MISCELLANEOUS PAPER N-75-8

STRUCTURAL ANALYSIS OF THE NEW ORLEANS INNER HARBOR NAVIGATIONAL CANAL LOCK WALL

by

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P. O. Box 631, Vicksburg, Miss. 39180

December 1975
Final Report

Approved For Public Release; Distribution Unlimited

Prepared for U. S. Army Engineer District, New Orleans
New Orleans, Louisiana 70160

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17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Lock walls
New Orleans Inner Harbor Navigational Canal
Structural analysis

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A beam theory structural analysis was conducted on the 50-year-old Inner Harbor Navigational Canal at New Orleans. A moment of 122 ft-kips on a wall section having an allowable moment capacity of 49 ft-kips was determined when the lock was dewatered. However, there was no evidence of structural failure during previous dewatering operations. The moment at the wall section, determined from a three-dimensional finite element analysis, was 89 ft-kips, and stresses in the wall were less than yield stresses for the materials. The ultimate moment (Continued)
20. ABSTRACT (Continued).

Capacity of the section was found to be 94 ft-kips. Using the ultimate moment capacity of the section, when a load factor of 1.7 is applied to the loading and a capacity reduction factor of 0.9 is applied to the section as required by American Concrete Institute design procedures, the ultimate moment capacity of the section would be exceeded. A procedure is outlined to monitor lock wall deflections during dewatering operations to determine impending failure.
PREFACE

The investigation reported herein was conducted by personnel of the Structures Division, Weapons Effects Laboratory (WEL), U. S. Army Engineer Waterways Experiment Station (WES), during the period October 1973 to March 1974, under the sponsorship of the U. S. Army Engineer District, New Orleans.

The project was under the general supervision of Mr. W. J. Flathau, Chief, WEL, and Mr. J. T. Ballard, Chief, Structures Division. Mr. P. J. Rieck of the Structures Division conducted the analysis and he and Mr. R. E. Walker prepared this report.

COL G. H. Hilt, CE, was Director of WES during the course of the investigation and preparation and publication of this report. Mr. F. R. Brown was Technical Director.
2.8 Counterfort models used in finite element computer runs------ 23
3.1 Lateral deflection of lock wall----------------------------- 29
3.2 Computation of moment on wall section---------------------- 30
3.3 Resultant moments along lock wall-------------------------- 31
3.4 Critical wall section stresses------------------------------- 32
4.1 Maximum lock wall moments during dewatering operations------ 36
CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

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<th>Multiply</th>
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<tr>
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<td>2.54</td>
<td>centimetres</td>
</tr>
<tr>
<td>square inches</td>
<td>6.4516</td>
<td>square centimetres</td>
</tr>
<tr>
<td>feet</td>
<td>0.3048</td>
<td>metres</td>
</tr>
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<td>miles (U. S. statute)</td>
<td>1.609344</td>
<td>kilometres</td>
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<tr>
<td>pounds (force) per square inch</td>
<td>6894.757</td>
<td>pascals</td>
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<tr>
<td>pounds (mass) per cubic foot</td>
<td>16.01846</td>
<td>kilograms per cubic metre</td>
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<tr>
<td>kips (force)</td>
<td>4448.222</td>
<td>newtons</td>
</tr>
<tr>
<td>foot-kips</td>
<td>1355.818</td>
<td>metre-newtons</td>
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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The Inner Harbor Navigational Canal (IHNC) is located in New Orleans, Louisiana, and extends a distance of 4-1/2 miles\(^1\) between the Mississippi River and Lake Ponchartrain. A vicinity map is shown in Figure 1.1. The canal and associated lock structure, which is located at the Mississippi River end of the canal, were constructed during the period 1919-1923.

The lock is a U-frame supported by counterforts at 15-foot intervals. It has a length of 420 feet, a width of 75 feet, and a depth of 55 feet. Principal views of the structure are shown in Figure 1.2. Copies of two original working drawings of the lock are presented in Appendix A.

The New Orleans District (NOD) conducted a stress analysis of the IHNC lock wall as part of a structural review program of the Corps of Engineers. The analysis conducted by NOD indicated that certain portions of the lock structure would be dangerously overstressed during dewatering operations. Specifically, moments developed in the lock wall were found to exceed the allowable moments by 250 percent. However, the lock has been dewatered on several occasions with no evidence of cracking or excessive deflections to suggest impending failure. This indicated that the structural analysis procedures used by NOD may be overly-conservative for this type of problem. According to the NOD analysis, the lock wall develops a moment of 122 ft-kips when the lock is dewatered and has an allowable moment capacity of 48 ft-kips. The NOD

\(^1\) A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.
stress analysis calculations are shown in Appendix B.

1.2 OBJECTIVES

The objectives of this report are to investigate the procedures used in the NOD analysis and to resolve the apparent conflict between the NOD analysis and the observed behavior of the lock structure.

1.3 APPROACH

A three-dimensional finite element computer code was used as a more refined method of analysis than that of NOD. While not a complete analysis in itself, the finite element method does not have many of the restrictions that the NOD analysis has. The assumptions and approximations used in both methods are investigated, and the results of both methods are compared with each other and with the actual behavior of the lock.
Figure 1.1 Vicinity map.
Figure 1.2 Principal views of the IHNC lock (half-structure).
CHAPTER 2

ANALYSIS OF LOCK WALL

The methods of analysis used by NOD and the U. S. Army Engineer
Waterways Experiment Station (WES) are fundamentally the same. Both
analyses use linear elastic theory and assume isotropic material behav­
or at any point within the structure.

The advantage of the finite element method of analysis is the fact
that it provides a computer-assisted solution to the modeling of the
three-dimensional geometry, material properties, boundary conditions,
and loadings associated with a complicated structure. The finite ele­
ment method solution is presented as stresses and displacements through­
out the structure. This allows the analyst to determine critical areas
quickly by comparing stresses obtained by the calculation with the
allowable stresses of the materials stipulated by design criteria.
Design conditions such as moments and shears are determined by investi­
gating the stress components along a specified section and performing
the necessary calculations.

The solutions of both NOD and WES methods are presented in the form
of moments along the height of the lock wall in Figure 3.3, page 31.

2.1 DESCRIPTION OF LOCK WALL

The IHNC lock wall is a reinforced concrete structure having a
20-foot-wide base section containing an 8- by 10-foot culvert and a wall
section 30 feet high tapering from a 4-foot thickness at the bottom to a
2-foot thickness at the top. An overhanging walkway and deck are at­
tached to the upper portion of the wall. The wall is supported by 3- by
3-foot reinforced concrete counterforts at 15-foot intervals along the
length of the structure. The entire wall rests on a pile-supported base
slab 10 feet thick. The outside of the lock wall is backfilled with
soil, with the water table midway up the wall. Figure 2.1 shows a cross
section of the lock. The reinforcing steel of the lock is detailed in a
working drawing in Appendix A.
The structure had a concrete design strength of 2,200 psi when built in 1919. In the following analysis, a modulus of elasticity of 29 million psi was used for the steel, and a modulus of 3 million psi was used for the concrete. The steel had an allowable stress of 22,000 psi.

2.2 DESCRIPTION OF LOADING

The loading consists of the at-rest lateral earth pressures above and below the water table. The same loading is used in both methods of analysis. The structure is considered to be dewatered. The soil loading is determined by 

\[ p_h = k_0 \gamma_{soil} h \]

above the water table with an additional loading, 

\[ p_h = k_0 \gamma_{submerged} + \gamma_{water} \]

below the water table where:

- \( p_h \) = horizontal earth pressure at \( x \) distance below the surface
- \( k_0 \) = coefficient of lateral earth pressure, 0.55
- \( \gamma_{soil} \) = soil density, 122 lb/ft\(^3\)
- \( h \) = distance below the surface, feet
- \( \gamma_{submerged} \) = \( \gamma_{soil} \) minus \( \gamma_{water} \)
- \( \gamma_{water} \) = water density, 62.4 lb/ft\(^3\)

The structure and loading are shown in Figure 2.2. The earth load varied linearly from zero at the top to a maximum of 32.3 psi at the culvert bottom.

Data on actual earth pressures on the sides of lock walls are available from measurements made on the Port Allen Lock and the Old River Lock as described in References 1 and 2. The loadings represent the active lateral earth pressure on the side of the lock in a dewatered condition. These data indicate that the design load described above may be somewhat conservative. Figure 2.3 compares the measured data with the assumed loading. The Port Allen and Old River Locks, however, are newer structures with different structural geometry and water tables. A firm conclusion as to whether the NOD loading is conservative or not is not possible. The effects of long-term settlement and numerous
dewatering and watering operations, along with the possibility of excessive moisture, justify the loading used by NOD in the analysis.

2.3 NOD ANALYSIS

The NOD investigation of the IHNC lock showed that when the lock is dewatered, moments develop in the lock wall which exceed the allowable moment capacity of the wall. The critical section occurs at the base of the wall just above the culvert section. Figure 2.4 shows a cross section at the 1-foot depth of the critical section of the wall and the steel reinforcing within. NOD determined the allowable moment capacity of the section to be 48.8 ft-kips. An analysis by WES, presented in Appendix C, found the ultimate moment capacity to be 94.3 ft-kips. The WES calculation used the ultimate strength method and should be modified with a capacity reduction factor of 0.9 for flexure as described in reference 3.

NOD used three approaches to determine the moment distribution along the lock wall. For the first approximation, the wall was assumed to be a fixed-end cantilever loaded by the at-rest lateral earth loading. For the second approximation, the wall was assumed to be a fixed-end beam supported by a pinned support at the location of the counterfort tie-in. These two methods are shown in Figure 2.5. A third approach was a moment distribution method. The results of the NOD and WES analyses are shown in Figure 3.3, page 31. The maximum moments determined by NOD were all found to be at the base of the wall and are listed below:

<table>
<thead>
<tr>
<th>Method of Analysis (NOD)</th>
<th>Maximum Moment, ft-kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cantilever beam</td>
<td>462</td>
</tr>
<tr>
<td>Fixed-pinned beam</td>
<td>122</td>
</tr>
<tr>
<td>Moment distribution</td>
<td>141</td>
</tr>
</tbody>
</table>

Note: Calculated moment capacity of section:
Allowable: 48.8 ft-kips
Ultimate: 94.3 ft-kips

The NOD analysis cannot account for the complete geometry of the lock wall. The methods used by NOD cannot include extra strength given
the wall by the counterfort and whaler (3-dimensional effects). Even then, however, the NOD results compare well with the WES finite element results (see Figure 3.3, page 31).

In all cases, however, the computed maximum moments exceed the allowable moment capacity of the lock wall. However, the structure has survived many dewaterings without evidence of impending failure.

2.4 FINITE ELEMENT ANALYSIS

A general three-dimensional structural analysis program, SAP IV (Reference 4), was used by WES to analyze the IHNC lock structure. The computer runs in the analysis were made on the Honeywell 635 computer facilities at WES. The computer program uses the finite element method to obtain a solution. The lock structure was modeled as an assemblage of three-dimensional elements. The elements are defined by their corners, or nodal points. The coordinates of the nodal points of an element define the element's size and position in the overall structure. Each element can represent a different material defined by a modulus of elasticity, \( E \); Poisson's ratio, \( \nu \); and a weight density. The two constants, \( E \) and \( \nu \), restrict the element to behave as a linear, elastic, or isotropic material. The individual element stiffnesses are added together to form a stiffness matrix for the entire structure, depending on the force-displacement characteristics of the individual elements. Boundary conditions are determined by specifying the displacements of the specific nodal points forming the boundaries of the lock. The structure loads are defined as specified external forces acting on the nodal points. The stiffness matrix and nodal point forces form a system of linear equations having the form of:

\[
F = kx
\]

where

- \( F \) = nodal point forces
- \( k \) = structure stiffness matrix
- \( x \) = nodal point displacements

This set of equations is solved for the nodal point displacements.
The solution of a finite element program consists of displacements at all the nodal points and stresses in all the elements. The amount of displacement of the nodal points is used to determine the strain within the elements. The material properties of the elements transform the strains into element stresses. As the number of elements used in defining the structure is increased, the solution converges to an "exact" solution based on linear elasticity theory.

Reinforced concrete is one of the most difficult structural materials to be modeled using finite element methods. A reasonable approximation can be made, however, if special attention is given to the results of the analysis to see if any section develops high tensile stress.

2.4.1 Lock Wall Grid. A typical section of the lock structure was used to form the finite element model. The typical section was determined by investigating the symmetrical features of the structure and by finding planes of zero normal strain (plane strain conditions). Nodal points lying within a plane of zero normal strain are restricted to displacements within the plane, forming a boundary.

Figure 2.6 shows the typical section of the lock used in the analysis. The nodal points were considered fixed along the bottom of the base slab to represent the restraining effects of the pile foundation. A perspective view of the finite element model or grid is shown in Figure 2.7. The grid consists of 704 elements and 1258 nodal points. A system of 3065 linear equations was solved to obtain a solution.

2.4.2 Lock Wall Loading. The same loading used by NOD (Figure 2.2) representing the at-rest lateral earth pressures was used in the finite element analysis. Every element in the grid which was "in contact" with the soil was given a loading representing the earth loading at that point.

The finite element problem complete with grid, boundary conditions, and loading can now be solved for the resulting deflections and stresses.

2.4.3 Lock Wall Analyses. Three finite element method computer runs were made by WES to analyze the IHNC lock structure. In all cases the earth loading, boundary conditions, and grid remained the same.
The differences in the analyses were due to the material property changes made in the elements approximating the strength of the lock counterfort.

The NOD computations showed that the moments that developed in the lock wall at the critical section were extremely dependent on the displacement allowed for the wall at the point of intersection of the line of action of the counterfort and the wall. This is seen by examining the NOD approximations and the resulting moments (Section 2.3). The unrestrained cantilever gave the highest moment; and the pinned support approximation, which allowed no displacement at all, gave the lowest moment. The stresses and moments resulting from a finite element analysis would also be determined by the displacement of the wall and would be directly dependent on the approximated strength of the counterfort. Figures 2.8a, b, and c show the changes made in the representations of the strength of the counterforts for the computer runs.

The first run assumed that the strength of the counterfort was determined by the tensile properties of the concrete, using an assumption that concrete does have a tensile stress limited to 10 to 20 percent of its compressive strength (see Reference 3).

The second run assumed that the counterfort concrete was badly deteriorated and that the counterfort was composed of steel, representing the actual amount of reinforcing in the counterfort as shown in the working drawings. This approximation allowed large strains to occur in the counterfort. A better approximation may have been to assume that the concrete in a portion of the counterfort was deteriorated rather than that in the entire counterfort.

The third computer run assumed that the combined strength of reinforcing steel and concrete acted to resist the loading. The counterfort is heavily reinforced, having 2.22% steel, and therefore needs to be modeled as reinforced concrete to represent the structure most accurately. The steel reinforcement was added to the model by overlaying one-dimensional truss elements on the original finite element grid corresponding to the position and direction of the steel in the counterfort as shown in the working drawings.
The strength changes were made in the counterfort with the prior knowledge that the counterfort would undergo a predominantly tensile loading. Reinforced concrete can be modeled very well when the response of the structure is known to represent an axial or bending condition.
Figure 2.1 Cross section of the IHNC lock structure.
Figure 2.2 Description of earth loading on lock wall.
Figure 2.3 Comparison of horizontal earth pressures on lock walls.
Figure 2.4 Section geometry at the point of critical moment.
Figure 2.5 Beam approximations used by NOD.
Figure 2.6 Typical section of lock used in finite element analysis.
Figure 2.7 Finite element grid of lock section.
a. **CONCRETE ONLY**  
NO REINFORCING STEEL

b. **REINFORCING STEEL ONLY**

Material Properties:

\[ E_C = 3 \times 10^6 \text{ PSI} \]
\[ E_S = 29 \times 10^6 \text{ PSI} \]
\[ \nu_C = 0.3 \]
\[ \nu_S = 0.3 \]

634 SQ IN. OF CONCRETE
14.1 SQ IN. OF STEEL  
(2.22% REINFORCING STEEL)

NOTE:  
**EC** = MODULUS OF ELASTICITY FOR CONCRETE, PSI  
**ES** = MODULUS OF ELASTICITY FOR STEEL, PSI  
**\nu_C** = POISSON'S RATIO FOR CONCRETE  
**\nu_S** = POISSON'S RATIO FOR STEEL

Figure 2.8 Counterfort models used in finite element computer runs.
CHAPTER 3

RESULTS OF FINITE ELEMENT ANALYSES

The finite element solutions using the SAP IV computer program consist of nodal point displacements and element stresses for all the nodal points and elements in the grid representing the structure. In general, three displacement components are associated with each nodal point, and six stress components are associated with each element. The entire displacement and stress field of this structure cannot be presented herein.

3.1 DEFLECTION OF LOCK WALL

The analysis showed that the lock wall had a lateral inward deflection resulting from the lateral earth loading. Figure 3.1 shows the lateral deflection of the wall taken at a section midway between the counterforts. The three deflection curves represent the three analyses made using the different counterfort materials. The displacements of the top of the lock, the point of counterfort tie-in, and the part of the lock wall located at the critical moment section are given in the following tabulation.

<table>
<thead>
<tr>
<th>Finite Element Analysis</th>
<th>Top of Wall</th>
<th>Strut Tie-in</th>
<th>Critical Moment Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete counterfort</td>
<td>0.21</td>
<td>0.16</td>
<td>0.06</td>
</tr>
<tr>
<td>Steel reinforcing counterfort</td>
<td>0.46</td>
<td>0.28</td>
<td>0.05</td>
</tr>
<tr>
<td>Reinforced concrete counterfort</td>
<td>0.19</td>
<td>0.15</td>
<td>0.06</td>
</tr>
</tbody>
</table>

The analysis proved the reinforced concrete to form the strongest counterfort, as expected, yielding the smallest displacements. The tensile properties of concrete make the concrete counterfort stronger than the counterfort using only reinforcing steel due to the much greater cross-sectional area of concrete.

By inspection, Figure 3.1 shows that the greatest curvature
associated with the deflection curves occurs at the critical moment section, at the base of the wall, where the wall broadens out over the culvert section.

3.2 MOMENTS ON THE LOCK WALL

The finite element analyses solved for resultant stresses in all the elements of the grid. Moments on a section are calculated by plotting the normal stress distribution across the specified section. The moment can be determined independently of the section material properties. When the stress distribution is linear, or nearly so, the section moment can be determined using the linear beam relation:

\[ M = \frac{1}{2} (\sigma_i - \sigma_o) \left( \frac{I}{c} \right) \]

\[ = \frac{bh^2(\sigma_i - \sigma_o)}{12} \]

where:

- \( M \) = section moment, ft-kips
- \( \sigma_i \) = vertical stress on chamber face, psi
- \( \sigma_o \) = vertical stress on outside face, psi
- \( I \) = moment of inertia of section, inches
- \( c \) = distance from center of section to outside faces, inches
- \( b \) = width of section, inches
- \( h \) = length of section, inches

Figure 3.2 shows a typical wall section on which the procedure above was used to determine the section moment.

This procedure was used to determine the lock wall moments along the length of the wall for the three finite element runs made. The wall moments are plotted in Figure 3.3. The moment results obtained by the NOD study are also plotted for comparison. In all cases, the maximum moments occur at the critical section determined by NOD at the base of the lock wall where the section broadens out over the culvert. The maximum moments and methods of analysis are listed here:
Method of Analysis | Maximum Moment, ft-kips
---|---
Cantilever beam | 462
Fixed-pinned beam | 122
Moment distribution | 141

Finite elements

Concrete counterfort | 97
Steel counterfort | 206
Reinforced concrete | 89

Section moment capacity: Ultimate: 94.3 ft-kips
Allowable: 48.8 ft-kips

All of the section moments calculated by NOD and WES exceed the NOD design-allowable moment of 48.8 ft-kips. The reinforced concrete finite element analysis moment value is within the ultimate strength capacity of 94.3 ft-kips.

3.3 COUNTERFORT STRESSES

The reinforced concrete model counterfort is the best representative of the actual counterfort. The analyses using the concrete and steel counterfort material properties were run independently to determine the contributory effects of the material in relation to the moments developed in the lock wall. The counterfort tensile stresses as determined by the three finite element analyses are presented below.

<table>
<thead>
<tr>
<th>Finite Element Analysis</th>
<th>Counterfort Stress, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete counterfort</td>
<td>350</td>
</tr>
<tr>
<td>Steel counterfort</td>
<td>10,800</td>
</tr>
<tr>
<td>Reinforced concrete counterfort</td>
<td></td>
</tr>
<tr>
<td>Concrete stress</td>
<td>290</td>
</tr>
<tr>
<td>Steel stress</td>
<td>2,800</td>
</tr>
</tbody>
</table>
The most questionable stress is the 350-psi tensile stress associated with the concrete counterfort. Reference 3 indicates that the concrete could sustain a 220- to 440-psi tensile stress, but the 440-psi value is more appropriate for short-time loadings. The stress in the steel counterfort has a low stress value compared with its 33,000-psi yield strength. This indicates that the additional strength provided by the steel in the reinforced concrete counterfort model helps to reduce the concrete stresses in the model to lower tensile levels.

It should be noted that the procedure used above is an approach to determine the actual strength of the lock as closely as possible and should not be used as a standard method of design.

3.4 CRITICAL SECTION STRESSES

The critical section of the lock wall occurs at its base where the wall broadens over the culvert section. The moments calculated at this section are the largest on the wall. The location of the section is shown in Figure 2.4.

The finite element analysis run did not include the effects of reinforcing steel in the wall. This exclusion resulted in higher wall deflections, stresses, and moments than if the effects of the steel were included. It is difficult to include the effects of the steel in the wall in the finite element analysis since the wall is doubly reinforced at varying intervals, and many more elements would be needed for an accurate representation. The percentage of steel in the lock wall is low, 0.13% tensile steel and 0.41% compression steel, and would not cause a significant change in the results.

The strains determined by the finite element program were then applied to the actual section geometry. The section included the reinforcing, and reinforcing stresses were calculated. The critical section stresses for the three finite element analyses made are shown in Figure 3.4.

The tensile concrete wall stress at the critical section resulting from the run using a steel counterfort was 530 psi, which would indicate
wall cracking if the actual counterfort had the strength as modeled. The concrete tensile stresses of the other two runs, 206 and 133 psi, were low enough so that cracking would not occur. In all three cases the steel reinforcing stress levels were low. Modeling of the lock wall reinforcing steel would have reduced the concrete tensile stresses.
Figure 3.1 Lateral deflection of lock wall.
a. LOCATION OF WALL SECTION

Note: \( h \) = LENGTH OF SECTION, IN.
\( M \) = SECTION MOMENT, FT-KIPS
\( \sigma_i \) = VERTICAL STRESS ON CHAMBER FACE, PSI
\( \sigma_o \) = VERTICAL STRESS ON OUTSIDE FACE, PSI

b. STRESS DISTRIBUTION ACROSS SECTION

\[
M = (\sigma_i - \sigma_o) \frac{h^2}{12}
\]
\[
= (-198 - 73) \frac{(46.6)^2}{12}
\]
\[
= -49,041 \text{ FT-LB} = -49.0 \text{ FT-KIPS}
\]

c. SECTION MOMENT CALCULATION

Figure 3.2 Computation of moment on wall section.
Figure 3.3 Resultant moments along lock wall.
**SECTION STRAIN, $\mu$IN./IN.**

**CHAMBER SIDE**

**LAND SIDE**

- **EL 10.0'**
- $+16$
- $-89$
- $-100$

**SECTION MOMENT:** 97 FT-KIPS

$E_S = 29 \times 10^6$ PSI

$E_C = 3 \times 10^6$ PSI

**SECTION STRESSES:**

1. **CONCRETE, COMPRESSION:** $-323$ PSI
2. **STEEL, COMPRESSION:** $-2580$ PSI
3. **STEEL, TENSION:** 464 PSI
4. **CONCRETE, TENSION:** 194 PSI

**NOTE:**

$A_S = \text{AREA OF TENSION REINFORCING STEEL, SQ IN.}$

$A_s' = \text{AREA OF COMPRESSION REINFORCING STEEL, SQ IN.}$

---

**CONCRETE COUNTERFORT MODEL**

Figure 3.4 Critical wall section stresses (sheet 1 of 3).
SECTION STRAIN, µIN./IN.

CHAMBER SIDE

LAND SIDE

EL +10.0'

(-156)

(+83)

SECTION GEOMETRY

SECTION MOMENT: -206 FT-KIPS

SECTION STRESSES:

1. CONCRETE, COMPRESSION: -580 PSI
2. STEEL, COMPRESSION: -4520 PSI
3. STEEL, TENSION: +2410 PSI
4. CONCRETE, TENSION: +530 PSI

b. STEEL COUNTERFORT MODEL

Figure 3.4 (sheet 2 of 3).
CHAMBER SIDE

SECTION STRAIN, μIN./IN.

LAND SIDE

EL +10.0'

48'

4''

4''

1 2 3 4

AS' = 3 NO.8 BARS
= 2.36 SQ IN.

AS = 1 NO.8 BAR
= 0.79 SQ IN.

E₀ = 29 x 10⁶ PSI

E₀ = 3 x 10⁶ PSI

SECTION GEOMETRY

SECTION MOMENT: - 89 FT-KIPS

SECTION STRESSES:
1 CONCRETE, COMPRESSION: - 307 PSI
2 STEEL, COMPRESSION: - 2500 PSI
3 STEEL, TENSION: + 400 PSI
4 CONCRETE, TENSION: + 173 PSI

c. REINFORCED CONCRETE COUNTERFORT

Figure 3.4 (sheet 3 of 3).
CHAPTER 4

DISCUSSION OF RESULTS

The finite element analysis of a three-dimensional reinforced concrete structure was performed. The reinforcing steel was modeled in the counterfort (2.22 percent tensile steel) and neglected in the lock wall (0.13 percent tensile steel, 0.41 percent compression steel). Inclusion of the lock wall reinforcing would have resulted in lower wall displacement and moments, but more elements would have been needed to model the structure and analysis cost would have been significantly increased. The critical section moments are listed below.

<table>
<thead>
<tr>
<th>Method of Analysis</th>
<th>Maximum Moment, ft-kips</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NOD</td>
</tr>
<tr>
<td>Cantilever beam</td>
<td>462</td>
</tr>
<tr>
<td>Fixed-pinned beam</td>
<td>141</td>
</tr>
<tr>
<td>Moment distribution</td>
<td></td>
</tr>
<tr>
<td>Steel counterfort</td>
<td>206</td>
</tr>
<tr>
<td>Concrete counterfort</td>
<td></td>
</tr>
<tr>
<td>Reinforced concrete counterfort</td>
<td>89</td>
</tr>
</tbody>
</table>

The finite element method using the reinforced concrete counterfort most accurately defined the lock structure. This analysis also resulted in the lowest moments and stresses at the critical section. The allowable moment of 48.8 ft-kips was still exceeded. Figure 4.1 shows the critical wall section moment predicted as a function of the total wall deflection (both walls of the lock) at the top of the lock during dewatering operations. The moment-wall displacement relationship was determined using the values obtained in the analysis using the reinforced concrete counterfort.
Figure 4.1 Maximum lock wall moments during dewatering operations.
CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The finite element analysis indicates that the IHNC lock structure is not safe during dewatering operations. The resulting lock structure wall moments exceed allowable values. The American Concrete Institute code of 1971 requires a load factor of 1.7 to be applied to the load and a capacity reduction factor of 0.9 on the calculated ultimate moment of the section. This would result in an ultimate wall moment of 151 ft-kips and a section moment capacity of 85 ft-kips, which would not be acceptable. The finite element analyses conducted assume the lock to be in good condition.

The loading used on the structure may be too severe, as no provision is made for reduction of load due to soil-structure displacement characteristics of back-filled cantilever walls. The lateral earth pressure data obtained by measurement of the Port Allen and Old River Locks indicate that the soil loading is conservative. However, there may be questions concerning these data since the sequence of backfilling operations on the locks tends to displace the wall inward and reduce the lateral earth pressures at lower levels. The backfilling of the IHNC lock in 1921 can be considered by now to be well compacted. The lock dewatering and waterings over the years will have activated both active and passive earth loads. It is concluded that this loading as determined by NOD and used in all the analyses is valid and accurate.

The compressive strength of concrete of 2200 psi is probably low. The natural curing conditions of the lock structure probably have increased this value considerably. This increased strength, however, is offset by the age of the 50-year-old structure.

The reinforcing steel in the IHNC lock wall seems to be designed to withstand the hydrostatic pressures from a full channel rather than the external earth loading.

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5.2 RECOMMENDATIONS

It is recommended that a core sample of the lock concrete be tested to determine the compressive strength of the 50-year-old concrete. A significant increase over the 2200-psi strength will give added assurance of the lock structure's strength.

Wall-to-wall measurements should also be made at the top of the lock if dewatering operations are undertaken. Figure 4.1 shows that if the total inward deflection, $\Delta$, exceeds 0.21 inch, the allowable moment of 48.8 ft-kips is exceeded. If the deflection exceeds 0.44 inch, the ultimate moment of 94.3 ft-kips would be exceeded.

It is still possible to conduct a more extensive analysis of the lock which utilizes earth load-structure interaction effects. Such an analysis could reduce the moment value in the critical section, due to active soil pressure conditions.
REFERENCES

1. W. C. Sherman, Jr., and C. C. Trahan; "Analysis of Data from Instrumentation Program, Port Allen Lock"; Technical Report S-68-7, September 1968; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.

2. W. C. Sherman, Jr., and C. C. Trahan; "Analysis of Data from Instrumentation Program, Old River Lock"; Technical Report S-72-10, June 1972; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.


4. K. Bathe, E. L. Wilson, and F. E. Peterson; "SAP IV: A Structural Analysis Program for Static and Dynamic Response of Linear Systems"; Report EERC 73-11, June 1973; University of California, Berkeley, California.
APPENDIX A

NEW ORLEANS DISTRICT WORKING DRAWINGS
GENERAL NOTES

(1) The design is based on the assumption that all plaque of the
work through east of the concrete shall be made. Keep the
lining. Strength is 5000 psi.

(2) Spacing bars shown in section, but not marked, to be 12 bar spaced.

(3) Panel protrusion left reverse and fixed. Special Section including sections on each side

(4) See drawing 432-437 for reference and details of drawings and assumptions of
design.

(5) Placement of spacers 430, #3 bars in which may be used.

(7) See drawing 432-437 for accurate details of work.
APPENDIX B
NEW ORLEANS DISTRICT STRESS ANALYSIS CALCULATIONS
Figure #1
WALL CONSIDERED ACTING AS A CANTILEVER

MOMENT IN WALL AT FOLLOWING ELEVATIONS

CONDITION: LOCK Dewatered

MOMENT DUE TO WALL OVER HANGE

\[
\begin{align*}
\text{M} & = \frac{WT}{2} (10 \times 10 \times 50) = 15^2 \\
& = 2.074 \\
\end{align*}
\]

\[
\begin{align*}
\text{EM} & = 2.074 \times 32.5 \times 4.5 \\
& = 296.25
\end{align*}
\]

MOMENT DUE TO WT OF CONE = 296.25

MOMENT DUE TO SOIL LOAD

\[
\begin{align*}
\text{EM} & = \frac{H \times \text{W} \times (\text{W} + 10 \times 10 \times 50)}{2} \\
& = \frac{4.5 \times 24.5 \times (24.5 + 100)}{2} \\
& = 164.32
\end{align*}
\]

MOMENTS AT EL. 20.5

\[
\begin{align*}
\text{EM} & = \frac{H \times \text{W} \times (\text{W} + 10 \times 10 \times 50)}{2} \\
& = \frac{4.5 \times 24.5 \times (24.5 + 100)}{2} \\
& = 164.32
\end{align*}
\]

MOMENT AT EL. 10.0

\[
\begin{align*}
\text{W} & = \text{H} \times (55 \times 10) \\
& = (10.5) \times (55 \times 10) \\
& = 526 \times \frac{55}{2} \\
& = 1842
\end{align*}
\]

\[
\begin{align*}
\text{EM} & = \frac{H \times \text{W} \times (\text{W} + 10 \times 10 \times 50)}{2} \\
& = \frac{10.5 \times 1842 \times (1842 + 100)}{2} \\
& = 3775.06
\end{align*}
\]

WALL OVER HANGING

\[
\begin{align*}
\text{EM} & = \frac{H \times \text{W} \times (\text{W} + 10 \times 10 \times 50)}{2} \\
& = \frac{10.5 \times 1842 \times (1842 + 100)}{2} \\
& = 3775.06
\end{align*}
\]

\[
\begin{align*}
\text{EM} & = 462.12
\end{align*}
\]

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MOMENT @ 5.5

Mom. Due to overturning = 21.96
0.55/16' * (24.1 + 15) = 24.1 x 2.16
0.55/16' * (24.1/2) = 24.1 x 0.75
h (55/26 + 26) / h (3/4) = (10.5/15) / 10.5 / 10.5

EM = 682.22

MOMENT CAPACITY OF LOCK WALL EL 32.5'
l = 2.2H10 KSI COMP
f = 1000 psi
f = 35000 psi
f = 18000 psi

From (A.T.I. 102 w)
E = w = $337 f_c
E = (150)'(337)200' / 2 = 2.84 x 10^6
M = E/2 = 5210 / (2.84 x 10^6) = 10,1 use 10

Check Reinforcement in tension

\[ \frac{1}{2} E + \frac{1}{2} (150)^2 \] = 2050

\[ \frac{1}{2} E + 12 (150)^2 (32.5) = 10 (23)(18 - 32) \]
\[ 6 H^2 + 14.35 H - 59.3 = 6108 - 3040 \]
\[ 6 H^2 + 38.55 H = 720.6 = 0 \]
\[ K^2 + 6.42 H - 120.1 = 0 \]

\[ H - 6.42 \pm \sqrt{(6.42)^2 - 4(1)(-120.1)} \]

\[ \frac{6.42 \pm \sqrt{2050}}{2} = 6.2 \]

STRESS VOLUME
\[ G = \frac{x}{2} (14 - 6 = \frac{14}{2} = 3.4) = 38.46 \times \frac{3}{4} = 29.4 \]
\[ G = \frac{14.1}{10} \times \frac{3}{4} = 20.4 \]
\[ C = \frac{16.2}{10} = 16.48 \]

\[ \frac{14.1}{10} = 1.76" \]
\[ \sqrt{28 - 1.5} = 26.44" \]

\[ M_2 = 233.5 \text{ KF} \]

\[ M_t = 10(18/10) = 2.34 \text{ KF} \]

\[ M_2 = 233.5 \text{ KF} \]

\[ M_t = 23 (18/10) = 23.6 \text{ KF} \]

SHEAR
\[ V = 6.6 H = 0.66 (18)(24.44) = 20.9 \text{ K} \]
CHAMBER FACE REINFORCEMENT IN TENSION
MOBILITY CAPACITY AT ELEV 20.5

\[
\begin{align*}
\frac{12}{2} k_d^2 + (2m-1)k_d \left( h_d - h_y \right) m e (d - k_d) \\
6w^2 + 19.78h/d + 0 = 10(9.5)(50 - 10) \\
6w^2 + 19.78h/d - 53.64 = 817.64 - 53.64 \\
6w^2 + 19.78h/d - 53.64 + 817.64 = 0 \\
k_d^2 + 38.5k_d - 930.5 = 0 \\
k_d^2 + 6.42k_d - 155 = 0 \\
k_d = \frac{6.42 \pm \sqrt{(6.42)^2 - 4(-155)}}{2} \\
k_d = 9.64 \text{ ft}
\end{align*}
\]

\[
T_A = \frac{1.2(16)}{1.4} = 12.4 \text{ kips}
\]

\[
c_1 = \frac{0.5(5.4)}{2} = 6.84 \text{ kips}
\]

\[
c_2 = \frac{0.5(5.4)}{2} = 6.84 \text{ kips}
\]

\[
C_1 = (6.84)(1.2) = 81.72 \text{ kips} \\
C_2 = (6.84)(1.2) = 81.72 \text{ kips}
\]

\[
M_C = \frac{C_1}{h_d} = 66.58 \left( \frac{9.5}{10} \right) = 186.42 \text{ kF}
\]

\[
M_3 = \frac{C_2}{h_d} = 42.4 \left( \frac{9.5}{10} \right) = 118.9 \text{ kF}
\]

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\[ M_e = C_1 h / \]  
\[ M_e = 73.4 \text{in}(42.74) / 147.3 \text{in} \]  
\[ M_e = 2.56(14.328) / 147.3 \text{in} \]  
\[ V_e = 0.066(14.328) / 31.98 \]

\[ M_e = C_2 h / \]  
\[ M_e = 73.4 \text{in}(42.74) / 147.3 \text{in} \]  
\[ M_e = 2.56(14.328) / 147.3 \text{in} \]  
\[ V_e = 0.066(14.328) / 31.98 \]
FOR LANDSIDE FACE REINFORCING INTENSITY (#8, 11")

MOMENT CAPACITY EL 32.5'

\[ 6k_d^2 + 10(1.35)(k_d - 0.65) = 38.785(0.23 - k_d) \]
\[ k_d^2 + 8.75k_d - 66 = 0 \]
\[ k_d = \frac{-8.75 \pm \sqrt{8.75^2 - 4(1)(-66)}}{2(1)} \]
\[ k_d = 3.79'' \]

STRESS VOLUME

\[ G = \frac{f_d}{f_d(4.85)} = \frac{10.1 \times \frac{8.25}{3}}{4.85} + \frac{31.2}{36.9} = 78.2 \]

\[ M_1 = 30.48kF \]

\[ M_2 + CW = 10.75(2.88)(4) = 79.58'' \]
\[ M_3 = 7.4(176)(0.63)(4) = 36.5'' \]
\[ M_4 = 28.12 + 25.88'' \]

AT 20.5''

\[ d = 3.7'' \]

\[ 6k_d^2 + 10(2.35)(k_d - 0.45) = 10(38.5)(0.23 - k_d) \]
\[ k_d^2 + 8.75k_d - 78 = 0 \]
\[ k_d = \frac{-8.75 \pm \sqrt{8.75^2 - 4(1)(-78)}}{2(1)} \]
\[ k_d = 5.78'' \]

\[ G = \frac{f_d}{f_d(5.48)} = \frac{12.06 \times \frac{54.1}{3}}{44.93} = 2.41 \]
\[ G = \frac{f_d}{f_d(5.48)} = \frac{12.06 \times \frac{54.1}{3}}{44.93} = 2.41 \]
\[ M_2 = 44.35(34.58/2) \]
\[ M_7 = 1.785(18)(34.58/4) = 40.72'' \]
**EL. 10.0**  
**NEGATIVE MOMENT**  

\[ K_d^2 + 8.75 \times K_d - 87.33 = 0 \]

\[ K_d = \frac{-8.75 \pm \sqrt{(8.75^2 - 4 \times 87.33)}}{2} = 5.54" \]

\[ C = \frac{L}{2} \times (5.54) = 35.64 \times \frac{5.54}{2} = 70.57 \]

\[ G = \frac{5.54}{3} \times (\frac{L}{2}) \times 1.35 = 14.58 \times 4 = \frac{58.32}{30.22} = 1.93 \]

\[ \varepsilon = \frac{58.32}{30.22} = 1.93" \quad \sqrt{2.93} = 44.43 = 44.43" \]

\[ M_c = 50.22 \times (41.43)(\frac{L}{2}) = 173.38 \text{ k}\cdot\text{ft} \]

\[ M_T = 785 \times (41.43)(\frac{L}{2}) = 48.78 \text{ k}\cdot\text{ft} \]

---

**SHEAR @ EL. 10 FOR WALL SPANNING BETWEEN WALL & TOP OF CULVERT WALL.**

**SHEAR DUE TO WALL SPANNING BETWEEN WALL & TOP OF CULVERT WALL.**

**SHEAR DUE TO WALL SPANNING BETWEEN WALL & TOP OF CULVERT WALL.**

\[ V = \frac{22.5 + \frac{7 \times 122}{20}}{2} \]

\[ V = \frac{54.05 \times 2}{2} = 22.5 \text{ kips/ft length} \]

\[ V = \frac{22.5}{20} \times 20 \text{ kips/ft length} \]

\[ \text{ALLOWABLE SHEAR @ EL. 10.0} \]

\[ V = 0.06 \times (40.38) = 2.42 \text{ kips/ft length} \]

\[ \text{SHEAR IS ADEQUATE} \]

---

**SHEET 7**
ANALYSIS OF WALL SUPPORTED BY WALER

ASSUMPTIONS:
1. SOIL UNIT WEIGHT OF 122 #/FT³
2. MOMENT OF INERTIA OF WALL CONSTANT.
3. LOCK DEWATERED.
4. WATER TABLE AT EL 20.5 CAIRO DATUM;
5. AT REST SOIL PRESSURE

NOTE: OVERHANG LOAD ASSUMED TO VARY LINEARLY FROM EL 45 TO EL 34 AND PRODUCES A MOMENT OF 21.96 FK AT SUPPORT (EL 34). (SEE SHEET 2)

MOMENTS DUE TO δ:

\[ M_{34} = \frac{0.07(10)}{2} = 0.35 \text{ ft-lb} \]
\[ M_{245} = \frac{0.07(24.9)}{2} = 0.86 \text{ ft-lb} \]
\[ M_{10} = \frac{0.07(24.9)(10)}{2} = 9.31 \text{ ft-lb} \]
\[ M_{10} = 375.35 \text{ kft} \]

MOMENT DUE TO θ:

\[ M_{10} = \frac{0.07(24.9)(10)}{2} = 9.31 \text{ ft-lb} \]
\[ M_{10} = 90.48 \text{ kft} \]
SOLUTION: WIT method.

FOR COMPUTATIONS, WE MUST SHOWN ON ITS SIDE.

1. ASSUME NO DEFLECTION AT SUPPORT.
2. REMOVE SUPPORT AND COMPUTE $c_P$.

- $t_2$ due to $M$.
  \[
  E I \theta_2 = \frac{21.04 \times 24.1}{2} = 527.84 \times 10^6
  \]
  \[
  E I \Delta_2 = \frac{527.84 \times 24.1}{2} = 324.48 \times 10^6
  \]

- $t_2$ due to $\theta$.
  \[
  E I \theta_2 = \frac{(4.86 \times 4.8)}{2} = 40.86 \times 10^6
  \]
  \[
  E I \Delta_2 = \frac{(4.86 \times 24.1)}{2} = 100.78 \times 10^6
  \]
  \[
  E I \Delta_2 = \theta_2 - \theta_2 = 527.84 \times 10^6
  \]

3. \[\eta_4 = \frac{\theta_3 - \theta_4}{\theta_4}\]

4. \[\Delta_4 = 4.44\]

5. \[E I \Delta_4 = 96.92 \times 10^6 = 8733 \times 10^6
  \]
  \[
  E I \Delta_5 = \frac{4.86 \times 24.1}{2} = 1724 \times 10^6
  \]
  \[
  E I \Delta_6 = 1724 \times 10^6 = 3228 \times 10^6
  \]
  \[
  E I \Delta_7 = \frac{24.1 	imes 10.5}{2} = 110.5 \times 10^6
  \]

6. \[E I \Delta_7 = \frac{108.5 \times 24.2}{2} = 2272 \times 10^6
  \]

7. $t_2$ due to $\theta$.
  \[
  E I \theta_2 = \frac{90.48 \times 24.2}{2} = 316.69 \times 10^6
  \]
  \[
  E I \Delta_7 = 316.69 \times 24.2 = 7673 \times 10^6
  \]

8. \[\eta_4 = \frac{\theta_4 - \theta_4}{\theta_4}\]

9. \[E I \theta_2 = \frac{452 \times 24.2}{2} = 48.61 \times 10^6
  \]
  \[
  E I \Delta_7 = 48.61 \times 24.2 = 1066.6 \times 10^6
  \]

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\[ EI \Delta_T = -6324 + 8733 + 32328 + 22725 + 6769 + 1025 \]
\[ EI \Delta_T = 65296 \text{ kF}^3 \]

**NOW APPLY A 1kip LOAD UP AT B AND COMPUTE \( \Delta_1 \) DUE TO 1kip LOAD.**

\[ EI \alpha = 24(24)(12) = 288 \text{ kF}^2 \]
\[ EI \delta = 288(16) = 4608 \text{ kF}^3 \]

**REACTION AT PT. B:**

\[ R = \frac{\Delta}{\delta} = \frac{65296}{4608} = 14.17 \text{ kips} \]

**WALL LOADING FOR Dewatered CONDITION**
MOMENT DIAGRAM CALCULATIONS:

ASSUME OVERHANG MOMENT VARIES FROM 0 KF AT EL 45 LINEARLY TO 21.96 KF AT EL 34:

FROM EL 45 TO EL 34

\[ M_x = \frac{21.96(t)}{11} - \frac{0.67(t)^3}{6} \]

\[ M_{23} = \frac{+21.96(t)}{11} - \frac{0.67(t)^3}{6} = +5.69 KF \]

\[ M_{40} = \frac{+21.96(t)}{11} - \frac{0.67(t)^3}{6} = +8.53 KF \]

\[ M_{77} = \frac{+21.96(t)}{11} - \frac{0.67(t)^3}{6} = +10.25 KF \]

\[ M_{94} = -21.96 - \frac{0.67(t)^3}{6} = +11.09 KF \]

FROM EL 34 TO EL 20.5

\[ M_x = -21.96 - 14.17(t-11) - \frac{0.67(t)^3}{6} \]

\[ M_{30} = -21.96 - 14.17(4) - \frac{0.67(4)^3}{6} = +19.05 KF \]

\[ M_{26} = -21.96 - 14.17(3) - \frac{0.67(3)^3}{6} = +58.72 KF \]

\[ M_{25} = -21.96 - 14.17(2) - \frac{0.67(2)^3}{6} = +60.16 KF \]

\[ M_{24} = -21.96 - 14.17(1) - \frac{0.67(1)^3}{6} = +60.24 KF \]

\[ M_{23} = -21.96 - 14.17(0) - \frac{0.67(0)^3}{6} = +58.92 KF \]

\[ M_{22} = -21.96 - 14.17(-1) - \frac{0.67(-1)^3}{6} = +49.03 KF \]

FROM EL 20.5 TO EL 10

\[ M_x = 21.96 + 14.17(x-1) - \frac{0.67(x)^3}{6} - (x-16.33) - \frac{(x-16)^3}{6} \]

\[ M_{21} = 21.96 + 14.17(9) - 20.10(x-16.33) - 0.016(x-16)^3 = +12.83 KF \]

\[ M_{18} = 21.96 + 14.17(6) - 20.10(x-10.67) - 0.016(x-16)^3 = +11.07 KF \]

\[ M_{15} = 21.96 + 14.17(3) - 20.10(3.5) - 0.016(3.5)^3 = -11.07 KF \]

\[ M_{12} = 21.96 + 14.17(0) - 20.10(0) - 0.016(0)^3 = 70.57 KF \]

\[ M_{0} = 21.96 + 14.17(-1) - 20.10(13.5) - 0.016(13.5)^3 = -12.26 KF \]
ANALYSIS OF STRUTS

3. DEWATERING CONDITION

3.1. Analysis of Strut

3.1.1. Analysis of Strut load

3.1.1.1. Strut in Tension

3.1.1.2. Strut in Compression

3.1.2. Analysis of Water Pressure

3.1.2.1. Water Pressure Calculation

3.1.2.1.1. Water Pressure in Strut

3.1.2.1.2. Water Pressure in Water Chamber

3.1.3. Analysis of Soil Pressure

3.1.3.1. Soil Pressure Calculation

3.1.3.1.1. Soil Pressure in Strut

3.1.3.1.2. Soil Pressure in Water Chamber

3.1.4. Analysis of Strut Stability

3.1.4.1. Strut Stability Calculation

3.1.4.1.1. Strut Stability in Tension

3.1.4.1.2. Strut Stability in Compression

3.1.5. Analysis of Water Pressure Overflow

3.1.5.1. Overflow Calculation

3.1.5.1.1. Overflow in Strut

3.1.5.1.2. Overflow in Water Chamber
INNER HARBOR NAVIGATION LOCK

Stresses in Chamber wall during dewatering.

Check of computations by E.I.H. (Sheets 2 thru 14)

\[ f = 122 \, \text{lb/ft}^2 \]
\[ p = .55 \times 122 = 67 \, \text{lb/ft}^2 \]

\[ p = .55 \times 60 + 62.5 = 96 \, \text{lb/ft}^2 \]
HNC. Lock.

Stress in Chamber Wall

It is assumed that the strut spaced at 15' intervals is hinged at B since the stiffness of the strut is less than that of the wall.

Fix End Moments:

\[
F_A = \frac{.737 \cdot 24^2 \cdot 12}{30} = 35.4
\]
\[+ 1.608 \cdot 12 \cdot 1.5 \cdot 24 = 30.9\]
\[+ 0.3045 \cdot 5.25 \cdot \frac{3.69 \cdot .191 \cdot .29}{30} = 0.9\]
\[
\frac{67.2}{30}
\]

\[
F_B = \frac{.737 \cdot 24^2 \cdot 12}{30} = 35.4
\]
\[+ 1.608 \cdot 12 \cdot \frac{1}{10} \cdot 24 = 46.4\]
\[+ 0.3045 \cdot 5.25 \cdot \frac{4.37 \cdot 6.20 \cdot 24 \cdot 3.5}{30} = 85.3\]
\[
\frac{22.0}{30}
\]

\[
\begin{array}{c|c|c}
A & 8 & 8 \\
-37.1 & -67.2 + 7.1 & +7.1 + 7.1 \\
-12.2 & 0 & +7.1 + 7.1 \\
\end{array}
\]
NC. Lock
Tress in Chamber Wall

Sheet 3

\[ R_A = 0.737 \times 12 = 8.844 \]
\[ 1.608 \times 12 \times \frac{2}{3} = 12.864 \]
\[ 1.608 \times 20.5 \div 24 = 1.366 \]
\[ \frac{23.074}{23.074} \]

\[ R_B = 0.732 \times 12 = 8.844 \]
\[ 1.608 \times 12 \times \frac{2}{3} = 6.932 \]
\[ 1.608 \times 3.5 \times 24 = 0.234 \]
\[ \frac{15.510}{15.510} \]
VNC. Lock
tree in Chamber Wall

For Pt. A Fixed and Pt. B Pinned

4.08 k

10.11 k

122.4 k

28.47 k

-123.4 k

1° 58' E

Horizontal Curve

1° 16' E

47.1 k
4.4.6. Lock

Firn in Chamber Wall:

Mom. Capacity of Wall

Wall at El. 10.0

\[ 6(Kd)^2 + 2.35 \cdot 19(Kd - 4) = 10 \cdot 7.85(44 - Kd) \]

\[ 6(Kd)^2 + 52.35Kd - 52.3 = 0 \]

\[ (Kd)^2 + 8.75Kd - 8.77 = 0 \]

\[ Kd = -4.37 + \sqrt{106.4} = 5.9'' \]

\[ C_1 = f_c \cdot 12 + 5.9 = 35.4f_c \cdot 35.4 \cdot \frac{5}{3}f_c = 70.0f_c \]

\[ C_2 = f_c + \frac{5.9}{3.9} \cdot 19.235 = 14.4f_c + 4.4f_c \cdot 4 = 576f_c \]

\[ \frac{49.8f_c \cdot 2}{Z = 127.6f_c} \cdot Z = 2.55'' \]

\[ dy = 44 - 2.55 = 41.45'' \]

\[ f_c = 1050, \quad f_c = 49.8 \cdot 41.45 \cdot 1.05 = 180'' \]

\[ f_c = 18,000, \quad f_c = 7.85 \cdot 18 \cdot 41.45 = 48.7'' \]

The fixed end moment calculated is 182.4", therefore the beam will not be fixed and a but either pin or fixed is a plastic hinge is placed with no moment.
VC Lock
Reinforced Chamber Wall

The ultimate capacity of moment at A will be

\[ M_{u} = 0.785 \times 40 \times \frac{44 - 1.03}{12} \]

With no steel in compression face:

\[ a = \frac{0.785 \times 40}{0.85 \times 3 + 12} = 1.03'' \]

\[ f_{u} = 0.785 \times 40 \times \frac{44 - 1.03}{12} = 112'' \] (Elastic limit 40)

With 3 - 2.8 compressive steel:

\[ f_{u} = 0.785 \times 40 \times \frac{40}{12} = 103.5'' \]

**Computed Moment** = 122.4'' > 103.5''

**Allowable Moment** = 48.7''

\[ \% \text{Stressed Above Mallow} = \frac{122.4}{48.7} = 2.51\% \]
HNC LOCK

Stress in Chamber Wall

WALL ACTING AS A SIMPLE BEAM

Assume Hinge at A: (\( \gamma_A = 0 \))

\[ \gamma_{\text{max}} = \frac{114.6 + 7.1 \times \frac{14}{24}}{24} = 119 \text{ k} \text{ at E1.20.} \]

Depth of wall at E1.20.0 = 3'-6"

\[
\begin{array}{c|c|c|c}
\text{Floor} & \text{Depth} & \text{E} & \text{F} \\
\hline
3'-8" & \frac{7}{12} & \times & \frac{3}{4}\text{"} \\
\hline
3'-6" & \frac{1}{4}\text{"} & \times & \frac{1}{2}\text{"} \\
\hline
\end{array}
\]

\[ p = \frac{785\times 3}{38\times 12} = 0,0052 \]

With \( f_c = 3000, \ y = 10, \ k = 0.27, \ f = 0.91 \)

\[ e = \frac{2 + \frac{119,000\times 12}{0.27\times 0.91\times 12\times 38}}{0.27\times 0.91\times 38} = 670 < 1050 \]

\[ f_s = \frac{119,000\times 12}{2.35\times 0.91\times 38} = 17,600 \text{ psi} \]
HNC Lock
Stress in column wall

Beam at E1 34.
Span 15', Load 15.51 + 4.05 = 19.56 k

\[ M_{\text{max}} = 19.56 \times 15 \div 12 = 36.8 \text{ k} \]

\[ A_0 = 3.93 \text{ in} \times 6.5 = 68'' \]

\[ 18 (K_d) + 19.3 (K_d - 9) = 10 \times 3.93 (K_d - 9) \]

\[ K_d = 10.05 \]

\[ C_1 = \frac{36}{2} \times 10.05 = 181.1 \text{ ft}^2, \quad 181.1 \text{ ft}^2 \times \frac{605}{3} = 607 \text{ ft}^2 \]

\[ C_2 = \frac{10.05 - 4}{10.05} \times 3.93 \times 17.45 \text{ ft}^2 = 180 \text{ ft}^2 \]

\[ f_d = 68 - 3.5 = 64.5'' \]

\[ f_c = \frac{368000 \times 12}{226.1 \times 64.5} = 300 \text{ psi} \]

\[ f_s = \frac{368000 \times 12}{3.93 \times 64.5} = 17400 \text{ psi} \]
4NC Lock

Stresses in Lock Walls

STRUT: $T = 19.56 \times 15 = 293.2 \text{kN}$

$T_3 = \frac{293.2 \times 40.56}{20.5} = 580 \text{kN}$

(assume $o at A$)

Lengthening of Strut:

$\delta = \frac{20.500}{30 \times 10^6} \text{ in/min.}$

Total $\delta = \frac{20.500 \times 33 \times 12}{30 \times 10^6} = 0.27''$

$\delta = \frac{580,000}{28.3} = 20,500 \text{ psi.}$

Strut is O.K. provided no moment is transferred to it from load on wall. Lengthening of strut would cause and bend moment at A, since A is considered.
HWC Lock

Strut in Lock Wall.

Rotation of joint at B due to
load on BA as shown on Sheet 3:

Load BM with H, loading and
find RB:

\[ 7 \times R_A \times 24 = \]

\[
\begin{array}{ccc}
46.0 \times 22.67 & 930 & \text{Ft Kips x Ft}^2 \\
113.1 \times 20.91 & 2360 & \\
166.4 \times 18.95 & 3150 & \\
202.7 \times 16.97 & 3440 & \\
223.0 \times 14.98 & 3350 & \\
228.6 \times 13.00 & 2970 & \\
222.1 \times 11.01 & 2440 & \\
212.9 \times 9.62 & 1830 & \\
172.2 \times 7.01 & 1220 & \\
132.9 \times 5.05 & 670 & \\
85.0 \times 3.10 & 263 & \\
29.5 \times 1.33 & 39 & \\
\hline
1819.9 & 22652 & \\
\end{array}
\]

\[ EI \frac{R_A \times 22652}{24} = 945 \text{ Ft Kips x Ft} \]

\[ R_B = \frac{874.4}{EI} \]

\[ G = \frac{874.4 \times 1000 \times 12}{3 \times 10^6 \times \frac{1}{12} \times 42^3 \times 12} = \frac{126}{3 \times 74 \times 10^3} = \frac{5.6}{10^4} \text{ rad/m} \]
INC Lock.

Rises in lock Walls.

Moment in strut due to rotation of wall:

\[ \theta = \frac{5.6}{10^3} \text{ radian} \]

\[ M = 4EK\theta \]

\[ M = 4 \times 3 \times 10^6 \times 36 \times 5.6 \times \frac{5.6}{12 \times 35 \times 12 \times 10^4 + 12 \times 10^2} \]

\[ M = \frac{1296 \times 1296 \times 5.6}{35 \times 144 + 10} = \frac{9 \times 1296}{62.5} = 176 \text{ kN} \]
Y NC Lock
Strut on Lock Wall

Moment assumed 0 at A.
AB and BC continuous.
Strut fixed at C

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
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<tbody>
<tr>
<td>-85.3</td>
<td>-672</td>
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<td>+620</td>
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<td>-7.2</td>
<td>-9.8</td>
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<td>+7.2</td>
<td>+9.0</td>
<td>-0.8</td>
</tr>
<tr>
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<tr>
<td>+7.1</td>
<td>+4.5</td>
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</tr>
<tr>
<td>-5.6</td>
<td>-11.7</td>
<td>+5.8</td>
</tr>
</tbody>
</table>

Moment on strut at B = 15 + 11.7 = 175 k"
Stresses in Lock Wall

**Strut**

16.5" x 8.5" = 27.5"

Tension in Strut = \( \frac{19.79 \times 15 \times 40.56}{20.5} \)

\( f_t = \frac{585000}{28.2} = 20,740 \text{ psi} \)

\( p = \frac{14.14}{36.275} = 0.014 \)

\( \eta = 10, \ K = 0.39 \ j = 0.87 \)

\( f_{\text{max}} = \frac{175,000 \times 12}{0.40 \times 6.87 \times 36 \times 27.5^2} = 440 \text{ psi} \)

\( f_3 = \frac{175,000 \times 12}{14.14 \times 6.87 \times 27.5} = 6,200 \text{ psi} \)

**Total Max Steel Strain** = 6,200 + 20,740

\( = 26,940 \text{ psi} \)

The moment in wall can be 0 at El. 40.0 without the wall failing. The steel stress in the strut will be 26,940 psi. However...
Strut: BC

Tension in BC = 19.2 × 15 × 90.56 = 587.20,50

A = 16 - 1/2'' bars = 28.27''

f = \frac{587000}{28.27} = 20800 psi

Hmax = 13.08 × 15 = 196.2 k

\frac{H}{6d^2} = \frac{196.200 × 12}{36 × 27.5^2} = 87

f_s = \frac{196.200 × 12}{14.135 × 864 × 27.5} = 7000 psi

Total Tensile stress in strut = 27,800 psi

Sheet 14 of 14
APPENDIX C
ULTIMATE MOMENT CAPACITY OF CRITICAL WALL SECTION
NOTATION

\( a \)  Depth of Whitney compressive stress block, inches
\( A_s \)  Area of tension reinforcing steel, square inches
\( A'_s \)  Area of compression reinforcing steel, square inches
\( d \)  Depth of section, inches
\( f'_c \)  Allowable stress for steel, psi
\( f_y \)  Yield stress for steel, psi
\( M_u \)  Ultimate moment of the section, ft-kips
\( N \)  Compressive force on the section, kips
\( N_T \)  Tensile force on the section, kips
\[ f' = 2,200 \text{ PSI} \]
\[ f_y = 33,000 \text{ PSI} \]

**Underreinforced Beam**

\[ d = 48" - 4" = 44" \]

1. **Ignore Compressive Steel**

\[ \bar{N}_c = \bar{N}_T = A_S f_y = 0.85 f'_c \cdot a \cdot b \]

\[ a = \frac{A_S f_y}{0.85 f'_c b} = \frac{0.79 (33,000)}{0.85 (2,200) (12)} = 1.16 \text{ IN.} \]

\[ \bar{M}_u = \bar{N}_T (d - a/2) \]

\[ = A_S f_y (44 - 1.16/2) = 0.79 (33,000) (43.4) \]

\[ = 1,132 \text{ IN.-KIPS} \]

\[ = 94.3 \text{ FT-KIPS} \]

**Using Ultimate Strength Analysis Methods**

And ignoring compressive steel, the section has an ultimate moment capacity of 94.3 FT-KIPS

\[^{a}P. M. \text{ FURGESON. SEE REFERENCE 3 OF MAIN TEXT.}\]
APPENDIX D
NOTATION
$A_s$  Area of tension reinforcing steel, square inches

$A'_s$  Area of compression reinforcing steel, square inches

$b$  Width of section, inches

$c$  Distance from center of section to outside faces, inches

$E$  Modulus of elasticity, psi

$E_c$  Modulus of elasticity for concrete, psi

$E_s$  Modulus of elasticity for steel, psi

$F$  Nodal point force

$h$  Length of section, inches

$h_x$  Distance below the surface, feet

$I$  Moment of inertia of section, inches $^4$

$k$  Structure stiffness matrix

$k_0$  Coefficient of lateral earth pressure, 0.55

$M$  Section moment, ft-kips

$\phi_{hx}$  Horizontal earth pressure at $x$ distance below the surface

$\phi_{hy}$  Horizontal earth pressure at $y$ distance below the water table

$x$  Nodal point displacement

$\gamma_{soil}$  Soil density, 122 lb/ft$^3$

$\gamma_{submerged}$  $\gamma_{soil}$ minus $\gamma_{water}$

$\gamma_{water}$  Water density, 62.4 lb/ft$^3$

$\nu$  Poisson's ratio

$\nu_c$  Poisson's ratio for concrete

$\nu_s$  Poisson's ratio for steel

$\sigma_i$  Vertical stress on chamber face, psi

$\sigma_o$  Vertical stress on outside face, psi
In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Rieck, Peter J
Structural analysis of the New Orleans Inner Harbor Navigational Canal lock wall, by Peter J. Rieck and Robert E. Walker. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1975.
80 p. illus. 27 cm. (U. S. Waterways Experiment Station. Miscellaneous paper N-75-8)
Prepared for U. S. Army Engineer District, New Orleans, New Orleans, Louisiana.
TA7.W34m no.N-75-8