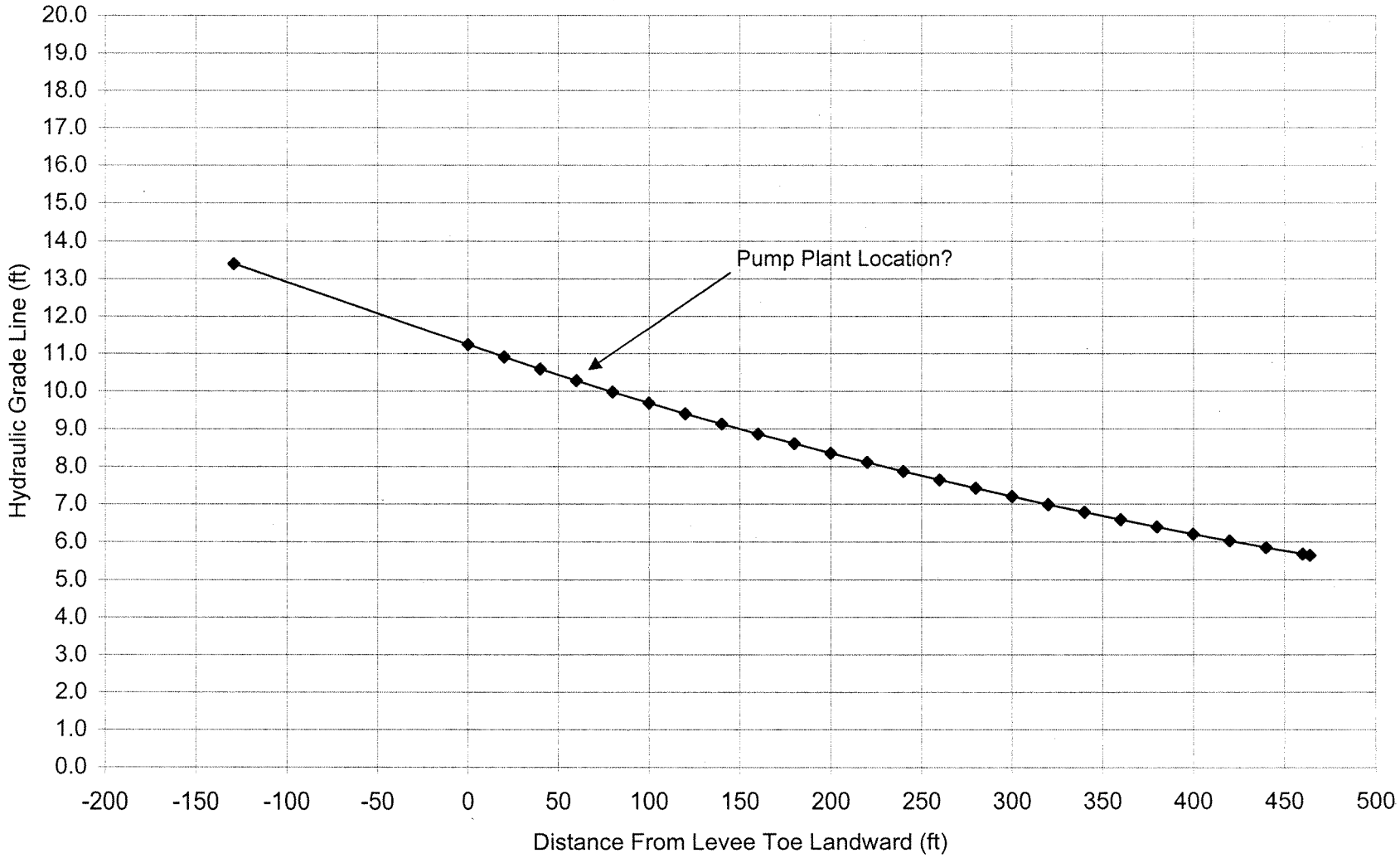
Elevation Top of levee (MSL)	761.7	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	14.7		
L1 (ft)	79		
L2 (ft)	50		
Le (ft)	673		
ho	12.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-129	14.70	761.70
	0	12.33	759.33
	20	11.97	758.97
	40	11.62	758.62
Pump Plant Location?	60	11.28	758.28
	80	10.95	757.95
	100	10.63	757.63
	120	10.32	757.32
	140	10.02	757.02
	160	9.73	756.73
	180	9.44	756.44
	200	9.17	756.17
	220	8.90	755.90
	240	8.64	755.64
	260	8.38	755.38
	280	8.14	755.14
	300	7.90	754.90
	320	7.67	754.67
	340	7.45	754.45
	360	7.23	754.23
	380	7.02	754.02
	400	6.81	753.81
	420	6.61	753.61
	440	6.42	753.42
	460	6.23	753.23
	464	6.19	753.19


Existing Conditions Hydraulic Grade Line for the
PBI Gordon Pump Plant Sta 289+60
Armourdale Levee Unit
Pump Plant Location?
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the PBI Gordon Pump Plant
Design Conditions n500+0

Elevation Top of levee (MSL)	761	n500+0	
Elevation at Landside Levee Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	13.4		
L1 (ft)	79		
L2 (ft)	50		
Le (ft)	673		
ho	11.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-129	13.40	760.40
	0	11.24	758.24
	20	10.91	757.91
	40	10.60	757.60
Pump Plant Location?	60	10.29	757.29
	80	9.98	756.98
	100	9.69	756.69
	120	9.41	756.41
	140	9.13	756.13
	160	8.87	755.87
	180	8.61	755.61
	200	8.35	755.35
	220	8.11	755.11
	240	7.87	754.87
	260	7.64	754.64
	280	7.42	754.42
	300	7.20	754.20
	320	6.99	753.99
	340	6.79	753.79
	360	6.59	753.59
	380	6.40	753.40
	400	6.21	753.21
	420	6.03	753.03
	440	5.85	752.85
	460	5.68	752.68
	464	5.65	752.65

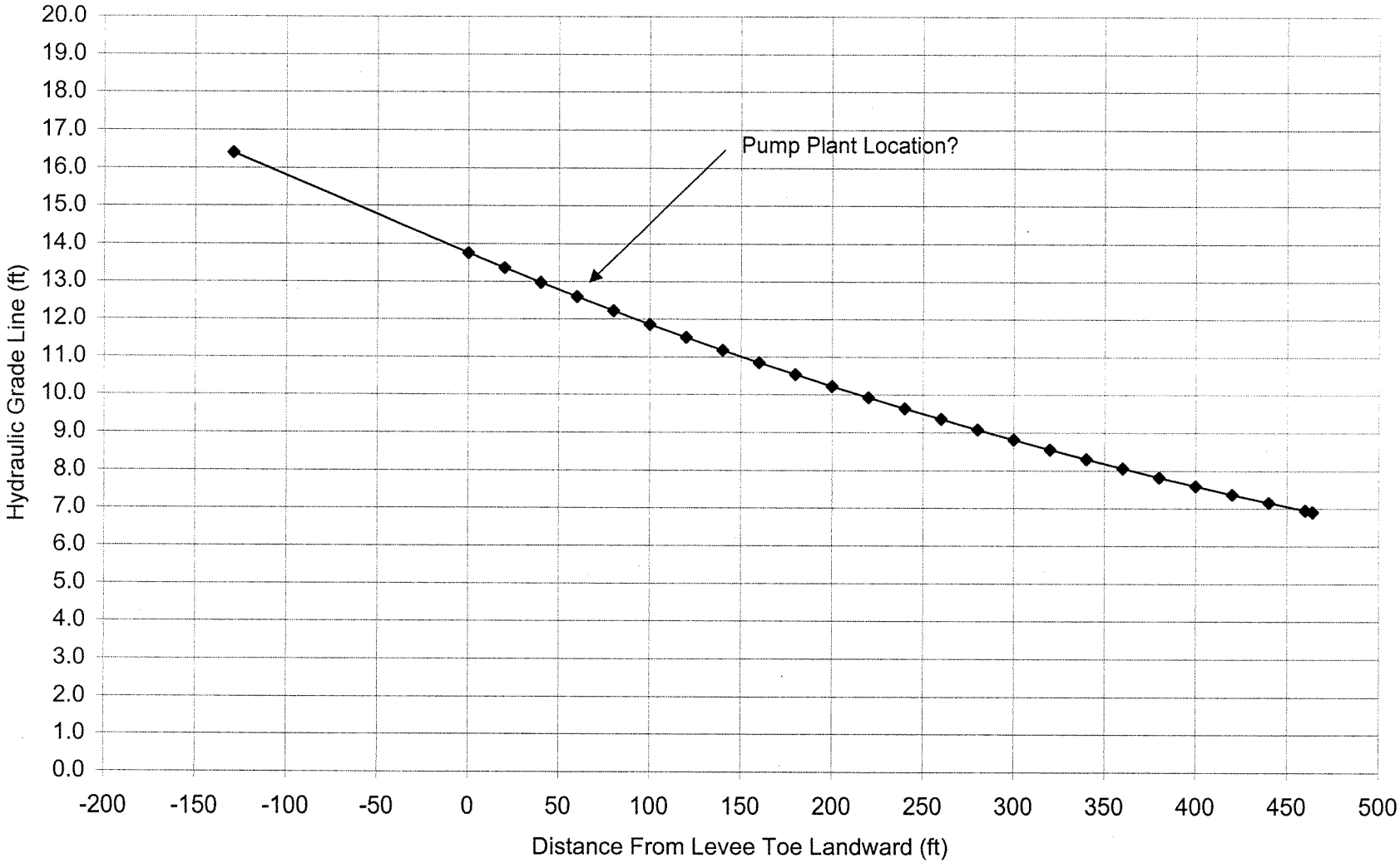
Design Condition n500+0 Hydraulic Grade Line for the
PBI Gordon Pump Plant Sta 289+60
Armourdale Levee Unit



Hydraulic Grade Line for the PBI Gordon Pump Plant
Design Conditions n500+3

Elevation New Top of levee (MSL)	764	n500 +3	
Elevation at Landside Levee Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	16.4		
L1 (ft)	79		
L2 (ft)	50		
Le (ft)	673		
ho	13.76		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-129	16.40	763.40
	0	13.76	760.76
	20	13.36	760.36
	40	12.97	759.97
Pump Plant Location?	60	12.59	759.59
	80	12.22	759.22
	100	11.86	758.86
	120	11.52	758.52
	140	11.18	758.18
	160	10.85	757.85
	180	10.53	757.53
	200	10.23	757.23
	220	9.93	756.93
	240	9.64	756.64
	260	9.35	756.35
	280	9.08	756.08
	300	8.81	755.81
	320	8.56	755.56
	340	8.31	755.31
	360	8.06	755.06
	380	7.83	754.83
	400	7.60	754.60
	420	7.38	754.38
	440	7.16	754.16
	460	6.95	753.95
	464	6.91	753.91

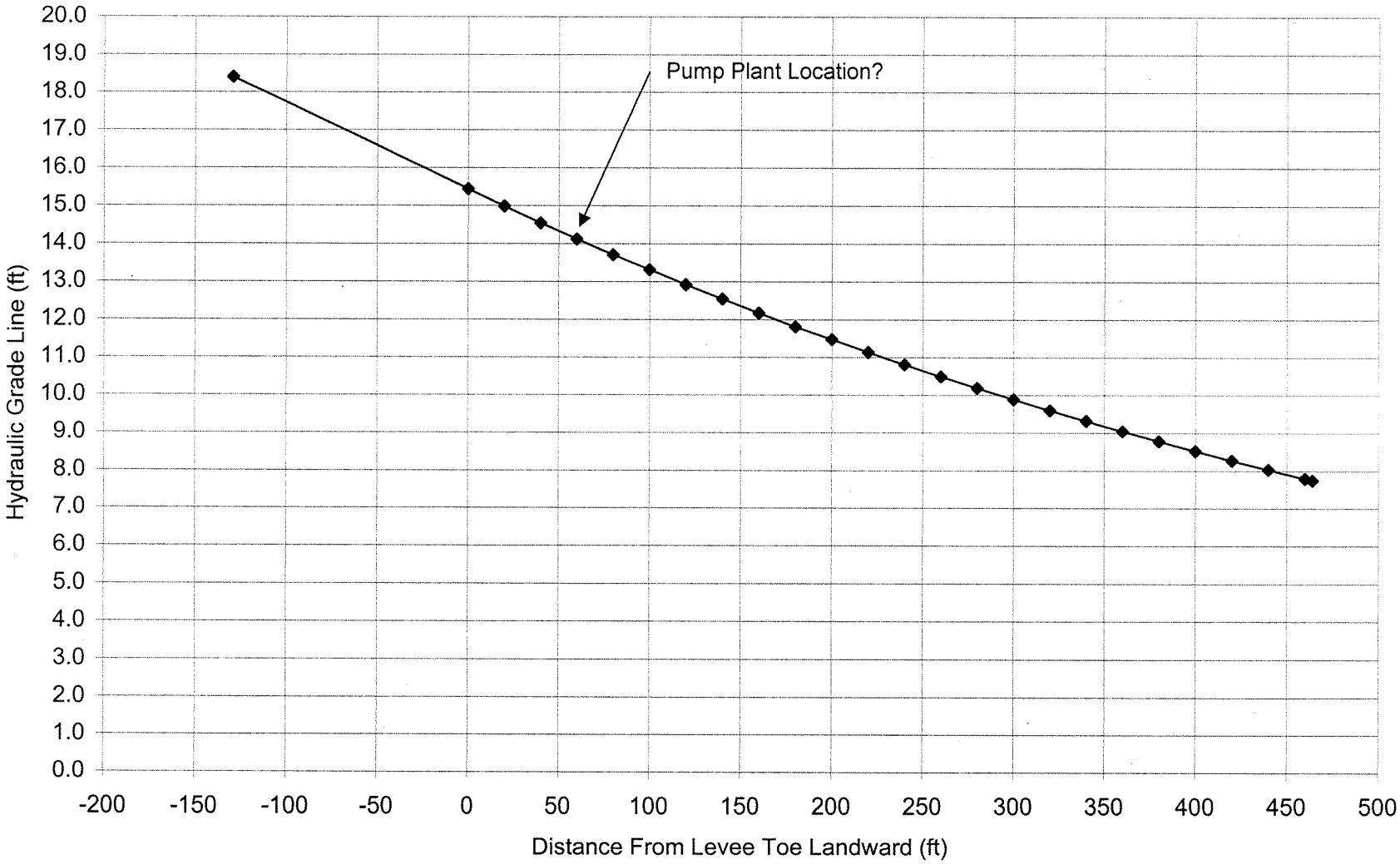
Design Condition n500+3 Hydraulic Grade Line for the
PBI Gordon Pump Plant Sta 289+60
Armourdale Levee Unit



Hydraulic Grade Line for the PBI Gordon Pump Plant
Design Conditions n500+5

Elevation New Top of levee (MSL)	766	n500 +5	
Elevation at Landside Levee Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	18.4		
L1 (ft)	79		
L2 (ft)	50		
Le (ft)	673		
ho	15.44		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-129	18.40	765.40
	0	15.44	762.44
	20	14.99	761.99
	40	14.55	761.55
Pump Plant Location?	60	14.12	761.12
	80	13.71	760.71
	100	13.31	760.31
	120	12.92	759.92
	140	12.54	759.54
	160	12.17	759.17
	180	11.82	758.82
	200	11.47	758.47
	220	11.14	758.14
	240	10.81	757.81
	260	10.49	757.49
	280	10.19	757.19
	300	9.89	756.89
	320	9.60	756.60
	340	9.32	756.32
	360	9.05	756.05
	380	8.78	755.78
	400	8.52	755.52
	420	8.28	755.28
	440	8.03	755.03
	460	7.80	754.80
	464	7.75	754.75

Design Condition n500+5 Hydraulic Grade Line for the
PBI Gordon Pump Plant Sta 289+60
Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the National Beef Pump Station Sta 295+80
Assuming No Wells

ARM National Beef PP 295+50 No Wells.xls

NOMENCLATURE

NATIONAL BEEF PUMP PLANT
FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD GMB
Date : 08/18/07

NOTES:
1. Replace floodwall with landside levee proposed
2. L_2 revised for design conditions due to new levee

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2L/R/C-1)}) / (e^{(2L/R/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

Elevation Used

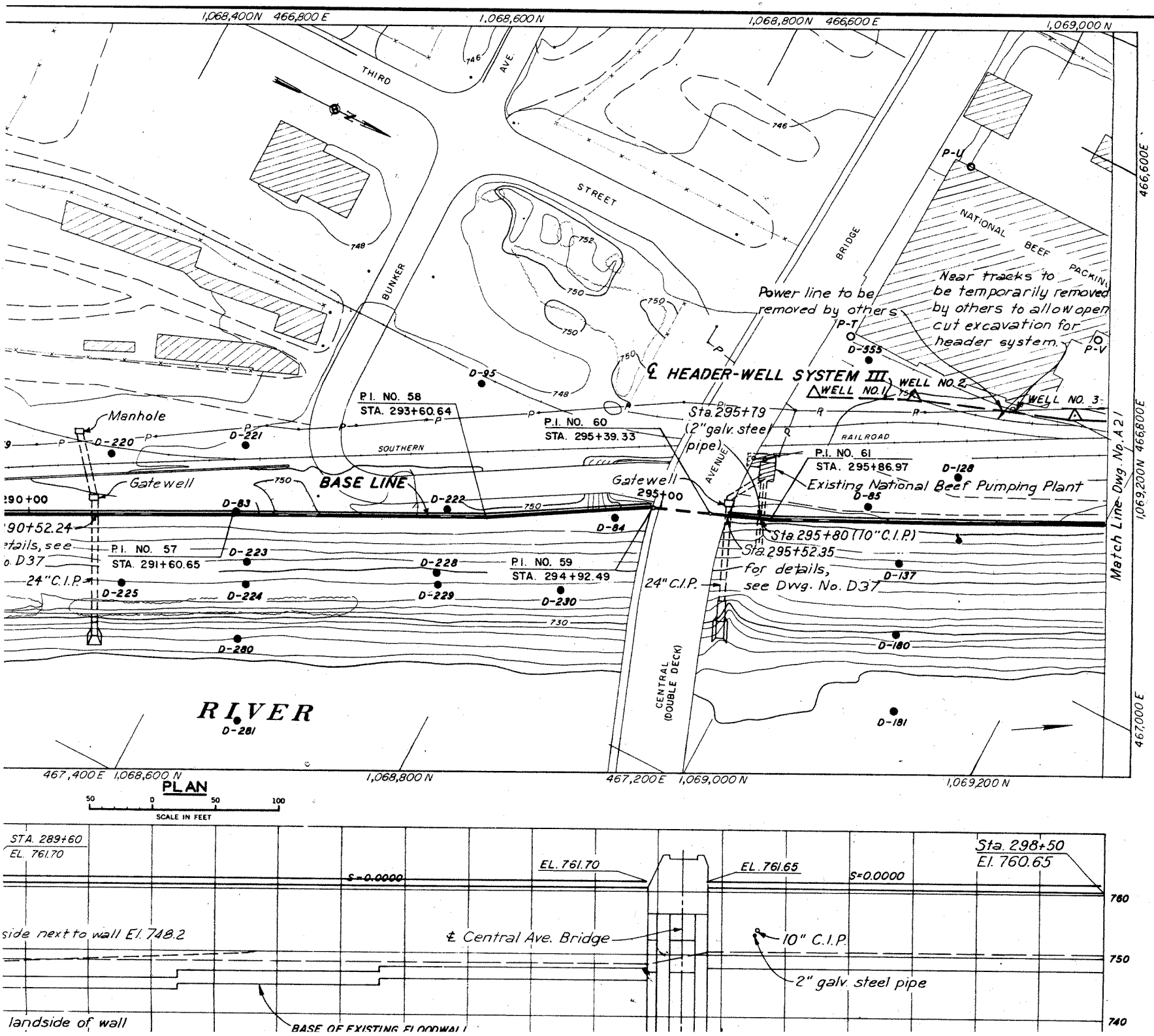
Top of Existing Levee:	761.7	(From O&M Manual)
500 year + 0':	760.4	(Lower than existing conditions)
500 year + 3':	763.4	
500 year + 5':	765.4	
Ground Surface Land Side:	750.0	(For existing conditions)
Ground Surface River Side:	738.0	(To obtain average thickness of blanket due to slope)
Bottom of Blanket:	726.0	
Top of Bedrock:	678.0	

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

			Input Parameters								
Station			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks
295+80	Existing Conditions		400	400	24.0	18.0	12.0	48.0	80	11.7	EC
295+80	n500 + 0		400	400	24.0	18.0	12.0	48.0	50	10.4	n500
295+80	n500 +3		400	400	24.0	18.0	12.0	48.0	50	13.4	n500+3
295+80	n500 + 5		400	400	24.0	18.0	12.0	48.0	50	15.4	n500+5

				Analysis										
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
480	679	79	20	679	778	10.21	0.43	0.84	1.98		
480	679	50	100	679	829	8.52	0.35	0.84	2.37		
480	679	50	100	679	829	10.98	0.46	0.84	1.84		
480	679	50	100	679	829	12.62	0.53	0.84	1.60		



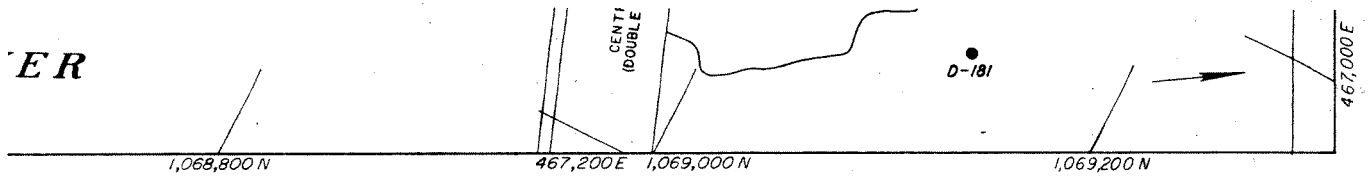
NATIONAL BEEF PUMP PLANT.

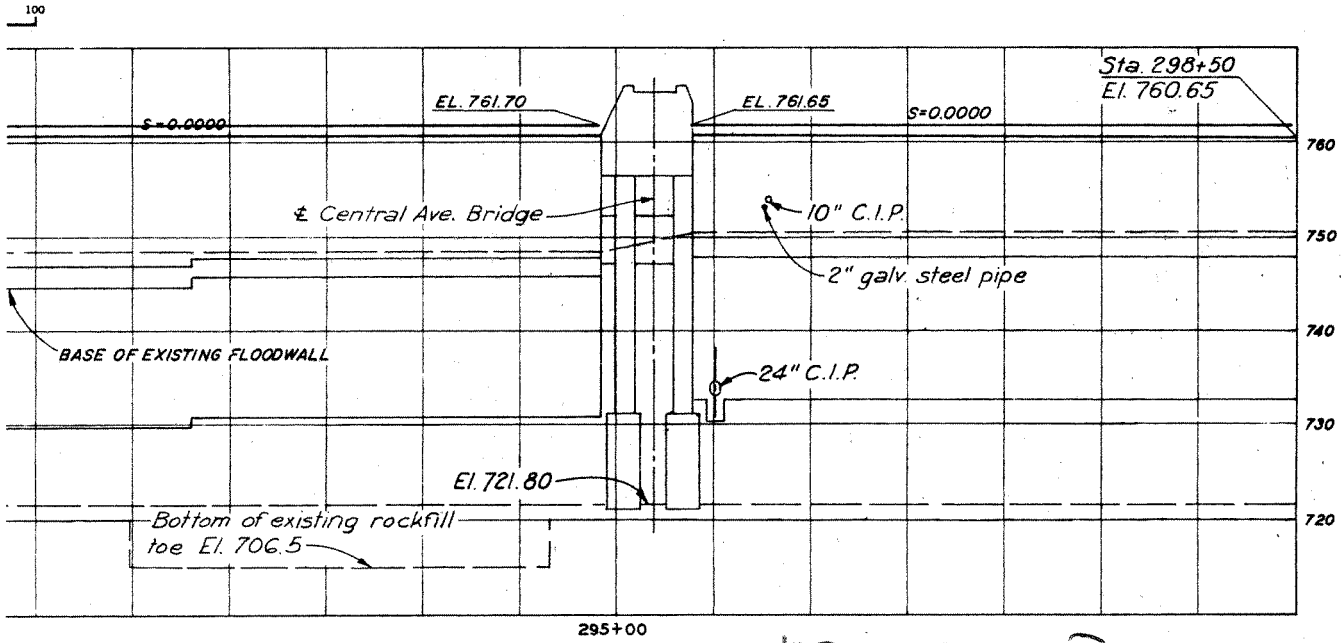
NOTES:

For General Notes, s

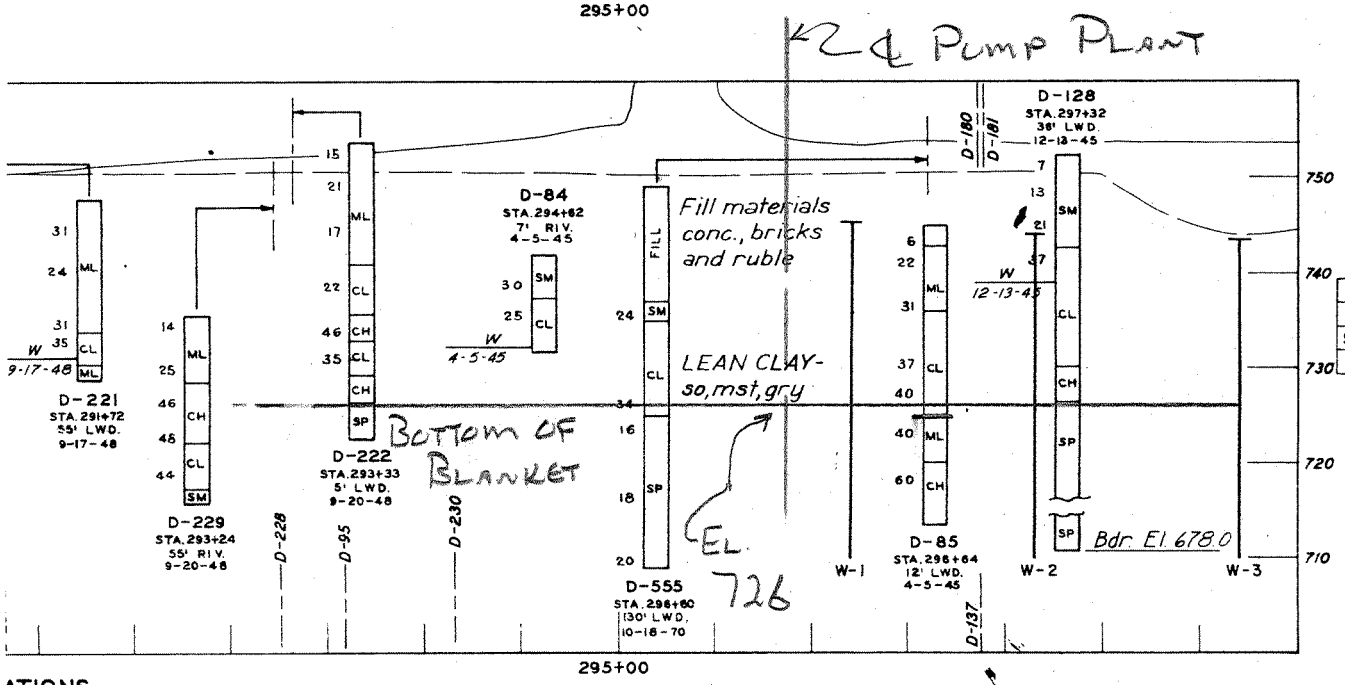
For details of relie piezometers, see Dwg.

For Header Profile Dwg. No. C1





NOTES:
For General Notes, see Dwg. No. A3
For details of relief wells, manholes & piezometers, see Dwg. Nos. C2 & C3
For Header Profile-Well System III, see Dwg. No. C1
For Legend - Existing Utilities, see Dwg. No. A3



ATIONS

RECORD DRAWING

MAY 1976

CONTRACT NO. DACW41-74-C-0093

SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" conditions	9/20/78	

REVISIONS

KANSAS RIVER, KANSAS

KANSAS CITY, KANSAS - 1962 MOD.
ARMOURDALE UNIT
PLAN, PROFILE AND UNDERGROUND EXPLORATIONS
STA. 284+00 TO STA. 298+50

In 109 sheets — Sheet No. 20 — Scale: as shown

Dwg. No. A20

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

U. S. ARMY
MAY 1973

Submitted: CHIEF, LOCAL PROTECTION SECTION
Recommended: CHIEF, FOUNDATIONS & MATLS. BR.
Approved: CHIEF, ENGINEERING DIVISION

COMPILED BY: T.A.K.-J.A.H. DRAWN BY: J.E.B. CHECKED BY: R.L.B.

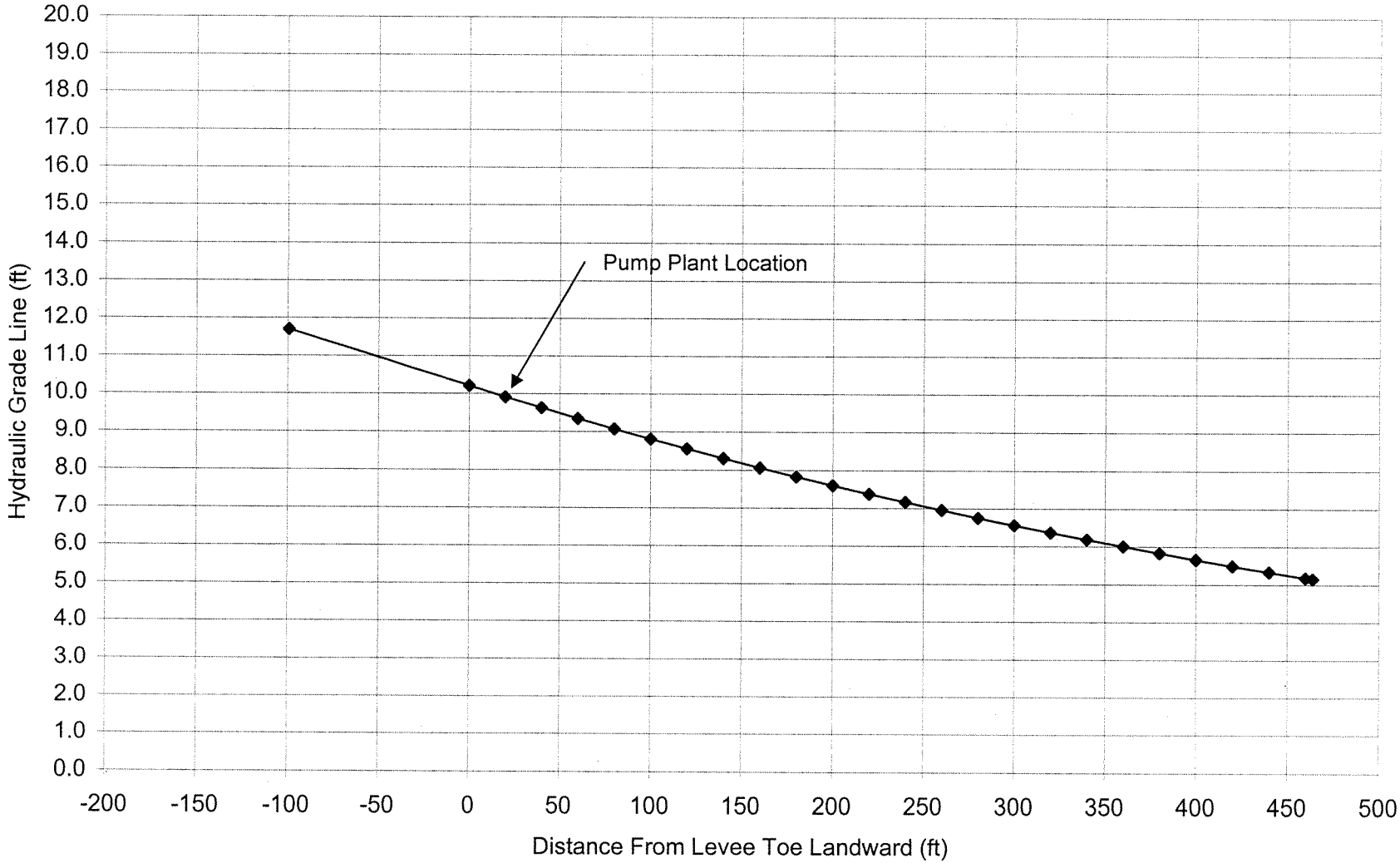
FILE NO. A-10-6020

O & M M PLATE NO. 182

Hydraulic Grade Line for the National Beef Pump Plant
Existing Conditions - Assuming No Wells

Elevation Top of levee (MSL)	761.65	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	11.7		
L1 (ft)	79		
L2 (ft)	20		
Le (ft)	679		
ho	10.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-99	11.70	761.70
	0	10.21	760.21
Pump Plant Location	20	9.91	759.91
	40	9.62	759.62
	60	9.34	759.34
	80	9.07	759.07
	100	8.81	758.81
	120	8.55	758.55
	140	8.31	758.31
	160	8.06	758.06
	180	7.83	757.83
	200	7.60	757.60
	220	7.38	757.38
	240	7.17	757.17
	260	6.96	756.96
	280	6.76	756.76
	300	6.56	756.56
	320	6.37	756.37
	340	6.19	756.19
	360	6.01	756.01
	380	5.83	755.83
	400	5.66	755.66
	420	5.50	755.50
	440	5.34	755.34
	460	5.18	755.18
	464	5.15	755.15

Existing Conditions Hydraulic Grade Line for the National Beef Pump Plant Sta 295+80 Armourdale Levee Unit - Assuming No Wells

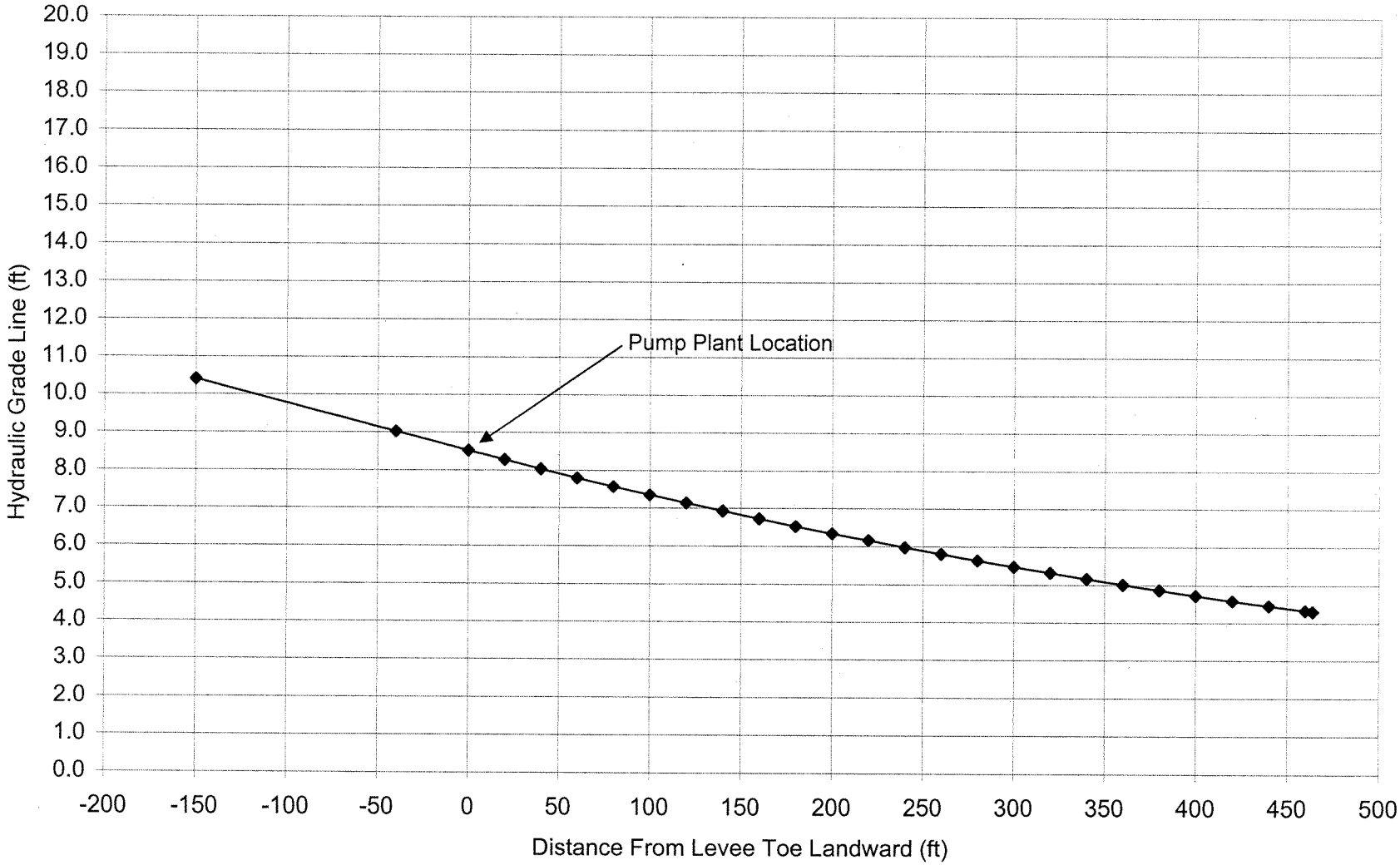


Hydraulic Grade Line for the National Beef Pump Plant
Design Conditions n500+0 - Assuming No Wells

Elevation Top of levee (MSL)	760.4	n500+0
Elevation at Landside Levee Toe (MSL)	750	
Elevation Bottom of Blanket (MSL)	726	
Driving Head (ft)	10.4	
L1 (ft)	50	
L2 (ft)	100	
Le (ft)	679	
ho	8.5	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-150	10.40	760.40
	-40	9.02	759.02
Pump Plant Location	0	8.52	758.52
	20	8.27	758.27
	40	8.03	758.03
	60	7.80	757.80
	80	7.57	757.57
	100	7.35	757.35
	120	7.14	757.14
	140	6.93	756.93
	160	6.73	756.73
	180	6.54	756.54
	200	6.35	756.35
	220	6.16	756.16
	240	5.98	755.98
	260	5.81	755.81
	280	5.64	755.64
	300	5.48	755.48
	320	5.32	755.32
	340	5.16	755.16
	360	5.01	755.01
	380	4.87	754.87
	400	4.73	754.73
	420	4.59	754.59
	440	4.46	754.46
	460	4.33	754.33
	464	4.30	754.30

Design Condition n500+0 Hydraulic Grade Line for the National Beef Pump Plant Sta 295+80 Armourdale Levee Unit - Assuming No Wells

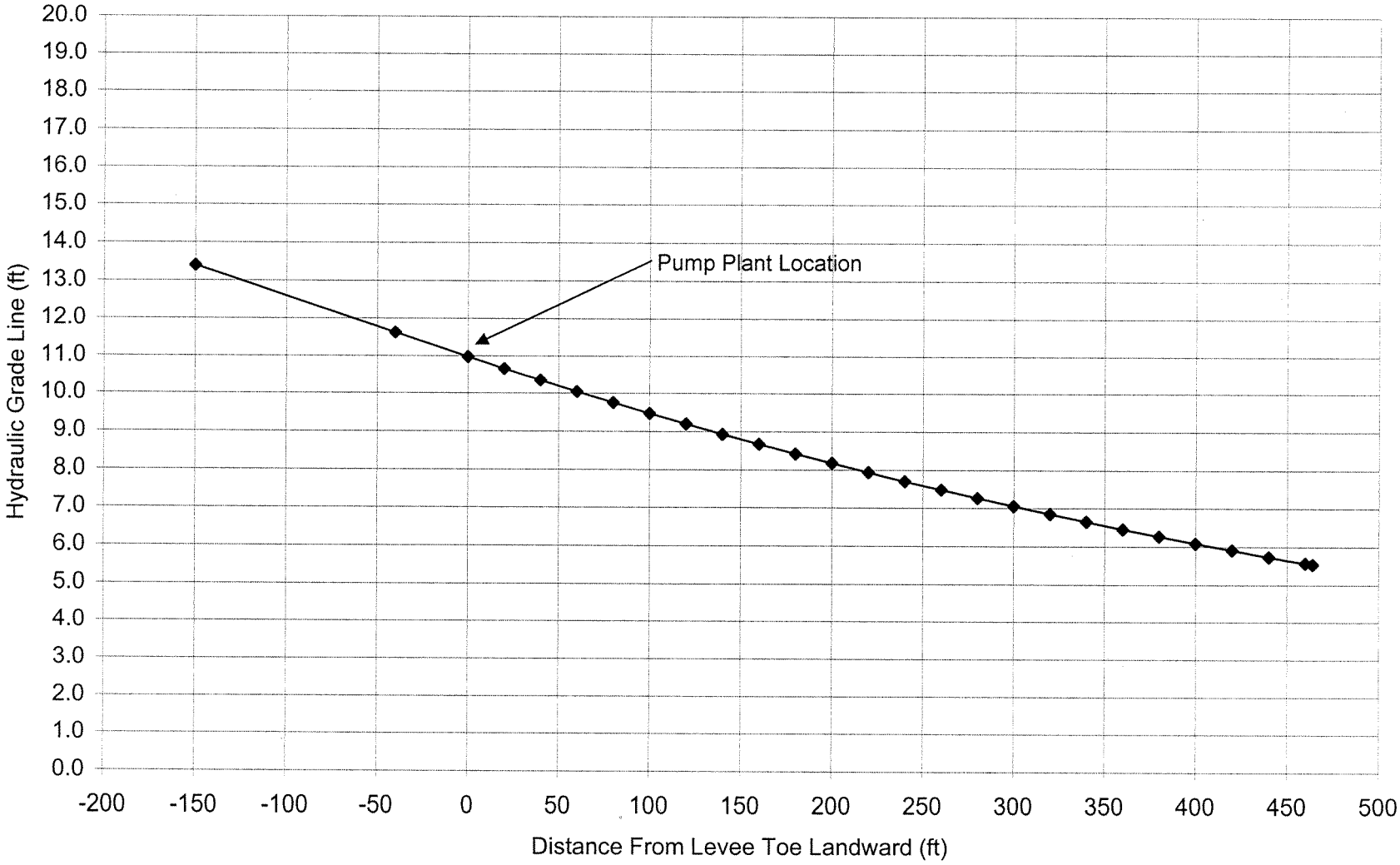


Hydraulic Grade Line for the National Beef Pump Plant
Design Conditions n500+3 - Assuming No Wells

Elevation New Top of levee (MSL)	763.4	n500 +3
Elevation at Landside Levee Toe (MSL)	750	
Elevation Bottom of Blanket (MSL)	726	
Driving Head (ft)	13.4	
L1 (ft)	50	
L2 (ft)	100	
Le (ft)	679	
ho	10.98	

	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-150	13.40	763.40
	-40	11.62	761.62
Pump Plant Location	0	10.98	760.98
	20	10.66	760.66
	40	10.35	760.35
	60	10.05	760.05
	80	9.76	759.76
	100	9.47	759.47
	120	9.20	759.20
	140	8.93	758.93
	160	8.67	758.67
	180	8.42	758.42
	200	8.18	758.18
	220	7.94	757.94
	240	7.71	757.71
	260	7.48	757.48
	280	7.27	757.27
	300	7.06	757.06
	320	6.85	756.85
	340	6.65	756.65
	360	6.46	756.46
	380	6.27	756.27
	400	6.09	756.09
	420	5.91	755.91
	440	5.74	755.74
	460	5.57	755.57
	464	5.54	755.54

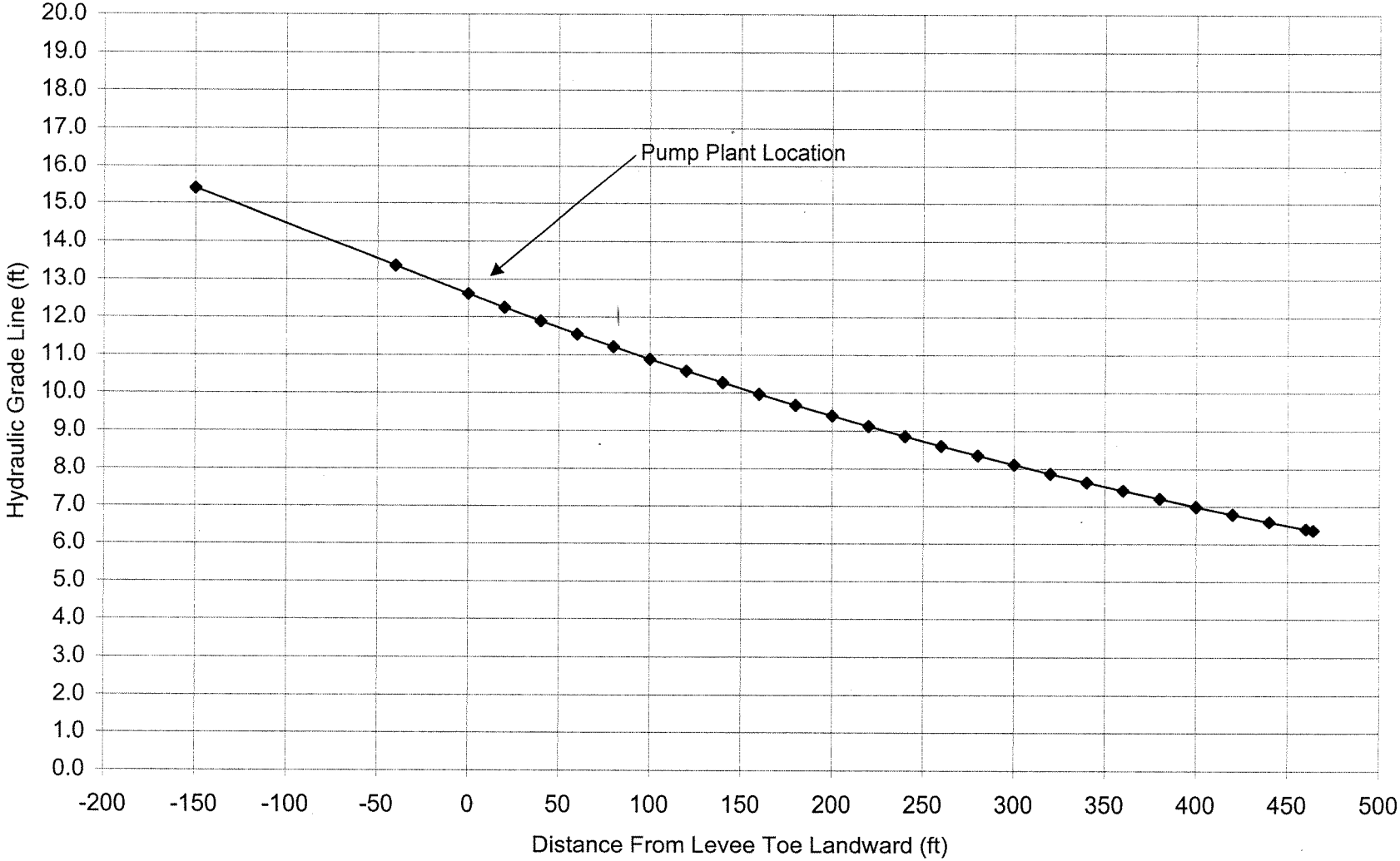
Design Condition n500+3 Hydraulic Grade Line for the National Beef Pump Plant Sta 295+80 Armourdale Levee Unit - Assuming No Wells



Hydraulic Grade Line for the National Beef Pump Plant
Design Conditions n500+5 - Assuming No Wells

Elevation New Top of levee (MSL)	765.4	n500 +5	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	15.4		
L1 (ft)	50		
L2 (ft)	100		
Le (ft)	679		
ho	12.62		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-150	15.40	765.40
	-40	13.36	763.36
Pump Plant Location	0	12.62	762.62
	20	12.25	762.25
	40	11.89	761.89
	60	11.55	761.55
	80	11.21	761.21
	100	10.89	760.89
	120	10.57	760.57
	140	10.26	760.26
	160	9.97	759.97
	180	9.68	759.68
	200	9.40	759.40
	220	9.12	759.12
	240	8.86	758.86
	260	8.60	758.60
	280	8.35	758.35
	300	8.11	758.11
	320	7.87	757.87
	340	7.65	757.65
	360	7.42	757.42
	380	7.21	757.21
	400	7.00	757.00
	420	6.80	756.80
	440	6.60	756.60
	460	6.41	756.41
	464	6.37	756.37

Design Condition n500+5 Hydraulic Grade Line for the National Beef Pump Plant Sta 295+80 Armourdale Levee Unit - Assuming No Wells



Armourdale Kansas - Hydraulic Grade Line for the Central Ave. Pump Station Sta 299+20
Assuming No Wells

ARM Central Ave PP 299+21 No Wells.xls

NOMENCLATURE

CENTRAL AVE PUMP PLANT
FEB 2007

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD
Date : 02/26/02 GMB

NOTES:
1. Replace floodwall with landside levee proposed
2. L_2 revised for design conditions due to new levee

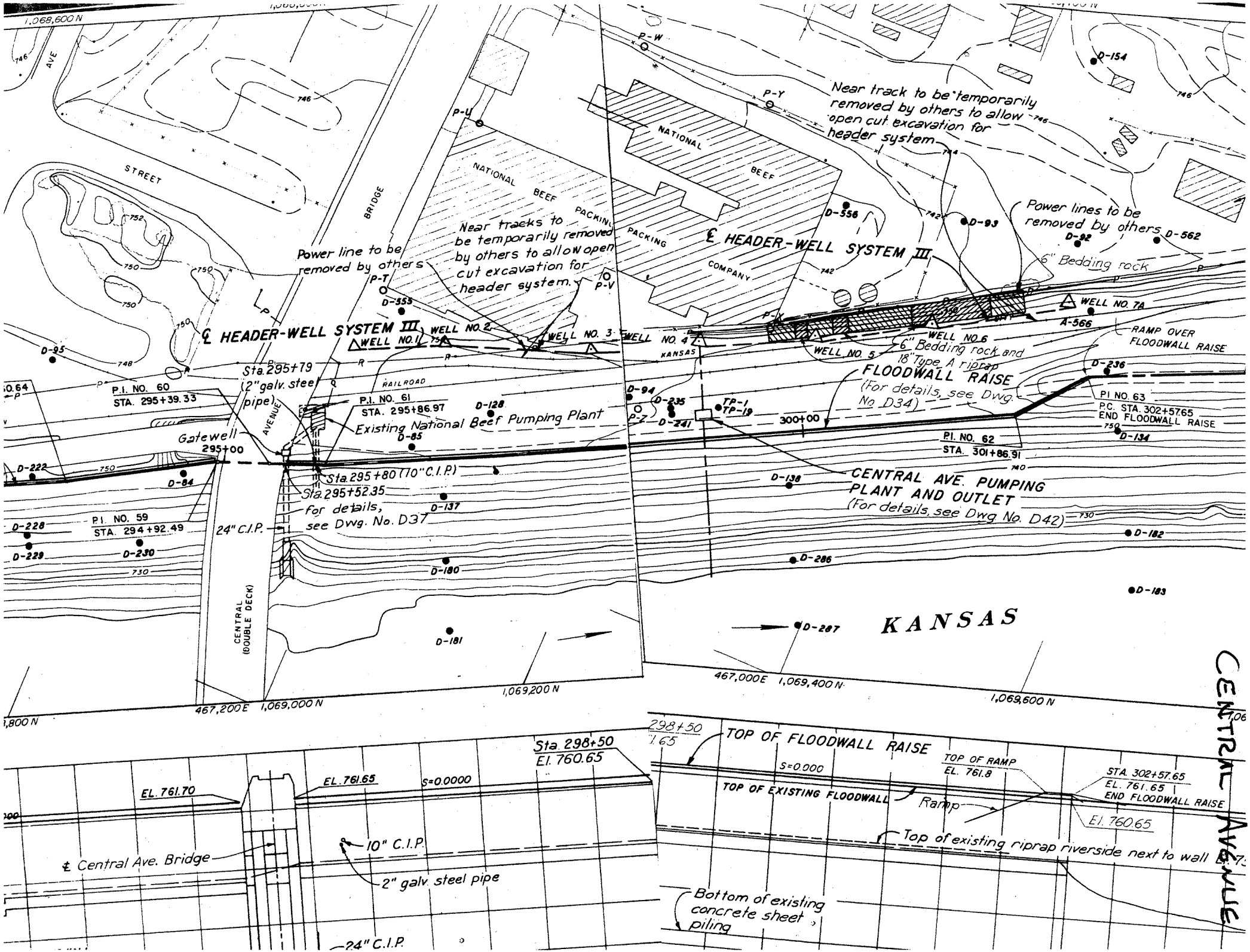
Elevation Used

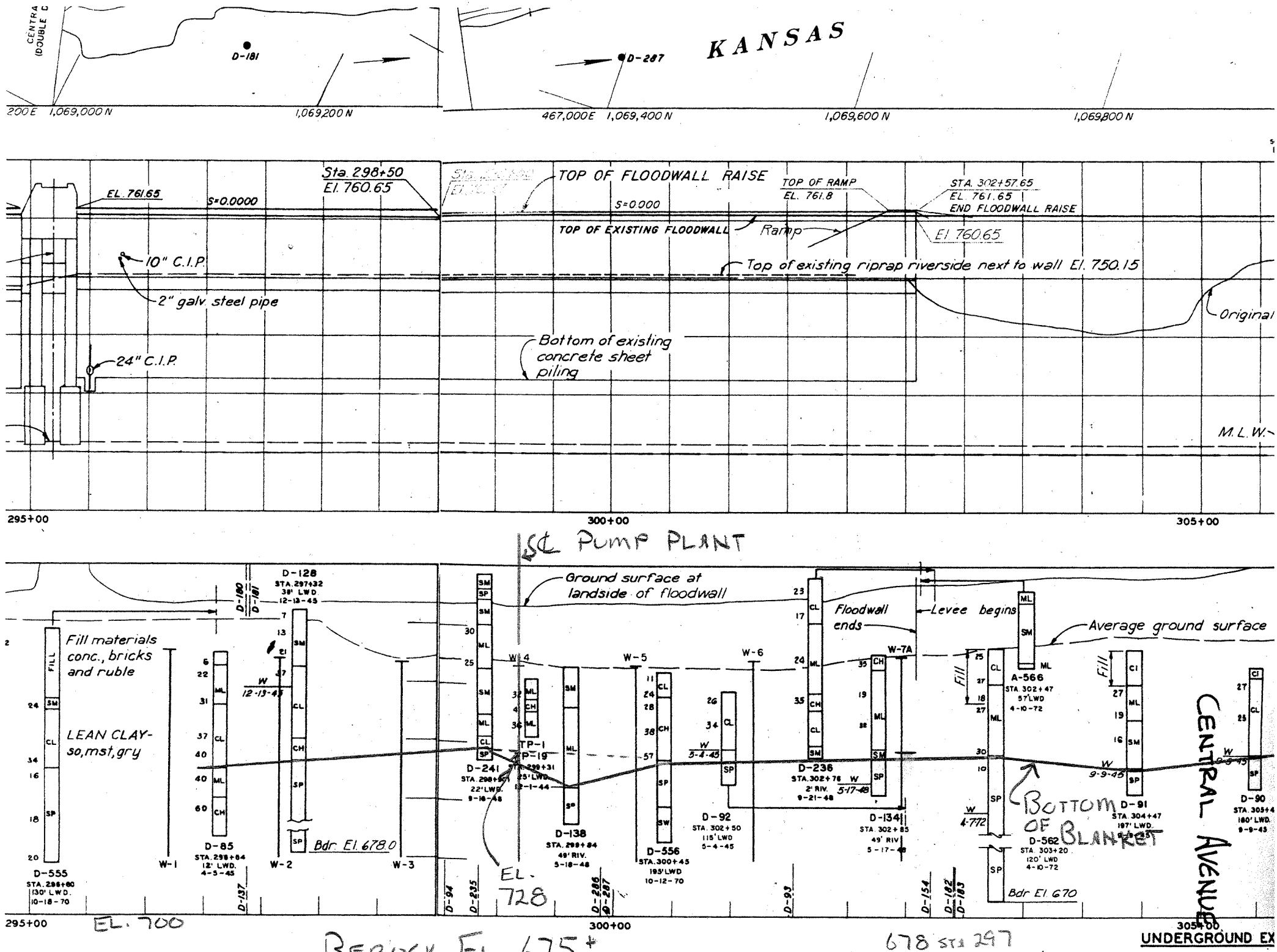
Top of Existing Levee:	761.7 (From O&M Manual)
500 year + 0':	760.1 (Lower than existing conditions)
500 year + 3':	763.1
500 year + 5':	765.1
Ground Surface Land Side:	750.0 (For existing conditions)
Ground Surface Land Side:	742.0 (For design conditions)
Ground Surface River Side:	735.0 (To obtain average thickness of blanket)
Bottom of Blanket:	728.0
Top of Bedrock:	675.0

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

			Input Parameters									Analysis											
Station			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks	C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
299+20	Existing Conditions		300	300	24.0	17.0	7.0	53.0	70	11.7	EC	334	618	69	20	618	707	10.23	0.43	0.84	1.97		
299+20	n500 + 0		300	300	14.0	10.5	7.0	53.0	40	18.1	n500	334	472	40	80	472	592	14.43	1.03	0.84	0.81		
299+20	n500 +3		300	300	14.0	10.5	7.0	53.0	40	21.1	n500+3	334	472	40	80	472	592	16.83	1.20	0.84	0.70		
299+20	n500 + 5		300	300	14.0	10.5	7.0	53.0	40	23.1	n500+5	334	472	40	80	472	592	18.42	1.32	0.84	0.64		

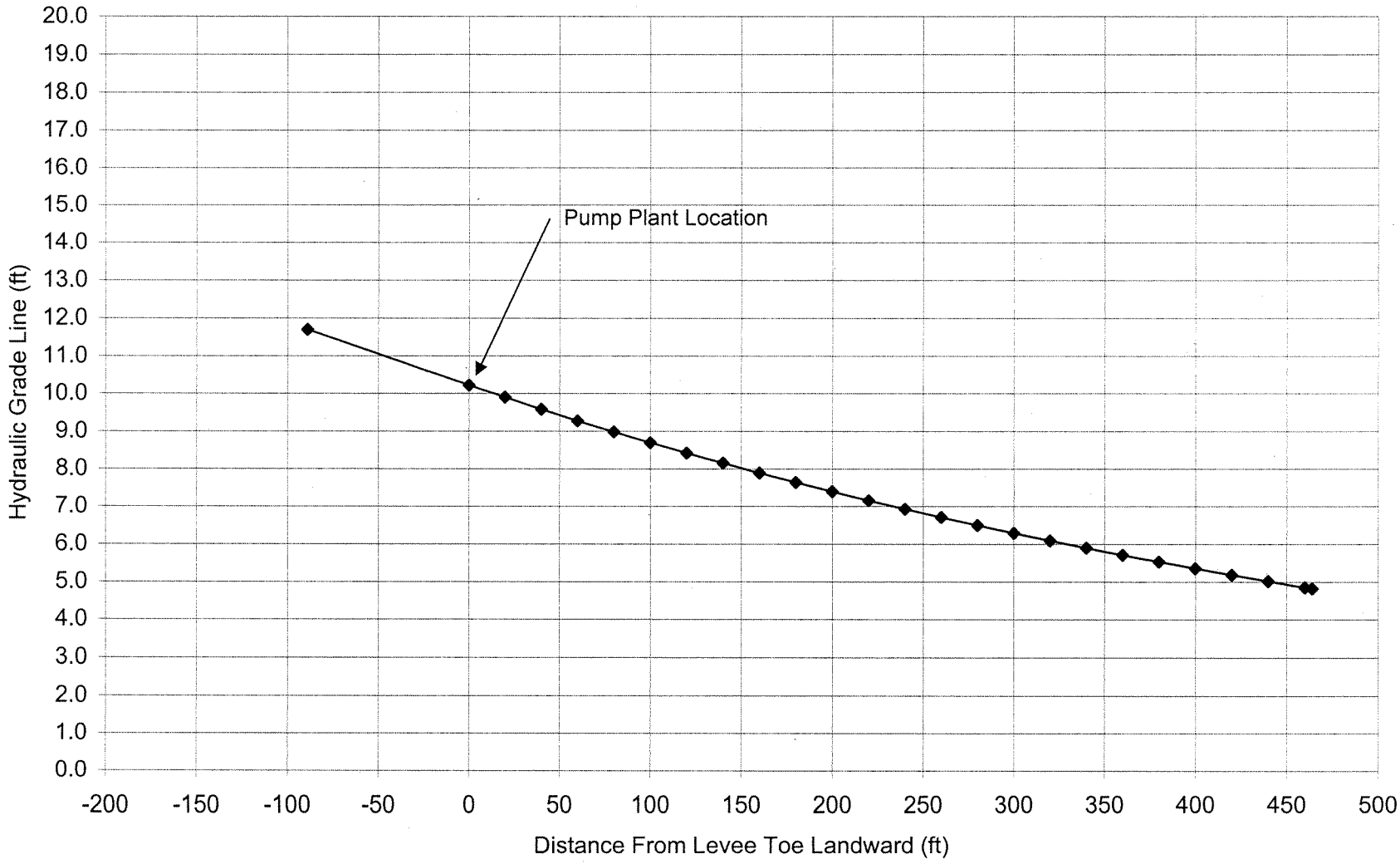

1,068,600 N
STREET
BRIDGE
NATIONAL BEEF PACKING COMPANY
NATIONAL BEEF
Near track to be temporarily removed by others to allow open cut excavation for header system
Power lines to be removed by others
Near tracks to be temporarily removed by others to allow open cut excavation for header system.
Power line to be removed by others
℄ HEADER-WELL SYSTEM III
WELL NO. 1
WELL NO. 2
WELL NO. 3
WELL NO. 4
WELL NO. 5
WELL NO. 6
WELL NO. 7A
6" Bedding rock
6" Bedding rock and 18" Type A riprap
FLOODWALL RAISE
(For details, see Dwg. No. D34)
RAMP OVER FLOODWALL RAISE
P.I. NO. 60
STA. 295+39.33
Sta. 295+79
(2" galv. steel pipe)
P.I. NO. 61
STA. 295+86.97
Existing National Beef Pumping Plant
Gatewell
295+00
300+00
P.I. NO. 62
STA. 301+86.91
P.I. NO. 63
P.C. STA. 302+57.65
END FLOODWALL RAISE
CENTRAL AVE. PUMPING PLANT AND OUTLET
(For details, see Dwg. No. D42)
Sta. 295+80 (10" C.I.P.)
Sta. 295+52.35
For details, see Dwg. No. D37
24" C.I.P.
P.I. NO. 59
STA. 294+92.49
CENTRAL (DOUBLE DECK)
KANSAS
467,200E
1,069,000 N
1,069,200 N
467,000E
1,069,400 N
1,069,600 N
Sta. 298+50
El. 760.65
EL. 761.70
EL. 761.65
S=0.0000
℄ Central Ave. Bridge
10" C.I.P.
2" galv. steel pipe
24" C.I.P.
298+50
TOP OF FLOODWALL RAISE
S=0.000
TOP OF RAMP
EL. 761.8
STA. 302+57.65
EL. 761.65
END FLOODWALL RAISE
TOP OF EXISTING FLOODWALL
Ramp
El. 760.65
Top of existing riprap riverside next to wall
Bottom of existing concrete sheet piling
CENTRAL AVENUE


CENTRA
(DOUBLE C
D-181
D-287
KANSAS
200 E 1,069,000 N
1,069,200 N
467,000E 1,069,400 N
1,069,600 N
1,069,800 N
EL. 761.65
S=0.0000
Sta. 298+50
El. 760.65
TOP OF FLOODWALL RAISE
TOP OF RAMP
EL. 761.8
STA. 302+57.65
EL. 761.65
END FLOODWALL RAISE
S=0.000
TOP OF EXISTING FLOODWALL
Ramp
El. 760.65
10" C.I.P.
Top of existing riprap riverside next to wall El. 750.15
2" galv steel pipe
Original
Bottom of existing concrete sheet piling
24" C.I.P.
M.L.W.
295+00
300+00
305+00
℄ PUMP PLANT
D-128
STA. 297+32
38' LWD
12-13-45
D-180
D-181
Ground surface at landside of floodwall
Floodwall ends
Levee begins
Average ground surface
Fill materials conc., bricks and ruble
LEAN CLAY- so,mst,gry
W-4
W-5
W-6
W-7A
A-566
STA. 302+47
57' LWD
4-10-72
D-555
STA. 296+80
130' LWD.
10-18-70
D-85
STA. 298+64
12' LWD.
4-5-45
W-1
W-2
W-3
Bdr El. 678.0
D-241
STA. 298+50
22' LWD.
9-16-48
TP-1
TP-19
D-138
STA. 299+84
49' RIV.
5-18-48
D-556
STA. 300+45
195' LWD
10-12-70
D-92
STA. 302+50
115' LWD
5-4-45
D-236
STA. 302+76
2' RIV.
9-21-48
D-134
STA. 302+85
49' RIV
5-17-48
D-562
STA. 303+20
120' LWD
4-10-72
Bdr El. 670
D-91
STA. 304+47
197' LWD.
D-90
STA. 305+4
180' LWD.
9-9-45
BOTTOM OF BLANKET
CENTRAL AVENUE
EL. 728
EL. 700
BEROCK EL 675±
678 STA 297
670 STA 302+50
UNDERGROUND EX

Hydraulic Grade Line for the Central Avenue Pump Plant
Existing Conditions - Assuming No Wells

Elevation Top of levee (MSL)	761.65	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	728		
Driving Head (ft)	11.7		
L1 (ft)	69		
L2 (ft)	20		
Le (ft)	618		
ho	10.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-89	11.70	761.70
Pump Plant Location	0	10.23	760.23
	20	9.90	759.90
	40	9.59	759.59
	60	9.28	759.28
	80	8.98	758.98
	100	8.70	758.70
	120	8.42	758.42
	140	8.15	758.15
	160	7.89	757.89
	180	7.64	757.64
	200	7.40	757.40
	220	7.16	757.16
	240	6.93	756.93
	260	6.71	756.71
	280	6.50	756.50
	300	6.29	756.29
	320	6.09	756.09
	340	5.90	755.90
	360	5.71	755.71
	380	5.53	755.53
	400	5.35	755.35
	420	5.18	755.18
	440	5.02	755.02
	460	4.86	754.86
	464	4.83	754.83

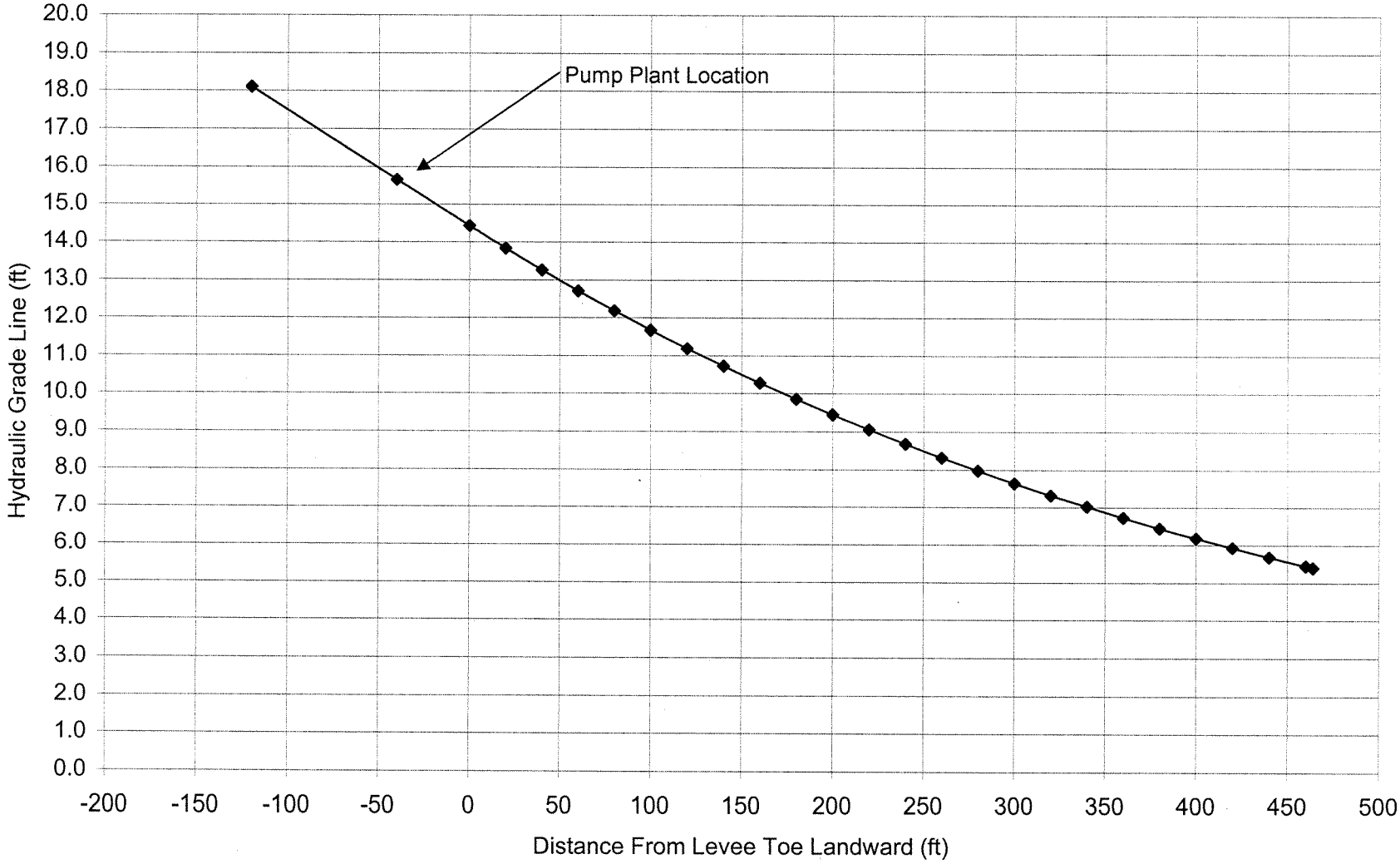
Existing Conditions Hydraulic Grade Line for the Central Avenue Pump Plant Sta 299+20 Armourdale Levee Unit - Assuming No Wells



Hydraulic Grade Line for the Central Avenue Pump Plant
Design Conditions n500+0 - Assuming No Wells

Elevation Top of levee (MSL)	760.1	n500+0	
Elevation at Landside Levee Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	728		
Driving Head (ft)	18.1		
L1 (ft)	40		
L2 (ft)	80		
Le (ft)	472		
ho	14.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	18.10	760.10
Pump Plant Location	-40	15.66	757.66
	0	14.43	756.43
	20	13.84	755.84
	40	13.26	755.26
	60	12.71	754.71
	80	12.18	754.18
	100	11.68	753.68
	120	11.19	753.19
	140	10.73	752.73
	160	10.28	752.28
	180	9.86	751.86
	200	9.45	751.45
	220	9.06	751.06
	240	8.68	750.68
	260	8.32	750.32
	280	7.97	749.97
	300	7.64	749.64
	320	7.33	749.33
	340	7.02	749.02
	360	6.73	748.73
	380	6.45	748.45
	400	6.18	748.18
	420	5.93	747.93
	440	5.68	747.68
	460	5.44	747.44
	464	5.40	747.40

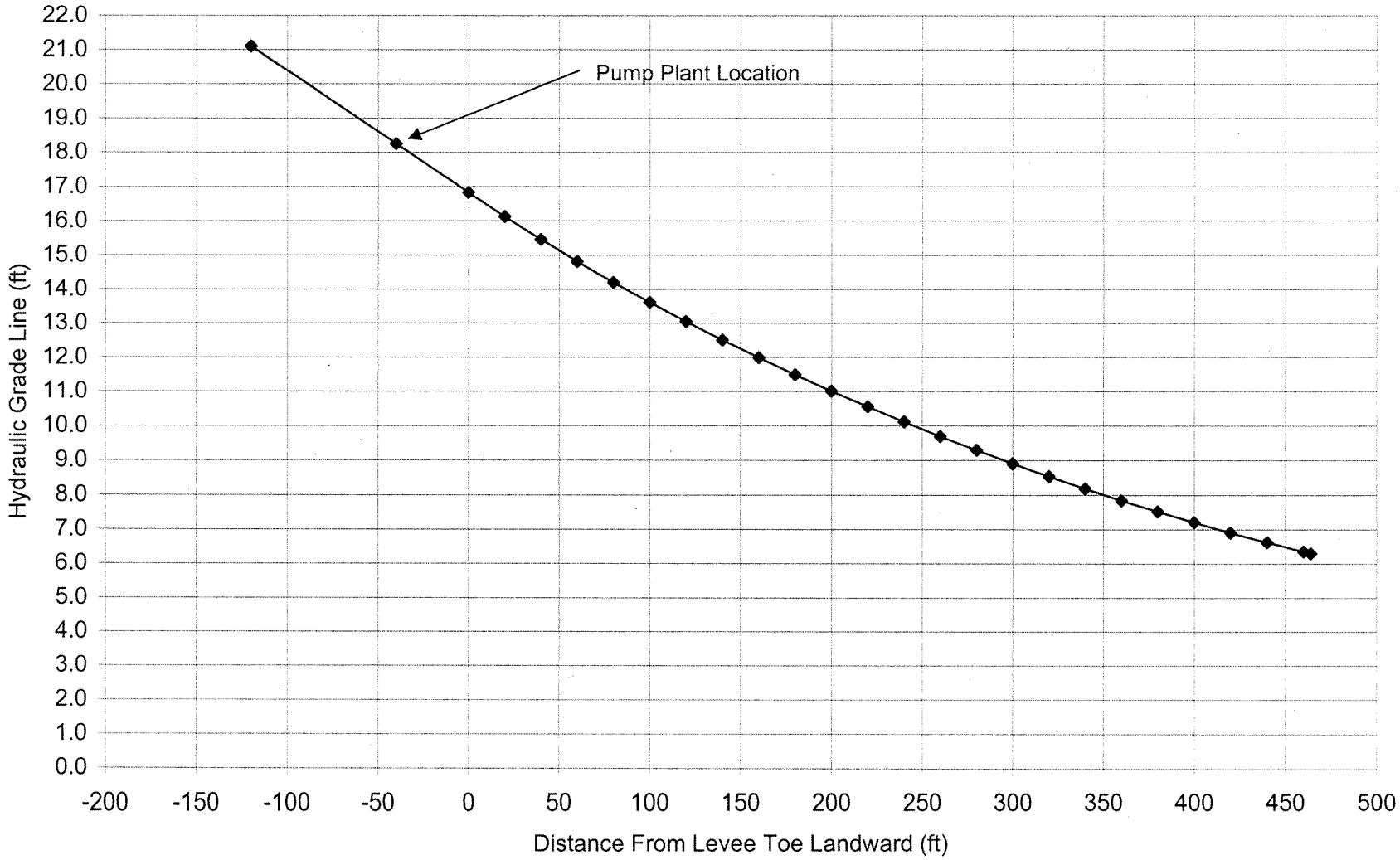
Design Condition n500+0 Hydraulic Grade Line for the Central Avenue Pump Plant Sta 299+20 Armourdale Levee Unit - Assuming No Wells



Hydraulic Grade Line for the Central Avenue Pump Plant
Design Conditions n500+3 - Assuming No Wells

Elevation New Top of levee (MSL)	763.1	n500 +3	
Elevation at Landside Levee Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	728		
Driving Head (ft)	21.1		
L1 (ft)	40		
L2 (ft)	80		
Le (ft)	472		
ho	16.83		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
	-120	21.10	763.10
Pump Plant Location	-40	18.25	760.25
	0	16.83	758.83
	20	16.13	758.13
	40	15.46	757.46
	60	14.82	756.82
	80	14.20	756.20
	100	13.61	755.61
	120	13.05	755.05
	140	12.51	754.51
	160	11.99	753.99
	180	11.49	753.49
	200	11.01	753.01
	220	10.56	752.56
	240	10.12	752.12
	260	9.70	751.70
	280	9.30	751.30
	300	8.91	750.91
	320	8.54	750.54
	340	8.19	750.19
	360	7.85	749.85
	380	7.52	749.52
	400	7.21	749.21
	420	6.91	748.91
	440	6.62	748.62
	460	6.35	748.35
	464	6.29	748.29

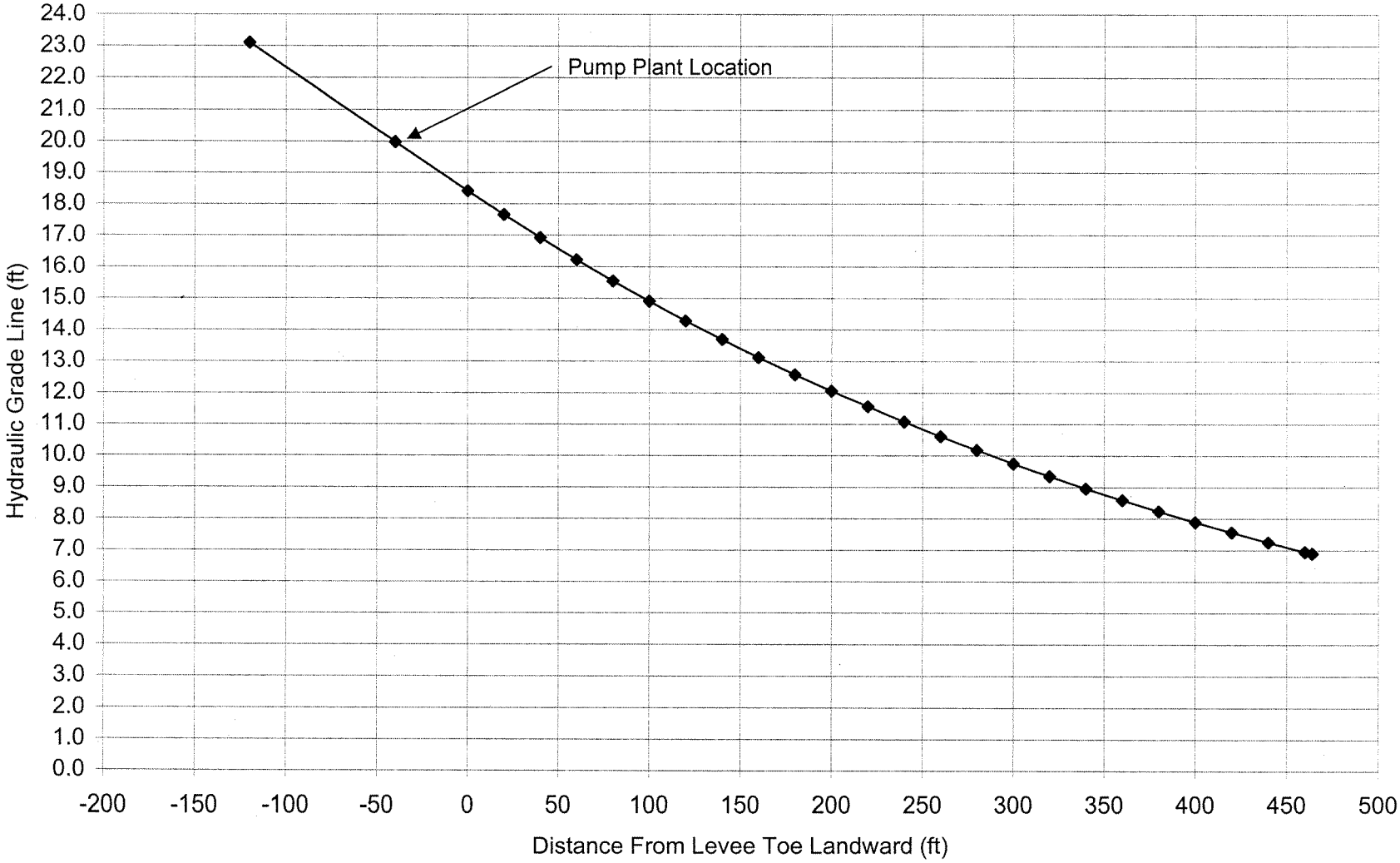
Design Condition n500+3 Hydraulic Grade Line for the Central Avenue Pump Plant Sta 299+20 Armourdale Levee Unit - Assuming No Wells



Hydraulic Grade Line for the Central Avenue Pump Plant
Design Conditions n500+5 - Assuming No Wells

Elevation New Top of levee (MSL)	765.1	n500 +5	
Elevation at Landside Levee Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	728		
Driving Head (ft)	23.1		
L1 (ft)	40		
L2 (ft)	80		
Le (ft)	472		
ho	18.42		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-120	23.10	765.10
Pump Plant Location	-40	19.98	761.98
	0	18.42	760.42
	20	17.66	759.66
	40	16.92	758.92
	60	16.22	758.22
	80	15.55	757.55
	100	14.90	756.90
	120	14.28	756.28
	140	13.69	755.69
	160	13.12	755.12
	180	12.58	754.58
	200	12.06	754.06
	220	11.56	753.56
	240	11.08	753.08
	260	10.62	752.62
	280	10.18	752.18
	300	9.75	751.75
	320	9.35	751.35
	340	8.96	750.96
	360	8.59	750.59
	380	8.23	750.23
	400	7.89	749.89
	420	7.56	749.56
	440	7.25	749.25
	460	6.95	748.95
	464	6.89	748.89

Design Condition n500+5 Hydraulic Grade Line for the Central Avenue Pump Plant Sta 299+20 Armourdale Levee Unit - Assuming No Wells



Kansas City's Levees Phase II Feasibility Study

Pump Station Structural Analysis

(EC-DS)

Parks, Marvin L NWK

From: Bolte, William G [William_G_Bolte@RL.gov]
Sent: Tuesday, November 28, 2006 10:13 AM
To: Parks, Marvin L NWK
Subject: RE: Pump Station Peer Review Back Checks

Attachments: image004.jpg; image005.jpg; image003.jpg

image004.jpg (35 KB) image005.jpg (52 KB) image003.jpg (52 KB)

Sounds good. I'd consider my peer review done. If you need me to sign something, otherwise just initial WGB on the PDF files.

From: Parks, Marvin L NWK [mailto:Marvin.L.Parks@nwk02.usace.army.mil]
Sent: Tuesday, November 28, 2006 8:07 AM
To: Bolte, William G
Cc: Sivakumar, Sinniah NWK; Parks, Marvin L NWK
Subject: RE: Pump Station Peer Review Back Checks

Bill;

Above is my response. The attached file has been updated to reflect my response.

Marvin

From: Bolte, William G [mailto:William_G_Bolte@RL.gov]
Sent: Tuesday, November 28, 2006 9:36 AM
To: Parks, Marvin L NWK
Cc: Sivakumar, Sinniah NWK
Subject: RE: Pump Station Peer Review Back Checks

Marvin,

I reviewed the attached files. One question remains. I didn't see a response or an update to the Shawnee Mission N500+3ft HGL. Based on the attached spreadsheet it appears the HGL may have in fact even changed (765.5 vs 764). Just coordinate with geotechs and make sure the correct number is used.

Bill

From: Sivakumar, Sinniah NWK [mailto:Sinniah.Sivakumar@nwk02.usace.army.mil]
Sent: Tuesday, November 28, 2006 6:12 AM
To: Bolte, William NWK; Bolte, William G
Cc: Parks, Marvin L NWK
Subject: FW: Pump Station Peer Review Back Checks

See below for your action.

From: Parks, Marvin L NWK
Sent: Tuesday, November 28, 2006 7:24 AM
To: Jansen, Ronald G NWK
Cc: Sivakumar, Sinniah NWK; Parks, Marvin L NWK; Muller, Paul D NWK
Subject: Pump Station Peer Review Back Checks

Ron;

Below is the information that I will forward to Bill Bolte for back check once I get his email address from Siv. Bill and I spoke by phone yesterday on this subject. This information is being provided to let you know where we are at in the review process and the actions that are in progress.

Regards;

Marvin

3253

Bill;

Attached are the Adobe files of the pump station structural calculations after peer review. These documents show your comments and my response. The following has to be kept in mind in regards to the analysis:

1. The files reflect the 500+3 HGL values that were posted by EC-GD in August or prior. Those values have since changed.

2. Because of the many changes to the HGL values necessitating calculation rework, the team decided in September to focus instead on existing conditions.

3. Although the HGL values used in final analysis is likely to change (especially pump stations with relief wells for the design condition), the methodology shown in the attached Adobe files will not change unless the peer review dictates otherwise.

Below is a screen shot of the files.

Also attached is the Excel spreadsheet used to keep track of the HGL values for EC and design conditions for reference.

In summary, Osage, KCS, Central and Shawnee pump stations have been reviewed but require back check. My response to your comments needs to be back checked. If my responses are acceptable, please use the following:

1. Select “Tools” from the Adobe menu.

2. Select “Commenting” -> “Stamps” -> “Dynamic”

3. Select “Reviewed”. The “Reviewed” stamp will appear. Place the stamp at the top of the first page using the mouse.

4. Double click on the stamp. A comment balloon will appear. Type “EC-DS peer review completed”.

Hope this makes sense. My number is 816.389.3253 if you have any questions.

Regards;

Marvin

Address K:\MissionProjects\civ\kansas_citys\Structures\Phase 2\Pump Stations\Pump Station Analysis\After Peer Review

Name	Size	Type	Date Modified
Central PS 299+20plus3.pdf	130 KB	Adobe Acrobat 7.0 Document	8/26/2006 1:51 PM
KCS PS 276+79 +3 .pdf	82 KB	Adobe Acrobat 7.0 Document	8/26/2006 1:05 PM
Osage PS 76+83 +3 .pdf	138 KB	Adobe Acrobat 7.0 Document	8/26/2006 12:29 PM
Shawnee PS 230+78+3.pdf	74 KB	Adobe Acrobat 7.0 Document	11/28/2006 10:02 AM
Central PS 299+20plus3 after peer review.xmcd	960 KB	Mathcad Document	8/26/2006 1:52 PM
KCS PS 276+79 +3 after peer review.xmcd	862 KB	Mathcad Document	8/26/2006 1:05 PM
Osage PS 76+83 +3 after peer review.xmcd	1,079 KB	Mathcad Document	8/26/2006 12:42 PM
Bill's Memo.pdf	16 KB	Adobe Acrobat 7.0 Document	11/29/2006 8:03 AM
KCL Phase II Pump Stations EC_DS Calculations....	429 KB	Adobe Acrobat 7.0 Document	11/29/2006 8:16 AM

KANSAS CITYS LEVEES PHASE II

Variables

$kip := 1000lb \quad plf := \frac{lb}{ft} \quad psf \equiv \frac{lb}{ft^2} \quad pcf := \frac{lb}{ft^3} \quad psi \equiv \frac{lb}{in^2} \quad ksi \equiv \frac{1000lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad kips := 1000lb \quad klf := \frac{kips}{ft}$

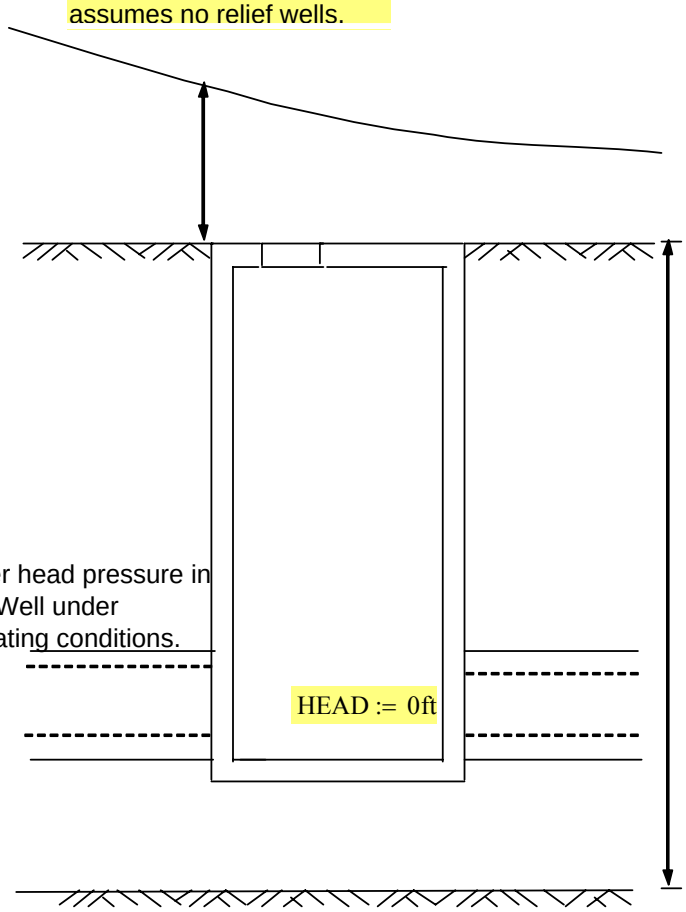
Notes:
1. Walls analyzed for two-way action, plate fixed along three edges, hinged along one edge (top slab connection), uniformly varying load. Figure 13 of the Bureau monographs.
2. Base slab analyzed for two-way action, plate fixed along four edges, uniform load. Figure 34 of Bureau monographs.
3. Two walls were analyzed.

REFERENCES
1. Moments and Reactions for Rectangular Plates, Monograph No. 27, Bueau of Reclamation, July 1963

Properties

NOTE: HGL Supplied assumes no relief wells.

Original Hydraulic Grade Line as supplied by Geotechnicals (Affects Uplift on Base Slab if in the landside of levee)



Elevation

$ELEV_1 := 753ft$

$HGL := 761.3ft - ELEV_1$

$HGL = 8.3\ ft$

Water head pressure in Wet Well under operating conditions.

$BLANKET := 753ft - 727ft$

$BLANKET = 26\ ft$

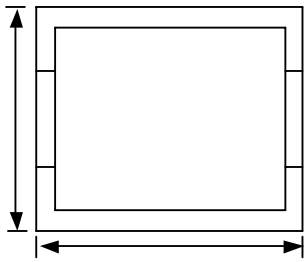
$HEAD := 0ft$

$ELEV_2 := 730ft$

Plan

$L_{1D} := 13.9167ft$ $L_{2D} := 9.25ft$

$L_{1E} := 14.167ft$ $L_{2E} := 9.5ft$



$ELEV_1 = 753\ ft$

$ELEV_2 = 730\ ft$

$H := ELEV_1 - ELEV_2$

$H = 23\ ft$

Wall Thickness 1 $D_{1D} := 1.25ft$

$D_{1E} := 1.5ft$

Clear Cover 1 $C_{c1} := 2.0in$

Floor Thickness $D_f := 1.5ft$

Clear Cover $C_{cf} := 3in$

$d_f := D_f - C_{cf} - 0.75in$ # 6

$d_f = 1.188\ ft$

Wall Thickness 2 $D_{2D} := 1.25ft$

$D_{2E} := 1.5ft$

Clear Cover 2 $C_{c2} := 2.0in$

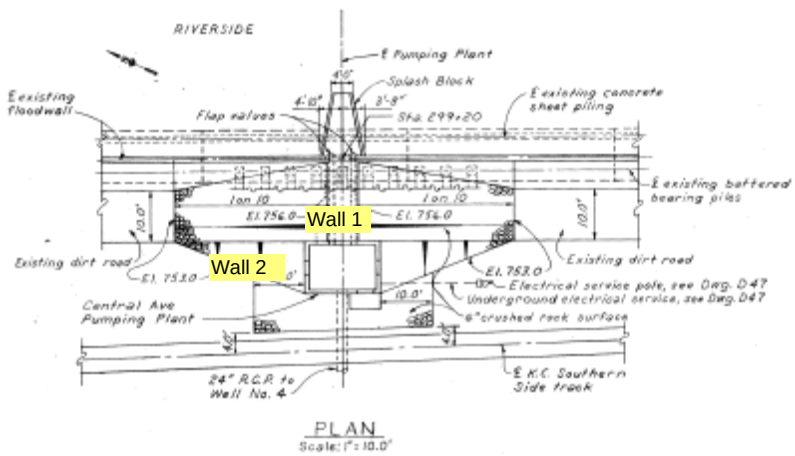
$d_{1D} := D_{1D} - C_{c1}$ $d_{2D} := D_{2D} - C_{c2}$

$d_{1E} := D_{1E} - C_{c1}$ $d_{2E} := D_{2E} - C_{c2}$

Lengths L1 and L2 are center to center distances

$d_{1D} = 13\ in$ $d_{2D} = 13\ in$

$d_{1E} = 16\ in$ $d_{2E} = 16\ in$



Wall 1 is parallel to the floodwall centerline

Aspect Ratios:

$$\frac{L_{1D} \cdot 0.5}{H} = 0.303 \quad \frac{L_{2D} \cdot 0.5}{H} = 0.201$$

$$\frac{L_{1E} \cdot 0.5}{H} = 0.308 \quad \frac{L_{2E} \cdot 0.5}{H} = 0.207$$

File A-10-6083 Excerpt

NOTE: The wall concrete clear cover is not shown on A-10-6085. 2.0 inch clear cover assumed.
The subscript "D" refers to wall section D as shown on A-10-6085. Likewise with section E.

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Soil Properties: $\phi := 26deg$, $\gamma := 125pcf$

$K_o := 1 - \sin(\phi)$

$K_o = 0.562$

Water Unit Weight: $\gamma_w := 62.4pcf$

Steel Properties: $F_y := 40ksi$

Concrete Properties: $f'_c := 3.00ksi$

Concrete Unit Weight: $\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength: $\phi_V := 1.0$

Flexural Strength: $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor: $\gamma_L := 1.0$ — Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor: $\gamma_H := 1.0$ — Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor: $\gamma_X := 1.0$ — Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

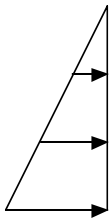
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALLS $K_o = 0.562$ BLANKET = 26 ft

Wall Loading

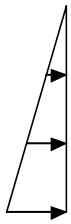
Soil or Soil & Water



$H_2 := H$

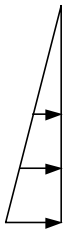
$H_2 = 23\ ft$

OR



$H_2 = 23\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot H - HEAD$

$H''_3 := H + HGL - HEAD$

$H_3 := if(H > BLANKET, H''_3, H'_3)$

$H_3 = 30.342\ ft$

$H'_3 = 30.342\ ft$

$H''_3 = 31.3\ ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_1 = 1615\ \frac{lb}{ft^2}$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3]$

$W_2 = 2702\ \frac{lb}{ft^2}$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W := if(W_1 > W_2, W_1, W_2)$ $W = 2702\ \frac{lb}{ft^2}$

The load on the wall at Sect D, A-10-6085 is:

$W_D := \frac{15.67ft \cdot W}{23ft}$

$W_D = 1840.88\ psf$

The load on the wall at Sect E, A-10-6085 is:

$W_E := \frac{21ft \cdot W}{23ft}$

$W_E = 2467.037\ psf$

WALL 1

SHEAR AT SECTION D

$d_{1D} = 1.083\ ft$

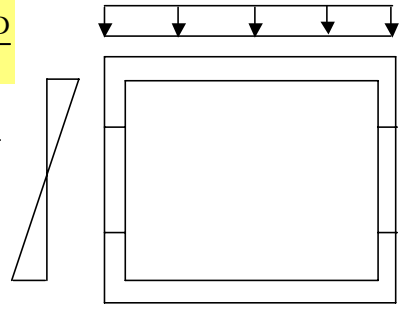
$L_{1D} = 13.917\ ft$

$THRUST_{1D} := \frac{W_D \cdot L_{2D}}{2}$ $\quad THRUST_{1D} = 8514\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_D \cdot \frac{L_{1D}}{2}$

$V'_{u1} = 12809\ \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_{1D}} \cdot \left(\frac{L_{1D}}{2} - \frac{D_{2D}}{2} - d_{1D} \right)$ $\quad V_{u1} = 9665\ \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_{1D} \cdot 12\ \frac{in}{ft}$ $\quad A_g = 180\ \frac{in^2}{ft}$

EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_{1D}}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_{1D} \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 17493\ \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD"\right)$

$Check1 = "OKAY"$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$

$FS_{S1} = 1.81$

WALL 1

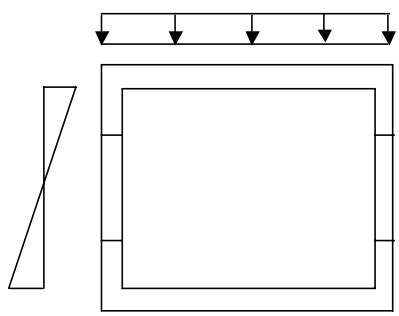
SHEAR AT SECTION E

$THRUST_{1E} := \frac{W \cdot L_{2E}}{2}$ $\quad THRUST_{1E} = 12834\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W \cdot \frac{L_{1E}}{2}$

$V'_{u1} = 19140\ \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_{1E}} \cdot \left(\frac{L_{1E}}{2} - \frac{D_{2E}}{2} - d_{1E} \right)$ $\quad V_{u1} = 13510\ \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_{1D} \cdot 12\ \frac{in}{ft}$ $\quad A_g = 180\ \frac{in^2}{ft}$

EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_{1E}}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_{1E} \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 21782\ \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD"\right)$

$Check1 = "OKAY"$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$

$FS_{S1} = 1.612$

Wall 1 Bending, moment distribution method-horizontal steel at Section D

Analyze such that <u>all</u> of the load is carried by the horizontal steel using moment distribution to determine the moment. Consider thrust in the analysis. The vertical reinforcement is for temperature and shrinkage only.

$W_D = 1840.88\ \text{psf}$

$FEM_{1D} := \frac{W_D \cdot L_{1D}^2}{12} \cdot 1\text{ft}$ $\quad FEM_{1D} = 29.711\ \text{kip}\cdot\text{ft}$ Fixed end moment, long span

$FEM_{2D} := \frac{W_D \cdot L_{2D}^2}{12} \cdot 1\text{ft}$ $\quad FEM_{2D} = 13.126\ \text{kip}\cdot\text{ft}$ Fixed end moment, short span

$DF_1 := \frac{L_{2D}}{L_{1D} + L_{2D}}$ $\quad DF_1 = 0.399$ Distribution factor for wall 1

$DF_2 := \frac{L_{1D}}{L_{1D} + L_{2D}}$ $\quad DF_2 = 0.601$ Distribution factor for wall 2

$EdgeMoment := FEM_{2D} + DF_2 \cdot (FEM_{1D} - FEM_{2D})$ $\quad EdgeMoment = 23.089\ \text{kip}\cdot\text{ft}$

"EdgeMoment" is the negative moment present at the end span connection after distribution.

$MidspanMoment := \frac{W_D \cdot L_{1D}^2}{8} \cdot 1\text{ft} - EdgeMoment$ $\quad MidspanMoment = 21.478\ \text{kip}\cdot\text{ft}$

"MidspanMoment" is the center-of-span (positive) moment.

Information for CASTR: $THRUST_{1D} = 8.51\ \frac{\text{kip}}{\text{ft}}$ $\quad EdgeMoment = 23.089\ \text{ft}\ \frac{\text{kip}\cdot\text{ft}}{\text{ft}}$

From CASTR Program *Cen3W1D.dat*

```
100 Central PS Cen3W1D M.Parks
110 Wall1 SectD n500+3
200 1
210 3.00 40.0 0.375
300 Wall 1 Hor steel at Sect D
400 12.0 15
500 1
510 1 0.80 12.66 6.0 6.0
600 8.514 23.1
```

Percent Design Strength Used in Tensile Zone

$Percent := 67.59\%$

$FoS' := \frac{1}{Percent \cdot .9}$

Note: Strength Reduction Factor (.9) not applied.

$FS_{EPV} := FoS'$

$FoS' = 1.64$ Wall 1, section D is OKAY.

Wall 1 Bending, moment distribution method-horizontal steel at Section E

Analyze such that all of the load is carried by the horizontal steel using moment distribution to determine the moment. Consider thrust in the analysis. The vertical reinforcement is for temperature and shrinkage only.

$W_E = 2467.037\,psf$

$FEM_{1E} := \frac{W_E \cdot L_{1E}^2}{12} \cdot 1ft$ $\quad FEM_{1E} = 41.262\,kip \cdot ft$ Fixed end moment, long span

$FEM_{2E} := \frac{W_E \cdot L_{2E}^2}{12} \cdot 1ft$ $\quad FEM_{2E} = 18.554\,kip \cdot ft$ Fixed end moment, short span

$DF_1 := \frac{L_{2E}}{L_{1E} + L_{2E}}$ $\quad DF_1 = 0.401$ Distribution factor for wall 1

$DF_2 := \frac{L_{1E}}{L_{1E} + L_{2E}}$ $\quad DF_2 = 0.599$ Distribution factor for wall 2

$EdgeMoment := FEM_{2E} + DF_2 \cdot (FEM_{1E} - FEM_{2E})$ $\quad EdgeMoment = 32.147\,kip \cdot ft$

"EdgeMoment" is the negative moment present at the end span connection after distribution.

$MidspanMoment := \frac{W_E \cdot L_{1E}^2}{8} \cdot 1ft - EdgeMoment$ $\quad MidspanMoment = 29.746\,kip \cdot ft$

"MidspanMoment" is the center-of-span (positive) moment.

Information for CASTR: $THRUST_{1E} = 12.83 \frac{kip}{ft}$ $\quad EdgeMoment = 32.147\ ft \frac{kip \cdot ft}{ft}$

From CASTR Program *Cen3W1E.dat*

```
100 Central PS Cen3W1E M.Parks
110 Wall1 SectE n500+3
200 1
210 3.00 40.0 0.375
300 Wall 1 Hor steel at Sect E
400 12.0 18
500 1
510 1 0.80 15.66 6.0 6.0
600 12.83 32.15
```

Percent Design Strength Used in Tensile Zone

$Percent := 70.45\%$

$FoS' := \frac{1}{Percent \cdot .9}$ Note: Strength Reduction Factor (.9) not applied.

$FS_{EPV} := FoS'$

$FoS' = 1.58$ Wall 1, section E is OKAY.

WALL 2

SHEAR AT SECTION D

$d_{2D} = 1.083\ ft$

$L_{2D} = 9.25\ ft$

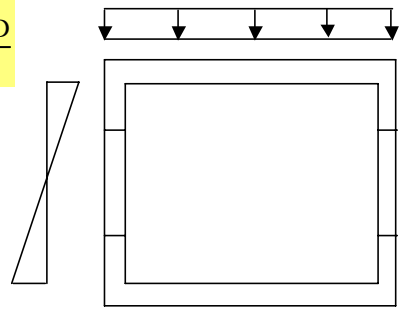
$W_D = 1840.88\ psf$

$THRUST_{2D} := \frac{W_D \cdot L_{1D}}{2}$ $\quad THRUST_{1D} = 8514\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u2} := W_D \cdot \frac{L_{2D}}{2}$

$V'_{u2} = 8514\ \frac{lb}{ft}$



$V_{u2} := 2 \frac{V'_{u2}}{L_{2D}} \cdot \left(\frac{L_{2D}}{2} - \frac{D_{1D}}{2} - d_{2D} \right)$ $\quad V_{u2} = 5369\ \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_{2D} \cdot 12\ \frac{in}{ft}$ $\quad A_g = 180\ \frac{in^2}{ft}$

EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_{2D}}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_{2D} \right) \right]$ ACI EQ (11-4)

$\phi V_{n2} = 17697\ \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$

Check1 = "OKAY"

Factor of Safety

$FS_{S1} := \frac{\phi V_{n2}}{V_{u2}}$

$FS_{S1} = 3.296$

WALL 2

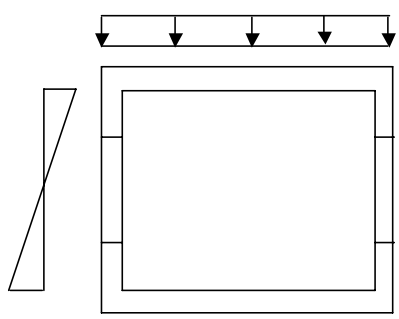
SHEAR AT SECTION E

$THRUST_{2E} := \frac{W \cdot L_{1E}}{2}$ $\quad THRUST_{2E} = 19140\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u2} := W \cdot \frac{L_{2E}}{2}$

$V'_{u1} = 19140\ \frac{lb}{ft}$



$V_{u2} := 2 \frac{V'_{u2}}{L_{2E}} \cdot \left(\frac{L_{2E}}{2} - \frac{D_{1E}}{2} - d_{2E} \right)$ $\quad V_{u1} = 13510\ \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_{2E} \cdot 12\ \frac{in}{ft}$ $\quad A_g = 216\ \frac{in^2}{ft}$

EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_{2E}}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_{2E} \right) \right]$ ACI EQ (11-4)

$\phi V_{n2} = 21964\ \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$

Check1 = "OKAY"

Factor of Safety $FS_{S1} := \frac{\phi V_{n2}}{V_{u2}}$ $FS_{S1} = 3.048$

Wall 2 Bending, moment distribution method-horizontal steel at Section D

Analyze such that all of the load is carried by the horizontal steel using moment distribution to determine the moment. Consider thrust in the analysis. The vertical reinforcement is for temperature and shrinkage only.

$W_D = 1840.88\ psf$ $L_{2D} = 9.25\ ft$ $W_D = 1840.88\ psf$

$FEM_{1D} := \frac{W_D \cdot L_{1D}^2}{12} \cdot 1ft$ $FEM_{1D} = 29.711\ kip \cdot ft$ Fixed end moment, long span

$FEM_{2D} := \frac{W_D \cdot L_{2D}^2}{12} \cdot 1ft$ $FEM_{2D} = 13.126\ kip \cdot ft$ Fixed end moment, short span

$DF_1 := \frac{L_{2D}}{L_{1D} + L_{2D}}$ $DF_1 = 0.399$ Distribution factor for wall 1

$DF_2 := \frac{L_{1D}}{L_{1D} + L_{2D}}$ $DF_2 = 0.601$ Distribution factor for wall 2

$EdgeMoment := FEM_{2D} + DF_2 \cdot (FEM_{1D} - FEM_{2D})$ $EdgeMoment = 23.089\ kip \cdot ft$

"EdgeMoment" is the negative moment present at the end span connection after distribution.

$MidspanMoment := \frac{W_D \cdot L_{2D}^2}{8} \cdot 1ft - EdgeMoment$ $MidspanMoment = -3.4\ kip \cdot ft$

"MidspanMoment" is the center-of-span (positive) moment.

Information for CASTR: $THRUST_{2D} = 12.81 \frac{kip}{ft}$ $EdgeMoment = 23.089\ ft \frac{kip \cdot ft}{ft}$

From CASTR Program *Cen3W2D.dat*

```
100 Central PS Cen3W2D M.Parks
110 Wall2 SectD n500+3
200 1
210 3.00 40.0 0.375
300 Wall 2 Hor steel at Sect D
400 12.0 15
500 1
510 1 0.80 12.66 6.0 6.0
600 12.81 23.1
```

Percent Design Strength Used in Tensile Zone

$Percent := 62.44\%$

$FoS' := \frac{1}{Percent \cdot .9}$ Note: Strength Reduction Factor (.9) not applied.

$FS_{EPV} := FoS'$

$FoS' = 1.78$ Wall 2, section D is OKAY.

Wall 2 Bending, moment distribution method-horizontal steel at Section E

Analyze such that <u>all</u> of the load is carried by the horizontal steel using moment distribution to determine the moment. Consider thrust in the analysis. The vertical reinforcement is for temperature and shrinkage only.

$W_E = 2467.037\,psf$

$FEM_{1E} := \frac{W_E \cdot L_{1E}^2}{12} \cdot 1ft$ $\quad FEM_{1E} = 41.262\,kip \cdot ft$ Fixed end moment, long span

$FEM_{2E} := \frac{W_E \cdot L_{2E}^2}{12} \cdot 1ft$ $\quad FEM_{2E} = 18.554\,kip \cdot ft$ Fixed end moment, short span

$DF_1 := \frac{L_{2E}}{L_{1E} + L_{2E}}$ $\quad DF_1 = 0.401$ Distribution factor for wall 1

$DF_2 := \frac{L_{1E}}{L_{1E} + L_{2E}}$ $\quad DF_2 = 0.599$ Distribution factor for wall 2

$EdgeMoment := FEM_{2E} + DF_2 \cdot (FEM_{1E} - FEM_{2E})$ $\quad EdgeMoment = 32.147\,kip \cdot ft$

"EdgeMoment" is the negative moment present at the end span connection after distribution.

$MidspanMoment := \frac{W_E \cdot L_{2E}^2}{8} \cdot 1ft - EdgeMoment$ $\quad MidspanMoment = -4.316\,kip \cdot ft$

"MidspanMoment" is the center-of-span (positive) moment.

Information for CASTR: $THRUST_{2E} = 19.14 \frac{kip}{ft}$ $\quad EdgeMoment = 32.147\,ft \frac{kip \cdot ft}{ft}$

From CASTR Program *Cen3W2E.dat*

```
100 Central PS Cen3W2E M.Parks
110 Wall2 SectE n500+3
200 1
210 3.00 40.0 0.375
300 Wall 2 Hor steel at Sect E
400 12.0 18
500 1
510 1 0.80 15.66 6.0 6.0
600 19.14 32.15
```

Percent Design Strength Used in Tensile Zone

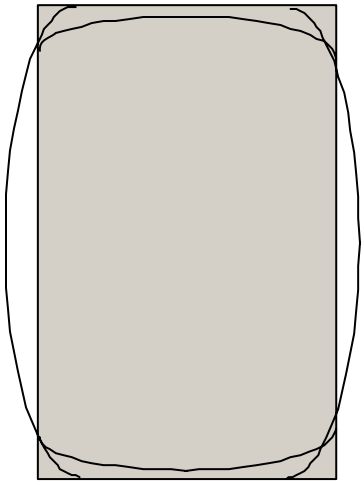
$Percent := 62.97\%$

$FoS' := \frac{1}{Percent \cdot .9}$

Note: Strength Reduction Factor (.9) not applied.

$FS_{EPV} := FoS'$

$FoS' = 1.76$ Wall 1, section E is OKAY.



Approximate shape of the moment curves. Note that the short span does not have a positive moment at mid-span at section D or E.

Summary. The controlling Factor of Safety is the long span (wall 1) moment at Sect E

	Wall 1		Wall 2	
	Sec D	Sec E	Sec D	Sec E
Shear	1.81	1.61	3.3	3.05
Moment	1.64	1.58	1.78	1.76

Uplift on Structure

INCLUDE THE SOIL OVER THE STEPS

$Wall_C := (14ft + 8in)(10ft + 0in)\cdot(7ft + 4in)$ $\qquad Wall_C\cdot 150pcf = 161.333\ kips$

$Wall_D := (15ft + 2in)(10ft + 6in)\cdot(7ft + 4in)$ $\qquad Wall_D\cdot 150pcf = 175.175\ kips$

$Wall_E := (15ft + 8in)(11ft + 0in)\cdot(8ft + 1.5in)$ $\qquad Wall_E\cdot 150pcf = 210.031\ kips$

$Interior1 := -[(22ft + 9.5in)\cdot(8ft + 0in)\cdot(12ft + 8in)]$ $\qquad Interior1\cdot 150pcf = -346.433\ kip$

$Topslab := \left[(14ft + 8in)\cdot(10ft) - (2.5ft\cdot 5ft\cdot 2) - 3.14\cdot(0.5ft)^2\cdot 2\right]\cdot 1ft$ $\qquad Topslab\cdot 150pcf = 18.014\ kip$

$abovegradewalls := [(14ft + 8in)\cdot(10ft) - (13ft + 4in)\cdot(8ft + 8in)]\cdot(9ft + 4in) - (3ft + 4in)\cdot 7.5ft\cdot 8in$

$abovegradewalls\cdot 150pcf = 41.056\ kip$

$BaseSlab := 11ft\cdot(15ft + 8in)$ $\qquad BaseSlab = 172.333\ ft^2$

$Baset := 1.5ft$ $\qquad Baset = 1.5\ ft$ $\qquad BaseSlabvol := BaseSlab\cdot Baset$

$Structure := (Wall_C + Wall_D + Wall_E + Interior1 + abovegradewalls + BaseSlabvol + Topslab)\cdot 150pcf$

$Structure = 297.951\ kip$

$HEAD := 0ft$ Depth of water in the pump plant

$Water := (12ft + 8in)\cdot 8ft\cdot HEAD\cdot\gamma_w$ Weight of water in the pump plant

$Water = 0$

$HGL = 8.3\ ft$

$H = 23\ ft$

Soil:

$Vol_CD := [(15ft + 2in)\cdot(10ft + 6in) - (14ft + 8in)\cdot 10ft]\cdot(7ft + 4in)$ $\qquad Vol_CD = 92.278\ ft^3$

$Vol_DE := [(15ft + 8in)\cdot 11ft - (15ft + 2in)\cdot(10.5ft)]\cdot(14ft + 4in)$ $\qquad Vol_DE = 187.528\ ft^3$

$V_{soil} := Vol_CD + Vol_DE$ $\qquad V_{soil} = 279.806\ ft^3$

$\gamma = 125\ pcf$ $\qquad \gamma_w = 62.4\ pcf$

Note that the bottom of sump pit elevation is elev 726.3 ft (below the bottom of blanket elevation of 727).

Dissipated Head, if structural foundation in blanket. $HGL = 8.3\ ft$ (portion of station above blanket)

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET}\cdot 25.25ft\right)\cdot\left[\left(BaseSlab - 60.5\cdot ft^2\right)\cdot\gamma_w\right] \qquad U_1 = 232.455\ kips \quad BLANKET = 26\ ft$$

The sump pit portion of the pump plant is exposed to full head uplift pressure.

Full Head, if structural foundation extends through blanket. (portion of station below the blanket)

$$U_2 := \left[(26.78ft + HGL)\cdot\left(60.5ft^2\right)\cdot\gamma_w\right] \qquad U_2 = 132.434\ kips$$

$$Uplift := U_1 + U_2 \qquad Uplift = 364.889\ kip$$

$$Stability := \frac{Structure + Water + V_{soil}\cdot(\gamma - \gamma_w)}{Uplift - V_{soil}\cdot\gamma_w} \qquad Stability = 0.908$$

Does not account for the fixed equimpment weight

$$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay") \qquad StabilityCheck = "Not\ Okay"$$

BASE SLAB ANALYSIS $W_{water} := 0lbf$ Conservative

SHEAR

Slab Strength

$$THRUST_S := \frac{W\cdot L_{2E}}{2} \qquad THRUST_S = 12834\ \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis (if necessary)

$$\frac{Structure + W_{water}}{BaseSlab} = 1728.924\ psf$$

$$W' := if\left(Stability < 1.0, \frac{Uplift}{BaseSlab}, \frac{Structure}{BaseSlab}\right) \qquad W' = 2117.342\ psf$$

$$\frac{Uplift}{BaseSlab} = 2117.342\ psf$$

$Slab_t := 1.5ft$ $L_{strip} := 14.17ft$ $Wall_t := 1.5ft$ $Slab_d := 14.25in$ effective depth of slab

$$V'_{ub} := W'\cdot\frac{L_{strip}}{2} \qquad V'_{ub} = 15001\ \frac{lb}{ft} \qquad W' = 2117.342\ \frac{lb}{ft^2}$$

$$L_{1E} = 14.167\ ft$$

$$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}}\cdot\left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right) \qquad V_{ub} = 10899\ \frac{lb}{ft}$$

$d_f = 14.25\ in$

$D_f = 1.5\ ft$

$A_g := D_f \cdot 12 \frac{in}{ft}$ $\quad A_g = 216 \frac{in^2}{ft}$

$Water = 0$

ANALYZE STRENGTH WITHOUT THE AID OF THRUST

$$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S \cdot 0}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f \right) \right]$$ ACI EQ (11-4) $\quad \phi V_{nb} = 18732 \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, "OKAY", "NO GOOD"\right)$ $\quad CheckBase = "OKAY"$

Factor of Safety $\quad FS_b := \frac{\phi V_{nb}}{V_{ub}}$ $\quad FS_b = 1.719$ $\quad 1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 1.03$

Slab Bending

Consider the sump pit base slab transition as fixed

$Lsump := 15ft + 8in - 5.5ft + 1.5ft$ $\quad Lsump = 11.667\ ft$ $\quad \frac{L_{2E}}{Lsump} = 0.814$ $\quad L_{1E} = 14.167\ ft$ $\quad L_{2E} = 9.5\ ft$

REBAR PARALLEL TO LONG WALL, Wall 1

ratio = 0.875 $\quad 0.0209 \cdot W' \cdot Lsump^2 = 6.023\ kip \cdot \frac{ft}{ft}$

$W' = 2117.342\ psf$ $\quad 0.0530 \cdot W' \cdot Lsump^2 = 15.274\ kip \cdot \frac{ft}{ft}$

Simple Conservative Analysis

$$\frac{W' \cdot (31ft)^2 \cdot 1ft}{8} = 254.346\ kip \cdot ft$$

Plate Analysis

$MidPlateMoment := 6.023 kip \cdot \frac{ft}{ft}$ My

$EdgePlateMoment := 15.27 kip \cdot \frac{ft}{ft}$ My

Steel Depth

$d_1 := 18in - 3in - 0.75in$

$d_2 := 18in - 3in - 0.75in$

$d_2 = 14.25\ in$

#6's @ 12 in. inside face (mid-plate moment)

A-10-6085, TOP REINF

$A_{mpl} := 0.44 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mpl} = 0.575\ in$

$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_1 - \frac{a_{mpl}}{2} \right)$ $\quad \phi M_{mpl} = 20.5 \frac{kip \cdot ft}{ft}$

#6's @ 12" Outside Face (edge-plate moment)

A-10-6085, BOTTOM REINF

$A_{epl} := 0.44 \frac{in^2}{ft}$ $\quad a_{epl} := \frac{A_{epl} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{epl} = 0.575\ in$

$\phi M_{epl} := \phi_B \cdot A_{epl} \cdot F_y \cdot \left(d_2 - \frac{a_{epl}}{2} \right)$ $\quad \phi M_{epl} = 20.5 \frac{kip \cdot ft}{ft}$

$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot MidPlateMoment, "OKAY", "NO GOOD"\right)$ $CheckMP = "OKAY"$

Factor of Safety $FS_{MPL} := \dfrac{\phi M_{mpl}}{MidPlateMoment}$ $FS_{MPL} = 3.4$

$CheckEPL := if\left(\phi M_{epl} > 1.5 \cdot EdgePlateMoment, "OKAY", "NO GOOD"\right)$ $CheckEPL = "NO GOOD"$

Factor of Safety $FS_{EPL} := \dfrac{\phi M_{epl}}{EdgePlateMoment}$ $FS_{EPL} = 1.341$

The thrust value= W $W = 2701.993\ psf$

Information for CASTR: $THRUST_{2D} := W \cdot 1ft^2$ $THRUST_{2D} = 2.702\ kip$ $EdgeMoment := 15.3 kip \cdot ft$

From CASTR Program *Cen3Bas.dat*

```
100 Central PS Cen3Bas M.Parks
110 Base slab n500+3
200 1
210 3.00 40.0 0.375
300 Base slab parallel to wall 1
400 12.0 18
500 1
510 1 0.44 14.25 6.0 6.0
600 2.7 15.3
```

Percent Design Strength Used in Tensile Zone

$Percent := 72.52\%$

$FoS' := \dfrac{1}{Percent \cdot .9}$

Note: Strength Reduction Factor (.9) not applied.

$FS_{EPV} := FoS'$

$FoS' = 1.53$

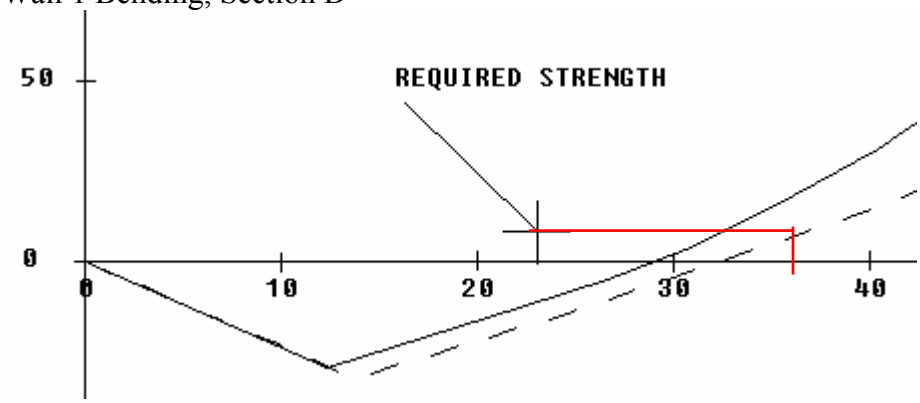
Base slab OKAY. The above base slab bending calculation are conservative. The base slab will not be exposed to the bending moment and shear used in analysis because water has to be in the pump station to maintain stability. The water would reduce the net moment and shear forces acting on the slab.

END OF ANALYSIS

Central Pump Station Interaction Diagrams.

The CASTR program output prints the percentage of permissible design strength used by the applied loading. The previous calculations used this value to determine the Factor of Safety. Under certain circumstances, using the CASTR output value of percent design strength used by the applied loading may create an conservative factor of safety (FoS') value. As an alternative, the section strength was obtained from the interaction diagram output created by CASTR. The interaction diagram strength value was compared to the demand moment to determine the section bending FoS'.

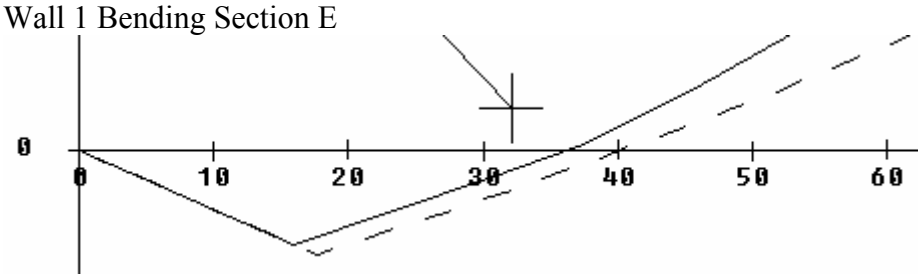
Wall 1 Bending, Section D



Cen3W1D.dat Interaction Diagram

Using interaction diagram to determine the factor of safety:
Strength = 36 k*ft (from interaction diagram)
Demand= 23.09 k*ft
FoS' = 36/23.09 = 1.56

Wall 1 Bending Section E

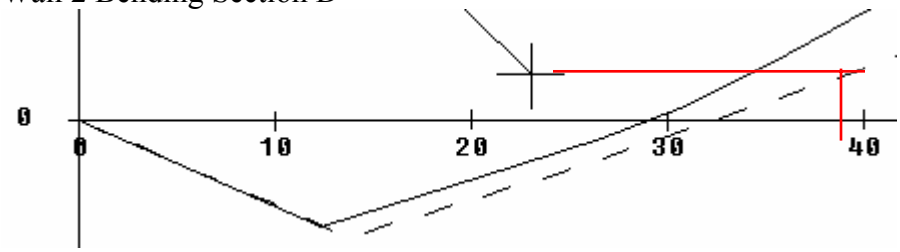


Cen3W1E.dat Interaction Diagram

Using interaction diagram to determine the factor of safety

Strength = 48 k*ft (from interaction diagram)
Demand = 32.15 k*ft
FoS' = 48/32.15 = 1.5

Wall 2 Bending Section D

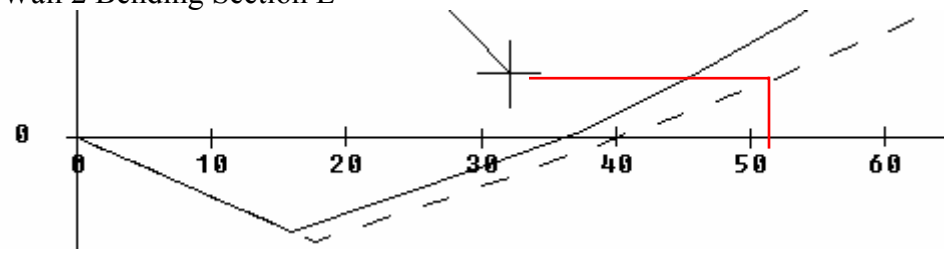


Cen3W2D.dat Interaction Diagram.

Using interaction diagram to determine the factor of safety

Strength = 38 k*ft
Demand = 23.1 k*ft
FoS' = 38/23.1 = 1.65

Wall 2 Bending Section E

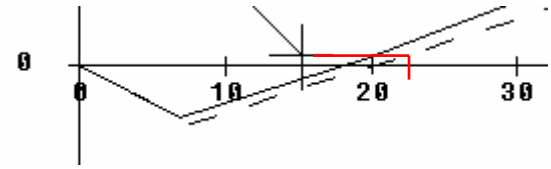


Cen3W2E.dat Interaction Diagram

Using interaction diagram to determine the factor of safety

Strength = 51 k*ft
Demand = 32.15 k*ft
FoS' = 1.59

Base Slab



Cen3Bas.dat Interaction Diagram
Strength = 23 k*ft
Demand = 15.3
FoS'= 1.5

Conclusion. The FoS' obtained using the interaction diagram values of section strength were more conservative. Wall 1, wall 2 and the base slab were okay for bending moment strength.

Variables

$kip := 1000lb \quad plf := \frac{lb}{ft} \quad psf \equiv \frac{lb}{ft^2} \quad pcf := \frac{lb}{ft^3} \quad psi \equiv \frac{lb}{in^2} \quad ksi \equiv \frac{1000lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad kips := 1000lb \quad klf := \frac{kips}{ft}$

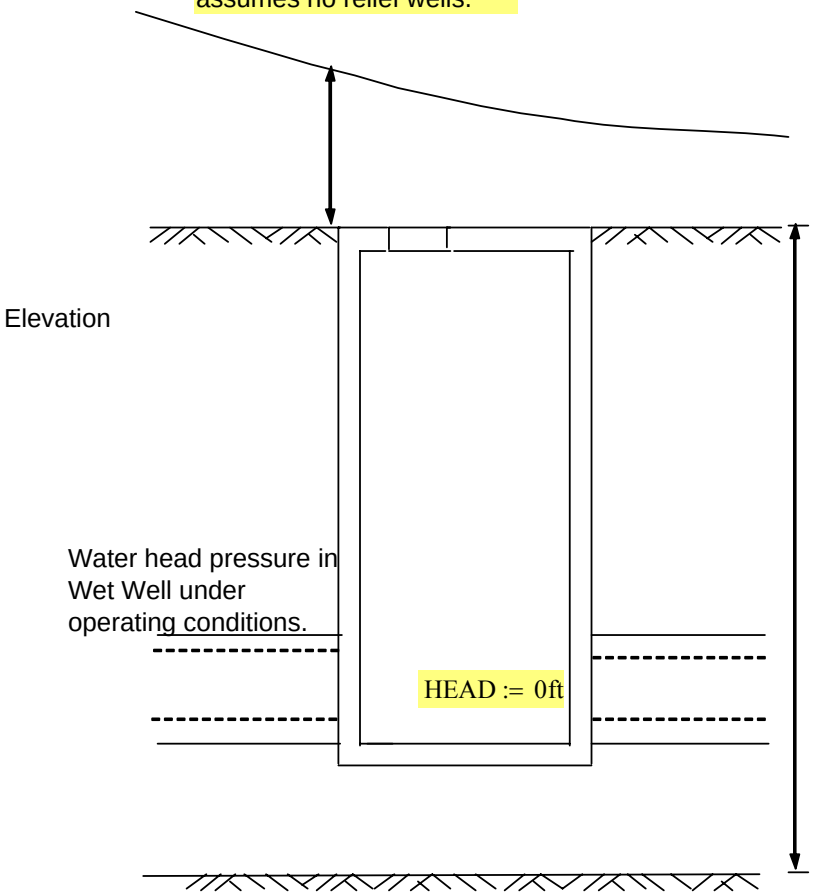
Notes:
1. Walls analyzed for two-way action, plate fixed along three edges, hinged along one edge (top slab connection), uniformly varying load. Figure 13 of the Bureau monographs.
2. Base slab analyzed for two-way action, plate fixed along four edges, uniform load. Figure 34 of Bureau monographs.
3. Two walls were analyzed.

REFERENCES
1. Moments and Reactions for Rectangular Plates, Monograph No. 27, Bueau of Reclamation, July 1963

Properties

NOTE: HGL Supplied assumes no relief wells.

Original Hydraulic Grade Line as supplied by Geotechnicals (Affects Uplift on Base Slab if in the landside of levee)



$ELEV_1 := 749.5ft$

$HGL := 763ft - ELEV_1$

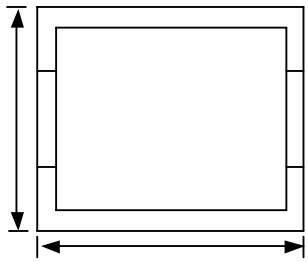
$HGL = 13.5\ ft$

$BLANKET := 750ft - 722ft$

$BLANKET = 28\ ft$

$ELEV_2 := 731ft$

Plan



$L_1 := 11ft + 3in$

$L_1 = 11.25\ ft$

$H := ELEV_1 - ELEV_2$

$H = 18.5\ ft$

Wall Thickness 1 $D_1 := 1.25ft$

Clear Cover 1 $C_{c1} := 2.0in$

$L_2 := 10ft + 9in$

$L_2 = 10.75\ ft$

Floor Thickness $D_f := 2.5ft$

Clear Cover $C_{cf} := 3in$

Wall Thickness 2 $D_2 := 1.25ft$

Clear Cover 2 $C_{c2} := 2.0in$

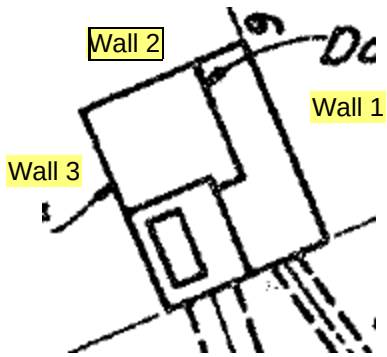
$l_1 := L_1 \quad d_1 := D_1 - C_{c1} \quad d_f := D_f - C_{cf}$

$l_2 := L_2 \quad d_2 := D_2 - C_{c2} \quad d_2 = 1.083\ ft$

Lengths L1 and L2 are center to center distances

Wall Layout

Wall 2 is parallel to the levee centerline



Levee Centerline

$\frac{L_1 \cdot 0.5}{H} = 0.304$

NORTH

$\frac{L_2 \cdot 0.5}{H} = 0.291$

File A-10-1989 Excerpt

Wall 1 is perpendicular to the levee centerline

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Soil Properties $\phi := 26deg$

$\gamma := 125pcf$

$K_o := 1 - \sin(\phi)$

$K_o = 0.562$

Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40ksi$

Concrete Properties $f'_c := 3.00ksi$

Concrete Unit Weight $\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

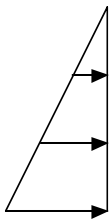
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALLS $K_o = 0.562$ BLANKET = 28 ft

Wall Loading

Soil or Soil & Water



$H_2 := H$

$H_2 = 18.5\ ft$

OR

$H_2 = 18.5\ ft$

&

$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot H - HEAD$

$H''_3 := H + HGL - HEAD$

$H_3 := if(H > BLANKET, H''_3, H'_3)$

$H_3 = 27.42\ ft$

$H'_3 = 27.42\ ft$

$H''_3 = 32\ ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_1 = 1299\ \frac{lb}{ft^2}$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3]$

$W_2 = 2361\ \frac{lb}{ft^2}$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W := if(W_1 > W_2, W_1, W_2)$

$W = 2361\ \frac{lb}{ft^2}$

SHEAR

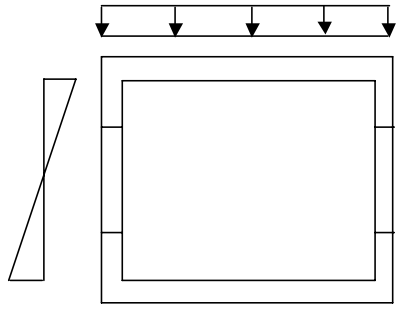
WALL 1

$THRUST_1 := \frac{W \cdot L_2}{2}$ $\quad THRUST_1 = 12693 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W \cdot \frac{L_1}{2}$

$V'_{u1} = 13283 \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$ $\quad V_{u1} = 9249 \frac{lb}{ft}$

$D_2 = 1.25\ ft$

$d_1 = 1.083\ ft$

$L_1 = 11.25\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 180 \frac{in^2}{ft}$

$\frac{L_1}{d_1} = 10.385$ EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 17691 \frac{lb}{ft}$

$Check1 := if(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD")$ $\quad Check1 = "OKAY"$

Factor of Safety $\quad FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$ $\quad FS_{S1} = 1.913$

If wall does not meet shear requirements when analyzed as 1ft wide strip. Consider Plate reactions.

NOT REQUIRED

From Figure 13 of the Bureau $\quad Ry := 0.2647 \cdot W \cdot (ELEV_1 - ELEV_2)$ $\quad Ry = 11.564 \frac{kip}{ft}$ $\quad V'_{u1} := 19.6 \frac{kips}{ft}$

$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - D_2 - d_1 \right)$ $\quad V_{u1} = 11470 \frac{lb}{ft}$

$Check1 := if(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD")$ $\quad Check1 = "OKAY"$

Factor of Safety $\quad FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$ $\quad FS_{S1} = 1.542$

$W = 2361.408 \frac{lb}{ft^2}$ $\quad$ ratio = 1/4

$0.0031 \cdot 2361 psf \cdot (ELEV_1 - ELEV_2)^2 = 2.505\ kip \cdot \frac{ft}{ft}$

Figure 13 monograph

$0.0107 \cdot 2361 psf \cdot (ELEV_1 - ELEV_2)^2 = 8.646\ kip \cdot \frac{ft}{ft}$

Wall 1 Bending

Vertical Steel

Simple Conservative Analysis

$$M_{u1} := \frac{W \cdot L_1^{\,2}}{12} \qquad M_{u1} = 24905 \frac{ft \cdot lb}{ft}$$

Plate Analysis

$$NegMidPlateMoment := 2.51 kip \cdot \frac{ft}{ft}$$ My

$$PosEdgePlateMoment := 8.65 kip \cdot \frac{ft}{ft}$$ My

#4's @ 12 in. inside face (mid-plate moment)

$$A_{mpv} := 0.20 \frac{in^2}{ft} \qquad b := 12 \frac{in}{ft} \qquad a_{mpv} := \frac{A_{mpv} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{mpv} = 0.261\ in$$

A-10-1989

$$\phi M_{mpv} := \phi_B \cdot A_{mpv} \cdot F_y \cdot \left(d_1 - \frac{a_{mpv}}{2}\right) \qquad \phi M_{mpv} = 8.6 \frac{kip \cdot ft}{ft}$$

#5's @ 12 in. outside face at wall base (edge-plate moment)

$$A_{epv} := 0.31 \frac{in^2}{ft} \qquad a_{epv} := \frac{A_{epv} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{epv} = 0.405\ in$$

A-10-1989

$$\phi M_{epv} := \phi_B \cdot A_{epv} \cdot F_y \cdot \left(d_1 - \frac{a_{epv}}{2}\right) \qquad \phi M_{epv} = 13.2 \frac{kip \cdot ft}{ft}$$

$$CheckMPV := if\left(\phi M_{mpv} > 1.5 \cdot NegMidPlateMoment, "OKAY", "NO\ GOOD"\right)$$ CheckMPV = "OKAY"

Factor of Safety

$$FS_{MPV} := \frac{\phi M_{mpv}}{NegMidPlateMoment}$$ $FS_{MPV} = 3.418$

$$CheckEPV := if\left(\phi M_{epv} > 1.5 \cdot PosEdgePlateMoment, "OKAY", "NO\ GOOD"\right)$$ CheckEPV = "OKAY"

Factor of Safety

$$FS_{EPV} := \frac{\phi M_{epv}}{PosEdgePlateMoment}$$ $FS_{EPV} = 1.529$

ratio = 1/4

$$0.0057 \cdot 2361 psf \cdot \left(ELEV_1 - ELEV_2\right)^2 = 4.606 kip \cdot \frac{ft}{ft}$$

Figure 13 monograph

$$0.0114 \cdot 2361 psf \cdot \left(ELEV_1 - ELEV_2\right)^2 = 9.212 kip \cdot \frac{ft}{ft}$$

Wall 1 Bending

Horizontal Steel

Plate Analysis

Simple Conservative Analysis

$$M_{u1} = 24905 \frac{ft \cdot lb}{ft}$$

$$NegMidPlateMoment2 := 4.61 kip \cdot \frac{ft}{ft}$$ Mx

$$PosEdgePlateMoment2 := 9.212 kip \cdot \frac{ft}{ft}$$ Mx

#5's @ 10 in. inside face
(mid-plate moment)

$A_{mph} := 0.37 \frac{in^2}{ft}$ $\quad a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mph} = 0.484\ in$

A-10-1989

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_1 - \frac{a_{mph}}{2}\right)$ $\quad \phi M_{mph} = 15.7 \frac{kip \cdot ft}{ft}$

#8's @ 12 in. outside face
#4's @ 12 in. (alt)
(edge-plate moment)

$A_{eph} := 0.99 \frac{in^2}{ft}$ $\quad a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{eph} = 1.294\ in$

A-10-1989

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_1 - \frac{a_{eph}}{2}\right)$ $\quad \phi M_{eph} = 40.8 \frac{kip \cdot ft}{ft}$

$CheckMPH := if\left(\phi M_{mph} > 1.5 \cdot NegMidPlateMoment2, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\quad CheckMPH = \text{"OKAY"}$

Factor of Safety $\quad FS_{MPH} := \frac{\phi M_{mph}}{NegMidPlateMoment2}$ $\quad FS_{MPH} = 3.413$

$CheckEPH := if\left(\phi M_{eph} > 1.5 \cdot PosEdgePlateMoment2, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\quad CheckEPH = \text{"OKAY"}$

Factor of Safety $\quad FS_{EPH} := \frac{\phi M_{eph}}{PosEdgePlateMoment2}$ $\quad FS_{EPH} = 4.425$

Controlling Factor of Safety

$FS_1 := \min\left(FS_{S1}, FS_{MPV}, FS_{EPV}, FS_{MPH}, FS_{EPH}\right)$ $\quad FS_1 = 1.529$

Controlling Mechanism

$Comment1 := if\left(FS_1 < FS_{S1}, \text{"Wall 1 Flexural Steel"}, \text{"Wall 1 Shear"}\right)$ $\quad Comment1 = \text{"Wall 1 Flexural Steel"}$

WALL 2

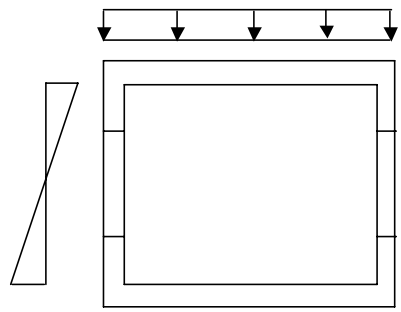
SHEAR

$THRUST_2 := \frac{W \cdot L_1}{2}$ $\quad THRUST_2 = 13283 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u2} := W \cdot \frac{L_2}{2}$

$V'_{u2} = 12693 \frac{lb}{ft}$



$V_{u2} := 2 \frac{V'_{u2}}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2 \right)$ $\quad V_{u2} = 8658 \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1 $\quad \frac{L_2}{d_2} = 9.923$

$A_g := D_2 \cdot 12 \frac{in}{ft}$ $\quad A_g = 180 \frac{in^2}{ft}$

EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2 \cdot 0}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_2 \right) \right]$ ACI EQ (11-4)

$d_2 = 1.083 \ ft$

$\phi V_{n2} = 17089 \frac{lb}{ft}$

$Check2 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$ $\quad Check2 = "OKAY"$

Factor of Safety $\quad FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$ $\quad FS_{S2} = 1.974$

$W = 2361.408 \ psf$

ratio = 1/4

$0.0031 \cdot 2361 psf \cdot (ELEV_1 - ELEV_2)^2 = 2.505 \ kip \cdot \frac{ft}{ft}$

Figure 13 monograph

$0.0107 \cdot 2361 psf \cdot (ELEV_1 - ELEV_2)^2 = 8.646 \ kip \cdot \frac{ft}{ft}$

Wall 2 Bending

Vertical Steel

Simple Conservative Analysis

$M_{u1} := \frac{W \cdot L_2^2}{12}$ $\quad M_{u1} = 22.74 \frac{kip \cdot ft}{ft}$

Plate Analysis

$NegMidPlateMoment := 2.51 kip \cdot \frac{ft}{ft}$ My

$PosEdgePlateMoment := 8.65 kip \cdot \frac{ft}{ft}$ My

#4's @ 12 in. Inside Face (mid-plate moment)

A-10-1989

$A_{mpv} := 0.20 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a_{mpv} := \frac{A_{mpv} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mpv} = 0.261 \ in$

$\phi M_{mpv} := \phi_B \cdot A_{mpv} \cdot F_y \cdot \left(d_1 - \frac{a_{mpv}}{2} \right)$ $\quad \phi M_{mpv} = 8.6 \frac{kip \cdot ft}{ft}$

#5's @ 12 in. outside face
(edge-plate moment)

A-10-1989

$A_{epv} := 0.31 \frac{in^2}{ft}$ $\quad a_{epv} := \frac{A_{epv} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{epv} = 0.405\ in$

$\phi M_{epv} := \phi_B \cdot A_{epv} \cdot F_y \cdot \left(d_1 - \frac{a_{epv}}{2}\right)$ $\quad \phi M_{epv} = 13.2 \frac{kip \cdot ft}{ft}$

$CheckMPV := if\left(\phi M_{mpv} > 1.5 \cdot NegMidPlateMoment, "OKAY", "NO\ GOOD"\right)$ $\quad CheckMPV = "OKAY"$

Factor of Safety $\quad FS_{MPV} := \frac{\phi M_{mpv}}{NegMidPlateMoment}$ $\quad FS_{MPV} = 3.418$

$CheckEPV := if\left(\phi M_{epv} > 1.5 \cdot PosEdgePlateMoment, "OKAY", "NO\ GOOD"\right)$ $\quad CheckEPV = "OKAY"$

Factor of Safety $\quad FS_{EPV} := \frac{\phi M_{epv}}{PosEdgePlateMoment}$ $\quad FS_{EPV} = 1.529$

ratio = 1/4 $\quad 0.0057 \cdot 2361 psf \cdot \left(ELEV_1 - ELEV_2\right)^2 = 4.606\ kip \cdot \frac{ft}{ft}$

Figure 13 monograph $\quad 0.0114 \cdot 2361 psf \cdot \left(ELEV_1 - ELEV_2\right)^2 = 9.212\ kip \cdot \frac{ft}{ft}$

Wall 2 Bending

Horizontal Steel

Simple Conservative Analysis

$M_{u1} = 22741 \frac{ft \cdot lb}{ft}$

Plate Analysis

$L_2 = 10.75\ ft$

$NegMidPlateMoment2 := 4.61 kip \cdot \frac{ft}{ft}$ Mx

$PosEdgePlateMoment2 := 9.2 kip \cdot \frac{ft}{ft}$ Mx

#5's @ 12 in. inside face
(mid-plate moment)

A-10-1989

$A_{mph} := 0.31 \frac{in^2}{ft}$ $\quad a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mph} = 0.405\ in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_1 - \frac{a_{mph}}{2}\right)$ $\quad \phi M_{mph} = 13.2 \frac{kip \cdot ft}{ft}$

#6's @ 12 in. outside face
#4's @ 12 in. (alt)
(edge-plate moment)

A-10-1989

$A_{eph} := 0.64 \frac{in^2}{ft}$ $\quad a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{eph} = 0.837\ in$

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_1 - \frac{a_{eph}}{2}\right)$ $\quad \phi M_{eph} = 26.8 \frac{kip \cdot ft}{ft}$

$CheckMPH := if\left(\phi M_{mph} > 1.5 \cdot NegMidPlateMoment2, "OKAY", "NO\ GOOD"\right)$ $\quad CheckMPH = "OKAY"$

Factor of Safety $\quad FS_{MPH} := \frac{\phi M_{mph}}{NegMidPlateMoment2}$ $\quad FS_{MPH} = 2.869$

$$CheckEPH := if\left(\phi M_{eph} > 1.5 \cdot PosEdgePlateMoment2, \text{"OKAY"}, \text{"NO GOOD"}\right)$$ $CheckEPH = \text{"OKAY"}$

Factor of Safety $$FS_{EPH} := \frac{\phi M_{eph}}{PosEdgePlateMoment2}$$ $FS_{EPH} = 2.917$

Controlling Factor of Safety

$$FS_2 := \min\left(FS_{S2}, FS_{MPV}, FS_{EPV}, FS_{MPH}, FS_{EPH}\right)$$ $FS_2 = 1.529$

Controlling Mechanism

$$Comment2 := if\left(FS_2 < FS_{S2}, \text{"Wall 2 Flexural Steel"}, \text{"Wall 2 Shear"}\right)$$ $Comment2 = \text{"Wall 2 Flexural Steel"}$

Uplift on Structure

STRUCTURE DOES NOT HAVE HEELS OR SOIL ON TOP OF IT

$Wall_1 := (12ft + 6in)(750ft - 731ft)(1ft + 9in)$ $\quad Wall_1 \cdot 150pcf = 62.344\,kips$

$Wall_2 := (9ft + 6in)(750ft - 731ft)(1ft + 9in)$ $\quad Wall_2 \cdot 150pcf = 47.381\,kips$

$Wall_3 := (12ft + 6in)(1ft + 3in)(750ft - 731ft)$ $\quad Wall_3 \cdot 150pcf = 44.531\,kips$

$Wall_4 := (9ft + 6in)(1ft + 3in)(750ft - 731ft)$ $\quad Wall_4 \cdot 150pcf = 33.844\,kips$

$GW_1 := (6ft + 5in)(1ft + 2in)(750ft - 731ft)$ $\quad GW_1 \cdot 150pcf = 21.335\,kips$

$GW_2 := (5ft)(1ft + 2in)(750ft - 730ft)$ $\quad GW_2 \cdot 150pcf = 17.5\,kips$

$GW := (760.7ft - 750ft)[(5ft + 2in + 1ft + 3in)(6ft + 5in) - 5ft \cdot (4ft + 6in)]$ $\quad GW \cdot 150pcf = 29.971\,kip$

$BS_1 := (730.75ft - 728.5ft)(5ft + 3in + 1ft + 2in)(6ft + 2in + 1ft + 3in)$ $\quad BS_1 \cdot 150pcf = 16.062\,kip$

$GWSlab := (730ft - 728.5ft)(6ft + 3in)(5ft + 9in)$ $\quad GWSlab \cdot 150pcf = 8085.938\,lb$

$WetSlab := (731ft - 728.5ft)(5ft + 3in + 1ft + 2in) \cdot 12ft$ $\quad WetSlab \cdot 150pcf = 28875\,lb$

$BaseSlab := 12ft \cdot 12.5ft$ $\quad BaseSlab = 150\,ft^2$

$Baset := 731ft - 728.5ft$ $\quad Baset = 2.5\,ft$

$Structure := (Wall_1 + Wall_2 + Wall_3 + Wall_4 + GW_1 + GW_2 + GW + BS_1 + GWSlab + WetSlab) \cdot 150pcf$

$Structure = 309.929\,kip$

$Water := HEAD \cdot BaseSlab$ $\quad Water = 0$

$Soil := 0lbf$

Dissipated Head, if structural foundation in blanket. $\quad HGL = 13.5\,ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot BaseSlab \cdot \gamma_w \qquad U_1 = 256.648\,kips$$

$BLANKET = 28\,ft$

$HGL = 13.5\,ft$

$H = 18.5\,ft$

Full Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL + Baset) \cdot (BaseSlab) \cdot \gamma_w\right] \qquad U_2 = 322.92\,kips$$

$$Uplift := if\left(H > BLANKET, U_2, U_1\right)$$

$Uplift = 256.648\,kips$

$$Stability := \frac{Structure + Water + Soil}{Uplift}$$

$Stability = 1.208$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not\ Okay")$ $\quad StabilityCheck = "Okay"$

Equipment weight and other neglected items will increase uplift factor of safety.

BASE SLAB ANALYSIS

$W_{water} := 0lbf$

Slab Strength

SHEAR

$THRUST_S := \frac{W \cdot L_1}{2}$ $\quad THRUST_S = 13283 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$\frac{Structure + W_{water}}{BaseSlab} = 2066.195\,psf$

$W' := if\left(Stability < 1.0, \frac{Uplift}{BaseSlab}, \frac{Structure}{BaseSlab}\right)$ $\quad W' = 2066.195\,psf$

$\frac{Uplift}{BaseSlab} = 1710.986\,psf$

$Slab_t := 2.25ft$ $\quad L_{strip} := 12.5ft$ $\quad Wall_t := 1.25ft$ $\quad Slab_d := 23.5in$ effective depth of slab

$V'_{ub} := W' \cdot \frac{L_{strip}}{2}$ $\quad V'_{ub} = 12914 \frac{lb}{ft}$ $\quad W' = 2066.195 \frac{lb}{ft^2}$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}} \cdot \left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $\quad V_{ub} = 7576 \frac{lb}{ft}$

$A_g := D_f 12 \frac{in}{ft}$ $\quad A_g = 360 \frac{in^2}{ft}$

$Water = 0$

ANALYZE STRENGTH WITHOUT THE AID OF THRUST

$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S \cdot 0}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f\right)\right]$ ACI EQ (11-4) $\quad \phi V_{nb} = 35492 \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, "OKAY", "NO GOOD"\right)$ $\quad CheckBase = "OKAY"$

Factor of Safety $\quad FS_b := \frac{\phi V_{nb}}{V_{ub}}$ $\quad FS_b = 4.685$ $\quad 1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 1.018$

Slab Bending

$\frac{L_2}{L_1} = 0.956$ $\quad L_1 = 11.25\ ft$ $\quad L_2 = 10.75\ ft$

REBAR PARALLEL TO LONG WALL

ratio = 1.0 $\quad 0.0213 \cdot W' \cdot L_2^{\ 2} = 5.086\ kip \cdot \frac{ft}{ft}$

$W' = 2066.195\ psf$ $\quad 0.05 \cdot W' \cdot L_2^{\ 2} = 11.939\ kip \cdot \frac{ft}{ft}$

Simple Conservative Analysis	Plate Analysis		Steel Depth
$\frac{W' \cdot (31ft)^2 \cdot 1ft}{8} = 248.202\ kip \cdot ft$	$MidPlateMoment := 5.086 kip \cdot \frac{ft}{ft}$	My	$d_1 := 27in - 3in - 0.5in$
	$EdgePlateMoment := 11.94 kip \cdot \frac{ft}{ft}$	My	$d_2 := 27in - 3in - 0.5in$

#5's @ 12 in. inside face (mid-plate moment)
A-10-1989, TOP REINF

$A_{mpl} := 0.31 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mpl} = 0.405\ in$

$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_1 - \frac{a_{mpl}}{2}\right)$ $\quad \phi M_{mpl} = 24.1 \frac{kip \cdot ft}{ft}$

#5's @ 12" Outside Face (edge-plate moment)
A-10-1989, BOTTOM REINF

$A_{epl} := 0.31 \frac{in^2}{ft}$ $\quad a_{epl} := \frac{A_{epl} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{epl} = 0.405\ in$

$\phi M_{epl} := \phi_B \cdot A_{epl} \cdot F_y \cdot \left(d_2 - \frac{a_{epl}}{2}\right)$ $\quad \phi M_{epl} = 24.1 \frac{kip \cdot ft}{ft}$

$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot MidPlateMoment, "OKAY", "NO\ GOOD"\right)$ $\quad CheckMP = "OKAY"$

Factor of Safety $\quad FS_{MPL} := \frac{\phi M_{mpl}}{MidPlateMoment}$ $\quad FS_{MPL} = 4.733$

$CheckEPL := if\left(\phi M_{epl} > 1.5 \cdot EdgePlateMoment, "OKAY", "NO\ GOOD"\right)$ $\quad CheckEPL = "OKAY"$

Factor of Safety $\quad FS_{EPL} := \frac{\phi M_{epl}}{EdgePlateMoment}$ $\quad FS_{EPL} = 2.016$

BASE SLAB REBAR PARALLEL TO THE SHORT SIDE

$L_1 = 11.25\ ft$ $\quad L_2 = 10.75\ ft$ $\quad \frac{L_2}{L_1} = 0.956$

REBAR PARALLEL TO SHORT WALL

ratio = 1.0

$0.0213 \cdot W' \cdot L_2^{\ 2} = 5.086\ kip \cdot \frac{ft}{ft}$

$0.05 \cdot W' \cdot L_2^{\ 2} = 11.939\ kip \cdot \frac{ft}{ft}$

Simple Conservative Analysis

$\frac{W' \cdot (31ft)^2 \cdot 1ft}{8} = 248.202\ kip \cdot ft$

Plate Analysis

$MidPlateMoment2 := 5.086 kip \cdot \frac{ft}{ft}$ My

$EdgePlateMoment2 := 11.94 kip \cdot \frac{ft}{ft}$ My

Steel Depth

$d_1 := 27in - 3in - 0.5in$

$d_2 := 27in - 3in - 0.5in$

#6's @ 12 in. inside face (mid-plate moment)

A-10-1989, TOP REINF

$A_{mps} := 0.31 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a_{mps} := \frac{A_{mps} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mps} = 0.405\ in$

$\phi M_{mps} := \phi_B \cdot A_{mps} \cdot F_y \cdot \left(d_1 - \frac{a_{mps}}{2}\right)$ $\quad \phi M_{mps} = 24.1 \frac{kip \cdot ft}{ft}$

#6's @ 12" Outside Face (edge-plate moment)

A-10-1989, BOTTOM REINF

$A_{eps} := 0.31 \frac{in^2}{ft}$ $\quad a_{eps} := \frac{A_{eps} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{eps} = 0.405\ in$

$\phi M_{eps} := \phi_B \cdot A_{eps} \cdot F_y \cdot \left(d_2 - \frac{a_{eps}}{2}\right)$ $\quad \phi M_{eps} = 24.1 \frac{kip \cdot ft}{ft}$

$CheckMP := if\left(\phi M_{mps} > 1.5 \cdot MidPlateMoment2, "OKAY", "NO\ GOOD"\right)$ $\quad CheckMP = "OKAY"$

Factor of Safety $\quad FS_{MPS} := \frac{\phi M_{mps}}{MidPlateMoment2}$ $\quad FS_{MPS} = 4.733$

$CheckEPS := if\left(\phi M_{eps} > 1.5 \cdot EdgePlateMoment2, "OKAY", "NO\ GOOD"\right)$ $\quad CheckEPS = "OKAY"$

Factor of Safety $\quad FS_{EPS} := \frac{\phi M_{eps}}{EdgePlateMoment2}$ $\quad FS_{EPS} = 2.016$

Controlling Factor of Safety

$FS_B := \min(FS_b, FS_{EPL}, FS_{EPS}, FS_{MPL}, FS_{EPL}, FS_{MPS})$ $\boxed{FS_B = 2.016}$

Controlling Mechanism

$CommentB := \text{if}(FS_B < FS_b, \text{"Base Flexural Steel"}, \text{"Base Shear"})$ $\boxed{CommentB = \text{"Base Flexural Steel"}}$

SUMMARY OF FACTORS OF SAFETY

Wall 1: $\boxed{FS_1 = 1.529}$

$\boxed{Comment1 = \text{"Wall 1 Flexural Steel"}}$

Wall 2: $\boxed{FS_2 = 1.529}$

$\boxed{Comment2 = \text{"Wall 2 Flexural Steel"}}$

Base Slab: $\boxed{FS_B = 2.016}$

$\boxed{CommentB = \text{"Base Flexural Steel"}}$

Uplift: $\boxed{Stability = 1.208}$

$\boxed{StabilityCheck = \text{"Okay"}}$

END OF ANALYSIS

The May 1971 Design Memorandum (DM) No. 3, page 52 states the following in regards to the KCS pump station outlet works:

"Station 276+79. A 42-inch diameter C.I.P. outfall sewer extending from the Kansas City Southern Plant crosses under the floodwall at this point. The pumping plant was completed in 1949 by the Government under the flood control program and is located landward of the retaining wall of the Kansas City Southern Railroad's depressed track area. The gatewell at the pumping plant houses a sluice gate. The gatewell will be raised 1.8 feet to the new top of the protection. About 115 feet of 42-inch C.I.P. and a concrete outlet structure with flap gate were installed riverward from the pumping plant. All are in good condition. A section thru the plant and outlet structure is shown on plate 66."

Drawing excerpts of the drainage system from the O&MM manual are attached showing the 42-inch cast iron pipe. The outlet has a flap gate. It is very likely the pipe was installed in the trench condition. The O&MM drawings do not show how the pipe was laid.

The article excerpt from DM 3 reported in 1971 that the drainage system "all are in good condition". The DM does not provide information as to what action was taken in order to come to this conclusion.

Background of CIP. EM 1110-2-2902 states "Ductile iron has replaced cast iron pipe in use and application. " (page 7-1). FEMA 484 Technical Manual: Conduits through Embankment Dams states on page 54 of its document "Cast-iron pipe has been used in the past for conduits through embankment dams, but is currently not considered acceptable for new embankment dam construction by any of the federal dam-building agencies. The disadvantages of using cast-iron pipe include:
* Cast-iron pipe is heavy and makes handling difficult.
* Joints are bell-and-spigot. Cast-iron pipe cannot be welded, so that flanged joints are not possible.
* Not normally commercially available in the realm of diameters of most conduits. Custom fabrication would be very expensive.
* Lacks a favorable shape for compaction of earthfill against the conduit.
* Cast-iron pipe is brittle and can crack, if not properly handled."

Specifications or inspection history of the CIP (other than the DM 3 entry above) was not available for review.

Given the date the outlet works was constructed (1949) it is possible the CIP has leadite joints. Leadite has a different coefficient of expansion than cast iron and facilitates pitting corrosion. The failure rate for leadite joint pipe is significantly higher than for lead joint pipe (EPA Distribution System Issue Paper, Deteriorating Buried Infrastructure Management Challenges and Strategies, May 2002). These (leadite) joints are more likely to leak relative to a rubber gasketed joint which came onboard in the pipe industry in the mid 1950s. Note that ductile iron pipe (DIP) is roughly twice as strong as CIP as determined by its mechanical properties.

Based on the information obtained, the following is recommended:

BASED ON THE ABOVE INFORMATION, ASSUME THE PIPE WILL REQUIRE REHABILIATION. THE REHABILITATION METHOD WILL BE A STEEL SLIPLINER. ALSO PERFORM A COMPLETE VISUAL INSPECTION OF THE PIPE PAYING SPECIAL ATTENTION TO THE JOINTS AND CRACKS. THIS ACTION SHOULD BE ACCOMPLISHED PRIOR TO DETERMINING WHAT ACTION IS NECESSARY TO ENSURE THAT THE OUTLET WORKS PERFORMS RELIABLY**.

**Be aware that if it is found during the visual inspection that the pipe joints are leaking, extra precautions shall be taken during the rehabilitation design if a slip liner will be used. When the liner is inserted, seepage into the conduit is eliminated, causing a change to the flow patterns within the embankment. The flow pattern change may be detrimental to the line of protection by prompting flow along CIP exterior surface areas. The seepage and potential for internal erosion or backward erosion piping along the conduit must be addressed by an acceptable means. This may include the installation of a filter diaphragm, collar or warrant conduit removal and replacement.

APPENDIX I

Kcs RAILROAD PUMP STATION OUTLET WORKS EXCERPT

RECORD DRAWINGS

VOLUME ONE

OPERATION AND MAINTENANCE MANUAL

KANSAS CITYS FLOOD CONTROL PROJECT

Missouri and Kansas Rivers

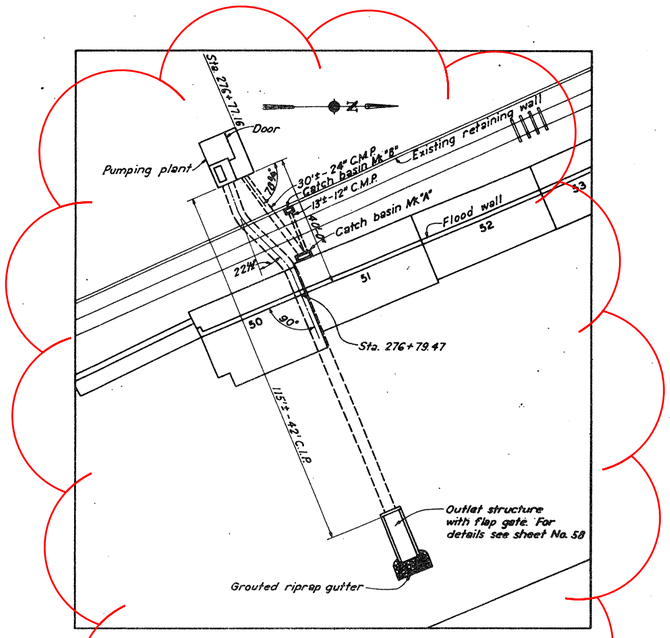
ARMOURDALE UNIT

FILE COPY
EST. & SPEC. SEC.

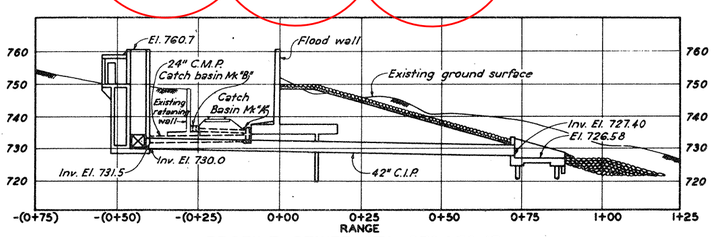
DEPARTMENT OF THE ARMY

KANSAS CITY DISTRICT, CORPS OF ENGINEERS

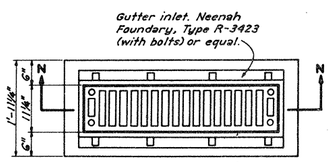
SCALE AS SHOWN
PER DRAWING



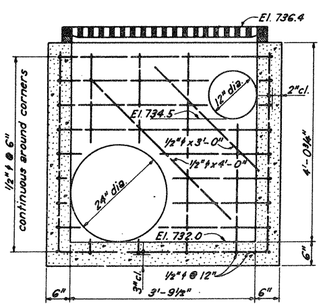
PLAN - SEWER STA. 276+79.47
Scale: 1" = 20'



PROFILE - SEWER STA. 276+79.47
Scale: 1" = 20'

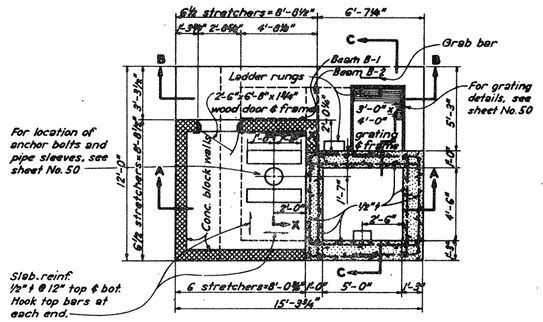


PLAN AT TOP

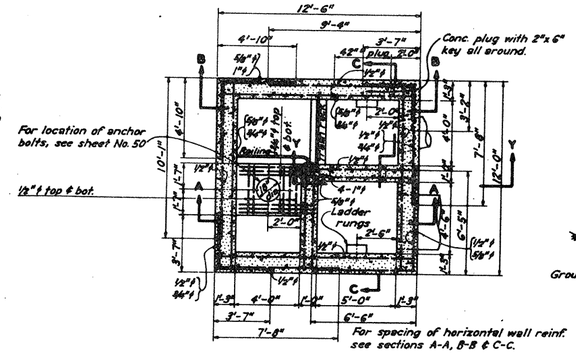


SECTION N-N

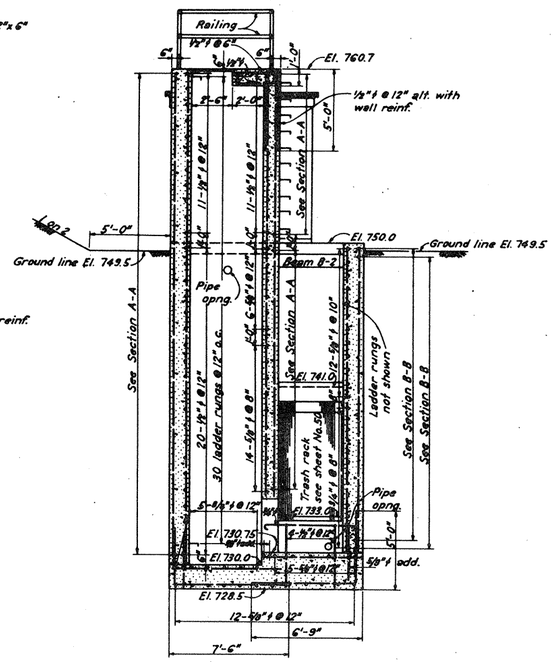
CATCH BASIN MARK "A"
Scale: ¾" = 1'-0"



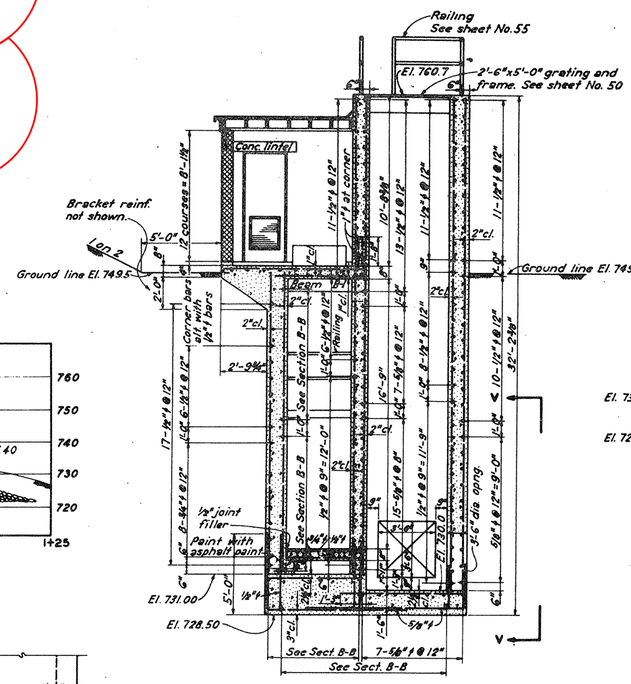
OPERATING FLOOR PLAN
Scale: ¼" = 1'-0"



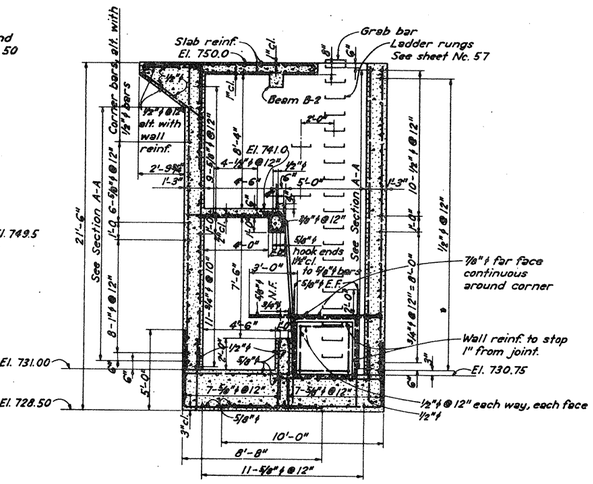
SECTIONAL PLAN OF SUBSTRUCTURE
Scale: ¼" = 1'-0"



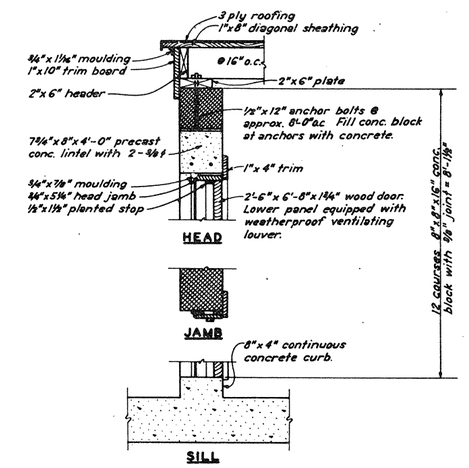
SECTION C-C
Scale: ¼" = 1'-0"



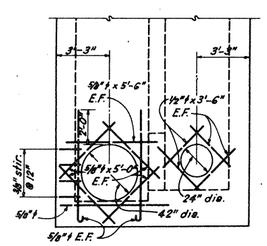
SECTION A-A
Scale 1/4" = 1'-0"



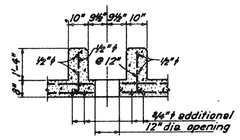
SECTION B-B
Scale: ¼" = 1'-0"



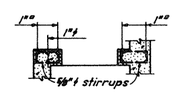
TYPICAL WALL AND DOOR SECTION
Scale: 1" = 1'-0"



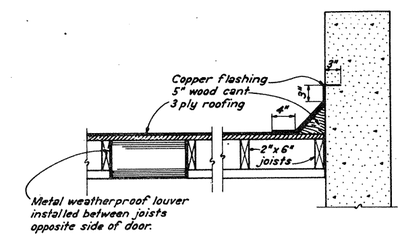
ELEVATION V-V
Scale: ¼" = 1'-0"



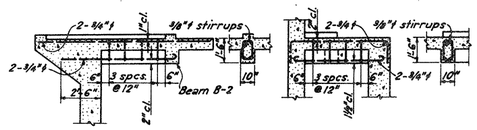
SECTION X-X
Scale: 5/8" = 1'-0"



SECTION W-W
Scale: ¼" = 1'-0"

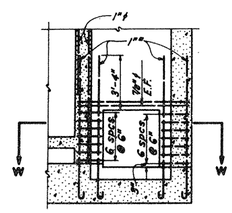


DETAILS OF LOUVER AND ROOF FLASHING AT DISCHARGE CHAMBER
Scale: 1" = 1'-0"



BEAM B-1
Scale: ¼" = 1'-0"

BEAM B-2
Scale: ¼" = 1'-0"



SECTION Y-Y
Showing reinf. around 3'-6" sq. opn'g.
Scale: ¼" = 1'-0"

NOTES:
All vertical reinf. ½"ϕ, unless otherwise noted, spaced as shown.
Reinf. at openings is additional.
Reinf. shall be lapped 40 dia. unless otherwise shown.
For details of Catch Basin Mk"B", see sheet No. 50.

RECORD DRAWING

APRIL 1951

NO.	DATE	BY	REVISION	APP'D.
2	4-16-51	R.L.G.	Minor revisions for "as built" conditions.	
1	5/9/49	R.L.G.	Revised in accordance with Add. #1, dated 6 Apr. 1949	

KANSAS CITY FLOOD CONTROL PROJECT

ARMOURDALE UNIT

PUMPING PLANT
STA. 276+79.47

In 62 sheets — Sheet No. 49 — Scale: as shown

OFFICE, DISTRICT ENGINEER
KANSAS CITY DISTRICT
KANSAS CITY, MO.
FEBRUARY 1949

Submitted: Chief, Design Branch
Recommended: Chief, Engineering Division
Approved: Col., C.of E., District Engineer

Drawn by: H.J.B. — Traced by: O.T. — Checked by: R.L.G. — To accompany specifications dated: FEBRUARY 1949 — File No. A-10-1989

APPENDIX I

RECORD DRAWINGS

VOLUME TWO

OPERATION AND MAINTENANCE MANUAL

KANSAS CITYS FLOOD CONTROL PROJECT

Missouri and Kansas Rivers

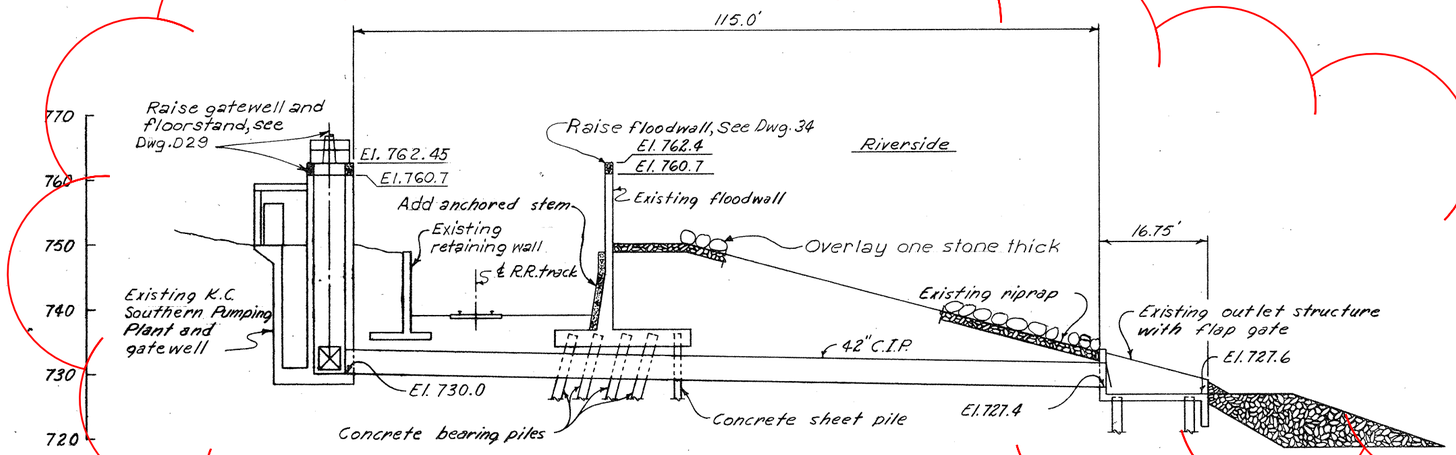
ARMOURDALE UNIT

DEPARTMENT OF THE ARMY

KANSAS CITY DISTRICT, CORPS OF ENGINEERS

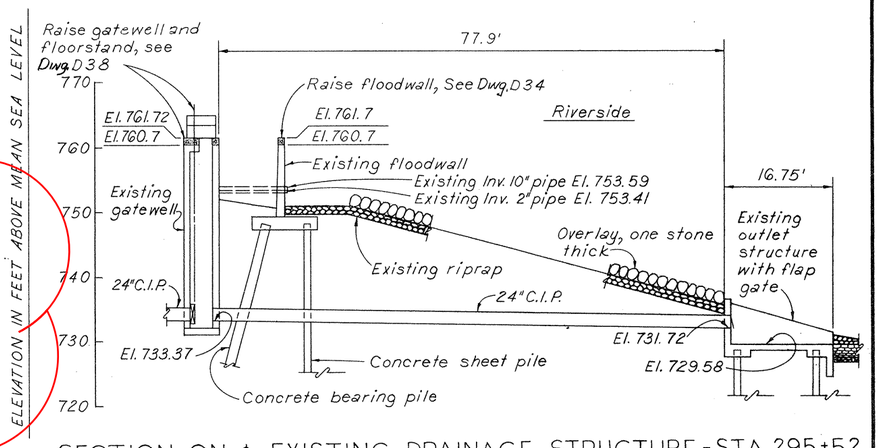
FILE COPY
EST. & SPEC. SEC.

SCALE AS SHOWN
PER DRAWING



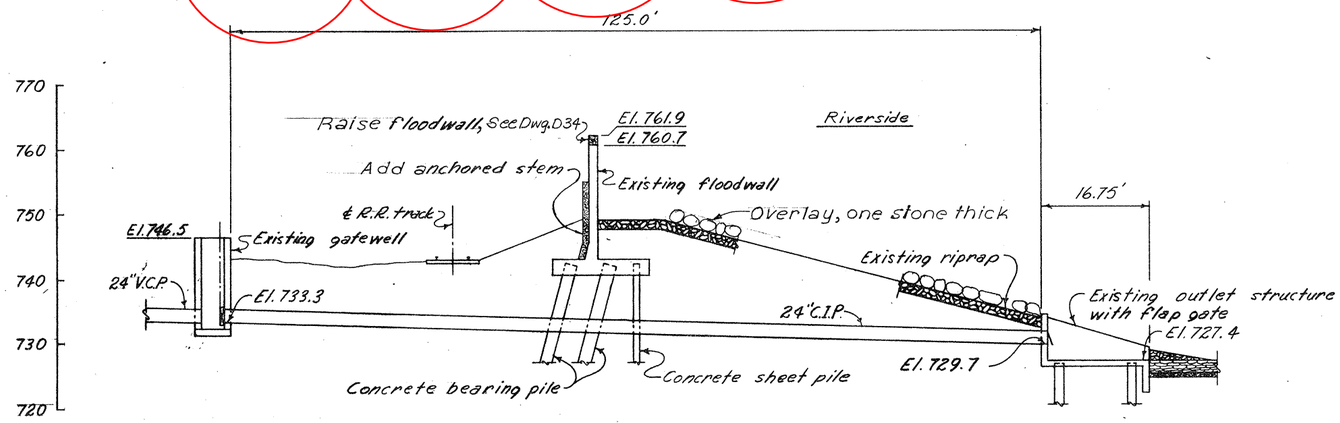
SECTION ON ℄ EXISTING DRAINAGE STRUCTURE - STA. 276 + 79

Scale: 1" = 10'



SECTION ON ℄ EXISTING DRAINAGE STRUCTURE - STA. 295 + 52

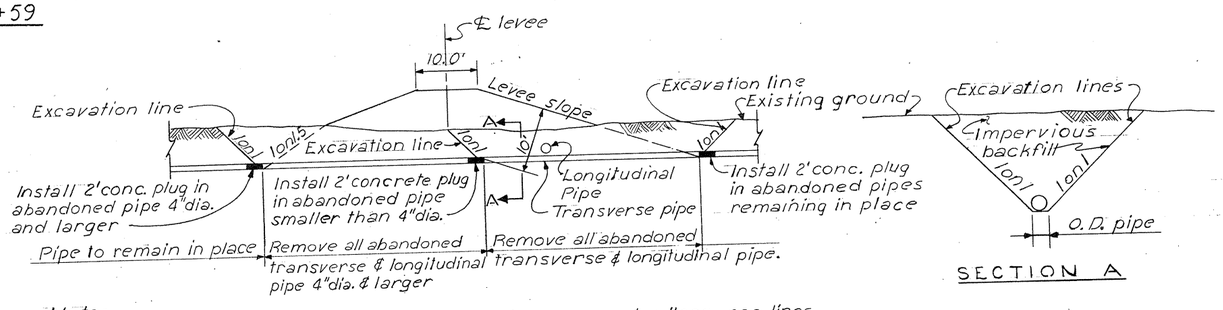
Scale: 1" = 10'



SECTION ON ℄ EXISTING DRAINAGE STRUCTURE - STA. 286 + 59

(No modification to drainage structures required)

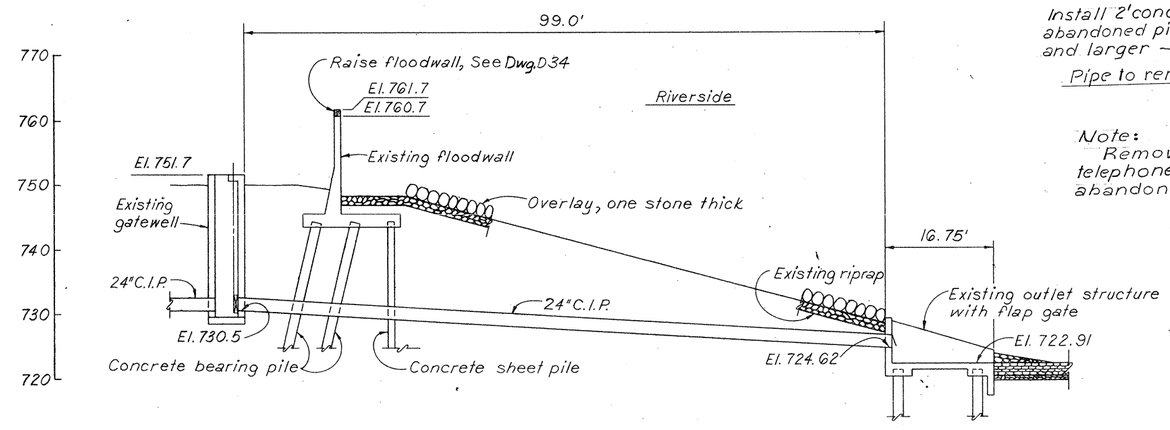
Scale: 1" = 10'



Note:
Removal of pipe shall include all abandoned sewers, water lines, gas lines, telephone conduits, manholes, valves boxes and headwalls, including those abandoned due to alterations by others, except where otherwise noted.

TYPICAL DETAILS FOR REMOVAL ABANDONED PIPES

Not to Scale



SECTION ON ℄ EXISTING DRAINAGE STRUCTURE - STA. 290 + 52

(No modification to drainage structures required)

Scale: 1" = 10'

NOTE:
For locations of Riprap see typ. sections, Dwgs. Nos. B-1 thru. B-4

RECORD DRAWING

MAY 1976

CONTRACT NO. DACW41-74-C-0093

SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" conditions	9/20/78	

REVISIONS

KANSAS RIVER, KANSAS

KANSAS CITY, KANSAS - 1962 MOD. ARMOURDALE UNIT

DRAINAGE STRUCTURES SECTIONS

In 109 sheets

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

Submitted: CHIEF, GENERAL STRUCTURAL SECT.

Sheet No. 81
Dwg. No. D37

Recommended: CHIEF, DESIGN BRANCH

Scale: as shown

U. S. ARMY
MAY 1973

Approved: CHIEF, ENGINEERING DIVISION

DESIGNED BY: J.H.D. DRAWN BY: J.F.V. CHECKED BY: R.S.H.

FILE NO. A-10-6078

O. & M. M. PLATE NO. 237

Variables

$kip := 1000lb \quad plf := \frac{lb}{ft} \quad psf \equiv \frac{lb}{ft^2} \quad pcf := \frac{lb}{ft^3} \quad psi \equiv \frac{lb}{in^2} \quad ksi \equiv \frac{1000lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad kips := 1000lb \quad klf := \frac{kips}{ft}$

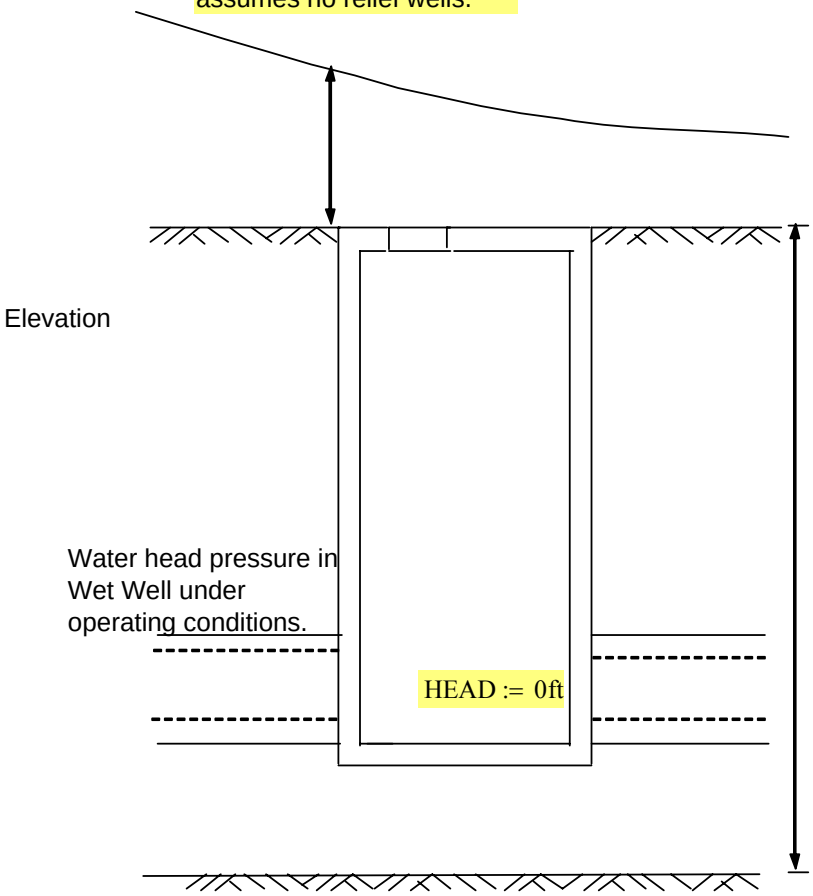
Notes:
1. Walls analyzed for two-way action, plate fixed along three edges, hinged along one edge (top slab connection), uniformly varying load. Figure 13 of the Bureau monographs.
2. Base slab analyzed for two-way action, plate fixed along four edges, uniform load. Figure 34 of Bureau monographs.
3. Two walls were analyzed.

REFERENCES
1. Moments and Reactions for Rectangular Plates, Monograph No. 27, Bueau of Reclamation, July 1963

Properties

NOTE: HGL Supplied assumes no relief wells.

Original Hydraulic Grade Line as supplied by Geotechnicals (Affects Uplift on Base Slab if in the landside of levee)



$ELEV_1 := 766.5ft$

$HGL := 772.20ft - ELEV_1$

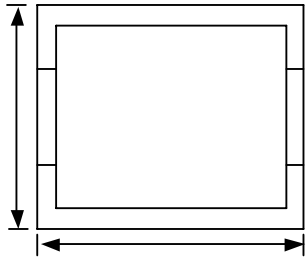
$HGL = 5.7ft$

$BLANKET := 760ft - 742ft$

$BLANKET = 18\ ft$

$ELEV_2 := 739.67ft$

Plan



$L_1 := 19ft + 10in$

$L_1 = 19.833\ ft$

$H := ELEV_1 - ELEV_2$

$H = 26.83\ ft$

Wall Thickness 1 $D_1 := 1.75ft$

Clear Cover 1 $C_{c1} := 2.5in$

$L_2 := 18ft + 8in$

$L_2 = 18.667\ ft$

Floor Thickness $D_f := 2ft$

Clear Cover $C_{cf} := 4in$

Wall Thickness 2 $D_2 := 1.75ft$

Clear Cover 2 $C_{c2} := 2.5in$

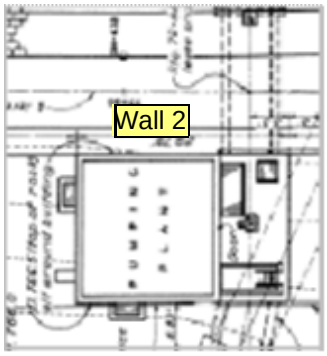
$l_1 := L_1 \quad d_1 := D_1 - C_{c1} \quad d_f := D_f - C_{cf}$

$l_2 := L_2 \quad d_2 := D_2 - C_{c2} \quad d_2 = 1.542\ ft$

Lengths L1 and L2 are center to center distances

Wall Layout

Wall 2 is parallel to the levee centerline



Levee Centerline

$\frac{L_1}{H} = 0.739$

Wall 1

Wall 3

NORTH

$\frac{L_2}{H} = 0.696$

Wall 1 is perpendicular to the levee centerline

File A-10-3112 Excerpt
Wall 4

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Soil Properties $\phi := 26deg$

$\gamma := 125pcf$

$K_o := 1 - \sin(\phi)$

$K_o = 0.562$

Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40ksi$

Concrete Properties $f'_c := 3.00ksi$

Concrete Unit Weight $\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

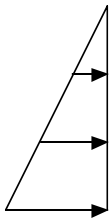
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALLS $K_o = 0.562$ $BLANKET = 18\ ft$

Wall Loading

Soil or Soil & Water



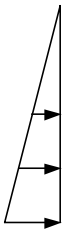
$H_2 := H$

$H_2 = 26.83\ ft$

OR

$H_2 = 26.83\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot H - HEAD$

$H''_3 := H + HGL - HEAD$

$H_3 := if(H > BLANKET, H''_3, H'_3)$

$H_3 = 32.53\ ft$

$H'_3 = 35.326\ ft$

$H''_3 = 32.53\ ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_1 = 1884\ \frac{lb}{ft^2}$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3]$

$W_2 = 2973\ \frac{lb}{ft^2}$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W := if(W_1 > W_2, W_1, W_2)$

$W = 2973\ \frac{lb}{ft^2}$

SHEAR

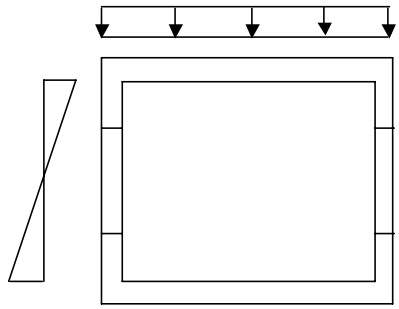
WALL 1

$THRUST_1 := \frac{W \cdot L_2}{2}$ $\quad THRUST_1 = 27749 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W \cdot \frac{L_1}{2}$

$V'_{u1} = 29484 \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$ $\quad V_{u1} = 22299 \frac{lb}{ft}$

$D_2 = 1.75\ ft$

$d_1 = 1.542\ ft$

$L_1 = 19.833\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 252 \frac{in^2}{ft}$

$\frac{L_1}{d_1} = 12.865$

EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 25658 \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO\ GOOD"\right)$

$Check1 = "NO\ GOOD"$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$

$FS_{S1} = 1.151$

If wall does not meet shear requirements when analyzed as 1ft wide strip. Consider Plate reactions.

From Figure 13 of the Bureau

$Ry := 0.2647 \cdot W \cdot \left(ELEV_1 - ELEV_2\right)$ $\quad Ry = 21.115 \frac{kip}{ft}$

$V'_{u1} := 21.12 \frac{kips}{ft}$

$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$

$V_{u1} = 15973 \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO\ GOOD"\right)$

$Check1 = "OKAY"$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$

$FS_{S1} = 1.606$

$W = 2973.16 \frac{lb}{ft^2}$

ratio = 3/8

$0.0063 \cdot 2973psf \cdot \left(ELEV_1 - ELEV_2\right)^2 = 13.483\ kip \cdot \frac{ft}{ft}$

Figure 13 monograph

$0.02 \cdot 2973psf \cdot \left(ELEV_1 - ELEV_2\right)^2 = 42.802\ kip \cdot \frac{ft}{ft}$

Wall 1 Bending

Vertical Steel

Simple Conservative Analysis

$M_{u1} := \frac{W \cdot L_1^2}{12}$ $M_{u1} = 97460 \frac{ft \cdot lb}{ft}$

Plate Analysis

$NegMidPlateMoment := 13.48 kip \cdot \frac{ft}{ft}$ My

$PosEdgePlateMoment := 42.80 kip \cdot \frac{ft}{ft}$ My

#7's @ 10 in. inside face (mid-plate moment)

A-10-3118

$A_{mpv} := 0.722 \frac{in^2}{ft}$ $b := 12 \frac{in}{ft}$ $a_{mpv} := \frac{A_{mpv} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mpv} = 0.944 in$

$\phi M_{mpv} := \phi_B \cdot A_{mpv} \cdot F_y \cdot \left(d_1 - \frac{a_{mpv}}{2}\right)$ $\phi M_{mpv} = 43.4 \frac{kip \cdot ft}{ft}$

#9's @ 9 in. outside face at wall base (edge-plate moment)

A-10-3120

$A_{epv} := 1.33 \frac{in^2}{ft}$ $a_{epv} := \frac{A_{epv} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{epv} = 1.739 in$

$\phi M_{epv} := \phi_B \cdot A_{epv} \cdot F_y \cdot \left(d_1 - \frac{a_{epv}}{2}\right)$ $\phi M_{epv} = 78.2 \frac{kip \cdot ft}{ft}$

$CheckMPV := if\left(\phi M_{mpv} > 1.5 \cdot NegMidPlateMoment, "OKAY", "NO GOOD"\right)$ $CheckMPV = "OKAY"$

Factor of Safety $FS_{MPV} := \frac{\phi M_{mpv}}{NegMidPlateMoment}$ $FS_{MPV} = 3.219$

$CheckEPV := if\left(\phi M_{epv} > 1.5 \cdot PosEdgePlateMoment, "OKAY", "NO GOOD"\right)$ $CheckEPV = "OKAY"$

Factor of Safety $FS_{EPV} := \frac{\phi M_{epv}}{PosEdgePlateMoment}$ $FS_{EPV} = 1.826$

ratio = 3/8 $0.0098 \cdot 2973 psf \cdot \left(ELEV_1 - ELEV_2\right)^2 = 20.973 kip \cdot \frac{ft}{ft}$

Figure 13 monograph $0.0207 \cdot 2973 psf \cdot \left(ELEV_1 - ELEV_2\right)^2 = 44.3 kip \cdot \frac{ft}{ft}$

Wall 1 Bending

Horizontal Steel

Plate Analysis

Simple Conservative Analysis

$M_{u1} = 97460 \frac{ft \cdot lb}{ft}$

$NegMidPlateMoment2 := 20.97 kip \cdot \frac{ft}{ft}$ Mx

$PosEdgePlateMoment2 := 44.3 kip \cdot \frac{ft}{ft}$ Mx

#9's @ 12 in. inside face (mid-plate moment)

A-10-3119

$A_{mph} := 1 \frac{in^2}{ft}$ $\quad a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mph} = 1.307\ in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_1 - \frac{a_{mph}}{2}\right)$ $\quad \phi M_{mph} = 59.5 \frac{kip \cdot ft}{ft}$

#9's @ 10 in. outside face (edge-plate moment)

A-10-3119

$A_{eph} := 1.199 \frac{in^2}{ft}$ $\quad a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{eph} = 1.567\ in$

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_1 - \frac{a_{eph}}{2}\right)$ $\quad \phi M_{eph} = 70.8 \frac{kip \cdot ft}{ft}$

$CheckMPH := if\left(\phi M_{mph} > 1.5 \cdot NegMidPlateMoment2, "OKAY", "NO\ GOOD"\right)$ $\quad CheckMPH = "OKAY"$

Factor of Safety $\quad FS_{MPH} := \frac{\phi M_{mph}}{NegMidPlateMoment2}$ $\quad FS_{MPH} = 2.837$

$CheckEPH := if\left(\phi M_{eph} > 1.5 \cdot PosEdgePlateMoment2, "OKAY", "NO\ GOOD"\right)$ $\quad CheckEPH = "OKAY"$

Factor of Safety $\quad FS_{EPH} := \frac{\phi M_{eph}}{PosEdgePlateMoment2}$ $\quad FS_{EPH} = 1.598$

Controlling Factor of Safety

$FS_1 := \min\left(FS_{S1}, FS_{MPV}, FS_{EPV}, FS_{MPH}, FS_{EPH}\right)$ $\quad FS_1 = 1.598$

Controlling Mechanism

$Comment1 := if\left(FS_1 < FS_{S1}, "Wall\ 1\ Flexural\ Steel", "Wall\ 1\ Shear"\right)$ $\quad Comment1 = "Wall\ 1\ Flexural\ Steel"$

WALL 2

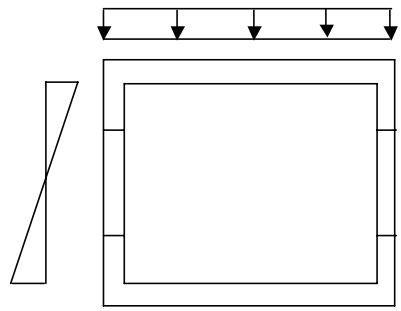
SHEAR

$THRUST_2 := \frac{W \cdot L_1}{2}$ $\quad THRUST_2 = 29484 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u2} := W \cdot \frac{L_2}{2}$

$V'_{u2} = 27749 \frac{lb}{ft}$



$V_{u2} := 2 \frac{V'_{u2}}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2 \right)$ $\quad V_{u2} = 20564 \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1 $\quad \frac{L_2}{d_2} = 12.108$

$A_g := D_2 \cdot 12 \frac{in}{ft}$ $\quad A_g = 252 \frac{in^2}{ft}$

EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_2 \right) \right]$ ACI EQ (11-4)

$d_2 = 1.542$ ft

$\phi V_{n2} = 25742 \frac{lb}{ft}$

$Check2 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$ $\quad Check2 = "NO GOOD"$

Factor of Safety $\quad FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$ $\quad FS_{S2} = 1.252$

If wall does not meet shear requirements when analyzed as 1ft wide strip. Consider Plate reactions.

From Figure 13 of the Bureau $\quad Ry := 0.2647 \cdot W \cdot (ELEV_1 - ELEV_2)$ $\quad Ry = 21.115 \frac{kip}{ft}$ $\quad V'_{u2} := 21.12 \frac{kips}{ft}$

$V_{u2} := 2 \frac{V'_{u2}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_1}{2} - d_2 \right)$ $\quad V_{u2} = 15973 \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$ $\quad Check1 = "OKAY"$

Factor of Safety $\quad FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$ $\quad FS_{S2} = 1.612$

ratio = 3/8

$$0.0063 \cdot 2973\text{psf} \cdot (ELEV_1 - ELEV_2)^2 = 13.483\ \text{kip} \cdot \frac{\text{ft}}{\text{ft}}$$

Figure 13 monograph

$$0.02 \cdot 2973\text{psf} \cdot (ELEV_1 - ELEV_2)^2 = 42.802\ \text{kip} \cdot \frac{\text{ft}}{\text{ft}}$$

Wall 2 Bending

Vertical Steel

Simple Conservative Analysis

Plate Analysis

$$M_{u1} := \frac{W \cdot L_2^{\,2}}{12} \qquad M_{u1} = 86.33\ \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

$$NegMidPlateMoment := 13.48\text{kip} \cdot \frac{\text{ft}}{\text{ft}}$$ My

$$PosEdgePlateMoment := 42.80\text{kip} \cdot \frac{\text{ft}}{\text{ft}}$$ My

#7's @ 12 in. Inside Face (mid-plate moment)

A-10-3118

$$A_{mpv} := 0.60\ \frac{\text{in}^2}{\text{ft}} \qquad b := 12\ \frac{\text{in}}{\text{ft}} \qquad a_{mpv} := \frac{A_{mpv} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{mpv} = 0.784\ \text{in}$$

$$\phi M_{mpv} := \phi_B \cdot A_{mpv} \cdot F_y \cdot \left(d_1 - \frac{a_{mpv}}{2}\right) \qquad \phi M_{mpv} = 36.2\ \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

#8's @ 9 in. outside face (edge-plate moment)

A-10-3120

$$A_{epv} := 1.047\ \frac{\text{in}^2}{\text{ft}} \qquad a_{epv} := \frac{A_{epv} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{epv} = 1.369\ \text{in}$$

$$\phi M_{epv} := \phi_B \cdot A_{epv} \cdot F_y \cdot \left(d_1 - \frac{a_{epv}}{2}\right) \qquad \phi M_{epv} = 62.2\ \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

$$CheckMPV := \text{if}\left(\phi M_{mpv} > 1.5 \cdot NegMidPlateMoment, \text{"OKAY"}, \text{"NO GOOD"}\right)$$ CheckMPV = "OKAY"

Factor of Safety

$$FS_{MPV} := \frac{\phi M_{mpv}}{NegMidPlateMoment}$$ $FS_{MPV} = 2.687$

$$CheckEPV := \text{if}\left(\phi M_{epv} > 1.5 \cdot PosEdgePlateMoment, \text{"OKAY"}, \text{"NO GOOD"}\right)$$ CheckEPV = "NO GOOD"

Factor of Safety

$$FS_{EPV} := \frac{\phi M_{epv}}{PosEdgePlateMoment}$$ $FS_{EPV} = 1.453$

Information for CASTR: $THRUST_2 = 29.48\ \frac{\text{kip}}{\text{ft}}$ $PosEdgePlateMoment = 42.8\ \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$

From CASTR Program Osa3w2EPV26deg.*dat*

```
100 Osa3w2EPV26deg M.Parks
110 wall 2 EPV 26 degree
200 1
210 3.00 40.0 0.375
300 12.0 x 21 with 1 bar
400 12.0 21
500 1
510 1 1.047 18.5 6.0 6.0
600 29.48 42.8
```

Percent Design Strength Used in Tensile Zone

$$Percent := 50.92\%$$

$$FoS' := \frac{1}{Percent \cdot .9}$$

Note: Strength Reduction Factor (.9) not applied.

$$FS_{EPV} := FoS'$$

$FoS' = 2.18$ Wall 2 is OKAY.

ratio = 3/8 $0.0098 \cdot 2973psf \cdot (ELEV_1 - ELEV_2)^2 = 20.973\,kip \cdot \frac{ft}{ft}$

Figure 13 monograph $0.0207 \cdot 2973psf \cdot (ELEV_1 - ELEV_2)^2 = 44.3\,kip \cdot \frac{ft}{ft}$

Wall 2 Bending

Horizontal Steel

Simple Conservative Analysis

$M_{u1} = 86332 \frac{ft \cdot lb}{ft}$

Plate Analysis

$NegMidPlateMoment2 := 20.97kip \cdot \frac{ft}{ft}$ Mx

$PosEdgePlateMoment2 := 44.3kip \cdot \frac{ft}{ft}$ Mx

$L_2 = 18.667\ ft$

#6's @ 12 in. inside face (mid-plate moment)

A-10-3119, Sec E-E

$A_{mph} := 0.442 \frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mph} = 0.578\ in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_2 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 26.8 \frac{kip \cdot ft}{ft}$

#11's @ 12 in. outside face (edge-plate moment)

A-10-3119, Sec F-F

$A_{eph} := 1.56 \frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{eph} = 2.039\ in$

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_2 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 90.9 \frac{kip \cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5 \cdot NegMidPlateMoment2, "OKAY", "NO GOOD")$ $CheckMPH = "NO\ GOOD"$

Factor of Safety $FS_{MPH} := \frac{\phi M_{mph}}{NegMidPlateMoment2}$ $FS_{MPH} = 1.279$

$CheckEPH := if(\phi M_{eph} > 1.5 \cdot PosEdgePlateMoment2, "OKAY", "NO GOOD")$ $CheckEPH = "OKAY"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{PosEdgePlateMoment2}$ $FS_{EPH} = 2.052$

Information for CASTR: $THRUST_2 = 29.48 \frac{kip}{ft}$ $NegMidPlateMoment2 = 20.97 \frac{kip \cdot ft}{ft}$

From CASTR Program Osa3w2MPH26deg.*dat*

```
100 Osa3w2MPH26deg M.Parks
110 wall 2 MPH 26 degree
200 1
210 3.00 40.0 0.375
300 12.0 X 21 with 1 bar
400 12.0 21
500 1
510 1 0.44 18.5 6.0 6.0
600 29.48 20.97
```

Percent Design Strength Used in Tensile Zone

$Percent := 22.3\%$

$FoS' := \frac{1}{Percent \cdot .9}$

Note: Strength Reduction Factor (.9) not applied.

$FS_{EPV} := FoS'$

$FoS' = 4.98$ Wall 2 is OKAY.

Controlling Factor of Safety

$FS_2 := \min(FS_{S2}, FS_{MPV}, FS_{EPV}, FS_{MPH}, FS_{EPH})$ $FS_2 = 1.279$

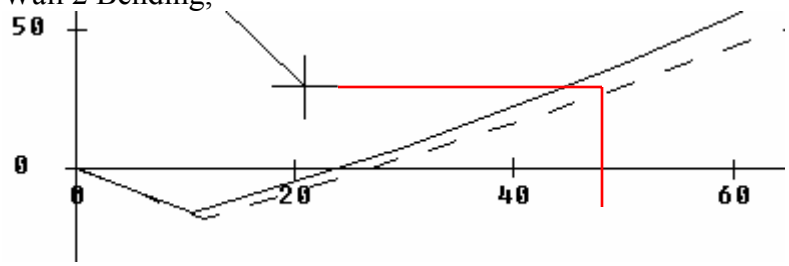
Controlling Mechanism

$Comment2 := \text{if}(FS_2 < FS_{S2}, \text{"Wall 2 Flexural Steel"}, \text{"Wall 2 Shear"})$ $Comment2 = \text{"Wall 2 Flexural Steel"}$

Osage Pump Station Interaction Diagrams.

The CASTR program output prints the percentage of permissible design strength used by the applied loading. The previous calculations used this value to determine the Factor of Safety. Under certain circumstances, using the CASTR output value of percent design strength used by the applied loading may create an conservative factor of safety (FoS') value. As an alternative, the section strength was obtained from the interaction diagram output created by CASTR. The interaction diagram strength value was compared to the demand moment to determine the section bending FoS'.

Wall 2 Bending,



Cen3W1D.dat Interaction Diagram

Using interaction diagram to determine the factor of safety:
Strength = 48 k*ft (from interaction diagram)
Demand= 21 k*ft
FoS' = 48/21 = 2.29

Conclusion. The FoS' obtained using the interaction diagram values of section strength were more conservative.

Uplift on Structure

Stability

$Wall1 := (21ft + 5.5in)(27ft + 10in)(1ft + 9in)$ $\quad Wall1 \cdot 150pcf = 156.78\,kips$

$Wall2 := (17ft + 0.5in)(27ft + 10in)(1ft + 9in)$ $\quad Wall2 \cdot 150pcf = 124.511\,kips$

$Wall3 := [3(1ft + 6in) + 7ft + 9.5in + 9ft + 2in](28ft + 7in)(1.5ft$ $\quad Wall3 \cdot 150pcf = 138.004\,kips$

$Wall4 := [6ft + 10.5in + 2(1.5ft) + 8ft + 8in](27ft + 10in)\,1.5ft$ $\quad Wall4 \cdot 150pcf = 116.117\,kips$

$Wall5 := (27ft + 10in)1.5ft \cdot 5.5ft$ $\quad Wall5 \cdot 150pcf = 34.444\,kips$

$Wall6 := (4ft + 3.5in)(27ft + 10in)\,1.5ft$ $\quad Wall6 \cdot 150pcf = 26.877\,kips$

$Wall7 := (21ft + 10in)1.5ft \cdot 5ft$ $\quad Wall7 \cdot 150pcf = 24.562\,kip$

$Wall8 := (9ft + 2in) \cdot 1.5ft \cdot 26.25ft$ $\quad Wall8 \cdot 150pcf = 54.141\,kip$

Wall 8 is the interior gatewell wall parallel with levee centerline.

Wall 7 is the wall above the skewed interior opening (Sect G on A-10-3118)

Wall 6 is the beam wall located on Wall 4

Wall 5 is part of the skewed wall connected to the interior gatewell wall

$Mason := [(21ft + 5.5in)(29ft + 9.5in) - (27ft + 9.5in)(19ft + 5.5in)](15ft + 10.5in) - 30ft^3 - 6ft^3$

$Mason \cdot 85pcf = 129.853\,kip$

$Roofjoist := 2.5in \cdot (27ft + 9.5in)(19ft + 5.5in) + 6 \cdot 0.5ft \cdot 1ft \cdot 19.25ft$ $\quad Roofjoist \cdot 150pcf = 25.562\,kip$

$GatewellRaiseWalls := (9.75ft + 6.25ft + 6ft + 1.8ft + 2ft)(0.75ft \cdot 4.417ft)$

$GatewellRaiseWalls \cdot 150pcf = 12.82\,kip$

$BaseSlab := (29ft + 9.5in)(21ft + 5.5in)$ $\quad BaseSlab = 639.28\,ft^2$

$Baset := 2.0ft$ $\quad BaseSlab \cdot Baset \cdot 150pcf = 191.784\,kips$

$TopSlab1 := 19ft \cdot 18ft \cdot 0.75ft - 21ft^3$ $\quad TopSlab1 \cdot 150pcf = 35.325\,kips$ (Holes taken out)

$GateWalls := [8ft \cdot (22ft + 7in)\,1.5ft]2$ $\quad GateWalls \cdot 150pcf = 81.3\,kips$

$PumpHouse := Mason \cdot 85pcf$ $\quad PumpHouse = 129.853\,kips$ Assumed

$Water := HEAD \cdot BaseSlab$ $\quad Water = 0\,kips$

$Walls := Wall1 + Wall2 + Wall3 + Wall4 + Wall5 + Wall6 + Wall7 + Wall8$

$Structure := (Walls + BaseSlab \cdot Baset + TopSlab1 + GateWalls) \cdot 150pcf + PumpHouse + Roofjoist \cdot 150pcf$ ...
$+ GatewellRaiseWalls \cdot 150pcf$

$Structure = 1152.08\,kips$

$SoilonLedge1 := 0$

$SoilonLedge2 := 0$

$Soil := SoilonLedge1 + SoilonLedge2$ $\quad Soil = 0\,kips$

$Structure + Water + Soil = 1152.08\,kips$

Dissipated Head, if structural foundation in blanket. $HGL = 5.7\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot BaseSlab \cdot \gamma_w \qquad U_1 = 1409.198\ kips$$

$BLANKET = 18\ ft$
$HGL = 5.7\ ft$

$H = 26.83\ ft$

Full Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL + Baset) \cdot (BaseSlab) \cdot \gamma_w\right] \qquad U_2 = 1377.438\ kips$$

$$Uplift := if\left(H > BLANKET, U_2, U_1\right)$$

$Uplift = 1377.438\ kips$

$$Stability := \frac{Structure + Water + Soil}{Uplift}$$

$Stability = 0.836$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $StabilityCheck = "Not\ Okay"$

Equipment weight and other neglected items will increase uplift factor of safety, but probably not high enough to be sufficient.

CALCULATE THE DEPTH OF WATER REQUIRED TO RAISE THE FACTOR OF SAFETY FOR FLOTATION EQUAL TO 1.1

$$A_{pumpit} := (9ft + 2in) \cdot 17ft + (6ft + 10in) \cdot 9ft + \frac{5ft \cdot (5.75ft)}{2} \qquad A_{pumpit} = 231.708\ ft^2$$

$$A_{gatewell} := (9ft + 2in) \cdot (8ft) \qquad A_{gatewell} = 73.333\ ft^2$$

$$A_{other} := (7ft + 9.5in) \cdot (8ft) + (8ft + 8in) \cdot (4ft + 3.5in) + 3.5ft \cdot 8ft \qquad A_{other} = 127.528\ ft^2$$

$$A_{total} := A_{pumpit} + A_{gatewell} + A_{other} \qquad A_{total} = 432.569\ ft^2 \qquad BaseSlab = 639.28\ ft^2$$

$HEAD := 12ft$ $\quad W_{water} := A_{total} \cdot HEAD \cdot \gamma_w \qquad W_{water} = 323.908\ kip$

$Soil = 0$

$$Stability := \frac{Structure + W_{water} + Soil}{Uplift}$$

$Stability = 1.072$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $StabilityCheck = "Not\ Okay"$

12 ft of water required in pumping station to meet Uplift Stability Criteria.

Based on current pump curves, 7.33ft of water is available under flood conditions.

$HEAD := 7.33ft$

$$W_{water} := A_{total} \cdot HEAD \cdot \gamma_w \qquad W_{water} = 197.854\ kip$$

$$Stability := \frac{Structure + W_{water} + Soil}{Uplift}$$

$Stability = 0.98$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $StabilityCheck = "Not\ Okay"$

Factor of Safety of less then 1.0 with 7.33ft in wet well. Structural modifications may be necessary.

BASE SLAB ANALYSIS

$W_{water} = 197.854\ kip$

Slab Strength

SHEAR

Thrust to be used in CASTR for thrust analysis (if necessary)

$THRUST_S := \frac{W \cdot L_1}{2}$ $\quad THRUST_S = 29484\ \frac{lb}{ft}$

$\frac{Structure + W_{water}}{BaseSlab} = 2111.648\ psf$

$W' := if\left(Stability < 1.0, \frac{Uplift}{BaseSlab}, \frac{Structure}{BaseSlab}\right)$ $\quad W' = 2154.672\ psf$

$\frac{Uplift}{BaseSlab} = 2154.672\ psf$

$Slab_t := 24in$ $\quad L_{strip} := 19.83ft$ $\quad Wall_t := 1.75ft$ $\quad Slab_d := 19in$ effective depth of slab

$V'_{ub} := W' \cdot \frac{L_{strip}}{2}$ $\quad V'_{ub} = 21364\ \frac{lb}{ft}$ $\quad W' = 2154.672\ \frac{lb}{ft^2}$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}} \cdot \left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $\quad V_{ub} = 16067\ \frac{lb}{ft}$

$A_g := D_f \cdot 12\ \frac{in}{ft}$ $\quad A_g = 288\ \frac{in^2}{ft}$

$Water = 0$

ANALYZE STRENGTH WITHOUT THE AID OF THRUST

$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S \cdot 0}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f\right)\right]$ ACI EQ (11-4) $\quad \phi V_{nb} = 26291\ \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\quad CheckBase = \text{"OKAY"}$

Factor of Safety $\quad FS_b := \frac{\phi V_{nb}}{V_{ub}}$ $\quad FS_b = 1.636$ $\quad 1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 1.051$

Slab Bending

$L_1 = 19.833\ ft$ $\quad L_2 := 102in$ $\quad \frac{L_2}{L_1} = 0.429$ $\quad$ NOTE THAT SLAB HAS BEAM

REBAR PARALLEL TO LONG WALL

ratio = 0.375

$0.0145 \cdot W' \cdot L_2^2 = 2.257\ kip \cdot \frac{ft}{ft}$

$0.0516 \cdot W' \cdot L_2^2 = 8.033\ kip \cdot \frac{ft}{ft}$

Simple Conservative Analysis	Plate Analysis		Steel Depth
$\frac{W' \cdot (31ft)^2 \cdot 1ft}{8} = 258.83\ kip \cdot ft$	$MidPlateMoment := 1.888 kip \cdot \frac{ft}{ft}$	My	$d_1 := 24in - 2in - 0.5in$
	$EdgePlateMoment := 6.72 kip \cdot \frac{ft}{ft}$	My	$d_2 := 24in - 3in - 0.5in$

#6's @ 12 in. inside face (mid-plate moment)
A-10-3120, TOP REINF

$A_{mpl} := 0.31 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mpl} = 0.405\ in$

$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_1 - \frac{a_{mpl}}{2}\right)$ $\quad \phi M_{mpl} = 22.0 \frac{kip \cdot ft}{ft}$

#5's @ 12" Outside Face (edge-plate moment)
A-10-3120, BOTTOM REINF

$A_{epl} := 0.31 \frac{in^2}{ft}$ $\quad a_{epl} := \frac{A_{epl} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{epl} = 0.405\ in$

$\phi M_{epl} := \phi_B \cdot A_{epl} \cdot F_y \cdot \left(d_2 - \frac{a_{epl}}{2}\right)$ $\quad \phi M_{epl} = 21.0 \frac{kip \cdot ft}{ft}$

$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot MidPlateMoment, "OKAY", "NO\ GOOD"\right)$ $\quad CheckMP = "OKAY"$

Factor of Safety $\quad FS_{MPL} := \frac{\phi M_{mpl}}{MidPlateMoment}$ $\quad FS_{MPL} = 11.656$

$CheckEPL := if\left(\phi M_{epl} > 1.5 \cdot EdgePlateMoment, "OKAY", "NO\ GOOD"\right)$ $\quad CheckEPL = "OKAY"$

Factor of Safety $\quad FS_{EPL} := \frac{\phi M_{epl}}{EdgePlateMoment}$ $\quad FS_{EPL} = 3.121$

BASE SLAB REBAR PARALLEL TO THE SHORT SIDE

$L_1 = 19.833\ ft$ $L_2 = 8.5\ ft$ $\frac{L_2}{L_1} = 0.429$ NOTE THAT SLAB HAS BEAM

REBAR PARALLEL TO SHORT WALL

ratio = 0.375

$0.0424 \cdot W' \cdot L_2^2 = 6.601\ kip \cdot \frac{ft}{ft}$

$0.0830 \cdot W' \cdot L_2^2 = 12.921\ kip \cdot \frac{ft}{ft}$

Simple Conservative Analysis

$\frac{W' \cdot (31ft)^2 \cdot 1ft}{8} = 258.83\ kip \cdot ft$

Plate Analysis

$MidPlateMoment2 := 5.5kip \cdot \frac{ft}{ft}$ My

$EdgePlateMoment2 := 10.8kip \cdot \frac{ft}{ft}$ My

Steel Depth

$d_1 := 24in - 2in - 0.5in$

$d_2 := 24in - 3in - 0.5in$

#6's @ 12 in. inside face (mid-plate moment)

A-10-3120, TOP REINF

$A_{mps} := 0.44 \frac{in^2}{ft}$ $b := 12 \frac{in}{ft}$ $a_{mps} := \frac{A_{mps} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mps} = 0.575\ in$

$\phi M_{mps} := \phi_B \cdot A_{mps} \cdot F_y \cdot \left(d_1 - \frac{a_{mps}}{2}\right)$ $\phi M_{mps} = 31.1 \frac{kip \cdot ft}{ft}$

#6's @ 12" Outside Face (edge-plate moment)

A-10-3120, BOTTOM REINF

$A_{eps} := 0.44 \frac{in^2}{ft}$ $a_{eps} := \frac{A_{eps} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{eps} = 0.575\ in$

$\phi M_{eps} := \phi_B \cdot A_{eps} \cdot F_y \cdot \left(d_2 - \frac{a_{eps}}{2}\right)$ $\phi M_{eps} = 29.6 \frac{kip \cdot ft}{ft}$

$CheckMP := if\left(\phi M_{mps} > 1.5 \cdot MidPlateMoment2, "OKAY", "NO\ GOOD"\right)$ CheckMP = "OKAY"

Factor of Safety $FS_{MPS} := \frac{\phi M_{mps}}{MidPlateMoment2}$ $FS_{MPS} = 5.657$

$CheckEPS := if\left(\phi M_{eps} > 1.5 \cdot EdgePlateMoment2, "OKAY", "NO\ GOOD"\right)$ CheckEPS = "OKAY"

Factor of Safety $FS_{EPS} := \frac{\phi M_{eps}}{EdgePlateMoment2}$ $FS_{EPS} = 2.745$

Controlling Factor of Safety

$$FS_B := \min\left(FS_b, FS_{EPL}, FS_{EPS}, FS_{MPL}, \cancel{FS_{EPL}}\right)$$

$FS_B = 1.636$

Controlling Mechanism

$$CommentB := \text{if}\left(FS_B < FS_b, \text{"Base Flexural Steel"}, \text{"Base Shear"}\right)$$

CommentB = "Base Shear"

SUMMARY OF FACTORS OF SAFETY

Wall 1: $FS_1 = 1.598$

Comment1 = "Wall 1 Flexural Steel"

Wall 2: $FS_2 = 1.279$

Comment2 = "Wall 2 Flexural Steel"

Base Slab: $FS_B = 1.636$

CommentB = "Base Shear"

Uplift: Stability = 0.98 HEAD = 7.33 ft

StabilityCheck = "Not Okay"

The uplift stability is 0.98 when the water in the pump station has a depth of 7.33 ft.
The uplift stability is 1.072 when the water in the pump station has a depth of 12 ft.
The uplift stability is 0.84 when the water in the pump station has a depth of 0 ft.

END OF ANALYSIS

Peer Reviewed By: Paul Muller 4/10/07
Note from reviewer: Marvin Parks and I had several discussions regarding bedding assumptions and came to the conclusion that the assumed values were appropriate. Also, PIPECAR was independently run and the values below were matched.

Computed By: Marvin Parks Date: 10-Apr-07
Checked By:

OSAGE OUTLET WORKS EVALUATION

The Osage sewer line was evaluated for strength when exposed to existing condition and n500+3 event loading as part of the KC Levees Feasibility Study. The calculated pipe D-loads for the specified events are provided in Table 1.

Pipe loading or demand is primarily controlled by the pipe bedding factor. Given that the Osage sewer pipe is a modified circular section and installation method shown in O&MM manual, a 1st Class bedding condition was assumed (embankment condtion). This results in a bedding factor, Bf = 2.8. When concrete pipe has been installed underground, the soil-structure system will continually show an increase in load capacity. Data gathered by the American Concrete Pipe Association (ACPA) has indicated an increase in concrete strength and an increase in load carrying capacity of 10 to 40 percent. Settlement and consolation will improve the soil structure surrounding the pipe, which also improves load carrying capacity (ACPA, Design Data 1).

For the n500+3 event loading and assuming the pipe is full of water, the design pipe demand is 3484 #/ft/ft. The pipe demand drops to 2680 #/ft/ft if the EM 1110-2-2104 hydraulic factor = 1.3 is waived. For the existing condition loading and assuming the pipe full of water, the design pipe demand is 3006 #/ft/ft. If the hydraulic load factor 1.3 is waived, the demand becomes 2312 #/ft/ft.

The design capacity of the pipe was calculated using PIPECAR software. PIPECAR determined the D-load capacity based on the inside reinforcement area, pipe diameter, wall thickness, steel reinforcement yield and ultimate tensile strength values provided by the user.

The PIPECAR calculated value of D-load capacity based on a 0.01 inch crack is 2502 #/ft/ft. This value was generated using the input values shown in Table 2 for a steel reinforcement yield strength equal to 40 ksi and concrete compressive strength of 3 ksi.

The concrete pipe section reinforcement area was obtained from the O&MM drawings. The concrete and reinforcement strength values were not specified in the drawing set. The values of strength were chosen based on the installation date (1950s, fy=40 ksi, f`c = 3 ksi). These values of strength for steel and concrete were common during this era of construction.

The sewer pipe was inspected in February 2007 by EC-DS personnel. The pipe did not show signs of distress or out of roundness. No longifudinal cracks were found. There were two isolated areas where longitudinal (temperature) reinforcement was exposed. The exposed areas were in the pipe invert were ongoing erosion is present. No steps in the concrete was observed during the inspection.

Based on the analysis conducted, field investigation and available information, the Osage pipe is BORDERLINE for the existing condition loading. The pipe design capacity is less than the pipe demand loading for existing conditions when the hydraulic factor = 1.0. To meet the design requirements of EM 1110-2-2902 for n500+3 event loading which requires the hydraulic factor = 1.3, it is recommended that the pipe be rehabilitated using either a steel slip liner or a CIPP sleeve. Prior to rehabilitation, the areas where reinforcement is exposed should be repaired.

Table 1. OSAGE OUTLET WORKS DESIGN PIPE DEMAND LOAD, INDIRECT METHOD, EMBANKMENT CONDTIONS

					Project:	Osage Sewer Line	
Si =	6	ft	**Water?**	1	**Date:**	4/9/2007	
bc =	8	ft			PIPECAR Capacity Values:		
Soil =	120	pcf			3EB =	2502	# / ft / ft
					Ultimate=	3261	# / ft / ft

Event	H_f	H_c, ft	B_f	Instal. Type	VAF	PL	VAF*PL	W_F	W_L	W_T	$D_{0.01}$	COMMENT
n500+3	1.0	31.33	2.8	2	1.4	30902	43263	1763	0	45027	2680	Hf = 1.0
n500+3	1.3	31.33	2.8	2	1.4	30902	43263	1763	0	45027	3484	Hf = 1.3
Exist Cond	1.0	26.73	2.8	2	1.4	26486	37081	1763	0	38844	2312	Hf=1.0
Exist Cond	1.3	26.73	2.8	2	1.4	26486	37081	1763	0	38844	3006	Hf=1.3
	1.3	20.5	2.8	2	1.4	20506	28708	1763	0	30471	2358	max Hc=20.5 for design

EM 1110-2-2902 Reference

Table 3-2
Bedding Factor Constants: Embankment

Projection Ratio p	Concrete Bedding X_a	Other Projection Bedding X_a
0	0.15	0
0.3	0.743	0.217
0.5	0.856	0.423
0.7	0.811	0.594
0.9	0.678	0.655
1.0	0.638	0.638
Type of Bedding X_p		
Ordinary		0.840
First Class		0.707
Concrete Cradle		0.505

EM Bedding factor determination		
Xa =	0.594	
Xp =	0.707	
Bf =	2.8	

ACPA

Vertical Arching Factor (VAF)

Type 1	VAF = 1.35
Type 2	VAF = 1.40
Type 3	VAF = 1.40
Type 4	VAF = 1.45

ACPA Reference

Illustration 4.19 Bedding Factors, Embankment Conditions, B_{fe}

Pipe Diameter	Standard Installation Type 1	Type 2	Type 3	Type 4
12 in.	4.4	3.2	2.5	1.7
24 in.	4.2	3.0	2.4	1.7
36 in.	4.0	2.9	2.3	1.7
72 in.	3.8	2.8	2.2	1.7
144 in.	3.6	2.8	2.2	1.7

Notes:

1. *For pipe diameters other than listed in Illustration 4.19, embankment condition factors, B_{fe} can be obtained by interpolation.*
2. *Bedding factors are based on the soils being placed with the minimum compaction specified in Illustrations 4.5 and 4.7 for each standard installation.*

Table 2. Osage Sewer Pipe Capacity Values, PIPECAR Software for 3-Edge Bearing.

Fu (ksi)	Fy (ksi)	f`c (ksi)	$D_{0.01}$ #/ft/ft	D_{ult} #/ft/ft	Reinf Type	Design Criteria	Comment
40	40	3.0	2502	3261	3D	Yield	Mean value of steel and concrete strength.
40	40	2.40	2283	3261	3D	Yield	
40	40	3.75	2612	3261	3D	Yield	
32	32	2.40	2007	2526	3D	Yield	
32	32	3.0	2007	2526	3D	Yield	
32	32	3.75	2007	2526	3D	Yield	
50	50	2.40	2283	4181	3D	Yield	
50	50	3.0	2502	4181	3D	Yield	
50	50	3.75	2748	4181	3D	Yield	

Notes:
1. Values obtained using ACPA PIPECAR software for 3 edge bearing capacity (indirect design) for D-load capacity for a 0.01 inch crack and the D-load at which collapse occurs. The program requires Fu, Fy, f'c, wall thickness, diameter and inside reinforcement area input values.
2. Reinforcement area = 0.72 in^2 /ft used to calculate the D-load capacity.
3. Values of pipe capacity for other values of steel reinforcement and concrete strength provided for risk and

APPENDIX I

RECORD DRAWINGS

VOLUME ONE

EXCERPTS FROM OPERATIONS AND MAINTENANCE MANUAL WHOSE TITLE PAGE IS SHOWN HERE. THE EXCERPTS ARE OF THE OSAGE PUMP STATION OUTLET WORKS.

OPERATION AND MAINTENANCE MANUAL

KANSAS CITYS FLOOD CONTROL PROJECT

Missouri and Kansas Rivers

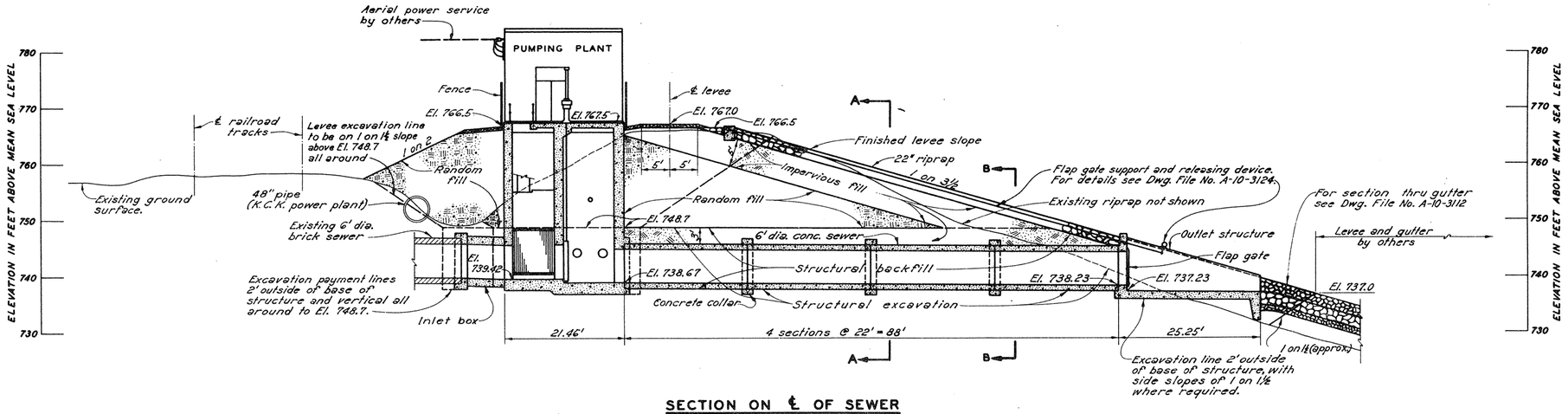
ARMOURDALE UNIT

FILE COPY
EST. & SPEC. SEC.

DEPARTMENT OF THE ARMY

KANSAS CITY DISTRICT, CORPS OF ENGINEERS

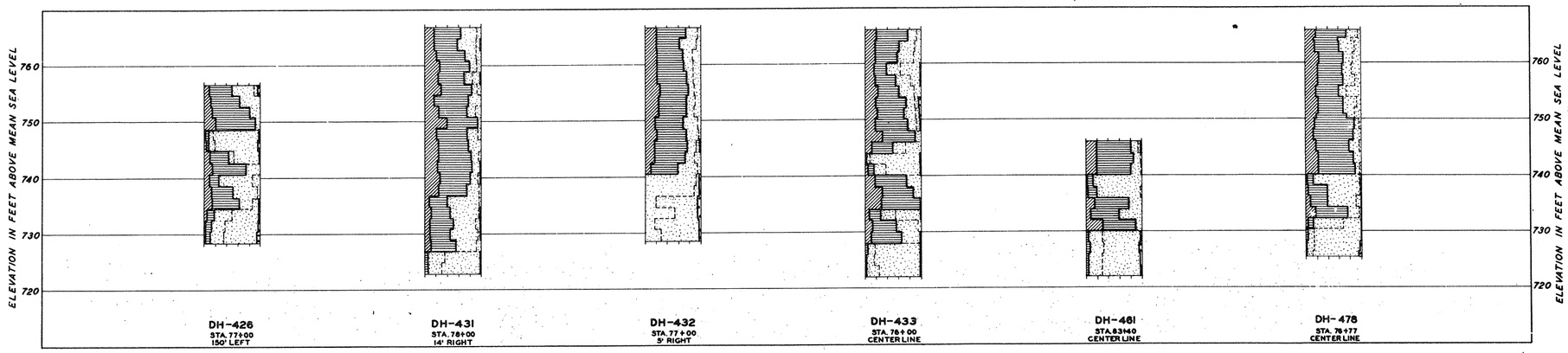
SCALE AS SHOWN
PER DRAWING

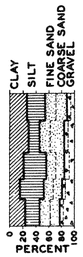


SECTION ON ℄ OF SEWER

Scale: 1" = 10'

Notes:
For Sections A-A and B-B see Drawing File No. A-10-3112.





LEGEND FOR UNDERGROUND EXPLORATIONS

Percentages of clay, silt, sand and gravel (Bureau of Soils Classification) are shown by the symbols indicated at left. Percentages of sample passing No. 4 & No. 50 sieves indicated by broken lines.

Drive hole and number _____ DH-426

RECORD DRAWING

AUGUST 1954

CONTRACT NO. DA-23-028-ENG-1540

KANSAS CITYS FLOOD CONTROL PROJECT

ARMOURDALE UNIT

OSAGE AVENUE PUMPING PLANT
SECTION THRU SEWER
AND UNDERGROUND EXPLORATIONS

In 14 sheets | Sheet No. 3 | Scale: as shown

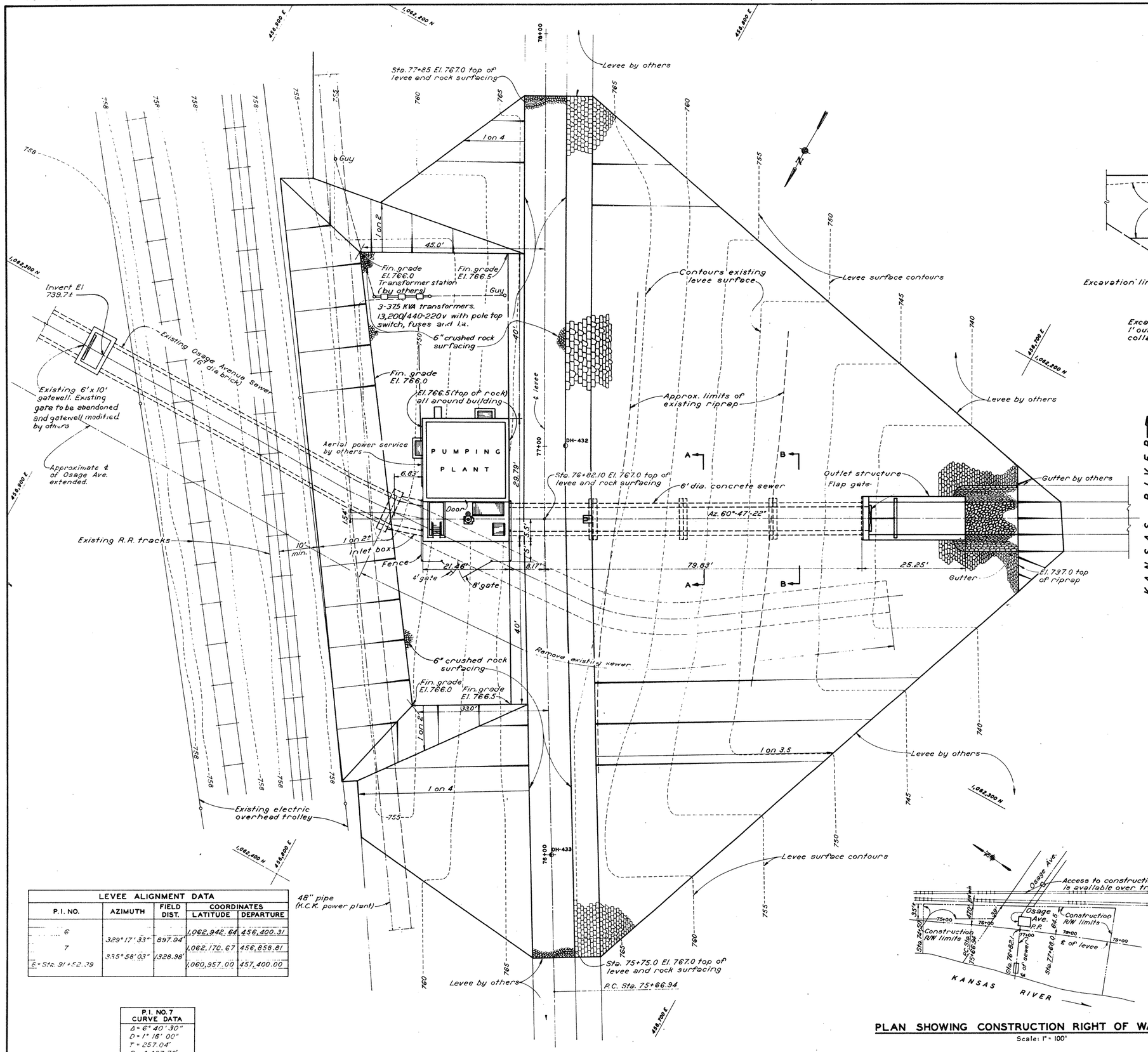
OFFICE, DISTRICT ENGINEER
KANSAS CITY DISTRICT

KANSAS CITY, MO.
APRIL 1952

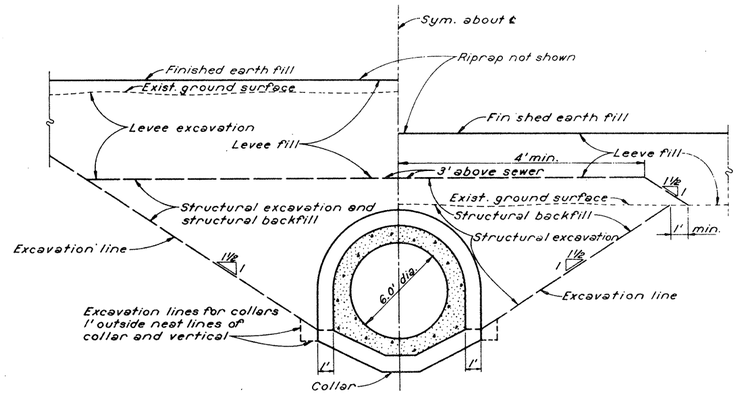
Submitted: R. P. Gilkey, Chief, Struct. Design
Recommended: Louis Gaal, Chief, Engineering Division
Approved: L. J. Lincoln, Col., C. of E., District Engineer

Drawn by: E.T.L. | Traced by: C.W.S. | Checked by: E.B.C. | To accompany specifications dated: SEPTEMBER 1952 | File No. A-10-3113

O & M.M. PLATE NO. 107

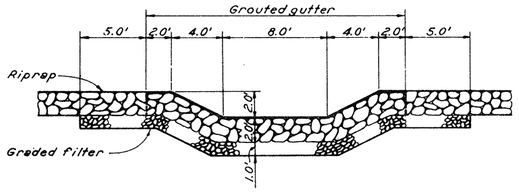


PLAN
Scale: 1" = 10'



SECTION A-A SECTION B-B

SHOWING EXCAVATION AND BACKFILL LIMITS
No Scale

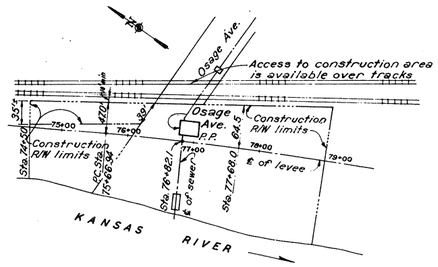


SECTION THRU GUTTER
No Scale

NOTES:
Levee and gutter indicated "by others" were completed subsequent to the completion of work under this contract.
Fin. grade indicates top of crushed rock surfacing.
For test hole data, see Dwg. File No. A-10-3113.
For Section on center line of sewer, see Dwg. File No. A-10-3113.
Bench Mark; Left bank of Kansas River at West Kansas Avenue Bridge. A chiseled square in northeast corner of bridge abutment El. 766.46

LEVEE ALIGNMENT DATA				
P.I. NO.	AZIMUTH	FIELD DIST.	COORDINATES LATITUDE	COORDINATES DEPARTURE
6			1,062,942.64	456,400.31
	329°17'33"	897.94'		
7			1,062,170.67	456,858.81
	335°58'03"	1,328.98'		
R = Sta. 91+52.39			1,060,957.00	457,400.00

P.I. NO. 7 CURVE DATA
Δ = 6° 40' 30"
D = 1° 16' 00"
T = 257.04'
R = 4,407.76'
L = 512.51'
P.C. = Sta. 75+66.94
P.T. = Sta. 80+79.45



PLAN SHOWING CONSTRUCTION RIGHT OF WAY LIMITS
Scale: 1" = 100'

RECORD DRAWING

AUGUST 1954

CONTRACT NO. DA-23-028-ENG-1540

NO.	DATE	BY	REVISIONS	APP'D.
1	8-15-54	E.B.C.	Minor revisions for "as built" conditions.	R.L.G.

KANSAS CITYS FLOOD CONTROL PROJECT

ARMOURDALE UNIT

OSAGE AVENUE PUMPING PLANT
STATION 76 + 82
GENERAL PLAN

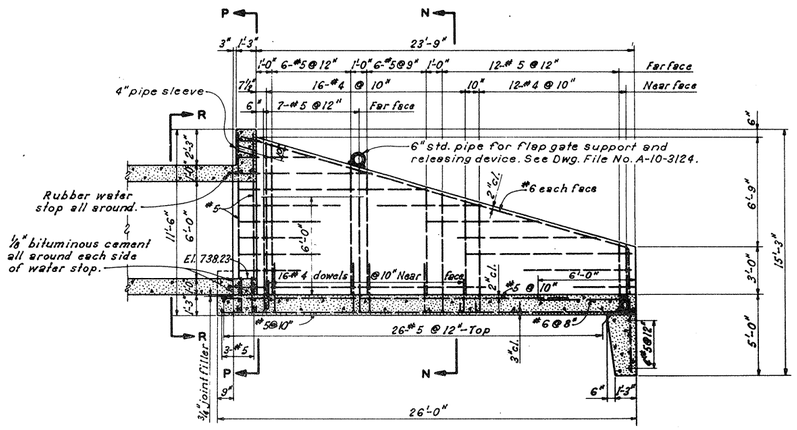
In 14 sheets Sheet No. 2 Scale: as shown

OFFICE, DISTRICT ENGINEER
KANSAS CITY DISTRICT
KANSAS CITY, MO.
APRIL 1952

Submitted: R. L. Gillis — Chief, Struct. Design
Recommended: — Chief, Engineering Division
Approved: L. J. Lincoln — Col., C. of E., District Engineer

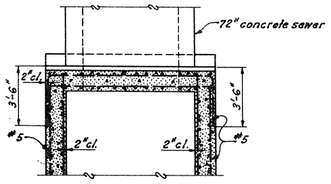
Drawn by: E.T.L. Traced by: G.R.P. Checked by: E.B.C. To accompany specifications dated: SEPTEMBER 1952 File No. A-10-3112

O & M.M. PLATE NO. 106

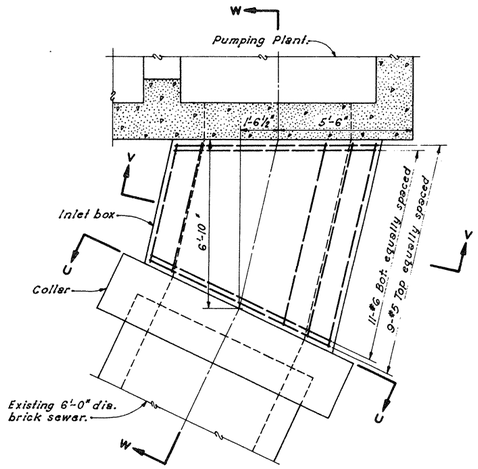


SECTION ON ℄

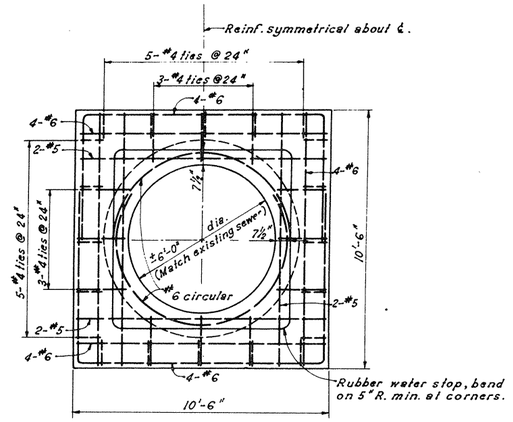
Note:
Longitudinal reinforcement for wingwalls #5 @ 12" each face.
Structure is symmetrical about ℄.



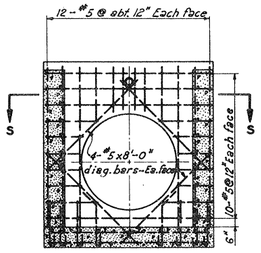
SECTION S-S



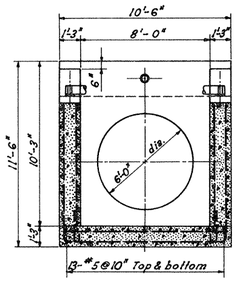
PLAN
SHOWING REINFORCEMENT IN TOP



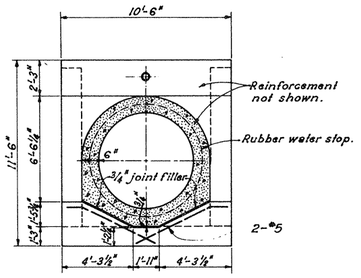
ELEVATION U-U



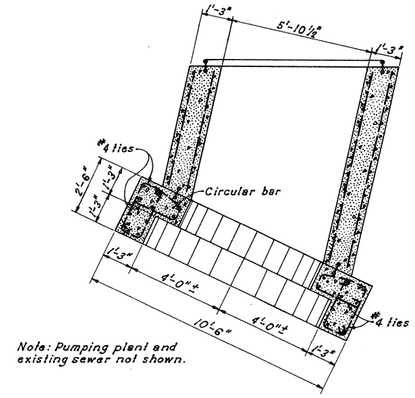
SECTION P-P



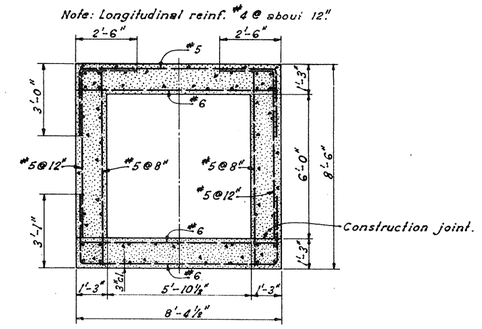
SECTION N-N



SECTION R-R



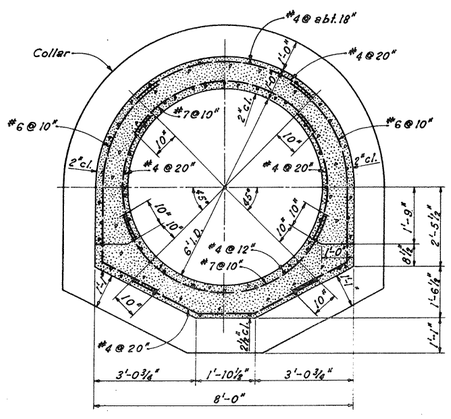
SECTION T-T



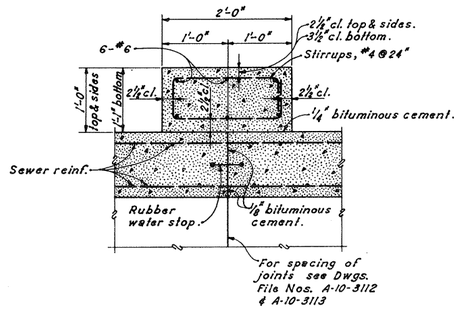
SECTION V-V

Note: Flap gate not shown.

OUTLET STRUCTURE
Scale: 1/4" = 1'-0"

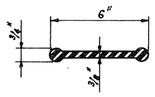


TYPICAL SECTION
Scale: 1/2" = 1'-0"

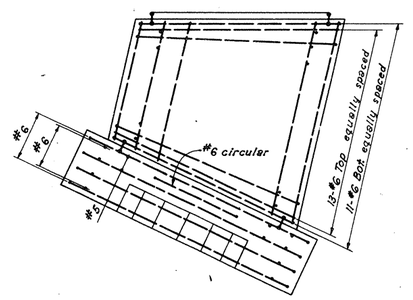


TYPICAL JOINT AND COLLAR
Scale: 1" = 1'-0"

DETAILS OF 6' DIA. SEWER



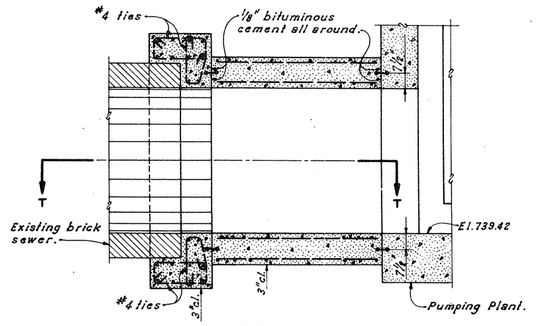
RUBBER WATER STOP DETAIL
Scale: 3" = 1'-0"



PLAN OF BASE

Notes:
Clear distance of reinforcement from surface of concrete shall be 2 1/2" unless otherwise shown.
All exposed concrete edges shall be chamfered 3/4".

INLET BOX
Scale: 3/8" = 1'-0"



SECTION W-W

RECORD DRAWING

AUGUST 1954
CONTRACT NO. DA-23-028-ENG-1540

NO.	DATE	BY	REVISIONS	APP'D
1	8-15-54	E.B.C.	Minor revisions for "as built" conditions.	R.L.G.

KANSAS CITY'S FLOOD CONTROL PROJECT

ARMOURDALE UNIT

OSAGE AVENUE PUMPING PLANT
STRUCTURAL

In 14 sheets — Sheet No. 12 — Scale: as shown

OFFICE, DISTRICT ENGINEER
KANSAS CITY DISTRICT
KANSAS CITY, MO.
APRIL 1952

Submitted: R. L. Gillis, Chief, Struct. Design
Recommended: Louis J. Ceol, Chief, Engineering Division
Approved: L. J. Lincoln, Col., Corps of E. District Engineer

Drawn by: J.L.N. Traced by: W.W.B. Checked by: E.B.C. To accompany specifications dated: SEPTEMBER 1952 File No. A-10-3122

Kansas Citys Phase II, Armourdale District
Shawnee Pump Station Analysis, Sta 230+78

I. Overview/Introduction

PRESENTATION OF THE ANALYSIS ASSUMPTIONS, CRITERIA AND RECOMMENDATIONS ARE LOCATED IN THE DESIGN ANALYSIS REPORT (DAR)

THE FOLLOWING CHECKS WERE ACCOMPLISHED ON THE SHAWNEE PUMPING STATION:
1. WEST WALL AT ELEVATION 734.25, MOMENT AND SHEAR
2. BASE SLAB MOMENT AND SHEAR
3. CROSS STRUT CAPACITY
4. FLOTATION STABILITY

ANALYSIS INDICATES THAT THE FS FOR SHEAR ON THE SOUTH SIDE OF THE EAST AND WEST WALLS AT ELEVATION 742 FEET OR BELOW IS LESS THAN 1.5 FOR THE n500+3 CONDITION

II. References

1. EC-GD email dated May 15, 2006, geotech parameters
2. Record Drawings, full-size, dated 6-25-75
3. Site Photographs dated July 2005

III. Analysis

PARAMETERS FOR ANALYSIS

$HGL_1 := 765.5ft$ n500+3 YEAR EVENT HYDRAULIC GRADE LINE ELEVATION — EC-GD

$ELEV_{east} := 766ft$ GROUND SURFACE ELEVATION AT THE EAST WALL

$ELEV_{west} := 763ft$ GROUND SURFACE ELEVATION ON THE WEST WALL

$ELEV_2 := 716ft$ TOP OF PUMPING STATION BASE SLAB ELEVATION

$TOB := 754ft$ TOP OF BLANKET ELEVATION PROVIDED BY EC-GD. ASSUME FOR SOIL LOADING ANALYSIS THAT THE TOP OF BLANKET ELEVATION IS AT THE GROUND SURFACE — EC-GD

$BOB := 729ft$ BOTTOM OF BLANKET ELEVATION — EC-GD

$\phi := 26deg$ SOIL FRICTION ANGLE — EC-GD

$\gamma := 125pcf$ SATURATED SOIL UNIT WEIGHT — EC-GD

$\gamma_w := 62.4pcf$ UNIT WEIGHT OF WATER

$HEAD := 0ft$ DEPTH OF WATER ON INTERIOR SIDE OF WALL

$K_o := 1 - \sin(\phi)$ $K_o = 0.562$

$Blanket_1 := TOB - BOB$ $Blanket_1 = 25.00\ ft$ — EC-GD

$c2 := 2in$ WALL CLEAR COVER

$c3 := 3in$ BASE SLAB CLEAR COVER

1. Loading

- **West Wall Loading Due to Soil Alone**

$ELEV := 734.25ft$ ELEVATION OF THE WALL SECTION TO BE CHECKED

$Depth := ELEV_{west} - ELEV$ $\quad Depth = 28.75\ ft$ $\quad K_o = 0.56$

$W_1 := K_o \times \gamma \times Depth$ $\quad W_1 = 2018.35\ psf$

- **West Wall Loading Due to Water and Soil**

$HGL := HGL_1 - ELEV_{west}$ $\quad HGL = 2.50\ ft$

$Blanket := ELEV_{west} - BOB$ $\quad Blanket = 34.00\ ft$ $\quad Depth = 28.75\ ft$

$$H'_3 := \frac{Blanket + HGL}{Blanket} \times Depth - HEAD \qquad H'_3 = 30.86\ ft$$

$$H''_3 := Depth + HGL - HEAD \qquad H''_3 = 31.25\ ft$$

$$H_3 := if\left(Depth > Blanket, H''_3, H'_3\right) \qquad H_3 = 30.86\ ft$$

THE BASE OF STRUCTURE IS BELOW THE BLANKET BASE ELEVATION

$$W_2 := K_o \times \left(\gamma - \gamma_w\right) Depth + \gamma_w \times H_3 \qquad W_2 = 2936.70\ psf \qquad W_1 = 2018.35\ psf$$

$$W := \max\left(W_1, W_2\right) \qquad W = 2936.70\ psf$$

EVALUATE A ONE FOOT STRIP OF WALL SECTION LOCATED AT THE CENTERLINE OF THE CROSS STRUT LOCATED AT ELEVATION 734.25. USE ENERCAL PROGRAM TO EVALUATE.
BASED ON THE RAM 3D MODEL, THE MAXIMUM MOMENT IN THE HORIZONTAL REINFORCEMENT OCCURS AT ELEVATIONS NEAR 734.

NOTE THAT IF Ko = 1.0, THEN THE PRESSURE AT ELEVATION 734.25 BECOMES 3.73 ksf

2. Base Slab

REFER TO THE RAM ANALYSIS FOR THE MOMENT VALUES

MOMENT ABOUT THE LOCAL "3" AXIS, BOTTOM STEEL

$$M_{u33b} := 52 \frac{kip \times ft}{ft}$$ MAX. MOMENT IN BOTTOM LAYER

$f_y := 60ksi$ $\quad f'_c := 4ksi$ $\quad d := 24in - 3.5in$

NO.9 AT 10" $\quad A_{s1b} := 1.20 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a := \frac{A_{s1b} \times f_y}{0.85 f'_c \times b}$ $\quad a = 1.76\ in$

$M_{n1b} := A_{s1b} \times f_y \times \left(d - \frac{a}{2}\right)$ $\quad M_{n1b} = 117.7 \frac{1}{ft} kip \times ft$

$CheckM1b := if\left(M_{n1b} > 1.5 \times M_{u33b}, "OKAY", "NO GOOD"\right)$ $\quad CheckM1b = "OKAY"$

Factor of Safety $\quad FS_{M1b} := \frac{M_{n1b}}{M_{u33b}}$ $\quad FS_{M1b} = 2.26$

MOMENT ABOUT THE LOCAL "3" AXIS, TOP STEEL

$M_{u33t} := 32 \frac{kip \times ft}{ft}$ MAX. MOMENT IN BOTTOM LAYER

$f_y := 60ksi$ $\quad f'_c := 4ksi$ $\quad d := 24in - 3.5in$

NO.9 AT 10" $\quad A_{s1t} := 1.09 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a := \frac{A_{s1t} \times f_y}{0.85 f'_c \times b}$ $\quad a = 1.60 in$

$M_{n1t} := A_{s1t} \times f_y \times \left(d - \frac{a}{2}\right)$ $\quad M_{n1t} = 107.4 \frac{1}{ft} kip \times ft$

$CheckM1t := if\left(M_{n1t} > 1.5 \times M_{u33t}, "OKAY", "NO GOOD"\right)$ $\quad CheckM1b = "OKAY"$

Factor of Safety $\quad FS_{M1t} := \frac{M_{n1t}}{M_{u33t}}$ $\quad FS_{M1t} = 3.35$

MOMENT ABOUT THE LOCAL "1" AXIS, BOTTOM STEEL

$M_{u11b} := 24 \frac{kip \times ft}{ft}$ MAX. MOMENT IN BOTTOM LAYER

$f_y := 60ksi$ $\quad f'_c := 4ksi$ $\quad d := 24in - 3.5in$

NO.9 AT 10" $\quad A_{s3b} := 0.60 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a := \frac{A_{s3b} \times f_y}{0.85 f'_c \times b}$ $\quad a = 0.88 in$

$M_{n3b} := A_{s3b} \times f_y \times \left(d - \frac{a}{2}\right)$ $\quad M_{n3b} = 60.2 \frac{1}{ft} kip \times ft$

$CheckM3b := if\left(M_{n3b} > 1.5 \times M_{u11b}, "OKAY", "NO GOOD"\right)$ $\quad CheckM3b = "OKAY"$

Factor of Safety $\quad FS_{M3b} := \frac{M_{n3b}}{M_{u11b}}$ $\quad FS_{M3b} = 2.51$

MOMENT ABOUT THE LOCAL "1" AXIS, TOP STEEL

$M_{u11t} := 41.75 \frac{kip \times ft}{ft}$ MAX. MOMENT IN BOTTOM LAYER

$f_y := 60ksi$ $f'_c := 4ksi$ $d := 24in - 3.5in$ $d = 20.50\ in$

NO.9 AT 10" $A_{s3t} := 0.60 \frac{in^2}{ft}$ $b := 12 \frac{in}{ft}$ $a := \frac{A_{s3t} \times f_y}{0.85 f'_c \times b}$ $a = 0.88\ in$

$M_{n3t} := A_{s3t} \times f_y \times \left(d - \frac{a}{2}\right)$ $M_{n3t} = 60.2 \frac{1}{ft} kip \times ft$

$CheckM3t := if\left(M_{n3t} > 1.5 \times M_{u11t}, "OKAY", "NO GOOD"\right)$ $CheckM3t = "NO\ GOOD"$

Factor of Safety $FS_{M3t} := \frac{M_{n3t}}{M_{u11t}}$ $FS_{M3t} = 1.44$

2. Stability

- **Input Parameters**

$W_s := 2808kip$ VALUE FOR RAM ADVANSE "SHAWNEE_REV4.AVW"

VOLUME OF SOIL OVER TOP OF STRUCTURE

SLAB

$A_{slab} := 7.5ft \times 46.5ft$ $A_{slab} = 348.75\ ft^2$

$D_{slab} := 766ft - 744.5ft$ $D_{slab} = 21.50\ ft$

$V_1 := A_{slab} \times D_{slab}$ $V_1 = 7498.13\ ft^3$

HEEL

$A_{heel} := (49.5ft \times 2 + 27ft \times 2) \times 1.5ft$ $A_{heel} = 229.50\ ft^2$

$D_{heel} := 764ft - 716ft$ $D_{heel} = 48.00\ ft$

$V_2 := A_{heel} \times D_{heel}$ $V_2 = 11016.00\ ft^3$

$V := V_1 + V_2$ $V = 18514.13\ ft^3$

UPLIFT

THE STRUCTURE'S BASE ELEVATION IS BELOW THE BOTTOM OF BLANKET ELEVATION. THEREFORE THE BLANKET THICKNESS DOES NOT IMPACT THE UPLIFT FORCE ACTING ON THE STRUCTURE.

$Head := HGL_1 - 714ft$ $HGL_1 = 765.50\ ft$

$Head = 51.50\ ft$

$Pressure := Head \times \gamma_w$ $Pressure = 3213.60\ psf$

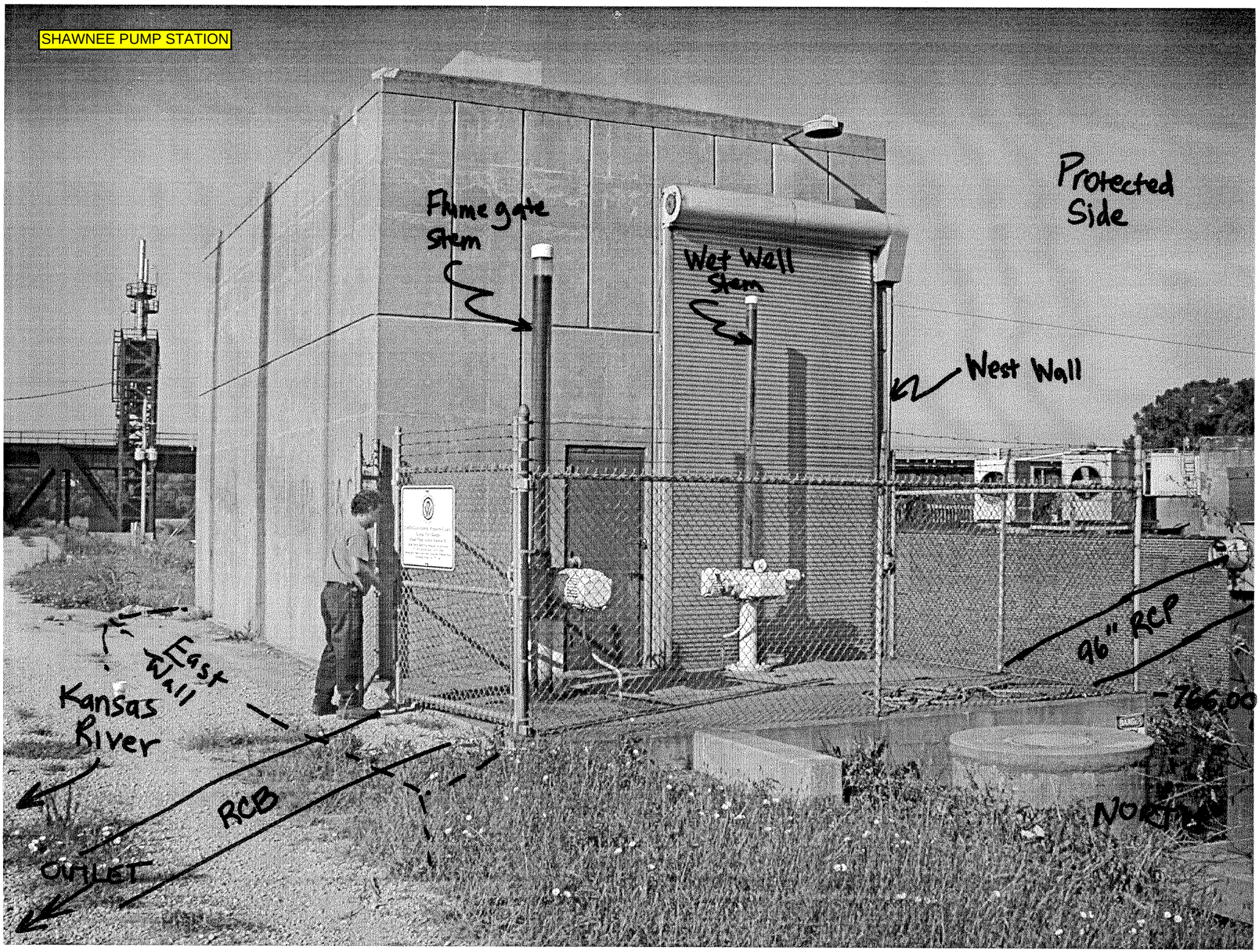
$Area := 1500ft^2$

$U := Pressure \times Area$ $U = 4820.40\ kip$

- **Factor of Safey for Flotation**

$$SF := \frac{W_s + V \times (\gamma - \gamma_w)}{U - V \times \gamma_w} \qquad SF = 1.08$$

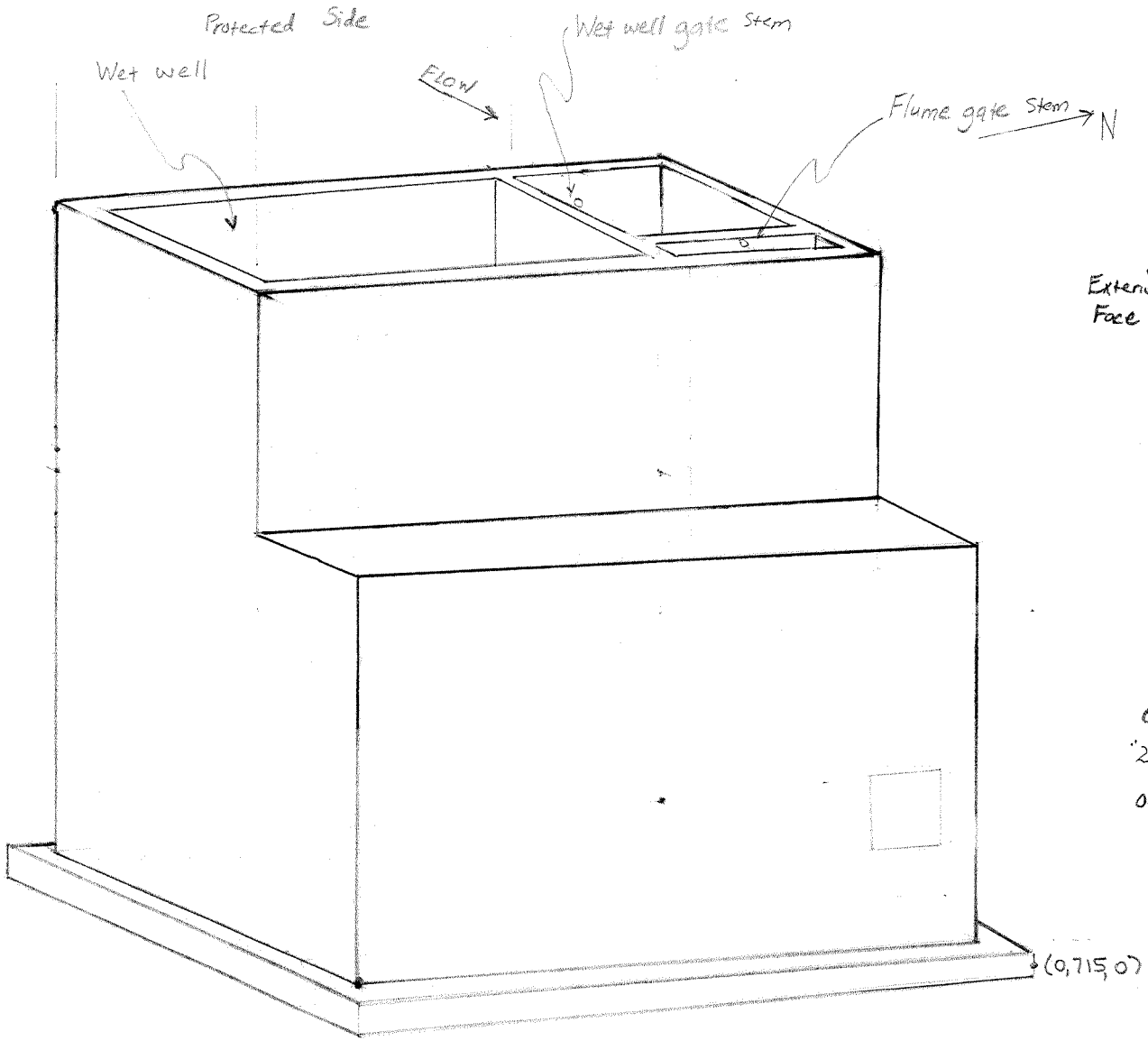
END OF ANALYSIS

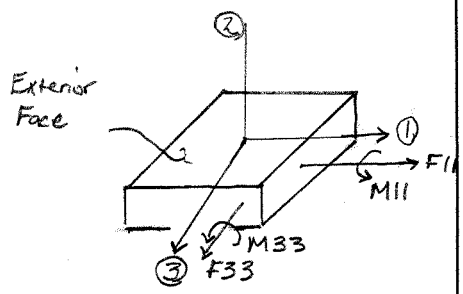

SHAWNEE PUMP STATION
Flume gate Stem
Wet Well Stem
Protected Side
West Wall
96" RCP
-766.00
East Wall
Kansas River
RCB
OUTLET
NORTH

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By MLP Date 5/25/06 Project KCL Sheet 6 of

Chkd. By _____ Date _____ Subject Shawnee Sheet _____ of _____





+ Sign Convention For Local Axis

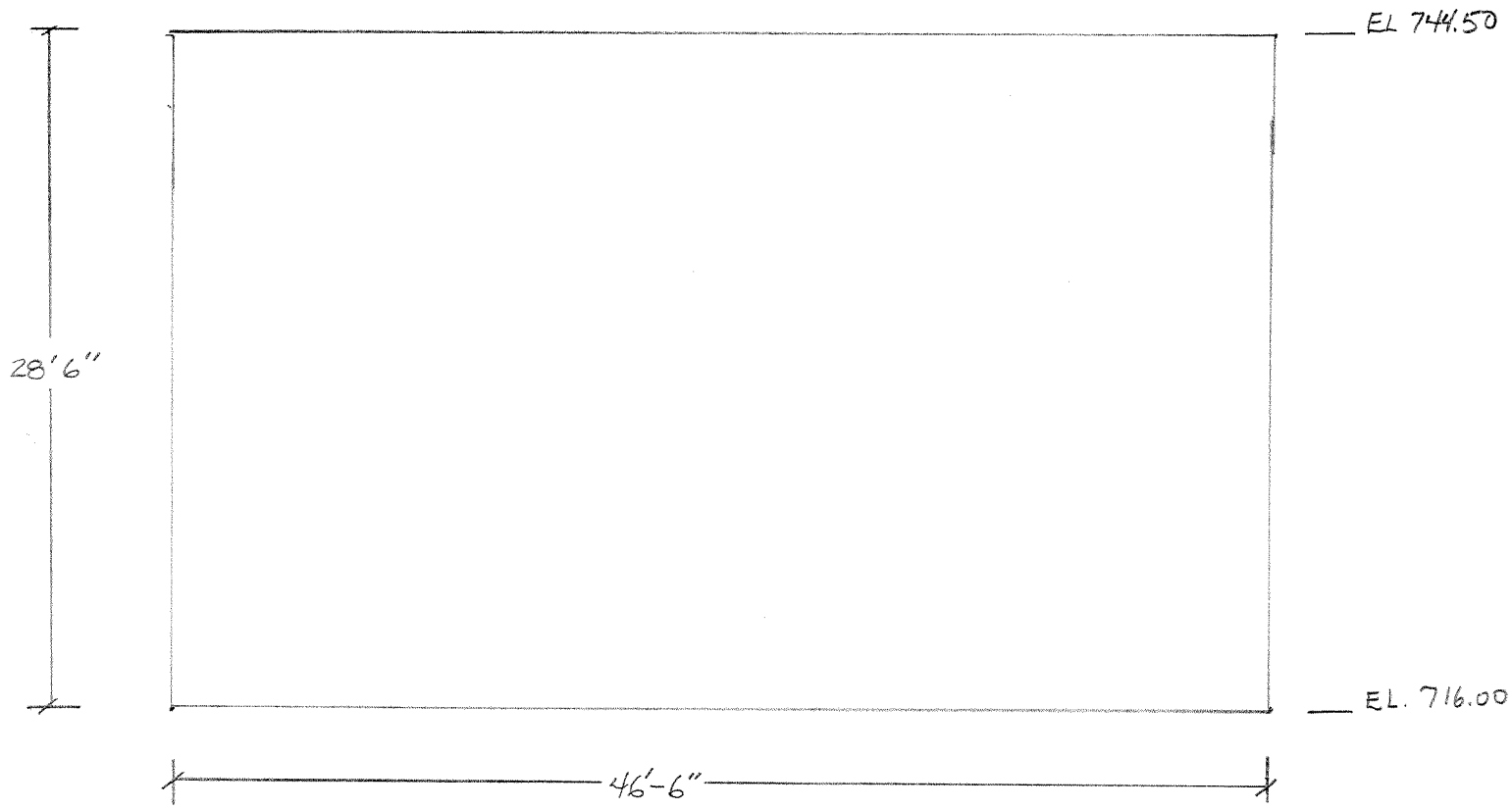
For this model, all exterior plates has the "2" local axis pointing outward

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By MP Date 5/31/06 Project KCL Sheet 7 of

Chkd. By Date Subject Shawnee Sheet of

Wall thickness: 1'6"



East Wall Scale 1/8" = 1'

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

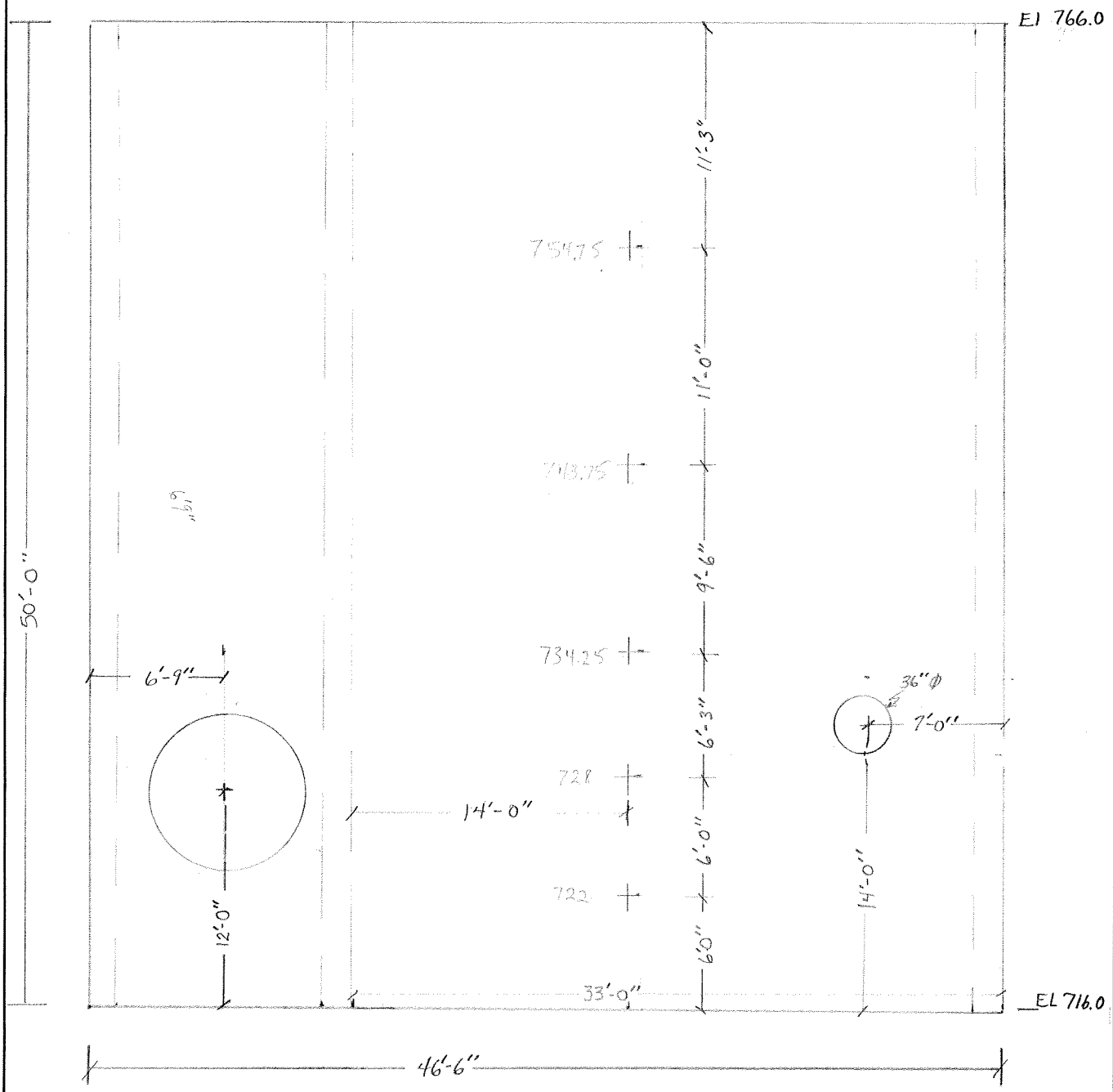
Comp. By MLP Date 5/31/06 Project KCL Sheet 8 of

Chkd. By Date Subject Shawnee Sheet of

Wall Thickness = 1'-6"

CROSS STRUT NODES

EL	NODE	NODE
722	124	540
728	135	539
734.25	146	538
754.75	168	536



West Wall Scale 1/8" = 1'

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Comp. By MP Date 6/1/06 Project KCL Phase II Sheet 1 of

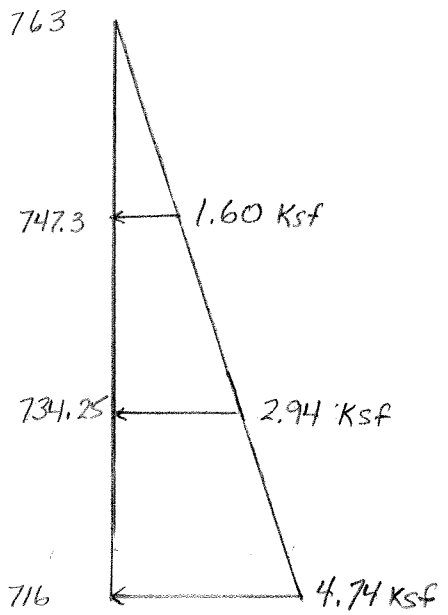
Chkd. By Date Subject Shawnee Sheet of

I. West Wall

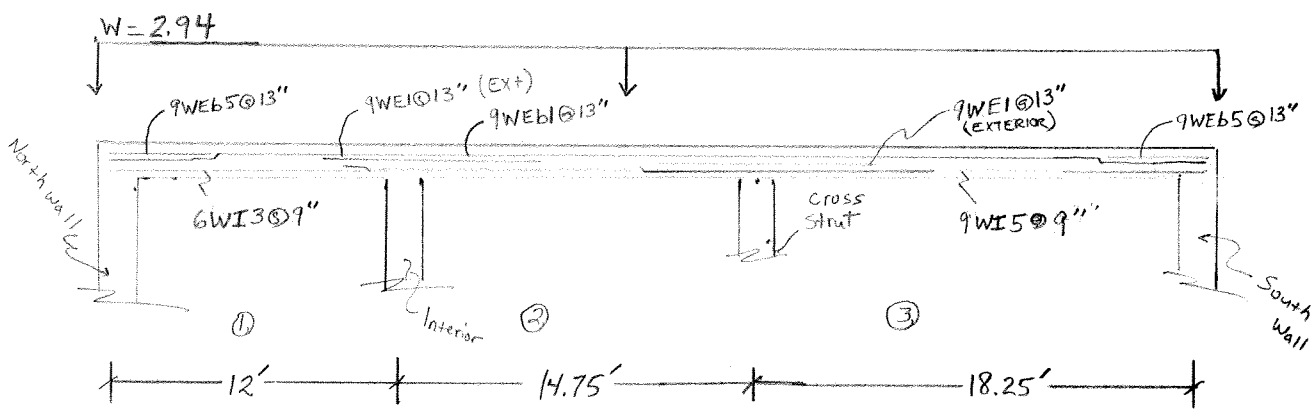
Consider a 1' horizontal strip located 18.25' above El. 716. Check the wall moment & shear strength.

El = 716 + 18.25' = 734.25' ∴ Refer to Mathcad file "Shawnee Pump Station.xmcd"

The load on the west wall at elevation 734.25 = 2,936.7 psf



Reinf. areas from sheets 8, 9 & 16



Center	0.59	1.33	1.33
Left	0.92		
Right		1.84	1.84	1.84

Concrete cover = 2" per Detail 13-10 on Sheet 13

Rebar Callout

WE1	8'-8"	#9
WEb1	4'3", 46'-0"	#9 ← Bar "hooks" into South Wall
WEb5	4'-3", 4'-3"	#9
WI7	36'-0"	#7
WI3	10'-6"	#6

WI5 36'-0"

Refer to Enercalc Shawnee.ecw

MRK FORM 1 DEC 80 222

Elev 734.25 $K_o = 0.56$ Load = 2.94 K/ft West Wall

Clear cover = 2" per Detail 13-10

$d_1 = 18 - \left(2 + \frac{1.128}{2}\right) = 15.4''$

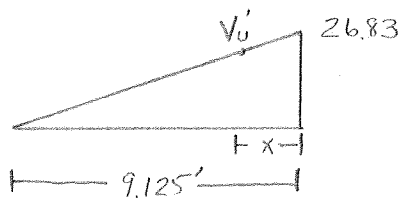
$d_2 = 2 + \frac{1.128}{2} = 2.56''$

	N. Wall to Int	Int to Strut	Strut To S. Wall	
Center	$\frac{39.72}{0.9} = 44.1$ ∴	$\frac{86.3}{0.9} = 95.9$ ∴	95.9	Mn
	$\frac{44.1}{17.64} = 2.5$	$\frac{95.9}{26.65} = 3.6$	$\frac{95.9}{40.8} = 2.35$	$\frac{Mn}{Mu}$
Left End	$\frac{61.09}{0.9} = 67.88$	$\frac{116.59}{0.9} = 129.5$	$\frac{116.59}{0.9} = 129.5$	Mn
	$\frac{67.88}{35.28} = 1.92$	$\frac{129.5}{53.3} = 2.42$	* $\frac{129.5}{81.6} = 1.59$	$\frac{Mn}{Mu}$
Right End	67.88	129.5	129.5	Mn
	$\frac{67.88}{35.28} = 1.92$	2.42	* $\frac{129.5}{81.6} = 1.59$	Mu/Mn

* Controls

Shear:

$V_u = 26.83K$ ∴ Go d" distance from support



$x = \frac{18}{2} + 15.4 = 24.4'' = 2.03'$

$\frac{26.83}{9.125} = \frac{V_u}{9.125 - 2.03} \Rightarrow V_u = 20.86K$

$V_c = 2\sqrt{4000}\,(12)15.4 = 23.38K$ ∴ $\frac{V_c}{V_u} = 1.12$ N.G.

West Wall @ 734.25 con't Shear

Thrust: $\frac{2.94(27)}{2} = 39.69\ K$ 27' = Length of North & South Walls @ Elev 734.25

Eqn 11-4 of ACI

$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}\ bd$ ∴ $N_u = 39.69$ ∴ $A_g = 12(18) = 216\ in^2$

$b = 12''$ $d = 15.4''$

$V_c = 2\left(1 + \frac{39{,}690}{2000(216)}\right)\sqrt{4000}\ (12)15.4 =$

$2.18\sqrt{4000}\ (12)15.4 = 25.52\ K$

$\frac{25.52}{20.86} = 1.22$ Still N.G.

Eqn 11-5 of ACI

$\rho_w = \frac{A_s}{bd} = \frac{1.33+1.84}{12(15.4)} = 0.0172$ ∴ $M_u = 81600\ \#\cdot ft = 979200\ \#\cdot in$

$V_u = 20860\ \#$

$V_c = \left(1.9\sqrt{4000} + 2500(0.0172)\ \frac{20860(15.4)}{979200}\right)12(15.4)$

$= \left(1.9\sqrt{4000} + 14.07\right)12(15.4)$

$= 24.81\ K$

$\frac{24.81}{20.86} = 1.19$ Still N.G.

FS for shear at subject location < 1.5

At what elevation does the FS for shear ≥ 1.5 for the west wall?

Backcalculating, @ Elev 742, the load is 2.15

$2.15\left(\frac{18.25}{2}\right) = 19.6\ K = V_u'$ ∴ V_u at "d" away $= \frac{19.6(9.125 - 2.03)}{9.125} = 15.25K = V_u$

Using Eqn 11-4, $N_u = \frac{2.15(27)}{2} = 29\ K$

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$$V_c = 2\left(1 + \frac{29000}{2000(216)}\right)\sqrt{4000}\,(12)\,15.4 =$$

$$= 2.13\sqrt{4000}\,(12)\,15.4 = 24.9\text{ K}$$

$$\frac{24.9}{15.25} = 1.63$$

FS for shear > 1.5 at El. 742 or higher for the west wall.

Punching Shear At Cross Strut 3 Located at EL 743.25

ACI 11.12

$d = 15.4'' \therefore d/2 = 7.7$

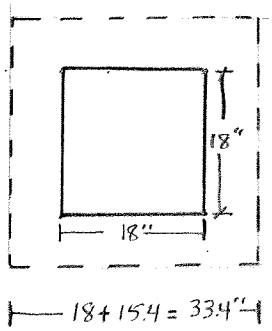
See Detail 13-7, Sheet 13

$b_0 = 33.4(4) = 133.6''$

$\beta_c = 1$

pressure = 2.94 KSF

Area of "b_0" $= (33.4)^2$
$= 1115.6 in^2$
$= 7.75 ft^2$



Load = Area x pressure
= 7.75 (2.94)
= 22.8 Kip

$2 + 4/\beta_c = 2 + 4 = 6 \therefore \frac{\alpha_s d}{b_0} = \frac{40(15.4)}{133.6} = 4.6$ Use Eqn 11-35

$V_c = 4\sqrt{4000}\ 133.6(15.4) = 520\ K$ FS >> 1.5 OKAY FOR PUNCHING SHEAR

FOR BEAM ACTION AT THE CROSS STRUT SUPPORT, THE FS IS <1.5 (SAME AS AT THE SOUTH WALL SUPPORT) AS REPORTED EARLIER

Multi-Span Concrete Beam

Description West wall elev. 734.25. Ko=0.56

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy	60,000.0 psi	Spans Considered Continuous Over Supports		ACI Dead Load Factor	1.00
f'c	4,000.0 psi	Stirrup Fy	0.0 psi	ACI Live Load Factor	1.00

Concrete Member Information

Description			N.wall to int.	Int. to strut	strut to S. wall
Span		ft	12.00	14.75	18.25
Beam Width		in	12.00	12.00	12.00
Beam Depth		in	18.00	18.00	18.00
End Fixity			Fix-Fix	Fix-Fix	Fix-Fix
Reinforcing	Center	Area	0.59in2	1.33in2	1.33in2
		Bar Depth	15.40in	15.40in	15.40in
	Left	Area	0.92in2	1.84in2	1.84in2
		Bar Depth	2.56in	2.56in	2.56in
	Right	Area	0.92in2	1.84in2	1.84in2
		Bar Depth	2.56in	2.56in	2.56in

Loads

Using Live Load This Span ??		No	No	No
Dead Load	k/ft	2.940	2.940	2.940
Live Load	k/ft			

Results

		Beam OK	Beam OK	Beam OK
Mmax @ Cntr	k-ft	17.64	26.65	40.80
@ X =	ft	6.00	7.37	9.12
Mn * Phi	k-ft	39.72	86.30	86.30
Max @ Left End	k-ft	-35.28	-53.30	-81.60
Mn * Phi	k-ft	61.09	116.59	116.59
Max @ Right End	k-ft	-35.28	-53.30	-81.60
Mn * Phi	k-ft	61.09	116.59	116.59
		Bending OK	Bending OK	Bending OK
Shear @ Left	k	17.64	21.68	26.83
Shear @ Right	k	17.64	21.68	26.83

Reactions & Deflections

DL @ Left	k	17.64	21.68	26.83
LL @ Left	k	0.00	0.00	0.00
Total @ Left	k	17.64	21.68	26.83
DL @ Right	k	17.64	21.68	26.83
LL @ Right	k	0.00	0.00	0.00
Total @ Right	k	17.64	21.68	26.83
Max. Deflection	in	-0.028	-0.083	-0.231
@ X =	ft	6.00	7.37	9.12
Inertia : Effective	in4	2,748.76	2,100.99	1,765.09

Shear Stirrups

Stirrup Rebar Area	in2	0.000	0.000	0.220
Spacing @ Left	in	0.00	0.00	0.00
Spacing @ .2*L	in	0.00	0.00	0.00
Spacing @ .4*L	in	Not Req'd	Not Req'd	Not Req'd
Spacing @ .6*L	in	Not Req'd	Not Req'd	Not Req'd
Spacing @ .8*L	in	0.00	0.00	0.00
Spacing @ Right	in	0.00	0.00	0.00

Description West wall elev. 734.25. Ko=0.56

Query Values

Location	ft	0.00	0.00	0.00
Moment	k-ft	-35.3	-53.3	-81.6
Shear	k	17.6	21.7	26.8
Deflection	in	0.0000	0.0000	0.0000

```
100 Program X0067
110 Investigation of Shawnee Pump Station Cross Strut 3
200 1
210 4.00 60.0 0.375
300 18 x 18 with 1 layer
400 18.0 18
500 1
510 3 1.56 14.3 3.7 14.3
600 449.7 19
```

See detail 13-7, Sheet 13

CASTR SHAWmem3.dat Output

Diagnostic Output:

```
Required Strength
-----------------

P(U) =     449.7 k              PHI        = .7000
M(U) =      19.0 k-ft           ASTOT/B*H  = .0144

Results
-------

Section falls within admissible range.

Does not meet input data reinforcement limit requirement.

Minimum D for no comp steel =  27.75 inches.

60.07% of design strength used in
compression (steel in tension) zone.
```

RAM OUTPUT, UNFACTORED AXIAL LOAD

RAM OUTPUT, UNFACTORED AXIAL LOAD

$$FS = \frac{1}{0.601(0.9)} = \underline{\underline{1.85}}$$

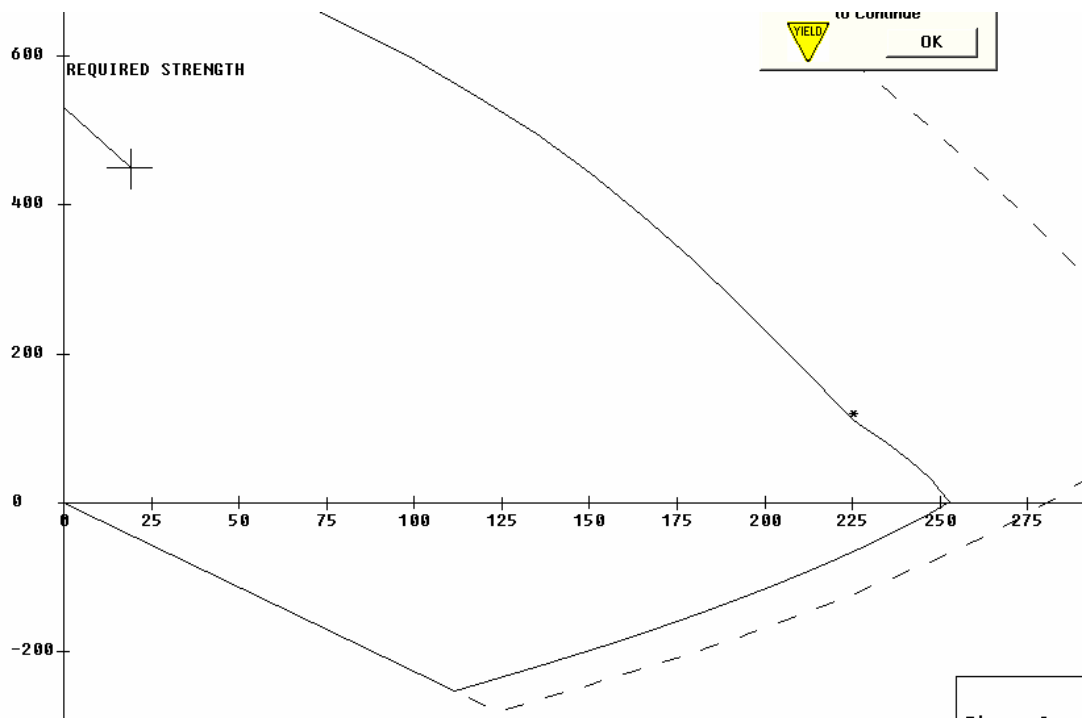
Accept

Shawnee Pump Station Interaction Diagram.

The CASTR program output prints the percentage of permissible design strength used by the applied loading. The previous calculations used this value to determine the Factor of Safety. Under certain circumstances, using the CASTR output value of percent design strength used by the applied loading may create an conservative factor of safety (FoS') value. As an alternative, the section strength was obtained from the interaction diagram output created by CASTR. The interaction diagram strength value was compared to the demand moment to determine the section bending FoS'.

```
100 Program X0067
110 Investigation of Shawnee Pump Station Cross Strut 3
200 1
210 4.00 60.0 0.375
300 18 x 18 with 1 layer
400 18.0 18
500 1
510 3 1.56 14.3 3.7 14.3
600 449.7 19
```

CASTR Input File, Cross Strut 3



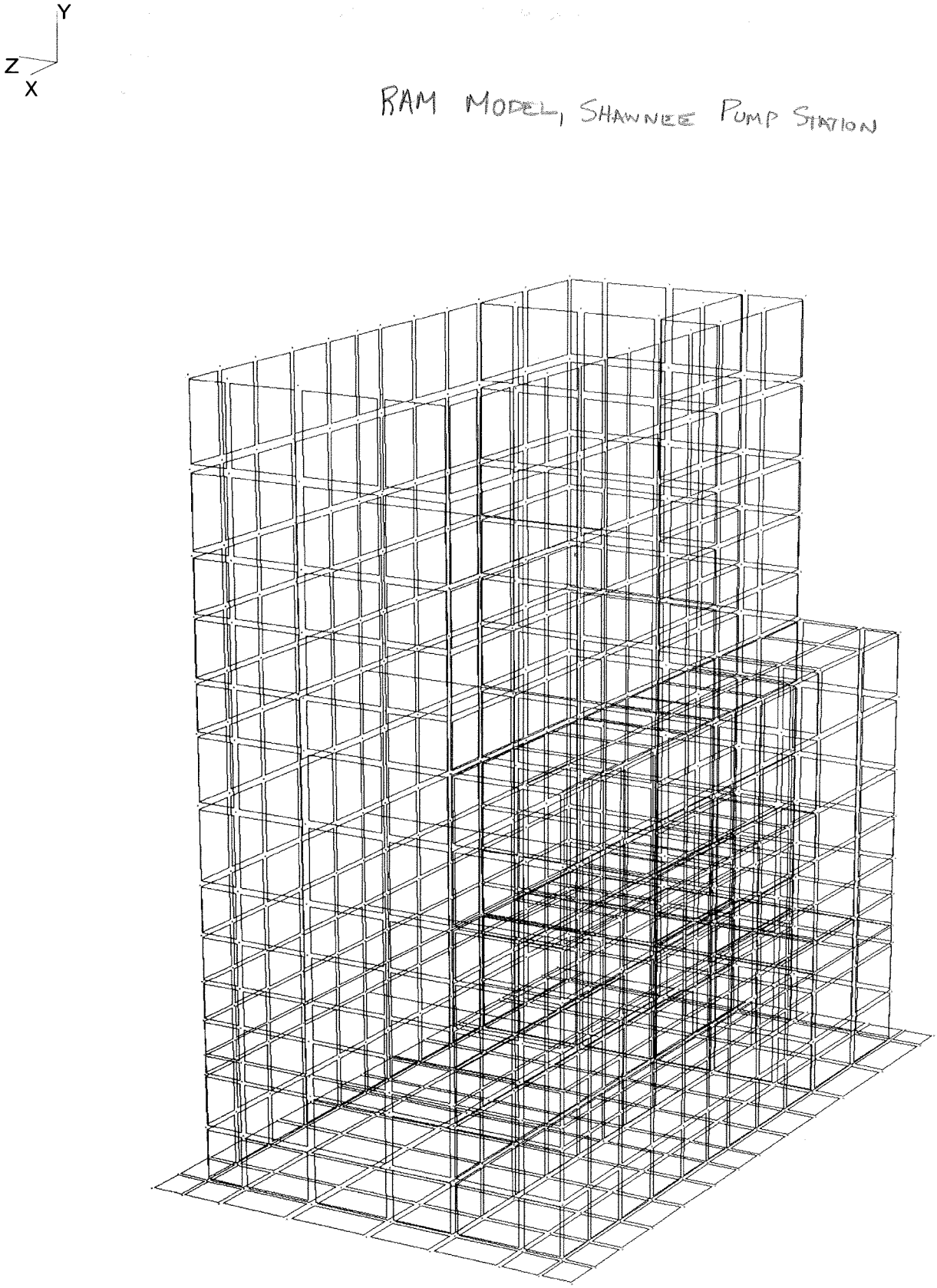
Shawmem3.dat Interaction Diagram

Conclusion. The required strength is well within the interaction diagram envelope.

RAM International

File : Shawnee_rev2.AVW
Condition: DL=Dead Load

Analysis (1st order)
Date: 6/13/2006 9:11:23 AM



RAM International

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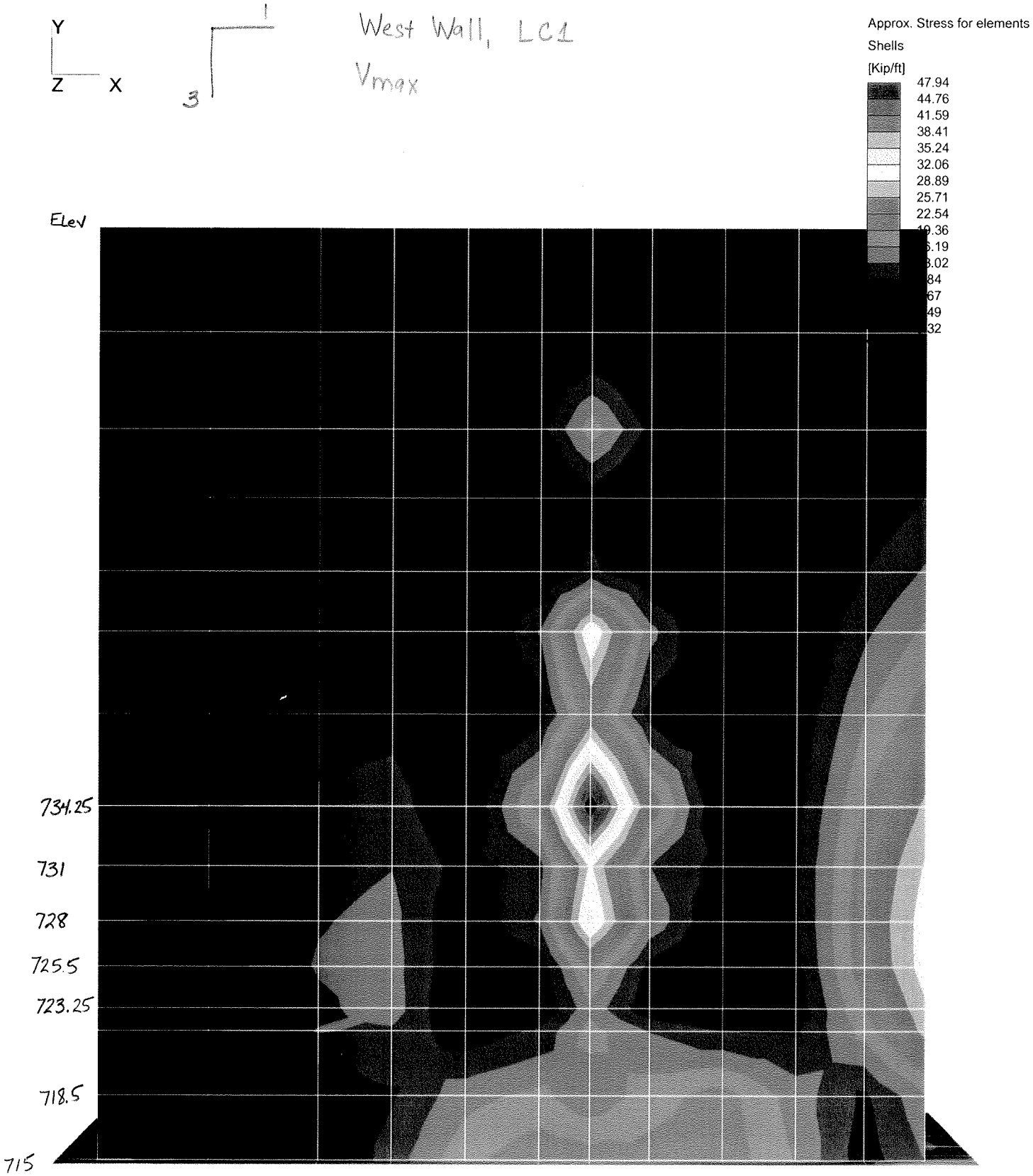
Condition: LC1=DL+SLV1+SLV2+SL

Analysis (1st order)

Date: 6/15/2006 7:09:12 AM



RAM International

File : Shawnee_rev4.AVW

Condition: LC1=DL+SLV1+SLV2+SL

Analysis (1st order)

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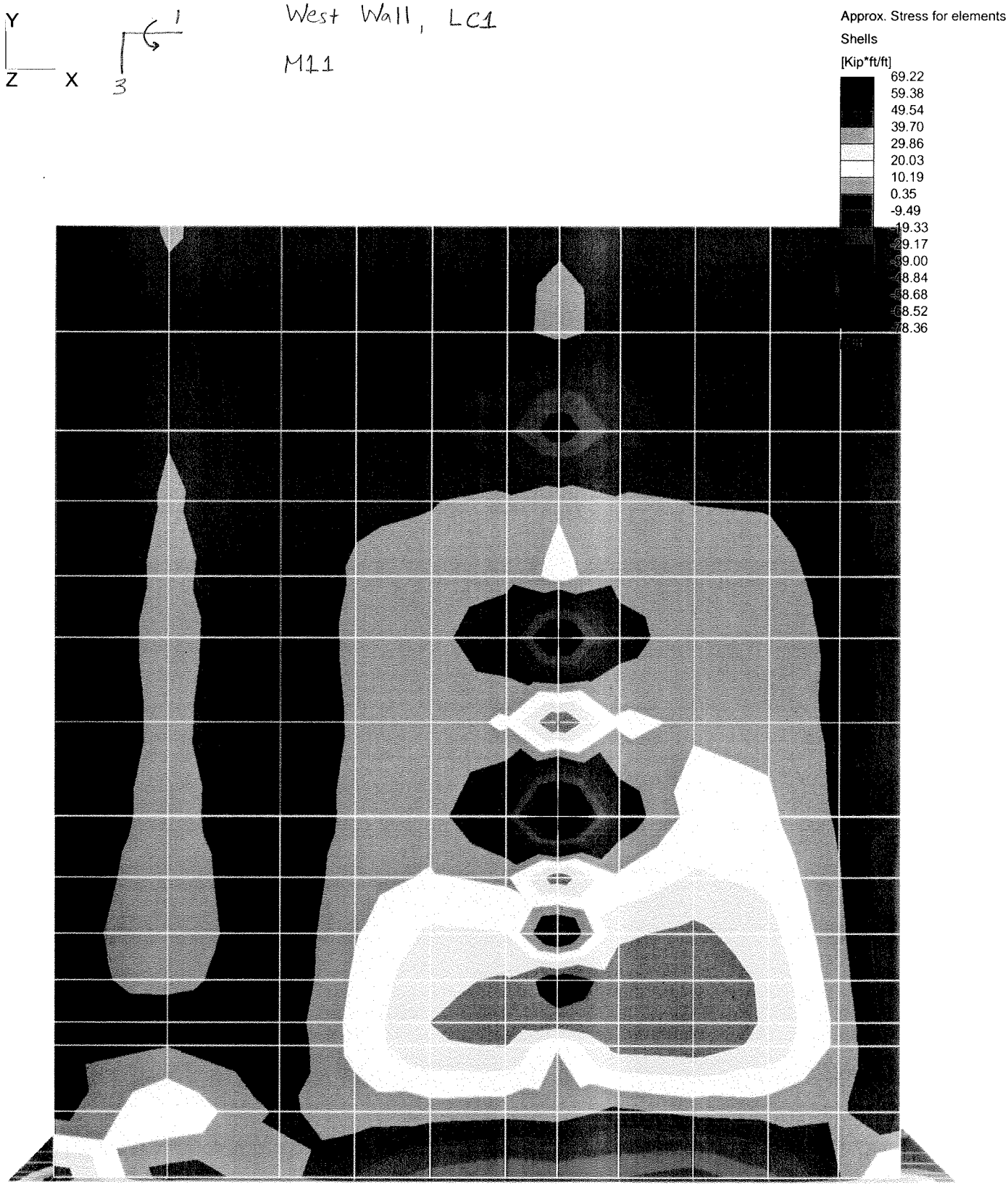

RAM International

File : Shawnee_rev4.AVW
Condition: LC1=DL+SLV1+SLV2+SL

Analysis (1st order)
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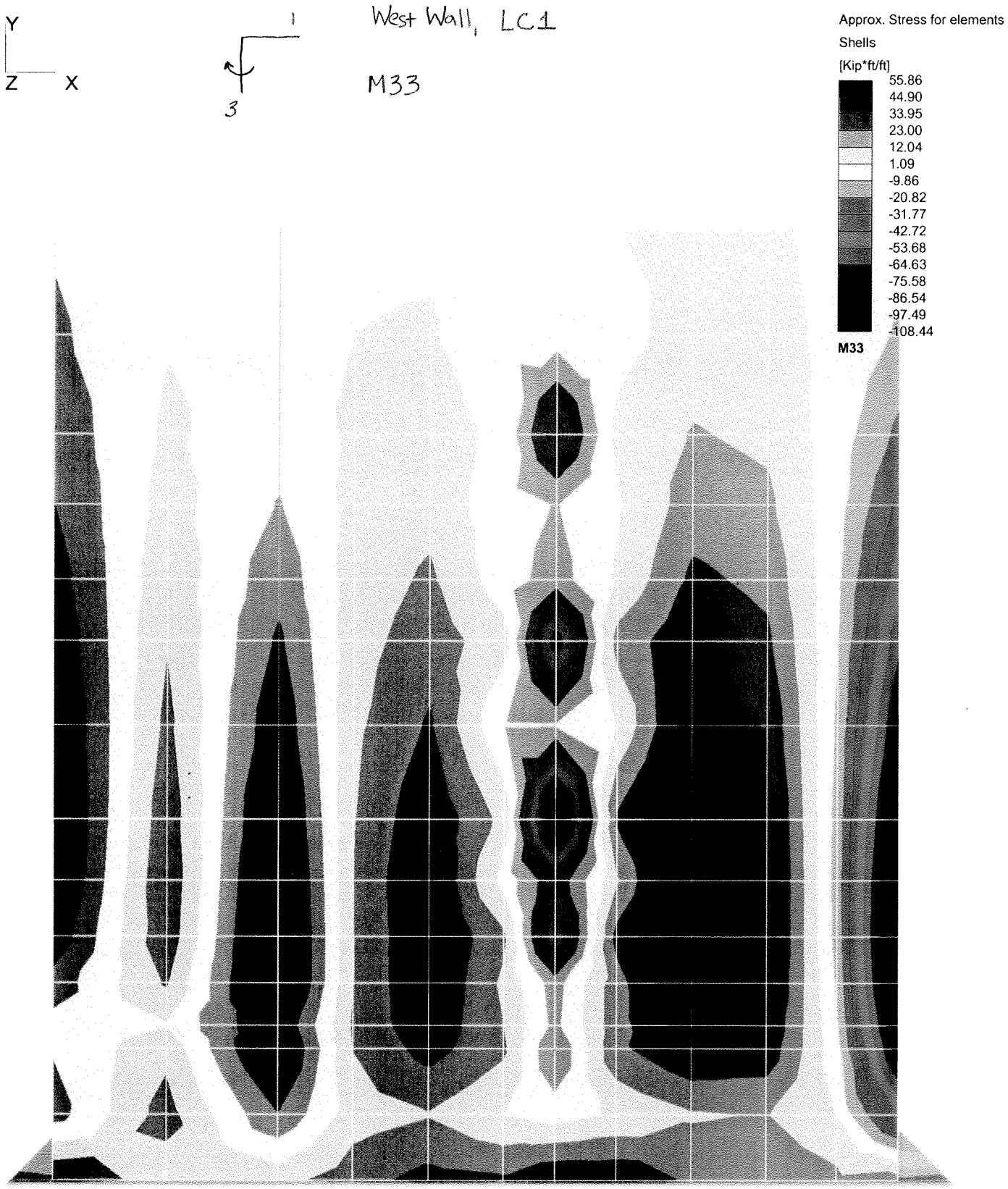

RAM International
File : Shawnee_rev4.AVW
Condition: LC1=DL+SLV1+SLV2+SL

Analysis (1st order)
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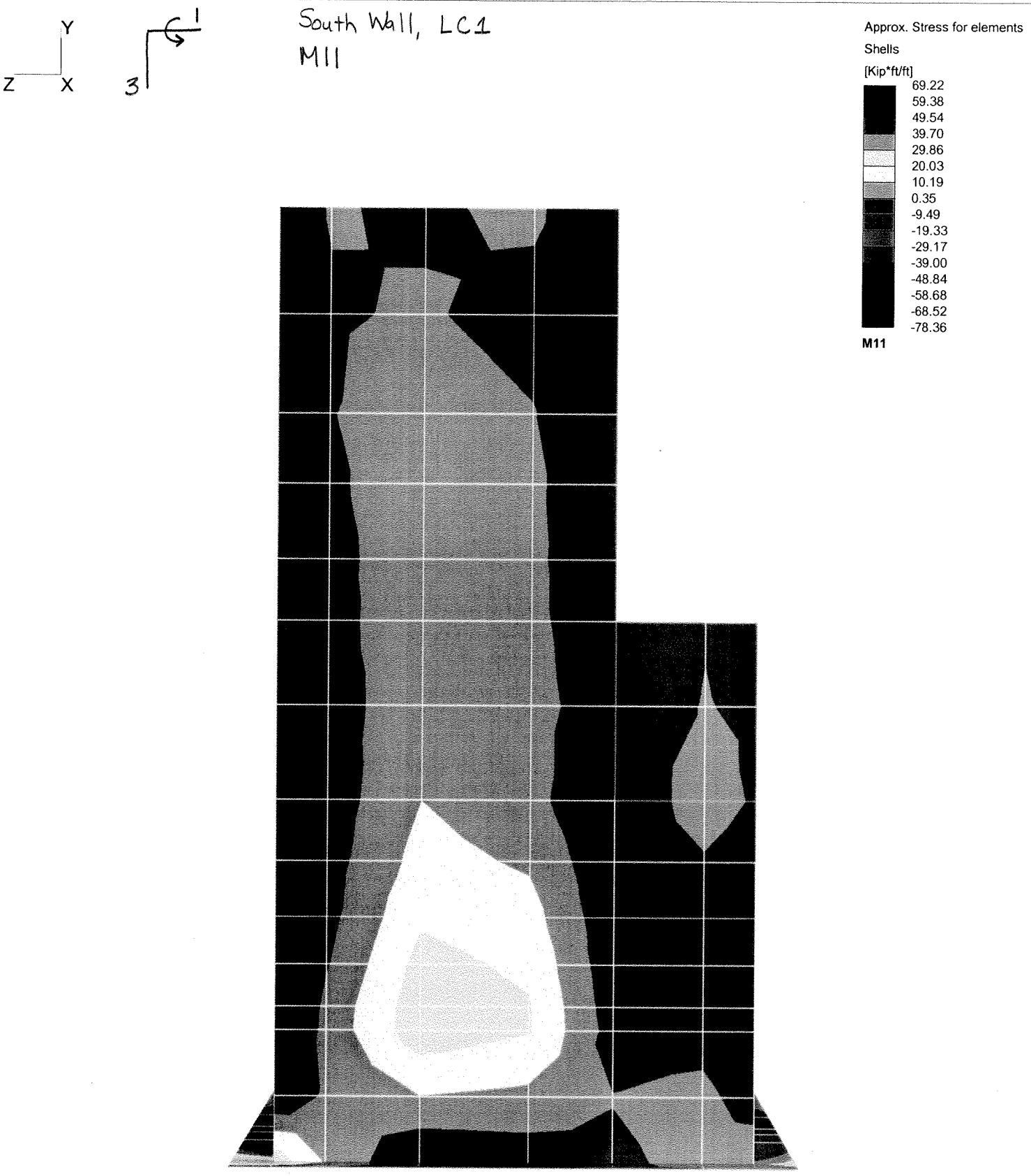

RAM International

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Analysis (1st order)

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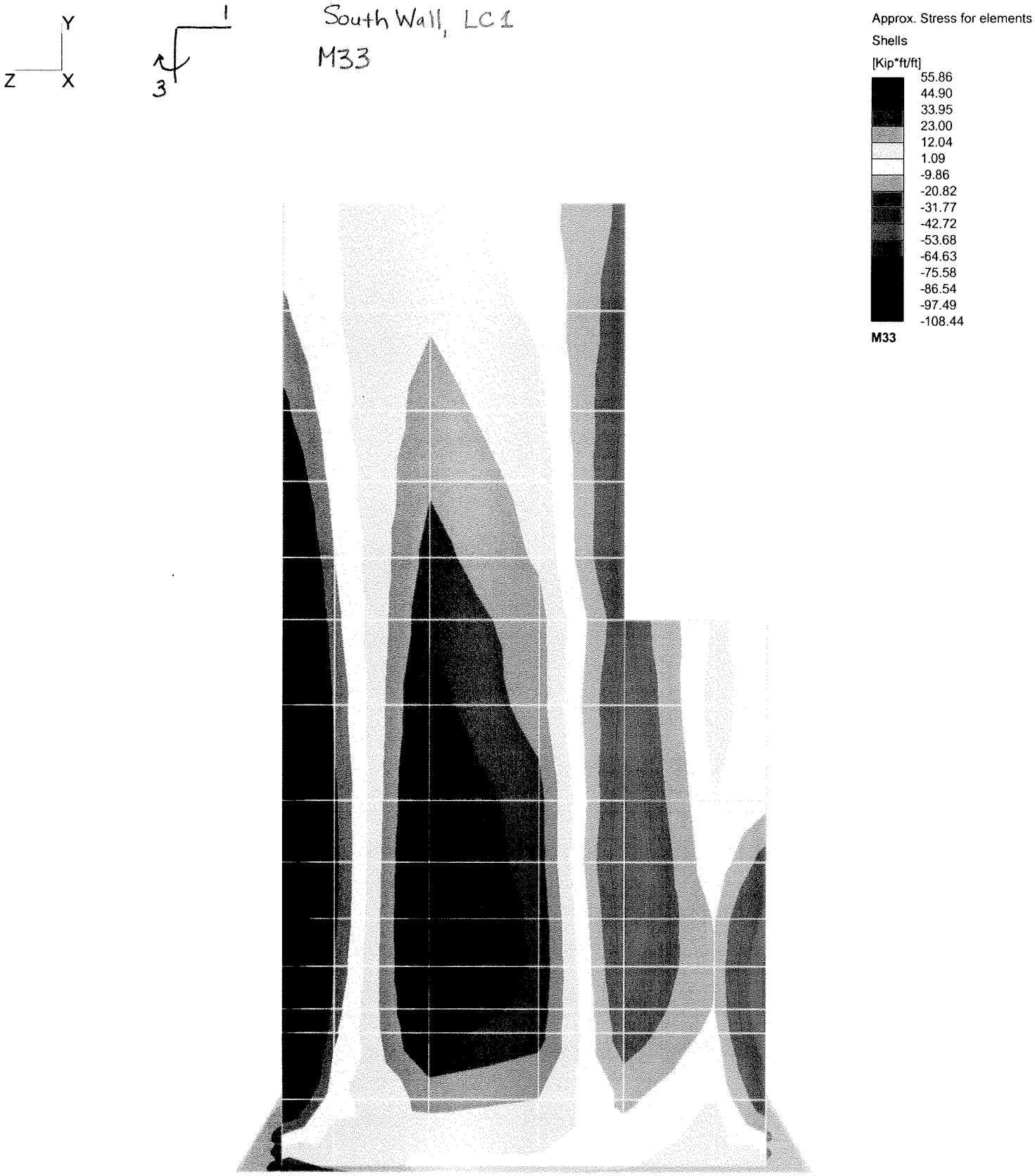


RAM International

File : Shawnee_rev4.AVW

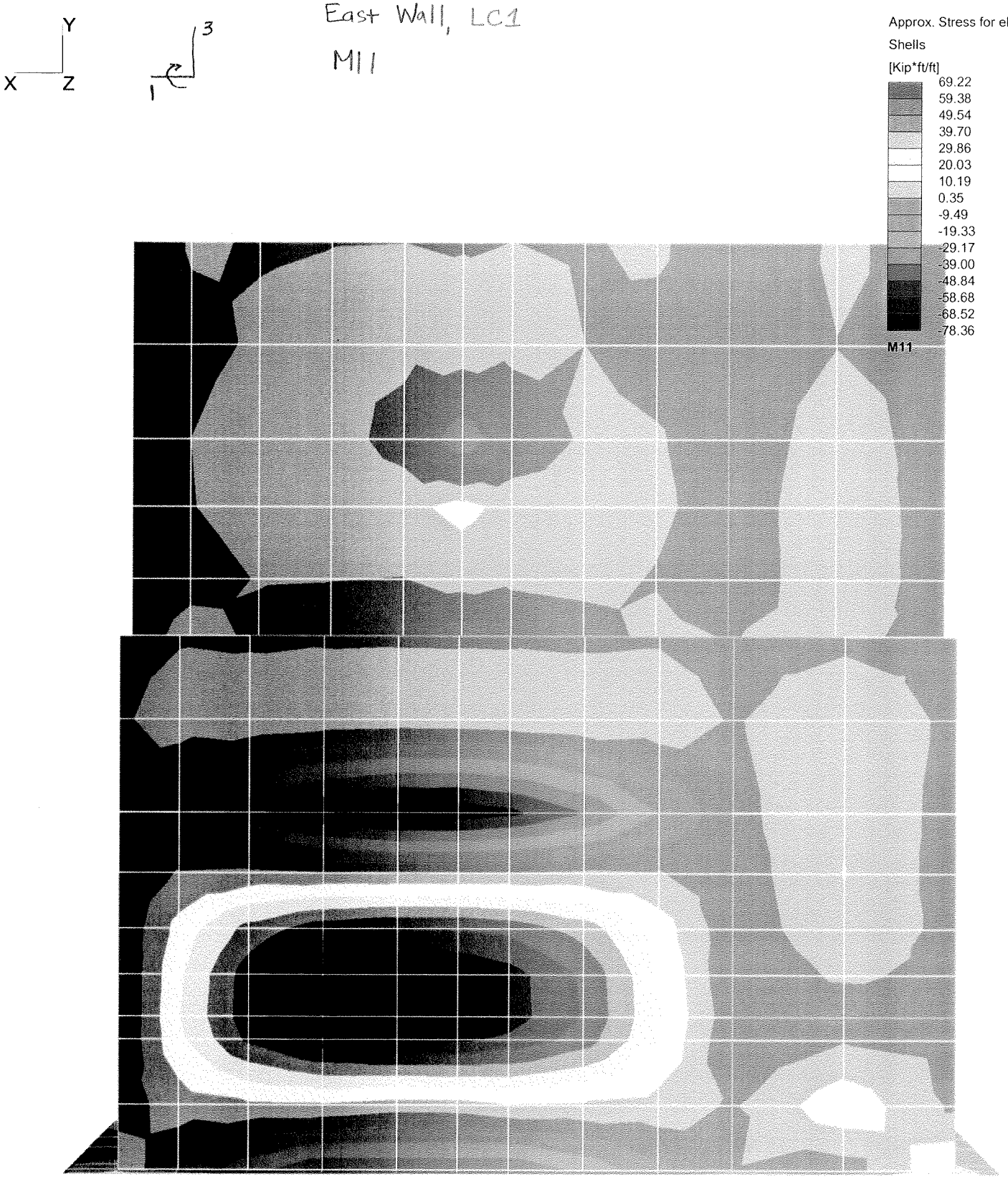
Condition: LC1=DL+SLV1+SLV2+SL

Analysis (1st order)

Date: 6/15/2006 7:04:06 AM

East Wall, LC1

M11



RAM International

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Condition: LC1=DL+SLV1+SLV2+SL

Analysis (1st order)

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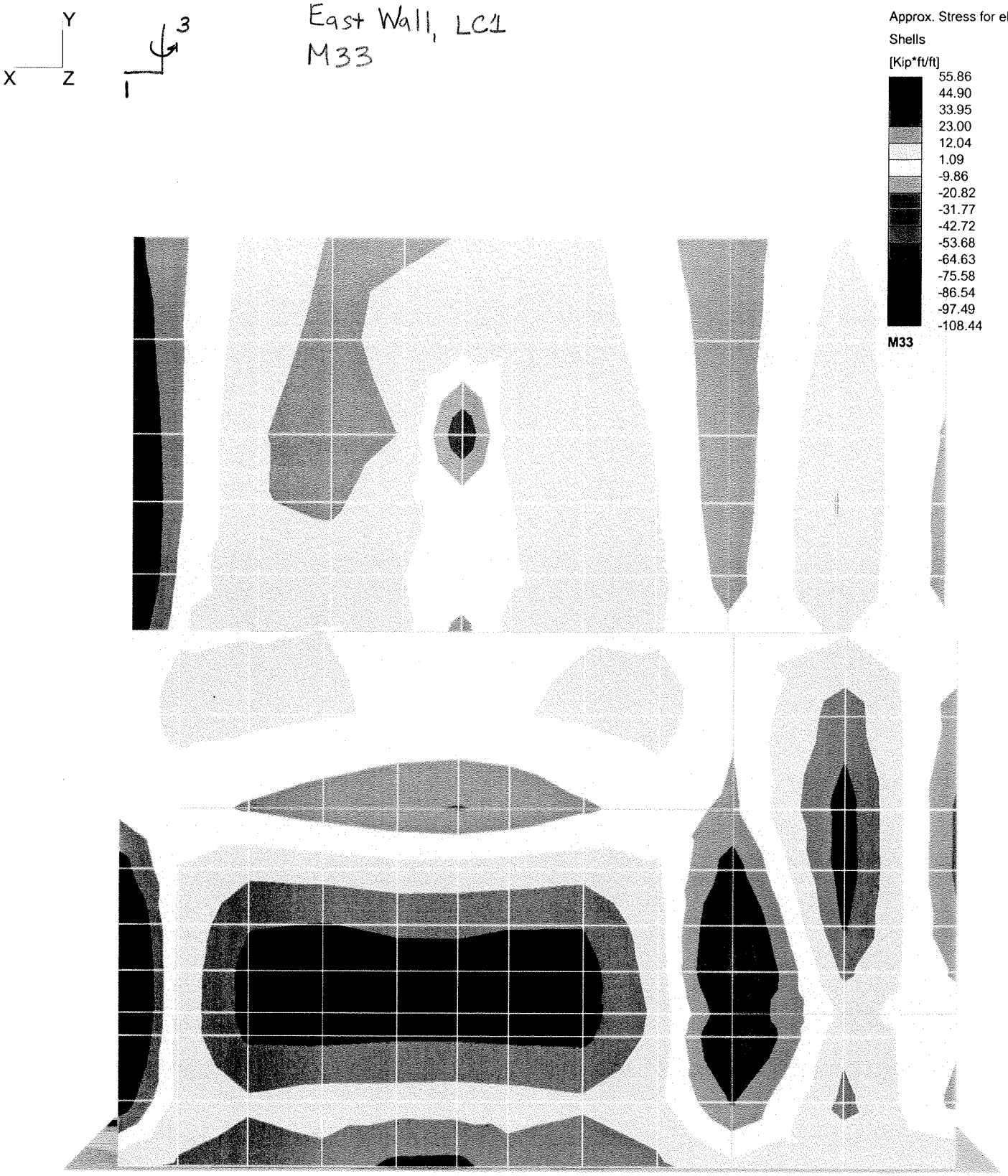
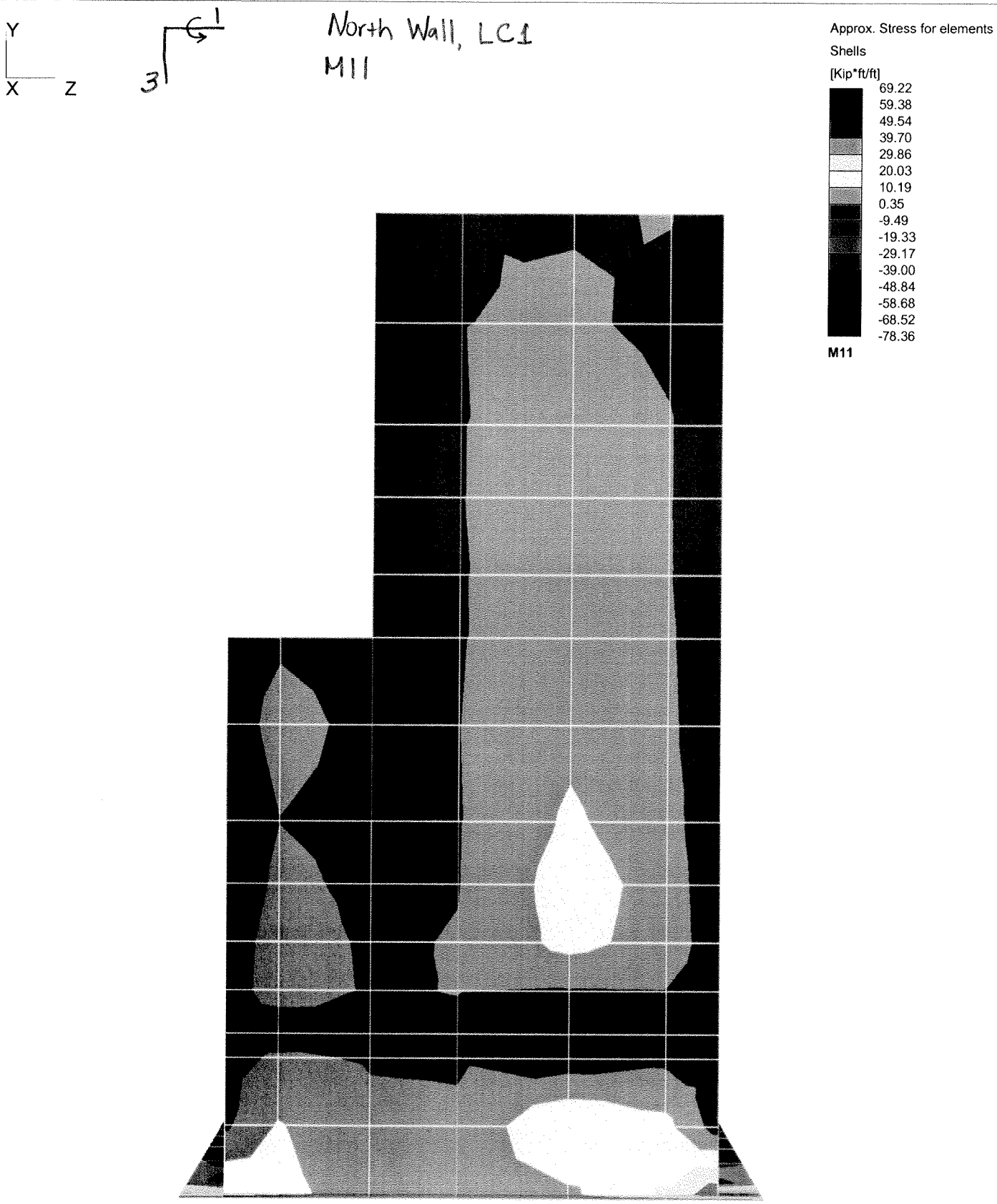


RAM International
File : Shawnee_rev4.AVW
Condition: LC1=DL+SLV1+SLV2+SL

Analysis (1st order)
Date: 6/15/2006 7:04:49 AM



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Analysis (1st order)

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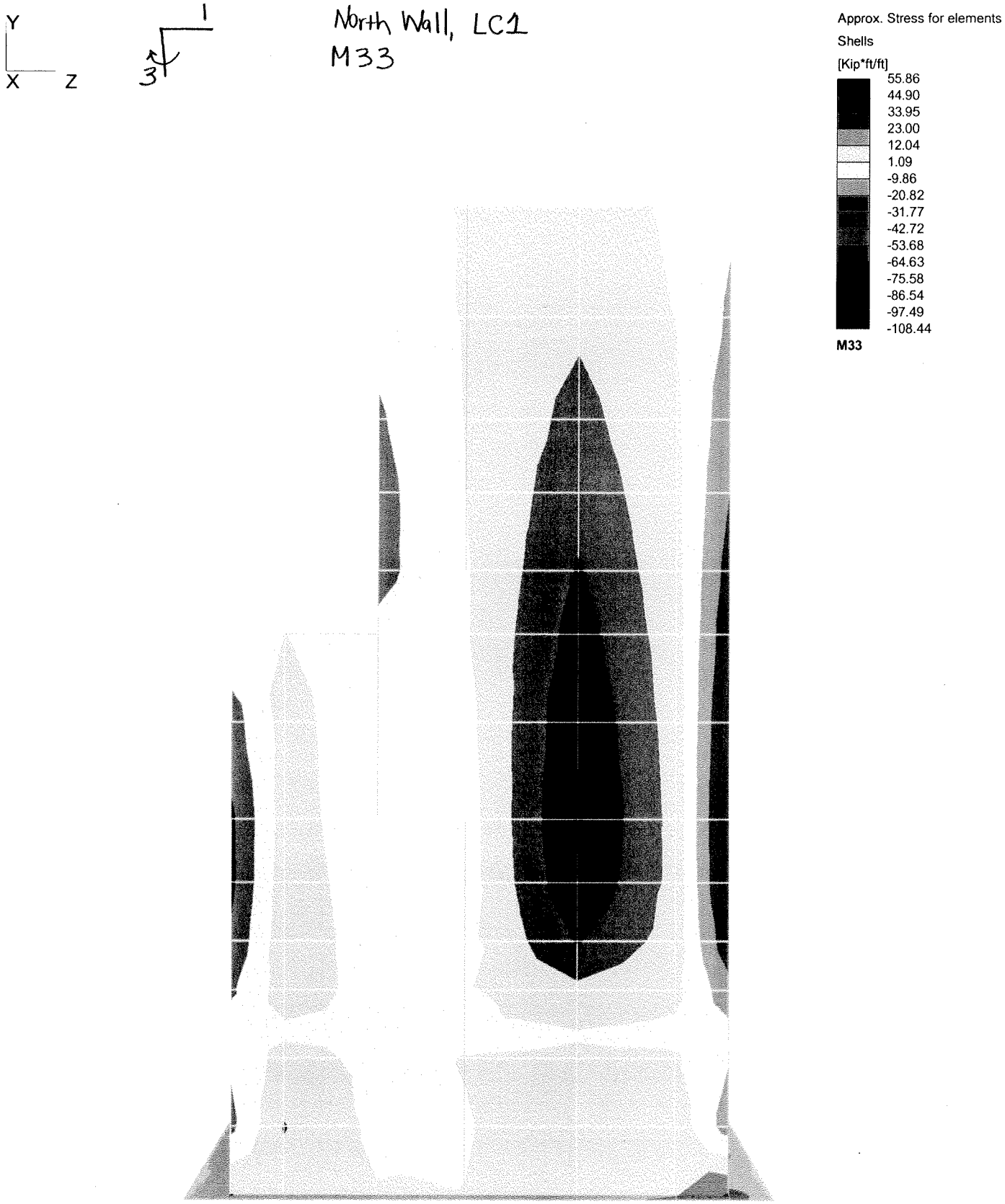


RAM International

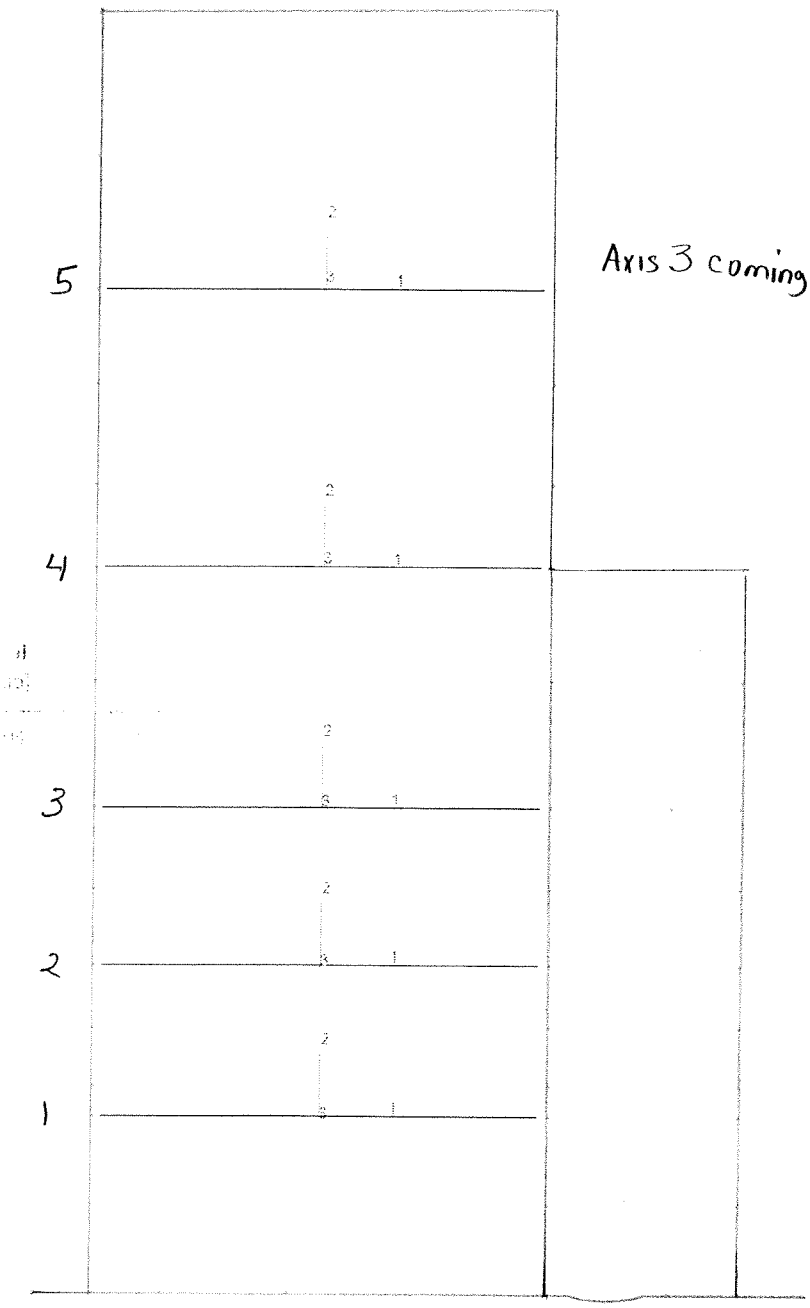
File : Shawnee_rev4.AVW
Condition: LC1=DL+SLV1+SLV2+SL

Analysis (1st order)
Date: 6/17/2006 6:55:51 PM

Y
Z X

Shawnee

Cross Struts



Axis 3 coming out of page

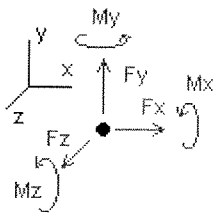
Axial = 449.7K
M_{11} = -19 K.ft

RAM International

File : C:\Documents and Settings\g5pexmlp\My Documents_PROJECTS\Civil\KC Levee Phase 2 04132006\RAM\Shawnee_rev4.AVW
Units system : English
Date : 6/17/2006 6:52:57 PM

Analysis Results

Reactions

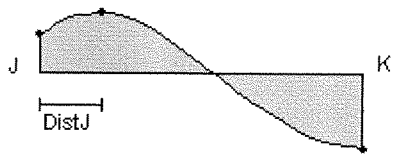


CROSS STRUT ANALYSIS

Direction of positive forces and moments

Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **LC1=DL+SLV1+SLV2+SL**						
SUM	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Points of interest in members



Considered points

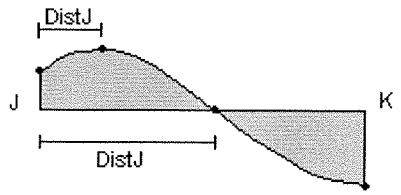
CONDITION **LC1=DL+SLV1+SLV2+SL**

Station	Dist to J [ft]	Axial [Kip]	Plane 1-2 Shear V2 [Kip]	M33 [Kip*ft]	Shear V3 [Kip]	Plane 1-3 M22 [Kip*ft]	Torsion [Kip*ft]
MEMBER **1**							
0%	0.000	-137.203	-0.911	11.369	0.400	-4.360	0.007
15%	2.700	-137.203	-0.036	12.648	0.400	-3.281	0.007
100%	18.000	-137.203	4.919	-24.704	0.400	2.836	0.007
MEMBER **2**							
0%	0.000	-255.700	-4.287	-18.979	0.651	-7.361	0.007
73%	13.050	-255.700	-0.060	9.384	0.651	1.130	0.007
100%	18.000	-255.700	1.543	5.713	0.651	4.350	0.007
MEMBER **3**							
0%	0.000	-449.675	-4.333	-20.982	0.616	-7.358	0.006
75%	13.500	-449.675	0.040	7.996	0.616	0.951	0.006

100%	18.000	-449.675	1.497	4.539	0.616	3.721	0.006
MEMBER **4**							
0%	0.000	-288.378	-3.566	-15.881	0.498	-5.727	0.004
60%	10.800	-288.378	-0.068	3.739	0.498	-0.346	0.004
100%	18.000	-288.378	2.264	-4.168	0.498	3.241	0.004
MEMBER **5**							
0%	0.000	-173.649	-3.662	-15.659	0.154	-2.473	0.006
63%	11.250	-173.649	-0.019	5.045	0.154	-0.740	0.006
100%	18.000	-173.649	2.168	-2.209	0.154	0.300	0.006

Points of interest and inflection points in members

Note: Inflection points are approximated



Considered points

CONDITION **LC1=DL+SLV1+SLV2+SL**

			Plane 1-2			Plane 1-3	
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[ft]	[Kip]	[Kip]	[Kip*ft]	[Kip]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	-137.203	-0.911	11.369	0.400	-4.360	0.007
15%	2.700	-137.203	-0.036	12.648	0.400	-3.281	0.007
61%	10.906	-137.203	2.622	2.040	0.400	0.000	0.007
65%	11.649	-137.203	2.862	0.003	0.400	0.297	0.007
100%	18.000	-137.203	4.919	-24.704	0.400	2.836	0.007
MEMBER **2**							
0%	0.000	-255.700	-4.287	-18.979	0.651	-7.361	0.007
31%	5.624	-255.700	-2.465	0.008	0.651	-3.702	0.007
63%	11.314	-255.700	-0.622	8.791	0.651	0.000	0.007
73%	13.050	-255.700	-0.060	9.384	0.651	1.130	0.007
100%	18.000	-255.700	1.543	5.713	0.651	4.350	0.007
MEMBER **3**							
0%	0.000	-449.675	-4.333	-20.982	0.616	-7.358	0.006
35%	6.351	-449.675	-2.276	0.003	0.616	-3.449	0.006
66%	11.955	-449.675	-0.461	7.671	0.616	0.000	0.006
75%	13.500	-449.675	0.040	7.996	0.616	0.951	0.006
100%	18.000	-449.675	1.497	4.539	0.616	3.721	0.006
MEMBER **4**							
0%	0.000	-288.378	-3.566	-15.881	0.498	-5.727	0.004
34%	6.203	-288.378	-1.557	0.006	0.498	-2.637	0.004
60%	10.800	-288.378	-0.068	3.739	0.498	-0.346	0.004
64%	11.495	-288.378	0.157	3.708	0.498	0.000	0.004
88%	15.816	-288.378	1.557	0.004	0.498	2.153	0.004
100%	18.000	-288.378	2.264	-4.168	0.498	3.241	0.004
MEMBER **5**							

0%	0.000	-173.649	-3.662	-15.659	0.154	-2.473	0.006
32%	5.729	-173.649	-1.807	0.006	0.154	-1.591	0.006
63%	11.250	-173.649	-0.019	5.045	0.154	-0.740	0.006
89%	16.055	-173.649	1.538	1.395	0.154	0.000	0.006
94%	16.884	-173.649	1.806	0.008	0.154	0.128	0.006
100%	18.000	-173.649	2.168	-2.209	0.154	0.300	0.006

Member forces at fixed stations

CONDITION **LC1=DL+SLV1+SLV2+SL**

	M33 [Kip*ft]	**V2** [Kip]	**M22** [Kip*ft]	**V3** [Kip]	**Axial** [Kip]	**Torsion** [Kip*ft]
MEMBER **1**						
0%	11.37	-0.91	-4.36	0.40	-137.20	0.01
25%	12.19	0.55	-2.56	0.40	-137.20	0.01
50%	6.45	2.00	-0.76	0.40	-137.20	0.01
75%	-5.85	3.46	1.04	0.40	-137.20	0.01
100%	-24.70	4.92	2.84	0.40	-137.20	0.01
MEMBER **2**						
0%	-18.98	-4.29	-7.36	0.65	-255.70	0.01
25%	-2.97	-2.83	-4.43	0.65	-255.70	0.01
50%	6.48	-1.37	-1.51	0.65	-255.70	0.01
75%	9.38	0.09	1.42	0.65	-255.70	0.01
100%	5.71	1.54	4.35	0.65	-255.70	0.01
MEMBER **3**						
0%	-20.98	-4.33	-7.36	0.62	-449.67	0.01
25%	-4.76	-2.88	-4.59	0.62	-449.67	0.01
50%	4.90	-1.42	-1.82	0.62	-449.67	0.01
75%	8.00	0.04	0.95	0.62	-449.67	0.01
100%	4.54	1.50	3.72	0.62	-449.67	0.01
MEMBER **4**						
0%	-15.88	-3.57	-5.73	0.50	-288.38	0.00
25%	-3.12	-2.11	-3.49	0.50	-288.38	0.00
50%	3.09	-0.65	-1.24	0.50	-288.38	0.00
75%	2.74	0.81	1.00	0.50	-288.38	0.00
100%	-4.17	2.26	3.24	0.50	-288.38	0.00
MEMBER **5**						
0%	-15.66	-3.66	-2.47	0.15	-173.65	0.01
25%	-2.46	-2.20	-1.78	0.15	-173.65	0.01
50%	4.18	-0.75	-1.09	0.15	-173.65	0.01
75%	4.27	0.71	-0.39	0.15	-173.65	0.01
100%	-2.21	2.17	0.30	0.15	-173.65	0.01

Forces envelope

Note.- **Ic** is the controlling load condition
Forces envelope for :
LC1=DL+SLV1+SLV2+SL

MEMBER 1

Station		Axial [Kip]	lc	Shear V2 [Kip]	lc	Shear V3 [Kip]	lc	Torsion [Kip*ft]	lc	M22 [Kip*ft]	lc	M33 [Kip*ft]	lc
0%	Max	-137.20	LC1	-0.91	LC1	0.40	LC1	0.01	LC1	-4.36	LC1	11.37	LC1
	Min	-137.20	LC1	-0.91	LC1	0.40	LC1	0.01	LC1	-4.36	LC1	11.37	LC1
25%	Max	-137.20	LC1	0.55	LC1	0.40	LC1	0.01	LC1	-2.56	LC1	12.19	LC1
	Min	-137.20	LC1	0.55	LC1	0.40	LC1	0.01	LC1	-2.56	LC1	12.19	LC1
50%	Max	-137.20	LC1	2.00	LC1	0.40	LC1	0.01	LC1	-0.76	LC1	6.45	LC1
	Min	-137.20	LC1	2.00	LC1	0.40	LC1	0.01	LC1	-0.76	LC1	6.45	LC1
75%	Max	-137.20	LC1	3.46	LC1	0.40	LC1	0.01	LC1	1.04	LC1	-5.85	LC1
	Min	-137.20	LC1	3.46	LC1	0.40	LC1	0.01	LC1	1.04	LC1	-5.85	LC1
100%	Max	-137.20	LC1	4.92	LC1	0.40	LC1	0.01	LC1	2.84	LC1	-24.70	LC1
	Min	-137.20	LC1	4.92	LC1	0.40	LC1	0.01	LC1	2.84	LC1	-24.70	LC1

MEMBER 2

Station		Axial [Kip]	lc	Shear V2 [Kip]	lc	Shear V3 [Kip]	lc	Torsion [Kip*ft]	lc	M22 [Kip*ft]	lc	M33 [Kip*ft]	lc
0%	Max	-255.70	LC1	-4.29	LC1	0.65	LC1	0.01	LC1	-7.36	LC1	-18.98	LC1
	Min	-255.70	LC1	-4.29	LC1	0.65	LC1	0.01	LC1	-7.36	LC1	-18.98	LC1
25%	Max	-255.70	LC1	-2.83	LC1	0.65	LC1	0.01	LC1	-4.43	LC1	-2.97	LC1
	Min	-255.70	LC1	-2.83	LC1	0.65	LC1	0.01	LC1	-4.43	LC1	-2.97	LC1
50%	Max	-255.70	LC1	-1.37	LC1	0.65	LC1	0.01	LC1	-1.51	LC1	6.48	LC1
	Min	-255.70	LC1	-1.37	LC1	0.65	LC1	0.01	LC1	-1.51	LC1	6.48	LC1
75%	Max	-255.70	LC1	0.09	LC1	0.65	LC1	0.01	LC1	1.42	LC1	9.38	LC1
	Min	-255.70	LC1	0.09	LC1	0.65	LC1	0.01	LC1	1.42	LC1	9.38	LC1
100%	Max	-255.70	LC1	1.54	LC1	0.65	LC1	0.01	LC1	4.35	LC1	5.71	LC1
	Min	-255.70	LC1	1.54	LC1	0.65	LC1	0.01	LC1	4.35	LC1	5.71	LC1

MEMBER 3

Station		Axial [Kip]	lc	Shear V2 [Kip]	lc	Shear V3 [Kip]	lc	Torsion [Kip*ft]	lc	M22 [Kip*ft]	lc	M33 [Kip*ft]	lc
0%	Max	-449.67	LC1	-4.33	LC1	0.62	LC1	0.01	LC1	-7.36	LC1	-20.98	LC1
	Min	-449.67	LC1	-4.33	LC1	0.62	LC1	0.01	LC1	-7.36	LC1	-20.98	LC1
25%	Max	-449.67	LC1	-2.88	LC1	0.62	LC1	0.01	LC1	-4.59	LC1	-4.76	LC1
	Min	-449.67	LC1	-2.88	LC1	0.62	LC1	0.01	LC1	-4.59	LC1	-4.76	LC1
50%	Max	-449.67	LC1	-1.42	LC1	0.62	LC1	0.01	LC1	-1.82	LC1	4.90	LC1
	Min	-449.67	LC1	-1.42	LC1	0.62	LC1	0.01	LC1	-1.82	LC1	4.90	LC1
75%	Max	-449.67	LC1	0.04	LC1	0.62	LC1	0.01	LC1	0.95	LC1	8.00	LC1
	Min	-449.67	LC1	0.04	LC1	0.62	LC1	0.01	LC1	0.95	LC1	8.00	LC1
100%	Max	-449.67	LC1	1.50	LC1	0.62	LC1	0.01	LC1	3.72	LC1	4.54	LC1
	Min	-449.67	LC1	1.50	LC1	0.62	LC1	0.01	LC1	3.72	LC1	4.54	LC1

MEMBER 4

Station		Axial [Kip]	lc	Shear V2 [Kip]	lc	Shear V3 [Kip]	lc	Torsion [Kip*ft]	lc	M22 [Kip*ft]	lc	M33 [Kip*ft]	lc
0%	Max	-288.38	LC1	-3.57	LC1	0.50	LC1	0.00	LC1	-5.73	LC1	-15.88	LC1
	Min	-288.38	LC1	-3.57	LC1	0.50	LC1	0.00	LC1	-5.73	LC1	-15.88	LC1
25%	Max	-288.38	LC1	-2.11	LC1	0.50	LC1	0.00	LC1	-3.49	LC1	-3.12	LC1
	Min	-288.38	LC1	-2.11	LC1	0.50	LC1	0.00	LC1	-3.49	LC1	-3.12	LC1
50%	Max	-288.38	LC1	-0.65	LC1	0.50	LC1	0.00	LC1	-1.24	LC1	3.09	LC1
	Min	-288.38	LC1	-0.65	LC1	0.50	LC1	0.00	LC1	-1.24	LC1	3.09	LC1
75%	Max	-288.38	LC1	0.81	LC1	0.50	LC1	0.00	LC1	1.00	LC1	2.74	LC1
	Min	-288.38	LC1	0.81	LC1	0.50	LC1	0.00	LC1	1.00	LC1	2.74	LC1
100%	Max	-288.38	LC1	2.26	LC1	0.50	LC1	0.00	LC1	3.24	LC1	-4.17	LC1
	Min	-288.38	LC1	2.26	LC1	0.50	LC1	0.00	LC1	3.24	LC1	-4.17	LC1

MEMBER **5**

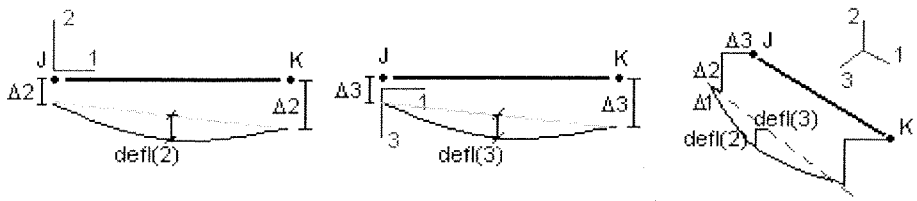
Station		Axial [Kip]	lc	Shear V2 [Kip]	lc	Shear V3 [Kip]	lc	Torsion [Kip*ft]	lc	M22 [Kip*ft]	lc	M33 [Kip*ft]	lc
0%	Max	-173.65	LC1	-3.66	LC1	0.15	LC1	0.01	LC1	-2.47	LC1	-15.66	LC1
	Min	-173.65	LC1	-3.66	LC1	0.15	LC1	0.01	LC1	-2.47	LC1	-15.66	LC1
25%	Max	-173.65	LC1	-2.20	LC1	0.15	LC1	0.01	LC1	-1.78	LC1	-2.46	LC1
	Min	-173.65	LC1	-2.20	LC1	0.15	LC1	0.01	LC1	-1.78	LC1	-2.46	LC1
50%	Max	-173.65	LC1	-0.75	LC1	0.15	LC1	0.01	LC1	-1.09	LC1	4.18	LC1
	Min	-173.65	LC1	-0.75	LC1	0.15	LC1	0.01	LC1	-1.09	LC1	4.18	LC1
75%	Max	-173.65	LC1	0.71	LC1	0.15	LC1	0.01	LC1	-0.39	LC1	4.27	LC1
	Min	-173.65	LC1	0.71	LC1	0.15	LC1	0.01	LC1	-0.39	LC1	4.27	LC1
100%	Max	-173.65	LC1	2.17	LC1	0.15	LC1	0.01	LC1	0.30	LC1	-2.21	LC1
	Min	-173.65	LC1	2.17	LC1	0.15	LC1	0.01	LC1	0.30	LC1	-2.21	LC1

Maximum forces at members

Condition : **LC1=DL+SLV1+SLV2+SL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-137.20	4.92	0.40	0.01	2.84	12.65
Min	-137.20	-0.91	0.40	0.01	-4.36	-24.70
MEMBER **2**						
Max	-255.70	1.54	0.65	0.01	4.35	9.38
Min	-255.70	-4.29	0.65	0.01	-7.36	-18.98
MEMBER **3**						
Max	-449.67	1.50	0.62	0.01	3.72	8.00
Min	-449.67	-4.33	0.62	0.01	-7.36	-20.98
MEMBER **4**						
Max	-288.38	2.26	0.50	0.00	3.24	3.74
Min	-288.38	-3.57	0.50	0.00	-5.73	-15.88
MEMBER **5**						
Max	-173.65	2.17	0.15	0.01	0.30	5.02
Min	-173.65	-3.66	0.15	0.01	-2.47	-15.66

Local deflections in members



Use definitions

Condition : **LC1=DL+SLV1+SLV2+SL**

Station	Axis 1 [in]	Axis 2 [in]	Axis 3 [in]	Rotation11 [Rad]	Defl. (2) [in]	Defl. (3) [in]
MEMBER **1**						
0%	0.076	-0.167	-0.011	-0.00012	-	-
25%	0.070	-0.192	-0.017	-0.00012	-0.03226 (L/6695)	-
50%	0.063	-0.179	-0.015	-0.00011	-0.02742 (L/7878)	-
75%	0.057	-0.147	-0.011	-0.00011	-	-
100%	0.051	-0.135	-0.009	-0.00010	-	-
MEMBER **2**						
0%	0.085	-0.169	-0.020	-0.00012	-	-
25%	0.073	-0.169	-0.031	-0.00012	-	-
50%	0.061	-0.180	-0.028	-0.00011	-0.02764 (L/7815)	-
75%	0.049	-0.172	-0.021	-0.00011	-0.02792 (L/7738)	-
100%	0.037	-0.136	-0.017	-0.00011	-	-
MEMBER **3**						
0%	0.032	-0.170	-0.030	-0.00012	-	-
25%	0.011	-0.162	-0.042	-0.00012	-	-
50%	-0.010	-0.171	-0.039	-0.00011	-	-
75%	-0.030	-0.165	-0.031	-0.00011	-	-
100%	-0.051	-0.136	-0.025	-0.00011	-	-
MEMBER **4**						
0%	-0.025	-0.171	-0.044	-0.00012	-	-
25%	-0.038	-0.160	-0.051	-0.00012	-	-
50%	-0.052	-0.160	-0.048	-0.00011	-	-
75%	-0.065	-0.152	-0.041	-0.00011	-	-
100%	-0.078	-0.137	-0.037	-0.00011	-	-
MEMBER **5**						
0%	-0.097	-0.171	-0.059	-0.00012	-	-
25%	-0.105	-0.165	-0.063	-0.00012	-	-
50%	-0.113	-0.167	-0.061	-0.00011	-	-
75%	-0.121	-0.158	-0.056	-0.00011	-	-
100%	-0.129	-0.136	-0.050	-0.00011	-	-

Maximum relative deflections

Remark.- Magnitude of deflections in absolute value.

CONDITION**LC1=DL+SLV1+SLV2+SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.03428	(L/6301)	32.50000	0.00661	(< L/10000)	30.00000
2	0.03122	(L/6920)	62.50000	0.01218	(< L/10000)	32.50000
3	0.02221	(L/9725)	65.00000	0.01371	(< L/10000)	32.50000
4	0.00785	(< L/10000)	63.52459	0.00983	(< L/10000)	31.96721
5	0.01475	(< L/10000)	62.50000	0.00718	(< L/10000)	40.00000

RAM International
File : Shawnee_rev4.AVW
Condition: LC3=DL+SLV1+SLV2+SL

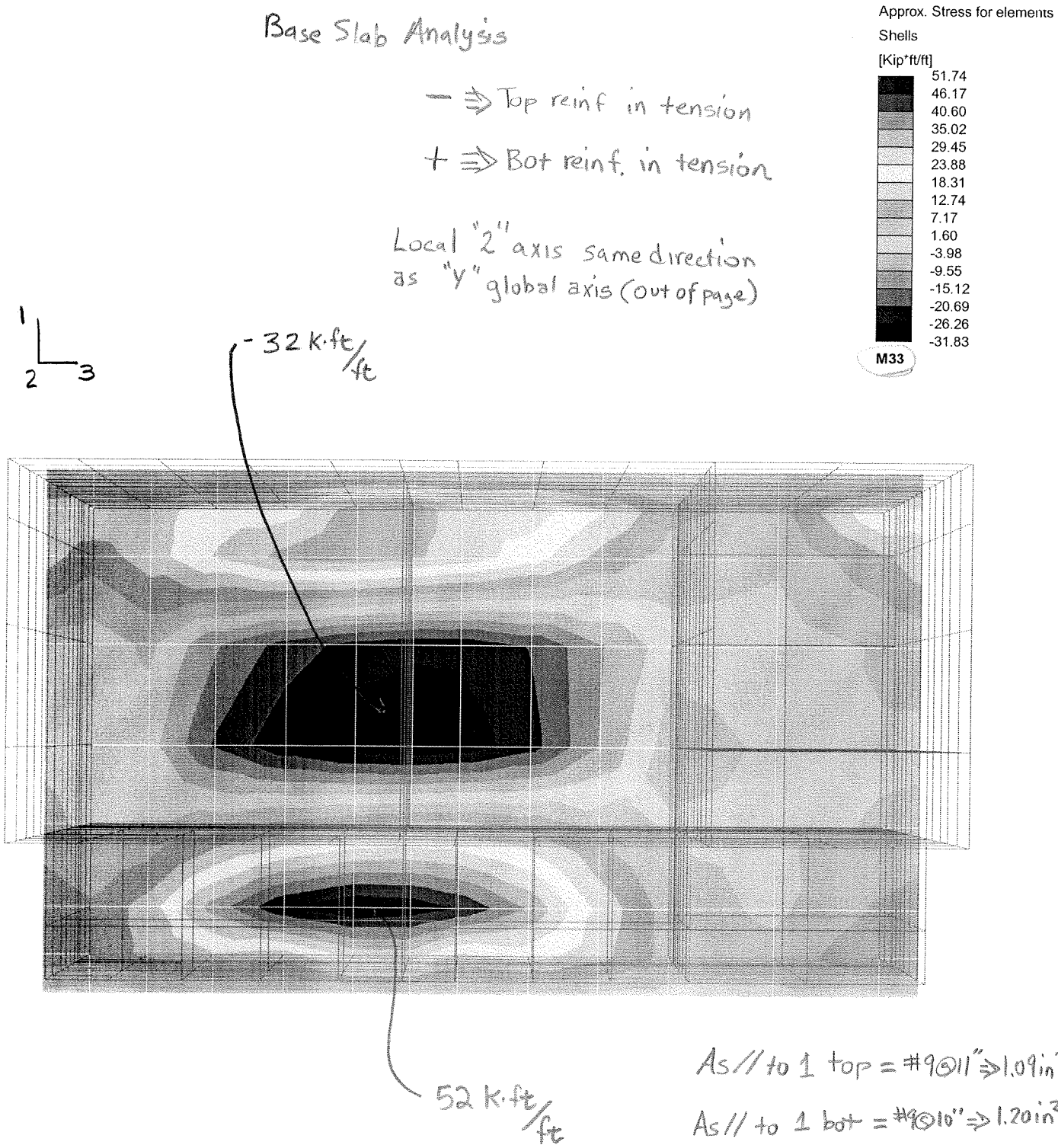
Analysis (1st order)
Date: 6/17/2006 9:42:02 PM

Base Slab Analysis

− ⇒ Top reinf in tension

+ ⇒ Bot reinf. in tension

Local "2" axis same direction as "y" global axis (out of page)



As // to 1 top = #9@11" ⇒ 1.09 in²

As // to 1 bot = #9@10" ⇒ 1.20 in²

RAM International
File : Shawnee_rev4.AVW
Condition: LC1=DL+SLV1+SLV2+SL+U

Analysis (1st order)
Date: 6/17/2006 9:33:07 PM

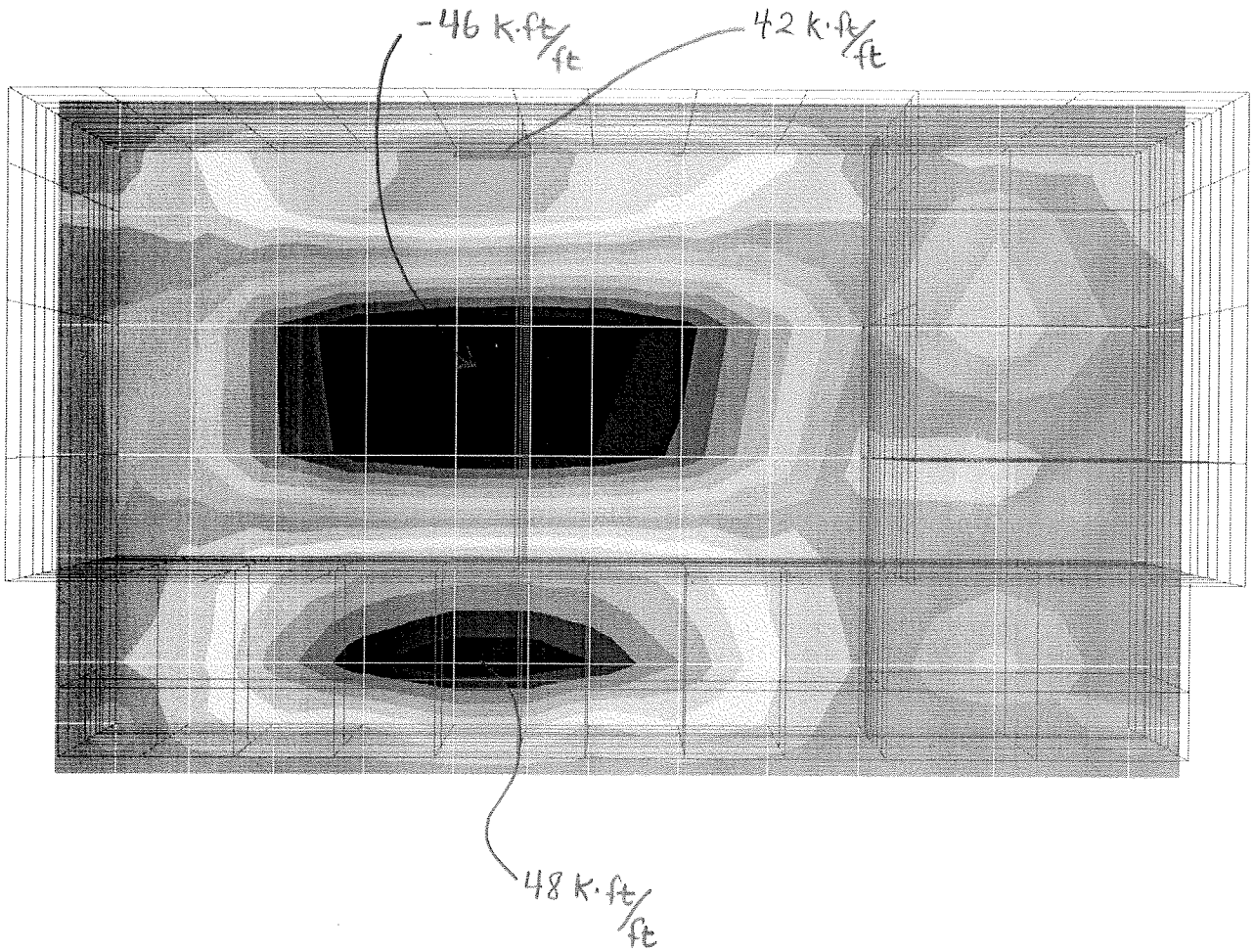
Base Slab

Approx. Stress for elements
Shells
[Kip*ft/ft]

47.22
41.05
34.89
28.73
22.56
16.40
10.24
4.07
-2.09
-8.25
-14.42
-20.58
-26.74
-32.91
-39.07
-45.24

M33

Local 2 axis same direction as the global Y axis

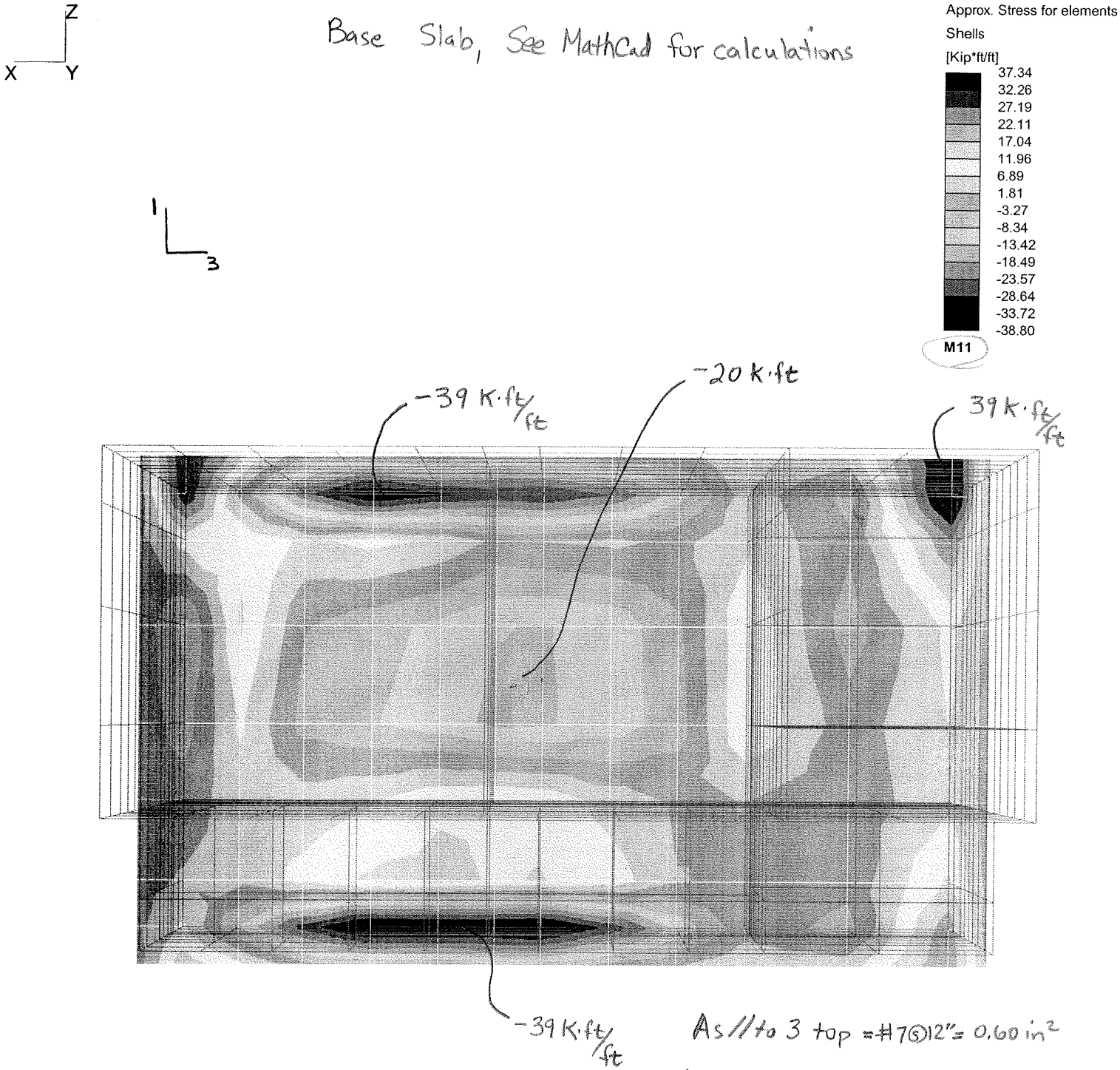


RAM International

File : Shawnee_rev4.AVW
Condition: LC3=DL+SLV1+SLV2+SL

Analysis (1st order)
Date: 6/17/2006 9:42:15 PM

Base Slab, See MathCad for calculations



As //to 3 top = #7@12" = 0.60 in²

As //to 3 bot = #7@12" = 0.60 in²

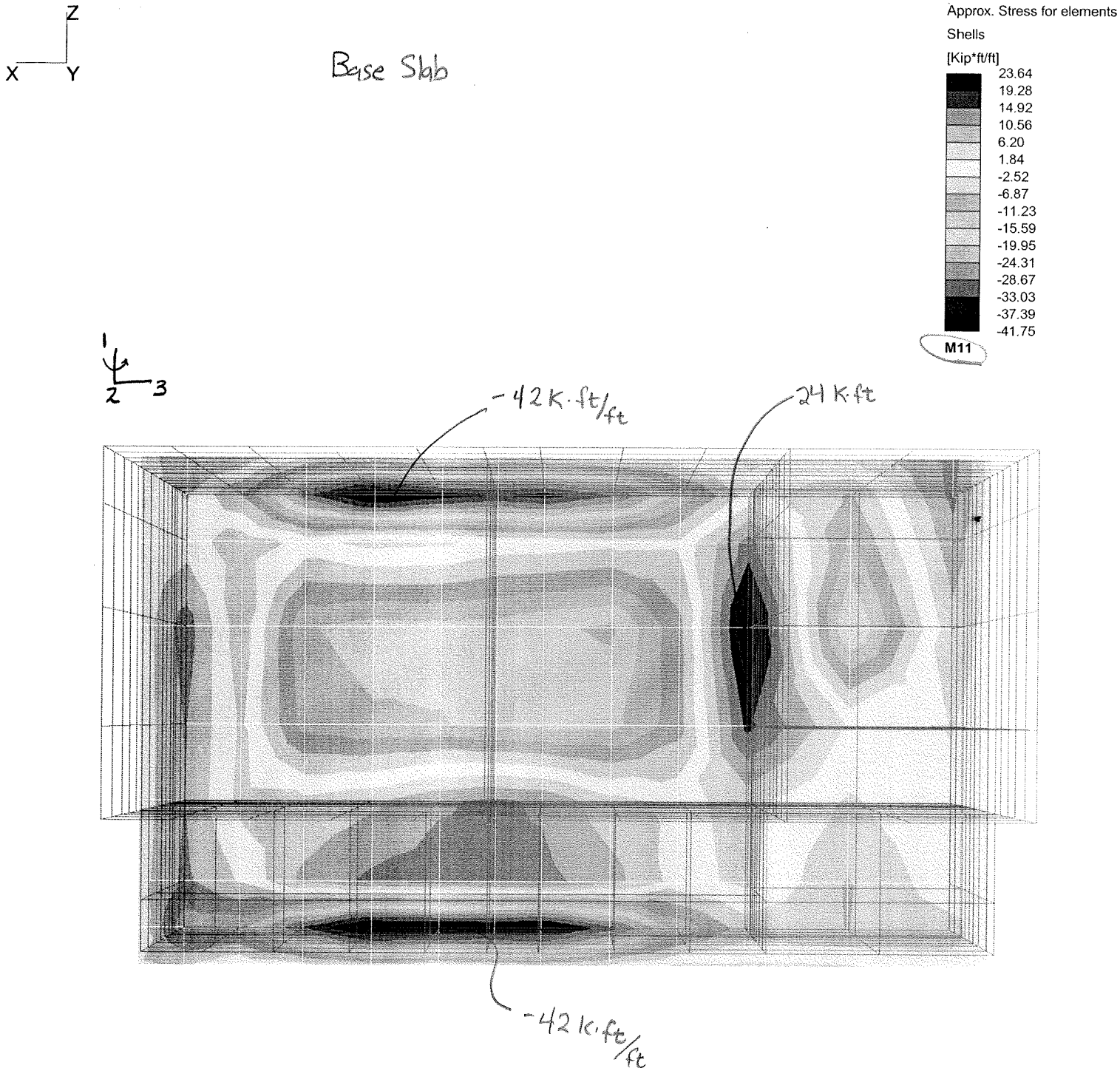
RAM International

File : Shawnee_rev4.AVW

Condition: LC1=DL+SLV1+SLV2+SL+U

Analysis (1st order)

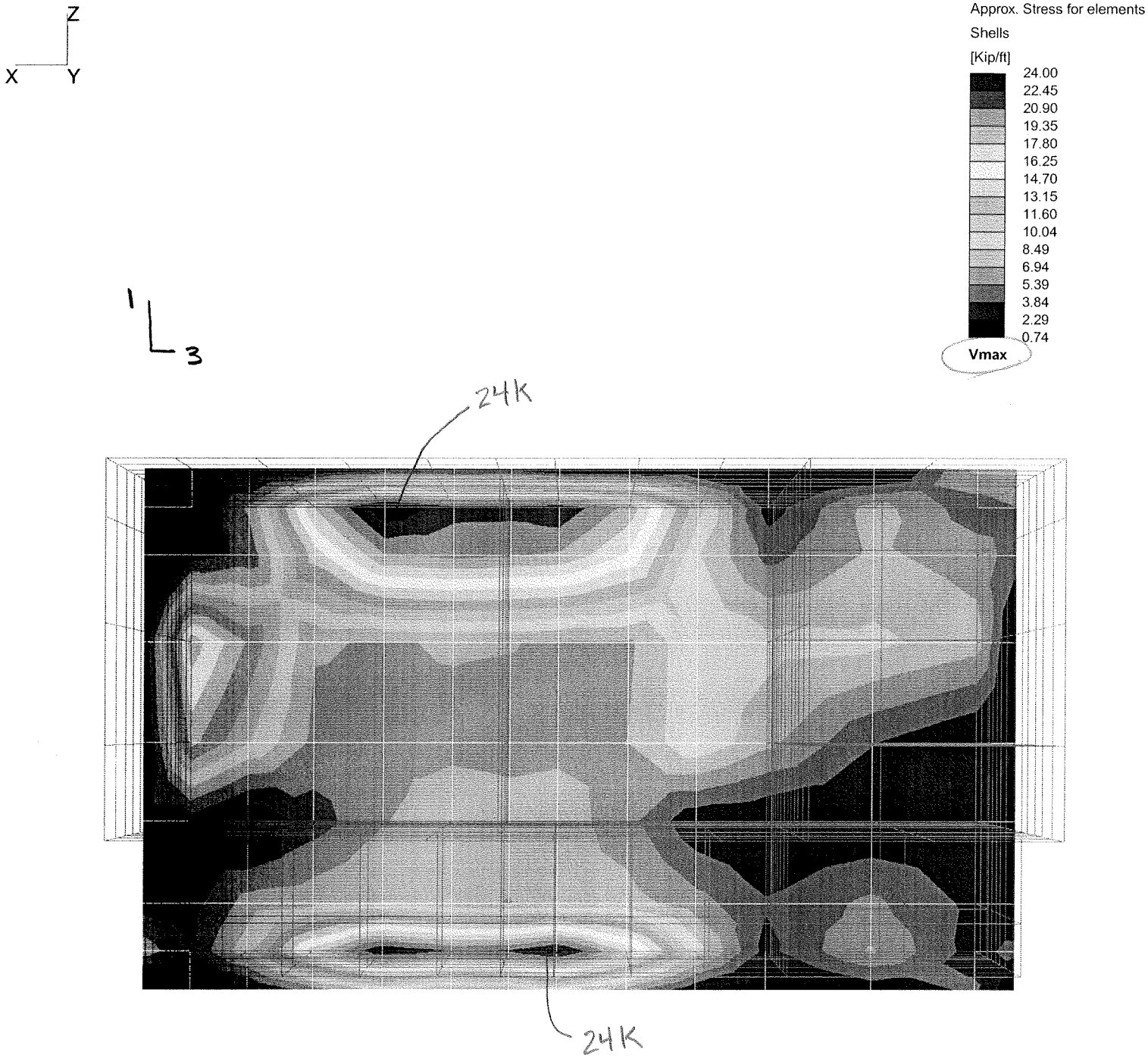
Date: 6/17/2006 9:33:24 PM



RAM International

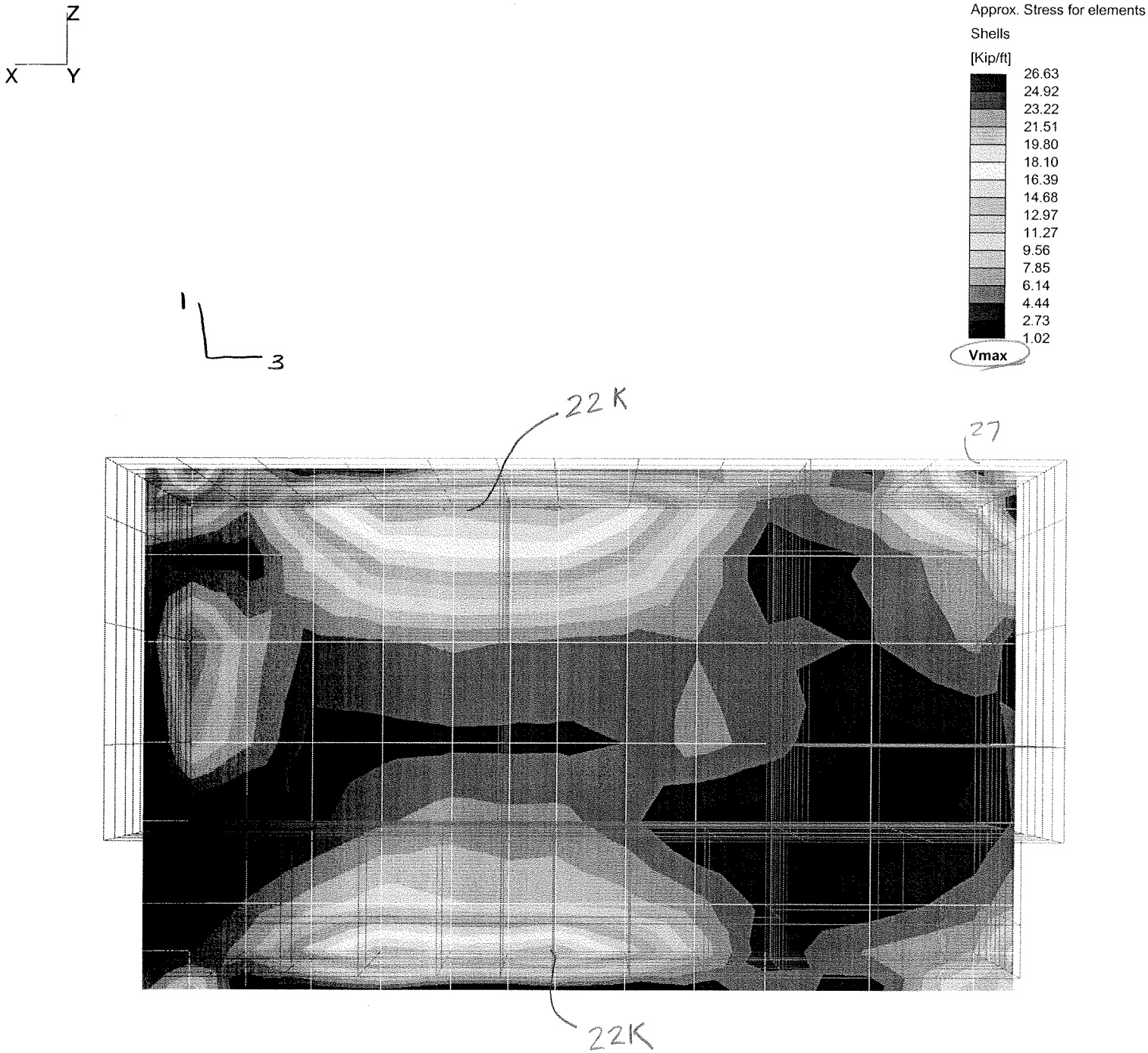
File : Shawnee_rev4.AVW
Condition: LC1=DL+SLV1+SLV2+SL+U

Analysis (1st order)
Date: 6/17/2006 10:29:26 PM



RAM International
File : Shawnee_rev4.AVW
Condition: LC3=DL+SLV1+SLV2+SL

Analysis (1st order)
Date: 6/17/2006 10:29:43 PM



KANSAS CITY LEVEES, ARMOURDALE UNIT

SHAWNEE AVENUE REINFORCED CONCRRETE BOX CULVERT (RCB) INVESTIGATION
NOTE: SCANNED IMAGES OF THE RCB ARE LOCATED AT THE END OF THIS DOCUMENT.

This document contains the investigation analysis that was completed on the RCB located at the Shawnee Pump Station. The RCB provides interior drainage of protected areas located landward of the line of protection. The RCB has an opening of 7.5 feet by 7.5 feet with 12 inch haunches. The RCB is located between the pump station and the outlet. The RCB does not have a flap gate on the outlet. A stamped copy of the Shawnee Pump Station drawings were used to determine the RCB design parameters. The RCB was constructed in the mid 1970s. The roof slab and sidewalls are 12 inches thick. The bottom slab is 13 inches thick. Number 9 reinforcement, spaced 9 inches on center, each face is used to provide strength against flexure. Number 5 longitudinal steel spaced 12 inches on center, each face was provided for temperature and shrinkage.

The RCB was visually inspected by EC-DS and EC-GC personnel in February 2007. The RCB did not exhibit indications of distress. However, a localized area of the box culvert had cracks that were leaking water as shown in Figure 1. The cracks are located approximately 25 feet from the outlet. All of the visible cracks were located in this area. The cracks go along the perimeter of the RCB (loop). No longitudinal cracks were observed. Since the cracks do not go longitudinal, it is not believed that the cracks were created as a result of external loading on the RCB.

The RCB was investigated for the design (n500+3) and existing condition events using CASE program CORTCUL. CORTCUL can be used for design or investigation of orthogonal, reinforced concrete culverts by either the working stress design (WSD) or strength design (SD) procedures. The user has the option of placing water in the conduit (internal water), vertical and horizontal load coefficients, and ground water elevation and surcharge.

For this investigation, the RCB was analyzed using SD to calculate the factors of safety shown in Table 3 for the loads and events described in Tables 1 and 2. Four CORTCUL model files were run (23078A thru 23078E). For each model, 3 load cases were investigated (total of 12 load cases). Table 3 shows only the events and load cases for which the structure did not not meet the minimum required factor of safety = 1.5.

CONCLUSIONS:
1. Investigation indicates that the RCB sidewalls and roof slab are inadequate in shear for the additional load created when the levee is raised 4.10 feet (n500+3 raise). This can be shown by the following simplified analysis*. Probability of failure analysis showed a reliability of 99.9% = R for the existing condition event roof shear (controls).

For the existing condition:
Shear demand on roof slab: 32.8 ft depth x 115 pcf = 3772 psf : 3772 psf x 9.5 ft = 35834 lbf = total load on roof slab. 35834 / 2 = 17.917 kips = Shear demand. (For load case 3 the shear demand = 1.5 x 17.9 kips = 26.8 kips.)

Shear capacity of roof slab: Assume no aid by axial thrust. 2 x sqrt 4000 x b x d = 12.75 kips = Vc where b=12 inches and d= 8.4.
Factor of Safety. Vc / V = 12.75 / 17.91 = 0.71 = Factor of Safety

* The above simplified calculation does not take into account the benefit provided by thrust and the haunches. Also, the shear demand is taken at the effective depth distance from the support face. Shear at d distance from the face will be lower. (The benefits provided by these parameters are taken into account with CORTCUL program)

2. The RCB is shown to be adequate for flexure for the existing and design (n500+3) events

3. BASED ON THIS ANALYSIS, IT IS RECOMMENDED THAT THE RCB UNDERGO REHABILITATION PRIOR TO PLACING ADDITIONAL EARTH LOADING ON THE STRUCTURE. THE REHABILITATION IS NECESSARY BECAUSE THE STRUCTURE DOES NOT PROVIDE ADEQUATE STRENGTH IN SHEAR. THE REHABILITATION RECOMMENDED IS A LINER MATERIAL PLACED INSIDE THE RCB. THE LINER WILL START APPROXIMATELY 50 FEET FROM THE OUTLET AND EXTEND TO THE PUMP STATION CONNECTION. ANNULAR AREAS CREATED BY THE LINER WILL BE FILLED WITH GROUT PER MANUFACTURER`S RECOMMENDATIONS.

Figure 1. Crack in RCB interior wall and roof slab leaking water. Picture was taken during site visit by EC-DS in Feb. 2007.

Table. 1. Shawnee Pump Station RCB Event Parameters Used in CORTCUL.

23078_	Event	Earth Depth[A], ft	Ground Water Location	Internal Water Depth[B], ft	Description
A	n500+3	36.9	RCB Invert	0	Local event. Storm flow drainage = 0 cfs. Levee embankment ground water at RCB invert elevation.
B	n500+3	36.9	Design Levee Crest	0	Local event. Storm flow drainage = 0 cfs. Levee embankment is saturated.
C	EC	32.8	RCB Invert	0	Local event. Storm flow drainage = 0 cfs. Levee embankment ground water at RCB invert elevation.
D	EC	32.8	Existing Levee Crest	0	Local event. Storm flow drainage = 0 cfs. Levee embankment is saturated.
E	n500+3	36.9	Design Levee Crest	7.5	Local event. RCB conduit full of water but not pressurized. Levee embankment is saturated.
G	EC	32.8	Existing Levee Crest	0	Same as D except the exterior reinforcement cover was reduced from 3.56 inches to 2.56 inches.
M	EC	32.8	Existing Levee Crest	0	Reliability analysis. Same as G except that concrete strength = 5 ksi (mean)
L	EC	32.8	Existing Levee Crest	0	Reliability analysis. Same as G except that concrete strength = 5.7 ksi (upper)
U	EC	32.8	Existing Levee Crest	0	Reliability analysis. Same as G except that concrete strength = 4.3 ksi (lower)

Notes:
A. Earth Depth = Levee crest elevation - Top of RCB roof elevation
B. Depth of water in the RCB
The Shawnee pump station conduit does not contain a flap gate. During a Kansas river flood event, the RCB will become pressurized with a head equal to the river elevation. This load case was not investigated and does not control because the internal water pressure created by the Kansas river will offset the external pressures created by the soil resulting in a lower net pressure acting on the RCB structure.
The RCB invert elevation = 724.00 ft. The existing condition top of levee elevation is 765.30. The n500+3 levee elevation is 769.40.
Files 23078M, L and U performed for reliability and probability of shear failure analysis for the existing condition event. Note that the steel strength was not changed because moment strength is okay.

Table 2. RCB Load Cases (LC) Investigated

LC	Vert. Coef.	Hor. Coef.	LC Description
1	1.00	0.40	Trench with active lateral pressure
2	1.00	0.57	Trench with at-rest lateral pressure (phi=26 degrees)
3	1.50	0.50	Embankment

Notes:
1. The embankment condition whereby the vertical coef. = 1.5 and horizontal coef. = 1.0 was not investigated. This load case was not investigated because the lateral earth pressures represented by a horizontal coef. = 1.0 have dissipated since construction. However, the vertical coef. = 1.50 could still be present after construction.
2. For all load cases the live load coef. = dead load coef. = 1.0
3. For all load cases the saturated unit weight = 120 pcf and moist unit weight = 115 pcf.
4. The 3 oad cases shown in Table 2 were investigated in each CORTCUL file.

Table 3. Shawnee Pump Station RCB RCB Investigation Significant Findings

CORTCUL File	Event	Load Case(s)	Member	Strength Parm.	FS	Status
23078A	n500+3	3	Roof	Shear	1.24	No Good
23078B	n500+3	1	Sidewalls	Shear	1.42	No Good
		2	Sidewalls		1.28	No Good
		3	Sidewalls		1.34	No Good
		1, 2	Roof		1.13	No Good
		3	Roof		0.92	No Good
23078C	EC	2	Roof	Shear	1.32	No Good
		3	Roof		1.36	No Good
23078D	EC	2	Sidewalls	Shear	1.41	No Good
		3	Sidewalls		1.48	Borderline
		1,2	Roof		1.26	No Good
		3	Roof		1.02	No Good
23078E	n500+3	2	Sidewalls	Shear	1.38	No Good
		3	Sidewalls		1.44	No Good
		1,2	Roof		1.13	No Good
		3	Roof		0.92	No Good
23078G	EC	1,2	Roof	Shear	1.4	No Good
		3	Roof		1.14	No Good
23078M	EC	3	Roof	Shear	1.25	No Good
23078U	EC	3	Roof	Shear	1.32	No Good
23078L	EC	1,2	Roof	Shear	1.44	No Good
		3	Roof		1.16	No Good

Notes:
1. Table 3 summarizes information obtained from CORTCUL software output. Refer to Tables 1 and 2 for details regarding the loading and load cases.
2. Factors of safety (FS) above 1.50 for the load cases and event investigated are not shown in Table 3.
3. Shear capacity determined by CORTCUL using ACI 318-02 equations 11-5 through 11-7. These equations are for shear strength provided by concrete when subjected to axial compression.
4. Moment capacity and demand determined using CORTCUL software.
4. Concrete strength used in analaysis = f`c = 4000 psi. Steel yield strength = 60,000 psi
5. Factors of safety for moment were greater than 1.5 for all load cases and events investigated.
6. 23078M, U and L were done for reliability purposes. Only the concrete strength was changed because the RCB is showing FS < 1.5 for shear only.

```
1000 4 23078A SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
1010   LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=724.0
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  4000.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     3.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   769.40   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   724.00
1130     1.00      .57      .00   724.00
1140     1.50      .50      .00   724.00
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1   1.33   1.33    1.33    .0    1.33   1.33
1200  11   1.33   1.33    1.33   1.33   1.33   1.33
1210  12   1.33   1.33    1.33   1.33   1.33   1.33
1220  21   1.33   1.33     .0    1.33   1.33   1.33
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/31/2007                                TIME: 16:12:27
```

CONCLUSIONS:
1. LOAD CASE 3, MEMBER 21 FS = 1.24
MEMBER INADEQUATE FOR LOAD
CASE 3 DEMAND LOAD.

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/31/2007                                TIME: 16:12:38

1. INPUT DATA

1.A.--HEADING

23078A SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=724.0
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  4000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
               (FT)      (IN)        (FT)       (FT)
       1        7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	769.40	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	724.00
2	1.00	.57	.00	724.00
3	1.50	.50	.00	724.00

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 3/31/2007 TIME: 16:12:41

2.A.--HEADING

23078A SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST EVENT:N500+3 IWAT=0 GWATEL=724.0
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	12.73	-19.83	12.73
AXIAL FORCE	(KIPS)	9.62	9.62	9.62
FLEXURE FACTOR OF SAFETY		17.07	3.06	17.07
STRENGTH REDUCTION (PHI)		.70	.81	.70
SHEAR FORCE AT D	(KIPS)	-13.53		13.53
SHEAR FS AT D (ACI63)		1.85		1.85
SHEAR FORCE AT 0.15LN	(KIPS)	-12.15		12.15
SHEAR FS AT 0.15LN (UI440)		2.05		2.05

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-17.46	-4.35	-18.17
AXIAL FORCE	(KIPS)	21.91	21.35	20.79
FLEXURE FACTOR OF SAFETY		11.76	12.79	11.46
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	5.76		-5.75
SHEAR FS AT D (ACI63)		2.29		2.21
SHEAR FORCE AT 0.15LN	(KIPS)	4.99		-5.07
SHEAR FS AT 0.15LN (UI440)		1.31		1.28

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	17.46	4.35	18.17
AXIAL FORCE	(KIPS)	21.91	21.35	20.79
FLEXURE FACTOR OF SAFETY		11.76	12.79	11.46

STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-5.76		5.75
SHEAR FS AT D (ACI63)	2.29		2.21
SHEAR FORCE AT 0.15LN (KIPS)	-4.99		5.07
SHEAR FS AT 0.15LN (UI440)	1.31		1.28

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-13.47	17.42	-13.47
AXIAL FORCE (KIPS)	8.78	8.78	8.78
FLEXURE FACTOR OF SAFETY	14.96	3.07	14.96
STRENGTH REDUCTION (PHI)	.70	.81	.70
SHEAR FORCE AT D (KIPS)	13.20		-13.20
SHEAR FS AT D (ACI63)	1.53		1.53
SHEAR FORCE AT 0.15LN (KIPS)	11.53		-11.53
SHEAR FS AT 0.15LN (UI440)	1.33		1.33

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	16.04	-16.51	16.04
AXIAL FORCE (KIPS)	13.78	13.78	13.78
FLEXURE FACTOR OF SAFETY	13.77	3.76	13.77
STRENGTH REDUCTION (PHI)	.70	.73	.70
SHEAR FORCE AT D (KIPS)	-13.53		13.53
SHEAR FS AT D (ACI63)	1.71		1.71
SHEAR FORCE AT 0.15LN (KIPS)	-12.15		12.15
SHEAR FS AT 0.15LN (UI440)	2.12		2.12

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.05	-.03	-19.41
AXIAL FORCE (KIPS)	21.91	21.35	20.79
FLEXURE FACTOR OF SAFETY	10.92	16.79	10.83
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	8.30		-8.11
SHEAR FS AT D (ACI63)	1.59		1.62
SHEAR FORCE AT 0.15LN (KIPS)	7.19		-7.14
SHEAR FS AT 0.15LN (UI440)	.91		.91

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.05	.03	19.41
AXIAL FORCE (KIPS)	21.91	21.35	20.79
FLEXURE FACTOR OF SAFETY	10.92	16.79	10.83
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-8.30		8.11
SHEAR FS AT D (ACI63)	1.59		1.62
SHEAR FORCE AT 0.15LN (KIPS)	-7.19		7.14
SHEAR FS AT 0.15LN (UI440)	.91		.91

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-16.16	14.73	-16.16
AXIAL FORCE (KIPS)	12.43	12.43	12.43
FLEXURE FACTOR OF SAFETY	12.68	3.68	12.68
STRENGTH REDUCTION (PHI)	.70	.74	.70
SHEAR FORCE AT D (KIPS)	13.20		-13.20
SHEAR FS AT D (ACI63)	1.56		1.56
SHEAR FORCE AT 0.15LN (KIPS)	11.53		-11.53

```
     SHEAR FS AT 0.15LN (UI440)          .52                                .52
```

1.28/0.85 = 1.51
OKAY

```
2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       16.81        -30.67          16.81
     AXIAL FORCE           (KIPS)       11.90         11.90          11.90
     FLEXURE FACTOR OF SAFETY           12.81          1.96          12.81
     STRENGTH REDUCTION  (PHI)            .70           .83            .70
     SHEAR FORCE AT D      (KIPS)      -19.73                        19.73
     SHEAR FS AT D (ACI63)              1.28                         1.28
     SHEAR FORCE AT 0.15LN (KIPS)      -17.72                        17.72
     SHEAR FS AT 0.15LN (UI440)         1.43                         1.43


MEMBER 11                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      -24.59         -8.63         -26.35
     AXIAL FORCE           (KIPS)       31.99         31.43          30.87
     FLEXURE FACTOR OF SAFETY            8.30          7.00           7.88
     STRENGTH REDUCTION  (PHI)            .70           .70            .70
     SHEAR FORCE AT D      (KIPS)        7.09                        -7.31
     SHEAR FS AT D (ACI63)              1.82                         1.68
     SHEAR FORCE AT 0.15LN (KIPS)       6.12                        -6.46
     SHEAR FS AT 0.15LN (UI440)         1.14                         1.08


MEMBER 12                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       24.59          8.63          26.35
     AXIAL FORCE           (KIPS)       31.99         31.43          30.87
     FLEXURE FACTOR OF SAFETY            8.30          7.00           7.88
     STRENGTH REDUCTION  (PHI)            .70           .70            .70
     SHEAR FORCE AT D      (KIPS)       -7.09                         7.31
     SHEAR FS AT D (ACI63)              1.82                         1.68
     SHEAR FORCE AT 0.15LN (KIPS)       -6.12                         6.46
     SHEAR FS AT 0.15LN (UI440)         1.14                         1.08


MEMBER 21                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      -18.61         27.20         -18.61
     AXIAL FORCE           (KIPS)       11.10         11.10          11.10
     FLEXURE FACTOR OF SAFETY           10.73          1.95          10.73
     STRENGTH REDUCTION  (PHI)            .70           .82            .70
     SHEAR FORCE AT D      (KIPS)       19.58                       -19.58
     SHEAR FS AT D (ACI63)              1.05                         1.05
     SHEAR FORCE AT 0.15LN (KIPS)       17.10                       -17.10
     SHEAR FS AT 0.15LN (UI440)           .92                          .92
```

1.05/0.85 = 1.24 NO GOOD

```
23078A SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=724.0
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


3.B.--RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
```

OUTPUT COMPLETE.

```
1000 4 23078B SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
1010   LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=769.40
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  4000.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     3.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   769.40   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   769.40
1130     1.00      .57      .00   769.40
1140     1.50      .50      .00   769.40
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1   1.33   1.33    1.33    .0    1.33   1.33
1200  11   1.33   1.33    1.33   1.33   1.33   1.33
1210  12   1.33   1.33    1.33   1.33   1.33   1.33
1220  21   1.33   1.33     .0    1.33   1.33   1.33
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/31/2007                                TIME: 17:14:56
```

CONCLUSIONS:
SIDEWALL AND BOTTOM MEMBERS FAIL TO MEET THE
REQUIRED FS FOR ALL LOAD CASES ANALYZED.

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/31/2007                                TIME: 17:15: 4

1. INPUT DATA

1.A.--HEADING

23078B SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=769.40
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  4000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
               (FT)      (IN)        (FT)       (FT)
       1        7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	769.40	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	769.40
2	1.00	.57	.00	769.40
3	1.50	.50	.00	769.40

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/31/2007                                TIME:  17:15: 7

2.A.--HEADING

23078B SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=769.40
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      19.60        -14.25          19.60
     AXIAL FORCE           (KIPS)      17.98         17.98          17.98
     FLEXURE FACTOR OF SAFETY          11.36          4.59          11.36
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     -14.07                        14.07
     SHEAR FS AT D (ACI63)              1.68                         1.68
     SHEAR FORCE AT 0.15LN (KIPS)     -12.64                        12.64
     SHEAR FS AT 0.15LN (UI440)         1.30                         1.30


MEMBER 11                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     -21.20          3.77         -21.31
     AXIAL FORCE           (KIPS)      22.79         22.23          21.66
     FLEXURE FACTOR OF SAFETY           9.91         12.67           9.87
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)      10.86                       -10.52
     SHEAR FS AT D (ACI63)              1.21                         1.25
     SHEAR FORCE AT 0.15LN (KIPS)       9.42                        -9.26
     SHEAR FS AT 0.15LN (UI440)          .70                          .70


MEMBER 12                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      21.20         -3.77          21.31
     AXIAL FORCE           (KIPS)      22.79         22.23          21.66
     FLEXURE FACTOR OF SAFETY           9.91         12.67           9.87
```

STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.86		10.52
SHEAR FS AT D (ACI63)	1.21		1.25
SHEAR FORCE AT 0.15LN (KIPS)	-9.42		9.26
SHEAR FS AT 0.15LN (UI440)	.70		.70

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.21	12.97	-19.21
AXIAL FORCE (KIPS)	16.15	16.15	16.15
FLEXURE FACTOR OF SAFETY	10.76	4.30	10.76
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	13.76		-13.76
SHEAR FS AT D (ACI63)	.96		.96
SHEAR FORCE AT 0.15LN (KIPS)	12.02		-12.02
SHEAR FS AT 0.15LN (UI440)	.52		.52

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	21.26	-12.59	21.26
AXIAL FORCE (KIPS)	20.07	20.07	20.07
FLEXURE FACTOR OF SAFETY	10.52	5.22	10.52
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-14.07		14.07
SHEAR FS AT D (ACI63)	1.70		1.70
SHEAR FORCE AT 0.15LN (KIPS)	-12.64		12.64
SHEAR FS AT 0.15LN (UI440)	1.32		1.32

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-21.99	5.93	-21.93
AXIAL FORCE (KIPS)	22.79	22.23	21.66
FLEXURE FACTOR OF SAFETY	9.57	9.57	9.57
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.13		-11.70
SHEAR FS AT D (ACI63)	1.09		1.13
SHEAR FORCE AT 0.15LN (KIPS)	10.53		-10.30
SHEAR FS AT 0.15LN (UI440)	.62		.63

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	21.99	-5.93	21.93
AXIAL FORCE (KIPS)	22.79	22.23	21.66
FLEXURE FACTOR OF SAFETY	9.57	9.57	9.57
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-12.13		11.70
SHEAR FS AT D (ACI63)	1.09		1.13
SHEAR FORCE AT 0.15LN (KIPS)	-10.53		10.30
SHEAR FS AT 0.15LN (UI440)	.62		.63

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-20.56	11.63	-20.56
AXIAL FORCE (KIPS)	17.98	17.98	17.98
FLEXURE FACTOR OF SAFETY	10.10	4.84	10.10
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	13.76		-13.76
SHEAR FS AT D (ACI63)	.96		.96
SHEAR FORCE AT 0.15LN (KIPS)	12.02		-12.02

```
     SHEAR FS AT 0.15LN (UI440)          .53                          .53


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     21.64        -19.68          21.64
     AXIAL FORCE           (KIPS)     19.13         19.13          19.13
     FLEXURE FACTOR OF SAFETY         10.25          3.15          10.25
     STRENGTH REDUCTION  (PHI)          .70           .71            .70
     SHEAR FORCE AT D      (KIPS)    -17.17                        17.17
     SHEAR FS AT D (ACI63)             1.39                         1.39
     SHEAR FORCE AT 0.15LN (KIPS)    -15.43                        15.43
     SHEAR FS AT 0.15LN (UI440)        1.08                         1.08


MEMBER 11                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)    -24.77          1.62         -25.41
     AXIAL FORCE           (KIPS)     27.84         27.27          26.71
     FLEXURE FACTOR OF SAFETY          8.43         13.14           8.29
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     11.53                       -11.30
     SHEAR FS AT D (ACI63)             1.14                         1.17
     SHEAR FORCE AT 0.15LN (KIPS)      9.99                        -9.96
     SHEAR FS AT 0.15LN (UI440)         .68                          .68


MEMBER 12                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     24.77         -1.62          25.41
     AXIAL FORCE           (KIPS)     27.84         27.27          26.71
     FLEXURE FACTOR OF SAFETY          8.43         13.14           8.29
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)    -11.53                        11.30
     SHEAR FS AT D (ACI63)             1.14                         1.17
     SHEAR FORCE AT 0.15LN (KIPS)      -9.99                        9.96
     SHEAR FS AT 0.15LN (UI440)         .68                          .68


MEMBER 21                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)    -21.79         17.87         -21.79
     AXIAL FORCE           (KIPS)     17.31         17.31          17.31
     FLEXURE FACTOR OF SAFETY          9.44          3.03           9.44
     STRENGTH REDUCTION  (PHI)          .70           .72            .70
     SHEAR FORCE AT D      (KIPS)     16.95                       -16.95
     SHEAR FS AT D (ACI63)              .78                          .78
     SHEAR FORCE AT 0.15LN (KIPS)      14.81                       -14.81
     SHEAR FS AT 0.15LN (UI440)         .43                          .43
```

0.78/0.85 = 0.92

```
23078B SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=769.40
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


3.B.--RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
```

OUTPUT COMPLETE.

```
1000 4 23078C SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
1010   LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=724.00
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  4000.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     3.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   765.30   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   724.00
1130     1.00      .57      .00   724.00
1140     1.50      .50      .00   724.00
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1   1.33   1.33    1.33    .0    1.33   1.33
1200  11   1.33   1.33    1.33   1.33   1.33   1.33
1210  12   1.33   1.33    1.33   1.33   1.33   1.33
1220  21   1.33   1.33     .0    1.33   1.33   1.33
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/31/2007                                TIME: 17:30:23
```

CONCLUSIONS:
THE RCB ROOF FS = 1.32 FOR LOAD CASE 2
(CONTROLS)

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/31/2007                                TIME: 17:30:35

1. INPUT DATA

1.A.--HEADING

23078C SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=724.00
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  4000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
                (FT)      (IN)        (FT)       (FT)
       1         7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	765.30	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	724.00
2	1.00	.57	.00	724.00
3	1.50	.50	.00	724.00

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 3/31/2007 TIME: 17:30:37

2.A.--HEADING

23078C SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=724.00
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	11.50	-17.75	11.50
AXIAL FORCE	(KIPS)	8.73	8.73	8.73
FLEXURE FACTOR OF SAFETY		18.91	3.43	18.91
STRENGTH REDUCTION (PHI)		.70	.80	.70
SHEAR FORCE AT D	(KIPS)	-12.15		12.15
SHEAR FS AT D (ACI63)		1.85		1.85
SHEAR FORCE AT 0.15LN	(KIPS)	-10.92		10.92
SHEAR FS AT 0.15LN (UI440)		2.27		2.27

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-15.71	-3.83	-16.24
AXIAL FORCE	(KIPS)	19.67	19.11	18.55
FLEXURE FACTOR OF SAFETY		13.07	14.42	12.83
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	5.22		-5.16
SHEAR FS AT D (ACI63)		2.52		2.47
SHEAR FORCE AT 0.15LN	(KIPS)	4.52		-4.55
SHEAR FS AT 0.15LN (UI440)		1.42		1.40

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	15.71	3.83	16.24
AXIAL FORCE	(KIPS)	19.67	19.11	18.55
FLEXURE FACTOR OF SAFETY		13.07	14.42	12.83

STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-5.22		5.16
SHEAR FS AT D (ACI63)	2.52		2.47
SHEAR FORCE AT 0.15LN (KIPS)	-4.52		4.55
SHEAR FS AT 0.15LN (UI440)	1.42		1.40

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-12.05	15.52	-12.05
AXIAL FORCE (KIPS)	7.86	7.86	7.86
FLEXURE FACTOR OF SAFETY	16.72	3.45	16.72
STRENGTH REDUCTION (PHI)	.70	.81	.70
SHEAR FORCE AT D (KIPS)	11.79		-11.79
SHEAR FS AT D (ACI63)	1.70		1.70
SHEAR FORCE AT 0.15LN (KIPS)	10.30		-10.30
SHEAR FS AT 0.15LN (UI440)	1.48		1.48

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	14.49	-14.76	14.49
AXIAL FORCE (KIPS)	12.50	12.50	12.50
FLEXURE FACTOR OF SAFETY	15.26	4.21	15.26
STRENGTH REDUCTION (PHI)	.70	.73	.70
SHEAR FORCE AT D (KIPS)	-12.15		12.15
SHEAR FS AT D (ACI63)	1.89		1.89
SHEAR FORCE AT 0.15LN (KIPS)	-10.92		10.92
SHEAR FS AT 0.15LN (UI440)	2.34		2.34

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-17.14	.06	-17.35
AXIAL FORCE (KIPS)	19.67	19.11	18.55
FLEXURE FACTOR OF SAFETY	12.14	18.76	12.11
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	7.51		-7.28
SHEAR FS AT D (ACI63)	1.75		1.81
SHEAR FORCE AT 0.15LN (KIPS)	6.51		-6.42
SHEAR FS AT 0.15LN (UI440)	.98		.99

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	17.14	-.06	17.35
AXIAL FORCE (KIPS)	19.67	19.11	18.55
FLEXURE FACTOR OF SAFETY	12.14	18.76	12.11
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-7.51		7.28
SHEAR FS AT D (ACI63)	1.75		1.81
SHEAR FORCE AT 0.15LN (KIPS)	-6.51		6.42
SHEAR FS AT 0.15LN (UI440)	.98		.99

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-14.48	13.10	-14.48
AXIAL FORCE (KIPS)	11.13	11.13	11.13
FLEXURE FACTOR OF SAFETY	14.16	4.14	14.16
STRENGTH REDUCTION (PHI)	.70	.74	.70
SHEAR FORCE AT D (KIPS)	11.79		-11.79
SHEAR FS AT D (ACI63)	1.12		1.12
SHEAR FORCE AT 0.15LN (KIPS)	10.30		-10.30

1.12/0.85 = 1.32

SHEAR FS AT 0.15LN (UI440)	.58	.58

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	15.15	-27.36	15.15
AXIAL FORCE (KIPS)	10.80	10.80	10.80
FLEXURE FACTOR OF SAFETY	14.23	2.20	14.23
STRENGTH REDUCTION (PHI)	.70	.82	.70
SHEAR FORCE AT D (KIPS)	-17.66		17.66
SHEAR FS AT D (ACI63)	1.42		1.42
SHEAR FORCE AT 0.15LN (KIPS)	-15.87		15.87
SHEAR FS AT 0.15LN (UI440)	1.59		1.59

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-22.06	-7.61	-23.51
AXIAL FORCE (KIPS)	28.63	28.07	27.51
FLEXURE FACTOR OF SAFETY	9.25	7.92	8.83
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	6.42		-6.56
SHEAR FS AT D (ACI63)	2.02		1.87
SHEAR FORCE AT 0.15LN (KIPS)	5.54		-5.80
SHEAR FS AT 0.15LN (UI440)	1.23		1.17

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	22.06	7.61	23.51
AXIAL FORCE (KIPS)	28.63	28.07	27.51
FLEXURE FACTOR OF SAFETY	9.25	7.92	8.83
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-6.42		6.56
SHEAR FS AT D (ACI63)	2.02		1.87
SHEAR FORCE AT 0.15LN (KIPS)	-5.54		5.80
SHEAR FS AT 0.15LN (UI440)	1.23		1.17

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-16.64	24.20	-16.64
AXIAL FORCE (KIPS)	9.94	9.94	9.94
FLEXURE FACTOR OF SAFETY	12.01	2.20	12.01
STRENGTH REDUCTION (PHI)	.70	.82	.70
SHEAR FORCE AT D (KIPS)	17.45		-17.45
SHEAR FS AT D (ACI63)	1.16		1.16
SHEAR FORCE AT 0.15LN (KIPS)	15.25		-15.25
SHEAR FS AT 0.15LN (UI440)	1.02		1.02

23078C SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=724.00
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

3.B.--RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

OUTPUT COMPLETE.

```
1000 4 23078D SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
1010   LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  4000.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     3.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   765.30   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   765.30
1130     1.00      .57      .00   765.30
1140     1.50      .50      .00   765.30
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1   1.33   1.33    1.33    .0    1.33   1.33
1200  11   1.33   1.33    1.33   1.33   1.33   1.33
1210  12   1.33   1.33    1.33   1.33   1.33   1.33
1220  21   1.33   1.33     .0    1.33   1.33   1.33
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/ 1/2007                                TIME:  9:29: 7
```

CONCLUSIONS:
SIDEWALL FS = 1.41 FOR LC 2 AND FS = 1.48 FOR LC 3
ROOF FS<1.5 FOR ALL 3 LC. THE LOWEST FS IS 1.02 FOR LC 3

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/ 1/2007                                TIME:  9:29:19

1. INPUT DATA

1.A.--HEADING

23078D SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  4000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
                (FT)      (IN)        (FT)       (FT)
       1         7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	765.30	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	765.30
2	1.00	.57	.00	765.30
3	1.50	.50	.00	765.30

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 4/ 1/2007 TIME: 9:29:21

2.A.--HEADING

23078D SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	17.69	-12.70	17.69
AXIAL FORCE	(KIPS)	16.31	16.31	16.31
FLEXURE FACTOR OF SAFETY		12.60	5.17	12.60
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-12.63		12.63
SHEAR FS AT D (ACI63)		1.86		1.86
SHEAR FORCE AT 0.15LN	(KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)		1.43		1.43

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-19.07	3.49	-19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		11.02	13.94	11.04
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	9.83		-9.45
SHEAR FS AT D (ACI63)		1.34		1.39
SHEAR FORCE AT 0.15LN	(KIPS)	8.52		-8.33
SHEAR FS AT 0.15LN (UI440)		.76		.77

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	19.07	-3.49	19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		11.02	13.94	11.04

STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-9.83		9.45
SHEAR FS AT D (ACI63)	1.34		1.39
SHEAR FORCE AT 0.15LN (KIPS)	-8.52		8.33
SHEAR FS AT 0.15LN (UI440)	.76		.77

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-17.22	11.51	-17.22
AXIAL FORCE (KIPS)	14.47	14.47	14.47
FLEXURE FACTOR OF SAFETY	12.01	4.86	12.01
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.28		-12.28
SHEAR FS AT D (ACI63)	1.07		1.07
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73
SHEAR FS AT 0.15LN (UI440)	.57		.57

1.07/0.85 = 1.26

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.19	-11.20	19.19
AXIAL FORCE (KIPS)	18.20	18.20	18.20
FLEXURE FACTOR OF SAFETY	11.66	5.85	11.66
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-12.63		12.63
SHEAR FS AT D (ACI63)	1.87		1.87
SHEAR FORCE AT 0.15LN (KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)	1.45		1.45

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.79	5.44	-19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	10.64	10.47	10.69
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.98		-10.52
SHEAR FS AT D (ACI63)	1.20		1.25
SHEAR FORCE AT 0.15LN (KIPS)	9.51		-9.26
SHEAR FS AT 0.15LN (UI440)	.68		.69

1.20/0.85 = 1.41

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.79	-5.44	19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	10.64	10.47	10.69
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.98		10.52
SHEAR FS AT D (ACI63)	1.20		1.25
SHEAR FORCE AT 0.15LN (KIPS)	-9.51		9.26
SHEAR FS AT 0.15LN (UI440)	.68		.69

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-18.44	10.29	-18.44
AXIAL FORCE (KIPS)	16.11	16.11	16.11
FLEXURE FACTOR OF SAFETY	11.26	5.46	11.26
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.28		-12.28
SHEAR FS AT D (ACI63)	1.07		1.07
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73

```
     SHEAR FS AT 0.15LN (UI440)          .58                          .58


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     19.52        -17.51          19.52
     AXIAL FORCE           (KIPS)     17.34         17.34          17.34
     FLEXURE FACTOR OF SAFETY         11.37          3.54          11.37
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)    -15.39                        15.39
     SHEAR FS AT D (ACI63)             1.53                         1.53
     SHEAR FORCE AT 0.15LN (KIPS)    -13.83                        13.83
     SHEAR FS AT 0.15LN (UI440)        1.18                         1.18


MEMBER 11                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)    -22.25          1.60         -22.71
     AXIAL FORCE           (KIPS)     24.94         24.38          23.81
     FLEXURE FACTOR OF SAFETY          9.39         14.71           9.28
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     10.43                       -10.15
     SHEAR FS AT D (ACI63)             1.26                         1.30
     SHEAR FORCE AT 0.15LN (KIPS)      9.03                        -8.95
     SHEAR FS AT 0.15LN (UI440)         .74                          .74


MEMBER 12                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     22.25         -1.60          22.71
     AXIAL FORCE           (KIPS)     24.94         24.38          23.81
     FLEXURE FACTOR OF SAFETY          9.39         14.71           9.28
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)    -10.43                        10.15
     SHEAR FS AT D (ACI63)             1.26                         1.30
     SHEAR FORCE AT 0.15LN (KIPS)      -9.03                        8.95
     SHEAR FS AT 0.15LN (UI440)         .74                          .74


MEMBER 21                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)    -19.52         15.85         -19.52
     AXIAL FORCE           (KIPS)     15.51         15.51          15.51
     FLEXURE FACTOR OF SAFETY         10.54          3.42          10.54
     STRENGTH REDUCTION  (PHI)          .70           .72            .70
     SHEAR FORCE AT D      (KIPS)     15.12                       -15.12
     SHEAR FS AT D (ACI63)              .87                          .87
     SHEAR FORCE AT 0.15LN (KIPS)      13.21                       -13.21
     SHEAR FS AT 0.15LN (UI440)         .47                          .47
```

1.26/0.85 = 1.48

0.87/0.85 = 1.02

```
23078D SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


3.B.--RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
```

OUTPUT COMPLETE.

```
1000 4 23078E SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
1010   LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=769.40
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  4000.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     3.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   769.40   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   769.40
1130     1.00      .57      .00   769.40
1140     1.50      .50      .00   769.40
1150   0
1160     1.00     1.00
1170   1
1180  731.5
1190   4
1200   1   1.33   1.33    1.33    .0    1.33   1.33
1210  11   1.33   1.33    1.33   1.33   1.33   1.33
1220  12   1.33   1.33    1.33   1.33   1.33   1.33
1230  21   1.33   1.33     .0    1.33   1.33   1.33
```

CONCLUSIONS:
COMPARABLE TO 23078B EXCEPT THE CONDUIT HAS 7.5 FT OF WATER.
THIS CONDITION DOES NOT CHANGE THE LOADING ON THE ROOF RELATIVE TO 23078B. THE FS FOR THE SIDEWALLS DO INCREASE BUT ARE STILL BELOW 1.5 FOR LC 2 AND 3.

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/ 1/2007                                TIME: 10:25:37




PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/ 1/2007                                TIME: 10:25:42

1. INPUT DATA

1.A.--HEADING

23078E SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=769.40
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  4000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
               (FT)      (IN)        (FT)       (FT)
       1        7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	769.40	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	769.40
2	1.00	.57	.00	769.40
3	1.50	.50	.00	769.40

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA

INTERNAL WATER ELEVATIONS

CELL NO	WATER ELEV (FT)
1	731.50

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

```
LOCAL COORDINATE SYSTEMS:
     HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
     VERTICAL MEMBERS  : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:
     POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
     POSITIVE BENDING MOMENT PRODUCES COMPRESSION
          ON PLUS Y FACE OF MEMBER
     POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
     POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
     POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
     UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
          INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
          IS DEFINED BY  FS = PHI * PN / P
               WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                      P = ACTUAL AXIAL FORCE
```

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 4/ 1/2007 TIME: 10:25:44

2.A.--HEADING

23078E SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 03302007
LOC: LEVEE CREST EVENT:N500+3 IWAT=0 GWATEL=769.40
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	18.34	-14.82	18.34
AXIAL FORCE	(KIPS)	16.75	16.75	16.75
FLEXURE FACTOR OF SAFETY		12.14	4.31	12.14
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-13.78		13.78
SHEAR FS AT D (ACI63)		1.70		1.70
SHEAR FORCE AT 0.15LN	(KIPS)	-12.38		12.38
SHEAR FS AT 0.15LN (UI440)		1.32		1.32

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-20.42	2.68	-20.97
AXIAL FORCE	(KIPS)	22.79	22.23	21.66
FLEXURE FACTOR OF SAFETY		10.24	14.64	10.04
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	9.97		-10.01
SHEAR FS AT D (ACI63)		1.32		1.32
SHEAR FORCE AT 0.15LN	(KIPS)	8.68		-8.77
SHEAR FS AT 0.15LN (UI440)		.76		.74

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	20.42	-2.68	20.97
AXIAL FORCE	(KIPS)	22.79	22.23	21.66
FLEXURE FACTOR OF SAFETY		10.24	14.64	10.04
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-9.97		10.01
SHEAR FS AT D (ACI63)		1.32		1.32
SHEAR FORCE AT 0.15LN	(KIPS)	-8.68		8.77
SHEAR FS AT 0.15LN (UI440)		.76		.74

MEMBER 21		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-18.61	13.58	-18.61
AXIAL FORCE	(KIPS)	15.63	15.63	15.63
FLEXURE FACTOR OF SAFETY		11.11	4.04	11.11
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	13.76		-13.76
SHEAR FS AT D (ACI63)		.96		.96
SHEAR FORCE AT 0.15LN	(KIPS)	12.02		-12.02
SHEAR FS AT 0.15LN (UI440)		.52		.52

0.96/0.85 = 1.13

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	20.00	-13.16	20.00
AXIAL FORCE	(KIPS)	18.84	18.84	18.84
FLEXURE FACTOR OF SAFETY		11.17	5.06	11.17
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-13.78		13.78
SHEAR FS AT D (ACI63)		1.72		1.72
SHEAR FORCE AT 0.15LN	(KIPS)	-12.38		12.38
SHEAR FS AT 0.15LN (UI440)		1.34		1.34

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-21.22	4.84	-21.59
AXIAL FORCE	(KIPS)	22.79	22.23	21.66
FLEXURE FACTOR OF SAFETY		9.90	11.00	9.73
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	11.24		-11.19
SHEAR FS AT D (ACI63)		1.17		1.18
SHEAR FORCE AT 0.15LN	(KIPS)	9.78		-9.81
SHEAR FS AT 0.15LN (UI440)		.67		.66

1.17/0.85 = 1.38

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	21.22	-4.84	21.59
AXIAL FORCE	(KIPS)	22.79	22.23	21.66
FLEXURE FACTOR OF SAFETY		9.90	11.00	9.73
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-11.24		11.19
SHEAR FS AT D (ACI63)		1.17		1.18
SHEAR FORCE AT 0.15LN	(KIPS)	-9.78		9.81
SHEAR FS AT 0.15LN (UI440)		.67		.66

MEMBER 21		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-19.96	12.23	-19.96
AXIAL FORCE	(KIPS)	17.45	17.45	17.45

```
     FLEXURE FACTOR OF SAFETY          10.41          4.65          10.41
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)      13.76                       -13.76
     SHEAR FS AT D (ACI63)               .96                          .96
     SHEAR FORCE AT 0.15LN (KIPS)      12.02                       -12.02
     SHEAR FS AT 0.15LN (UI440)          .52                          .52
```

0.96/0.85 = 1.13

```
2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER  1                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     20.38        -20.25          20.38
     AXIAL FORCE           (KIPS)     17.90         17.90          17.90
     FLEXURE FACTOR OF SAFETY         10.87          3.07          10.87
     STRENGTH REDUCTION  (PHI)          .70           .72            .70
     SHEAR FORCE AT D      (KIPS)    -16.88                        16.88
     SHEAR FS AT D (ACI63)             1.40                         1.40
     SHEAR FORCE AT 0.15LN (KIPS)    -15.17                        15.17
     SHEAR FS AT 0.15LN (UI440)        1.76                         1.76

MEMBER 11                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)    -23.99           .53         -25.06
     AXIAL FORCE           (KIPS)     27.84         27.27          26.71
     FLEXURE FACTOR OF SAFETY          8.66         13.14           8.39
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     10.63                       -10.79
     SHEAR FS AT D (ACI63)             1.24                         1.22
     SHEAR FORCE AT 0.15LN (KIPS)      9.25                        -9.47
     SHEAR FS AT 0.15LN (UI440)         .73                          .71
```

1.22/0.85 =1.44

```
MEMBER 12                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     23.99          -.53          25.06
     AXIAL FORCE           (KIPS)     27.84         27.27          26.71
     FLEXURE FACTOR OF SAFETY          8.66         13.14           8.39
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)    -10.63                        10.79
     SHEAR FS AT D (ACI63)             1.24                         1.22
     SHEAR FORCE AT 0.15LN (KIPS)      -9.25                         9.47
     SHEAR FS AT 0.15LN (UI440)         .73                          .71

MEMBER 21                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)    -21.18         18.48         -21.18
     AXIAL FORCE           (KIPS)     16.79         16.79          16.79
     FLEXURE FACTOR OF SAFETY          9.71          2.94           9.71
     STRENGTH REDUCTION  (PHI)          .70           .73            .70
     SHEAR FORCE AT D      (KIPS)     16.95                       -16.95
     SHEAR FS AT D (ACI63)              .78                          .78
     SHEAR FORCE AT 0.15LN (KIPS)      14.81                       -14.81
     SHEAR FS AT 0.15LN (UI440)         .42                          .42
```

0.78/0.85 = 0.92

```
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

3.B.--RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE




OUTPUT COMPLETE.
```

```
1000 4 23078G SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
1010   LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  4000.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     2.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   765.30   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   765.30
1130     1.00      .57      .00   765.30
1140     1.50      .50      .00   765.30
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1   1.33   1.33    1.33    .0    1.33   1.33
1200  11   1.33   1.33    1.33   1.33   1.33   1.33
1210  12   1.33   1.33    1.33   1.33   1.33   1.33
1220  21   1.33   1.33     .0    1.33   1.33   1.33
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/10/2007                                TIME: 12:50:42


PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/10/2007                                TIME: 12:50:51

1. INPUT DATA

1.A.--HEADING

23078G SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  4000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
                (FT)      (IN)        (FT)       (FT)
       1         7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 2.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	765.30	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	765.30
2	1.00	.57	.00	765.30
3	1.50	.50	.00	765.30

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
IS DEFINED BY FS = PHI * PN / P
WHERE PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
P = ACTUAL AXIAL FORCE

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 4/10/2007 TIME: 12:50:54

2.A.--HEADING

23078G SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	17.69	-12.70	17.69
AXIAL FORCE	(KIPS)	16.31	16.31	16.31
FLEXURE FACTOR OF SAFETY		12.89	5.17	12.89
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-12.45		12.45
SHEAR FS AT D (ACI63)		2.08		2.08
SHEAR FORCE AT 0.15LN	(KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)		2.32		2.32

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-19.07	3.49	-19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		11.32	14.86	11.30
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	9.68		-9.33
SHEAR FS AT D (ACI63)		1.49		1.54
SHEAR FORCE AT 0.15LN	(KIPS)	8.52		-8.33
SHEAR FS AT 0.15LN (UI440)		1.99		2.02

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	19.07	-3.49	19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		11.32	14.86	11.30

STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-9.68		9.33
SHEAR FS AT D (ACI63)	1.49		1.54
SHEAR FORCE AT 0.15LN (KIPS)	-8.52		8.33
SHEAR FS AT 0.15LN (UI440)	1.99		2.02

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-17.22	11.51	-17.22
AXIAL FORCE (KIPS)	14.47	14.47	14.47
FLEXURE FACTOR OF SAFETY	12.31	4.86	12.31
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.11		-12.11
SHEAR FS AT D (ACI63)	1.19		1.19
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73
SHEAR FS AT 0.15LN (UI440)	1.51		1.51

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.19	-11.20	19.19
AXIAL FORCE (KIPS)	18.20	18.20	18.20
FLEXURE FACTOR OF SAFETY	11.92	5.85	11.92
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-12.45		12.45
SHEAR FS AT D (ACI63)	2.10		2.10
SHEAR FORCE AT 0.15LN (KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)	2.35		2.35

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.79	5.44	-19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	10.89	11.52	10.95
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.81		-10.38
SHEAR FS AT D (ACI63)	1.33		1.39
SHEAR FORCE AT 0.15LN (KIPS)	9.51		-9.26
SHEAR FS AT 0.15LN (UI440)	1.78		1.82

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.79	-5.44	19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	10.89	11.52	10.95
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.81		10.38
SHEAR FS AT D (ACI63)	1.33		1.39
SHEAR FORCE AT 0.15LN (KIPS)	-9.51		9.26
SHEAR FS AT 0.15LN (UI440)	1.78		1.82

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-18.44	10.29	-18.44
AXIAL FORCE (KIPS)	16.11	16.11	16.11
FLEXURE FACTOR OF SAFETY	11.54	5.46	11.54
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.11		-12.11
SHEAR FS AT D (ACI63)	1.19		1.19
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73

SHEAR FS AT 0.15LN (UI440)	1.53	1.53

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.52	-17.51	19.52
AXIAL FORCE (KIPS)	17.34	17.34	17.34
FLEXURE FACTOR OF SAFETY	11.63	3.54	11.63
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-15.17		15.17
SHEAR FS AT D (ACI63)	1.72		1.72
SHEAR FORCE AT 0.15LN (KIPS)	-13.83		13.83
SHEAR FS AT 0.15LN (UI440)	1.92		1.92

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-22.25	1.60	-22.71
AXIAL FORCE (KIPS)	24.94	24.38	23.81
FLEXURE FACTOR OF SAFETY	9.70	14.71	9.50
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.27		-10.02
SHEAR FS AT D (ACI63)	1.40		1.44
SHEAR FORCE AT 0.15LN (KIPS)	9.03		-8.95
SHEAR FS AT 0.15LN (UI440)	1.94		1.94

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	22.25	-1.60	22.71
AXIAL FORCE (KIPS)	24.94	24.38	23.81
FLEXURE FACTOR OF SAFETY	9.70	14.71	9.50
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.27		10.02
SHEAR FS AT D (ACI63)	1.40		1.44
SHEAR FORCE AT 0.15LN (KIPS)	-9.03		8.95
SHEAR FS AT 0.15LN (UI440)	1.94		1.94

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.52	15.85	-19.52
AXIAL FORCE (KIPS)	15.51	15.51	15.51
FLEXURE FACTOR OF SAFETY	10.80	3.42	10.80
STRENGTH REDUCTION (PHI)	.70	.72	.70
SHEAR FORCE AT D (KIPS)	14.91		-14.91
SHEAR FS AT D (ACI63)	.97		.97
SHEAR FORCE AT 0.15LN (KIPS)	13.21		-13.21
SHEAR FS AT 0.15LN (UI440)	1.24		1.24

23078G SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

3.B.--RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

OUTPUT COMPLETE.

```
1000 4 23078L SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
1010   LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  4300.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     2.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   765.30   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   765.30
1130     1.00      .57      .00   765.30
1140     1.50      .50      .00   765.30
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1   1.33   1.33    1.33    .0    1.33   1.33
1200  11   1.33   1.33    1.33   1.33   1.33   1.33
1210  12   1.33   1.33    1.33   1.33   1.33   1.33
1220  21   1.33   1.33     .0    1.33   1.33   1.33
```

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PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/10/2007                                TIME: 13:33:45


PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/10/2007                                TIME: 13:34: 1

1. INPUT DATA

1.A.--HEADING

23078L SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  4300.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .84
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
               (FT)      (IN)        (FT)       (FT)
       1        7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 2.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	765.30	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	765.30
2	1.00	.57	.00	765.30
3	1.50	.50	.00	765.30

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
IS DEFINED BY FS = PHI * PN / P
WHERE PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
P = ACTUAL AXIAL FORCE

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 4/10/2007 TIME: 13:34: 5

2.A.--HEADING

23078L SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	17.69	-12.70	17.69
AXIAL FORCE	(KIPS)	16.31	16.31	16.31
FLEXURE FACTOR OF SAFETY		13.54	5.37	13.54
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-12.45		12.45
SHEAR FS AT D (ACI63)		2.16		2.16
SHEAR FORCE AT 0.15LN	(KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)		2.39		2.39

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-19.07	3.49	-19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		11.93	15.69	11.89
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	9.68		-9.33
SHEAR FS AT D (ACI63)		1.53		1.59
SHEAR FORCE AT 0.15LN	(KIPS)	8.52		-8.33
SHEAR FS AT 0.15LN (UI440)		2.05		2.09

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	19.07	-3.49	19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		11.93	15.69	11.89

STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-9.68		9.33
SHEAR FS AT D (ACI63)	1.53		1.59
SHEAR FORCE AT 0.15LN (KIPS)	-8.52		8.33
SHEAR FS AT 0.15LN (UI440)	2.05		2.09

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-17.22	11.51	-17.22
AXIAL FORCE (KIPS)	14.47	14.47	14.47
FLEXURE FACTOR OF SAFETY	12.90	5.03	12.90
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.11		-12.11
SHEAR FS AT D (ACI63)	1.22		1.22
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73
SHEAR FS AT 0.15LN (UI440)	1.56		1.56

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.19	-11.20	19.19
AXIAL FORCE (KIPS)	18.20	18.20	18.20
FLEXURE FACTOR OF SAFETY	12.54	6.18	12.54
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-12.45		12.45
SHEAR FS AT D (ACI63)	2.18		2.18
SHEAR FORCE AT 0.15LN (KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)	2.43		2.43

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.79	5.44	-19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	11.47	12.12	11.51
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.81		-10.38
SHEAR FS AT D (ACI63)	1.37		1.43
SHEAR FORCE AT 0.15LN (KIPS)	9.51		-9.26
SHEAR FS AT 0.15LN (UI440)	1.84		1.87

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.79	-5.44	19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	11.47	12.12	11.51
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.81		10.38
SHEAR FS AT D (ACI63)	1.37		1.43
SHEAR FORCE AT 0.15LN (KIPS)	-9.51		9.26
SHEAR FS AT 0.15LN (UI440)	1.84		1.87

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-18.44	10.29	-18.44
AXIAL FORCE (KIPS)	16.11	16.11	16.11
FLEXURE FACTOR OF SAFETY	12.10	5.77	12.10
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.11		-12.11
SHEAR FS AT D (ACI63)	1.22		1.22
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73

SHEAR FS AT 0.15LN (UI440)	1.58	1.58

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.52	-17.51	19.52
AXIAL FORCE (KIPS)	17.34	17.34	17.34
FLEXURE FACTOR OF SAFETY	12.21	3.68	12.21
STRENGTH REDUCTION (PHI)	.70	.71	.70
SHEAR FORCE AT D (KIPS)	-15.17		15.17
SHEAR FS AT D (ACI63)	1.78		1.78
SHEAR FORCE AT 0.15LN (KIPS)	-13.83		13.83
SHEAR FS AT 0.15LN (UI440)	1.98		1.98

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-22.25	1.60	-22.71
AXIAL FORCE (KIPS)	24.94	24.38	23.81
FLEXURE FACTOR OF SAFETY	10.25	15.53	10.01
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.27		-10.02
SHEAR FS AT D (ACI63)	1.44		1.48
SHEAR FORCE AT 0.15LN (KIPS)	9.03		-8.95
SHEAR FS AT 0.15LN (UI440)	2.00		2.00

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	22.25	-1.60	22.71
AXIAL FORCE (KIPS)	24.94	24.38	23.81
FLEXURE FACTOR OF SAFETY	10.25	15.53	10.01
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.27		10.02
SHEAR FS AT D (ACI63)	1.44		1.48
SHEAR FORCE AT 0.15LN (KIPS)	-9.03		8.95
SHEAR FS AT 0.15LN (UI440)	2.00		2.00

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.52	15.85	-19.52
AXIAL FORCE (KIPS)	15.51	15.51	15.51
FLEXURE FACTOR OF SAFETY	11.31	3.54	11.31
STRENGTH REDUCTION (PHI)	.70	.72	.70
SHEAR FORCE AT D (KIPS)	14.91		-14.91
SHEAR FS AT D (ACI63)	.99		.99
SHEAR FORCE AT 0.15LN (KIPS)	13.21		-13.21
SHEAR FS AT 0.15LN (UI440)	1.27		1.27

23078L SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

3.B.--RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

OUTPUT COMPLETE.

```
1000 4 23078M SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
1010   LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  5000.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     2.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   765.30   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   765.30
1130     1.00      .57      .00   765.30
1140     1.50      .50      .00   765.30
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1   1.33   1.33    1.33    .0    1.33   1.33
1200  11   1.33   1.33    1.33   1.33   1.33   1.33
1210  12   1.33   1.33    1.33   1.33   1.33   1.33
1220  21   1.33   1.33     .0    1.33   1.33   1.33
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PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/10/2007                                TIME: 13:19:22


PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/10/2007                                TIME: 13:19:28

1. INPUT DATA

1.A.--HEADING

23078M SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  5000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .80
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
               (FT)      (IN)        (FT)       (FT)
       1        7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 2.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	765.30	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	765.30
2	1.00	.57	.00	765.30
3	1.50	.50	.00	765.30

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
IS DEFINED BY FS = PHI * PN / P
WHERE PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
P = ACTUAL AXIAL FORCE

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 4/10/2007 TIME: 13:19:30

2.A.--HEADING

23078M SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	17.69	-12.70	17.69
AXIAL FORCE	(KIPS)	16.31	16.31	16.31
FLEXURE FACTOR OF SAFETY		14.98	5.78	14.98
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-12.45		12.45
SHEAR FS AT D (ACI63)		2.33		2.33
SHEAR FORCE AT 0.15LN	(KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)		2.56		2.56

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-19.07	3.49	-19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		13.30	17.61	13.20
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	9.68		-9.33
SHEAR FS AT D (ACI63)		1.63		1.69
SHEAR FORCE AT 0.15LN	(KIPS)	8.52		-8.33
SHEAR FS AT 0.15LN (UI440)		2.19		2.22

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	19.07	-3.49	19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		13.30	17.61	13.20

STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-9.68		9.33
SHEAR FS AT D (ACI63)	1.63		1.69
SHEAR FORCE AT 0.15LN (KIPS)	-8.52		8.33
SHEAR FS AT 0.15LN (UI440)	2.19		2.22

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-17.22	11.51	-17.22
AXIAL FORCE (KIPS)	14.47	14.47	14.47
FLEXURE FACTOR OF SAFETY	14.19	5.39	14.19
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.11		-12.11
SHEAR FS AT D (ACI63)	1.30		1.30
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73
SHEAR FS AT 0.15LN (UI440)	1.66		1.66

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.19	-11.20	19.19
AXIAL FORCE (KIPS)	18.20	18.20	18.20
FLEXURE FACTOR OF SAFETY	13.89	6.90	13.89
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-12.45		12.45
SHEAR FS AT D (ACI63)	2.35		2.35
SHEAR FORCE AT 0.15LN (KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)	2.59		2.59

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.79	5.44	-19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	12.75	13.50	12.77
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.81		-10.38
SHEAR FS AT D (ACI63)	1.46		1.52
SHEAR FORCE AT 0.15LN (KIPS)	9.51		-9.26
SHEAR FS AT 0.15LN (UI440)	1.96		2.00

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.79	-5.44	19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	12.75	13.50	12.77
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.81		10.38
SHEAR FS AT D (ACI63)	1.46		1.52
SHEAR FORCE AT 0.15LN (KIPS)	-9.51		9.26
SHEAR FS AT 0.15LN (UI440)	1.96		2.00

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-18.44	10.29	-18.44
AXIAL FORCE (KIPS)	16.11	16.11	16.11
FLEXURE FACTOR OF SAFETY	13.34	6.42	13.34
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.11		-12.11
SHEAR FS AT D (ACI63)	1.30		1.30
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73

SHEAR FS AT 0.15LN (UI440)	1.68	1.68

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.52	-17.51	19.52
AXIAL FORCE (KIPS)	17.34	17.34	17.34
FLEXURE FACTOR OF SAFETY	13.49	3.97	13.49
STRENGTH REDUCTION (PHI)	.70	.72	.70
SHEAR FORCE AT D (KIPS)	-15.17		15.17
SHEAR FS AT D (ACI63)	1.92		1.92
SHEAR FORCE AT 0.15LN (KIPS)	-13.83		13.83
SHEAR FS AT 0.15LN (UI440)	2.11		2.11

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-22.25	1.60	-22.71
AXIAL FORCE (KIPS)	24.94	24.38	23.81
FLEXURE FACTOR OF SAFETY	11.46	17.47	11.14
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.27		-10.02
SHEAR FS AT D (ACI63)	1.53		1.57
SHEAR FORCE AT 0.15LN (KIPS)	9.03		-8.95
SHEAR FS AT 0.15LN (UI440)	2.13		2.13

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	22.25	-1.60	22.71
AXIAL FORCE (KIPS)	24.94	24.38	23.81
FLEXURE FACTOR OF SAFETY	11.46	17.47	11.14
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.27		10.02
SHEAR FS AT D (ACI63)	1.53		1.57
SHEAR FORCE AT 0.15LN (KIPS)	-9.03		8.95
SHEAR FS AT 0.15LN (UI440)	2.13		2.13

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.52	15.85	-19.52
AXIAL FORCE (KIPS)	15.51	15.51	15.51
FLEXURE FACTOR OF SAFETY	12.41	3.81	12.41
STRENGTH REDUCTION (PHI)	.70	.74	.70
SHEAR FORCE AT D (KIPS)	14.91		-14.91
SHEAR FS AT D (ACI63)	1.06		1.06
SHEAR FORCE AT 0.15LN (KIPS)	13.21		-13.21
SHEAR FS AT 0.15LN (UI440)	1.36		1.36

23078M SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

3.B.--RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

OUTPUT COMPLETE.

```
1000 4 23078U SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
1010   LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  5700.0 60000.0   150.0  .3750  .0000
1060   1   7.50  12.00  724.00   7.50
1070     2.56     2.56     2.56
1080    12.00    12.00    13.00
1090   1
1100   765.30   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   765.30
1130     1.00      .57      .00   765.30
1140     1.50      .50      .00   765.30
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1   1.33   1.33    1.33    .0    1.33   1.33
1200  11   1.33   1.33    1.33   1.33   1.33   1.33
1210  12   1.33   1.33    1.33   1.33   1.33   1.33
1220  21   1.33   1.33     .0    1.33   1.33   1.33
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/10/2007                                TIME: 13:31:12


PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  4/10/2007                                TIME: 13:31:17

1. INPUT DATA

1.A.--HEADING

23078U SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST  EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  5700.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .77
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 60000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
               (FT)      (IN)        (FT)       (FT)
       1        7.50     12.00      724.00      7.50


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 2.56     ROOF SLAB      = 12.00
     INTERIOR ROOF/END WALLS = 2.56     EXTERIOR WALLS = 12.00
     INTERIOR BASE SLAB      = 2.56     BASE SLAB      = 13.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	765.30	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	765.30
2	1.00	.57	.00	765.30
3	1.50	.50	.00	765.30

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	1.33	1.33	1.33	.00	1.33	1.33
11	1.33	1.33	1.33	1.33	1.33	1.33
12	1.33	1.33	1.33	1.33	1.33	1.33
21	1.33	1.33	.00	1.33	1.33	1.33

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 4/10/2007 TIME: 13:31:24

2.A.--HEADING

23078U SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	17.69	-12.70	17.69
AXIAL FORCE	(KIPS)	16.31	16.31	16.31
FLEXURE FACTOR OF SAFETY		16.33	6.13	16.33
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	-12.45		12.45
SHEAR FS AT D (ACI63)		2.49		2.49
SHEAR FORCE AT 0.15LN	(KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)		2.71		2.71

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-19.07	3.49	-19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		14.58	19.54	14.44
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	9.68		-9.33
SHEAR FS AT D (ACI63)		1.72		1.78
SHEAR FORCE AT 0.15LN	(KIPS)	8.52		-8.33
SHEAR FS AT 0.15LN (UI440)		2.31		2.35

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	19.07	-3.49	19.06
AXIAL FORCE	(KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY		14.58	19.54	14.44

STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-9.68		9.33
SHEAR FS AT D (ACI63)	1.72		1.78
SHEAR FORCE AT 0.15LN (KIPS)	-8.52		8.33
SHEAR FS AT 0.15LN (UI440)	2.31		2.35

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-17.22	11.51	-17.22
AXIAL FORCE (KIPS)	14.47	14.47	14.47
FLEXURE FACTOR OF SAFETY	15.38	5.70	15.38
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.11		-12.11
SHEAR FS AT D (ACI63)	1.37		1.37
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73
SHEAR FS AT 0.15LN (UI440)	1.76		1.76

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.19	-11.20	19.19
AXIAL FORCE (KIPS)	18.20	18.20	18.20
FLEXURE FACTOR OF SAFETY	15.15	7.55	15.15
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-12.45		12.45
SHEAR FS AT D (ACI63)	2.51		2.51
SHEAR FORCE AT 0.15LN (KIPS)	-11.35		11.35
SHEAR FS AT 0.15LN (UI440)	2.75		2.75

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.79	5.44	-19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	13.95	14.85	13.95
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.81		-10.38
SHEAR FS AT D (ACI63)	1.54		1.60
SHEAR FORCE AT 0.15LN (KIPS)	9.51		-9.26
SHEAR FS AT 0.15LN (UI440)	2.07		2.12

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.79	-5.44	19.62
AXIAL FORCE (KIPS)	20.45	19.89	19.33
FLEXURE FACTOR OF SAFETY	13.95	14.85	13.95
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.81		10.38
SHEAR FS AT D (ACI63)	1.54		1.60
SHEAR FORCE AT 0.15LN (KIPS)	-9.51		9.26
SHEAR FS AT 0.15LN (UI440)	2.07		2.12

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-18.44	10.29	-18.44
AXIAL FORCE (KIPS)	16.11	16.11	16.11
FLEXURE FACTOR OF SAFETY	14.48	6.89	14.48
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	12.11		-12.11
SHEAR FS AT D (ACI63)	1.37		1.37
SHEAR FORCE AT 0.15LN (KIPS)	10.73		-10.73

SHEAR FS AT 0.15LN (UI440)	1.79	1.79

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.52	-17.51	19.52
AXIAL FORCE (KIPS)	17.34	17.34	17.34
FLEXURE FACTOR OF SAFETY	14.66	4.21	14.66
STRENGTH REDUCTION (PHI)	.70	.74	.70
SHEAR FORCE AT D (KIPS)	-15.17		15.17
SHEAR FS AT D (ACI63)	2.05		2.05
SHEAR FORCE AT 0.15LN (KIPS)	-13.83		13.83
SHEAR FS AT 0.15LN (UI440)	2.24		2.24

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-22.25	1.60	-22.71
AXIAL FORCE (KIPS)	24.94	24.38	23.81
FLEXURE FACTOR OF SAFETY	12.60	19.40	12.20
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	10.27		-10.02
SHEAR FS AT D (ACI63)	1.62		1.66
SHEAR FORCE AT 0.15LN (KIPS)	9.03		-8.95
SHEAR FS AT 0.15LN (UI440)	2.25		2.26

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	22.25	-1.60	22.71
AXIAL FORCE (KIPS)	24.94	24.38	23.81
FLEXURE FACTOR OF SAFETY	12.60	19.40	12.20
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-10.27		10.02
SHEAR FS AT D (ACI63)	1.62		1.66
SHEAR FORCE AT 0.15LN (KIPS)	-9.03		8.95
SHEAR FS AT 0.15LN (UI440)	2.25		2.26

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.52	15.85	-19.52
AXIAL FORCE (KIPS)	15.51	15.51	15.51
FLEXURE FACTOR OF SAFETY	13.42	4.03	13.42
STRENGTH REDUCTION (PHI)	.70	.75	.70
SHEAR FORCE AT D (KIPS)	14.91		-14.91
SHEAR FS AT D (ACI63)	1.12		1.12
SHEAR FORCE AT 0.15LN (KIPS)	13.21		-13.21
SHEAR FS AT 0.15LN (UI440)	1.44		1.44

23078U SHAWNEE PUMP STA. OUTLET WORKS M.PARKS 04102007
LOC: LEVEE CREST EVENT:EC IWAT=0 GWATEL=765.3
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

3.B.--RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

OUTPUT COMPLETE.

US Army Corps of Engineers

Armourdale District
Shawnee RCB Sta 230+78
Probability of Failure

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. "Factors of Safety and Reliability in Geotechnical Engineering" by J. Michael Duncan published in the Journal of Geotechnical and Geoenvironmental Engineering in April 2000
2. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996

III. Situation

1. Shawnee pump station reinforced concrete box culvert has shear FS < 1.5 for the existing condition.
2. Load Case 23078M, L and U from CORTCUL provided input values

IV. Variable Definitions

FS_D = Shear Factor of Safety under mean concrete parameters
FS_{Su} = Shear Factor of Safety due to the upper bound value of the Concrete Strength
FS_{Sl} = Shear Factor of Safety due to the lower bound value of the Concrete Strength
σ_F = Standard Deviation of the Sliding Factor of Safety
V_F = Coefficient of Variation of the Sliding Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

V. Caclulating Factors of Safety

Shear demand: $V_{u2} := 14.91 \frac{kip}{ft}$ Shear demand. Calculations found at: 23078G

Shear strength: $V_{n2} := 17.01 \frac{kip}{ft}$ Shear capacity. Thrust was taken into account. = 14.91 x 0.97/0.85

$FS_{S2} := \frac{V_{n2}}{V_{u2}}$ $FS_{S2} = 1.141$ Factor of safety.

$f'_c := 4000psi$ Design Concrete Strength $F_y := 60ksi$ Steel yield strength

$b := \frac{12in}{ft}$ $\phi_V := 1.0$ $d_2 := 9.44in$ $D_2 := 12in$

Mean Concrete Strength

$f'_{cM} := f'_c \cdot 1.25$

$f'_{cM} = 5000\,psi$

Based on FEMA 310, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength.

$\phi V_{n2} := \frac{1.06 \cdot 14.91 \frac{kip}{ft}}{0.85}$ CORTCUL 23078M OUTPUT, LOAD CASE 3 MEMBER 21

$\phi V_{n2} = 18594 \frac{lb}{ft}$

$FS_D := \frac{\phi V_{n2}}{V_{u2}}$ $FS_D = 1.247$

Upper Concrete Strength

$$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$$

$$f'_{cU} = 5700\,\text{psi}$$

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$$\phi V_{n2} := \frac{1.12 \cdot 14.91\,\frac{\text{kip}}{\text{ft}}}{0.85}$$

CORTCUL 23078U, LOAD CASE 3, MEMBER 21

$$\phi V_{n2} = 19646\,\frac{\text{lb}}{\text{ft}}$$

$$FS_{Su} := \frac{\phi V_{n2}}{V_{u2}}$$

$$FS_{Su} = 1.318$$

Lower Concrete Strength

$$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$$

$$f'_{cL} = 4300\,\text{psi}$$

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$$\phi V_{n2} := \frac{0.97 \cdot 14.91 \frac{\text{kip}}{\text{ft}}}{0.85}$$

$$\phi V_{n2} = 17015 \frac{\text{lb}}{\text{ft}}$$

$$FS_{Sl} := \frac{\phi V_{n2}}{V_{u2}}$$

$$FS_{Sl} = 1.141$$

VI. Probability of Failure Calculation

$\Delta F_S := FS_{Su} - FS_{Sl}$ $\qquad \Delta F_S = 0.176$

$\sigma_F := \sqrt{\left(\frac{\Delta F_S}{2}\right)^2}$ $\qquad \sigma_F = 0.088$

$V_F := \frac{\sigma_F}{FS_D}$ $\qquad V_F = 0.071$

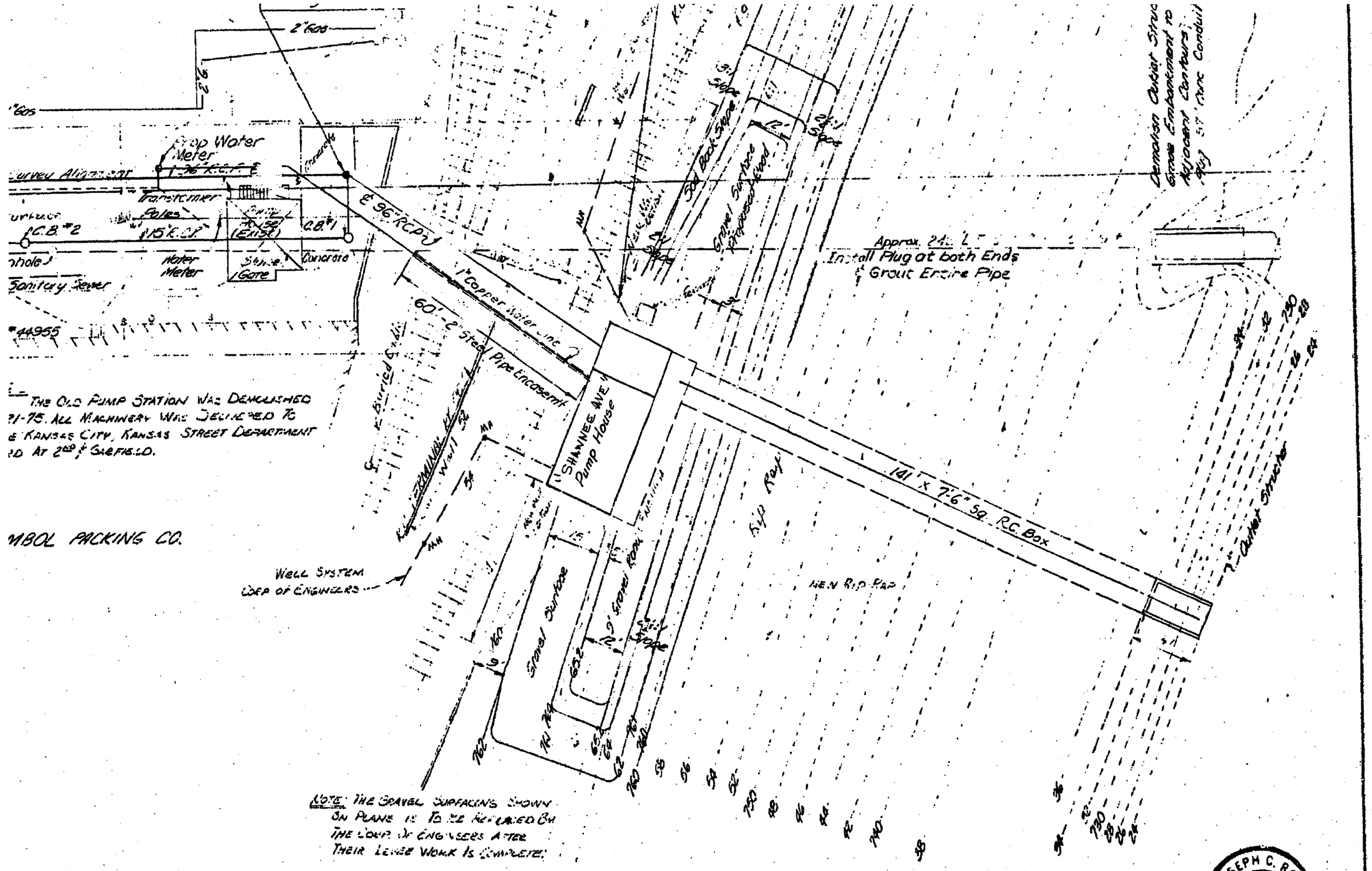
$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\qquad \beta_{LN} = 3.089$

$R := \mathrm{cnorm}\left(\beta_{LN}\right)$ $\qquad R = 99.9\,\%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 0.100404\,\%$

END OF ANALYSIS



Shawnee Avenue Record Drawings
Sheet 1 of 23
Schlup, Becker & Brennan
Consulting Engineers

Joseph C. Roth
Licensed
5843
Kansas
Professional Engineer

C.8

11

22

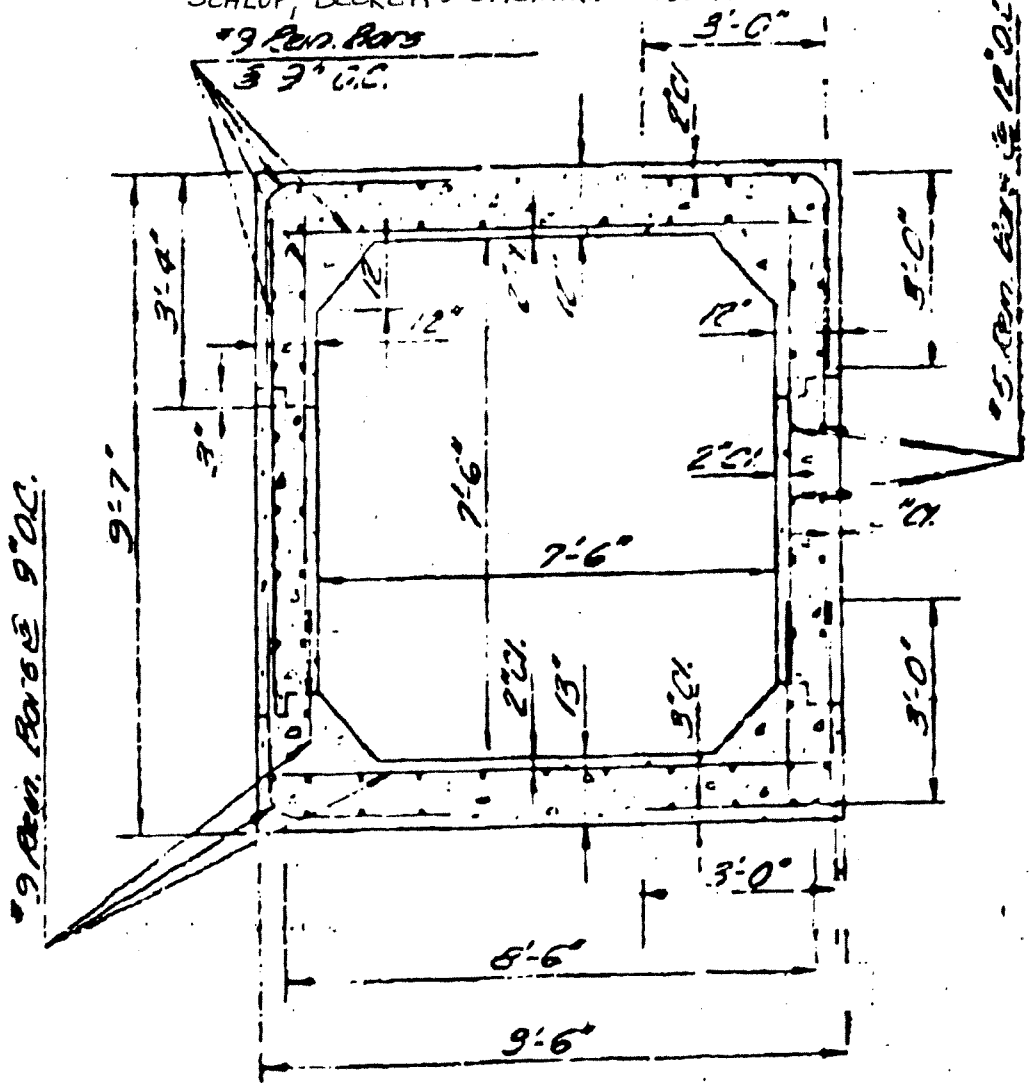
SHAWNEE AVENUE

RECORD DRAWINGS

SHEET 3 OF 23

6-25-75

SCHLUP, BECKER & BRENNAN CONSULTING ENGINEERS



EFFLUENT STRUCTURE SECTION

~~SCALE: 1/8" = 1'-0"~~

SEE SHEET NO. 1 & 2 PLAN & PROFILE

No Single Section of Effluent Structure shall be longer than 30' in Length. See above Collar Detail.

Variables

$kip := 1000lb \quad plf := \frac{lb}{ft} \quad psf \equiv \frac{lb}{ft^2} \quad pcf := \frac{lb}{ft^3} \quad psi \equiv \frac{lb}{in^2} \quad ksi \equiv \frac{1000lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad kips := 1000lb \quad klf := \frac{kips}{ft}$

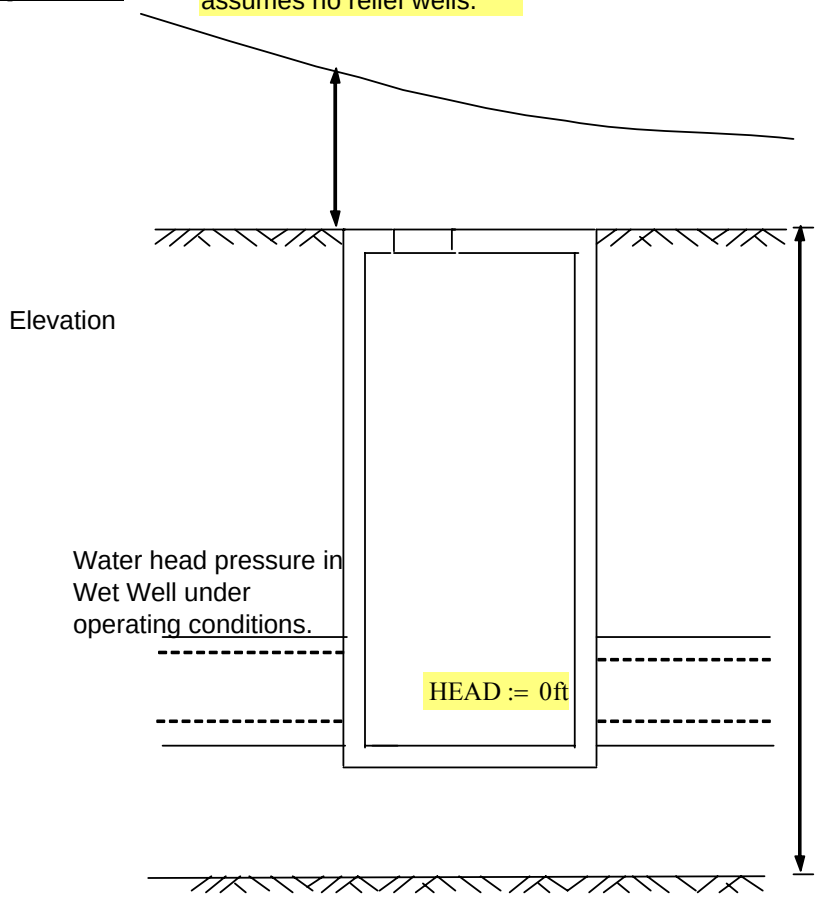
STRUCTURAL DRAWINGS OF THE 12TH STREET PUMP STATION COULD NOT BE FOUND. STRUCTURAL DRAWINGS OF THE MILL STREET PUMP STATION WERE AVAILABLE. THE 12TH STREET PUMP STATION AND MILL STREET PUMP STATION WERE BUILT ~ 1916. FROM THE DRAWINGS AVAILABLE, THE PUMP STATIONS ARE VERY SIMILIAR. BASED ON THESE FINDINGS, THE 12TH STREET PUMP STATION WILL BE ANALYZED USING THE SAME REINFORCEMENT AREAS, CONCRETE STRENGTH AND STEEL REINFORCEMENT STRENGTH. THE AVAILABLE 12TH STREET DRAWINGS DO INDICATE THE WETWELL WALL THICKNESS AND HEIGHT.

REFERENCES
1. Structural drawings not available.
2. DR&G Engineers drawings (pump modification), 1995.

Top of existing levee elev = 769.5
500 year +3 ft top of levee elevation = 774.4

Properties

NOTE: HGL Supplied assumes no relief wells.



$ELEV_1 := 760ft$

$HGL := 768.2ft - ELEV_1$

$HGL = 8.2\ ft$

$BLANKET := 760ft - 734ft$

$BLANKET = 26\ ft$

The 1995 drawings were generated as a result of pump replacement done by DR&G Engineers.
The structure undergone significant modifications during the 1995 pump replacement. The single pump was replaced with two pumps.

Refering to the 1995 drawings:

Wall 1 Parameters Wet well	Wall 2 Parameters Wet well
West wall	North wall
$L_1 := 13ft + 0in$	$L_2 := 7ft + 2in$
$Hwall_1 := 29.5ft$	$Hwall_2 := 29.5ft$
$D_1 := 10in$	$D_2 := 10in$
$C_{c1} := 2.5in$	$C_{c2} := 2.5in$

NOTE: The 12th Street pump station exterior walls other than the wet well walls were not analyzed. Structural drawings of these walls are not available. Structural drawings of the Mill Street pump station walls are available and assumed to be similar in design to the 12th St pump station walls. The Mill Street walls were analyzed and were reliable for 767.2 = HGL event when sand infill is not present. Both the 12th St and Mill St pump stations have sand infill on the interior side of the walls (earth load on the outside).
Based on this information, the 12th Street walls enclosing the sand infill material are considered reliable.

Hwall is measured from grade and may not be the actual wall height. The Hwall value is used to determine the wall loads.

BASE SLAB

The base slab reinforcement is not shown on the available drawings. Assume that the base slab has #6 at 12 inches.

Floor Thickness $D_f := 10in$ Clear Cover $C_{cf} := 3in$ $d_f := D_f - C_{cf} - \frac{0.75in}{2}$ $d_f = 6.625\ in$

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - C_{c1} - 0.375in$ $d_1 = 7.125\ in$

WALL 2

$d_2 := D_2 - C_{c2} - 0.25in$ $d_2 = 7.25\ in$

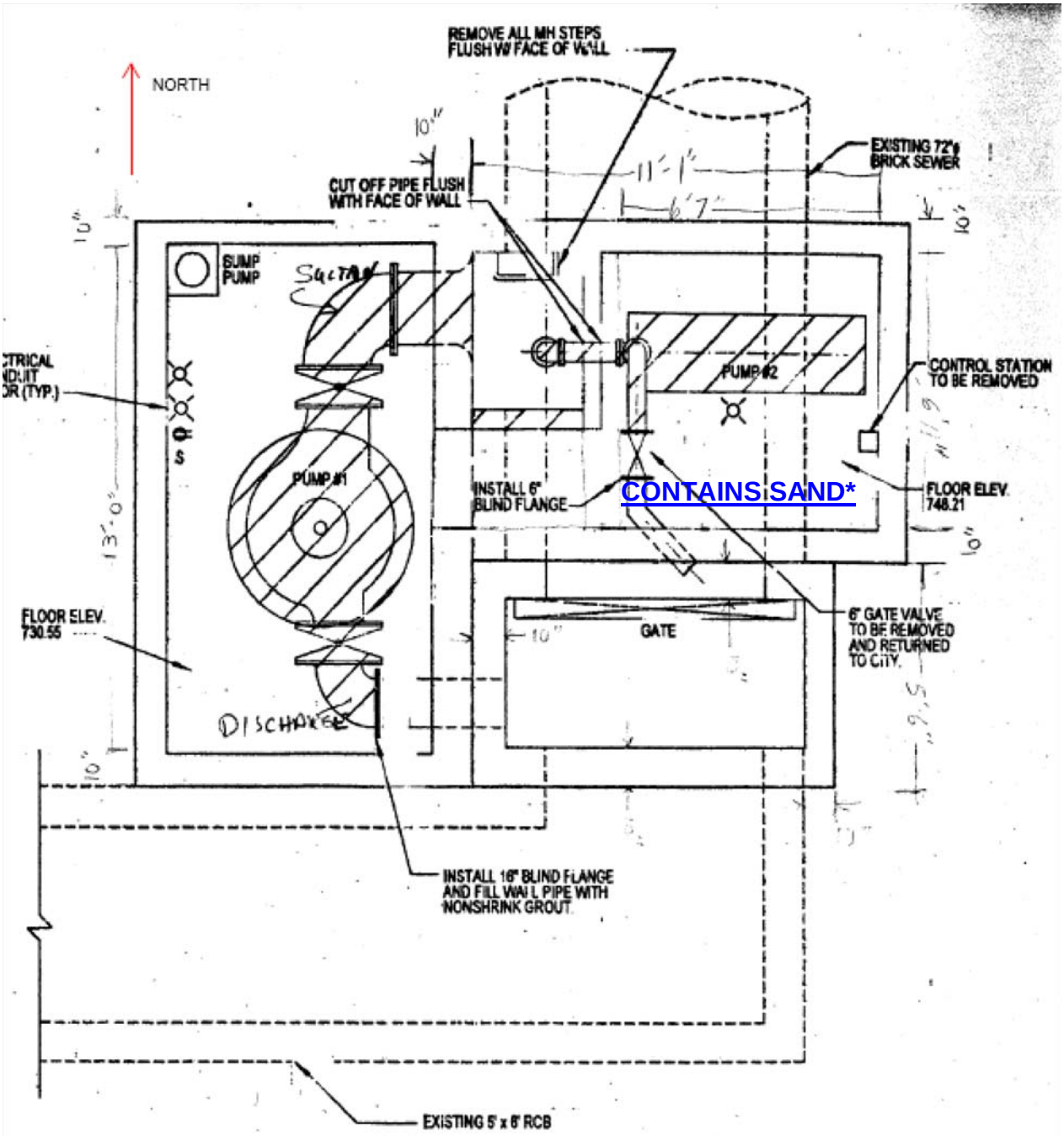


Figure 1. 12th Street Pump StationPlan View Drawing Sheet 2 of 15 DR&G Engineers, 1995

* Existing area below grade filled with sand as part of the 1995 pump replacement. This is shown on the DR&G Engineers drawings. As a result of the modifications, access to this area is no longer possible.

Assumptions and Criteria

- As-built drawings give fc=1350 and fs=20,000. These are working stress parameters. For this analysis, f'c=3000 psf and fy=40,000 psf.

Soil Properties	$\phi := 26deg$	$K_o := 1 - \sin(\phi)$	Steel Properties	$F_y := 40ksi$
	$\gamma := 120pcf$	$K_o = 0.562$	Concrete Properties	$f'_c := 3.00ksi$
Water Unit Weight	$\gamma_w := 62.4pcf$		Concrete Unit Weight	$\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

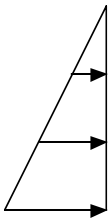
WALL 1, (West wall)

$K_o = 0.562$ $BLANKET = 26\ ft$ $HGL = 8.2\ ft$

$Hwall_1 = 29.5\ ft$

Wall Loading

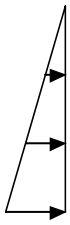
Soil or Soil & Water



$H_2 := Hwall_1$

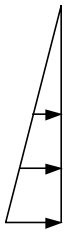
$H_2 = 29.5\ ft$

OR



$H_2 = 29.5\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_1 - HEAD$

$H''_3 := Hwall_1 + HGL - HEAD$

$H_3 := if(Hwall_1 > BLANKET, H''_3, H'_3)$

$H_3 = 37.7\ ft$

$H'_3 = 38.804\ ft$

$H''_3 = 37.7\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 1988\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3]$

$W_w = 3307\ \frac{lb}{ft^2}$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_1 := if(W_s > W_w, W_s, W_w)$

$W_1 = 3307\ \frac{lb}{ft^2}$

WALL 1 (West Wall) SHEAR

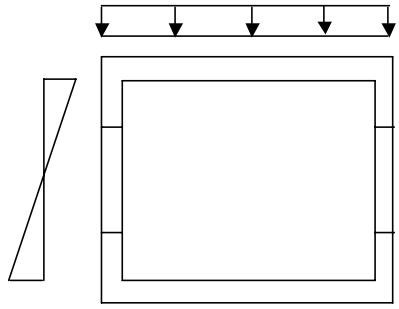
$L_2 = 7.167\ ft$

$THRUST_1 := \frac{W_1 \cdot L_2}{2}$ $THRUST_1 = 11849\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_1 \cdot \frac{L_1}{2}$

$V'_{u1} = 21494\ \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$ $V_{u1} = 18153\ \frac{lb}{ft}$

$D_2 = 0.833\ ft$

$d_1 = 7.125\ in$

$L_1 = 13\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\frac{L_1}{d_1} = 21.895$ EM 1110-2-2104 EQ (5-1) Does not Apply

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $A_g = 120\ \frac{in^2}{ft}$

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 9828\ \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD"\right)$

$Check1 = "NO GOOD"$

$\frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 0.049$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$ $FS_{S1} = 0.541$

ANALYZE WALL 1 (West wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_1 - \frac{(W_1 - 0psf)\cdot 3ft}{29.5ft} = 2970.515\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{2971psf\cdot L_1^2}{8}$ $M_{mid} = 62.762\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_1^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$

#6's @ 6 in. inside face (mid-plate moment) $A_{mph} := 0.884\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 1.156\,in$

$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 19.3\frac{kip\cdot ft}{ft}$

no bars outside face (edge-plate moment) $A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO\ GOOD")$ CheckMPH = "NO GOOD"

Factor of Safety $FS_{MPH1} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH1} = 0.307$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO\ GOOD")$ CheckEPH = "NO GOOD"

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3306.8\,psf$

$$W_1 - \frac{(W_1 - 0psf)\cdot 3ft}{29.5ft} = 2970.515\,psf$$

Thrust present 3 ft from bottom of wall.

$$THRUST := \frac{2971psf \cdot L_2}{2}$$

$$THRUST = 10646.083 \frac{lb}{ft}$$

$d_1 = 7.125\ in$ $A_g = 0.833\ ft$

$C_{c1} = 2.5\ in$

Information for CASTR: $THRUST = 10.65 \frac{kip}{ft}$ $M_{mid} = 62.762 \frac{kip \cdot ft}{ft}$

```
100 12thw1MPH767.2 M.Parks
110 Wall 1 MPM HGL=768.2
200 1
210 3.00 40.0 0.375
300 12.0 X 12 with 1 bar inside face
400 12.0 12.0
500 1
510 1 0.884 7.125 6.0 6.0
600 10.7 62.8
```

Percent Design Strength Used in Tensile Zone

$Percent := 340.31\%$

$$FoS' := \frac{1}{Percent \cdot .9}$$

Note: Strength Reduction Factor (.9) not applied.

$FoS' = 0.33$

$M_{mid} = 62.762 \frac{kip \cdot ft}{ft}$

CASTR output: The section falls outside the admissible range and does not meet input data reinforcement limit requirement.

From interaction diagram:

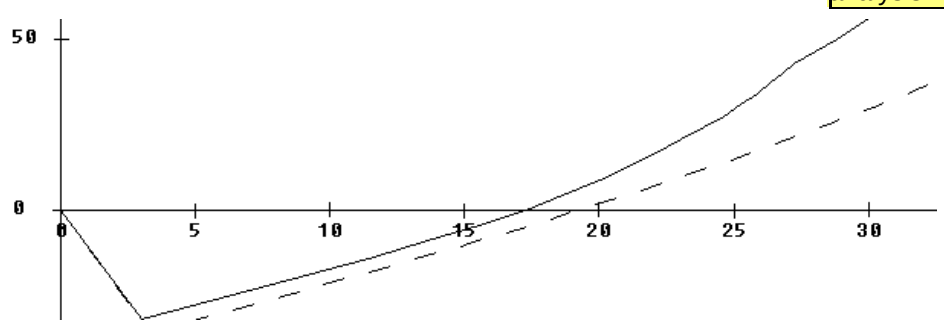
Nominal Strength

$$M_{mph_int} := 23 \frac{kip \cdot ft}{ft}$$

$$FS_{MPH1} := \frac{M_{mph_int}}{M_{mid}}$$

$$FS_{MPH1} = 0.366$$

Below 1.5. Perform reliability analysis



Reliability analysis (12th Street Wall 1 Com Bending and Thrust PoF.xmcd) indicates that wall 1 is unreliable, PF = 100%.

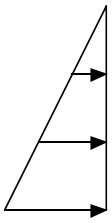
WALL 2, (North wall)

$K_o = 0.562$ $\quad$ $BLANKET = 26\ ft$ $\quad$ $HGL = 8.2\ ft$

$Hwall_2 = 29.5\ ft$ $\quad$ $\gamma_w = 62.4\ pcf$

Wall Loading

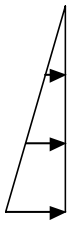
Soil or Soil & Water



$H_2 := Hwall_2$

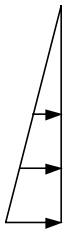
$H_2 = 29.5\ ft$

OR



$H_2 = 29.5\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_2 - HEAD$

$H''_3 := Hwall_2 + HGL - HEAD$

$H_3 := if\left(Hwall_2 > BLANKET, H''_3, H'_3\right)$

$H_3 = 37.7\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 1988\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_w = 3306.8\ psf$

$H'_3 = 38.804\ ft$

$H''_3 = 37.7\ ft$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_2 := if\left(W_s > W_w, W_s, W_w\right)$

$W_2 = 3307\ \frac{lb}{ft^2}$

WALL 2, (North Wall)

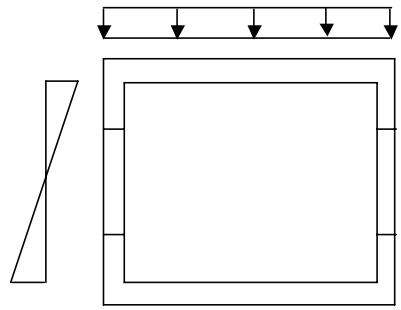
SHEAR

$$THRUST_2 := \frac{W_1 \cdot L_1}{2} \quad THRUST_2 = 21494 \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis (if necessary or applicable)

$$V'_{u2} := W_2 \cdot \frac{L_2}{2}$$

$$V'_{u2} = 11849 \frac{lb}{ft}$$



$$V_{u2} := 2 \frac{V'_{u2}}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2 \right) \quad V_{u2} = 8474 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$D_2 = 10 \, in$$

$$A_g := D_2 \cdot 12 \frac{in}{ft} \quad A_g = 120 \frac{in^2}{ft}$$

EM 1110-2-2104 EQ (5-1) Does not Apply

$$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_2 \right) \right]$$

ACI EQ (11-4)

$$\phi V_{n2} = 10384 \frac{lb}{ft}$$

$$d_2 = 7.25 \, in$$

$$\frac{THRUST_2 \cdot \frac{in^2}{lb}}{A_g} = 179.118$$

$$Check2 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$$

Check2 = "NO GOOD"

Factor of Safety

$$FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$$

$$FS_{S2} = 1.225$$

ANALYZE WALL 2 (North wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\,\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_2 - \frac{(W_2 - 0psf)\cdot 3ft}{29.5ft} = 2970.515\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{2971psf\cdot L_2^{\,2}}{8}$ $M_{mid} = 19.074\,\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_2^{\,2}}{12}$ $M_{end} = 0\,\frac{kip\cdot ft}{ft}$

#4's @ 6 in. inside face (mid-plate moment) $A_{mph} := 0.393\,\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 0.514\,in$

$$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right) \qquad \phi M_{mph} = 9.0\,\frac{kip\cdot ft}{ft}$$

no bars outside face (edge-plate moment) $A_{eph} := 0\,\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right) \qquad \phi M_{eph} = 0.0\,\frac{kip\cdot ft}{ft}$$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO GOOD")$ $CheckMPH = "NO GOOD"$

Factor of Safety $FS_{MPH2} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH2} = 0.472$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO GOOD")$ $CheckEPH = "NO GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3306.8\,psf$

$$W_2 - \frac{(W_2 - 0psf)\cdot 3ft}{29.5ft} = 2970.515\,psf$$

Thrust present 3 ft from bottom of wall.

$$THRUST := \frac{2971psf \cdot L_1}{2}$$ $$THRUST = 19311.5 \frac{lb}{ft}$$

$d_1 = 7.125\,in$ $A_g = 0.833\ ft$

$C_{c1} = 2.5\,in$

Loads from initial screening calculation: $M_u := 19.1kip \cdot ft$ $F_y = 40\,ksi$ $f'_c = 3000\,psi$

$Thrust := 19.3kip$

Compute Strength and Factor of Safety When Thrust is Included

NOTE: Reassign variables to allow use of the following routine

Steel Modulus of Elasticity: $E_s := 29000ksi$

Section Info: Thickness of Section, $D := 10in$ $b := 1ft$ (1 foot strip)

$A_1 := 0.393in^2$ $A_2 := 0in^2$ ONLY 1 LAYER OF STEEL PRESENT

$d_1 := d_2$ $d_2 := d_2$ Distance from compression face to bar centerline

$d_1 = 7.25\,in$

$$\beta 1 := if\left[f'_c \le 4ksi, 0.85, if\left[\left(f'_c\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_c}{ksi} - 4\right)\right]\right]$$ $\beta 1 = 0.85$

Assume c=0.2*D $i := 1..2$

Given $c := .2 \cdot D$ $c = 2\,in$ $Thrust = 19.3\,kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_y \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_y \\ -F_y - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_y \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $c = 1.346\,in$ $a := if\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right)$ $a = 1.144\,in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s \cdot \varepsilon s_i > F_y, F_y, \text{if}(E_s \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'_c, 0ksi) \cdot A_i$ $\qquad$ $Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.013	-40	ksi	0.393	in^2	-15.72	kip	35.37	kip·in
0.604		-0.013	-40		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 35\ kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 155\ kip \cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 15.87\ kip \cdot ft$

$FS_{MPH2} := \frac{M_n}{M_u}$

$Check := \text{if}(FS_{MPH2} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\qquad$ $FS_{MPH2} = 0.831$ $\qquad$ Check = "NO GOOD"

Reliability analysis was performed (12th St Wall 2 Com Bending and Thrust_PoF.xmcd) The calculated reliability is Pf = 72.81%. Based on these results, Wall 2 is **NOT** considered reliable.

Uplift on Structure

Refer to Figure 2 for the identified areas concerning uplift calculations.

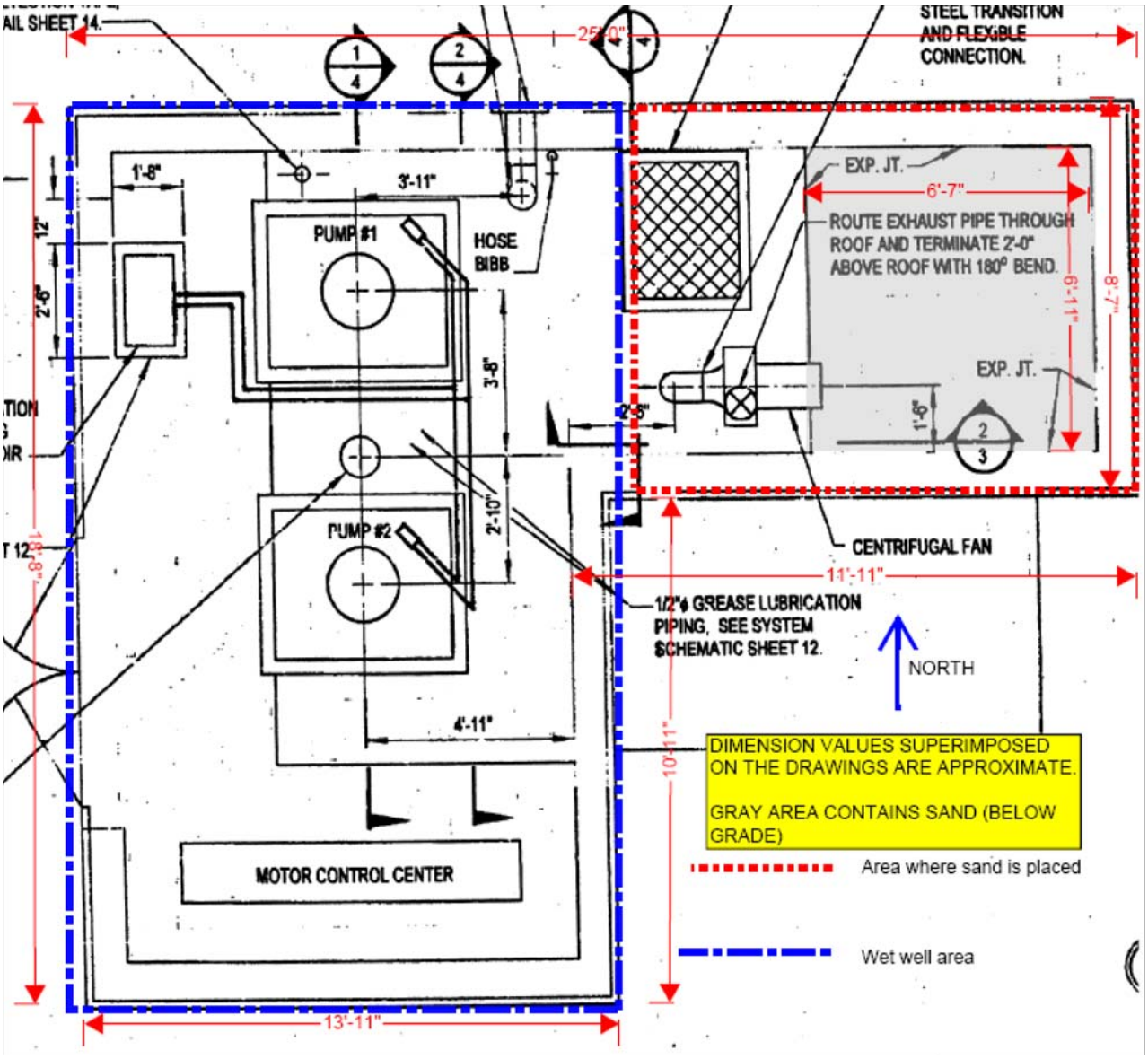


Figure 2. Ground Floor Elev. 762.17, 12th St Pump Station after 1995 modifications (DR&G Engineers) .

1. Wet well weight.

The wetwell weight includes all of the following within the outlined area in Figure 2:
1. Wetwell walls
2. Top slab within the identified wetwell area (Note that the top slab extends beyond the wetwell footprint).
3. Concrete wall above grade within the wetwell area.
4. Wetwell base slab
5. Estimated weight of steel used to rehabilitate the wetwell for strength deficiencies.

The above grade concrete wall size and top slab area is not shown on the DR&G Engineers drawings. The values used in calculations are conservative (low) estimate values when used to calculate uplift.

1. Wet well weight, con't.

Top Slab

$Top := 1.25ft \cdot (18ft + 8in) \cdot (13ft + 11in)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 48.708\,kip$

West Wall

$hwest := (762.17ft - 1.25ft) - (730.55ft - 10in)$ $\quad hwest = 31.203\ ft$

$West := hwest \cdot 13ft \cdot 10in$ $\quad Wwest := West \cdot 150pcf$ $\quad Wwest = 50.705\,kip$

East Wall

$heast := hwest$ $\quad heast = 31.203\ ft$

$East := heast \cdot 13ft \cdot 10in$ $\quad Weast := East \cdot 150pcf$ $\quad Weast = 50.705\,kip$

South Wall

$hsouth := hwest$ $\quad hsouth = 31.203\ ft$

$South := hsouth \cdot (7ft + 2in + 20in)\ 10in$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 34.454\,kip$

North Wall

$hnorth := hwest$ $\quad hnorth = 31.203\ ft$

$North := South$ $\quad Wnorth := Wsouth$ $\quad Wnorth = 34453.681\ lb$

Base Slab

$Wbase := 10in \cdot (7ft + 2in) \cdot 13ft \cdot 150pcf$ $\quad Wbase = 11.646\,kip$

North Wall above grade

$hnorth_{ag} := 5ft$ $\quad hnorth_{ag} = 5\ ft$

$North_{ag} := hnorth_{ag} \cdot 12in \cdot (13ft + 11in)$ $\quad Wnorth_{ag} := North_{ag} \cdot 150pcf$ $\quad Wnorth_{ag} = 10.437\,kip$

South Wall above Grade

$Wsouth_{ag} := Wnorth_{ag}$ $\quad Wsouth_{ag} = 10.437\,kip$

West Wall above Grade

$hwest_{ag} := 5ft$

$West_{ag} := hwest_{ag} \cdot (4ft + 6in) \cdot 12in$ $\quad Wwest_{ag} := West_{ag} \cdot 150pcf$ $\quad Wwest_{ag} = 3.375\,kip$

East Wall above Grade

$heast := 5ft$

$East_{ag} := (11ft + 9in) \cdot (12in) \cdot heast$ $\quad Weast_{ag} := East_{ag} \cdot 150pcf$ $\quad Weast_{ag} = 8.813\,kip$

1. Wet well weight, con't.

$Wsteel := 12kip$ Estimated weight of steel placed in wetwell for strength rehabilitation

Wet Well Weight

$Wwetwell := Wtop + Wwest + Weast + Wsouth + Wnorth + Wbase ... + Wnorth_{ag} + Wsouth_{ag} + Wwest_{ag} + Weast_{ag} + Wsteel$ $Wwetwell = 275.735\ kip$

2. Uplift On Wetwell Structure

$Awetwell := (13ft + 20in)\cdot(7ft + 2in + 20in)$ $Awetwell = 129.556\ ft^2$ Area that uplift is acting on

$H := ELEV_1 - (730.55ft - 10in)$ $H = 30.283\ ft$ $ELEV_1 = 760\ ft$

Bottom of blanket elev = 734

Bottom of wetwell elev = 729.72

Head, if structural foundation in blanket. $HGL = 8.2\ ft$

$U_1 := \left(\frac{BLANKET + HGL}{BLANKET}\cdot H\right)\cdot Awetwell\cdot\gamma_w$ $U_1 = 322.031\ kips$

$BLANKET = 26\ ft$
$HGL = 8.2\ ft$

$H = 30.283\ ft$

Head, if structural foundation extends through blanket.

$U_2 := \left[(H + HGL)\cdot(Awetwell)\cdot\gamma_w\right]$ $U_2 = 311.11\ kips$

$\left(\frac{BLANKET + HGL}{BLANKET}\cdot H\right) = 39.834\ ft$

$Uplift_{wet} := if\left(H > BLANKET, U_2, U_1\right)$

The uplift force acting on the wetwell structure $Uplift_{wet} = 311.11\ kips$

3. Wetwell Structure Factor of Safety for Flotation

Weight of water that is present during the event

$HEAD := 3ft$ $Wellarea := 13ft\cdot(7ft + 2in)$ $Wellarea = 93.167\ ft^2$

$Water := HEAD\cdot Wellarea\cdot 62.4pcf$ $Water = 17.441\ kip$

$heel := 0ft$ The wet well does not have heels

Volume of Soil above the heels

$volsoil := (1ft - 1ft)\cdot heel + (1ft - 1ft)\cdot heel$ $volsoil = 0\ ft^3$

$volsoil\cdot\left(\gamma - \gamma_w\right) = 0\ kip$ $volsoil\cdot\gamma_w = 0\ kip$

$Wwetwell + Water + volsoil\cdot\left(\gamma - \gamma_w\right) = 293.176\ kip$ $Uplift_{wet} = 311.11\ kip$

$Stability := \frac{Wwetwell + Water + volsoil\cdot\left(\gamma - \gamma_w\right)}{Uplift_{wet} - volsoil\cdot\gamma_w}$ $Stability = 0.942$ $(H + HGL) = 38.483\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $StabilityCheck = "Not Okay"$

Side Friction As Aid To Wetwell Factor of Safety Against Uplift

SOIL

$\phi := 26deg$ INTERNAL FRICTION ANGLE

$\gamma := 120pcf$ SOIL UNIT WEIGHT

$\gamma_w := 62.4pcf$ WATER UNIT WEIGHT

$\gamma_{sub} := (\gamma - \gamma_w)$ SUBMERGED SOIL UNIT WEIGHT $\gamma_{sub} = 57.6\,pcf$

$K_o := 1 - \sin(\phi)$ AT-REST SOIL COEFFICIENT. ASSUME THAT THE WALLS DO NOT MOVE

$K_o = 0.562$

$\mu := 0.25$ FRICTION COEFICIENT BETWEEN STRUCTURE AND SOIL.

$U := 2401psf$ UPLIFT PRESSURE ACTING ON BASE OF STRUCTURE

$c := 5ft$ ASSUMED CRACK DEPTH

STRUCTURE

$H := 29.5ft$ DEPTH OF STRUCTURE BELOW GRADE

$per := 13ft \cdot 2 + 7.167ft \cdot 2$ STRUCTURE PERIMETER $per = 40.334\ ft$

$W_s := 275.74kip$ STRUCTURE DEAD WEIGHT

$W_{water} := 17.44kip$ WEIGHT OF WATER LOCATED IN STRUCTURE.

$Soil := 0kip$ WEIGHT OF SOIL LOCATED ON TOP OF THE BASE SLAB HEEL OR WALL STEP. EQUAL TO ZERO IF NO HEEL IS PRESENT.

UPLIFT PARAMETERS:

$U_{s_ec} := 311.11kip$ UPLIFT FORCE FOR THE EXISTING CONDITION

V. Calculation of Pv

1. Effective Lateral Load that Contributes to Side Friction

$\gamma = 120\,\text{pcf}$ $\quad c = 5\ \text{ft}$ $\quad H = 29.5\ \text{ft}$

$R_1 := (H - c)\cdot\left(\frac{c + H}{2}\right)\cdot\gamma \qquad R_1 = 1576.27\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

$R_2 := (H - c)\cdot U\cdot\left(\frac{c}{2\cdot H} + \frac{1}{2}\right) \qquad R_2 = 1069.103\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

$R_1 - R_2 = 507.167\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

2. Force Created By Side Friction, Pv

$K_o = 0.562$ $\quad \mu = 0.25$ $\quad per = 40.334\ \text{ft}$

$P_v := (R_1 - R_2)\cdot K_o\cdot\mu\cdot per \qquad P_v = 92.41\ \text{kip}$

Stability Without the Aid of Side Friction, Existing Condition

$Stability_{nofric_ec} := \frac{W_s + W_{water} + Soil}{U_{s_ec}} \qquad Stability_{nofric_ec} = 0.942$

Stability with the Aid of Side Friction, Existing Conditon

$Stability_{fric_ec} := \frac{W_s + W_{water} + Soil + P_v}{U_{s_ec}} \qquad Stability_{fric_ec} = 1.239$

The uplift factor of safety for various HGL elevations is shown below. The wetwell factor of safety for the existing condition is adequate when skin friction is considered.

Uplift factors of safety versus HGL

HGL	FS no skin friction	FS with skin friction	
768.2	0.942	1.24	Existing Condition
771	0.878	1.11	
772	0.858	1.07	n500+3 Event

Assuming that the skin friction CANNOT be taken into account perform the following analysis... .

Determine the amount of additional weight required to achieve FS = 1.1 of the wetwell structure

$Wreqdwetwell := Uplift_{wet} \cdot 1.1 - \left[Wwetwell + Water + volsoil \cdot \left(\gamma - \gamma_w\right)\right]$ $\quad Wreqdwetwell = 49.045\,kip$

4. Structure containing sand.

$Sa := (6ft + 7in) \cdot (6ft + 11in) \cdot (760.92ft - 748.71ft)$ $\quad Sa = 555.979\,ft^3$

$Wsa := 100pcf \cdot Sa$ $\quad Wsa = 55.598\,kip$

Top Slab

$Top := 1.25ft \cdot (11ft + 1in) \cdot (8ft + 7in)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 17.837\,kip$

East Wall

$heast := (762.17ft - 1.25ft - 748.71ft)$ $\quad heast = 12.21\ ft$

$East := heast \cdot (6ft + 11in) \cdot 10in$ $\quad Weast := East \cdot 150pcf$ $\quad Weast = 10.557\,kip$

South Wall

$hsouth := heast$ $\quad hsouth = 12.21\ ft$

$South := hsouth \cdot (11ft + 1in) \cdot 10in$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 16.916\,kip$

North Wall

$hnorth := heast$ $\quad hnorth = 12.21\ ft$

$North := South$ $\quad Wnorth := Wsouth$ $\quad Wnorth = 16.916\,kip$

Base Slab

$Wbase := (748.71ft - 748.21ft) \cdot (11ft + 1in)(6ft + 11in + 20in) \cdot 150pcf$ $\quad Wbase = 7.135\,kip$

Interior Wall

$hint := heast$ $\quad hint = 12.21\ ft$

$Int := heast \cdot (6ft + 11in) \cdot (1.5ft)$ $\quad Wint := Int \cdot 150pcf$ $\quad Wint = 19.002\,kip$

Above grade concrete walls

North Wall above grade

$hnorth_{ag} := 5ft$ $\quad hnorth_{ag} = 5\ ft$

$North_{ag} := hnorth_{ag} \cdot 12in \cdot (11ft + 1in)$ $\quad Wnorth_{ag} := North_{ag} \cdot 150pcf$ $\quad Wnorth_{ag} = 8.312\,kip$

4. Structure containing sand, con't

South Wall above Grade

$Wsouth_{ag} := Wnorth_{ag}$ $\quad Wsouth_{ag} = 8.312\ kip$

East Wall above Grade

$heast_{ag} := 5ft$

$East_{ag} := heast_{ag} \cdot (6ft + 11in) \cdot 12in$ $\quad Weast_{ag} := East_{ag} \cdot 150pcf$ $\quad Weast_{ag} = 5.187\ kip$

$Wbldg := 3kip$ Estimated weight of steel building enclosure, (conservative)

Structure Containing Sand Weight

$Wsand := Wsa + Wtop + Weast + Wsouth + Wnorth + Wbase + Wint \ldots + Wnorth_{ag} + Wsouth_{ag} + Weast_{ag} + Wbldg$ $\quad Wsand = 168.773\ kip$

5. Uplift On Sand Structure

$Asand := (11 \cdot ft + 11 \cdot in) \cdot (6 \cdot ft + 11 \cdot in + 20 \cdot in)$ $\quad Asand = 102.285\ ft^2$ Area uplift acts on

$ELEVsand := 748.21ft$ Elevation at bottom of sand structure base slab

Bottom of blanket elev = 734

Bottom of wetwell elev = 729.72

$H := ELEV_1 - ELEVsand$ $\quad H = 11.79\ ft$ $\quad ELEV_1 = 760\ ft$

Head, if structural foundation in blanket. $\quad HGL = 8.2\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot Asand \cdot \gamma_w \qquad U_1 = 98.983\ kips$$

$BLANKET = 26\ ft$
$HGL = 8.2\ ft$

Head, if structural foundation extends through blanket.

$H = 11.79\ ft$

$$U_2 := \left[(H + HGL) \cdot (Asand) \cdot \gamma_w\right] \qquad U_2 = 127.588\ kips$$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 15.508\ ft$$

$$Uplift_{sand} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the sand structure $\quad Uplift_{sand} = 98.983\ kips$

6. Sand Structure Factor of Safety for Flotation

Weight of soil on the sand structure heels

$volsoil := (1ft - 1ft) \cdot heel + (1ft - 1ft) \cdot heel$ $\quad volsoil = 0\ ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 0\ kip$ $\quad volsoil \cdot \gamma_w = 0\ kip$

$$Stability := \frac{Wsand + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{sand} - volsoil \cdot \gamma_w}$$

$Stability = 1.705$ $\quad (H + HGL) = 19.99\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $\quad StabilityCheck = "Okay"$

7. Amount of weight available to assist the wetwell flotation.

$$Wsandforwetwell := Wsand - \frac{1.10}{Stability} \cdot \left[Wsand + volsoil \cdot \left(\gamma - \gamma_w\right)\right]$$ $Wsandforwetwell = 59.891\ kip$

8. Determine if the pump station (without gatewell) meets the required factor of safety for uplift

To satisfy the overall factor of safety against flotation, the amount of weight available in the sand structure (Wsandforwetwell) must be greater than the amount of weight required such that the wetwell structure meets the required factor of safety (Wreqdwetwell).

$PumpStationmeetsuplift := if(Wsandforwetwell \geq Wreqdwetwell, "okay", "NO\ GOOD")$

$PumpStationmeetsuplift = "okay"$

This is assuming that the force can be transfered in shear at the wetwell and sand structure interface. Determine the length of wall required to transfer the shear force.

Wall Shear Strength

$Wsandforwetwell = 59.891\ kip$	Amount of sand weight available to increase the wetwell FS for flotation
$Wreqdwetwell = 49.045\ kip$	This is the amount of shear that has to be transferred in the walls
$t_{wall} := 10in$	Wall thickness
$d_2 = 7.25\ in$	Effective depth of the wall reinforcement
$f'_c = 3\ ksi$	Concrete compressive strength

$$hreqd := \frac{Wreqdwetwell}{0.85 \cdot 2 \cdot \sqrt{f'_c} \cdot d_2 \cdot \frac{\sqrt{lb}}{in}}$$ $hreqd = 6.054\ ft$

$hnorth + hsouth = 24.42\ ft$

$Wallokforshear := if[hreqd \leq (hnorth + hsouth), "okay", "NO\ GOOD"]$ $Wallokforshear = "okay"$

9. Consider the gatewell attached to the station.

$Wgate := [[(5.5ft \cdot 10ft) - (3.833ft \cdot 8.33ft)] \cdot (764ft - 729.72ft) + 10in \cdot (3.833ft \cdot 8.33ft)] \cdot 150pcf - (5ft \cdot 6ft \cdot 10in) \cdot 150pcf$

$Wgate = 118.873\ kip$

$Agate := (5.5ft + 0in) \cdot (10ft + 0in)$ $\quad Agate = 55\ ft^2$ $\quad$ Area that uplift is acting on

$H := ELEV_1 - (729.72ft)$ $\quad H = 30.28\ ft$ $\quad ELEV_1 = 760\ ft$

Bottom of blanket elev = 734

Bottom of wetwell elev = 729.72

Head, if structural foundation in blanket. $\quad HGL = 8.2\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot Agate \cdot \gamma_w \qquad U_1 = 136.696\ kips$$

$BLANKET = 26\ ft$

$HGL = 8.2\ ft$

$H = 30.28\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (Agate) \cdot \gamma_w\right] \qquad U_2 = 132.063\ kips$$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 39.83\ ft$$

$$Uplift_{gate} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the wetwell structure $\quad Uplift_{gate} = 132.063\ kips$

$$Stability_{gate} := \frac{Wgate}{Uplift_{gate}} \qquad Stability_{gate} = 0.9$$

$(H + HGL) = 38.48\ ft$

ASSUMES THE GATEWELL HAS NO HEELS (CONSERVATIVE)

Amount of weight required to achieve FS=1.10

$Wreqdgate := \left(Uplift_{gate} \cdot 1.10\right) - Wgate$ $\quad Wreqdgate = 26.397\ kip$

Place sand in the gatewell as a fix....

$hsand := 5ft$

$Wgatesand := (3.833ft \cdot 8.33ft \cdot hsand) \cdot 100pcf$ $\quad Wgatesand = 15.964\ kip$

Amount of weight left over to aid pump station

$Wgateavail := Wgatesand - Wreqdgate$ $\quad Wgateavail = -10.432\ kip$

$Wsandforwetwell = 59.891\ kip$ $\quad Wreqdwetwell = 49.045\ kip$

Weight required such that the entire pump station (with gatewell) meets FS = 1.1 for uplift

$Wreqd := Wreqdwetwell - Wsandforwetwell$ $\quad Wreqd = -10.846\ kip$

The addditional weight placed in the gatewell (sand) must be equal to or greater than Wreqd

$PumpStationmeetsupliftwithgatesand := if(Wgateavail \geq Wreqd, "okay", "NO GOOD")$

$PumpStationmeetsupliftwithgatesand = "okay"$

IN SUMMARY, IF 5 FT OR MORE OF SAND IS PLACED IN THE GATEWELL, THE PUMP STATION (WITH GATEWELL) HAS A FS EQUAL TO OR GREATER THAN 1.1. NOTE THAT THE GATEWELL DIMENSIONS USED IN THE CALCULATIONS ARE APPROXIMATE. THE ACTUAL AMOUNT WILL REQUIRE VERIFICATION OF THE GATEWELL WALL THICKNESS AND SIZE.

Wet Well BASE SLAB ANALYSIS

$Water = 17.441\ kip$ WEIGHT OF WATER

$BaseSlab := Awetwell$ $BaseSlab = 129.556\ ft^2$ $Uplift_{wet} = 311.11\ kip$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$ $\frac{Wwetwell + Water}{BaseSlab} = 2262.934\ psf$

$W' := if\left(Stability < 1.0, \frac{Uplift_{wet}}{BaseSlab}, \frac{Wwetwell}{BaseSlab}\right)$ $W' = 2128.314\ psf$ $\frac{Uplift_{wet}}{BaseSlab} = 2401.36\ psf$

$W' = 2128.314\ psf$

$W' := 2401psf$ $\frac{Wwetwell + volsoil\cdot(\gamma - \gamma_w)}{BaseSlab} = 2128.314\ psf$

$Slab_t := 10in$ $L_{strip} := (7ft + 2in + 20in)$ $Wall_t := D_1$ $Slab_d := d_f$ effective depth of slab

Analyze slab with one-way action

$L := 7ft + 2in$ $Slab_d = 6.625\ in$ $D_1 = 10\ in$

$V'_{ub} := \frac{W'\cdot L}{2}$ $V'_{ub} = 8603.583\ \frac{lb}{ft}$ $d_f = 6.625\ in$

$D_f = 0.833\ ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}}\cdot\left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 6716\ \frac{lb}{ft}$

$A_g := Slab_t\cdot 12\frac{in}{ft}$ $A_g = 120\frac{in^2}{ft}$ $Water = 17440.8\ lb$

$\phi V_{nb} := \phi_V\cdot\left[2\cdot\left(1 + \frac{THRUST_S}{2000\cdot A_g}\cdot\frac{in^2}{lb}\right)\cdot\left(\sqrt{f'_c}\cdot\frac{lb^{.5}}{in}\right)\cdot\left(12\cdot\frac{in}{ft}\cdot d_f\right)\right]$ ACI EQ (11-4) $\phi V_{nb} = 8709\ \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5V_{ub}, "OKAY", "NO GOOD"\right)$ $CheckBase = "NO\ GOOD"$

Factor of Safety $FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$ $FS_{Sb} = 1.297$

Slab Bending

Assume simple span moment (conservative). One-way action, investigate the short span moment capacity.

$M_b := \frac{L^2 \cdot W'}{8}$ $\quad M_b = 15.415 \frac{kip \cdot ft}{ft}$

#6's @ 12 in.

$A_{mpl} := 0.442 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mpl} = 0.578\ in$

$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_f - \frac{a_{mpl}}{2} \right)$ $\quad \phi M_{mpl} = 9.3 \frac{kip \cdot ft}{ft}$

$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot M_b, "OKAY", "NO\ GOOD"\right)$ $\quad CheckMP = "NO\ GOOD"$

Factor of Safety $\quad FS_{Mb} := \frac{\phi M_{mpl}}{M_b}$ $\quad FS_{Mb} = 0.606$

Again, it is unknown what reinforcement exists in the wetwell slab.

FACTORS OF SAFETY SUMMARY

Five walls and the base slab were evaluated for strength. A summary of the results are below. The reliability analysis calculations are in separate MathCad files.

Analysis indicates that the wetwell walls are unreliable. Based on the Wall 2 calculations, the East and South wet well walls are also considered unreliable. The East and South wet well walls have the same span length, thickness and reinforcement as the North wet well wall (Wall 2) and are exposed to lateral loading equal to the lateral loading present on Wall 2.

The available drawings do not indicate reinforcement present in the wet well base slab. Assuming that #6 @ 12" are present in the base slab, the base slab factor of safety <1.0. Based on these results, the base slab is also considered unreliable.

WALL 1 (WEST WETWELL WALL)

BAD

Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S1} = 0.541$

Moment

$FS_{MPH1} = 0.366$ Mid-plate moment

$FS_{EPH1} =$ ▮ Edge-of-plate moment (not applicable)

RELIABILITY ANALYSIS INDICATES THAT WALL 1 IS UNRELIABLE, Pf = 100%.

WALL 2 (NORTH WETWELL WALL)

BAD

Shear

$FS_{S2} = 1.225$ Reliability analysis indicates the wall being reliable for shear(Pf<0.0001)

Moment

$FS_{MPV2} =$ ▮ not applicable

$FS_{EPV2} =$ ▮ not applicable

$FS_{MPH2} = 0.831$ Reliability analysis indicates that the wall is uneliable for moment, Pf= 72.81%

$FS_{EPH2} =$ ▮ not applicable

OVERALL, RELIABILITY ANALYSIS INDICATES THAT WALL 2 IS UNRELIABLE.

BASE SLAB STATUS: SUSPECT. The base slab reinforcement is unknown.

Shear

$FS_{Sb} = 1.297$

Moment

$FS_{Mb} = 0.606$ Analyzed as a simple span, one-way action.

STABILITY GOOD - SEE NOTE

NOTE: The wetwell stability is adequate (FS=1.10) IF skin friction is taken into account. Analyzed by itselft, the gatewell factor of safety is 0.9 when no addtional weight (sand) is placed in the structure nor the benefit of skin friction considered. When skin friction is considered, the gatewell factor of safety becomes 1.08 (calculations not shown). Based on these analysis, the pump station factor of safety against is okay for the existing condtion.

Wetwell Uplift factors of safety versus HGL

HGL	FS no skin friction	FS with skin friction	
768.2	0.942	1.24	Existing Condition $1.24 - 0.942 = 0.298$
771	0.878	1.11	
772	0.858	1.07	n500+3 Event

Probability of Failure
12th Street Pump StationWall 1

STA 129+20

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the screening criteria derived from the EM and therefore, the following reliability analysis was performed to calculate the reliability. See the MathCad analysis for the complete existing condition strength check with water to the top of protection.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength
3. Material Properties used:
   Concrete Design Strength, $f'_c := 3000 \cdot psi$; Mean Concrete Strength, $f'_{cM} := 1.25 f'_c$
   Steel Design Strength, $F_y := 40 \cdot ksi$; Mean Steel Strength, $F_{yM} := 1.25 F_y$
4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. Caclulating Factors of Safety

Condition under consideration from strength check: Wall Flexural Steel (from mathcad strength analysis).

Design Concrete Strength

Steel Modulus of Elasticity: $E_s := 29000ksi$

Section Info: Thickness of Section, $D := 10in$ $b := 1ft$ (1 foot strip)

$A_1 := 0.884in^2$ $A_2 := 0in^2$ ONLY 1 LAYER OF STEEL PRESENT

$C_1 := 2.5in$ $C_2 := 2.5in$ Distance from compression face to bar centerline

$d_1 := D - C_1 - 0.375in$ $d_2 := C_2 + 0.375in$

$d_1 = 7.125\ in$

Loads from initial screening calculation: $M_u := 62.8kip \cdot ft$

$Thrust := 10.7kip$

Mean Concrete Strength and Steel Yield Strength

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $i := 1..2$

Given $c := .2\cdot D$ $c = 2\ in$ $Thrust = 10.7\ kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \\ F_{yM} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{array}\right]\cdot A_i\right] \ldots$$
$$+ 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := Find(c)$ $c = 1.689\ in$ $a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right)$ $a = 1.435\ in$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}(E_s\cdot\varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s\cdot\varepsilon s_i))$

$Fs_i := fs_i\cdot A_i - \text{if}(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi)\cdot A_i$ $\qquad$ $Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.594	ft	-0.01	-50	ksi	0.884	in^2	-44.2	kip	93.925	kip·in
0.24		-0.002	-50		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 94\,kip\cdot in$

$Mc := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 235\,kip\cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 27.42\,kip\cdot ft$

$FS_B := \frac{M_n}{M_u}$ $\quad$ $Check := \text{if}(FS_B > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\quad$ $FS_B = 0.437$ $\qquad$ Check = "NO GOOD"

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM}\cdot 0.14$ $\qquad$ $f'_{cU} = 4275\,psi$

$\beta 1 := \text{if}\left[f'_{cU} \le 4ksi, 0.85, \text{if}\left[(f'_{cU}) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cU}}{ksi} - 4\right)\right]\right]$ $\qquad$ $\beta 1 = 0.836$

Assume c=0.2*D $\qquad$ $i := 1..2$

Given $\qquad$ $c := .2\cdot D$ $\qquad$ $c = 2\,in$ $\qquad$ $Thrust = 10.7\,kip$

Upper Concrete Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ +-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \\ F_{yM}-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM}-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{array}\right]\cdot A_i\right] \ldots$$

$$+\,0.85\cdot f'_{cU}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 1.506\ \text{in}$ $\quad a := \text{if}\left(\beta1\cdot c > D, D, \beta1\cdot c\right)$ $\quad a = 1.259\ \text{in}$

Find Moment:

$\varepsilon s_i := \left(c-d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s\cdot\varepsilon s_i < 0-F_{yM}, 0-F_{yM}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cU}, 0\text{ksi}\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2}-d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.594 ft	-0.011	-50 ksi	0.884 in^2	-44.2 kip	93.925 kip·in
0.24	-0.003	-50	0	0	0

$Mst_U := \sum_{i=1}^{2} Ms_i$ $\quad Mst_U = 94\ \text{kip}\cdot\text{in}$

$Mc_U := 0.85\cdot f'_{cU}\cdot a\cdot b\cdot\left(\frac{D}{2}-\frac{a}{2}\right)$ $\quad Mc_U = 240\ \text{kip}\cdot\text{in}$

$M_{nU} := \left(Mc_U + Mst_U\right)$ $\quad M_{nU} = 27.82\ \text{kip}\cdot\text{ft}$

$FS_{BU} := \frac{M_{nU}}{M_u}$ $\quad Check_U := \text{if}\left(FS_{BU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BU} = 0.443$ $\quad Check_U = \text{"NO GOOD"}$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\qquad f'_{cL} = 3225\,psi$

$$\beta 1 := \text{if}\left[f'_{cL} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cL}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cL}}{ksi} - 4\right)\right]\right]$$

$\beta 1 = 0.85$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2 \cdot D \qquad c = 2\,in \qquad Thrust = 10.7\,kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \\ F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots$$

$$+ 0.85 \cdot f'_{cL} \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \qquad c = 1.963\,in \qquad a := \text{if}\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right) \qquad a = 1.669\,in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(E_s \cdot \varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s \cdot \varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cL}, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.594	ft	-0.008	-50	ksi	0.884	in^2	-44.2	kip	93.925	kip·in
0.24		-0.001	-50		0		0		0	

Lower Concrete Strength, con't

$$Mst_L := \sum_{i=1}^{2} Ms_i \qquad Mst_L = 94\,kip\cdot in$$

$$Mc_L := 0.85\cdot f'_{cL}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right) \qquad Mc_L = 229\,kip\cdot in$$

$$M_{nL} := \left(Mc_L + Mst_L\right) \qquad M_{nL} = 26.88\,kip\cdot ft$$

$$FS_{BL} := \frac{M_{nL}}{M_u} \qquad Check_L := if\left(FS_{BL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$FS_{BL} = 0.428$ $\qquad$ $Check_L = \text{"NO GOOD"}$

Upper Steel Yield Strength

$$F_{yU} := F_{yM} + F_{yM}\cdot 0.14 \qquad F_{yU} = 57\,ksi$$

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2\cdot D \qquad c = 2\,in \qquad Thrust = 10.7\,kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yU} \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \\ F_{yU} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yU} \\ -F_{yU} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yU} \end{array}\right]\cdot A_i\right] \ldots$$
$$+ 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := Find(c) \qquad c = 1.879\,in \qquad a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right) \qquad a = 1.597\,in$

Upper Steel Yield Strength, con't

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad fs_i := \text{if}(E_s\cdot\varepsilon s_i > F_{yU}, F_{yU}, \text{if}(E_s\cdot\varepsilon s_i < 0 - F_{yU}, 0 - F_{yU}, E_s\cdot\varepsilon s_i))$

$Fs_i := fs_i\cdot A_i - \text{if}(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi)\cdot A_i$ $\qquad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.594 ft	-0.008	-57 ks	0.884 in^2	-50.388 kip	107.074 kip·in
0.24	-0.002	-40.389	0	0	0

$Mst_{cU} := \sum_{i=1}^{2} Ms_i$ $\qquad Mst_{cU} = 107\,kip\cdot in$

$Mc_{cU} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad Mc_{cU} = 257\,kip\cdot in$

$M_{ncU} := (Mc_{cU} + Mst_{cU})$ $\qquad M_{ncU} = 30.31\,kip\cdot ft$

$FS_{BcU} := \frac{M_{ncU}}{M_u}$ $\qquad Check_{cU} := \text{if}(FS_{BcU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$

$FS_{BcU} = 0.483$ $\qquad Check_{cU} = \text{"NO GOOD"}$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM}\cdot 0.14$ $\qquad F_{yL} = 43\,ksi$

$\beta 1 := \text{if}\left[f'_{cM} \le 4ksi, 0.85, \text{if}\left[(f'_{cM}) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right]$ $\qquad \beta 1 = 0.85$

Assume c=0.2*D $\qquad i := 1..2$

Given $\qquad c := .2\cdot D$ $\qquad c = 2\,in$ $\qquad Thrust = 10.7\,kip$

Lower Steel Yield Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2} \left[\left[\begin{cases} (c - d_i)\cdot\frac{0.003}{c}\cdot E_s \ldots + -\text{if}(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0) & \text{if } (c - d_i)\cdot\frac{0.003}{c}\cdot E_s < F_{yL} \\ F_{yL} - \text{if}(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0) & \text{if } (c - d_i)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yL} \\ -F_{yL} - \text{if}(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0) & \text{if } (c - d_i)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yL} \end{cases} \right] \cdot A_i \right] \ldots + 0.85\cdot f'_{cM}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 1.498\ \text{in}$ $\quad a := \text{if}(\beta1\cdot c > D, D, \beta1\cdot c)$ $\quad a = 1.274\ \text{in}$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}(E_s\cdot\varepsilon s_i > F_{yL}, F_{yL}, \text{if}(E_s\cdot\varepsilon s_i < 0 - F_{yL}, 0 - F_{yL}, E_s\cdot\varepsilon s_i))$

$Fs_i := fs_i\cdot A_i - \text{if}(\beta1\cdot c > d_i, 0.85\cdot f'_{cM}, 0\text{ksi})\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.594	ft	-0.011	-43	ksi	0.884	in^2	-38.012	kip	80.776	kip·in
0.24		-0.003	-43		0		0		0	

$Mst_{cL} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cL} = 81\ \text{kip}\cdot\text{in}$

$Mc_{cL} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cL} = 213\ \text{kip}\cdot\text{in}$

$M_{ncL} := (Mc_{cL} + Mst_{cL})$ $\quad M_{ncL} = 24.44\ \text{kip}\cdot\text{ft}$

$FS_{BcL} := \frac{M_{ncL}}{M_u}$ $\quad Check_{cL} := \text{if}(FS_{BcL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$

$FS_{BcL} = 0.389$ $\quad Check_{cL} = \text{"NO GOOD"}$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{BcU} - FS_{BcL}$ $\Delta F_{Fy} = 0.093$

$\Delta F_{fc} := FS_{BU} - FS_{BL}$ $\Delta F_{fc} = 0.015$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2} \qquad \sigma_F = 0.047$$

$$V_F := \frac{\sigma_F}{FS_B} \qquad V_F = 0.108$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_B}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = -7.724$$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 5.66 \times 10^{-13}\,\%$

$M_u = 62.8\,\mathrm{kip \cdot ft}$

$\mathrm{Thrust} = 10.7\,\mathrm{kip}$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 100\,\%$

Note that P_F becomes 1% when Moment = 20 k*ft and Thrust= 5 kip

Probability of Failure
12th Street Pump StationWall 2

STA 129+20

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the screening criteria derived from the EM and therefore, the following reliability analysis was performed to calculate the reliability. See the MathCad analysis for the complete existing condition strength check with water to the top of protection.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength
3. Material Properties used:
   Concrete Design Strength, $f'_c := 3000 \cdot psi$; Mean Concrete Strength, $f'_{cM} := 1.25 f'_c$
   Steel Design Strength, $F_y := 40 \cdot ksi$; Mean Steel Strength, $F_{yM} := 1.25 F_y$
4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. Caclulating Factors of Safety

Condition under consideration from strength check: Wall Flexural Steel (from mathcad strength analysis).

Design Concrete Strength

Steel Modulus of Elasticity: $E_s := 29000ksi$

Section Info: Thickness of Section, $D := 10in$ $b := 1ft$ (1 foot strip)

$A_1 := 0.393in^2$ $A_2 := 0in^2$ ONLY 1 LAYER OF STEEL PRESENT

$C_1 := 2.5in$ $C_2 := 2.5in$ Distance from compression face to bar centerline

$d_1 := D - C_1 - 0.25in$ $d_2 := C_2 + 0.375in$

$d_1 = 7.25\ in$

Loads from initial screening calculation: $M_u := 19.1kip \cdot ft$

$Thrust := 19.3kip$

Mean Concrete Strength and Steel Yield Strength

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right]$$ $\beta 1 = 0.85$

Assume c=0.2*D $i := 1 .. 2$

Given $c := .2 \cdot D$ $c = 2\ in$ $Thrust = 19.3\ kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & \\ F_{yM} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_{cM} \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $c = 1.198\ in$ $a := if\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right)$ $a = 1.018\ in$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}(E_s\cdot\varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s\cdot\varepsilon s_i))$

$Fs_i := fs_i\cdot A_i - \text{if}(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi)\cdot A_i$ $\qquad$ $Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.015	-50	ksi	0.393	in^2	-19.65	kip	44.213	kip·in
0.24		-0.004	-50		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 44\,kip\cdot in$

$Mc := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 175\,kip\cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 18.26\,kip\cdot ft$

$FS_B := \frac{M_n}{M_u}$ $\quad$ $Check := \text{if}(FS_B > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\quad$ $FS_B = 0.956$ $\qquad$ $Check = \text{"NO GOOD"}$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM}\cdot 0.14$ $\qquad$ $f'_{cU} = 4275\,psi$

$\beta 1 := \text{if}\left[f'_{cU} \le 4ksi, 0.85, \text{if}\left[(f'_{cU}) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cU}}{ksi} - 4\right)\right]\right]$ $\qquad$ $\beta 1 = 0.836$

Assume c=0.2*D $\qquad$ $i := 1..2$

Given $\qquad$ $c := .2\cdot D$ $\qquad$ $c = 2\,in$ $\qquad$ $Thrust = 19.3\,kip$

Upper Concrete Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ +-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \\ F_{yM}-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM}-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{array}\right]\cdot A_i\right] \ldots$$

$$+0.85\cdot f'_{cU}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 1.068\ \text{in}$ $\quad a := \text{if}\left(\beta1\cdot c > D, D, \beta1\cdot c\right)$ $\quad a = 0.893\ \text{in}$

Find Moment:

$\varepsilon s_i := \left(c-d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s\cdot\varepsilon s_i < 0-F_{yM}, 0-F_{yM}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cU}, 0\text{ksi}\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2}-d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.604 ft	-0.017	-50 ksi	0.393 in^2	-19.65 kip	44.213 kip·in
0.24	-0.005	-50	0	0	0

$Mst_U := \sum_{i=1}^{2} Ms_i$ $\quad Mst_U = 44\ \text{kip}\cdot\text{in}$

$Mc_U := 0.85\cdot f'_{cU}\cdot a\cdot b\cdot\left(\frac{D}{2}-\frac{a}{2}\right)$ $\quad Mc_U = 177\ \text{kip}\cdot\text{in}$

$M_{nU} := \left(Mc_U + Mst_U\right)$ $\quad M_{nU} = 18.46\ \text{kip}\cdot\text{ft}$

$FS_{BU} := \frac{M_{nU}}{M_u}$ $\quad Check_U := \text{if}\left(FS_{BU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BU} = 0.967$ $\quad Check_U = \text{"NO GOOD"}$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\qquad f'_{cL} = 3225\,psi$

$$\beta 1 := \text{if}\left[f'_{cL} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cL}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cL}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2 \cdot D \qquad c = 2\,in \qquad Thrust = 19.3\,kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \\ F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_{cL} \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \qquad c = 1.393\,in \qquad a := \text{if}\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right) \qquad a = 1.184\,in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(E_s \cdot \varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s \cdot \varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cL}, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.013	-50	ksi	0.393	in^2	-19.65	kip	44.213	kip·in
0.24		-0.003	-50		0		0		0	

Lower Concrete Strength, con't

$$Mst_L := \sum_{i=1}^{2} Ms_i \qquad Mst_L = 44\,kip\cdot in$$

$$Mc_L := 0.85\cdot f'_{cL}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right) \qquad Mc_L = 172\,kip\cdot in$$

$$M_{nL} := \left(Mc_L + Mst_L\right) \qquad M_{nL} = 17.99\,kip\cdot ft$$

$$FS_{BL} := \frac{M_{nL}}{M_u} \qquad Check_L := if\left(FS_{BL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$FS_{BL} = 0.942$ $\qquad$ $Check_L = \text{"NO GOOD"}$

Upper Steel Yield Strength

$$F_{yU} := F_{yM} + F_{yM}\cdot 0.14 \qquad F_{yU} = 57\,ksi$$

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2\cdot D \qquad c = 2\,in \qquad Thrust = 19.3\,kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yU} \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \\ F_{yU} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yU} \\ -F_{yU} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yU} \end{array}\right]\cdot A_i\right] \ldots$$
$$+ 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := Find(c) \qquad c = 1.283\,in \qquad a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right) \qquad a = 1.09\,in$

Upper Steel Yield Strength, con't

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := \text{if}(E_s \cdot \varepsilon s_i > F_{yU}, F_{yU}, \text{if}(E_s \cdot \varepsilon s_i < 0 - F_{yU}, 0 - F_{yU}, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cM}, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.014	-57	ksi	0.393	in^2	-22.401	kip	50.402	kip·in
0.24		-0.004	-57		0		0		0	

$Mst_{cU} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cU} = 50\,kip \cdot in$

$Mc_{cU} := 0.85 \cdot f'_{cM} \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cU} = 186\,kip \cdot in$

$M_{ncU} := (Mc_{cU} + Mst_{cU})$ $\quad M_{ncU} = 19.68\,kip \cdot ft$

$FS_{BcU} := \frac{M_{ncU}}{M_u}$ $\quad Check_{cU} := \text{if}(FS_{BcU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$

$FS_{BcU} = 1.03$ $\quad Check_{cU} = \text{"NO GOOD"}$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14$ $\quad F_{yL} = 43\,ksi$

$\beta 1 := \text{if}\left[f'_{cM} \le 4ksi, 0.85, \text{if}\left[(f'_{cM}) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right]$ $\quad \beta 1 = 0.85$

Assume c=0.2*D $\quad i := 1 .. 2$

Given $\quad c := .2 \cdot D$ $\quad c = 2\,in$ $\quad Thrust = 19.3\,kip$

Lower Steel Yield Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2}\left[\left[\begin{cases}(c-d_i)\cdot\frac{0.003}{c}\cdot E_s \ldots + -\text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } (c-d_i)\cdot\frac{0.003}{c}\cdot E_s < F_{yL} \\ F_{yL} - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } (c-d_i)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yL} \\ -F_{yL} - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } (c-d_i)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yL}\end{cases}\right]\cdot A_i\right] \ldots + 0.85\cdot f'_{cM}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 1.113\ \text{in}$ $\quad a := \text{if}(\beta1\cdot c > D, D, \beta1\cdot c)$ $\quad a = 0.946\ \text{in}$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yL}, F_{yL}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yL}, 0 - F_{yL}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cM}, 0\text{ksi}\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.604 ft	-0.017	-43 ksi	0.393 in^2	-16.899 kip	38.023 kip·in
0.24	-0.005	-43	0	0	0

$Mst_{cL} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cL} = 38\ \text{kip}\cdot\text{in}$

$Mc_{cL} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cL} = 164\ \text{kip}\cdot\text{in}$

$M_{ncL} := (Mc_{cL} + Mst_{cL})$ $\quad M_{ncL} = 16.82\ \text{kip}\cdot\text{ft}$

$FS_{BcL} := \frac{M_{ncL}}{M_u}$ $\quad Check_{cL} := \text{if}(FS_{BcL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$

$FS_{BcL} = 0.881$ $\quad Check_{cL} = \text{"NO GOOD"}$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{BcU} - FS_{BcL}$ $\Delta F_{Fy} = 0.15$

$\Delta F_{fc} := FS_{BU} - FS_{BL}$ $\Delta F_{fc} = 0.025$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2}$$ $\sigma_F = 0.076$

$$V_F := \frac{\sigma_F}{FS_B}$$ $V_F = 0.079$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_B}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$$ $\beta_{LN} = -0.607$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 27.19\,\%$

$M_u = 19.1\,\mathrm{kip \cdot ft}$

$\mathrm{Thrust} = 19.3\,\mathrm{kip}$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 72.81\,\%$

Note that P_F becomes 1% when Moment = 13 k*ft and Thrust= 13 kip

CONCLUSIONS

The existing condition HGL elevation is 768.2. The existing wetwell walls and base slab are inadequate for the **HGL=768.2** event loading. The pump station is adequate for uplift stability for the **HGL=768.2** event loading when the benefit of skin friction is taken into account. The reliability analysis indicates that Wall 1 and Wall 2 are unreliable for the existing condition event loading.

NOTE: The wetwell stability is adequate (FS=1.10) IF skin friction is taken into account. Analyzed by itself, the gatewell factor of safety is 0.9 when no addtional weight (sand) is placed in the structure nor the benefit of skin friction considered. When skin friction is considered, the gatewell factor of safety becomes 1.08 (calculations not shown). Based on these analysis, the pump station factor of safety against is okay for the existing condtion.

Wetwell Uplift factors of safety versus HGL

HGL	FS no skin friction	FS with skin friction	
768.2	0.942	1.24	Existing Condition $1.24 - 0.942 = 0.298$
771	0.878	1.11	
772	0.858	1.07	n500+3 Event

END OF ANALYSIS

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $pcf := \frac{lb}{ft^3}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $kips := 1000lb$ $klf := \frac{kips}{ft}$

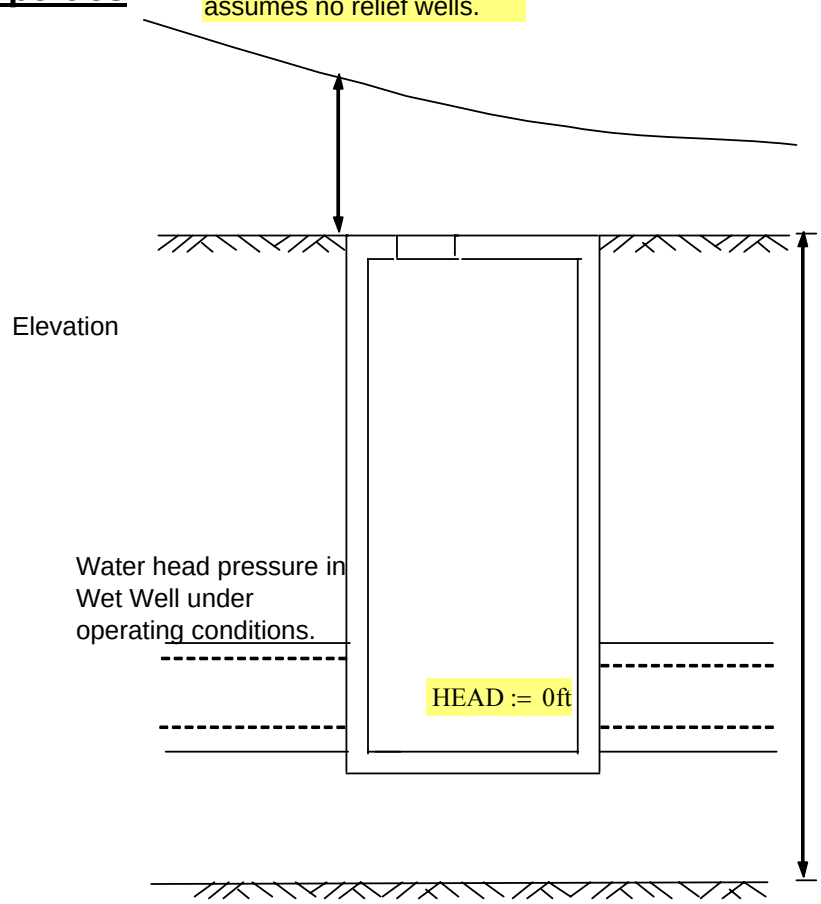
STRUCTURAL DRAWINGS OF THE 12TH STREET PUMP STATION COULD NOT BE FOUND. STRUCTURAL DRAWINGS OF THE MILL STREET PUMP STATION WERE AVAILABLE. THE 12TH STREET PUMP STATION AND MILL STREET PUMP STATION WERE BUILT ~ 1916. FROM THE DRAWINGS AVAILABLE, THE PUMP STATIONS ARE VERY SIMILIAR. BASED ON THESE FINDINGS, THE 12TH STREET PUMP STATION WILL BE ANALYZED USING THE SAME REINFORCEMENT AREAS, CONCRETE STRENGTH AND STEEL REINFORCEMENT STRENGTH. THE AVAILABLE 12TH STREET DRAWINGS DO INDICATE THE WETWELL WALL THICKNESS AND HEIGHT.

REFERENCES
1. Structural drawings not available.
2. DR&G Engineers drawings (pump modification), 1995.

Top of existing levee Elevation = 769.5
500 year +3 ft Levee Elevation = 774.4

Properties

NOTE: HGL Supplied assumes no relief wells.



$ELEV_1 := 760ft$

$HGL := 772ft - ELEV_1$

$HGL = 12\ ft$

$BLANKET := 760ft - 734ft$

$BLANKET = 26\ ft$

The 1995 drawings were generated as a result of pump replacement done by DR&G Engineers.
The structure undergone significant modifications during the 1995 pump replacement. The single pump was replaced with two pumps.

Refering to the 1995 drawings:

Wall 1 Parameters Wet well	Wall 2 Parameters Wet well
West wall	North wall
$L_1 := 13ft + 0in$	$L_2 := 7ft + 2in$
$Hwall_1 := 29.5ft$	$Hwall_2 := 29.5ft$
$D_1 := 10in$	$D_2 := 10in$
$C_{c1} := 2.5in$	$C_{c2} := 2.5in$

NOTE: The 12th Street pump station exterior walls other than the wet well walls were not analyzed. Structural drawings of these walls are not available. Structural drawings of the Mill Street pump station walls are available and assumed to be similar in design to the 12th St pump station walls. The Mill Street walls were analyzed and were reliable for 767.2 = HGL event when sand infill is not present. Both the 12th St and Mill St pump stations have sand infill on the interior side of the walls (earth load on the outside).
Based on this information, the 12th Street walls enclosing the sand infill material are considered reliable.

Hwall is measured from grade and may not be the actual wall height. The Hwall value is used to determine the wall loads.

BASE SLAB

The base slab reinforcement is not shown on the available drawings. Assume that the base slab has #6 at 12 inches.

Floor Thickness $D_f := 10in$ Clear Cover $C_{cf} := 3in$ $d_f := D_f - C_{cf} - \frac{0.75in}{2}$ $d_f = 6.625\ in$

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - C_{c1} - 0.375in$ $d_1 = 7.125\ in$

WALL 2

$d_2 := D_2 - C_{c2} - 0.25in$ $d_2 = 7.25\ in$

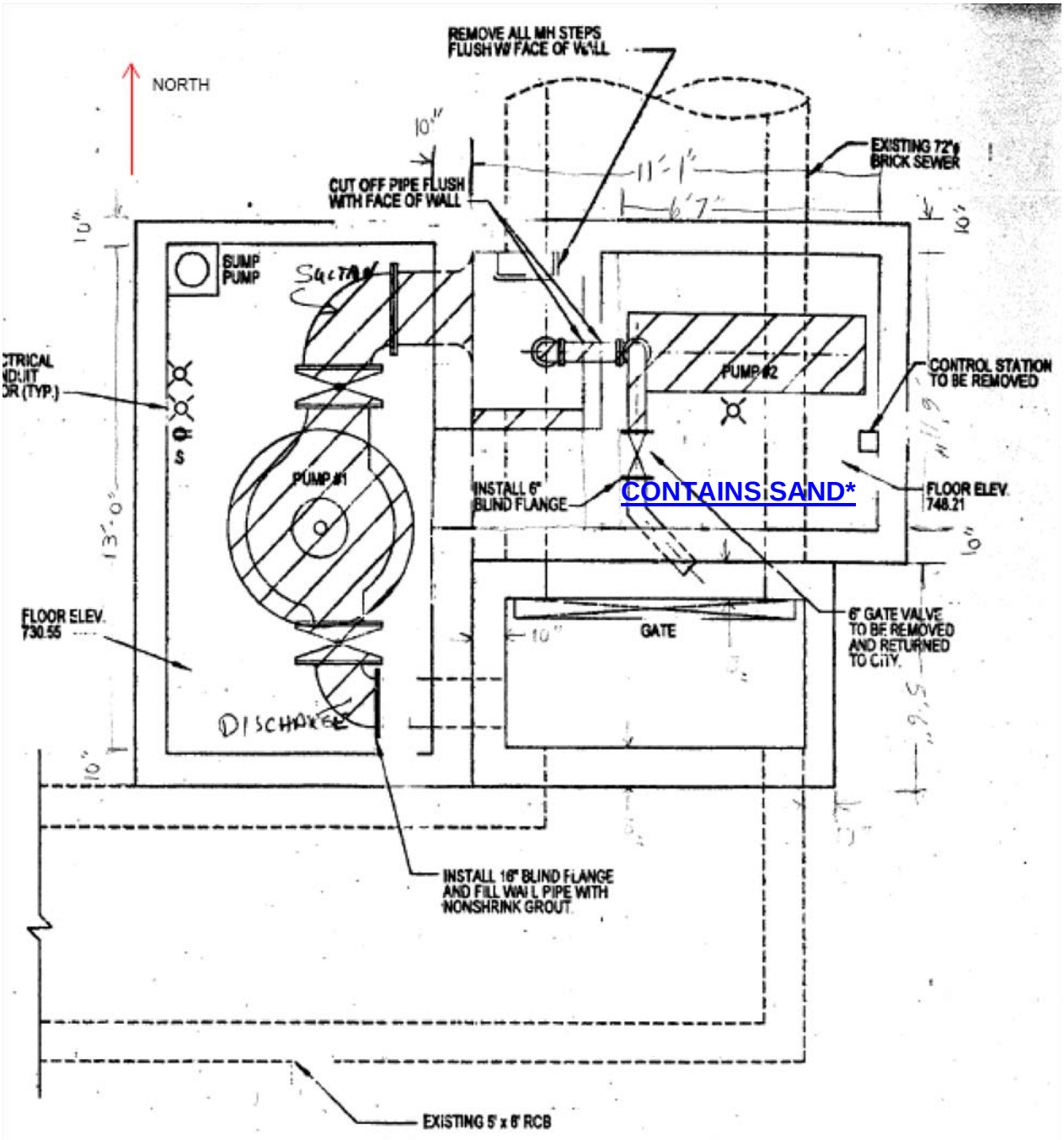


Figure 1. 12th Street Pump StationPlan View Drawing Sheet 2 of 15 DR&G Engineers, 1995

* Existing area below grade filled with sand as part of the 1995 pump replacement. This is shown on the DR&G Engineers drawings. As a result of the modifications, access to this area is no longer possible.

Assumptions and Criteria

- As-built drawings give fc=1350 and fs=20,000. These are working stress parameters. For this analysis, f'c=3000 psf and fy=40,000 psf.

Soil Properties	$\phi := 26deg$	$K_o := 1 - \sin(\phi)$	Steel Properties	$F_y := 40ksi$
	$\gamma := 120pcf$	$K_o = 0.562$	Concrete Properties	$f'_c := 3.00ksi$
Water Unit Weight	$\gamma_w := 62.4pcf$		Concrete Unit Weight	$\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Analysis

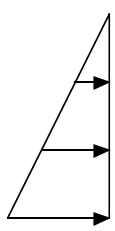
WALL 1, (West wall)

$K_o = 0.562$ $BLANKET = 26\ ft$ $HGL = 12\ ft$

$Hwall_1 = 29.5\ ft$

Wall Loading

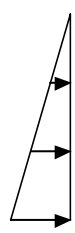
Soil or Soil & Water



$H_2 := Hwall_1$

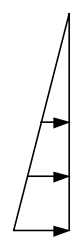
$H_2 = 29.5\ ft$

OR



$H_2 = 29.5\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_1 - HEAD$

$H''_3 := Hwall_1 + HGL - HEAD$

$H_3 := if(Hwall_1 > BLANKET, H''_3, H'_3)$

$H_3 = 41.5\ ft$

$H'_3 = 43.115\ ft$

$H''_3 = 41.5\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 1988\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3]$

$W_w = 3544\ \frac{lb}{ft^2}$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_1 := if(W_s > W_w, W_s, W_w)$

$W_1 = 3544\ \frac{lb}{ft^2}$

WALL 1 (West Wall) SHEAR

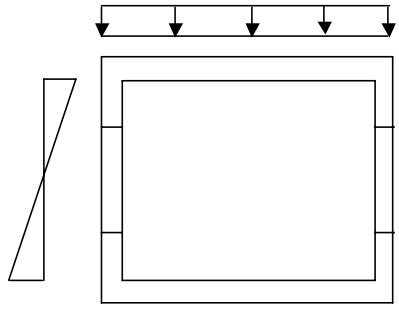
$L_2 = 7.167\ ft$

$THRUST_1 := \frac{W_1 \cdot L_2}{2}$ $\quad THRUST_1 = 12699\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_1 \cdot \frac{L_1}{2}$

$V'_{u1} = 23035\ \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$ $\quad V_{u1} = 19455\ \frac{lb}{ft}$

$D_2 = 0.833\ ft$

$d_1 = 7.125\ in$

$L_1 = 13\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\frac{L_1}{d_1} = 21.895$ EM 1110-2-2104 EQ (5-1) Does not Apply

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 120 \frac{in^2}{ft}$

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 9862\ \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD"\right)$

$Check1 = "NO GOOD"$

$\frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 0.053$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$

$FS_{S1} = 0.507$

ANALYZE WALL 1 (West wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_1 - \frac{(W_1 - 0psf)\cdot 3ft}{29.5ft} = 3183.521\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{3184psf\cdot L_1^2}{8}$ $M_{mid} = 67.262\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_1^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$

#6's @ 6 in. inside face (mid-plate moment) $A_{mph} := 0.884\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 1.156\,in$

$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 19.3\frac{kip\cdot ft}{ft}$

no bars outside face (edge-plate moment) $A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO\ GOOD")$ CheckMPH = "NO GOOD"

Factor of Safety $FS_{MPH1} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH1} = 0.287$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO\ GOOD")$ CheckEPH = "NO GOOD"

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3543.92\,psf$

$$W_1 - \frac{(W_1 - 0psf)\cdot 3ft}{29.5ft} = 3183.521\,psf$$

Thrust present 3 ft from bottom of wall.

$$THRUST := \frac{3184psf \cdot L_2}{2}$$

$$THRUST = 11409.333 \frac{lb}{ft}$$

$d_1 = 7.125\ in$ $A_g = 0.833\ ft$

$C_{c1} = 2.5\ in$

Information for CASTR: $THRUST = 11.41 \frac{kip}{ft}$ $M_{mid} = 67.262 \frac{kip \cdot ft}{ft}$

```
100 12thw1MPH767.2 M.Parks
110 wall 1 MPM HGL=772
200 1
210 3.00 40.0 0.375
300 12.0 X 12 with 1 bar inside face
400 12.0 12.0
500 1
510 1 0.884 7.125 6.0 6.0
600 11.4 67.3
```

Percent Design Strength Used in Tensile Zone

$Percent := 364.8\%$

$$FoS' := \frac{1}{Percent \cdot .9}$$

Note: Strength Reduction Factor (.9) not applied.

$FoS' = 0.30$

$M_{mid} = 67.262 \frac{kip \cdot ft}{ft}$

CASTR output: The section falls outside the admissible range and does not meet input data reinforcement limit requirement.

From interaction diagram:

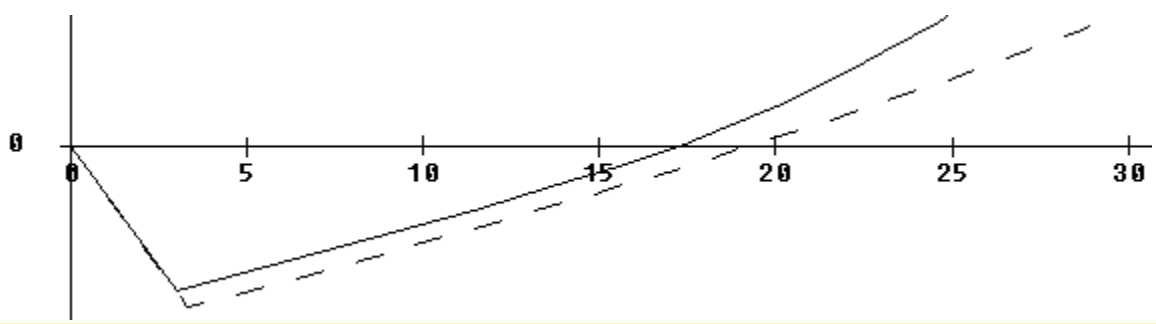
Nominal Strength

$$M_{mph_int} := 23 \frac{kip \cdot ft}{ft}$$

$$FS_{MPH1} := \frac{M_{mph_int}}{M_{mid}}$$

$$FS_{MPH1} = 0.342$$

Below 1.5. Perform reliability analysis



Reliability analysis (12th Street Wall 1 Com Bending and Thrust PoF.xmcd) indicates that wall 1 is unreliable, PF = 100%.

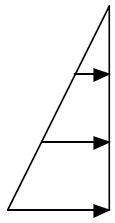
WALL 2, (North wall)

$K_o = 0.562$ $\quad$ $BLANKET = 26\ ft$ $\quad$ $HGL = 12\ ft$

$Hwall_2 = 29.5\ ft$ $\quad$ $\gamma_w = 62.4\ pcf$

Wall Loading

Soil or Soil & Water



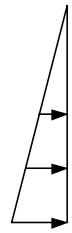
$H_2 := Hwall_2$

$H_2 = 29.5\ ft$

OR

$H_2 = 29.5\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_2 - HEAD$

$H''_3 := Hwall_2 + HGL - HEAD$

$H_3 := if\left(Hwall_2 > BLANKET, H''_3, H'_3\right)$

$H_3 = 41.5\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 1988\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_w = 3543.92\ psf$

$H'_3 = 43.115\ ft$

$H''_3 = 41.5\ ft$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_2 := if\left(W_s > W_w, W_s, W_w\right)$

$W_2 = 3544\ \frac{lb}{ft^2}$

WALL 2, (North Wall)

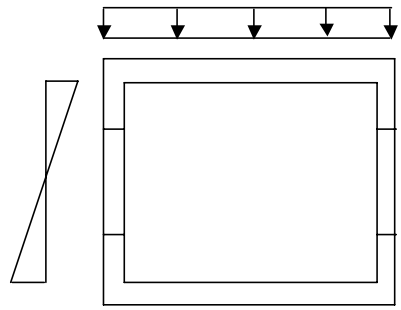
SHEAR

$$THRUST_2 := \frac{W_1 \cdot L_1}{2} \qquad THRUST_2 = 23035 \ \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis (if necessary or applicable)

$$V'_{u2} := W_2 \cdot \frac{L_2}{2}$$

$$V'_{u2} = 12699 \ \frac{lb}{ft}$$



$$V_{u2} := 2 \frac{V'_{u2}}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2 \right) \qquad V_{u2} = 9081 \ \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$D_2 = 10 \ in$$

$$A_g := D_2 \cdot 12 \frac{in}{ft} \qquad A_g = 120 \frac{in^2}{ft}$$

EM 1110-2-2104 EQ (5-1) Does not Apply

$$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_2 \right) \right]$$

ACI EQ (11-4)

$$d_2 = 7.25 \ in$$

$$\frac{THRUST_2 \cdot \frac{in^2}{lb}}{A_g} = 191.962$$

$$\phi V_{n2} = 10445 \ \frac{lb}{ft}$$

$$Check2 := if\left(\phi V_{n2} > 1.5 V_{u2}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

Check2 = "NO GOOD"

Factor of Safety

$$FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$$

$$FS_{S2} = 1.15$$

ANALYZE WALL 2 (North wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_2 - \frac{(W_2 - 0psf)\cdot 3ft}{29.5ft} = 3183.521\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{3184psf\cdot L_2^2}{8}$ $M_{mid} = 20.442\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_2^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$

#4's @ 6 in. inside face (mid-plate moment) $A_{mph} := 0.393\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 0.514\,in$

$$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right) \qquad \phi M_{mph} = 9.0\frac{kip\cdot ft}{ft}$$

no bars outside face (edge-plate moment) $A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right) \qquad \phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO GOOD")$ $CheckMPH = "NO GOOD"$

Factor of Safety $FS_{MPH2} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH2} = 0.44$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO GOOD")$ $CheckEPH = "NO GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3543.92\,psf$

$$W_2 - \frac{(W_2 - 0psf)\cdot 3ft}{29.5ft} = 3183.521\,psf$$

Thrust present 3 ft from bottom of wall.

$$THRUST := \frac{3184psf \cdot L_1}{2}$$ $$THRUST = 20696 \frac{lb}{ft}$$

$d_1 = 7.125\,in$ $A_g = 0.833\ ft$

$C_{c1} = 2.5\,in$

Loads from initial screening calculation: $M_u := 20.42 kip \cdot ft$ $F_y = 40\,ksi$ $f'_c = 3000\,psi$

$Thrust := 20.69 kip$

Compute Strength and Factor of Safety When Thrust is Included

NOTE: Reassign variables to allow use of the following routine

Steel Modulus of Elasticity: $E_s := 29000 ksi$

Section Info: Thickness of Section, $D := 10in$ $b := 1ft$ (1 foot strip)

$A_1 := 0.393 in^2$ $A_2 := 0 in^2$ ONLY 1 LAYER OF STEEL PRESENT

$d_1 := d_2$ $d_2 := d_2$ Distance from compression face to bar centerline

$d_1 = 7.25\,in$

$$\beta 1 := if\left[f'_c \le 4ksi, 0.85, if\left[\left(f'_c\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_c}{ksi} - 4\right)\right]\right]$$ $\beta 1 = 0.85$

Assume c=0.2*D $i := 1..2$

Given $c := .2 \cdot D$ $c = 2\,in$ $Thrust = 20.69\,kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_y \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_y \\ -F_y - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_y \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $c = 1.4\,in$ $a := if\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right)$ $a = 1.19\,in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s \cdot \varepsilon s_i > F_y, F_y, \text{if}(E_s \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'_c, 0ksi) \cdot A_i$ $\qquad$ $Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.013	-40	ksi	0.393	in^2	-15.72	kip	35.37	kip·in
0.604		-0.013	-40		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 35\ kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 160\ kip \cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 16.31\ kip \cdot ft$

$FS_{MPH2} := \frac{M_n}{M_u}$

$Check := \text{if}(FS_{MPH2} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\qquad$ $FS_{MPH2} = 0.799$ $\qquad$ Check = "NO GOOD"

Reliability analysis was performed (12th St Wall 2 Com Bending and Thrust_PoF.xmcd) The calculated reliability is Pf > 72.81%. Based on these results, Wall 2 is **NOT** considered reliable.

Uplift on Structure

Refer to Figure 2 for the identified areas concerning uplift calculations.

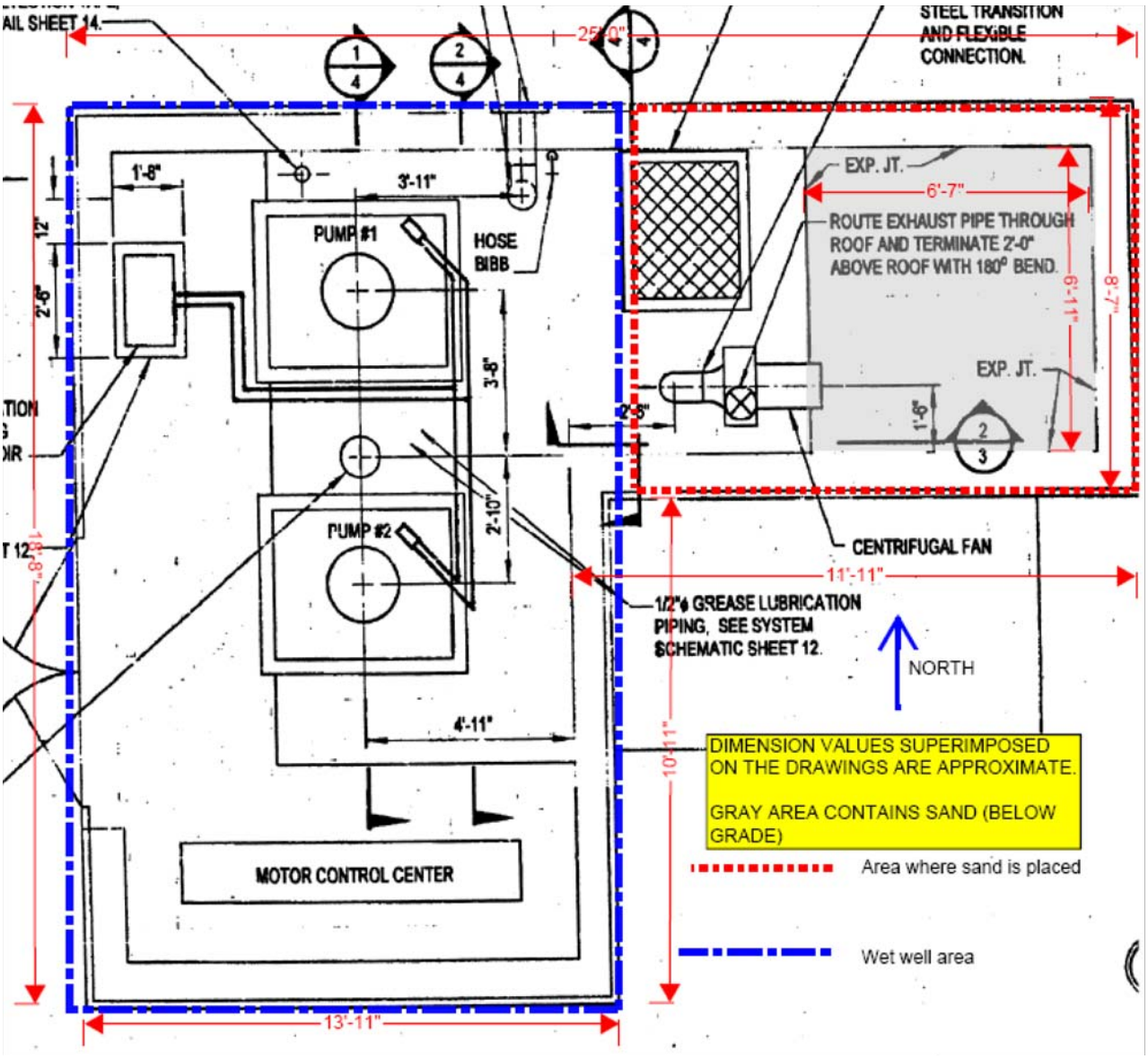


Figure 2. Ground Floor Elev. 762.17, 12th St Pump Station after 1995 modifications (DR&G Engineers) .

1. Wet well weight.

The wetwell weight includes all of the following within the outlined area in Figure 2:
1. Wetwell walls
2. Top slab within the identified wetwell area (Note that the top slab extends beyond the wetwell footprint).
3. Concrete wall above grade within the wetwell area.
4. Wetwell base slab
5. Estimated weight of steel used to rehabilitate the wetwell for strength deficiencies.

The above grade concrete wall size and top slab area is not shown on the DR&G Engineers drawings. The values used in calculations are conservative (low) estimate values when used to calculate uplift.

1. Wet well weight, con't.

Top Slab

$Top := 1.25ft \cdot (18ft + 8in) \cdot (13ft + 11in)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 48.708\,kip$

West Wall

$hwest := (762.17ft - 1.25ft) - (730.55ft - 10in)$ $\quad hwest = 31.203\ ft$

$West := hwest \cdot 13ft \cdot 10in$ $\quad Wwest := West \cdot 150pcf$ $\quad Wwest = 50.705\,kip$

East Wall

$heast := hwest$ $\quad heast = 31.203\ ft$

$East := heast \cdot 13ft \cdot 10in$ $\quad Weast := East \cdot 150pcf$ $\quad Weast = 50.705\,kip$

South Wall

$hsouth := hwest$ $\quad hsouth = 31.203\ ft$

$South := hsouth \cdot (7ft + 2in + 20in)\ 10in$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 34.454\,kip$

North Wall

$hnorth := hwest$ $\quad hnorth = 31.203\ ft$

$North := South$ $\quad Wnorth := Wsouth$ $\quad Wnorth = 34453.681\ lb$

Base Slab

$Wbase := 10in \cdot (7ft + 2in) \cdot 13ft \cdot 150pcf$ $\quad Wbase = 11.646\,kip$

North Wall above grade

$hnorth_{ag} := 5ft$ $\quad hnorth_{ag} = 5\ ft$

$North_{ag} := hnorth_{ag} \cdot 12in \cdot (13ft + 11in)$ $\quad Wnorth_{ag} := North_{ag} \cdot 150pcf$ $\quad Wnorth_{ag} = 10.437\,kip$

South Wall above Grade

$Wsouth_{ag} := Wnorth_{ag}$ $\quad Wsouth_{ag} = 10.437\,kip$

West Wall above Grade

$hwest_{ag} := 5ft$

$West_{ag} := hwest_{ag} \cdot (4ft + 6in) \cdot 12in$ $\quad Wwest_{ag} := West_{ag} \cdot 150pcf$ $\quad Wwest_{ag} = 3.375\,kip$

East Wall above Grade

$heast := 5ft$

$East_{ag} := (11ft + 9in) \cdot (12in) \cdot heast$ $\quad Weast_{ag} := East_{ag} \cdot 150pcf$ $\quad Weast_{ag} = 8.813\,kip$

1. Wet well weight, con't.

Wet Well Weight

$Wsteel := 12kip$ Estimated weight of steel placed in wetwell for strength rehabilitation

$Wwetwell := Wtop + Wwest + Weast + Wsouth + Wnorth + Wbase ... + Wnorth_{ag} + Wsouth_{ag} + Wwest_{ag} + Weast_{ag} + Wsteel$ $Wwetwell = 275.735\ kip$

2. Uplift On Wetwell Structure

$Awetwell := (13ft + 20in)\cdot(7ft + 2in + 20in)$ $Awetwell = 129.556\ ft^2$ Area that uplift is acting on

$H := ELEV_1 - (730.55ft - 10in)$ $H = 30.283\ ft$ $ELEV_1 = 760\ ft$

Bottom of blanket elev = 734

Bottom of wetwell elev = 729.72

Head, if structural foundation in blanket. $HGL = 12\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET}\cdot H\right)\cdot Awetwell\cdot\gamma_w \qquad U_1 = 357.812\ kips$$

$BLANKET = 26\ ft$
$HGL = 12\ ft$

$H = 30.283\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL)\cdot(Awetwell)\cdot\gamma_w\right] \qquad U_2 = 341.83\ kips$$

$$\left(\frac{BLANKET + HGL}{BLANKET}\cdot H\right) = 44.26\ ft$$

$$Uplift_{wet} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the wetwell structure $Uplift_{wet} = 341.83\ kips$

3. Wetwell Structure Factor of Safety for Flotation

Weight of water that is present during the event

$HEAD := 3ft$ $Wellarea := 13ft\cdot(7ft + 2in)$ $Wellarea = 93.167\ ft^2$

$Water := HEAD\cdot Wellarea\cdot 62.4pcf$ $Water = 17.441\ kip$

$heel := 0ft$ The wet well does not have heels

Volume of Soil above the heels

$volsoil := (1ft - 1ft)\cdot heel + (1ft - 1ft)\cdot heel$ $volsoil = 0\ ft^3$

$volsoil\cdot(\gamma - \gamma_w) = 0\ kip$ $volsoil\cdot\gamma_w = 0\ kip$

$Wwetwell + Water + volsoil\cdot(\gamma - \gamma_w) = 293.176\ kip$ $Uplift_{wet} = 341.83\ kip$

$$Stability_{wet} := \frac{Wwetwell + Water + volsoil\cdot(\gamma - \gamma_w)}{Uplift_{wet} - volsoil\cdot\gamma_w}$$

$Stability_{wet} = 0.858$ $(H + HGL) = 42.283\ ft$

$StabilityCheck := if(Stability_{wet} \geq 1.1, "Okay", "Not Okay")$ $StabilityCheck = "Not Okay"$

Side Friction As Aid To Wetwell Factor of Safety Against Uplift

SOIL

$\phi := 26deg$ INTERNAL FRICTION ANGLE

$\gamma := 120pcf$ SOIL UNIT WEIGHT

$\gamma_w := 62.4pcf$ WATER UNIT WEIGHT

$\gamma_{sub} := (\gamma - \gamma_w)$ SUBMERGED SOIL UNIT WEIGHT $\gamma_{sub} = 57.6\,pcf$

$K_o := 1 - \sin(\phi)$ AT-REST SOIL COEFFICIENT. ASSUME THAT THE WALLS DO NOT MOVE

$K_o = 0.562$

$\mu := 0.25$ FRICTION COEFICIENT BETWEEN STRUCTURE AND SOIL.

$U := 2638psf$ UPLIFT PRESSURE ACTING ON BASE OF STRUCTURE

$c := 5ft$ ASSUMED CRACK DEPTH

STRUCTURE

$H := 29.5ft$ DEPTH OF STRUCTURE BELOW GRADE

$per := 13ft \cdot 2 + 7.167ft \cdot 2$ STRUCTURE PERIMETER $per = 40.334\ ft$

$W_s := 275.74kip$ STRUCTURE DEAD WEIGHT

$W_{water} := 17.44kip$ WEIGHT OF WATER LOCATED IN STRUCTURE.

$Soil := 0kip$ WEIGHT OF SOIL LOCATED ON TOP OF THE BASE SLAB HEEL OR WALL STEP. EQUAL TO ZERO IF NO HEEL IS PRESENT.

UPLIFT PARAMETERS:

$U_{s_ec} := 341.83kip$ UPLIFT FORCE FOR THE EXISTING CONDITION

V. Calculation of Pv

1. Effective Lateral Load that Contributes to Side Friction

$\gamma = 120\,\text{pcf}$ $\quad c = 5\ \text{ft}$ $\quad H = 29.5\ \text{ft}$

$R_1 := (H - c)\cdot\left(\frac{c + H}{2}\right)\cdot\gamma$ $\quad R_1 = 1576.27\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

$R_2 := (H - c)\cdot U\cdot\left(\frac{c}{2\cdot H} + \frac{1}{2}\right)$ $\quad R_2 = 1174.633\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

$R_1 - R_2 = 401.637\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

2. Force Created By Side Friction, Pv

$K_o = 0.562$ $\quad \mu = 0.25$ $\quad per = 40.334\ \text{ft}$

$P_v := (R_1 - R_2)\cdot K_o\cdot\mu\cdot per$ $\quad P_v = 73.181\,\text{kip}$

<u>Stability Without the Aid of Side Friction, Existing Condition</u>

$Stability_{nofric_ec} := \frac{W_s + W_{water} + Soil}{U_{s_ec}}$ $\quad Stability_{nofric_ec} = 0.858$

<u>Stability with the Aid of Side Friction, Existing Conditon</u>

$Stability_{fric_ec} := \frac{W_s + W_{water} + Soil + P_v}{U_{s_ec}}$ $\quad Stability_{fric_ec} = 1.072$

The uplift factor of safety for various HGL elevations is shown below. The wetwell factor of safety for the n500+3 is 1.07 (borderline) when skin friction is considered.

Wetwell Uplift factors of safety versus HGL			
HGL	FS no skin friction	FS with skin friction	
768.2	0.942	1.24	Existing Condition
771	0.878	1.11	
772	0.858	1.07	n500+3 Event

Assuming that the skin friction CANNOT be taken into account perform the following analysis... .

Determine the amount of additional weight required to achieve FS = 1.1 of the wetwell structure

$Wreqdwetwell := Uplift_{wet} \cdot 1.1 - \left[Wwetwell + Water + volsoil \cdot \left(\gamma - \gamma_w\right)\right]$ $\quad Wreqdwetwell = 82.837\,kip$

$Water = 17.441\,kip$

4. Structure containing sand.

$Sa := (6ft + 7in) \cdot (6ft + 11in) \cdot (760.92ft - 748.71ft)$ $\quad Sa = 555.979\,ft^3$

$Wsa := 100pcf \cdot Sa$ $\quad Wsa = 55.598\,kip$

Top Slab

$Top := 1.25ft \cdot (11ft + 1in) \cdot (8ft + 7in)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 17.837\,kip$

East Wall

$hwest = 31.203\ ft$

$heast := (762.17ft - 1.25ft - 748.71ft)$ $\quad heast = 12.21\ ft$

$East := heast \cdot (6ft + 11in) \cdot 10in$ $\quad Weast := East \cdot 150pcf$ $\quad Weast = 10.557\,kip$

South Wall

$hsouth := heast$ $\quad hsouth = 12.21\ ft$

$South := hsouth \cdot (11ft + 1in) \cdot 10in$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 16.916\,kip$

North Wall

$hnorth := heast$ $\quad hnorth = 12.21\ ft$

$North := South$ $\quad Wnorth := Wsouth$ $\quad Wnorth = 16.916\,kip$

Base Slab

$Wbase := (748.71ft - 748.21ft) \cdot (11ft + 1in)(6ft + 11in + 20in) \cdot 150pcf$ $\quad Wbase = 7.135\,kip$

Interior Wall

$hint := heast$ $\quad hint = 12.21\ ft$

$Int := heast \cdot (6ft + 11in) \cdot (1.5ft)$ $\quad Wint := Int \cdot 150pcf$ $\quad Wint = 19.002\,kip$

Above grade concrete walls

North Wall above grade

$hnorth_{ag} := 5ft$ $\quad hnorth_{ag} = 5\ ft$

$North_{ag} := hnorth_{ag} \cdot 12in \cdot (11ft + 1in)$ $\quad Wnorth_{ag} := North_{ag} \cdot 150pcf$ $\quad Wnorth_{ag} = 8.312\,kip$

4. Structure containing sand, con't

South Wall above Grade

$Wsouth_{ag} := Wnorth_{ag}$ $\qquad Wsouth_{ag} = 8.312\ kip$

East Wall above Grade

$heast_{ag} := 5ft$

$East_{ag} := heast_{ag} \cdot (6ft + 11in) \cdot 12in$ $\qquad Weast_{ag} := East_{ag} \cdot 150pcf$ $\qquad Weast_{ag} = 5.187\ kip$

$Wbldg := 3kip$ Estimated weight of steel building enclosure, (conservative)

Structure Containing Sand Weight

$$Wsand := Wsa + Wtop + Weast + Wsouth + Wnorth + Wbase + Wint \ldots + Wnorth_{ag} + Wsouth_{ag} + Wwest_{ag} + Wbldg$$

$Wsand = 166.96\ kip$

5. Uplift On Sand Structure

$Asand := (11 \cdot ft + 11 \cdot in) \cdot (6 \cdot ft + 11 \cdot in + 20 \cdot in)$ $\qquad Asand = 102.285\ ft^2$ Area uplift acts on

$ELEVsand := 748.21ft$ Elevation at bottom of sand structure base slab

Bottom of blanket elev = 734

Bottom of wetwell elev = 729.72

$H := ELEV_1 - ELEVsand$ $\qquad H = 11.79\ ft$ $\qquad ELEV_1 = 760\ ft$

Head, if structural foundation in blanket. $\qquad HGL = 12\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot Asand \cdot \gamma_w \qquad U_1 = 109.981\ kips$$

$BLANKET = 26\ ft$
$HGL = 12\ ft$

Head, if structural foundation extends through blanket.

$H = 11.79\ ft$

$$U_2 := \left[(H + HGL) \cdot (Asand) \cdot \gamma_w\right] \qquad U_2 = 151.841\ kips$$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 17.232\ ft$$

$$Uplift_{sand} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the sand structure $\qquad Uplift_{sand} = 109.981\ kips$

6. Sand Structure Factor of Safety for Flotation

Weight of soil on the sand structure heels

$volsoil := (1ft - 1ft) \cdot heel + (1ft - 1ft) \cdot heel$ $\qquad volsoil = 0\ ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 0\ kip$ $\qquad volsoil \cdot \gamma_w = 0\ kip$

$$Stability_{sand} := \frac{Wsand + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{sand} - volsoil \cdot \gamma_w}$$

$Stability_{sand} = 1.518$ $\qquad (H + HGL) = 23.79\ ft$

$StabilityCheck := if\left(Stability_{sand} \geq 1.1, \text{"Okay"}, \text{"Not Okay"}\right)$ $\qquad StabilityCheck = \text{"Okay"}$

7. Amount of weight available to assist the wetwell flotation.

$$Wsandforwetwell := Wsand - \frac{1.10}{Stability_{sand}} \cdot \left[Wsand + volsoil \cdot \left(\gamma - \gamma_w\right)\right] \qquad Wsandforwetwell = 45.981\ kip$$

8. Determine if the pump station (without gatewell) meets the required factor of safety for uplift

To satisfy the overall factor of safety against flotation, the amount of weight available in the sand structure (Wsandforwetwell) must be greater than the amount of weight required such that the wetwell structure meets the required factor of safety (Wreqdwetwell).

$$PumpStationmeetsuplift := if(Wsandforwetwell \geq Wreqdwetwell, "okay", "NO\ GOOD")$$

$$PumpStationmeetsuplift = "NO\ GOOD"$$

This is assuming that the force can be transfered in shear at the wetwell and sand structure interface. Determine the length of wall required to transfer the shear force.

Wall Shear Strength

$Wsandforwetwell = 45.981\ kip$ — Amount of sand weight available to increase the wetwell FS for flotation

$Wreqdwetwell = 82.837\ kip$ — This is the amount of shear that has to be transferred in the walls

$t_{wall} := 10in$ — Wall thickness

$d_2 = 7.25\ in$ — Effective depth of the wall reinforcement

$f'_c = 3\ ksi$ — Concrete compressive strength

$$hreqd := \frac{Wreqdwetwell}{0.85 \cdot 2 \cdot \sqrt{f'_c} \cdot d_2 \cdot \frac{\sqrt{lb}}{in}} \qquad hreqd = 10.226\ ft$$

$$hnorth + hsouth = 24.42\ ft$$

$$Wallokforshear := if[hreqd \leq (hnorth + hsouth), "okay", "NO\ GOOD"] \qquad Wallokforshear = "okay"$$

9. Consider the gatewell attached to the station.

$Wgate := [[(5.5ft \cdot 10ft) - (3.833ft \cdot 8.33ft)] \cdot (764ft - 729.72ft) + 10in \cdot (3.833ft \cdot 8.33ft)] \cdot 150pcf - (5ft \cdot 6ft \cdot 10in) \cdot 150pcf$

$Wgate = 118.873\ kip$

$Agate := (5.5ft + 0in) \cdot (10ft + 0in)$ $\quad Agate = 55\ ft^2$ $\quad$ Area that uplift is acting on

$H := ELEV_1 - (729.72ft)$ $\quad H = 30.28\ ft$ $\quad ELEV_1 = 760\ ft$

Bottom of blanket elev = 734

Bottom of wetwell elev = 729.72

Head, if structural foundation in blanket. $\quad HGL = 12\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot Agate \cdot \gamma_w \qquad U_1 = 151.884\ kips$$

$BLANKET = 26\ ft$
$HGL = 12\ ft$

$H = 30.28\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (Agate) \cdot \gamma_w\right] \qquad U_2 = 145.105\ kips$$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 44.255\ ft$$

$$Uplift_{gate} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the wetwell structure $\quad Uplift_{gate} = 145.105\ kips$

$$Stability_{gate} := \frac{Wgate}{Uplift_{gate}} \qquad Stability_{gate} = 0.819$$

$(H + HGL) = 42.28\ ft$

ASSUMES THE GATEWELL HAS NO HEELS (CONSERVATIVE)

Amount of weight required to achieve FS=1.10

$Wreqdgate := \left(Uplift_{gate} \cdot 1.10\right) - Wgate$ $\quad Wreqdgate = 40.743\ kip$

Place sand in the gatewell as a fix....

$hsand := 25ft$

$Wgatesand := (3.833ft \cdot 8.33ft \cdot hsand) \cdot 100pcf$ $\quad Wgatesand = 79.822\ kip$

Amount of weight left over to aid pump station

$Wgateavail := Wgatesand - Wreqdgate$ $\quad Wgateavail = 39.08\ kip$

$Wsandforwetwell = 45.981\ kip$ $\quad Wreqdwetwell = 82.837\ kip$

Weight required such that the entire pump station (with gatewell) meets FS = 1.1 for uplift

$Wreqd := Wreqdwetwell - Wsandforwetwell$ $\quad Wreqd = 36.856\ kip$

The addditional weight placed in the gatewell (sand) must be equal to or greater than Wreqd

$PumpStationmeetsupliftwithgatesand := if(Wgateavail \geq Wreqd, "okay", "NO\ GOOD")$

$PumpStationmeetsupliftwithgatesand = "okay"$

IN SUMMARY, IF 25 FT OR MORE OF SAND IS PLACED IN THE GATEWELL, THE PUMP STATION (WITH GATEWELL) HAS A FS EQUAL TO OR GREATER THAN 1.1. NOTE THAT THE GATEWELL DIMENSIONS USED IN THE CALCULATIONS ARE APPROXIMATE. THE ACTUAL AMOUNT WILL REQUIRE VERIFICATION OF THE GATEWELL WALL THICKNESS AND SIZE.

Wet Well BASE SLAB ANALYSIS

$Water = 17.441\,kip$ WEIGHT OF WATER

$BaseSlab := Awetwell$ $BaseSlab = 129.556\ ft^2$ $Uplift_{wet} = 341.83\,kip$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$ $\frac{Wwetwell + Water}{BaseSlab} = 2262.934\,psf$

$W' := if\left(Stability_{wet} < 1.0, \frac{Uplift_{wet}}{BaseSlab}, \frac{Wwetwell}{BaseSlab}\right)$ $W' = 2638.48\,psf$ $\frac{Uplift_{wet}}{BaseSlab} = 2638.48\,psf$

$W' = 2638.48\,psf$

$W' := 2638psf$ $\frac{Wwetwell + volsoil\cdot(\gamma - \gamma_w)}{BaseSlab} = 2128.314\,psf$

$Slab_t := 10in$ $L_{strip} := (7ft + 2in + 20in)$ $Wall_t := D_1$ $Slab_d := d_f$ effective depth of slab

Analyze slab with one-way action

$L := 7ft + 2in$ $Slab_d = 6.625\,in$ $D_1 = 10\,in$

$V'_{ub} := \frac{W'\cdot L}{2}$ $V'_{ub} = 9452.833\ \frac{lb}{ft}$ $d_f = 6.625\,in$

$D_f = 0.833\ ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}}\cdot\left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 7379\ \frac{lb}{ft}$

$A_g := Slab_t\cdot 12\frac{in}{ft}$ $A_g = 120\ \frac{in^2}{ft}$ $Water = 17440.8\ lb$

BASE SLAB SHEAR CON'T

$$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f \right) \right]$$

ACI EQ (11-4)

$\phi V_{nb} = 8709 \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, \text{"OKAY"}, \text{"NO GOOD"}\right)$

CheckBase = "NO GOOD"

Factor of Safety

$$FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$$

$FS_{Sb} = 1.18$

Slab Bending

Assume simple span moment (conservative). One-way action, investigate the short span moment capacity.

$$M_b := \frac{L^2 \cdot W'}{8}$$

$M_b = 16.936 \frac{kip \cdot ft}{ft}$

#6's @ 12 in.

$A_{mpl} := 0.442 \frac{in^2}{ft}$ $b := 12 \frac{in}{ft}$ $a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mpl} = 0.578\ in$

$$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_f - \frac{a_{mpl}}{2} \right)$$

$\phi M_{mpl} = 9.3 \frac{kip \cdot ft}{ft}$

$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot M_b, \text{"OKAY"}, \text{"NO GOOD"}\right)$

CheckMP = "NO GOOD"

Factor of Safety

$$FS_{Mb} := \frac{\phi M_{mpl}}{M_b}$$

$FS_{Mb} = 0.551$

Again, it is unknown what reinforcement exists in the wetwell slab.

FACTORS OF SAFETY SUMMARY

Five walls and the base slab were evaluated for strength. A summary of the results are below. The reliability analysis calculations are in separate MathCad files.

Analysis indicates that the wetwell walls are unreliable. Based on the Wall 2 calculations, the East and South wet well walls are also considered unreliable. The East and South wet well walls have the same span length, thickness and reinforcement as the North wet well wall (Wall 2) and are exposed to lateral loading equal to the lateral loading present on Wall 2.

The available drawings do not indicate reinforcement present in the wet well base slab. Assuming that #6 @ 12" are present in the base slab, the base slab factor of safety <1.0. Based on these results, the base slab is also considered unreliable.

WALL 1 (WEST WETWELL WALL)

BAD

Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S1} = 0.507$

Moment

$FS_{MPH1} = 0.342$ Mid-plate moment

$FS_{EPH1} =$ ▮ Edge-of-plate moment (not applicable)

RELIABILITY ANALYSIS INDICATES THAT WALL 1 IS UNRELIABLE, Pf = 100%.

WALL 2 (NORTH WETWELL WALL)

BAD

Shear

$FS_{S2} = 1.15$ Reliability analysis indicates the wall being reliable for shear(Pf<0.0001)

Moment

$FS_{MPV2} =$ ▮ not applicable

$FS_{EPV2} =$ ▮ not applicable

$FS_{MPH2} = 0.799$ Reliability analysis indicates that the wall is uneliable for moment, Pf> 72.81%

$FS_{EPH2} =$ ▮ not applicable

OVERALL, RELIABILITY ANALYSIS INDICATES THAT WALL 2 IS UNRELIABLE.

BASE SLAB STATUS: SUSPECT. The base slab reinforcement is unknown.

Shear

$FS_{Sb} = 1.18$

Moment

$FS_{Mb} = 0.551$ Analyzed as a simple span, one-way action.

STABILITY MODIFICATION RECOMMENDED - SEE NOTE

NOTE: The wetwell stability is BORDERLINE (FS=1.07) IF skin friction is taken into account. Analyzed by itself, the gatewell factor of safety is 0.82 when no addtional weight (sand) is placed in the structure nor the benefit of skin friction considered. When skin friction is considered, the gatewell factor of safety becomes 0.94 (calculations not shown). Based on these analysis, the pump station gatewell should be modified by adding a slab and placing sand on the slab such that the required factor of safety against uplift of 1.10 is attained.

Wetwell Uplift factors of safety versus HGL

HGL	FS no skin friction	FS with skin friction	
768.2	0.942	1.24	Existing Condition
771	0.878	1.11	
772	0.858	1.07	n500+3 Event

RECOMMENDATIONS

The existing wetwell walls and base slab are inadequate for the **HGL=768.2** event loading. The existing structure is adequate for uplift stability for the **HGL=768.2** event loading when skin friction is taken into account. To ensure reliability of the structure during the event, the following actions are recommended:

1. Strengthen the wetwell walls and base slab using the design similar to the Strong Avenue pump station rehabilitation. This design is essentially a braced excavation design using uprights and horizontal beam bracing placed inside the wet well. The available drawings indicate that this method of rehabilitation is constructable at the 12th Street pump station. To facilitate installation of the steel, the above grade building structure, pumps and associated equipment will have to be removed temporarily. The existing at grade top slab will have to be removed (at least partially) in order to install the steel structural members. It is advised that temporary wetwell wall bracing be provided prior to removing the top slab. The Strong Ave fix is shown in "Strong Ave 273+41fix.mcd" file. After installling the steel structural members, replace the top slab, reinstall equipment and building enclosure structure.

2. Preliminary analysis indicates that the W-section uprights should not be spaced more than 5 ft on center such that the wet well walls are not overloaded for the **HGL=772** event.

3. The W-section uprights shall be supported at the top and bottom as a mimimum by horizontal beams or struts.

4. To reduce the required W-section size, intermediate struts are permissible.

5. The design shall effectively transfer the earth loading acting on a wetwell wall to a wall that is also earth loaded.

6. To address the uplift, remove the existing slide gate in the gatewell. <u>The slide gate is no longer used to maintain pump station operations</u>. Place a concrete floor above the conduit opening that will adequately support sand placed on it. To ensure adequate transfer of the uplift loads from the gatewell structure to the remaining pump station, anchor the gatewell walls to the pump station walls. A possible method of anchorage can be achieved by drilling holes and installing expansion anchors. Place sand on the installed concrete floor and cap with concrete. Preliminary analysis indicates that 25 feet of sand will be necessary in order achieve an overall FS against uplift of 1.10 for an **HGL=772** event (5 feet of sand for **HGL=768.2**) when the benefit of skin friction is NOT taken into account.

7. OBTAIN AND VERIFY ALL DIMENSIONS PRIOR TO CREATION OF THE FINAL DESIGN. OBTAIN CONCRETE SAMPLES OF THE WET WELL WALLS TO DETERMINE THE ACTUAL CONCRETE STRENGTH PRIOR TO FINAL DESIGN. THE Strong Ave 273+71fix.mcd file referenced earlief is attached (EXHIBIT 3).

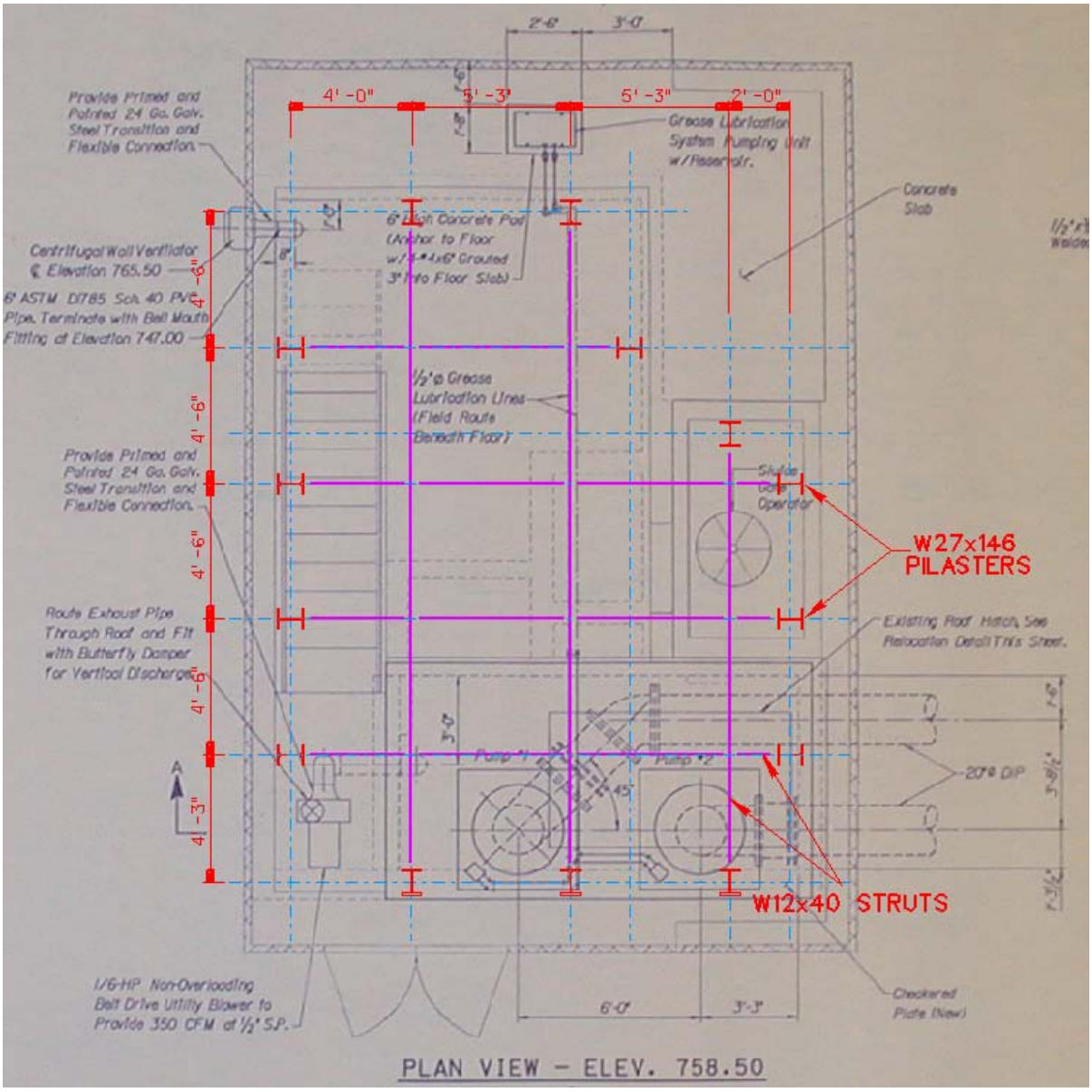
US Army Corps of Engineers

EXHIBIT 3
Argentine Flood Unit
Pump Station Analysis for Strong Ave. Sta. 273+41
Strong Ave. and Kaw River 4-14-1916 Sheet 1 of 2
Mechanical Plan and Sections, DR&G As Built Drawings, 1-17-94
Foundation Modification with Water at top of Protection (Existing Conditions)

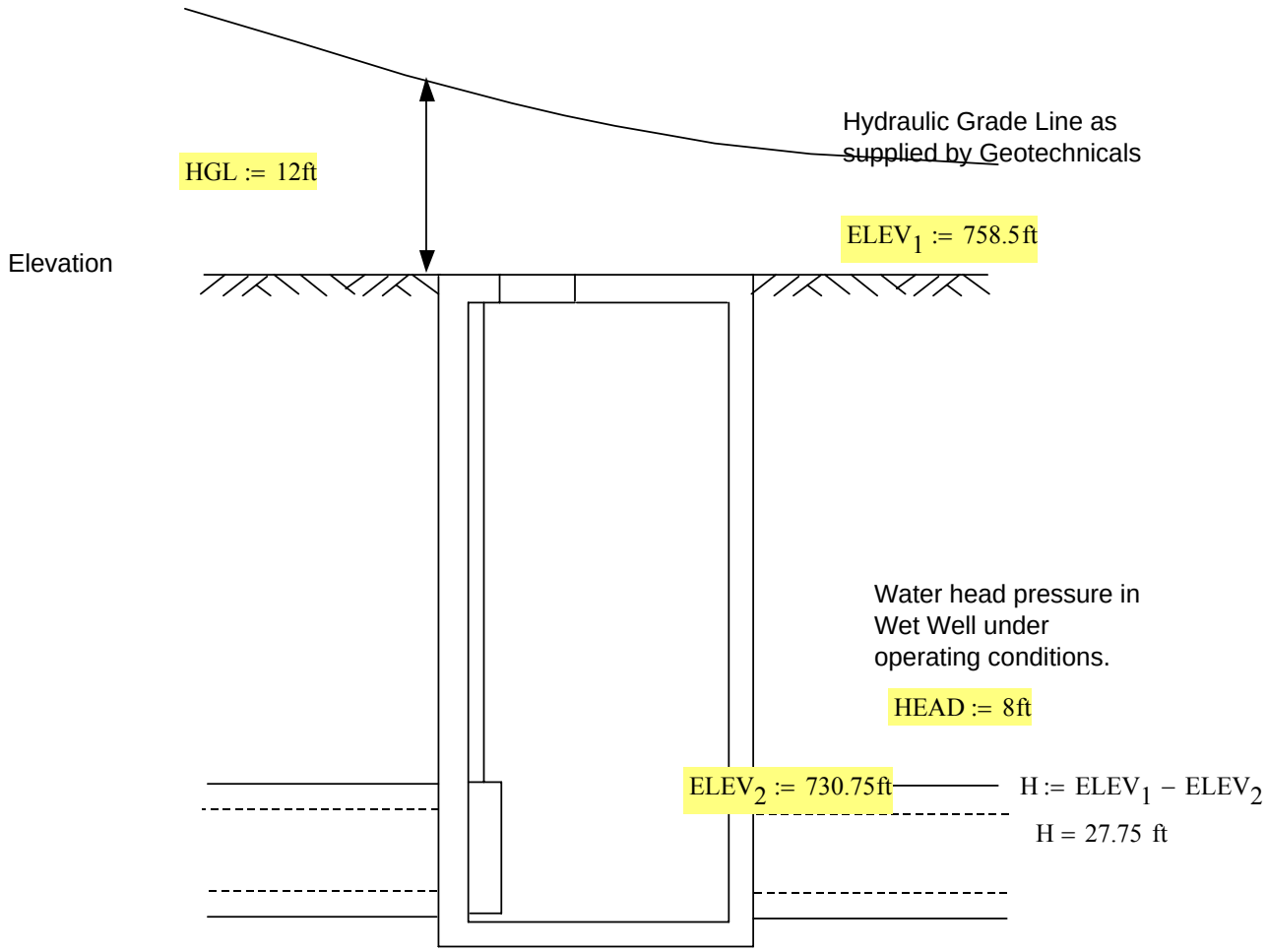
Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $pcf := \frac{lb}{ft^3}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $kips := 1000lb$ $klf := \frac{kips}{ft}$

A Series of Steel W-Section pilaters spaced approximately 5 ft on center with strut braces is purposed to address the foundation strength issues at the Strong Avenue pump station.



Properties



Assumptions

- Steel W-Section Pilasters will be embeded into the existing floor and an additional 10" of concrete added to the basement foundation slab for uplift and floor strength concerns.
- The steel pilasters will be braced just below the existing floor elevation with a steel struts.

Soil Properties $\phi := 28deg$

$\gamma := 120pcf$

$K_o := 1 - \sin(\phi)$

$K_o = 0.531$

Steel Properties $F_y := 50ksi$

Water Unit Weight $\gamma_w := 62.4pcf$

Load & Resistance Factor Design

Strength Reduction Factors

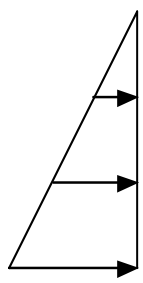
Shear Strength	$\phi_V := .85$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.
Flexural Strength	$\phi_B := 0.90$	
Load Factors		Load Multiplication Factor EM 1110-2-2104 (3-1)
Dead and Live Load Factor	$\gamma_L := 1.6$	Hydraulic Factor EM 1110-2-2104 (3-2)
Hydraulic Load Factor	$\gamma_H := 1.3$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)
Extreme Case Factor	$\gamma_X := .75$	

Analysis

WALLS

Wall Loading

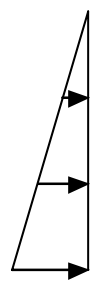
Soil or Soil & Water



$L_2 := H$

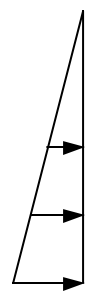
$L_2 = 27.75\ ft$

OR



$L_2 = 27.75\ ft$

&



$L_3 := H + HGL - HEAD$

$L_3 = 31.75\ ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot L_2$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma - \gamma_w) \cdot L_2 + \gamma_w \cdot L_3\right]$

$W_1 = 3675\ \frac{lb}{ft^2}$

$W_2 = 4414\ \frac{lb}{ft^2}$

Check to determine if water to top of gatewell with reduction factor, γ_X, or soil loading is worst case

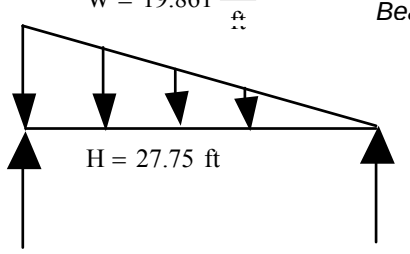
$W' := if(W_1 > W_2, W_1, W_2)$

$W' = 4414\ \frac{lb}{ft^2}$

Tributary Width $Width := 4.5ft$

Loading $W := W' \cdot Width$

$W = 19.861\ \frac{kip}{ft}$



Reaction and Moments from AISC Manual of Steel Construction

Beam Diagrams and Formulas

$$\phi M_{Max} := \frac{2\left(\frac{W \cdot H}{2}\right) \cdot H}{9 \cdot \sqrt{3}} \qquad \phi M_{Max} = 981.1\ ft \cdot kip$$

$$R_2 := \frac{\frac{W \cdot H}{2}}{3} \qquad R_2 = 91.857\ kips$$

PILASTERS

From AISC Manual of Steel Construction Beam Tables

Unbraced Length: $H = 27.75\ ft$

Design Moment: $\phi M_{Max} = 981.1\ ft \cdot kip$

Yield Strength: $F_y = 50\ ksi$

Use W 27x146

STRUTS

From AISC Manual of Steel Construction Beam Tables

Unbraced Length: $Width = 4.5\ ft$

Axial Load: $R_2 = 91.9\ kip$

Yield Strength: $F_y = 50\ ksi$

Use W 12x40

NOTE: Does not take into account floor load (combined axial and bending) but W 12x40 is good estimate for feasibility study. Additional Analysis to be performed at time of Plans and Specifications.

END OF EXHIBIT 3

CALCULATED BY Marvin Parks
PEER REVIEWED BY Paul Muller
April 10, 2007

Peer Reviewed by Paul Muller 4/09/07
No backcheck required as changes based on review notes will only improve results or are only suggestions for presentation but won't change results.

12920A.DAT

```
1000 4 12920A 12TH ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
1010   LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=730.0
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT
1040 I    S
1050  3000.0 40000.0   150.0  .3750  .0000
1060   1   6.00    .00  730.00   5.00
1070     3.25     2.88     2.88
1080    16.00    14.00    16.00
1090   1
1100   774.35   120.00   115.00
1110   3  62.40
1120     1.00      .40     .00   730.00
1130     1.00      .57     .00   730.00
1140     1.50      .50     .00   730.00
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1    .59    .20     .59    .20    .59    .20
1200  11    .20    .31     .20    .31    .20    .31
1210  12    .31    .20     .31    .20    .31    .20
1220  21    .20    .59     .20    .59    .20    .59
```

SUMMARY OF RCB ANALYSIS

LC	FILE	MEMBER	FS	PARAMETER	EVENT	STATUS
3	12920A	Bottom	2.13	Positive Moment	n500+3	Okay
2	12920B	Sidewall	3.14	Negative Moment	n500+3	Okay

The 12th Street outlet works RCB was evaluated for strength using CASE software CORTCUL. Based on the analysis, the 12th Street RCB structure is considered reliable for the existing condition and n500+3 event.

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME: 20:59: 0
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME: 20:59: 7

1. INPUT DATA

1.A.--HEADING

12920A 12TH ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=730.0
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  3000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 40000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
                (FT)      (IN)        (FT)       (FT)
       1         6.00       .00      730.00      5.00


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.25     ROOF SLAB      = 16.00
     INTERIOR ROOF/END WALLS = 2.88     EXTERIOR WALLS = 14.00
     INTERIOR BASE SLAB      = 2.88     BASE SLAB      = 16.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	774.35	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	730.00
2	1.00	.57	.00	730.00
3	1.50	.50	.00	730.00

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	.59	.20	.59	.20	.59	.20
11	.20	.31	.20	.31	.20	.31
12	.31	.20	.31	.20	.31	.20
21	.20	.59	.20	.59	.20	.59

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME:  20:59: 9

2.A.--HEADING

12920A 12TH ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=730.0
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       4.52        -10.31           4.52
     AXIAL FORCE           (KIPS)       8.71          8.71           8.71
     FLEXURE FACTOR OF SAFETY          12.22          3.63          12.22
     STRENGTH REDUCTION  (PHI)           .70           .79            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      -8.30                         8.30
     SHEAR FS AT 0.15LN (UI440)         7.77                         7.77
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 11                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      -7.51          1.27          -7.08
     AXIAL FORCE           (KIPS)      17.28         16.75          16.23
     FLEXURE FACTOR OF SAFETY           5.67         14.96           5.98
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)       4.10                        -3.89
     SHEAR FS AT 0.15LN (UI440)         9.70                        ------
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER
          ------ - SHEAR FS IS GREATER THAN TEN
```

```
MEMBER 12                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       7.51          -1.27           7.08
     AXIAL FORCE           (KIPS)      17.28          16.75          16.23
     FLEXURE FACTOR OF SAFETY           5.67          14.96           5.98
     STRENGTH REDUCTION  (PHI)           .70            .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                        NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      -4.10                          3.89
     SHEAR FS AT 0.15LN (UI440)         9.70                        ------
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER
          ------ - SHEAR FS IS GREATER THAN TEN

MEMBER 21                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      -4.25           9.68          -4.25
     AXIAL FORCE           (KIPS)       7.81           7.81           7.81
     FLEXURE FACTOR OF SAFETY          11.93           3.79          11.93
     STRENGTH REDUCTION  (PHI)           .70            .80            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                        NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)       7.80                         -7.80
     SHEAR FS AT 0.15LN (UI440)         8.21                          8.21
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER
```

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

```
MEMBER  1                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       6.94          -7.88           6.94
     AXIAL FORCE           (KIPS)      12.36          12.36          12.36
     FLEXURE FACTOR OF SAFETY           6.88           7.13           6.88
     STRENGTH REDUCTION  (PHI)           .70            .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                        NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      -8.30                          8.30
     SHEAR FS AT 0.15LN (UI440)         8.00                          8.00
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER

MEMBER 11                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      -8.28           4.12          -7.88
     AXIAL FORCE           (KIPS)      17.28          16.75          16.23
     FLEXURE FACTOR OF SAFETY           4.53          10.87           4.64
     STRENGTH REDUCTION  (PHI)           .70            .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                        NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)       5.80                         -5.58
     SHEAR FS AT 0.15LN (UI440)         6.85                          7.08
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER

MEMBER 12                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       8.28          -4.12           7.88
     AXIAL FORCE           (KIPS)      17.28          16.75          16.23
     FLEXURE FACTOR OF SAFETY           4.53          10.87           4.64
     STRENGTH REDUCTION  (PHI)           .70            .70            .70
```

```
     SHEAR FORCE AT D      (KIPS)     NNNNNN                      NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                      NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      -5.80                        5.58
     SHEAR FS AT 0.15LN (UI440)         6.85                        7.08
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 21                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      -6.64          7.29          -6.64
     AXIAL FORCE           (KIPS)      11.16         11.16          11.16
     FLEXURE FACTOR OF SAFETY           6.36          7.55           6.36
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                      NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                      NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)       7.80                       -7.80
     SHEAR FS AT 0.15LN (UI440)         8.43                        8.43
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       5.35        -16.12           5.35
     AXIAL FORCE           (KIPS)      10.86         10.86          10.86
     FLEXURE FACTOR OF SAFETY          10.95          2.13          10.95
     STRENGTH REDUCTION  (PHI)           .70           .82            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                      NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                      NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)     -12.03                        12.03
     SHEAR FS AT 0.15LN (UI440)         5.46                        5.46
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 11                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     -10.47           .43         -10.06
     AXIAL FORCE           (KIPS)      25.08         24.55          24.03
     FLEXURE FACTOR OF SAFETY           4.21         10.21           4.38
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                      NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                      NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)       5.10                       -4.88
     SHEAR FS AT 0.15LN (UI440)         8.26                        8.58
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 12                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      10.47          -.43          10.06
     AXIAL FORCE           (KIPS)      25.08         24.55          24.03
     FLEXURE FACTOR OF SAFETY           4.21         10.21           4.38
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                      NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                      NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      -5.10                        4.88
     SHEAR FS AT 0.15LN (UI440)         8.26                        8.58
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER
```

```
MEMBER 21                            LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      -5.07         15.51          -5.07
     AXIAL FORCE           (KIPS)       9.78          9.78           9.78
     FLEXURE FACTOR OF SAFETY          10.91          2.16          10.91
     STRENGTH REDUCTION  (PHI)           .70           .83            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      11.52                       -11.52
     SHEAR FS AT 0.15LN (UI440)         5.64                         5.64
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER
```

```
12920A 12TH ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=730.0
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST 3=EMBANKMENT

3.B.--RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
```

OUTPUT COMPLETE.

```
1000 4 12920B 12TH ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
1010   LOC: LEVEE CREST  EVENT:N500+3 NO IWAT=0 GWATEL=774.35
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST | 3=EMBANKMENT

1040 I    S
1050  3000.0 40000.0   150.0  .3750  .0000
1060   1   6.00    .00  730.00   5.00
1070     3.25     2.88     2.88
1080    16.00    14.00    16.00
1090   1
1100   774.35   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   774.35
1130     1.00      .57      .00   774.35
1140     1.50      .50      .00   774.35
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1    .59    .20     .59    .20    .59    .20
1200  11    .20    .31     .20    .31    .20    .31
1210  12    .31    .20     .31    .20    .31    .20
1220  21    .20    .59     .20    .59    .20    .59
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME: 21: 2:35


PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME: 21: 2:40

1. INPUT DATA

1.A.--HEADING

12920B 12TH ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 NO IWAT=0 GWATEL=774.35
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST | 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  3000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 40000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
                (FT)      (IN)        (FT)       (FT)
       1         6.00       .00      730.00      5.00


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.25     ROOF SLAB      = 16.00
     INTERIOR ROOF/END WALLS = 2.88     EXTERIOR WALLS = 14.00
     INTERIOR BASE SLAB      = 2.88     BASE SLAB      = 16.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	774.35	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	774.35
2	1.00	.57	.00	774.35
3	1.50	.50	.00	774.35

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	.59	.20	.59	.20	.59	.20
11	.20	.31	.20	.31	.20	.31
12	.31	.20	.31	.20	.31	.20
21	.20	.59	.20	.59	.20	.59

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME:  21: 2:43

2.A.--HEADING

12920B 12TH ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 NO IWAT=0 GWATEL=774.35
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST | 3=EMBANKMENT


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                          LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       9.36         -6.04           9.36
     AXIAL FORCE           (KIPS)      16.04         16.04          16.04
     FLEXURE FACTOR OF SAFETY           4.71         10.69           4.71
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      -8.62                         8.62
     SHEAR FS AT 0.15LN (UI440)         7.54                         7.54
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 11                          LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      -9.28          6.80          -8.92
     AXIAL FORCE           (KIPS)      17.95         17.43          16.90
     FLEXURE FACTOR OF SAFETY           3.54          6.94           3.56
     STRENGTH REDUCTION  (PHI)           .70           .70            .70
     SHEAR FORCE AT D      (KIPS)     NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)       7.54                        -7.30
     SHEAR FS AT 0.15LN (UI440)         5.30                         5.44
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER
```

```
MEMBER 12                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)        9.28          -6.80           8.92
     AXIAL FORCE           (KIPS)       17.95          17.43          16.90
     FLEXURE FACTOR OF SAFETY            3.54           6.94           3.56
     STRENGTH REDUCTION  (PHI)            .70            .70            .70
     SHEAR FORCE AT D      (KIPS)      NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)             NNNNNN                        NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)       -7.54                          7.30
     SHEAR FS AT 0.15LN (UI440)          5.30                          5.44
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 21                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       -9.02           5.48          -9.02
     AXIAL FORCE           (KIPS)       14.57          14.57          14.57
     FLEXURE FACTOR OF SAFETY            4.30          11.77           4.30
     STRENGTH REDUCTION  (PHI)            .70            .70            .70
     SHEAR FORCE AT D      (KIPS)      NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)             NNNNNN                        NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)        8.12                         -8.12
     SHEAR FS AT 0.15LN (UI440)          7.92                          7.92
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       10.57          -4.83          10.57
     AXIAL FORCE           (KIPS)       17.87          17.87          17.87
     FLEXURE FACTOR OF SAFETY            4.05          11.97           4.05
     STRENGTH REDUCTION  (PHI)            .70            .70            .70
     SHEAR FORCE AT D      (KIPS)      NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)             NNNNNN                        NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)       -8.62                          8.62
     SHEAR FS AT 0.15LN (UI440)          7.64                          7.64
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 11                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)       -9.67           8.22          -9.32
     AXIAL FORCE           (KIPS)       17.95          17.43          16.90
     FLEXURE FACTOR OF SAFETY            3.16           5.09           3.14
     STRENGTH REDUCTION  (PHI)            .70            .70            .70
     SHEAR FORCE AT D      (KIPS)      NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)             NNNNNN                        NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)        8.39                         -8.14
     SHEAR FS AT 0.15LN (UI440)          4.76                          4.88
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 12                              LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)        9.67          -8.22           9.32
     AXIAL FORCE           (KIPS)       17.95          17.43          16.90
     FLEXURE FACTOR OF SAFETY            3.16           5.09           3.14
     STRENGTH REDUCTION  (PHI)            .70            .70            .70
     SHEAR FORCE AT D      (KIPS)      NNNNNN                        NNNNNN
     SHEAR FS AT D (ACI63)             NNNNNN                        NNNNNN
```

```
     SHEAR FORCE AT 0.15LN (KIPS)      -8.39                          8.14
     SHEAR FS AT 0.15LN (UI440)         4.76                          4.88
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 21                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)    -10.22          4.29         -10.22
     AXIAL FORCE           (KIPS)     16.25         16.25          16.25
     FLEXURE FACTOR OF SAFETY          3.67         13.36           3.67
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)    NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)           NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      8.12                         -8.12
     SHEAR FS AT 0.15LN (UI440)        8.02                          8.02
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     9.78         -8.96           9.78
     AXIAL FORCE           (KIPS)    17.12         17.12          17.12
     FLEXURE FACTOR OF SAFETY          4.72          7.08           4.72
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)    NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)           NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)    -10.49                         10.49
     SHEAR FS AT 0.15LN (UI440)        6.25                          6.25
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 11                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)    -10.77          6.38         -10.42
     AXIAL FORCE           (KIPS)     21.86         21.34          20.81
     FLEXURE FACTOR OF SAFETY          3.32          7.41           3.34
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)    NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)           NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      8.04                         -7.80
     SHEAR FS AT 0.15LN (UI440)        5.12                          5.25
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 12                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     10.77         -6.38          10.42
     AXIAL FORCE           (KIPS)     21.86         21.34          20.81
     FLEXURE FACTOR OF SAFETY          3.32          7.41           3.34
     STRENGTH REDUCTION  (PHI)          .70           .70            .70
     SHEAR FORCE AT D      (KIPS)    NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)           NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)      -8.04                          7.80
     SHEAR FS AT 0.15LN (UI440)        5.12                          5.25
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 21                           LEFT END      CENTERLINE     RIGHT END
```

```
BENDING MOMENT        (K-FT)      -9.43          8.41          -9.43
AXIAL FORCE           (KIPS)      15.56         15.56          15.56
FLEXURE FACTOR OF SAFETY           4.30          7.44           4.30
STRENGTH REDUCTION  (PHI)           .70           .70            .70
SHEAR FORCE AT D      (KIPS)     NNNNNN                       NNNNNN
SHEAR FS AT D (ACI63)            NNNNNN                       NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)       9.99                        -9.99
SHEAR FS AT 0.15LN (UI440)         6.49                         6.49
     NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
          APPLY FOR THIS MEMBER
```

```
12920B 12TH ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 NO IWAT=0 GWATEL=774.35
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2 AT-REST | 3=EMBANKMENT

3.B.--RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE



OUTPUT COMPLETE.
```

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $pcf := \frac{lb}{ft^3}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $kips := 1000lb$ $klf := \frac{kips}{ft}$

This analysis is for the Existing Condition HGL elevation 767.2

Top of existing levee El = 768.7
Bottom of blanket El = 725.0

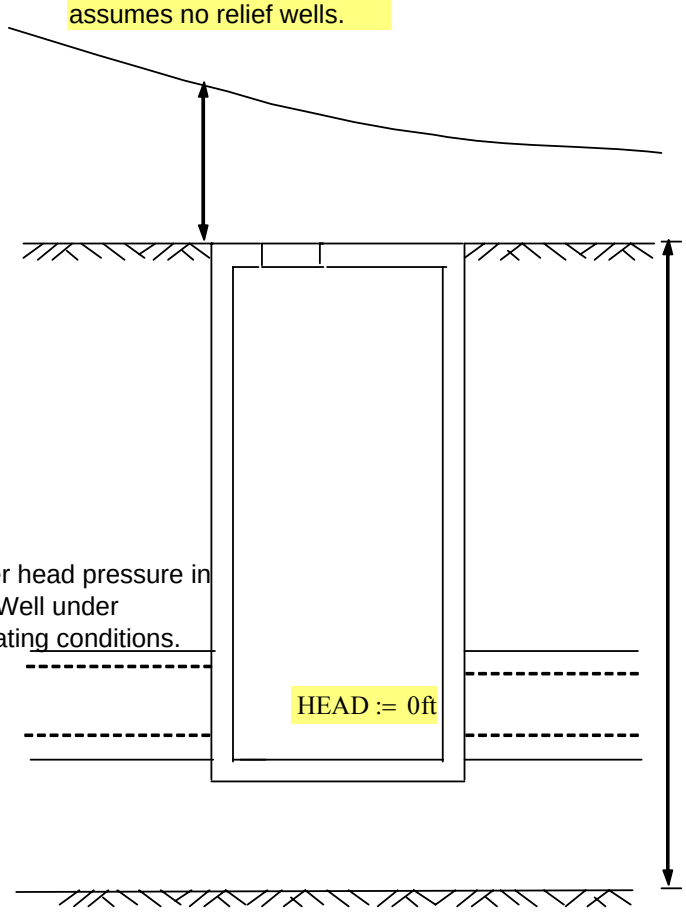
REFERENCES
1. Drawings, dated 1914 and 1916.
2. DR&G Engineers drawings (pump modification), 1995.

Properties

NOTE: HGL Supplied assumes no relief wells.

Original Hydraulic Grade Line as supplied by Geotechnicals (Affects Uplift on Base Slab if in the landside of levee)

Top of levee elev = 768.3 ft



Elevation

$ELEV_1 := 761ft$

$HGL := 767.2ft - ELEV_1$

$HGL = 6.2\ ft$

Water head pressure in Wet Well under operating conditions.

$BLANKET := 761ft - 725ft$

$BLANKET = 36\ ft$

$HEAD := 0ft$

Structural drawings dated 1914, 1916 and 1995 were obtained. The 1995 drawings were generated as a result of pump replacement done by DR&G Engineers.
The structure has undergone significant modifications. Records indicate that in the 1910s, the wetwell area was enlarged. The enlargement was probably necessitated by the size of the pump. An interior wall was also added as part of the modification. In the 1990s, the single pump was replaced with two pump. According to the drawing information found, the exterior wall modifications was limited to the 1910s modification.

Refering to the 1916 drawings, modifications to the "wall in place" was accomplished by removing an existing south wall and extending the west wall, thus increasing the wet well area.

Wall 1 Parameters Wet well	Wall 2 Parameters Wet well	Wall 3 Parameters In place wall	Wall 4 Parameters In place wall	Wall 5 Parameters In place wall
West wall	North wall	North wall	East wall	South wall
$L_1 := 13ft + 4in$	$L_2 := 7ft$	$L_3 := 22ft$	$L_4 := 13ft$	$L_5 := 6ft + 7in$
$Hwall_1 := 30.25ft$	$Hwall_2 := 30.25ft$	$Hwall_3 := 13.5ft$	$Hwall_4 := 13.5ft$	$Hwall_5 := 13.5ft$
$D_1 := 1ft$	$D_2 := 10in$	$D_3 := 13in$	$D_4 := 12in$	$D_5 := 10in$
$C_{c1} := 2.5in$	$C_{c2} := 2.5in$	$C_{c3} := 2.5in$	$C_{c4} := 2.5in$	$C_{c5} := 2.5in$
			Square Bars	Square Bars

Hwall is measured from grade and may not be the actual wall height. The Hwall value is used to determine the wall loads.
Sand was placed in the interior area created by the "In place" walls as part of the 1995 modification.

BASE SLAB

The base slab reinforcement is not shown on the available drawings. Assume that the base slab has #6 at 12 inches.

Floor Thickness $D_f := 10in$ Clear Cover $C_{cf} := 3in$ $d_f := D_f - C_{cf} - \frac{0.75in}{2}$ $d_f = 6.625\ in$

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - C_{c1} - 0.375in$ $d_1 = 9.125\ in$

WALL 3

$d_3 := D_3 - C_{c3} - 0.25in$ $d_3 = 10.25\ in$

WALL 2

$d_2 := D_2 - C_{c2} - 0.25in$ $d_2 = 7.25\ in$

WALL 4

$d_4 := D_4 - C_{c4} - 0.375in$ $d_4 = 9.125\ in$

WALL 5

$d_5 := D_5 - C_{c5} - \frac{5}{16}in$ $d_5 = 7.188\ in$

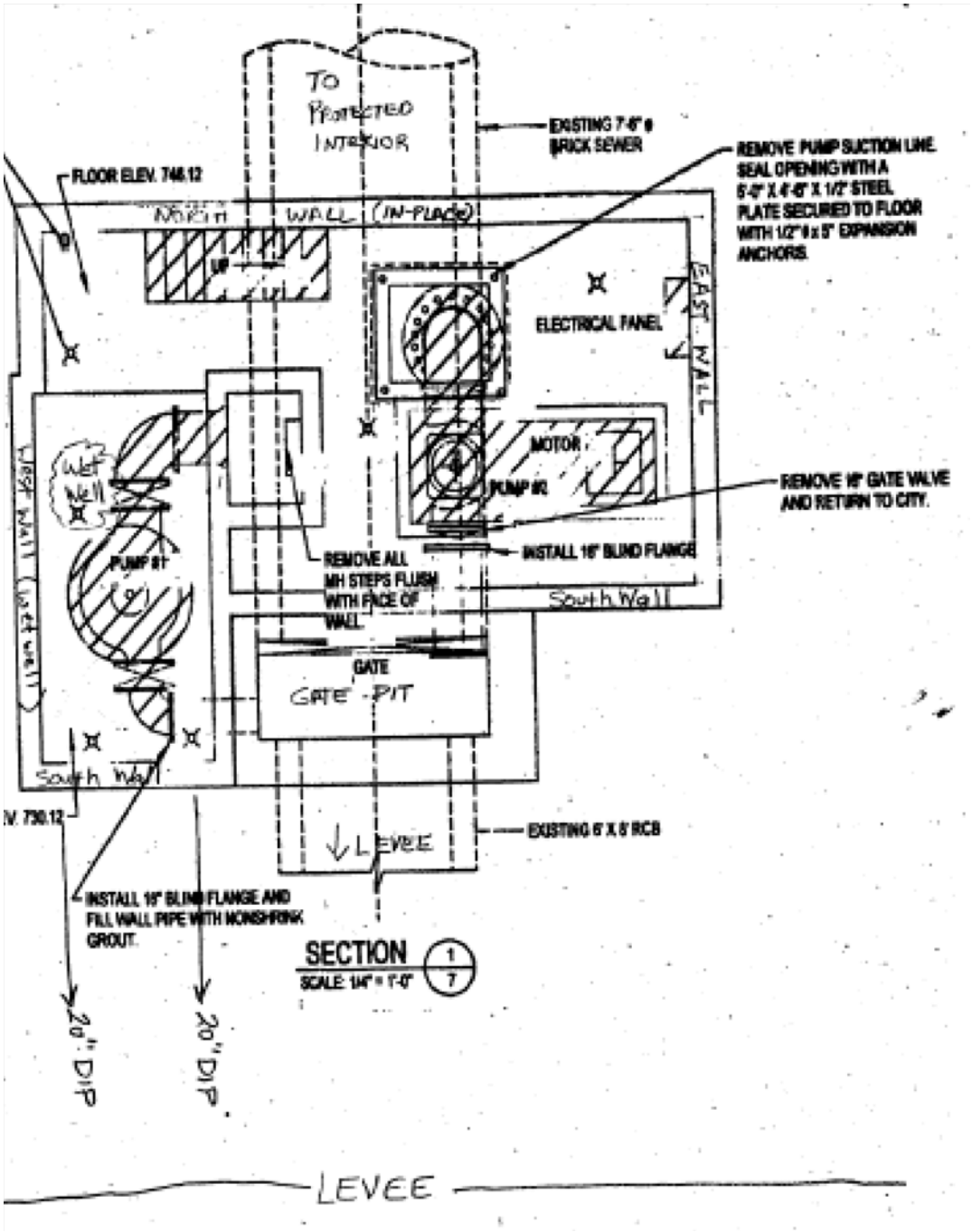


Figure 1. Mill Street Pump StationPlan View Drawing Sheet 7 of 17 DR&G Engineers, 1995

Assumptions and Criteria

- As-built drawings give fc=1350 and fs=20,000. These are working stress parameters. For this analysis, f'c=3000 psf and fy=40,000 psf.

Soil Properties	$\phi := 26deg$	$K_o := 1 - \sin(\phi)$	Steel Properties	$F_y := 40ksi$
	$\gamma := 120pcf$	$K_o = 0.562$	Concrete Properties	$f'_c := 3.00ksi$
Water Unit Weight	$\gamma_w := 62.4pcf$		Concrete Unit Weight	$\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

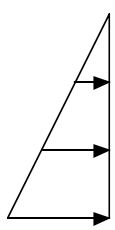
WALL 1, (West wall)

$K_o = 0.562$ $BLANKET = 36\ ft$ $HGL = 6.2\ ft$

$Hwall_1 = 30.25\ ft$

Wall Loading

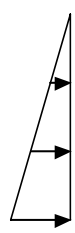
Soil or Soil & Water



$H_2 := Hwall_1$

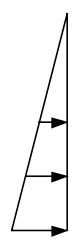
$H_2 = 30.25\ ft$

OR



$H_2 = 30.25\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_1 - HEAD$

$H''_3 := Hwall_1 + HGL - HEAD$

$H_3 := if(Hwall_1 > BLANKET, H''_3, H'_3)$

$H_3 = 35.46\ ft$

$H'_3 = 35.46\ ft$

$H''_3 = 36.45\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 2039\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3]$

$W_w = 3191\ \frac{lb}{ft^2}$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_1 := if(W_s > W_w, W_s, W_w)$

$W_1 = 3191\ \frac{lb}{ft^2}$

WALL 1 (West Wall) SHEAR

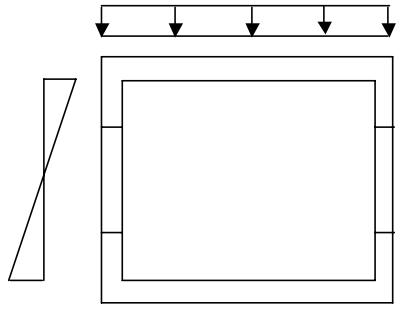
$L_2 = 7\ ft$

$THRUST_1 := \frac{W_1 \cdot L_2}{2}$ $\quad THRUST_1 = 11169\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_1 \cdot \frac{L_1}{2}$

$V'_{u1} = 21275\ \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$ $\quad V_{u1} = 17519\ \frac{lb}{ft}$

$D_2 = 0.833\ ft$

$d_1 = 9.125\ in$

$L_1 = 13.333\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\frac{L_1}{d_1} = 17.534$ EM 1110-2-2104 EQ (5-1) Does not Apply

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 144 \frac{in^2}{ft}$

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 12460\ \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD"\right)$

$Check1 = "NO GOOD"$

$\frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 0.039$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$

$FS_{S1} = 0.711$

ANALYZE WALL 1 (West wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_1 - \frac{(W_1 - 0psf)\cdot 3ft}{30.25ft} = 2874.779\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{2875psf\cdot L_1^2}{8}$ $M_{mid} = 63.889\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_1^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$

#6's @ 6 in. inside face (mid-plate moment) $A_{mph} := 0.884\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 1.156\,in$

$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 25.2\frac{kip\cdot ft}{ft}$

no bars outside face (edge-plate moment) $A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO\ GOOD")$ CheckMPH = "NO GOOD"

Factor of Safety $FS_{MPH1} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH1} = 0.394$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO\ GOOD")$ CheckEPH = "NO GOOD"

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3191.269\,psf$

$$W_1 - \frac{(W_1 - 0psf)\cdot 3ft}{30.25ft} = 2874.779\,psf$$

Thrust present 3 ft from bottom of wall.

$$THRUST := \frac{2875psf \cdot L_2}{2}$$

$$THRUST = 10062.5 \frac{lb}{ft}$$

$d_1 = 9.125\ in$ $A_g = 1\ ft$

$C_{c1} = 2.5\ in$

Information for CASTR:

$$THRUST = 10.06 \frac{kip}{ft}$$

$$M_{mid} = 63.889 \frac{kip \cdot ft}{ft}$$

```
100 Millw1MPH767.2 M.Parks
110 Wall 1 MPM HGL=767.2
200 1
210 3.00 40.0 0.375
300 12.0 X 12 with 1 bar inside face
400 12.0 12.0
500 1
510 1 0.884 9.125 6.0 6.0
600 10.06 63.9
```

Percent Design Strength Used in Tensile Zone

$Percent := 269\%$

$$FoS' := \frac{1}{Percent \cdot .9}$$

Note: Strength Reduction Factor (.9) not applied.

$FoS' = 0.41$

$$M_{mid} = 63.889 \frac{kip \cdot ft}{ft}$$

CASTR output: The section falls outside the admissible range and does not meet input data reinforcement limit requirement.

From interaction diagram:

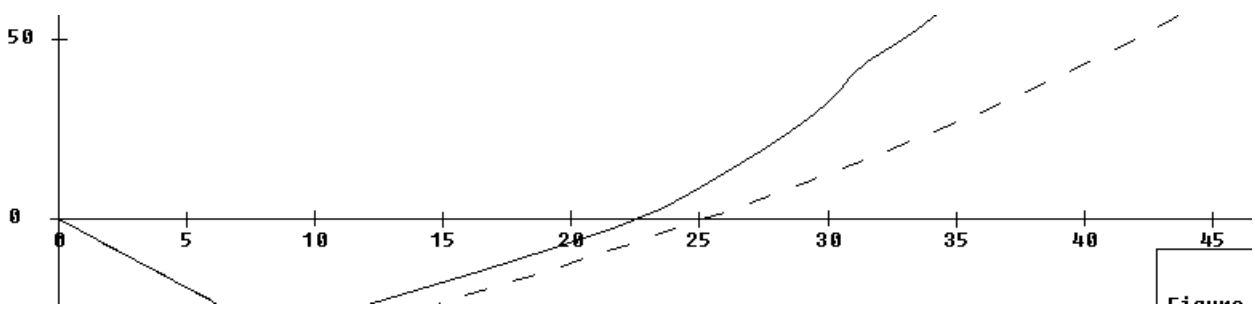
Nominal Strength

$$M_{mph_int} := 28 \frac{kip \cdot ft}{ft}$$

$$FS_{MPH1} := \frac{M_{mph_int}}{M_{mid}}$$

$$FS_{MPH1} = 0.438$$

Below 1.5. Perform reliability analysis



Reliability analysis indicates that wall 1 is unreliable, PF = 100%.

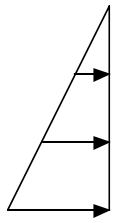
WALL 2, (North wall)

$K_o = 0.562$ $\quad$ $BLANKET = 36\ ft$ $\quad$ $HGL = 6.2\ ft$

$Hwall_2 = 30.25\ ft$ $\quad$ $\gamma_w = 62.4\ pcf$

Wall Loading

Soil or Soil & Water



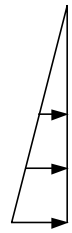
$H_2 := Hwall_2$

$H_2 = 30.25\ ft$

OR

$H_2 = 30.25\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_2 - HEAD$

$H''_3 := Hwall_2 + HGL - HEAD$

$H_3 := if\left(Hwall_2 > BLANKET, H''_3, H'_3\right)$

$H_3 = 35.4597\ ft$

$H'_3 = 35.46\ ft$

$H''_3 = 36.45\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 2039\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_w = 3191.27\ psf$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_2 := if\left(W_s > W_w, W_s, W_w\right)$

$W_2 = 3191\ \frac{lb}{ft^2}$

WALL 2, (North Wall)

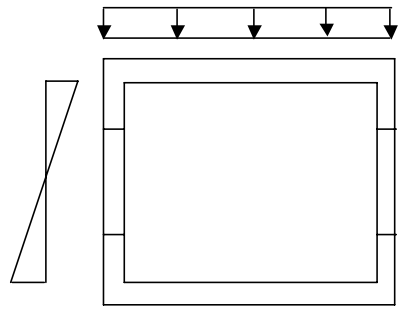
SHEAR

$$THRUST_2 := \frac{W_1 \cdot L_1}{2} \quad THRUST_2 = 21275 \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis (if necessary or applicable)

$$V'_{u2} := W_2 \cdot \frac{L_2}{2}$$

$$V'_{u2} = 11169 \frac{lb}{ft}$$



$$V_{u2} := 2 \frac{V'_{u2}}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2 \right) \quad V_{u2} = 7646 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$D_2 = 10 \, in$$

$$A_g := D_2 \cdot 12 \frac{in}{ft} \quad A_g = 120 \frac{in^2}{ft}$$

EM 1110-2-2104 EQ (5-1) Does not Apply

$$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_2 \right) \right]$$

ACI EQ (11-4)

$$\phi V_{n2} = 10375 \frac{lb}{ft}$$

$$d_2 = 7.25 \, in$$

$$\frac{THRUST_2 \cdot \frac{in^2}{lb}}{A_g} = 177.293$$

$$Check2 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$$

Check2 = "NO GOOD"

Factor of Safety

$$FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$$

$$FS_{S2} = 1.357$$

ANALYZE WALL 2 (North wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\,\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_2 - \frac{(W_2 - 0psf)\cdot 3ft}{30.25ft} = 2874.779\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{2875psf\cdot L_2^{\,2}}{8}$ $M_{mid} = 17.609\,\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_2^{\,2}}{12}$ $M_{end} = 0\,\frac{kip\cdot ft}{ft}$

#4's @ 6 in. inside face (mid-plate moment) $A_{mph} := 0.393\,\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 0.514\,in$

$$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right) \qquad \phi M_{mph} = 11.6\,\frac{kip\cdot ft}{ft}$$

no bars outside face (edge-plate moment) $A_{eph} := 0\,\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right) \qquad \phi M_{eph} = 0.0\,\frac{kip\cdot ft}{ft}$$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO GOOD")$ $CheckMPH = "NO GOOD"$

Factor of Safety $FS_{MPH2} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH2} = 0.66$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO GOOD")$ $CheckEPH = "NO GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3191.269\,psf$

$$W_2 - \frac{(W_2 - 0psf)\cdot 3ft}{30.25ft} = 2874.779\,psf$$

Thrust present 3 ft from bottom of wall.

$$THRUST := \frac{2875psf \cdot L_1}{2}$$ $$THRUST = 19166.667 \frac{lb}{ft}$$

$d_1 = 9.125\,in$ $A_g = 0.833\ ft$

$C_{c1} = 2.5\,in$

Loads from initial screening calculation: $M_u := 17.61\,kip \cdot ft$ $F_y = 40\,ksi$ $f'_c = 3000\,psi$

$Thrust := 19.17\,kip$

Compute Strength and Factor of Safety When Thrust is Included

NOTE: Reassign variables to allow use of the following routine

Steel Modulus of Elasticity: $E_s := 29000\,ksi$

Section Info: Thickness of Section, $D := 10\,in$ $b := 1\,ft$ (1 foot strip)

$A_1 := 0.393\,in^2$ $A_2 := 0\,in^2$ ONLY 1 LAYER OF STEEL PRESENT

$d_1 := d_2$ $d_2 := d_2$ Distance from compression face to bar centerline

$d_1 = 7.25\,in$

$$\beta 1 := if\left[f'_c \le 4ksi, 0.85, if\left[\left(f'_c\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_c}{ksi} - 4\right)\right]\right]$$ $\beta 1 = 0.85$

Assume c=0.2*D $i := 1 .. 2$

Given $c := .2 \cdot D$ $c = 2\,in$ $Thrust = 19.17\,kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_y \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_y \\ -F_y - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_y \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $c = 1.341\,in$ $a := if\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right)$ $a = 1.14\,in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s \cdot \varepsilon s_i > F_y, F_y, \text{if}(E_s \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'_c, 0ksi) \cdot A_i$ $\qquad$ $Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.013	-40	ksi	0.393	in^2	-15.72	kip	35.37	kip·in
0.604		-0.013	-40		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 35\ kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 155\ kip \cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 15.83\ kip \cdot ft$

$FS_{MPH2} := \frac{M_n}{M_u}$

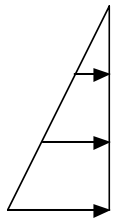
$Check := \text{if}(FS_{MPH2} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\qquad$ $FS_{MPH2} = 0.899$ $\qquad$ Check = "NO GOOD"

Reliability analysis was performed The calculated reliability is Pf = 34.91%. Based on these results, Wall 2 is **NOT** considered reliable.

WALL 3, (North wall) In place wall $K_o = 0.562$ $BLANKET = 36\ ft$ $HGL = 6.2\ ft$

Wall Loading

Soil or Soil & Water $Hwall_3 = 13.5\ ft$



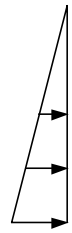
$H_2 := Hwall_3$

$H_2 = 13.5\ ft$

OR

$H_2 = 13.5\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_3 - HEAD$

$H''_3 := Hwall_3 + HGL - HEAD$

$H_3 := if\left(Hwall_3 > BLANKET, H''_3, H'_3\right)$

$H_3 = 15.825\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 909.8\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_w = 1424\ psf$

$H'_3 = 15.825\ ft$

$H''_3 = 19.7\ ft$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_{3ext} := if\left(W_s > W_w, W_s, W_w\right)$ $W_{3ext} = 1424\ psf$

Sand Loading (Interior Side of Wall)

The wall has sand placed on the other side. The sand reduces the net load acting on the wall.

$\phi_s := 32deg$ $K_{os} := 1 - \sin\left(\phi_s\right)$ $K_{os} = 0.47$

$\gamma_{sand} := 100pcf$

$H_{s3} := Hwall_3$ $H_{s3} = 13.5\ ft$

$W_{int3} := K_{os} \cdot \gamma_{sand} \cdot H_{s3}$ $W_{int3} = 634.609\ psf$ $W_{3ext} = 1424.203\ psf$

$W_3 := W_{3ext} - W_{int3}$ $W_3 = 789.594\ psf$ The net load acting on the wall

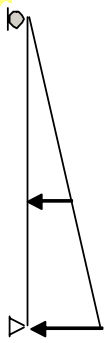
WALL 3, North Wall (In place wall)

SHEAR $L_3 = 22\ ft$ Thrust cannot be used when section spans vertically (no aid from earth loading). Model as a simple beam with increasing uniform load load case. The section spans vertically.

$$THRUST_1 := \frac{W_3 \cdot L_4 \cdot 0}{2} \qquad THRUST_1 = 0 \qquad V_{u1} := \frac{W_3 \cdot Hwall_3}{6} - \frac{W_3 \cdot Hwall_3}{2} \cdot \frac{d_3^{\,2}}{Hwall_3^{\,2}} \qquad V_{u1} = 1755.249\ \frac{lb}{ft}$$

$$V'_{u1} := W_3 \cdot \frac{L_3}{3}$$

$$V'_{u1} = 5790\ \frac{lb}{ft}$$



Elevation View

$$V_{u1} := 2\,\frac{V'_{u1}}{L_3} \cdot \left(\frac{L_3}{2} - \frac{D_2}{2} - d_3\right)$$

disabled

$$V_{u1} = 1755\ \frac{lb}{ft}$$

$D_2 = 0.833\ ft$

$d_1 = 0.76\ ft$

$L_1 = 13.333\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$A_g := D_1 \cdot 12\,\frac{in}{ft} \qquad A_g = 144\,\frac{in^2}{ft} \qquad \frac{L_1}{d_1} = 17.534$$

EM 1110-2-2104 EQ (5-1) Does not Apply

$$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1\right)\right]$$

ACI EQ (11-4)

$$\phi V_{n1} = 11995\ \frac{lb}{ft}$$

$$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

Check1 = "OKAY"

Factor of Safety

$$FS_{S3} := \frac{\phi V_{n1}}{V_{u1}}$$

$$FS_{S3} = 6.834$$

ANALYZE WALL 3 FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the vertical reinforcement carries all of the lateral loads $b := 12\,\frac{in}{ft}$

Evaluate the wall where the moment is greatest.

Wall 3 height is 13.5 ft ::: Wall 1 height is 30.25 ft::::Wall 2 height is 30.25 ft
Wall 4 height is 13.5 ft:::: Wall 5 height is 13.5 ft $Hwall_3 = 13.5\ ft$

The maximum moment occurs at $0.5774 \cdot Hwall_3 = 7.795\ ft$ from top of wall.

or $Hwall_3 - 7.79ft = 5.71\ ft$ from bottom of wall.

The maximum moment $M_{max} := \frac{0.128}{2} W_3 \cdot Hwall_3^{\,2}$ $M_{max} = 9.21\,\frac{kip \cdot ft}{ft}$ $W_3 = 789.594\,psf$

The end-span positive moment $M_{end} := \frac{0psf \cdot L_1^{\,2}}{12}$ $M_{end} = 0\,\frac{kip \cdot ft}{ft}$ no negative reinforcement

#6s @ 8 in. inside face (mid-plate moment) $A_{mph} := 0.663\,\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mph} = 0.867\,in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_3 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 21.7\,\frac{kip \cdot ft}{ft}$

no bars in outside face (edge-plate moment) $A_{eph} := 0\,\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{eph} = 0\,in$

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 0.0\,\frac{kip \cdot ft}{ft}$

$CheckMPH := if\left(\phi M_{mph} > 1.5 \cdot M_{max}, "OKAY", "NO\ GOOD"\right)$ $CheckMPH = "OKAY"$

Factor of Safety $FS_{MPH3} := \frac{\phi M_{mph}}{M_{max}}$ $FS_{MPH3} = 2.356$

$CheckEPH := if\left(\phi M_{eph} > 1.5 \cdot M_{end}, "OKAY", "NO\ GOOD"\right)$ $CheckEPH = "NO\ GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

$W_3 = 789.594\,psf$

NOTE: Thrust cannot be used to assist Wall 3 section strength because the reinforcement spans vertically. The end-of span moment is 0 and thus the FS_{EPH} is not relevant.

WALL 4, East Wall (In place wall)

$W_3 = 790 \frac{lb}{ft^2}$

$L_4 = 13\ ft$

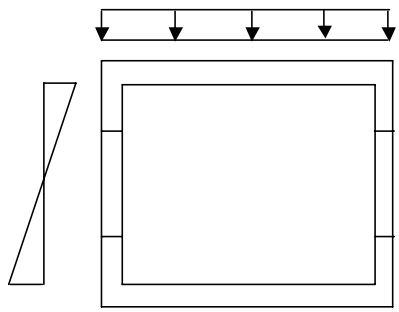
SHEAR

$THRUST_1 := \frac{W_3 \cdot L_3}{2}$ $\quad THRUST_1 = 8686 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_3 \cdot \frac{L_4}{2}$

$V'_{u1} = 5132 \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_4} \cdot \left(\frac{L_4}{2} - \frac{D_3}{2} - d_4 \right)$ $\quad V_{u1} = 4104 \frac{lb}{ft}$

$D_2 = 0.833\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$d_1 = 0.76\ ft$

$L_1 = 13.333\ ft$

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 144 \frac{in^2}{ft}$

$\frac{L_1}{d_1} = 17.534$ EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_4 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 12357 \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, \text{"OKAY"}, \text{"NO GOOD"}\right)$

Check1 = "OKAY"

Factor of Safety

$FS_{S4} := \frac{\phi V_{n1}}{V_{u1}}$

$FS_{S4} = 3.011$

ANALYZE WALL 4 (East wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom $W_3 = 789.594\,psf$

$$W_3 - \frac{(W_3 - 0psf)\cdot 3ft}{13.5ft} = 614.128\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{615psf\cdot L_4^2}{8}$ $M_{mid} = 12.992\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_2^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$ no negative reinforcement

0.75" @ 7 in. inside face (mid-plate moment) Square bars $A_{mph} := 0.964\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 1.26\,in$

$$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_4 - \frac{a_{mph}}{2}\right) \qquad \phi M_{mph} = 27.3\frac{kip\cdot ft}{ft}$$

no bars outside face (edge-plate moment) $A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right) \qquad \phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO GOOD")$ $CheckMPH = "OKAY"$

Factor of Safety $FS_{MPH4} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH4} = 2.101$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO GOOD")$ $CheckEPH = "NO GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Loads from initial screening calculation: $M_u := 13kip \cdot ft$ $F_y = 40\,ksi$ $f'_c = 3000\,psi$

$Thrust := 8.6kip$

Compute Strength and Factor of Safety When Thrust is Included

NOTE: Reassign variables to allow use of the following routine

Steel Modulus of Elasticity: $E_s := 29000ksi$

Section Info: Thickness of Section, $D := 12in$ $b := 1ft$ (1 foot strip)

$A_1 := 0.964in^2$ $A_2 := 0in^2$ ONLY 1 LAYER OF STEEL PRESENT

$d_1 := d_4$ $d_2 := d_4$ Distance from compression face to bar centerline

$d_1 = 9.125\,in$

$$\beta 1 := if\left[f'_c \le 4ksi, 0.85, if\left[\left(f'_c\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_c}{ksi} - 4\right)\right]\right]$$

$\beta 1 = 0.85$

Assume c=0.2*D $i := 1..2$

Given $c := .2\cdot D$ $c = 2.4\,in$ $Thrust = 8.6\,kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_y \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \\ F_y - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_y \\ -F_y - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_y \end{array}\right]\cdot A_i\right] \ldots$$

$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := Find(c)$ $c = 1.813\,in$ $a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right)$ $a = 1.541\,in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s \cdot \varepsilon s_i > F_y, F_y, \text{if}(E_s \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'_c, 0ksi) \cdot A_i$ $\qquad$ $Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.76	ft	-0.012	-40	ksi	0.964	in^2	-38.56	kip	120.5	kip·in
0.76		-0.012	-40		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 120\ kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 247\ kip \cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 30.59\ kip \cdot ft$

$FS_{MPH4} := \frac{M_n}{M_u}$

$Check := \text{if}(FS_{MPH4} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\qquad$ $FS_{MPH4} = 2.353$ $\qquad$ Check = "OKAY"

WALL 5, South Wall (In place wall)

$W_3 = 790 \frac{lb}{ft^2}$

$L_4 = 13\ ft$

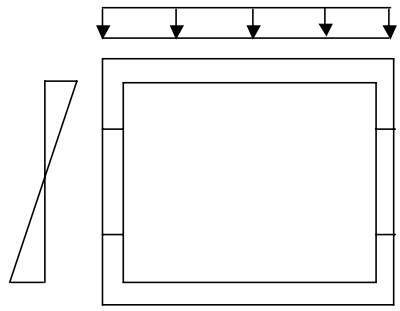
SHEAR

$THRUST_1 := \frac{W_3 \cdot L_4}{2}$ $\quad THRUST_1 = 5132 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_3 \cdot \frac{L_5}{2}$

$V'_{u1} = 2599 \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_4} \cdot \left(\frac{L_4}{2} - \frac{D_3}{2} - d_4 \right)$ $\quad V_{u1} = 2078 \frac{lb}{ft}$

$D_2 = 0.833\ ft$

$d_1 = 0.76\ ft$

$L_1 = 13.333\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 144 \frac{in^2}{ft}$

$\frac{L_1}{d_1} = 17.534$ EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_4 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 12209 \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO\ GOOD"\right)$ $\quad Check1 = "OKAY"$

Factor of Safety

$FS_{S5} := \frac{\phi V_{n1}}{V_{u1}}$ $\quad FS_{S5} = 5.874$

ANALYZE WALL 5 (South wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_3 - \frac{(W_3 - 0psf)\cdot 3ft}{13.5ft} = 614.128\ psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{615psf\cdot L_5^2}{8}$ $M_{mid} = 3.332\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_1^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$

5/8" @ 7 in. inside face (mid-plate moment) Square bars

$A_{mph} := 0.669\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 0.875\ in$

$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_5 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 15.1\frac{kip\cdot ft}{ft}$

no bars outside face (edge-plate moment)

$A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\ in$

$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO\ GOOD")$ $CheckMPH = "OKAY"$

Factor of Safety $FS_{MPH5} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH5} = 4.518$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO\ GOOD")$ $CheckEPH = "NO\ GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Uplift on Structure Refer to Figure 2 for the identified areas concerning uplift calculations

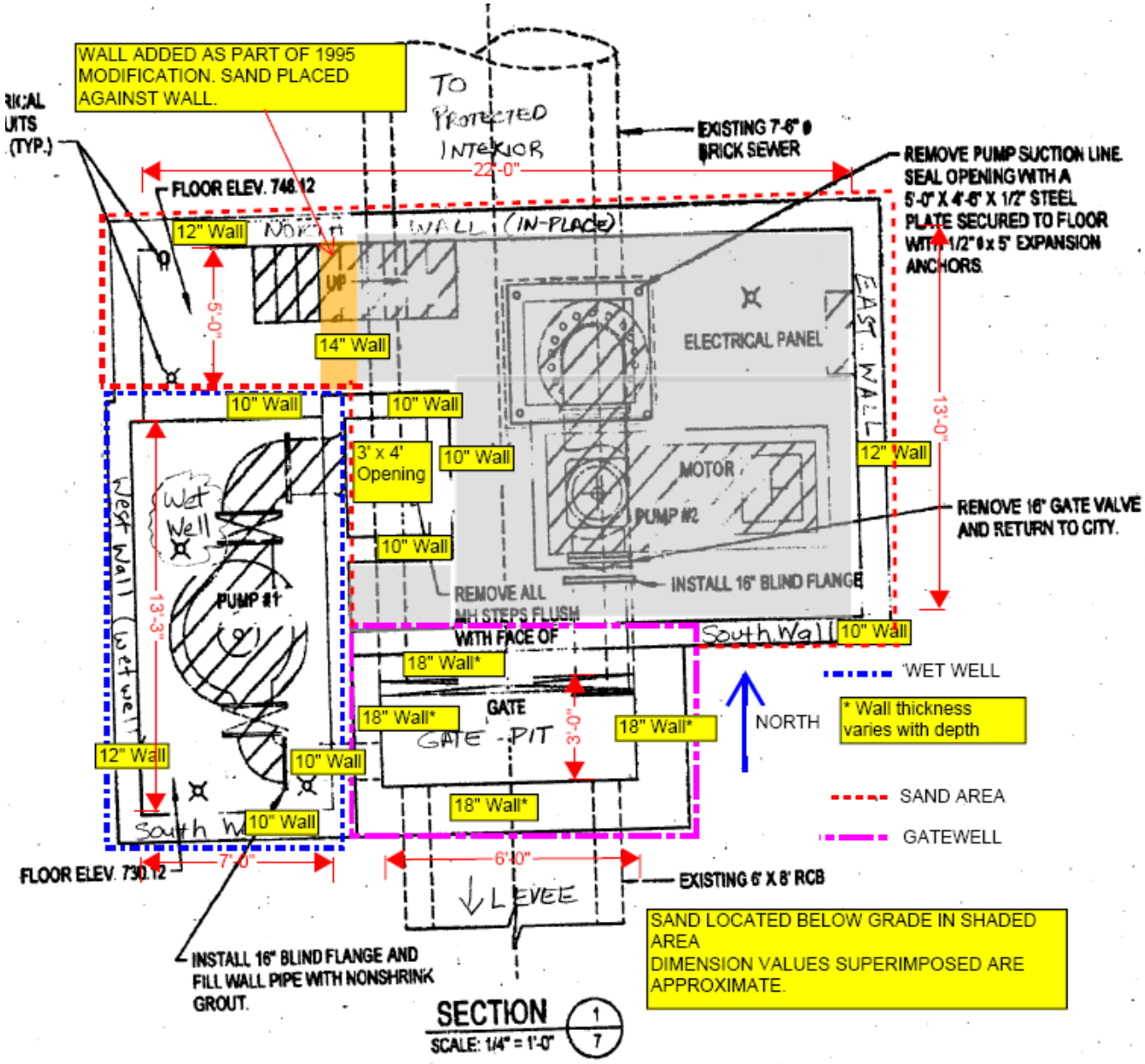


Figure 2. Section cut between elevation 748.12 and 755, Mill St Pump Station after 1995 modifications (DR&G Engineers).

Wet well weight.

The wetwell weight includes all of the following within the outlined area in Figure 2:

1. Wetwell walls
2. Top slab within the identified wetwell area (Note that the top slab extends beyond the wetwell footprint).
3. Concrete wall above grade within the wetwell area.
4. Wetwell base slab
5. Estimated weight of steel used to rehabilitate the wetwell for strength deficiencies.

The above grade concrete wall size and top slab area is not shown on the DR&G Engineers drawings. The values used in calculations are conservative (low) estimate values when used to calculate uplift.

Uplift on Wetwell Structure

The 1995 DR&G drawings show sand placed in the pump station.

Weight of wetwell.The equipment weight was not included.

Top Slab

$Top := 15in \cdot (7ft + 12in + 10in) \cdot (13.33ft + 12in + 10in)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 25.114\,kip$

Walls

$West := 30.25ft \cdot 13.33ft \cdot 1ft$ $\quad Wwest := West \cdot 150pcf$ $\quad Wwest = 60.485\,kip$

$South := 30.25ft \cdot 7ft \cdot 10in$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 26.469\,kip$

$East := West$ $\quad Weast := Wwest$ $\quad Weast = 60.485\,kip$

$North := Hwall_2 \cdot L_2 \cdot D_2$ $\quad Wnorth := North \cdot 150pcf$ $\quad Wnorth = 26.469\,kip$

Base Slab

$Base := 10 \cdot in \cdot (7 \cdot ft + 1 \cdot ft + 10 \cdot in) \cdot (13.33 \cdot ft + 10 \cdot in + 10 \cdot in) + 14in \cdot 5ft \cdot 4.5ft$ $\quad 5ft \cdot 4.5ft = 22.5\,ft^2$

$Base = 136.642\,ft^3$ $\quad Wbase := Base \cdot 150pcf$ $\quad Wbase = 20.496\,kip$

$Basearea := (7 \cdot ft + 1 \cdot ft + 10 \cdot in) \cdot (13.33 \cdot ft + 10 \cdot in + 10 \cdot in)$ $\quad Basearea = 132.471\,ft^2$

South Wall above grade

$hsouth_{ag} := 2ft$

$South_{ag} := hsouth_{ag} \cdot 12in \cdot (7ft + 12in + 10in)$ $\quad Wsouth_{ag} := South_{ag} \cdot 150pcf$ $\quad Wsouth_{ag} = 2650\,lb$

West Wall above Grade

$hwest_{ag} := 2ft$

$West_{ag} := hwest_{ag} \cdot (3ft + 0in) \cdot 12in$ $\quad Wwest_{ag} := West_{ag} \cdot 150pcf$ $\quad Wwest_{ag} = 0.9\,kip$

$Wsteel := 12kip$ Estimated weight of steel placed in wetwell for strength rehabilitation

Wetwell Weight

$Wwetwell := Wtop + Wwest + Wsouth + Weast + Wnorth + Wbase \ldots + Wsouth_{ag} + Wwest_{ag} + Wsteel$ $\quad Wwetwell = 235.068\,kip$

Wetwell Structure Factor of Safety for Flotation

Weight of water that is present during the event

$Wellarea := 81ft^2$ EC-GD documentation

Bottom of wetwell base elevation

$Elevwetwellbot := 730.12ft - 10in - 14in$

$HEAD := 3ft$

$Elevwetwellbot = 728.12\ ft$

bottom of blanket at 725

$Water := HEAD \cdot Wellarea \cdot 62.4pcf$ $Water = 15.163\ kip$

$heel := 0ft$ $heel = 0$ The wet well does not have heels

$upliftarea := Basearea$ $upliftarea = 132.471\ ft^2$

$H := ELEV_1 - Elevwetwellbot$ $H = 32.88\ ft$ $ELEV_1 = 761\ ft$ Grade elevation

Weight of soil on the wetwell heels

$volsoil := (1ft - 1ft) \cdot heel + (1ft - 1ft) \cdot heel$ $volsoil = 0\ ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 0\ kip$ $volsoil \cdot \gamma_w = 0\ kip$

Head, if structural foundation in blanket. $HGL = 6.2\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot upliftarea \cdot \gamma_w$$ $U_1 = 318.6\ kips$

$BLANKET = 36\ ft$
$HGL = 6.2\ ft$

Head, if structural foundation extends through blanket.

$H = 32.88\ ft$

$$U_2 := \left[(H + HGL) \cdot (upliftarea) \cdot \gamma_w\right]$$ $U_2 = 323.042\ kips$

$$Uplift_{wet} := if\left(H > BLANKET, U_2, U_1\right)$$ $Uplift_{wet} = 318.6\ kips$ $\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 38.543\ ft$

The uplift force acting on the wetwell structure $Uplift_{wet} = 318.6\ kips$

$$Stability := \frac{Wwetwell + Water + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{wet} - volsoil \cdot \gamma_w}$$ $Stability = 0.785$ $(H + HGL) = 39.08\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $StabilityCheck = "Not\ Okay"$

Determine the amount of additional weight required to achieve FS = 1.1 of the wetwell structure

$$Wreqdwetwell := Uplift_{wet} \cdot 1.1 - \left[Wwetwell + Water + volsoil \cdot (\gamma - \gamma_w)\right]$$ $Wreqdwetwell = 100.229\ kip$

$Water = 15.163\ kip$

Structure containing sand.

$Sa := (22ft - 7ft - 14in)\cdot(5ft + 0in)\cdot(760ft - 748.12ft) \ldots + (10ft - 4ft)\cdot(7ft + 2in)\cdot(760ft - 748.12ft) \ldots + (3ft + 10in)\cdot(1ft + 7in)\cdot(760ft - 748.12ft)$ $\quad Sa = 1404.645\ ft^3$

$Wsa := 100pcf\cdot Sa$ $\quad Wsa = 140.465\ kip$

NOTE: See "Recommendations" for more details as to the considerations that must be taken into account during design if Wsa is to be considered as an aid against uplift.

Top Slab

$Top := 1.25ft\cdot(22ft - 7ft - 14in + 12in)\cdot(13ft + 10in + 12in)$ $\quad Wtop := Top\cdot 150pcf$ $\quad Wtop = 41.255\ kip$

East Wall

$heast := (13ft + 6in)$ $\quad heast = 13.5\ ft$

$East := heast\cdot(13ft + 0in)\cdot 12in$ $\quad Weast := East\cdot 150pcf$ $\quad Weast = 26.325\ kip$

South Wall

$hsouth := heast$ $\quad hsouth = 13.5\ ft$

$South := hsouth\cdot(6ft + 7in + 12in)\cdot 10in$ $\quad Wsouth := South\cdot 150pcf$ $\quad Wsouth = 12.797\ kip$

North Wall

$hnorth := heast$ $\quad hnorth = 13.5\ ft$

$North := hnorth\cdot(22ft + 2ft)\cdot 13in$ $\quad Wnorth := North\cdot 150pcf$ $\quad Wnorth = 52.65\ kip$

West Wall

$hwest := heast$

$West := hwest\cdot(5ft)\cdot 12in$ $\quad Wwest := West\cdot 150pcf$ $\quad Wwest = 10.125\ kip$

Interior Wall

$hint := 760ft - 748.12ft$ $\quad hint = 11.88\ ft$

$Int := hint\cdot 14in\cdot 5ft$ $\quad Wint := Int\cdot 150pcf$ $\quad Wint = 10.395\ kip$

Base Slab

$Wbase := [(6in)\cdot(22ft + 0in)\cdot 5ft + (7ft + 2in)\cdot(10ft + 4in + 10in + 3ft)\cdot 6in]\cdot 150pcf$ $\quad Wbase = 15.865\ kip$

$Wbldg := 3kip$ Estimated weight of steel building enclosure, (conservative)

Structure Containing Sand Weight

$Wsand := Wsa + Wtop + Weast + Wsouth + Wnorth + Wbase + Wint \ldots + Wbldg + Wwest$ $\quad Wsand = 312.876\ kip$

Structure containing sand, con't

Uplift On Sand Structure

$Asand := (11 \cdot ft + 11 \cdot in) \cdot (6 \cdot ft + 11 \cdot in + 20 \cdot in)$ $\quad Asand = 102.285\ ft^2$ Area uplift acts on

$ELEVsand := 747.6 ft$ Elevation at bottom of sand structure base slab

Bottom of blanket elev = 729.3

Bottom of wetwell elev = 725

$H := ELEV_1 - ELEVsand$ $\quad H = 13.4\ ft$ $\quad ELEV_1 = 761\ ft$

Head, if structural foundation in blanket. $\quad HGL = 6.2\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot Asand \cdot \gamma_w \qquad U_1 = 100.256\ kips$$

$BLANKET = 36\ ft$
$HGL = 6.2\ ft$

$H = 13.4\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (Asand) \cdot \gamma_w\right] \qquad U_2 = 125.098\ kips$$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 15.708\ ft$$

$$Uplift_{sand} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the sand structure $\quad Uplift_{sand} = 100.256\ kips$

Sand Structure Factor of Safety for Flotation

<u>Weight of soil on the sand structure heels</u>

$volsoil := (1ft - 1ft) \cdot heel + (1ft - 1ft) \cdot heel$ $\quad volsoil = 0\ ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 0\ kip$ $\quad volsoil \cdot \gamma_w = 0\ kip$

$$Stability := \frac{Wsand + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{sand} - volsoil \cdot \gamma_w}$$

$Stability = 3.121$ $\quad (H + HGL) = 19.6\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $\quad StabilityCheck = "Okay"$

Amount of weight available to assist the wetwell flotation .

$$Wsandforwetwell := Wsand - \frac{1.10}{Stability} \cdot \left[Wsand + volsoil \cdot (\gamma - \gamma_w)\right] \qquad Wsandforwetwell = 202.595\ kip$$

Determine if the pump station (without gatewell) meets the required factor of safety for uplift

To satisfy the overall factor of safety against flotation, the amount of weight available in the sand structure (Wsandforwetwell) must be greater than the amount of weight required such that the wetwell structure meets the required factor of safety (Wreqdwetwell).

$PumpStationmeetsuplift := if(Wsandforwetwell \geq Wreqdwetwell, "okay", "NO GOOD")$

$PumpStationmeetsuplift = "okay"$

Wall Shear Strength

This is assuming that the force can be transfered in shear at the wetwell and sand structure interface. Determine the length of wall required to transfer the shear force.

$Wsandforwetwell = 202.595\ kip$ Amount of sand weight available to increase the wetwell FS for flotation

$Wreqdwetwell = 100.229\ kip$ This is the amount of shear that has to be transferred in the walls

$t_{wall} := 10in$ Wall thickness

$d_e := 7.25in$ Effective depth of the wall reinforcement

The south and east wetwell walls are attached to the gatewell. Both of these walls are 10 inches in thickness.

$f'_c = 3\ ksi$ Concrete compressive strength

$$hreqd := \frac{Wreqdwetwell}{0.85 \cdot 2 \cdot \sqrt{f'_c} \cdot d_2 \cdot \frac{\sqrt{lb}}{in}}$$

$hnorth = 13.5\ ft$

$hreqd = 12.373\ ft$

$Availwallength := 20ft$ Length of a wall that is available to transfer the shear force. Based on the west wetwall height.

$Wallokforshear := if[hreqd \le (Availwallength), "okay", "NO\ GOOD"]$ $Wallokforshear = "okay"$

Consider the gatewell attached to the station.

$Wgate := [[(6ft \cdot 9ft) - (3ft \cdot 6ft)] \cdot (764ft - 729ft) + 10in \cdot (9ft \cdot 6ft)] \cdot 150pcf - (5ft \cdot 6ft \cdot 10in) \cdot 150pcf$

$Wgate = 192\ kip$

$Agate := (5.5ft + 0in) \cdot (10ft + 0in)$ $Agate = 55\ ft^2$ Area that uplift is acting on

$H := ELEV_1 - (729ft)$ $H = 32\ ft$ $ELEV_1 = 761\ ft$

Bottom of blanket elev = 729.3

Bottom of wetwell elev = 725

Head, if structural foundation in blanket. $HGL = 6.2\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot Agate \cdot \gamma_w$$ $U_1 = 128.738\ kips$

$BLANKET = 36\ ft$
$HGL = 6.2\ ft$

$H = 32\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (Agate) \cdot \gamma_w\right]$$ $U_2 = 131.102\ kips$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 37.511\ ft$$

$$Uplift_{gate} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the wetwell structure $Uplift_{gate} = 128.738\ kips$

$Stability_{gate} := \frac{Wgate}{Uplift_{gate}}$ $Stability_{gate} = 1.491$ $(H + HGL) = 38.2\ ft$

ASSUMES THE GATEWELL HAS NO HEELS (CONSERVATIVE)

Amount of weight required to achieve FS=1.10

$Wreqdgate := (Uplift_{gate} \cdot 1.10) - Wgate$ $Wreqdgate = -50.388\ kip$

Place sand in the gatewell as a fix. This may be necessary if the uplift force acting on the wetwell cannot be transferred in whole to the sand structure base slab, i.e. the gatewell will assist in resisting the uplift force. This will be achieved by placing a slab in the existing gatewell and placing sand above the slab.

$hsand := 0ft$

$Wgatesand := (3.833ft \cdot 8.33ft \cdot hsand) \cdot 100pcf$ $Wgatesand = 0\ kip$

Amount of weight left over to aid pump station

$Wgateavail := Wgatesand - Wreqdgate$ $Wgateavail = 50.388\ kip$ $Wreqdgate = -50388.053\ lb$

$Wsandforwetwell = 202.595\ kip$ $Wreqdwetwell = 100.229\ kip$

Weight required such that the entire pump station (with gatewell) meets FS = 1.1 for uplift

$Wreqd := Wreqdwetwell - Wsandforwetwell$ $Wreqd = -102.366\ kip$

The addditional weight placed in the gatewell (sand) must be equal to or greater than Wreqd

$PumpStationmeetsupliftwithgatesand := if(Wgateavail \geq Wreqd, "okay", "NO GOOD")$

$PumpStationmeetsupliftwithgatesand = "okay"$

Wet Well BASE SLAB ANALYSIS

$W_{water} := 15.16kip$ WEIGHT OF WATER

$BaseSlab := upliftarea$ $BaseSlab = 132.471\ ft^2$ $Uplift_{wet} = 318.6\ kip$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$

$W' := if\left(Stability < 1.0, \frac{Uplift_{wet}}{BaseSlab}, \frac{Wwetwell}{BaseSlab}\right)$ $W' = 1774.491\ psf$

$W' = 1774.491\ psf$

$\frac{Wwetwell + W_{water}}{BaseSlab} = 1888.932\ psf$

$\frac{Uplift_{wet}}{BaseSlab} = 2405.062\ psf$

$\frac{Wwetwell + volsoil \cdot (\gamma - \gamma_w)}{BaseSlab} = 1774.491\ psf$

$W' := 2406psf$

$Slab_t := 10in$ $L_{strip} := 8ft$ $Wall_t := D_1$ $Slab_d := d_f$ effective depth of slab

Analyze slab with one-way action

$Slab_d = 6.625\ in$

$D_1 = 12\ in$

$L := 7ft$

$V'_{ub} := \frac{W' \cdot L}{2}$ $V'_{ub} = 8421\ \frac{lb}{ft}$

$d_f = 6.625\ in$

$D_f = 0.833\ ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}} \cdot \left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 6206\ \frac{lb}{ft}$

$A_g := Slab_t \cdot 12\frac{in}{ft}$ $A_g = 120\ \frac{in^2}{ft}$

$Water = 15163.2\ lb$

$$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f \right) \right]$$

ACI EQ (11-4)

$$\phi V_{nb} = 8709 \frac{lb}{ft}$$

$$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, "OKAY", "NO GOOD"\right)$$

CheckBase = "NO GOOD"

Factor of Safety

$$FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$$

$$FS_{Sb} = 1.403$$

Slab Bending

Assume simple span moment (conservative). One-way action, investigate the short span moment capacity.

$$M_b := \frac{L^2 \cdot W'}{8} \qquad M_b = 14.737 \frac{kip \cdot ft}{ft}$$

#6's @ 12 in.

$$A_{mpl} := 0.442 \frac{in^2}{ft} \quad b := 12 \frac{in}{ft} \quad a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{mpl} = 0.578\ in$$

$$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_f - \frac{a_{mpl}}{2} \right) \qquad \phi M_{mpl} = 9.3 \frac{kip \cdot ft}{ft}$$

$$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot M_b, "OKAY", "NO GOOD"\right)$$

CheckMP = "NO GOOD"

Factor of Safety

$$FS_{Mb} := \frac{\phi M_{mpl}}{M_b}$$

$$FS_{Mb} = 0.633$$

Again, it is unknown what reinforcement exists in the wetwell slab.

FACTORS OF SAFETY SUMMARY

Five walls and the base slab were evaluated for strength. A summary of the results are below. The reliability analysis calculations are in separate MathCad files.

Analysis indicates that the wetwell walls are unreliable. Based on the Wall 2 calculations, the East and South wet well walls are also considered unreliable. The East and South wet well walls have the same span length, thickness and reinforcement as the North wet well wall (Wall 2) and are exposed to lateral loading equal to the lateral loading present on Wall 2.

The available drawings do not indicate reinforcement present in the wet well base slab. Assuming that #6 @ 12" are present in the base slab, the base slab factor of safety <1.0. Based on these results, the base slab is also considered unreliable.

WALL 1 (WEST WETWELL WALL)

BAD

Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S1} = 0.711$

Moment

$FS_{MPH1} = 0.438$ Mid-plate moment

$FS_{EPH1} =$ ▮ Edge-of-plate moment (not applicable)

RELIABILITY ANALYSIS INDICATES THAT WALL 1 IS UNRELIABLE.

WALL 2 (NORTH WETWELL WALL)

BAD

Wall 2 analyzed as a 2D model using RAM Advanse s/w. Wall 2 has intermediate supports.

Shear

$FS_{S2} = 1.357$ Reliability analysis indicates the wall being reliable for shear(Pf<0.0001)

Moment

$FS_{MPV2} =$ ▮ not applicable

$FS_{EPV2} =$ ▮ not applicable

$FS_{MPH2} = 0.899$ Reliability analysis indicates that the wall is uneliable for moment, Pf= 34.91%

$FS_{EPH2} =$ ▮ not applicable

OVERALL, RELIABILITY ANALYSIS INDICATES THAT WALL 2 IS UNRELIABLE.

WALL 3 (IN PLACE NORTH WALL) GOOD

Wall 3 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S3} = 6.834$

Moment

$FS_{MPH3} = 2.356$ Reliability analysis, Pf = 0.0154%

$FS_{EPH3} =$ [redacted]

SAND IS PLACED WITHIN THE INTERIOR AREAS OF WALL 3, ANALYSIS INDICATES THAT REHABILITATION IS NOT NECESSARY.

WALL 4 (IN PLACE EAST WALL) GOOD

Wall 4 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S4} = 3.011$

Moment

$FS_{MPH4} = 2.353$ Reliability analysis indicates that wall 4 is reliable, Pf ~ 0

$FS_{EPH3} =$ [redacted]

SAND IS PLACED WITHIN THE INTERIOR AREAS OF WALL 4, ANALYSIS INDICATES THAT REHABILITATION IS NOT NECESSARY.

WALL 5 (IN PLACE SOUTH WALL) GOOD

Wall 5 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S5} = 5.874$

Moment

$FS_{MPH5} = 4.518$

$FS_{EPH3} =$ [redacted]

SAND IS PLACED WITHIN THE INTERIOR AREAS OF WALL 5, ANALYSIS INDICATES THAT REHABILITATION IS NOT NECESSARY.

BASE SLAB STATUS: SUSPECT. The base slab reinforcement is unknown.

Shear

$FS_{Sb} = 1.403$

Moment

$FS_{Mb} = 0.633$ Analyzed as a simple span, one-way action.

STABILITY

GOOD - SEE NOTE

NOTE: The overall pump station stability is adequate (FS=1.10) given adequate transfer of the uplift force from the wetwell walls to the gatewell and "sand structure".

Probability of Failure
Mill Street Pump StationWall 1

STA 156+75

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the screening criteria derived from the EM and therefore, the following reliability analysis was performed to calculate the reliability. See the MathCad analysis for the complete existing condition strength check with water to the top of protection.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength
3. Material Properties used:
   Concrete Design Strength, $f'_c := 3000 \cdot psi$; Mean Concrete Strength, $f'_{cM} := 1.25 f'_c$
   Steel Design Strength, $F_y := 40 \cdot ksi$; Mean Steel Strength, $F_{yM} := 1.25 F_y$
4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

END OF ANALYSIS AND RECOMMENDATIONS

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. Caclulating Factors of Safety

Condition under consideration from strength check: Wall Flexural Steel (from mathcad strength analysis).

Design Concrete Strength

Steel Modulus of Elasticity: $E_s := 29000ksi$

Section Info: Thickness of Section, $D := 12in$ $b := 1ft$ (1 foot strip)

$A_1 := 0.884in^2$ $A_2 := 0in^2$ ONLY 1 LAYER OF STEEL PRESENT

$C_1 := 2.5in$ $C_2 := 2.5in$ Distance from compression face to bar centerline

$d_1 := D - C_1 - 0.375in$ $d_2 := C_2 + 0.375in$

Loads from initial screening calculation: $M_u := 63.9kip \cdot ft$

$Thrust := 10.06kip$

Mean Concrete Strength and Steel Yield Strength

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $i := 1..2$

Given $c := .2 \cdot D$ $c = 2.4\,in$ $Thrust = 10.06\,kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \\ F_{yM} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{array}\right]\cdot A_i\right] \ldots$$
$$+ 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := Find(c)$ $c = 1.669\,in$ $a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right)$ $a = 1.419\,in$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}(E_s\cdot\varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s\cdot\varepsilon s_i))$

$Fs_i := fs_i\cdot A_i - \text{if}(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi)\cdot A_i$ $\qquad$ $Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.76 ft	-0.013	-50 ksi	0.884 in^2	-44.2 kip	138.125 kip·in
0.24	-0.002	-50	0	0	0

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 138\ kip\cdot in$

$Mc := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 287\ kip\cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 35.43\ kip\cdot ft$

$FS_B := \frac{M_n}{M_u}$ $\quad$ $Check := \text{if}(FS_B > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\quad$ $FS_B = 0.555$ $\qquad$ Check = "NO GOOD"

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM}\cdot 0.14$ $\qquad$ $f'_{cU} = 4275\ psi$

$\beta 1 := \text{if}\left[f'_{cU} \le 4ksi, 0.85, \text{if}\left[(f'_{cU}) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cU}}{ksi} - 4\right)\right]\right]$ $\qquad$ $\beta 1 = 0.836$

Assume c=0.2*D $\qquad$ $i := 1 .. 2$

Given $\qquad$ $c := .2\cdot D$ $\qquad$ $c = 2.4\ in$ $\qquad$ Thrust = 10.06 kip

Upper Concrete Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ +-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \\ F_{yM}-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM}-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{array}\right]\cdot A_i\right] \ldots$$

$$+\,0.85\cdot f'_{cU}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 1.488\ \text{in}$ $\quad a := \text{if}\left(\beta1\cdot c > D, D, \beta1\cdot c\right)$ $\quad a = 1.244\ \text{in}$

Find Moment:

$\varepsilon s_i := \left(c-d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s\cdot\varepsilon s_i < 0-F_{yM}, 0-F_{yM}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cU}, 0\text{ksi}\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2}-d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.76	ft	-0.015	-50	ksi	0.884	in^2	-44.2	kip	138.125	kip·in
0.24		-0.003	-50		0		0		0	

$Mst_U := \sum_{i=1}^{2} Ms_i$ $\quad Mst_U = 138\ \text{kip}\cdot\text{in}$

$Mc_U := 0.85\cdot f'_{cU}\cdot a\cdot b\cdot\left(\frac{D}{2}-\frac{a}{2}\right)$ $\quad Mc_U = 292\ \text{kip}\cdot\text{in}$

$M_{nU} := \left(Mc_U + Mst_U\right)$ $\quad M_{nU} = 35.83\ \text{kip}\cdot\text{ft}$

$FS_{BU} := \frac{M_{nU}}{M_u}$ $\quad Check_U := \text{if}\left(FS_{BU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BU} = 0.561$ $\quad Check_U = \text{"NO GOOD"}$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\qquad f'_{cL} = 3225\,psi$

$$\beta 1 := \text{if}\left[f'_{cL} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cL}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cL}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2 \cdot D \qquad c = 2.4\,in \qquad Thrust = 10.06\,kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \\ F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots$$

$$+ 0.85 \cdot f'_{cL} \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \qquad c = 1.941\,in \qquad a := \text{if}\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right) \qquad a = 1.649\,in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(E_s \cdot \varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s \cdot \varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cL}, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.76	ft	-0.011	-50	ksi	0.884	in^2	-44.2	kip	138.125	kip·in
0.24		-0.001	-50		0		0		0	

Lower Concrete Strength, con't

$$Mst_L := \sum_{i=1}^{2} Ms_i \qquad Mst_L = 138\ kip\cdot in$$

$$Mc_L := 0.85\cdot f'_{cL}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right) \qquad Mc_L = 281\ kip\cdot in$$

$$M_{nL} := \left(Mc_L + Mst_L\right) \qquad M_{nL} = 34.91\ kip\cdot ft$$

$$FS_{BL} := \frac{M_{nL}}{M_u} \qquad Check_L := if\left(FS_{BL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$FS_{BL} = 0.546$ $\qquad$ $Check_L = \text{"NO GOOD"}$

Upper Steel Yield Strength

$$F_{yU} := F_{yM} + F_{yM}\cdot 0.14 \qquad F_{yU} = 57\ ksi$$

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2\cdot D \qquad c = 2.4\ in \qquad Thrust = 10.06\ kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{l} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots \quad \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yU} \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) \\ F_{yU} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) \quad \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yU} \\ -F_{yU} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) \quad \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yU} \end{array}\right]\cdot A_i\right] \ldots$$
$$+ 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := Find(c) \qquad c = 1.859\ in \qquad a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right) \qquad a = 1.58\ in$

Upper Steel Yield Strength, con't

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(E_s \cdot \varepsilon s_i > F_{yU}, F_{yU}, if(E_s \cdot \varepsilon s_i < 0 - F_{yU}, 0 - F_{yU}, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cM}, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.76 ft	-0.012	-57 ks	0.884 in^2	-50.388 kip	157.462 kip·in
0.24	-0.002	-41.892	0	0	0

$Mst_{cU} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cU} = 157\ kip \cdot in$

$Mc_{cU} := 0.85 \cdot f'_{cM} \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cU} = 315\ kip \cdot in$

$M_{ncU} := (Mc_{cU} + Mst_{cU})$ $\quad M_{ncU} = 39.37\ kip \cdot ft$

$FS_{BcU} := \frac{M_{ncU}}{M_u}$ $\quad Check_{cU} := if(FS_{BcU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$

$FS_{BcU} = 0.616$ $\quad Check_{cU} = \text{"NO GOOD"}$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14$ $\quad F_{yL} = 43\ ksi$

$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[(f'_{cM}) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right]$ $\quad \beta 1 = 0.85$

Assume c=0.2*D $\quad i := 1 .. 2$

Given $\quad c := .2 \cdot D$ $\quad c = 2.4\ in$ $\quad Thrust = 10.06\ kip$

Lower Steel Yield Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2} \left[\left[\begin{cases} (c - d_i)\cdot\frac{0.003}{c}\cdot E_s \ldots + -\text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } (c - d_i)\cdot\frac{0.003}{c}\cdot E_s < F_{yL} \\ F_{yL} - \text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } (c - d_i)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yL} \\ -F_{yL} - \text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } (c - d_i)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yL} \end{cases} \right] \cdot A_i \right] \ldots + 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 1.479\ \text{in}$ $\quad a := \text{if}(\beta 1\cdot c > D, D, \beta 1\cdot c)$ $\quad a = 1.257\ \text{in}$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yL}, F_{yL}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yL}, 0 - F_{yL}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0\text{ksi}\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.76 ft	-0.016	-43 ksi	0.884 in^2	-38.012 kip	118.787 kip·in
0.24	-0.003	-43	0	0	0

$Mst_{cL} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cL} = 119\ \text{kip}\cdot\text{in}$

$Mc_{cL} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cL} = 258\ \text{kip}\cdot\text{in}$

$M_{ncL} := (Mc_{cL} + Mst_{cL})$ $\quad M_{ncL} = 31.42\ \text{kip}\cdot\text{ft}$

$FS_{BcL} := \frac{M_{ncL}}{M_u}$ $\quad Check_{cL} := \text{if}(FS_{BcL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$

$FS_{BcL} = 0.492$ $\quad Check_{cL} = \text{"NO GOOD"}$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{BcU} - FS_{BcL}$ $\Delta F_{Fy} = 0.124$

$\Delta F_{fc} := FS_{BU} - FS_{BL}$ $\Delta F_{fc} = 0.014$

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$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2}$$ $\sigma_F = 0.063$

$$V_F := \frac{\sigma_F}{FS_B}$$ $V_F = 0.113$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_B}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$$ $\beta_{LN} = -5.296$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 5.92 \times 10^{-6}\,\%$ $M_u = 63.9\,\mathrm{kip \cdot ft}$

$\mathrm{Thrust} = 10.06\,\mathrm{kip}$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 100\,\%$

When Moment=24 k*ft and Thrust=3.5 kip, the P_F value becomes 1%

Probability of Failure
Mill Street Pump StationWall 2

STA 156+75

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the screening criteria derived from the EM and therefore, the following reliability analysis was performed to calculate the reliability. See the MathCad analysis for the complete existing condition strength check with water to the top of protection.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength
3. Material Properties used:
   Concrete Design Strength, $f'_c := 3000 \cdot psi$; Mean Concrete Strength, $f'_{cM} := 1.25 f'_c$
   Steel Design Strength, $F_y := 40 \cdot ksi$; Mean Steel Strength, $F_{yM} := 1.25 F_y$
4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. Caclulating Factors of Safety

Condition under consideration from strength check: Wall Flexural Steel (from mathcad strength analysis).

Design Concrete Strength

Steel Modulus of Elasticity: $E_s := 29000ksi$

Section Info: Thickness of Section, $D := 10in$ $b := 1ft$ (1 foot strip)

$A_1 := 0.393in^2$ $A_2 := 0in^2$ ONLY 1 LAYER OF STEEL PRESENT

$C_1 := 2.5in$ $C_2 := 2.5in$ Distance from compression face to bar centerline

$d_1 := D - C_1 - 0.25in$ $d_2 := C_2 + 0.375in$

$d_1 = 7.25\ in$

Loads from initial screening calculation: $M_u := 17.61kip \cdot ft$

$Thrust := 19.17kip$

Mean Concrete Strength and Steel Yield Strength

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $i := 1..2$

Given $c := .2\cdot D$ $c = 2\ in$ $Thrust = 19.17\ kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \\ F_{yM} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{array}\right]\cdot A_i\right] \ldots$$
$$+ 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := Find(c)$ $c = 1.194\ in$ $a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right)$ $a = 1.015\ in$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}(E_s\cdot\varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s\cdot\varepsilon s_i))$

$Fs_i := fs_i\cdot A_i - \text{if}(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi)\cdot A_i$ $\qquad$ $Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.015	-50	ksi	0.393	in^2	-19.65	kip	44.213	kip·in
0.24		-0.004	-50		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 44\,kip\cdot in$

$Mc := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 174\,kip\cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 18.22\,kip\cdot ft$

$FS_B := \frac{M_n}{M_u}$ $\quad$ $Check := \text{if}(FS_B > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\quad$ $FS_B = 1.035$ $\qquad$ Check = "NO GOOD"

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM}\cdot 0.14$ $\qquad$ $f'_{cU} = 4275\,psi$

$\beta 1 := \text{if}\left[f'_{cU} \le 4ksi, 0.85, \text{if}\left[(f'_{cU}) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cU}}{ksi} - 4\right)\right]\right]$ $\qquad$ $\beta 1 = 0.836$

Assume c=0.2*D $\qquad$ $i := 1..2$

Given $\qquad$ $c := .2\cdot D$ $\qquad$ $c = 2\,in$ $\qquad$ Thrust = 19.17 kip

Upper Concrete Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ +-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \\ F_{yM}-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM}-\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{array}\right]\cdot A_i\right] \ldots$$

$$+\,0.85\cdot f'_{cU}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 1.065\ \text{in}$ $\quad a := \text{if}\left(\beta1\cdot c > D, D, \beta1\cdot c\right)$ $\quad a = 0.89\ \text{in}$

Find Moment:

$\varepsilon s_i := \left(c-d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s\cdot\varepsilon s_i < 0-F_{yM}, 0-F_{yM}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cU}, 0\text{ksi}\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2}-d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.604 ft	-0.017	-50 ksi	0.393 in^2	-19.65 kip	44.213 kip·in
0.24	-0.005	-50	0	0	0

$Mst_U := \sum_{i=1}^{2} Ms_i$ $\quad Mst_U = 44\ \text{kip}\cdot\text{in}$

$Mc_U := 0.85\cdot f'_{cU}\cdot a\cdot b\cdot\left(\frac{D}{2}-\frac{a}{2}\right)$ $\quad Mc_U = 177\ \text{kip}\cdot\text{in}$

$M_{nU} := \left(Mc_U + Mst_U\right)$ $\quad M_{nU} = 18.42\ \text{kip}\cdot\text{ft}$

$FS_{BU} := \frac{M_{nU}}{M_u}$ $\quad Check_U := \text{if}\left(FS_{BU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BU} = 1.046$ $\quad Check_U = \text{"NO GOOD"}$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\qquad f'_{cL} = 3225\,psi$

$$\beta 1 := if\left[f'_{cL} \le 4ksi, 0.85, if\left[\left(f'_{cL}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cL}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2 \cdot D \qquad c = 2\,in \qquad Thrust = 19.17\,kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & \\ F_{yM} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_{cL} \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c) \qquad c = 1.388\,in \qquad a := if\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right) \qquad a = 1.18\,in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := if\left(E_s \cdot \varepsilon s_i > F_{yM}, F_{yM}, if\left(E_s \cdot \varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - if\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cL}, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.013	-50	ksi	0.393	in^2	-19.65	kip	44.213	kip·in
0.24		-0.003	-50		0		0		0	

Lower Concrete Strength, con't

$$Mst_L := \sum_{i=1}^{2} Ms_i \qquad Mst_L = 44\,kip\cdot in$$

$$Mc_L := 0.85\cdot f'_{cL}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right) \qquad Mc_L = 171\,kip\cdot in$$

$$M_{nL} := \left(Mc_L + Mst_L\right) \qquad M_{nL} = 17.95\,kip\cdot ft$$

$$FS_{BL} := \frac{M_{nL}}{M_u} \qquad Check_L := if\left(FS_{BL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$FS_{BL} = 1.019$ $\qquad$ $Check_L = \text{"NO GOOD"}$

Upper Steel Yield Strength

$$F_{yU} := F_{yM} + F_{yM}\cdot 0.14 \qquad F_{yU} = 57\,ksi$$

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2\cdot D \qquad c = 2\,in \qquad Thrust = 19.17\,kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yU} \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \\ F_{yU} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yU} \\ -F_{yU} - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yU} \end{array}\right]\cdot A_i\right] \ldots$$
$$+ 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := Find(c) \qquad c = 1.279\,in \qquad a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right) \qquad a = 1.087\,in$

Upper Steel Yield Strength, con't

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := \text{if}(E_s \cdot \varepsilon s_i > F_{yU}, F_{yU}, \text{if}(E_s \cdot \varepsilon s_i < 0 - F_{yU}, 0 - F_{yU}, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cM}, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.604 ft	-0.014	-57 ksi	0.393 in^2	-22.401 kip	50.402 kip·in
0.24	-0.004	-57	0	0	0

$Mst_{cU} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cU} = 50\,kip \cdot in$

$Mc_{cU} := 0.85 \cdot f'_{cM} \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cU} = 185\,kip \cdot in$

$M_{ncU} := (Mc_{cU} + Mst_{cU})$ $\quad M_{ncU} = 19.64\,kip \cdot ft$

$FS_{BcU} := \frac{M_{ncU}}{M_u}$ $\quad Check_{cU} := \text{if}(FS_{BcU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$

$FS_{BcU} = 1.115$ $\quad Check_{cU} = \text{"NO GOOD"}$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14$ $\quad F_{yL} = 43\,ksi$

$\beta 1 := \text{if}\left[f'_{cM} \le 4ksi, 0.85, \text{if}\left[(f'_{cM}) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right]$ $\quad \beta 1 = 0.85$

Assume c=0.2*D $\quad i := 1 .. 2$

Given $\quad c := .2 \cdot D$ $\quad c = 2\,in$ $\quad Thrust = 19.17\,kip$

Lower Steel Yield Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yL} \\ +-\text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \\ F_{yL} - \text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yL} \\ -F_{yL} - \text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yL} \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85\cdot f'_{cM}\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 1.109\ \text{in}$ $\quad a := \text{if}\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right)$ $\quad a = 0.943\ \text{in}$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yL}, F_{yL}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yL}, 0 - F_{yL}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0\text{ksi}\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.017	-43	ksi	0.393	in^2	-16.899	kip	38.023	kip·in
0.24		-0.005	-43		0		0		0	

$Mst_{cL} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cL} = 38\ \text{kip}\cdot\text{in}$

$Mc_{cL} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cL} = 163\ \text{kip}\cdot\text{in}$

$M_{ncL} := \left(Mc_{cL} + Mst_{cL}\right)$ $\quad M_{ncL} = 16.78\ \text{kip}\cdot\text{ft}$

$FS_{BcL} := \frac{M_{ncL}}{M_u}$ $\quad Check_{cL} := \text{if}\left(FS_{BcL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BcL} = 0.953$ $\quad Check_{cL} = \text{"NO GOOD"}$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{BcU} - FS_{BcL}$ $\Delta F_{Fy} = 0.162$

$\Delta F_{fc} := FS_{BU} - FS_{BL}$ $\Delta F_{fc} = 0.027$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2} \qquad \sigma_F = 0.082$$

$$V_F := \frac{\sigma_F}{FS_B} \qquad V_F = 0.08$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_B}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = 0.388$$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 65.09\ \%$

$M_u = 17.61\ \mathrm{kip \cdot ft}$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$\mathrm{Thrust} = 19.17\ \mathrm{kip}$

$P_F := 1 - R$ $P_F = 34.91\ \%$

When the moment=13 k*ft and thrust=19.17 k the P_F value becomes 1%

CONCLUSIONS

The existing condition HGL elevation is 767.2. The existing wetwell walls and base slab are inadequate for the **HGL=767.2** event loading. The existing structure is also inadequate for uplift stability for the **HGL=767.2** event loading. The reliability analysis indicates that Wall 1 and Wall 2 are unreliable for the existing condition event loading. Wall 1 controls for strength.

Backcheck of peer review by:
Paul Muller 4/9/07

Comp by: MLP
Chkd by:

US Army Corps of Engineers

Armourdale District n500+3 Event
Mill Street Pump Station Analysis Sta. 156+75, HGL=770.9

KANSAS CITYS LEVEES PHASE II

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $pcf := \frac{lb}{ft^3}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $kips := 1000lb$ $klf := \frac{kips}{ft}$

This analysis is for the n500+3 event that has an HGL elevation = 770.9

Top of existing levee El = 768.7
Bottom of blanket El = 725.0
Ground El = 760.0

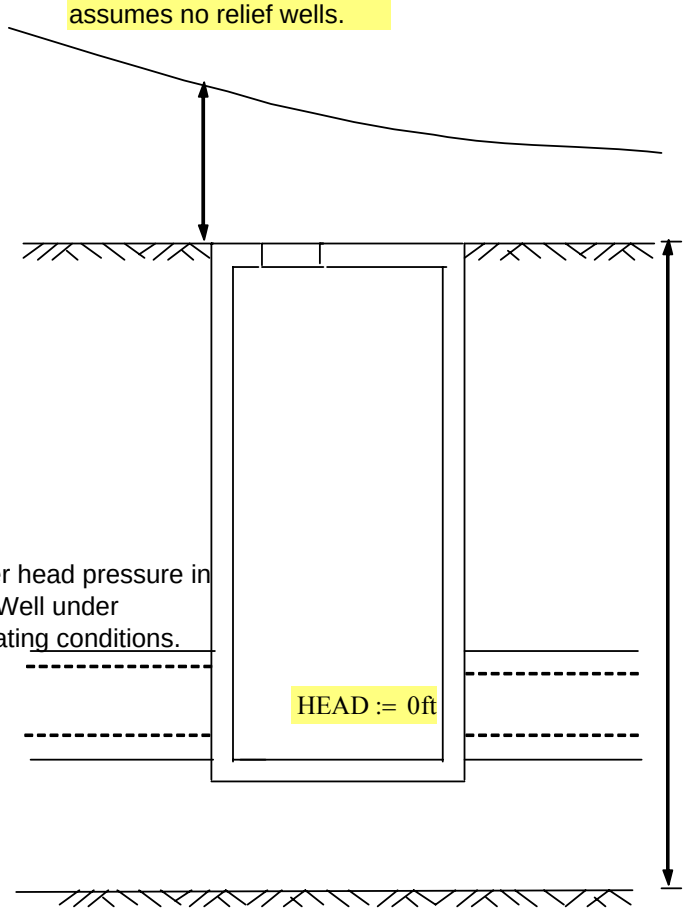
REFERENCES
1. Drawings, dated 1914 and 1916.
2. DR&G Engineers drawings (pump modification), 1995.

Properties

NOTE: HGL Supplied assumes no relief wells.

Original Hydraulic Grade Line as supplied by Geotechnicals (Affects Uplift on Base Slab if in the landside of levee)

Top of levee elev = 768.3 ft



Elevation

$ELEV_1 := 761ft$

$HGL := 770.9ft - ELEV_1$

$HGL = 9.9\ ft$

Water head pressure in Wet Well under operating conditions.

$BLANKET := 761ft - 725ft$

$BLANKET = 36\ ft$

$HEAD := 0ft$

Structural drawings dated 1914, 1916 and 1995 were obtained. The 1995 drawings were generated as a result of pump replacement done by DR&G Engineers.
The structure has undergone significant modifications. Records indicate that in the 1910s, the wetwell area was enlarged. The enlargement was probably necessitated by the size of the pump. An interior wall was also added as part of the modification. In the 1990s, the single pump was replaced with two pump. According to the drawing information found, the exterior wall modifications was limited to the 1910s modification.

Refering to the 1916 drawings, modifications to the "wall in place" was accomplished by removing an existing south wall and extending the west wall, thus increasing the wet well area.

Wall 1 Parameters Wet well	Wall 2 Parameters Wet well	Wall 3 Parameters In place wall	Wall 4 Parameters In place wall	Wall 5 Parameters In place wall
West wall	North wall	North wall	East wall	South wall
$L_1 := 13ft + 4in$	$L_2 := 7ft$	$L_3 := 22ft$	$L_4 := 13ft$	$L_5 := 6ft + 7in$
$Hwall_1 := 30.25ft$	$Hwall_2 := 30.25ft$	$Hwall_3 := 13.5ft$	$Hwall_4 := 13.5ft$	$Hwall_5 := 13.5ft$
$D_1 := 1ft$	$D_2 := 10in$	$D_3 := 13in$	$D_4 := 12in$	$D_5 := 10in$
$C_{c1} := 2.5in$	$C_{c2} := 2.5in$	$C_{c3} := 2.5in$	$C_{c4} := 2.5in$	$C_{c5} := 2.5in$
			Square Bars	Square Bars

Hwall is measured from grade and may not be the actual wall height. The Hwall value is used to determine the wall loads.
Sand was placed in the interior area created by the "In place" walls as part of the 1995 modification.

BASE SLAB

The base slab reinforcement is not shown on the available drawings. Assume that the base slab has #6 at 12 inches.

Floor Thickness $D_f := 10in$ Clear Cover $C_{cf} := 3in$ $d_f := D_f - C_{cf} - \frac{0.75in}{2}$ $d_f = 6.625\ in$

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - C_{c1} - 0.375in$ $d_1 = 9.125\ in$

WALL 3

$d_3 := D_3 - C_{c3} - 0.25in$ $d_3 = 10.25\ in$

WALL 2

$d_2 := D_2 - C_{c2} - 0.25in$ $d_2 = 7.25\ in$

WALL 4

$d_4 := D_4 - C_{c4} - 0.375in$ $d_4 = 9.125\ in$

WALL 5

$d_5 := D_5 - C_{c5} - \frac{5}{16}in$ $d_5 = 7.188\ in$

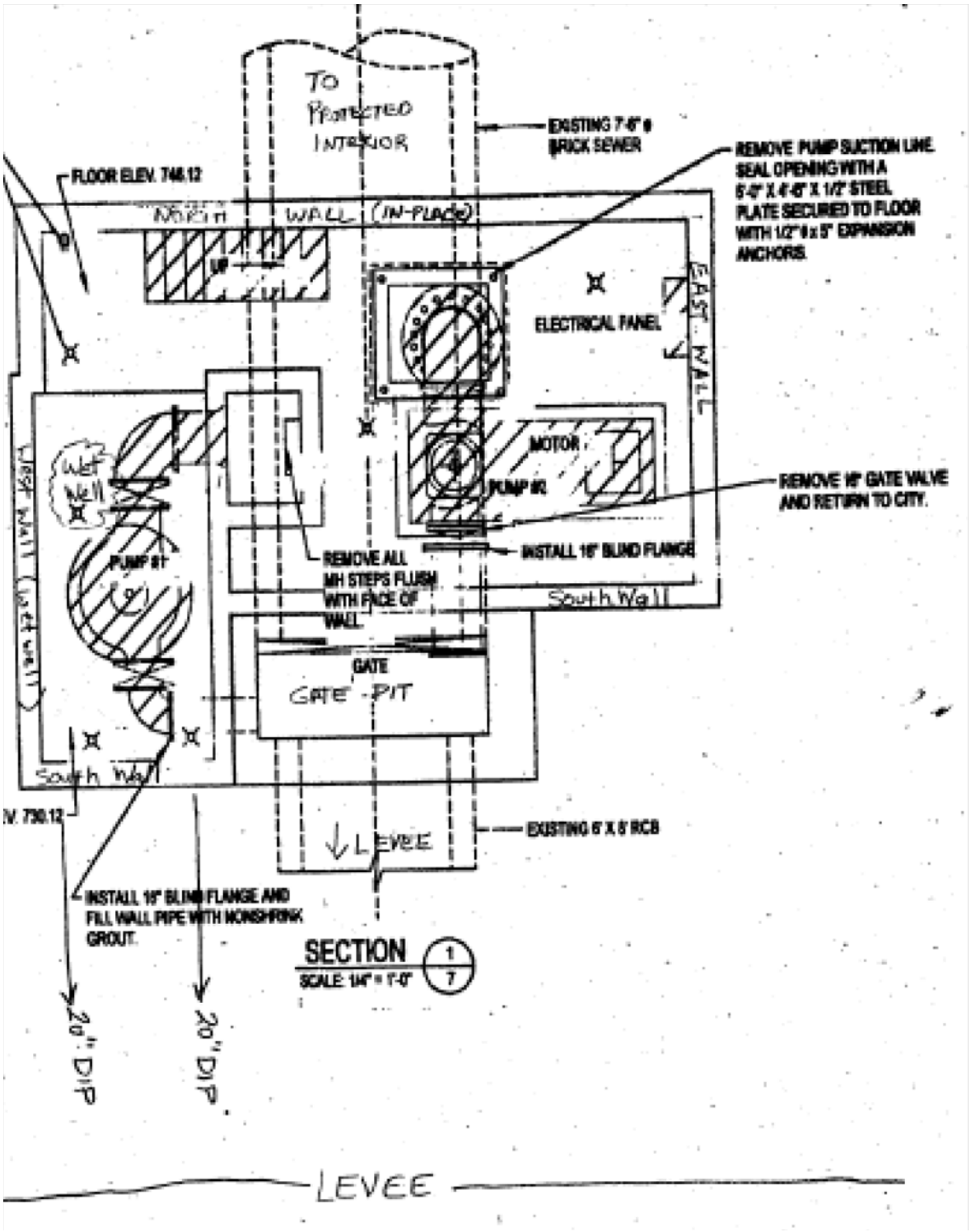


Figure 1. Mill Street Pump StationPlan View Drawing Sheet 7 of 17 DR&G Engineers, 1995

Assumptions and Criteria

- As-built drawings give fc=1350 and fs=20,000. These are working stress parameters. For this analysis, f'c=3000 psf and fy=40,000 psf.

Soil Properties	$\phi := 26deg$	$K_o := 1 - \sin(\phi)$	Steel Properties	$F_y := 40ksi$
	$\gamma := 120pcf$	$K_o = 0.562$	Concrete Properties	$f'_c := 3.00ksi$
Water Unit Weight	$\gamma_w := 62.4pcf$		Concrete Unit Weight	$\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

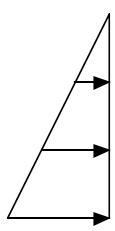
WALL 1, (West wall)

$K_o = 0.562$ $BLANKET = 36\ ft$ $HGL = 9.9\ ft$

$Hwall_1 = 30.25\ ft$

Wall Loading

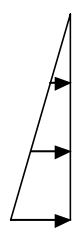
Soil or Soil & Water



$H_2 := Hwall_1$

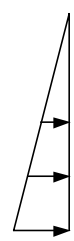
$H_2 = 30.25\ ft$

OR



$H_2 = 30.25\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_1 - HEAD$

$H''_3 := Hwall_1 + HGL - HEAD$

$H_3 := if(Hwall_1 > BLANKET, H''_3, H'_3)$

$H_3 = 38.569\ ft$

$H'_3 = 38.569\ ft$

$H''_3 = 40.15\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 2039\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3]$

$W_w = 3385\ \frac{lb}{ft^2}$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_1 := if(W_s > W_w, W_s, W_w)$

$W_1 = 3385\ \frac{lb}{ft^2}$

WALL 1 (West Wall) SHEAR

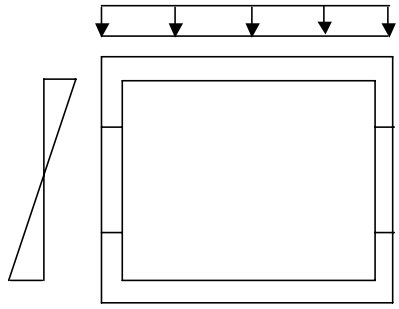
$L_2 = 7\ ft$

$THRUST_1 := \frac{W_1 \cdot L_2}{2}$ $\quad THRUST_1 = 11848\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_1 \cdot \frac{L_1}{2}$

$V'_{u1} = 22568\ \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$ $\quad V_{u1} = 18584\ \frac{lb}{ft}$

$D_2 = 0.833\ ft$

$d_1 = 9.125\ in$

$L_1 = 13.333\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\frac{L_1}{d_1} = 17.534$ EM 1110-2-2104 EQ (5-1) Does not Apply

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 144 \frac{in^2}{ft}$

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 12489\ \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD"\right)$

$Check1 = "NO GOOD"$

$\frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 0.041$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$

$FS_{S1} = 0.672$

ANALYZE WALL 1 (West wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_1 - \frac{(W_1 - 0psf)\cdot 3ft}{30.25ft} = 3049.543\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{3050psf\cdot L_1^2}{8}$ $M_{mid} = 67.778\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_1^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$

#6's @ 6 in. inside face (mid-plate moment) $A_{mph} := 0.884\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 1.156\,in$

$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 25.2\frac{kip\cdot ft}{ft}$

no bars outside face (edge-plate moment) $A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO\ GOOD")$ CheckMPH = "NO GOOD"

Factor of Safety $FS_{MPH1} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH1} = 0.372$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO\ GOOD")$ CheckEPH = "NO GOOD"

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3385.272\,psf$

$$W_1 - \frac{(W_1 - 0psf)\cdot 3ft}{30.25ft} = 3049.543\,psf$$

Thrust present 3 ft from bottom of wall.

$$THRUST := \frac{3050psf \cdot L_2}{2}$$

$$THRUST = 10675 \frac{lb}{ft}$$

$d_1 = 9.125\ in$ $A_g = 1\ ft$

$C_{c1} = 2.5\ in$

Information for CASTR:

$$THRUST = 10.67 \frac{kip}{ft}$$

$$M_{mid} = 67.778 \frac{kip \cdot ft}{ft}$$

```
100 Millw1MPH767.2 M.Parks
110 Wall 1 MPM HGL=767.2
200 1
210 3.00 40.0 0.375
300 12.0 X 12 with 1 bar inside face
400 12.0 12.0
500 1
510 1 0.884 9.125 6.0 6.0
600 10.67 67.8
```

Percent Design Strength Used in Tensile Zone

$$Percent := 285.4\%$$

$$FoS' := \frac{1}{Percent \cdot .9}$$

Note: Strength Reduction Factor (.9) not applied.

$$FoS' = 0.39$$

$$M_{mid} = 67.778 \frac{kip \cdot ft}{ft}$$

CASTR output: The section falls outside the admissible range and does not meet input data reinforcement limit requirement.

From interaction diagram:

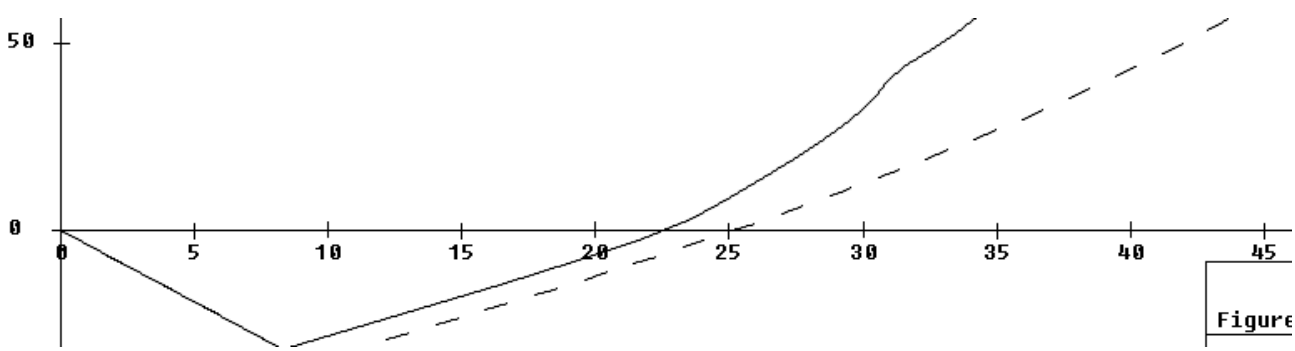
Nominal Strength

$$M_{mph_int} := 28 \frac{kip \cdot ft}{ft}$$

$$FS_{MPH1} := \frac{M_{mph_int}}{M_{mid}}$$

$$FS_{MPH1} = 0.413$$

Below 1.5.



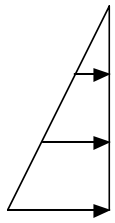
WALL 2, (North wall)

$K_o = 0.562$ $\quad$ $BLANKET = 36\ ft$ $\quad$ $HGL = 9.9\ ft$

$Hwall_2 = 30.25\ ft$ $\quad$ $\gamma_w = 62.4\ pcf$

Wall Loading

Soil or Soil & Water



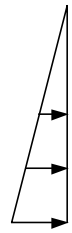
$H_2 := Hwall_2$

$H_2 = 30.25\ ft$

OR

$H_2 = 30.25\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_2 - HEAD$

$H''_3 := Hwall_2 + HGL - HEAD$

$H_3 := if\left(Hwall_2 > BLANKET, H''_3, H'_3\right)$

$H_3 = 38.5687\ ft$

$H'_3 = 38.569\ ft$

$H''_3 = 40.15\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 2039\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_w = 3385.27\ psf$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_2 := if\left(W_s > W_w, W_s, W_w\right)$

$W_2 = 3385\ \frac{lb}{ft^2}$

WALL 2, (North Wall)

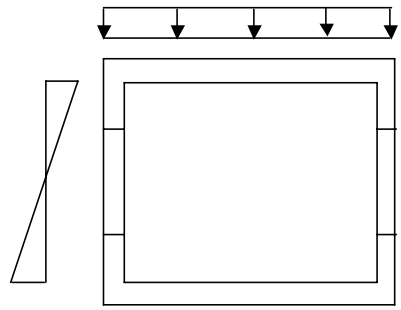
SHEAR

$$THRUST_2 := \frac{W_1 \cdot L_1}{2} \quad THRUST_2 = 22568 \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis (if necessary or applicable)

$$V'_{u2} := W_2 \cdot \frac{L_2}{2}$$

$$V'_{u2} = 11848 \frac{lb}{ft}$$



$$V_{u2} := 2 \frac{V'_{u2}}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2 \right) \quad V_{u2} = 8111 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$D_2 = 10 \, in$$

$$A_g := D_2 \cdot 12 \frac{in}{ft} \quad A_g = 120 \frac{in^2}{ft}$$

EM 1110-2-2104 EQ (5-1) Does not Apply

$$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_2 \right) \right]$$

ACI EQ (11-4)

$$\phi V_{n2} = 10427 \frac{lb}{ft}$$

$$d_2 = 7.25 \, in$$

$$\frac{THRUST_2 \cdot \frac{in^2}{lb}}{A_g} = 188.071$$

$$Check2 := if\left(\phi V_{n2} > 1.5 V_{u2}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

Check2 = "NO GOOD"

Factor of Safety

$$FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$$

$$FS_{S2} = 1.286$$

ANALYZE WALL 2 (North wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\,\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_2 - \frac{(W_2 - 0psf)\cdot 3ft}{30.25ft} = 3049.543\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{3050psf\cdot L_2^{\,2}}{8}$ $M_{mid} = 18.681\,\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_2^{\,2}}{12}$ $M_{end} = 0\,\frac{kip\cdot ft}{ft}$

#4's @ 6 in. inside face (mid-plate moment) $A_{mph} := 0.393\,\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 0.514\,in$

$$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right) \qquad \phi M_{mph} = 11.6\,\frac{kip\cdot ft}{ft}$$

no bars outside face (edge-plate moment) $A_{eph} := 0\,\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right) \qquad \phi M_{eph} = 0.0\,\frac{kip\cdot ft}{ft}$$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO GOOD")$ $CheckMPH = "NO GOOD"$

Factor of Safety $FS_{MPH2} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH2} = 0.622$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO GOOD")$ $CheckEPH = "NO GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3385.272\,psf$

$$W_2 - \frac{(W_2 - 0psf)\cdot 3ft}{30.25ft} = 3049.543\,psf$$

Thrust present 3 ft from bottom of wall.

$$THRUST := \frac{3050psf \cdot L_1}{2}$$ $$THRUST = 20333.333 \frac{lb}{ft}$$

$d_1 = 9.125\,in$ $A_g = 0.833\ ft$

$C_{c1} = 2.5\,in$

Loads from initial screening calculation: $M_u := 17.61\,kip \cdot ft$ $F_y = 40\,ksi$ $f'_c = 3000\,psi$

$Thrust := 19.17\,kip$

Compute Strength and Factor of Safety When Thrust is Included

NOTE: Reassign variables to allow use of the following routine

Steel Modulus of Elasticity: $E_s := 29000\,ksi$

Section Info: Thickness of Section, $D := 10\,in$ $b := 1\,ft$ (1 foot strip)

$A_1 := 0.393\,in^2$ $A_2 := 0\,in^2$ ONLY 1 LAYER OF STEEL PRESENT

$d_1 := d_2$ $d_2 := d_2$ Distance from compression face to bar centerline

$d_1 = 7.25\,in$

$$\beta 1 := if\left[f'_c \le 4ksi, 0.85, if\left[\left(f'_c\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_c}{ksi} - 4\right)\right]\right]$$ $\beta 1 = 0.85$

Assume c=0.2*D $i := 1 .. 2$

Given $c := .2 \cdot D$ $c = 2\,in$ $Thrust = 19.17\,kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_y \\ F_y - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_y \\ -F_y - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & if\ \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_y \end{array}\right] \cdot A_i\right] \ldots + 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $c = 1.341\,in$ $a := if\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right)$ $a = 1.14\,in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s \cdot \varepsilon s_i > F_y, F_y, \text{if}(E_s \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'_c, 0ksi) \cdot A_i$ $\qquad$ $Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.604	ft	-0.013	-40	ksi	0.393	in^2	-15.72	kip	35.37	kip·in
0.604		-0.013	-40		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 35\ kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 155\ kip \cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 15.83\ kip \cdot ft$

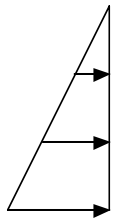
$FS_{MPH2} := \frac{M_n}{M_u}$

$Check := \text{if}(FS_{MPH2} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\qquad$ $FS_{MPH2} = 0.899$ $\qquad$ Check = "NO GOOD"

WALL 3, (North wall) In place wall $K_o = 0.562$ $BLANKET = 36\ ft$ $HGL = 9.9\ ft$

Wall Loading

Soil or Soil & Water $Hwall_3 = 13.5\ ft$



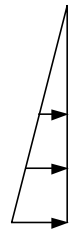
$H_2 := Hwall_3$

$H_2 = 13.5\ ft$

OR

$H_2 = 13.5\ ft$

&



$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_3 - HEAD$

$H''_3 := Hwall_3 + HGL - HEAD$

$H_3 := if\left(Hwall_3 > BLANKET, H''_3, H'_3\right)$

$H_3 = 17.212\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 909.8\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_w = 1511\ psf$

$H'_3 = 17.212\ ft$

$H''_3 = 23.4\ ft$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_{3ext} := if\left(W_s > W_w, W_s, W_w\right)$ $W_{3ext} = 1511\ \frac{lb}{ft^2}$

Sand Loading (Interior Side of Wall)

The wall has sand placed on the other side. The sand reduces the net load acting on the wall.

$\phi_s := 32deg$ $K_{os} := 1 - \sin\left(\phi_s\right)$ $K_{os} = 0.47$

$\gamma_{sand} := 100pcf$

$H_{s3} := Hwall_3$ $H_{s3} = 13.5\ ft$

$W_{int3} := K_{os} \cdot \gamma_{sand} \cdot H_{s3}$ $W_{int3} = 634.609\ psf$ $W_{3ext} = 1510.783\ psf$

$W_3 := W_{3ext} - W_{int3}$ $W_3 = 876.174\ psf$ The net load acting on the wall

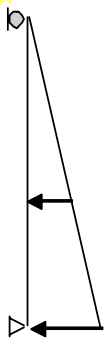
WALL 3, North Wall (In place wall)

SHEAR $L_3 = 22\ ft$ Thrust cannot be used when section spans vertically (no aid from earth loading). Model as a simple beam with increasing uniform load load case. The section spans vertically.

$$THRUST_1 := \frac{W_3 \cdot L_4 \cdot 0}{2} \qquad THRUST_1 = 0 \qquad V_{u1} := \frac{W_3 \cdot Hwall_3}{6} - \frac{W_3 \cdot Hwall_3}{2} \cdot \frac{d_3^{\,2}}{Hwall_3^{\,2}} \qquad V_{u1} = 1947.714\ \frac{lb}{ft}$$

$$V'_{u1} := W_3 \cdot \frac{L_3}{3}$$

$$V'_{u1} = 6425\ \frac{lb}{ft}$$



Elevation View

$$V_{u1} := 2\,\frac{V'_{u1}}{L_3} \cdot \left(\frac{L_2}{2} - \frac{D_2}{2} - d_3\right)$$

disabled

$$V_{u1} = 1948\ \frac{lb}{ft}$$

$D_2 = 0.833\ ft$

$d_1 = 0.76\ ft$

$L_1 = 13.333\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$A_g := D_1 \cdot 12\,\frac{in}{ft} \qquad A_g = 144\,\frac{in^2}{ft} \qquad \frac{L_1}{d_1} = 17.534$$

EM 1110-2-2104 EQ (5-1) Does not Apply

$$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1\right)\right]$$

ACI EQ (11-4)

$$\phi V_{n1} = 11995\ \frac{lb}{ft}$$

$$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

Check1 = "OKAY"

Factor of Safety

$$FS_{S3} := \frac{\phi V_{n1}}{V_{u1}}$$

$$FS_{S3} = 6.159$$

ANALYZE WALL 3 FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the vertical reinforcement carries all of the lateral loads $b := 12\,\frac{in}{ft}$

Evaluate the wall where the moment is greatest.

Wall 3 height is 13.5 ft ::: Wall 1 height is 30.25 ft::::Wall 2 height is 30.25 ft
Wall 4 height is 13.5 ft:::: Wall 5 height is 13.5 ft $Hwall_3 = 13.5\ ft$

The maximum moment occurs at $0.5774 \cdot Hwall_3 = 7.795\ ft$ from top of wall.

or $Hwall_3 - 7.79ft = 5.71\ ft$ from bottom of wall.

The maximum moment $M_{max} := \frac{0.128}{2} W_3 \cdot Hwall_3^{\ 2}$ $M_{max} = 10.22\,\frac{kip \cdot ft}{ft}$

The end-span positive moment $M_{end} := \frac{0psf \cdot L_1^{\ 2}}{12}$ $M_{end} = 0\,\frac{kip \cdot ft}{ft}$ no negative reinforcement

#6s @ 8 in. inside face (mid-plate moment) $A_{mph} := 0.663\,\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mph} = 0.867\ in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_3 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 21.7\,\frac{kip \cdot ft}{ft}$

no bars in outside face (edge-plate moment) $A_{eph} := 0\,\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{eph} = 0\ in$

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 0.0\,\frac{kip \cdot ft}{ft}$

$CheckMPH := if\left(\phi M_{mph} > 1.5 \cdot M_{max}, "OKAY", "NO\ GOOD"\right)$ $CheckMPH = "OKAY"$

Factor of Safety $FS_{MPH3} := \frac{\phi M_{mph}}{M_{max}}$ $FS_{MPH3} = 2.123$

$CheckEPH := if\left(\phi M_{eph} > 1.5 \cdot M_{end}, "OKAY", "NO\ GOOD"\right)$ $CheckEPH = "NO\ GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

$W_3 = 876.174\ psf$

NOTE: Thrust cannot be used to assist Wall 3 section strength because the reinforcement spans vertically. The end-of span moment is 0 and thus the FS_{EPH} is not relevant.

WALL 4, East Wall (In place wall)

$W_3 = 876 \frac{lb}{ft^2}$

$L_4 = 13\ ft$

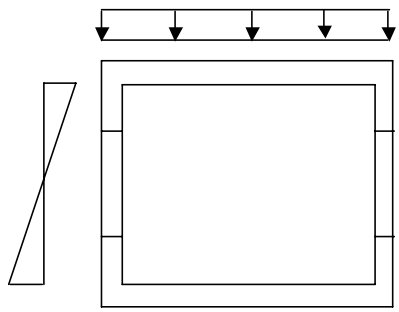
SHEAR

$THRUST_1 := \frac{W_3 \cdot L_3}{2}$ $\quad THRUST_1 = 9638 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_3 \cdot \frac{L_4}{2}$

$V'_{u1} = 5695 \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_4} \cdot \left(\frac{L_4}{2} - \frac{D_3}{2} - d_4 \right)$ $\quad V_{u1} = 4554 \frac{lb}{ft}$

$D_2 = 0.833\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$d_1 = 0.76\ ft$

$L_1 = 13.333\ ft$

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 144 \frac{in^2}{ft}$

$\frac{L_1}{d_1} = 17.534$ EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_4 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 12397 \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\quad Check1 = \text{"OKAY"}$

Factor of Safety

$FS_{S4} := \frac{\phi V_{n1}}{V_{u1}}$ $\quad FS_{S4} = 2.722$

ANALYZE WALL 4 (East wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom $W_3 = 876.174\,psf$

$$W_3 - \frac{(W_3 - 0psf)\cdot 3ft}{13.5ft} = 681.468\,psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{682psf\cdot L_4^2}{8}$ $M_{mid} = 14.407\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_2^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$ no negative reinforcement

0.75" @ 7 in. inside face (mid-plate moment) Square bars $A_{mph} := 0.964\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 1.26\,in$

$$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_4 - \frac{a_{mph}}{2}\right) \qquad \phi M_{mph} = 27.3\frac{kip\cdot ft}{ft}$$

no bars outside face (edge-plate moment) $A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\,in$

$$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right) \qquad \phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO GOOD")$ $CheckMPH = "OKAY"$

Factor of Safety $FS_{MPH4} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH4} = 1.895$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO GOOD")$ $CheckEPH = "NO GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Loads from initial screening calculation: $M_u := 14.41\,kip\cdot ft$ $F_y = 40\,ksi$ $f'_c = 3000\,psi$

$Thrust := 9.6kip$

Compute Strength and Factor of Safety When Thrust is Included

NOTE: Reassign variables to allow use of the following routine

Steel Modulus of Elasticity: $E_s := 29000ksi$

Section Info: Thickness of Section, $D := 12in$ $b := 1ft$ (1 foot strip)

$A_1 := 0.964in^2$ $A_2 := 0in^2$ ONLY 1 LAYER OF STEEL PRESENT

$d_1 := d_4$ $d_2 := d_4$ Distance from compression face to bar centerline

$d_1 = 9.125\,in$

$$\beta 1 := if\left[f'_c \le 4ksi, 0.85, if\left[\left(f'_c\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_c}{ksi} - 4\right)\right]\right]$$

$\beta 1 = 0.85$

Assume c=0.2*D $i := 1..2$

Given $c := .2\cdot D$ $c = 2.4\,in$ $Thrust = 9.6\,kip$

$$Thrust = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_y \\ +-if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \\ F_y - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_y \\ -F_y - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_y \end{array}\right]\cdot A_i\right] \ldots$$

$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := Find(c)$ $c = 1.852\,in$ $a := if\left(\beta 1\cdot c > D, D, \beta 1\cdot c\right)$ $a = 1.574\,in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(E_s \cdot \varepsilon s_i > F_y, F_y, \text{if}(E_s \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, E_s \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'_c, 0ksi) \cdot A_i$ $\qquad$ $Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.76	ft	-0.012	-40	ksi	0.964	in^2	-38.56	kip	120.5	kip·in
0.76		-0.012	-40		0		0		0	

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 120\ kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 251\ kip \cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 30.96\ kip \cdot ft$

$FS_{MPH4} := \frac{M_n}{M_u}$

$Check := \text{if}(FS_{MPH4} > 1.5, \text{"OKAY"}, \text{"NO GOOD"})$ $\qquad$ $FS_{MPH4} = 2.149$ $\qquad$ Check = "OKAY"

WALL 5, South Wall (In place wall)

$W_3 = 876 \frac{lb}{ft^2}$

$L_4 = 13\ ft$

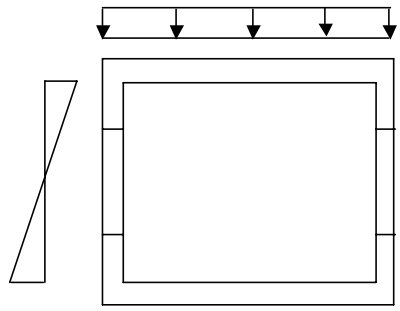
SHEAR

$THRUST_1 := \frac{W_3 \cdot L_4}{2}$ $\quad THRUST_1 = 5695 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_3 \cdot \frac{L_5}{2}$

$V'_{u1} = 2884 \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_4} \cdot \left(\frac{L_4}{2} - \frac{D_3}{2} - d_4 \right)$ $\quad V_{u1} = 2306 \frac{lb}{ft}$

$D_2 = 0.833\ ft$

$d_1 = 0.76\ ft$

$L_1 = 13.333\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $\quad A_g = 144 \frac{in^2}{ft}$

$\frac{L_1}{d_1} = 17.534$ EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_4 \right) \right]$ ACI EQ (11-4)

$\phi V_{n1} = 12232 \frac{lb}{ft}$

$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO\ GOOD"\right)$

Check1 = "OKAY"

Factor of Safety

$FS_{S5} := \frac{\phi V_{n1}}{V_{u1}}$ $\quad FS_{S5} = 5.304$

ANALYZE WALL 5 (South wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\frac{in}{ft}$

Analyze as a simple span. The reinforcement is not developed in the orthaganol walls.

Evaluate the wall 3 feet from bottom

$$W_3 - \frac{(W_3 - 0psf)\cdot 3ft}{13.5ft} = 681.468\ psf$$

Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{682psf\cdot L_5^2}{8}$ $M_{mid} = 3.695\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{0psf\cdot L_1^2}{12}$ $M_{end} = 0\frac{kip\cdot ft}{ft}$

5/8" @ 7 in. inside face (mid-plate moment) Square bars

$A_{mph} := 0.669\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 0.875\ in$

$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_5 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 15.1\frac{kip\cdot ft}{ft}$

no bars outside face (edge-plate moment)

$A_{eph} := 0\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 0\ in$

$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 0.0\frac{kip\cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO\ GOOD")$ CheckMPH = "OKAY"

Factor of Safety $FS_{MPH5} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH5} = 4.074$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO\ GOOD")$ CheckEPH = "NO GOOD"

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 0$

Uplift on Structure

Refer to Figure 2 for the identified areas concerning uplift calculations

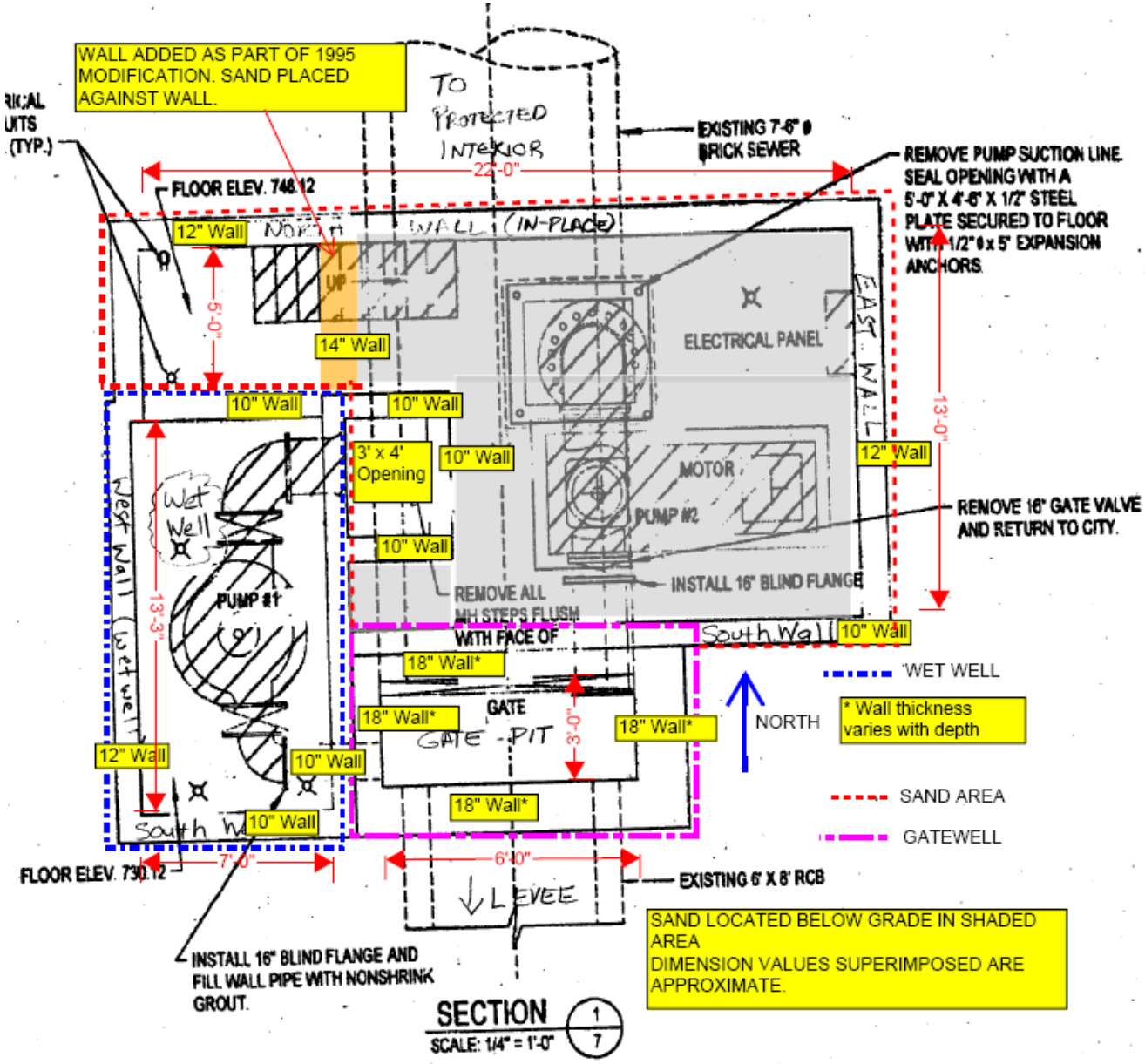


Figure 2. Section cut between elevation 748.12 and 755, Mill St Pump Station after 1995 modifications (DR&G Engineers).

Wet well weight.

The wetwell weight includes all of the following within the outlined area in Figure 2:

1. Wetwell walls
2. Top slab within the identified wetwell area (Note that the top slab extends beyond the wetwell footprint).
3. Concrete wall above grade within the wetwell area.
4. Wetwell base slab
5. Estimated weight of steel used to rehabilitate the wetwell for strength deficiencies.

The above grade concrete wall size and top slab area is not shown on the DR&G Engineers drawings. The values used in calculations are conservative (low) estimate values when used to calculate uplift.

Uplift on Wetwell Structure

The 1995 DR&G drawings show sand placed in the pump station.

Weight of wetwell.The equipment weight was not included.

Top Slab

$Top := 15in \cdot (7ft + 12in + 10in) \cdot (13.33ft + 12in + 10in)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 25.114\,kip$

Walls

$West := 30.25ft \cdot 13.33ft \cdot 1ft$ $\quad Wwest := West \cdot 150pcf$ $\quad Wwest = 60.485\,kip$

$South := 30.25ft \cdot 7ft \cdot 10in$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 26.469\,kip$

$East := West$ $\quad Weast := Wwest$ $\quad Weast = 60.485\,kip$

$North := Hwall_2 \cdot L_2 \cdot D_2$ $\quad Wnorth := North \cdot 150pcf$ $\quad Wnorth = 26.469\,kip$

Base Slab

$Base := 10 \cdot in \cdot (7 \cdot ft + 1 \cdot ft + 10 \cdot in) \cdot (13.33 \cdot ft + 10 \cdot in + 10 \cdot in) + 14in \cdot 5ft \cdot 4.5ft$ $\quad 5ft \cdot 4.5ft = 22.5\,ft^2$

$Base = 136.642\,ft^3$ $\quad Wbase := Base \cdot 150pcf$ $\quad Wbase = 20.496\,kip$

$Basearea := (7 \cdot ft + 1 \cdot ft + 10 \cdot in) \cdot (13.33 \cdot ft + 10 \cdot in + 10 \cdot in)$ $\quad Basearea = 132.471\,ft^2$

South Wall above grade

$hsouth_{ag} := 2ft$

$South_{ag} := hsouth_{ag} \cdot 12in \cdot (7ft + 12in + 10in)$ $\quad Wsouth_{ag} := South_{ag} \cdot 150pcf$ $\quad Wsouth_{ag} = 2650\,lb$

West Wall above Grade

$hwest_{ag} := 2ft$

$West_{ag} := hwest_{ag} \cdot (3ft + 0in) \cdot 12in$ $\quad Wwest_{ag} := West_{ag} \cdot 150pcf$ $\quad Wwest_{ag} = 0.9\,kip$

$Wsteel := 12kip$ Estimated weight of steel placed in wetwell for strength rehabilitation

Wetwell Weight

$Wwetwell := Wtop + Wwest + Wsouth + Weast + Wnorth + Wbase \ldots + Wsouth_{ag} + Wwest_{ag} + Wsteel$ $\quad Wwetwell = 235.068\,kip$

Wetwell Structure Factor of Safety for Flotation

Weight of water that is present during the event

$Wellarea := 81ft^2$ EC-GD documentation

Bottom of wetwell base elevation

$Elevwetwellbot := 730.12ft - 10in - 14in$

$HEAD := 3ft$

$Elevwetwellbot = 728.12\ ft$

bottom of blanket at 725

$Water := HEAD \cdot Wellarea \cdot 62.4pcf$ $Water = 15.163\ kip$

$heel := 0ft$ $heel = 0$ The wet well does not have heels

$upliftarea := Basearea$ $upliftarea = 132.471\ ft^2$

$H := ELEV_1 - Elevwetwellbot$ $H = 32.88\ ft$ $ELEV_1 = 761\ ft$ Grade elevation

Weight of soil on the wetwell heels

$volsoil := (1ft - 1ft) \cdot heel + (1ft - 1ft) \cdot heel$ $volsoil = 0\ ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 0\ kip$ $volsoil \cdot \gamma_w = 0\ kip$

Head, if structural foundation in blanket. $HGL = 9.9\ ft$

$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot upliftarea \cdot \gamma_w$ $U_1 = 346.534\ kips$

$BLANKET = 36\ ft$
$HGL = 9.9\ ft$

Head, if structural foundation extends through blanket.

$H = 32.88\ ft$

$U_2 := \left[(H + HGL) \cdot (upliftarea) \cdot \gamma_w\right]$ $U_2 = 353.626\ kips$

$Uplift_{wet} := if(H > BLANKET, U_2, U_1)$ $Uplift_{wet} = 346.534\ kips$ $\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 41.922\ ft$

The uplift force acting on the wetwell structure $Uplift_{wet} = 346.534\ kips$

$Stability := \frac{Wwetwell + Water + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{wet} - volsoil \cdot \gamma_w}$ $Stability = 0.722$ $(H + HGL) = 42.78\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $StabilityCheck = "Not\ Okay"$

Determine the amount of additional weight required to achieve FS = 1.1 of the wetwell structure

$Wreqdwetwell := Uplift_{wet} \cdot 1.1 - \left[Wwetwell + Water + volsoil \cdot (\gamma - \gamma_w)\right]$ $Wreqdwetwell = 130.956\ kip$

$Water = 15.163\ kip$

Structure containing sand.

$Sa := (22ft - 7ft - 14in)\cdot(5ft + 0in)\cdot(760ft - 748.12ft) \ldots + (10ft - 4ft)\cdot(7ft + 2in)\cdot(760ft - 748.12ft) \ldots + (3ft + 10in)\cdot(1ft + 7in)\cdot(760ft - 748.12ft)$ $\quad Sa = 1404.645\ ft^3$

$Wsa := 100pcf\cdot Sa$ $\quad Wsa = 140.465\ kip$

NOTE: See "Recommendations" for more details as to the considerations that must be taken into account during design if Wsa is to be considered as an aid against uplift.

Top Slab

$Top := 1.25ft\cdot(22ft - 7ft - 14in + 12in)\cdot(13ft + 10in + 12in)$ $\quad Wtop := Top\cdot 150pcf$ $\quad Wtop = 41.255\ kip$

East Wall

$heast := (13ft + 6in)$ $\quad heast = 13.5\ ft$

$East := heast\cdot(13ft + 0in)\cdot 12in$ $\quad Weast := East\cdot 150pcf$ $\quad Weast = 26.325\ kip$

South Wall

$hsouth := heast$ $\quad hsouth = 13.5\ ft$

$South := hsouth\cdot(6ft + 7in + 12in)\cdot 10in$ $\quad Wsouth := South\cdot 150pcf$ $\quad Wsouth = 12.797\ kip$

North Wall

$hnorth := heast$ $\quad hnorth = 13.5\ ft$

$North := hnorth\cdot(22ft + 2ft)\cdot 13in$ $\quad Wnorth := North\cdot 150pcf$ $\quad Wnorth = 52.65\ kip$

West Wall

$hwest := heast$

$West := hwest\cdot(5ft)\cdot 12in$ $\quad Wwest := West\cdot 150pcf$ $\quad Wwest = 10.125\ kip$

Interior Wall

$hint := 760ft - 748.12ft$ $\quad hint = 11.88\ ft$

$Int := hint\cdot 14in\cdot 5ft$ $\quad Wint := Int\cdot 150pcf$ $\quad Wint = 10.395\ kip$

Base Slab

$Wbase := [(6in)\cdot(22ft + 0in)\cdot 5ft + (7ft + 2in)\cdot(10ft + 4in + 10in + 3ft)\cdot 6in]\cdot 150pcf$ $\quad Wbase = 15.865\ kip$

$Wbldg := 3kip$ Estimated weight of steel building enclosure, (conservative)

Structure Containing Sand Weight

$Wsand := Wsa + Wtop + Weast + Wsouth + Wnorth + Wbase + Wint \ldots + Wbldg + Wwest$ $\quad Wsand = 312.876\ kip$

Structure containing sand, con't

Uplift On Sand Structure

$Asand := (11 \cdot ft + 11 \cdot in) \cdot (6 \cdot ft + 11 \cdot in + 20 \cdot in)$ $\quad Asand = 102.285\ ft^2$ Area uplift acts on

$ELEVsand := 747.6 ft$ Elevation at bottom of sand structure base slab

Bottom of blanket elev = 729.3

Bottom of wetwell elev = 725

$H := ELEV_1 - ELEVsand$ $\quad H = 13.4\ ft$ $\quad ELEV_1 = 761\ ft$

Head, if structural foundation in blanket. $\quad HGL = 9.9\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot Asand \cdot \gamma_w \qquad U_1 = 109.046\ kips$$

$BLANKET = 36\ ft$
$HGL = 9.9\ ft$

$H = 13.4\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (Asand) \cdot \gamma_w\right] \qquad U_2 = 148.714\ kips$$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 17.085\ ft$$

$$Uplift_{sand} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the sand structure $\quad Uplift_{sand} = 109.046\ kips$

Sand Structure Factor of Safety for Flotation

<u>Weight of soil on the sand structure heels</u>

$volsoil := (1ft - 1ft) \cdot heel + (1ft - 1ft) \cdot heel$ $\quad volsoil = 0\ ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 0\ kip$ $\quad volsoil \cdot \gamma_w = 0\ kip$

$$Stability := \frac{Wsand + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{sand} - volsoil \cdot \gamma_w}$$

$Stability = 2.869$ $\quad (H + HGL) = 23.3\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $\quad StabilityCheck = "Okay"$

Amount of weight available to assist the wetwell flotation .

$$Wsandforwetwell := Wsand - \frac{1.10}{Stability} \cdot \left[Wsand + volsoil \cdot (\gamma - \gamma_w)\right] \qquad Wsandforwetwell = 192.925\ kip$$

Determine if the pump station (without gatewell) meets the required factor of safety for uplift

To satisfy the overall factor of safety against flotation, the amount of weight available in the sand structure (Wsandforwetwell) must be greater than the amount of weight required such that the wetwell structure meets the required factor of safety (Wreqdwetwell).

$PumpStationmeetsuplift := if(Wsandforwetwell \geq Wreqdwetwell, "okay", "NO GOOD")$

$PumpStationmeetsuplift = "okay"$

Wall Shear Strength

This is assuming that the force can be transfered in shear at the wetwell and sand structure interface. Determine the length of wall required to transfer the shear force.

$Wsandforwetwell = 192.925\ kip$ Amount of sand weight available to increase the wetwell FS for flotation

$Wreqdwetwell = 130.956\ kip$ This is the amount of shear that has to be transferred in the walls

$t_{wall} := 10in$ Wall thickness

$d_e := 7.25in$ Effective depth of the wall reinforcement

The south and east wetwell walls are attached to the gatewell. Both of these walls are 10 inches in thickness.

$f'_c = 3\ ksi$ Concrete compressive strength

$$hreqd := \frac{Wreqdwetwell}{0.85 \cdot 2 \cdot \sqrt{f'_c} \cdot d_e \cdot \frac{\sqrt{lb}}{in}}$$

$Hwall_1 = 30.25\ ft$

$hreqd = 16.166\ ft$

$Availwallength := 20ft$

Length of a wall that is available to transfer the shear force. Based on the west wetwall height.

$Wallokforshear := if[hreqd \le (Availwallength), "okay", "NO\ GOOD"]$

$Wallokforshear = "okay"$

Consider the gatewell attached to the station.

$Wgate := [[(6ft \cdot 9ft) - (3ft \cdot 6ft)] \cdot (764ft - 729ft) + 10in \cdot (9ft \cdot 6ft)] \cdot 150pcf - (5ft \cdot 6ft \cdot 10in) \cdot 150pcf$

$Wgate = 192\ kip$

$Agate := (5.5ft + 0in) \cdot (10ft + 0in)$ $Agate = 55\ ft^2$ Area that uplift is acting on

$H := ELEV_1 - (729ft)$ $H = 32\ ft$ $ELEV_1 = 761\ ft$

Bottom of blanket elev = 729.3

Bottom of wetwell elev = 725

Head, if structural foundation in blanket. $HGL = 9.9\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot Agate \cdot \gamma_w$$

$U_1 = 140.026\ kips$

$BLANKET = 36\ ft$
$HGL = 9.9\ ft$

$H = 32\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (Agate) \cdot \gamma_w\right]$$

$U_2 = 143.801\ kips$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 40.8\ ft$$

$$Uplift_{gate} := if\left(H > BLANKET, U_2, U_1\right)$$

The uplift force acting on the wetwell structure $Uplift_{gate} = 140.026\ kips$

$Stability_{gate} := \frac{Wgate}{Uplift_{gate}}$ $Stability_{gate} = 1.371$ $(H + HGL) = 41.9\ ft$

ASSUMES THE GATEWELL HAS NO HEELS (CONSERVATIVE)

Amount of weight required to achieve FS=1.10

$Wreqdgate := (Uplift_{gate} \cdot 1.10) - Wgate$ $Wreqdgate = -37.972\ kip$

Sand placement may be necessary if the uplift force acting on the wetwell cannot be transferred entirely to the the sand structure base slab. The capability of whether the transfer of uplift force can occur from the wall to the base could not be established by a field visit. The gatewell will assist in resisting the uplift force. This will be achieved by placing a slab in the existing gatewell and placing sand above the installed slab. From calculations, the gatewell without sand will provide 37.9 kips of resistance (adjusted for uplift on the gatewell base slab). When hsand = 23 feet, Wgatesand = 73.4 kip. (Wgatesand does not include the gatewell weight.) 37.9 + 73.4 = 111.3 kips of resistance. From earlier calculations, it was established that 131 kips of resistance was necessary. 131 - 111.3 = 19.7 or 20 kips of additional resistance. This resistance can come from the wall weight enclosing the sand. In summary, this method is a feasible means of acquiring the necessary weight and effective transfer of uplfit forces.

$hsand := 23ft$

$Wgatesand := (3.833ft \cdot 8.33ft \cdot hsand) \cdot 100pcf$ $Wgatesand = 73.436\ kip$

Amount of weight left over to aid pump station

$Wgateavail := Wgatesand - Wreqdgate$ $Wgateavail = 111.408\ kip$

$Wsandforwetwell = 192.925\ kip$ $Wreqdwetwell = 130.956\ kip$

Weight required such that the entire pump station (with gatewell) meets FS = 1.1 for uplift

$Wreqd := Wreqdwetwell - Wsandforwetwell$ $Wreqd = -61.969\ kip$

The addditional weight placed in the gatewell (sand) must be equal to or greater than Wreqd

$PumpStationmeetsupliftwithgatesand := if(Wgateavail \geq Wreqd, "okay", "NO\ GOOD")$

$PumpStationmeetsupliftwithgatesand = "okay"$

Wet Well BASE SLAB ANALYSIS

$W_{water} := 15.16kip$ WEIGHT OF WATER

$BaseSlab := upliftarea$ $BaseSlab = 132.471\ ft^2$ $Uplift_{wet} = 346.534\ kip$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$

$W' := if\left(Stability < 1.0, \frac{Uplift_{wet}}{BaseSlab}, \frac{Wwetwell}{BaseSlab}\right)$ $W' = 1774.491\ psf$

$W' = 1774.491\ psf$

$\frac{Wwetwell + W_{water}}{BaseSlab} = 1888.932\ psf$

$\frac{Uplift_{wet}}{BaseSlab} = 2615.933\ psf$

$\frac{Wwetwell + volsoil \cdot (\gamma - \gamma_w)}{BaseSlab} = 1774.491\ psf$

$W' := 2616psf$

$Slab_t := 10in$ $L_{strip} := 8ft$ $Wall_t := D_1$ $Slab_d := d_f$ effective depth of slab

Analyze slab with one-way action

$Slab_d = 6.625\ in$

$D_1 = 12\ in$

$L := 7ft$

$V'_{ub} := \frac{W' \cdot L}{2}$ $V'_{ub} = 9156\ \frac{lb}{ft}$

$d_f = 6.625\ in$

$D_f = 0.833\ ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}} \cdot \left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 6748\ \frac{lb}{ft}$

$A_g := Slab_t \cdot 12\ \frac{in}{ft}$ $A_g = 120\ \frac{in^2}{ft}$

$Water = 15163.2\ lb$

$$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f \right) \right]$$

ACI EQ (11-4)

$$\phi V_{nb} = 8709 \frac{lb}{ft}$$

$$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, "OKAY", "NO GOOD"\right)$$

CheckBase = "NO GOOD"

Factor of Safety

$$FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$$

$$FS_{Sb} = 1.291$$

Slab Bending

Assume simple span moment (conservative). One-way action, investigate the short span moment capacity.

$$M_b := \frac{L^2 \cdot W'}{8} \qquad M_b = 16.023 \frac{kip \cdot ft}{ft}$$

#6's @ 12 in.

$$A_{mpl} := 0.442 \frac{in^2}{ft} \quad b := 12 \frac{in}{ft} \quad a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{mpl} = 0.578\ in$$

$$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_f - \frac{a_{mpl}}{2} \right) \qquad \phi M_{mpl} = 9.3 \frac{kip \cdot ft}{ft}$$

$$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot M_b, "OKAY", "NO GOOD"\right)$$

CheckMP = "NO GOOD"

Factor of Safety

$$FS_{Mb} := \frac{\phi M_{mpl}}{M_b}$$

$$FS_{Mb} = 0.583$$

Again, it is unknown what reinforcement exists in the wetwell slab.

FACTORS OF SAFETY SUMMARY

Five walls and the base slab were evaluated for strength. A summary of the results are below. The reliability analysis calculations are in separate MathCad files.

Analysis indicates that the wetwell walls are unreliable. Based on the Wall 2 calculations, the East and South wet well walls are also considered unreliable. The East and South wet well walls have the same span length, thickness and reinforcement as the North wet well wall (Wall 2) and are exposed to lateral loading equal to the lateral loading present on Wall 2.

The available drawings do not indicate reinforcement present in the wet well base slab. Assuming that #6 @ 12" are present in the base slab, the base slab factor of safety <1.0. Based on these results, the base slab is also considered unreliable.

WALL 1 (WEST WETWELL WALL)

BAD

Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S1} = 0.672$

Moment

$FS_{MPH1} = 0.413$ Mid-plate moment

$FS_{EPH1} =$ ▮ Edge-of-plate moment (not applicable)

REHABILITATION OF WALL 1 WILL BE NECESSARY.

WALL 2 (NORTH WETWELL WALL)

BAD

Wall 2 analyzed as a 2D model using RAM Advanse s/w. Wall 2 has intermediate supports.

Shear

$FS_{S2} = 1.286$ Reliability analysis indicates the wall being reliable for shear(Pf<0.0001)

Moment

$FS_{MPV2} =$ ▮ not applicable

$FS_{EPV2} =$ ▮ not applicable

$FS_{MPH2} = 0.899$ Reliability analysis indicates that the wall is uneliable for moment, Pf= 34.91%

$FS_{EPH2} =$ ▮ not applicable

REHABILITATION OF WALL 2 WILL BE NECESSARY.

WALL 3 (IN PLACE NORTH WALL) GOOD

Wall 3 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S3} = 6.159$

Moment

$FS_{MPH3} = 2.123$ Reliability analysis, Pf = 0.0154%

$FS_{EPH3} =$ [redacted]

SAND IS PLACED WITHIN THE INTERIOR AREAS OF WALL 3, ANALYSIS INDICATES THAT REHABILITATION IS NOT NECESSARY.

WALL 4 (IN PLACE EAST WALL) GOOD

Wall 4 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S4} = 2.722$

Moment

$FS_{MPH4} = 2.149$ Reliability analysis indicates that wall 4 is reliable, Pf ~ 0

$FS_{EPH3} =$ [redacted]

SAND IS PLACED WITHIN THE INTERIOR AREAS OF WALL 4, ANALYSIS INDICATES THAT REHABILITATION IS NOT NECESSARY.

WALL 5 (IN PLACE SOUTH WALL) GOOD

Wall 5 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S5} = 5.304$

Moment

$FS_{MPH5} = 4.074$

$FS_{EPH3} =$ [redacted]

SAND IS PLACED WITHIN THE INTERIOR AREAS OF WALL 5, ANALYSIS INDICATES THAT REHABILITATION IS NOT NECESSARY.

BASE SLAB STATUS: SUSPECT. The base slab reinforcement is unknown.

Shear

$FS_{Sb} = 1.291$

Moment

$FS_{Mb} = 0.583$ Analyzed as a simple span, one-way action.

STABILITY

GOOD - SEE NOTE

NOTE: The overall pump station stability is adequate (FS=1.10) given adequate transfer of the uplift force from the wetwell walls to the gatewell and "sand structure".

SEE NEXT PAGE FOR THE RECOMMENDATIONS

RECOMMENDATIONS

The existing wetwell walls and base slab are inadequate for the **HGL=767.2** event loading. The existing structure is also inadequate for uplift stability for the **HGL=770.9** event loading. To ensure reliability of the structure during the event, the following actions are recommended:

1. Strengthen the wetwell walls and base slab using the design similar to the Strong Avenue pump station rehabilitation. This design is essentially a braced excavation design using uprights and horizontal beam bracing placed inside the wet well. The available drawings indicate that this method of rehabilitation is constructable at the Mill Street pump station. To facilitate installation of the steel, the above grade building structure, pumps and associated equipment will have to be removed temporarily. The existing at grade top slab will have to be removed (at least partially) in order to install the steel structural members. It is advised that temporary wetwell wall bracing be provided prior to removing the top slab. The Strong Ave fix is shown in "Strong Ave 273+41fix.mcd" file. After installling the steel structural members, replace the top slab, reinstall equipment and building enclosure structure.

2. Preliminary analysis indicates that the W-section uprights should not be spaced more than 5 ft on center such that the wet well walls are not overloaded for the **HGL=770.9** event. The final spacing of the uprights will be determined during design.

3. The W-section uprights shall be supported at the top and bottom as a mimimum by horizontal beams or struts.

4. To reduce the required W-section size, intermediate struts are permissible.

5. The design shall effectively transfer the earth loading acting on a wetwell wall to a wall that is also earth loaded.

6. The portion of the pump station containing sand has enough weight to sufficiently offset the total uplift forces for the HGL = 770.9 event. <u>However</u>, the wetwell portion of the pump station structure by itself has a factor of safety against uplift equal to 0.72. To achieve the minimum uplift factor of safety of the wetwell, 131 kips of weight is necessary. The portion of the pump station containing sand has 192 kips of weight available. In order for the sand structure to provide support against uplift forces acting on the wetwell base slab, the uplift force has to be transfered primarily through the wetwell walls as shear. The shear force will be transferred to the attached gatewell walls and north "in-place" wall. As part of the design, the effectiveness of the existing anchorage of the wetwell walls to the gatewell and north "in-place" wall should be checked. If the load cannot be easily transferred to the sand structure, consider placing sand in the gatewell structure. (This methodology is recommended for the 12th Street pump station.) To place sand in the gatewell, install floor in gatewell and fill with sand.. <u>The gatewell slide gate is no longer used to maintain pump station operations</u>. Place the concrete floor above the conduit opening such that it will adequately support sand placed on it. To ensure adequate transfer of the uplift loads from the gatewell structure to the remaining pump station, anchor the gatewell walls to the pump station wetwell walls.

7. OBTAIN AND VERIFY ALL DIMENSIONS PRIOR TO CREATION OF THE FINAL DESIGN. OBTAIN CONCRETE SAMPLES OF THE WET WELL WALLS TO DETERMINE THE ACTUAL CONCRETE STRENGTH PRIOR TO FINAL DESIGN.

The Strong Avenue pump station fix referenced earlier is attached as Exhibit 1.

US Army Corps of Engineers

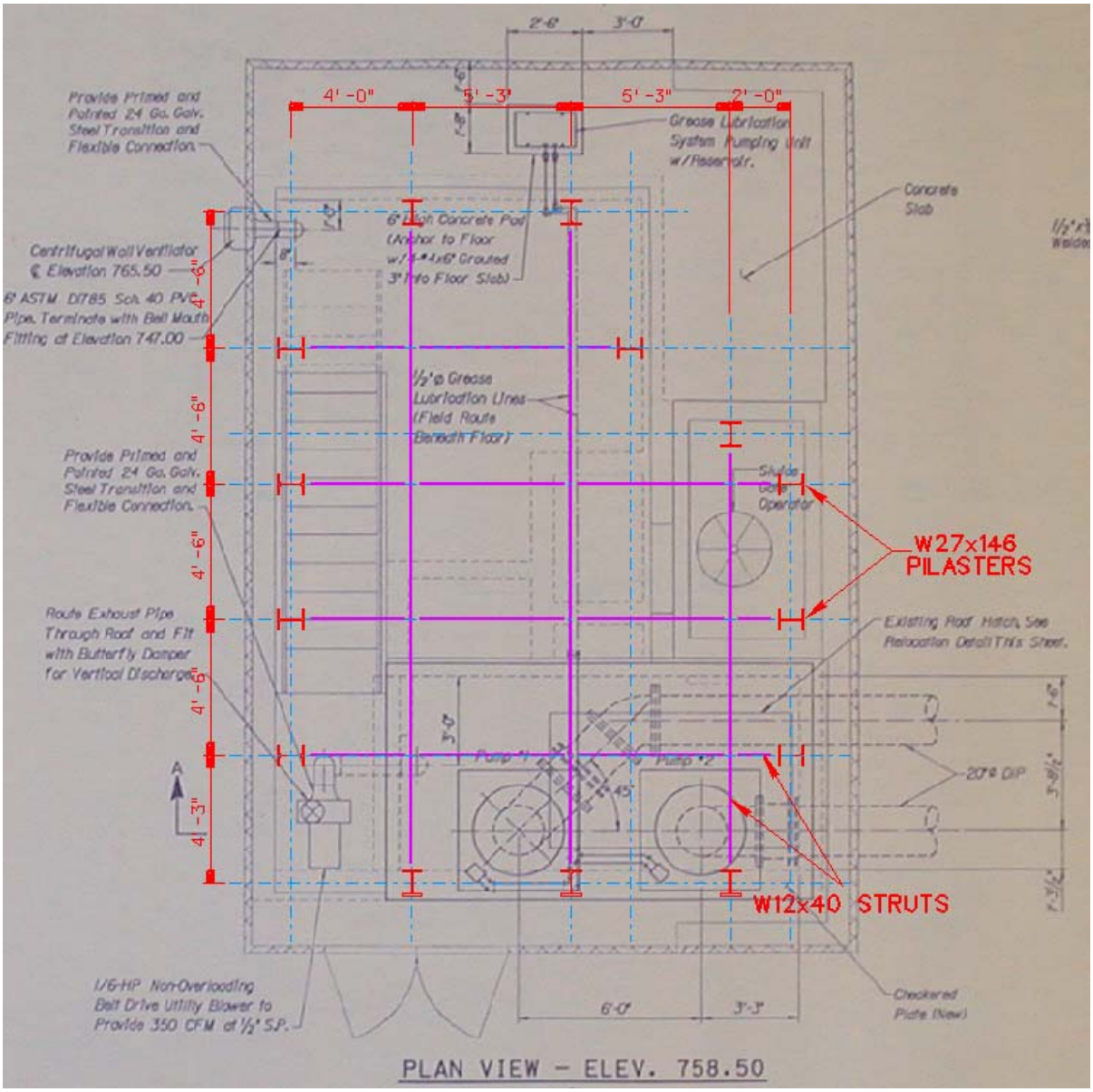
EXHIBIT 1
Argentine Flood Unit
Pump Station Analysis for Strong Ave. Sta. 273+41
Strong Ave. and Kaw River 4-14-1916 Sheet 1 of 2
***Mechanical Plan and Sections*, DR&G As Built Drawings, 1-17-94**

Foundation Modification with Water at top of Protection (Existing Conditions)

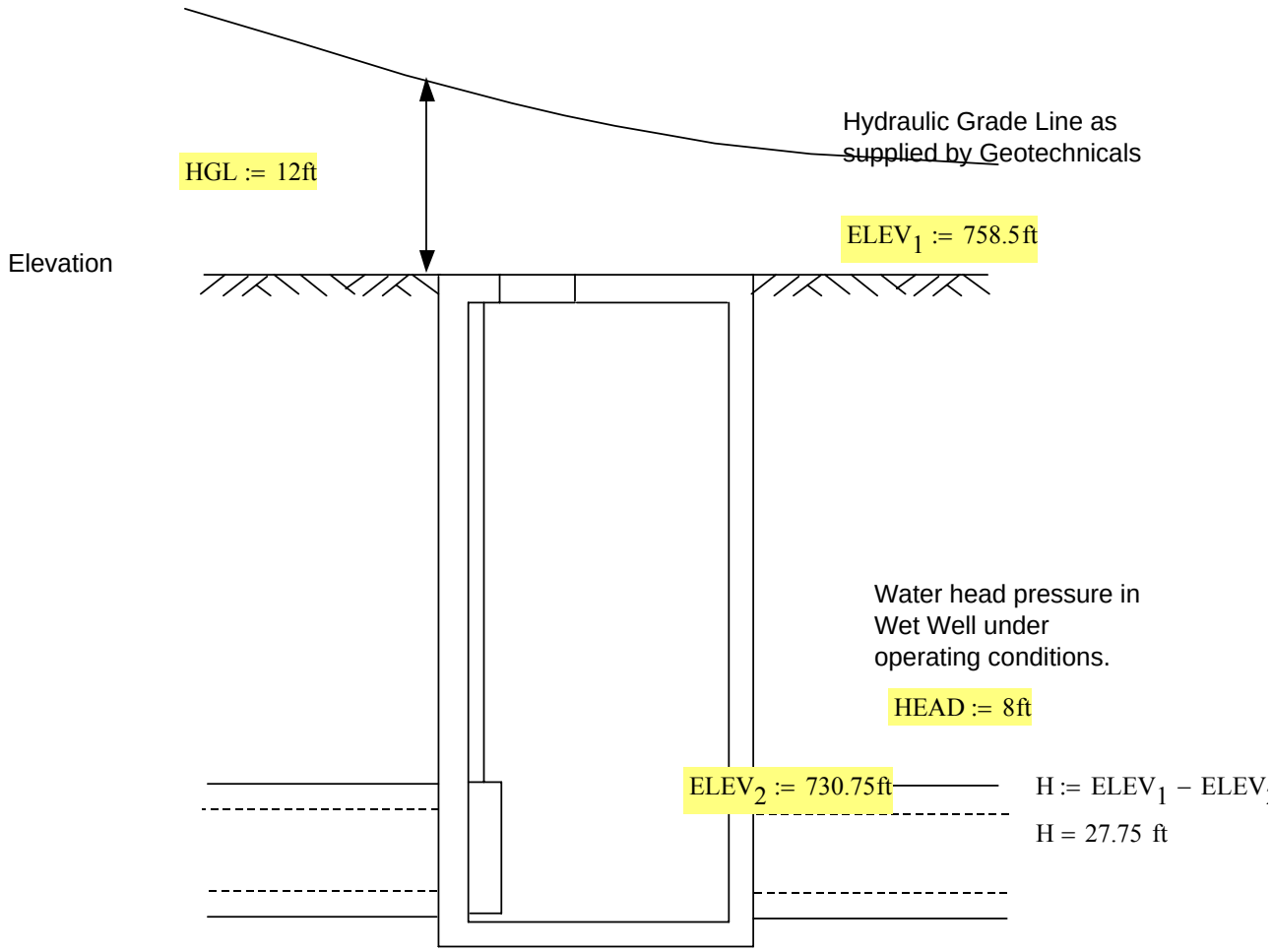
Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $pcf := \frac{lb}{ft^3}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $kips := 1000lb$ $klf := \frac{kips}{ft}$

A Series of Steel W-Section pilaters spaced approximately 5 ft on center with strut braces is purposed to address the foundation strength issues at the Strong Avenue pump station.



Properties



Assumptions

- Steel W-Section Pilasters will be embeded into the existing floor and an additional 10" of concrete added to the basement foundation slab for uplift and floor strength concerns.
- The steel pilasters will be braced just below the existing floor elevation with a steel struts.

Soil Properties $\phi := 28deg$

$\gamma := 120pcf$

$K_o := 1 - \sin(\phi)$

$K_o = 0.531$

Steel Properties $F_y := 50ksi$

Water Unit Weight $\gamma_w := 62.4pcf$

Load & Resistance Factor Design

Strength Reduction Factors

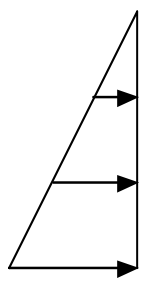
Shear Strength	$\phi_V := .85$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.
Flexural Strength	$\phi_B := 0.90$	
Load Factors		
Dead and Live Load Factor	$\gamma_L := 1.6$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1.3$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := .75$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Analysis

WALLS

Wall Loading

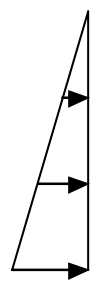
Soil or Soil & Water



$L_2 := H$

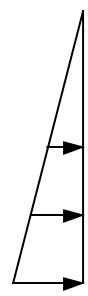
$L_2 = 27.75\ ft$

OR



$L_2 = 27.75\ ft$

&



$L_3 := H + HGL - HEAD$

$L_3 = 31.75\ ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot L_2$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma - \gamma_w) \cdot L_2 + \gamma_w \cdot L_3\right]$

$W_1 = 3675\ \frac{lb}{ft^2}$

$W_2 = 4414\ \frac{lb}{ft^2}$

Check to determine if water to top of gatewell with reduction factor, γ_X, or soil loading is worst case

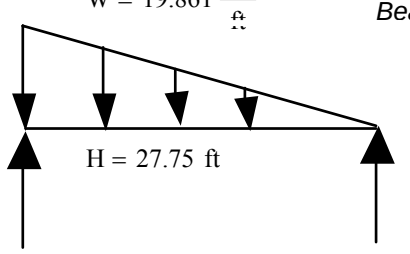
$W' := if(W_1 > W_2, W_1, W_2)$

$W' = 4414\ \frac{lb}{ft^2}$

Tributary Width $Width := 4.5ft$

Loading $W := W' \cdot Width$

$W = 19.861\ \frac{kip}{ft}$



Reaction and Moments from AISC Manual of Steel Construction

Beam Diagrams and Formulas

$$\phi M_{Max} := \frac{2\left(\frac{W \cdot H}{2}\right) \cdot H}{9 \cdot \sqrt{3}} \qquad \phi M_{Max} = 981.1\ ft \cdot kip$$

$$R_2 := \frac{\frac{W \cdot H}{2}}{3} \qquad R_2 = 91.857\ kips$$

PILASTERS

From AISC Manual of Steel Construction Beam Tables

Unbraced Length: $H = 27.75\ ft$

Design Moment: $\phi M_{Max} = 981.1\ ft \cdot kip$

Yield Strength: $F_y = 50\ ksi$

Use W 27x146

STRUTS

From AISC Manual of Steel Construction Beam Tables

Unbraced Length: $Width = 4.5\ ft$

Axial Load: $R_2 = 91.9\ kip$

Yield Strength: $F_y = 50\ ksi$

Use W 12x40

NOTE: Does not take into account floor load (combined axial and bending) but W 12x40 is good estimate for feasibility study. Additional Analysis to be performed at time of Plans and Specifications.

MILL STREET RCB ANALYSIS

CALCULATED BY Marvin Parks
PEER REVIEWED BY Paul Muller
Peer review completed on 10 April 2007.

LC	FILE	MEMBER	FS	PARAMETER	EVENT	STATUS
3	15675A	Bottom	2.04	Positive Moment	n500+3	Okay
2	15675B	Sidewall	1.88	Positive Moment	n500+3	Okay

The Mill Street outlet works RCB was evaluated for strength using CASE software CORTCUL. Based on the analysis, the Mill Street RCB structure is considered reliable for the existing condition and n500+3 events.

```
1000 4 15675A MILL ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
1010   LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=729.20
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2=AT-REST | 3=EMBANKMENT

1040 I    S
1050  3000.0 40000.0   150.0  .3750  .0000
1060   1   8.00    .00  729.20   6.00
1070     3.25     2.94     2.94
1080    18.00    15.00    18.00
1090   1
1100   775.30   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   729.20
1130     1.00      .57      .00   729.20
1140     1.50      .50      .00   729.20
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1    .60    .44     .60    .22    .60    .44
1200  11    .44    .20     .20    .31    .40    .31
1210  12    .20    .44     .31    .20    .31    .40
1220  21    .40    .60     .20    .60    .40    .60
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME: 21:13:51
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME: 21:14: 1

1. INPUT DATA

1.A.--HEADING

15675A MILL ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=729.20
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2=AT-REST | 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  3000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 40000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
                (FT)      (IN)        (FT)       (FT)
       1         8.00       .00      729.20      6.00


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.25     ROOF SLAB      = 18.00
     INTERIOR ROOF/END WALLS = 2.94     EXTERIOR WALLS = 15.00
     INTERIOR BASE SLAB      = 2.94     BASE SLAB      = 18.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	775.30	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	729.20
2	1.00	.57	.00	729.20
3	1.50	.50	.00	729.20

This is the n500+5 elevation. Since the RCB checked okay for this elevation, the RCB is good for the n500+3 event. CORTCUL was not rerun. Marvin Parks

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	.60	.44	.60	.22	.60	.44
11	.44	.20	.20	.31	.40	.31
12	.20	.44	.31	.20	.31	.40
21	.40	.60	.20	.60	.40	.60

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE: 3/13/2007 TIME: 21:14:26

2.A.--HEADING

15675A MILL ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST EVENT:N500+3 IWAT=0 GWATEL=729.20
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2=AT-REST | 3=EMBANKMENT

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	8.51	-13.03	8.51
AXIAL FORCE	(KIPS)	11.36	11.36	11.36
FLEXURE FACTOR OF SAFETY		6.97	3.71	6.97
STRENGTH REDUCTION (PHI)		.70	.77	.70
SHEAR FORCE AT D	(KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)		NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN	(KIPS)	-10.05		10.05
SHEAR FS AT 0.15LN (UI440)		7.22		7.22

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
APPLY FOR THIS MEMBER

MEMBER 11		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	-11.45	4.40	-10.74
AXIAL FORCE	(KIPS)	20.20	19.45	18.70
FLEXURE FACTOR OF SAFETY		4.08	10.95	4.18
STRENGTH REDUCTION (PHI)		.70	.70	.70
SHEAR FORCE AT D	(KIPS)	6.00		-5.65
SHEAR FS AT D (ACI63)		4.23		4.47
SHEAR FORCE AT 0.15LN	(KIPS)	5.57		-5.28
SHEAR FS AT 0.15LN (UI440)		5.47		5.73

MEMBER 12		LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT	(K-FT)	11.45	-4.40	10.74

AXIAL FORCE (KIPS)	20.20	19.45	18.70
FLEXURE FACTOR OF SAFETY	4.08	10.95	4.18
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-6.00		5.65
SHEAR FS AT D (ACI63)	4.23		4.47
SHEAR FORCE AT 0.15LN (KIPS)	-5.57		5.28
SHEAR FS AT 0.15LN (UI440)	5.47		5.73

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-8.00	11.96	-8.00
AXIAL FORCE (KIPS)	9.99	9.99	9.99
FLEXURE FACTOR OF SAFETY	6.37	3.95	6.37
STRENGTH REDUCTION (PHI)	.70	.78	.70
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	9.31		-9.31
SHEAR FS AT 0.15LN (UI440)	7.72		7.72

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	13.03	-8.51	13.03
AXIAL FORCE (KIPS)	16.11	16.11	16.11
FLEXURE FACTOR OF SAFETY	4.04	9.69	4.04
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	-10.05		10.05
SHEAR FS AT 0.15LN (UI440)	7.20		7.20

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-13.43	8.98	-12.76
AXIAL FORCE (KIPS)	20.20	19.45	18.70
FLEXURE FACTOR OF SAFETY	2.91	5.72	2.84
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	8.51		-8.10
SHEAR FS AT D (ACI63)	2.99		3.12
SHEAR FORCE AT 0.15LN (KIPS)	7.89		-7.56
SHEAR FS AT 0.15LN (UI440)	3.86		4.00

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	13.43	-8.98	12.76
AXIAL FORCE (KIPS)	20.20	19.45	18.70
FLEXURE FACTOR OF SAFETY	2.91	5.72	2.84
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-8.51		8.10
SHEAR FS AT D (ACI63)	2.99		3.12
SHEAR FORCE AT 0.15LN (KIPS)	-7.89		7.56
SHEAR FS AT 0.15LN (UI440)	3.86		4.00

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-12.42	7.54	-12.42

AXIAL FORCE (KIPS)	14.28	14.28	14.28
FLEXURE FACTOR OF SAFETY	3.77	10.91	3.77
STRENGTH REDUCTION (PHI)	.73	.70	.73
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	9.31		-9.31
SHEAR FS AT 0.15LN (UI440)	7.68		7.68

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	10.14	-20.87	10.14
AXIAL FORCE (KIPS)	14.15	14.15	14.15
FLEXURE FACTOR OF SAFETY	6.27	2.04	6.27
STRENGTH REDUCTION (PHI)	.70	.81	.70
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	-14.47		14.47
SHEAR FS AT 0.15LN (UI440)	5.12		5.12

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-15.53	4.18	-14.84
AXIAL FORCE (KIPS)	29.15	28.40	27.65
FLEXURE FACTOR OF SAFETY	3.18	8.93	3.26
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	7.47		-7.09
SHEAR FS AT D (ACI63)	3.53		3.71
SHEAR FORCE AT 0.15LN (KIPS)	6.94		-6.62
SHEAR FS AT 0.15LN (UI440)	4.67		4.86

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	15.53	-4.18	14.84
AXIAL FORCE (KIPS)	29.15	28.40	27.65
FLEXURE FACTOR OF SAFETY	3.18	8.93	3.26
STRENGTH REDUCTION (PHI)	.70	.70	.70
SHEAR FORCE AT D (KIPS)	-7.47		7.09
SHEAR FS AT D (ACI63)	3.53		3.71
SHEAR FORCE AT 0.15LN (KIPS)	-6.94		6.62
SHEAR FS AT 0.15LN (UI440)	4.67		4.86

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-9.57	19.86	-9.57
AXIAL FORCE (KIPS)	12.51	12.51	12.51
FLEXURE FACTOR OF SAFETY	5.72	2.08	5.72
STRENGTH REDUCTION (PHI)	.70	.82	.70
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	13.73		-13.73
SHEAR FS AT 0.15LN (UI440)	5.33		5.33

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

```
15675A MILL ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=729.20
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2=AT-REST | 3=EMBANKMENT


3.B.--RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE




OUTPUT COMPLETE.
```

```
1000 4 15675B MILL ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
1010   LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=775.30
1020   3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1030   1=TRENCH, PHI=26DEG, ACTIVE | 2=AT-REST | 3=EMBANKMENT

1040 I    S
1050  3000.0 40000.0   150.0  .3750  .0000
1060   1   8.00    .00  729.20   6.00
1070     3.25     2.94     2.94
1080    18.00    15.00    18.00
1090   1
1100   775.30   120.00   115.00
1110   3  62.40
1120     1.00      .40      .00   775.30
1130     1.00      .57      .00   775.30
1140     1.50      .50      .00   775.30
1150   0
1160     1.00     1.00
1170   0
1180   4
1190   1    .60    .44     .60    .22    .60    .44
1200  11    .44    .20     .20    .31    .40    .31
1210  12    .20    .44     .31    .20    .31    .40
1220  21    .40    .60     .20    .60    .40    .60
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME: 21:15:11


PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME: 21:15:18

1. INPUT DATA

1.A.--HEADING

15675B MILL ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=775.30
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2=AT-REST | 3=EMBANKMENT


1.B.--MODE AND PROCEDURE
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


1.C.--MATERIAL PROPERTIES

     CONCRETE:
          ULTIMATE STRENGTH     =  3000.      (PSI)
          ULTIMATE STRAIN       =      .003
          COMP. BLOCK RATIO     =      .85
          UNIT WEIGHT           =   150.      (PCF)


     REINFORCEMENT:
          YIELD STRENGTH        = 40000.      (PSI)
          MODULUS OF ELASTICITY =    29.E+06  (PSI)
          MAXIMUM REINF RATIO   =      .38


     STRENGTH REDUCTION FACTOR  = VARIABLE


1.D.--GEOMETRY
     NO OF      CELL     HAUNCH      INVERT      CELL
     CELLS     HEIGHT     WIDTH       ELEV      WIDTH
                (FT)      (IN)        (FT)       (FT)
       1         8.00       .00      729.20      6.00


     REINFORCEMENT COVER (IN):          THICKNESS (IN):
     EXTERIOR SURFACES       = 3.25     ROOF SLAB      = 18.00
     INTERIOR ROOF/END WALLS = 2.94     EXTERIOR WALLS = 15.00
     INTERIOR BASE SLAB      = 2.94     BASE SLAB      = 18.00


1.E.--LOAD DATA

     1.E.1.--STANDARD LOAD CASES
```

SOIL DATA:

LAYER NO	ELEV AT TOP OF LAYER (FT)	SATURATED UNIT WEIGHT (PCF)	MOIST UNIT WEIGHT (PCF)
1	775.30	120.00	115.00

STANDARD LOAD CASE DATA
WATER UNIT WEIGHT = 62.4 (PCF)

LOAD CASE	PRESSURE COEFFICIENTS VERTICAL	HORIZONTAL	SURFACE SURCHARGE (PSF)	GROUND WATER ELEVATION (FT)
1	1.00	.40	.00	775.30
2	1.00	.57	.00	775.30
3	1.50	.50	.00	775.30

This is the n500+5 elevation. Since the RCB checked okay for this elevation, the RCB is good for the n500+3 event. CORTCUL was not rerun. Marvin Parks

1.E.2--SPECIAL LOAD CASES
NO SPECIAL LOAD CASES

1.E.3.--LOAD FACTORS FOR ACI STRENGTH DESIGN:
LIVE LOAD FACTOR = 1.00
DEAD LOAD FACTOR = 1.00

1.E.4--INTERNAL WATER DATA
NO INTERNAL WATER

1.F.--REINFORCEMENT AREAS (SQIN) FOR INVESTIGATION

MEMBER NO	LEFT END TOP	LEFT END BOTTOM	CENTERLINE TOP	CENTERLINE BOTTOM	RIGHT END TOP	RIGHT END BOTTOM
1	.60	.44	.60	.22	.60	.44
11	.44	.20	.20	.31	.40	.31
12	.20	.44	.31	.20	.31	.40
21	.40	.60	.20	.60	.40	.60

SCHEMATIC OF CULVERT:

```
*--21--*
!      !
!      !
11     12
!      !
!      !
*---1--*
```

LOCAL COORDINATE SYSTEMS:
HORIZONTAL MEMBERS: ORIGIN AT LEFT END, X-AXIS TO RIGHT, Y-AXIS UP
VERTICAL MEMBERS : ORIGIN AT BOTTOM, X-AXIS UP, Y-AXIS TO LEFT

SIGN CONVENTIONS:

```
POSITIVE LATERAL LOAD ACTS IN PLUS Y DIRECTION
POSITIVE BENDING MOMENT PRODUCES COMPRESSION
     ON PLUS Y FACE OF MEMBER
POSITIVE SHEAR TENDS TO MOVE MEMBER IN PLUS Y DIRECTION
POSITIVE AXIAL LOAD ACTS IN PLUS X DIRECTION
POSITIVE AXIAL INTERNAL FORCE IS COMPRESSION
UNLESS OTHERWISE NOTED, FACTOR OF SAFETY FOR
     INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE
     IS DEFINED BY  FS = PHI * PN / P
          WHERE  PN = ULTIMATE STRENGTH AT ACTUAL ECCENTRICITY
                 P = ACTUAL AXIAL FORCE
```

```
PROGRAM CORTCUL - DESIGN/INVESTIGATION OF ORTHOGONAL CULVERTS
DATE:  3/13/2007                                TIME:  21:15:22

2.A.--HEADING

15675B MILL ST. PUMP STA. OUTLET WORKS M.PARKS 03122007
LOC: LEVEE CREST  EVENT:N500+3 IWAT=0 GWATEL=775.30
3 LOAD CASES:1. CV=1.0, CH=0.4,| 2. CH=1.0 CH=0.57 | 3. CV=1.5, CH=0.5
1=TRENCH, PHI=26DEG, ACTIVE | 2=AT-REST | 3=EMBANKMENT


2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 1
      INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE


MEMBER  1                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      17.55         -4.82          17.55
     AXIAL FORCE           (KIPS)      20.93         20.93          20.93
     FLEXURE FACTOR OF SAFETY           2.90         13.22           2.90
     STRENGTH REDUCTION  (PHI)           .71           .70            .71
     SHEAR FORCE AT D      (KIPS)     NNNNNN                       NNNNNN
     SHEAR FS AT D (ACI63)            NNNNNN                       NNNNNN
     SHEAR FORCE AT 0.15LN (KIPS)     -10.44                        10.44
     SHEAR FS AT 0.15LN (UI440)         7.15                         7.15
          NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT
               APPLY FOR THIS MEMBER


MEMBER 11                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)     -15.70         13.39         -15.07
     AXIAL FORCE           (KIPS)      20.98         20.23          19.48
     FLEXURE FACTOR OF SAFETY           2.24          2.54           2.18
     STRENGTH REDUCTION  (PHI)           .73           .71            .74
     SHEAR FORCE AT D      (KIPS)      11.06                       -10.58
     SHEAR FS AT D (ACI63)              2.31                         2.40
     SHEAR FORCE AT 0.15LN (KIPS)      10.26                        -9.89
     SHEAR FS AT 0.15LN (UI440)         2.99                         3.08


MEMBER 12                           LEFT END      CENTERLINE     RIGHT END
     BENDING MOMENT        (K-FT)      15.70        -13.39          15.07
```

AXIAL FORCE (KIPS)	20.98	20.23	19.48
FLEXURE FACTOR OF SAFETY	2.24	2.54	2.18
STRENGTH REDUCTION (PHI)	.73	.71	.74
SHEAR FORCE AT D (KIPS)	-11.06		10.58
SHEAR FS AT D (ACI63)	2.31		2.40
SHEAR FORCE AT 0.15LN (KIPS)	-10.26		9.89
SHEAR FS AT 0.15LN (UI440)	2.99		3.08

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-16.82	3.95	-16.82
AXIAL FORCE (KIPS)	18.64	18.64	18.64
FLEXURE FACTOR OF SAFETY	2.68	15.28	2.68
STRENGTH REDUCTION (PHI)	.75	.70	.75
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	9.70		-9.70
SHEAR FS AT 0.15LN (UI440)	7.59		7.59

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 2
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	19.81	-2.56	19.81
AXIAL FORCE (KIPS)	23.31	23.31	23.31
FLEXURE FACTOR OF SAFETY	2.53	13.97	2.53
STRENGTH REDUCTION (PHI)	.72	.70	.72
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	-10.44		10.44
SHEAR FS AT 0.15LN (UI440)	7.26		7.26

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-16.69	15.69	-16.08
AXIAL FORCE (KIPS)	20.98	20.23	19.48
FLEXURE FACTOR OF SAFETY	2.02	1.88	1.94
STRENGTH REDUCTION (PHI)	.74	.76	.76
SHEAR FORCE AT D (KIPS)	12.31		-11.81
SHEAR FS AT D (ACI63)	2.07		2.15
SHEAR FORCE AT 0.15LN (KIPS)	11.42		-11.03
SHEAR FS AT 0.15LN (UI440)	2.93		2.76

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	16.69	-15.69	16.08
AXIAL FORCE (KIPS)	20.98	20.23	19.48
FLEXURE FACTOR OF SAFETY	2.02	1.88	1.94
STRENGTH REDUCTION (PHI)	.74	.76	.76
SHEAR FORCE AT D (KIPS)	-12.31		11.81
SHEAR FS AT D (ACI63)	2.07		2.15
SHEAR FORCE AT 0.15LN (KIPS)	-11.42		11.03
SHEAR FS AT 0.15LN (UI440)	2.93		2.76

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-19.04	1.74	-19.04

AXIAL FORCE (KIPS)	20.79	20.79	20.79
FLEXURE FACTOR OF SAFETY	2.34	15.64	2.34
STRENGTH REDUCTION (PHI)	.75	.70	.75
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	9.70		-9.70
SHEAR FS AT 0.15LN (UI440)	7.69		7.69

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

2.B.--SUMMARY OF RESULTS FOR STANDARD LOAD CASE 3
INVESTIGATION USING ACI STRENGTH DESIGN PROCEDURE

MEMBER 1	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	18.36	-8.75	18.36
AXIAL FORCE (KIPS)	22.33	22.33	22.33
FLEXURE FACTOR OF SAFETY	2.82	9.19	2.82
STRENGTH REDUCTION (PHI)	.71	.70	.71
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	-12.65		12.65
SHEAR FS AT 0.15LN (UI440)	5.95		5.95

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

MEMBER 11	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-17.74	13.28	-17.12
AXIAL FORCE (KIPS)	25.46	24.71	23.96
FLEXURE FACTOR OF SAFETY	2.09	3.36	2.04
STRENGTH REDUCTION (PHI)	.70	.70	.72
SHEAR FORCE AT D (KIPS)	11.80		-11.30
SHEAR FS AT D (ACI63)	2.20		2.29
SHEAR FORCE AT 0.15LN (KIPS)	10.94		-10.56
SHEAR FS AT 0.15LN (UI440)	2.89		2.97

MEMBER 12	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	17.74	-13.28	17.12
AXIAL FORCE (KIPS)	25.46	24.71	23.96
FLEXURE FACTOR OF SAFETY	2.09	3.36	2.04
STRENGTH REDUCTION (PHI)	.70	.70	.72
SHEAR FORCE AT D (KIPS)	-11.80		11.30
SHEAR FS AT D (ACI63)	2.20		2.29
SHEAR FORCE AT 0.15LN (KIPS)	-10.94		10.56
SHEAR FS AT 0.15LN (UI440)	2.89		2.97

MEMBER 21	LEFT END	CENTERLINE	RIGHT END
BENDING MOMENT (K-FT)	-17.61	7.91	-17.61
AXIAL FORCE (KIPS)	19.90	19.90	19.90
FLEXURE FACTOR OF SAFETY	2.61	10.17	2.61
STRENGTH REDUCTION (PHI)	.74	.70	.74
SHEAR FORCE AT D (KIPS)	NNNNNN		NNNNNN
SHEAR FS AT D (ACI63)	NNNNNN		NNNNNN
SHEAR FORCE AT 0.15LN (KIPS)	11.91		-11.91
SHEAR FS AT 0.15LN (UI440)	6.23		6.23

NNNNNN - ACI63 SHEAR PROCEDURE DOES NOT APPLY FOR THIS MEMBER

OUTPUT COMPLETE.

US Army Corps of Engineers

KANSAS CITYS LEVEES PHASE II

Variables

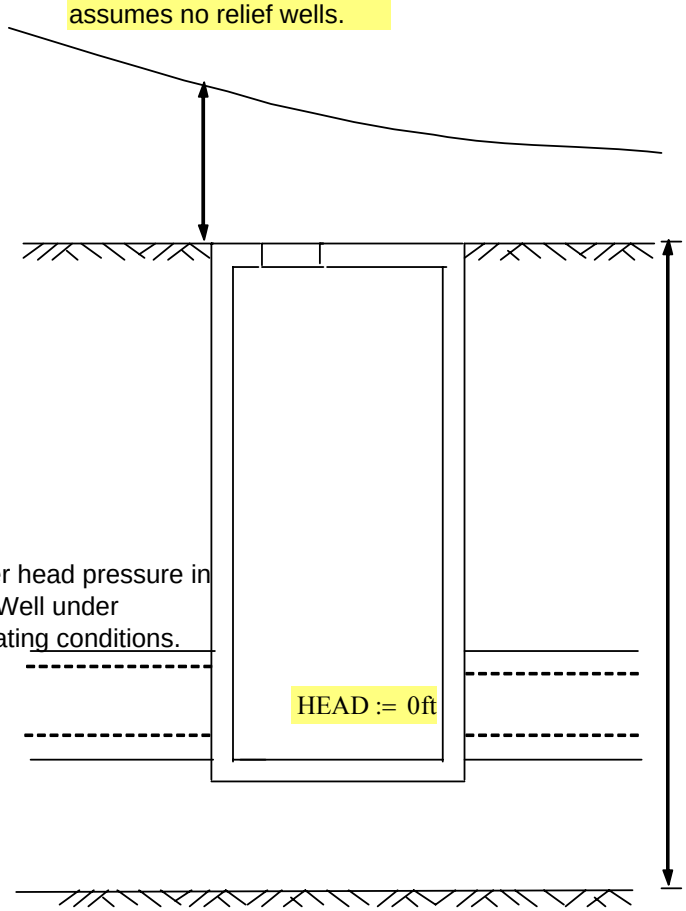
$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $pcf := \frac{lb}{ft^3}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $kips := 1000lb$ $klf := \frac{kips}{ft}$

HGL = 769.5 is the hydraulic grade line elevation for the n500+3 event. The HGL = 765.7 is the hydraulic grade line elevation for the existing condition event. This document is an analysis of the pump station for the existing condition grade line = 765.7 elevation.
For the n500+3 event, the pump station walls are adequate for strength (FS>1.5). Therefore an existing condition analysis of the walls was not conducted. However, the pump station base slab is inadequate for strength for the n500+3 event. Therefore, the pump station base slab will be evaluated for strength for the existing condition load condition. Also, the pump station will be evaluated for uplift stability for the existing condition since it did not meet the minimum requirements for the n500+3 event either.

REFERENCES
1. Moments and Reactions for Rectangular Plates, Monograph No. 13, Bueau of Reclamation, July 1963
2. RAM Advance finite element analysis, Version 7.0

Properties

NOTE: HGL Supplied assumes no relief wells.



Original Hydraulic Grade Line as supplied by Geotechnicals (Affects Uplift on Base Slab if in the landside of levee)

$ELEV_1 := 758ft$ grade elevation

$HGL := 765.7ft - ELEV_1$

$HGL = 7.7\ ft$

$BLANKET := 755ft - 720ft$

$BLANKET = 35\ ft$

For blanket thickness, do not go up to grade elevation as confirmed with EC-GD

BLANKET = blanket thickness

Dscn4803.jpg

Double click on jpg box to view photo of pump station.
The south wall is parallel and closest to the levee centerline. The station utilities enter the building thru the south wall as shown in the photo.
The station entry doors are in the east wall.
The stem riser shown in the photo is the 72" gate stem riser.

Wall 1 Parameters East wall	Wall 2 Parameters North wall	Wall 3 Parameters South wall	Wall 4 Parameters West wall
$L_1 := 21ft$	$L_2 := 34.5ft$	$L_3 := 34.5ft$	$L_4 := 21ft$
$Hwall_1 := 30.56ft$	$Hwall_2 := 35.06ft$	$Hwall_3 := 30.56ft$	$Hwall_4 := 33.06ft$
$D_1 := 2ft$	$D_2 := 2ft$	$D_3 := 2ft$	$D_4 := 2ft$
$C_{c1} := 2in$	$C_{c2} := 2in$	$C_{c3} := 2in$	$C_{c4} := 2in$

Wall height calculations
Top of wall elevation, 758 ft

$758ft - 727.44ft = 30.56$ $758ft - 722.94ft = 35.06$ $758ft - 727.44ft = 30.56$ ft $758ft - 724.94ft = 33.06$ ft

Floor Thickness $D_f := 2ft$ Clear Cover $C_{cf} := 3in$ $d_f := D_f - C_{cf} - \frac{1.25in}{2}$ $d_f = 20.375$ in

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - C_{c1} - 0.5in$ $d_1 = 21.5$ in

WALL 3

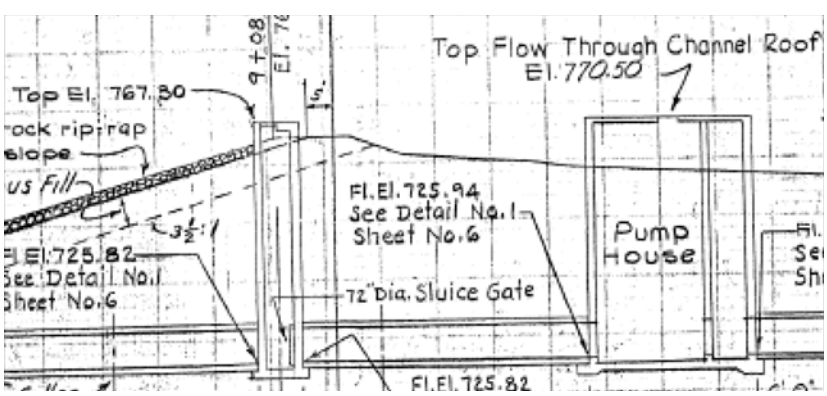
$d_3 := D_3 - C_{c3} - 0.5in$ $d_3 = 21.5$ in

WALL 2

$d_2 := D_2 - C_{c2} - 0.5in$ $d_2 = 21.5$ in

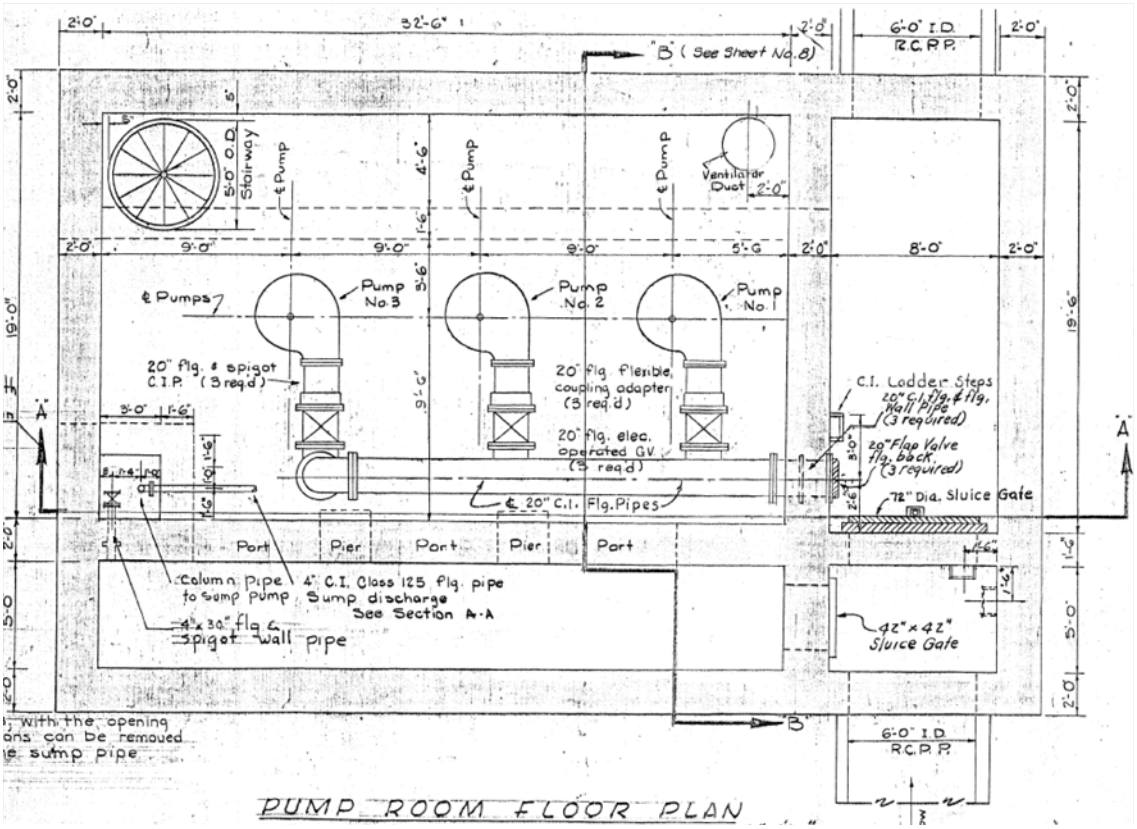
WALL 4

$d_4 := D_4 - C_{c4} - 0.5in$ $d_4 = 21.5$ in



Cross-Sectional Drawing

South wall



West wall

Plan View Drawing

Assumptions and Criteria

- As-built drawings give fc=1350 and fs=20,000. These are working stress parameters. For this analysis, f'c=3000 psf and fy=40,000 psf.

Soil Properties $\phi := 26deg$ $\gamma := 115pcf$

$K_o := 1 - \sin(\phi)$

$K_o = 0.562$

Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40ksi$

Concrete Properties $f'_c := 3.00ksi$

Concrete Unit Weight $\gamma_c := 150pcf$

Strength Reduction Factors

Load & Resistance Factor Design

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

WALL EVALUATION

The walls were not evaluated for strength for the existing condition loading condition. The walls met the FS=1.5 requirement for the n500+3 event. To review the wall analysis, review **5th PS 185+70 +3 HGL 769_5.xmcd** file.

Uplift on Structure

Stability

$Wall1 := 30ft \cdot 2ft \cdot (758.50ft - 720ft)$ $\quad Wall1 \cdot 150pcf = 346.5\ kips$

$Wall2 := 42.5ft \cdot 2ft \cdot (758.5ft - 720.94ft)$ $\quad Wall2 \cdot 150pcf = 478.89\ kips$

$Wall3 := 42.5ft \cdot 2ft \cdot (758.5ft - 722.94ft)$ $\quad Wall3 \cdot 150pcf = 453.39\ kips$

$Wall4 := 30ft \cdot 2ft \cdot (758.50ft - 722.94ft)$ $\quad Wall4 \cdot 150pcf = 320.04\ kips$

$Int1 := 26ft \cdot 2ft \cdot (758.50ft - 724.94ft)$ $\quad Int1 \cdot 150pcf = 261.768\ kips$

$Int2 := 32.5ft \cdot 2ft \cdot (758.50ft - 725.94ft)$ $\quad Int2 \cdot 150pcf = 317.46\ kips$

$Int3 := 8ft \cdot 1.5ft \cdot (758.50ft - 724.94ft)$ $\quad Int3 \cdot 150pcf = 60.408\ kip$

$PumpFloor := 19ft \cdot 1.5ft \cdot 32.5ft$ $\quad PumpFloor \cdot 150pcf = 138.937\ kip$

$Gateslab := 26ft \cdot 2ft \cdot 8ft$ $\quad Gateslab \cdot 150pcf = 62.4\ kip$

$heel1 := 1.5ft \cdot 3ft \cdot 30ft$ $\quad heel1 \cdot 150pcf = 20.25\ kip$

$heel2 := 1.5ft \cdot 2ft \cdot 30ft$ $\quad heel2 \cdot 150pcf = 13.5\ kip$

$base := 32.5ft \cdot 2ft \cdot 19.5ft$ $\quad base \cdot 150pcf = 190.125\ kip$

Above ground concrete building structure:

Refer to "Motor Room Floor Plan" on Sheet 3 of the drawings.
The above grade structure has 1 ft thick walls and the height = 12 ft.

$Bldgwalls := [[(46 \cdot ft \cdot 23 \cdot ft) - (44.5 \cdot ft \cdot 21 \cdot ft)] \cdot 12 \cdot ft$ $\quad Bldgwalls \cdot 150pcf = 222.3\ kip$

$Bldgroof := 2.5in \cdot [26ft \cdot (38ft + 2in)]$ $\quad Bldgroof \cdot 150pcf = 31.01\ kip$

$$Structure := \begin{pmatrix} Wall1 + Wall2 + Wall3 + Wall4 + Int1 + Int2 + Int3 + PumpFloor + Gateslab + heel1 \ldots \\ + heel2 + base + Bldgwalls + Bldgroof \end{pmatrix} \cdot 150pcf$$

$Structure = 2916.979\ kip$

$Waterarea := 19.5ft \cdot 32.5ft$ $\quad Waterarea = 633.75\ ft^2$ $\quad HEAD := 3ft$

$Water := HEAD \cdot Waterarea \cdot 62.4pcf$ $\quad Water = 118.638\ kip$

$heel := 1.5ft \cdot 30ft$ $\quad heel = 45\ ft^2$

$upliftarea := 49.5ft \cdot 30ft$ $\quad upliftarea = 1485\ ft^2$

$H := 758ft - 720.94ft$ $\quad H = 37.06\ ft$

$volsoil := (758ft - 724.94ft) \cdot heel + (758ft - 722.94ft) \cdot heel$ $\quad volsoil = 3065.4\ ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 161.24\ kip$ $\quad volsoil \cdot \gamma_w = 191.281\ kip$

Uplift on Structure, con't

Head, if structural foundation in blanket.

$HGL = 7.7\ ft$

bottom of blanket at 720

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot upliftarea \cdot \gamma_w$$

$U_1 = 4189.636\ kips$

$BLANKET = 35\ ft$

$HGL = 7.7\ ft$

$H = 37.06\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (upliftarea) \cdot \gamma_w\right]$$

$U_2 = 4147.641\ kips$

$$Uplift := if\left(H > BLANKET, U_2, U_1\right)$$

$Uplift = 4147.641\ kips$

$$\left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 45.213\ ft$$

$$Stability := \frac{Structure + Water + volsoil \cdot \left(\gamma - \gamma_w\right)}{Uplift - volsoil \cdot \gamma_w}$$

$Stability = 0.808$

$(H + HGL) = 44.76\ ft$

$$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not\ Okay")$$

$StabilityCheck = "Not\ Okay"$

Consider Side Friction As Aid To Structure Factor of Safety Against Uplift

SOIL

$\phi := 26deg$ INTERNAL FRICTION ANGLE

$\gamma := 120pcf$ SOIL UNIT WEIGHT

$\gamma_w := 62.4pcf$ WATER UNIT WEIGHT

$\gamma_{sub} := (\gamma - \gamma_w)$ SUBMERGED SOIL UNIT WEIGHT $\gamma_{sub} = 57.6\,pcf$

$K_o := 1 - \sin(\phi)$ AT-REST SOIL COEFFICIENT. ASSUME THAT THE WALLS DO NOT MOVE

$K_o = 0.562$

$\mu := 0.25$ FRICTION COEFICIENT BETWEEN STRUCTURE AND SOIL.

$U := 2793psf$ UPLIFT PRESSURE ACTING ON BASE OF STRUCTURE

$c := 5ft$ ASSUMED CRACK DEPTH

STRUCTURE

$H := 30.5ft$ DEPTH OF STRUCTURE BELOW GRADE

$per := 30ft \cdot 2 + 46.5ft \cdot 2$ STRUCTURE PERIMETER $per = 153\ ft$

$W_s := 2917kip$ STRUCTURE DEAD WEIGHT

$W_{water} := 118.6kip$ WEIGHT OF WATER LOCATED IN STRUCTURE.

$Soil := 161.2kip$ WEIGHT OF SOIL LOCATED ON TOP OF THE BASE SLAB HEEL OR WALL STEP. EQUAL TO ZERO IF NO HEEL IS PRESENT.

UPLIFT PARAMETERS:

$U_{s_ec} := 4147.64kip$ UPLIFT FORCE FOR THE EXISTING CONDITION

V. Calculation of Pv

1. Effective Lateral Load that Contributes to Side Friction

$\gamma = 120\,\text{pcf}$ $\quad c = 5\ \text{ft}$ $\quad H = 30.5\ \text{ft}$

$R_1 := (H - c)\cdot\left(\frac{c + H}{2}\right)\cdot\gamma \qquad R_1 = 1688.162\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

$R_2 := (H - c)\cdot U\cdot\left(\frac{c}{2\cdot H} + \frac{1}{2}\right) \qquad R_2 = 1288.261\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

$R_1 - R_2 = 399.901\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

2. Force Created By Side Friction, Pv

$K_o = 0.562$ $\quad \mu = 0.25$ $\quad per = 153\ \text{ft}$

$P_v := (R_1 - R_2)\cdot K_o\cdot\mu\cdot per \qquad P_v = 276.4\,\text{kip}$

Stability Without the Aid of Side Friction, Existing Condition

$Stability_{nofric_ec} := \frac{W_s + W_{water} + Soil}{U_{s_ec} - volsoil\cdot\gamma_w} \qquad Stability_{nofric_ec} = 0.808$

Stability with the Aid of Side Friction, Existing Conditon

$Stability_{fric_ec} := \frac{W_s + W_{water} + Soil + P_v}{U_{s_ec} - volsoil\cdot\gamma_w} \qquad Stability_{fric_ec} = 0.878$

The uplift factor of safety for various HGL elevations is shown below. To achieve a factor of safety equal to 1.1 (with skin friction present) the HGL cannot exceed elevation 758.25.

Uplift factors of safety versus HGL

HGL	FS no skin friction	FS with skin friction
765.7	0.808	0.878
764	0.842	0.923
763	0.863	0.952
762.5	0.873	0.966
761.75	0.89	0.99
761.25	0.902	1.00
758.75	0.965	1.09

Existing Condition

Amount of additional weight required to achieve FS=1.10

$$Reqdadditional := \left(Uplift - volsoil \cdot \gamma_w\right) \cdot 1.10 - \left[Structure + Water + volsoil \cdot \left(\gamma - \gamma_w\right)\right]$$

$$Reqdadditional = 1139.812\,kip$$

Investigate the length of heel required to achieve the required FS = 1.10

$heel_{prop} := 6ft$ $\quad footprint := \left(49.5ft + 2heel_{prop}\right) \cdot \left(30ft + 2heel_{prop}\right)$ $\quad footprint = 2583\ ft^2$

$theel := 2.5ft$ $\quad V_{soil} := \left(footprint - 1485ft^2\right) \cdot 35ft$ $\quad V_{soil} = 38430\ ft^3$

$W_{heel} := \left(footprint - 1485ft^2\right) \cdot theel \cdot 150pcf$ $\quad W_{heel} = 411.75\,kip$

$Uplift_{mod} := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot footprint \cdot \gamma_w$ $\quad Uplift_{mod} = 5997.478\,kip$

$V_{soil} \cdot \left(\gamma - \gamma_w\right) = 2213.568\,kip$ $\quad V_{soil} \cdot \gamma_w = 2398.032\,kip$

$Stability_{mod} := \frac{Structure + Water + V_{soil} \cdot \left(\gamma - \gamma_w\right) + W_{heel}}{Uplift_{mod} - V_{soil} \cdot \left(\gamma_w\right)}$ $\quad Stability_{mod} = 1.573$

Design of the heel not accomplished for the existing condition event.

BASE SLAB ANALYSIS

$W_{water} := 118kip$ WEIGHT OF WATER IN STATION

$BaseSlab := upliftarea$ $BaseSlab = 1485\ ft^2$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$

$W' := if\left(Stability < 1.0, \frac{Uplift}{BaseSlab}, \frac{Structure}{BaseSlab}\right)$ $W' = 2793.024\,psf$

$W' = 2793.024\,psf$

$\frac{Structure + W_{water}}{BaseSlab} = 2043.757\,psf$

$\frac{Uplift}{BaseSlab} = 2793.024\,psf$

$\frac{Structure + volsoil \cdot (\gamma - \gamma_w)}{BaseSlab} = 2083.196\,psf$

$W' := 2606psf$ $2793psf - 62.4pcf \cdot 3ft = 2605.8\,psf$

$Slab_t := 24in$ $L_{strip} := 19.83ft$ $Wall_t := D_f$ $Slab_d := d_f$ effective depth of slab

$ratio := \frac{13ft}{32.5ft}$ $ratio = 0.4$ Analyze slab with one-way action

$L := 13ft$

Rx controls.

$V'_{ub} := \frac{W' \cdot L}{2}$ $V'_{ub} = 16939\ \frac{lb}{ft}$ $a := 19.5ft$ $d_f = 20.375\ in$

$D_f = 2\ ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}} \cdot \left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 12330\ \frac{lb}{ft}$

$A_g := D_f 12 \frac{in}{ft}$ $A_g = 288\ \frac{in^2}{ft}$

$Water = 118638\ lb$

Base Slab Shear, con't

$$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f \right) \right]$$

ACI EQ (11-4)

$$\phi V_{nb} = 26784 \frac{lb}{ft}$$

$$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

CheckBase = "OKAY"

Factor of Safety

$$FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$$

$$FS_{Sb} = 2.172$$

Slab Bending

Assume simple span moment (conservative). One-way action, investigate the short span moment capacity.

$$M_b := \frac{L^2 \cdot W'}{8} \qquad M_b = 55.052 \frac{kip \cdot ft}{ft}$$

#10's @ 12 in.
(mid-plate moment)

TOP REINF

$$A_{mpl} := 1.27 \frac{in^2}{ft} \qquad b := 12 \frac{in}{ft} \qquad a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{mpl} = 1.66\ in$$

$$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_f - \frac{a_{mpl}}{2} \right) \qquad \phi M_{mpl} = 82.7 \frac{kip \cdot ft}{ft}$$

$$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot M_b, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

CheckMP = "OKAY"

Factor of Safety

$$FS_{Mb} := \frac{\phi M_{mpl}}{M_b}$$

$$FS_{Mb} = 1.503$$

FACTORS OF SAFETY SUMMARY, Existing Conditon

Three exterior walls and the base slab were evaluate for strength.

WALL 1 (EAST WALL) STATUS: OKAY

WALL 2 (NORTH WALL) STATUS: OKAY

WALL 3 (SOUTH WALL) STATUS: OKAY

STABILITY STATUS: NO GOOD FOR EXISTING EVENT

Uplift factors of safety versus HGL		
HGL	**FS no skin friction**	**FS with skin friction**
765.7	0.808	0.878
764	0.842	0.923
763	0.863	0.952
762.5	0.873	0.966
761.75	0.89	0.99
761.25	0.902	1.00
758.75	0.965	1.09

$0.878 - 0.808 = 0.07$
$0.923 - 0.842 = 0.081$
$0.952 - 0.863 = 0.089$
$0.966 - 0.873 = 0.093$
$0.99 - 0.89 = 0.1$

$1.0 - 0.902 = 0.098$
$1.09 - 0.965 = 0.125$

Stability = 0.808 HEAD = 3 ft HGL = 7.7 ft

BASE SLAB STATUS: OKAY

Shear

$FS_{Sb} = 2.172$

Moment

$FS_{Mb} = 1.503$ Analyzed as a simple span, one-way action.

Conclusions

For the n500+3 event:
1.The pump station base slab has a FS for strength of 1.4. The required factor of safety is 1.5. The base slab will require thickening or some type of strengthening to achieve the design level of strength against the uplift forces.
2. The pump station wall strength is adequate.
3. The pump station uplift factor of safety is inadequate. The flotation FS = 0.74. The required additional weight is 1543 kips in order to achieve a FS =1.10. Without additional weight, a heel extension of 10 ft is necessary to achieve the required factor of safety against flotation.

For the existing condition event:
1. The pump station wall strength is adequate. No reliability analysis was required.
2. The pump station base slab strength is adequate. No reliability analysis was required.
3. The pump station uplift factor of safety is inadequate. The flotation FS= 0.81 when the benefit of skin friction is neglected. If the benefit of skin friction is taken into account, the existing condition uplift factor of safety is 0.88. The required additional weight is 1156 kips in order to achieve a FS = 1.10.

Other:
1. The HGL for the n500+3 event was lowered from 770.7 to 769.5. The bottom of blanket elevation was lowered from 723 to 720. The top of blanket elevation is elevation 755.0, whereas before it was 758. These updated values were provided by EC-GD.
2. The station has a 1.5 foot heel along the east and west walls. Whether the station has a heel along the north wall is questionable. Sheet 4 of the drawings show a heel whereas Sheet 7 of the drawings do not show a heel. The existance of heels along the north and south walls should be verified during PED. For this analysis, a heel was considered NOT present along the north or south walls. The presence of a heel will improve the uplift factor of safety.

Factors of Safety for Uplift Versus HGL (no skin friction)

HGL	FS	Event
769.5	0.74	n500+3
765.7	0.81	existing condition
758	0.99	
755	1.08	
754	1.11	

Table at left based on the following:
Water depth = 3 feet
Equipment weight not included
Heel along the east and west walls only
NO BENEFIT FROM SKIN FRICTION ASSUMED.

Analysis indicates that the HGL cannot be greater than 754.5 in order to achieve a factor of safety against uplift greater than 1.10 when no modifications are done to the station assuming no benefit from skin friction.

The uplift factor of safety for various HGL elevations is shown below. To achieve a factor of safety equal to 1.1 (**with skin friction present**) the HGL cannot exceed elevation 758.25.

Uplift factors of safety versus HGL			
HGL	**FS no skin friction**	**FS with skin friction**	
765.7	0.808	0.878	Existing Condition
764	0.842	0.923	
763	0.863	0.952	
762.5	0.873	0.966	
761.75	0.89	0.99	
761.25	0.902	1.00	
758.75	0.965	1.09	

EC-GD Provided Input Values (7 March 2007)

Armourdale Levee Unit						
Pump Station	Existing Relief Wells?	EC-GD Comps	EC-GD Peer Review	EC-GD Backchecl and Release to EC-DS	Landside Blanket Thickness (ft)	Design Condition n500+3 HGL at Pump Station (assuming no wells)
Osage 76+83*	N	RSK	GMB	RSK	24	763
Shawnee Ave 230+78	Y	RSK	GMB	RSK	25	766.1
KCS RR 276+79	Y	RSK	GMB	RSK	28***	762.3
Central Avenue 299+21	Y	RSK	GMB	RSK	14	760.2
12th Street 129+20	N	RSK	GMB	RSK	26	772
Mill Street 156+75	N	RSK	GMB	RSK	35	770.9
5th Street 185+70	N	RSK	GMB	RSK	35	769.5
MW Cold Storage 194+60	Y	RSK	GMB	RSK	28**	768
PBI Gordon 289+59	N	RSK	GMB	RSK	27	759.6
National Beef 295+52	Y	RSK	GMB	RSK	24	761

*Design condition for fully penetrating seepage cutoff wall from station 62+00 to 83+00
**Blanket thickness based upon building basement elevation and not adjacent ground surface
***Calculations ignore the slot - assumed to be filled in for design

EC-GD Soil Parameters, con't

22 November 2006
rsk

Kansas City Levees – Armourdale
Geotechnical Design Parameters

Material	Unit Weight (pcf)		Shear Strength			
	Moist	Saturated	Undrained		Drained	
			c (psf)	ϕ (deg)	c' (psf)	ϕ' (deg)
Levee Fill	115	120	1000	0	0	29
Foundation Clay (CL)	110	115	500	0	0	26
Foundation Clay (CH)	105	110	500	0	100	17
Foundation Silt (ML)	110	115	500	0	0	28
Foundation Sand (SP)	115	120	N/A	N/A	0	32

Notes:

1. Undrained shear strengths were estimated from limited existing test data and typical values for the materials. Foundation strength data was increased from the existing test data slightly to account for an increase in material strength under the footprint of the existing levee due to consolidation from the weight of the levee.
2. Drained strength information was taken for the most part from Armourdale Unit, Design Memorandum No. 3, test data performed in 1962.

End of analysis.

Variables

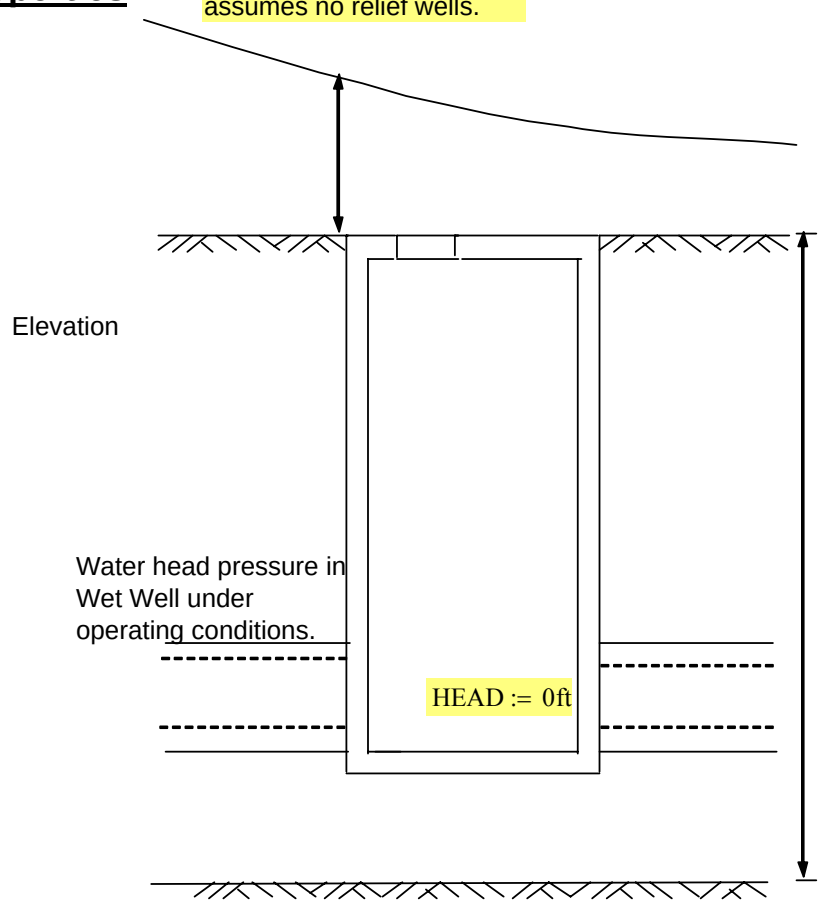
$kip := 1000lb \quad plf := \frac{lb}{ft} \quad psf \equiv \frac{lb}{ft^2} \quad pcf := \frac{lb}{ft^3} \quad psi \equiv \frac{lb}{in^2} \quad ksi \equiv \frac{1000lb}{in^2} \quad pcf := \frac{lb}{ft^3} \quad kips := 1000lb \quad klf := \frac{kips}{ft}$

HGL = 769.5 is the hydraulic grade line elevation for the n500+3 event. This document analyzes the pump station when exposed to n500+3 event loading whereby the HGL = 769.5.

REFERENCES
1. Moments and Reactions for Rectangular Plates, Monograph No. 13, Bueau of Reclamation, July 1963
2. RAM Advance finite element analysis, Version 7.0

Properties

NOTE: HGL Supplied assumes no relief wells.



Original Hydraulic Grade Line as supplied by Geotechnicals (Affects Uplift on Base Slab if in the landside of levee)

$ELEV_1 := 758ft$ grade elevation

$HGL := 769.5ft - ELEV_1$

$HGL = 11.5\ ft$

$BLANKET := 755ft - 720ft$

$BLANKET = 35\ ft$

For blanket thickness, do not go up to grade elevation as confirmed with EC-GD

$HEAD := 0ft$

BLANKET = blanket thickness

Dscn4803.jpg

Double click on jpg box to view photo of pump station.
The south wall is parallel and closest to the levee centerline. The station utilities enter the building thru the south wall as shown in the photo.
The station entry doors are in the east wall.
The stem riser shown in the photo is the 72" gate stem riser.

Wall 1 Parameters East wall	Wall 2 Parameters North wall	Wall 3 Parameters South wall	Wall 4 Parameters West wall
$L_1 := 21ft$	$L_2 := 34.5ft$	$L_3 := 34.5ft$	$L_4 := 21ft$
$Hwall_1 := 30.56ft$	$Hwall_2 := 35.06ft$	$Hwall_3 := 30.56ft$	$Hwall_4 := 33.06ft$
$D_1 := 2ft$	$D_2 := 2ft$	$D_3 := 2ft$	$D_4 := 2ft$
$C_{c1} := 2in$	$C_{c2} := 2in$	$C_{c3} := 2in$	$C_{c4} := 2in$

Wall height calculations
Top of wall elevation, 758 ft

$758ft - 727.44ft = 30.56$ $758ft - 722.94ft = 35.06$ $758ft - 727.44ft = 30.56$ ft $758ft - 724.94ft = 33.06$ ft

Floor Thickness $D_f := 2ft$ Clear Cover $C_{cf} := 3in$ $d_f := D_f - C_{cf} - \frac{1.25in}{2}$ $d_f = 20.375$ in

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - C_{c1} - 0.5in$ $d_1 = 21.5$ in

WALL 3

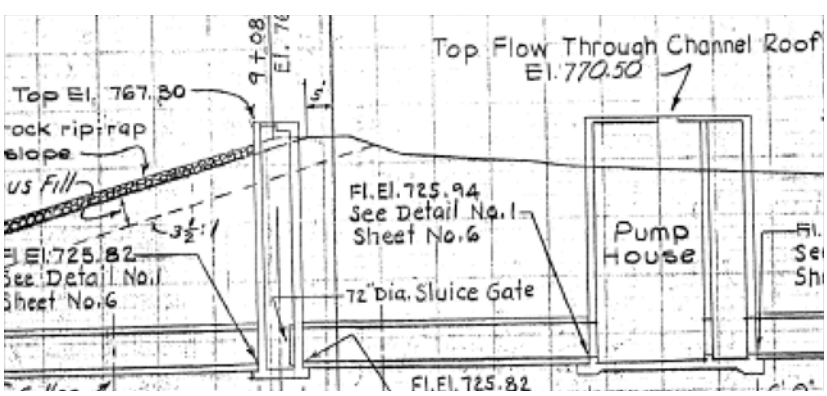
$d_3 := D_3 - C_{c3} - 0.5in$ $d_3 = 21.5$ in

WALL 2

$d_2 := D_2 - C_{c2} - 0.5in$ $d_2 = 21.5$ in

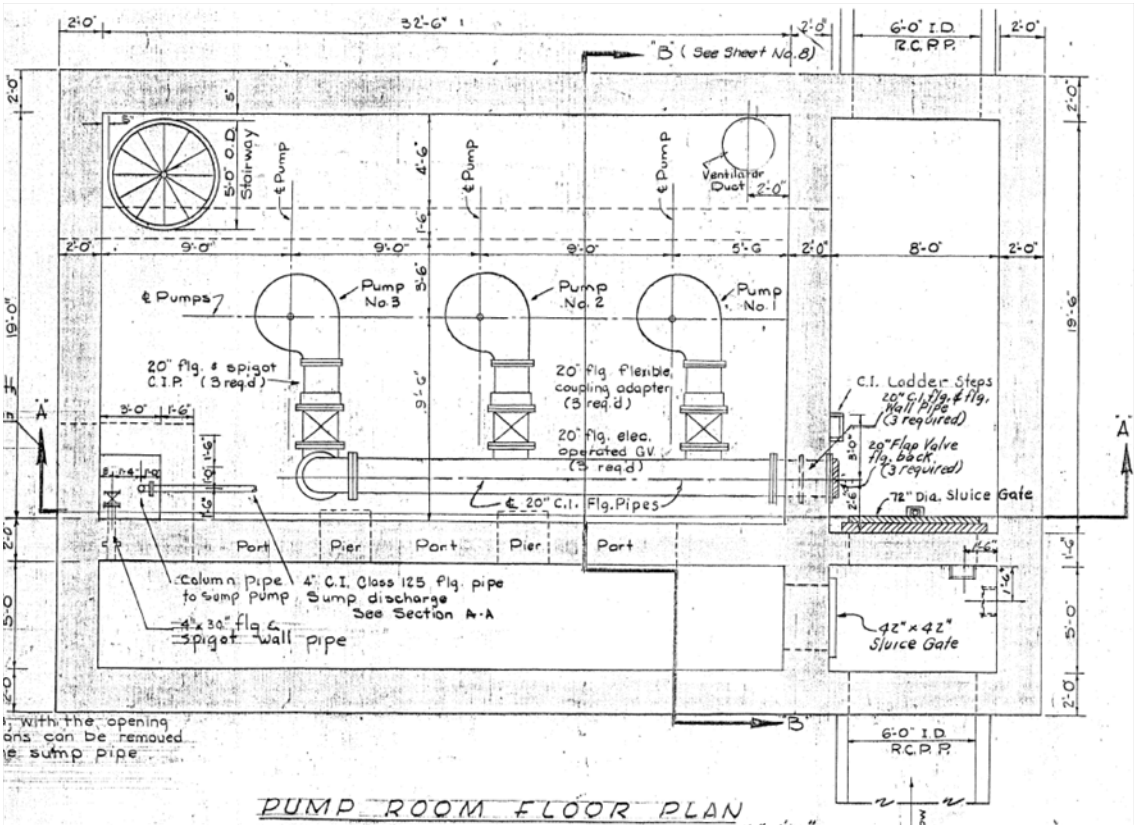
WALL 4

$d_4 := D_4 - C_{c4} - 0.5in$ $d_4 = 21.5$ in



Cross-Sectional Drawing

South wall



West wall

Plan View Drawing

Assumptions and Criteria

- As-built drawings give fc=1350 and fs=20,000. These are working stress parameters. For this analysis, f'c=3000 psf and fy=40,000 psf.

Soil Properties $\phi := 26deg$

$\gamma := 115pcf$

$K_o := 1 - \sin(\phi)$

$K_o = 0.562$

Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40ksi$

Concrete Properties $f'_c := 3.00ksi$

Concrete Unit Weight $\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ — Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ — Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ — Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

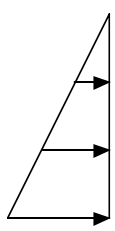
WALL 1, (East wall)

$K_o = 0.562$ $\quad$ $BLANKET = 35\ ft$ $\quad$ $HGL = 11.5\ ft$

$Hwall_1 = 30.56\ ft$ $\quad$ $HEAD = 0$

Wall Loading

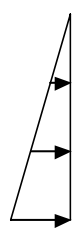
Soil or Soil & Water



$H_2 := Hwall_1$

$H_2 = 30.56\ ft$

OR



$H_2 = 30.56\ ft$

$$H'_3 := \frac{BLANKET + HGL + 3ft}{BLANKET} \cdot (Hwall_1 - 3ft) - HEAD$$

&

$H''_3 := Hwall_1 + HGL - HEAD$

$H_3 := if(Hwall_1 > BLANKET, H''_3, H'_3)$

$H_3 = 38.978\ ft$

$H'_3 = 38.978\ ft$

$H''_3 = 42.06\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$W_s = 1974\ \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3]$

$W_w = 3335\ \frac{lb}{ft^2}$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_1 := if(W_s > W_w, W_s, W_w)$ $\quad$ $W_1 = 3335\ \frac{lb}{ft^2}$

$BLANKET + HGL = 46.5\ ft$

NOTE: The H`$_3$ equation (equated above) was modified to account for the blanket terminating 3 feet below the grade elevation.

$$H'_3 = \frac{BLANKET + HGL + 3ft}{BLANKET} \cdot (Hwall_1 - 3ft) - HEAD$$

$BLANKET + HGL + 3ft = 49.5\ ft$

$BLANKET = 35\ ft$

$\frac{BLANKET + HGL + 3ft}{BLANKET} \cdot Hwall_1 = 43.221\ ft$ $\quad$ $\gamma_w \cdot H_3 = 2432.209\ psf$

WALL 1 (East Wall) SHEAR

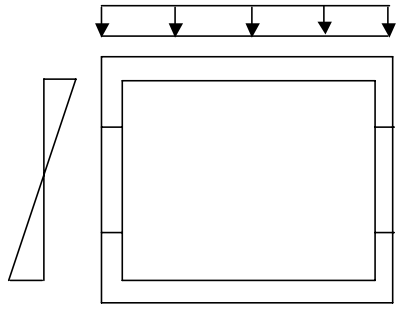
$L_2 = 34.5\ ft$

$THRUST_1 := \frac{W_1 \cdot L_2}{2}$ $THRUST_1 = 57529\ \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis (if necessary)

$V'_{u1} := W_1 \cdot \frac{L_1}{2}$

$V'_{u1} = 35018\ \frac{lb}{ft}$



$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1\right)$ $V_{u1} = 25707\ \frac{lb}{ft}$

$D_2 = 2\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$d_1 = 21.5\ in$

$L_1 = 21\ ft$

$A_g := D_1 \cdot 12 \frac{in}{ft}$ $A_g = 288\ \frac{in^2}{ft}$

$\frac{L_1}{d_1} = 11.721$ EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1\right)\right]$ ACI EQ (11-4)

$\phi V_{n1} = 31085\ \frac{lb}{ft}$

$Check1 := if(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD")$

$Check1 = "NO GOOD"$

$\frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 0.1$

Factor of Safety

$FS_{S1} := \frac{\phi V_{n1}}{V_{u1}}$ $FS_{S1} = 1.209$

From Monograph figure 13, using interpolation

Ry $\frac{0.34 - 0.25}{0.375 - 0.25} \cdot (0.2647 - 0.1950) + 0.1950 = 0.245$ $Ry := 0.2452 \cdot W_1 \cdot (ELEV_1 - 727.44ft)$ $Ry = 24.99\ \frac{kip}{ft}$

$V'_{u1} := 25 \frac{kips}{ft}$

$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1\right)$ $V_{u1} = 18353\ \frac{lb}{ft}$

$Check1 := if(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO GOOD")$

$Check1 = "OKAY"$

Factor of Safety

$FS_{S1y} := \frac{\phi V_{n1}}{V_{u1}}$ $FS_{S1y} = 1.694$

From Monograph figure 13, using interpolation

Rx $\frac{0.34 - 0.25}{0.375 - 0.25} \cdot (0.2120 - 0.1517) + 0.1517 = 0.195$ $Rx := 0.1951 \cdot W_1 \cdot (ELEV_1 - 727.44ft)$ $Rx = 19.884\ \frac{kip}{ft}$

$V'_{u1} := 20 \frac{kips}{ft}$

$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1\right)$ $V_{u1} = 14683\ \frac{lb}{ft}$

The thrust present at y/b =0.4 is as follows:

$0.4 \cdot (30.56\,ft) = 12.224\ ft$ Distance from bottom where the maximum shear occurs using plate analysis.

$W_1 = 3335.003\,psf$

$$W_1 - \frac{(W_1 - 0psf) \cdot 12.2ft}{30.56ft} = 2003.621\,psf$$

Thrust present 12.2 ft from bottom of wall.

$$THRUST := \frac{2003psf \cdot L_2}{2} \qquad THRUST = 34551.75\ \frac{lb}{ft} \qquad d_1 = 21.5\,in \qquad A_g = 2\ ft$$

$$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1\right)\right]$$

ACI EQ (11-4)

$$\frac{THRUST}{2000 \cdot A_g} \cdot \frac{in^2}{lb} = 0.06$$

$$\phi V_{n1} = 29957.828\ \frac{lb}{ft} \qquad V_{u1} = 14682.54\ \frac{lb}{ft} \qquad \frac{THRUST \cdot \frac{in^2}{lb}}{A_g} = 119.971$$

$$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, "OKAY", "NO\ GOOD"\right)$$

$Check1 = "OKAY"$

Factor of Safety

$$FS_{S1x} := \frac{\phi V_{n1}}{V_{u1}}$$

$FS_{S1x} = 2.04$

$$FS_{S1} := \min\left(FS_{S1x}, FS_{S1y}\right) \qquad FS_{S1} = 1.694$$

SHEAR FACTOR OF SAFETY FOR WALL 1

Wall 1 Bending, East Wall-Vertical Reinforcement

The value of a/b = 0.34. Interpolate the values provided in Figure 13 monograph.

My, vertical reinf. mid span moment

$$\frac{0.34 - 0.25}{0.375 - 0.25}\cdot(0.0063 - 0.0031) + 0.0031 = 0.0054 \quad 0.0054\cdot W_1\cdot\left(ELEV_1 - 727.44ft\right)^2 = 16.819\,kip\cdot\frac{ft}{ft}$$

My, vertical reinf. end of span moment

$$\frac{0.34 - 0.25}{0.375 - 0.25}\cdot(0.0200 - 0.0107) + 0.0107 = 0.0174 \quad 0.0174\cdot W_1\cdot\left(ELEV_1 - 727.44ft\right)^2 = 54.194\,kip\cdot\frac{ft}{ft}$$

Vertical Steel

Simple Conservative Analysis

Plate Analysis

$NegMidPlateMoment := 16.82kip\cdot\frac{ft}{ft}$ My

$PosEdgePlateMoment := 54.2kip\cdot\frac{ft}{ft}$ My

#5's @ 12 Inside Face (mid-plate moment)

$A_{mpv} := 0.307\frac{in^2}{ft}$ $b := 12\frac{in}{ft}$ $a_{mpv} := \frac{A_{mpv}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mpv} = 0.401\ in$

$\phi M_{mpv} := \phi_B\cdot A_{mpv}\cdot F_y\cdot\left(d_1 - \frac{a_{mpv}}{2}\right)$ $\phi M_{mpv} = 21.8\frac{kip\cdot ft}{ft}$

#6's @ 12 in. outside face (edge-plate moment)

$A_{epv} := 0.442\frac{in^2}{ft}$ $a_{epv} := \frac{A_{epv}\cdot F_y}{0.85f'_c\cdot b}$ $a_{epv} = 0.578\ in$

$\phi M_{epv} := \phi_B\cdot A_{epv}\cdot F_y\cdot\left(d_1 - \frac{a_{epv}}{2}\right)$ $\phi M_{epv} = 31.3\frac{kip\cdot ft}{ft}$

$CheckMPV := if\left(\phi M_{mpv} > 1.5\cdot NegMidPlateMoment, "OKAY", "NO\ GOOD"\right)$ CheckMPV = "NO GOOD"

Factor of Safety $FS_{MPV} := \frac{\phi M_{mpv}}{NegMidPlateMoment}$ $FS_{MPV} = 1.296$

$CheckEPV := if\left(\phi M_{epv} > 1.5\cdot PosEdgePlateMoment, "OKAY", "NO\ GOOD"\right)$ CheckEPV = "NO GOOD"

Factor of Safety $FS_{EPV} := \frac{\phi M_{epv}}{PosEdgePlateMoment}$ $FS_{EPV} = 0.577$

Wall 1 Bending, East Wall-Horizontal Reinforcement

The value of a/b = 0.34. Interpolate the values provided in Figure 13 monograph.

Mx, horizontal reinf. mid span moment

$$\frac{0.34 - 0.25}{0.375 - 0.25}\cdot(0.0098 - 0.0057) + 0.0057 = 0.0087 \qquad 0.0087\cdot W_1\cdot\left(ELEV_1 - 727.44ft\right)^2 = 27.097\,kip\cdot\frac{ft}{ft}$$

Mx, horizontal reinf. end of span moment

$$\frac{0.34 - 0.25}{0.375 - 0.25}\cdot(0.0207 - 0.0114) + 0.0114 = 0.0181 \qquad 0.0181\cdot W_1\cdot\left(ELEV_1 - 727.44ft\right)^2 = 56.374\,kip\cdot\frac{ft}{ft}$$

Horizontal Steel Plate Analysis

Simple Conservative Analysis

$NegMidPlateMoment2 := 27.1kip\cdot\frac{ft}{ft}$ Mx

$PosEdgePlateMoment2 := 56.4kip\cdot\frac{ft}{ft}$ Mx

#6's @ 12 in. inside face (mid-plate moment)

$A_{mph} := 0.442\,\frac{in^2}{ft}$ $\quad a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $\quad a_{mph} = 0.578\,in$

$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right)$ $\quad \phi M_{mph} = 31.3\,\frac{kip\cdot ft}{ft}$

#9's @ 6 in. outside face (edge-plate moment)

$A_{eph} := 2\,\frac{in^2}{ft}$ $\quad a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $\quad a_{eph} = 2.614\,in$

$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right)$ $\quad \phi M_{eph} = 134.6\,\frac{kip\cdot ft}{ft}$

$CheckMPH := if\left(\phi M_{mph} > 1.5\cdot NegMidPlateMoment2, "OKAY", "NO GOOD"\right)$ $\quad CheckMPH = "NO\ GOOD"$

Factor of Safety $\quad FS_{MPH} := \frac{\phi M_{mph}}{NegMidPlateMoment2}$ $\quad FS_{MPH} = 1.153$

$CheckEPH := if\left(\phi M_{eph} > 1.5\cdot PosEdgePlateMoment2, "OKAY", "NO GOOD"\right)$ $\quad CheckEPH = "OKAY"$

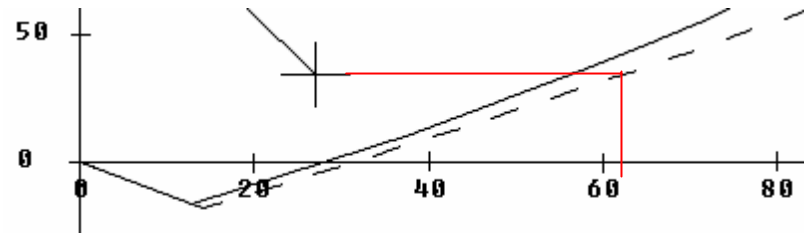
Factor of Safety $\quad FS_{EPH} := \frac{\phi M_{eph}}{PosEdgePlateMoment2}$ $\quad FS_{EPH} = 2.387$

TAKE INTO ACCOUNT THE THRUST THAT IS PRESENT. THRUST PROVIDES BENEFIT.

$THRUST = 34551.75 \frac{lb}{ft}$

$NegMidPlateMoment2 := 27.1kip \cdot \frac{ft}{ft}$

```
100 5th3w1MPH7695 M.Parks
110 wall 1 MPM HGL=769.5
200 1
210 3.00 40.0 0.375
300 12.0 X 24 with 1 bar inside face
400 12.0 24
500 1
510 1 0.442 21.625 6.0 6.0
600 34.5 27.1
```



From interaction diagram:

Nominal Strength

$M_{mph_int} := 62 \frac{kip \cdot ft}{ft}$

$FoS' := \frac{M_{mph_int}}{NegMidPlateMoment2}$

$FoS' = 2.288$

Based on interaction diagram, the section is adequate

OKAY.

ANALYZE WALL 1 FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\,\frac{in}{ft}$

Evaluate the wall 5 feet from bottom

$$W_1 - \frac{(W_1 - 0psf)\cdot 5ft}{30.56ft} = 2789.355\,psf$$ Load present on the wall.

The mid-span negative moment $M_{mid} := \frac{2790psf\cdot L_1^{\,2}}{24}$ $M_{mid} = 51.266\,\frac{kip\cdot ft}{ft}$

The end-span positive moment $M_{end} := \frac{2790psf\cdot L_1^{\,2}}{12}$ $M_{end} = 102.533\,\frac{kip\cdot ft}{ft}$

#6's @ 12 in. inside face (mid-plate moment) $A_{mph} := 0.442\,\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 0.578\,in$

$$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right)$$ $\phi M_{mph} = 31.3\,\frac{kip\cdot ft}{ft}$

#9's @ 6 in. outside face (edge-plate moment) $A_{eph} := 2\,\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 2.614\,in$

$$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right)$$ $\phi M_{eph} = 134.6\,\frac{kip\cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO GOOD")$ $CheckMPH = "NO GOOD"$

Factor of Safety $FS_{MPH} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH} = 0.61$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO GOOD")$ $CheckEPH = "NO GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 1.313$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_1 = 3335.003\,psf$

$$W_1 - \frac{(W_1 - 0psf)\cdot 5ft}{30.56ft} = 2789.355\,psf$$ Thrust present 5 ft from bottom of wall.

$$THRUST := \frac{2790psf \cdot L_2}{2}$$

$$THRUST = 48127.5 \frac{lb}{ft}$$

$d_1 = 21.5\,in$ $A_g = 2\ ft$

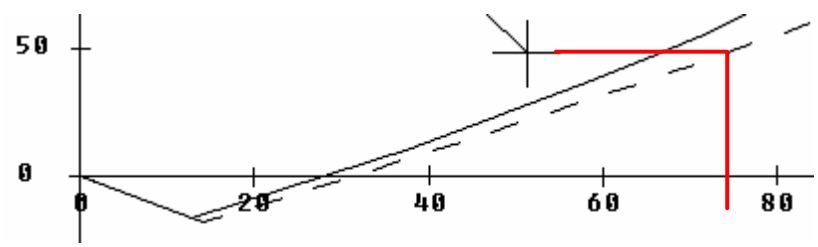
$C_{c1} = 2\,in$

Information for CASTR:

$$THRUST = 48.13 \frac{kip}{ft}$$

$$M_{mid} = 51.266 \frac{kip \cdot ft}{ft}$$

```
100 5thw1MPH7695A M.Parks
110 wall 1 MPM HGL=769.5
200 1
210 3.00 40.0 0.375
300 12.0 X 24 with 1 bar inside face
400 12.0 24
500 1
510 1 0.442 21.625 6.0 6.0
600 48.13 51.3
```



From interaction diagram:

Nominal Strength

$$M_{mph_int} := 77 \frac{kip \cdot ft}{ft}$$

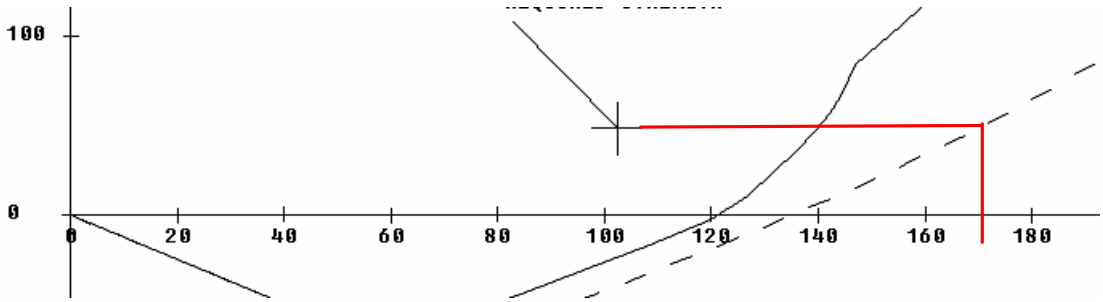
$$FS_{MPH1} := \frac{M_{mph_int}}{M_{mid}}$$

$$FS_{MPH1} = 1.502$$

Must evaluate the end-of-span location as well.

Information for CASTR: $THRUST = 48.13 \frac{kip}{ft}$ $M_{end} = 102.533 \frac{kip \cdot ft}{ft}$

```
100 5thW1EPH7695A M.Parks
110 Wall 1 MPM HGL=769.5
200 1
210 3.00 40.0 0.375
300 12.0 x 24 with 1 bar outside face
400 12.0 24
500 1
510 1 2.0 21.625 6.0 6.0
600 48.13 102.5
```



From interaction diagram:

Nominal Strength

$M_{eph_int} := 171 \frac{kip \cdot ft}{ft}$ $FS_{EPH1} := \frac{M_{eph_int}}{M_{end}}$ $FS_{EPH1} = 1.668$ Based on interaction diagram, the section is adequate

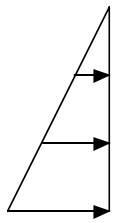
WALL 2, (North wall)

$K_o = 0.562$ $\quad$ $BLANKET = 35\ ft$ $\quad$ $HGL = 11.5\ ft$

$Hwall_2 = 35.06\ ft$ $\quad$ $\gamma_w = 62.4\ pcf$

Wall Loading

Soil or Soil & Water



$H_2 := Hwall_2$

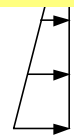
$H_2 = 35.06\ ft$

OR

$H_2 = 35.06\ ft$

$$H'_3 := \frac{BLANKET + HGL + 3ft}{BLANKET} \cdot (Hwall_2 - 3ft) - HEAD$$

&



$H''_3 := Hwall_2 + HGL - HEAD$

$H_3 := if(Hwall_2 > BLANKET, H''_3, H'_3)$

$H_3 = 46.56\ ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$

$H_3 := 45.34ft$ The base of wall is ABOVE the blanket base

$H'_3 = 45.342\ ft$

$H''_3 = 46.56\ ft$

$W_s = 2264\ \frac{lb}{ft^2}$

$$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3\right]$$

$W_w = 3864.95\ psf$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$W_2 := if(W_s > W_w, W_s, W_w)$ $\quad$ $W_2 = 3865\ \frac{lb}{ft^2}$

$BLANKET + HGL + 3ft = 49.5\ ft$ $\quad$ $K_o \cdot (\gamma - \gamma_w) \cdot H_2 = 1035.731\ psf$ $\quad$ $H'_3 = 45.342\ ft$

$BLANKET = 35\ ft$ $\quad$ $\gamma_w \cdot H_3 = 2829.216\ psf$

NOTE: WALL 2 CONTAINS SUPPORTS AS SHOWN ON SHEET 4 OF 12, SECTION "B-B". THE VALUES OF DEMAND SHEAR AND MOMENT WERE OBTAINED USING FINITE ELEMENT SOFTWARE, RAM ANALYSIS. THE NORTH WALL WAS MODELED IN RAM ANALYSIS AS A 2D WALL WITH THE SUPPORTS PLACED AT X, Y COORDINATES (16,10) AND (20,10). THE RAM ANALYSIS SOFTWARE FILE NAME IS **5thStreetNorthWall withsupports.AVW** Excerpts from this file are at the end of this document.

Calculate two points to use in RAM Advanse for lateral loading:

Bottom: $\gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma - \gamma_w) \cdot (H_2 - 0.5ft) + \gamma_w \cdot (H_3 - 0.5ft)\right] = 3819\ psf$

Top: $\gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma - \gamma_w) \cdot (0.5ft) + \gamma_w \cdot (0)\right] = 14.771\ psf$

Define equation using y=mx+b to determine the intermediate points (straight line)

NOTE 1
Where "Note 1" is shown for Wall 2 analysis, the assigned demand value was obtained from RAM Advanse using a W_2 input value = 4114 psf. This load value was obtained using an HGL=770.7 input value for the n500+3 event. The HGL value for the n500+3 event was subsequently lowered by EC-GD from HGL=770.7 to HGL = 769.5 resulting in a lower W_2 = 3865 value. Since the wall was adequate for the HGL= 770.7 event, the RAM analysis was not reaccomplished for the lower loads.

WALL 2, (North Wall)

SHEAR

$THRUST_2 := \frac{W_1 \cdot L_1}{2}$ $\quad THRUST_2 = 35018 \frac{lb}{ft}$

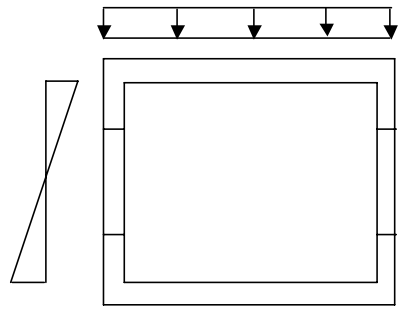
Thrust to be used in CASTR for thrust analysis (if necessary)

THE VALUE OF SHEAR, V`u2 WAS OBTAINED FROM RAM ANALYSIS, Vmax.

SEE NOTE 1

$V'_{u2} := 23 \frac{kip}{ft}$

$V'_{u2} = 23000 \frac{lb}{ft}$



$V_{u2} := 2 \frac{V'_{u2}}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2 \right)$ $\quad V_{u2} = 19278 \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$A_g := D_2 \cdot 12 \frac{in}{ft}$ $\quad A_g = 288 \frac{in^2}{ft}$

EM 1110-2-2104 EQ (5-1) Does not Apply

$\phi V_{n2} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_2 \right) \right]$ ACI EQ (11-4)

$d_2 = 21.5\,in$

$\frac{THRUST_2 \cdot \frac{in^2}{lb}}{A_g} = 121.589$

$\phi V_{n2} = 29981 \frac{lb}{ft}$

$Check2 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$ $\quad Check2 = "OKAY"$

Factor of Safety $\quad FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$ $\quad FS_{S2} = 1.555$

Need to check punching shear at the supports. Ref. ACI 11.12.1.2

$width := 18in$ support width (square)

Note: The centerline distance between the supports is approximately 4 ft.

ACI 318-02 11.10.1 Design for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in 11.12. Per 11.12.2.1 for nonprestressed slabs and footings, Equation 11-35 controls for Vc (smallest)

$b_o := (width + d_2) \cdot 4$ $\quad b_o = 158\,in$

Note that the wall contains shear reinforcement per V-8 reinforcement detail. The limiting value of Vc becomes: $V_c = 6 \cdot \sqrt{f'_c} \cdot b_o \cdot d_2$

Eqn 11-35:

$V_c := 4 \cdot \sqrt{f'_c} \cdot b_o \cdot d_2 \cdot \frac{lb^{0.5}}{in}$ $\quad V_c = 744.245\,kip$ Shear capacity when shear reinforcement is not considered.

The demand shear present at the support from Ram Advanse Output portion of this document: $V_u = 403.6kip$ (Node 430 FZ)

$\frac{1.0 \cdot V_c}{403.6kip} = 1.844$ OKAY

WALL 2 VERTICAL STEEL

MOMENTS USING FIGURE 13 MONOGRAPH (SUPPORTS NOT PRESENT ARE PROVIDED FOR COMPARISON ONLY-NOT USED IN THE ANALYSIS OF WALL 2). MONOGRAPH VALUES OF MOMENT ARE COMPARABLE TO THE RAM ANALYSIS MODEL WHEN THE WALL IS MODELED WITHOUT SUPPORTS.

Figure 13 monograph

$ELEV_1 = 758\ ft$ ratio = 1/2

midspan moment $0.0116 \cdot W_2 \cdot (ELEV_1 - 722.94ft)^2 = 55.109\ kip \cdot \frac{ft}{ft}$

$ELEV_1 - 722.94ft = 35.06\ ft$

end of span moment $0.0320 \cdot W_2 \cdot (ELEV_1 - 722.94ft)^2 = 152.026\ kip \cdot \frac{ft}{ft}$

THE SUPPORTS LOWER SIGNIFICANTLY THE EDGE PLATE NEGATIVE MOMENT. (COMPARE MONOGRAPH MOMENTS WITH RAM ANALYSIS MOMENT VALUES.)

THE VALUES OF MOMENT (M11) WERE OBTAINED FROM RAM ANALYSIS. M11 IS THE MOMENT ABOUT THE LOCAL "1" AXIS. THE "1" AXIS IS HORIZONTAL.

Wall 2 Bending (North Wall)

Simple Conservative Analysis — Vertical Steel — Plate Analysis — **SEE NOTE 1**

$NegMidPlateMoment := 38kip \cdot \frac{ft}{ft}$ My

$PosEdgePlateMoment := 52kip \cdot \frac{ft}{ft}$ My

#8's @ 12 in. + #6's@12 in.
Inside Face
(mid-plate moment)

$A_{mpv} := 1.227 \frac{in^2}{ft}$ $b := 12 \frac{in}{ft}$ $a_{mpv} := \frac{A_{mpv} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mpv} = 1.604\ in$

$\phi M_{mpv} := \phi_B \cdot A_{mpv} \cdot F_y \cdot \left(d_1 - \frac{a_{mpv}}{2}\right)$ $\phi M_{mpv} = 84.7 \frac{kip \cdot ft}{ft}$

#10's @ 12 in. outside face
(edge-plate moment)

$A_{epv} := 1.267 \frac{in^2}{ft}$ $a_{epv} := \frac{A_{epv} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{epv} = 1.656\ in$

$\phi M_{epv} := \phi_B \cdot A_{epv} \cdot F_y \cdot \left(d_1 - \frac{a_{epv}}{2}\right)$ $\phi M_{epv} = 87.3 \frac{kip \cdot ft}{ft}$

$CheckMPV := if(\phi M_{mpv} > 1.5 \cdot NegMidPlateMoment, "OKAY", "NO\ GOOD")$ $CheckMPV = "OKAY"$

Factor of Safety $FS_{MPV2} := \frac{\phi M_{mpv}}{NegMidPlateMoment}$ $FS_{MPV2} = 2.228$

$CheckEPV := if(\phi M_{epv} > 1.5 \cdot PosEdgePlateMoment, "OKAY", "NO\ GOOD")$ $CheckEPV = "OKAY"$

Factor of Safety $FS_{EPV2} := \frac{\phi M_{epv}}{PosEdgePlateMoment}$ $FS_{EPV2} = 1.679$

Wall 2 Bending, North Wall

HORIZONTAL STEEL

MOMENTS USING FIGURE 13 MONOGRAPH (SUPPORTS NOT PRESENT. USE FOR COMPARISON ONLY-NOT USED IN THE ANALYSIS OF WALL 2). NOMOGRAPH VALUES OF MOMENT ARE COMPARABLE TO THE RAM ANALYSIS MODEL WHEN THE WALL IS MODELED WITHOUT SUPPORTS.

ratio = 1/2 $0.0119 \cdot W_2 \cdot (ELEV_1 - 722.94ft)^2 = 56.535\,kip \cdot \frac{ft}{ft}$ mid span moment

Figure 13 monograph $0.0269 \cdot W_2 \cdot (ELEV_1 - 722.94ft)^2 = 127.797\,kip \cdot \frac{ft}{ft}$ end of span moment

THE VALUES OF MOMENT (M33) WERE OBTAINED FROM RAM ANALYSIS. M33 IS THE MOMENT ABOUT THE LOCAL "3" AXIS. THE "3" AXIS IS VERTICAL.

Horizontal Steel

Simple Conservative Analysis

Plate Analysis **SEE NOTE 1**

$L_2 = 34.5\ ft$

$NegMidPlateMoment2 := 32.4kip \cdot \frac{ft}{ft}$ Mx

$PosEdgePlateMoment2 := 67kip \cdot \frac{ft}{ft}$ Mx

#10's @ 12 in. inside face (mid-plate moment)

$A_{mph} := 1.267 \frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mph} = 1.656\ in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_2 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 87.3 \frac{kip \cdot ft}{ft}$

#9's @ 6 in. outside face (edge-plate moment)

$A_{eph} := 2.0 \frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{eph} = 2.614\ in$

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_2 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 134.6 \frac{kip \cdot ft}{ft}$

$CheckMPH := if(\phi M_{mph} > 1.5 \cdot NegMidPlateMoment2, "OKAY", "NO\ GOOD")$ $CheckMPH = "OKAY"$

Factor of Safety $FS_{MPH2} := \frac{\phi M_{mph}}{NegMidPlateMoment2}$ $FS_{MPH2} = 2.695$

$CheckEPH := if(\phi M_{eph} > 1.5 \cdot PosEdgePlateMoment2, "OKAY", "NO\ GOOD")$ $CheckEPH = "OKAY"$

Factor of Safety $FS_{EPH2} := \frac{\phi M_{eph}}{PosEdgePlateMoment2}$ $FS_{EPH2} = 2.009$

WALL 3, (South wall)

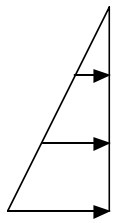
$K_o = 0.562$ $\quad$ $BLANKET = 35\ ft$ $\quad$ $HGL = 11.5\ ft$

$Hwall_3 = 30.56\ ft$

The south wall was analyzed using nomographs. RAM Advanse was not used analyze the south wall.

Wall Loading

Soil or Soil & Water



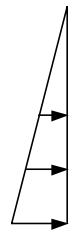
$H_2 := Hwall_3$

$H_2 = 30.56\ ft$

OR

$H_2 = 30.56\ ft$

&



$$H'_3 := \frac{BLANKET + HGL}{BLANKET} \cdot Hwall_3 - HEAD$$

$$H''_3 := Hwall_3 + HGL - HEAD$$

$$H_3 := if\left(Hwall_3 > BLANKET, H''_3, H'_3\right)$$

$H_3 = 40.601\ ft$

$H'_3 = 40.601\ ft$

$H''_3 = 42.06\ ft$

$$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_2$$

$$W_s = 1973.8\ \frac{lb}{ft^2}$$

$$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$$

$$W_w = 3436\ \frac{lb}{ft^2}$$

Check to determine if water to top of wetwell with reduction factor, γ_X, or soil loading is worst case

$$W_3 := if\left(W_s > W_w, W_s, W_w\right)$$

$$W_3 = 3436\ \frac{lb}{ft^2}$$

WALL 3, South Wall

SHEAR

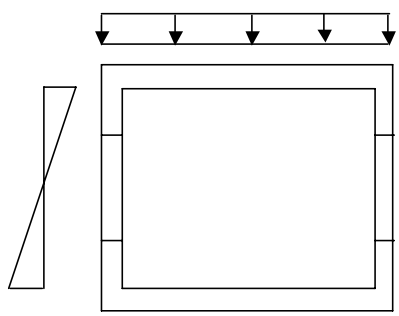
$L_3 = 34.5\ ft$

$$THRUST_1 := \frac{W_1 \cdot L_1}{2} \qquad THRUST_1 = 35018\ \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis (if necessary)

$$V'_{u1} := W_3 \cdot \frac{L_3}{2}$$

$$V'_{u1} = 59276\ \frac{lb}{ft}$$



$$V_{u1} := 2 \frac{V'_{u1}}{L_3} \cdot \left(\frac{L_3}{2} - \frac{D_2}{2} - d_3\right)$$

$$V_{u1} = 49683\ \frac{lb}{ft}$$

$D_2 = 2\ ft$

$d_1 = 1.792\ ft$

$L_1 = 21\ ft$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$A_g := D_1 \cdot 12\ \frac{in}{ft} \qquad A_g = 288\ \frac{in^2}{ft}$$

$$\frac{L_1}{d_1} = 11.721$$

EM 1110-2-2104 EQ (5-1) Does not Apply

$$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1\right)\right]$$

ACI EQ (11-4)

$$\phi V_{n1} = 29981\ \frac{lb}{ft}$$

$$Check1 := if\left(\phi V_{n1} > 1.5 V_{u1}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$Check1 = \text{"NO GOOD"}$

Factor of Safety

$$FS_{S3} := \frac{\phi V_{n1}}{V_{u1}}$$

$FS_{S3} = 0.603$

Analyze wall as 1ft wide strip, considering plate reactions.

From Monograph figure 13, using interpolation

Ry $\frac{0.56 - 0.5}{0.75 - 0.5} \cdot (0.3828 - 0.3225) + 0.3225 = 0.337$ $Ry := 0.337 \cdot W_3 \cdot (ELEV_1 - 727.44ft)$ $Ry = 35.39 \frac{kip}{ft}$

Rx $\frac{0.56 - 0.5}{0.75 - 0.5} \cdot (0.2616 - 0.2450) + 0.2450 = 0.249$ $Rx := 0.249 \cdot W_3 \cdot (ELEV_1 - 727.44ft)$ $Rx = 26.148 \frac{kip}{ft}$

$V'_{u1} := 35.4 \frac{kips}{ft}$

$V_{u1} := 2 \frac{V'_{u1}}{L_3} \cdot \left(\frac{L_3}{2} - \frac{D_2}{2} - d_3 \right)$ $V_{u1} = 29671 \frac{lb}{ft}$

$Check1 := if(\phi V_{n1} > 1.5V_{u1}, "OKAY", "NO GOOD")$ $Check1 = "NO GOOD"$

Factor of Safety $FS_{S3} := \frac{\phi V_{n1}}{V_{u1}}$ $FS_{S3} = 1.01$

SHEAR REINFORCEMENT IS PRESENT IN THE NORTH AND SOUTH WALLS. THE SHEAR REINFORCEMENT IS SHOWN ON SHEET 4, REBAR CALLOUT V-8.

$\phi V_c := \phi V_{n1}$

$V_{u1} = 29671.014 \frac{lb}{ft}$ $\phi V_{n1} = 29.981 \frac{kip}{ft}$

$\phi Vs := 1.5V_{u1} - \phi V_c$ $\phi Vs = 14.526 \frac{kip}{ft}$ The amount of shear resistance that must be provide by the steel in order to have a FS =1.5

For a 1 foot section

$Av := \frac{0.2in^2}{ft}$ shear reinforcement area- #4 at 12 in per V-8 detail on sheet 4

$s_2 := 12in$ shear reinforcement spacing

$\phi V_s := \frac{Av \cdot d_3 \cdot F_y}{s_2}$ $\phi V_s = 14.333 \frac{1}{ft} kip$

$FS_{S3} := \frac{\phi V_{n1} + \phi V_s}{V_{u1}}$ $FS_{S3} = 1.494$ OKAY

Wall 3 Bending, South Wall, vertical reinf.

The value of a/b = 0.56. Interpolate the values provided in Figure 13 monograph.

My, vertical reinf. mid span moment

$$\frac{0.56 - 0.5}{0.75 - 0.5}\cdot(0.0198 - 0.0116) + 0.0116 = 0.0136 \qquad 0.0136\cdot W_3\cdot\left(ELEV_1 - 727.44ft\right)^2 = 43.645\,kip\cdot\frac{ft}{ft}$$

My, vertical reinf. end of span moment

$$\frac{0.56 - 0.5}{0.75 - 0.5}\cdot(0.0505 - 0.032) + 0.032 = 0.0364 \qquad 0.0364\cdot W_3\cdot\left(ELEV_1 - 727.44ft\right)^2 = 116.815\,kip\cdot\frac{ft}{ft}$$

Vertical Steel

Plate Analysis

$NegMidPlateMoment := 43.65\,kip\cdot\frac{ft}{ft}$ My

$PosEdgePlateMoment := 116.82\,kip\cdot\frac{ft}{ft}$ My

#8's @ 12 in. + #6's@12 in.
Inside Face
(mid-plate moment)

$A_{mpv} := 1.227\,\frac{in^2}{ft}$ $b := 12\,\frac{in}{ft}$ $a_{mpv} := \frac{A_{mpv}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mpv} = 1.604\,in$

$\phi M_{mpv} := \phi_B\cdot A_{mpv}\cdot F_y\cdot\left(d_1 - \frac{a_{mpv}}{2}\right)$ $\phi M_{mpv} = 84.7\,\frac{kip\cdot ft}{ft}$

#10's @ 12 in. outside face
(edge-plate moment)

$A_{epv} := 1.27\,\frac{in^2}{ft}$ $a_{epv} := \frac{A_{epv}\cdot F_y}{0.85f'_c\cdot b}$ $a_{epv} = 1.66\,in$

$\phi M_{epv} := \phi_B\cdot A_{epv}\cdot F_y\cdot\left(d_1 - \frac{a_{epv}}{2}\right)$ $\phi M_{epv} = 87.5\,\frac{kip\cdot ft}{ft}$

$CheckMPV := if\left(\phi M_{mpv} > 1.5\cdot NegMidPlateMoment, "OKAY", "NO\ GOOD"\right)$ $CheckMPV = "OKAY"$

Factor of Safety $FS_{MPV3} := \frac{\phi M_{mpv}}{NegMidPlateMoment}$ $FS_{MPV3} = 1.939$

$CheckEPV := if\left(\phi M_{epv} > 1.5\cdot PosEdgePlateMoment, "OKAY", "NO\ GOOD"\right)$ $CheckEPV = "NO\ GOOD"$

Factor of Safety $FS_{EPV3} := \frac{\phi M_{epv}}{PosEdgePlateMoment}$ $FS_{EPV3} = 0.749$

Wall 3 Bending, South Wall, horizontal reinf.

The value of a/b = 0.56. Interpolate the values provided in Figure 13 monograph.

My, horizontal reinf. mid span moment

No change, although the max. moment changed position, up from y/b=0.4 to y/b=0.6

$$0.0119 \cdot W_3 \cdot (ELEV_1 - 727.44ft)^2 = 38.19\ kip \cdot \frac{ft}{ft}$$

My, vertical reinf. end of span moment

$$\frac{0.56 - 0.5}{0.75 - 0.5} \cdot (0.0302 - 0.0269) + 0.0269 = 0.0277$$

$$0277 \cdot W_3 \cdot (ELEV_1 - 727.44ft)^2 = 88.895\ kip \cdot \frac{ft}{ft}$$

Horizontal Steel $\quad NegMidPlateMoment2 := 38.19 kip \cdot \frac{ft}{ft}$ Mx

Simple Conservative Analysis $\quad PosEdgePlateMoment2 := 88.9 kip \cdot \frac{ft}{ft}$ Mx

#10's @ 12 in. inside face (mid-plate moment)

$$A_{mph} := 1.267 \frac{in^2}{ft} \qquad a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{mph} = 1.656\ in$$

$$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_1 - \frac{a_{mph}}{2}\right) \qquad \phi M_{mph} = 87.3 \frac{kip \cdot ft}{ft}$$

#9's @ 6 in. outside face (edge-plate moment)

$$A_{eph} := 2 \frac{in^2}{ft} \qquad a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b} \qquad a_{eph} = 2.614\ in$$

$$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_1 - \frac{a_{eph}}{2}\right) \qquad \phi M_{eph} = 134.6 \frac{kip \cdot ft}{ft}$$

$$CheckMPH := if\left(\phi M_{mph} > 1.5 \cdot NegMidPlateMoment2, "OKAY", "NO\ GOOD"\right) \qquad CheckMPH = "OKAY"$$

Factor of Safety $\quad FS_{MPH3} := \frac{\phi M_{mph}}{NegMidPlateMoment2} \qquad FS_{MPH3} = 2.286$

$$CheckEPH := if\left(\phi M_{eph} > 1.5 \cdot PosEdgePlateMoment2, "OKAY", "NO\ GOOD"\right) \qquad CheckEPH = "OKAY"$$

Factor of Safety $\quad FS_{EPH3} := \frac{\phi M_{eph}}{PosEdgePlateMoment2} \qquad FS_{EPH3} = 1.514$

Wall 3, con't

Thrust available.

The thrust present at y/b =0.4 is as follows:

$0.4 \cdot (30.56\,\text{ft}) = 12.224\ \text{ft}$ Distance from bottom where the maximum shear occurs using plate analysis.

$W_3 = 3436.305\,\text{psf}$

$$W_3 - \frac{(W_3 - 0\text{psf}) \cdot 12.2\text{ft}}{30.56\,\text{ft}} = 2064.482\,\text{psf}$$

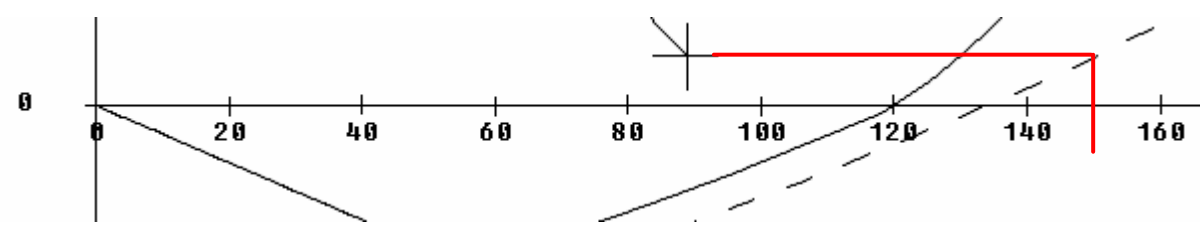
Thrust present 12.2 ft from bottom of wall.

$$\text{THRUST} := \frac{2064\text{psf} \cdot L_1}{2} \qquad \text{THRUST} = 21672\ \frac{\text{lb}}{\text{ft}}$$

Information for CASTR: $\text{THRUST} = 21672\ \frac{\text{lb}}{\text{ft}}$ $\text{PosEdgePlateMoment2} = 88.9\ \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$

From CASTR Program 5th3w3EPH26deg.*dat*

```
100 5thw3EPH7695 M.Parks
110 wall 3 EPH HGL=769.5
200 1
210 3.00 40.0 0.375
300 12.0 x 24 with 1 bar outside face
400 12.0 24
500 1
510 1 2.0 21.44 6.0 6.0
600 21.6 88.9
```



From interaction diagram:

Nominal Strength

$$M_{eph_int} := 150\ \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \qquad FS_{EPH3} := \frac{M_{eph_int}}{\text{PosEdgePlateMoment2}}$$

Based on interaction diagram, the section is adequate

$$FS_{EPH3} = 1.687$$

ANALYZE WALL 3 FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries all of the lateral loads $b := 12\,\frac{in}{ft}$

Evaluate the wall 5 feet from bottom

$$W_3 - \frac{(W_3 - 0psf)\cdot 5ft}{30.56ft} = 2874.082\,psf$$

Load present on the wall.

Wall 3 height is 30.56 ft
Wall 1 height is 30.56 ft
Wall 2 height is 35.06 ft
Wall 4 height is 33.06 ft

The mid-span negative moment $M_{mid} := \frac{2874psf\cdot L_1^{\,2}}{24}$ $M_{mid} = 52.81\,\frac{kip\cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{2874psf\cdot L_1^{\,2}}{12}$ $M_{end} = 105.62\,\frac{kip\cdot ft}{ft}$

#10's @ 12 in. inside face (mid-plate moment) $A_{mph} := 1.267\,\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{mph} = 1.656\,in$

$$\phi M_{mph} := \phi_B\cdot A_{mph}\cdot F_y\cdot\left(d_1 - \frac{a_{mph}}{2}\right) \qquad \phi M_{mph} = 87.3\,\frac{kip\cdot ft}{ft}$$

#9's @ 6 in. outside face (edge-plate moment) $A_{eph} := 2\,\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph}\cdot F_y}{0.85f'_c\cdot b}$ $a_{eph} = 2.614\,in$

$$\phi M_{eph} := \phi_B\cdot A_{eph}\cdot F_y\cdot\left(d_1 - \frac{a_{eph}}{2}\right) \qquad \phi M_{eph} = 134.6\,\frac{kip\cdot ft}{ft}$$

$CheckMPH := if(\phi M_{mph} > 1.5\cdot M_{mid}, "OKAY", "NO\ GOOD")$ $CheckMPH = "OKAY"$

Factor of Safety $FS_{MPH} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH} = 1.653$

$CheckEPH := if(\phi M_{eph} > 1.5\cdot M_{end}, "OKAY", "NO\ GOOD")$ $CheckEPH = "NO\ GOOD"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH} = 1.275$

Evaluate the flexural capacity taking into account the thrust acting on the orthagonal wall at that elevation

$W_3 = 3436.305\,psf$

$$W_3 - \frac{(W_3 - 0psf)\cdot 5ft}{30.56ft} = 2874.082\,psf$$

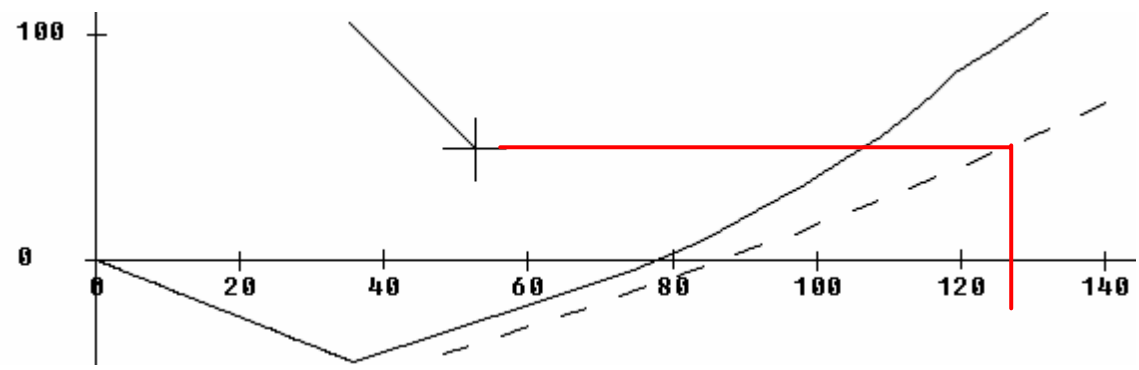
Thrust present 5 ft from bottom of wall.

$$THRUST := \frac{2874psf \cdot L_2}{2}$$ $$THRUST = 49576.5 \frac{lb}{ft}$$ $$d_1 = 21.5\,in \qquad A_g = 2\ ft$$

Information for CASTR: $THRUST = 49.58 \frac{kip}{ft}$ $M_{mid} = 52.81 \frac{kip \cdot ft}{ft}$

```
100 5thw3MPH7695 M.Parks
110 Wall 3 MPH HGL=769.5
200 1
210 3.00 40.0 0.375
300 12.0 X 24 with 1 bar outside face
400 12.0 24
500 1
510 1 1.267 21.44 6.0 6.0
600 49.5 52.8
```

$$M_{mid} = 52.81 \frac{kip \cdot ft}{ft}$$



From interaction diagram:

Nominal Strength

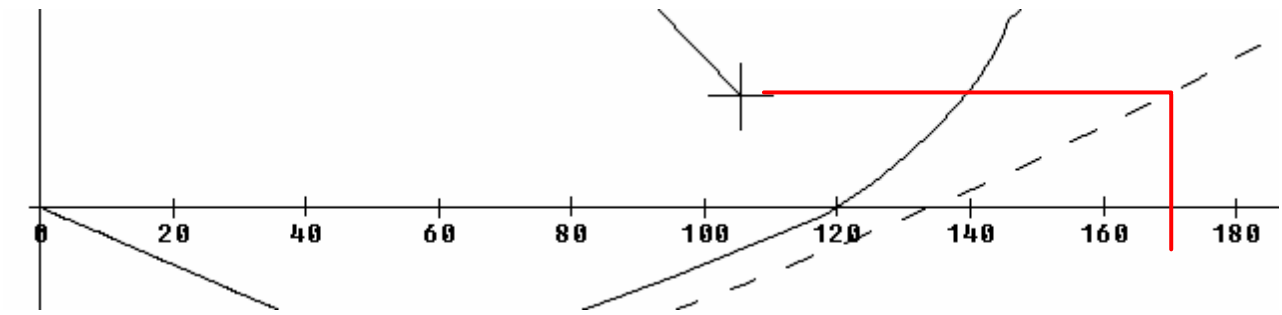
$$M_{mph_int} := 128 \frac{kip \cdot ft}{ft}$$ $$FS_{MPH3} := \frac{M_{mph_int}}{M_{mid}}$$ OKAY

$$FS_{MPH3} = 2.424$$

Evaluate the end-of-span location as well.

Information for CASTR: $THRUST = 49.58 \frac{kip}{ft}$ $M_{end} = 105.62 \frac{kip \cdot ft}{ft}$

```
100 5thW3EPH7695A M.Parks
110 Wall 3 EPH HGL=769.5
200 1
210 3.00 40.0 0.375
300 12.0 X 24 with 1 bar outside face
400 12.0 24
500 1
510 1 2.0 21.44 6.0 6.0
600 49.5 105.6
```



From interaction diagram:

Nominal Strength

$M_{eph_int} := 170 \frac{kip \cdot ft}{ft}$ $FS_{EPH3} := \frac{M_{eph_int}}{M_{end}}$ $FS_{EPH3} = 1.61$ Based on interaction diagram, the section is adequate

Uplift on Structure

Stability

$Wall1 := 30ft \cdot 2ft \cdot (758.50ft - 720ft)$ $\quad Wall1 \cdot 150pcf = 346.5\ kips$

$Wall2 := 42.5ft \cdot 2ft \cdot (758.5ft - 720.94ft)$ $\quad Wall2 \cdot 150pcf = 478.89\ kips$

$Wall3 := 42.5ft \cdot 2ft \cdot (758.5ft - 722.94ft)$ $\quad Wall3 \cdot 150pcf = 453.39\ kips$

$Wall4 := 30ft \cdot 2ft \cdot (758.50ft - 722.94ft)$ $\quad Wall4 \cdot 150pcf = 320.04\ kips$

$Int1 := 26ft \cdot 2ft \cdot (758.50ft - 724.94ft)$ $\quad Int1 \cdot 150pcf = 261.768\ kips$

$Int2 := 32.5ft \cdot 2ft \cdot (758.50ft - 725.94ft)$ $\quad Int2 \cdot 150pcf = 317.46\ kips$

$Int3 := 8ft \cdot 1.5ft \cdot (758.50ft - 724.94ft)$ $\quad Int3 \cdot 150pcf = 60.408\ kip$

$PumpFloor := 19ft \cdot 1.5ft \cdot 32.5ft$ $\quad PumpFloor \cdot 150pcf = 138.937\ kip$

$Gateslab := 26ft \cdot 2ft \cdot 8ft$ $\quad Gateslab \cdot 150pcf = 62.4\ kip$

$heel1 := 1.5ft \cdot 3ft \cdot 30ft$ $\quad heel1 \cdot 150pcf = 20.25\ kip$

$heel2 := 1.5ft \cdot 2ft \cdot 30ft$ $\quad heel2 \cdot 150pcf = 13.5\ kip$

$base := 32.5ft \cdot 2ft \cdot 19.5ft$ $\quad base \cdot 150pcf = 190.125\ kip$

Above ground concrete building structure:

Refer to "Motor Room Floor Plan" on Sheet 3 of the drawings.
The above grade structure has 1 ft thick walls and the height = 12 ft.

$Bldgwalls := [[(46 \cdot ft \cdot 23 \cdot ft) - (44.5 \cdot ft \cdot 21 \cdot ft)] \cdot 12 \cdot ft$ $\quad Bldgwalls \cdot 150pcf = 222.3\ kip$

$Bldgroof := 2.5in \cdot [26ft \cdot (38ft + 2in)]$ $\quad Bldgroof \cdot 150pcf = 31.01\ kip$

$$Structure := \begin{pmatrix} Wall1 + Wall2 + Wall3 + Wall4 + Int1 + Int2 + Int3 + PumpFloor + Gateslab + heel1 \ldots \\ + heel2 + base + Bldgwalls + Bldgroof \end{pmatrix} \cdot 150pcf$$

$Structure = 2916.979\ kip$

$Waterarea := 19.5ft \cdot 32.5ft$ $\quad Waterarea = 633.75\ ft^2$ $\quad HEAD := 3ft$

$Water := HEAD \cdot Waterarea \cdot 62.4pcf$ $\quad Water = 118.638\ kip$

$heel := 1.5ft \cdot 30ft$ $\quad heel = 45\ ft^2$

$upliftarea := 49.5ft \cdot 30ft$ $\quad upliftarea = 1485\ ft^2$

$H := 758ft - 720.94ft$ $\quad H = 37.06\ ft$

$volsoil := (758ft - 724.94ft) \cdot heel + (758ft - 722.94ft) \cdot heel$ $\quad volsoil = 3065.4\ ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 161.24\ kip$ $\quad volsoil \cdot \gamma_w = 191.281\ kip$

Uplift on Structure, con't

Head, if structural foundation in blanket.

$HGL = 11.5\ ft$ bottom of blanket at 720

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET}\cdot H\right)\cdot upliftarea\cdot\gamma_w$$

$U_1 = 4562.484\ kips$

$BLANKET = 35\ ft$

$HGL = 11.5\ ft$

$H = 37.06\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL)\cdot(upliftarea)\cdot\gamma_w\right]$$

$U_2 = 4499.764\ kips$

$$Uplift := if\left(H > BLANKET, U_2, U_1\right)$$

$Uplift = 4499.764\ kips$ $\left(\frac{BLANKET + HGL}{BLANKET}\cdot H\right) = 49.237\ ft$

$$Stability := \frac{Structure + Water + volsoil\cdot\left(\gamma - \gamma_w\right)}{Uplift - volsoil\cdot\gamma_w}$$

$Stability = 0.742$ $(H + HGL) = 48.56\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $StabilityCheck = "Not\ Okay"$

Consider Side Friction As Aid To Structure Factor of Safety Against Uplift

SOIL

$\phi := 26deg$ INTERNAL FRICTION ANGLE

$\gamma := 120pcf$ SOIL UNIT WEIGHT

$\gamma_w := 62.4pcf$ WATER UNIT WEIGHT

$\gamma_{sub} := (\gamma - \gamma_w)$ SUBMERGED SOIL UNIT WEIGHT $\gamma_{sub} = 57.6\,pcf$

$K_o := 1 - \sin(\phi)$ AT-REST SOIL COEFFICIENT. ASSUME THAT THE WALLS DO NOT MOVE

$K_o = 0.562$

$\mu := 0.25$ FRICTION COEFICIENT BETWEEN STRUCTURE AND SOIL.

$U := 3030psf$ UPLIFT PRESSURE ACTING ON BASE OF STRUCTURE

$c := 5ft$ ASSUMED CRACK DEPTH

STRUCTURE

$H := 30.5ft$ DEPTH OF STRUCTURE BELOW GRADE

$per := 30ft \cdot 2 + 46.5ft \cdot 2$ STRUCTURE PERIMETER $per = 153\ ft$

$W_s := 2917kip$ STRUCTURE DEAD WEIGHT

$W_{water} := 118.6kip$ WEIGHT OF WATER LOCATED IN STRUCTURE.

$Soil := 161.2kip$ WEIGHT OF SOIL LOCATED ON TOP OF THE BASE SLAB HEEL OR WALL STEP. EQUAL TO ZERO IF NO HEEL IS PRESENT.

UPLIFT PARAMETERS:

$U_{s_n500} := 4500kip$ UPLIFT FORCE FOR THE n500+3 CONDITION

V. Calculation of Pv

1. Effective Lateral Load that Contributes to Side Friction

$\gamma = 120\,\text{pcf}$ $\quad c = 5\ \text{ft}$ $\quad H = 30.5\ \text{ft}$

$R_1 := (H - c)\cdot\left(\frac{c + H}{2}\right)\cdot\gamma$ $\quad R_1 = 1688.162\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

$R_2 := (H - c)\cdot U\cdot\left(\frac{c}{2\cdot H} + \frac{1}{2}\right)$ $\quad R_2 = 1397.577\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

$R_1 - R_2 = 290.585\ \frac{s^2}{ft}\ \frac{lbf}{ft}$

2. Force Created By Side Friction, Pv

$K_o = 0.562$ $\quad \mu = 0.25$ $\quad per = 153\ \text{ft}$

$P_v := \left(R_1 - R_2\right)\cdot K_o\cdot\mu\cdot per$ $\quad P_v = 200.845\ \text{kip}$

<u>Stability Without the Aid of Side Friction, n500+3 Conditon</u>

$$Stability_{nofric_n500} := \frac{W_s + W_{water} + Soil}{U_{s_n500} - volsoil\cdot\gamma_w} \qquad Stability_{nofric_n500} = 0.742$$

<u>Stability with the Aid of Side Friction, n500+3 Conditon</u>

$$Stability_{fric_n500} := \frac{W_s + W_{water} + Soil + P_v}{U_{s_n500} - volsoil\cdot\gamma_w} \qquad Stability_{fric_n500} = 0.789$$

$0.789 - 0.742 = 0.047$ Increase in factor of safety when skin friction is taken into account

$Uplift = 4499.764\,kip$ $\quad volsoil = 3065.4\,ft^3$ $\quad Structure = 2916.979\,kip$ $\quad Water = 3687.382\,\frac{s^2}{ft}\,lbf$

Amount of additional weight required to achieve FS=1.10

$$Reqdadditional := \left(Uplift - volsoil \cdot \gamma_w\right) \cdot 1.10 - \left[Structure + Water + volsoil \cdot \left(\gamma - \gamma_w\right)\right]$$

$$Reqdadditional = 1527.147\,kip$$

Investigate the length of heel required to achieve the required FS = 1.10

$heel_{prop} := 10ft$ $\quad footprint := \left(49.5ft + 2heel_{prop}\right) \cdot \left(30ft + 2heel_{prop}\right)$ $\quad footprint = 3475\,ft^2$

$theel := 2.5ft$ $\quad V_{soil} := \left(footprint - 1485ft^2\right) \cdot 35ft$ $\quad V_{soil} = 69650\,ft^3$

$$W_{heel} := \left(footprint - 1485ft^2\right) \cdot theel \cdot 150pcf \qquad W_{heel} = 746.25\,kip$$

$$Uplift_{mod} := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot footprint \cdot \gamma_w \qquad Uplift_{mod} = 8786.667\,kip$$

$$V_{soil} \cdot \left(\gamma - \gamma_w\right) = 4011.84\,kip \qquad V_{soil} \cdot \gamma_w = 4346.16\,kip$$

$$Stability_{mod} := \frac{Structure + Water + V_{soil} \cdot \left(\gamma - \gamma_w\right) + W_{heel}}{Uplift_{mod} - V_{soil} \cdot \left(\gamma_w\right)} \qquad Stability_{mod} = 1.755$$

The heel will have to be at least 10 in length in order to achieve a FS = 1.10. The heel will have to be placed along the full perimeter of the station less the areas occupied by the pipe. Skin friction will not be taken into account in design for the n500+3 event.

BASE SLAB ANALYSIS

$W_{water} := 118kip$ WEIGHT OF WATER IN STATION

$BaseSlab := upliftarea$ $BaseSlab = 1485\ ft^2$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$

$W' := if\left(Stability < 1.0, \frac{Uplift}{BaseSlab}, \frac{Structure}{BaseSlab}\right)$ $W' = 3030.144\,psf$

$W' = 3030.144\,psf$

$\frac{Structure + W_{water}}{BaseSlab} = 2043.757\,psf$

$\frac{Uplift}{BaseSlab} = 3030.144\,psf$

$\frac{Structure + volsoil \cdot (\gamma - \gamma_w)}{BaseSlab} = 2083.196\,psf$

$W' := 2843psf$ $3030psf - 62.4pcf \cdot 3ft = 2842.8\,psf$

$Slab_t := 24in$ $L_{strip} := 19.83ft$ $Wall_t := D_f$ $Slab_d := d_f$ effective depth of slab

$ratio := \frac{13ft}{32.5ft}$ $ratio = 0.4$ Analyze slab with one-way action

$L := 13ft$

Rx controls.

$V'_{ub} := \frac{W' \cdot L}{2}$ $V'_{ub} = 18479.5\ \frac{lb}{ft}$ $a := 19.5ft$ $d_f = 20.375\ in$

$D_f = 2\ ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}} \cdot \left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 13451\ \frac{lb}{ft}$

$A_g := D_f\, 12\, \frac{in}{ft}$ $A_g = 288\ \frac{in^2}{ft}$

$Water = 118638\ lb$

Base Slab Shear, con't

$$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f \right) \right]$$

ACI EQ (11-4)

$\phi V_{nb} = 26784 \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, "OKAY", "NO GOOD"\right)$

CheckBase = "OKAY"

Factor of Safety

$FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$

$FS_{Sb} = 1.991$

Slab Bending

Assume simple span moment (conservative). One-way action, investigate the short span moment capacity.

$M_b := \frac{L^2 \cdot W'}{8}$ $\quad M_b = 60.058 \frac{kip \cdot ft}{ft}$ $\quad W' = 2843\, psf$

#10's @ 12 in.
(mid-plate moment)

TOP REINF

$A_{mpl} := 1.27 \frac{in^2}{ft}$ $\quad b := 12 \frac{in}{ft}$ $\quad a_{mpl} := \frac{A_{mpl} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_{mpl} = 1.66\, in$

$\phi M_{mpl} := \phi_B \cdot A_{mpl} \cdot F_y \cdot \left(d_f - \frac{a_{mpl}}{2} \right)$

$\phi M_{mpl} = 82.7 \frac{kip \cdot ft}{ft}$

$CheckMP := if\left(\phi M_{mpl} > 1.5 \cdot M_b, "OKAY", "NO GOOD"\right)$

CheckMP = "NO GOOD"

Factor of Safety

$FS_{Mb} := \frac{\phi M_{mpl}}{M_b}$

$FS_{Mb} = 1.378$

To address the uplift issue, the base slab may require thickening.

Check the strut capacity.

From RAM Advanse, the axial load is $P_u := 404kip$

$Ag := 18in \cdot 18in$ $Ag = 324\,in^2$ $Ast := 1.767in^2$

$F_y = 40000\,psi$ $f'_c = 3000\,psi$

$P_n := 0.8 \cdot \left[0.85 \cdot f'_c \cdot (Ag - Ast) + F_y \cdot Ast\right]$ $P_n = 713.899\,kip$ tie reinforcement ACi 318-02, 10.3.6.2

The strut end moments are small and the strut length is 5 ft.

$FSstrut := \frac{P_n}{P_u}$ $FSstrut = 1.767$ OKAY

FACTORS OF SAFETY SUMMARY, n500+3

Three walls and the base slab were evaluate for strength.

WALL 1 (EAST WALL) STATUS: OKAY

Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S1} = 1.694$

Moment

$FS_{MPH1} = 1.502$ Mid-plate moment

$FS_{EPH1} = 1.668$ Edge-of-plate moment

WALL 2 (NORTH WALL) STATUS: OKAY

Wall 2 analyzed as a 2D model using RAM Advanse s/w. Wall 2 has intermediate supports.

Shear

$FS_{S2} = 1.555$

Moment

$FS_{MPV2} = 2.228$

$FS_{EPV2} = 1.679$ Controls.

$FS_{MPH2} = 2.695$

$FS_{EPH2} = 2.009$

WALL 3 (SOUTH WALL) STATUS: OKAY

Wall 3 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S3} = 1.494$

Moment

$FS_{MPH3} = 2.424$

$FS_{EPH3} = 1.61$

STABILITY STATUS: NO GOOD

Stability = 0.742 HEAD = 3 ft HGL = 11.5 ft

Analysis indicates that a 10 ft heel extension (minimum) is necessary in order to achieve a FS = 1.1

BASE SLAB STATUS: NO GOOD

Shear

$FS_{Sb} = 1.991$

Moment

$FS_{Mb} = 1.378$ Analyzed as a simple span, one-way action.

Rehabilitation of the base slab to resist the uplift forces will be necessary.

Conclusions

For the n500+3 event:
1.The pump station base slab has a FS for strength of 1.4. The required factor of safety is 1.5. The base slab will require thickening or some type of strengthening to achieve the design level of strength against the uplift forces.
2. The pump station wall strength is adequate.
3. The pump station uplift factor of safety is inadequate. The flotation FS = 0.74. The required additional weight is 1543 kips in order to achieve a FS =1.10. Without additional weight, a heel extension of 10 ft is necessary to achieve the required factor of safety against flotation.

For the existing condition event:
1. The pump station wall strength is adequate. No reliability analysis was required.
2. The pump station base slab strength is adequate. No reliability analysis was required.
3. The pump station uplift factor of safety is inadequate. The flotation FS= 0.81. The required additional weight is 1156 kips in order to achieve a FS = 1.10.

Other:
1. The HGL for the n500+3 event was lowered from 770.7 to 769.5. The bottom of blanket elevation was lowered from 723 to 720. The top of blanket elevation is elevation 755.0, whereas before it was 758. These updated values were provided by EC-GD.
2. The station has a 1.5 foot heel along the east and west walls. Whether the station has a heel along the north wall is questionable. Sheet 4 of the drawings show a heel whereas Sheet 7 of the drawings do not show a heel. The existance of heels along the north and south walls should be verified during PED. For this analysis, a heel was considered NOT present along the north or south walls. The presence of a heel will improve the uplift factor of safety.
Factors of Safety for Uplift Versus HGL

HGL	FS	Event
769.5	0.74	n500+3
765.7	0.81	existing condition
758	0.99	
755	1.08	
754	1.11	

Table based on the following:
Water depth = 3 feet
Equipment weight not included
Heel along the east and west walls only

Analysis indicates that the HGL cannot be greater than 754.5 in order to achieve a factor of safety against uplift greater than 1.10 when no modifications are done to the station.

EC-GD Provided Input Values (7 March 2007)

Armourdale Levee Unit						
Pump Station	Existing Relief Wells?	EC-GD Comps	EC-GD Peer Review	EC-GD Backchecl and Release to EC-DS	Landside Blanket Thickness (ft)	Design Condition n500+3 HGL at Pump Station (assuming no wells)
Osage 76+83*	N	RSK	GMB	RSK	24	763
Shawnee Ave 230+78	Y	RSK	GMB	RSK	25	766.1
KCS RR 276+79	Y	RSK	GMB	RSK	28***	762.3
Central Avenue 299+21	Y	RSK	GMB	RSK	14	760.2
12th Street 129+20	N	RSK	GMB	RSK	26	772
Mill Street 156+75	N	RSK	GMB	RSK	35	770.9
5th Street 185+70	N	RSK	GMB	RSK	35	769.5
MW Cold Storage 194+60	Y	RSK	GMB	RSK	28**	768
PBI Gordon 289+59	N	RSK	GMB	RSK	27	759.6
National Beef 295+52	Y	RSK	GMB	RSK	24	761

*Design condition for fully penetrating seepage cutoff wall from station 62+00 to 83+00
**Blanket thickness based upon building basement elevation and not adjacent ground surface
***Calculations ignore the slot - assumed to be filled in for design

EC-GD Soil Parameters, con't

22 November 2006
rsk

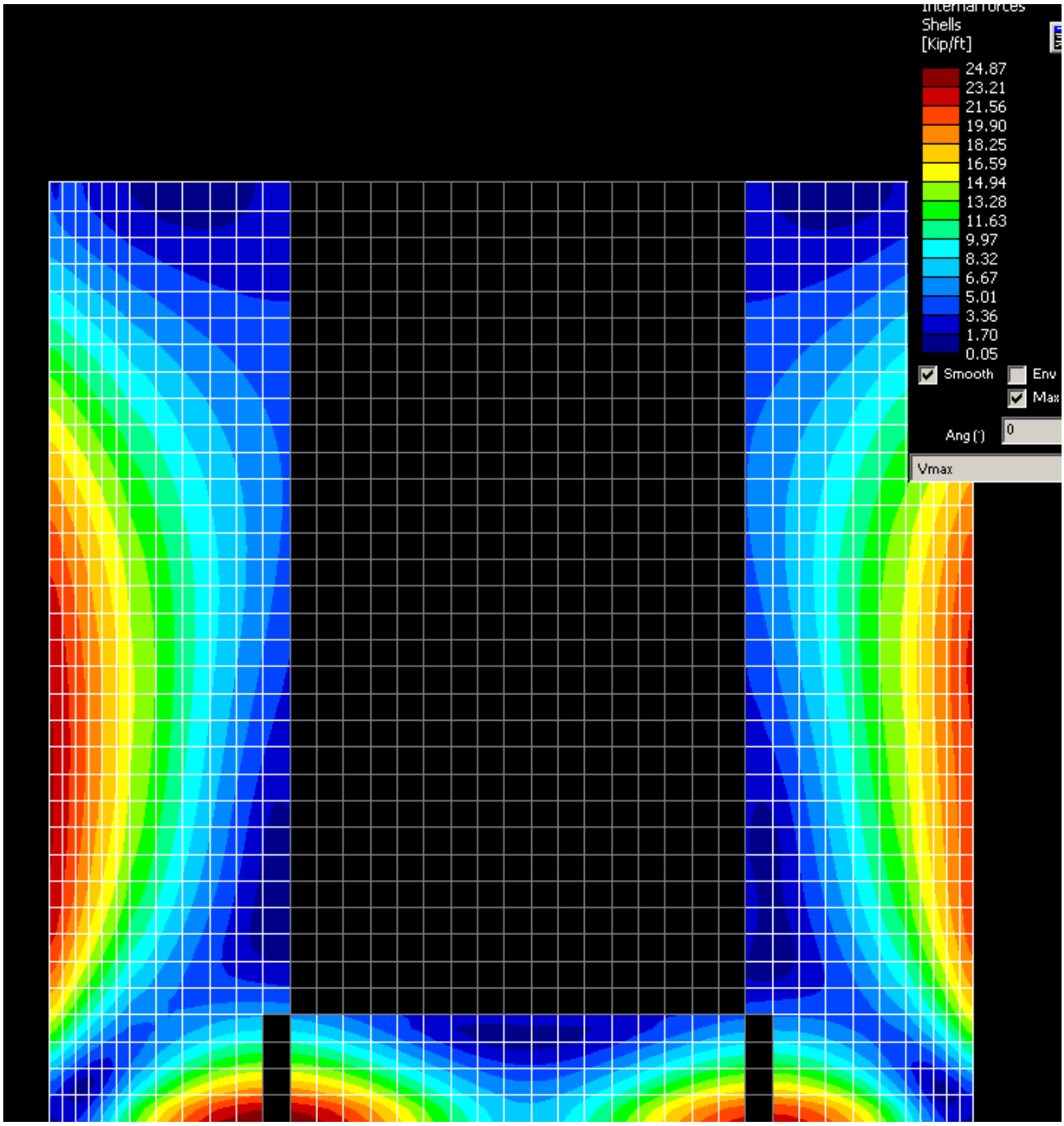
Kansas City Levees – Armourdale
Geotechnical Design Parameters

Material	Unit Weight (pcf)		Shear Strength			
	Moist	Saturated	Undrained		Drained	
			c (psf)	ϕ (deg)	c' (psf)	ϕ' (deg)
Levee Fill	115	120	1000	0	0	29
Foundation Clay (CL)	110	115	500	0	0	26
Foundation Clay (CH)	105	110	500	0	100	17
Foundation Silt (ML)	110	115	500	0	0	28
Foundation Sand (SP)	115	120	N/A	N/A	0	32

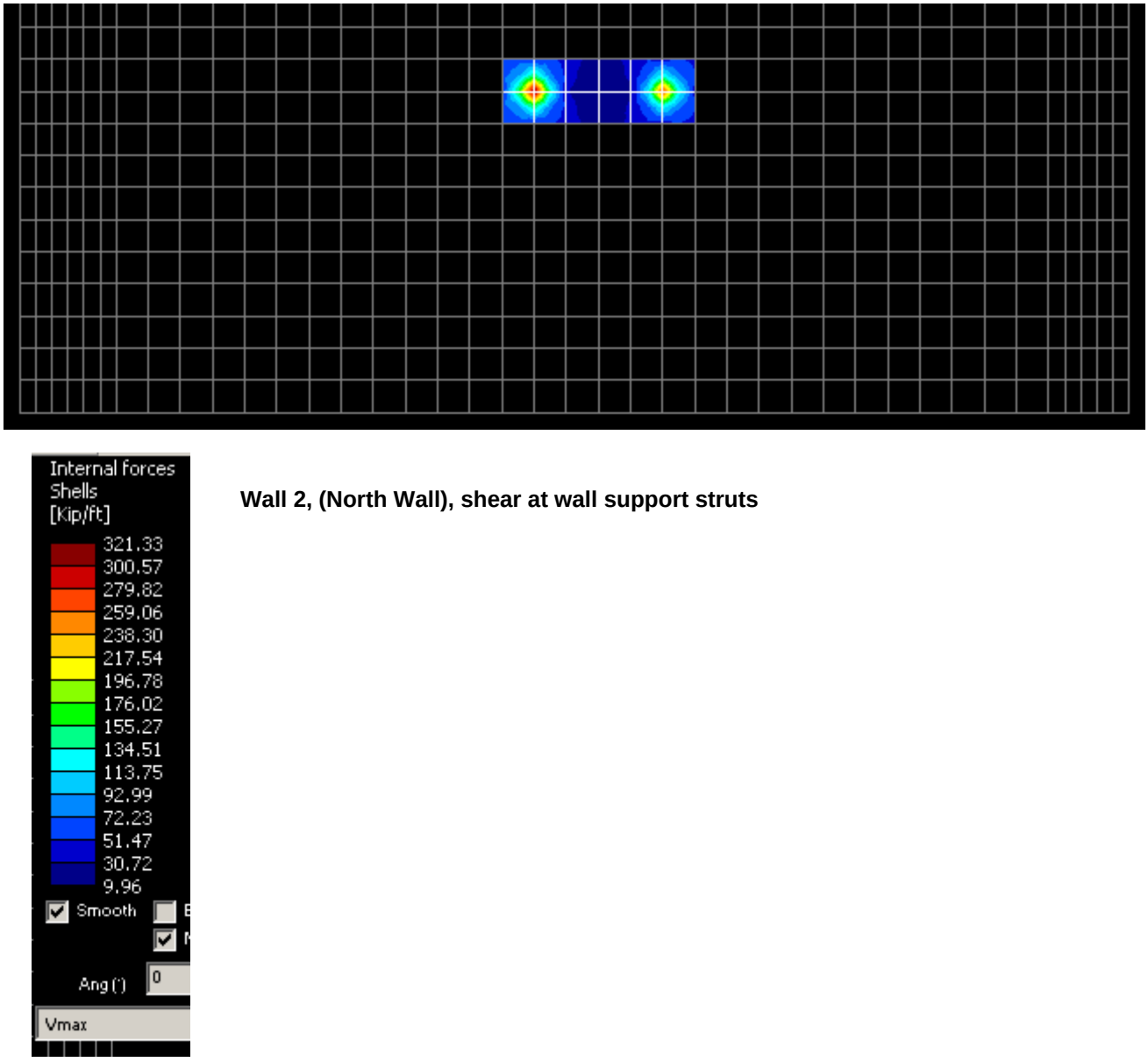
Notes:

1. Undrained shear strengths were estimated from limited existing test data and typical values for the materials. Foundation strength data was increased from the existing test data slightly to account for an increase in material strength under the footprint of the existing levee due to consolidation from the weight of the levee.
2. Drained strength information was taken for the most part from Armourdale Unit, Design Memorandum No. 3, test data performed in 1962.

RAM Advanse Output

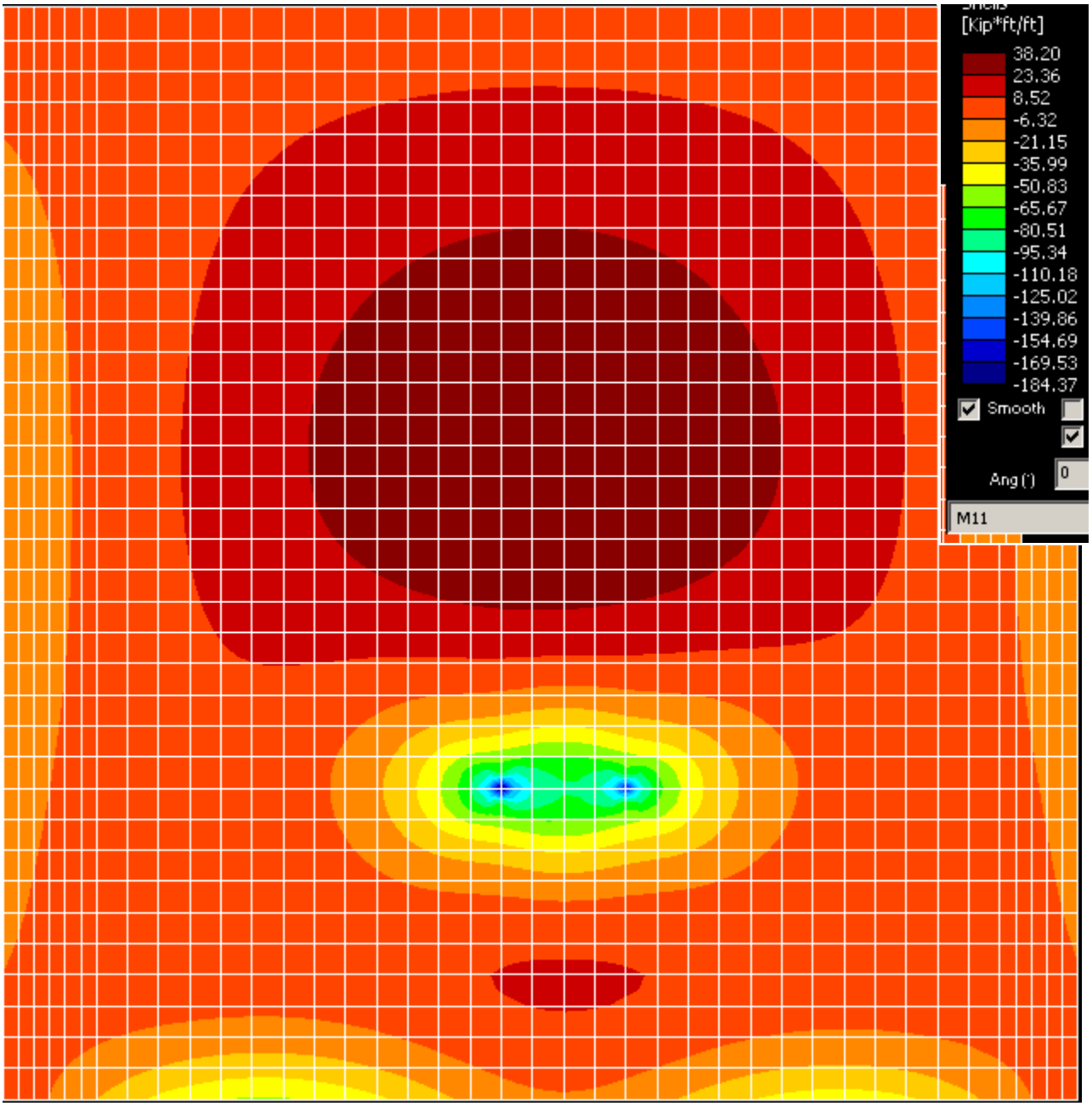


Wall 2, (North Wall), $V`_{u2}$ (shear at wall supports not shown)



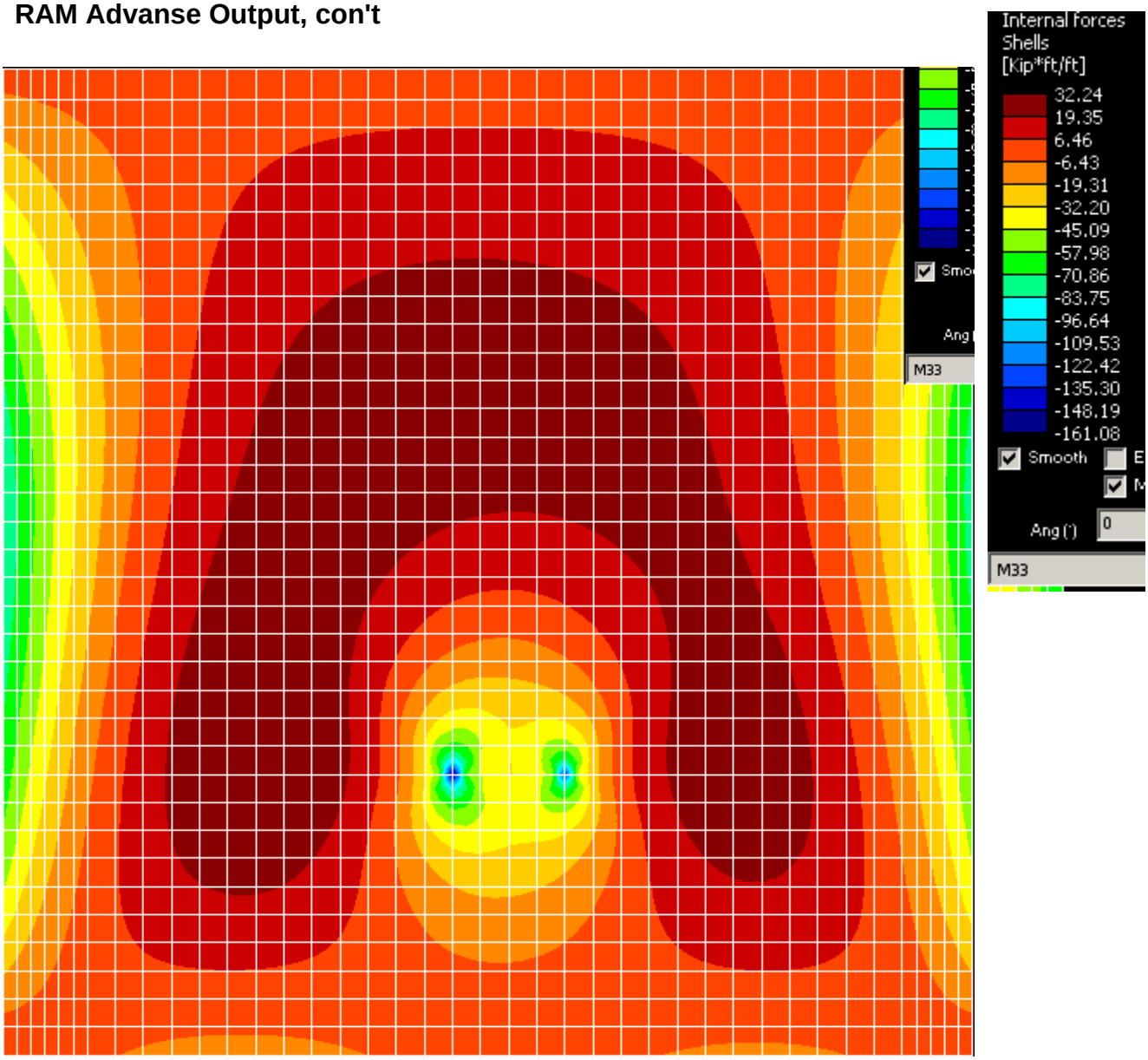
Wall 2, (North Wall), shear at wall support struts

RAM Advanse Output, con't



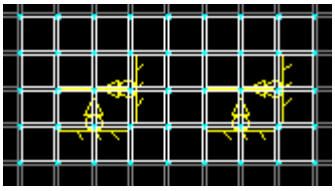
Wall 2, moment M11, moment about the local "1" axis is horizontal (vertical steel)

RAM Advanse Output, con't



Wall 2, moment M33, moment about the local "3" axis is vertical (horizontal steel)

RAM Advanse Output, con't

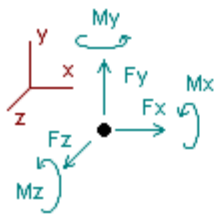


Supports used to model strut reactions.

Node 430 is the left support
Node 434 is the right support

Z axis perpendicular to the plane formed by the wall.

Reactions



Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **SL=Lateral Load on Wall**						
430	0.00000	0.00000	403.59520	0.00000	0.00000	0.00000
434	0.00000	0.00000	337.97754	0.00000	0.00000	0.00000
SUM	0.00000	0.00000	741.57274	0.00000	0.00000	0.00000

Envelope for nodal reactions

Note.- **lc** is the controlling load condition

End of analysis.

5th Street Pump Station Outlet Works Analysis (STA 185+70)

I. Introduction and Background

THE 5TH STREET PUMP STATION OUTLET WORKS LOCATED IN ARMOURDALE, KC LEVEES, CONSIST OF CONCRETE PRESSURE PIPE ACCORDING TO THE TRUMAN AND SCHLUP STAMPED DESIGN DRAWINGS. THE PIPE REQUIRES EVALUATION AS TO WHETHER IT CAN SUSTAIN THE ADDITIONAL LOADS FOR THE n500+3 DESIGN EVENT. THIS DOCUMENT OUTLINES THE ACTIONS TAKEN TO OBTAIN INFORMATION ABOUT THE PIPE AND RECOMMENDATIONS BASED ON THOSE FINDINGS.

II. Conclusions and Recommendations

THE PRICE BROTHERS PRESSURE PIPE COMPANY WAS CONTACTED IN AN EFFORT TO FIND THE CONCRETE PRESSURE PIPE DESIGN SPECIFICATIONS. PRICE BROTHERS MANUFACTURES PRESSURE PIPE AND HAS A DATABASE OF PRESSURE PIPE THAT HAS BEEN MANUFACTURED AND SOLD EITHER BY THEIR COMPANY OR BY COMPANIES THEY BOUGHT OUT. ONE OF THE COMPANIES THEY BOUGHT OUT WAS INTERPACE WHICH HAD AN OFFICE IN THE KANSAS CITY AREA UNTIL 1990. (INCIDENTLY PRICE BROTHERS WAS RECENTLY BOUGHT OUT BY HANSON).

MR. RICK DEREMIAH, A PRODUCT ENGINEER FOR PRICE BROTHERS (973.226.8728) LOOKED IN THEIR DATABASE FOR THE CONCRETE PRESSURE PIPE AT 5TH STREET. HE DID NOT FIND PIPE THAT MATCHED THE SIZE AND LOCATION. HE STATED THAT WITHOUT THE BENEFIT OF PIPE DESIGN INFORMATION, IT WOULD BE IMPOSSIBLE TO CONFIRM THE PIPE'S ABILITY TO HANDLE THE ADDITIONAL EARTH COVER.

FROM THE INFORMATION THAT I PROVIDED PRICE BROTHERS, THEY BELIEVE THE PIPE IS A AWWA C302 PIPE. THIS PIPE HAS STEEL REINFOCEMENT (VERSUS STEEL PIPE).

BASED ON SUBSEQUENT CONVERSATION WITH MR. DEREMIAH AND ASSUMING THAT THE PIPE SPECIFICATIONS CANNOT BE OBTAINED, IT WILL BE ASSUMED FOR FEASIBILITY PURPOSES THAT THE PIPE WILL REQUIRE REHABILITAION TO MEET THE ADDITIONAL LOAD REQUIREMENTS. THE REHABILITATION WILL REQUIRE INSTALLATION OF A STEEL LINER.

THE RECOMMENDATIONS ARE AS FOLLOWS.
1. ASSUME FOR FEASIBILITY COST PURPOSES, A STEEL LINER PIPE WILL BE INSTALLED IN THE EXISTING RCP PIPE AND THE STEEL PIPE WILL ASSUME ALL OF THE LOADING.
2. PRIOR TO PLACING ADDITIONAL LEVEE MATERIAL, THE PIPE SHALL UNDERGO A FULL VISUAL INSPECTION.
3. IF DURING VISUAL INSPECTION, THE PIPE IS SHOWN TO BE OVERLOADED (LONGITUDINAL CRACKS AND OUT-OF-ROUNDNESS) OR DETERIOTED, INSTALL THE STEEL LINER PIPE. GROUT AREAS BETWEEN THE STEEL AND CONCRETE PIPE.
4. IF THE VISUAL INSPECTION DOES NOT REVEAL PIPE DISTRESS, CONTINUE MONITORING.
5. AFTER THE ADDITIONAL EARTH LOAD (LEVEE RAISE) MATERIAL IS ADDED, REINSPECT THE PIPE. IF DISTRESS IS FOUND, INSTALL THE STEEL LINER PIPE AND GROUT BETWEEN THE STEEL PIPE AND THE CONCRETE PIPE.
6. PERFORM AN ANNUAL INSPECTION OF THE PIPE FOR A PERIOD OF TEN YEARS. IDENTIFY LOCATIONS TO MAKE OUT-OF-ROUNDESS CHECKS AND INSPECT FOR CRACKS. DOCUMENT THE INSPECTION MEASUREMENTS AND FINDINGS. THE OUT-OF-ROUNDNESS MEASUREMENTS SHOULD INCLUDE A VERTICAL (INVERT TO CROWN) AND SPRINGLINE (HORIZONTAL) MEASUREMENT.
7. ALL INSPECTION WORK SHALL BE IN ACCORDANCE WITH APPLICABLE OSHA STANDARDS. GIVEN THAT SANITARY SEWER MAY BE PRESENT, THE PIPE AREAS ARE CLASSIFIED AS CONFINED SPACE, PERMIT REQUIRED.
8. A SIGNIFICANT AMOUNT OF MATERIAL HAS ERODED FROM UNDER THE OUTLET STRUCTURE. TO PREVENT FAILURE OF THE OUTLET STRUCTURE, MATERIAL SHOULD BE PLACED UNDER THE OUTLET BASE SLAB.

THE ABOVE RECOMMENDATIONS ARE BASED ON THE FOLLOWING:
1. PIPE BEDDING TYPICALLY IMPROVES OVER TIME, REDUCING THE PIPE LOADS.
2. THE EFFECTS OF ADDITIONAL SOIL LOAD ON THE PIPE MAY TAKE A SIGNIFICANT AMOUT OF TIME. THEREFORE INTERIOR AREAS OF THE PIPE SHOULD BE INSPECTED ANNUALLY FOR CRACKS AND OUT OF ROUNDESS.
3. RISKS ARE ASSOCIATED WITH REMOVING A SEGMENT OF PIPE AND TESTING. WILL THE SEGMENT REMOVED BE A REPRESENTATIVE SAMPLE? REMOVAL REQUIRES REMOVAL AND REPLACEMENT OF THE OUTLET STRUCTURE. THERE IS THE RISK OF A SIGNIFICANT FLOOD EVENT THAT MAY OCCUR DURING PIPE TESTING WHICH COULD PUT THE LEVEE AT RISK.

III. References

1. Design Memorandum 3, Armourdale Unit, Department of the Army, Kansas City District, Corps of Engineers, May 1971
2. Price Brothers Pressure Pipe Company.
Contacts: Rick Dermiah, Product Engineer, 937-226-8728 (Dayton, OH)
Field Representative Ron Maybell, 816-769-1490 (KCMO)
Tom Thorton 937-226-8902 (Dayton, OH)
3. Truman and Schulp Design Drawings, 1962

IV. Background Information

1. Loading

$Invert := 725.82ft$ PIPE INVERT ELEVATION AT LEVEE CROWN

$Dia := 6ft$ INSIDE PIPE DIAMETER

$thick := 6in$ PIPE WALL THICKNESS (ASSUMED)

- Original Design

$TopofLevee_{orig} := 764.23ft$ ORIGINAL "TOP OF LEVEE" ELEVATION

$Cover_{orig} := TopofLevee_{orig} - (Invert + Dia + thick)$

$Cover_{orig} = 31.91\ ft$ DEPTH OF COVER ABOVE THE PIPE CROWN

- 1964 Mod

$TopofLevee_{64mod} := 767.6ft$ "TOP OF LEVEE" ELEVATION FOR THE 1964 MOD

$Cover_{64mod} := TopofLevee_{64mod} - (Invert + Dia + thick)$

$Cover_{64mod} = 35.28\ ft$ DEPTH OF COVER ABOVE THE PIPE CROWN

- 1964 Mod, con't

$$Increaseaboveorig1 := \frac{Cover_{64mod} - Cover_{orig}}{Cover_{orig}}$$

$Increaseaboveorig1 = 10.56\ \%$ ADDITIONAL LOADING RELATIVE TO THE ORIGINAL COVER. DESIGN MEMORANDUM STATES "THE EXISTING PIPE IS IN GOOD CONDITION AND HAS BEEN CHECKED AND FOUND CAPABLE OF WITHSTANDING THE ADDITONAL LOAD DUE TO THE PROPOSED LEVEE RAISE." CALCULATIONS ASSOCIATED WITH THIS STATEMENT COULD NOT BE LOCATED.

THE EXISTING LEVEE ELEVATION IS $TopofLevee_{64mod} = 767.60\ ft$

- n500+3 Event Loading

$TopofLevee_{n500plus3} := 772.0ft$

$Cover_{n500plus3} := TopofLevee_{n500plus3} - (Invert + Dia + thick)$

$Cover_{n500plus3} = 39.68\ ft$ DEPTH OF COVER ABOVE THE PIPE CROWN

$$Increaseaboveorig2 := \frac{Cover_{n500plus3} - Cover_{orig}}{Cover_{orig}}$$

$Increaseaboveorig2 = 24.35\ \%$ ADDITIONAL LOADING RELATIVE TO THE ORIGINAL COVER

$$Increaseabove64mod := \frac{Cover_{n500plus3} - Cover_{64mod}}{Cover_{64mod}}$$

$Increaseabove64mod = 12.47\ \%$ ADDITIONAL LOADING RELATIVE TO THE APPROVED 64 MOD. THE 64 MOD ELEVATION IS THE CURRENT TOP OF LEVEE ELEVATION.

2. Supporting Documentation

From: Deremiah, Rick [rderemiah@PriceBrothers.com] Sent: Thu 3/29/2007 1:48 PM
To: Parks, Marvin L NWK
Cc: Jansen, Ronald G NWK; McKissack, Grady L NWK; Corkill, Melissa R NWK; Bellew, Glen M NWK; Gibbs, Don; Maybell, Ron; Thornton, Tom
Subject: RE: Pressure Pipe Inquiry, 5th Street Pump Station

Mr. Parks:

We have closely examined our past project database for The Lock Joint Pipe Company / Interpace Corporation to see if we could identify this project. Unfortunately, we cannot find specific reference to a project that fits the information you have provided. The closest project was:

Job Number TU-65-47 (1965 manufacture)
Kansas City, KS
9047' - 72" reinforced concrete non-cylinder pipe (AWWA C302 type; SP-1)
Engineer: Black and Veatch
Name: Northeast Industrial District Interceptor Contract No. 19

It is very possible that this pipe may have been supplied by another pipe manufacturer.

Without the benefit of pipe design information, it will be impossible for us to confirm the pipe's ability to handle additional earth cover.

--Rick

Rick Deremiah, PE
Manager, Design Engineering
Price Brothers Company / Hanson Pipe and Precast
333 West First Street, Suite 700
Dayton, OH 45402
Phone: 937-226-8728
Cell: 937-479-4117
FAX: 937-226-8711
rderemiah@pricebrothers.com
www.pipesite.com

2. Supporting Documentation, con't

-----Original Message-----
From: Parks, Marvin L NWK [mailto:Marvin.L.Parks@nwk02.usace.army.mil]
Sent: Thursday, March 29, 2007 1:51 PM
To: Deremiah, Rick
Cc: Jansen, Ronald G NWK; Parks, Marvin L NWK; McKissack, Grady L NWK; Corkill, Melissa R NWK; Bellew, Glen M NWK
Subject: Pressure Pipe Inquiry, 5th Street Pump Station

Dear Mr. Deremiah;
Thank you for your assistance. To summarize what we have:

1. Location: Kansas City, Kansas Armourdale Industrial Park

2. Background. The drawings specify "6`-0" concrete pressure pipe. The total pipe length is approximately 220 feet. The pipe provides interior storm sewer drainage of protected area. During flood events on the Kansas river a pump station pumps interior water from the interior areas to the river. The pump station is "5th Street pump station" (STA 185+70)

3. The pipe was constructed and placed in 1962-1963 timeframe.

4. Truman Schlup Consulting Engineers provided the design.

5. Loading. The original design height of fill was 31.9 feet. In the 1970s, the levee was raised. The resulting height of fill is 35.3 feet.
The proposed levee raise will create a fill height of 39.7 feet. These measurements are from the levee crown to the pipe crown.

6. Army Corps of Engineer document, Design Memorandum 3 dated May 1971 address the 1970s levee raise mentioned in paragraph 5 above. This document states "The existing pipe is in good condition and has been checked and found capable of withstanding the additional load due to the proposed levee raise."

7. Therefore, the additional fill above the pipe crown relative to the 1970s raise is 4.4 ft. This represents an additional 12.4% load due to a levee raise relative to 1970s raise and a 24.4% additional load relative to the original 1962 design.

8. The drainage system has a flap gate. Entry into the conduit requires specialized equipment and is likely a confined space area. Excavation would be necessary in order to view the pipe exterior areas. It is not known how long the pipe segments are.

9. My understanding from our conversation earlier today is that prestress pipe lengths are usually longer than other types of pipes.
Prestress pipes come in 16-20 foot lengths whereas the maximum length of other pipe types is around 12 feet. Prestress pipe has a rough exterior relative to other pipes.

2. Supporting Documentation, con't

10. The bottom line is that in order to assess the pipe strength, a design data sheet (or similar) has to be available or some other means of finding the pipe design has to be determined. Concrete pressure pipe strength cannot be evaluated like an RCP using the Indirect Method because pressure pipe is not conveniently categorized by class. This requires use of the Direct Method whereby the pipe reinforcement, wall size, concrete strength has to be known in order to determine the pipe strength.

11. In the event that no information can be found on the pipe what are your recommendations for evaluating the pipe? My thoughts are that testing would have to be done on an existing pipe segment. This would require replacement of an existing pipe segment. Does this approach make sense or are nondestructive methods available? Are there any other options out there?

Again thank you for your assistance. If your records provide design sheets or any other additional information, please notify me. Mr.
Thorton did mention that a 66" pipe was used on the Armourdale project.
Any information on this pipe may be of further benefit.

Regards
Marvin Parks
Structural Engineer
Kansas City District
Army Corps of Engineers
816.389.3253
FAX 816.389.2013

END OF ANALYSIS.

US Army Corps of Engineers

Probability of Failure

5th Ave. Pump Station - Station 185+70

Armourdale Unit

SUMMARY:
THE 5TH AVE. STREET PUMP STATION FACTOR OF SAFETY (FS) FOR UPLIFT STABILITY IS 0.88 FOR AN EXISTING CONDITION EVENT (WATER ELEVATION = 767.6) WHERE THE HGL= 765.70. FOR THIS EVENT, THE PROBALITY OF FAILURE (POF) IS 100 PERCENT AS SHOWN IN FIGURE 1. THE TOP OF LEVEE ELEVATION IS 767.60 FT. THE POF FOR UPLIFT IS 100 PERCENT FOR WATER SURFACE ELEVATIONS 764.0 AND HIGHER. AT WATER SURFACE ELEVATION 763.00 FT, THE POF IS 93.9 PERCENT. FOR WATER SURFACE ELEVATION 762.0 FT, THE POF IS 2.5 PERCENT. BY INTERPOLATION, THE POF IS 1.23 PERCENT WHEN THE WATER SURFACE ELEVATION IS 761.00 FT. THE POF VALUES DO TAKE INTO ACCOUNT SKIN FRICTION BETWEEN THE SOIL AND CONCRETE STRUCTURE.

Part I. Vary the Geotechnical Parameters and Determine the Corresponding Factor of Uplift Factor of Safety Values

CARRY FORWARD THE UPLIFT FACTOR OF SAFETY VALUES CALCULATED EARLIER

Uplift factors of safety versus HGL

HGL	FS no skin friction	FS with skin friction
765.7	0.808	0.878
764	0.842	0.923
763	0.863	0.952
762.5	0.873	0.966
761.75	0.89	0.99
761.25	0.902	1.00
758.75	0.965	1.09

VALUES OBTAINED FROM EC-DS ANALYSIS OF PUMP STATION:

$Structure := 2917kip$ — STRUCTURE DEAD WEIGHT

$Water := 118.6kip$ — WEIGHT OF WATER IN THE STRUCTURE

$BaseSlab := 1485ft^2$ — AREA OF THE SLAB THAT UPLIFT FORCES ACT ON

$\gamma := 115pcf$ — SOIL UNIT WEIGHT

$volsoil := 3065.4ft^3$ — VOLUME OF SOIL OVER HEELS

$\gamma_w := 62.4pcf$ — WATER UNIT WEIGHT

$H := 37.06ft$ — HEIGHT OF STRUCTURE BELOW GRADE

$P_V =$ ▮ — SKIN FRICTION (VARIES)

GEOTECHNICAL PARAMETERS:

$x := 45ft$ — EFFECTIVE RIVERSIDE SEEPAGE LENGTH (CONSTANT), (x_1 IN GEOTECH SPREADSHEET)

$L_e =$ ▮ — EFFECTIVE LANDSIDE SEEPAGE LENGTH (VARIES)

$\frac{K_f}{K_b} =$ ▮ — PERMEABILITY RATIO (VARIES)

$D_b =$ ▮ — IMPERVIOUS BLANKET THICKNESS (VARIES)

$D_f =$ ▮ — PERVIOUS BLANKET THICKNESS (VARIES)

TOP OF EXISTING LEVEE ELEVATION = 767.6

NOTE THAT THE EC-GD BASE THE HGL ON ELEVATION 755 (LEVEE TOE) WHEREAS EC-DS CALCULATIONS HGL IS BASED ON GRADE ELEVATION 758 AT PUMP STATION AS SHOWN ON THE PLANS. TO ACCOUNT FOR THE DIFFERENCE, 3 FEET WAS SUBTRACTED FROM THE ho VALUES PROVIDED BY EC-GD.

Part I. Water Elevation = 762.0

WATER SURFACE ELEVATION $Flood := 762ft$ $P_v := 370.7kip$

Mean HGL $HGL := 5.96ft - 3ft$ $HGL = 2.96\ ft$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right] \qquad Uplift = 3708.41\ kip$$

$$FS_{mean} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{mean} = 1.014$$

Kf / Kb +1Std

$HGL := 6.1ft - 3ft$ $HGL = 3.10\ ft$ $P_v := 367.9kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Kfu} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Kfu} = 1.01$$

Kf / Kb -1Std

$HGL := 5.71ft - 3ft$ $HGL = 2.71\ ft$ $P_v := 375.6kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{KfL} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{KfL} = 1.02$$

Db +1Std

$HGL := 6.03ft - 3ft$ $HGL = 3.03\ ft$ $P_v := 369.3kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Dbu} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Dbu} = 1.01$$

Db -1Std

$HGL := 5.85ft - 3ft$ $HGL = 2.85\ ft$ $P_v := 372.9kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{DbL} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{DbL} = 1.02$$

Part I. Water Elevation = 762.0, con't

Flood = 762.00 ft

Df +1Std

$HGL := 6.02ft - 3ft$ $\quad HGL = 3.02\ ft$ $\quad P_v := 369.5kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{Dfu} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{Dfu} = 1.01$

Df -1Std

$HGL := 5.88ft - 3ft$ $\quad HGL = 2.88\ ft$ $\quad P_v := 372.3kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{DfL} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{DfL} = 1.02$

Part I. Water Elevation = 760.0

WATER SURFACE ELEVATION $Flood1 := 760ft$ $P_v := 404.7kip$

Mean HGL $HGL := 4.25ft - 3ft$ $HGL = 1.25\ ft$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right] \qquad Uplift = 3549.96\ kip$$

$$FS_{mean1} := \frac{Structure + Water + volsoil \times (\gamma - \gamma_w) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{mean1} = 1.072$$

Kf / Kb +1Std

$HGL := 4.35ft - 3ft$ $HGL = 1.35\ ft$ $P_v := 402.7kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Kfu1} := \frac{Structure + Water + volsoil \times (\gamma - \gamma_w) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Kfu1} = 1.07$$

Kf / Kb -1Std

$HGL := 4.08ft - 3ft$ $HGL = 1.08\ ft$ $P_v := 408.1kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{KfL1} := \frac{Structure + Water + volsoil \times (\gamma - \gamma_w) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{KfL1} = 1.08$$

Db +1Std

$HGL := 4.31ft - 3ft$ $HGL = 1.31\ ft$ $P_v := 403.5kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Dbu1} := \frac{Structure + Water + volsoil \times (\gamma - \gamma_w) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Dbu1} = 1.07$$

Db -1Std

$HGL := 4.18ft - 3ft$ $HGL = 1.18\ ft$ $P_v := 406.1kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{DbL1} := \frac{Structure + Water + volsoil \times (\gamma - \gamma_w) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{DbL1} = 1.07$$

Part I. Water Elevation = 760.0 con't.

$Flood1 = 760.00\ ft$

Df +1Std

$HGL := 4.3ft - 3ft$ $\quad HGL = 1.30\ ft$ $\quad P_v := 403.7kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{Dfu1} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{Dfu1} = 1.07$

Df -1Std

$HGL := 4.2ft - 3ft$ $\quad HGL = 1.20\ ft$ $\quad P_v := 405.7kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{DfL1} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{DfL1} = 1.07$

Part I. Water Elevation = 764.0

WATER SURFACE ELEVATION $Flood2 := 764ft$ $P_v := 336.8kip$

Mean HGL $HGL := 7.66ft - 3ft$ $HGL = 4.66\ ft$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right] \qquad Uplift = 3865.94\ kip$$

$$FS_{mean2} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{mean2} = 0.962$$

Kf / Kb +1Std

$HGL := 7.84ft - 3ft$ $HGL = 4.84\ ft$ $P_v := 333.3kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Kfu2} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Kfu2} = 0.96$$

Kf / Kb -1Std

$HGL := 7.34ft - 3ft$ $HGL = 4.34\ ft$ $P_v := 343.2kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{KfL2} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{KfL2} = 0.97$$

Db +1Std

$HGL := 7.76ft - 3ft$ $HGL = 4.76\ ft$ $P_v := 334.9kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Dbu2} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Dbu2} = 0.96$$

Db -1Std

$HGL := 7.52ft - 3ft$ $HGL = 4.52\ ft$ $P_v := 339.6kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{DbL2} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{DbL2} = 0.97$$

Part I. Water Elevation = 764.0 con't.

$Flood2 = 764.00\ ft$

Df +1Std

$HGL := 7.73ft - 3ft \qquad HGL = 4.73\ ft \qquad P_v := 335.5kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{Dfu2} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{Dfu2} = 0.96$

Df -1Std

$HGL := 7.56ft - 3ft \qquad HGL = 4.56\ ft \qquad P_v := 338.9kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{DfL2} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{DfL2} = 0.96$

Part I. Water Elevation = 763.0

WATER SURFACE ELEVATION $Flood3 := 763ft$ $P_v := 353.8kip$

Mean HGL $HGL := 6.81ft - 3ft$ $HGL = 3.81\ ft$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right] \qquad Uplift = 3787.18\ kip$$

$$FS_{mean3} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{mean3} = 0.987$$

Kf / Kb +1Std

$HGL := 6.97ft - 3ft$ $HGL = 3.97\ ft$ $P_v := 350.6kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Kfu3} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Kfu3} = 0.98$$

Kf / Kb -1Std

$HGL := 6.52ft - 3ft$ $HGL = 3.52\ ft$ $P_v := 359.5kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{KfL3} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{KfL3} = 1.00$$

Db +1Std

$HGL := 6.9ft - 3ft$ $HGL = 3.90\ ft$ $P_v := 352kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Dbu3} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Dbu3} = 0.98$$

Db -1Std

$HGL := 6.69ft - 3ft$ $HGL = 3.69\ ft$ $P_v := 356.2kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{DbL3} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{DbL3} = 0.99$$

Part I. Water Elevation = 763.0, con't

$Flood3 = 763.00\ ft$

Df +1Std

$HGL := 6.88ft - 3ft \qquad HGL = 3.88\ ft \qquad P_v := 352.4kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{Dfu3} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{Dfu3} = 0.99$

Df -1Std

$HGL := 6.72ft - 3ft \qquad HGL = 3.72\ ft \qquad P_v := 355.6kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{DfL3} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{DfL3} = 0.99$

Part I. Water Elevation = 762.5

WATER SURFACE ELEVATION $Flood4 := 762.5ft$ $P_v := 362.1kip$

Mean HGL $HGL := 6.39ft - 3ft$ $HGL = 3.39\ ft$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right] \qquad Uplift = 3748.26\ kip$$

$$FS_{mean4} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{mean4} = 1.001$$

Kf / Kb +1Std

$HGL := 6.54ft - 3ft$ $HGL = 3.54\ ft$ $P_v := 359.1kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Kfu4} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Kfu4} = 1.00$$

Kf / Kb -1Std

$HGL := 6.12ft - 3ft$ $HGL = 3.12\ ft$ $P_v := 367.5kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{KfL4} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{KfL4} = 1.01$$

Db +1Std

$HGL := 6.47ft - 3ft$ $HGL = 3.47\ ft$ $P_v := 360.5kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{Dbu4} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{Dbu4} = 1.00$$

Db -1Std

$HGL := 6.27ft - 3ft$ $HGL = 3.27\ ft$ $P_v := 364.5kip$

$$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$$

$$FS_{DbL4} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w} \qquad FS_{DbL4} = 1.00$$

Part I. Water Elevation = 762.5, con't

$Flood4 = 762.50\ ft$

NOTE: VALUES FOR 762.5 WATER ELEVATION WERE INTERPOLATED FROM EC-GD PROVIDED VALUES OF 762.0 AND 763.0 FT.

Df +1Std

$HGL := 6.45ft - 3ft \qquad HGL = 3.45\ ft \qquad P_v := 360.9kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{Dfu4} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{Dfu4} = 1.00$

Df -1Std

$HGL := 6.30ft - 3ft \qquad HGL = 3.30\ ft \qquad P_v := 363.9kip$

$Uplift := \left[(H + HGL) \times (BaseSlab) \times \gamma_w\right]$

$$FS_{DfL4} := \frac{Structure + Water + volsoil \times \left(\gamma - \gamma_w\right) + P_v}{Uplift - volsoil \times \gamma_w}$$

$FS_{DfL4} = 1.00$

FACTORS OF SAFETY SUMMARY

WATER AT ELEVATION 762

$FS_{mean} = 1.01$ WATER SURFACE ELEVATION $Flood = 762.00$ ft

$FS_{Kfu} = 1.01$ $FS_{Dbu} = 1.01$ $FS_{Dfu} = 1.01$

$FS_{KfL} = 1.02$ $FS_{DbL} = 1.02$ $FS_{DfL} = 1.02$

WATER AT ELEVATION 760

$FS_{mean1} = 1.07$ WATER SURFACE ELEVATION $Flood1 = 760.00$ ft

$FS_{Kfu1} = 1.07$ $FS_{Dbu1} = 1.07$ $FS_{Dfu1} = 1.07$

$FS_{KfL1} = 1.08$ $FS_{DbL1} = 1.07$ $FS_{DfL1} = 1.07$

WATER AT ELEVATION 764

$FS_{mean2} = 0.96$ WATER SURFACE ELEVATION $Flood2 = 764.00$ ft

$FS_{Kfu2} = 0.96$ $FS_{Dbu2} = 0.96$ $FS_{Dfu2} = 0.96$

$FS_{KfL2} = 0.97$ $FS_{DbL2} = 0.97$ $FS_{DfL2} = 0.96$

WATER AT ELEVATION 763

$FS_{mean3} = 0.99$ WATER SURFACE ELEVATION $Flood3 = 763.00$ ft

$FS_{Kfu3} = 0.98$ $FS_{Dbu3} = 0.98$ $FS_{Dfu3} = 0.99$

$FS_{KfL3} = 1.00$ $FS_{DbL3} = 0.99$ $FS_{DfL3} = 0.99$

WATER AT ELEVATION 762.5

$FS_{mean4} = 1.00$ WATER SURFACE ELEVATION $Flood4 = 762.50$ ft

$FS_{Kfu4} = 1.00$ $FS_{Dbu4} = 1.00$ $FS_{Dfu4} = 1.00$

$FS_{KfL4} = 1.01$ $FS_{DbL4} = 1.00$ $FS_{DfL4} = 1.00$

Part II. Probability of Failure

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996

III. Situation

1. XXXXXX Pump Station Uplift Calculations
2. Analysis of Existing Conditions
3. Mathcad spreadsheets used to determine the Factors of Safety.
4. From varying hydraulic gradelines as supplied by geotechnical input.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Ku} = Factor of Safety due to the upper bound value of the Permiablility Ratio
FS_{Kl} = Factor of Safety due to the lower bound value of the Permiablility Ratio
FS_{Bu} = Factor of Safety due to the upper bound value Blanket Thickness
FS_{Bl} = Factor of Safety due to the lower bound value Blanket Thickness
FS_{Fu} = Factor of Safety due to the upper bound value Foundation Thickness
FS_{Fl} = Factor of Safety due to the lower bound value Foundation Thickness
ΔF_K = Difference in Factors of Safety due to the change in Permiablility Ratio
ΔF_B = Difference in Factors of Safety due to the change in Blanket Thickness
ΔF_F = Difference in Factors of Safety due to the change in Foundation Thickness
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. Probability of Failure Calculation, Water at Elev 762.0

FACTORS OF SAFETY FOR UPLIFT, WATER X FEET BELOW TOP OF LEVEE

$FS_{mean} = 1.01$ WATER SURFACE ELEVATION Flood = 762.00 ft

$FS_{Kfu} = 1.01$ $FS_{Dbu} = 1.01$ $FS_{Dfu} = 1.01$

$FS_{KfL} = 1.02$ $FS_{DbL} = 1.02$ $FS_{DfL} = 1.02$

$\Delta F_K := FS_{KfL} - FS_{Kfu}$ $\Delta F_K = 0.01$

$\Delta F_B := FS_{DbL} - FS_{Dbu}$ $\Delta F_B = 0.01$

$\Delta F_F := FS_{DfL} - FS_{Dfu}$ $\Delta F_F = 0.00$

$$\sigma_F := \sqrt{\left(\frac{\Delta F_K}{2}\right)^2 + \left(\frac{\Delta F_B}{2}\right)^2 + \left(\frac{\Delta F_F}{2}\right)^2} \qquad \sigma_F = 0.01$$

$$V_F := \frac{\sigma_F}{FS_{mean}} \qquad V_F = 0.01$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_{mean}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = 1.97$$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 97.54\,\%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 2.46\,\%$

VI. Probability of Failure Calculation, Water at Elev 760.0

FACTORS OF SAFETY FOR UPLIFT, WATER X + 1 FEET BELOW TOP OF LEVEE

$FS_{mean1} = 1.07$ WATER SURFACE ELEVATION $Flood1 = 760.00$ ft

$FS_{Kfu1} = 1.07$ $FS_{Dbu1} = 1.07$ $FS_{Dfu1} = 1.07$

$FS_{KfL1} = 1.08$ $FS_{DbL1} = 1.07$ $FS_{DfL1} = 1.07$

$\Delta F_K := FS_{KfL1} - FS_{Kfu1}$ $\Delta F_K = 0.01$

$\Delta F_B := FS_{DbL1} - FS_{Dbu1}$ $\Delta F_B = 0.00$

$\Delta F_F := FS_{DfL1} - FS_{Dfu1}$ $\Delta F_F = 0.00$

$$\sigma_F := \sqrt{\left(\frac{\Delta F_K}{2}\right)^2 + \left(\frac{\Delta F_B}{2}\right)^2 + \left(\frac{\Delta F_F}{2}\right)^2} \qquad \sigma_F = 0.01$$

$$V_F := \frac{\sigma_F}{FS_{mean1}} \qquad V_F = 0.01$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_{mean1}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = 13.31$$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 100\,\%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_{F1} := 1 - R$ $P_{F1} = 0\,\%$

VII. Probability of Failure Calculation, Water at Elev 764.0

FACTORS OF SAFETY FOR UPLIFT, WATER X + 2 FEET BELOW TOP OF LEVEE

$FS_{mean2} = 0.96$ WATER SURFACE ELEVATION $Flood2 = 764.00$ ft

$FS_{Kfu2} = 0.96$ $FS_{Dbu2} = 0.96$ $FS_{Dfu2} = 0.96$

$FS_{KfL2} = 0.97$ $FS_{DbL2} = 0.97$ $FS_{DfL2} = 0.96$

$\Delta F_K := FS_{KfL2} - FS_{Kfu2}$ $\Delta F_K = 0.01$

$\Delta F_B := FS_{DbL2} - FS_{Dbu2}$ $\Delta F_B = 0.01$

$\Delta F_F := FS_{DfL2} - FS_{Dfu2}$ $\Delta F_F = 0.01$

$$\sigma_F := \sqrt{\left(\frac{\Delta F_K}{2}\right)^2 + \left(\frac{\Delta F_B}{2}\right)^2 + \left(\frac{\Delta F_F}{2}\right)^2} \qquad \sigma_F = 0.01$$

$$V_F := \frac{\sigma_F}{FS_{mean2}} \qquad V_F = 0.01$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_{mean2}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = -4.37$$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 6.21 \times 10^{-4}\,\%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_{F2} := 1 - R$ $P_{F2} = 99.9994\,\%$

VIII. Probability of Failure Calculation, Water at Elev 763.0

FACTORS OF SAFETY FOR UPLIFT, WATER X FEET BELOW TOP OF LEVEE

$FS_{mean3} = 0.99$ WATER SURFACE ELEVATION $Flood3 = 763.00$ ft

$FS_{Kfu3} = 0.98$ $FS_{Dbu3} = 0.98$ $FS_{Dfu3} = 0.99$

$FS_{KfL3} = 1.00$ $FS_{DbL3} = 0.99$ $FS_{DfL3} = 0.99$

$\Delta F_K := FS_{KfL3} - FS_{Kfu3}$ $\Delta F_K = 0.01$

$\Delta F_B := FS_{DbL3} - FS_{Dbu3}$ $\Delta F_B = 0.01$

$\Delta F_F := FS_{DfL3} - FS_{Dfu3}$ $\Delta F_F = 0.00$

$$\sigma_F := \sqrt{\left(\frac{\Delta F_K}{2}\right)^2 + \left(\frac{\Delta F_B}{2}\right)^2 + \left(\frac{\Delta F_F}{2}\right)^2} \qquad \sigma_F = 0.01$$

$$V_F := \frac{\sigma_F}{FS_{mean3}} \qquad V_F = 0.01$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_{mean3}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = -1.55$$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 6.08\,\%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_{F3} := 1 - R$ $P_{F3} = 93.92\,\%$

VIV. Probability of Failure Calculation, Water at Elev 762.5

FACTORS OF SAFETY FOR UPLIFT, WATER X FEET BELOW TOP OF LEVEE

$FS_{mean4} = 1.00$ WATER SURFACE ELEVATION $Flood4 = 762.50$ ft

$FS_{Kfu4} = 1.00$ $FS_{Dbu4} = 1.00$ $FS_{Dfu4} = 1.00$

$FS_{KfL4} = 1.01$ $FS_{DbL4} = 1.00$ $FS_{DfL4} = 1.00$

$\Delta F_K := FS_{KfL4} - FS_{Kfu4}$ $\Delta F_K = 0.01$

$\Delta F_B := FS_{DbL4} - FS_{Dbu4}$ $\Delta F_B = 0.01$

$\Delta F_F := FS_{DfL4} - FS_{Dfu4}$ $\Delta F_F = 0.00$

$$\sigma_F := \sqrt{\left(\frac{\Delta F_K}{2}\right)^2 + \left(\frac{\Delta F_B}{2}\right)^2 + \left(\frac{\Delta F_F}{2}\right)^2} \qquad \sigma_F = 0.01$$

$$V_F := \frac{\sigma_F}{FS_{mean4}} \qquad V_F = 0.01$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_{mean4}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = 0.07$$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 52.68\,\%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_{F4} := 1 - R$ $P_{F4} = 47.32\,\%$

VV. Probability of Failure Summary

WATER SURFACE ELEVATION	PROBABILITY OF FAILURE
Flood = 762.00 ft	P_F = 2.46 %
Flood1 = 760.00 ft	P_{F1} = 0.0000 %
Flood2 = 764.00 ft	P_{F2} = 99.9994 %
Flood3 = 763.00 ft	P_{F3} = 93.92 %
Flood4 = 762.50 ft	P_{F4} = 47.32 %

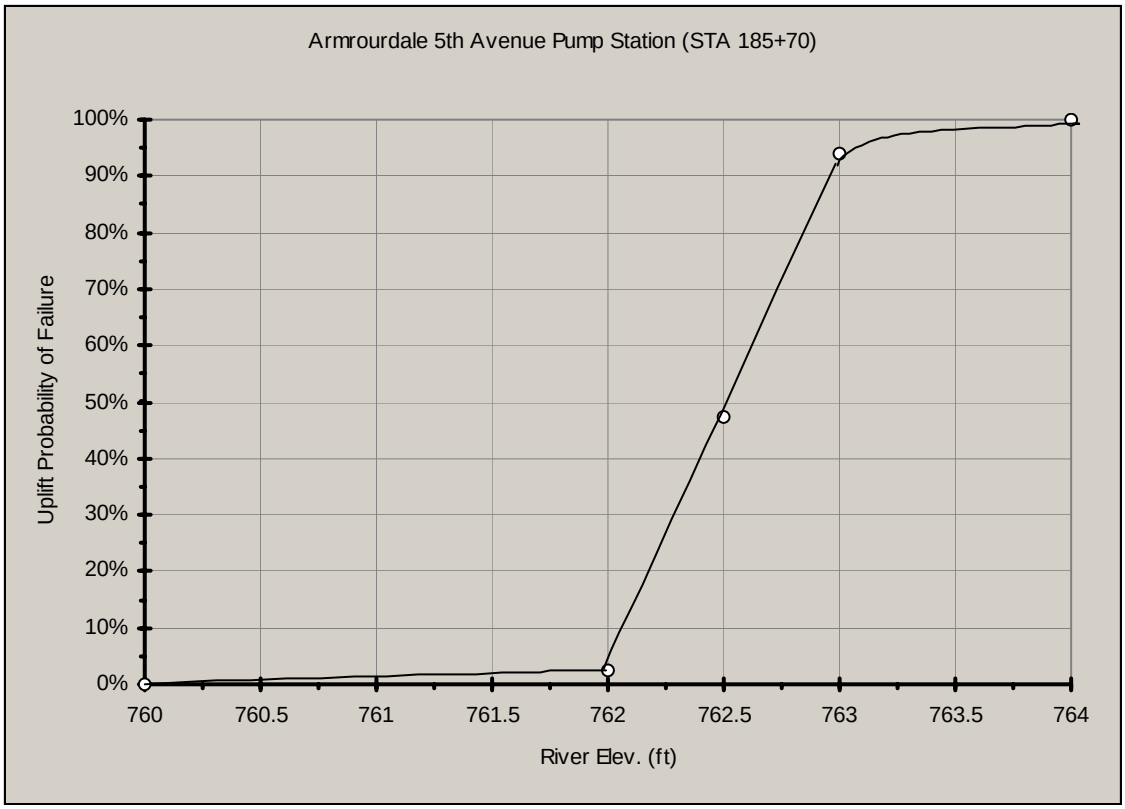


FIGURE 1. 5TH AVENUE PUMP STATION UPLIFT PROBALITY OF FAILURE VERSUS RIVER ELEVATION. SKIN FRICTION IS TAKEN INTO ACCOUNT. THE TOP OF LEVEE ELEVATION IS 767.60 FEET.

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

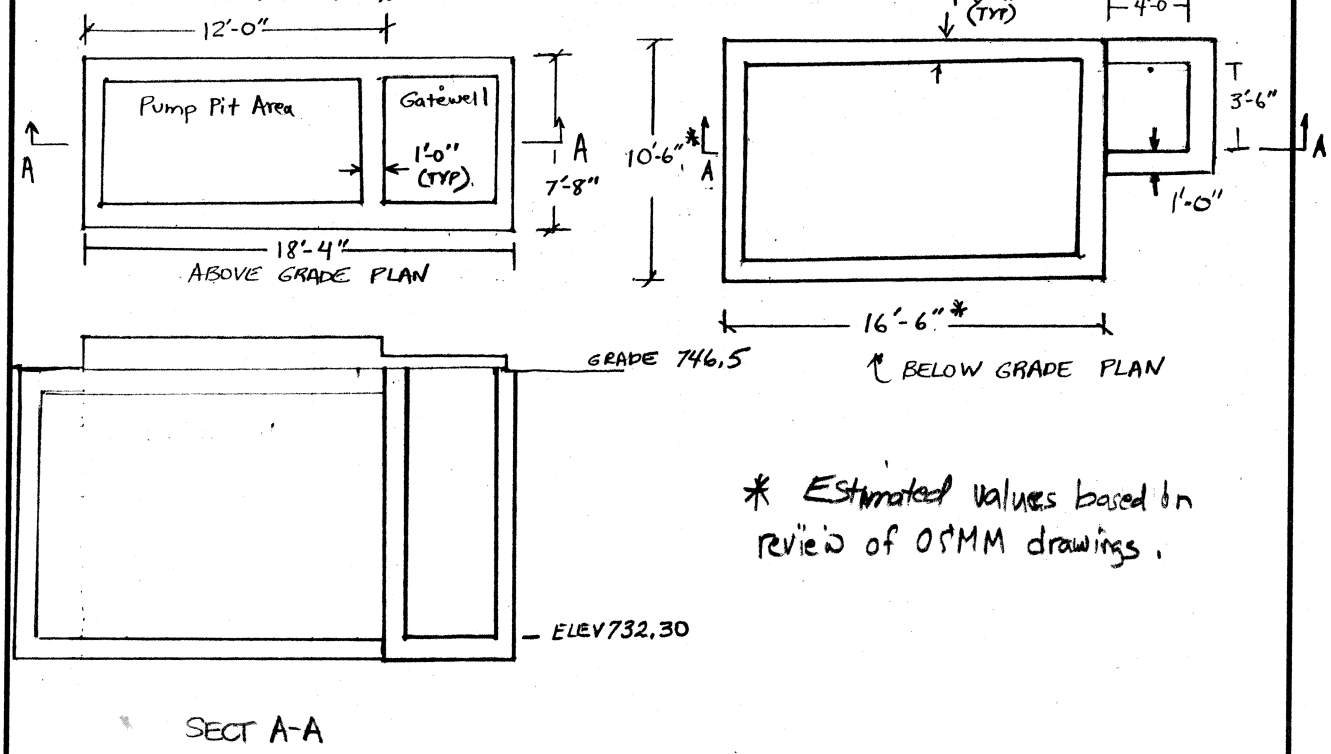
Comp. By MP Date 5/7/07 Project KC LEVEE STA 286+59.12 Sheet 1 of 6

Chkd. By TM Date 5/8/07 Subject PBI PUMP STATION ANALYSIS Sheet 1 of 6

I.

INTRODUCTION / BACKGROUND

The PBI Gordon pump station was evaluated as part of the KC Levee feasibility study. Structural drawings of the station could not be found. On February 15, 2007 a site visit to the pump station was accomplished. During the inspection, visible portions of the station were measured. Measurements taken are shown in "ABOVE GRADE PLAN"



* Estimated values based on review of OSMM drawings.

It is not known if the pump pit has heels. Measurements with "*" were obtained from OSMM PLATE NO. 54 (File A-10-1994) and are approximate only using scale. The actual dimensions were not provided. OSMM PLATE NO 56 provides the gatewell dimensions (FILE A-10-1996). The gatewell was constructed in the 1940s.

Greg Dent of PBI Gordon was contacted in efforts to obtain more information about the pump pit. Mr. Dent stated that a general site plan drawing dated 1/14/24 shows a pit. PBI Gordon has been at its current location approximately 66 years.

Based on the available information, it is believed that the pump pit was constructed in the 1920s. In the 1940s a gatewell was added to the structure as evidenced by the OSMM drawings. Plate 54 indicates that below-grade the gatewell and pump pit share a common wall.

I.

INTRODUCTION / BACKGROUND cont

Sometime after construction of the gatewell, the above grade concrete structure was built. Records of this construction could not be found. These assumptions will be used to evaluate the pump station

II. UPLIFT

Area exposed to uplift = A = 10.5 (16.5) + 5(5.5) = 173.25 + 27.5
= 200.75 ft²

Weight of structure

Base slab: W_{Slab} = 10.5(16.5)(1)150 + 5(5.5)1(150) = 25988 + 4125
= 30113 #

Pit Walls: [(10.5)16.5 − 8.5(14.5)](746.5 − 732.30) 150 = 50(14.20)150
= 710.00 (150)
= 106,500 #

Gatewell Walls: [5.5(5) − 4(3.5)] 14.20 (150) = 28755 #

Pump pit top slab weight not included due to lack of information

TOTAL Weight = W_s = 30113 + 106,500 + 28755 = 165368 #

GEOTECH INFO: ("ARM PBI Gordon PP 289+60.xls")

Bottom of Blanket El = 720.00
Top of Blanket El = 746.500
HGL, n500+3 Event = 759.60
HGL, existing condition = 758.30
Top of existing levee El = 761.70

Blanket Thickness = 746.5 − 720.0
= 26.5'

Bottom of structure El = 731.30

II. UPLIFT con't

A. n500+3 EVENT

H_1 = HGL - Bottom of Blanket = 759.6 - 720 = 39.6'

H_2 = 26.5'

H_3 = 746.5 - 731.30 = 15.20'

$$FS_{n500+3} = \frac{W_s}{U} \quad \therefore \quad U = \frac{H_1}{H_2} H_3 \, A \, \gamma_w = \frac{39.6}{26.5}(15.20)\,200.75\,(62.4)$$

$$= 284533\# \uparrow$$

$$= \frac{165368}{284533} = 0.58$$

Amount of total weight req'd to achieve $FS \geq 1.10$ $\therefore$ $1.1 = \frac{W_s'}{284.5}$

$W_s' = 312.95K$ $\therefore$ Amount of additional weight req'd : 312.95 - 165.4 = 147.6 K

Consider side friction:

$c = 5'$ = crack length

$\mu = 0.25$ = coef. of friction

$\gamma_w = 62.4$ pcf unit weight of water

$\phi = 26°$

$\gamma = 120$ pcf unit weight of soil

$K_0 = 1 - \sin\phi = 0.56$

$$P = \frac{U}{A} = \frac{284.5}{200.75} = 1417.2 \text{ psf}$$

per = 16.5(2) + 10.5 + 5(2) + 5.6 + (10.5 - 5.5) = 64.1 ft

H = 746.5 - 731.30 = 15.20'

$$R_1 = (H-c)\left(\frac{c+H}{2}\right)\gamma = (15.2-5)\left(\frac{5+15.20}{2}\right)120 = 12362 \text{ \#/ft}$$

$$R_2 = (H-c)\,p\left(\frac{c}{2H} + \frac{1}{2}\right) = (10.2)\,1417.2\left(\frac{5}{2(15.20)} + \frac{1}{2}\right) = 9605 \text{ \#/ft}$$

$$P_V = (R_1 - R_2)\,K_0\,\mu \cdot per = (12362 - 9605)\,0.56\,(0.25)\,64.1 = 24740 \#$$

II UPLIFT, n500+3 event con't

Taking skin friction into account, the n500+3 FS becomes:

$$FS_{n500+3} = \frac{W_S + P_V}{U} = \frac{165.4 + 24.7}{284.5} = 0.67$$

Amount of additional weight reqd

$$312.95 - (165.4 + 24.7) = 122.9\,K$$

If concrete is placed in the pump pit, what depth is necessary?

w/o skin friction: $8.5(14.5)d(150) = 147{,}600\,\#$

$$18487.5d = 147{,}600$$

$$d = 8\text{ ft}$$

with skin friction benefit: $18487.5d = 122900\,\#$

$$\boxed{d = 6.75\text{ ft}}$$

The gatewell will be abandoned. It should be filled w/ concrete to maintain stability during a flood event.

B. Existing Condition Event

$$H_1 = 758.3 - 720 = 38.3 \Rightarrow U = \frac{38.3}{26.5}(15.20)200.75(62.4)$$

$$= 275.2\,K$$

$$FS_{exist\ cond} = \frac{W_S}{U} = \frac{165.4}{275.2} = 0.60 \qquad \therefore P = \frac{U}{A} = \frac{275.2}{200.75} = 1371\,psf$$

Amount of additional weight req'd $= 275.2(1.1) - 165.4 = 137.3\,K$

If side friction is taken into account

$R_1 = 12362\ \#/ft$ (page 3)

$$R_2 = (H-c)P\left(\frac{c}{2H} + \frac{1}{2}\right) = (10.2)1371\left(\frac{5}{2(15.20)} + \frac{1}{2}\right) = 9292\ \#/ft$$

$$P_V = (R_1 - R_2)K_0\,\mu \cdot per = (12362 - 9292)0.56(0.25)64.1 = 27550\,\#$$

II. UPLIFT, EXISTING CONDITIONS, CON'T

$$FS_{exist\,cond} = \frac{W_s + P_v}{U} = \frac{165.4 + 27.5}{275.2} = 0.70$$

Amount of additional weight reqd = 275.2(1.1) − (165.4 + 27.5) = 109.8K

If concrete is placed in the pump pit only, what depth is necessary?

w/o skin friction: 8.5(14.5)d(150) = 137300#

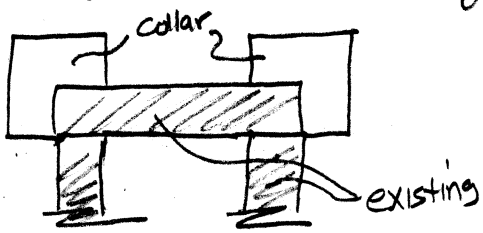
$\boxed{d = 7.5\text{ ft}}$

with skin friction: 8.5(14.5)d(150) = 109,800#

$\boxed{d = 6\text{ ft}}$

III RECOMMENDATIONS

1. Based on the available information, the PBI Gordon pump station can remain in place & perform reliably for the n500+3 event given the following caveats are followed.
2. Assume that the shear force that develops in the pump pit/gatewell wall cannot be developed for economic assessment purposes. If enough weight cannot be placed in the pump pit to achieve the necessary factor of safety against uplift, then the pump pit/gatewell wall shear strength needs to be determined & wall modified as necessary to achieve the required strength.
3. Verify the below grade pump pit dimensions by taking field measurements. (this could not be done during the Feb 15, 2007 site visit because of the cold conditions & inassessibility).
4. Analysis indicates that 6.75 feet of concrete fill is necessary in the pump pit. This is feasible since the inlet pipe is located 7 ft above the base slab. The 6.75 feet of concrete material should be used for cost purposes only. For design, the actual amount should be determined upon verification of the pump pit dimensions.
5. The pump pit concrete strength is considered adequate, given the depth of concrete fill, reducing the wall height under load. As an added measure of safety, braces could be installed against the walls (similar to braced excavation) since the concrete wall strength is unknown. Assume f_y=40ksi & f'_c=3ksi given the structure age,

III RECOMMENDATIONS con't

6. In the event that the necessary depth of concrete cannot be achieved, a concrete collar may be considered.