Chapter A-15

ARGENTINE UNIT PUMP STATION ANALYSIS

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A-15.1 SUMMARY

The following summarizes the findings and analysis conducted by the St. Louis and Kansas City District Corps of Engineers staff, including the Argentine Unit Federal Pump Station Modification and the Argentine Unit Private Pump Station Modification sections of this chapter. Additional details are provided following the summary.

A-15.1.1 Sources of Information

Corps staff visually inspected every pump station studied, though printed information was unavailable or limited for several of the facilities. Information for the Turner, ConAgra and Bulk Mail pump stations was readily available from either the owner or the Kaw Valley Drainage District and included as-built drawings. Strong Avenue pump station information was provided by a local consultant, and included circa 1916 as-built drawings and more recent drawings showing later modifications. Information for the Argentine station was not available, and is believed to have been lost in the 1951 flood. Original as-built drawings for the Santa Fe station were also unavailable and believed to be lost in the 1951 flood, though a local consultant was able to provide recent drawings showing modifications to the facility.

A-15.1.2 Recommendations

The analysis of the three major pump stations along the line of protection resulted in the recommendations below. Required modifications to Turner, Argentine and Strong Ave stations are cost-shared as part of the proposed Federal project. Required modifications to private pump stations with no formal easement (ConAgra, Bulk Mail) are the responsibility of the owners. Required modifications to private pump stations with formal easements/crossing rights (likely Santa Fe) are also typically cost-shared as part of the Federal project.

Turner Pump Station. The mechanical and structural evaluation indicates that Turner Pump Station does not meet flotation criteria for the existing levee with water to the top of protection. The use of relief wells to draw down the hydraulic grade line (HGL) is the most effective alternative to allow the pump station to meet criteria for the existing conditions evaluation. Approximately seven relief wells will be positioned around the perimeter of the pump station and will discharge (free flowing) into a collector pipe. The collector pipe will transfer the water to the pump station. This water will be evacuated from the pump station with the interior drainage.

For any levee raise, an existing pump will need to be replaced by a higher capacity pump and the top of the discharge chamber will need to be raised to coincide with the top of the levee. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new motor actuated gate hoist, and installation of new stem guides.

The Turner Pump Station was designed and constructed as part of the initial 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 recommended.

Argentine Pump Station. An existing conditions structural evaluation was performed for this pump station, but the analysis process was hampered by the lack of information regarding the pump station and inlet RCB. Based on a variety of possible assumptions, the analysis showed that there is significant concern about the flotation stability of the pump station with water at top of levee. Insufficient information was available to perform a strength analysis, but results of the Strong Avenue Pump Station strength analysis reinforced the judgment that this pump station would not meet strength criteria for the existing conditions evaluation. The uncertainty associated with the structural details of the pump station, the results of the existing conditions flotation analysis, the presence of chemicals in the surrounding soil, and the age of the pump station resulted in the conclusion that this pump station and inlet and outlet culverts should be replaced.

The Argentine Pump Station was designed and constructed prior to the initial Corps of Engineers involvement in the flood control project. Record drawings were not available. Additional testing during the next project phase is not recommended since the pump station would be replaced.

Strong Avenue Pump Station. A strength evaluation with water to the top of existing levee was performed for the Strong Avenue Pump Station. The analysis shows that the current pump station does not meet criteria for water at the top of the existing level of protection. Detailed drawings of the substructure were discovered and did provide some confidence to the strength analysis. The inlet culvert is a 78-inch brick sewer that may have been in place since the turn of the 20th century. Again, no detailed drawings were available. No conclusions were made concerning the brick sewer. A field investigation of the brick sewer by local entities is recommended.

The outlet culvert consists of an older portion (approximately 100 feet) and a newer portion (approximately 30 feet) and a new gatewell. The newer outlet culvert and gatewell meet criteria for the existing conditions, while the older culvert does not meet strength criteria.

A steel pilaster and braced strut design is proposed for strengthening the pump station foundation walls, along with a thickened reinforced slab to address floor strength and station uplift concerns.

The most feasible alternative for strengthening of the older culvert is to line the culvert with a new pipe. Information gathered from Kaw Valley Drainage District suggests that the inlet and outlet culverts are oversized for the flows that pass through this pump station. It is believed that a reduction in the flow area of the outlet will not compromise the ability of the system to pass interior drainage. This is reinforced by the reduction in drainage area and the diversion of flows to the Ruby Street system. Exact flow information is not available (the owner of the system does not know how much water flows through the system during interior storm events).

The Strong Avenue Pump Station was designed and constructed prior to the initial Corps of Engineers involvement in the flood control project. Since record drawings were available and used extensively for the analysis, forming the basis for the recommended structural improvements, additional testing during the next project phase is not required.

Bulk Mail Pump Station. Strength analysis of the walls revealed factors of safety greater then 1.5 for water up to the nominal 500-year plus 5 ft raise alternative. The foundation wall strengths are adequate for a levee raise. Flotation analysis showed that the pump plant meets Corps guidance of a 1.1 factor of safety for water at the top of existing levee height. However, for a raise alternative to occur, relief wells will need to be installed to draw down uplift pressures. The mechanical evaluation indicates that, with the current pumps, raising the discharge several feet to accommodate a levee raise will cause only minor losses in pumping rates and efficiencies. It is recommended that, due to the age of the existing lines and local disturbances during construction, the existing discharge lines be replaced when the levee is modified. If the existing twin 12" diameter discharge lines are replaced with 14" diameter lines (the next larger standard size) pumping rates may increase slightly.

The Bulk Mail Pump Station was designed and constructed by the Corps of Engineers for the U.S. Postal Service after completion of the original Corps of Engineers flood control system. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not recommended.

ConAgra Pump Station. Strength analysis of the walls revealed factors of safety greater then 1.5 for water up to the nominal 500-year plus 5 ft raise alternative. Flotation analysis revealed the plant to meet flotation criteria up to the nominal 500-year plus 5 ft raise. The mechanical evaluation indicates that, with the current pumps, raising the discharge several feet to accommodate a levee raise will cause only minor losses in pumping rates and efficiencies. It is recommended that, due to the age of the existing line and local disturbances during construction, the existing discharge line be replaced when the levee is modified. If the existing 14" diameter discharge line is replaced with a new 16" diameter line (the next larger standard size), pumping rates may increase slightly.

The ConAgra Pump Station was designed and constructed by private interests after completion of the original Corps of Engineers flood control system. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not recommended.

Santa Fe Pump Station. Strength analysis of the foundation walls revealed factors of safety greater then 1.5 for water up to the nominal 500-year plus 5 ft raise alternative. Flotation analysis showed that the plant meets flotation criteria up to the nominal 500-year plus 5 ft raise. The mechanical evaluation indicates that, with the current pumps, raising the discharge several feet to accommodate a levee raise will have no effect on pumping rates and efficiencies since the current arrangement does not pump "up and over" the levee.

The Santa Fe Pump Station was designed and constructed prior to the original Corps of Engineers flood control system. Drawings showing 36 inch wall thicknesses, as determined by recent modifications, were available. However, drawings showing the size, spacing and orientation of rebar were never located. Based on the 36 inch wall thickness, additional non-destructive testing is not necessary. The assumption of reinforcing ratios of 0.5% (very conservative) still yielded satisfactory factors of safety.

A-15.1.3 Information Used in the Economic Model

A probability of failure curve for the Turner Pump Station was not included in the economic model. It is recognized that the potential for damages is present, but it is also

understood that the other pump stations have a greater likelihood of failure and as a result will be used for the economic model. Little is known about the Argentine Pump Station's substructure, but five assumptions were made to evaluate the range of possible conditions. The most likely assumption appears to be that the two pump pits act independently to resist uplift. For a more detailed description of assumptions, see the Argentine Unit Federal Pump Station Modification section of this chapter. Varying the hydraulic grade line by plus and minus one standard deviation (as supplied by the geotechnical members of the study team), a 100% probability of failure was calculated for water at the top of levee. Taking into consideration the pump plants performance during the 1993 flood event, a 25% probability of failure (lower percentage than was calculated) is assumed reasonable for the 1993 flood elevation (approximately nine feet below top of levee). The Argentine Pump Station probability of failure curve is used in the economic model.

Strength analysis of the Strong Avenue Pump Station foundation walls was performed using mean steel and concrete strengths of 37.6 ksi and 3750 psi, as used throughout this study. Results of the analysis revealed a 100% probability of failure with water at the top of existing levee. Taking into consideration the pump plants performance during the 1993 flood event, a 25% probability of failure (lower percentage than was calculated) is assumed reasonable for the 1993 flood elevation. The Strong Avenue Pump Station probability of failure curve is used in the economic model.

A-15.1.4 Summary Table of Pump Station Modifications

Table A-15.1 summarizes required modifications to the Argentine Unit pump stations to accommodate a levee raise up to and including the 500-year plus 5 ft level.

TABLE A-15.1
Summary of Pump Station Modifications

Name	Ownership	Required Modifications	Further Testing Recommended?
Turner	Kaw Valley Drainage District	Relief wells Larger pumps	No
Argentine	Kaw Valley Drainage District	Abandon and replace	No
Strong Ave	Wyandotte Co / Kansas City Kansas Unified Government	Major structural rehab	No
Bulk Mail	Private (US Postal Service)	Relief wells Larger discharge	No
ConAgra	Private (ConAgra)	Larger discharge	No
Santa Fe	Private (Railroad)	None	No

A-15.2 FEDERAL PUMP STATION MODIFICATIONS

The Argentine Levee Unit includes three Federal pump stations. These pump stations are referred to as the Turner, Argentine (also known as 5th Avenue, or Main Street), and Strong Avenue Pump Stations. These three pump stations were determined to have a "high" potential for impact due to the proposed levee raise. Recognizing the potential for significant construction costs associated with modification or replacement of the pump stations to meet existing and future conditions scenarios, the project team performed a feasibility level analysis of these pump stations. The recommendations presented herein are based on this additional planning effort and were used in the development of an economic benefit to cost ratio.

A-15.2.1 Background

Several study efforts have been performed to inventory and analyze the pump stations under the existing and future conditions. These study efforts include an inventory of the various pump stations and associated drainage features within the overall study area, discussion with levee district engineering and operational personnel, field investigations and evaluations, and various analyses performed by the project team

Pump Station Inventory. An inventory of all study area drainage features was completed in June, 2001. An inventory of existing designs was also performed in order to compare those designs with current operating conditions. Problem areas or "red flags" were identified for future consideration. The report includes spreadsheets of the existing pump stations with a significant amount of information related to existing pumping capacities.

In a follow-on effort, St. Louis District Corps of Engineer staff were asked to examine the pump stations along the Kansas River Units and identify the expected level of impact to each pump station that might result if the units were raised (a 3 ft raise was assumed for purposes of analysis). This led to a categorization of the stations under a "high", "medium", and "low" level of analysis and design effort.

These products were refined into a features inventory and are included in the General chapter of the feasibility engineering appendix.

Overall Approach to Pump Station Analysis. As the study progressed, the team was able to focus its efforts on those stations located within the Kansas River levee units of interest. The Argentine levee unit was first to be analyzed for a potential raise. This unit is examined under the interim feasibility reporting efforts. Analysis was aimed at understanding possible weaknesses in these stations and incorporating necessary modifications and/or replacements into overall Argentine Unit recommendations.

A risk based analysis has been completed for the Turner, Argentine and Strong Avenue stations. The reliability of these pump stations has been incorporated into the HEC-FDA model.

The "No Federal Action" (future condition) scenario is likely to be no change to the existing condition of the stations. If an existing condition reliability problem is found, it is highly unlikely that the sponsor will undertake an independent modification/replacement of any of these three Argentine stations without Federal involvement.

It is important to note that all other Missouri River and Kansas River pump stations have been reviewed using sponsor interviews (sponsors are very familiar with

most stations due to their O&M responsibilities), site observations during the study, annual inspection records, review of existing design and specification documentation (where available), and consideration of station performance during the 1993 flood. The review has shown no known structural or major pumping capacity problems at this time. Study funding was not budgeted for a detailed quantitative review of these remaining stations. However, it is likely that the engineering appendix for the final feasibility report will involve detailed risk based-analysis of the Armourdale Unit and CID-KS Unit pump stations as both of these units will be reviewed for possible raises.

A-15.2.2 Existing and Future Conditions Evaluation

The St. Louis District Corps of Engineers was engaged to perform a structural and mechanical evaluation of the Turner station. The Turner Pump Station analysis is included as Exhibit A-15.2 in the Supplemental Exhibits section of this chapter.

The four alternatives initially considered for the pump station evaluations include:

- The nominal 500-year flood plus zero feet
- The nominal 500-year flood plus three feet
- The nominal 500-year flood plus five feet
- The no raise (existing conditions scenario with flood to top of protection)

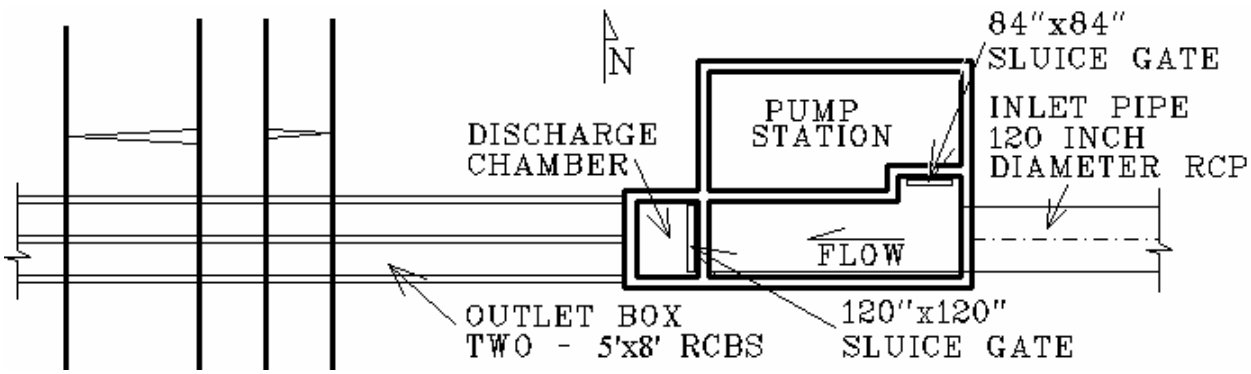
The goal of the pump station analysis was to evaluate the pump station performance for existing conditions with flood and for the future conditions with flood. The process had to be reformulated due to schedule and budget restraints. The effort was reduced to include only a detailed analysis of the Turner Pump Station. The pump station was evaluated for the existing condition (no raise) scenario and one future conditions scenario. The project team determined that the future conditions analysis should be based on the nominal 500-year flood plus three feet. The St. Louis District pump station analysis was performed using this criterion.

The project team then participated in a series of pump station review meetings that focused on the Turner Pump Station analysis results and on formulation of a defendable engineering analysis and planning evaluation approach to the two remaining pump stations (Argentine and Strong Avenue Pump Stations) within the Argentine Unit. Prior to developing an approach to the Argentine and Strong Avenue Pump Stations, the Turner Pump Station analysis was reviewed and discussed. The project team then compared the remaining two pump stations with Turner Pump Station. This comparison showed that, in general, each pump station is unique. After careful consideration of the available information for the Argentine and Strong Avenue Pump Stations, it was determined that an additional limited engineering analysis should be performed for the two remaining pump stations. An existing conditions flotation analysis was performed for the Argentine Pump Station and a strength analysis was performed on the Strong Avenue Pump Station. These analyses, coupled with the information developed for Turner Pump Station, would be used to support modification or replacement of the remaining two pump stations. The analyses of these pump stations are summarized below.

Turner Pump Station. The Turner Pump Station allows for evacuation of interior storm water by gravity during low flow events on the Kansas River and by pumps

during high flow events on the Kansas River. The pump station serves commercial and residential areas. The pump station is located at station 60+40 and was constructed in 1950. The station is situated approximately 44 feet landward of the levee toe. Exhibit A-15.1 shows the layout of the Turner Pump Station.

EXHIBIT A-15.1
Turner Pump Station Features



The Turner Pump Station consists of a reinforced concrete substructure with an un-reinforced masonry and brick veneer superstructure above grade. The substructure includes a pump sump adjacent to a discharge chamber located on the landside of the levee. The water is evacuated from the discharge chamber through two 5 feet by 8 feet reinforced concrete box (RCB) culverts. There are four pumps that discharge through steel pipes into the discharge chamber. There are two cast iron sluice gates. There is a 108-inch by 108-inch sluice gate located in the discharge chamber that is used to block gravity flow during high water levels on the Kansas River. There is an 84-inch by 84-inch sluice gate located between the pump sump and the gravity flow sewer that allows water to get to the pumps when the pump station is in operation. There is a 120-inch diameter reinforced concrete pipe (RCP) inlet culvert that delivers the interior storm water runoff to the pump station.

Mechanical Evaluation. A pump station pump capacity analysis was performed for current (existing) conditions and for future with levee raise conditions. The capacity analysis showed that one pump should be removed and replaced with a higher capacity pump. The detailed analysis is included in Exhibit A-15.2 in the Supplemental Exhibits.

Current Pumping Conditions. River levels, sump levels, and capacities were determined from the Supplement on Interior Drainage (1950), feasibility inventory, and from site visits to the pump station including discussions with Kaw Valley Drainage District personnel. There were two critical design Kansas River flood levels determined from these sources. The original design flood level of elevation 753.0 feet and the design maximum flood level of elevation 774.3 feet. System head loss curves were developed for each of the resulting static head conditions. The system head loss curves were plotted on the pump curves for each of the three size pumps. A summary of the existing pump

station capacities is shown in Table A-15.2 (the sump levels reflect interior flooding conditions).

TABLE A-15.2
Turner Pump Station Capacities for Current Conditions

	River Level El. 753.0	River Level El. 774.3
Sump El. 738.3		
Pump No. 1	14,250 gpm	8,150 gpm
Pump No. 2	25,150 gpm	17,500 gpm
Pump No. 3	12,200 gpm	12,200 gpm
Pump No. 4	12,200 gpm	12,200 gpm
Total Capacity	63,800 gpm (142.2 cfs)	50,050 gpm (111.5 cfs)

	River Level El. 753.0	River Level El. 774.3
Sump El. 743.6		
Pump No. 1	15,200 gpm	10,100 gpm
Pump No. 2	26,750 gpm	20,500 gpm
Pump No. 3	12,300 gpm	12,300 gpm
Pump No. 4	12,300 gpm	12,300 gpm
Total Capacity	66,550 gpm (148.3 cfs)	55,200 gpm (123.0 cfs)

Future Pumping Conditions. The Design River Level at the Turner Pump Station will become elevation 780.8 feet. The sump levels of elevation 738.3 feet and elevation 743.6 feet remain the same. System head loss curves were developed for each of the resulting static head conditions. The system head loss curves were plotted on the pump curves for each of the three size pumps. A summary of the existing pump station capacities is shown in Table A-15.3 (the sump levels reflect interior flooding conditions).

TABLE A-15.3
Turner Pump Station Capacities for Future Conditions

	River Level El. 780.8
Sump El. 738.3	
Pump No. 1	Off the curve
Pump No. 2	13,000 gpm
Pump No. 3	12,150 gpm
Pump No. 4	12,150 gpm
Total Capacity	37,300 gpm (83.1 cfs)

	River Level El. 780.8
Sump El. 743.6	
Pump No. 1	7,300 gpm
Pump No. 2	16,800 gpm
Pump No. 3	12,250 gpm
Pump No. 4	12,250 gpm
Total Capacity	48,600 gpm (108.3 cfs)

Capacity at Design River Condition**.** The analysis provided above indicates a decrease in pumping capacity from the existing Maximum Design River (elevation 774.3 feet) to the proposed Maximum Design River (elevation 780.8 feet) of the following:

Sump Level 738.3	Station Capacity Decrease – 12,750 gpm
Sump Level 743.6	Station Capacity Decrease – 6,600 gpm

There are not pump replacement requirements for the existing conditions scenario. It is recommended that Pump No. 1 be replaced with a new pump to mitigate the change in pump capacity due to the future conditions scenario. The new submersible pump will have a rated capacity of 12,750 gpm @ 46.0 feet.

Discharge Chamber. There are no recommended modifications to the discharge chamber or components for the existing conditions scenario. There are some modifications recommended for the future conditions scenario.

The proposed future condition levee raise will result in the top of the discharge chamber being raised. When the discharge chamber is raised, the 108" x 108" gravity drain sluice gate stem will be too short and will have to be replaced. New stem guides will be required. The electric motor actuated sluice gate hoist will need to be relocated to the top of the discharge chamber. The 84" x 84" sluice gate, located inside the pump station, will not be affected by the levee raise.

Details for raising the discharge chamber are included in Exhibit A-15.2. The existing discharge chamber top slab will need to be removed and the side walls will be extended using cast-in-place reinforced concrete. The side wall extensions will be doweled into the existing structure. A new reinforced concrete top slab will be constructed.

Structural Evaluation. The structural evaluation focused on flotation stability and strength. The analysis of the Turner Pump Station showed that the pump station did not meet flotation criteria for the existing or future conditions. Additionally, the analysis showed that all of the structural members meet strength criteria for the existing conditions scenario and only one of the structural members does not meet strength criteria for the future conditions scenario.

There is one location on the discharge chamber that was found to not meet strength criteria for the future conditions scenario. This wall will need to be strengthened for the future conditions scenario. Further discussion of this is provided in the Strength Analysis paragraph. The detailed structural analysis is included in Exhibit A-15.2 in the Supplemental Exhibits section.

Flotation Analysis. The pump station was modeled in the Corps of Engineers CASE program 3DSAD to determine pump station weights. The 3DSAD program was also used to determine soil bearing pressures at the base. Factor of safety against flotation was determined using computed weights from the 3DSAD models and uplift pressures were determined from the hydraulic grade line (HGL). A summary of results is provided in Table A-15.4.

TABLE A-15.4
Flotation and Bearing Pressure Results

Load Condition	Load Description	FS Against Flotation	Max bearing pressure (ksf)
0	Existing Station Only, NO GW raise	n/a	2.137 (net)
00	Existing Station Only – UPLIFT for Existing Condition with HGL 12.3 ft above grade	0.95	n/a
01	Existing Station Only + GW raise	n/a	2.248 (net)
A	Existing Station + GW raise + soil	n/a	2.858
B	Existing Station + GW raise 500+3 (FULL HGL)	0.90	0.486
C	Existing Station + GW raise 500+3 (HGL –7 ft)	1.08	0.312
D	Existing Station + GW raise 500+3 (HGL –8 ft)	1.11	0.616
E	Existing Station + GW raise 500+3 (HGL –9 ft)	1.15	0.613

(Notes: The FS for uplift includes 8 ft of water in pump station; the term "GW" is short for Gatewell)

The above table summarizes the existing conditions with flood and the future 500-plus 3 feet condition with flood. The analysis shows that the pump station does not meet the flotation criteria for the existing conditions. Load conditions B through E quantify how effective a relief well system would have to be to allow the pump station to meet flotation criteria for the future 3 foot raise scenario. Essentially, a relief well system would have to be constructed around the perimeter of the pump station in a manner that would lower the HGL a minimum of 8 feet to allow the pump station to meet flotation criteria. A safety factor against flotation of 1.1 or greater was considered acceptable for these load conditions.

The bearing pressure analysis shows that the pump station meets the allowable bearing pressure criteria for the existing conditions. The pump station will meet allowable bearing pressure criteria for the future conditions if relief wells are installed and are as effective as assumed. The allowable bearing pressure varies depending upon a flood or non-flood condition. The allowable bearing pressure for the nominal 500-year plus 5 feet raise is 2,000 pounds per square foot (psf) compared to an allowable bearing pressure of 2,500 psf for the nominal 500-year plus zero feet raise. The allowable bearing pressure for the existing condition with no flood is considerably higher than 2,800 psf based on past experience with deep founded structures along the Missouri and Kansas River units.

Strength Analysis. The existing conditions strength analysis shows that the pump station does not require modification. The future conditions strength

investigation found the majority of the elements of the pump station to be adequate to accommodate additional loading. Therefore, for these elements, no action is necessary to accommodate the levee raise.

An exception to this is select portions of the east wall of the discharge chamber. The suspect area is located in the negative moment region of a wall segment on the east wall of the discharge chamber at elevation 767.5 feet. This wall segment was found to have a factor of safety equal to 1.21 (no load factors or strength reduction factors were applied). If the strength factor of safety of a structural element is below 1.5, then strengthening of the element is recommended.

Outlet and Inlet Culvert Evaluation. A flotation and a strength evaluation was performed for the two 5 feet by 8 feet outlet RCB culverts for existing and future conditions. The RCB culverts met criteria for the existing conditions, but not for future conditions. This analysis showed that the RCB culverts would be overstressed if additional fill was placed over the culverts when raising the height of the existing levee. This fill would increase soil pressures on the culverts. The structural analysis of the RCB culverts is provided in the Structural Analysis – Argentine Raise chapter of this appendix.

Further analysis of the outlet RCB culverts showed that the RCBs meet strength criteria if the line of protection is raised using a floodwall where the RCB crosses under the levee. Analysis of this type of measure (referred to as "deloading") is included in the Structural Analysis – Argentine Raise chapter of this appendix.

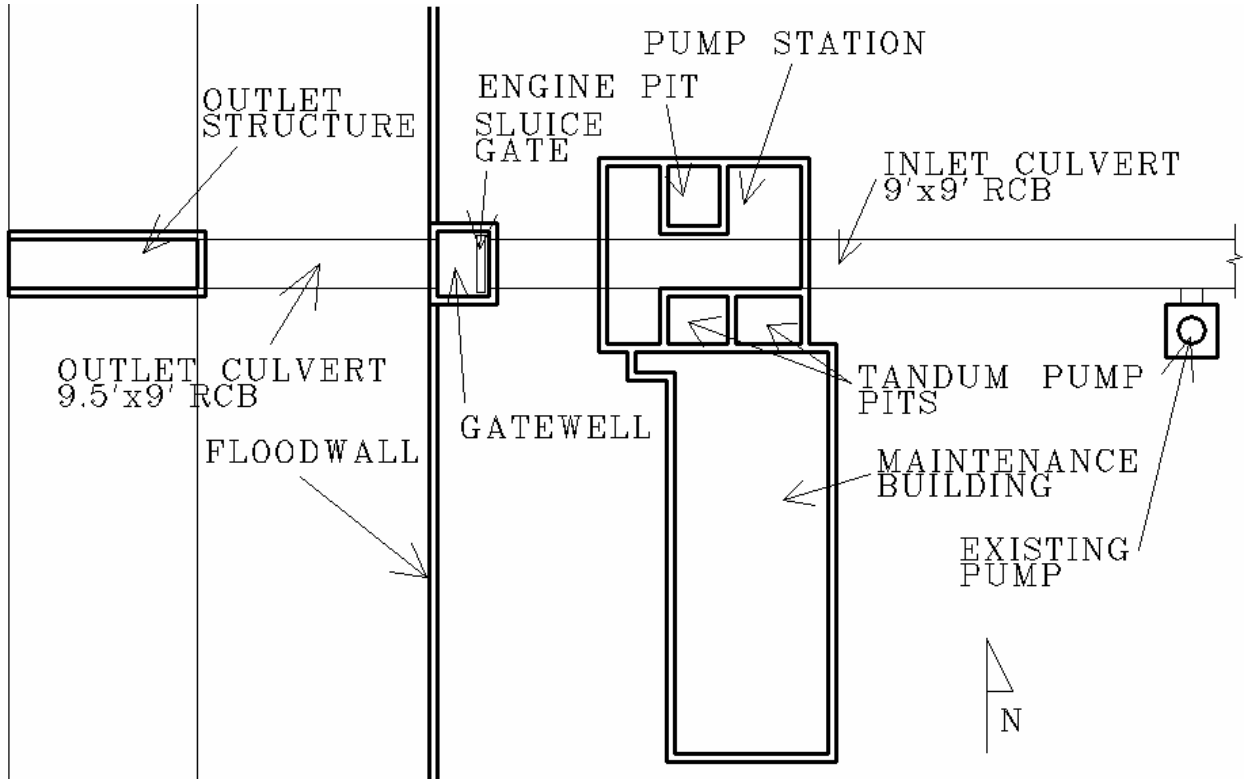
The inlet culvert is a large RCP. This pipe was constructed when the pump station was constructed. Based on field inspection of this pipe, the project team believes this pipe is adequate to handle the change in ground water pressures that would result from the proposed levee raise. This pipe should be evaluated during the next phase of the project.

Argentine Pump Station. The Argentine Pump Station serves a similar purpose as the Turner Pump Station. The Argentine Pump Station is located at station 253+14 near the Santa Fe Railroad property and is located 18.5 feet landward of a floodwall system. This pump station was constructed prior to the 1920s. The exact date of construction is unknown. Exhibit A-15.3 shows the various features associated with the Argentine Pump Station.

Generally, there is very little detailed information available concerning the substructure of the pump station. The substructure appears to be constructed from a combination of a reinforced concrete frame with masonry infill. The superstructure is an un-reinforced masonry structure with brick veneer. The substructure includes two pump pits and an engine pit. Water enters the pump station through a 9 feet by 9 feet RCB. Water is evacuated by gravity flow through a gatewell into a 9.5 feet by 9 feet RCB. During high Kansas River flows, a sluice gate is closed in the gatewell and water is backed up into the pump pits. Water is then conveyed through pump discharge pipes into the gatewell.

A field investigation of the outlet and inlet culvert was performed by the project team. During this investigation, the project team members discovered that there are petroleum products in the soil around the box and pump station. Research into this site revealed that this contamination is gasoline, diesel, and benzene and is being monitored.

EXHIBIT A-15.3
Argentine Pump Station Features



Structural Evaluation. There are some drawings available for the Argentine Pump Station that show general structural layout, but do not show any detail. The lack of detail prevented direct comparison with the Turner Pump Station evaluation. The Turner Pump Station evaluation showed that flotation was a critical issue which drove modification requirements for the pump station. The strength evaluation resulted in minor modifications to the Turner Pump Station.

The project team concluded that a sensitivity analysis of flotation could be performed for the Argentine Pump Station and that these results would provide a basis for determining if the Argentine Pump Station would require modification for the existing and future conditions.

Additionally, the project team concluded that inferences concerning the strength performance of this pump station could be made based on the strength analysis results for the Strong Avenue Pump Station since both of these pump stations were built in the same time period. The analysis is included as Exhibit A-15.4. The following assumptions were made concerning the pump station. The number of assumptions made is reflective of the lack of information available concerning the pump station. The Argentine Pump Station was evaluated against each assumption for the existing condition with water to top of levee.

- Assumption 1: The substructure was assumed to consist of concrete walls along the footprint of the station. The walls were assumed to support the superstructure and continue below grade to a depth equal to the location of the box culvert sewer. The substructure was assumed to have a continuous base slab over the full footprint of the station, on which uplift pressures could act. The Operation and Maintenance Manual (O&MM) drawings indicate that there could be soil located within the substructure above the box culvert sewer and around the original manhole, and sluice gate chase. For Assumption 1, this soil was assumed to be in place and contained within the assumed substructure walls. It was also assumed that the soil was saturated, as this part of the substructure was not watertight. Two analyses were performed with one assuming 8 ft of water in the box culvert and a second assuming zero feet of water in the box culvert.
- Assumption 2: Assumption 2 is the same as Assumption 1, except the soil within the substructure is not included.
- Assumption 3: The O&MM drawings elude to a condition in which the superstructure of the station is supported on a subgrade system of beams and "columns", with some type of masonry fill between the columns (at the perimeter of the station footprint). Most of the masonry was likely removed or disturbed during modifications of the plant, and are assumed not to be intact. The O&MM drawings identify the sewer, pump pits, engine pits with straight lines, and therefore these features are assumed to be concrete, as opposed to masonry. In contrast to Assumption 1, Assumption 3 does not assume a full base slab over the footprint of the station, but only at locations of the sewer and the pump and engine pits. The perimeter substructure walls of masonry are neglected from the uplift calculation. The weight of the superstructure is included based upon a simple percentage of the overall footprint area to the area assumed for uplift. The soil above the sewer is assumed to be in place, as in Assumption 1. Two analyses were performed with one assuming 8 ft of water in the box culvert and a second assuming zero feet of water in the box culvert.
- Assumption 4: Assumption 4 is the same as Assumption 3, except the soil within the substructure is not included.
- Assumption 5: The tandem pump pits and the engine pit are assumed to act independently to resist the uplift applied to their respective base areas. The pits are assumed to go down to same level as the box sewer.

Based upon the lack of information on the existing station, and the age and condition of the station, it is difficult to determine the actual factor of safety against flotation for the Argentine Pump Station. The flotation analysis of each assumption results in a large safety factor spread. An acceptable safety factor against flotation is 1.1 or greater for the analyzed assumptions. The results of the flotation analysis are listed in Table A-15.5.

A risk and uncertainty (R&U) analysis of flotation stability for the Argentine Pump Station was performed. The R&U analysis was performed on the five assumptions

developed to evaluate the pump station. Uncertainty was measured by applying a mean and standard deviation to H_o (distance of hydraulic grade line above grade). The mean and standard deviation of H_o were estimated using a geotechnical Taylor Series Method (TSM) analysis. The flotation stability analysis was set up as a capacity versus demand equation and evaluated using a separate structural TSM analysis. The mean and standard deviation for H_o were used in this TSM analysis. Results are summarized below.

TABLE A-15.5
Argentine Flotation Analysis Results

Loading Condition	Factor of Safety Against (Flotation)	Probability of Failure (Flotation)
Assumption 1 A	1.22	0 %
Assumption 1 B	1.13	0 %
Assumption 2 A	0.83	100 %
Assumption 2 B	0.75	100 %
Assumption 3 A	0.87	100 %
Assumption 3 B	0.77	100 %
Assumption 4 A	0.68	100 %
Assumption 4 B	0.58	100 %
Assumption 5 (pump pits)	0.67	100 %
Assumption 5 (old engine pit)	0.74	100 %

The Argentine Pump Station meets criteria for Assumptions 1A and 1B, but exhibits significant reliability concerns for all other assumptions. For a more detailed description of the risk and uncertainty methodology developed and employed in this study, see the Structural Analysis chapters of this appendix.

Outlet and Inlet Culvert Evaluation. A flotation and strength evaluation of the outlet RCB culvert and gatewell was performed. The RCB meets flotation and strength criteria as long as additional fill is not placed over the RCB to raise the line of protection. At this location, the line of protection consists of a floodwall; thus, additional fill will not be used to raise the levee.

The gatewell, which is located in line with the floodwall, does meet flotation criteria. However, the floodwall adjacent to the gatewell does not meet bearing pressure criteria. Additionally, calculations show that a base extension allows the floodwall to meet bearing pressure criteria.

The inlet RCB is a 9 foot by 9 foot culvert. There are no detailed drawings for this RCB, thus a strength analysis was not performed. The connection with the new pump station inlet pipe will be made at a distance back from the levee where the ground water pressure change due to the proposed raise is minimal.

Mechanical Evaluation. A detailed mechanical evaluation of the Argentine Pump Station pump capacity was not performed. The analysis of Turner Pump Station assumed the existing pump capacity is adequate and used this as a benchmark against the

future conditions with 3 foot raise scenario. This resulted in a recommendation for replacement of an existing pump with a new pump. Using this same logic, it is recommended that one existing pump be replaced with a new pump if the levee is raised.

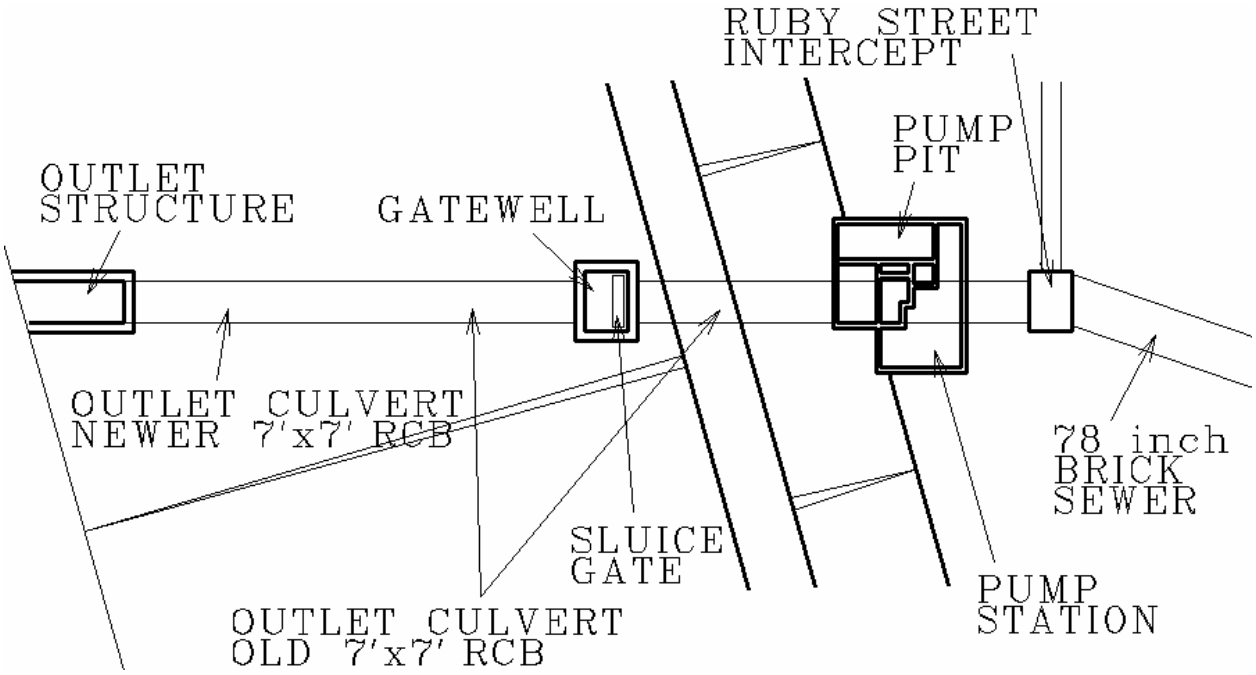
Strong Avenue Pump Station. The Strong Avenue Pump Station is located at station 273+14 and is located 32 feet from the landside levee toe. The Strong Avenue Pump Station was constructed prior to 1916. The exact date of construction is unknown. The role of this pump station has evolved over time. Initially the pump station removed storm and sewer water from a 607 acre residential and commercial area. During the 1960s, the Ruby Street Intercept was constructed. This intercept redirects low flows to a wastewater treatment plant. A significant amount of flow that originally went to the Strong Avenue Pump Station has been diverted to the Ruby Street system. At some point, the stormwater contribution area was reduced to 175 acres, further reducing the pump station demand. Kaw Valley Drainage District indicated that there are no interior flooding issues resulting from interior drainage issues at this pump station.

The pump capacities of the pumps in the Strong Avenue Pump Station are roughly 8,000 gpm each. Both may be operated concurrently. Pumps were sized based on physical dimensions of the wet well, not per detailed inflow analysis. It appears that the pumps are more than adequate for exiting and future conditions, based on the reduction in contributing drainage area and anecdotal evidence.

Detailed drawings of the substructure were located; however, visual inspection of the substructure interior indicates that the structure has been modified several times. There is evidence that masonry/brick walls have been covered by concrete and the finish on the concrete surfaces indicates different types of forms (plywood sheets and small boards).

Exhibit A-15.5 shows the various features of the Strong Avenue Pump Station. The detailed drawings show the structure to be a reinforced concrete box. The superstructure is a pre-engineered metal building. Water is delivered to the pump station from a 78-inch brick sewer. As combined sewer and stormwater exceed the capacity of the intercept, water passes through the pump station into a 7-foot by 7-foot outlet reinforced concrete box (RCB) culvert. The outlet RCB culvert includes 60 feet of old RCB that connects the pump station with the gatewell. From the gatewell to the river, there is an additional 40 feet of old RCB culvert which transitions to 30 feet of new RCB. During high Kansas River stages, the sluice gate in the gatewell is closed. Water is then pumped from the pump station into the gatewell which acts as a discharge chamber.

EXHIBIT A-15.5
Strong Avenue Pump Station Features



Structural Evaluation. Reviews of the pump station detailed drawings indicate that the substructure is comprised of reinforced concrete walls which vary from 10 to 24 inches in thickness. Using judgment gained by analysis of the Turner and Argentine Pump Stations, the project team concluded that this structure will not satisfy either flotation or strength criteria under existing conditions with flood. The structural evaluation focused primarily on the strength of the pump station walls to confirm this conclusion.

A strength analysis was performed on pump pit walls. The aspect ratio of these walls is such that the earth and water pressures will be carried laterally to the side walls. Review of the detailed drawings showed that these walls are reinforced with #4 reinforcing bars at 6 inches horizontally. This analysis showed that the walls do not meet the existing conditions load case. The analysis is included as Exhibit A-15.6 in the Supplemental Exhibits section.

Outlet Culvert Evaluation. Detailed drawings of the 7-foot by 7-foot outlet RCB and gatewell were located. Using this information, these systems were analyzed for existing conditions with flood. The analysis showed that the older portions of the outlet RCB do not meet strength criteria.

A flotation and strength analysis of the gatewell showed that the gatewell meets criteria for the existing conditions with flood. No modification of this structure is required for this condition. The gatewell will need to be raised if the levee is raised. The gatewell will be raised by removing the existing sluice gate hoist, stem and guides. The

top slab will then be demolished. The walls will be extended using cast-in-place concrete. A new top slab will be constructed. A new sluice gate stem, motor actuated gate hoist and stem guides will be installed as well.

Inlet Culvert Evaluation. The pump station receives inflow from a 78-inch brick sewer. It appears that, under low flow conditions, the 78-inch sewer flows are directed into the Ruby Street system. Under certain interior storm events, overflow is directed into the Strong Avenue Pump Station. There is no information regarding the age of the 78-inch brick sewer, construction documents, or inspection documents. There was some verbal communication that a portion was inspected in the past and it was believed that the sewer is in good condition. A flotation or strength analysis was not performed on the 78-inch brick sewer. Inspection of the sewer by local entities to determine the condition of the sewer is recommended.

Pump Station Plan Evaluation Results. The limited mechanical and structural evaluation of Turner, Argentine and Strong Avenue pump stations showed that if the existing structures were not modified, these pump stations would not meet flotation criteria for existing conditions with flood. Additionally, this analysis indicates that the Argentine and Strong Avenue Pump Station do not meet strength criteria for the existing conditions. All of the pump stations will require a certain degree of modification to meet any future condition with flood scenario. The results of the evaluations of the pump stations are provided in Tables A-15.6 and A-15.7. The weakest link or critical element is identified.

TABLE A-15.6
Existing Conditions Structural Evaluation Results

Pump Station	Flotation				Strength				Critical Element that is Driving Modification/Replacement Scenario
	Pump Station	Gatewell	Inlet RCB/Pipe	Outlet RCB	Pump Station	Gatewe ll	Inlet RCB/Pipe	Outlet RCB	
Turner (Sta. 60+40)	**No Good**	**NA**	**Not Checked**	**OK**	**OK**	**OK**	**Not Checked**	**OK**	Flotation of Pump Station
Argentine (Sta. 253+14)	**No Good**	**No Good**	**Not Checked**	**OK**	**Not Checked**	**OK**	**Not Checked**	**OK**	Flotation of Pump Station (also strength of pump station and interior inlet culvert is suspect, but detailed information is lacking)
Strong Avenue (Sta. 273+14)	**No Good**	**OK**	**Not Checked**	**OK** (30' New RCB) **No Good** (100' Older RCB)	**No Good**	**OK**	**Not Checked**	**OK** (30' New RCB) **No Good** (100' Older RCB)	Pump Station and Outlet RCB Strength (also, flotation of pump station is suspect)

TABLE A-15.7
Existing and Future Conditions Mechanical Evaluation Results

Pump Station	Pump Capacity Analysis		Sluice Gate	
	Existing	Future	Existing	Future
Turner (Sta. 60+40)	OK	NG	OK	Lengthen Stem
Argentine (Sta. 253+14)	Not Checked	Not Checked	OK	Lengthen Stem
Strong Avenue (Sta. 273+14)	OK	OK	OK	Lengthen Stem

A-15.2.3 Alternatives Evaluation

Several alternatives were developed to solve the flotation stability issues. The list of alternatives to solve flotation stability issues for the pump stations include the following (this list is not specific to any one pump station):

- placement of relief wells around the perimeter of a pump station
- use of piles to anchor a pump station to the soil
- extending the base of the pump stations
- adding weight inside or on the outside of the pump station substructures by placing concrete in these areas
- abandonment of the pump stations

Alternatives for increasing the strength of the various components (pump station members, inlet and outlet pipes, and gatewells) included (this list is not specific to any one pump station):

- locally strengthening pump station members as necessary
- forming additional reinforced concrete walls on the inside of the pump station
- lining (with a new pipe) the reinforced concrete boxes
- use of fiber reinforced polymer (FRP) to reinforce box culverts
- removal of concrete and installation of dowels and new concrete to reinforce box culvert bottom slabs
- abandonment of the inlet and outlet pipes

Turner Pump Station. The mechanical and structural evaluation indicates that Turner Pump Station does not meet flotation criteria for the existing conditions with flood. The remaining structural and mechanical features meet criteria for the existing conditions evaluation. The use of relief wells to draw down the hydraulic grade line (HGL) is the most effective alternative to allow the pump station to meet criteria for the

existing conditions evaluation. Approximately 7 relief wells will be required to reduce the effective HGL by at least 8 feet. The wells will be positioned around the perimeter of the pump station and will discharge (free flowing) into a collector pipe. The collector pipe will transfer the water to the pump station. This water will be evacuated from the pump station with the interior drainage. Preliminary design calculations for the relief well system are included as Exhibit A-15.7 in the Supplemental Exhibits section.

The results for the future conditions indicate that an existing pump will need to be replaced by a higher capacity (12,750 gpm) pump and that the top of the discharge chamber will need to be raised to coincide with the top of the levee. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new motor actuated gate hoist, and installation of new stem guides.

Argentine Pump Station. An existing conditions structural evaluation was performed for this pump station. This analysis showed that there is significant concern about the flotation stability of the pump station and floodwall.

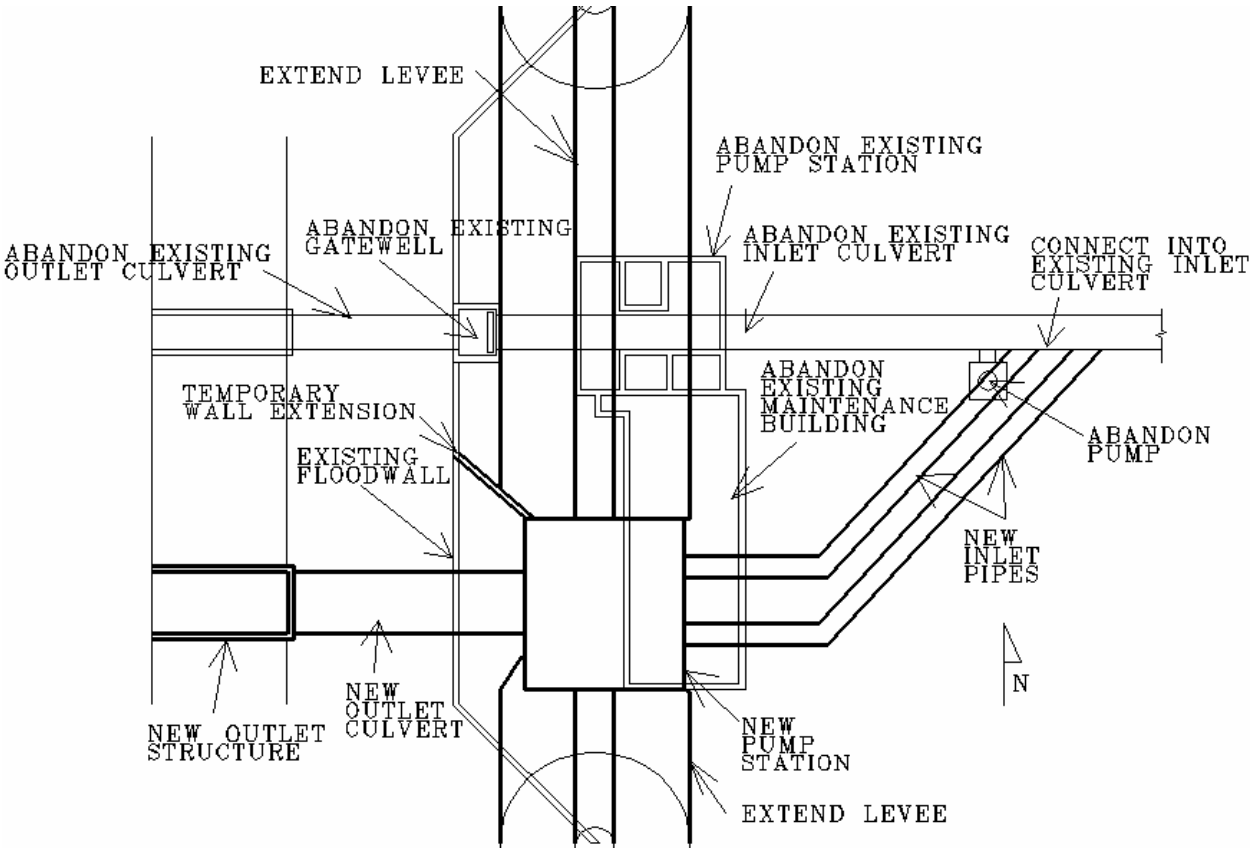
The analysis process was frustrated by the lack of information regarding the pump station and inlet RCB. Review of the drawings indicated the substructure was originally a combination of reinforced concrete and masonry. Several non-documented modifications to the pump station have occurred over the last 90 years. Furthermore, the results of the Strong Avenue Pump Station strength analysis reinforced the judgment that this pump station would not meet strength criteria for the existing conditions evaluation.

During a site investigation, team members discovered that petroleum products were in the surrounding soil. Further investigation into this issue showed that the soil surrounding the pump station is contaminated by gasoline, diesel, and benzene. These types of chemicals are known to degrade relief well screens and result in the break down of the relief well process.

The project team struggled with identifying a viable alternative for allowing the pump station to meet existing conditions criteria. Relief wells were considered, but the reliability of the relief wells over time and the ramifications of pumping ground water from a contaminated site were considered too risky and costly.

The uncertainty associated with the structural details of the pump station, the results of the existing conditions flotation analysis, the presence of chemicals in the surrounding soil, and the age of the pump station resulted in the conclusion that this pump station should be replaced. Replacement of the pump station would allow for construction of a reliable pump station, and inlet and outlet culverts. Contaminated soils would need to be excavated, but these could be removed to a disposal area that was designated to receive these soils. This solution did not require pumping of contaminated groundwater. A schematic plan of the recommended alternative is shown in Exhibit A-15.8.

EXHIBIT A-15.8
Argentine Pump Station Replacement Schematic



A construction sequence was developed for construction of the new plant. The sequence is as follows:

- demolition of existing maintenance building
- build new inlet pipe (use deep trench braced excavation)
- construct new pump station (possibly incorporate shoring into the pump station)
- extend existing levee to pump station and create positive tie-in
- construct temporary floodwall from pump station to existing floodwall
- excavate and construct outlet pipe from pump station to river (use cofferdam as necessary to prevent the Kansas River from flooding construction site)
- tie-in new inlet pipe to existing inlet culvert on landside
- abandon existing pump station (the pump station may be abandoned by removing the top 3 feet and backfilling over the area with soil)

- abandon existing inlet, and outlet culverts, as well as the gatewell (the existing inlet pipe, gatewell, and outlet pipes will need to be removed or filled with grout - may also require filling of voids with grout along the outside of the structures)
- extend remaining portion of levee to pump station and create positive tie-in

Strong Avenue Pump Station. An existing conditions strength evaluation was performed for the Strong Avenue Pump Station. This analysis shows that the pump station does not meet criteria for the existing conditions. Site investigations of this pump station showed that repairs and modifications to the pump station may have occurred since 1916 and these modifications are not documented. Detailed drawings of the substructure were discovered and did provide some confidence to the strength analysis.

The inlet culvert is a 78-inch brick sewer that may have been in place since the turn of the 20th century. Again, no detailed drawings were available. No conclusions were made concerning the brick sewer. A field investigation of the brick sewer by local entities is recommended. Brick sewers often perform well over time. Providing a strength liner is a standard method for rehabilitating brick sewers.

The outlet culvert consists of an older portion (approximately 100 feet) and a newer portion (approximately 30 feet) and a new gatewell. The newer outlet culvert and gatewell meet criteria for the existing conditions. The older culvert does not meet strength criteria.

The most feasible alternative for strengthening of the older culvert is to line the culvert with a new pipe. Information gathered from Kaw Valley Drainage District suggests that the inlet and outlet culverts are oversized for the flows that pass through this pump station. It is believed that a reduction in the flow area of the outlet will not compromise the ability of the system to pass interior drainage. This is reinforced by the reduction in drainage area and the diversion of flows to the Ruby Street system. Exact flow information is not available (the owner of the system does not know how much water flows through the system during interior storm events).

The most feasible method for strengthening the pump station is to use a brace and strut system to strengthen the walls and thicken the floor slab. The brace and struts will be structural steel. The floor will be thickened with concrete. This will address strength, as well as flotation stability of the pump station.

No Action Scenario. Several issues were discussed, including no physical modification of the pump stations for the existing and/or future conditions scenarios. The premise of this discussion was to determine if flood fighting a flotation or strength failure of a pump station or component (inlet or outlet box) was reasonable. The project team had developed considerable experience with flood fighting during the Missouri River floods of 1993 and 1995. Based on this experience, it was determined that a flood fighting scenario at a pump station could lead to catastrophic failure of the levee system. For this reason, a “no action” scenario is not recommended.

The scenario for flood fighting a failure at a pump plant was developed as follows. If a flood event occurred, the pump stations would remain active until flotation or strength failure occurred. A flood fight would ensue that would allow the pump station to fill with water to prevent sub-grade soils from being piped into the pump

station and then expelled into the river. This movement of soils through the pump stations could result in catastrophic failure of the levee unit. The project team reasoned that relief wells could be placed around the perimeter of the pump station to control the ground water pressures and to prevent the hydraulic grade line from extending above grade near the pump station vicinity. Pumps would be used to remove water from the relief wells. This would control the water levels around the pump station, and in the pump station, thus preventing interior flooding.

The arguments against this approach were as follows. Allowing for a flotation or strength failure of a pump station adjacent to a levee allows for a high degree of uncertainty and a high potential for failure. Prediction of this outcome is difficult. Qualitatively, this type of failure would be rated as a "high" potential for leading to catastrophic failure. The use of portable pumps to remove water directly from the relief wells adjacent to the pump station is not desirable since the portable pumps may not be available or reliable. The need to monitor this type of situation 24 hours a day results in tremendous impact to the sponsor and to the Kansas City District's personnel and financial resources.

During the 1993 flood, a pump station located along the North Kansas City Levee uplifted and separated from an inlet pipe. By coincidence, two Kansas City District employees were inspecting other issues associated with this levee near the time that this failure occurred. The failure was investigated and discovered quickly, allowing for an immediate flood fight response. The pump station was shut down and water was pumped from existing relief wells located at the landward toe of the levee. This resulted in the loss of riverside riprap. This failure and subsequent flood fighting scenario are not desirable since there is a greater potential for levee failure. The likelihood that a failure of this nature is detected quickly is not great.

A-15.2.4 Recommended Alternatives Summary

Turner Pump Station. The basis for modification to the pump station for existing conditions is as follows:

- A flotation analysis showed that this pump station does not meet existing conditions criteria
- Conclusion is to add seven relief wells, and collector pipes around the perimeter of the pump station
- A strength analysis indicates that a portion of the discharge chamber wall does not meet current criteria
- Conclusion is to strengthen this portion of the discharge chamber by thickening the walls

The basis for modification to this pump station for future conditions is as follows:

- If the pump station does not meet existing conditions flotation criteria, the pump station will not meet future conditions criteria
- Conclusion is to:
    - add seven relief wells, and collector pipes around the perimeter of the pump station

- strengthen a portion of the discharge chamber wall by thickening the wall
- raise the discharge chamber to the proposed levee raise height
- replace the existing sluice gate guide stem, motor actuated gate hoist, and replace the stem guides
- replace existing Pump No. 1 with a new submersible pump

Argentine Pump Station. The basis for modification to the pump station for the existing and future conditions is as follows:

- A flotation analysis showed that this pump station does not meet existing conditions criteria
- The age of the pump station (constructed prior to the 1920s) and non-documented modifications to the pump station result in uncertainty concerning the reliability of the pump station strength
- A flotation analysis showed the gatewell does not meet existing conditions criteria
- The presence of gasoline, diesel and benzene in the soil surrounding the pump station reduces the reliability of relief wells and will result in the pumping of contaminated groundwater
- Conclusion is to construct a new pump station

Strong Avenue Pump Station. The basis for modification to the pump station for existing conditions is as follows:

- A strength analysis showed that this pump station does not meet existing conditions criteria
- The age of the pump station (constructed prior to 1916) and non-documented modifications to the pump station result in uncertainty concerning the reliability of the pump station strength
- Using judgment, the pump station does not meet flotation criteria for the existing conditions
- A strength analysis of the older outlet culvert shows that this culvert does not meet existing conditions criteria
- Conclusion is to:
  - install (on the interior of the pump station) a brace and strut system to strengthen the walls and thicken the floor slab
  - line the interior of the existing older outlet culvert with a new pipe

The basis for modification to this pump station for future conditions is as follows:

- If the pump station and outlet culvert do not meet existing conditions strength criteria, they will not meet future conditions criteria
- Conclusion is to:
  - install (on the interior of the pump station) a brace and strut system to strengthen the walls and thicken the floor slab
  - line the interior of the existing older outlet culvert with a new pipe

- raise the gatewell to the proposed levee raise height
- replace the existing sluice gate guide stem, motor actuated gate hoist, and replace the stem guides

A-15.3 PRIVATE PUMP STATION MODIFICATIONS

The following documents the structural and mechanical/hydraulic investigations of the private pump stations that impact the Argentine levee unit.

A-15.3.1 Bulk Mail Pump Station

The Bulk Mail pump station was constructed in 1972 and is owned by the Post Office. It is located at levee station 131+36. The system provides for gravity drainage through the levee to be utilized under low river stages. Under river stages sufficiently high to impede gravity drainage, flow is pumped up and over the levee and discharged into a gatewell.

Corps of Engineers civil and structural engineers (Jansen & Davis, respectively) met with Paul Wolf, the owner's representative, on 13 Apr 2005 to view/inspect the pump station. Mr. Wolf provided mechanical and structural drawings. Pump curves were not available.

The pump station is reinforced concrete (RC) construction. A 48" diameter reinforced concrete pipe (RCP) with gatewell passes through the levee, allowing gravity drainage under conditions of locally heavy rainfall with low river stages. Under river stages sufficiently high to impede gravity drainage, two identical pumps pressurize twin 12" diameter welded steel discharge line running up and over the levee and discharging into the top of the gatewell. Each pump is coupled to a 60hp motor, though specific manufacturer, model, impeller, and flow rates are unknown. Based solely on comparison to the ConAgra system and expected velocities and head losses in the discharge lines, total flow rates are expected to be 5,000-6,000 gpm per pump, for a total flow rate of 10,000-12,000 gpm. The pump station drawings show the gravity drainage pipe and gatewell as new features; i.e. privately-constructed for purposes of evacuating local runoff on the owner's property.

Structural Evaluation. Drawings were located for the Bulk Mail Pump Station detailing wall thicknesses and reinforcing steel sizes. Steel and concrete strengths were not given, so they were estimated to be 60 ksi and 4 ksi respectively (typical strengths for that time period). Strength analysis of the walls revealed factors of safety greater then 1.5 for water up to the nominal 500-year plus 5 ft raise alternative. The foundation wall strengths are adequate for a levee raise. Flotation analysis showed the pump plant meets Corps guidance of a 1.1 factor of safety for water at the top of existing levee height. For a raise alternative to occur, however, relief wells will need to be installed to draw down uplift pressures. Refer to Exhibit A-15.9 in the Supplemental Exhibits section for calculations and information relating to structural analysis.

Mechanical Evaluation. The mechanical evaluation for the Bulk Mail station was limited to general comparisons to the ConAgra station, since only limited mechanical information was available. The comparison is valid since the construction, capacity, dimensions, and operation of the two stations are similar. Based upon these similarities, pumping rates are expected to decrease slightly, along with slightly increased head, for a levee raise with the same twin 12" diameter pipes. As with the ConAgra station, since the

entire length of discharge piping will be removed and replaced to accommodate the levee raise, the owner may consider upgrading the new discharge line to the next larger standard size, 14" diameter. Similar calculations for twin 14" diameter line would likely show a slight increase in overall pumping capacity, as compared to the existing condition. In other words, the smaller friction losses in the larger diameter pipe would be expected to more than offset the increased elevation head. Since excavation costs would be nearly identical for 12" or 14" lines, and material costs only slightly higher for the larger pipe, the incremental cost for the larger pipe is expected to be minimal.

A-15.3.2 ConAgra Pump Station

The ConAgra pump station (formerly Swift Eckridge) was constructed in 1979 and is owned by ConAgra. It is located at levee station 145+00. The system provides for gravity drainage through the levee to be utilized under low river stages. Under river stages sufficiently high to impede gravity drainage, flow is pumped up and over the levee and discharged into a gatewell.

Corps of Engineers civil & structural engineers (Jansen & Davis, respectively) met with Bill Kennedy, the owner's representative, on 13 Apr 2005 to view/inspect the pump station. Mr. Kennedy provided mechanical drawings and pump curves. Complete plans, including structural drawings, were later provided by the Kaw Valley Drainage District (KVDD). Pertinent mechanical drawings and information are included in the Supplemental Exhibits section as Exhibit A-15.10.

The pump station is reinforced concrete (RC) construction. A 36" diameter reinforced concrete pipe (RCP) with gatewell passes through the levee, allowing gravity drainage under conditions of locally heavy rainfall with low river stages. Under river stages sufficiently high to impede gravity drainage, two identical pumps pressurize a single 14" diameter ductile iron (DI) discharge line running up and over the levee and discharging into the top of the gatewell. Each pump is reportedly a KMD model 12CKMD, 40 horsepower (hp) with 15" diameter impeller rated for 4,000 gallons per minute (gpm) at "32' HD", presumably 32' total dynamic head (TDH). The pump station construction drawings show the gravity drainage pipe and gatewell as existing features, however it is unclear whether the pipe and gatewell were privately-constructed (though prior to the pump station) or part of an earlier levee modification by the KVDD.

Structural Evaluation. Drawings were located for the ConAgra Pump Station detailing wall thicknesses and reinforcing steel sizes. Steel and concrete strengths were not given, so they were estimated to be 60ksi and 4ksi respectively (typical strengths of that time period). Strength analysis of the walls revealed factors of safety greater then 1.5 for water up to the nominal 500-year plus 5 ft raise alternative. Flotation analysis revealed the plant to meet flotation criteria up to the nominal 500-year plus 5 ft raise. Refer to Exhibit A-15.11 in the Supplemental Exhibits section for additional structural analysis details.

Mechanical Evaluation. The mechanical evaluation first considered the existing condition, with the two pumps operating in parallel and discharge line lengths and elevations scaled from construction drawings. The elevation head (EH) was scaled at approximately 20', and length of discharge line at 144 lineal feet (lf). Friction head (Hf)

was then calculated for the system using the Hazen & Williams (H/W) formula[1] for a variety of flows to define the system curve, assuming a H/W friction factor of 100. Velocity head and local losses (also known as minor losses) were assumed to be insignificant, and were therefore neglected. This system curve was then plotted against the pump curve to identify the operating point based on information scaled from drawings and for comparison to the stated operating point. Since the two pumps are designed for parallel operation, double the stated flow per pump was assumed for reference (i.e. 8,000 gpm @ 32' TDH). The stated operating point (8,000 gpm @ 32' TDH) compared favorably with the calculated operating point (8,000 gpm @ 33' TDH). Refer to the pump curve provided by the owner, with the operating point plotted (Exhibit A-15.12 in the Supplemental Exhibits section).

The mechanical evaluation then considered the future condition (also shown in Exhibit A-15.12 as system curve plotted on pump curve) with the same two pumps operating in parallel, but with discharge line lengths and elevations increased to accommodate a levee raise. The new EH was estimated at 26', and length of discharge line at 160 lf. Hf was again calculated for the system using the Hazen & Williams (H/W) formula for a variety of flows to define the system curve, assuming a H/W friction factor of 120[2]. Velocity head and local losses (also known as minor losses) were neglected. This system curve was then plotted against the pump curve to identify the operating point based on information scaled from drawings and for comparison to the stated operating point. Although the new calculated operating point (7,900 gpm @ 34' TDH) is slightly lower flow/higher head than the previous stated operating point, pump efficiencies are nearly identical and the loss of capacity is very small. Since the entire length of discharge piping will be removed and replaced to accommodate the levee raise, the owner may consider upgrading the new discharge line to the next larger standard size, 16" diameter. Similar calculations for a 16" diameter line actually show a slight increase in overall pumping capacity, as compared to the existing condition. In other words, the smaller friction losses in the larger diameter pipe more than offset the increased elevation head. Since excavation costs would be nearly identical for 14" or 16" line, and material costs are only slightly higher for the larger pipe, the incremental cost for the larger pipe is expected to be minimal. See Exhibit A-15.13 in the Supplemental Exhibits for typical "up and over" details.

A-15.3.3 Santa Fe Pump Station

The Santa Fe pump station was constructed in 1924 and is owned by the Burlington Northern & Santa Fe Railway Company (BNSF). It is located at levee station 258+36. The system was originally designed to use a reinforced concrete box (RCB) gravity drainage system under periods of locally heavy rainfall with low river stages. Under river stages sufficiently high to impede gravity drainage, flow would be pumped

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[1] Although the Darcy-Weisbach equation is commonly accepted as a more accurate method, Hazen & Williams is sufficient for less precise estimates when considering relatively clear water at about 50 degrees Farenheit.

[2] A friction factor of 100 is commonly applied to DI pipe older than 10-20 years or of unknown coating & condition. Newer pipe with modern coatings commonly achieve friction factors of up to 150, deteriorating to 120 over 15-20 years of service.

into a chamber inside of the pump station and flow through the gravity line against the head of the river.

Corps of Engineers civil & structural engineers (Jansen, Davis, and Wright, respectively) met with Ernie Quinto, KVDD, Paul Schakel, the owner's representative, and Arthur Hutt, the owner's engineer, on 15 Apr 2005 to view/inspect the pump station. Structural drawings were unavailable; however Mr Hutt later provided mechanical drawings showing slab and wall thickness and other pertinent details gained from previous work on the pump station. Pump curves were not requested.

Per owner and engineer, the design was modified in 1994 and is presently used to pump industrial wastewaters containing oil and grease to the BNSF pre-treatment facilities prior to discharge to the Kansas City sanitary sewer. Normally, all wastewater flows are pumped to the pre-treatment holding facilities. The railroad has the ability to pump into the RCB or divert flow directly by opening the river sluice gate under flood conditions. These options are reportedly used very infrequently (i.e. only under extreme local flooding conditions).

Structural Evaluation. Drawings were located for the Santa Fe Pump Station detailing approximate wall thicknesses. Steel reinforcing sizes were not called out. A 3% steel-to-concrete reinforcing ratio was assumed (typical for reinforced concrete design). Steel and concrete strengths were not given, so they were estimated to be 30 ksi and 3 ksi respectively (typical strengths for that time period). Strength analysis of the foundation walls revealed factors of safety greater then 1.5 for water up to the nominal 500-year plus 5 ft raise alternative. Flotation analysis showed that the plant meets flotation criteria up to the nominal 500-year plus 5 ft raise. Refer to Exhibit A-15.14 in the Supplemental Exhibits section for structural analysis calculations.

Mechanical Evaluation. A detailed mechanical evaluation was not conducted. Raising the levee in this area will have no effect on pumping rates and efficiencies since the current arrangement does not pump "up and over" the levee.

A-15.4 SUPPLEMENTAL EXHIBITS

EXHIBIT A-15.2

Argentine Unit – Turner Pump Station Analysis

KANSAS CITY, MISSOURI AND KANSAS
SEVEN LEVEES FEASIBILITY STUDY

TURNER AVE. PUMP STATION EVALUATION BY ST. LOUIS DISTRICT

STRUCTURAL SUMMARY NARRATIVE

Total Pages: 137 (including this page)

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II. Structural Summary Narrative
   a. Section 1. Turner Pump Station Stability
   b. Uplift Computations/Results
   c. Section 2. Turner Pump Station Strength
   d. Quantity Estimate
   e. Hydraulic Grade Line Computations/Results
III. CENWK EC-GD Foundation Bearing Capacity Commentary to St. Louis Report
IV. Turner Avenue Pump Station As-Built Drawings

This Adobe file document, file name **Turner_PS_Structural_Final.pdf**, is the completed "Final" Turner Avenue Pump Station Evaluation Structural Summary Narrative. The Structural Summary Narrative in this document was created by the St. Louis District in response to NWK Dr. Checks comments generated during review of the "initial" St. Louis Structural Summary Narrative. (The initial St. Louis Structural Summary Narrative does not have a date posted. The first page of the initial report has "0_Turner PS STRUCT SUMMARY Narrative.doc" written in the footer). All comments posted in Dr. Checks were resolved as of March 15, 2005 and are reflected in this Turner_PS_Structural_Final.pdf file.

As an aid to future referencing, items contained in the initial St. Louis Structural Summary Narrative document that did not require revision have been included in the Turner_PS_Structural_Final.pdf file (As-Built drawings, HGL analysis, etc).

In regards to Dr. Checks comment (Id 679691) addressing the need for an additional retaining wall to prevent additional pressure on the pump station walls, the following information is provided for future reference: *"The existing RCB will not be strengthened. The De-Loading method will be used and as a result, the additional retaining walls specified in the initial St. Louis Structural Summary Narrative will not be necessary."* This information will also be provided in Black & Veatch's Argentine Pump Station Report.

KANSAS CITYS, MISSOURI AND KANSAS
SEVEN LEVEES FEASIBILITY STUDY

PUMP STATION EVALUATIONS BY ST. LOUIS DISTRICT

STRUCTURAL SUMMARY NARRATIVE

(Revised 11/2004 due to KC Review [v2])
(Revised 1/2005 after KC Summit #1 12/1/04 [v3])

Levee Unit:	Argentine
Pump Station:	Turner Ave. (Kaw Valley No. 4)
Levee Station:	Sta. 60+40
Type of Raise:	Levee Raise
Flood Event:	500yr+3
Proposed Protection:	Elevation 780.8

1. Data. The data gathered and used to evaluate the pump station includes the following:

1.1 O&MM Drawings. Construction drawings of the original pump station and the modifications to the pump station where the retaining wall and new pumps were added were provided in the Operation and Maintenance Manual drawings for the Argentine Unit. These O&MM drawings were provided by the Kansas City District in scanned image format (*.tif files).

1.2 Hydraulic Grade Line. Hydraulic gradient at design flood was provided and coordinated with the geotechnical engineer in Kansas City District.

1.3 Profile of Proposed Levee Section.Profile of proposed raised levee section was provided by and coordinated with Kansas City District. Note: at summit meeting #1, the use of a flood wall at the pump station location for the purposes of de-loading the box culvert was discussed. It is understood that the current plan includes a wall at the location of the pump station.

1.4 Geotechnical/soils data.The geotechnical soils data used in the structural investigation of the pump station were provided by the Kansas City District.

1.5 Structural Criteria.

- Issue Resolution Paper, Kansas Citys, Missouri and Kansas River Levees System, dated August 2003 provided by Kansas City District.
- EM 1110-2-2104 Strength Design for Reinforced-Concrete Hydraulic Structures, dated June 1992
- Draft EM 1110-2-2100 Stability Analysis of Concrete Structures
- EC 1110-2-60058 Stability Analysis of Concrete Structures

2. Pump Station Stability.

2.1 Floatation Stability. Reference the stability computations for further information.

Flotation stability due to increased uplift pressures on the pump station which are based upon the hydraulic gradient line (HGL) was investigated. Sliding and overturning was not included in the scope of the stability investigation based upon collective engineering decision that flotation will control and that sliding and overturning should not be a significant issue. Therefore, uplift was initially investigated for the with

project 500+3 condition. Initial findings for the full HGL case indicated that the required factor of safety for uplift could not be met. In addition, after summit #1, flotation stability was investigated for the existing conditions – a flood producing a HGL approximately 12.3 feet above grade will also result in unsatisfactory uplift performance. The additional uplift calculation also identified the effect of the 8 ft of water in the pump station. (ie, determined the factor of safety with and without the 8 ft water)

2.2 Stability Criteria Requirements. The required factor of safety were provided by the Kansas City District in Table 3 of the Issue Paper dated Aug 2003 for this specific project. The factors of safety are summarized below:

Recommended Flotation Stability Factors of Safety		
Load Condition Category	**Return Period**	**Factor of Safety**
Usual	10 yrs	1.3
Unusual	300 yrs	1.2
Extreme	Top of Protection	1.1

For the analysis of the levee raise for Turner pump station, the top of protection category will be applicable, and therefore the factor of safety of 1.1 against flotation will be the minimum. In addition, the 1.1 factor of safety is applicable to the flood condition for the existing conditions.

2.3 Loading Conditions.
The following loading conditions were considered:

Loading Condition	**Loading Condition Description**
0	Existing Station only with no gatewell raise. Soil Loads not included. Investigation for bearing pressures only.
00	(Added after summit #1, 12/1/04) Existing Station only with no gatewell raise. UPLIFT investigated for flood case of the existing conditions. Distance of the HGL above grade ranged from 0 to 15 feet, with 12.3 feet applicable to existing conditions. This existing condition, without project load case was investigated after the initial analysis of the with project 500+3 condition, and was not run in 3DSAD. Simple uplift calculations were documented in excel spreadsheet. Results of the full range were also plotted in a chart with distance HGL above grade vs. uplift factor of safety.
01	Existing station with gatewell raise for 500+3 protection raise. Soil loads not included.
A	Existing station with gatewell raise. Soil loads for existing condition are included.
B	Existing station with gatewell raise. Also includes proposed 500+3 protection raise and flood level to top of protection. Full HGL considered. 8 feet water in station.
C	Same as B with HGL reduced by 7 feet (HGL -7)
D	Same as B with HGL reduced by 8 feet (HGL -8)
E	Same as B with HGL reduced 9 feet (HGL –9)

2.4 Uplift and Bearing Pressure Determination. The pump station was modeled in the Corps of Engineers CASE program 3DSAD to determine pump station weights. 3DSAD will also be used to determine soil bearing pressures at the base. Factor of safety for uplift was determined using computed weights from the 3DSAD models and uplift pressures determined from the HGL. An initial evaluation by the NWK geotech engineer identified an allowable bearing pressure of 2000psf. A summary of results is shown:

Load Condition	**Load Description**	**FS Uplift**	**Max brg pressure (ksf)**
0	Existing Station Only, NO GW raise	n/a	2.137 (net)
00	Existing Station Only – UPLIFT for Existing Condition with HGL 12.3 ft above grade	0.95	n/a
01	Existing Station Only + GW raise	n/a	2.248 (net)
A	Existing Station + GW raise + soil	n/a	2.858
B	Existing Station + GW raise 500+3 (FULL HGL)	0.90	0.486
C	Existing Station + GW raise 500+3 (HGL –7 ft)	1.08	0.312
D	Existing Station + GW raise 500+3 (HGL –8 ft)	1.11	0.616
E	Existing Station + GW raise 500+3 (HGL –9 ft)	1.15	0.613

(note: above FS for uplift include 8 ft water in pump station)

Reference attached summary TABLE 1-1, which primarily summarizes the with project 500+3 condition. UPLIFT (or flotation) factor of safety was calculated as 0.90 for load condition B, which is the station experiencing full hydraulic gradient (HGL) per the 500yr +3ft level. This factor of safety as compared to the minimum 1.1 is unacceptable.

Two distinct alternatives to alleviate the potential uplift problem as stated above include the following:

a. ADD WEIGHT TO STRUCTURE. For the with project 500+3 condition, an addition of approximately 860 kips would be required to be added to the dead weight of the structure to achieve a 1.1 factor of safety. For the without project existing condition, assuming a 12.3 ft above grade HGL, the additional weight required is approximately 700 kips.
    i. CONCRETE. For the with project 500+3 condition, this is about 5710 cubic feet of concrete. This is approximately 3 feet of new concrete inside the station over the entire base area of the pump station, approximately 1900 square feet. From a practical, operational, and functional standpoint, this is not feasible.
    ii. SOIL. Engaging additional soil dead weight is also an alternative to attempt to increase the dead weight. The extension would have to be increased to an impracticable length before any significant benefit to the uplift factor of safety was realized. In addition, the drawback of this is concept is the extensive excavation that would be required directly adjacent to the pump station, which in turn could produce sliding or overturning stability problems.

NOTE: After summit #1 (12/1/04), base extension was re-investigated for the existing condition based upon HGL 12.3 above grade. The base was assumed to be extended both in the east and in the north direction, and resulting affect on uplift factor of safety documented. Results of this are included in table “UPLIFT - Existing BASE EXT-E”, Table 3-2 “UPLIFT - Existing BASE EXT-NE” and PLOT 2 “UPLIFT PLOT - BASE EXT”

b. REDUCE UPLIFT VIA RELIEF WELLS. In order to achieve a 1.1 factor of safety, the uplift pressures would have to be reduced by lowering the HGL via a relief well system.

Based upon discussions and coordination with NWK, it was generally agreed that the most feasible and economic option would be to reduce the HGL thru a relief well system.

2.5 If the HGL is drawn down to reduce the uplift pressures, a higher factor of safety against uplift will be achieved. Initially, the minimum amount of drawdown for the with project 500+3 condition was back calculated based upon the minimum 1.1 factor of safety required. This minimum amount of drawdown was determined to be approximately 7 to 8 feet for the 500+3 condition. Three drawdown cases for loading conditions C, D, and E were evaluated for drawdowns of 7, 8, and 9 feet respectively. The results of which are shown and summarized in the table in paragraph 2.4 above.
The results of the drawdown by relief wells on the factor of safety, for both the with project 500+3 condition and the without project existing condition are illustrated in the plot entitled "TURNER PUMP STATION – UPLIFT FACTOR OF SAFETY (_Turner Stability.v3.xls spreadsheet tab "UPLIFT PLOT").

2.6 Recommendation. In order to achieve the required factor of safety for uplift, it is recommended that the uplift pressures on the pump station be reduced by means of relief well installation. The recommended magnitude of the HGL drawdown is minimum 8 feet, but possibly 9 feet, from the with project 500+3 condition, depending upon the final desired factor of safety. This recommendation has been coordinated with the geotechnical engineers in Kansas City District. Kansas City District will develop required relief well system and related costs. It is further recommended that in order to adequately monitor the stability of the station in flood conditions, that a water monitoring system be incorporated into the operational procedures for this station.

3. Pump Station Strength.

3.1 Structural Elements. Select portions of the pump station received a structural strength capacity check. The strength check investigated the existing member's capacity to accommodate any change in loading due to the proposed levee raise. Overall, the general approach for strength checks was to make simple, and possibly conservative, assumptions to simplify analysis. This approach was used to quickly reach a determination of potential strength problems or issues, at the same time as minimizing engineering effort due to funding restraints. All elements and portions were analyzed using strength design procedures per EM 1110-2-2104. Portions of the structure checked were determined to be worst case-controlling representatives of the structure based upon engineering judgment. General portions checked include: Exterior Wall, Collar Beam, Base Slab, and Discharge Chamber Wall. Reference strength computation narratives for each element for detailed information: (a brief summary follows).

NOTE: After summit #1 (12/1/04), existing members were investigated based upon the approach outlined in the STRUCTURAL ANALYSIS CHAPTER A-11 of the report prepared by the Kansas City District which is summarized here:

> Load and strength reduction factors were not used in the analysis of the existing structures. This implies that if an existing structure has a calculated Factor of Safety of less than 1.0, the structure has ceased to function as designed. When considering an allowable factor of safety for existing structures, several allowable reductions can be taken into account. EM 1110-2-2104 allows for a 25% reduction in load for short duration loads with a low probability of occurrence, which would apply to flood events with a return period of greater that 300 years, A "performance" factor can also be applied to take into account the previous behavior of the existing structure. Knowing that the existing structure has performed well under loading and not shown visible signs of distress, it is assumed a 15% reduction in factor of safety is acceptable. Combining the design factor with the frequency and performance factors produces an approximate 1.5 factor of safety for existing hydraulic structures in extreme loading conditions.

3.2 Material Assumptions. Material properties could not be determined for the existing documentation on Turner Pump Station. As a result assumptions consistent with the assumptions for the remainder of the Argentine Unit were used. ACI recommends the use of 3000 psi concrete strength for construction around the late 1940s and 1950s. As concrete ages it continues to hydrate and gain strength. FEMA 310, Section 4.2.4.4 states, "Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." (FEMA 310, 1999) Using this information, a mean concrete compressive strength, f'c of 3750 psi was assumed for the existing concrete in the pump station. In addition, an assumed reinforcing steel yield strength of 40 ksi was used.

3.3 Strength Result Summary.

A general summary of elements investigated using unfactored approach (as described above) for with project 500+3 condition (results after summit #1 12/1/04):
Table 3.2-1

Structural Element	**Flexural Check Factor of Safety Mn/Mmax**	**Shear Check (Conc only) Factor of Safety Vn/Vmax**	**Shear Check (Conc + steel) Factor of Safety Vn/Vmax**	**Flexure/Axial Interaction**	**ACI 318 10.5.1 Min met ?**
PS Exterior Wall	2.92	1.57	-	OK	OK
PS Collar Beam (FULL HGL)	2.36	1.40*	1.93	-	OK
PS Collar Beam (HGL –8 ft)	-	-	-	-	-
Base Slab	2.22	2.33	-	-	NO
Discharge Chamber Walls A at El. 776.8	5.99**	11.8	-	-	NO
Discharge Chamber Walls B at El. 772.0	1.83**	5.20	-	-	NO
Discharge Chamber Walls C at El. 767.5	1.21**	3.44	-	-	NO
Discharge Chamber Walls D at El. 762.0	1.63**	2.43	-	-	NO
Discharge Chamber Walls E at El. 757.0	1.64**	3.10	-	-	NO
Discharge Chamber Walls F at El. 752.0	1.84**	2.56	-	-	NO
Discharge Chamber Walls G at El. 746.5	2.02	2.15	-	-	NO
Discharge Chamber Wall Beam	2.35	1.67*	-	-	OK

Notes:

* = factor of safety for shear at distance d from support. Otherwise, factor of safety for shear at support

** = factor of safety for negative moment at ends of member

3.4 Pump Station Exterior Wall & Collar Beam. The exterior wall was investigated essentially at two locations, the vertical span portion of the wall which spans from base slab to collar beam, and the collar beam.

The exterior wall was modeled as a simple 1 ft strip beam spanning vertically, with fixed ends, loaded laterally with soil loads and increased water pressures due to levee raise and full HGL. The factors of safety for moment and shear were both above 1.5, therefore the exterior wall beam strip was found to be adequate to accommodate additional loads for the with project 500+3 condition.

Note that these members did meet the ACI minimums for tension steel.

The collar beam was investigated based upon the spans on the south side of the station. For simplicity, the collar beam is assumed to span horizontally and is investigated as a three-span span continuous beam with fixed ends. The loading on the beam consisted of soil loads and increased water pressures due to levee raise and full HGL with a tributary width equal to the beam width plus 3 feet above and below the beam. Using the unfactored approach , the factors of safety for both moment and shear were above 1.5 for the 500+3 condition. The shear check is based upon contribution from stirrups for shear steel. The collar beam therefore was found to be adequate.

Note that the collar beam did meet the ACI minimums for tension steel.

3.5 Base Slab. For the loading condition of flood event with the proposed levee raise, the base slab of the pump station will be investigated for strength to resist the resulting uplift pressures. The uplift pressures used to evaluate the base slab strength will conservatively be based upon the full HGL for the with-project 500+3 condition. The base slab is assumed to span in the north south direction.

The unfactored factor of safety approach also indicated that the slab was adequate. The factors of safety of flexure and shear were both above 1.5.

Note this member does NOT meet ACI 10.5.1 minimum for tension steel.

3.6 Discharge Chamber Interior Wall. For simplicity, the wall is assumed to span horizontally and is investigated as a single span 1-foot strip beam with fixed ends. Maximum positive moment in center (tension face on outside) was computed and maximum negative moment at ends (tension face on inside) was computed. As shown in the table above, wall segments A, B, and D thru G (per the section diagram in the computations) were found to be adequate in flexure and shear. Note that the negative moment at the ends of the member controlled the flexural check. The factors of safety for flexure and shear for these segments of the wall were above the 1.5.

Note, however that the tension steel did NOT meet the ACI minimum for these members.

However, wall segments C did not meet the factor of safety of 1.5 for the end or negative moment region. The factor of safety is above one, so it can be concluded that a complete failure and collapse of this member is not eminent under the 500+3 condition, but would be overstressed. It is recommended that this end region of the wall, where the negative flexural factor of safety is below 1.5, be strengthened to achieve the minimum 1.5 factor of safety.

The integral “beam” in the east wall of the gatewell/discharge chamber just above the sluice opening was also investigated for increased lateral loads due to water pressure and found to be adequate The beam was found to be adequate for lateral moment capacity, and the factors of safety were above 1.5.

The tension steel in this member did meet the ACI minimum.

3.7 Recommendation. Overall, from a feasibility level of effort standpoint, strength investigation found the majority of the elements of the pump station to be adequate to accommodate additional loading.

Therefore, for these elements no action is necessary to accommodate the levee raise. However, some of the members as noted in the summary table above do not meet ACI minimum for tension steel.

One of the exceptions to this were select portions of the east wall of the discharge chamber, as previously discussed (and as discussed in the computation narrative for this wall). However, based upon the extremely low probability of the flood to top of protection loading condition for which these walls were investigated, and based upon the fact that it was found that the nominal capacity was not exceeded by actual unfactored loads (i.e., member will not fail), it is being recommended that no modification be necessary to increase the structural strength these specific wall segments. It is recommended that these walls, in addition to the entire structure be carefully monitored during any flood event.

4. Discharge Chamber/Gatewell.

4.1 Required Structural Raise. Height of discharge chamber/gatewell will need to be raised to the same height as the line of protection.

Existing Conditions. The original top of the gatewell was at El. 772.0. Per the Argentine O&M plates, the protection level at this location was raised from approximately 772 to 776.8, and the gatewell was also raised accordingly. The 1962 raised the top of gatewell 4.8 feet from El. 772 to El. 776.8. The raise was accomplished by simply "adding" on to the top of the structure with reinforced cast-in-place concrete. Therefore the current top of the existing gatewell is at El. 776.8.

Proposed Change. With the proposed level of protection raise, the top of the gatewell must also be raised to equal the top of protection. The proposed 500+3 level will raise the top of the structure 4 feet, to El. 780.8. Assume that the raise will be of similar construction to that of the raise in 1962.

	Elevation (ft)	**Difference (ft)**
Proposed Gatewell raise	780.8	0
Existing	776.8	4
"Pre 1962"	772.0	8.8

4.2 Discharge Pipes. Discharge pipe from new pumps will need to be raised or put through the raised section of the discharge chamber/gatewell. The discharge pipes will remain at the existing elevation, and be incorporated into the wall of the gatewell raise.

4.3 Gate Operator. Electric motor actuated sluice gate operator needs to be raised and a longer stem installed.

4.4 Wall Strength. Structural strength of the discharge chamber/gatewell walls was investigated due to increased groundwater pressures at design flood level. See above.

5. Levee Profile/Retaining Wall

5.1 An earthen levee raise will require that the existing retaining wall directly adjacent to the pump station along the pumps will need to be raised and extended. Increased soil loads may require that this wall be replaced. However, the current approach is to install a new wall on the levee crown to achieve the with-project 500+3 protection adjacent to the pump station. This is being done for purposes of deloading the box culvert beneath the levee. This will eliminate the need for any retaining wall modifications directly adjacent to the station.

SECTION 1.
Turner Pump Station
STABILITY

Levee Unit: Argentine
Pump Station: Turner Ave. (Kaw Valley No 4)
Levee Station: 60+40
Flood Event: 500+3
Proposed Protection: EL. 780.8

Revisions:

1. Stability – Flotation/Uplift and Bearing Pressure Investigation (REVISED NOV 2004)

2. Additional Uplift Computations added Jan 2005 as result of KC summit meeting #1 12/1/2004 – added comps for existing station under existing condition flood included at the end of this section (REVISED Jan 2005)

NOTES:

1. Loading. Per conversation with Scott Loehr and John Hall of CENWK, the new levee embankment will NOT be placed against existing structures. Geotech in NWK plan to add new retaining wall to retain the new levee embankment and keep it off of the existing pump station. Therefore assume that no additional soil loads on the structure. Loading change for the structure will be due to increased water pressures only. Further information at Summit #1 clarified this :

   Profile of proposed raised levee section was provided by and coordinated with Kansas City District. Note: at summit meeting #1, the use of a flood wall at the pump station location for the purposes of de-loading the box culvert was discussed. For this reason, it is understood that the plan includes a wall at the location of the pump station. This will eliminate additional soil loads on the pump station.

2. The pump station was modeled in the Corps of Engineers CASE program 3DSAD to determine pump station weights. 3DSAD will also be used to determine soil bearing pressures at the base. The geometry of the station is modeled with appropriate material densities used for each portion of the structure. Material unit weights were assumed as follows for input into the 3DSAD model:

Material	**Unit Weight (kcf)**
Reinforced concrete	0.150
Water	0.0625
Brick Walls of superstructure	0.120
Roof of superstructure	0.130
Moist soil	0.110
Submerged (total) soil	0.116

3. Mechanical equipment weights were also included in the dead weight determination of the structure. These weights were taken from actual equipment weights when available from the mechanical engineer, and otherwise estimated based upon engineering judgment.
   Mechanical weights assumed in the models are as follows:

ITEM	**WT (k)**
84”x84” Sluice gate	8.5
108”x108” Sluice gate	10.5
Original East pump (24”)	4.6
Original East engine	3.0
Original West pump (36”)	13.2
Original West engine	2.5
Sump pump + piping	4.8
New pump #1 + motor	14.7
New pump #2 + motor	14.7

4. The loading conditions as noted and defined in the pump station STRUCTURAL SUMMARY narrative were investigated.

5. 3DSAD files used:

TPSF0.TXT = Define geometry and dead weight of Station, including both superstructure and substructure, and mechanical weights.

TPSF1.TXT = Define geometry and dead weight of gatewell raise. This is the required raised portion of the gatewell from existing to protection level of EL. 780.8.

TPSLCA.TXT = Defines soil loads on pump station for existing loading condition A. (revised Nov 2004)

TPSLCB.TXT = Defines soil and water loads on pump station for the loading condition B. (revised Nov 2004)

TPSLCC.TXT = Defines soil and water loads on pump station for the loading condition C. (revised Nov 2004)

TPSLCD.TXT = Defines soil and water loads on pump station for the loading condition D. (revised Nov 2004)

TPSLCE.TXT = Defines soil and water loads on pump station for the loading condition E. (revised Nov 2004)

TPSANA0.TXT = Analysis input file for loading condition 0.
TPSANA01.TXT = Analysis input file for loading condition 01.
TPSANA.TXT = Analysis input file for loading condition A. (revised Nov 2004)
TPSANB.TXT = Analysis input file for loading condition B. (revised Nov 2004)
TPSANC.TXT = Analysis input file for loading condition C. (revised Nov 2004)
TPSAND.TXT = Analysis input file for loading condition D. (revised Nov 2004)
TPSANE.TXT = Analysis input file for loading condition E. (revised Nov 2004)

A print of the input files for all of the above follows. Output of the loads and analysis runs for loading conditions A, B &D also are printed. A summary table, TABLE 1-1, of the all inputs and results from the 3DSAD model run is included on the attached Excel spreadsheet (Turner Stability.xls). A simplified summary table is provided defined in the pump station STRUCTURAL SUMMARY narrative.

6. Reference attached summary TABLE 1-1. The results of the loading conditions 0 and 01 indicate that the additional deadweight due to the addition and raise of the gatewell has minimal impact on the bearing pressures.

7. Reference attached summary TABLE 1-1. The results of loading condition A indicate that under typical or normal conditions with no uplift pressure present from flood water that bearing pressures under the existing station range from approximately 1.490 ksf to 2.858 ksf. The maximum bearing pressure of 2.858 ksf occurs at the northeast corner of the pump station base (point 704 in the 3DSAD model shown in the "TURNER PS – BASE PLAN" sketch included herein.

8. Reference attached summary TABLE 1-1. UPLIFT (or flotation) factor of safety was calculated as 0.90 for load condition B, which is the station experiencing full hydraulic gradient (HGL) per the 500yr +3ft level. This factor of safety as compared to the minimum 1.1 is unacceptable.

TURNER PUMP STATION
3DSAD element descriptions

Filename	ID	Description
TPSF0.txt	BASE	3' pump station base with hole for sump
	HO01	-1' void in top of BASE for GW base slab
	SUBA	2.5' thick sump base
	HO02	-1 void in SUBA sump base
	LOWA	mass concrete for lower walls, top of base to top of collar
	HO03	void in LOWA for gatewell
	HO04	void in LOWA for south chamber
	HO05	void in LOWA for north chamber
	HO06	void in LOWA for 10' intake pipe east wall
	HO07	void in LOWA for 7'x7' opening in interior wall
	HO08	void in LOWA for 9'x9' opening in gatewell wall
	HO09	void in LOWA for north 5'x7' opening in gatewell wall
	HO10	void in LOWA for south 5'x7' opening in gatewell wall
	SOBT	additional concrete for collar beyond wall thickness on south wall
	NOBT	additional concrete for collar beyond wall thickness on north wall
	WEBT	additional concrete for collar beyond wall thickness on west wall
	EABT	additional concrete for collar beyond wall thickness on east wall
	STR1	southwest interior strut beam
	STR2	northwest interior strut beam
	STR3	southeast interior strut beam
	STR4	northeast interior strut beam
	CEBT	additional concrete for collar on interior wall
	UPWA	mass concrete for upper walls, top of collar to top of substructure
	HO11	void in UPWA for gatewell
	HO12	void in UPWA for south chamber
	HO13	void in UPWA for north chamber
	DBM1	deck beam B-1 south span
	DBM2	deck beam B-1 south span
	DBM3	deck beam B-1 south span
	DBM4	deck beam B-1 south span
	DBM5	deck beam B-3 south span
	DBM6	deck beam B-1 north span
	DBM7	deck beam B-1 north span
	DBM8	deck beam B-1 north span
	DBM9	deck beam B-1 north span
	DBM10	deck beam B-2 north span
	DBM11	deck beam B-2 north span
	HO14	void in slab at EL. 762 for 36" pump
	HO15	void in slab at EL. 762 for added pumps
	HO16	void in slab at EL. 762 for 24" pump
	HO17	void in slab at EL. 762 for stairs
	STRW	mass stairs
	HO18	void for stairs
	PLFM	platform in south chamber at bottom of stairs
	HO19	void in iterior wall for stairs and platform

	HO20	misc voind in interior wall
	HO21	void iin interior wall for 24" discharge pipe
	HO22	misc void in interior wall
	HO23	void in interior wall for 36" discharge pipe
	HO24	void in gatewell wall for discharge pipe
	HO25	void in gatewell wall for discharge pipe
	TGTW	mass concrete for gatewell above 762 to 772
	HO26	void in TGTW for gatewell 762 to 772
	DB12	gatwell deckbeams
	DB13	gatwell deckbeams
	BRWA	mass brick superstructure
	HO27	void in BRWA for superstructure floor to roof
	ROOF	roof of superstructure
	BRP1	mass brick parapet above roof
	HO28	void in BRP1 for brick parapet
	BRP2	brick parapet west wall adjacent to gatewell
	HO29	misc void in brick superstructure walls
	HO30	misc void in brick superstructure walls
	HO31	misc void in brick superstructure walls
	HO32	misc void in brick superstructure walls
	HO33	misc void in brick superstructure walls
	HO34	misc void in brick superstructure walls
	HO35	misc void in brick superstructure walls
	ARW1	west air vent at south wall substructure
	ARW2	east air vent at south wall substructure
	HO36	misc void in ARW1 & ARW2
	HO37	misc void in ARW1 & ARW3
	HO38	misc void in ARW1 & ARW4
	HO39	misc void in ARW1 & ARW5
	HO40	voids in south wall for air vents
	HO41	voids in south wall for air vents
	HO42	voids in south wall for air vents
	HO43	voids in south wall for air vents
	GTWE	mass concrete for gatewell original extension above 772
	HO44	void in GTWE for original gatewell extension
	DB14	original gatewell extension deckbeams
	DB15	original gatewell extension deckbeams
	W4	mechanical equipment
	W5	mechanical equipment
	W6	mechanical equipment
	W7	mechanical equipment
	W8	mechanical equipment
	W9	mechanical equipment
	W10	mechanical equipment
	W11	mechanical equipment
	W12	mechanical equipment
	W13	mechanical equipment
TPSF1.txt	GWE2	mass conrete for proposed gatewell extension 776.8 to 780.8

	HO45	void in GWE2 for proposed gatewell extension
	DB16	proposed gatewell extension deckbeams
	DB17	proposed gatewell extension deckbeams
	HO46	void in proposed gatewell extension for extiing 18" discharge pipes
	HO47	void in proposed gatewell extension for extiing 18" discharge pipes
TPSLCA.txt	W1A	soil vertical load above base extension - east
	W1AV	soil void for concrete collar - east (EABT)
	W1B	soil vertical load above base extension - south
	W1BV	soil void for concrete collar - south (SOBT)
	W1C	soil vertical load above base extension - north
	W1CV	soil void for concrete collar - north (NOBT)
	W1D	soil vertical load above base extension - GWsouth
	W1E	soil vertical load above base extension - GWnorth
	W1F	soil vertical load above base extension - west (lower below collar)
	W1F2	soil vertical load above base extension - west (upper below collar)
	W14A	water vertical load - 4' in sump pit
	F1	soil horizontal load - west
	F2	soil horizontal load - east
	F2A	soil horizontal load - south
	F2B	soil horizontal load - north
TPSLCB.txt	W1A	soil vertical load above base extension - east
TPSLCC.txt	W1AV	soil void for concrete collar - east (EABT)
TPSLCD.txt	W1B	soil vertical load above base extension - south
TPSLCE.txt	W1BV	soil void for concrete collar - south (SOBT)
	W1C	soil vertical load above base extension - north
	W1CV	soil void for concrete collar - north (NOBT)
	W1D	soil vertical load above base extension - GWsouth
	W1E	soil vertical load above base extension - GWnorth
	W1F	soil vertical load above base extension - west
	W1F2	soil vertical load above base extension - west (upper below collar)
	W14A	water vertical load - 4' in sump pit
	F1	soil horizontal load - west
	F2	soil horizontal load - east
	F2A	soil horizontal load - south
	F2B	soil horizontal load - north
	W2	water vertical load in gatewell
	W14B	water vertical load station, south chamber
	W14C	water vertical load station, north chamber
	F3	water horizontal load - east
	F4	water horizontal load - west
	F3A	water horizontal load - south
	F3B	water horizontal load - north

TPSF0.TXT input file:

```
100 XY
110 POINTS 12
120 1 0 -18.33 0
122 400 9 -18.33 0
124 401 9 -19.33 0
130 2 54.83 -19.33 0
140 3 54.83 19.33 0
150 4 9 19.33 0
160 5 9 -1.17 0
170 6 0 -1.17 0
180 7 20.33 -2.17 0
190 8 20.33 1.83 0
200 9 24.33 1.83 0
210 10 24.33 -2.17 0
220 BLOCK -Z BASE .15 3 1
230 1 1
240 8 1 400 401 2 3 4 5 6
250 1 1
260 4 7 8 9 10
270 POINTS 4
280 11 2 -15.33 2
290 12 2 -4.17 2
300 13 11 -4.17 2
310 14 11 -15.33 2
320 BLOCK +Z HO01 .15 1
330 1 1
340 4 11 12 13 14
350 POINTS 8
360 15 18.83 -3.67 -2.5
370 16 25.83 -3.67 -2.5
380 17 25.83 3.33 -2.5
390 18 18.83 3.33 -2.5
400 19 20.33 -2.17 -1
410 20 20.33 1.83 -1
420 21 24.33 1.83 -1
430 22 24.33 -2.17 -1
440 BLOCK -Z SUBA .15 2.5
450 1 1
460 4 15 16 17 18
470 BLOCK +Z HO02 .15 1
480 1 1
490 4 19 20 21 22
500 POINTS 6
510 23 0 -17.33 3
520 24 52.83 -17.33 3
530 25 52.83 17.33 3
540 26 11 17.33 3
550 27 11 -2.17 3
560 28 0 -2.17 3
570 BLOCK -Z LOWA .15 13.75
580 1 1
590 6 23 24 25 26 27 28
600 POINTS 4
610 29 2 -15.33 3
620 30 2 -4.17 3
630 31 11 -4.17 3
640 32 11 -15.33 3
650 BLOCK +Z HO03 .15 13.75
660 1 1
670 4 29 30 31 32
680 POINTS 6
690 33 13 -15.33 3
700 34 13 -4.17 3
710 35 39.5 -4.17 3
720 36 39.5 -0.17 3
730 37 50.83 -0.17 3
740 38 50.83 -15.33 3
750 BLOCK +Z HO04 .15 13.75
760 1 1
770 6 33 34 35 36 37 38
780 POINTS 6
790 39 13 -2.17 3
800 40 13 15.33 3
810 41 50.83 15.33 3
820 42 50.83 1.83 3
830 43 37.83 1.83 3
840 44 37.83 -2.17 3
850 BLOCK +Z HO05 .15 13.75
860 1 1
870 6 39 40 41 42 43 44
880 YZ
890 POINTS 4
900 45 50.83 -9.75 3
910 46 50.83 -14.75 8
920 47 50.83 -9.75 13
930 48 50.83 -4.75 8
940 CIRC 45 48 5
950 CIRC 48 47 5
960 CIRC 47 46 5
970 CIRC 46 45 5
980 BLOCK +Z HO06 .15 2
990 1 1
1000 4 45 46 47 48
1010 XZ
1020 POINTS 4
1030 49 40.67 -0.17 3.75
1040 50 47.67 -0.17 3.75
1050 51 47.67 -0.17 10.75
1060 52 40.67 -0.17 10.75
1070 BLOCK +Z HO07 .15 2
1080 1 1
1090 4 49 50 51 52
1100 YZ
1110 POINTS 4
1120 53 11 -5.25 3
1130 54 11 -14.25 3
1140 55 11 -14.25 12
1150 56 11 -5.25 12
1160 BLOCK +Z HO08 .15 2
1170 1 1
1180 4 53 54 55 56
1190 POINTS 8
1200 57 0 -4.17 3
1210 58 0 -9.17 3
1220 59 0 -9.17 10.63
1230 60 0 -4.17 10.63
1240 61 0 -10.33 3
1250 62 0 -15.33 3
1260 63 0 -15.33 10.63
1270 64 0 -10.33 10.63
1280 BLOCK +Z HO09 .15 2
1290 1 1
1300 4 57 58 59 60
1310 BLOCK +Z HO10 .15 2
1320 1 1
1330 4 61 62 63 64
1340 POINTS 8
1350 65 11 -17.33 13.75
1360 66 11 -17.33 16.75
1370 67 11 -18.33 16.75
1380 68 11 -18.33 13.75
1390 69 9 18.33 13.75
1400 70 9 18.33 16.75
1410 71 9 17.33 16.75
1420 72 9 17.33 13.75
1430 BLOCK -Z SOBT .15 43.83
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1440 1 1
1450 4 65 66 67 68
1460 BLOCK -Z NOBT .15 45.83
1470 1 1
1480 4 69 70 71 72
1490 XZ
1500 POINTS 8
1510 73 9 -2.17 13.75
1520 74 9 -2.17 16.75
1530 75 11 -2.17 16.75
1540 76 11 -2.17 13.75
1550 77 52.83 -17.33 13.75
1560 78 52.83 -17.33 16.75
1570 79 54.83 -17.33 16.75
1580 80 54.83 -17.33 13.75
1590 BLOCK -Z WEBT .15 19.5
1600 1 1
1610 4 73 74 75 76
1620 BLOCK -Z EABT .15 34.67
1630 1 1
1640 4 77 78 79 80
1650 POINTS 8
1660 81 24.67 -15.33 14.08
1670 82 24.67 -15.33 16.42
1680 83 26.33 -15.33 16.42
1690 84 26.33 -15.33 14.08
1700 85 24.67 -1.67 14.08
1710 86 24.67 -1.67 16.42
1720 87 26.33 -1.67 16.42
1730 88 26.33 -1.67 14.08
1740 BLOCK -Z STR1 .15 11.17
1750 1 1
1760 4 81 82 83 84
1770 BLOCK -Z STR2 .15 17
1780 1 1
1790 4 85 86 87 88
1800 POINTS 8
1810 89 37.83 -15.33 14.08
1820 90 37.83 -15.33 16.42
1830 91 39.5 -15.33 16.42
1840 92 39.5 -15.33 14.08
1850 93 37.83 1.83 14.08
1860 94 37.83 1.83 16.42
1870 95 39.5 1.83 16.42
1880 96 39.5 1.83 14.08
1890 BLOCK -Z STR3 .15 11.17
1900 1 1
1910 4 89 90 91 92
1920 BLOCK -Z STR4 .15 13.5
1930 1 1
1940 4 93 94 95 96
1950 YZ
1960 POINTS 4
1970 97 13 -2.17 13.75
1980 98 13 -1.67 13.75
1990 99 13 -1.67 16.75
2000 100 13 -2.17 16.75
2010 BLOCK -Z CEBT .15 24.83
2020 1 1
2030 4 97 98 99 100
2040 XY
2050 POINTS 6
2060 101 0.33 -17 16.75
2070 102 52.5 -17 16.75
2080 103 52.5 17 16.75
2090 104 11.33 17 16.75
2100 105 11.33 -2.5 16.75
2110 106 0.33 -2.5 16.75
2120 BLOCK -Z UPWA .15 15.52
2130 1 1
2140 6 101 102 103 104 105 106
2150 POINTS 4
2160 107 2 -15.33 16.75
2170 108 2 -4.17 16.75
2180 109 11.33 -4.17 16.75
2190 110 11.33 -15.33 16.75
2200 BLOCK +Z HO11 .15 15.52
2210 1 1
2220 4 107 108 109 110
2230 POINTS 6
2240 111 13 -15.33 16.75
2250 112 13 -4.17 16.75
2260 113 39.5 -4.17 16.75
2270 114 39.5 -0.17 16.75
2280 115 50.83 -0.17 16.75
2290 116 50.83 -15.33 16.75
2300 BLOCK +Z HO12 .15 14.85
2310 1 1
2320 6 111 112 113 114 115 116
2330 POINTS 6
2340 117 13 -2.5 16.75
2350 118 13 15.33 16.75
2360 119 50.83 15.33 16.75
2370 120 50.83 1.5 16.75
2380 121 37.83 1.5 16.75
2390 122 37.83 -2.5 16.75
2400 BLOCK +Z HO13 .15 14.85
2410 1 1
2420 6 117 118 119 120 121 122
2430 XZ
2440 POINTS 16
2450 123 18.58 -15.33 30.60
2460 124 19.58 -15.33 30.60
2470 125 19.58 -15.33 31.60
2480 126 18.58 -15.33 31.60
2490 127 25.25 -15.33 30.60
2500 128 26.25 -15.33 30.60
2510 129 26.25 -15.33 31.60
2520 130 25.25 -15.33 31.60
2530 131 31.92 -15.33 30.60
2540 132 32.92 -15.33 30.60
2550 133 32.92 -15.33 31.60
2560 134 31.92 -15.33 31.60
2570 135 38.5 -15.33 30.60
2580 136 39.5 -15.33 30.60
2590 137 39.5 -15.33 31.60
2600 138 38.5 -15.33 31.60
2610 BLOCK -Z DBM1 .15 11.17
2620 1 1
2630 4 126 125 124 123
2640 BLOCK -Z DBM2 .15 11.17
2650 1 1
2660 4 130 129 128 127
2670 BLOCK -Z DBM3 .15 11.17
2680 1 1
2690 4 134 133 132 131
2700 BLOCK -Z DBM4 .15 11.17
2710 1 1
2720 4 138 137 136 135
2730 POINTS 4
2740 139 44.67 -15.33 30.44
2750 140 45.67 -15.33 30.44
2760 141 45.67 -15.33 31.60
2770 142 44.67 -15.33 31.60
2780 BLOCK -Z DBM5 .15 15.17
2790 1 1
2800 4 142 141 140 139
2810 POINTS 16
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2820 143 16.17 -2.5 29.52
2830 144 17.17 -2.5 29.52
2840 145 17.17 -2.5 31.60
2850 146 16.17 -2.5 31.60
2860 147 20.83 -2.5 29.52
2870 148 21.83 -2.5 29.52
2880 149 21.83 -2.5 31.60
2890 150 20.83 -2.5 31.60
2900 151 30.5 -2.5 29.52
2910 152 31.5 -2.5 29.52
2920 153 31.5 -2.5 31.60
2930 154 30.5 -2.5 31.60
2940 155 35.17 -2.5 29.52
2950 156 36.17 -2.5 29.52
2960 157 36.17 -2.5 31.60
2970 158 35.17 -2.5 31.60
2980 BLOCK -Z DBM6 .15 17.83
2990 1 1
3000 4 146 145 144 143
3010 BLOCK -Z DBM7 .15 17.83
3020 1 1
3030 4 150 149 148 147
3040 BLOCK -Z DBM8 .15 17.83
3050 1 1
3060 4 154 153 152 151
3070 BLOCK -Z DBM9 .15 17.83
3080 1 1
3090 4 158 157 156 155
3100 POINTS 8
3110 159 42.83 1.5 30.44
3120 160 43.83 1.5 30.44
3130 161 43.83 1.5 31.60
3140 162 42.83 1.5 31.60
3150 163 46.83 1.5 30.44
3160 164 47.83 1.5 30.44
3170 165 47.83 1.5 31.60
3180 166 46.83 1.5 31.60
3190 BLOCK -Z DB10 .15 13.83
3200 1 1
3210 4 162 161 160 159
3220 BLOCK -Z DB11 .15 13.83
3230 1 1
3240 4 166 165 164 163
3250 POINTS 12
3260 167 17.17 2 32.27
3270 168 17.17 2 31.60
3280 169 20.83 2 31.60
3290 170 20.83 2 32.27
3300 171 31.5 2 32.27
3310 172 31.5 2 31.60
3320 173 35.17 2 31.60
3330 174 35.17 2 32.27
3340 175 43.83 2 32.27
3350 176 43.83 2 31.60
3360 177 46.83 2 31.60
3370 178 46.83 2 32.27
3380 BLOCK +Z HO14 .15 13.33
3390 1 1
3400 4 167 168 169 170
3410 BLOCK +Z HO15 .15 13.33
3420 1 1
3430 4 171 172 173 174
3440 BLOCK +Z HO16 .15 13.33
3450 1 1
3460 4 175 176 177 178
3470 POINTS 4
3480 179 47.83 2 32.27
3490 180 47.83 2 31.60
3500 181 50.83 2 31.60
3510 182 50.83 2 32.27
3520 BLOCK +Z HO17 .15 10.67
3530 1 1
3540 4 179 180 181 182
3550 YZ
3560 POINTS 4
3570 183 47.17 12.67 31.10
3580 184 47.17 1.5 31.10
3590 185 47.17 1.5 19.02
3600 186 47.17 1.83 19.02
3610 BLOCK -Z STRW .15 3.67
3620 1 1
3630 4 183 184 185 186
3640 POINTS 3
3650 187 47.83 12.25 31.10
3660 188 47.83 1.5 19.31
3670 189 47.83 1.5 31.10
3680 BLOCK +Z HO18 .15 3
3690 1 1
3700 3 187 188 189
3710 POINTS 4
3720 190 39.5 -0.17 18.52
3730 191 39.5 -0.17 19.02
3740 192 39.5 -3.17 19.02
3750 193 39.5 -3.17 18.52
3760 BLOCK -Z PLFM .15 11.33
3770 1 1
3780 4 190 191 192 193
3790 POINTS 4
3800 194 47.83 1.5 19.02
3810 195 47.83 -0.17 19.02
3820 196 47.83 -0.17 26.02
3830 197 47.83 1.5 26.02
3840 BLOCK +Z HO19 .15 3
3850 1 1
3860 4 194 195 196 197
3870 XZ
3880 POINTS 8
3890 198 50.83 -0.17 29.10
3900 199 50.83 -0.17 31.10
3910 200 47.83 -0.17 31.10
3920 201 47.83 -0.17 29.10
3930 202 39.5 -0.17 26.60
3940 203 42.33 -0.17 26.60
3950 204 42.33 -0.17 28.94
3960 205 39.5 -0.17 28.94
3970 BLOCK +Z HO20 .15 1.67
3980 1 1
3990 4 198 199 200 201
4000 BLOCK +Z HO21 .15 1.67
4010 1 1
4020 4 202 203 204 205
4030 POINTS 8
4040 206 13 -4.17 29.10
4050 207 16.17 -4.17 29.10
4060 208 16.17 -4.17 31.10
4070 209 13 -4.17 31.10
4080 210 17.33 -4.17 22.10
4090 211 20.67 -4.17 22.10
4100 212 20.67 -4.17 25.44
4110 213 17.33 -4.17 25.44
4120 BLOCK +Z HO22 .15 1.67
4130 1 1
4140 4 206 207 208 209
4150 BLOCK +Z HO23 .15 1.67
4160 1 1
4170 4 210 211 212 213
4180 YZ
4190 POINTS 4
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4200 214 11.33 -7 22.37
4210 215 11.33 -8.5 23.87
4220 216 11.33 -7 25.37
4230 217 11.33 -5.5 23.87
4240 CIRC 214 217 1.5
4250 CIRC 217 216 1.5
4260 CIRC 216 215 1.5
4270 CIRC 215 214 1.5
4280 BLOCK +Z HO24 .15 1.67
4290 1 1
4300 4 214 215 216 217
4310 POINTS 4
4320 218 11.33 -7 26.87
4330 219 11.33 -8 27.87
4340 220 11.33 -7 28.87
4350 221 11.33 -6 27.87
4360 CIRC 218 221 1
4370 CIRC 221 220 1
4380 CIRC 220 219 1
4390 CIRC 219 218 1
4400 BLOCK +Z HO25 .15 1.67
4410 1 1
4420 4 218 219 220 221
4430 XZ
4440 POINTS 4
4450 222 0.5 -16.83 32.27
4460 223 0.5 -16.83 42.27
4470 224 13 -16.83 42.27
4480 225 13 -16.83 32.27
4490 BLOCK -Z TGTW .15 14.17
4500 1 1
4510 4 222 223 224 225
4520 POINTS 8
4530 226 1.67 -15.67 32.27
4540 227 11.83 -15.67 32.27
4550 228 11.83 -15.67 41.52
4560 229 6.67 -15.67 41.52
4570 230 6.67 -15.67 39.77
4580 231 5.67 -15.67 39.77
4590 232 5.67 -15.67 42.27
4600 233 1.67 -15.67 42.27
4610 BLOCK +Z HO26 .15 11.83
4620 1 1
4630 8 226 227 228 229 230 231 232 233
4640 YZ
4650 POINTS 8
4660 234 6.67 -8.25 40.74
4670 235 6.67 -8.25 41.52
4680 236 6.67 -9.25 41.52
4690 237 6.67 -9.25 40.74
4700 238 6.67 -10.25 40.74
4710 239 6.67 -10.25 41.52
4720 240 6.67 -11.25 41.52
4730 241 6.67 -11.25 40.74
4740 BLOCK -Z DB12 .15 5.16
4750 1 1
4760 4 234 235 236 237
4770 BLOCK -Z DB13 .15 5.16
4780 1 1
4790 4 238 239 240 241
4800 XY
4810 POINTS 6
4820 242 13 -16.83 32.27
4830 243 52.33 -16.83 32.27
4840 244 52.33 2 32.27
4850 245 11.96 2 32.27
4860 246 11.96 -2.67 32.27
4870 247 13 -2.67 32.27
4880 BLOCK -Z BRWA .12 11.10
4890 1 1
4900 6 242 243 244 245 246 247
4910 POINTS 4
4920 248 13 -15.79 32.27
4930 249 51.29 -15.79 32.27
4940 250 51.29 0.96 32.27
4950 251 13 0.96 32.27
4960 BLOCK +Z HO27 .12 11.10
4970 1 1
4980 4 251 250 249 248
4990 POINTS 4
5000 252 12.67 -16.12 43.38
5010 253 51.62 -16.12 43.38
5020 254 51.62 1.29 43.38
5030 255 12.67 1.29 43.38
5040 BLOCK -Z ROOF .13 0.33
5050 1 1
5060 4 252 253 254 255
5070 POINTS 8
5080 256 11.96 -16.83 43.38
5090 257 52.33 -16.83 43.38
5100 258 52.33 2 43.38
5110 259 11.96 2 43.38
5120 260 12.67 -16.12 43.38
5130 261 51.62 -16.12 43.38
5140 262 51.62 1.29 43.38
5150 263 12.67 1.29 43.38
5160 BLOCK -Z BRP1 .12 1.75
5170 1 1
5180 4 256 257 258 259
5190 BLOCK +Z HO28 .12 1.75
5200 1 1
5210 4 263 262 261 260
5220 POINTS 4
5230 264 11.96 -16.83 42.28
5240 265 13 -16.83 42.28
5250 266 13 -2.67 42.28
5260 267 11.96 -2.67 42.28
5270 BLOCK -Z BRP2 .12 1.10
5280 1 1
5290 4 264 265 266 267
5300 XZ
5310 POINTS 4
5320 268 28.92 -16.83 32.27
5330 269 34.92 -16.83 32.27
5340 270 34.92 -16.83 39.27
5350 271 28.92 -16.83 39.27
5360 BLOCK +Z HO29 .12 1.04
5370 1 1
5380 4 268 269 270 271
5390 POINTS 8
5400 272 19 -16.83 33.4
5410 273 23 -16.83 33.4
5420 274 23 -16.83 37.4
5430 275 19 -16.83 37.4
5440 276 41.33 -16.83 33.4
5450 277 45.33 -16.83 33.4
5460 278 45.33 -16.83 37.4
5470 279 41.33 -16.83 37.4
5480 BLOCK +Z HO30 .12 1.04
5490 1 1
5500 4 272 273 274 275
5510 BLOCK +Z HO31 .12 1.04
5520 1 1
5530 4 276 277 278 279
5540 POINTS 4
5550 280 26.42 0.96 33.4
5560 281 30.42 0.96 33.4
5570 282 30.42 0.96 37.4
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5580 283 26.42 0.96 37.4
5590 BLOCK +Z HO32 .12 1.04
5600 1 1
5610 4 280 281 282 283
5620 POINTS 4
5630 284 38.33 0.96 32.27
5640 285 41.33 0.96 32.27
5650 286 41.33 0.96 39.27
5660 287 38.33 0.96 39.27
5670 BLOCK +Z HO33 .12 1.04
5680 1 1
5690 4 284 285 286 287
5700 YZ
5710 POINTS 4
5720 288 11.96 0.17 33.4
5730 289 11.96 -1.83 33.4
5740 290 11.96 -1.83 37.4
5750 291 11.96 0.17 37.4
5760 BLOCK +Z HO34 .12 1.04
5770 1 1
5780 4 288 289 290 291
5790 POINTS 4
5800 292 51.29 -5.42 33.4
5810 293 51.29 -9.42 33.4
5820 294 51.29 -9.42 37.4
5830 295 51.29 -5.42 37.4
5840 BLOCK +Z HO35 .12 1.04
5850 1 1
5860 4 292 293 294 295
5870 XY
5880 POINTS 8
5890 296 13 -17 28.44
5900 297 13 -19.5 28.44
5910 298 24.83 -19.5 28.44
5920 299 24.83 -17 28.44
5930 300 39.5 -17 28.44
5940 301 39.5 -19.5 28.44
5950 302 51.33 -19.5 28.44
5960 303 51.33 -17 28.44
5970 BLOCK -Z ARW1 .15 2.67
5980 1 1
5990 4 296 297 298 299
6000 BLOCK -Z ARW2 .15 2.67
6010 1 1
6020 4 300 301 302 303
6030 POINTS 16
6040 304 13.5 -17 28.44
6050 305 18.5 -17 28.44
6060 306 18.5 -19 28.44
6070 307 13.5 -19 28.44
6080 308 19.33 -17 28.44
6090 309 24.33 -17 28.44
6100 310 24.33 -19 28.44
6110 311 19.33 -19 28.44
6120 312 40 -17 28.44
6130 313 45 -17 28.44
6140 314 45 -19 28.44
6150 315 40 -19 28.44
6160 316 45.83 -17 28.44
6170 317 50.83 -17 28.44
6180 318 50.83 -19 28.44
6190 319 45.83 -19 28.44
6200 BLOCK +Z HO36 .15 2.67
6210 1 1
6220 4 304 305 306 307
6230 BLOCK +Z HO37 .15 2.67
6240 1 1
6250 4 308 309 310 311
6260 BLOCK +Z HO38 .15 2.67
6270 1 1
6280 4 312 313 314 315
6290 BLOCK +Z HO39 .15 2.67
6300 1 1
6310 4 316 317 318 319
6320 XZ
6330 POINTS 16
6340 320 14 -17 31.6
6350 321 14 -17 29.6
6360 322 18 -17 29.6
6370 323 18 -17 31.6
6380 324 19.83 -17 31.6
6390 325 19.83 -17 29.6
6400 326 23.83 -17 29.6
6410 327 23.83 -17 31.6
6420 328 40.5 -17 31.6
6430 329 40.5 -17 29.6
6440 330 44.5 -17 29.6
6450 331 44.5 -17 31.6
6460 332 46.33 -17 31.6
6470 333 46.33 -17 29.6
6480 334 50.33 -17 29.6
6490 335 50.33 -17 31.6
6500 BLOCK +Z HO40 .15 1.67
6510 1 1
6520 4 320 321 322 323
6530 BLOCK +Z HO41 .15 1.67
6540 1 1
6550 4 324 325 326 327
6560 BLOCK +Z HO42 .15 1.67
6570 1 1
6580 4 328 329 330 331
6590 BLOCK +Z HO43 .15 1.67
6600 1 1
6610 4 332 333 334 335
6620 XZ
6630 POINTS 4
6640 336 0.5 -16.83 42.27
6650 337 0.5 -16.83 47.07
6660 338 11.96 -16.83 47.07
6670 339 11.96 -16.83 42.27
6680 BLOCK -Z GTWE .15 14.17
6690 1 1
6700 4 336 337 338 339
6710 POINTS 8
6720 340 1.67 -15.67 42.27
6730 341 10.79 -15.67 42.27
6740 342 10.79 -15.67 46.32
6750 343 6.67 -15.67 46.32
6760 344 6.67 -15.67 44.57
6770 345 5.67 -15.67 44.57
6780 346 5.67 -15.67 47.07
6790 347 1.67 -15.67 47.07
6800 BLOCK +Z HO44 .15 11.83
6810 1 1
6820 8 340 341 342 343 344 345 346 347
6830 YZ
6840 POINTS 8
6850 348 6.67 -8.25 45.57
6860 349 6.67 -8.25 46.32
6870 350 6.67 -9.25 46.32
6880 351 6.67 -9.25 45.57
6890 352 6.67 -10.25 45.57
6900 353 6.67 -10.25 46.32
6910 354 6.67 -11.25 46.32
6920 355 6.67 -11.25 45.57
6930 BLOCK -Z DB14 .15 4.12
6940 1 1
6950 4 348 349 350 351
```
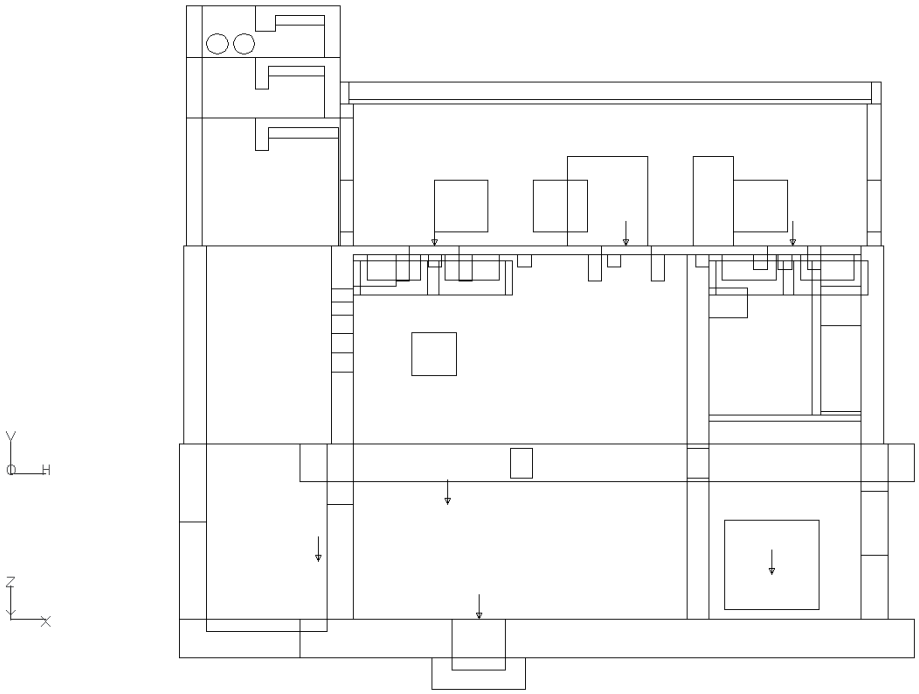
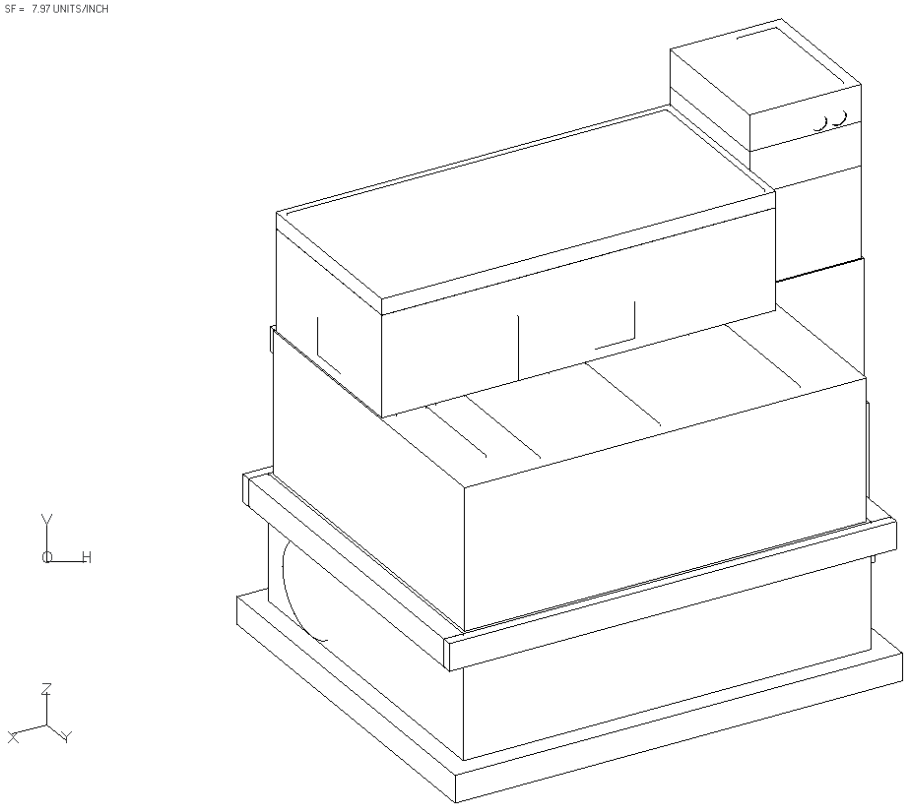

```
6960 BLOCK -Z DB15 .15 4.12
6970 1 1
6980 4 352 353 354 355
7890 PTLD W4 44.17 -0.67 6.5 0 0 -8.5
7900 PTLD W5 10.33 -9.75 7.5 0 0 -10.5
7910 PTLD W6 19 6.58 32.27 0 0 -19.9
7920 PTLD W7 45.75 10.33 32.27 0 0 -9.4
7930 PTLD W8 19 -4.17 32.27 0 0 -2.5
7940 PTLD W9 45.75 -7 32.27 0 0 -3.0
7950 PTLD W10 22.33 -0.17 3 0 0 -4.8
7960 PTLD W11 20 -1 12 0 0 -13.6
7970 PTLD W12 33.33 10.33 32.27 0 0 -17.7
7980 PTLD W13 33.33 4.33 32.27 0 0 -17.7
20000 BACKGROUND BLACK
```

TPSF1.TXT input file:

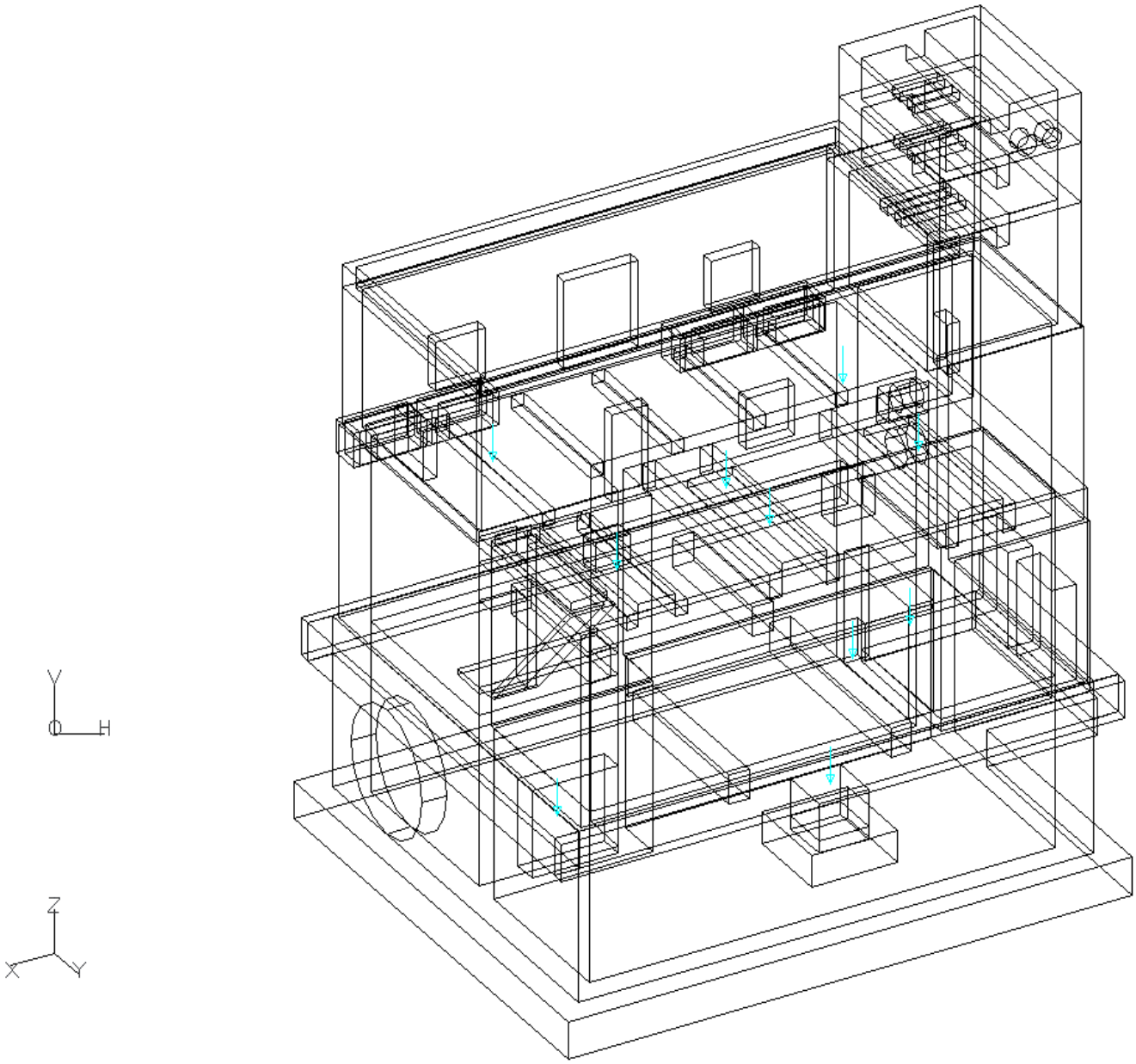
```
6990 XZ
7000 POINTS 4
7010 356 0.5 -16.83 47.07
7020 357 0.5 -16.83 51.07
7030 358 11.96 -16.83 51.07
7040 359 11.96 -16.83 47.07
7050 BLOCK -Z GWE2 .15 14.17
7060 1 1
7070 4 356 357 358 359
7080 POINTS 8
7090 360 1.67 -15.67 47.07
7100 361 10.79 -15.67 47.07
7110 362 10.79 -15.67 50.32
7120 363 7.17 -15.67 50.32
7130 364 7.17 -15.67 49.07
7140 365 5.67 -15.67 49.07
7150 366 5.67 -15.67 51.07
7160 367 1.67 -15.67 51.07
7170 BLOCK +Z HO45 .15 11.83
7180 1 1
7190 8 360 361 362 363 364 365 366 367
7200 YZ
7202 POINTS 8
7210 368 7.17 -8.25 49.57
7220 369 7.17 -8.25 50.32
7230 370 7.17 -9.25 50.32
7240 371 7.17 -9.25 49.57
7250 372 7.17 -10.25 49.57
7260 373 7.17 -10.25 50.32
7270 374 7.17 -11.25 50.32
7280 375 7.17 -11.25 49.57
7290 BLOCK -Z DB16 .15 3.63
7300 1 1
7310 4 368 369 370 371
7320 BLOCK -Z DB17 .15 3.63
7330 1 1
7340 4 372 373 374 375
7350 XZ
7360 POINTS 4
7370 376 2.83 -3.84 47.32
7380 377 3.62 -3.84 48.11
7390 378 2.83 -3.84 48.90
7400 379 2.04 -3.84 48.11
7410 CIRC 376 379 0.7917
7420 CIRC 379 378 0.7917
7430 CIRC 378 377 0.7917
7440 CIRC 377 376 0.7917
7450 BLOCK +Z HO46 .15 1.17
7460 1 1
7470 4 376 377 378 379
7480 POINTS 4
7490 380 4.83 -3.84 47.32
7500 381 5.62 -3.84 48.11
7510 382 4.83 -3.84 48.90
7520 383 4.04 -3.84 48.11
7530 CIRC 380 383 0.7917
7540 CIRC 383 382 0.7917
7550 CIRC 382 381 0.7917
7560 CIRC 381 380 0.7917
7570 BLOCK +Z HO47 .15 1.17
7580 1 1
7590 4 380 381 382 383
20000 BACKGROUND BLACK
```



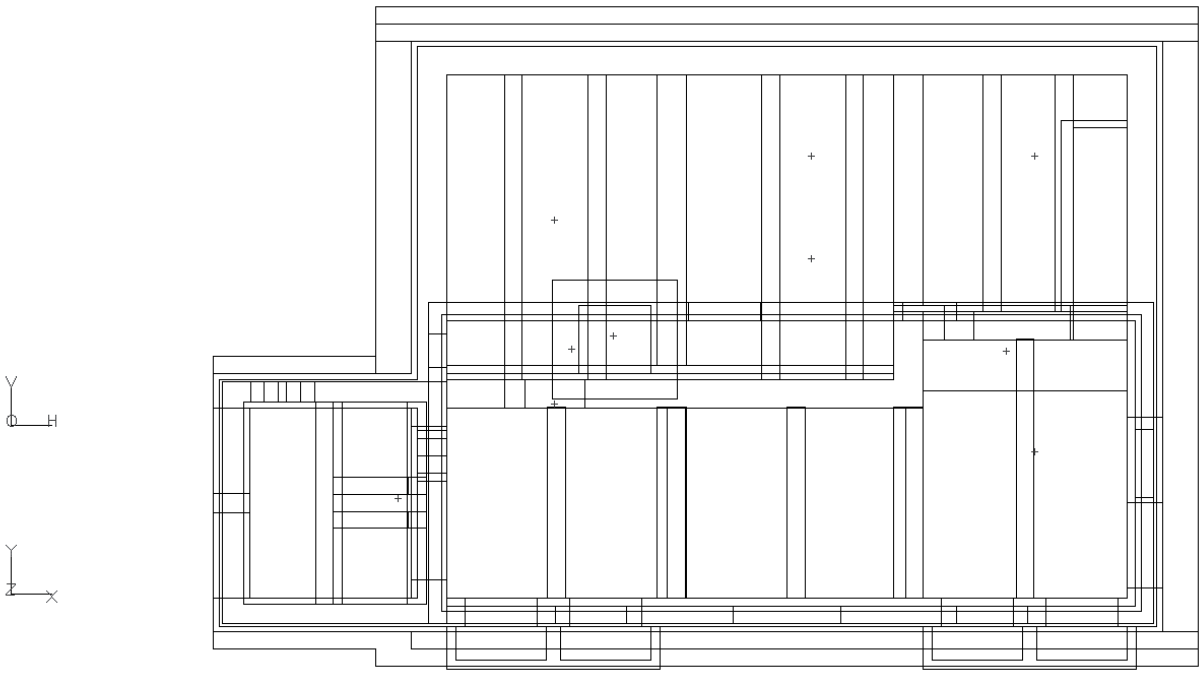
SF = 7.97 UNITS/INCH



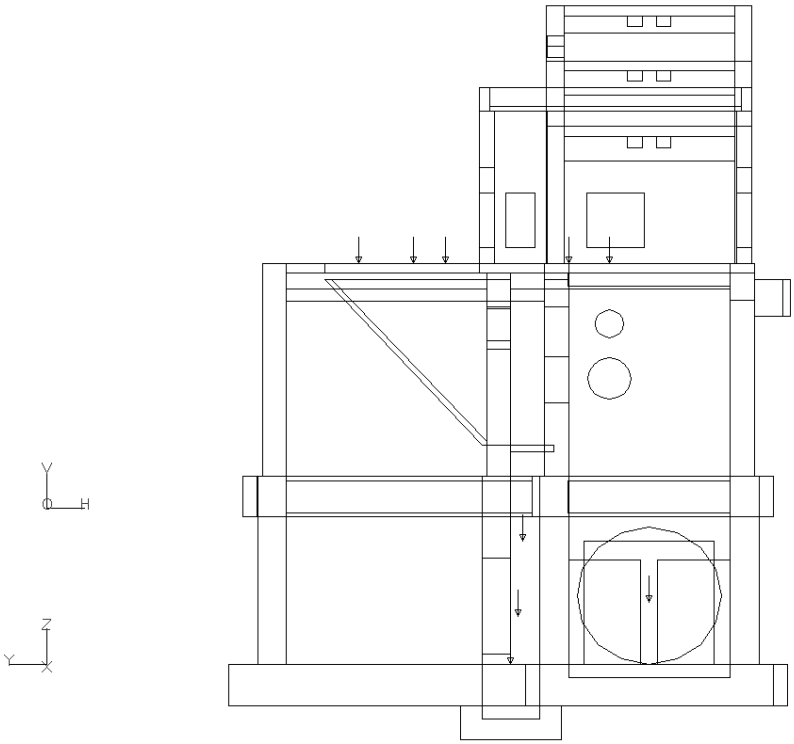
SF = 10.69 UNITS/INCH



SF = 10.69 UNITS/INCH



SF = 6.92 UNITS/INCH



SF = 7.97 UNITS/INCH

Loading Condition definition file TPSLCA.TXT:

```
100 XZ
110 POINTS 6
120 514 54.83 -19.33 3
130 515 52.83 -19.33 3
140 516 52.83 -19.33 16.75
150 517 52.5 -19.33 16.75
151 614 52.5 -19.33 32.27
152 615 54.83 -19.33 32.27
160 BLOCK -Z W1A 0.110 38.8
170 1 1
180 6 514 515 516 517 614 615
200 POINTS 4
210 616 52.83 -17.33 13.75
220 617 52.83 -17.33 16.75
230 618 54.83 -17.33 16.75
240 619 54.83 -17.33 13.75
250 BLOCK +Z W1AV .110 34.67
260 1 1
270 4 616 617 618 619
280 YZ
290 POINTS 6
300 518 9 -19.33 3
310 519 9 -17.33 3
320 520 9 -17.33 16.75
330 521 9 -17.00 16.75
332 522 9 -17.00 32.27
333 523 9 -19.33 32.27
340 BLOCK -Z W1B 0.110 43.67
350 1 1
360 6 518 519 520 521 522 523
370 POINTS 4
380 620 11 -17.33 13.75
390 621 11 -17.33 16.75
400 622 11 -18.33 16.75
410 623 11 -18.33 13.75
420 BLOCK +Z W1BV .110 43.83
430 1 1
440 4 620 621 622 623
490 POINTS 6
500 624 9 19.33 3
510 625 9 17.33 3
520 626 9 17.33 16.75
530 627 9 17.00 16.75
532 628 9 17.00 32.27
533 629 9 19.33 32.27
540 BLOCK -Z W1C 0.110 43.67
550 1 1
560 6 624 625 626 627 628 629
570 POINTS 4
580 630 9 18.33 13.75
590 631 9 18.33 16.75
600 632 9 17.33 16.75
610 633 9 17.33 13.75
620 BLOCK +Z W1CV .110 45.83
630 1 1
640 4 630 631 632 633
645 XZ
650 POINTS 4
660 526 0 -18.33 3
670 527 0 -18.33 32.27
680 528 9 -18.33 32.27
690 529 9 -18.33 3
700 BLOCK -Z W1D 0.110 1.17
710 1 1
720 4 526 527 528 529
730 POINTS 4
740 530 0 -2.33 3
750 531 0 -2.33 32.27
760 532 9 -2.33 32.27
770 533 9 -2.33 3
780 BLOCK -Z W1E 0.110 1.17
790 1 1
800 4 530 531 532 533
810 XZ
820 POINTS 4
830 634 9 -2.17 3
840 635 9 -2.17 13.75
850 636 11.00 -2.17 13.75
860 637 11.00 -2.17 3
870 BLOCK -Z W1F 0.110 19.5
880 1 1
890 4 634 635 636 637
900 POINTS 4
910 638 9 -2.5 16.75
920 639 9 -2.5 32.27
930 640 11.33 -2.5 32.27
940 641 11.33 -2.5 16.75
950 BLOCK -Z W1F2 0.110 19.5
960 1 1
970 4 638 639 640 641
990 XY
1000 POINTS 4
1010 538 20.33 -2.17 -1
1020 539 20.33 1.83 -1
1030 540 24.33 1.83 -1
1040 541 24.33 -2.17 -1
1050 BLOCK -Z W14A 0.0625 4
1060 1 1
1070 4 541 540 539 538
2100 XZ
2110 POINTS 4
2120 500 -10 -17.17 0
2130 501 -13.15 -17.17 0
2140 502 -10.49 -17.17 32.27
2150 503 -10 -17.17 32.27
2160 BLOCK +X F1 1 34.34
2170 1 1
2180 4 500 501 502 503
2190 POINTS 3
2200 504 60 -17.17 0
2210 505 60 -17.17 32.27
2220 506 62.02 -17.17 0
2230 BLOCK -X F2 1 34.34
2240 1 1
2250 3 504 505 506
2254 YZ
2255 POINTS 3
2260 904 0 -30 0
2265 905 0 -30 32.27
2270 906 0 -32.02 0
2275 BLOCK +Y F2A 1 52.83
2280 1 1
2285 3 904 905 906
2355 POINTS 3
2360 907 11 30 0
2365 908 11 30 32.27
2370 909 11 32.02 0
2375 BLOCK -Y F2B 1 41.83
2380 1 1
2385 3 907 908 909
20000 BACKGROUND BLACK
```

Loading Condition Input file
TPSANA.TXT

```
100 POINTS 8
110 700 0 -18.33 0
120 701 9 -18.33 0
130 702 9 -19.33 0
140 703 54.83 -19.33 0
150 704 54.83 19.33 0
160 705 9 19.33 0
170 706 9 -1.17 0
180 707 0 -1.17 0
190 BASE
200 8 700 701 702 703 704
210 705 706 707
220 UPLIFT 8
230 700 0
240 701 0
250 702 0
260 703 0
270 704 0
280 705 0
290 706 0
300 707 0
310 PHI 26.6
320 CASE ANA 2
330 897.6 358.52 -4275.76 1499.47 136438.27 1971.86
```

Loading Condition definition file TPSLCB.TXT:

```
100 XZ
110 POINTS 6
120 514 54.83 -19.33 3
130 515 52.83 -19.33 3
140 516 52.83 -19.33 16.75
150 517 52.5 -19.33 16.75
151 614 52.5 -19.33 32.27
152 615 54.83 -19.33 32.27
160 BLOCK -Z W1A 0.116 38.8
170 1 1
180 6 514 515 516 517 614 615
200 POINTS 4
210 616 52.83 -17.33 13.75
220 617 52.83 -17.33 16.75
230 618 54.83 -17.33 16.75
240 619 54.83 -17.33 13.75
250 BLOCK +Z W1AV .116 34.67
260 1 1
270 4 616 617 618 619
280 YZ
290 POINTS 6
300 518 9 -19.33 3
310 519 9 -17.33 3
320 520 9 -17.33 16.75
330 521 9 -17.00 16.75
332 522 9 -17.00 32.27
333 523 9 -19.33 32.27
340 BLOCK -Z W1B 0.116 43.67
350 1 1
360 6 518 519 520 521 522 523
370 POINTS 4
380 620 11 -17.33 13.75
390 621 11 -17.33 16.75
400 622 11 -18.33 16.75
410 623 11 -18.33 13.75
420 BLOCK +Z W1BV .116 43.83
430 1 1
440 4 620 621 622 623
490 POINTS 6
500 624 9 19.33 3
510 625 9 17.33 3
520 626 9 17.33 16.75
530 627 9 17.00 16.75
532 628 9 17.00 32.27
533 629 9 19.33 32.27
540 BLOCK -Z W1C 0.116 43.67
550 1 1
560 6 624 625 626 627 628 629
570 POINTS 4
580 630 9 18.33 13.75
590 631 9 18.33 16.75
600 632 9 17.33 16.75
610 633 9 17.33 13.75
620 BLOCK +Z W1CV .116 45.83
630 1 1
640 4 630 631 632 633
645 XZ
650 POINTS 4
660 526 0 -18.33 3
670 527 0 -18.33 32.27
680 528 9 -18.33 32.27
690 529 9 -18.33 3
700 BLOCK -Z W1D 0.116 1.17
710 1 1
720 4 526 527 528 529
730 POINTS 4
740 530 0 -2.33 3
750 531 0 -2.33 32.27
760 532 9 -2.33 32.27
770 533 9 -2.33 3
780 BLOCK -Z W1E 0.116 1.17
790 1 1
800 4 530 531 532 533
810 XZ
820 POINTS 4
830 634 9 -2.17 3
840 635 9 -2.17 13.75
850 636 11.00 -2.17 13.75
860 637 11.00 -2.17 3
870 BLOCK -Z W1F 0.116 19.5
880 1 1
890 4 634 635 636 637
900 POINTS 4
910 638 9 -2.5 16.75
920 639 9 -2.5 32.27
930 640 11.33 -2.5 32.27
940 641 11.33 -2.5 16.75
950 BLOCK -Z W1F2 0.116 19.5
960 1 1
970 4 638 639 640 641
990 XY
1000 POINTS 4
1010 538 20.33 -2.17 -1
1020 539 20.33 1.83 -1
1030 540 24.33 1.83 -1
1040 541 24.33 -2.17 -1
1050 BLOCK -Z W14A 0.0625 4
1060 1 1
1070 4 541 540 539 538
2100 XZ
2110 POINTS 4
2120 500 -10 -17.17 0
2130 501 -11.53 -17.17 0
2140 502 -10.24 -17.17 32.27
2150 503 -10 -17.17 32.27
2160 BLOCK +X F1 1 34.34
2170 1 1
2180 4 500 501 502 503
2190 POINTS 3
2200 504 60 -17.17 0
2210 505 60 -17.17 32.27
2220 506 60.98 -17.17 0
2230 BLOCK -X F2 1 34.34
2240 1 1
2250 3 504 505 506
2254 YZ
2255 POINTS 3
2260 904 0 -30 0
2265 905 0 -30 32.27
2270 906 0 -30.98 0
2275 BLOCK +Y F2A 1 52.83
2280 1 1
2285 3 904 905 906
2355 POINTS 3
2360 907 11 30 0
2365 908 11 30 32.27
2370 909 11 30.98 0
2375 BLOCK -Y F2B 1 41.83
2380 1 1
2385 3 907 908 909
2400 XZ
2410 POINTS 4
2420 534 2 -15.33 2
2430 535 2 -15.33 50.32
2440 536 11.17 -15.33 50.32
```

```
2450 537 11.17 -15.33 2
2460 BLOCK -Z W2 0.0625 11.17
2470 1 1
2480 4 534 535 536 537
2500 XY
2510 POINTS 6
2520 542 13 -15.33 3
2530 543 13 -4.17 3
2540 544 39.5 -4.17 3
2550 545 39.5 -0.17 3
2560 546 50.83 -0.17 3
2570 547 50.83 -15.33 3
2580 BLOCK -Z W14B 0.0625 8
2590 1 1
2600 6 547 546 545 544 543 542
2610 POINTS 6
2620 548 13 -2.17 3
2630 549 13 15.33 3
2640 550 50.83 15.33 3
2650 551 50.83 1.83 3
2670 552 37.83 1.83 3
2680 553 37.83 -2.17 3
2690 BLOCK -Z W14C 0.0625 8
3000 1 1
3010 6 553 552 551 550 549 548
21100 XZ
21260 POINTS 3
21270 507 70 -17.17 0
21280 508 70 -17.17 32.27
21290 509 72.9 -17.17 0
21300 BLOCK -X F3 1 34.34
21310 1 1
21320 3 507 508 509
21330 POINTS 4
21340 510 -20 -17.17 0
21350 511 -22.97 -17.17 0
21360 512 -20.96 -17.17 32.27
21370 513 -20 -17.17 32.27
21380 BLOCK +X F4 1 34.34
21390 1 1
21400 4 510 511 512 513
22000 YZ
22255 POINTS 3
22260 804 0 -60 0
22265 805 0 -60 32.27
22270 806 0 -62.9 0
22275 BLOCK +Y F3A 1 52.83
22280 1 1
22285 3 804 805 806
22355 POINTS 3
22360 807 11 60 0
22365 808 11 60 32.27
22370 809 11 62.9 0
22375 BLOCK -Y F3B 1 41.83
22380 1 1
22385 3 807 808 809
25000 BACKGROUND BLACK
```

Loading Condition Input file TPSANB.TXT

```
100 POINTS 8
110 700 0 -18.33 0
120 701 9 -18.33 0
130 702 9 -19.33 0
140 703 54.83 -19.33 0
150 704 54.83 19.33 0
160 705 9 19.33 0
170 706 9 -1.17 0
180 707 0 -1.17 0
190 BASE
200 8 700 701 702 703 704
210 705 706 707
220 UPLIFT 8
230 700 2.97
240 701 2.96
250 702 2.96
260 703 2.92
270 704 2.92
280 705 2.96
290 706 2.96
300 707 2.97
310 PHI 26.6
320 CASE ANB 2
330 1008.42 688.64 -5179.62 873.67 162861.38
3787.53
```

Loading Condition definition file TPSLCC.TXT:

```
100 XZ
110 POINTS 6
120 514 54.83 -19.33 3
130 515 52.83 -19.33 3
140 516 52.83 -19.33 16.75
150 517 52.5 -19.33 16.75
151 614 52.5 -19.33 32.27
152 615 54.83 -19.33 32.27
160 BLOCK -Z W1A 0.116 38.8
170 1 1
180 6 514 515 516 517 614 615
200 POINTS 4
210 616 52.83 -17.33 13.75
220 617 52.83 -17.33 16.75
230 618 54.83 -17.33 16.75
240 619 54.83 -17.33 13.75
250 BLOCK +Z W1AV .116 34.67
260 1 1
270 4 616 617 618 619
280 YZ
290 POINTS 6
300 518 9 -19.33 3
310 519 9 -17.33 3
320 520 9 -17.33 16.75
330 521 9 -17.00 16.75
332 522 9 -17.00 32.27
333 523 9 -19.33 32.27
340 BLOCK -Z W1B 0.116 43.67
350 1 1
360 6 518 519 520 521 522 523
370 POINTS 4
380 620 11 -17.33 13.75
390 621 11 -17.33 16.75
400 622 11 -18.33 16.75
410 623 11 -18.33 13.75
420 BLOCK +Z W1BV .116 43.83
430 1 1
440 4 620 621 622 623
490 POINTS 6
500 624 9 19.33 3
510 625 9 17.33 3
520 626 9 17.33 16.75
530 627 9 17.00 16.75
532 628 9 17.00 32.27
533 629 9 19.33 32.27
540 BLOCK -Z W1C 0.116 43.67
550 1 1
560 6 624 625 626 627 628 629
570 POINTS 4
580 630 9 18.33 13.75
590 631 9 18.33 16.75
600 632 9 17.33 16.75
610 633 9 17.33 13.75
620 BLOCK +Z W1CV .116 45.83
630 1 1
640 4 630 631 632 633
645 XZ
650 POINTS 4
660 526 0 -18.33 3
670 527 0 -18.33 32.27
680 528 9 -18.33 32.27
690 529 9 -18.33 3
700 BLOCK -Z W1D 0.116 1.17
710 1 1
720 4 526 527 528 529
730 POINTS 4
740 530 0 -2.33 3
750 531 0 -2.33 32.27
760 532 9 -2.33 32.27
770 533 9 -2.33 3
780 BLOCK -Z W1E 0.116 1.17
790 1 1
800 4 530 531 532 533
810 XZ
820 POINTS 4
830 634 9 -2.17 3
840 635 9 -2.17 13.75
850 636 11.00 -2.17 13.75
860 637 11.00 -2.17 3
870 BLOCK -Z W1F 0.116 19.5
880 1 1
890 4 634 635 636 637
900 POINTS 4
910 638 9 -2.5 16.75
920 639 9 -2.5 32.27
930 640 11.33 -2.5 32.27
940 641 11.33 -2.5 16.75
950 BLOCK -Z W1F2 0.116 19.5
960 1 1
970 4 638 639 640 641
990 XY
1000 POINTS 4
1010 538 20.33 -2.17 -1
1020 539 20.33 1.83 -1
1030 540 24.33 1.83 -1
1040 541 24.33 -2.17 -1
1050 BLOCK -Z W14A 0.0625 4
1060 1 1
1070 4 541 540 539 538
2100 XZ
2110 POINTS 4
2120 500 -10 -17.17 0
2130 501 -11.53 -17.17 0
2140 502 -10.24 -17.17 32.27
2150 503 -10 -17.17 32.27
2160 BLOCK +X F1 1 34.34
2170 1 1
2180 4 500 501 502 503
2190 POINTS 3
2200 504 60 -17.17 0
2210 505 60 -17.17 32.27
2220 506 60.98 -17.17 0
2230 BLOCK -X F2 1 34.34
2240 1 1
2250 3 504 505 506
2254 YZ
2255 POINTS 3
2260 904 0 -30 0
2265 905 0 -30 32.27
2270 906 0 -30.98 0
2275 BLOCK +Y F2A 1 52.83
2280 1 1
2285 3 904 905 906
2355 POINTS 3
2360 907 11 30 0
2365 908 11 30 32.27
2370 909 11 30.98 0
2375 BLOCK -Y F2B 1 41.83
2380 1 1
2385 3 907 908 909
2400 XZ
2410 POINTS 4
2420 534 2 -15.33 2
2430 535 2 -15.33 50.32
2440 536 11.17 -15.33 50.32
```

```
2450 537 11.17 -15.33 2
2460 BLOCK -Z W2 0.0625 11.17
2470 1 1
2480 4 534 535 536 537
2500 XY
2510 POINTS 6
2520 542 13 -15.33 3
2530 543 13 -4.17 3
2540 544 39.5 -4.17 3
2550 545 39.5 -0.17 3
2560 546 50.83 -0.17 3
2570 547 50.83 -15.33 3
2580 BLOCK -Z W14B 0.0625 8
2590 1 1
2600 6 547 546 545 544 543 542
2610 POINTS 6
2620 548 13 -2.17 3
2630 549 13 15.33 3
2640 550 50.83 15.33 3
2650 551 50.83 1.83 3
2670 552 37.83 1.83 3
2680 553 37.83 -2.17 3
2690 BLOCK -Z W14C 0.0625 8
3000 1 1
3010 6 553 552 551 550 549 548
21100 XZ
21260 POINTS 3
21270 507 70 -17.17 0
21280 508 70 -17.17 32.27
21290 509 72.46 -17.17 0
21300 BLOCK -X F3 1 34.34
21310 1 1
21320 3 507 508 509
21330 POINTS 4
21340 510 -20 -17.17 0
21350 511 -22.54 -17.17 0
21360 512 -20.52 -17.17 32.27
21370 513 -20 -17.17 32.27
21380 BLOCK +X F4 1 34.34
21390 1 1
21400 4 510 511 512 513
22000 YZ
22255 POINTS 3
22260 804 0 -60 0
22265 805 0 -60 32.27
22270 806 0 -62.46 0
22275 BLOCK +Y F3A 1 52.83
22280 1 1
22285 3 804 805 806
22355 POINTS 3
22360 807 11 60 0
22365 808 11 60 32.27
22370 809 11 62.46 0
22375 BLOCK -Y F3B 1 41.83
22380 1 1
22385 3 807 808 809
25000 BACKGROUND BLACK
```

Loading Condition Input file TPSANC.TXT

```
100 POINTS 8
11100 POINTS 8
110 700 0 -18.33 0
120 701 9 -18.33 0
130 702 9 -19.33 0
140 703 54.83 -19.33 0
150 704 54.83 19.33 0
160 705 9 19.33 0
170 706 9 -1.17 0
180 707 0 -1.17 0
190 BASE
200 8 700 701 702 703 704
210 705 706 707
220 UPLIFT 8
230 700 2.54
240 701 2.53
250 702 2.53
260 703 2.48
270 704 2.48
280 705 2.53
290 706 2.53
300 707 2.54
310 PHI 26.6
320 CASE ANC 2
330 770.17 610.55 -5179.62 1713.68 157676.11
3358.01
```

Loading Condition definition file TPSLCD.TXT:

```
100 XZ
110 POINTS 6
120 514 54.83 -19.33 3
130 515 52.83 -19.33 3
140 516 52.83 -19.33 16.75
150 517 52.5 -19.33 16.75
151 614 52.5 -19.33 32.27
152 615 54.83 -19.33 32.27
160 BLOCK -Z W1A 0.116 38.8
170 1 1
180 6 514 515 516 517 614 615
200 POINTS 4
210 616 52.83 -17.33 13.75
220 617 52.83 -17.33 16.75
230 618 54.83 -17.33 16.75
240 619 54.83 -17.33 13.75
250 BLOCK +Z W1AV .116 34.67
260 1 1
270 4 616 617 618 619
280 YZ
290 POINTS 6
300 518 9 -19.33 3
310 519 9 -17.33 3
320 520 9 -17.33 16.75
330 521 9 -17.00 16.75
332 522 9 -17.00 32.27
333 523 9 -19.33 32.27
340 BLOCK -Z W1B 0.116 43.67
350 1 1
360 6 518 519 520 521 522 523
370 POINTS 4
380 620 11 -17.33 13.75
390 621 11 -17.33 16.75
400 622 11 -18.33 16.75
410 623 11 -18.33 13.75
420 BLOCK +Z W1BV .116 43.83
430 1 1
440 4 620 621 622 623
490 POINTS 6
500 624 9 19.33 3
510 625 9 17.33 3
520 626 9 17.33 16.75
530 627 9 17.00 16.75
532 628 9 17.00 32.27
533 629 9 19.33 32.27
540 BLOCK -Z W1C 0.116 43.67
550 1 1
560 6 624 625 626 627 628 629
570 POINTS 4
580 630 9 18.33 13.75
590 631 9 18.33 16.75
600 632 9 17.33 16.75
610 633 9 17.33 13.75
620 BLOCK +Z W1CV .116 45.83
630 1 1
640 4 630 631 632 633
645 XZ
650 POINTS 4
660 526 0 -18.33 3
670 527 0 -18.33 32.27
680 528 9 -18.33 32.27
690 529 9 -18.33 3
700 BLOCK -Z W1D 0.116 1.17
710 1 1
720 4 526 527 528 529
730 POINTS 4
740 530 0 -2.33 3
750 531 0 -2.33 32.27
760 532 9 -2.33 32.27
770 533 9 -2.33 3
780 BLOCK -Z W1E 0.116 1.17
790 1 1
800 4 530 531 532 533
810 XZ
820 POINTS 4
830 634 9 -2.17 3
840 635 9 -2.17 13.75
850 636 11.00 -2.17 13.75
860 637 11.00 -2.17 3
870 BLOCK -Z W1F 0.116 19.5
880 1 1
890 4 634 635 636 637
900 POINTS 4
910 638 9 -2.5 16.75
920 639 9 -2.5 32.27
930 640 11.33 -2.5 32.27
940 641 11.33 -2.5 16.75
950 BLOCK -Z W1F2 0.116 19.5
960 1 1
970 4 638 639 640 641
990 XY
1000 POINTS 4
1010 538 20.33 -2.17 -1
1020 539 20.33 1.83 -1
1030 540 24.33 1.83 -1
1040 541 24.33 -2.17 -1
1050 BLOCK -Z W14A 0.0625 4
1060 1 1
1070 4 541 540 539 538
2100 XZ
2110 POINTS 4
2120 500 -10 -17.17 0
2130 501 -11.53 -17.17 0
2140 502 -10.24 -17.17 32.27
2150 503 -10 -17.17 32.27
2160 BLOCK +X F1 1 34.34
2170 1 1
2180 4 500 501 502 503
2190 POINTS 3
2200 504 60 -17.17 0
2210 505 60 -17.17 32.27
2220 506 60.98 -17.17 0
2230 BLOCK -X F2 1 34.34
2240 1 1
2250 3 504 505 506
2254 YZ
2255 POINTS 3
2260 904 0 -30 0
2265 905 0 -30 32.27
2270 906 0 -30.98 0
2275 BLOCK +Y F2A 1 52.83
2280 1 1
2285 3 904 905 906
2355 POINTS 3
2360 907 11 30 0
2365 908 11 30 32.27
2370 909 11 30.98 0
2375 BLOCK -Y F2B 1 41.83
2380 1 1
2385 3 907 908 909
2400 XZ
2410 POINTS 4
```

```
2420 534 2 -15.33 2
2430 535 2 -15.33 50.32
2440 536 11.17 -15.33 50.32
2450 537 11.17 -15.33 2
2460 BLOCK -Z W2 0.0625 11.17
2470 1 1
2480 4 534 535 536 537
2500 XY
2510 POINTS 6
2520 542 13 -15.33 3
2530 543 13 -4.17 3
2540 544 39.5 -4.17 3
2550 545 39.5 -0.17 3
2560 546 50.83 -0.17 3
2570 547 50.83 -15.33 3
2580 BLOCK -Z W14B 0.0625 8
2590 1 1
2600 6 547 546 545 544 543 542
2610 POINTS 6
2620 548 13 -2.17 3
2630 549 13 15.33 3
2640 550 50.83 15.33 3
2650 551 50.83 1.83 3
2670 552 37.83 1.83 3
2680 553 37.83 -2.17 3
2690 BLOCK -Z W14C 0.0625 8
3000 1 1
3010 6 553 552 551 550 549 548
21100 XZ
21260 POINTS 3
21270 507 70 -17.17 0
21280 508 70 -17.17 32.27
21290 509 72.40 -17.17 0
21300 BLOCK -X F3 1 34.34
21310 1 1
21320 3 507 508 509
21330 POINTS 4
21340 510 -20 -17.17 0
21350 511 -22.47 -17.17 0
21360 512 -20.46 -17.17 32.27
21370 513 -20 -17.17 32.27
21380 BLOCK +X F4 1 34.34
21390 1 1
21400 4 510 511 512 513
22000 YZ
22255 POINTS 3
22260 804 0 -60 0
22265 805 0 -60 32.27
22270 806 0 -62.40 0
22275 BLOCK +Y F3A 1 52.83
22280 1 1
22285 3 804 805 806
22355 POINTS 3
22360 807 11 60 0
22365 808 11 60 32.27
22370 809 11 62.40 0
22375 BLOCK -Y F3B 1 41.83
22380 1 1
22385 3 807 808 809
25000 BACKGROUND BLACK
```

Loading Condition Input file TPSAND.TXT

```
100 POINTS 8
110 700 0 -18.33 0
120 701 9 -18.33 0
130 702 9 -19.33 0
140 703 54.83 -19.33 0
150 704 54.83 19.33 0
160 705 9 19.33 0
170 706 9 -1.17 0
180 707 0 -1.17 0
190 BASE
200 8 700 701 702 703 704
210 705 706 707
220 UPLIFT 8
230 700 2.47
240 701 2.46
250 702 2.46
260 703 2.42
270 704 2.42
280 705 2.46
290 706 2.46
300 707 2.47
310 PHI 26.6
320 CASE AND 2
330 731.38 599.90 -5179.62 1828.24 156901.31
3299.44
```

Loading Condition definition file TPSLCE.TXT:

```
100 XZ
110 POINTS 6
120 514 54.83 -19.33 3
130 515 52.83 -19.33 3
140 516 52.83 -19.33 16.75
150 517 52.5 -19.33 16.75
151 614 52.5 -19.33 32.27
152 615 54.83 -19.33 32.27
160 BLOCK -Z W1A 0.116 38.8
170 1 1
180 6 514 515 516 517 614 615
200 POINTS 4
210 616 52.83 -17.33 13.75
220 617 52.83 -17.33 16.75
230 618 54.83 -17.33 16.75
240 619 54.83 -17.33 13.75
250 BLOCK +Z W1AV .116 34.67
260 1 1
270 4 616 617 618 619
280 YZ
290 POINTS 6
300 518 9 -19.33 3
310 519 9 -17.33 3
320 520 9 -17.33 16.75
330 521 9 -17.00 16.75
332 522 9 -17.00 32.27
333 523 9 -19.33 32.27
340 BLOCK -Z W1B 0.116 43.67
350 1 1
360 6 518 519 520 521 522 523
370 POINTS 4
380 620 11 -17.33 13.75
390 621 11 -17.33 16.75
400 622 11 -18.33 16.75
410 623 11 -18.33 13.75
420 BLOCK +Z W1BV .116 43.83
430 1 1
440 4 620 621 622 623
490 POINTS 6
500 624 9 19.33 3
510 625 9 17.33 3
520 626 9 17.33 16.75
530 627 9 17.00 16.75
532 628 9 17.00 32.27
533 629 9 19.33 32.27
540 BLOCK -Z W1C 0.116 43.67
550 1 1
560 6 624 625 626 627 628 629
570 POINTS 4
580 630 9 18.33 13.75
590 631 9 18.33 16.75
600 632 9 17.33 16.75
610 633 9 17.33 13.75
620 BLOCK +Z W1CV .116 45.83
630 1 1
640 4 630 631 632 633
645 XZ
650 POINTS 4
660 526 0 -18.33 3
670 527 0 -18.33 32.27
680 528 9 -18.33 32.27
690 529 9 -18.33 3
700 BLOCK -Z W1D 0.116 1.17
710 1 1
720 4 526 527 528 529
730 POINTS 4
740 530 0 -2.33 3
750 531 0 -2.33 32.27
760 532 9 -2.33 32.27
770 533 9 -2.33 3
780 BLOCK -Z W1E 0.116 1.17
790 1 1
800 4 530 531 532 533
810 XZ
820 POINTS 4
830 634 9 -2.17 3
840 635 9 -2.17 13.75
850 636 11.00 -2.17 13.75
860 637 11.00 -2.17 3
870 BLOCK -Z W1F 0.116 19.5
880 1 1
890 4 634 635 636 637
900 POINTS 4
910 638 9 -2.5 16.75
920 639 9 -2.5 32.27
930 640 11.33 -2.5 32.27
940 641 11.33 -2.5 16.75
950 BLOCK -Z W1F2 0.116 19.5
960 1 1
970 4 638 639 640 641
990 XY
1000 POINTS 4
1010 538 20.33 -2.17 -1
1020 539 20.33 1.83 -1
1030 540 24.33 1.83 -1
1040 541 24.33 -2.17 -1
1050 BLOCK -Z W14A 0.0625 4
1060 1 1
1070 4 541 540 539 538
2100 XZ
2110 POINTS 4
2120 500 -10 -17.17 0
2130 501 -11.53 -17.17 0
2140 502 -10.24 -17.17 32.27
2150 503 -10 -17.17 32.27
2160 BLOCK +X F1 1 34.34
2170 1 1
2180 4 500 501 502 503
2190 POINTS 3
2200 504 60 -17.17 0
2210 505 60 -17.17 32.27
2220 506 60.98 -17.17 0
2230 BLOCK -X F2 1 34.34
2240 1 1
2250 3 504 505 506
2254 YZ
2255 POINTS 3
2260 904 0 -30 0
2265 905 0 -30 32.27
2270 906 0 -30.98 0
2275 BLOCK +Y F2A 1 52.83
2280 1 1
2285 3 904 905 906
2355 POINTS 3
2360 907 11 30 0
2365 908 11 30 32.27
2370 909 11 30.98 0
2375 BLOCK -Y F2B 1 41.83
2380 1 1
2385 3 907 908 909
2400 XZ
2410 POINTS 4
```

```
2420 534 2 -15.33 2
2430 535 2 -15.33 50.32
2440 536 11.17 -15.33 50.32
2450 537 11.17 -15.33 2
2460 BLOCK -Z W2 0.0625 11.17
2470 1 1
2480 4 534 535 536 537
2500 XY
2510 POINTS 6
2520 542 13 -15.33 3
2530 543 13 -4.17 3
2540 544 39.5 -4.17 3
2550 545 39.5 -0.17 3
2560 546 50.83 -0.17 3
2570 547 50.83 -15.33 3
2580 BLOCK -Z W14B 0.0625 8
2590 1 1
2600 6 547 546 545 544 543 542
2610 POINTS 6
2620 548 13 -2.17 3
2630 549 13 15.33 3
2640 550 50.83 15.33 3
2650 551 50.83 1.83 3
2670 552 37.83 1.83 3
2680 553 37.83 -2.17 3
2690 BLOCK -Z W14C 0.0625 8
3000 1 1
3010 6 553 552 551 550 549 548
21100 XZ
21260 POINTS 3
21270 507 70 -17.17 0
21280 508 70 -17.17 32.27
21290 509 72.34 -17.17 0
21300 BLOCK -X F3 1 34.34
21310 1 1
21320 3 507 508 509
21330 POINTS 4
21340 510 -20 -17.17 0
21350 511 -22.41 -17.17 0
21360 512 -20.39 -17.17 32.27
21370 513 -20 -17.17 32.27
21380 BLOCK +X F4 1 34.34
21390 1 1
21400 4 510 511 512 513
22000 YZ
22255 POINTS 3
22260 804 0 -60 0
22265 805 0 -60 32.27
22270 806 0 -62.34 0
22275 BLOCK +Y F3A 1 52.83
22280 1 1
22285 3 804 805 806
22355 POINTS 3
22360 807 11 60 0
22365 808 11 60 32.27
22370 809 11 62.340 0
22375 BLOCK -Y F3B 1 41.83
22380 1 1
22385 3 807 808 809
25000 BACKGROUND BLACK
```

Loading Condition Input file TPSANE.TXT

```
100 POINTS 8
110 700 0 -18.33 0
120 701 9 -18.33 0
130 702 9 -19.33 0
140 703 54.83 -19.33 0
150 704 54.83 19.33 0
160 705 9 19.33 0
170 706 9 -1.17 0
180 707 0 -1.17 0
190 BASE
200 8 700 701 702 703 704
210 705 706 707
220 UPLIFT 8
230 700 2.41
240 701 2.40
250 702 2.40
260 703 2.35
270 704 2.35
280 705 2.40
290 706 2.40
300 707 2.41
310 PHI 26.6
320 CASE ANE 2
330 692.60 589.25 -5179.62 1942.79 156066.92 3240.87
```

Loading Output from 3DSAD for structure
(Existing station with gatewell raise.) TPSF0.TXT + TPSF1.TXT

NAME	FX MX	FY MY	FZ MZ
BASE	0.00	0.00	-859.60
	676.38	25597.94	0.00
HO01	0.00	0.00	15.07
	-146.89	-97.93	0.00
SUBA	0.00	0.00	-18.38
	3.12	410.31	0.00
HO02	0.00	0.00	2.40
	-0.41	-53.59	0.00
LOWA	0.00	0.00	-3334.21
	3353.44	97326.16	0.00
HO03	0.00	0.00	207.16
	-2019.79	-1346.52	0.00
HO04	0.00	0.00	964.22
	-8692.67	-32011.74	0.00
HO05	0.00	0.00	1258.18
	9002.74	-38823.20	0.00
HO06	0.00	0.00	23.56
	-229.73	-1221.22	0.00
HO07	0.00	0.00	14.70
	12.20	-649.30	0.00
HO08	0.00	0.00	24.30
	-236.93	-291.60	0.00
HO09	0.00	0.00	11.45
	-76.34	-11.45	0.00
HO10	0.00	0.00	11.45
	-146.84	-11.45	0.00
SOBT	0.00	0.00	-19.72
	351.67	649.20	0.00
NOBT	0.00	0.00	-20.62
	-367.72	658.20	0.00
WEBT	0.00	0.00	-17.55
	-133.03	175.50	0.00
EABT	0.00	0.00	-31.20
	-0.16	1679.66	0.00
STR1	0.00	0.00	-6.51
	63.42	165.96	0.00
STR2	0.00	0.00	-9.91
	-67.65	252.58	0.00
STR3	0.00	0.00	-6.55
	63.81	253.16	0.00
STR4	0.00	0.00	-7.91
	-67.90	305.97	0.00

NAME	FX MX	FY MY	FZ MZ
CEBT	0.00	0.00	-5.59
	10.73	141.99	0.00
UPWA	0.00	0.00	-3630.00
	3620.33	106165.80	0.00
HO11	0.00	0.00	242.40
	-2363.38	-1615.58	0.00
HO12	0.00	0.00	1041.36
	-9388.08	-34572.68	0.00
HO13	0.00	0.00	1386.64
	9696.25	-42816.55	0.00
DBM1	0.00	0.00	-1.68
	16.33	31.97	0.00
DBM2	0.00	0.00	-1.68
	16.33	43.14	0.00
DBM3	0.00	0.00	-1.68
	16.33	54.32	0.00
DBM4	0.00	0.00	-1.68
	16.33	65.34	0.00
DBM5	0.00	0.00	-2.64
	20.44	119.23	0.00
DBM6	0.00	0.00	-5.56
	-35.69	92.73	0.00
DBM7	0.00	0.00	-5.56
	-35.69	118.66	0.00
DBM8	0.00	0.00	-5.56
	-35.69	172.45	0.00
DBM9	0.00	0.00	-5.56
	-35.69	198.43	0.00
DB10	0.00	0.00	-2.41
	-20.25	104.27	0.00
DB11	0.00	0.00	-2.41
	-20.25	113.90	0.00
HO14	0.00	0.00	4.90
	42.49	-93.16	0.00
HO15	0.00	0.00	4.92
	42.60	-163.89	0.00
HO16	0.00	0.00	4.02
	34.82	-182.18	0.00
HO17	0.00	0.00	3.22
	23.60	-158.69	0.00
STRW	0.00	0.00	-38.24
	-199.85	1873.84	0.00

NAME	FX MX	FY MY	FZ MZ
HO18	0.00	0.00	28.52
	144.96	-1406.75	0.00
PLFM	0.00	0.00	-2.55
	4.26	115.14	0.00
HO19	0.00	0.00	5.26
	3.50	-259.50	0.00
HO20	0.00	0.00	1.50
	1.00	-74.14	0.00
HO21	0.00	0.00	1.66
	1.10	-67.87	0.00
HO22	0.00	0.00	1.59
	-5.30	-23.16	0.00
HO23	0.00	0.00	2.79
	-9.32	-53.10	0.00
HO24	0.00	0.00	1.77
	-12.39	-21.54	0.00
HO25	0.00	0.00	0.79
	-5.51	-9.57	0.00
TGTW	0.00	0.00	-265.69
	2589.12	1793.39	0.00
HO26	0.00	0.00	168.99
	-1648.45	-1126.06	0.00
DB12	0.00	0.00	-0.60
	5.28	5.58	0.00
DB13	0.00	0.00	-0.60
	6.49	5.58	0.00
BRWA	0.00	0.00	-992.93
	7316.75	32303.38	0.00
HO27	0.00	0.00	854.29
	-6334.55	-27461.09	0.00
ROOF	0.00	0.00	-29.09
	215.71	935.14	0.00
BRP1	0.00	0.00	-159.64
	1183.69	5131.47	0.00
HO28	0.00	0.00	142.41
	-1055.93	-4577.61	0.00
BRP2	0.00	0.00	-1.94
	18.95	24.26	0.00
HO29	0.00	0.00	5.24
	-85.49	-167.31	0.00
HO30	0.00	0.00	2.00
	-32.57	-41.93	0.00

```
                         ---------
NAME              FX                FY                FZ
                  MX                MY                MZ


HO31              0.00              0.00              2.00
                 -32.57           -86.52              0.00
HO32              0.00              0.00              2.00
                  2.96           -56.75              0.00
HO33              0.00              0.00              2.62
                  3.88          -104.39              0.00
HO34              0.00              0.00              1.00
                 -0.83           -12.46              0.00
HO35              0.00              0.00              2.00
                 -14.82          -103.45              0.00
ARW1              0.00              0.00            -11.84
                 216.17           224.04              0.00
ARW2              0.00              0.00            -11.84
                 216.17           537.93              0.00
HO36              0.00              0.00              4.01
                 -72.09           -64.08              0.00
HO37              0.00              0.00              4.01
                 -72.09           -87.43              0.00
HO38              0.00              0.00              4.01
                 -72.09          -170.21              0.00
HO39              0.00              0.00              4.01
                 -72.09          -193.56              0.00
HO40              0.00              0.00              2.00
                 -32.39           -32.06              0.00
HO41              0.00              0.00              2.00
                 -32.39           -43.75              0.00
HO42              0.00              0.00              2.00
                 -32.39           -85.17              0.00
HO43              0.00              0.00              2.00
                 -32.39           -96.85              0.00
GTWE              0.00              0.00           -116.92
                1139.38           728.41              0.00
HO44              0.00              0.00             67.76
                -661.01          -408.71              0.00
DB14              0.00              0.00             -0.46
                  4.06              4.05              0.00
DB15              0.00              0.00             -0.46
                  4.98              4.05              0.00
W4                0.00              0.00             -8.50
                  5.70            375.44              0.00
W5                0.00              0.00            -10.50
                 102.38            108.46              0.00
```

```
                    ---------
NAME          FX              FY              FZ
              MX              MY              MZ


W6              0.00            0.00          -19.90
             -130.94          378.10            0.00
W7              0.00            0.00           -9.40
              -97.10          430.05            0.00
W8              0.00            0.00           -2.50
               10.43           47.50            0.00
W9              0.00            0.00           -3.00
               21.00          137.25            0.00
W10             0.00            0.00           -4.80
                0.82          107.18            0.00
W11             0.00            0.00          -13.60
               13.60          272.00            0.00
W12             0.00            0.00          -17.70
             -182.84          589.94            0.00
W13             0.00            0.00          -17.70
              -76.64          589.94            0.00
GWE2            0.00            0.00          -97.43
              949.48          607.01            0.00
HO45            0.00            0.00           54.59
             -532.55         -325.85            0.00
DB16            0.00            0.00           -0.41
                3.57            3.67            0.00
DB17            0.00            0.00           -0.41
                4.39            3.67            0.00
HO46            0.00            0.00            0.34
               -1.12           -0.97            0.00
HO47            0.00            0.00            0.34
               -1.12           -1.66            0.00

TOTAL           0.00            0.00        -3249.41
             5613.55        90949.14            0.00
                    ---------
```

Loading Output from 3DSAD for Load Condition A. TPSLCA.TXT

```
                    ---------
 NAME          FX              FY              FZ
               MX              MY              MZ


 W1A            0.00            0.00         -271.71
             -19.02        14600.58            0.00
 W1AV           0.00            0.00           22.88
               0.11        -1231.75            0.00
 W1B            0.00            0.00         -305.81
            5576.86         9429.69            0.00
 W1BV           0.00            0.00           14.46
            -257.89         -476.08            0.00
 W1C            0.00            0.00         -305.81
           -5576.86         9429.69            0.00
 W1CV           0.00            0.00           15.12
            269.66         -482.68            0.00
 W1D            0.00            0.00          -33.90
            601.62          152.57            0.00
 W1E            0.00            0.00          -33.90
             59.16          152.57            0.00
 W1F            0.00            0.00          -46.12
            -349.57          461.17            0.00
 W1F2           0.00            0.00          -77.57
            -562.36          788.46            0.00
 W14A           0.00            0.00           -4.00
               0.68           89.32            0.00
 F1          2016.84            0.00            0.00
               0.00        24614.84            0.00
 F2         -1119.23            0.00            0.00
               0.00       -12039.25            0.00
 F2A            0.00         1721.87            0.00
          -18521.61            0.00        45483.28
 F2B            0.00        -1363.35            0.00
           14665.13            0.00       -43511.42

TOTAL         897.60          358.52        -1026.35
           -4114.08        45489.13         1971.86
                    ---------
```

Analysis Output from 3DSAD for Load Condition A. TPSANA.TXT

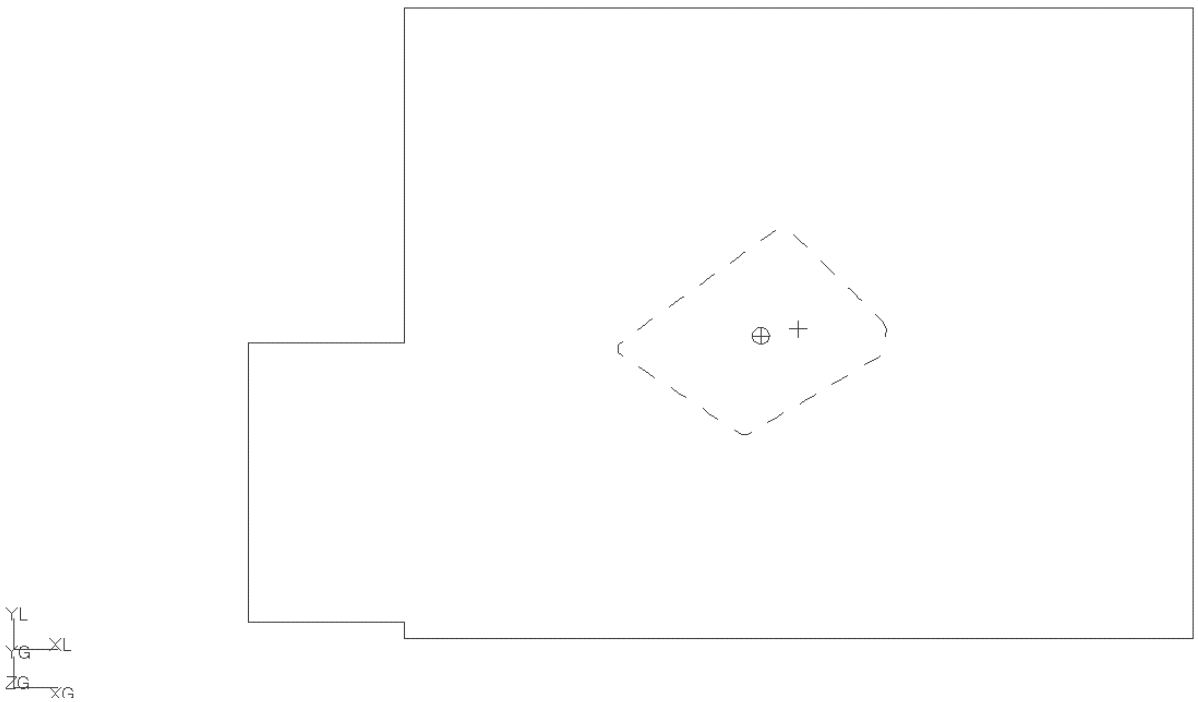
```
                    LOAD CASE ANA  CATEGORY =    2


                     FINAL BASE AREA PROPERTIES

  AREA =     1926.228      IXP =     230285.8      IYP =     425614.5
  XBAR =       29.717     YBAR =       -0.782     ZBAR =        0.000
 XYANG =        0.000    ZNANG =        0.000    PXANG =       11.440


                 SUMMARY OF FINAL FORCES AND MOMENTS

                            --INPUT--

   FX =      897.600       FY =      358.520       FZ =    -4275.760
   MX =       1499.5       MY =     136438.3       MZ =       1971.9

                        --COMPUTED UPLIFT--

   FX =        0.000       FY =        0.000       FZ =        0.000
   MX =          0.0       MY =          0.0       MZ =          0.0

                            --TOTAL--

   FX =      897.600       FY =      358.520       FZ =    -4275.760
   MX =       1499.5       MY =     136438.3       MZ =       1971.9


            FINAL IN PLANE COORDINATES AND BASE PRESSURES

 NAME                X               Y               Z           PRESSURE


 700               0.000         -18.330          0.000           1.490
 701               9.000         -18.330          0.000           1.689
 702               9.000         -19.330          0.000           1.685
 703              54.830         -19.330          0.000           2.695
 704              54.830          19.330          0.000           2.858
 705               9.000          19.330          0.000           1.848
706               9.000          -1.170          0.000           1.761
707               0.000          -1.170          0.000           1.563

                FINAL SHEAR FRICTION FACTOR OF SAFETY

 PHI =         26.60   SHRSTR =          0.00   SANGLE =         21.77

            FACTOR OF SAFETY =          2.22


                        EFFECTIVE BASE = 100.0%
```

Final Kern Plot for Load Condition A:

LOAD CASE ANA
CATEGORY = 2

ECC. - LOCAL
X ECC. = 2.19
Y ECC. = 0.43

R DIST = 3.83



Loading Output from 3DSAD for Load Condition B. TPSLCB.TXT

NAME	FX MX	FY MY	FZ MZ
W1A	0.00	0.00	-286.53
	-20.06	15396.97	0.00
W1AV	0.00	0.00	24.13
	0.12	-1298.93	0.00
W1B	0.00	0.00	-322.49
	5881.05	9944.04	0.00
W1BV	0.00	0.00	15.25
	-271.96	-502.05	0.00
W1C	0.00	0.00	-322.49
	-5881.05	9944.04	0.00
W1CV	0.00	0.00	15.95
	284.37	-509.01	0.00
W1D	0.00	0.00	-35.75
	634.43	160.89	0.00
W1E	0.00	0.00	-35.75
	62.39	160.89	0.00
W1F	0.00	0.00	-48.63
	-368.64	486.33	0.00
W1F2	0.00	0.00	-81.80
	-593.03	831.47	0.00
W14A	0.00	0.00	-4.00
	0.68	89.32	0.00
F1	980.71	0.00	0.00
	0.00	11979.61	0.00
F2	-542.99	0.00	0.00
	0.00	-5840.82	0.00
F2A	0.00	835.36	0.00
	-8985.74	0.00	22066.07
F2B	0.00	-661.43	0.00
	7114.78	0.00	-21109.43
W2	0.00	0.00	-309.34
	3014.47	2036.97	0.00
W14B	0.00	0.00	-233.75
	2107.31	7760.42	0.00
W14C	0.00	0.00	-305.01
	-2182.48	9411.68	0.00
F3	-1606.82	0.00	0.00
	0.00	-17284.02	0.00
F4	2177.52	0.00	0.00
	0.00	29144.43	0.00
F3A	0.00	2472.00	0.00
	-26590.41	0.00	65297.76

```
                         ---------
 NAME            FX                FY                FZ
                 MX                MY                MZ


 F3B              0.00          -1957.29             0.00
             21053.89              0.00        -62466.87

TOTAL          1008.42            688.64         -1930.21
             -4739.88          71912.24          3787.53
                         ---------
```

Analysis Output from 3DSAD for Load Condition B.
TPSANB.TXT

```
                    LOAD CASE ANB  CATEGORY =    2


                    COMPUTED BASE AREA PROPERTIES

  AREA =     1926.226      IXP =     230285.3      IYP =     425614.1
  XBAR =       29.717     YBAR =       -0.782     ZBAR =        0.000
 XYANG =        0.000    ZNANG =        0.000    PXANG =       11.440


                    SUMMARY OF FORCES AND MOMENTS

                              --INPUT--

   FX =     1008.420       FY =      688.640       FZ =    -5179.620
   MX =        873.7       MY =     162861.4       MZ =       3787.5

                          --COMPUTED UPLIFT--

   FX =        0.000       FY =        0.000       FZ =     5667.389
   MX =      -4468.8       MY =    -168044.7       MZ =          0.0

                              --TOTAL--

   FX =     1008.420       FY =      688.640       FZ =      487.769
   MX =      -3595.1       MY =      -5183.3       MZ =       3787.5
```

COMPUTED IN PLANE COORDINATES AND BASE PRESSURES

NAME	X	Y	Z	PRESSURE
700	0.000	-18.330	0.000	-1.065
701	9.000	-18.330	0.000	-0.873
702	9.000	-19.330	0.000	-0.883
703	54.830	-19.330	0.000	0.096
704	54.830	19.330	0.000	0.486
705	9.000	19.330	0.000	-0.493
706	9.000	-1.170	0.000	-0.700
707	0.000	-1.170	0.000	-0.892

COMPUTED SHEAR FRICTION FACTOR OF SAFETY

PHI = 26.60 SHRSTR = 0.00 SANGLE = 34.33

FACTOR OF SAFETY = 0.20

Loading Output from 3DSAD for Load Condition C. TPSLCC.TXT

NAME	FX MX	FY MY	FZ MZ
W1A	0.00	0.00	-286.53
	-20.06	15396.97	0.00
W1AV	0.00	0.00	24.13
	0.12	-1298.93	0.00
W1B	0.00	0.00	-322.49
	5881.05	9944.04	0.00
W1BV	0.00	0.00	15.25
	-271.96	-502.05	0.00
W1C	0.00	0.00	-322.49
	-5881.05	9944.04	0.00
W1CV	0.00	0.00	15.95
	284.37	-509.01	0.00
W1D	0.00	0.00	-35.75
	634.43	160.89	0.00
W1E	0.00	0.00	-35.75
	62.39	160.89	0.00
W1F	0.00	0.00	-48.63
	-368.64	486.33	0.00
W1F2	0.00	0.00	-81.80
	-593.03	831.47	0.00
W14A	0.00	0.00	-4.00
	0.68	89.32	0.00
F1	980.71	0.00	0.00
	0.00	11979.61	0.00
F2	-542.99	0.00	0.00
	0.00	-5840.82	0.00
F2A	0.00	835.36	0.00
	-8985.74	0.00	22066.07
F2B	0.00	-661.43	0.00
	7114.78	0.00	-21109.43
W2	0.00	0.00	-309.34
	3014.47	2036.97	0.00
W14B	0.00	0.00	-233.75
	2107.31	7760.42	0.00
W14C	0.00	0.00	-305.01
	-2182.48	9411.68	0.00
F3	-1363.02	0.00	0.00
	0.00	-14661.71	0.00
F4	1695.47	0.00	0.00
	0.00	21336.84	0.00
F3A	0.00	2096.93	0.00
	-22556.04	0.00	55390.39

```
                         ---------
 NAME            FX                FY                FZ
                 MX                MY                MZ


 F3B              0.00          -1660.32             0.00
             17859.53              0.00        -52989.02

TOTAL           770.17            610.55         -1930.21
            -3899.87          66726.97          3358.01
                         ---------
```

Analysis Output from 3DSAD for Load Condition C.
TPSANC.TXT

```
                    LOAD CASE ANC  CATEGORY =    2


                     FINAL BASE AREA PROPERTIES

  AREA =     1132.523      IXP =      73044.1      IYP =     187428.7
  XBAR =       38.376     YBAR =        3.920     ZBAR =        0.000
 XYANG =        0.000    ZNANG =        0.000    PXANG =      -41.575


                 SUMMARY OF FINAL FORCES AND MOMENTS

                              --INPUT--

  FX =      770.170       FY =      610.550       FZ =    -5179.620
  MX =       1713.7       MY =     157676.1       MZ =       3358.0

                          --COMPUTED UPLIFT--

  FX =        0.000       FY =        0.000       FZ =     4858.467
  MX =      -3961.1       MY =    -143899.2       MZ =          0.0

                              --TOTAL--

  FX =      770.170       FY =      610.550       FZ =     -321.153
  MX =      -2247.4       MY =      13776.9       MZ =       3358.0
```

FINAL IN PLANE COORDINATES AND BASE PRESSURES

NAME	X	Y	Z	PRESSURE
703-702	43.408	-19.330	0.000	0.000
703	54.830	-19.330	0.000	0.196
704	54.830	19.330	0.000	0.812
705	9.000	19.330	0.000	0.024
705-706	9.000	17.828	0.000	0.000

FINAL SHEAR FRICTION FACTOR OF SAFETY

PHI = 26.60 SHRSTR = 0.00 SANGLE = 38.41

FACTOR OF SAFETY = 0.16

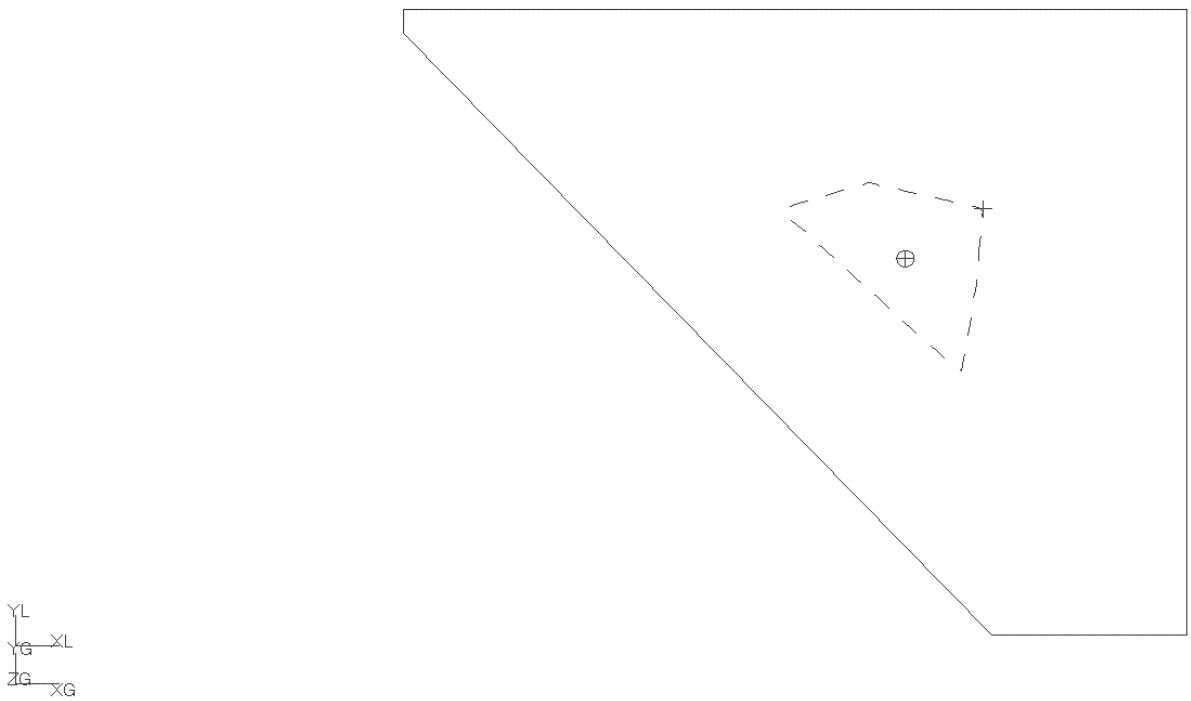
EFFECTIVE BASE = 58.8%

Final Kern Plot for Load Condition C:

LOAD CASE ANC
CATEGORY = 2

ECC. - LOCAL
X ECC. = 4.52
Y ECC. = 3.08

R DIST = 0.00



Loading Output from 3DSAD for Load Condition D. TPSLCD.TXT

NAME	FX MX	FY MY	FZ MZ
W1A	0.00	0.00	-286.53
	-20.06	15396.97	0.00
W1AV	0.00	0.00	24.13
	0.12	-1298.93	0.00
W1B	0.00	0.00	-322.49
	5881.05	9944.04	0.00
W1BV	0.00	0.00	15.25
	-271.96	-502.05	0.00
W1C	0.00	0.00	-322.49
	-5881.05	9944.04	0.00
W1CV	0.00	0.00	15.95
	284.37	-509.01	0.00
W1D	0.00	0.00	-35.75
	634.43	160.89	0.00
W1E	0.00	0.00	-35.75
	62.39	160.89	0.00
W1F	0.00	0.00	-48.63
	-368.64	486.33	0.00
W1F2	0.00	0.00	-81.80
	-593.03	831.47	0.00
W14A	0.00	0.00	-4.00
	0.68	89.32	0.00
F1	980.71	0.00	0.00
	0.00	11979.61	0.00
F2	-542.99	0.00	0.00
	0.00	-5840.82	0.00
F2A	0.00	835.36	0.00
	-8985.74	0.00	22066.07
F2B	0.00	-661.43	0.00
	7114.78	0.00	-21109.43
W2	0.00	0.00	-309.34
	3014.47	2036.97	0.00
W14B	0.00	0.00	-233.75
	2107.31	7760.42	0.00
W14C	0.00	0.00	-305.01
	-2182.48	9411.68	0.00
F3	-1329.78	0.00	0.00
	0.00	-14304.08	0.00
F4	1623.44	0.00	0.00
	0.00	20204.43	0.00
F3A	0.00	2045.79	0.00
	-22005.86	0.00	54039.52

```
                         ---------
 NAME              FX                FY                FZ
                   MX                MY                MZ


 F3B               0.00          -1619.82              0.00
               17423.91              0.00         -51696.71

TOTAL            731.38            599.90          -1930.21
               -3785.31          65952.17           3299.44
                         ---------
```

Analysis Output from 3DSAD for Load Condition D.
TPSAND.TXT

```
                    LOAD CASE AND  CATEGORY =    2


                     FINAL BASE AREA PROPERTIES

   AREA =     1758.263      IXP =     205427.0      IYP =     319269.5
   XBAR =       31.900     YBAR =        0.406     ZBAR =        0.000
  XYANG =        0.000    ZNANG =        0.000    PXANG =       -6.315


                 SUMMARY OF FINAL FORCES AND MOMENTS

                              --INPUT--

    FX =      731.380       FY =      599.900       FZ =    -5179.620
    MX =       1828.2       MY =     156901.3       MZ =       3299.4

                           --COMPUTED UPLIFT--

    FX =        0.000       FY =        0.000       FZ =     4709.117
    MX =      -3779.8       MY =    -139534.2       MZ =          0.0

                              --TOTAL--

    FX =      731.380       FY =      599.900       FZ =     -470.503
    MX =      -1951.5       MY =      17367.1       MZ =       3299.4
```

FINAL IN PLANE COORDINATES AND BASE PRESSURES

NAME	X	Y	Z	PRESSURE
703-702	20.223	-19.330	0.000	0.000
703	54.830	-19.330	0.000	0.268
704	54.830	19.330	0.000	0.616
705	9.000	19.330	0.000	0.260
706	9.000	-1.170	0.000	0.076
707	0.000	-1.170	0.000	0.006
707-700	0.000	-1.828	0.000	0.000

FINAL SHEAR FRICTION FACTOR OF SAFETY

PHI = 26.60 SHRSTR = 0.00 SANGLE = 39.36

FACTOR OF SAFETY = 0.25

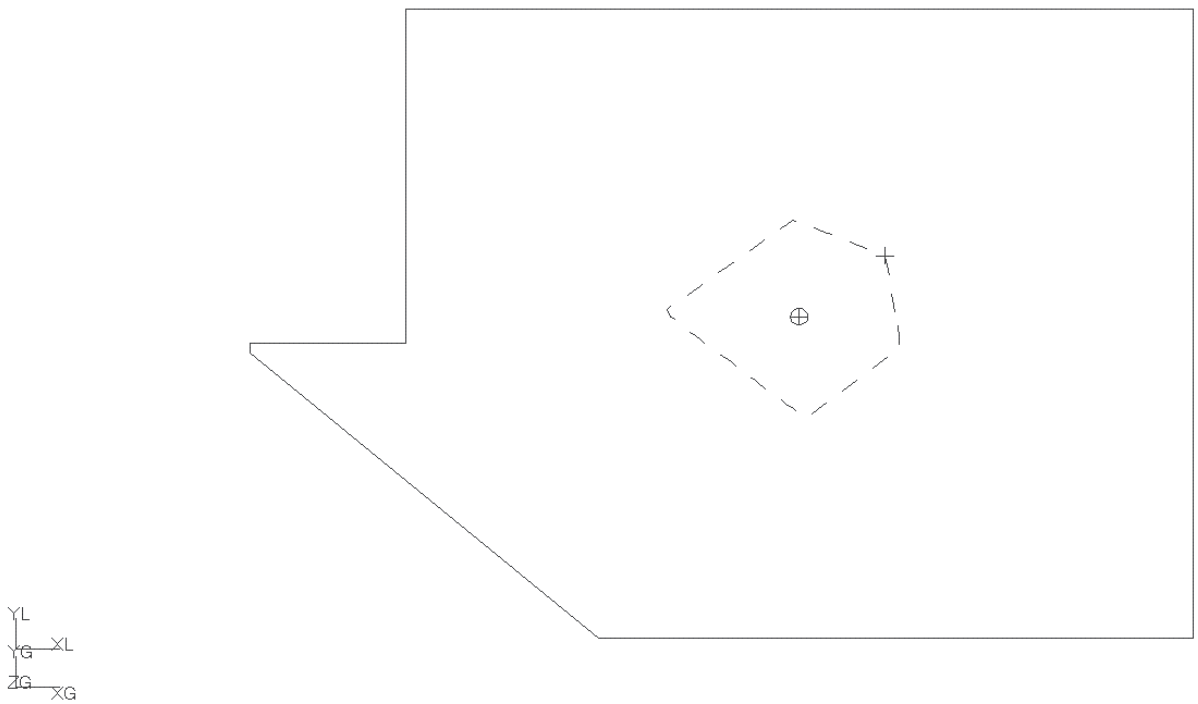
EFFECTIVE BASE = 91.3%

Final Kern Plot for Load Condition D:

LOAD CASE AND
CATEGORY = 2

ECC. - LOCAL
X ECC. = 5.01
Y ECC. = 3.74

R DIST = -0.01



Loading Output from 3DSAD for Load Condition E. TPSLCE.TXT

NAME	FX	FY	FZ
	MX	MY	MZ
W1A	0.00	0.00	-286.53
	-20.06	15396.97	0.00
W1AV	0.00	0.00	24.13
	0.12	-1298.93	0.00
W1B	0.00	0.00	-322.49
	5881.05	9944.04	0.00
W1BV	0.00	0.00	15.25
	-271.96	-502.05	0.00
W1C	0.00	0.00	-322.49
	-5881.05	9944.04	0.00
W1CV	0.00	0.00	15.95
	284.37	-509.01	0.00
W1D	0.00	0.00	-35.75
	634.43	160.89	0.00
W1E	0.00	0.00	-35.75
	62.39	160.89	0.00
W1F	0.00	0.00	-48.63
	-368.64	486.33	0.00
W1F2	0.00	0.00	-81.80
	-593.03	831.47	0.00
W14A	0.00	0.00	-4.00
	0.68	89.32	0.00
F1	980.71	0.00	0.00
	0.00	11979.61	0.00
F2	-542.99	0.00	0.00
	0.00	-5840.82	0.00
F2A	0.00	835.36	0.00
	-8985.74	0.00	22066.07
F2B	0.00	-661.43	0.00
	7114.78	0.00	-21109.43
W2	0.00	0.00	-309.34
	3014.47	2036.97	0.00
W14B	0.00	0.00	-233.75
	2107.31	7760.42	0.00
W14C	0.00	0.00	-305.01
	-2182.48	9411.68	0.00
F3	-1296.53	0.00	0.00
	0.00	-13946.47	0.00
F4	1551.41	0.00	0.00
	0.00	19012.42	0.00
F3A	0.00	1994.64	0.00
	-21455.69	0.00	52688.47

```
                         ---------
 NAME            FX                FY                FZ
                 MX                MY                MZ


 F3B              0.00          -1579.33             0.00
             16988.29              0.00        -50404.24

TOTAL           692.60           589.25         -1930.21
            -3670.76         65117.78          3240.87
                         ---------
```

Analysis Output from 3DSAD for Load Condition E.
TPSANE.TXT

```
                    LOAD CASE ANE  CATEGORY =    2


                     FINAL BASE AREA PROPERTIES

  AREA =     1926.226      IXP =     230285.3      IYP =     425614.1
  XBAR =       29.717     YBAR =       -0.782     ZBAR =        0.000
 XYANG =        0.000    ZNANG =        0.000    PXANG =       11.440


                  SUMMARY OF FINAL FORCES AND MOMENTS

                              --INPUT--

   FX =      692.600       FY =      589.250       FZ =    -5179.620
   MX =       1942.8       MY =     156066.9       MZ =       3240.9

                          --COMPUTED UPLIFT--

   FX =        0.000       FY =        0.000       FZ =     4579.457
   MX =      -3621.8       MY =    -135630.8       MZ =          0.0

                              --TOTAL--

   FX =      692.600       FY =      589.250       FZ =     -600.163
   MX =      -1679.0       MY =      20436.1       MZ =       3240.9
```

FINAL IN PLANE COORDINATES AND BASE PRESSURES

NAME	X	Y	Z	PRESSURE
700	0.000	-18.330	0.000	0.006
701	9.000	-18.330	0.000	0.055
702	9.000	-19.330	0.000	0.047
703	54.830	-19.330	0.000	0.298
704	54.830	19.330	0.000	0.613
705	9.000	19.330	0.000	0.362
706	9.000	-1.170	0.000	0.195
707	0.000	-1.170	0.000	0.145

FINAL SHEAR FRICTION FACTOR OF SAFETY

PHI = 26.60 SHRSTR = 0.00 SANGLE = 40.39

FACTOR OF SAFETY = 0.33

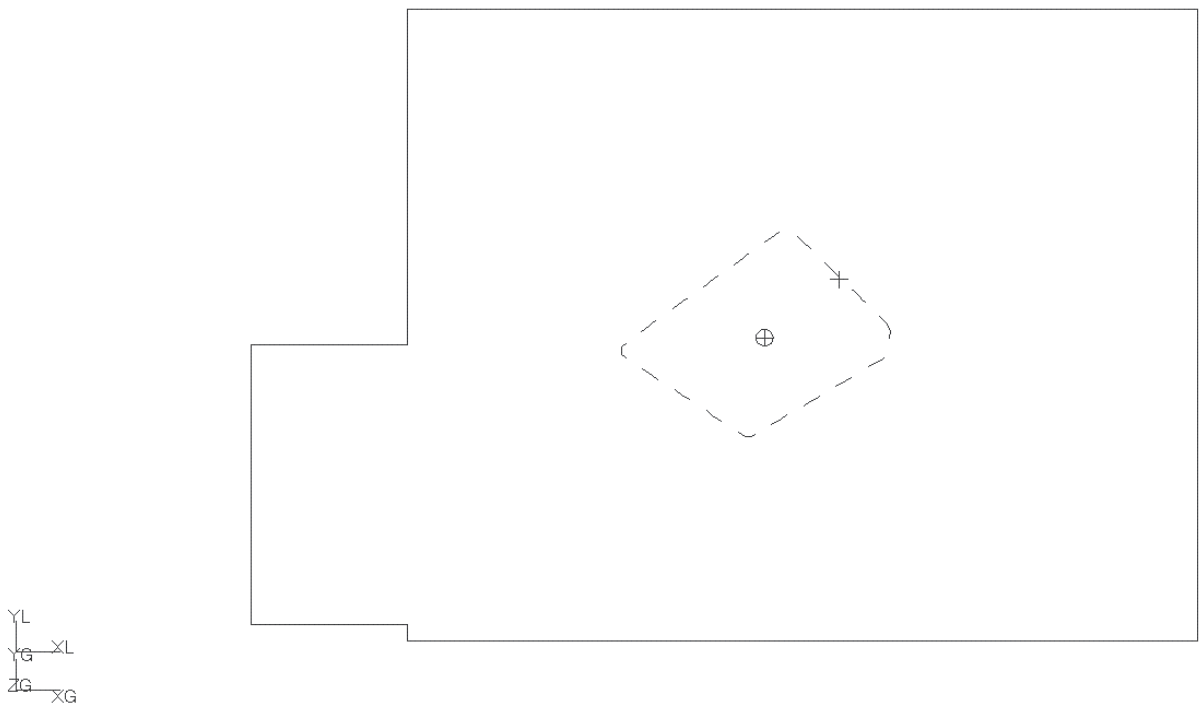
EFFECTIVE BASE = 100.0%

Final Kern Plot for Load Condition E:

LOAD CASE ANE
CATEGORY = 2

ECC. - LOCAL
X ECC. = 4.33
Y ECC. = 3.58

R DIST = 0.10



UPLIFT COMPUTATIONS/RESULTS
(printout from _Turner Stability.v3.xls)

TABLE 1-1. - TURNER Pump Station Bearing Pressure and Uplift SUMMARY (Results of 3DSAD)

Load Condition	Load Condition Description	Load (force) file	Load case file	Analysis Input File	Structure	Mech Equip	Backfill	Water in Station	HGL	RESULT (F=FINAL, I=INTERMEDIATE)	FX (k)	FY (k)	FZ (k)	MX (k-ft)	MY (k-ft)	MZ (k-ft)	Base Point 700 (ksf)	Base Point 701 (ksf)	Base Point 702 (ksf)	Base Point 703 (ksf)	Base Point 704 (ksf)	Base Point 705 (ksf)	Base Point 706 (ksf)	Base Point 707 (ksf)	Max Pressure (ksf)	Min Pressure (ksf)	Effective Base	Input FZ	Uplift FZ	Flotation Factor of Safety SF_f
		filename (*.txt)																												
0	**Existing Station Only - GW raise**	TPSF0	-	TPSAN0	X	X				F	0.00	0.00	-3206.44	5190.89	90663.27	0.00	2.137	2.045	2.055	1.588	1.215	1.681	1.879	1.971	2.137	1.215	100.0%	-3206.440	0.000	N/A
00	**Existing Station Only - GW raise - FLOOD for Existing Conditions (note 1)**																													(note 1)
01	**Existing Station Only + GW raise**	TPSF0 + TPSF1	-	TPSAN01	X	X				F	0.00	0.00	-3249.41	5613.55	90949.14	0.00	2.248	2.136	2.147	1.577	1.155	1.725	1.949	2.061	2.248	1.155	100.0%	-3249.410	0.000	N/A
A	**Existing Station + GW raise + soil**	TPSF0 + TPSF1			X	X	X			F	0.00	0.00	-3249.41	5613.55	90949.14	0.00														
			TPSLCA								897.60	358.52	-1026.35	-4114.08	45489.13	1971.86														
				TPSANA							897.60	358.52	-4275.76	1499.47	136438.27	1971.86	1.490	1.689	1.685	2.695	2.858	1.848	1.761	1.563	2.858	1.490	100.0%	-4275.760	0.000	N/A
B	**Existing Station + GW raise**	TPSF0 + TPSF1	-		X	X					0.00	0.00	-3249.41	5613.55	90949.14	0.00														
	Proposed 500+3 HGL		TPSLCB		X	X					1008.42	688.64	-1930.21	-4739.88	71912.24	3787.53														
				TPSANB			X	8	F	I	1008.42	688.64	-5179.62	873.67	162861.38	3787.53	-1.065	-0.873	-0.883	0.096	0.486	-0.493	-0.700	-0.892	0.486	-1.065	<1%	-5179.620	5667.389	0.90
C	**Existing Station + GW raise**	TPSF0 + TPSF1	-		X	X					0.00	0.00	-3249.41	5613.55	90949.14	0.00														
	Proposed 500+3, HGL (-7)		TPSLCC		X	X					770.17	610.55	-1930.21	-3899.87	66726.97	3358.01														
				TPSANC			X	8	-7	I	770.17	610.55	-5179.62	1713.68	157676.11	3358.01	-0.244	-0.160	-0.169	0.257	0.586	0.160	-0.014	-0.098	0.586	-0.244	<1%	-5179.620	4829.865	
							X	8	-7	F	770.17	610.55	-5179.62	1713.68	157676.11	3358.01	0.000	0.000	0.000	0.196	0.312	0.024	0.000	0.000	0.312	0.000	58.8%	-5179.620	4858.467	1.08
D	**Existing Station + GW raise**	TPSF0 + TPSF1	-		X	X					0.00	0.00	-3249.41	5613.55	90949.14	0.00														
	Proposed 500+3, HGL (-8)		TPSLCD		X	X					731.38	599.90	-1930.21	-3785.31	65952.17	3299.44														
				TPSAND			X	8	-8	I	731.38	599.90	-5179.62	1828.24	156901.31	3299.44	-0.115	-0.050	-0.058	0.275	0.597	0.264	0.093	0.028	0.597	-0.115	<1%	-5179.620	4704.275	
							X	8	-8	F	731.38	599.90	-5179.62	1828.24	156901.31	3299.44	0.000	0.000	0.000	0.268	0.616	0.260	0.076	0.006	0.616	0.000	91.3%	-5179.620	4709.117	1.11
E	**Existing Station + GW raise**	TPSF0 + TPSF1	-		X	X					0.00	0.00	-3249.41	5613.55	90949.14	0.00														
	Proposed 500+3, HGL (-9)		TPSLCE		X	X					692.60	589.25	-1930.21	-3670.76	65117.78	3240.87														
				TPSANE			X	8	-9	F	692.60	589.25	-5179.62	1942.79	156066.92	3240.87	0.006	0.055	0.047	0.298	0.613	0.362	0.195	0.145	0.613	0.006	100.0%	-5179.620	4579.457	1.15

Note 1. Load condition 00 - This Existing Condition Load Case was investigated for uplift only based upon an FLOOD condition with a HGL 12.3 feet above grade. 3DSAD was not run for this load case, due to simple uplift being investigated. A range for the distance above grade to the HGL was run with the existing station. Results are shown in the table lablel "UPLIFT - Existing" and graphic results in form of a plot are in "UPLIFT PLOT".

Turner Pump Station

LC00 - Existing Station Only - FLOOD - (Uplift Check Only)

	Use per Scott Loehr (NWK)	Dist. GHL above grade (ft)	Top of Grade (ft)	HGL EL. (ft)	EL. Bot of PS (ft)	AVG uplift (ksf)	Area base (ft^2)	UPLIFT (k)
		9	762	771	729.7	2.58	1926.71	4972.115994
		9.5	762	771.5	729.7	2.61	1926.71	5032.325681
		10	762	772	729.7	2.64	1926.71	5092.535369
		11	762	773	729.7	2.71	1926.71	5212.954744
		12	762	774	729.7	2.77	1926.71	5333.374119
Existing Conditions =	x	12.3	762	774.3	729.7	2.79	1926.71	5369.499931
		13	762	775	729.7	2.83	1926.71	5453.793494

LCA - Existing Soil, no Flood

		Ko	UNIT WT kcf	Lateral depth (ft)	pressure (psf)	Uplift head (ft)	uplift (ksf)
soil	P1	0.75	0.11	5.91	0.49		
	P3	0.75	0.11	38.16	3.15		
	P5	0.57	0.11	32.27	2.02		
water	P2	1	0.0625	0	0.00		
	P4	1	0.0625	0	0.00		
	P6	1	0.0625	0	0.00		
	P7	1	0.0625			0	0.00
	P8	1	0.0625			0	0.00
	P9	1	0.0625			0	0.00
	P10	1	0.0625			0	0.00
	P11	1	0.0625			0	0.00

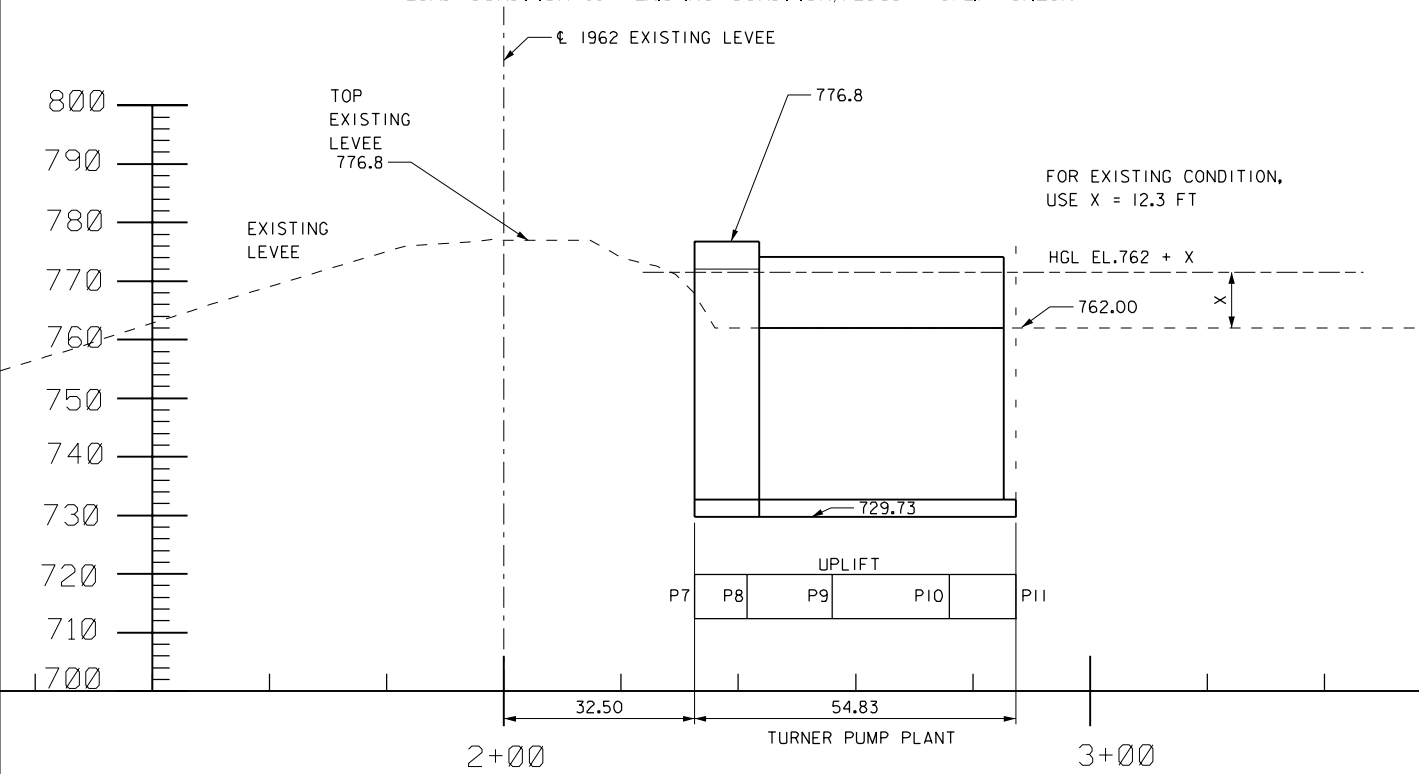
LCB - 500+3 Flood, Full HGL

		Ko	UNIT WT kcf	Lateral depth (ft)	pressure (psf)	Uplift head (ft)	uplift (ksf)
soil	P1	0.75	0.0535	5.91	0.24		
	P3	0.75	0.0535	38.16	1.53		
	P5	0.57	0.0535	32.27	0.98		
water	P2	1	0.0625	15.31	0.96		
	P4	1	0.0625	47.58	2.97		
	P6	1	0.0625	46.38	2.90		
	P7	1	0.0625			47.58	2.97
	P8	1	0.0625			47.43	2.96
	P9	1	0.0625			47.18	2.95
	P10	1	0.0625			46.85	2.93
	P11	1	0.0625			46.67	2.92

		LCC - 500+3 Flood, HGL			**-7**				
				Lateral		Uplift			
		Ko	UNIT WT	depth	pressure	head (ft)	uplift		
			kcf	(ft)	(psf)	(ft)	(ksf)		
soil	P1	0.75	0.0535	5.91	0.24				
	P3	0.75	0.0535	38.16	1.53				
	P5	0.57	0.0535	32.27	0.98				
water	P2	1	0.0625	8.31	0.52				
	P4	1	0.0625	40.58	2.54				
	P6	1	0.0625	39.38	2.46				
	P7	1	0.0625			40.58	2.54		
	P8	1	0.0625			40.43	2.53		
	P9	1	0.0625			40.18	2.51		
	P10	1	0.0625			39.85	2.49		
	P11	1	0.0625			39.67	2.48		
Pressures based upon nominal HGL reductions									
	FULL	-7		-8		-9		-10	
	depth	depth	pressure	depth	pressure	depth	pressure	depth	pressure
	(ft)	(ft)	(psf)	(ft)	(psf)	(ft)	(psf)	(ft)	(psf)
P2	15.31	8.31	0.52	7.31	0.46	6.31	0.39	5.31	0.33
P4	47.58	40.58	2.54	39.58	2.47	38.58	2.41	37.58	2.35
P6	46.38	39.38	2.46	38.38	2.40	37.38	2.34	36.38	2.27
P7	47.58	40.58	2.54	39.58	2.47	38.58	2.41	37.58	2.35
P8	47.43	40.43	2.53	39.43	2.46	38.43	2.40	37.43	2.34
P9	47.18	40.18	2.51	39.18	2.45	38.18	2.39	37.18	2.32
P10	46.85	39.85	2.49	38.85	2.43	37.85	2.37	36.85	2.30
P11	46.67	39.67	2.48	38.67	2.42	37.67	2.35	36.67	2.29
AVG	47.142	40.142	2.508875	39.142	2.446375	38.142	2.383875	37.142	2.321375

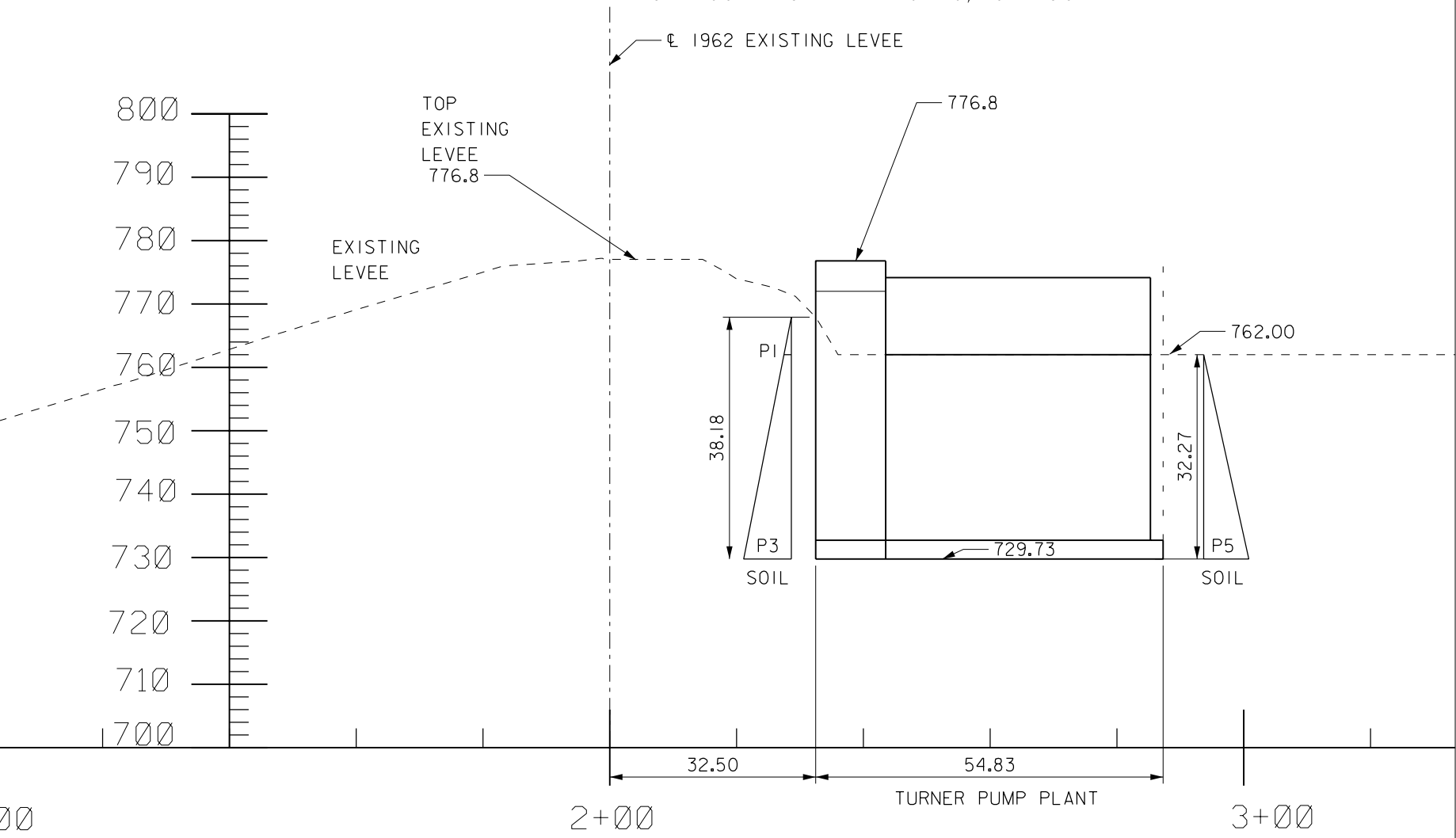
STA. 60+40
EXISTING, TOL=776.8

LOAD CONDITION 00 – EXISTING CONDITION, FLOOD – UPLIFT CHECK



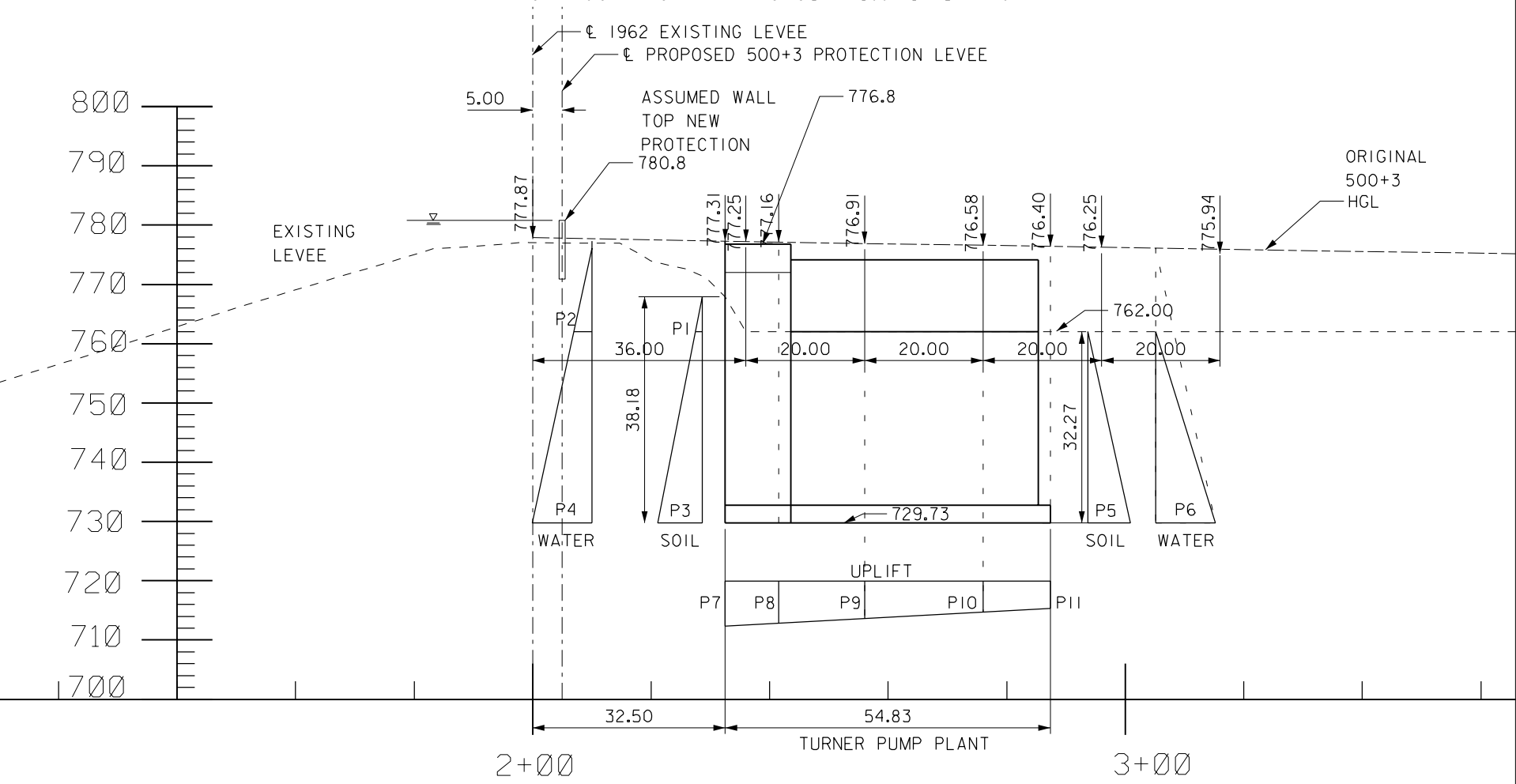
STA. 60+40
500+3, TOL=780.8

LOAD CONDITION A – EXISTING, NO FLOOD



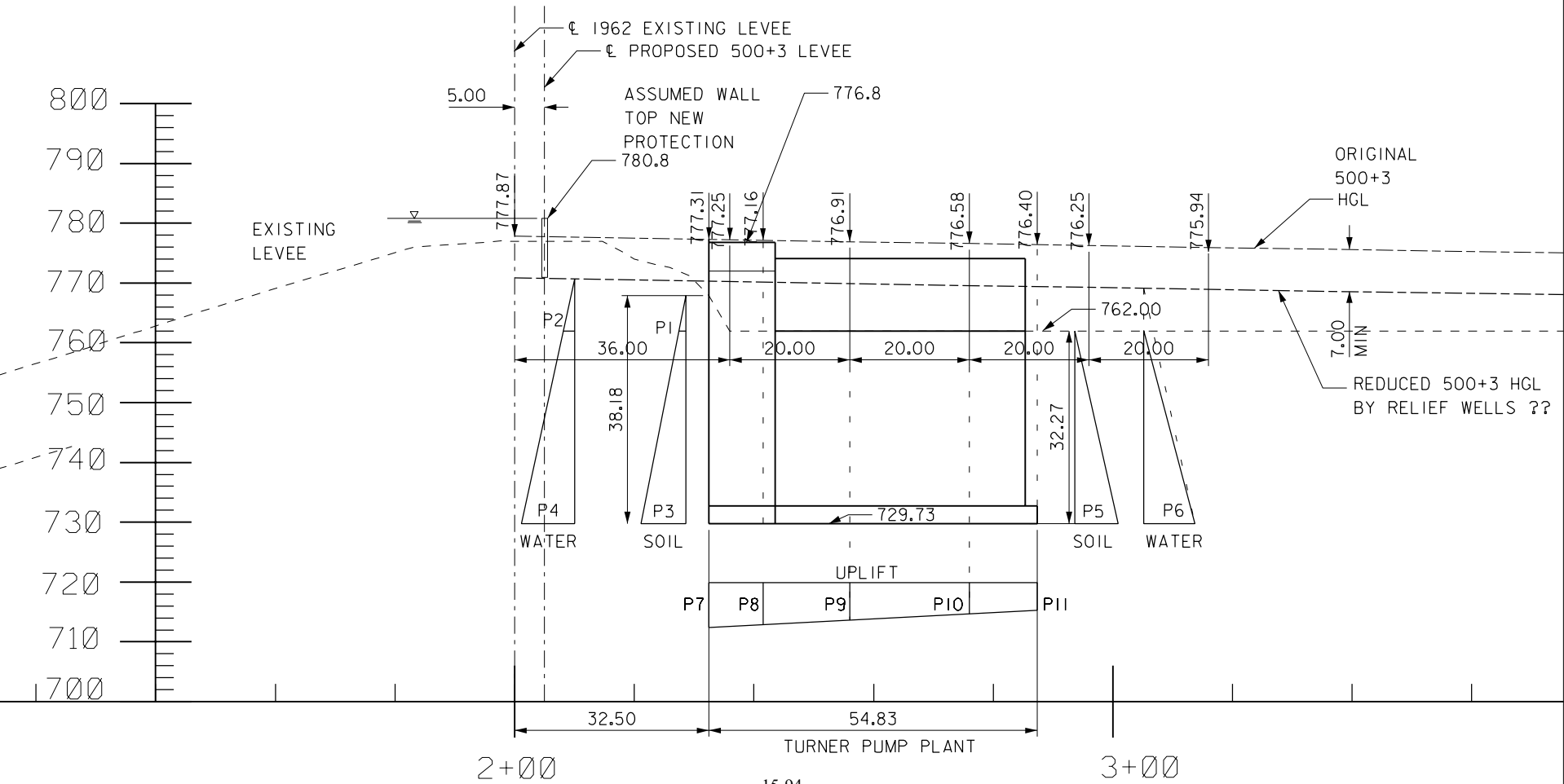
STA. 60+40
500+3, TOL=780.8

LOAD CONDITION B – PROPOSED 500+3 FULL HGL



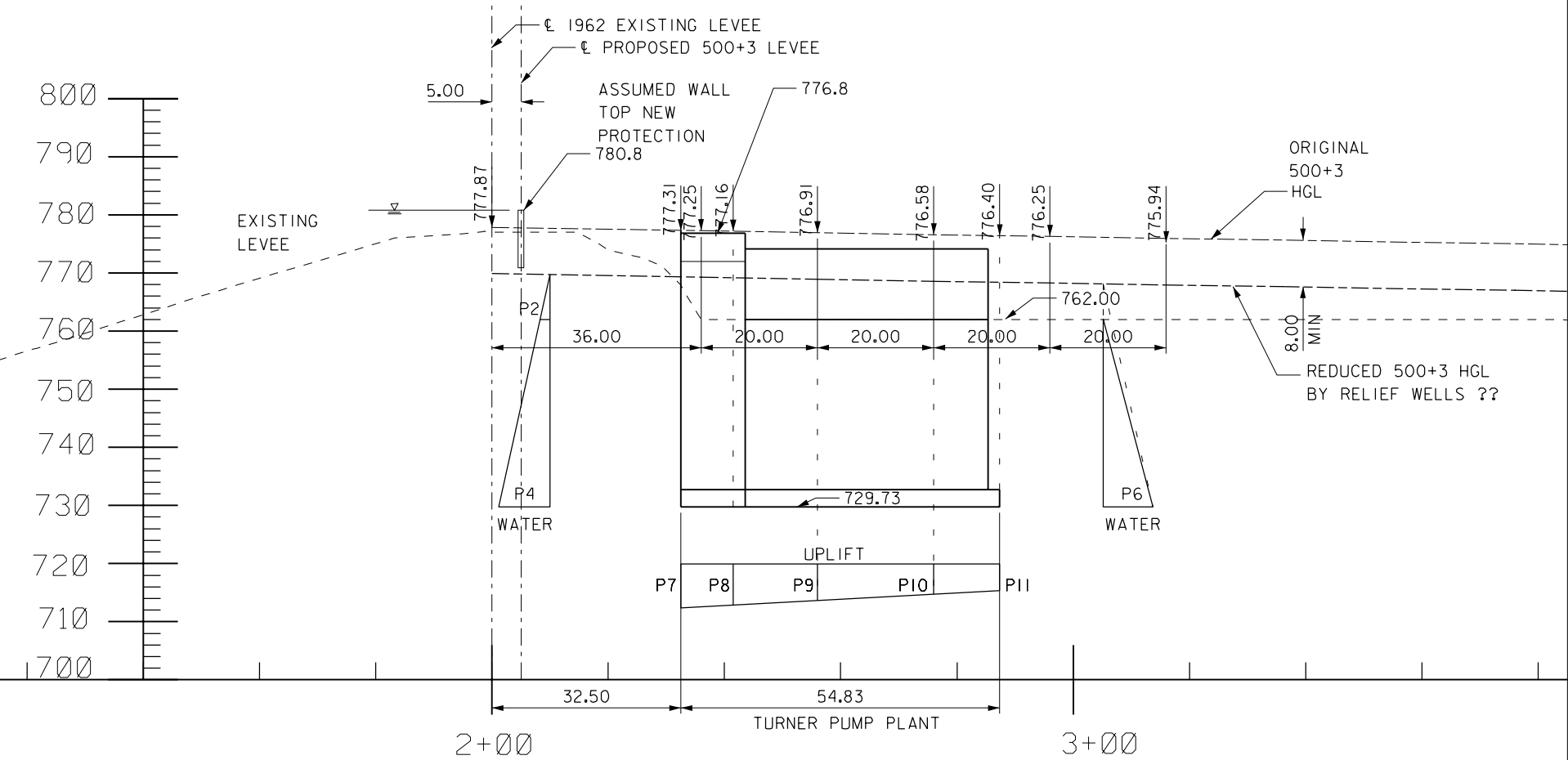
STA. 60+40
500+3, TOL=780.8

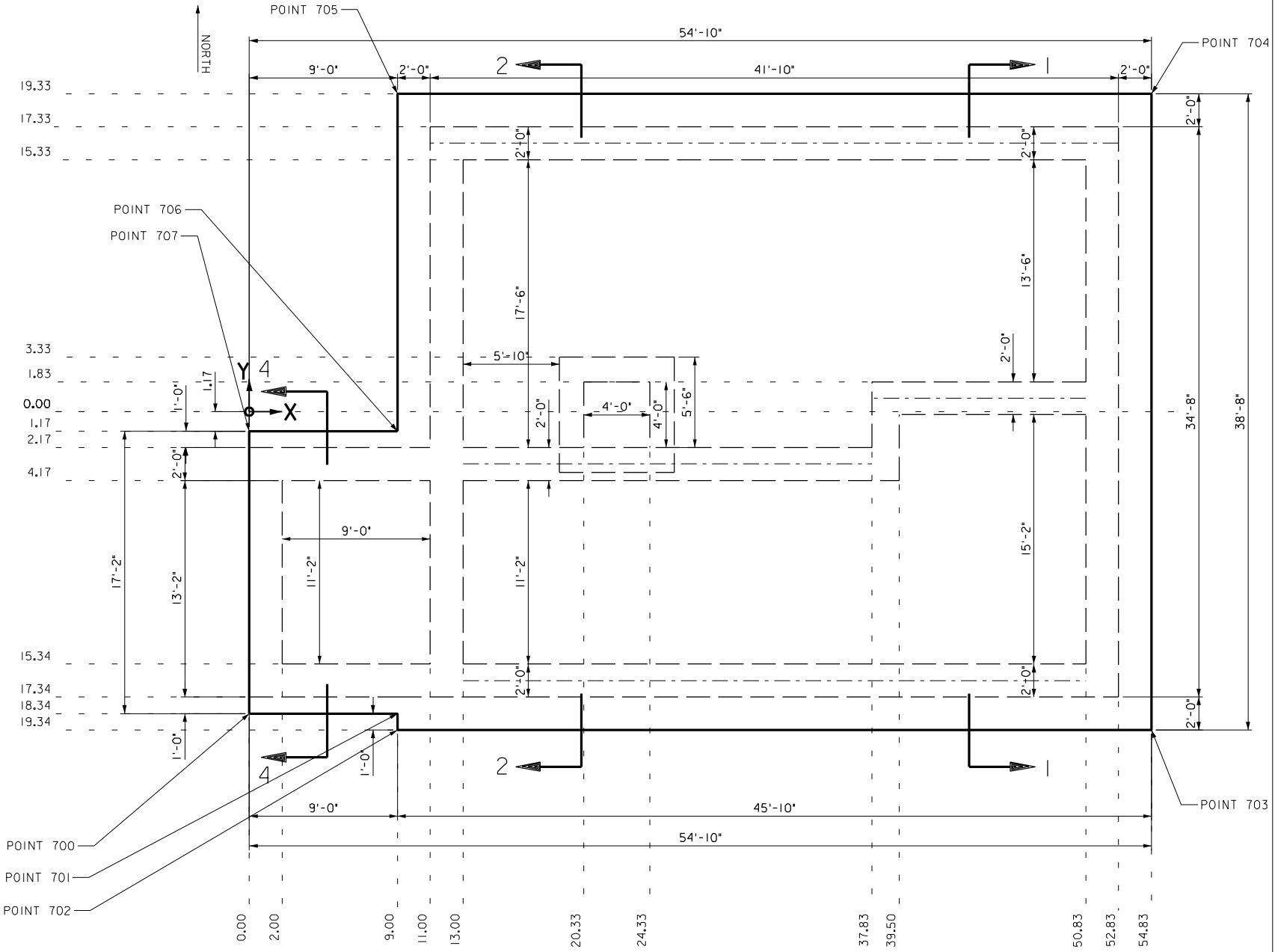
LOAD CONDITION C – PROPOSED 500+3 HGL (–7 FT)



STA. 60+40
500+3, TOL=780.8

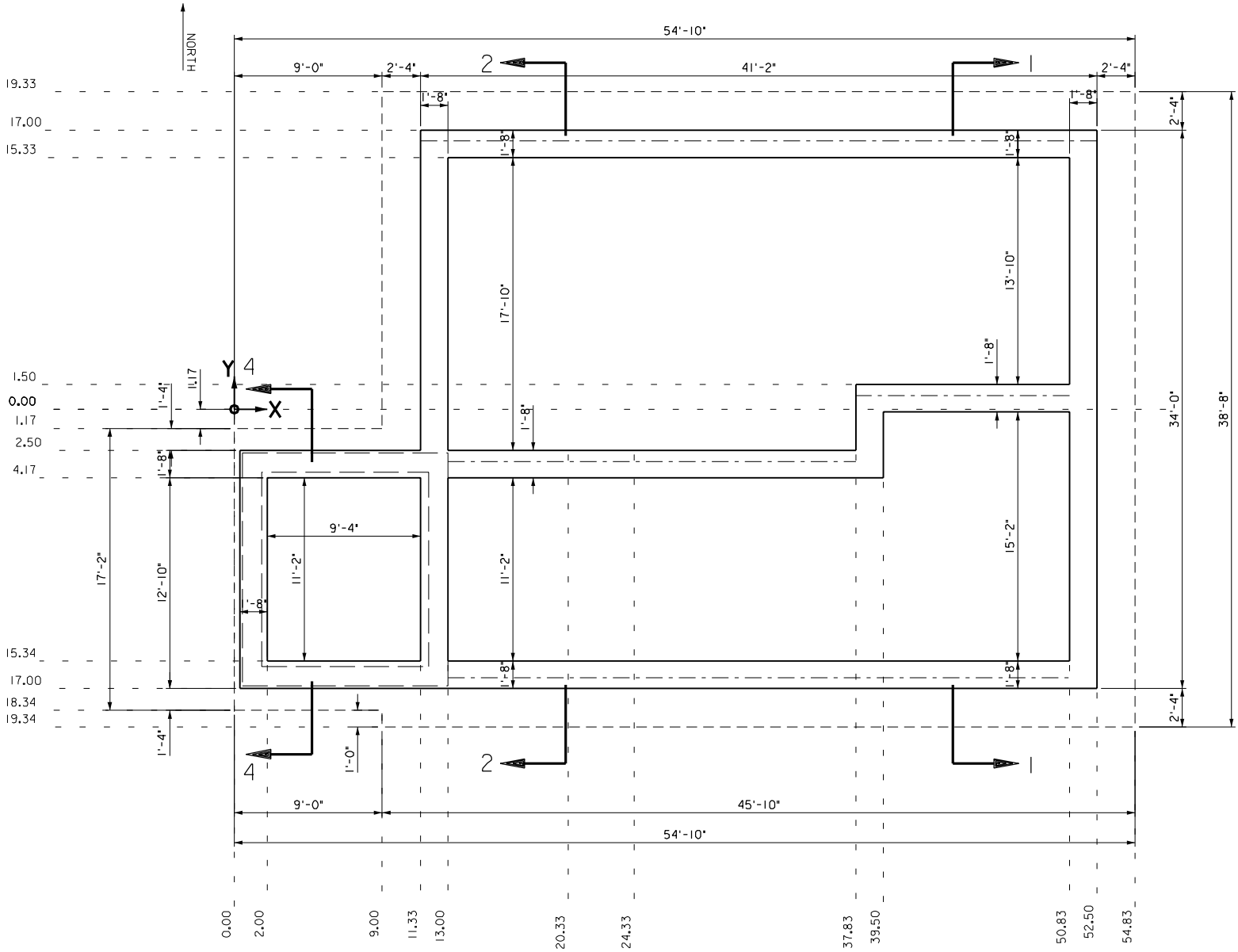
LOAD CONDITION D – PROPOSED 500+3 HGL (–8 FT)





TURNER PS - BASE PLAN
(EL. 729.73 TO 746.48)

REF: O&MM PLATE NO. 74-76



TURNER PS - PLAN SECTION
(EL. 746.48 TO EL. 762.0)

REF: O&MM PLATE NO. 74-76

TURNER PUMP STATION - 500 + 3 - FLOOD										
UPLIFT CONDITION A - Assumes 8 ft of water in pump station										
FSf =	Ws + Wc + S									
	(U-Wg)									
Ws =	Weight of structure (including wieghts of fixed equipment, soil above top surface of structure)									
Wc =	Weight of water contained in structure (controlled by mechanical)									
S =	surcharge loads									
U =	uplift forces acting on structure									
Wg =	weight of water above top surface of the structure (controlled by gravity)									
			total							
			wt							
Average Distance HGL above grade (ft)			(k)							
							LOAD CASE B	**LOAD CASE C**	**LOAD CASE D**	**LOAD CASE E**
FSf =							**0.9**	**1.08**	**1.11**	**1.15**
Ws + Wc + S							4598.7	4598.74	4598.74	4598.74
(U-Wg)							5086.5	4277.59	4128.24	3998.58
Ws							**3746.6**	**3746.6**	**3746.6**	**3746.6**
Structure + Equip										
	Existing Station		3206.4				3206.4	3206.44	3206.44	3206.44
	Gatewell Extension		43.0				43.0	42.97	42.97	42.97
				vol	water	buoyant				
					wt	soil wt				
Soil Portion of:				(ft^3)	(k)	(k)				
	W1A	soil vertical load above base extension - east	286.5	2470.1	154.4	132.1	132.1	132.15	132.15	132.15
	W1AV	soil void for concrete collar - east (EABT)	-24.1	-208.0	-13.0	-11.1	-11.1	-11.13	-11.13	-11.13
	W1B	soil vertical load above base extension - south	322.5	2780.1	173.8	148.7	148.7	148.73	148.73	148.73
	W1BV	soil void for concrete collar - south (SOBT)	-15.3	-131.5	-8.2	-7.0	-7.0	-7.03	-7.03	-7.03
	W1C	soil vertical load above base extension - north	322.5	2780.1	173.8	148.7	148.7	148.73	148.73	148.73
	W1CV	soil void for concrete collar - north (NOBT)	-16.0	-137.5	-8.6	-7.4	-7.4	-7.36	-7.36	-7.36
	W1D	soil vertical load above base extension - GWsouth	35.8	308.2	19.3	16.5	16.5	16.49	16.49	16.49
	W1E	soil vertical load above base extension - GWnorth	35.8	308.2	19.3	16.5	16.5	16.49	16.49	16.49
	W1F	soil vertical load above base extension - west (lower below collar)	48.6	419.2	26.2	22.4	22.4	22.43	22.43	22.43
	W1F2	soil vertical load above base extension - west (upper below collar)	81.8	705.2	44.1	37.7	37.7	37.73	37.73	37.73

Wc							**852.1**	**852.1**	**852.1**	**852.1**
	W14A	water vertical load - 4' in sump pit			4.0		4.0	4.00	4.00	4.00
	W2	water vertical load in gatewell			309.3		309.3	309.34	309.34	309.34
	W14B	water vertical load station, south chamber			233.8		233.8	233.75	233.75	233.75
	W14C	water vertical load station, north chamber			305.0		305.0	305.01	305.01	305.01
S							**0.0**	**0.0**	**0.0**	**0.0**
							0.0	0.00	0.00	0.00
U							**5667.4**	**4858.47**	**4709.12**	**4579.46**
Wg							**580.9**	**580.9**	**580.9**	**580.9**
Water portion of:										
	W1A	soil vertical load above base extension - east			154.4		154.4	154.38	154.38	154.38
	W1AV	soil void for concrete collar - east (EABT)			-13.0		-13.0	-13.00	-13.00	-13.00
	W1B	soil vertical load above base extension - south			173.8		173.8	173.76	173.76	173.76
	W1BV	soil void for concrete collar - south (SOBT)			-8.2		-8.2	-8.22	-8.22	-8.22
	W1C	soil vertical load above base extension - north			173.8		173.8	173.76	173.76	173.76
	W1CV	soil void for concrete collar - north (NOBT)			-8.6		-8.6	-8.59	-8.59	-8.59
	W1D	soil vertical load above base extension - GWsouth			19.3		19.3	19.26	19.26	19.26
	W1E	soil vertical load above base extension - GWnorth			19.3		19.3	19.26	19.26	19.26
	W1F	soil vertical load above base extension - west (lower below collar)			26.2		26.2	26.20	26.20	26.20
	W1F2	soil vertical load above base extension - west (upper below collar)			44.1		44.1	44.07	44.07	44.07

TURNER PUMP STATION - 500 + 3 - FLOOD										
UPLIFT CONDITION B - Assumes 0 ft of water in pump station - DRY STATION										
FSf =	Ws + Wc + S									
	(U-Wg)									
Ws =	Weight of structure (including wieghts of fixed equipment, soil above top surface of structure)									
Wc =	Weight of water contained in structure (controlled by mechanical)									
S =	surcharge loads									
U =	uplift forces acting on structure									
Wg =	weight of water above top surface of the structure (controlled by gravity)									
			total	vol	water	buoyant				
			wt		wt	soil wt				
			(k)	(ft^3)	(k)	(k)				
							LOAD CASE B	**LOAD CASE C**	**LOAD CASE D**	**LOAD CASE E**
FSf =							**-0.9**	**-0.86**	**-0.86**	**-0.86**
Ws + Wc + S							497.2	497.23	497.23	497.23
(U-Wg)							-580.9	-580.88	-580.88	-580.88
Ws							**497.2**	**497.2**	**497.2**	**497.2**
Structure + Equip										
	Existing Station		0.0				0.0	0.00	0.00	0.00
	Gatewell Extension		0.0				0.0	0.00	0.00	0.00
Soil Portion of:										
	W1A	soil vertical load above base extension - east	286.5	2470.1	154.4	132.1	132.1	132.15	132.15	132.15
	W1AV	soil void for concrete collar - east (EABT)	-24.1	-208.0	-13.0	-11.1	-11.1	-11.13	-11.13	-11.13
	W1B	soil vertical load above base extension - south	322.5	2780.1	173.8	148.7	148.7	148.73	148.73	148.73
	W1BV	soil void for concrete collar - south (SOBT)	-15.3	-131.5	-8.2	-7.0	-7.0	-7.03	-7.03	-7.03
	W1C	soil vertical load above base extension - north	322.5	2780.1	173.8	148.7	148.7	148.73	148.73	148.73
	W1CV	soil void for concrete collar - north (NOBT)	-16.0	-137.5	-8.6	-7.4	-7.4	-7.36	-7.36	-7.36
	W1D	soil vertical load above base extension - GWsouth	35.8	308.2	19.3	16.5	16.5	16.49	16.49	16.49
	W1E	soil vertical load above base extension - GWnorth	35.8	308.2	19.3	16.5	16.5	16.49	16.49	16.49
	W1F	soil vertical load above base extension - west (lower below collar)	48.6	419.2	26.2	22.4	22.4	22.43	22.43	22.43
	W1F2	soil vertical load above base extension - west (upper below collar)	81.8	705.2	44.1	37.7	37.7	37.73	37.73	37.73

Wc							**0.0**	**0.0**	**0.0**	**0.0**
	W14A	water vertical load - 4' in sump pit			4.0					
	W2	water vertical load in gatewell			309.3					
	W14B	water vertical load station, south chamber			233.8					
	W14C	water vertical load station, north chamber			305.0					
S							**0.0**	**0.0**	**0.0**	**0.0**
							0.0	0.00	0.00	0.00
U							**0.0**	**0.00**	**0.00**	**0.00**
Wg							**580.9**	**580.9**	**580.9**	**580.9**
Water portion of:										
	W1A	soil vertical load above base extension - east			154.4		154.4	154.38	154.38	154.38
	W1AV	soil void for concrete collar - east (EABT)			-13.0		-13.0	-13.00	-13.00	-13.00
	W1B	soil vertical load above base extension - south			173.8		173.8	173.76	173.76	173.76
	W1BV	soil void for concrete collar - south (SOBT)			-8.2		-8.2	-8.22	-8.22	-8.22
	W1C	soil vertical load above base extension - north			173.8		173.8	173.76	173.76	173.76
	W1CV	soil void for concrete collar - north (NOBT)			-8.6		-8.6	-8.59	-8.59	-8.59
	W1D	soil vertical load above base extension - GWsouth			19.3		19.3	19.26	19.26	19.26
	W1E	soil vertical load above base extension - GWnorth			19.3		19.3	19.26	19.26	19.26
	W1F	soil vertical load above base extension - west (lower below collar)			26.2		26.2	26.20	26.20	26.20
	W1F2	soil vertical load above base extension - west (upper below collar)			44.1		44.1	44.07	44.07	44.07

TURNER PUMP STATION - 500 + 3 - FLOOD
UPLIFT CONDITION A - Assumes 8 ft of water in pump station

FSf = Ws + Wc + S / (U-Wg)

Ws = Weight of structure (including wieghts of fixed equipment, soil above top surface of structure)
Wc = Weight of water contained in structure (controlled by mechanical)
S = surcharge loads
U = uplift forces acting on structure
Wg = weight of water above top surface of the structure (controlled by gravity)

			total																			
			wt																			
Average Distance HGL above grade (ft)			(k)				0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
													LOAD CASE E	**LOAD CASE D**	**LOAD CASE C**							**LOAD CASE B**
FSf =							**1.39**	**1.34**	**1.30**	**1.25**	**1.21**	**1.18**	**1.14**	**1.11**	**1.08**	**1.05**	**1.02**	**0.99**	**0.97**	**0.94**	**0.92**	**0.90**
Ws + Wc + S							4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7	4598.7
(U-Wg)							3307.5	3427.9	3548.3	3668.7	3789.1	3909.6	4030.0	4150.4	4270.8	4391.2	4511.7	4632.1	4752.5	4872.9	4993.3	5113.8
Ws							**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**
Structure + Equip																						
Existing Station			3206.4				3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4
Gatewell Extension			43.0				43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0
				vol	water wt	buoyant soil wt																
Soil Portion of:				(ft^3)	(k)	(k)																
	W1A	soil vertical load above base extension - east	286.5	2470.1	154.4	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1
	W1AV	soil void for concrete collar - east (EABT)	-24.1	-208.0	-13.0	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1
	W1B	soil vertical load above base extension - south	322.5	2780.1	173.8	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7
	W1BV	soil void for concrete collar - south (SOBT)	-15.3	-131.5	-8.2	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0
	W1C	soil vertical load above base extension - north	322.5	2780.1	173.8	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7
	W1CV	soil void for concrete collar - north (NOBT)	-16.0	-137.5	-8.6	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4
	W1D	soil vertical load above base extension - GWsouth	35.8	308.2	19.3	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
	W1E	soil vertical load above base extension - GWnorth	35.8	308.2	19.3	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
	W1F	soil vertical load above base extension - west (lower below collar)	48.6	419.2	26.2	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4
	W1F2	soil vertical load above base extension - west (upper below collar)	81.8	705.2	44.1	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7
Wc							**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**
	W14A	water vertical load - 4' in sump pit			4.0		4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	W2	water vertical load in gatewell			309.3		309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3
	W14B	water vertical load station, south chamber			233.8		233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8
	W14C	water vertical load station, north chamber			305.0		305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0
S							**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
							0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
U							**3888.3**	**4008.8**	**4129.2**	**4249.6**	**4370.0**	**4490.4**	**4610.9**	**4731.3**	**4851.7**	**4972.1**	**5092.5**	**5213.0**	**5333.4**	**5453.8**	**5574.2**	**5694.6**
		Distance HGL above grade (ft)					0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
		Top of Grade	762.0				762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0
		HGL EL.					762.0	763.0	764.0	765.0	766.0	767.0	768.0	769.0	770.0	771.0	772.0	773.0	774.0	775.0	776.0	777.0
		EL. Bot of PS	729.7				729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7
		AVG uplift					2.02	2.08	2.14	2.21	2.27	2.33	2.39	2.46	2.52	2.58	2.64	2.71	2.77	2.83	2.89	2.96
		Area base	1926.7				1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7
		UPLIFT					3888.3	4008.8	4129.2	4249.6	4370.0	4490.4	4610.9	4731.3	4851.7	4972.1	5092.5	5213.0	5333.4	5453.8	5574.2	5694.6
Wg							**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**
Water portion of:																						
	W1A	soil vertical load above base extension - east			154.4		154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4
	W1AV	soil void for concrete collar - east (EABT)			-13.0		-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0
	W1B	soil vertical load above base extension - south			173.8		173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8
	W1BV	soil void for concrete collar - south (SOBT)			-8.2		-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2
	W1C	soil vertical load above base extension - north			173.8		173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8
	W1CV	soil void for concrete collar - north (NOBT)			-8.6		-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6
	W1D	soil vertical load above base extension - GWsouth			19.3		19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3
	W1E	soil vertical load above base extension - GWnorth			19.3		19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3
	W1F	soil vertical load above base extension - west (lower below collar)			26.2		26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2
	W1F2	soil vertical load above base extension - west (upper below collar)			44.1		44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1

TURNER PUMP STATION - 500 + 3 - FLOOD
UPLIFT CONDITION B - Assumes 0 ft of water in pump station - DRY STATION

FSf = Ws + Wc + S / (U-Wg)

Ws = Weight of structure (including wieghts of fixed equipment, soil above top surface of structure)
Wc = Weight of water contained in structure (controlled by mechanical)
S = surcharge loads
U = uplift forces acting on structure
Wg = weight of water above top surface of the structure (controlled by gravity)

			total																			
			wt																			
Average Distance HGL above grade (ft)			(k)				0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
													LOAD CASE E	**LOAD CASE D**	**LOAD CASE C**							**LOAD CASE B**
FSf =							**1.13**	**1.09**	**1.06**	**1.02**	**0.99**	**0.96**	**0.93**	**0.90**	**0.88**	**0.85**	**0.83**	**0.81**	**0.79**	**0.77**	**0.75**	**0.73**
Ws + Wc + S							3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6	3746.6
(U-Wg)							3307.5	3427.9	3548.3	3668.7	3789.1	3909.6	4030.0	4150.4	4270.8	4391.2	4511.7	4632.1	4752.5	4872.9	4993.3	5113.8
Ws							**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**	**3746.6**
Structure + Equip																						
Existing Station			3206.4				3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4
Gatewell Extension			43.0				43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0
				vol	water wt	buoyant soil wt																
Soil Portion of:				(ft^3)	(k)	(k)																
	W1A	soil vertical load above base extension - east	286.5	2470.1	154.4	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1
	W1AV	soil void for concrete collar - east (EABT)	-24.1	-208.0	-13.0	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1
	W1B	soil vertical load above base extension - south	322.5	2780.1	173.8	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7
	W1BV	soil void for concrete collar - south (SOBT)	-15.3	-131.5	-8.2	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0
	W1C	soil vertical load above base extension - north	322.5	2780.1	173.8	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7
	W1CV	soil void for concrete collar - north (NOBT)	-16.0	-137.5	-8.6	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4
	W1D	soil vertical load above base extension - GWsouth	35.8	308.2	19.3	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
	W1E	soil vertical load above base extension - GWnorth	35.8	308.2	19.3	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
	W1F	soil vertical load above base extension - west (lower below collar)	48.6	419.2	26.2	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4
	W1F2	soil vertical load above base extension - west (upper below collar)	81.8	705.2	44.1	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7
Wc							**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
	W14A	water vertical load - 4' in sump pit					0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	W2	water vertical load in gatewell					0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	W14B	water vertical load station, south chamber					0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	W14C	water vertical load station, north chamber					0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
S							**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
							0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
U							**3888.3**	**4008.8**	**4129.2**	**4249.6**	**4370.0**	**4490.4**	**4610.9**	**4731.3**	**4851.7**	**4972.1**	**5092.5**	**5213.0**	**5333.4**	**5453.8**	**5574.2**	**5694.6**
		Distance HGL above grade (ft)					0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
		Top of Grade	762.0				762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0
		HGL EL.					762.0	763.0	764.0	765.0	766.0	767.0	768.0	769.0	770.0	771.0	772.0	773.0	774.0	775.0	776.0	777.0
		EL. Bot of PS	729.7				729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7
		AVG uplift					2.02	2.08	2.14	2.21	2.27	2.33	2.39	2.46	2.52	2.58	2.64	2.71	2.77	2.83	2.89	2.96
		Area base	1926.7				1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7
		UPLIFT					3888.3	4008.8	4129.2	4249.6	4370.0	4490.4	4610.9	4731.3	4851.7	4972.1	5092.5	5213.0	5333.4	5453.8	5574.2	5694.6
Wg							**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**
Water portion of:																						
	W1A	soil vertical load above base extension - east			154.4		154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4
	W1AV	soil void for concrete collar - east (EABT)			-13.0		-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0
	W1B	soil vertical load above base extension - south			173.8		173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8
	W1BV	soil void for concrete collar - south (SOBT)			-8.2		-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2
	W1C	soil vertical load above base extension - north			173.8		173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8
	W1CV	soil void for concrete collar - north (NOBT)			-8.6		-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6
	W1D	soil vertical load above base extension - GWsouth			19.3		19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3
	W1E	soil vertical load above base extension - GWnorth			19.3		19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3
	W1F	soil vertical load above base extension - west (lower below collar)			26.2		26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2
	W1F2	soil vertical load above base extension - west (upper below collar)			44.1		44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1

TURNER PUMP STATION LOAD CASE 00 - EXISTING CONDITION - FLOOD
UPLIFT CONDITION A - Assumes 8 ft of water in pump station

FSf = Ws + Wc + S / (U-Wg)

Ws = Weight of structure (including wieghts of fixed equipment, soil above top surface of structure)
Wc = Weight of water contained in structure (controlled by mechanical)
S = surcharge loads
U = uplift forces acting on structure
Wg = weight of water above top surface of the structure (controlled by gravity)

distance HGL above grade (ft)						**0.0**	**1.0**	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**	**15.0**	**16.0**
FSf =						**1.38**	**1.33**	**1.28**	**1.24**	**1.20**	**1.17**	**1.13**	**1.10**	**1.07**	**1.04**	**1.01**	**0.98**	**0.96**	**0.93**	**0.91**	**0.89**	**0.87**
Ws + Wc + S						4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8	4555.8
(U-Wg)						3307.5	3427.9	3548.3	3668.7	3789.1	3909.6	4030.0	4150.4	4270.8	4391.2	4511.7	4632.1	4752.5	4872.9	4993.3	5113.8	5234.2
Ws						**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**
			total wt (k)	vol (ft^3)	water wt (k)	buoyant soil wt (k)																
Structure + Equip																						
Existing Station			3206.4				3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4
Gatewell Extension			43.0																			
Soil Portion of:																						
	W1A	soil vertical load above base extension - east	286.5	2470.1	154.4	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1
	W1AV	soil void for concrete collar - east (EABT)	-24.1	-208.0	-13.0	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1
	W1B	soil vertical load above base extension - south	322.5	2780.1	173.8	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7
	W1BV	soil void for concrete collar - south (SOBT)	-15.3	-131.5	-8.2	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0
	W1C	soil vertical load above base extension - north	322.5	2780.1	173.8	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7
	W1CV	soil void for concrete collar - north (NOBT)	-16.0	-137.5	-8.6	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4
	W1D	soil vertical load above base extension - GWsouth	35.8	308.2	19.3	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
	W1E	soil vertical load above base extension - GWnorth	35.8	308.2	19.3	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
	W1F	soil vertical load above base extension - west (lower below collar)	48.6	419.2	26.2	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4
	W1F2	soil vertical load above base extension - west (upper below collar)	81.8	705.2	44.1	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7

Wc							**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**	**852.1**
	W14A	water vertical load - 4' in sump pit			4.0		4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	W2	water vertical load in gatewell			309.3		309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3	309.3
	W14B	water vertical load station, south chamber			233.8		233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8	233.8
	W14C	water vertical load station, north chamber			305.0		305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0	305.0
S							**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
						0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
U							**3888.3**	**4008.8**	**4129.2**	**4249.6**	**4370.0**	**4490.4**	**4610.9**	**4731.3**	**4851.7**	**4972.1**	**5092.5**	**5213.0**	**5333.4**	**5453.8**	**5574.2**	**5694.6**	**5815.1**
		Distance HGL above grade (ft)					0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
		Top of Grade	762.0				762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0
		HGL EL.					762.0	763.0	764.0	765.0	766.0	767.0	768.0	769.0	770.0	771.0	772.0	773.0	774.0	775.0	776.0	777.0	778.0
		EL. Bot of PS	729.7				729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7
		AVG uplift					2.02	2.08	2.14	2.21	2.27	2.33	2.39	2.46	2.52	2.58	2.64	2.71	2.77	2.83	2.89	2.96	3.02
		Area base	1926.7				1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7
		UPLIFT					3888.3	4008.8	4129.2	4249.6	4370.0	4490.4	4610.9	4731.3	4851.7	4972.1	5092.5	5213.0	5333.4	5453.8	5574.2	5694.6	5815.1
Wg							**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**
Water portion of:																							
	W1A	soil vertical load above base extension - east			154.4		154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4
	W1AV	soil void for concrete collar - east (EABT)			-13.0		-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0
	W1B	soil vertical load above base extension - south			173.8		173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8
	W1BV	soil void for concrete collar - south (SOBT)			-8.2		-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2
	W1C	soil vertical load above base extension - north			173.8		173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8
	W1CV	soil void for concrete collar - north (NOBT)			-8.6		-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6
	W1D	soil vertical load above base extension - GWsouth			19.3		19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3
	W1E	soil vertical load above base extension - GWnorth			19.3		19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3
	W1F	soil vertical load above base extension - west (lower below collar)			26.2		26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2
	W1F2	soil vertical load above base extension - west (upper below collar)			44.1		44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1

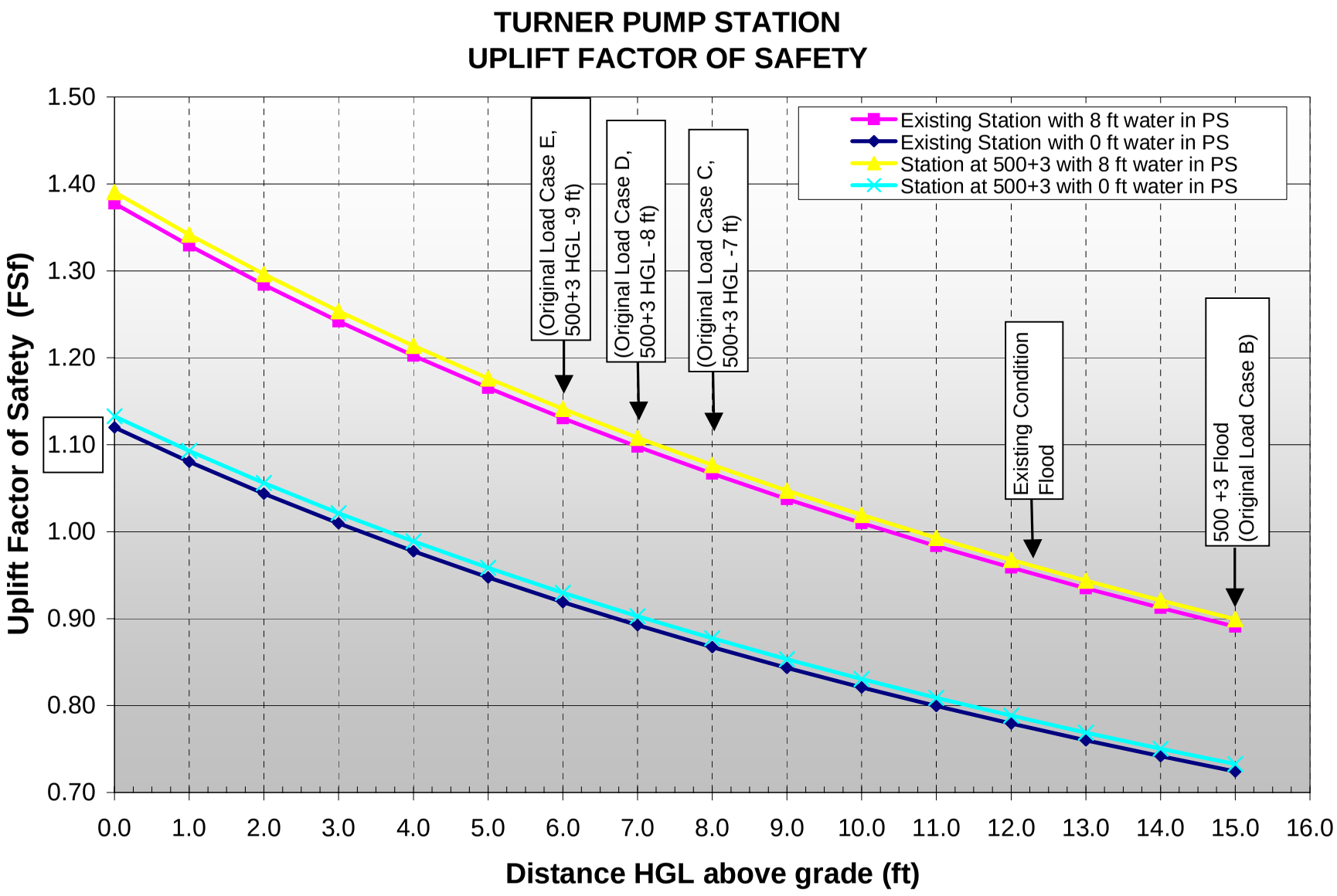
TURNER PUMP STATION LOAD CASE 00 - EXISTING CONDITION - FLOOD
UPLIFT CONDITION B - Assumes 0 ft of water in pump station - DRY STATION

FSf = Ws + Wc + S / (U-Wg)

Ws = Weight of structure (including wieghts of fixed equipment, soil above top surface of structure)
Wc = Weight of water contained in structure (controlled by mechanical)
S = surcharge loads
U = uplift forces acting on structure
Wg = weight of water above top surface of the structure (controlled by gravity)

distance HGL above grade (ft)						**0.0**	**1.0**	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**	**11.0**	**12.0**	**13.0**	**14.0**	**15.0**	**16.0**
FSf =						**1.12**	**1.08**	**1.04**	**1.01**	**0.98**	**0.95**	**0.92**	**0.89**	**0.87**	**0.84**	**0.82**	**0.80**	**0.78**	**0.76**	**0.74**	**0.72**	**0.71**
Ws + Wc + S						3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7	3703.7
(U-Wg)						3307.5	3427.9	3548.3	3668.7	3789.1	3909.6	4030.0	4150.4	4270.8	4391.2	4511.7	4632.1	4752.5	4872.9	4993.3	5113.8	5234.2
Ws						**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**	**3703.7**
			total wt (k)	vol (ft^3)	water wt (k)	buoyant soil wt (k)																
			0.0	0.0	0.0	0.0																
Structure + Equip			0.0	0.0	0.0	0.0																
Existing Station			3206.4	0.0	0.0	0.0	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4	3206.4
Gatewell Extension			43.0	0.0	0.0	0.0																
			0.0	0.0	0.0	0.0																
Soil Portion of:			0.0	0.0	0.0	0.0																
	W1A	soil vertical load above base extension - east	286.5	2470.1	154.4	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1	132.1
	W1AV	soil void for concrete collar - east (EABT)	-24.1	-208.0	-13.0	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1	-11.1
	W1B	soil vertical load above base extension - south	322.5	2780.1	173.8	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7
	W1BV	soil void for concrete collar - south (SOBT)	-15.3	-131.5	-8.2	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0	-7.0
	W1C	soil vertical load above base extension - north	322.5	2780.1	173.8	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7	148.7
	W1CV	soil void for concrete collar - north (NOBT)	-16.0	-137.5	-8.6	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4	-7.4
	W1D	soil vertical load above base extension - GWsouth	35.8	308.2	19.3	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
	W1E	soil vertical load above base extension - GWnorth	35.8	308.2	19.3	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
	W1F	soil vertical load above base extension - west (lower below collar)	48.6	419.2	26.2	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4	22.4
	W1F2	soil vertical load above base extension - west (upper below collar)	81.8	705.2	44.1	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7	37.7
			0.0	0.0	0.0	0.0																

Wc		0.0	0.0	0.0	0.0	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
		0.0	0.0	0.0	0.0																	
W14A	water vertical load - 4' in sump pit	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
W2	water vertical load in gatewell	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
W14B	water vertical load station, south chamber	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
W14C	water vertical load station, north chamber	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		0.0	0.0	0.0	0.0																	
		0.0	0.0	0.0	0.0																	
S		0.0	0.0	0.0	0.0	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		0.0	0.0	0.0	0.0																	
U		0.0	0.0	0.0	0.0	**3888.3**	**4008.8**	**4129.2**	**4249.6**	**4370.0**	**4490.4**	**4610.9**	**4731.3**	**4851.7**	**4972.1**	**5092.5**	**5213.0**	**5333.4**	**5453.8**	**5574.2**	**5694.6**	**5815.1**
	Distance HGL above grade (ft)	0.0	0.0	0.0	0.0	0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
	Top of Grade	762.0	0.0	0.0	0.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0	762.0
	HGL EL.	0.0	0.0	0.0	0.0	762.0	763.0	764.0	765.0	766.0	767.0	768.0	769.0	770.0	771.0	772.0	773.0	774.0	775.0	776.0	777.0	778.0
	EL. Bot of PS	729.7	0.0	0.0	0.0	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7	729.7
	AVG uplift	0.0	0.0	0.0	0.0	2.02	2.08	2.14	2.21	2.27	2.33	2.39	2.46	2.52	2.58	2.64	2.71	2.77	2.83	2.89	2.96	3.02
	Area base	1926.7	0.0	0.0	0.0	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7	1926.7
	UPLIFT	0.0	0.0	0.0	0.0	3888.3	4008.8	4129.2	4249.6	4370.0	4490.4	4610.9	4731.3	4851.7	4972.1	5092.5	5213.0	5333.4	5453.8	5574.2	5694.6	5815.1
		0.0	0.0	0.0	0.0																	
		0.0	0.0	0.0	0.0																	
Wg		0.0	0.0	0.0	0.0	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**	**580.9**
		0.0	0.0	0.0	0.0																	
Water portion of:		0.0	0.0	0.0	0.0																	
W1A	soil vertical load above base extension - east	0.0	0.0	154.4	0.0	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4	154.4
W1AV	soil void for concrete collar - east (EABT)	0.0	0.0	-13.0	0.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0	-13.0
W1B	soil vertical load above base extension - south	0.0	0.0	173.8	0.0	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8
W1BV	soil void for concrete collar - south (SOBT)	0.0	0.0	-8.2	0.0	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2	-8.2
W1C	soil vertical load above base extension - north	0.0	0.0	173.8	0.0	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8	173.8
W1CV	soil void for concrete collar - north (NOBT)	0.0	0.0	-8.6	0.0	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6	-8.6
W1D	soil vertical load above base extension - GWsouth	0.0	0.0	19.3	0.0	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3
W1E	soil vertical load above base extension - GWnorth	0.0	0.0	19.3	0.0	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3
W1F	soil vertical load above base extension - west (lower below collar)	0.0	0.0	26.2	0.0	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2	26.2
W1F2	soil vertical load above base extension - west (upper below collar)	0.0	0.0	44.1	0.0	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1	44.1



_Turner Stability.v3.xls
UPLIFT PLOT

SECTION 2.
Turner Pump Station
STRENGTH

Levee Unit: Argentine
Pump Station: Turner Ave. (Kaw Valley No 4)
Levee Station: 60+40
Flood Event: 500+3
Proposed Protection: EL. 780.8

Strength Investigation of Exterior Walls & Collar Beam

(Revised 11/2004 due to KC Review [v2])
(Revised 1/2005 after KC Summit #1 12/1/04 [v3])

NOTES:

EXTERIOR WALL

1. Loading. The current approach is to install a new wall on the levee crown to achieve the with-project 500+3 protection adjacent to the pump station. This is being done for purposes of deloading the box culvert beneath the levee. Therefore assume that no additional soil loads on the structure. Loading change for the structure will be due to increased water pressures only.

2. The pressure change due to increased water loads for the with-project 500+3 condition will be the greatest at the lower elevations near the bottom of the station. Therefore investigate the lower portion of the walls as they will be considered the critical location. The exterior walls will see the greatest load due to the existing soil pressures and the increased water pressure from the proposed hydraulic grade line. The investigations of the lower wall assumed full HGL. Soil pressures were computed using buoyant soil weights and a conservative value for at-rest lateral coefficient, Ko of 1.0.

3. Strength Investigation. The section of wall investigated in a section approximately at the third point on the south side of the station, which is approximately at the location of section 2 as shown on the OM&M plates. Investigate the lower portion of the south exterior wall. This wall also carries vertical axial load from the superstructure (building) of the pump station. Interaction of flexural and axial loading was also investigated. Assume that the lower portion of the wall spans vertically from the base to the collar elevation. This agrees with the fact that the main reinforcing steel in the walls in located vertically on the far inside and outside faces. The wall was investigated assuming a simple 1 foot strip beam approach. The tension steel is 1-inch square bars at 8-inch spacing the clear cover is assumed to be 2.5 inches.

   Initially a strength design approach to the investigation was used. Strength investigation is per EM 1110-2-2104. Per the EM, load factors used include the single load factor of 1.7, the hydraulic load factor of 1.3 and the extreme event factor of 0.75. The member was modeled in CASE program CFRAME to determine internal moments and shears developed from lateral loading.

   After KC summit #1 12/1/04, an unfactored, factor of safety approach was used.

4. Material Properties. Material properties for all wall segments assumed to be f'c=3.75 ksi and fy=40 ksi.

5. Member Assumptions. For simplicity, the wall is assumed to span vertically and is investigated as a single span 1-foot strip beam with fixed ends. The tension steel is 1-inch square bars at 8-inch spacing the clear cover is assumed to be 2.5 inches.

6. Results.
   FACTORED. Results of strength investigation for the moment and shear capacities are shown on the attached Excel spreadsheet/table TABLE 2-1. The wall is adequate to accommodate the lateral loads for moment and shear. The wall was also investigated for interaction of axial load and flexure and is shown in the attached hand computation. The result of the interaction check indicates the wall is acceptable.

UNFACTORED. Also, after the KC summit #1, 12/1/04, the unfactored factor of safety approach also indicated that the wall was adequate. The factors of safety of flexure and shear were both above 1.5. Results for this approach are also shown in the TABLE 2-1.

Note this member does meet ACI 10.5.1 minimum for tension steel.

COLLAR BEAM

1. Subsequently, since it was assumed that the lower portion of the wall spans vertically from base to collar location, the collar beam was also investigated for increased loads.

2. Loading. The investigations of the collar beam assumed full HGL. Soil pressures were computed using buoyant soil weights and a conservative value for at-rest lateral coefficient, Ko of 1.0. It was assumed that the collar beam carries load from approximately 3 feet each side of the beam, i.e. the tributary width of the beam was assumed to be the width of the beam itself plus three feet above and below the beam.

3. Strength Investigation. The collar beam along the south wall was investigated. . Strength investigation is per EM 1110-2-2104. Per the EM, load factors used include the single load factor of 1.7, the hydraulic load factor of 1.3 and the extreme event factor of 0.75. The member was modeled in CASE program CFRAME to determine internal moments and shears developed from lateral loading.

   After KC summit #1 12/1/04, an unfactored, factor of safety approach was used.

4. Material Properties. Material properties assumed to be f'c=3.75 ksi and fy=40 ksi.

5. Member Assumptions. For simplicity, the collar beam is assumed to span horizontally and is investigated as a three-span span continuous beam with fixed ends. The beam is modeled as shown in the OM&M plates. The beam is essentially 3 ft by 3ft with 2 rows of seven 1 1/8-inch square bars at each vertical face. The clear cover is assumed to be 3 inches.

6. Results.

   FACTORED. Results of strength investigation for the moment and shear capacities are shown on the attached Excel spreadsheet/table **TABLE 2-2**. The wall is adequate to accommodate the lateral loads for moment. However, for shear the beam appears to be inadequate. Note that only member 1 of the modeled beam was checked for shear assuming it was the controlling case due it having the largest span. The contribution of shear steel, 5/8-inch stirrups at the spacing indicated on OM&M plate 74, was taken in to account, and the factored shear loads still exceeded the shear capacity of the member.

   The shear loads used and summarized in TABLE 2-2 were conservatively based upon water pressures resulting from the full HGL. For floatation stability of the station, the HGL will be draw down by relief wells by a recommended minimum 8 feet. Therefore an additional check of the shear in the collar beam was made using a reduced HGL. The results of this check are included in the excel spreadsheet/table **TABLE 2-3**. It appear from this table that the overall shear capacity, taking into account the contribution of the concrete and the shear steel, that this member is adequate in shear within the clear span region of the member.

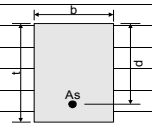
   UNFACTORED. Also, after the KC summit #1, 12/1/04, the unfactored factor of safety approach also indicated that the beam was adequate. The factors of safety of flexure and shear were both above 1.5. Results for this approach are also shown in the TABLE 2-2.

   Note this member does meet ACI 10.5.1 minimum for tension steel.

Comp by SMH
Checked by ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40 **TABLE 2-1**
Description/Info:
500+3, exterior wall strength check
Assume 1ft strip; Beam = 12" x 24" with 1" sq bar @ 8" o.c. (As=1.5 sq in.) each face

Investigation of Moment and Shear Capacity per EM 1110-2-2104



	Strength Design Checks using LRFD		Strength Checks with UNFACTORED LOADS & Safety Factors	
	Full HGL		Full HGL	
Item/member description	**Lwr wall in face**	**Lwr wall out face**	**Lwr wall in face**	**Lwr wall out face**
Max Moment from CFRAME (k-in)	204.4	415.6	204.4	415.6
Max shear from CFRAME (k)	18.18	19.65	18.18	19.65
Material Properties				
f'c (ksi)	3.75	3.75	3.75	3.75
fy (ksi)	40	40	40	40
Es (ksi)	29000	29000	29000	29000
Member Properties				
span(ft)	10.75	10.75	10.75	10.75
b (in)	12	12	12	12
Member thickness (in)	24	24	24	24
cover+1/2db, d' (estimate) (in)	3	3	3	3
As (in^2)	1.5	1.5	1.5	1.5
Loading				
Max Moment, M (ft-k)	17.03	34.63	17.03	34.63
Max Shear, V (k)	18.18	19.65	18.18	19.65
Single Load Factor	1.70	1.70	1.00	1.00
Hydraulic Load Factor	1.30	1.30	1.00	1.00
Extreme Case load Factor	0.75	0.75	1.00	1.00
Max Mu (ft-k)	28.23	57.40	17.03	34.63
Max Vu (k)	30.13	32.57	18.18	19.65
Check Steel Yield				
d (in)	21.00	21.00	21.00	21.00
a (in)	1.57	1.57	1.57	1.57
c (in)	1.85	1.85	1.85	1.85
es	0.0311	0.0311	0.0311	0.0311
ey	0.0014	0.0014	0.0014	0.0014
Steel yield ?	YES	YES	YES	YES
Check Flexure				
phiMn (ft-k)	90.97	90.97	101.08	101.08
phiMn > Mu ?	OK	OK	OK	OK
Factor of Safety, FS (Mn/Mmax)			5.93	2.92
FS>1.5 ?			YES	YES
Check Shear per ACI				
phiVc (k) (ACI)	26.23	26.23	30.86	30.86
phiVc > Max Vu ?	NO	NO	YES	YES
Factor of Safety, FS (Vn/Vmax)			1.70	1.57
FS>1.5 ?			YES	YES
Check Shear at distance d from support				
Vu at distance d	20.32	21.97	12.26	13.25
phiVc > Vu ?	YES	YES	YES	YES
Factor of Safety, FS (Vn/Vmax)			2.52	2.33
FS>1.5 ?			YES	YES
Check Steel Ratio				
Beta1	0.85	0.85	0.85	0.85
rho bal	0.04640	0.04640	0.04640	0.04640
rho actual	0.00595	0.00595	0.00595	0.00595
Recommended limit, 0.25*rho bal	0.01160	0.01160	0.01160	0.01160
rho act < recommended limit?	YES	YES	YES	YES
Maximum permitted limit, 0.375*rho bal	0.01740	0.01740	0.01740	0.01740
rho act < MAX rho?	YES	YES	YES	YES
Check ACI Minimums				
ACI 318 10.5.1 minimum (3f'c^0.5/fy)bd	1.16	1.16	1.16	1.16
As> min ACI 10.5.1 ?	YES	YES	YES	YES
ACI 318 10.5.1 minimum (200/fy)bd	1.26	1.26	1.26	1.26
As> min ACI 10.5.1 ?	YES	YES	YES	YES

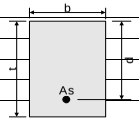
Comp by SMH
Checked by ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40

TABLE 2-2

Description/Info:
500+3, collar beam strength check
3' x3' beam with 7-1 1/8-inch square bars in each vertical face (As=8.86 sq in)

Investigation of Moment and Shear Capacity per EM 1110-2-2104



	Strength Design Checks using LRFD						Strength Checks with UNFACTORED LOADS & Safety Factors					
	Full HGL		Full HGL		Full HGL		Full HGL		Full HGL		Full HGL	
Item/member description	member 1 in face	member 1 out face	member 2 in face	member 2 out face	member 3 in face	member 3 out face	member 1 in face	member 1 out face	member 2 in face	member 2 out face	member 3 in face	member 3 out face
Max Moment from CFRAME (k-in)	2369	4732	2260	4638	2331	4657	2369	4732	2260	4638	2331	4657
Max shear from CFRAME (k)	175.8	175.8	173.1	173.1	174.4	174.4	175.8	175.8	173.1	173.1	174.4	174.4
Material Properties												
f'c (ksi)	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75
fy (ksi)	40	40	40	40	40	40	40	40	40	40	40	40
Es (ksi)	29000	29000	29000	29000	29000	29000	29000	29000	29000	29000	29000	29000
Member Properties												
span(ft)	13.42	13.42	13.25	13.25	13.33	13.33	13.42	13.42	13.25	13.25	13.33	13.33
b (in)	36	36	36	36	36	36	36	36	36	36	36	36
Member thickness (in)	36	36	36	36	36	36	36	36	36	36	36	36
cover+1/2db, d' (estimate) (in)	3	3	3	3	3	3	3	3	3	3	3	3
As (in^2)	8.86	8.86	8.86	8.86	8.86	8.86	8.86	8.86	8.86	8.86	8.86	8.86
Loading												
Max Moment, M (ft-k)	197.42	394.33	188.33	386.50	194.25	388.08	197.42	394.33	188.33	386.50	194.25	388.08
Max Shear, V (k)	175.8	175.8	173.1	173.1	174.4	174.4	175.8	175.8	173.1	173.1	174.4	174.4
Single Load Factor	1.70	1.70	1.70	1.70	1.70	1.70	1.00	1.00	1.00	1.00	1.00	1.00
Hydraulic Load Factor	1.30	1.30	1.30	1.30	1.30	1.30	1.00	1.00	1.00	1.00	1.00	1.00
Extreme Case load Factor	0.75	0.75	0.75	0.75	0.75	0.75	1.00	1.00	1.00	1.00	1.00	1.00
Max Mu (ft-k)	327.22	653.61	312.16	640.62	321.97	643.25	197.42	394.33	188.33	386.50	194.25	388.08
Max Vu (k)	291.39	291.39	286.91	286.91	289.07	289.07	175.80	175.80	173.10	173.10	174.40	174.40
Check Steel Yield												
d (in)	33.00	33.00	33.00	33.00	33.00	33.00	33.00	33.00	33.00	33.00	33.00	33.00
a (in)	3.09	3.09	3.09	3.09	3.09	3.09	3.09	3.09	3.09	3.09	3.09	3.09
c (in)	3.63	3.63	3.63	3.63	3.63	3.63	3.63	3.63	3.63	3.63	3.63	3.63
es	0.0242	0.0242	0.0242	0.0242	0.0242	0.0242	0.0242	0.0242	0.0242	0.0242	0.0242	0.0242
ey	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014
Steel yield ?	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
Check Flexure												
phiMn (ft-k)	836.09	836.09	836.09	836.09	836.09	836.09	928.99	928.99	928.99	928.99	928.99	928.99
phiMn > Mu ?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Factor of Safety, FS (Mn/Mmax)							4.71	2.36	4.93	2.40	4.78	2.39
FS>1.5 ?							YES	YES	YES	YES	YES	YES
Check Shear per ACI												
phiVc (k) (ACI)	123.67	123.67	123.67	123.67	123.67	123.67	145.50	145.50	145.50	145.50	145.50	145.50
phiVc > Max Vu ?	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
Factor of Safety, FS (Vn/Vmax)							0.83	0.83	0.84	0.84	0.83	0.83
FS>1.5 ?							NO	NO	NO	NO	NO	NO
Check Shear at distance d from support												
Vu at distance d	171.97	171.97	167.82	167.82	169.80	169.80	103.75	103.75	101.25	101.25	102.44	102.44
phiVc > Vu ?	NO	NO	NO	NO	NO	NO	YES	YES	YES	YES	YES	YES
Factor of Safety, FS (Vn/Vmax)							1.40	1.40	1.44	1.44	1.42	1.42
FS>1.5 ?							NO	NO	NO	NO	NO	NO

15-113

Comp by SMH
Checked by ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40

TABLE 2-2

Description/Info:
500+3, collar beam strength check
3' x3' beam with 7-1 1/8-inch square bars in each vertical face (As=8.86 sq in)

Investigation of Moment and Shear Capacity per EM 1110-2-210

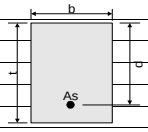
	Strength Design Checks using LRFD						Strength Checks with UNFACTORED LOADS & Safety Factors					
	Full HGL		Full HGL		Full HGL		Full HGL		Full HGL		Full HGL	
Item/member description	**member 1 in face**	**member 1 out face**	**member 2 in face**	**member 2 out face**	**member 3 in face**	**member 3 out face**	**member 1 in face**	**member 1 out face**	**member 2 in face**	**member 2 out face**	**member 3 in face**	**member 3 out face**
Max Moment from CFRAME (k-in)	**2369**	**4732**	**2260**	**4638**	**2331**	**4657**	**2369**	**4732**	**2260**	**4638**	**2331**	**4657**
Max shear from CFRAME (k)	**175.8**	**175.8**	**173.1**	**173.1**	**174.4**	**174.4**	**175.8**	**175.8**	**173.1**	**173.1**	**174.4**	**174.4**
Assumed contibution from shear steel (at dist d)												
stirrup As (in^2)	0.62						0.62	0.62	0.62	0.62	0.62	0.62
stirrup spacing, s(in)	15						15	15	15	15	15	15
phiVs = phi*(As*fy*d)/s	46.376						54.56	54.56	54.56	54.56	54.56	54.56
phiVn = phiVc + phiVs	170.05						200.06	200.06	200.06	200.06	200.06	200.06
phiVn > Vu ?	**NO**						**YES**	**YES**	**YES**	**YES**	**YES**	**YES**
Factor of Safety, FS (Vn/Vmax)							1.93	1.93	1.98	1.98	1.95	1.95
FS>1.5 ?							**YES**	**YES**	**YES**	**YES**	**YES**	**YES**
Check Steel Ratio												
Beta1	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
rho bal	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640
rho actual	0.00746	0.00746	0.00746	0.00746	0.00746	0.00746	0.00746	0.00746	0.00746	0.00746	0.00746	0.00746
Recommended limit, 0.25*rho bal	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160
rho act < recommended limit?	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**
Maximum permitted limit, 0.375*rho bal	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740
rho act < MAX rho?	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**
Check ACI Minimums												
ACI 318 10.5.1 minimum (3f'c^0.5/fy)bd	5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46
As> min ACI 10.5.1 ?	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**
ACI 318 10.5.1 minimum (200/fy)bd	5.94	5.94	5.94	5.94	5.94	5.94	5.94	5.94	5.94	5.94	5.94	5.94
As> min ACI 10.5.1 ?	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**

Comp by SMH
Checked by ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40 **TABLE 2-3**
Description/Info:
500+3 with HGL -8, collar beam strength check - SHEAR ONLY

Investigation of Moment and Shear Capacity per EM 1110-2-2104

	Strength Design Checks using LRFD		Strength Checks with UNFACTORED LOADS & Safety Factors	
	HGL -8		HGL -8	
Item/member description	**member 1 in face**	**member 1 out face**	**member 1 in face**	**member 1 out face**
Max Moment from CFRAME (k-in)				
Max shear from CFRAME (k)	**144.7**	**144.7**	**144.7**	**144.7**
Material Properties				
f'c (ksi)	**3.75**	**3.75**	**3.75**	**3.75**
fy (ksi)	**40**	**40**	**40**	**40**
Es (ksi)	**29000**	**29000**	**29000**	**29000**
Member Properties				
span(ft)	**13.42**	**13.42**	**13.42**	**13.42**
b (in)	**36**	**36**	**36**	**36**
Member thickness (in)	**36**	**36**	**36**	**36**
cover+1/2db, d' (estimate) (in)	**3**	**3**	**3**	**3**
As (in^2)	**8.86**	**8.86**	**8.86**	**8.86**
Loading				
Max Moment, M (ft-k)	0.00	0.00	0.00	0.00
Max Shear, V (k)	144.7	144.7	144.7	144.7
Single Load Factor	1.70	1.70	1.00	1.00
Hydraulic Load Factor	1.30	1.30	1.00	1.00
Extreme Case load Factor	0.75	0.75	1.00	1.00
Max Mu (ft-k)	0.00	0.00	0.00	0.00
Max Vu (k)	239.84	239.84	144.70	144.70
Check Steel Yield				
d (in)	33.00	33.00	33.00	33.00
a (in)	3.09	3.09	3.09	3.09
c (in)	3.63	3.63	3.63	3.63
es	0.0242	0.0242	0.0242	0.0242
ey	0.0014	0.0014	0.0014	0.0014
Steel yield ?	**YES**	**YES**	**YES**	**YES**
Check Shear per ACI				
phiVc (k) (ACI)	123.67	123.67	145.50	145.50
phiVc > Max Vu ?	**NO**	**NO**	**YES**	**YES**
Factor of Safety, FS (Vn/Vmax)			1.01	1.01
FS>1.5 ?			**NO**	**NO**
Check Shear at distance d from support				
Vu at distance d	141.55	141.55	85.40	85.40
phiVc > Vu ?	**NO**	**NO**	**YES**	**YES**
Factor of Safety, FS (Vn/Vmax)			1.70	1.70
FS>1.5 ?			**YES**	**YES**
Check Steel Ratio				
Beta1	0.85	0.85	0.85	0.85
rho bal	0.04640	0.04640	0.04640	0.04640
rho actual	0.00746	0.00746	0.00746	0.00746
Recommended limit, 0.25*rho bal	0.01160	0.01160	0.01160	0.01160
rho act < recommended limit?	**YES**	**YES**	**YES**	**YES**
Maximum permitted limit, 0.375*rho bal	0.01740	0.01740	0.01740	0.01740
rho act < MAX rho?	**YES**	**YES**	**YES**	**YES**
Check ACI Minimums				
ACI 318 10.5.1 minimum (3f'c^0.5/fy)bd	5.46	5.46	5.46	5.46
As> min ACI 10.5.1 ?	**YES**	**YES**	**YES**	**YES**
ACI 318 10.5.1 minimum (200/fy)bd	5.94	5.94	5.94	5.94
As> min ACI 10.5.1 ?	**YES**	**YES**	**YES**	**YES**



_TPS.Strength Checks.v3.xls
Collar Beam(HGL-8)

Levee Unit: Argentine
Pump Station: Turner Ave. (Kaw Valley No 4)
Levee Station: 60+40
Flood Event: 500+3
Proposed Protection: EL. 780.8

Strength Investigation of Base Slab

(Revised 11/2004 due to KC Review [v2])
(Revised 1/2005 after KC Summit #1 12/1/04 [v3])

NOTES:

1. Loading. For the loading condition of flood event with the proposed levee raise, the base slab of the pump station will be investigated for strength to resist the resulting uplift pressures.

2. The uplift pressures used to evaluate the base slab strength will conservatively be based upon the full HGL for the with-project 500+3 condition. The dead weight of the slab itself and the weight of the water in the pump station (assumed to be a minimum of 8 ft per mechanical) will be included. The reactions from the exterior walls (as determined in the exterior wall investigation) were also included in the investigation.

3. Strength Investigation. The base slab is assumed to span in the north south direction as indicated in the included sketch. The location which is approximately at the location of section 2 as shown on the OM&M plates will be assumed to control based upon the largest slab span between the north wall and the interior wall. The slab will be investigated assuming a simple 1 foot strip beam approach.

   Initially, a strength investigation is per EM 1110-2-2104. Per the EM, load factors used include the single load factor of 1.7, the hydraulic load factor of 1.3 and the extreme event factor of 0.75. The member was modeled in CASE program CFRAME to determine internal moments and shears developed from lateral loading.

   After KC summit #1 12/1/04, an unfactored, factor of safety approach was used.

4. Material Properties. Material properties for the base segment assumed to be f'c=3.75 ksi and fy=40 ksi.

5. Member Assumptions. For simplicity, the base is assumed to span horizontally and is investigated as a double span 1-foot strip beam with fixed ends. The tension steel is 1-inch round bars at 8-inch spacing and the clear cover is assumed to be 2.5 inches.

6. Results.

   FACTORED. Results of strength investigation for the moment and shear capacities are shown on the attached Excel spreadsheet/table TABLE 3-1. The slab appears to be adequate to accommodate the loads for moment and shear.

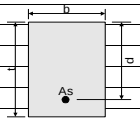
   UNFACTORED. Also, after the KC summit #1, 12/1/04, the unfactored factor of safety approach also indicated that the slab was adequate. The factors of safety of flexure and shear were both above 1.5. Results for this approach are also shown in the TABLE 3-1.

   Note this member does NOT meet ACI 10.5.1 minimum for tension steel.

Comp by SMH
Checked by ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40 **TABLE 3-1**
Description/Info:
500+3, base slab strength check
Assume 1ft strip; Beam = 12" x 36" with 1" dia bar @ 8" o.c. (As=1.185 sq in.) each face

Investigation of Moment and Shear Capacity per EM 1110-2-2104



	Strength Design Checks using LRFD		Strength Checks with UNFACTORED LOADS & Safety Factors	
	Full HGL		Full HGL	
Item/member description	**Lwr wall top face**	**Lwr wall bot face**	**Lwr wall top face**	**Lwr wall bot face**
Max Moment from CFRAME (k-in)	**690.8**	**606.7**	**690.8**	**606.7**
Max shear from CFRAME (k)	**20.8**	**20.8**	**20.8**	**20.8**
Material Properties				
f'c (ksi)	**3.75**	**3.75**	**3.75**	**3.75**
fy (ksi)	**40**	**40**	**40**	**40**
Es (ksi)	**29000**	**29000**	**29000**	**29000**
Member Properties				
span(ft)	**17.5**	**17.5**	**17.5**	**17.5**
b (in)	**12**	**12**	**12**	**12**
Member thickness (in)	**36**	**36**	**36**	**36**
cover+1/2db, d' (estimate) (in)	**3**	**3**	**3**	**3**
As (in^2)	**1.185**	**1.185**	**1.185**	**1.185**
Loading				
Max Moment, M (ft-k)	57.57	50.56	57.57	50.56
Max Shear, V (k)	20.8	20.8	20.8	20.8
Single Load Factor	1.70	1.70	1.00	1.00
Hydraulic Load Factor	1.30	1.30	1.00	1.00
Extreme Case load Factor	0.75	0.75	1.00	1.00
Max Mu (ft-k)	95.42	83.80	57.57	50.56
Max Vu (k)	34.48	34.48	20.80	20.80
Check Steel Yield				
d (in)	33.00	33.00	33.00	33.00
a (in)	1.24	1.24	1.24	1.24
c (in)	1.46	1.46	1.46	1.46
es	0.0649	0.0649	0.0649	0.0649
ey	0.0014	0.0014	0.0014	0.0014
Steel yield ?	**YES**	**YES**	**YES**	**YES**
Check Flexure				
phiMn (ft-k)	115.11	115.11	127.90	127.90
phiMn > Mu ?	**OK**	**OK**	**OK**	**OK**
Factor of Safety, FS (Mn/Mmax)			2.22	2.53
FS>1.5 ?			**YES**	**YES**
Check Shear per ACI				
phiVc (k) (ACI)	41.22	41.22	48.50	48.50
Factor of Safety, FS (Vn/Vmax)			2.33	2.33
FS>1.5 ?			**YES**	**YES**
phiVc > Max Vu ?	**YES**	**YES**	**YES**	**YES**
Check Shear at distance d from support				
Vu at distance d	23.64	23.64	14.26	14.26
phiVc > Vu ?	**YES**	**YES**	**YES**	**YES**
Factor of Safety, FS (Vn/Vmax)			3.40	3.40
FS>1.5 ?			**YES**	**YES**
Check Steel Ratio				
Beta1	0.85	0.85	0.85	0.85
rho bal	0.04640	0.04640	0.04640	0.04640
rho actual	0.00299	0.00299	0.00299	0.00299
Recommended limit, 0.25*rho bal	0.01160	0.01160	0.01160	0.01160
rho act < recommended limit?	**YES**	**YES**	**YES**	**YES**
Maximum permitted limit, 0.375*rho bal	0.01740	0.01740	0.01740	0.01740
rho act < MAX rho?	**YES**	**YES**	**YES**	**YES**
Check ACI Minimums				
ACI 318 10.5.1 minimum (3f'c^0.5/fy)bd	1.82	1.82	1.82	1.82
As> min ACI 10.5.1 ?	**NO**	**NO**	**NO**	**NO**
Ratio As/As min	0.65	0.65	0.65	0.65
ACI 318 10.5.1 minimum (200/fy)bd	1.98	1.98	1.98	1.98
As> min ACI 10.5.1 ?	**NO**	**NO**	**NO**	**NO**
Ratio As/As min	0.60	0.60	0.60	0.60

Levee Unit: Argentine
Pump Station: Turner Ave. (Kaw Valley No 4)
Levee Station: 60+40
Flood Event: 500+3
Proposed Protection: EL. 780.8

Discharge Chamber/Gatewell Raise

(Revised 11/2004 due to KC Review [v2])
(Revised 1/2005 after KC Summit #1 12/1/04 [v3])

NOTES:

1. The discharge chamber (gatewell) will require structural raise to equal the new proposed levee height for the flood event and proposed protection elevation noted above.

2. Existing Conditions. The original top of the gatewell was at El. 772.0. Per the Argentine O&M plates, the protection level at this location was raised from approximately 772 to 776.8, and the gatewell was also raised accordingly. The 1962 raised the top of gatewell 4.8 feet from El. 772 to El. 776.8. The raise was accomplished by simply "adding" on to the top of the structure with reinforced cast-in-place concrete. Therefore the current top of the existing gatewell is at El. 776.8.

3. Proposed Change. With the proposed level of protection raise, the top of the gatewell must also be raised to equal the top of protection. The proposed 500+3 level will raise the top of the structure 4 feet, to El. 780.8. Assume that the raise will be of similar construction to that of the raise in 1962. Details and quantities related to this raise are attached.

	Elevation (ft)	**Difference (ft)**
Proposed Gatewell raise	780.8	0
Existing	776.8	4
"Pre 1962"	772.0	8.8

4. Loading. The current approach is to install a new wall on the levee crown to achieve the with-project 500+3 protection adjacent to the pump station. This is being done for purposes of deloading the box culvert beneath the levee. Therefore assume that no additional soil loads on the structure. Loading change for the structure will be due to increased water pressures only.

5. The sluice gate at the gatewell is on the landside between the gatewell/discharge chamber and the pump station. Therefore, assume that the interior of the gatewell will see river head. At max flood condition the water elevation in the gatewell will be equal to the river elevation of 780.8. By inspection, the critical wall in the discharge chamber/gatewell is the common wall between the gatewell and pump station (ie the east wall).

6. Strength Investigation. The east wall will be investigated for structural strength based upon its judged criticality. This wall will have water loads from the inside of the gatewell, but will have essentially no loading in the opposite direction on the other side of the wall to counteract the water pressures. Strength check will be performed on the existing concrete east wall of the gatewell based upon internal water level at EL. 780.8. The wall section will be checked at various elevations based upon the changes in member properties relating to wall thickness and reinforcement. (Reference O&M Plate 76). A summary of the wall "segments" investigated are listed here and are graphically shown in the attached sketch. The integral beam (as identified in the sketch) in the wall directly above the sluice opening was also investigated for the increased horizontal water pressures.

   Initially, a strength investigation per EM 1110-2-2104 was used. Load factors used include the single load factor of 1.7, the hydraulic load factor of 1.3 and the extreme event factor of 0.75.

After KC summit #1 12/1/04, an unfactored, factor of safety approach was used.

Wall Segment	Lowest Elevation(ft)	Wall Thickness (in)	Reinforcing
A	776.8	14	#5@12”
B	772.0	14	#5@12”
C	767.5	14	5/8” DIA @ 12”
D	762.0	14	7/8” DIA @ 12”
E	757.0	20	3/4” DIA @ 12”
F	752.0	20	7/8” DIA @ 12”
G	746.5	20	1” DIA @ 12”

7. Material Properties. Material properties for all existing wall segments assumed to be f’c=3.75 ksi and fy=40 ksi, except for the proposed raised construction in which f’c=4 ksi and fy=60 ksi were assumed.

8. Member Assumptions. For simplicity, the wall is assumed to span horizontally and is investigated as a single span 1-foot strip beam with fixed ends. Maximum positive moment in center (tension face on outside) computed as $wl^2/24$ and maximum negative moment at ends (tension face on inside) computed as $wl^2/12$. The span was assumed to be the clear span between adjacent walls.

9. Results.

FACTORED. Results of strength investigation shown on Excel spreadsheet/table TABLE 4-1. There is also a hand calculation included for wall segment F that verifies the spreadsheet calculations.
All wall segments meet the EM criteria for moment at the center of the span (positive moment) and for shear. However wall segments B thru F do not meet the EM criteria for moment capacity for the end or negative moment regions. A sub-summary table is shown below for wall segments B thru F. The corresponding ratios of factored moment (Mu) to moment capacity (ϕMn) are shown in the table and range from 1.01 to 1.53. Strength reduction factor, ϕ of 0.9 was used. The factored moment is only 1% over the factored moment capacity for walls B and F, and can be considered acceptable. Walls D and E could also be considered acceptable, as the factored moment is less than 15% over the factored moment capacity for this extremely rare loading event.

	B	**C**	**D**	**E**	**F**
phiMn > Max Mu ?	NO	NO	NO	NO	NO
RATIO Mu to phiMn	1.01	1.53	1.14	1.13	1.01
RATIO phiMn to Mmax	1.64	1.08	1.46	1.47	1.65
RATIO Mn to Mmax	1.82	1.20	1.62	1.63	1.83

Therefore, the remaining wall segment left in question is wall C in which the factored moment is 53% over the factored moment capacity. It is recommended that this region of the wall C be strengthened.

UNFACTORED.

Structural Element	**Flexural Check Factor of Safety Mn/Mmax**	**Shear Check (Conc only) Factor of Safety Vn/Vmax**	**Shear Check (Conc + steel) Factor of Safety Vn/Vmax**	**Flexure/Axial Interaction**	**ACI 318 10.5.1 Min met ?**
Discharge Chamber Walls A at El. 776.8	5.99**	11.8	-	-	NO
Discharge Chamber Walls B at El. 772.0	1.83**	5.20	-	-	NO
Discharge Chamber Walls C at El. 767.5	1.21**	3.44	-	-	NO
Discharge Chamber Walls D at El. 762.0	1.63**	2.43	-	-	NO
Discharge Chamber Walls E at El. 757.0	1.64**	3.10	-	-	NO
Discharge Chamber Walls F at El. 752.0	1.84**	2.56	-	-	NO
Discharge Chamber Walls G at El. 746.5	2.02	2.15	-	-	NO
Discharge Chamber Wall Beam	2.35	1.67*	-	-	OK

Notes:

* = factor of safety for shear at distance d from support. Otherwise, factor of safety for shear at support
** = factor of safety for negative moment at ends of member

As shown in the table above, wall segments A, B, and D thru G (per the section diagram in the computations) were found to be adequate in flexure and shear. Note that the negative moment at the ends of the member controlled the flexural check. The factors of safety for flexure and shear for these segments of the wall were above the 1.5.

Note, however that the tension steel did NOT meet the ACI minimum for these members.

However, wall segments C did not meet the factor of safety of 1.5 for the end or negative moment region. The factor of safety is above one, so it can be concluded that a complete failure and collapse of this member is not eminent under the 500+3 condition, but would be overstressed. It is recommended that this end region of the wall, where the negative flexural factor of safety is below 1.5, be strengthened to achive the minimum 1.5 factor of safety.

The wall "beam" just above the sluice opening was found to be adequate and results are summarized in the attached Excel spreadsheet/table TABLE 4-2. The beam was found to be adequate for lateral moment capacity. The shear reinforcement had to be accounted for to adequately resist the factored shear loads.

QUANTITY SUMMARY FOR GATEWELL RAISE

Grating
1 ¼" deep = 4'-2.25"(6'-0") — 25 ft^3

Ladder Rungs
Preformed (Cast Steel) — 3 each

Ladder
Fabricated (1 ½" DIA steel pipe w/ 1" DIA pipe rungs) — 4 linear ft

Railing
Repair and replace (1 ½" DIA pipe) — 13 linear ft

Railing
Remove and replace (2 rail, 42 in high) — 46 linear ft

Grout Holes
Drill 1 ½" DIA holes x 1.0 ft deep — 32 each

Non-Shrink Grout
1 ½" DIA holes x 1ft deep = — 0.4ft^3

Sluice Gate Stem
add extension — 4 linear ft

Flap Gates
Low head gates (18") — 2 each

Concrete — 10.6 yd^3

overall	14.17	11.46	4	649.55
void	-4	11.83	4	-189.28
void	-1.5	11.83	2	-35.49
void	-3.63	9.83	3.25	-115.97
void	-3.63	2	2.5	-18.15
chamfer	-0.125	0.125	28.33	-0.44
chamfer	-0.125	0.125	22.42	-0.35
chamfer	-0.125	0.125	24.17	-0.38
chamfer	-0.125	0.125	8	-0.13
pipe voids	0.7917	1.17	-2	-4.61

TOTAL	284.76	cu ft
	10.55	cu yd

Reinforcing steel 1,941 lbs

	qty	length	wt/ft	
#5	22	2.5	1.04	57.37
#5	65	3.88	1.04	263.04
#5	30	5.5	1.04	172.10
#5	10	13.83	1.04	144.25
#5	10	15.83	1.04	165.11
#5	10	16.08	1.04	167.71
#5	10	13.25	1.04	138.20
#4	26	6	0.67	104.21
#4	13	14	0.67	121.58
#8	4	14	2.67	149.52
#8	4	18	2.67	192.24
#7	6	6.1	2.04	74.81
#7	6	8.58	2.04	105.23
#3	14	4.83	0.38	25.43
#3	12	6.67	0.38	30.10
#5	3	4.67	1.04	14.61
#5	4	3.75	1.04	15.65

TOTAL 1941.13 lbs

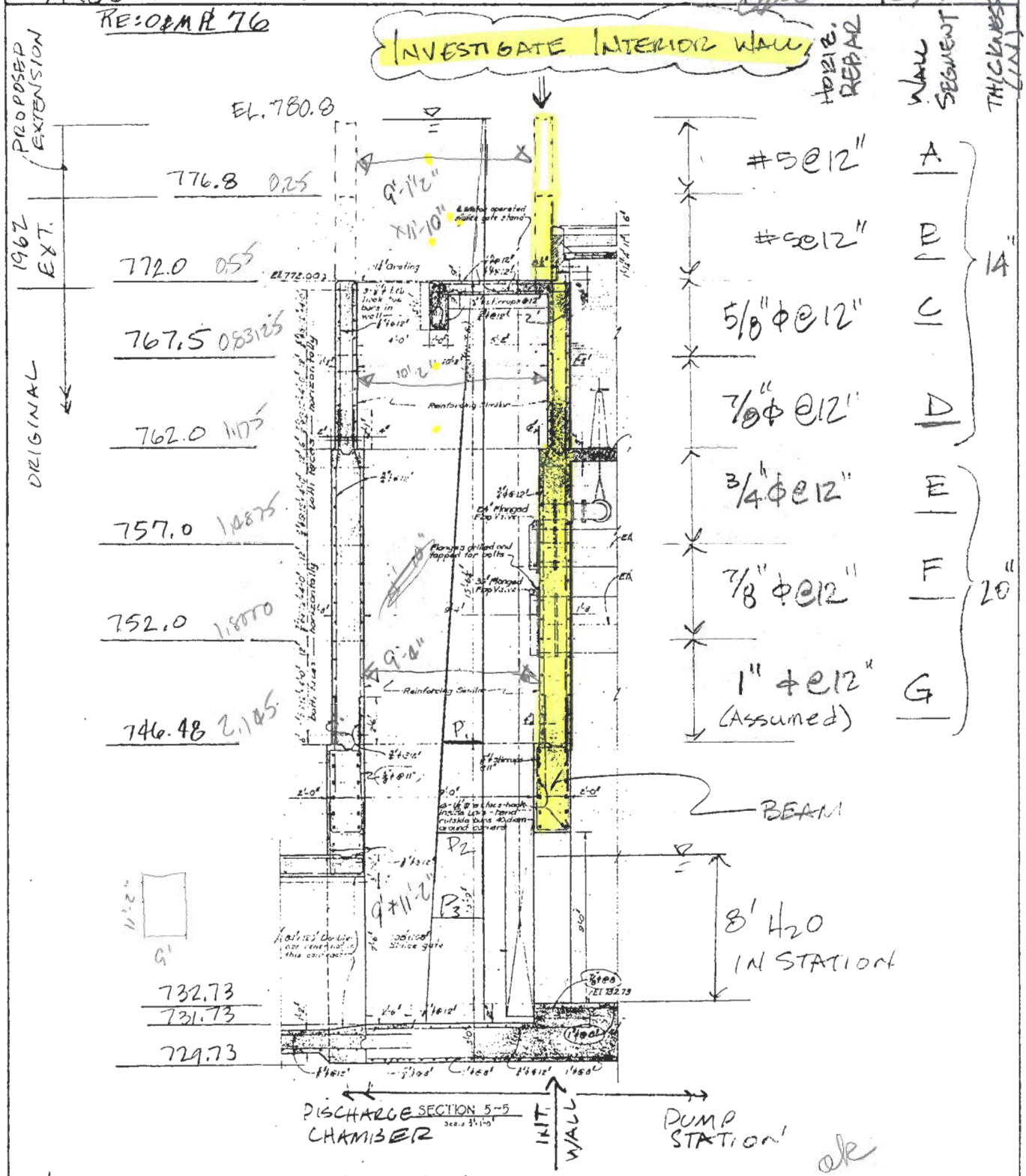
Structural Steel

grate frame L1 ½ x1 ½ x ¼ “	80.2 lbs
bar 1 ½ x ¼ x 0’-7”	14.8 lbs
bar 2 x ½ x 5’-6”	37.8 lbs
pipe sleeve 4” DIA x 0’-9”	8.1 lbs
TOTAL	140.5 lbs

PROJECT		COMPUTED BY	DATE
KC DISTRICT – SEVEN LEVEES	Page ___ of ___	SMH	3/24/04
SUBJECT		CHECKED BY	DATE
ARGENTINE – TURNER DISCHARGE CHAMBER			5/6/04

RE: O&M # 76

INVESTIGATE INTERIOR WALL



– INVESTIGATE WALL BETWEEN DISCHARGE CHAMBER AND PUMP STATION. THIS WILL CONSTITUTES WORSE CASE CONDITION DUE TO FULL RIVER HEAD IN DISCHARGE CHAMBER AND MIN. 8' HEAD IN PUMP STATION.

– FOR SIMPLICITY, ASSUME WALL SPANS HORIZONTALLY

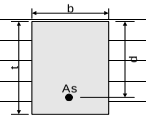
– ASSUME CONTINUITY AT CORNERS – ASSUME 1-FT STRIP

Comp by _SMH
Checked by _ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40 **TABLE 4-1**
Description/Info:
500+3, Discharge Chamber - Sidewalls
Investigate assuming 1-ft strip - beam, single span
Assumes continuity at corners - ie, assume fixed ends

Investigation of Moment and Shear Capacity per EM 1110-2-2104

Pool Elevation (ft)	780.8													
	Strength Design Checks using LRFD							Strength Checks with UNFACTORED LOADS & Safety Factors						
Wall Segment	A	B	C	D	E	F	G	A	B	C	D	E	F	G
Elevation on wall (ft)	776.8	772	767.5	762	757	752	746.5	776.8	772	767.5	762	757	752	746.5
Material Properties														
f'c (ksi)	4	3.75	3.75	3.75	3.75	3.75	3.75	4	3.75	3.75	3.75	3.75	3.75	3.75
fy (ksi)	60	40	40	40	40	40	40	60	40	40	40	40	40	40
Es (ksi)	29000	29000	29000	29000	29000	29000	29000	29000	29000	29000	29000	29000	29000	29000
Member Properties														
clear span (ft)	11.83	11.83	11.83	11.83	11.17	11.17	11.17	11.83	11.83	11.83	11.83	11.17	11.17	11.17
b (in)	12	12	12	12	12	12	12	12	12	12	12	12	12	12
Member thickness (in)	14	14	14	14	20	20	20	14	14	14	14	20	20	20
cover+1/2db (estimate), d' (in)	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
As (in^2)	0.31	0.31	0.31	0.6	0.44	0.6	0.79	0.31	0.31	0.31	0.6	0.44	0.6	0.79
Loading														
water depth (ft)	4.00	8.80	13.30	18.80	23.80	28.80	34.30	4.00	8.80	13.30	18.80	23.80	28.80	34.30
Water pressure (ksf)	0.25	0.55	0.83	1.18	1.49	1.80	2.14	0.25	0.55	0.83	1.18	1.49	1.80	2.14
Max Moment, M (ft-k) [at ends wl^2/12]	2.92	6.41	9.69	13.70	15.47	18.72	22.29	2.92	6.41	9.69	13.70	15.47	18.72	22.29
Moment, M1 (ft-k) [at center wl^2/24]	1.46	3.21	4.85	6.85	7.73	9.36	11.14	1.46	3.21	4.85	6.85	7.73	9.36	11.14
Max Shear, V (k)	1.48	3.25	4.92	6.95	8.31	10.05	11.97	1.48	3.25	4.92	6.95	8.31	10.05	11.97
Single Load Factor	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Hydraulic Load Factor	1.30	1.30	1.30	1.30	1.30	1.30	1.30	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Extreme Case Load Factor	0.75	0.75	0.75	0.75	0.75	0.75	0.75	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Max Mu at ends (ft-k)	4.83	10.63	16.07	22.71	25.64	31.02	36.94	2.92	6.41	9.69	13.70	15.47	18.72	22.29
M1u at center	2.42	5.32	8.03	11.36	12.82	15.51	18.47	1.46	3.21	4.85	6.85	7.73	9.36	11.14
Max Vu (k)	2.45	5.39	8.15	11.52	13.77	16.66	19.84	1.48	3.25	4.92	6.95	8.31	10.05	11.97
Check Steel Yield														
d (in)	11.50	11.50	11.50	11.50	17.50	17.50	17.50	11.50	11.50	11.50	11.50	17.50	17.50	17.50
a (in)	0.46	0.32	0.32	0.63	0.46	0.63	0.83	0.46	0.32	0.32	0.63	0.46	0.63	0.83
c (in)	0.54	0.38	0.38	0.74	0.54	0.74	0.97	0.54	0.38	0.38	0.74	0.54	0.74	0.97
es	0.0613	0.0875	0.0875	0.0437	0.0940	0.0681	0.0510	0.0613	0.0875	0.0875	0.0437	0.0940	0.0681	0.0510
ey	0.0021	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014	0.0021	0.0014	0.0014	0.0014	0.0014	0.0014	0.0014
Steel yield ?	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES



15-124

Comp by _SMH
Checked by _ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40 **TABLE 4-1**
Description/Info:
500+3, Discharge Chamber - Sidewalls
Investigate assuming 1-ft strip - beam, single span
Assumes continuity at corners - ie, assume fixed ends

Investigation of Moment and Shear Capacity per EM 1110-2-2104

Pool Elevation (ft)	780.8													
	Strength Design Checks using LRFD							Strength Checks with UNFACTORED LOADS & Safety Factors						
Wall Segment	A	B	C	D	E	F	G	A	B	C	D	E	F	G
Elevation on wall (ft)	776.8	772	767.5	762	757	752	746.5	776.8	772	767.5	762	757	752	746.5
Check Flexure at Max Moment [ends]														
phiMn (ft-k)	15.72	10.54	10.54	20.14	22.80	30.94	40.50	17.47	11.72	11.72	22.37	25.33	34.37	45.00
phiMn > Max Mu ?	YES	NO	NO	NO	NO	NO	YES	YES	YES	YES	YES	YES	YES	YES
RATIO Mn to Mmax		1.83	1.21	1.63	1.64	1.84								
Factor of Safety, FS (Mn/Mmax)								5.99	1.83	1.21	1.63	1.64	1.84	2.02
FS>1.5 ?								YES	YES	NO	YES	YES	YES	YES
Check Flexure at Max Moment [center]														
phiMn (ft-k)	15.72	10.54	10.54	20.14	22.80	30.94	40.50	17.47	11.72	11.72	22.37	25.33	34.37	45.00
phiMn > M1u [center]?	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
Factor of Safety, FS (Mn/Mmax)								11.98	3.65	2.42	3.27	3.28	3.67	4.04
FS>1.5 ?								YES	YES	YES	YES	YES	YES	YES
Check Shear per ACI														
phiVc (k) (ACI)	14.84	14.37	14.37	14.37	21.86	21.86	21.86	17.46	16.90	16.90	16.90	25.72	25.72	25.72
phiVc > Vu ?	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
Factor of Safety, FS (Vn/Vmax)								11.80	5.20	3.44	2.43	3.10	2.56	2.15
FS>1.5 ?								YES	YES	YES	YES	YES	YES	YES
Check Shear at distance d from support														
Vu at distance d	2.05	4.52	6.83	9.65	10.17	12.31	14.66	1.24	2.73	4.12	5.82	6.14	7.43	8.85
phiVc > Vu ?	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
Factor of Safety, FS (Vn/Vmax)								14.09	6.20	4.10	2.90	4.19	3.46	2.91
FS>1.5 ?								YES	YES	YES	YES	YES	YES	YES
Check Shear-special straight members														
l/d	12.34	12.34	12.34	12.34	7.66	7.66	7.66	12.34	12.34	12.34	12.34	7.66	7.66	7.66
check 1.25 > l/d > 9 ?	NO	NO	NO	NO	YES	YES	YES	NO	NO	NO	NO	YES	YES	YES
EM EQN 5-1 apply ?	NO	NO	NO	NO	YES	YES	YES	NO	NO	NO	NO	YES	YES	YES
Adjacent clear span (ft)					9.33	9.33	9.33					9.33	9.33	9.33
Nu (k)	n/a	n/a	n/a	n/a	9.42	11.40	13.57	n/a	n/a	n/a	n/a	9.42	11.40	13.57
Ag (sq in)	n/a	n/a	n/a	n/a	240.00	240.00	240.00	n/a	n/a	n/a	n/a	240.00	240.00	240.00
phiVc (k) (EM EQN 5-1)	n/a	n/a	n/a	n/a	44.59	45.12	45.69	n/a	n/a	n/a	n/a	52.46	53.08	53.76
limit phiVc	n/a	n/a	n/a	n/a	94.89	94.89	94.89	n/a	n/a	n/a	n/a	111.64	111.64	111.64
phiVc	n/a	n/a	n/a	n/a	44.59	45.12	45.69	n/a	n/a	n/a	n/a	52.46	53.08	53.76
phiVc > Vu ?	n/a	n/a	n/a	n/a	YES	YES	YES	n/a	n/a	n/a	n/a	YES	YES	YES
Factor of Safety, FS (Vn/Vmax)								n/a	n/a	n/a	n/a	8.55	7.15	6.08
FS>1.5 ?								n/a	n/a	n/a	n/a	YES	YES	YES
Check Steel Ratio														
Beta1	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
rho bal	0.02851	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640	0.02851	0.04640	0.04640	0.04640	0.04640	0.04640	0.04640
rho actual	0.00225	0.00225	0.00225	0.00435	0.00210	0.00286	0.00376	0.00225	0.00225	0.00225	0.00435	0.00210	0.00286	0.00376
Recommended limit, 0.25*rho bal	0.00713	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160	0.00713	0.01160	0.01160	0.01160	0.01160	0.01160	0.01160
rho act < recommended limit?	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
Maximum permitted limit, 0.375*rho bal	0.01069	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740	0.01069	0.01740	0.01740	0.01740	0.01740	0.01740	0.01740
rho act < MAX rho?	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES

15-125

Comp by SMH
Checked by ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40
Description/Info:
500+3, Discharge Chamber - Sidewalls
Investigate assuming 1-ft strip - beam, single span
Assumes continuity at corners - ie, assume fixed ends

TABLE 4-1

Investigation of Moment and Shear Capacity per EM 1110-2-2104

Pool Elevation (ft)	780.8						
	Strength Design Checks using LRFD						
Wall Segment	**A**	**B**	**C**	**D**	**E**	**F**	**G**
Elevation on wall (ft)	**776.8**	**772**	**767.5**	**762**	**757**	**752**	**746.5**
Check ACI Minimums							
ACI 318 10.5.1 minimum (3f'c^0.5/fy)bd	0.44	0.63	0.63	0.63	0.96	0.96	0.96
As> min ACI 10.5.1 ?	**NO**	**NO**	**NO**	**NO**	**NO**	**NO**	**NO**
Ratio As/As min	0.71	0.49	0.49	0.95	0.46	0.62	0.82
ACI 318 10.5.1 minimum (200/fy)bd	0.46	0.69	0.69	0.69	1.05	1.05	1.05
As> min ACI 10.5.1 ?	**NO**	**NO**	**NO**	**NO**	**NO**	**NO**	**NO**
Ratio As/As min	0.67	0.45	0.45	0.87	0.42	0.57	0.75

Strength Checks with UNFACTORED LOADS & Safety Factors						
A	**B**	**C**	**D**	**E**	**F**	**G**
776.8	**772**	**767.5**	**762**	**757**	**752**	**746.5**
0.44	0.63	0.63	0.63	0.96	0.96	0.96
NO	**NO**	**NO**	**NO**	**NO**	**NO**	**NO**
0.71	0.49	0.49	0.95	0.46	0.62	0.82
0.46	0.69	0.69	0.69	1.05	1.05	1.05
NO	**NO**	**NO**	**NO**	**NO**	**NO**	**NO**
0.67	0.45	0.45	0.87	0.42	0.57	0.75

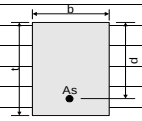
15-126

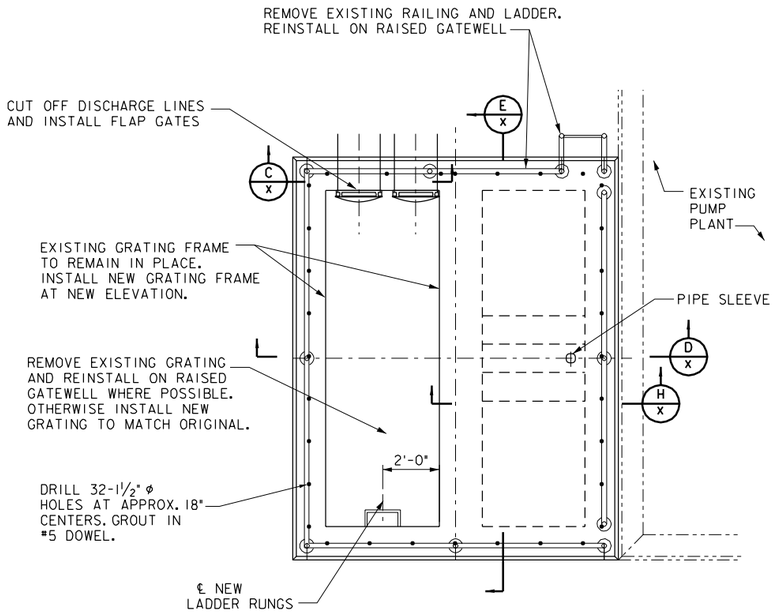
Comp by SMH
Checked by ALS

Project:
KC Dist - Seven Levees - Argentine Unit - Turner PS Sta. 60+40 **TABLE 4-2** **TABLE 4-2**
Description/Info:
500+3, Discharge Chamber - Wall "Beam"
Investigate assuming single span
Assumes continuity ie, assume fixed ends

Investigation of Moment and Shear Capacity per EM 1110-2-2104

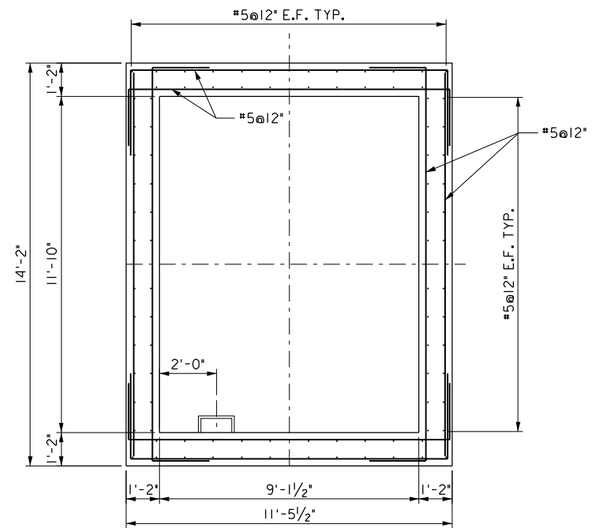
	Strength Design Checks using LRFD		**Strength Checks with UNFACTORED LOADS & Safety Factors**	
	Full HGL		**Full HGL**	
Item/member description	**HORIZ**		**HORIZ**	
Max Moment from CFRAME (k-in)	**2644**		**2644**	
Max shear from CFRAME (k)	**118.3**		**118.3**	
Material Properties				
f'c (ksi)	**3.75**		**3.75**	
fy (ksi)	**40**		**40**	
Es (ksi)	**29000**		**29000**	
Member Properties				
span(ft)	**11.17**		**11.17**	
b (in)	**51**		**51**	
Member thickness (in)	**24**		**24**	
cover+1/2db, d' (estimate) (in)	**2.5**		**2.5**	
As (in^2)	**7.56**		**7.56**	
Loading				
Max Moment, M (ft-k)	220.33		220.33	
Max Shear, V (k)	118.3		118.3	
Single Load Factor	1.70		1.00	
Hydraulic Load Factor	1.30		1.00	
Extreme Case load Factor	0.75		1.00	
Max Mu (ft-k)	365.20		220.33	
Max Vu (k)	196.08		118.30	
Check Steel Yield				
d (in)	21.50		21.50	
a (in)	1.86		1.86	
c (in)	2.19		2.19	
es	0.0265		0.0265	
ey	0.0014		0.0014	
Steel yield ?	**YES**		**YES**	
Check Flexure				
phiMn (ft-k)	466.53		518.36	
phiMn > Mu ?	**OK**		**OK**	
Factor of Safety, FS (Mn/Mmax)			2.35	
FS>1.5 ?			**YES**	
Check Shear per ACI				
phiVc (k) (ACI)	114.15		134.29	
phiVc > Max Vu ?	**NO**		**YES**	
Factor of Safety, FS (Vn/Vmax)			1.14	
FS>1.5 ?			**NO**	
Check Shear at distance d from support				
Vu at distance d	133.18		80.35	
phiVc > Vu ?	**NO**		**YES**	
Factor of Safety, FS (Vn/Vmax)			1.67	
FS>1.5 ?			**YES**	
Assumed contibution from shear steel (at dist d)				
stirrup As (in^2)	0.62		0.62	
stirrup spacing, s(in)	11		11	
phiVs = phi*(As*fy*d)/s	41.2018182		48.4727273	
phiVn = phiVc + phiVs	155.35		182.77	
phiVn > Vu ?	**YES**		**YES**	
Check Steel Ratio				
Beta1	0.85		0.85	
rho bal	0.04640		0.04640	
rho actual	0.00689		0.00689	
Recommended limit, 0.25*rho bal	0.01160		0.01160	
rho act < recommended limit?	**YES**		**YES**	
Maximum permitted limit, 0.375*rho bal	0.01740		0.01740	
rho act < MAX rho?	**YES**		**YES**	
Check ACI Minimums				
ACI 318 10.5.1 minimum (3f'c^0.5/fy)bd	5.04		5.04	
As> min ACI 10.5.1 ?	**YES**		**YES**	
ACI 318 10.5.1 minimum (200/fy)bd	5.4825		5.4825	
As> min ACI 10.5.1 ?	**YES**		**YES**	





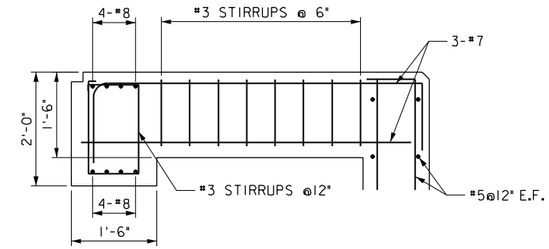
PLAN OF PROPOSED EL. 780.8

SCALE: 3/8" = 1'-0"



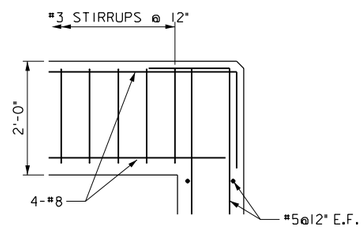
SECTION F

SCALE: 3/8" = 1'-0"



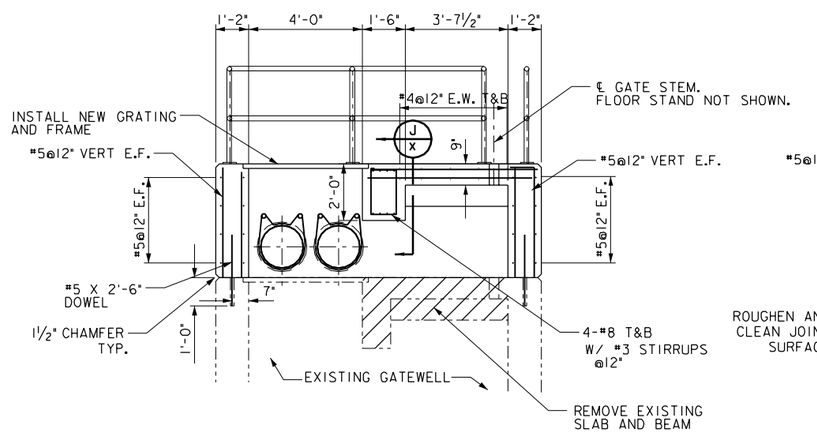
SECTION H

SCALE: 3/4" = 1'-0"



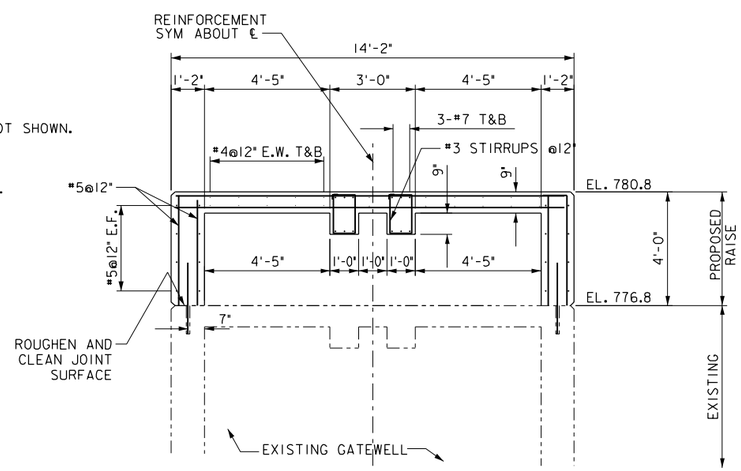
SECTION J

SCALE: 3/4" = 1'-0"



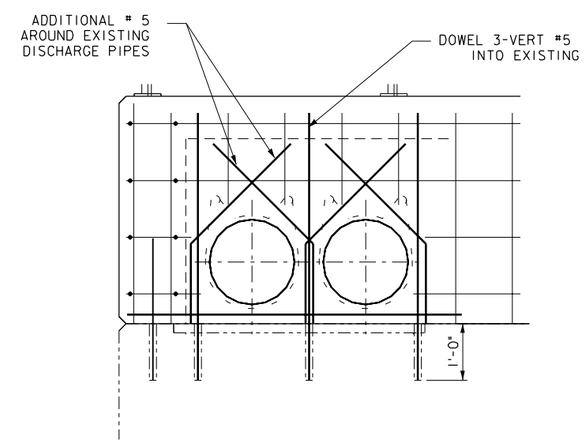
SECTION D

SCALE: 3/8" = 1'-0"



SECTION E

SCALE: 3/8" = 1'-0"



SECTION C

SCALE: 3/4" = 1'-0"

RAISED GATEWELL - STA 60+40

REFERENCE ARGENTINE UNIT O&MM PLATE NO. 120

US Army Corps of Engineers

Symbol	Description / Revisions	Date	Appr.

Designed by: X
Drawn by: X
Ckd by: X
Reviewed by: X
APPROVED:
Date:
SUBMITTED:
File name: $$$$DESIGN$FILE$SPECIFICATION$$$$
Plot date: $$$$DATE
Plot time: $$$$TIME
X
SOL. NO. X- X-

U.S. ARMY ENGINEER DIVISION CORPS OF ENGINEERS X

KANSAS CITY DISTRICT - SEVEN LEVEES
ARGENTINE UNIT, 500+3
TURNER GATEWELL/DISCHARGE CHAMBER RAISE
STRUCTURAL
X X X

S-X
DRAWING NUMBER

END

PROJECT		COMPUTED BY	DATE
KANSAS CITY DIST. - COE	Page — of —	M. STOHL	9 MAR 04
SUBJECT		CHECKED BY	DATE
TURNER PUMP. STA. - GATEWELL RAISE		[illegible]	3/9/04

QUANTITY ESTIMATE

CONCRETE

14.17 (11.46)(4.0)	=	649.6 FT.^{3}
- 4.0 (11.83)(4.0)	=	-189.3
- 1.5 (11.83)(2.0)	=	- 35.5
- 3.63 (9.83)(3.25)	=	-116.0
- 3.63 (2.0)(2.5)	=	- 18.2
- 0.125^2 (28.33 + 22.42)	=	- 0.8
- 0.125^2 (24.17 + 8.0)	=	- 0.5
- 3.14 ($0.79(17^2)$)(2)(1.17)	=	- 4.6

TOTAL = 284.7 FT.^{3}/27 = **10.5 YDS.^{3}**

REINFORCING STEEL

#5 (VERT.)	= 3 (4.67)(1.043)	=	14.6 LBS.
#5 x 2'-6" DWL'S.	= 22 (2.5)(1.043)	=	57.4
#5 (VERT.)	= 65 (3.88)(1.043)	=	263.0
#5 (VERT.)	= 30 (5.5)(1.043)	=	172.1
#5 (HORIZ.)	= 10 (13.83)(1.043)	=	144.2
#5 (HORIZ.)	= 10 (15.83)(1.043)	=	165.1
#5 (HORIZ.)	= 10 (16.08)(1.043)	=	167.8
#5 (HORIZ.)	= 10 (13.25)(1.043)	=	138.2
#4 (HORIZ.)	= 26 (6.0)(0.668)	=	104.2
#4 (HORIZ.)	= 13 (14.0)(0.668)	=	121.6
#8 (HORIZ.)	= 4 (14.0)(2.670)	=	149.5
#8 (HORIZ.)	= 4 (18.0)(2.670)	=	192.2
#7 (HORIZ.)	= 6 (6.08)(2.044)	=	74.6
#7 (HORIZ.)	= 6 (8.58)(2.044)	=	105.2
#3 (STIR.)	= 14 (4.83)(0.376)	=	25.4
#3 (STIR.)	= 12 (6.67)(0.376)	=	30.1
#5 (VERT.)	= 4 (3.75)(1.043)	=	15.6

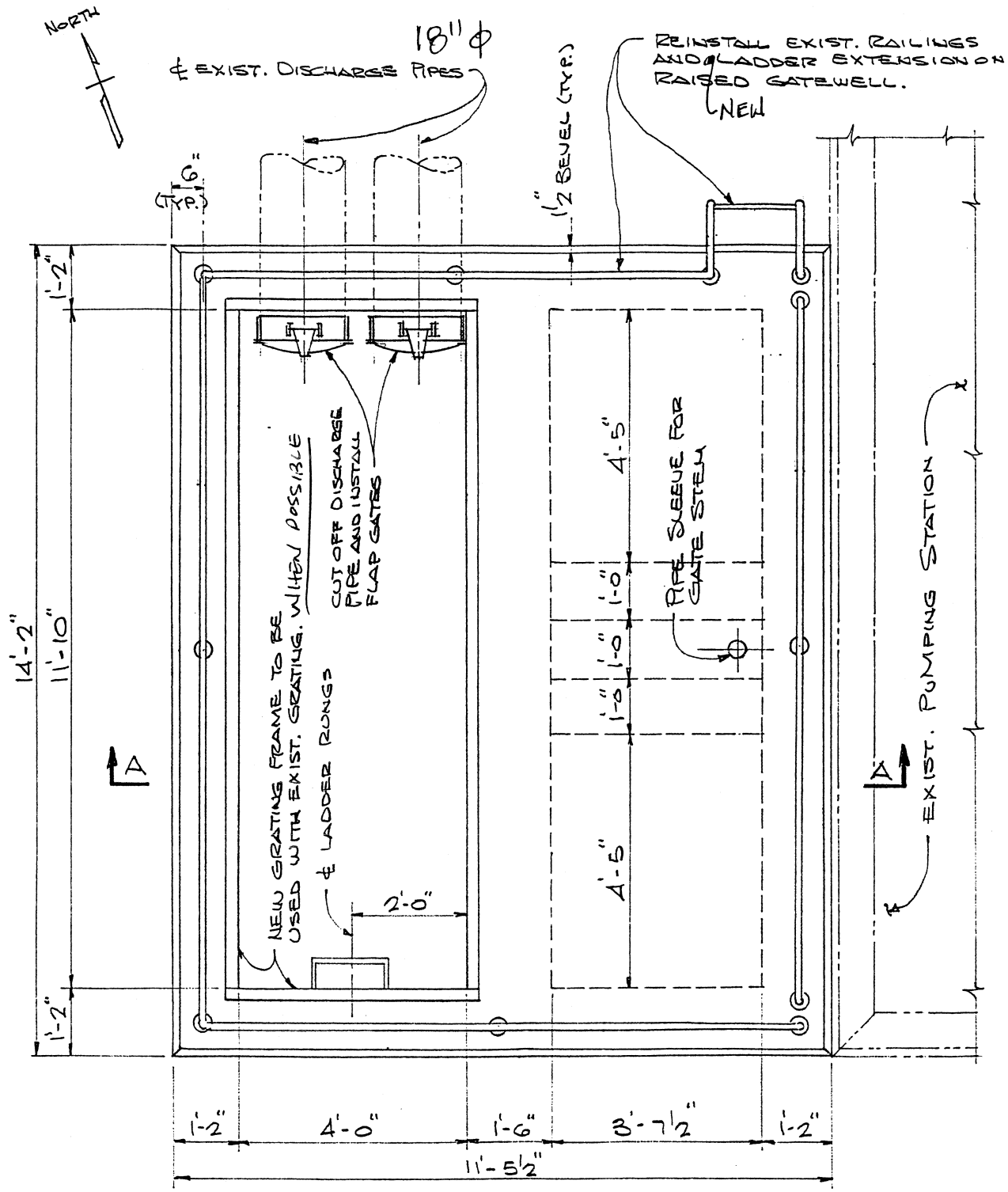
TOTAL = 1,940.8 LBS. = **1,941 LBS.**

STRUCTURAL STEEL

L - 1½" x 1½" x ¼"	= 2.5 (32.08)	= 80.2 LBS.	(GRATE FRAME)
BAR 1½" x ¼" x 0'-7"	= 1.28 (0.58)(20)	= 14.8	(STRAP ANCHORS)
BAR 2" x ½" x 5'-6"	= 3.4 (5.5)(2)	= 37.4	(ANCHORS FOR 18" ϕ PIPES)
PIPE (4" ϕ x 0'-9")	= 10.79 (0.75)	= 8.1	(GATE STEM SLEEVE)

TOTAL = 140.5 LBS. = **141 LBS.**

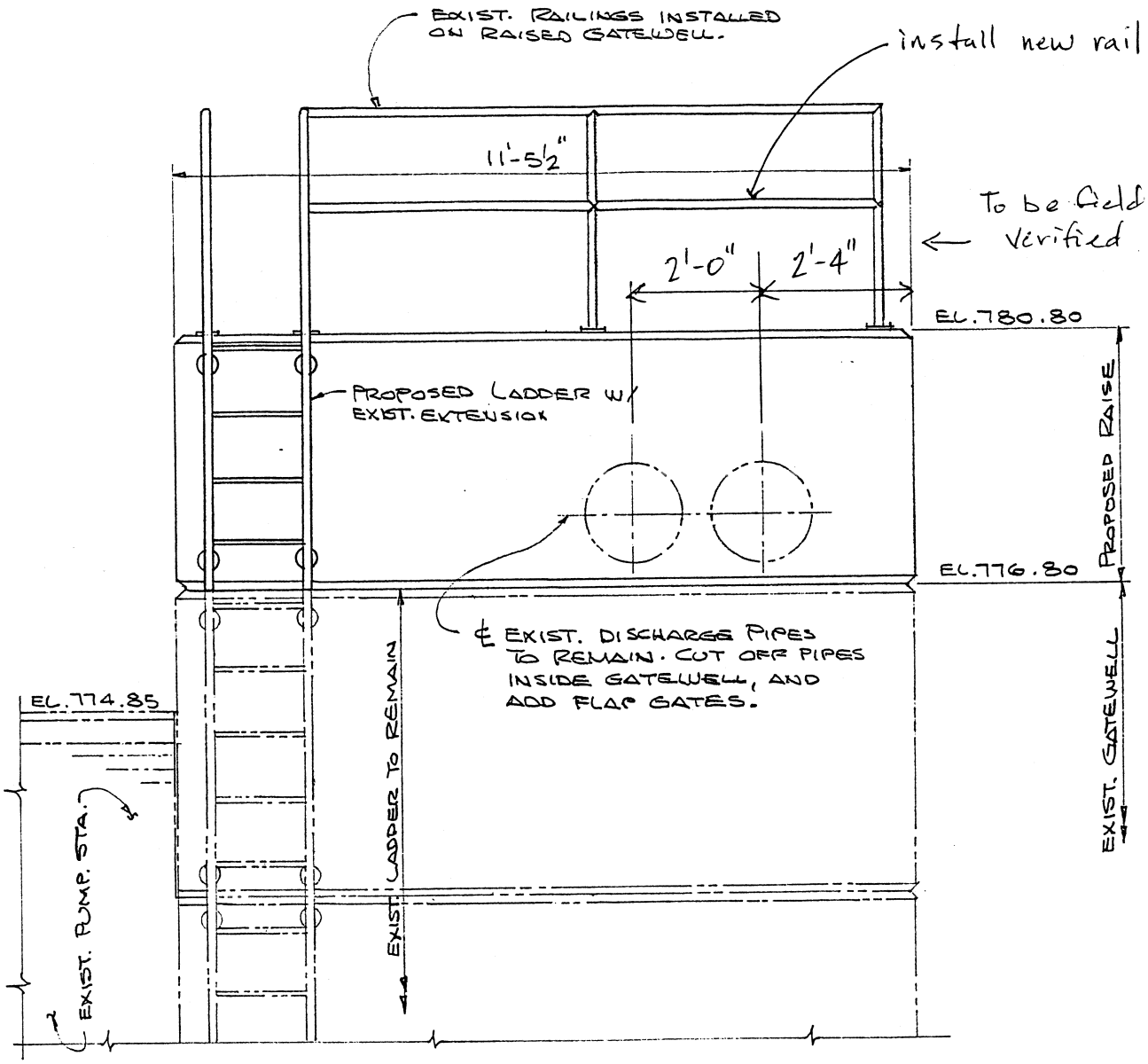
PROJECT		COMPUTED BY	DATE
KANSAS CITY DIST. - COE	Page 1 of 3	M. STOHL	2 MAR 04
SUBJECT		CHECKED BY	DATE
TURNER PUMP. STA. - GATEWELL RAISE		SMH	3 MAR 04



PLAN OF RAISED GATEWELL

SCALE: 3/8" = 1'-0"

PROJECT		COMPUTED BY	DATE
KANSAS CITY DIST. - COE	Page 2 of 3	M. STOHL	2 MAR 04
SUBJECT		CHECKED BY	DATE
TURNER PUMP. STA. - GATEWELL RAISE		SMH	3 MAR 04

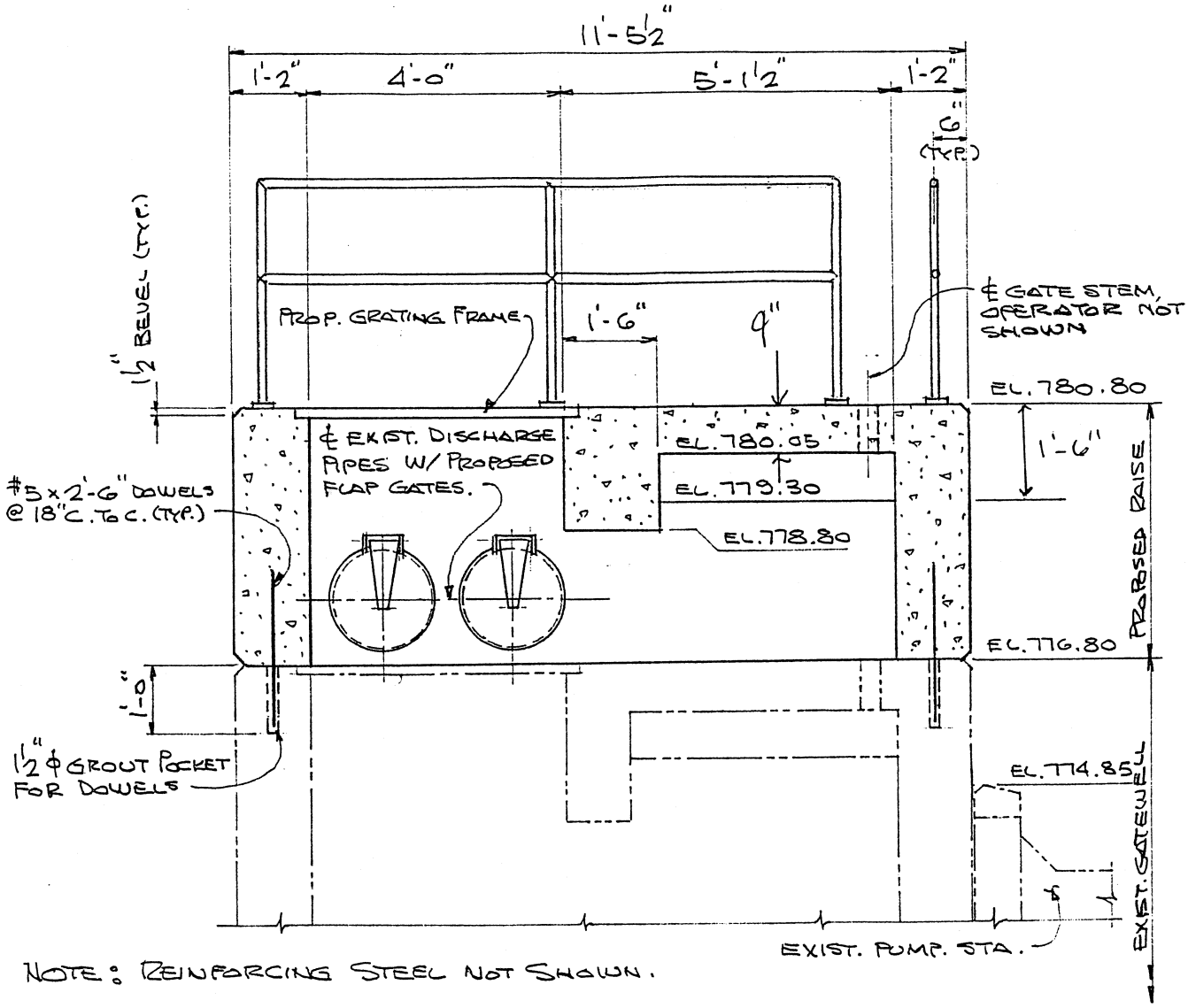


NORTH ELEVATION

SCALE: 3/8" = 1'-0"

NOTE: SLUICE GATE OPERATOR NOT SHOWN. THE GATE STEM SHALL BE LENGTHENED, AND THE EXIST. OPERATOR INSTALLED ON THE RAISED GATEWELL.

PROJECT		COMPUTED BY	DATE
KANSAS CITY DIST. - COE	Page 3 of 3	M. STAHL	2 MAR 04
SUBJECT		CHECKED BY	DATE
TURNER PUMP. STA. - GATEWELL RAISE		SMH	3 MAR 04



NOTE: REINFORCING STEEL NOT SHOWN.

SECTION A-A
SCALE: 3/8" = 1'-0"

PROJECT	Page ___ of ___	COMPUTED BY	DATE
SUBJECT		CHECKED BY	DATE

GATEWELL - MAIN BEAM

MOMENT CAPACITY OF EXISTING BEAM:

BM = 12" x 30" w/ A_s = 3-#7 (1.8 in²)

Assuming f'_c = 3ksi, f_y = 40ksi

ϕM_n = 139.45 'k

Assuming f'_c = 4ksi, f_y = 60 ksi

ϕM_n = 207.98

"REPLACEMENT" BEAM - CHECK MOMENT CAPACITY

BM = 18" x 24" w/ 4-#8 (A_s = 3.16 in²)

Using f'_c = 4ksi & f_y = 60 ksi,

ϕM_n = 276.59 'k > ϕM_n existing

Assume ok. for Moment Capacity.

PROJECT	Page ___ of ___	COMPUTED BY	DATE
SUBJECT		CHECKED BY	DATE

GATEWELL - MAIN BEAM

SHEAR CAPACITY OF EXISTING BEAM

BEAM = 12" x 30" w/ A_s = 3-#7 & stirrups #3 @ 12"

Assume d = 30 - 3 = 27", f'_c = 3ksi, f_y = 40 ksi

$V_n = V_c + V_s$

$V_c = 2\sqrt{3000}(12)(27) = 35.5^k$

$V_s = \frac{A_v(f_y)d}{s} = \frac{0.22(40)27}{12} = 19.8^k$

$\phi V_n = 0.85(35.5 + 19.8) = 47.0^k$

"REPLACEMENT" BEAM - CHECK SHEAR CAPACITY

BEAM = 18" x 24" w/ 4-#8 & stirrups #3 @ 12"

Use f_y = 60 ksi f'_c = 4 ksi Assume d = 24 - 3 = 21"

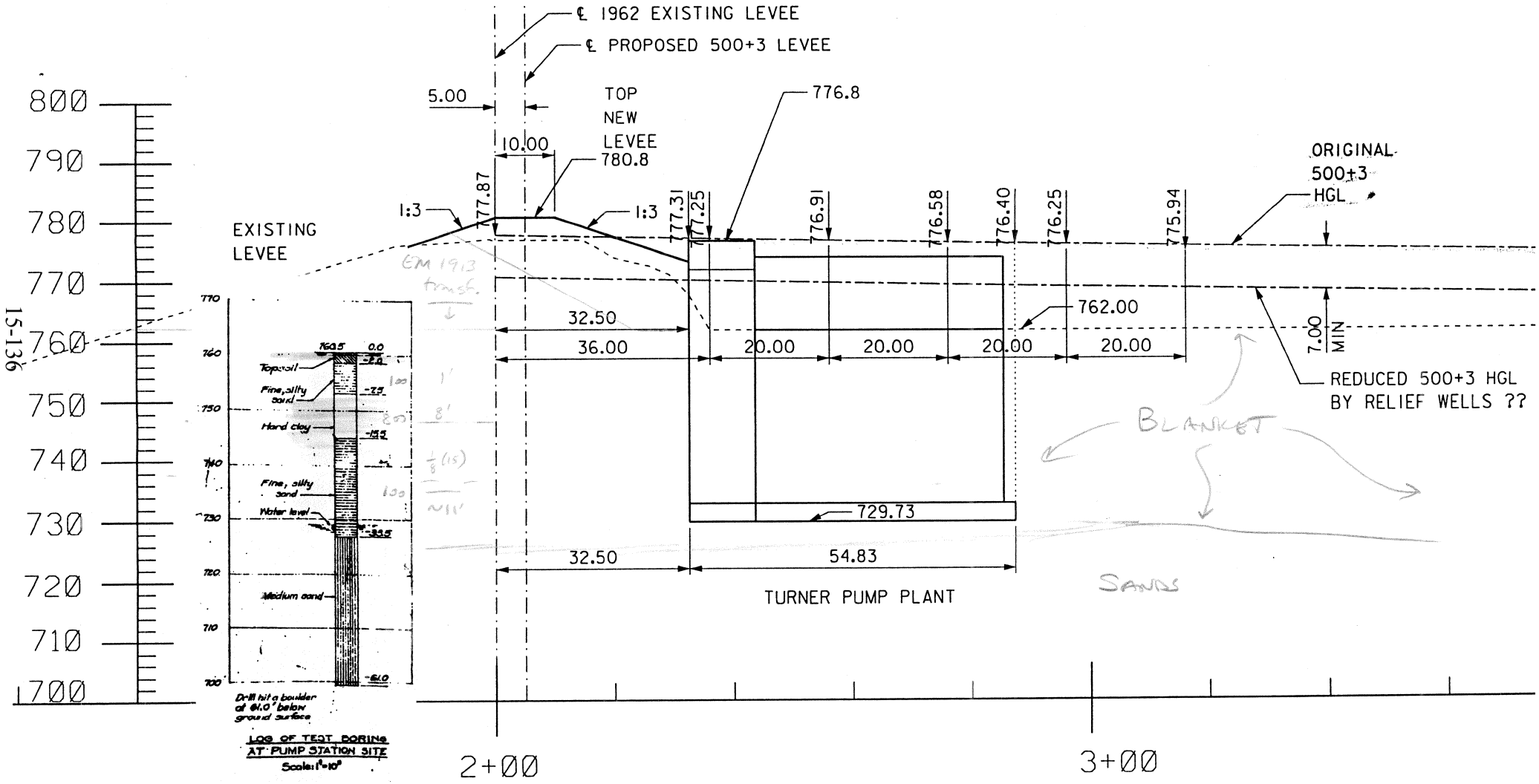
$V_c = 2\sqrt{4000}(18)(21) = 47.8^k$

$V_s = \frac{0.22(60)21}{12} = 23.1^k$

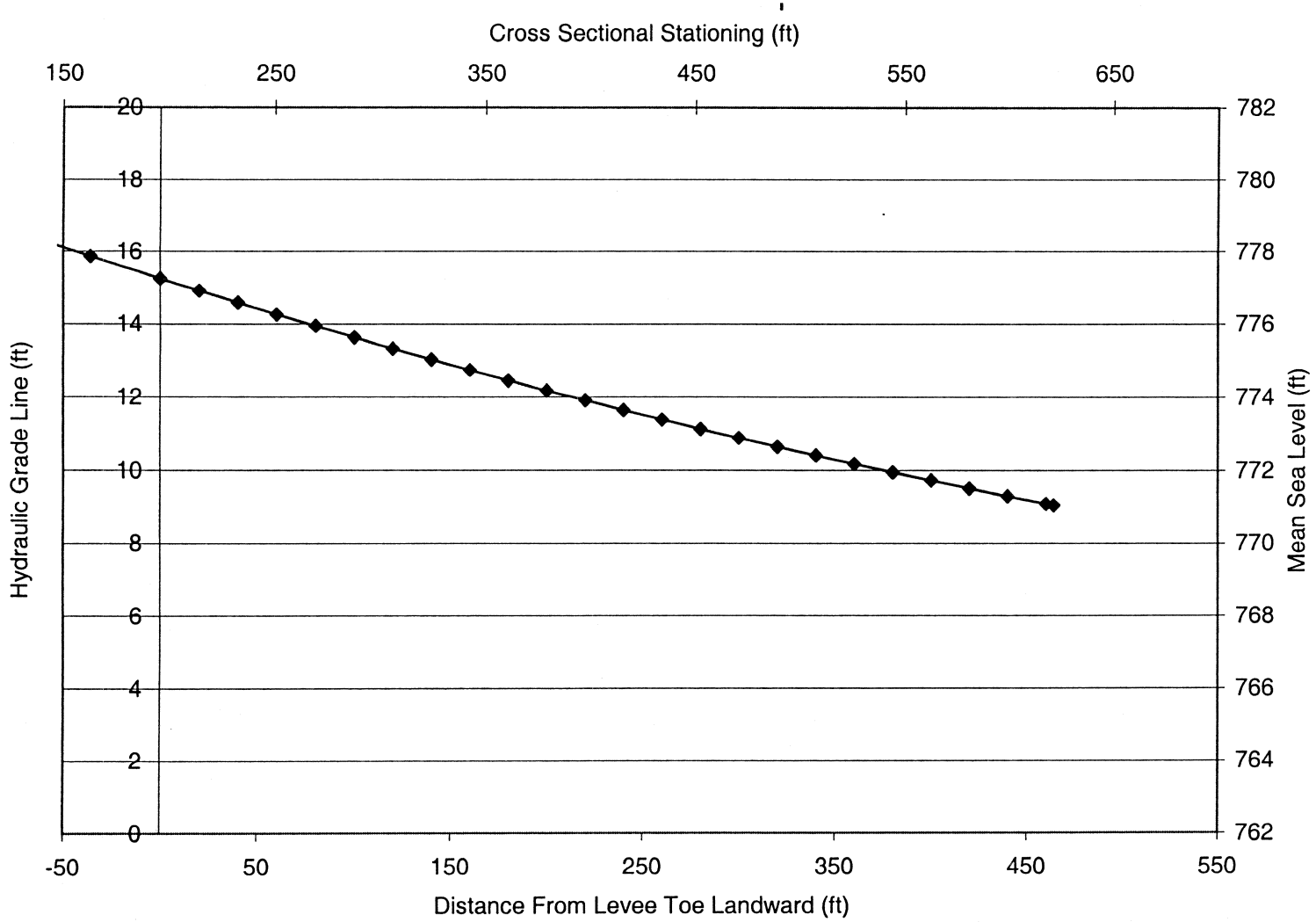
$\phi V_n = 0.85(47.8 + 23.1) = 60.3^k$ > ϕV_n existing beam

Assume <u>ok</u> for Shear Capacity.

STA. 60+40
500+3, TOL=780.8



Hydraulic Grade Line for the Turner Pump Station Based on Cross Section 6
File (kcarg-pump-Turner-Xsect.dgn) DRAFT
At Base of Blanket (33 ft thick at Pump Plant) 500 yr +3



FROM J. LOEHR
2/23/04

Hydralic Grade Line for the Turner Pump Station Based on Cross Section 6 File (kcarg-pump-Turner-Xsect.dgn)			
Elevation New Top of levee (MSL)	780.8	500yr+3	
Elevation at Levee Toe (MSL)	762		
Elevation Bottom of Blanket (MSL)	729		
Driving Head (ft)	18.8		
L1 (ft)	100		
L2 (ft)	114		
Le (ft)	890		
ho	15.25		
Cross Section Coordinate	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
CL Levee at 200			
	-207	18.8	780.80
200	-36	15.87	777.87
236	0	15.25	777.25
256	20	14.91	776.91
276	40	14.58	776.58
296	60	14.26	776.26
316	80	13.94	775.94
336	100	13.63	775.63
356	120	13.33	775.33
376	140	13.03	775.03
396	160	12.74	774.74
416	180	12.46	774.46
436	200	12.18	774.18
456	220	11.91	773.91
476	240	11.65	773.65
496	260	11.39	773.39
516	280	11.13	773.13
536	300	10.89	772.89
556	320	10.64	772.64
576	340	10.41	772.41
596	360	10.18	772.18
616	380	9.95	771.95
636	400	9.73	771.73
656	420	9.51	771.51
676	440	9.30	771.30
696	460	9.10	771.10
700	464	9.05	771.05

Hydralic Grade Line for the Turner Pump Station Based on Cross Section 6 (kcarg-pump-Turner-Xsect.dgn) **File**

ARG turner pump station 500

UNDERSEEPAGE ANALYSIS

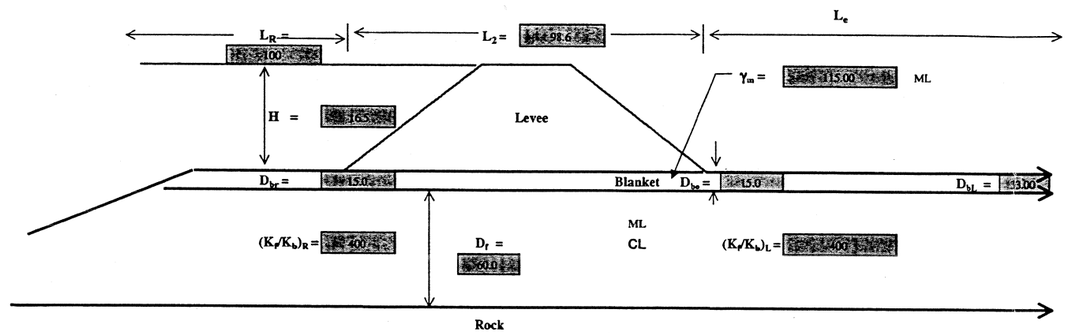
Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

MRL S L-142

Name : Sommers EC-GD
Date : 1/22/2003

Revised and Checked by : Loehr 2/23/2004

Revised to reflect criteria developed for L-142



Recommended Permeability Ratio For Foundation Blanket Materials Chapter Underseepage Revision dated Oct 1998	
Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

NOMENCLATURE

$(K_f/K_b)_R$ = riverside permeability
$(K_f/K_b)_L$ = landside permeability
D_{br} = riverside blanket thickness
D_{bo} = levee toe blanket thickness
D_{bL} = landside blanket thickness
D_f = thickness of pervious foundation
L_R = length of riverside blanket
L_L = length of landside blanket
H = max head or levee height
$H(W_T)$ = head above tailwater at end of underseepage berm
$H(W_T) = H(W_T/2)*e^{(-WT/(2*CL))}$
$H(W_{T/2})$ = head above tailwater midpoint of underseepage berm
$= H * L_e / L'_t$
W_T = berm width
i_o = seepage gradient
C_r = riverside effective length coefficient
C_L = landside effective length coefficient
where $C = [(Kf/Kb) * Df * Db]^{1/2}$
H'o = head above tailwater at levee toe (w/ berm)
i_c = critical seepage gradient
L_1 = riverside effective length
where $L_1 = C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
L_2 = levee base width
L_e = landside effective length
L_t = total effective length
L'_t = Total Effective Length + 1/2 of Berm
t' = required Berm thickness at toe
Sc = calc'd slope of underseepage berm
q = seepage /unit Length
Q = cummulated seepage
$FS_i = i_c / i_o$

Station	Begin Station	End Station	Input Parameters								Remarks
	Feet	Feet	$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL}	D_{bo}	D_{br}	D_f	L_R	H	
			First row is tied to above graphs! Change Graphics only!								
61+00			400	400	33.0	15.0	15.0	60.0	100	16.5	
61+00	500yr Nominal		400	400	33.0	15.0	15.0	60.0	100	16.5	60+40 Elev 778.53'
61+00	500 yr +3		400	400	33.0	15.0	15.0	60.0	100	18.8	60+40 Elev 780.79'
61+00	500 yr + 5		400	400	33.0	15.0	15.0	60.0	100	20.8	60+40 Elev 782.79'

Analysis of Without Berm Conditions

C_R	C_L	L_1	L_2	L_e	L_t	h_o	I_o	i_c	Check #1	Check #1	Remarks
Feet	Feet	Feet	Feet	Feet	Feet	Feet	Ft/Ft	Ft/Ft	Full Head Toe FS_i (need 1.1)	Reduced Head Toe FS_i (need 1.5)	
600	890	99	99	890	1088	13.51	0.90	0.84	0.94	1.14	< 1.1 and < 1.5 Need control
600	890	99	99	890	1088	13.50	0.90	0.84	0.93	1.14	< 1.1 and < 1.5 Need control
600	890	99	108	890	1097	15.25	1.02	0.84	0.83	0.98	< 1.1 and < 1.5 Need control
600	890	99	114	890	1103	16.77	1.12	0.84	0.75	0.88	< 1.1 and < 1.5 Need control

PLANS FOR PUMP STATION NO. 4

KAW VALLEY DRAINAGE DISTRICT

KANSAS CITY, KANSAS

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BLACK & VEATCH

CONSULTING ENGINEERS

KANSAS CITY, MO.

1950

SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" cond.	2/9/78	[illegible]

REVISIONS

KANSAS CITYS FLOOD CONTROL PROJECT

ARGENTINE UNIT

KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT

KANSAS CITY

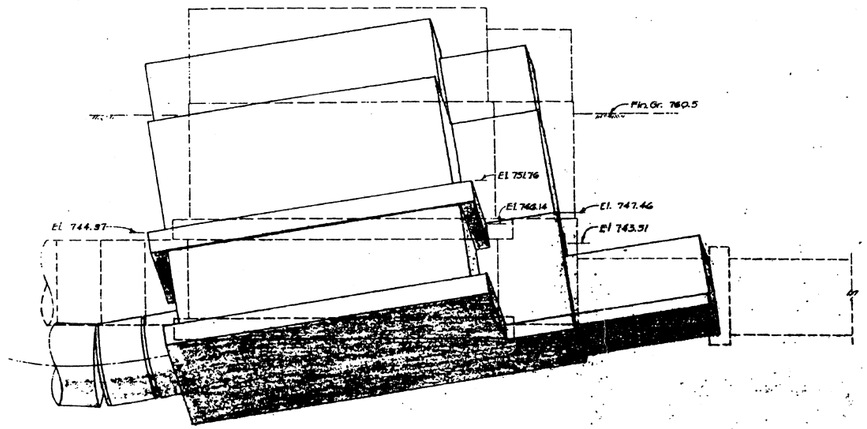
FILE NO. A-10-6391

FEBRUARY 1978

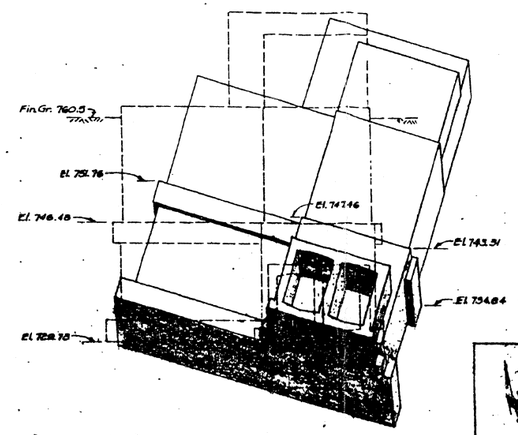
RECORD DRAWING

NOVEMBER 1954

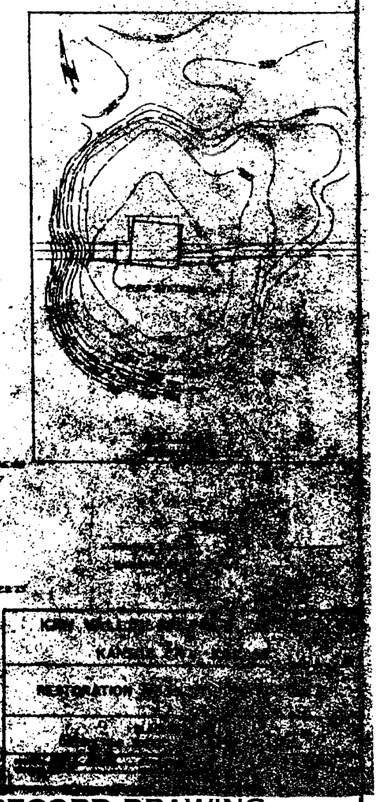
CONTRACT NO. DA-23-028-ENG-481

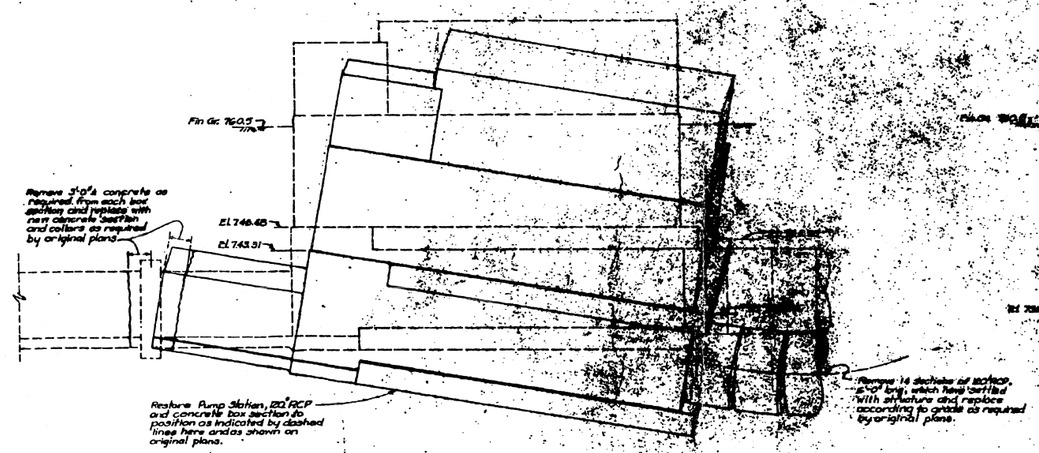


NORTH ELEVATION
Scale: 1/8" = 1'-0"

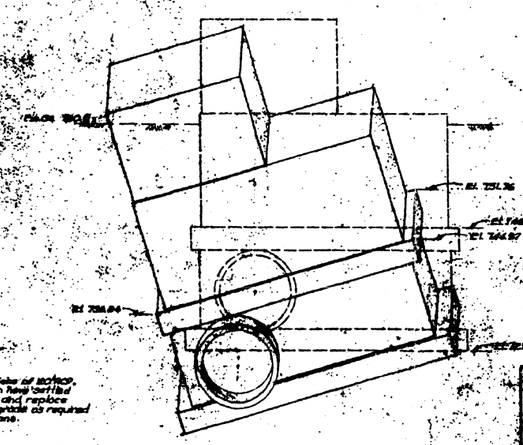


WEST ELEVATION
Scale: 1/8" = 1'-0"





SOUTH ELEVATION
Scale: 1/8" = 1'-0"



EAST ELEVATION
Scale: 1/8" = 1'-0"

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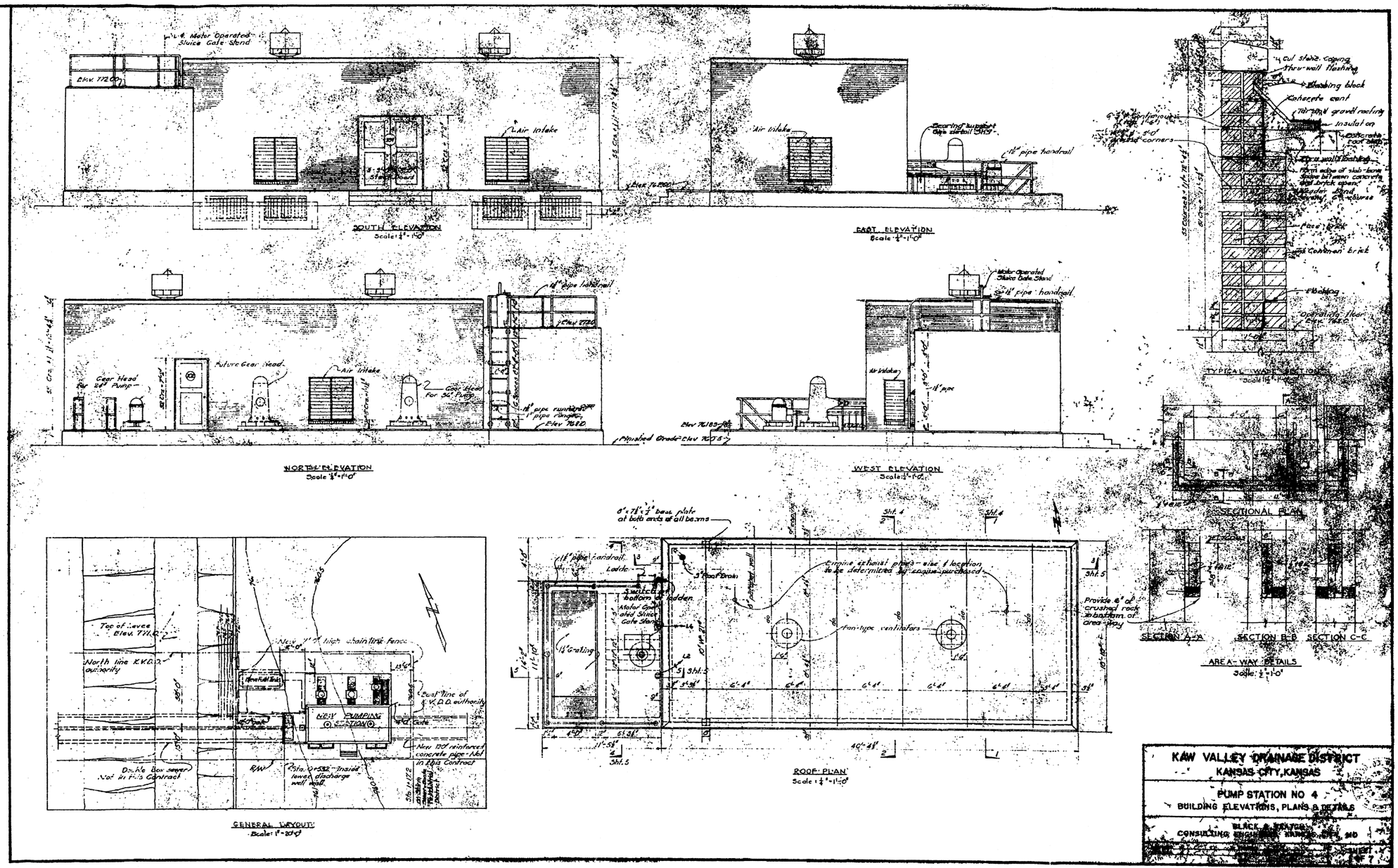
KANSAS CITY'S FLOOD CONTROL
ARGENTINE UNIT
KANSAS RIVER, KANSAS
U. S. ARMY ENGINEER DISTRICT
KANSAS CITY
FILE NO A-10-6392
FEBRUARY 1976



KAW VALLEY DRAINAGE DISTRICT
KANSAS CITY, KANSAS

PUMP STATION NO 4
BUILDING ELEVATIONS, PLANS & DETAILS

BLACK & VEATCH
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SHEET 7 OF 7

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ARGENTINE UNIT

KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT
KANSAS CITY

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FEBRUARY 1978

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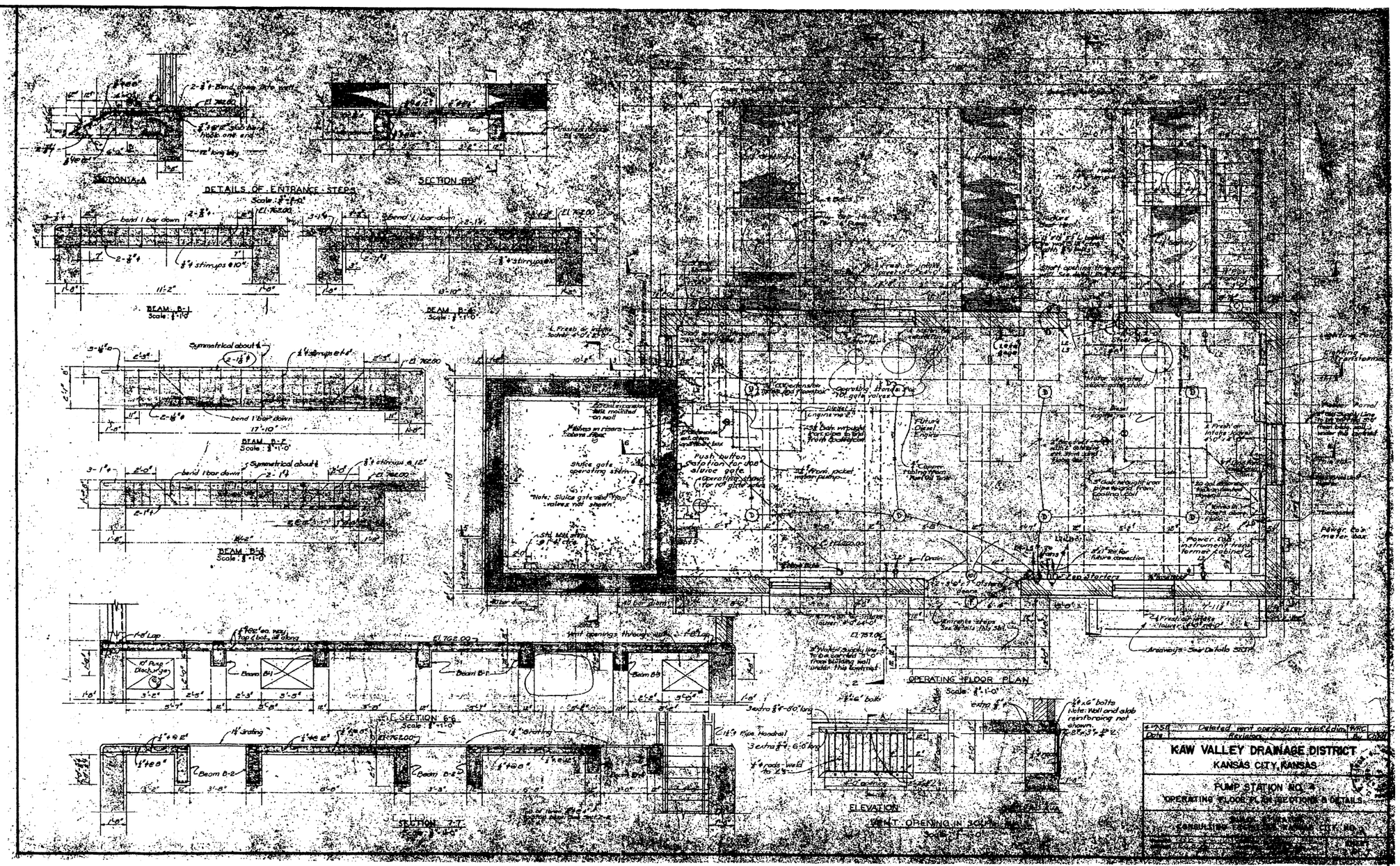
NOVEMBER 1954
CONTRACT NO. DA-23-028 ENG-[illegible]

O. & M.M. PLATE NO. 72



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KANSAS CITYS FLOOD CONTROL PROJECT

ARGENTINE UNIT

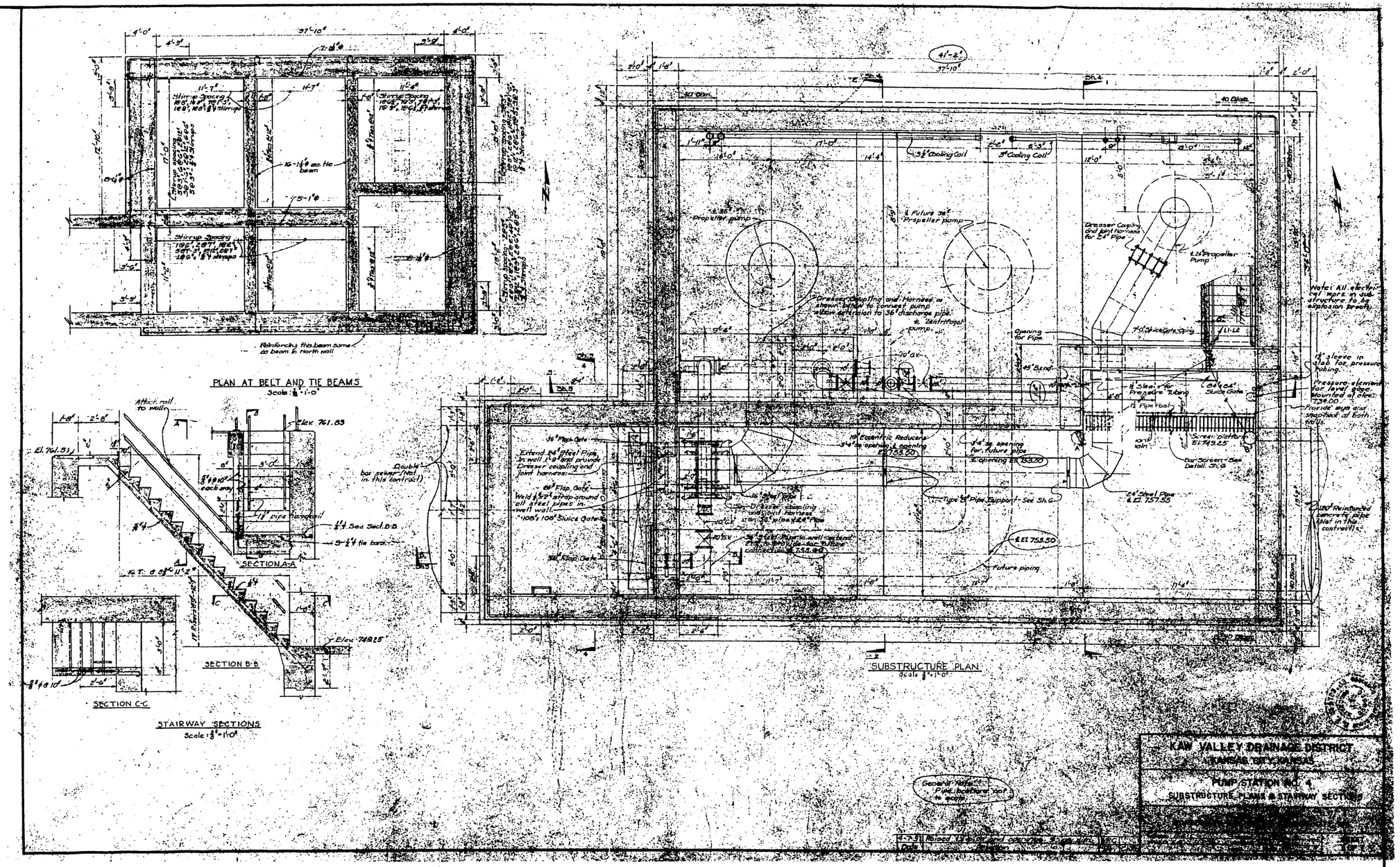
KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT
KANSAS CITY

FILE NO. A-10-6394

FEBRUARY 1978

RECORD DRAWING



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ARGENTINE UNIT

KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT
KANSAS CITY

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FEBRUARY 1978

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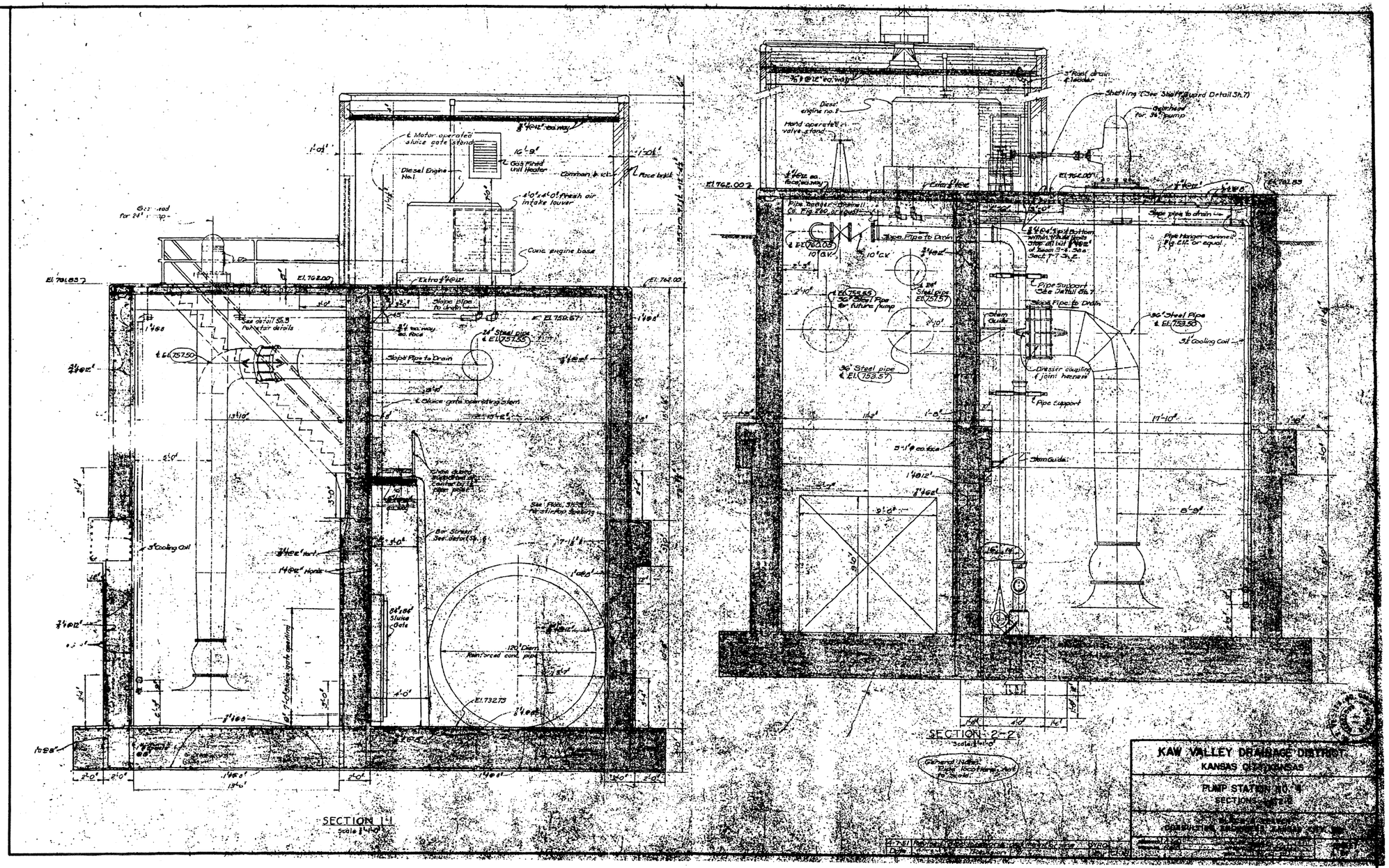

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KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT
KANSAS CITY

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FEBRUARY 1978

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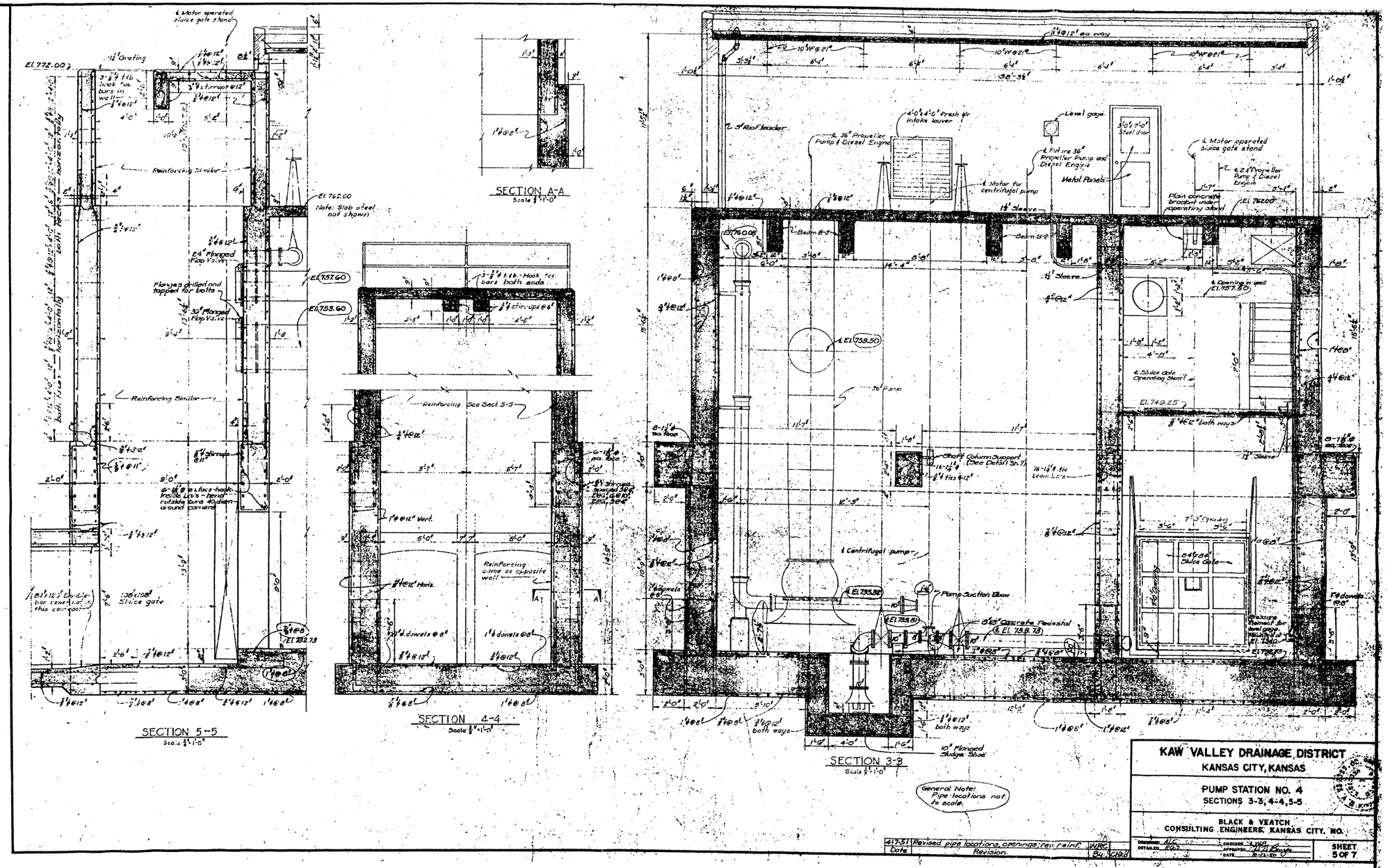
NOVEMBER 1954

CONTRACT NO. DA-[illegible]-ENG-[illegible]



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KANSAS CITY

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FEBRUARY 1978

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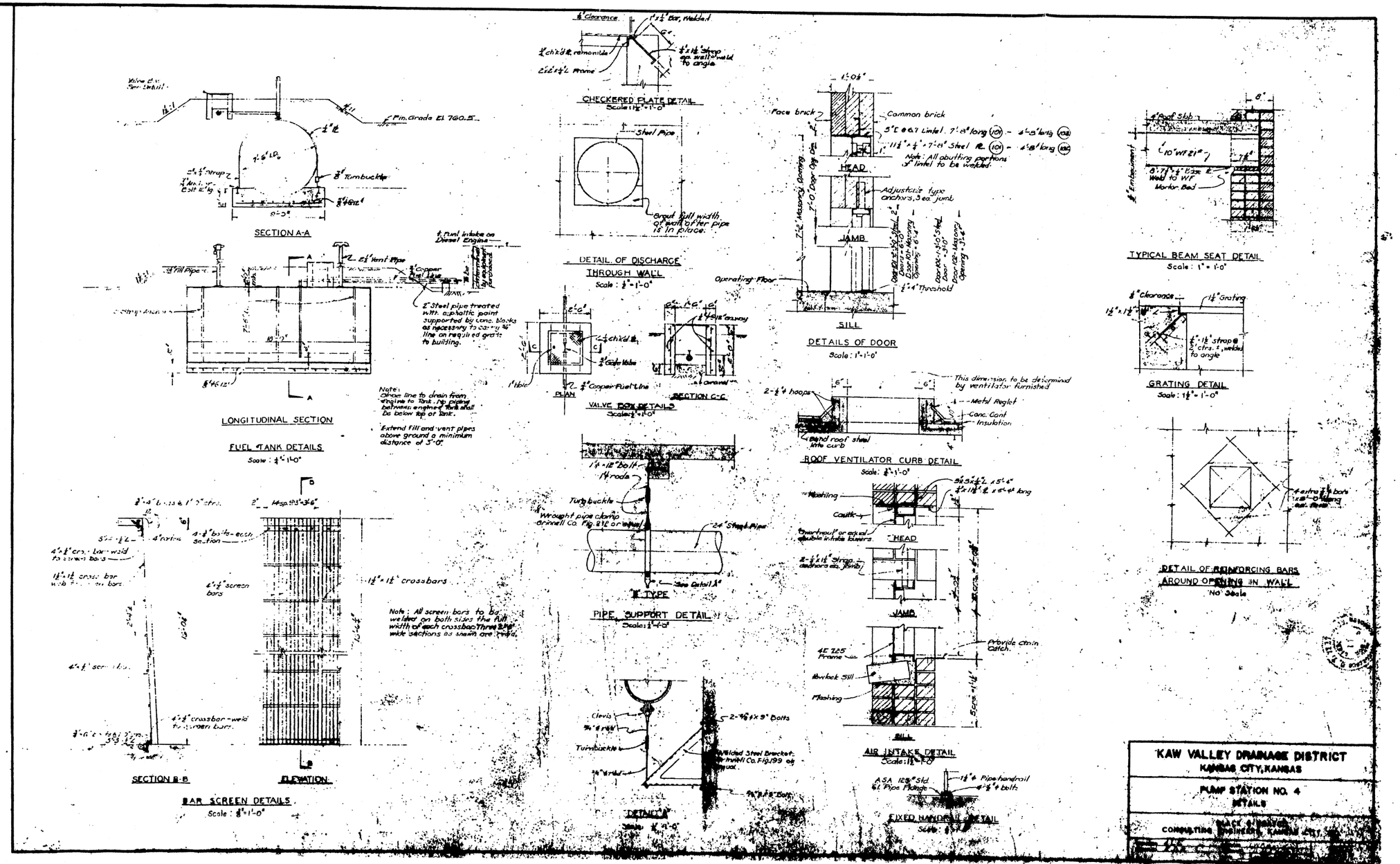
NOVEMBER 1978

CONTRACTING DIVISION

O.&M.M. PLATE NO. 76



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KANSAS RIVER, KANSAS

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KANSAS CITY

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FEBRUARY 1978

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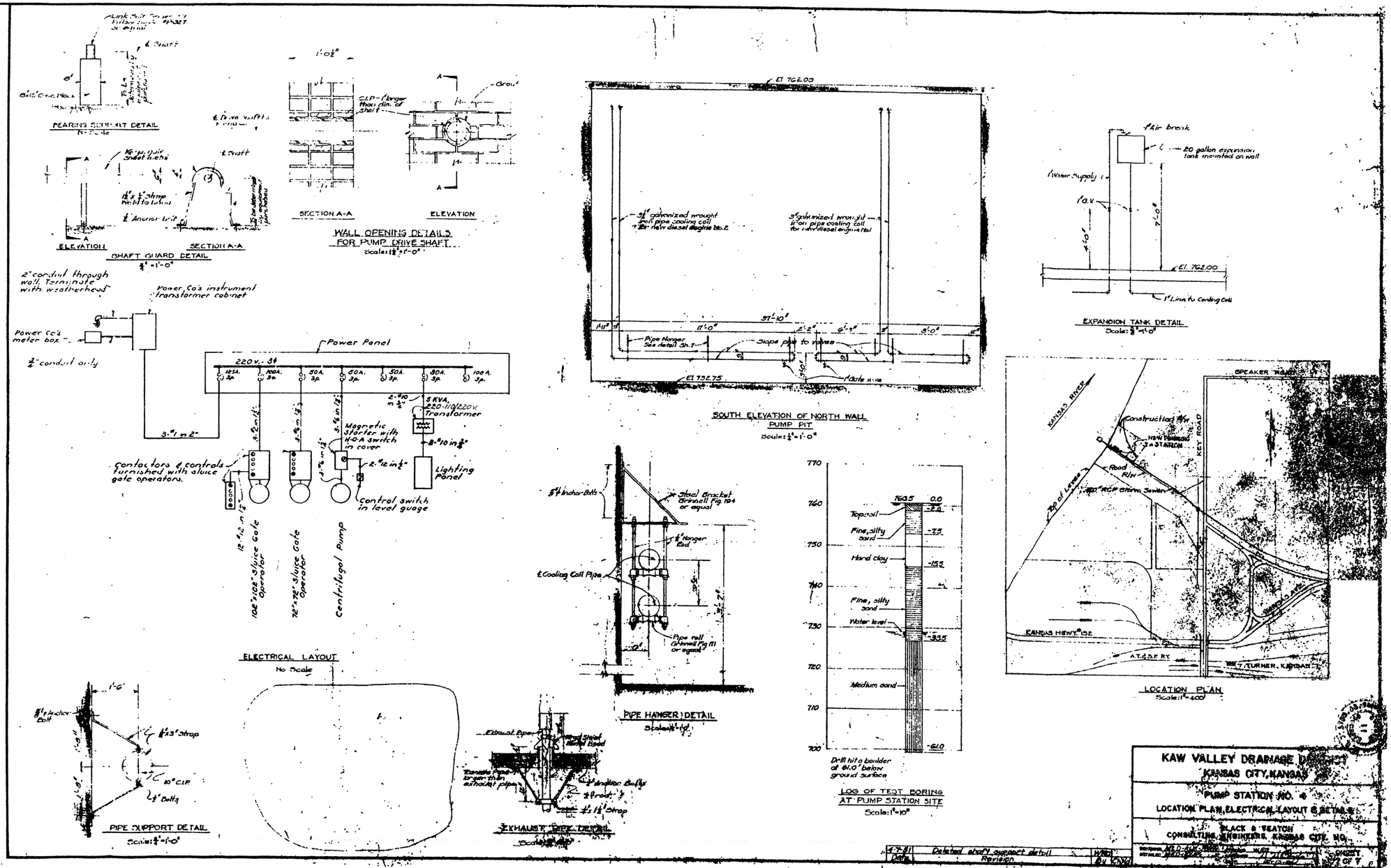
NOVEMBER 1954



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KANSAS CITY

FILE NO. A-10-6399

FEBRUARY 1978

RECORD DRAWING
NOVEMBER 1954
CONTRACT NO. DA-23-028-ENG-481

Subject: Turner Pump Plant

Allowable bearing capacity for the foundation below the existing pump plant : There is no existing boring data Standard penetration results upon which to base capacity. The recommended allowable bearing capacity of 2000 psf is considered conservative (provided in early August 2003) and should serve as the allowable for the 500 year nominal flood plus 5 feet. Based on the Armourdale studies an allowable of 2500 psf for the nominal 500 year is recommended. This capacity should result in settlement less than 2 inches total.

Scott Loehr, P.E.
Geotechnical Engineer
October 2, 2003

PLANS FOR PUMP STATION NO. 4

KAW VALLEY DRAINAGE DISTRICT

KANSAS CITY, KANSAS

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BLACK & VEATCH

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KANSAS CITY, MO.

1950

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KANSAS CITYS FLOOD CONTROL PROJECT

ARGENTINE UNIT

KANSAS RIVER, KANSAS

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KANSAS CITY

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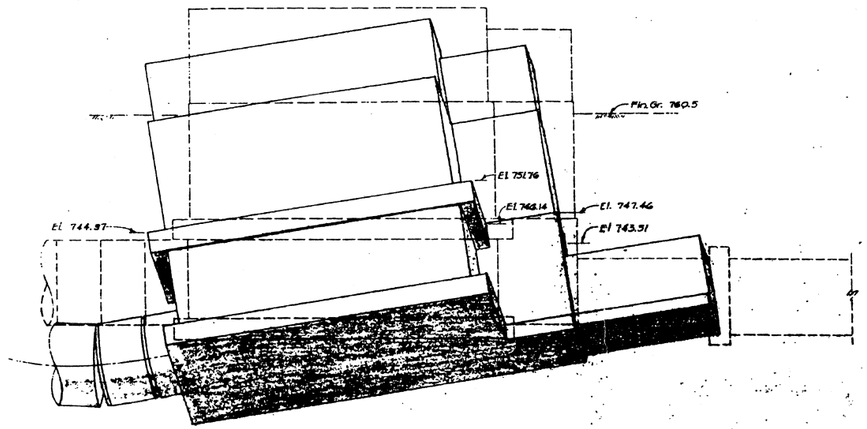
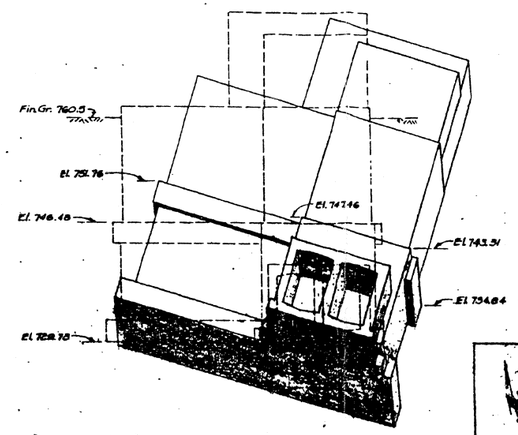


NORTH ELEVATION
Scale: 1/8" = 1'-0"



WEST ELEVATION
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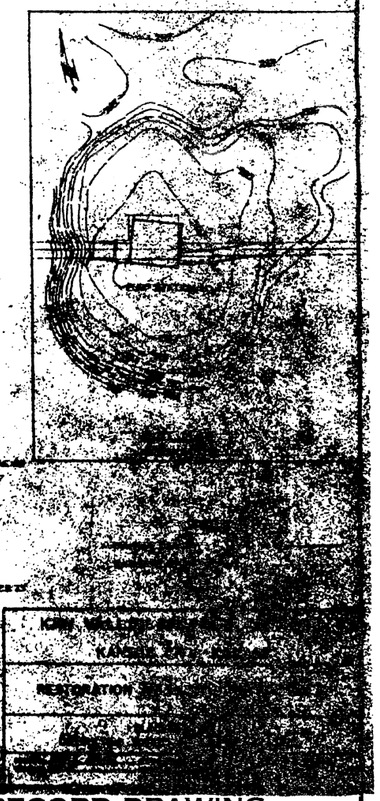
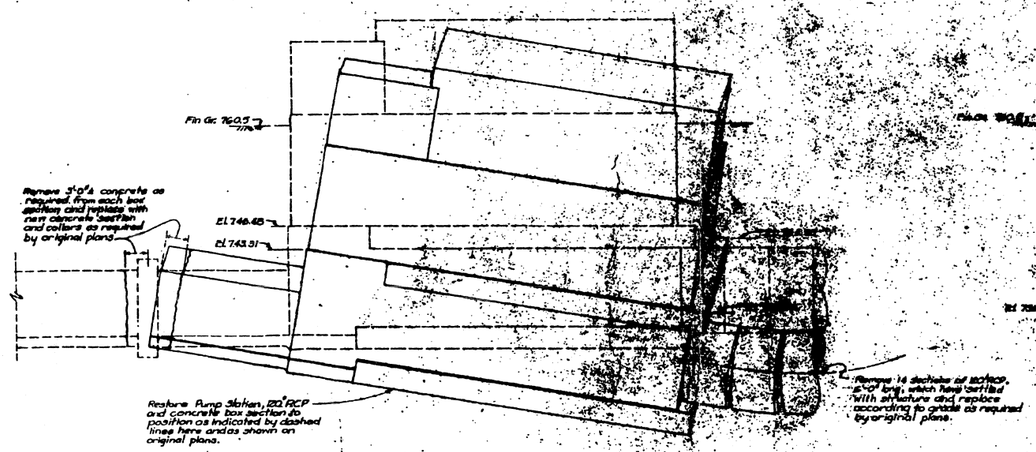
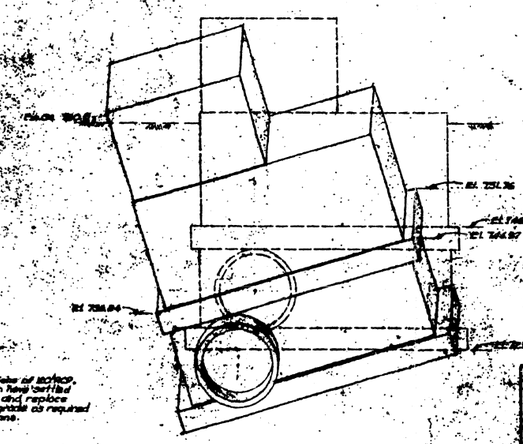




SOUTH ELEVATION
Scale: 1/8" = 1'-0"



EAST ELEVATION
Scale: 1/8" = 1'-0"

RECORD DRAWING
NOVEMBER 1954
CONTRACT NO. DA-23-028-ENG-481

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KANSAS RIVER, KANSAS
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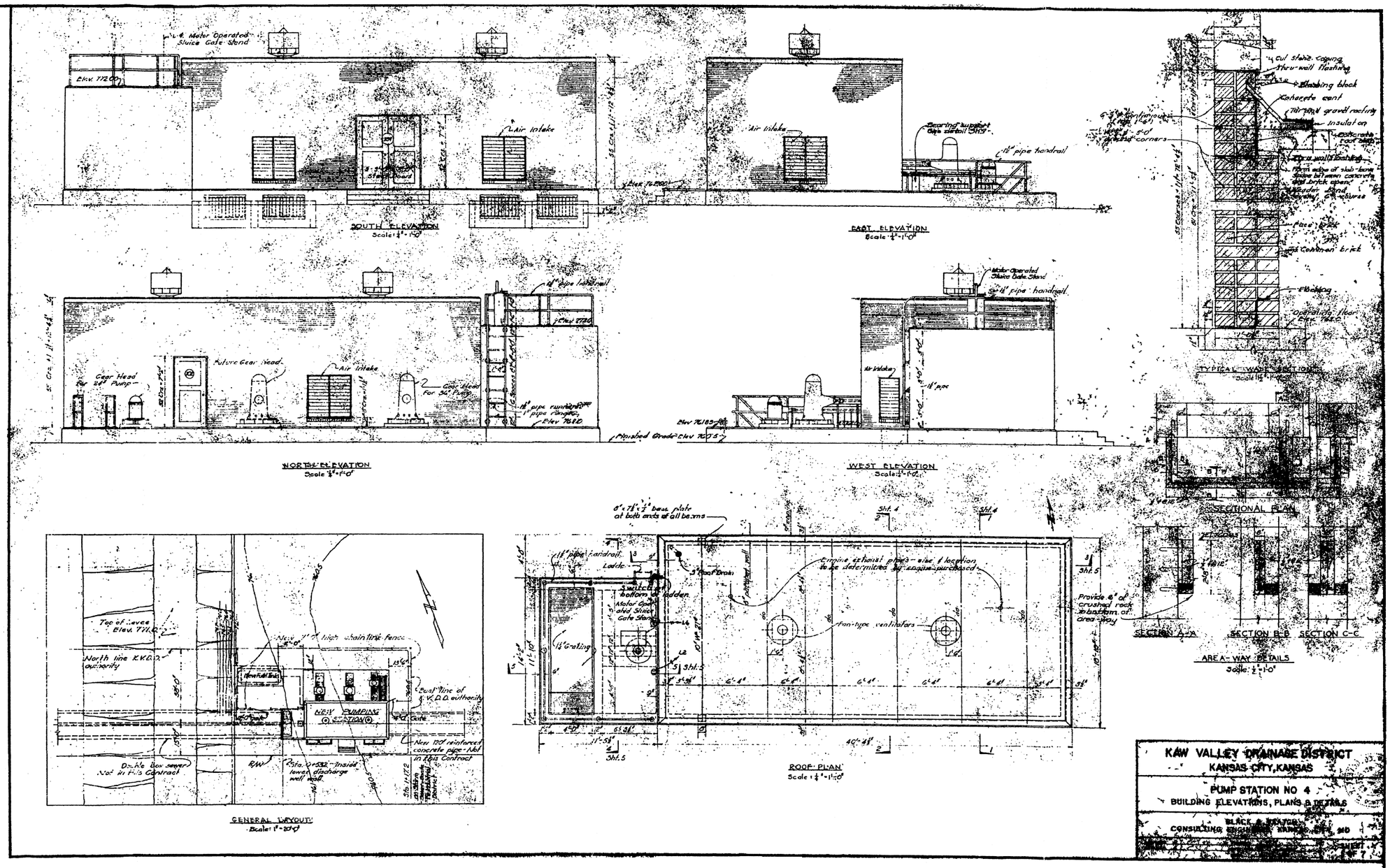
KANSAS CITY
FILE NO A-10-6392
FEBRUARY 1976



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ARGENTINE UNIT

KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT
KANSAS CITY

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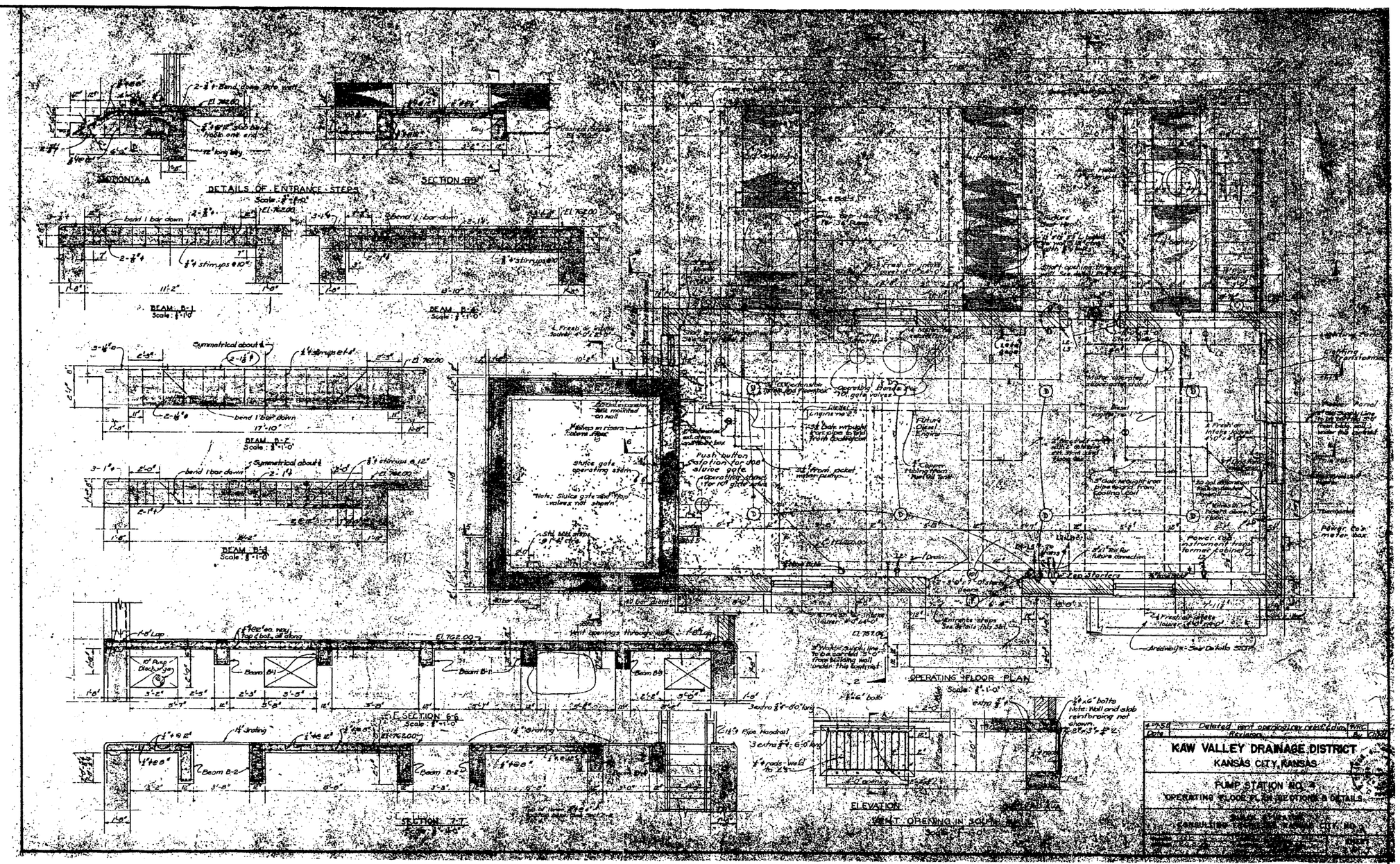
FEBRUARY 1978

RECORD DRAWING

NOVEMBER 1954

CONTRACT NO. DA-23-028 ENG-[illegible]

O. & M.M. PLATE NO. 72



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ARGENTINE UNIT

KANSAS RIVER, KANSAS

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KANSAS CITY

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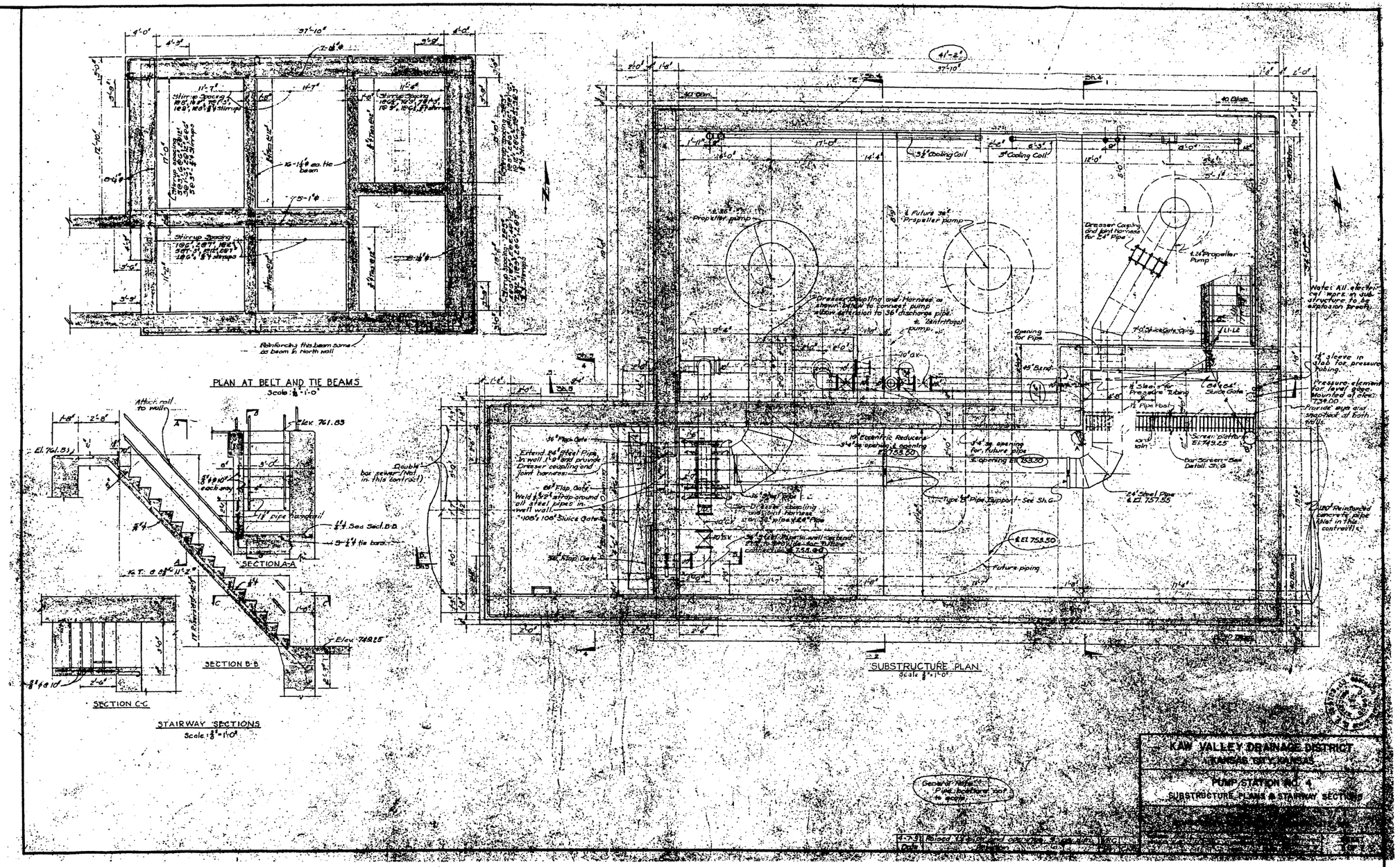
FEBRUARY 1978

RECORD DRAWING



SYM.	DESCRIPTION	DATE	APP'D.
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KANSAS CITYS FLOOD CONTROL PROJECT

ARGENTINE UNIT

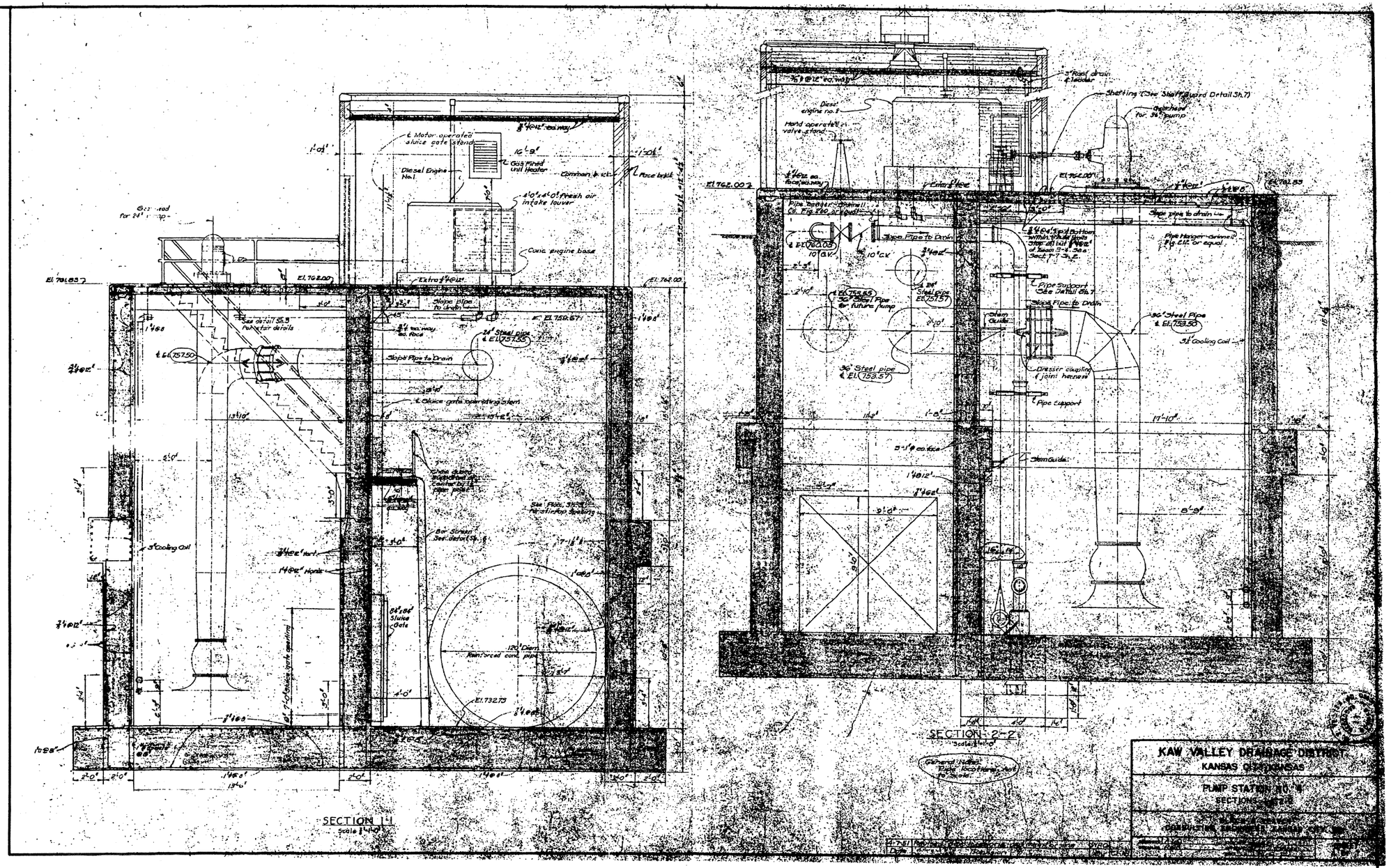
KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT
KANSAS CITY

FILE NO. A-10-6395

FEBRUARY 1978

RECORD DRAWING



SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" cond.		

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KANSAS CITYS FLOOD CONTROL PROJECT

ARGENTINE UNIT

KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT

KANSAS CITY

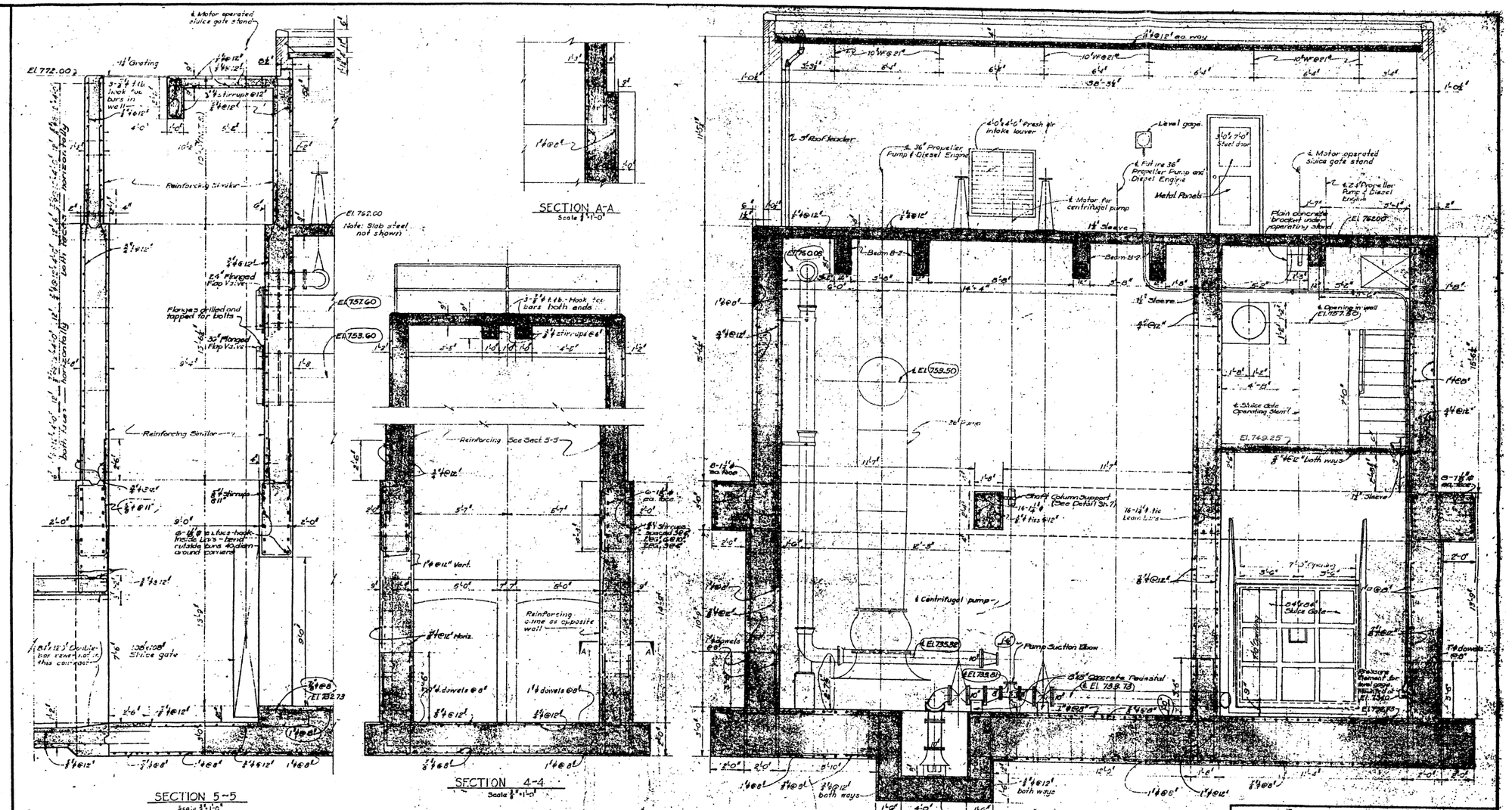
FILE NO. A-10-6396

FEBRUARY 1978

RECORD DRAWING

NOVEMBER 1954

CONTRACT NO. DA-25-025-ENG-56



Revised for "As Built" cond.

SYM.	DESCRIPTION	DATE	APP'D.

REVISED BY CORPS OF ENGINEERS

KANSAS CITYS FLOOD CONTROL PROJECT

ARGENTINE UNIT

KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT

KANSAS CITY

FILE NO. A-10-6397

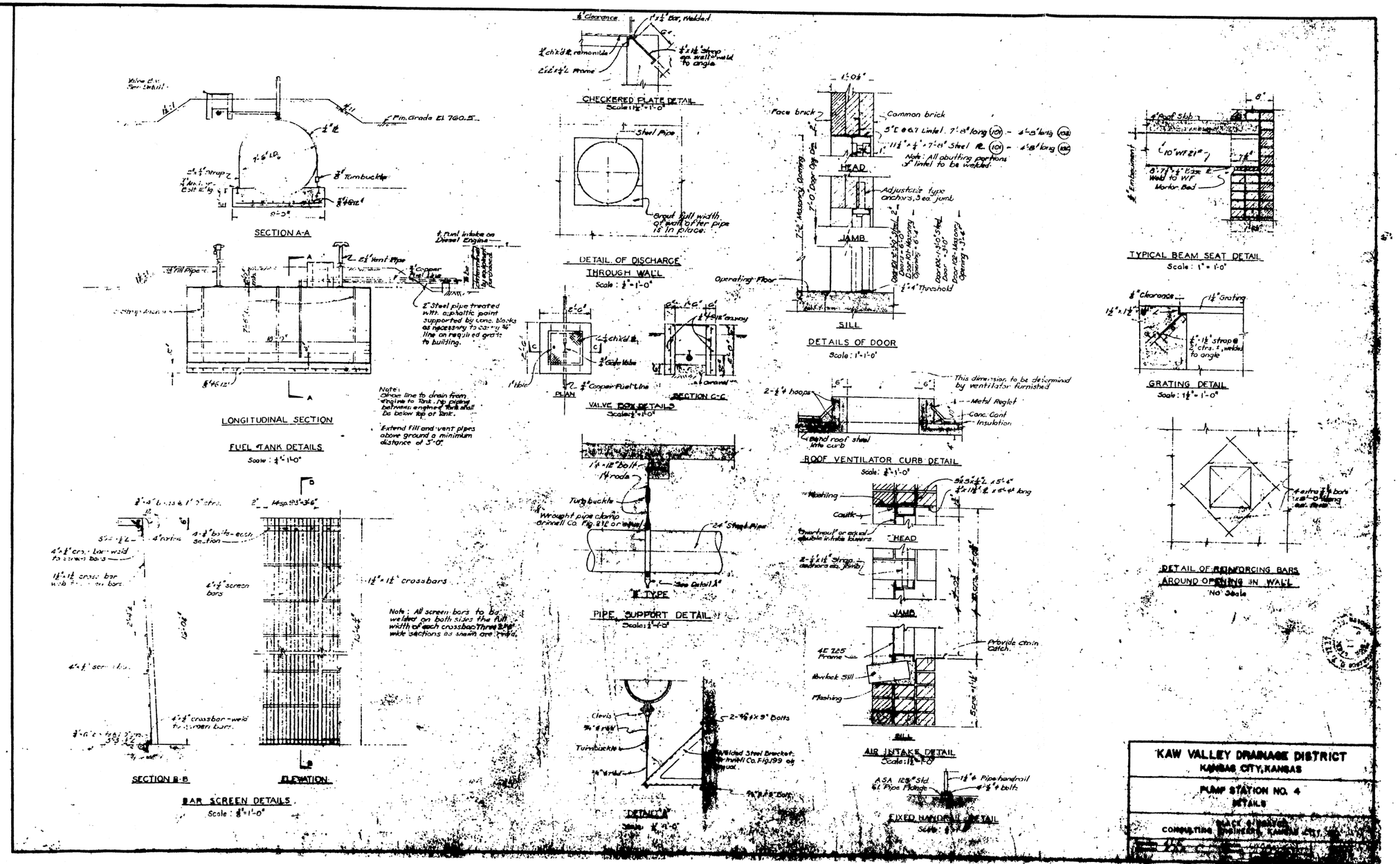
FEBRUARY 1978

RECORD DRAWING

NOVEMBER 1983

CONTRACTING DIVISION

O.&M.M. PLATE NO. 76



SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" cond.	2/9/78	
	REVISED BY CORPS OF ENGINEERS		

KANSAS CITYS FLOOD CONTROL PROJECT

ARGENTINE UNIT

KANSAS RIVER, KANSAS

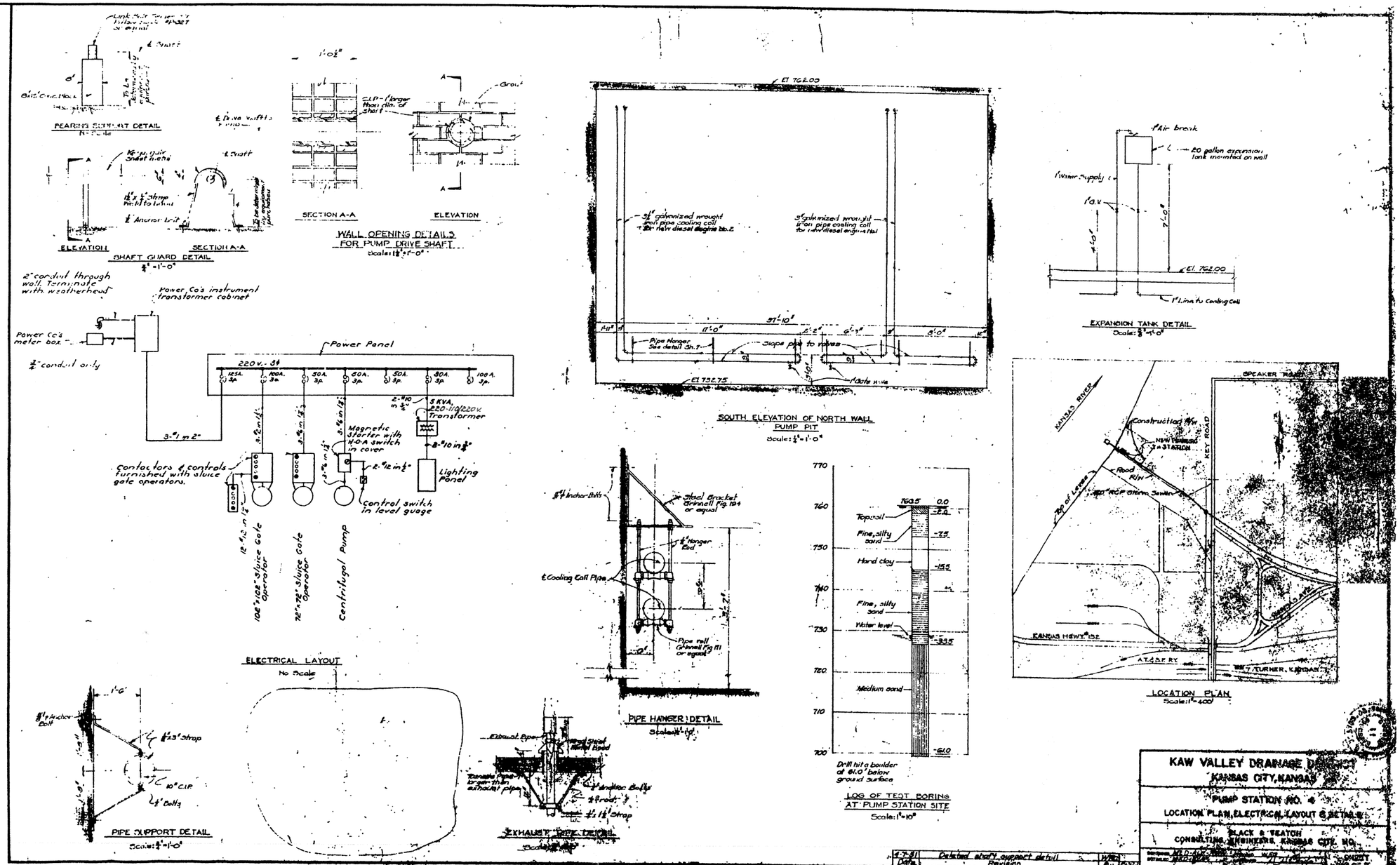
U. S. ARMY ENGINEER DISTRICT
KANSAS CITY

FILE NO. A-10-6398

FEBRUARY 1978

RECORD DRAWING

NOVEMBER 1954



SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" cond.	2/7/78	[illegible]

REVISED BY CORPS OF ENGINEERS

KANSAS CITYS FLOOD CONTROL PROJECT

ARGENTINE UNIT

KANSAS RIVER, KANSAS

U. S. ARMY ENGINEER DISTRICT
KANSAS CITY

FILE NO. A-10-6399

FEBRUARY 1978

RECORD DRAWING
NOVEMBER 1954
CONTRACT NO. DA-23-028-ENG-481

KANSAS CITYS, MISSOURI AND KANSAS
SEVEN LEVEES FEASIBILITY STUDY

PUMP STATION EVALUATIONS BY ST. LOUIS DISTRICT
TURNER PUMP STATION
PUMP STATION MECHANICAL SUMMARY

Levee Unit:	Argentine
Levee Station:	Sta. 60+40
Type of Raise:	Levee Raise
Flood Event:	500yr+3
Proposed Protection Level:	Elevation 780.8
No. of Pumps:	4

1. Data. The data gathered and used to evaluate the pump station includes the following:

1.1 O&MM Drawings. Construction drawings of the original pump station and the modifications to the pump station where the retaining wall and new pumps were added were provided in the Operation and Maintenance Manual drawings for the Argentine Unit. These O&MM drawings were provided by the Kansas City District in scanned image format (*.tif files).

1.2 Existing Pump Operating Condition. The Turner Pump Station has four pumping units. Two of these pumps (Pump Nos. 1 and 2) were installed when the pump station was originally constructed. These pumps are Fairbanks-Morse vertical line-shaft axial flow pumps driven by diesel engines through right angle speed reducers. The pumps discharge through steel discharge pipes into the discharge chamber. The two other pumps (Pump Nos. 3 and 4) were installed recently. These pumps are vertical line-shaft mixed flow pumps driven by vertical electric motors. These pumps discharge through steel discharge pipes, which were installed outside of the pump station, and terminate at the top of the existing discharge chamber. The original pump curves for Pump Nos. 1 and 2 are not available. St. Louis District records were checked for pump stations constructed in the same time period as the Turner Pump Station. The Cahokia Pump Station, which was constructed as part of the East St. Louis Flood Protection Project in the early 1950's, was found to have Fairbanks-Morse vertical pumps that were the same type, size, speed and horsepower. The pump curves for these pumps were in the files. These curves were assumed to be similar to those for Pump Nos. 1 and 2. The pump curves for Pump Nos. 3 and 4 were given to the Kansas City District by the Kaw Valley Levee Distirct. CHARTS 15-1-1 thru 15-1-5 give details of the existing pumps.

1.3 Existing Mechanical Equipment. The Turner Pump Station has two cast iron sluice gates. There is a 108" x 108" sluice gate located in the discharge chamber that is used to block gravity flow during high water levels on the Kansas River. This sluice gate is operated using an electric motor actuated sluice gate hoist located at the top of the discharge chamber. There is a 84" x 84" sluice gate located between the pump sump from the gravity flow sewer that allows water to get to the pumps when the pump station is in operation. This sluice gate is operated using an electric motor actuated sluice gate hoist located on the pump station operating floor.

2. **Pump Station Pump Capacity Analysis.**

2.1 Current Pumping Conditions. River levels, sump levels and capacities were determined from the Supplement on Interior Drainage (1950), the Existing Conditions Report and from site visits to the pump station including discussions with Kaw Valley Levee District personnel. There were two critical design Kansas River flood levels determined from these sources. The original design flood level of El. 753.0 and the design maximum flood level of El. 774.3. System head loss curves were developed for each of the resulting static head conditions. The system head loss curves were plotted on the pump curves for each of

the three size pumps. The computations of the system head loss values and the plotting of these resulting curves on the existing pump curves can be found on CHARTS 15-2-1 thru 15-2-21. Following is a summary of the existing pump station capacities –

	River Level El. 753.0	River Level El. 774.3
Sump El. 738.3		
Pump No. 1	14,050 gpm	8,150 gpm
Pump No. 2	25,150 gpm	17,500 gpm
Pump No. 3	12,200 gpm	12,200 gpm
Pump No. 4	12,200 gpm	12,200 gpm
Total Capacity	63,600 gpm (144.7 cfs)	50,050 gpm (111.5 cfs)

	River Level El. 753.0	River Level El. 774.3
Sump El. 743.6		
Pump No. 1	14,950 gpm	9,900 gpm
Pump No. 2	26,750 gpm	20,500 gpm
Pump No. 3	12,300 gpm	12,300 gpm
Pump No. 4	12,300 gpm	12,300 gpm
Total Capacity	66,300 gpm (150.5 cfs)	55,000 gpm (122.5 cfs)

2.2 Future Pumping Conditions. The Design River Level at the Turner Pump Station River will become El. 780.8. The sump levels of El. 738.3 and El. 743.6 remain the same. System head loss curves were developed for each of the resulting static head conditions. The system head loss curves were plotted on the pump curves for each of the three size pumps. The computations of the system head loss values and the plotting of these resulting curves on the existing pump curves can be found on CHARTS 15-3-1 thru 15-3-11. Following is a summary of the existing pump station capacities –

	River Level El. 780.8
Sump El. 738.3	
Pump No. 1	Off the curve
Pump No. 2	13,000 gpm
Pump No. 3	12,150 gpm
Pump No. 4	12,150 gpm
Total Capacity	37,300 gpm (83.1 cfs)

River Level El. 780.8

Sump El. 743.6

Pump No. 1	7,700 gpm
Pump No. 2	16,800 gpm
Pump No. 3	12,250 gpm
Pump No. 4	12,250 gpm
Total Capacity	49,000 gpm (109.2 cfs)

2.3 Capacity at Design River Condition. The analysis provided above indicates a decrease in pumping capacity from the existing Maximum Design River (El. 774.3) to the proposed Maximum Design River (El. 780.8) of the following –

Sump Level 738.3	Station Capacity Decrease – 12,750 gpm
Sump Level 743.6	Station Capacity Decrease – 6,000 gpm

2.4 Alternatives. A number of alternatives were considered to mitigate for this decrease in pump station capacity. These alternatives included modifying the existing pumps to increase their capacity, installing a new (5th) pump and replacing one of the existing pumps with a new, higher capacity unit. Since Pump No. 1 was determined to be unusable at the higher design river elevation it was determined that replacement of this pump would provide the best solution. Replacement of this pump resulted in two alternatives types of pumps to be installed. The first would be to replace the existing pump with a similar type pump that was larger. The second would be to replace the existing pump with an electric submersible pump. For the vertical pump, data and costs were obtained from Fairbanks-Morse Pump. For the electric submersible pump, data and costs were obtained from Flygt Pump. Following are advantages/disadvantages for each of these alternatives –

Similar Type Vertical Pump (w/gear and diesel engine) -

Advantages	Disadvantages
Similar Layout	Increase in Discharge Pipe Size
Operating Costs Not Dependent on Utility Rates	Increase in Unit Size and Weight (analyze operating floor structure)

Electric Submersible Pump -

Advantages	Disadvantages
Less Pump Unit Maintenance	Operating Costs Dependent on Utility Rates
Same Size Discharge Pipe	Increased Electrical Equipment
Weight Reduction at the Operating Level with Pump Installed on the Sump Floor	

2.5 Recommended Plan. The recommended plan is to replace Pump No. 1 with an electric submersible pump. This type of pump with the recommended installation configuration can be found on Chart 15-4-1. The cost estimate for this installation including the increased electrical equipment can be found on Chart 15-4-2.

3. **Pump Station Sluice Gates.**

3.1 Effects of Levee Raise. The levee raise will result in the top of the discharge chamber being raised. When the discharge chamber is raised the 108” x 108” gravity drain sluice gate stem will be too short and will have to be replaced. The electric motor actuated sluice gate hoist will need to be relocated to the top of the discharge chamber addition. The 84” x 84” sluice gate, located inside the pump station, will not be affected by the levee raise.

3.2 Recommended Plan. It is recommended that the existing stem on the 108” x 108” gravity drain be replaced with a new stainless steel stem. New gate stem guides should also be provided at correct spacing along the discharge chamber wall. In order to assure that the stem operates efficiently with the electric motor actuated gate hoist it is recommended that the existing electric motor actuated gate hoist be replaced with a new one.

4. **Cost Estimates.**

4.1 Pump Replacement. Cost estimates are presented at the end of this report for both alternatives for replacing Pump No. 1 – replacement with a similar type pump/diesel engine and replacement with an electric motor driven submersible pump (recommended plan).

4.2 Gravity Drain Sluice Gate. A cost estimate is present at the end of this report for installing a new longer gate stem and for providing a new electric motor actuated gate hoist at the top of the new discharge chamber.

PROJECT KC Flood Reduction Study	Page 1 of 5	COMPUTED BY SAF	DATE 2/12/04
SUBJECT Determine Existing Conditions		CHECKED BY	DATE

1. Existing Conditions -

Design Water Surface - use 2'-6" below the top of the discharge chamber

Top of Discharge Chamber - El. 776.8
2.5
El. 774.3

Sump Water Elev. -

Pump No 1 - 8'-0" from sewer invert

Pump No 2 - 10'-0" from sewer invert

Sewer Invert - El. 732.75 ⇒ El. Use 740.75 as the Design Sump Elev.

Existing Pumps -

Pump #1 - Fairbanks - Morse - Niagra Propeller Pump
Size - 18 Single Stage Speed - 870 rpm

Curve not available for existing pump

Use curve for a Fairbank Morse Figure 6360 Propeller Pump
Size - 18 Single Stage Speed - 870 rpm
used at Cahokia Pump Station in the E. St Louis Flood Protection Project
Installed in 1950 See Chart 15-1-3

Pump #2 - Fairbanks Morse Figure 6360
Size - 24 Single Stage Speed - 740 rpm

Curve not available for existing pump

Use curve for a Fairbanks Morse Figure 6360 Propeller Pump
Size - 24 Single Stage Speed - 700 rpm
used at Cahokia Pump Station in the E. St Louis Flood Protection Project
Installed in 1950 See Chart 15-1-4

CHART 15-1-1

Pump #3 + Pump No. 4 -

Vertiline Pump - Layne Bowler Pump Co
10,300 gpm 97 ft 1180 rpm 18-in Discharge
Curve provided - See Chart 15-1-5

Original Pump Station Capacity (Pump Nos. 1 and 2) -

At Design River El. 774.3 per data in the Existing Conditions Report

Sump Level 738.3 - 18 cfs (8078.4 gpm)

Sump Level 743.6 - 35.5 cfs (15,932.4 gpm)

At River El. 753.0 per the Supplement on Interior Drainage (1950)

Capacity - 29,500 gpm (65.7 cfs)

Local Interests at that time desired another 26,750 gpm (59.6 cfs)

CHART 15-1-2

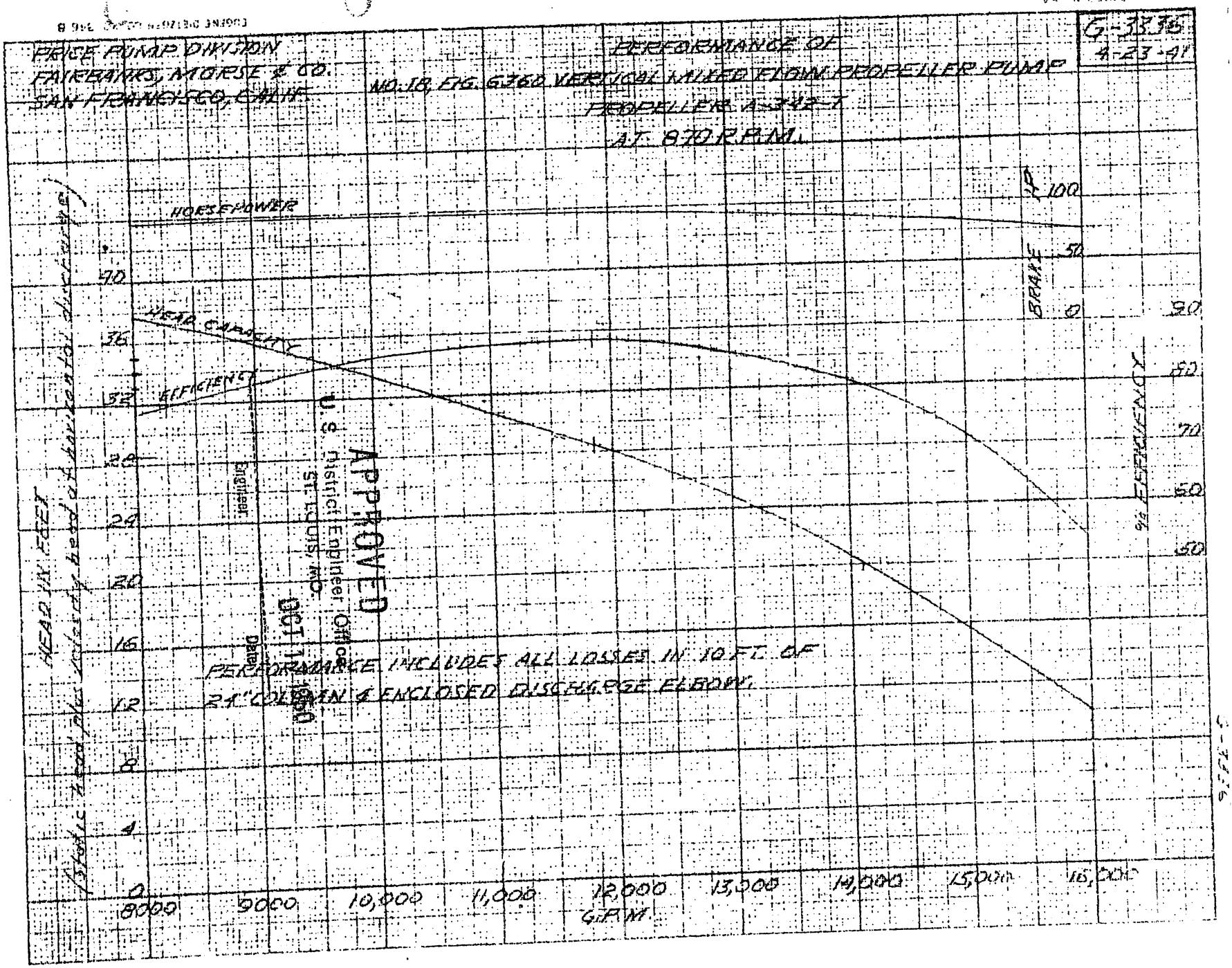
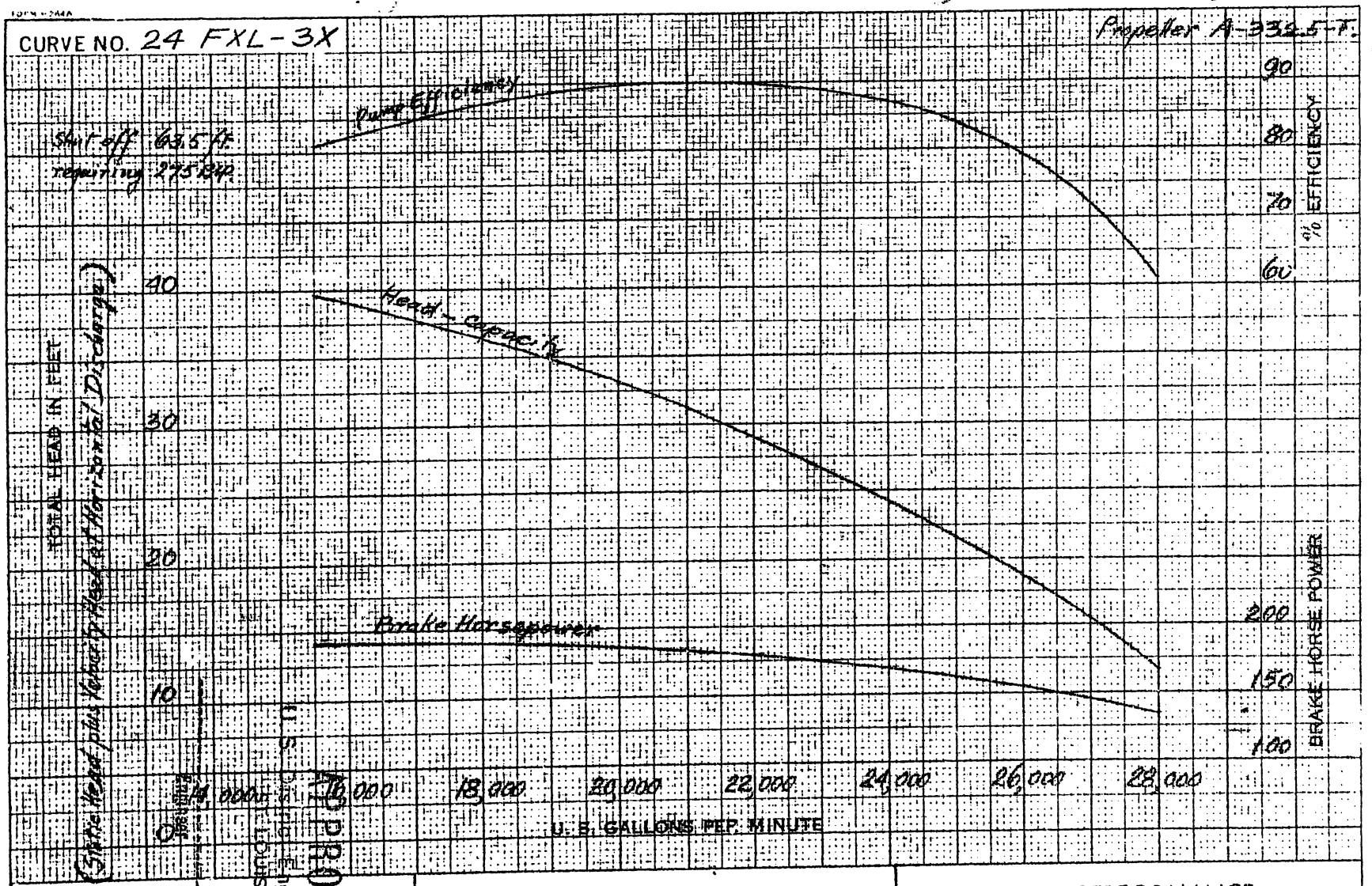

PRICE PUMP DIVISION
FAIRBANKS, MORSE & CO.
SAN FRANCISCO, CALIF.
PERFORMANCE OF
NO. 18 FIG. 6760 VERTICAL MIXED FLOW PROPELLER PUMP
PROPELLER A-342-1
AT 870 R.P.M.
G-3336
4-23-41
HORSEPOWER
HEAD CAPACITY
EFFICIENCY
HP
BRAKE
EFFICIENCY %
HEAD IN FEET
(STATIC HEAD PLUS LOSSES IN DISCHARGE PIPE)
PERFORMANCE INCLUDES ALL LOSSES IN 10 FT. OF
24" COLUMN & ENCLOSED DISCHARGE ELBOW.
GPM
8000
9000
10,000
11,000
12,000
13,000
14,000
15,000
16,000
APPROVED
U.S. District Engineer Office
St. Louis, Mo.
OCT 1 1940
Chief Engineer
Detail

CHART 1-3



The pump is guaranteed for the use of the conditions specified, other points on the curve are approximate. Capacity, head and efficiency guarantees are contingent on the pump being furnished with the specified amount of clear, fresh non aerated water at a temperature of not to exceed 85 degrees Fahrenheit.

9-26-47

FAIRBANKS, MORSE & CO.

POMONA WORKS

POMONA • CALIFORNIA

FIELD PERFORMANCE

One Stage No 24 Fig. 6360 Mixed Flow Propeller

Pump Operated at 700 R.P.M.

All Losses in 20 Ft. of 36" Column & Enclosed Type Disch. Elbow are charged against pump.

CHART 15

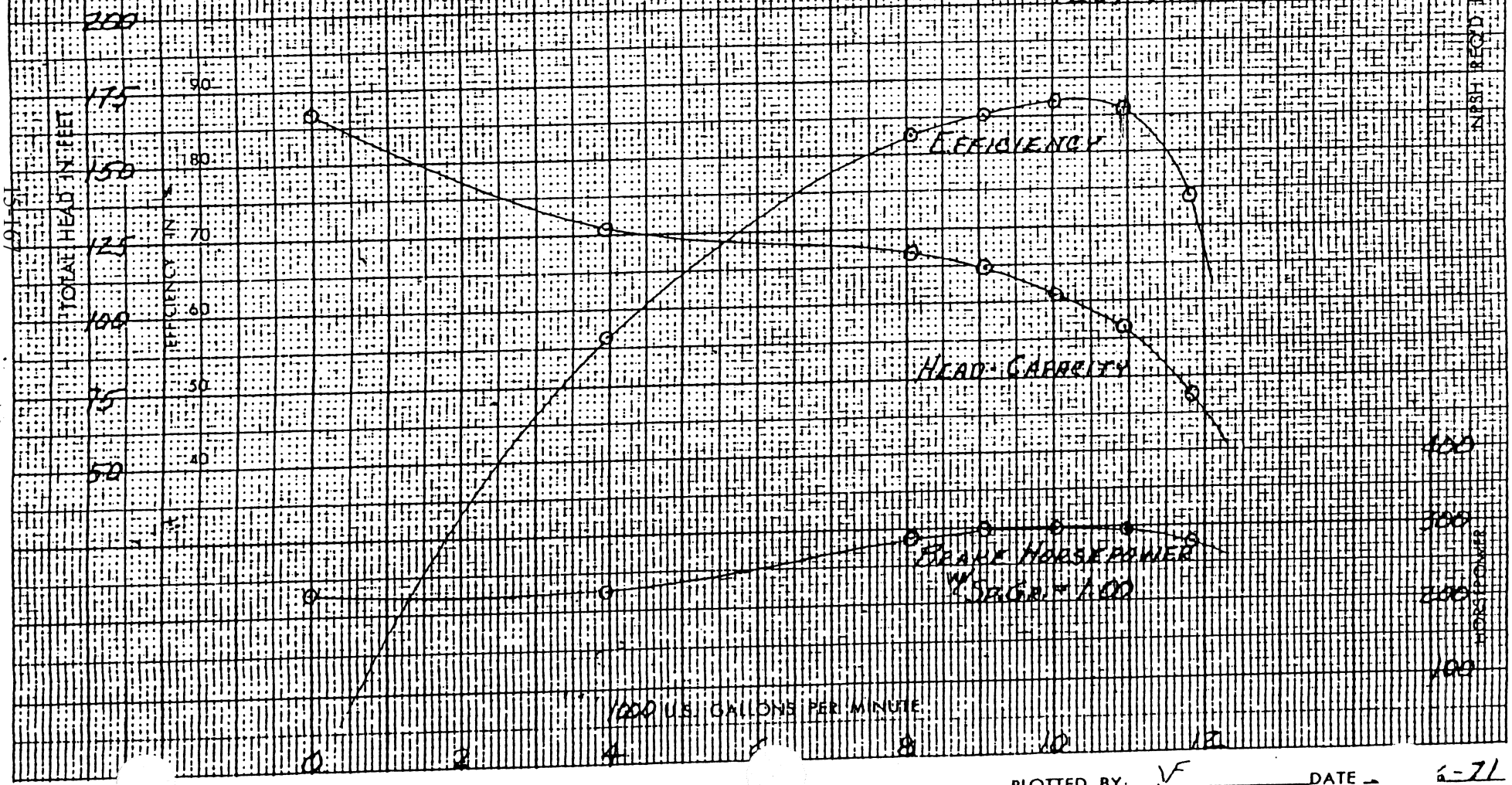
Turner Pump Station
Station 60 + 40

CURVE NO. 107

LAYNE & BOWLER PUMP COMPANY
LOS ANGELES 22, CALIF.

LAYNE & BOWLER VERTI-LINE PUMPS

LABORATORY PERFORMANCE
1 STAGE 285KKL PUMP
1180 R.P.M.
FOR
~~Johnson County Water Dist #1~~
KANSAS
Low Service Pump #3
Pump D27897-A
Test 71-107



PLOTTED BY: VF DATE 6-71
CERTIFIED BY W. F. P. DATE 3-11-71

CHART 15-1-2

STEP 1 - Determine pump operating capacities at -

River Elevations - El. 774.3 and El. 753.0

Sump Elevations - El. 738.3 and El. 743.6

Pump # 1 -

Static Heads

1. River El 774.3 - Sump El. 738.3 = 36.0 ft
2. River El 774.3 - Sump El. 743.6 = 30.7 ft
3. River El 753.0 - Sump El. 738.3 = 14.7 ft
4. River El 753.0 - Sump El. 743.6 = 9.4 ft

Develop System Loss Curves for each static head condition

24 in discharge pipe - 42 ft (22.624 in. ID)

1 - 30° Miter L/D = 8 k = 0.10

1 - 90° Elbow L/D = 20 k = 0.24

1 - Discharge flap gate

Use Hazen and Williams for friction losses in pipe

$$h_f = 0.002083\, L \left(\frac{100}{C}\right)^{1.85} \times \frac{Q^{1.85}}{d^{4.8655}}$$

where,

L = length of pipe

C = friction factor ; 100 for steel pipe

Q - flow in gpm

d - pipe diameter (in.) = 22.624 in

CHART 15-2-1

Pipe Length -

Straight Pipe	42.0
30° Miter 2×8	16.0
90° Elbow 2×20	40.0
	98.0 ft

Example - for 10,000 gpm

$$h_f = 0.002083\,(98)\left(\frac{100}{100}\right)^{1.85} \times \frac{(10{,}000)^{1.85}}{(22.624)^{4.8655}}$$

$$= 1.94 \text{ ft}$$

System Losses = h_f + vel. head + flap gate loss

flap gate loss - use the max. for a 24-inch gate = 0.1 ft (see Chart 15-2-21)

vel. head = $V^2/2g = V^2/2(32.174)$

for 10,000 gpm $V = 7.98$ ft/sec vel. head = $(7.98)^2/2(32.174) = 0.99$ ft

System Loss for 10,000 gpm = 1.94 + 0.99 + 0.1 = 3.03 ft

System Loss for each of the 4 static heads

1. 36.0 + 3.03 = 39.03 ft
2. 30.7 + 3.03 = 33.73 ft
3. 14.7 + 3.03 = 17.73 ft
4. 9.4 + 3.03 = 13.43 ft

A set of data with friction head losses and system head losses was developed. The corresponding system head loss curves were then plotted on the pump curve being used for Pump No. 1

CHART 15-2-3 is the spreadsheet of the system head losses at the various static heads

CHARTS 15-2-4 - 15-2-7 are the system loss curves for each static head

CHART 15-2-2

TURNER PUMP STATION

SYSTEM HEAD LOSSES - OPERATING PUMP NO. 1

FLOW (gpm)	VELOCITY (ft/sec)	VELOCITY HEAD (ft.)	FRICTION LOSS (ft.)	FLAP GATE LOSS (ft.)	TOTAL SYSTEM LOSSES (ft.)	STATIC HEAD = 36.0	STATIC HEAD = 30.7	STATIC HEAD = 14.7	STATIC HEAD = 9.4
8000	6.39	0.63	0.87	0.10	1.60	37.60	32.30	16.30	11.00
9000	7.18	0.80	1.60	0.10	2.50	38.50	33.20	17.20	11.90
10000	7.98	0.99	1.94	0.10	3.03	39.03	33.73	17.73	12.43
11000	8.78	1.20	2.32	0.10	3.62	39.62	34.32	18.32	13.02
12000	9.58	1.43	2.72	0.10	4.25	40.25	34.95	18.95	13.65
13000	10.38	1.67	3.16	0.10	4.93	40.93	35.63	19.63	14.33
14000	11.17	1.94	3.62	0.10	5.66	41.66	36.36	20.36	15.06
15000	11.97	2.23	4.11	0.10	6.44	42.44	37.14	21.14	15.84
16000	12.77	2.53	4.64	0.10	7.27	43.27	37.97	21.97	16.67

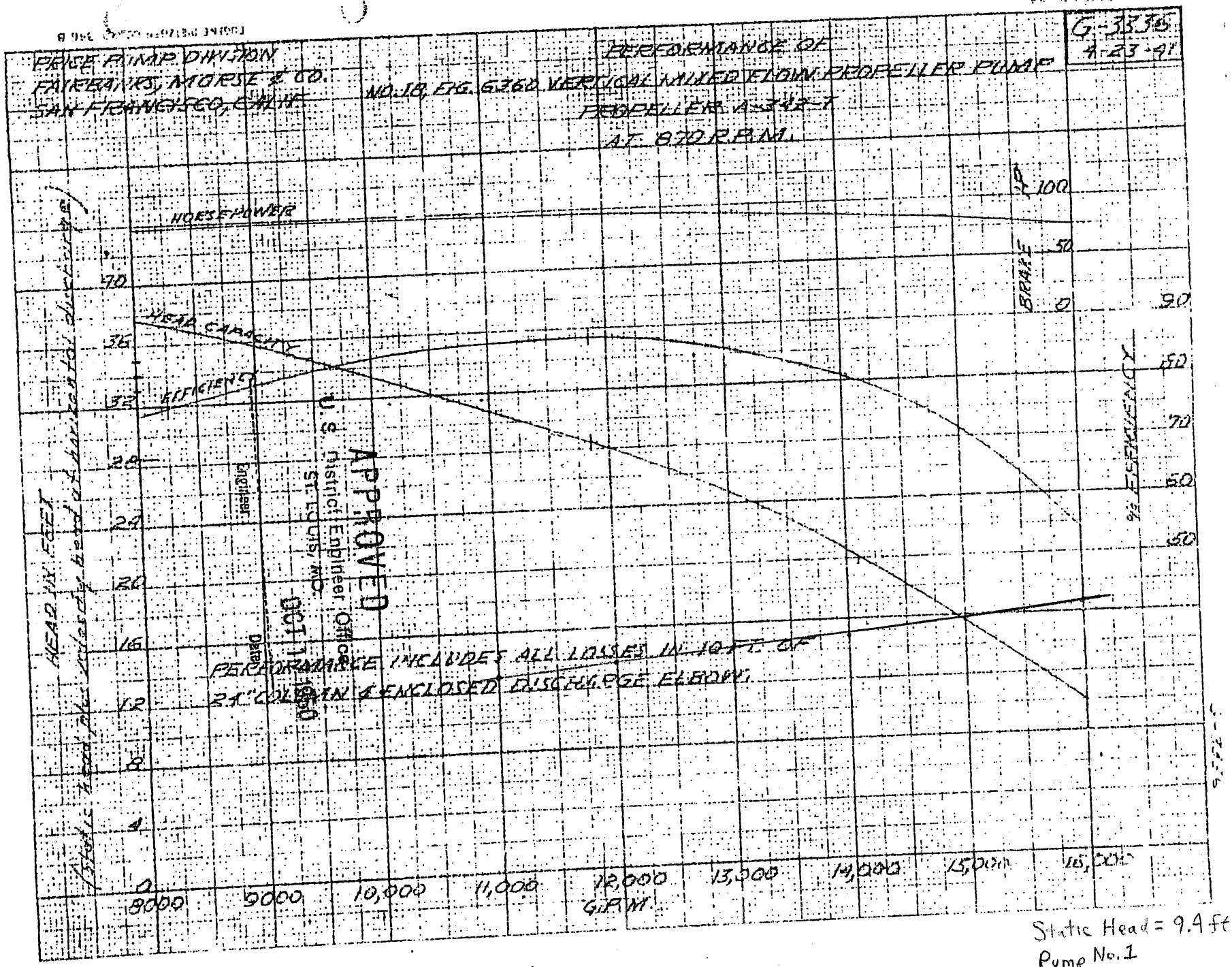
CHART 15-2-3

PR 48

G-3336
4-23-91

PRICE PUMP DIVISION
FAIRBANKS, MORSE & CO.
SAN FRANCISCO, CALIF.

PERFORMANCE OF
NO. 18 FIG. 6360 VERTICAL MIXED FLOW PROPELLER PUMP
PROPELLER A-342-7
AT 870 R.P.M.



PERFORMANCE INCLUDES ALL LOSSES IN INLET OF
24" COLUMN & ENCLOSED DISCHARGE ELBOW.

APPROVED
U.S. District Engineer, Office
St. Louis, Mo.
OCT 11 1990
Date

Static Head = 9.4 ft
Pump No. 1

CHRR -2-4

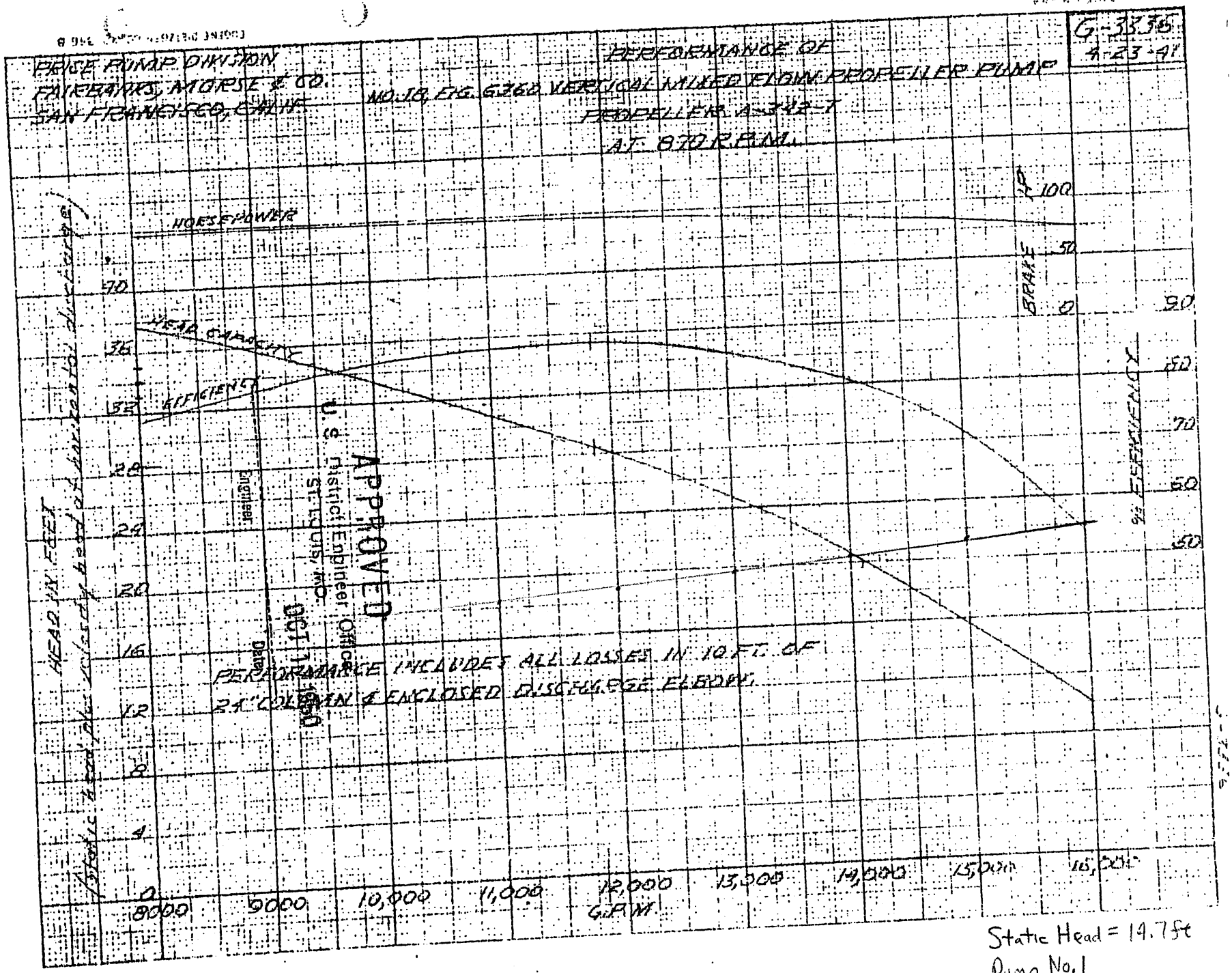
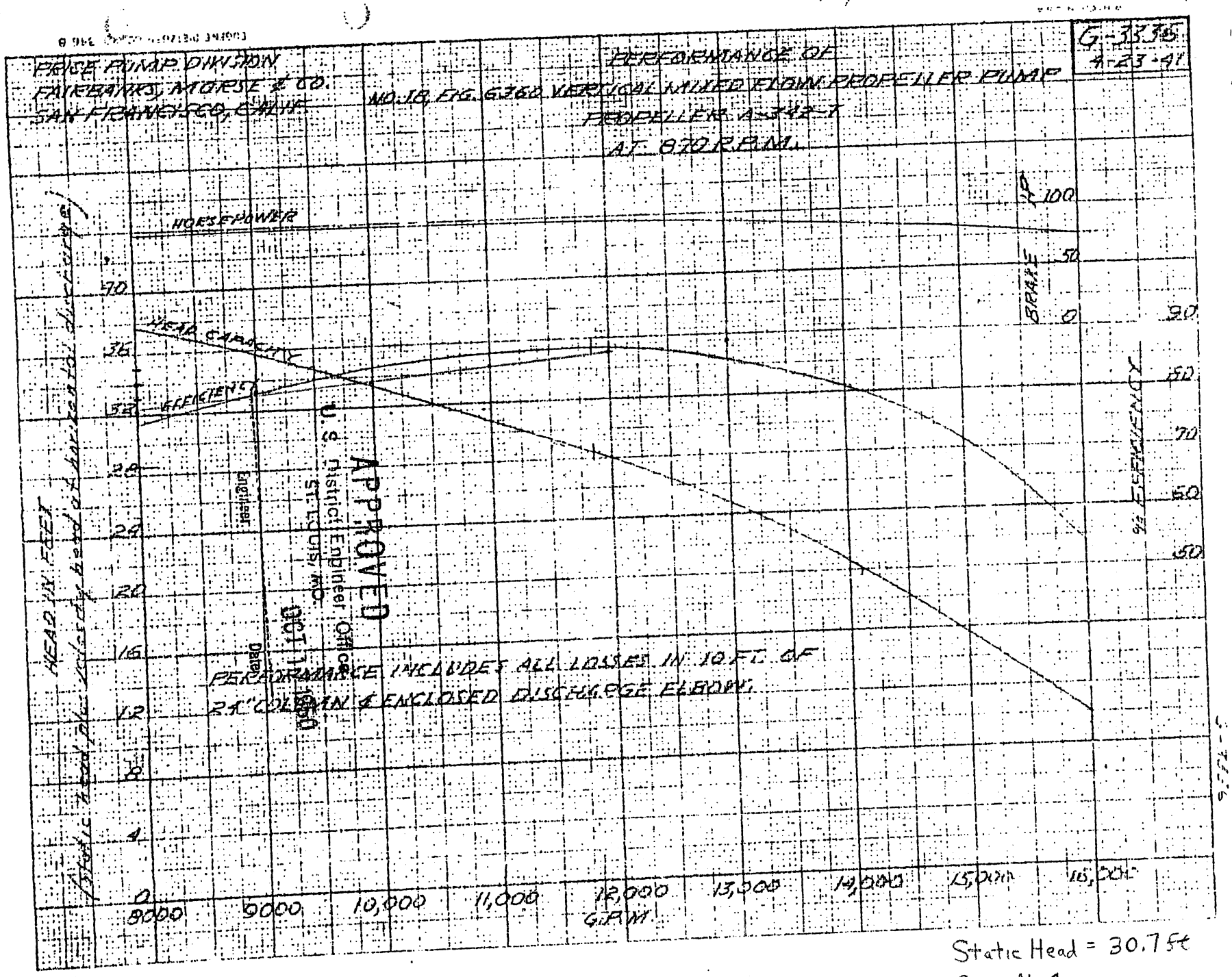

G-3335
4-23-41
PEERLESS PUMP DIVISION
FAIRBANKS, MORSE & CO.
SAN FRANCISCO, CALIF.
PERFORMANCE OF
NO. 18 FIG. G-260 VERTICAL MIXED FLOW PROPELLER PUMP
PROPELLER A-343-1
AT 870 R.P.M.
HORSEPOWER
HEAD DYNAMIC
EFFICIENCY
BRAKE
HP
HEAD IN FEET
EFFICIENCY %
G.P.M.
PERFORMANCE INCLUDES ALL LOSSES IN 10 FT. OF
24" COLUMN & ENCLOSED DISCHARGE ELBOWS.
APPROVED
U.S. District Engineer Office
St. Louis, Mo.
OCT 1 1950
Engineer
Date
Static Head = 14.7 ft
Pump No. 1

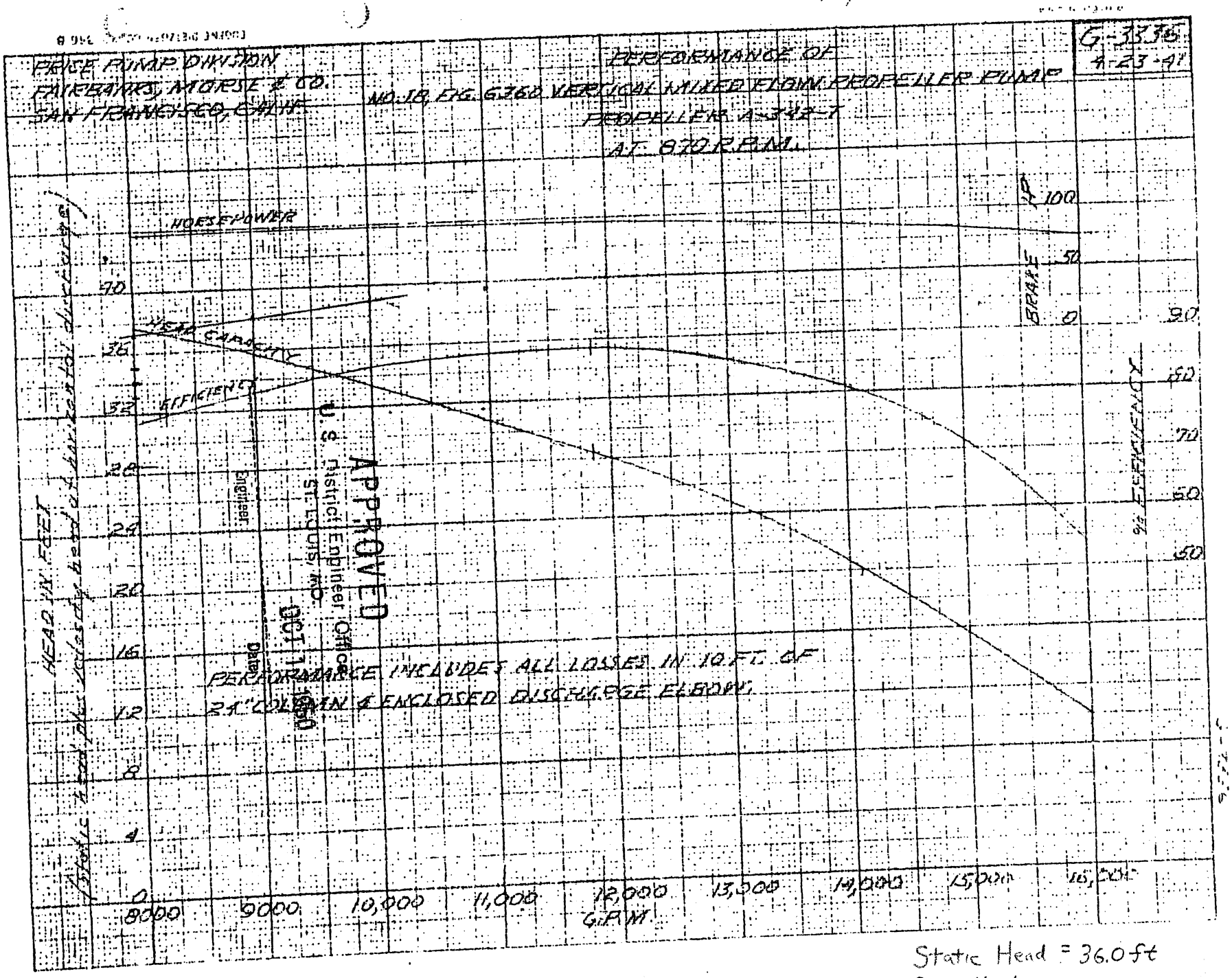
CHART 15-2-5



Static Head = 30.7 ft
Pump No. 1

ART 15-2-6

FR 49



Static Head = 36.0 ft
Pump No. 1

Pump #2

Static Heads - The ℄ of the pump discharge is at El. 757.6 (above El 753.0)

1. River El. 774.3 - Sump El. 738.3 = 36.0 ft
2. River El. 774.3 - Sump El. 743.6 = 30.7 ft
3. Flap Gate El. 757.6 - Sump El. 738.3 = 19.3 ft
4. Flap Gate El. 757.6 - Sump El. 743.6 = 14.0 ft

Develope System Loss Curves for each static head condition

36-in discharge pipe - 13.0 ft long (34.5 in. ID)

1 - 90° Elbow (L/D = 20) 1 - Discharge Flap Gate

Hazen-Williams for friction loss in pipe

$$h_f = 0.002083\, L \left(\frac{100}{C}\right)^{1.85} \times \frac{Q^{1.85}}{d^{4.8655}}$$

where, L = length of pipe, ft. Q = flow, gpm

C = 100 (Cameron Hydraulic Data) d = pipe diameter (in.) = 34.5 in.

Pipe Length (L) -

Straight Pipe - 13.0 ft

90° Elbow (3×20) - 60.0 ft

73.0 ft.

Example - for 20,000 gpm

$$h_f = 0.002083(73)\left(\frac{100}{100}\right)^{1.85} \frac{(20{,}000)^{1.85}}{(34.5)^{4.8655}}$$

$$= 0.45 \text{ ft}$$

System Losses = h_f + vel. head + flap gate loss

flap gate loss - use the max. loss for a 36-in gate = 0.15 ft (CHART 15-2-21)

vel head = $v^2/2g$

CHART 15-2-8

for 20,000 gpm $v = 6.87$ ft/sec

vel. head $= (6.87)^2/2(32.174) = 0.73$ ft.

System Loss (20,000 gpm) $= 0.45 + 0.73 + 0.15 = 1.33$ ft.

System Loss for each of the 4 static head conditions

1. $36.0 + 1.33 = 37.33$ ft
2. $30.7 + 1.33 = 32.03$ ft
3. $19.3 + 1.33 = 20.63$ ft
4. $14.0 + 1.33 = 15.33$ ft

A set of data with friction head losses and system head losses was developed. The corresponding system head loss curves were then plotted on the pump curve being used for Pump No. 2.

CHART 15-2-10 is the spreadsheet of the system head loss curves at the various static heads.

CHARTS 15-2-11 - 15-2-14 are the system loss curves for each static head condition

Pump #3 and Pump #4

Develop System Head Loss Curves for each static head condition.

The ℄ of the pump discharge pipes is at the top of the discharge chamber
Therefore the river (discharge) elevation should be -

Top of Discharge Chamber -	El. 776.80
1/2 of 18-in Discharge Pipe -	0.75
	El. 777.55

Static Head -

1. Discharge El. 777.55 - Sump El. 738.3 = 39.25 ft
2. Discharge El. 777.55 - Sump El. 743.6 = 33.95 ft

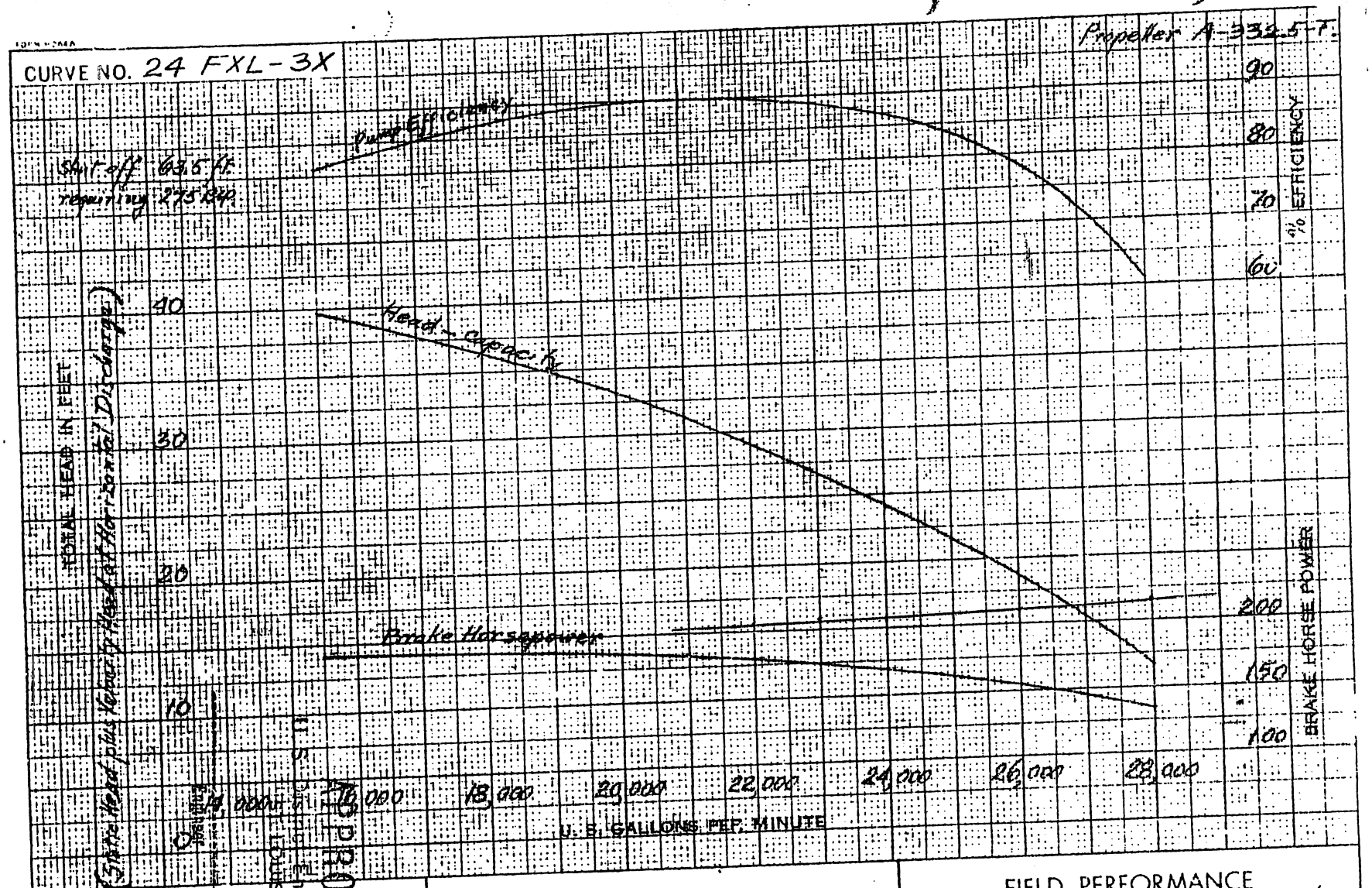
CHART 15-2-9

TURNER PUMP STATION

SYSTEM HEAD LOSSES - OPERATING PUMP NO. 2

FLOW (gpm)	VELOCITY (ft/sec)	VELOCITY HEAD (ft.)	FRICTION LOSS (ft.)	FLAP GATE LOSS (ft.)	TOTAL SYSTEM LOSSES (ft.)	STATIC HEAD = 36.0	STATIC HEAD = 30.7	STATIC HEAD = 19.3	STATIC HEAD = 14.0
14000	4.81	0.36	0.23	0.15	0.74	36.74	31.44	20.04	14.74
15000	5.15	0.41	0.27	0.15	0.83	36.83	31.53	20.13	14.83
16000	5.49	0.47	0.30	0.15	0.92	36.92	31.62	20.22	14.92
17000	5.83	0.53	0.34	0.15	1.01	37.01	31.71	20.31	15.01
18000	6.18	0.59	0.37	0.15	1.12	37.12	31.82	20.42	15.12
19000	6.52	0.66	0.41	0.15	1.22	37.22	31.92	20.52	15.22
20000	6.86	0.73	0.45	0.15	1.34	37.34	32.04	20.64	15.34
21000	7.21	0.81	0.50	0.15	1.45	37.45	32.15	20.75	15.45
22000	7.55	0.89	0.54	0.15	1.58	37.58	32.28	20.88	15.58
23000	7.89	0.97	0.59	0.15	1.71	37.71	32.41	21.01	15.71
24000	8.24	1.05	0.64	0.15	1.84	37.84	32.54	21.14	15.84
25000	8.58	1.14	0.69	0.15	1.98	37.98	32.68	21.28	15.98
26000	8.92	1.24	0.74	0.15	2.12	38.12	32.82	21.42	16.12
27000	9.27	1.33	0.79	0.15	2.27	38.27	32.97	21.57	16.27
28000	9.61	1.44	0.85	0.15	2.43	38.43	33.13	21.73	16.43

CHART 15-2-10



The pump is guaranteed for the use of conditions specified, other points on the curve are approximate. Capacity, head and efficiency guarantees are contingent on the pump being furnished with the specified amount of clear, fresh non-aerated water at a temperature not to exceed 85 degrees Fahrenheit 9-26-47

Over Small Outlet

U.S. Patent Office

FAIRBANKS, MORSE & CO.

POMONA WORKS

POMONA • CALIFORNIA

FIELD PERFORMANCE

One Stage No 24 Fig. 6360 Mixed Flow Propeller

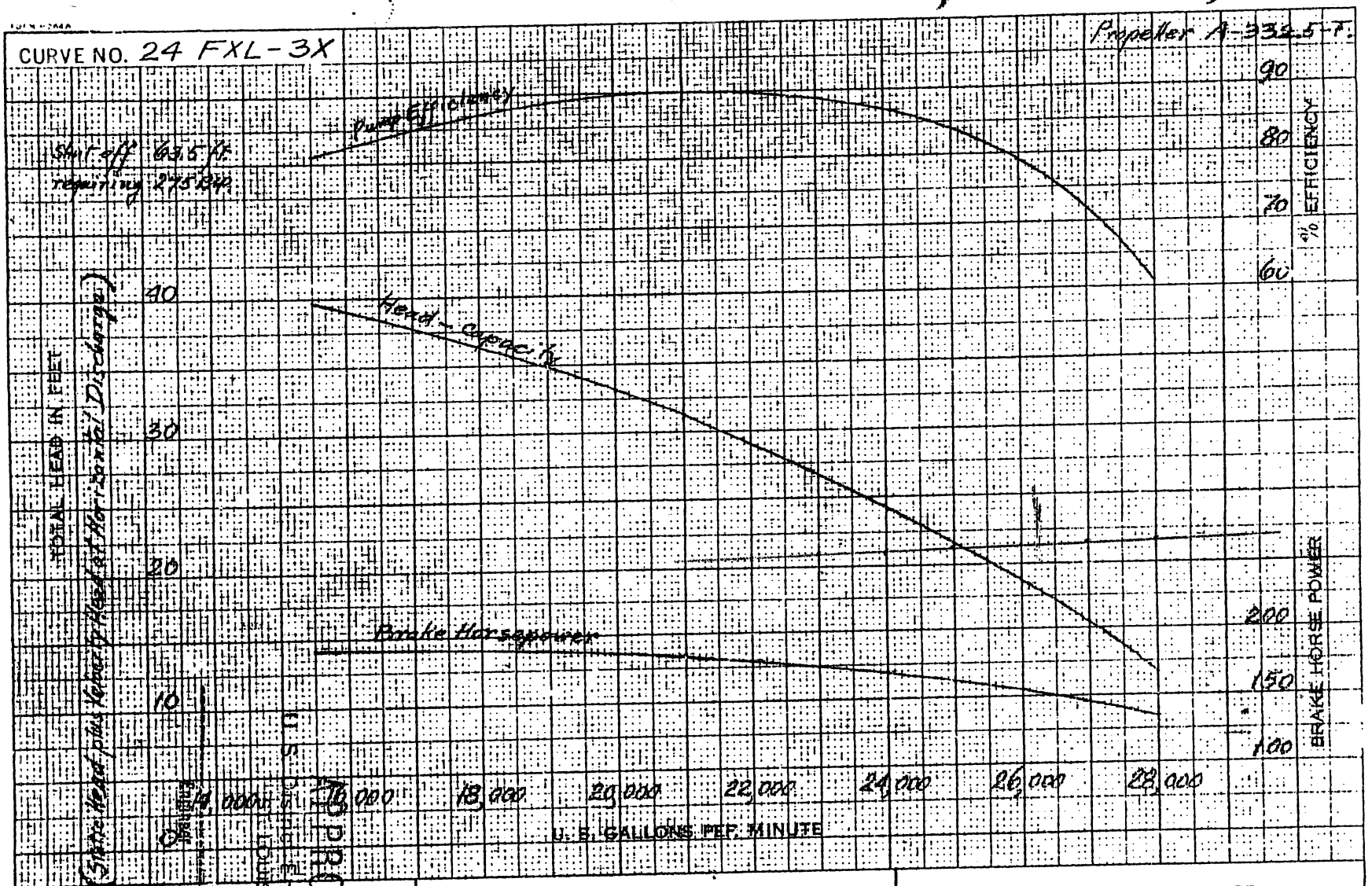
Pump Operated at 700 R.P.M.

All Losses in 20 Ft. of 36" Column & Enclosed Type Disch. Elbow are charged against pump.

Static Head = 14.0 ft

Pump No. 2

CHART 15-2-11



The pump is guaranteed for the set of conditions specified, other points on the curve are approximate. Capacity, head and efficiency guarantees are contingent on the pump being furnished with the specified amount of clear, fresh non aerated water at a temperature of not to exceed 85 degrees Fahrenheit 9-26-47

FAIRBANKS, MORSE & CO.

POMONA WORKS

POMONA • CALIFORNIA

FIELD PERFORMANCE

One Stage No 24 Fig. 6360 Mixed Flow Propeller

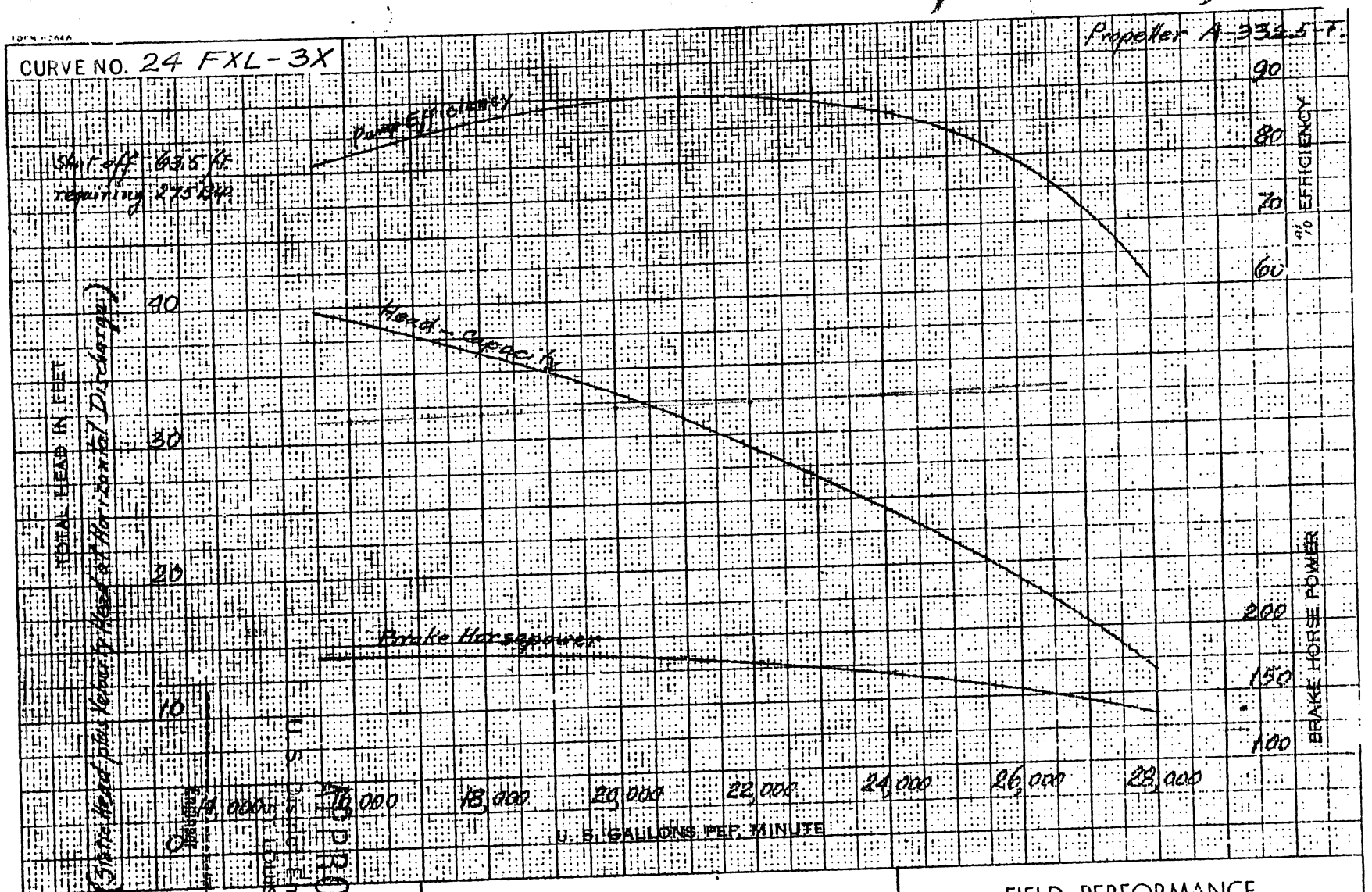
Pump Operated at 700 R.P.M.

All Losses in 20 Ft. of 36" Column & Enclosed Type Disch. Elbow are charged against pump

Static Head = 19.3 ft

Pump No. 2

CHART 15-2-12



The pump is guaranteed for the use of conditions specified, other points on the curve are approximate. Capacity, head and efficiency guarantees are contingent on the pump being furnished with the specified amount of clear, fresh non aerated water at a temperature of not to exceed 85 degrees Fahrenheit

9-26-47 K.

Date Shipped 10-10-50

Over Speed Test Engineer 5-11

FAIRBANKS, MORSE & CO.

POMONA WORKS

POMONA • CALIFORNIA

FIELD PERFORMANCE

One Stage No 24 Fig. 6360 Mixed Flow Propeller

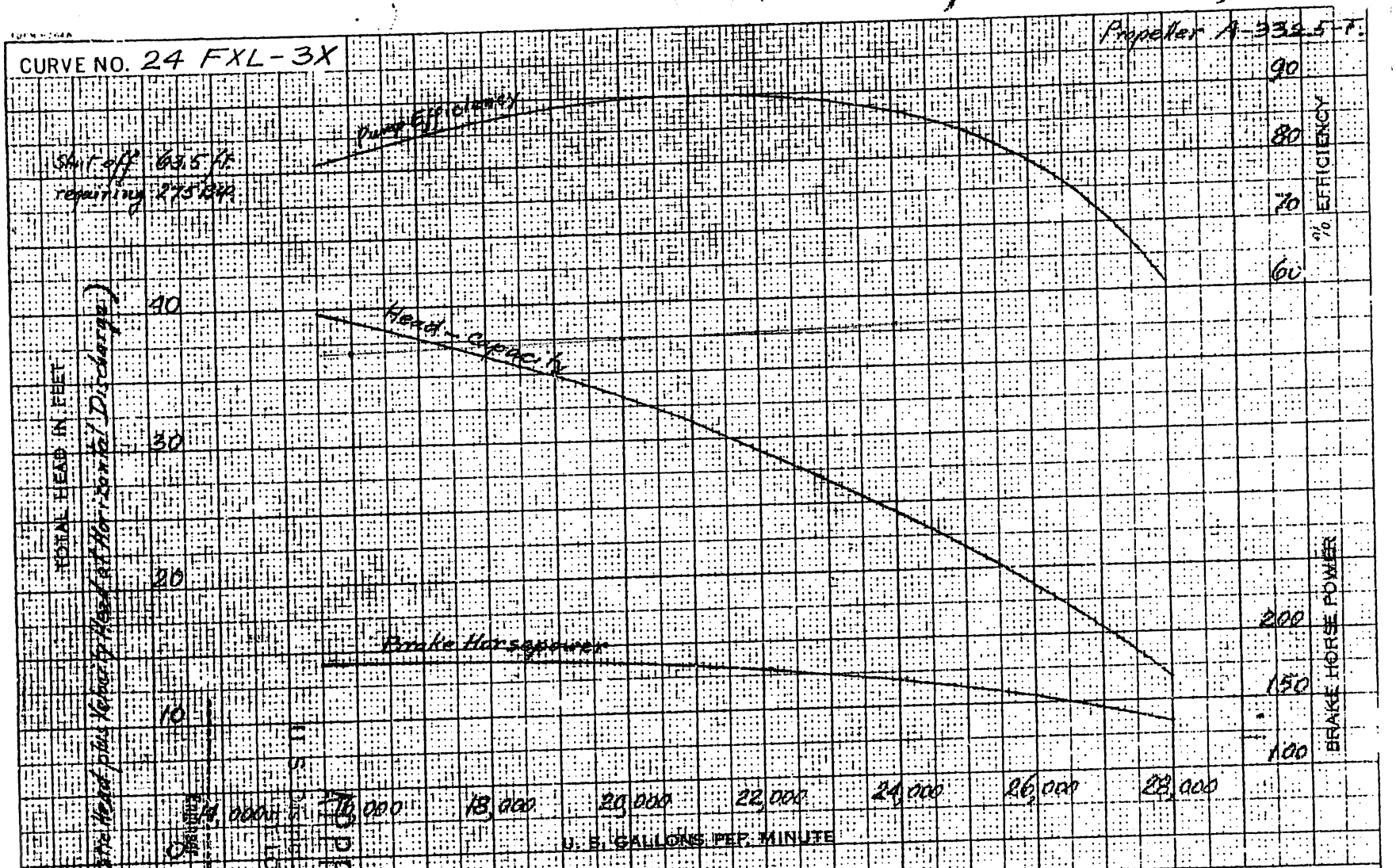
Pump Operated at 700 R.P.M.

All Losses in 20 Ft. of 36" Column & Enclosed Type Disch. Elbow are charged against pump.

Static Head = 30.7 Ft

Pump No. 2

CHART 15-2-13



The pump is guaranteed for the set of conditions specified, other points on the curve approximate. Capacity, head and efficiency guarantees are contingent on the pump being furnished with the specified amount of clear, fresh non aerated water at a temperature of not to exceed 85 degrees Fahrenheit

9-26-47

FAIRBANKS, MORSE & CO.

POMONA WORKS

POMONA • CALIFORNIA

FIELD PERFORMANCE

One Stage No 24 Fig. 6360 Mixed Flow Propeller

Pump Operated at 700 R.P.M.

All Losses in 20 Ft. of 36" Column & Enclosed Type Disch. Elbow are charged against pump

Static Head = 36.0 ft

Pump No. 2

CHART 15-2-14

18-in discharge pipe - 42 ft. (16.876 in. ID)

Note: This length is the longest (worst case) of the 2 discharge pipes

3 - 45° Elbows L/D = 20 K = 0.24

Use Hazen-Williams for friction loss in pipe

$$h_f = 0.002083\, L\left(\frac{100}{C}\right)^{1.85} \times \frac{Q^{1.85}}{d^{4.8655}}$$

where,

L = length of pipe, ft

C = 100

Q = flow, gpm

d = pipe diameter = 18.876 in.

Pipe Length -

Straight pipe	42.0 ft
90° Elbows 3×1.5×20	90.0 ft
	132.0 ft

Example - for 8,000 gpm

$$h_f = 0.002083\,(132)\left(\frac{100}{100}\right)^{1.85} \times \frac{(8{,}000)^{1.85}}{(16.876)^{4.8655}}$$

$$= 4.88 \text{ ft}$$

System Losses = h_f + vel. head

vel. head = $v^2/2g$

for 8,000 gpm v = 11.5 ft/sec

vel. head = $(11.5)^2/2(32.174) = 2.06$ ft.

System Loss for 8,000 gpm = 4.88 + 2.06 = 6.94 ft

CHART 15-2-15

System Loss for each of the 2 static heads

1. 39.25 + 6.94 = 46.19 ft

2. 33.95 + 6.94 = 40.89 ft

A set of data with friction head losses and system head losses was developed. The corresponding system head loss curves were then plotted on the pump curve for Pump Nos. 3 and 4

CHART 15-2-18 is the spreadsheet of the system head losses at the two static heads

CHARTS 15-2-19 - 15-2-20 are the system loss curves for the two static heads

Pump Station Capacities -

At River Elev. 753.0 and Sump Elev 743.6

Pump No. 1 -	14,950 gpm
Pump No. 2 -	26,750 gpm
Pump No. 3 -	12,300 gpm
Pump No. 4 -	12,300 gpm
Total -	66,300 gpm (150.5 cfs)

At River Elev. 753.0 and Sump Elev. 738.3

Pump No. 1 -	14,050 gpm
Pump No. 2 -	25,150 gpm
Pump No 3 -	12,200 gpm
Pump No. 4 -	12,200 gpm
	63,600 gpm (144.7 cfs)

At River Elev. 774.3 and Sump Elev 743.6

Pump No. 1 -	9,900 gpm
Pump No 2 -	20,500 gpm
Pump No 3 -	12,300 gpm
Pump No. 4 -	12,300 gpm
	55,000 gpm (122.5 cfs)

CHART 15-2-16

PROJECT KC Flood Reduction Study	Page 17 of 21	COMPUTED BY DDF	DATE 3/4/04
SUBJECT Determine Pump Conditions		CHECKED BY	DATE

At River Elev. 774.3 and Sump Elev. 738.3

Pump No. 1 -	8150 gpm
Pump No. 2 -	17500 gpm
Pump No 3 -	12200 gpm
Pump No 4 -	12,200 gpm
	50,050 gpm (111.5 cfs)

CHART 15-2-17

TURNER PUMP STATION

SYSTEM HEAD LOSSES - OPERATING
PUMP NOS. 3 and 4

FLOW (gpm)	VELOCITY (ft/sec)	VELOCITY HEAD (ft.)	FRICTION LOSS (ft.)	FLAP GATE LOSS (ft.)	TOTAL SYSTEM LOSSES (ft.)	STATIC HEAD = 39.25	STATIC HEAD = 33.95
3000	4.30	0.29	0.80	0.00	1.08	40.33	35.03
4000	5.74	0.51	1.35	0.00	1.87	41.12	35.82
5000	7.17	0.80	2.05	0.00	2.85	42.10	36.80
6000	8.61	1.15	2.87	0.00	4.02	43.27	37.97
7000	10.04	1.57	3.81	0.00	5.38	44.63	39.33
8000	11.48	2.05	4.88	0.00	6.93	46.18	40.88
9000	12.91	2.59	6.07	0.00	8.66	47.91	42.61
10000	14.34	3.20	7.38	0.00	10.58	49.83	44.53
11000	15.78	3.87	8.80	0.00	12.67	51.92	46.62
12000	17.21	4.60	10.34	0.00	14.94	54.19	48.89
13000	18.65	5.40	11.99	0.00	17.39	56.64	51.34

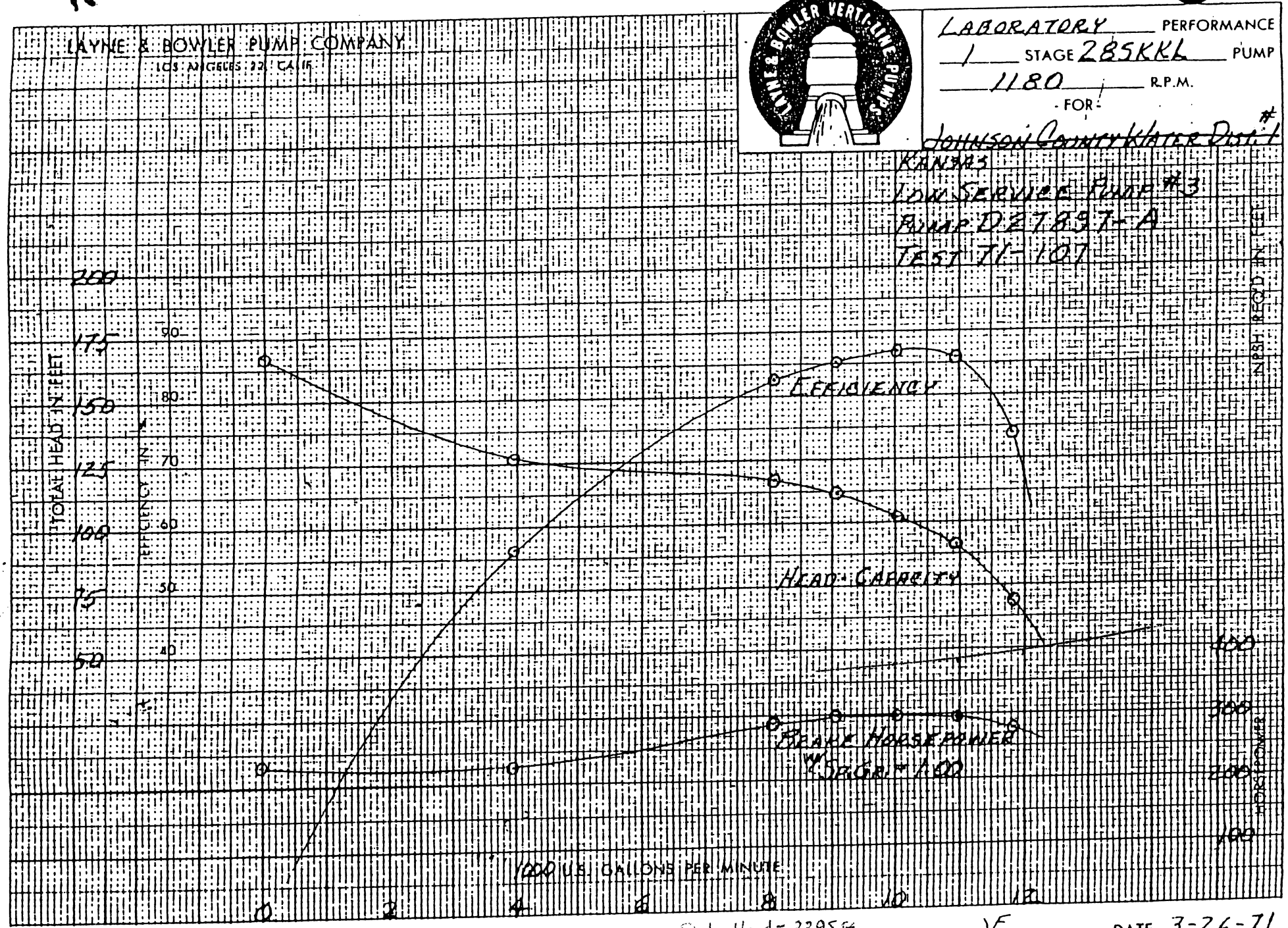
CHART 15-2-18

Station 60 + 40

CURVE NO. 107

LAYNE & BOWLER PUMP COMPANY
LOS ANGELES 22, CALIF.

LABORATORY PERFORMANCE
1 STAGE 285KKL PUMP
1180 R.P.M.
FOR:
Johnson County Water Dist #1
KANSAS
LOW SERVICE PUMP #3
PUMP D2789T-A
TEST TL-107



Static Head = 33.95 ft
Pump Nos. 3+4

PLOTTED BY: V DATE 3-26-71

CERTIFIED BY [illegible] DATE 3-27-71

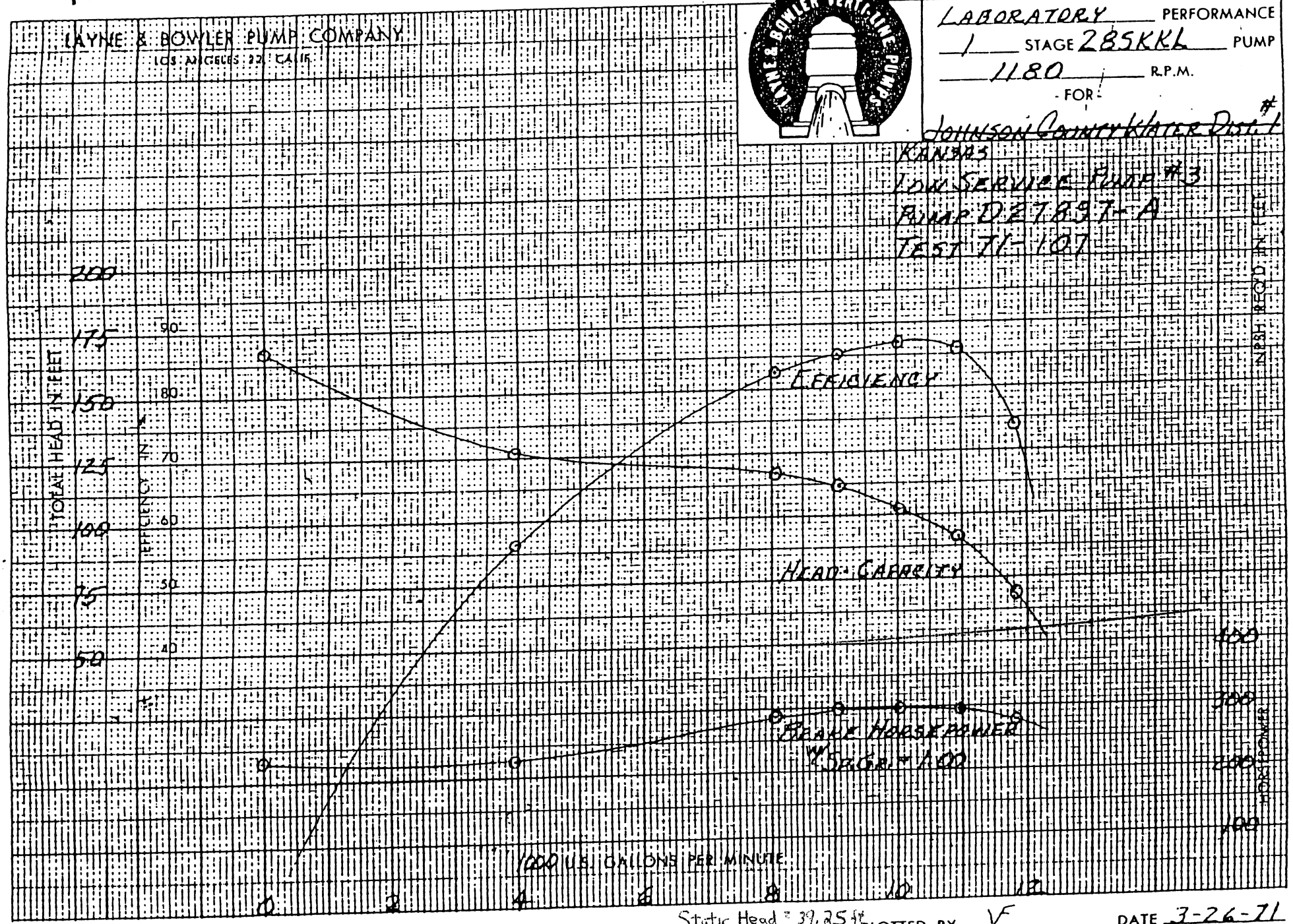
CHART 15-2-19

Turner Pump Station
Station 60+40

CURVE NO. 107

LAYNE & BOWLER PUMP COMPANY
LOS ANGELES 22, CALIF.

LABORATORY PERFORMANCE
1 STAGE 285KKL PUMP
1180 R.P.M.
FOR
~~Johnson County Water Dist #1~~
KANSAS
Low Service Pump #3
Pump D27897-A
Test 71-107



Static Head = 39.25 ft
Pump Nos. 3+4

CHART 15-2-20

PLOTTED BY: VF DATE 3-26-71
CERTIFIED BY [illegible] DATE 3-26-71

Loss of Head Through Flap Gates

Tests conducted on flap gates show that the loss of head due to the flap riding on the water is very small compared with other losses in the hydraulic structure. Of these, the entrance loss is usually considerably more critical than loss at the flap gate on the outlet end of the conduit.

The Hydraulic Laboratory of the State University of Iowa conducted a series of tests some years ago to determine the amount of head lost by water discharging through Hydro Flap Gates Model 10C (formerly Armco-Calco). The gates—18, 24 and 30 inches in diameter—were supplied from commerical stock.

The following passage is excerpted from the report of Professor Floyd A. Nagler, Associate Professor of Mechanics and Hydraulics, who supervised the investigation.

"Based on these experiments the following empirical formula was derived to express the loss in head through Calco Gates of varying sizes and with different velocities of flow:

L—Loss of head in feet
v—Velocity of flow through gate in feet per second
d—Diameter of outlet in feet
e—Base of natural logarithms

$$L = \frac{4v^2}{g} \times e^{\left(\frac{-1.15v}{\sqrt{d}}\right)}$$

"It may be concluded from these experiments that the Calco Gate in its hydraulic characteristics is all that the manufacturers have claimed for it. The small loss in head obtained through these gates demonstrates that their installation has but little effect on the discharged capacity of drainage outlets."

Medium and heavy duty flap gates have heavier flaps or covers than the gate model tested. As a result, head losses through these gates may be slightly more than those indicated by the test.

LOSS OF HEAD THROUGH FLAP GATES

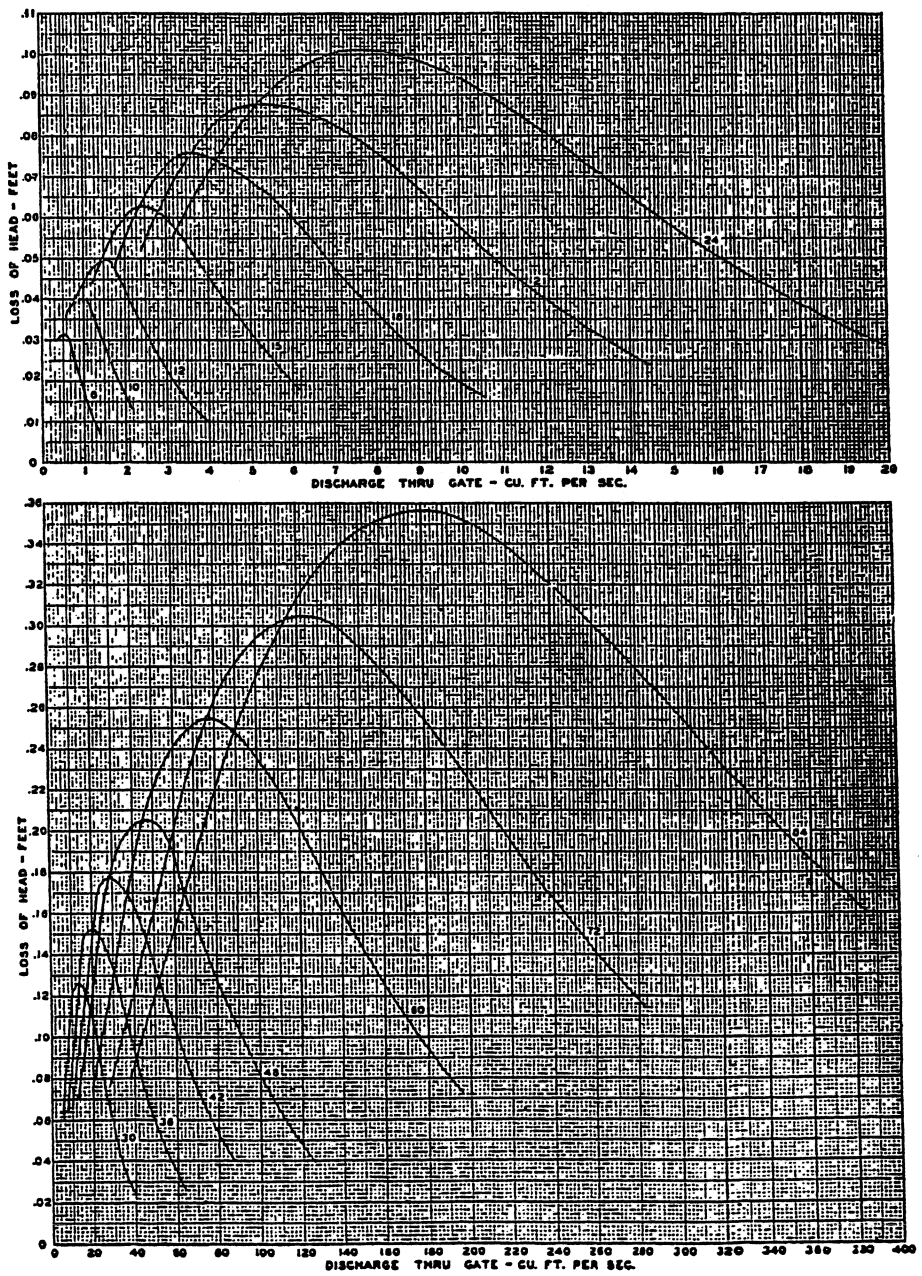


CHART 15-2-21

Future Design Water Elev. with the 500 yr + 3' level of protection - El. 780.8

Pump No. 1

The new static head conditions are -

1. River El. 780.8 - Sump Elev. 738.3 = 42.3 ft

2. River El. 780.8 - Sump Elev. 743.6 = 37.2 ft

CHART 15-3-3 is the spreadsheet of the system head losses at the two new static heads

CHARTS 15-3-6 and 15-3-7 are the system loss curves at each of these static heads

Pump No 2

The new static head conditions are -

1. 42.3 ft

2. 37.2 ft

CHART 15-3-4 is the spreadsheet of the system head losses at these two new static heads

CHARTS 15-3-8 and 15-3-9 are the system loss curves at each of these static heads

Pump Nos. 3 and 4

With the discharge chamber being raised these 18-inch discharge pipes will have a flap gate installed on the end. The maximum head loss for an 18-inch flap gate is .08 ft

The new static head conditions are -

1. 42.3 ft 2. 37.2 ft

CHART 15-3-1

PROJECT KC Flood Reduction Study	Page 2 of 11	COMPUTED BY DJF	DATE 3/16/04
SUBJECT Future Conditions		CHECKED BY	DATE

Pump Station Capacities -

At River El. 780.8 and Sump El. 743.6

Pump No1 - 7,700 gpm (Off the chart slightly)
Pump No2 - 16,800 gpm
Pump No3 - 12,250 gpm
Pump No4 - 12,250 gpm
49,000 gpm (109.2 cfs)

At River El. 780.8 and Sump El. 738.3

Pump No. 1 - Too far off the chart
Pump No 2 - 13,000 gpm (Off the chart slightly)
Pump No 3 - 12,150 gpm
Pump No 4 - 12,150 gpm
37,300 gpm (83.1 cfs)

CHART 15-3-2

TURNER PUMP STATION

SYSTEM HEAD LOSSES - OPERATING

PUMP NO. 1

FLOW (gpm)	VELOCITY (ft/sec)	VELOCITY HEAD (ft.)	FRICTION LOSS (ft.)	FLAP GATE LOSS (ft.)	TOTAL SYSTEM LOSSES (ft.)	STATIC HEAD = 42.3	STATIC HEAD = 37.2
8000	6.39	0.63	0.87	0.10	1.60	43.90	38.80
9000	7.18	0.80	1.60	0.10	2.50	44.80	39.70
10000	7.98	0.99	1.94	0.10	3.03	45.33	40.23
11000	8.78	1.20	2.32	0.10	3.62	45.92	40.82
12000	9.58	1.43	2.72	0.10	4.25	46.55	41.45
13000	10.38	1.67	3.16	0.10	4.93	47.23	42.13
14000	11.17	1.94	3.62	0.10	5.66	47.96	42.86
15000	11.97	2.23	4.11	0.10	6.44	48.74	43.64
16000	12.77	2.53	4.64	0.10	7.27	49.57	44.47

CHART 15-3-3

TURNER PUMP STATION

SYSTEM HEAD LOSSES - OPERATING

PUMP NO. 2

FLOW (gpm)	VELOCITY (ft/sec)	VELOCITY HEAD (ft.)	FRICTION LOSS (ft.)	FLAP GATE LOSS (ft.)	TOTAL SYSTEM LOSSES (ft.)	STATIC HEAD = 42.3	STATIC HEAD = 37.2
14000	4.81	0.36	0.23	0.15	0.74	43.04	37.94
15000	5.15	0.41	0.27	0.15	0.83	43.13	38.03
16000	5.49	0.47	0.30	0.15	0.92	43.22	38.12
17000	5.83	0.53	0.34	0.15	1.01	43.31	38.21
18000	6.18	0.59	0.37	0.15	1.12	43.42	38.32
19000	6.52	0.66	0.41	0.15	1.22	43.52	38.42
20000	6.86	0.73	0.45	0.15	1.34	43.64	38.54
21000	7.21	0.81	0.50	0.15	1.45	43.75	38.65
22000	7.55	0.89	0.54	0.15	1.58	43.88	38.78
23000	7.89	0.97	0.59	0.15	1.71	44.01	38.91
24000	8.24	1.05	0.64	0.15	1.84	44.14	39.04
25000	8.58	1.14	0.69	0.15	1.98	44.28	39.18
26000	8.92	1.24	0.74	0.15	2.12	44.42	39.32
27000	9.27	1.33	0.79	0.15	2.27	44.57	39.47
28000	9.61	1.44	0.85	0.15	2.43	44.73	39.63

CHART 15-3-4

TURNER PUMP STATION

SYSTEM HEAD LOSSES - OPERATING
PUMP NOS. 3 and 4

FLOW (gpm)	VELOCITY (ft/sec)	VELOCITY HEAD (ft.)	FRICTION LOSS (ft.)	FLAP GATE LOSS (ft.)	TOTAL SYSTEM LOSSES (ft.)	STATIC HEAD = 42.5	STATIC HEAD = 37.2
3000	4.30	0.29	0.80	0.08	1.16	43.66	38.36
4000	5.74	0.51	1.35	0.08	1.95	44.45	39.15
5000	7.17	0.80	2.05	0.08	2.93	45.43	40.13
6000	8.61	1.15	2.87	0.08	4.10	46.60	41.30
7000	10.04	1.57	3.81	0.08	5.46	47.96	42.66
8000	11.48	2.05	4.88	0.08	7.01	49.51	44.21
9000	12.91	2.59	6.07	0.08	8.74	51.24	45.94
10000	14.34	3.20	7.38	0.08	10.66	53.16	47.86
11000	15.78	3.87	8.80	0.08	12.75	55.25	49.95
12000	17.21	4.60	10.34	0.08	15.02	57.52	52.22
13000	18.65	5.40	11.99	0.08	17.47	59.97	54.67

CHART 15-3-5

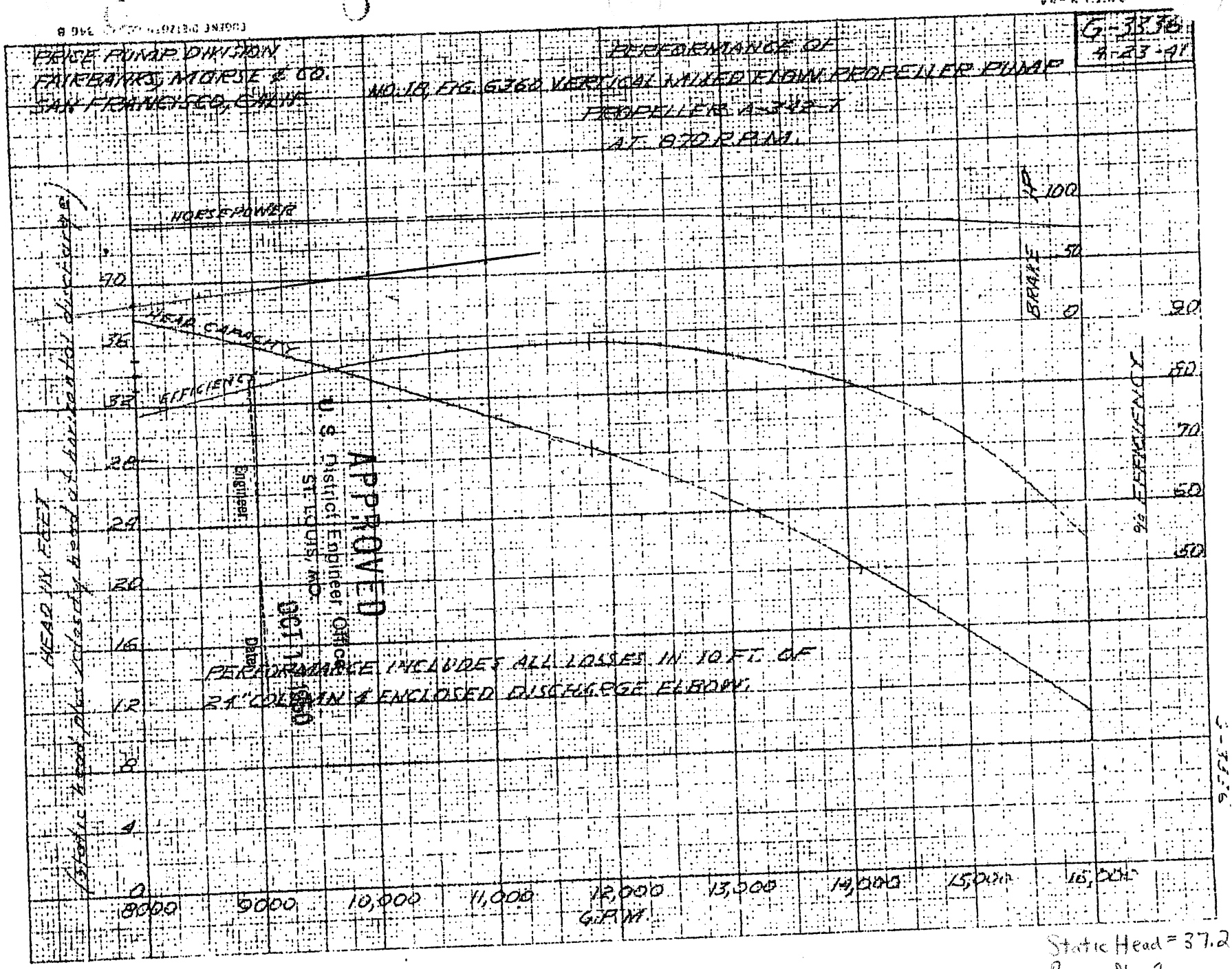
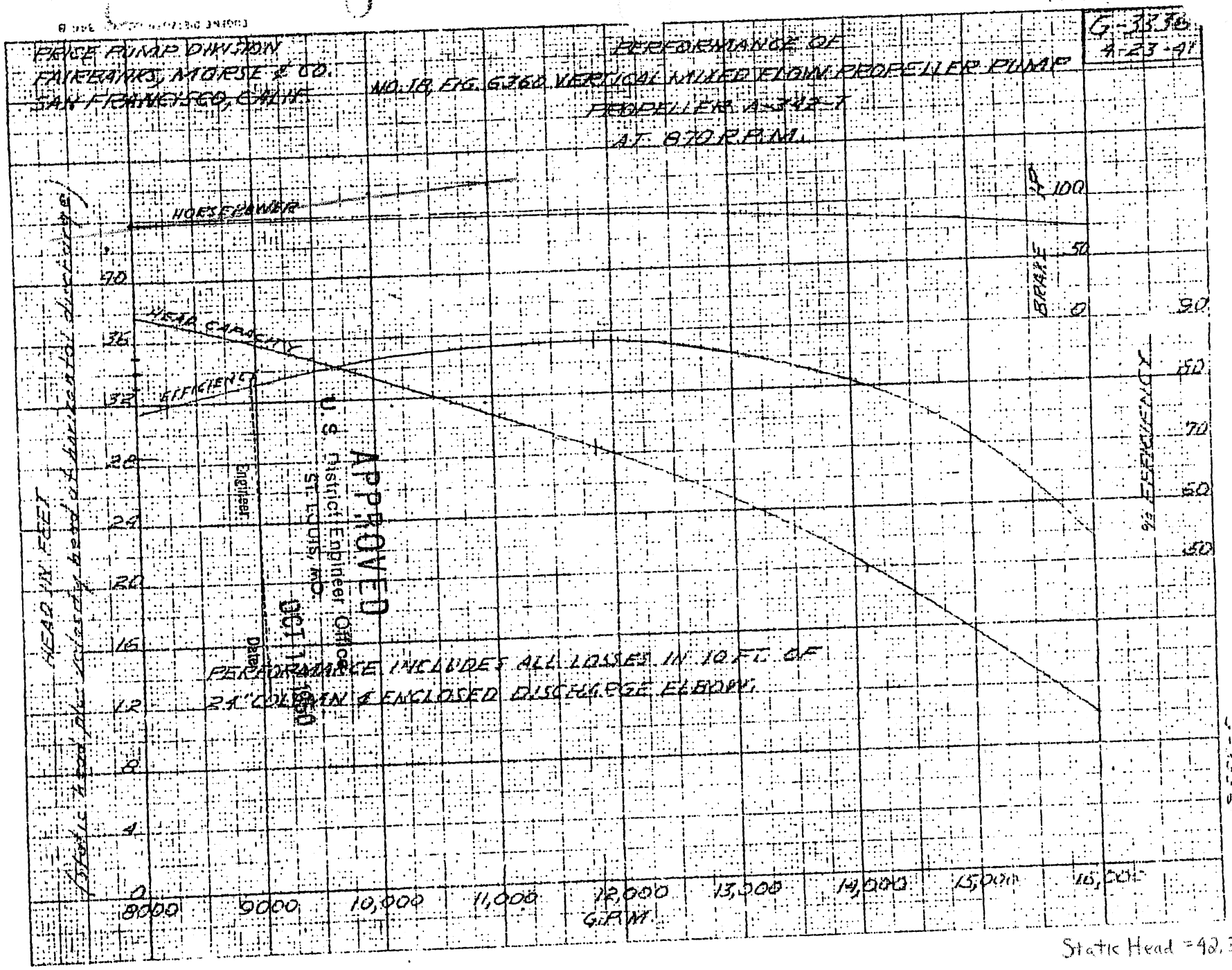
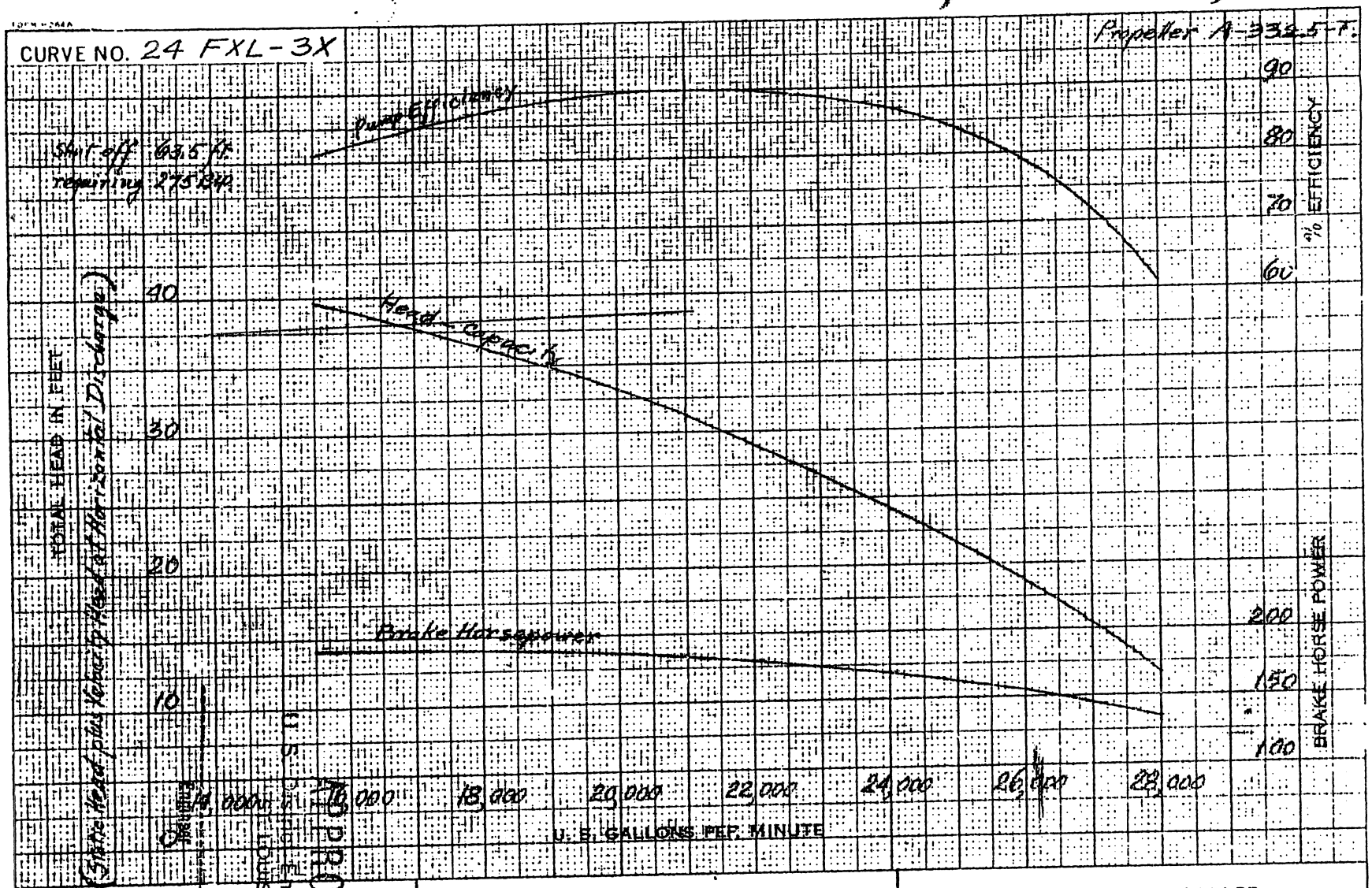

PEERLESS PUMP DIVISION
FAIRBANKS, MORSE & CO.
SAN FRANCISCO, CALIF.
PERFORMANCE OF
NO. 18 FIG. 6360 VERTICAL MIXED FLOW PROPELLER PUMP
PROPELLER A-342-T
AT 870 R.P.M.
G-3536
4-23-41
HORSEPOWER
HEAD CAPACITY
EFFICIENCY
HEAD IN FEET
(Static Head plus Losses by Pump on Discharge)
BRAKE HP
% EFFICIENCY
GPM
PERFORMANCE INCLUDES ALL LOSSES IN 10 FT OF
24" COLUMN & ENCLOSED DISCHARGE ELBOW.
APPROVED
U.S. District Engineer Office
ST. LOUIS, MO.
OCT 1 1940
Engineer
Dated
Static Head = 37.2 ft.
Pump No. 1

CHART 15-3-6



Static Head = 42.35 ft
Pump No. 1

CHART 15-3-7



The pump is guaranteed for the set of conditions specified, other points on the curve are approximate. Capacity, head and efficiency guarantees are contingent on the pump being furnished with the specified amount of clear, fresh non aerated water at a temperature of not to exceed 85 degrees Fahrenheit

9-26-47

FAIRBANKS, MORSE & CO.

POMONA WORKS

POMONA • CALIFORNIA

FIELD PERFORMANCE

One Stage No 24 Fig. 6360 Mixed Flow Propeller

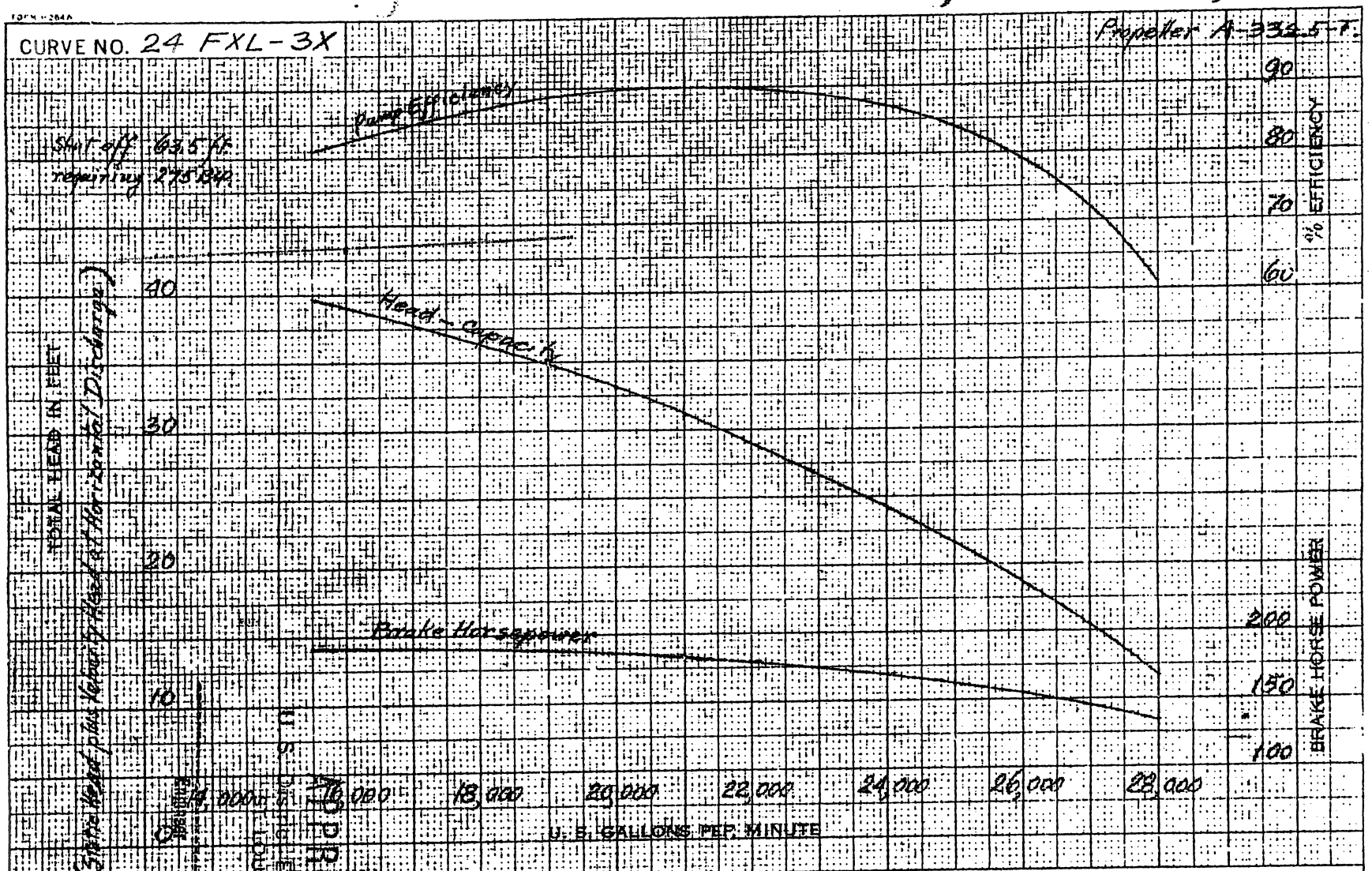
Pump Operated at 700 R.P.M.

All Losses in 20 Ft. of 36" Column & Enclosed Type Disch. Elbow are charged against pump

Static Head = 37.2 ft

Pump No. 2

CHART 15-3-8



The pump is guaranteed for the set of conditions specified, other points on the curve are approximate. Capacity, head and efficiency guarantees are contingent on the pump being furnished with the specified amount of clear, fresh non aerated water at a temperature of not to exceed 85 degrees Fahrenheit.

9-26-47

FAIRBANKS, MORSE & CO.

POMONA WORKS

POMONA • CALIFORNIA

FIELD PERFORMANCE

One Stage No 24 Fig. 6360 Mixed Flow Propeller

Pump Operated at 700 R.P.M.

All Losses in 20 Ft. of 36" Column & Enclosed Type Disch. Elbow are charged against pump.

Static Head = 42.3 ft

Pump No. 2

CHART 15-3-9

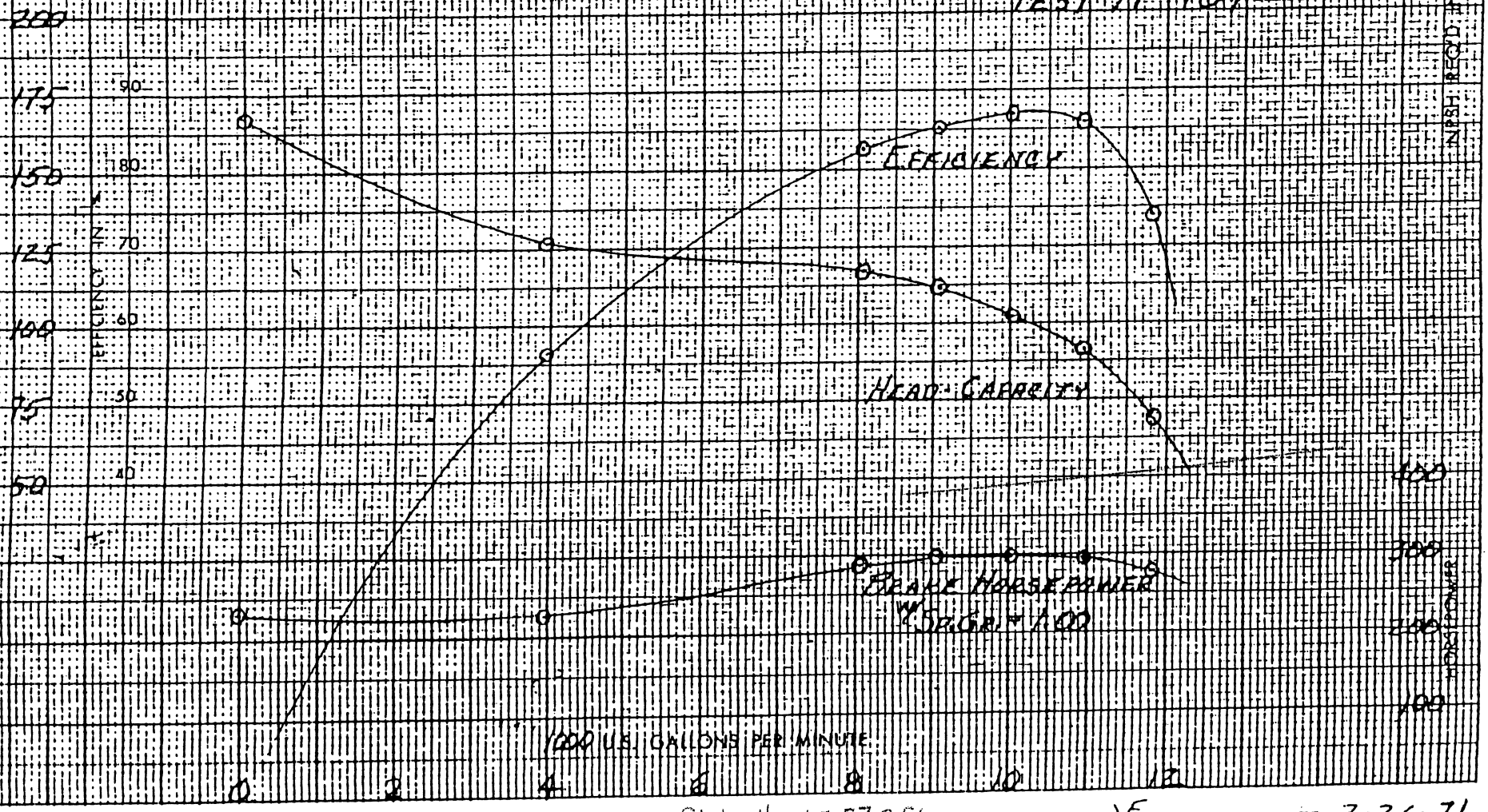
Station 60 + 40

CURVE NO. 107

LAYNE & BOWLER PUMP COMPANY
LOS ANGELES 22, CALIF.

LAYNE & BOWLER VERTI-LINE PUMPS

LABORATORY PERFORMANCE
1 STAGE 285KKL PUMP
1180 R.P.M.
FOR
~~Johnson County Water Dist #1~~
KANSAS
Low Service Pump #3
Pump D27897-A
Test 71-107



Static Head = 37.2 ft
Pump Nos. 3 and 4

CHART 15-3-10

PLOTTED BY: VF DATE 3-26-71
CERTIFIED BY W. F. P. DATE 3-26-71

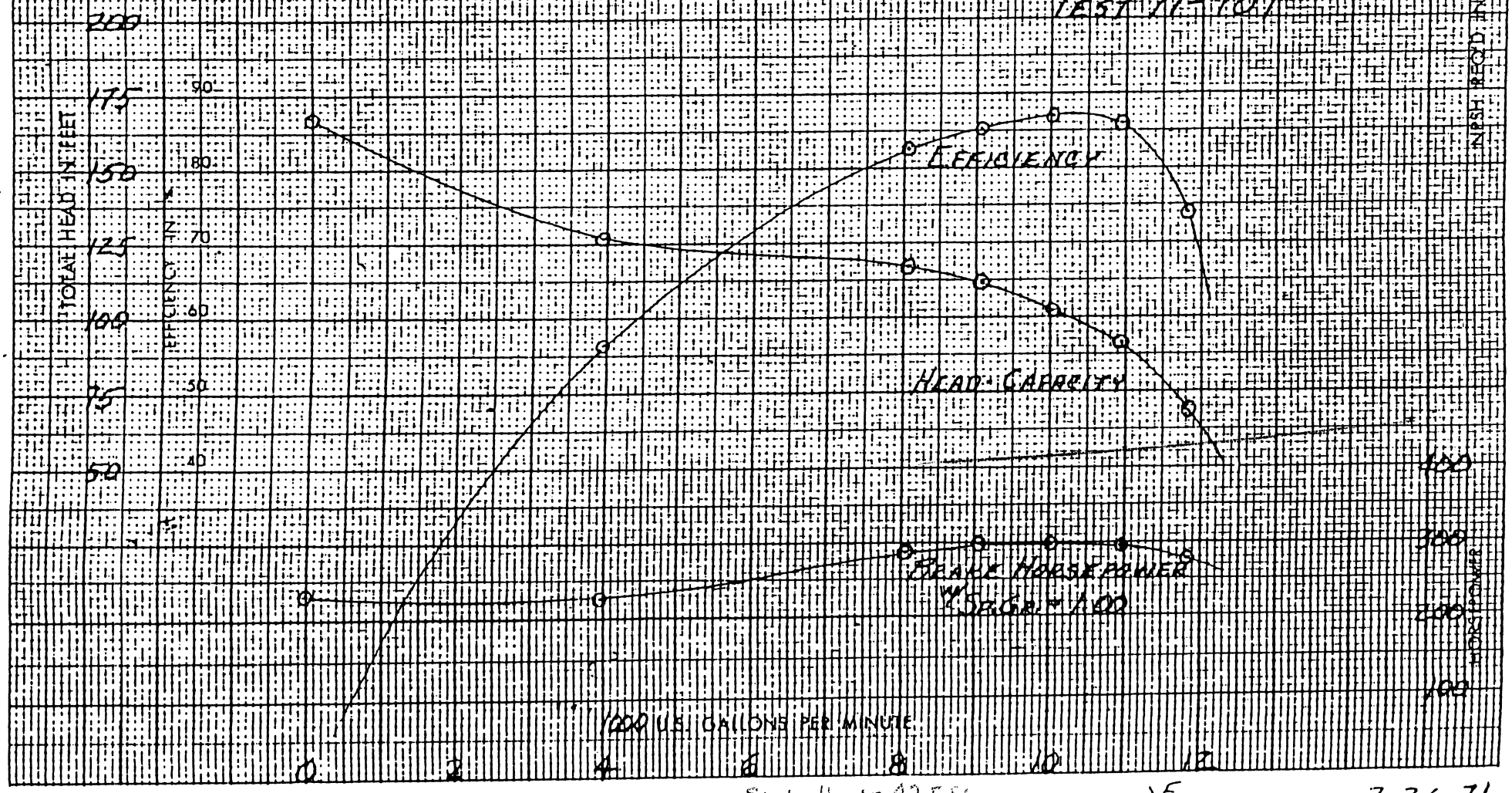
Turner Pump Station Proj. Chart
Station 60 + 40

CURVE NO. 107

LAYNE & BOWLER PUMP COMPANY
LOS ANGELES 22, CALIF.

LAYNE & BOWLER VERTI-LINE PUMPS

LABORATORY PERFORMANCE
1 STAGE 285KKL PUMP
1180 R.P.M.
FOR
~~Johnson County Water Dist. #1~~
Kansas
Low Service Pump #3
Pump D27897-A
Test 71-107



Static Head = 42.5 ft
Pump Nos. 3 and 4

CHART 15-3-11

PLOTTED BY: VF DATE 3-26-71
CERTIFIED BY: W.F.P. DATE 3-26-71

EXHIBIT A-15.4

Argentine Unit – Argentine Pump Station Analysis

KANSAS CITYS, MISSOURI AND KANSAS
SEVEN LEVEES FEASIBILITY STUDY
PUMP STATION EVALUATIONS
Argentine Pump Station Existing Conditions Uplift Check - NOTES

Levee Unit:	Argentine
Pump Station:	Argentine
Levee Station:	Sta. 253+14
Type of Raise:	None – Existing Conditions

1. Data. The data gathered and used to evaluate the pump station includes the following:

1.1 Hydraulic Grade Line. Hydraulic gradient was provided and coordinated with the geotechnical engineer in Kansas City District. For the existing conditions, the HGL is provided to be approximately 12 feet above existing grade on the landside of the levee, at the location of the pump station. Backup calculations and information are included with this exhibit.

1.2 O&MM Drawings.
Reference O&MM Plate Numbers: 44, 49, 52, 53, 124 - 125
Construction drawings of the original pump station were not available. Modifications to the pump station where the flood wall and new discharge chamber were added are shown in the Operation and Maintenance Manual drawings for the Argentine Unit. These O&MM drawings however, do not completely detail the pump station itself, particularly the substructure. Therefore, in order to capture a range of possible geometry and substructure conditions, the following assumptions were made and uplift analysis performed on each. To extent possible, member sizes, configuration, etc. were scaled off of and interpreted from the existing O&MM drawings.

<u>ASSUMPTION 1</u>

The substructure was assumed to consist of concrete walls along the footprint of the station. The walls were assumed to support the superstructure and continue below grade to a depth equal to the location of the box culvert sewer. The substructure was assumed to have a continuous base slab over the full footprint of the station, on which uplift pressures could act. The O&MM drawings indicate that there could be soil located within the substructure above the box culvert sewer and around the original manhole, and sluice gate chase. For assumption 1 this soil was assumed to be in place and contained within the assumed substructure walls. It was also assumed that the soil was saturated, as this part of the substructure was not watertight. The enclosed hand comps document basic member size assumptions and determinations.

Assumption 1, Uplift Condition A – assumes 8 ft f water in the box culvert sewer
Assumption 1, Uplift Condition B – assumes zero ft water in box culvert sewer

<u>ASSUMPTION 2</u>

Assumption 2 is the same as Assumption 1, except the soil within the substructure is not included.

<u>ASSUMPTION 3</u>

The O&MM drawings suggest a condition in which the superstructure of the station is supported on a subgrade system of beams and "columns", with some type of masonry fill between the columns (at the perimeter of the station footprint). Most of the masonry was likely removed or disturbed during modifications of the plant, and is assumed not to be intact. The O&MM drawings identify the sewer, pump pits, engine pits with straight lines, and therefore these features are assumed to be concrete, as opposed to masonry. In contrast to Assumption 1, Assumption 3 does not assume a full base slab over the footprint of the station, but only at locations of the sewer and the pump and engine pits. The perimeter substructure walls of masonry are neglected in the uplift calculation. The weight of the superstructure is included based upon a simple percentage of the overall footprint area to the area assumed for uplift. The soil above the sewer is assumed to be in place, as in Assumption 1.

Assumption 3, Uplift Condition A – assumes 8 ft of water in the box culvert sewer
Assumption 3, Uplift Condition B – assumes zero ft water in box culvert sewer

ASSUMPTION 4

Assumption 4 is the same as Assumption 3, except the soil within the substructure is not included.

ASSUMPTION 5

Assumption 5 is a simple assumption in which the tandem pump pits and the engine pit act independently to resist the uplift applied to their respective base areas. The pits are assumed to go down to same level as the box sewer.

Results and calculation are provided in the enclosed tables and plots.

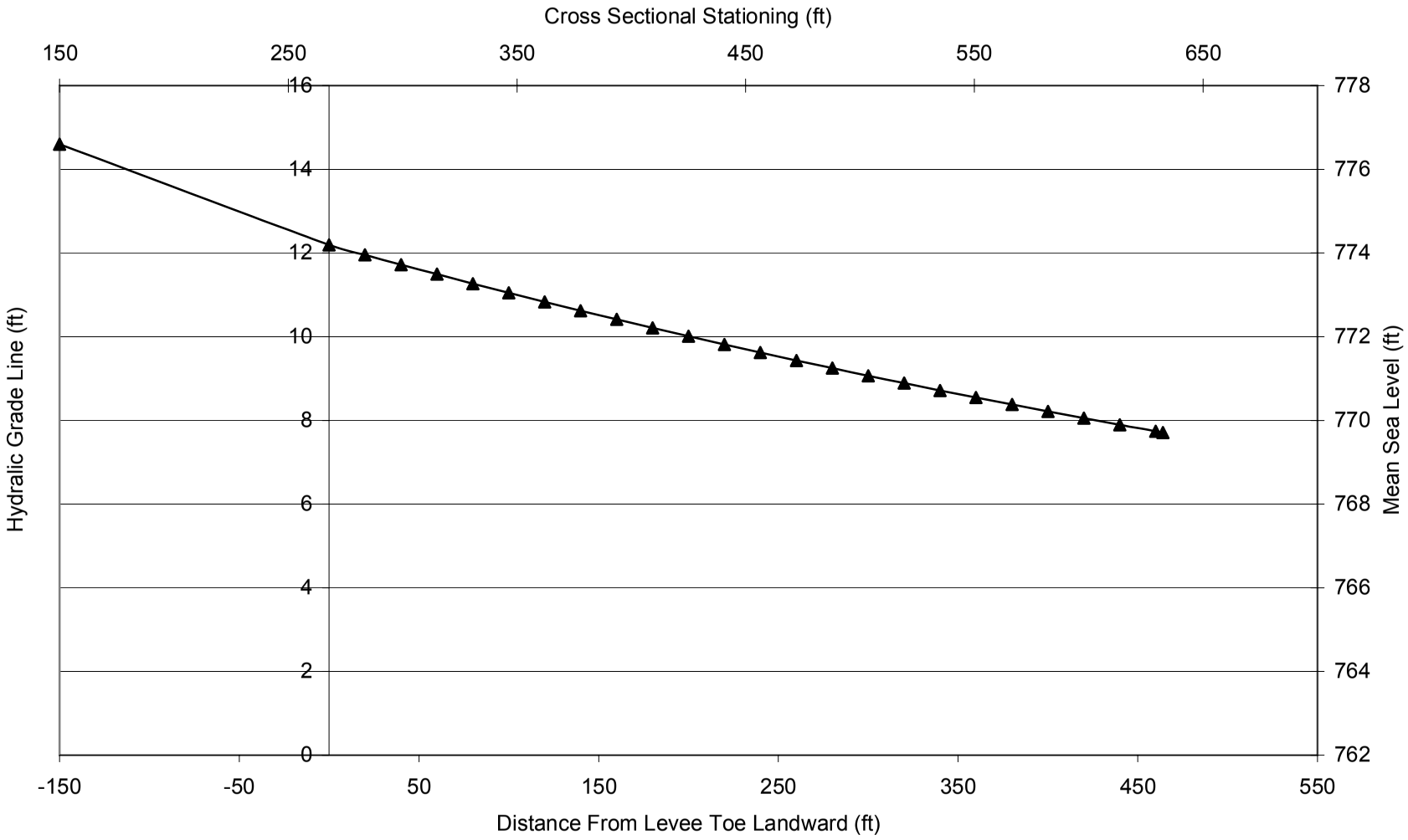
Based upon the lack of information of the existing station, and the age and condition of the station, it is difficult to determine to any confidence level what the actual factor of safety for uplift truly is for Argentine. Unfortunately the results of the varying assumptions noted result in widely varying factors of safety for uplift for the existing conditions. It could be construed that the actual conditions most closely resemble some cross between the conditions of Assumptions 3 & 4 and Assumption 5.

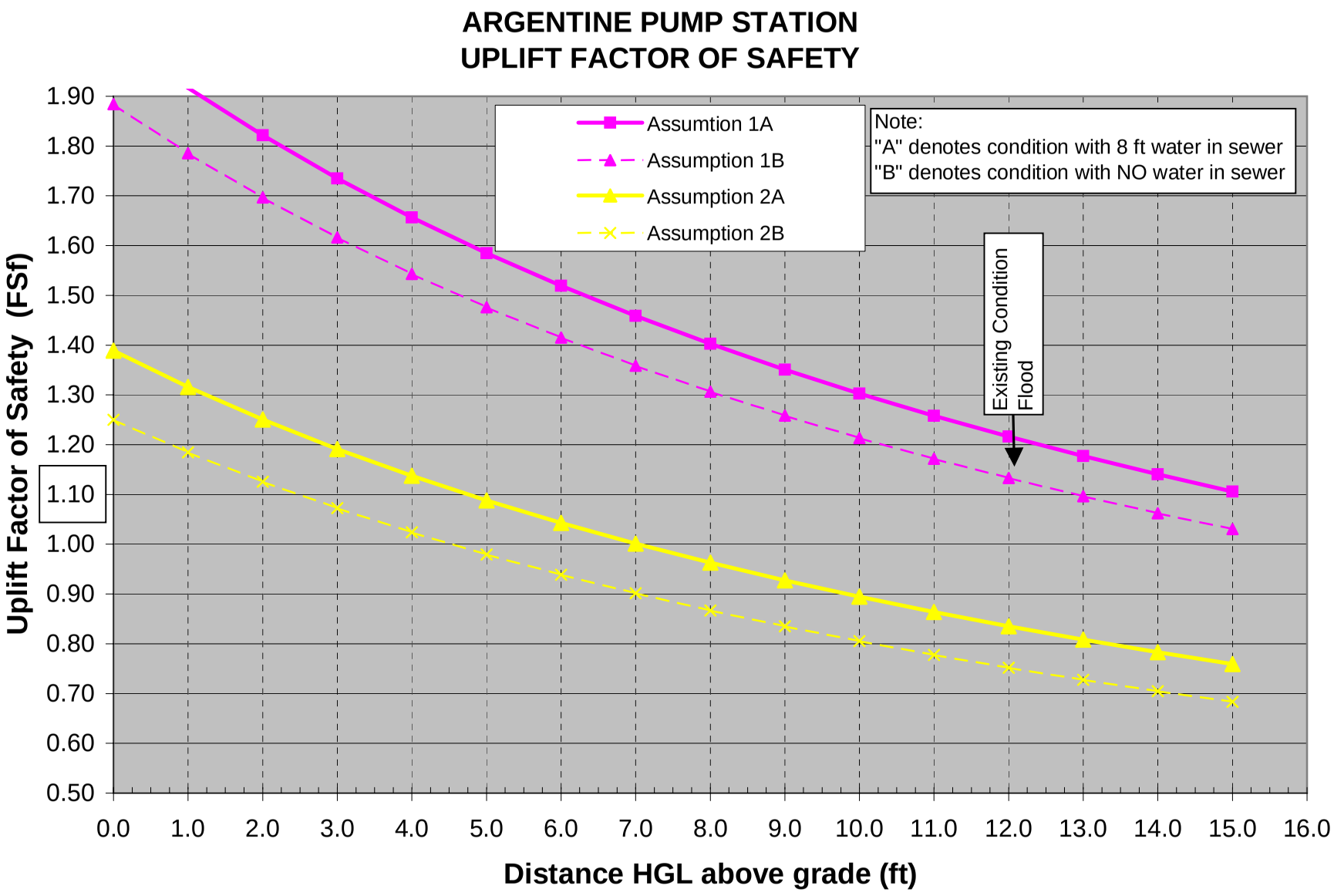
- A Summary of the uplift factors of safety based upon existing condition with HGL 12 feet above grade: Note: target factor of safety to meet criteria is 1.1

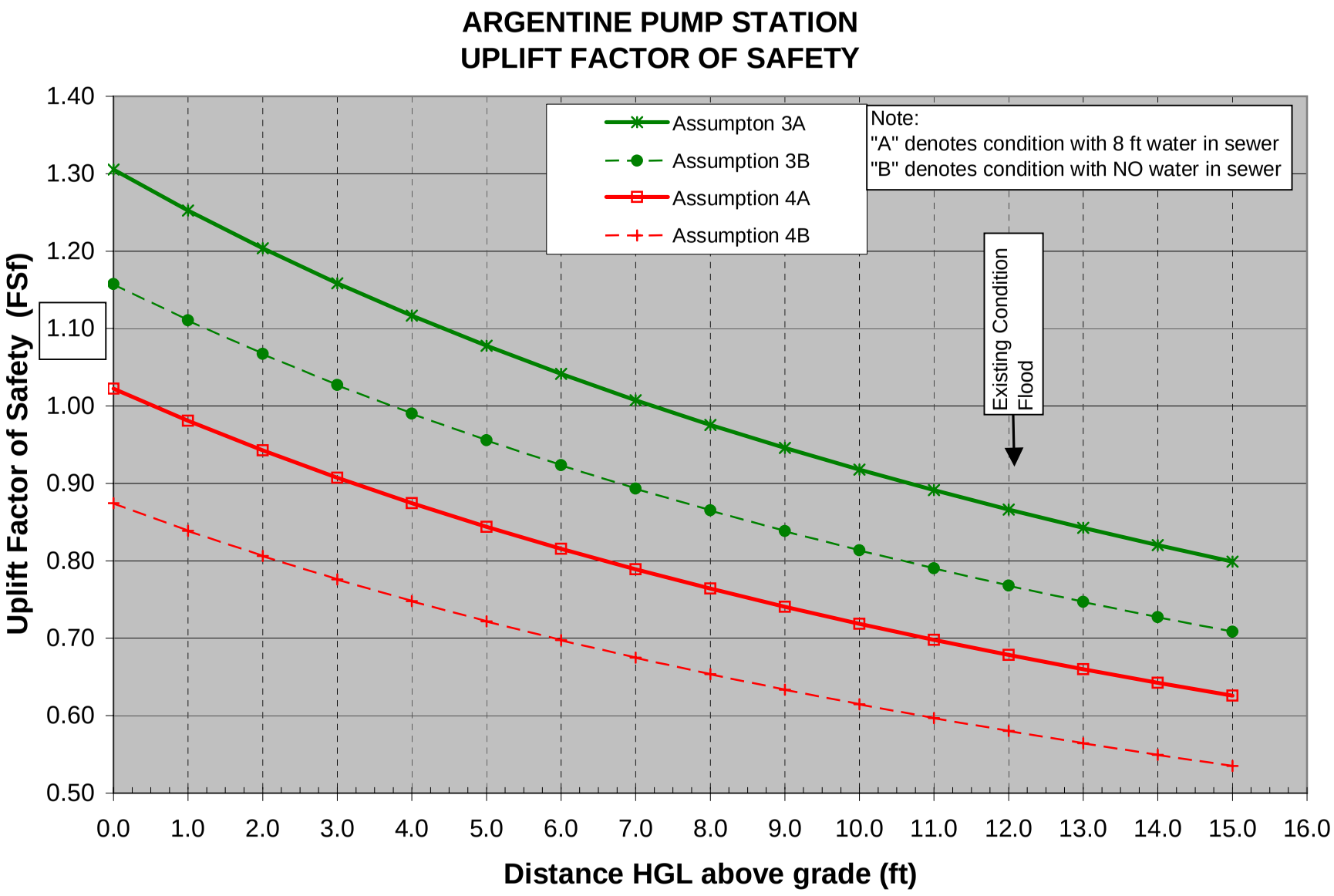
condition	Factor of Safety (UPLIFT)
Assumption 1 A	1.22
Assumption 1 B	1.13
Assumption 2 A	0.83
Assumption 2 B	0.75
Assumption 3 A	0.87
Assumption 3 B	0.77
Assumption 4 A	0.68
Assumption 4 B	0.58
Assumption 5 (pump pits)	0.67
Assumption 5 (old engine pit)	0.74

Hydraulic Grade Line for the Argentine Pump Station		
Elevation New Top of levee (MSL)	776.6	
Elevation at Levee Toe (MSL)	762	
Elevation Bottom of Blanket (MSL)	737	
Driving Head (ft)	14.6	
L1 (ft)	100	
L2 (ft)	100	
Le (ft)	1012	
ho	12.19	
Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
-150	14.6	776.60
0	12.19	774.19
20	11.96	773.96
40	11.72	773.72
60	11.49	773.49
80	11.27	773.27
100	11.05	773.05
120	10.83	772.83
140	10.62	772.62
160	10.41	772.41
180	10.21	772.21
200	10.01	772.01
220	9.81	771.81
240	9.62	771.62
260	9.43	771.43
280	9.25	771.25
300	9.07	771.07
320	8.89	770.89
340	8.72	770.72
360	8.55	770.55
380	8.38	770.38
400	8.21	770.21
420	8.05	770.05
440	7.90	769.90
460	7.74	769.74
464	7.71	769.71

Hydraulic Grade Line for the Argentine Pump Station




ARGENTINE PUMP STATION
UPLIFT FACTOR OF SAFETY
Uplift Factor of Safety (FSf)
1.90
1.80
1.70
1.60
1.50
1.40
1.30
1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
Distance HGL above grade (ft)
Assumtion 1A
Assumption 1B
Assumption 2A
Assumption 2B
Note:
"A" denotes condition with 8 ft water in sewer
"B" denotes condition with NO water in sewer
Existing Condition
Flood


ARGENTINE PUMP STATION
UPLIFT FACTOR OF SAFETY
Uplift Factor of Safety (FSf)
1.40
1.30
1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
Distance HGL above grade (ft)
Assumpton 3A
Assumption 3B
Assumption 4A
Assumption 4B
Note:
"A" denotes condition with 8 ft water in sewer
"B" denotes condition with NO water in sewer
Existing Condition
Flood

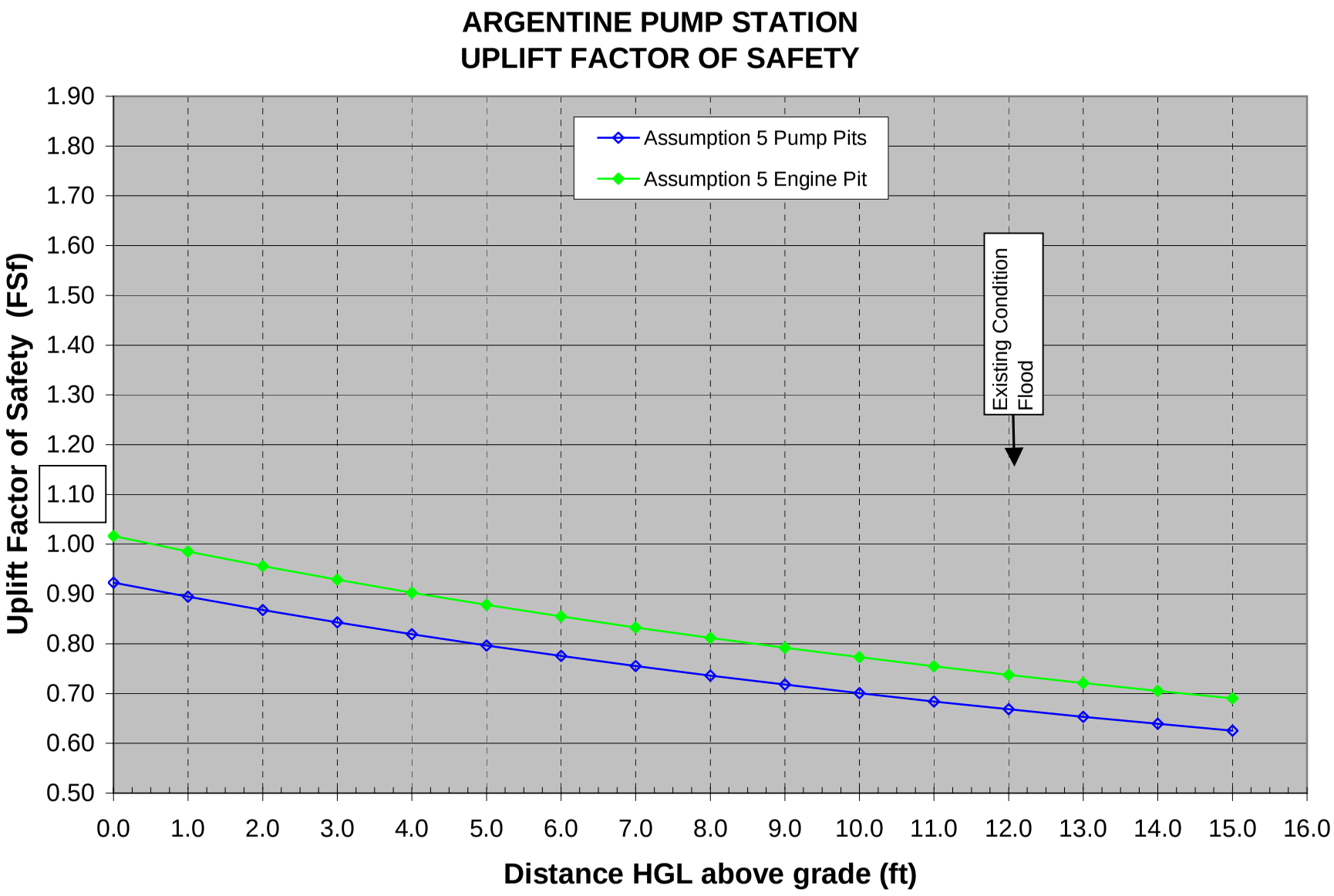

ARGENTINE PUMP STATION
UPLIFT FACTOR OF SAFETY
Assumption 5 Pump Pits
Assumption 5 Engine Pit
Existing Condition Flood
Uplift Factor of Safety (FSf)
1.90
1.80
1.70
1.60
1.50
1.40
1.30
1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
Distance HGL above grade (ft)

EXHIBIT A-15.6

Argentine Unit – Strong Avenue Pump Station Analysis

US Army Corps of Engineers

EXHIBIT 3
Argentine Flood Unit
Pump Station Analysis for Strong Ave. Sta. 273+41
Strong Ave. and Kaw River 4-14-1916 Sheet 1 of 2

Comp by: WGB
Chkd by: LAS
Except for last page.

KANSAS RIVER LEVEES

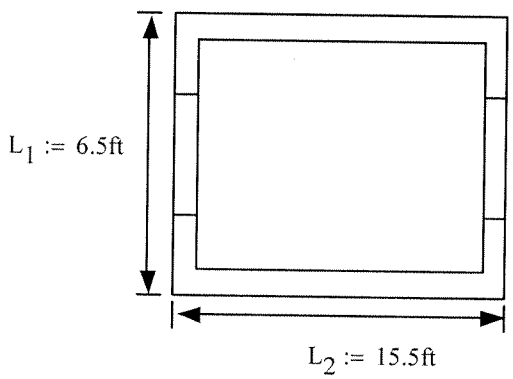
Existing Conditions, Mean Material Values

Results used for Probablity of Failure Calcs. in *Strong Avenue Probability of Failure Analysis.mcd*

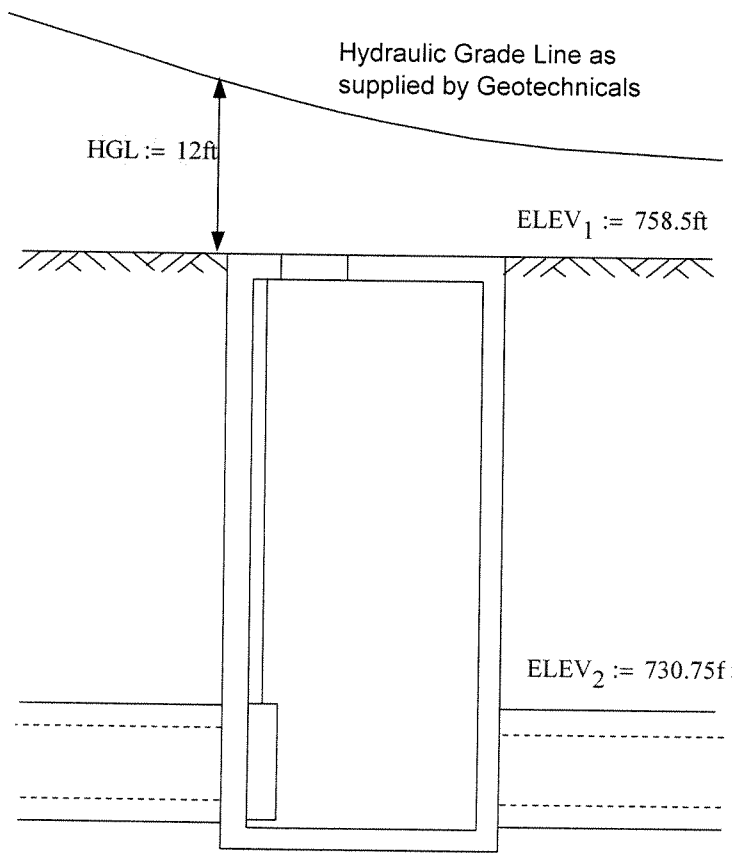
Variables

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

Properties



Plan



Elevation

Water head pressure in Wet Well under operating conditions. $HEAD := 8ft$

$H := ELEV_1 - ELEV_2$

$H = 27.75\,ft$

Wall Thickness 1 $D_1 := 12in$

Clear Cover 1 $C_{c1} := 5in$

Floor Thickness $D_f := 10in$

$d_1 := D_1 - C_{c1}$

Wall Thickness 2 $D_2 := 24in$

Clear Cover $C_{cf} := 5in$

$l_2 := L_2 - 2 \cdot D_1$

$d_f := D_f - C_{cf}$

Clear Cover 2 $C_{c2} := 5in$

$d_2 := D_2 - C_{c2}$

$l_1 := L_1 - 2 \cdot D_2$

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. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi.
- The Concrete Reinforcing Steel Institute *Engineering Data Report Number 48* recommends using 33 ksi yield strengths for rebar of this time period. It should be noted that this is a minimum yield strength, not an average value. To obtain an average value it is assumed that actual steel strengths varied at least 1 standard of deviation from the minimum required strength. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete is 14%. As a result the mean Steel Yield strength shall be taken as 37.6 ksi.

Soil Properties $\phi := 28deg$

$\gamma := 120pcf$ $K_o := 1 - \sin(\phi)$

Water Unit Weight $\gamma_w := 62.4pcf$ $K_o = 0.531$

Steel Properties $F_y := 33 \cdot 1.14 \cdot ksi$

Concrete Properties $f'_c := 3.75ksi$

Concrete Unit Weight $\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

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

Load Factors

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

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

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

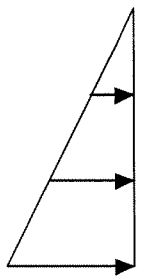
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALLS

Wall Loading

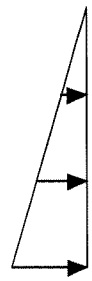
Soil or Soil & Water



$L_2 := H$

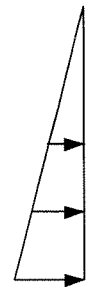
$L_2 = 27.75\,ft$

OR



$L_2 = 27.75\,ft$

&



$L_3 := H + HGL - HEAD$

$L_3 = 31.75\,ft$

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

$W_1 = 1767 \frac{lb}{ft^2}$

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

$W_2 = 2829 \frac{lb}{ft^2}$

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

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

$W = 2829 \frac{lb}{ft^2}$

WALL 1

SHEAR

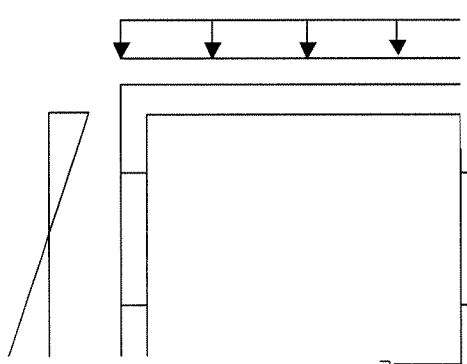
$THRUST_1 := \frac{W \cdot L_2}{2}$

$THRUST_1 = 39255 \frac{lb}{ft}$

Thrust to be used in CASTR for thrust analysis

$V'_{u1} := W \cdot \frac{l_1}{2}$

$V'_{u1} = 3536 \frac{lb}{ft}$



$V_{u1} := V'_{u1} \cdot \frac{\left(\frac{l_1}{2} - d_1\right)}{\frac{l_1}{2}}$

$V_{u1} = 1886 \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\frac{l_1}{d_1} = 4.286$

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

$\phi V_{n1} := \phi_V \cdot \left[3.3 \cdot \left(\sqrt{f'_c}\right)\left(12in \cdot d_1\right) \cdot \frac{lb^{.5}}{in \cdot ft} + \frac{THRUST_1 \cdot d_1}{4 \cdot l_1}\right]$

ACI 11.10.6 EQ (11-31)

$\phi V_{n1} = 19265 \frac{lb}{ft}$

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

Check1 = "OKAY"

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

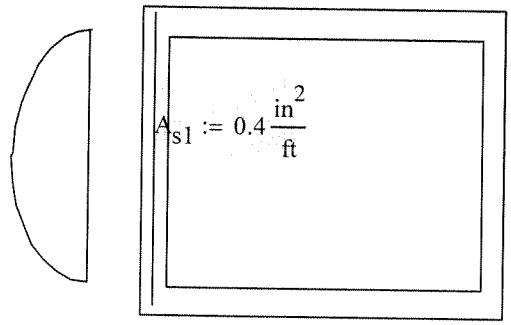
$FS_{S1} = 10.214$

Bending

NOTE: Discontinuous steel at corners. Elements act as simply supported beams with M=W*L^2/8 at midspan.

$$M_{u1} := \frac{W \cdot l_1^{\,2}}{8}$$

$$M_{u1} = 2210 \frac{ft \cdot lb}{ft}$$



$$A_{s1} := 0.4 \frac{in^2}{ft}$$

1/2" Bars @ 6 in. Steel

$$b := 12 \frac{in}{ft} \quad a := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \quad a = 0.393\,in$$

$$\phi M_{n1} := \phi_B \cdot A_{s1} \cdot F_y \cdot \left(d_1 - \frac{a}{2}\right)$$

$$\phi M_{n1} = 8.5 \frac{kip \cdot ft}{ft}$$

Check2 determines if the negative steel is capable of carrying the fixed end moment. A 1.5 existing conditions factor of safety is considered acceptable and will not require risk analysis.

$$Check2 := if\left(\phi M_{n1} > 1.5 M_{u1}, "OKAY", "NO\ GOOD"\right)$$

Check2 = "OKAY"

Factor of Safety $\quad FS_{B1} := \frac{\phi M_{n1}}{M_{u1}} \quad FS_{B1} = 3.86$

Controlling Factor of Safety

$$FS_1 := \min\left(FS_{S1}, FS_{B1}\right)$$

Controlling Mechanism

$$Comment1 := if\left(FS_{B1} > FS_{S1}, "Wall\ 1\ Shear", "Wall\ 1\ Flexural\ Steel"\right)$$

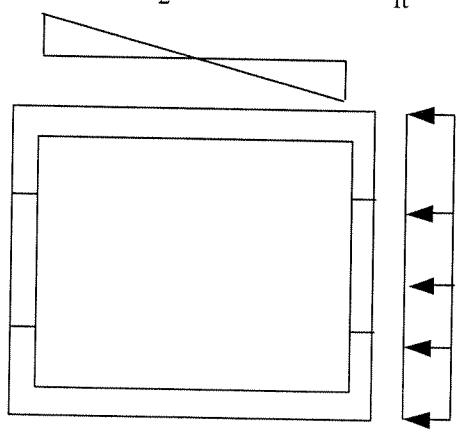
Factor of Safety $\quad FS_1 = 3.86$

Controlling Mechanism $\quad Comment1 = "Wall\ 1\ Flexural\ Steel"$

WALL 2

SHEAR

$V'_{u2} := W \cdot \frac{l_2}{2}$ $\quad V'_{u2} = 19097 \frac{lb}{ft}$



Thrust to be used in CASTR for thrust analysis

$THRUST_2 := \frac{W \cdot L_1}{2}$ $\quad THRUST_2 = 9195 \frac{lb}{ft}$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\frac{l_2}{d_2} = 8.526$ EM 1110-2-2104 EQ (5-1)
Does not Apply

$V_{u2} := V'_{u2} \cdot \frac{\left(\frac{l_2}{2} - d_2\right)}{\frac{l_2}{2}}$ $\quad V_{u2} = 14618 \frac{lb}{ft}$

$\phi V_{n2} := \phi_V \cdot \left[3.3 \cdot \left(\sqrt{f'_c}\right)\left(12in \cdot d_2\right) \cdot \frac{lb^{.5}}{in \cdot ft} + \frac{THRUST_2 \cdot d_2}{4 \cdot l_2}\right]$ ACI 11.10.6 EQ (11-31)

$\phi V_{n2} = 46345 \frac{lb}{ft}$

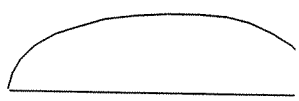
$Check3 := if\left(\phi V_{n2} > 1.5 V_{u2}, "OKAY", "NO GOOD"\right)$ $\quad$ Check3 = "OKAY"

Factor of Safety $\quad FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$ $\quad FS_{S2} = 3.17$

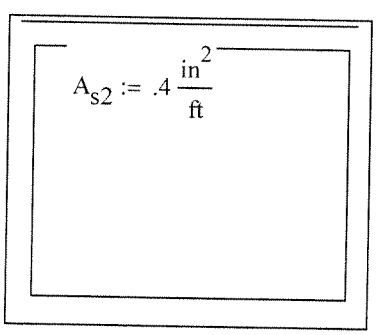
Bending

NOTE: Discontinuous steel at corners. Elements act as simply supported beams with M=W*L²/8 at midspan.

$M_{u2} := \frac{W \cdot l_2^2}{8}$



$M_{u2} = 64453 \frac{ft \cdot lb}{ft}$



$A_{s2} := .4 \frac{in^2}{ft}$

1/2" Bars @ 6 in. Steel

$b := 12 \frac{in}{ft}$ $\quad a := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a = 0.393 in$ $\quad d_2 = 19 in$

$\phi M_{n2} := \phi_B \cdot A_{s2} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right)$ $\quad \phi M_{n2} = 23.6 \frac{kip \cdot ft}{ft}$

Check4 determines if the negative steel is capable of carrying the fixed end moment. A 1.5 existing conditions factor of safety is considered acceptable and will not require risk analysis.

$Check4 := if(\phi M_{n2} > 1.5M_{u2}, "OKAY", "NO GOOD")$ $\quad$ $Check4 = "NO GOOD"$

Factor of Safety $\quad FS_{B2} := \frac{\phi M_{n2}}{M_{u2}}$ $\quad FS_{B2} = 0.366$

Shear

$FS_S := FS_{S2}$ $\quad FS_S = 3.17$

Bending

$FS_B := FS_{B2}$ $\quad FS_B = 0.366$

Controlling Factor of Safety

$FS_2 := \min(FS_S, FS_B)$

Controlling Mechanism

$Comment2 := if(FS_B > FS_S, "Wall\ 2\ Shear", "Wall\ 2\ Flexural\ Steel")$

Factor of Safety $\quad FS_2 = 0.366$

Controlling Mechanism $\quad Comment2 = "Wall\ 2\ Flexural\ Steel"$

Controlling Factor of Safety

$FoS := \min(FS_{S1}, FS_{B1}, FS_{S2}, FS_{B2})$ $\quad FoS = 0.366$

Controlling Mechanism

$Mechanism := if(FS_2 > FoS, Comment1, Comment2)$ $\quad Mechanism = "Wall\ 2\ Flexural\ Steel"$

The project team has descided 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 for this is that the load factor (live load neglecting hydraulic structure) divided by the strength reduction factor is approximately 1.75. Because of the more detailed load analysis using combined load distribution factors (initial load and levee raise) and relative certainty of existing steel it has been decided that a Factor of Safety of 1.5 will be acceptable.

$Result := if(FoS > 1.5, "Acceptable", "Consider\ Moment\ Thrust\ Interaction")$

$Result = "Consider\ Moment\ Thrust\ Interaction"$

Further Analysis with Thrust Considerations

If the Factor of Safety is less than 1.5 additional flexural capacity shall be considered as a result of the axial loading.

$$THRUST := if(FoS > 1.5, 0, if(FoS < FS_{B1}, THRUST_2, THRUST_1))$$

$$M_u := if(FoS > 1.5, 0, if(FoS < FS_{B1}, M_{u2}, M_{u1}))$$

Information for CASTR: $THRUST = 9.19 \frac{kip}{ft}$ $M_u = 64.453 \frac{kip \cdot ft}{ft}$

From CASTR Program *STRGAVEMEAN.dat*

Percent Design Strength Used in Tensile Zone

$Percent := 178.17\%$

$FoS' := \frac{1}{Percent \cdot .9}$ Note: Strength Reduction Factor (.9) not applied. $FoS' = 0.62$

US Army Corps of Engineers

EXHIBIT 3
Argentine Flood Unit
Pump Station Analysis for Strong Ave. Sta. 273+41

Strong Ave. and Kaw River 4-14-1916 Sheet 1 of 2
Mechanical Plan and Sections, DR&G As Built Drawings,
1-17-94

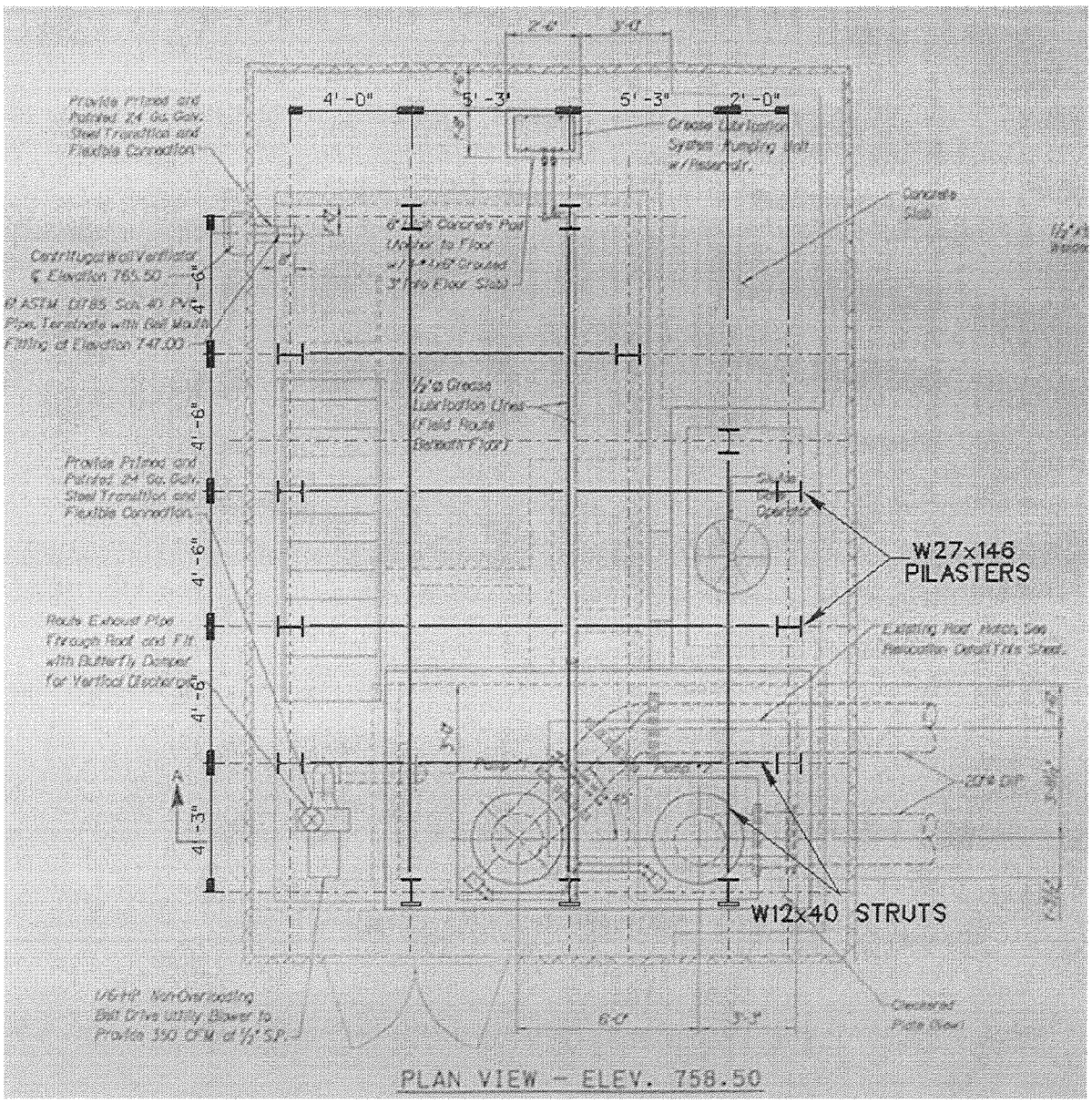
Comp by: WGB
Chkd by: LAS

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

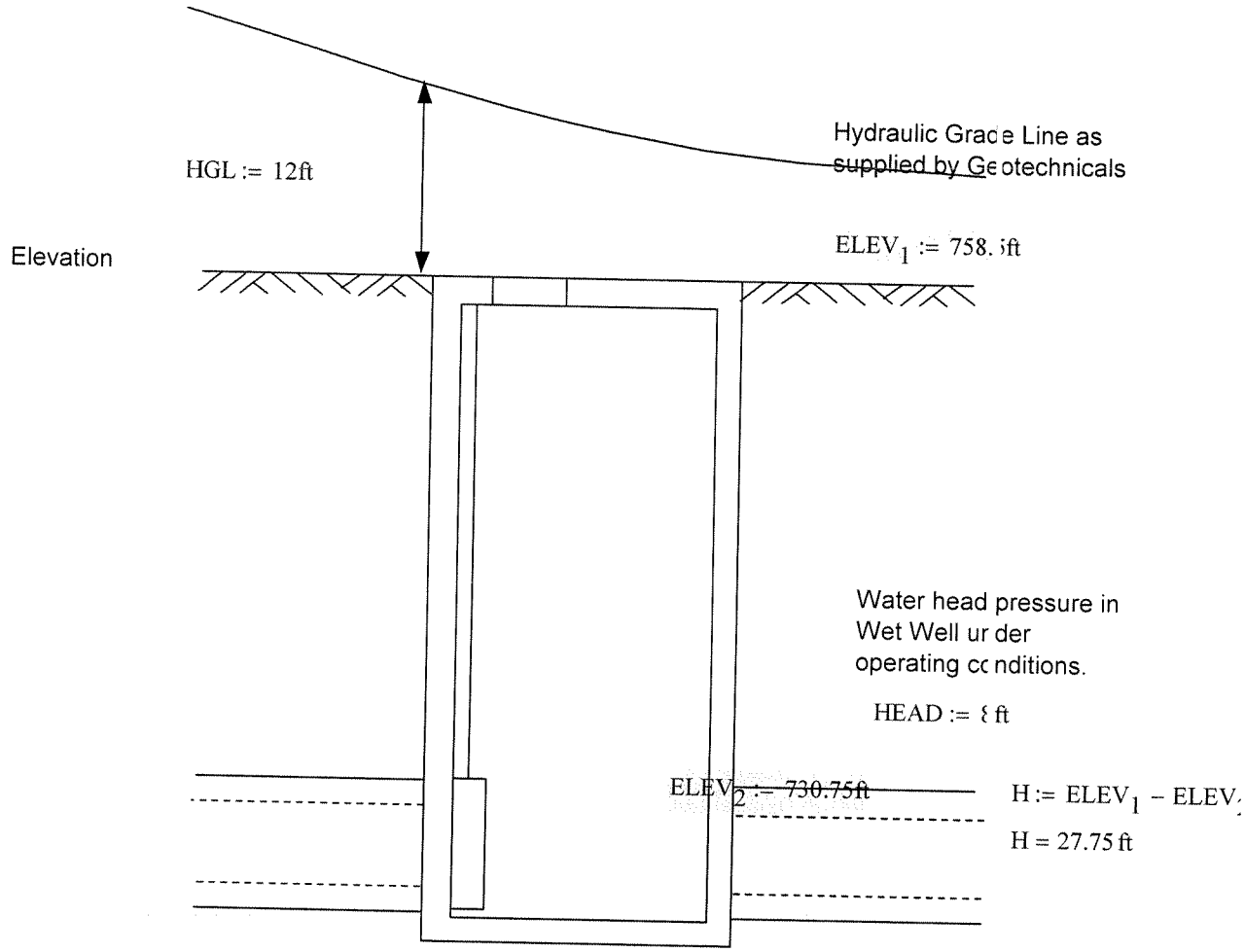
Variables

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

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



Properties



Assumptions

- Steel W-Section Pilasters will be embeded into the existing floor and an additional 10" of concrete added to the basement foundation slab for uplift and floor strength concerns.
- The steel pilasters will be braced just below the existing floor elevation with a steel struts.

Soil Properties $\phi := 28deg$

$\gamma := 120pcf$ $K_o := 1 - \sin(\phi)$ $K_o = 0.531$

Steel Properties $F_y := 50ksi$

Water Unit Weight $\gamma_w := 62.4pcf$

Load & Resistance Factor Design

Strength Reduction Factors

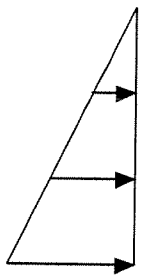
Shear Strength	$\phi_V := .85$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) not applied.
Flexural Strength	$\phi_B := 0.90$	
Load Factors		
Dead and Live Load Factor	$\gamma_L := 1.6$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1.3$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := .75$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Analysis

WALLS

Wall Loading

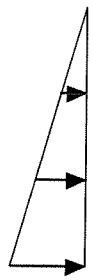
Soil or Soil & Water



$L_2 := H$

$L_2 = 27.75\,ft$

OR



$L_2 = 27.75\,ft$

&

$L_3 := H + HGL - HEAD$

$L_3 = 31.75\,ft$

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

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

$W_1 = 3675 \frac{lb}{ft^2}$

$W_2 = 4414 \frac{lb}{ft^2}$

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

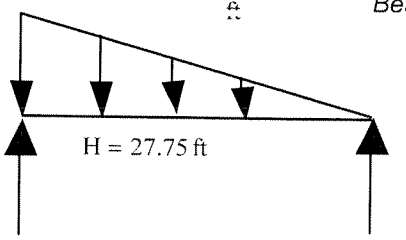
$W' := if(W_1 > W_2, W_1, W_2)$

$W' = 4414 \frac{lb}{ft^2}$

Loading $W := W' \cdot Width$

Tributary Width $Width := 4.5ft$

$W = 19.861 \frac{kip}{ft}$



Reaction and Moments from AISC Manual of Steel Construction

Beam Diagrams and Formulas

$$\phi M_{Max} := \frac{2\left(\frac{W \cdot H}{2}\right) \cdot H}{9 \cdot \sqrt{3}} \qquad \phi M_{Max} = 981.1\,ft \cdot kip$$

$$R_2 := \frac{\left(\frac{W \cdot H}{2}\right)}{3} \qquad R_2 = 91.857\,kips$$

PILASTERS

From AISC Manual of Steel Construction Beam Tables

Unbraced Length: $H = 27.75\,ft$

Design Moment: $\phi M_{Max} = 981.1\,ft \cdot kip$

Yield Strength: $F_y = 50\,ksi$

Use W 27x146

STRUTS

From AISC Manual of Steel Construction Beam Tables

Unbraced Length: $Width = 4.5\,ft$

Axial Load: $R_2 = 91.9\,kip$

Yield Strength: $F_y = 50\,ksi$

Use W 12x40

NOTE: Does not take into account floor load (combined axial and bending) but W 12x40 is good estimate for feasibility study. Additional Analysis to be performed at time of Plans and Specifications.

EXHIBIT A-15.7

Additional Turner, Argentine, and Strong Avenue Pump Plant Calculations

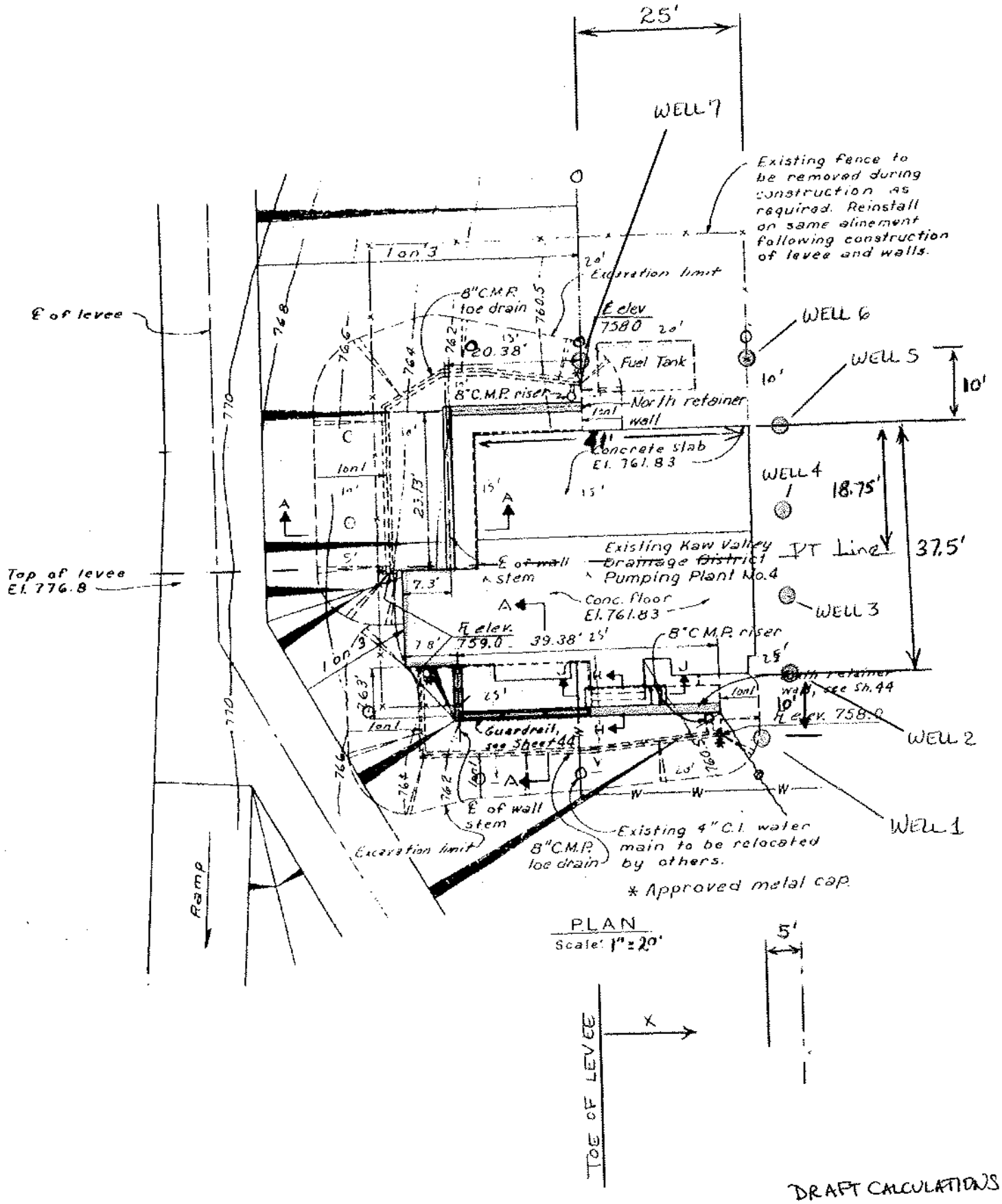
GEOTECHNICAL SUPPORT
TURNER PUMP PLANT

The geotechnical design section was tasked by the design team to determine the following items needed for evaluation of the Turner Pump Plant.

1) An underseepage analysis was needed to determine the amount of excessive had above grade that the foundation sands would produce on the pump plant for the existing condition Turner Pump plant and the 500 design water surface plus 3 feet of overbuild future condition alternative. This information was determined using existing boring information and characterizing the area near Station 60+40 to develop the EC and FC levee foundation blanket of silts and clays and the underlying foundation sand thickness. The hydraulic top of levee set in the future conditions report was used to determine the amount of river head on the foundation for the n500 + 3 case and the as built drawings with supplemental survey information was used to determine the river head for the existing conditions. The foundation soil parameters were developed using Kansas City district underseepage criteria outline in the geotechnical chapter in the FC report. It was determined that an excessive head of 15.25 feet would be present at the base of the foundation for the n500 + 3 levee raise.

2) A recommendation for relief of the excessive head developed for future condition n500 + 3 alternate were assigned as part of this scope of work. It was reported by the St. Louis District Corps that marginal Factor of safety with respect to uplift resulted due to the excessive head in the foundation sands below the Turner Pump plant. This work was approached using a system of relief wells to reduce the foundation pressure in the foundation sands. The original model used 4 wells. One well was assigned to each corner of the pump plant structure. The existing Turner pump plant is built 15 feet into the landside slopes of the levee. The wells could be designed to provide adequate pressure relief but the costs to excavate into the existing levee in order to install the wells would be very high. Temporary flood protection would also be required during removal of the landside levee slope. Another series of wells were modeled to be place at convenient locations that did not interfere with the levee. This required the wells to be placed landside of the levee. Seven relief wells were modeled to control the foundation pressure. The parameters used to model the flow and pressure relief of the wells were based on historic research of wells placed along the landside toe of existing levees and their performance during high river stages. This information provided a guide to the expected magnitude of relief wells flow for given foundation conditions and given river head. It was found that most well flows ranged form 0.75 cfs to 1.5 cfs for the expected head conditions model at the Turner pump plant. Historic pump test data provided a range of expected foundation permeability values for the model. The model uses fully penetrating relief wells to bedrock. The pressure relief was calculated for the mid well location that would provide the highest pressure values after reduction of the gradient.

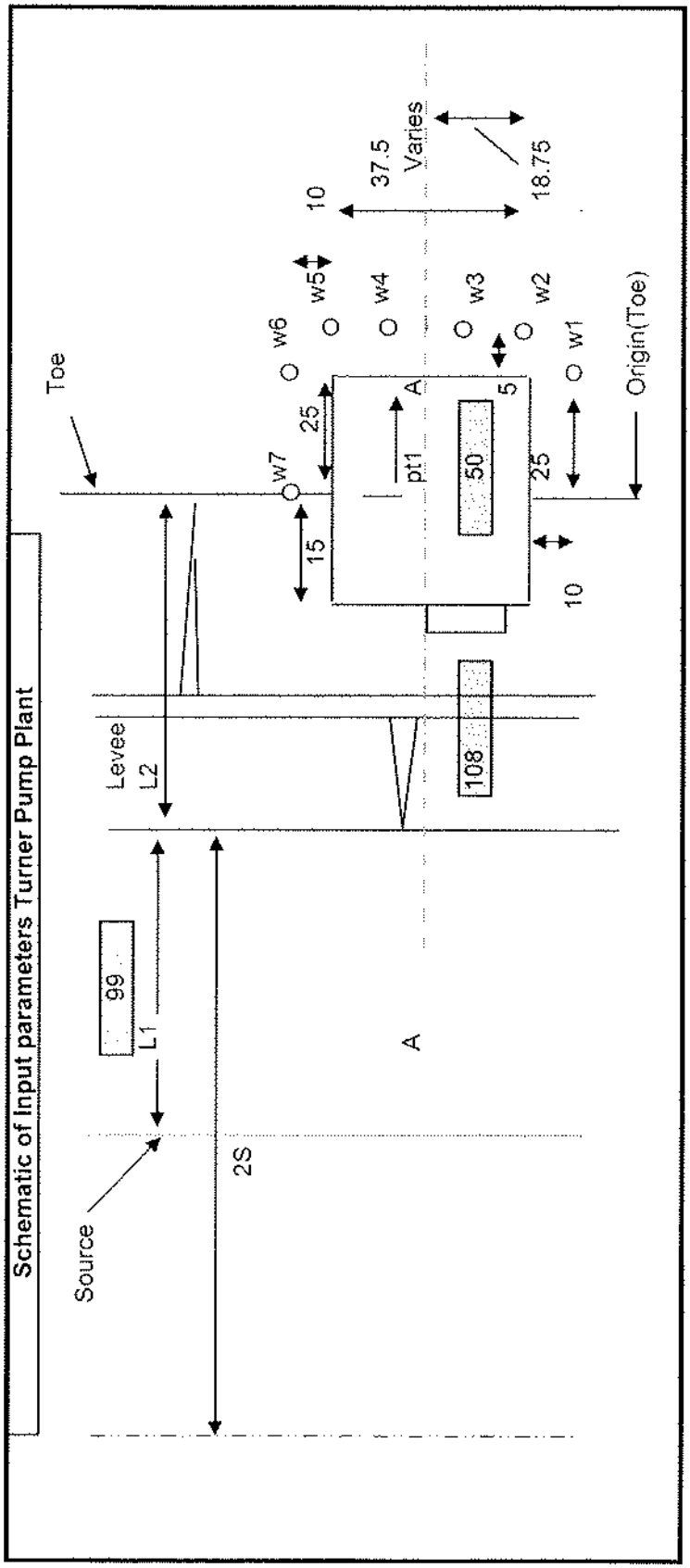
The resulting relief wells spacing is shown on the enclosure and calculations are provided. Although the final number of wells was increased from 4 to 7, the total discharge need from the wells to drop the pressure and average of eight feet from the river induced gradient was increased form 6 to 7 cfs. The well discharge invert was held at 1.5 feet below grade. Future geotechnical design can improve on the number of wells or invert elevation to adjust for structural and mechanical designers needs. Seven relief wells penetrating to a depth of 75 feet were used for the feasibility cost estimate. A header system was added to the estimate along with modification to the pump pant to keep the discharge evacuated for the pipe system into the pump plant during a flood event.



DRAFT CALCULATIONS

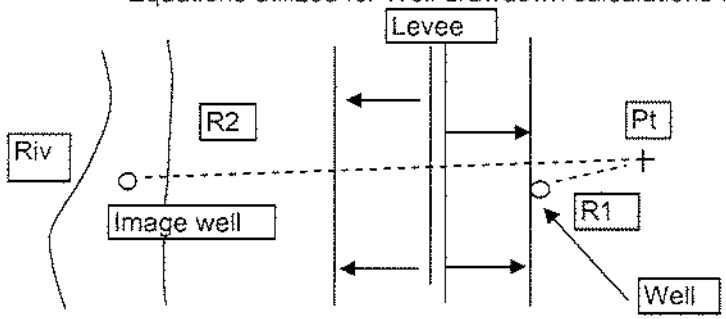
7 WELL ALTERNATE

TURNER PUMP PLANT STA 60+40



7 Well Alternative
ARGENTINE LEVEE
UNIT
3/11/2005 LOETHR
EC-GD

Equations utilized for Well drawdown calculations : Drawdown = (Q/(2*pi*k*Df)) ln (R2/R1)



Well Design Calculations

Loehr EC-GD 3/11/2005

Well Drawdown Influence Table

Turner - Argentine KS: Station 60+40

	ic =	0.84	Well losses	1.3	(entrance, velocity bends)	
Ground = 0			Discharge elev loss	-1.5	Dtbl=	15	Feet
Location of points at the base of the blanket perpendicular to the center well			DD at Toe	6.74	Qw =	1	cfs
			X	890	k =	0.0052	ft /sec
			h0	15.25	Df =	60.0	feet

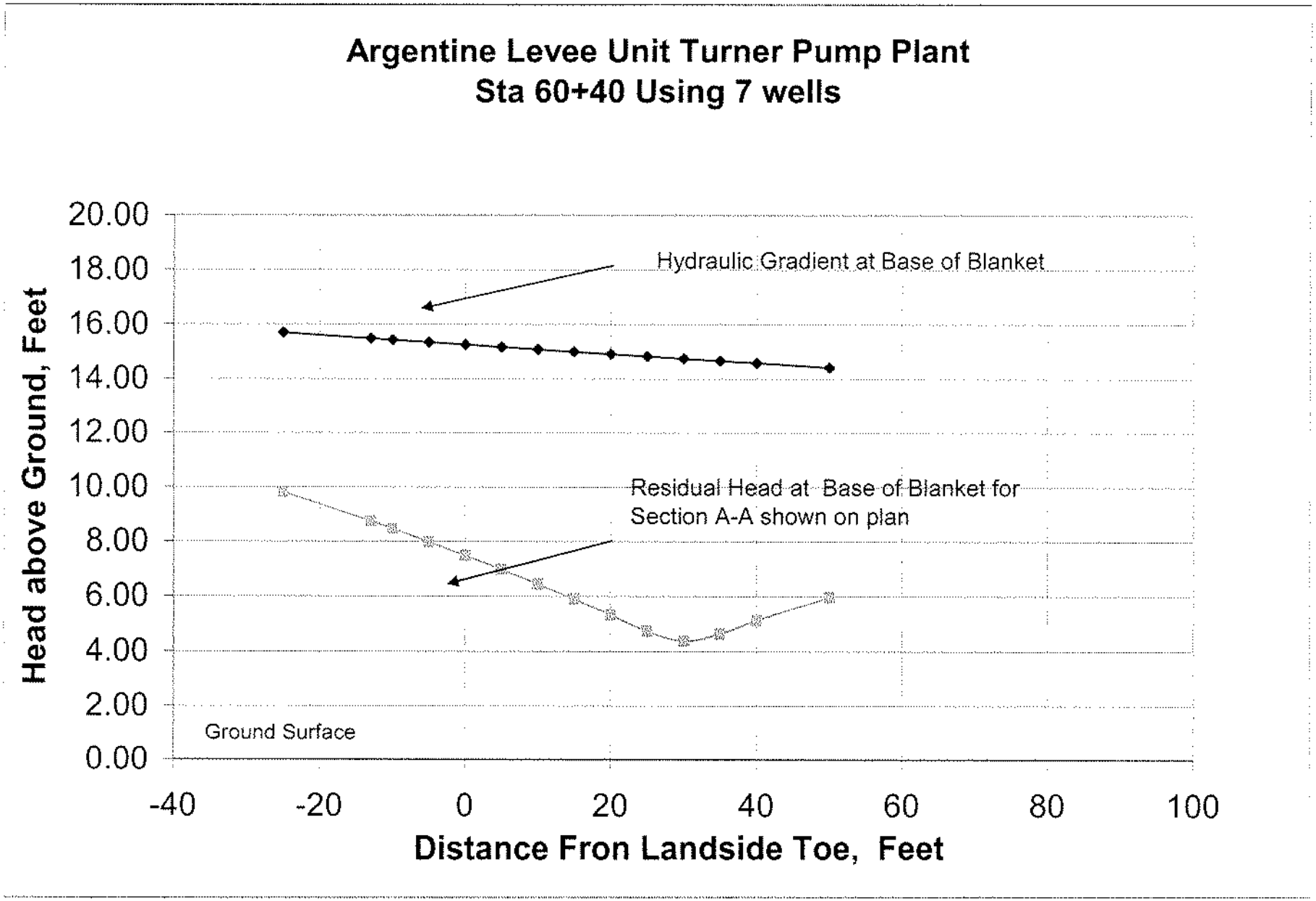
Distance gx	Head hx	Well 0 wx	DDwx Feet	Residual Head Feet	Gradient Factor of Safety
-25	15.68	-25	5.88	9.80	1.97
-13	15.47	-13	6.74	8.73	1.44
-10	15.42	-10	6.97	8.45	1.49
-5	15.34	-5	7.36	7.98	1.58
0	15.25	0	7.77	7.48	1.68
5	15.16	5	8.19	6.97	1.81
10	15.08	10	8.63	6.45	1.95
15	15.00	15	9.1	5.90	2.14
20	14.91	20	9.59	5.32	2.37
25	14.83	25	10.09	4.74	2.66
30	14.74	30	10.36	4.38	2.87
35	14.66	35	10.02	4.64	2.71
40	14.58	40	9.46	5.12	2.46
50	14.42	50	8.45	5.97	2.11

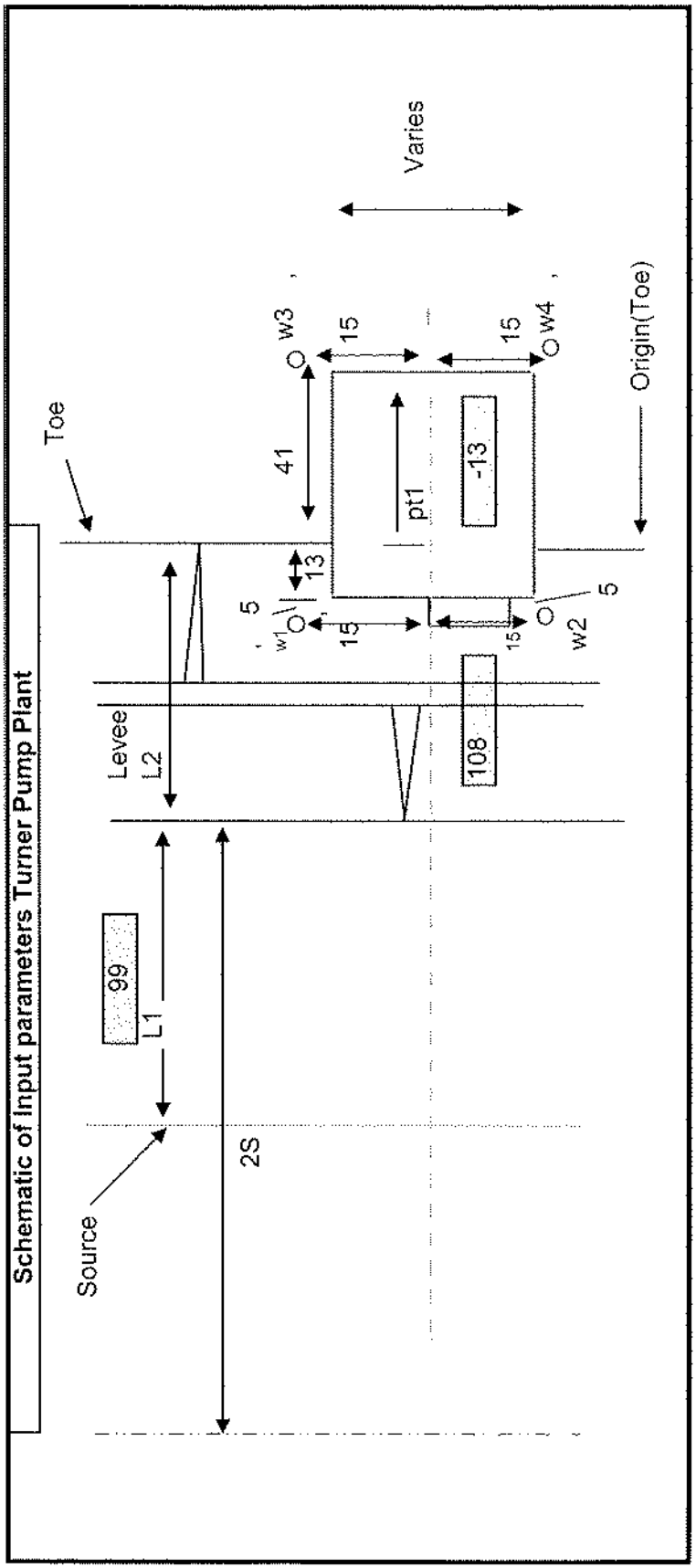
PT1 data

Example Calculation

well 1		well 6		well 3		well 4	
S =	232	same as well 1		S =	237	Use well 3	
r2-1=	440			r2-1=	444		
r1-1=	38.1			r1-1=	31		
DDw1=	1.25	DDw2=	1.25	DDw2=	1.36	DDw2=	1.36
well 2		well 5		well 7			
S=	237	same as well 2		S =	207		
r2-3 =	445			r2-1=	415		
r1-3=	35			r1-1=	29		
DDw3=	1.29	DDw4=	1.29	DDw7=	1.36		

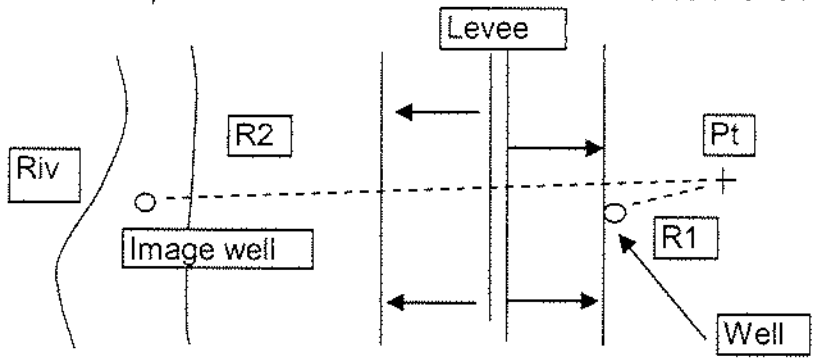
Dist x	DDwx	Headlosses	well number	Net DDwx
0	1.25	-0.2	1	1.05
	1.29	-0.2	2	1.09
	1.36	-0.2	3	1.16
	1.36	-0.2	4	1.16
	1.29	-0.2	5	1.09
	1.25	-0.2	6	1.05
	1.36	-0.2	7	1.16
		PT1 Sum DD =	7.77	feet


Argentine Levee Unit Turner Pump Plant
Sta 60+40 Using 7 wells
Hydraulic Gradient at Base of Blanket
Residual Head at Base of Blanket for
Section A-A shown on plan
Ground Surface
Head above Ground, Feet
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
-40
-20
0
20
40
60
80
100
Distance Fron Landside Toe, Feet



4 WELL ALTERNATIVE
ARGENTINE LEVEE
UNIT
3/11/2005 LOEHR
EC·GD

Equations utilized for Well drawdown calculations : Drawdown = (Q/(2*pi*k*Df)) ln (R2/R1)



Well Design Calculations

Loehr EC-GD 3/24/2004

Well Drawdown Influence Table

Turner - Argentine KS: Station 60+40

	ic =	0.84	Well losses	1.3	(entrance, velocity bends)		
Ground = 0			Discharge elev loss	-1.5	Dbl=	15	Feet
Location of points at the base of the blanket perpendicular to the center well			DD at Toe	8.84	Qw =	1.5	cfs
			X_L	890	k =	0.0052	ft /sec
			h0	15.25	Df =	60.0	feet
Distance	**Head**	**Well 0**	**DDwx**	**Residual Head**	**Gradient Factor**		
gx	**hx**	**wx**	**Feet**	**Feet**	**of Safety**		
-18	**15.56**	**-18**	**8.76**	**6.80**	**2.84**		
-13	**15.47**	**-13**	**8.84**	**6.63**	**1.90**		
14	**15.01**	**14**	**8.72**	**6.29**	**2.00**		
41	**14.56**	**41**	**9.17**	**5.39**	**2.34**		
51	**14.40**	**51**	**8.73**	**5.67**	**2.22**		

Example Calculation

		PT1				
		data	well 1		well 2	
			S =	189		
			r2-1=	383		
			r1-1=	15.8		
			DDw1=	2.44	DDw2=	2.44
			well 3		well4	
			S=	248		
			r2-3 =	442		
			r1-3=	56		
			DDw3=	1.58	DDw4=	1.58
	Dist x	DDwx	Headlosses	Number	Net DDwx	
	-13	2.44	-0.2	1	2.64	
		2.44	-0.2	1	2.64	
		1.58	-0.2	1	1.78	
		1.58	-0.2	1	1.78	
			PT1 Sum DD =	8.84	feet	

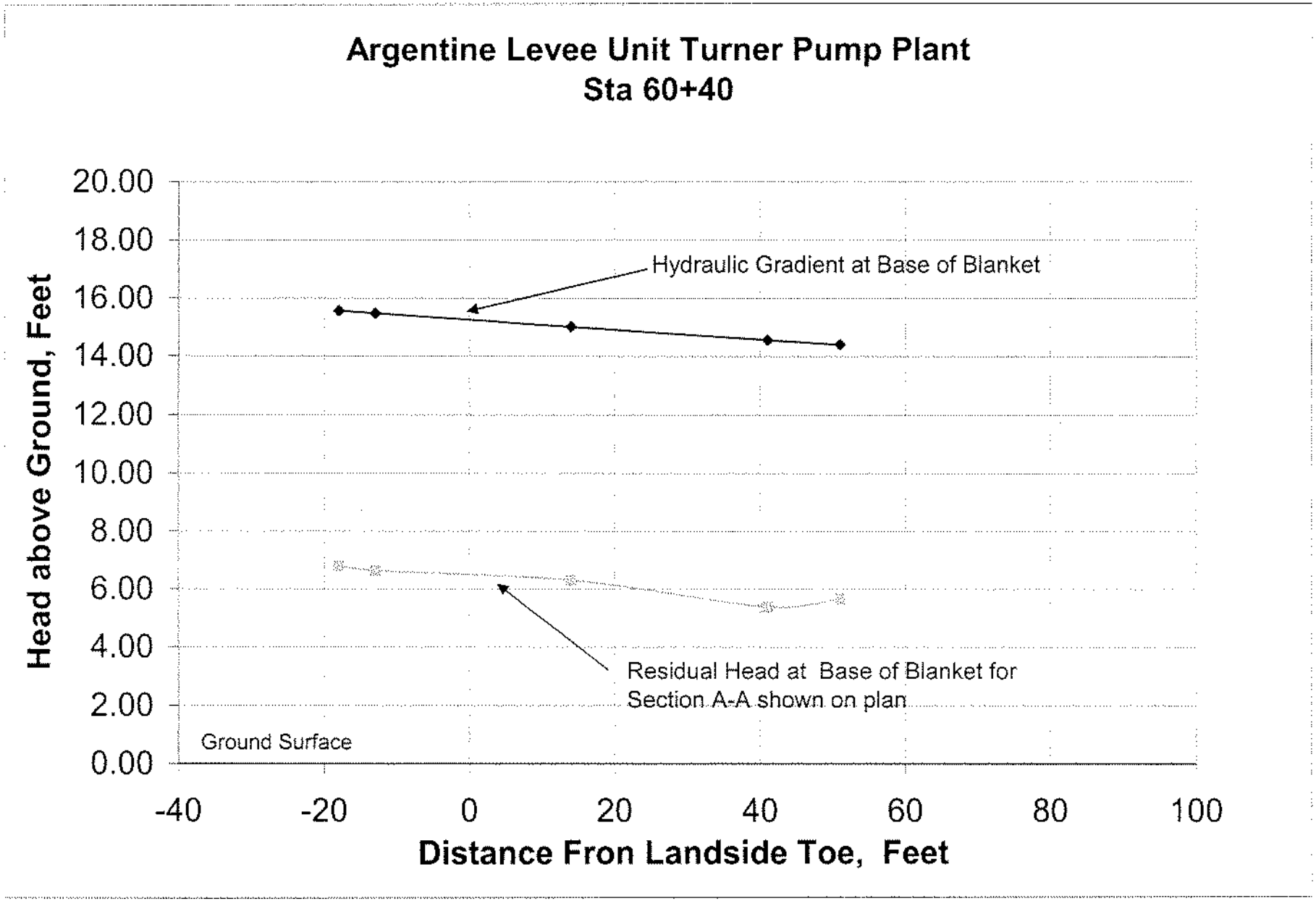

Argentine Levee Unit Turner Pump Plant
Sta 60+40
Head above Ground, Feet
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
-40
-20
0
20
40
60
80
100
Distance Fron Landside Toe, Feet
Hydraulic Gradient at Base of Blanket
Residual Head at Base of Blanket for
Section A-A shown on plan
Ground Surface

EXHIBIT A-15.9

Argentine Unit – Bulk Mail Pump Station Analysis

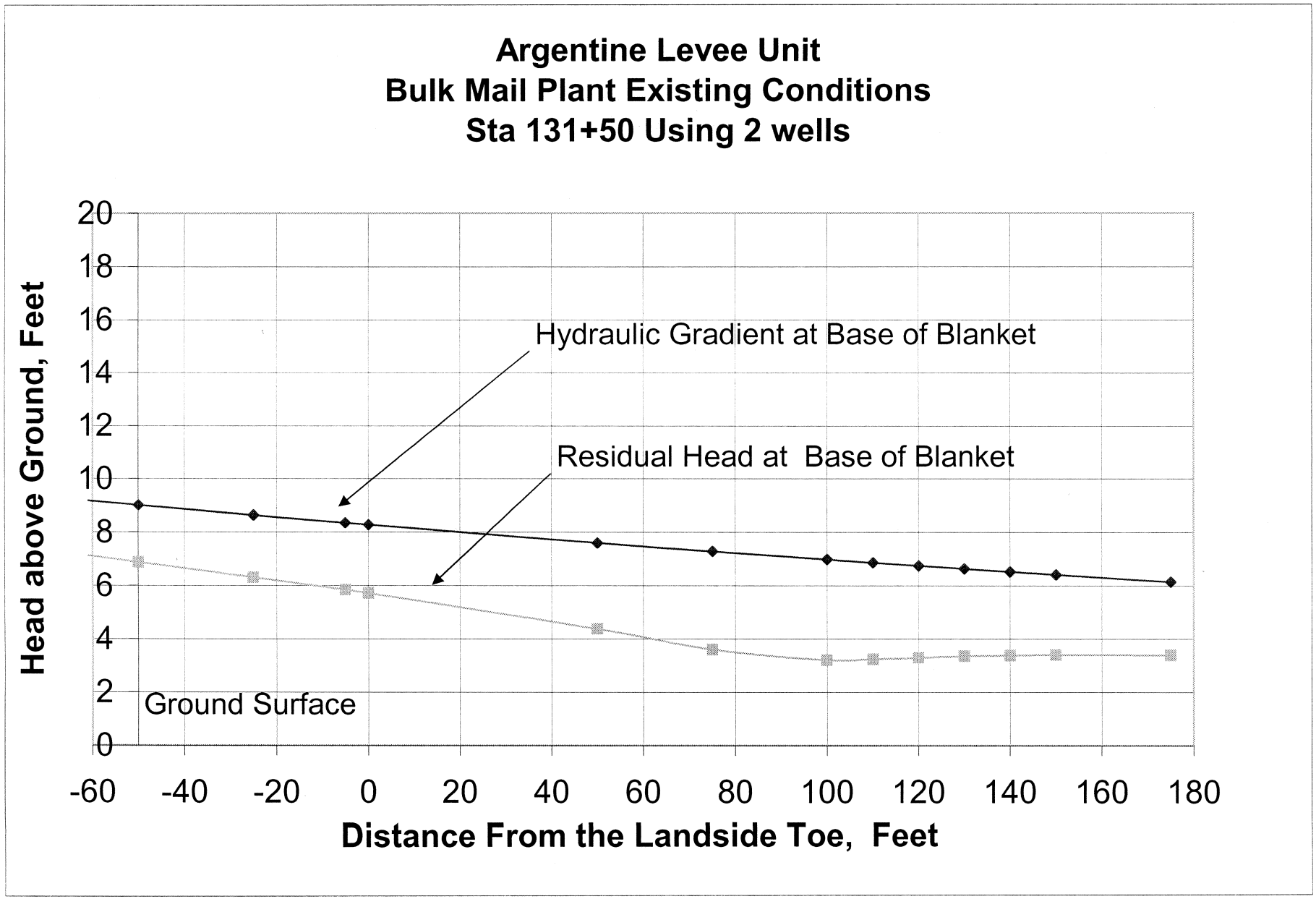
Bulk Mail Geotechnical Analysis and Design of Relief Wells

Existing Condition Conclusions:

For the existing Bulk Mail plant and anticipated water to the top of levee, 2 relief wells spaced 50 feet apart on the center of the pump plant, 10 feet landside of the pump plant, can reduce the gradient to below 4 feet above the top of ground. The Discharge of each well is anticipated to be 1 cfs and the invert elevation of the collector system must be kept 1.5 below existing top of ground or lower.

Future Conditions n500+0 Raise Conclusions:

For the existing Bulk Mail plant and anticipated water to the top of levee after a raise to the n500+0 (1.92 feet), 2 relief wells spaced 50 feet apart on the center of the pump plant, 10 feet landside of the pump plant, can reduce the gradient to below 4 feet above the top of ground. The Discharge of each well is anticipated to be 1 cfs and the invert elevation of the collector system must be kept 1.75 below existing top of ground or lower.


Argentine Levee Unit
Bulk Mail Plant Existing Conditions
Sta 131+50 Using 2 wells
Head above Ground, Feet
20
18
16
14
12
10
8
6
4
2
0
Hydraulic Gradient at Base of Blanket
Residual Head at Base of Blanket
Ground Surface
-60
-40
-20
0
20
40
60
80
100
120
140
160
180
Distance From the Landside Toe, Feet

		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	929.346	929.346	929.346		
Example Calculation			r1-1=	166.9	166.9	166.9		
			DDw1=	0.78	0.78	0.78		
			well 2	x < 0	0 < x < wx	0 < x < wx		
			S=	547	547	547		
			r2-2 =	929.346	929.346	929.346		
			r1-2=	166.9	166.9	166.9		
			DDw2=	0.78	0.78	0.78		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	-75	0.78	-0.2	0.78	0.78	0.78	1	
		0.78	-0.2	0.78	0.78	0.78	2	
			PT1 Sum DD =	1.96	1.96	1.96		

		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	979.329	979.329	979.329		
Example Calculation			r1-1=	117.7	117.7	117.7		
			DDw1=	0.96	0.96	0.96		
			well 2	x < 0	0 < x < wx	0 < x < wx		
			S=	547	547	547		
			r2-2 =	979.329	979.329	979.329		
			r1-2=	117.7	117.7	117.7		
			DDw2=	0.96	0.96	0.96		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	-25	0.96	-0.2	0.96	0.96	0.96	1	
		0.96	-0.2	0.96	0.96	0.96	2	
			PT1 Sum DD =	2.32	2.32	2.32		

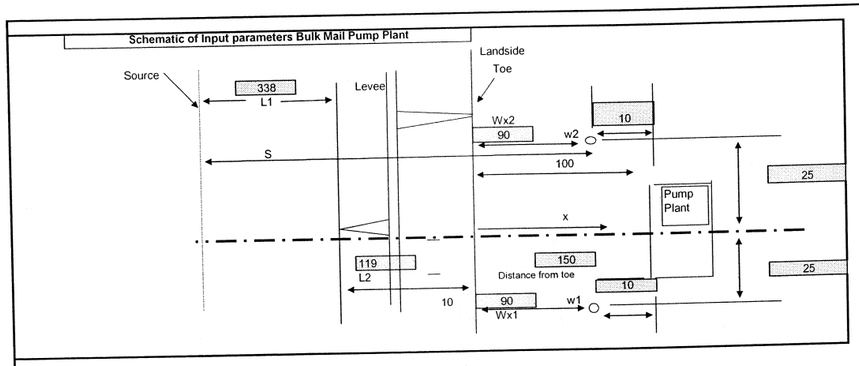
		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	1054.307	1054.307	1054.307		
Example Calculation			r1-1=	47.2	47.2	47.2		
			DDw1=	1.41	1.41	1.41		
			well 2	x < 0	0 < x < wx	0 < x < wx		
			S=	547	547	547		
			r2-2 =	1054.307	1054.307	1054.307		
			r1-2=	47.2	47.2	47.2		
			DDw2=	1.41	1.41	1.41		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	50	1.41	-0.2	1.41	1.41	1.41	1	
		1.41	-0.2	1.41	1.41	1.41	2	
			PT1 Sum DD =	3.22	3.22	3.22		

		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	1094.296	1094.296	1094.296		
Example Calculation			r1-1=	25.0	25.0	25.0		
			DDw1=	1.71	1.71	1.71		
			well 2	x < 0	0 < x < wx	0 < x < wx		
			S=	547	547	547		
			r2-2 =	1094.296	1094.296	1094.296		
			r1-2=	25.0	25.0	25.0		
			DDw2=	1.71	1.71	1.71		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	90	1.71	-0.2	1.71	1.71	1.71	1	
		1.71	-0.2	1.71	1.71	1.71	2	
			PT1 Sum DD =	3.83	3.83	3.83		

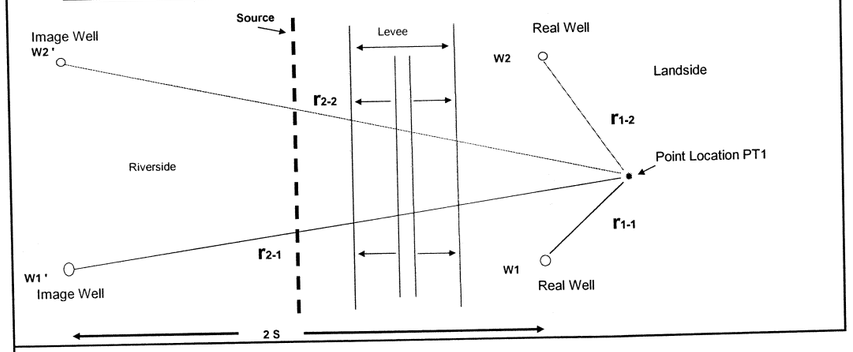
		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	1114.291	1114.291	1114.291		
Example Calculation			r1-1=	32.0	32.0	32.0		
			DDw1=	1.61	1.61	1.61		
			well 2	x < 0	0 < x < wx	0 < x < wx		
			S=	547	547	547		
			r2-2 =	1114.291	1114.291	1114.291		
			r1-2=	32.0	32.0	32.0		
			DDw2=	1.61	1.61	1.61		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	110	1.61	-0.2	1.61	1.61	1.61	1	
		1.61	-0.2	1.61	1.61	1.61	2	
			PT1 Sum DD =	3.62	3.62	3.62		

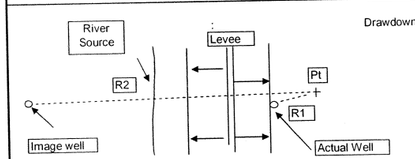
		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	1154.281	1154.281	1154.281		
Example Calculation			r1-1=	65.0	65.0	65.0		
			DDw1=	1.30	1.30	1.30		
			well 2	x < 0	0 < x < wx	0 < x < wx		
			S=	547	547	547		
			r2-2 =	1154.281	1154.281	1154.281		
			r1-2=	65.0	65.0	65.0		
			DDw2=	1.30	1.30	1.30		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	150	1.30	-0.2	1.30	1.30	1.30	1	
		1.30	-0.2	1.30	1.30	1.30	2	
			PT1 Sum DD =	3.01	3.01	3.01		

Schematic of Input parameters Bulk Mail Pump Plant



Schematic of Input parameters





Drawdown = (Q/(2*pi*k*Df)) ln (R2/R1)

Well Design Calculations

Loehr EC-GD 5/4/2005

Well Drawdown Influence Table

Bulk Mail - Argentine KS: Station 131+50

ic =	0.84	Well losses	1.3	(entrance, velocity bends)		
Ground = 0		Disch Elev Loss	-1.5	Dbl=	12.8	Feet
Location of points at the base of the blanket perpendicular to the center well		DD at Toe	2.55	Qw =	1	cfs
		X_L	588	k =	0.0052	ft /sec
		h0	8.27	Df =	67.5	feet

Distance x	Head hx	x < 0 Rvsd of Ldsd Toe	0 < x < Wx Between Toe and Well	x > Wx Ldsd of Well	DDwx Feet	Residual Head Feet	Maximum Target Head
-75	9.40	-75			1.96	7.44	
-50	9.00	-50			2.13	6.87	
-25	8.63	-25			2.32	6.31	
-5	8.34	-5			2.5	5.84	
0	8.27	0			2.55	5.72	
50	7.60		50.00		3.22	4.38	
75	7.28		75.00		3.68	3.60	
100	6.98			100	3.77	3.21	4.00
110	6.86			110	3.62	3.24	4.00
120	6.74			120	3.45	3.29	4.00
130	6.63			130	3.28	3.35	4.00
140	6.52			140	3.14	3.38	
150	6.41			150	3.01	3.40	
175	6.14			175	2.75	3.39	

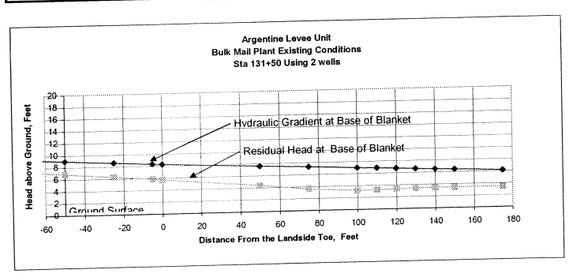
Designed By:	Loehr 05/04/2005
Checked By:	Jones 06/15/2005

Kf/Kb =	400
LR =	385 feet

Example Calculation

PT1 data	well 1	x < 0	0 < x < wx	x > wx
	S =	547	547	547
	r2-1=	1154.281	1154.281	1154.281
	r1-1=	65.0	65.0	65.0
	DDw1=	1.30	1.30	1.30
	well 2	x < 0	0 < x < wx	0 < x < wx
	S=	547	547	547
	r2-2 =	1154.281	1154.281	1154.281
	r1-2=	65.0	65.0	65.0
	DDw2=	1.30	1.30	1.30

Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well
150	1.30	-0.2	1.30	1.30	1.30	1
	1.30	-0.2	1.30	1.30	1.30	2
		PT1 Sum DD =	3.01	3.01	3.01	



The source, S, location is considered to be the location riverside of the flood protection that the full head is considered to be acting on the foundation sands below a finite blanket thickness. The source location is dependent on the blanket thickness, depth of foundation sands, permeability of the blanket and sand or exposure of river to the sands. S = L1+ L2 + Dist to well (landside).

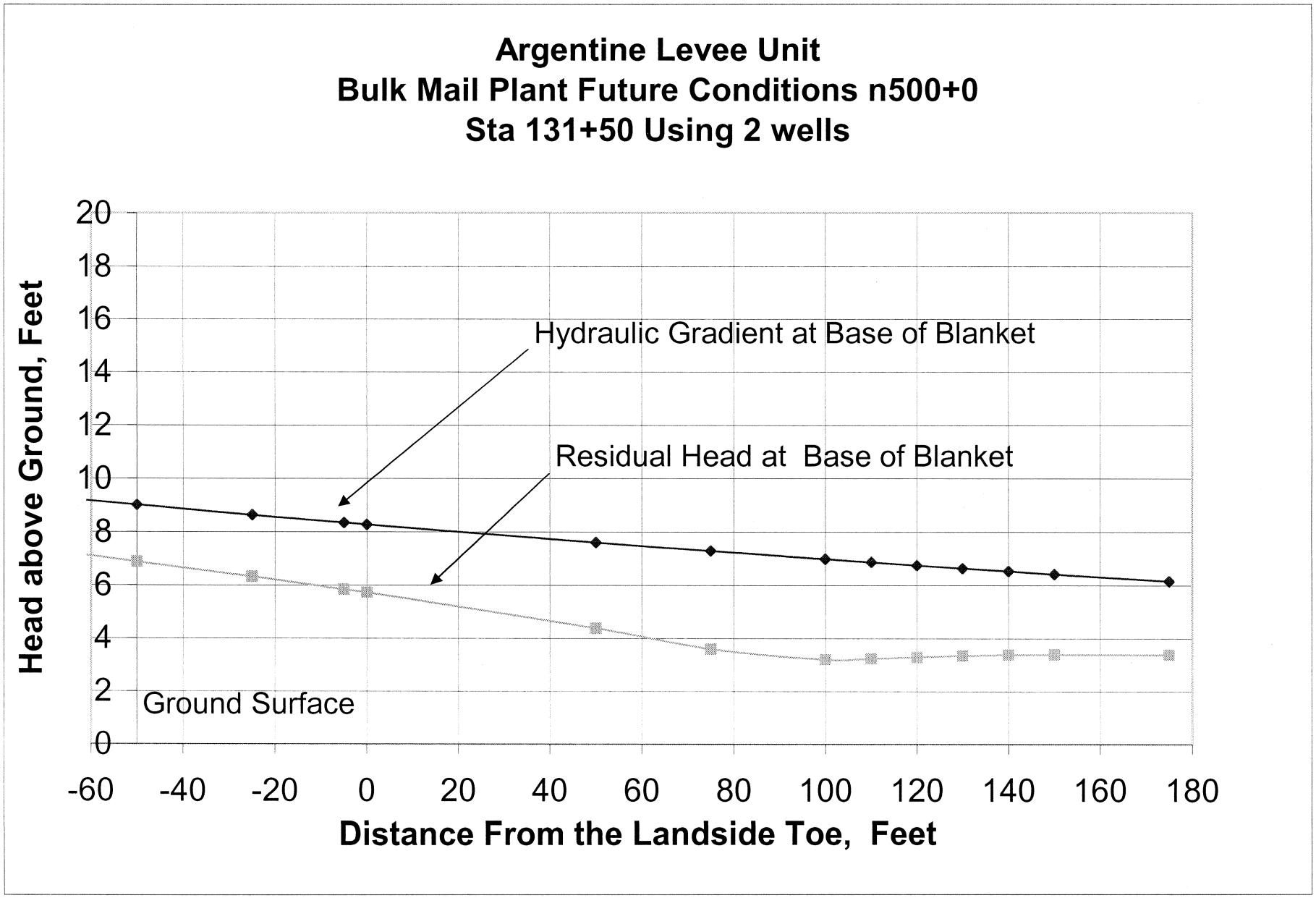
The image well is located at a distance equal to two source lengths from the real well.

Knowing the location of the image well and the real well allows the calculation of the expected drawdown of the hydraulic gradient at any point location being considered.

The drawdown is calculate using the equation : (Q/(2*pi*k*Df)) ln (R2/R1). Where Q = expected flow from the well dependent on the head of the flood event and well characteristics; k is the coefficient of permeability of the sand foundation; and Df is the depth of the foundation sand. R2 is the distance fomr the point to the image well

Definitions

gx :	Distance from the Landside toe used for plotting of the hydraulic head above ground, hx
hx :	Head above ground surface due to the river without wells considered
wx :	Distance from the Landside toe to the well use din the calculations
DDwx	The drawdown at the point location due to all of the wells.
Residual Head	The amount of head above the gorund surface at the point being considered after the drawdown of the well has been subtracted.
Well Losses	The net amount that the head of water will rise at the point being considered considering entrance losses, velocity losses and elevation head location for the outlet of the well. Note that the head is considerably affected by the outlet elevation.
xL :	The effective length of the seepage path due to blanket thickness, and permeability of the blanket and underlying foundaiton sands
Qw :	The expected discharge of the well based on expected head and seepage path of the system.
k:	The permeability assigned to the foundation sands.
Dbl :	The thickness of the foundaiton blanket
Df :	The thickness of the foundation sands.
L1	Calculated Distance that the river provides the full head to the foundation sand below the blanket where L1 = C * (e^(2*LR/C-1)/(e^(2*LR/C+1) and C = (Df * Dbl * Kf/Kb)^0.5 with LR equal to the distance to the river source.


Argentine Levee Unit
Bulk Mail Plant Future Conditions n500+0
Sta 131+50 Using 2 wells
Head above Ground, Feet
20
18
16
14
12
10
8
6
4
2
0
Hydraulic Gradient at Base of Blanket
Residual Head at Base of Blanket
Ground Surface
-60
-40
-20
0
20
40
60
80
100
120
140
160
180
Distance From the Landside Toe, Feet

		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	954.338	954.338	954.338		
Example Calculation			r1-1=	142.2	142.2	142.2		
			DDw1=	0.86	0.86	0.86		
			well 2	x < 0	0 < x < wx	x wx		
			S=	547	547	547		
			r2-2 =	954.338	954.338	954.338		
			r1-2=	142.2	142.2	142.2		
			DDw2=	0.86	0.86	0.86		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	-50	0.86	-0.45	0.86	0.86	0.86	1	
		0.86	-0.45	0.86	0.86	0.86	2	
			PT1 Sum DD =	2.63	2.63	2.63		

		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	999.323	999.323	999.323		
Example Calculation			r1-1=	98.2	98.2	98.2		
			DDw1=	1.05	1.05	1.05		
			well 2	x < 0	0 < x < wx	x wx		
			S=	547	547	547		
			r2-2 =	999.323	999.323	999.323		
			r1-2=	98.2	98.2	98.2		
			DDw2=	1.05	1.05	1.05		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	-5	1.05	-0.45	1.05	1.05	1.05	1	
		1.05	-0.45	1.05	1.05	1.05	2	
			PT1 Sum DD =	3.00	3.00	3.00		

		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	1079.300	1079.300	1079.300		
Example Calculation			r1-1=	29.2	29.2	29.2		
			DDw1=	1.64	1.64	1.64		
			well 2	x < 0	0 < x < wx	x wx		
			S=	547	547	547		
			r2-2 =	1079.300	1079.300	1079.300		
			r1-2=	29.2	29.2	29.2		
			DDw2=	1.64	1.64	1.64		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	75	1.64	-0.45	1.64	1.64	1.64	1	
		1.64	-0.45	1.64	1.64	1.64	2	
			PT1 Sum DD =	4.18	4.18	4.18		

		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	1094.296	1094.296	1094.296		
Example Calculation			r1-1=	25.0	25.0	25.0		
			DDw1=	1.71	1.71	1.71		
			well 2	x < 0	0 < x < wx	x wx		
			S=	547	547	547		
			r2-2 =	1094.296	1094.296	1094.296		
			r1-2=	25.0	25.0	25.0		
			DDw2=	1.71	1.71	1.71		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	90	1.71	-0.45	1.71	1.71	1.71	1	
		1.71	-0.45	1.71	1.71	1.71	2	
			PT1 Sum DD =	4.33	4.33	4.33		

		PT1						
		data	well 1	x < 0	0 < x < wx	x > wx		
			S =	547	547	547		
			r2-1=	1124.288	1124.288	1124.288		
Example Calculation			r1-1=	39.1	39.1	39.1		
			DDw1=	1.52	1.52	1.52		
			well 2	x < 0	0 < x < wx	x wx		
			S=	547	547	547		
			r2-2 =	1124.288	1124.288	1124.288		
			r1-2=	39.1	39.1	39.1		
			DDw2=	1.52	1.52	1.52		
	Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well	
	120	1.52	-0.45	1.52	1.52	1.52	1	
		1.52	-0.45	1.52	1.52	1.52	2	
			PT1 Sum DD =	3.95	3.95	3.95		

Hydraulic Grade Line for the Bulk Mail Location Station 131+50

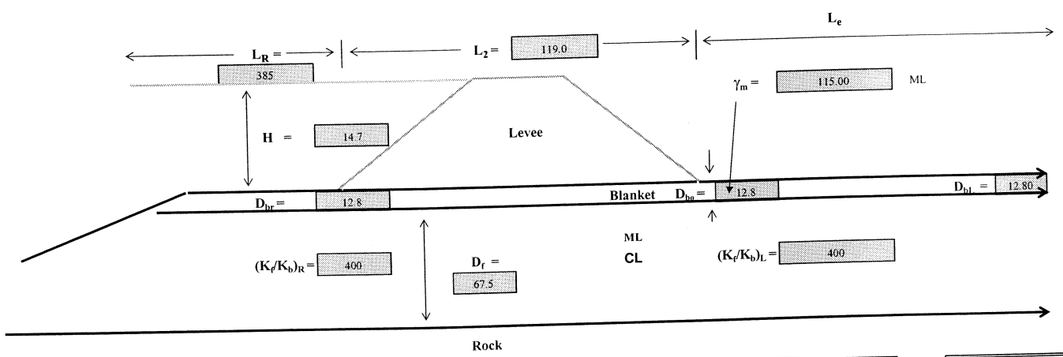
UNDERSEEPAGE ANALYSIS

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

Argentine Levee

Name : Loehr EC-GD
Date : 04/28/2005
Revised to reflect criteria developed for L-142

Revised and Checked by : Jones EC-GD 06/15/2005



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

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

NOMENCLATURE

$(K_f/K_b)_R$ = riverside permeability
$(K_f/K_b)_L$ = landside permeability
D_{br} = riverside blanket thickness

D_{bo} = levee toe blanket thickness
D_{bL} = landside blanket thickness
D_f = thickness of pervious foundation
L_R = length of riverside blanket
L_L = length of landside blanket
H = max head or levee height
$H(W_T)$ = head above tailwater at end of underseepage berm
$H(W_T) = H(W_T/2)*e^{(-WT/(2*CL))}$
$H(W_{T/2})$ = head above tailwater midpoint of underseepage berm
$= H * L_e / L'_t$
W_T = berm width
i_o = seepage gradient
C_r = riverside effective length coefficient
C_L = landside effective length coefficient
where $C = [(Kf/Kb) * Df * Db]^{1/2}$
H'o = head above tailwater at levee toe (w/ berm)
i_c = critical seepage gradient
L_1 = riverside effective length
where $L_1 = C * (e^{(2LR/C-1)}) / (e^{(2LR/C+1)})$
L_2 = levee base width
L_e = landside effective length
L_t = total effective length
L'_t = Total Effective Length + 1/2 of Berm

t' = required Berm thickness at toe
Sc = calc'd slope of underseepage berm
q = seepage /unit Length

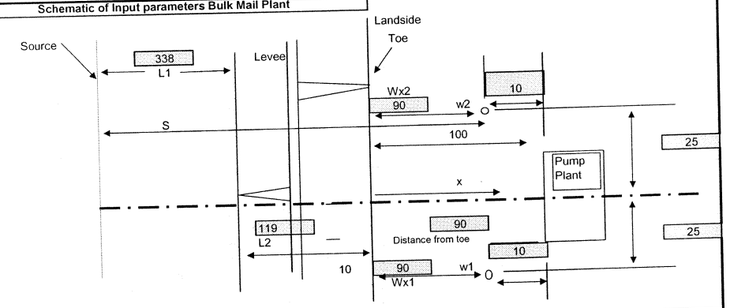
Q = cummulated seepage
$FS_i = i_c / i_o$

Station	Begin Station	End Station	$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL}	D_{bo}	D_{br}	D_f	L_R	H	Remarks
	Feet	Feet	First row is tied to above graphs! Change Graphics only!								
131+50	existing levee		400	400	12.8	12.8	12.8	67.5	385	14.7	Using 774.7-760
131+50	500yr Nominal		400	400	12.8	12.8	12.8	67.5	385	16.62	Using 776.62-760
131+50	500 yr +3		400	400	12.8	12.8	12.8	67.5	385	18.50	Using 778.5-760
131+50	500 yr + 5		400	400	12.8	12.8	12.8	67.5	385	20.50	Using 780.5-760

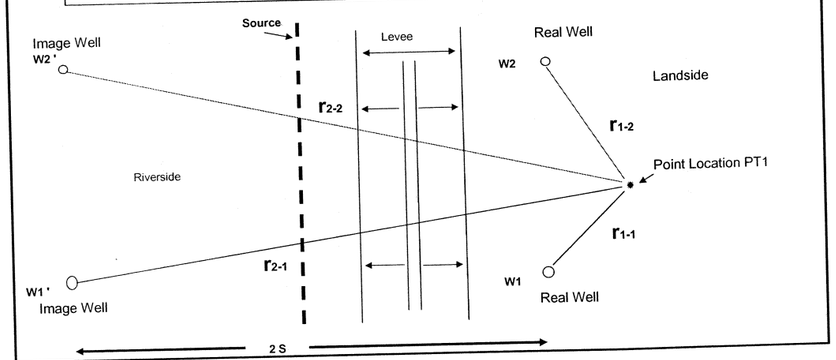
Analysis of Without Berm Conditions

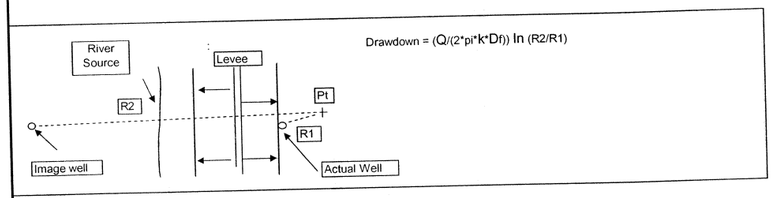
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	Check #1 Full Head Toe FS_i (need 1.1)	Check #1 Reduced Head Toe FS_i (need 1.5)	Remarks
588	588	338	119	588	1045	8.27	0.65	0.84	1.30	1.64	
588	588	338	119	588	1045	9.35	0.73	0.84	1.15	1.40	
588	588	338	119	588	1045	10.41	0.81	0.84	1.03	1.23	< 1.1 and < 1.5 Need control
588	588	338	119	588	1045	11.53	0.90	0.84	0.93	1.09	< 1.1 and < 1.5 Need control

Schematic of Input parameters Bulk Mail Plant



Schematic of Input parameters





Drawdown = (Q/(2*pi*k*Df)) ln (R2/R1)

Well Design Calculations

Loehr EC-GD 5/4/2005

Well Drawdown Influence Table

ill - Argentine KS: Station 131+50 n500+0 Future Cc

ic =	0.84	Well losses	1.3	(entrance, velocity bends)		
Ground = 0		Disch Elev Loss	-1.75	Dbl=	12.8	Feet
Location of points at the base of the blanket perpendicular to the center well		DD at Toe	3.05	Qw =	1	cfs
		X_L	588	k =	0.0052	ft /sec
		h0	9.35	Df =	67.5	feet

Distance x	Head hx	x < 0 Rvsd of Ldsd Toe	0 < x < Wx Between Toe and Well	x > Wx Ldsd of Well	DDwx Feet	Residual Head Feet	Maximum Target Head
-75	10.62	-75			2.46	8.16	
-50	10.18	-50			2.63	7.55	
-25	9.76	-25			2.82	6.94	
-5	9.43	-5			3	6.43	
0	9.35	0			3.05	6.30	
50	8.59		50.00		3.72	4.87	
75	8.23		75.00		4.18	4.05	
100	7.89			100	4.27	3.62	4.00
110	7.76			110	4.2	3.56	4.00
120	7.62			120	3.95	3.67	4.00
130	7.50			130	3.78	3.72	4.00
140	7.37			140	3.64	3.73	
150	7.24			150	3.51	3.73	
175	6.94			175	3.25	3.69	

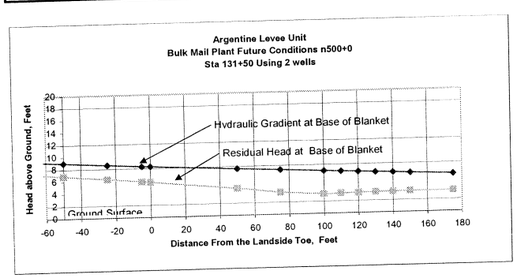
Designed By:	Loehr 05/04/2005
Checked By:	Jones 06/15/2005

Kf/Kb =	400
LR =	385 feet

Example Calculation

PT1 data				
well 1	x < 0	0 < x < wx	x > wx	
S =	547	547	547	
r2-1=	1094.296	1094.296	1094.296	
r1-1=	25.0	25.0	25.0	
DDw1=	1.71	1.71	1.71	
well 2	x < 0	0 < x < wx	x wx	
S=	547	547	547	
r2-2 =	1094.296	1094.296	1094.296	
r1-2=	25.0	25.0	25.0	
DDw2=	1.71	1.71	1.71	

Dist x	DDwx	Headlosses	x < 0	0 < x < wx	x > wx	Well
90	1.71	-0.45	1.71	1.71	1.71	1
	1.71	-0.45	1.71	1.71	1.71	2
PT1 Sum DD =			4.33	4.33	4.33	



The source, S, location is considered to be the location riverside of the flood protection that the full head is considered to be acting on the foundation sands below a finite blanket thickness. The source location is dependent on the blanket thickness, depth of foundation sands, permeability of the blanket and sand or exposure of river to the sands. S = L1+ L2 + Dist to well (landside).

The image well is located at a distance equal to two source lengths from the real well.

Knowing the location of the image well and the real well allows the calculation of the expected drawdown of the hydraulic gradient at any point location being considered.

The drawdown is calculate using the equation : (Q/(2*pi*k*Df)) ln (R2/R1). Where Q = expected flow from the well dependent on the head of the flood event and well characteristics; k is the coefficient of permeability of the sand foundation; and Df is the depth of the foundation sand. R2 is the distance fomr the point to the image well

Definitions

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k:	The permeability assigned to the foundation sands.
Dbl :	The thickness of the foundaiton blanket
Df :	The thickness of the foundation sands.
L1	Calculated Distance that the river provides the full head to the foundation sand below the blanket where L1 = C * (e^(2*LR/C-1)/(e^(2*LR/C+1) and C = (Df * Dbl * Kf/Kb)^0.5 with LR equal to the distance to the river source.

KANSAS RIVER LEVEES

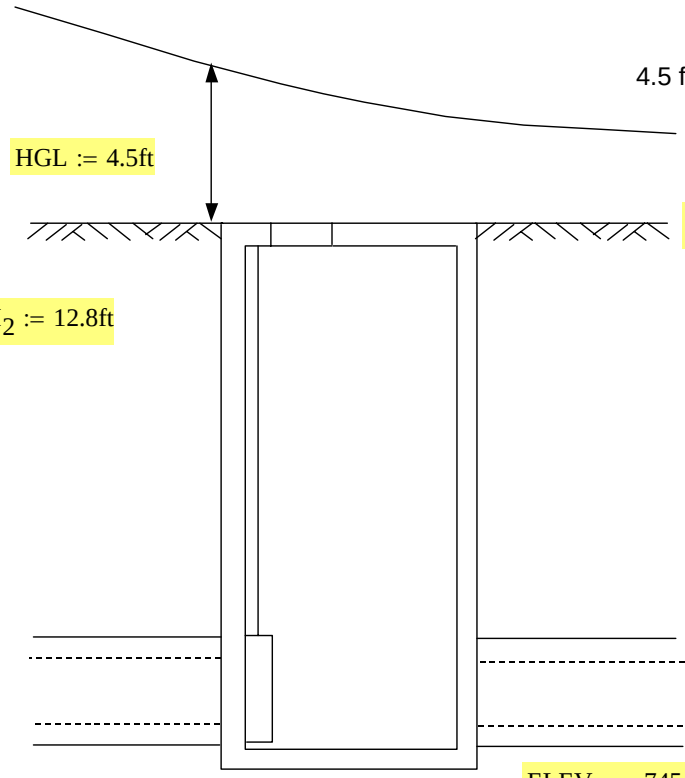
Extreme Conditions, Maximum Allowable Hydraulic Grade Line (~Existing Levee Height)

Variables

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

Properties

4.5 ft HGL is acceptable.



$HGL := 4.5ft$

Elevation $ELEV_1 := 763ft$

Blanket Thicknes $H_2 := 12.8ft$

Water head pressure in Wet Well under operating conditions.

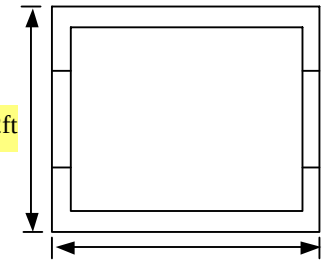
$HEAD := 0ft$

$ELEV_2 := 745.18ft$

$H := ELEV_1 - ELEV_2$

$H = 17.82\,ft$

Plan



$L_1 := 12ft$

$L_2 := 14.66ft$

Wall Thickness 1 $D_1 := 15in$

Clear Cover 1 $C_{c1} := 3in$

Floor Thickness $D_f := 18in$

Wall Thickness 2 $D_2 := 15in$

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

Clear Cover 2 $C_{c2} := 3in$

$l_2 := L_2 - 2 \cdot D_1$

$d_2 := D_2 - C_{c2}$

$d_1 := D_1 - C_{c1}$

$d_f := D_f - C_{cf}$

$l_1 := L_1 - 2 \cdot D_2$

$h := H - 2 \cdot D_f$

$H_1 := H + HGL$ $H_3 := H$

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. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi.
- The Concrete Reinforcing Steel Institute *Engineering Data Report Number 48* recommends using 33 ksi yield strengths for rebar of this time period. 60 KSI steel

Soil Properties $\phi := 28deg$

$\gamma := 120pcf$

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

$K_o = 0.531$

Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 60ksi$

Concrete Properties $f'_c := 4ksi$

Concrete Unit Weight $\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

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

Load Factors

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

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

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

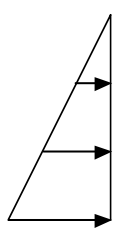
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALLS

Wall Loading

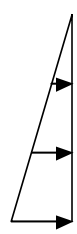
Soil or Soil & Water



$L_2 := H$

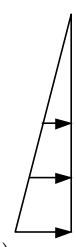
$L_2 = 17.82\,ft$

OR



$L_2 = 17.82\,ft$

&



$L'_3 := H_3 \cdot \left(\frac{H_1}{H_2}\right)$

$L_3 := \text{if}\left(H > H_2, H + HGL, L'_3\right)$

$L_3 = 22.32\,ft$

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

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

$W_1 = 1134 \frac{lb}{ft^2}$

$W_2 = 1937 \frac{lb}{ft^2}$

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

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

$W = 1937 \frac{lb}{ft^2}$

WALL 1

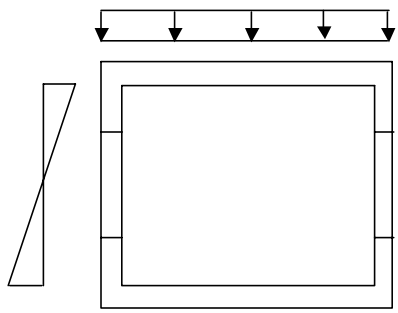
SHEAR

$$THRUST_1 := \frac{W \cdot L_2}{2} \qquad THRUST_1 = 17262 \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis

$$V'_{u1} := W \cdot \frac{l_1}{2}$$

$$V'_{u1} = 9202 \frac{lb}{ft}$$



$$V_{u1} := V'_{u1} \cdot \frac{\left(\frac{l_1}{2} - d_1\right)}{\frac{l_1}{2}} \qquad V_{u1} = 7265 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$\frac{l_1}{d_1} = 9.5$$

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

$$\phi V_{n1} := \phi_V \cdot \left[3.3 \cdot \left(\sqrt{f'_c}\right)\left(12in \cdot d_1\right) \cdot \frac{lb^{.5}}{in \cdot ft} + \frac{THRUST_1 \cdot d_1}{4 \cdot l_1}\right]$$

ACI 11.10.6 EQ (11-31)

$$\phi V_{n1} = 30509 \frac{lb}{ft}$$

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

Check1 = "OKAY"

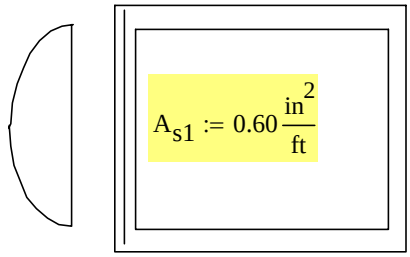
Factor of Safety $\quad FS_{S1} := \frac{\phi V_{n1}}{V_{u1}} \qquad FS_{S1} = 4.199$

Bending

NOTE: Discontinuous steel at corners. Elements act as simply supported beams with M=W*L^2/8 at midspan.

$$M_{u1} := \frac{W \cdot l_1^{\,2}}{8}$$

$$M_{u1} = 21855 \frac{ft \cdot lb}{ft}$$



$$A_{s1} := 0.60 \frac{in^2}{ft}$$

#7 Bars @ 12 in. Steel

$$b := 12 \frac{in}{ft} \qquad a := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a = 0.882\,in$$

$$\phi M_{n1} := \phi_B \cdot A_{s1} \cdot F_y \cdot \left(d_1 - \frac{a}{2}\right)$$

$$\phi M_{n1} = 34.7 \frac{kip \cdot ft}{ft}$$

A 1.5 existing conditions factor of safety is considered acceptable and will not require risk analysis.

$$Check2 := if\left(\phi M_{n1} > 1.5 M_{u1}, "OKAY", "NO GOOD"\right)$$

Check2 = "OKAY"

Factor of Safety $\quad FS_{B1} := \frac{\phi M_{n1}}{M_{u1}} \qquad FS_{B1} = 1.587$

Controlling Factor of Safety

$$FS_1 := \min(FS_{S1}, FS_{B1})$$

Controlling Mechanism

$$Comment1 := \text{if}\left(FS_{B1} > FS_{S1}, \text{"Wall 1 Shear"}, \text{"Wall 1 Flexural Steel"}\right)$$

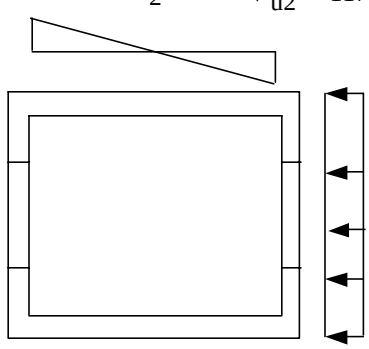
Factor of Safety $FS_1 = 1.587$

Controlling Mechanism $Comment1 = \text{"Wall 1 Flexural Steel"}$

WALL 2

SHEAR

$$V'_{u2} := W \cdot \frac{l_2}{2} \qquad V'_{u2} = 11779 \frac{lb}{ft}$$



$$THRUST_2 := \frac{W \cdot L_1}{2}$$

$$THRUST_2 = 11624 \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis

$$V_{u2} := V'_{u2} \cdot \frac{\left(\frac{l_2}{2} - d_2\right)}{\frac{l_2}{2}} \qquad V_{u2} = 9842 \frac{lb}{ft}$$

Shear at Distance d from Support
CI 318R-11.1.3.1

$$\frac{l_2}{d_2} = 12.16$$

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

$$\phi V_{n2} := \phi_V \cdot \left[3.3 \cdot \left(\sqrt{f'_c}\right)\left(12in \cdot d_2\right) \cdot \frac{lb^{.5}}{in \cdot ft} + \frac{THRUST_2 \cdot d_2}{4 \cdot l_2}\right]$$

ACI 11.10.6 EQ (11-31)

$$\phi V_{n2} = 30293 \frac{lb}{ft}$$

$$Check3 := \text{if}\left(\phi V_{n2} > 1.5 V_{u2}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

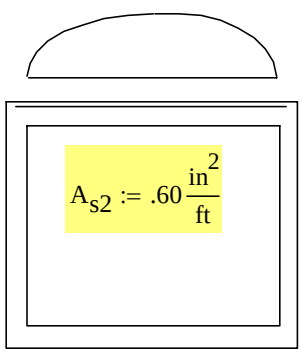
$Check3 = \text{"OKAY"}$

Factor of Safety $FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$ $FS_{S2} = 3.078$

Bending

NOTE: First assume elements act as simply supported beams with M=W*L^2/8 at midspan.

$$M_{u2} := \frac{W \cdot l_2^2}{8} \qquad M_{u2} = 35808 \frac{ft \cdot lb}{ft}$$



$$A_{s2} := .60 \frac{in^2}{ft}$$

#7 Bars @ 12 in. Steel

$$b := 12 \frac{in}{ft} \qquad a := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a = 0.882 in$$

$$d_2 = 12 in$$

$$\phi M_{n2} := \phi_B \cdot A_{s2} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right)$$

$$\phi M_{n2} = 34.7 \frac{kip \cdot ft}{ft}$$

Check4 A 1.5 existing conditions factor of safety is considered acceptable and will not require risk analysis.

$Check4 := if\left(\phi M_{n2} > 1.5M_{u2}, "OKAY", "NO GOOD"\right)$

$Check4 = "NO GOOD"$

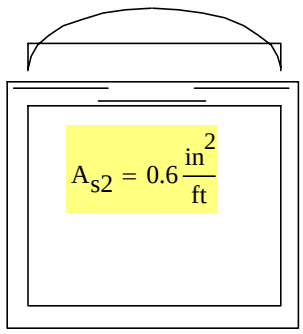
Factor of Safety $FS_{B2} := \frac{\phi M_{n2}}{M_{u2}}$ $FS_{B2} = 0.968$

NOTE: Continuity of Reinforcement at corners. If the structure acts as a frame (fixed ends for elements) the moments will be M=W*L^2/12 and M=W*L^2/24. Otherwise the elements act as simply supported beams with M=W*L^2/8 at midspan.

$M_{uMIN2} := \frac{W_1 \cdot l_2^2}{24}$ $M_{uMIN2} = 7\frac{kip \cdot ft}{ft}$

$M_{uMAX2} := \frac{W \cdot l_2^2}{12}$ $M_{uMIN2} = 7\frac{kip \cdot ft}{ft}$

$A_{sMAX2} := 0.79\frac{in^2}{ft}$



$A_{s2} = 0.6\frac{in^2}{ft}$

8's @ 12" Continuious Corner Steel
7's @ 12" Interior

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

$a := \frac{A_{sMAX2} \cdot F_y}{0.85f'_c \cdot b}$ $a = 1.162\,in$

$\phi M_{nMAX2} := \phi_B \cdot A_{sMAX2} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right)$

$\phi M_{nMAX2} = 45.1\frac{kip \cdot ft}{ft}$

Check5 and Check6 determine if the wall acts as a fixed end member.

Check5 determines if the negative steel is capable of carrying the fixed end moment.

$Check5 := if\left(\phi M_{nMAX2} > M_{uMAX2}, "OKAY", "NO GOOD"\right)$

$Check5 = "OKAY"$

Factor of Safety $FS'_{B2} := \frac{\phi M_{nMAX2}}{M_{uMAX2}}$ $FS'_{B2} = 1.889$

$a := \frac{A_{s2} \cdot F_y}{0.85f'_c \cdot b}$ $a = 0.882\,in$

$\phi M_{nMIN2} := \phi_B \cdot A_{s2} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right)$

$\phi M_{nMIN2} = 34.7\frac{kip \cdot ft}{ft}$

Check6 determines if the positive steel is capable of carrying the center span moment with fixed ends.

$Check6 := if\left(\phi M_{nMIN2} > M_{uMIN2}, "OKAY", "NO GOOD"\right)$

$Check6 = "OKAY"$

Factor of Safety $FS''_{B2} := \frac{\phi M_{nMIN2}}{M_{uMIN2}}$ $FS''_{B2} = 4.961$

NOTE: Checks performed to determine if there is a higher Factor of Safety for the pinned condition or

fixed end condition.

Bending Controlling Factor of Safety

$FS_{B2} := \max\left(FS_{B2}, \min\left(FS'_{B2}, FS''_{B2}\right)\right)$ $\quad FS_{B2} = 1.889$

Shear

$FS_S := FS_{S2}$ $\quad FS_S = 3.078$

Bending

$FS_B := FS_{B2}$ $\quad FS_B = 1.889$

Controlling Factor of Safety

$FS_2 := \min\left(FS_S, FS_B\right)$

Controlling Mechanism

$Comment2 := \text{if}\left(FS_B > FS_S, \text{"Wall 2 Shear"}, \text{"Wall 2 Flexural Steel"}\right)$

Factor of Safety $\quad FS_2 = 1.889$

Controlling Mechanism $\quad Comment2 = \text{"Wall 2 Flexural Steel"}$

Controlling Factor of Safety

$FoS := \min\left(FS_{S1}, FS_{B1}, FS_{S2}, FS_{B2}\right)$ $\quad FoS = 1.587$

Controlling Mechanism

$Mechanism := \text{if}\left(FS_2 > FoS, Comment1, Comment2\right)$ $\quad Mechanism = \text{"Wall 1 Flexural Steel"}$

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 of the more detailed load analysis using combined load distribution factors (initial load and levee raise) and relative certainty of existing steel it has been decided that a Factor of Safety of 1.5 will be acceptable.

Uplift Calculations

If the Factor of Safety of greater then 1.3 is required for usual flood events (10 year), and greater then 1.1 is required for extreme flood events (Top of Protection).

Weight of Structure

$W_{Structure} := \left(L_1 \cdot L_2 \cdot H - l_1 \cdot l_2 \cdot h\right) \cdot \gamma_c$ $\qquad W_{Structure} = 314.793\,kip$

Weight of Water in Structure

Assume no water in structure. $\qquad W_{Water} := 0kip$

Surcharge

$Surcharge := 50psf \cdot L_1 \cdot L_2 + 2000lb \cdot 2$ $\qquad Surcharge = 14692\,lb$

Uplift

$Uplift := L_1 \cdot L_2 \cdot L_3 \cdot \gamma_w$ $\qquad Uplift = 297.83\,kip$

Weight of Surcharge Water Above Top Surface

No surcharge water. $\qquad W_{SurWater} := 0lb$

$$SF_{UPLIFT} := \frac{W_{Structure} + W_{Water} + Surcharge}{Uplift - W_{SurWater}} \qquad SF_{UPLIFT} = 1.106$$

$Result_{Uplift} := if\left(SF_{UPLIFT} > 1.1, \text{"Uplift Acceptable"}, \text{"Uplift Concerns"}\right)$

$Result_{Uplift} = \text{"Uplift Acceptable"}$

EXHIBIT A-15.10

Argentine Unit – ConAgra Pump Station Plans

A. STORM WATER DRAINAGE:

1. The present area runoff is served by existing 36" diameter sewer, starting approximately 40' outside northeast property line, and runs under dike to river. Sewer is controlled by U.S.C.O. ENG. GAUGE NO. 2 valve.

2. Existing sewer starts on entering side with concrete headwall, invert elevation 750.1 (U.S. Datum), and is terminated with existing outlet headwall, invert elevation ±742.5'. Total length of 36" sewer is approximately 225'.

3. Existing sewer will be extended to property line, entering headwall will be connected with new pumphouse to accomodate two 4,000 GPM, 40 HP, 32' HD. storm water pumps.

4. In normal operation storm water will flow under gravity through 36" diameter sewer to the river.

5. When river water level will rise, and the U.S.C.O. ENG. will close gate No. 2 the storm water will be pumped out by two 4,000 GPM each pumps through 14" discharge pipe to GAUGE NO. 2 well. Excess of run off will build up to provided detention basin.

B. STORM WATER PUMPS P-36 and P-37

1. Provide (2) single stage sewage non-clog vertical pumps, complete with base, pump casing, brackets, flexible drive shaft, with all necessary couplings and universal connector, electric motor and mounting base. Each pump shall have 4,000 GPM capacity at 32 ft. head, 40 HP motor, 875 RPM, 460 volts, 60 Hertz, 3 Phase. Pump size shall be 10 x 10 x 15, with enclosed 15" Diameter impeller. Pumps shall have double thrust bearings, stainless steel shaft and automatic stuffing box grease seal lubricator. Pumps shall be as manufactured by Aurora Pumps, Model No. 612, Worthington Type KMS and Economy Pump Corp. are acceptable manufacturers.

C. PUMP CONTROLS

1. Provide pedestal mounted float switches and automatic electric alternator to control duplex pump operation; cuts in first one pump then the other and both pumps if load requires. Electrical contractor shall provide magnetic starters and disconnects.

D. PIPING MATERIALS

1. A. PUMP DISCHARGE PIPING SHALL BE DUCTILE CAST IRON PIPE, MECHANICAL JOINT, WITH COAL TAR VARNISH OUTSIDE AND INSIDE, TO MEET ANSI A21.51 (AWWA C151) AND FEDERAL SPEC. WW-P-421 C FOR TYPE III PIPE.
B. MECHANICAL JOINT ELBOWS SHALL BE USED TO CONNECT PIPE AND FOLLOW EXACT LEVEE SLOPE, AS SHOWN ON PROFILE 4/M4

2. INSIDE PUMP HOUSE FITTINGS SHALL BE CLASS 125 C.I. TO MEET ANSI A21.10 (AWWA C110) WITH COAL TAR VARNISH OUTSIDE AND INSIDE.

3. 36"ϕ RCP (REINF. CONC. PIPE) SHALL CONFORM TO ASTM C76-78 CLASS IV REQUIREMENTS. JOINTS SHALL CONFORM TO ASTM C 361. RUBBER AND STEEL WATERTIGHT.

GENERAL NOTES

1. ALL WORK SHALL BE IN ACCORDANCE WITH PLANS OF ENGINEERS LATEST MANUAL, EM 110-2-2102 AND EM 110-2-2102, AND SPECIFICATIONS.

2. AT CONNECTION ON NEW PIPES TO EXISTING STRUCTURES, AT LEAST 6" OF CONCRETE SHOULD BE REMOVED AROUND THE WALL OPENINGS. THE EXISTG. REINFMT. SHOULD BE LEFT IN PLACE AND USED IN THE NEW WORK. AN EXPANSION TYPE CONCRETE WITH A MIN. COMPRESSIVE STRENGTH OF 3000 PSI @ 28 DAYS SHOULD BE PLACED IN THE WALL OPENINGS AROUND THE NEW PIPES

3A. FOR PIPE EXCAVATION AND BACKFILL. SEE SPEC. SECTION 15100 SITE UTILITIES.
B. PERVIOUS BACKFILL SHOULD NOT BE ALLOWED WITHIN THE IMPERVIOUS ZONE OF THE EXISTING NATURAL BLANKET.
C. IMPERVIOUS MAT'L (I.E. >50% PASSING #200 SIEVE) SHOULD BE USED FOR BACKFILL OR THE EXCAVATED MAT'LS. MAY BE SELECTIVELY STOCKPILED AND REPLACED IN THE BACKFILL AS NECESSARY TO MAINTAIN THE EXIST. ENDTH. ZONING.
D. BEFORE CONNECTING 36"ϕ RCP TO EXIST. SYSTEM, SYSTEM SHALL BE CLEANED AND ALL DEBRIS REMOVED.

NO	DATE	REVISION RECORD	BY	APPD
5	12-30-92	ADDED WASTEWATER ADDITION	RR	
4	4-25-80	REVISED PUMP DISCHARGE PIPE	BP	EJZ
3	9-25-79	LOCATED 14" PUMP DISCH PIPE	TCS	
2	9-7-79	RELOCATED 14" DISCHARGE PIPE	EGK	
1	8-27-79	ADDEND DWG #1	TCS	EJZ
0	8-16-79	FOR CONSTRUCTION		

Globe Engineering Company
222 N. Dearborn Street
Chicago, Illinois 60601
(312) 431-6800

DESIGNED: T.C. SZTABA
DRAWN: B.M. BONDOC
CHECKED: T.C. SZTABA
APPROVED:

NORTH

LATEST MANUFACTURER'S DRAWINGS, EM 1110-2-2102, AND SHOP PLAN FOR PUMPING STATION PERTAIN TO.

2. AT CONNECTION ON NEW PIPES TO EXISTING STRUCTURES, AT LEAST 6" OF CONCRETE SHOULD BE REMOVED AROUND THE WALL OPENINGS. THE EXIST'G. REINFMT. SHOULD BE LEFT IN PLACE AND USED IN THE NEW WORK. AN EXPANSION TYPE CONCRETE WITH A MIN. COMPRESSIVE STRENGTH OF 3000 PSI @ 28 DAYS SHOULD BE PLACED IN THE WALL OPENINGS AROUND THE NEW PIPES

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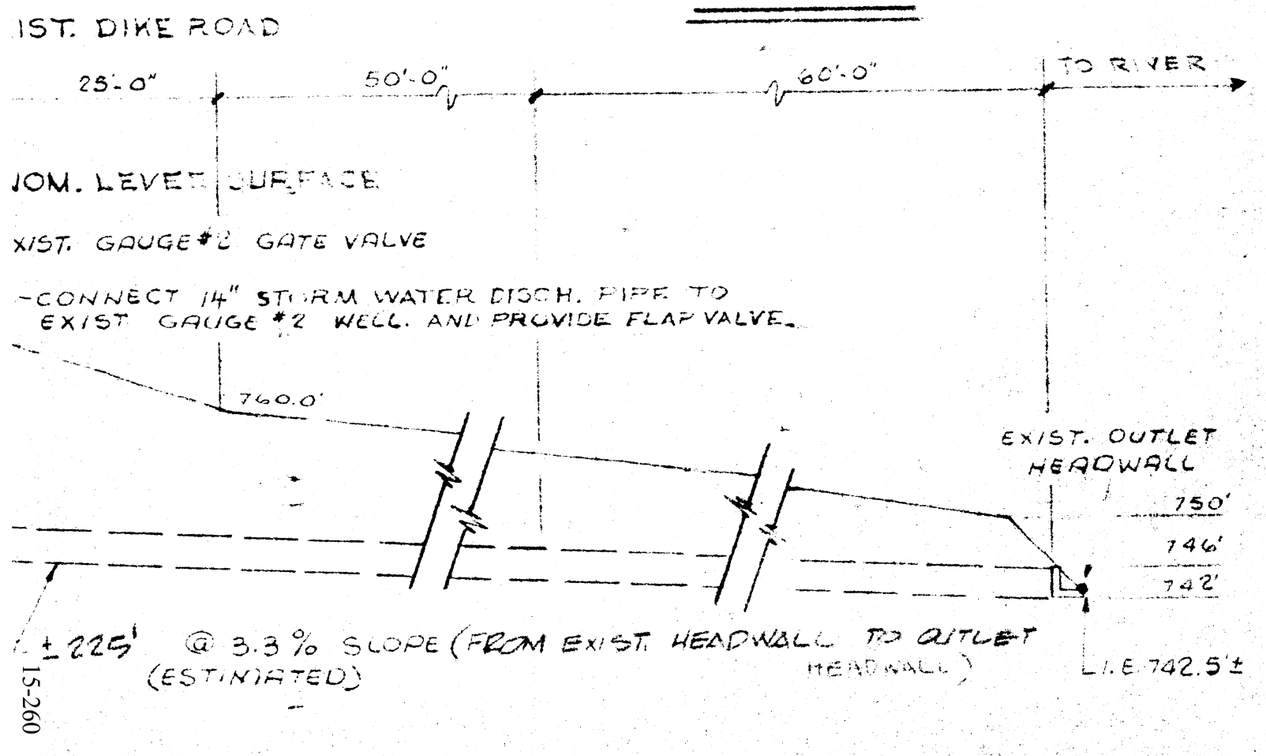
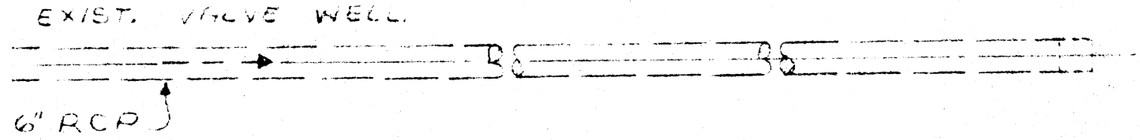
D. BEFORE CONNECTING 30" & RCP TO EXIST SYSTEM, SYSTEM SHALL BE CLEANED AND ALL DEBRIS REMOVED.

RIVERSIDE

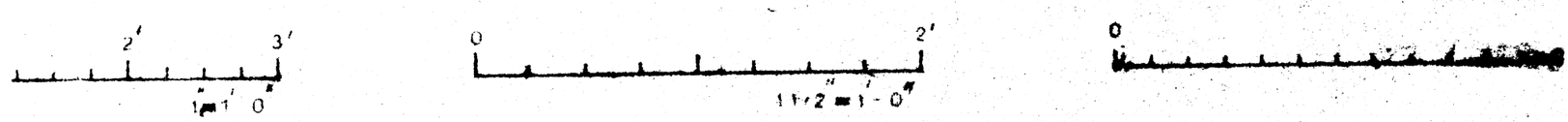


15-260

SEWER PROFILE



ORM SEWER



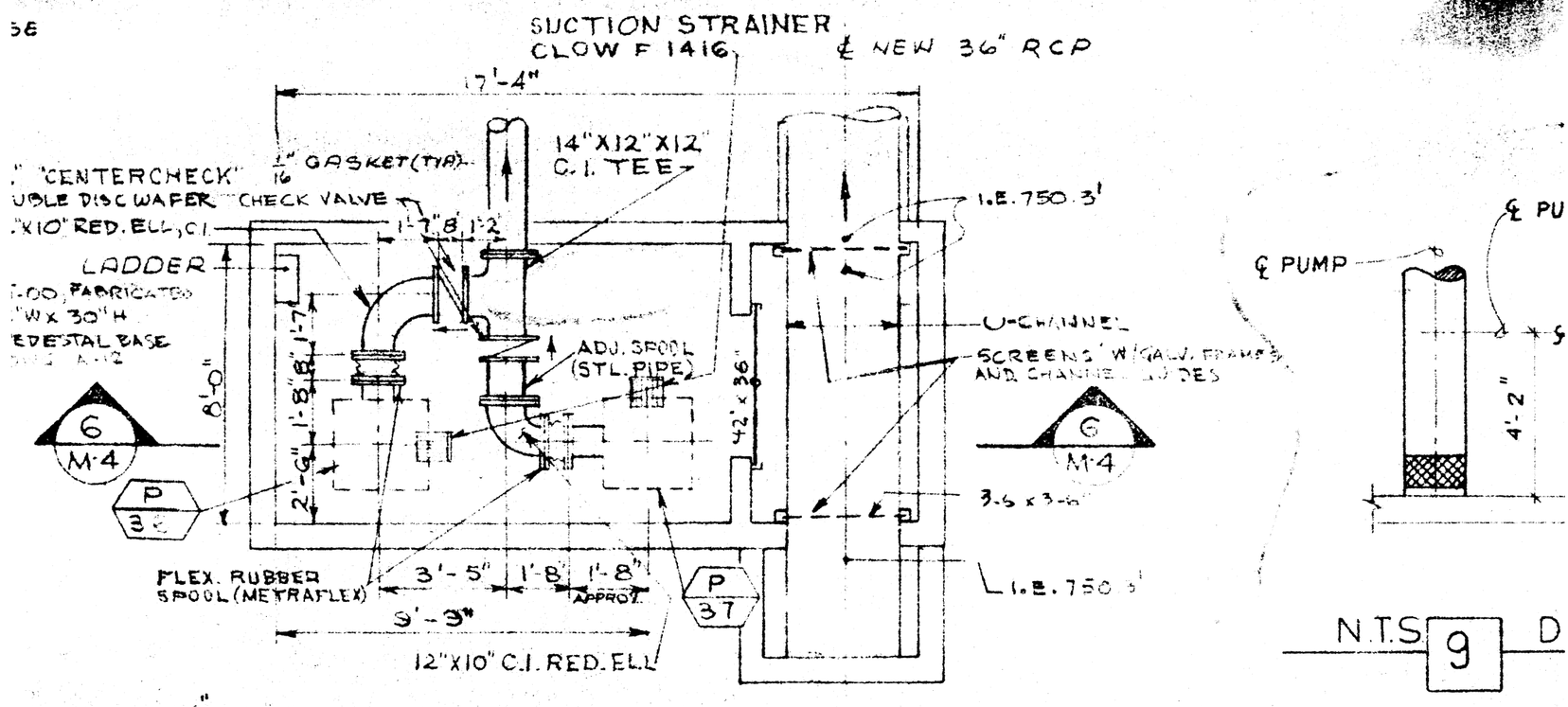
NORTH

FINISH GRADE & STORM WATER
SITE PLAN

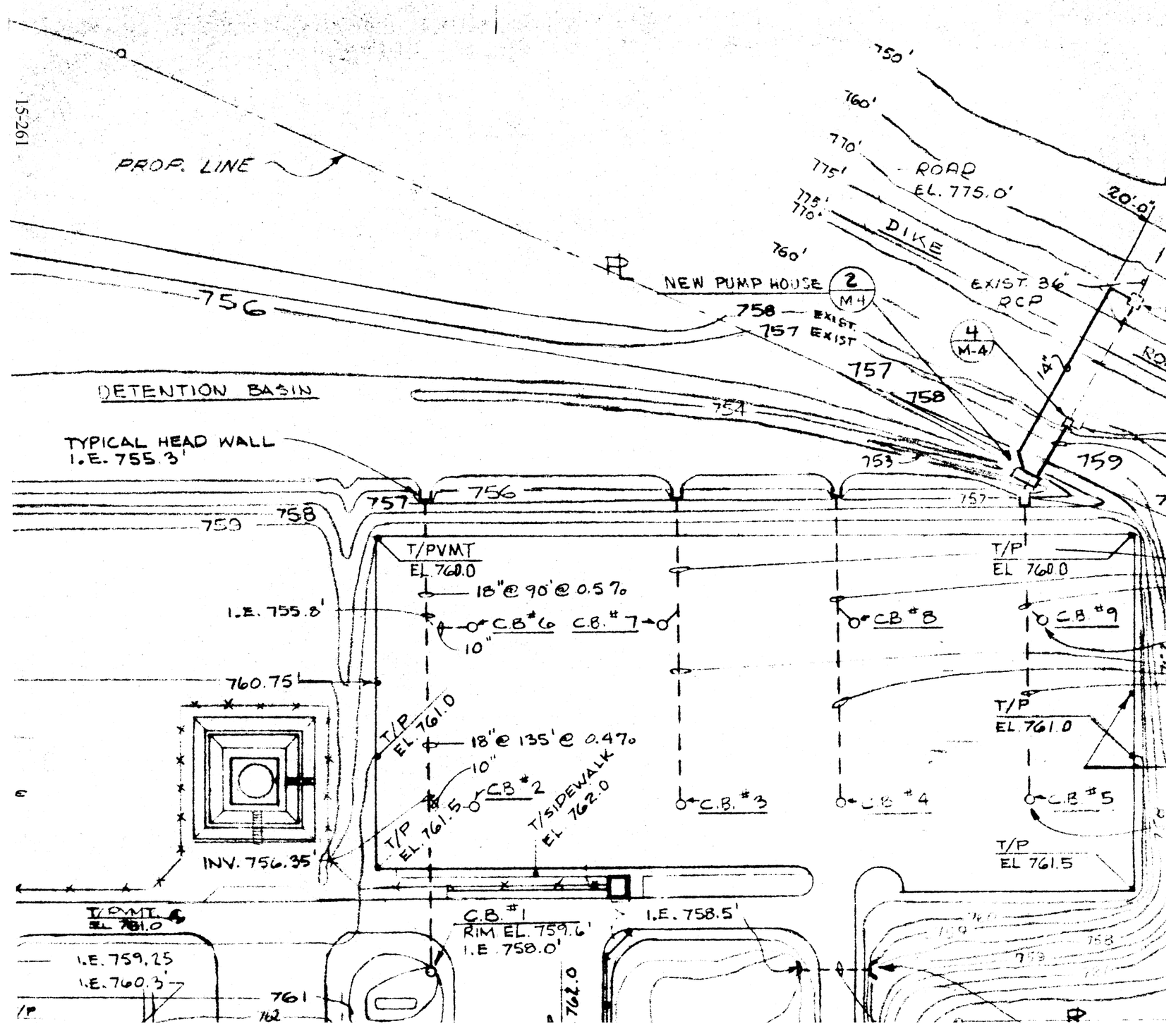
MIDWEST BREAKFAST STRIP PLANT
TURNER INDUSTRIAL DISTRICT
SPEAKER ROAD PROJECT
KANSAS CITY, KANSAS

PROJECT NO. S0996
DWG. NO. 504

M-4
OF SHEETS



1/4" = 1'-0" 2 PLAN - STORM WATER PUMPHOUSE
PUMP LEVEL



15-261

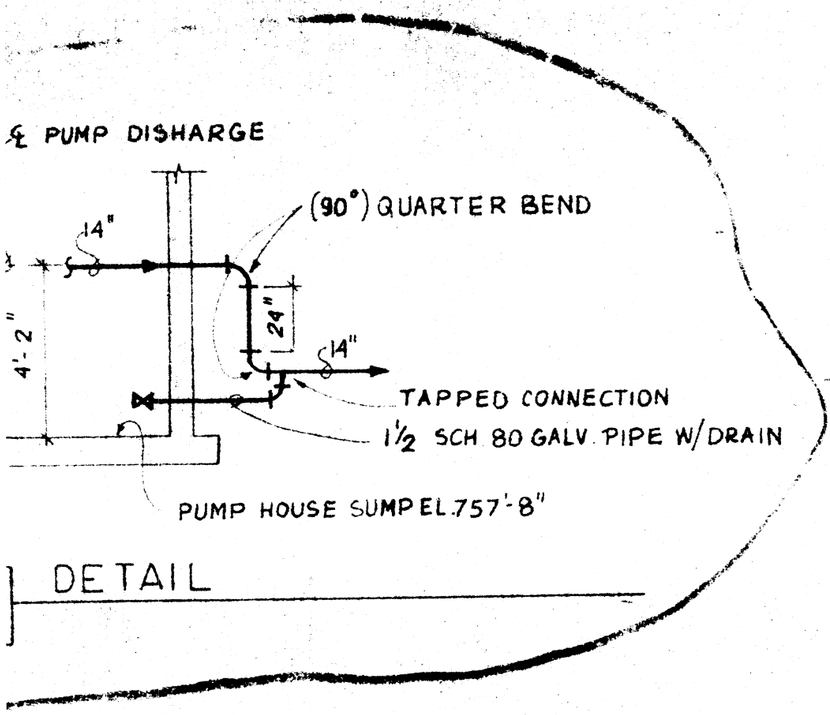
760.75
T/P EL 761.0
18" @ 135' @ 0.47%
10"
C.B. #2
T/P EL 761.5
T/SIDEWALK EL 762.0
INV. 756.35'
C.B. #3
C.B. #4
C.B. #5
T/P EL 761.0
T/P EL 761.5
T/PVMT. EL 761.0
C.B. #1
RIM EL. 759.6'
I.E. 758.0'
I.E. 758.5'
I.E. 759.25
I.E. 760.3'
761
762
T/P EL 762.0
T/CONC. EL 763.0
760
764
I.E. 758.2
18 Φ CULVERT - 40
763
765
NSPOUTS
INV EL 762.0
BLDG FIN FL EL 767.0'
T/CONC & GRATE EL 762.75
INV. EL 761.75
MUD BASIN
TYP. DOWNSPOUTS
T/PVMT. EL 762.5
RATE & CONC. 762.75
T. RAMP T/SIDEWALK EL 763.0
BUILDING ADDITION
OFFICE F.F.L.EL. 767.0'
ROOF DRAIN OUTLET
T/PVMT. EL 762.5
15-262
T/PVMT. EL 763.4
I.E. 760.7

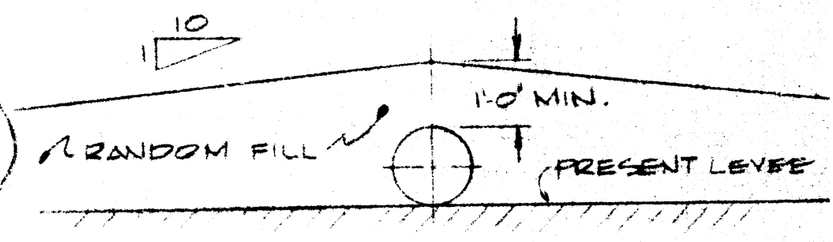
SH GRADE & STORM WATER SITE PLAN

STORM INLET

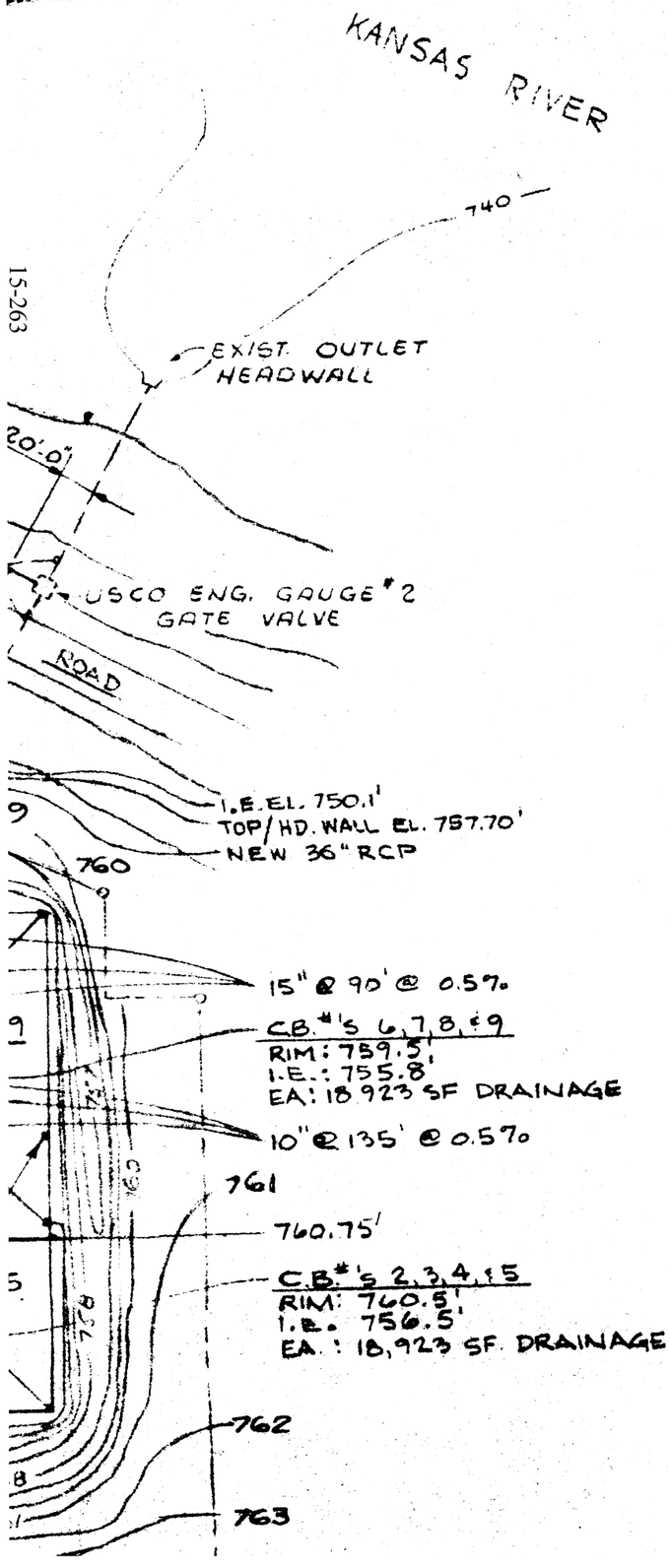
40' 0 5' 10' 20'
1" = 8' - 0"

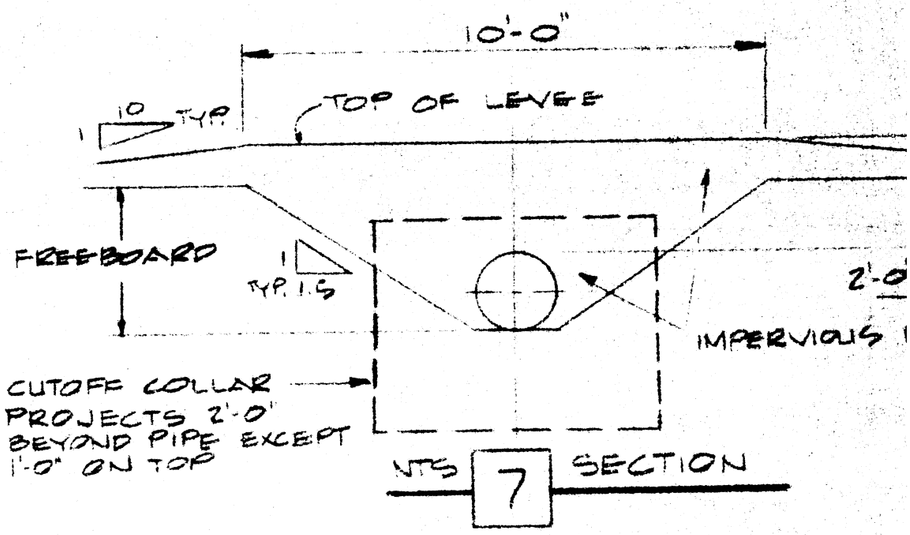
0 2 4' 8' 12'
1" = 4' - 0"



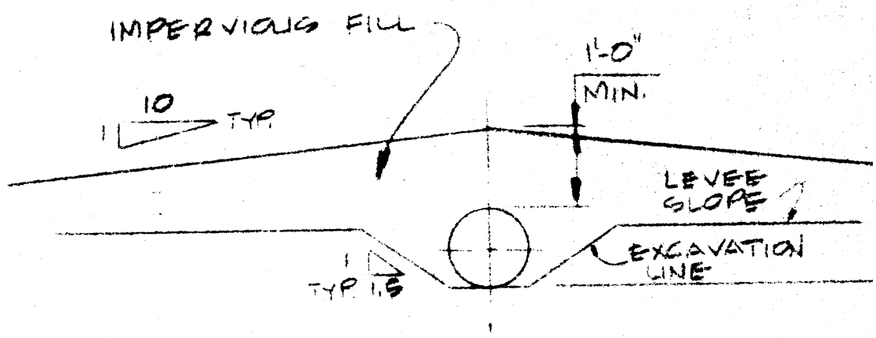


NTS 8 SECTION



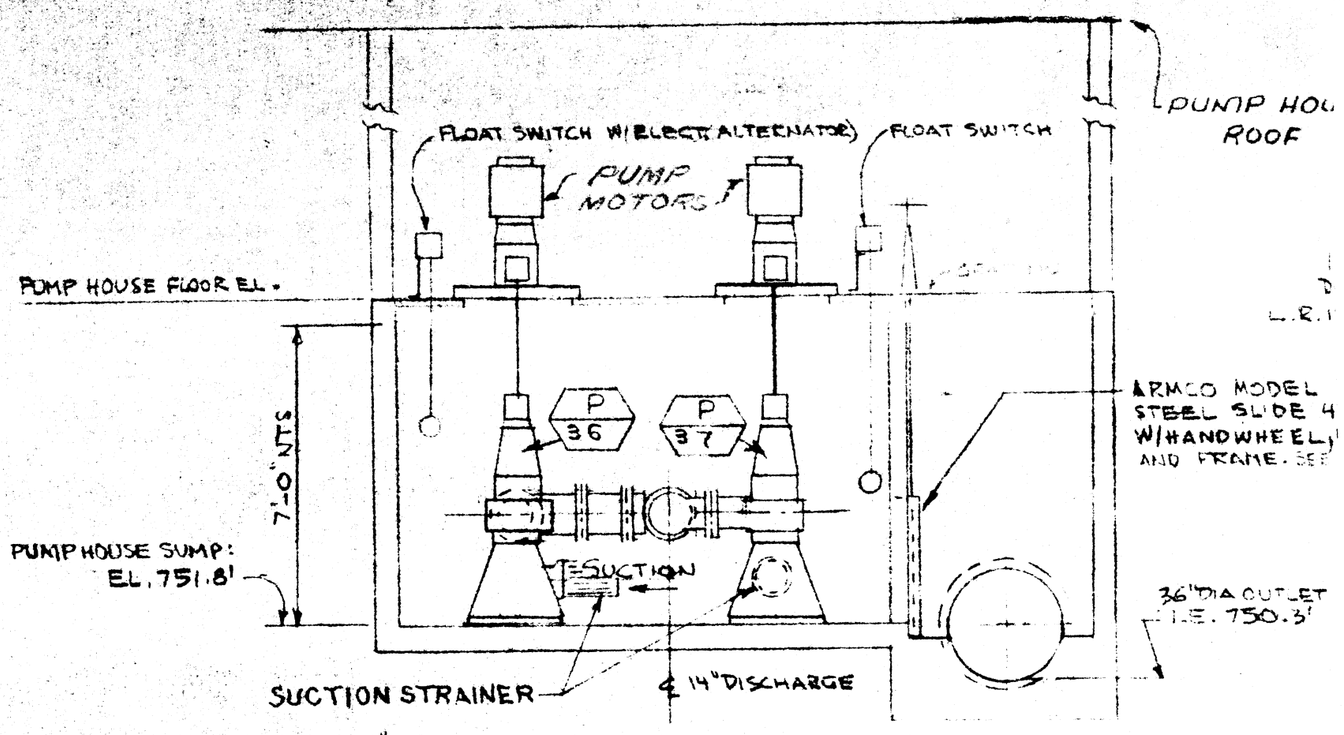


NTS 7 SECTION

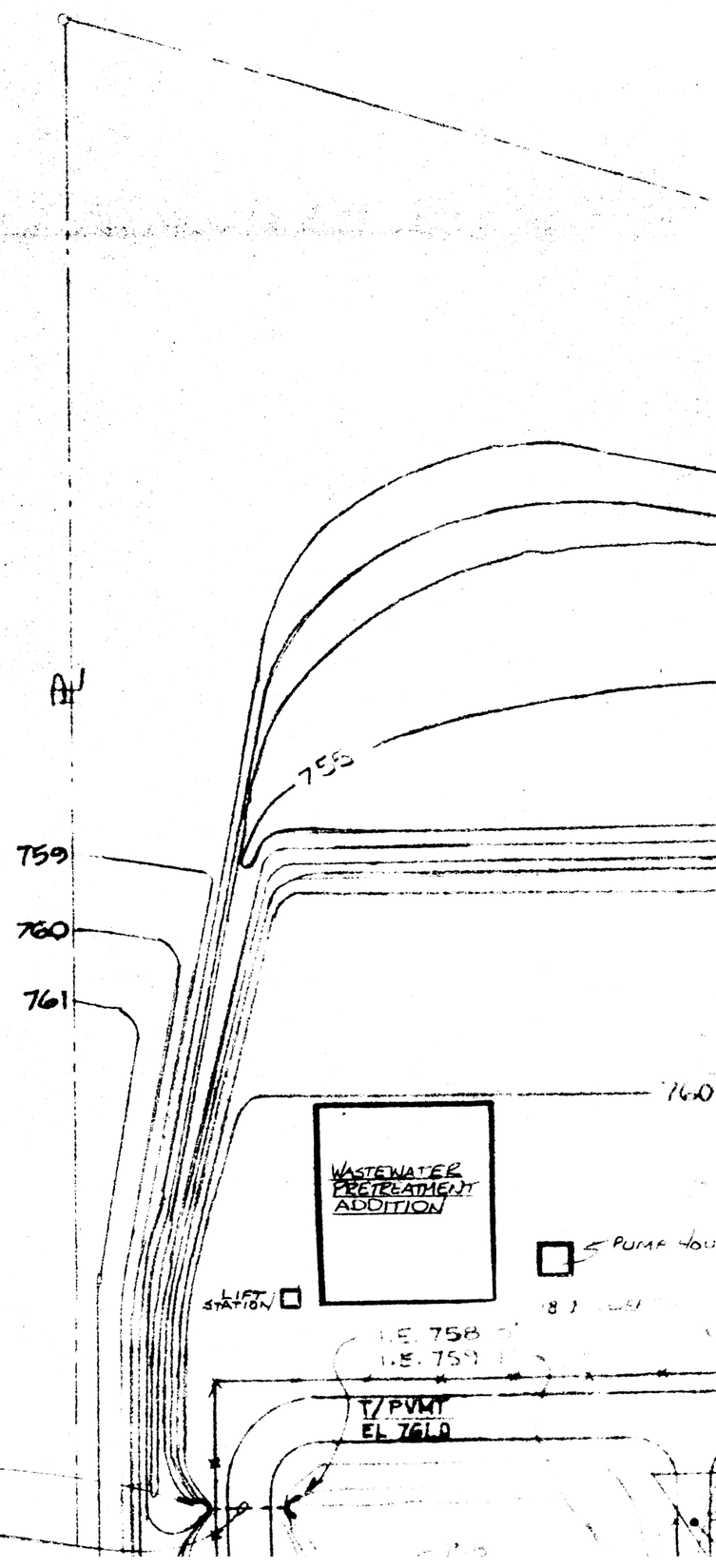


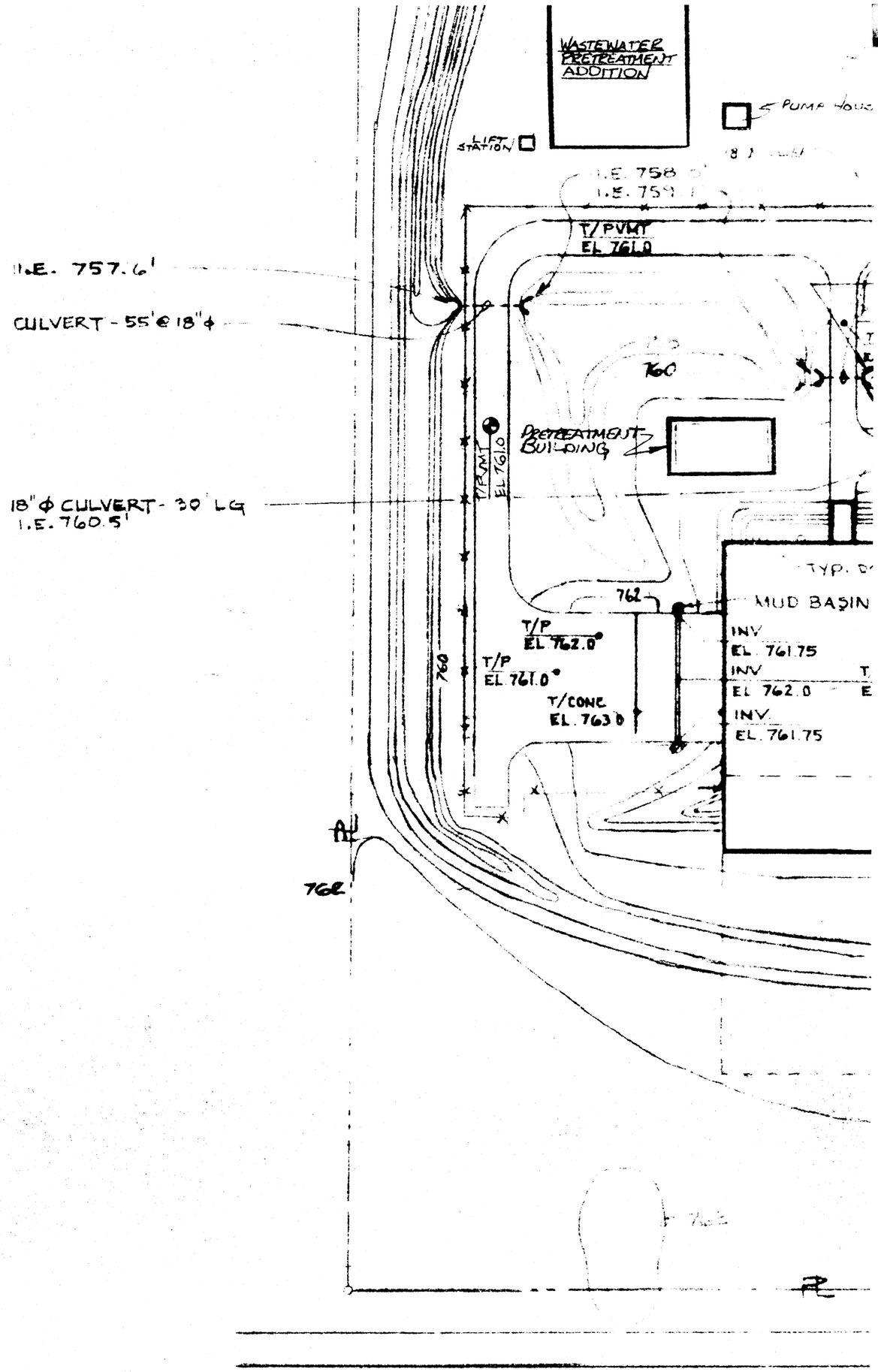
NTS 5 SECTION

15-263



1/4" = 1'-0" 6 SECTION





1" = 80'-0"

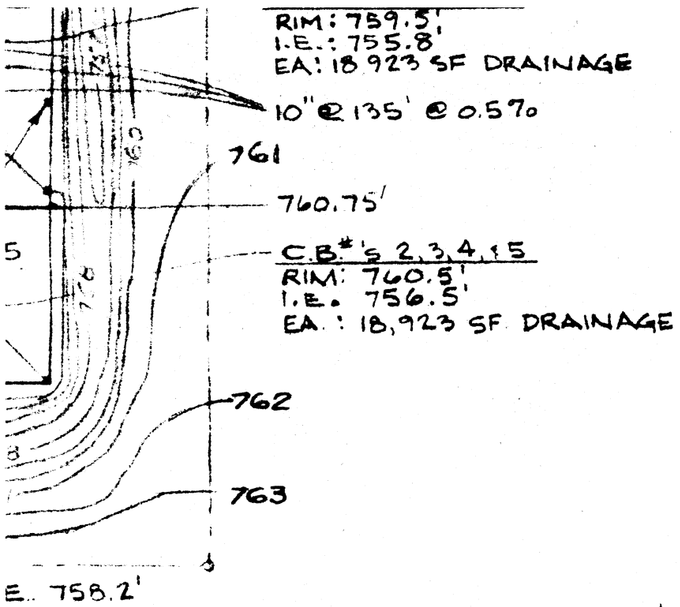
1

IF THIS SHEET IS NOT 30"x42" IT IS A REDUCED PRINT SCALE ACCORDINGLY

0 20' 20'

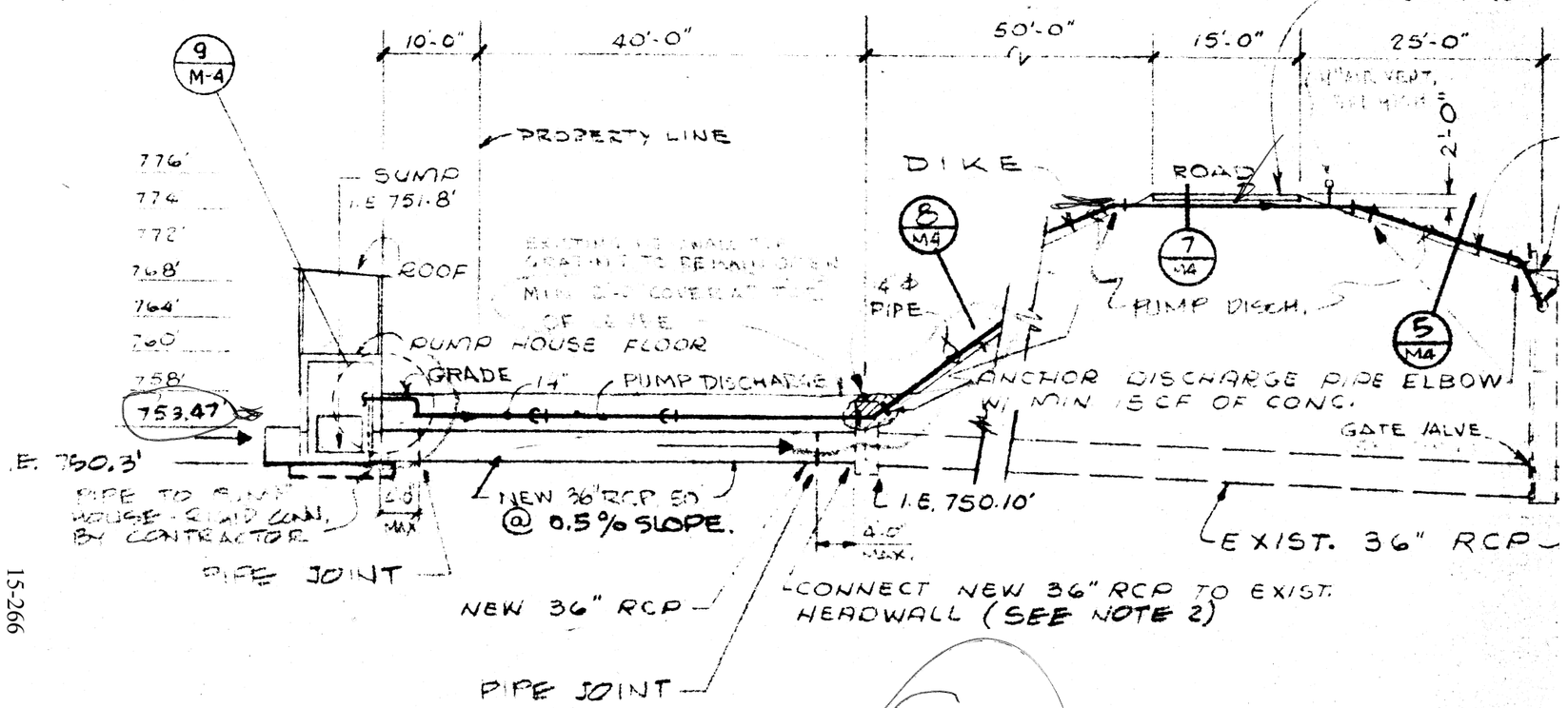
SCALE

15-265

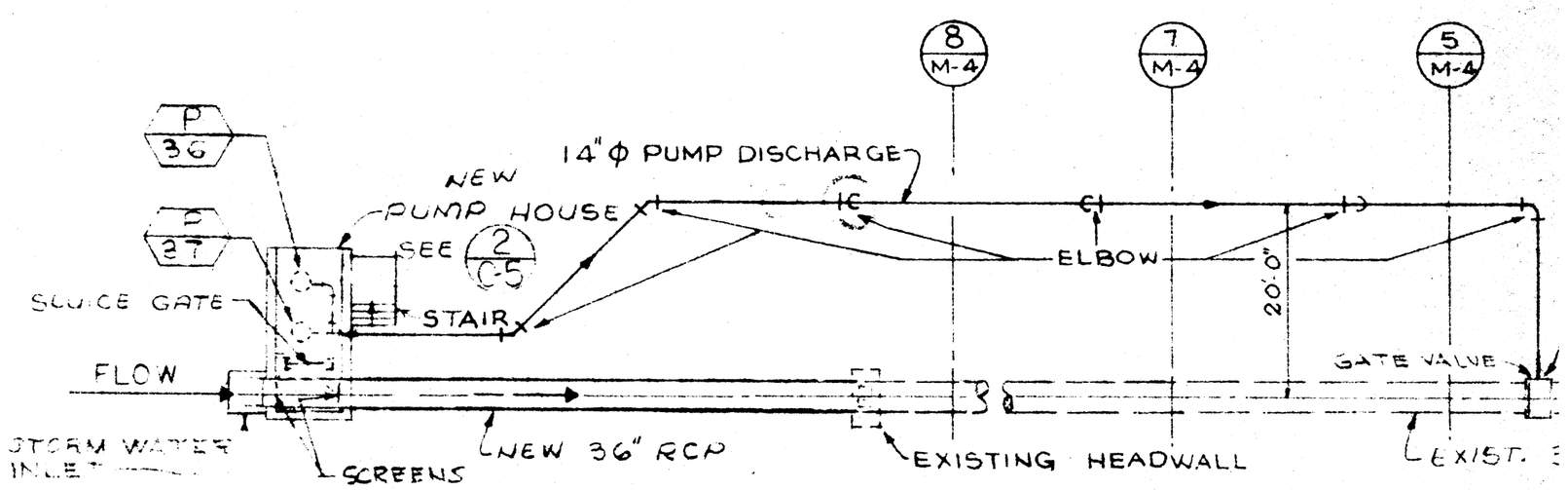


NTS **5** SECTION

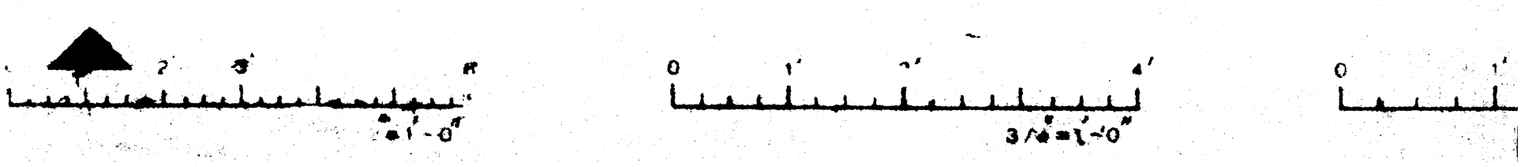
LANDSIDE



(HORIZ.) 1" = 15'-0"
(VERT.) 1" = 14'-0" **4** EXIST. & NEW STORM



1" = 15'-0" **3** PLAN — EXIST. & NEW ST



15-266

EXHIBIT A-15.11

Argentine Unit – ConAgra Pump Station Analysis

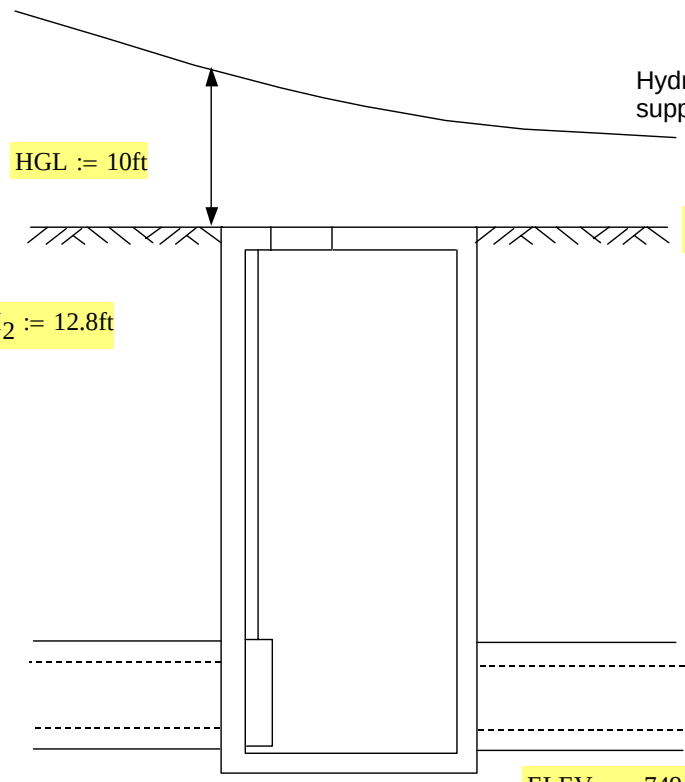
KANSAS RIVER LEVEES

Extreme Conditions, Nominal 500yr + 5ft

Variables

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

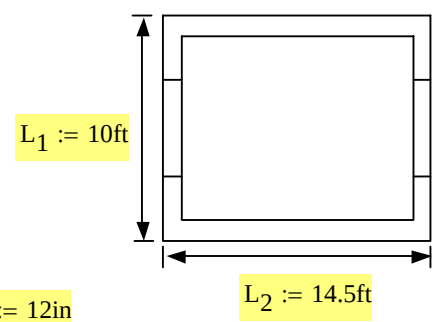
Properties



$H := ELEV_1 - ELEV_2$

$H = 7.86\,ft$

Plan



Wall Thickness 1 $D_1 := 12in$

Clear Cover 1 $C_{c1} := 2in$

Wall Thickness 2 $D_2 := 12in$

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

Floor Thickness $D_f := 32in$

Clear Cover $C_{cf} := 2in$

$d_1 := D_1 - C_{c1}$

$d_f := D_f - C_{cf}$

$l_2 := L_2 - 2 \cdot D_1$

$d_2 := D_2 - C_{c2}$

$l_1 := L_1 - 2 \cdot D_2$

$h := H - 2 \cdot D_f$

$H_1 := H + HGL$ $H_3 := H$

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. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi.
- The Concrete Reinforcing Steel Institute *Engineering Data Report Number 48* recommends using 33 ksi yield strengths for rebar of this time period. It should be noted that this is a minimum yield strength, not an average value. To obtain an average value it is assumed that actual steel strengths varied at least 1 standard of deviation from the minimum required strength. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete is 14%. As a result the mean Steel Yield strength shall be taken as 37.6 ksi.

Soil Properties $\phi := 28deg$ $\gamma := 120pcf$

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

$K_o = 0.531$

Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40ksi$

Concrete Properties $f'_c := 3ksi$

Concrete Unit Weight $\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

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

Load Factors

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

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

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

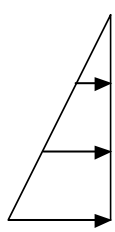
Analysis

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

WALLS

Wall Loading

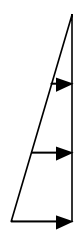
Soil or Soil & Water



$L_2 := H$

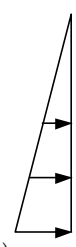
$L_2 = 7.86\,ft$

OR



$L_2 = 7.86\,ft$

&



$L'_3 := H_3 \cdot \left(\frac{H_1}{H_2}\right)$

$L_3 := \text{if}\left(H > H_2, H + HGL, L'_3\right)$

$L_3 = 10.967\,ft$

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

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

$W_1 = 500 \frac{lb}{ft^2}$

$W_2 = 925 \frac{lb}{ft^2}$

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

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

$W = 925 \frac{lb}{ft^2}$

WALL 1

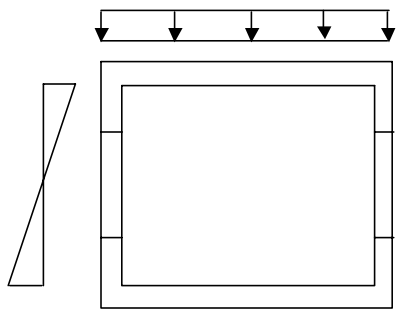
SHEAR

$$THRUST_1 := \frac{W \cdot L_2}{2} \qquad THRUST_1 = 3633 \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis

$$V'_{u1} := W \cdot \frac{l_1}{2}$$

$$V'_{u1} = 3698 \frac{lb}{ft}$$



$$V_{u1} := V'_{u1} \cdot \frac{\left(\frac{l_1}{2} - d_1\right)}{\frac{l_1}{2}} \qquad V_{u1} = 2928 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$\frac{l_1}{d_1} = 9.6$$

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

$$\phi V_{n1} := \phi_V \cdot \left[3.3 \cdot \left(\sqrt{f'_c}\right)\left(12in \cdot d_1\right) \cdot \frac{lb^{.5}}{in \cdot ft} + \frac{THRUST_1 \cdot d_1}{4 \cdot l_1}\right]$$

ACI 11.10.6 EQ (11-31)

$$\phi V_{n1} = 21784 \frac{lb}{ft}$$

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

Check1 = "OKAY"

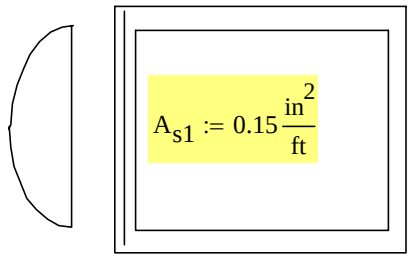
Factor of Safety $\quad FS_{S1} := \frac{\phi V_{n1}}{V_{u1}} \qquad FS_{S1} = 7.441$

Bending

NOTE: Discontinuous steel at corners. Elements act as simply supported beams with M=W*L^2/8 at midspan.

$$M_{u1} := \frac{W \cdot l_1^{\,2}}{8}$$

$$M_{u1} = 7396 \frac{ft \cdot lb}{ft}$$



$$A_{s1} := 0.15 \frac{in^2}{ft}$$

#4's Bars @ 16 in. Steel

$$b := 12 \frac{in}{ft} \qquad a := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a = 0.196 in$$

$$\phi M_{n1} := \phi_B \cdot A_{s1} \cdot F_y \cdot \left(d_1 - \frac{a}{2}\right)$$

$$\phi M_{n1} = 5.0 \frac{kip \cdot ft}{ft}$$

Check2 determines if the negative steel is capable of carrying the fixed end moment. A 1.5 existing conditions factor of safety is considered acceptable and will not require risk analysis.

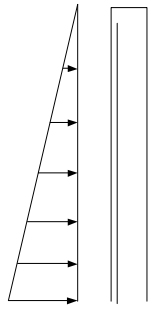
$$Check2 := if\left(\phi M_{n1} > 1.5 M_{u1}, "OKAY", "NO GOOD"\right)$$

Check2 = "NO GOOD"

Factor of Safety $\quad FS_{B1} := \frac{\phi M_{n1}}{M_{u1}} \qquad FS_{B1} = 0.669$

Cantilever Calculations

Wall could act as cantilever.



$$M_{u1c} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{W \cdot (H - D_f)}{2} \cdot \frac{(H - D_f)}{3} \right]$$

$$M_{u1c} = 4.2 \frac{kip \cdot ft}{ft}$$

$$A_{s1c} := 0.20 \frac{in^2}{ft}$$

#4's Bars @ 12 in. Steel

$$W = 924.54\,psf$$

$$b := 12 \frac{in}{ft} \quad a := \frac{A_{s1c} \cdot F_y}{0.85 f'_c \cdot b} \quad a = 0.261\,in \quad d_2 = 10\,in$$

$$\phi M_{n1c} := \phi_B \cdot A_{s1c} \cdot F_y \cdot \left(d_1 - \frac{a}{2} \right) \qquad \phi M_{n1c} = 6.6 \frac{kip \cdot ft}{ft}$$

$$Check2c := if\left(\phi M_{n1c} > 1.5 M_{u1c}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$$Check2c = \text{"OKAY"}$$

Factor of Safety $\quad FS_{B1c} := \frac{\phi M_{n1c}}{M_{u1c}} \qquad FS_{B1c} = 1.583$

Shear $\quad FS_S := FS_{S1} \qquad FS_S = 7.441$

Bending $\quad FS_B := \max\left(FS_{B1}, FS_{B1c}\right) \qquad FS_B = 1.583$

Controlling Factor of Safety

$$FS_1 := \min\left(FS_S, FS_B\right)$$

Controlling Mechanism

$$Comment1 := if\left(FS_B > FS_S, \text{"Wall 2 Shear"}, \text{"Wall 1 Flexural Steel"}\right)$$

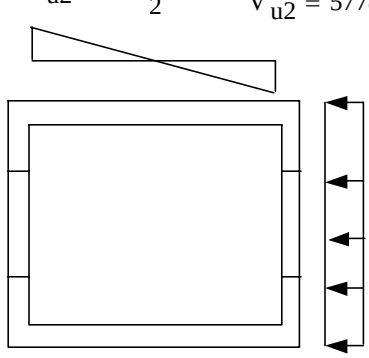
Factor of Safety $\quad FS_1 = 1.583$

Controlling Mechanism $\quad Comment1 = \text{"Wall 1 Flexural Steel"}$

WALL 2

SHEAR

$$V'_{u2} := W \cdot \frac{l_2}{2} \qquad V'_{u2} = 5778 \frac{lb}{ft}$$



$$THRUST_2 := \frac{W \cdot L_1}{2}$$

$$THRUST_2 = 4623 \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis

$$V_{u2} := V'_{u2} \cdot \frac{\left(\frac{l_2}{2} - d_2\right)}{\frac{l_2}{2}} \qquad V_{u2} = 5008 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$\frac{l_2}{d_2} = 15$$

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

$$\phi V_{n2} := \phi_V \cdot \left[3.3 \cdot \left(\sqrt{f'_c}\right)\left(12in \cdot d_2\right) \cdot \frac{lb^{.5}}{in \cdot ft} + \frac{THRUST_2 \cdot d_2}{4 \cdot l_2} \right]$$ ACI 11.10.6 EQ (11-31)

$$\phi V_{n2} = 21767 \frac{lb}{ft}$$

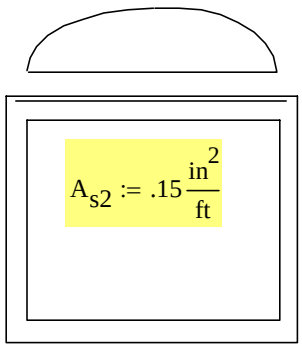
$$Check3 := if\left(\phi V_{n2} > 1.5V_{u2}, "OKAY", "NO GOOD"\right)$$ Check3 = "OKAY"

Factor of Safety $FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$ $FS_{S2} = 4.346$

Bending

NOTE: Discontinuous steel at corners. Elements act as simply supported beams with M=W*L^2/8 at midspan.

$$M_{u2} := \frac{W \cdot l_2^{\,2}}{8}$$



$$M_{u2} = 18057 \frac{ft \cdot lb}{ft}$$

$$A_{s2} := .15 \frac{in^2}{ft}$$

#4's Bars @ 16 in. Steel

$$b := 12 \frac{in}{ft} \quad a := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \quad a = 0.196\,in$$

$$d_2 = 10\,in$$

$$\phi M_{n2} := \phi_B \cdot A_{s2} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right)$$

$$\phi M_{n2} = 5.0 \frac{kip \cdot ft}{ft}$$

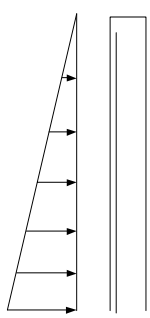
Check4 determines if the negative steel is capable of carrying the fixed end moment. A 1.5 existing conditions factor of safety is considered acceptable and will not require risk analysis.

$$Check4 := if\left(\phi M_{n2} > 1.5M_{u2}, "OKAY", "NO GOOD"\right)$$ Check4 = "NO GOOD"

Factor of Safety $FS_{B2} := \frac{\phi M_{n2}}{M_{u2}}$ $FS_{B2} = 0.274$

Cantilever Calculations

Wall could act as cantilever.



$$M_{u2c} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{W \cdot \left(H - D_f\right)}{2} \cdot \frac{\left(H - D_f\right)}{3} \right]$$

$$M_{u2c} = 4.2 \frac{kip \cdot ft}{ft}$$

$$A_{s2c} := 0.20 \frac{in^2}{ft}$$

#4's Bars @ 12 in. Steel

$$W = 924.54\,psf$$

$$b := 12 \frac{in}{ft} \quad a := \frac{A_{s2c} \cdot F_y}{0.85 f'_c \cdot b} \quad a = 0.261\,in \quad d_2 = 10\,in$$

$$\phi M_{n2c} := \phi_B \cdot A_{s2c} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right) \qquad \phi M_{n2c} = 6.6 \frac{kip \cdot ft}{ft}$$

$$Check5 := if\left(\phi M_{n2c} > 1.5 M_{u2c}, "OKAY", "NO GOOD"\right) \qquad Check5 = "OKAY"$$

Factor of Safety $\quad FS_{B2c} := \frac{\phi M_{n2c}}{M_{u2c}} \qquad FS_{B2c} = 1.583$

Shear

$$FS_S := FS_{S2} \qquad FS_S = 4.346$$

Bending

$$FS_B := \max\left(FS_{B2}, FS_{B2c}\right) \qquad FS_B = 1.583$$

Controlling Factor of Safety

$$FS_2 := \min\left(FS_S, FS_B\right)$$

Controlling Mechanism

$$Comment2 := if\left(FS_B > FS_S, "Wall\ 2\ Shear", "Wall\ 2\ Flexural\ Steel"\right)$$

Factor of Safety $\quad FS_2 = 1.583$

Controlling Mechanism $\quad Comment2 = "Wall\ 2\ Flexural\ Steel"$

Controlling Factor of Safety

$$FoS := \min\left(FS_1, FS_2\right) \qquad FoS = 1.583$$

Controlling Mechanism

$$Mechanism := if\left(FS_2 > FoS, Comment1, Comment2\right) \qquad Mechanism = "Wall\ 2\ Flexural\ Steel"$$

Uplift Calculations

If the Factor of Safety of greater then 1.3 is required for usual flood events (10 year), and greater then 1.1 is required for extreme flood events (Top of Protection).

Weight of Structure

$$W_{Structure} := \left(L_1 \cdot L_2 \cdot H - l_1 \cdot l_2 \cdot h\right) \cdot \gamma_c \qquad W_{Structure} = 54.769\,kip$$

Weight of Water in Structure

Assume no water in structure. $\quad W_{Water} := 0kip$

Surcharge

$$Surcharge := 25psf \cdot L_1 \cdot L_2 \qquad Surcharge = 1965\,lb$$

Uplift

$$Uplift := L_1 \cdot L_2 \cdot L_3 \cdot \gamma_w \qquad Uplift = 53.79\,kip$$

Weight of Surcharge Water Above Top Surface

No surcharge water. $\quad W_{SurWater} := L_1 \cdot L_2 \cdot 25psf$

$$SF_{UPLIFT} := \frac{W_{Structure} + W_{Water} + Surcharge}{Uplift - W_{SurWater}} \qquad SF_{UPLIFT} = 1.095$$

$$Result_{Uplift} := if\left(SF_{UPLIFT} > 1.1, "Uplift\ Acceptable", "Uplift\ Concerns"\right) \qquad Result_{Uplift} = "Uplift\ Concerns"$$

NOTE: Close enough to 1.1 Allowable safety factor.

EXHIBIT A-15.12

Argentine Unit – ConAgra Pump Curve

FAX

ConAgra Foods

ConAgra Foods Retail Products Company
Refrigerated Foods Group
4612 Speaker Road
P. O. Box 2924
Kansas City, KS 66110
913-281-3200
FAX: 913-371-3872

To: Ron Jansen **Fax:** 816 426 5462

From: Bill Kennedy

Date: 4-22-05 **Pages including cover sheet:** 2 (Two)

Re: Pump Curve **cc:**

Pump Curve Sheet - Food Pumps

Con Agra Foods

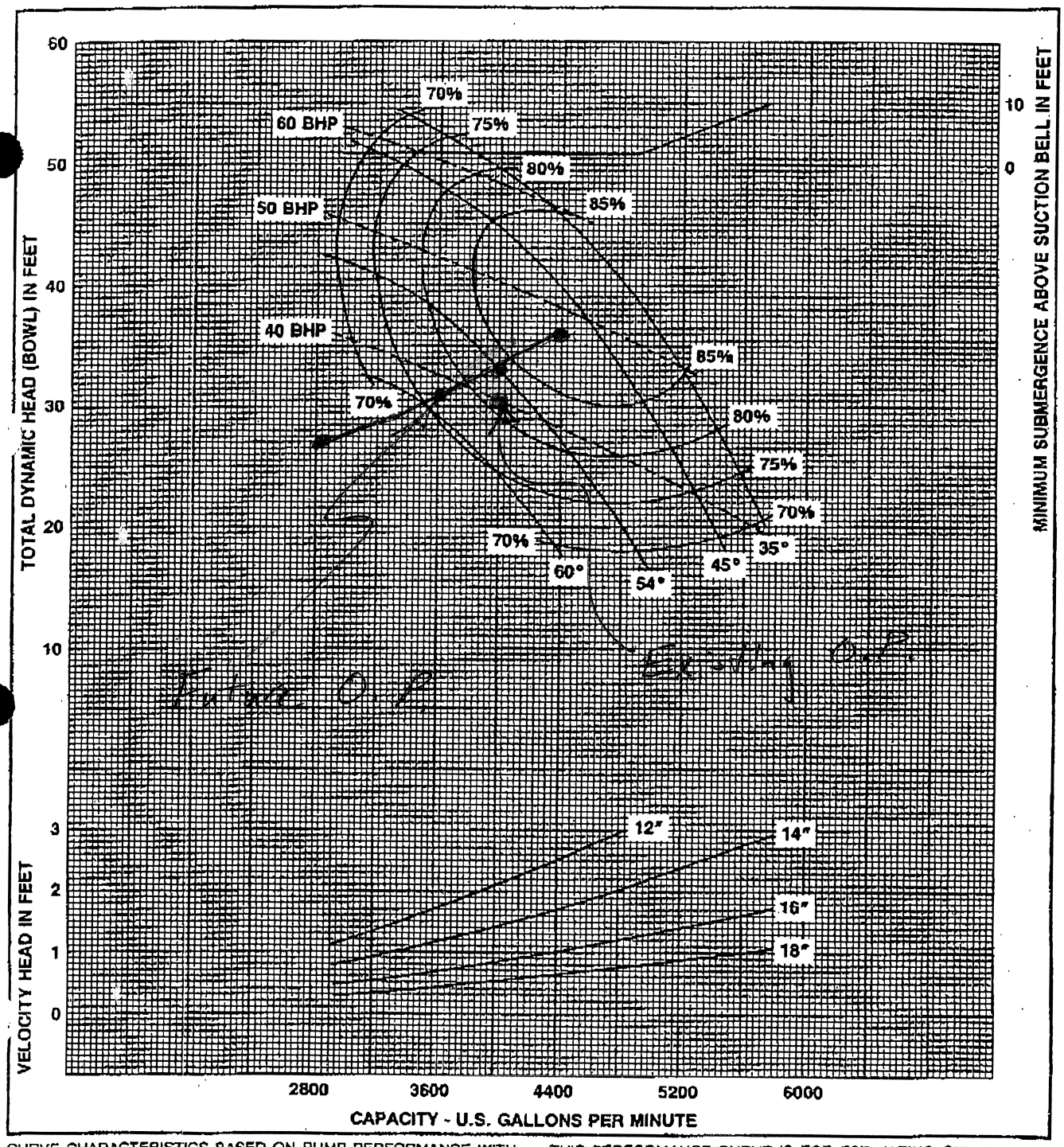
Bill

913-233-3252

FLOWSERVE

5310 Taneytown Pike
Taneytown, MD 21787

MODEL 12CKMD 1760 RPM

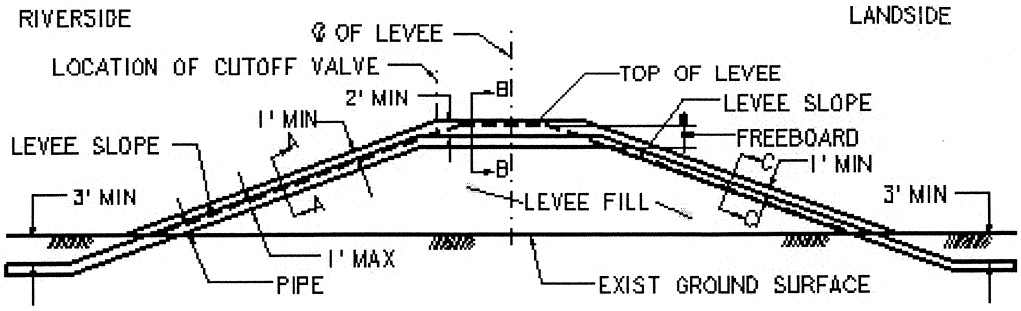


CURVE CHARACTERISTICS BASED ON PUMP PERFORMANCE WITH SPECIFIED AMOUNT OF CLEAR, NON-AERATED, FRESH COLD WATER.

THIS PERFORMANCE CURVE IS FOR ESTIMATING ONLY. CURVE SHOWS SINGLE STAGE PERFORMANCE. FOR MULTIPLE, VAR-

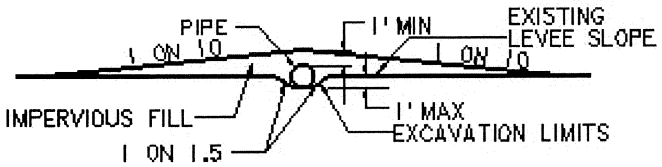
EXHIBIT A-15.13

Typical Pump Station Discharge "Up and Over" Detail



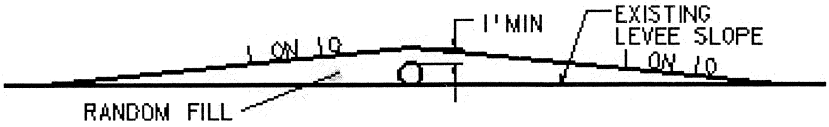
PROFILE OF PIPELINE CROSSING

NTS



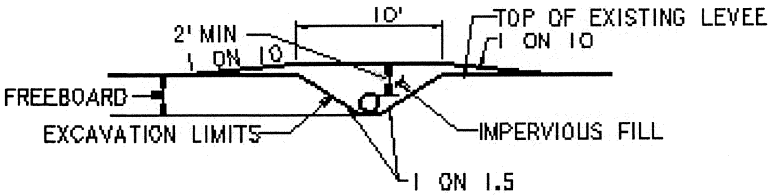
SECTION A (RIVERSIDE SLOPE)

NTS



SECTION C (LANDSIDE SLOPE)

NTS



SECTION B (LEVEE CREST)

NTS

NOTE:

IF PERMISSION IS GRANTED FOR A PIPELINE OR UTILITY TO CROSS THE LEVEE BELOW THE THE FREEBOARD, A POSITIVE CUTOFF VALVE OR CLOSURE GATE WILL BE INSTALLED AT THE RIVERSIDE SHOULDER OF THE LEVEE. POSITIVE CUTOFF MAY BE REQUIRED EVEN THOUGH THE CROSSING IS WITHIN THE FREEBOARD, IF IT IS DETERMINED TO BE NECESSARY BY THE CORPS OF ENGINEERS

PIPELINE CROSSING A LEVEE

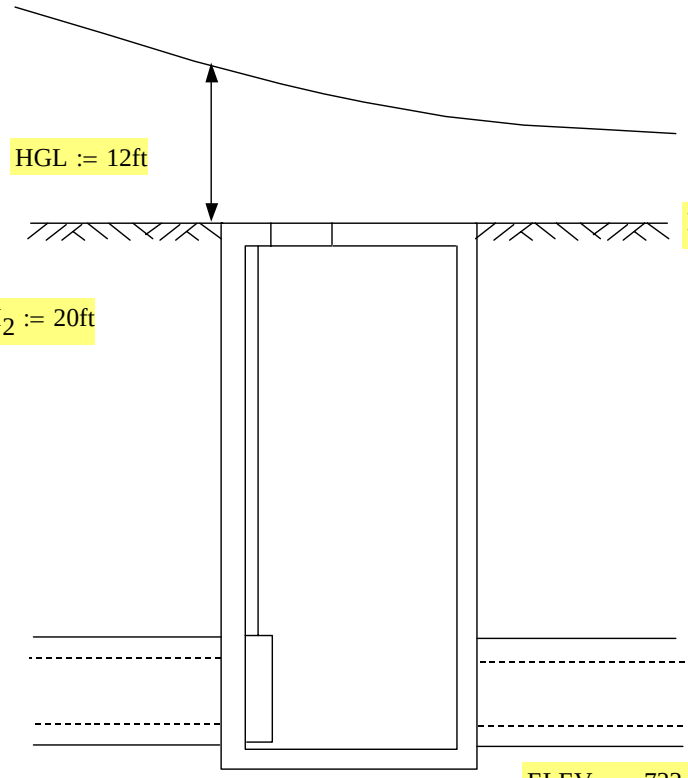
EXHIBIT A-15.14

Argentine Unit – Santa Fe Pump Station Analysis

Variables

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

Properties



$HGL := 12ft$

$ELEV_1 := 766.93ft$

Elevation

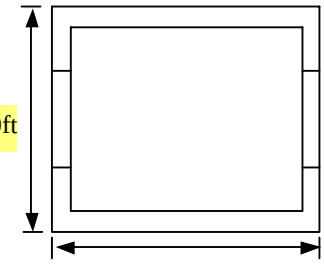
Blanket Thicknes $H_2 := 20ft$

Water head pressure in Wet Well under operating conditions.

$HEAD := 0ft$

$ELEV_2 := 733.6ft + 7ft$

Plan



$L_1 := 20ft$

$L_2 := 10ft$

$H := ELEV_1 - ELEV_2$

$H = 26.33\,ft$

Wall Thickness 1 $D_1 := 36in$

Clear Cover 1 $C_{c1} := 3in$

Wall Thickness 2 $D_2 := 36in$

Clear Cover 2 $C_{c2} := 3in$

Floor Thickness $D_f := 7ft$

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

$l_2 := L_2 - 2 \cdot D_1$

$d_2 := D_2 - C_{c2}$

$d_1 := D_1 - C_{c1}$

$d_f := D_f - C_{cf}$

$l_1 := L_1 - 2 \cdot D_2$

$h := H - 2 \cdot D_f$

$H_1 := H + HGL$ $H_3 := H$

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. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi.
- The Concrete Reinforcing Steel Institute *Engineering Data Report Number 48* recommends using 33 ksi yield strengths for rebar of this time period. 60 KSI steel

Soil Properties $\phi := 28deg$

$\gamma := 120pcf$

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

$K_o = 0.531$

Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40ksi$

Concrete Properties $f'_c := 4ksi$

Concrete Unit Weight $\gamma_c := 150pcf$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

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

Load Factors

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

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

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

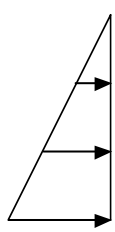
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) not applied for analysis of existing conditions.

Analysis

WALLS

Wall Loading

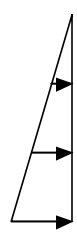
Soil or Soil & Water



$L_2 := H$

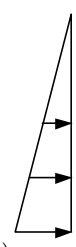
$L_2 = 26.33\,ft$

OR



$L_2 = 26.33\,ft$

&



$L'_3 := H_3 \cdot \left(\frac{H_1}{H_2}\right)$

$L_3 := \text{if}\left(H > H_2, H + HGL, L'_3\right)$

$L_3 = 38.33\,ft$

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

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

$W_1 = 1676 \frac{lb}{ft^2}$

$W_2 = 3196 \frac{lb}{ft^2}$

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

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

$W = 3196 \frac{lb}{ft^2}$

WALL 1

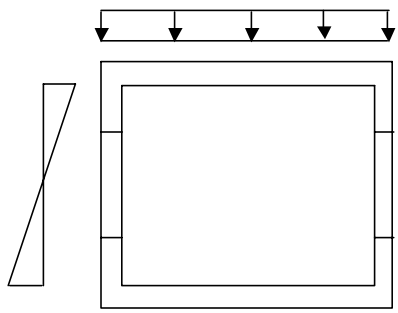
SHEAR

$$THRUST_1 := \frac{W \cdot L_2}{2} \qquad THRUST_1 = 42081 \frac{lb}{ft}$$

Thrust to be used in CASTR for thrust analysis

$$V'_{u1} := W \cdot \frac{l_1}{2}$$

$$V'_{u1} = 22375 \frac{lb}{ft}$$



$$V_{u1} := V'_{u1} \cdot \frac{\left(\frac{l_1}{2} - d_1\right)}{\frac{l_1}{2}} \qquad V_{u1} = 13585 \frac{lb}{ft}$$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$$\frac{l_1}{d_1} = 5.091$$

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

$$\phi V_{n1} := \phi_V \cdot \left[3.3 \cdot \left(\sqrt{f'_c}\right)\left(12in \cdot d_1\right) \cdot \frac{lb^{.5}}{in \cdot ft} + \frac{THRUST_1 \cdot d_1}{4 \cdot l_1}\right]$$

ACI 11.10.6 EQ (11-31)

$$\phi V_{n1} = 84716 \frac{lb}{ft}$$

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

Check1 = "OKAY"

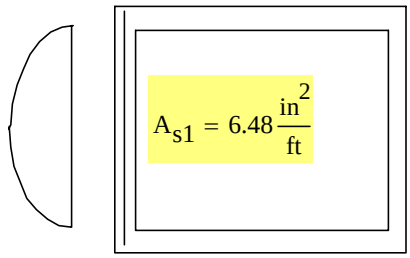
Factor of Safety $\quad FS_{S1} := \frac{\phi V_{n1}}{V_{u1}} \qquad FS_{S1} = 6.236$

Bending

NOTE: Discontinuous steel at corners. Elements act as simply supported beams with M=W*L^2/8 at midspan.

$$M_{u1} := \frac{W \cdot l_1^{\,2}}{8}$$

$$M_{u1} = 78312 \frac{ft \cdot lb}{ft}$$



Assume 3% reinforcing steel by volume with half on each face.

$$A_{s1} := .03 \frac{D_1}{2}$$

$$b := 12 \frac{in}{ft} \qquad a := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a = 6.353 in$$

$$\phi M_{n1} := \phi_B \cdot A_{s1} \cdot F_y \cdot \left(d_1 - \frac{a}{2}\right)$$

$$\phi M_{n1} = 644.2 \frac{kip \cdot ft}{ft}$$

A 1.5 existing conditions factor of safety is considered acceptable and will not require risk analysis.

$$Check2 := if\left(\phi M_{n1} > 1.5 M_{u1}, "OKAY", "NO GOOD"\right)$$

Check2 = "OKAY"

Factor of Safety $\quad FS_{B1} := \frac{\phi M_{n1}}{M_{u1}} \qquad FS_{B1} = 8.226$

Controlling Factor of Safety

$$FS_1 := \min(FS_{S1}, FS_{B1})$$

Controlling Mechanism

$$Comment1 := \text{if}\left(FS_{B1} > FS_{S1}, \text{"Wall 1 Shear"}, \text{"Wall 1 Flexural Steel"}\right)$$

Factor of Safety $FS_1 = 6.236$

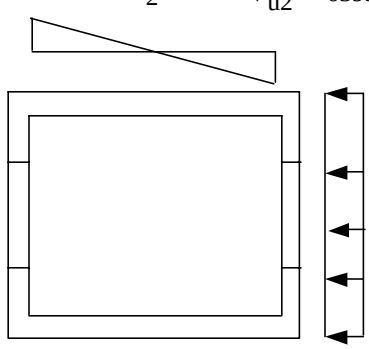
Controlling Mechanism $Comment1 = \text{"Wall 1 Shear"}$

WALL 2

SHEAR

Thrust to be used in CASTR for thrust analysis

$$V_{u2} := W \cdot \frac{l_2}{2} \qquad V_{u2} = 6393 \frac{lb}{ft}$$



$$THRUST_2 := \frac{W \cdot L_1}{2}$$

$$THRUST_2 = 31964 \frac{lb}{ft}$$

$$\frac{l_2}{d_2} = 1.455$$

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

$$\phi V_{n2} := \phi_V \cdot \left[3.3 \cdot \left(\sqrt{f'_c}\right)\left(12in \cdot d_2\right) \cdot \frac{lb^{.5}}{in \cdot ft} + \frac{THRUST_2 \cdot d_2}{4 \cdot l_2}\right]$$

ACI 11.10.6 EQ (11-31)

$$\phi V_{n2} = 88143 \frac{lb}{ft}$$

$$Check3 := \text{if}\left(\phi V_{n2} > 1.5 V_{u2}, \text{"OKAY"}, \text{"NO GOOD"}\right)$$

$Check3 = \text{"OKAY"}$

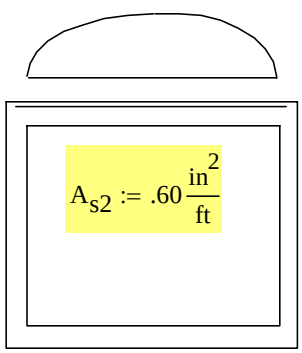
Factor of Safety $FS_{S2} := \frac{\phi V_{n2}}{V_{u2}}$ $FS_{S2} = 13.788$

Bending

NOTE: First assume elements act as simply supported beams with M=W*L^2/8 at midspan.

Assume 3% reinforcing steel by volume with half on each face.

$$M_{u2} := \frac{W \cdot l_2^2}{8} \qquad M_{u2} = 6393 \frac{ft \cdot lb}{ft}$$



$$A_{s2} := .03 \frac{D_2}{2}$$

$$A_{s2} := .60 \frac{in^2}{ft}$$

#7 Bars @ 12 in. Steel

$$b := 12 \frac{in}{ft} \qquad a := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a = 0.588 in$$

$$d_2 = 33 in$$

$$\phi M_{n2} := \phi_B \cdot A_{s2} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right)$$

$$\phi M_{n2} = 65.4 \frac{kip \cdot ft}{ft}$$

Check4 A 1.5 existing conditions factor of safety is considered acceptable and will not require risk analysis.

$Check4 := if\left(\phi M_{n2} > 1.5M_{u2}, "OKAY", "NO GOOD"\right)$ $\quad Check4 = "OKAY"$

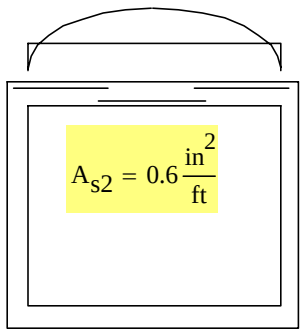
Factor of Safety $\quad FS_{B2} := \frac{\phi M_{n2}}{M_{u2}}$ $\quad FS_{B2} = 10.232$

NOTE: Continuity of Reinforcement at corners. If the structure acts as a frame (fixed ends for elements) the moments will be M=W*L^2/12 and M=W*L^2/24. Otherwise the elements act as simply supported beams with M=W*L^2/8 at midspan.

$M_{uMIN2} := \frac{W_1 \cdot l_2^2}{24}$ $\quad M_{uMIN2} = 1.1\frac{kip \cdot ft}{ft}$

$M_{uMAX2} := \frac{W \cdot l_2^2}{12}$ $\quad M_{uMIN2} = 1.1\frac{kip \cdot ft}{ft}$

$A_{sMAX2} := 0.79\frac{in^2}{ft}$



$A_{s2} = 0.6\frac{in^2}{ft}$

8's @ 12" Continuious Corner Steel
7's @ 12" Interior

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

$a := \frac{A_{sMAX2} \cdot F_y}{0.85f'_c \cdot b}$ $\quad a = 0.775\,in$

$\phi M_{nMAX2} := \phi_B \cdot A_{sMAX2} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right)$

$\phi M_{nMAX2} = 85.9\frac{kip \cdot ft}{ft}$

Check5 and Check6 determine if the wall acts as a fixed end member.

Check5 determines if the negative steel is capable of carrying the fixed end moment.

$Check5 := if\left(\phi M_{nMAX2} > M_{uMAX2}, "OKAY", "NO GOOD"\right)$ $\quad Check5 = "OKAY"$

Factor of Safety $\quad FS'_{B2} := \frac{\phi M_{nMAX2}}{M_{uMAX2}}$ $\quad FS'_{B2} = 20.151$

$a := \frac{A_{s2} \cdot F_y}{0.85f'_c \cdot b}$ $\quad a = 0.588\,in$

$\phi M_{nMIN2} := \phi_B \cdot A_{s2} \cdot F_y \cdot \left(d_2 - \frac{a}{2}\right)$

$\phi M_{nMIN2} = 65.4\frac{kip \cdot ft}{ft}$

Check6 determines if the positive steel is capable of carrying the center span moment with fixed ends.

$Check6 := if\left(\phi M_{nMIN2} > M_{uMIN2}, "OKAY", "NO GOOD"\right)$ $\quad Check6 = "OKAY"$

Factor of Safety $\quad FS''_{B2} := \frac{\phi M_{nMIN2}}{M_{uMIN2}}$ $\quad FS''_{B2} = 58.534$

NOTE: Checks performed to determine if there is a higher Factor of Safety for the pinned condition or

fixed end condition.

Bending Controlling Factor of Safety

$$FS_{B2} := \max\left(FS_{B2}, \min\left(FS'_{B2}, FS''_{B2}\right)\right) \qquad FS_{B2} = 20.151$$

Shear

$$FS_S := FS_{S2} \qquad FS_S = 13.788$$

Bending

$$FS_B := FS_{B2} \qquad FS_B = 20.151$$

Controlling Factor of Safety

$$FS_2 := \min\left(FS_S, FS_B\right)$$

Controlling Mechanism

$$Comment2 := \text{if}\left(FS_B > FS_S, \text{"Wall 2 Shear"}, \text{"Wall 2 Flexural Steel"}\right)$$

Factor of Safety $\qquad FS_2 = 13.788$

Controlling Mechanism $\qquad Comment2 = \text{"Wall 2 Shear"}$

Controlling Factor of Safety

$$FoS := \min\left(FS_{S1}, FS_{B1}, FS_{S2}, FS_{B2}\right) \qquad FoS = 6.236$$

Controlling Mechanism

$$Mechanism := \text{if}\left(FS_2 > FoS, Comment1, Comment2\right) \qquad Mechanism = \text{"Wall 1 Shear"}$$

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 of the more detailed load analysis using combined load distribution factors (initial load and levee raise) and relative certainty of existing steel it has been decided that a Factor of Safety of 1.5 will be acceptable.

Uplift Calculations

If the Factor of Safety of greater then 1.3 is required for usual flood events (10 year), and greater then 1.1 is required for extreme flood events (Top of Protection).

Weight of Structure

$W_{Structure} := \left(L_1 \cdot L_2 \cdot H - l_1 \cdot l_2 \cdot h\right) \cdot \gamma_c$ $\qquad W_{Structure} = 1976.235\,kip$

Weight of Water in Structure

Assume no water in structure. $\qquad W_{Water} := 0kip$

Surcharge

$Surcharge := 50psf \cdot L_1 \cdot L_2 + 2000lb \cdot 2$ $\qquad Surcharge = 30330\,lb$

Uplift

$Uplift := L_1 \cdot L_2 \cdot L_3 \cdot \gamma_w$ $\qquad Uplift = 1259.518\,kip$

Weight of Surcharge Water Above Top Surface

No surcharge water. $\qquad W_{SurWater} := 0lb$

$$SF_{UPLIFT} := \frac{W_{Structure} + W_{Water} + Surcharge}{Uplift - W_{SurWater}} \qquad SF_{UPLIFT} = 1.593$$

$Result_{Uplift} := if\left(SF_{UPLIFT} > 1.1, \text{"Uplift Acceptable"}, \text{"Uplift Concerns"}\right)$

$Result_{Uplift} = \text{"Uplift Acceptable"}$