Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Final Feasibility Report

Chapter A-7
CENTRAL INDUSTRIAL DISTRICT UNIT PUMP STATION ANALYSIS

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Chapter A-7
CENTRAL INDUSTRIAL DISTRICT 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 were unavailable or limited for several of the facilities. Ownership and maintenance of the pump stations is by the City of Kansas City (KCMO), Unified Government of Wyandotte County / Kansas City, Kansas (UG/KCK), the Kaw Valley Drainage District (KVDD), or private, as indicated below.

TABLE A-7.1
Summary of Pump Station Available Information

Name	Ownership	Station	As-Builts?	Pump Curves?
CENTRAL INDUSTRIAL DISTRICT (CID) Unit Pump Stations:				
Broadway	KCMO	24+77	Partial	Yes
Santa Fe	KCMO	52+85	Yes	Yes
Ohio	UG/KCK	83+52	Yes	Yes
Mistletoe	UG/KCK	37+06.5	Yes	Yes
New Central	UG/KCK	58+12	Yes	No
Stockyards #3	KVDD	74+21	Yes	Yes
Gateway 2000	Private	80+90	Yes	Yes
Stockyards #1/Field pump house	KVDD	98+05	No	No
Kemper Arena	KCMO	106+49	Yes	Yes

A-7.2 Presentation of Supporting Information and Calculations

Supporting information for the pump station analysis is presented in following Exhibits included at this end of this chapter:

Exhibit 1 – Pump Station Information and Photographs
Exhibit 2 – Geotechnical Data (Hydraulic Grade Line Caluclations)
Exhibit 3 – Structural Calculations

A-7.3 Methods of Analysis

Pump stations are generally evaluated both for structural reliability and, in cases of relief wells servicing actively pumped relief well systems, hydraulic capacity. No CID pump stations handle seepage flow from relief wells discharging below grade, therefore hydraulic capacity was not analyzed unless the local Sponsor indicated problems. Detailed analyses were conducted for existing conditions in an effort to define the risk of failure under current project conditions; and for the n500+3 design loading to determine the required modifications to accommodate the higher loading associated with a prospective levee or floodwall raise. Two other design

conditions, n500+0 and n500+5 were also considered, but primarily by engineering judgment rather than detailed analysis. The purposes of the two additional design points is 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 the case of Stockyards #1/ Field pump house, details of the original construction were not available. Normally, with a lack of design documentation, assumptions would be made based on standard practice at the time of construction or previous findings on similar structures. In this particular case, since the pump station was deemed hydraulically unnecessary, it was not analyzed. 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 or proposed 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.

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 CID line of protection resulted in the findings and recommendations below. Existing and future condition findings and recommendations are also presented in Tables A-7.2 and A-7.3, respectively.

Broadway Pump Station. The Broadway Pump Station is owned and operated by the City of Kansas City, MO and is located on the CID MO unit. It handles interior drainage and uncontrolled seepage flow. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. The floodwall section in this area is not expected to change, i.e. there is no raise proposed.

- The structural evaluation indicates that the station is marginally acceptable for strength, and acceptable for flotation, with water to the top of the existing line of protection.
- The existing pump discharge configuration is a discharge through flap valves into a discharge chamber connected to the gravity discharge pipe passing under the floodwall and discharging to the Missouri River. The existing gravity discharge is a 54” reinforced concrete pipe. Corps staff were unable to locate drawings so as to determine the strength of the 54” line.
- The Broadway pump station was designed and constructed as part of the original Corps of Engineers flood control project. Since record drawings for the structure were available and used for the analysis, additional testing during the next project phase is not required. The discharge line, however, should be inspected and / or tested to determine its strength.
- N500+0 design conditions would essentially be existing conditions, since the existing line of protection is already roughly above this level. Only inspection of the discharge pipe would be required.
- N500+3 and N500+5 design conditions would involve raising the line of protection by several feet. Since strength is marginal under existing conditions, improvements such as bracing or addition of a web wall would likely be required. Stability would likely be acceptable under this condition.

Santa Fe Pump Station. The Santa Fe Pump Station is owned and operated by the City of Kansas City, MO and is located on the CID MO unit. It handles interior drainage and uncontrolled seepage flow. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. The floodwall section in this area is not expected to change, i.e. there is no raise proposed.

- The structural evaluation indicates that the station is acceptable for strength, and marginally acceptable for flotation, with water to the top of the existing line of protection.
- The existing pump discharge configuration is a discharge through flap valves into a discharge chamber connected to the gravity discharge pipe passing under the floodwall and discharging to the Missouri River. The existing gravity discharge is a 10' reinforced concrete pipe. Strength of the discharge pipe was found to be acceptable under existing conditions.
- The Santa Fe 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.
- N500+0 design conditions would essentially be existing conditions, since the existing line of protection is already roughly above this level. No modifications would be required.
- N500+5 design conditions would involve raising the line of protection by several feet. Strength would likely be acceptable under this condition. While stability was considered marginal under existing conditions, the decrease in factor of safety for stability would likely be very small for this design condition. Therefore, stability would also likely be acceptable, and only the discharge chamber and extension of the sluice gate stem would be required.

Ohio Pump Station (KCK FPS #1). The Ohio Pump Station is owned and operated by the Unified Government of Wyandotte County / City of Kansas City, KS. While it is physically located in Kansas, the discharge crosses the state line and CID MO unit, and flows to the Missouri River. It handles interior drainage and uncontrolled seepage flow. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. The levee section in this area is not expected to change, i.e. there is no levee raise proposed.

- Structural evaluations indicate that the pump station is acceptable for flotation with water to the top of the existing line of protection. Strength of the pump station is unacceptable for existing conditions. Structural modifications are required to meet strength criteria. These required modifications are discussed in detail in Exhibit 4 – Structural Calculations.
- The existing pump discharge configuration is a discharge through flap valves into a discharge chamber connected to the gravity discharge pipe passing under the levee and discharging to the Missouri River. The existing gravity discharge is a 42" reinforced concrete pipe. Strength of the discharge pipe was found to be acceptable under existing conditions.
- The Ohio pump station was designed and constructed prior to Corps of Engineers involvement in the flood control project, and was later modified by the Corps. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.
- N500+0 design conditions would essentially be existing conditions, since the existing line of protection is already above this level. Since strength is

unacceptable for existing conditions, similar structural modifications as discussed above would be required.

- N500+5 design conditions would involve raising the line of protection by several feet. Since strength is unacceptable for existing conditions, similar structural modifications as discussed above would be required. Stability would likely be acceptable under this condition. Therefore, only raising of the discharge chamber and extension of the sluice gate stem would be required.

Mistletoe Pump Station. The Mistletoe Pump Station is owned and operated by the Kaw Valley Drainage District and is located on the CID KS unit. It handles interior drainage and uncontrolled seepage flow. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. A floodwall raise of roughly 1.5 feet (n500+3 level of protection) is proposed in this area.

- Structural evaluations indicate that the pump station is acceptable for strength and flotation with water to the top of the existing and proposed lines of protection.
- The existing pump discharge configuration is a discharge through flap valves into a discharge chamber connected to the gravity discharge pipe passing under the levee and discharging to the Kansas River. The existing gravity discharge is an 18" cast iron pipe. Although detailed inspection of the discharge line was not possible, unprotected ferrous pipe may experience pitting corrosion. For this reason, replacement of the pipe or lining with a cured in placed pipe (CIPP) designed for the fully deteriorated condition is recommended. The Mistletoe station is integral with the floodwall. For any line of protection raise, the top of the discharge chamber will need to be raised to coincide with the line of protection. Raising of the discharge chamber will require extension of the sluice gate stem.
- The Mistletoe pump station was designed and constructed prior to Corps of Engineers involvement in the flood control project, and was later modified by the Corps. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.
- N500+0 design conditions would essentially be existing conditions, since the existing floodwall is already roughly 1.5 feet above this level. Only replacement or rehabilitation of the discharge pipe would be required.
- N500+5 design conditions would involve raising the line of protection by roughly 3.5 feet. Since both strength and stability have more than adequate factors of safety for the N500+3 condition, both would likely be acceptable for this condition. Therefore, only raising of the discharge chamber and extension of the sluice gate stem would be required.

New Central Ave Pump Station (KCK FPS #16). The New Central Ave. Pump Station is owned and operated by the Unified Government of Wyandotte County / City of Kansas City, KS and is located on the CID KS unit. It handles interior drainage. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. A levee raise of 2 feet (n500+3 level of protection) is proposed in this area.

- Structural evaluations indicate that the pump station is acceptable for strength and flotation with water to the top of the existing and proposed lines of protection

- The existing pump discharge configuration is a gravity drain under the levee and twin ductile iron pump discharge lines "up and over" the levee, discharging into a gatewell on the riverside levee crest. The existing gravity discharge is a 84" reinforced concrete pipe. Strength of the discharge pipe was found to be acceptable under both existing and proposed conditions. Strength and stability of the gatewell is adequate. 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. Pumps will also require replacement with more powerful pumps because the discharge lines are at a higher elevation.
- The New Central Ave. pump station was designed and constructed in 1987 by local interests after the Corps of Engineers involvement in the flood control project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.
- N500+0 design conditions would essentially be existing conditions, since the existing floodwall is already roughly 1 foot above this level.
- N500+5 design conditions would involve raising the line of protection by roughly 4 feet. Strength would be marginally acceptable because the factor of safety would likely dip slightly below 1.5. Stability has a more than adequate factor of safety for n500+3 and would likely be acceptable for this condition. Therefore, raising of the discharge chamber, extension of the sluice gate stem, relocating the up and over pipes, and new pumps would be required.

Stockyards #3 Pump Station. The Stockyards Pump Station is owned and operated by the Kaw Valley Drainage District and is located on the CID KS unit. It handles interior drainage and seepage flow from relief wells between Station 65+00 and 82+00. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. A levee raise of 2 feet (n500+3 level of protection) is proposed in this area.

- Structural evaluations indicate that the pump station is unacceptable for flotation with water to the top of the existing line of protection. Addition of weight or control of subsurface hydrostatic pressure is required to alleviate flotation issues. Strength of the pump station is also unacceptable for existing conditions. Structural modifications are required to meet strength criteria. Abandon the station and the relief well system in place and replace the entire system with area fill and re-grade.
- The existing pump discharge configuration is a gatewell at the pump station and a pressurized pipe under the levee. Strength of the discharge pipe was found to be acceptable under existing conditions. The existing gravity discharge is a 24" cast iron pipe. Stability and the strength of the gatewell is unacceptable, for existing conditions. The pump station will be abandoned in place. The 24" pipe will be grouted as will all incoming pipes that service the relief well system. All mechanical and electrical equipment in the pump station will be removed.

- The Stockyards 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.
- N500+0 design conditions would essentially be existing conditions, since the existing conditions are 1 foot above this level. If there would be no levee raise, this station would not be abandoned and it's floatation and discharge pipe problems would be addressed.
- N500+5 design conditions would involve raising the level of protection by 4 feet. Since the pump station is to be abandoned and filled in at n500+3, then it would also be abandoned and filled in for this design condition.

Gateway 2000 Pump Station. The Gateway 2000 Pump Station is owned and operated by private interests and is located on the CID KS unit. It was designed to handle interior drainage in the area surrounding Gateway 2000. Structural evaluations indicate that the pump station is acceptable for flotation with water to the top of the existing line of protection. Strength of the pump station is also acceptable for existing conditions. A levee raise of 3.6 feet (n500+3 level of protection) is proposed in this area.

- The existing pump discharge configuration is a discharge through flap valves into a gatewell/discharge chamber connected to the gravity discharge pipe passing under the levee. Strength of the discharge pipe was found to be acceptable under existing conditions. The existing gravity discharge is a 60" reinforced concrete pipe. Stability of the gatewell is acceptable and the strength of the gatewell is acceptable, for existing conditions. Strength and stability of the gatewell is adequate. For any line of protection raise, the top of the gatewell will need to be raised to coincide with the new line of protection. Strength of the base slab will need to be increased by modifying. Adding FRP to the base slab is a recommended method for modification. Raising the gatewell will also require replacement of the sluice gate stem, installation of a new gate hoist, and installation of a new stem.
- The Gateway 2000 pump station was designed and constructed in 1994 by local interests, after the Corps of Engineers involvement in the flood control project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.
- N500+0 design conditions would involve raising the level of protection by 0.5 feet. Since the pump station is adequate at protection level n500+3 it would be adequate at this design condition.
- N500+5 design condition would involve raising the level of protection by 5.5 feet. Since the pump station is more than adequate at the protection level n500+3 it will likely be adequate at this design condition. The gatewell will likely require additional strengthening as indicated for N500+3 conditions.

Stockyards #1/ Field Pump House Pump Station. The Stockyards #1 Pump Station is owned and operated by the Kaw Valley Drainage District and is located on the CID KS unit. The station was originally designed to handle a small amount of interior drainage. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore

hydraulic analyses were not conducted. A floodwall raise of 2 feet (n500+3 level of protection) is proposed in this area.

- The existing pump discharge pumps to a 24" cast iron pipe. Stability and strength are adequate under existing conditions. This pump station pumps are likely inadequate and have not needed to run for over 20 years. This pump station will be abandoned in place. The 24" cast iron pipe and valve manhole will be grouted. All mechanical equipment will be removed.
- N500+0 design conditions would essentially be existing conditions, since the existing conditions are 1 foot above this level. The pump station would still be abandoned in place.
- N500+5 design conditions would involve raising the levee protection by 4 feet. The pump station would still be abandoned in place.

Kemper Arena Pump Station. The Kemper Arena Pump Station is owned and operated by the City of Kansas City, Missouri, and is located in the CID KS unit. The Station handles interior drainage from Kemper Arena and the American Royal area. Structural evaluations indicate that the 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. Strength of the pump station is acceptable for existing conditions. Structural modifications are required to meet strength criteria.

- The pump station discharges into a 6' x 6' reinforced concrete box the passes under the floodwall, with a landside gatewell, at Sta. 106+49. Strength of the discharge pipe was found to be acceptable under existing conditions. Stability of the gatewell is acceptable and the strength of the gatewell is marginally unacceptable, for existing conditions. Structural modification, such as adding FRP to strengthen the walls, is required to meet strength criteria.
- Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.

A-7.5 Reliabilities

Probability of failure curves for Broadway and Ohio pump stations were used in the economic model.

Calculations for the Broadway pump station revealed a 0.8% probability of failure for wall strength with water at the top of the flood protection.

Calculations for the Ohio pump station revealed a 100% probability of failure with water at top of flood protection, driven primarily by inadequate strength of key structural members.

TABLE A-7.2

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
Broadway (KCMO)	24+77	1.8	1.11 R=99.15%	2.46	2.0	1.22	54" RCP Requires Insp	
Santa Fe (KCMO)*	52+85	2.64	1.53	3.98	2.2	1.28[A]	120" RCP	Structure is supported by piles. Piles were not used to benefit against uplift.
Ohio (KVDD)	83+52	1.92	0.44 Not Reliable	0.76 Reliability not performed	3.63	1.59	42"RCP	Deterioration of station interior components due to corrosive environment observed during 2007 inspection
Mistletoe (KVDD)	37+06.5	1.61	2.25	3.54	2.13	1.62	18"CIP Requires Insp.	
New Central (KVDD)	58+12	1.59	2.19	4.16	2.86	2.0	84" RCP	
Stockyards #3 (KVDD)	74+21	1.54	1.97	1.76	2.1	1.16[B]	24" CIP Requires Insp.	
Gateway (KVDD)	80+90	2.3	>1.5	2.53	1.87	1.61	60"RCP	
Kemper Arena (KCMO)	106+49	2.77	1.99	1.71	1.92	1.15	6' x 6' RCB Shear 2.45 Moment 2.91	Station stability of concern with HGL above EC elevation

Strength: 1.5 FS required per established criteria
Stability: 1.1 FS required per established criteria
*Pump station is pile supported. The piles were not analyzed.

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
R=Reliability
TBD: To Be Determined

"Superscripted Notes:
A. After use of soil friction on walls. Before soil frction, the FS = 1.02
B. After use of soil fricitn on walls. Before soil friction the FS = 0.9

TABLE A-7.3

Revision Date: 5/6/2008

FUTURE CONDITIONS CID PUMP STATION ANALYSIS/REHAB PROGRESS STATUS

Draft - subject to final reviews

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	Outlet Pipe Reliability	Pump Station Peer Reviewed	Pump Sta Backchecked	Pump Sta Ready for ITR	Outlet Works Peer Reviewed	Outlet Works Backchecked	Outlet Works Ready for ITR	Comments	HGL_{FC} - HGL_{EC} (ft)	BOB_{FC} - BOB_{EC} (ft)
Broadway (KCMO)	24+77						54" RCP Requires Insp	**On hold**							0	
Santa Fe (KCMO)*	52+85						120" RCP	**On hold**							0	
Ohio (KVDD)	83+52	1.92	FS=0.44 Braced Frame in Wetwell	FS=0.76 R/C Overlay/ Shorten pump intakes	3.63	1.59	42" RCP	X	X	X	X	X	X	Address deterioration. Check with Paul/Katrina about attached gatewell. Verify that no additional fill will be placed over pipe. Pipe is okay with no additional fill.	0	0
Mistletoe (KVDD)	37+06.5	1.56*	2.19*	3.44*	2.39*	1.58*	18"CIP Requires Insp.	X	X	X	X	X	X	Recommend CIPP for economic analysis due to pipe age and unknown condition.	1.13	-3
New Central (KVDD)	58+12	1.50*	2.09*	4.00*	2.75*	1.95*	84" RCP	X	X	X	X	X	X		1.33	-2
Stockyards #3 (KVDD)	74+21						24" CIP Requires Insp.	**On hold**			`			Recommend CIPP for economic analysis due to pipe age and unknown condition.	2.07	2
Gateway (KVDD)	80+90	2.13*	>1.5	2.47*	1.82*	1.43*	60"RCP	X	X	X	X	X	X		2.84	3
Kemper Arena (KCMO)	106+49						6' x 6' RCB Shear 2.45 Moment 2.91	**On hold**							unk	

*** Deterministic Value was used to obtain the values shown, per e-mail sent by M.Parks on 5/9/08.**

Rehabilitation Required

FS equal to or greater than 1.5, no rehabilitation required

Strength: 1.5 FS required per established criteria for shear and moment
Stability: 1.1 FS required per established criteria
*Pump station is pile supported. The piles were not analyzed.

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

x

A-7.6 Analysis of New Relief Well Flows

The following is a conceptual analysis of the impacts that proposed relief wells will impose on existing infrastructure in two locations located along the Kansas River, in both Missouri and Kansas. Maps of each location are included at the end of the discussion.

Kansas River. Sta. 107+00 - Sta. 116+70

Eighteen (18) surface discharging relief wells are proposed along the Kansas River between Sta. 107+00 and Sta. 116+70, adjacent to the American Royal Building (south of Kemper Arena) and the American Crane and Tractor building, south of the intersection of American Royal Drive and Genessee Street. The proposed quantity of stormwater that will be discharged by the relief wells is 18 cfs (8,100 gpm).

The well discharge elevations will be approximately 753. The top elevation of the current floodwall within this area is approximately 765. Therefore, the Kansas River will still have 12 feet of available freeboard before cresting the current floodwall prior to surface discharging occurring at this location. Once the river elevation equals the elevation of the relief wells, the wells will begin discharging to the surface.

Once the wells begin to discharge, stormwater will begin to pond within the existing rail yard and green space north and east of the existing American Crane and Tractor building. Approximately 91,000 cubic feet of storage is available in this area. Per the current information available for this area, it does not appear that an existing storm sewer system is available to drain this low area. However, 18 cfs is a relatively small amount of stormwater and should be able to be handled by most storm sewer systems, with minimal ponding occurring if there is in fact an existing drainage system available that drains this existing low area. Assuming full flow and no existing storm sewer (worst case), this low area would take approximately 1.4 hours to fill up, prior to overflowing onto the curb and gutter system along American Royal Drive.

Upon overflowing onto American Royal Drive, the existing enclosed storm sewer system along American Royal Drive has the capacity to handle the discharge from the wells. However, if the stormwater does not get into the enclosed system due to additional rainfall falling within the drainage area or clogging of the existing inlets, the stormwater will overtop the curb and pond in the parking lot west of the American Royal Building. There is approximately 31,000 cubic feet of storage available in this area. This low area is currently drained by an existing enclosed storm sewer system. Assuming that this inlet is clogged, it would take approximately 30 minutes to fill this area and then begin to flood the existing adjacent buildings. Presumably the existing inlets will be free of clutter and allow proper drainage of the well discharge. There are numerous inlets along American Royal Drive and the parking lot west of the American Royal Building that has adequate capacity to handle the additional stormwater. The vertical information for the existing storm sewer was not available for this analysis. The assumption was that the pipes were laid to the absolute minimum slope approved by APWA, which is 0.08% for the 36" RCP that drains this area. Assuming no additional surface runoff other than the wells, the 36" RCP at a 0.08% slope has adequate capacity to handle the additional flow.

Therefore, based on the information available, the existing enclosed storm sewer downstream of Sta. 107+00-Sta. 116+70 has sufficient capacity to handle the additional 18 cfs from the surface discharge from the proposed relief wells, assuming that no additional surface runoff was occurring with the drainage area.

During the instance where the relief wells are flowing and a small rainfall event occurs, the existing enclosed storm sewer system would be required to handle an additional 9 cfs of surface runoff for a total runoff amount of 27 cfs. Assuming that the existing storm sewer system has a slope greater than 0.20%, the existing system will have adequate capacity to handle the relief well discharge, in addition to the small rainfall event runoff.

Kansas River, Sta. 127+00 - Sta. 166+83

Eighty three (83) surface discharging relief wells are proposed along the Kansas River between Sta. 127+00 and Sta. 166+83, adjacent to existing railroad buildings and infrastructure. The specific uses and vacancies for these buildings is not known at this time. The proposed quantity of stormwater that will be discharged by the relief wells is 108 cfs (48,474 gpm). These wells release stormwater into two watersheds, split by the KC Terminal Railroad Bridge at approximately Sta. 132+00. For the purpose of this report, the total 108 cfs has been prorated between each drainage area by stationing. Therefore, approximately 94.4 cfs (42,369 gpm) will drain to the south and approximately 13.6 cfs (6,104 gpm) will flow to the north.

Sta. 127+00- Sta. 132+00

The well discharge elevations for this area will be approximately 758, which is seven feet below the current top of floodwall elevation of 765.

As the river begins to rise above the top elevations of the well, stormwater will begin to discharge and eventually start to pool along Genessee Street. Per the current available information, there is an existing storm drainage system, but the details of said system are not known. It appears that this enclosed storm sewer discharges directly into the Turkey Creek Outfall Sewer. This area of ponding has an estimated storage volume of 156,000 cubic feet and will be two feet deep at its deepest point, if the existing storm sewer system is non existent or inundated. If the stormwater does begin to pond, it will take 3.2 hours for this area to fill up and close Genessee Street. If the existing storm sewer system happened to be clogged or inundated and does not allow the proper passage of stormwater, nearby buildings will begin to experience flooding. However, the drainage area for this low area is quite significant. If there are no current flooding issues at this location, the enclosed storm sewer system should be adequately sized to handle the 13.4 cfs from the relief wells.

Therefore, presuming the enclosed storm sewer exists and is sufficiently sized for its drainage area, the area between Sta. 127+00 and Sta. 166+83 should not experience significant flooding due to the additional discharge from the relief wells assuming that no additional surface runoff was occurring with the drainage area. Unfortunately, there is insufficient utility and topographical information to determine if a small rainfall event over the drainage area while the relief wells are flowing will cause any adverse flooding.

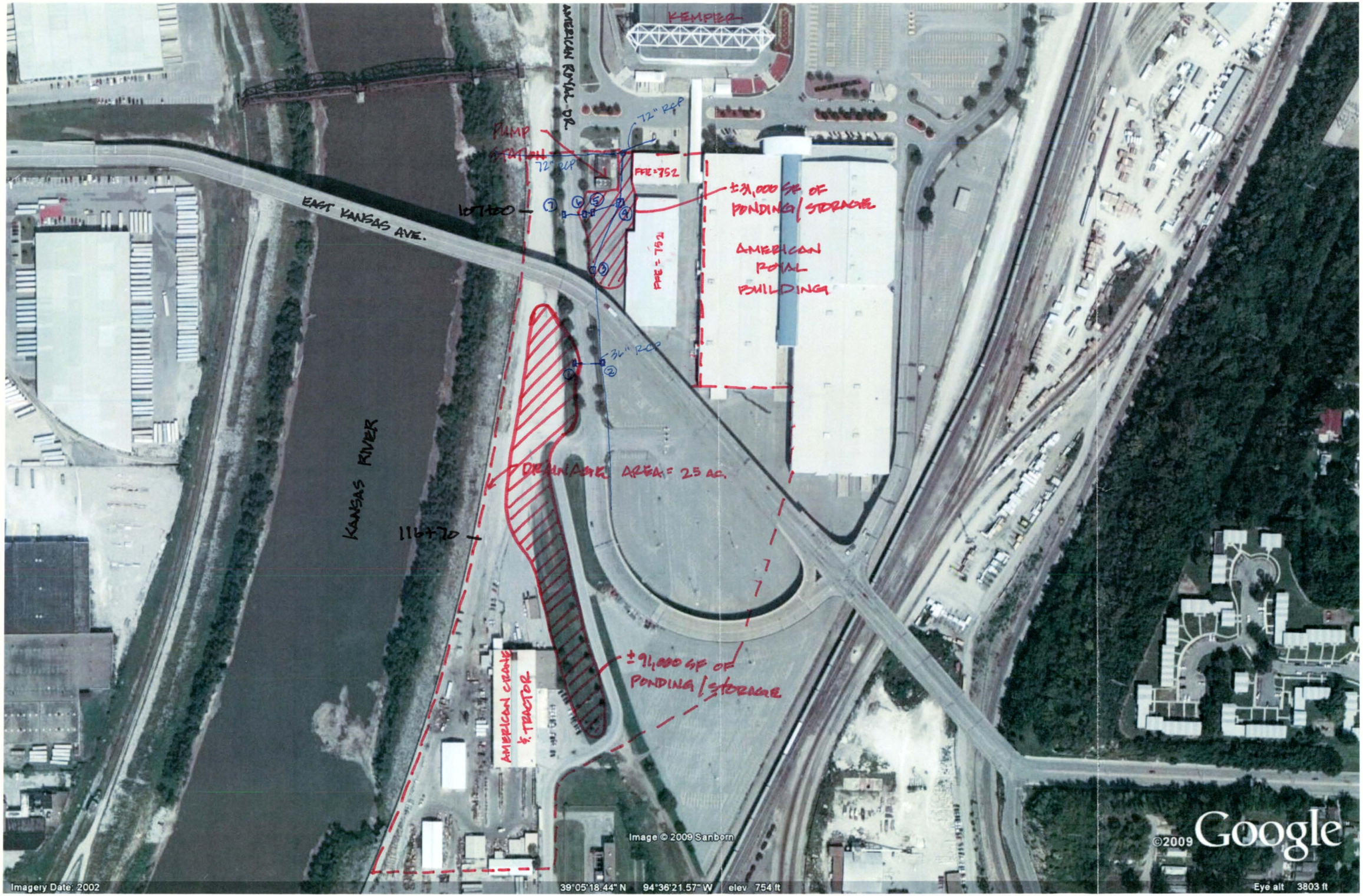
Sta. *132+00* - Sta. 166+83

The well discharge elevations for this area will range between 753 and 756, which is 9 - 13 feet below the current top of floodwall elevation of 765 for this stretch of the river.

Once stormwater begins to discharge from the relief wells, it will travel through the existing rail yard and eventually start to pond along the southeast side of the rail yard. Approximately 303,000 cubic feet of storage (at a depth of one foot) is available for the stormwater to collect. Assuming no direct outfall from this low area, it would take 0.9 hours for this area to fill up and then begin to adversely affect nearby buildings. However, per storm sewer atlas maps, numerous existing enclosed storm sewer systems are located in the drainage area of these relief wells. However, as built conditions for these sewers were not available to analyze their current capacities. Similar to the northern drainage area, the southern drainage area is quite large, roughly 80 acres. Assuming that the existing storm sewer systems have been sized to handle the normal rainfall events, they should have adequate capacity to handle the 94.4 cfs that is proposed to discharge from the relief wells. Based on a quick Rational Method calculation, 80 acres of undeveloped land and a five minute time of concentration (worst case scenario) would produce over 175 cfs of runoff during the typical 10-year storm event. Presuming the storm sewers are adequately sized to handle the 10-year storm event, the enclosed system would be adequately sized to handle the additional stormwater generated from the relief wells. If the existing storm sewers are slightly undersized, the broad, large storage area should provide enough ponding for the stormwater to safely discharge without adversely affecting any buildings. Therefore, presuming the enclosed storm sewer is adequately sized to handle the 10-year storm event for its drainage area, the area between Sta. 132+00 and Sta. 166+83 should not experience significant flooding or infrastructure damage due to the additional discharge from the relief wells assuming that no additional surface runoff was occurring with the drainage area. Unfortunately, there is insufficient utility and topographical information to determine if a small rainfall event over the drainage area while the relief wells are flowing will cause any adverse flooding.

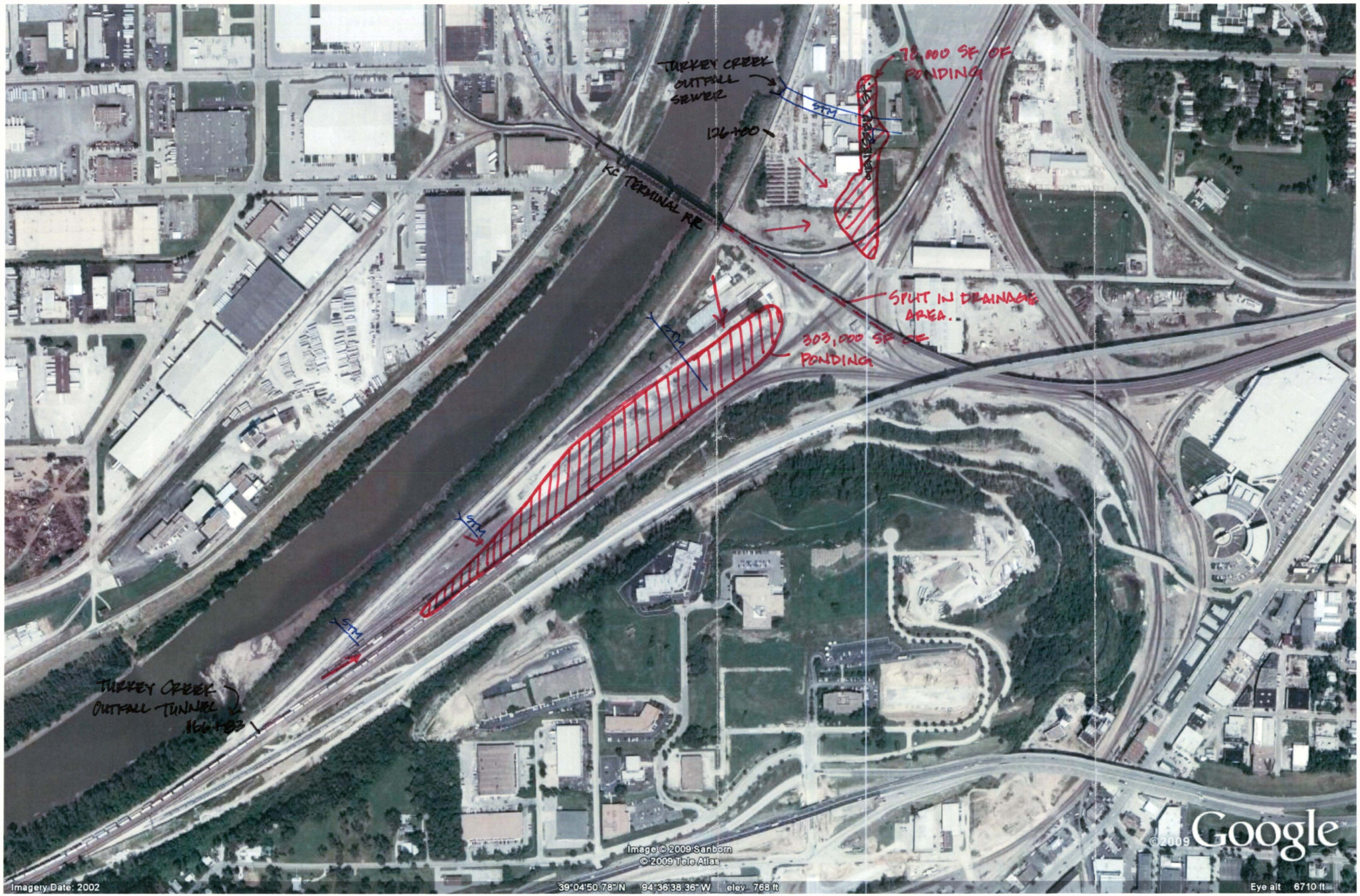
Relief Well Flow Recommendation

Based on the discussion presented above, it is recommended to allow the new relief wells to discharge at ground surface. No work is proposed to provide a new collector system nor modify or construct pump stations specifically for these flows.



STA. 107+00 — 116+70

07/14/09 SPM



STA. 127+00 - 166+83

07/15/09 SPM

EXHIBIT 1 - PUMP STATION INFORMATION AND PHOTOGRAPHS

1. Broadway

    a. Year Constructed: 1949[11]
    b. Owner/Operator: KCMO
    c. Levee Station: 24+75 (MO)
    d. Distance From Flood Protection Centerline: 35ft[7]
    e. Station Capacity: 3,450gpm[1]
    f. Pumps: --
    g. Contributing Flows:
        i. Seepage flow (from seepage system between Sta. 22+82 and Sta. 0+00)[1] NOTE: According to KCMO staff no seepage flow comes into pump station [11]
        ii. Storm runoff (flow is contributed by the railroad yards area, 28ac, in the northeastern part of the Central Industrial District)[1]
    h. River Discharge: 54" RCP[1]
    i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under levee[7]
    j. Drawing Availability:
        i. Feb 1981 – General Plan View[7]
        ii. Dec 1955 – Gatewell Plans[7]
    k. General
        i. There is a floodwall at the pump plant location. [3]
        ii. The floodplain contains one small depression available for storage of surface runoff and seepage. The area which is protected by the floodwall is occupied by the Missouri Pacific Railroad yards and the Missouri Produce Company's railroad yards. [3]
        iii. Seepage begins at 29.5 and is, therefore, negligible at design river stage. [3]
        iv. The storm runoff is greater in design flood stage because there would be no infiltration over approximately one-half of the normally pervious area. [3]
        v. The City of Kansas City, Missouri stated that the Broadway Pump Station drains the hillside area and the railroad tracks, so there is almost always flow going through the station. However, the only time that the pumps were needed was in 1993. The pumps are the original pumps. [3]
        vi. There were plans to rehabilitate the pump station a few years ago, but the work was cancelled. The City did note, though, that new flap gates were installed because they had already been ordered. The City does not know if the remainder of the plans will be carried out in the future. [3]
        vii. The conditions of the service area for this pump plant have not gone through any significant changes since the design of the plant. The estimated percent impervious is from visual inspection of 1996 aerial photographs. [3]

2. Santa Fe Pump Station

    a. Year Constructed: 1949[11]
    b. Owner/Operator: KCMO[3]
    c. Levee Station: 52+85 (MO)
    d. Distance From Flood Protection Centerline: 45ft[7]
    e. Station Capacity: 42,500gpm[1]
    f. Pumps: --
    g. Contributing Flows:
        i. Seepage flow from the seepage system between Sta. 78+00 to Sta. 22+82. [1]
        ii. Storm sewer system flow contributed by a bottoms area, 546ac, and a high bluff area in the northern portion of the Central Industrial District. [1]
    h. River Discharge: 10' RCP[1]
    i. Discharge Arrangement: Gatewell & pressurized pipe under floodwall[7]
    j. Drawing Availability:
        i. Feb 1981 – General Plan View[7]
        ii. Dec 1955 – Gatewell Plans[7]
    k. General
        i. There is a floodwall at this pump plant location. [3]
        ii. The Armour Packing Plant area has a pump plant that is adequate, but the sewer surcharges in the area where the flow would head to the Santa Fe Pump Plant. Therefore, the Armour Packing Plant area is considered undrained. [3]
        iii. In 1962, modifications were done to divert the sanitary flow to the West Side Sewage Treatment Plant. In doing so, the capacity of the stormwater side of the pump plant was decreased. Two of the pumps which were used previously for stormwater pumping became dedicated to sanitary flow pumping. [3]
        iv. HNTB performed a study on the Santa Fe Pump Station in 1994, which was followed by some renovation of the plant in about 1995. Pumps were rehabilitated, but capacities were not increased. [3]
        v. The drainage area determined for the 1994 HNTB report is slightly larger than that originally estimated by the COE. It was assumed in the report that the lesser area may have deducted non-contributing depressed areas that were present at the time. [3]
        vi. A small amount of additional development has taken place in the Santa Fe Pump Station service area which is reflected in the percent impervious. The estimated percent impervious was determined by visual inspection of 1996 aerial photographs. As the capacities of the pump station were not increased in 1994, the deficiencies still exist. [3]
        vii. The pumping capacity may or may not be adequate based on the impacts of the increased percent impervious. [3]

3. Ohio (KCK FPS 1)

a. Year Constructed: 1945 (Reconstructed 1950) [4]
b. Owner/Operator: KCK
c. Levee Station: 83+52 (MO) [1]
d. Distance From Flood Protection Centerline: 170ft[7]
e. Station Capacity: 32,650gpm[4]
f. Pumps:
    i. Worthington Impeller Centrifugal 16" design capacity 8,400gpm 870RPM, drive 100hp[4] Appears to have been removed.[9]
    ii. Fairbanks-Morse impeller axial flow 16" design cap. 6,800gpm 1,200RPM, drive 50hp[4]
    iii. Fairbanks-Morse impeller axial flow[4] 24" design cap. 17,4500gpm@17.3TDH 880RPM, drive 100hp[7]
g. Contributing Flows:
    i. Seepage flow from the area between sta. 00+00 and approx. sta. 26+00, plus the seepage from approx. sta. 46+00 to approx. sta. 65+00. [1]
    ii. Storm sewer system flow contributed by the area, 110ac, just south and southwest of the junction of the Missouri and Kansas Rivers. [1]
h. River Discharge: 42" RCP[1]
i. Discharge Arrangement: Gatewell & pressurized pipe under levee[7]
j. Drawing Availability:
    i. Oct 1950 – Plan & Profile[7]
    ii. Jul 1950 – Mechanical, Electrical & Structural Details[7]
    iii. Oct 1943 – Structural plan [5]
k. General
    i. Approximately half of the topography is very flat, with the remainder having moderate slopes. There is one small depression located near the state line available for surface storage. [3]
    ii. The outlet is located at mile 378.43 on the Missouri River. Critical stage is 22.0 feet (where the river begins to back up into the pipe), but the pumps are not started until 28.0 feet. [3]
    iii. As noted in the 1948 "Supplement on Interior Drainage", one of the stormwater pumps cannot be depended on for any discharge in the Design Flood Condition as the total dynamic head is in excess of the maximum head for which it is rated. [3]
    iv. The City of Kansas City, Kansas owns and operates the pump plant. They call it Flood Pump Station #1. The equipment listed for the plant includes the original pumps, although some rehabilitation work may have been done in recent years. HNTB is in the process of retrieving information pertaining to the possible rehabilitation. [3]
    v. Sanitary flow now goes to the treatment plant and is, therefore, not included here. There is a minor reduction with respect to total flow contributing to the plant as a result. [3]
    vi. Commercial development has contributed to a greater amount of impervious area within the service area. The estimated percent impervious is from visual inspection of 1996 aerial photography. The

greater impervious area would result in an increase in runoff from the area. [3]

vii. The total flow reaching the plant remains the same in the Design River Condition because the sewer system capacity controls. [3]

4. Mistletoe Pump Station

a. Year Constructed: --
b. Owner/Operator: KVDD
c. Levee Station: 37+07 (KS) [1]
d. Distance From Flood Protection Centerline: 0ft[7]
e. Station Capacity: 3150gpm[1]
f. Pumps:
    i. 8” 2,660gpm@15ft RDH 700RPM Centrifugal Sewage, dry pit[7] 15hp motor[10]
    ii. 8” 2,660gpm@15ft RDH 700RPM Centrifugal Sewage, dry pit[7] 15hp motor[10]
    iii. Sump Pump 20gpm@20ftTDH 116RPM[7]
g. Contributing Flows:
    i. Seepage flow the area between approx. Sta 26+00 and approx. Sta. 46+00. [1] Floodwall Toe Drain[10]
    ii. Storm runoff flow contributed by the area, 7ac, immediately adjacent to the pump station. [1]
h. River Discharge: 18” CIP[1]
i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under floodwall[7]
j. Drawing Availability:
    i. July 1981 – General Plan & Profile[7]
    ii. July 1950 – Mechanical, Electrical, Structural & Architectural Details[7]
k. General
    i. The ponding that would occur in the plant at the design flood stage should not be damaging. It should be about 17 minutes in duration and cover about .003 acre-feet. [3]
    ii. The plant is owned and operated by KAW Valley Drainage District. Larry Brennan, manager of the district, stated that the only time the Mistletoe Plant ever ran was in 1993. [3]
    iii. The only change with respect to plant design is that the sanitary flow has been diverted to the treatment plant. This will decrease the total flow draining to the pump plant. There have been no significant changes with regard to the stormwater contributing area of the pump plant. Estimated percent impervious was determined from visual inspection of 1996 aerial photography. [3]

5. New Central Avenue (KCK FPS 16)

    a. Year Constructed: 1987[3]
    b. Owner/Operator: KCK
    c. Levee Station: 58+12 (KS) [1]
    d. Distance From Flood Protection Centerline: 40ft[5]
    e. Station Capacity: --
    f. Pumps:
        i. Cascade 1 stage 24MF 700RPM 19,000gpm @ 25.5ft head, drive 145hp[4]
        ii. Cascade 1 stage 24MF 700RPM 19,000gpm @ 25.5ft head, drive 145hp[4]
        iii. Cascade 1 stage 24MF 700RPM 19,000gpm @ 25.5ft head, drive 145hp[4]
        iv. Sump Pump 3” discharge 200 gpm at 19ft head[4]
    g. Contributing Flows:
        i. Storm sewer system flow [1]
    h. River Discharge: 84” RCP[1]
    i. Discharge Arrangement: Gravity drainage under levee pressurized pipe place up and over levee discharging into gatewell on riverside of the levee[9]
    j. Drawing Availability:
        i. Sep 1988 record drawings[5]
    k. General
        i. When the Kansas River stage is low enough to permit gravity flow, the Central Avenue Pump Plant is on standby. Water entering the stormwater collection system flows through the station inlet to the river by gravity. However, once the critical stage is reached, the gatewell sluice gate is closed and stormwater is pumped to the riverside of the gatewell sluice gate. This creates a high enough head in the wetwell to permit gravity flow to the river (going through the same outlet pipe as in the former gravity flow case). [3]
        ii. The City of Kansas City, Kansas calls this Flood Pump Station #16. [3]
        iii. The plant has a sump pump rated at 200 gpm for 19 feet of head. [3]
        iv. The pump plant was designed for certain flows at total dynamic head conditions. It is thought that the drainage area conditions have not changed in the past 15 years. [3]

6. Stockyards #3

    a. Year Constructed: ~1950
    b. Owner/Operator: KVDD
    c. Levee Station: 74+21 (KS) [1]
    d. Distance From Flood Protection Centerline: 85ft[7]
    e. Station Capacity: 10,300gpm[1]
    f. Pumps:
        i. 14” 5710 WX Fairbanks Morse pump, drive 50hp 860RPM[7]
        ii. 14” 5710 WX Fairbanks Morse pump, drive 50hp 860RPM[7]
        iii. 14” Sumbmersible Fairbanks Morse Pump, 47.5hp 705RPM[10]

iv. Sump Pump

g. Contributing Flows:
   i. Seepage flow for the area between station 65+00 and station 82+00 coming through relief wells. (1)
   ii. Storm runoff drainage from the west side of the Central Industrial District (Kansas), less than 47ac. (1)
h. River Discharge: 24" CIP(1)
i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under levee(7)
j. Drawing Availability:
   i. July 1980 – General Plan & Profile, Other Details (12 sheets) (7)
k. General
   i. It was found by the COE that excessive damage occurs from the extended duration of ponding caused by high seepage rates over long periods and not by ponding of short duration caused by storm runoff. The designed pumping capacity was based upon this finding. (3)
   ii. KAW Valley calls this pump plant the Stockyards Pump Plant (used to be called Stockyards #3 Pump Plant). (3)
   iii. The Stockyards Pump Plant used to take relief well seepage flow, along with storm drainage, from the stockyards area. The construction of the Gateway 2000 facility cut off some of that storm drainage area, so the overall flow to the plant has decreased. In addition, the sanitary flow was diverted to the treatment plant. The estimated percent impervious is from visual inspection of 1996 aerial photography. (3)
   iv. The pump capacities are noted because they were previously inadequate. It is not currently known if the decreased drainage area and runoff will be enough to nullify that inadequacy. (3)
   v. New submersible pump installed in 1995.(10)

7. Gateway 2000

a. Year Constructed: 1994
b. Owner/Operator: Gateway 2000
c. Levee Station: 80+90 (KS) (1)
d. Distance From Flood Protection Centerline: 13ft
e. Station Capacity: 9,500gpm(1)
f. Pumps:
   i. Flygt submersible L3531/705 14,100@8ft of head and 9,500gpm@25ft of head. (8)
   ii. Flygt submersible L3531/705 14,100@8ft of head and 9,500gpm@25ft of head.(8)
g. Contributing Flows:
   i. Storm sewer system flow contributed by the Gateway 2000 property, approximately 80ac. (1)
h. River Discharge: 60" RCP(1)

i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under the levee.
j. Drawing Availability:
    i. Oct 1996 Plans – complete set[8]
k. General
    i. The Gateway 2000 Pump Plant was constructed in 1994 to serve the area surrounding the Gateway 2000 facilities. [3]
    ii. The pump plant is located in Kansas and discharge through the CID Kansas Unit. However, a portion of the contributing service area is located in Missouri. [3]
    iii. The pump plant was designed to have certain pumping capacities at given river stages. Therefore, the exact service area information was not obtainable. It was estimated from drainage area mapping provided by the designer of the plant. The estimated percent impervious was determined by visual inspection of 1996 aerial photographs. However, the conditions of the approximate service area have not changed since the pump plant construction. [3]
    iv. Replaced Stockyards pump station #2 (pigeon roost)[10]

8. Stockyards #1

a. Year Constructed: N/A
b. Owner/Operator: KVDD
c. Levee Station: 98+05 (KS) [1]
d. Distance From Flood Protection Centerline: 75ft[7]
e. Station Capacity: N/A
f. Pumps: N/A
g. Contributing Flows:
    i. Storm sewer system flow from the west side of the Central Industrial District (Kansas), less than 14ac. [1]
h. River Discharge: 24” CIP[1]
i. Discharge Arrangement:
j. Drawing Availability:
    i. July 1980 – General Plan & Profile[7]
k. General
    i. The capacity of the pump is actually 4.23 cfs, but the lack of power from the motor brings that down to the 1.47 cfs. [3]
    ii. There is a total storage of 0.44 acre-feet required for ponding, with a maximum duration of 110 minutes. Part of this flow would drain to Valve No. 7. The ponding should not cause damage. [3]
    iii. The outlet for the pump plant is at Kansas River Mile 1.92. [3]
    iv. KAW Valley Drainage District calls this plant the Field Pump House (formerly called the Stockyards Pump House #1). Larry Brennan said that the pumps have not needed to run for over 20 years. [3]

v. Part of the drainage area contributing flow to the pump plant was cut off by Kemper Arena. The percent impervious estimated by visual inspection of 1996 aerial photographs shows no significant change, but the total flow from the smaller drainage area must be significantly less judging from the fact that it does not run very often. (3)
vi. The pump is likely still inadequate. (3)
vii. Pump pit, gatewell and valve box are all brick and motor construction. Pump pit is approximately 2' dia., both the gatewell and valve box are approximately 2'x2' in size.(10)
viii. Gate valve is located in discharge conduit at the levee. This valve is kept closed by KVDD. (10)
ix. Pump station has not been operational recently. During the 1993 flood crews tried, without success, to operate the pump. (10)
x. KVDD thought that most overland flow has been diverted to Kemper Pump Plant. (10)

9. Kemper (Area)

a. Year Constructed: --
b. Owner/Operator: KCMO
c. Levee Station: 106+49 (KS) (1)
d. Distance From Flood Protection Centerline: 180ft(7)
e. Station Capacity: --
f. Pumps:
    i. Four 36" column type, drive 350hp(12)
    ii. Sump pump(12)
g. Contributing Flows:
    i. Storm runoff flow from the Kemper Arena and American Royal area, 51ac. (1)
h. River Discharge: 6'x6' RCB(1)
i. Discharge Arrangement: --
j. Drawing Availability:
    i. Jul 1980 – General Plan & Profile
k. General
    i. The pump plant is located in Missouri, with almost all of the drainage area in Missouri also. However, the discharge eventually goes through the Central Industrial District levee unit in Kansas. (3)
    ii. A representative of the City of Kansas City, Missouri noted that the pump plant was built in 1975 and has never had more than two pumps running. There are a total of four pumps at the plant. (3)
    iii. HNTB has been in contact with the designers in charge of runoff disposal from the new Butler Manufacturing building which will be constructed to the north of Kemper Arena. The runoff totals shown are preliminary estimates from the designers. Information is not readily available on the original design of the plant, but it is known that there are four vertical

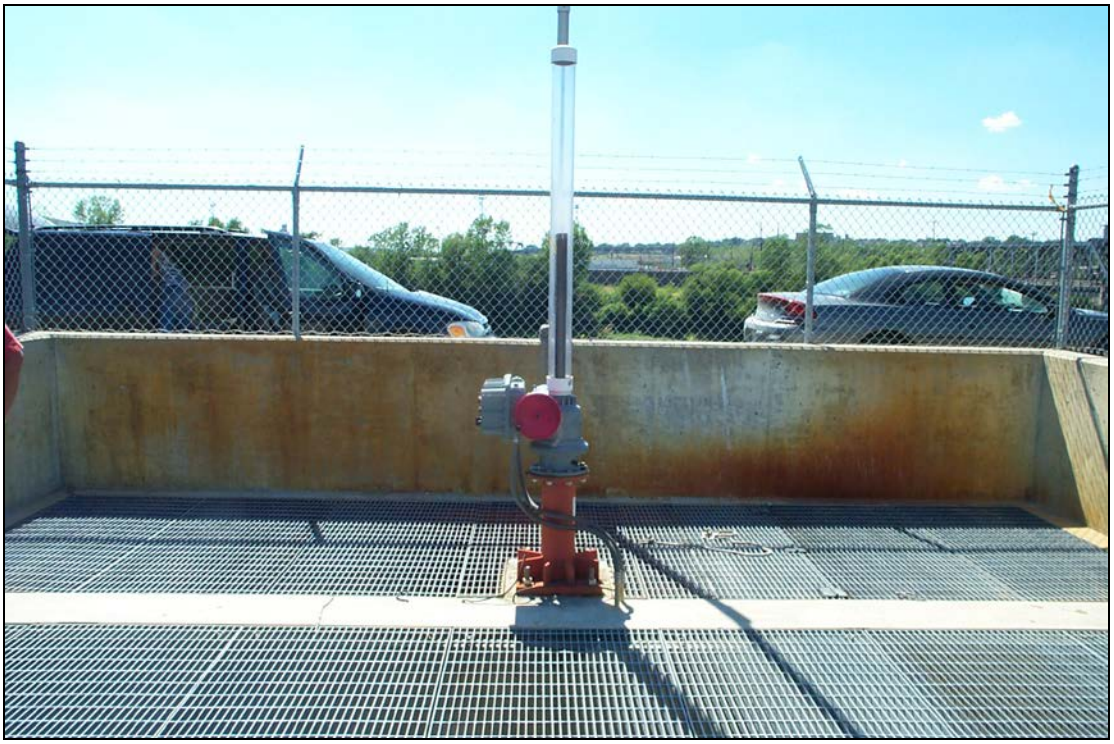
turbine pumps rated at 31,000 gpm each at 37 feet of head. There is a low river condition at which water is pumped to a point to be gravity discharged and a high river condition at the elevation noted. [(3)]

iv. Current conditions are similar to design conditions for the Kemper Arena Pump Plant, with one exception. When the Butler Manufacturing building was constructed in the early 2000's, north of Kemper Arena, additional drainage was routed to the pump plant. A new 48" line from the site ties into the 6'x6' box between Kemper Arena and American Royal which feeds into the Kemper pump plant. With the exception of the Golden Ox parking lot, most of the area from Genessee Street and I-670 drains southward to the new pipe. An existing stormwater discharge pipe at the Livestock Exchange building was relocated around the new parking garage east of the Exchange building. The pipe was routed back to its original outlet location on the river. The project caused an approximate increase of flow to Kemper Pump Plant of 12.65 acres with a percent impervious of 87. The estimated percent impervious for present conditions was determined by visual inspection of 1996 aerial photographs. [(3)]

10. Pump Station Information 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. Verbal Conversations with Ernie Quinto
7. O&M Drawings located at E:\sec\LocalProtectionProjects\KansasCity
8. Gateway pump station asbuilts supplied by Gateway (Chris Nijsse)
9. Site visit with KCK 25-Jun-05
10. Site visit with KVDD 04-Aug-05 (Ernie Quinto & Larry Brennen)
11. Site visit with KCMO 15-Sep-05 (Mort Dasjordi & Steve Zulke)
12. Periodic General Inspection Report No. 1, March 1981

Gateway 2000 Pump Station



Mistletoe Pump Station

Central Avenue Pump Station

Ohio Avenue Pump Station

Stockyards #1/Field Pump Station

Stockyards #3 Pump Station

Broadway Pump Station

Kemper Pump Station

Santa Fe Pump Station

EXHIBIT 2 – GEOTECHNICAL DATA

Hydraulic Gradeline Calculations for CID Pump Stations

Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Missouri - Hydraulic Grade Line for the Broadway Pump Station Sta 24+76.9

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.

PDT
Name : Kuzniakowski EC-GD
Date : 7/26/2007

Peer Review
Checked by :
Date :

NOTES:
1. Existing condition: Pump Station 30 ft from Wall Centerline
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise
4. Blanket details are not well known, very heterogeneous, mostly sand

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	759.00
500 year + 0':	
500 year + 3':	
500 year + 5':	
Ground Surface Land Side:	752
Ground Surface River Side:	750
Bottom of Blanket:	745 (Assumed, based upon limited available information, blanket consists of mostly debris, no aquifer identified)
Top of Bedrock:	720 (Assumed, based upon limited available information)
Ground Surface at Pump Station:	750

Note: Ignore all but the existing condition calculations

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

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Input Parameters

Station	Design Condition	Permeability Ratio: River Side (K_f/K_{br})	Permeability Ratio: Land Side (K_f/K_{bl})	Blanket Thickness (ft): River Side z_{br}	Blanket Thickness (ft): Protection z_{bo}	Blanket Thickness (ft): Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Protection L_2	Seepage Length (ft): Land Side L_3	Driving Head (ft) H
24+76.9	Existing Condition	100	100	5.0	6.1	7.3	25.0	70	15	1000	6.8
24+76.9	n500 + 0	100	100	5.0	6.1	7.3	25.0	70	15	1000	-752.3
24+76.9	n500 +3	100	100	5.0	6.1	7.3	25.0	70	15	1000	-752.3
24+76.9	n500 + 5	100	100	5.0	6.1	7.3	25.0	70	15	1000	-752.3

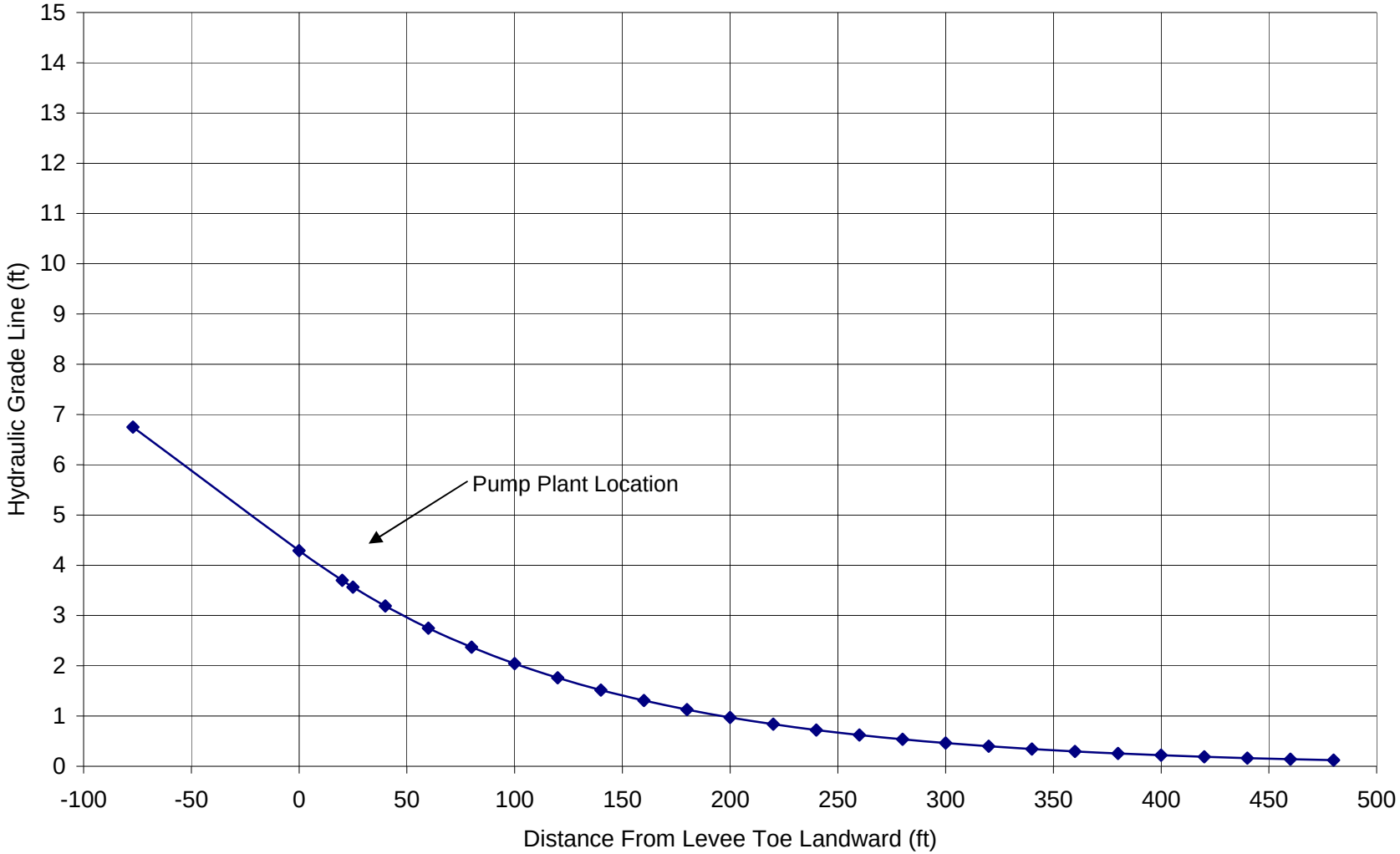
Analysis

Factors: c_r	Factors: c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
0.008944	0.007428	62	135	4.3	0.59	0.84	1.42	
0.008944	0.007428	62	135	-478.3	-65.98	0.84	-0.01	
0.008944	0.007428	62	135	-478.3	-65.98	0.84	-0.01	
0.008944	0.007428	62	135	-478.3	-65.98	0.84	-0.01	

Hydraulic Grade Line for the Broadway Pump Plant
Existing Conditions

Elevation Top of Protection (MSL)	759	Existing Conditions	
Elevation at Landside Toe (MSL)	752.25		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	6.75		
c_l	0.007428		
x_1 (ft)	62		
L_2 (ft)	15		
x_3 (ft)	135		
h_o	4.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-77	6.75	759.00
	0	4.29	756.54
	20	3.70	755.95
Pump Plant Location	25	3.56	755.81
	40	3.19	755.44
	60	2.75	755.00
	80	2.37	754.62
	100	2.04	754.29
	120	1.76	754.01
	140	1.52	753.77
	160	1.31	753.56
	180	1.13	753.38
	200	0.97	753.22
	220	0.84	753.09
	240	0.72	752.97
	260	0.62	752.87
	280	0.54	752.79
	300	0.46	752.71
	320	0.40	752.65
	340	0.34	752.59
	360	0.30	752.55
	380	0.26	752.51
	400	0.22	752.47
	420	0.19	752.44
	440	0.16	752.41
	460	0.14	752.39
	480	0.12	752.37

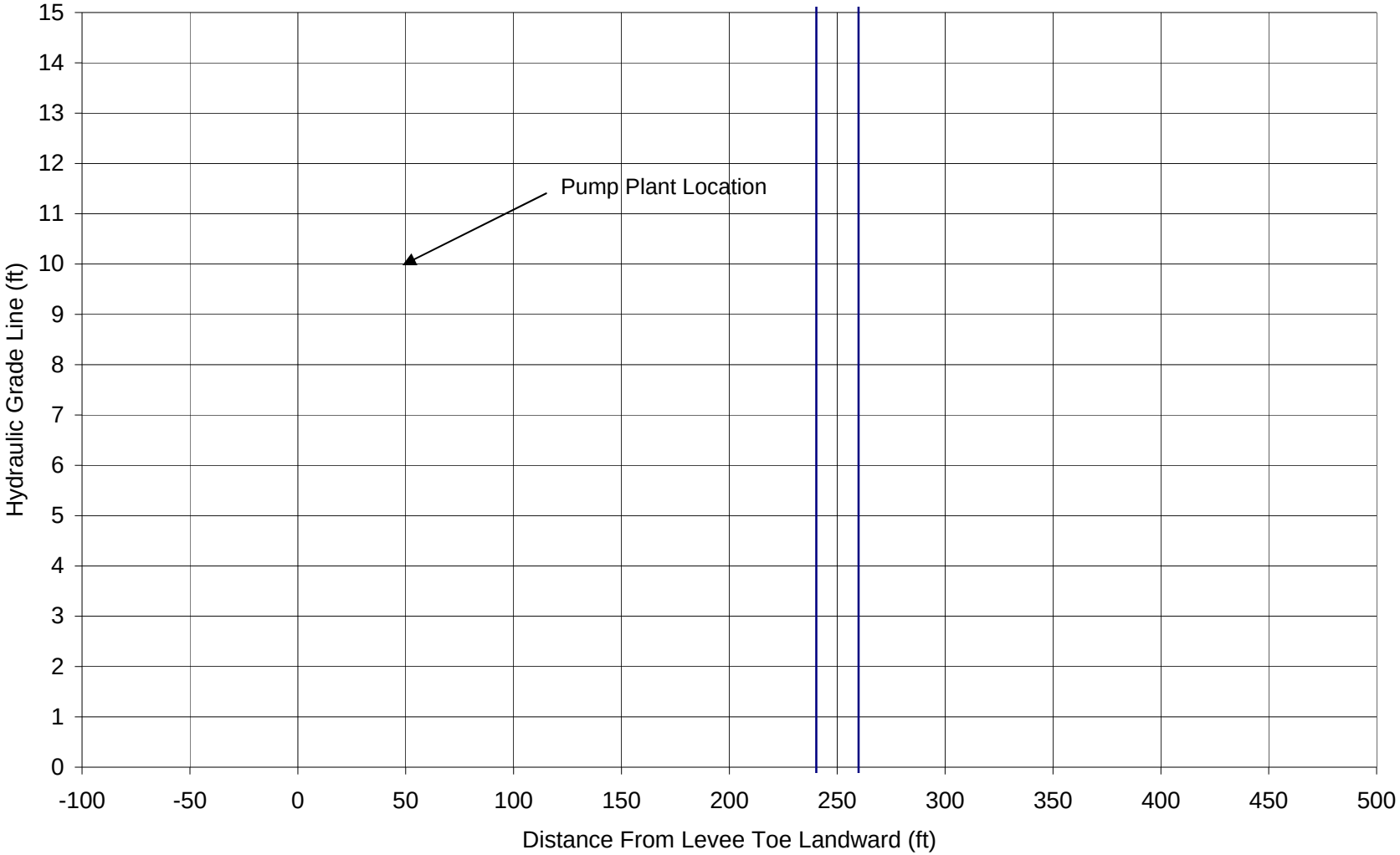
Existing Conditions Hydraulic Grade Line for the
Broadway Pump Plant Sta 24+76.9
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Broadway Pump Plant
n500+0 Loading Conditions

Elevation Top of Protection (MSL)	759	Existing Conditions	
Elevation at Landside Toe (MSL)	752.25		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-752.25		
c_l	0.007428		
x_1 (ft)	62		
L_2 (ft)	15		
x_3 (ft)	135		
h_o	-478.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-77	-752.25	0.00
	0	-478.34	273.91
	20	-412.31	339.94
Pump Plant Location	25	-397.28	354.97
	40	-355.39	396.86
	60	-306.33	445.92
	80	-264.04	488.21
	100	-227.59	524.66
	120	-196.17	556.08
	140	-169.09	583.16
	160	-145.75	606.50
	180	-125.63	626.62
	200	-108.28	643.97
	220	-93.34	658.91
	240	-80.45	671.80
	260	-69.35	682.90
	280	-59.77	692.48
	300	-51.52	700.73
	320	-44.41	707.84
	340	-38.28	713.97
	360	-32.99	719.26
	380	-28.44	723.81
	400	-24.51	727.74
	420	-21.13	731.12
	440	-18.21	734.04
	460	-15.70	736.55
	480	-13.53	738.72

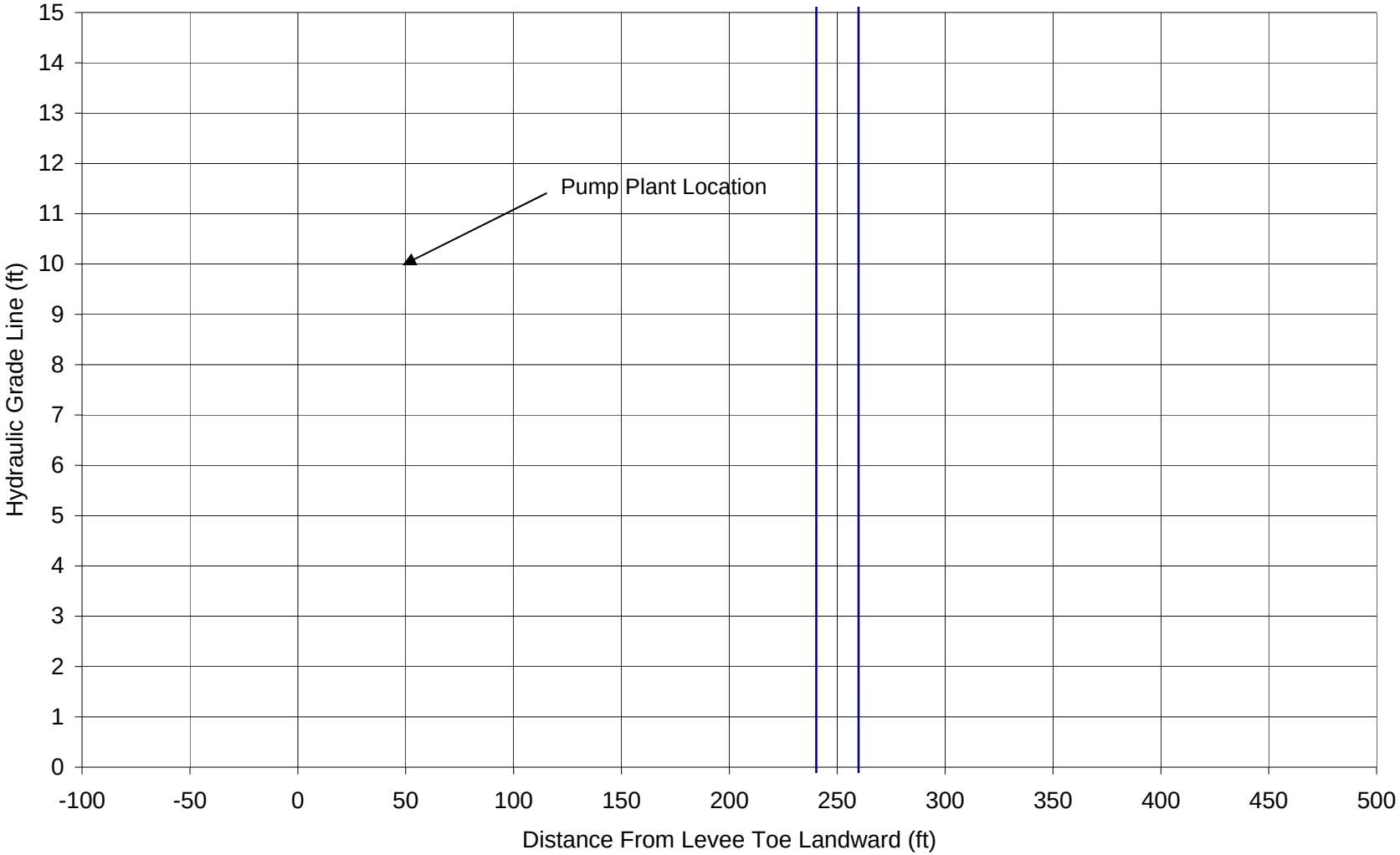
n500+0 Loading Conditions Hydraulic Grade Line for the
Broadway Pump Plant Sta 24+76.9
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Broadway Pump Plant
n500+3 Loading Conditions

Elevation Top of Protection (MSL)	759	Existing Conditions	
Elevation at Landside Toe (MSL)	752.25		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-752.25		
c_l	0.007428		
x_1 (ft)	62		
L_2 (ft)	15		
x_3 (ft)	135		
h_o	-478.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-77	-752.25	0.00
	0	-478.34	273.91
	20	-412.31	339.94
Pump Plant Location	25	-397.28	354.97
	40	-355.39	396.86
	60	-306.33	445.92
	80	-264.04	488.21
	100	-227.59	524.66
	120	-196.17	556.08
	140	-169.09	583.16
	160	-145.75	606.50
	180	-125.63	626.62
	200	-108.28	643.97
	220	-93.34	658.91
	240	-80.45	671.80
	260	-69.35	682.90
	280	-59.77	692.48
	300	-51.52	700.73
	320	-44.41	707.84
	340	-38.28	713.97
	360	-32.99	719.26
	380	-28.44	723.81
	400	-24.51	727.74
	420	-21.13	731.12
	440	-18.21	734.04
	460	-15.70	736.55
	480	-13.53	738.72

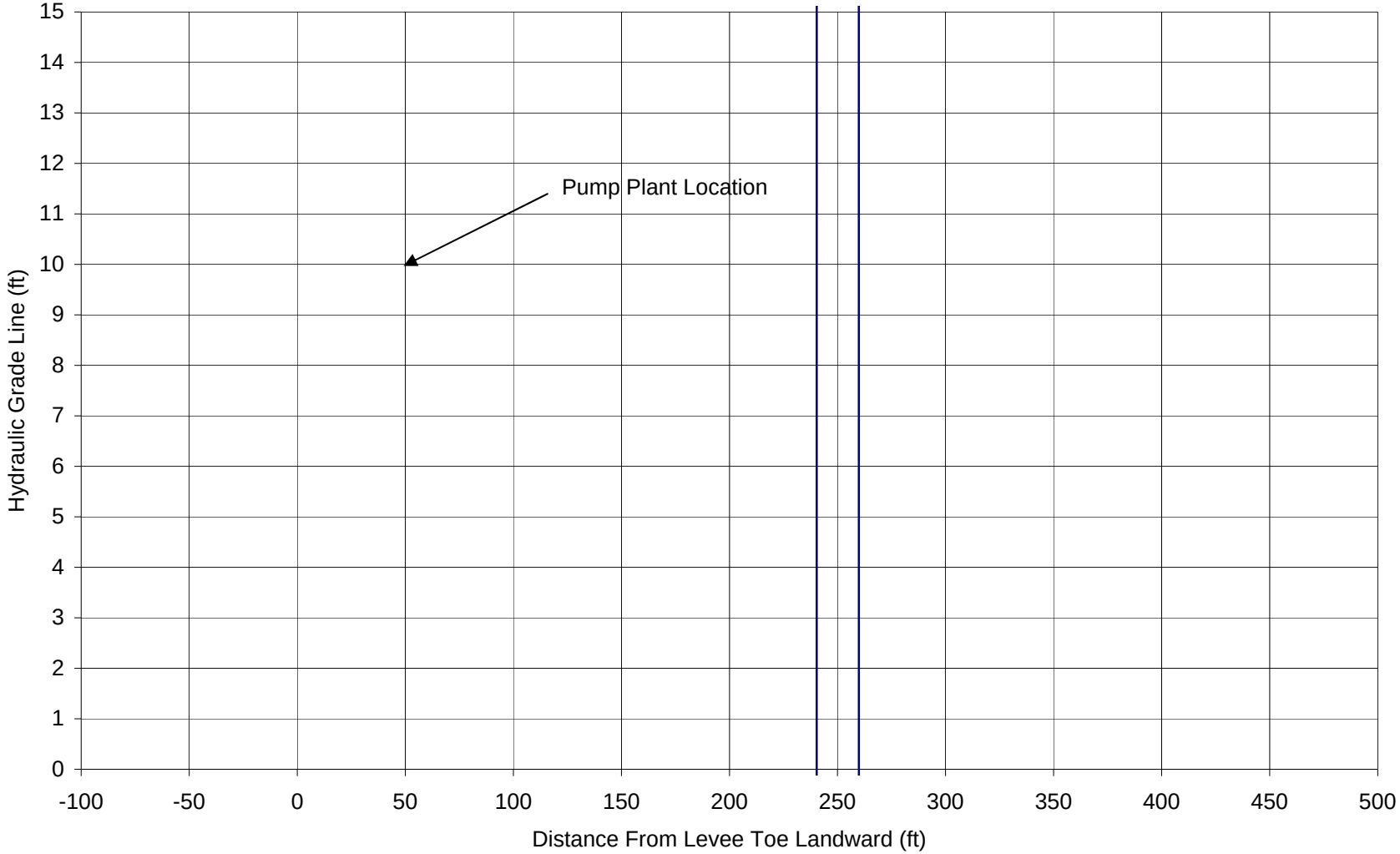
n500+3 Loading Conditions Hydraulic Grade Line for the
Broadway Pump Plant Sta 24+76.9
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Broadway Pump Plant
n500+5 Loading Conditions

Elevation Top of Protection (MSL)	759	Existing Conditions	
Elevation at Landside Toe (MSL)	752.25		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-752.25		
c_l	0.007428		
x_1 (ft)	62		
L_2 (ft)	15		
x_3 (ft)	135		
h_o	-478.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-77	-752.25	0.00
	-40	-620.46	131.79
	0	-478.34	273.91
	20	-412.31	339.94
Pump Plant Location	25	-397.28	354.97
	40	-355.39	396.86
	60	-306.33	445.92
	80	-264.04	488.21
	100	-227.59	524.66
	120	-196.17	556.08
	140	-169.09	583.16
	160	-145.75	606.50
	180	-125.63	626.62
	200	-108.28	643.97
	220	-93.34	658.91
	240	-80.45	671.80
	260	-69.35	682.90
	280	-59.77	692.48
	300	-51.52	700.73
	320	-44.41	707.84
	340	-38.28	713.97
	360	-32.99	719.26
	380	-28.44	723.81
	400	-24.51	727.74
	420	-21.13	731.12
	440	-18.21	734.04
	460	-15.70	736.55
	480	-13.53	738.72

n500+5 Loading Conditions Hydraulic Grade Line for the
Broadway Pump Plant Sta 24+76.9
Central Industrial District Kansas Levee Unit



Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the Mistletoe Pump Station Sta 37+07

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.

PDT
Name : Kuzniakowski EC-GD
Date : 6/19/2007

Peer Review
Checked by : EC-GD
Date :

NOTES:
1. Existing condition: Pump Station integral with floodwall
2. Floodwall raise proposed
3. L_2 not revised for minor change due to raise
4. Approx. 5 feet of trash fill exists landside, so reduce blanket thickness by 5 feet.

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	760.65
500 year + 0':	759.00
500 year + 3':	762.00
500 year + 5':	764.00
Ground Surface Land Side:	745
Ground Surface River Side:	736
Bottom of Blanket:	722
Top of Bedrock:	670
Ground Surface at Pump Station:	755

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

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Input Parameters

Station	Design Condition	Permeability Ratio: River Side (K_f/K_{br})	Permeability Ratio: Land Side (K_f/K_{bl})	Blanket Thickness (ft): River Side z_{br}	Blanket Thickness (ft): Protection z_{bo}	Blanket Thickness (ft): Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Protection L_2	Seepage Length (ft): Land Side L_3	Driving Head (ft) H
37+07	Existing Condition	800	800	14.0	18.5	23.0	52.0	0	100	1000	15.7
37+07	n500 + 0	800	800	14.0	18.5	23.0	52.0	0	100	1000	14.0
37+07	n500 +3	800	800	14.0	18.5	23.0	52.0	0	100	1000	17.0
37+07	n500 + 5	800	800	14.0	18.5	23.0	52.0	0	100	1000	19.0

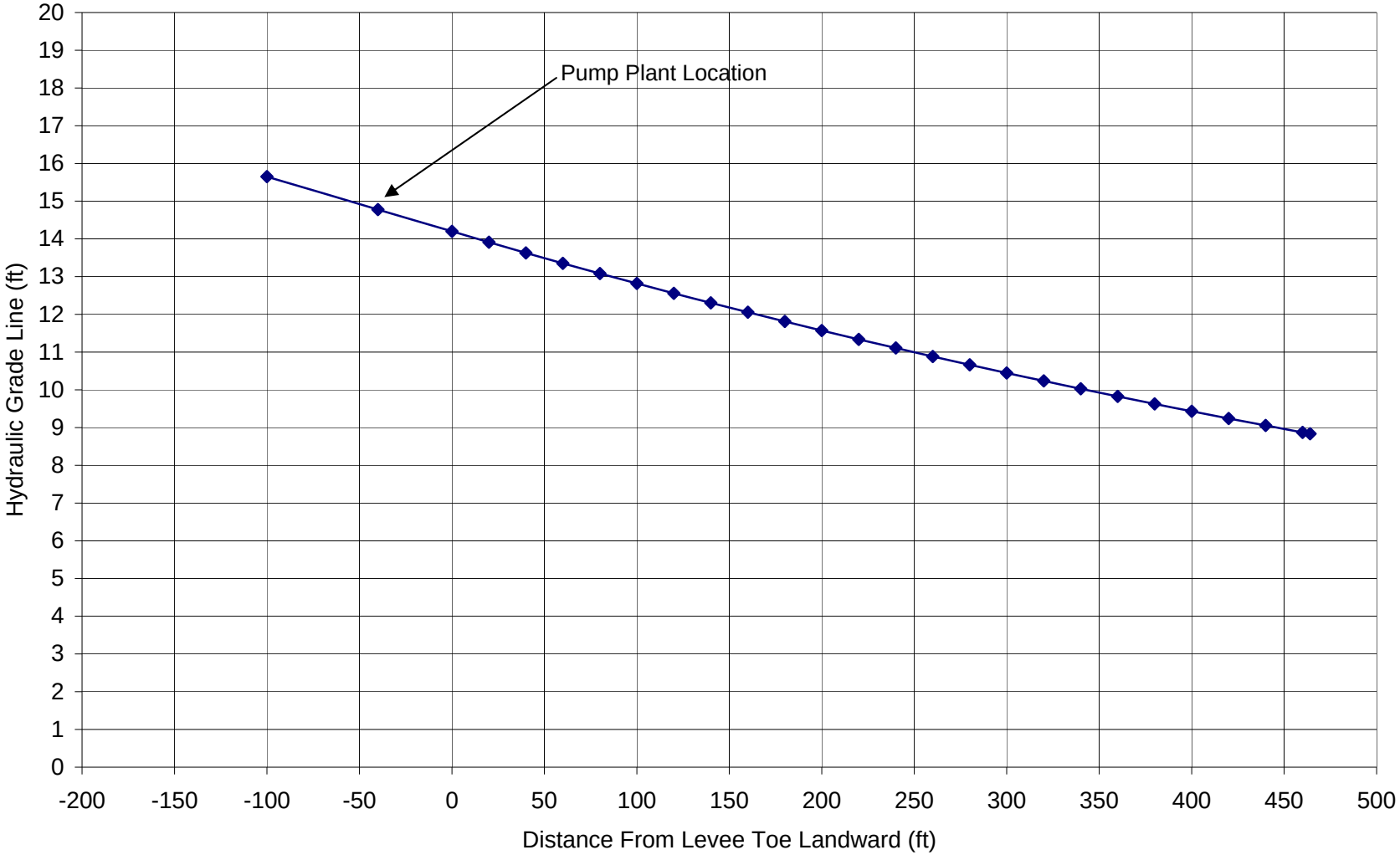
Analysis

Station	Design Condition	Factors: c_r	Factors: c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
37+07	Existing Condition	0.001310	0.001022	0	978	14.2	0.62	0.84	1.37	
37+07	n500 + 0	0.001310	0.001022	0	978	12.7	0.55	0.84	1.53	
37+07	n500 +3	0.001310	0.001022	0	978	15.4	0.67	0.84	1.26	
37+07	n500 + 5	0.001310	0.001022	0	978	17.2	0.75	0.84	1.12	

Hydraulic Grade Line for the Mistletoe Pump Plant
Existing Conditions

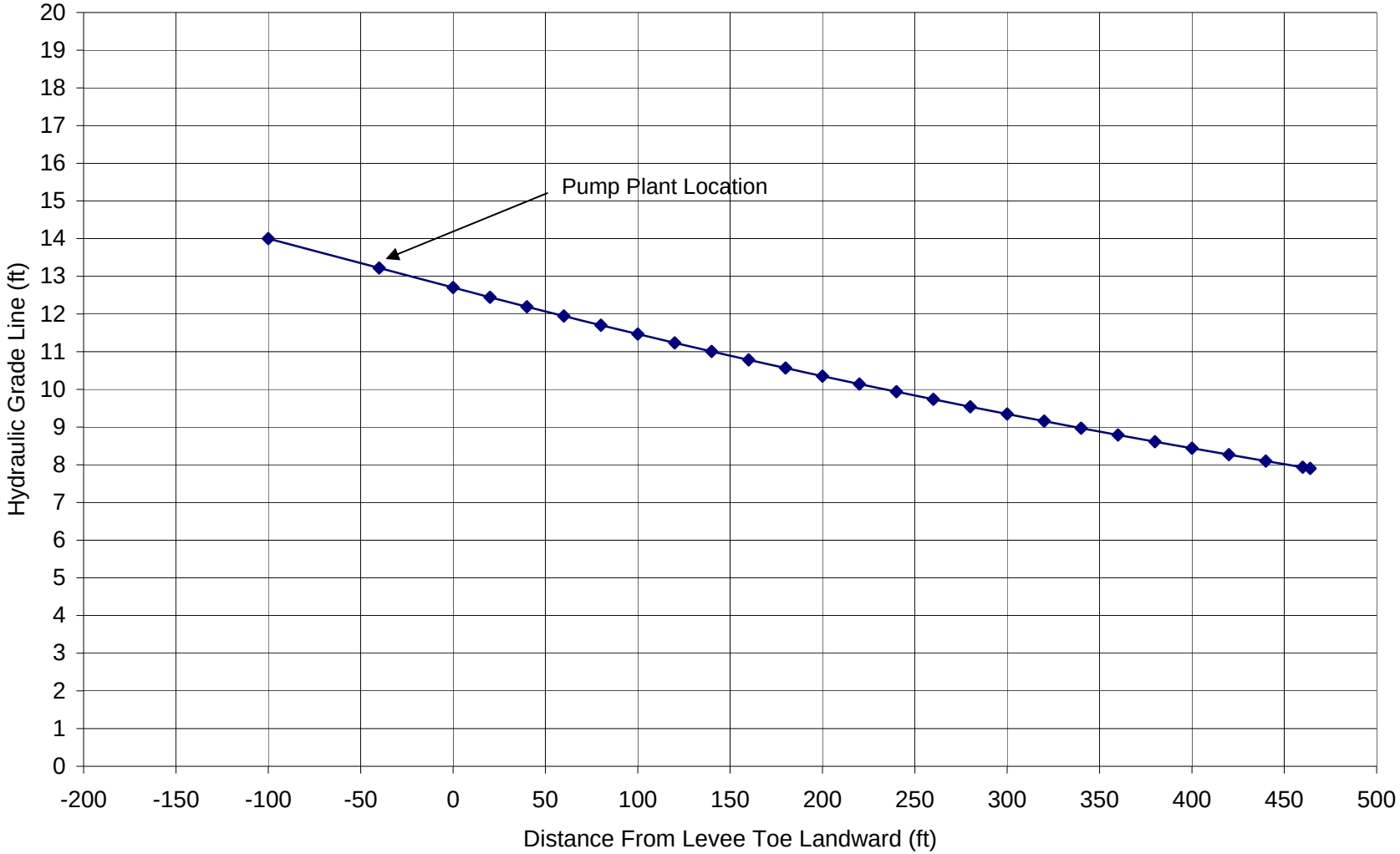
Elevation Top of Protection (MSL)	760.65	Existing Conditions	
Elevation at Landside Toe (MSL)	745		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.65		
c_l	0.001022		
x_1 (ft)	0		
L_2 (ft)	100		
x_3 (ft)	978		
h_o	14.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-100	15.65	760.65
Pump Plant Location	-40	14.78	759.78
	0	14.20	759.20
	20	13.91	758.91
	40	13.63	758.63
	60	13.35	758.35
	80	13.08	758.08
	100	12.82	757.82
	120	12.56	757.56
	140	12.31	757.31
	160	12.06	757.06
	180	11.81	756.81
	200	11.57	756.57
	220	11.34	756.34
	240	11.11	756.11
	260	10.88	755.88
	280	10.66	755.66
	300	10.45	755.45
	320	10.24	755.24
	340	10.03	755.03
	360	9.83	754.83
	380	9.63	754.63
	400	9.43	754.43
	420	9.24	754.24
	440	9.05	754.05
	460	8.87	753.87
	464	8.84	753.84

Existing Conditions Hydraulic Grade Line for the
Mistletoe Pump Plant Sta 37+07
Central Industrial District Kansas Levee Unit



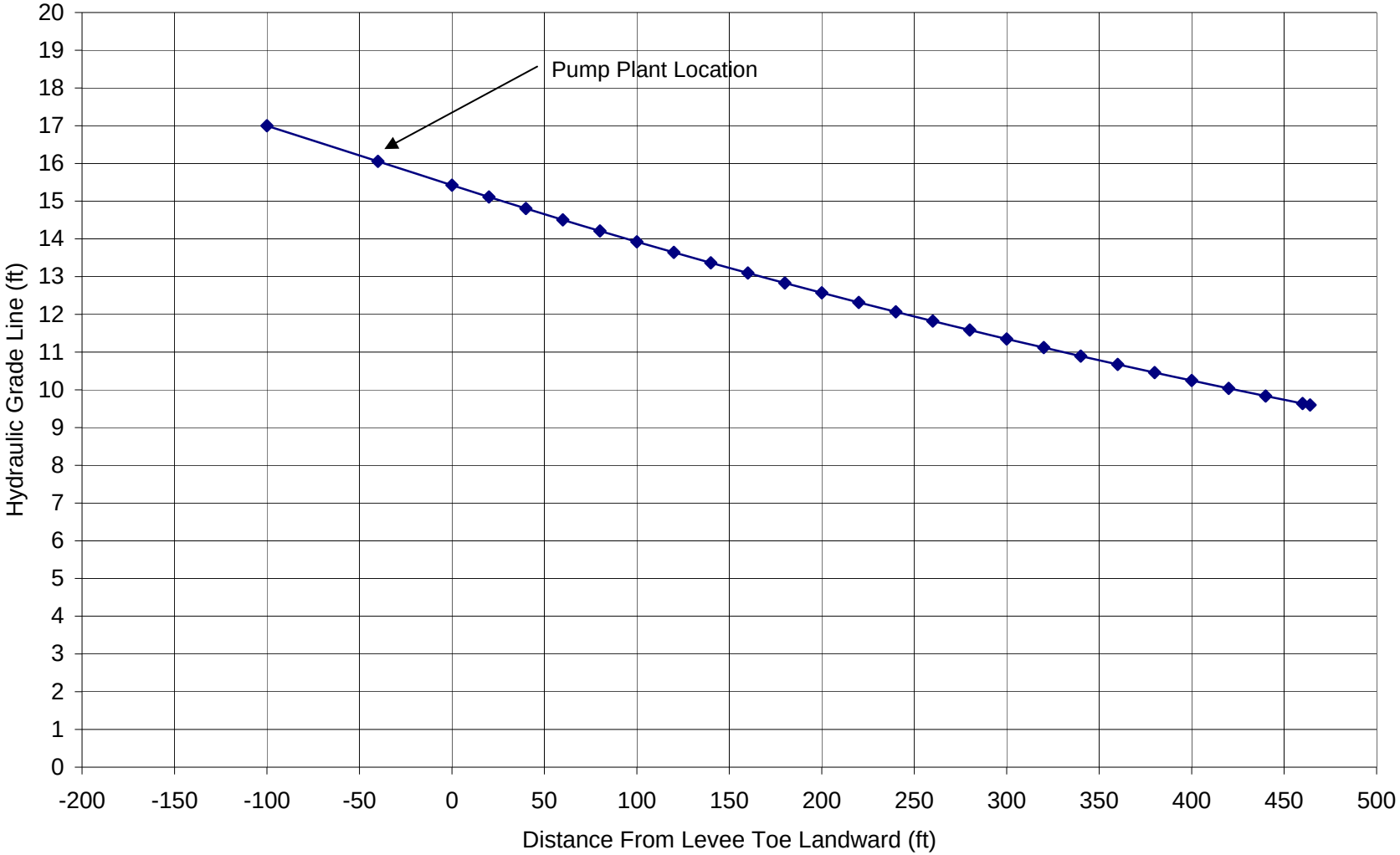
Hydraulic Grade Line for the Mistletoe Pump Plant Loading Conditions			n500+0
Elevation Top of Protection (MSL)	759.00	Existing Conditions	
Elevation at Landside Toe (MSL)	745		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	14		
c_l	0.001022		
x_1 (ft)	0		
L_2 (ft)	100		
x_3 (ft)	978		
h_o	12.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-100	14.00	759.00
Pump Plant Location	-40	13.22	758.22
	0	12.70	757.70
	20	12.44	757.44
	40	12.19	757.19
	60	11.95	756.95
	80	11.70	756.70
	100	11.47	756.47
	120	11.24	756.24
	140	11.01	756.01
	160	10.78	755.78
	180	10.57	755.57
	200	10.35	755.35
	220	10.14	755.14
	240	9.94	754.94
	260	9.74	754.74
	280	9.54	754.54
	300	9.35	754.35
	320	9.16	754.16
	340	8.97	753.97
	360	8.79	753.79
	380	8.61	753.61
	400	8.44	753.44
	420	8.27	753.27
	440	8.10	753.10
	460	7.94	752.94
	464	7.90	752.90

n500+0 Loading Conditions Hydraulic Grade Line for the
Mistletoe Pump Plant Sta 37+07
Central Industrial District Kansas Levee Unit



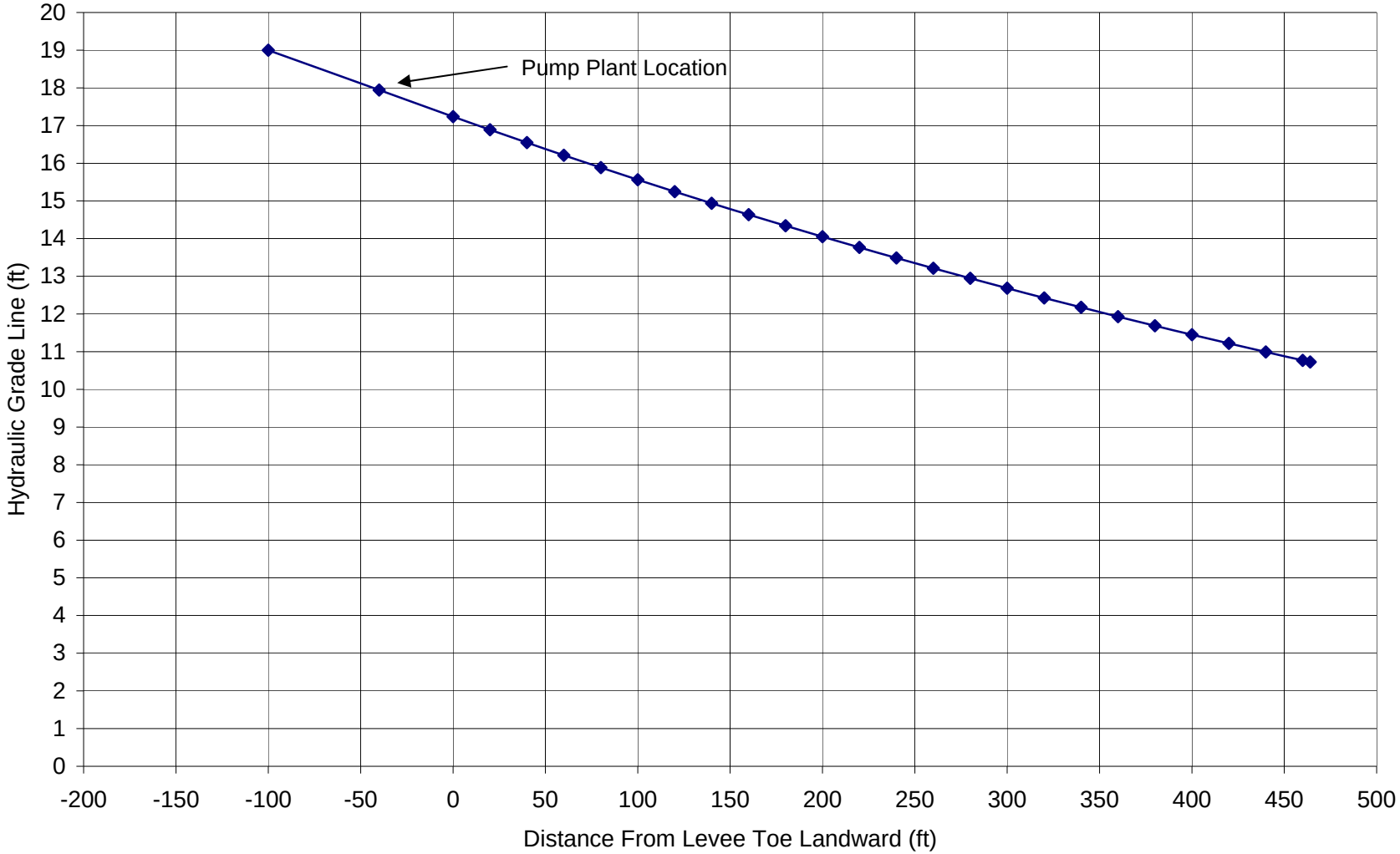
Hydraulic Grade Line for the Mistletoe Pump Plant Loading Conditions			n500+3
Elevation Top of Protection (MSL)	762.00	Existing Conditions	
Elevation at Landside Toe (MSL)	745		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	17		
c_l	0.001022		
x_1 (ft)	0		
L_2 (ft)	100		
x_3 (ft)	978		
h_o	15.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-100	17.00	762.00
Pump Plant Location	-40	16.05	761.05
	0	15.42	760.42
	20	15.11	760.11
	40	14.81	759.81
	60	14.51	759.51
	80	14.21	759.21
	100	13.92	758.92
	120	13.64	758.64
	140	13.37	758.37
	160	13.10	758.10
	180	12.83	757.83
	200	12.57	757.57
	220	12.32	757.32
	240	12.07	757.07
	260	11.82	756.82
	280	11.58	756.58
	300	11.35	756.35
	320	11.12	756.12
	340	10.89	755.89
	360	10.67	755.67
	380	10.46	755.46
	400	10.25	755.25
	420	10.04	755.04
	440	9.84	754.84
	460	9.64	754.64
	464	9.60	754.60

n500+3 Loading Conditions Hydraulic Grade Line for the
Mistletoe Pump Plant Sta 37+07
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Mistletoe Pump Plant Loading Conditions			n500+5
Elevation Top of Protection (MSL)	764.00	Existing Conditions	
Elevation at Landside Toe (MSL)	745		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	19		
c_l	0.001022		
x_1 (ft)	0		
L_2 (ft)	100		
x_3 (ft)	978		
h_o	17.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-100	19.00	764.00
Pump Plant Location	-40	17.94	762.94
	0	17.24	762.24
	20	16.89	761.89
	40	16.55	761.55
	60	16.21	761.21
	80	15.88	760.88
	100	15.56	760.56
	120	15.25	760.25
	140	14.94	759.94
	160	14.64	759.64
	180	14.34	759.34
	200	14.05	759.05
	220	13.77	758.77
	240	13.49	758.49
	260	13.21	758.21
	280	12.95	757.95
	300	12.68	757.68
	320	12.43	757.43
	340	12.18	757.18
	360	11.93	756.93
	380	11.69	756.69
	400	11.45	756.45
	420	11.22	756.22
	440	10.99	755.99
	460	10.77	755.77
	464	10.73	755.73

n500+5 Loading Conditions Hydraulic Grade Line for the
Mistletoe Pump Plant Sta 37+07
Central Industrial District Kansas Levee Unit



Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Missouri - Hydraulic Grade Line for the Santa Fe Pump Station Sta 52+86.7

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.

PDT
Name : Kuzniakowski EC-GD
Date : 7/26/2007

Peer Review
Checked by :
Date :

NOTES:
1. Existing condition: Pump Station 55 ft from Wall Centerline
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise
4. Blanket details are not well known, very heterogeneous, mostly sand

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	759.70
500 year + 0':	
500 year + 3':	
500 year + 5':	
Ground Surface Land Side:	750
Ground Surface River Side:	748
Bottom of Blanket:	745 (Based upon limited available information; significant sand fill)
Top of Bedrock:	670 (Assumed, based upon limited available information)
Ground Surface at Pump Station:	748

Note: Ignore all but the existing condition calculations

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

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Input Parameters

Station	Design Condition	Permeability Ratio: River Side (K_f/K_{br})	Permeability Ratio: Land Side (K_f/K_{bl})	Blanket Thickness (ft): River Side z_{br}	Blanket Thickness (ft): Protection z_{bo}	Blanket Thickness (ft): Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Protection L_2	Seepage Length (ft): Land Side L_3	Driving Head (ft) H
52+86.7	Existing Condition	100	100	3.0	4.0	5.0	75.0	70	13	1000	9.7
52+86.7	n500 + 0	100	100	3.0	4.0	5.0	75.0	70	13	1000	-750.0
52+86.7	n500 +3	100	100	3.0	4.0	5.0	75.0	70	13	1000	-750.0
52+86.7	n500 + 5	100	100	3.0	4.0	5.0	75.0	70	13	1000	-750.0

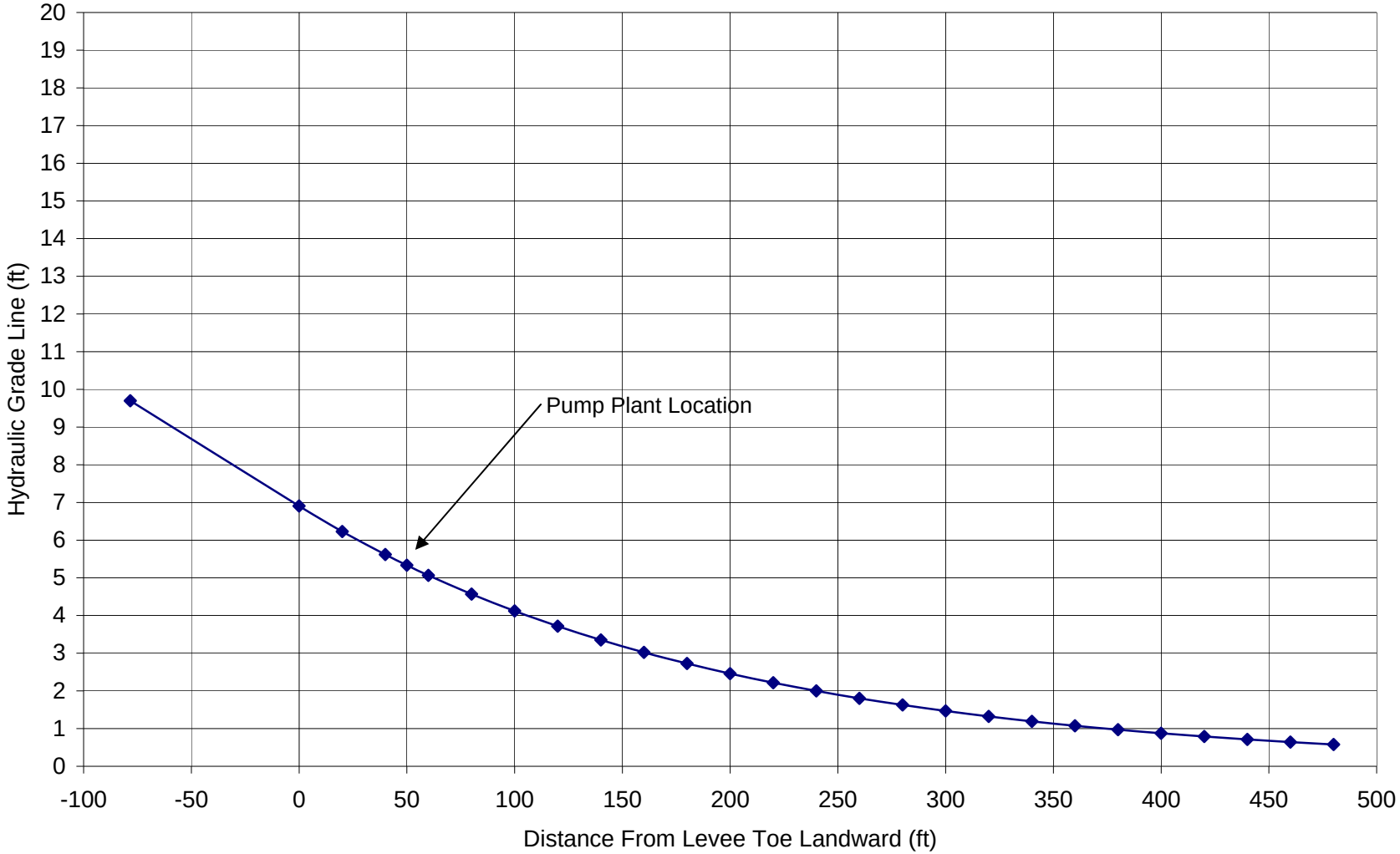
Analysis

Factors: c_r	Factors: c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
0.006667	0.005164	65	194	6.9	1.38	0.84	0.61	
0.006667	0.005164	65	194	-534.0	-106.80	0.84	-0.01	
0.006667	0.005164	65	194	-534.0	-106.80	0.84	-0.01	
0.006667	0.005164	65	194	-534.0	-106.80	0.84	-0.01	

Hydraulic Grade Line for the Santa Fe Pump Plant
Existing Conditions

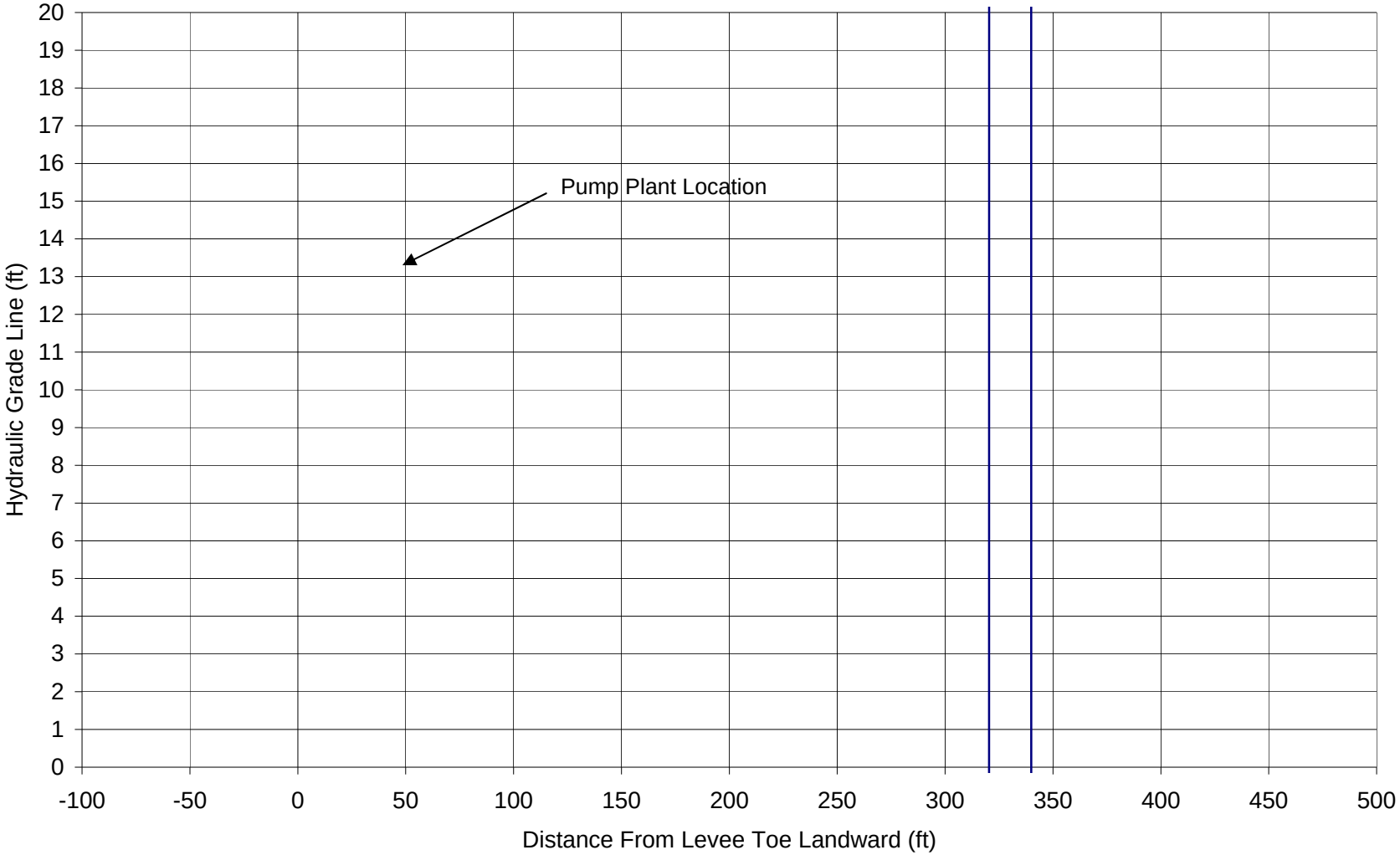
Elevation Top of Protection (MSL)	759.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	9.7		
c_l	0.005164		
x_1 (ft)	65		
L_2 (ft)	13		
x_3 (ft)	194		
h_o	6.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-78	9.70	759.70
	0	6.91	756.91
	20	6.23	756.23
	40	5.62	755.62
Pump Plant Location	50	5.33	755.33
	60	5.07	755.07
	80	4.57	754.57
	100	4.12	754.12
	120	3.72	753.72
	140	3.35	753.35
	160	3.02	753.02
	180	2.73	752.73
	200	2.46	752.46
	220	2.22	752.22
	240	2.00	752.00
	260	1.80	751.80
	280	1.63	751.63
	300	1.47	751.47
	320	1.32	751.32
	340	1.19	751.19
	360	1.08	751.08
	380	0.97	750.97
	400	0.88	750.88
	420	0.79	750.79
	440	0.71	750.71
	460	0.64	750.64
	480	0.58	750.58

Existing Conditions Hydraulic Grade Line for the
Santa Fe Pump Plant Sta 52+86.7
Central Industrial District Kansas Levee Unit



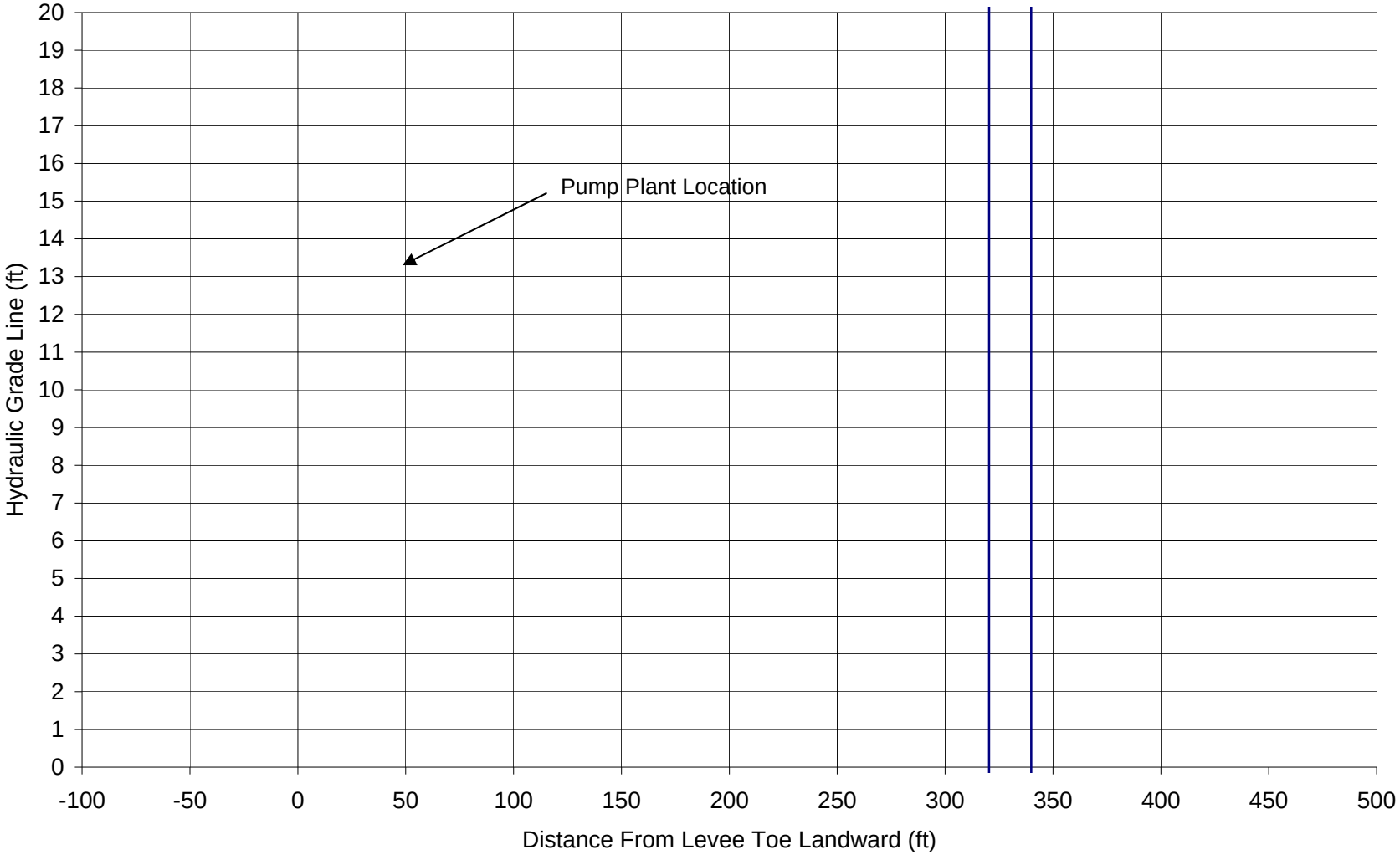
Hydraulic Grade Line for the Santa Fe Pump Plant Loading Conditions			n500+0
Elevation Top of Protection (MSL)	759.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-750		
c_l	0.005164		
x_1 (ft)	65		
L_2 (ft)	13		
x_3 (ft)	194		
h_o	-534.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-78	-750.00	0.00
	0	-534.01	215.99
	20	-481.61	268.39
	40	-434.35	315.65
Pump Plant Location	50	-412.49	337.51
	60	-391.73	358.27
	80	-353.29	396.71
	100	-318.63	431.37
	120	-287.36	462.64
	140	-259.16	490.84
	160	-233.73	516.27
	180	-210.80	539.20
	200	-190.11	559.89
	220	-171.46	578.54
	240	-154.63	595.37
	260	-139.46	610.54
	280	-125.78	624.22
	300	-113.43	636.57
	320	-102.30	647.70
	340	-92.26	657.74
	360	-83.21	666.79
	380	-75.05	674.95
	400	-67.68	682.32
	420	-61.04	688.96
	440	-55.05	694.95
	460	-49.65	700.35
	480	-44.78	705.22

n500+0 Loading Conditions Hydraulic Grade Line for the
Santa Fe Pump Plant Sta 52+86.7
Central Industrial District Kansas Levee Unit



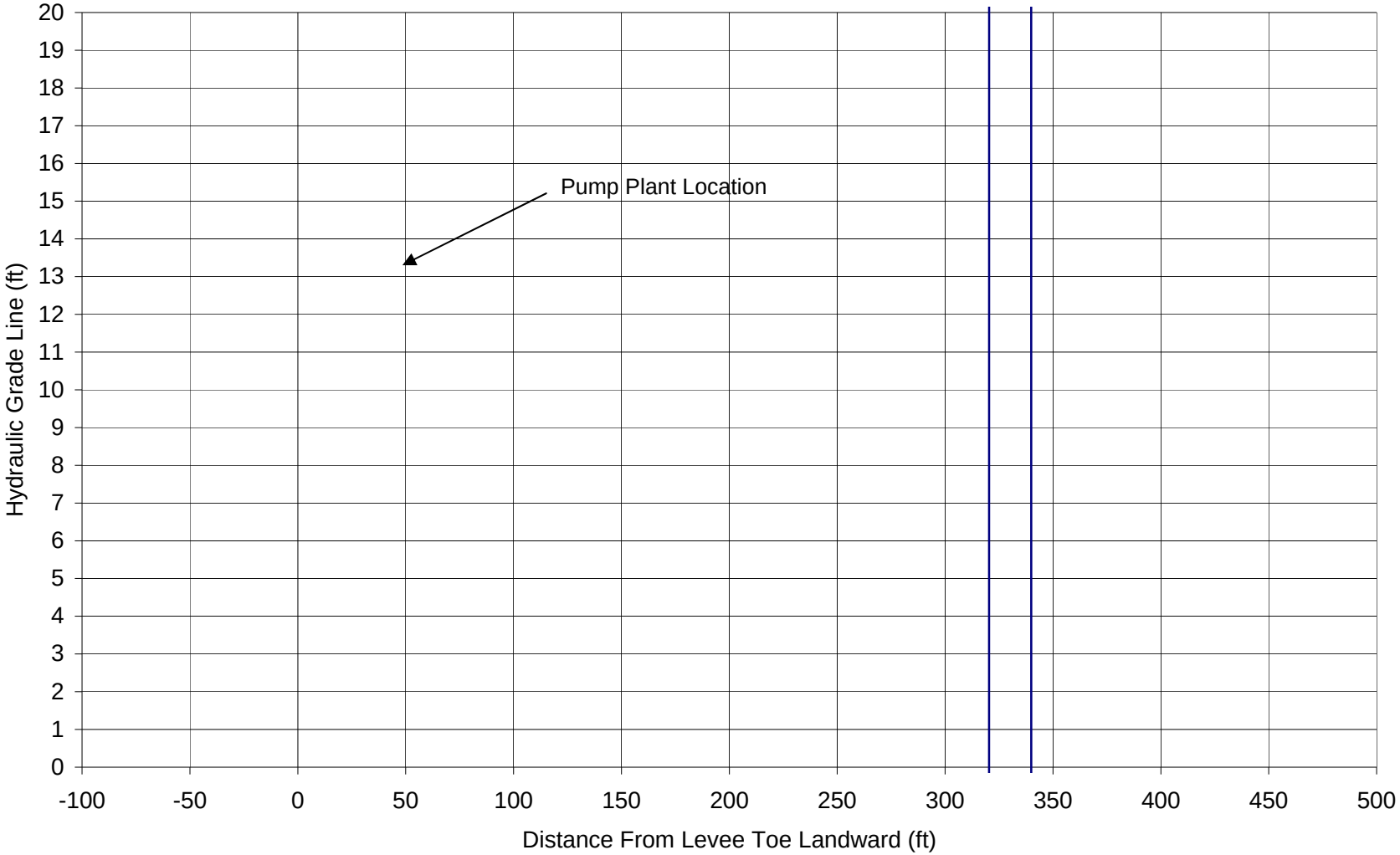
Hydraulic Grade Line for the Santa Fe Pump Plant Loading Conditions			n500+3
Elevation Top of Protection (MSL)	759.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-750		
c_l	0.005164		
x_1 (ft)	65		
L_2 (ft)	13		
x_3 (ft)	194		
h_o	-534.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-78	-750.00	0.00
	0	-534.01	215.99
	20	-481.61	268.39
	40	-434.35	315.65
Pump Plant Location	50	-412.49	337.51
	60	-391.73	358.27
	80	-353.29	396.71
	100	-318.63	431.37
	120	-287.36	462.64
	140	-259.16	490.84
	160	-233.73	516.27
	180	-210.80	539.20
	200	-190.11	559.89
	220	-171.46	578.54
	240	-154.63	595.37
	260	-139.46	610.54
	280	-125.78	624.22
	300	-113.43	636.57
	320	-102.30	647.70
	340	-92.26	657.74
	360	-83.21	666.79
	380	-75.05	674.95
	400	-67.68	682.32
	420	-61.04	688.96
	440	-55.05	694.95
	460	-49.65	700.35
	480	-44.78	705.22

n500+3 Loading Conditions Hydraulic Grade Line for the
Santa Fe Pump Plant Sta 52+86.7
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Santa Fe Pump Plant Loading Conditions			n500+5
Elevation Top of Protection (MSL)	759.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-750		
c_l	0.005164		
x_1 (ft)	65		
L_2 (ft)	13		
x_3 (ft)	194		
h_o	-534.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-78	-750.00	0.00
	-40	-644.31	105.69
	0	-534.01	215.99
	20	-481.61	268.39
	40	-434.35	315.65
Pump Plant Location	50	-412.49	337.51
	60	-391.73	358.27
	80	-353.29	396.71
	100	-318.63	431.37
	120	-287.36	462.64
	140	-259.16	490.84
	160	-233.73	516.27
	180	-210.80	539.20
	200	-190.11	559.89
	220	-171.46	578.54
	240	-154.63	595.37
	260	-139.46	610.54
	280	-125.78	624.22
	300	-113.43	636.57
	320	-102.30	647.70
	340	-92.26	657.74
	360	-83.21	666.79
	380	-75.05	674.95
	400	-67.68	682.32
	420	-61.04	688.96
	440	-55.05	694.95
	460	-49.65	700.35
	480	-44.78	705.22

n500+5 Loading Conditions Hydraulic Grade Line for the
Santa Fe Pump Plant Sta 52+86.7
Central Industrial District Kansas Levee Unit



Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the New Central Pump Station Sta 60+00

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.

PDT
Name : Kuzniakowski EC-GD
Date : 6/19/2007

Peer Review
Checked by : EC-GD
Date :

NOTES:
1. Existing condition: Pump Station at Landward Toe of Levee
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise
4. Approx. 10 feet of trash fill exists landside, so reduce blanket thickness by 5 feet.

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	762.10
500 year + 0':	760.90
500 year + 3':	763.90
500 year + 5':	765.90
Ground Surface Land Side:	750 (Subtract 5' from $z_{bl\ due\ to\ debris\ fill}$, not for H)
Ground Surface River Side:	738
Bottom of Blanket:	722
Top of Bedrock:	672
Ground Surface at Pump Station:	750

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

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Input Parameters

Station	Design Condition	Permeability Ratio: River Side (K_f/K_{br})	Permeability Ratio: Land Side (K_f/K_{bl})	Blanket Thickness (ft): River Side z_{br}	Blanket Thickness (ft): Protection z_{bo}	Blanket Thickness (ft): Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Protection L_2	Seepage Length (ft): Land Side L_3	Driving Head (ft) H
58+12	Existing Condition	400	400	16.0	19.5	23.0	50.0	0	85	1000	12.1
58+12	n500 + 0	400	400	16.0	19.5	23.0	50.0	0	85	1000	10.9
58+12	n500 +3	400	400	16.0	19.5	23.0	50.0	0	85	1000	13.9
58+12	n500 + 5	400	400	16.0	19.5	23.0	50.0	0	85	1000	15.9

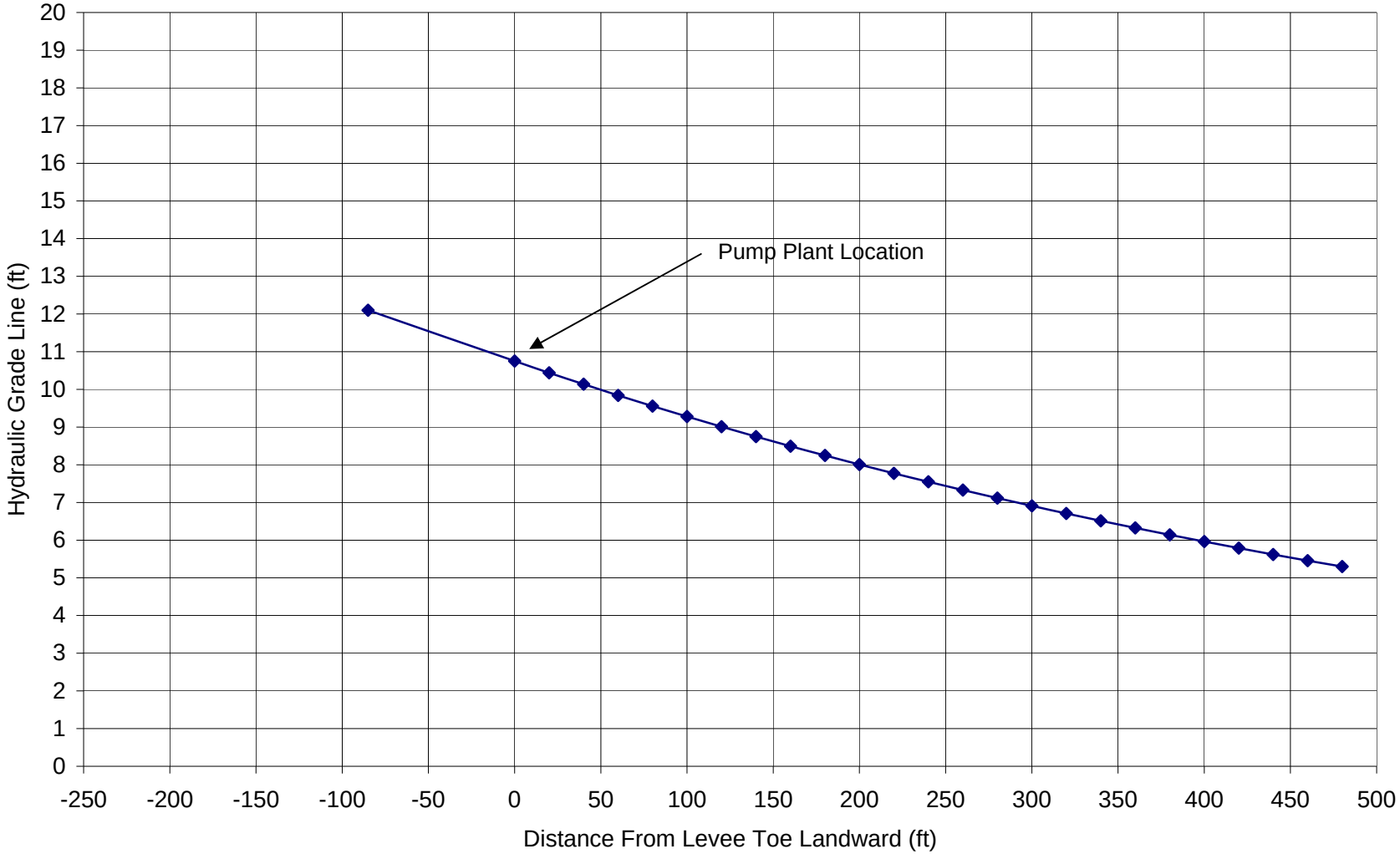
Analysis

Station	Design Condition	Factors: c_r	Factors: c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
58+12	Existing Condition	0.001768	0.001474	0	678	10.8	0.47	0.84	1.80	
58+12	n500 + 0	0.001768	0.001474	0	678	9.7	0.42	0.84	2.00	
58+12	n500 +3	0.001768	0.001474	0	678	12.4	0.54	0.84	1.57	
58+12	n500 + 5	0.001768	0.001474	0	678	14.1	0.61	0.84	1.37	

Hydraulic Grade Line for the New Central Pump Plant
Existing Conditions

Elevation Top of Protection (MSL)	762.1	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	12.1		
c_l	0.001474		
x_1 (ft)	0		
L_2 (ft)	85		
x_3 (ft)	678		
h_o	10.8		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-85	12.10	762.10
Pump Plant Location	0	10.75	760.75
	20	10.44	760.44
	40	10.14	760.14
	60	9.84	759.84
	80	9.56	759.56
	100	9.28	759.28
	120	9.01	759.01
	140	8.75	758.75
	160	8.49	758.49
	180	8.25	758.25
	200	8.01	758.01
	220	7.77	757.77
	240	7.55	757.55
	260	7.33	757.33
	280	7.12	757.12
	300	6.91	756.91
	320	6.71	756.71
	340	6.51	756.51
	360	6.32	756.32
	380	6.14	756.14
	400	5.96	755.96
	420	5.79	755.79
	440	5.62	755.62
	460	5.46	755.46
	480	5.30	755.30

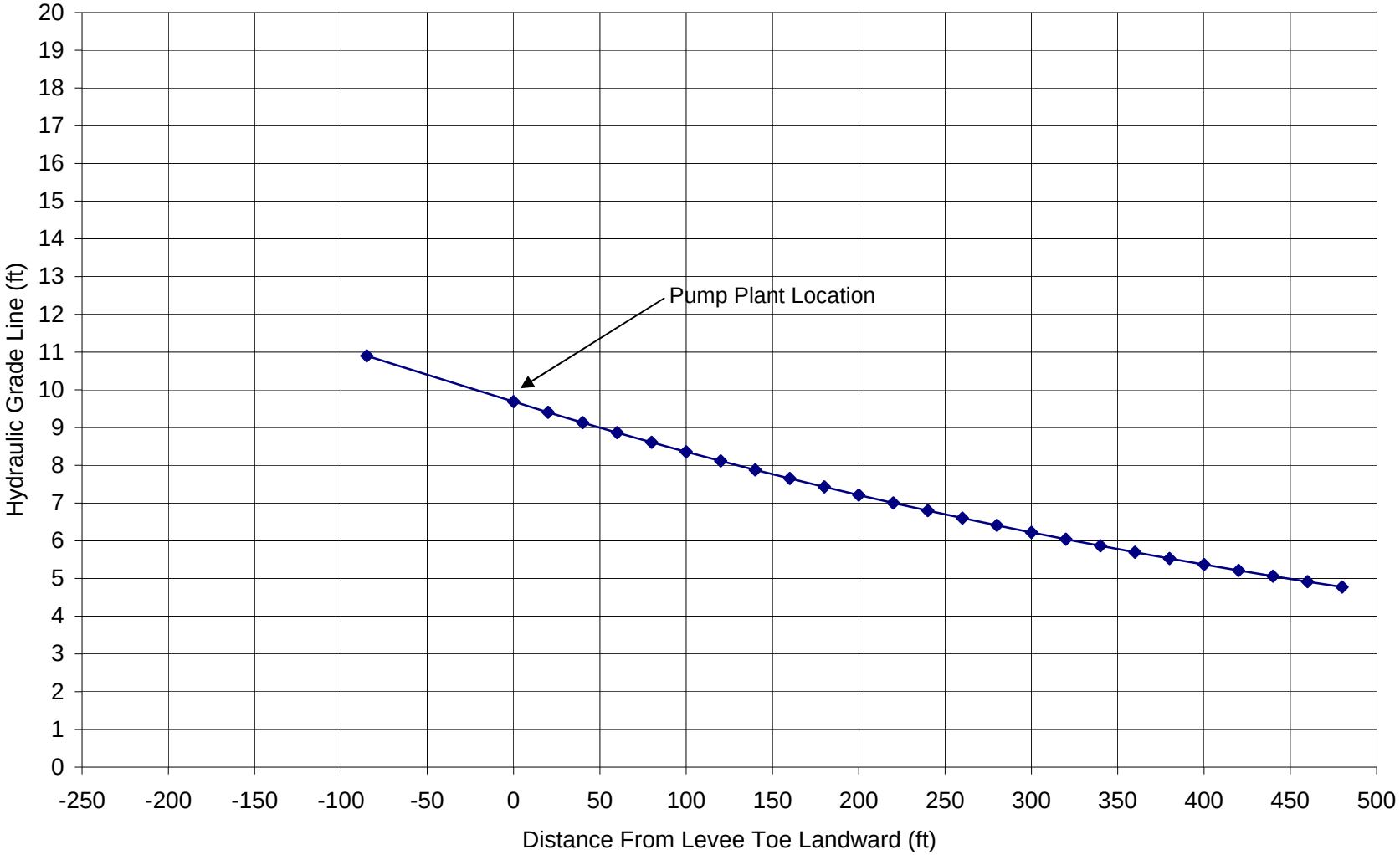
Existing Conditions Hydraulic Grade Line for the
New Central Pump Plant Sta 60+00
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the New Central Pump Plant
n500+0 Loading Conditions

Elevation Top of Protection (MSL)	760.90	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	10.9		
c_l	0.001474		
x_1 (ft)	0		
L_2 (ft)	85		
x_3 (ft)	678		
h_o	9.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-85	10.90	760.90
Pump Plant Location	0	9.69	759.69
	20	9.40	759.40
	40	9.13	759.13
	60	8.87	758.87
	80	8.61	758.61
	100	8.36	758.36
	120	8.12	758.12
	140	7.88	757.88
	160	7.65	757.65
	180	7.43	757.43
	200	7.21	757.21
	220	7.00	757.00
	240	6.80	756.80
	260	6.60	756.60
	280	6.41	756.41
	300	6.22	756.22
	320	6.04	756.04
	340	5.87	755.87
	360	5.70	755.70
	380	5.53	755.53
	400	5.37	755.37
	420	5.21	755.21
	440	5.06	755.06
	460	4.92	754.92
	480	4.77	754.77

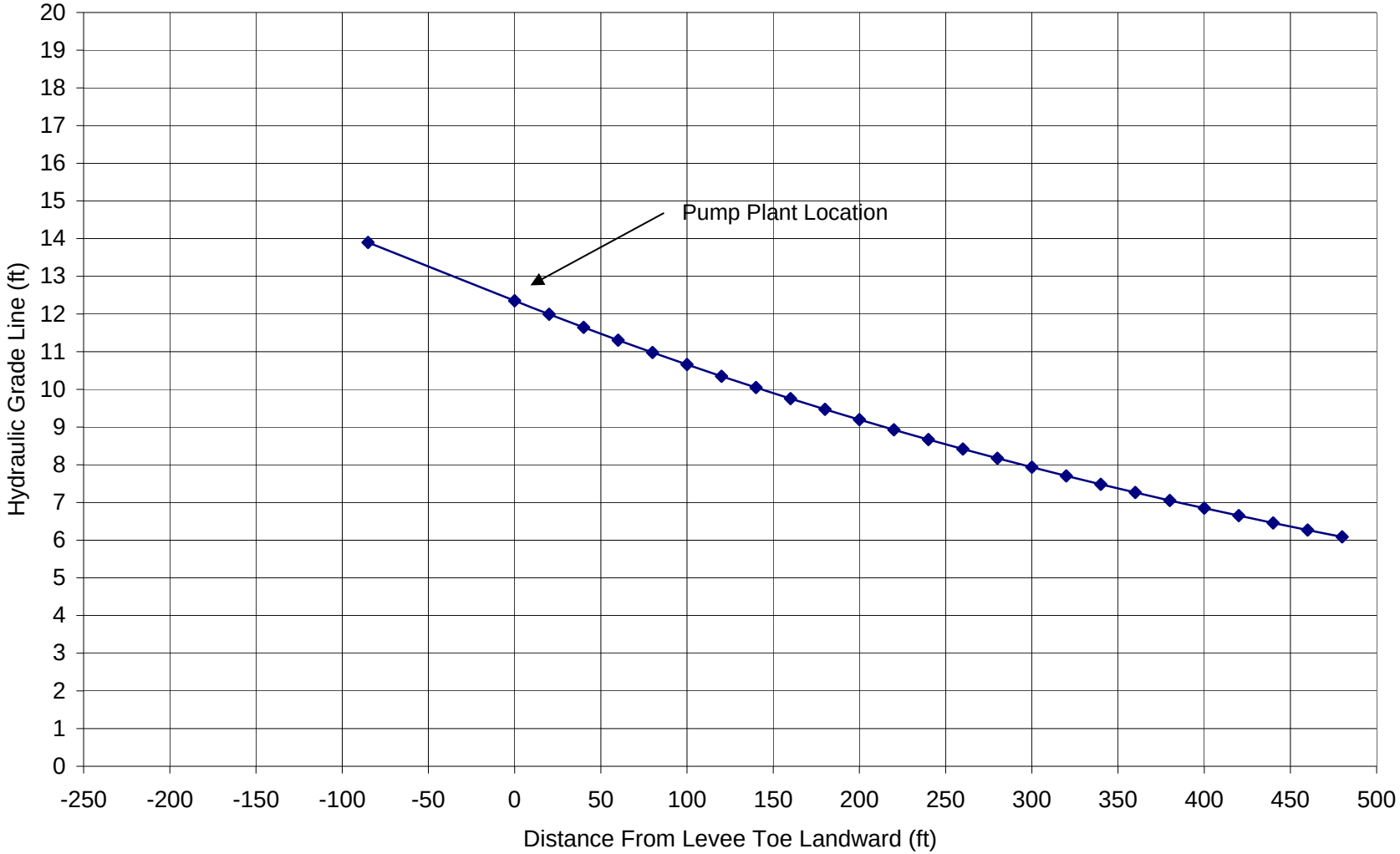
n500+0 Loading Conditions Hydraulic Grade Line for the
New Central Pump Plant Sta 60+00
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the New Central Pump Plant
n500+3 Loading Conditions

Elevation Top of Protection (MSL)	763.90	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	13.9		
c_l	0.001474		
x_1 (ft)	0		
L_2 (ft)	85		
x_3 (ft)	678		
h_o	12.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-85	13.90	763.90
Pump Plant Location	0	12.35	762.35
	20	11.99	761.99
	40	11.64	761.64
	60	11.31	761.31
	80	10.98	760.98
	100	10.66	760.66
	120	10.35	760.35
	140	10.05	760.05
	160	9.76	759.76
	180	9.47	759.47
	200	9.20	759.20
	220	8.93	758.93
	240	8.67	758.67
	260	8.42	758.42
	280	8.17	758.17
	300	7.94	757.94
	320	7.71	757.71
	340	7.48	757.48
	360	7.26	757.26
	380	7.05	757.05
	400	6.85	756.85
	420	6.65	756.65
	440	6.46	756.46
	460	6.27	756.27
	480	6.09	756.09

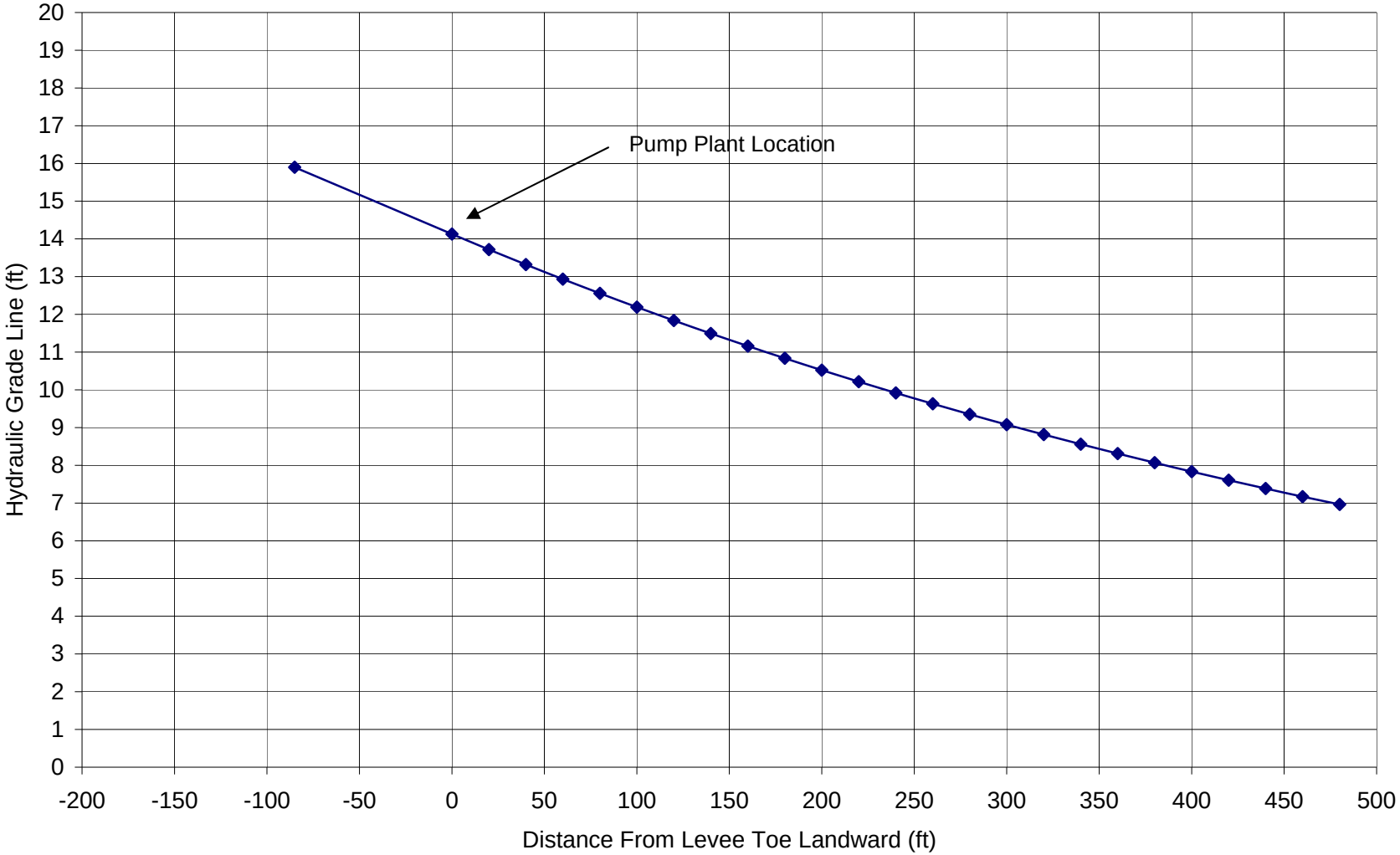
n500+3 Loading Conditions Hydraulic Grade Line for the
New Central Pump Plant Sta 60+00
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the New Central Pump Plant
n500+5 Loading Conditions

Elevation Top of Protection (MSL)	765.90	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.9		
c_l	0.001474		
x_1 (ft)	0		
L_2 (ft)	85		
x_3 (ft)	678		
h_o	14.1		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-85	15.90	765.90
Pump Plant Location	0	14.13	764.13
	20	13.72	763.72
	40	13.32	763.32
	60	12.93	762.93
	80	12.56	762.56
	100	12.19	762.19
	120	11.84	761.84
	140	11.49	761.49
	160	11.16	761.16
	180	10.84	760.84
	200	10.52	760.52
	220	10.22	760.22
	240	9.92	759.92
	260	9.63	759.63
	280	9.35	759.35
	300	9.08	759.08
	320	8.81	758.81
	340	8.56	758.56
	360	8.31	758.31
	380	8.07	758.07
	400	7.83	757.83
	420	7.61	757.61
	440	7.39	757.39
	460	7.17	757.17
	480	6.96	756.96

n500+5 Loading Conditions Hydraulic Grade Line for the
New Central Pump Plant Sta 60+00
Central Industrial District Kansas Levee Unit



Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the Stockyards No. 3 Pump Station Sta 74+22

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.

PDT
Name : Kuzniakowski EC-GD
Date : 6/19/2007

Peer Review
Checked by : EC-GD
Date :

NOTES:
1. Existing condition: Pump Station at Landside Toe of Levee
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	762.50
500 year + 0':	762.30
500 year + 3':	765.30
500 year + 5':	767.30
Ground Surface Land Side:	747
Ground Surface River Side:	738
Bottom of Blanket:	722
Top of Bedrock:	672
Ground Surface at Pump Station:	742

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

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Input Parameters

Station	Design Condition	Permeability Ratio: River Side (K_f/K_{br})	Permeability Ratio: Land Side (K_f/K_{bl})	Blanket Thickness (ft): River Side z_{br}	Blanket Thickness (ft): Protection z_{bo}	Blanket Thickness (ft): Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Protection L_2	Seepage Length (ft): Land Side L_3	Driving Head (ft) H
74+22	Existing Condition	500	500	16.0	20.5	25.0	50.0	0	120	1000	15.5
74+22	n500 + 0	500	500	16.0	20.5	25.0	50.0	0	120	1000	15.3
74+22	n500 +3	500	500	16.0	20.5	25.0	50.0	0	120	1000	18.3
74+22	n500 + 5	500	500	16.0	20.5	25.0	50.0	0	120	1000	20.3

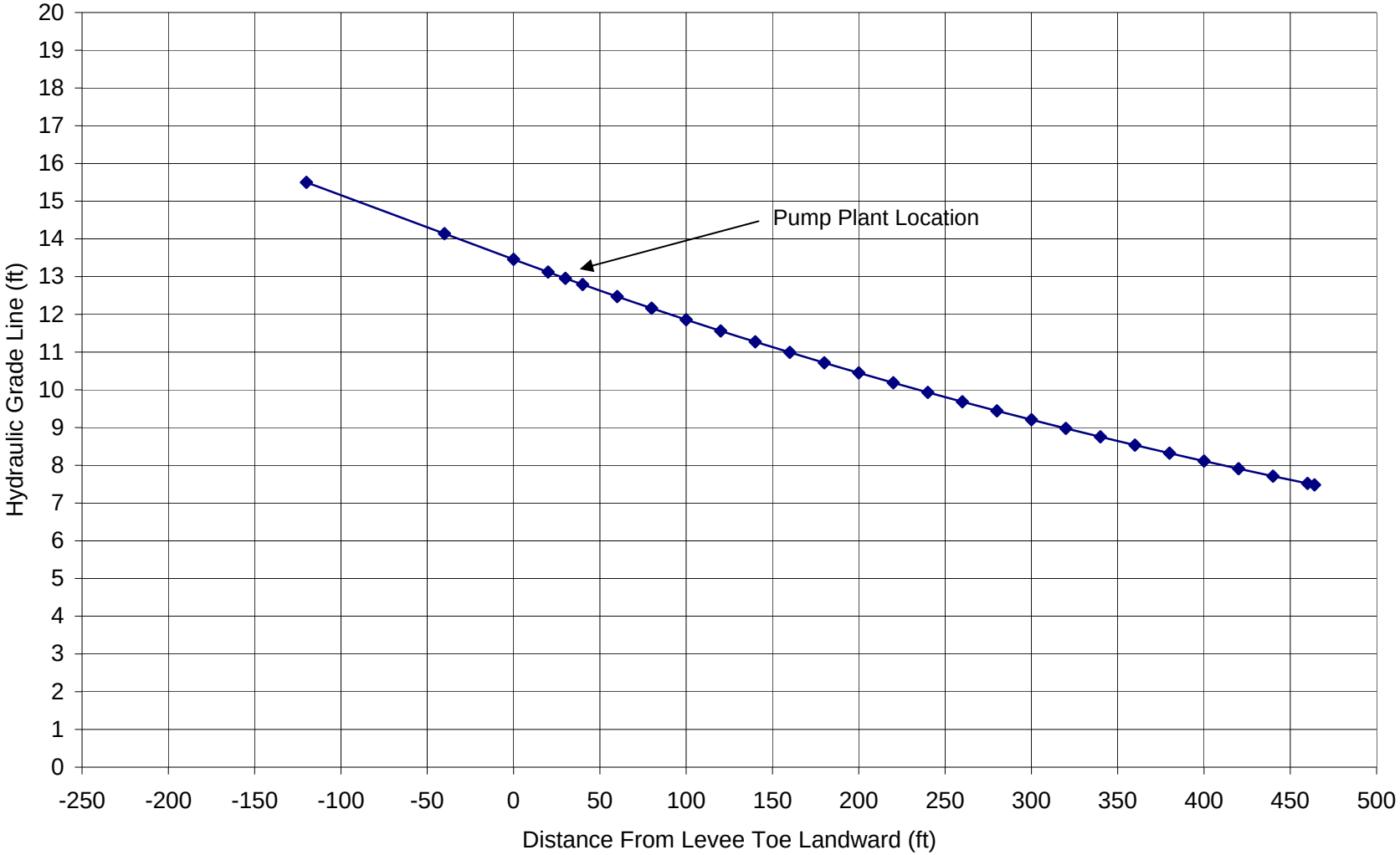
Analysis

Station	Design Condition	Factors: c_r	Factors: c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
74+22	Existing Condition	0.001581	0.001265	0	791	13.5	0.54	0.84	1.57	
74+22	n500 + 0	0.001581	0.001265	0	791	13.3	0.53	0.84	1.59	
74+22	n500 +3	0.001581	0.001265	0	791	15.9	0.64	0.84	1.33	
74+22	n500 + 5	0.001581	0.001265	0	791	17.6	0.70	0.84	1.20	

Hydraulic Grade Line for the Stockyards No. 3 Pump Plant
Existing Conditions

Elevation Top of Protection (MSL)	762.5	Existing Conditions	
Elevation at Landside Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.5		
c_l	0.001265		
x_1 (ft)	0		
L_2 (ft)	120		
x_3 (ft)	791		
h_o	13.5		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	15.50	762.50
	-40	14.14	761.14
	0	13.46	760.46
	20	13.12	760.12
Pump Plant Location	30	12.96	759.96
	40	12.79	759.79
	60	12.47	759.47
	80	12.16	759.16
	100	11.86	758.86
	120	11.56	758.56
	140	11.27	758.27
	160	10.99	757.99
	180	10.72	757.72
	200	10.45	757.45
	220	10.19	757.19
	240	9.93	756.93
	260	9.69	756.69
	280	9.44	756.44
	300	9.21	756.21
	320	8.98	755.98
	340	8.75	755.75
	360	8.53	755.53
	380	8.32	755.32
	400	8.11	755.11
	420	7.91	754.91
	440	7.71	754.71
	460	7.52	754.52
	464	7.48	754.48

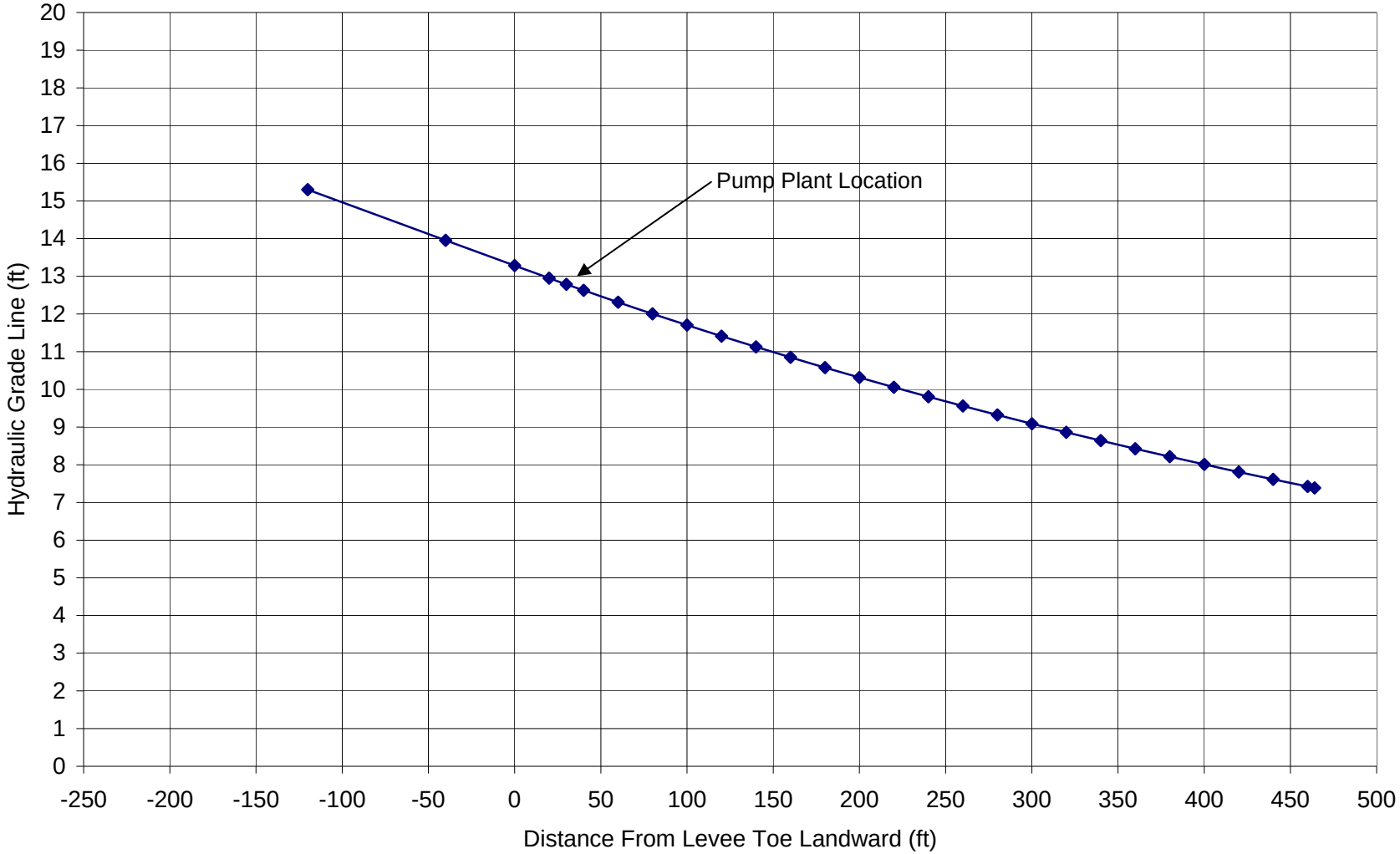
Existing Conditions Hydraulic Grade Line for the
Stockyards No. 3 Pump Plant Sta 74+22
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Stockyards No. 3 Pump Plant n500+0 Loading Conditions

Elevation Top of Protection (MSL)	762.30	Existing Conditions	
Elevation at Landside Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.3		
c_l	0.001265		
x_1 (ft)	0		
L_2 (ft)	120		
x_3 (ft)	791		
h_o	13.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	15.30	762.30
	-40	13.96	760.96
	0	13.28	760.28
	20	12.95	759.95
Pump Plant Location	30	12.79	759.79
	40	12.63	759.63
	60	12.31	759.31
	80	12.01	759.01
	100	11.71	758.71
	120	11.41	758.41
	140	11.13	758.13
	160	10.85	757.85
	180	10.58	757.58
	200	10.31	757.31
	220	10.06	757.06
	240	9.81	756.81
	260	9.56	756.56
	280	9.32	756.32
	300	9.09	756.09
	320	8.86	755.86
	340	8.64	755.64
	360	8.42	755.42
	380	8.21	755.21
	400	8.01	755.01
	420	7.81	754.81
	440	7.61	754.61
	460	7.42	754.42
	464	7.39	754.39

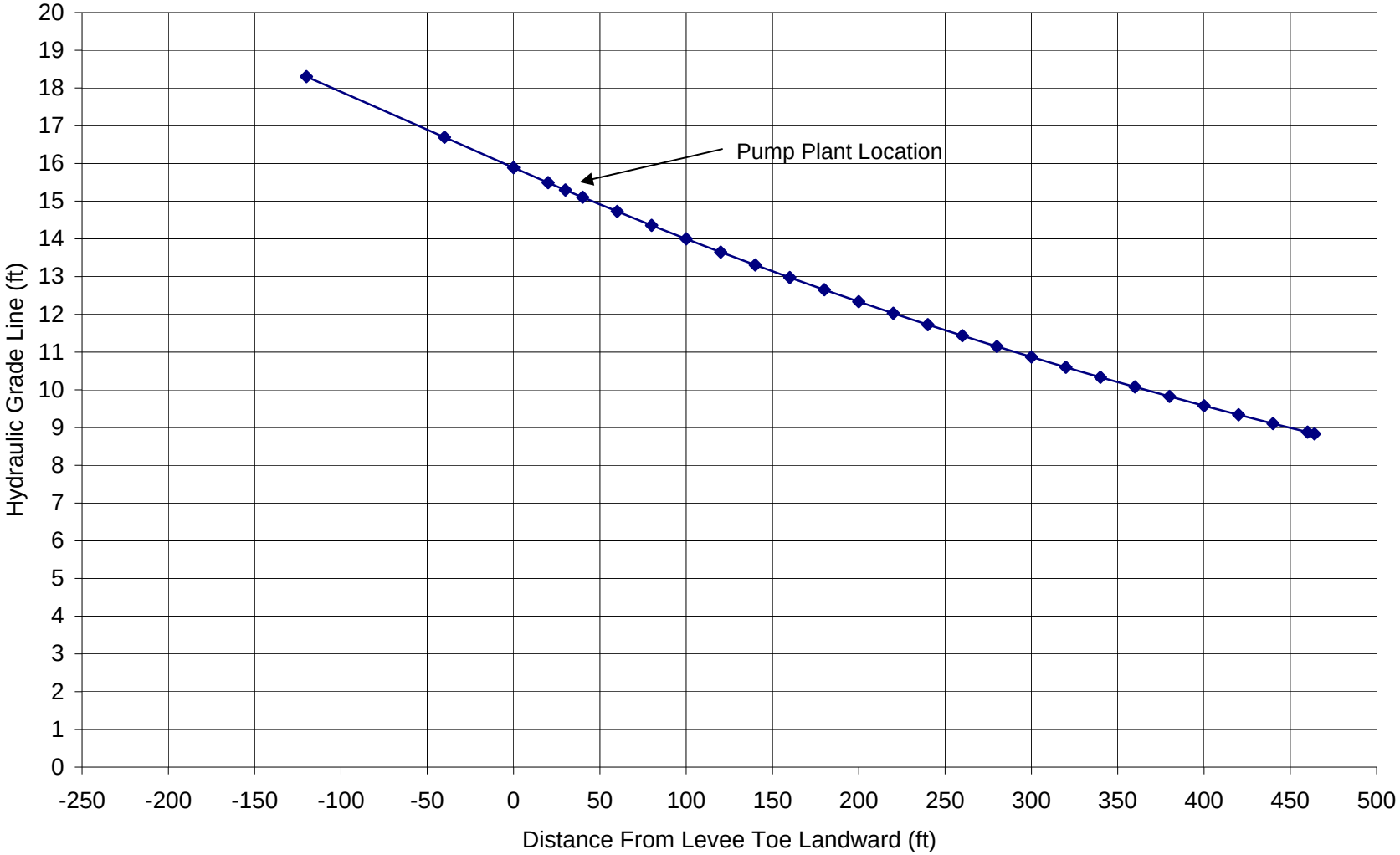
n500+0 Loading Conditions Hydraulic Grade Line for the Stockyards No. 3 Pump Plant Sta 74+22 Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Stockyards No. 3 Pump Plant n500+3 Loading Conditions

Elevation Top of Protection (MSL)	765.30	Existing Conditions	
Elevation at Landside Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	18.3		
c_l	0.001265		
x_1 (ft)	0		
L_2 (ft)	120		
x_3 (ft)	791		
h_o	15.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	18.30	765.30
	-40	16.69	763.69
	0	15.89	762.89
	20	15.49	762.49
Pump Plant Location	30	15.30	762.30
	40	15.10	762.10
	60	14.73	761.73
	80	14.36	761.36
	100	14.00	761.00
	120	13.65	760.65
	140	13.31	760.31
	160	12.98	759.98
	180	12.65	759.65
	200	12.34	759.34
	220	12.03	759.03
	240	11.73	758.73
	260	11.44	758.44
	280	11.15	758.15
	300	10.87	757.87
	320	10.60	757.60
	340	10.33	757.33
	360	10.08	757.08
	380	9.82	756.82
	400	9.58	756.58
	420	9.34	756.34
	440	9.11	756.11
	460	8.88	755.88
	464	8.83	755.83

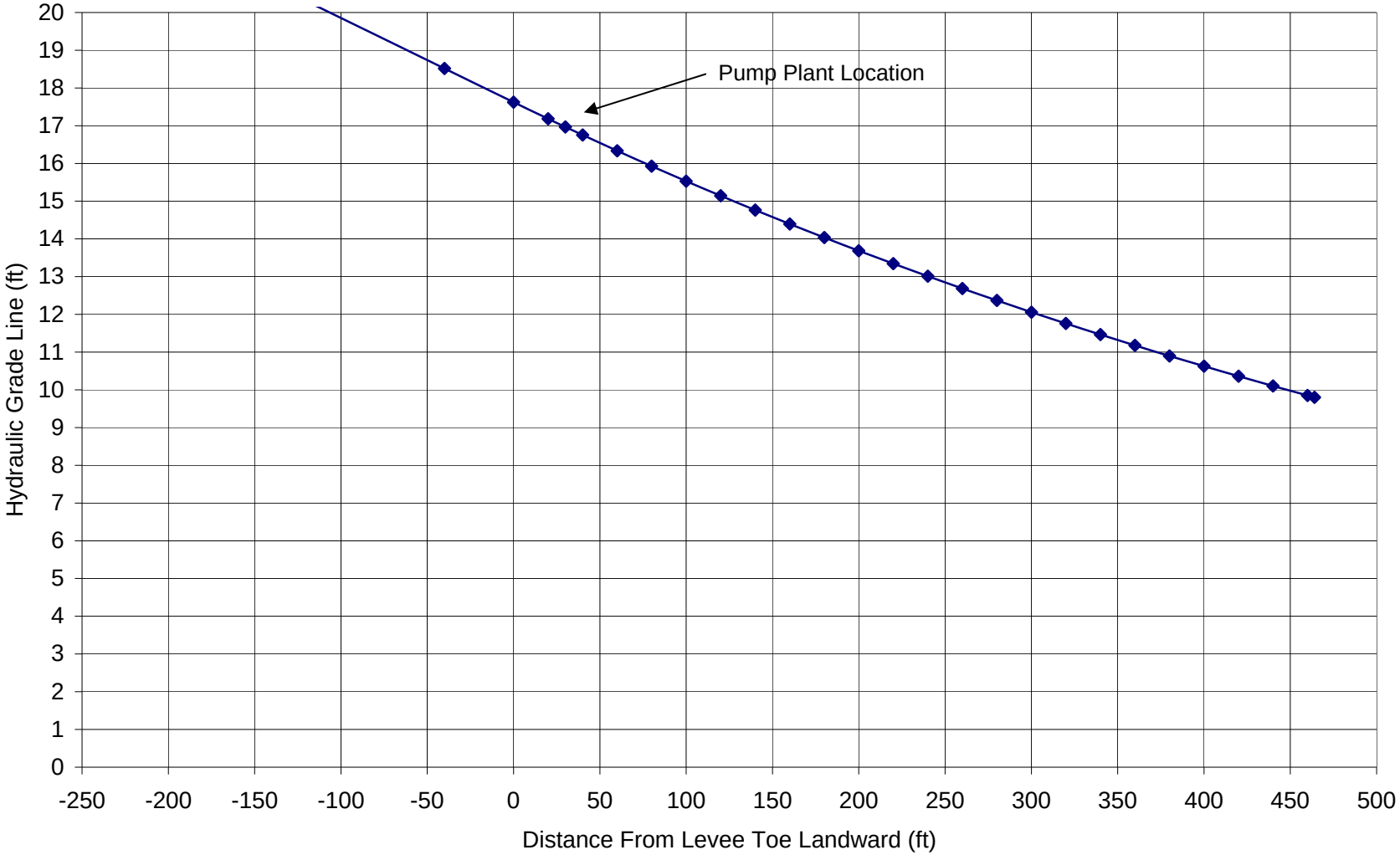
n500+3 Loading Conditions Hydraulic Grade Line for the Stockyards No. 3 Pump Plant Sta 74+22 Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Stockyards No. 3 Pump Plant n500+5 Loading Conditions

Elevation Top of Protection (MSL)	767.30	Existing Conditions	
Elevation at Landside Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	20.3		
c_l	0.001265		
x_1 (ft)	0		
L_2 (ft)	120		
x_3 (ft)	791		
h_o	17.6		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	20.30	767.30
	-40	18.52	765.52
	0	17.62	764.62
	20	17.18	764.18
Pump Plant Location	30	16.97	763.97
	40	16.76	763.76
	60	16.34	763.34
	80	15.93	762.93
	100	15.53	762.53
	120	15.14	762.14
	140	14.76	761.76
	160	14.40	761.40
	180	14.04	761.04
	200	13.69	760.69
	220	13.34	760.34
	240	13.01	760.01
	260	12.69	759.69
	280	12.37	759.37
	300	12.06	759.06
	320	11.76	758.76
	340	11.46	758.46
	360	11.18	758.18
	380	10.90	757.90
	400	10.63	757.63
	420	10.36	757.36
	440	10.10	757.10
	460	9.85	756.85
	464	9.80	756.80

n500+5 Loading Conditions Hydraulic Grade Line for the Stockyards No. 3 Pump Plant Sta 74+22 Central Industrial District Kansas Levee Unit



Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the Gateway Station Sta 80+90

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.

NOTES:
1. Existing condition: Pump Station on Landside Crest of Levee Section
2. Raise proposed currently unknown
3. L_2 not revised for minor change due to raise
4. Floodwalls Exist along this reach. This analysis does NOT include drawdown effects of wells.

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	762.50
500 year + 0':	762.90
500 year + 3':	765.90
500 year + 5':	767.90
Ground Surface Land Side:	742
Ground Surface River Side:	738
Bottom of Blanket:	716
Top of Bedrock:	656
Ground Surface at Pump Station:	760

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

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Input Parameters

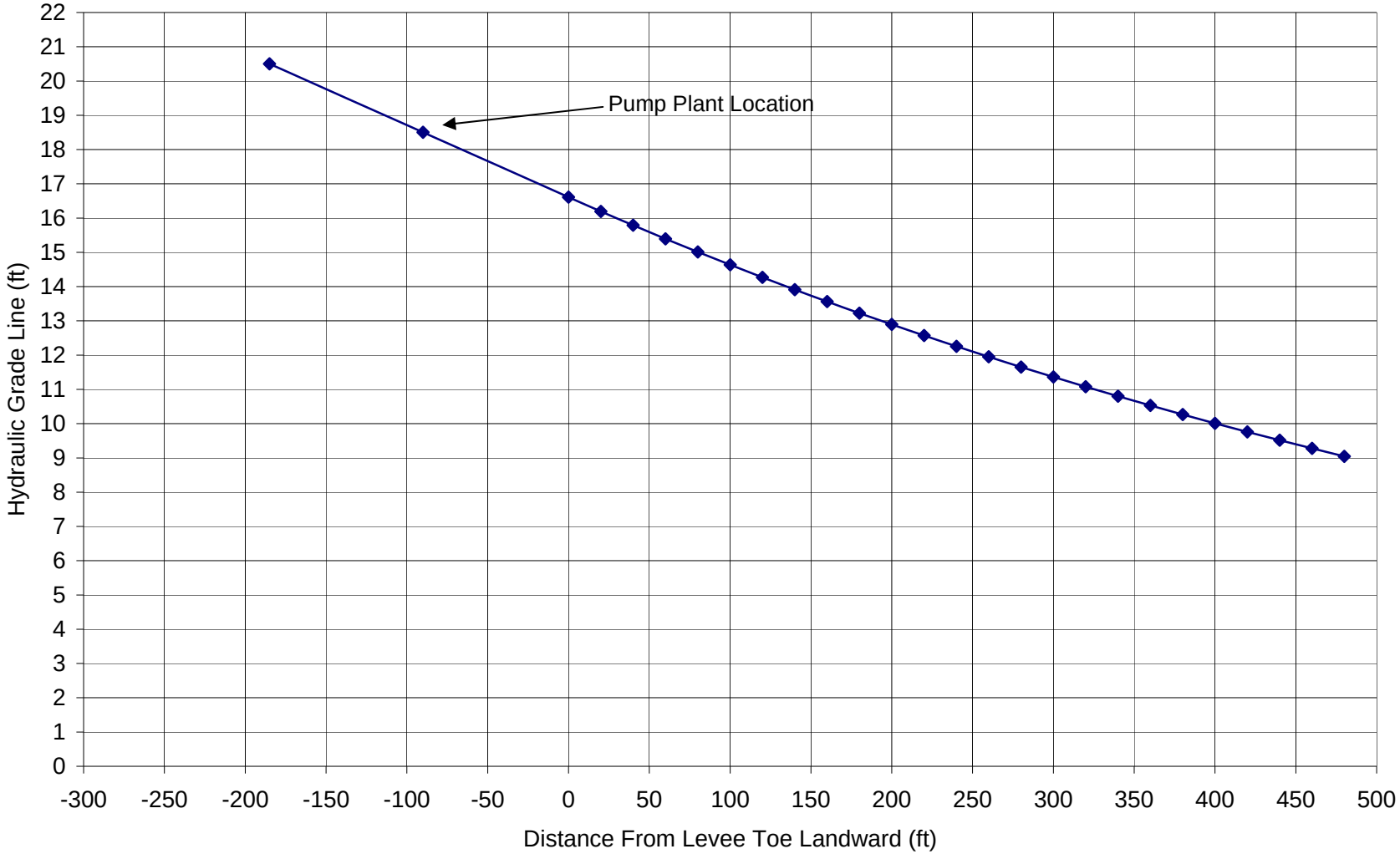
Station	Design Condition	Permeability Ratio: River Side (K_f/K_{br})	Permeability Ratio: Land Side (K_f/K_{bl})	Blanket Thickness (ft): River Side z_{br}	Blanket Thickness (ft): Protection z_{bo}	Blanket Thickness (ft): Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Protection L_2	Seepage Length (ft): Land Side L_3	Driving Head (ft) H
80+90	Existing Condition	400	400	22.0	24.0	26.0	60.0	0	185	1000	20.5
80+90	n500 + 0	400	400	22.0	24.0	26.0	60.0	0	185	1000	20.9
80+90	n500 +3	400	400	22.0	24.0	26.0	60.0	0	185	1000	23.9
80+90	n500 + 5	400	400	22.0	24.0	26.0	60.0	0	185	1000	25.9

Analysis

Factors: c_r	Factors: c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
0.001376	0.001266	0	790	16.6	0.64	0.84	1.32	
0.001376	0.001266	0	790	16.9	0.65	0.84	1.29	
0.001376	0.001266	0	790	19.4	0.74	0.84	1.13	
0.001376	0.001266	0	790	21.0	0.81	0.84	1.04	

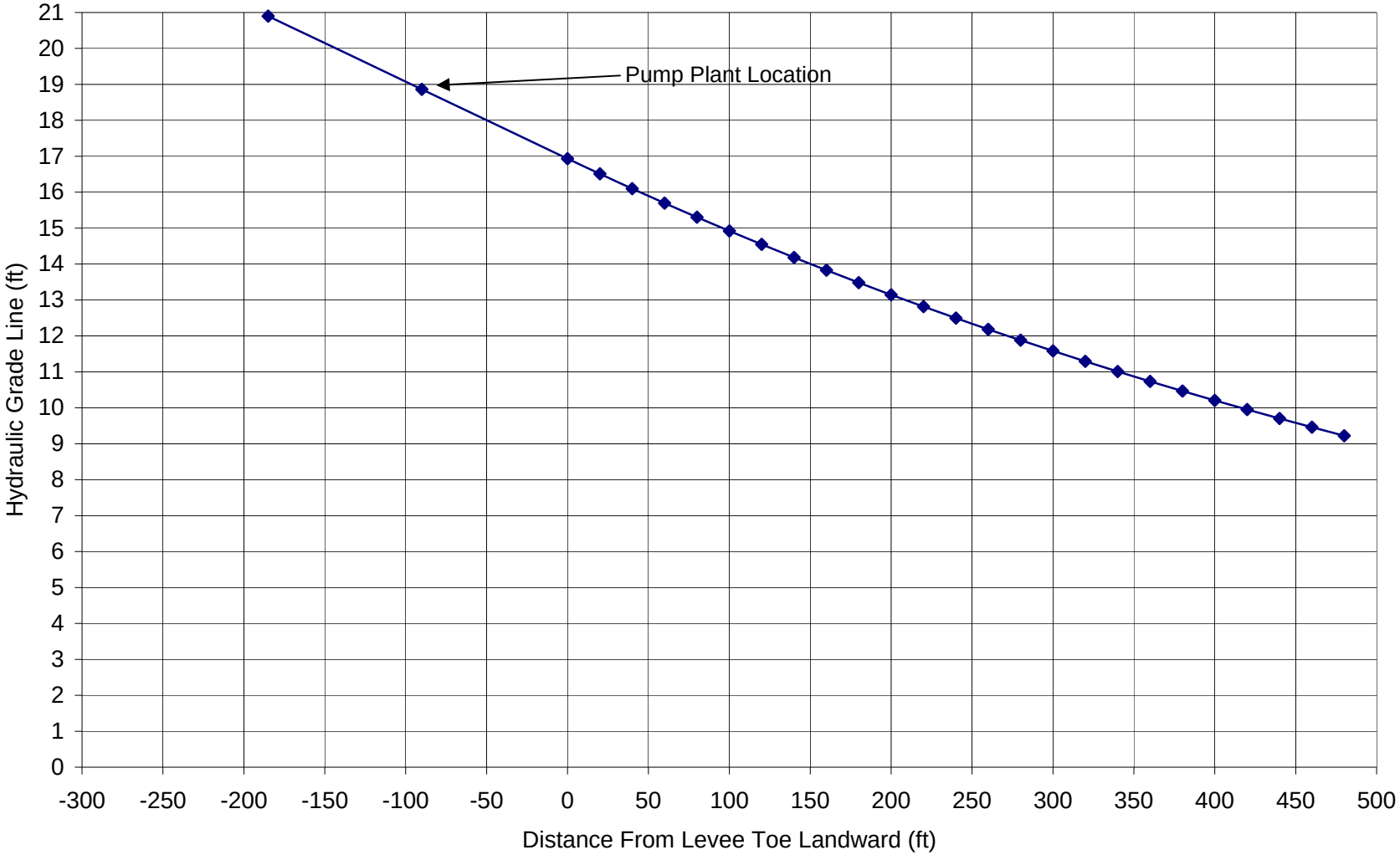
Hydraulic Grade Line for the Gateway Pump Plant Conditions			Existing
Elevation Top of Protection (MSL)	762.5	Existing Conditions	
Elevation at Landside Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	716		
Driving Head (ft)	20.5		
c_l	0.001266		
x_1 (ft)	0		
L_2 (ft)	185		
x_3 (ft)	790		
h_o	16.6		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-185	20.50	762.50
Pump Plant Location	-90	18.50	760.50
	0	16.61	758.61
	20	16.19	758.19
	40	15.79	757.79
	60	15.40	757.40
	80	15.01	757.01
	100	14.63	756.63
	120	14.27	756.27
	140	13.91	755.91
	160	13.56	755.56
	180	13.23	755.23
	200	12.89	754.89
	220	12.57	754.57
	240	12.26	754.26
	260	11.95	753.95
	280	11.65	753.65
	300	11.36	753.36
	320	11.08	753.08
	340	10.80	752.80
	360	10.53	752.53
	380	10.27	752.27
	400	10.01	752.01
	420	9.76	751.76
	440	9.52	751.52
	460	9.28	751.28
	480	9.05	751.05

Existing Conditions Hydraulic Grade Line for the
Gateway Pump Plant Sta 80+90
Central Industrial District Kansas Levee Unit



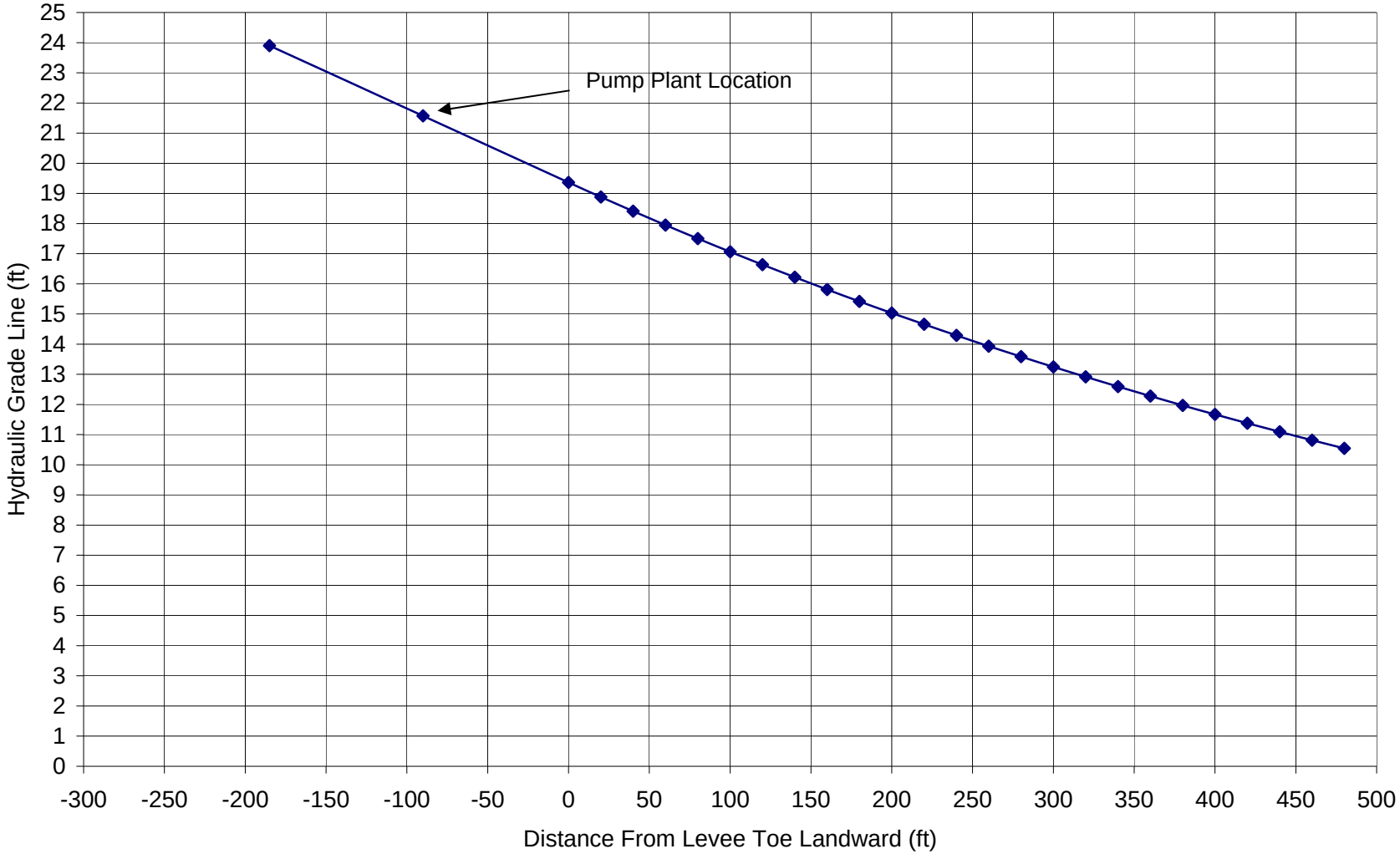
Hydraulic Grade Line for the Gateway Pump Plant Loading Conditions			n500+0
Elevation Top of Protection (MSL)	762.90	Existing Conditions	
Elevation at Landside Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	716		
Driving Head (ft)	20.9		
c_l	0.001266		
x_1 (ft)	0		
L_2 (ft)	185		
x_3 (ft)	790		
h_o	16.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-185	20.90	762.90
Pump Plant Location	-90	18.86	760.86
	0	16.93	758.93
	20	16.51	758.51
	40	16.10	758.10
	60	15.70	757.70
	80	15.30	757.30
	100	14.92	756.92
	120	14.55	756.55
	140	14.18	756.18
	160	13.83	755.83
	180	13.48	755.48
	200	13.15	755.15
	220	12.82	754.82
	240	12.50	754.50
	260	12.18	754.18
	280	11.88	753.88
	300	11.58	753.58
	320	11.29	753.29
	340	11.01	753.01
	360	10.74	752.74
	380	10.47	752.47
	400	10.21	752.21
	420	9.95	751.95
	440	9.70	751.70
	460	9.46	751.46
	480	9.22	751.22

n500+0 Loading Conditions Hydraulic Grade Line for the
Gateway Pump Plant Sta 80+90
Central Industrial District Kansas Levee Unit



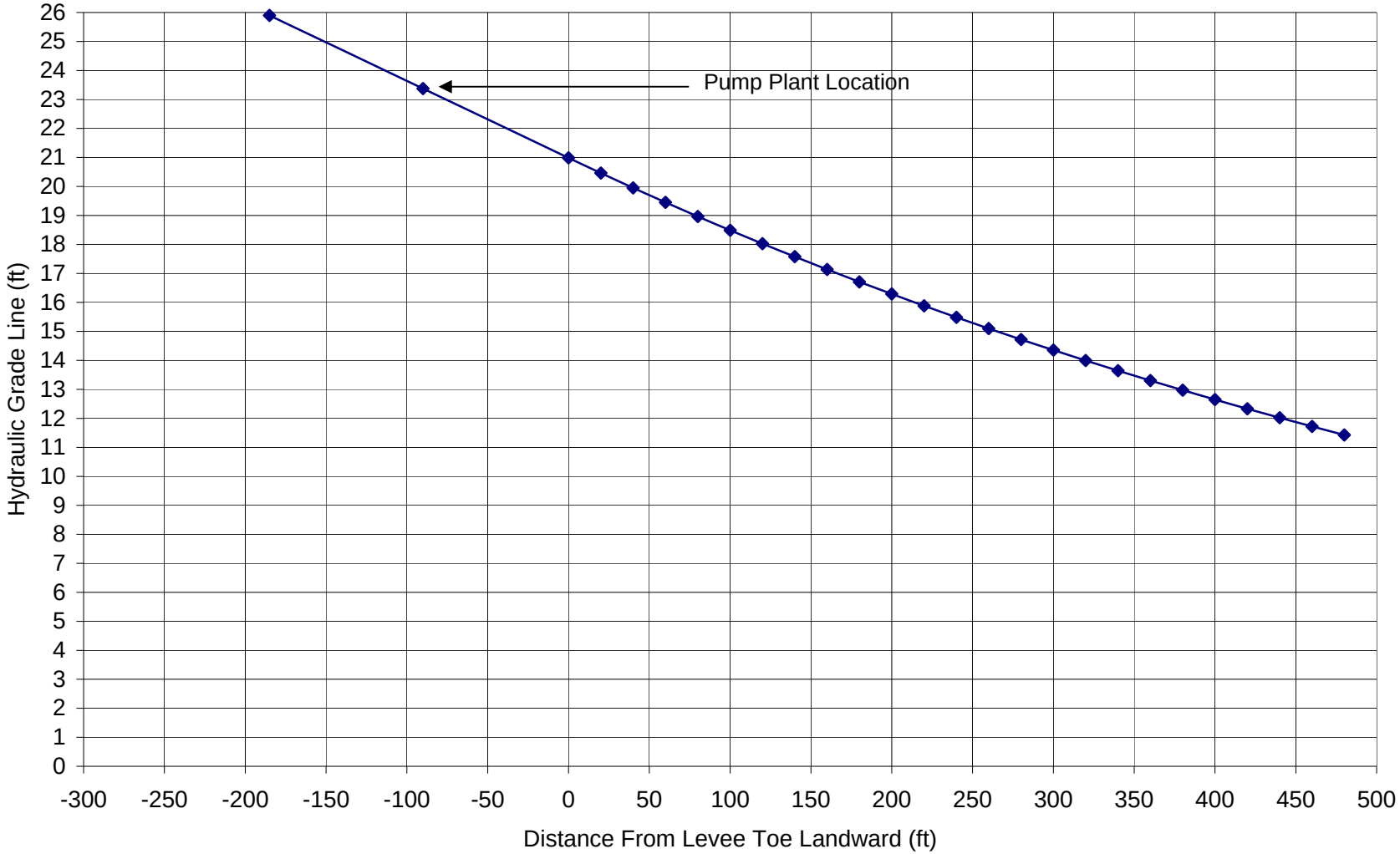
Hydraulic Grade Line for the Gateway Pump Plant Loading Conditions			n500+3
Elevation Top of Protection (MSL)	765.90	Existing Conditions	
Elevation at Landside Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	716		
Driving Head (ft)	23.9		
c_l	0.001266		
x_1 (ft)	0		
L_2 (ft)	185		
x_3 (ft)	790		
h_o	19.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-185	23.90	765.90
Pump Plant Location	-90	21.57	763.57
	0	19.36	761.36
	20	18.88	760.88
	40	18.41	760.41
	60	17.95	759.95
	80	17.50	759.50
	100	17.06	759.06
	120	16.64	758.64
	140	16.22	758.22
	160	15.81	757.81
	180	15.42	757.42
	200	15.03	757.03
	220	14.66	756.66
	240	14.29	756.29
	260	13.93	755.93
	280	13.59	755.59
	300	13.25	755.25
	320	12.91	754.91
	340	12.59	754.59
	360	12.28	754.28
	380	11.97	753.97
	400	11.67	753.67
	420	11.38	753.38
	440	11.09	753.09
	460	10.82	752.82
	480	10.55	752.55

n500+3 Loading Conditions Hydraulic Grade Line for the
Gateway Pump Plant Sta 80+90
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Gateway Pump Plant Loading Conditions			n500+5
Elevation Top of Protection (MSL)	767.90	Existing Conditions	
Elevation at Landside Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	716		
Driving Head (ft)	25.9		
c_l	0.001266		
x_1 (ft)	0		
L_2 (ft)	185		
x_3 (ft)	790		
h_o	21.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-185	25.90	767.90
Pump Plant Location	-90	23.38	765.38
	0	20.99	762.99
	20	20.46	762.46
	40	19.95	761.95
	60	19.45	761.45
	80	18.96	760.96
	100	18.49	760.49
	120	18.03	760.03
	140	17.58	759.58
	160	17.14	759.14
	180	16.71	758.71
	200	16.29	758.29
	220	15.88	757.88
	240	15.49	757.49
	260	15.10	757.10
	280	14.72	756.72
	300	14.35	756.35
	320	14.00	756.00
	340	13.65	755.65
	360	13.30	755.30
	380	12.97	754.97
	400	12.65	754.65
	420	12.33	754.33
	440	12.02	754.02
	460	11.72	753.72
	480	11.43	753.43

n500+5 Loading Conditions Hydraulic Grade Line for the
Gateway Pump Plant Sta 80+90
Central Industrial District Kansas Levee Unit



Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the Ohio Avenue Pump Station Sta 83+52 (CID-MO Stationing)

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.

PDT
Name : Kuzniakowski EC-GD
Date : 6/19/2007

Peer Review
Checked by : EC-GD
Date :

NOTES:
1. Existing condition: Pump Station on Landside of Levee
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise
4. Shallow buried collector along landside toe of levee. Not expected to have much effect on hgl

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	760.55
500 year + 0':	
500 year + 3':	
500 year + 5':	
Ground Surface Land Side:	754
Ground Surface River Side:	752
Bottom of Blanket:	725
Top of Bedrock:	670
Ground Surface at Pump Station:	754

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

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Input Parameters

Station	Design Condition	Permeability Ratio: River Side (K_f/K_{br})	Permeability Ratio: Land Side (K_f/K_{bl})	Blanket Thickness (ft): River Side z_{br}	Blanket Thickness (ft): Protection z_{bo}	Blanket Thickness (ft): Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Protection L_2	Seepage Length (ft): Land Side L_3	Driving Head (ft) H
83+52	Existing Condition	400	400	27.0	28.0	29.0	55.0	250	70	1000	6.5
83+52	n500 + 0	400	400	27.0	28.0	29.0	55.0	250	70	1000	-754.0
83+52	n500 +3	400	400	27.0	28.0	29.0	55.0	250	70	1000	-754.0
83+52	n500 + 5	400	400	27.0	28.0	29.0	55.0	250	70	1000	-754.0

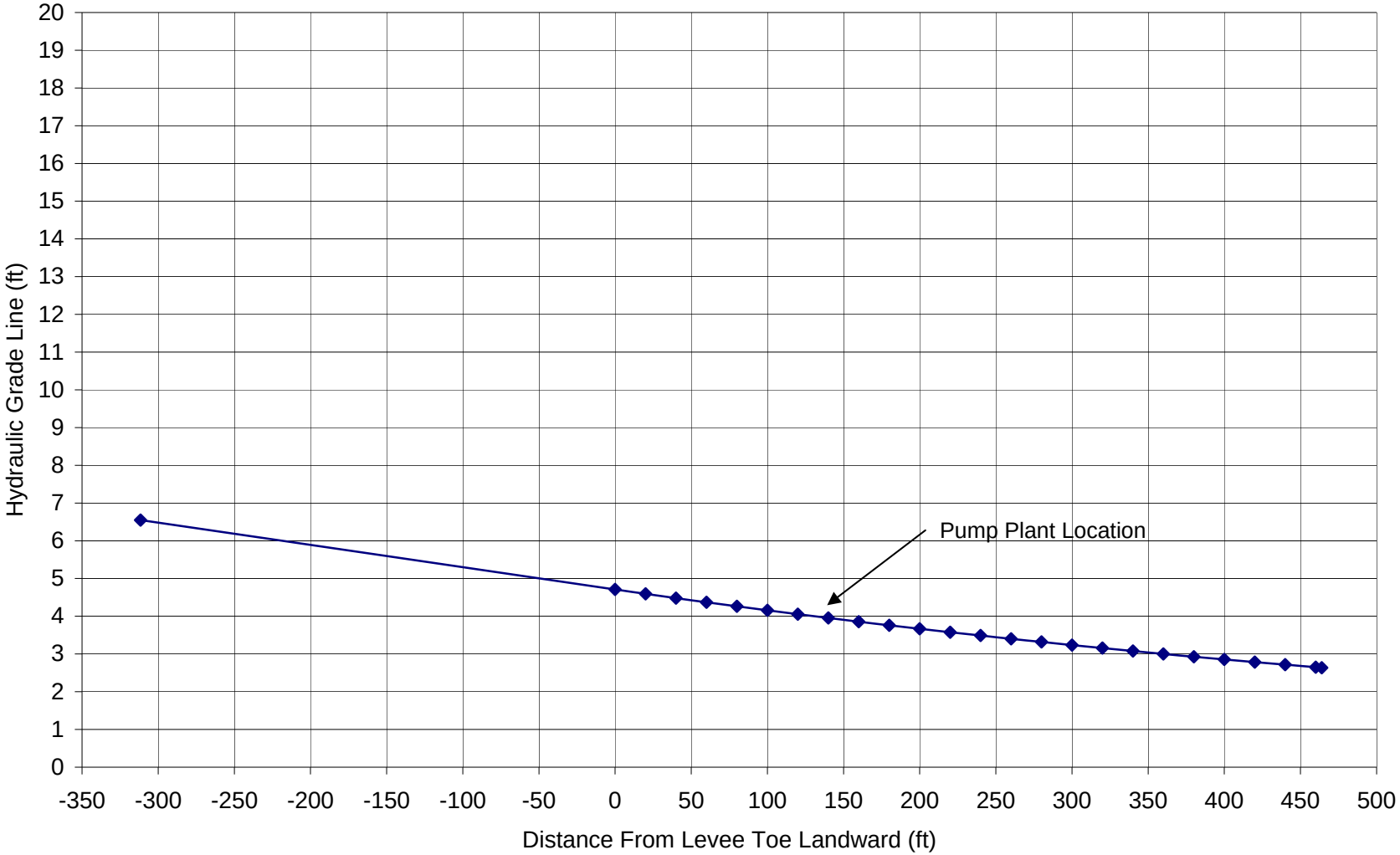
Analysis

Station	Design Condition	Factors: c_r	Factors: c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
83+52	Existing Condition	0.001297	0.001252	242	799	4.7	0.16	0.84	5.19	
83+52	n500 + 0	0.001297	0.001252	242	799	-542.4	-18.70	0.84	-0.05	
83+52	n500 +3	0.001297	0.001252	242	799	-542.4	-18.70	0.84	-0.05	
83+52	n500 + 5	0.001297	0.001252	242	799	-542.4	-18.70	0.84	-0.05	

Hydraulic Grade Line for the Ohio Avenue Pump Plant
Existing Conditions

Elevation Top of Protection (MSL)	760.55	Existing Conditions	
Elevation at Landside Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	6.55		
c_l	0.001252		
x_1 (ft)	242		
L_2 (ft)	70		
x_3 (ft)	799		
h_o	4.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-312	6.55	760.55
	0	4.71	758.71
	20	4.60	758.60
	40	4.48	758.48
	60	4.37	758.37
	80	4.26	758.26
	100	4.16	758.16
	120	4.05	758.05
Pump Plant Location	140	3.95	757.95
	160	3.86	757.86
	180	3.76	757.76
	200	3.67	757.67
	220	3.58	757.58
	240	3.49	757.49
	260	3.40	757.40
	280	3.32	757.32
	300	3.24	757.24
	320	3.16	757.16
	340	3.08	757.08
	360	3.00	757.00
	380	2.93	756.93
	400	2.86	756.86
	420	2.79	756.79
	440	2.72	756.72
	460	2.65	756.65
	464	2.64	756.64

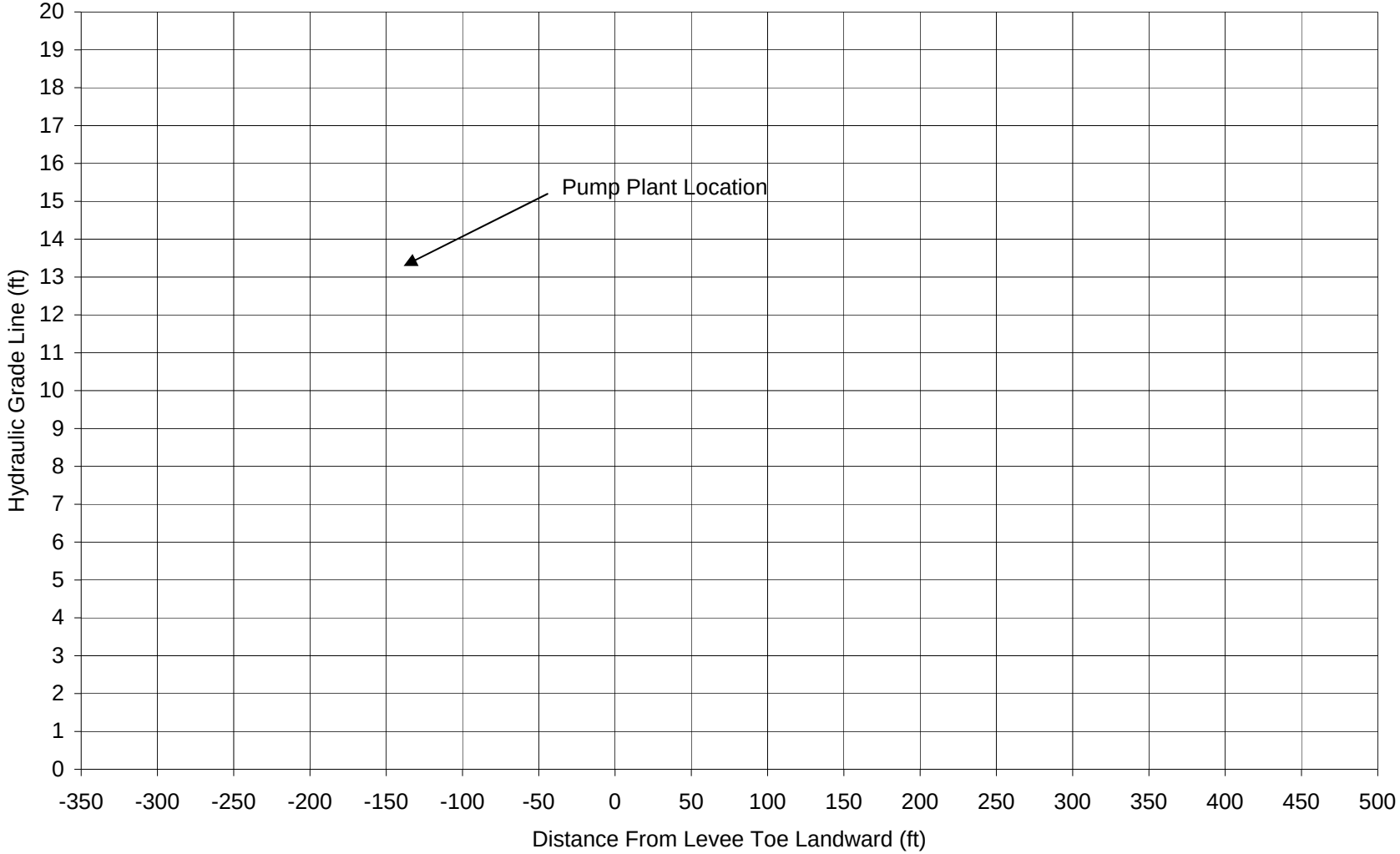
Existing Conditions Hydraulic Grade Line for the
Ohio Avenue Pump Plant Sta 83+52 (CID-MO Stationing)
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Ohio Avenue Pump Plant
n500+0 Loading Conditions

Elevation Top of Protection (MSL)	760.55	Existing Conditions	
Elevation at Landside Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	-754		
c_l	0.001252		
x_1 (ft)	242		
L_2 (ft)	70		
x_3 (ft)	799		
h_o	-542.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-312	-754.00	0.00
	-40	-569.57	184.43
	0	-542.41	211.59
	20	-529.00	225.00
	40	-515.92	238.08
	60	-503.16	250.84
	80	-490.72	263.28
	100	-478.58	275.42
	120	-466.75	287.25
Pump Plant Location	140	-455.21	298.79
	160	-443.95	310.05
	180	-432.97	321.03
	200	-422.26	331.74
	220	-411.82	342.18
	240	-401.64	352.36
	260	-391.71	362.29
	280	-382.02	371.98
	300	-372.57	381.43
	320	-363.36	390.64
	340	-354.38	399.62
	360	-345.61	408.39
	380	-337.07	416.93
	400	-328.73	425.27
	420	-320.60	433.40
	440	-312.67	441.33
	460	-304.94	449.06
	464	-303.42	450.58

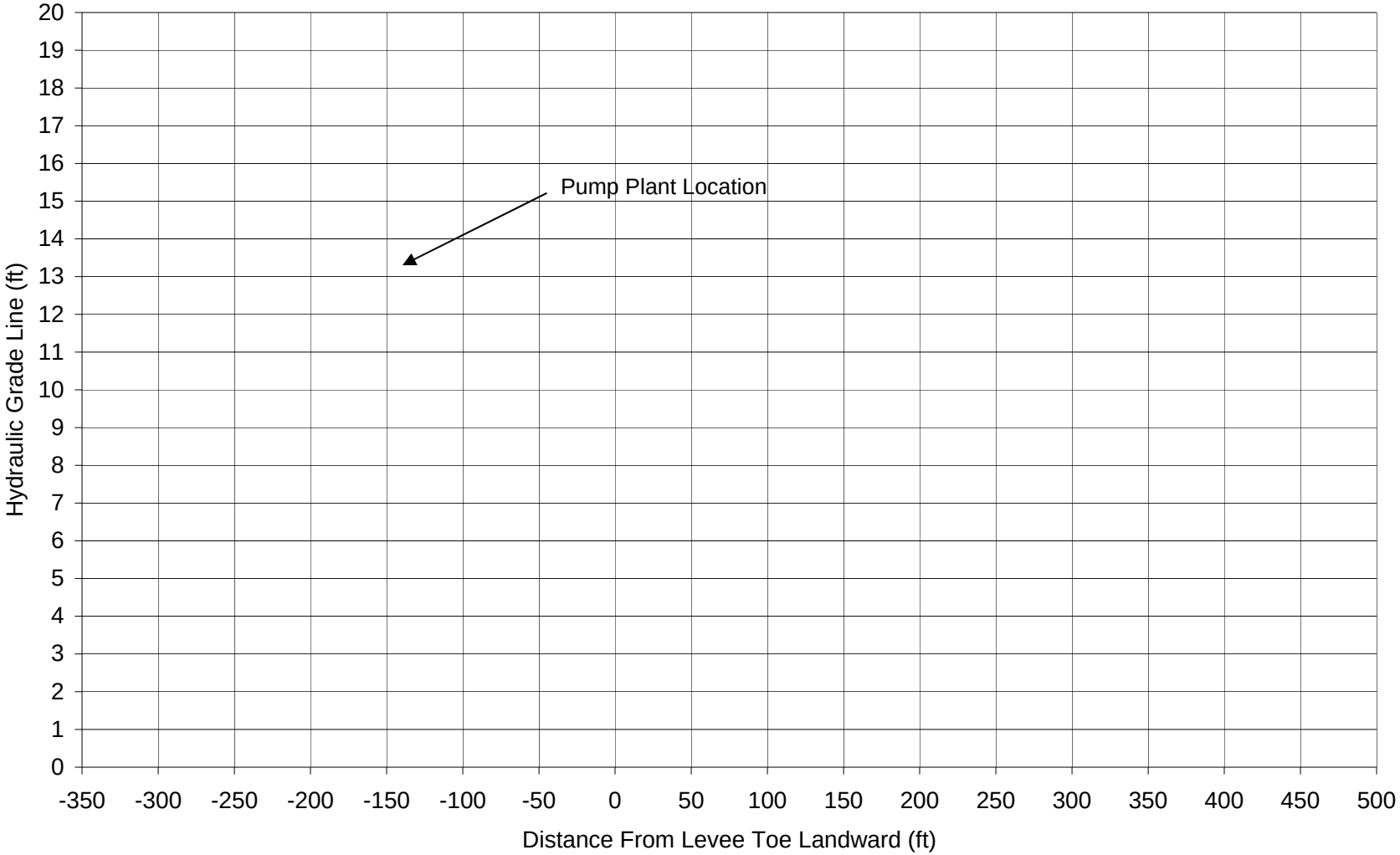
n500+0 Loading Conditions Hydraulic Grade Line for the
Ohio Avenue Pump Plant Sta 83+52 (CID-MO Stationing)
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Ohio Avenue Pump Plant
n500+3 Loading Conditions

Elevation Top of Protection (MSL)	760.55	Existing Conditions	
Elevation at Landside Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	-754		
c_l	0.001252		
x_1 (ft)	242		
L_2 (ft)	70		
x_3 (ft)	799		
h_o	-542.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-312	-754.00	0.00
	-40	-569.57	184.43
	0	-542.41	211.59
	20	-529.00	225.00
	40	-515.92	238.08
	60	-503.16	250.84
	80	-490.72	263.28
	100	-478.58	275.42
	120	-466.75	287.25
Pump Plant Location	140	-455.21	298.79
	160	-443.95	310.05
	180	-432.97	321.03
	200	-422.26	331.74
	220	-411.82	342.18
	240	-401.64	352.36
	260	-391.71	362.29
	280	-382.02	371.98
	300	-372.57	381.43
	320	-363.36	390.64
	340	-354.38	399.62
	360	-345.61	408.39
	380	-337.07	416.93
	400	-328.73	425.27
	420	-320.60	433.40
	440	-312.67	441.33
	460	-304.94	449.06
	464	-303.42	450.58

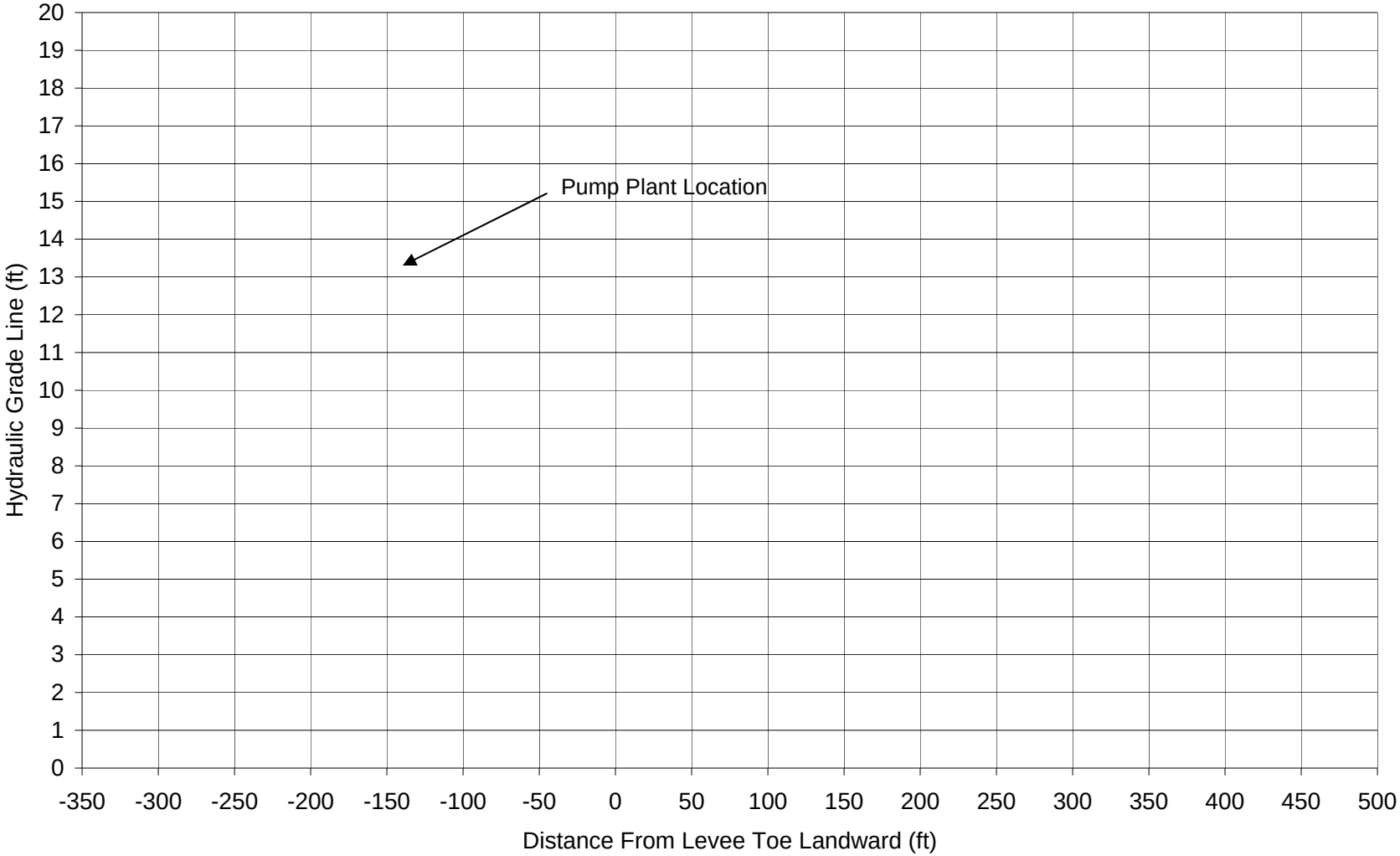
n500+3 Loading Conditions Hydraulic Grade Line for the
Ohio Avenue Pump Plant Sta 83+52 (CID-MO Stationing)
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Ohio Avenue Pump Plant
n500+5 Loading Conditions

Elevation Top of Protection (MSL)	760.55	Existing Conditions	
Elevation at Landside Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	-754		
c_l	0.001252		
x_1 (ft)	242		
L_2 (ft)	70		
x_3 (ft)	799		
h_o	-542.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-312	-754.00	0.00
	0	-542.41	211.59
	20	-529.00	225.00
	40	-515.92	238.08
	60	-503.16	250.84
	80	-490.72	263.28
	100	-478.58	275.42
	120	-466.75	287.25
Pump Plant Location	140	-455.21	298.79
	160	-443.95	310.05
	180	-432.97	321.03
	200	-422.26	331.74
	220	-411.82	342.18
	240	-401.64	352.36
	260	-391.71	362.29
	280	-382.02	371.98
	300	-372.57	381.43
	320	-363.36	390.64
	340	-354.38	399.62
	360	-345.61	408.39
	380	-337.07	416.93
	400	-328.73	425.27
	420	-320.60	433.40
	440	-312.67	441.33
	460	-304.94	449.06
	464	-303.42	450.58

n500+5 Loading Conditions Hydraulic Grade Line for the Ohio Avenue Pump Plant Sta 83+52 (CID-MO Stationing) Central Industrial District Kansas Levee Unit



Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Missouri - Hydraulic Grade Line for the Kemper Arena Pump Station Sta 106+49 (KS Sta)

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.

PDT
Name : Kuzniakowski EC-GD
Date : 7/27/2007

Peer Review
Checked by :
Date :

NOTES:
1. Existing condition: Pump Station 180 ft from Wall Centerline
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	765.00
500 year + 0':	764.70
500 year + 3':	767.70
500 year + 5':	769.70
Ground Surface Land Side:	750 (Subtract 5' for fill in blanket thickness, but not for H)
Ground Surface River Side:	740
Bottom of Blanket:	720
Top of Bedrock:	670
Ground Surface at Pump Station:	750

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

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Input Parameters

Station	Design Condition	Permeability Ratio: River Side (K_f/K_{br})	Permeability Ratio: Land Side (K_f/K_{bl})	Blanket Thickness (ft): River Side z_{br}	Blanket Thickness (ft): Protection z_{bo}	Blanket Thickness (ft): Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Protection L_2	Seepage Length (ft): Land Side L_3	Driving Head (ft) H
106+49	Existing Condition	300	300	20.0	22.5	25.0	50.0	0	30	1000	15.0
106+49	n500 + 0	300	300	20.0	22.5	25.0	50.0	0	30	1000	14.7
106+49	n500 +3	300	300	20.0	22.5	25.0	50.0	0	30	1000	17.7
106+49	n500 + 5	300	300	20.0	22.5	25.0	50.0	0	30	1000	19.7

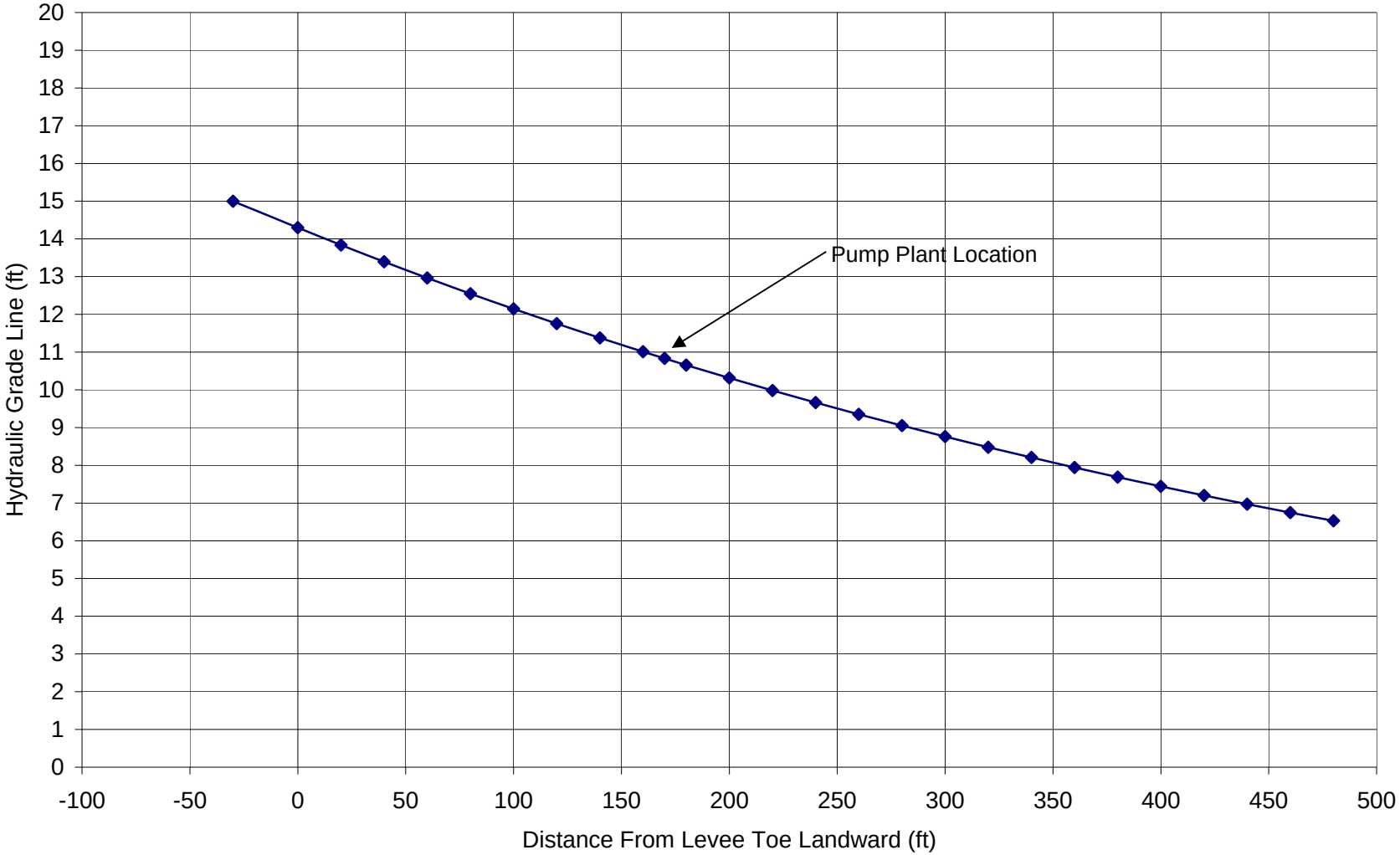
Analysis

Factors: c_r	Factors: c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
0.001826	0.001633	0	612	14.3	0.57	0.84	1.47	
0.001826	0.001633	0	612	14.0	0.56	0.84	1.50	
0.001826	0.001633	0	612	16.9	0.67	0.84	1.25	
0.001826	0.001633	0	612	18.8	0.75	0.84	1.12	

Hydraulic Grade Line for the Kemper Arena Pump Plant
Existing Conditions

Elevation Top of Protection (MSL)	765.0	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	15		
c_l	0.001633		
x_1 (ft)	0		
L_2 (ft)	30		
x_3 (ft)	612		
h_o	14.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-30	15.00	765.00
	0	14.30	764.30
	20	13.84	763.84
	40	13.40	763.40
	60	12.96	762.96
	80	12.55	762.55
	100	12.15	762.15
	120	11.75	761.75
	140	11.38	761.38
	160	11.01	761.01
Pump Plant Location	170	10.83	760.83
	180	10.66	760.66
	200	10.32	760.32
	220	9.98	759.98
	240	9.66	759.66
	260	9.35	759.35
	280	9.05	759.05
	300	8.76	758.76
	320	8.48	758.48
	340	8.21	758.21
	360	7.94	757.94
	380	7.69	757.69
	400	7.44	757.44
	420	7.20	757.20
	440	6.97	756.97
	460	6.75	756.75
	480	6.53	756.53

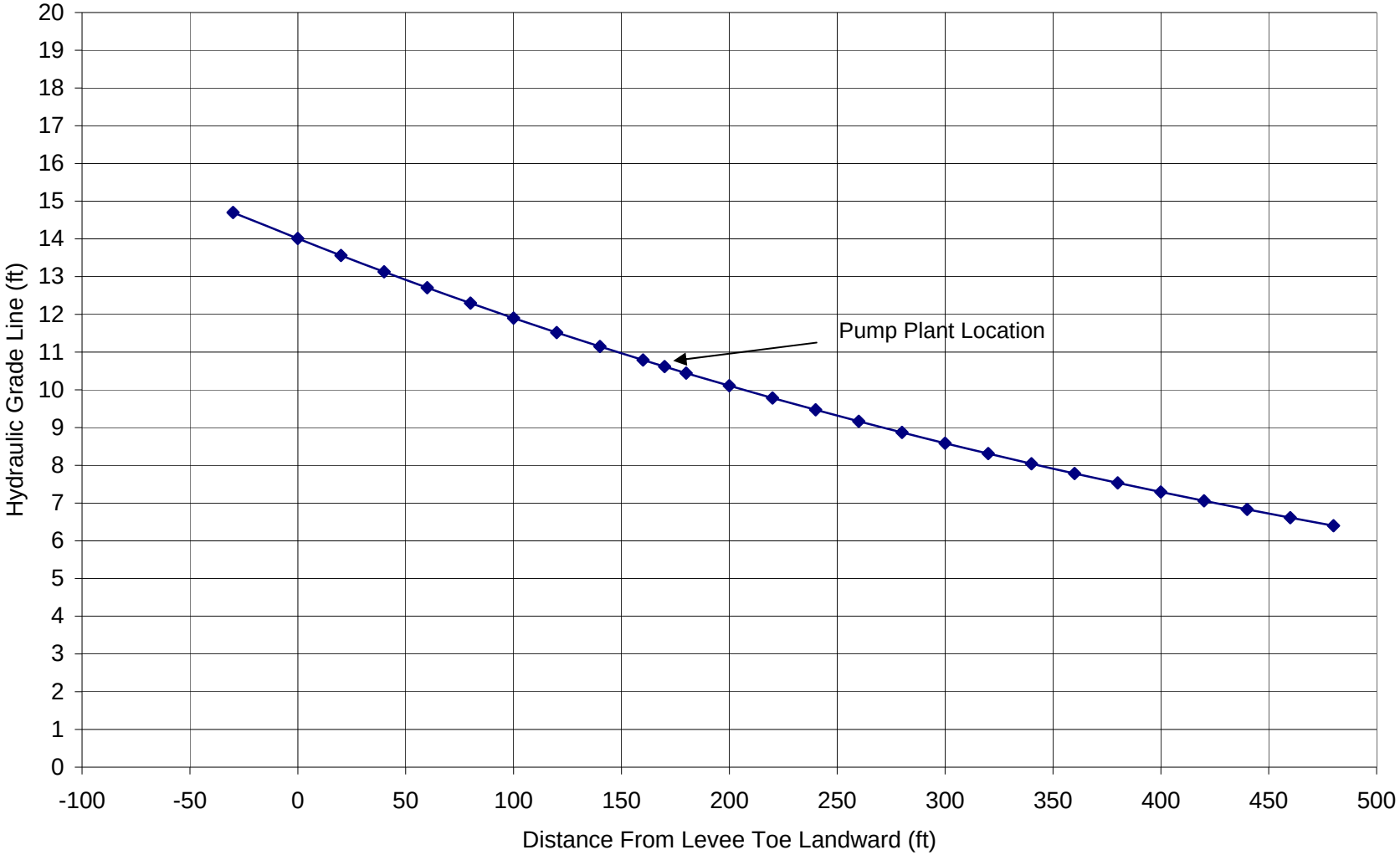
Existing Conditions Hydraulic Grade Line for the
Kemper Arena Pump Plant Sta 106+49
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Kemper Arena Pump Plant
n500+0 Loading Conditions

Elevation Top of Protection (MSL)	764.70	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	14.7		
c_l	0.001633		
x_1 (ft)	0		
L_2 (ft)	30		
x_3 (ft)	612		
h_o	14.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-30	14.70	764.70
	0	14.01	764.01
	20	13.56	763.56
	40	13.13	763.13
	60	12.71	762.71
	80	12.30	762.30
	100	11.90	761.90
	120	11.52	761.52
	140	11.15	761.15
	160	10.79	760.79
Pump Plant Location	170	10.62	760.62
	180	10.44	760.44
	200	10.11	760.11
	220	9.78	759.78
	240	9.47	759.47
	260	9.17	759.17
	280	8.87	758.87
	300	8.59	758.59
	320	8.31	758.31
	340	8.04	758.04
	360	7.78	757.78
	380	7.53	757.53
	400	7.29	757.29
	420	7.06	757.06
	440	6.83	756.83
	460	6.61	756.61
	480	6.40	756.40

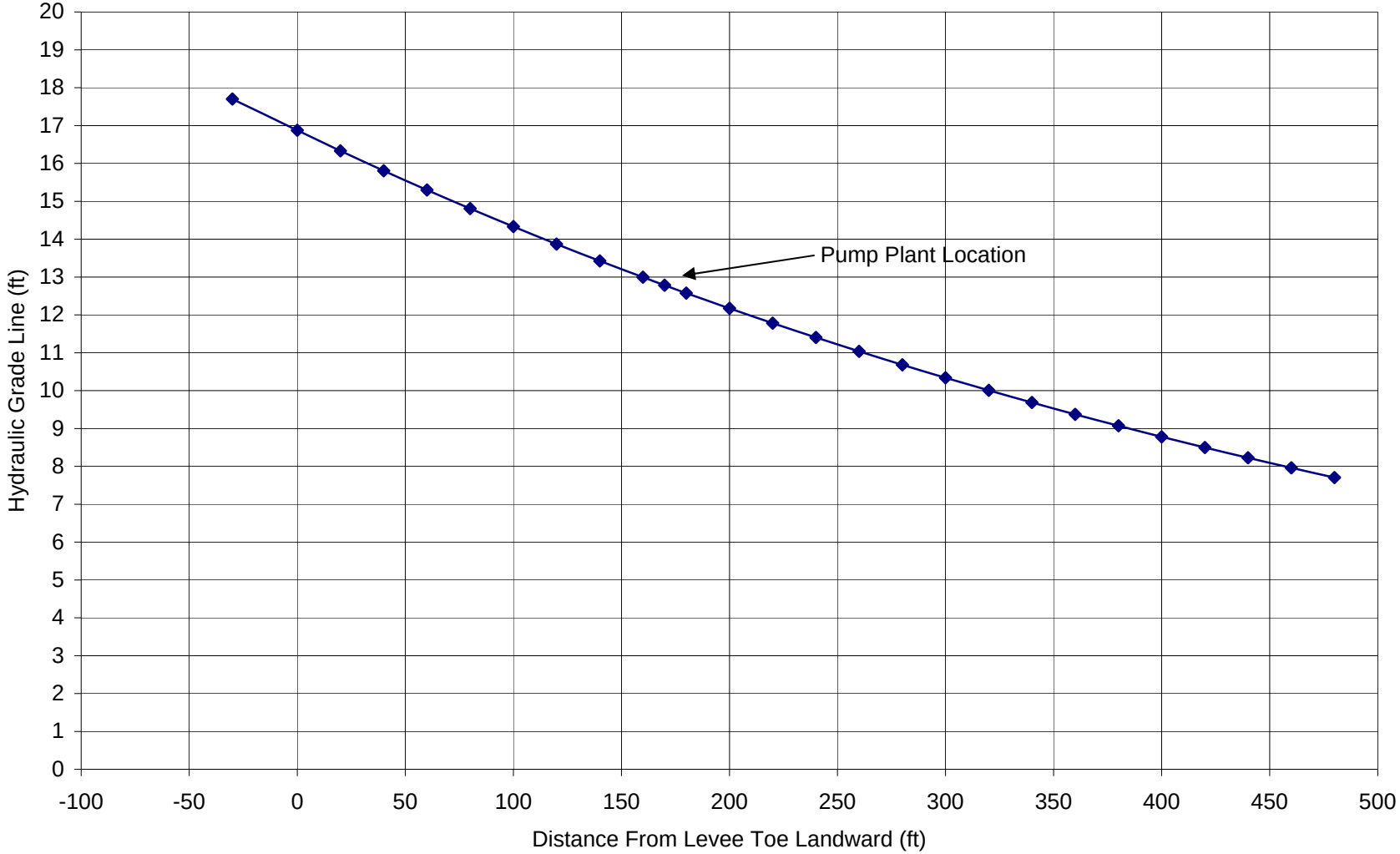
n500+0 Loading Conditions Hydraulic Grade Line for the
Kemper Arena Pump Plant Sta 106+49
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Kemper Arena Pump Plant
n500+3 Loading Conditions

Elevation Top of Protection (MSL)	767.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	17.7		
c_l	0.001633		
x_1 (ft)	0		
L_2 (ft)	30		
x_3 (ft)	612		
h_o	16.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-30	17.70	767.70
	0	16.87	766.87
	20	16.33	766.33
	40	15.81	765.81
	60	15.30	765.30
	80	14.81	764.81
	100	14.33	764.33
	120	13.87	763.87
	140	13.42	763.42
	160	12.99	762.99
Pump Plant Location	170	12.78	762.78
	180	12.58	762.58
	200	12.17	762.17
	220	11.78	761.78
	240	11.40	761.40
	260	11.04	761.04
	280	10.68	760.68
	300	10.34	760.34
	320	10.01	760.01
	340	9.68	759.68
	360	9.37	759.37
	380	9.07	759.07
	400	8.78	758.78
	420	8.50	758.50
	440	8.23	758.23
	460	7.96	757.96
	480	7.71	757.71

n500+3 Loading Conditions Hydraulic Grade Line for the
Kemper Arena Pump Plant Sta 106+49
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Kemper Arena Pump Plant
n500+5 Loading Conditions

Elevation Top of Protection (MSL)	769.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	19.7		
c_l	0.001633		
x_1 (ft)	0		
L_2 (ft)	30		
x_3 (ft)	612		
h_o	18.8		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-30	19.70	769.70
	0	18.78	768.78
	20	18.18	768.18
	40	17.59	767.59
	50	17.31	767.31
	60	17.03	767.03
	80	16.48	766.48
	100	15.95	765.95
	120	15.44	765.44
	140	14.94	764.94
	160	14.46	764.46
Pump Plant Location	170	14.23	764.23
	180	14.00	764.00
	200	13.55	763.55
	220	13.11	763.11
	240	12.69	762.69
	260	12.28	762.28
	280	11.89	761.89
	300	11.51	761.51
	320	11.14	761.14
	340	10.78	760.78
	360	10.43	760.43
	380	10.10	760.10
	400	9.77	759.77
	420	9.46	759.46
	440	9.15	759.15
	460	8.86	758.86
	480	8.58	758.58

n500+5 Loading Conditions Hydraulic Grade Line for the
Kemper Arena Pump Plant Sta 106+49
Central Industrial District Kansas Levee Unit

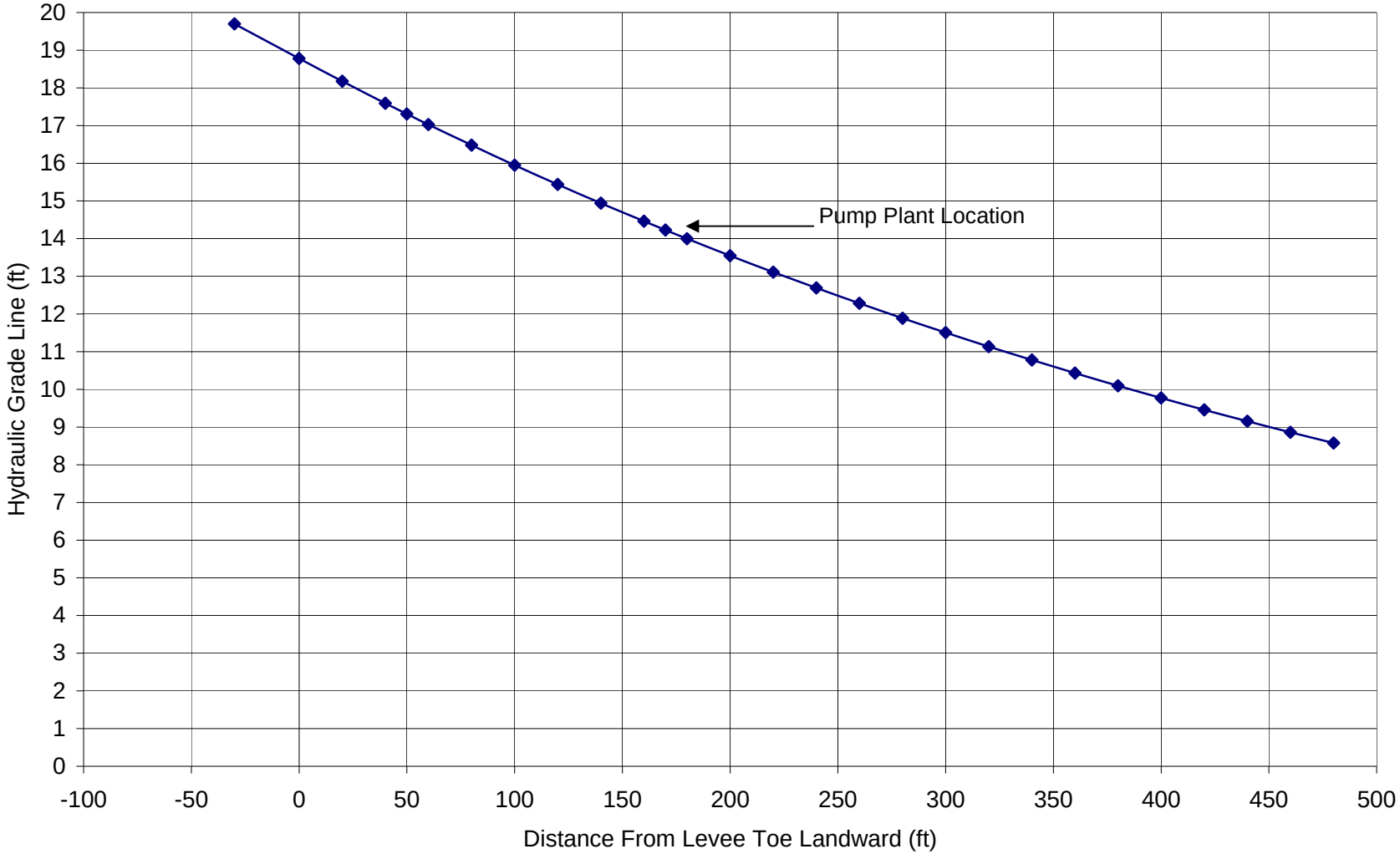


EXHIBIT 3 – STRUCTURAL ANALYSIS

CID Pump Stations

Kansas City's Levees Phase II Feasibility Study

Pump Station Structural Analysis

(EC-DS)

CENWK-EC-DS (1110-2-1150a) 13 May 2008
(PARKS/3253/ad)
Checked by EHF 5/13/08

MEMORANDUM FOR Ron Jansen, EC-GC

SUBJECT: Deterministic Method of Obtaining Pump Station Strength Reliability for a Given Set of Pump Station Factors of Safety and Geotechnical Parameters, KCL CID KVDD Pump Station Feasibility Study.

1. BACKGROUND: On April 29, 2008 a meeting was held with Lamar McKissack, Ron Jansen, Randy Kuzniakowski and Marvin Parks. The purpose of the meeting was to determine a course of action given the hydraulic grade line (HGL) elevation change posted by EC-GD on April 25, 2008. The geotechnical input parameters are used in-part to determine a pump station's reliability for a given event. Upon conclusion of the meeting it was decided to use the factors of safety (FS) acquired from previously published HGL values and make a deterministic assessment as to how recently published or future published HGL values will impact the station's reliability. This MFR outlines the methodology used and the information obtained by the described deterministic method.

2. Bottom of Blanket Elevation (BOB). The change in lateral wall loading W was calculated for a given HGL for two different blanket thicknesses for the New Central, Gateway and Mistletoe pump stations. The calculation results are shown in Table 1. For the pump stations evaluated the effect of varying the blanket thickness 3 feet or less was minimal.

Table 1. Comparison of Wall Loading for Various Values of BOB and HGL values.

HGL	BOB,ft	W, (plf)	Pump Station	Comments
761.94	716	3005	Gateway	The effect on wall loading is minimal when the BOB is decreased 3 feet
761.94	713	3007		
757.9	716	2829		
757.9	713	2841		
760	722	2159	Mistletoe	The effect on wall loading is minimal when the BOB is increased 3 feet.
760	725	2173		
760	725	2143		
760.3	722	2755	New Central	The effect on wall loading is minimal when the BOB is increased 2 feet.
760.3	724	2786		

Highlighted BOBvalues are the current values as of April 24, 2008.

3. HGL Values. The HGL and BOB values used in the pump station analysis are posted in Table 2.

Table 2. HGL and BOB values.

Pump Station	Sta.	EC HGL / BOB	FC HGL/BOB	Status
Ohio	83+52	757.95 / 725	757.95 / 725	Calculated by direct analysis
		SAME	SAME	The HGL values did not change for Ohio
Mistletoe	37+06.5	759.9 / 725	759.3 / 722	Calculated by direct analysis
		759.8 / 722	761.1 / 722	Revised as of 4/25/08. Not calculated by direct analysis.
New Central	58+12	760.0 / 724	760.3 / 722	Calculated by direct analysis
		760.8 / 722	762.4 / 722	Revised as of 4/25/08. Not calculated by direct analysis.
Stockyards #3	74+21	757.7 / 720	760.53 / 722	FC not calculated
		760 / 722	762.3 / 722	Revised as of 4/25/08. Not calculated by direct analysis.
Gateway	80+90	757.9 / 713	761.94 / 716	Calculated by direct analysis
		760.5 / 716	763.6 / 716	Revised as of 4/25/08. Not calculated by direct analysis.

4. EC Strength Deterministic Analysis. The percent increase in wall loading for the EC using the revised calculations (04/25/08) are posted in Table 3. The percent change in wall loading W was subtracted from the direct calculated value of FS to obtain the deterministic FS value for EC. The lowest (controlling) strength FS for the particular pump station was used.

Table 3. Change in Wall Loading for the EC event and Deterministic FS Strength Values.

Pump Station	Sta.	EC HGL and W (plf)		W Increase (%)	FS	
Mistletoe	37+06.5	759.9	2143	0	1.61	Station considered reliable
		759.8	2143		1.61	
New Central	58+12	760.0	2769	0.4	1.56	Station considered reliable for n500+3 event using revised HGL based on deterministic analysis.
		760.8	2781		1.55	
Stockyards #3	74+21	757.7				Station borderline. On hold pending further instruction.
		760				
Gateway	80+90	757.9	2841	<5.46	1.61	Station considered reliable based on older FC HGL value that is higher than the revised HGL value
		760.5	<3005		>1.5	

Highlighted values are the revised HGL values as of 4/25/08

5. FC Strength Deterministic Analysis. The percent increase in wall loading for the FC using the revised calculations (04/25/08) are posted in Table 4. The percent change in wall loading W was subtracted from the direct calculated value of FS to obtain the deterministic FS value for FC. The lowest (controlling) strength FS for the particular pump station was used.

Table 4. Change in Wall Loading for the FC event and Deterministic FS Strength Values.

Pump Station	Sta.	FC HGL and W (plf)		W increase (%)	FS	
Mistletoe	37+06.5	760.0	2143	2.55	1.61	Station considered reliable for n500+3 event based on deterministic analysis
		761.1	2199		1.56	
New Central	58+12	760.3	2755	3.81	1.56	Station considered reliable for n500+3 event using revised HGL based on deterministic analysis.
		762.4	2864		1.50	
Stockyards #3	74+21	760.53				Station not considered reliable. On hold pending further instruction.
		762.3				
Gateway	80+90	761.94	3005	2.37	1.87	Station considered reliable for n500+3 event using revised HGL based on deterministic analysis.
		763.6	3078		1.82	

Highlighted values are the revised HGL values as of 4/25/08

Fernandez, Eduardo H NWK

From: Kuzniakowski, Randall S NWK
Sent: Sunday, April 20, 2008 8:04 AM
To: Fernandez, Eduardo H NWK
Subject: RE: Mistletoe PS, New Central, Gateway

We really haven't thought about this much, Glen is still trying to finish existing conditions underseepage stuff, but here is my best guess.....

No fill will be placed on the riverside at these locations.

Randy

-----Original Message-----
From: Fernandez, Eduardo H NWK
Sent: Friday, April 18, 2008 2:28 PM
To: Kuzniakowski, Randall S NWK
Cc: Parks, Marvin L NWK
Subject: Mistletoe PS, New Central, Gateway

Randy,

Do you envision putting any fill above Mistletoe, New Central, Gateway outlet works?

Variables

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

$b := 1ft$

REFERENCES
1. Drawings, dated 1948 Operation and Maintenance Manual V2
2. HGL, blanket elevations, levee elevations supplied EC-G

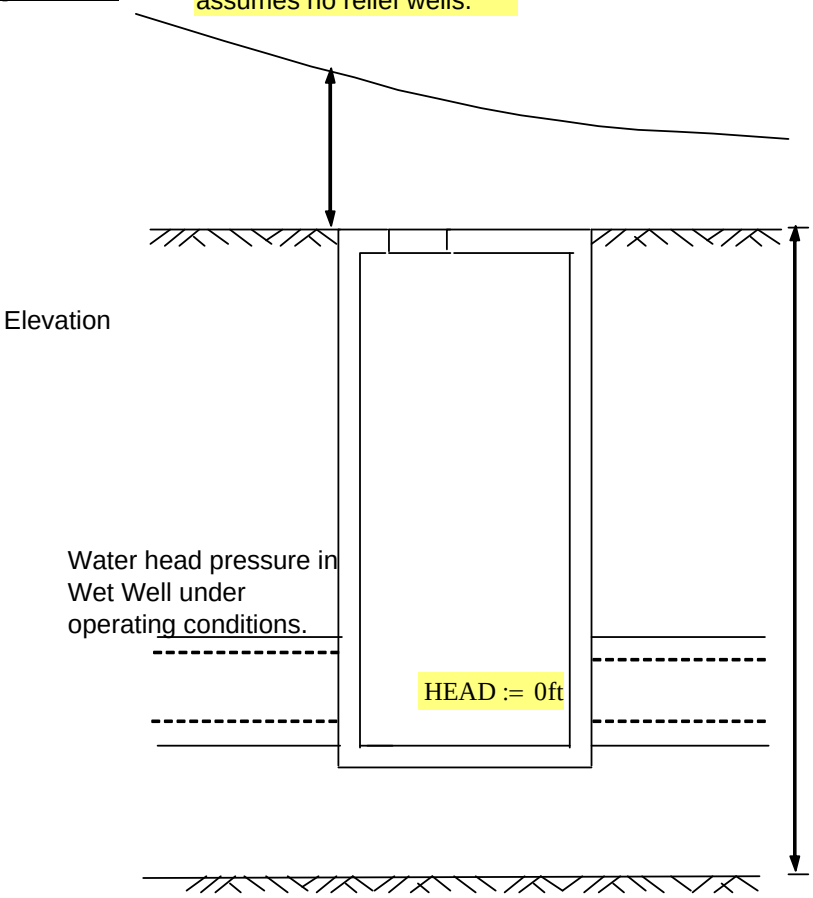
Top of existing levee El = 760.6
Bottom of blanket El = 725
Ground El = 754

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 = 760.6 ft



$ELEV_1 := 754ft$

$HGL := 757.95ft - ELEV_1$

$HGL = 3.95\,ft$

$BLANKET := 754ft - 725ft$

$BLANKET = 29\,ft$

$ELEV_2 := 731.58ft$

$H := ELEV_1 - ELEV_2$

$H = 22.42\,ft$

Structural drawings dated 1943 and 1947were obtained. This structure, A.K.A KCK Flood Pump Station 1, is designed for a pump capacity of 32,650gpm. Originally constructed in 1945, a new 17,450gpm was added in 1950. This pump station receives flow from seepage, storm sewer flow and runoff. It has a 42" RCP outlet pipe that discharges into the river.

Since structure is relatively symetrical and no significant variances in wall dimensions are apparent, north and south wall can be treated as identical as well as the east and west walls. This will allow for an efficient means to determine strength/stability.

Wall 1 Parameters

South wetwall wall

$L_1 := 11.25\,ft$

$H = 22.42\ ft$

$D_1 := 15in$

$C_{c1} := 2in$

Wall 2 Parameters

West wetwall wall

$L_2 := 14.25\,ft$

$H = 22.42\ ft$

$D_2 := 15in$

$C_{c2} := 2in$

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 shown on sectional plan pg A-10-1359

5's @ 12" O.C. E.W. E.F.

Assume 3" Clear Cover on slab

Floor Thickness $D_f := 18in$ Clear Cover $C_{cf} := 3in$ $d_f := D_f - C_{cf} - \frac{.31in}{2}$ $d_f = 14.845 \cdot in$

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - \left(C_{c1} + \frac{.5}{2}in\right)$

$d_1 = 12.75 \cdot in$

WALL 2

$d_2 := D_2 - \left[C_{c2} + \left(\frac{.5}{2}in\right)\right]$

$d_2 = 12.75 \cdot in$

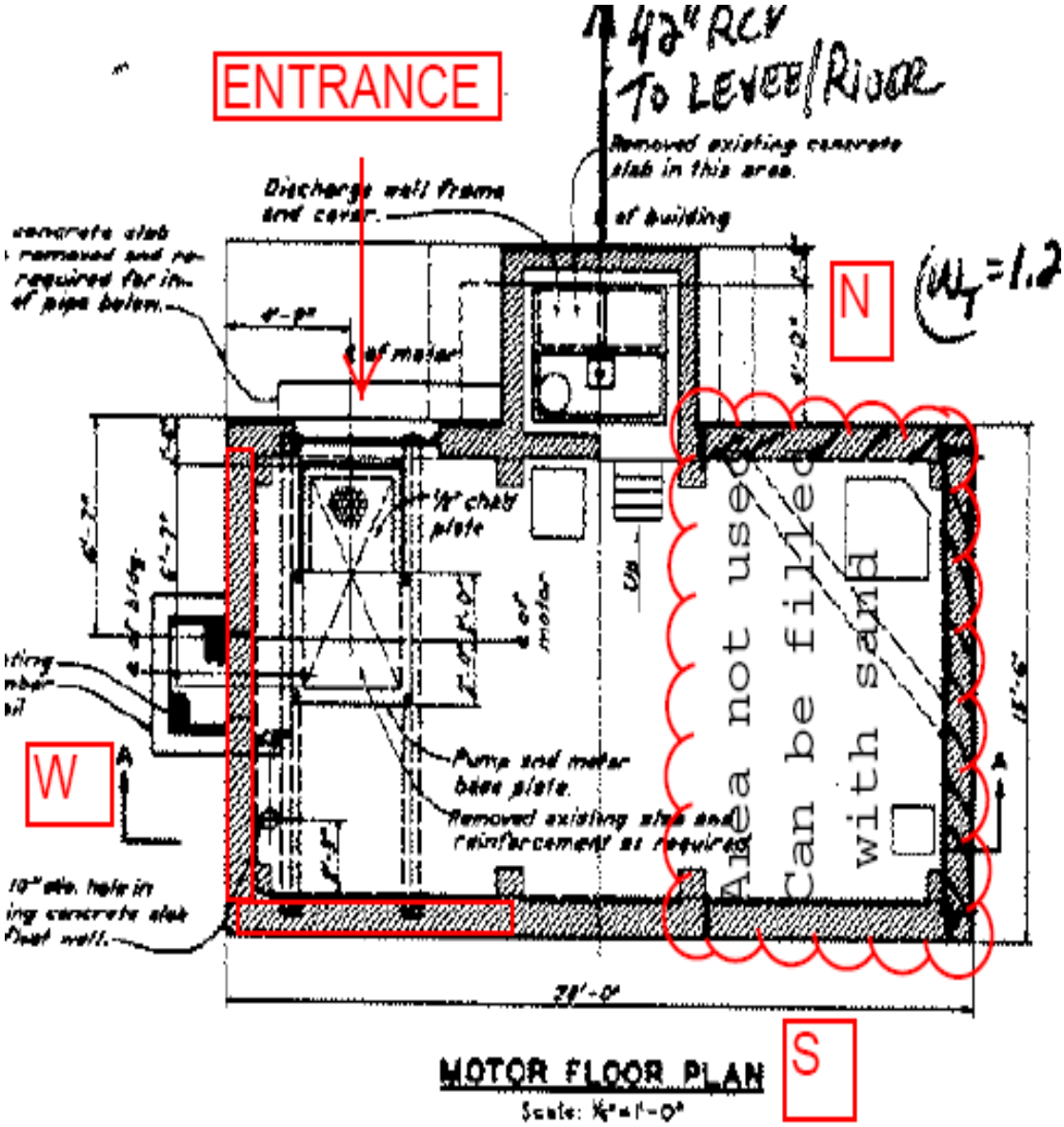


Figure 1. Ohio Pump Station 83+52 Plan View Drawing Sheet 5 USACE O&M CID 1948

Assumptions and Criteria

- Steel and Concrete strengths were not stated on the design drawings. For this analysis, assume f'c=3000 psf and fy=40,000 psf given the period the station was constructed.

Soil Properties

$\phi := 21deg$

$\gamma := 110pcf$

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

$K_o = 0.642$

Water Unit Weight

$\gamma_w := 62.4pcf$

Steel Properties

$F_y := 40ksi$

Concrete Properties

$f'_c := 3.00ksi$

Concrete Unit Weight

$\gamma_c := 150pcf$

Steel Modulus of Elasticity: $E_s := 29000ksi$

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)

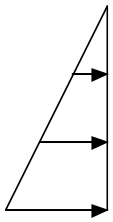
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALL 1, (South wall) In place wall $K_o = 0.642$ $BLANKET = 29\,ft$ $HGL = 3.95\,ft$

Wall Loading

Soil or Soil & Water

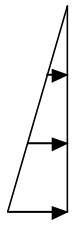


$H = 22.42\,ft$

$H_2 := H$

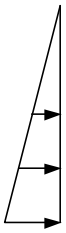
$H_2 = 22.42\,ft$

OR



$H_2 = 22.42\,ft$

&



$H'_1 := \frac{BLANKET + HGL}{BLANKET} \cdot H - HEAD$

$H''_1 := H + HGL - HEAD$

$H_1 := if(H_2 > BLANKET, H''_1, H'_1)$

$H_1 = 25.474\,ft$

$H'_1 = 25.474\,ft$

$H''_1 = 26.37\,ft$

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

$W_s = 1582.4 \cdot psf$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_1]$

$W_w = 2274 \cdot psf$

$W_{1ext} := if(W_s > W_w, W_s, W_w)$ $W_{1ext} = 2274.307 \cdot psf$

SHEAR

WALL 1, South Wall

$L_{tw} := 7.13ft$

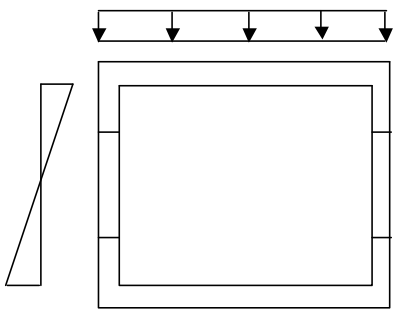
Ltw stands for length of thrust wall

$$THRUST_1 := \frac{5 \cdot W_{1ext} \cdot L_{tw}}{8}$$

$$V'_{u1} := W_{1ext} \cdot \frac{L_1}{2}$$

$$V'_{u1} = 12793 \frac{lb}{ft}$$

$$THRUST_1 = 10134.881 \frac{lb}{ft}$$



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

$$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$$

$$V_{u1} = 8955 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$A_g := D_1 \cdot 12 \frac{in}{ft} \qquad A_g = 180 \cdot \frac{in^2}{ft}$$

$$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$$ ACI EQ (11-4)

$$\phi V_{n1} = 17232 \frac{lb}{ft}$$

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

Check1 = "OKAY"

Factor of Safety

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

$$FS_{S1} = 1.924$$

ANALYZE WALL 1 (FOR BENDING AS A "STRIP" 12 INCHES WIDE)

Assume that the horizontal reinforcement carries all of the lateral loads

$b := 12\,\frac{in}{ft}$

$L_1 = 11.25\,ft$

The mid-span negative moment $M_{mid} := \frac{W_{1ext} \cdot L_1^2}{24}$ $M_{mid} = 11.993 \cdot \frac{kip \cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{W_{1ext} \cdot L_1^2}{12}$ $M_{end} = 23.987 \cdot \frac{kip \cdot ft}{ft}$

#4's @ 12in. inside face (mid-plate moment) $A_{mph} := .20\,\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mph} = 0.261 \cdot in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_1 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 8.4 \cdot \frac{kip \cdot ft}{ft}$

5 @ 12in bars outside face (edge-plate moment) $A_{eph} := .31\,\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{eph} = 0.405 \cdot in$

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 13.0 \cdot \frac{kip \cdot ft}{ft}$

$CheckMPH1 := if\left(\phi M_{mph} > 1.5 \cdot M_{mid}, "OKAY", "NO\ GOOD"\right)$ CheckMPH1 = "NO GOOD"

Factor of Safety $FS_{MPH1} := \frac{\phi M_{mph}}{M_{mid}}$ $FS_{MPH1} = 0.701$

$CheckEPH1 := if\left(\phi M_{eph} > 1.5 \cdot M_{end}, "OKAY", "NO\ GOOD"\right)$ CheckEPH1 = "NO GOOD"

Factor of Safety $FS_{EPH1} := \frac{\phi M_{eph}}{M_{end}}$ $FS_{EPH1} = 0.541$

Loads from initial screening calculation: $M_u := 24 kip \cdot ft$

$Thrust_2 := 4.75 kip$ "Half of the thrust coming from exterior wall (9500/2) per M.P. 2/14/08"

$F_y = 40 \cdot ksi \qquad f'_c = 3000 \cdot psi$

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, $D := 15in$ $\qquad b := 1ft$ (1 foot strip)

$A_1 := .20in^2 \qquad A_2 := .31in^2$

$d_1 := 2.25in \qquad d_2 := 12.8in$ Distance from compression face to bar centerline

$$\beta 1 := if\left[f'_c \le 4ksi, 0.85, if\left[\left(f'_c\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_c}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1..2$

Given $\qquad c := .2\cdot D_1 \qquad c = 3\cdot in \qquad Thrust_2 = 4.75\cdot kip$

$$Thrust_2 = \sum_{i=1}^{2}\left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_y \\ + -if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \\ F_y - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_y \\ -F_y - if\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & if\ \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_y \end{array}\right]\cdot A_i\right] \ldots$$
$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := Find(c) \qquad c = 0.659\cdot in \qquad a := if(\beta 1\cdot c > D, D, \beta 1\cdot c) \qquad a = 0.56\cdot in$

Find Moment:

$$\varepsilon s_i := \left(c - d_i\right)\cdot\frac{0.003}{c} \qquad fs_i := if\left(E_s\cdot\varepsilon s_i > F_y, F_y, if\left(E_s\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, E_s\cdot\varepsilon s_i\right)\right)$$

$$Fs_i := fs_i\cdot A_i - if\left(\beta 1\cdot c > d_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.188 ft	-0.007	-40 ·ksi	0 ·in²	0 ·kip	0 ·kip·in
1.067	-0.055	-40	0.31	-12.4	65.72

$$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 66 \cdot kip \cdot in$$

$$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right) \qquad Mc = 124 \cdot kip \cdot in$$

$$M_n := (Mc + Mst) \qquad M_n = 15.79 \cdot kip \cdot ft$$

$$FS_{EPH1} := \frac{M_n}{M_u} \qquad FS_{EPH1} = 0.658$$

$$Check := if\left(FS_{EPH1} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check = \text{"NO GOOD"}$$

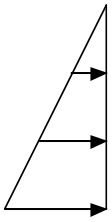
"POF Analysis on seperate MathCad sheet"

WALL 2, West Wall

$K_o = 0.642$ BLANKET = 29 ft HGL = 3.95 ft

Wall Loading

Soil or Soil & Water

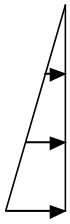


$H = 22.42$ ft

$H_1 := H$

$H_1 = 22.42$ ft

OR



$H_1 = 22.42$ ft

&

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

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

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

$H_3 = 25.474$ ft

$H'_3 = 25.474$ ft

$H''_3 = 26.37$ ft

$$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_1$$

$$W_s = 1582 \frac{lb}{ft^2}$$

$$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma - \gamma_w) \cdot H_1 + \gamma_w \cdot H_3\right]$$

$$W_w = 2274 \frac{lb}{ft^2}$$

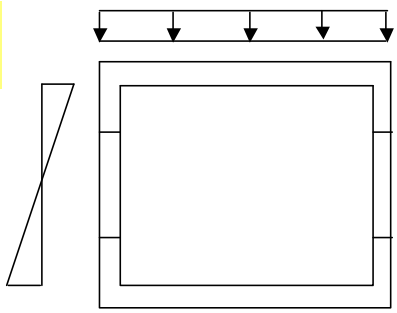
$$W_2 := if\left(W_s > W_w, W_s, W_w\right) \qquad W_2 = 2274 \frac{lb}{ft^2}$$

$L_{ntw} := 5.75ft$

$THRUST_2 := \frac{W_2 \cdot L_{ntw}}{2}$ $THRUST_2 = 6539 \frac{lb}{ft}$

$V'_u := W_2 \cdot \frac{L_{ntw}}{2}$

$V'_u = 6539 \frac{lb}{ft}$



$V_u := 2 \frac{V'_u}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2 \right)$ $V_u = 4990 \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$D_2 = 15 \cdot in$

$A_g := D_2 \cdot 12 \frac{in}{ft}$ $A_g = 180 \cdot \frac{in^2}{ft}$

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

$\phi V_n := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_2 \right) \right]$ ACI EQ (11-4)

$d_2 = 12.75 \cdot in$

$\frac{THRUST_2 \cdot \frac{in^2}{lb}}{A_g} = 36.326$

$\phi V_n = 17065 \frac{lb}{ft}$

$Check2 := if\left(\phi V_n > 1.5 V_u, "OKAY", "NO GOOD"\right)$

Check2 = "OKAY"

Factor of
Safety

$FS_{S2} := \frac{\phi V_n}{V_u}$

$FS_{S2} = 3.42$

ANALYZE WALL 2 (West Wall) FOR BENDING AS A "STRIP" 12 INCHES WIDE

Assume that the horizontal reinforcement carries <u>all</u> of the lateral loads

$b := 12 \frac{in}{ft}$

The mid-span negative moment

$M_{mid2} := \frac{W_2 \cdot L_2^2}{24}$

$M_{mid2} = 19.243 \cdot \frac{kip \cdot ft}{ft}$

The end-span positivie moment

$M_{end} := \frac{W_2 \cdot L_2^2}{12}$

$M_{end} = 38.486 \cdot \frac{kip \cdot ft}{ft}$

4's @ 12in. inside face
(mid-plate moment)

$A_{mph} := .20 \frac{in^2}{ft}$

$a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$

$a_{mph} = 0.261 \cdot in$

$$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_2 - \frac{a_{mph}}{2}\right)$$

$$\phi M_{mph} = 8.4 \cdot \frac{kip \cdot ft}{ft}$$

5's at 12in.
(edge-plate moment)

$$A_{eph} := .31 \frac{in^2}{ft}$$

$$a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$$

$$a_{eph} = 0.405 \cdot in$$

$$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_2 - \frac{a_{eph}}{2}\right)$$

$$\phi M_{eph} = 13.0 \cdot \frac{kip \cdot ft}{ft}$$

$$CheckMPH := if\left(\phi M_{mph} > 1.5 \cdot M_{mid2}, "OKAY", "NO\ GOOD"\right)$$

CheckMPH = "NO GOOD"

Factor of Safety

$$FS_{MPH2} := \frac{\phi M_{mph}}{M_{mid2}}$$

$$FS_{MPH2} = 0.437$$

$$CheckEPH := if\left(\phi M_{eph} > 1.5 \cdot M_{end}, "OKAY", "NO\ GOOD"\right)$$

CheckEPH = "NO GOOD"

Factor of Safety

$$FS_{EPH2} := \frac{\phi M_{eph}}{M_{end}}$$

$$FS_{EPH2} = 0.337$$

$$THRUST := \frac{W_2 \cdot L_2}{2}$$

$$THRUST = 16204.438 \frac{lb}{ft}$$

$$M_{u2} := 38 \cdot kip \cdot ft$$

$$Thrust_2 := 6.5 \cdot kip$$

$$F_y = 40 \cdot ksi \qquad f'_c = 3000 \cdot psi$$

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, $D := 15in$ $\qquad b := 1ft$ (1 foot strip)

$$A_4 := .20in^2 \qquad A_2 := .31in^2$$

$$d_1 := 2.25in \qquad d_2 := 12.8in$$

Distance from compression face to bar centerline

$$\beta 1 := \text{if}\left[f'_c \le 4ksi, 0.85, \text{if}\left[\left(f'_c\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_c}{ksi} - 4\right)\right]\right]$$

$\beta 1 = 0.85$

Assume c=0.2*D

$i := 1..2$

Given $\quad c := .2\cdot D_1 \quad c = 3\cdot in \quad Thrust_2 = 6.5\cdot kip$

$$Thrust_2 = \sum_{i=1}^{2}\left[\left[\begin{array}{l} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots \quad \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_y \\ + -\text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \\ F_y - \text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_y \\ -F_y - \text{if}\left(d_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_y \end{array}\right]\cdot A_i\right] \ldots$$

$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c) \quad c = 0.727\cdot in \quad a := \text{if}(\beta 1\cdot c > D, D, \beta 1\cdot c) \quad a = 0.618\cdot in$

Find Moment:

$$\varepsilon s_i := \left(c - d_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, E_s\cdot\varepsilon s_i\right)\right)$$

$$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > d_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.188 ft	-0.006	-40 ·ksi	0 ·in²	0 ·kip	0 ·kip·in
1.067	-0.05	-40	0.31	-12.4	65.72

$$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 66\cdot kip\cdot in$$

$$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right) \qquad Mc = 136\cdot kip\cdot in$$

$$M_n := (Mc + Mst) \qquad M_n = 16.8\cdot kip\cdot ft$$

$$FS_{EPH2a} := \frac{M_n}{M_{u2}} \qquad FS_{EPH2a} = 0.442$$

$$Check := \text{if}\left(FS_{EPH2a} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check = \text{"NO GOOD"}$$

Uplift on Structure Refer to Figure 2 for the identified areas concerning uplift calculations

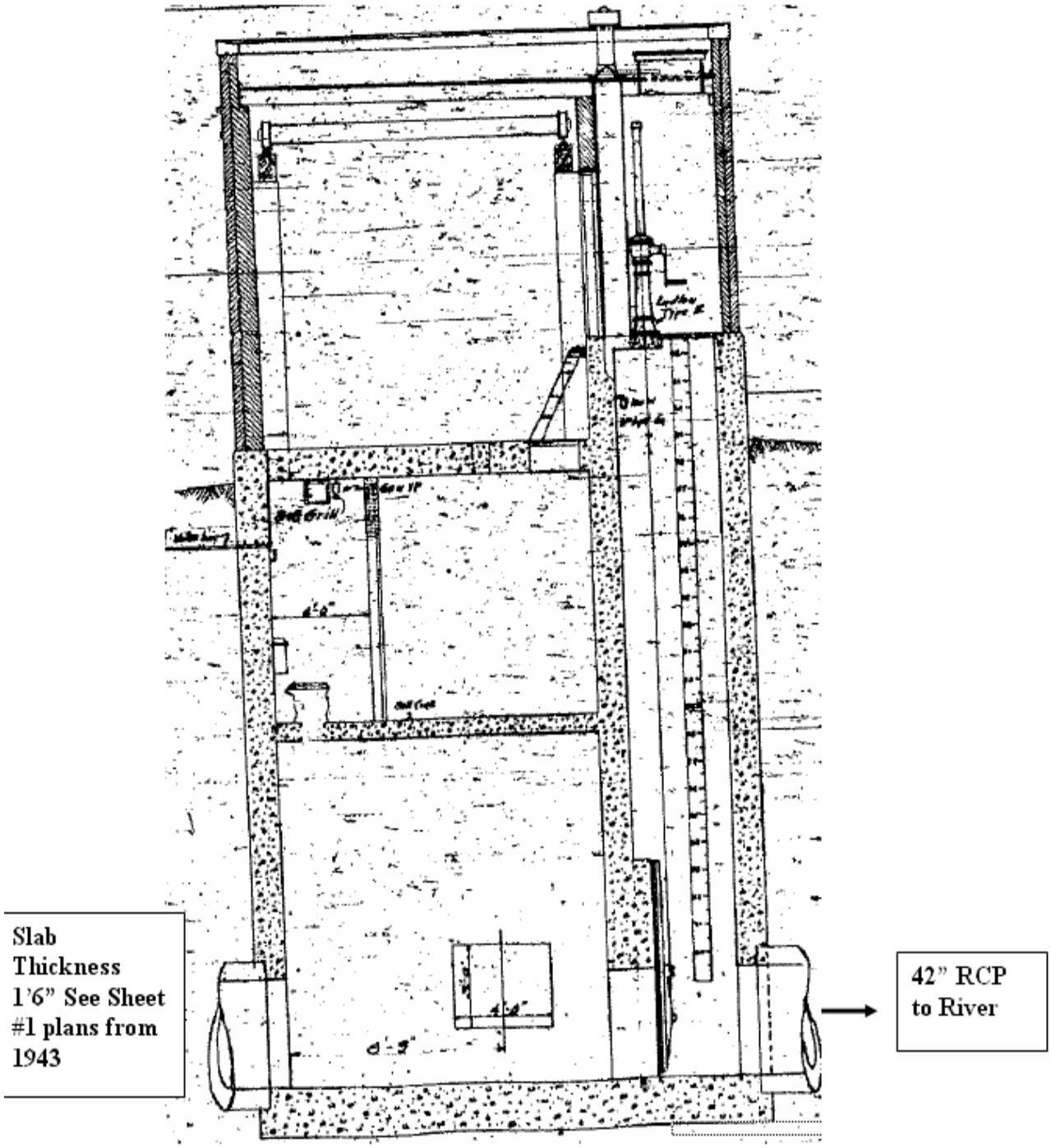


Figure 2. Ohio Ave. PumpStation Section cut facing West.

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.
3. Concrete wall above grade within the wetwell area.
4. Wetwell base slab

Uplift on Structure

Weight of wetwell.The equipment weight was not included.

Top Slab

$Top := 8in \cdot (13.5ft) \cdot (11.5ft)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 15.525 \cdot kip$

Walls

$West := 19.42ft \cdot 13ft \cdot 1.25ft$ $\quad Wwest := West \cdot 150pcf$ $\quad Wwest = 47.336 \cdot kip$

$South := 19.42ft \cdot 11.5ft \cdot 1.25ft$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 41.874 \cdot kip$

$East := West$ $\quad Weast := Wwest$ $\quad Weast = 47.336 \cdot kip$

$North := South$ $\quad Wnorth := North \cdot 150pcf$ $\quad Wnorth = 41.874 \cdot kip$

Base Slab

$Base := 1.5 \cdot ft \cdot (13ft) \cdot (11.5ft)$

$Base = 224.25 \cdot ft^3$ $\quad Wbase := Base \cdot 150pcf$ $\quad Wbase = 33.638 \cdot kip$

$Basearea := (13ft) \cdot (11.5ft)$ $\quad Basearea = 149.5\ ft^2$

South Wall above grade

$hsouth_{ag} := 14.5ft$

$South_{ag} := hsouth_{ag} \cdot 8in \cdot (11.5ft)$ $\quad Wsouth_{ag} := South_{ag} \cdot 150pcf$ $\quad Wsouth_{ag} = 16.675 \cdot kip$

$Wnorth_{ag} := Wsouth_{ag}$

West Wall above Grade

$hwest_{ag} := 14.5ft$

$West_{ag} := hwest_{ag} \cdot (13ft) \cdot 15in$ $\quad Wwest_{ag} := West_{ag} \cdot 150pcf$ $\quad Wwest_{ag} = 35.344 \cdot kip$

$Weast_{ag} := Wwest_{ag}$

Gatewell Above Grade "Gatewell does not sit above wetwell"

$Wgatewell := 0ft \cdot 0ft \cdot 0ft$ $\quad Wgatewell := 0 \cdot 150pcf$ $\quad Wgatewell = 0 \cdot kip$

Wetwell Weight

$$Wwetwell := Wtop + Wwest + Wsouth + Weast + Wnorth + Wbase \ldots + Wsouth_{ag} + Wwest_{ag} + Weast_{ag} + Wnorth_{ag} + Wgatewell$$

$Wwetwell = 331.621 \cdot kip$

Wetwell Structure Factor of Safety for Flotation

Weight of water that is present during the event

$Wellarea := 149.5 ft^2$

Bottom of wetwell base elevation

$Elevwetwellbot := 731.58 ft$

$HEAD := 5 ft$

bottom of blanket at 725

$Water := HEAD \cdot Wellarea \cdot 62.4 pcf$ $\quad Water = 46.644 \cdot kip$

$heel := 0 ft$ $\quad heel = 0$ The wet well does not have heels

$upliftarea := Basearea$ $\quad upliftarea = 149.5\ ft^2$ $\quad ELEV_1 = 754\ ft$

$H := ELEV_1 - Elevwetwellbot$ $\quad H = 22.42\ ft$ Grade elevation

Weight of soil on the wetwell heels

$volsoil := (1ft - 1ft) \cdot heel + (1ft - 1ft) \cdot heel$ $\quad volsoil = 0 \cdot ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 0 \cdot kip$ $\quad volsoil \cdot \gamma_w = 0 \cdot kip$

Head, if structural foundation in blanket. $\quad HGL = 3.95\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot upliftarea \cdot \gamma_w$$

$U_1 = 237.64 \cdot kips$ $\quad BLANKET = 29\ ft$ $\quad HGL = 3.95\ ft$

Head, if structural foundation extends through blanket. $\quad H = 22.42\ ft$

$$U_2 := \left[(H + HGL) \cdot (upliftarea) \cdot \gamma_w\right]$$

$U_2 = 246 \cdot kips$

$$Uplift_{wet} := if\left(H > BLANKET, U_2, U_1\right)$$

$$Uplift_{wet} = 237.64 \cdot kips \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 25.474\ ft$$

The uplift force acting on the wetwell structure $\quad Uplift_{wet} = 237.64 \cdot kips$

$$Stability := \frac{Wwetwell + Water + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{wet} - volsoil \cdot \gamma_w}$$

$Stability = 1.592$ $\quad (H + HGL) = 26.37\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $\quad StabilityCheck = "Okay"$

Wet Well BASE SLAB ANALYSIS

$W_{water} := Water$ WEIGHT OF WATER

$BaseSlab := upliftarea$ $BaseSlab = 149.5\,ft^2$ $Uplift_{wet} = 237.64 \cdot kip$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$ $\frac{Wwetwell + W_{water}}{BaseSlab} = 2530.202 \cdot psf$

$W' := if\left(Stability < 1.0, \frac{Uplift_{wet}}{BaseSlab}, \frac{Wwetwell}{BaseSlab}\right)$ $W' = 2218.202 \cdot psf$ $\frac{Uplift_{wet}}{BaseSlab} = 1589.563 \cdot psf$

$W' = 2218.202 \cdot psf$ $\frac{Wwetwell + volsoil \cdot (\gamma - \gamma_w)}{BaseSlab} = 2218.202 \cdot psf$

Note: This was also used in the skin friction calculations as U.

$Slab_t := D_f$ $L_{strip} := 26.75\,ft$ $Wall_t := D_1$ $Slab_d := d_f$ effective depth of slab

Analyze slab with two-way action

$Slab_d = 14.845 \cdot in$

$D_1 = 15 \cdot in$

$V'_{ub} := \frac{W' \cdot L_{strip}}{2}$ $V'_{ub} = 29668.456 \frac{lb}{ft}$

$d_f = 14.845 \cdot in$

$D_f = 1.5\,ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}} \cdot \left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 25538 \frac{lb}{ft}$

$A_g := Slab_t \cdot 12 \frac{in}{ft}$ $A_g = 216 \cdot \frac{in^2}{ft}$

$Water = 46644\,lb$

$$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f \right) \right]$$

ACI EQ (11-4)

$\phi V_{nb} = 19514 \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, "OKAY", "NO GOOD"\right)$

CheckBase = "NO GOOD"

Factor of Safety

$$FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$$

$FS_{Sb} = 0.764$

"Based on Moment Strength Reliability analysis, shear Reliability was not performed"

Base Slab Analysis

$l_2 := 5.63 ft$ $\quad l_1 := 14.25 ft$

Plate Loading (Bending)

a/b=L_2/L_1

$\frac{l_2}{l_1} = 0.395$

M.x = Horizontal steel

$A_{slb} := .31 in^2$ (OF Steel)

$\frac{l_2}{l_1} = 0.395$

From plates tables:

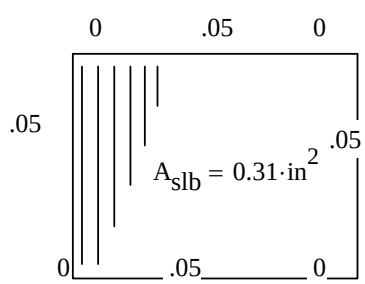
a/b	Coefficient
1	.0500
.875	.0592
.75	.0686
.5	.082
.375	.083

$ab_k :=$ $\quad Coeff_k :=$ $\quad k := 1..5$

ab_k	$Coeff_k$
.375	.083
.5	.082
.75	.069
.875	.0592
1	.0500

Plate was analyzed as being fix supported on all sides (uniform load). (Bur. of Rec. pg.40)

5's @ 12" O.C.



$$W_{UNIFORM} := \frac{Uplift_{wet}}{upliftarea}$$

$$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right)$$

$Coefficient = 0.0828$

$W_{UNIFORM} = 1589.563 \cdot psf$ $\quad b = 1 ft$

$l_2 = 5.63 ft$

$$M_{uF} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$$

$M_{uF} = 4.174 \cdot kip \cdot ft$

$d_f = 1.237 ft$

$$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b}$$

$a = 0.405 \cdot in$

$$\phi M_{nF} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$$

$\phi M_{nF} = 15.13 ft \cdot kip$

$Slab_Bending_Check := if\left(\phi M_{nF} > 1.5 M_{uF}, "OK", "NO GOOD"\right)$

Slab_Bending_Check = "OK"

Factor of Safety

$$FS_{slabmoment} := \frac{\phi M_{nF}}{M_{uF}}$$

$FS_{slabmoment} = 3.625$

Base Slab Analysis

Plate Loading

M.x= Horizontal Steel

$l_2 := 5.63ft$ $l_1 := 14.25ft$

$k := 1..5$

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.395$

From plates tables:

a/b	Coefficient
1	.0213
.875	.0267
.75	.0324
.5	.0411
.375	.0424

$ab_k :=$

.375
.5
.75
.875
1.0

$Coeff_k :=$

.0424
.0411
.0324
.0267
.0213

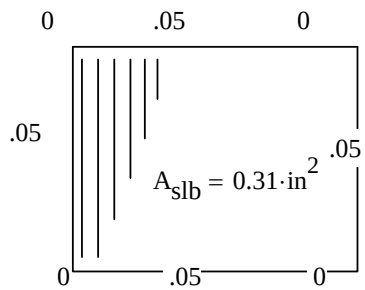
$A_{slb} := .31in^2$ (IF Steel)

$\frac{l_2}{l_1} = 0.395$

Plate was analyzed as being fix supported on all sides (uniform load). (Bur. of Rec. pg.40)

$W_{UNIFORM} := \frac{Uplift_{wet}}{upliftarea}$

5's @ 12" O.C.



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right)$

$Coefficient = 0.0422$

$W_{UNIFORM} = 1589.563 \cdot psf$ $b = 1\ ft$

$l_2 = 5.63\ ft$

$M_{uF} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$

$M_{uF} = 2.126 \cdot kip \cdot ft$ $d_f = 1.237\ ft$

$a := \frac{A_{slb} \cdot F_y}{0.85f'_c \cdot b}$ $a = 0.405 \cdot in$

$\phi M_{nF} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nF} = 15.13\ ft \cdot kip$

$Slab_Bending_Check := if\left(\phi M_{nF} > 1.5M_{uF}, "OK", "NO\ GOOD"\right)$ Slab_Bending_Check = "OK"

Factor of Safety $FS_{slabmoment1} := \frac{\phi M_{nF}}{M_{uF}}$

$Check := if\left(FS_{slabmoment1} > 1.5, "OKAY", "NO\ GOOD"\right)$ $FS_{slabmoment1} = 7.118$

Check = "OKAY"

FACTORS OF SAFETY SUMMARY

WALL 1 (WEST WETWELL WALL= EAST WETWELL WALL)

BAD

Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S1} = 1.924$

Probabilty of Failure Analysis conducted

Moment

$FS_{EPH1} = 0.658$

WALL 2 (SOUTH WETWELL WALL = NORTH WETWELL WALL)

BAD

Wall 2 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

Shear

$FS_{S2} = 3.42$

Probabilty of Failure Analysis Conducted

Moment

$FS_{EPH2a} = 0.442$

BAD

BASE SLAB

Shear

$FS_{Sb} = 0.764$

"Based on Moment Strength Reliability analysis, shear Reliability was not performed"

Moment

Analyzed using Two Way Action pin supported edges

$FS_{slabmoment} = 3.625$

Horizontal

$FS_{slabmoment1} = 7.118$

Vertical

STABILITY

Stability = 1.592

RECOMMENDATIONS

The existing wetwell walls and base slab are inadequate for the **HGL=757.95** future conditions event loading. To ensure reliability of the structure during the event, the following actions are recommended:

1. Strengthen the wetwell walls 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 Ohio Ave. pump station. To facilitate installation of the steel, the metal roofing system will have to be removed temporarily. The existing at grade top slab can be left in place as W-sections can be fished through the existing pump hole left open from the 1962 modifications, in order to install the steel structural members. The Ohio Ave fix is shown in "FC_OHIO_FIX_STA 83+52 HGL 758 B4Peer.mcd" file. After installing the steel structural members, replace the metal roofing system as required.
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=757.95** event.
3. The W-section uprights shall be supported at the top and bottom as a minimum 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 base slab shear, place an 1-foot thick reinforced concrete floor overlay the existing floor. The additional concrete flooring will necessitate the reduction in height of the 24-inch intake pipe, but should not affect existing pump performance per Ron Jansen (EC-GC).
7. During the site visit on June 18, 2007, it was noticed that the two of the original floor beams sustaining the load of the large pump, show signs of severe deterioration or rotting. Due to this deterioration, sometime ago a newer beam was put in place next to the deteriorated beam nearest the interior wall to remedy. However, the floor beam nearest the exterior wall is showing signs of severe deterioration as well, and therefore it is recommended that a similar remedy be executed to address this area of concern. Prior to design of remedy, it will be necessary to obtain all drawings of the prior fix to cross reference loading and strength requirements.
8. 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.

END OF ANALYSIS AND RECOMMENDATIONS

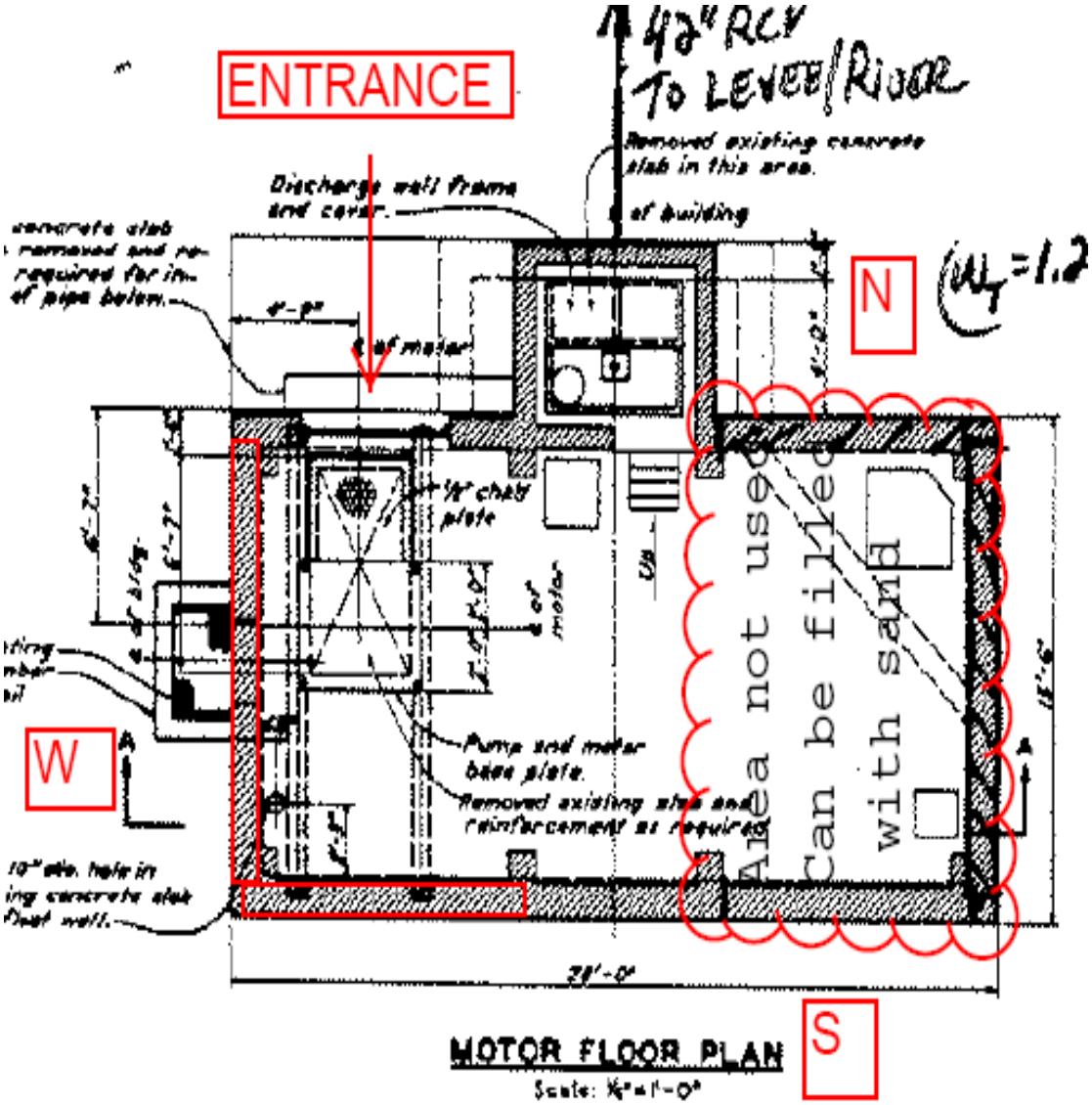
Foundation Modification with Water at top of Protection (Future Conditions)

Variables

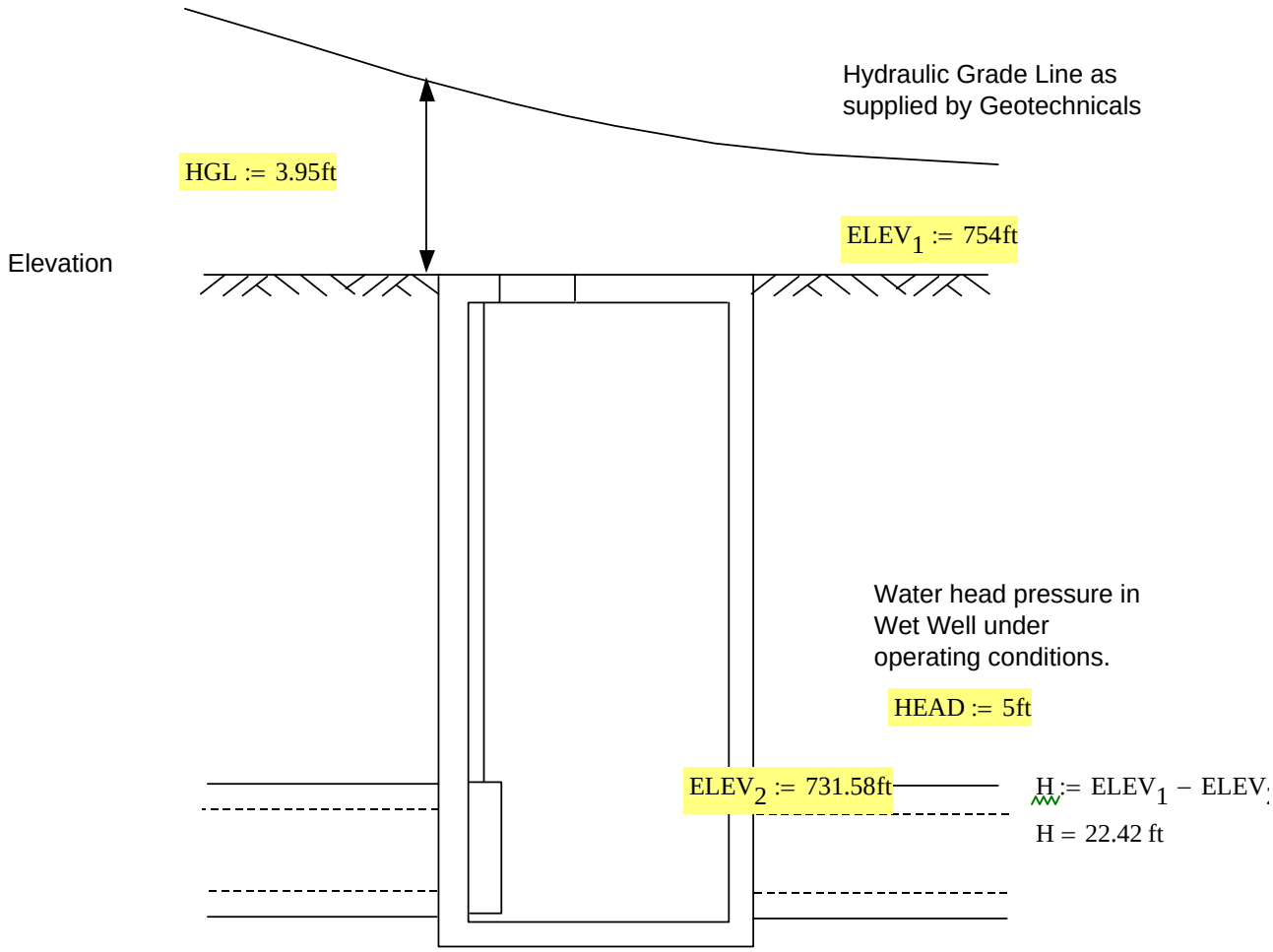
$kips := 1000lb$

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

A Series of Steel W-Section pilaters spaced approximately 5 ft on center with strut braces is proposed to address the foundation strength issues at the Ohio Avenue pump station.



Properties



Assumptions

- Steel W-Section Pilasters will be embeded into the existing floor and an additional 12" of concrete added to the basement foundation slab for floor strength concerns.
- The steel pilasters will be braced just below the existing floor elevation with a steel struts.

Soil Properties $\phi := 21deg$

$\gamma := 110pcf$

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

$K_o = 0.642$

Steel Properties $F_y := 50ksi$

Water Unit Weight $\gamma_w := 62.4pcf$

Load & Resistance Factor Design

Strength Reduction Factors

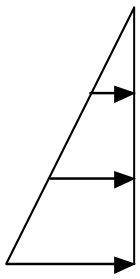
Shear Strength	$\phi_V := .85$	Note: Strength Reduction Factors
Flexural Strength	$\phi_B := 0.90$	
Load Factors		
Dead and Live Load Factor	$\gamma_L := 1.6$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1.3$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := .75$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Analysis

WALLS

Wall Loading

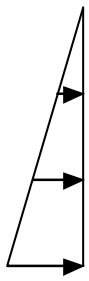
Soil or Soil & Water



$L_2 := H$

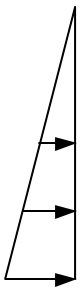
$L_2 = 22.42\ ft$

OR



$L_2 = 22.42\ ft$

&



$L_3 := 25.47ft$

Note: Taken from FC Calc's page # 5 (Ohio PS)

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot L_2$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma - \gamma_w) \cdot L_2 + \gamma_w \cdot L_3\right]$

$W_1 = 3291 \frac{lb}{ft^2}$

$W_2 = 3548 \frac{lb}{ft^2}$

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

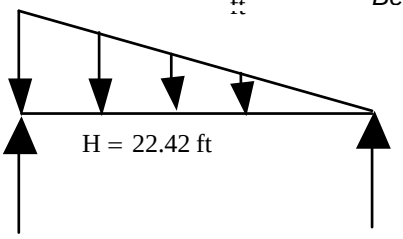
$W' := if(W_1 > W_2, W_1, W_2)$

$W' = 3548 \frac{lb}{ft^2}$

Tributary Width $Width := 5ft$

Loading $W := W' \cdot Width$

$W = 17.738 \cdot \frac{kip}{ft}$



Reaction and Moments from AISC Manual of Steel Construction

Beam Diagrams and Formulas

$$M_u := \frac{2\left(\frac{W \cdot H}{2}\right) \cdot H}{9 \cdot \sqrt{3}} \qquad M_u = 572 \cdot ft \cdot kip$$

$$R_2 := \frac{\left(\frac{W \cdot H}{2}\right)}{3} \qquad R_2 = 66.28 \cdot kips$$

PILASTERS

From AISC Manual of Steel Construction Beam Tables

Unbraced Length: $H = 22.42\ ft$

Design Moment: $\phi M_n := 574 kip \cdot ft$

Use W 18x76

Table 3-10 pg 3-119, Intermediatte braces must be present such that the unbraced length can not be more than 12-feet.

Yield Strength: $F_y = 50 \cdot ksi$

STRUTS

From AISC Manual of Steel Construction Beam Tables

Unbraced Length: $Width = 5\ ft$

Axial Load: $R_2 = 66.3 \cdot kip$

Use W 10x33

Yield Strength: $F_y = 50 \cdot ksi$

NOTE: Does not take into account floor load (combined axial and bending) but W 10x33 is good estimate for feasibility study. Additional Analysis to be performed at time of Plans and Specifications.

Probability of Failure
Ohio Pump StationWall 1

Comp by: EHF
Chkd by: MP 5/6/08

STA 83+52

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.
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, $D := 15in$ $b := 1ft$ (1 foot strip)

$A_1 := .31in^2$ $A_2 := .20in^2$

$C_1 := 2.0in$ $C_2 := 2.0in$ Distance from compression face to bar centerline

$d_1 := D - C_1 - .25in$ $d_2 := C_2 + .25in$

$d_1 = 12.75 \cdot in$

Refer to EC_OHIO_STA 83+52 HGL 758 b4peer.xmcd

Loads from initial screening calculation: $M_u := 24kip \cdot ft$

$Thrust := 4.75kip$ "Half of the thrust coming from exterior wall (9500/2) per M.P. 2/14/08"

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 $c := .2 \cdot D$ $c = 3 \cdot in$ $Thrust = 4.75 \cdot kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & if \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ F_{yM} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & if \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & if \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots + 0.85 \cdot f'_{cM} \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $c = 0.93 \cdot in$ $a := if(\beta 1 \cdot c > D, D, \beta 1 \cdot c)$ $a = 0.791 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi\right)\cdot A_i$ $\qquad$ $Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.03	-50 ·ksi	0.31 ·in^2	-15.5 ·kip	81.375 ·kip·in
0.188	-4.255·10	-50	0.2	-10	-52.5

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 29\cdot kip\cdot in$

$Mc := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 215\cdot kip\cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 20.32\cdot kip\cdot ft$

$FS_B := \frac{M_n}{M_u}$ $\quad$ $Check := \text{if}\left(FS_B > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\quad$ $FS_B = 0.846$ $\qquad$ 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.

$f'_{cU} := f'_{cM} + f'_{cM}\cdot 0.14$ $\qquad$ $f'_{cU} = 4275\cdot psi$

$\beta 1 := \text{if}\left[f'_{cU} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cU}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cU}}{ksi} - 4\right)\right]\right]$ $\qquad$ $\beta 1 = 0.836$

Assume c=0.2*D $\qquad$ $i := 1 .. 2$

Given $\qquad$ $c := .2\cdot D$ $\qquad$ $c = 3\cdot in$ $\qquad$ $Thrust = 4.75\cdot kip$

Upper Concrete Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2} \left[\begin{cases} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots + -\text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ F_{yM} - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM} - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{cases} \right]\cdot A_i \ldots + 0.85\cdot f'_{cU}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 0.83\cdot in$ $\quad a := \text{if}(\beta1\cdot c > D, D, \beta1\cdot c)$ $\quad a = 0.694\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cU}, 0ksi\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.043	-50 ·ksi	0.31 $\cdot in^2$	-15.5 ·kip	81.375 ·kip·in
0.188	$-5.137\cdot10^{-3}$	-50	0.2	-10	-52.5

$Mst_U := \sum_{i=1}^{2} Ms_i$ $\quad Mst_U = 29\cdot kip\cdot in$

$Mc_U := 0.85\cdot f'_{cU}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_U = 216\cdot kip\cdot in$

$M_{nU} := \left(Mc_U + Mst_U\right)$ $\quad M_{nU} = 20.44\cdot kip\cdot ft$

$FS_{BU} := \frac{M_{nU}}{M_u}$ $\quad Check_U := \text{if}\left(FS_{BU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BU} = 0.852$ $\quad Check_U = \text{"NO GOOD"}$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\qquad f'_{cL} = 3225 \cdot psi$

$$\beta 1 := \text{if}\left[f'_{cL} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cL}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cL}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2 \cdot D \qquad c = 3 \cdot in \qquad Thrust = 4.75 \cdot kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{l} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots \quad \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) \\ F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) \quad \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) \quad \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_{cL} \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \qquad c = 1.082 \cdot in \qquad a := \text{if}(\beta 1 \cdot c > D, D, \beta 1 \cdot c) \qquad a = 0.92 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(E_s \cdot \varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s \cdot \varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cL}, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.032	-50 ·ksi	0.31 $\cdot in^2$	-15.5 ·kip	81.375 ·kip·in
0.188	$-3.239 \cdot 10^{-3}$	-50	0.2	-10	-52.5

Lower Concrete Strength, con't

$$Mst_L := \sum_{i=1}^{2} Ms_i \qquad Mst_L = 29 \cdot kip \cdot in$$

$$Mc_L := 0.85 \cdot f'_{cL} \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right) \qquad Mc_L = 213 \cdot kip \cdot in$$

$$M_{nL} := \left(Mc_L + Mst_L\right) \qquad M_{nL} = 20.15 \cdot kip \cdot ft$$

$$FS_{BL} := \frac{M_{nL}}{M_u} \qquad Check_L := if\left(FS_{BL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$$FS_{BL} = 0.84 \qquad Check_L = \text{"NO GOOD"}$$

Upper Steel Yield Strength

$$F_{yU} := F_{yM} + F_{yM} \cdot 0.14 \qquad F_{yU} = 57 \cdot ksi$$

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2 \cdot D \qquad c = 3 \cdot in \qquad Thrust = 4.75 \cdot kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yU} \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & \\ F_{yU} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yU} \\ -F_{yU} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yU} \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_{cM} \cdot \beta 1 \cdot c \cdot b$$

$$c := Find(c) \qquad c = 1.04 \cdot in \qquad a := if\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right) \qquad a = 0.884 \cdot in$$

Upper Steel Yield Strength, con't

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yU}, F_{yU}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yU}, 0 - F_{yU}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi\right)\cdot A_i$ $\qquad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.034	-57 ·ksi	0.31 $\cdot in^2$	-17.67 ·kip	92.767 ·kip·in
0.188	$-3.489\cdot10^{-3}$	-57	0.2	-11.4	-59.85

$Mst_{cU} := \sum_{i=1}^{2} Ms_i$ $\qquad Mst_{cU} = 33\cdot kip\cdot in$

$Mc_{cU} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad Mc_{cU} = 239\cdot kip\cdot in$

$M_{ncU} := \left(Mc_{cU} + Mst_{cU}\right)$ $\qquad M_{ncU} = 22.63\cdot kip\cdot ft$

$FS_{BcU} := \frac{M_{ncU}}{M_u}$ $\qquad Check_{cU} := \text{if}\left(FS_{BcU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BcU} = 0.943$ $\qquad Check_{cU} = \text{"NO GOOD"}$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM}\cdot 0.14$ $\qquad F_{yL} = 43\cdot ksi$

$\beta 1 := \text{if}\left[f'_{cM} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right]$ $\qquad \beta 1 = 0.85$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2\cdot D$ $\qquad c = 3\cdot in$ $\qquad Thrust = 4.75\cdot kip$

Lower Steel Yield Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2}\left[\left[\begin{array}{ll}\left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots + -\text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yL} \\ F_{yL} - \text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yL} \\ -F_{yL} - \text{if}\left(d_i<\beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } \left(c-d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yL}\end{array}\right]\cdot A_i\right] \ldots + 0.85\cdot f'_{cM}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 0.821\cdot in$ $\quad a := \text{if}(\beta1\cdot c > D, D, \beta1\cdot c)$ $\quad a = 0.698\cdot in$

Find Moment:

$\varepsilon s_i := \left(c-d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yL}, F_{yL}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yL}, 0 - F_{yL}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.044	-43 ·ksi	0.31 $\cdot in^2$	-13.33 ·kip	69.982 ·kip·in
0.188	$-5.226\cdot10^{-3}$	-43	0.2	-8.6	-45.15

$Mst_{cL} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cL} = 25\cdot kip\cdot in$

$Mc_{cL} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cL} = 191\cdot kip\cdot in$

$M_{ncL} := \left(Mc_{cL} + Mst_{cL}\right)$ $\quad M_{ncL} = 17.97\cdot kip\cdot ft$

$FS_{BcL} := \frac{M_{ncL}}{M_u}$ $\quad Check_{cL} := \text{if}\left(FS_{BcL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BcL} = 0.749$ $\quad Check_{cL} = \text{"NO GOOD"}$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{BcU} - FS_{BcL}$ $\Delta F_{Fy} = 0.194$

$\Delta F_{fc} := FS_{BU} - FS_{BL}$ $\Delta F_{fc} = 0.012$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2} \qquad \sigma_F = 0.097$$

$$V_F := \frac{\sigma_F}{FS_B} \qquad V_F = 0.115$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_B}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = -1.511$$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 6.54 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 93.46 \cdot \%$

END OF ANALYSIS

US Army Corps of Engineers

Probability of Failure
Ohio Pump StationWall 2

Comp by: EHF
Chkd by: MP 5/7/08

STA 83+52

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.
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, $D := 15in$ $b := 1ft$ (1 foot strip)

$A_1 := .31in^2$ $A_2 := .20in^2$

$C_1 := 2.0in$ $C_2 := 2.0in$ Distance from compression face to bar centerline

$d_1 := D - C_1 - .25in$ $d_2 := C_2 + .25in$

$d_1 = 12.75 \cdot in$ $d_2 = 2.25 \cdot in$

Refer to EC_OHIO_STA 83+52 HGL 758 b4peer.xmcd

Loads from initial screening calculation: $M_u := 38kip \cdot ft$

$Thrust := 6.5kip$

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 $c := .2 \cdot D$ $c = 3 \cdot in$ $Thrust = 6.5 \cdot kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & if \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ F_{yM} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & if \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & if \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots + 0.85 \cdot f'_{cM} \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $c = 0.984 \cdot in$ $a := if(\beta 1 \cdot c > D, D, \beta 1 \cdot c)$ $a = 0.837 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad$ $fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi\right)\cdot A_i$ $\qquad$ $Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.03	-50 ·ksi	0.31 ·in^2	-15.5 ·kip	81.375 ·kip·in
0.188	$-3.858\cdot10$	-50	0.2	-10	-52.5

$Mst := \sum_{i=1}^{2} Ms_i$ $\qquad$ $Mst = 29\cdot kip\cdot in$

$Mc := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 227\cdot kip\cdot in$

$M_n := (Mc + Mst)$ $\qquad$ $M_n = 21.29\cdot kip\cdot ft$

$FS_B := \frac{M_n}{M_u}$ $\quad$ $Check := \text{if}\left(FS_B > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\quad$ $FS_B = 0.56$ $\qquad$ 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.

$f'_{cU} := f'_{cM} + f'_{cM}\cdot 0.14$ $\qquad$ $f'_{cU} = 4275\cdot psi$

$\beta 1 := \text{if}\left[f'_{cU} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cU}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cU}}{ksi} - 4\right)\right]\right]$ $\qquad$ $\beta 1 = 0.836$

Assume c=0.2*D $\qquad$ $i := 1 .. 2$

Given $\qquad$ $c := .2\cdot D$ $\qquad$ $c = 3\cdot in$ $\qquad$ $Thrust = 6.5\cdot kip$

Upper Concrete Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2} \left[\begin{cases} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots + -\text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yM} \\ F_{yM} - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yM} \\ -F_{yM} - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cU}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yM} \end{cases} \right]\cdot A_i \ldots + 0.85\cdot f'_{cU}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 0.878\cdot in$ $\quad a := \text{if}(\beta1\cdot c > D, D, \beta1\cdot c)$ $\quad a = 0.734\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cU}, 0ksi\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.041	-50 ·ksi	0.31 $\cdot in^2$	-15.5 ·kip	81.375 ·kip·in
0.188	$-4.692\cdot10^{-3}$	-50	0.2	-10	-52.5

$Mst_U := \sum_{i=1}^{2} Ms_i$ $\quad Mst_U = 29\cdot kip\cdot in$

$Mc_U := 0.85\cdot f'_{cU}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_U = 228\cdot kip\cdot in$

$M_{nU} := \left(Mc_U + Mst_U\right)$ $\quad M_{nU} = 21.43\cdot kip\cdot ft$

$FS_{BU} := \frac{M_{nU}}{M_u}$ $\quad Check_U := \text{if}\left(FS_{BU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BU} = 0.564$ $\quad Check_U = \text{"NO GOOD"}$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\qquad f'_{cL} = 3225 \cdot psi$

$$\beta 1 := \text{if}\left[f'_{cL} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cL}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cL}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2 \cdot D \qquad c = 3 \cdot in \qquad Thrust = 6.5 \cdot kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{l} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots \quad \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yM} \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) \\ F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) \quad \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yM} \\ -F_{yM} - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cL}, 0\right) \quad \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yM} \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_{cL} \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \qquad c = 1.144 \cdot in \qquad a := \text{if}(\beta 1 \cdot c > D, D, \beta 1 \cdot c) \qquad a = 0.973 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(E_s \cdot \varepsilon s_i > F_{yM}, F_{yM}, \text{if}\left(E_s \cdot \varepsilon s_i < 0 - F_{yM}, 0 - F_{yM}, E_s \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'_{cL}, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.03	-50 ·ksi	0.31 $\cdot in^2$	-15.5 ·kip	81.375 ·kip·in
0.188	$-2.898 \cdot 10^{-3}$	-50	0.2	-10	-52.5

Lower Concrete Strength, con't

$$Mst_L := \sum_{i=1}^{2} Ms_i \qquad Mst_L = 29 \cdot kip \cdot in$$

$$Mc_L := 0.85 \cdot f'_{cL} \cdot a \cdot b \cdot \left(\frac{D}{2} - \frac{a}{2}\right) \qquad Mc_L = 224 \cdot kip \cdot in$$

$$M_{nL} := \left(Mc_L + Mst_L\right) \qquad M_{nL} = 21.11 \cdot kip \cdot ft$$

$$FS_{BL} := \frac{M_{nL}}{M_u} \qquad Check_L := if\left(FS_{BL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$$FS_{BL} = 0.556 \qquad Check_L = \text{"NO GOOD"}$$

Upper Steel Yield Strength

$$F_{yU} := F_{yM} + F_{yM} \cdot 0.14 \qquad F_{yU} = 57 \cdot ksi$$

$$\beta 1 := if\left[f'_{cM} \le 4ksi, 0.85, if\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right] \qquad \beta 1 = 0.85$$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2 \cdot D \qquad c = 3 \cdot in \qquad Thrust = 6.5 \cdot kip$

$$Thrust = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s < F_{yU} \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & \\ F_{yU} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \ge F_{yU} \\ -F_{yU} - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'_{cM}, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot E_s \le -F_{yU} \end{array}\right] \cdot A_i\right] \ldots$$
$$+ 0.85 \cdot f'_{cM} \cdot \beta 1 \cdot c \cdot b$$

$$c := Find(c) \qquad c = 1.094 \cdot in \qquad a := if\left(\beta 1 \cdot c > D, D, \beta 1 \cdot c\right) \qquad a = 0.93 \cdot in$$

Upper Steel Yield Strength, con't

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$ $\qquad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yU}, F_{yU}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yU}, 0 - F_{yU}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi\right)\cdot A_i$ $\qquad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.032	-57 ·ksi	0.31 $\cdot in^2$	-17.67 ·kip	92.767 ·kip·in
0.188	$-3.17\cdot10^{-3}$	-57	0.2	-11.4	-59.85

$Mst_{cU} := \sum_{i=1}^{2} Ms_i$ $\qquad Mst_{cU} = 33\cdot kip\cdot in$

$Mc_{cU} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\qquad Mc_{cU} = 250\cdot kip\cdot in$

$M_{ncU} := \left(Mc_{cU} + Mst_{cU}\right)$ $\qquad M_{ncU} = 23.6\cdot kip\cdot ft$

$FS_{BcU} := \frac{M_{ncU}}{M_u}$ $\qquad Check_{cU} := \text{if}\left(FS_{BcU} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BcU} = 0.621$ $\qquad Check_{cU} = \text{"NO GOOD"}$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM}\cdot 0.14$ $\qquad F_{yL} = 43\cdot ksi$

$\beta 1 := \text{if}\left[f'_{cM} \le 4ksi, 0.85, \text{if}\left[\left(f'_{cM}\right) > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'_{cM}}{ksi} - 4\right)\right]\right]$ $\qquad \beta 1 = 0.85$

Assume c=0.2*D $\qquad i := 1 .. 2$

Given $\qquad c := .2\cdot D$ $\qquad c = 3\cdot in$ $\qquad Thrust = 6.5\cdot kip$

Lower Steel Yield Strength, con't

$$\text{Thrust} = \sum_{i=1}^{2} \left[\left[\begin{array}{ll} \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ldots & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s < F_{yL} \\ +-\text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \\ F_{yL} - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \ge F_{yL} \\ -F_{yL} - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'_{cM}, 0\right) & \text{if } \left(c - d_i\right)\cdot\frac{0.003}{c}\cdot E_s \le -F_{yL} \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85\cdot f'_{cM}\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\quad c = 0.874\cdot in$ $\quad a := \text{if}(\beta1\cdot c > D, D, \beta1\cdot c)$ $\quad a = 0.743\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right)\cdot\frac{0.003}{c}$ $\quad fs_i := \text{if}\left(E_s\cdot\varepsilon s_i > F_{yL}, F_{yL}, \text{if}\left(E_s\cdot\varepsilon s_i < 0 - F_{yL}, 0 - F_{yL}, E_s\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta1\cdot c > d_i, 0.85\cdot f'_{cM}, 0ksi\right)\cdot A_i$ $\quad Ms_i := Fs_i\cdot\left(\frac{D}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
1.062 ft	-0.041	-43 ·ksi	0.31 $\cdot in^2$	-13.33 ·kip	69.982 ·kip·in
0.188	$-4.719\cdot10^{-3}$	-43	0.2	-8.6	-45.15

$Mst_{cL} := \sum_{i=1}^{2} Ms_i$ $\quad Mst_{cL} = 25\cdot kip\cdot in$

$Mc_{cL} := 0.85\cdot f'_{cM}\cdot a\cdot b\cdot\left(\frac{D}{2} - \frac{a}{2}\right)$ $\quad Mc_{cL} = 203\cdot kip\cdot in$

$M_{ncL} := \left(Mc_{cL} + Mst_{cL}\right)$ $\quad M_{ncL} = 18.96\cdot kip\cdot ft$

$FS_{BcL} := \frac{M_{ncL}}{M_u}$ $\quad Check_{cL} := \text{if}\left(FS_{BcL} > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_{BcL} = 0.499$ $\quad Check_{cL} = \text{"NO GOOD"}$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{BcU} - FS_{BcL}$ $\qquad \Delta F_{Fy} = 0.122$

$\Delta F_{fc} := FS_{BU} - FS_{BL}$ $\qquad \Delta F_{fc} = 8.384 \times 10^{-3}$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2} \qquad \sigma_F = 0.061$$

$$V_F := \frac{\sigma_F}{FS_B} \qquad V_F = 0.109$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_B}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = -5.376$$

$R := \mathrm{cnorm}(\beta_{LN})$ $\qquad R = 3.81 \times 10^{-6} \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 100 \cdot \%$

END OF ANALYSIS

Computed By: Eddie Fernandez Date: 24-Feb-08
Checked By: MP 5/6/08

OHIO OUTLET WORKS EVALUATION

The Ohio outlet pipe was evaluated for strength when exposed to existing condition 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 Santa Fe outlet pipe is a modified circular section and installation method shown in O&MM manual, a 1st Class bedding condition was assumed (embankment condition). 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 existing condition loading and assuming the pipe full of water, the design pipe demand is 2777 #/ft/ft. If the hydraulic load factor 1.3 is waived, the demand becomes 2136 #/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 2871 #/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 outlet pipe was inspected in July 2007 by EC-DS personnel. The pipe did not show signs of distress or out of roundness. No longitudinal cracks were found. There were two isolated areas where reinforcement was exposed. A circumferential crack of approximately 1.5-2-inches in width was discovered mid-way through the pipe section most likely caused by settlement.

Based on the analysis conducted, field investigation and available information, the Ohio pipe is OKAY for the existing condition loading.

Table 1. OHIO OUTLET WORKS DESIGN PIPE DEMAND LOAD, INDIRECT METHOD, EMBANKMENT CONDTIONS

					Project:	Ohio 42" concrete pipe
Si =	3.5	ft	**Water?**	1	**Date:**	2/24/2008
bc =	5.17	ft			PIPECAR Capacity Values:	
Soil =	110	pcf			3EB =	2871 # / ft / ft
					Ultimate=	3425 # / 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
Exist Cond	1.0	25	2.8	2	1.4	14525	20335	600	0	20935	2136	Hf=1.0
Exist Cond	1.3	25	2.8	2	1.4	14525	20335	600	0	20935	2777	Hf=1.3

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.*

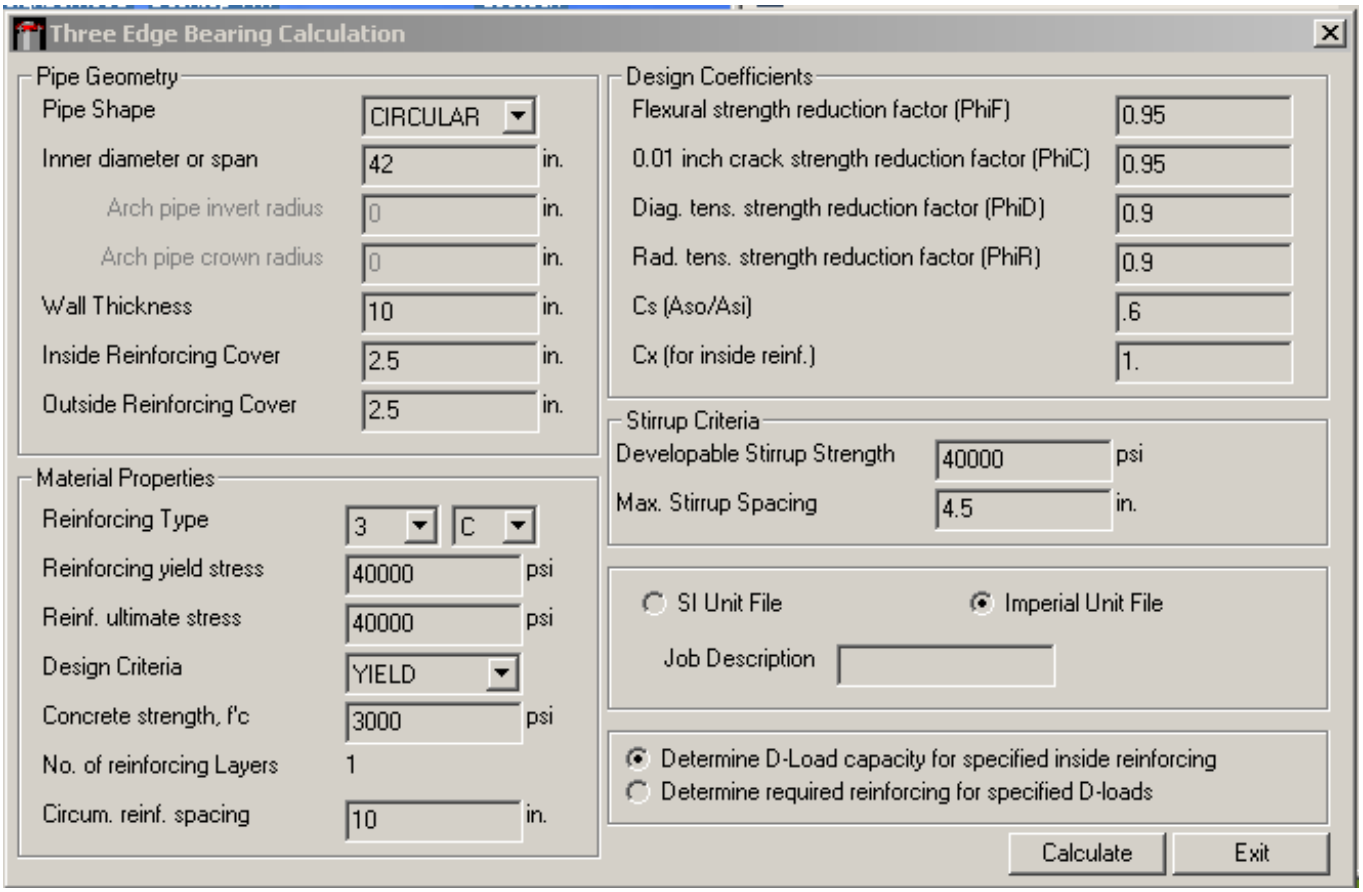
Table 2. Ohio Pipe Capacity Values, PIPECAR Software for 3-Edge Bearing.

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	2871	3347	3D	Yield	Mean value of steel and concrete strength.
40	40	2.40	2871	3347	3D	Yield	
40	40	3.75	2871	3347	3D	Yield	
32	32	2.40	2223	2418	3D	Yield	
32	32	3.0	2223	2418	3D	Yield	
32	32	3.75	2223	2418	3D	Yield	
50	50	2.40	3680	4276	3D	Yield	
50	50	3.0	3680	4276	3D	Yield	
50	50	3.75	3680	4276	3D	Yield	

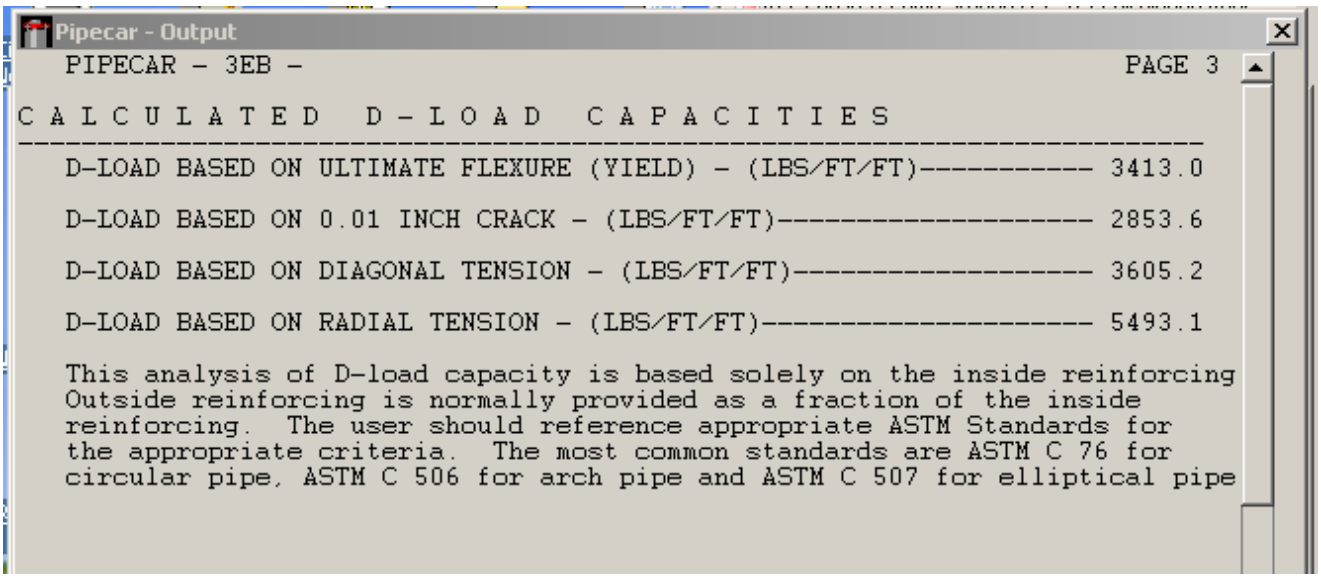
Notes:

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 = 0.37 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 reliability study in the event this information is required.

OHIO PUMP STATION CONDUIT INPUT PARAMETERS



OHIO PUMP STATION CONDUIT D0.01 STRENGTH.



Pipecar - Output

```
   PIPECAR - 3EB -                                                    PAGE 3

C A L C U L A T E D   D - L O A D   C A P A C I T I E S
-----------------------------------------------------------------------------
   D-LOAD BASED ON ULTIMATE FLEXURE (YIELD) - (LBS/FT/FT)----------- 3413.0

   D-LOAD BASED ON 0.01 INCH CRACK - (LBS/FT/FT)--------------------- 2853.6

   D-LOAD BASED ON DIAGONAL TENSION - (LBS/FT/FT)------------------- 3605.2

   D-LOAD BASED ON RADIAL TENSION - (LBS/FT/FT)--------------------- 5493.1

   This analysis of D-load capacity is based solely on the inside reinforcing
   Outside reinforcing is normally provided as a fraction of the inside
   reinforcing.  The user should reference appropriate ASTM Standards for
   the appropriate criteria.  The most common standards are ASTM C 76 for
   circular pipe, ASTM C 506 for arch pipe and ASTM C 507 for elliptical pipe
```

Next page

PIPECAR - THREE-EDGE BEARING - D-LOAD ANALYSIS
42 INCH SPAN CIRCULAR PIPE

****

S P E C I F I E D R E I N F O R C I N G A R E A S

INSIDE REINFORCING AREA, Asi = .368 SQ.IN./FT

C A P A C I T Y R E D U C T I O N F A C T O R S

PhiF = .95 PhiC = .95 PhiD = .9 PhiR = .9

D E S I G N C O E F F I C I E N T S

Cm = 1. Cmo = 1. Cs = .6 Cx = 1. Cmp = 1.

P I P E S P E C I F I C A T I O N S

WALL THICKNESS (IN.)	10.00
INSIDE REINFORCING COVER (IN.)	2.50
OUTSIDE REINFORCING COVER (IN.)	2.50
NO. OF REINFORCING LAYERS	1.
PIPE WEIGHT (LBS/FT)	1716.
CONCRETE STRENGTH (PSI)	3000.
REINFORCING YIELD STRENGTH (PSI)	40000.
REINFORCING ULTIMATE STRENGTH (PSI)	40000.
DESIGN CRITERIA	YIELD
SPACING OF CIRCUMFERENTIAL REINF. (IN.)	10.00

TYPE 3 REINFORCEMENT
WELDED DEFORMED WIRE FABRIC, DEFORMED BARS
OR ANY REINFORCEMENT WITH STIRRUPS

Ohio Gatewell 83+52 Existing Condition Strength and Uplift Stability Analysis:

Large openings create condition for which load and section analyzed should be first taken above the opening. See page 8 for loading calculation.

Major tasks assocaited with this analysis:

1. Obtain the geometry for the gatewell and levee which are taken from the Record drawings.
2. Obtain the geotechnical parameters which are obtained from the Geotechnical Engineer.
3. Perform uplift analysis assuming gate is empty.

Calculation Convention Notes:

1. All input values are highlighted in *yellow*
2. Results are highlighted in green
3. Assumption notes are displayed in red

Hydraulic Gradeline:

HGL is the same for "Existing Condition" and for "Future Condition" (n500+3).

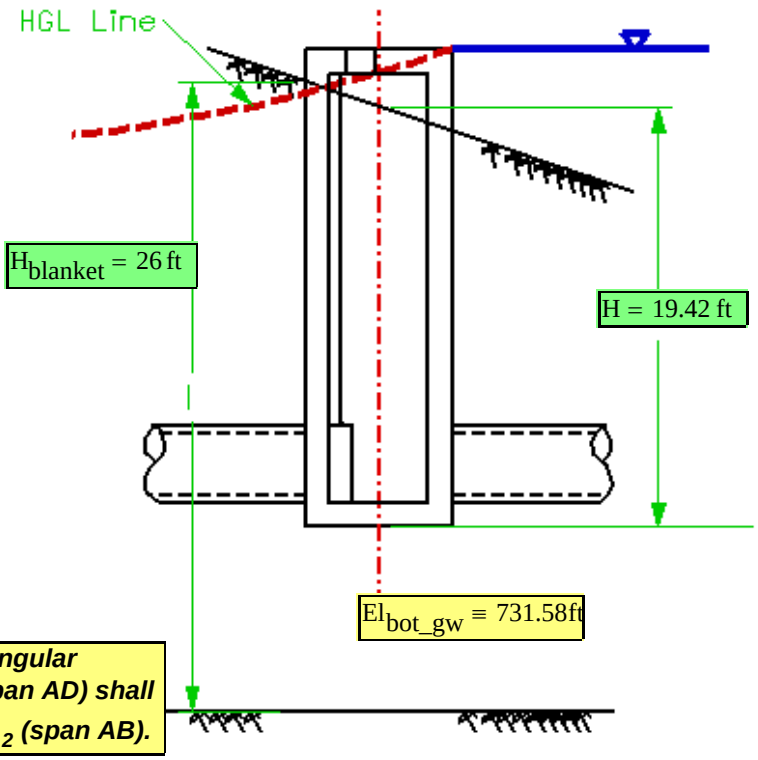
Soil Elevation Assumed by geotech: $El_{soil_geotech} \equiv 751ft$

Height of the HGL using geotech datum: $HGL_{geotech} \equiv 757.95ft$

Elevation of bottom of blanket: $El_{bot_blanket} \equiv 725ft$

Input Values

From record drwg $El_{top_gw} \equiv 759.3ft$



$H_{blanket} \equiv El_{soil_geotech} - El_{bot_blanket}$

The HGL from the geotech needs to be adjusted using the soil elevation at the gatewell as the datum:

$HGL := HGL_{geotech} \cdots + -El_{soil_geotech}$

$HGL = 6.95\,ft$

Water head pressure in Wet Well under operating conditions.

$HEAD := 5ft$

Buried Height of

$H \equiv El_{soil_geotech} - El_{bot_gw}$

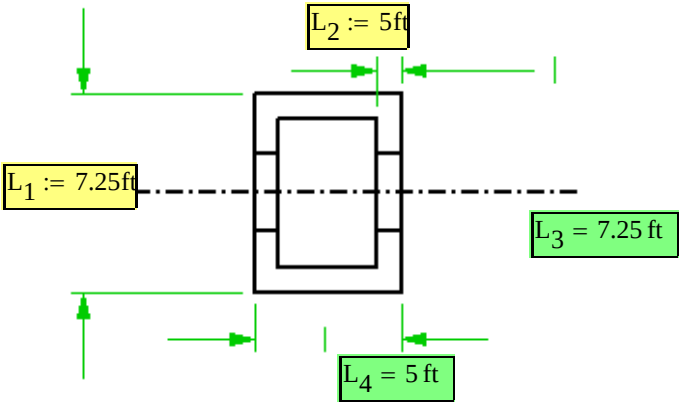
$H = 19.42\,ft$

NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

Projected Height of Gatewell above soil

$H' := El_{top_gw} - El_{soil_geotech}$

$H' = 8.3\,ft$



Width analyzed: $b := 12in$

$L_3 := L_1$

Floor Thickness $D_f := 18in$

$L_4 := L_2$

Slab clear cover $C_{cf} := 3in$

Slab effective depth $d_f := D_f - C_{cf}$

$d_f = 15 \cdot in$

Note: Upper case "L" represents the out-to-out dimension.
Lower case "l" representst the clear span dimension.

Length_Check :=	"Ok" if $L_1 \geq L_2$
	"NG - Assign larger dim to L1" otherwise

Length_Check = "Ok"

Wall Thickness

$D_1 := 15in$ Wall 1 (Span AD) wall thickness

$D_2 := 15in$ Wall 2 (Span AB) wall thickness

$D_3 := D_1$

$D_4 := D_2$

Wall 3 (Span BC) must be of the same thickness as Wall 1 (Span AD)
Wall 4 (Span CD) must be of the same thickness as Wall 2 (Span AB)

Clear Spans

$l_1 := L_1 - 2 \cdot D_2$ $l_1 = 4.75\,ft$ Clear span wall AD

$l_2 := L_2 - 2D_1$ $l_2 = 2.5\,ft$ Clear span of wall AB

$l_3 := l_1$ $l_3 = 4.75\,ft$ Clear span of wall BC

$l_4 := l_2$ $l_4 = 2.5\,ft$ Clear span of wall CD

Concrete Section Areas

$b = 1\,ft$

$A_{g1} := D_1 \cdot b$ $A_{g1} = 1.25\,ft^2$

$A_{g2} := D_2 \cdot b$ $A_{g2} = 1.25\,ft^2$

$A_{g3} := A_{g1}$ $A_{g3} = 1.25\,ft^2$

$A_{g4} := A_{g2}$ $A_{g4} = 1.25\,ft^2$

$d_f = 15 \cdot in$

Base Slab Reinforcment

Reinforcement information provided by the record drawings. $Bar_Sp_{slb} := 12in$

$BarNo_{slb} :=$ 5 6 $BarNo_{slb} = 5$

Base Slab Reinforcement

$$A_{slb} := \frac{A_{slb}}{\frac{Bar_Sp_{slb}}{12in}}$$

$A_{slb} = 0.31 \cdot in^2$

Wall Reinforcement Areas At Elevation Bot of Gatewell:

Reinforcement information provided by the record drawings

I. WALL AD

$L_1 = 7.25\,ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar_Spacing_{ADout} := 12in$

$Bar_Spacing_{ADin} := 12in$

$Bar_No_{ADout} :=$ 5 6 $Bar_No_{ADout} = 5$

$Bar_No_{ADin} :=$ 4 5 $Bar_No_{ADin} = 4$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_{ADout} := \frac{A_{out}}{\frac{Bar_Spacing_{ADout}}{12in}}$$

$$A_{ADin} := \frac{A_{in}}{\frac{Bar_Spacing_{ADin}}{12in}}$$

$A_{out} = 0.31 \cdot in^2$

$A_{in} = 0.2 \cdot in^2$

$A_{ADout} = 0.31 \cdot in^2$

$A_{ADin} = 0.2 \cdot in^2$

II. WALL AB

$L_2 = 5\,ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar_Spacing_{ABout} := 12in$

$Bar_Spacing_{ABin} := 12in$

$Bar_No_{ABout} :=$ 5 6 $Bar_No_{ABout} = 5$

$Bar_No_{ABin} :=$ 4 5 $Bar_No_{ABin} = 4$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_{ABout} := \frac{A_{out}}{\frac{Bar_Spacing_{ABout}}{12in}} \qquad A_{ABin} := \frac{A_{in}}{\frac{Bar_Spacing_{ABin}}{12in}} \qquad A_{out} = 0.31 \cdot in^2 \qquad A_{in} = 0.2 \cdot in^2$$

$$A_{ABout} = 0.31 \cdot in^2 \qquad A_{ABin} = 0.2 \cdot in^2$$

Reinforcement Bar Diameters

Reinforcement Clear Cover

Wall AD

$coverADout := 2in$

$coverADin := 2in$

Wall AB

$coverABout := 2in$

$coverABin := 2in$

Effective Depth:

These effective depths are used for the AD end analysis

$D_1 = 15 \cdot in \qquad diaADin = 0.5 \cdot in \qquad diaADout = 0.625 \cdot in$

$$d_{AD_in_end} := coverADin + \frac{diaADin}{2} \qquad d_{AD_out_end} := D_1 - coverADout - \frac{diaADout}{2}$$

$$d_{AD_in_end} = 2.25 \cdot in \qquad d_{AD_out_end} = 12.688 \cdot in$$

These effective depths are used for the AB end analysis

$diaABin = 0.5 \cdot in \qquad diaABout = 0.625 \cdot in \qquad D_2 = 1.25\,ft$

$$d_{AB_in_end} := coverABin + \frac{diaABin}{2} \qquad d_{AB_out_end} := D_2 - coverABout - \frac{diaABout}{2}$$

$$d_{AB_in_end} = 2.25 \cdot in \qquad d_{AB_out_end} = 12.688 \cdot in$$

These effective depths are used for the AD mid analysis

$diaADin = 0.5 \cdot in \qquad diaADout = 0.625 \cdot in$

$$d_{AD_in_mid} := D_1 - coverADin - \frac{diaADin}{2} \qquad d_{AD_out_mid} := coverADout + \frac{diaADout}{2}$$

$$d_{AD_in_mid} = 12.75 \cdot in \qquad d_{AD_out_mid} = 2.313 \cdot in$$

These effective depths are used for the AB mid analysis

$diaABin = 0.5 \cdot in$ $diaABout = 0.625 \cdot in$

$d_{AB_in_mid} := D_2 - coverABin - \frac{diaABin}{2}$

$d_{AB_in_mid} = 12.75 \cdot in$

$d_{AB_out_mid} := coverABout + \frac{diaABout}{2}$

$d_{AB_out_mid} = 2.313 \cdot in$

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 $\phi_B := 21deg$ $\gamma_{soil} := 110pcf$

At Rest Soil Pressure $K_o := 1 - \sin(\phi_B)$ $K_o = 0.642$

Active Soil Pressure: $K_a := \tan\left(45deg - \frac{\phi_B}{2}\right)^2$ $K_a = 0.472$

Concrete Unit Weight $\gamma_c := 150pcf$ Concrete Properties $f'_c := 3.0ksi$ Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40.0ksi$ Modulus of Elasticity: $Es := 29000ksi$

$\beta 1 := if\left(f'_c \le 4ksi, 0.85, if\left(f'_c > 8ksi, 0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4ksi}{ksi}\right)\right)$ $\beta 1 = 0.85$ $f'_c = 3000\,psi$

Load & Resistance Factor Design Resistance and Load Factor Values

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

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)

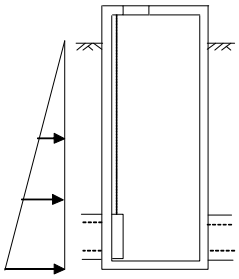
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) and Strength Reduction Factors (.85 for shear, 0.90 for bending) are not applied for in this initial evaluation. Instead a factor of safety is computed for the existing structure. If a strength concern is found, a design approach (applying these factors directly) will be taken for any recommended solutions.

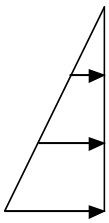
Wall Loading

Wall Loading

WALLS

Soil





$H = 19.42\ ft$

$H_2 := H$

$H_2 = 19.42\ ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H_2$

$W_1 = 1371 \cdot psf$

Soil & Water

$H_2 = 19.42\ ft$

&

$H_3 := \text{if}\left[H < H_{blanket}, \frac{H}{H_{blanket}} \cdot \left(HGL + H_{blanket}\right) - HEAD, H + HGL - HEAD\right]$

$H_3 = 19.611\ ft$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma_{soil} - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_2 = 1817 \cdot psf$

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

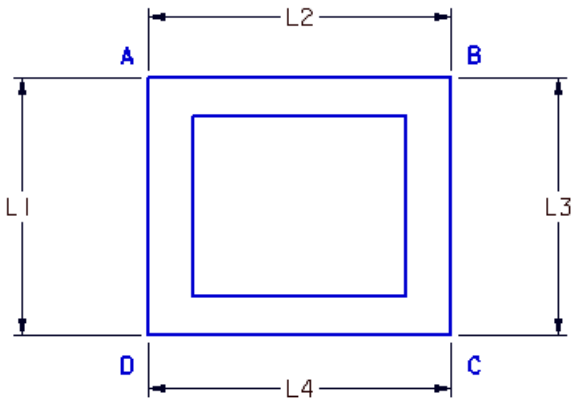
$W := \text{if}\left(W_1 > W_2, W_1, W_2\right) \cdot b$ $W = 1817 \cdot plf$

Moment Distribution of Gatewell Walls

NOTE: Check continuity of reinforcement at corners. If continuity is present, use moment distribution to determine the demand moment values. Otherwise, analyze the gatewell wall sections as simply supported beams.

The following calculations determine the distributed moment based on relative stiffness. In order for the calculations to be valid, the following must be true:
1. The structure has four walls. The walls that are parallel to each other are of the same thickness.
2. The EI value is based on the wall length and the wall thickness.
3. The walls are orthagonal.
4. The walls are exposed to uniform loading W, and $W = 1817 \cdot plf$ is the same for each wall.

Determine Moment and Shear:



Design Section Length

$L_1 = 7.25\,ft$ $L_2 = 5\,ft$ $L_3 = 7.25\,ft$ $L_4 = 5\,ft$

Moment of Inertia:

$b = 1\,ft$ $D_1 = 1.25\,ft$ $D_2 = 1.25\,ft$

$D_4 = 1.25\,ft$ $D_3 = 1.25\,ft$

$I_{beam1} := \frac{b \cdot D_1^3}{12}$ $I_{beam1} = 3375 \cdot in^4$

$I_{beam2} := \frac{b \cdot D_2^3}{12}$ $I_{beam2} = 3375 \cdot in^4$

$I_{beam3} := I_{beam1}$ $I_{beam3} = 3375 \cdot in^4$

$I_{beam4} := I_{beam2}$ $I_{beam4} = 3375 \cdot in^4$

Enter End Conditions:

All carryover factors are equal to 0.5 (idealize as a continuous beam over supports A, B, C and D).

$COF_A := 0.5$ $COF_B := COF_A$ $COF_C := COF_A$ $COF_D := COF_A$

$A := COF_A$ $B := COF_B$ $C := COF_C$ $D' := COF_D$

Fixed End Moments For A Uniform Load Between Fixed Supports using center of support to center of support distance per ACI 318-02 8.7.2 :

$L_1 = 7.25\,ft$ $L_2 = 60 \cdot in$ $L_3 = 7.25\,ft$ $L_4 = 5\,ft$ $W = 1816.853 \cdot plf$

$M_{fix_1} := \frac{W \cdot (L_1 - D_2)^2}{12}$ $M_{fix_1} = 5.451 \cdot kip \cdot ft$ $M_{fix_2} := \frac{W \cdot (L_2 - D_1)^2}{12}$ $M_{fix_2} = 2.129 \cdot kip \cdot ft$

$M_{fix_3} := \frac{W \cdot (L_3 - D_2)^2}{12}$ $M_{fix_3} = 5.451 \cdot kip \cdot ft$ $M_{fix_4} := \frac{W \cdot (L_4 - D_1)^2}{12}$ $M_{fix_4} = 2.129 \cdot kip \cdot ft$

Distribution Factors

	M_{AD}	M_{AB}	M_{BA}	M_{BC}	M_{CB}	M_{CD}	M_{DC}
COF	0.50	0.50	0.50	0.50	0.50	0.50	0.50
DF	0.41	0.59	0.59	0.41	0.41	0.59	0.59
FEM	-5.45	2.13	-2.13	5.45	-5.45	2.13	-2.13
D#1	1.356	1.966	-1.966	-1.356	1.356	1.966	-1.966
CO#1	-0.678	-0.983	0.983	0.678	-0.678	-0.983	0.983
DF#2	0.678	0.983	-0.983	-0.678	0.678	0.983	-0.983
CO#2	-0.339	-0.491	0.491	0.339	-0.339	-0.491	0.491
DF#3	0.339	0.491	-0.491	-0.339	0.339	0.491	-0.491
CO#3	-0.169	-0.246	0.246	0.169	-0.169	-0.246	0.246
DF#4	0.169	0.246	-0.246	-0.169	0.169	0.246	-0.246
CO#3	-0.085	-0.123	0.123	0.085	-0.085	-0.123	0.123
DF#4	0.085	0.123	-0.123	-0.085	0.085	0.123	-0.123
	-4.09	4.09	-4.09	4.09	-4.09	4.09	-4.09

Attach proper units to distributed values

$M_{ADend} := M'_{AD} \cdot ft \cdot kip$ $M_{ADend} := M'_{AB} \cdot ft \cdot kip$

$M_{CBend} := M'_{CB} \cdot ft \cdot kip$ $M_{DCend} := M'_{DC} \cdot ft \cdot kip$

End-of-Span Moments After Distribution

$M_{ADend} = -4.095 \cdot kip \cdot ft$ $M_{ADend} = -4.095 \cdot kip \cdot ft$

$M_{CBend} = -4.095 \cdot kip \cdot ft$ $M_{DCend} = -4.095 \cdot kip \cdot ft$

Note that the end-of-span moment value is the same at each joint because of uniform and equal loading present on all spans.

$W = 1816.853 \cdot plf$

Note: L_1 represents span AD, L_2 represents span AB, L_3 represents span BC and L_4 represents span CD

Mid-Span Moment Values

Moment Envelope Values

$$M_{UADin} := \left[\frac{W\cdot(L_1 - D_2)^2}{8} - |M_{ADend}|\right] \qquad M_{UADin} = 4.081\cdot kip\cdot ft \qquad \frac{W\cdot(L_1 - D_2)^2}{8} = 8.176\cdot kip\cdot ft$$

$$M_{UABin} := \left[\frac{W\cdot(L_2 - D_1)^2}{8} - |M_{ADend}|\right] \qquad M_{UABin} = -0.901\cdot kip\cdot ft \qquad \frac{W\cdot(L_2 - D_1)^2}{8} = 3.194\cdot kip\cdot ft$$

$$M_{UCBin} := \left[\frac{W\cdot(L_3 - D_2)^2}{8} - |M_{CBend}|\right] \qquad M_{UCBin} = 4.081\cdot kip\cdot ft \qquad \frac{W\cdot(L_3 - D_2)^2}{8} = 8.176\cdot kip\cdot ft$$

$$M_{UDCin} := \left[\frac{W\cdot(L_4 - D_1)^2}{8} - |M_{DCend}|\right] \qquad M_{UDCin} = -0.901\cdot kip\cdot ft \qquad \frac{W\cdot(L_4 - D_1)^2}{8} = 3.194\cdot kip\cdot ft$$

The demand moment will be taken at the support face IAW ACI 318-02, 8.7.2 and 8.7.3.

$$M_{UADface} := \left[M_{ADend} + \frac{W\cdot D_2}{8}\cdot(2L_1 - 3D_2)\right]$$

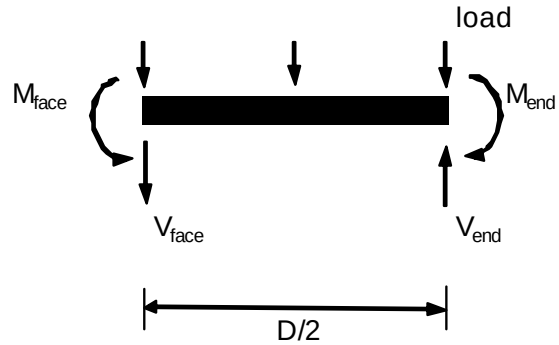
$$M_{UADface} = -1.043\cdot kip\cdot ft$$

$$M_{UABface} := \left[M_{ADend} + \frac{W\cdot D_1}{8}\cdot(2\cdot L_2 - 3D_1)\right]$$

$$M_{UABface} = -2.321\cdot kip\cdot ft$$

$$M_{UCBface} := M_{UADface} \qquad M_{UDCface} := M_{UABface}$$

Free Body Diagram

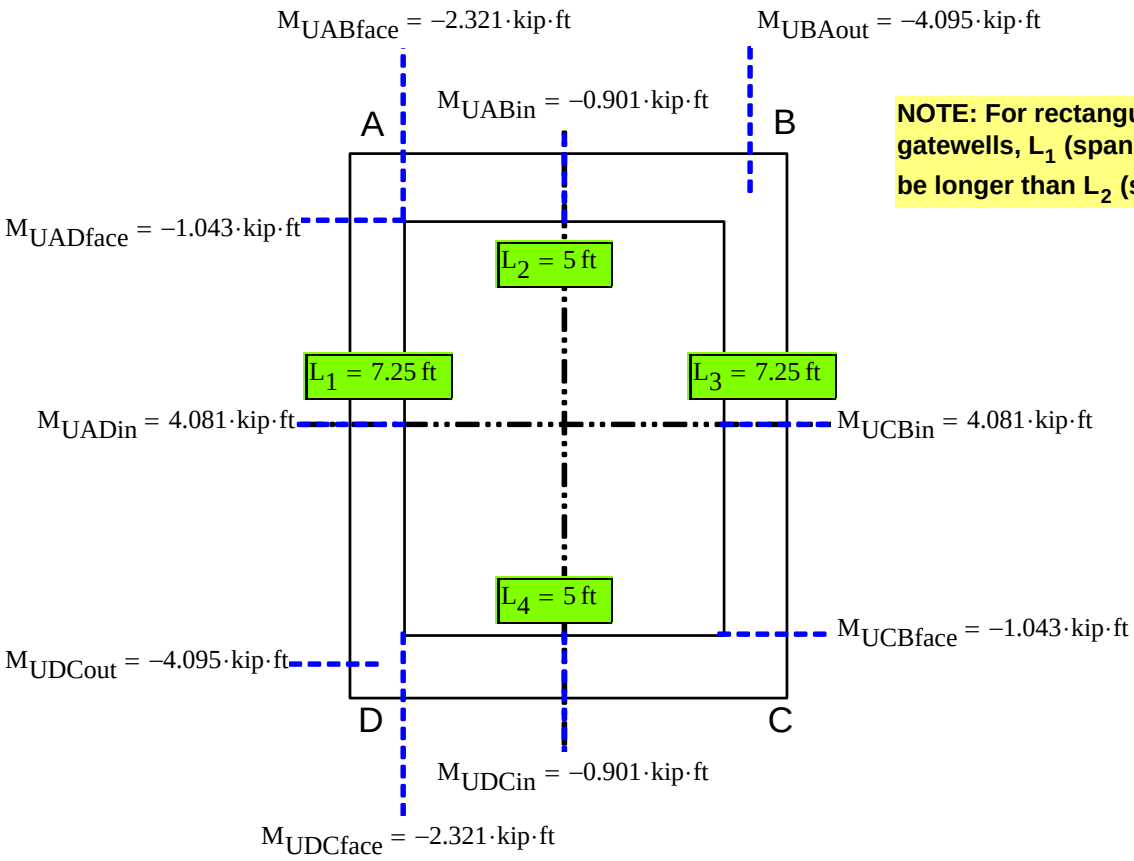


"D" represents the orthagonal wall thickness

$M_{UDCout} := M_{DCend}$ $M_{UDCout} = -4.095 \cdot kip \cdot ft$

$M_{UABout} := M_{ADend}$ $M_{UABout} = -4.095 \cdot kip \cdot ft$ $M_{UBAout} := M_{ADend}$

FACTORED MOMENT DISTRIBUTION VALUES



NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

The "L" values are the "outside-to-outside" dimensions.

Ensure the reinforcement size entered earlier reflects the steel that will be in tension at the mid-span.
For example:
IF: The mid-span moment demand value obtained from moment distribution is negative.
THEN: The steel located near the wall's exterior face is in tension. Select the reinforcement size that is present near the wall exterior face.

Wall Thrust

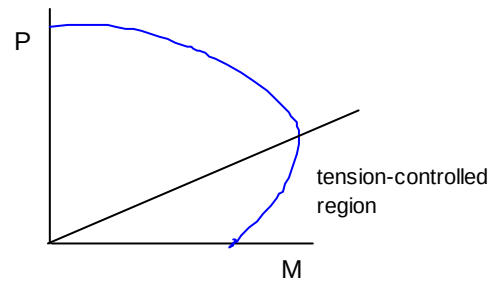
Thrust was considered when evaluating the wall section moment and shear capacity. A higher value of thrust within the tension-controlled region of the interaction diagram results in a higher nominal strength value. The calculated thrust is based upon the active (lower) value of lateral loading. This is deemed conservative relative to the at-rest (higher) value of lateral loading. Note that for lateral loading on the walls, the at-rest loading condition was used.

$K_a = 0.472$ $\gamma_{soil} = 110 \cdot pcf$ $H_2 = 19.42\ ft$

$$W_{1active} := \gamma_L \cdot \gamma_H \cdot K_a \cdot \gamma_{soil} \cdot H_2$$

$$W_{2active} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_a \cdot \left(\gamma_{soil} - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$$

$W_{1active} = 1009.045 \cdot psf$ $W_{2active} = 1660.375 \cdot psf$



$$Wactive := if\left(W_{1active} > W_{2active}, W_{1active}, W_{2active}\right) \cdot b$$

$Wactive = 1660.375 \cdot plf$ $W = 1816.853 \cdot plf$

Total Trust Acting On Wall AD:

$$P_{ADthrust} := \frac{Wactive \cdot L_2}{2}$$

$P_{ADthrust} = 4.151 \cdot kip$

Total Trust Acting On Wall AB:

$$P_{ABthrust} := \frac{Wactive \cdot L_1}{2}$$

$P_{ABthrust} = 6.019 \cdot kip$

Wall AD (Wall 1) Analysis

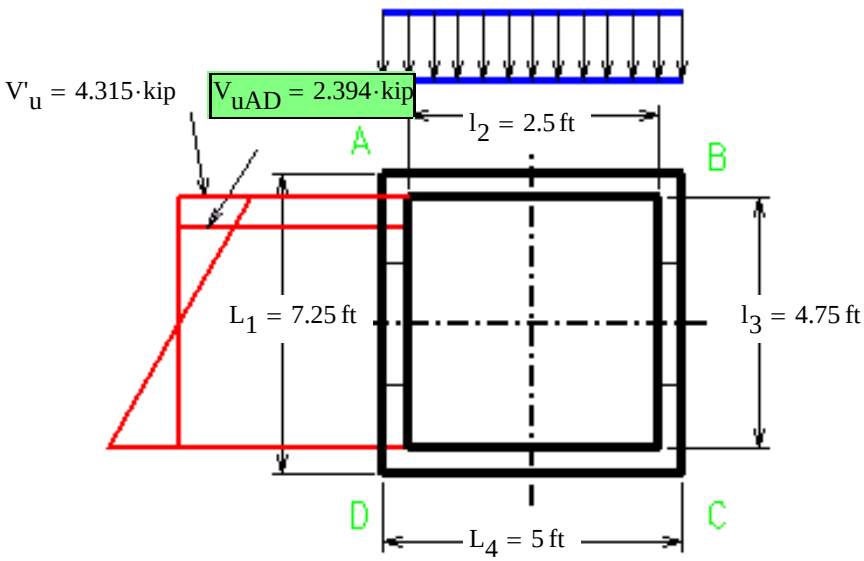
Shear Demand:

$W = 1816.853 \cdot plf \qquad l_1 = 4.75\,ft$

$V'_u := W \cdot \frac{l_1}{2} \qquad V'_u = 4.315 \cdot kip$

$d_{AD_out_end} = 12.688 \cdot in$

$V_{uAD} := V'_u \cdot \frac{\frac{l_1}{2} - d_{AD_out_end}}{\frac{l_1}{2}} \qquad V_{uAD} = 2.394 \cdot kip \qquad W = 1816.853 \cdot plf$



Shear Capacity and Factor of Safety:

$d_{AD_out_end} = 12.688 \cdot in \qquad \frac{P_{ADthrust}}{2000 \cdot A_{g1}} = 0.012\,psi \qquad f'_c = 3 \cdot ksi \quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$\phi V_{n_AD} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{P_{ADthrust}}{2000 \cdot A_{g1}} \cdot \frac{in^2}{lbf} \right) \cdot \left(\sqrt{f'_c \cdot psi} \right) \cdot \left(b \cdot d_{AD_out_end} \right) \right]$ ACI EQ (11-4)

$\phi V_{n_AD} = 16.87 \cdot kip$

$V_{uAD} = 2.394 \cdot kip$

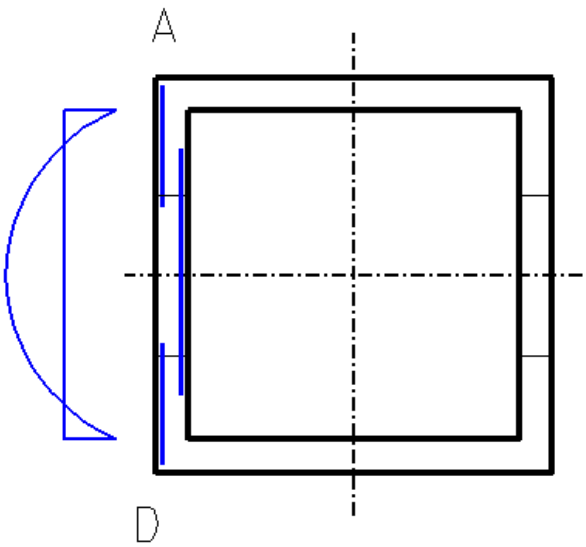
Factor of Safety $FS_{ADshear} := \frac{\phi V_{n_AD}}{V_{uAD}}$

$FS_{ADshear} = 7.047$

DEPTH OF EQUIVALENT RECTANGULAR STRESS BLOCK

Wall AD Con't

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\ ft$ $W = 1.817 \cdot klf$ $L_1 = 7.25\ ft$



$A_{ADout} = 0.31 \cdot in^2$ $a_{AD_out} := \frac{A_{ADout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ADin} = 0.2 \cdot in^2$ $a_{AD_in} := \frac{A_{ADin} \cdot F_y}{0.85 f'_c \cdot b}$

$a_{AD_out} = 0.405 \cdot in$ $a_{AD_in} = 0.261 \cdot in$

Negative Moment Redistribution

Wall AD Con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85 \qquad f'_c = 3000\,psi$

Beta one value is equal to 0.85 for f'c equal to or less than 4000 psi

$c_{AD_out} := \frac{a_{AD_out}}{\beta 1}$ $\quad c_{AD_out} = 0.477 \cdot in$ $\quad c_{AD_in} := \frac{a_{AD_in}}{\beta 1}$ $\quad c_{AD_in} = 0.308 \cdot in$

The net tensile strain values

$$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AD_out_end}}{c_{AD_out}} - 1 \right) \qquad \varepsilon_{tout} = 0.077$$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AD end support and increase the positive mid-span moment (if applicable)

$$R_{nface} := \frac{|M_{UADface}|}{\phi_B \cdot b \cdot d_{AD_out_end}^2} \qquad R_{nface} = 6.48 \cdot psi \qquad M_{UADface} = -1.043 \cdot kip \cdot ft$$

$M_{UADin} = 4.081 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\dfrac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UADin} > 0) \\ 0 & \text{otherwise} \end{cases} \qquad \varepsilon'_{tface} = 0.999$$

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$$\varepsilon'_ADfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases} \qquad \varepsilon'_ADfacedelta = 0.200 \qquad \varepsilon'_{tface} \cdot 10 = 9.992$$

$$RedistAD := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases} \qquad RedistAD = \text{"Permitted"}$$

IF: The strain is greater than or equal to 0.0075 and the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UADface} := \begin{cases} M_{UADface} \cdot (1 - \varepsilon'_ADfacedelta) & \text{if } \varepsilon'_ADfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADface} = -0.835 \cdot kip \cdot ft \qquad M_{UADface} = -1.043 \cdot kip \cdot ft$

Wall AD Con't

$$M'_{UADout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'_ADfacedelta) & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -3.276 \cdot kip \cdot ft$ $M_{UADface} = -1.043 \cdot kip \cdot ft$ $M_{UDCout} = -4.095 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UADin} := \begin{cases} \frac{W \cdot (L_1 - D_2)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UADout} & \text{if } M'_{UADout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -3.276 \cdot kip \cdot ft$

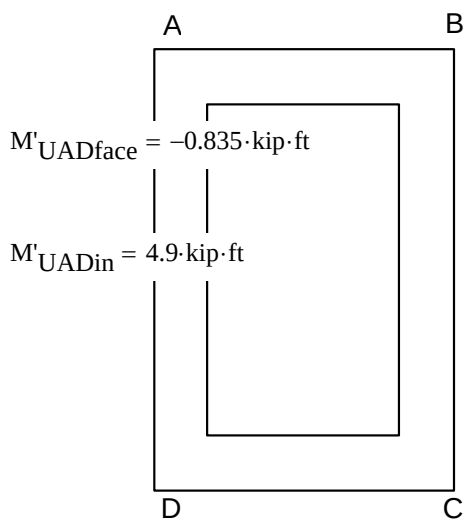
$M'_{UADin} = 4.9 \cdot kip \cdot ft$

$$\frac{W \cdot (L_1 - D_2)^2}{8} = 8.176 \cdot kip \cdot ft$$

$M_{UADin} = 4.081 \cdot kip \cdot ft$

Moments on Span AD After Redistribution

RedistAD = "Permitted"



Note: If redistribution is not allowed, "N/A" is shown.

Wall AD Magnified Moment Check

Wall AD Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k l_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, K values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UADface}$ $\qquad$ $M_2 := M_{UADface}$

$M_1 = -1.043 \cdot kip \cdot ft$ $\qquad$ $M_2 = -1.043 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_1$ $\qquad$ $l_u = 7.25\ ft$

Radius of Gyration, r:

Thickness of Wall: $D_1 = 1.25\ ft$

10.11.2 — It shall be permitted to take the radius of gyration r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_1$ $\qquad$ $r = 0.375\ ft$

$Slender_Ratio := k \cdot \frac{l_u}{r}$ $\qquad$ $Slender_Ratio = 11.6$

$$ADSlenderness_Check := if\left[Slender_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], \text{"Ok"}, \text{"Consider Slenderness using ACI 10.10.1"}\right]$$

$ADSlenderness_Check = \text{"Ok"}$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AD Mid-Span Nominal Moment Capacity

$L_1 = 7.25\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 15 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ADin} = 4$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADin = 2 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar_No_{ADin} = 4$ $diaADin = 0.5 \cdot in$

$row := 2$

Iterations $i := 1 .. 2$ $i =$

1
2

Bar area at i $A_1 := A_{ADout}$ $A_2 := A_{ADin}$ $A_i =$

0.31
0.2

$\cdot in^2$

$coverADout = 2 \cdot in$ $diaADout = 0.625 \cdot in$ $coverADin = 2 \cdot in$ $diaADin = 0.5 \cdot in$

$$dd_1 := \begin{cases} coverADout + \dfrac{diaADout}{2} & \text{if } M_{UADin} > 0ft \cdot kip \\ coverADin + \dfrac{diaADin}{2} & \text{otherwise} \end{cases}$$

$M_{UADin} = 4.081 \cdot kip \cdot ft$

$dd_1 = 2.313 \cdot in$

$$dd_2 := \begin{cases} D_1 - \left(coverADin + \dfrac{diaADin}{2}\right) & \text{if } M_{UADin} > 0ft \cdot kip \\ D_1 - \left(coverADout + \dfrac{diaADout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 12.75 \cdot in$

Wall AD, Midspan Column Interaction Results

Wall AD Midspan

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.25\,ft \quad b = 1\,ft \quad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \quad c = 3 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{l} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \le -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B$$

$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \quad c = 0.944 \cdot in \quad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \quad a = 0.802 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.193 ft	-0.004	-40 ·ksi	0.31 $\cdot in^2$	-12.4 ·kip	-64.325 ·kip·in
1.063	-0.038	-40	0.2	-8	42

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = -22 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 174 \cdot kip \cdot in$

$\phi M_{nADin1} := (Mc + Mst) \qquad \phi M_{nADin1} = 12.66 \cdot kip \cdot ft$

$$\phi M_{nADin} := \begin{cases} \phi M_{nADin1} & \text{if } M_{UADin} > 0ft \cdot kip \\ \phi M_{nADin1} \cdot (-1) & \text{otherwise} \end{cases}$$

Wall AD Mid-Span Nomimal Moment Strength $\quad \phi M_{nADin} = 12.663 \cdot kip \cdot ft$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADmid}

Wall AD Mid-Span Con't

$\phi M_{nADin} = 12.663 \cdot kip \cdot ft$

$M_{UADin} = 4.081 \cdot kip \cdot ft$

$$FS_{ADin} := \left(\left| \frac{\phi M_{nADin}}{M_{UADin}} \right| \right)$$

$FS_{ADin} = 3.103$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ADmid}$

$\phi M_{nADin} = 12.663 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UADin} = 4.9 \cdot kip \cdot ft$

$$FS'_{ADin} := \begin{cases} \left| \frac{\phi M_{nADin}}{M'_{UADin}} \right| & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADin} = 2.584$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADinspanCheck := if (ADSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ADinspanCheck = "OKAY"

Wall AD End-of-Span Nominal Moment Capacity

$L_1 = 7.25\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 15 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ADout} = 5$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADout = 2 \cdot in$ Cover -in

$Bar_No_{ADout} = 5$ $diaADin = 0.5 \cdot in$

$A_1 := A_{ADin}$ $A_2 := A_{ADout}$ $A_i =$

0.2
0.31

$\cdot in^2$

$coverADout = 2 \cdot in$ $diaADout = 0.625 \cdot in$

$dd_1 := d_{AD_in_end}$ $dd_1 = 2.25 \cdot in$

$dd_2 := d_{AD_out_end}$ $dd_2 = 12.688 \cdot in$

Wall AD, End-of-Span Column Interaction Results

Wall AD End-of-Span

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.25\,ft \quad b = 1\,ft$ $\quad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \quad c = 3 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \le -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B$$
$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \quad c = 0.944 \cdot in \quad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \quad a = 0.802 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.188 ft	-0.004	-40 ·ksi	0.2 $\cdot in^2$	-8 ·kip	-42 ·kip·in
1.057	-0.037	-40	0.31	-12.4	64.325

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 22 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 174 \cdot kip \cdot in$

$\phi M_{nADface1} := (Mc + Mst) \qquad \phi M_{nADface1} = 16.38 \cdot kip \cdot ft$

$\phi M_{nADface} := \begin{cases} \phi M_{nADface1} & \text{if } M_{UADface} > 0ft \cdot kip \\ \phi M_{nADface1} \cdot (-1) & \text{otherwise} \end{cases} \qquad M_{UADface} = -1.043 \cdot kip \cdot ft$

Wall AD End-of-Support Nominal Moment Strength $\quad \phi M_{nADface} = -16.384 \cdot kip \cdot ft$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADface}

$\phi M_{nADface} = -16.384 \cdot kip \cdot ft$ $M_{UADface} = -1.043 \cdot kip \cdot ft$

$$FS_{ADface} := \left(\left| \frac{\phi M_{nADface}}{M_{UADface}} \right| \right)$$

$FS_{ADface} = 15.707$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ADface}$

$$FS'_{ADface} := \begin{cases} \left| \frac{\phi M_{nADface}}{M'_{UADface}} \right| & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADface} = 19.633$

The end-of-span factor of safety always increases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADoutspanCheck := if (ADSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

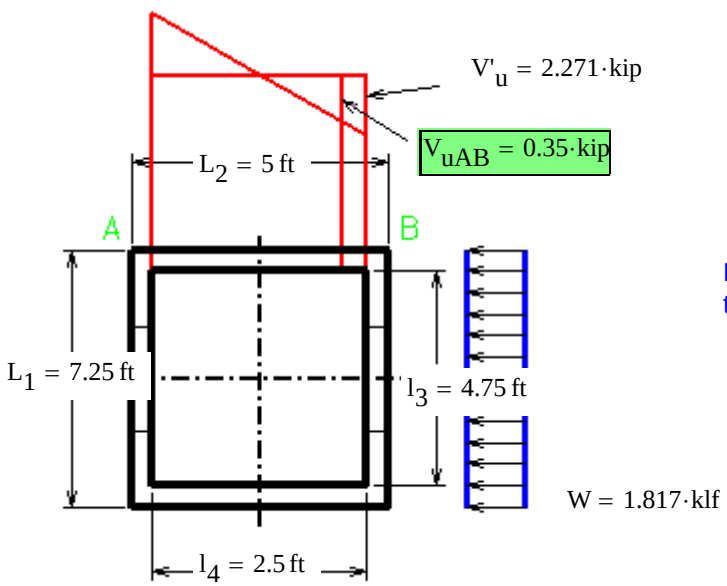
ADoutspanCheck = "OKAY"

Wall AB (Wall 2) Analysis

Shear Demand:

$W = 1816.853\,ft\cdot psf$ $\quad l_2 = 2.5\,ft$ $\quad d_{AB_out_end} = 12.688\cdot in$

$V'_u := W\cdot\frac{l_2}{2}$ $\quad V'_u = 2.271\cdot kip$ $\quad V_{uAB} := V'_u\cdot\frac{\frac{l_2}{2} - d_{AB_out_end}}{\frac{l_2}{2}}$ $\quad V_{uAB} = 0.35\cdot kip$



Total Thrust into Wall2:

$P_{ABthrust} = 6.019\cdot kip$

Note: Refer to wall AD calculations for derivation of the thrust values acting on wall AB.

Shear Capacity and Factor of Safety

$d_{AB_out_end} = 12.688\cdot in$ $\quad \frac{P_{ABthrust}}{2000\cdot A_{g2}} = 0.017\,psi$ $\quad f'_c = 3000\,psi$ $\quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$$\phi V_{n_AB} := \phi_V\cdot\left[2\cdot\left(1 + \frac{P_{ABthrust}}{2000\cdot A_{g2}}\cdot\frac{in^2}{lbf}\right)\cdot\left(\sqrt{f'_c\cdot psi}\right)\cdot\left(b\cdot d_{AB_out_end}\right)\right]$$

ACI EQ (11-4) $\quad \phi V_{n_AB} = 16.957\cdot kip$

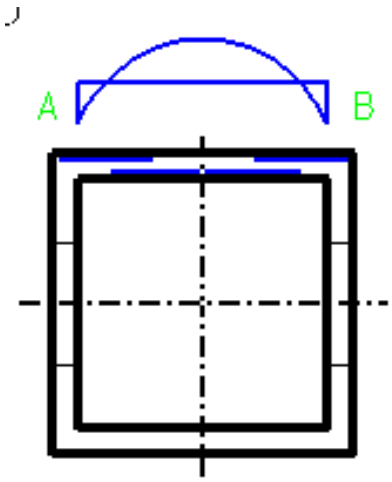
$V_{uAB} = 0.35\cdot kip$

Factor of Safety $\quad FS_{ABshear} := \frac{\phi V_{n_AB}}{V_{uAB}}$ $\quad FS_{ABshear} = 48.432$

Wall AB con't

Depth of Equivalent Rectangular Stress Block

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\ ft$ $W = 1.817 \cdot klf$ $L_1 = 7.25\ ft$



$A_{ABout} = 0.31 \cdot in^2$ $aABout := \frac{A_{ABout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ABin} = 0.2 \cdot in^2$ $aABin := \frac{A_{ABin} \cdot F_y}{0.85 f'_c \cdot b}$

$aABout = 0.405 \cdot in$ $aABin = 0.261 \cdot in$

Negative Moment Redistribution

Wall AB con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85$ $\quad f'_c = 3000\,psi$

$cABout := \frac{aABout}{\beta 1}$ $\quad cABout = 0.477 \cdot in$ $\quad cABin := \frac{aABin}{\beta 1}$ $\quad c_{AD_in} = 0.308 \cdot in$

The net tensile strain values

$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AB_out_end}}{cABout} - 1 \right)$ $\quad \varepsilon_{tout} = 0.077$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AB end support and increase the positive mid-span moment (if applicable)

$R_{nface} := \frac{|M_{UABface}|}{\phi_B \cdot b \cdot d_{AB_out_end}^2}$ $\quad R_{nface} = 14.416 \cdot psi$ $\quad M_{UABface} = -2.321 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\frac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UABin} > 0) \\ 0 & \text{otherwise} \end{cases}$$

$\varepsilon'_{tface} = 0$

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$$\varepsilon'_ABfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases}$$

$\varepsilon'_ABfacedelta = 0.000$ $\quad \varepsilon'_{tface} \cdot 10 = 0$

$$RedistAB := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases}$$

RedistAB = "Not Permitted"

IF: The strain is greater than or equal to 0.0075 and the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UABface} := \begin{cases} M_{UABface} \cdot (1 - \varepsilon'_ABfacedelta) & \text{if } \varepsilon'_ABfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABface} = \text{"N/A"} \cdot kip \cdot ft$ $\quad M_{UABface} = -2.321 \cdot kip \cdot ft$

Wall AB con't

$$M'_{UABout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'_ABfacedelta) & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABout} = \text{"N/A"} \cdot kip \cdot ft$ $M_{UABface} = -2.321 \cdot kip \cdot ft$ $M_{UDCout} = -4.095 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UABin} := \begin{cases} \dfrac{W \cdot (L_2 - D_1)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UABout} & \text{if } M'_{UABout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

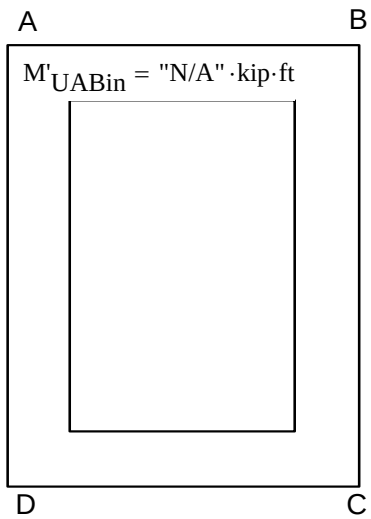
$M'_{UABin} = \text{"N/A"} \cdot kip \cdot ft$

$$\frac{W \cdot (L_2 - D_1)^2}{8} = 3.194 \cdot kip \cdot ft \quad M_{UABin} = -0.901 \cdot kip \cdot ft$$

$M'_{UABin} = \text{"N/A"} \cdot kip \cdot ft$

Moments on Span AD After Redistribution RedistAB = "Not Permitted"

$M'_{UABface} = \text{"N/A"} \cdot kip \cdot ft$



Note: If redistribution is not allowed, "N/A" is shown.

Wall AB Magnified Moment Check

Wall AB Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k l_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, K values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UABface}$ $\qquad$ $M_2 := M_{UABface}$

$M_1 = -2.321 \cdot kip \cdot ft$ $\qquad$ $M_2 = -2.321 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_2$ $\qquad$ $l_u = 5\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_2 = 1.25\,ft$

10.11.2 — It shall be permitted to take the radius of gyration r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_2$ $\qquad$ $r = 0.375\,ft$

$Slender_Ratio := k \cdot \frac{l_u}{r}$ $\qquad$ $Slender_Ratio = 8$

$$ABSlenderness_Check := if\left[Slender_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], "Ok", "Consider\ Slenderness\ using\ ACI\ 10.10.1"\right]$$

$ABSlenderness_Check = "Ok"$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AB Mid-Span Nominal Moment Capacity

$L_2 = 5\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 15 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ABin} = 4$ **Tension** bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABin = 2 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar_No_{ABin} = 4$ $diaABin = 0.5 \cdot in$

$diaABout = 0.625 \cdot in$

Bar area at i $A_1 := A_{ABout}$ $A_2 := A_{ABin}$ $A_i =$

$$\begin{array}{|r|} \hline 0.31 \\ \hline 0.2 \\ \hline \end{array} \cdot in^2$$

$coverABout = 2 \cdot in$ $diaABout = 0.625 \cdot in$ $coverABin = 2 \cdot in$ $diaABin = 0.5 \cdot in$

$$dd_1 := \begin{cases} coverABout + \dfrac{diaABout}{2} & \text{if } M_{UABin} > 0ft \cdot kip \\ coverABin + \dfrac{diaABin}{2} & \text{otherwise} \end{cases}$$

$M_{UABin} = -0.901 \cdot kip \cdot ft$

$dd_1 = 2.25 \cdot in$

$$dd_2 := \begin{cases} D_2 - \left(coverABin + \dfrac{diaABin}{2}\right) & \text{if } M_{UABin} > 0ft \cdot kip \\ D_2 - \left(coverABout + \dfrac{diaABout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 12.688 \cdot in$

$coverABout = 2 \cdot in$ $diaABout = 0.625 \cdot in$

$dd_1 := d_{AB_out_mid}$ $dd_1 = 2.313 \cdot in$

$dd_2 := d_{AB_in_mid}$ $dd_2 = 12.75 \cdot in$

Wall AB, Mid-Span Column Interaction Results

Wall AB Mid-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 15\cdot in \quad b = 1\,ft$ $\quad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3\cdot in$

$$P_{ABthrust} = \left[\sum_{i=1}^{row}\left[\left[\begin{array}{ll} \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ +-\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{array}\right]\cdot A_i\right] \ldots \\ +0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b \end{array}\right]\cdot\phi_B$$

$c := \text{Find}(c) \quad c = 1.016\cdot in \quad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \quad a = 0.863\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.193 ft	-0.004	-40 ·ksi	0.31 $\cdot in^2$	-12.4 ·kip	-64.325 ·kip·in
1.062	-0.035	-40	0.2	-8	42

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = -22\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 187\cdot kip\cdot in$

$\phi M_{nABin1} := (Mc + Mst) \qquad \phi M_{nABin1} = 13.7\cdot kip\cdot ft$

$$\phi M_{nABin} := \begin{cases} \phi M_{nABin1} & \text{if } M_{UABin} > 0ft\cdot kip \\ \phi M_{nABin1}\cdot(-1) & \text{otherwise} \end{cases}$$

Wall AB Mid-Span Nominal Moment Strength $\qquad \phi M_{nABin} = -13.701\cdot kip\cdot ft$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABmid}

Wall AB Mid-Span Con't

$\phi M_{nABin} = -13.701 \cdot kip \cdot ft$

$M_{UABin} = -0.901 \cdot kip \cdot ft$

$$FS_{ABin} := \left(\left| \frac{\phi M_{nABin}}{M_{UABin}} \right| \right)$$

$FS_{ABin} = 15.203$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ABmid}$

$\phi M_{nABin} = -13.701 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UABin} = \text{"N/A"} \cdot kip \cdot ft$

$$FS'_{ABin} := \begin{cases} \left| \frac{\phi M_{nABin}}{M'_{UABin}} \right| & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABin} = \text{"N/A"}$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Not Permitted"

ABinspanCheck := if (ABSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ABinspanCheck = "OKAY"

Wall AB End-of-Span Nominal Moment Capacity

$L_2 = 5\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 15 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ABout} = 5$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABout = 2 \cdot in$ Cover -in

$Bar_No_{ABout} = 5$ $diaABin = 0.5 \cdot in$

$A_1 := A_{ABin}$ $A_2 := A_{ABout}$

$A_i =$

0.2
0.31

$\cdot in^2$

<==The first value of A1 is the compression steel, A2 is the tension steel

$coverABout = 2 \cdot in$ $diaABout = 0.625 \cdot in$

$dd_1 := d_{AB_in_end}$ $dd_1 = 2.25 \cdot in$ Effective depth of the compression steel

$dd_2 := d_{AB_out_end}$ Effective depth of the tension steel

Wall AB, End-of-Span Column Interaction Results

Wall AB End-of-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 15\cdot in \quad b = 1\,ft$ $\quad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3\cdot in$

$$P_{ABthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ +-\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{array}\right]\cdot A_i\right] \ldots \right]\cdot\phi_B$$

$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c) \quad c = 1.016\cdot in \quad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \quad a = 0.863\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$		$\varepsilon s_i =$	$fs_i =$		$A_i =$		$Fs_i =$		$Ms_i =$	
0.188	ft	-0.004	-40	·ksi	0.2	$\cdot in^2$	-8	·kip	-42	·kip·in
1.057		-0.034	-40		0.31		-12.4		64.325	

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 22\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 187\cdot kip\cdot in$

$\phi M_{nABface1} := (Mc + Mst) \qquad \phi M_{nABface1} = 17.42\cdot kip\cdot ft$

$$\phi M_{nABface} := \begin{cases} \phi M_{nABface1} & \text{if } M_{UABface} > 0ft\cdot kip \\ \phi M_{nABface1}\cdot(-1) & \text{otherwise} \end{cases}$$

Wall AB End-of-Span Nominal Moment Strength $\quad \phi M_{nABface} = -17.422\cdot kip\cdot ft$

Wall AB End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABface}

$\phi M_{nABface} = -17.422 \cdot kip \cdot ft$ $M_{UABface} = -2.321 \cdot kip \cdot ft$

$$FS_{ABface} := \left(\left| \frac{\phi M_{nABface}}{M_{UABface}} \right| \right)$$

$FS_{ABface} = 7.507$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ABface}$

$\varepsilon'_ABfacedelta = 0$

$$FS'_{ABface} := \begin{cases} \left| \frac{\phi M_{nABface}}{M'_{UABface}} \right| & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABface} = \text{"N/A"}$

The end-of-span factor of safety always increases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Not Permitted"

ABoutspanCheck := if (ABSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ABoutspanCheck = "OKAY"

Strength Analysis Results Summary

Span AD Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ADface} = 15.707$ $FS_{ADin} = 3.103$ $FS'_{ADface} = 19.633$ $FS'_{ADin} = 2.584$

$FS_{ADno_redist} := \min(FS_{ADface}, FS_{ADin})$ $FS'_{ADredist} := \min(FS'_{ADface}, FS'_{ADin})$

$FS_{ADno_redist} = 3.103$ $FS'_{ADredist} = 2.584$

$$FS_{ADmoment} := \begin{cases} \max(FS_{ADno_redist}, FS'_{ADredist}) & \text{if } FS'_{ADredist} \neq \text{"N/A"} \\ FS_{ADno_redist} & \text{otherwise} \end{cases}$$

$FS_{ADin} = 3.103$

$FS_{ADmoment} = 3.103$

$$AD_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Recommended"} & \text{otherwise} \end{cases}$$

AD_Recommendation1 = "Perform Simple Beam Analysis"

$$AD_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ADmoment} < 1.5 \\ \text{"Span AD OKAY for Moment Strength"} & \text{if } FS_{ADmoment} \geq 1.5 \end{cases}$$

AD_Recommendation2 = "Span AD OKAY for Moment Strength"

A listing of the span AD moment demand-capacity values, factors of safety and recommendations are provided in the following table

δ Scalar Conversion Equations To Allow Text Entry Into Tables

Scalar Conversion Equations To Allow Text Entry Into Tables

Span AD Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-1.04	-16.38	15.71	4.08	12.66	3.10
Redistribution of Negative Moments	-0.83		19.63	4.90		2.58
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AD OKAY for Moment Strength						

N/A: Not Applicable

Span AB Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ABno_redist} := \min(FS_{ABface}, FS_{ABin})$ $FS'_{ABredist} := \min(FS'_{ABface}, FS'_{ABin})$

$FS_{ABno_redist} = 7.507$ $FS'_{ABredist} = \text{"N/A"}$

$$FS_{ABmoment} := \begin{cases} \max(FS_{ABno_redist}, FS'_{ABredist}) & \text{if } FS'_{ABredist} \neq \text{"N/A"} \\ FS_{ABno_redist} & \text{otherwise} \end{cases}$$

$FS_{ABin} = 15.203$

$FS_{ABmoment} = 7.507$

$$AB_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Recommended"} & \text{otherwise} \end{cases}$$

AB_Recommendation1 = "Perform Simple Beam Analysis"

$$AB_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ABmoment} < 1.5 \\ \text{"Span AB OKAY for Moment Strength"} & \text{if } FS_{ABmoment} \geq 1.5 \end{cases}$$

AB_Recommendation2 = "Span AB OKAY for Moment Strength"

A listing of the span AB moment demand-capacity values, factors of safety and recommendations are provided in the following table

AB Scalar Conversion for Table

Span AB Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-2.32	-17.42	7.51	-0.90	-13.70	15.20
Redistribution of Negative Moments	N/A		N/A	N/A		N/A
Slenderness Check	Ok					
Redistribution of Negative Moments	Not Permitted					
Recommendations: Perform Simple Beam Analysis Span AB OKAY for Moment Strength						

N/A: Not Applicable

Shear Factor of Safety Analysis Summary and Recommendations

Span AD

$$FS_{ADshear} = 7.047$$

Span AB

$$FS_{ABshear} = 48.432$$

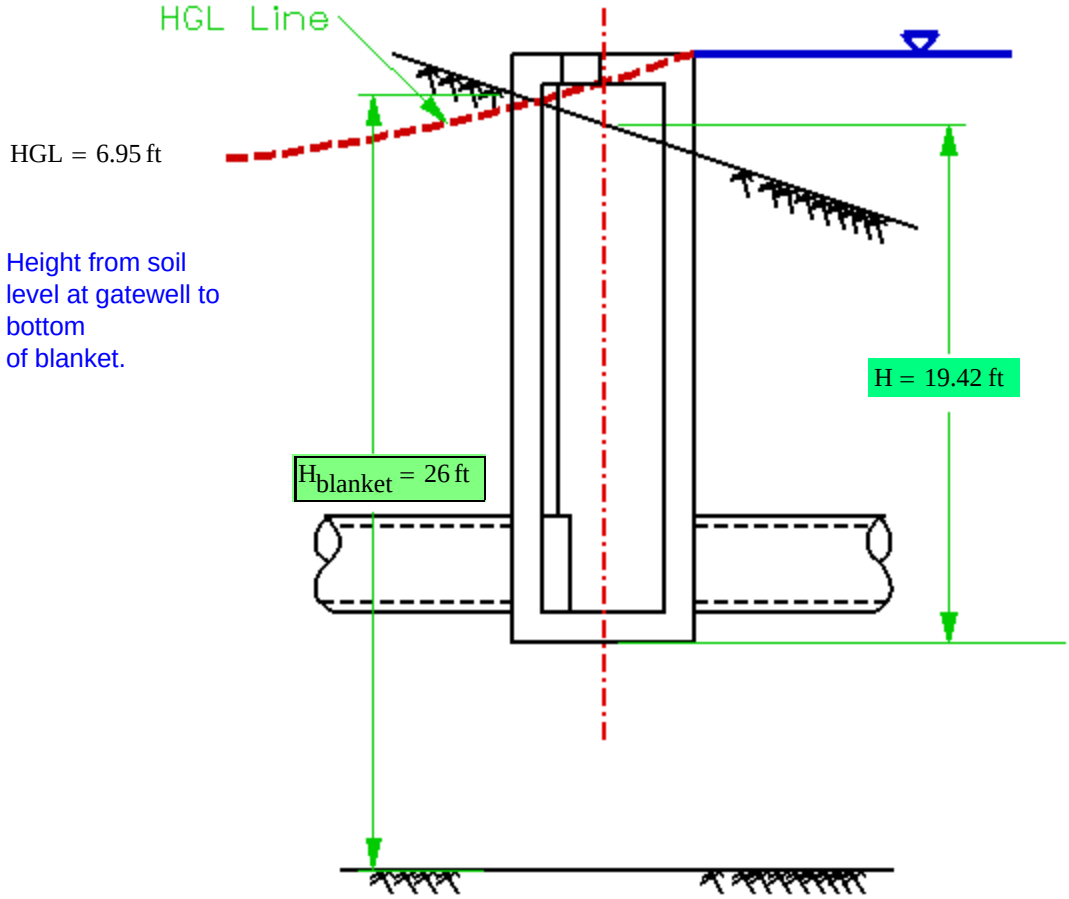
NOTES TO STRENGTH SUMMARY RESULTS:

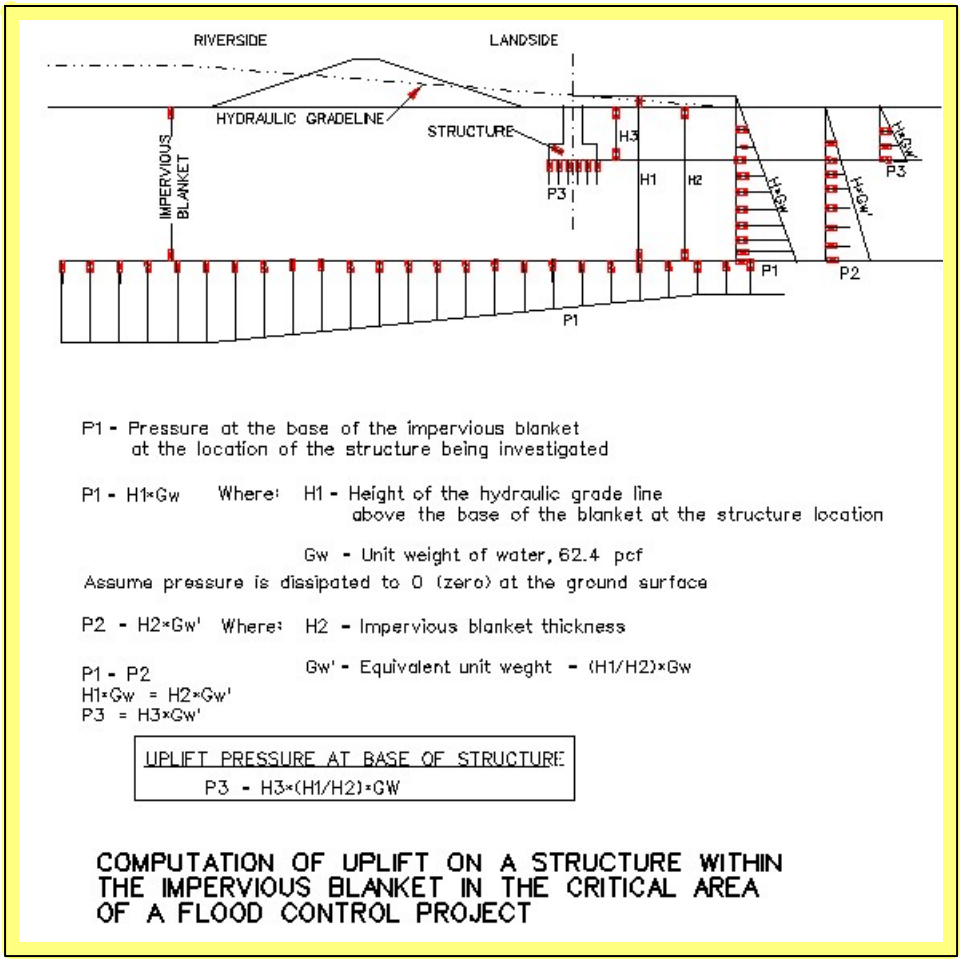
1. If simple beam analysis is performed on one wall, it must be performed on all four gatewell walls. This is required because fixity is lost when an adjacent wall is analyzed as a simple span. Stability should be considered when the gatewell is analyzed as having pinned supports.
2. Moment redistribution is not permitted when the mid-span and end-support moment values are negative.
3. Moment redistribution is not permitted when the reinforcement strain is less than 0.0075
4. Simple span analysis is recommended when the mid-span moment factor of safety is greater than 2.0. Final decision to use simple span analysis rests with the engineer.
5. Reliability analysis is recommended when the 1.0 < FS < 1.5
6. Caution is provided in the recommendations when the FS < 1.0

Uplift Analysis:

Uplift Using Local Protection Guidance

UPLIFT





<< =from KC levees uplift guidance.

$A_{base} := L_1 \cdot L_2$ $\qquad A_{base} = 36.25\ ft^2$

Dissipated Head, if structural foundation in blanket.

$H_{blanket} = 26\ ft$ $\quad HGL = 6.95\ ft$ $\quad H = 19.42\ ft$ $\quad L_1 = 7.25\ ft$ $\quad L_2 = 5\ ft$

$$U_1 := \left(\frac{H_{blanket} + HGL}{H_{blanket}} \cdot H\right) \cdot L_1 \cdot L_2 \cdot \gamma_w \qquad U_1 = 55.7 \cdot kip$$

Full Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (L_1 \cdot L_2) \cdot \gamma_w\right] \qquad U_2 = 59.6 \cdot kip$$

Uplift

$H = 19.42\,ft$ $H_{blanket} = 26\,ft$ $H' = 8.3\,ft$

$U := if(H > H_{blanket}, U_2, U_1)$ $U = 55.7 \cdot kip$

$U_{uplift_Method1} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(\frac{U}{L_1 \cdot L_2}\right)$ $U_{uplift_Method1} = 1536 \cdot psf$

$Walls := (L_1 \cdot L_2 - l_1 \cdot l_2) \cdot (H + H') \cdot \gamma_c$ $Slabs := 2 \cdot (l_1 \cdot l_2) \cdot D_f \cdot \gamma_c$ $W_S := \gamma_L \cdot \gamma_H \cdot \gamma_X (Walls + Slabs)$ $W_S = 106.7 \cdot kip$

$W_{UNIFORM} := \frac{W_S}{L_1 \cdot L_2}$ $W_{UNIFORM} = 2943 \cdot psf$

Surcharge Loads $S := 0lb$

Weight of Surcharge Water $W_g := 0kip$

$W_S = 106.695 \cdot kip$ $S = 0$ $W_g = 0$

Final Uplift Factor of Safety:

$U_{uplift_Method1} \cdot (L_1 \cdot L_2) = 55.67 \cdot kip$

ETL 1110-2-307 $FS_{Uplift} := \left[\frac{W_S + S}{U_{uplift_Method1} \cdot (L_1 \cdot L_2) - W_g}\right]$ $FS_{Uplift} = 1.917$

NOTE: EM 1110-2-2100 defines 1.1 Factor of Safety acceptable for extreme loading case. The gatewells have minimal overhangs to produce weight of water above structure, W_G, therefore W_G ignored (conservative).

Base Slab Analysis

Plate Loading

$k := 1 .. 5$

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.526$

From plates tables:

a/b	Coefficient
1	.05
.875	.059
.75	.069
.625	.077
.5	.082

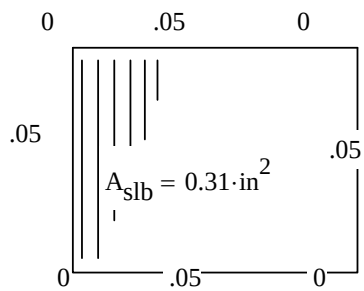
$ab_k :=$

0.5
.625
.75
.875
1

$Coeff_k :=$

.082
.077
.069
.059
.05

$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right)$



$W_{UNIFORM} = 2943.31 \cdot psf$ $b = 1\,ft$

$l_2 = 2.5\,ft$

$M_{uF} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$

$M_{uF} = 1.489 \cdot kip \cdot ft$

$d_f = 1.25\,ft$

$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b}$ $a = 0.405 \cdot in$

$\phi M_{nF} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nF} = 15.291\,ft \cdot kip$

$Slab_Bending_Check := if\left(\phi M_{nF} > M_{uF}, "OK", "NO\ GOOD"\right)$ $Slab_Bending_Check = "OK"$

Factor of Safety $FS_{slabmoment} := \frac{\phi M_{nF}}{M_{uF}}$ $FS_{slabmoment} = 10.268$

**********Overall Factors of Safety**********

Uplift Stability

$FS_{Uplift} = 1.9$

Strength

Wall AD $L_1 = 7.25\ ft$

Wall AB $L_2 = 5\ ft$

WALL MOMENT

The overall wall moment factor of safety, $FS_{wallmoment}$

$FS_{ADmoment} = 3.103$ $FS_{ABmoment} = 7.507$

$FS_{wallmoment} := \min(FS_{ADmoment}, FS_{ABmoment})$ $FS_{wallmoment} = 3.103$

WALL SHEAR

$FS_{ADshear} = 7.047$ $FS_{ABshear} = 48.432$

$FS_{wallshear} := \min(FS_{ADshear}, FS_{ABshear}) = 7.047$ $FS_{wallshear} = 7.047$

BASE SLAB MOMENT FACTOR OF SAFETY

$FS_{slabmoment} = 10.268$

Summary :

$FS_{strength} := \min(FS_{wallmoment}, FS_{wallshear}, FS_{slabmoment})$ $FS_{strength} = 3.1$

$$Mechanism := \begin{cases} \text{"Wall Bending"} & \text{if } FS_{strength} = FS_{wallmoment} \\ \text{"Wall Shear"} & \text{if } FS_{strength} = FS_{wallshear} \\ \text{"Base Slab Moment"} & \text{otherwise} \end{cases}$$

Mechanism = "Wall Bending"

$FS_{Uplift} = 1.9$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. The reasoning being the load factor (live load neglecting hydraulic structure) divided by the strength reduction factor is approximately 1.75. Because this is an existing structure in the field that is routinely inspected with no visible history of problems, a factor of safety of 1.5 is deemed acceptable. If modifications are required for strength, the modified structure must be evaluated from a standard design approach (include factors - no FS calculation)

END OF ANALYSIS

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By _____ Date _____ Project _____ Sheet ____ of ____

Chkd. By RB Date 2/25/08 Subject Mistletoe Comments Sheet ____ of ____

For Mistletoe the HGI & bottom of blanket BOB elevation value changed. The HGI went down & the BOB value went down too. The change in wall loading shown on your pdf file is based on a change in HGI alone. The new wall loading should be calculated taking into account the revised HGI and BOB elevation.

It is unlikely that the above revisions will make a large difference.

We need to document ~~wh~~ how the W = 2143 plf (original) value was obtained as well as the revised value.

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By EAF Date 4/18/08 Project Mistletoe P.S. Sheet 1 of 1

Chkd. By _____ Date _____ Subject F.C. Load w/new Hgl of 759.29' Sheet _____ of _____

Calculate new load on wall due to change in Hgl elevation from 760' to 759.29'.

$\boxed{W_{(760)} = 2143 \text{ plf using Hgl of } 760'}$

$K_a = .642$

$\gamma_s = 110 \text{ pcf}$

$\gamma_w = 62.4 \text{ pcf}$

Approximate loading:

Water load $= W_w = (759.29 - 737)\gamma_w = 22.29(62.4) = 1390.9 \text{ plf}$

Soil Load $= W_s = (756 - 737)(110 - 62.4)(.642) = 580.6 \text{ plf}$

$W = W_s + W_w = \underline{1971.5 \text{ plf}}$

The original Hgl value of 760' was used because information available from Geotechnical was DRAFT Hgl's. Since then the Hgl's have been revised and been reviewed, and the new Hgl is actually lower than the DRAFT Hgl. Therefore in light of this new information the Mistletoe P.S. is considered Reliable for future Conditions F.C.

EAF 4/18/08

EC-GD CONCURS 4/22/08 RSK

The BOB elevation ~~wont~~ changed. How did this change impact W?

MRK FORM 222

Parks, Marvin L NWK

From: Parks, Marvin L NWK
Sent: Friday, April 25, 2008 3:32 PM
To: Fernandez, Eduardo H NWK
Subject: Mistletoe FC Peer Review

Eddie;
I have looked over the FC analysis. I have the following.

For Mistletoe, the HGL and bottom of blanket (BOB) elevation values posted by EC-GD have been revised. The HGL for the FC has gone down and the BOB elevation has also gone down.

The Adobe file outlines the effect on wall loading with revised (lower) HGL value. Since the HGL went down, the loading also went down. However, the calculations should take into account the change in BOB elevation value. Recommend that the calculations include the revised HGL and BOB values in determining the new wall loading.

I could not find the calculations used to obtain the 2143 plf-W value. (I looked in the folder). We should have this calculation documented. A cut and paste from one of the MathCad sheets showing those equations should work.

Additional fill is not assumed. The 18" CIP should be inspected due to its age and type of material. Place the pipe recommendations in the EC file in the FC file.
Marvin

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

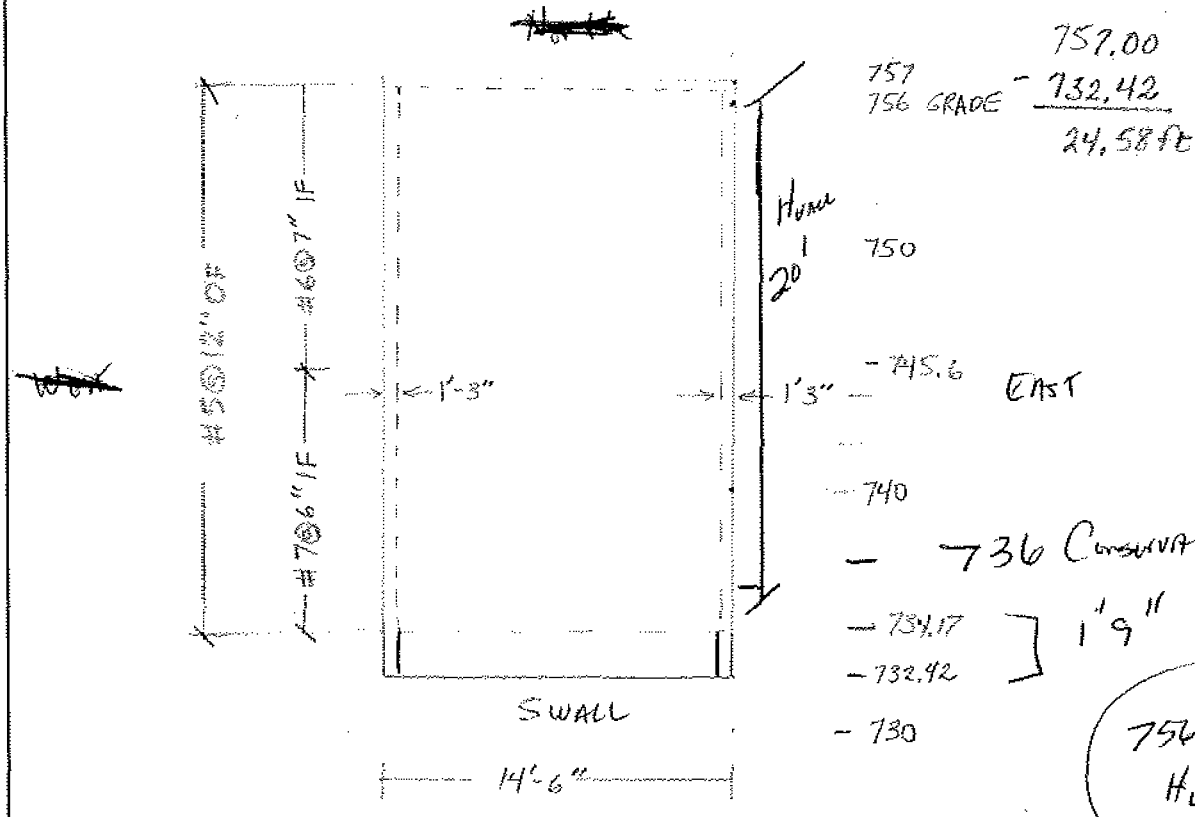
Comp. By MP Date 6/7/07 Project KCL Phase 2 CID Pump Stations Sheet 3 of

Chkd. By Date Subject Mattetoe Analysis Sheet of

South Pump Pit Wall

760 — HGL EL

Wall Thickness = 1'-3"



Vertical Reinf. #5 @ 12" EF ∴ clear cover 2½" EF (2" EFF Sheet #35)

Horizontal see A-A

Preliminary Analysis:

Assume 1 way action, w/ + reinf. carrying all of the load given the reinforcement design

Approximate Loading

Blanket thickness = 756 - 725 = 31' = H_2 ∴ HGL_{elev} = 760

H_1 = 760 - 725 = 35'

H_3 = 756 - 732.42 = 23.6'

$K_o = 1 - \sin\phi = 1 - \sin 26°$
$= 0.56$

Water Load = $W_w = (760 - 736)\gamma_w = 24'(62.4\ lb/ft^3) = 1498$ plf

Soil Load = $W_s = (756 - 736)(120 - 62.4)(0.56) = 645$ plf

$W = W_w + W_s = 2143$ plf

14.5 - 1.25 = 13.25

13.25 ℄ of wall to ℄

$M = \frac{wL^2}{8} = \frac{2143(13.25)}{8} = 47.0$ k·ft

$A_s = 1.2\ in^2$ ∴ $f_y = 40$ ksi ∴ $f_c = 3$ ksi

$d = 15" - (2.5 + \frac{0.875}{2}) = 12.06$

$a = \frac{A_s f_y}{0.85 f_c b} = \frac{1.2(40)}{0.85(3)12} = 1.569"$

Capacity ∴ $M_n = A_s f_y (d - a/2)$

$= 1.2(40)(12.06 - \frac{1.57}{2})$

$= 541$ k·in $= 45.1$ k·ft

1745 lb/ft = 1.74 kip/ft

FORM

Variables

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

REFERENCES
1. Drawings, dated 1948 Operation and Maintenance Manual V2
2. HGL, blanket elevations, levee elevations supplied EC-G

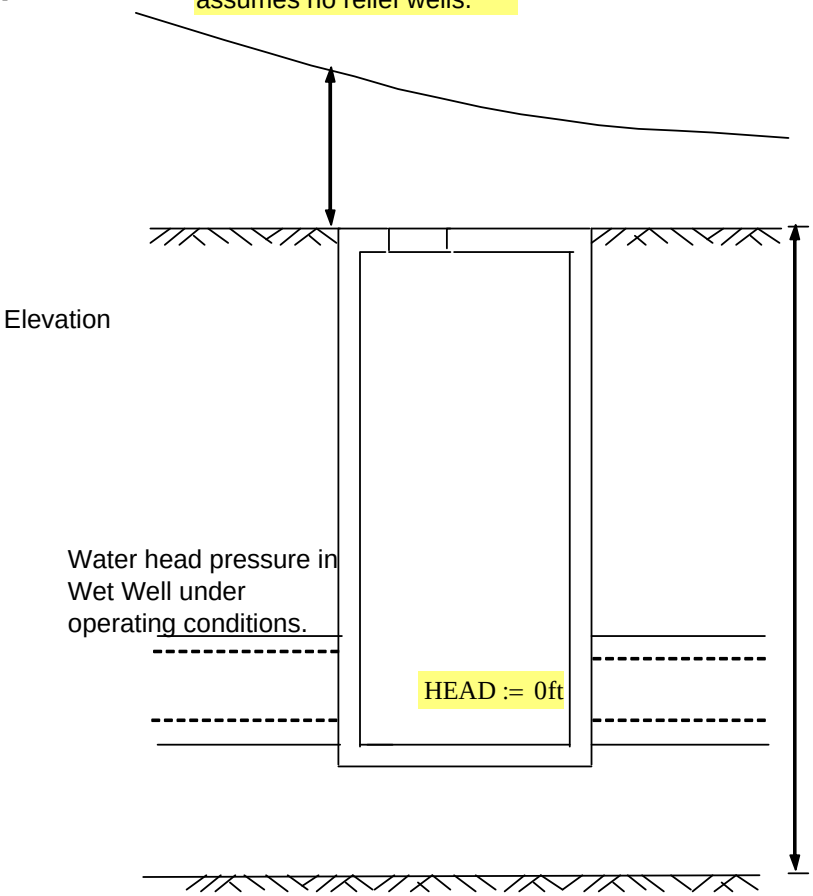
Top of existing levee El = 762
Bottom of blanket El = 722
Ground El = 756

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 = 762 ft



$ELEV_1 := 756ft$

$HGL := 759.29ft - ELEV_1$

$HGL = 3.29\,ft$

$BLANKET := 756ft - 722ft$

$BLANKET = 34\,ft$

$ELEV_2 := 737ft$

$H := ELEV_1 - ELEV_2$

$H = 19\,ft$

Structural drawings dated 1948 were obtained. This structure was last operated for flood control purposes in 1993. The west wall is integrated within the levee wall, and was analyzed using EnerCalc and is considered reliable. The slab was anaylyzed for uplift (see calculations below)

Wall 1 Parameters

West wall

$L_1 := 18.25\,ft$

$H = 19\,ft$

$D_1 := 15in$

$C_{c1} := 2.5in$

Wall 2 Parameters

East Wall

$L_2 := 18.25\,ft$

$H = 19\,ft$

$D_2 := 15in$

$C_{c2} := 2.5in$

$C_{cfi} := 2.5in$

Ccf= 2.5" for I.F. on base slab.

BASE SLAB

Floor Thickness $D_f := 21in$

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

$$d_{fo} := D_f - C_{cfo} - \frac{.625in}{2} \qquad d_{fo} = 17.688 \cdot in$$

$$d_{fi} := D_f - C_{cfi} - \frac{.625in}{2} \qquad d_{fi} = 18.188 \cdot in$$

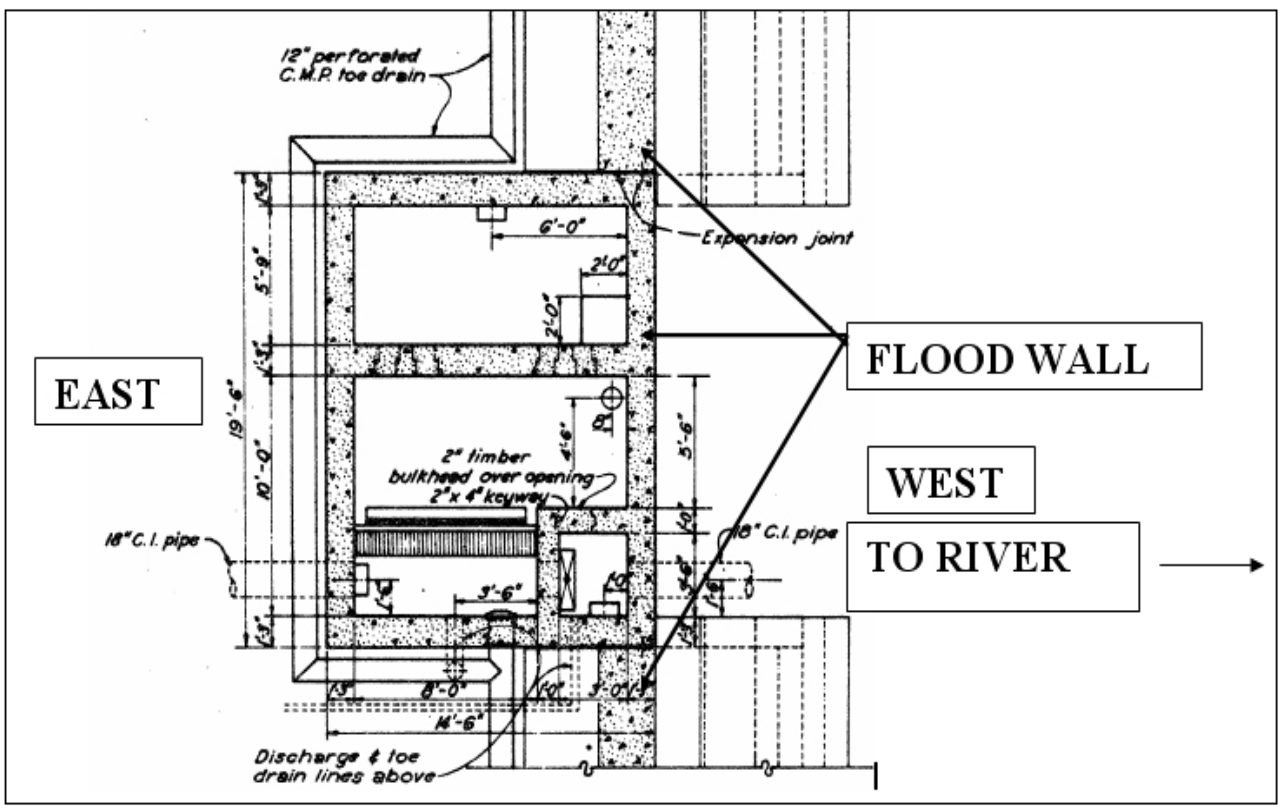


Figure 1. Mistletoe Pump Station 37+06.5 Plan View Drawing Sheet 47 (A-10-1877) USACE O&M CID 1948

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 $\phi := 21deg$

$\gamma := 110pcf$

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

$K_o = 0.642$

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)

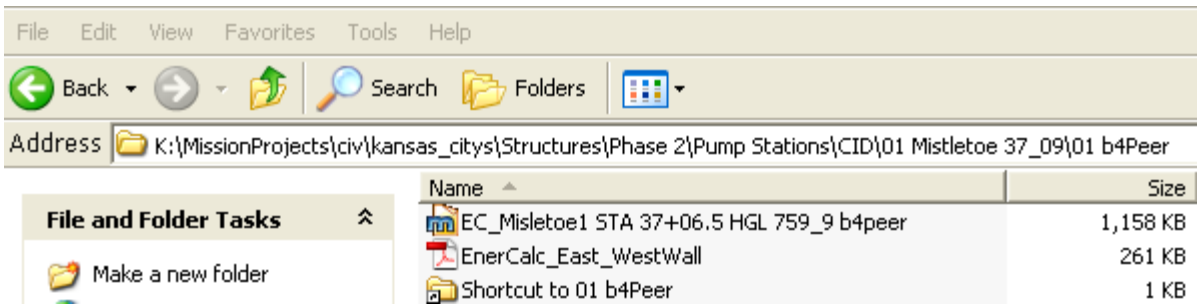
Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

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

West Wall and East Wall both were found to be reliable according to the EnerCalc model's found in white CID Binder and electronic copy in found in below folder EnerCalc_East_WestWall.pdf which contains the results for both walls.



WALL 1 (WEST WETWELL WALL)

$\phi V_n := 20kip$ $\quad V_u := 7.1kip$

Shear

$Wall1_shear_Check := if(\phi V_n > 1.5V_u, "OK", "NO\ GOOD")$ $\quad$ $Wall1_shear_Check = "OK"$

Factor of Safety $\quad FS_{Wall1_shear} := \frac{\phi V_n}{V_u}$

$FS_{Wall1_shear} = 2.817$

Moment

$FS_{MPV3} =$ not applicable

$FS_{EPV2} =$ not applicable

$\phi M_{nf} := 11.12kip\ ft$

$M_{uF} := 4.95kip \cdot ft$

$Wall1_Bending_Check := if(\phi M_{nf} > 1.5M_{uF}, "OK", "NO\ GOOD")$ $\quad$ $Wall1_Bending_Check = "OK"$

Factor of Safety $\quad FS_{Wall1} := \frac{\phi M_{nf}}{M_{uF}}$ $\quad$ $FS_{Wall1} = 2.246$

WALL 2 (EAST WETWELL WALL)

West Wall 2 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

SEE ENERCALC Sheet "Highlighted values"

West Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

$\phi V_n := 20.5kip$ $V_u := 12.76kip$

Shear

$Wall1_shear2_Check := if(\phi V_n > 1.5V_u, "OK", "NO\ GOOD")$ $Wall1_shear2_Check = "OK"$

Factor of Safety $FS_{Wall2_shear} := \frac{\phi V_n}{V_u}$

$FS_{Wall2_shear} = 1.607$

Moment

$FS_{MPV3} =$ not applicable

$FS_{EPV2} =$ not applicable

$\phi M_{nf} := 31.4kip\ ft$

$M_{uF} := 12.75kip \cdot ft$

$Wall1_Bending_Check := if(\phi M_{nf} > 1.5M_{uF}, "OK", "NO\ GOOD")$ $Wall1_Bending_Check = "OK"$

Factor of Safety $FS_{Wall2} := \frac{\phi M_{nf}}{M_{uF}}$

$FS_{Wall2} = 2.463$

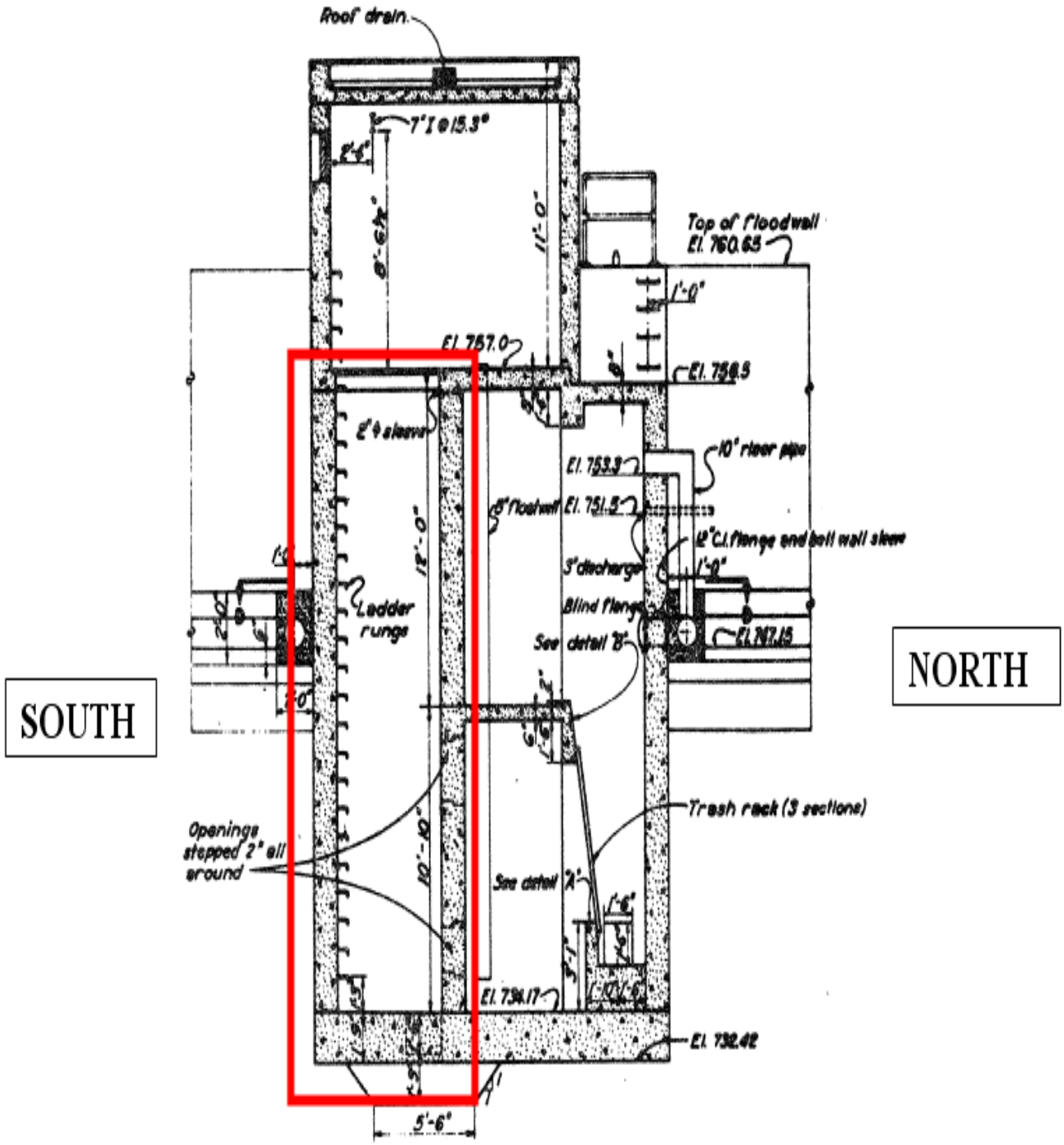


Figure 2 Mistletoe Pump Station 37+06.5 Plan View Drawing Sheet 7 (A-10-1877)
USACE O&M CID 1948

Uplift on Structure

Wet well weight.

The wetwell weight includes all of the following (See Figure# 2):
1. Wetwell walls
2. Top slab within the identified wetwell area
3. Concrete wall above grade within the wetwell area.
4. Wetwell base slab

Weight of wetwell.The equipment weight was not included.

Top Slab

$Top := 8in \cdot (7ft) \cdot (13.25ft)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 9.275 \cdot kip$

Walls

$West := 19ft \cdot 7ft \cdot 1.25ft$ $\quad Wwest := West \cdot 150pcf$ $\quad Wwest = 24.938 \cdot kip$

$South := 19ft \cdot 13.25ft \cdot 1.25ft$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 47.203 \cdot kip$

$East := West$ $\quad Weast := Wwest$ $\quad Weast = 24.938 \cdot kip$

$North := South$ $\quad Wnorth := North \cdot 150pcf$ $\quad Wnorth = 47.203 \cdot kip$

Base Slab

$Base := 1.5 \cdot ft \cdot (7ft) \cdot (13.25ft)$

$Base = 139.125 \cdot ft^3$ $\quad Wbase := Base \cdot 150pcf$ $\quad Wbase = 20.869 \cdot kip$

$Basearea := (7ft) \cdot (13.25ft)$ $\quad Basearea = 92.75\ ft^2$

South Wall above grade

$hsouth_{ag} := 11ft$

$South_{ag} := hsouth_{ag} \cdot 8in \cdot (13.25ft)$ $\quad Wsouth_{ag} := South_{ag} \cdot 150pcf$ $\quad Wsouth_{ag} = 14.575 \cdot kip$

$Wnorth_{ag} := Wsouth_{ag}$

West Wall above Grade

$hwest_{ag} := 11ft$

$West_{ag} := hwest_{ag} \cdot (7ft) \cdot 8in$ $\quad Wwest_{ag} := West_{ag} \cdot 150pcf$ $\quad Wwest_{ag} = 7.7 \cdot kip$

$Weast_{ag} := Wwest_{ag}$

Gatewell Above Grade

$Gatewell := 0ft \cdot [(4.5ft \cdot 6ft) - (2.5ft \cdot 5ft)]$ $\quad Wgatewell := Gatewell \cdot 150pcf$ $\quad Wgatewell = 0 \cdot kip$

Wetwell Weight

$$Wwetwell := Wtop + Wwest + Wsouth + Weast + Wnorth + Wbase \ldots + Wsouth_{ag} + Wwest_{ag} + Weast_{ag} + Wnorth_{ag} + Wgatewell$$

$Wwetwell = 218.975 \cdot kip$

Wetwell Structure Factor of Safety for Flotation

Weight of water that is present during the event

$Wellarea := 92.75 ft^2$

Bottom of wetwell base elevation

$Elevwetwellbot := 734 ft$

$HEAD := 1.33 ft$

bottom of blanket at 722

$Water := HEAD \cdot Wellarea \cdot 62.4 pcf$ $\quad Water = 7.698 \cdot kip$

$heel := 0 ft$ $\quad heel = 0$ The wet well does not have heels

$upliftarea := Basearea$ $\quad upliftarea = 92.75\ ft^2$

$H := ELEV_1 - Elevwetwellbot$ $\quad H = 22\ ft$ $\quad ELEV_1 = 756\ ft$ Grade elevation

Weight of soil on the wetwell heels

$volsoil := (1ft - 1ft) \cdot heel + (1ft - 1ft) \cdot heel$ $\quad volsoil = 0 \cdot ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 0 \cdot kip$ $\quad volsoil \cdot \gamma_w = 0 \cdot kip$

Head, if structural foundation in blanket. $\quad HGL = 3.29\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot upliftarea \cdot \gamma_w \qquad U_1 = 139.648 \cdot kips$$

$BLANKET = 34\ ft$
$HGL = 3.29\ ft$

Head, if structural foundation extends through blanket.

$H = 22\ ft$

$$U_2 := \left[(H + HGL) \cdot (upliftarea) \cdot \gamma_w\right] \qquad U_2 = 146.368 \cdot kips$$

$$Uplift_{wet} := if\left(H > BLANKET, U_2, U_1\right) \qquad Uplift_{wet} = 139.648 \cdot kips \qquad \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 24.129\ ft$$

The uplift force acting on the wetwell structure $\quad Uplift_{wet} = 139.648 \cdot kips$

$$Stability := \frac{Wwetwell + Water + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{wet} - volsoil \cdot \gamma_w}$$

$Stability = 1.623$ $\quad (H + HGL) = 25.29\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $\quad StabilityCheck = "Okay"$

Wet Well BASE SLAB ANALYSIS

$W_{water} := Water$ WEIGHT OF WATER

$BaseSlab := upliftarea$ $BaseSlab = 92.75\ ft^2$ $Uplift_{wet} = 139.648 \cdot kip$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$ $\frac{Wwetwell + W_{water}}{BaseSlab} = 2443.908 \cdot psf$

$W' := if\left(Stability < 1.0, \frac{Uplift_{wet}}{BaseSlab}, \frac{Wwetwell}{BaseSlab}\right)$ $W' = 2360.916 \cdot psf$ $\frac{Uplift_{wet}}{BaseSlab} = 1505.639 \cdot psf$

$W' = 2360.916 \cdot psf$ $\frac{Wwetwell + volsoil \cdot (\gamma - \gamma_w)}{BaseSlab} = 2360.916 \cdot psf$

$Slab_t := D_f$ $Wall_t := D_1$ $Slab_d := d_{fi}$ effective depth of slab

The longer length of slab was chosen. By inspection will carry the most moment.

$Slab_d = 18.188 \cdot in$

$D_1 = 15 \cdot in$

$L_s := 10ft$

$V'_{ub} := \frac{W' \cdot L_s}{2}$ $V'_{ub} = 11804.582 \frac{lb}{ft}$

$d_{fi} = 18.188 \cdot in$

$D_f = 1.75\ ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_s}{2}} \cdot \left(\frac{L_s}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 6751 \frac{lb}{ft}$

$A_g := Slab_t \cdot 12 \frac{in}{ft}$ $A_g = 252 \cdot \frac{in^2}{ft}$

$Water = 7697.508\ lb$

$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_{fi}\right)\right]$ ACI EQ (11-4) $\phi V_{nb} = 23908 \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5V_{ub}, "OKAY", "NO GOOD"\right)$ $\quad CheckBase = "OKAY"$

Factor of Safety $\quad FS_{Sb} := \dfrac{\phi V_{nb}}{V_{ub}} \quad FS_{Sb} = 3.542$

Slab Bending

Negative reinforcement. One-way action,

$b := 12\dfrac{in}{ft}$

The mid-span negative moment $\quad M_{mid} := \dfrac{W' \cdot L_s^2}{24} \quad M_{mid} = 9.837 \cdot \dfrac{kip \cdot ft}{ft}$

The end-span positivie moment $\quad M_{end} := \dfrac{W' \cdot L_s^2}{12} \quad M_{end} = 19.674 \cdot \dfrac{kip \cdot ft}{ft}$

5's @ 10.5". (I.F) inside face (see Sectional A-A A-10-1355) (mid-plate moment)

$A_{mph} := .35\dfrac{in^2}{ft} \quad a_{mph} := \dfrac{A_{mph} \cdot F_y}{0.85f'_c \cdot b} \quad a_{mph} = 0.458 \cdot in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_{fi} - \dfrac{a_{mph}}{2}\right) = 20.952 \cdot \dfrac{kip \cdot ft}{ft}$

6's at 6" (O.F) (see sectional A-10-1355) (edge-plate moment)

$A_{eph} := .88\dfrac{in^2}{ft} \quad a_{eph} := \dfrac{A_{eph} \cdot F_y}{0.85f'_c \cdot b} \quad a_{eph} = 1.15 \cdot in$

$\phi M_{ephs} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_{fo} - \dfrac{a_{eph}}{2}\right) \quad \phi M_{ephs} = 50.2 \cdot \dfrac{kip \cdot ft}{ft}$

$CheckMPHS := if\left(\phi M_{mph} > 1.5 \cdot M_{mid}, "OKAY", "NO GOOD"\right)$ $\quad CheckMPHS = "OKAY"$

Factor of Safety $\quad FS_{MPHS} := \dfrac{\phi M_{mph}}{M_{mid}} \quad FS_{MPHS} = 2.13$

$CheckEPHS := if\left(\phi M_{ephs} > 1.5 \cdot M_{end}, "OKAY", "NO GOOD"\right)$ $\quad CheckEPHS = "OKAY"$

Factor of Safety $\quad FS_{EPHS} := \dfrac{\phi M_{ephs}}{M_{end}} \quad FS_{EPHS} = 2.551$

FACTORS OF SAFETY SUMMARY

Since structure is symetrical two walls and the base slab were evaluated for strength. A summary of the results are below. .

Analysis indicates that the wetwell walls as reliable. Based on the Wall 1 and 2 calculations, the North and South wet well walls were not analyzed.

WALL 1 (WEST WETWELL WALL) SEE ENERCALC Sheet "Highlighted values" GOOD

West Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

$\phi V_n := 20kip$ $V_u := 7.1kip$

Shear

$Wall1_shear_Check := if(\phi V_n > 1.5V_u, "OK", "NO GOOD")$ Wall1_shear_Check = "OK"

Factor of Safety $FS_{Wall1_shear} := \frac{\phi V_n}{V_u}$

$FS_{Wall1_shear} = 2.817$

Moment

$FS_{MPV3} =$ not applicable

$\phi M_{nf} := 11.12kip\ ft$

$M_{uF} := 4.95kip \cdot ft$

$FS_{EPV2} =$ not applicable

$Wall1_Bending_Check := if(\phi M_{nf} > 1.5M_{uF}, "OK", "NO GOOD")$ Wall1_Bending_Check = "OK"

Factor of Safety $FS_{Wall1} := \frac{\phi M_{nf}}{M_{uF}}$

$FS_{Wall1} = 2.246$

WALL 2 (EAST WETWELL WALL) GOOD

West Wall 2 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

SEE ENERCALC Sheet "Highlighted values"

West Wall 1 moment evaluated such that the horizontal reinforcement assumes all of the lateral loading.

$\phi V_n := 20.5kip$ $V_u := 12.76kip$

Shear

$Wall1_shear2_Check := if\left(\phi V_n > 1.5V_u, "OK", "NO\ GOOD"\right)$ Wall1_shear2_Check = "OK"

Factor of Safety $FS_{Wall2_shear} := \frac{\phi V_n}{V_u}$

$FS_{Wall2_shear} = 1.607$

Moment

$FS_{MPV3} =$ not applicable

$\phi M_{nf} := 31.4 kip\ ft$

$M_{uF} := 12.75 kip \cdot ft$

$FS_{EPV2} =$ not applicable

$Wall1_Bending_Check := if\left(\phi M_{nf} > 1.5M_{uF}, "OK", "NO\ GOOD"\right)$ Wall1_Bending_Check = "OK"

Factor of Safety $FS_{Wall2} := \frac{\phi M_{nf}}{M_{uF}}$

$FS_{Wall2} = 2.463$

BASE SLAB

GOOD

Shear

$FS_{Sb} = 3.542$

Moment

$FS_{MPHS} = 2.13$

$FS_{EPHS} = 2.551$

$FS_{Wall2} = 2.463$

STABILITY

Stability = 1.623

RECOMMENDATIONS

END OF ANALYSIS AND RECOMMENDATIONS

WESTWALL

Title :
Dsgnr:
Description :

Scope :

Job #
Date: 11:49AM, 21 AUG 07

Multi-Span Concrete Beam

Description Mistletoe Pump Station West Walll strength calc's

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy	40,000.0 psi	Spans Considered Continuous Over Supports		ACI Dead Load Factor	1.00
f'c	3,000.0 psi	Stirrup Fy	40,000.0 psi	ACI Live Load Factor	1.00

Concrete Member Information

Description			end to pump pit wa	pit wall to gatwell	ell wall to North w
Span		ft	4.75	6.63	4.63
Beam Width		in	12.00	12.00	12.00
Beam Depth		in	15.00	15.00	15.00
End Fixity			Fix-Pin	Pin-Pin	Pin-Fix
Reinforcing	Center	Area	0.31in2	0.31in2	0.31in2
		Bar Depth	12.19in	12.19in	12.19in
	Left	Area	0.88in2	0.88in2	0.88in2
		Bar Depth	2.88in	2.88in	2.88in
	Right	Area	0.88in2	0.88in2	0.88in2
		Bar Depth	2.88in	2.88in	2.88in

$\frac{\phi V_n}{V_u} = \frac{635}{7.1} = 89$

$\frac{20}{7} = 2.9$

Loads

Using Live Load This Span ??		Yes	Yes	Yes
Dead Load	k/ft	2.143	2.143	2.143
Live Load	k/ft			

Results

		Beam OK	Beam OK	Beam OK
Mmax @ Cntr	k-ft	1.49	4.95	1.39
@ X =	ft	1.96	3.31	2.78
Mn * Phi	k-ft	11.12	11.12	11.12
Max @ Left End	k-ft	-2.61	-6.86	-6.79
Mn * Phi	k-ft	30.44	30.44	30.44
Max @ Right End	k-ft	-6.86	-6.79	-2.35
Mn * Phi	k-ft	30.44	30.44	30.44
		Bending OK	Bending OK	Bending OK
Shear @ Left	k	4.20	7.11	5.92
Shear @ Right	k	5.98	7.09	4.00

$\phi V_n = 2\sqrt{f'_c}\, b_w d$
$= 2\sqrt{3000}(15)(12.19)$
$= 635$ kips
OK

Reactions & Deflections

DL @ Left	k	4.20	13.10	13.02
LL @ Left	k	0.00	0.00	0.00
Total @ Left	k	4.20	13.10	13.02
DL @ Right	k	13.10	13.02	4.00
LL @ Right	k	0.00	0.00	0.00
Total @ Right	k	13.10	13.02	4.00
Max. Deflection	in	-0.000	-0.003	0.000
@ X =	ft	1.80	3.31	0.71
Inertia : Effective	in4	3,375.00	3,375.00	3,375.00

Shear Stirrups

Stirrup Rebar Area	in2	0.400	0.400	0.400
Spacing @ Left	in	Not Req'd	Not Req'd	Not Req'd
Spacing @ .2*L	in	Not Req'd	Not Req'd	Not Req'd
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	Not Req'd	Not Req'd	Not Req'd
Spacing @ Right	in	Not Req'd	Not Req'd	Not Req'd

Rev: 580000
User: KW-0607206, Ver 5.8.0, 1-Dec-2003
(c)1983-2003 ENERCALC Engineering Software

Multi-Span Concrete Beam

Description Mistletoe Pump Station West Walll strength calc's

Query Values

Location	ft	0.00	0.00	0.00
Moment	k-ft	-2.6	-6.9	-6.8
Shear	k	4.2	7.1	5.9
Deflection	in	0.0000	0.0000	0.0000

EAST WALL
GOOD

Title : MISTLETOE P.S.
Dsgnr: EHF
Description :

Job #
Date: 3:09PM, 5 JUL 07

Scope :

Multi-Span Concrete Beam

Description East Wall (Misletoe Pump Station) Steel Reinforcement sizes taken from A-10-1355 of O&M Manual. Note: Different steel sizes (I.F.) for Section A-A and Section B-B. The 2.143
TAKEN From M. Parks Calc's dated 6/7/07 White Binder (CJD)

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy	40,000.0 psi	Spans Considered Continuous Over Supports		ACI Dead Load Factor	1.00
f'c	3,000.0 psi	Stirrup Fy	40,000.0 psi	ACI Live Load Factor	1.00

Concrete Member Information

Description			wall to pump wall	wall to North wall
Span		ft	7.00	11.25
Beam Width		in	12.00	12.00
Beam Depth		in	15.00	15.00
End Fixity			Fix-Pin	Pin-Fix
Reinforcing	Center	Area	0.44in2	0.88in2
		Bar Depth	12.50in	12.50in
	Left	Area	1.45in2	1.45in2
		Bar Depth	2.40in	2.40in
	Right	Area	1.45in2	1.45in2
		Bar Depth	2.40in	2.40in

0.75

$\phi V_N = 2\sqrt{f'_c}\, b_w d$

$\phi V_N = 2\sqrt{3000}\,(15)(12.0)$

$\phi V_N = 650\,k$

$\phi V_N = \frac{487.14}{20.5}$

1.6

20.5

$V_u \phi$

Loads

Using Live Load This Span ??		No	No
Dead Load	k/ft	2.143	2.143
Live Load	k/ft		

Results

		Beam OK	Beam OK
Mmax @ Cntr	k-ft	3.02	~~12.75~~
@ X =	ft	2.66	5.32
Mn * Phi	k-ft	16.10	~~31.44~~
Max @ Left End	k-ft	-4.48	-17.29
Mn * Phi	k-ft	50.63	50.63
Max @ Right End	k-ft	-17.29	~~-25.26~~
Mn * Phi	k-ft	50.63	~~50.63~~
		Bending OK	Bending OK
Shear @ Left	k	5.67	11.35
Shear @ Right	k	9.33	~~12.76~~

$M_N =$

M_x

Reactions & Deflections

DL @ Left	k	5.67	20.68
LL @ Left	k	0.00	0.00
Total @ Left	k	5.67	20.68
DL @ Right	k	20.68	12.76
LL @ Right	k	0.00	0.00
Total @ Right	k	20.68	12.76
Max. Deflection	in	0.002	-0.044
@ X =	ft	5.74	5.32
Inertia : Effective	in4	2,525.15	1,410.13

Shear Stirrups

Stirrup Rebar Area	in2	0.000	0.400
Spacing @ Left	in	Not Req'd	6.30
Spacing @ .2*L	in	Not Req'd	Not Req'd
Spacing @ .4*L	in	Not Req'd	Not Req'd
Spacing @ .6*L	in	Not Req'd	Not Req'd
Spacing @ .8*L	in	Not Req'd	6.30
Spacing @ Right	in	0.00	6.30

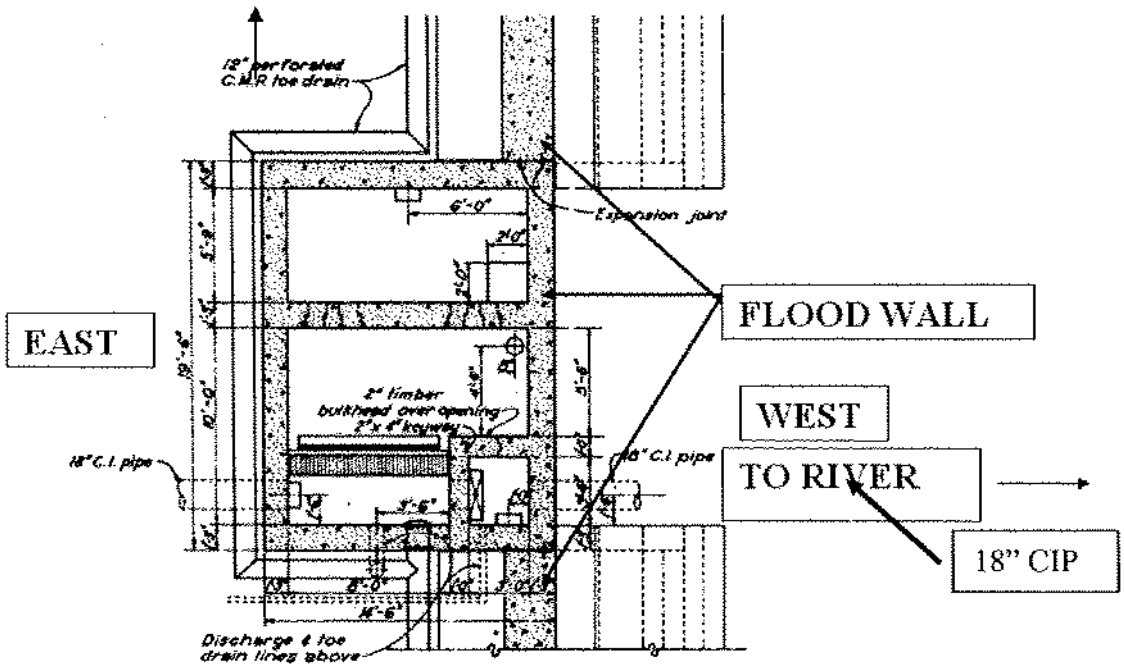
Description East Wall (Misletoe Pump Station) Steel Reinforcement sizes taken from A-10-1355 of O&M Manual. Note: Different steel sizes (I.F.) for Section A-A and Section B-B. The 2.143

Query Values

Location	ft	0.00	0.00
Moment	k-ft	-4.5	-17.3
Shear	k	5.7	11.3
Deflection	in	0.0000	0.0000

Mistletoe Pump Station Sta. 37+06.5
Outlet Works Evaluation

EHF 2/22/2008



Summary:

The Mistletoe pump station outlet works consists of a 18-inch diameter cast-iron-pipe. A flap gate is located on the pipe outlet structure. Visible portions of the pipe outlet structure and flap gate were inspected by CENWK-EC-DS personnel on July 30, 2007. The outlet structure appeared to be in good condition with no signs of distress evident. The flap gate appeared to be in satisfactory condition. The Mistletoe pump station was constructed in the late 1940's. The pipe was inaccessible during the July 30, 2007 site-visit because the flap gate was closed prohibiting pipe entry and safety concerning possible confined entry conditions. It is recommended that the pipe be visually inspected via CCTV. If the pipe is determined to be in distress, pipe rehabilitation should be instituted. For cost analysis, it should be assumed that the pipe will require rehabilitation given its age and lack of information in regards to its construction and no inspection history.

CID District
New Central Pump Station F.C.(n500+3) Analysis Sta. 58+12, HGL=760.3'
KANSAS CITYS LEVEES PHASE II

Variables

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

REFERENCES
1. Drawings, dated 1986 Black and Veatch
2. HGL, blanket elevations, levee elevations supplied EC-G

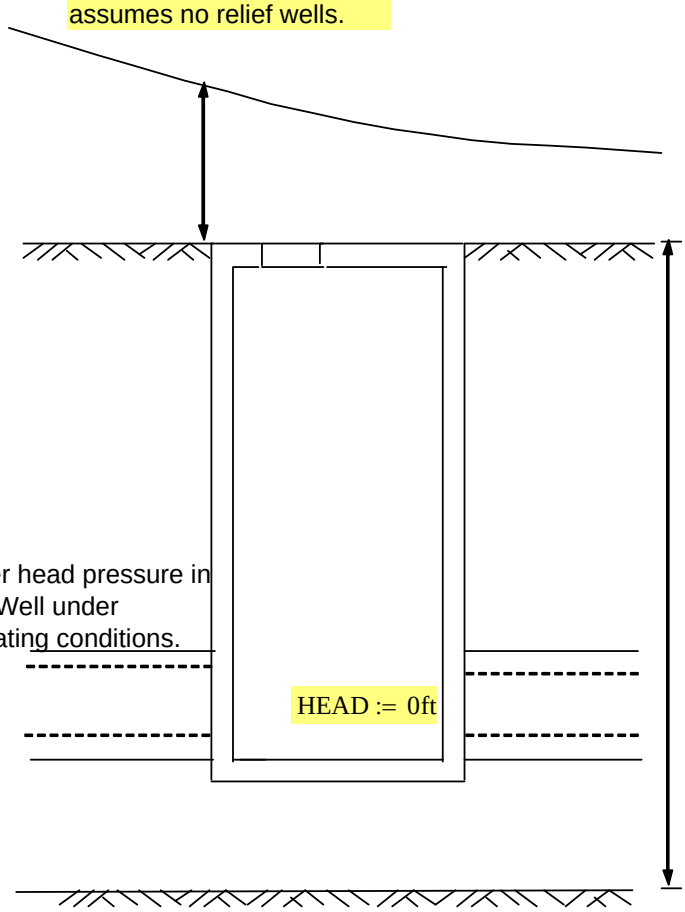
Top of existing levee El = 763.9
Bottom of blanket El = 722
Ground El = 752

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 = 763.9 ft



Elevation

$ELEV_1 := 752ft$

$HGL := 760.3ft - ELEV_1$

$HGL = 8.3\,ft$

Water head pressure in Wet Well under operating conditions.

$BLANKET := 752ft - 722ft$

$BLANKET = 30\,ft$

$HEAD := 0ft$

$ELEV_2 := 727ft$

$H := ELEV_1 - ELEV_2$

$H = 25\,ft$

Elev2 taken at bottom of wetwell and is used to determine load on structure. Unlike other pump stations previously analyzed, New Central's 84"RCP is confined to the west wall. Therfore, for analysis we use the the actual bottom elevation of structure 727' for a better represenation of uniform load on wall.

Structural drawings dated 1986 were obtained from Black & Veatch. The City of Kansas City, KS calls this structure Flood Pump Station #16. The pump was designed to allow gravity flow when the Kansas River stage is low enough and therefore places the pump system on stand by. Water entering the stormwater collection system flows through the station inlet to the river by gravity. However, once the critcal stage is reached, the gatewell is closed and stormwater is pumped to the riverside of the gatewell sluice gate. This creates a high enough head in the wetwell to permit gravity flow to the river through the same outlet pipe.
Since structure is relatively symetrical and no significant variances in wall dimensions are apparent, east and south west wall can be treated as identical as well as the west and south west walls. This will allow for an efficient means to determine strength/stability.

Wall 1 Parameters

East wall

$L_1 := 31.2ft$

$Hwall_1 := 25ft$

$D_1 := 36in$

$C_{c1} := 4in$

Wall 2 Parameters

North East Wall

$L_2 := 48ft$

$Hwall_2 := 25ft$

$D_2 := 36in$

$C_{c2} := 4in$

BASE SLAB

The base slab reinforcement is shown on sheet 7 of 14.

Bar size floor = # 10's @ 6" T&B

Floor Thickness $D_f := 54in$ Clear Cover $C_{cf} := 6in$ $d_f := D_f - C_{cf} - \frac{1.27in}{2}$ $d_f = 47.365 \cdot in$

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - \left(C_{c1} + \frac{1.0}{2}in\right)$

$d_1 = 31.5 \cdot in$

WALL 2

$d_2 := D_2 - \left[C_{c2} + \left(\frac{1.0}{2}in\right)\right]$

$d_2 = 31.5 \cdot in$

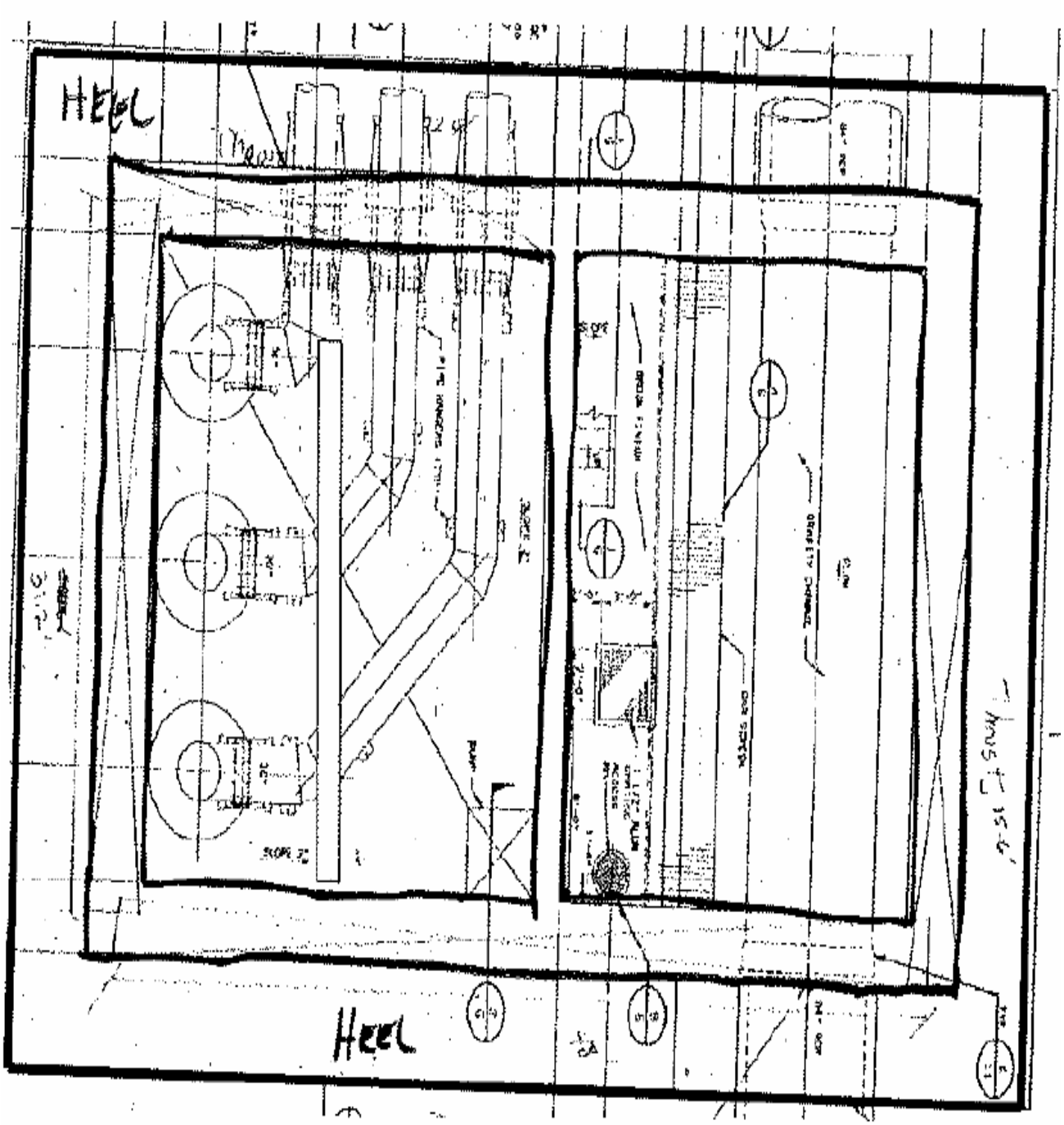


Figure 1. New Central Pump Station 58+12 Plan View Drawing Sheet #5 of 14 B&V 1986.

Assumptions and Criteria

- Steel reinforcement yield strength = 60ksi provided on Sheet 10 of 14 of the construction drawings. Given the time period of construction (1980's), the concrete f'c =4ksi.

Soil Properties	$\phi := 21deg$	$K_o := 1 - \sin(\phi)$	Steel Properties	$F_y := 60ksi$
	$\gamma := 110pcf$	$K_o = 0.642$	Concrete Properties	$f'_c := 4.00ksi$
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)

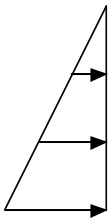
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALL 1, (East wall) In place wall $K_o = 0.642$ $BLANKET = 30\,ft$ $HGL = 8.3\,ft$

Wall Loading

Soil or Soil & Water



$H = 25\,ft$

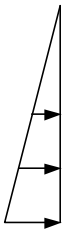
$H_2 := H$

$H_2 = 25\,ft$

OR

$H_2 = 25\,ft$

&



$H'_1 := \frac{BLANKET + HGL}{BLANKET} \cdot H - HEAD$

$H''_1 := H + HGL - HEAD$

$H_1 := if(H_2 > BLANKET, H''_1, H'_1)$

$H_1 = 31.917\,ft$

$H'_1 = 31.917\,ft$

$H''_1 = 33.3\,ft$

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

$W_s = 1764.5 \cdot psf$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_1]$

$W_w = 2755 \cdot psf$

$W_{1ext} := if(W_s > W_w, W_s, W_w)$ $W_{1ext} = 2755.142 \cdot psf$

SHEAR

WALL 1, East Wall

$L_{tw} := 24ft$ Ltw stands for length of thrust wall

$$THRUST_1 := \frac{W_{1ext} \cdot L_{tw}}{2}$$

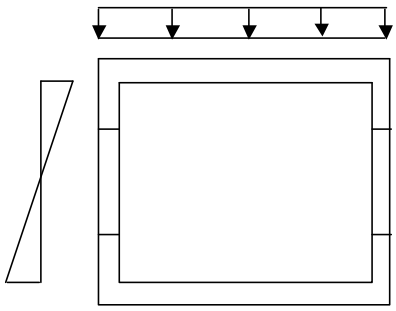
From Sheet No.5 of 14

$$V'_{u1} := W_{1ext} \cdot \frac{L_1}{2}$$

$$V'_{u1} = 42980 \frac{lb}{ft}$$

$$THRUST_1 = 33061.706 \frac{lb}{ft}$$

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



$$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$$

$$V_{u1} = 31615 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$A_g := D_1 \cdot 12 \frac{in}{ft} \qquad A_g = 432 \cdot \frac{in^2}{ft}$$

$$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$$ ACI EQ (11-4)

$$\phi V_{n1} = 49643 \frac{lb}{ft}$$

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

Check1 = "OKAY"

Factor of Safety

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

$$FS_{S1} = 1.57$$

Wall Analysis East Wall 1

$l_2 := 15.6ft$

Plate Loading (Bending)

$l_1 := 25ft$

$a/b = L_2/L_1$

$\frac{l_2}{l_1} = 0.624$

From plates tables:

a/b	Coefficient
1.5	.0232
1.0	.0289
.75	.0302
.5	.0269
.375	.0207

$ab_k :=$ $Coeff_k :=$ $k := 1..5$

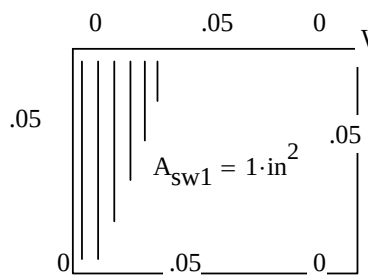
ab_k	$Coeff_k$
.375	.0207
.50	.0269
.75	.0302
1.0	.0289
1.5	.0232

M.x = Horizontal exterior steel

$A_{sw1} := 1in^2$ (End of Span)

Using plate analysis (pg. 19 Bur. of Rec) fixed on all sides hinged at top.

9's @ 12" O.C.



$W_{UNIFORM} := W_{1ext}$

$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right)$

$Coefficient = 0.0285$

$W_{UNIFORM} = 2755.142 \cdot psf$

$b := 1ft$

$l_2 = 15.6\,ft$

$M_{uF} := Coefficient \cdot W_{UNIFORM} \cdot l_1^2 \cdot b$

$M_{uF} = 49.139 \cdot kip \cdot ft$

$a := \frac{A_{sw1} \cdot F_y}{0.85 f'_c \cdot b}$ $a = 1.471 \cdot in$

$d_f = 3.947\,ft$

$\phi M_{nF} := \phi_B \cdot A_{sw1} \cdot F_y \cdot \left(d_1 - \frac{a}{2}\right)$ $\phi M_{nF} = 153.824\,ft \cdot kip$

$Wall1_Bending_Check := if\left(\phi M_{nF} > 1.5 M_{uF}, "OK", "NO\ GOOD"\right)$ $Wall1_Bending_Check = "OK"$

Factor of Safety $FS_{wallmomentA} := \frac{\phi M_{nF}}{M_{uF}}$

$FS_{wallmomentA} = 3.13$

Wall Analysis East Wall 1

$l_2 := 15.6ft$

Plate Loading (Bending)

$l_1 := 25ft$

$a/b = L_2/L_1$

$\frac{l_2}{l_1} = 0.624$

From plates tables:

a/b	Coefficient
1.5	.0637
1.0	.0593
.75	.0505
.5	.0320
.375	.0200

$ab_k :=$ $Coeff_k :=$ $k := 1..5$

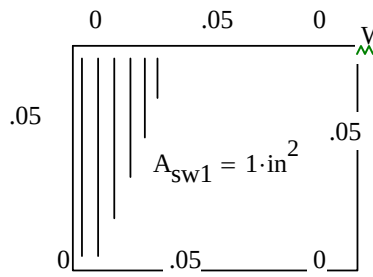
ab_k	$Coeff_k$
.375	.0200
.5	.0320
.75	.0505
1	.0593
1.5	.0637

M.y = Vertical exterior steel

$A_{sw1} := 1in^2$ (End of Span)

Using plate analysis (pg. 19 Bur. of Rec) fixed on all sides hinged at top.

9's @ 12"
O.C.



$W_{UNIFORM} := W_{1ext}$ $\quad$ $Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right)$

$Coefficient = 0.0412$

$W_{UNIFORM} = 2755.142 \cdot psf$ $\quad$ $b = 1\,ft$

$l_2 = 15.6\,ft$

$M_{uF} := Coefficient \cdot W_{UNIFORM} \cdot l_1^2 \cdot b$

$M_{uF} = 70.904 \cdot kip \cdot ft$

$a := \frac{A_{sw1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad$ $a = 1.471 \cdot in$

$\phi M_{nF} := \phi_B \cdot A_{sw1} \cdot F_y \cdot \left(d_1 - \frac{a}{2}\right)$ $\quad$ $\phi M_{nF} = 153.824\,ft \cdot kip$

$Wall1a_Bending_Check := if\left(\phi M_{nF} > 1.5 M_{uF}, "OK", "NO\ GOOD"\right)$

Factor of Safety $\quad$ $FS_{wallmomentB} := \frac{\phi M_{nF}}{M_{uF}}$

$Wall1a_Bending_Check = "OK"$

$FS_{wallmomentB} = 2.169$

Wall Analysis East Wall 1

$l_2 := 15.6ft$

Plate Loading
(Bending)

$l_1 := 25ft$

$\frac{l_2}{l_1} = 0.624$

a/b=L_2/L_1

M.x = Horizontal interior steel

$A_{sw1} := .79in^2$ (Middle of Span)

From plates tables:

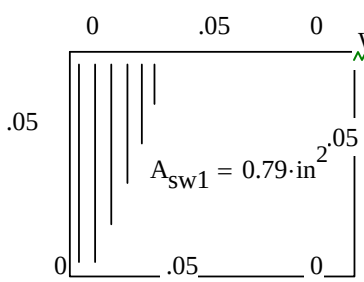
a/b	Coefficient
1.5	.0089
1.0	.0098
.75	.0119
.5	.0119
.375	.0098

$ab_k :=$ $\quad$ $Coeff_k :=$ $\quad$ $k := 1..5$

ab_k	$Coeff_k$
.375	.0098
.50	.0119
.75	.0119
1.0	.0098
1.5	.0089

Using plate analysis (pg. 19 Bur. of Rec) fixed on all sides hinged at top.

8's @ 12"
O.C.



$W_{UNIFORM} := W_{1ext}$ $\quad$ $Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right)$

$Coefficient = 0.0119$

$W_{UNIFORM} = 2755.142 \cdot psf$ $\quad$ $b := 1ft$

$l_2 = 15.6\,ft$

$M_{uF} := Coefficient \cdot W_{UNIFORM} \cdot l_1^2 \cdot b$

$M_{uF} = 20.491 \cdot kip \cdot ft$

$d_f = 3.947\,ft$

$a := \frac{A_{sw1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad$ $a = 1.162 \cdot in$

$\phi M_{nF} := \phi_B \cdot A_{sw1} \cdot F_y \cdot \left(d_1 - \frac{a}{2}\right)$ $\phi M_{nF} = 122.131\ ft \cdot kip$

$Wall1_Bending_Check := if\left(\phi M_{nF} > 1.5 M_{uF}, "OK", "NO\ GOOD\right)$ $Wall1_Bending_Check = "OK"$

Factor of Safety $FS_{wallmomentC} := \frac{\phi M_{nF}}{M_{uF}}$ $FS_{wallmomentC} = 5.96$

Wall Analysis East Wall 1

$l_2 := 15.6ft$

Plate Loading (Bending) $l_1 := 25ft$

From plates tables:

$\frac{l_2}{l_1} = 0.624$

a/b=L_2/L_1

a/b	Coefficient
1.5	.0297
1.0	.0259
.75	.0198
.5	.0116
.375	.0063

$ab_k :=$ $Coeff_k :=$ $k := 1 .. 5$

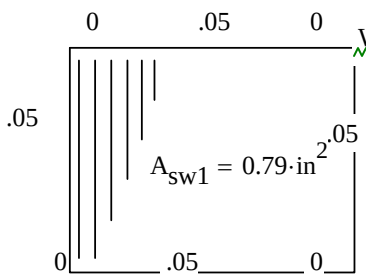
ab_k	$Coeff_k$
.375	.0063
.50	.0116
.75	.0198
1.0	.0259
1.5	.0297

M.x = Vertical interior steel

$A_{sw1} := .79in^2$ (Middle of Span)

Using plate analysis (pg. 19 Bur. of Rec) fixed on all sides hinged at top.

8's @ 12" O.C.



$A_{sw1} = 0.79 \cdot in^2$

$W_{UNIFORM} := W_{1ext}$ $Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right)$

$Coefficient = 0.0157$

$W_{UNIFORM} = 2755.142 \cdot psf$ $b := 1ft$

$l_2 = 15.6\ ft$

$M_{uF} := Coefficient \cdot W_{UNIFORM} \cdot l_1^2 \cdot b$

$M_{uF} = 26.978 \cdot kip \cdot ft$ $d_f = 3.947\ ft$

$a := \frac{A_{sw1} \cdot F_y}{0.85 f'_c \cdot b}$ $a = 1.162 \cdot in$

$\phi M_{nF} := \phi_B \cdot A_{sw1} \cdot F_y \cdot \left(d_1 - \frac{a}{2}\right)$ $\phi M_{nF} = 122.131\ ft \cdot kip$

$Wall1_Bending_Check := if\left(\phi M_{nF} > 1.5 M_{uF}, "OK", "NO\ GOOD\right)$ $Wall1_Bending_Check = "OK"$

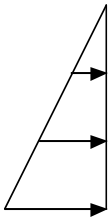
Factor of Safety $FS_{wallmomentD} := \frac{\phi M_{nF}}{M_{uF}}$ $FS_{wallmomentD} = 4.527$

WALL 2, North East Wall

$K_o = 0.642$ BLANKET = 30 ft HGL = 8.3 ft

Wall Loading

Soil or Soil & Water

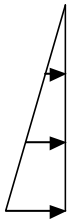


$H = 25\,ft$

$H_1 := H$

$H_1 = 25\,ft$

OR



$H_1 = 25\,ft$

&

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

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

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

$H_3 = 31.917\,ft$

$H'_3 = 31.917\,ft$

$H''_3 = 33.3\,ft$

$W_s := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma \cdot H_1$

$W_s = 1764 \frac{lb}{ft^2}$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_1 + \gamma_w \cdot H_3]$

$W_w = 2755 \frac{lb}{ft^2}$

$W_2 := if(W_s > W_w, W_s, W_w)$

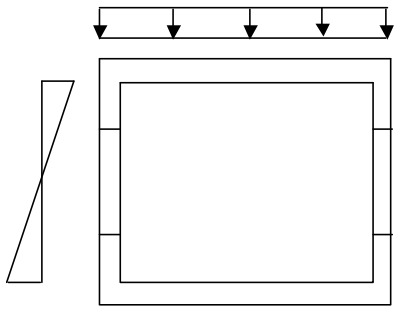
$W_2 = 2755 \frac{lb}{ft^2}$

$L_{ntw} := 15.6ft$

$THRUST_2 := \frac{W_2 \cdot L_{ntw}}{2}$ $THRUST_2 = 21490 \frac{lb}{ft}$

$V'_u := W_2 \cdot \frac{L_2}{2}$

$V'_u = 66123 \frac{lb}{ft}$



$V_u := 2 \frac{V'_u}{L_2} \cdot \left(\frac{L_2}{2} - \frac{D_1}{2} - d_2\right)$

$V_u = 54758 \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$D_2 = 36 \cdot in$

$A_g := D_2 \cdot 12 \frac{in}{ft}$ $A_g = 432 \cdot \frac{in^2}{ft}$

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

$\phi V_n := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_2}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1\right)\right]$ ACI EQ (11-4)

$d_2 = 31.5 \cdot in$

$\frac{THRUST_2 \cdot \frac{in^2}{lb}}{A_g} = 49.746$

$\phi V_n = 49003 \frac{lb}{ft}$

$Check1 := if(\phi V_n > 1.5 V_u, "OKAY", "NO GOOD")$

Check1 = "NO GOOD"

Factor of Safety

$FS_{S2} := \frac{\phi V_n}{V_u}$

$FS_{S2} = 0.895$

Due to the complexity of the wall design, RAM ADVANSE was used to model this wall (see appendix). The following are the calc's gained from RAM to check for strength FOS.

From RAM Vmax = 30k/ft

Therefore - $\phi V_n = 49 \cdot \frac{kip}{ft}$ $V_u := 30 \frac{kip}{ft}$

$Check2 := if\left(\phi V_n > 1.5V_u, "OKAY", "NO\ GOOD"\right)$ $Check2 = "OKAY"$

Factor of Safety $FS_{S2a} := \frac{\phi V_n}{V_u}$ $FS_{S2a} = 1.633$

WALL 2, North East Wall, Cont'd

$b := 12 \frac{in}{ft}$

#8's @ 12in. inside face (mid-plate moment) $A_{mph} := .79 \frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{mph} = 1.162 \cdot in$

$\phi M_{mph} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_1 - \frac{a_{mph}}{2}\right)$ $\phi M_{mph} = 122.1 \cdot \frac{kip \cdot ft}{ft}$

9 @ 12in bars outside face (edge-plate moment) $A_{eph} := 1.0 \frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$ $a_{eph} = 1.471 \cdot in$

$\phi M_{eph} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_1 - \frac{a_{eph}}{2}\right)$ $\phi M_{eph} = 153.8 \cdot \frac{kip \cdot ft}{ft}$

From RAM M11 Vertical Reinforcement Mend= 70 kip-ft , Mmid= 30 kip-ft

$M_{end} := 70 \frac{kip \cdot ft}{ft}$

$M_{mid} := 30 \frac{kip \cdot ft}{ft}$

$CheckMPH1 := if\left(\phi M_{mph} > 1.5 \cdot M_{mid}, "OKAY", "NO\ GOOD"\right)$

$CheckMPH1 = "OKAY"$

Factor of Safety $FS_{MPH} := \frac{\phi M_{mph}}{M_{mid}}$

$FS_{MPH} = 4.071$

$CheckEPH1 := if\left(\phi M_{eph} > 1.5 \cdot M_{end}, "OKAY", "NO\ GOOD"\right)$

$CheckEPH1 = "OKAY"$

Factor of Safety $FS_{EPH} := \frac{\phi M_{eph}}{M_{end}}$

$FS_{EPH} = 2.197$

From RAM M33 Horizontal Reinforcement Mend= 55 kip-ft , Mmid= 30 kip-ft

$M_{end} := 55 \frac{kip \cdot ft}{ft}$

$M_{mid} := 30 \frac{kip \cdot ft}{ft}$

$CheckMPH1 := if\left(\phi M_{mph} > 1.5 \cdot M_{mid}, "OKAY", "NO\ GOOD"\right)$

$CheckMPH1 = "OKAY"$

Factor of Safety $FS_{MPH1} := \frac{\phi M_{mph}}{M_{mid}}$

$FS_{MPH1} = 4.071$

$CheckEPH1 := if\left(\phi M_{eph} > 1.5 \cdot M_{end}, "OKAY", "NO\ GOOD"\right)$

$CheckEPH1 = "OKAY"$

Factor of Safety $FS_{EPH1} := \frac{\phi M_{eph}}{M_{end}}$

$FS_{EPH1} = 2.797$

Uplift on Structure Refer to Figure 2 for the identified areas concerning uplift calculations

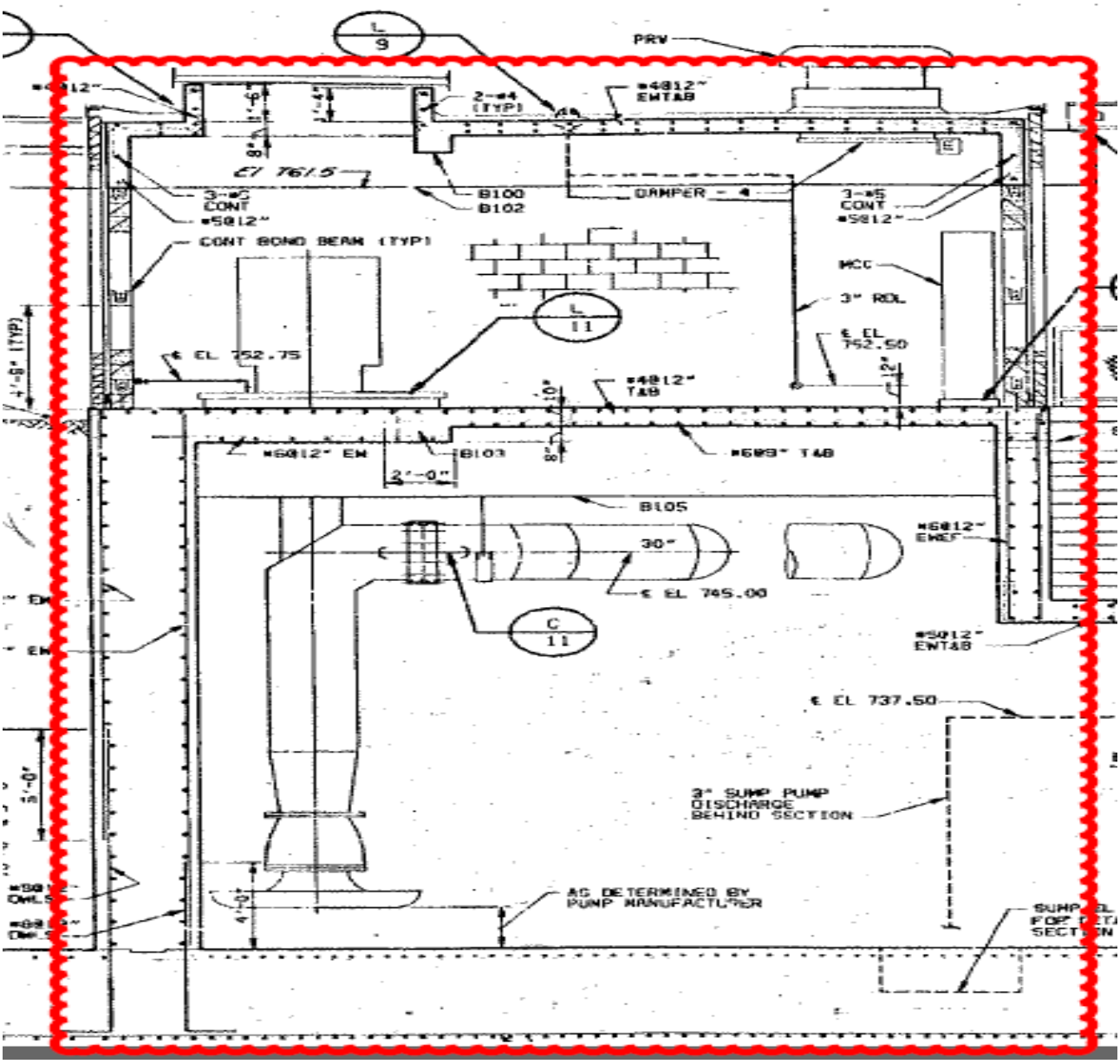


Figure 2. New Central Section cut facing North East.

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
3. Concrete wall above grade within the wetwell area.
4. Wetwell base slab

.

Uplift on Structure

Weight of wetwell.The equipment weight was not included.

Top Slab

$Top := 12in \cdot (28.16ft) \cdot (23ft)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 97.152 \cdot kip$

WallsA

$West := 25ft \cdot 31.2ft \cdot 3ft$ $\quad Wwest := West \cdot 150pcf$ $\quad Wwest = 351 \cdot kip$

$South := 25ft \cdot 48ft \cdot 3ft$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 540 \cdot kip$

$East := West$ $\quad Weast := Wwest$ $\quad Weast = 351 \cdot kip$

$North := South$ $\quad Wnorth := North \cdot 150pcf$ $\quad Wnorth = 540 \cdot kip$

Base Slab.A

$Base := 4.5 \cdot ft \cdot (44ft) \cdot (29ft)$

$Base = 5742 \cdot ft^3$ $\quad Wbase := Base \cdot 150pcf$ $\quad Wbase = 861.3 \cdot kip$

$Basearea := (44ft) \cdot (29ft)$ $\quad Basearea = 1276\, ft^2$

South Wall above grade

$hsouth_{ag} := 13ft$

$South_{ag} := hsouth_{ag} \cdot 8in \cdot (23ft)$ $\quad Wsouth_{ag} := South_{ag} \cdot 120pcf$ $\quad Wsouth_{ag} = 23.92 \cdot kip$

$Wnorth_{ag} := Wsouth_{ag}$

West Wall above Grade

$hwest_{ag} := 13ft$

$West_{ag} := hwest_{ag} \cdot (28.16ft) \cdot 8in$ $\quad Wwest_{ag} := West_{ag} \cdot 120pcf$ $\quad Wwest_{ag} = 29.286 \cdot kip$

$Weast_{ag} := Wwest_{ag}$

Gatewell Above Grade

$Gatewell := 0ft \cdot 0ft \cdot 0ft$ $\quad Wgatewell := Gatewell \cdot 150pcf$

$Wgatewell = 0 \cdot kip$

Wetwell Weight

$$Wwetwell := Wtop + Wwest + Wsouth + Weast + Wnorth + Wbase \ldots + Wsouth_{ag} + Wwest_{ag} + Weast_{ag} + Wnorth_{ag} + Wgatewell$$

$Wwetwell = 2846.865 \cdot kip$

Wetwell Structure Factor of Safety for Flotation

Weight of water that is present during the event

$Wellarea := 1276ft^2$

Bottom of wetwell base elevation

$Elevwetwellbot := 727ft$

$HEAD := 4.75ft$

bottom of blanket at 722

$Water := HEAD \cdot Wellarea \cdot 62.4pcf$ $\quad Water = 378.206 \cdot kip$

$heel_1 := 4.9ft$

The wet well does have heels

$heel_2 := 5ft$

$upliftarea := Basearea$ $\quad upliftarea = 1276\,ft^2$

$H := ELEV_1 - Elevwetwellbot$ $\quad H = 25\,ft$ $\quad ELEV_1 = 752\,ft$ Grade elevation

Weight of soil on the wetwell heels

$volsoil_1 := (24.5ft \cdot 29ft \cdot heel_1) \cdot 2$ $\quad volsoil_1 = 6962.9 \cdot ft^3$

$volsoil_1 \cdot (\gamma - \gamma_w) = 331.434 \cdot kip$ $\quad volsoil_1 \cdot \gamma_w = 434.485 \cdot kip$

$volsoil_2 := 24.5ft \cdot 34.17ft \cdot heel_2$ $\quad volsoil_2 = 4185.825 \cdot ft^3$

$volsoil_2 \cdot (\gamma - \gamma_w) = 199.245 \cdot kip$ $\quad volsoil_2 \cdot \gamma_w = 261.195 \cdot kip$

$Total_volsoil_1 := volsoil_1 \cdot (\gamma - \gamma_w) + volsoil_2 \cdot (\gamma - \gamma_w) = 530.679 \cdot kips$

$Total_volsoil_2 := volsoil_1 \cdot (\gamma_w) + volsoil_2 \cdot (\gamma_w) = 695.68 \cdot kips$

Head, if structural foundation in blanket.

$HGL = 8.3\,ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot upliftarea \cdot \gamma_w$$

$U_1 = 2541.282 \cdot kips$

$BLANKET = 30\,ft$
$HGL = 8.3\,ft$

Head, if structural foundation extends through blanket.

$H = 25\,ft$

$U_2 := [(H + HGL) \cdot (upliftarea) \cdot \gamma_w]$ $\quad U_2 = 2651.426 \cdot kips$

$Uplift_{wet} := if(H > BLANKET, U_2, U_1)$ $\quad Uplift_{wet} = 2541.282 \cdot kips$ $\quad \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 31.917\,ft$

The uplift force acting on the wetwell structure $\quad Uplift_{wet} = 2541.282 \cdot kips$

$$Stability := \frac{Wwetwell + Water + Total_volsoil_1}{Uplift_{wet} - Total_volsoil_2}$$

$Stability = 2.035$ $\quad (H + HGL) = 33.3\,ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $\quad StabilityCheck = "Okay"$

Wet Well BASE SLAB ANALYSIS

$W_{water} := Water$ WEIGHT OF WATER

$BaseSlab := upliftarea$ $BaseSlab = 1276\,ft^2$ $Uplift_{wet} = 2541.282 \cdot kip$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$

$\frac{Wwetwell + W_{water}}{BaseSlab} = 2527.485 \cdot psf$

$W' := if\left(Stability < 1.0, \frac{Uplift_{wet}}{BaseSlab}, \frac{Wwetwell}{BaseSlab}\right)$ $W' = 2231.085 \cdot psf$

$W' = 2231.085 \cdot psf$

$\frac{Uplift_{wet}}{BaseSlab} = 1991.6 \cdot psf$

$\frac{Wwetwell + Total_volsoil_1}{BaseSlab} = 2646.978 \cdot psf$

$W' := 2406psf$

$Slab_t := D_f$ $L_{strip} := 25.25ft$ $Wall_t := D_1$ $Slab_d := d_f$ effective depth of slab

$Slab_d = 47.365 \cdot in$

$D_1 = 36 \cdot in$

$V'_{ub} := \frac{W' \cdot L_{strip}}{2}$ $V'_{ub} = 30375.75 \frac{lb}{ft}$

$d_f = 47.365 \cdot in$

$D_f = 4.5\,ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_{strip}}{2}} \cdot \left(\frac{L_{strip}}{2} - \frac{Wall_t}{2} - Slab_d\right)$

$V_{ub} = 17270 \frac{lb}{ft}$

$A_g := Slab_t \cdot 12 \frac{in}{ft}$ $A_g = 648 \cdot \frac{in^2}{ft}$

$Water = 378206.4\,lb$

$$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f \right) \right]$$

ACI EQ (11-4)

$$\phi V_{nb} = 71895 \frac{lb}{ft}$$

$$CheckBase := if\left(\phi V_{nb} > 1.5 V_{ub}, "OKAY", "NO GOOD"\right)$$

$$CheckBase = "OKAY"$$

Factor of Safety

$$FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$$

$$FS_{Sb} = 4.163$$

Slab Bending

$$b := 12 \frac{in}{ft}$$

$$W_s := Uplift_{wet}$$

The mid-span negative moment

$$M_{mid} := \frac{W' \cdot L_{strip}^2}{24}$$

$$M_{mid} = 63.916 \cdot \frac{kip \cdot ft}{ft}$$

The end-span positivie moment

$$M_{end} := \frac{W' \cdot L_{strip}^2}{12}$$

$$M_{end} = 127.831 \cdot \frac{kip \cdot ft}{ft}$$

8's @ 6 in. (I.F) inside face
(mid-plate moment)

$$A_{mph} := 1.58 \frac{in^2}{ft}$$

$$a_{mph} := \frac{A_{mph} \cdot F_y}{0.85 f'_c \cdot b}$$

$$a_{mph} = 2.324 \cdot in$$

$$\phi M_{mphs} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_f - \frac{a_{mph}}{2} \right) = 365.006 \cdot \frac{kip \cdot ft}{ft}$$

8's at 6 (O.F)
(edge-plate moment)

$$A_{eph} := 1.58 \frac{in^2}{ft}$$

$$a_{eph} := \frac{A_{eph} \cdot F_y}{0.85 f'_c \cdot b}$$

$$a_{eph} = 2.324 \cdot in$$

$$\phi M_{ephs} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_f - \frac{a_{eph}}{2} \right)$$

$$\phi M_{ephs} = 365.0 \cdot \frac{kip \cdot ft}{ft}$$

$$CheckMPHSb := if\left(\phi M_{mphs} > 1.5 \cdot M_{mid}, "OKAY", "NO GOOD"\right)$$

$$CheckMPHSb = "OKAY"$$

Factor of Safety

$$FS_{MPHSb} := \frac{\phi M_{mphs}}{M_{mid}}$$

$$FS_{MPHSb} = 5.711$$

$$CheckEPHSb := if\left(\phi M_{ephs} > 1.5 \cdot M_{end}, "OKAY", "NO GOOD"\right)$$

$$CheckEPHSb = "OKAY"$$

Factor of Safety

$$FS_{EPHSb} := \frac{\phi M_{ephs}}{M_{end}}$$

$$FS_{EPHSb} = 2.855$$

FACTORS OF SAFETY SUMMARY

WALL 1 (EAST WALL)

GOOD

Shear

$FS_{S1} = 1.57$

Moment

$FS_{wallmomentA} = 3.13$

$FS_{wallmomentB} = 2.169$

$FS_{wallmomentC} = 5.96$

$FS_{wallmomentD} = 4.527$

GOOD

WALL 2 (NORTH EAST WALL)

$FS_{S2a} = 1.633$

$FS_{MPH} = 4.071$

$FS_{EPH} = 2.197$

$FS_{MPH1} = 4.071$

$FS_{EPH1} = 2.797$

BASE SLAB GOOD

Shear

$FS_{Sb} = 4.163$

Moment

$FS_{MPHSb} = 5.711$

$FS_{EPHSb} = 2.855$

STABILITY

Stability = 2.035

SEE NEXT PAGE FOR THE RECOMMENDATIONS

RECOMMENDATIONS

END OF ANALYSIS AND RECOMMENDATIONS

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Units system: English
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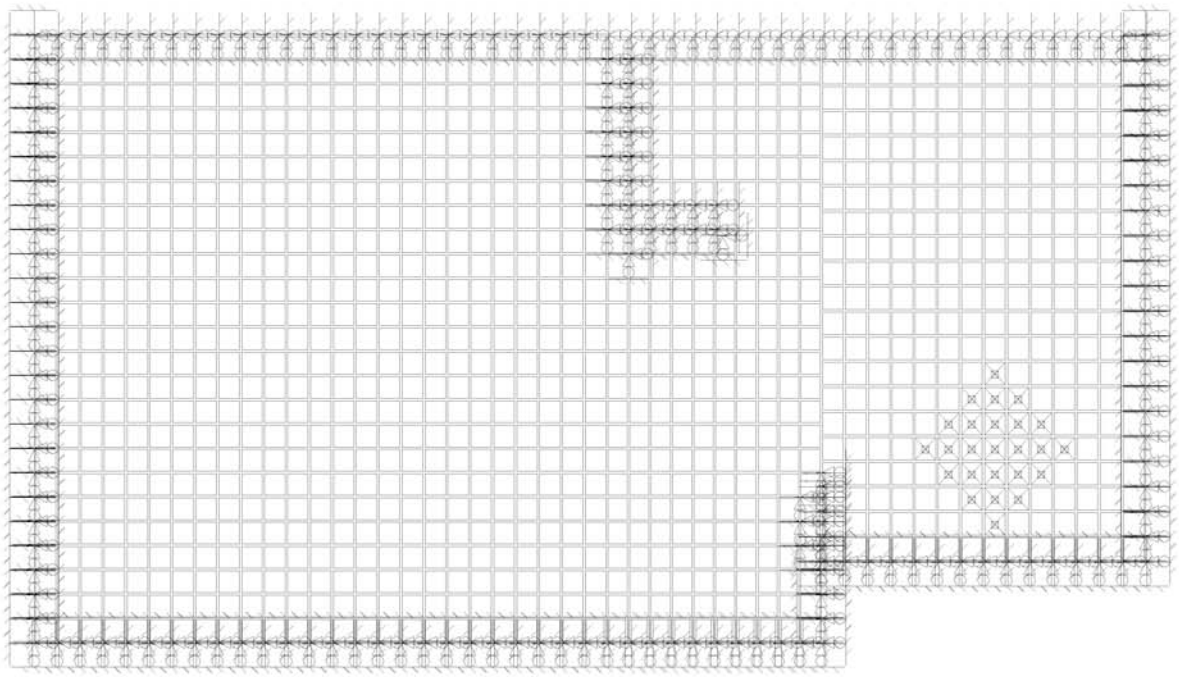
New Central Pump Station Landside (Northeast) Wall



New Central Wall Support Conditions

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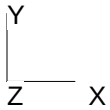

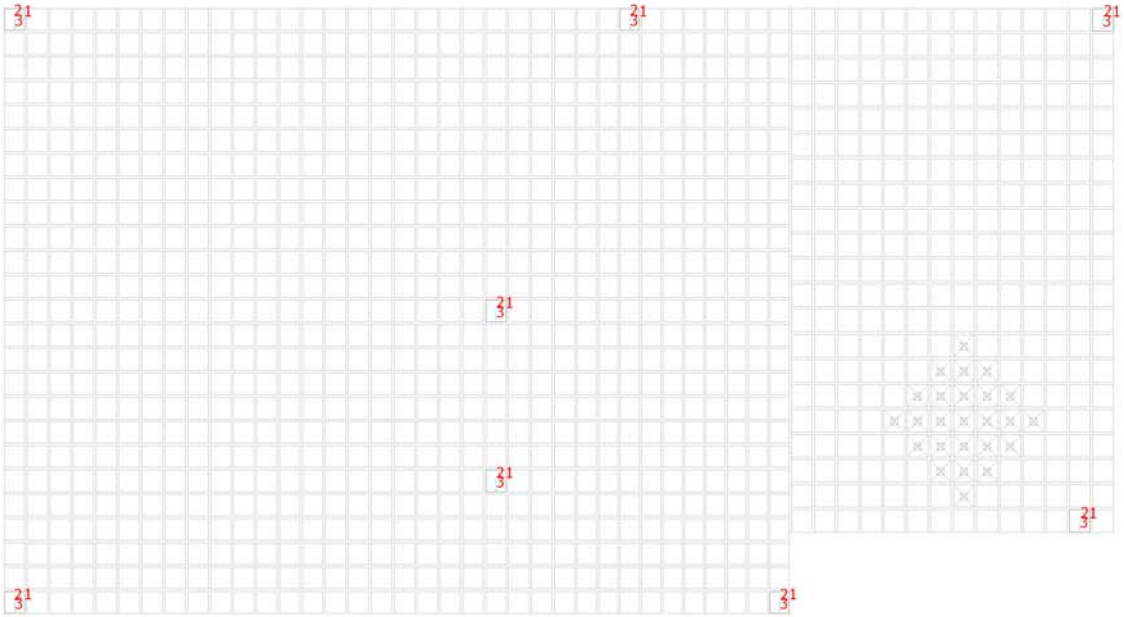

New Central pump station local axes orientation. M11 moment is a moment acting on the vertical steel. M33 moment is a moment acting on the horizontal steel.

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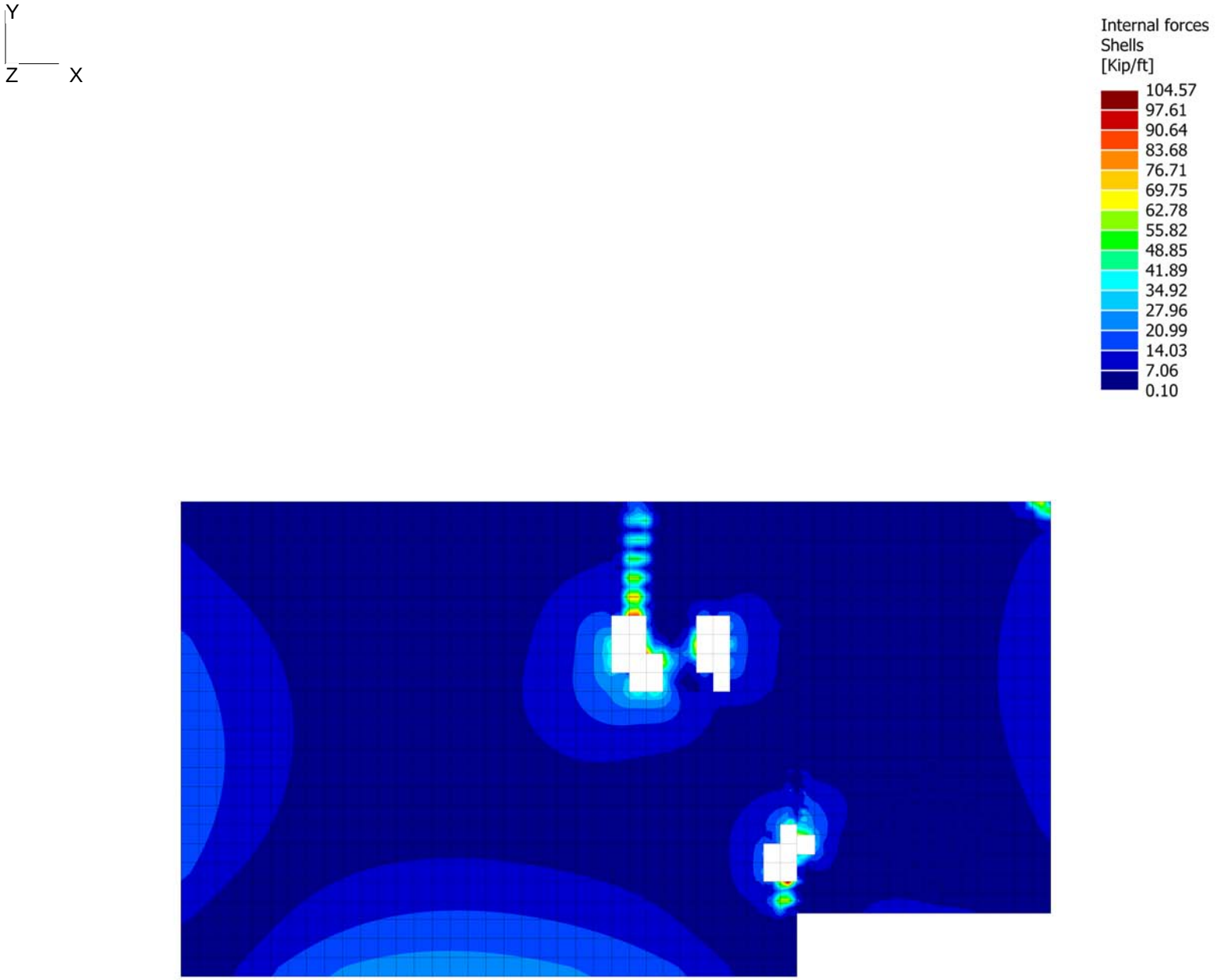

New Central pump station landside wall shear contour, Vmax for existing conditions.

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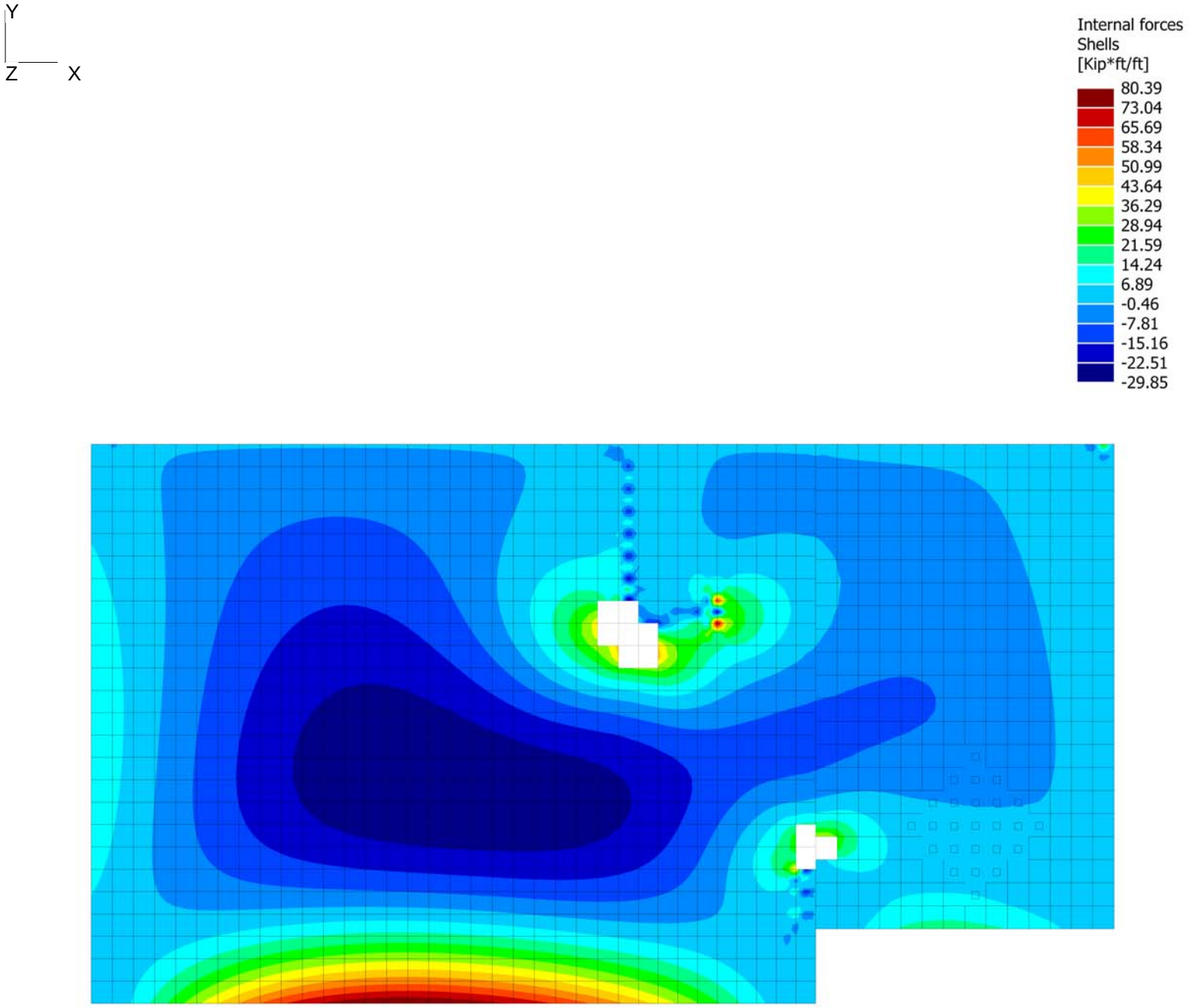

New Central pump station M11 moment contours for existing condition.
M11 is moment acting on the vertical reinforcement.

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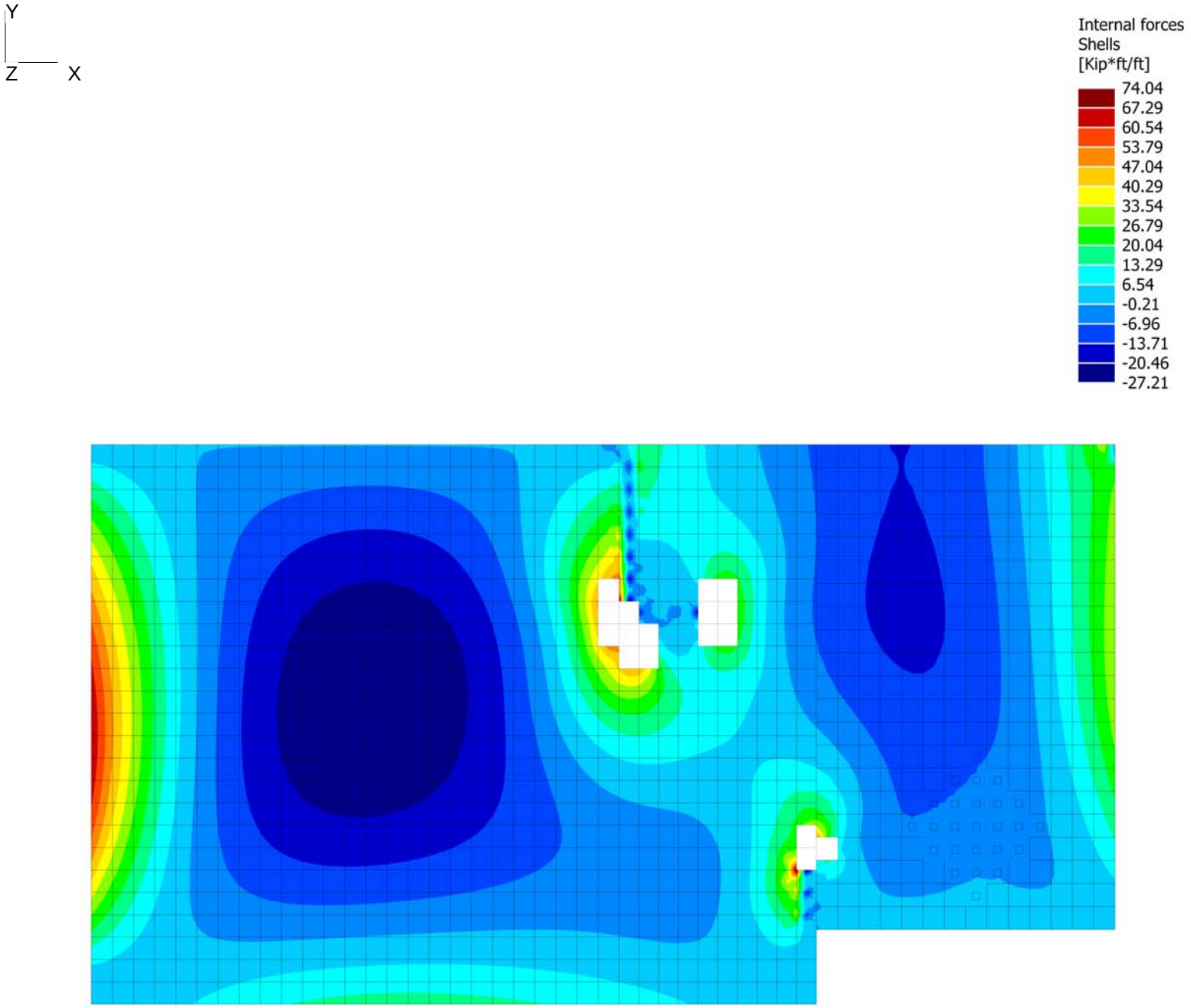

New Central pump station M33 contour lines for the existing condition.
M33 is the moment acting on the horizontal reinforcement.

US Army Corps of Engineers

Pipe Strength Design

I. Design Criteria for Strength

Refer to figure 3-2 from EM 1110-2-2902 for the embankment conditions

Class B Wall Thickness was used and Concrete Cradle Bedding. Wall B pipe thickness assumed in calculations. (The wall thickness does not impact the pipe strength, only Class). The stamped drawings show that Class IV pipe was used in construction with a concrete cradle. The pipe section extending from the pump station to the outlet was evaluated. An embankment load condition is assumed to be present.

II. Assigned Variables

The values given below remain constant:

1 Kip equals 1000 lbs. $Kip := 1000 \cdot lb$

inside pipe diameter (d_2): $d_2 := 7 \cdot ft$

$d_2 = 84 \cdot in$

weight of water: $\delta_W := 0.0624 \cdot \frac{Kip}{ft^3}$

concrete density: $\delta_C := 0.150 \cdot \frac{Kip}{ft^3}$

soil density: $\delta_S := 0.120 \cdot \frac{Kip}{ft^3}$

Wall thickness of pipe:
Class A $ClassA := 7in$
Class B $ClassB := 8in$
Class C $ClassC := 8.75in$

Pipe weight for given class:

$W_{ClassA} := 2.09 \cdot \frac{Kip}{ft}$ $W_{ClassB} := 2.41 \cdot \frac{Kip}{ft}$ $W_{ClassC} := 2.66 \cdot \frac{Kip}{ft}$

inside pipe area:

$$Areapipe := \left(\frac{d_2}{2}\right)^2 \cdot \pi \qquad \pi = 3.142$$

$$Areapipe = 38.485\,ft^2 \qquad d_2 = 7\,ft$$

interior pipe volume:

$$Volpipe := Areapipe \cdot 1 \cdot ft$$

$$Volpipe = 287.884\,gal$$

weight of water in pipe:

$$W_{WATER_PIPE} := Volpipe \cdot \delta_W$$

$$W_{WATER_PIPE} = 2401.43\,lb$$

Weight of arch OVER pipe

$$W_{arch} := 0\,\frac{lb}{ft}$$

III. Pipe Strength Design, Embankment Condition:

$ClassB = 8 \cdot in$

Refer to figure 3-2 from EM 1110-2-2902 for the embankment conditions

Input the values of highway loads present for pipe size and depth: $H_h := 27 .. 32$

$W_{L_{H_h}} :=$

$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
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$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$

A. Design Loads When Concrete Cradle Bedding Is Present

Refer to figure 3-2 from EM 1110-2-2902 for the embankment conditions

Variables:

The wall thickness used for design is "B"

Inside Pipe Diameter: $d_2 = 7\,ft$

Outside Pipe Diameter: $b_c := d_2 + 2ClassB$ $ClassB = 0.667\,ft$

$b_c = 8.333\,ft$

Projection Ratio: $p := 0.7$ Use the maximum value of p for 1st Class Bedding is 0.7

Depth of Pipe, Centerline of Pipe to Final Grade: $H_h := 27 .. 32$

Soil Density: $\delta_S = 120 \frac{lb}{ft^3}$

Water Density: $\delta_W = 62.4 \frac{lb}{ft^3}$

Live Load on Pipe: W_L

Earth Load on Pipe: W_E

Fluid Load on Pipe: $W_{WATER_PIPE} := \pi \cdot \left(\frac{d_2}{2}\right)^2 \cdot \delta_W$

$W_{WATER_PIPE} = 2401.433 \frac{lb}{ft}$

Bedding Factor X_P: $X_P := 0.505$

Bedding Factor X_A: $X_A := 0.811$

Bedding Factor: $B_f := \frac{1.431}{X_P - \frac{X_A}{3}}$

$B_f = 6.098$

Total Weight on Pipe: W_T

Hydraulic Factor: $H_f := 1.3$

Design Load: $D_{0.01}$

$$W_{E_{H_h}} := H_h \cdot b_c \cdot \delta_S \cdot 1.5 \cdot ft$$

$$W_{T_{H_h}} := W_{WATER_PIPE} + W_{E_{H_h}} + W_{L_{H_h}}$$

$$D_{0.01_CC_{H_h}} := \frac{H_f \cdot W_{T_{H_h}}}{d_2 \cdot B_f}$$

The Design Loads for Concrete Cradle Bedding at Various Values of Pipe Depth H_h:

$p = 0.7$ $d_2 = 7\,ft$ $B_f = 6.098$ $X_A = 0.811$

$X_P = 0.505$ $b_c = 8.333\,ft$ $W_{WATER_PIPE} = 2401\,\frac{lb}{ft}$

$W_{E_{H_h}} =$

$4 \cdot 10^4$	$\frac{lb}{ft}$
$4 \cdot 10^4$	
$4 \cdot 10^4$	
$4 \cdot 10^4$	
$5 \cdot 10^4$	
$5 \cdot 10^4$	

$W_{T_{H_h}} =$

42901	$\frac{lb}{ft}$
44401	
45901	
47401	
48901	
50401	

$D_{0.01_CC_{H_h}} =$

1307	$\frac{lb}{ft^2}$
1352	
1398	
1444	
1489	
1535	

$D_{0.01}$-CC is the design load using a hydraulic factor equal to 1.3.

The greatest pipe depth:

$$\frac{b_c}{2} = 4.167\ \text{ft}$$

Per ASTM C 76
"*Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe*"

Class	D-load to produce 0.01-in. crack
I	800
II	1000
III	1350
IV	**2000**
V	3000

$$D_{0.01_CC_{27}} = 1307\ \frac{\text{lb}}{\text{ft}^2}$$

D-load present at x feet with Concrete Cradle bedding

$$D_{0.01_CC_{28}} = 1352\ \frac{\text{lb}}{\text{ft}^2}$$

$$D_{0.01_CC_{29}} = 1398\ \frac{\text{lb}}{\text{ft}^2}$$

$$D_{0.01_CC_{30}} = 1444\ \frac{\text{lb}}{\text{ft}^2}$$

$$D_{0.01_CC_{31}} = 1489\ \frac{\text{lb}}{\text{ft}^2}$$

$$D_{0.01_CC_{32}} = 1535\ \frac{\text{lb}}{\text{ft}^2}$$

Summary: On July 30, 2007 the New Central PS was inspected by USACE. Due to accessibility issues the RCP was unable to be entered for a more thorough investigation. The 84" RCP is considered to be RELIABLE for Existing Conditions. The ASTM C76 specified D0.01 crack capacity of the New Central 84 inch diameter RCP is 2000 lb/ft/ft. For the existing condition (depth at springline = 29.4), the pipe demand is 1400 lb/ft/ft. The pipe demand exceeds the pipe capacity when top of levee elevation is is 773' or greater(See spreadsheet in New Central file under B4 peer review) or when an additional 43 feet of cover is placed over the RCP.

Variables

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

REFERENCES
1. Drawings, dated 1986 Black and Veatch
2. HGL, blanket elevations, levee elevations supplied EC-G

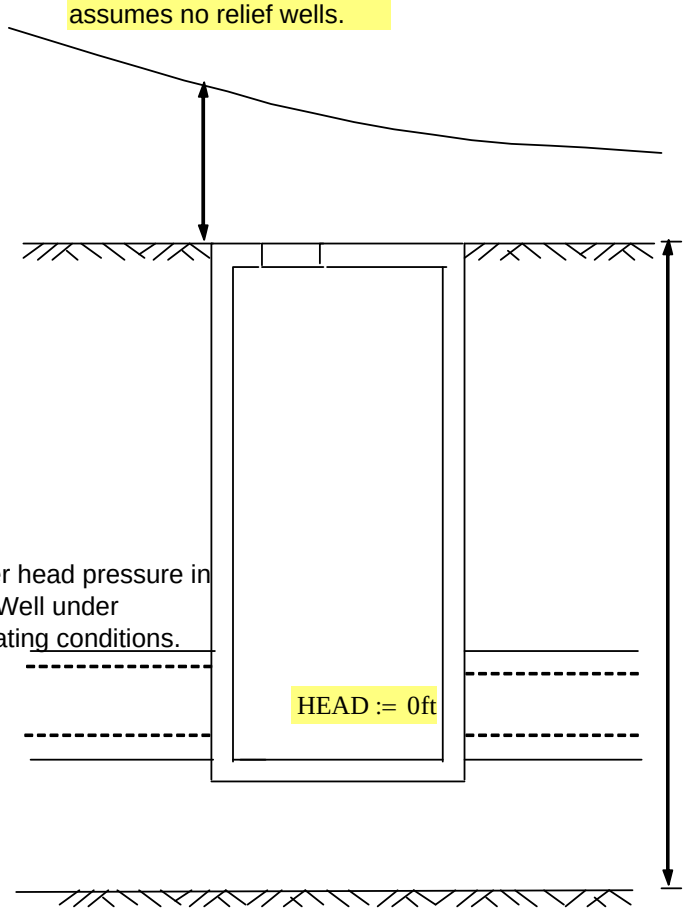
Top of existing levee El = 765.90
Bottom of blanket El = 716
Ground El = 762.50

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 = 765.90 ft



Elevation

$ELEV_1 := 762.50ft$

$HGL := 761.94ft - ELEV_1$

$HGL = -0.56\,ft$

Water head pressure in Wet Well under operating conditions.

$BLANKET := 762.50ft - 716ft$

$BLANKET = 46.5\,ft$

$HEAD := 0ft$

$ELEV_2 := 729.9ft$

$H := ELEV_1 - ELEV_2$

$H = 32.6\,ft$

$H_w := 754.9ft - ELEV_2$

$H_w = 25\,ft$

Hw is height of 4 foot thick walls to bottom of well (North and West)

The Gateway 2000 Pump Plant was constructed in 1994 to serve the area surrounding the Gateway 2000 facilities. The pump plant is located in Kansas and discharges through the CID Kansas Unit, However, a portion of the contributing service area is located in Missouri. The pump plant was designed to have certain pumping capacities at the given river stages. The station contains two 9600gpm submersible pumps, and has a discharge pipe at 60"RCP.

Since structure is relatively symetrical and no significant variances in wall dimensions are apparent, east and west wall can be treated as identical as well as the north and south walls. This will allow for an efficient means to determine strength/stability.

Wall 1 Parameters
In place wall

North wall

$L_1 := 33.1ft$

$H_w = 25\,ft$

$D_1 := 48in$

$C_{c1} := 2in$

Wall 2 Parameters
In place wall

West Wall

$L_2 := 25ft$

$H_w = 25\,ft$

$D_2 := 48in$

$C_{c2} := 2in$

Lengths are taken center to center on wall

Analyze 4' thick walls only

BASE SLAB

The base slab reinforcement is shown on sheet 11 of 25.

Bar size floor = # 6 @ 6" EF EW

Floor Thickness $D_f := 48in$ Clear Cover $C_{cf} := 2in$ $d_f := D_f - C_{cf} - \frac{.88in}{2}$ $d_f = 45.56 \cdot in$

EFFECTIVE DEPTHS

WALL 1

$d_1 := D_1 - \left(C_{c1} + \frac{.75}{2}in\right)$

$d_1 = 45.625 \cdot in$

WALL 2

$d_2 := D_2 - \left[C_{c2} + \left(\frac{.75}{2}in\right)\right]$

$d_2 = 45.625 \cdot in$

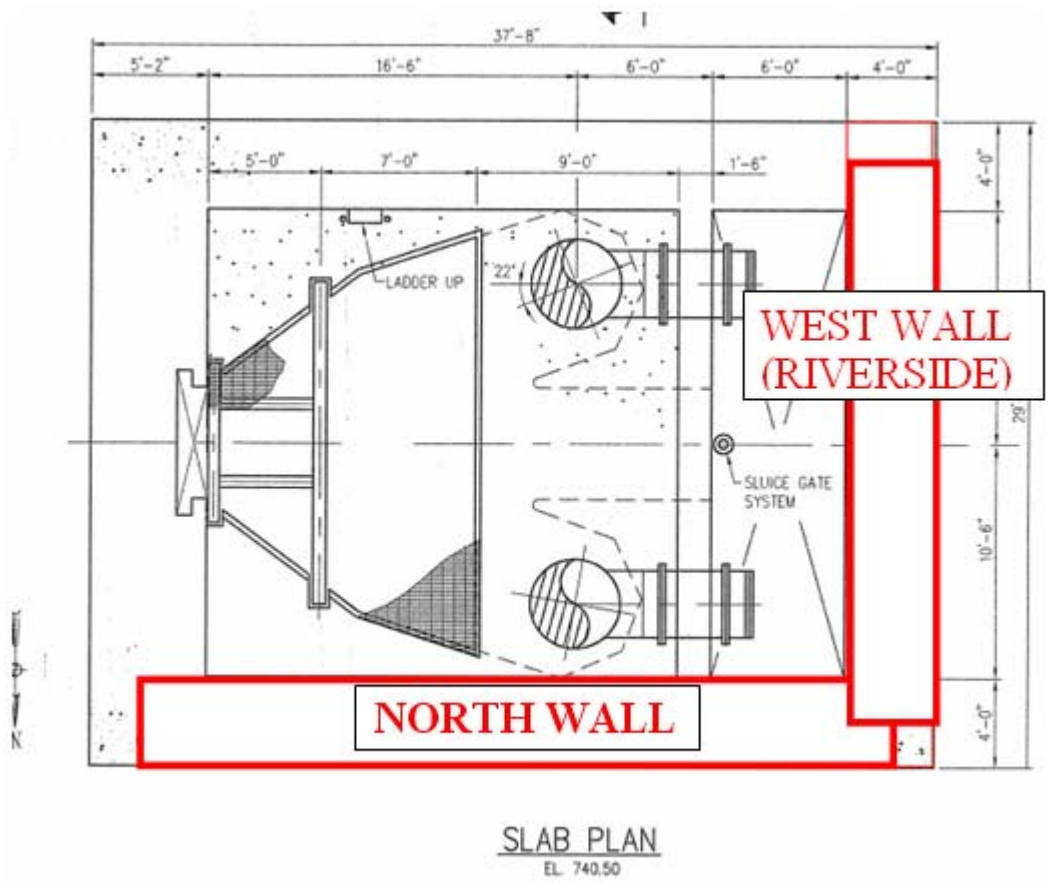


Figure 1. Gateway 2000 P. S. 80+90 Plan View Drawing Sheet # 9 of 25 T&B 1996

Assumptions and Criteria

Per drawing C11, all reinforcement is GR60 & concrete minimum strength is 4000psi.

Soil Properties	$\phi := 21deg$	$K_o := 1 - \sin(\phi)$	Steel Properties	$F_y := 60ksi$
	$\gamma := 110pcf$	$K_o = 0.642$	Concrete Properties	$f'_c := 4.00ksi$
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)

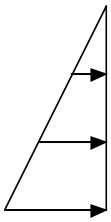
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALL 1, (North wall) In place wall $K_o = 0.642$ $BLANKET = 46.5\,ft$ $HGL = -0.56\,ft$

Wall Loading

Soil or Soil & Water

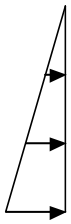


$H = 32.6\,ft$

$H_2 := H$

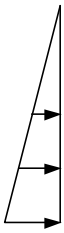
$H_2 = 32.6\,ft$

OR



$H_2 = 32.6\,ft$

&



$H'_1 := \frac{BLANKET + HGL}{BLANKET} \cdot H - HEAD$

$H''_1 := H + HGL - HEAD$

$H_1 := if(H_2 > BLANKET, H''_1, H'_1)$

$H_1 = 32.207\,ft$

$H'_1 = 32.207\,ft$

$H''_1 = 32.04\,ft$

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

$W_s = 2300.9 \cdot psf$

$W_w := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot [K_o \cdot (\gamma - \gamma_w) \cdot H_2 + \gamma_w \cdot H_1]$

$W_w = 3005 \cdot psf$

$W_{1ext} := if(W_s > W_w, W_s, W_w)$ $W_{1ext} = 3005.401 \cdot psf$

SHEAR

WALL 1, North Wall

$L_{tw} := 12.5ft$

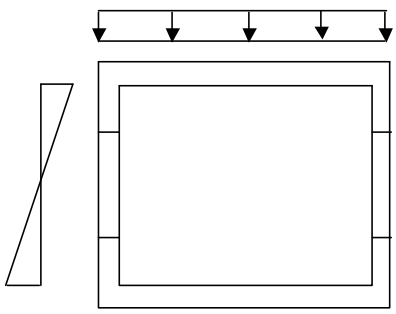
Ltw stands for length of thrust wall

$$THRUST_1 := \frac{W_{1ext} \cdot L_{tw}}{2}$$

$$V'_{u1} := W_{1ext} \cdot \frac{L_1}{2}$$

$$V'_{u1} = 49739 \frac{lb}{ft}$$

$$THRUST_1 = 18783.754 \frac{lb}{ft}$$



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

$$V_{u1} := 2 \frac{V'_{u1}}{L_1} \cdot \left(\frac{L_1}{2} - \frac{D_2}{2} - d_1 \right)$$

$$V_{u1} = 32302 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$A_g := D_1 \cdot 12 \frac{in}{ft} \qquad A_g = 576 \cdot \frac{in^2}{ft}$$

$$\phi V_{n1} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_1}{2000 \cdot A_g} \cdot \frac{in^2}{lb} \right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in} \right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_1 \right) \right]$$ ACI EQ (11-4)

$$\phi V_{n1} = 70383 \frac{lb}{ft}$$

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

Check1 = "OKAY"

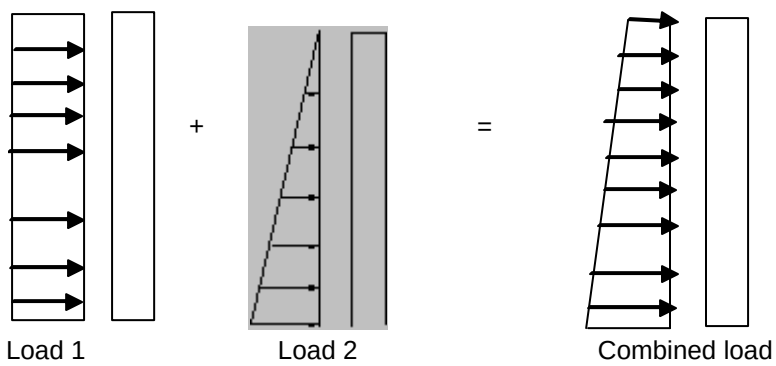
Factor of Safety

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

$$FS_{S1} = 2.179$$

Wall Analysis Wall 1

"Analyze as two-way action recognizing that the steel reinf. area is equal EF & EW. By considering only the portion of the wall (Hw) that has the 4' thickness creates a trapezoidal load which necessitates the need to combine loads "



Load Case 1: $ELEV_1 - 754.90ft = 7.6\,ft$

Assuming no internal wall "Very Conservative"

$a := 16.6ft$ $\quad b := 25ft$ $\quad \frac{a}{b} = 0.664$ $\quad k := 1..7$

$Load_1 := 7.6ft \cdot (\gamma - \gamma_w) \cdot K_o + \gamma_w \cdot 3ft = 419.317 \cdot psf$

$Load_2 := W_{1ext} = 3005.401 \cdot psf$

Using plate analysis (pg. 7 Bur. of Rec) fixed on all sides free at top.

$Combined_{load} := Load_2 - Load_1 = 2586.084 \cdot psf$

Plate Loading (Bending) $\quad l_1 := 25ft \quad l_2 := 16.6ft$

$\frac{l_2}{l_1} = 0.664$

a/b=L_2/L_1 $\quad \frac{l_2}{l_1} = 0.664$

$\frac{l_2}{l_1} = 0.664$

M.x = Horizontal steel
M.y = Vertical steel

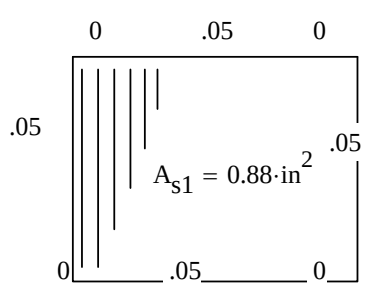
$A_{s1} := .88in^2$ (Interior steel and end steel of Span)

$ab_k :=$	$CoeffMx_k :=$	$CoeffMy_k :=$
.125	.0052	.0033
.25	.0209	.0135
.375	.0476	.0288
.5	.0852	.0534
.75	.1788	.1212
1	.2613	.2043
1.5	.3304	.3508

$CMx := linterp\left(ab, CoeffMx, \frac{a}{b}\right) \quad CMx = 0.147$

$CMy := linterp\left(ab, CoeffMy, \frac{a}{b}\right) \quad CMy = 0.098$

6's @ 6" O.C.



$Coefficient_x := linterp\left(ab, CoeffMx, \frac{l_2}{l_1}\right)$

$Coefficient_x = 0.1466$

$Coefficient_y := linterp\left(ab, CoeffMy, \frac{l_2}{l_1}\right)$

$Coefficient_y = 0.0979$

$b_1 := 1ft$

$l_2 = 16.6\,ft$

$M_{uF.x} := Coefficient_x \cdot Load_1 \cdot l_1^2 \cdot b_1 = 38.42 \cdot kip \cdot ft$

$M_{uFy} := Coefficient_y \cdot Load_1 \cdot l_1^2 \cdot b_1 = 25.651 \cdot kip \cdot ft$

Load Case 2:

Using plate analysis (pg. 10 Bur. of Rec) fixed on all sides free at top.

$k := 1 .. 7$

$\frac{a}{b} = 0.664$

From plates tables:

Interpolation :

$ab_k :=$	$CoeffMx_k :=$	$CoeffMy1_k :=$
.125	.0038	.003
.25	.0114	.0107
.375	.0208	.0200
.5	.0277	.0325
.75	.0433	.0584
1	.0644	.0845
1.5	.0857	.1262

$CMx := linterp\left(ab, CoeffMx, \frac{a}{b}\right)$ $CMx = 0.038$

$CMy1 := linterp\left(ab, CoeffMy1, \frac{a}{b}\right)$ $CMy1 = 0.049$

Since Mx (exterior steel) coefficient is greater for Load Case 1 (0.1466) than 2 use this to calculate MuFx. Since the My for the vertical steel is greater than the MuFx in both load cases, this controls and therefore you must add the two load cases together to determine MuFy (max) or TotMom.vert and check for strength FOS.

$M_{uFy1} := CMy1 \cdot Combined_{load} \cdot l_1^2 \cdot b_1 = 79.991 \cdot kip \cdot ft$

"Therefore Total Moment at Vertical Steel"

$TotMom_{vert} := M_{uFy} + M_{uFy1} = 105.642 \cdot kip \cdot ft$

$a := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $a = 0.052 \cdot in$

$M_{uF.x} = 38.42 \cdot kip \cdot ft$

$d_f = 3.797\,ft$

$\phi M_{nF} := \phi_B \cdot A_{s1} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nF} = 200.35\,ft \cdot kip$

$Wall1_Bending_Check := if\left(\phi M_{nF} > 1.5 M_{uF.x}, "OK", "NO GOO\right.$ $Wall1_Bending_Check = "OK"$

Factor of Safety $FS_{wall1moment1} := \frac{\phi M_{nF}}{M_{uF.x}}$ $FS_{wall1moment1} = 5.215$

$TotMom_{vert}$

$a := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $a = 0.052 \cdot in$ $d_f = 3.797\ ft$

$\phi M_{nF} := \phi_B \cdot A_{s1} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nF} = 200.35\ ft \cdot kip$

$Wall1_Bending_Check := if\left(\phi M_{nF} > 1.5 TotMom_{vert}, "OK", "NO\ GOOD"\right)$ $Wall1_Bending_Check = "OK"$

Factor of Safety $FS_{wall1moment1a} := \frac{\phi M_{nF}}{1.5 \cdot TotMom_{vert}}$ $FS_{wall1moment1a} = 1.264$

FOS= 1.31 without internal wall support. Since an intermediate wall support is present, the wall is considered reliable for moment when exposed to EC loading.

WALL 2, West Wall

The west wall has the same reinf. design and thickness as the north wall but has a shorter length. The support conditions for the west wall are also the same as the north wall. Based on these parameters and results obtained from analyzing the north wall, the west wall is adequate for shear and moment strength when exposed to existing condition (EC) loading.

Uplift on Structure Refer to Figure 2 for the identified areas concerning uplift calculations

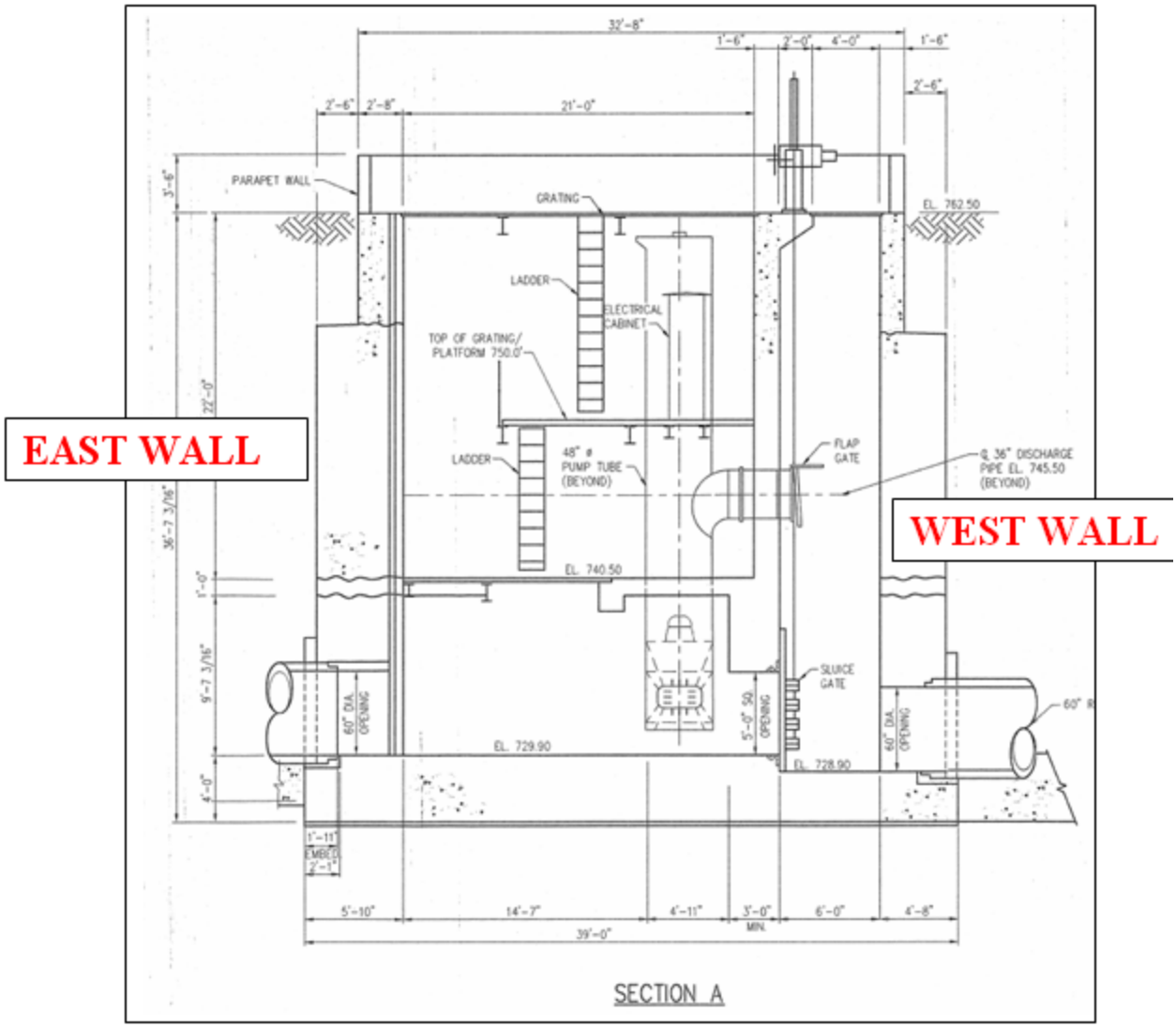


Figure 2. Gateway 2000 P.S. Section cut facing North sheet #10 of 25.

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

Uplift on Structure

Weight of wetwell.The equipment weight was not included.

Top Slab

$Top := 12in \cdot (0ft) \cdot (0ft)$ $\quad Wtop := Top \cdot 150pcf$ $\quad Wtop = 0 \cdot kip$

Walls

$West := 32.5ft \cdot 25ft \cdot 4ft$ $\quad Wwest := West \cdot 150pcf$ $\quad Wwest = 487.5 \cdot kip$

$South := 32.5ft \cdot 25ft \cdot 5.17ft$ $\quad Wsouth := South \cdot 150pcf$ $\quad Wsouth = 630.094 \cdot kip$

$East := West$ $\quad Weast := Wwest$ $\quad Weast = 487.5 \cdot kip$

$North := South$ $\quad Wnorth := North \cdot 150pcf$ $\quad Wnorth = 630.094 \cdot kip$

Base Slab.A

$Base := 4 \cdot ft \cdot (30.33ft) \cdot (39ft)$

$Base = 4731.48 \cdot ft^3$ $\quad Wbase := Base \cdot 150pcf$ $\quad Wbase = 709.722 \cdot kip$

$Basearea := (30.33ft) \cdot (39ft)$ $\quad Basearea = 1182.87\ ft^2$

South Wall above grade

$hsouth_{ag} := 0ft$

$South_{ag} := hsouth_{ag} \cdot 8in \cdot (23ft)$ $\quad Wsouth_{ag} := South_{ag} \cdot 120pcf$ $\quad Wsouth_{ag} = 0 \cdot kip$

$Wnorth_{ag} := Wsouth_{ag}$

West Wall above Grade

$hwest_{ag} := 13ft$

$West_{ag} := hwest_{ag} \cdot (28.16ft) \cdot 8in$ $\quad Wwest_{ag} := West_{ag} \cdot 120pcf$ $\quad Wwest_{ag} = 29.286 \cdot kip$

$Weast_{ag} := Wwest_{ag}$

Gatewell Above Grade

$Wgatewell := 0ft \cdot 5ft \cdot 5ft$ $\quad Wgatewell := Wgatewell \cdot 150pcf$ $\quad Wgatewell = 0$

Gatewell attached to structure

Wetwell Weight

$$Wwetwell := Wtop + Wwest + Wsouth + Weast + Wnorth + Wbase \ldots + Wsouth_{ag} + Wwest_{ag} + Weast_{ag} + Wnorth_{ag} + Wgatewell$$

$Wwetwell = 3003.482 \cdot kip$

Wetwell Structure Factor of Safety for Flotation

Weight of water that is present during the event

$Wellarea := Basearea$ $\quad Wellarea = 1182.87\ ft^2$

Bottom of wetwell base elevation

$Elevwetwellbot := 730ft$

$HEAD := 5.1ft$

bottom of blanket at 713

$Water := HEAD \cdot Wellarea \cdot 62.4pcf$ $\quad Water = 376.437 \cdot kip$

$heel := .67ft$ $\quad heel = 0.67\ ft$ The wet well does have heels

$upliftarea := Basearea$ $\quad upliftarea = 1182.87\ ft^2$

$H := ELEV_1 - Elevwetwellbot$ $\quad H = 32.5\ ft$ $\quad ELEV_1 = 762.5\ ft$ Grade elevation

Weight of soil on the wetwell heels

$volsoil := 5ft \cdot 32.6ft \cdot heel + (5ft \cdot 28.16ft) \cdot heel + 7.6ft \cdot 2.4ft \cdot 32.5ft$ $\quad volsoil = 796.346 \cdot ft^3$

$volsoil \cdot (\gamma - \gamma_w) = 37.906 \cdot kip$ $\quad volsoil \cdot \gamma_w = 49.692 \cdot kip$

Head, if structural foundation in blanket.

$HGL = -0.56\ ft$

$$U_1 := \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) \cdot upliftarea \cdot \gamma_w$$

$U_1 = 2369.971 \cdot kips$

$BLANKET = 46.5\ ft$
$HGL = -0.56\ ft$
$H = 32.5\ ft$

Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (upliftarea) \cdot \gamma_w\right]$$

$U_2 = 2357.526 \cdot kips$

$$Uplift_{wet} := if\left(H > BLANKET, U_2, U_1\right)$$

$$Uplift_{wet} = 2369.971 \cdot kips \quad \left(\frac{BLANKET + HGL}{BLANKET} \cdot H\right) = 32.109\ ft$$

The uplift force acting on the wetwell structure $\quad Uplift_{wet} = 2369.971 \cdot kips$

$$Stability := \frac{Wwetwell + Water + volsoil \cdot (\gamma - \gamma_w)}{Uplift_{wet} - volsoil \cdot \gamma_w}$$

$Stability = 1.473$ $\quad (H + HGL) = 31.94\ ft$

$StabilityCheck := if(Stability \geq 1.1, "Okay", "Not Okay")$ $\quad StabilityCheck = "Okay"$

Wet Well BASE SLAB ANALYSIS

$W_{water} := Water$ WEIGHT OF WATER

Determine the amount of additional weight required to achieve FS = 1.1 of the wetwell structure

$BaseSlab := upliftarea$ $BaseSlab = 1182.87\ ft^2$ $Uplift_{wet} = 2369.971 \cdot kip$

SHEAR

$THRUST_S := 0kip$ $THRUST_S = 0$ $\frac{Wwetwell + W_{water}}{BaseSlab} = 2857.388 \cdot psf$

$W' := if\left(Stability < 1.0, \frac{Uplift_{wet}}{BaseSlab}, \frac{Wwetwell}{BaseSlab}\right)$ $W' = 2539.148 \cdot psf$ $\frac{Uplift_{wet}}{BaseSlab} = 2003.577 \cdot psf$

$W' = 2539.148 \cdot psf$ $\frac{Wwetwell + volsoil \cdot (\gamma - \gamma_w)}{BaseSlab} = 2571.194 \cdot psf$

$Slab_t := D_f$ $L_{strip} := 1ft$ $Wall_t := D_1$ $Slab_d := d_f$ effective depth of slab

Analyze slab with two-way action

$Slab_d = 45.56 \cdot in$

$D_1 = 48 \cdot in$

$L_s := 33.1ft$

$V'_{ub} := \frac{W' \cdot L_s}{2}$ $V'_{ub} = 42022.904 \frac{lb}{ft}$

$d_f = 45.56 \cdot in$

$D_f = 4\ ft$

$V_{ub} := \frac{V'_{ub}}{\frac{L_s}{2}} \cdot \left(\frac{L_s}{2} - \frac{Wall_t}{2} - Slab_d\right)$ $V_{ub} = 27304 \frac{lb}{ft}$

$A_g := Slab_t \cdot 12 \frac{in}{ft}$ $A_g = 576 \cdot \frac{in^2}{ft}$

$Water = 376436.549\ lb$

$\phi V_{nb} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{THRUST_S}{2000 \cdot A_g} \cdot \frac{in^2}{lb}\right) \cdot \left(\sqrt{f'_c} \cdot \frac{lb^{.5}}{in}\right) \cdot \left(12 \cdot \frac{in}{ft} \cdot d_f\right)\right]$ ACI EQ (11-4) $\phi V_{nb} = 69155 \frac{lb}{ft}$

$CheckBase := if\left(\phi V_{nb} > 1.5V_{ub}, "OKAY", "NO\ GOOD"\right)$ $CheckBase = "OKAY"$

Factor of Safety $FS_{Sb} := \frac{\phi V_{nb}}{V_{ub}}$ $FS_{Sb} = 2.533$

Slab Bending

Assume that the horizontal reinforcement carries all of the lateral loads

$b := 12\frac{in}{ft}$

The mid-span negative moment $M_{mid} := \frac{W' \cdot L_s^2}{24}$ $M_{mid} = 115.913 \cdot \frac{kip \cdot ft}{ft}$

The end-span positivie moment $M_{end} := \frac{W' \cdot L_s^2}{12}$ $M_{end} = 231.826 \cdot \frac{kip \cdot ft}{ft}$

6's @ 6 in. (I.F) inside face
(mid-plate moment)

$A_{mph} := .88\frac{in^2}{ft}$ $a_{mph} := \frac{A_{mph} \cdot F_y}{0.85f'_c \cdot b}$ $a_{mph} = 1.294 \cdot in$

$\phi M_{mphs} := \phi_B \cdot A_{mph} \cdot F_y \cdot \left(d_f - \frac{a_{mph}}{2}\right)$

6's at 6 (O.F)
(edge-plate moment)

$A_{eph} := .88\frac{in^2}{ft}$ $a_{eph} := \frac{A_{eph} \cdot F_y}{0.85f'_c \cdot b}$ $a_{eph} = 1.294 \cdot in$

$\phi M_{ephs} := \phi_B \cdot A_{eph} \cdot F_y \cdot \left(d_f - \frac{a_{eph}}{2}\right)$ $\phi M_{ephs} = 197.6 \cdot \frac{kip \cdot ft}{ft}$

$CheckMPHSb := if\left(\phi M_{mphs} > 1.5 \cdot M_{mid}, "OKAY", "NO\ GOOD"\right)$ $CheckMPHSb = "OKAY"$

Factor of Safety $FS_{MPHSb} := \frac{\phi M_{mphs}}{M_{mid}}$ $FS_{MPHSb} = 1.705$

$CheckEPHSb := if\left(\phi M_{ephs} > 1.5 \cdot M_{end}, "OKAY", "NO\ GOOD"\right)$ $CheckEPHSb = "NO\ GOOD"$

Factor of Safety $FS_{EPHSb} := \frac{\phi M_{ephs}}{M_{end}}$ $FS_{EPHSb} = 0.852$

Base Slab Analysis

$k := 1..7$

$a := 30.33$ $b := 39ft$

$\frac{a}{b} = 0.778$

Edge Moments:

Plate Loading
(Bending)

$a/b=L_2/L_1$

M.x = Horizntal steel

$A_{slb} := .88in^2$ (Middle of Span)

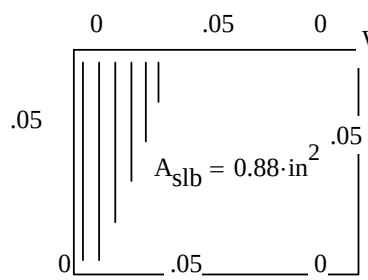
$ab_k :=$	$CoeffMx_k :=$	$CoeffMy_k :=$
.125	.0038	.003
.375	.0830	.0516
.5	.0815	.0538
.625	.0765	.0547
.75	.0686	.0546
.875	.0592	.0530
1.0	.0500	.0500

Using plate analysis (pg.40 Bur. of Rec) fixed on all sides.

$CMx := linterp\left(ab, CoeffMx, \frac{a}{b}\right)$ $CMx = 0.067$

$CMy := linterp\left(ab, CoeffMy, \frac{a}{b}\right)$ $CMy = 0.054$

6's @ 6" O.C
E.F. & E.W..



$W_{UNIFORM} := W'$

$W_{UNIFORM} = 2539.148 \cdot psf$

$b := 1ft$

$l_2 = 16.6\ ft$

$a := \frac{A_{slb} \cdot F_y}{0.85f'_c \cdot b}$ $a = 1.294 \cdot in$

$M_{uF} := CMx \cdot W_{UNIFORM} \cdot l_1^2 \cdot b$

$M_{uF} = 105.561 \cdot kip \cdot ft$

$d_f = 3.797\ ft$

$\phi M_{nF} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nF} = 197.617\ ft \cdot kip$

$Wall1_Bending_Check := if\left(\phi M_{nF} > 1.5M_{uF}, "OK", "NO\ GOOD"\right)$ $Wall1_Bending_Check = "OK"$

Factor of Safety $FS_{slabmoment} := \frac{\phi M_{nF}}{M_{uF}}$

$FS_{slabmoment} = 1.872$

Recognizing that CMy is less than CMx which corresponds to a higher FOS, therefore CMx controls since the same steel is going EF and EW one calculation can be done for two-way action.

FACTORS OF SAFETY SUMMARY

WALL 1 (NORTH WALL) GOOD

Shear

$FS_{S1} = 2.179$

Moment

$FS_{wall1moment1} = 5.215$

$FS_{wall1moment1a} = 1.264$

FOS= 1.3 without internal wall support. Since an intermediate wall support is present, the wall is considered reliable for moment when exposed to FC loading.

GOOD

WALL 2 (WEST WALL)

The west wall has the same reinf. design and thickness as the north wall but has a shorter length. The support conditions for the west wall are also the same as the north wall. Based on these parameters and results obtained from analyzing the north wall, the west wall is adequate for shear and moment strength when exposed to existing condition (EC) loading.

BASE SLAB

GOOD

Shear

$FS_{Sb} = 2.533$

Moment

Using Two Way Action

$FS_{slabmoment} = 1.872$

STABILITY

$Stability = 1.473$ Good

SEE NEXT PAGE FOR THE RECOMMENDATIONS

RECOMMENDATIONS

END OF ANALYSIS AND RECOMMENDATIONS

US Army Corps of Engineers

Gateway Pipe Strength Design Analysis

I. Design Criteria for Strength

Refer to figure 3-2 from EM 1110-2-2902 for the embankment conditions

Class B Wall Thichness was used and Concrete Cradle Bedding. Wall B pipe thickness assumed in calculations. (The wall thickness does not impact the pipe strength, only Class). The stamped drawings show that Class V pipe was used in construction with a concrete cradle. The pipe section extending from the pump station to the outlet was evaluated. An embankment load condition is assumed to be present.

See sheet # 5 of 25 T&B drawings set

II. Assigned Variables

The values given below remain constant:

1 Kip equals 1000 lbs. $Kip := 1000 \cdot lb$

inside pipe diameter (d_2): $d_2 := 5 \cdot ft$

weight of water: $\delta_W := 0.0624 \cdot \frac{Kip}{ft^3}$

concrete density: $\delta_C := 0.150 \cdot \frac{Kip}{ft^3}$

soil density: $\delta_S := 0.120 \cdot \frac{Kip}{ft^3}$

$d_2 = 60 \cdot in$

Wall thickness of pipe:
Class A
Class B
Class C

$ClassA := 5in$

$ClassB := 6in$

$ClassC := 6.75in$

Pipe weight for given class:

$W_{ClassA} := 1.1 \cdot \frac{Kip}{ft}$ $W_{ClassB} := 1.3 \cdot \frac{Kip}{ft}$ $W_{ClassC} := 1.5 \cdot \frac{Kip}{ft}$

inside pipe area:

$Areapipe := \left(\frac{d_2}{2}\right)^2 \cdot \pi$ $\pi = 3.142$

$Areapipe = 19.635\, ft^2$ $d_2 = 5\, ft$

interior pipe volume:

$Volpipe := Areapipe \cdot 1 \cdot ft$

$Volpipe = 146.88\, gal$

weight of water in pipe:

$W_{WATER_PIPE} := Volpipe \cdot \delta_W$

$W_{WATER_PIPE} = 1225.22\, lb$

Weight of arch OVER pipe $W_{arch} := 0 \frac{lb}{ft}$

III. Pipe Strength Design, Embankment Condition:

$ClassB = 6 \cdot in$

Refer to figure 3-2 from EM 1110-2-2902 for the embankment conditions

Input the values of highway loads present for pipe size and depth: $H_h := 27 .. 32$

$W_{L_{H_h}} :=$

$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
$0 \cdot \frac{Kip}{ft}$
Kip

A. Design Loads When Concrete Cradle Bedding Is Present

Refer to figure 3-2 from EM 1110-2-2902 for the embankment conditions

Variables:

The wall thickness used for design is "B"

Inside Pipe Diameter: $d_2 = 5\,ft$

Outside Pipe Diameter: $b_c := d_2 + 2ClassB$ $\quad ClassB = 0.5\,ft$

$b_c = 6\,ft$

Projection Ratio: $p := 0.7$ Use the maximum value of p for 1st Class Bedding is 0.7

Depth of Pipe, Centerline of Pipe to Final Grade: $H_h := 27 .. 32$

Soil Density: $\delta_S = 120 \frac{lb}{ft^3}$

Water Density: $\delta_W = 62.4 \frac{lb}{ft^3}$

Live Load on Pipe: W_L

Earth Load on Pipe: W_E

Fluid Load on Pipe: $W_{WATER_PIPE} := \pi \cdot \left(\frac{d_2}{2}\right)^2 \cdot \delta_W$

$W_{WATER_PIPE} = 1225.221 \frac{lb}{ft}$

Bedding Factor X_P: $X_P := 0.505$

Bedding Factor X_A: $X_A := 0.811$

Bedding Factor: $B_f := \frac{1.431}{X_P - \frac{X_A}{2}}$

$0 \cdot \frac{1}{ft}$

$B_f = 6.098$

Total Weight on Pipe: W_T

Hydraulic Factor: $H_f := 1.3$

Design Load: $D_{0.01}$

$$W_{E_{Hh}} := H_h \cdot b_c \cdot \delta_S \cdot 1.5 \cdot ft$$

$$W_{T_{Hh}} := W_{WATER_PIPE} + W_{E_{Hh}} + W_{L_{Hh}}$$

$$D_{0.01_CC_{Hh}} := \frac{H_f \cdot W_{T_{Hh}}}{d_2 \cdot B_f}$$

The Design Loads for Concrete Cradle Bedding at Various Values of Pipe Depth H_h:

$p = 0.7$ $d_2 = 5\,ft$ $B_f = 6.098$ $X_A = 0.811$

$X_P = 0.505$ $b_c = 6\,ft$ $W_{WATER_PIPE} = 1225\,\frac{lb}{ft}$

$W_{E_{Hh}}$ =	
$3 \cdot 10^4$	$\frac{lb}{ft}$
$3 \cdot 10^4$	
$3 \cdot 10^4$	
$3 \cdot 10^4$	
$3 \cdot 10^4$	
$3 \cdot 10^4$	

$W_{T_{Hh}}$ =	
30385	$\frac{lb}{ft}$
31465	
32545	
33625	
34705	
35785	

$D_{0.01_CC_{Hh}}$ =	
1296	$\frac{lb}{ft^2}$
1342	
1388	
1434	
1480	
1526	

$D_{0.01}$-CC is the design load using a hydraulic factor equal to 1.3.

The greatest pipe depth:

$$\frac{b_c}{2} = 3\,\text{ft}$$

Per ASTM C 76
"*Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe*"

Class	D-load to produce 0.01-in. crack
I	800
II	1000
III	1350
IV	2000
V	**3000**

$$D_{0.01_CC_{27}} = 1296\,\frac{\text{lb}}{\text{ft}^2}$$

D-load present at x feet with Concrete Cradle bedding

$$D_{0.01_CC_{28}} = 1342\,\frac{\text{lb}}{\text{ft}^2}$$

$$D_{0.01_CC_{29}} = 1388\,\frac{\text{lb}}{\text{ft}^2}$$

$$D_{0.01_CC_{30}} = 1434\,\frac{\text{lb}}{\text{ft}^2}$$

$$D_{0.01_CC_{31}} = 1480\,\frac{\text{lb}}{\text{ft}^2}$$

$$D_{0.01_CC_{32}} = 1526\,\frac{\text{lb}}{\text{ft}^2}$$

The ASTM C76 specified D0.01 crack capacity of the Gateway 60" RCP is 3000 lb/ft/ft. For the existing condition (depth at springline = 31' (731.9')), the pipe demand is 1480 lb/ft/ft. The pipe demand exceeds the pipe capacity when top of levee elevation is is 796.9' or greater (See spreadsheet in Gateway file under B4 peer review).

Summary: On 7/30/07, the Gateway P.S. and 60" RCP were visited for observation. At that time no issues regarding the 60" RCP were found. The Gateway 60" RCP is considered reliable for Existing Conditions. For a Class V RCP, the ASTM C76 D-load to produce a 0.01 crack is 3000psf. At the current grade elevation the D-load is 1480psf and will not reach capacity until an elevation of 796.9' is achieved.