Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study

Engineering Appendix to the Final Feasibility Study

Chapter A-15

CID-MO Structural Analysis

Chapter A-15
CID-MO Structural Analysis

Table of Contents

A-15.1 Overview 1
A-15.2 Assumptions 1
A-15.3 Soil Material Properties 2
A-15.4 CID-MO Existing Conditions 3
A-15.4.1 Floodwalls Existing Conditions 3
A-15.4.1.1 Floodwall Existing Conditions Summary 4
A-15.4.2 Floodwall Structures Remediation 5
A-15.4.3 Drainage Structures Existing Conditions 5
A-15.4.4 Closure Structures Existing Conditions 7
A-15.5 RCBs and Pipes Associated with Gatewells 9
A-15.6 References: 9

LIST OF TABLES

Table A-15-1: CID-MO Soil Properties 3
Table A-15-2: CID-MO Existing Spread Footing Floodwall Conditions 5
Table A-15-3: CID-MO Existing Pile Footing Floodwall Conditions 5
Table A-15-4: CID-MO Existing Gatewell Conditions 7
Table A-15-5: CID-MO Closure Summary of Results for Existing Conditions…… 9

Chapter A-15
CID-MO Structural Analysis

A-15.1 Overview

This chapter addresses the structural features (floodwalls, gatewells, and closure structures) for the Missouri Central Industrial District (CID-MO) unit which includes existing conditions only because a raise is not being pursued at this time. The existing condition evaluation includes assigning reliability to the structures and recommending remedies if required. Elevations reference the NGVD29 datum as information was obtained from record drawings. For general structural methodology refer to the general structural chapter.

A-15.2 Assumptions

See the General Structural chapter for general KC Levees Structural Feasibility Assumptions. The following assumptions are in addition to those general assumptions, applying specifically to the CID Missouri Unit.

1. To assess flexible behavior of a closure's pile cap and localized load effects on piles, a simplified torsional analysis is considered sufficient for the current scope and budget. A finite element model is required for more precision, and confidence in decisions regarding the reliability of the structures. A maximum width of 4 ft, centered about each abutment, was selected due to pile spacing, concentrated loads at the abutment, minimum horizontal steel, and the absence of torsion reinforcement. Using a 4 ft width assumes rotation so that torsion is minimized (note, torsion reliability, with respect to ¼ τ_{cr}, was less than 99.8% for water at top of the wall). The abutment width is only 2.5 ft wide, and placement of primary reinforcement was limited to within the abutment width. Considering sill heights, footer thicknesses, and that torsional reliability was near 99%; however, distribution of stresses beyond the 2.5 ft width is reasonable.
2. Due to smaller geotechnical strengths, only the long-term load case of the seemingly critical closure and floodwall was analyzed with consideration of pile strengths.
3. The footer's heel and toe maximum moments occur at the stem.
4. Loads are capable of distributing throughout the pilecap for a width of 4 ft, centered about the abutments. It is noted that the rebar detailing has primary steel only under abutment (typically 2 ft 6 in). USACE program CPGA was used to analyze the pile forces, which assumes a rigid cap.
5. The pile cap under the closure width is adequate for strength and in good condition.
6. For a consistent risk and economic comparison with other units' T-wall results, axial tension in the heel of T-walls was not analyzed, shear and flexure in the floodwall keys were not analyzed, and shear friction at construction joints with only residual rebar capacity not used to resist bending was not checked.
7. Heels of T-walls were analyzed without contribution of key forces producing a pure moment into the heel. This moment is usually small in comparison with heel forces. Also, the heel was analyzed for net vertical pressures only.

8. Based on closure structure analysis, the following checks were not performed on the floodwalls because they proved not to control for the closures, therefore likely not control for the floodwalls: tension pile embedment, punching shear of pile through pile-cap, vertical and horizontal bearing of pile and pile-cap, spread footer stability with short-term strengths, horizontal shear in pile-cap (at sheetpile) and horizontal shear in all piles.

9. For a consistent risk and economic comparison of gate well stability calculations with other units' results, a portion of soil skin friction is assumed aiding in the resistance of flotation. A crack is assumed in the top five feet of soil, which eliminates any contribution of the soil in resisting uplift.

A-15.3 Soil Material Properties

The expected soil properties used for the CID-MO calculations are located in Table A-15-1 These values were used for general soil properties in the structural calculations. The allowable bearing capacity values were used in the structural calculations and was arrived at by dividing the Ultimate Bearing Capacity from the table below by the factor of safety listed. Soil to structure friction and cohesion interaction was typically neglected for stability and strength calculations. However, for gatewells and spread footing founded floodwalls, this behavior was considered under geotechnical guidance.

	Embankment	Fill/Debris	Foundation Blanket	Foundation Sand
Friction Angle (drained shear strength)	29°	20°	22°	30°
Cohesion	-	-	-	-
Moist unit wt.	115pcf	110pcf	110pcf	115pcf
Saturated unit wt.	120pcf	115pcf	115pcf	120pcf
Wall friction coefficient (used in gatwell uplift)	0.25	0.25	0.25	0.25
Ultimate Bearing Capacity – compression (psf) FS=1.7 (Extreme Loading Condition)			3,400 (Sta 0+00 to 7+00) 3,610 (Sta 7+00 to 7+50) 3,663 (Sta 7+50 to 10+25) 40,000 (Sta 10+25 to 23+00)	

Table A-15-1: CID-MO Soil Properties

A-15.4 CID-MO Existing Conditions

A-15.4.1 Floodwalls Existing Conditions

Floodwalls on the CID-MO unit consist of spread footing floodwalls and pile founded floodwalls. For each of these types of walls the existing condition was evaluated. The evaluation entailed analysis of existing reliability and recommending solutions for any existing condition deficiencies. See the structural analysis appendix chapter for further information on reliability analysis.

The existing CID-MO floodwalls are as follows:

- Sta. 0+00 to 22+31.71 spread footing floodwall constructed in 1947.
- Sta. 22+81.46 to 78+12.22 pile foundation floodwall constructed in 1947.

This does not include walls that are part of a closure structure (see closure structures section). The floodwall is interrupted in locations where the wall ties into bridge piers or roadway ramp. The floodwall ties into existing ground at Sta. 0+00 and ties into an earthen levee at Sta. 78+12.22.

The soil elevations both landside and riverside of the wall for the majority of the walls were field verified and where there were differences, the field noted elevations were used in the analysis.

Clarification to pile details on the drawings - The record drawing (A-10-825) shows only 18” square piles with and without tapers and the precast concrete sheet pile section. The last revision date on that sheet is 6/23/1947. Another drawing that apparently preceeded the record drawing shows 16” and 18” square piles without tapers and the precast concrete sheet pile section. All other drawings in the record drawings (as builts) show that there are 16” and 18” piles that were used – including the driven pile lengths listed on record drawing A-10-1178. So the following assumptions were made for the feasibility analysis:

1. 16” piles not tapered and as detailed
2. All 18” piles tapered – looking at 21’ long pile (each type wall has at least 1 monolith with the 21’ long piles) (therefore used 15”x15” dimensions for properties –because of taper)
3. used the precast sheet pile as detailed on the record drawings

The controlling probability of failure results for the spread footing walls is based on the controlling walls; “12ft” wall for strength reliability (wall between Sta 0+00 to 3+49.16) and “Especial” wall (wall between Sta. 20+23.77 and 20+65.77)for sliding reliability. The controlling probability of failure results for the pile footing walls is based on the wall “R” (wall between Sta. 22+81.46 to 24+54.76) which controlled for pile geotechnical compression capacity. The pile capacities changed very little as the water level lowered down the wall, so the probability of failure analysis assumed the same pile capacity at each location.

A-15.4.1.1 Floodwall Existing Conditions Summary

CID-MO FLOODWALLS - Spread footing Floodwalls Existing Conditions with Water to Top of Wall							
		Factors of Safety					
Station	Wall Type	OVERTURNING % Base in Compression **(25% Req'd)**	BEARING **(2.0 Req'd)**	SLIDING **(1.3 Req'd)**	STRENGTH Wall/ Heel/Toe **(1.5 Req'd)**	Mechanism	Comments
0+00.00 to 3+49.16	**12'**	72%	3.2	1.1	**0.9**	**Flexural Bottom Steel in Toe**	**86% reliability for strength calculated**
4+09.81 to 7+03.97	11'	83%	3.3	1.0	1.6	Flexural Bottom Steel in Toe	
7+03.97 to 7+45.97	13'	100%	3.3	2.6	1.1	Flexural Top Steel in Heel	
7+45.97 to 8+29.97	15'	95%	2.8	1.4	1.1	Flexural Top Steel in Heel	
8+29.97 to 11+93.90	16'	68%	2.2	0.9	1.1	Flexural Top Steel in Heel	
11+93.90 to 13+61.90	L (Monolith 29)	90%	21.6	2.5	2.0	Flexural Bottom Steel in Toe	
13+61.90 to 14+45.90	M (Monolith 31)	100%	22.3	3.3	2.2	Flexural Bottom Steel in Toe	
15+19.77 to 16+87.77	J (Monolith 34)	77%	19.3	2.4	1.8	Flexural Bottom Steel in Toe	
16+87.77 to 18+13.77	H (Monolith 38)	80%	22.4	2.2	1.9	Flexural Bottom Steel in Toe	
18+13.77 to 18+97.77	G (Monolith 40)	61%	20.4	1.7	1.9	Flexural Bottom Steel in Toe	
18+97.77 to 19+39.77	F	33%	12.1	1.6	1.5	Flexural Bottom Steel in Toe	
19+39.77 to 20+23.77	Fspecial	0%	0.0	0.4	2.2	Flexural Riverside Steel in Stem	
20+23.77 to 20+65.77	**Especial**	**0%**	**0.0**	**0.0**	**2.0**	**Flexural Bottom Steel in Toe**	**0% reliability for sliding**
20+65.77 to 22+31.71	E (Monolith 47)	0%	0.0	0.7	1.6	Flexural Bottom Steel in Toe	

Table A-15-2: CID-MO Existing Spread Footing Floodwall Conditions

CID-MO FLOODWALLS - Pile Foundation Floodwalls - Existing Conditions with Water to Top of Wall								
	Factors of Safety							Comments
		Concrete Sheet Piling			Concrete Pile			
Wall Type	STRENGTH Wall/ Pile Cap **(1.5 Req'd)**	STRENGTH **(1.5 Req'd)**	Geotech allowable Axial Compression and Tension loads **(1.0 required)**		STRENGTH **(1.5 Req'd)**	Geotech allowable Axial Compression and Tension loads **(1.0 required)**		
R	1.4	5	9.1	tension	11.4	0.94	compression	**96% reliability calculated for concrete pile geotechnical compression capacity**
Q	2.0	2.2	2.7	tension	4.0	1.23	compression	
Q-Monolith 54	1.2	2.4	1.2	tension	4.3	0.90	compression	
P	2.1	7	12.4	tension	7.4	1.54	compression	
N	2.2	2.2	3.4	tension	5.1	1.01	compression	
S	1.0	3	14.7	tension	19.4	1.46	compression	93% reliability calculated for heel strength

Table A-15-3: CID-MO Existing Pile Footing Floodwall Conditions

A-15.4.2 Floodwall Structures Remediation

As a result of the findings noted in the floodwall existing conditions analysis, repair is recommended for approximately 292-feet of flood wall (between Sta 19+39.77 and 22+31.71). A preliminary recommended repair may be to:

1. reconstruct the heel key by demolishing the end of the heel at the existing unreinforced key
2. extending the heel key approximately 1-foot deeper than the existing key depth (total key depth of 2-feet) and reinforcing the key for E Special and F special walls
3. extending the heel approximately 2 feet riverward and constructing a new reinforced key approximately 4-feet.

This repair will allow the key to be considered for sliding resistance for the stability of the floodwall.

The 12-foot spread footing floodwall has a reliability of 69%. The analysis was based upon the maximum exposed stem height based upon field verified measurements. Much of the 12-foot wall has much less exposed stem since it is near the end of the protection where the groundline tapers up to the top of the wall (see record drawings). That coupled with a reasonably high reliability when water is to the top of the wall and the fact that once water is 1-foot down from the top of the wall, reliability is 98%, results in low risk of failure of the existing 12-foot spread footing floodwall. Similar results can be expected at the other walls with a low strength factor of safety. Some additional measures may be needed to address sliding in the area from Sta. 0+00 to 11+93, but they are expected to be minimal in nature.

A-15.4.3 Drainage Structures Existing Conditions

The structural section of the Engineering Appendix for the Kansas City, Missouri and Kansas City, Kansas Flood Protection Project contains an evaluation of the existing gatewells' abilities to meet Nominal 500 yr flood event. The three CID-MO gatewells and one manhole were analyzed with water to top of levee (worst case) only to determine their reliability for strength and flotation. The results of this analysis are shown in Table A-15-4.

The three gatewells and one manhole along the CID-MO unit were analyzed for flotation and strength requirements to determine if the gatewells could withstand water seepage pressures (HGL's) with water to top of flood walls. Results are summarized in Table A-15.4. All values are based on the nominal 500 yr or water to top of levee (worst case).

Existing Conditions with Water to Top of Protection HGL's From Geotech May 2011					
Station	Flotation Factor of Safety (>1.1 Req'd)	Strength Factor of Safety (>1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability (%)	Comment
14+97 (GW)	N/A	1.4	Wall Bending	99.86	The bottom of GW is the floodwall toe, therefore uplift is not analyzed. The floodwall stability is analyzed seperately.
14+99 (MH)	1.9	16.1	Base Slab Moment	N/A	MH is 38.5' upstream of the gatewell, Line of Creep Method used for uplift pressures.
24+76.9	1.3	2.0	Wall Bending	N/A	HGL change. Gatewell is attached to Broadway pump station. Uplift not a concern.
41+84.9	1.3	2.9	Base Slab Moment	N/A	No Change from previous HGL estimate. Gatewell atatched to floodwall, uplift not a concern.
52+85	N/A	N/A	N/A	N/A	Gatewell removed and rebuilt in Santa Fe Pump Station (Sheet #7, A-10 -1617) Gatewell should be analyzed in unison with the Pump station.

Table A-15-4: CID-MO Existing Gatewell Conditions

Gatewell at Sta. 14+97 showed a 1.4 factor of safety for wall flexure, slightly less than the minimum requirement of 1.5. A probability of failure (POF) analysis was conducted which resulted in the wall being 99.86% reliable for strength. Flotation was not analyzed for this gatewell because the bottom of the gatewell is the toe of the floodwall. The floodwall stability was analyzed separately..

A-15.4.4 Closure Structures Existing Conditions

There were seven closure structures analyzed for the feasibility study: 1) M10 (STA 5+23.95), 2) M18 (STA 8+67.90), 3) M32 (STA 14+80.09), 4) M145 (STA 63+15.28), 5) M 158 (STA 68+90.14), 6) M162 (STA 70+70.66), and 7) a levee closure (STA 80+18.62). The FS results from the deterministic analysis of key structural components are summarized in the table below. Pertaining to pile-founded closures (M145, M158, M162, and the Levee Closure), considering a simplified approach to qualify torsion, the geotechnical pullout strength of the tension piles (precast sheetpile) was critical. Reliability is 36% with water elevation at Tow of Wall (TOW); 85% with water elevation at 1' below TOW; and 99.5% with water elevation at 2' below TOW. With water elevation below 2 ft from TOW, the reliability is considered 99.8%. M145 was the most critical of all the pile-founded monoliths, and its results were used to establish POF plots vs water elevation for all closures.

Since the closures were not found 99.8% reliable, without adjustment for structure degradation, a review in original design assumptions was performed. Like the present analysis, it seems that the original designers included water level to top of the wall. Unlike the present analysis, the original designers apparently only used undrained geotechnical strengths for the pile capacities (whereas the feasibility study included lower drained strengths), considered the closure's pile caps as rigid (whereas torsion was qualified in the feasibility study), but very conservatively ignored any efficiency of the cutoff wall to mitigate seepage and uplift pressures (whereas the cutoff wall efficiency was assumed 50% in the feasibility study).

Setting aside remediation measures that may be borne out of the results from the feasibility analysis, the following recommendations are made in light of the site visit and analysis:

1. Most closures need maintenance at the abutments, including cleaning and painting embedded steel.
2. If possible, inspect pile caps for cracks and corrosion of rebar, and repair if necessary.
3. Inspect timber stop logs and repair/replace if needed.
4. Seal cracks and joints with appropriate sealant for exposed conditions and movement of concrete.
5. Should a finite element model, or other more detailed analysis, be performed, it is recommended that a 5% to 15% reduction in strength capacity be applied given the observations made during the site visit. Site visit observations can be found in the appendix with closure structure calculations.

Table A-15-5: CID-MO Closure Summary of Results for Existing Conditions - Summary of Probable Failure Mode Results[1]

Monolith	Heel Trsv Moment at Stem FS[2]	Heel Trsv Shear (stem or cntr pile)	Pile Cap Heel Long Moment at Cntr Pile (Pos or Neg)[4]	Stem Moment at Base[5]	Stem Shear at base	Toe Trsv Moment at Stem[8]	Toe Trsv Shear (stem or pile)	Tension Pile's Rebar development into Pile Cap	Max Compressive Pile Punching Shear	Critical Pile Bearing onto Cap (Loaded Vert or Horz)	Cutoff Pile Horz Shear through Pile Cap	Cutoff Pile Geotechnical Pullout	Cutoff Pile Combined Axial and Moment	Shear through Cutoff Pile	Max Compr Pile Geotech Bearing Capacity	Max Compr Pile Combined Axial and Moment	Shear through Square Pile	Spreadfooter Key Moment	Spreadfooter Key Shear	Guide Post Moment at Top of Footer	Guide Post Shear at Top of Footer	Guide Post Moment at Sill EL	Guide Post Shear at Sill EL	Postwell Strength Anaplsis[9]	Interior Post Strength (shear or moment)	Stop logs Moment	Stop logs Shear	Stop logs bearing	Sill Strength (shear or moment)
M10	2.01	8.13	NA	2.67	2.89	1.76	6.01	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	2.55	6.28	1.70	2.51	2.30	5.33	NA	NA	CMLevee	CM32	18.20	4.96
M18	1.62	6.32	NA	2.51	1.50	1.59	5.94	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	3.10	5.77	2.48	1.55	7.80	4.70	NA	NA	CMLevee	CM32	18.20	2.11
M32	3.16	5.45	NA	2.78	2.15	1.36	4.36	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	2.00	1.94	1.60	2.89	NA	NA	CMLevee	4.00	18.20	4.20
M145	1.09	1.39	2.25	2.50	1.55	3.11	1.77	5.75	2.80	13.35	1.57	**0.96**	1.07	1.22	2.23	1.90	1.79	NA	NA	2.30	2.59	NA	NA	NA	NA	CMLevee	CM32	18.20	3.13
M158	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	NA	NA	2.48	1.57	3.30	2.75	NA	NA	CMLevee	CM32	18.20	3.50
M162	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	NA	NA	2.90	1.67	6.90	4.56	1.67	1.98	CMLevee	CM32	18.20	2.99
Mlevee	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	NA	NA	2.10	2.54	5.10	5.94	NA	NA	1.81	CM32	18.20	4.86

Pile Cap Torsion Assessment[3]

Monolith	FS	Reliability (at Water EL) TOW	TOW-1ft
M145	0.95	79.7%	99.8%
M158	1.05	98.9%	99.8%
M162	0.94	75.2%	99.8%
Mlevee	1.37	99.8%	

Spread Footer Torsion Assessment[3]

Monolith	FS	Reliability (at Water EL) TOW
M10	1.4	99.8%
M18	3.0	CM10
M32	3.6	CM10

Spread Footing Stability Results (Existing Condition with Water to Top of Wall)[6]:

Monolith	FS Sliding	Req FS	% Base Compressed	Req %	FS Bearing	Req FS
M10 (drained Strengths)	1.35	1.3	96%	25%	3.2	2
M10 (undrained Strengths)	5.89	1.3	79%	25%	3.2	2
M18 (drained Strengths)	2.84	1.3	100%	25%	2.62	2
M18 (undrained Strengths)	18.4	1.3	100%	25%	2.69	2
M32 (drained Strengths)	1.59	1.3	100%	25%	21.5	2
M32 (undrained Strenghts were judged from inspection to not control)						

Risk and Reliability Results:

The single most critical component of the gaps (lowest FS and highest COV) was the geotechnical pullout of the cutoff wall piles.

Cutoff Wall Pullout Reliability Results (based upon M145 only)

	TOW[10]	TOW-1ft	TOW-2ft	TOW-3ft
M145	36.4%	84.8%	99.5%	99.8%

Notes:

1 Some failure mechanisms are judged to be critical on one or two closures. For these mechanisms, a designation of CMxxx is used to refer the reader to the results of the critical monolith. Analysis was not performed for non-critical mechanisms.
2 FS reported are for water EL at Top of Wall. For Factors of Safety greater than 1.5, a reliability of 99.8% is assigned.
3 FS and Reliability is computed with respect to a capacity equal to 1/4 of cracking Torsion.
4 Longitudinal Shear in pile cap across center row of piles is adequate by inspection (by comparing to capacity in primary load direction).
5 The stem's Temperature and Shrinkage reinforcement is only 74% of current USACE criteria.
6 A 2D analysis in CTWALL was performed for results. Rotation in Plan due to eccentric loading on M18 and M32 (Z shape), was found to posit a minor stress on neighboring walls should shear resistance be exceeded by footer.
7 Strength FS for spread footer gaps is critical of drained and undrained results.
8 The stem at M32 has an abrupt change in cross-sectional area. Flexure was checked at this height and a FS was found to be 2.2.
9 Considers critical result from flexure, shear resistance from shear friction and concrete, and bearing.
10 TOW elevation for M145 is 759.9 ft.

A-15.5 RCBs and Pipes Associated with Gatewells

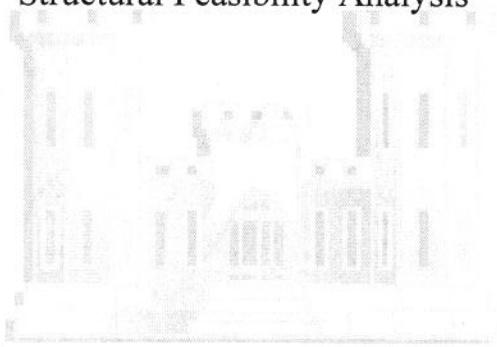
There was no analysis done on the CID-MO pipes associated with the gatewells because there is no future conditions analysis (raise) for this unit. Existing pipes are assumed to be functioning at current state. Inspections and maintenance will be required for upkeep to system. The levee sponsor is responsible for maintaining all structures per the O&M (Operation and Maintenance) Manual.

A-15.6 References:

1. CID-MO Plans for Construction of Broadway and Santa Fe Pumping Plants (October 1947)
2. Record Drawings; Appendix I Record Drawings; Operations and Maintenance Manual; Kansas Citys Flood Control Project; Missouri and Kansas Rivers; Central Industrial Unit – Missouri Section; Sheet dates vary
3. Computations; Central Industrial District – Mo. Section Design Computations, dated 1946.

KC Levees
CID MO Closures

Structural Feasibility Analysis



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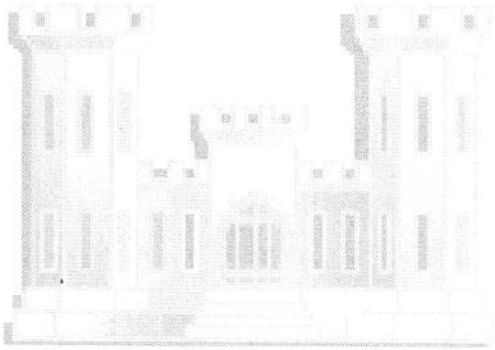
Order of Appearance

	Title	Page
1.	Summary	3
2.	Site Visit	8
3.	Pile Cap Analysis	12
4.	Spread Footer Analysis	176
5.	Load and Capacity Analysis for other Structural Components	280
6.	Geotechnical Referenced Data	379
7.	Incomplete GTSTRUDL Analysis	401

SUMMARY



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KC Levees Feasibility Study

CID MO – Closure Analysis Summary

This document is a summary of findings, assumptions, and recommendations for the KC Levees, CID MO Closures.

The work contained herein is to provide Feasibility Analysis results only; it does not replace a deterministic analysis, or answer the questions that only a deterministic analysis can. The closures were analyzed, without factors of safety and with consistent assumptions, in order to evaluate the relative risk and consequences for economic and risk-informed decision-making purposes. Risk and Reliability studies do not replace deterministic analyses, nor do such studies confirm the satisfaction of any design criteria, past or present. They simply provide additional information for the decision-maker with respect to the possible performance of the structure for the loads under consideration. This provides a risk-informed decision with respect to project repairs or improvements.

Assumptions to Analysis.

The following lists major assumptions in the feasibility study.

1. The closures were analyzed based on dimensions, quantities, and conditions represented by record drawings. Deviations from plans cannot be verified per scope and budget. This is a consistent assumption for relative risk and reliability assessment for KC Levees feasibility study. A site visit was conducted, but its findings are limited to observable deficiencies above ground. In general, the condition of the closures is fair. Some significant cracking, separation, and rotation have been observed which may impact the closure's ability to perform in a peak flood.
2. The risk analysis reflects a structure in the condition as shown on plans, or during an inspection, at the time of the extreme (water to top of flood protection) flood. Secondary and incremental load effects are not considered for the feasibility analysis. Permanent deformation or damages from any less than extreme floods that occur prior to the extreme flood are expected to have been repaired.
3. Construction practices are considered good, and all specifications noted on plans satisfied.
4. Materials, such as reinforcing bar, are in good condition.
5. Soil profiles are represented accurately by drawings. Soil is adequately compacted and fill type and strength parameters supplied by Geotechnical Section is correct.
6. Concrete compression zone is without voids.
7. Some structural components were not analyzed. Only components judged to be critical based upon engineering experience were analyzed for feasibility. Components that were not analyzed include, but may not be limited to, minimum rebar embedment lengths, structure capacity at rebar cutoff locations, various cut-off wall efficiencies, and various soil resistances.

8. Cut-off walls are 50% efficient, and are of adequate strength and good condition. The upstream face of structural wedges was analyzed with a Line of Creep, reduction beginning at top of ground. This is the default method in CTWALL.
9. The Sand Bag Closure (STA 1+53.14) requires no Structural Analysis.
10. To assess flexible behavior of a closure's pile cap and localized load effects on piles, a simplified torsional analysis is considered sufficient for the current scope and budget. A finite element model is required for more precision, and confidence in decisions regarding the reliability of the structures. A maximum width of 4 ft, centered about each abutment, was selected due to pile spacing, concentrated loads at the abutment, minimum horizontal steel, and the absence of torsion reinforcement. Using a 4 ft width assumes rotation so that torsion is minimized (note, torsion reliability, with respect to ¼ τ_{cr}, was less than 99.8% for water at top of the wall). The abutment width is only 2.5 ft wide, and placement of primary reinforcement was limited to within the abutment width. Considering sill heights, footer thicknesses, and that torsional reliability was near 99%; however, distribution of stresses beyond the 2.5 ft width is reasonable.
11. For Risk and Reliability analysis, the strength of structural materials has an expected value of 125% of their design strength, based upon FEMA 310. Structural materials are assumed to have a 14% standard deviation. Geotechnical materials are assumed to have an 18% standard deviation.
12. Due to smaller geotechnical strengths, only the long-term load case of the seemingly critical closure was analyzed with consideration of pile strengths.
13. The footer's heel and toe maximum moments occur at the stem.
14. Loads are capable of distributing throughout the pilecap for a width of 4 ft, centered about the abutments. It is noted that the rebar detailing has primary steel only under abutment (typically 2 ft 6 in). USACE program CPGA was used to analyze the pile forces, which assumes a rigid cap.
15. The pile cap under the closure width is adequate for strength and in good condition.

Results of Risk & Reliability.

The method of analysis begins with a deterministic analysis (without any load or resistant factors) of critical components to establish an expected Factor of Safety (FS). A Reliability analysis was performed for the critical structural components to establish a probability of failure (POF). The critical structural components were chosen based upon lowest FS and largest COV. Sometimes more than one component can control for development of a POF curve at different water surface elevations. The Reliability was determined based upon the "Reliability Index," β, which assumes a lognormal distribution, and relative to FS of 1.0. For structural components whose factors of safety met or exceeded 1.5, or for geotechnical components whose FS met or exceeded 1.7, Reliability was assigned 99.8%.

The FS results from the deterministic analysis of key structural components are summarized in the attached spreadsheet. Results of any Reliability Analyses are summarized below:

4 C/K/437

Sheetpile Pullout. Pertaining to pile-founded closures (M145, M158, M162, and the Levee Closure), considering a simplified torsional analysis, the geotechnical pullout strength of the tension piles (sheetpile) was critical. Reliability is 36% with water elevation at Tow of Wall (TOW); 85% with water elevation at 1' below TOW; and 99.5% with water elevation at 2' below TOW. With water elevation below 2 ft from TOW, the reliability is considered 99.8%. M145 was the most critical of all the pile-founded monoliths, and its results were used to establish POF plots vs water elevation.

Comparison with Original Design.

Since the closures were not found 99% reliable, without adjustment for structure degradation, a comparison in design assumptions was performed. Like the present analysis, it seems that the original designers included water level to top of the wall. Unlike the present analysis, the original designers apparently only used undrained geotechnical strengths for the pile capacities (whereas the feasibility study included lower drained strengths), considered the closure's pile caps as rigid (whereas torsion was qualified in the feasibility study), but very conservatively ignored any efficiency of the cutoff wall to mitigate seepage and uplift pressures (whereas the cutoff wall efficiency was assumed 50% in the feasibility study). A few pages of original design comps were copied and attached for reference.

Recommendations.

1. Most closures need maintenance at the abutments, including cleaning and painting embedded steel.
2. If possible, inspect pile caps for cracks and corrosion of rebar.
3. Inspect timber stop logs and repair/replace if needed.
4. Seal cracks and joints with appropriate sealant for exposed conditions and movement of concrete.
5. Should a finite element analysis be performed, it is recommended that a 5% to 15% reduction in strength capacity be applied given the observations made during the site visit.

By Frederick R. Sheffield, P.E.
08 August 2008 (REV SEP 2011)
✓ KSM 11/16/11

(ORIGINAL CALCS)

Control Approval Symbol KE-6 5A/D/

CORPS OF ENGINEERS - U. S. ARMY
Office of the District Engineer
Kansas City 8, Missouri

Return to Struc. Design Section

Sheet 1 of 3
By JHT date 1-24-46
Ckd. [illegible] date 1-24-46

COMPUTATIONS

Project C10 Mo.

Design for General - 12'-13'-14'-15'-16' Pile Brg Floodwalls

Design 12', 13', 14', 15' & 16' pile bearing flood walls with a sheet pile cut-off wall at the heel and a drain at the toe.

Assume:

- Earth scoured away to level of river on river side
- Sat. Earth on land side
- No horizontal load from earth on landside
- Uplift: A straight line reduction in uplift from full head at the sheet pile cut-off wall to the static head in the toe drain at the toe with no percolation loss thru the sheet pile
- Four conditions of loading have been considered on all walls.
  1. Water to top of wall - uplift - moments in piles due to dirty in base included
  2. Same as 1 but piles not considered fixed in base hence no moments from piles
  3. Water to top of wall - no uplift - moments in piles due to dirty in base included
  4. Same as 3 but piles not considered fixed in base hence no moments from piles.

In addition the 15' wall was checked for the above cases with R.R. surcharge

Control Approval Symbol KE-6

ORIGINAL CALCS

5B/

CORPS OF ENGINEERS - U. S. ARMY
Office of the District Engineer
Design Section Kansas City 8, Missouri

Sheet 2 of 3
By JHT date 1-24-46
Ckd. CCR date 1-29-46

COMPUTATIONS

Project CID. Mo. Design for General - Pile Bearing Floodwalls

and for the cases of R.R. surcharge - no water on riverside of wall and consequently no uplift - with and - without moments in base from footin piles. These additional cases of loading showed that case 2 above with surcharge was the critical case for the steel in the top of the base at the riverside of the stem but that the increase in steel reqd. by this case over that required in the 4 cases of loading for all the walls was only 2.5 %. For this reason R.R. surcharge was not included in the design loadings.

- Conc. sheet piles between consecutive bearing piles are considered to be equivalent to one bearing pile for vertical load carrying capacities.
- Horizontal thrust of water is resisted by bearing piles and sheet piles, the load being divided between them in proportion to their stiffnesses.
- Where moments and deflections in the footin piles have been included in the computations they have been taken from data (attached)

ORIGINAL CALCS

Control Approval Symbol KE-6

5C/

CORPS OF ENGINEERS - U. S. ARMY
Office of the District Engineer
Design Section Kansas City 8, Missouri

Sheet 3 of 3
By HT date 1-24-46
Ckd. _____ date _____

COMPUTATIONS

Project G.I.D. Mo. Design for General Pile Bearing Floodwalls.

Compiled by HAN. from formulas presented by Cummings in A.S.C.E. Transactions of 1937.

— Allowable loads

Vertical loads on piling

16" □ Conc. pile — 25 Tons.
18" □ " " — 30 "
10" Conc. Sh. pile — 6 " per lin ft.
10" " " — " Uplift per lin. ft.

Horizontal loads

16" □ Conc. pile — 5 Tons.
18" □ " " — 5 Tons.
10" " Sh. pile — 2 Tons Lin. Ft.

— Design stresses

Steel — 18000#/□" Tension.
Conc. 1050#/□" Compr.
n = 10.
V = 60#/□" (No anchorage of long. reinf.)
V = 90#/□" (Spec. " " " "

(ORIGINAL CALCS) Control Approval Symbol KE-6

5D/10

CORPS OF ENGINEERS - U. S. ARMY Sheet 2 of ____
Office of the District Engineer By RRD date 1-22-46
Design Section Kansas City 8, Missouri Ckd. JPC date 4-1-46

COMPUTATIONS

Subject C.I.D. Mo. Design for 15' FLOOD WALL - FULL UPLIFT - NO SURCHARGE

	↓	↑	→	←	ARM	↺	↻
Vert. Forces							
Concrete 150 x 3 x 1	450				11.5	5200	
" 150 x .5 x 1	80				9.67	800	
" 150 x 13 x 2	3900				6.5	25400	
" 150 x 1 x 13	1950				4.0	7800	
" 150 x .33 x 13	650				4.72	3100	
Water 62.5 x 7.83 x 13	6360				9.08	57700	
62.5 x .33 x 13	270				4.85	1300	
Earth 125 x 3.5 x 2	880				1.75	1500	
	14540				7.09	102800	
Uplift		8880					69400
ΣV	5660					33400	
Hor. Forces							
Water 1000 x 8				8000	4.33		34600
Uplift 864 x 1			860		0.5		400
" 68 x .5			30		0.67		
" 250 x 2			500		1.33	700	
Piles ① & ②			3400		0.00		
Pile ③			3210		1.00		3200
ΣH			0				37500

Defl. per 3.0^K load 18" conc. brg. pile = .005556 x 3 = .016668"
" " 1.0^K " 10" " sh. " = .017637"

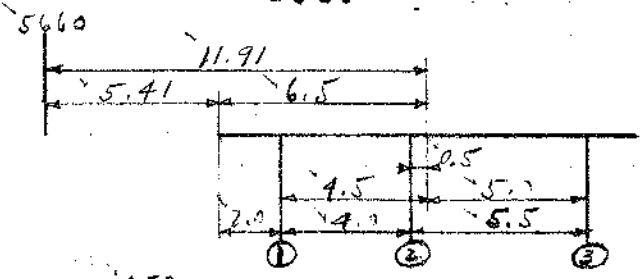
Hor. Shear in Brg. Piles = $\frac{6610 \times .017637}{.017637 + .016668}$ = 3400 51.44%

" " " sh. " = $\frac{6610 \times .016668}{0.034305}$ = 3210 48.56%

Moment in Brg. " = 4865 x 3.4 = 16500'#
" " sh. " = 3111 x 3.21 = 10000'# / 26500'#

Stability $a = \frac{33400 - 37500 - 26500}{5660} = -5.41'$ = Dist. outside Toe Resultant cuts Base.

Pile Loads



CG = (2+6+11.5)/3 = 6.5 from Toe
$\Sigma d^2 = 4.5^2 + 0.5^2 + 5^2 = 45.5$

① $\frac{5660}{3}\left(1 + \frac{3 \times 11.91}{45.5} \times 4.5\right)$ = 8550# per L.F. or 51300# per pile (3.53)

② 1887(1 + .78527 x 0.5) = 2620# " " (.39)

③ 1887(1 - .78527 x 5.0) = -5530 (3.93)
1810
3650 / 30 = 122# per a'

Case I

Comp. By F.R.S. Date ____ Project ____ CID MO Flood Protection ____ Sheet 7 of 937
Chkd. By KSM Date 9/2011 Subject Summary of Closures ____ Sheet ____ of ____

	Closure Type	Wall STA Begin	Wall STA End	Closes	Width (ft)	Height above ground	Monolith	On Piles?	Comments
1	Sand Bag	1+38.14	1+68.14	Pavement	30	4' 6"	NA	No	Requires No Structural Analysis
2	Stop Log	5+11.95	5+35.97	Pavement	24.02	6' 7"	10	No	
3	Stop Log	8+57.90	8+77.90	Railroad	22.5	8' 7 3/4"	18	No	"Z" Monolith
4	Stop Log	14+70.09	14+90.09	Railroad	22.5	11' 11"	32	No	"Z" Monolith
5	Stop Log	63+05.28	63+25.28	Railroad	22.5	11' 0"	145	Yes	"Z" Monolith
6	Stop Log	68+80.14	69+00.14	Railroad	22.5	9' 9 1/2"	158	Yes	"Z" Monolith
7	Stop Log	70+53.14	70+88.14	Street	35	9' 1 1/2"	162	Yes	Has Center Guide Post
8	Stop Log	80+04.12	80+33.14	Street	29.04	6' 10 1/2"	NA	Yes	Through Levee

	Strength Checks							
	Guide Posts Meet FS?	Center Post Meet FS?	Postwell Meet FS?	Stoplogs Meet FS?	Can torsion be ignored? ACI 11.6.1	Key Meet FS?	Sill Meet FS?	12" Wide Sill Meet FS?
M10	Y	NA	NA	Y	Y	Y	Y	NA
M18	Y	NA	NA	Y	Y	Y	Y	Y
M32	Y	NA	NA	Y	Y	NA	Y	Y
M145	Y	NA	NA	Y	**NO**	NA	NA	Y
M158	Y	NA	NA	Y	**NO**	NA	Y	Y
M162	Y	Y	Y	Y	**NO**	NA	Y	NA
Levee	Y	NA	NA	Y	Y	NA	Y	NA

	Stability		Bearing
	Sliding FS	OT Base in Compression	Meet FS?
M10	1.35	79%	Y
M18	2.84	100%	Y
M32	1.59	100%	Y
M145	Stability and other strength parameters for pile founded		
M158	closures were performed for M145 Abutment (3' pile cap		
M162	width). A POF was performed for sheetpile pullout.		
Levee			

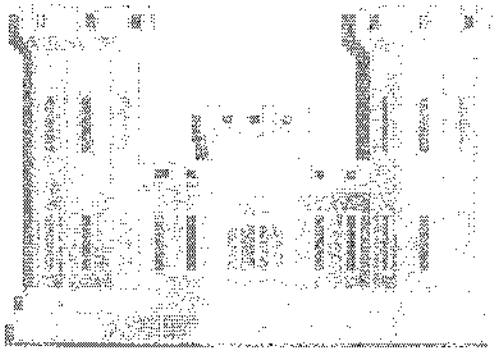
Notes:

Width is measured from out to out of abutments. Each abutment is 2' 6" wide, except for Levee Stop Log, width = 2' 6 1/4", and Monolith 10, width DS = 2' 0" and width US = 2' 0 1/4"

STA is measured from outside of abutments, or to CL of PI at Intersections.

"Z" Monolith refers to the Z shape the monolith makes in plan.

SITE VISIT



Subject: 05 May 2008 Site Visit to CID MO Closures
Prepared By: Frederick Sheffield
Date: 06 May 2008

The following notes were made:

M145. Height above ground = 10'-7".
The North Abutment appeared to have some mechanical damage. One inch deep gashes were observed. The N. Abutment was skew to the floodwall, as if partially rotated about its primary axis. The maximum relative deflection from the abutment and the wall was about 4 ½" at the top. The base of the abutment was not deflected away from the wall. The top 12" of the abutment appears to have been repaired in the past.
The South Abutment has minor cracking from mid-height and down. Minor map cracking perceived at base. Embedded steel rusting and pitting observed at base.

M32. Height above ground = 11"-10". Stoplogs were placed; possibly abandoned closure.
Minor horizontal and vertical cracks observed at abutments, measuring 0.01" to 0.016" in width. There is a possible hollow spot near the ground elevation of N. Abutment.

M18. Height above ground = 8'-2"; N. Abutment has 11'-2" exposure on side of abutment (possible exposure on protected side). Stoplogs were placed; possibly abandoned closure.
Vertical cracks observed at abutments, measuring 0.013" wide. N. Abutment observed leaning toward river (base is flush with floodwall at base, top deflection approximately 6" to 7"). Direction of lean is towards abutment side with 11'-2" exposed height.

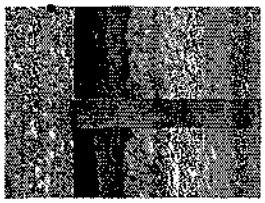
M10. Height above ground = 5'-7'. Clear width between abutments = 20'. Stoplogs were placed; possibly abandoned closure.
Top of abutments had "C" shape crack that measured 0.06" wide. It seems possible that these cracks could allow for degradation of vertical reinforcement. Each abutment had noticeable spalls on floodside. Vertical cracking typical of both abutments, each face. Embedded steel is rusting at base for first 18" from sill.

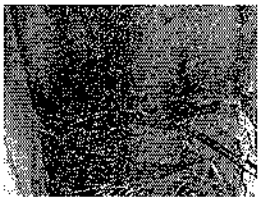
M158. Height above ground = 9'-1". Gated for vehicular use.
N. Abutment has Horizontal rebar visible and rusting.
Many horizontal and vertical cracks visible. Several patches (approx 2" dia) on each abutment.

M162. Height above ground = 8'-9". Gated for vehicular use.
E. Abutment has cracking on protected side. Metal trap for embedded steel is visible and rusting.
W. Abutment has fewer cracks than E. Abutment. 4 grout patches (approx 6" dia) on flood side. Embedded steel at base is delaminating from rust.

MLevee. Height above ground = 5'-11 ½"

Levee slopes on wall and abutment on floodside (increasing load), more so than landside.
W. Abutment is separated and tilting away from wall – 2" at base, 3" at top. The gap between the abutment and wall has been filled at one time with grout, but the grout is mostly disintegrated and missing.
Reference M10 "C" shape cracking at abutment. No crack in plan view of abutments noticed at this monolith. The top of the abutments at other closures could not be viewed for inspection of cracking.


M130 REL DEF.JPG

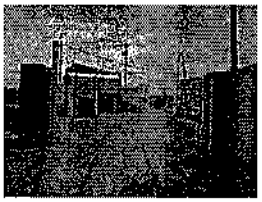

M145 Emb STL Rust at Base.JPG

M145 N Abt REL DEF.JPG

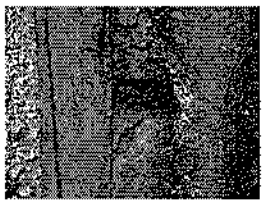
M145 S Abt Crcking typ Flood side.JPG

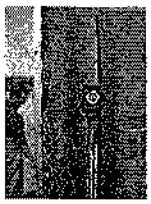
M145 stnding dry side.JPG

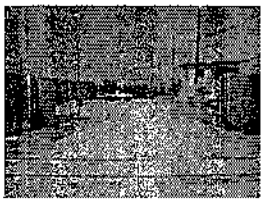
M158 N Abt popout at Rebar.JPG


M158 stnding Landside.JPG

M158 Typ of many small Patchs.JPG


M162 E Abut Rust at Emb metal 2.JPG


M162 E Abut Rust at Emb metal.JPG


M162 standing landside.JPG

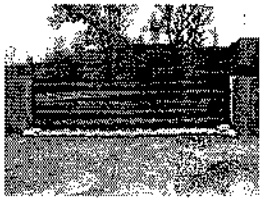
M162 W Abt Delaminating at Emb metal.JPG

MLev standing Landside.JPG

MLev W Abt leaning away from wall.JPG

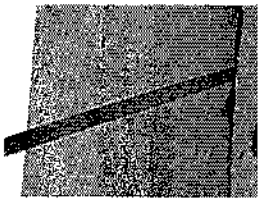
M10 E Abt standing Floodside.JPG

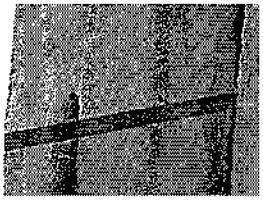
M10 E Abut stnding Landside.JPG

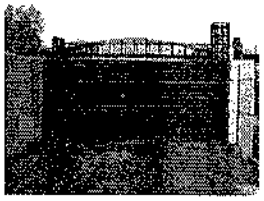

M10 standing Floodside.JPG

M10 Top of Abt.JPG

M10 W Abut stnding Landside.JPG

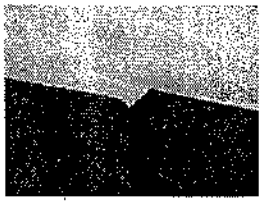

M18 N Abt tilting to river 1.JPG


M18 N Abt tilting to river 2.JPG


M18 stnding Floodside.JPG

M32 Possible delamination N Abt.JPG

M32 stnding fldside.JPG


M130 higher than neighboring Mono.JPG