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ENGINEERING CONDITION SURVEY AND STRUCTURAL INVESTIGATION OF EMSWORTH LOCKS AND DAM, OHIO RIVER

by

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Final Report

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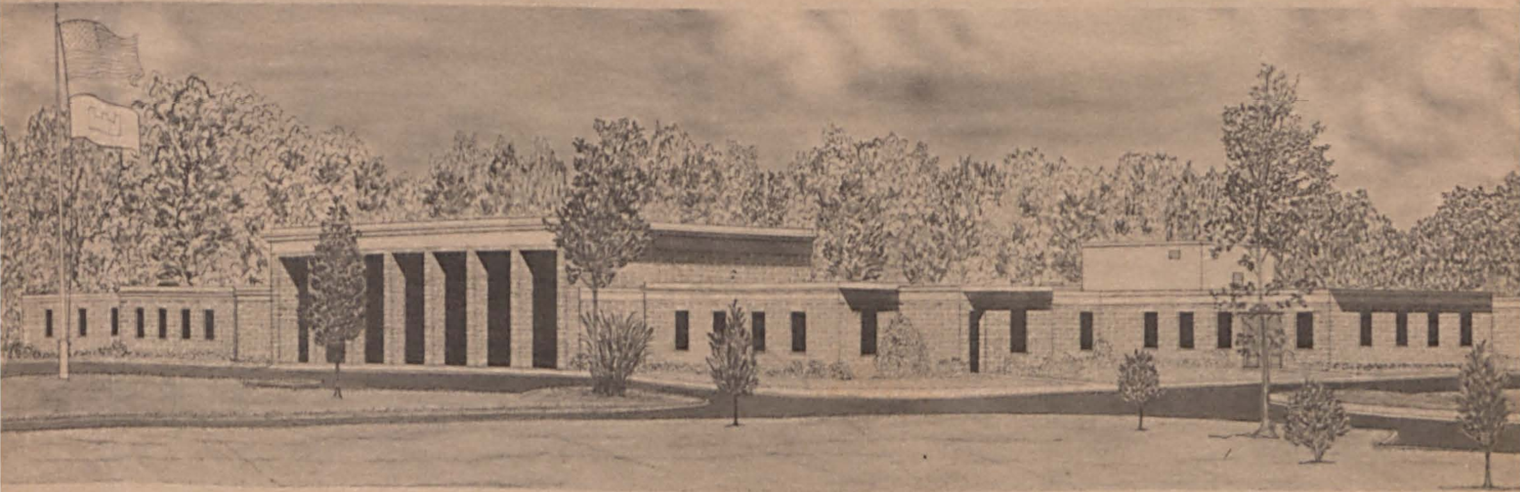

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The Concrete Laboratory at the U. S. Army Engineer Waterways Experiment Station was contracted to prepare an engineering condition survey and structural investigation for Locks and Dam 3, Monongahela River, and Emsworth and Montgomery Locks and Dams, Ohio River. This report gives the results of an engineering condition survey and a structural analysis of Emsworth Locks and Dam, Ohio River.

In general, the monoliths on the land wall do not meet present day criteria

(Continued)

for overturning, sliding, or base pressures. Also, some monoliths in the middle and river walls do not meet present day stability requirements. In fact, the stability analysis of M-22 along with the visual observation of a 1-1/2 in. separation between the ceiling of the emptying culverts and the middle wall indicates that there has been some movement of these middle wall monoliths.

The main concern for concrete integrity is the cracked, spalled, and deteriorated surface concrete which will allow accelerated deterioration reducing the effective section of the monoliths increasing the already excessive tensile stresses. In general, if corrective measures are not taken, this will surely cause maintenance expense and will also reduce the life of the concrete monoliths. The compressive stresses are larger than indicated by the stress analysis and can also cause problems in deteriorated concrete.

From the deteriorated condition of the surface of the lock monoliths, it is evident that some action must be initiated. Since corrective action is needed, a feasibility study should be made to determine what action is necessary which will provide the most economical and adequate lock usage over a period of 30 to 50 years. For this reason, it is recommended that a feasibility study be made considering the following alternatives:

_a_. Minimum maintenance and protection of the locks and dam from weathering with expected replacement when needed as determined by periodic inspections.

_b_. Rehabilitation of locks and dam.

_c_. Replacement of locks and dam.

The above recommendations may be affected by a total structural and operational evaluation. In fact, this study does not evaluate the steel gates, bridge work, lock gates, or appurtenant mechanical or electrical facilities; these will be considered by the Pittsburgh District in the overall evaluation of the locks and dam.

PREFACE

The work of a engineering condition survey and structural investigation for Emsworth Locks and Dam located on the Ohio River was conducted for the U. S. Army Engineer District, Pittsburgh, Corps of Engineers, by the Concrete Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES).

The contract was monitored by the Pittsburgh District Office with main assistance from Messrs. J. Colletti, H. Ferguson, J. Gribar, and S. Long.

The cooperation and assistance of all personnel at the District Office were greatly appreciated.

The study was performed under the direction of Messrs. B. Mather, J. M. Scanlon, and J. E. McDonald, CL. The structural analysis was performed by Messrs. C. E. Pace, R. L. Campbell, E. F. O'Neil, J. T. Peatross, and Major H. Beardslee. The material properties were obtained by Messrs. R. L. Stowe, F. S. Stewart, and J. B. Eskridge. The report was written by Dr. Carl E. Pace.

The Director of WES during the conduct of the program and the preparation and publication of this report was COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Mr. F. R. Brown was Technical Director.

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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:

Multiply	By	To Obtain
inches	2.540000 E-02	metre
feet	3.048000 E-01	metre
pounds (mass)	4.535924 E-01	kilogram
pounds (force)	4.448222 E+00	newton
pounds (mass) per cubic foot	1.601846 E+01	kilogram per cubic metre
pounds (force) per square inch	6.894757 E-03	megapascals
tons (force) per square foot	9.576052 E-03	megapascals
feet per second	3.048000 E-01	metre per second

ENGINEERING CONDITION SURVEY AND
STRUCTURAL INVESTIGATION OF EMSWORTH LOCKS AND DAM
OHIO RIVER

SECTION 1: INTRODUCTION

1.1 This report presents the results of an engineering condition survey and a structural analysis of Emsworth Locks and Dam (Figures 1.1 and 2.1) on the Ohio River. The investigation was conducted from October 1974 to August 1976 by the Waterways Experiment Station (WES) for the U. S. Army Engineer District, Pittsburgh (ORP). Authorization for the investigation was given in DA Form 2544, dated 23 October 1974, issued by ORP.

1.2 ORP initiated the investigation of Emsworth Locks and Dam by their Periodic Inspection Report.[1] The report reviews the construction and the general condition of the locks and dam with attention to specific problem areas. The results of the periodic report warranted further study. A condition survey[2] was conducted for ORP by WES to determine the quality of the concrete and locate cracks that could affect the structural integrity of the locks and dam. The present study was then initiated to determine if there exists a need to consider the rehabilitation or replacement of the structures. If this need exists, a separate study will be initiated to study the feasibility of rehabilitation or replacement.

Location of Study Area

1.3 Emsworth Locks and Dam are located on the Ohio River about five miles north of Pittsburgh, downstream of the confluence of the Allegheny and Monongahela Rivers, and consist of two structures; one on each side of Neville Island. Two locks, a dam, a fixed weir, and an abutment are located across the main channel 6.2 miles below the head of the Ohio River at Pittsburgh. The second structure is a six-gated dam across the back channel at river mile 6.8. The locks are located on the right bank adjacent to the Borough of Emsworth on the narrow floodplain developed by Lowries Run. The Pennsylvania Railroad with its main line tracks is immediately adjacent to the locks.

Purpose and Approach

1.4 The purpose of this report is to present the findings and conclusions drawn from the condition survey and structural investigation of Emsworth Locks and Dam. This study does not evaluate the steel gates, bridge work, lock gates, or appurtenant mechanical or electrical facilities; these will be considered by the Pittsburgh district when the overall condition of the locks and dam is evaluated.

1.5 This investigation is limited to:

a. Foundation evaluation.

b. Structural property determination of foundation and concrete.

c. Stability analysis of selected monoliths of the locks and dams.

d. Stress analysis of selected monoliths.

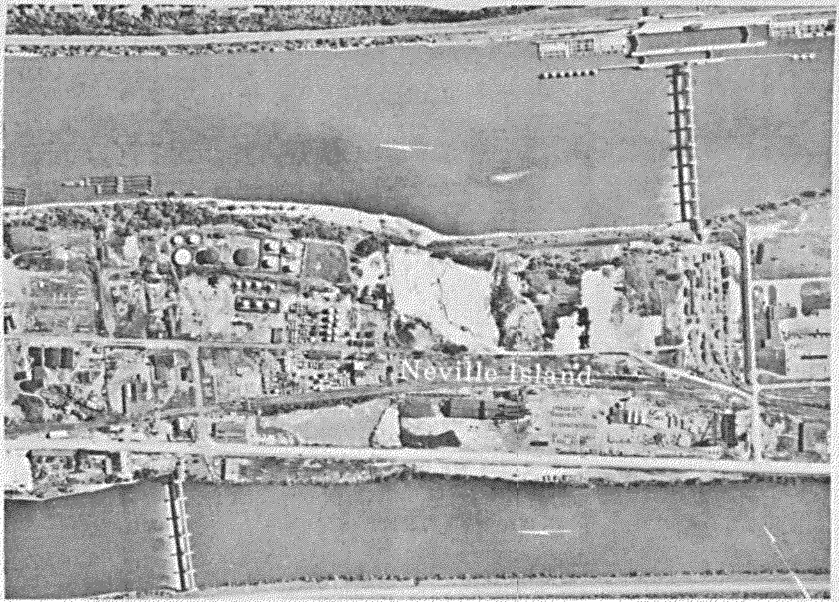
1.6 The evaluation of the structures as given in this report is relative to the above specific studies; although, concrete integrity, concrete deterioration, conditions which may cause immediate failure, existance and extent of structural cracking, and the alignment or settlement of the various structural monoliths are given consideration.

Historical Construction

1.7 The original structures of Emsworth Locks and Dam were built during 1919 to 1922 and replaced Ohio River Locks and Dams No. 1 and 2 which consisted of movable dams and single locks. The original dam at Emsworth was an uncontrolled overflow weir. This dam was replaced in 1936-1937 by a vertical-lift, gate controlled, low-sill structure to provide a deeper and more stable pool at Pittsburgh. This increased the head differential on the lock monoliths above that for which they were orginally designed. This created a harbor with a shoreline of approximately 50 miles in the Pittsburgh area. The Emsworth pool extends to Lock 2 on the Allegheny River (mile 6.7) and extends to Lock 2 on the Monongahela River (mile 11.2).

1.8 The new dam uses a portion of the old structure for a stilling basin. The dam is in two parts with Neville Island forming the barrier between the main and back channel structures. The overall length of the main channel dam, from the river wall to the abutment, is 967.42 ft, including a 34.42-ft weir with a crest at elevation 709.0 adjacent to the river wall. The back channel dam has an overall length of 750 ft from abutment to abutment. The navigation pool is controlled by eight gated sections in the main channel and six in the back channel, each 100 ft in length with a damming height of 12 ft above the sill at elevation 698.0.

1.9 The Emsworth Locks are dual, adjacent, parallel chambers with horizontally framed mitering gates for the 56-ft lock and vertically framed gates for the 110-ft lock. The upper guide wall and both the upper and lower guard walls of the locks were extended when the dam was rebuilt in 1936-1937. The land chamber has clear dimensions of 110 ft by 600 ft and the river chamber has clear dimensions of 56 ft by 360 ft. The lift between lower pool (elevation 692.0) and upper pool (elevation 710.0) is 18 ft. Depths to the poiree dam foundation control the lower lock approaches and will accommodate drafts of 12.9 ft. The structural and mechanical make-up of the locks and dam is presented in Appendix VI of Reference 1. Since the locks and dam have been built, various repairs using gunite and concrete surface overlays have been used to protect the deteriorating surfaces. These repairs have deteriorated and the original concrete is exposed in many areas.



26 AUG. 1963

27 AUG. 1963

Figure 1.1 Aerial view of Emsworth Locks and Dam

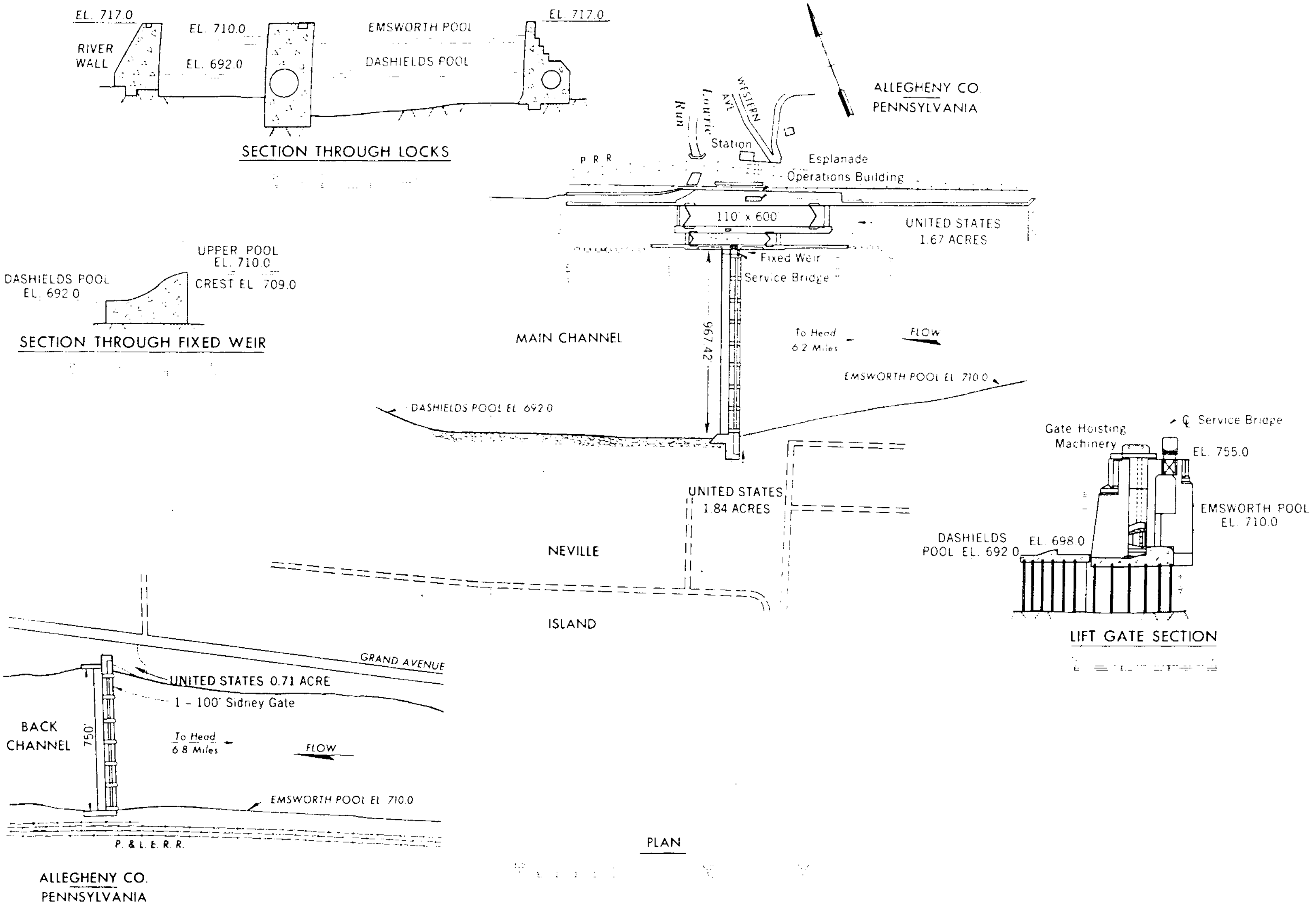


Figure 1.2 Plan and sections of Emsworth Locks and Dam

SECTION 2: SURFACE CONDITION OF CONCRETE

2.1 The land, middle, and river wall monoliths are in the advanced stages of deterioration; especially the concrete which is 1-1/2 ft to 4-1/2 ft below the surface. The concrete from 1-1/2 to 4-1/2 ft is fragmented to such a degree that it could not be tested and any compressive strengths applies to that below these depths. The top surface of the walls shows spalled areas, numerous random cracks, and signs of extreme weathering.

2.2 The upper guide wall addition, which was constructed in 1937 and resurfaced in 1957, is not as deteriorated as the original guide wall. The original guide wall is gouged, badly spalled, weathered, and deteriorated.

2.3 The esplanade has experienced differential settlement causing low areas to develop and hold water (Figure 2.1). This condition is bad for foot traffic and results in water seeping into the foundation causing accelerated settlement and backfill saturation to a higher elevation than would be. This subjects the land wall monoliths to greater loadings thereby affecting their stability.

2.4 The chamber walls were resurfaced in 1931 but the "gunite" surfacing is worn or deteriorated and in a lot of areas has loosened and been removed or has fallen away from the walls (Figure 2.2). At times, some of the loose gunite has to be removed by the lock personnel to eliminate it falling and hurting someone.

2.5 There are many horizontal and vertical cracks in the walls of the locks. The exterior surface has been replaced, repaired, and resurfaced to such an extent that, in general, a correlation of the interior and exterior cracks was unsuccessful. An indication of interior cracks was determined from boreholes, core examinations, and pulse velocity measurements. All of these indicators are extremely useful but the borehole and core data gave only isolated observations.

2.6 The width of the cracks in the top surface[2] was as wide as 1/4-in. and as wide as 1/2-in. in the vertical wall surfaces. The open lift lines and open joints varied in width from 1 to 6-in. with the larger widths being in the open vertical joints. These openings are of concern because of their high susceptibility to frost action. The crack locations given in Reference 2 are presented in Figure 2.3. The concrete is non-air entrained which will allow progressive frost damage.

2.7 The main problem that is apparent concerning the original guide, land, middle, and river walls is that there has been considerable spalling and weathering of the concrete surface. The open cracks allow water penetration which results in accelerated deterioration due to freezing and thawing. This means that accelerated deterioration will occur if corrective action is not taken.

2.8 The surface condition suggests that the total surface of the original walls has to be rehabilitated to check accelerated deterioration and to make the use of the locks feasible for a time beyond that necessary for structure replacement.

2.9 The feasibility consideration of rehabilitation or replacement is not an objective of this report but the deteriorated surface condition of the concrete suggests that this study is necessary. Of course, any consideration of replacement or rehabilitation must include the structural as well as operational performance of the locks and dam as related to present and future needs.

Figure 2.1 Differential settlement in the Esplanade which holds water

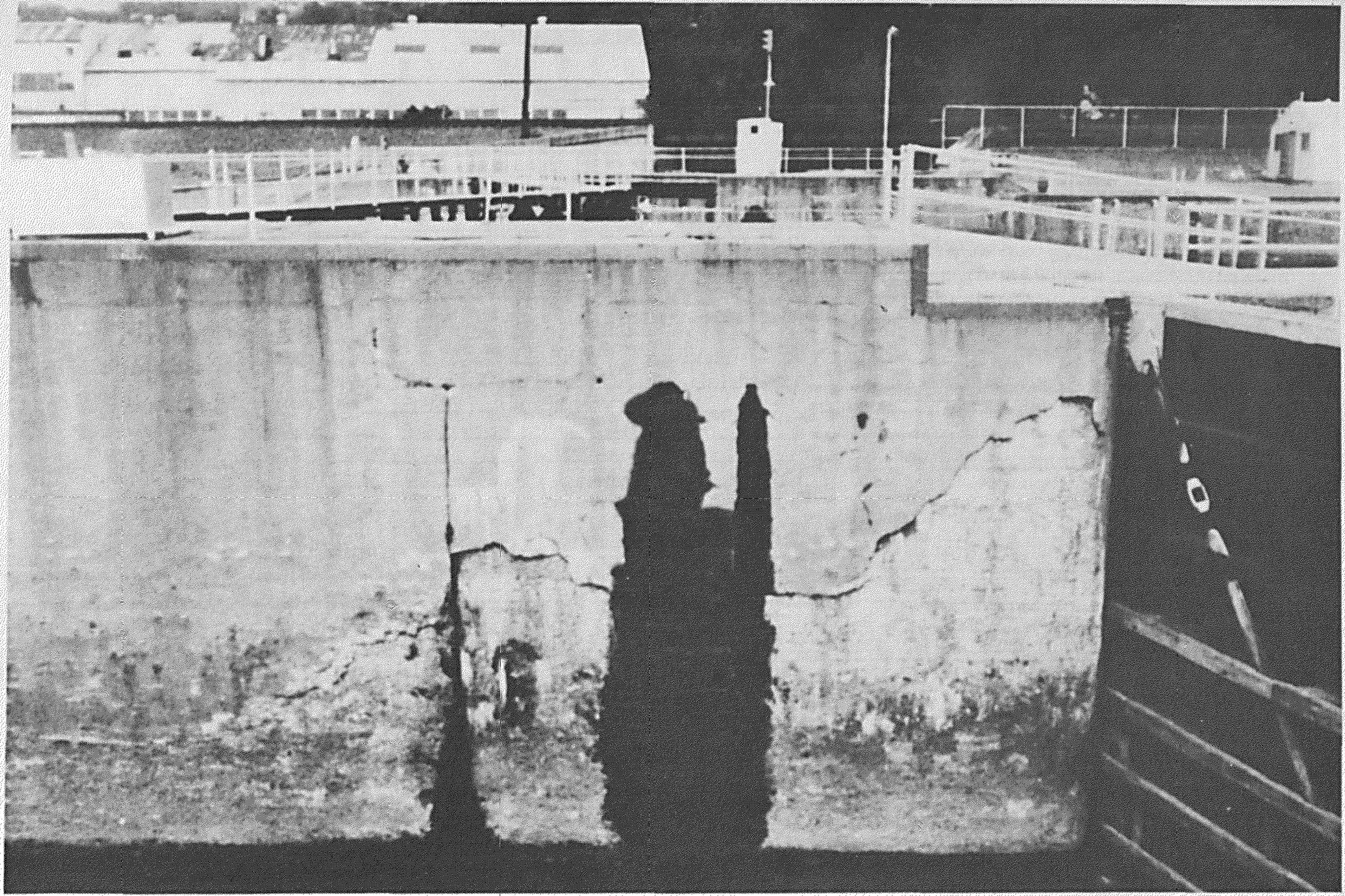


Figure 2.2 Deteriorated shotcrete resurfacing

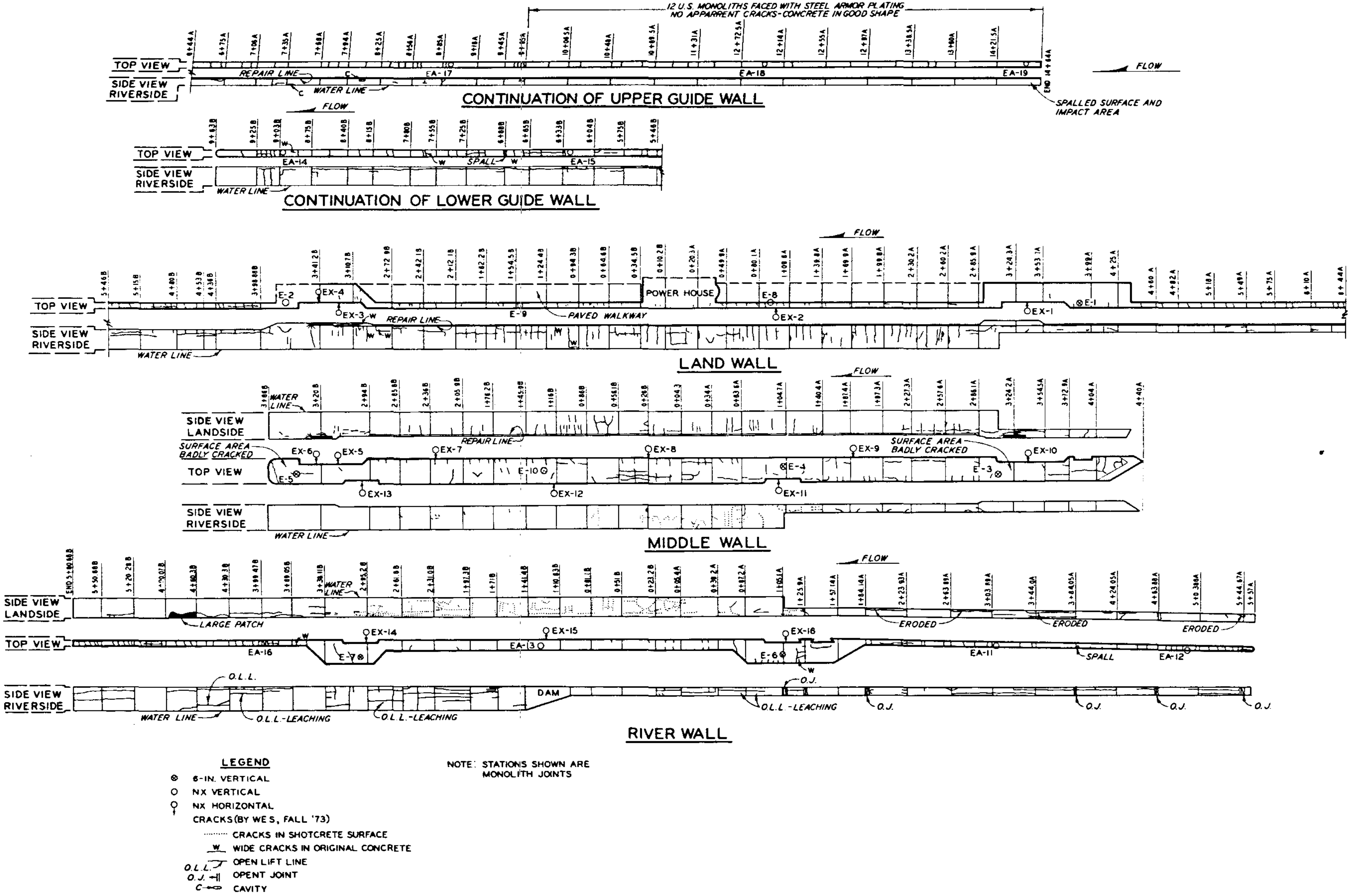


Figure 2.3 Crack Survey and Bore Hole locations on Emsworth Locks and Dams, Ohio River

SECTION 3: GENERAL CONDITION OF FOUNDATION

3.1 In the past, foundation inadequacies have not caused any structural or operational problems. The main considerations concerning the foundation are:

- a. In general, the contact between monoliths and foundation is tight.
- b. Even though the foundation is badly fractured, it is adequate for the construction at Emsworth Locks and Dam.
- c. The fractures are local and do not fit into any pattern of weakness which could result in some overall sliding or failure at the Lock and Dam.

3.2 The concern at Emsworth Locks and Dam is not with the foundation but is with the surface concrete deterioration and design inadequacies as are discussed in this report.

3.3 Core boring information, core logs, and borehole camera data are given in Reference 2.

SECTION 4: CONCRETE INTEGRITY

4.1 The main concern for the concrete integrity at Emsworth Locks is the first 1-1/2 to 4-1/2 ft of deteriorated concrete. Freezing and thawing action in the non-air-entrained concrete has caused damage to the top few feet of the exposed surface. This deterioration (Figure 3.1) will accelerate if corrective action is not taken. The action to take will depend upon the most feasible alternative of rehabilitation or replacement by considering the total locks and dam situation.

4.2 In any case, if the locks and dam are to be operational for over six to eight years, the cracks, spalled areas, open lift and joint lines, and deteriorated surfaces should be repaired by either armor plating or sealing whichever is appropriate. Also, where possible, concrete at critically stressed areas should be checked for deterioration. The general areas where stress concentrations occur will be discussed in Section VII. The locations of surface cracks, cavities, eroded areas, open monolith joints, and open lift lines are shown in Figure 2.3. The vertical and horizontal core locations are also shown in Figure 2.3. If the weathering is allowed to continue, deterioration will progress causing problems which will make replacement of the structure necessary at a much earlier time than if the surfaces were sealed.

4.3 Visual observation of surface cracks (Figure 2.3) does not provide adequate information about their depth; therefore, supplemental methods such as soniscope investigation, direct examination of cores, and indirect examination of core holes by photography were necessary. These examinations were completed and their results reported in Reference 2. A relative position correlation of interior and exterior cracking was found in only one location. This evidence reinforces the concept that the main consideration is to stop concrete deterioration and assure adequate strength in places where stress is important (Chapter 7). The borehole photography showed a lot of honeycomb or void-space areas in the concrete which indicates imperfect consolidation of placed concrete. The borehole observations are local and far apart, therefore, conclusions from this data concerning specific monoliths cannot be made.

4.4 Concrete below the outer few feet of exterior surface has an average compressive strength of 4000 psi.[2] This strength is more than adequate for a gravity structure except in some isolated areas of tensile stress (tensile strength ≈ 10% compressive strength).

4.5 There are isolated problems with the adequacy of the concrete. There tends to be a concentration of cracks, especially at the corners, in the horizontal cutouts of each wall where the gate arms are recessed (Figure 3.1).

4.6 The rebound hammer tests show that the bond of the shot concrete to the underlying concrete is consistent and generally good. This was substantiated by horizontal cores in the small lock chamber. There is definitely a problem with the shotcrete surface, because, a lot of the surface has been worn away or removed and at times loose pieces can be seen hanging from the walls. These two nearly conflicting views leads to the conclusion that the concrete is being loosened from the lock walls by river traffic and is, therefore, of value for only a limited period of time.

4.7 The top of a monolith on the lower guard wall was sheared and shifted more than 5 in. when the monolith was hit by a work boat. This points up the extreme forces which can be applied by impact to the lock monoliths.

4.8 If the structures are not replaced, the monoliths of the locks and dam are in immediate need of having the numerous random cracks and spalled areas sealed. The main question here is whether to make permanent repairs or to just neatly seal the surfaces from the weather. This consideration can only be answered by a feasibility study of either:

a. Maintenance with expected replacement when needed as determined by periodic inspections.

b. Complete rehabilitation.

c. Replacement of the structures.

4.9 It may be more economical, as far as the concrete integrity is concerned, to seal the deteriorated surfaces in order to add life to the structures and delay replacement until periodic inspections show that the feasibility breakpoint has been reached and replacement is necessary. If a consistent treatment of complete rehabilitation is considered for the locks and dam along with a consideration of the operational life of the structure, replacement will probably be more feasible. This is a guess; therefore, the feasibility study needs to be made.

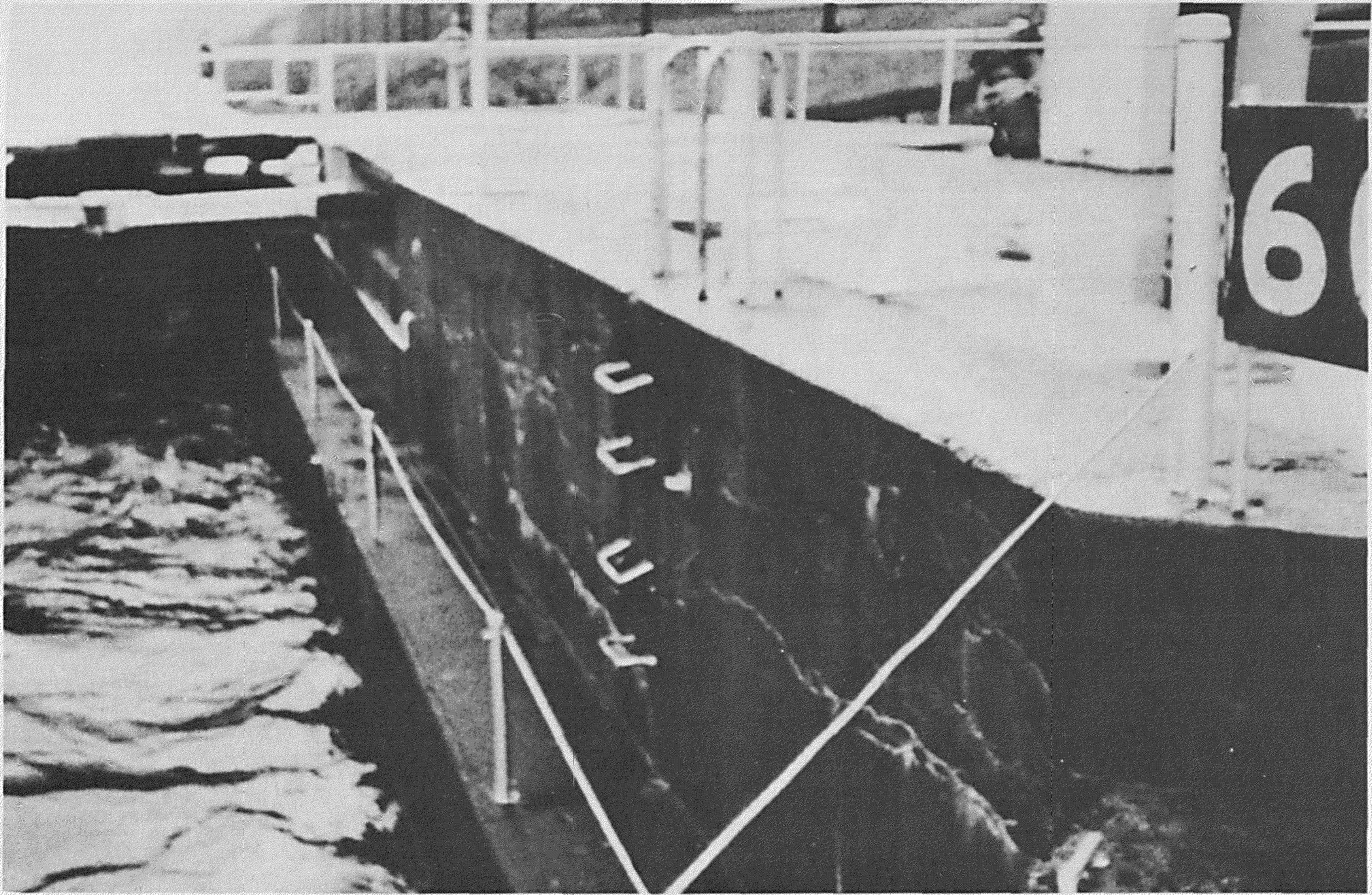


Figure 3.1 Concrete surface deterioration

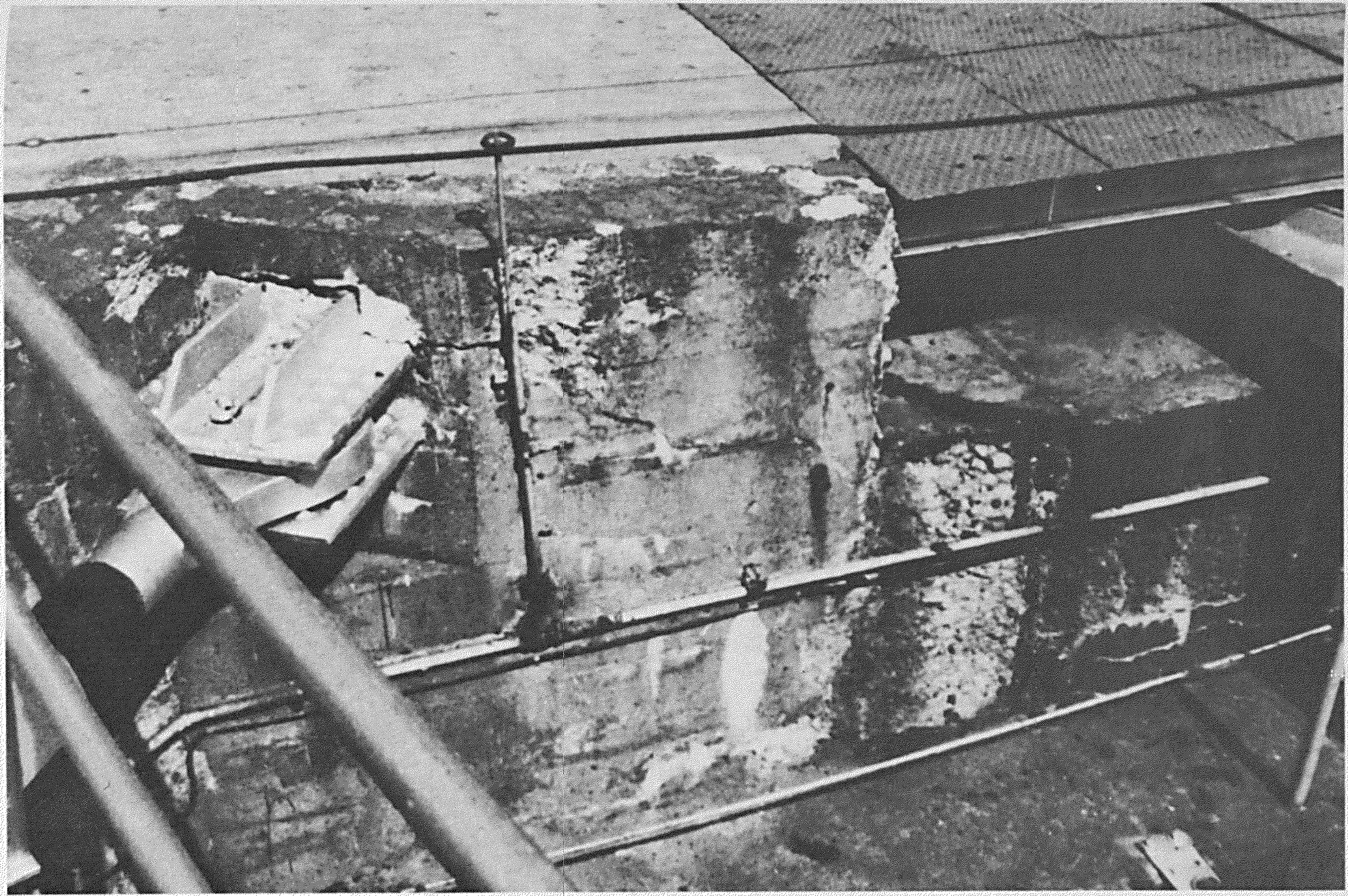


Figure 3.2 Cracks at gate arm recesses

SECTION 5: LABORATORY TESTS

Introduction and Problem Statement

5.1 The structural investigation requires the use of physical property data for the backfill, foundation, and concrete. The material properties of unit weight, compressive strength, triaxial and direct shear strengths, and various elastic constants are needed.

Material Properties

5.2 The gravity walls are supported on component rock; therefore, the "at rest" pressure coefficient should be used for obtaining horizontal pressures. It may be that the actual horizontal pressure coefficient is lower than the "at rest" value, but the only way to get actual values is to make a number of tests at the lock and dam site. The scope of this work in time and funding is not such that this type of testing is possible. Since the railroad track is at the back of the landwall monoliths, the vibration will ~~decrease internal soil resistance and~~ cause the backfill to be supported more than normally by the monoliths. In this respect, it would not be safe to consider less than 0.5 for the horizontal pressure coefficient. More discussion concerning the selection of the horizontal pressure coefficient is given in Appendix C. The unit weight of the backfill is given in Table 5.1.

5.3 The concrete properties were obtained from cores. The tests yielded the following information:

- a. Compressive strength, qu, and unit weights, γ.
- b. Modulus of elasticity, E.
- c. Poisson's ratio, μ.
- d. Shear modulus, G.

5.4 The unit weights for the foundation rock cores were obtained using measured volumes and weights. The average value is given in Table 5.1. The unconfined and triaxial compression test specimens were prepared according to standard method of test for triaxial strength of undrained rock core specimens, CRD-C 147.[3] The specimens were cut with a diamond-blade saw and the cut surfaces were ground flat to 0.001 in.; specimens were checked for parallel ends and the perpendicularity of ends to the axis of the specimen. Two vertically and three horizontally mounted linear potentiometers, respectively, were used to measure the vertical and diameter change during compression testing. The displacement measurements were then used to calculate the axial strain, ε_a , and the diametric strain, ε_d . The modulus of elasticity, Poisson's ratio, and shear modulus were calculated from the stress-strain data. Axial specimen load was applied with a 440,000-lb-capacity universal testing machine. Confining pressure for the triaxial compression test was applied by a hand-operated electrohydraulic pump.

5.5 The direct shear test specimens were prepared according to applicable portions of the standard method of test for shear strength, CRD-C 90.[3] The direct shear tests on intact shale were conducted using normal loads, σ_n , of 33, 66, and 100 psi. Tensile test specimens were prepared according to standard method of test for splitting tensile strength of concrete specimens, CRD-C 77.[3]

5.6 At the concrete-foundation rock interface it is required to know the coefficient of sliding friction and the cohesion. A multistage triaxial test was conducted to obtain these values.

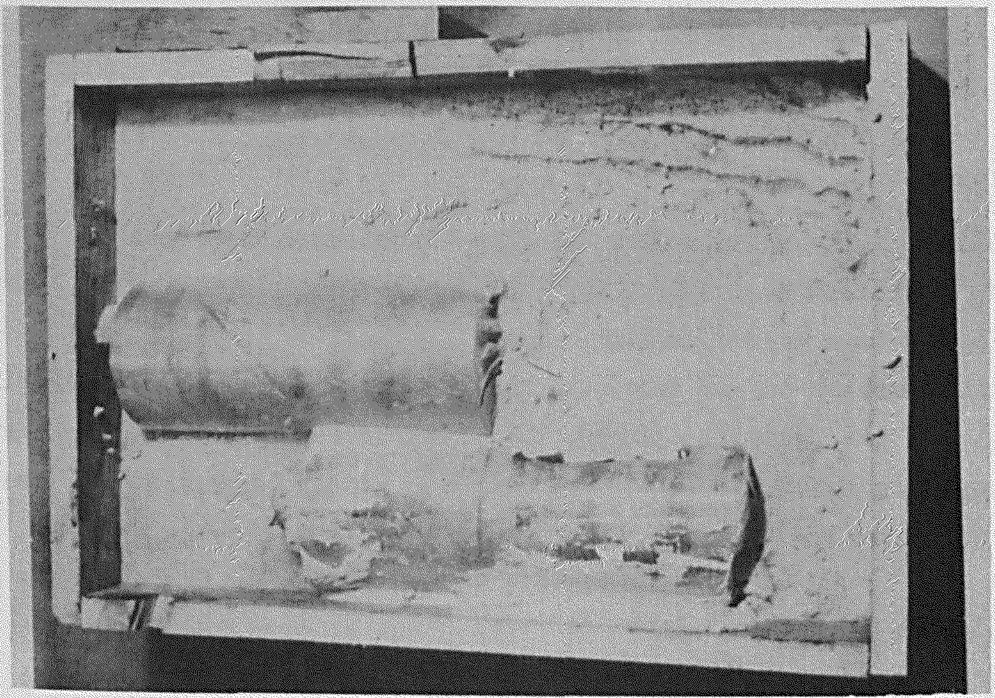
5.7 The multistage triaxial test was run in the same pressure chamber as the standard triaxial tests. The weights of the piston, swivels, and specimen end platens were accounted for in obtaining the axial load on the specimen. Seven stages were run, including confining (σ_3) pressures of 10, 35, 65, 105, 150, 200, and 300 psi. The sawed surfaces were oriented at an angle of 45 deg from the longitudinal axis of the core.

5.8 Figure 5.1a depicts the orientation of the cores and the method used to cut the cores to insure that the surfaces would reasonably match. The cores were aligned parallel to each other and located relative to each other such that the required portions of the concrete and rock would be obtained. They were then hydrostoned in a wooden box. Figure 5.1b shows the two cores after the 45-deg saw cut was made. When the specimens were removed from the hydrostone, the concrete and shale surfaces were checked for alignment and found to match quite well; when held to the light, you could only see through about 10 percent of the contact area.

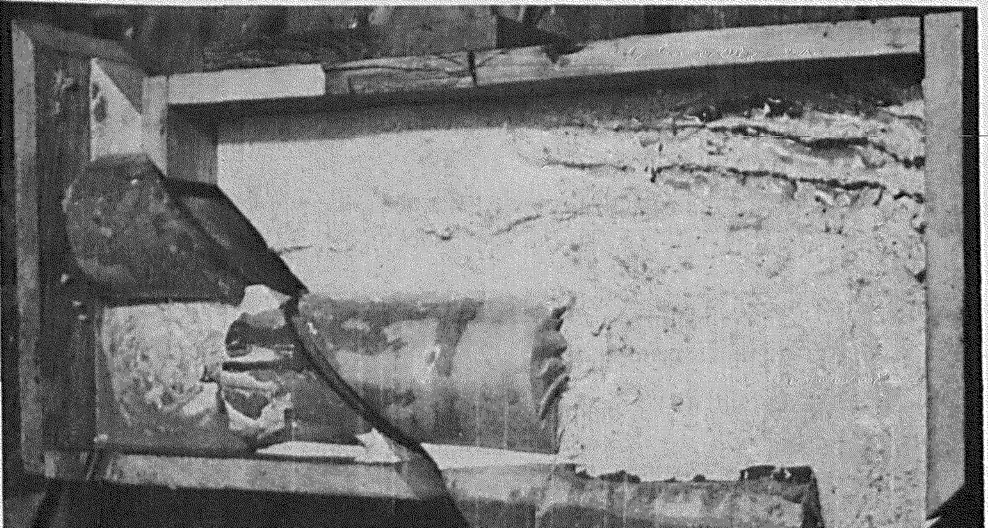
5.9 The concrete and rock core logs are given in Reference 2 along with a discussion of the petrographic analysis of the concrete and rock material.

Table 5.1: Material Parameters

	Foundation	Concrete	Backfill
Index Properties			
Drained Unit Weight, lb/ft^3			112.7
Sustained Unit Weight, lb/ft^3	158.5	146.8	
Submerged Unit Weight, lb/ft^3			132.8
Compressive Strength, psi	6270	4000	
Shear Strength, psi			
Intact	C = 945 Φ= 53° 15'		
Concrete on Rock	C = 0 Φ= 30° 30'		
Modulus of Elasticity x 10^6 psi	0.51	3.47	
Poisson's Ratio	0.12	0.16	
Shear Modulus x 10^6 psi	0.228	1.53	



a. Core Layout



b. Cut Cores Matched

Figure 5.1. Orientation of Cores for Cutting Parallel Surfaces

SECTION 6: STABILITY ANALYSIS

Introduction and Problem Statement

6.1 One main consideration in determining the structural adequacy of the locks and dam is the stability of the various monoliths when subjected to possible loading conditions. The stability study involved analyzing selected monoliths of the locks and dam to determine if they have adequate resistance against overturning, sliding, and base pressures. In this study, only one monolith of each typical configuration and loading was analyzed. The conclusions determined from these data are adequate for an evaluation of all monoliths.

6.2 In addition to the condition survey report,[2] a field survey and examination of Emsworth Locks and Dam were conducted. From the field survey and examination, no relative settlement or misalignment of monoliths were detected. Bench marks and alignment plugs have recently been installed on the locks; therefore, alignment and settlement can be monitored and any movement detected. The resurfacing on most of the monoliths has deteriorated and in many places it is already absent from the concrete surface. The concrete surfaces are badly deteriorated and will be a concern in areas of stress concentrations which will be discussed in Section 7.

6.3 The objective of the stability analysis is to determine whether or not the monoliths of this old structure meets the present-day criteria of desired safety against overturning, sliding, and excessive base pressures. The present-day criteria are set forth by the Corps of Engineers in their Engineering Manuals and Technical Letters. These are the standards which set forth the current state of the art for the design or analysis of Civil Works structures. Any advances in the state of the art which reflect needed changes in these criteria are a separate consideration and are to be used only by approval of the Chief of Engineers. Even with the criteria from the Engineering Manuals and Technical Letters, engineering judgment will have to be used in certain aspects of the analysis. In these considerations, it is important not to relax engineering concepts to include variables which are not dependable because, during infrequent but special conditions, they could cause failure.

6.4 The first information needed in order to start a stability analysis is the physical geometry of the various monoliths. This is needed in the actual analysis as well as in the selection of the monoliths to be analyzed. When analyzing an old lock, it is important to determine the as-built construction. In this case, no as-built plans were available; therefore, other means were used to determine construction variations from that originally planned. The construction photographs show that the original upper guide wall has some monoliths with 10 ft base widths; this could cause them to be susceptible to stability problems. This narrow base width is considered in the analysis of Monolith L-19. In other cases, it was not possible to establish any differences between the as-built and planned construction from the construction photographs. Borehole data were used to determine the concrete-rock interface. When this interface was significantly different from the planned construction, the depth of the monolith was made to correspond with the borehole data.

6.5 After ~~the monoliths for analysis are selected~~ and their geometry determined, possible loading conditions must then be determined. A summary of the loading and criteria used in the stability analyses are given below and a more detailed explanation is given in Appendix C.

6.6 The surface elevations of normal upper and lower pools are 710.0 and 692.0, respectively. The saturation levels used in the backfill are given in Table C.1. These are the saturation levels used as design standards by ORP. The unit weight of backfill material was 112.7 and 132.8 lb/ft^3 for the drained and submerged weights, respectively. The horizontal force exerted by the backfill material on the land wall monoliths was used as a coefficient times the vertical soil pressure at that depth. A lower bound "at-rest" coefficient of 0.5 was used. The location of the resultant soil pressure was considered to be 0.45H above the monolith base. This height was used because upward sloping backfill, railroad vibrations, and surcharge loading were located close to the back of the land wall monoliths.

6.7 The unit weight of concrete was used as 146.8 lb/ft^3 which was an average of many measurements obtained from cores. The boat impact was:

- a. Lock chamber wall: 800 lb/ft but not less than 40,000 lb per monolith.
- b. Other walls: 2,500 lb/ft but not less than 120,000 lb per monolith.

6.8 The hawser pull was considered as 24,000 lb distributed over a monolith 30 ft in length. The boat impact and hawser pull are considered as acting 5 ft above the waterline and are combined with the most severe normal loading conditions.

6.9 An allowable base pressure of 20 k/ft^2 was used. A wind loading of 30 lb/ft^2 was used when applicable. For sliding the cohesion value (c) was 0 and the angle of sliding friction (ϕ) between the concrete and foundation was 30.5°.

6.10 Resistance to overturning was considered adequate if the resultant fell outside the kern but within the middle half of the base for normal operation cases using "at-rest" earth pressure coefficients. The resultant for the extreme maintenance condition using at-rest earth pressure was considered adequate if it fell outside the kern but within the middle half of the base.

6.11 The criteria for determining resistance to sliding are given in ETL 1110-2-184 and the safety factors are listed in ETL 1110-2-22.

Results

6.12 A summary of the stability analyses is given in Table C.1. A discussion of the stability of the individual monoliths is given below. Since the inadequacy of the monoliths is the factor which is significant, the monoliths which are inadequate in stability are the only ones discussed. The monolith numbering and stationing is presented in Figure 6.1.

- a. Monolith L-19 is inadequate for overturning, sliding, and base pressures. Under normal design considerations, monoliths in the original guide wall do not meet present-day criteria.

b. Under normal operation, Monolith L-34 is inadequate for resistance to overturning, sliding, and base pressures and is very inadequate in a maintenance condition for overturning, sliding, and base pressures.

c. Monolith L-37 is inadequate for overturning and sliding.

d. Monolith L-52 is inadequate for overturning, sliding, and base pressures even during normal operation and is very inadequate in the maintenance case.

e. The stability of Monolith L-56 can logically be reasoned as sufficient from the stability computations of the other monoliths and from the stabilizing effect of the wall connected to it from Lowery Run.

f. Monolith M-8 is inadequate for overturning in the normal operating case.

g. In the middle wall, Monolith M-22 is inadequate in both the normal operation and maintenance condition for overturning, sliding, and base pressures. When the river chamber was dewatered in 1968, a 1-1/2 in. wide separation was discovered at the intersection of the ceiling of the emptying culverts and the middle wall monoliths, of which M-22 is typical. This separation was parallel to the middle wall and continuous across all five emptying barrels. Temperature effects were considered and for high values of coefficients of linear expansion a contraction of 1-1/2 in. is unreasonable. Table C.1 shows that the safety factors against sliding for these middle wall monoliths are 1.4 and 0.92 for normal operation and dewatering the 110-ft lock chamber, respectively. Therefore, some movement of these monoliths could have taken place toward the land during unusual impact loading, ice loadings, or when the 110-ft lock chamber was dewatered. It is conceivable that movement could have occurred at least until sufficient resistance was developed between the 110-ft lock floor slab, other supporting projections, or friction between adjacent monoliths.

h. Monolith M-25 is inadequate for overturning in the normal and maintenance cases. It is somewhat inadequate for sliding in both cases.

i. R-4 is inadequate for overturning and is somewhat inadequate for sliding.

j. R-14 is inadequate for sliding in the maintenance case.

k. R-17 is inadequate for overturning in the normal operation case.

l. R-24 is inadequate for overturning and sliding in the normal operation case.

m. R-27 is inadequate for overturning and sliding during normal operation.

6.13 An excellent analysis of typical and end piers is given in Reference 1 which shows they are stable; these results are not reproduced in this report.

6.14 In general, the monoliths on the land wall do not meet present day criteria for overturning, sliding, or base pressures. Also some monoliths in the middle and river walls do not meet the present day stability requirements.

6.15 There are two acceptable approaches to this situation when considering only the stability of monoliths. One approach is to say the monoliths do not meet the criteria and examine the feasibility of modifying the construction or replacing the locks and dam. The other approach is to give consideration to the length of time the monoliths have been in service without excessive relative settlement or misalignment, and to schedule periodic inspections of the locks and dam so that any potential trouble can be detected and corrective action taken. The periodic inspection has merit because minimum maintenance can be performed to protect the monoliths from weathering, and decisions of replacement made when conditions warrant such action. The minimum maintenance and inspection are valid considerations because the feasibility of complete rehabilitation or replacement will probably lead to a replacement of the structure. Rehabilitation or replace should be considered, taking into account the total condition (operational and structural) of the locks and dam.

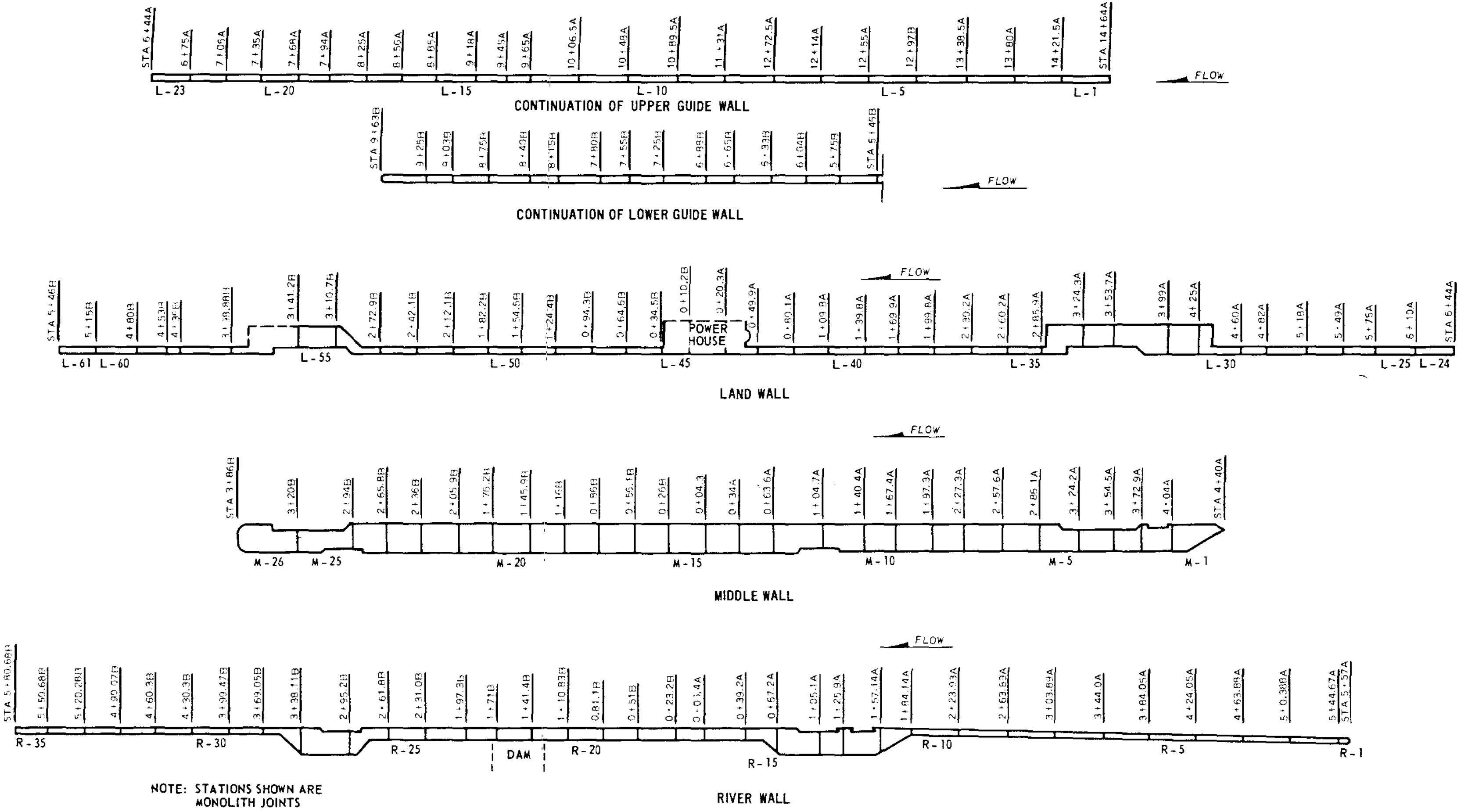


Figure 6.1 Top views of guide, land, middle, and river walls showing monolith numbering and stationing, Emsworth Locks and Dam, Ohio River

SECTIONS 7: STRESS ANALYSIS

Introduction and Problem Statement

7.1 In the structural evaluation of Emsworth Locks, a two-dimensional plane strain finite-element analysis was used to determine stresses within selected structural monoliths.

7.2 It is becoming increasingly important to understand the phenomenon of stress distribution in structures and not depend entirely on average stress approximations as has been done in conventional design. Knowledge of the total stress field is important in order that stress concentrations and decisions for concrete reinforcement can be handled wisely and economically. This is especially important when considering that raw materials are being depleted and should be used wisely and not at a rate in excess of that which is absolutely necessary. Conventional analysis usually leads to a safe but overly conservative design because the whole stress field is not known and observations of stress concentrations cannot be delineated, studied, and adequately reinforced. The finite-element analysis adds a new dimension or advantage in this respect. Finite-element calculations do not make conventional design obsolete; in fact, it is a supplement, making a combination which is much better than either separately. It is important to consider stress distribution in areas of stress concentration when evaluating old structures which have cracked and are deteriorating.

Finite Element Analysis

7.3 the finite-element analysis is used to get some idea of the magnitude of compressive- and tensile-stress concentrations within the monoliths under normal operation and maintenance conditions. The finite-element solution gives good results as long as the model adequately represents the actual situation and as long as any assumptions made can logically be seen or proven to be adequate. In the following analysis, elements were made continuous under the monolith which allowed tension between the base and foundation which is, of course, unrealistic. The tension effect dissipates rapidly but will decrease the compressive stresses at the base-foundation interface on the opposite side of the monolith. This effect can be eliminated but the time and funding required to do this trial and error solution are beyond the scope of this project.

7.4 The loads applied on the two-dimensional sections are presented in Figures 7.2, 7.9, 7.15 and 7.19. In a two-dimensional analysis of the monoliths, such factors as changes in geometry and loading along the monoliths lengths can only be approximated. Localized loading (gate anchorages, impact, hawser, etc.) will not give realistic stresses if applied on a one-foot length of monolith. In order to obtain more realistic stress values, concentrated loadings were considered by using a per foot load obtained from distributing the total load over a length or a portion of the length as given by a 45° distribution. The 45° distribution originates at the point of load concentration and extends in the direction of loading until its sides intersect the outer edge of the monolith. This can be done in both the horizontal and vertical planes with the shortest distance between intersections being the more critical. The distance between intersections in the more critical plane was used as the length over which to distribute the concentrated compressive loads and one-half this type distribution was used for concentrated tensile loads. The maximum compressive values were at the intersection of the base and foundation and a 45° distribution will give as reasonable a spread of the load to the foundation as any assumed distribution.

7.5 The maximum tensile stresses are around culverts at changes in geometry, at hawser locations, and at anchorages. The maximum is rather localized at the point of application and will only be relieved by deformation tending to spread the load over the section of concrete which is being separated from the monolith by tension. A 45° distribution of this tensile load will given concentrated stresses which are too low; therefore, an approximation of one-half the 45° distribution was used in the analysis.

7.6 An important concept is that changes in geometry and loading along the monolith length make the problem a three-dimensional analysis and approximations have to be made in a two-dimensional analysis. In the following work, the two-dimensional analysis is used to obtain some idea of maximum stresses in the monoliths. Three-dimensional analysis should be used in a detailed evaluation of stress distribution which is not the objective of this study.

7.7 Average elevations of soil behind the monoliths were considered as was done in the stability analysis. In making stress and displacement calculations, the backfill was not used as part of the grided medium. There were two reasons for this:

- a. Many elaborate tests of backfill material would be required to define the backfill properties precisely. This was not done because the vertical and horizontal backfill loads, which are obtained by using unit weights and coefficient of at-rest-earth pressure, are within the accuracy of the study.
- b. The finite-element grid would become very large.

The density of the backfill material was used to get vertical loads. The coefficient of at-rest-earth pressure and the density of the backfill material were used to obtain the horizontal loads. The water pressure from saturation level was taken into account. The loads are then applied at node points of the finite-element grid.

7.8 One consideration which must be made in the stress analysis is the effect of uplift on the base of the monolith. In certain cases the effect will be negligible, but in others it could be substantial; therefore, the effect must be included. The important concept concerning uplift is that it is a support condition, and its effect (small or large) is dependent on its distribution. Specific loadings on a structure cause a specific distribution of pressure under the monolith base. The uplift will change this distribution, thereby affecting the support condition of the monolith. It can be seen that the pressure distribution under the monolith affects the stress in the structure only by deformations resulting from the support condition. By looking at free body force diagrams of a monolith, in fact, a section an infinitesimal distance above the base (in rigid body analysis), can be taken and the upper part of the monolith considered as a free body. The analysis will then not even see the pressure distribution at the base; therefore, the distribution affects the stress analysis through deformations which are taken into account in the finite-element study. Uplift could have significant effect where there are large culverts close to the base of the monolith and the distribution is such as to load the slab to increase stresses. A reasonable way to handle the uplift is to put a silt of foundation material below the structure of thickness such that the deflection of the monolith at the base is less than the slit thickness in order that problems in code solutions, such as negative element areas, will not be encountered.

7.9 The stresses given in the finite-element computations follow the nomenclature given in Figure 7.1 below.

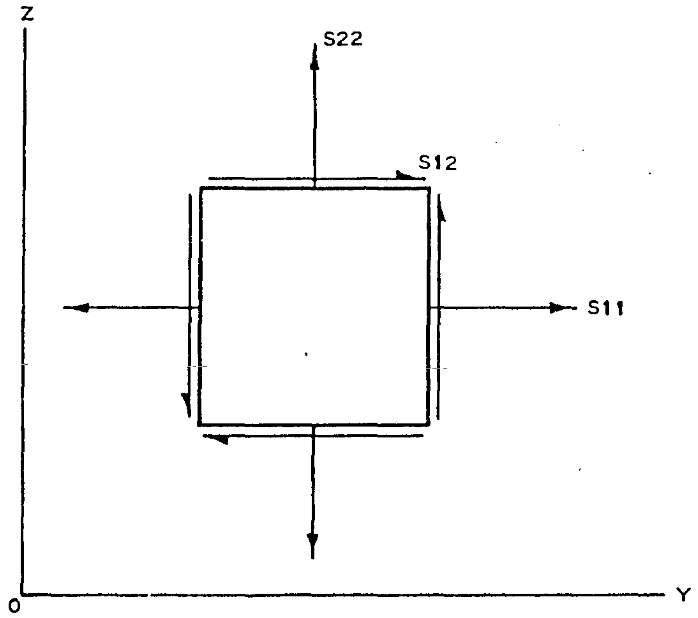


Figure 7.1. Stress Nomenclature

7.10 The stress distributions in this section of the report show the major and minor principal stresses in their respective directions at the centroid of the element. The arrows denote tension if directed toward the centroid, and compression if directed away from it. The values printed on the stress plot are the element number, minimum principal stress, and maximum principal stress. A positive sign indicates tension and a negative sign indicates compression.

7.11 Monoliths within the lock which will have maximum tensile and compressive stresses were selected for analysis.

7.12 Areas of maximum tensile stress in monolith L-56 are around the culverts and at gate anchorages. The loadings for L-56 are given in Figure 7.2. The total stress distribution for the normal operating and the dewatered cases is given in Figures 7.3 and 7.4, respectively. The maximum tensile stress is at the top of the culverts for the dewatered case and at the sides of the culvert during normal operation. The monolith sections and depicted areas of stress concentration are given in Figure 7.5. The tensile stress at the top and bottom of the culverts is in the range of 100 psi for the dewatered case and approximately 200 psi at the sides of the culverts for normal operation. These stress plots are given in Figure 7.6 and 7.7 for the normal operating and dewatered cases, respectively. At gate anchorages (Figure 7.8) the maximum tensile stresses occur in the normal operating cases and are approximately 360 psi. The compressive stress is largest at the toe of the monolith but is about one-tenth of the maximum compressive concrete strength. This is within the allowable for the concrete but is above the 20 ksf for the foundation.

7.13 The normal and dewatered conditions are both considered for M-21. The loadings and total stress distributions are given in Figures 7.9, 7.10, and 7.11. The areas depicting stress concentrations are given in Figure 7.12. The stresses in these areas are given in Figures 7.13 and 7.14, respectively, for the normal operating and dewatered cases. The maximum tensile stresses around the conduits are approximately 300 and 220 psi, respectively, for the normal operating and the dewatered cases.

7.14 The maximum compressive stress is ≈510 psi and 450 psi for normal operating and dewatered cases, respectively. These stresses occur at the landside of the monolith base and foundation intersection. The compressive stresses will be larger than this because of the nonrealistic tensile stress, on the opposite corner.

7.15 Stresses for monoliths M-25 are shown for only the normal operating case; the dewatered condition stresses are approximately 10 percent higher. The monolith loading and the total stress distribution are given in Figures 7.15 and 7.16 respectively. The monolith shape and depicted areas of stress concentration are given in Figure 7.17. The maximum compressive stress is approximately 450 psi as given in Figure 7.18. These compressive stresses at the base of the monolith will be larger because the tension between the structure and foundation in reality does not exist. If this tension was eliminated the monolith would rotate onto the toe increasing the compressive stresses.

7.16 In monolith R-27, the stresses for the normal operation case are approximately equal to those for the dewatered case; therefore, the stress is given for only the normal operation case. The loadings and total stress distribution are given in Figures 7.19 and 7.20, respectively. The area of stress concentration is depicted in Figure 7.21. Figure 7.22 gives the stresses which occur in the depicted area. The maximum compressive stress is approximately 450 psi and the compressive stresses at the base of the monolith will be larger as previously explained in the discussion of M-21 and M-25.

7.17 The stress analysis only gives an indication of what can be expected as maximum tension and compressive stresses. Cracking can be expected in the areas where tension exists. The tensile stresses determined by this analysis can cause problems in deteriorated concrete because the magnitude of these stresses are excessive for nondeteriorated, nonreinforced concrete.

7.18 The analysis shows stress concentrations at the base-foundation interface which results from differences in properties of the foundation and concrete. The concentrations are of importance only where the overturning criteria is inadequate causing large stresses on a reduced base area. In the evaluation of base-foundation interface stresses those from the stability analysis should be the ones which are considered applicable. The comparison of percent effective base between stability and stress computations is invalid because tension is allowed at the base-foundation interface in the stress calculations. The foundation stresses are very large for many of the monoliths as can be seen from the results of typical stability computations (Table C.1). This is a negative factor when considering the adequacy of the lock monoliths at Emsworth.

7.19 Much of the concrete cracking is at changes of geometry. The stress analysis substantuates this as an area of stress concentrations even in these massive lock monoliths.

7.20 As previously stated, the main concern for concrete integrity is the cracked, spalded and deteriorated surface concrete. This will allow concrete deterioration at an accelerated rate reducing the section of the concrete monoliths resulting in an increase in tensile stresses which are already too high.

7.21 In general if corrective measures are not taken this will surely cause maintenance expense and will also reduce the life of the concrete monoliths.

7.22 In general the above concerns are real and should be considered in a feasibility analysis of structure rehabititation or replacement.

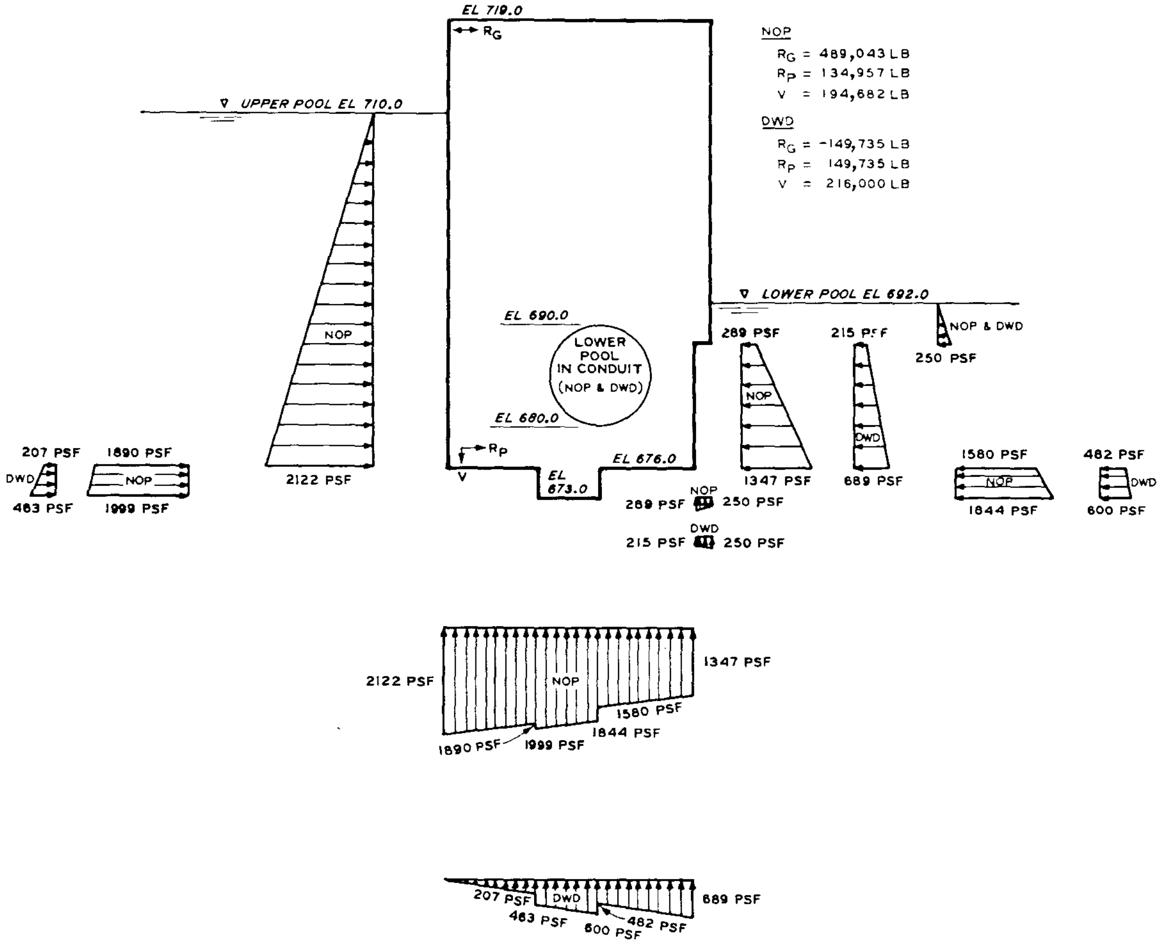


Figure 7.2 Loading-Monolith L-56

L56, NOP

EMSWORTH

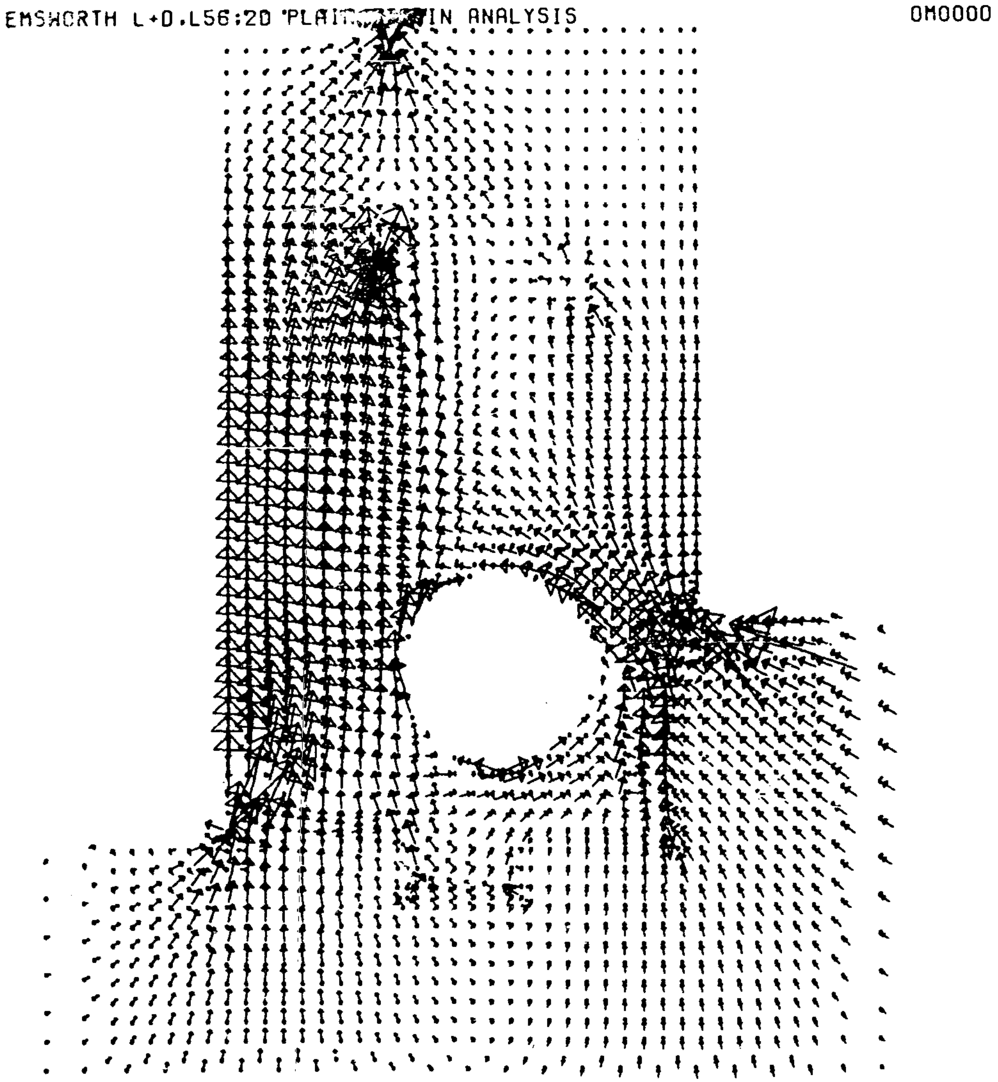


Figure 7.3 Total stress distribution, L-56, normal operating case

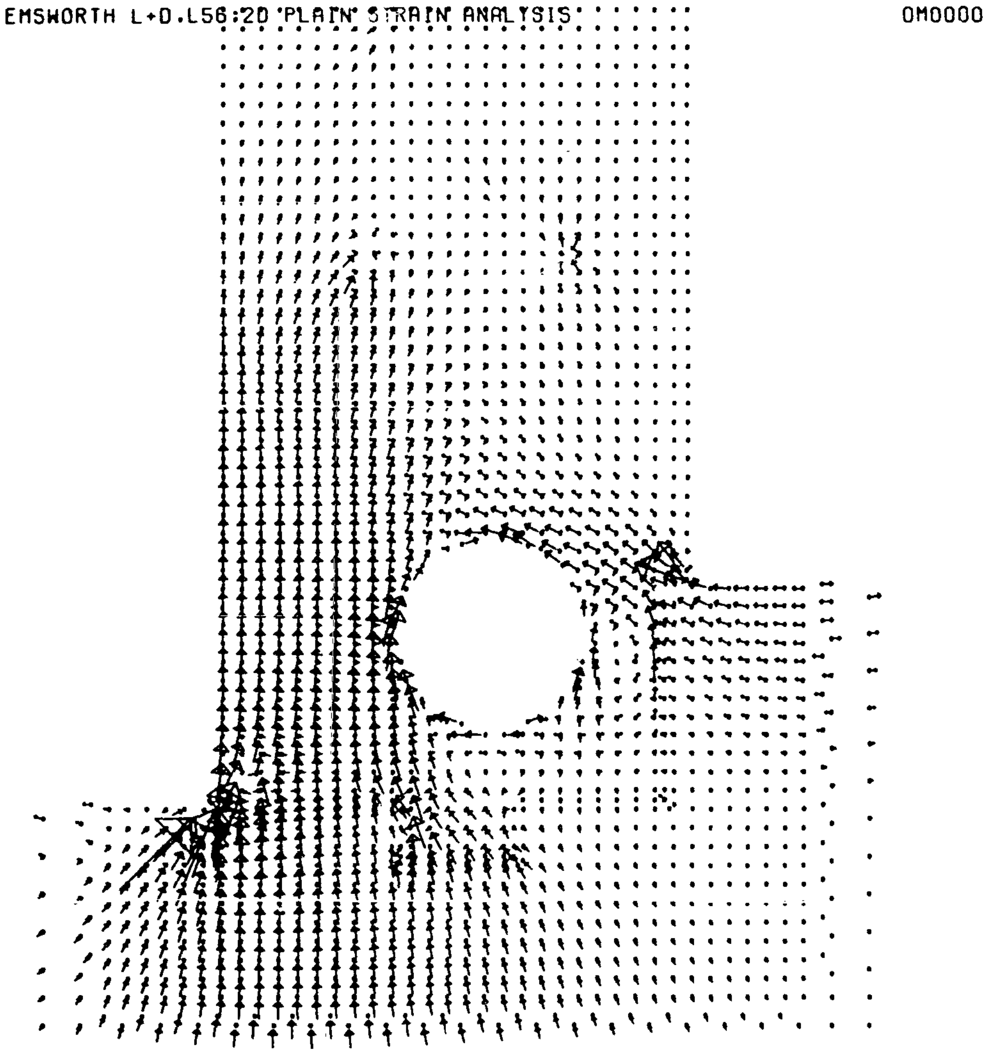


Figure 7.4 Total stress distribution, L-56, dewatered case

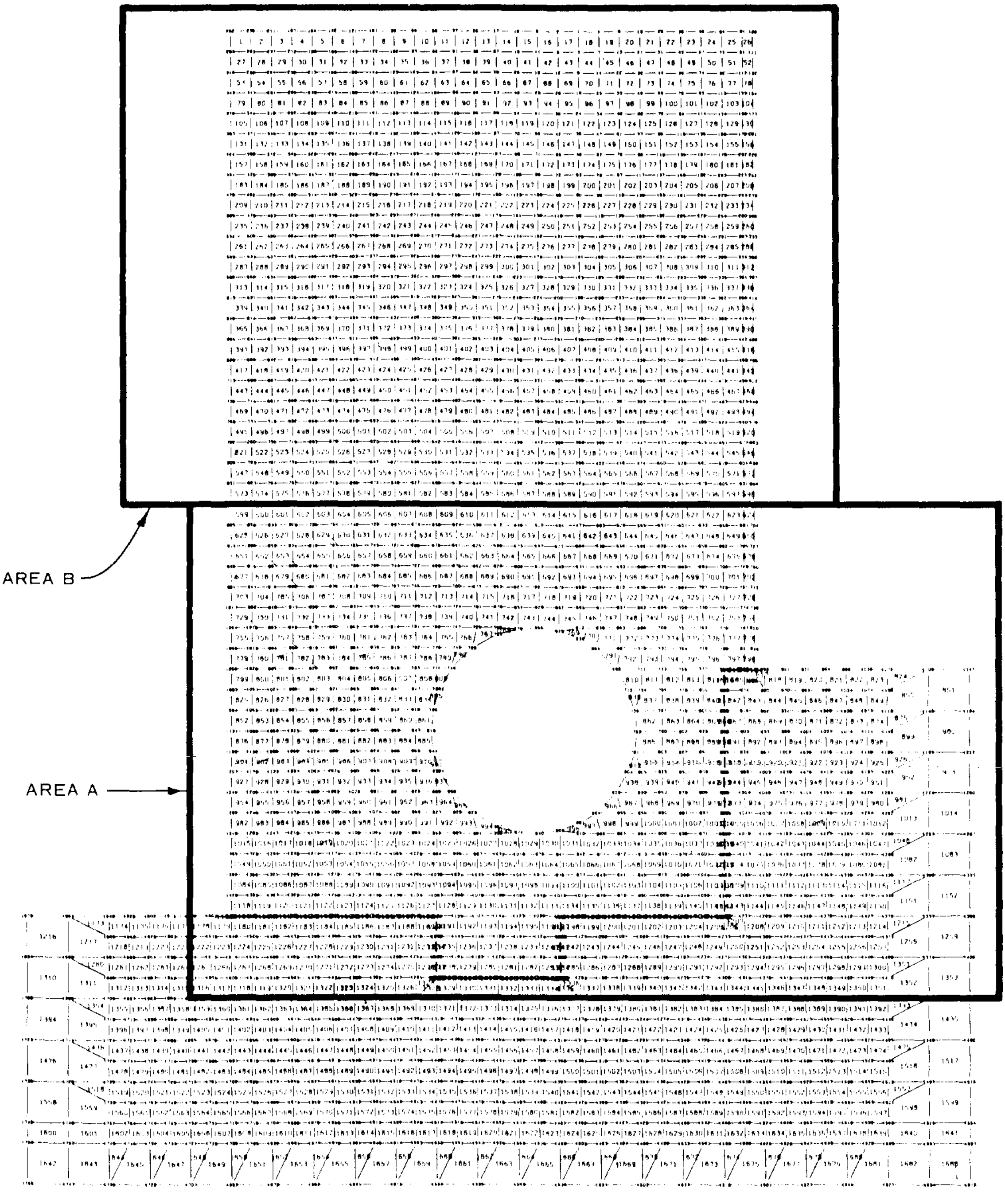


Figure 7.5 Landwall monolith, L-56, depicting stress concentration area for presentation

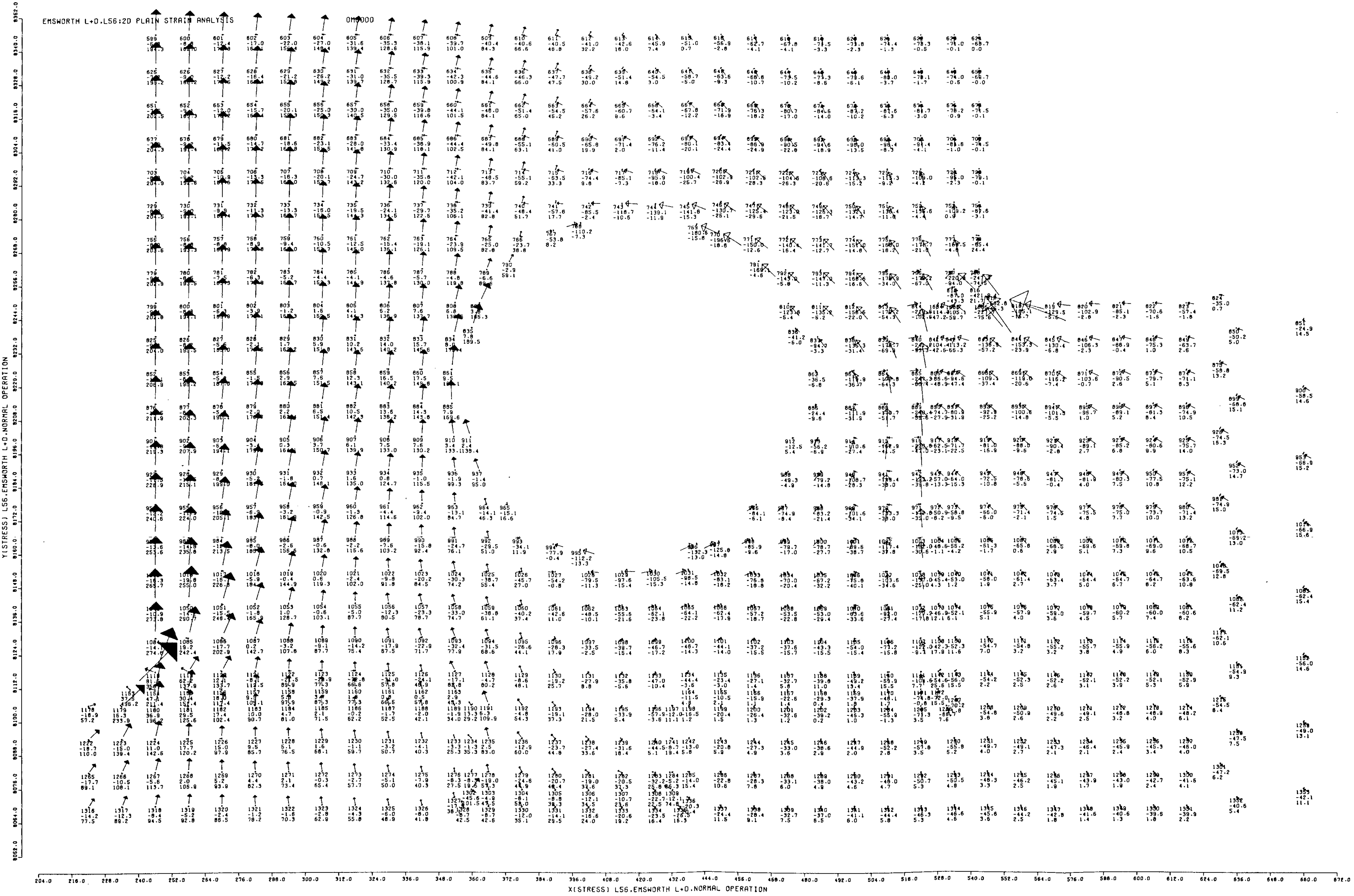


Figure 7.6 Monolith L-56, area of stress concentration as depicted by Area "A," Figure 7.5. Normal operating case

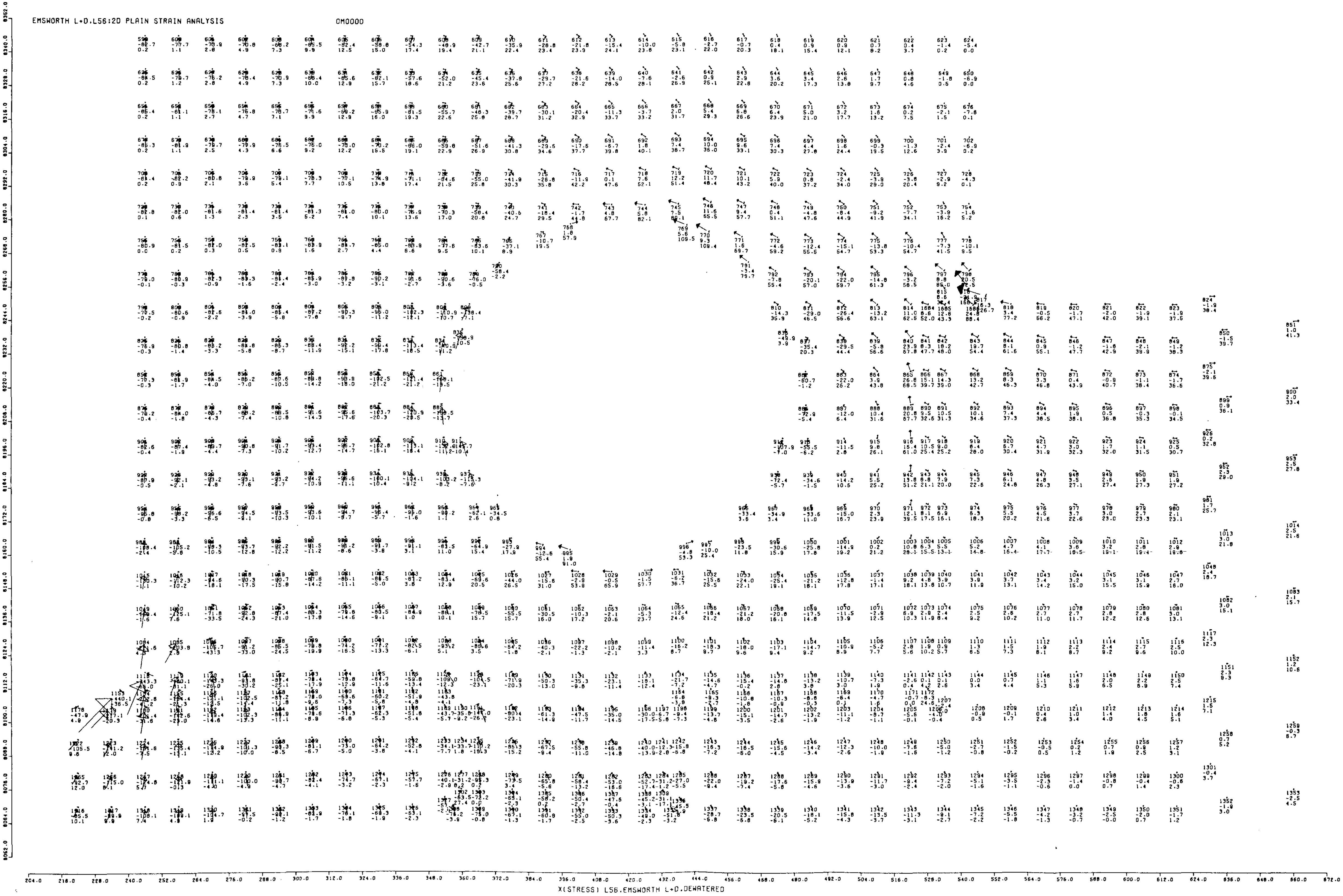


Figure 7.7 Monolith L-56, area of stress concentration as depicted by Area "A," Figure 7.5. Dewatered case

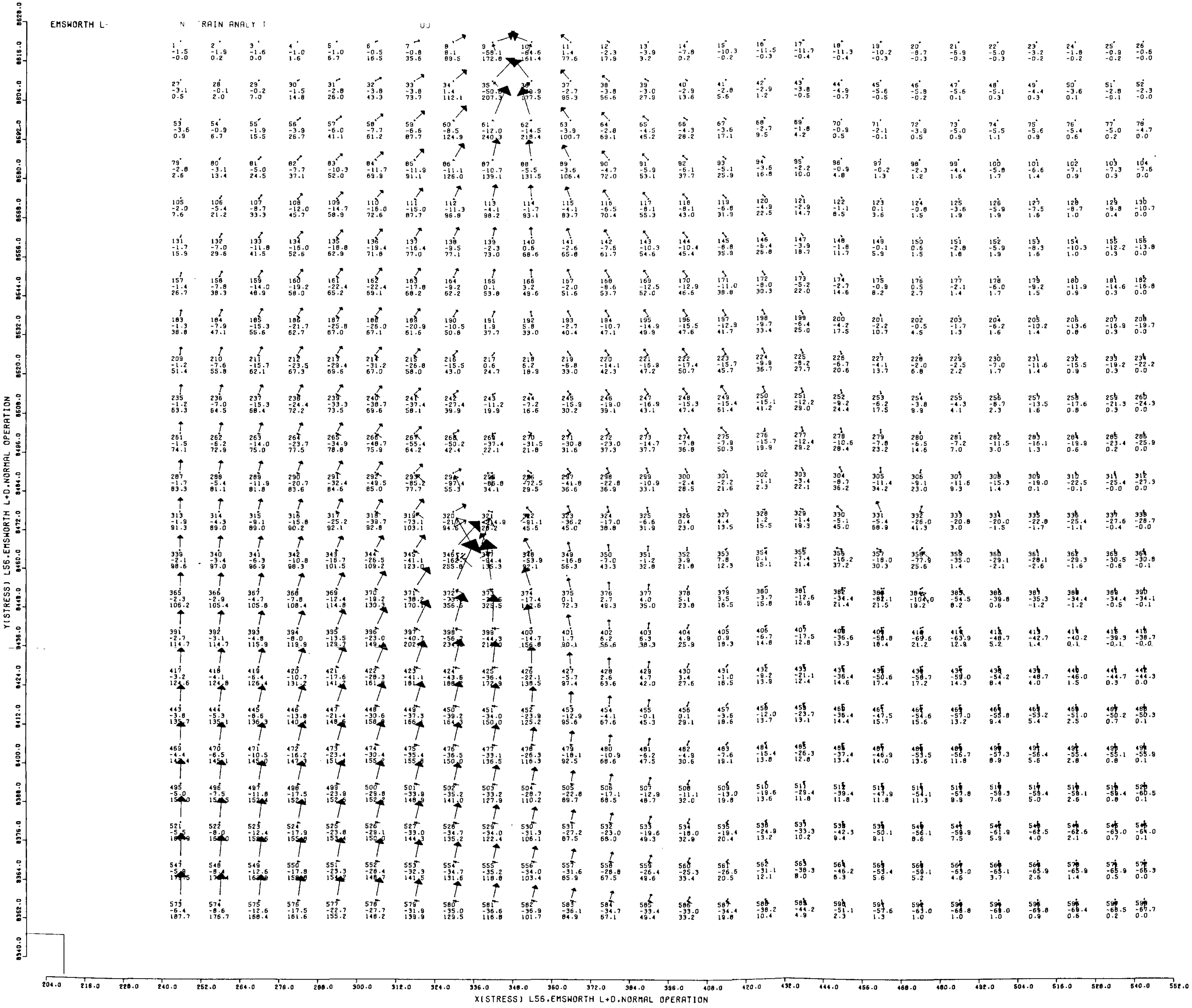


Figure 7.8 Monolith L-56, stress concentrations at gate anchorages as depicted by "Area B," Figure 7.5. Normal operation case

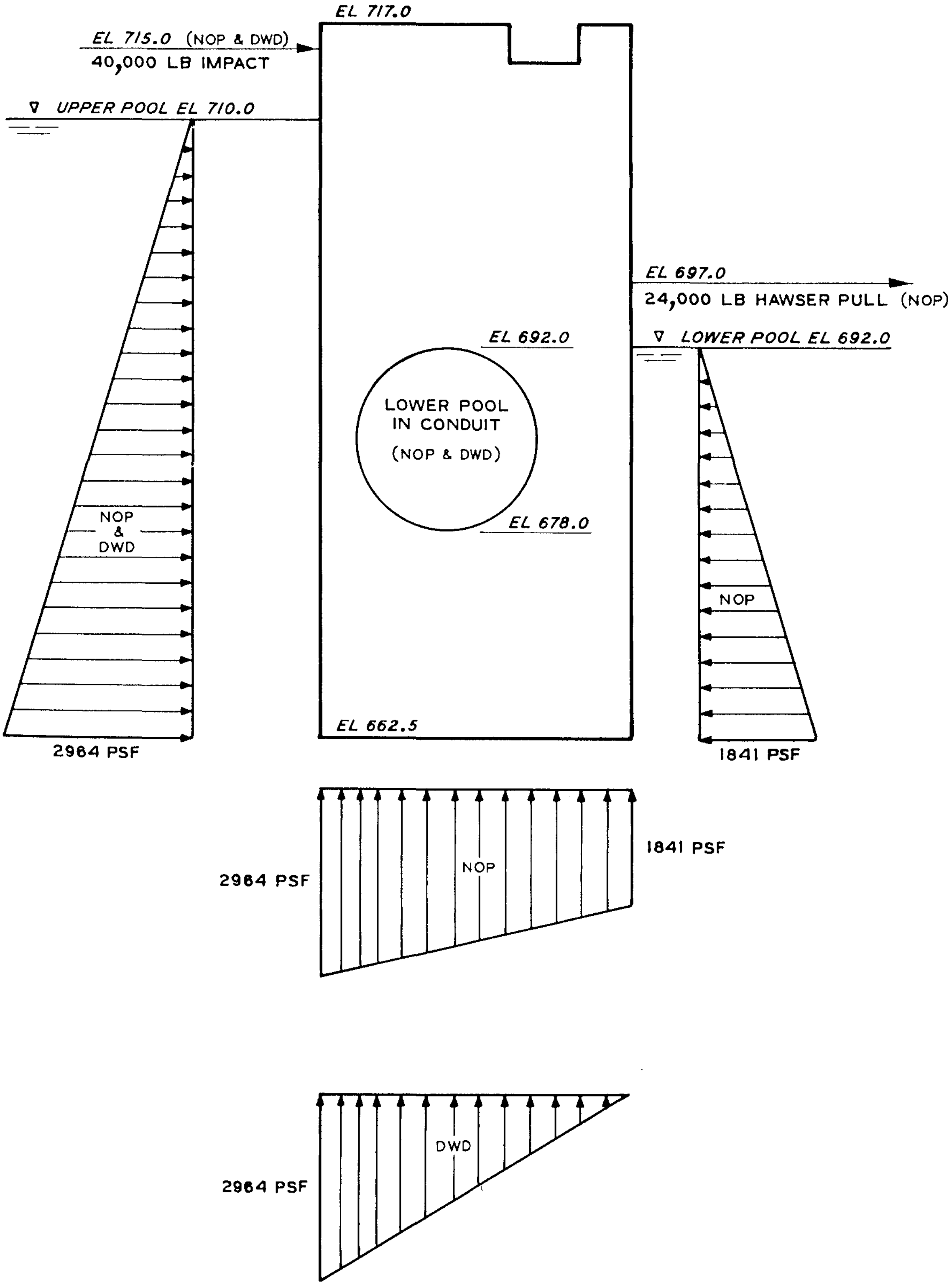


Figure 7.9 Loading-Monolith M-21

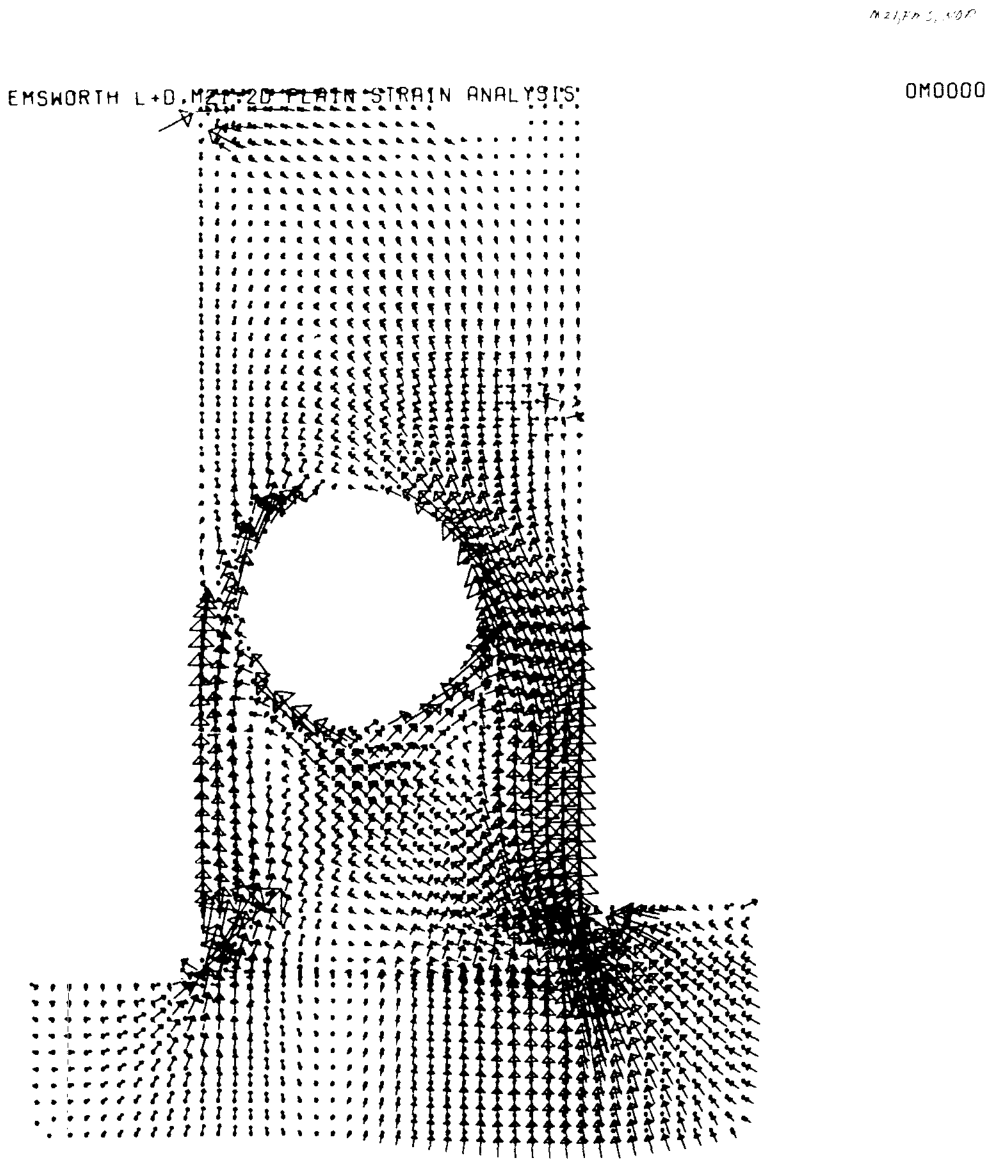


Figure 7.10 Total stress distribution, M-21, normal operating case

M21 FIGS, DWD

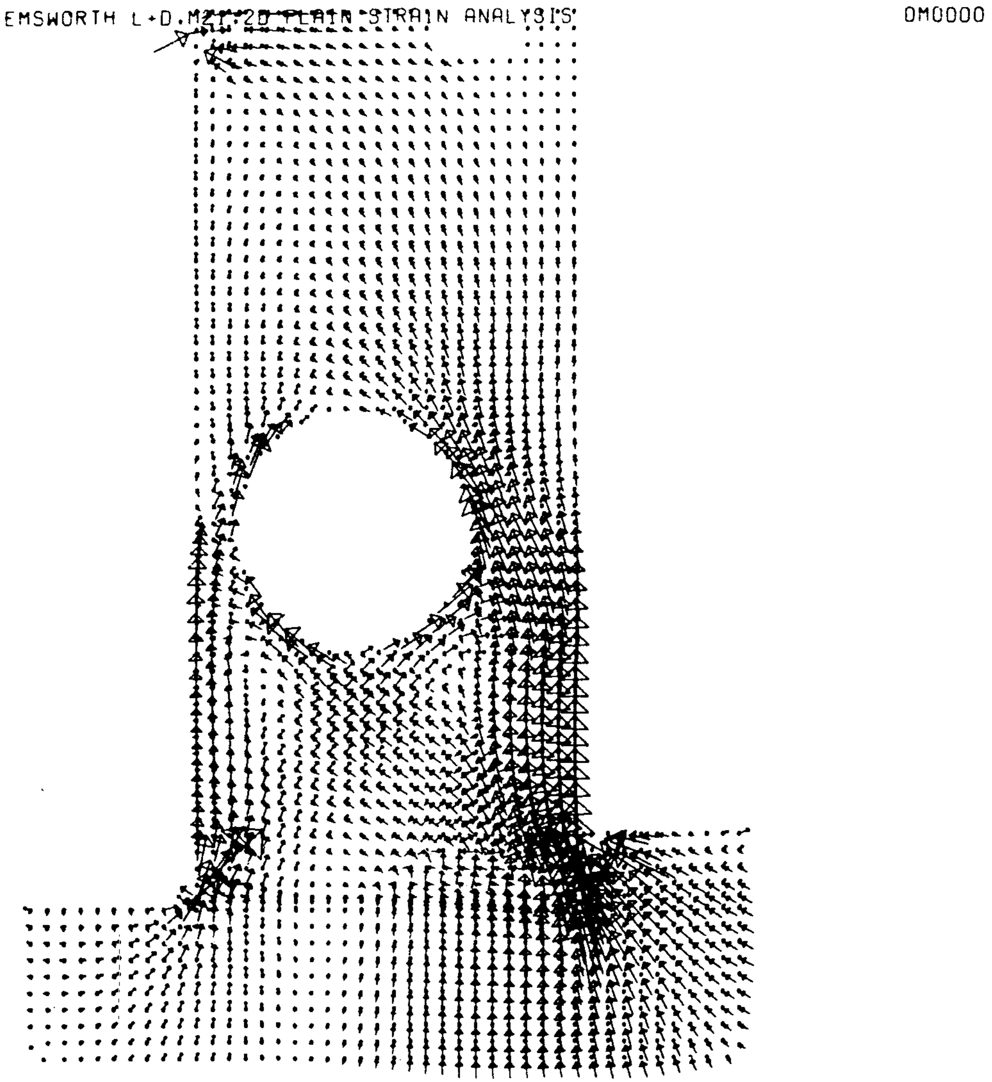


Figure 7.11 Total stress distribution, M-21, dewatered case

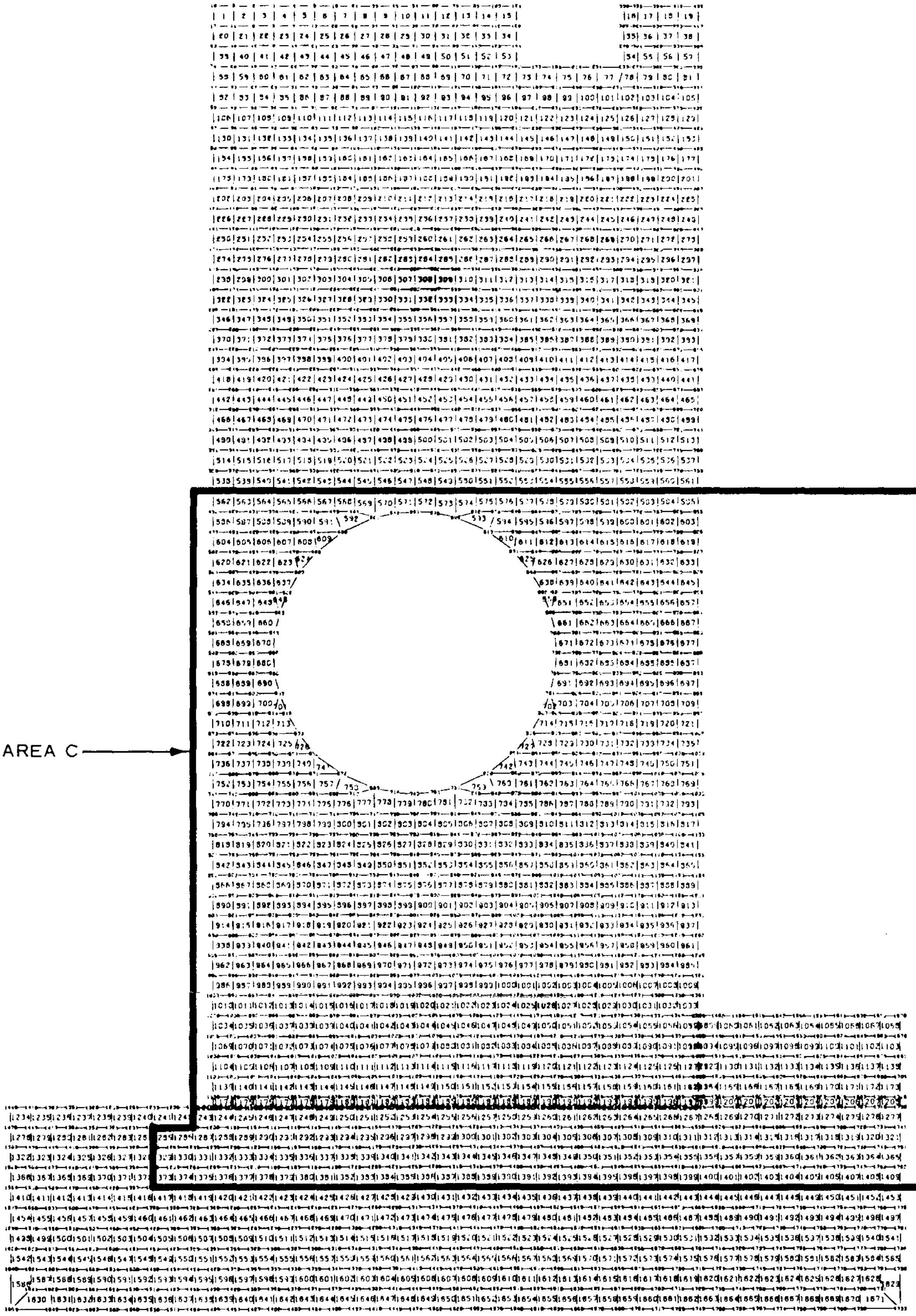


Figure 7.12 Middle wall monolith, M-21, depicting stress concentration area for presentation

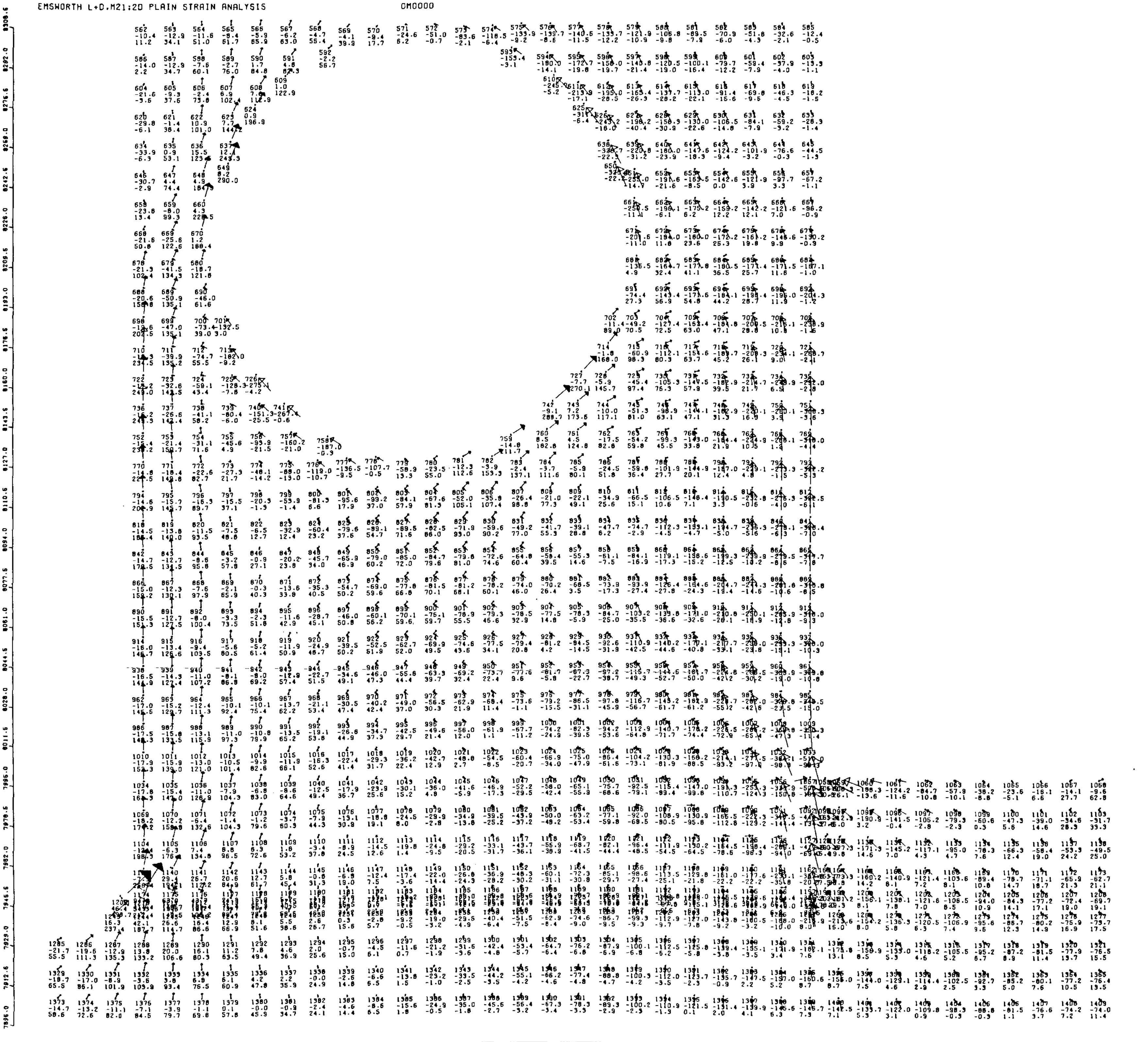


Figure 7.13 Monolith M-21, area of stress concentration as depicted by Area "C," Figure 7.12. Normal operating case

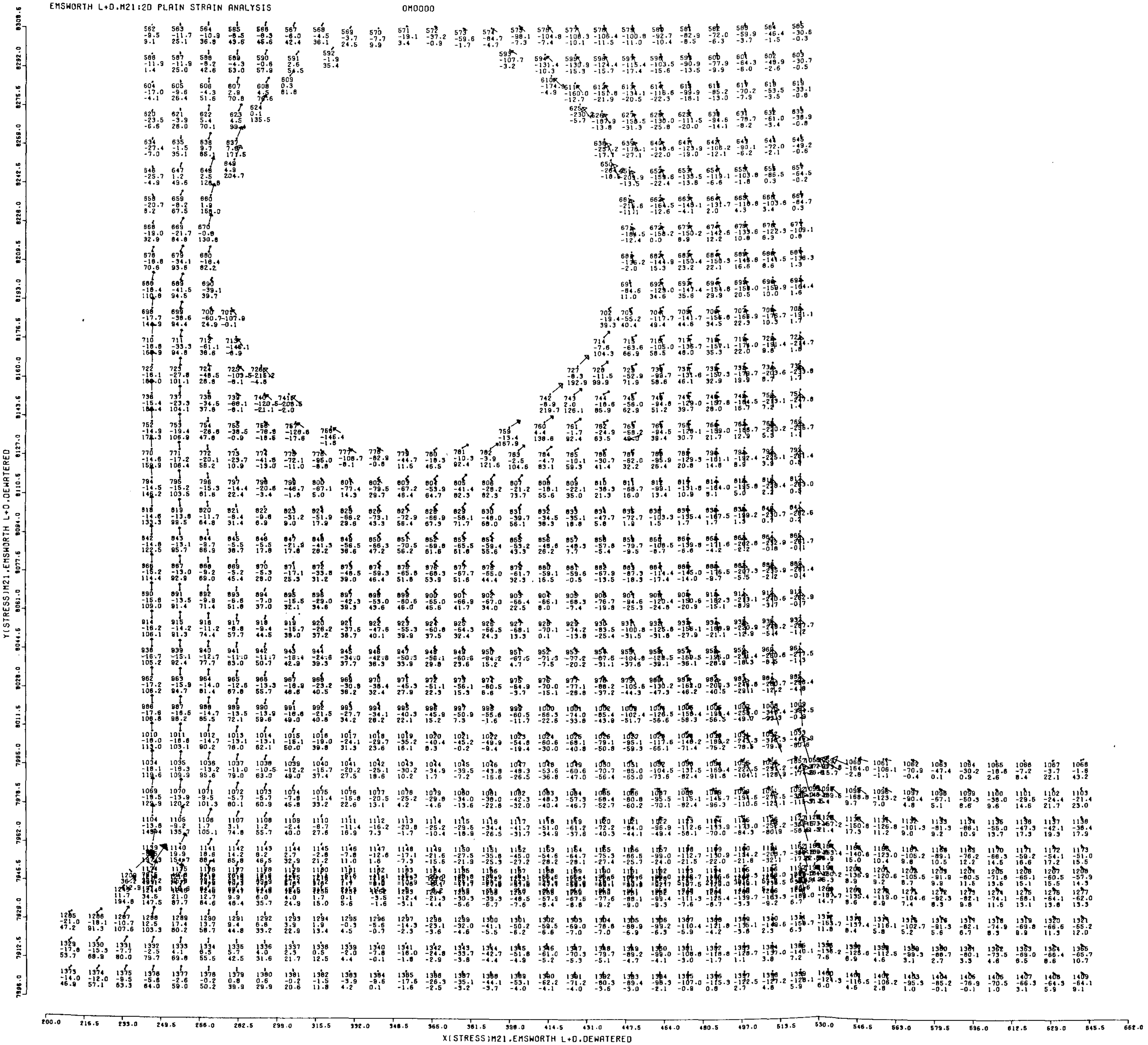


Figure 7.14 Monolith M-21, area of stress concentration as depicted by Area "C," Figure 7.12. Dewatered case

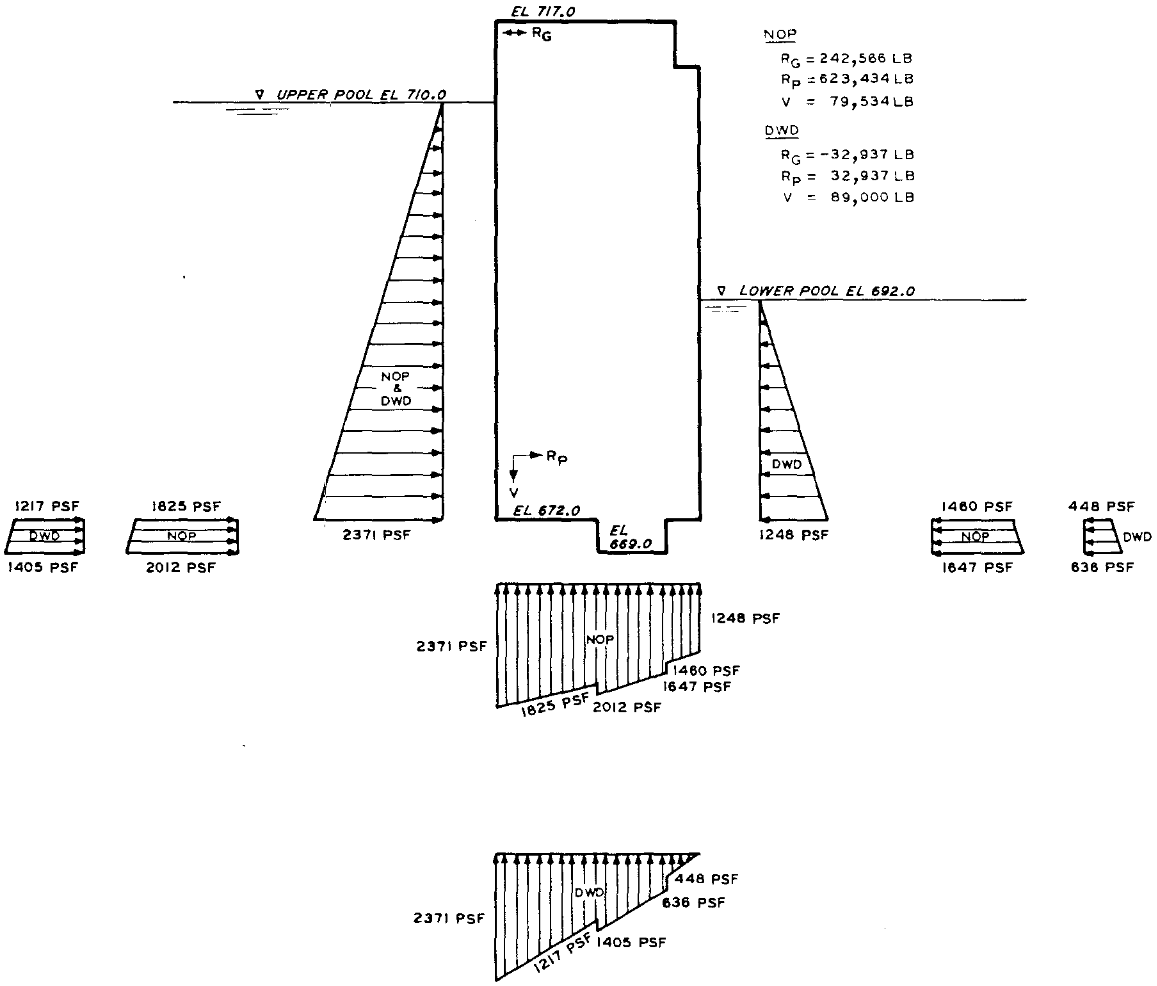


Figure 7.15 Loading, Monolith M-25

M25, EMS., NOP

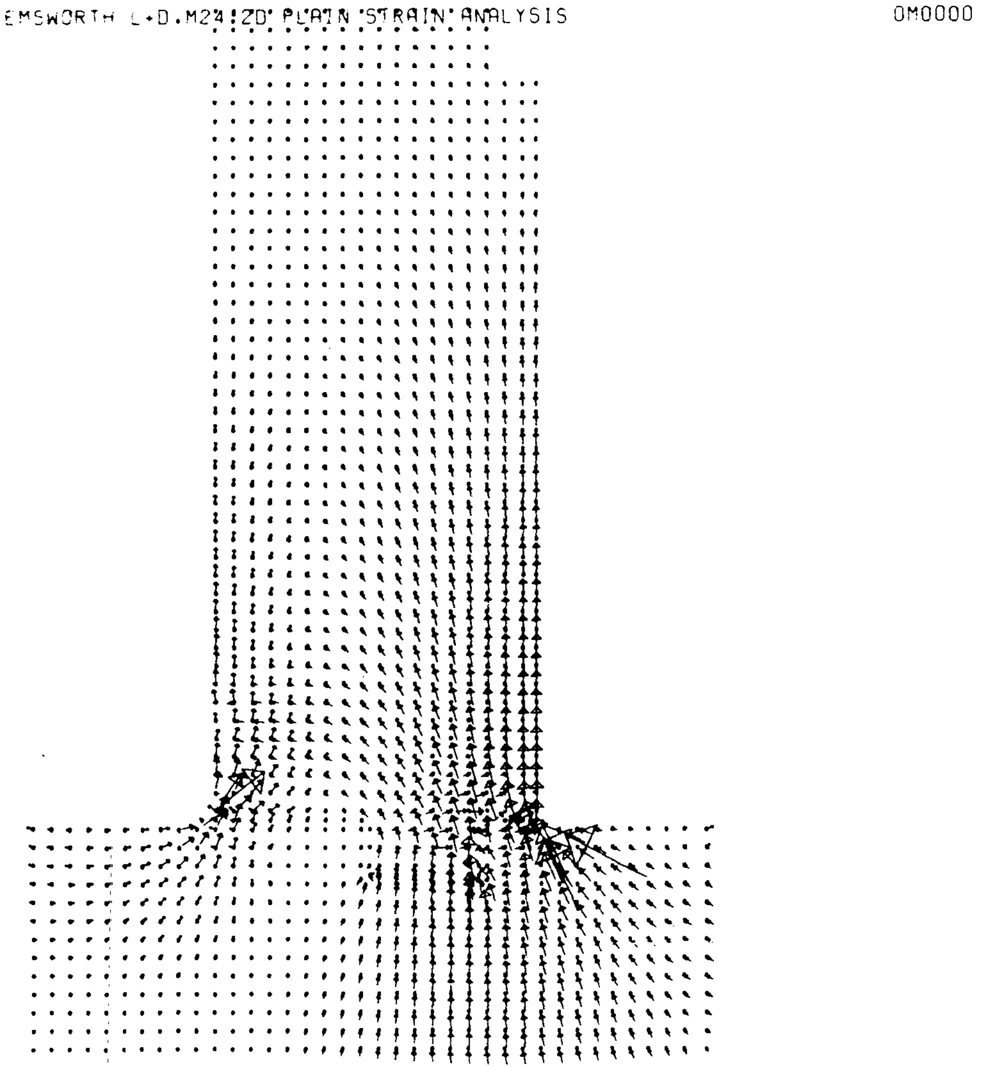


Figure 7.16 Total stress distribution, M-25, normal operating case

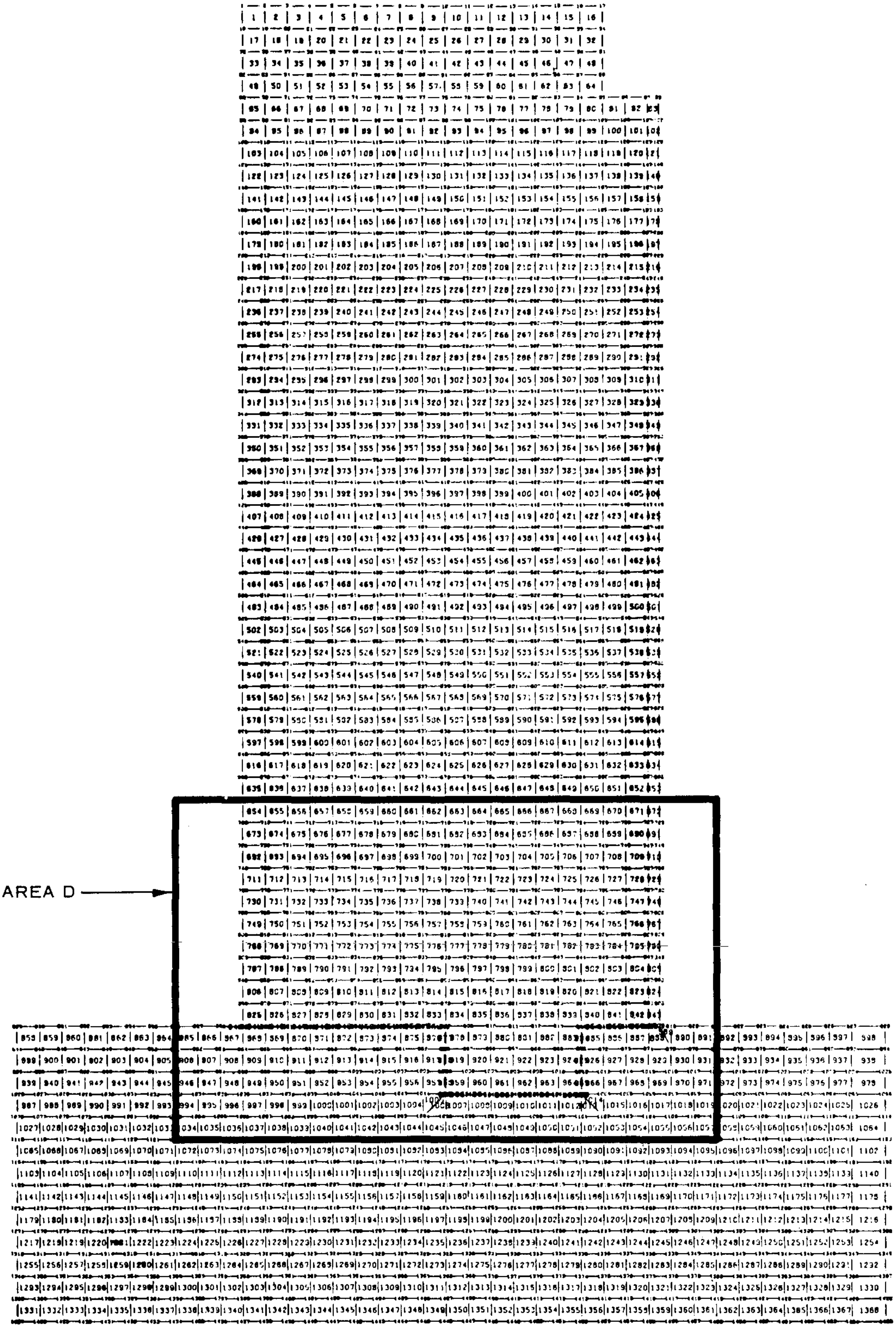


Figure 7.17 Middle wall monolith, M-25, depicting stress concentration area for presentation

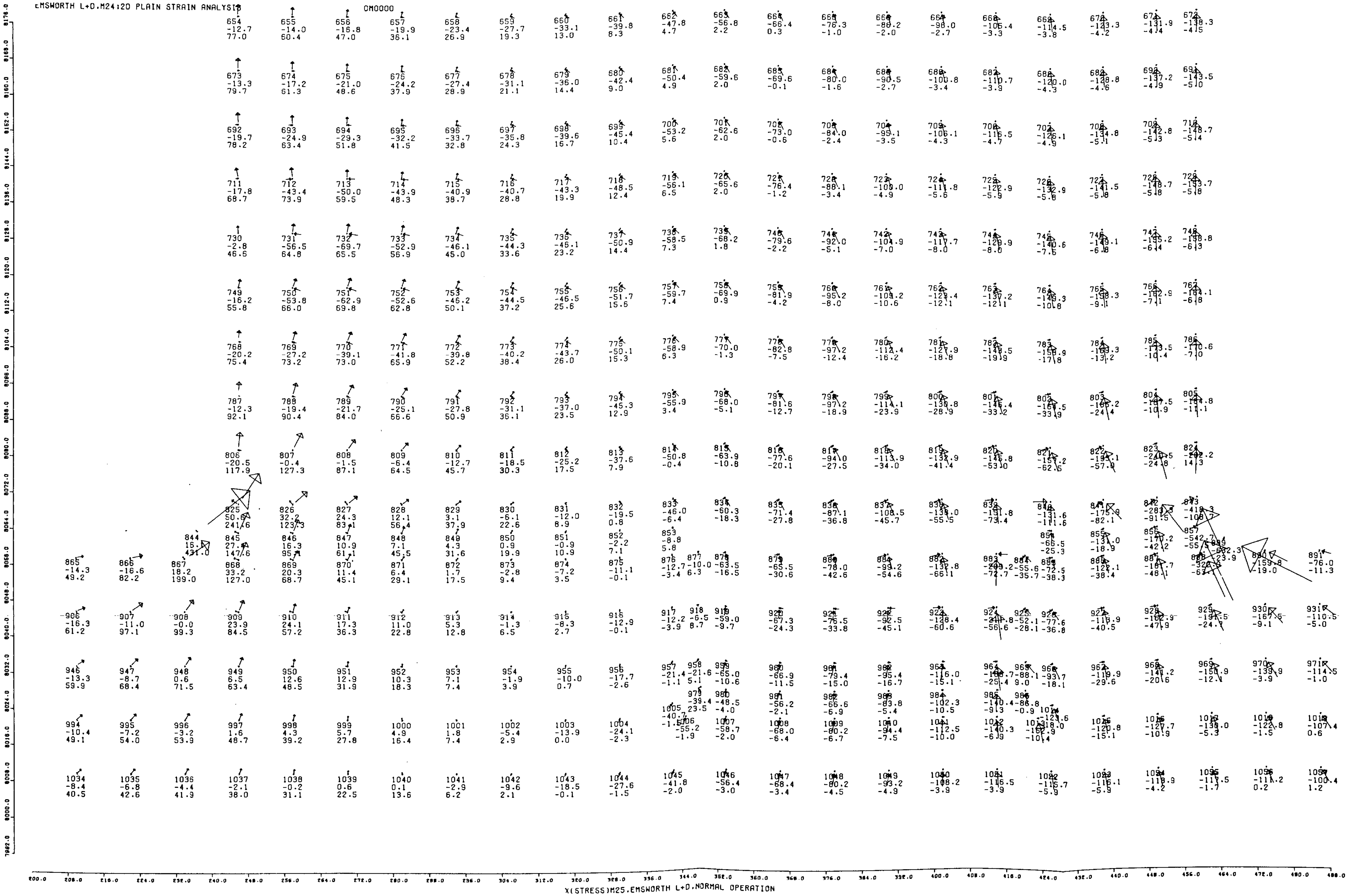


Figure 7.18 Monolith M-25, area of stress concentration as depicted by Area "D," Figure 7.17. Normal operating case

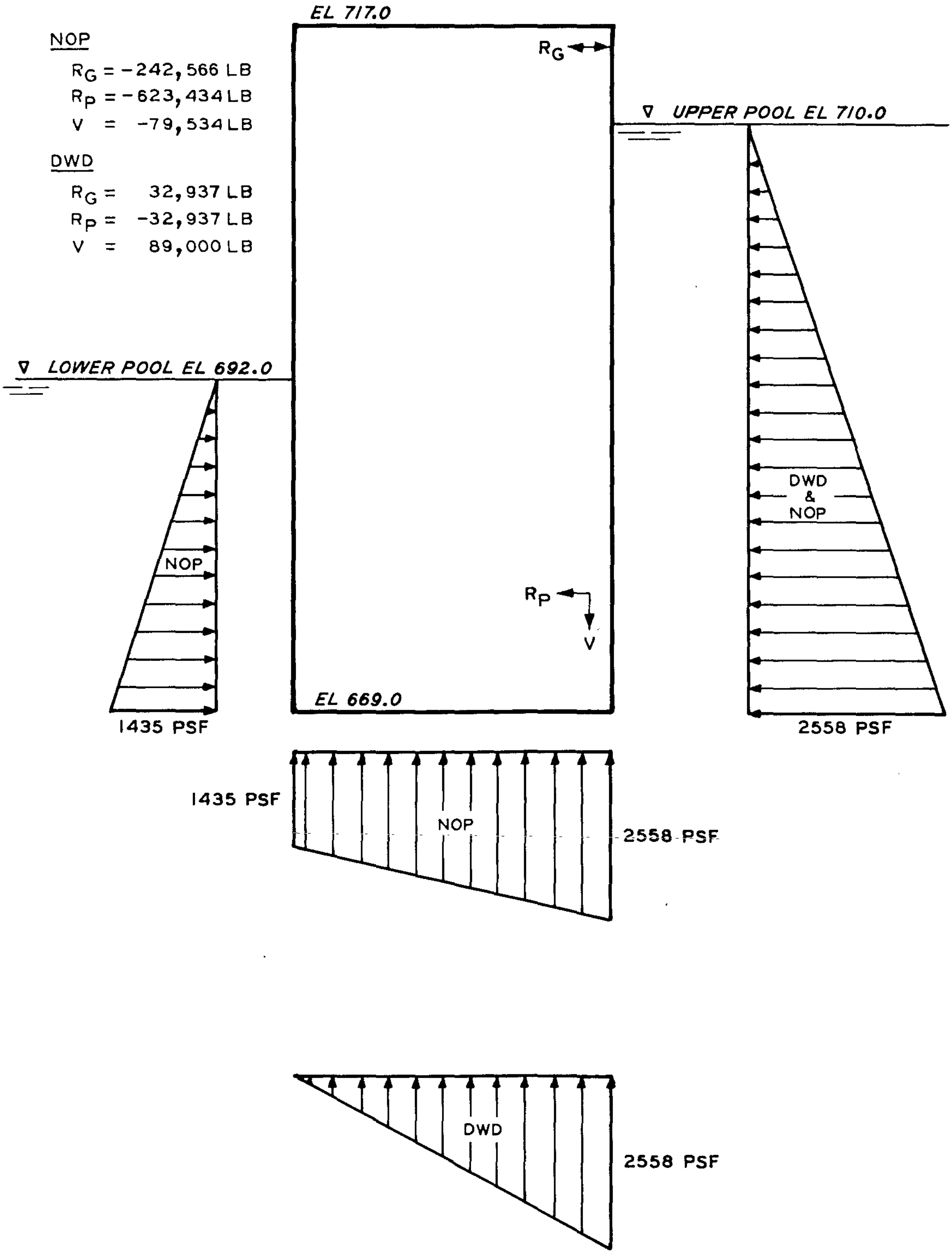


Figure 7.19 Loading, Monolith R-27

R27,Ems,NOP

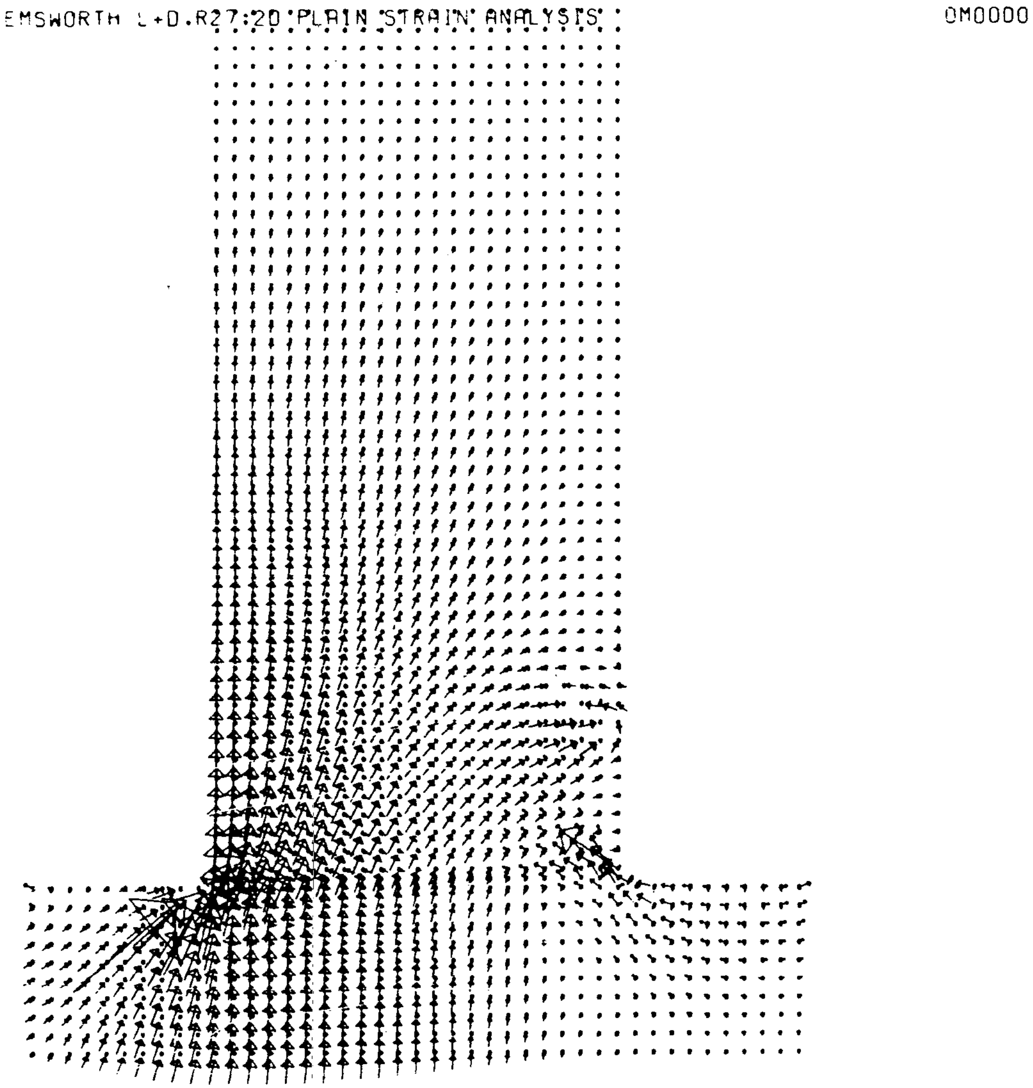


Figure 7.20 Total stress distribution, R-27, normal operation

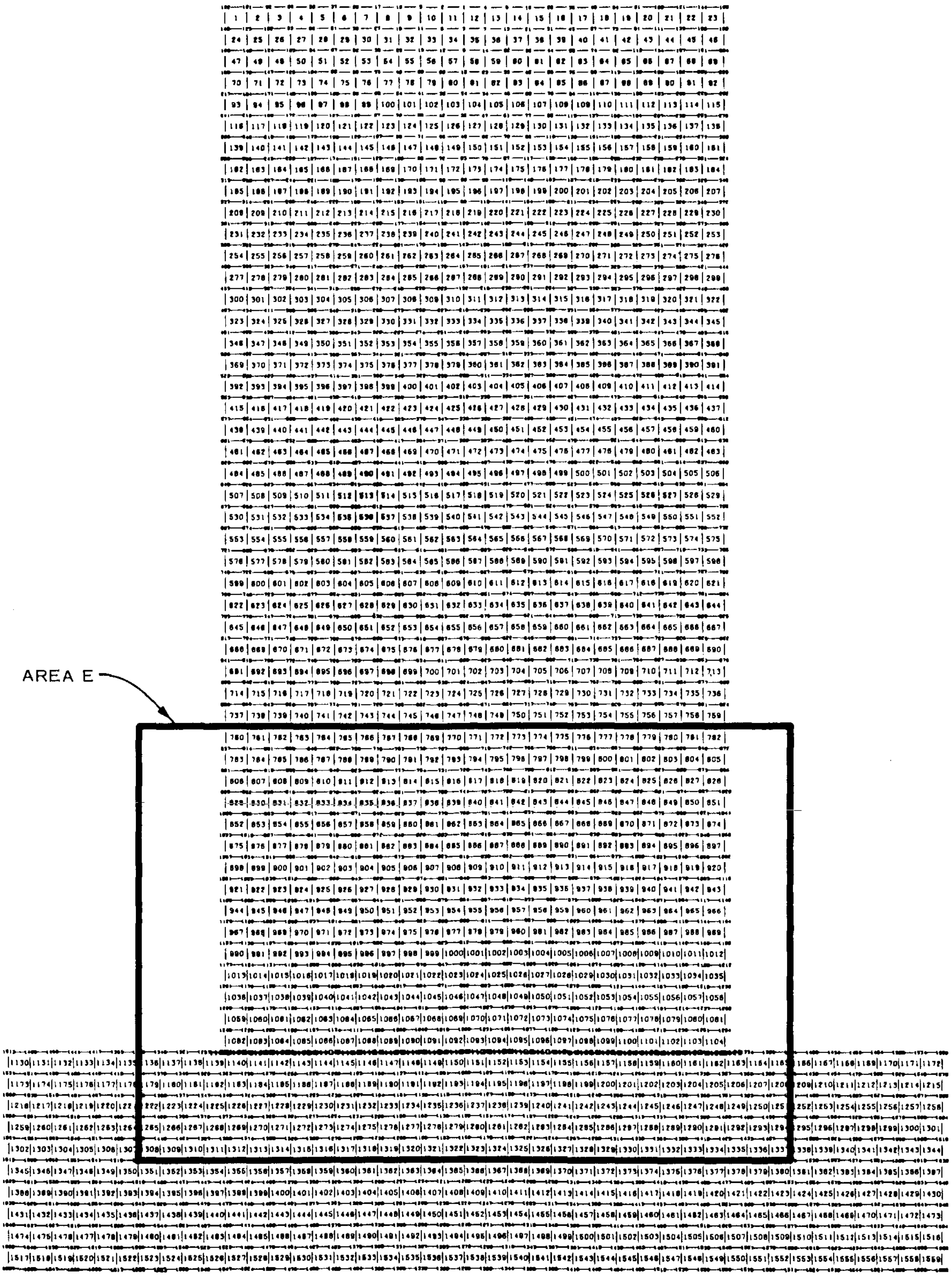


Figure 7.21 River wall monolith, R-27, depicting stress concentration area for presentation

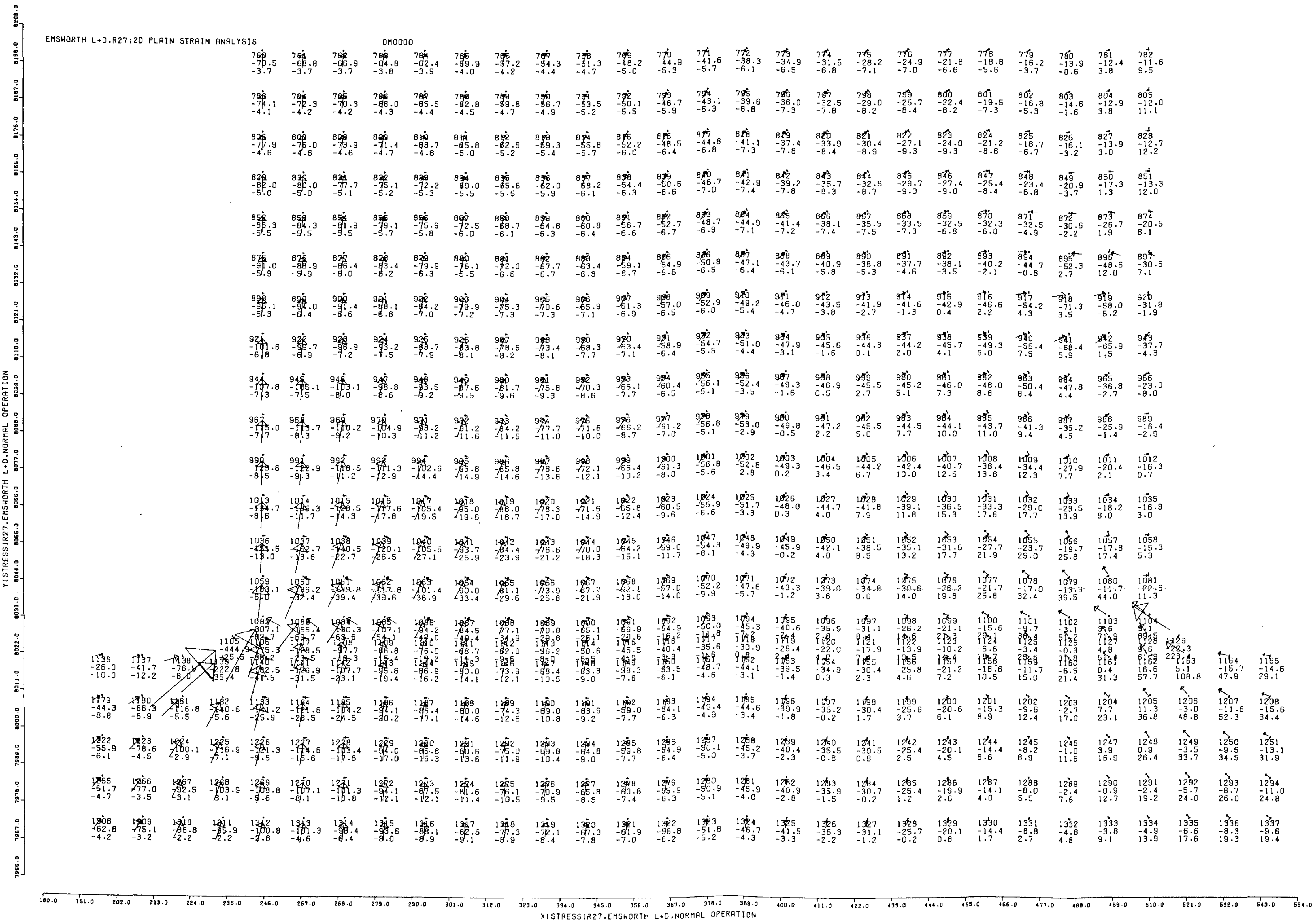


Figure 7.22 Monolith R-27, area of stress concentration as depicted by Area "E," Figure 7.21, normal operation case

SECTION 8: CONCLUSIONS AND RECOMMENDATIONS

8.1 In general, the monoliths on the land wall do not meet present day criteria for overturning, sliding, or base pressures. Also, some monoliths in the middle and river walls do not meet present day stability requirements. In fact, the stability analysis of M-22 along with the visual observation of a 1-1/2 in. separation between the ceiling of the emptying culverts and the middle wall indicates that there has been some movement of these middle wall monoliths.

8.2 The main concern for concrete integrity is the cracked, spaulded and deteriorated surface concrete which will allow accelerated deterioration reducing the effective section of the monoliths increasing the already excessive tensile stresses. In general, if corrective measures are not taken, this will surely cause maintenance expense and will also reduce the life of the concrete monoliths. The compressive stresses are larger than indicated by the stress analysis and can also cause problems in deteriorated concrete.

8.3 From the deteriorated condition of the surface of the lock monoliths, it is evident that some action must be initiated. Since corrective action is needed, a feasibility study should be made to determine what action is necessary that will provide the most economical and adequate lock usage over a period of 30 to 50 years. For this reason, it is recommended that a feasibility study be made considering the following alternatives:

- a. Minimum maintenance and protection of the locks and dam from weathering with expected replacement when needed as determined by periodic inspections.
- b. Rehabilitation of locks and dam.
- c. Replacement of locks and dam.

8.4 The above recommendations may be affected by a total structural and operational evaluation. In fact, this study does not evaluate the steel gates, bridge work, lock gates, or appurtenant mechanical or electrical facilities; these will be considered by the Pittsburgh District in the overall evaluation of the locks and dam.

REFERENCES

1. Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures, First Periodic Inspection Report, Emsworth Locks and Dam, Ohio River, Pennsylvania, Feb 1972.

2. Condition of Emsworth Locks and Dam, Ohio River, Pennsylvania, US Army Engineer Waterways Experiment Station, Vicksburg, MS, Mar 1974.

3. US Army Engineer Waterways Experiment Station, Corps of Engineers, Handbook for Concrete and Cement, with quarterly supplements, Vicksburg, MS, Aug 1949.

APPENDIX C: STABILITY ANALYSIS

THE FIRST AND ONLY APPENDIX IN THIS REPORT

Introduction

C.1 In the stability analysis, the monoliths of the locks and dam were checked for adequacy against overturning, sliding, and excessive base pressures.

C.2 In general, the stability study was done in accordance with the applicable portions of the following Engineer Manuals and Engineer Technical Letters.

a. EM 1110-2-2502, Retaining Walls.

b. EM 1110-2-2602, Planning and Design of Navigation Lock Walls and Appurtenances.

c. EM 1110-2-2607, Navigation Dam Masonry.

d. ETL 1110-2-22, Design of Navigation Lock Gravity Walls.

e. ETL 1110-2-184, Gravity Dam Design Stability.

C.3 The summary sheets and stability computations are given in Table C.1, and Figures C-1 through C-21, respectively.

Applied Loads

C.4 The lock and dam monoliths were investigated for two case loadings as given below:

a. Normal operating condition:

(1) Upper guide, land, and lower guide wall monoliths: the most critical loadings of upper pool, lower pool, and saturation level in backfill. Also, dead load, uplift, tow impact, hawser pull, wind, and gate loads were used when applicable.

(2) Middle and river wall monoliths: Normal lower and upper pools, uplift, impact, hawser pull, wind, and gate loads as applicable were considered in this case.

b. Maintenance or dewatered condition: Backfill, gate, dead loads, and uplift were considered. The saturation levels in the backfill were used as given in Table C.1. Impact, hawser pull, and wind loads were applied according to the situation.

C.5 The standard procedure was to analyze three-dimensional monoliths unless the geometry was uniform enough or could be closely approximated in order that a two-dimensional section of unit depth could be used to represent the stability of the total monolith. All sections were viewed from upstream looking downstream. Forces acting toward the right, downward, and clockwise moments are considered positive. In all cases, the lower left-hand corner of the monolith was used as the center of moments.

C.6 Approximations were necessary concerning several significant factors which affect the stability analysis; these approximations are discussed below.

C.7 The soil behind some land wall monoliths sloped higher than the monolith itself which creates a surcharge loading affecting horizontal and, in some cases, the vertical pressures acting on the monolith. Also, the railroad is located directly behind the land wall monoliths which vibrates the backfill and effects the horizontal load on these monoliths. Both vertical and horizontal pressures were calculated using average fill height. For sloping backfill, the average height used was that over the area for which the vertical pressure was calculated. The horizontal soil pressure was obtained using the average of the backfill surface of the top of the monolith and the height directly behind the monolith.

C.8 In the case of Emsworth Locks and Dam, with the gravity walls supported on component rock, the "at-rest" pressure coefficient is used as the coefficient of horizontal pressure. A lower bound coefficient of at-rest pressure was used. The only way to get experimental values would be to make a number of tests at the lock and dam site using the actual backfill material. The scope of this work in time and funding was not such that this type of testing was possible. On this basis, it was decided to estimate a lower bound value. This lower bound was obtained by considering the value for sand (from dense to loosely compacted) as 0.45 to 0.55; for silt, 0.6; and for clay, from 0.7 to 1.0. It is reasonable, therefore, to use a lower bound at-rest earth coefficient of 0.5. Since the railroad track is located directly behind the land wall monoliths and the backfill slopes upward behind part of the upper guide wall, the tendency is for the horizontal earth pressure to be increased. This makes it even more reasonable to not consider horizontal earth pressure coefficients less than 0.5.

C.9 It was concluded from EM 1110-2-2502, that the magnitude of horizontal soil force on the landside of the monolith can be computed by using a linear distribution of earth pressure. The location of the resultant horizontal soil force will not be at the centroid of this linear pressure distribution; it will be somewhat higher. The resultant location was used as 0.45H above the base because of the railroad loading and, in some cases, upward sloping backfill behind the monoliths.

C.10 The unit weight of concrete, drained backfill material, and submerged backfill material was used as 146.8 lb/ft^3, 112.7 lb/ft^3, and 132.8 lb/ft^3, respectively.

C.11 Boat impact loads were applied on the basis of design loads used for locks previously constructed with considerations given in EM 1110-2-2602. The loads which were used are:

- a. Lock chamber walls: 800 lb/ft but not less than 40,000 lb per monolith.
- b. Other walls: 2500 lb/ft but not less than 120,000 lb per monolith.

The boat impact was considered as acting 5 ft above the waterline and was combined with the most severe normal loading conditions.

C.12 A hawser pull of 24,000 lb was applied 5 ft above pool height and was considered distributed over a monolith length of about 30 ft.

C.13 When considering gate load, hawser pull, impact loads, etc., which act on a localized area of the monolith, the loads were distributed on a per foot basis when a two-dimensional stability analysis was made. This is accurate enough for stability analysis but is not accurate enough when considering localized stresses.

C.14 Ice loads would make some case loadings more critical.

Design Criteria

C.15 The monoliths were checked for overturning by considering where the resultant intersected the monolith base.

C.16 Resistance to overturning was considered adequate if the resultant fell outside the kern but within the middle half of the base for normal operation cases using "at-rest" earth pressure coefficients. The resultant for the extreme maintenance condition using "at-rest" earth pressures was considered adequate if it fell outside the kern but within the middle half of the base.

C.17 The criteria for determining resistance to sliding are given in ETL 1110-2-184 and the safety factors are listed in ETL 1110-2-22.

C.18 There is no problems in engineering concepts if the total base pressure is compressive because for massive-rigid structures it can be obtained rather accurately by $f = P/A \pm Mc/I$ considering the total projected area of the base. The problem arises when the monolith just rests on a foundation and part of the base is in tension, which in reality cannot exist. If the total base is used in the analysis when part of the area is noneffective (shows tension), the equilibrium equations are not even satisfied. The way to determine the base pressures is to consider only the effective part of the monolith base which is in compression. This will be done and the effective area for a rectangular base is derived below.

C.19 Consider the resultant force "x" distance from the left toe of the monolith and solve the equation $f = P/A - Mc/I$ when the stress (f) equals zero.

$$\frac{P_y}{A} - \frac{Mc}{I} = 0$$

$$\frac{P_y}{d} - \frac{\left(\frac{d}{2} - x\right) P_y \frac{d}{2}}{\frac{d^3}{12}} = 0$$

solving $d = 3x$ valid for $b > d > 0$.

C.20 The above derivation is for a two-dimensional section with a unit depth of 1 ft. The stress is then:

$$P_y\ (x) = \frac{f(3x)}{2}\ \frac{1}{3}\ (3x)$$

$$f = \frac{2}{3}\ \frac{P_y}{x}$$

If the resultant falls outside the base, the monolith should began overturning. By conventional design, the resultant falls outside the base for some of the lock monoliths. This is, in reality, not the case because the monoliths are in relatively good alignment.

C.21 In as many years as the lock has been in operation, the monoliths have not shown excessive settlement or misalignment; therefore, the resultant of all forces acting on them must fall within the base. This means that the conventional analysis is not considering some factor or factors. These factors are probably ones which are not dependable enough at this point of study to be justified in good engineering design. For example, such factors could be:

- a. The force required to shear a failure wedge from behind the monolith as would have to happen for tilting of the monolith to begin.
- b. The degree of uplift, which we are using in the design, may be greater then the actual situation.
- c. A refinement in parameters and calculation methods is needed to more accurately obtain a horizontal soil force against the monoliths.

C.22 There are no criteria for calculating pressures when the resultant falls outside the base; all the pressure would be on the toe of the monolith giving large pressures; therefore, a value of ∞ is given for these base pressures in Table C-1.

C.23 The above is supplemental information for stability considerations and makes no analyses or conclusions concerning the monoliths of Emsworth Locks and Dam. The analyses and conclusions are given in Section 6.

Table C.1: Summary of Stability Analysis Results

		Percent Effective Base		Sliding Safety Factor		Foundation Pressure, k/sf	
Monolith	Cases Considered	Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
L-3	Normal operation	100	95	4	4.7	20	3.1
L-19	Normal operation	100	0	4	0.8	20	∞
L-34	Normal operation	100	60	4	3.0	20	30.0
	Maintenance	75	1	2-2/3	1.9	20	∞
L-37	Normal operation	100	58	4	2.6	20	14.5
	Maintenance	75	43	2-2/3	2.1	20	20.0
L-52	Normal operation (with impact)	100	35	4	1.8	20	23.1
	Normal operation (with hawser)	100	29	4	1.7	20	27.9
	Maintenance	75	11	2-2/3	1.3	20	78.9
L-68	Normal operation (with impact)	100	100	4	4.3	20	4.6
	Normal operation (with hawser)	100	99	4	21.6	20	5.8
M-5	Normal operation	100	100	4	28.4	20	6.2
	Maintenance	75	100	2-2/3	4.6	20	11.8
M-8	Normal operation	100	75	4	3.2	20	10.1
	Maintenance	75	93	2-2/3	3.2	20	8.8

(Continued)

Table C.1

Monolith	Cases Considered	Percent Effective Base		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
M-12	Normal operation	100	100	4	4.5	20	7.8
	Maintenance (110-ft lock)	75	100	2-2/3	4.1	20	8.5
	Maintenance (56-ft lock)	75	100	2-2/3	3.8	20	9.8
M-22	Normal operation	100	39	4	1.4	20	23.3
	Maintenance	75	0	2-2/3	0.9	20	∞
M-25	Normal operation (condition 1)	100	50	4	3.5	20	16.5
	Dewatered (condition 1)	75	43	2-2/3	2.7	20	20.8
	Normal operation (condition 2)	100	21	4	2.0	20	54.8
	Dewatered (condition 2)	75	14	2-2/3	1.7	20	97.6
R-4	Normal operation	100	61	4	3.9	20	6.1
R-14	Normal operation	100	100	4	101.3	20	4.3
	Maintenance	75	100	2-2/3	1.9	20	11.5
R-17	Normal operation	100	70	4	4.8	20	8.3
	Maintenance	75	73	2-2/3	4.6	20	8.7
R-24	Normal operation	100	39	4	1.2	20	20.6
	Maintenance	75	93	2-2/3	2.6	20	11.2

(Continued)

Table C.1 (Continued)

Monolith	Cases Considered	Percent Effective Base		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
R-27	Normal operation (1)	100	62	4	1.9	20	18.1
	Maintenance	75	100	2-2/3	3.9	20	11.8
	Normal operation (2)	100	100	4	94.8	20	7.5
R-32	Normal operation	100	100	4	6.6	20	4.9
Upper guard wall cells	Normal operation	100	100	4	29.0	20	3.1

Table C.2

Saturation Levels to Use in the Backfill of the Land Wall Monoliths

Sections of Land Side Lock Wall	Saturation Elevations for Normal Operating Conditions	Saturation Elevations for Extreme Maintenance Conditions
Upper guide wall monolith	One-half way between upper pool and the top of lock wall	--
Upper gate monoliths	Upper pool elevation	Upper pool elevation
Lock chamber monoliths	One-half way between upper pool and lower pool elevations	Three-fourths way between upper pool and lower pool elevations
Lower gate monoliths	One-half way between upper pool and lower pool elevations	Three-fourths way between upper pool and lower pool elevations
Lower guide wall monoliths	One-half way between upper pool and lower pool elevations	One-half way between upper pool and lower pool elevations

SUBJECT: LANDWALL - UPPER GUIDE WALL MONOLITH L-3

COMPUTED BY:

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DATE:

STATION: 13+80.0A TO 13+38.5A = 41.5

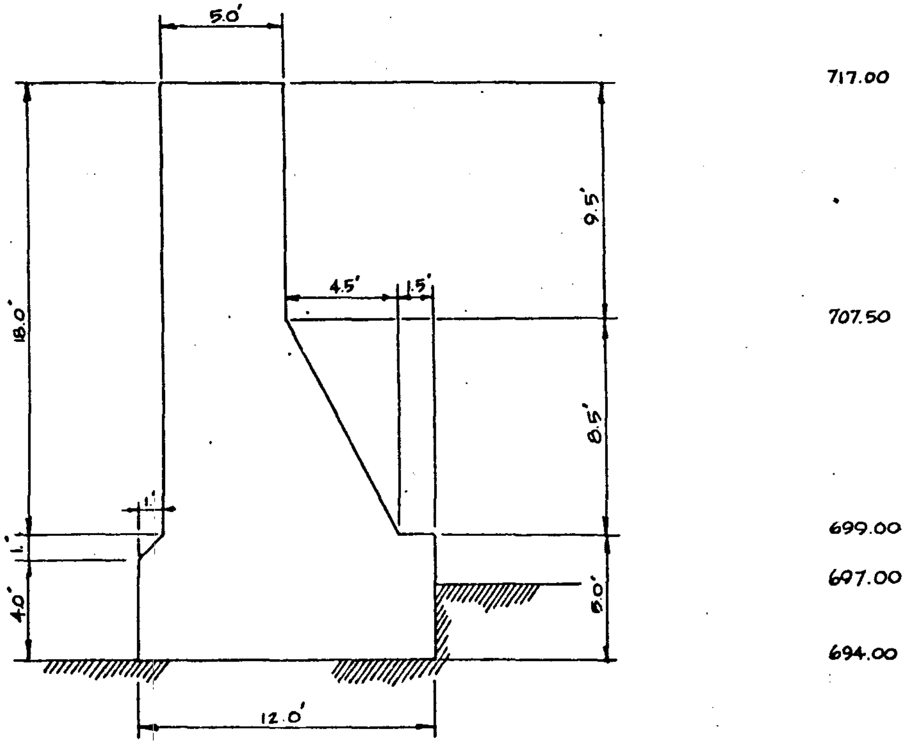

FIGURE C-1 LANDWALL - UPPER GUIDE WALL MONOLITH L-3

WES FORM NO 1253A DECEMBER 1966

SUBJECT: LANDWALL UPPER GUIDE WALL MONOLITH L-3

COMPUTED BY: | DATE:
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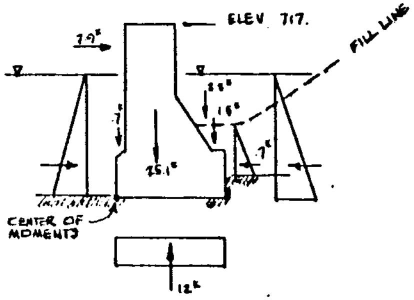
NOT TO SCALE



UPPER POOL = 710.00

FILL ELEV = 703.25

ITEM	FACTORS	Fv	FH	ARM	MOMENT
W CONC	[1488][(5)(23) + (1)(4) + (1)(1)(½) + (8.5)(4.5)(½) + (6)(5)]	25.1		4.87	122
W WATER LOCK	[.0625][(11)(1) + (1)(1)(½)]	0.7		0.50	0
W WATER LAND	[0625][(2.5)(6) + (2.25)(4.25)(½) + (3.75)(4.25)]	2.2		9.29	20
W EARTH	[1328][(2.25)(4.25)(½) (1.5)(4.25)]	1.8		10.60	16
	P WATER LOCK = - P WATER LAND HORIZ. FORCES CANCEL				
P EARTH	[.0703][(½)(703.25 - 697)²(.5)]		-0.7	5.81	- 4
UPLIFT	[.0625][(16)(12)]	-12.0		6.00	- 72
IMPACT	2.9 KIPS		2.9	21.00	61
		17.5	2.2		143

CASE I NORMAL OPERATIONS

UPPER POOL = SATURATION ELEV. = 710.0'
FILL ELEV (AT MONOLITH FACE) = 703.25'

$$e = \frac{143}{17.5} = 8.2'$$

$$\% \text{ Effective Base} = \frac{11.4}{12} \times 100 = 95\%$$

FIGURE C-1 LANDWALL UPPER GUIDE WALL MONOLITH L-3 (CONTINUED)

SUBJECT: LANDWALL UPPER GUIDE WALL MONOLITH L-3

COMPUTED BY:

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SLIDING

$R = \Sigma F_V \tan \phi$

$R = (17.5)(0.5890) = 10.31$

$S_{SF} = 10.31/2.2 = 4.69$

BASE PRESSURE

$f = \frac{2}{3}\frac{P}{e}$

$= \frac{2}{3}\left(\frac{17.5}{3.8}\right)$

$= 307$ KIPS/SQ FT.

FIGURE C-1 LANDWALL UPPER GUIDE WALL MONOLITH L-3 (CONCLUDED)

SUBJECT: LANDWALL - UPPER GUIDE WALL MONOLITH - L-19

COMPUTED BY:

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DATE:

STATION: 7+94.0A TO 7+68.0A = 26

MONOLITH L-19 AS CONSTRUCTED

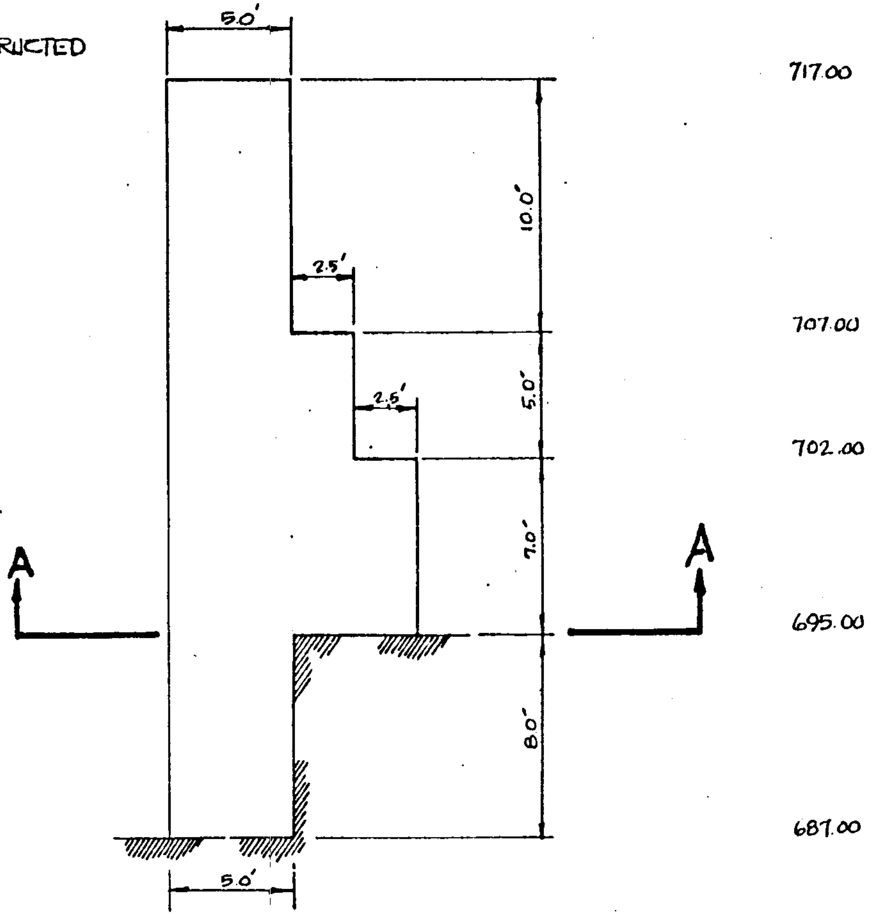


FIGURE C-2 LANDWALL- UPPER GUIDE WALL MONOLITH L-19

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LANDWALL - UPPER GUIDE WALL MONOLITH L-19

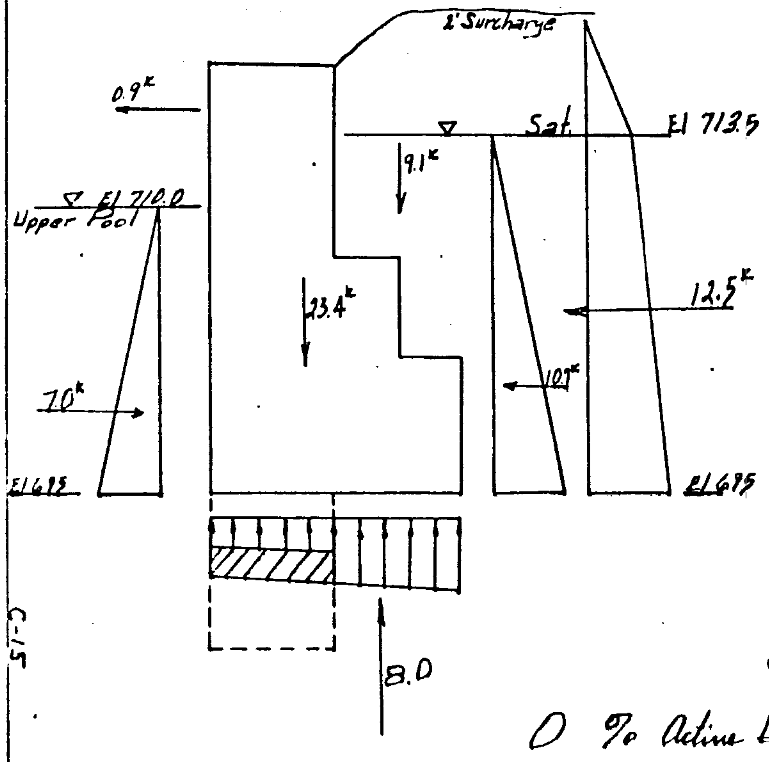
COMPUTED BY:

DATE:

CHECKED BY:

DATE:

Not to Scale



SECTION A-A

ITEM	FACTORS	F_V	F_H	ARM	MOMENTS
W_{CONC}	$(0.1488)[(5)(22)+(2.5)(12)+(2.5)(7)]$	23.4		3.93	92
$W_{EARTH + WATER}$	$(0.1127)[(5.5)(5)]+(0.1328)[(6.5)(5)+(5)(2.5)]$	9.1		7.72	70
$P_{WATER\ LOCK}$	$(0.0625)(710-695)^2(1/2)$		7.0	5.00	35
$P_{WATER\ LAND}$	$(0.0625)(713.5-695)^2(1/2)$		-10.7	6.17	-66
P_{EARTH}	$(0.1127)[(1/3)(5.5)^2(1/2)-(1/2)(2)(5)+(5.5)(8.5)(1/2)]$ $+[0.0704](1/2)(18.5)^2(1/2)$		-12.5	10.80	-135
Uplift	$(0.0625)[(6.75)(5)+(1.75)(5)(1/2)+(7.5)(5)$ $+(0.875)(5)(1/2)]$	-8.0		6.02	-48
HAWSER	0.923 KIPS		-0.9	20.00	-18
		24.5	-17.1		-70

$e = \frac{-70}{24.5} = -2.86'$

0 % Active Base (considering zero moment in stem)

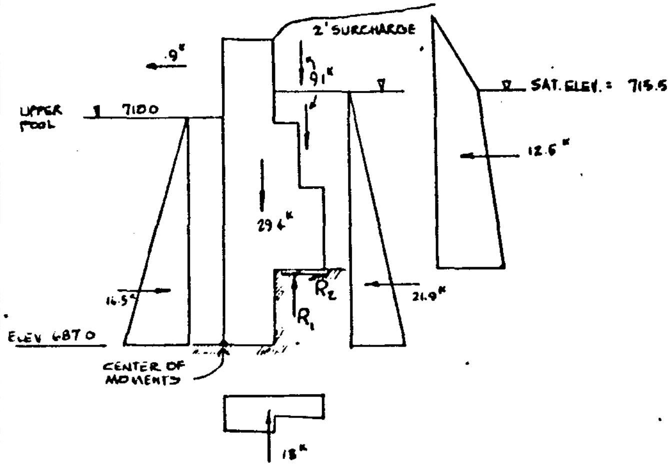
This is a stress consideration because overturning at this section would have to break the lower slice off.

The location of the resultant at A-A means that the normal and horizontal forces must be transferred through the stem; $R_1 = 0$. Since the cohesion is zero R_2 is also zero. See the location and direction of R_1 and R_2 on next page.

FIGURE C-2 LANDWALL - UPPER GUIDE WALL MONOLITH L-19 (CONTINUED)

SUBJECT: LANDWALL UPPER GUIDE WALL MONOLITH L-19	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENTS
$W_{CONC.}$	$[.1488][(5)(30)+(2.5)(12)+(2.5)(7)]$	29.4		3.62	106
$W_{EARTH+WATER}$	$[.1127][(5.5)(5)]+[.1328][(6.5)(5)+(5)(2.5)]$	9.1		7.72	70
$P_{WATER\ LOCK}$	$[.0625][(710-687)^2(1/2)]$		16.5	7.67	127
$P_{WATER\ LAND}$	$[.0625][(715.5-687)^2(1/2)]$		-21.9	8.83	-194
P_{EARTH}	$[.1127][(1/2)(5.5)^2(.5)-(1/2)(2)^2(.5)]+(5.5)(18.5)(.1127)(.5)+[.0704](1/2)(18.5)^2(.5)$		-12.5	18.80	-235
UPLIFT	$[.0625][(17.53)(5)+(1/2)(5)(.97)+(23)(5)+(1/2)(5)(.97)]$	-13.0		4.69	-61
HAWSER	.923 KIPS		-.9	28.00	-25
		25.5	-18.8		-212

$$e = \frac{-212}{25.5} = -8.3'$$

CASE I NORMAL OPERATIONS

UPPER POOL IN RIVER = 710.0
SATURATION ELEV = 713.5

The stability of the total monolith should then be considered about the lower base. Reactions R_1 and R_2 must be considered on the total free body. They were calculated assuming no moment in the stem at section A-A and by assuming R_2 to be the horizontal force based on the pressure under the step and the coefficient of sliding friction.

0% Active Base.

FIGURE C-2 LANDWALL - UPPER GUIDE WALL MONOLITH L-19 (CONTINUED)

SUBJECT: LANDWALL - UPPER GUIDE WALL MONOLITH L-19	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

SLIDING @ SECTION A-A

$R = \Sigma F_V \tan\phi$ on Shearing of Stem

$R = (0.075)(5)(144) = 54^K$

$S_{SF} = \frac{54}{17.1} = 3.16$

BASE PRESSURES

BASE PRESSURES ARE VERY LARGE

$R = \Sigma F \tan\phi + cA$

SLIDING TOTAL MONOLITH

$R = \Sigma F_V \tan\phi$

$= (25.5)(0.5890)$

$= 15.02^K$

$S_{SF} = 15.02/18.8 = 0.80$

FIGURE C-2 LANDWALL - UPPER GUIDE WALL MONOLITH L-19 (CONCLUDED)

SUBJECT:	COMPUTED BY:	DATE:
LAND WALL - UPPER GATE MONOLITH L-34	CHECKED BY:	DATE:

STATION: 2+24.3A TO 2+85.9A = 61.6

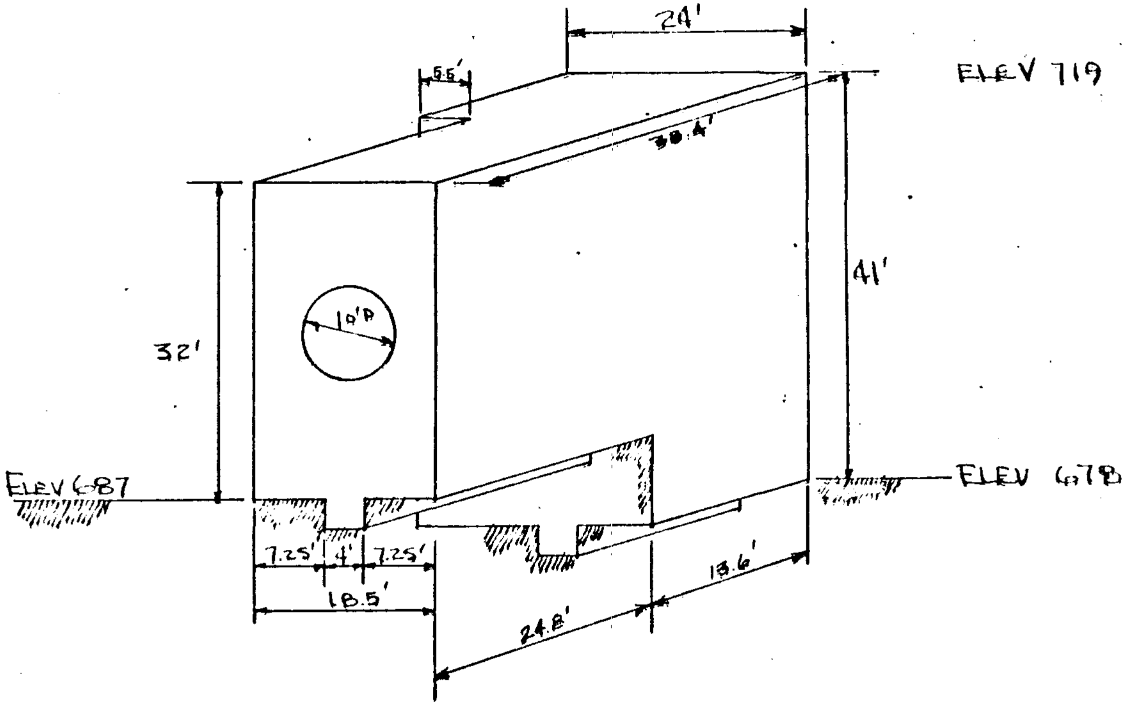
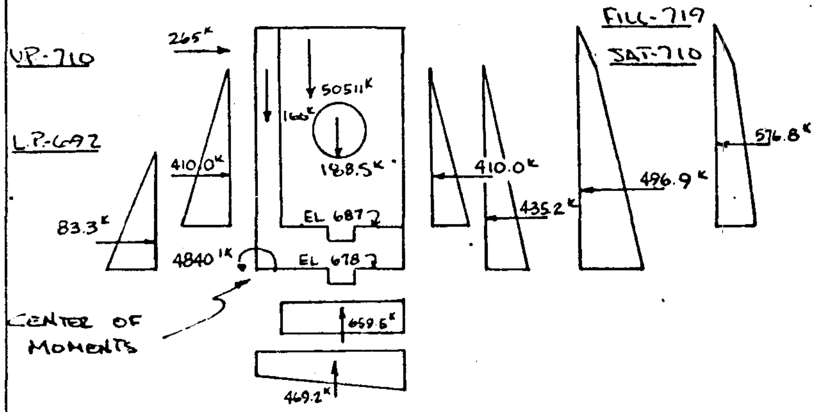


FIGURE C-3 LANDWALL - UPPER GATE MONOLITH L-34

WES FORM NO. 1253A
DECEMBER 19

SUBJECT: LAND WALL- UPPER GATE MONOLITH L-34	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1488] [(24)(41)(13.60)+(3)(4)(13.60) − $\pi(5)^2$(13.60) +(18.5)(32)(24.80)+(3)(4)(24.80) − $(\pi)(5)^2$(24.80)]	3794.9		13.31	50,511
P_W(UP)	($\frac{1}{2}$)$(710-687)^2$(.0625)(24.80)		410.0	16.67	6835
P_W(DOWN)	($\frac{1}{2}$)$(692-678)^2$(.0625)(13.60)		83.3	4.67	389
P_W(UP)	($\frac{1}{2}$)$(710-687)^2$(.0625)(24.80)		-410.0	16.67	-6835
P_W(DOWN)	($\frac{1}{2}$)$(710-678)^2$(.0625)(13.60)		-435.2	10.67	-4644
P_E(UP)	($\frac{1}{2}$)$(719-710)^2$(.1127)(24.80)(.5) +(719-710)(710-687)(.1127)(.5)(24.80)+($\frac{1}{2}$)$(710-687)^2$(.0704)(.5)(24.80)		-576.8	23.4	-13497
P_E(DOWN)	($\frac{1}{2}$)$(719-710)^2$(.1127)(13.60)(.5) +(719-710)(710-678)(.1127)(.5)(13.60)+($\frac{1}{2}$)$(710-678)^2$(.0704)(.5)(13.60)		-496.9	18.45	− 9168
W_W	$(\pi)(5)^2$(38.4)(.0625)	188.5		14.75	2780
UPLIFT(UP)	(710-687)(18.5)(.0625)(24.80)	-659.5		14.75	− 9728
UPLIFT(DOWN)	(692-678)(24)(.0625)(13.60)+($\frac{1}{2}$)(710-692)(24)(.0625)(13.60)	-469.2		13.56	− 6362
$GATE_S$			265.0	38.08	10,091
$GATE_V$		160		2.67	427
$GATE_M$					- 4840
		3015	-1160.6		15959

CASE I: NORMAL OPERATIONS

SATURATION- 710
L.P.- 692- IN LOCK

$$e = 15959 / 3015 = 5.29'$$

$$\% \text{ Effective Base} \approx \frac{(10.37)(24.8)+(15.87)(13.6)}{(18.5)(24.8)+(13.6)(24)} \times 100 \approx 60\%$$

FIGURE C-3 LANDWALL UPPER GATE MONOLITH L-34 (CONTINUED)

SUBJECT: LAND WALL- UPPER GATE MONOLITH L-34

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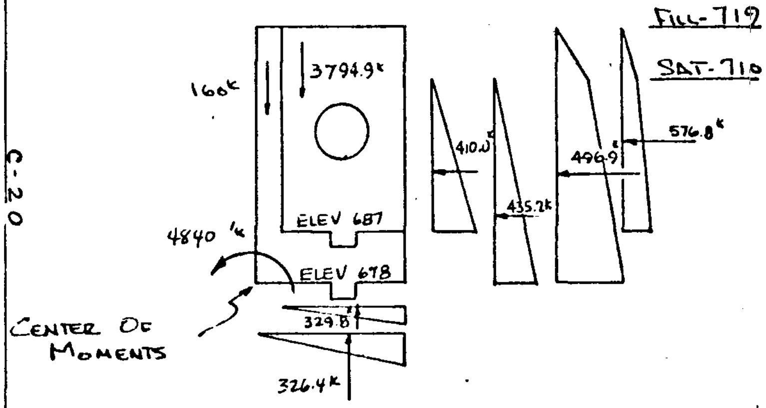
NOT TO SCALE



ITEM	FACTORS	Fv	FH	ARM	MOMENT
WCONC		3794.9		13.31	50511
Pw(UP)			-410.0	16.67	-6835
Pw(DOWN)			-435.2	11.67	-4644
PE(UP)			-576.8	23.40	-13497
PE(DOWN)			-496.9	18.45	-9168
UPLIFT(UP)	(½)(710-687)(.0625)(18.5)(24.80)	-329.8		17.83	-5880
UPLIFT(DOWN)	(½)(710-687)(24)(.0625)(13.60)	-326.4		16.00	-5222
GATEV		160.0		2.67	427
GATEM					-4840
		3298.7	-1918.9		852

e = 852 / 3298.7 = 0.26'

CASE II: MAINTENANCE CONDITION

DEWATER LOCK
SATURATION - 710

$$\% \text{ Effective Base} \approx \frac{(0.78)(13.6)}{785.2} \times 100 \approx 1\%$$

FIGURE C-3 LANDWALL UPPER GATE MONOLITH L-34 (CONTINUED)

SUBJECT: LAND WALL - UPPER GATE MONOLITH L-34

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

SLIDING

CASE I: NORMAL OPERATIONS

$R = V \tan \phi + \text{KEY RESISTANCE}$

$R = (3015.0)(.5890) + (4)(38.4)(0.075)(144)$

$R = 1775.8 + 1658.9 = 3434.7$

$S_{SF} = \frac{3434.7}{1160.6} = 3.0$

CASE II: MAINTENANCE CONDITION

$R = (3298.7)(.5890) + (4)(38.4)(0.075)(144)$

$R = 1942.9 + 1658.9 = 3601.8$

$S_{SF} = \frac{3601.8}{1918.9} = 1.9$

BASE PRESSURE

CASE I: NORMAL OPERATIONS

$f = \frac{2}{3} \frac{P}{e}$

avg. monolith width $= \frac{(24)(13.6) + (18.5)(24.8)}{38.4} = 20.45$

then $24 - 20.45 = 3.55$

assume an effective e as $5.29 - 3.55 = 1.74'$

$f = \frac{2}{3} \frac{3051}{(1.74)(38.4)} = 30$ KIPS/SQ FT.

CASE II: MAINTENANCE CONDITION

BASE PRESSURE IS VERY LARGE

FIGURE C-3 LANDWALL - UPPER GATE MONOLITH L-34 (CONCLUDED)

SUBJECT:	COMPUTED BY:	DATE:
LAND WALL - UPPER CHAMBER MONOLITH L-37	CHECKED BY:	DATE:

STATION 1+99.8A TO 2+30.2A = 30.4

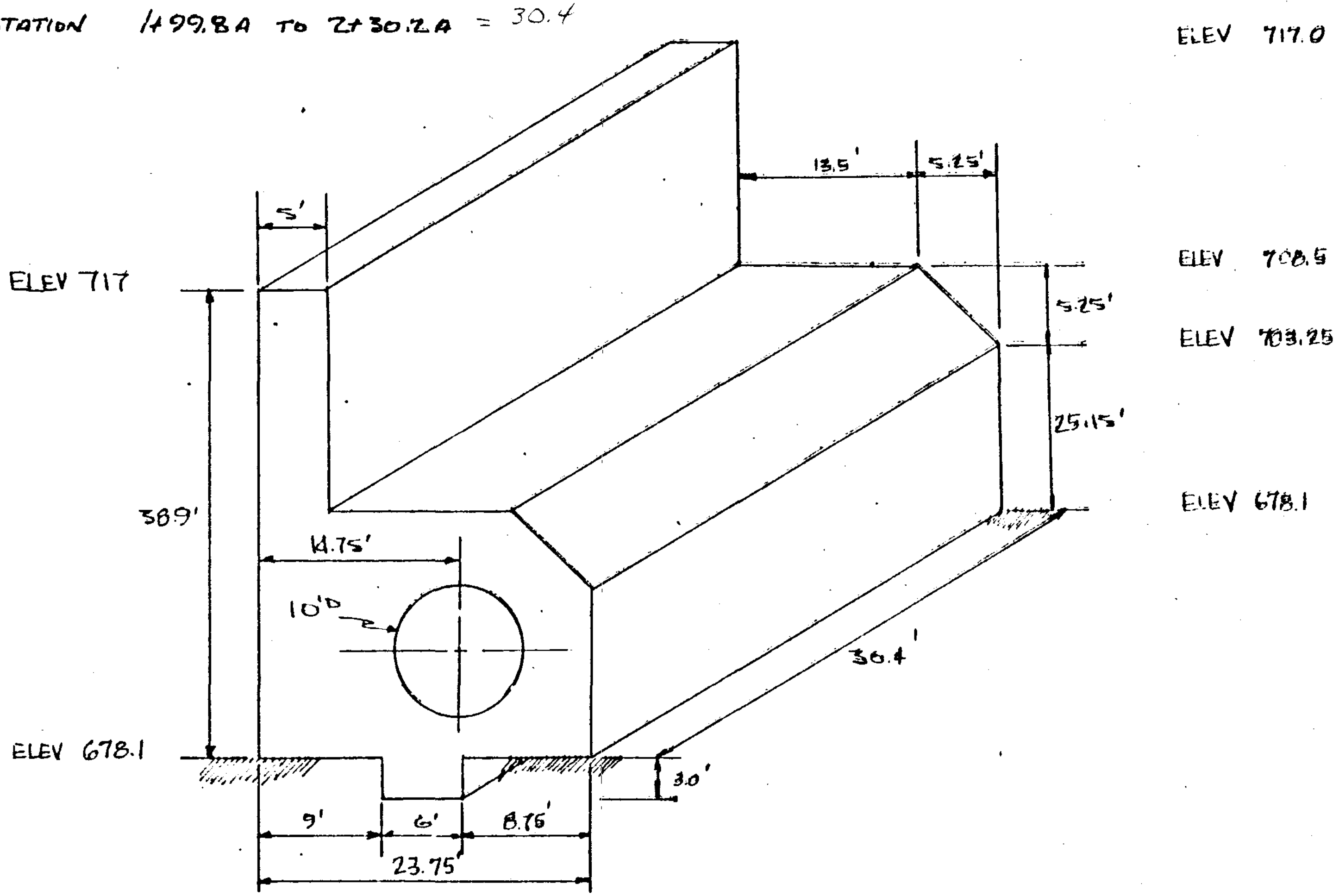


FIGURE C-4 LANDWALL - UPPER CHAMBER MONOLITH L-37

SUBJECT

LANDWALL - UPPER CHAMBER MONOLITH L-37

COMPUTED BY: DATE

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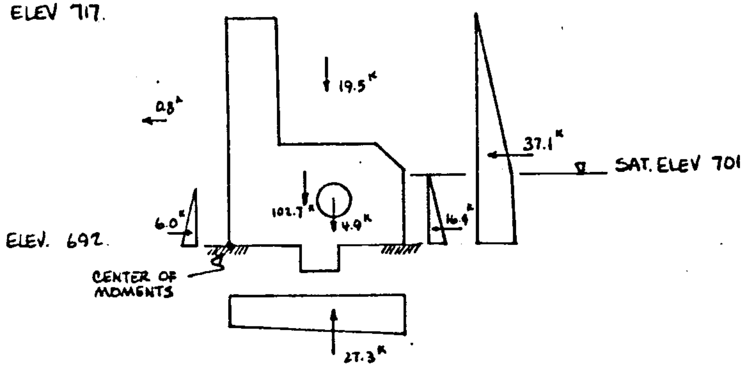
NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[.1488][(5)(38.9)+(18.75)(30.4)+(3)(6)-(1/2)(5.25)(5.25)-(\pi)(5)^2]$	102.7		10.77	1106
$P_{WATER-LAND}$	$(1/2)(701-678.1)^2(.0625)$		-16.4	7.63	-125
$P_{WATER-LOCK}$	$(1/2)(692-678.1)^2(.0625)$		6.0	4.63	28
P_{EARTH}	$[.1127][(717-701)^2(1/2)(.5)+(717-701)(701-678.1)(.5)]+(0704)(701-678.1)^2(1/2)(.5)$		-37.1	14.78	-548
W_{WATER}	$\pi(5)^2(.0625)$	4.9		14.75	72
W_{EARTH}	$[.1127][(717-708.5)(18.75)+(5.25)^2(1/2)]$	19.5		14.97	292
UPLIFT	$[.0625][(692-678.1)(23.75)+(701-692)(1/2)(23.75)]$	-27.3		12.85	-351
HAWSER	$0.8^K/FT$		-0.8	18.90	-15
		99.8	-48.3		459

$$e = \frac{459}{99.8} = 4.6'$$

CASE I NORMAL OPERATIONS

LOWER POOL IN LOCK 692.0
SATURATION ELEVATION 701.0

$$\% \text{ Effective Base} = \frac{13.8}{23.75} \times 100 = 58\%$$

FIGURE C-4 LANDWALL - UPPER CHAMBER MONOLITH L-37

SUBJECT

LANDWALL - UPPER CHAMBER MONOLITH L-37

COMPUTED BY:
DATE:
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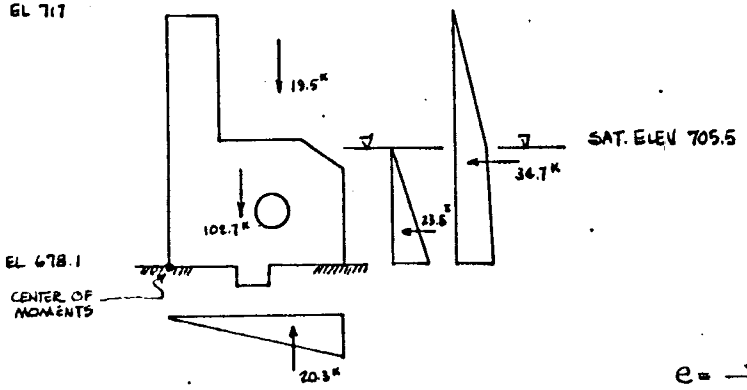
DATE:

NOT TO SCALE



ITEM	FACTORS	Fv	FH	ARM	MOMENT
W CONC	$[.1488][(5)(38.9)+(18.75)(30.4)+(3)(6)-(1/2)(5.25)(5.25)-(\pi)(5)^2]$	102.7		10.77	1106
P WATER	$(1/2)(705.5-678.1)^2(0.0625)$		-23.5	9.13	- 215
P EARTH	$[.1127][(717-705.5)^2(1/2)(.5)+(717-705.5)(705.5-678.1)(.5)]+(.0704)(705.5-678.1)^2(1/2)(.5)$		-34.7	14.78	- 513
W EARTH	$[.1127][(717-708.5)(18.75)+(5.25)^2(1/2)]$	19.5		14.97	292
UPLIFT	$(1/2)(23.75)(705.5-678.1)(0.0625)$	- 20.3		15.83	- 321
		101.9	-58.2		349

$$e = \frac{349}{101.9} = 3.4'$$

$$\% \text{ Effective Base} = \frac{10.2}{23.75} \times 100 = 43\%$$

CASE II MAINTENANCE CONDITION

NO WATER IN CULVERT
LOCK DEWATERED

FIGURE C-4 LANDWALL - UPPER CHAMBER MONOLITH L-37 (CONTINUED)

SUBJECT LAND WALL - UPPER CHAMBER L-37

COMPUTED BY | DATE
CHECKED BY | DATE

C-25

SLIDING

CASE I: NORMAL OPERATIONS

$R = V\tan\phi + \text{KEY RESISTANCE}$

$R = (99.8)(.5890) + (6)(.075)(144)$

$R = 58.8 + 64.8 = 123.6$

$S_{SF} = \dfrac{123.6}{48.3} = 2.6$

CASE II: MAINTENANCE CONDITION

$R = (101.9)(.5890) + 64.8$

$R = 60.0 + 64.8 = 124.8$

$S_{SF} = \dfrac{124.8}{58.2} = 2.1$

BASE PRESSURE

CASE I: NORMAL OPERATIONS

$f = \left(\frac{2}{3}\right)\left(\frac{P}{e}\right)$

$f = \left(\frac{2}{3}\right)\left(\frac{99.8}{4.6}\right) = 14.5 \text{ K/FT}^2$

CASE II: MAINTENANCE CONDITION

$f = \left(\frac{2}{3}\right)\left(\frac{101.9}{3.4}\right) = 20.0 \text{ K/FT}^2$

FIGURE C-4 LANDWALL UPPER CHAMBER MONOLITH L-37

SUBJECT:

LANDWALL - LOWER CHAMBER MONOLITH L-52

COMPUTED BY:

DATE:

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DATE:

STATION 2+42.1B TO 2+12.1B = 30'

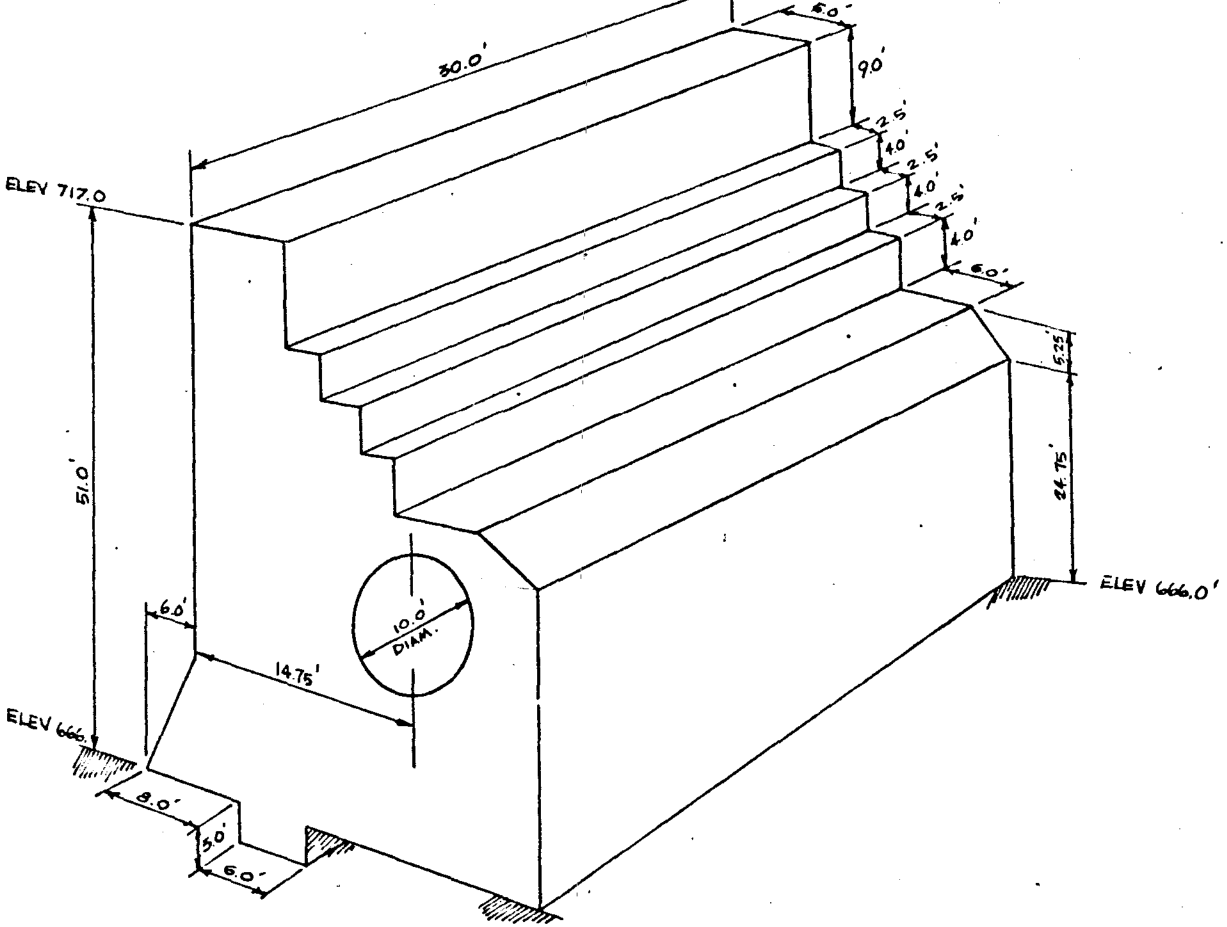


FIGURE C-5 LANDWALL - LOWER CHAMBER MONOLITH L-52

SUBJECT

LANDWALL - LOWER CHAMBER MONOLITH L-52

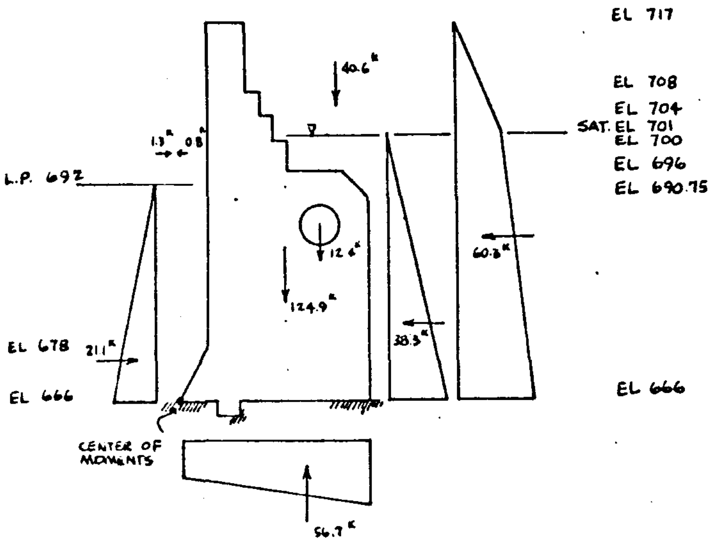
COMPUTED BY:

DATE

CHECKED BY:

DATE

NOT TO SCALE



ITEM	FACTORS	Fv	FH	ARM	MOMENT
WCONC	$[.1488][(51)(5)+(42)(2.5)+(38)(2.5)+(34)(2.5)+(30)(6)+(24.75)(5.25)+(1/2)(5.25)^2+(1/2)(12)(6)-\pi(5)^2+(3)(6)]$	124.9		15.24	1903
WEARTH	$[.1127][(9)(18.75)+(4)(16.25)+(3)(13.75)]+[.1328][(1)(13.75)+(4)(11.25)+(1/2)(5.25)^2$	40.6		21.90	889
WWATER	$[.0625][(6)(14)+(1/2)(12)(6)+\pi(5)^2]$	12.4		9.84	122
PWATER-LAND	$[.0625][35]^2(1/2)$		-38.3	11.67	- 447
PWATER-LOCK	$[.0625][26]^2(1/2)$		21.1	8.67	183
PEARTH	$[.1127][(717-701)^2(1/2)(.5)+(717-701)(701-666)(.5)+[.0704][(701-666)^2(1/2)(.5)]$		-60.3	22.95	- 1384
UPLIFT	$[.0625][(692-666)(29.75)+(1/2)(29.75)(701-692)]$	- 56.7		15.61	- 885
HAWSER	0.8 ᴷ/FT		- 0.8	31.00	- 25
IMPACT	40ᴷ/30' = 1.3 ᴷ/FT		1.3	31.00	41
	① WITH IMPACT NO HAWSER	121.2	- 76.2		422
	② WITH HAWSER NO IMPACT	121.2	- 78.3		356

CASE I NORMAL OPERATIONS

LOWER POOL IN LOCK
SATURATION ELEV = 701.0

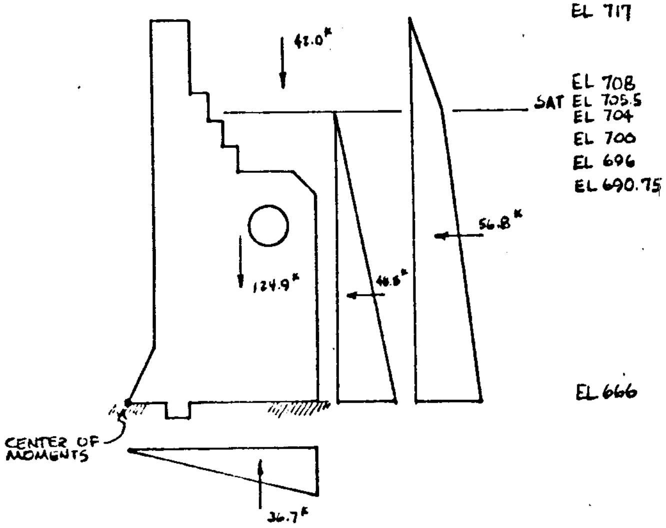
$$e_{①} = \frac{422}{121.2} = 3.5' \qquad \% \text{ Effective Base} = \frac{10.5}{29.75} \times 100 = 35\%$$

$$e_{②} = \frac{356}{121.2} = 2.9' \qquad \% \text{ Effective Base} = \frac{8.7}{29.75} \times 100 = 29\%$$

FIGURE C-5 LANDWALL - LOWER CHAMBER MONOLITH L-52 (CONTINUED)

SUBJECT	COMPUTED BY:	DATE
LANDWALL - LOWER CHAMBER MONOLITH L-52	CHECKED BY:	DATE

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		124.9		15.24	1903
W_{EARTH}	$[.1127][(9)(18.75)+(2.5)(16.25)]+$ $[.1328][(1.5)(16.25)+(4)(13.75)+$ $(4)(11.25)+(1/2)(5.25)^2]$	42.0		21.91	920
P_{WATER}	$(1/2)(705.5-666)^2(.0625)$		-48.8	13.17	-643
P_{EARTH}	$[.1127][(717-705.5)^2(1/2)(.5)+$ $(717-705.5)(705.5-666)(.5)]+$ $[.0704](705.5-666)^2(1/2)(.5)$		-56.8	22.95	-1304
UPLIFT	$[.0625][705.5-666](29.75)(1/2)$	-36.7		19.83	- 728
		130.2	-105.6		148

$$e = \frac{148}{130.2} = 1.1$$

$$\% \text{ Effective Base} = \frac{3.3}{29.75} \times 100 = 11\%$$

CASE II MAINTENANCE CONDITION

DEWATERED LOCK
SATURATION ELEV 705.5

FIGURE C-5 LANDWALL - LOWER CHAMBER MONOLITH L-52

SUBJECT: LANDWALL - LOWER CHAMBER MONOLITH L-52	COMPUTED BY:	DATE
	CHECKED BY:	DATE

SLIDING CASE I

$R = \Sigma V \tan\phi$ + KEY RESISTANCE

$= (121.2)(0.5890) + (6)(144)(0.075)$

$R = 71.4 + 64.8 = 136.2$

① $S_{SF} = \frac{136.2}{76.2} = 1.8$

② $S_{SF} = \frac{136.2}{78.3} = 1.7$

CASE II MAINTENANCE

$R = (130.2)(.5890) + 64.8$

$R = 76.7 + 64.8 = 141.5$

$S_{SF} = \frac{141.5}{105.6} = 1.3$

BASE PRESSURE CASE I

$f = \frac{2}{3}\frac{P}{e}$

① $f = \frac{2}{3}\frac{121.2}{3.5} = 23.1$ K/SQ FT.

② $f = \frac{2}{3}\frac{121.2}{2.9} = 27.9$ K/SQ FT

CASE II MAINTENANCE

$f = \frac{2}{3}\frac{130.2}{1.1} = 78.9$ K/SQ. FT.

FIGURE C-5 LANDWALL - LOWER CHAMBER MONOLITH L-52 (CONCLUDED)

SUBJECT LANDWALL - LOWER GUIDE WALL MONOLITH L-68

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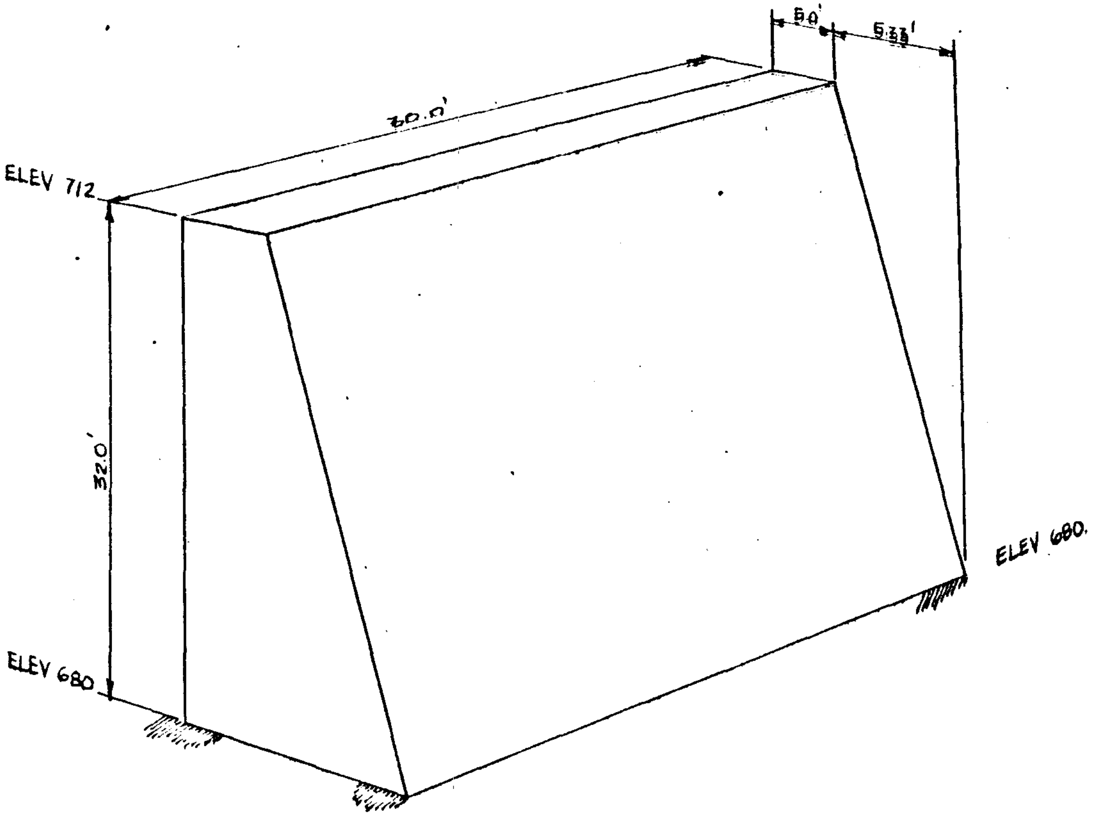


FIGURE C-6 LANDWALL - LOWER GUIDE WALL MONOLITH L-68

SUBJECT

LANDWALL - LOWER GUIDE WALL MONOLITH L-68

COMPUTED BY

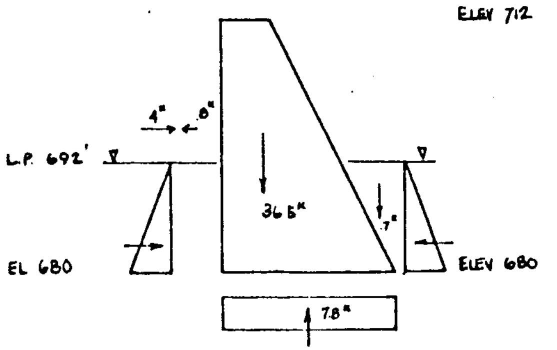
DATE

CHECKED BY

DATE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[.1488][(5)(32)+(1/2)(5.33)(32)]$	36.5		4.00	146
W_{WATER}	$[.0625][(1/2)(2)(12)]$	0.7		9.66	7
P_{WATER}	$P_{WATER\ LAND} = -P_{WATER\ RIVER}$ FORCES CANCEL				
U_{PLIFT}	$[.0625][692-680][10.33]$	-7.8		5.17	-40
IMPACT	$120^{K}/_{MON} \div 30^{FT}/_{MON} = 4^{K}/_{FT}$		4	17.00	68
HAWSER	$.8^{K}/_{FT}$		-.80	17.00	-14
	WITH IMPACT ①	29.4	4		181
	WITH HAWSER ②	29.4	-0.8		99

NOT TO SCALE



CASE I NORMAL OPERATIONS

LOWER POOL BOTH SIDES 692

$$e_{①} = \frac{181}{29.4} = 6.2$$

% Effective Base = 100%

$$e_{②} = \frac{99}{29.4} = 3.4$$

% Effective Base $= \frac{10.2}{10.33} \times 100 = 99\%$

FIGURE C-6 LANDWALL - LOWER GUIDE WALL MONOLITH L-68 (CONTINUED)

SUBJECT: LANDWALL - LOWER GUIDE WALL MONOLITH L-68

COMPUTED BY:

DATE:

CHECKED BY:

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SLIDING

$R = \Sigma V \tan\phi + \text{KEY RESISTANCE}$

$R = (29.4)(.5890) + 0$

$R = 17.3$

① $S_{SF} = \frac{17.3}{4} = 4.3$

② $S_{SF} = \frac{17.3}{0.8} = 21.6$

BASE PRESSURE

CASE I

① $f = \frac{P}{A} \pm \frac{Mc}{I}$

$= \frac{29.4}{10.33} + \frac{(29.4)(6.2-5.165)(5.165)(12)}{(10.33)^3}$

$= 2.85 + 1.71$

$= 4.56$ K/SQ FT.

② $f = \frac{2}{3}\frac{P}{e}$

$= \frac{2}{3}\frac{(29.4)}{(3.4)}$

$= 5.76$ K/SQ FT.

FIGURE C-6 LANDWALL LOWER GUIDE WALL MONOLITH L-68

SUBJECT: MIDDLE WALL- UPPER GATE MONO M-5	COMPUTED BY: CHECKED BY:	DATE: DATE:

STATION 3+24.2A TO 2+86.1A = 38.0'

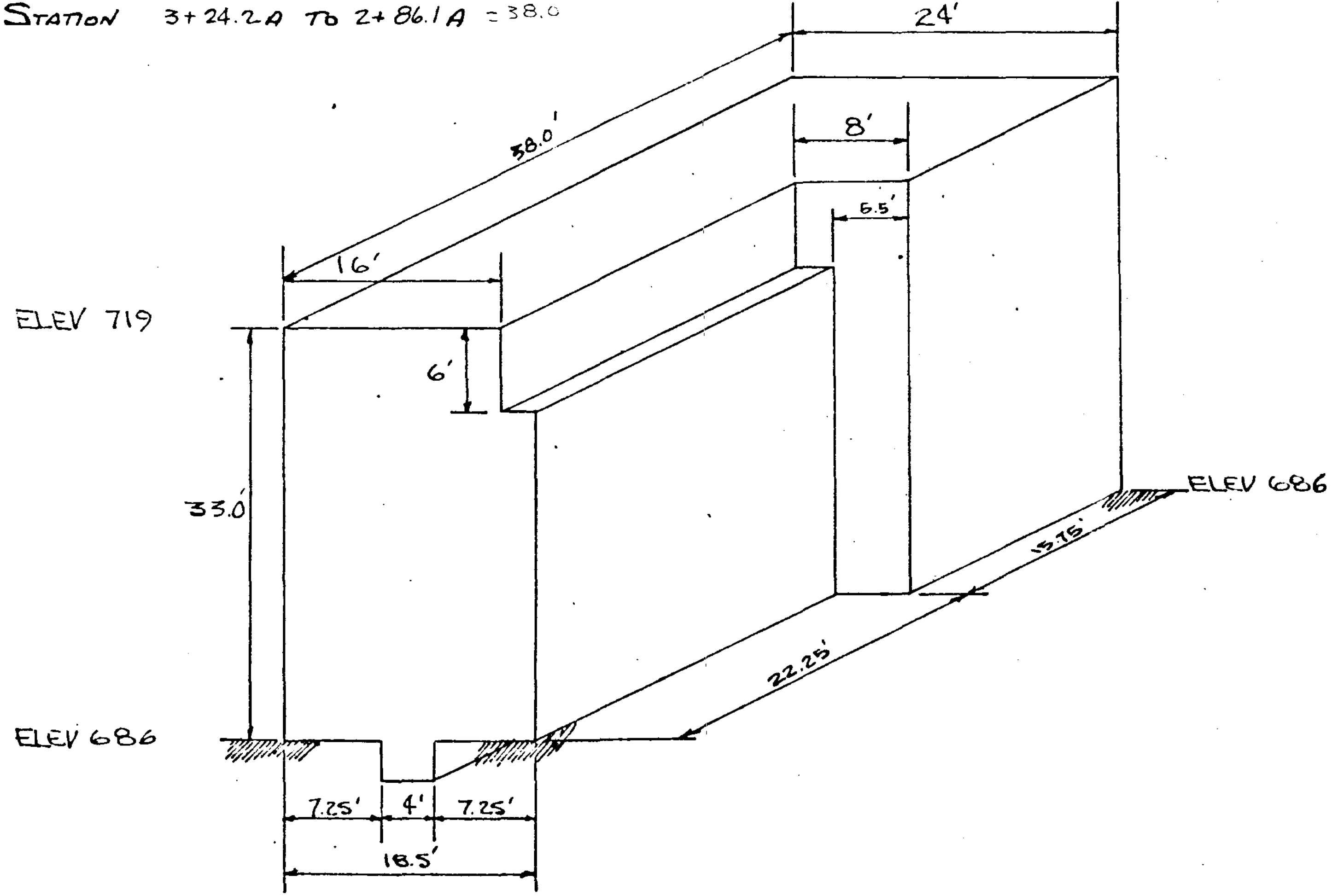
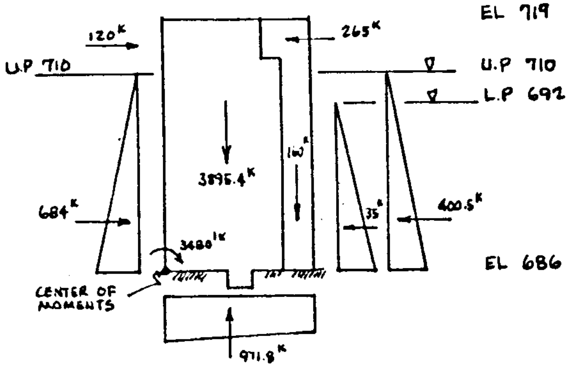


FIGURE C-7 MIDDLE WALL - UPPER GATE MONOLITH M-5

SUBJECT	COMPUTED BY	DATE
MIDDLE WALL - UPPER GATE MONOLITH M-5	CHECKED BY	DATE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[.1488][(16)(33)(22.25)+(2.5)(27)(22.25)+(4)(3)(38)+(24)(33)(15.75)$	3895.4		10.45	40707
$P_{WATER\ RIVER}$	$(1/2)(710-686)^2(.0625)(38)$		684.0	8.00	5472
$P_{WATER\ LAND}$	$(1/2)(710-686)^2(.0625)(22.25)$		-400.5	8.00	-3204
$P_{WATER\ LAND}$	$(1/2)(692-686)^2(.0625)(15.75)$		- 17.7	2.00	- 35
UPLIFT	$[.0625][(710-686)(18.5)(22.25)+(692-686)(15.75)(24)+(1/2)(710-692)(24)(15.75)$	- 971.8		9.38	- 9115
GATE S			-265	30.08	- 7971
GATE V		160		21.33	3413
GATE M					4840
IMPACT	$3.16\ ^K/_{FT} \times 38\ FT$		120	29.00	3480
		3038.8	120.8		37587

$$e = \frac{37587}{3038.8} = 12.38'$$

100% Effective Base

CASE I NORMAL OPERATION

LOW POOL 692 IN 110' LOCK

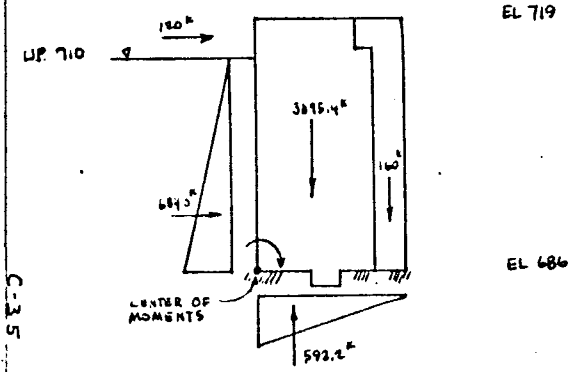
FIGURE C-7 MIDDLE WALL - UPPER GATE MONOLITH M-5 (CONTINUED)

SUBJECT: MIDDLE WALL - UPPER GATE MONOLITH M-5

COMPUTED BY: DATE

CHECKED BY: DATE

NOT TO SCALE



CASE II MAINTENANCE CONDITION

DEWATERED 110' LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		3895.4		10.45	40707
P_{WATER}			684.0	8.00	5472
UPLIFT	$[.0625][(\frac{1}{2})(710-686)(18.5)(22.25)+(\frac{1}{2})(710-686)(24.0)(15.75)]$	-592.2		7.02	-4170
GATE V		160		21.33	3413
GATE M					4840
IMPACT	$(3.16\ ^{K}/_{FT})(38\ FT)$		120.0	29.00	3480
		3463.2	804.0		53742

$$e = \frac{53742}{3463.2} = 15.51'$$

100% Effective Base.

FIGURE C-7 MIDDLE WALL - UPPER GATE MONOLITH M-5 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT MIDDLE WALL UPPER GATE MONOLITH M5

COMPUTED BY

DATE

CHECKED BY

DATE

SLIDING

CASE I NORMAL OPERATION

$$R = \sum V \tan\phi + \text{KEY RESISTANCE}$$

$$= (3038.8)(.5890) + (4)(38)(.075)(144)$$

$$R = 1769.8 + 1641.6 = 3451.4$$

$$SSF = \frac{3431.4}{120.8} = 28.4$$

CASE II MAINTENANCE CONDITION

$$R = (3463.2)(.5890) + 1641.6$$

$$= 2039.9 + 1641.6 = 3681.5$$

$$SSF = \frac{3681.5}{804.0} = 4.58$$

BASE PRESSURE

CASE I NORMAL OPERATION

$$f = \frac{P}{A} \pm \frac{Mc}{I}$$

$$= \frac{3038.8}{789.6} + \frac{(3038.8)(12.38-10.56)(24-10.56)}{31374.1}$$

$$= 3.85 + 2.37 = 6.22\ ^{K}/\text{SQ FT.}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{P}{A} \pm \frac{Mc}{I}$$

$$= \frac{3463.2}{789.6} + \frac{(3463.2)(15.51-10.56)(24-10.56)}{31374.1}$$

$$= 4.39 + 7.37 = 11.76\ ^{K}/\text{SQ FT.}$$

FIGURE C-7 MIDDLE WALL - UPPER GATE MONOLITH M-5 (CONCLUDED)

SUBJECT: MIDDLE WALL - UPPER CHAMBER MONOLITH M-8	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

STATION 2+27.3A TO 1+97.3A = 30'

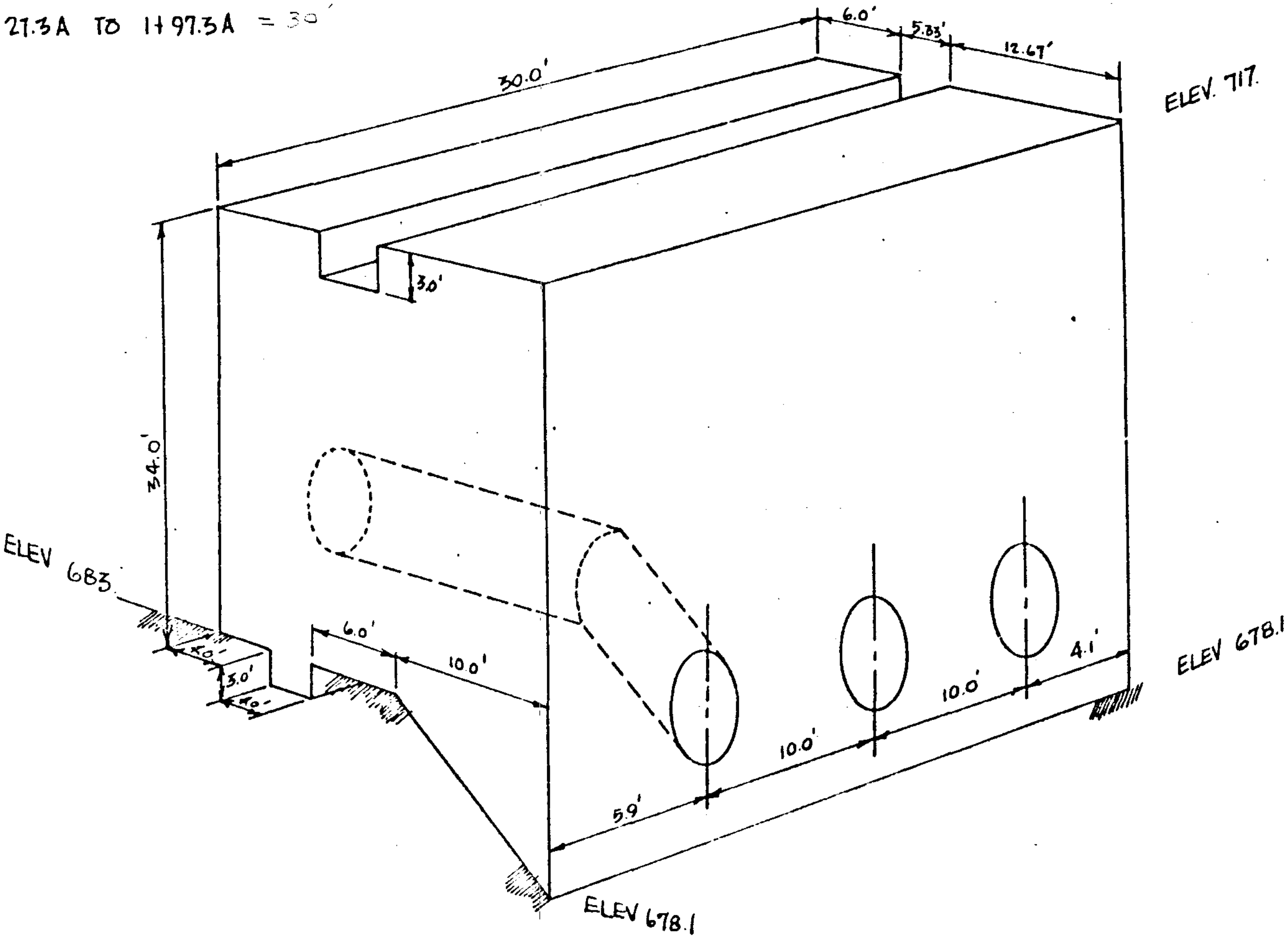


FIGURE C-8 MIDDLE WALL - UPPER CHAMBER MONOLITH M-8

C-37

SUBJECT

MIDDLE WALL – UPPER CHAMBER MONOLITH M-8

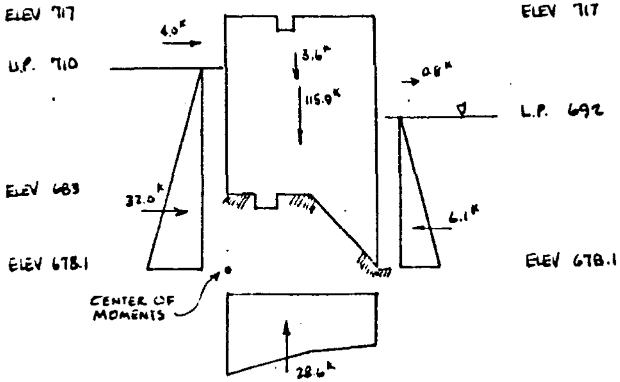
COMPUTED BY

DATE

CHECKED BY

DATE

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[[.1488][(6)(34)(30)+(5.33)(31)(30)+(12.67)(34)(30)+(3)(4)(30)+(1/2)(10)(5)(30)-(\pi)(2.71)^2(14)(3)-(\pi)(2.71)^2(11.2)(3)]\div 30]$	115.9		12.23	1417
$P_{WATER\ RIVER}$	$(1/2)(710-678.1)^2(.0625)$		32.0	10.67	341
$P_{WATER\ LOCK}$	$(1/2)(692-678.1)^2(.0625)$		-6.1	4.67	-28
$W_{WATER\ CULVERTS}$	$[[.0625][\pi][2.71]^2[3](14+11.2)/30]$	3.6		12.22	44
UPLIFT	$[.0625][(14)(10)+(1/2)(2.5)(10)+(16.5)(14)+(1/2)(27-16.5)(14)]$	-28.6		10.49	-300
HAWSER	(.8 K/FT)		0.8	19.00	15
IMPACT	(4 K/FT)		4.0	37.00	148
		90.9	30.7		1637

$$e = \frac{1637}{90.9} = 18.0'$$

$$\%\ \text{Effective Base} = \frac{18}{24} \times 100 = 75\%$$

CASE I NORMAL OPERATIONS

LOWER POOL IN LOCK
UPPER POOL IN RIVER

FIGURE C-8 MIDDLE WALL – UPPER CHAMBER MONOLITH M-8 (CONTINUED)

SUBJECT

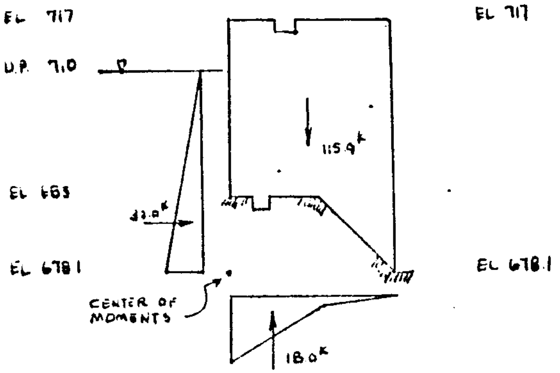
MIDDLE WALL – UPPER CHAMBER MONOLITH M-B

COMPUTED BY | DATE

CHECKED BY | DATE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		115.9		12.23	1417
$P_{WAT\ RIVER}$	$(1/2)(710-678.1)^2(.0625)$		32.0	10.67	341
UPLIFT	$[.0625][(8.38)(14)+(1/2)(18.62)(14)+(1/2)(8.38)(10)]$	-18.0		7.44	-134
		97.9	32.0		1624

NOT TO SCALE



$$e = \frac{1624}{97.9} = 16.58'$$

$$\%\ Effective\ Base = \frac{22.26}{24} \times 100 = 93\%$$

CASE II MAINTENANCE CONDITION

LOCK DEWATERED

FIGURE C-B MIDDLE WALL – UPPER CHAMBER MONOLITH M-B (CONTINUED)

1253A

SUBJECT: MIDDLE WALL - UPPER CHAMBER MONOLITH M-8

COMPUTED BY:

DATE:

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DATE:

SLIDING

CASE I NORMAL OPERATION

$$R = \Sigma V \tan\phi + \text{KEY RESISTANCE}$$

$$= (90.8)(.5890) + (4)(144)(.075)$$

$$= 53.4 + 43.2$$

$$R = 96.68$$

$$S_{SF} = \frac{96.68}{30.7} = 3.15$$

CASE II MAINTENANCE CONDITION

$$R = (97.9)(.5890) + (4)(144)(.075)$$

$$= 57.66 + 43.2$$

$$R = 100.86$$

$$S_{SF} = \frac{100.86}{32.0} = 3.15$$

BASE PRESSURE

CASE I NORMAL OPERATION

$$f = \frac{2}{3}\cdot\frac{P}{e}$$

$$= \frac{(2)(90.8)}{(3)(6)} = 10.1 \text{ K/SQ. FT.}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{2}{3}\cdot\frac{P}{e}$$

$$= \frac{(2)(97.9)}{(3)(7.42)} = 8.8 \text{ K/SQ. FT.}$$

FIGURE C-8 MIDDLE WALL - UPPER CHAMBER MONOLITH M-8 (CONCLUDED)

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - INTERMEDIATE GATE MONO M-12	CHECKED BY:	DATE:

STATION 1+04.7A TO 0+63.6A = 41.1

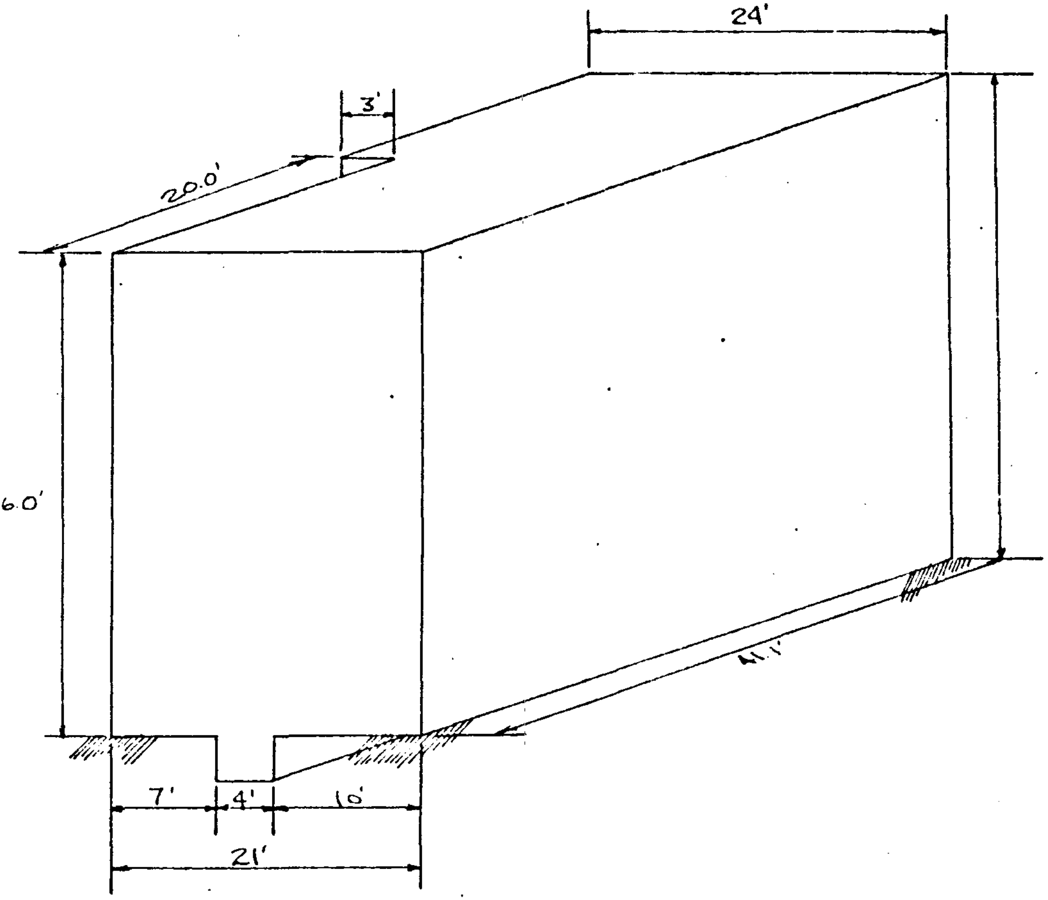


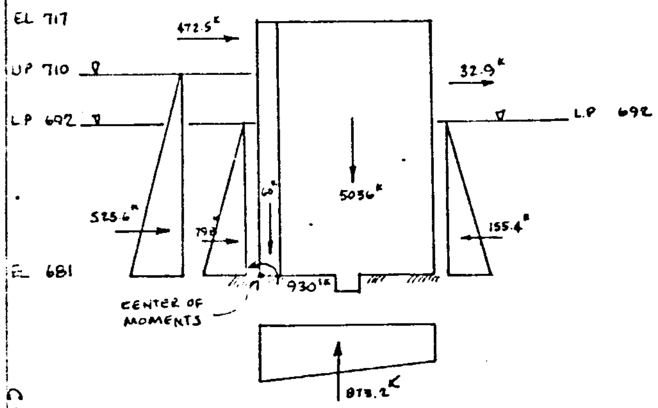
FIGURE C-9 MIDDLE WALL INTERMEDIATE GATE MONOLITH M-12

SUBJECT: MIDDLE WALL - INTERMEDIATE GATE MONOLITH M-12

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[.1488][(21)(36)(20)+(21.1)(24)(36)+(3)(4)(41.1)]$	5036.0		12.67	63806
P_{WATER} (110' LOCK)	$(\frac{1}{2})(692-681)^2(41.1)(.0625)$		-155.4	3.76	- 570
P_{WATER} (56' LOCK)	$(\frac{1}{2})(710-681)^2(20)(.0625)$		525.6	9.67	5083
P_{WATER} (56' LOCK)	$(\frac{1}{2})(692-681)^2(21.1)(.0625)$		79.8	3.67	293
UPLIFT	$[.0625][(692-681)(21)(20)+(\frac{1}{2})(710-692)(21)(20)+(692-681)(21.1)(24)]$	-873.2		11.95	- 10439
GATE S			472.5	15.13	7149
GATE V		60		1.5	90
GATE M					- 930
HAWSER	$(.8^{K}/_{FT})(41.1^{FT})$		32.9	16	526
		4222.8	955.4		65008

CASE I NORMAL OPERATION

LOWER POOL IN 110' LOCK
UPPER POOL IN 56' LOCK

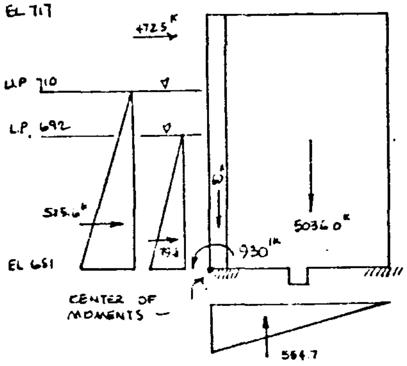
$$e = \frac{65008}{4222.8} = 15.39'$$

100% Effective Base

FIGURE C-9 MIDDLE WALL INTERMEDIATE GATE MONOLITH M-12 (CONTINUED)

SUBJECT	COMPUTED BY:	DATE
MIDDLE WALL - INTERMEDIATE GATE MONOLITH M-12	CHECKED BY:	DATE:

NOT TO SCALE



CASE II MAINTENANCE 110' LOCK

U.P. IN 56' LOCK

ITEM		F_V	F_H	ARM	MOMENT
W_{CONC}		5036.0		12.67	63806
P_{WATER} (56'L)			525.6	9.67	5083
P_{WATER} (56'L)			79.8	3.67	293
UPLIFT	$[.0625][(1/2)(710-681)(21)(20)+(1/2)(692-681)(24)(21.1)]$	-554.7		9.37	5199
GATE S			472.5	15.13	7149
GATE V		60		1.5	90
GATE M					- 930
		4541.3	1077.9		70292

$$e = \frac{70292}{4541.3} = 15.47'$$

100% Effective Base

FIGURE C-9 MIDDLE WALL - INTERMEDIATE GATE MONOLITH M-12 (CONTINUED)

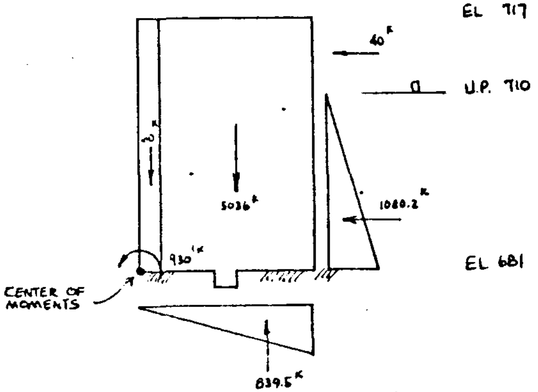
SUBJECT: MIDDLE WALL - INTERMEDIATE GATE MONOLITH M-12

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENTS
W_{CONC}		5036.0		12.67	63806
$P_{WATER\,(110'L)}$	$[.062](1/2)(710-681)^2(41.1)$		-1080.2	9.67	-10446
UPLIFT	$[.0625][(1/2)(710-681)(21)(20)+$ $(1/2)(692-681)(24)(21.1)]$	-839.5		16.45	-13812
GATE V		60		1.50	90
GATE M					-930
IMPACT			-40	34.00	-1360
		4256.5	-1120.2		37348



$$e = \frac{37348}{4256.5} = 8.77'$$

100% Effective Base

CASE III MAINTENANCE 56' LOCK

UPPER POOL IN 110' LOCK

FIGURE C-9 MIDDLE WALL - INTERMEDIATE GATE MONOLITH M-12 (CONTINUED)

SUBJECT: MIDDLE WALL - INTERMEDIATE GATE MONOLITH M-12

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

CASE I NORMAL OPERATIONS

$R = \sum V \tan\phi$ + KEY RESISTANCE

$= (4222.8)(.5890) + (4)(41.1)(.075)(144)$

$= 2487 + 1776 = 4263$

$SSF = \frac{4263}{955.4} = 4.5$

CASE II MAINTENANCE 110' LOCK

$R = (4541.3)(.5890) + 1776$

$= 2675 + 1776 = 4451$

$SSF = \frac{4451}{1077.9} = 4.1$

CASE III MAINTENANCE 56' LOCK

$R = (4256.5)(.5890) + 1776$

$= 2507 + 1776 = 4283$

$SSF = \frac{4283}{1120.2} = 3.8$

BASE PRESSURES

CASE I NORMAL OPERATIONS

$f = \frac{P}{A} \pm \frac{Mc}{I}$

$= \frac{4222.8}{926.4} + \frac{(4222.8)(2.71)(11.32)}{40259}$

$= 4.56 + 3.22$ = 7.78 K/SQ FT.

CASE II MAINTENANCE 110' LOCK

$f = \frac{4541.3}{926.4} + \frac{(4541.3)(2.79)(11.32)}{40259}$

$= 4.90 + 3.56$ = 8.46 K/SQ FT

CASE III MAINTENANCE 56' LOCK

$f = \frac{4256.5}{926.4} + \frac{(4256.5)(3.91)(12.68)}{40259}$

$= 4.59 + 5.24$ = 9.83 K/SQ FT

FIGURE C-9 MIDDLE WALL - INTERMEDIATE GATE MONOLITH M-12 (CONCLUDED)

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - LOWER CHAMBER AT CULVERTS MONOLITH M-22	CHECKED BY:	DATE:

STATION 2+05.9 B TO 2+36.0 B = 30'

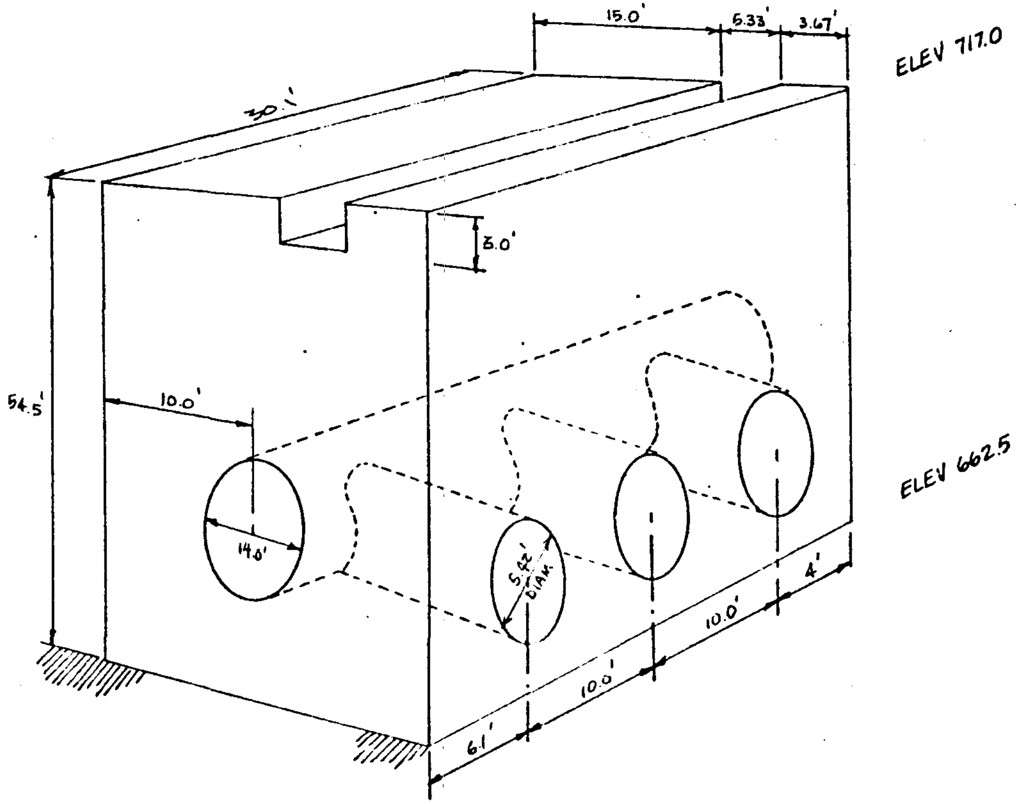
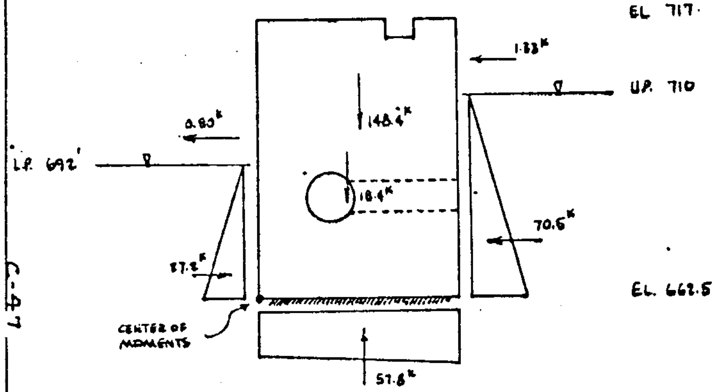


FIGURE C-10 MIDDLE WALL - LOWER CHAMBER AT CULVERTS MONOLITH M-22

SUBJECT: MIDDLE WALL - LOWER CHAMBER MONOLITH AT CULVERTS M-22

COMPUTED BY:
DATE:
CHECKED BY:
DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$	$[.1488][(54.5)(24)(30.1) - (5.33)(3)(3\ \ ) - (\pi)(7)^2(30.1) - (3)(\pi)(2.71)^2(7) - (13)(12)^2(2)]/30.1$	148.4		12.45	1849
$W_{WATER\ IN\ CULVERTS}$	$[.0625][(\pi)(7)^2(30.1) + (\pi)(2.71)^2(7)(2) + (13)(12)^2(2)]/30.1$	18.4		10.16	187
P_{WATER} (110' LOCK)	$(1/2)(710-662.5)^2(.0625)$		- 70.5	15.83	-1116
P_{WATER} (56' LOCK)	$(1/2)(692-662.5)^2(.0625)$		27.2	9.83	267
UPLIFT	$[.0625][(29.5)(24) + (1/2)(18)(24)]$	-57.8		12.94	- 748
IMPACT	1.33 K/FT		- 1.33	52.50	- 70
HAWSER	0.80 K/FT		- 0.8	34.50	- 28
		109.0	-45.4		340

$$e = \frac{340}{109.0} = 3.12'$$

$$\%\ \text{Effective Base} = \frac{9.36}{24} \times 100 = 39\%$$

CASE I NORMAL OPERATIONS

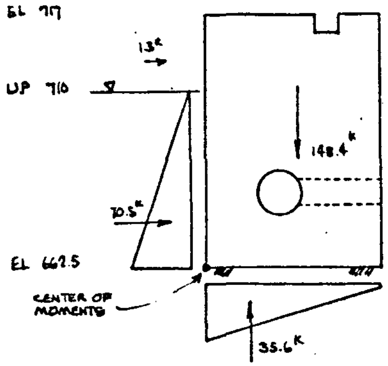
LOWER POOL IN 56' LOCK
UPPER POOL IN 110' LOCK

FIGURE C-10 MIDDLE WALL - LOWER CHAMBER MONOLITH AT CULVERTS M-22

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - LOWER CHAMBER MONOLITH AT CULVERTS M-22	CHECKED BY:	DATE:

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		148.4		12.45	1848
P_{WATER} (SL'L.)			70.5	15.83	1116
UPLIFT	(1/2) (710-662.5)(.0625)(24)	-35.6		8.00	- 285
IMPACT	1.3 K/FT		1.3	52.5	68
		112.8	71.8		2747



$$e = \frac{2747}{112.8} = 24.35$$

0% Effective Base

CASE II MAINTENANCE CONDITION

DEWATERED 110' LOCK

FIGURE C-10 MIDDLE WALL - LOWER CHAMBER MONOLITH AT CULVERTS M-22 (CONTINUED)

SUBJECT: MIDDLE WALL - LOWER CHAMBER MONOLITH AT CULVERTS M-22	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

SLIDING

CASE I NORMAL OPERATIONS

$R = \Sigma V \tan\phi$ + KEY RESISTANCE

$= (109.0)(.5890) + 0$

$R = 64.2$

$SSF = \frac{64.2}{45.4} = 1.4$

CASE II MAINTENANCE CONDITION

$R = (112.8)(.5890) + 0$

$R = 66.4$

$SSF = \frac{66.4}{71.8} = 0.92$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$f = \frac{2}{3}\frac{P}{e}$

$= \frac{(2)(109.0)}{(3)(3.12)} = 23.3$ K/SQ FT.

CASE II MAINTENANCE CONDITION

BASE PRESSURES ARE VERY LARGE

FIGURE C-10 MIDDLE WALL - LOWER CHAMBER MONOLITH AT CULVERTS M-22 (CONCLUDED)

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - LOWER GATE MONOLITH M-25	CHECKED BY:	DATE:

STATION 3+20.0 B TO 2+94.0 B = 26

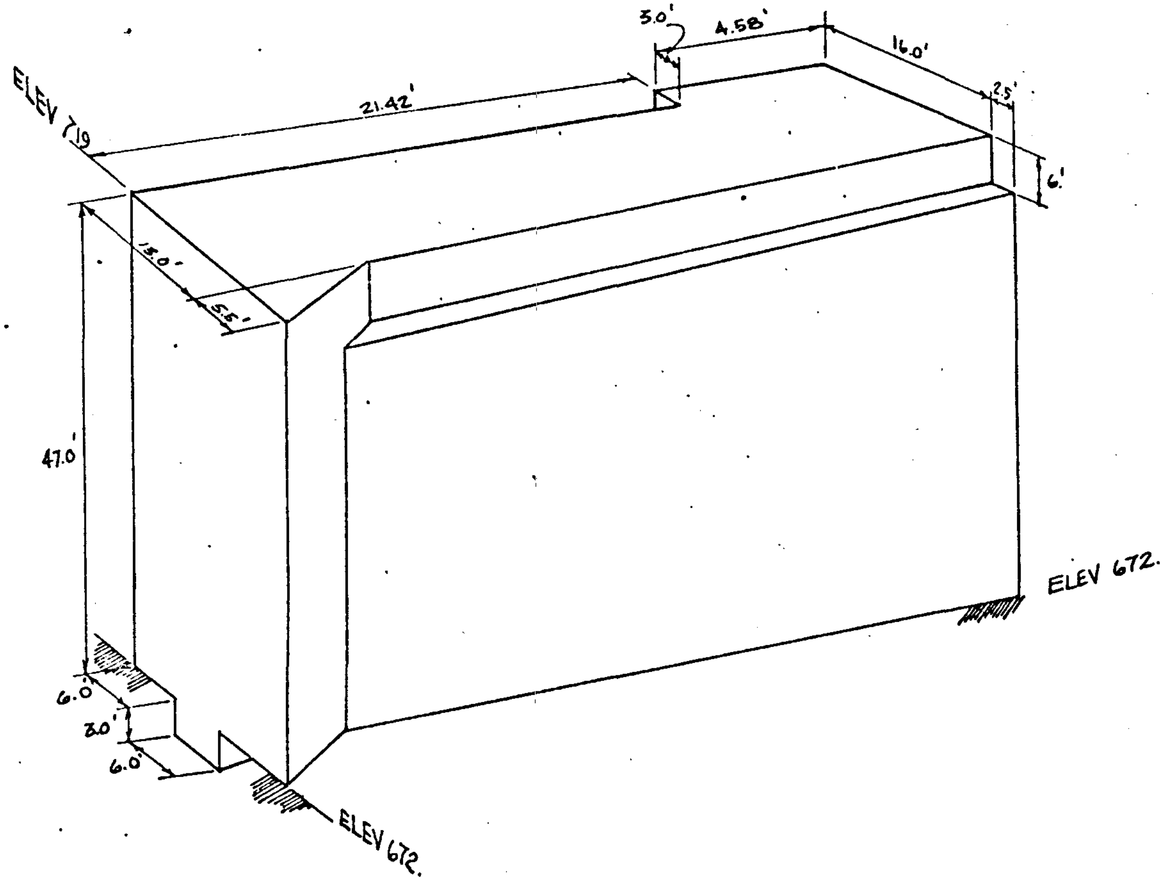


FIGURE C-11 MIDDLE WALL - LOWER GATE MONOLITH M-25

SUBJECT: MIDDLE WALL – LOWER GATE MONOLITH M-25

COMPUTED BY:

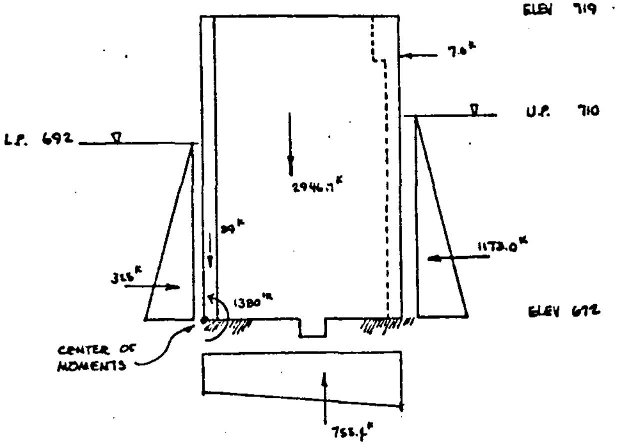
DATE:

CHECKED BY:

DATE:

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W CONC.	[.1488][(4.58)(16)(47) + (21.42)(13)(47) + (3)(4)(26) + (18.5)(2.5)(41) + (½)(7.5)(5.5)(47) + (½)(2.5)(1.83)(41)]	2946.7		10.46	30843
P WATER (56' LOCK)	$(½)(692 - 672)^2(.0625)(26)$		325.0	6.67	2168
P WATER (110' LOCK)	$(½)(710 - 672)^2(.0625)(26)$		-1173.0	12.67	-14862
UPLIFT	[.0625][(692-672)(15.5)(21.42) + (710-692)(½)(15.5)(21.42) + (692-672)(18.5)(4.58) + (710-692)(½)(18.5)(4.58)]	-765.4		11.28	-8519
GATE V		89.0		1.50	134
GATE M					-1380
WIND			-7.0	42.50	-298
		2280.3	-855.0		8086



$$e = \frac{8086}{2280.3} = 3.55'$$

$$\%\ \text{Effective Base} \approx \frac{(10.65)(4.58) + (7.65)(21.42)}{(21.42)(15.5) + (4.58)(18.5) + \frac{(3)(3)}{2}} \times 100$$

$$\approx \frac{212.64}{421.24} \times 100 \approx 50\%$$

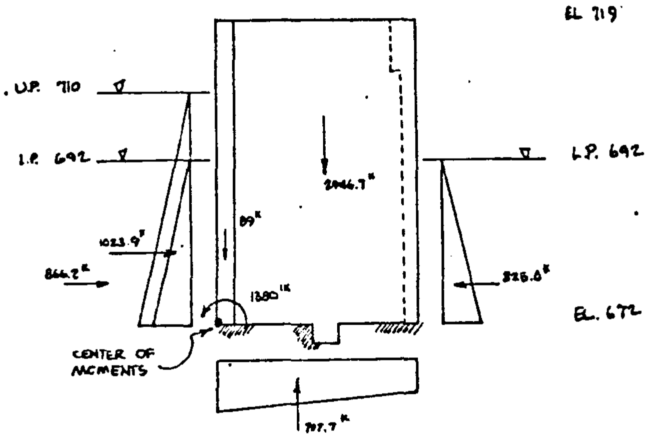
CASE I NORMAL OPERATION

LOWER POOL, 692., BOTH SIDES OF 56' LOCK GATE
UPPER POOL, 710, IN 110' LOCK

FIGURE C-11 MIDDLE WALL – LOWER GATE MONOLITH M-25 (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL LOWER GATE MONOLITH M-25	CHECKED BY:	DATE:

NOT TO SCALE



CASE II NORMAL OPERATION

UPPER POOL IN 56' LOCK
LOWER POOL BELOW 56' LOCK
LOWER POOL IN 110' LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W CONC		2946.7		10.46	30843
P_{WATER} (56' LOCK)	$[.0625][(1/2)(710-672)^2(21.42)+(692-672)^2(1/2)(4.58)]$		1023.9	12.33	12629
P_{WATER} (110' LOCK)	$[.0625][(1/2)(692-672)^2(26)]$		-325.0	6.67	-2168
UPLIFT	$[.0625][(692-672)(15.6)(21.42)+(710-692)(1/2)(15.5)(21.42)+(692-672)(18.5)(4.58)]$	-707.7		9.84	-6967
GATE S			866.2	8.61	7458
GATE V		89.0		1.50	134
GATE M					-1380
		2328.0	1565.1		40549

$$e = \frac{40549}{2328.0} = 17.41'$$

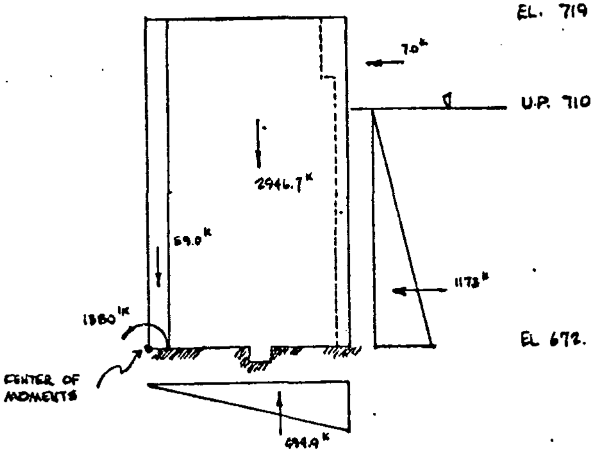
$$\% \text{ Effective Base} \approx \frac{(3.27)(26)+4.5}{421.24} \times 100$$

$$\approx 21\%$$

FIGURE C-11 MIDDLE WALL LOWER GATE MONOLITH M-25 (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - LOWER GATE MONOLITH M-25	CHECKED BY:	DATE:

NOT TO SCALE



CASE I MAINTENANCE CONDITION

UPPER POOL IN 110' LOCK
56' LOCK DEWATERED

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		2946.7		10.46	30843
P_{WATER} (110' LOCK)			-1173	12.67	-14862
UPLIFT	$[.0625][(1/2)(710-672)(15.5)(21.42)$ $+(1/2)(710-672)(18.5)(4.58)$	-494.9		13.13	6496
WIND			-7.0	42.60	-298
GATE V		89.0		1.50	134
GATE M					-1380
		1540.8	-1180.0		7941

$$e = \frac{7941}{1540.8} = 3.15'$$

$$\% \text{ Effective Base} \approx \frac{(9.39)(4.58) + (6.39)(21.42)}{421.24} \times 100$$

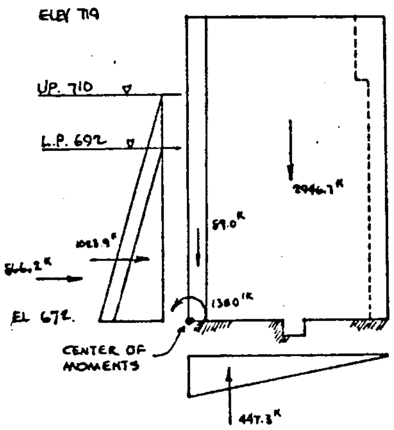
$$\approx 43\%$$

FIGURE C-11 MIDDLE WALL LOWER GATE MONOLITH M-25 (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - LOWER GATE MONOLITH M-25	CHECKED BY:	DATE:

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
N_{CONC}		2946.7		10.46	30843
P_{WATER}(SL'L)			1023.9	12.33	12629
UPLIFT	$[.0625][(1/2)(710-672)(15.5)(21.42) + (1/2)(692-672)(18.5)(4.58)]$	-447.3		7.93	-3548
GATE S			866.2	8.61	7458
GATE V		89.0		1.50	134
GATE M					-1380
		2588.4	1890.1		46136



$$e = \frac{46136}{2588.4} = 17.82'$$

$$\%\ \text{Effective Base} \approx \frac{(2.04)(26) + 4.5}{421.24} \times 100$$

$$\approx 14\%$$

CASE II MAINTENANCE CONDITION

110' LOCK DEWATERED
UPPER POOL IN 56' LOCK
LOWER POOL BELOW 56' LOCK.

FIGURE C-11 MIDDLE WALL - LOWER GATE MONOLITH M-25 (CONTINUED)

SUBJECT: MIDDLE WALL – LOWER GATE MONOLITH M-25

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \sum V \tan\phi + \text{KEY RESISTANCE}$$

$$= (2280.3)(.5890) + (6)(26)(.075)(144)$$

$$= 1343 + 1685$$

$$R = 3028$$

$$SSF = \frac{3028}{855} = 3.5$$

CASE I MAINTENANCE CONDITION

$$R = (2540.8)(.5890) + 1685$$

$$= 1496 + 1685$$

$$R = 3181$$

$$SSF = \frac{3181}{1180} = 2.7$$

CASE II NORMAL OPERATIONS

$$R = (2328.0)(.5890) + 1685$$

$$= 1371 + 1685$$

$$R = 3056$$

$$SSF = \frac{3056}{1565.1} = 2.0$$

CASE II MAINTENANCE CONDITION

$$R = (2588.4)(.5890) + 1685$$

$$= 1525 + 1685$$

$$R = 3210$$

$$SSF = \frac{3210}{1890.1} = 1.7$$

BASE PRESSURES

CASE I NORMAL OPERATIONS

$$f = \frac{2}{3}\,\frac{P}{e}$$

$$= \left[\frac{(2)(2280.3)}{(3)(3.55)}\right]\left[\frac{1}{26}\right]$$

$$f = 16.5 \text{ K/SQ FT.}$$

CASE I MAINTENANZE CONDITION

$$f = \left[\frac{(2)(2540.8)}{(3)(3.13)}\right]\left[\frac{1}{26}\right]$$

$$f = 20.8 \text{ K/SQ. FT.}$$

CASE II NORMAL OPERATIONS

$$f = \left[\frac{(2)(2328.0)}{(3)(1.09)}\right]\left[\frac{1}{26}\right]$$

$$f = 54.8 \text{ K/SQ. FT.}$$

CASE II MAINTENANZE CONDITION

$$f = \left[\frac{(2)(2588.1)}{(3)(0.68)}\right]\left[\frac{1}{26}\right]$$

$$f = 97.6 \text{ K/SQ. FT.}$$

FIGURE C-11 MIDDLE WALL – LOWER GATE MONOLITH M-25 (CONCLUDED

SUBJECT: RIVER WALL UPPER GUARD WALL R-4	COMPUTED BY: CHECKED BY:	DATE: DATE:

STATION 4+63.88 A TO 4+24.05 A = 39.83

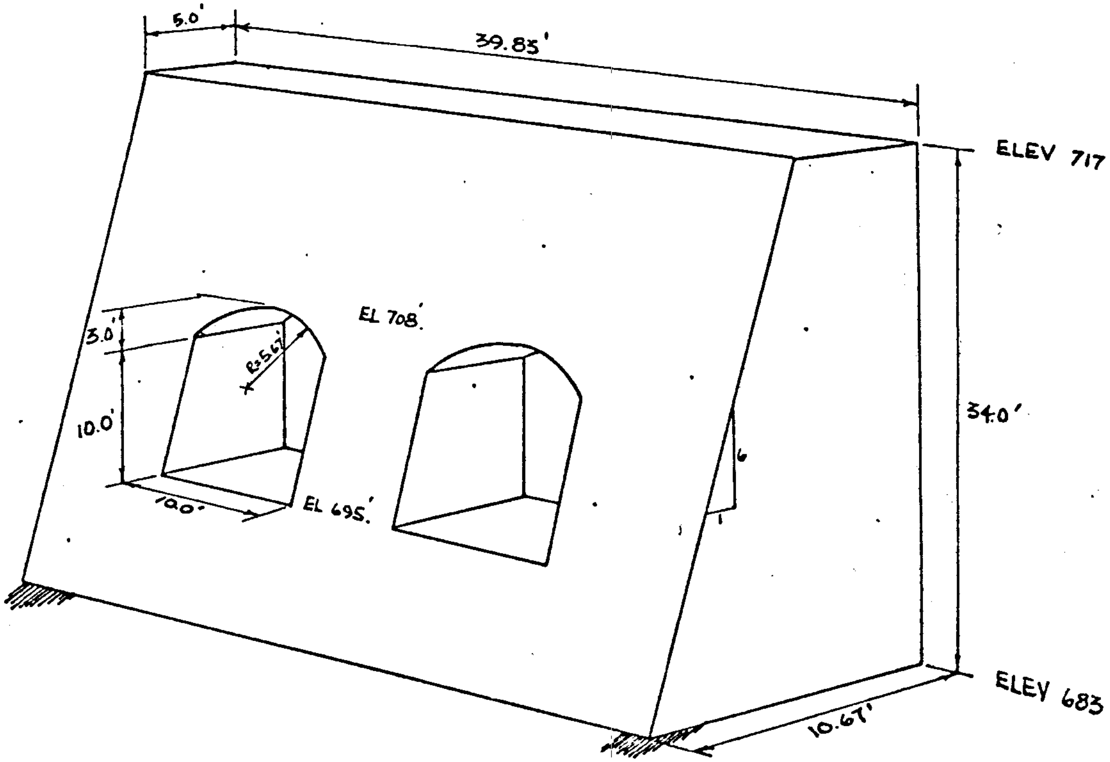
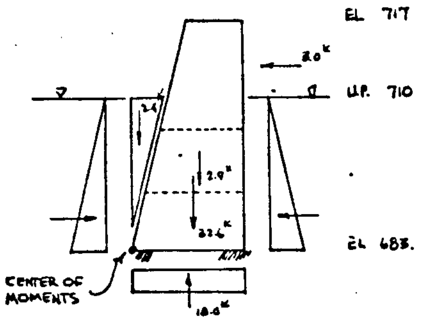


FIGURE C-12 RIVER WALL - UPPER GUARD WALL R-4

SUBJECT: RIVER WALL - UPPER GUARD WALL R-4	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

NOT TO SCALE



NORMAL OPERATION

UPPER POOL 710

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$	$[.1488][(5)(34)(39.83) + (1/2)(567)(34)(39.83) - (\frac{(3.1416)(5.67)^2}{2} + (7.33)(10))(6.5)(2) - (\frac{(3.1416)(5.67)^2}{2} + (7.33)(10))(1/2)(2.17)(2)] / 39.83$	32.6		6.56	213
$W_{WATER\ IN\ CULVERTS}$	$[.0625][2][[\frac{3.1416(5.67)^2}{2} + 7.33(10)](6.5) + [\frac{3.1416(5.67)^2}{2} + 7.33(10)](1/2)(2.17)] / 39.83$	2.9		6.90	20
$W_{WATER\ ON\ RIVER\ FACE}$	$(1/2)(710-683)(2.83)(.0625)$	2.4		0.94	2
UPLIFT	$(710-683)(10.67)(.0625)$	-18.0		5.34	- 96
IMPACT			- 3	32.00	- 96
P_{WATER}	UPPER POOL BOTH SIDES OF MONOLITH FORCES CANCEL EACH OTHER				
		19.9	-3		43

$$e = \frac{43}{19.9} = 2.16'$$

$$\%\ \text{Effective Base} = \frac{6.48}{10.67} \times 100 = 61\%$$

FIGURE C-12 RIVER WALL - UPPER GUARD WALL R-4 (CONTINUED)

SUBJECT: RIVER WALL UPPER GUARD WALL R-4	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

SLIDING

$R = \sum V \tan \phi$ + KEY RESISTANCE

$= (19.9)(.5890) + 0$

$= 11.72$

$S_{SF} = \frac{11.72}{3} = 3.9$

BASE PRESSURE

$f = \frac{2}{3} \frac{P}{e}$

$= \frac{(2)}{(3)} \frac{(19.9)}{(2.16)}$

$= 6.14\ ^{K}/\text{SQ.FT.}$

FIGURE C-12 RIVER WALL UPPER GUARD WALL R-4 (CONCLUDED)

WES FORM NO. 1253A
DECEMBER 1964

SUBJECT:	COMPUTED BY:	DATE:
River Wall - Upper Gate Mono R-14	CHECKED BY:	DATE:

STATION 0+67.2A TO 1+05.2A = 39

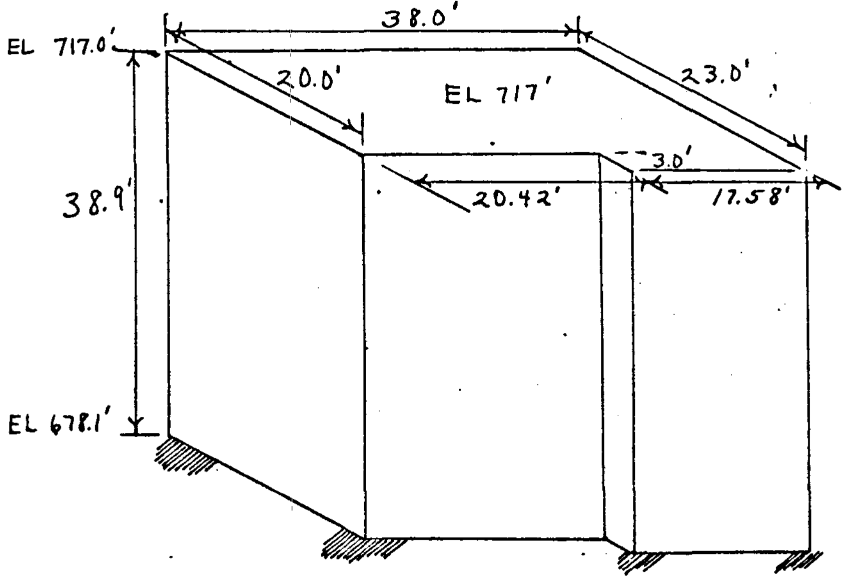
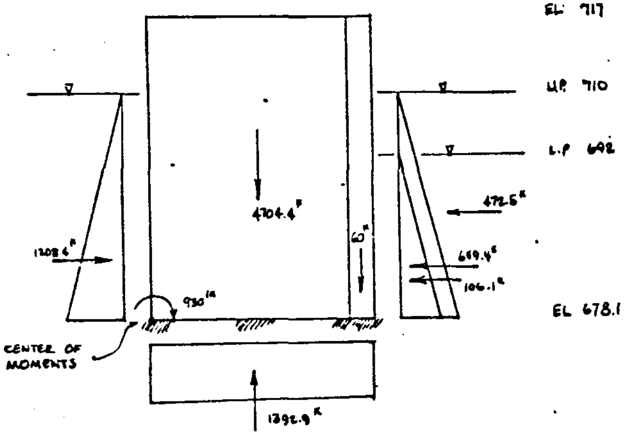


FIGURE C-13 RIVER WALL - UPPER GATE MONOLITH R-14

SUBJECT: RIVER WALL - UPPER GATE MONOLITH R-14	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

C-90

NOT TO SCALE



CASE I NORMAL OPERATIONS

UPPER POOL ABOVE GATE
LOWER POOL IN 56' LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$	$[.1488][(38)(20)(38.9)+(3)(17.58)(38.9)]$	4704.4		10.75	50,572
P_{WATER} (RIVER)	$(1/2)(710.-678.1)^2(.0625)(38)$		1208.4	10.63	12845
P_{WATER} (LOCK)	$(1/2)(710-678.1)^2(.0625)(20.42)$		-649.4	10.63	-6903
P_{WATER} (LOCK)	$(1/2)(692-678.1)^2(.0625)(17.58)$		-106.1	4.63	-491
UPLIFT	$[.0625][(710-678.1)(20)(20.42)+(692-678.1)(23)(17.58)+(1/2)(710-692)(23)(17.58)]$	-1392.9		16.11	-22445
GATE S			-472.5	18.03	-8519
GATE V		60.0		21.5	1290
GATE M					930
		3371.5	-19.6		35798

$$e = \frac{35798}{3371.5} = 10.62'$$

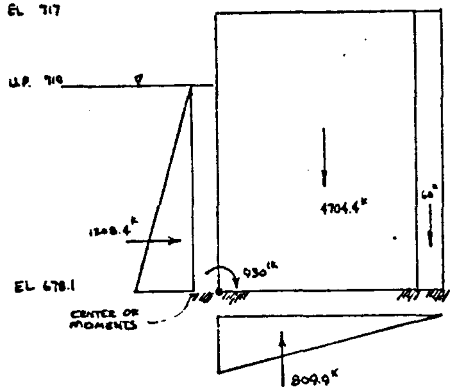
100% Effective Base

FIGURE C-13 RIVER WALL UPPER GATE MONOLITH R-14 (CONTINUED)

SUBJECT: RIVER WALL - UPPER GATE MONOLITH R-14	COMPUTED BY:	DATE:	
	CHECKED BY:	DATE:	

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$		4704.4		10.75	50572
$P_{WATER\,(RNGR)}$			1208.4	10.63	12845
UPLIFT	$[.0625][(1/2)(710-678.1)(20)(20.42)$ $+(1/2)(710-678.1)(23)(17.58)]$	-809.9		7.33	5937
GATE V		60		21.5	1290
GATE M					930
		3954.5	1208.4		59700



$$e = \frac{59700}{3954.5} = 15.1'$$

100% Effective Base

CASE II MAINTENANCE CONDITION

DEWATERED 56' LOCK

FIGURE C-13 RIVER WALL - UPPER GATE MONOLITH R-14 (CONTINUED)

SUBJECT: RIVER WALL - UPPER GATE MONOLITH R-14

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \Sigma V \tan\phi + \text{KEY RESISTANCE}$$

$$= (3371.5)(.5890) + 0$$

$$R = 1986$$

$$S_{SF} = \frac{1986}{19.6} = 101.3$$

CASE II MAINTENANCE CONDITION

$$R = (3954.5)(.5890) + 0$$

$$= 2329$$

$$S_{SF} = \frac{2329}{1208.4} = 1.93$$

BASE PRESSURE

CASE I NORMAL OPERATION

$$f = \frac{P}{A} \pm \frac{Mc}{I}$$

$$= \frac{3371.5}{812.7} + \frac{(3371.5)(.13)(10.75)}{31895}$$

$$= 4.15 + 0.15$$

$$= 4.30 \text{ K/SQ.FT.}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{P}{A} \pm \frac{Mc}{I}$$

$$= \frac{3954.5}{812.7} + \frac{(3954.5)(4.35)(12.25)}{31895}$$

$$= 4.87 + 6.60$$

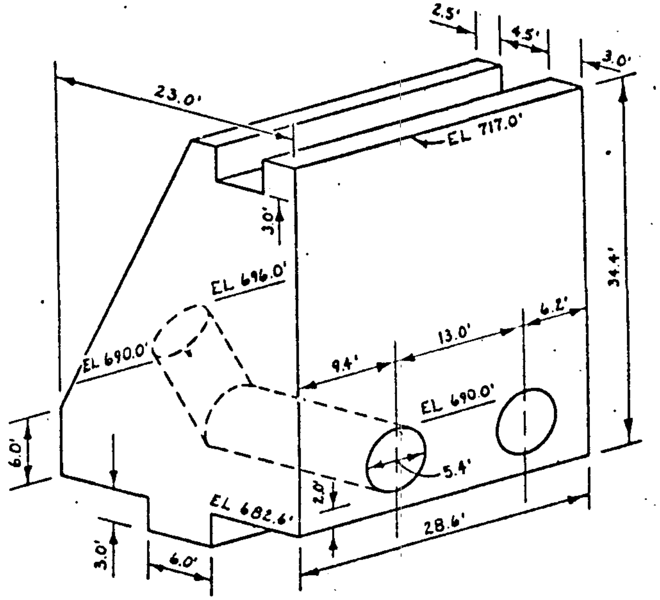
$$= 11.47 \text{ K/SQ.FT.}$$

FIGURE C-13 RIVER WALL- UPPER GATE MONOLITH R-14 (CONCLUDED)

SUBJECT: RIVER WALL - UPPER CHAMBER MONOLITH R-17	COMPUTED BY: CHECKED BY:	DATE: DATE:

STATION 0+05.4A TO 0+23.2 B = 28.6

C-63



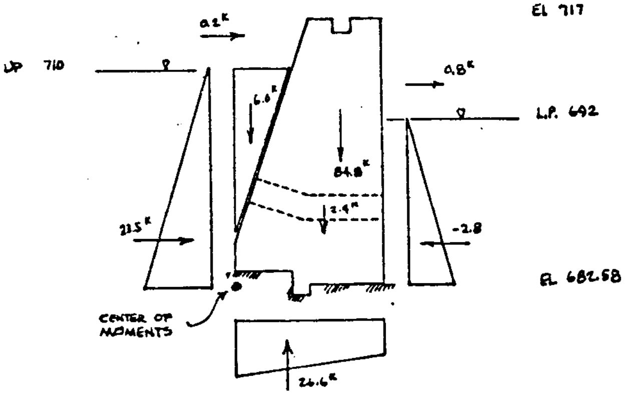
NOTE: CULVERT SHOWN IS TYPICAL

FIGURE C-14 RIVER WALL - UPPER CHAMBER MONOLITH R-17

SUBJECT: RIVER WALL UPPER CHAMBER MONOLITH R-17	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

NOT TO SCALE

C-64



CASE I NORMAL OPERATIONS

LOWER POOL IN 56' LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$	$[.1488][(23)(33)(28.6) - (1/2)(13)(27)(28.6) - (3)(4.5)(28.6) + (1.42)(14.5)(28.6) + (3)(6)(28.6) - (\pi)(2.71)^2(24.1)(2)]/28.6$	84.8		13.66	1159
$W_{WATER\ RIVER}$	$(1/2)(9.63)(710-690)(.0625)$	6.0		3.21	19
$W_{WATER\ IN\ CULVERTS}$	$(\pi)(2.71)^2(24.1)(2)(.0625)/28.6$	2.4		12.15	29
$P_{WATER-RIVER}$	$(1/2)(710-682.58)^2(.0625)$		23.5	9.14	215
$P_{WATER-LOCK}$	$(1/2)(692-682.58)^2(.0625)$		-2.8	3.14	-9
UPLIFT	$[.0625][(22.77)(8.5) + (1/2)(3.23)(8.5) + (10.22)(6) + (1/2)(1.75)(6) + (9.41)(8.5) + (1/2)(5.03)(8.5)]$	-26.6		9.55	-254
HAWSER			0.8	14.42	12
WIND	$(717-710)(1)(.03)$		0.2	30.92	6
		66.6	21.7		1177

$$e = \frac{1177}{66.6} = 17.67'$$

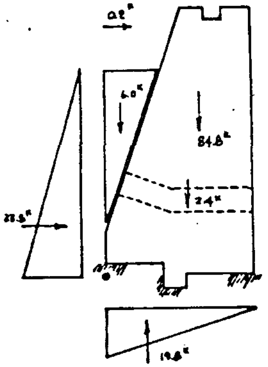
$$\%\ Effective\ Base = \frac{15.99}{23} \times 100 = 70\%$$

FIGURE C-14 RIVER WALL - UPPER CHAMBER MONOLITH R-17 (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
RIVER WALL UPPER CHAMBER MONOLITH R-17		
	CHECKED BY:	DATE:

ITEM	FACTORS	Fv	FH	ARM	MOMENT
Wconc.		84.8		13.66	1159
Wwater, river		6.0		3.21	19
Wwater in culvert		2.4		12.15	29
Pwater, river			23.5	9.14	215
Uplift	[.0625][(18.34)(8.5) + (½)(7.66)(8.5) + (13.37)(6) + (½)(54)(6) + (½)(7.6)(8.5)]	−19.8		7.62	−151
Wind			0.2	30.92	6
		73.4	23.7		1277

NOT TO SCALE



CASE II MAINTENANCE CONDITION

56' LOCK DEWATERED

$$e = \frac{1277}{73.4} = 17.4'$$

$$\% \text{ Effective Base} = \frac{16.8}{23} \times 100 = 73\%$$

FIGURE C-14 RIVER WALL - UPPER CHAMBER MONOLITH R-17 (CONTINUED)

SUBJECT: RIVER WALL - UPPER CHAMBER MONOLITH R-17

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

CASE I NORMAL OPERATIONS

$R = \Sigma V \tan\phi$ + KEY RESISTANCE

$= (66.6)(.5890) + (6)(.075)(144)$

$= 39.23 + 64.8$

$R = 104.03$

$S_{SF} = \frac{104.03}{21.75} = 4.8$

CASE II MAINTENANCE CONDITION

$R = (73.4)(.5890) + 64.8$

$= 43.23 + 64.8$

$R = 108.03$

$S_{SF} = \frac{108.03}{23.7} = 4.6$

BASE PRESSURES

CASE I NORMAL OPERATION

$f = \frac{2}{3}\frac{P}{e}$

$= \left(\frac{2}{3}\right)\left(\frac{66.6}{5.33}\right)$

$= 8.33$ K/SQ. FT.

CASE II MAINTENANCE CONDITION

$f = \frac{2}{3}\frac{P}{e}$

$= \left(\frac{2}{3}\right)\left(\frac{73.4}{5.6}\right)$

$= 8.7$

FIGURE C-14 RIVER WALL UPPER CHAMBER MONOLITH R-17 (CONCLUDED)

SUBJECT: RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-24	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

STATION 2+31.0 B TO 1+97.3 B

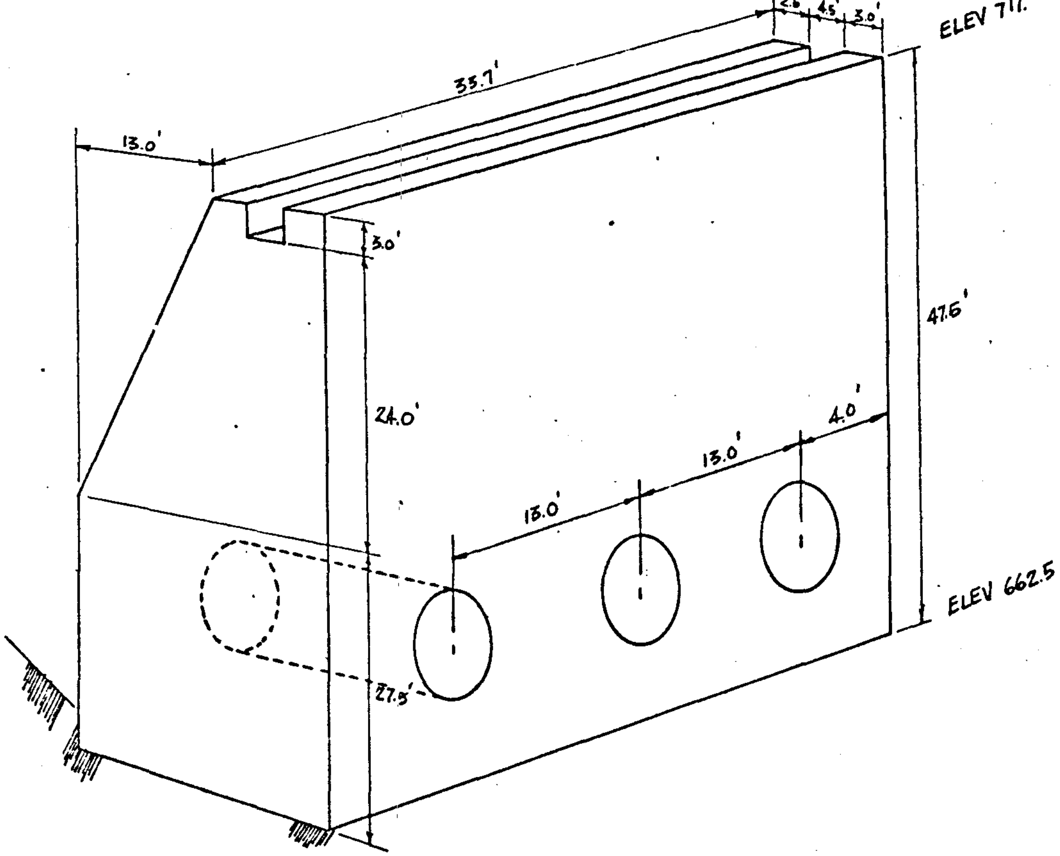
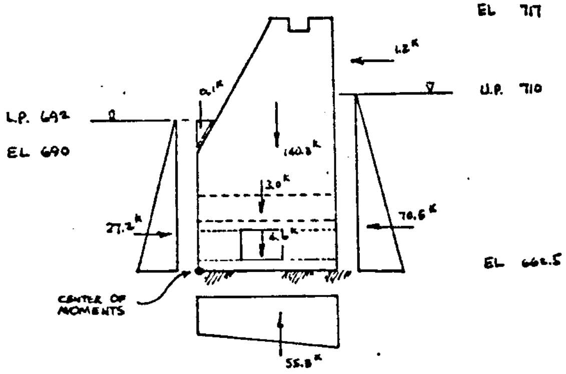


FIGURE C-15 RIVER WALL- TYPICAL LOWER CHAMBER MONOLITH R-24

WES FORM NO. 1253A
DECEMBER 1964

SUBJECT:	COMPUTED BY:	DATE:
RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-24	CHECKED BY:	DATE:

NOT TO SCALE



CASE I NORMAL OPERATION

UPPER POOL IN 56' LOCK
LOWER POOL IN RIVER

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[.1488][(23)(54.5)(33.7) - (1/2)(13)(27)(33.7) - (3)(4.5)(33.7) - (\pi)(2.71)^2(23)(3) - (8)(12)(26)]/33.7$	140.3		12.75	1789
$W_{WATER\ IN\ CULVERTS}$	$[.0625][(\pi)(2.71)^2(23)(3)]/33.7$ $[.0625][(8)(12)(26)]/33.7$	3.0 4.6		11.5 11.5	35 53
$P_{WATER,\ RIVER}$	$(1/2)(692. - 662.5)^2(.0625)$		27.2	9.83	267
$P_{WATER,\ LOCK}$	$(1/2)(710 - 662.5)^2(.0625)$		-70.5	15.83	-1116
$W_{WATER,\ RIVER}$	$(1/2)(.96)(2)(.0625)$	0.1		0.32	0
UPLIFT	$[.0625][(692 - 662.5)(23) + (1/2)(710 - 692)(23)]$	-55.3		12.40	-686
IMPACT			-1.2	52.50	-63
		92.7	-44.5		279

$$e = \frac{279}{92.7} = 3.0$$

$$\%\ \text{Effective Base} = \frac{9}{23} \times 100 = 39\%$$

FIGURE C-15 RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-24 (CONTINUED)

SUBJECT: RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-24

COMPUTED BY:

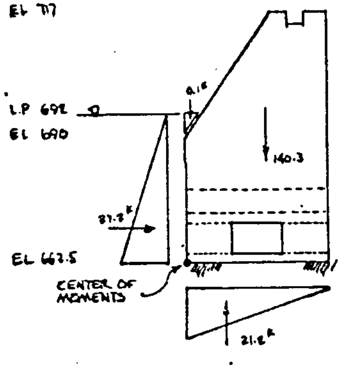
DATE:

CHECKED BY:

DATE:

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		140.3		12.74	1789
$P_{WATER, RIVER}$			27.2	9.83	267
$W_{WATER, RIVER}$		0.1		0.32	0
U_{PLIFT}	$(1/2)(692-662.5)(23)(.0625)$	-21.2		7.67	- 163
		119.2	27.2		1893



$$e = \frac{1893}{119.2} = 15.88'$$

$$\% \text{ Effective Base} = \frac{21.36}{23} \times 100 = 93\%$$

CASE II MAINTENANCE CONDITION

LOWER POOL IN RIVER
DEWATERED LOCK

FIGURE C-15 RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-24

SUBJECT: RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-24

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

CASE I NORMAL OPERATIONS

$R = \sum V \tan \phi$ + KEY RESISTANCE

$= (92.7)(.5890) + 0$

$R = 54.6$

$SSF = \frac{54.6}{44.5} = 1.2$

CASE II MAINTENANCE CONDITION

$R = (119.2)(.\ 890) + 0$

$R = 70.2$

$SSF = \frac{70.2}{27.2} = 2.6$

BASE PRESSURE

CASE I NORMAL OPERATION

$f = \frac{2}{3} \frac{P}{e}$

$= \left(\frac{2}{3}\right)\left(\frac{92.7}{3.00}\right)$

$= 20.6$ K/SQ. FT.

CASE II MAINTENANCE CONDITION

$f = \left(\frac{2}{3}\right)\left(\frac{119.2}{7.12}\right)$

$= 11.2$ K/SQ. FT.

FIGURE C-15 RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-24

SUBJECT: RIVER WALL - LOWER GATE MONOLITH R-27	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

STATION 2+95.2 B TO 3+38.11 B = 37.8

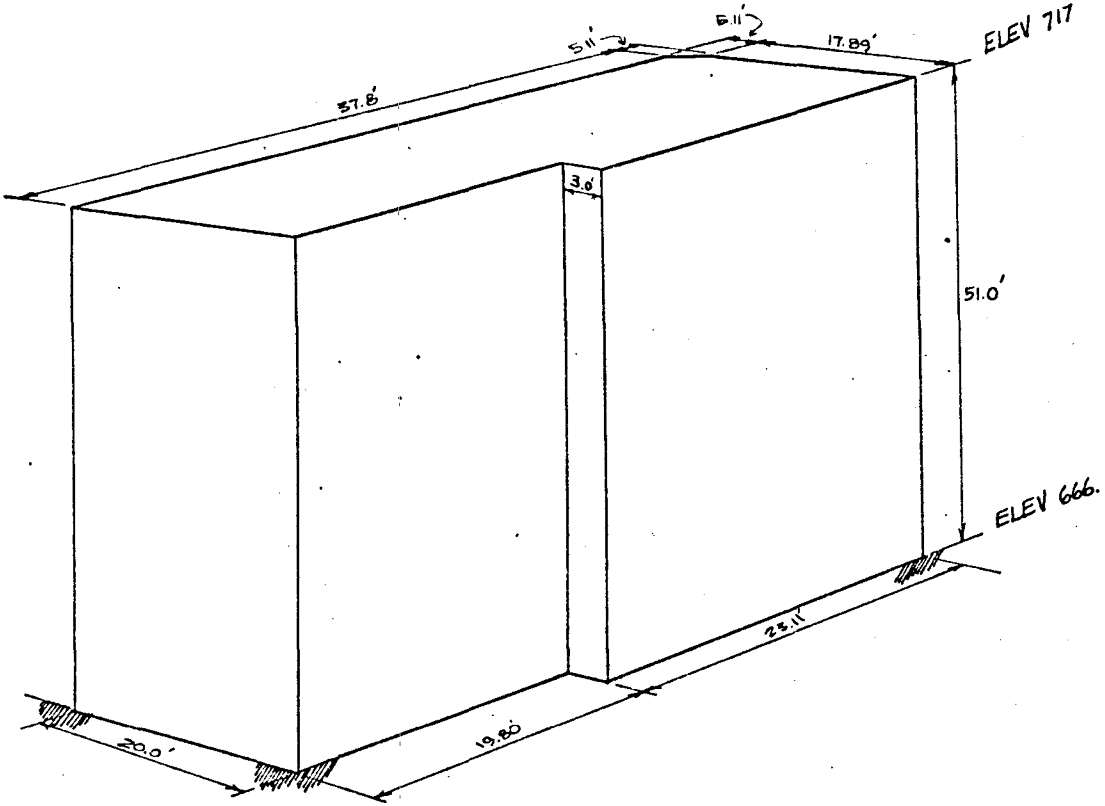
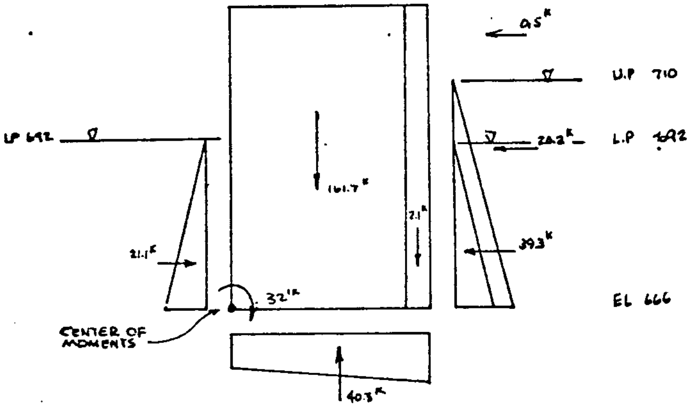


FIGURE C-16 RIVER WALL - LOWER GATE MONOLITH R-27

SUBJECT:	COMPUTED BY:	DATE:
RIVER WALL - LOWER GATE MONOLITH R-27	CHECKED BY:	DATE:

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1488][(20)(51)(19.8) + (23)(51)(23.11) - (½)(5.11)(5.11)(51)]/42.91	161.7		10.99	1777
$P_{WATER-RIVER}$	(½)(692 - 666)²(.0625)		21.1	8.67	183
$P_{WATER,\ LOCK}$	[(½)(710 - 666)²(.0625)(19.8) + (½)(692 - 666)²(.0625)(23.11)] ÷ [42.91]		-39.3	12.93	- 508
UPLIFT	[.0625][(692-666)(23)(23.11) + (692-666)(20)(19.8) + (½)(710-692)(20)(19.8)]/42.91	- 40.3		11.18	- 451
WIND	[.03][(7)(19.8) + (25)(23.11)]/42.91		- 0.5	40.25	- 20
GATE S	866.2/42.91 = 20.2ᴷ		-20.2	24.61	- 497
GATE Y	89/42.91 = 2.1ᴷ	2.1		21.50	45
GATE M	1380/42.91 = 32				32
		123.5	-38.9		561



CASE I NORMAL OPERATION

UPPER POOL IN 56' LOCK
LOWER POOL BELOW 56' LOCK

$$e = \frac{561}{123.5} = 4.54'$$

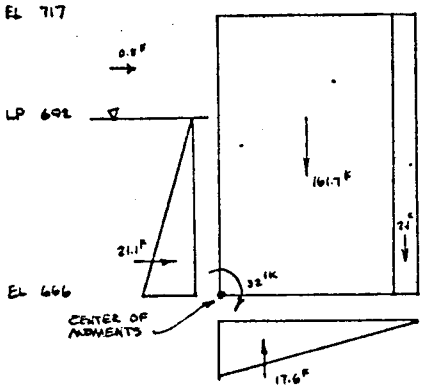
$$\%\ \text{Effective Base} = \frac{(13.62)(42.91) - (5.11)(5.11)(\tfrac{1}{2})}{(23)(23.11) - 13.06 + (20)(19.80)} \times 100$$

$$= 62\%$$

FIGURE C-16 RIVER WALL - LOWER GATE MONOLITH R-27 (CONTINUED)

SUBJECT: RIVER WALL - LOWER GATE MONOLITH R-27	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		161.7		10.99	1777
$P_{WATER\ RIVER}$	$(1/2)(692-666)^2(.0625)$		21.1	8.67	183
U_{PLIFT}	$(1/2)(692-666)(.0625)((20)(19.8)+(23)(23.11))/42.91$	-17.6		7.24	-127
GATE V	89/42.91 = 2.1	2.1		21.50	45
GATE M	1380/42.91 = 32				32
WIND	[.05](717-692)		0.8	38.50	31
		146.2	21.9		1941

$$e = \frac{1941}{146.2} = 13.28'$$

100% Effective Base

CASE I MAINTENANCE

LOWER POOL IN RIVER
LOCK DEWATERED

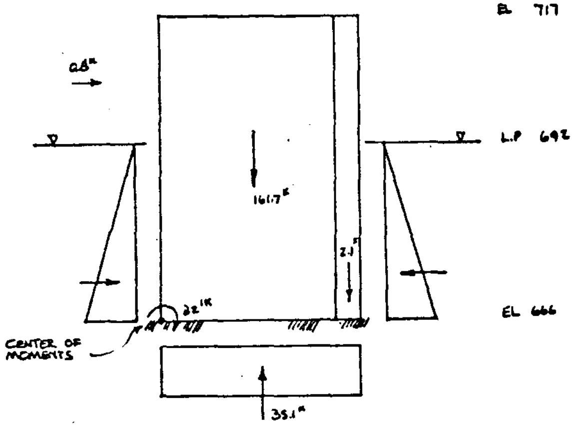
FIGURE C-16 RIVER WALL - LOWER GATE MONOLITH R-27 (CONTINUED)

WES FORM NO. 1283A
DECEMBER [illegible]

PAGE 3 OF 5

SUBJECT: RIVER WALL - LOWER GATE MONOLITH _ R-27	COMPUTED BY:	DATE:	
	CHECKED BY:	DATE:	

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		161.7		10.99	1777
P_{WATER}	$P_{WATER, RIVER} = -P_{WATER, LOCK}$ FORCES CANCEL				
UPLIFT	[.0625][692-666]((20)(19.8)+(23)(23.11))/42.91	-35.1		10.86	- 381
WIND	[.03][(717-692)]		0.8	38.50	31
GATE V	89/42.91 = 2.1	2.1		21.50	45
GATE M	1380/42.91 = 32				32
		128.7	0.8		1504



$$e = \frac{1504}{128.7} = 11.68'$$

100% Effective Base

CASE II NORMAL OPERATION

LOWER POOL IN 56' LOCK
LOWER POOL BELOW LOCK
LOWER POOL IN RIVER

FIGURE C-16 RIVER WALL LOWER GATE MONOLITH R-27 (CONTINUED)

SUBJECT: RIVER WALL - LOWER GATE MONOLITH R-27	COMPUTED BY: CHECKED BY:	DATE: DATE:

SLIDING

CASE I NORMAL OPERATIONS

$R = \Sigma V \tan\phi$ + KEY RESISTANCE

$= (123.5)(.5890) + 0$

$R = 72.74$

$SSF = \frac{72.74}{38.9} = 1.87$

CASE I MAINTENANCE CONDITION

$R = (146.2)(.5890) + 0$

$R = 86.11$

$SSF = \frac{86.11}{21.9} = 3.93$

CASE II NORMAL OPERATIONS

$R = (128.7)(.5890) + 0$

$R = 75.80$

$SSF = \frac{75.8}{0.8} = 94.75$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$f = \frac{2}{3}\frac{P}{e}$

$= \left(\frac{2}{3}\right)\left(\frac{123.5}{4.54}\right)$

$= 18.14$ K/SQ.FT.

CASE I MAINTENANCE CONDITION

$f = \frac{P}{A} \pm \frac{Mc}{I}$

$= \frac{146.2}{21.62} + \frac{(146.2)(13.28-10.85)(23-10.85)}{865.58}$

$= 6.76 + 4.99$

$= 11.75$ K/SQ. FT

CASE II NORMAL OPERATIONS

$f = \frac{128.7}{21.62} + \frac{(128.7)(11.68-10.85)(23-10.85)}{865.58}$

$= 5.95 + 1.50$

$= 7.45$ K/SQ.FT.

FIGURE C-16 RIVER WALL - LOWER GATE MONOLITH R-27 (CONCLUDED)

SUBJECT:	COMPUTED BY:	DATE:
River Wall - Lower Guard Wall R-32	CHECKED BY:	DATE:

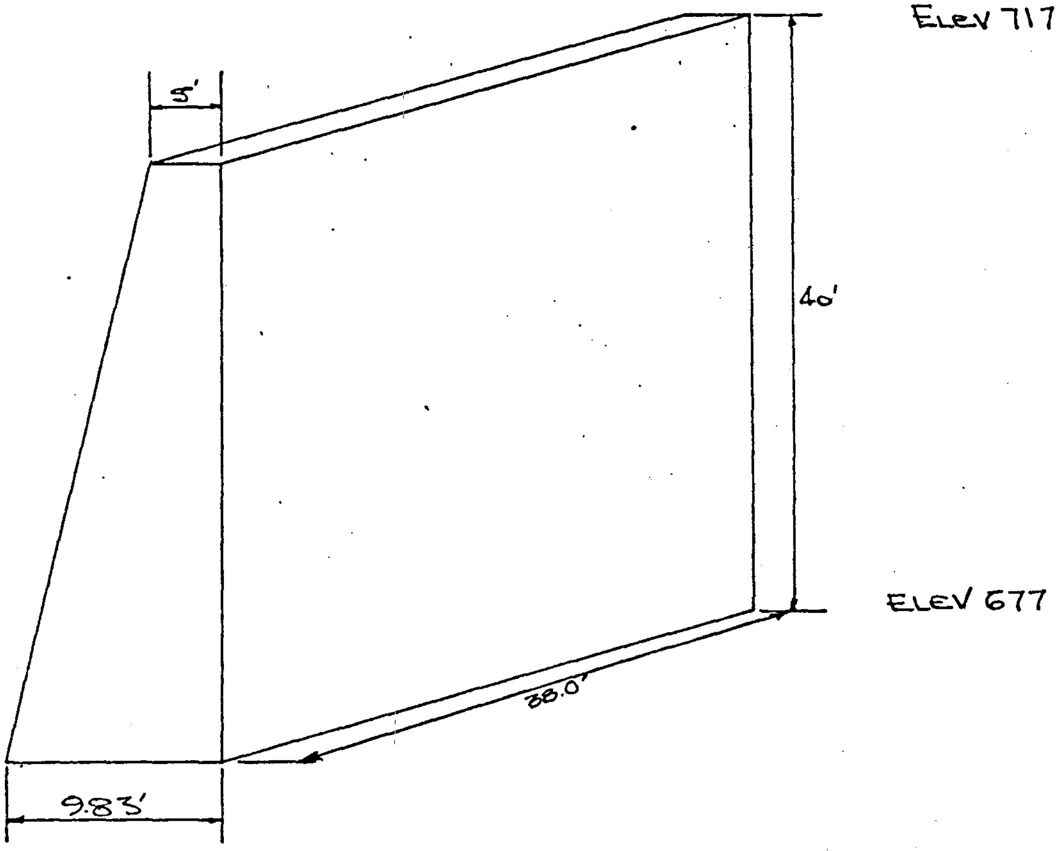


FIGURE C-17 RIVER WALL - LOWER GUARD WALL R-32

WES FORM NO. 1253A
DECEMBER 1964

SUBJECT: River Wall - Lower Guard Wall R-32

COMPUTED BY:

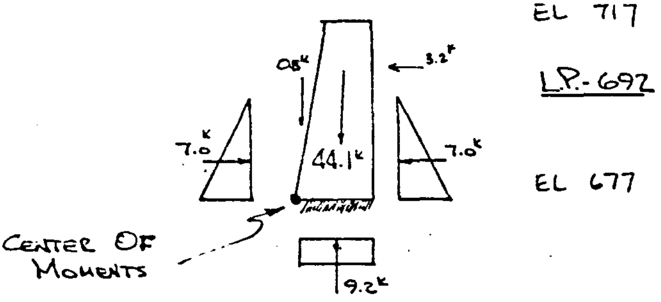
DATE:

CHECKED BY:

DATE:

Item	Factors	Fv	FH	Arm	Moment
Wconc	(717 - 677)(5)(.1488) 1/2 (4.83)(717 - 677)(.1488)	44.1		5.99	264
Pw (River)	1/2 $(692-677)^2$ (.0625)		7.0	5	35
Wwater (River)	1/2 (15)(1.81)(.0625)	0.8		0.60	0
Uplift	(9.83)(692 - 677)(.0625)	-9.2		4.92	-45.
Pw (Lock)	(1/2 $(692-677)^2$ (.0625)		-7.0	5	-35
Impact	3.2		-3.2	20	-64
		35.7	-3.2		155

NOT TO SCALE



Normal Operations
Lower Pool - 692

e = 155 / 35.7 = 4.34'

100% Effective Base

FIGURE C-17 RIVER WALL - LOWER GUARD WALL R-32

SUBJECT: River Wall - Lower Guard Wall R-32	COMPUTED BY: CHECKED BY:	DATE: DATE:

Sliding

$R = V \tan \phi$

$R = (35.7)(0.5890) = 21.0$

$S_{S\text{-}F} = 21.0 / 3.2 = 6.56$

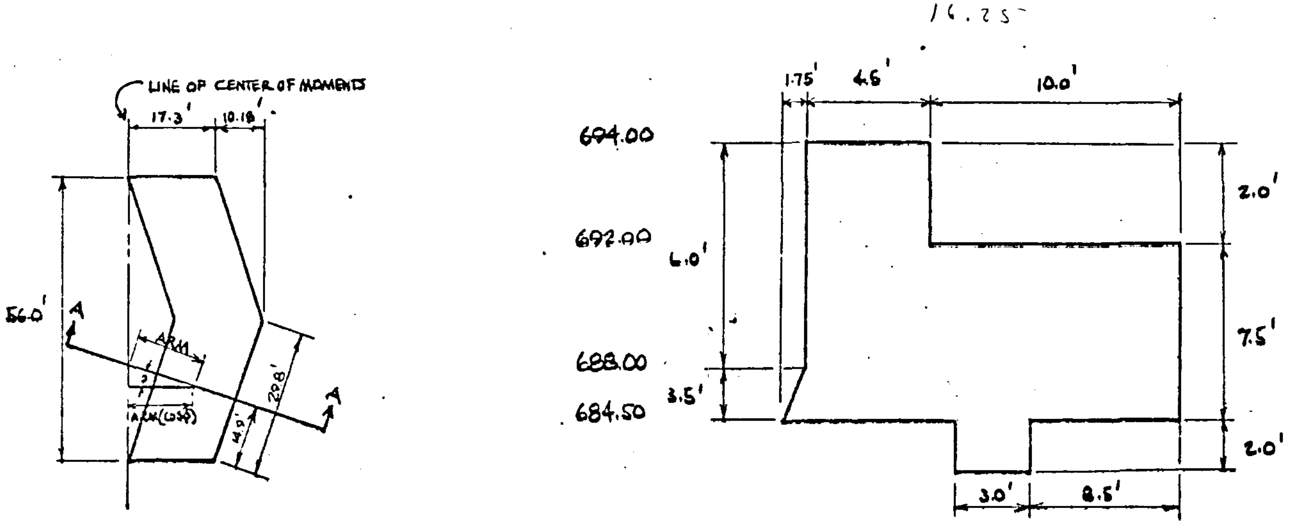
Base Pressure

$f = \frac{P}{A} \pm \frac{Mc}{I}$

$f = \frac{35.7}{9.83} + \frac{(35.7)(.58)(4.92)(12)}{(9.85)3} = 4.91 \text{ K/FT}^2$

FIGURE C-17 RIVER WALL - LOWER GUARD WALL R-32

SUBJECT: UPPER MITER SILL (56' LOCK)	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:



SECTION AA

FIGURE C-18 UPPER MITER SILL (56' LOCK)

SUBJECT: UPPER MITER SILL (56' LOCK)

COMPUTED BY:

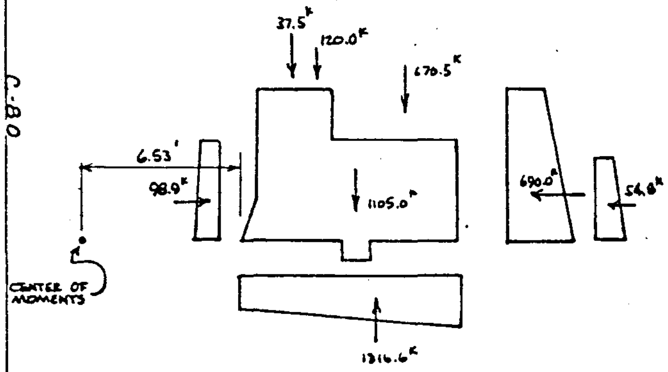
DATE:

CHECKED BY:

DATE:

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	ARM × cos φ	MOMENT
W_{CONC}	$[.1488][2][29.8][(692-684.5)(14.5)+(694-692)(4.5)+(688-684.5)(1/2)(.5)+(2)(3)(2)]$	1105.0			14.07	15547
W_{WATER}, ABOVE GATE	$[.0625](10)(2)(29.8)(710-692)$	670.5			16.70	11197
W_{WATER}, IN LOCK	$[.0625][2][29.8][(692-688)(1.75)+(688-684.5)(1.75)(1/2)]$	37.5			6.87	258
W_{GATE}	60^K/leaf × 2 = 120^K	120.0			9.25	1110
P_{WATER} ABOVE GATE	$[\cos\phi][.0625][2][29.8][(694-684.5)(16)+(1/2)(694-6)^2]$		-690.0	4.39		-3029
P_{WATER}, IN LOCK	$[\cos\phi][.0625][2][29.8][1/2](692-684.5)^2$		98.9	2.50		247
P_{earth} { conc above lock	$[.1488](2)(29.8)(690-684.5)(.5)(1)(\cos\phi)+[.0754](1/2)(690-684.5)^2(.5)(2)(29.8)(\cos\phi)$		-54.8	2.21		-121
Uplift	$[.0625][2][28.9][(710-692)(16.25)+(1/2)(692-684.5)(16.25)]$	-1316.6			14.21	-18709
		616.4	-645.9			6500



$$e = \frac{6500}{616.4} = 10.54$$

By iterative means the active base area was found to be 936.5 SF

Total base area = 968.8

PERCENT ACTIVE BASE = (936.5/968.8)(100) = 96.67 %

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{28359}{21859} = 1.30$$

CASE I NORMAL OPERATIONS
UPPER POOL ABOVE SILL
LOWER POOL IN LOCK

φ = 20° 0'

FIGURE C-18 UPPER MITER SILL (56' LOCK)

SUBJECT:

UPPER MITER SILL (56' LOCK)

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	ARM × COS φ	MOMENT
W_{CONC}		1105.0			14.07	15547
W_{GATE}		120.0			9.25	1110
$P_{EARTH \& CONC}$			-54.8	2.21		− 121
		1225.0	-54.8			16536

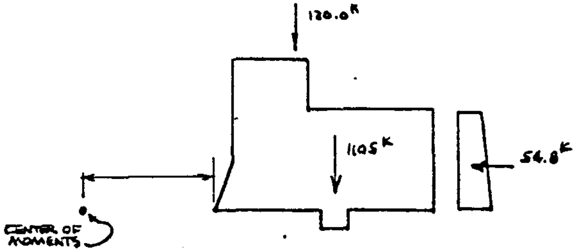
$e = \frac{16536}{1225.0} = 13.50$

By iterative means the active base area was found to be 968.8

Total base area = 968.8 SF

PERCENT ACTIVE BASE = (968.8/968.8)(100) = 100%

$$F.S. = \frac{\Sigma \text{STABILIZING } M}{\Sigma \text{OVERTURNING } M} = \frac{16657}{121} = 137.6$$



CASE II MAINTENANCE CONDITION

LOCK DEWATERED

φ = 20°0'

FIGURE C-18 UPPER MITER SILL (56' LOCK)

SUBJECT:
UPPER MITER SILL (56' LOCK)

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

CASE I NORMAL OPERATIONS

$R = \Sigma Fv \, \mathrm{Tan}\,\phi + \text{key Resistance}$

$= (616.4)(.5890) + (3)(2)(29.8)(.075)(144)$

$= 2294$

$SSF = 2294/645.9$

$= 3.55$

CASE II MAINTENANCE CONDITION

$R = (1225)(.5890) + (3)(2)(29.8)(.075)(144)$

$= 721.5 + 1931$

$= 2652.5$

$SSF = 2652.5/54.8$

$= 48.4$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{P}{A} + \frac{Mc}{I}$$

$$= \frac{616.4}{936.5} + \frac{(616.4)(13.34-10.54)(13.34)}{28118.2}$$

$$= 0.66 + 0.82$$

1.48 KSF

CASE II MAINTENANCE CONDITION

$$f = \frac{P}{A} + \frac{Mc}{I}$$

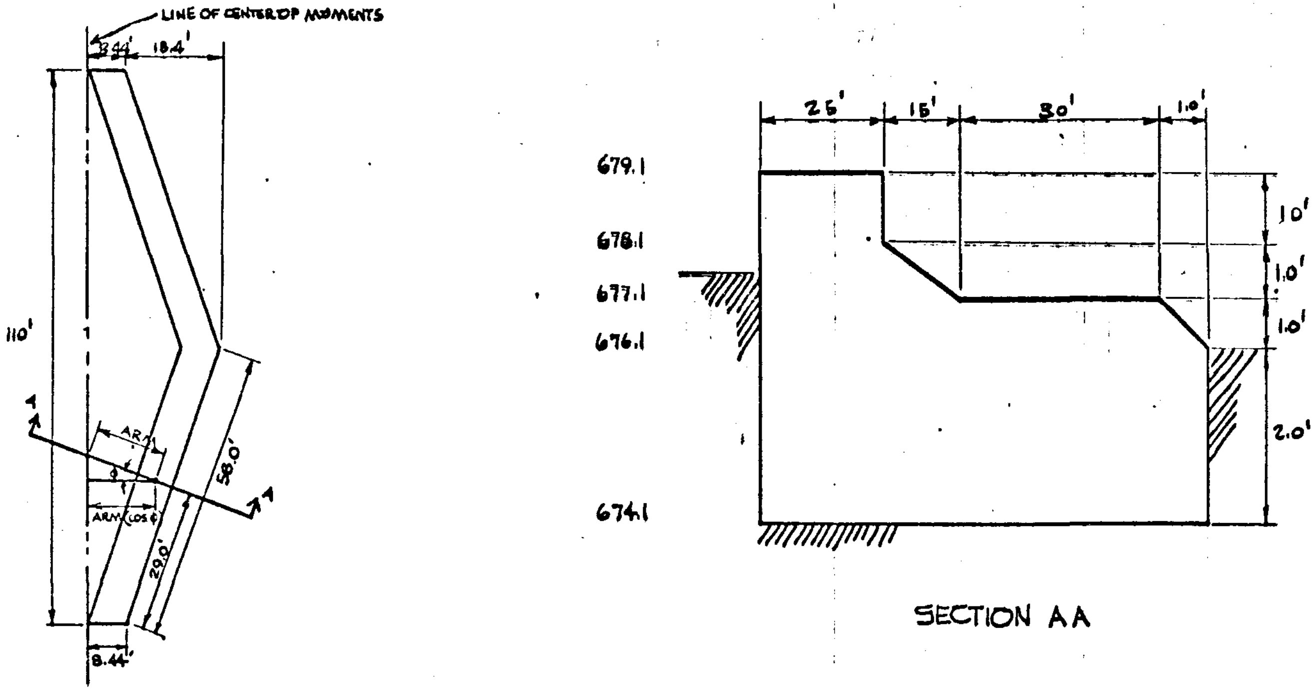
$$= \frac{1225}{968.8} + \frac{1225(13.74-13.6)(13.74)}{82585}$$

$$= 1.26 + 0.12$$

$= 1.38$ KSF

FIGURE C-18 UPPER MITER SILL (56' LOCK)

SUBJECT:	COMPUTED BY:	DATE:
LOWER MITER SILL (110' LOCK)	CHECKED BY:	DATE:

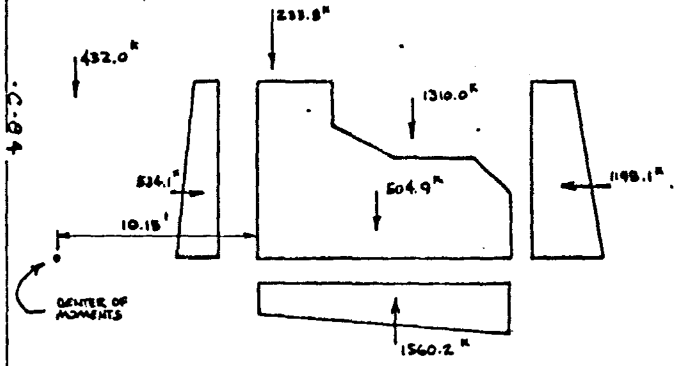


SECTION AA

FIGURE C-19 LOWER MITER SILL (110' LOCK)

SUBJECT: LOWER MITER SILL (110' LOCK)	COMPUTED BY:	DATE:	
	CHECKED BY:	DATE:	

NOT TO SCALE



CASE I NORMAL OPERATIONS

$\phi = 18^\circ 30$

UPPER POOL IN LOCK

LOWER POOL BELOW LOCK

ITEM	FACTORS	F_V	F_H	ARM	ARM × cos φ	MOMENT
W_{CONC}	[.1488][58][2][(2.5)(679.1-674.1)+ (5.5)(677.1-674.1) + (1/2)(1.5)(1) - (1/2) $(1)^2$]	504.9			12.89	6508
W_{WATER} (ABOVE GATE)	[.0625][58][2][(5.5)(710-678.1)+ (4)(1) + (1/2)(1.5)(1) + (1/2)$(1)^2$]	1310.0			14.62	19152
$W_{WATER, BELOW G.}$	[.0625](2)(58)(692-679.1)(2.5)	233.8			10.87	2541
W_{GATE}	216ᴷ/leaf × 2	432.0			4.22	1823
$P_{WATER, IN LOCK}$	[.0625][cos φ][(30.9)(679.1-674.1) (2)(58) + (1/2)$(679.1-674.1)^2$(2)(58)]		-1148.1	2.44		- 2801
$P_{WATER, BELOW L.}$	[.0625][cos φ][2][58] [(12.9) (679.1-674.1) + (1/2)$(679.1-674.1)^2$]		534.1	2.36		1260
UPLIFT	[.0625][2][58][(692-674.1)(8)+ (1/2)(710-692)(8)]	-1560.2			13.84	- 21593
		920.5	- 614.0			6890

$$e = \frac{6890}{920.5} = 7.49' \text{ from center of moments}$$

PERCENT ACTIVE BASE = By iterative means it was found that part of the base was not in compression. the active base area is 592.8SF

total area = 928.4

(592.8/928.4)(100) = 63.85%

$$F.S. \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{31284}{24394} = 1.28$$

FIGURE C-19 LOWER MITER SILL (110' LOCK)

SUBJECT: LOWER MITER SILL (110' LOCK)	COMPUTED BY:
	DATE:
	CHECKED BY:
	DATE:

NOT TO SCALE

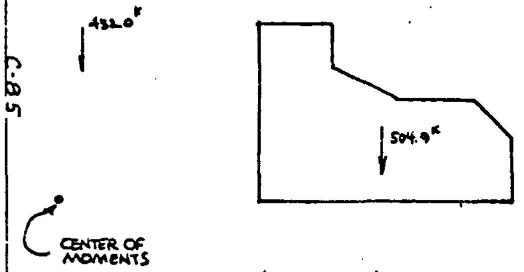
ITEM	FACTORS	F_V	F_H	ARM	ARM x cos φ	MOMENT
W CONC		504.9			12.98	6508
W GATE		432.0			4.22	1823
		936.9	0			8331

$$e = \frac{8331}{936.9} = 8.9'$$

Percent active base by iterative means the active base area was found to be 778.4 SF
Total base area 928.4

$$(778.4/928.4)(100) = 83.84\%$$

$$F.S \quad \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{8331}{0} = \infty$$



φ = 18°20'
LOCK DEWATERED

CASE II MAINTENANCE CONDITION

FIGURE C-19 LOWER MITER SILL (110' LOCK)

SUBJECT: LOWER MITER SILL (110' LOCK)	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \sum F_v \tan\theta + \text{Key Resistance}$$
$$= (920.5)(.5890) + (8)(2)(58)(.075)(144)$$
$$= 542.2 + 10{,}022.4$$
$$= 10564.6$$

$$SSF = 10564.6/614$$
$$= 17.21$$

CASE II MAINTENANCE CONDITION

$$R = 936.9(.5890) + 10022.4$$
$$= 10574.2$$

$$SSF = 10574.2/0$$
$$= \infty$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{920.5}{592.8} + \frac{(920.5)(9.84-7.49)(9.84)}{8554.9}$$
$$= 1.56 + 2.49$$
$$= 4.05 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{936.9}{778.4} + \frac{(936.9)(11.75-8.9)(11.75)}{17809.8}$$
$$= 1.20 + 1.76$$
$$= 2.96 \text{ KSF}$$

FIGURE C-19 LOWER MITER SILL (110' LOCK)

SUBJECT:	COMPUTED BY:	DATE:
CELLS OF EXTENSION TO UPPER GUARD WALL	CHECKED BY:	DATE:

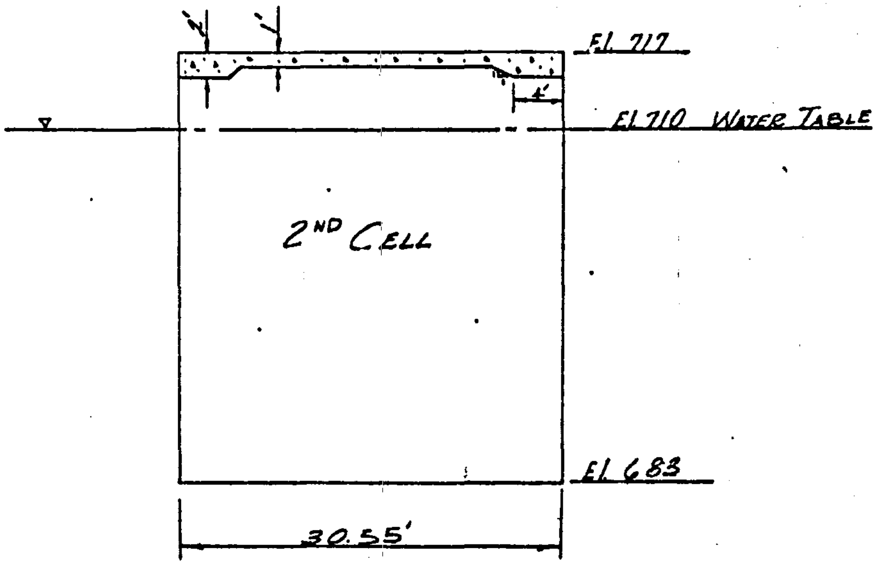
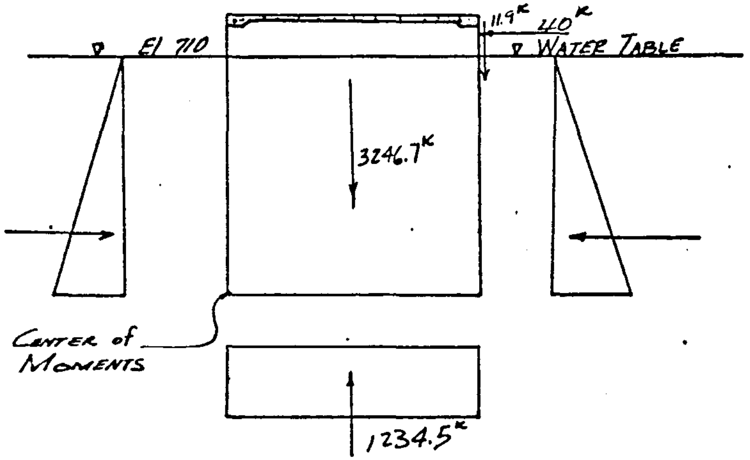


FIGURE C-20 CELL OF EXTENSION TO UPPER GUARD WALL

SUBJECT: CELLS OF EXTENSION TO UPPER GUARD WALL	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

No Scale



NORMAL OPERATION

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC} & FILL	$\left(\frac{3.14}{4}\right)(30.55-9)^2\left[0.1328(710-683)+(.1127)(717-710-1)+.1513\right]$	1608		$\frac{30.55}{2}$	24,574
	$\frac{3.14}{4}\left(30.55^2-(30.55-9)^2\right)\left[0.1328(710-683)+.1127(717-710-2)+2(.1513)\right]$	1638.7		$\frac{30.55}{2}$	25,031
UPLIFT	$(1.685)\left(\frac{3.14}{4}\right)(30.55)^2$	-1234.5		$\frac{30.55}{2}$	-18,857
TIMBER		11.9		31.73	378
IMPACT			40	32	-1280
		2024.1	40		29,846

$$e = \frac{29,846}{2024.1} = 14.75'$$

100% Effective Base

FIGURE C-20 CELL OF EXTENSION TO UPPER GUARD WALL

SUBJECT: CELLS OF EXTENSION TO UPPER GUARD WALL

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

$$S_{SF} = \frac{(2024.1)(.5658)}{40} \doteq 29$$

BASE PRESSURE

$$f = \frac{2024.1}{732.6} + \frac{(2024.1)(0.525)(15.275)}{42736}$$

$$= 2.76 + 0.38 = 3.14 \ ^{K}/_{\square'}$$

CELLS OF EXTENSION TO LOWER GUARD WALL WERE ANALYZED FOR STABILITY AND THEY WERE AS STABLE AS THE UPPER GUARD WALL CELLS.

FIGURE C-20 CELL OF EXTENSION TO UPPER GUARD WALL

SUBJECT: ABUTMENT WALL	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

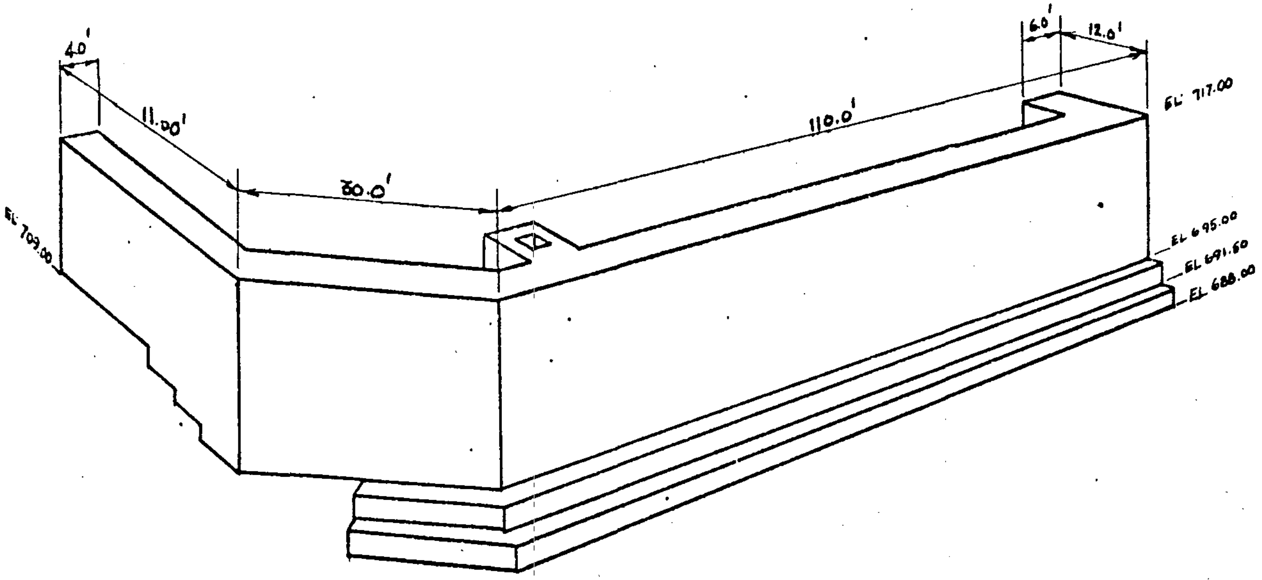
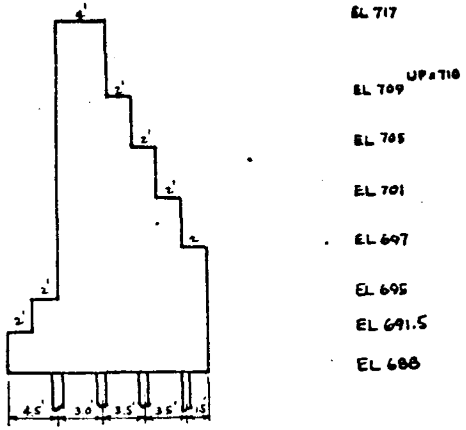


FIGURE C-21 ABUTMENT WALL

SUBJECT:
ABUTMENT WALL

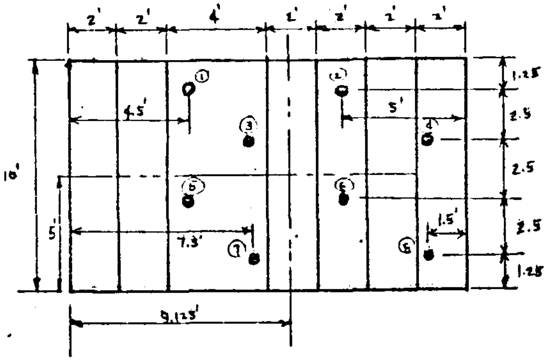
COMPUTED BY: | DATE:
CHECKED BY: | DATE:



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$	[.15][(2)(10)(691.5-688)+(2)(10)(695-688)+(4)(10)(717-688)+(2)(10)(707-688)+(2)(10)(705-688)+(2)(10)(701-688)+(2)(10)(697-688)]	298.5		8.83	2635.5
$W_{BACKFILL}$	[(.115)(1)+(.1208)(716-710)+(.1378)(710-709)](2)(10)+ [(.115)(1)+(.1208)(6)+(.1378)(5)](2)(10)+ [(.115)(1)+(.1208)(6)+(.1378)(9)](2)(10)+ [(.115)(1)+(.1208)(6)+(.1378)(13)](2)(10)	144.4		12.96	1872
$W_{WATER, RIVER}$	[.0625](2)(692-691.5)(10)	06		1.00	1
$P_{BACKFILL}$	[.115]$(\frac{1^2}{2})$(10) + (.115)(1)(10)(716-688) + (.5)(.1208)$(\frac{6^2}{2})$(10) + (.5)(.1208)(6)(10)(710-688) + [(.5)(75.5)+62.4]$\frac{(22)^2}{2}$(10)		-365.4	9.25	-3382
$P_{WATER, RIVER}$	[.0625]$\frac{(692-688)^2}{2}$(10)		5.0	1.33	7
UPLIFT	[.0625][(692-688)(16)(10) + (710-692)(1/2)(16)(10)]	-129.8		9.85	1278
		313.7	-360.4		-1599.78

$$\bar{X} = 5'$$

$$\bar{Y} = \frac{\Sigma M}{\Sigma F_V} = \frac{1599.78}{313.7} = 5.10'$$



LOCATION FOR CENTROID FOR PILES

AREA A	$\bar{Y}$	$\bar{X}$	$A\bar{Y}$	$A\bar{X}$
$2(\frac{\pi D^2}{4}) = 1A$	4.5	8.75	A(4.5)	A(8.75)
1A	7.5	6.25	A(7.5)	A(6.25)
1A	11.0	3.75	A(11)	A(3.75)
1A	14.5	1.25	A(14.5)	A(1.25)
4A			A(37.5)	A(20)

$$y = \frac{\Sigma A\bar{y}}{\Sigma A} = \frac{(2)(A)(37.5)}{(2)(A)(4)} = 9.125$$

$$x = \frac{\Sigma A\bar{x}}{\Sigma A} = \frac{20(A)}{4(A)} = 5$$

MOMENT OF INERTIA ABOUT CENTROID OF PILES

PILE NO	$\frac{\pi D^4}{64} = I_o$	$\frac{\pi D^2}{4} = A$	y	x	$I_{YY} = I_o + Ax^2$	$I_{XX} = I_o + Ay^2$
1	0.0491	0.7854	(9.125 - 4.5)	(5 - 1.25)	11.09	16.84
2	"	"	(6.875 - 5)	(5 - 1.25)	11.09	2.81
3	"	"	(9.125 - 7.5)	(5 - 3.75)	1.28	2.12
4	"	"	(6.875 - 1.5)	(5 - 3.75)	1.28	22.73
5	"	"	(9.125 - 4.5)	(5 - 3.75)	1.28	16.84
6	"	"	(6.875 - 5)	(5 - 3.75)	1.28	2.81
7	"	"	(9.125 - 7.5)	(5 - 1.25)	11.09	2.12
8	"	"	(6.875 - 1.5)	(5 - 1.25)	11.09	22.73
					49.48	87.0

FIGURE C-21 ABUTMENT WALL

SUBJECT: ABUTMENT WALL

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

HORIZONTAL PILE LOADS

$$\text{LOAD PER PILE} = \frac{\text{TOTAL HORIZONTAL LOAD}}{\text{NO. OF PILES}}$$

$$= \frac{360.4}{8}$$

$$= 45.0\ ^{K}/\text{PILE}$$

VERTICAL PILE LOADS

$$f = \frac{P}{A} + \frac{Mc}{I}$$

$$= \frac{\Sigma F_v}{A_{pile}} + \frac{\Sigma F_v \bar{y} c}{I_{xx}}$$

$$= \frac{313.7}{8(.785)} + \frac{313.7(9.125 - 5.1)(6.875 - 1.5)}{89}$$

$$= 49.95 + 76.25$$

$$= 126.20 \text{ KSF} \quad \text{or} \quad 99 \frac{K}{\text{PILE}} < 100 \frac{K}{\text{PILE}} \quad \text{O.K.}$$

THE HORIZONTAL LOAD OF 45 KIPS/PILE IS GREATER THAN 8 KIPS/PILE ALLOWABLE. THE 45 KIPS/PILE WAS OBTAINED BY USING THE MOST CRITICAL SECTION WHICH HAS FILL TO THE TOP OF THE ABUTMENT. THIS SITUATION DOES NOT EXIST FOR ALL PILES. FOR THE LENGTH OF ABUTMENT PARALLEL TO FLOW, 51 PILES HAVE NO HORIZONTAL LOAD, 9 HAVE FROM 0 TO 45 KIPS/PILE, AND 25 HAVE 45 KIPS/PILE. IT IS BEST TO CONSIDER THE TOTAL ABUTMENT WHICH IS BEING PUSHED TOWARDS THE FIXED DAM. CONSIDERING THE TOTAL ABUTMENT THE HORIZONTAL FORCE PER PILE IS:

$$\frac{(.707)(\frac{.1207}{4})(16^2 - 10^2)(25) + \frac{(12.5)}{6}(36 + (4)(9) + 0) + (36)(34)}{107 \text{ PILES}} \simeq 14 \text{ KIPS/ PILE}$$

CONSIDERING THAT THE CONCENTRATION OF HORIZONTAL LOAD IS DIRECTLY ON THE OTHER SIDE OF THE ABUTMENT FROM THE FIXED DAM, CONSIDERABLE LOAD CAN BE SUPPORTED BY THE DAM ALONG ITS AXIS, WHICH IS PERPINDICULAR TO FLOW. CONSIDERING THIS SUPPORT, THE ABOVE HORIZONTAL LOADING WILL BE NO PROBLEM.

FIGURE C-21 ABUTMENT WALL

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.

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1. Condition survey. 2. Dams. 3. Emsworth Locks and Dam. 4. Locks. 5. Stability analysis. 6. Stress analysis. 7. Structural evaluation. I. U. S. Army Engineer District, Pittsburgh. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper C-76-8)

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