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

Chapter A-12

STRUCTURAL ANALYSIS EXISTING CONDITIONS

CHAPTER A-12
STRUCTURAL ANALYSIS – EXISTING CONDITIONS

A-12.1 INTRODUCTION

The structural aspects of the levee units included in this section consist of floodwalls, retaining walls, and closure structures for openings in levees and floodwalls. A brief discussion of these features by levee unit is provided. Refer to the spreadsheet listings in the General chapter for additional information. The structural analysis involved an assessment of the existing condition of the structures. The assessment was based on visual observation, dated construction plans, historical data, discussions with the Corps of Engineers and Levee District personnel familiar with and involved in the inspection, operation, and maintenance of the levee units. Detailed engineering analysis was used in the floodwall assessment to determine a probability of failure required as input into the HEC-FDA model. Based on available historic data, site inspections, and engineering judgment time dependent deterioration of structures has not been included in the probability of failure analysis. Probability of failure analysis will not be used for design.

A-12.2 DESCRIPTION OF THE LEVEE UNITS – STRUCTURAL ASPECTS

A-12.2.1 Argentine Unit

There are two concrete floodwalls in this unit. One floodwall begins at Station 251+65 and continues to Station 253+92. This wall protects the Argentine Boulevard Pumping Plant. The other floodwall begins at Station 276+70 and extends east, or downstream, along the Santa Fe railroad tracks to Station 287+92. Both walls are inverted cantilever T-walls on spread footing foundation.

There are two existing openings in the line of protection that have closure structures in the Argentine Unit. Both of these are openings for railroads and are at Stations 29+02 and 288+57. Timber stoplogs are used to close the railroad openings. There used to be a third structure (a sandbag closure gap) at Kansas Avenue near the West Kansas Avenue Bridge. However, a bridge replacement and raise eliminated the need for the structure.

There are no retaining walls in the Argentine Unit.

A-12.2.2 Armourdale Unit

There are three concrete floodwalls in this unit. The first floodwall begins at Station 60+30 and continues to Station 77+78. The second floodwall begins at Station 246+90 and continues to Station 250+50. Both of these walls are inverted cantilever T-walls on spread footing foundations. The third wall begins at Station 257+65 and continues to Station 302+58. The third wall is an inverted T-wall on concrete pile foundation.

There are four existing openings with closure structures in the Armourdale Unit. Two are stoplog closures and the remaining two are sandbag closures. All stoplogs are of timber construction. A previously utilized closure structure at Station 62+35 has been abandoned and a gap at Station 249+54 has been closed with a floodwall. Further information can be found in Section A-2.4 containing the spreadsheet feature listings.

The Armourdale Unit has two retaining walls landside of the floodwalls. These are located from Station 60+30 to 63+10 and from Station 257+65 to 259+13.

A-12.2.3 Birmingham Unit

There are three floodwall segments at the northern end of the unit, south of the bend in Big Shoal Creek that were constructed as part of the Northland Park Development. One wall begins at Station 544+76 and extends to Station 546+19. There is a 120" sanitary sewer with an existing gatewell that crosses under the wall. The gatewell was raised (approximately 7' extended in height) to the height of protection of the newly constructed wall. The wall is as an inverted concrete T-wall on a spread footing foundation. The next wall spans between two railroad gaps in the line of protection. This wall begins at Station 555+53 and continues to Station 557+90 (stationing is approximate) and is also an inverted concrete T-wall on spread footing foundation. There is a third segment of wall beginning at Station 558+05 and ending at Station 558+50 (stationing is approximate), which is the downstream end of the line of protection. This wall is also an inverted concrete T-wall on spread footing foundation.

There are four openings with closure structures within the Birmingham Unit. All of the closure structures are at railroad crossings. The openings located at Stations 3+41 and 435+05 are closed with sandbags. A third sandbag gap is shown on the O&M Drawings at Station 557+89, but was eliminated with the Northland Park construction. Two stoplog closure structures were built with the floodwall associated with Northland Park. The first of these two closure structures is at Station 555+27. This gap is approximately 42' wide by 10'-9" high across two railroad tracks. The second closure structure is across a single track at Station 557+95 and is approximately 20'-7" wide by 12'-6" high.

There are no retaining walls in the Birmingham Unit.

A-12.2.4 Central Industrial District-Kansas Unit

There are three reaches of concrete floodwall in this unit. The first reach of floodwall begins at Station 26+73, near the James Street Bridge, and extends upstream to Station 40+31, near the Kansas City Southern Railroad Bridge. The second reach of floodwall extends from Station 74+36 to Station 77+28. The last reach of floodwall begins at Station 102+73, near the Chicago, Rock Island and Pacific Railroad Bridge, and extends upstream to the end of the unit, (Station 166+25) near the Seventh Street Bridge. Except for 16' of inverted cantilever T-wall on spread footing foundation (Station 75+76 to Station 75 +92) all of the floodwalls are inverted T-walls on pile foundations. The first reach of floodwall is supported by creosoted wood bearing piles. The second reach of floodwall, not on a spread footing foundation, is supported by concrete bearing piles. The third reach of floodwall is supported by timber piles. Refer to the spreadsheet listing in the General chapter for additional information in a sub-reach breakdown of floodwall segments. A modification was authorized in 1962. Under this authorization, the floodwalls were raised and buttresses were added from Station 102+73 to Station 166+25. The floodwall raises, at the time of construction, were considered the maximum that was possible without replacing the existing walls with new walls.

There are three existing openings in the CID-Kansas Unit line of protection that have closure structures. Two of these are timber stoplog closure structures across railroad track gaps (Kansas City Terminal Railroad Station 132+19 and Santa Fe Railroad Yards Station 168+00). The third is a sandbag closure at a railroad gap (Union Pacific Station 76+20). The nearby Missouri Pacific Railroad gap is high enough to not need a closure structure. A previously utilized closure structure at the James Street Bridge was eliminated with a bridge raise.

There are no retaining walls in the CID-Kansas Unit.

A-12.2.5 Central Industrial District-Missouri Unit

There are two concrete floodwalls in this unit. The two floodwalls are actually one wall separated by the Hannibal Bridge. The first floodwall begins at Station 0+00 and extends westerly to Station 22+32. This wall is an inverted cantilever T-wall supported on a spread footing foundation. From Station 11+94 to Station 22+32, the wall is founded on rock. The second wall begins at Station 22+82 and extends westerly to Station 78+12. The second wall is an inverted T-wall on concrete pile foundation.

The CID-Missouri Unit has eight openings with closure structures. All except one of the openings has timber stoplog closure structures. The other closure is a sandbag gap. Five of the eight closure structures are at railroad crossings and the other three are road crossings. For locations of these structures, see the spreadsheet listings in the General chapter of this appendix.

There are no retaining walls in the CID-Missouri Unit.

A-12.2.6 East Bottoms Unit

The East Bottoms Unit has one floodwall beginning at Station 57+14 that extends east, or downstream, to Station 74+56. This is an inverted cantilever concrete T-wall on a spread footing foundation. There are two additional short concrete floodwall sections that incorporate stoplog closure structures. These are from Station 472+55 to Station 473+35 and from Station 475+08 to Station 478+78.

The two openings mentioned above use timber stoplogs for closure and are at the Kansas City Terminal and Kansas City Southern Railroads, respectfully. There is one other closure structure in the East Bottoms Unit. This structure at the KCP&L Power Plant intake gates, Station 65+13, uses timber stoplogs for closure.

There are no retaining walls in the East Bottoms Unit.

A-12.2.7 Fairfax-Jersey Creek Unit

There are two floodwalls in this unit. The first reach of floodwall begins at Station 2+58 and extends upstream to Station 28+51. This wall is a concrete capped sheet pile I-wall. The second reach of floodwall begins at Station 287+86 and continues to Station 302+32. This wall is an inverted cantilever concrete T-wall on concrete pile foundation.

The Fairfax-Jersey Creek Unit has five active and two permanently closed closure structures. Three of the openings in the line of protection are at railroad tracks and utilize timber stoplogs for closure. There is an additional timber stoplog closure structure at a vehicular crossing. The fifth closure structure is a sandbag gap at the Union Pacific

Railroad tracks, Station 2+45. Refer to the spreadsheets in the General chapter for more detail on these closure structures.

There are no retaining walls in the Fairfax-Jersey Creek Unit.

A-12.2.8 North Kansas City-Airport Unit

The North Kansas City Airport Unit has one concrete floodwall, one concrete retaining wall and no closure structures. The walls are in the vicinity of the Broadway Bridge. The floodwall is an inverted T-wall supported on concrete piles that begins at Station 203+48 and continues to Station 208+82, with the last twelve feet extending into the levee. The retaining wall begins at Station 201+94 and continues to Station 203+48, the start of the floodwall. The retaining wall is a transition section from floodwall to levee. The retaining wall is required as there is insufficient room for a full levee section due to the proximity of the road to the line of protection. The road that circles the airport is located directly on the landside of the wall and pavement goes all the way to the stem of the wall. The wall consists of 4-40' long monoliths supported on concrete piles except for the short end at the levee section.

A-12.2.9 North Kansas City-Lower Unit

There are two significant floodwalls in this unit. They are located from Station 411+30 upstream to Station 412+55 and from Station 415+60 upstream to Station 417+42. Both of these walls are located along the Hillside Ditch. The walls are concrete inverted cantilever T-walls on spread footing foundations. In addition to these significant floodwalls, there are three minor floodwalls integrated with bridge headwalls along the Hillside Ditch at N. Cherry, N. Holmes, and Vernon Streets.

There are four openings with closure structures in the North Kansas City-Lower Unit. One closure structure located at Station13+17 is an aluminum stoplog gap at the Burlington-Northern Railroad tracks. There are two other aluminum stoplog closure structures. These are at Holmes Street (Station 463+33) and Cherry Street (Station 468+16). The fourth closure structure is a sandbag gap across Antioch Road (Station 421+90). Antioch Road is also Missouri State Highway 1 at this location.

There are no retaining walls in the North Kansas City-Lower Unit.

A-12.3 FLOODWALL ANALYSIS

A-12.3.1 General

An input requirement of the HEC-FDA program model is the reliability or probability of failure for levee sections with water at various elevations against the levee. Where floodwall exists as the line of protection instead of levees, the same information for floodwalls that was required of levees is needed for the model. Detailed risk and uncertainty analyses were done to determine the probability of failure of the floodwalls under existing conditions. The study was narrowed to the most likely failure modes using engineering judgment. The decision was made to consider only stability failure. Stability is a more likely mode of failure than structural capacity failure. Unfactored loads and unreduced strengths were used in the analysis. Factored loads and reduced strengths are used for design and are not appropriate for a probability of failure analysis.

A-12.3.2 Methodology

The economics model requires a probability of failure curve for varying water levels. No Corps of Engineers criteria were found that specifies the methodology for risk and uncertainty analysis of concrete structures. The method adopted for calculating a probability of failure is that outlined for geotechnical engineering in "Factors of Safety and Reliability in Geotechnical Engineering", by J. Michael Duncan, published in the Journal of Geotechnical and Geoenvironmental Engineering, April 2000. The use of this method provides consistency between the structural and geotechnical analyses. The method is described further in the Floodwalls on Spread Footing paragraph below. To produce a probability of failure curve, critical sections of each floodwall were analyzed (stability factor of safety determined) using strength and unit weight. Next, the soil parameters were varied to plus and minus one standard deviation from the mean one at a time and the factor of safety was recomputed. A Taylor series expansion was used to compute a probability of failure. If a probability of failure greater than 2% resulted, then the water elevation was lowered in 1-foot increments and the floodwall was reanalyzed until the probability of failure obtained was less than 2%. For purposes of floodwall analysis, the desire is to establish whether there is a 95% reliability of passing the flood. For a no probability of failure assumption, a slightly higher reliability is believed necessary. A 2% probability of failure was decided upon as an appropriate non-failure threshold.

A-12.3.3 Data Gathering

Soil parameters were obtained from geotechnical members of the project team. Various methods were used to obtain the floodwall data. The Operation & Maintenance (O&M) Manuals, Design Manuals, correspondence files, and old calculations (where available) were researched. Some dimensions were obtained by scaling the drawings. Site visits were made to confirm the wall dimensions, location, cross-section (where visible), and fill cover based on exposed stem height. Where data was unavailable, assumptions were made using engineering judgment for typical sizes, material strengths, and design and construction practices common to the time of construction. The walls were categorized by foundation type into walls founded on spread footing, sheet piling, and piles. In general, each different cross-section for a reach of floodwall was analyzed. This was done because it is difficult to ascertain by observation which cross-section, when there is more than one, is likely to have the highest probability of failure.

A-12.3.4 Floodwalls on Spread Footing

The failure modes looked at for these walls were sliding and overturning stability. Failure due to bearing was not analyzed since it normally does not control in the design of walls on spread footing foundations. The stability factor of safety of a floodwall is a function of the soil parameters. The uncertainty inherent in determining the soil parameters provides a means to find a probability of failure. From experience on the Missouri River Levee Project L-142 Criteria Study (KCD-COE), it was determined through analysis that the unit weight and the soil shear strength have a noticeable effect on a floodwall's factor of safety. Varying the concrete density has only a minor effect on the factor of safety. Geotechnical members of the project team provided a mean value and a standard deviation above and below the mean for both unit weight and soil shear strength for the soils present in the areas of the floodwalls. Using the mean and a plus and minus standard deviation for shear strength and moist unit weight, sliding factors of safety were computed (starting with the water at the top of the wall). Each floodwall cross-section was analyzed using the Corps of Engineers CASE project program CTWALL. This was done by varying one parameter at a time and analyzing each section under that condition. The computed sliding factors of safety were then used in a Taylor series reliability analysis, as outlined by Duncan, to get a probability of failure. A probability of failure was calculated for sliding stability only. Overturning was checked during the stability analysis, but does not have the well defined benchmark for failure that sliding has (a factor of safety of less than one defined for failure). Any location that overturning was shown to be a concern, sliding was also a concern. If the analysis indicated more than a 2% probability of failure, the water was lowered in one-foot increments and reanalyzed. This data was then used to develop a curve of probability of failure versus water elevation. Some of the floodwalls with large quantities of fill added on top of the design ground level required special considerations. This condition caused large factors of safety and, in some instances, large variances in factors of safety. When large variances are entered into the Taylor Series, an unacceptable probability of failure can result despite a very safe structure with a rather large factor of safety. A decision was made that if the factor of safety using mean values is greater than or equal to 2 and the factor of safety using the mean minus one standard deviation is greater than or equal to 1.5, it is not necessary to run the Taylor Series analysis. In these cases, a very low probability of failure was assigned to the cross-section. (A more refined reliability analysis procedure has since been developed that will be used in subsequent risk and uncertainty studies as discussed in paragraph A-12.7.)

A-12.3.5 Floodwalls on Piles

The failure of walls on piles was defined as a demand/capacity ratio of greater than one for the piles. Structural bending moment and pile axial load were calculated for water against the wall at different elevations. These were then compared to the moment capacity and axial capacity (pile skin friction and tip bearing) to compute a capacity/demand factor of safety. The pile capacities were calculated by varying the soil strengths similar to what was done for walls on spread footings. Geotechnical members of the study team provided the axial capacities, which were calculated using Lyman Reese's program APILE and what is referred to in the user's guide as the U.S. Army Corps Method. The calculated factors of safety were entered into a Taylor series reliability analysis similar to what was done for walls on spread footings to obtain a probability of failure curve. The same procedure was used for the moment capacity. For concrete piles, concrete compressive strength was varied as well as the soil parameters. The Corps of Engineers CASE project program CPGA was used to analyze the existing pile groups. CPGA required input of pile properties such as type of material (concrete, timber or steel), the shape (square or circular), strength of the material, cross-section and length. Some of the other inputs needed in the program were fixity of the piles, soil properties, compression capacity and tension capacity.

At times, CPGA's output displayed a pile failure, piles too short message. Hand calculations were performed to verify the output results from CPGA were correct and corresponded to the ultimate pile load, not the load at pile failure.

All walls on timber piles were found from the analyses to have a high probability of failure. As a result, it was not considered necessary to proceed with a strength analysis.

A-12.3.6 Floodwalls on Concrete Capped Sheet Piling

There is one concrete capped sheet pile I-wall that is located in the Fairfax-Jersey Creek Unit. The sheet pile wall was analyzed with CWALSHT, a Corps of Engineers CASE project program. The CWALSHT program is used to determine the required pile length with a design factor of safety for active and passive earth pressures. As the structure is existing, the factor of safety for passive earth pressures was varied to arrive at a factor of safety corresponding to the length of pile. A factor of safety of one was used for active earth pressures. The unit soil unit weight and shear strengths were varied to come up with the input for a Taylor series reliability analysis to achieve a probability of failure curve. No actual curve was developed, as the factor of safety calculated was 1.5 with mean values and water to the top of the wall. The resulting probability of failure was less than 2% (0.9%).

A-12.3.7 Results

The walls not discussed below were found to have no or a low probability of failure. It should be noted that a probability of failure analysis was not done for the entire wall system. Instead, the controlling (most critical) cross section was used to represent an entire reach of floodwall. It was difficult to tell by observation which sections were the most critical, so for all wall reaches, an analysis was made for each different type of cross-section (58 total) using what appeared to be the most critical location of each cross-section. A summary of the analysis showing probability of failure at the critical locations of each levee unit, by foundation type, is shown in Exhibit A-12.1. The Argentine floodwall between Stations 276+70 and 287+91 was found to have a 12.9% probability of failure with water to the top of the wall. The probability of failure of this wall reduces rapidly to 0.4% with water two feet below the top of the wall. The Armourdale wall between Stations 246+90 and 250+50 was found to have a very high probability of failure with water within three feet of the top of the wall. The Birmingham Unit wall at the Big Shoal Creek end of the unit was shown to have a 58% probability of failure with water to the top of the wall, reducing to 8% with water two feet from the top of the wall. This further reduced to 1% with water 3 feet below the top of the wall. The Birmingham Unit wall was constructed as part of the Northland Park development. This part of the line of protection is several feet higher than the original levee construction, so the levee would be overtopped before water reaches the top of the wall. The East Bottoms Unit floodwall between Station 64+48 and Station 74+56 was found to have an 8% probability of failure with water to the top of the wall, reducing to 0.8% with water one foot below the top of the wall. The CID-Kansas floodwall analysis for the sections on piles is inconclusive. Based on available information, the analysis of these walls indicated a high likelihood of failure or unsatisfactory performance before water reaches the top of the walls. No construction records have been found and it is felt that what is shown on the as-built drawings is not correct. Site visits have shown significant wall modifications (buttresses, stem extensions, etc.) which the original as-builts do not document. If the actual diameter and length of piles can be determined for these walls, there is a possibility that there could be a level of satisfactory performance expected. Otherwise, the walls are considered deficient. Non-destructive site testing is now under consideration to obtain and/or verify the actual lengths of the piles. A more detailed assessment of the all CID and Armourdale floodwalls will be performed under Phase 2 of this feasibility study.

Non-destructive testing was used to determine the piles' lengths and diameters supporting the Fairfax-Jersey Creek floodwall at the Board of Public Utilities power plant. Subsequent calculations revealed the potential for serious concerns about the walls reliability as water approaches the top of wall. An in-depth discussion of the BPU Floodwalls analysis and findings can be found in Fairfax-Jersey Creek portion of this appendix.

Analyses showed essentially no probability of failure for floodwalls in the North Kansas City-Airport and CID-Missouri Units before water would overtop the walls.

Site visits uncovered no visual observation of distress or major structural problems with any of the floodwalls or retaining walls. However, minor lateral displacement has occurred with the two short floodwalls along the Hillside Ditch in the North Kansas City-Lower Unit and the floodwalls in the Birmingham Unit. Exhibit A-12.2 shows the displacement at one of the North Kansas City floodwalls. Because this wall functions as a retaining wall, this minor displacement has little or not effect on the reliability of the wall to function as a floodwall.

The only other problem, captured during data gathering outside of the detailed engineering analysis, was the failure and subsequent repair of the floodwall at Station 40+00 in the CID-Kansas Unit. The CID-Kansas Unit will be more closely evaluated in Phase two of the feasibility study.

A-12.4 RETAINING WALL ANALYSIS

Retaining walls exist in the Armourdale and North Kansas City-Airport Levee Units. These walls were observed to be in good condition. There is approximately 428 linear feet of wall in the Armourdale Unit, in two sections. All of the Armourdale retaining walls are landside of floodwalls. No detailed structural analysis was made as they are not part of the line of protection and are not a concern of the existing conditions assessment. The retaining wall in the North Kansas City-Airport Unit consists of 4-40' long monoliths and is a transition section from floodwall to levee section. A preliminary analysis was made of the tallest section (wall stem height of 19') that has earth fill to the top of the wall on the riverside. Assuming no resisting force on the landside of the wall, the foundation pile capacity was found to be adequate. A very low probability of failure is assumed.

A-12.5 CLOSURE STRUCTURES

Closure structures consist of sandbag gaps and stoplog gaps. All of the stoplog structures are on spread footing foundation; there are no pile foundation stoplog gaps. No operational problems or structural problems have been observed or reported relating to the stoplog closures. As stated in the section on walls, a probability of failure was determined only when required for the economics model. Floodwalls were considered to have a greater probability of failure than the closure structures. Therefore, a risk and uncertainty analysis was not performed for closure structures. A probability of failure analysis would be similar to that used for the floodwalls. A probability of failure analysis would need to consider the reliability of stoplogs as well as stability performance. The structural performance of closure structures will be evaluated if modifications are planned for the adjacent floodwalls or levees.

A-12.6 SUMMARY

A literature and file search of data pertaining to design and construction of each levee unit was completed, followed by a site visit for visual verification and inspection. No visually observed or historical problems were noted other than minor movement of the floodwalls. These minor movements have occurred in the Hillside Ditch area of the North Kansas City-Lower Unit and in the Birmingham Unit. One wall failure and subsequent repair was noted at Station 40+00 in the CID Kansas Unit. In addition, a detailed probability of failure analysis was performed for floodwalls as required input into a HEC-FDA program model. The detailed probability of failure analysis indicated potential problems with pile supported floodwalls in the Fairfax-Jersey Creek and CID-Kansas Units. It also indicated potential problems with spread footing foundation walls at one location in the Armourdale Unit. Non-destructive testing is being considered to obtain better information, or verify the accuracy of the pile information used. The probability of failure analysis indicated potential problems with a floodwall at the Big Shoal Creek end of the Birmingham Unit line of protection, with water within the top two feet of wall. However, this is not considered a concern as this wall has been constructed higher than required and the levee should be overtopped before water reaches the top of the wall. No problems have been observed or reported with any of the closure structures or retaining walls.

A-12.7 EXISTING CONDITIONS ADDENDUM TO ARGENTINE ANALYSIS

While developing the Future Conditions for the Kansas Citys, Missouri and Kansas Flood Damage Reduction Feasibility Study, it was determined that additional information should be added to the discussion provided in this chapter for the Argentine Unit in an attempt to more accurately define the level of existing flood protection.

The Argentine Unit is located in Wyandotte County, Kansas on the right bank of the Kansas River between approximate Kansas River miles 10.1 and 4.75. The levee begins at Station 0+00 along the Barber Creek tieback and travels along the Santa Fe Railroad embankment to Station 29+02 where a stop log gap spans six Santa Fe railroad lines. Timber stop logs are used to close the railroad openings. Moving east, or downstream, the levee continues to Station 251+65 where a floodwall protecting the Argentine Boulevard Pump Station starts and then ends at Station 253+92. Earthen levee then continues to Station 276+70 where the second of Argentines two floodwalls extends east, adjacent to the Santa Fe Railroad tracks, to Station 287+92. Both walls are inverted cantilever T-walls on spread footing foundations. A second stop log closure structure continues to Station 288+57, crossing the same six lines of Santa Fe Railroad track and also using timber stop logs for closure.

Seventeen major gatewell closure structures are scattered along the length of the levee. Minor outlets with valve boxes associated with the pressure lines crossing the levee are addressed in the Civil Design chapter of this appendix. Four reinforced concrete box (RCB) culverts pass under the levee and service the pump plants at Stations

60+40 (Turner Station), 253+14 (Argentine Blvd.), 258+36 (Santa Fe Yards), and 273+41 (Strong Ave.).

A-12.7.1 Criteria

Stability Requirements. Structural stability criterion used in this addendum can be seen in Table A-12.1 below. It is based upon draft EM 1110-2-2100 – *Stability Analysis of Concrete Structures*, dated 30 May 2001, with the exception of the extreme load condition. There is some concern with the extreme load condition categories as specified in EM 1110-2-2100. The Missouri River L-142 Design Criteria Issue Resolution Paper (2002) addressed these issues and put forth more stringent guidelines for recommended extreme load condition stability criteria. That criterion is used herein.

TABLE A-12.1
Stability Criterion

Recommended Sliding Stability Factor of Safety		
Load Condition Category	**Return Period**	**Factor of Safety**
Usual	10 yrs	2
Unusual	300 yrs	1.5
Extreme	Top of Protection	1.3*

Recommended Rotational Stability Percent of Base in Compression		
Load Condition Category	**Return Period**	**Percent of Base in Compression**
Usual	10 yrs	100%
Unusual	300 yrs	75%
Extreme	Top of Protection	25% *

Recommended Maximum Allowable Bearing Capacity % Increase in Allowable Bearing Capacity		
Load Condition Category	**Return Period**	**% Increase in Allowable Bearing Capacity**
Usual	10 yrs	0%
Unusual	300 yrs	15%
Extreme	Top of Protection	50%

Recommended Flotation Stability Factor 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

* Stability requirements increased from value in draft EM 1110-2-2100

Strength Requirements. For new structures designed with the Strength Design Method, loads are increased by multiplying service loads by appropriate load factors and nominal strengths are decreased by appropriate strength reduction factors. Load factors required by EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, are a dead and live load factor of 1.7 and a hydraulic factor of 1.3. Combining these gives a total load factor of 2.2. The strength reduction factor for flexure, the typical controlling failure mechanism, is 0.90. Dividing the load factor by the strength reduction factor gives an overall factor of safety of about 2.45 for a new design.

Load and strength reduction factors were not used in the original analysis of the existing structures. This implies that if an existing structure has a calculated Factor of Safety of less then 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 than 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 load 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.

Uncertainty Analysis. For structures not meeting deterministic strength and stability criterion, a risk and uncertainty analysis was performed. A Taylor Series Method (TSM) of analysis is used in the calculation of structural risk and uncertainty. The TSM is appropriate when data is normally distributed, when parameters display a linear relationship, and when degradation over time is not a consideration. Because of the limited availability of data and with no information to suggest otherwise, an assumption of normal distributions for input data is reasonable and consistent with guidance provided in ETL 1110-2-547 (paragraph B-6.c). Examples of non-linear behavior for which the TSM should not be used include overturning stability analysis when the resultant is outside the kern of the base. Examples of degradation over time, which were not considered for the execution of this study, would include scour around piles, reactive concrete, sliding movement, and deteriorating drainage systems that affect uplift.

Risk Calculation. a. For strength calculations, uncertainty is measured by applying a mean and standard deviation to the concrete and steel strengths. The selected mean and normal standard deviation are based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

b. For stability calculations, uncertainty is considered by applying a mean and standard deviation to the soil unit weight and shear strength, and is based on values provided by the geotechnical engineers working on the study.

c. Failure is defined as the capacity to demand ratio (factor of safety) less than 1.0, or in other words, when the demand (loads) exceed the capacity (structural or geotechnical).

Material Properties. a. For the screening portion of the Kansas Citys Flood Damage Reduction Feasibility Study, the following structural properties will be used.

The American Concrete Institute recommended the use of a 3,000 psi concrete strength around the 1940's and 1950's, the typical timeframe of construction for most of the levee structures in these feasibility studies. Limited design documentation and as-built drawings have been discovered that support the 3,000 psi original design strength assumption. For earlier concrete strengths, little information exists. It is currently assumed that 2000 psi concrete strengths are appropriate. If additional research information is discovered, this value will be updated.

b. Knowing the time period of construction (~1940's – 1950's) and based upon the Portland Cement Association's pamphlet *Engineered Concrete Structures*, 1997, an assumed reinforcing steel design yield strength, F_y, of 40 ksi is used for most computations, unless known or stated otherwise. This number has also been verified in the limited original design documents that have been found. For earlier structures (~1900's), the Concrete Reinforcing Steel Institute in *Engineering Data Report 48* suggests 33 ksi steel is typical.

c. Based on FEMA 310, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength. For reinforced concrete structures Harr suggests a 14% standard deviation.

Concrete Strength Variation

1940's-1950's: $-\sigma = 3225$, $\mu = 3750$, $+\sigma = 4275$ (3000 psi min)
1900's-1920's: $-\sigma = 2150$, $\mu = 2500$, $+\sigma = 2850$ (2000 psi min)

Steel Strength Variation

1940's-1950's: $-\sigma = 43$, $\mu = 50$, $+\sigma = 57$ (40 ksi min)
1900's-1920's: $-\sigma = 35.5$, $\mu = 41.25$, $+\sigma = 47.0$ (33 ksi min)

A-12.7.2 Floodwalls

For the original Existing Conditions analysis, probability of failure calculations were performed for the floodwalls based on stability criteria checked using the Army Corps of Engineers CASE project program CTWALL. The structural probability of failure curve was based on the sliding stability of the floodwall at Station 276+70.

The existing floodwalls' strengths were checked using unfactored loads and unreduced strengths to determine the adequacy of the existing reinforcement to sustain the increased loading. A summary of results is displayed in Table A-12.2. The factor of safety based on sliding stability is less than that for strength for the floodwall at Station 276+70. As a result, it has been assumed that sliding stability controls the probability of failure. This is consistent with the assumption made in the original Existing Conditions analysis.

TABLE A-12.2
Floodwall Performance

Argentine Unit Floodwalls Extreme Condition (Water at Top of Wall)			
Criteria	**Required**	**Floodwall 251+56**	**Floodwall 276+70**
Sliding Stability	> 1.3 Factor of Safety	>> 2.0	1.5
Rotational Stability	> 25% Base in Compression	100%	38.35%
Bearing Pressure	< 150% Increase in Allowable Bearing Pressure	42%	110%
Strength	> 1.5 Factor of Safety for Existing Structure	1.58	1.68

A-12.7.3 Stop Log Gaps

Using the same stability criteria as required for floodwalls and described in Table A-12.3, the two stop log gaps at Stations 29+02 and 288+57 were also reviewed for strength and stability requirements.

TABLE A-12.3
Stop Log Gap Performance

Stop Log Gaps Extreme Condition (Water at Top of Wall)			
Criteria	Required	**Stop Log 29+02**	**Stop Log 288+57**
Sliding Stability	> 1.3 Factor of Safety	>> 2.0	>> 2.0
Rotational Stability	> 25% Base in Compression	100%	100%
Bearing Pressure	< 150% Increase in Allowable Bearing Pressure	40%	47%
Strength	> 1.5 Factor of Safety for Existing Structure	2.54	2.09

A-12.7.4 Gatewells

The seventeen gatewells along the Argentine unit were analyzed for uplift and strength requirements. Uplift Factors of Safety were calculated using Draft EM 1110-2-2100. Strength factors of safety are based on unfactored loads and unreduced strengths. Results are summarized in Table A-12.4. All values are based on the water at the top of structure.

TABLE A-12.4
Gatewell & Outlet Summary

Station	Exterior Dimensions (ft)	Pipe	Uplift Factor of Safety > 1.1 (Extreme)	Strength Factor of Safety > 1.5
35+10	6 x 6.5	36 " RCP	1.4	2.2
60+40	11.5 x 14	2 - 5' x 8' RCB	N/C*	N/C*
97+70	5.5 x 5.5	8" CIP	1.5	2.7
131+37	6.5 x 7.5	12" SP	N/C*	N/C*
131+50	8.5 x 9.25	48" RCP	1.3	2.2
145+00	6 x 6.5	36" RCP	1.4	2.2
190+00	7.83 x 10.33	60" RCP	1.6	2.5
218+17	7 x 7.33	36" RCP	1.2	2.4
247+32	5.5 x 5.5	24" CIP	1.5	3.06
253+14	13 x 14	9' x 9.5' RCB	1.1	1.9
258+36	6 x 12.5	4' x 5.5' RCB	N/C*	N/C*
273+41	10 x 12	7' x 7' RCB	1.3	2.8
280+48	6 x 6.5	36" RCP	1.4	2.2
284+35	6.75 x 14	10' x 10' Spillway	1.4	2.1
288+10	6 x 6	6" CIP	1.5	4.1

*Not Computed. Gatewell located on landside of levee.

Probabilities of failure were not computed for gatewell structures.

A-12.7.5 Reinforced Concrete Box Culverts

In the 1970's, the Argentine Unit was raised approximately 5 feet and the four box culverts were loaded with additional fill. No modifications were made to the boxes to accommodate the additional fill.

Reinforced Concrete Box Loading. The assumption was made that the critical load condition riverside of the closure sluice gate is not dependent on the water level and is only dependent on the height of earthen levee above each box. Boxes 253+14 and 273+ 41 have no flap gates, so as they are flooded there is equal water pressure on both the inside and outside of the box. It is assumed, and has been verified, that the flap gates on 60+40 and 258+36 will be closed only after the boxes have been submerged (verified with Larry Brennan of KVDD 7/22/04). This being the case, the boxes will operate with nearly equal water pressures inside and out during flood loading.

For the soil loading, EM 1110-2-2902 *Conduits, Culverts, and Pipes* (1997) defines two possible loading conditions for the buried Reinforced Concrete Boxes; either a trench condition or an embankment condition loading. The boxes appear to be originally constructed under embankment loading conditions. It was the opinion of several Army Corps of Engineer geotechnical engineers (in both the Kansas City and St Paul offices) that the loading stress would not dissipate with time and consolidation. It must be assumed that the RCBs are still experiencing an embankment loading condition from the initial levee construction. For the 1970's raise, an additional load was placed on the boxes. This additional loading was rationalized to represent a trench loading. Using this logic, the four boxes were analyzed using a superimposed loading approach. The initial levee fill is calculated as an embankment load in combination with the levee raise modeled as a trench load.

Reinforced Concrete Box Capacity. Each RCB was analyzed as three pieces: a roof member, a wall member, and a floor member. The Turner Station (60+40), Argentine (253+14), and Santa Fe Yards (258+36) RCBs all have continuous reinforcing steel at the corners and their members are modeled using fixed-end restraints. Calculations were performed to determine if sufficient positive moment (midspan) and negative moment (end) reinforcing was present in the boxes to carry the loadings required to meet fixed end conditions. If there was insufficient negative moment reinforcing to carry the load, it was assumed the corner reinforcing steel would yield but still carry load. After the negative reinforcement yields, the ends will continue to carry their maximum yield moment until a third plastic hinge forms at the midspan and the member fails. The overall factor of safety for a member with yielded end-restraints was then based on the capacity of the midspan steel to carry additional load. The Strong Avenue (273+41) RCB does not have continuous steel at the corners and was analyzed using a pinned restraint condition. Additional capacity from negative steel yielding was not included.

Probability of Failure. A Taylor series expansion was used for the computation of probabilities of failure based on either varying concrete and steel strengths or varying soil moist unit weights and shear strengths. Based on research presented by Milton E. Harr in his book *Reliability Based Design in Civil Engineering,* a coefficient of variation for Reinforced Concrete Grade 40 of 14% is typical. It should be noted that the design steel strength is a minimum yield strength, not a mean value. To obtain an average value it was assumed that actual steel strengths varied at least 1 standard of deviation from the minimum required strength. Soil parameters and their variations were supplied by the geotechnical members of the project team. Mean, upper, and lower bound values are summarized in Table A-12.5 below.

TABLE A-12.5
Probability of Failure Values

	Lower Boundary	Mean	Upper Boundary
Soil Unit Weight (pcf)	106	115	124
Soil Shear Strength (Degrees)	26.7	29.9	33.1
Steel Yield Strength, f_y (ksi)	40.0	45.6	51.2
Concrete Compressive Strength, $f`_c$ (psi)	3225	3750	4275

A comparison of the resulting probabilities of failure for varying concrete and steel strengths versus soil moist unit weights and shear strengths showed varying material properties had a greater effect then varying soil properties on the probability of failure. Consequently, varying material strengths were used in the calculation of box culvert probabilities of failure.

Results. Table A-12.6 summarizes the results of the flexural strength analysis of the four RCB's in the Argentine Unit. The range of risks and factors of safety for the Strong Avenue RCB illustrates the possible loading conditions. Analyzed under the combined embankment and trench loading produces a factor of safety of 0.75. If the entire fill above the box is modeled as a trench load, the resulting factor of safety becomes 1.07. For either loading, the probability of failure is unacceptable and modifications are required.

TABLE A-12.6
RCB Summary

Name / Station	Description	Risk of Failure (Existing Conditions)	Factor of Safety (Existing Conditions)	Loading Condition	Restraint Conditions	Comments
Turner Station 60+40	Twin 5' x 8' RCB	N/A*	2.21	Combined Trench and Embankment Loading	Yielding	Field observations on 07-April-04 collaborate the overall functionality of the RCB. A significant depression noted just riverside of the sluice gate must be monitored.
Argentine 253+14	9.5' x 9' RCB	N/A*	2.32	Combined Trench and Embankment Loading	Yielding	Field observations on 07-April-04 collaborate the overall functionality of RCB. Significant concrete patches applied over approximately 100 ft of box interior.
Santa Fe Yards 258+36	4' x 5' RCB	N/A*	5.00	Combined Trench and Embankment Loading	Yielding	No field observations attempted due to high Factor of Safety.
Strong Ave. 273+41	7' x 7' RCB	26.9 % - 99.5 %**	1.07 - 0.75**	Trench Loading – Combined Loading	Pinned	Limited observations on 07-April-04 due to sewer diversion.

* Risk Analysis Not Performed. For Factor's of Safety above 2.0 Probabilities of Failure are minimal.
** Range in Risk due to possible range of soil loading conditions.

As the probability of failure for each RCB is independent of the water elevation, the Strong Avenue RCB probability of failure curve will not be input into the economic model for potential damages. The estimated cost of strengthening will be included in the economic model.

RCB 273+41 Modification. Based on the strength analysis, the Strong Avenue RCB requires flexural strengthening. 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).

A-12.8 SUPPLEMENTAL EXHIBITS

EXHIBIT A-12.1
Significant Probability of Failure By Levee Unit

Spread Footing Founded Floodwalls

Levee Unit	River	River Mile	Station Range	Elevation of Top of Floodwall	Probability of Failure at Distances Below Top of Wall						
					Top	1 ft. below	2 ft. below	3 ft. below	4 ft. below	5 ft. below	6 ft. below
Argentine	Kansas River	4.376	276+70 to 287+91	771.0	12.9%	2.6%	0.4%				
Armourdale	Kansas River	2.067	60+30 to 77+78	764.1	94.3%	71.7%	27.4%	3.2%	0.1%		
	Kansas River	2.044	246+90 to 250+50	763.9	99.8%	99.5%	93.0%	52.8%	7.7%	0.2%	
Birmingham	Big Shoal Creek	6.324	558+05 to 558+50	752.6	58.0%	27.9%	7.8%	1.0%			
East Bottoms	Missouri River	364.391	64+48 to 74+56	755.95	7.9%	0.8%					

Pile Founded Floodwalls

Levee Unit	River	River Mile	Station Range	Elevation of Top of Floodwall	Probability of Failure at Distances Below Top of Wall						
					Top	1 ft. below	2 ft. below	3 ft. below	4 ft. below	5 ft. below	6 ft. below
Fairfax/Jersey Creek	Missouri River	373.417	287+86 to 302+32	765.8	51.6%	48.2%	39.2%	31.0%	33.1%	0.1%	
CID-Kansas	Kansas River	0.544	26+73 to 40+31	760.8	72.1%	63.2%	54.1%	42.8%	39.2%	57.0%	52.8%

All of these numbers are PRELIMINARY.

EXHIBIT A-12.2
Movement at a Floodwall Monolith Joint in the North Kansas City-Lower Unit