MISCELLANEOUS PAPER C-76-9

ENGINEERING CONDITION SURVEY AND STRUCTURAL INVESTIGATION OF LOCKS AND DAM 3, MONONGAHELA RIVER

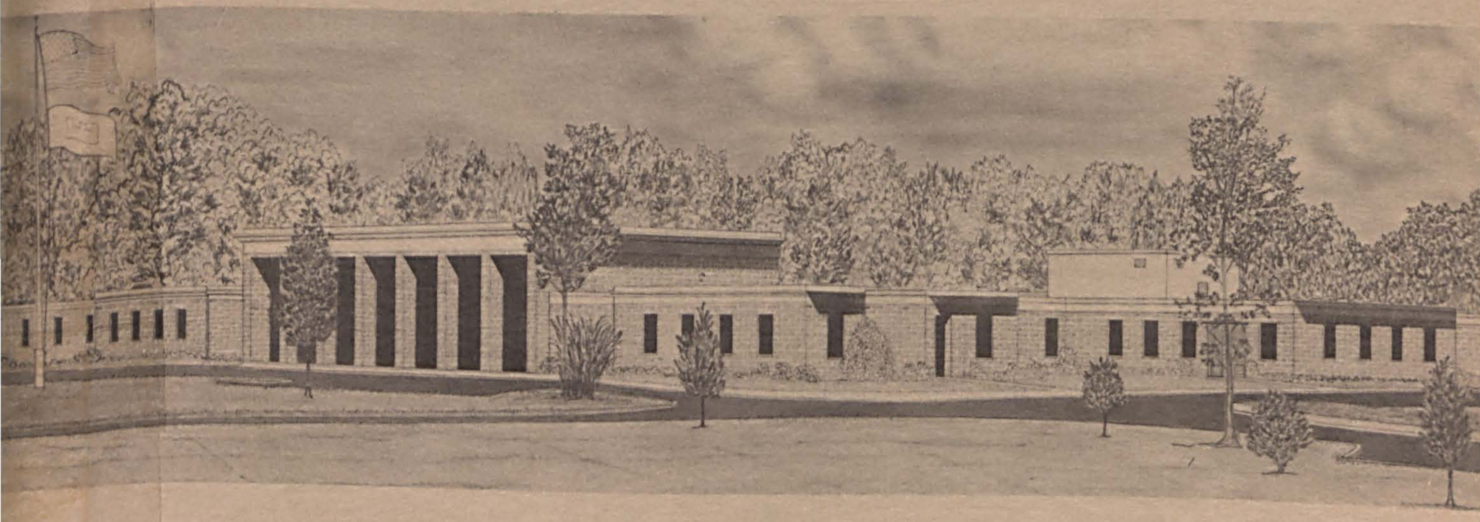
by

Carl E. Pace, Richard L. Stowe,
and Alan D. Buck

Concrete Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

August 1976

Final Report



Prepared for U. S. Army Engineer District, Pittsburgh
Pittsburgh, Pennsylvania 15222

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.

Pace, Carl E

Engineering condition survey and structural investigation of Locks and Dam 3, Monongahela River, by Carl E. Pace, Richard L. Stowe, and Alan D. Buck. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1976.

1 v. (various pagings) illus. 27 cm. (U. S. Waterways Experiment Station. Miscellaneous paper C-76-9)

Sponsored by U. S. Army Engineer District, Pittsburgh, Pittsburgh, Pennsylvania.

Includes bibliography.

1. Condition survey. 2. Dams. 3. Locks. 4. Locks and Dam 3. 5. Stability analysis. 6. Stress analysis. 7. Structural evaluation. I. Buck, Alan D., joint author. II. Stowe, Richard L., joint author. III. U. S. Army Engineer District, Pittsburgh. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper C-76-9)

TA7.W34m no.C-76-9

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Miscellaneous Paper C-76-9	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) ENGINEERING CONDITION SURVEY AND STRUCTURAL INVESTIGATION OF LOCKS AND DAM 3, MONONGAHELA RIVER		5. TYPE OF REPORT & PERIOD COVERED Final report 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Carl E. Pace, Richard L. Stowe, and Alan D. Buck		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Army Engineer Waterways Experiment Station Concrete Laboratory P. O. Box 631, Vicksburg, Miss. 39180		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U. S. Army Engineer District, Pittsburgh Federal Building, 1000 Liberty Avenue Pittsburgh, Pa. 15222		12. REPORT DATE August 1976 13. NUMBER OF PAGES 257
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

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

18. SUPPLEMENTARY NOTES

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

Condition survey	Stability analysis
Dams	Stress analysis
Locks	Structural evaluation
Locks and Dam 3	

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

The results of an engineering condition survey and a structural analysis of Locks and Dam 3, Monongahela River, are given.

External cracking of the concrete in the lock walls is extensive. Many horizontal and vertical cracks, although isolated and not fitting into a total failure picture, can result in localized failures, especially in critically stressed regions. The outer 2 to 6 ft of concrete as described in the core and photo logs are highly deteriorated. The same condition probably exists in the outer surface of the walls to lower pool. (Continued)

20. ABSTRACT (Continued).

The internal concrete is generally of adequate strength for current design criteria. One exception is a small zone in the downstream gate monolith of the middle wall; a compressive strength of 1150 psi was obtained on the concrete from this zone. The concrete is in a critically stressed area; immediate remedial action is recommended. Pressure grouting as an interim measure is suggested. There is one possible continuous crack in gate monolith R-23; it runs for a depth of 35 ft and possibly extends for two-thirds of the monolith to the riverside.

The borehole data indicate a consistent pattern of fractured concrete especially in the lower portions of the monoliths. This can be a significant factor tending to produce problems even at moderately high stresses.

The foundation appears to be in good condition over most of the lock site. Several local zones of highly fractured rock and one zone where a monolith bears on coal, R-12, are the only zones considered as possible weak rock. During dewatering of the locks, these local foundation conditions could contribute to serious failures of lock wall sections. Such failures could occur due to inadequate sliding resistance of some foundation materials at or near the concrete-foundation contact. There are no detectable continuous discontinuities, zones of fracture, or seams of weak material; bedding planes are the only detectable continuous features over the lock site.

The stability analysis reflects deficiencies in many areas:

a. In general the monoliths in the land wall do not comply with current criteria for overturning, sliding, or allowable base pressures.
b. Some middle wall monoliths do not have adequate resistance to sliding.
c. Some monoliths in the river wall do not have adequate resistance to sliding, and where coal seams underlie monoliths critical situations can exist if the river lock is ever dewatered. Bearing pressures are excessive in the upper guard wall causing it to tilt riverward.

The stability analysis for the land wall monoliths shows severe inadequacies and as with the guard wall and the abutment, it is probable that stability problems will develop. These problems can cause considerable delays to navigation through these locks.

The upper guard wall has excessive bearing pressures between the upper and lower timber cribbing sections and has tilted riverward. There are probably some rocks between the cribbing members which result in more bearing area and, therefore, reduce the calculated stresses considerably. However, the tilting of the guard wall shows that the bearing pressures are still excessive. Tow impact can cuase this guard wall to fail.

Cracking and deterioration of the exposed concrete are severe problems and are significant at stress concentrations. Stress analysis shows that gouges and concrete deterioration can become problems in the upper guide wall. Excessive stress concentrations exist in the structural slab over the filling and emptying flume. Tensile stresses indicate magnitudes that are excessive for even good quality concrete. Compressive stresses indicate magnitudes that would not be considered excessive for sound, uncracked concrete; however, for badly deteriorated concrete, they are excessive. Stress concentrations also exist at gate anchorages. As the concrete at Locks and Dam 3 continues to deteriorate, stress magnitudes under existing loading conditions, as has been observed, will become a problem.

The locks have apparently functioned well for over 70 years, but this cannot be expected to continue indefinitely because of accelerating deterioration due to concrete cracking, leaching, and exposure to a freezing-and-thawing environment. These locks and dam will have to be replaced or rehabilitated.

It is recommended that a study be initiated immediately to evaluate rehabilitation or replacement of Locks and Dam 3 on the Monongahela River.

THE CONTENTS OF THIS REPORT ARE NOT TO BE USED FOR ADVERTISING, PUBLICATION, OR PROMOTIONAL PURPOSES. CITATION OF TRADE NAMES DOES NOT CONSTITUTE AN OFFICIAL ENDORSEMENT OR APPROVAL OF THE USE OF SUCH COMMERCIAL PRODUCTS.

PREFACE

The work of a detailed engineering condition survey and structural investigation for Locks and Dam 3 located on the Monongahela 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 property and foundation investigation was performed by Messrs. R. L. Stowe, A. D. Buck, G. S. Wong, F. S. Stewart, and J. B. Eskridge. The report was written by Messrs. Pace, Stowe, and Buck.

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

CONTENTS

CONTENTS (CONTINUED)

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-ol	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 LOCKS AND DAM 3 MONONGAHELA RIVER

SECTION 1: INTRODUCTION

This report presents the results of an engineering condition survey and a structural analysis of Locks and Dam 3 on the Monongahela River. The investigation was conducted during the period from October 1974 through June 1975 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.

Background information pertinent to inspection and evaluation of completed Civil Works structures, and previous reports concerning periodic reports and a condition survey is presented in Appendix I.

Location of Study Area

Locks and Dam 3 is on the Monongahela River and is located in Allegheny County just upstream of the town of Elizabeth, Pennsylvania. Locks and Dam 3 is about 24 miles upstream from Pittsburgh, Pennsylvania, where the confluence of the Monongahela River and the Allegheny form the Ohio River, and about 105 miles below the head at Fairmont, West Virginia.

Purpose and Approach

The purpose of this report is to present the findings and conclusions drawn from the condition survey and structural investigation of Locks and Dam 3. The overall evaluation of Locks and Dam 3 included an evaluation of three component parts, the foundation, the concrete structures, and the concrete-foundation interaction. All parts were studied relative to the structural integrity, existence of any critical conditions which could cause immediate failure, deterioration of materials, existence of severe cracking, and movement.

The principal work items were a condition survey, evaluation of rock core borings and concrete, evaluation of field and laboratory testing, stability analysis of typical monoliths, a finite element analysis of specific monolith cross sections, and a final evaluation of the existing structure. This report presents the results derived from the principal work items under appropriate Sections of the text and in Appendices that address the foundation condition, the concrete structure, and the structure-foundation interaction. The last work item, a final evaluation of the existing structure, is presented in Section VIII: Conclusions and Recommendations.

Historical Construction

Locks and Dam 3 were constructed from 1905 to 1907 and have been operational since 20 May 1907. The two parallel navigation locks were originally constructed with 56-ft wide chambers 360 ft in length. The landward lock chamber was extended to 720-ft length in 1924; therefore, the downstream guide wall was constructed at that time. The upstream guide wall and guard wall were extended in 1926.

The upstream guide wall has never had wall armor; therefore, it has been badly gouged. The addition of a filling and emptying flume covered by a structural slab and supporting beams was added in the land wall monoliths. This addition resulted in construction joints at the interface of the original and newer additions which could cause the structures to act as separate units and not as monoliths. The units can be overturned much easier than the total monolithic structure because the effective base area is reduced and loads can be applied at the top of a unit by strut action (for example, through the structural slab) making them deficient in resistance to overturning. In certain case loadings these additions can also result in overstressing.

In some cases the as-built structure is substantially different from the planned construction. This is especially true for the downstream land wall monoliths and for the lower guide wall monoliths. A hanging wall is constructed on the river side of some of the land wall monoliths. This construction causes the monoliths to be less resistant to overturning; especially when this hanging wall extends deeper than the base as originally planned. Also, stress analysis shows that this hanging wall can act as a cantilever, creating a path of tensile stress up through the monolith.

The horizontal surfaces of the land wall were resurfaced a few years ago. The vertical face of the land wall has been refaced two or three times. The vertical surfaces of the middle and river walls and the horizontal surface of the middle wall have been resurfaced. In many places the resurfacing is gone and in most cases there is advanced deterioration.

The upper guard wall extension, added in 1926, is a concrete cap supported on a rock filled crib. Due to the low bearing area of the interface of the timber cribbing sections and settlement of rock fill in the crip sections there can be stability and stress problems with this construction.

The 56-ft lock miter gates are kept in relatively good repair because they are replaced by spare gates on a rotating replacement program. The operating machinery for the landward chamber miter gates are hydraulically operated and the riverward chamber miter gate machinery is air operated which does cause some problems in cold weather with ice forming in the lines.

A severe and expensive problem is encountered in dewatering the locks because the filling and emptying valves have no provisions for being bulkheaded. There is no provision for emergency closures at this project and it was originally planned to use cofferbeam and needles as closures for maintenance.

The non-navigable fixed crest dam spans 688 ft from the river face of the river wall to the river face of the abutment. The dam is constantly underwater. The abutment next to the dam is supported on pilings.

The locks and dam essentially do not even have temperature steel in the concrete monoliths. The concrete is nonair-entrained and is therefore more susceptible to weathering. It is amazing that the structure has functioned so well for over 70 years, but it is fast approaching a condition where its remaining life is the time necessary for a replacement structure to be built.

SECTION 2: SURFACE CONDITION OF CONCRETE

Surface Cracks

There are many horizontal and vertical cracks in the walls of the locks. Most of the cracks are isolated and do not fit into a total failure picture but can result in localized failures, especially in critically stressed regions. Recently, a localized failure due to tow impact did occur in the lower guide wall and as gouges and concrete deterioration continues failures of this type will become more frequent. This local failure probably initiated from surface cracks.

Many of the surface cracks show the results of leaching as evidenced by leakage from a horizontal crack at about station 2+90B on the riverside of the river wall. Evidence of leakage was observed in a near vertical crack between stations 3+82B and 4+70.5B on the same lock wall. Borehole camera data indicate that the near vertical crack may be continuous at least part way through the monolith. The borehole camera traces a vertical crack 1/8 to 1/4 in. wide for 35 ft in the concrete, or 71 percent of the depth of the monolith. The possibility is suggested in Appendix II and explored in detail in Appendix IV.

A good example of leaching in original concrete just below an overlay was observed for a distance of about 50 ft on the lower guide wall. This area has many tight horizontal and vertical cracks; most of which are covered with white calcium carbonate leached from the concrete. In almost all cases the surface cracks cannot be related to internal cracks because of the many repairs to both vertical and horizontal surfaces over the life of the locks. There is no way of knowing whether visible cracks are confined to the thickness of the repair or are extensions of internal cracks. However, it is known that the surface cracks permit water to percolate into the concrete and during severe weather conditions causes deterioration by the process of freezing and thawing.

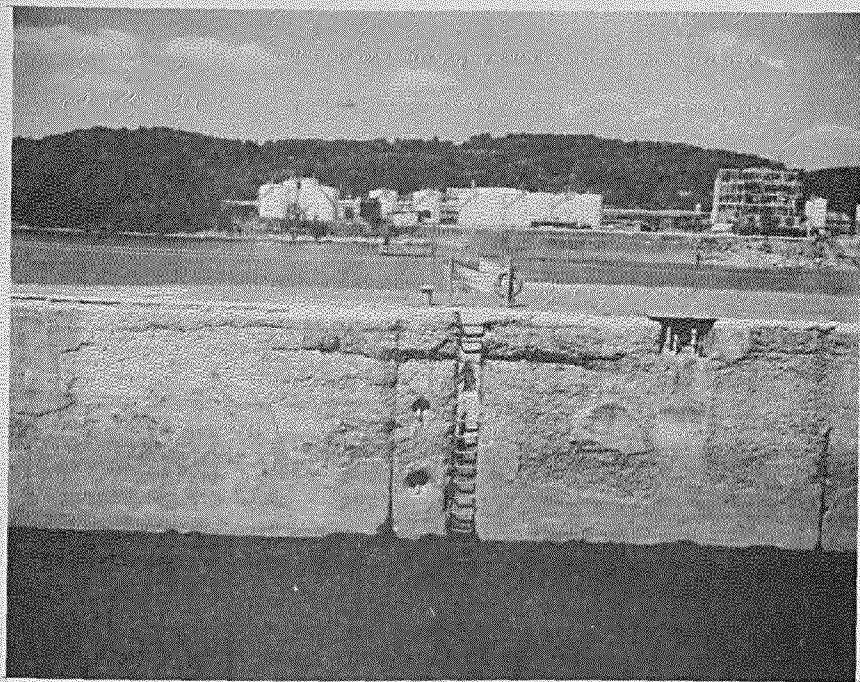
A 6-in.-wide crack is described in Reference[1] and is located at station 4+44.6B just above the waterline. The possible extent of this crack is one-third the way through M-16 (see Appendix II). Monolith M-16 is a gate monolith and should be considered as a critically stressed monolith.

Structural cracking has occurred in the roof structure over the landward flume. The roof forms the open deck of the work area and is of a concrete beam and slab construction. Observed cracking in beams renders the area unsafe for other than very light loaded vehicles or equipment.

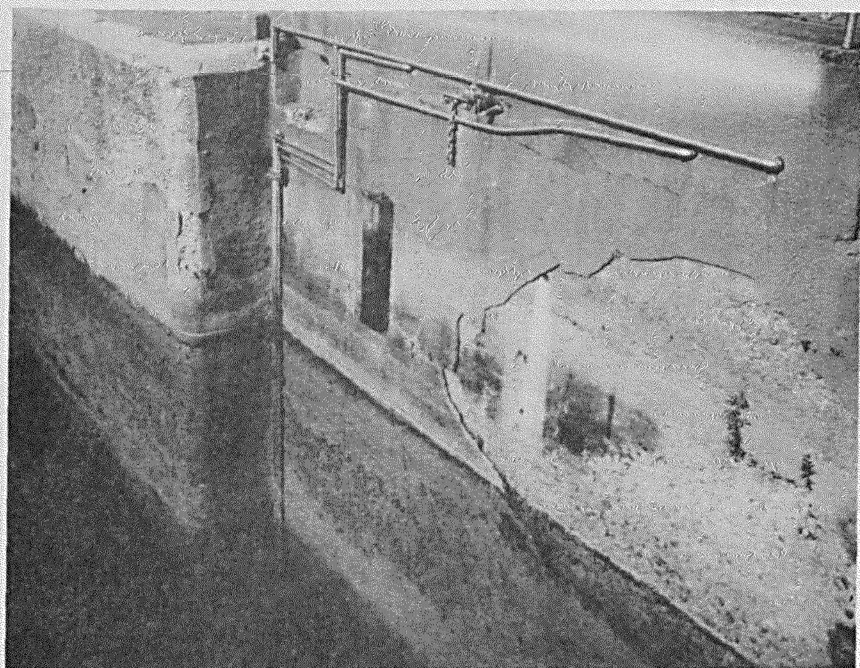
Deteriorated Concrete

The fact that essentially all of the original concrete surfaces of these locks have been masked by repair work during the 50 to 70 years of their existence is evidence of deterioration by weathering and lock usage. The rate of deterioration will not decrease with longevity, rather it will accelerate. Both core logs and borehole photography records show that the outer 2 to 6 ft of concrete is badly weathered; it can reasonably be assumed that the greater portions of the tops and the exposed sides of the lock walls are in a similar condition.

Figure 2.1 shows the downstream areas of the chamber face of the river wall. They illustrate the appearance of the gunite, the areas of old concrete where the gunite cover has come off, and some opening between the gunite and the underlying concrete in the gate recess. Figure 2.2shows an area on the top of the river wall near the chamber face where the overlying concrete repair has spalled off exposing the old concrete beneath it. The aggregate particles in the old concrete are loose in their sockets and pieces of mortar can be removed by hand; this area of loose aggregate and fragmented mortar looks like and probably represents damage due to long-time frost action. Figure 2.3 illustrates the damaged surface of the guard wall just above the upper gate of the large chamber. This type of damage is common in the guard wall surfaces, and is caused by continued impact and scraping by boats and barges upon frost-weakened concrete. The amount of deteriorated concrete in the lock wall is not surprising in view of the length of time the nonair-entrained concrete has been exposed to the severe climate where freezing and thawing are common occurrences. Continued maintenance and repair of the exterior concrete will be required at Lock and Dam 3.



a. Chamber Wall of River Wall at about Sta. 2+50B
Showing Loss of Gunite Cover



b. Downstream Gate Recess of River Wall
Showing Loss of Gunite Cover

Figure 2.1. River Wall Showing Loss of Gunite Cover.

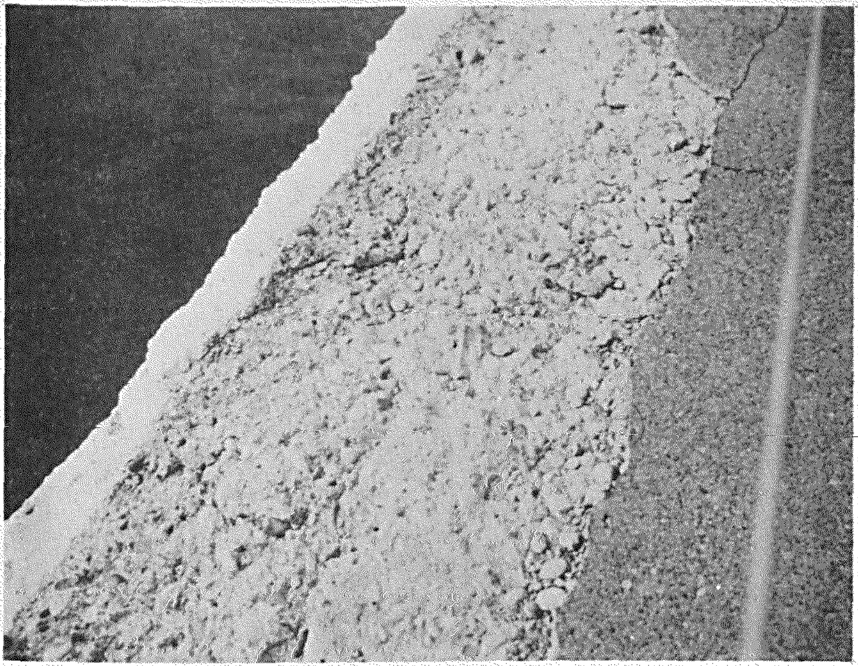


Figure 2.2. Top Surface of River Wall on Chamber Side Showing Spalling.

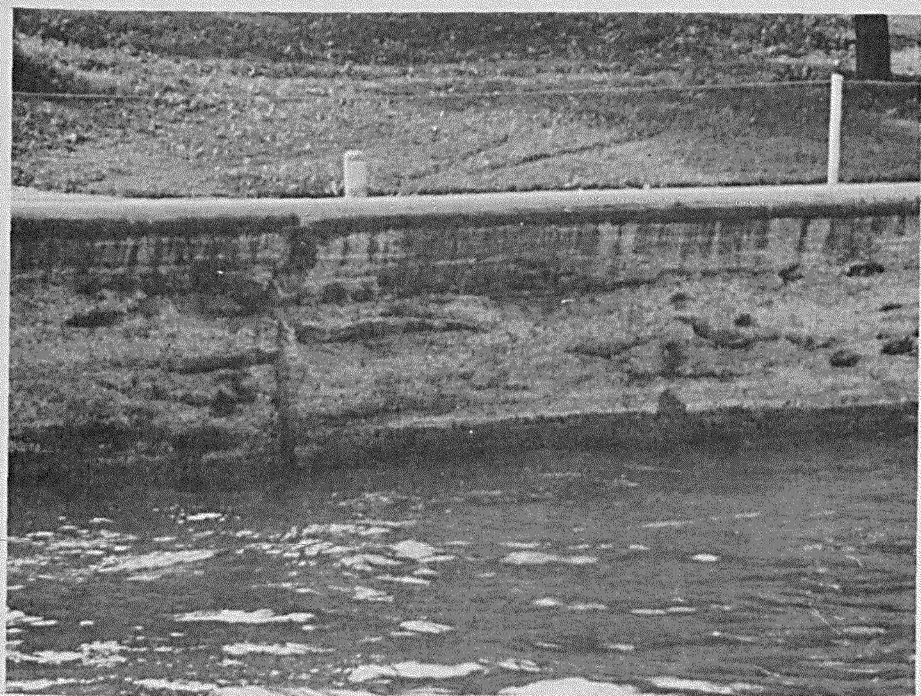


Figure 2.3. Upstream Guard Wall Near Upper Gate at about Sta. 0+00.

SECTION 3: FOUNDATION CONDITION

Introduction and Problem Statement

No major problems within the foundation have been previously identified. However, as an integral part of the structure, the foundation condition was investigated and evaluated relative to stability. Core boring information, core logs, borehole camera data, and physical property test results were used in evaluating the foundation condition.

Core Drilling

The locations of the vertical drill holes from which 6-in.-diameter core was recovered were selected so that core from critically stressed regions would be obtained. Both concrete and foundation cores were recovered. Six of eight drill holes were located adjacent to gates and in the gate monoliths; two were located in the land wall. Ten vertical NX-size drill holes were put down for supplemental concrete and foundation core information and for making borehole camera runs. The relative location of the 18 drill holes is presented in Figure 3.1. Detail information about the holes is presented in Appendix III. The core from all but holes M-7 and M-8 were stored at the U. S. Army Engineer Division Laboratory, Ohio River (ORDL). A brief description of the regional and local geology at the Locks and Dam 3 site is presented in Appendix III.

Summary of Weak Zones

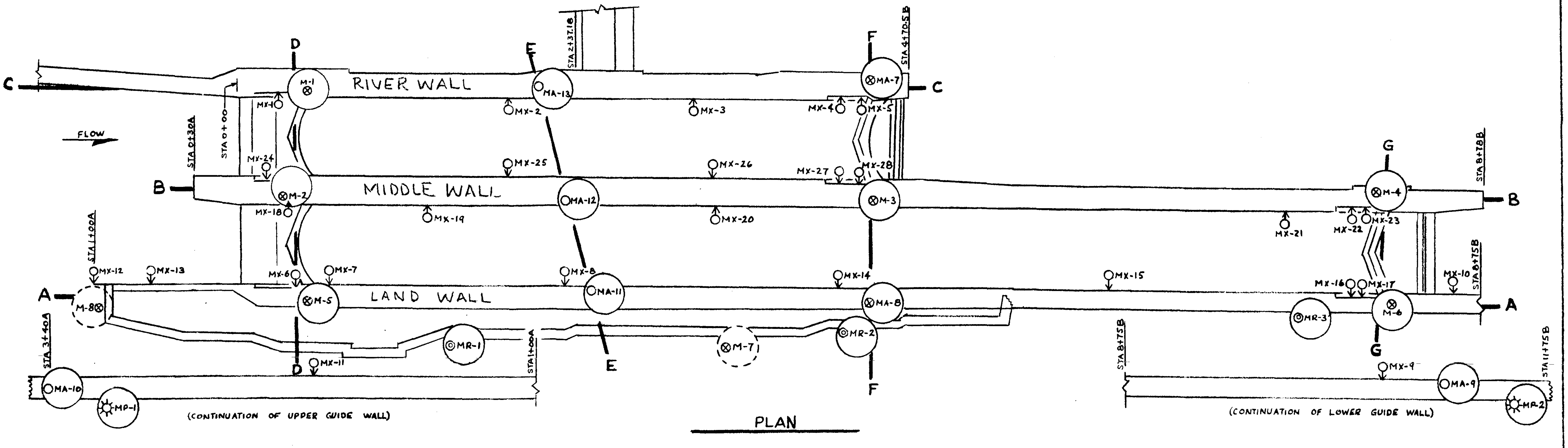
The foundation drilling disclosed several zones of possible weakness in the foundation rock. The foundation rocks recovered by drilling are essentially flat-lying, cyclic sediments consisting chiefly of shale, siltstone, limestone, and indurated clay; coal is present in minor quantities. Five core logs (M-1, M-2, MA-7, MA-10, and MA-12) indicate that for an average depth of 8.5 ft below the concrete, the rock is weathered or badly fractured. Holes M-1 and MA-7 are adjacent to the upstream and downstream gates, respectively. The borehole photo log of hole MA-7 shows that there is a poor contact between the concrete and foundation rock; in addition, the log shows that the rock contains voids and is highly fractured. Hole MA-7 is located in monolith R-23 and the first few feet of the underlying foundation rock is considered a weak zone. The photo log of hole MA-7 shows a vertical crack of from 1/8 to 1/4 in. wide extending from a depth of 2.4 ft beneath the top of the monolith to a depth of 37.4 ft. It is possible that the fractured foundation rock has contributed to the formation of the vertical fracture in this monolith.

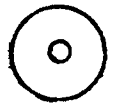
Holes M-2 and MA-12 are in the middle wall with M-2 located adjacent to the upper gate. The photo log of MA-12 indicates a good contact between the concrete and the foundation rock. Because of the zone of fractured rock in these two holes, monolith M-2 and M-9 are considered to be resting on several feet of weakened foundation rock.

The photo logs show a poor contact between the concrete and rock in hole MA-10 which is located at station 3+40A in the landside upper guide wall. In addition, the log shows that the rock is highly fractured with numerous voids for a 6-ft-deep zone just under the concrete. The foundation under the section of wall in which hole MA-10 is drilled is considered a possible weak zone.

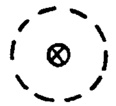
No plastic clay seams or other very weak seams were detected in the foundation. Coal was noted to underlie monolith R-12 and quite low shear values ($\phi = 15^{\circ}$, $C = 0$) were assumed for the stability analysis of this monolith. These low values should approach residual values if residual shear parameters are assumed for the coal member.

Fragmented foundation rock appears to be localized over the lock site. If the locks are dewatered then there is the possibility of major failures due to sliding of monolith R-12 and of those sections underlaid with highly fractured rock.





Drilled in 1973.



Drilled in Fall 1974.

Core locations at Lock and Dam No. 3. Partial reproduction of Drawing 037-L3-10/1.1 dated 23 Mar 73. Twenty vertical hole locations are circled. The relative locations of seven sections are shown and identified by letter.

Figure 3.1

SECTION 4: CONCRETE INTEGRITY

The concrete portion of the borehole photo logs, field soniscope results, and core logs of the 6-in.-diameter core were reexamined for any additional information that would aid in assessing the interior concrete.[1] This reexamination was concerned with trying to correlate interior with exterior cracks, extent of honeycombing and determining if unsound concrete existed in critically stressed regions such as gate monoliths.

Interior and Exterior Cracks

All cracks described in the borehole photo logs, detected by soniscope tests, and described in the core logs were considered when the original correlation between internal and external cracks was made. One probable correlation was found during the reexamination. Correlation between internal and external cracks was not possible due to the large amount of overlay repairs. There is no way of knowing with regard to most of the cracks whether a visible crack is restricted to the repair or is an extension of a crack that is present in the underlying concrete.

~~A great many internal cracks were recorded on the core log~~ sheets as breaks or fractures. Many of these features were described as either stained, weathered or leached. Staining, weathering and leaching are caused by percolating water through the concrete. One area in particular has a large number of these features. The area is in the middle wall in the downstream gate monolith, M-16. The core log representing the concrete core from hole M-3 shows that in the first 20 ft of concrete there are 6 of 16 breaks that are either stained, weathered or show signs of leaching. In the second 20 ft of concrete 6 of 11 breaks are either stained, weathered, or show signs of leaching. Assuming that 5 ft lifts were used to obtain the 40 ft of concrete and checking the log to see if any of the stained, weathered, or leached surfaces coincided with lift joints, we find that only 2 out of the 12 features coincide with lift joints.

One possible correlation between an interior and an exterior crack in monolith R-23 is discussed in Appendix IV.

Honeycombing

Honeycombing was found to occur in a few of the holes and existed in the older concrete. The honeycombing is a placement defect and not due to deterioration. However, the presence of honeycombing serves as structural weaknesses and does contribute to deterioration by allowing avenues of flow for sulphate bearing water or other types of acid waters. The lateral extent of the honeycombed areas cannot be ascertained. Several of the core logs and the borehole photography records indicate honeycombing at and near the concrete-foundation rock contact. Again the lateral extent cannot be determined.

Inadequate Concrete

The lowest compressive strengths reported in Reference 1 represent concrete recovered from drill hole M-3 which is in monolith M-16; M-16 is a downstream gate monolith in the middle wall. The strengths are 1450, 2100, and 1150 psi at el* 727.8, 719.8, and 715.8, respectively. Within 2 ft of each of these elevations either stained or weathered breaks exist. Honeycombed areas are present between the high and low elevations just mentioned. The extent of low strength concrete cannot be determined. However, the concrete within el 727.8 and 715.8 should be considered inadequate, based on present day design criteria. The concrete is in a critically stressed area and immediate remedial action is suggested.

The upper 2 to 6 ft of the original structure is now of low quality after 66 years of exposure to frost action. The same condition probably exists in the outer surface of the walls from above high pool elevation (726.9 ft) part of the way to lower pool el 718.7 ft. Beneath these top and wall surfaces the quality of the concrete is better, aside from scattered volumes of honeycomb, and probably of adequate quality for a gravity structure. On the other hand, it must be recollected that a concrete lock in a severe climate may manifest more cracking as a result of freezing and thawing near the top of the structure but local regions of high void content in the walls of the chanbers allow saturation of

---

* All elevations cited herein are in feet referred to mean sea level.

less porous adjoining concrete which will then be highly vulnerable to freezing and thawing of greater than average severity. This process can reasonably be considered as occurring in critical stress regions such as gate monoliths.

SECTION 5: LABORATORY TESTS

Introduction and Problem Statement

Physical property data for the backfill material, the foundation material, and the original and newer concrete were required for the structural investigation. Most of the material properties were obtained from laboratory tests conducted during this investigation; however, previously determined data for the concrete were incorporated where appropriate. The material properties required consisted of unit weights, compressive strengths, triaxial and direct shear strengths, and various elastic constants.

Backfill Material

Shear strength and unit weight values were desired for the backfill. In regard to shear strength, a wet sieve analysis of a portion of one of the samples indicated that there was too much granular material in it to permit a quick valid inplace shear test to determine strength. Nor was it feasible to test remolded material for strength since, as already mentioned, these tests would not be representative. Inplace shear strength tests were not considered due to money and time limitations. Therefore, no shear strength data could be obtained in the time allotted.

Two approaches were used to obtain a unit weight value for the backfill material. In one case, the unit weight of the two combined grab samples from hole M-7 was determined in its field moisture condition, and this value was modified to account for the large rock fragments that were not present. It was estimated that the larger rock would make up 10 percent of the total backfill material. The specific gravity of the rock fragments obtained during the sieving process had been determined, and a unit weight of the rock had been calculated. A unit weight for backfill was calculated as 90 percent of the measured unit weight of the backfill sample plus 10 percent of the calculated unit weight of the rock fragments.

The other method of obtaining a unit weight value was as follows:

a. The material passing the No. 10 sieve was defined as soil; its unit weight was estimated to be 90 lb/cu ft.

b. The amount of rock and amount of soil in the backfill sample were calculated from the partial sieve analysis. This value was modified to account for the estimated 10 percent of large rock fragments not present in the sample. This gave a value of 65 percent rock and 35 percent soil in the total backfill material. The unit weight of the backfill was then calculated as 65 $\pm$ 5 percent of the calculated unit weight of rock plus 35 $\pm$ 5 percent of the calculated unit weight of the soil.

To obtain a saturated unit weight value for the backfill, excess water was added to the sample and maintained for about 60 hours. The unit weight of this wetted material was then measured and a new value calculated from 90 percent of measured unit weight of backfill sample plus 10 percent of the calculated unit weight of the rock fragments.

A value for the field moisture content of the backfill was obtained by drying the sieved material passing No. 10 and using differences between as-received and dry weights. The moisture content was calculated based on the dry weight.

Concrete Property Tests

A limited number of 6-in.-diameter concrete cores that were recovered from the drilling effort in 1973 were received from the ORDL. Of the cores received, only three were suitable for compressive testing. One unconfined compression test and two triaxial tests were conducted in accordance with applicable portions of test methods CRD-C 19-72 and CRD-C 93-70,[2] respectively. Unit weights were obtained using measured volumes and weights of the cylinders. The tests yielded the following information:

a. Marginal verification of compressive strength, q_u, and unit weights, γ, presented in Reference 3.

b. Modulus of elasticity, E.

c. Poisson's ratio, ν.

d. Shear modulus, G.

Rock Property Tests

As mentioned earlier, the gray shale recovered from drill holes M-7 and M-8 comprised 95 percent of the core recovered at L/D 3 during the WES drilling effort. The core recovered in 1973 was not used for testing. The grayish shale that was tested is uniform, competent rock that is adjacent to the concrete under this structure in almost all cases, and its use for testing is considered appropriate. There were no unusual planes of weakness, such as bedding plane faults, detected in the shale from M-7 and M-8.

The unit weights were obtained using measured volumes and weights of the core. 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.[2] 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.

The direct shear test specimens were prepared according to applicable portions of the standard method of test for shear strength, CRD-C 90.[2] 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.[2]

Due to heavy work load schedules at WES when this program was initiated, the sliding friction tests could not be run. Since a coefficient of sliding friction value and a cohesion value were required for the stability analysis, in particular, a value for a concrete-foundation rock was needed; a multistage triaxial test was conducted. A piece of concrete core recovered from the Montgomery Dam during the drilling efforts of 1973 and a piece of shale from hole M-7, el 704, were used for testing.

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.

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 shale 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.

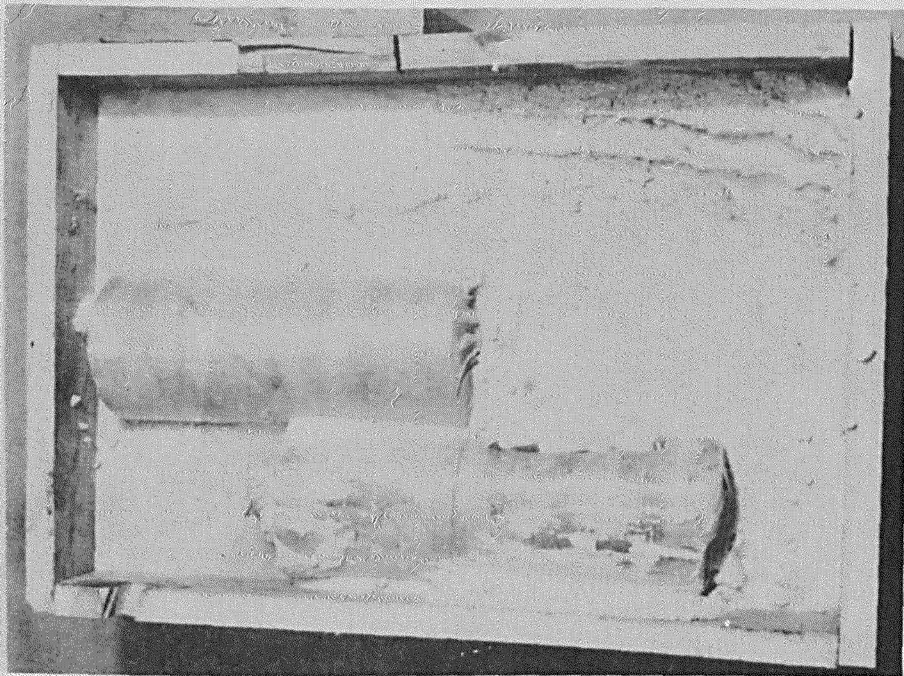
Petrographic Examination

The following four hand samples of rock from cores M-7 and M-8 were used for a petrographic examination:

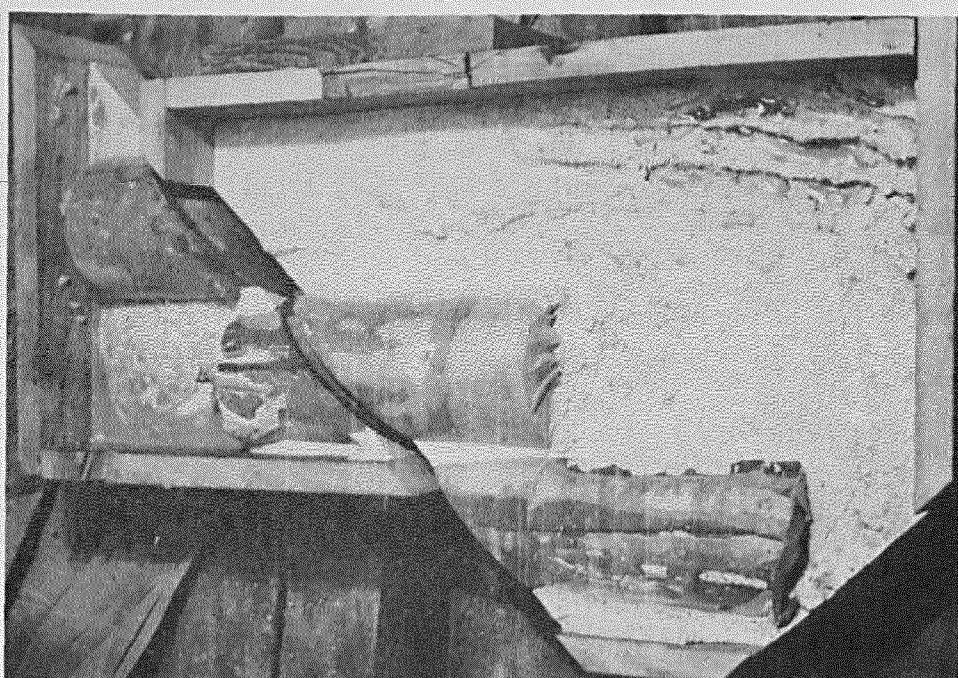
Hole	Depth, ft	Material
M-7	34.0	Grayish Shale
M-7	42.5	Grayish Shale
M-7	44.0	Carbonaceous Black Shale
M-8	45.0	Grayish Shale

The samples were allowed to dry, and the color was determined.[3] Dried pieces were placed in water to see if they would slake. The rocks were examined with a stereomicroscope.

Portions of each of the four rock samples were ground to pass a No. 325 sieve (44 μm). These samples were X-rayed as backpacked powders. In addition, slurries of these powders were placed on glass slides and allowed to dry; these powder films were then X-rayed in the air-dry condition, after heating to 300 C for one hour, and after treatment with glycerol. All of the X-ray patterns were made with an XRD-700 diffractometer using nickel-filtered copper radiation.



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

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 adequate resistance against overturning, sliding, and excessive base pressure exists. In this study, only one monolith of each typical configuration and loading was analyzed. The conclusions determined for these data are adequate for all monoliths.

In addition to the condition survey report (Reference 4), a field survey and examination of Locks and Dam 3 were conducted. The absence of protective wall armor has resulted in the walls, especially the upper guide wall, being gouged (Figure 6.1) by tows and construction joints being weathered and worn extensively (Figure 6.2). The concrete loss was not extensive enough to contribute substantially to stability considerations but is an important factor in stress analysis. The overall alignment was good except for the upper guard wall and the dam abutment; although a detailed examination of construction photographs shows that in the past there has been relative movement of some lock monoliths. The upstream guard wall has settled and rotated (Figure 6.3) approximately 6 in. toward the river. Settlement for structures of this type which are supported on timber cribbing is not unusual, and this case could lead to a structural failure. Lock personnel report visible movement in the guard wall as a result of large tow impacts. It is, therefore, possible that the wall could be overturned as the rotation toward the river increases. The dam abutment has a cant toward the river, but this movement seems to have stopped and is not, at present, as critical in stability as the upper guard wall.

Critical stability situations can exist for monoliths in the land wall where additions to these monoliths have resulted in construction joints. If adequate bond between old and new concrete at these joints is not maintained, the structure will not act as a monolith but will act as separate units. The units can be overturned much easier than the total monolithic structure because the effective base area is reduced and loads can be applied at the top of a unit by strut action (for example, through the structural slab) making them deficient in resistance to overturning.

Stability computations were made to give indications of how safe the monoliths are in relation to overturning, sliding, and base pressures. Conditions such as concrete cracking, spalling, and other distress areas which affect stress considerations more than that of stability are considered in Section VII. Critical stability problems which are mentioned above will be discussed in this section.

Results

The summary of the stability analysis is presented in Table 6.1. The stability analysis for typical monoliths is given in Appendix VI by Figures VI.1 through VI.21.

It is good to remember that the conclusions which follow for the stability analysis are based on several factors which are not conservative (See Appendix VI). Some of these factors are:

- a. Backfill saturation levels used in the calculations are not as high as normal design levels nor as high as can be expected after overtopping and fast dropping of pool levels.
- b. A lower bound at-rest-pressure coefficient is used.
- c. Concrete density of 150 lb/cu ft is used which is higher than the actual value.

The land wall monoliths do not meet the present-day criteria of desired safety against overturning as can be seen in Table 6.1. In fact, some of the force resultants fall outside the base of the monolith. This indicates that factors not dependable enough to be justified in good engineering design are keeping these monoliths stable. It is important that one does not relax the engineering criteria to include variables which are not dependable because during infrequent but special conditions failure could occur; it is just not good engineering. Some river wall monoliths are also not stable in overturning.

The safety factors against sliding are not adequate for the land wall monoliths, and if the lock is ever dewatered for maintenance, some are critical. This makes dewatering for maintenance of the lock unsafe. As old and deteriorated as this lock and dam is, it would be presumptuous to assume that dewatering will not be required, especially if the lock and dam is kept in operation for periods longer than necessary for replacement or complete rehabilitation. The safety factors against sliding are also inadequate in many cases for the middle wall, and river wall monoliths.

The stability analysis for various monoliths will be considered in detail. The monoliths, their numbering and stationing are shown in Figures 6.4 through 6.7. Monolith L-18 is representative of the upper guide wall monoliths. It does not have adequate resistance against overturning; in fact, for normal operation, when hawser pull is included and for the flood case, the resultant of the external loads falls outside the base. It is not even adequate for overturning when impact loads are considered. It does not have adequate safety factors against sliding. For normal operation and flood conditions, the safety factors are from 0.92 to 1.26 and should be a minimum of 4. The base pressures are not compressive over the total base and are excessive.

The upper landwall gate monolith (L-23) is adequate in overturning because of the added weight of the powerhouse. The safety factor for sliding is inadequate for both the normal operation and maintenance conditions. The base pressures are greater than the allowable 20 ksf.

Monolith L-32 is in the upper chamber on landside and shows typical construction of monoliths containing the filling and emptying flume. This monolith was checked for stability by considering it as a unit and by considering it as made up of separate units. It is reasonable that these landwall structures may act as separate units because when the lock was extended and additions added to the original monoliths, construction joints were made as shown in Figure VI.4 of Appendix VI. When L-32 is considered as a monolith, it is deficient in resistance to sliding. In figure VI.4, pages 1 through 4, of appendix VI, it is seen that if the monolith acts as separate units the riverside unit is totally inadequate for overturning and sliding. This is a realistic possibility and should be considered for possible lock failures in these land wall structures. The base pressures for this case are much larger than the allowable.

Monolith L-37 is a land wall chamber monolith which when considered acting as a unit, is adequate for overturning but can be deficient if construction joints cause it to act as separate units. Even as a unit it is very inadequate for sliding.

Monolith L-42 is a lower chamber land wall monolith. From core logs and construction photographs, it was determined that the as-built structure varied significantly from the planned construction. The stability analysis is computed for the geometry of the as-built monolith. The monolith is insufficient in resistance to overturning and sliding and the base pressure is excessive. The inadequacies for sliding become more critical and in some cases will become extremely critical if the lock is ever dewatered.

Monolith L-46 is a lower gate bay land wall monolith. The monolith was built of different geometry than is shown on the plans and is checked for stability by using the as-built section. It is inadequate in resistance to overturning; in fact, even for normal operation, the resultant falls outside the base. It does not have an adequate factor of safety against sliding and maximum base pressures are excessive.

Monolith L-55 is in the lower guide wall. Whether one considers the planned construction or the approximate as-built construction, the monolith does not have adequate resistance against overturning and is insufficient in factor of safety for sliding. The planned and the approximate as-built structures are analyzed and both presented in Appendix VI. It also has excessive base pressures. If deterioration at the lift joint occurs such that bond is lost there will be a critical situation for overturning.

Some middle wall and river wall lock monoliths are not adequate in safety factor against sliding. This is especially true for the river wall monoliths if this lock is ever dewatered for maintenance. Critical sliding situations will exist if the base of the monolith is located at or near a coal seam.

There are two monoliths which are supported above coal seams which are considered weaker in shear than the rock that is under most of the monoliths. The coal seam is at the concrete interface of monolith R-12 and 3 ft below monolith M-16. Monolith R-12 and monolith M-15 (close to monolith M-16) were checked for safety against sliding using lower bound values of $\phi = 15^{\circ}$ and $C = 0^{\circ}$.

Monolith R-12 is adequate (Figure VI.16, page 4 of 4) for normal operation, but if the lock is dewatered for maintenance, a critical sliding condition exists.

The coal seam does not affect the sliding of monolith M-15 more than it is affected at the concrete-shale interface.

The excessive bearing pressures which exist at the interface of the upper and lower timber cribbing sections of monolith R-2 (Section B-B, Figure 6.15) of the upper guard wall, especially when tow impact is considered, logically leads to the expectation of settlement and tilting of the wall toward the river. Such settlement and tilting has, in fact, occurred and, as has been previously stated, could lead to failure of this guard wall.

The sill and dam abutment are supported on piling. The abutment does not have adequate safety against either the horizontal or axial forces. It has excessive compressive force in piling on the riverside and large tensile forces in the landside piling. This could well explain why the abutment has tilted riverward. This tilting could continue if the soil from the landside yields such that it continues to exert the horizontal force for which it has proved a capability by the fact that the monolith has tilted. (See pages VI-95 to VI-101 of Appendix VI.)

Some of the worst cases of instability (the upper guard wall and the abutment) have actually shown physical signs of overturning. This substantiates the stability analysis and leads to the belief that other inadequacies can lead to failures.

In general, the stability of the monoliths is inadequate.

TABLE 6.1

Summary of Stability Analysis Results

Monolith	Cases Considered	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft: Minimum Allowable	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft: Actual	Sliding Safety Factor: Minimum Allowable	Sliding Safety Factor: Actual	Foundation Pressure, k/sf: Allowable	Foundation Pressure, k/sf: Actual
L-18	Normal Operation	2.50	0.87	4	1.26	20	19.31
	Normal Operating Neglecting Impact	2.50	-2.82	4	0.92	20	∞
	Flood Condition	1.67	-1.09	4	1.09	20	∞
L-23	Normal Operation	≈10.00	19.93	4	3.41	20	28.00
	Maintenance Condition	≈10.00	18.02	2-2/3	2.04	20	31.00
L-32	Normal Operation	8.25	7.75	4	2.07	20	6.68
	Normal Operation Neglecting Impact	2.50	7.46	4	1.98	20	6.94
	Maintenance Condition	1.67	5.93	2-2/3	1.66	20	9.06
L-32 w/construction joints	Normal Operation	3.75	0.28	4	1.43	20	134.05
	Normal Operation Neglecting Impact	3.75	0.96	4	1.50	20	39.10
	Maintenance	2.50	0.97	2-2/3	1.27	20	49.48
L-37	Normal Operation	7.50	10.69	4	2.72	20	4.83
	Maintenance Condition	5.00	8.82	2-2/3	1.74	20	7.00
L-42	Normal Operation	4.00	-0.06	4	1.64	20	∞
	Normal Operation Neglecting Impact	4.00	1.88	4	1.31	20	17.55
	Maintenance Condition	2.67	0.02	4	1.11	20	1760.00
L-46	Normal Operation	4.00	-0.17	4	2.29	20	∞
	Normal Operation Neglecting Impact	4.00	-1.25	4	2.01	20	∞
	Maintenance Condition	2.67	-2.08	2-2/3	1.16	20	∞

(Continued)

TABLE 6.1 (Continued)

Monolith	Cases Considered	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
L-55	Normal Operation $K_r = 0.5$	3.25	-2.25	4	2.05	20	∞
	Normal Operation Neglecting Impact	3.25	-3.39	4	1.89	20	∞
	Normal Operation $K = 0.8$	3.25	-7.61	4	1.30	20	∞
	Normal Operation Neglecting Impact	3.25	-8.76	4	1.24	20	∞
	Normal Operation $K_A = 0.33$	3.25	0.78	4	3.04	20	37.26
	Normal Operation Neglecting Impact	3.25	-0.37	4	2.70	20	∞
	Flood Condition $K_r = 0.5$	2.17	-2.09	4	2.62	20	∞
	Flood Condition $K_r = 0.8$	2.17	-7.29	4	1.63	20	∞
	Flood Condition $K_A = 0.33$	2.17	0.83	4	3.96	20	26.10
L-55	Normal Operation	2.75	-4.13	4	1.16	20	∞
	Normal Operation Neglecting Impact	2.75	-1.76	4	1.50	20	∞
	Maintenance Condition	1.83	-6.93	2-2/3	0.95	20	∞
M-2	Normal Operation	6.67	10.12	4	64.13	20	3.13
	Normal Operation Neglecting Impact	6.67	10.51	4	192.40	20	3.48
	Maintenance Condition	5.00	6.93	2-2/3	2.17	20	6.70
	Maintenance Condition Neglecting Impact	5.00	7.28	2-2/3	2.30	20	6.34
M-6	Normal Operation	6.67	6.94	4	13.90	20	5.39
	Maintenance Condition	5.00	13.95	2-2/3	7.81	20	6.86
M-15	Normal Operation	6.67	14.42	4	2.25	20	7.58
	Normal Operation Neglecting Impact	6.67	14.79	4	2.13	20	7.94
	Maintenance Condition	5.00	7.62	2-2/3	2.17	20	6.05

(Continued)

TABLE 6.1 (Continued)

Monolith	Cases Considered	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft: Minimum Allowable	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft: Actual	Sliding Safety Factor: Minimum Allowable	Sliding Safety Factor: Actual	Foundation Pressure, k/sf: Allowable	Foundation Pressure, k/sf: Actual
M-20	Normal Operation	5.33	5.29	4	5.1	20	5.65
	Flood Condition	4.00	8.13	4	∞	20	2.42
	Maintenance Condition	4.00	9.15	2-2/3	6.56	20	4.94
M-25	Normal Operation	6.00	6.95	4	9.03	20	5.10
	Normal Operation Neglecting Impact	6.00	6.59	4	8.30	20	5.46
	Maintenance Condition	4.50	10.07	2-2/3	9.00	20	3.48
	Maintenance Condition Neglecting Impact	4.50	9.74	2-2/3	9.83	20	4.24
R-2	Normal Operation Section AA	3.33	3.80	4	2.94	20	24.18
	Normal Operation Neglecting Impact	3.33	4.89	4	27.40	20	14.99
	Normal Operation Section BB	3.33	2.69	4	3.74	20	939.93
	Normal Operation Neglecting Impact	3.33	4.77	4	34.93	20	170.01
	Normal Operation Section CC	4.83	4.77	4	6.11	20	23.68
	Normal Operation Neglecting Impact	4.83	7.05	4	57.00	20	17.63
	Flood Condition Section AA	2.50	5.00	4	∞	20	8.62
	Flood Condition Section BB	2.50	5.00	4	∞	20	61.13
	Flood Condition Section CC	3.62	7.36	4	∞	20	13.86
R-12*	Normal Operation	6.67	9.50	4	33.18	20	4.27
	Normal Operation Neglecting Impact	6.67	9.83	4	99.55	20	3.90
	Maintenance Condition	5.00	15.14	2-2/3	0.70	20	12.22

* Borehole M-1 shows coal at the monolith base. Sliding factors of safety were calculated using lower bound values of $\phi = 15^{\circ}$ and $C = 0^{\circ}$.

(Continued)

TABLE 6.1 (Continued

Monolith	Cases Considered	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
R-15	Normal Operation	6.00	14.79	4	2.46	20	11.61
	Normal Operation Neglecting Impact	6.00	15.55	4	2.24	20	15.21
	Maintenance Condition	4.50	17.06	2-2/3	1.25	20	47.52
R-21	Normal Operation	6.00	6.69	4	2.19	20	5.17
	Normal Operation Neglecting Impact	6.00	7.62	4	2.39	20	4.27
	Maintenance Condition	4.50	14.10	2-2/3	2.23	20	11.68
R-23	Normal Operation	6.67	4.63	4	2.39	20	7.99
	Normal Operation Neglecting Impact	6.67	5.17	4	2.53	20	7.16
	Maintenance Condition	5.00	12.46	2-2/3	1.96	20	5.08

Figure 6.1. Gouged Upper Guide Wall

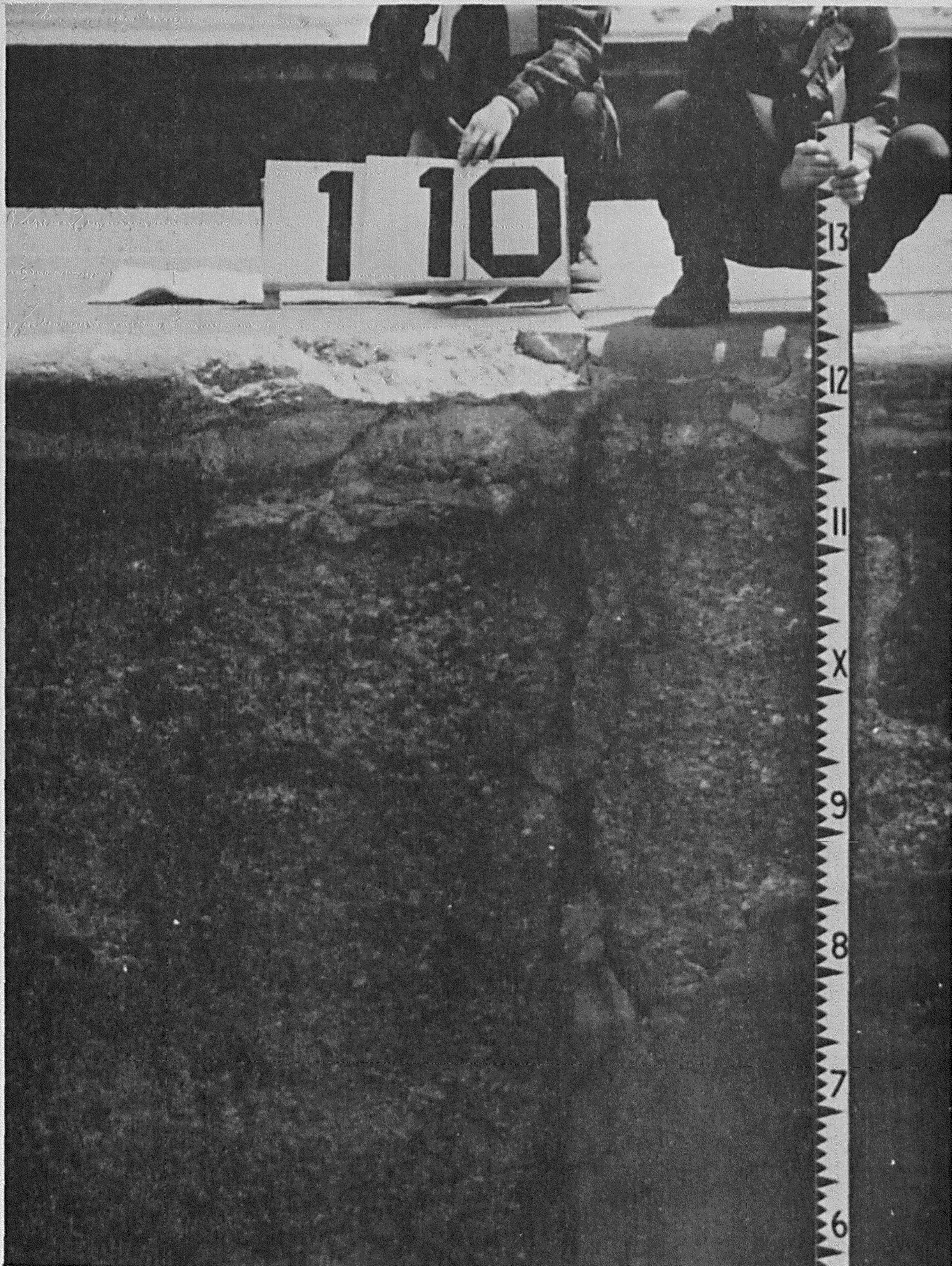


Figure 6.2. Deteriorated Construction Joint

Figure 6.3. Tilted Upper Guard Wall

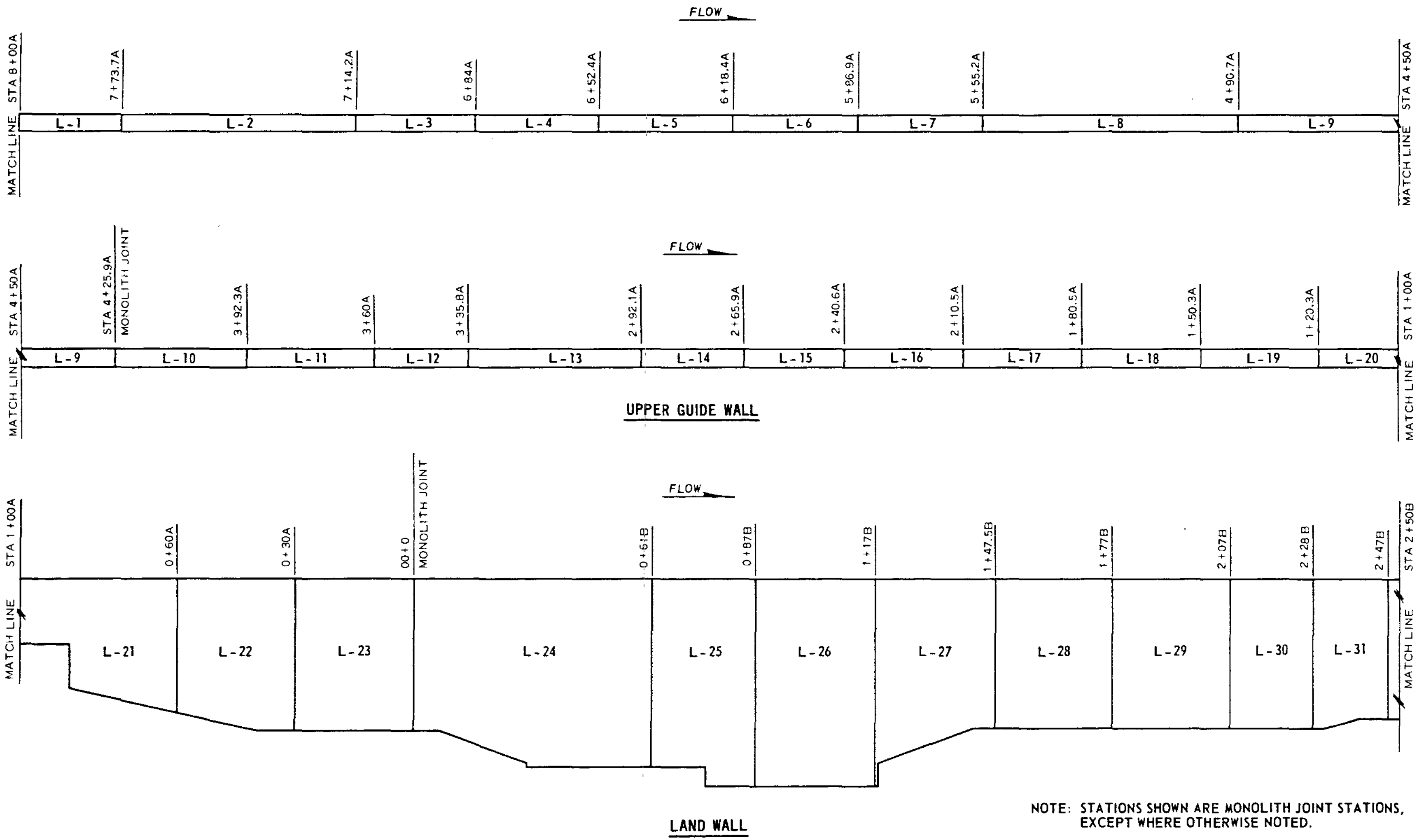


Figure 6.4. Top views of upper guide and land wall showing monolith numbering and stationing, Locks and Dam 3, Monongahela River

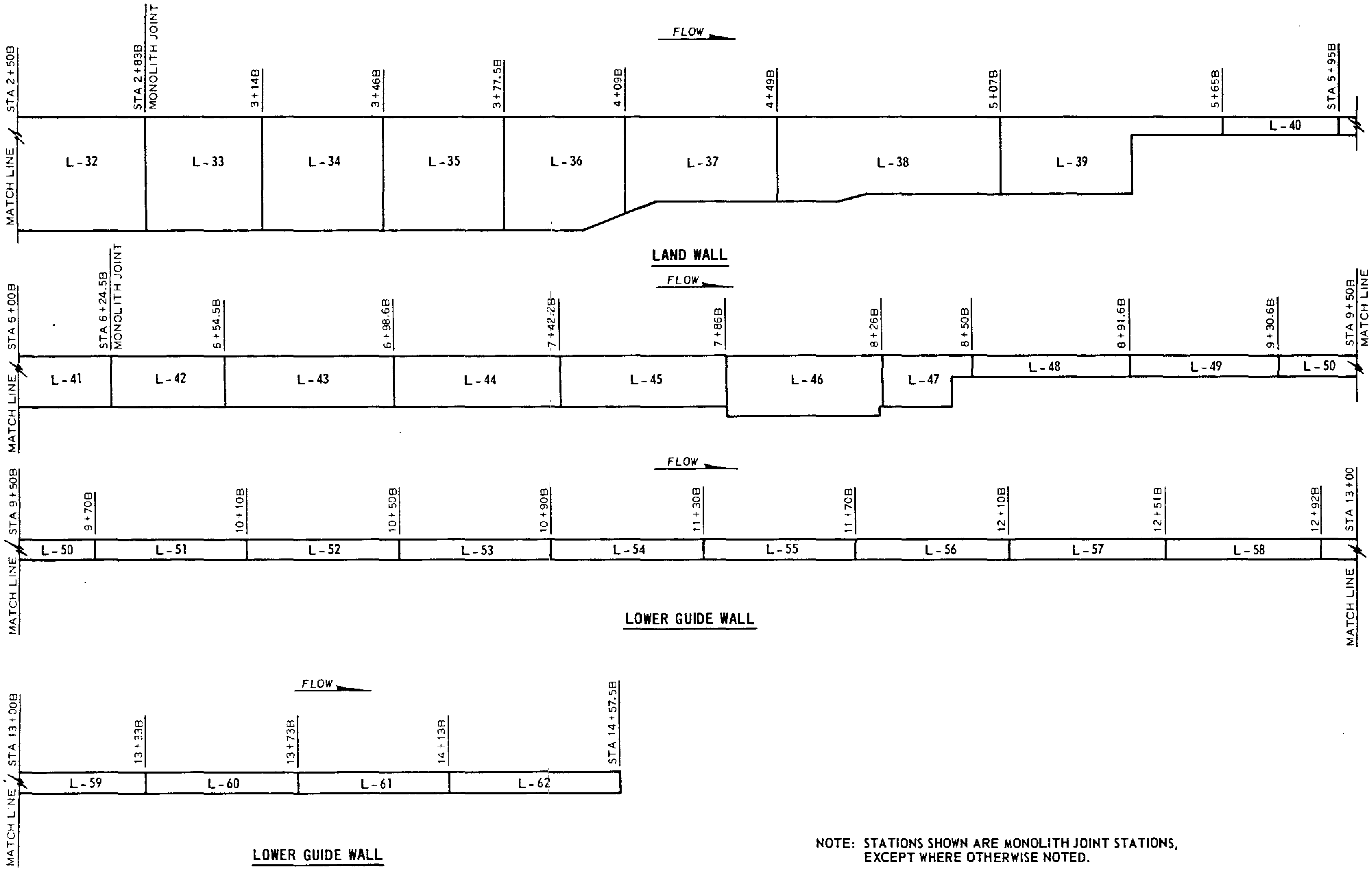


Figure 6.5. Top views of land and lower guide wall showing monolith numbering and stationing, Locks and Dam 3, Monongahela River

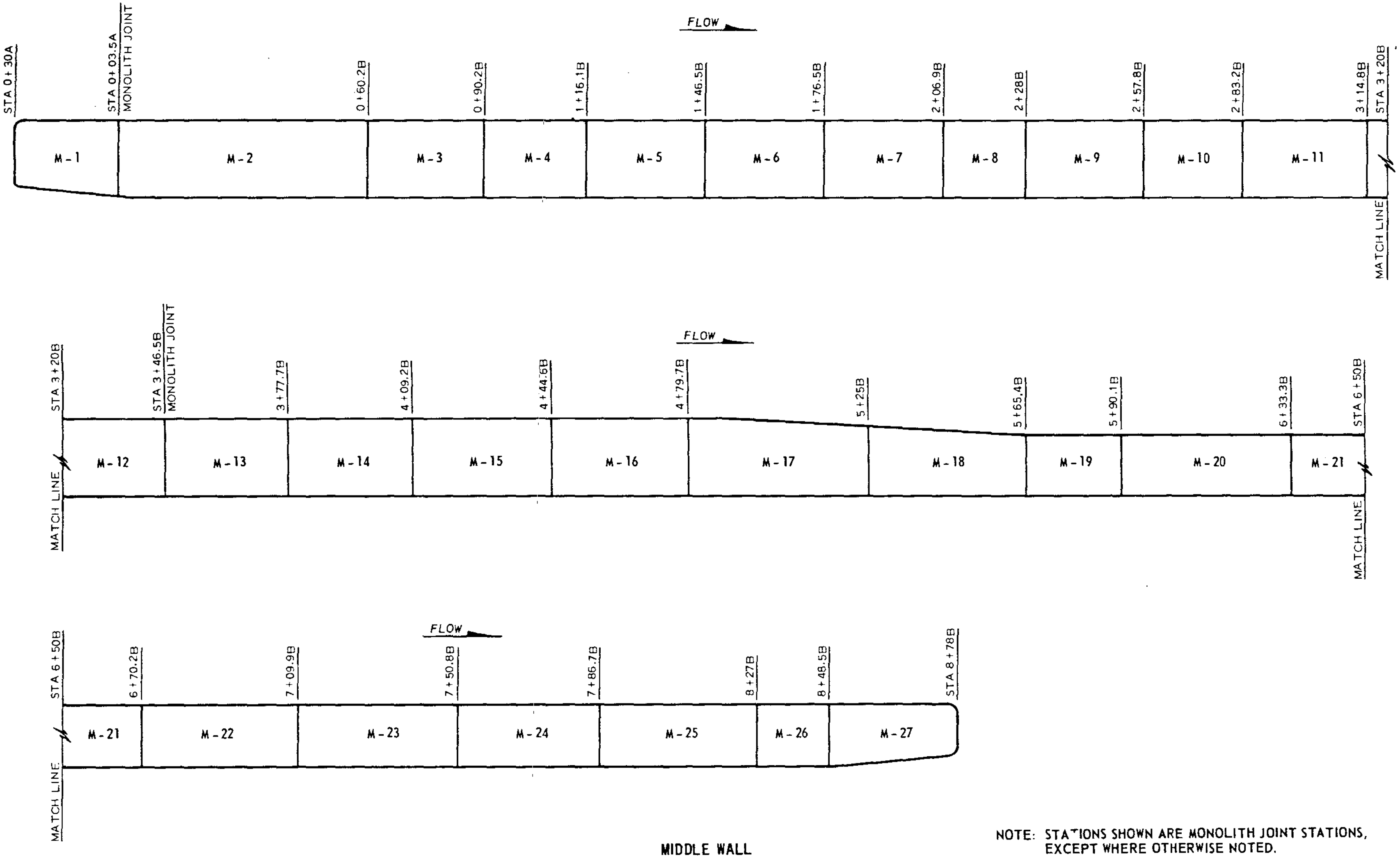


Figure 6.6. Top view of middle wall showing monolith numbering and stationing, Locks and Dam 3, Monongahela River

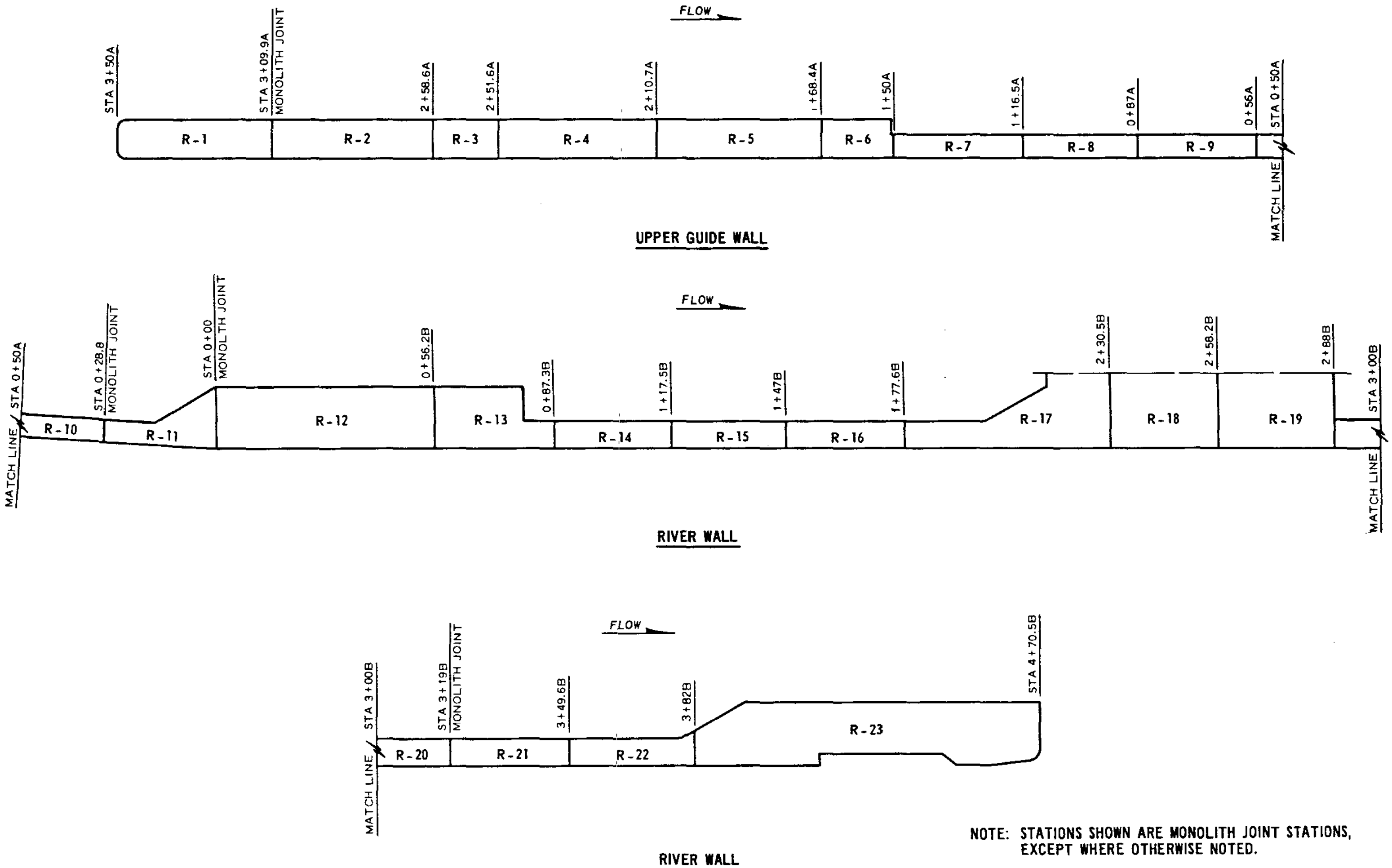


Figure 6.7. Top view of upper guide and river wall showing monolith numbering and stationing, Locks and Dam 3, Monongahela River

SECTION 7: STRESS ANALYSIS

Introduction and Problem Statement

In the structural evaluation of L/D 3, a two-dimentional plane strain finite element analysis as well as conventional design was used to determine stresses within selected structural monoliths.

It is becoming increasingly important to understand the phenomenon of stress flow 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 our 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, they are a supplement, making a combination which is much better than either separately. It is important to consider stress distributions in areas of stress concentration when evaluating old structures which have cracked and are deteriorating.

Critical Stress Areas

The stress analysis will be of maximum benefit in considering critical concentrations in areas which may, under deteriorated conditions, begin affecting the overall operation of the lock. Some areas of concern are:

a. Gouges in the upper guide wall due to tow alignment (Figure 6.1).

b. Stress concentrations within the structural slab over the filling and emptying flume of the land wall monoliths (Figure 7.1).

c. Stresses at the outer ends of the bottom slab of the filling and emptying flume of the land wall monoliths.

d. Gate anchor locations where cracking and deterioration will accelerate unless rehabilitation is undertaken (Figure 7.2).

While it is true that the locks have functioned for more than 70 years, it is not so much the previous condition of the lock that is of concern, but it is what may logically be expected in the future, considering the cracking and deterioration of concrete, especially in critically stressed regions where accelerated deterioration is inevitable if no corrective action is taken.

The gouges in the upper guide wall are as much as 3 ft deep and under certain loading conditions can cause overstressing in tension and possibly failure. If the consideration of stress in the upper guide wall is made by formulating and solving the depth of slit or gouge required to produce excessive tensile stress in good quality concrete, some idea of gouge effect can be determined. The gouges are considered at elevations of either the first three steps at the back and from the top of the guide wall. The calculations are given for maximum gouge depth at the first step and results from similar calculations tabulated for the next two lower steps.

The free body diagram and calculations at the first step are as follows:

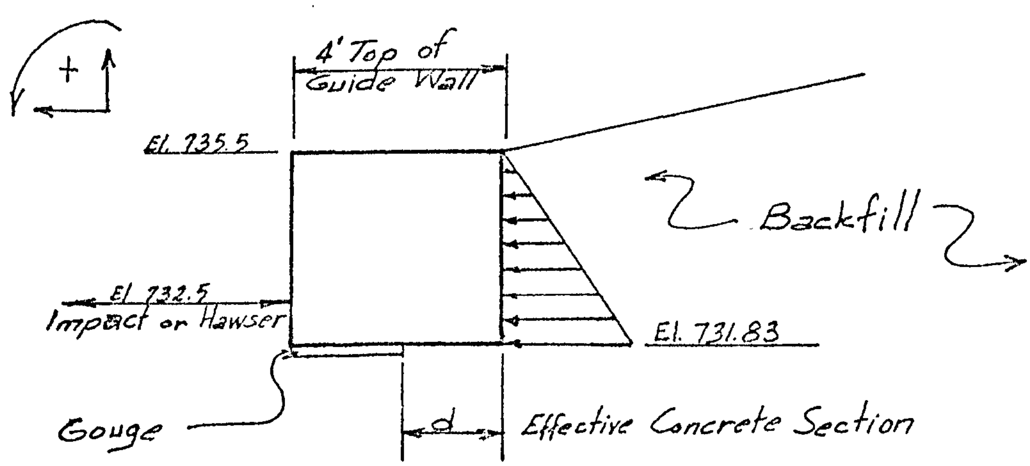


Table 7.1

Force and Moment Calculations for Stress Analysis

Item	Calculation	F_V	F_H	$\bar{y}$	M in-lb
Soil	$67.5 \frac{(3.67)^2}{2}$		+455	0.45(3.67)(12)	+9,017
Impact			-4000	(0.67)(12)	-32,160
Hawser			+800	(0.67)(12)	+6,432
Concrete	4(3.67)(150)	-2202		$24 - \frac{d}{2}$	

$$\sigma = \frac{P}{A} \pm \frac{Mc}{I} = \frac{P}{A} \pm \frac{c}{I} \quad (\Sigma M) = \frac{P}{d(12)} \pm \frac{d/2(\Sigma M)}{\frac{12d^3}{12}}$$

$$= \frac{P}{12d} + \frac{0.5(\Sigma M)}{d^2}$$

$$\sigma_{ultimate} = 3640 \times 10\% = 364 \text{ psi}$$

With Impact

$$364\ d^2 = \frac{-2202d}{12} + 0.5\ (9017 - 32,160 + (2202(24 - \frac{d}{2}))$$

$$364\ d^2 = -734d + 14,853$$

$$d^2 + 2.02\ d - 40.8 = 0$$

$$d = \frac{-2.02 \pm \sqrt{(2.02)^2 + (4)(40.8)}}{2} \approx 5.5''$$

Gouge can be 42.5"

With Hawser Pull

$$364\ d^2 = -734\ d + 0.5 \quad 9017 + 6432 + 2202(24 - \frac{d}{2})$$

$$d^2 + 2.02\ d - 93.8 = 0$$

$$d \approx 9''$$

Gouge can be 39"   Governs

Table 7.2

Gouge Depths for Overstressing in Tension

Gouge Position	Elevation ft	Minimum Gouge Depth for 360 psi Tensile Stress in Concrete, in.
Step 1	731.83	39
Step 2	728.16	44
Step 3	724.5	49

The above calculations considered the hawser pull and impact loads equally distributed along a 30-ft monolith. Since these loads are concentrated over a very small section, the guide wall could sustain damage because of gouges. This is especially true if transverse cracks cause short sections of the guide wall to be effective in resisting the applied loads. In fact, after this analysis and conclusions had been made, it was learned from communications with Pittsburgh District personnel that a failure of this type had occurred by tow impact at the end of the lower guide wall.

The slabs over the filling and emptying flume of the land wall monoliths are badly cracked and deteriorated (Figure 6.2) and probably cannot be feasibly rehabilitated making it safe for normal operation of men and equipment.

The location of miter gate anchors is cracked, and accelerated deterioration is taking place through leaching and freezing-and-thawing action. Due to the actions of cracking, weathering, leaching, and freezing and thawing, the concrete is of questionable quality in the initial 2 to 6 ft beneath the exposed surface, but much of the critical stress areas are in this 2- to 6-ft region. The stress problems and hence effects on normal operation are going to be in regions as have been discussed: gouged sections, structural slab over filling and emptying flume, gate anchors, etc. This report considers only the stability and stress analysis of the concrete monoliths and does not consider deterioration of machinery or any other operational components of the locks or dam.

Finite Element Analysis

The finite element analysis will be generally conducted as described below:

a. By using an example problem:

   (1) Examine the concept of uplift at the base of the monoliths.

   (2) Make some checks of agreement between stresses from conventional calculations with those obtained from finite element calculations.

b. Study areas of tensile stress under possible loading conditions.

c. Use monolith M-6 to study the extent of foundation needed to produce negligible effect on the stresses and displacements within the structure.

d. Using monoliths L-23 and L-32 study stress concentrations in in the slabs over and under the filling and emptying flume of the land wall monoliths.

e. Study the effect of concentrated gate loads using L46.

f. Analyze stress in a river wall monolith.

g. The condition of the upper guard wall will be assessed from the stability analysis and good engineering judgment.

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 properties precisely and the fill is random making it impractical to carry the material definition to the actual situation. The vertical and horizontal 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 and 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.

The submerged weight of the backfill material was calculated from $\gamma_{sub} = 5/8\ \gamma_{dry}$ which is a valid assumption for calculations when the specific gravity is 2.7 as it is in this case (Reference 5). The drained weight of the backfill was about 135 lb/cu ft, and using the above calculations assuming $\gamma_{dry} = \gamma_{drained}$, the saturated weight will be 148 lb/cu ft.

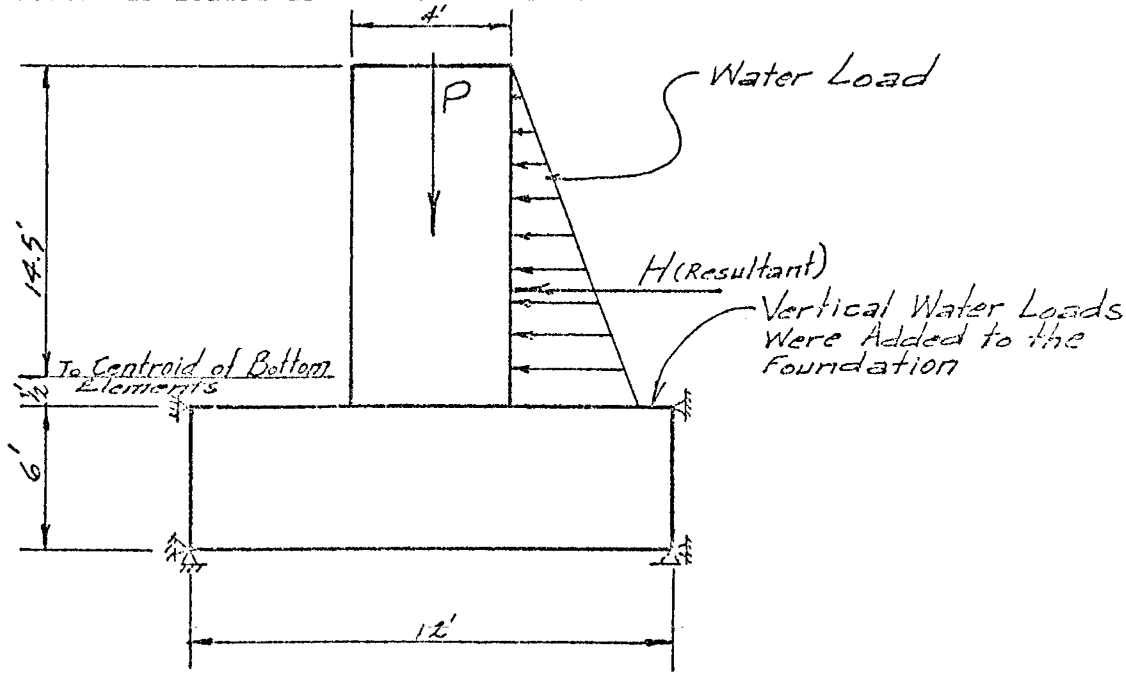
One consideration which must be made in the stress analysis is the effect of uplift on the base of the monolith. In certain cases, the effect will be negligible, but in others, it could be substantial; therefore, the effect must be included. The important concept concerning uplift is that it is a support 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 from top to bottom of 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.

An example problem of a general monolith shape was analyzed with and without uplift forces. A reasonable way to handle the uplift is to put a slit 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. The finite element grid stress and displacement plots for the example problem considering no uplift is given in Appendix VII by Figures VII.1, 2, and 3, respectively. The grid stress and displacement plots from calculations considering uplift are given in Appendix VII by Figures VII.4, 5, and 6, respectively. The uplift made little difference in this particular structural configuration and loading (Table 7.3, page 7.8)

Conventional stress calculations were made and compared to the finite element results. The calculations were made for elements 57 and 60 which are close to the base of the structure and for elements 33 and 36 at a higher elevation. The conventional calculations were obtained by using the standard formula:

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

The structure was loaded as illustrated below:



The distance from the top of the structure to elements 57 and 60 is 14.5 ft. The conventional calculations are computed as follows:

$$H = -\frac{(14.5)^2\ (62.4)\ (1)}{2} = -6560 \text{ lb.}$$

$$M = -\ (6560)\ \frac{(14.5)}{3}\ (12) = -380{,}480 \text{ in.-lb}$$

$$\frac{P}{A} = -\ \frac{(4)\ (14.5)\ (1)\ (150)}{(4)\ (12)\ (1)\ (12)} = -15.1 \text{ lb/in.}^2$$

$$f = -\ \frac{P}{A} \pm \frac{Mc}{I} = -15.1 \pm \frac{(-380{,}480)\ (1.5)\ (12)\ (12)}{(12)\ (48)^3}$$

$$= -15.1 \pm 61.9$$

$$f_{57} = -15.1 - 61.9 = -77.0 \text{ psi}$$

$$f_{60} = -15.1 + 61.9 = 46.8 \text{ psi}$$

The calculated values from the finite element stress analysis which give the total stress and displacement fields are available. The results of the finite element analysis compare well with the above computations. The calculations for vertical stresses in elements 33 and 36 were computed in a manner similar to that above and the results and comparisons are given in Table 7.3.

Table 7. 3

Comparison of Conventional and Finite Element Computations

	Stress, psi		
		Finite Element Calculations	
Element	Conventional Calculation	Uplift	No Uplift
57	-77.0	-84.69	-84.47
60	46.8	46.73	46.66
33	-21.3	-21.69	-21.69
36	3.6	3.99	3.99

The stresses given in the finite element computations follow the nomenclature given in Figure 7.3 below.

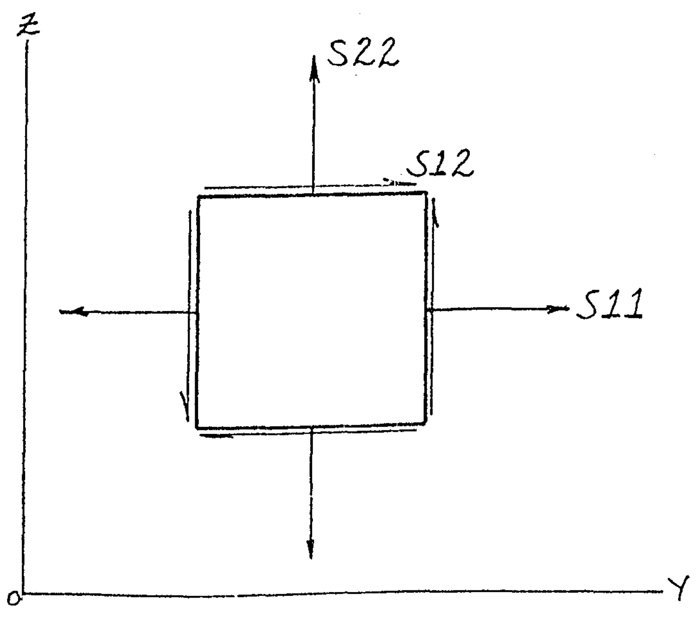
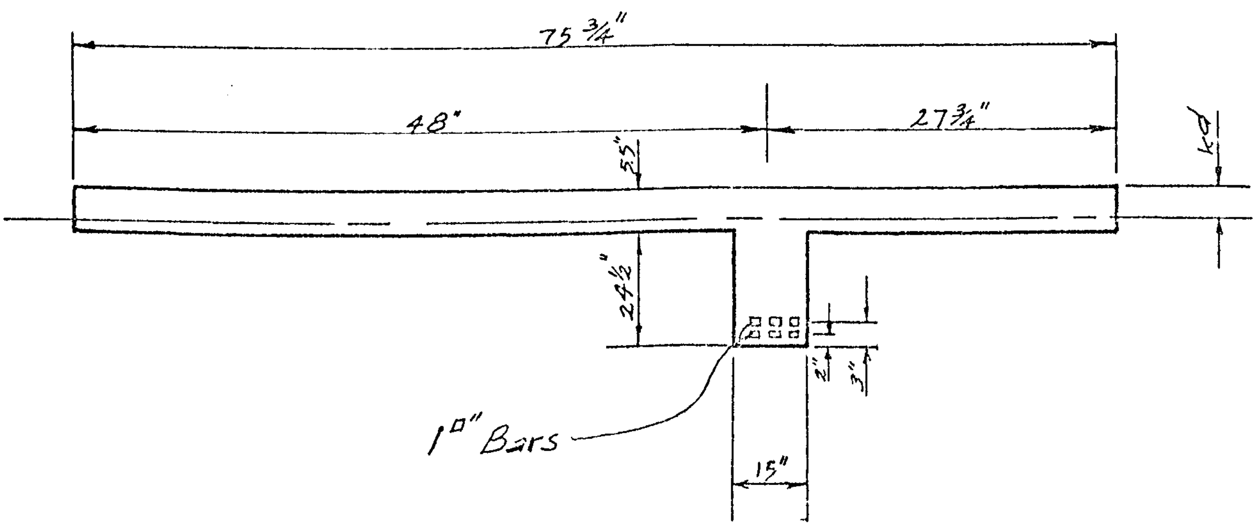


Figure 7.3. Stress Nomenclature

The stress component which compares with the conventional calculations is S22. The finite element calculations are almost self-explanatory because it gives the input data as well as the total stress and displacement on all elements. In other words, much more stress information is given than that of the stress component S22.

The stress flow plots 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 indicates compression. The displacement plot shows the vertical and horizontal displacement components at the original node position.

The two-dimensional stress analysis of sections which cross the filling and emptying flume on landside must take into account the variation in slab section parallel to the lock along the top of the flume. This is done by using the flexural capacity of the slab extending half-way between beams either side of the beam at which the specific section is being considered. This section is used to obtain the depth of a 1-ft rectangular section with equivalent per foot flexural capacity of the original slab section. This is done for monolith L-23 and monolith L-32. The calculations for monolith L-23 are given below and monolith L-32 is proportioned in the same manner.



Determine kd

$$(75.75)kd\ \frac{kd}{2} - (8)\ (6)\ \ 27 - kd\ = 0$$

$$37.875\ kd^2 + 48\ kd - 1296 = 0$$

$$kd^2 + 1.2673\ kd - 34.2178 = 0$$

$$kd = \frac{-1.2673 + \ (1.2673)^2 + (4)\ (34.2178)}{2}$$

$$= 5.25''$$

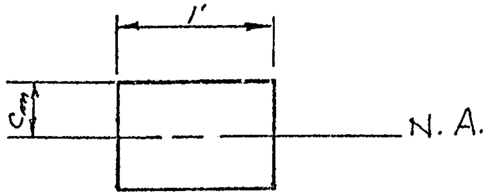
Determine moment of inertia of total section

$$I_{actual} = I_A = \frac{75.75\ (5.25)^3}{3} + 48(27 - 5.25)^2$$

$$= 26{,}361 \text{ in.}^4$$

Let moment of inertia of the model be represented by I_m

Then



$$I_m = \frac{12\ (2C_m)^3}{12} = 8\ C_m^3$$

$$\frac{(I_m) \text{ per foot}}{(C_m) \text{ per foot}} = \frac{(I_A) \text{ per foot}}{(C_A) \text{ per foot}}$$

$$8C_m^2 = \frac{26{,}361}{(5.25)\ (6.3125)}$$

or

$$= \frac{26{,}361}{(22.75)\ (6.3125)}$$

C_m = 9.97" or 4.79" depending on which C_A is used.

Use the depth of slab over the filling or emptying flume as that calculated with C_A to the outer concrete fiber. The slab depth over the filling and emptying flume should then be: $d = 2C_m = 9.58''$.

Another consideration in a finite element analysis is grid size for modeling the actual structure. Based on experience, a grid size was selected which was certain to give good results. In actuality, the element size could probably have been larger and still obtain adequate results. If time had permitted, results from various grid sizes could have been compared for selected monoliths to define optimum grid size.

Diagrams presenting the monolith loadings as used in the stress analysis are presented in Figures 7.4 through 7.9. For convenience in reading the report, 8- by 10-1/2-in. grid size plots are shown in this article giving areas of stress concentrations. These stress concentration plots are then presented in separate figures. Larger size grids are given in Appendix VII in case reference to actual node and element numbers is desired.

The grid depicting the areas of maximum stress for monolith L-18 is presented in Figure 7.10. These areas of stress concentration are then shown in Figures 7.11 and 7.12. Several important concepts become clear when the stresses in L-18 were studied. One concept is that the normal conventional design does not give maximum stress values. The S22 component is usually obtained, and it is generally not maximum or minimum principal stress. For example, when considering tensile stress in elements 64, 71, 78, 85, 92, 93, and 101, the maximum stresses are from 7 to 86 percent more than the S22 value. This shows that conventional computations can be misleading when obtaining stresses for design. This could cause the gouge depths necessary for failure in this upper guide wall (Table 7.1) to be reduced.

Another important concept is that the finite element stresses which are given in this report are for the centroid of the elements. The normal stresses at other points in the element, for example at its sides, may be much larger than those at the centroid. In other words, the center of elements are selected points in the monolith and other points may have larger stresses. For example, the maximum tensile stress at the side of element 453 is approximately 23 percent larger than the maximum tensile stress at its centroid.

The location of the compressive stress concentrations are indicated in Figure 7.10 and is given in Figure 7.12. The maximum compressive stresses in L-18 are low ($\approx$550 psi) when considering good quality concrete but the maximum tensile stresses are higher than the allowable even for good concrete. The allowable tensile stress for 4000-psi concrete is about 102 psi where L-18 has tensile values over 300 psi.

It is important to keep in mind that the conventional calculations, because of calculating a different component of the stress tensor, can vary significantly from the maximum finite element computation. Also, in areas of stress concentrations (for example, at the steps in the backs of the monolith) the values from the finite element analysis differ significantly (15 percent or more) from conventional calculations. This is true because conventional design does not give the total stress distribution; therefore, the finite element analysis is a definite advantage in these calculations. The above discussion considers good quality concrete; although compressive and tensile stresses will become much more significant if the concrete is deteriorated as it is in most of Locks and Dam 3.

There are shear stresses of approximately 100 psi in and at the extremes of the monolith base which in deteriorated sections could become important as a failure mode. The amount of foundation used under L-18 is adequate because the stresses at the lower boundary are decreased significantly from those at the base of the structure. The displacement plot for L-18 is given in Figure VII.8 of Appendix VII. The benefits obtained from the displacement plots are that the total fields of x and y node point translations are presented. The relative displacement is significant to stress and not the total displacement; therefore, the displacement plots have values only as stated above.

The grid for L-23 (Figure 7.13) shows the region of critical stress in the structural slab over the filling and emptying flume. This stress presentation is given in Figure 7.14. The large grid for L-23 is given in Appendix VII by Figure VII.9. The displacement plot is given in Figure VII.10 of Appendix VII. The stress plot for only the case of normal operation is given. The reason for this is that the stresses for the dewatered or maintenance condition are about the same as those for normal operation.

Tensile stresses of over 500 psi are produced in the upper part and at the ends of the structural slab. There is no reinforcing in this area; therefore, the tensile stresses are way above the allowable. The high tensile stresses of over 600 psi in the lower part of the structural slab can be carried by the steel unless this steel rusts and becomes ineffective. Areas of steel are now exposed and rusting. The tensile stresses on the riverside of the structural slab will only exist if the slab is compressed tight enough against the monolith to cause the construction joint to be ineffective and allow shear forces to carry moment and, therefore, tensile stresses into the slab. In fact, the slab is compressed against the wall causing spalling; therefore, the above possibility is realistic.

As deterioration continues one can expect this slab to become hazardous for even light loading.

Monolith L-32 is very similar to L-23 and the maximum stresses in the structural slab are approximately half those in L-23. For this reason stresses and displacement are not presented for L-32. The grid which was used in the analysis of L-32 is given in Figure VII.11 of Appendix VII.

The grid of L-46 depicting the critical stress areas is shown in Figure 7.15. These depicted areas "E" and "F" are presented in Figures 7.16 and 7.17, respectively. The grid and displacement vector plots are given in Appendix VII by Figures VII.12 and VII.13.

The land wall lower gate bay monolith, L-46, has tensile stresses as high as 430 psi at the beginning of the hanging wall on the river face. In the hanging wall on the river face toward the base of the monolith, tensile stresses are as high as 520 psi. The land wall gate loads push the monolith and produce tension by beam action in the river face of the monolith. These tensile stresses are above the allowable and can cause structural problems especially as the concrete deteriorates. It is good to keep in mind that in some cases the maximun tensile stresses are not reflected in the stress vector plots unless it occurs at the centroid of the element. Compressive stresses are as high as ≈1200 psi at the back and base of the monolith.

The stresses as distributed in three dimensions will relieve these high values to some degree but this relief can only be determined by a three-dimensional finite element stress analysis. Shear stresses in portions of the hanging wall are as high as 190 psi which could cause structural problems in this old structure.

The middle wall has a pronounced degree of concrete deterioration; therefore, monolith M-6 was selected for stress concentration studies. Also, varying amounts of foundation was used with M-6 in the finite element analysis to examine the influence of foundation extent on structural stresses. The analysis of M-6 was made with no foundation, structure plus some foundation (first addition) and structure plus even more foundation (first addition plus second addition). Stresses are presented for the analysis of structure and largest foundation addition. The grid showing the area of greatest stress concentration is presented in Figure 7.18 and the stress vector plot for this area is given in Figure 7.19. The grids for structure, structure plus first addition, and structure plus first addition plus second addition are given in Figures VII.14 through VII.16.

The highest tensile stresses are approximately 80 psi and compressive stresses are in the range of 100 psi. Tensile stresses of this magnitude can cause cracking in deteriorated concrete. The highest tensile stresses will be larger than 80 psi in isolated regions because only a per foot impact load is applied and in actuality it is concentrated over a small area of the monolith.

A comparison of stresses from the three analyses with varying amounts of foundation is given in Table 7.4. If no foundation is used the structure stresses can be affected considerably. The stresses for the two varying amounts of foundation do not vary considerably and they give a concept of the amount of foundation needed for a reasonable finite element analysis. No attempt will be made to specifically define the foundation needed because this will vary with structural geometry and loading.

Stresses in the bottom slab of the filling and emptying flume of the land wall monoliths and at gate anchorages were not critical.

No displacement plots for M-6 are shown because enough of these have been given to give the general concept of displacement of the lock monoliths.

The grid of R-15 depicting the critical stress area is shown in Figure 7.20. The stress plot for this area is presented in Figure 7.21. The larger grid is given in Figure VII.17. The largest tensile stresses of approximately 120 psi occur at the top and to the sides of the lower culvert. It is common that locks have cracks in these upper corners.

In general, there are tensile stresses in critical regions of various monoliths which are excessive for even good quality concrete. The compressive stresses indicate magnitudes that would not be considered excessive for sound, uncracked concrete; however, for badly deteriorated concrete, they are excessive.

Table 7.4

Comparison of Tensile and Compressive Stresses of Selected Elements in Monolith M-6 With Varying Amounts of Foundation in the Analysis

Element No.	Structure Only		Structure + First Addition		Structure + First Foundation + Second Foundation	
	Maximum psi	Minimum psi	Maximum psi	Minimum psi	Maximum psi	Minimum psi
426	-16.9	-81.0	-19.1	-89.7	-19.7	-91.2
429	69.1	-42.0	76.6	-46.8	76.6	-49.9
463	0.2	-44.4	0.0	-37.4	7.8	-38.8
511	-1.7	-77.1	-17.8	-104.4	-20.2	-112.2
546	-8.5	-46.3	-18.0	-92.9	-16.4	-90.0

Figure 7.1. Cracking and Deterioration in Structural Slab Over Filling and Emptying Flume

Figure 7.2. Deteriorated Locations at Gate Anchorages

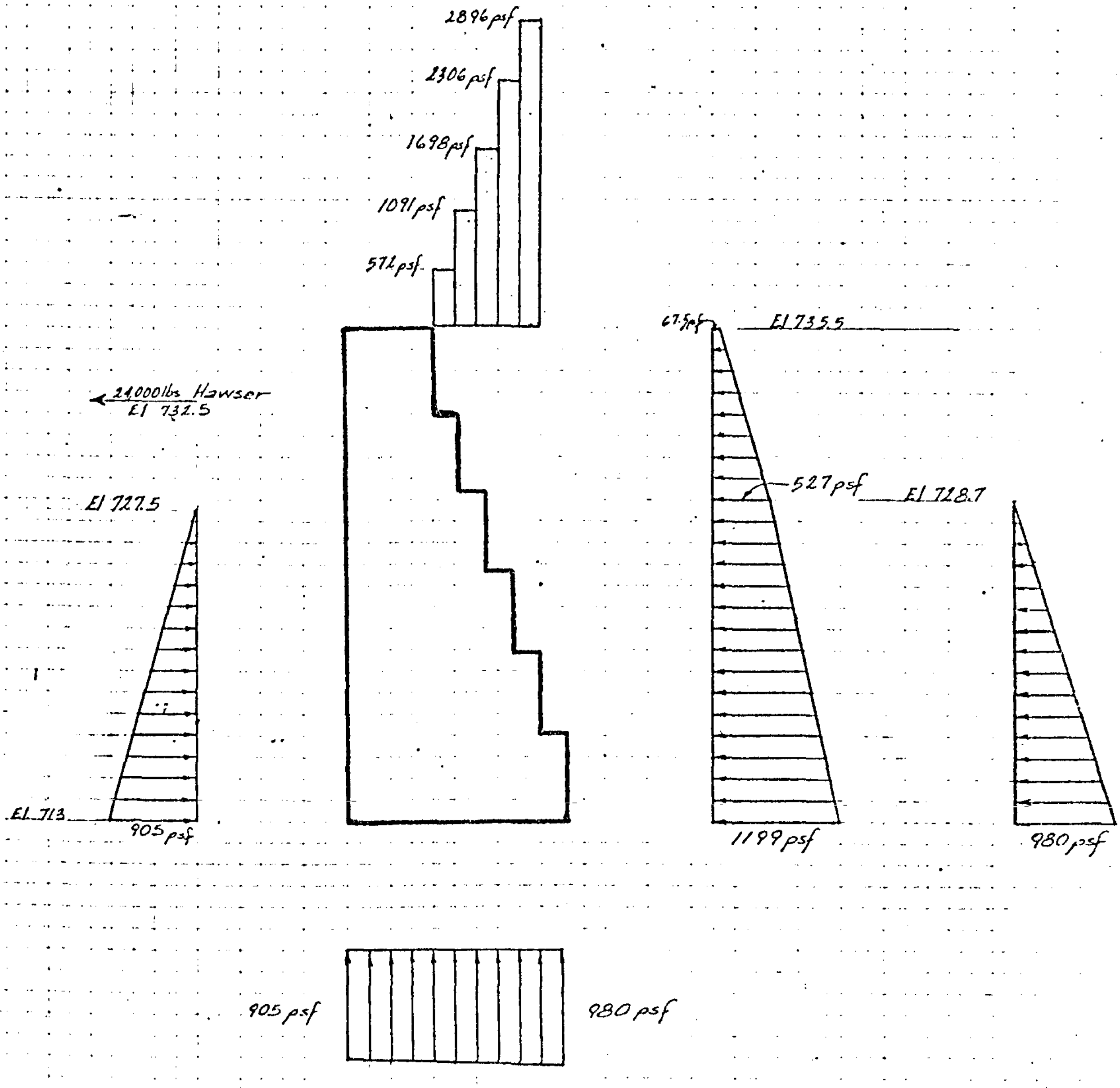


FIGURE 7.4 LOADING FOR MONOLITH L-18, STRESS ANALYSIS

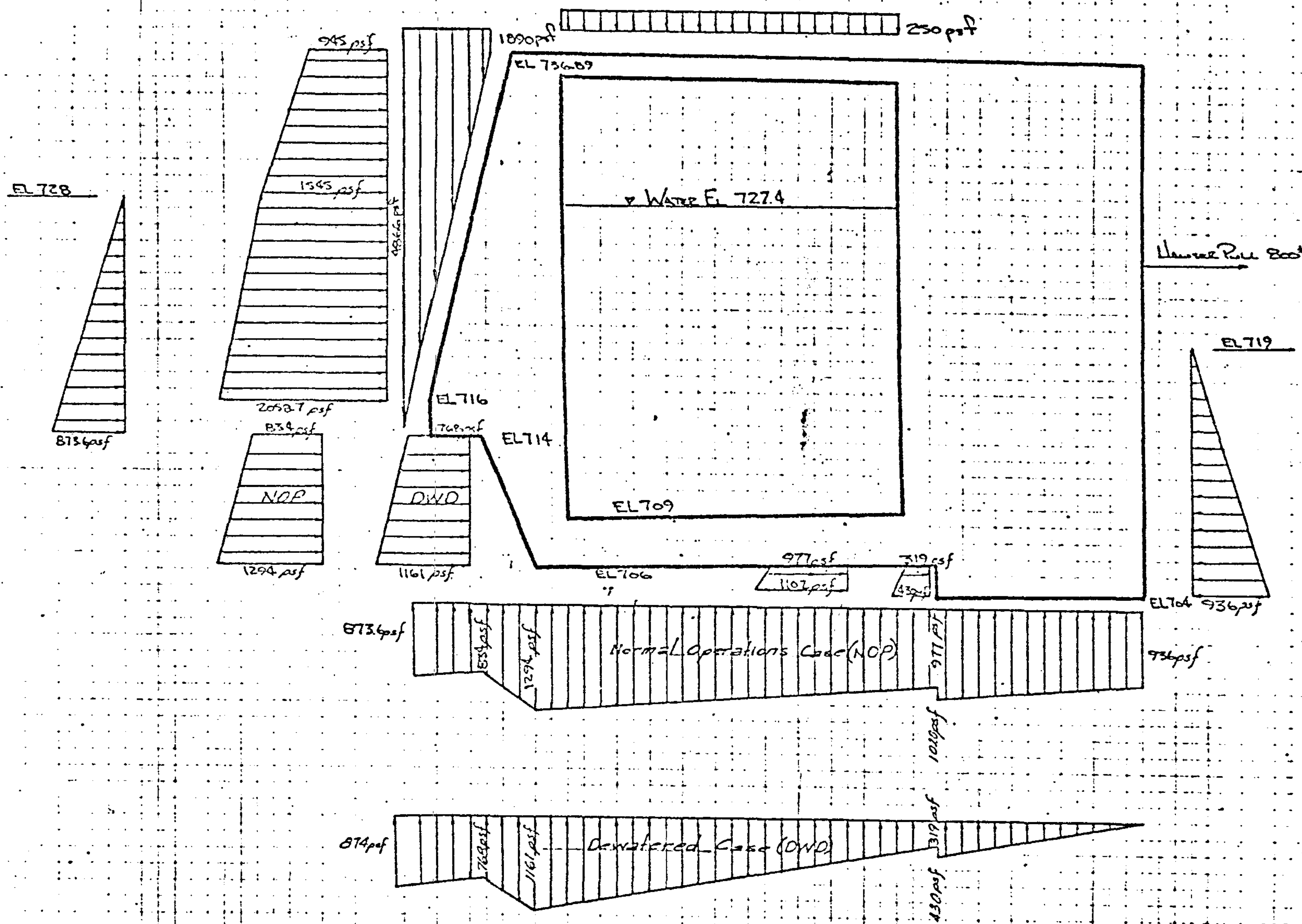


FIGURE 7.5 LOADING FOR MONOLITH L-23, STRESS ANALYSIS

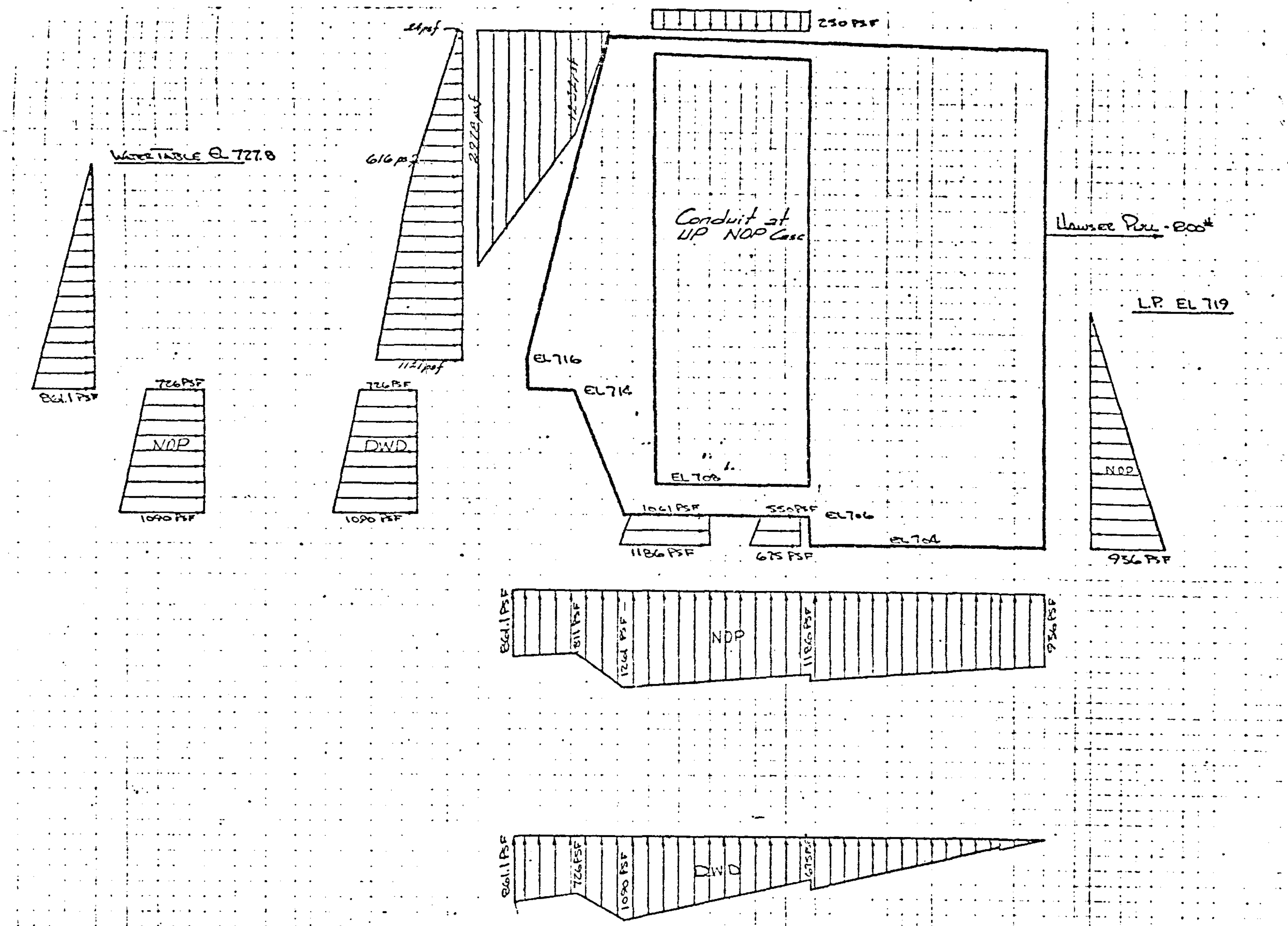


FIGURE 7.6 LOADING FOR MONOLITH L-32, STRESS ANALYSIS

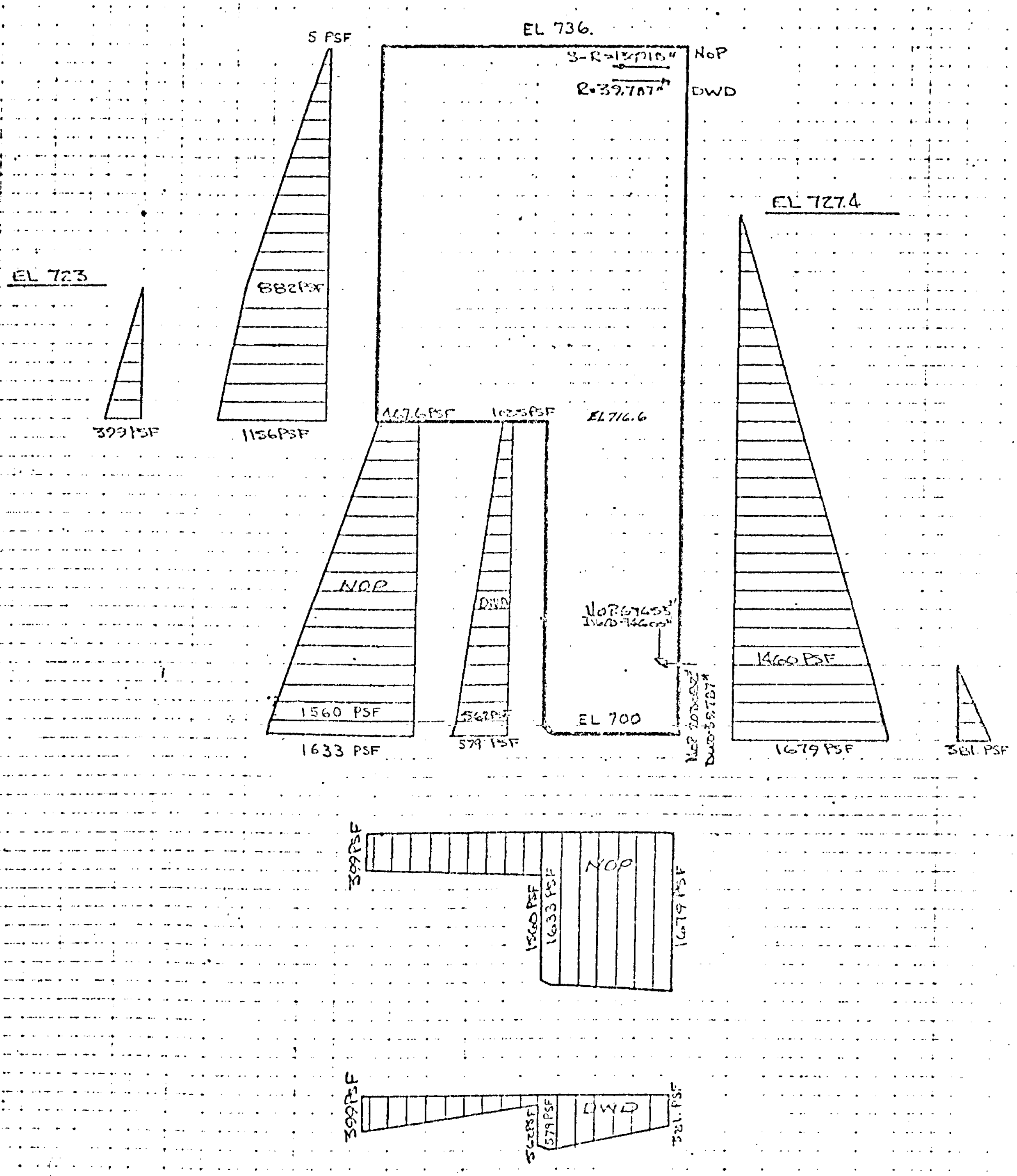


FIGURE 7.7 LOADING FOR MONOLITH L-46, STRESS ANALYSIS

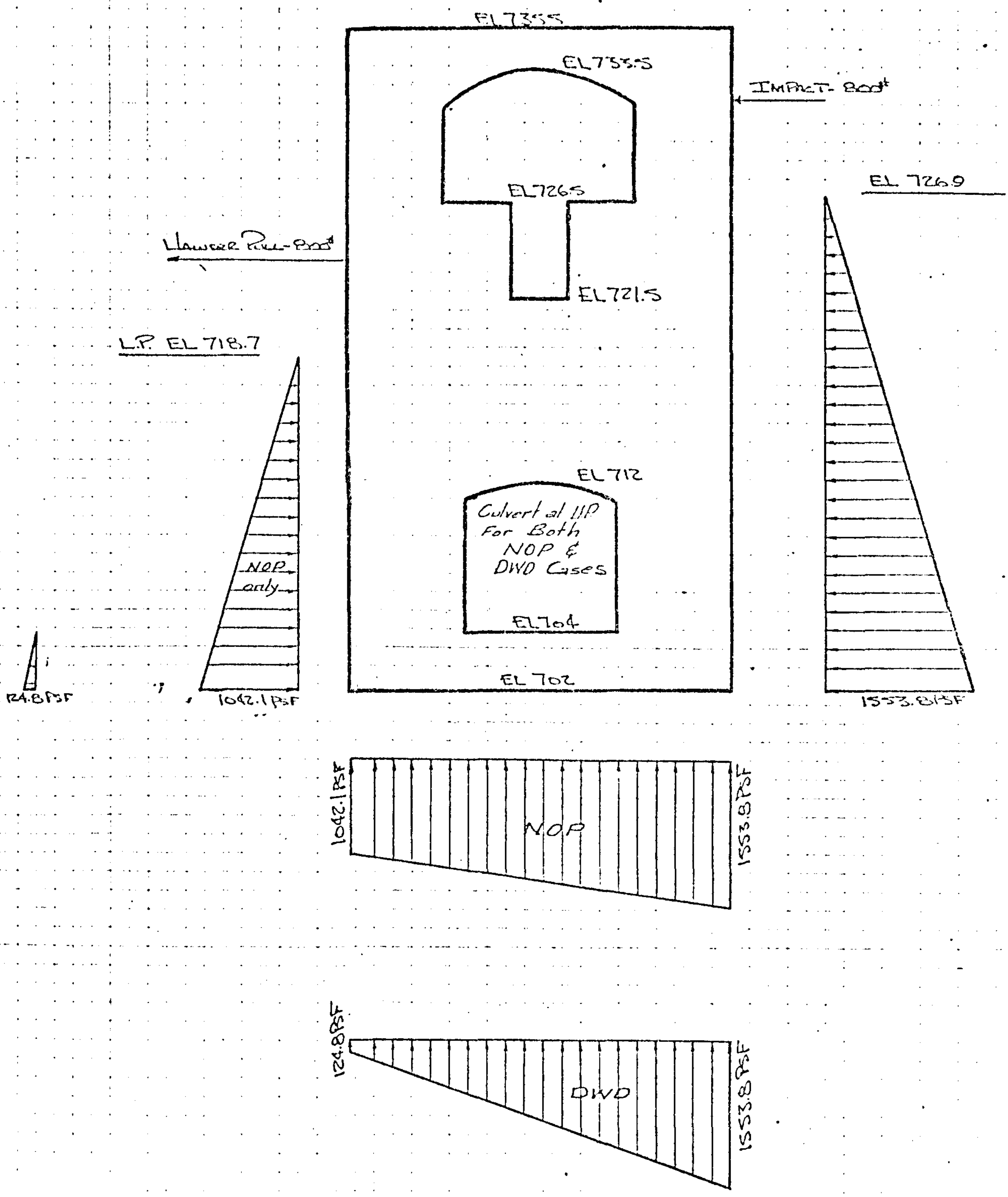


FIGURE 7.8 LOADING FOR MONOLITH M-6, STRESS ANALYSIS

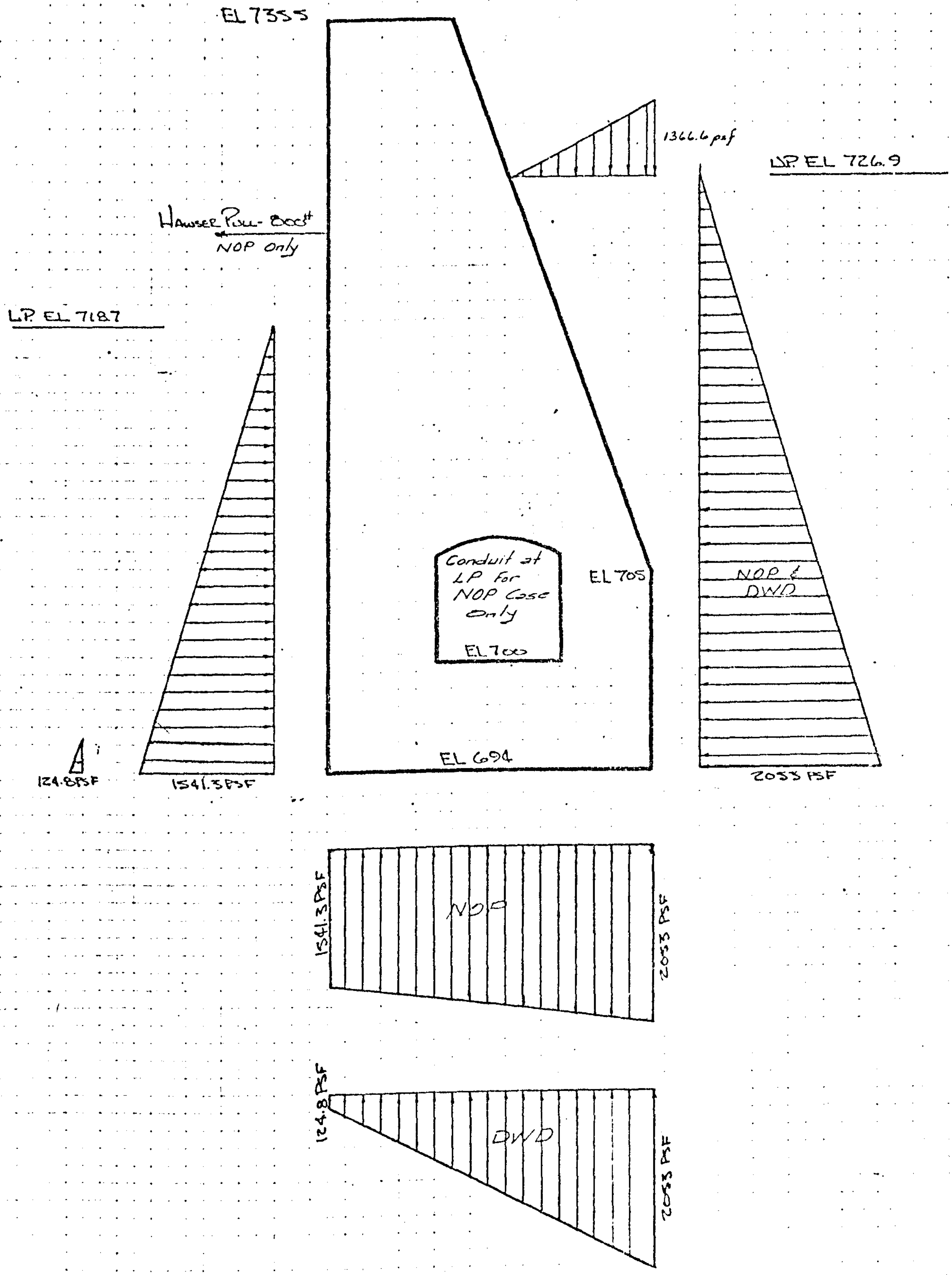


FIGURE 7.9 LOADING FOR MONOLITH R-15, STRESS ANALYSIS

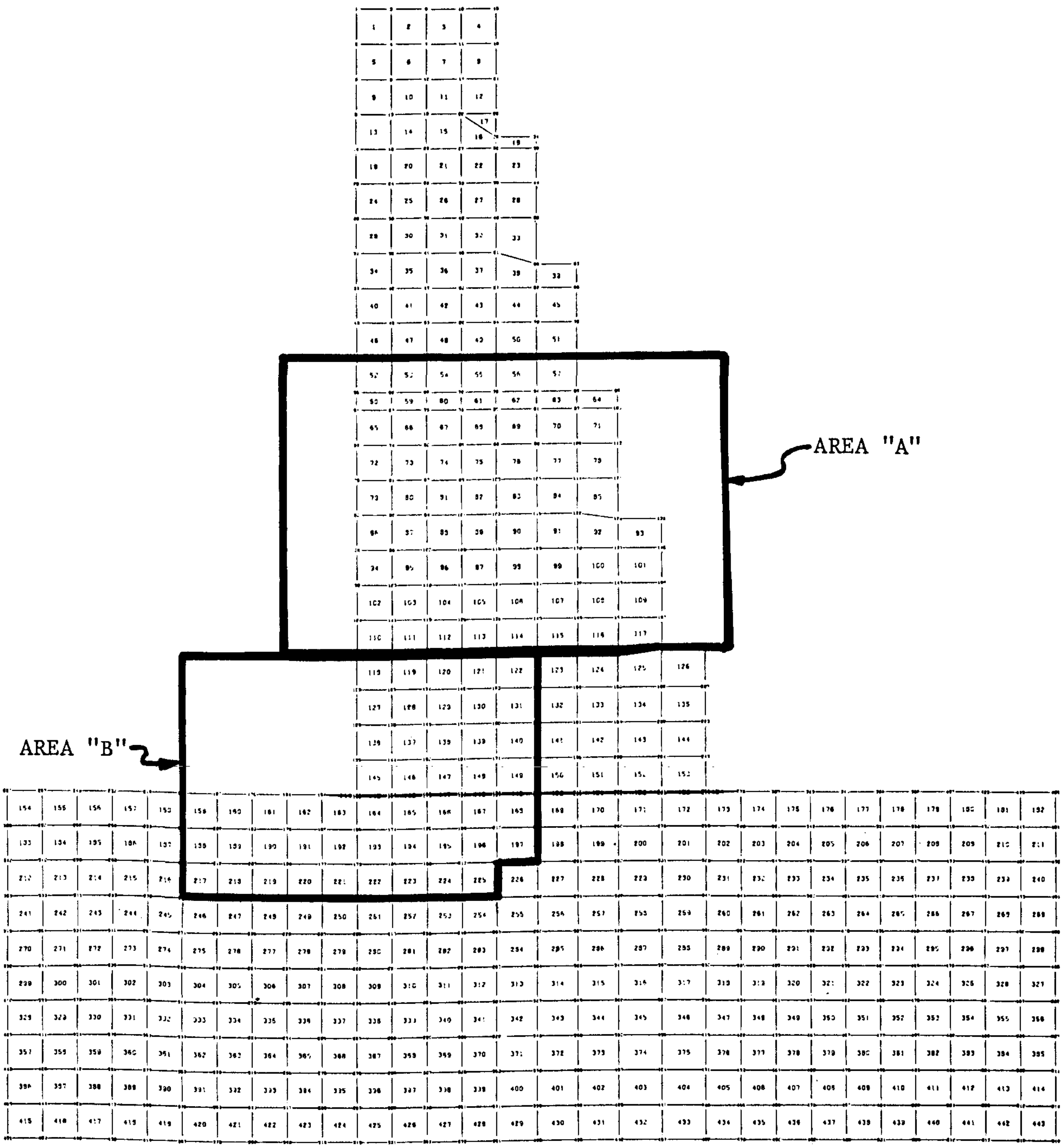


Figure 7.10. Grid - Upper Guide Wall Monolith, L-18, Depicting Stress Concentration Areas for Presentation

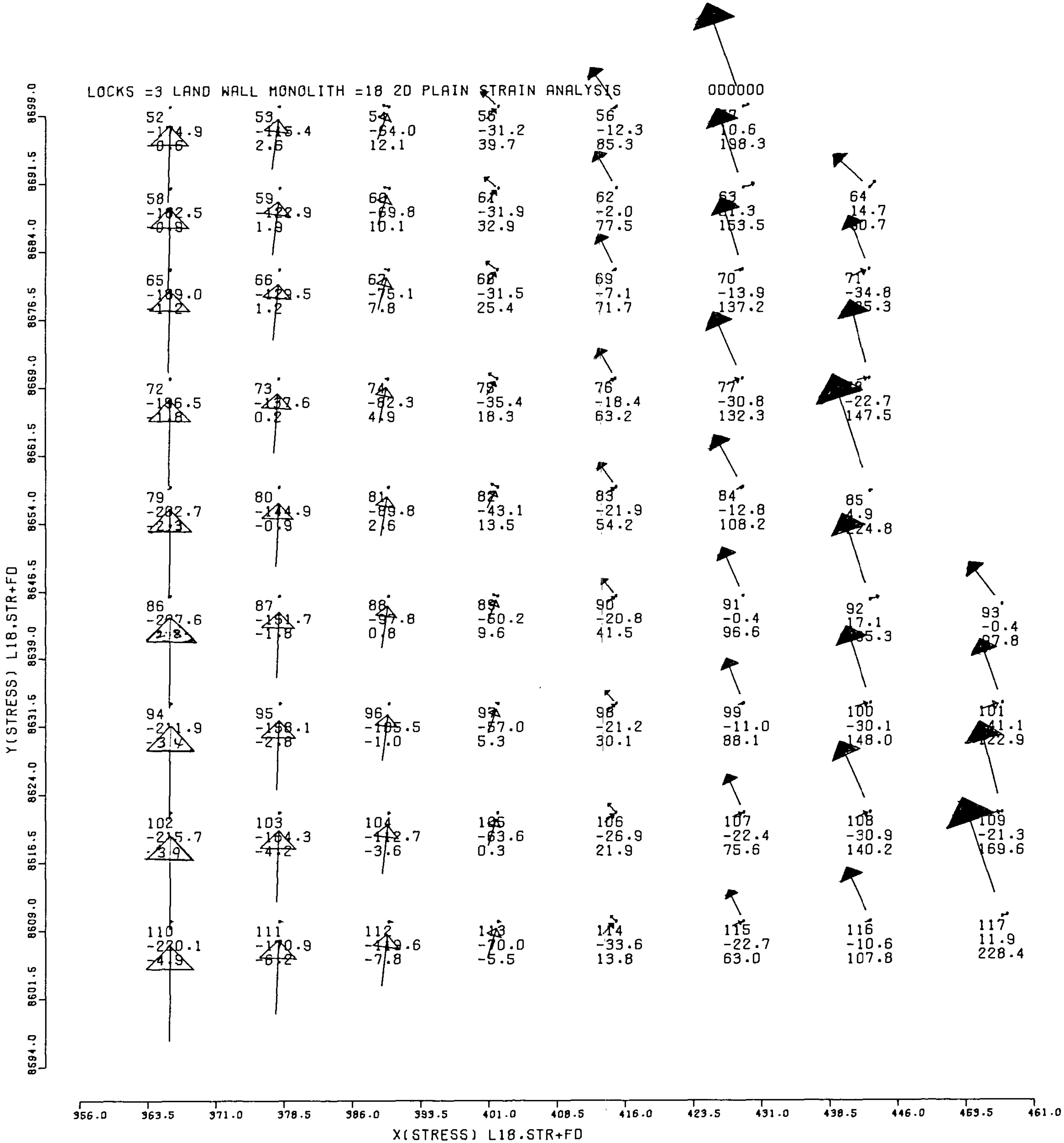


Figure 7.11. Stress vector plot of area "A" as depicted in Figure 7.10.
Monolith L-18

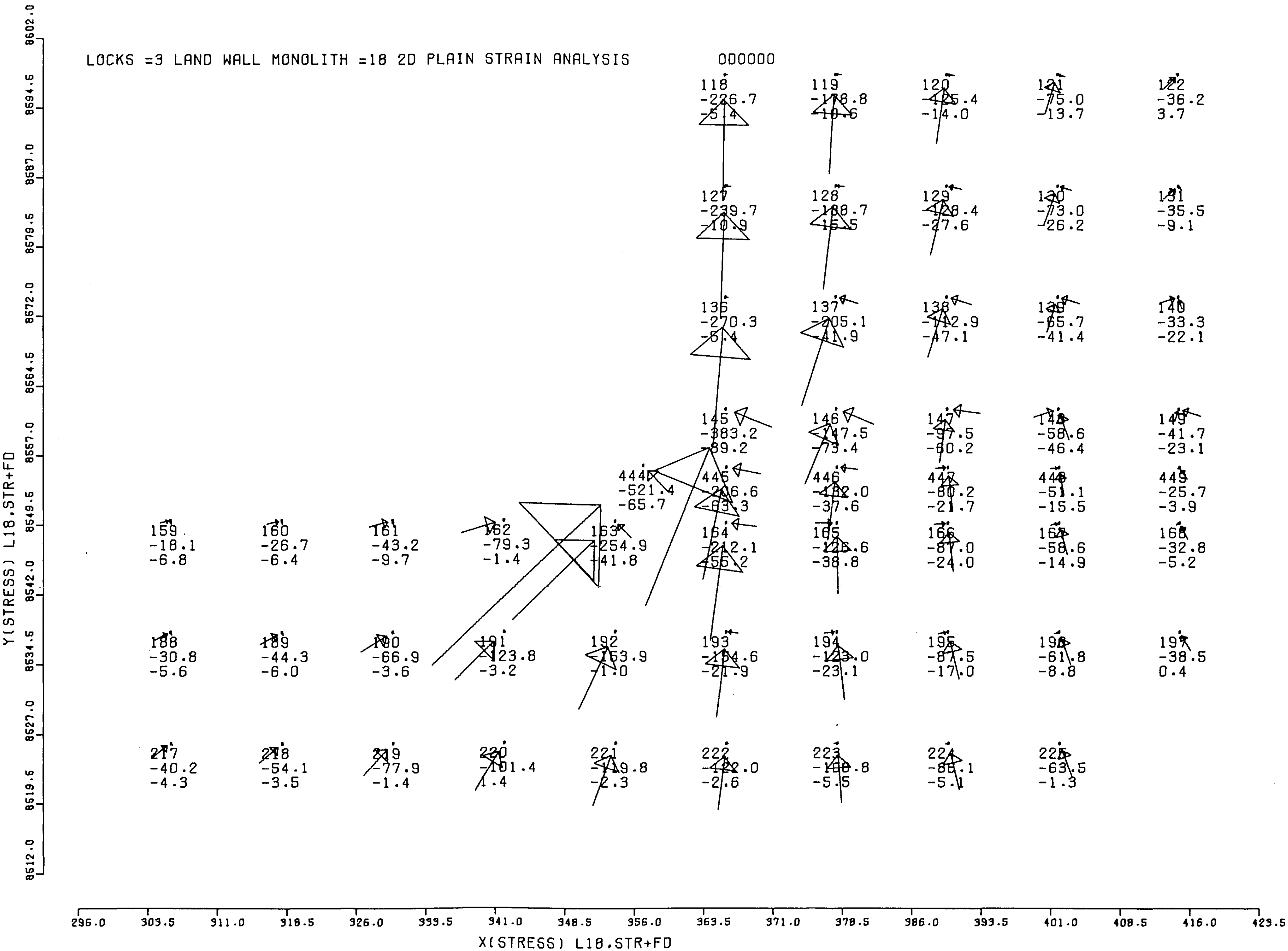


Figure 7.12. Stress vector plot of area "B" as depicted in Figure 7.10.
Monolith L-18

Figure 7.12.

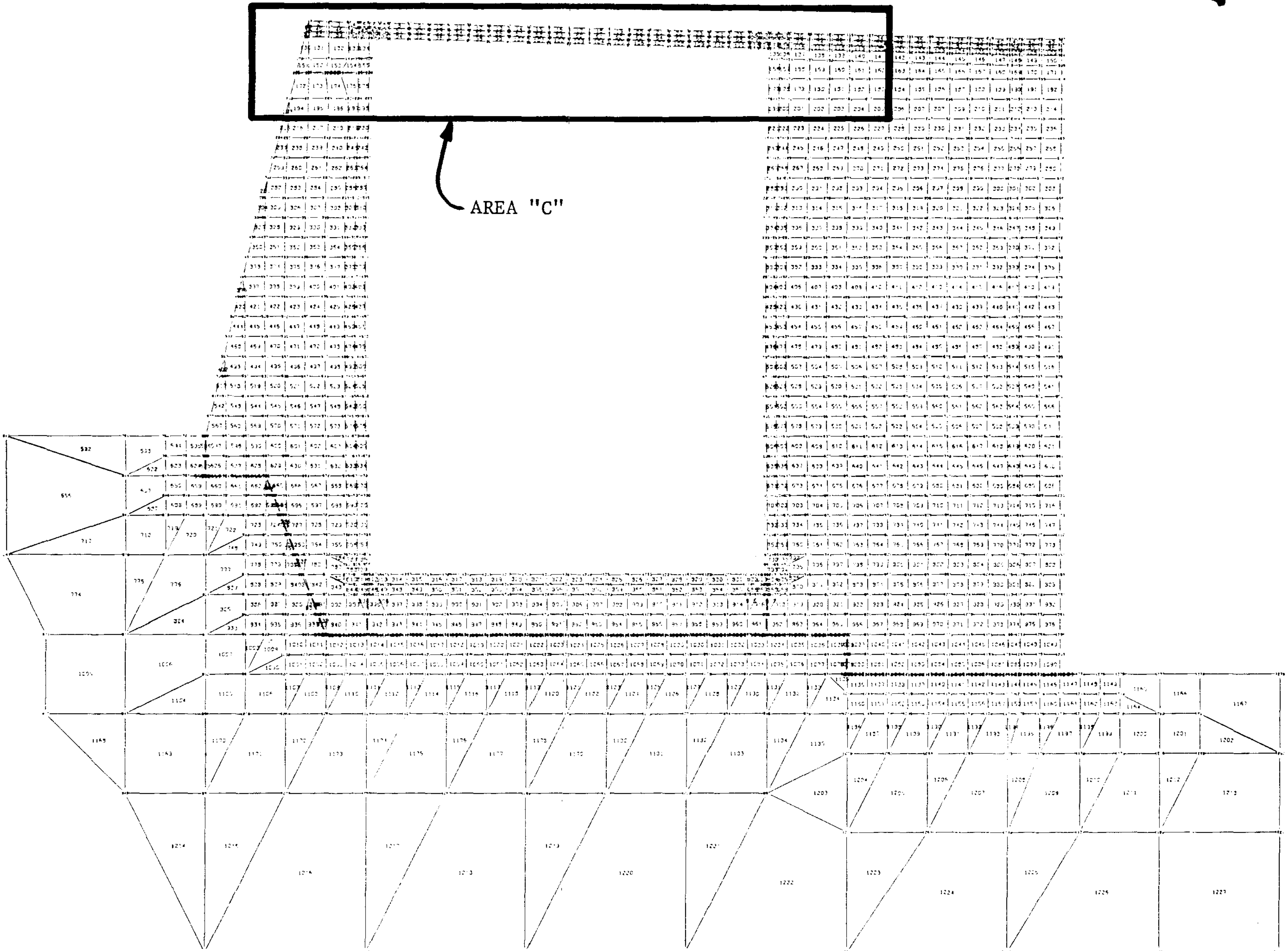


Figure 7.13. Grid - Land Wall, Upper Gate Bay Monolith, L-23, Depicting Stress Concentration Area for Presentation

Figure 7.14. Stress vector plot of area "C" as depicted in Figure 7.13.
Monolith L-23

Figure 7.14.

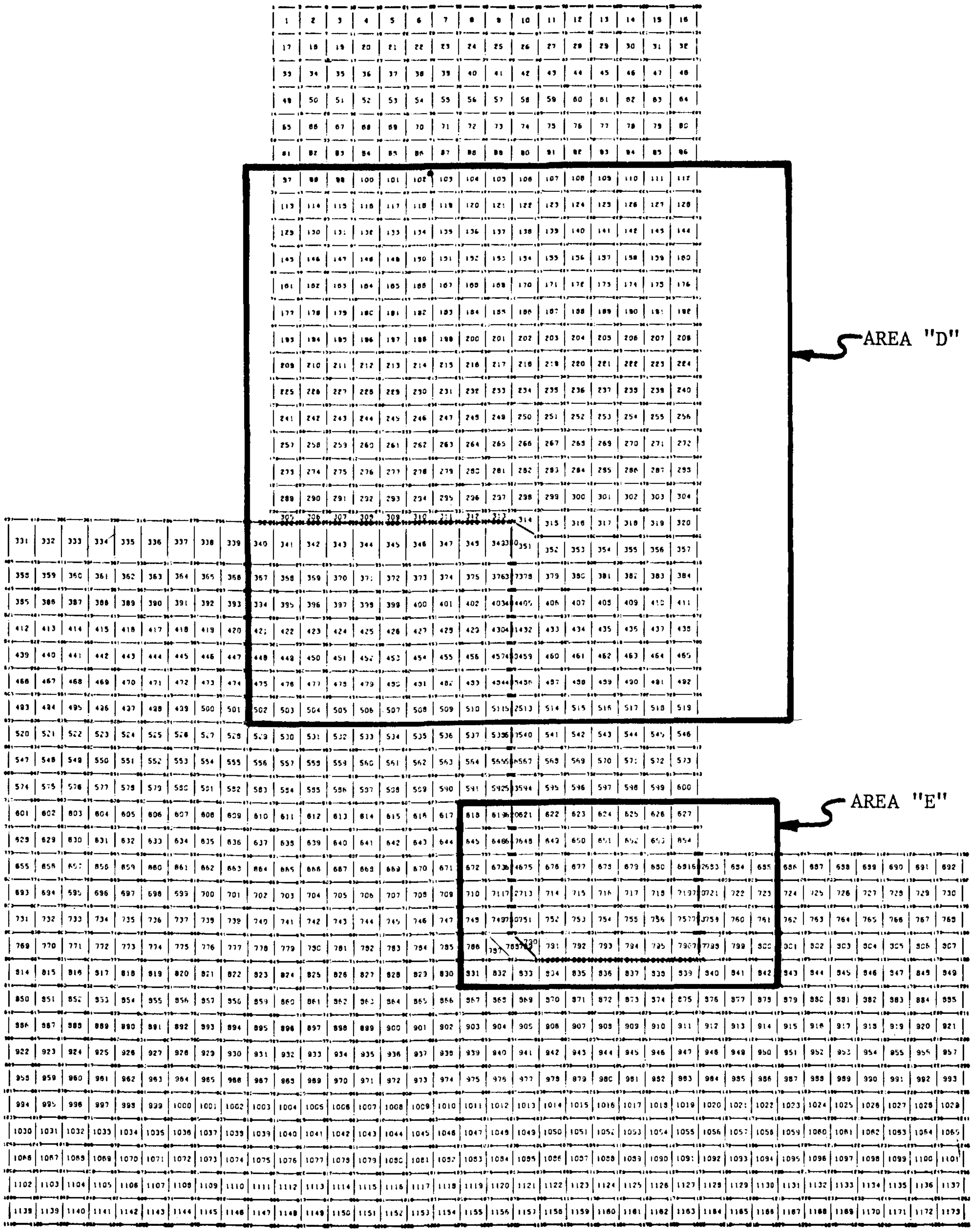


Figure 7.15. Grid - Land Wall, Lower Gate Monolith, L-46, Depicting Stress Concentration Areas for Presentation

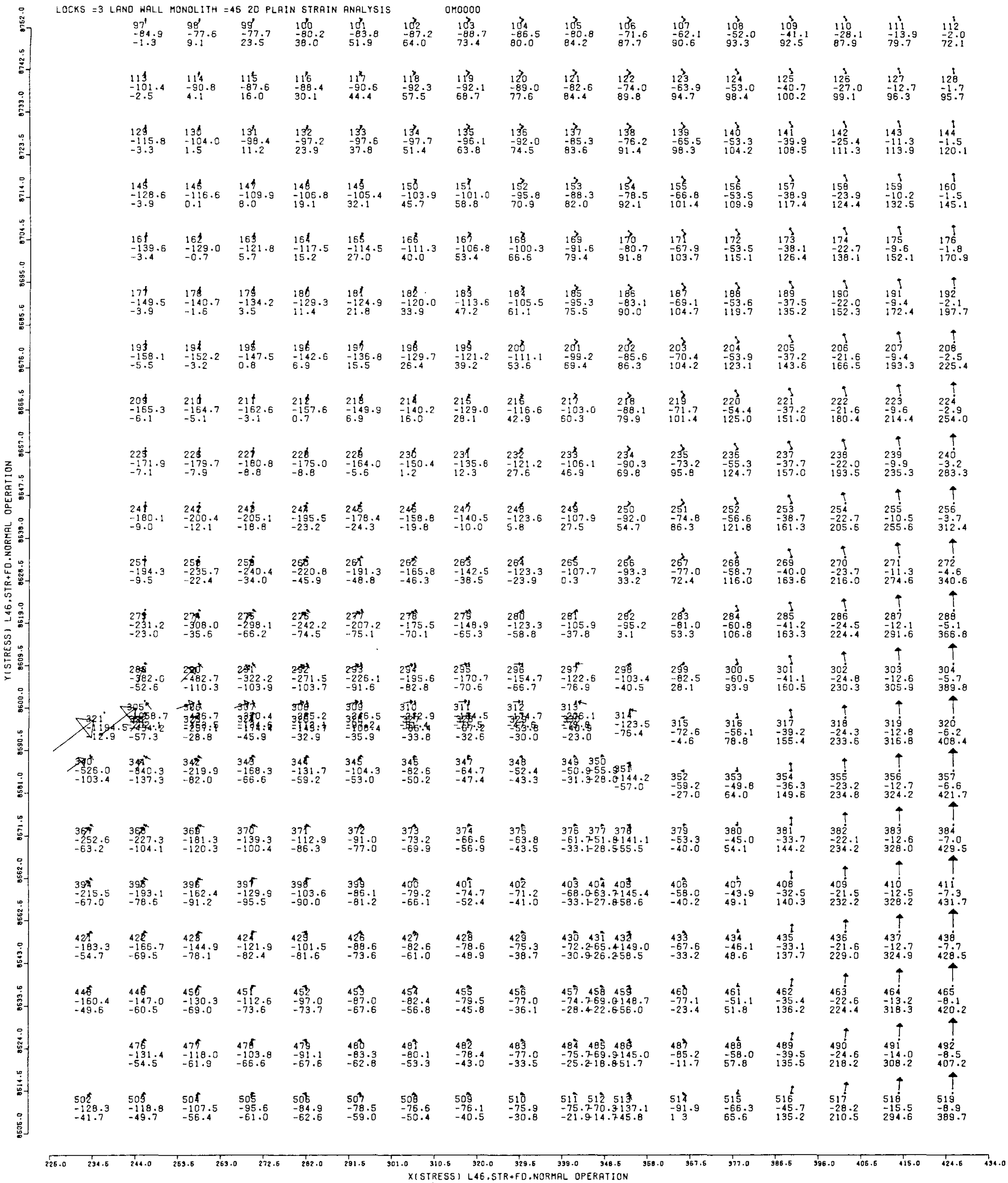


Figure 7.16.

Figure 7.16. Stress vector plot of area "D" as depicted in Figure 7.14

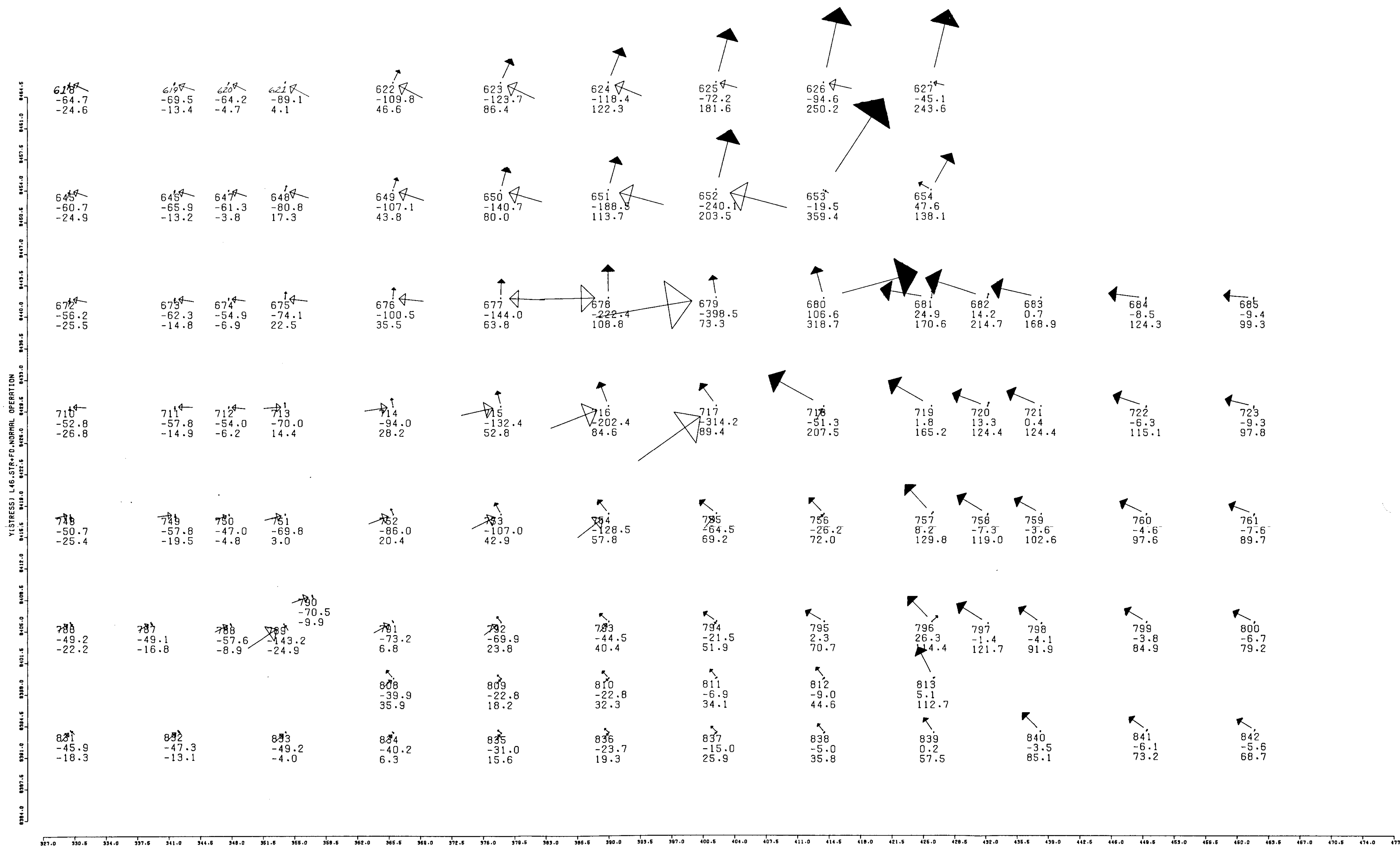


Figure 7.17. Stress vector plot of area "E" as depicted in Figure 7.14 Figure 7.17.

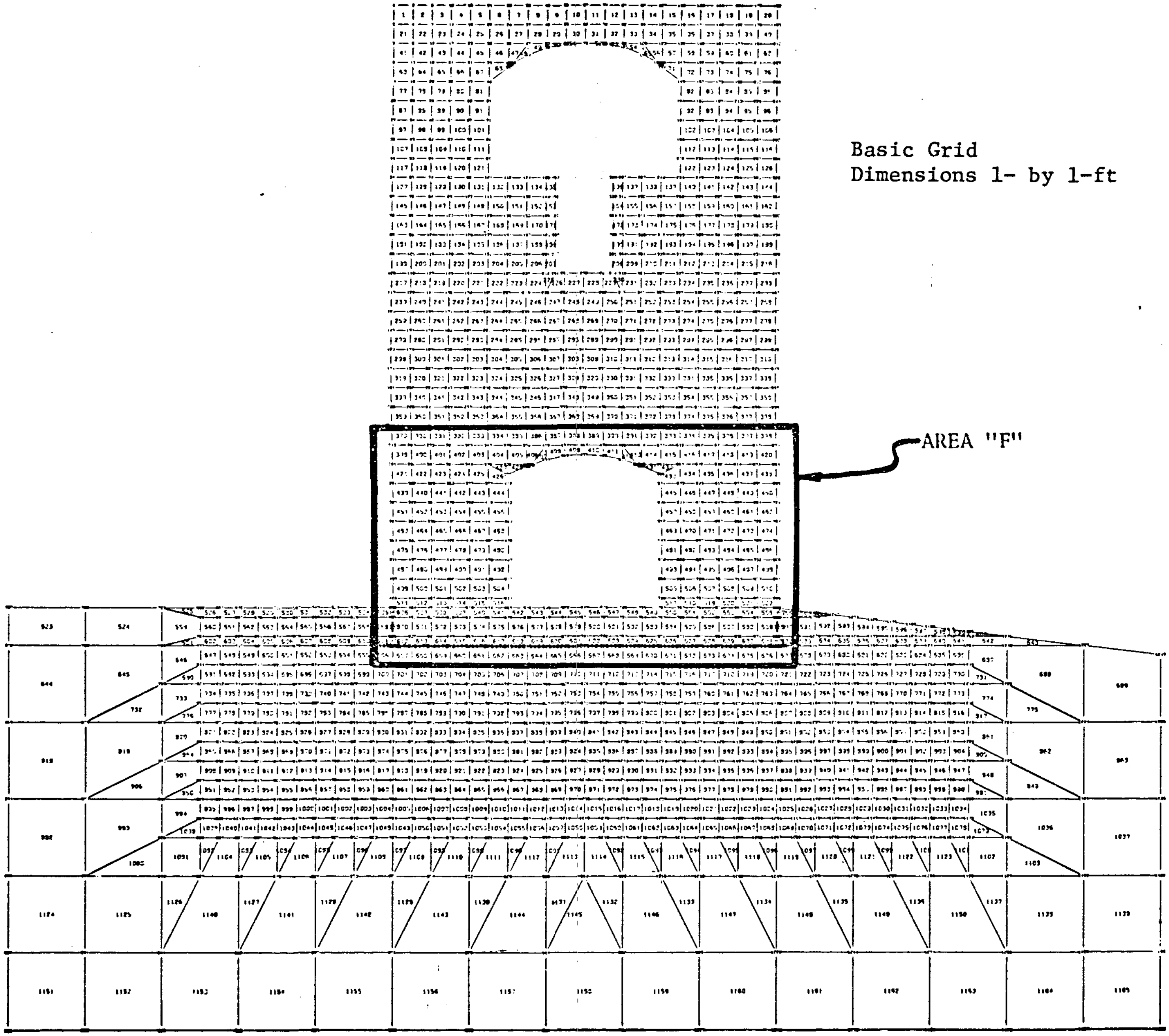


Figure 7.19. Grid-middle wall monolith, M-6, depicting stress concentration area for presentation

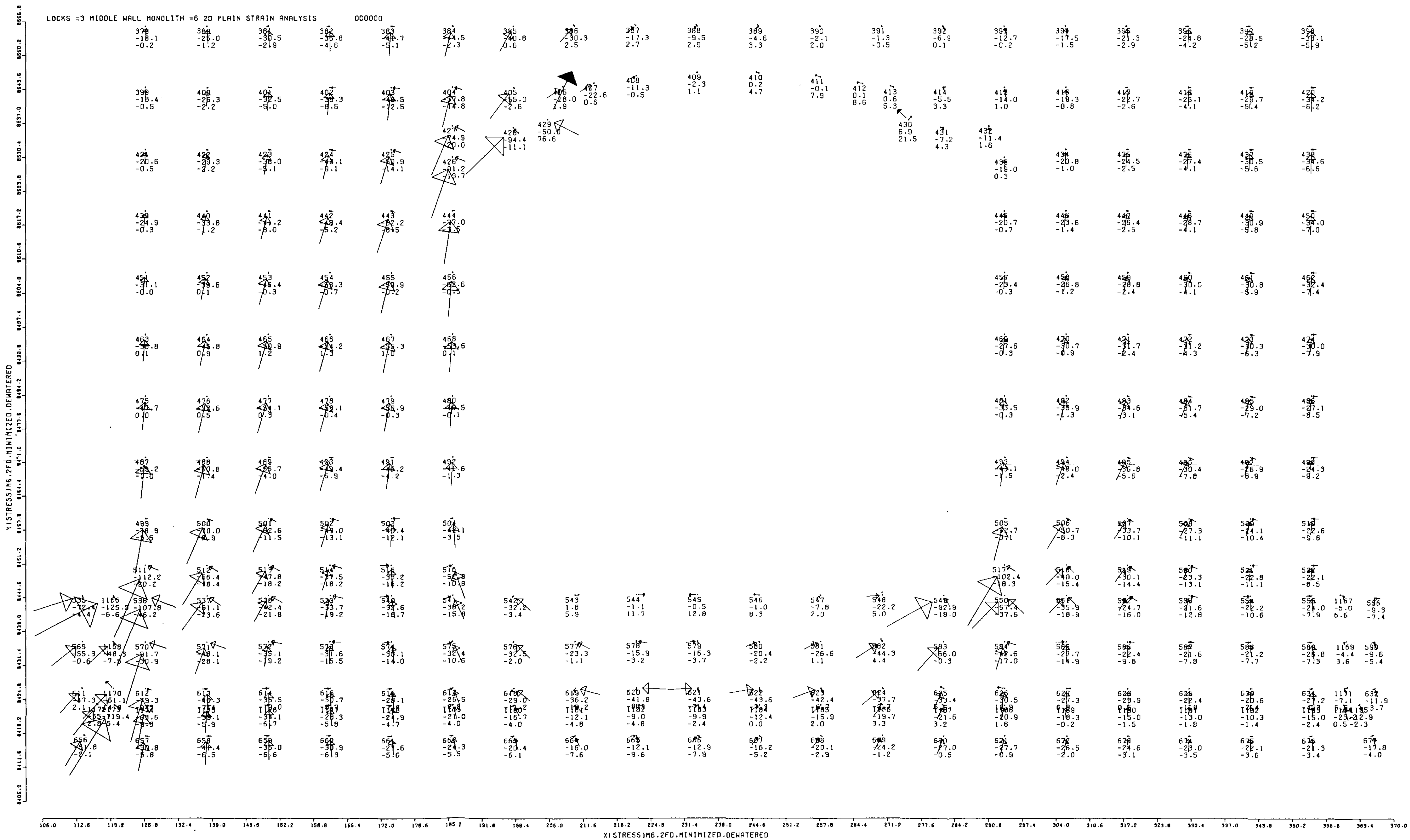


Figure 7.18. Stress Vector Plot of Area "F" as Depicted in Figure 7.19

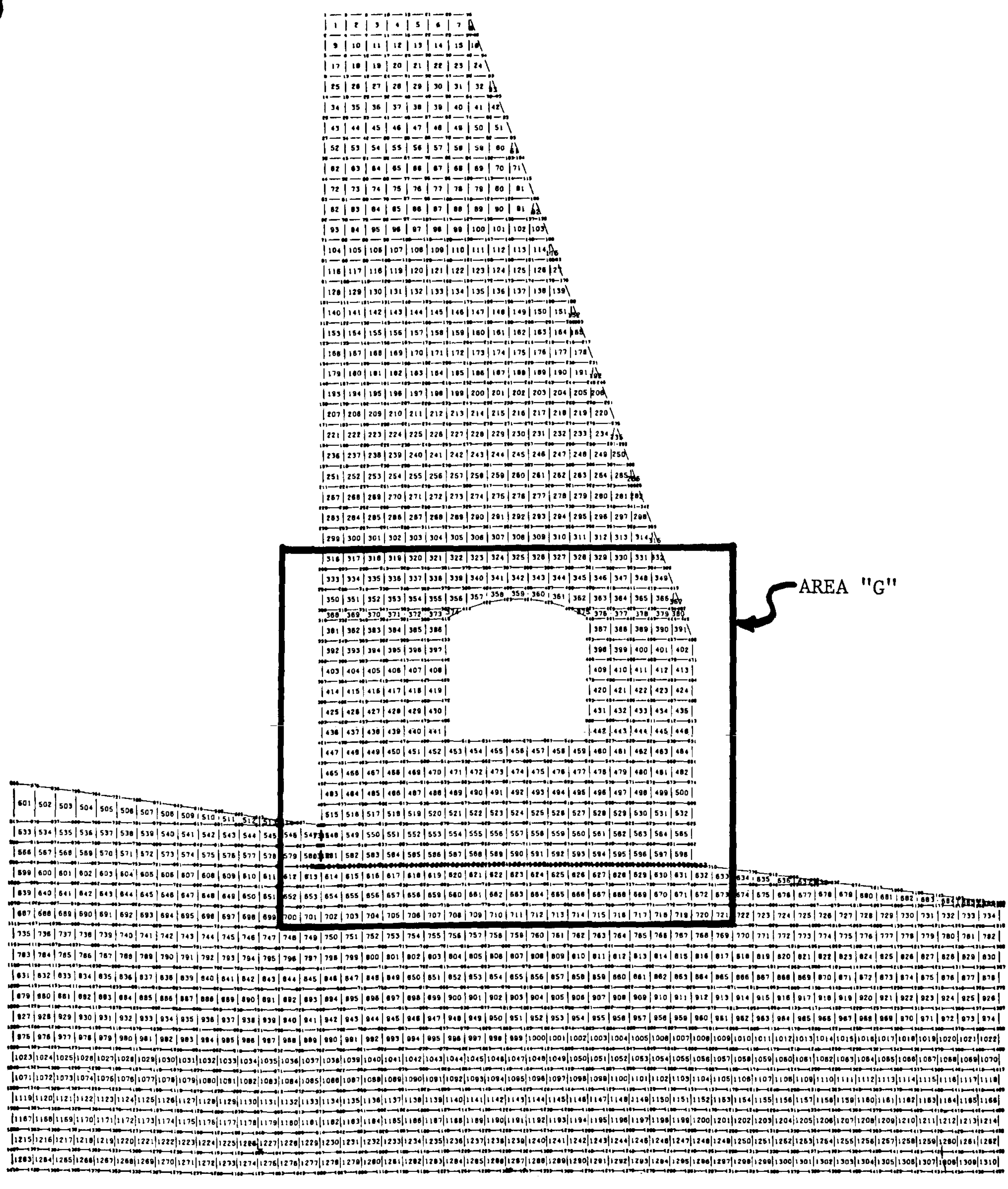


Figure 7.20. Grid - River Wall Monolith, R-15,
Depicting Stress Concentration Area for Presentation

SECTION 8: CONCLUSIONS AND RECOMMENDATIONS

External cracking of the concrete in the lock walls is extensive. It is concluded that the many horizontal and vertical cracks, while isolated and do not fit into a total failure picture, can result in localized failures, especially in critically stressed regions. The outer 2 to 6 ft of concrete as described in the core logs and the photo logs are highly deteriorated. The same condition probably exists in the outer surface of the walls to lower pool.

The internal concrete is generally of adequate strength for present day design criteria. One exception is a small zone in the downstream gate monolith of the middle wall; a compressive strength of 1150 psi was obtained on the concrete from this zone. The concrete is in a critically stressed area and immediate remedial action is recommended. Pressure grouting as an interim measure is suggested. There is one possible continuous crack in a gate monolith, R-23; it runs for a depth of 35 ft and possibly extends for two-thirds of the monolith to the riverside.

The borehole data indicate a consistent pattern of fractured concrete especially in the lower portions of the monoliths. This can be a significant factor tending to produce problems even at moderately high stresses.

The foundation appears to be in good condition over the greater majority of the lock site. Several local zones of highly fractured rock and one zone where a monolith bears on coal, R-12, are the only zones considered as possible weak rock. During dewatering of the locks, these local foundation conditions could contribute to serious failures of lock wall sections. Such failures could occur due to inadequate sliding resistance of some foundation materials at or near the concrete-foundation contact. There are no detectable continuous discontinuities, zones of fracture, or seams of weak material; bedding planes are the only detectable continuous features over the lock site.

The stability analysis reflects deficiencies in many areas.

a. In general the monoliths in the land wall do not comply with present day criteria for overturning, sliding or allowable base pressures.

b. Some middle wall monoliths do not have adequate resistance to sliding.

c. Some monoliths in the river wall do not have adequate resistance against sliding and where coal seams underlie monoliths critical situations can exist if the river lock is ever dewatered. Bearing pressures are excessive in the upper guard wall causing it to tilt riverward.

d. The abutment has excessive compressive, tensile, and horizontal loads on its pile foundation and this has resulted in it tilting riverward.

The stability analysis for the land wall monoliths shows severe inadequacies and as with the guard wall and the abutment, it is probable that stability problems will develop. These problems can cause considerable delays to navigation through these locks.

The upper guard wall has excessive bearing pressures between the upper and lower timber cribbing sections and has tilted riverward. There are probably some rocks between the cribbing members which result in more bearing area and, therefore, reduces the calculated stresses considerably. However, the tilting of the guard wall shows that the bearing pressures are still excessive. Tow impact can cause this guard wall to fail.

Cracking and deterioration of the exposed concrete are severe problems and are significant at stress concentrations. The stress analysis shows that gouges and concrete deterioration can become problems in the upper guide wall. There are excessive stress concentrations in the structural slab over the filling and emptying flume. The stress analysis indicates magnitudes of tensile stress that are excessive for even good quality concrete. The compressive stresses indicate magnitudes that would not be considered excessive for sound, uncracked concrete; however, for badly deteriorated concrete, they are excessive. Stress concentrations also exist at gate anchorages. As the concrete at Locks and Dam 3 continues to deteriorate, the stress magnitudes under existing loading conditions, as has been observed, will become a problem.

The locks have apparently functioned well for over 70 years, but this cannot be expected to continue indefinitely due to the accelerating deterioration due to concrete cracking, leaching, and exposure to a freezing-and-thawing environment. It is evident from this evaluation that these locks and dam will either have to be replaced or completely rehabilitated.

It is recommended that a study be initiated immediately to evaluate rehabilitation or replacement of Locks and Dam 3 on the Monongahela River.

REFERENCES

1. Condition of Two Locks at Lock and Dam No. 3, Monongahela River, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., 17 December 1973.

2. U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, Handbook for Concrete and Cement, with quarterly supplements, Vicksburg, Miss., August 1949.

3. The Rock-Color Chart Committee, National Research Council, Rock-Color Chart, Washington, D. C., 1963.

4. Preparation of an Engineering Condition Survey and Structural Investigation for Locks and Dam 3, Monongahela River; Emsworth Locks and Dam, Ohio River; Montgomery Locks and Dam, Ohio River; Appendix "A", by U. S. Army Engineer District, Pittsburgh.

5. Hough, B. K., Basic Soils Engineering, The Ronald Press Company, New York, Second Edition, 1957.

6. Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures, Lock and Dam 3, Pre-Inspection Data, First Periodic Inspection, 4 November 1971, U. S. Army Engineer District, Pittsburgh, Corps of Engineers, Pittsburgh, Pa.

7. U. S. Army Corps of Engineers, "Tests for Strength Characteristic of a Schistose Gneiss," MRD Lab No. 64/126, U. S. Army Corps of Engineers, Missouri River Division Laboratory, Omaha, Neb., May 1965.

APPENDIX I

BACKGROUND

APPENDIX I

BACKGROUND

1. During recent years, much emphasis has been placed on structural safety of Corps of Engineers structures, i.e., dams and locks, and programs have been established by the Office, Chief of Engineers and appropriate regulations disseminated to the various Corps offices implementing the program. These regulations include ER 1110-2-99, "Inspection and Evaluation of Corps of Engineers Bridges," and ER 1110-2-100, "Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures." In complying with these regulations, the various District offices have been required to reanalyze and reevaluate various structures within their jurisdiction.

2. The Pittsburgh District office initiated the investigation of Locks and Dam 3 by their Periodic Inspection Report.[6] This report reviews the general condition of Locks and Dam 3 with attention to specific problem areas. The results of the periodic report warranted a further detailed study. An additional study, a condition survey,[1] was conducted for the Pittsburgh District, ORP, by the Waterways Experiment Station, WES, in 1973. The purpose of the condition survey was to determine the quality of the concrete and locate cracks that could affect the stability of the locks. The major work units of the condition survey were:

Crack Survey
Borehole Photography
Soniscope Survey

The crack survey located and identified surface cracks. The results of the borehole camera investigation, the field soniscope study, and the core logs supplied information about the existence and location of interior cracks. The laboratory testing supplied information about the physical properties of the intact concrete cores. The results of the condition survey are briefly summarized in the following paragraphs.

a. Crack Survey. The crack surveys of the two locks were conducted by the ORP in the spring of 1973 and by WES representatives in early fall 1973. The significance of the cracks cannot be interpreted with respect to the distance that they penetrate, because most of the visible concrete surfaces were surfaces of repairs. The operational history of Lock No. 3 Monongahela River prepared by the ORP in 1971 includes 11 items that are clearly identified as repairs to faces or tops of the river wall, middle wall, land wall, lower guide wall, upper guard wall, and upper and lower guide walls, and other items that may have been included in other repairs to the walls. As a consequence the value of the crack survey is greatly reduced because there is no way to know with regard to most of the cracks whether a visible crack is restricted to the repair or is an extension of a crack that is present in the underlying concrete. For the reasons described above little use could be made of these data in the analysis of the extent of surface and interior cracking.

b. Borehole Photography. The interpretations of the borehole film records clearly show that the top portion, surface to about 2 ft, of concrete is in excellent condition and represents replaced concrete, not original concrete. The portion from about 2 ft to about 6 ft is highly weathered and fractured. Although specific areas were photographed, it is reasonable to believe that the top portions just under the replacement concrete of the river, mid, and land walls are in the same deteriorated condition. An exceptionally high number of honeycombed areas were detected on the film as well as open fractures. Most of the fractures were coated with various colored stains. The stained fractures indicate percolating water action. The concrete core logs indicate similar staining on many broken surfaces in the concrete. Of the seven drilled holes that were photographed, four showed poor to no contact between the concrete and rock. In hole MA-10 the rock just below the concrete-rock contact is highly fractured with numerous voids for a 6-ft depth. The core logs show that this interval was represented by gravel and by core loss.

c. Soniscope Survey. The interpretation of the soniscope results was hampered by the presence of interfaces behind all vertical surfaces. These interfaces resulted from the presence of sometimes unknown numbers of layers of patching and shotcreting. If a reading was obtained it was considered a valid representation of concrete in the path traversed by the compressional wave. When no reading was obtained it could be interpreted to mean:

(1) An open crack or the presence of a void.

(2) Lack of bond between repair and underlying concrete.

(3) Concrete of such poor quality that the signal is entirely attenuated.

Although probably causes (1) and (2) are variations on one theme, and are believed to be more significant than (3), no one of the explanations can be assigned to any specific instance where no reading was obtained.

3. In general the compressional wave velocities computed for the middle wall represented velocities through concrete that is generally considered sound concrete. However, of 110 locations, 15 resulted in no readings, 1 velocity is considered as representing concrete of questionable quality, and 2 locations gave velocities considered to represent concrete of very poor quality.

4. The velocities computed for the river wall were, in general, slightly lower than those obtained from the middle wall. Of 46 locations, 6 resulted in no reading and 11 locations (24 percent) represented what is considered as generally poor concrete. Velocity measurements through the river wall near drilled holes MA-4 and MA-7, stations 4+25B to 4+40B, were very low. Three of thirteen velocities were considered as representing good concrete. The area just described is within the riverside gate monolith. The core logs of these two holes show broken core and core loss within the depth interval through which the velocity readings were made.

APPENDIX II

SURFACE CONDITION OF CONCRETE

APPENDIX II

SURFACE CONDITION OF CONCRETE

Surface Cracks

A field crack survey was conducted by ORP in the spring of 1973[6] as part of its periodic inspection and continuing evaluation of Lock and Dam 3. The WES conducted a crack survey as part of its investigation in the summer of 1973.[1] The WES crack survey was mainly concerned with ascertaining if structural cracks existed in the lock walls. A structural crack is defined in Reference 1 as a crack that separates or tends to separate what was originally one concrete element into two or more. The crack survey provided information about the location of the existing surface cracks but did not provide any direct information about depths to which the surface cracks extended into the concrete. The reason for not determining depth of surface cracks was mainly due to the extensive amount of overlay of gunite and concrete on the lock walls.

The extent of cracking found during the ORP and the WES field crack surveys is presented in Figures II.1 through 4; the figures are reproduced from Reference 1. The results show that most of the cracks in the tops of the chamber portion of the land and middle walls are about 1/2 in. wide while those in the corresponding portion of the river wall are about 1/8 in. wide. Cracks in the tops of the guide walls are generally 1/32 to 5/64 in. wide. Most of the vertical cracks in the lock walls were observed with binoculars and telescopes and were not measured.

In the upper and lower land guide walls the majority of surface cracks appear to be located in areas where heavy boat damage is reported. Embedded pieces of wood in the lower guide wall from about station 8+60B to about 12+70B are shown in Figure II.2. These areas are not hazardous at the present time; however, when the wood rots and falls out concrete can more easily be removed by tow or boat impact.

As seen in Figure II.3 the middle walls contain a goodly number of cracks on both the top and sides of the lock walls. The riverside wall contains three open lift joints; however, the lateral extent that these joints remain open is not known. Two open lift joints are between station 0+95B and 1+45B; the third is between station 1+78B and 2+10B. Other horizontal cracks are depicted on the river and landside of the middle wall with the longest one from station 3+46.5B to 4+09.2B. A number of cavities are shown which are assumed to be due to deteriorating gunite and/or concrete.

A 6-in.-wide crack at station 4+44.6B in the gate monolith M-16 begins at the monolith joint and extends downward at an angle of about 45 degrees. The extent of the crack is not known; however, the extent of this crack should be determined for the following reasons:

<u>a</u>. The crack is in a gate monolith.

<u>b</u>. Monolith M-16 is the monolith from which the low-strength concrete cores were obtained; compressive strengths of 1150, 1450, and 2100 psi were obtained as reported in Reference 1 . If the crack goes into the monolith, it may be contributing to the low strength concrete by allowing water to leach or deteriorate the interior concrete.

<u>c</u>. Possible evidence of the crack is described in the core log of hole M-3, which is presented in Reference 1 . At a depth of 10 to 16 ft, the authors estimate that the crack begins 10 ft from the top of the wall. Features like weathered breaks, honeycombing, and mud on a weathered break are described. These features are strong evidence that the crack extends at least one-third the way through monolith M-16.

The number of surface cracks described in the upper guide and river wall appears to be less than the number of surface cracks described elsewhere in the lock walls. A greater number of horizontal cracks are present near the waterline than report to be present in the middle wall. At about station 3+00B a horizontal crack at about the elevation where a lift joint should be located, leaching and leakage are described. This is the first time that leakage was described in Reference 1 . At about station 4+68B (see Figure II.4) leaching and leaking are described for a near vertical crack that begins at the junction of a horizontal crack located several feet below the top of the river wall. The nearly vertical crack is located downstream of the downstream gate. If water leaks from this crack at a point above the waterline, then it can be assumed that the leakage is due to water from high pool in the lock chamber.

Rebound Hammer Results

Rebound hammer tests were conducted as described in Appendix B of Reference 1. The tests were made to evaluate the bond of the gunite to the concrete. Seven test sites were on gunite cover and five test sites were located near areas where the gunite cover was missing, or were located near cracks in the gunite, or both. There was no significant difference between the test results obtained on the two different gunite surfaces. These results indicate that the repair contact for the areas tested are fairly tight.

Deteriorated Zones

Considering the length of time that the lock walls have been exposed to as severe a climate as exists in the Pittsburgh area, the amount of damage due to freezing and thawing of the nonair-entrained concrete is not surprising. The top 2 to 6 ft of all of the 6-in.-diameter cores, except the core from hole M-4, from concrete placed in 1923-1924 contained fairly closely spaced parallel fractures and did not provide lengths great enough for laboratory velocity or normal compressive strength tests. Subparallel fractures parallel to a free surface are a form of failure characteristic of damage by freezing and thawing to nonair-entrained concrete. A goodly amount of surface deterioration is present at Lock and Dam 3. Concrete in the gate monoliths is continuing to deteriorate and could cause localized failures which would impair normal traffic flow for an unspecified length of time.

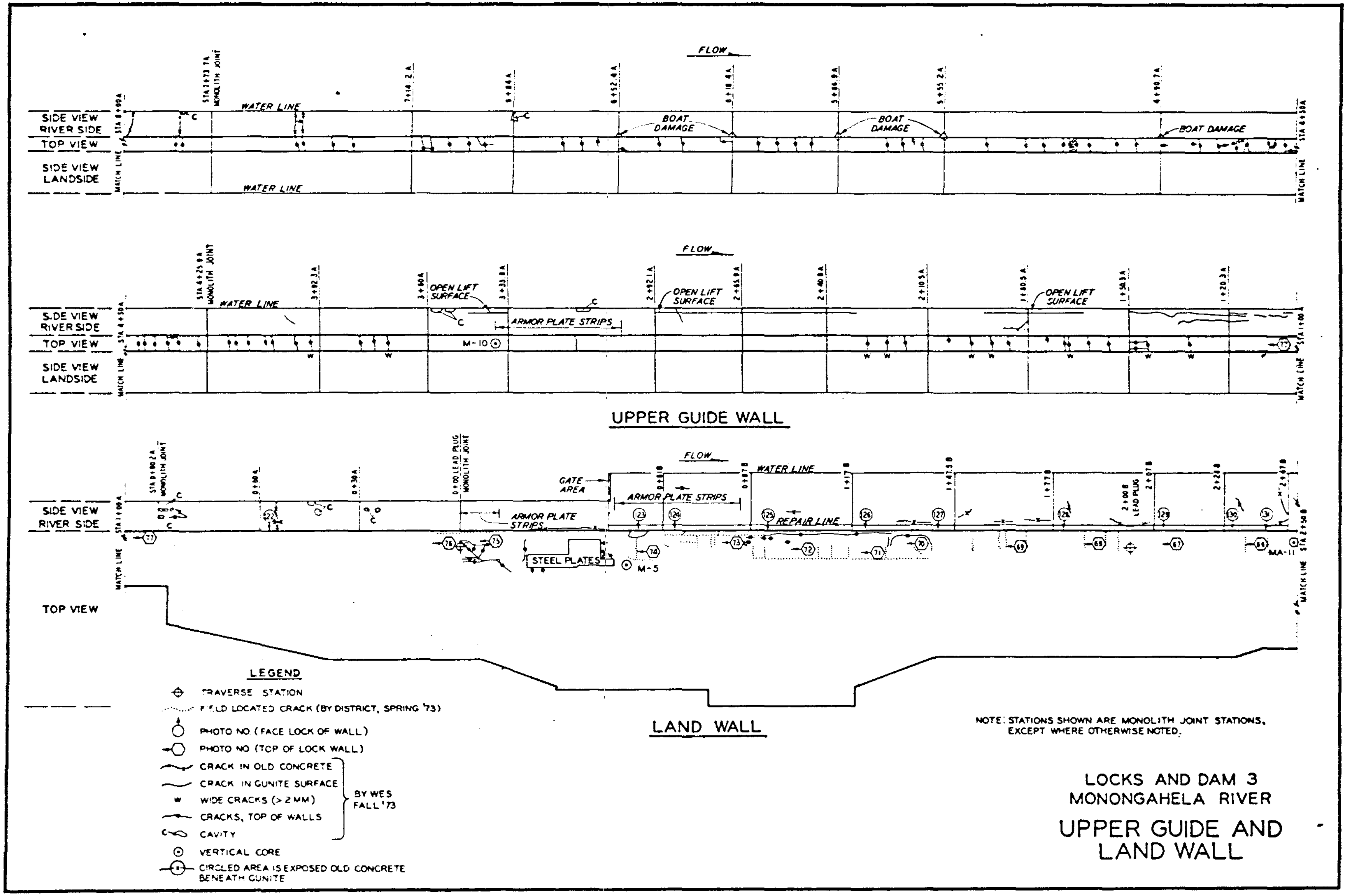

FLOW
SIDE VIEW RIVER SIDE
TOP VIEW
SIDE VIEW LANDSIDE
WATER LINE
BOAT DAMAGE
OPEN LIFT SURFACE
ARMOR PLATE STRIPS
UPPER GUIDE WALL
GATE AREA
REPAIR LINE
STEEL PLATES
M-5
M-10
LAND WALL
LEGEND
TRAVERSE STATION
FIELD LOCATED CRACK (BY DISTRICT, SPRING '73)
PHOTO NO. (FACE LOCK OF WALL)
PHOTO NO (TOP OF LOCK WALL)
CRACK IN OLD CONCRETE
CRACK IN GUNITE SURFACE
WIDE CRACKS (> 2 MM)
CRACKS, TOP OF WALLS
CAVITY
BY WES FALL '73
VERTICAL CORE
CIRCLED AREA IS EXPOSED OLD CONCRETE BENEATH GUNITE
NOTE: STATIONS SHOWN ARE MONOLITH JOINT STATIONS, EXCEPT WHERE OTHERWISE NOTED.
LOCKS AND DAM 3
MONONGAHELA RIVER
UPPER GUIDE AND LAND WALL

Figure II.1

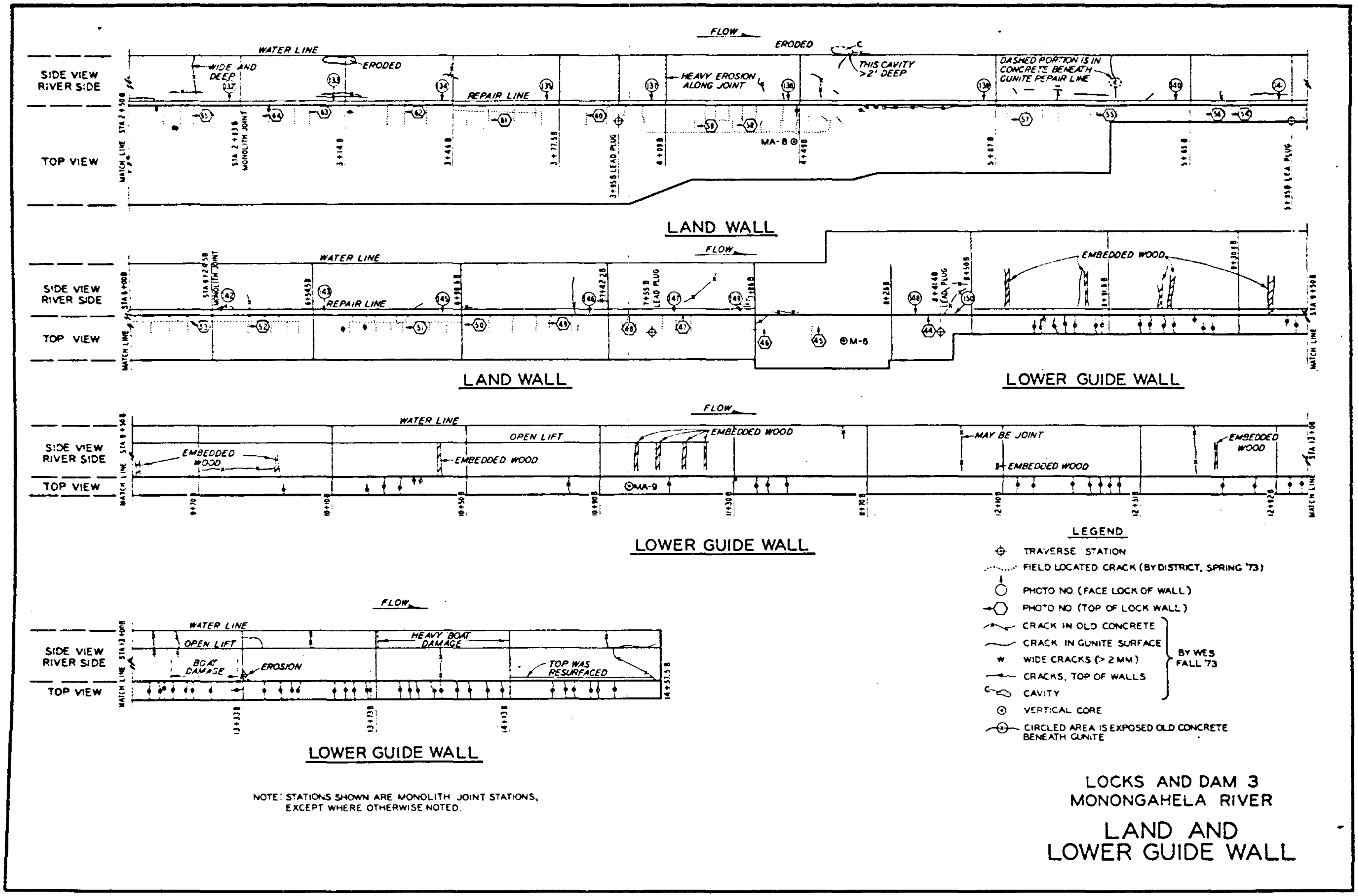

FLOW
WATER LINE
WIDE AND DEEP
ERODED
THIS CAVITY >2' DEEP
HEAVY EROSION ALONG JOINT
REPAIR LINE
DASHED PORTION IS IN CONCRETE BENEATH GUNITE REPAIR LINE
SIDE VIEW RIVER SIDE
TOP VIEW
MATCH LINE
STA 2+838 MONOLITH JOINT
LEAD PLUG
MA-8
LAND WALL
EMBEDDED WOOD
M-6
LOWER GUIDE WALL
OPEN LIFT
MAY BE JOINT
MA-9
HEAVY BOAT DAMAGE
BOAT DAMAGE
EROSION
TOP WAS RESURFACED
LEGEND
TRAVERSE STATION
FIELD LOCATED CRACK (BY DISTRICT, SPRING '73)
PHOTO NO (FACE LOCK OF WALL)
PHOTO NO (TOP OF LOCK WALL)
CRACK IN OLD CONCRETE
CRACK IN GUNITE SURFACE
WIDE CRACKS (> 2 MM)
CRACKS, TOP OF WALLS
CAVITY
BY WES FALL '73
VERTICAL CORE
CIRCLED AREA IS EXPOSED OLD CONCRETE BENEATH GUNITE
NOTE: STATIONS SHOWN ARE MONOLITH JOINT STATIONS, EXCEPT WHERE OTHERWISE NOTED.
LOCKS AND DAM 3
MONONGAHELA RIVER
LAND AND LOWER GUIDE WALL

Figure II.2

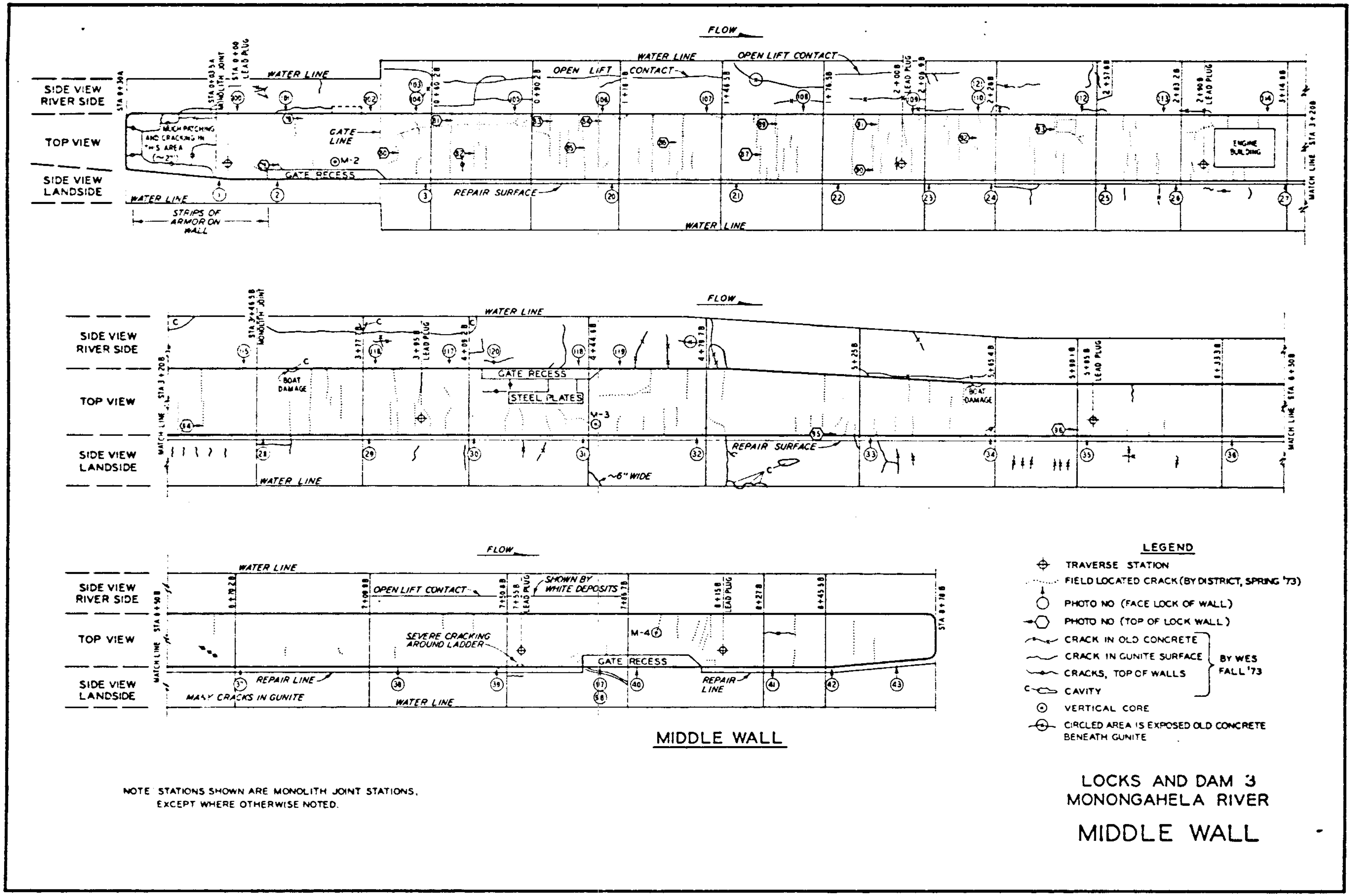

FLOW
SIDE VIEW RIVER SIDE
TOP VIEW
SIDE VIEW LANDSIDE
WATER LINE
OPEN LIFT CONTACT
GATE LINE
GATE RECESS
REPAIR SURFACE
STRIPS OF ARMOR ON WALL
ENGINE BUILDING
BOAT DAMAGE
STEEL PLATES
~6" WIDE
SEVERE CRACKING AROUND LADDER
SHOWN BY WHITE DEPOSITS
REPAIR LINE
MANY CRACKS IN GUNITE
MIDDLE WALL
LEGEND
TRAVERSE STATION
FIELD LOCATED CRACK (BY DISTRICT, SPRING '73)
PHOTO NO (FACE LOCK OF WALL)
PHOTO NO (TOP OF LOCK WALL)
CRACK IN OLD CONCRETE
CRACK IN GUNITE SURFACE
CRACKS, TOP OF WALLS
BY WES FALL '73
CAVITY
VERTICAL CORE
CIRCLED AREA IS EXPOSED OLD CONCRETE BENEATH GUNITE
NOTE STATIONS SHOWN ARE MONOLITH JOINT STATIONS, EXCEPT WHERE OTHERWISE NOTED.
LOCKS AND DAM 3
MONONGAHELA RIVER
MIDDLE WALL

Figure II.3

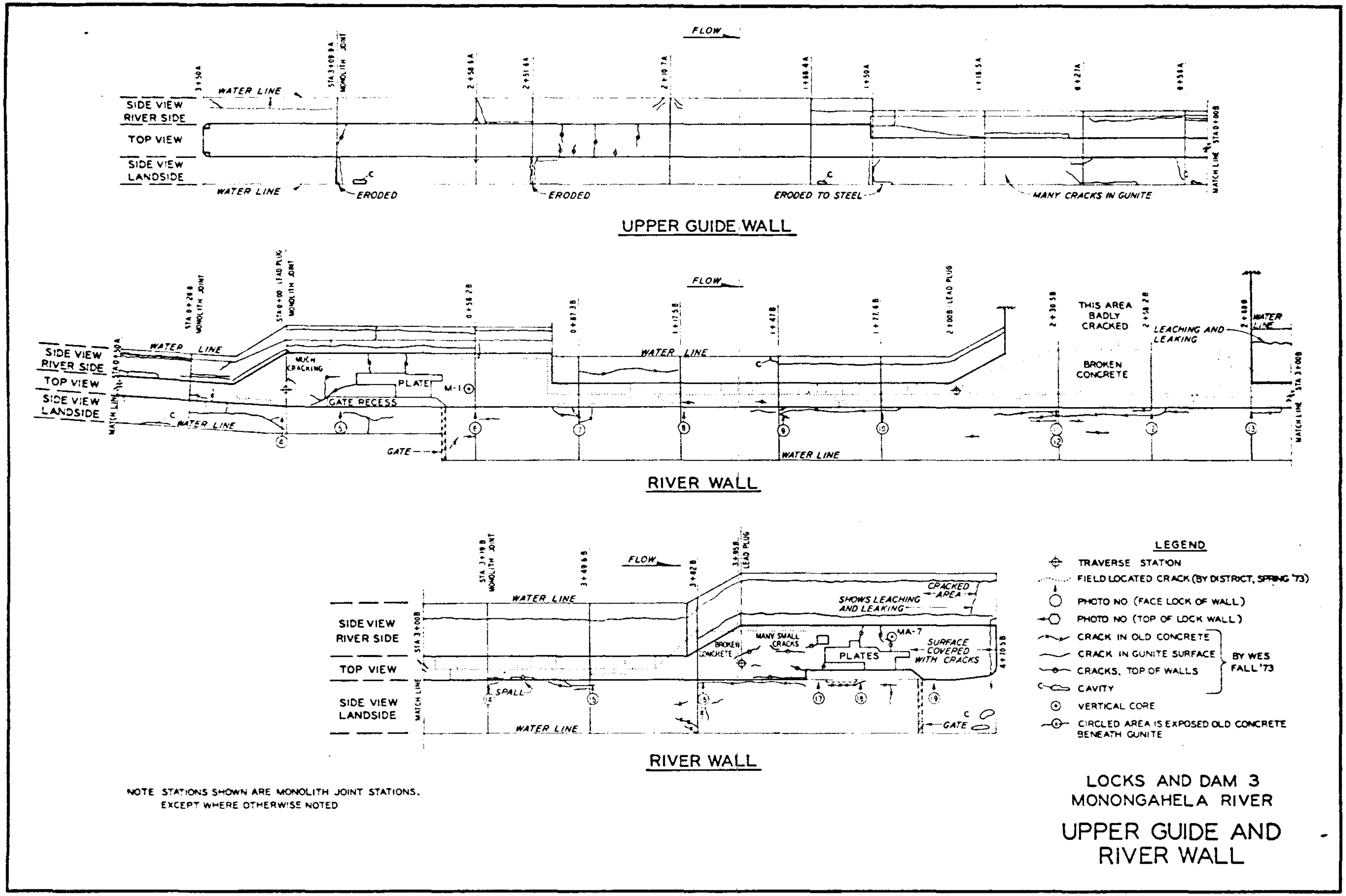

UPPER GUIDE WALL
RIVER WALL
RIVER WALL
FLOW
SIDE VIEW RIVER SIDE
TOP VIEW
SIDE VIEW LANDSIDE
WATER LINE
ERODED
ERODED TO STEEL
MANY CRACKS IN GUNITE
MATCH LINE
GATE RECESS
PLATE
GATE
THIS AREA BADLY CRACKED
BROKEN CONCRETE
LEACHING AND LEAKING
SHOWS LEACHING AND LEAKING
CRACKED AREA
MANY SMALL CRACKS
PLATES
SURFACE COVERED WITH CRACKS
SPALL
LEGEND
TRAVERSE STATION
FIELD LOCATED CRACK (BY DISTRICT, SPRING '73)
PHOTO NO. (FACE LOCK OF WALL)
PHOTO NO. (TOP OF LOCK WALL)
CRACK IN OLD CONCRETE
CRACK IN GUNITE SURFACE
CRACKS, TOP OF WALLS
CAVITY
BY WES FALL '73
VERTICAL CORE
CIRCLED AREA IS EXPOSED OLD CONCRETE BENEATH GUNITE
NOTE STATIONS SHOWN ARE MONOLITH JOINT STATIONS, EXCEPT WHERE OTHERWISE NOTED
LOCKS AND DAM 3
MONONGAHELA RIVER
UPPER GUIDE AND RIVER WALL

Figure II.4

APPENDIX III
FOUNDATION CONDITION

APPENDIX III: FOUNDATION CONDITION

Geology

Locks and Dam 3 is located on the Monongahela River just upstream from Elizabeth, Pennsylvania. The river flows in a series of meanders through the Kanawha section of the Appalachian Plateaus province within a valley cut in the Conemough and Monongahela formations of the Pennsylvanian System. Regional relief is variable between el 705 in the bed of the Monongahela River to el 1356 in the uplands. The valley at its widest point is about 2600 ft with the river comprising 1050 ft. The flat-lying rocks in the region consist chiefly of cyclic sediments composed of indurated clay, shale, limestone, and siltstone; minor amounts of coal are present. All of these rocks are members of the Middle Conemough formation. The Murrysville Anticline is the only major geologic structure in the vicinity of Lock and Dam 3. It parallels the river and lies just westward of the dam abutment. The overburden at the Lock and Dam is composed chiefly of alluvial soils in the river bottom, with colluvial deposits on the lock side and alluvial deposits in the flood plain on the abutment side. The colluvial deposits are composed of sandy, gravelly clay and sandy clay. The alluvial deposits are composed of silty, sandy gravel.

Core Drilling

Eighteen vertical cores were removed from Locks and Dam 3 and logged in the field in the latter part of 1973. These cores, along with others, were moved to the ORDL and stored. In late 1974, an examination of the 6-in.-diameter rock core recovered from Locks and Dam 3 in 1973 revealed that a majority of the weaker cores were not suitable for testing due to deterioration and drying. It is most important to ascertain the physical properties of the weaker foundation rock for the stability analysis;

therefore, it was suggested that two additional holes be drilled at Locks and Dam 3. It was anticipated that adequate samples of the weaker rock could be recovered with this drilling effort.

Two more vertical cores were drilled during the fall of 1974. The relative location is presented in Figure 3.1, Section III, and designated as M-7 and M-8. During the drilling operation, the cores were logged and steps taken to preserve the cores for testing purposes. The cores were immediately sealed in wax after removal from the core barrel. The core was exposed to the out-of-doors environment less than four minutes. About 15 ft of core per hole were returned to the WES and stored in the moist curing room while opened for inspection and during test sample selection.

Typical photographs of the backfill material and rock core recovered from M-8 and M-7 are presented in Figures III.1 and III.2 and Figures III.3 and III.4, respectively.

The backfill contains both large and small pieces of shaly rock and alluvial soil taken from the excavation of the foundation. The finer portion tends to be somewhat clayey. Two grab samples of the backfill from depths of 3 and 7 ft in hole M-7 were sealed in plastic bags and returned to WES. Since it was not practical to take large representative samples, the rock larger than a couple of inches was not included in the grab samples; therefore, the backfill samples are not totally representative of the inplace material.

Core Logs and Geologic Cross Section

The core logs were examined and geologic cross section drawn up to aid in selecting representative samples for testing. A total of 20 core logs and 7 borehole photo logs were reviewed. Copies of the original core logs and photo logs are given in Reference 1 . The 20 core logs are identified in Table III.1. Since the rock cores were logged by several people, there were some differences in terminology.

The logs were redrawn to a more convenient scale and arranged in seven groups by elevation and by location in the structure. The groupings are described in the following tabulation by sections.

Identification of Section	General Location of Section	Approx Section Length, ft	No. of Core Logs	List of Core Logs
		Up- and Downstream		
A	Along Land Wall	1600	13	MA-10, MP-1, M-8, M-5, MR-1, MA-11, M-7, MR-2 MA-8, MR-3, M-6, MA-9, and MP-3
B	Along Middle Wall	750	4	M-2, MA-12, M-3, and M-4
C	Along River Wall	400	3	M-1, MA-13, and MA-7
		Across Stream Flow		
D	Normal to River at Upper Gates	150	3	M-5, M-2, and M-1
E	Normal to River Near the Dam	150	3	MA-11, MA-12, and MA-13
F	Normal to River at Lower Gate of Small Lock	150	3	MA-8, M-3, and MA-7
G	Normal to River at Lower Gate of Large Lock	75	2	M-6 and M-4

Preexisting breaks, fracture zones and compositional features of the foundation material are shown on the logs in Sections A through G, Figures III.5 through III.11. Figure 3.1, Section III, indicates the orientation of Sections A through G. These features were placed on the logs after study of the original logs and the borehole photo logs for the seven holes where borehole photographs were made (MA-7 through MA-13).

Discussion of Core Logs

The foundation rocks recovered by drilling are essentially flat-lying, cyclic sediments consisting mainly of indurated clay, shale, siltstone, and limestone; minor amounts of coal are present in the foundation.

The normal sequence of rocks encountered going down a typical hole appears to be as follows:

a. Several feet of a medium-gray (N5),[3] hard shale or siltstone rock that does not fragment on drying. When this rock comes from the hole with its natural moisture content it is bluish-green in color, but as indicated above, it is grayish when dry. This is the rock that made up about 95 percent of that available for testing in cores M-7 and M-8. A number of the logs indicate fragmented rock at and for about 9 ft below the contact of concrete and rock. The fragmented zone appears not to be continuous over the lock site and could be localized zones caused by excavation techniques such as blasting. Three hand samples of this type of rock from holes M-7 and M-8 were selected for a petrographic examination. They represented a vertical interval of 11 ft and a horizontal interval of 250 ft. They were all similar in appearance and in composition. The composition of the three samples will be discussed in Section V.

b. A few feet of black carbonaceous fissile shale which does fragment on drying were found at greater depths. There was about 1.7 ft of this material in short pieces in the bottom of core M-7. None of it was present in M-8. There is no additional fragmentation once dried fragments are rewetted. This rock differs in composition from the grayish shale by containing organic material, more siderite, a small amount of a mixed-layer clay (probably montmorillonite and clay-mica), and smaller amounts of the other minerals.

c. Six to twelve inches of coal occurs at about el 690. None of this was available for testing.

d. Another few feet of the black carbonaceous shale were next in depth.

e. Several feet of limestone were found below the coal. Apparently this material can be or appear to be a limy shale or clay. None of this was available for testing.

f. The cycle then starts over with the hard medium-gray shale.

Discussion of Geologic Cross Sections

Sections A through G (Figures III.5 through III.11) provide graphic representation of the cores. The cores are properly aligned by elevation and by position in the structure. In trying to study geologic data such as those provided by the logs in these sections, a marker bed is used for correlation. The coal bed is the most distinctive material encountered and therefore used as the marker bed. Unfortunately, the coal did not show up in any of the borehole camera results for seven holes (MA-7 through MA-13).

The brittle nature of the coal could have caused it to shatter during drilling and fall from the walls of the core holes, thus leaving a void which should appear as a void in the photo logs. There are several void areas described in the borehole photo logs at approximate elevations where coal occurs in the core logs. Therefore, the contact between concrete and rock was taken as the distinctive feature for the borehole data. Comparison of the location of this contact in the core logs and the borehole camera logs indicated reasonably good agreement for all of the MA holes except MA-10; there was a 10-ft discrepancy for that hole. Since some loss of core usually occurred, the contact between rock and concrete was located according to the camera data when there were discrepancies; both locations of these contacts are shown on the geologic cross sections (Sections A through G). The effect of correcting the concrete-rock contact for core MA-10 was to produce a much better alignment of the coal bed with its counterpart in the other holes. This appears reasonable when consideration is given to the fact that no major faulting has thus far been described in the available literature. Comments about each of the seven sections follow.

a. Section A. This section along the land wall includes 13 holes over a span of about 1600 ft. The correlation by lithology is considered satisfactory. While there are old breaks and/or fracture zones in most of the cores, they do not seem to be continuous. The borehole photo logs tend to substantiate this belief. The coal, which is probably one of the weakest materials in the rock sequence, is always at least 15 ft below the contact between concrete and rock.

b. Section B. This section along the middle wall includes four cores over a span of about 750 ft. The lithologic correlation is better than in Section A; there are no continuous breaks or fracture zones. The weak coal bed is at least 11 ft below the concrete-rock contact in three of the holes (M-2, MA-12, and M-4); it is within 3 ft of this contact in hole M-3 which is at Station 4+48B in monolith 16.

c. Section C. This section along the river wall includes three cores over a span of about 400 ft. The lithologic correlation is not good. A couple of inches of coal are present in hole M-1 between 688 and 689 ft as the contact with the overlying concrete. The coal is missing or was missed in the middle core log (MA-13) and was probably removed by excavation during construction from the area penetrated by hole MA-7. No continuous cracks are indicated.

d. Sections D, E, F, and G. These represent 11 of the same 20 cores. They have been arranged in groups of two to three cores along lines normal to the direction of the river; the length of the sections ranges from about 75 to 150 ft. Study of these sections did not yield additional information pertinent to an evaluation of the foundation condition. The sections are presented to complete the record and for possible future use.

The hard gray shale was selected for physical property testing because it represented about 95 percent of the rock that was adequately preserved. In our opinion, it does not represent the weakest rock type described in the previous logs. The indurated clay, coal, and possibly other shales are considered as rocks which should be tested, but because they were not preserved and because the more recent drilling did not supply these weaker rocks, the hard gray shale was all that was available. Generally, the less competent material was deep enough under the locks not to exert any detectable influence. However, because weaker rocks are known to exist in the foundation, the stability of monolith R-12 was checked using assumed values of $\phi = 15^{\circ}$ and $C = 0$. The ϕ is assumed to represent the angle of friction of concrete on coal. The results of this particular stability check are presented in Section VI.

Borehole Photo Logs

The borehole photo logs were briefly reexamined for supplemental information concerning the foundation condition. In particular, an examination of the borehole photo logs was made to see if any discontinuities reflected the Murrysville Anticline. The anticline is about parallel to the river and lies slightly landward of the dam abutment. This would align the anticline in a N-S direction.

The majority of discontinuities are striking NE within a range of from 10 to 70 degrees and appear to be subdivided into two groups. One group is represented by "fractures" bearing from N10E to N30E and have low angle dips of from 15 to 30 degrees E. The other group is represented by "fractures" bearing from N55E to N70E and have dips up to 40 degrees E and SE. These discontinuities could possibly be associated with the NS trending anticline but adequate data are not available to make a conclusive statement. Joint sets or any other regular or repetitive structural feature in the foundation with the exception of bedding planes are not in evidence.

The general condition of the foundation is considered good. The core boring information indicates no detectable continuous discontinuities except relatively horizontal bedding planes, zones of fracture, or seams of weak material. There are, however, localized zones of fractured rock which are considered weak zones. These zones are discussed in Section III.

Table III.1

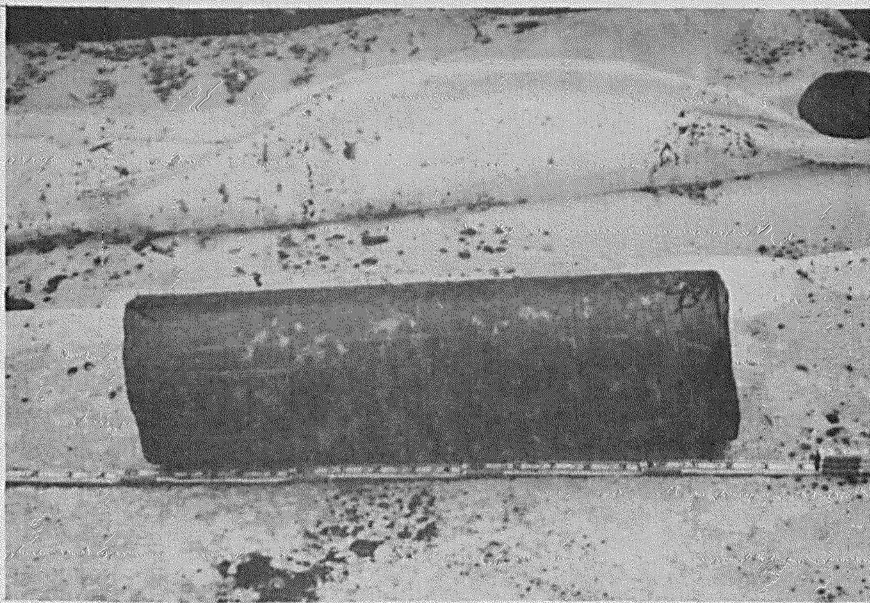
Identification of Core Logs

Drill Hole No.	Nominal Core Diameter	Material at Top of Hole	Material Represented in Core Log
M-1 M-2 M-3 M-4 M-5 M-6	6 in.	Concrete	Concrete and Rock
MA-7 MA-8 MA-9 MA-10 MA-11 MA-12 MA-13	NX (2-1/8 in.)	Concrete	Concrete and Rock
MP-1 MP-2	NX (2-1/8 in.)	Backfill	Backfill and Rock
MR-1 MR-2	NX (2-1/8 in.) NX (2-1/8 in.)	Backfill Backfill	Backfill and Rock
MR-3	NX (2-1/8 in.)	Concrete	Concrete Slab, Backfill, and Rock
M-7 M-8	6 in. 6 in.	Backfill Backfill	Backfill and Rock

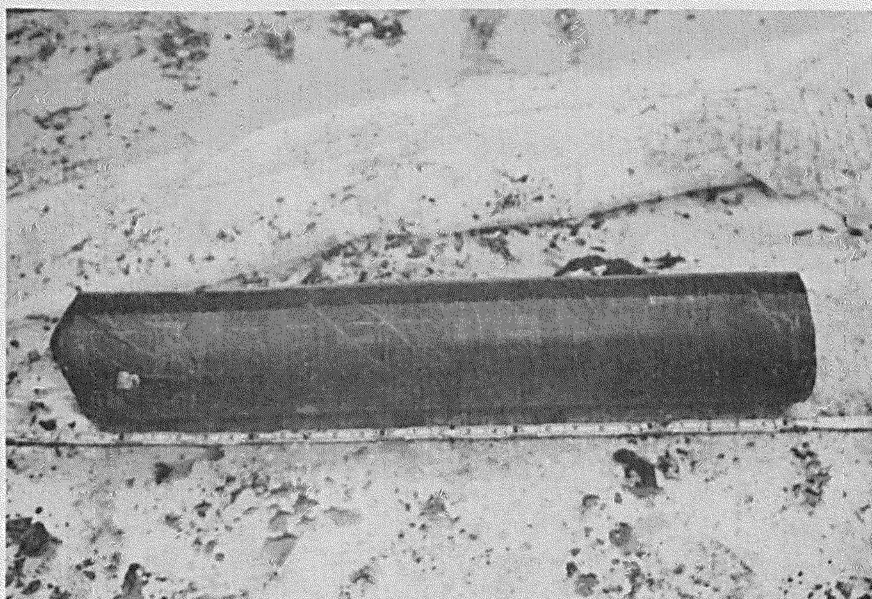
a. Backfill Recovered with Auger

b. Backfill Recovered with 6-in.-Diameter Core Barrel

Figure III.1. Photographs Showing Backfill (M-8).



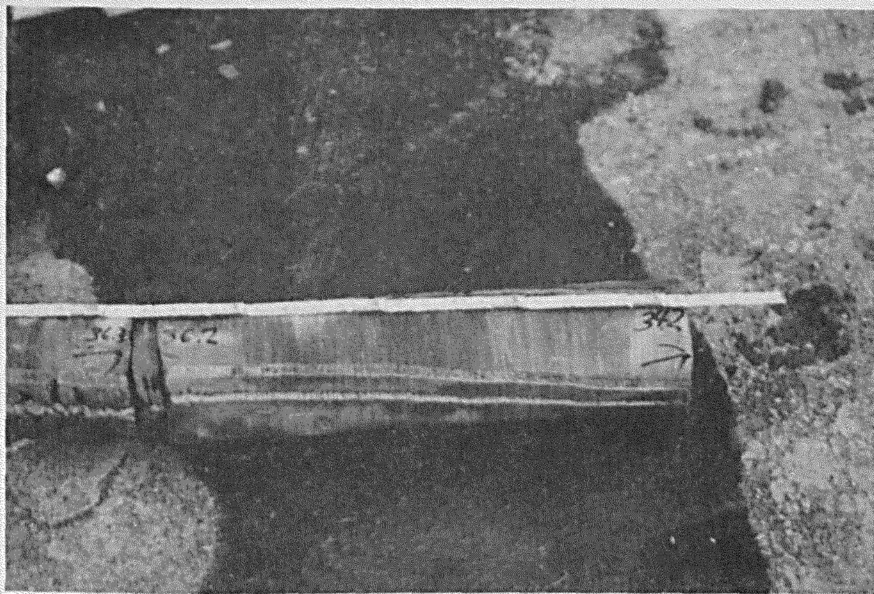
a. Blue-Green Shale



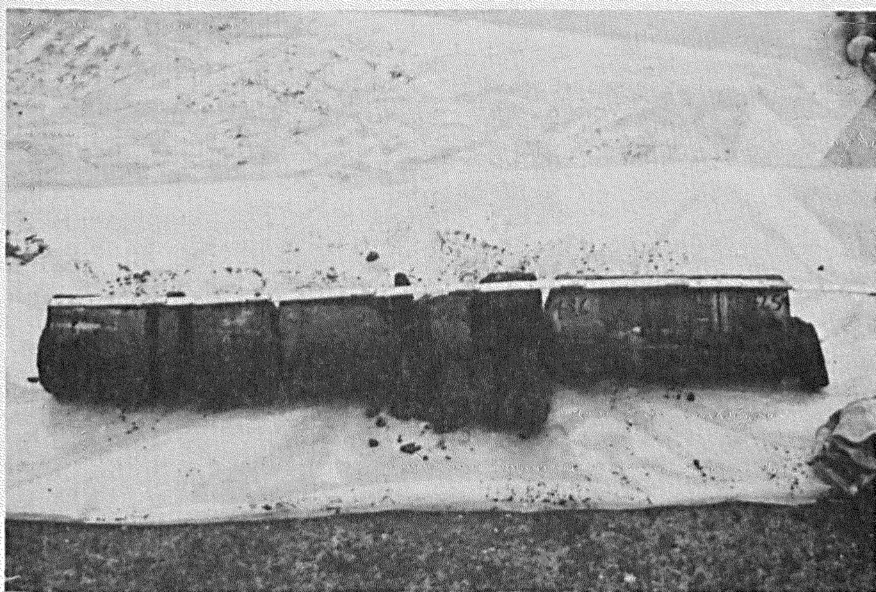
b. Blue-Green Shale

Figure III.2. Core Recovered with 6-in.-Diameter Core Barrel (M-8).

Figure III.3. Backfill Removed with 6-in.-Diameter Core Barrel (M-7).



a. Blue-Green Shale



b. Black Shale

Figure III.4. Core Recovered with 6-in.-Diameter Core Barrel (M-7).

LEGEND FOR SECTION A-G

Clay

Indurated Clay (In. Clay) and Claystone (Clst)

Compaction Shale (Cm Sh)

Shale (Sh)

Limy Shale (Limy Sh)

Limestone (Ls)

Siltstone (Sltst)

Coal

Backfill (Bf)

Concrete (Conc)

Topsoil

Silt Shale (Slt Sh)

Break

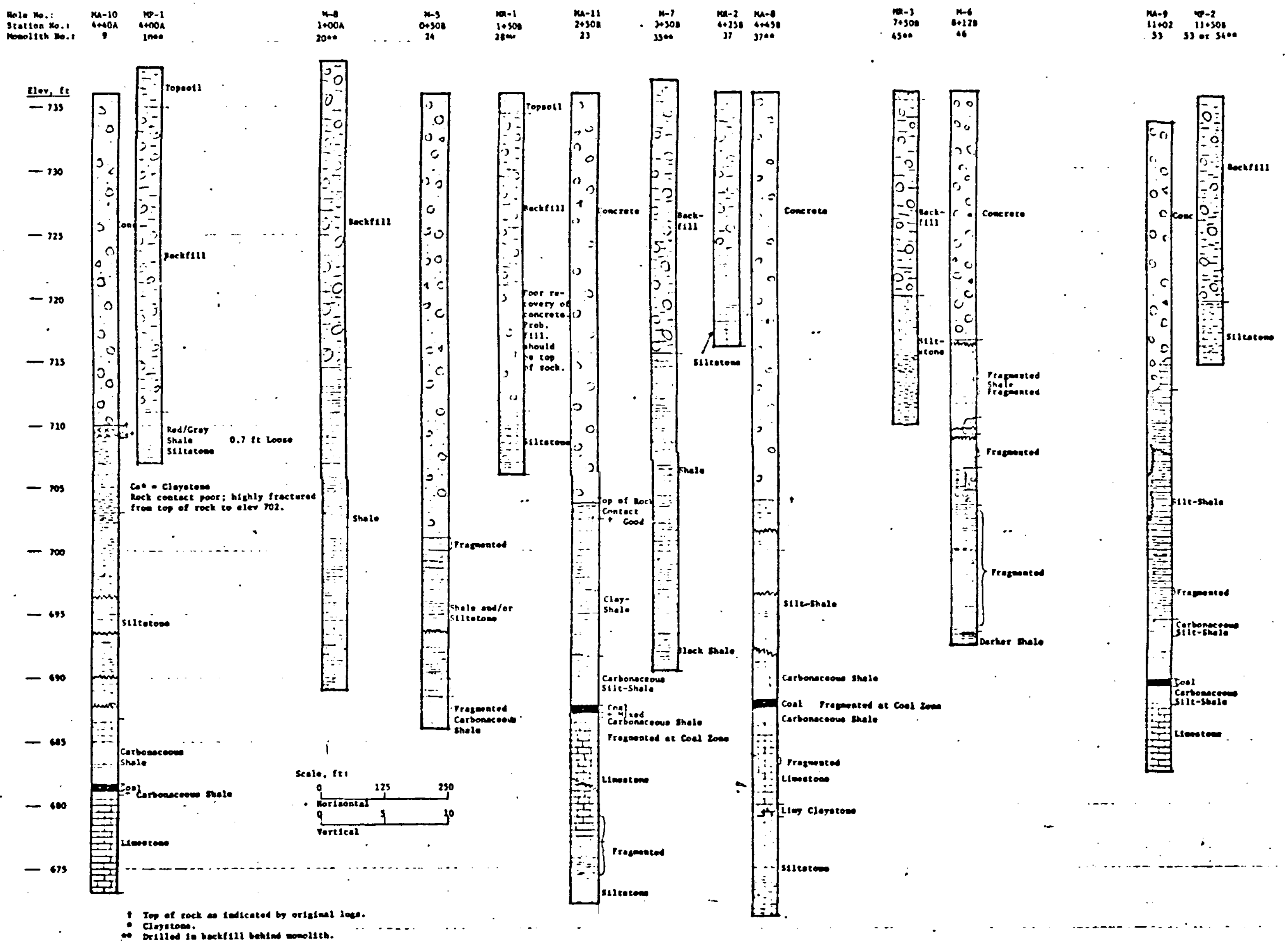
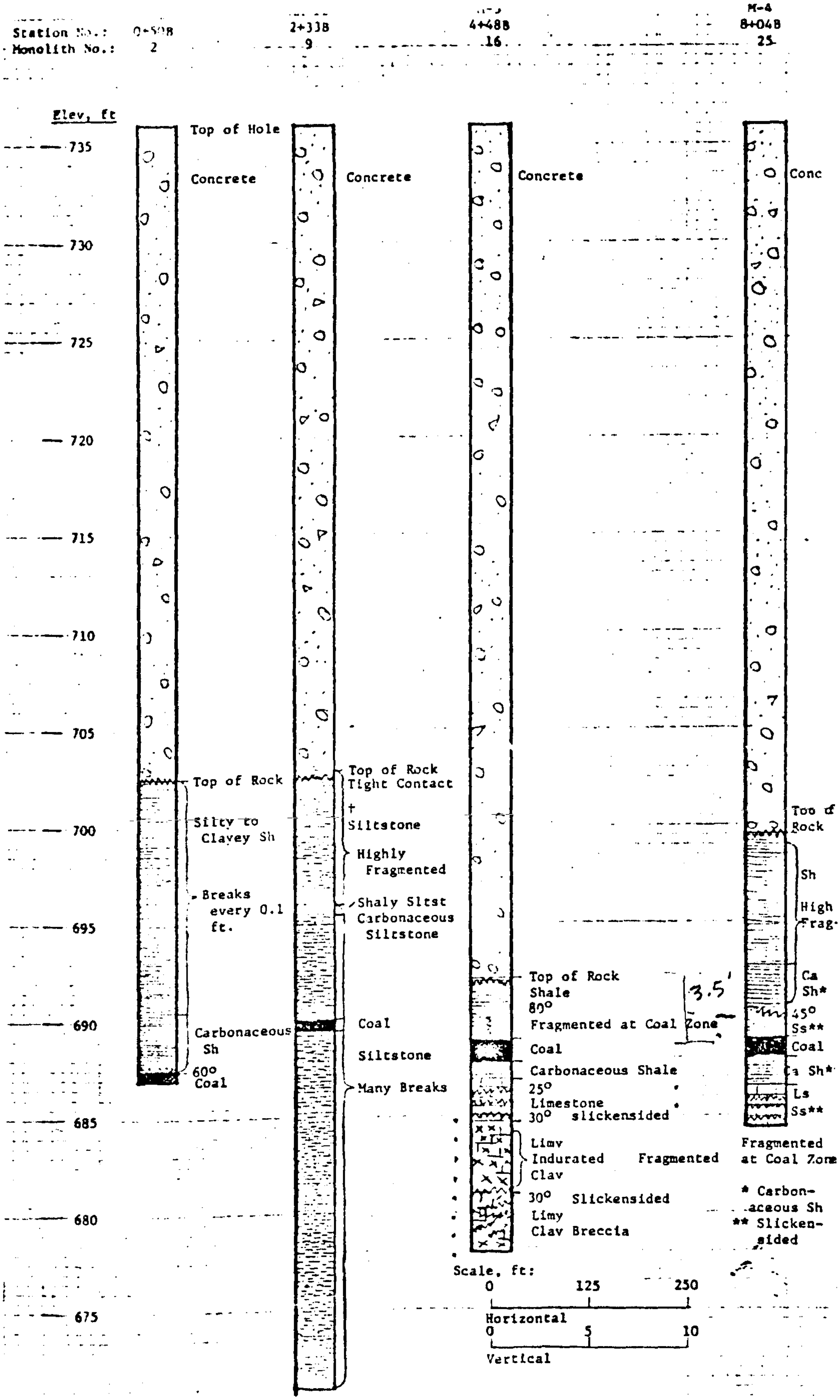
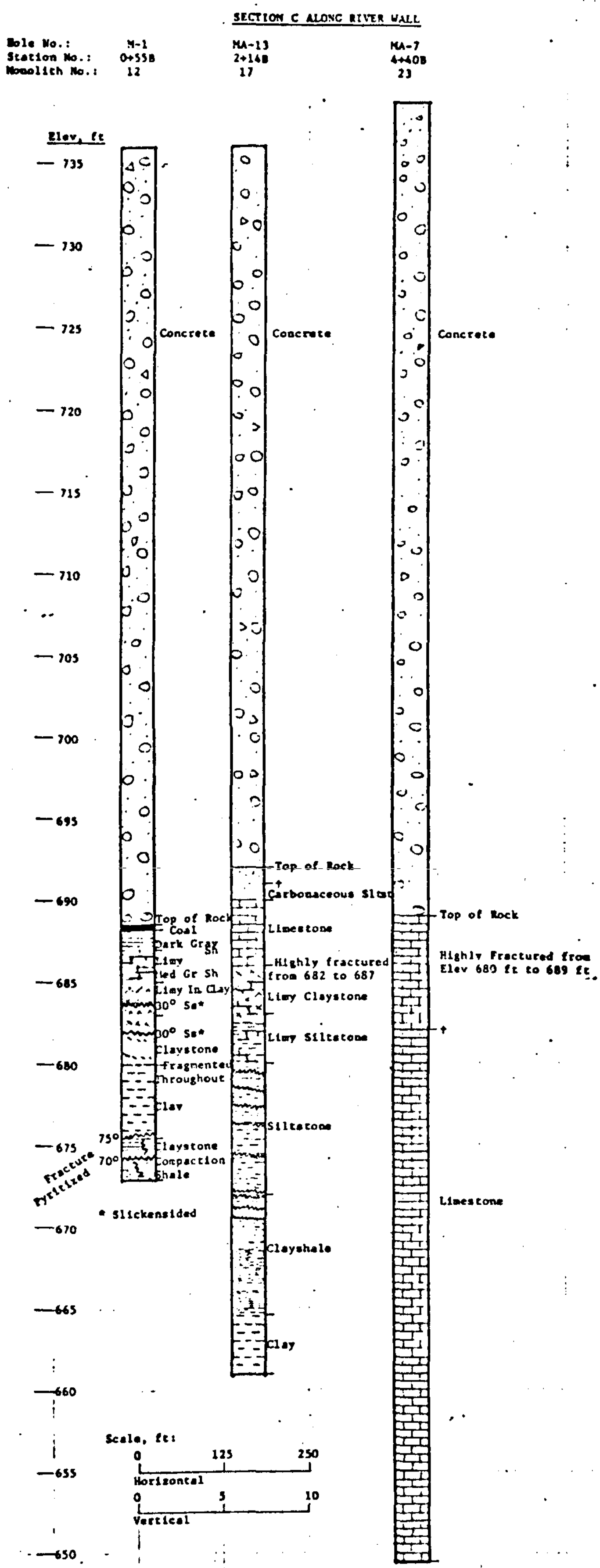

Hole No.: MA-10 MP-1 M-8 M-5 MR-1 MA-11 M-7 MR-2 MA-8 MR-3 M-6 MA-9 MP-2
Station No.: 4+40A 4+00A 1+00A 0+50B 1+50B 2+50B 3+50B 4+25B 4+45B 7+50B 8+12B 11+02 11+50B
Monolith No.: 9 1** 20** 24 28** 23 33** 37 37** 45** 46 53 53 or 54**
Elev, ft
735
730
725
720
715
710
705
700
695
690
685
680
675
Topsoil
Backfill
Concrete
Conc
Red/Grey Shale Siltstone
0.7 ft Loose
Ca* = Claystone
Rock contact poor; highly fractured from top of rock to elev 702.
Poor recovery of concrete. Prob. fill. should be top of rock.
Siltstone
Shale
Top of Rock Contact Good
Fragmented
Shale and/or Siltstone
Clay-Shale
Black Shale
Silt-Shale
Darker Shale
Carbonaceous Silt-Shale
Carbonaceous Shale
Fragmented Carbonaceous Shale
Coal
Coal Mixed Carbonaceous Shale
Fragmented at Coal Zone
Limestone
Limy Claystone
Scale, ft:
0 125 250
Horizontal
0 5 10
Vertical
† Top of rock as indicated by original logs.
* Claystone.
** Drilled in backfill behind monolith.

Figure III.5



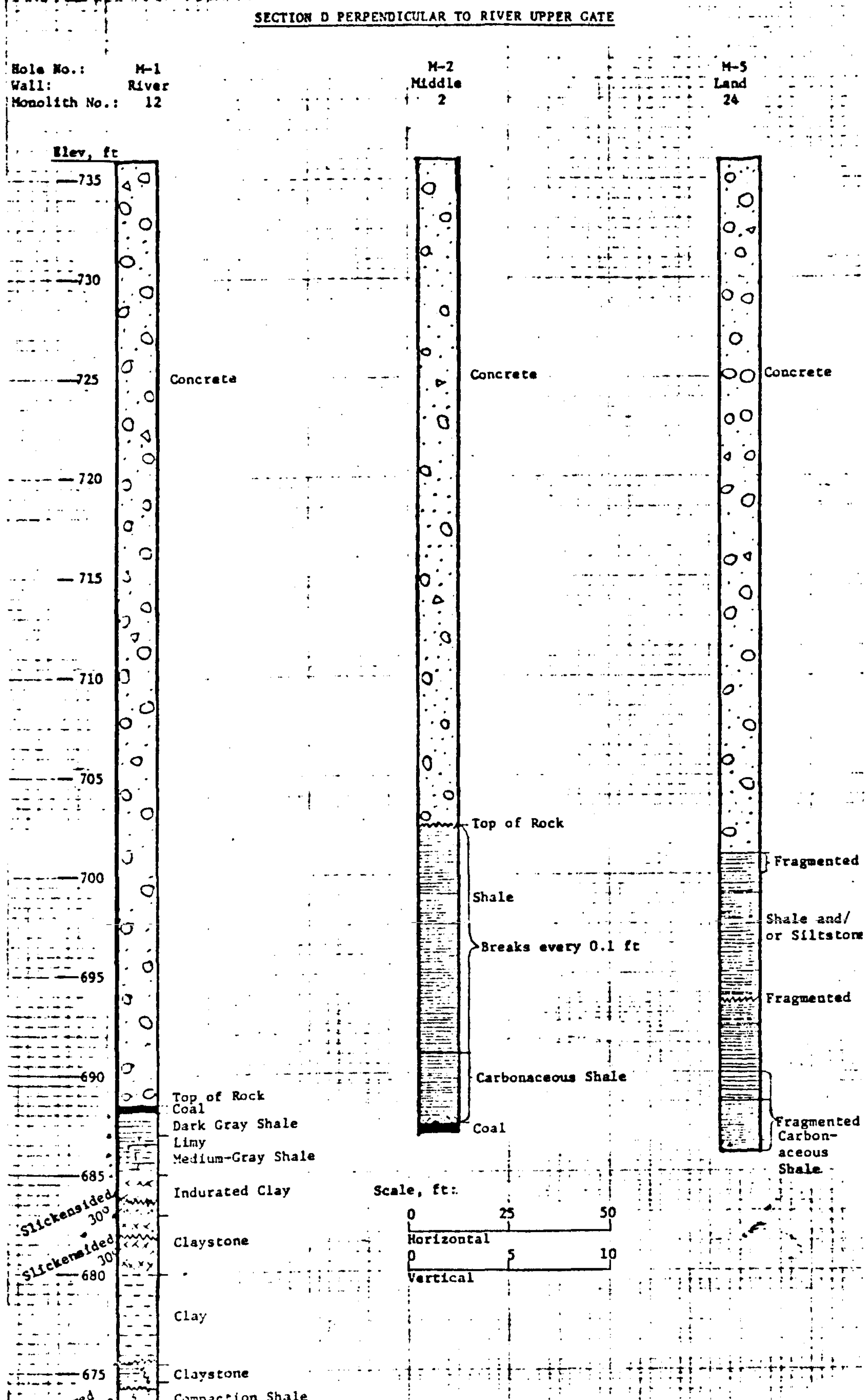
† Top of rock as indicated by original logs.

Figure III.6



† Top of rock as indicated by original logs.

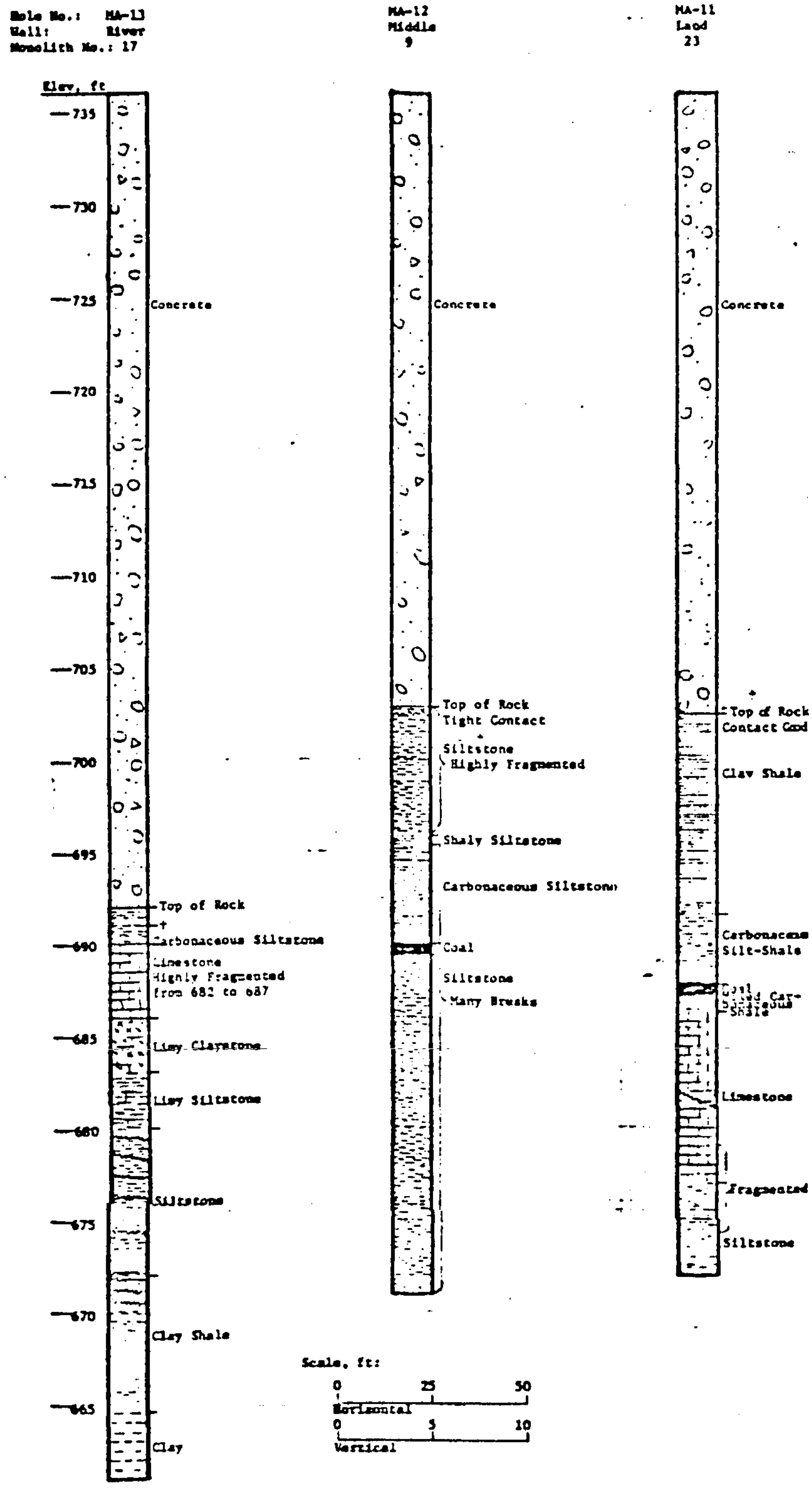
Figure III.7



SECTION D PERPENDICULAR TO RIVER UPPER GATE
Hole No.: M-1 M-2 M-5
Wall: River Middle Land
Monolith No.: 12 2 24
Elev, ft
735
730
725
720
715
710
705
700
695
690
685
680
675
Concrete
Top of Rock
Coal
Dark Gray Shale
Limy
Medium-Gray Shale
Indurated Clay
Slickensided 30°
Claystone
Slickensided 30°
Clay
Claystone
Compaction Shale
Fractured 75° and 70° Pyritized
Concrete
Top of Rock
Shale
Breaks every 0.1 ft
Carbonaceous Shale
Coal
Concrete
Fragmented
Shale and/ or Siltstone
Fragmented
Fragmented Carbon-aceous Shale
Scale, ft:
0 25 50
Horizontal
0 5 10
Vertical

Figure III.8

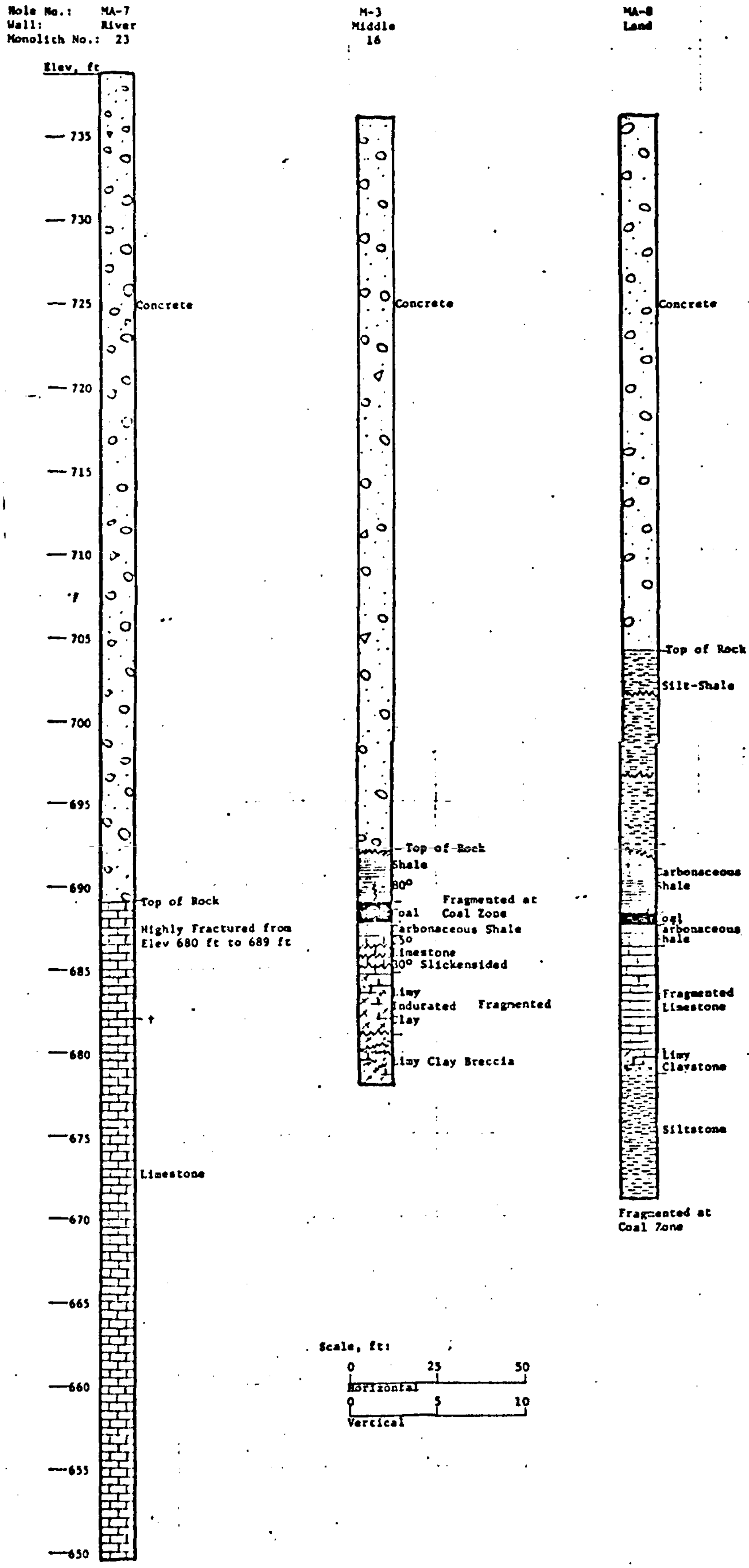
SECTION E PERPENDICULAR TO RIVER NEAR DAM



† Top of rock as indicated by original logs.

Figure III.9

SECTION F PERPENDICULAR TO RIVER NEAR LOWER GATE OF SMALL CHAMBER



† Top of rock as indicated by original logs.

Figure III.10

SECTION G PERPENDICULAR TO RIVER NEAR LOWER GATE OF BIG CHAMBER

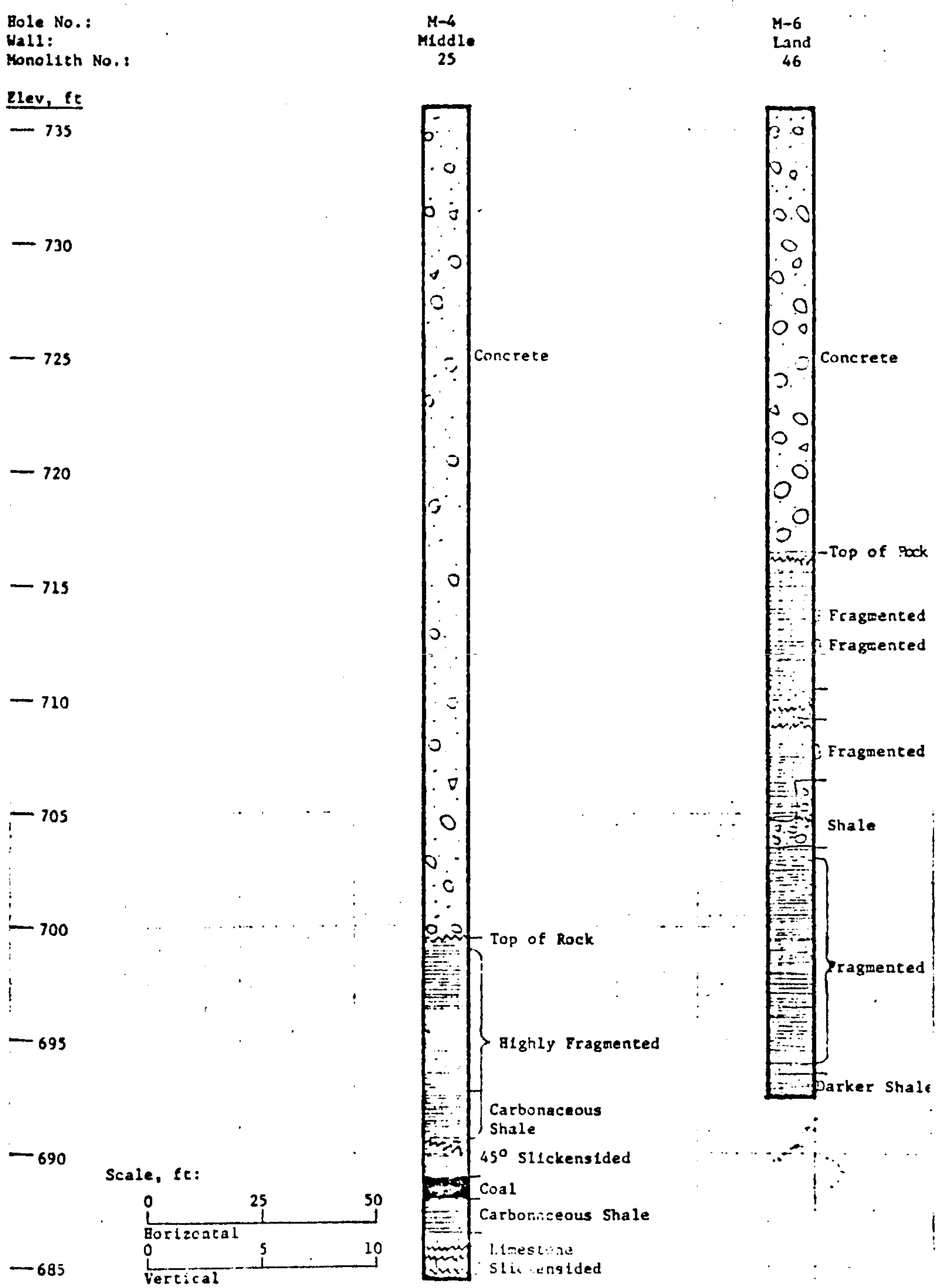


Figure III.11

APPENDIX IV

CONCRETE INTEGRITY

APPENDIX IV

CONCRETE INTEGRITY

Borehole Photo Logs

The borehole photo logs were reviewed for supplemental information concerning the condition of the internal concrete.

The photo logs are quite useful in assessing the internal concrete. The lateral extent of any detectable feature such as honeycombing, fractures, or leached areas cannot be determined without extensive work. A feature in one borehole cannot necessarily be related to a similar feature at the same elevation in another monolith. The reason for this is that the NX boreholes in which camera data were obtained are widely spaced. Four holes were photographed in the landside wall, one in the upper and lower guide wall each, and two in the land wall; one hole was photographed in the middle wall and two holes were photographed in the river wall. Several hundred feet separate most of the holes. If a feature does show up that appears to be continuous, it is probably coincidental. For example, in holes M-1, M-2, M-4, and M-5 at or very near el 712, the concrete is described in the core logs to be badly honeycombed, containing voids, slightly weathered, and well weathered. Certainly these features could not be related to a specific time in the construction of the locks because holes M-1, M-2, and M-5 were drilled in concrete cast in 1902-05 while hole M-4 was drilled in concrete cast in 1923-24.

The borehole camera records indicate, in most cases, intact concrete in those areas where the core logs indicate loose aggregate and high core loss. Recovery of loose aggregate tends to relate to honeycombed concrete broken up in drilling. The core log for the vertically drilled NX hole, MA-7, shows core losses of from 0 to 100 percent. The 100-percent core loss occurred in the first 2-1/2 ft of the hole. A 60-percent core loss occurred at about 18 ft into the concrete. The photo logs indicate "good concrete" for both of these areas of high core loss. A reasonable explanation for this apparent conflict is as follows.

If reasonable care is used in coring concrete in good condition, the recovery of NX core and of 6-in.-diameter core is similar. This is not true with concrete in poor condition where quality and amount of core recovery increases with increasing core diameter. The condition of concrete in these locks is poor enough that the recovery of NX core was not comparable to the recovery of 6-in.-diameter core at certain depth intervals over the lock walls.

One major vertical crack which extends from a depth of 2.4 to 37.4 ft was detected by the borehole camera in hole MA-7. Hole MA-7 is in the downstream river wall gate monolith, R-23. The hole is located between station 3+82B and 4+70.5B. The directional information about this crack in Reference 1 shows that it strikes NW so, if continuous, it may be exposed in the river face of the wall and the end face or possibly the land face downstream of the gate recess. Figure II.4 in Appendix II shows a near vertical crack between station 3+82B and 4+70.5B on the river side of this monolith. This is the same crack described in Appendix II as showing signs of leaching and leaking. The evidence is very strong that the crack observed in hole MA-7 is continuous to the riverside of the monolith.

The photo logs show three instances of poor contact between the concrete and foundation rock. In holes MA-7 and MA-10, the rock directly below the concrete is highly fractured. Numerous voids for a zone 6 ft in length are described in the photo log of hole MA-10. The extent of the fractured material is not known but remedial action should be considered to guard against possible failure in these particular zones.

Some drilling water loss was noted in Reference 1 and probably indicates localized rather than general leakage; some or all of the loss could be along construction joints. Drill water was not observed to be coming out of the walls through such joints.

Concrete Test Results

The compressive strength and unit weight test results presented in Reference 1 were reviewed. In general, the compressive strength of concrete can be used as an indicator of concrete quality. Most of the strength results of concrete recovered from el 731.8 through 702.2 (see Appendix E of Reference 1) in holes M-1, M-2, M-3, M-4, M-5, and M-6 would normally be considered as representing concrete of high quality. Of these 34 strength results, only four (about 12 percent) represent inplace concrete having a compressive strength less than 2500 psi. A 2500-psi concrete is considered acceptable for a gravity structure. The four samples having strengths below 2500 psi are considered to represent a small portion of the concrete in the locks for the elevation interval cited above.

Should one of these low strengths represent concrete in a critically stressed region such as a gate monolith, a local failure could occur. As mentioned in Section IV, three of these four low strengths represent inplace concrete in monolith M-16 which is the downstream gate monolith in the middle wall. The three strengths are considered quite low and are 1150, 1450, and 2100 psi. Granted, these are compressive strengths and a total compressive stress of say 1150 psi may never be placed on the concrete in monolith M-16. However, the tensile and shear components of a 1150-psi concrete could be exceeded in this monolith.

The tensile and shear strength of normal weight concrete is about 0.1 and 0.16 of the compressive strength, respectively; for the 1150-psi concrete, the tensile and shear strength would be 115 and 184 psi. The gate loads for M-16 are the same as for L-46 whose tensile stresses were considerably above the allowable; 102 psi for the same location as the 1150 psi concrete in M-16. Even though the loading and geometry is different for L-46 and M-16, the low strength concrete in M-16 could be stressed above its allowable and as deterioration continues structural problems can develop.

The unit weights presented in Appendix E of Reference 1 are considered reasonable and indicate that the unit weight of the inplace concrete is generally consistent.

The general observations presented in Appendix E of Reference 1 also address the question of the quality of the internal concrete:

"Based on the results of the laboratory logging, the petrographic examination, and the unconfined compression tests, the following observations relative to the condition of the concrete are noted:

"1. There is no evidence of any alkali-aggregate reaction in the concrete.

"2. The large amount of ettringite present in the voids suggests a sulfate reaction with the concrete; however, only in a few small areas does it appear that the sulfate attack has actually caused any deterioration of the concrete."

In general, the quality of the interior concrete is good. However, poor quality concrete does exist in a gate monolith in the middle lock wall. It is areas of low quality concrete such as present in monolith M-16 that can cause a structural failure. Deterioration of the concrete is continuing in the lock walls, and it is quite possible that within a 10-year period the poor concrete in monolith M-16 will become worse and contribute towards a structural failure.

APPENDIX V

TEST RESULTS AND ANALYSIS OF RESULTS

APPENDIX V

TEST RESULTS AND ANALYSIS OF RESULTS

Backfill Material

The unit weight values obtained using the two approaches mentioned in Section V were compared and a drained value of 135 lb/ft^3 was selected for the backfill. The undrained unit weight was slightly higher than the drained unit weight, but considered inconclusive and therefore not used in the structural analysis.

Tests of the two backfill samples from hole M-7 gave the following results:

a. Grading:

Sieves	Percent Retained	Cumulative Recalculated to Include 10% of Larger Rock
1 in.	27	24
No. 4	25	23
No. 10	6	5
Passing No. 10	42	38

b. Specific gravity of rock: The value for the combined rock larger than No. 10 sieve was 2.53.

c. Moisture content: This value was 16.2 percent.

Concrete Test Results

As mentioned earlier, there were a limited number of cores available for testing. One core was tested in unconfined compression, and its strength was 3070 psi. This value is well within the range of strength values presented in Reference 1. The stress-strain relations for this specimen and the triaxial specimens are presented in Figure V.1. A modulus of elasticity was computed as an initial tangent value and was 3.33×10^6 psi.

The initial modulus was selected because it would more nearly be representative of the inplace concrete; i.e., concrete under rather low stress conditions as normally associated with a gravity structure. A Poisson's ratio was calculated and found to be 0.13.

The unit weights of the three concrete specimens are tabulated below.

Hole No.	Elevation, ft	Unit Weight, lb/ft^3
M-1	728.2	144.6
M-2	715.2	141.3
M-3	702.9	145.7

The unit weights are reasonable and are well within the range of unit weights presented in Reference 1.

The stress-strain relations for the two specimens tested under triaxial conditions are presented in Figure V.1. An E and ν were calculated for the specimen tested with a confining pressure of 500 psi and found to be 4.00×10^6 psi and 0.20, respectively. Figure V.2 presents the principal stress difference $(\sigma_1 - \sigma_3)$ versus principal strain difference $(\varepsilon_a - \varepsilon_d)$ from which an initial tangent shear modulus was computed. The shear modulus is 1.18×10^6 psi and compares reasonably with the shear modulus calculated using:

$$G = \frac{E}{2\,(1 + \nu)} \qquad (1)$$

The triaxial data were basically used to obtain a shear modulus for the concrete as it was required for the structural analysis. The E's presented in Reference 1 and the E's determined during this study plus Poisson's ratio are tabulated below.

	Original Concrete		Newer Concrete	
	E, 10^6 psi	v	E, 10^6 psi	v
	4.00	0.2*	5.11	0.2*
	4.04			
	3.33	0.13		
	3.05			
Avg	3.6	0.165	5.11	0.2

* Assumed value.

The average E and v for the original concrete were used to compute a G value using Equation 1; comparative results are tabulated below.

G Calculated From	G
Triaxial Data	1.18×10^6 psi
Equation 1	1.55×10^6 psi

An approximate difference of 25 percent exists, which is considered reasonable.

Rock Test Results

The results of the unit weight, unconfined compression, tensile, and standard triaxial tests are presented in Table V.1.

The unit weights are consistent and reasonable for medium-hard dense shale. The lowest unit weight was 164.0 lb/ft^3 and the highest was 171.2 lb/ft^3. The average unit weight was 168.7 lb/ft^3, and the range was 7.2 lb/ft^3.

Three unconfined compression tests were conducted with the stress-strain relation being recorded for two specimens. The σ-ε results are presented in Figure V.3. The average strength is 3960 psi. During testing, water was observed coming out of the specimen from bedding planes and small vertical cracks at a stress level of about 800 psi. This stress level corresponded to the point on the σ-ε curve where the curve started to become concave downward. Water continued to be extruded from the specimens until a stress level corresponding to the second linear portion of the $\sigma-\varepsilon$ curve. The initial portion of the curve was linear to about 800 psi. This is a typical response for rock core representing none or little stress relieving upon being removed from its in-situ position. The middle portion of the curve reflects increased strain as free water was extruded from the core. The final portion of the curve is very nearly the same as the initial portion; the slopes are almost identical. The moduli were taken as initial tangent values due to the relatively low stress states assumed for the foundation; the average $E = 0.59 \times 10^6$ psi, which appears reasonable for this type of shale. Poisson's ratios were calculated for the initial portions of the curves; the average $\nu = 0.17$. Presented in Figure V.4 the $\sigma-\varepsilon$ relation for the test check specimen; the shale shows hysteresis upon unloading. The test check specimen was run to verify the ε response obtained under triaxial testing.

The average of two tensile splitting strength tests is 700 psi, which is reasonable for the competent medium-gray shale.

Three standard undrained triaxial tests were conducted on the shale from hole M-8 using σ_3's of 100, 500, and 1000 psi. The $\sigma-\varepsilon$ data for the triaxial tests are presented in Figures V.5-V.7. The $\sigma-\varepsilon$ relationship is the same as that observed for the specimens tested in unconfined compression. The initial E and ν are similar to the E and ν as were calculated for the specimens tested in unconfined compression. Shear moduli were calculated from principal stress difference ($\sigma_1 - \sigma_3$) and principal strain difference ($\varepsilon_a - \varepsilon_d$) data. The average shear modulus is 0.24×10^6 psi.

Figure V.8 presents the Mohr's diagram for the results obtained from the strength tests. The angle of internal friction (ϕ) for the initial portion of the failure envelope, defined as being tangent to the tensile stress circle and the average circle of three unconfined stress circles, is 45 deg. The cohesion (c) is 850 psi. At a $\sigma_n \simeq 700$ psi, the failure envelope has an angle of 31 deg. The initial ϕ and the c are reasonable for the competent shale.

Six direct shear tests were conducted for peak strengths on intact shale recovered from near the contact between the concrete and the foundation material in Hole No. M-7. Specimens were selected from a depth interval of 9.4 to 10.7 ft below the concrete-rock contact. Most of the pieces of core closer to the contact were of an inadequate length for testing. Two series of three specimens each were tested using normal stresses, σ_n, of 33, 66, and 100 psi (2.37, 4.75, and 7.20 tons/ft^2). The range of σ_n is well within the range of σ_n's anticipated at Locks and Dam 3.

The results of the first and second series of direct shear tests are presented in Figures V.9 and V.10, respectively. The shear stress (τ)-shear deformation are considered reasonable for the intact competent shale. Figure V.11 is a composite plot of the two test series. A least squares fit was applied to the data which yielded a $\phi = 68^{\circ}55'$ and $c = 112$ psi (8.06 tons/ft^2).

The results of the multistage test can be used to calculate the shearing stress (τ) across an established surface for various values of the normal stress (σ_n). The coefficient of friction (ϕ_j) on the surface can then be determined. When the principal stresses are known and the τ and the σ_n on a surface at an angle θ with respect to the principal plane are required, the following equations can be used to calculate these stresses:

$$\sigma_n = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta , \tag{2}$$

and

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta . \tag{3}$$

The values of σ_n and τ and then plotted, and values of ϕ_j and cohesion (c) are determined.

Results of the multistage test are presented in Figure V.12. Figure V.12 shows the stress circles for the seven loading stages used during the test. The τ and σ_n were plotted on the stress circles and connected to form the strength envelope for the sawed surface. The least squares method was used to compute the equation of the strength envelope as given below.

$$\tau = 0.0 + 0.6322\ \sigma_n$$

The equation yields a coefficient of friction of $32^o30'$ and a cohesion of 0.0 psi. It is interesting to note that the coefficient of friction calculated from stresses on the sawed surface is nearly equal to the coefficient of friction obtained for the intact samples at a normal stress level above 1000 psi. Similar results were reported[7] for SSD intact rock and room-dry sawed and open natural joints.

Presented in Figure V.13 are the failure envelopes representing the shale tested in triaxial and direct shear and the concrete on rock tested using the multistage triaxial test.

Petrographic Examination Results

Composition of Three Hand Samples

They were composed of chlorite, clay-mica, and kaolinite as the clay minerals and quartz, plagioclase feldspar, and a little siderite (iron carbonate) as the nonclay minerals. While this rock is fine-grained, it is considerably coarser-grained than the black shale.

Recommended Values for Structural Analysis

The structural analysis should consider rock type and bedrock structural features and backfill described herein.

Recommended Parameters

	Shale	Concrete Original	Concrete Newer	Backfill
Index Properties				
Drained Unit Weight, lb/ft^3				135
Wet Unit Weight, lb/ft^3	168.7	150.0	150.0	148
Moisture Content, pct	2.14	--	--	--
Compressive Strength, psi	3960	3640	4300	--
Tensile Strength, psi	700	--	--	--
Shear Strength, psi				
Intact	C = 8.06 tsf $\phi = 68^{o}9'$	--	--	--
Concrete on Rock	C = 0.0 $\phi = 32^{o}30'$	--	--	--
Modulus of Elasticity x 10^6 psi	0.58	3.6	5.11	--
Poisson's Ratio	0.18	0.17	0.20	--
Shear Modulus x 10^6 psi	0.24	1.55	--	--

Table V.1

Compression Test Results of Foundation Rock

Hole No.	Depth, ft	Wet Unit Weight, lb/ft^3	Min Prin Stress σ_3 psi	Max Prin Stress σ_1 psi	Prin Stress Difference $\sigma_1 - \sigma_3$ psi	Modulus of Elasticity, E, x 10^6 psi	Poisson's Ratio v	Shear Modulus from TX Test G, x 10^6 psi	Moisture Content
M-7	30.3-31.2	168.4							
	31.8-33.0	168.1							
	33.0-34.2	170.4							
	43.5	164.0							
M-8	34.2-35.1		0	--*		0.57	--		
	35.2-36.3	168.6							
	36.3-37.1		-770	0	-770				1.78
	37.1-38.0	168.9	1000	6930	5930	0.57	0.26	0.21	
	38.0-40.5	171.2	100	5750	5650	0.63	0.16	0.25	2.40
	40.5-41.5	169.5	500	5160	4660	0.58	0.1	0.25	2.64
	43.2-44.6	168.0	0	4430	4430				
	44.8-45.8		-620	0	-620				1.86
	47.8-48.7	170.0							
M-7	36.2-37.2	--	0	4240	4240	0.57	0.2	--	2.11
	42.5-43.5	--	0	3220	3220	0.57	0.2	--	2.04
Avg		168.7		3960		0.58	0.18	0.24	2.14

* Test check specimen not taken to maximum stress.

SPECIMEN No	ELEVATION Ft.	MODULUS ELASTICITY x 10⁶ psi	POISSON'S RATIO	σ_3 ksi	σ_1 ksi	$\sigma_1 - \sigma_3$ ksi
M-2	715.2	3.33	0.13	0.0	3.07	3.07
M-1	728.2	4.00	0.20	0.5	6.89	6.39
M-4	702.9	--	--	1.0	8.00	7.00

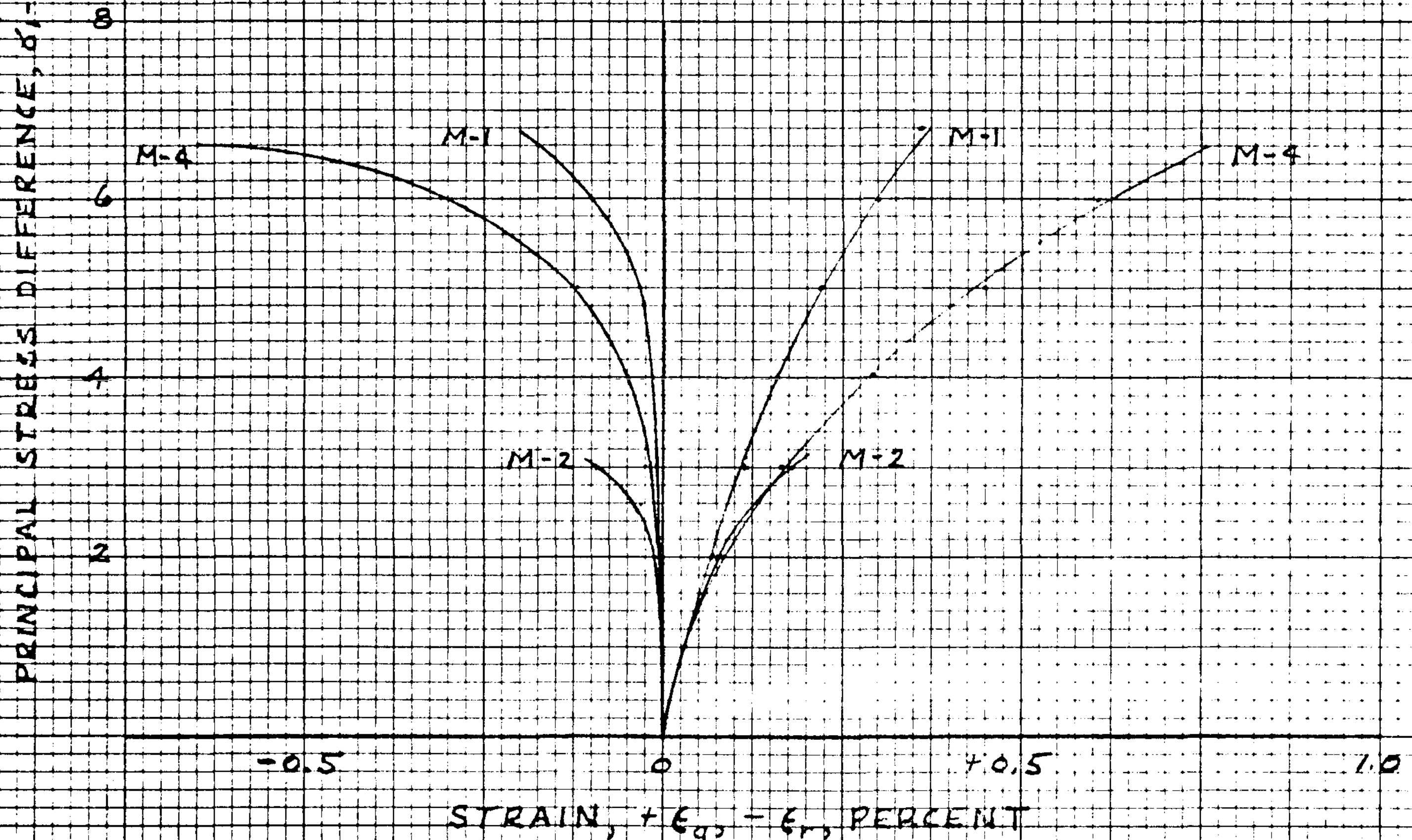


Figure V.1 TRIAXIAL TEST RESULTS OF CONCRETE CORES, $(\sigma_1 - \sigma_3)$ vs $(\epsilon_a \& \epsilon_r)$, LOCK & DAM No. 3

SPECIMEN No	ELEVATION ft	SHEAR MODULUS $\times 10^6$ psi	σ_3 ksi	σ_1 ksi	$\sigma_1 - \sigma_3$ ksi
M-1	728.2	1.18	0.5	6.89	6.39
M-4	702.9	1.18	1.0	8.00	7.00

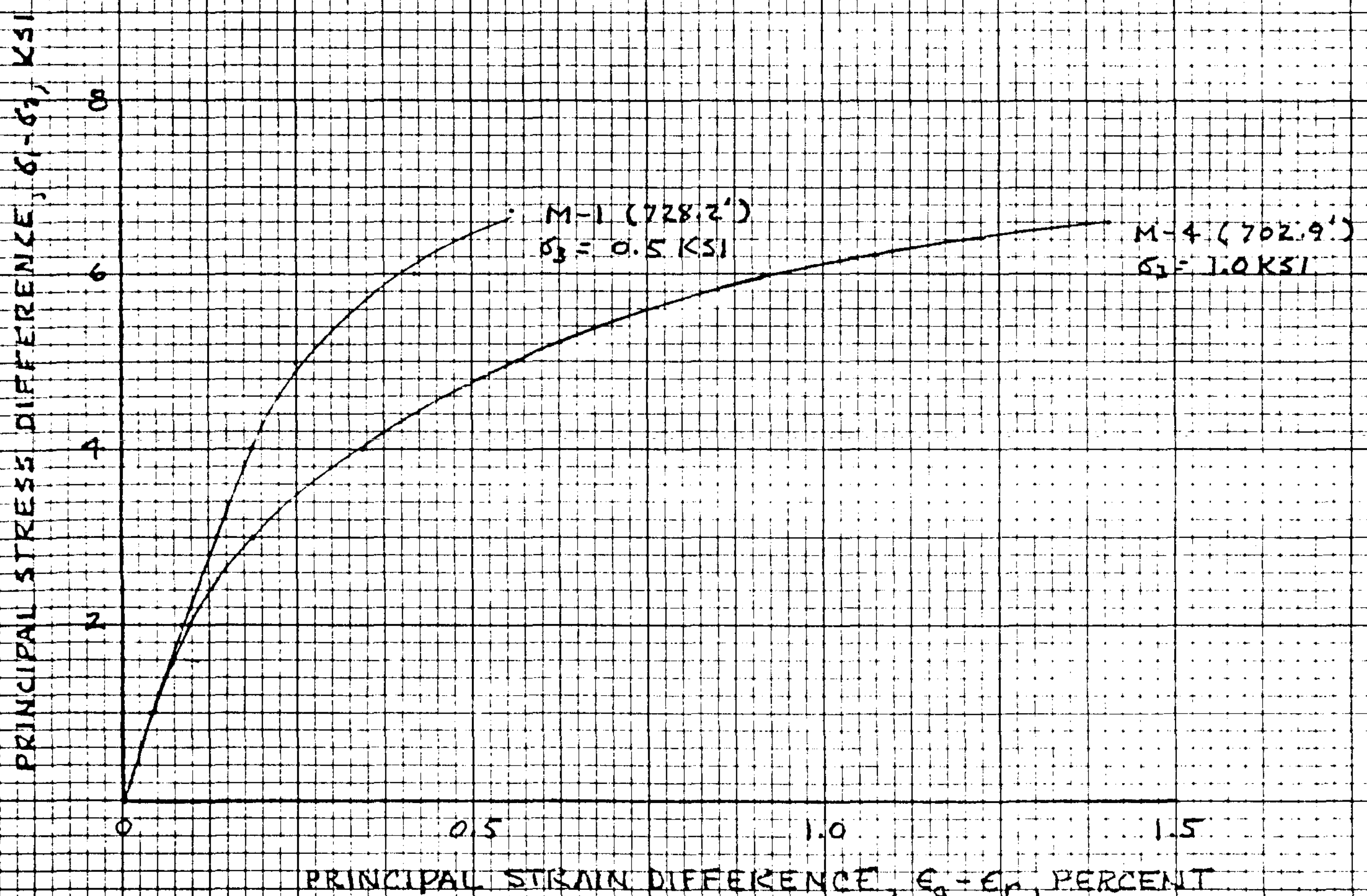


Figure V.2 TRIAXIAL TEST RESULTS OF CONCRETE CORES, $(\sigma_1-\sigma_3)$ vs $(\epsilon_a-\epsilon_r)$, LOCK & DAM No 3

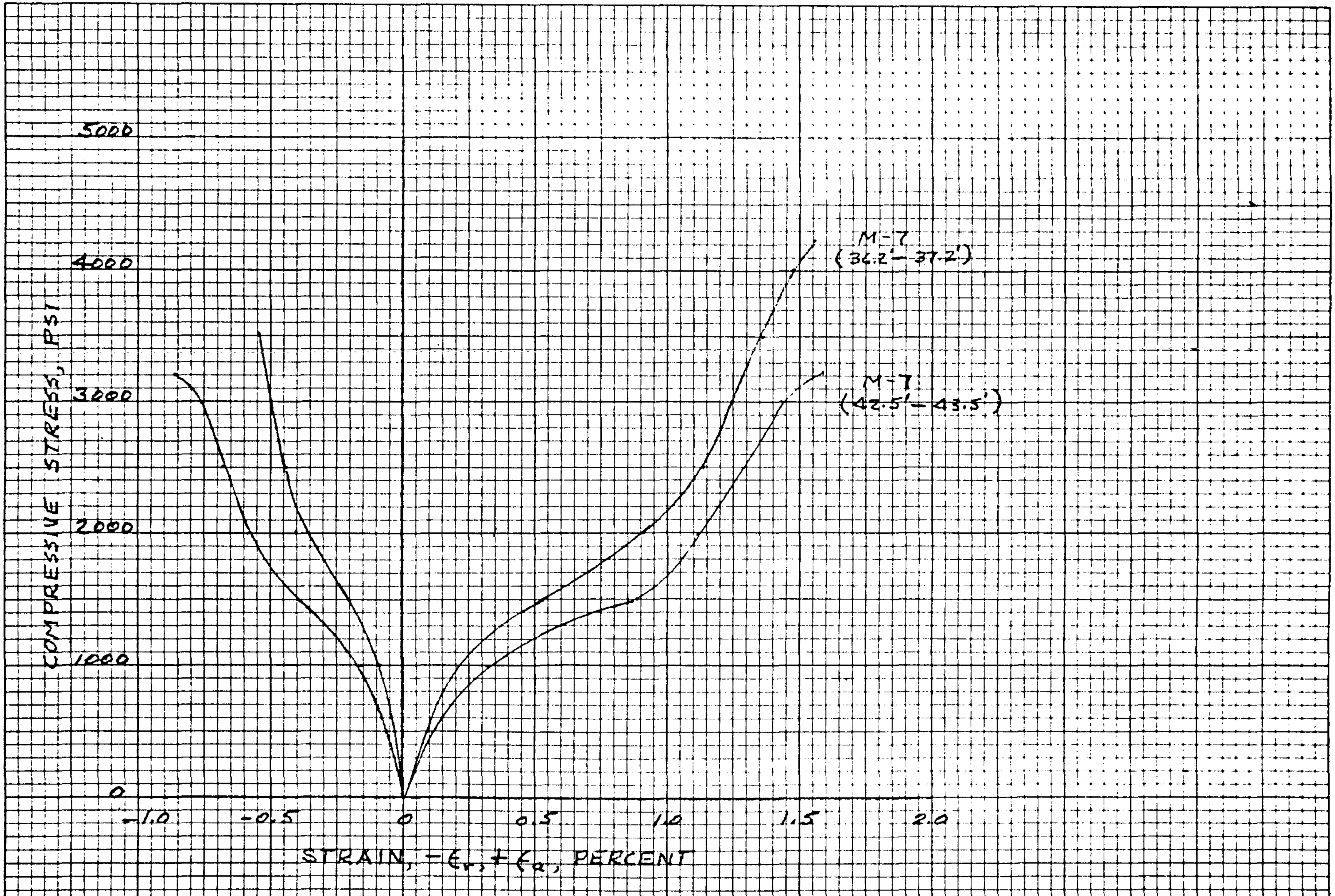


Figure V.3 STRESS-STRAIN RELATION FROM UNCONFINED TEST ON SHALE

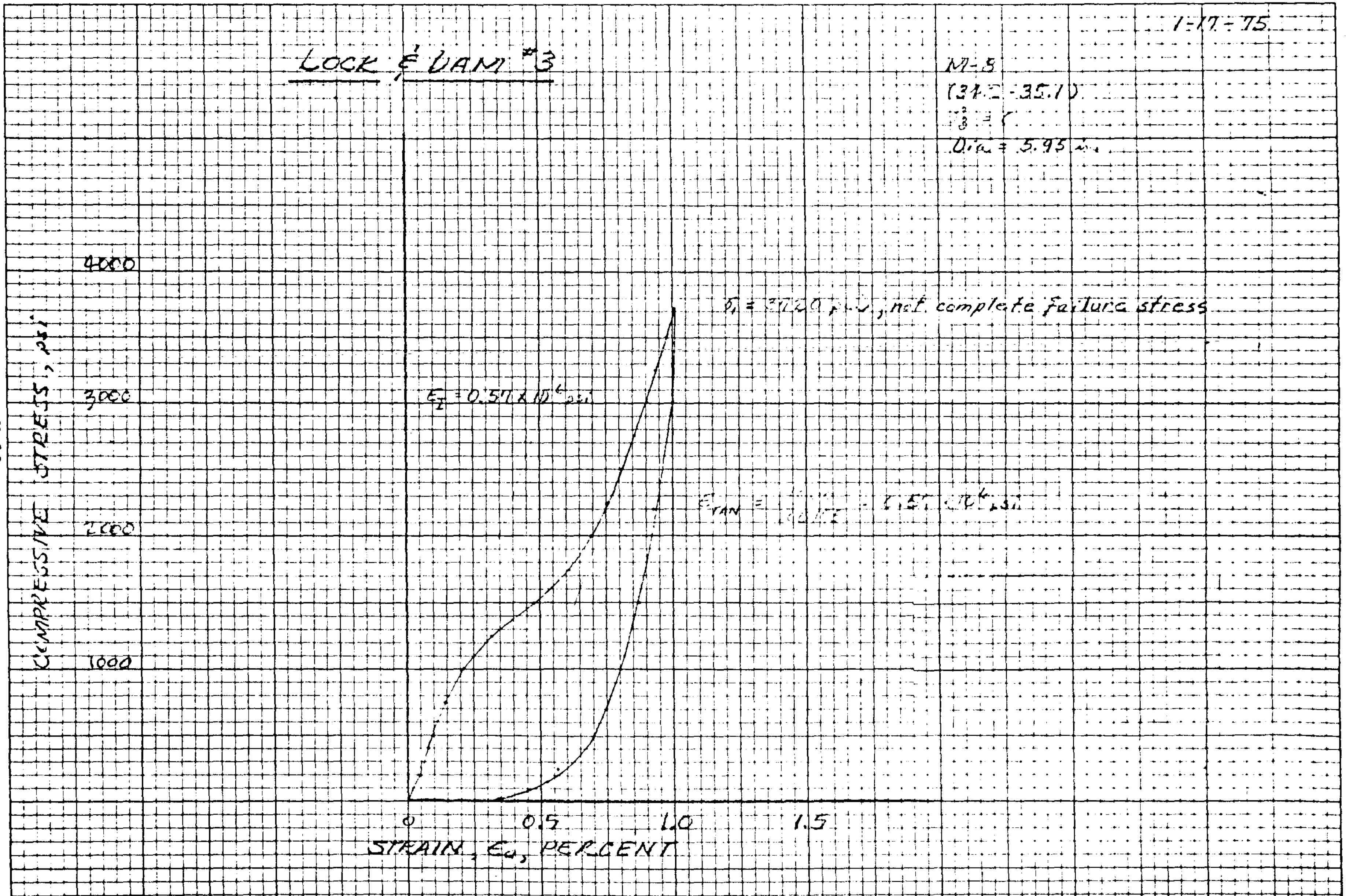


Figure V.4 TYPICAL COMPRESSIVE STRENGTH TEST RESULT, SHALE CORE

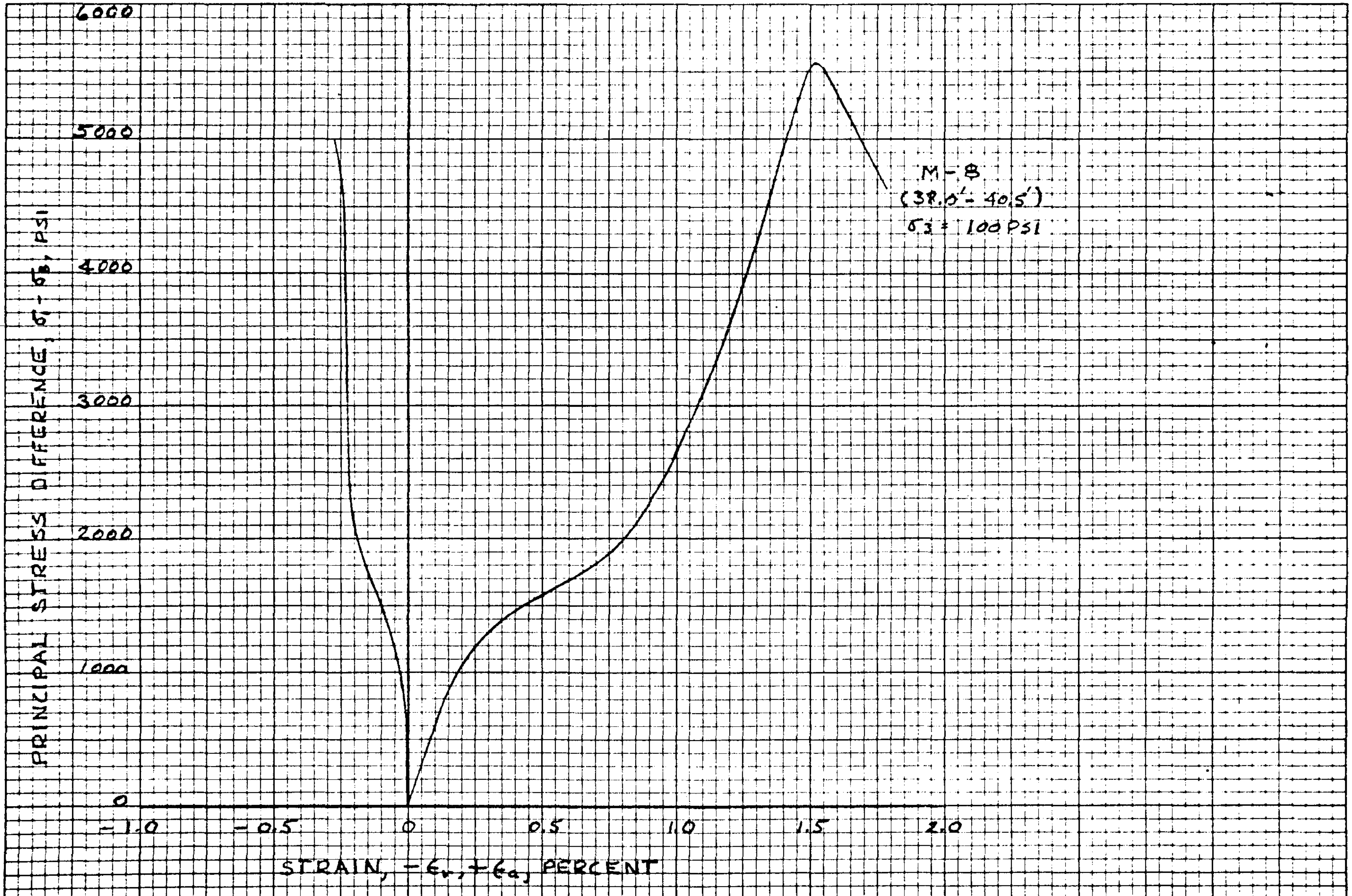


Figure V.5 TRIAXIAL TEST RESULTS OF SHALE, σ_3 = 100 psi, HOLE M-8

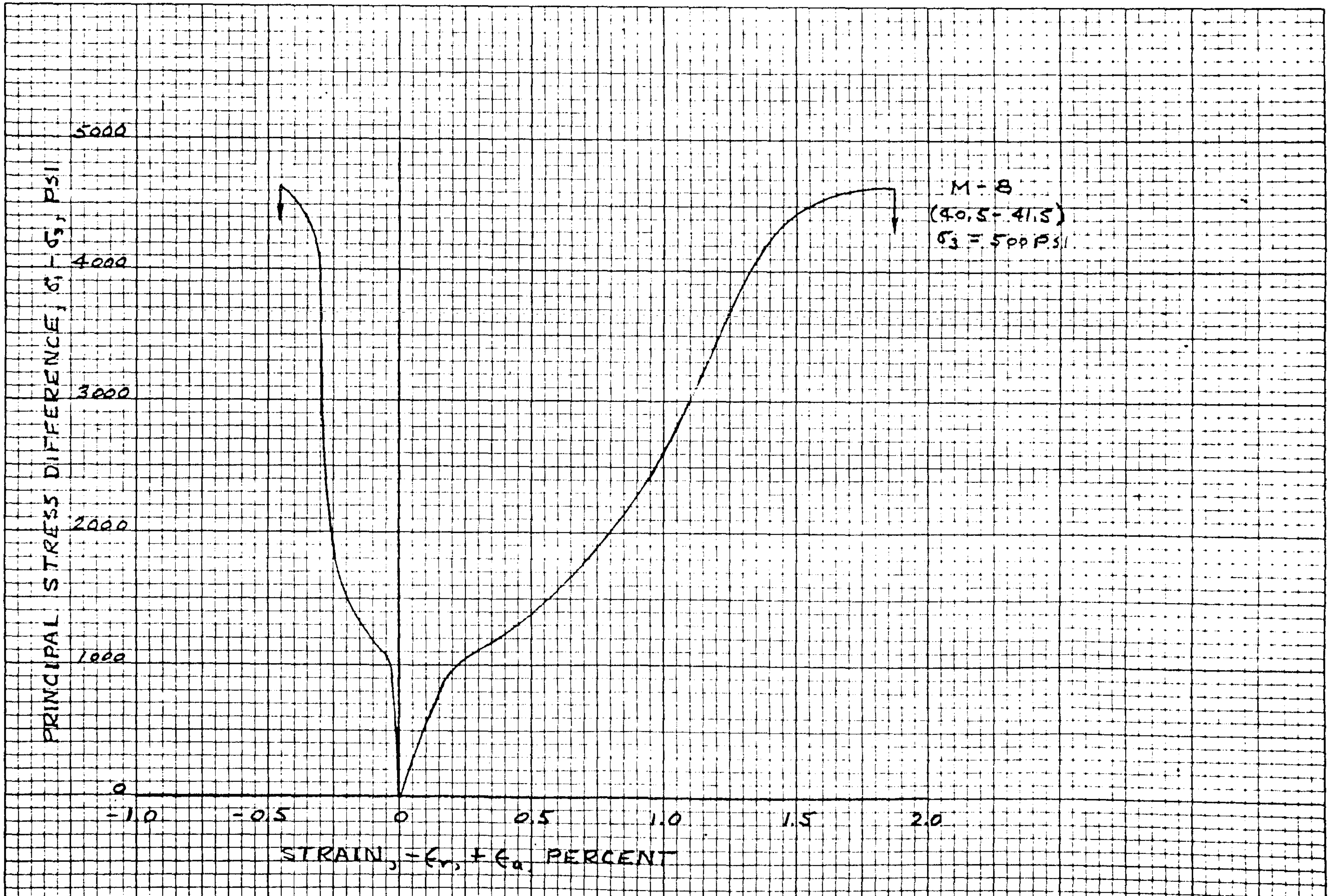
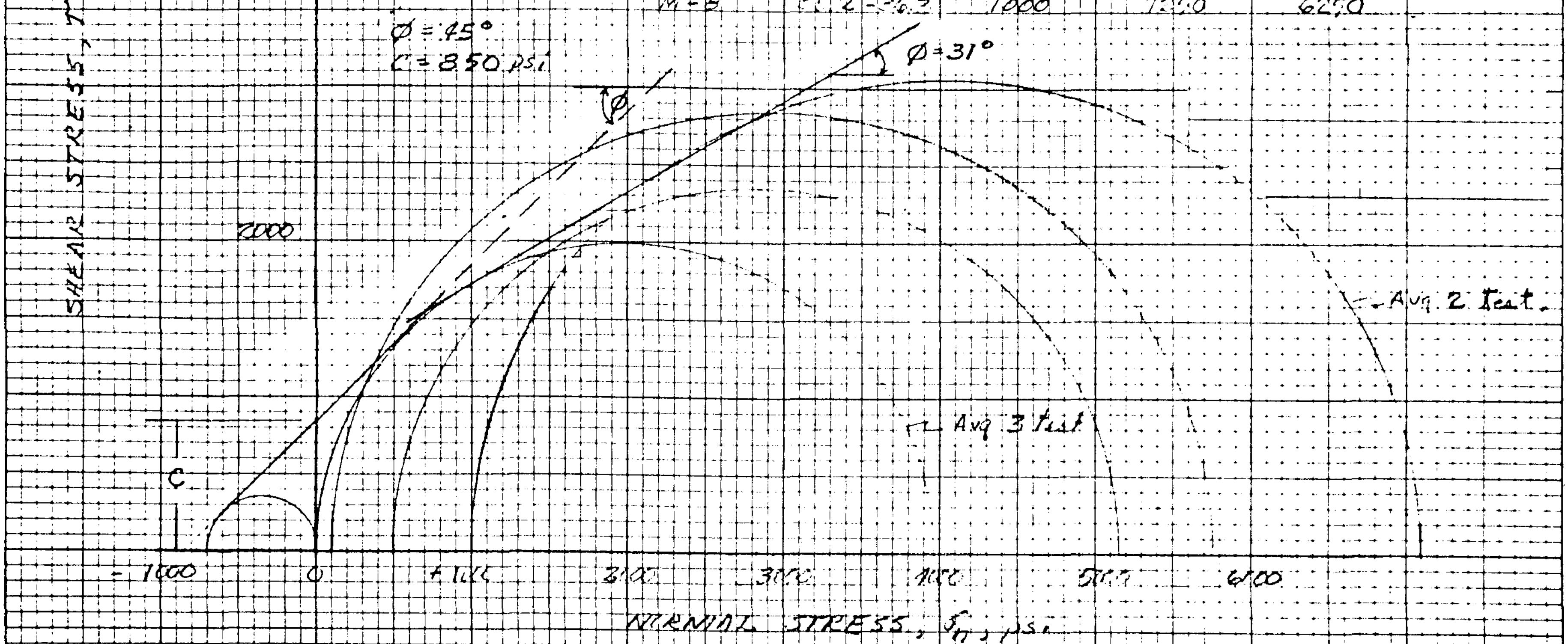


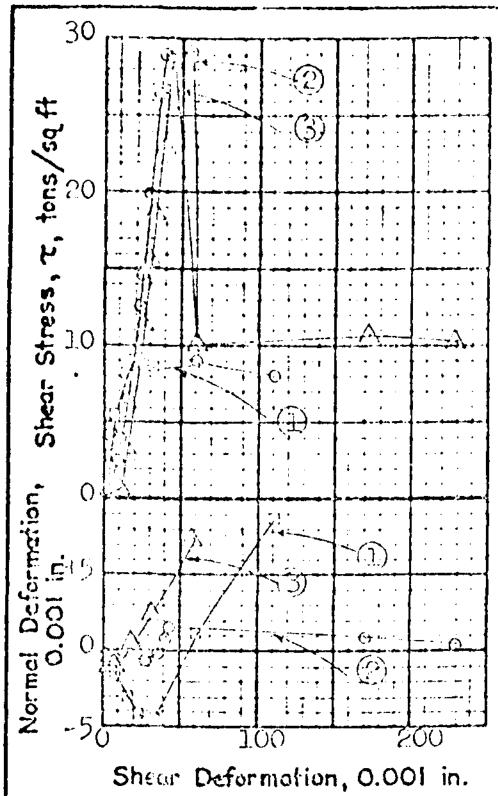
Figure V.6 TRIAXIAL TEST RESULTS OF SHALE, σ_3 = 500 psi HOLE M-8

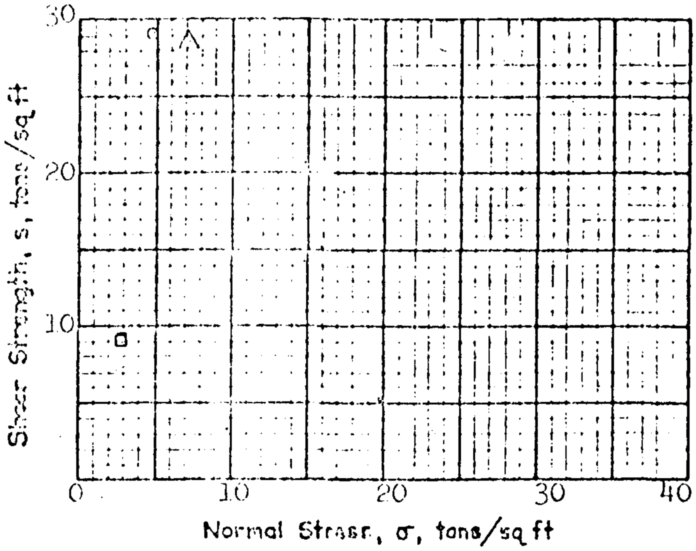
Figure V.7 TRIAXIAL TEST RESULTS OF SHALE, σ_3=1000 psi, HOLE M-8



DRILL HOLE	DEPTH ft	MINOR PRIN. STRESS, σ_3 psi	MAJ. PRIN. STRESS, σ_1 psi	PRIN. STRESS DIFFERENCE $\sigma_1 - \sigma_3$, psi
M-8	37.1-38.0	1000	6930	5930
M-8	39.0-40.5	100	5750	5650
M-8	40.5-41.5	500	5160	4660
M-8	36.3-37.1	0	- 770	- 770
M-8	44.8-45.8	0	- 620	- 620
M-8	43.2-44.6	0	4430	4430
M-8	[illegible]	1000	7250	6250

Figure V.8 MOHR DIAGRAM FOR SHALE FROM HOLE M-8, LOCK & DAM 3





SHEAR STRENGTH PARAMETERS

	Maximum	Residual	
ϕ =			
$\tan \phi$ =			
c =			tons/sq ft

Test No.		1	2	3			
Wet density, lb/cu ft	γ_d	163.8	168.1	168.4			
Water content	w	1.7 %	1.7 %	1.4 %	%	%	%
Normal stress, tons/sq ft	σ	2.37	4.75	7.20			
Maximum shear stress, tons/sq ft	τ_f	8.95	29.27	28.61			
Time to failure, minutes	t_f	10	45	23			
Residual shear stress, tons/sq ft	τ_r						
Initial Diameter, Inches	Do	5.84	5.88	5.66			
Initial Height, Inches	Ho	5.34	3.82	5.10			

Description of material *Medium-gray hard shale*

Remarks

Specimen No	Elevation, ft
M-7-1	705.?-705.4
M-7-2	705.1-704.8
M-7-3	706.1-705.7

Project *Lock & Dam 3*

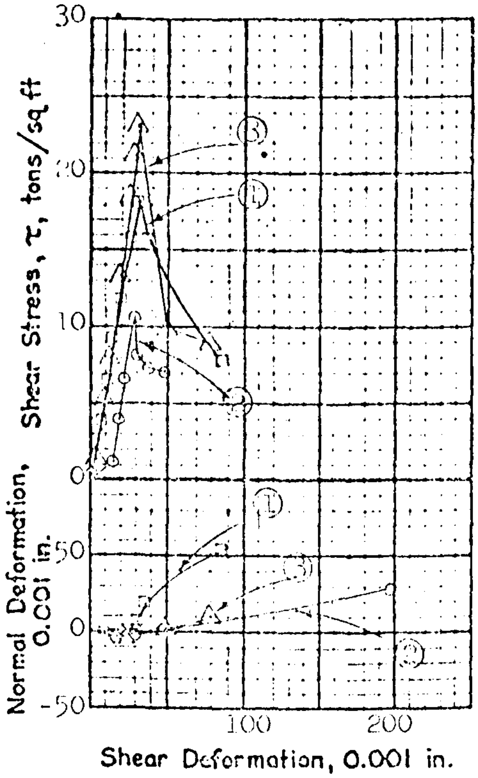
Area

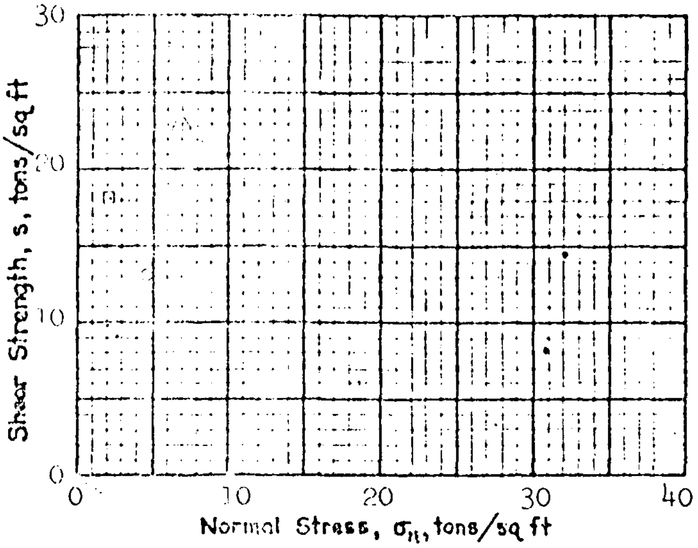
Boring No. *M-7* | Sample No.

Depth *30.8 to 32.1 ft* El *705.7 to 705.7* | Date

DIRECT SHEAR TEST REPORT (ROCK)

Figure V.4 DIRECT SHEAR TEST RESULTS, SPECIMENS 1, 2 & 3





SHEAR STRENGTH PARAMETERS

	Maximum	Residual	
ϕ =			
tan ϕ =			
c =			tons/sq ft

Test No.		4	5	6			
Wet density, lb/cu ft	γ_d	169.6	167.6	171.2			
Water content	w	1.7 %	1.8 %	1.5 %	%	%	%
Normal stress, tons/sq ft	σ_n	2.37	4.75	7.20			
Maximum shear stress, tons/sq ft	τ_f	18.13	13.21	23.12			
Time to failure, minutes	t_f	31	8	22			
Residual shear stress, tons/sq ft	τ_r						
Initial Diameter, Inches	Do	5.92	5.89	5.88			
Initial Height, Inches	Ho	4.00	4.94	3.96			

Description of material *Medium-gray hard shale*

Remarks

Specimen No	Elevation, ft
M-7-4	698.4 - 698.1
M-7-5	697.6 - 697.3
M-7-6	697.3 - 697.0

Project *Lock & Dam 3*

Area

Boring No. *M-7*

Sample No.

Depth *38.5 to 39.4 ft*
El *698.4 to 697.0*

Date 19 March 1975

DIRECT SHEAR TEST REPORT (ROCK)

WES FORM NO.

Figure V.10 DIRECT SHEAR TEST RESULTS, SPECIMENS 4, 5 & 6

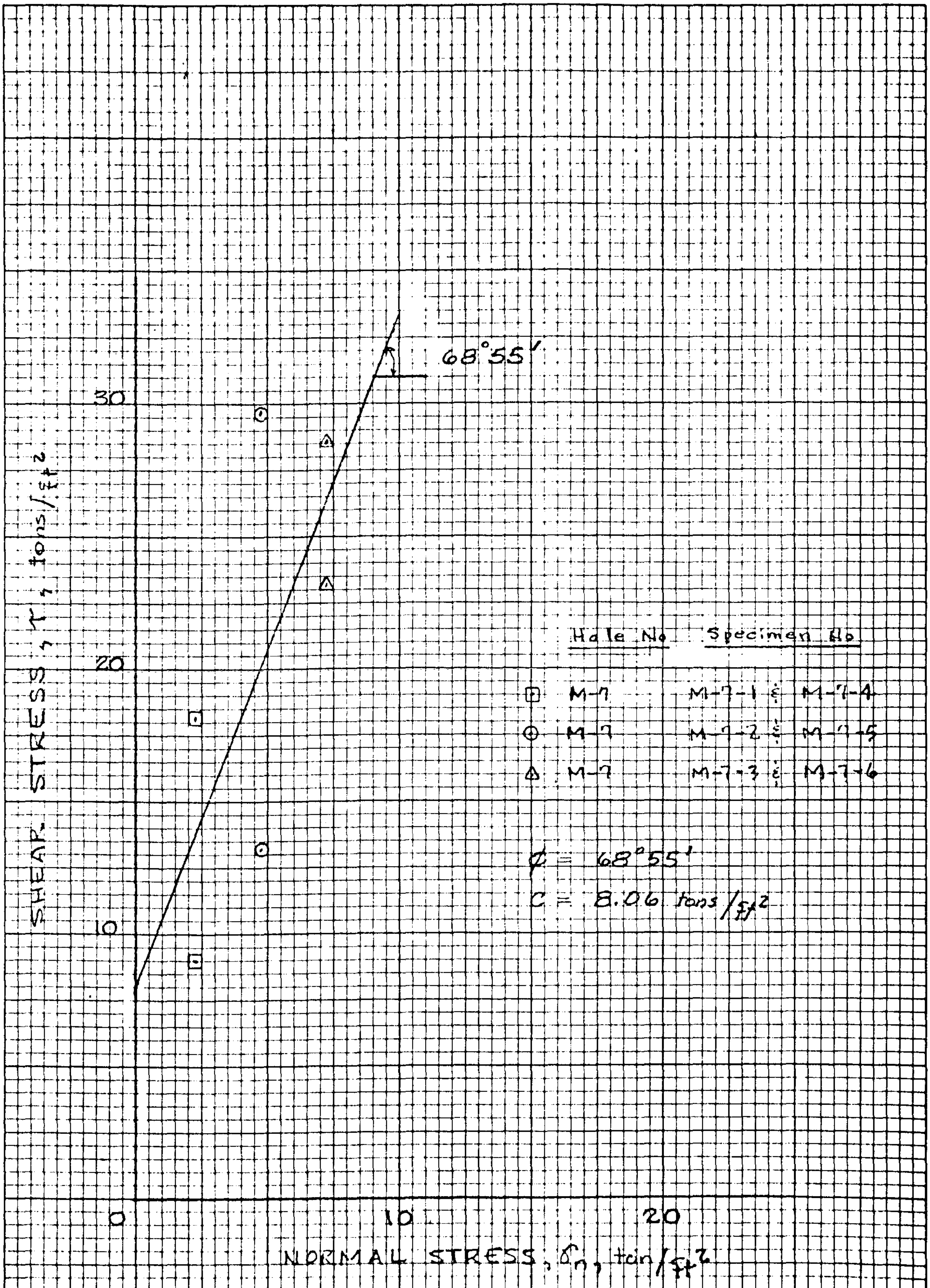


Figure V.11 FAILURE ENVELOPE FOR INTACT SHALE TESTED IN DIRECT SHEAR

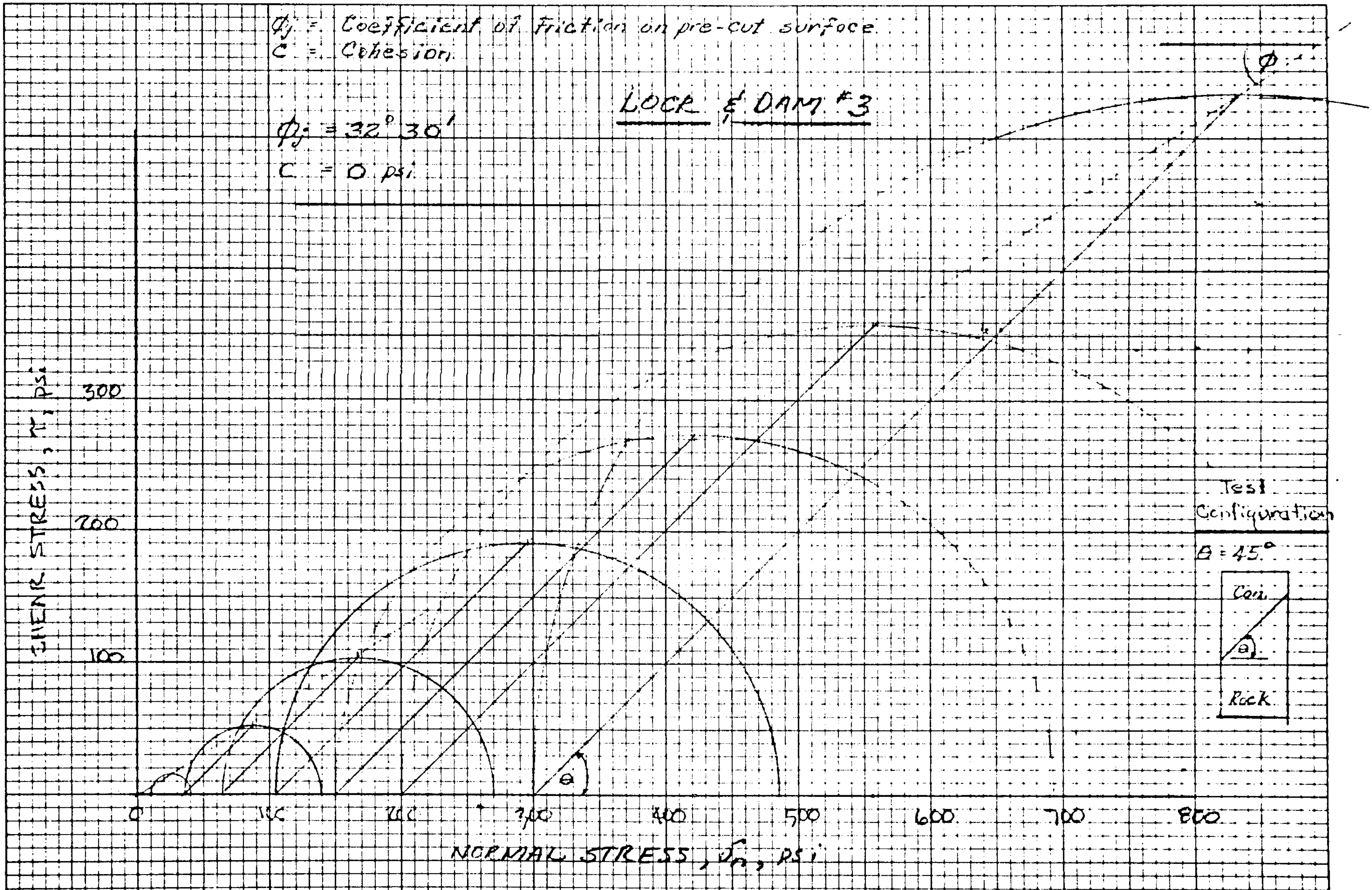


Figure V.12 MOHR'S DIAGRAM, CONCRETE ON SHALE CORE

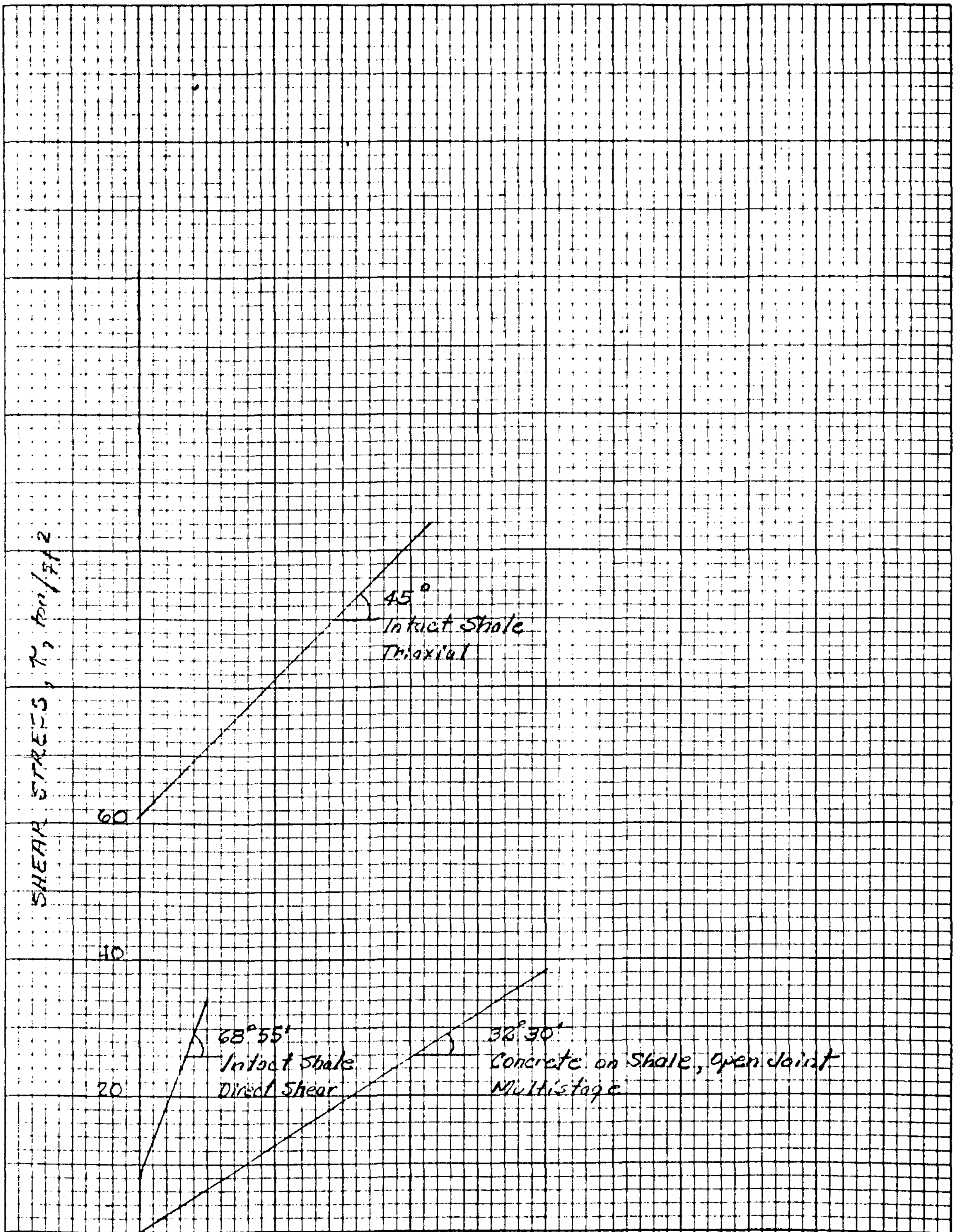


Figure V.13 FAILURE ENVELOPES TRAXIAL, MULTISTAGE & DIRECT SHEAR

APPENDIX VI

STABILITY ANALYSIS

APPENDIX VI

STABILITY ANALYSIS

Introduction

In the stability analysis the monoliths of the Locks and Dam were checked for adequacy against overturning, sliding, and excessive base pressures.

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.

From a field survey and examination of Locks and Dam 3 and from an examination of construction photographs, there were some apparent problems of structural instability. Computations were made to give indications of how safe the monoliths are in relation to overturning, sliding, and base pressures. The summary sheet and stability computations are given in Table VI.1 and Figures VI.1 through VI.21, respectively.

Applied Loads

The lock and dam monoliths were investigated for three case loadings unless it was apparent that certain cases were not critical. The three case loadings are:

a. Normal operating condition:

(1) Upper guide, land wall, and lower guide wall monoliths: The most critical loadings of upper pool, lower pool, and saturation level in backfill were taken from available piezometer data. Also, dead load, uplift, tow impact, hawser pull, wind, and gate loads were used when applicable. Except where specifically indicated, tow impact is included in all calculations for normal operations.

(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 taken to be the same as for the normal operating condition as described above. Impact, hawser pull, and wind loads were applied according to the situation.

c. Flood condition: Some monoliths were checked for stability during maximum flood conditions until it seemed apparent that this was not the most critical loading condition.

The standard procedure was to analyze two-dimensional sections of unit depth except for a three-dimensional analysis of monolith L-23. 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.

Approximations were necessary concerning several significant factors which affect the stability analysis; these approximations will be discussed below.

The soil behind the land wall monoliths was higher or sloped higher than the monolith itself which creates a surcharge loading affecting horizontal and in some cases the vertical pressure acting on the monolith. Both the 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.

In this case, with 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. At the outset of the investigation, at-rest coefficients between the values of 0.5 and 0.8 were considered. Monolith L-55 was chosen to apply a range of at-rest coefficients using 0.5 as minimum and 0.8 as a maximum. It was found that 0.5 produced a sliding safety factor that was unacceptable and, therefore, concluded that a factor of 0.8 would produce a more unacceptable value.

The random backfill was mainly noncohesive which made obtaining an intact sample virtually impossible. Tests on a remolded sample for "at-rest" pressure coefficients would not be reliable. This is partially true because of large pieces of rock in the fill, the effect of which would be hard to determine in any reasonable size test sample. 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/or funding was not such that this type of testing was possible. On this basis, it was decided to estimate a lower bound value as follows. 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. The backfill at Locks and Dam 3 contains a substantial amount of clay. It is reasonable, therefore, to use a lower bound at-rest earth coefficient of 0.5. Also, considering that the backfill slopes upwards which tends to increase any horizontal pressure coefficient, it would be unreasonable to use a lower bound value less than 0.5.

From EM 1110-2-2502, it was concluded 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, but will be somewhat higher. The resultant location used was as suggested in the above Engineer Manual considering walls supported on rock foundations:

a. 0.38H above the base for horizontal or downward sloping backfill.

b. 0.45H above the base for upward sloping backfill.

As stated in the case loadings, the saturation level in the backfill was used as the most critical from available piezometer data. The worst combination from available data (over approximately a year period) of pool and saturation levels were taken because it is a fact that the monoliths had to resist this particular combination of loads; whereas, we do not know what saturation level corresponds to normal upper and lower pool elevations. This loading may not be the most critical because it was obtained from measurements taken over approximately a year and, during this short span of time, it could not be the worst loading to which the monoliths will ever be subjected. In some cases the normal design criteria for saturation levels give results which are a lot more critical than those given by the piezometer data. There was really no way of knowing whether the reality was closer to the available data or to normal design criteria; available data were used giving less severe conditions in many cases. The saturation levels used are not on the safe side because the locks are overtopped which saturates the backfill and in many cases the pool levels drop fast leaving high saturation levels in the backfill. This results in maximum horizontal thrust toward the river even more than the normal design criteria for saturation levels. When piezometer readings were not located close to the monolith being considered, the saturation level was interpolated between piezometer locations.

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.

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.

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 for the two-dimensional stability but is not as accurate when considering stress because of localized stresses which may become critical in certain cases.

Ice loads would make some case loadings more critical.

Design Criteria

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

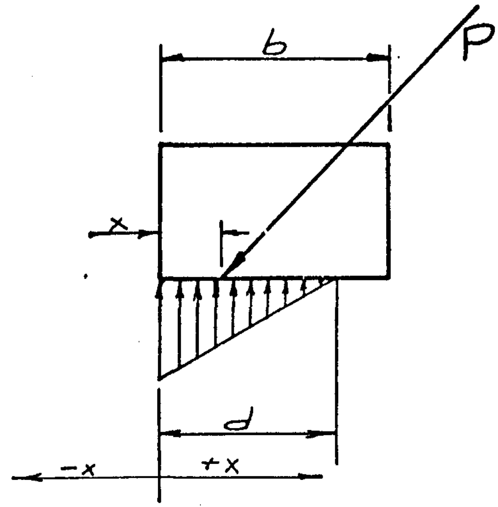
Resistance to overturning was considered adequate if the resultant fell outside the kern but within the middle half of the base for normal operation cases using "at-rest" earth pressure coefficients. The resultant for the extreme maintenance and for maximum flood conditions using "at rest" earth pressures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

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

The unit weight of concrete, drained backfill material, and submerged backfill material was used as 150 lb/cu ft, 135 lb/cu ft, and 148 lb/cu ft, respectively. Actually, the unit weight of the concrete is a little lower than 150 lb/cu ft which makes the stability computations not quite as critical as would have been if we had used its actual unit weight.

There is no problem in engineering concepts if the total base pressure is compressive because for massive-rigid structures it can be obtained rather accurately by $f = \frac{P}{A} \pm \frac{Mc}{I}$ considering the total 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--that which is in compression. This will be done and the effective area is derived below.

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



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

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

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

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

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

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

If the resultant falls outside the base, the monolith should begin overturning. By conventional design, the resultant falls outside the base for many monoliths at Locks and Dam 3. This is in reality not the case because the monoliths are in relatively good alignment except for the upper guard wall and the abutment which are not the main considerations in point. In more than 70 years, most monoliths have not showed 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 than the actual situation.

c. A refinement in parameters and calculation methods is needed to more accurately obtain a horizontal soil force against the monoliths.

There is 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 VI.1. The base pressures which can be calculated by normal conventional design, compare relatively well with those calculated near the base in the finite element analysis. The above is supplemental information for stability considerations and makes no analysis or conclusion concerning the monoliths at Locks and Dam 3. The analysis and conclusions are given in Section VI.

TABLE VI.1

Summary of Stability Analysis Results

Monolith	Cases Considered	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
L-18	Normal Operation	2.50	0.87	4	1.26	20	19.31
	Normal Operating Neglecting Impact	2.50	-2.82	4	0.92	20	∞
	Flood Condition	1.67	-1.09	4	1.09	20	∞
L-23	Normal Operation	≈10.00	19.93	4	3.41	20	28.00
	Maintenance Condition	≈10.00	18.02	2-2/3	2.04	20	31.00
L-32	Normal Operation	8.25	9.30	4	2.23	20	5.99
	Normal Operation Neglecting Impact	8.25	9.03	4	2.13	20	6.17
	Maintenance Condition	5.50	7.55	2-2/3	1.78	20	7.64
L-32 w/construction joints	Normal Operation	3.75	0.28	4	1.43	20	134.05
	Normal Operation Neglecting Impact	3.75	0.96	4	1.50	20	39.10
	Maintenance	2.50	0.97	2-2/3	1.27	20	49.48
L-37	Normal Operation	7.50	10.69	4	2.72	20	4.83
	Maintenance Condition	5.00	8.82	2-2/3	1.74	20	7.00
L-42	Normal Operation	0.56	0.11	4	5.74	20	164.00
	Normal Operation Neglecting Impact	0.56	5.44	4	4.10	20	510.00
	Maintenance Condition	0.38	-1.64	2-2/3	1.21	20	∞
L-46	Normal Operation	1.81	-1.22	4	2.73	20	∞
	Normal Operation Neglecting Impact	1.81	0.14	4	3.44	20	176.00
	Maintenance Condition	1.21	-2.19	2-2/3	1.20	20	∞

(Continued)

TABLE VI.1 (Continued)

Monolith	Cases Considered	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
L-55	Normal Operation K_r = 0.5	3.25	-2.25	4	2.05	20	∞
	Normal Operation Neglecting Impact	3.25	-3.39	4	1.89	20	∞
	Normal Operation K = 0.8	3.25	-7.61	4	1.30	20	∞
	Normal Operation Neglecting Impact	3.25	-8.76	4	1.24	20	∞
	Normal Operation K_A = 0.33	3.25	0.78	4	3.04	20	37.26
	Normal Operation Neglecting Impact	3.25	-0.37	4	2.70	20	∞
	Flood Condition K_r = 0.5	2.17	-2.09	4	2.62	20	∞
	Flood Condition K_r = 0.8	2.17	-7.29	4	1.63	20	∞
	Flood Condition K_A = 0.33	2.17	0.83	4	3.96	20	26.10
L-55	Normal Operation	0.50	-5.22	4	1.29	20	∞
	Normal Operation Neglecting Impact	0.50	-0.92	4	1.97	20	∞
	Maintenance Condition	0.33	-8.78	2-2/3	0.92	20	∞
M-2	Normal Operation	6.67	10.12	4	64.13	20	3.13
	Normal Operation Neglecting Impact	6.67	10.51	4	192.40	20	3.48
	Maintenance Condition	5.00	6.93	2-2/3	2.17	20	6.70
	Maintenance Condition Neglecting Impact	5.00	7.28	2-2/3	2.30	20	6.34
M-6	Normal Operation	6.67	6.94	4	13.90	20	5.39
	Maintenance Condition	5.00	13.95	2-2/3	7.81	20	6.86
M-15	Normal Operation	6.67	14.42	4	2.25	20	7.58
	Normal Operation Neglecting Impact	6.67	14.79	4	2.13	20	7.94
	Maintenance Condition	5.00	7.62	2-2/3	2.17	20	6.05

(Continued)

TABLE VI.1 (Continued)

Monolith	Cases Considered	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft: Minimum Allowable	Actual	Sliding Safety Factor: Minimum Allowable	Actual	Foundation Pressure, k/sf: Allowable	Actual
M-20	Normal Operation	5.33	5.29	4	5.1	20	5.65
	Flood Condition	4.00	8.13	4	∞	20	2.42
	Maintenance Condition	4.00	9.15	2-2/3	6.56	20	4.94
M-25	Normal Operation	6.00	6.95	4	9.03	20	5.10
	Normal Operation Neglecting Impact	6.00	6.59	4	8.30	20	5.46
	~~Maintenance Condition~~	~~4.50~~	~~10.07~~	~~2-2/3~~	~~9.00~~	~~20~~	~~3.48~~
	Maintenance Condition Neglecting Impact	4.50	9.74	2-2/3	9.83	20	4.24
R-2	Normal Operation Section AA	3.33	3.80	4	2.94	20	24.18
	Normal Operation Neglecting Impact	3.33	4.89	4	27.40	20	14.99
	Normal Operation Section BB	3.33	2.69	4	3.74	20	939.93
	Normal Operation Neglecting Impact	3.33	4.77	4	34.93	20	170.01
	Normal Operation Section CC	4.83	4.77	4	6.11	20	23.68
	Normal Operation Neglecting Impact	4.83	7.05	4	57.00	20	17.63
	Flood Condition Section AA	2.50	5.00	4	∞	20	8.62
	Flood Condition Section BB	2.50	5.00	4	∞	20	61.13
	Flood Condition Section CC	3.62	7.36	4	∞	20	13.86
R-12*	Normal Operation	6.67	9.50	4	33.18	20	4.27
	Normal Operation Neglecting Impact	6.67	9.83	4	99.55	20	3.90
	Maintenance Condition	5.00	15.14	2-2/3	0.70	20	12.22

* Borehole M-1 shows coal at the monolith base. Sliding factors of safety were calculated using lower bound values of $\phi = 15^{o}$ and $C = 0^{o}$.

(Continued)

TABLE VI.1 (Continued)

Monolith	Cases Considered	Distance From River Toe of Monolith to Where Resultant Intersects Monolith Base, ft		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
R-15	Normal Operation	6.00	14.79	4	2.46	20	11.61
	Normal Operation Neglecting Impact	6.00	15.55	4	2.24	20	15.21
	Maintenance Condition	4.50	17.06	2-2/3	1.25	20	47.52
R-21	Normal Operation	6.00	6.69	4	2.19	20	5.17
	Normal Operation Neglecting Impact	6.00	7.62	4	2.39	20	4.27
	Maintenance Condition	4.50	14.10	2-2/3	2.23	20	11.68
R-23	Normal Operation	6.67	4.63	4	2.39	20	7.99
	Normal Operation Neglecting Impact	6.67	5.17	4	2.53	20	7.16
	Maintenance Condition	5.00	12.46	2-2/3	1.96	20	5.08

SUBJECT: Upper Guide Wall - L18

COMPUTED BY: DATE:

CHECKED BY: DATE:

Station: 1+50.3A to 1+80.5A

Length = 30.2 ft

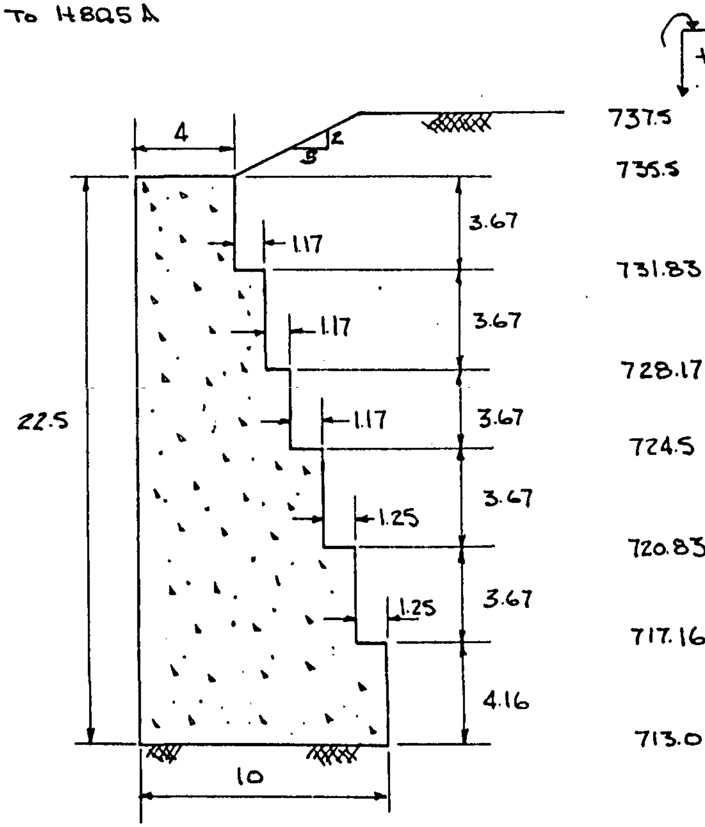


FIGURE VI-1 LAND WALL - UPPER GUIDE WALL L-18

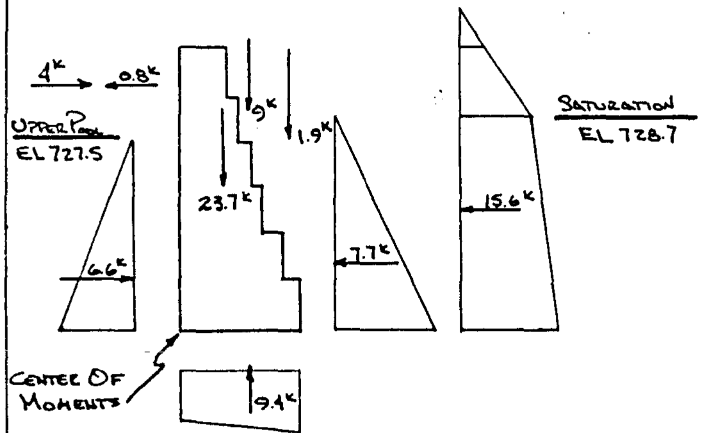
Scale 1:50

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: UPPER GUIDE WALL - L18

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

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[0.150][(4)(22.5)+(1.17)(18.83)+(1.17)(15.16)+(1.17)(11.49)+(1.25)(7.83)+(1.25)(4.16)][1]	23.7		3.82	91
P_{WATER}	$(727.5-713)^2(1)(.0624)(1/2)$		6.6	4.83	32
P_{WATER}	$(728.7-713)^2(1)(.0624)(1/2)$		-7.7	5.23	-40
HAWSER	(24)(1/30.2)		-0.8	19.5	-16
UPLIFT	(14.5)(.0624)(1)(10)+(1/2)(1.2)(.0624)(10)	-9.4		5.07	-48
P_{EARTH}	$(736.5-728.7)^2(1/2)(.135)(1)(.5) - (1/2)(.135)(736.5-733.5)^2(1)(.5)+(736.5-728.7)(728.7-713)(.135)(1)(.5)+(1/2)(728.7-713)^2(.0856)(1)(.5)$		-15.6	10.58	-165
W_{WATER}	[(1.17)(.0624)(728.7-728.17)+(1.17)(.0624)(728.7-724.5)+(1.25)(.0624)(728.7-720.83)+(1.25)(.0624)(728.7-717.16)][1]	1.9		9.14	17
W_{EARTH}	(1.17)(.135)(735.7-731.83)(1)+(1.17)(.135)(736.2-728.7)(1)+(1.17)(.0856)(728.7-728.17)(1)+(1.17)(.135)(736.7-728.7)(1)+(1.17)(.0856)(728.7-724.5)(1)+(1.25)(.135)(737.2-728.7)(1)+(1.25)(728.7-720.83)(1)(.0856)+(1.25)(.135)(737.5-728.7)(1)+(1.25)(.0856)(728.7-717.16)(1)	9.		6.43	58
IMPACT	(3.97)(1)		3.97	19.5	77
	①	25.2	-12.7		22
	NEGLECTING IMPACT ②	25.2	-17.5		-71

CASE I: NORMAL OPERATIONS

ASSUMPTIONS:
UPPER POOL EL 727.5
SATURATION EL 728.7
$K_r = 0.5$

$e_1 = 22/25.2 = 0.87$ FT

$e_2 = -71/25.2 = -2.82$ FT

FIGURE VI-1 LAND WALL - UPPER GUIDE WALL L-18 (CONTINUED)

SUBJECT: Upper Guide Wall - L18

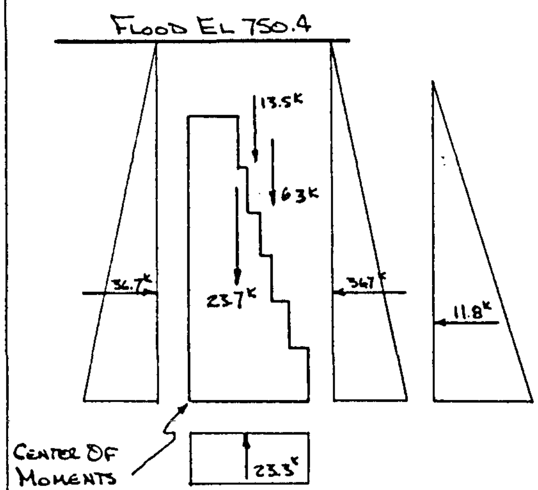
COMPUTED BY:

DATE:

CHECKED BY:

DATE:

Not To Scale



Item	Factors	F_V	F_H	Arm	Moment
W_{conc}		23.7		3.82	91
P_{water}	(750.4-735.5)(.0624)(735.5-713)(1) + (1/2)(735.5-713)^2(.0624)(1)		36.7	9.6	352
P_{water}			-36.7	9.6	-352
Uplift	(750.4-713)(.0624)(10)(1)	-23.3		5.	-117
W_{water}	(4)(.0624)(750.4-735.5)(1) + (1.17)(.0624)(750.4-731.83)(1) + (1.17)(.0624)(750.4-728.17)(1) + (1.17)(.0624)(750.4-724.5)(1) + (1.25)(.0624)(750.4-720.83)(1) + (1.25)(.0624)(750.4-717.16)(1)	13.5		5.9	80
W_{earth}	(735.7-731.83)(1)(1.17)(.0856) + (736.2-728.17)(1)(1.17)(.0856) + (736.7-724.5)(1.17)(.0856)(1) + (737.2-720.83)(1.25)(1)(.0856) + (737.5-717.16)(1.25)(.0856)(1)	6.3		7.8	49
P_{earth}	(736.5-713)^2(1/2)(1)(.0856)(.5)		-11.8	10.58	-125
		20.2	-11.8		-22

$e = -22/20.2 = -1.09$ FT

Case II: Flood Conditions

Assumptions:
Flood El 750.4
$K_r = 0.5$

FIGURE VI-1 LAND WALL - UPPER GUIDE WALL L-18 (CONTINUED)

SUBJECT: Upper Guide Wall - L18

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

Sliding

Case I: Normal Operations

$R = \Sigma V \tan\phi$

$R = (25.2)(.63707) = 16.05^k$

$S_{SF} = 16.05/-12.7 = -1.26$

Neglecting Impact $S_{SF} = 16.05/-17.5 = -0.92$

Case II: Flood Conditions

$R = (20.2)(.63707) = 12.87^k$

$S_{S\text{-}F} = 12.87/-11.8 = -1.09$

Foundation Pressure

Case I: Normal Operations

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

Including Impact $f = \left(\frac{2}{3}\right)\left(\frac{25.2}{.87}\right) = 19.31 \quad k/ft^2$

Fig. VI-1 Land Wall - Upper Guide Wall L-18 (Continued)

WES FORM NO. 1253A

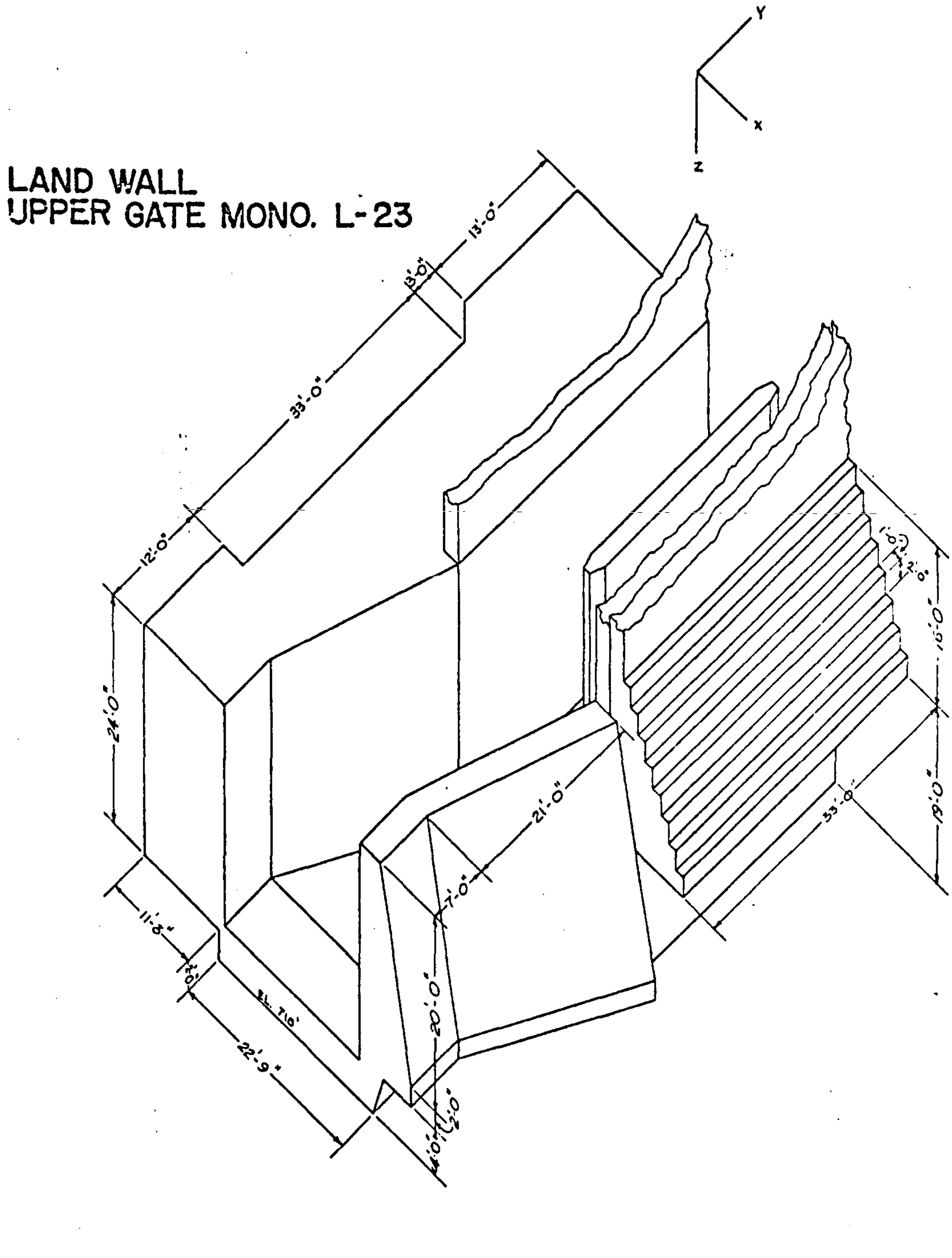


FIGURE VI-2 THREE DIMENSIONAL VIEW - LAND WALL
UPPER GATE MONOLITH (L-23)

Page 2 of 7

ITEM	FACTORS	F_V	F_H	F_D	ARM_Z	ARM_X	ARM_Y	M_{ZH}	M_{XV}	M_{YV}
W_{CONC}	[(16)(3)(33)+(14)(1)(33)+(12)(1)(33)+(10)(11)(33)+(8)(1)(33)+(6)(1)(33)+(4)(1)(33)+(2)(1)(33)+(2)(35)(22.5)+(2)(35)(24)+(2)(35)(33)+(16)(13)(34)+(13)(20)(34)+(3)(11)(20)+(3)(3)(1/2)(23)+(2)(9)(2)(33)+(2)(.65)(35)(9)+(2)(.65)(8.5)+(2)(.65)(6.83)+(4)(12)(24)+(4)(2)(22)+(.65)(1.25)(20)+(3)(23)(4)+(1/2)(20)(5)(4)+(5)(2)(4)+(1/2)(4)(8)+(12)(3)(24)+(22)(2)(3)+(3)(3)(23)+(1/2)(20)(5)(3)+(5)(2)(3)+(.65)(20)(33)+(12)(24)(5)+(26.6)(5)(22)+(1)(5)(22)+(7.12)(5)+(1/2)(1.9)(5)(20)+(5)(5)(1/2)(20)+(5)(5)(2)+(5)(5)(1/2)(1.25)+(3)(14)(1)+(9)(14)(24)+(24.9)(23.25)(14)+(5.65)(26.11)(14)+(5.39)(14)(1/2)(23.25)+(5)(21.25)(1/2)(14)+(5)(2.68)(14)(1/2)] [0.150]	6320.7				17.98	34.62		113646	218823
P_{EARTH_1}	$[(750-728.3)^2(1/2)(.135)(.5)+(750-728.3)(728.3-725)(.135)(.5)+(728.3-725)^2(1/2)(.0856)(.5)][33]$		-692		20.2			-13978		
P_{EARTH_2}	$[(750-728.3)^2(1/2)(.135)(.5)(21)-(750-736)^2(1/2)(.135)(.5)(21)+(750-728.3)(728.3-713.48)(.135)(.5)(21)+(728.3-713.48)^2(.5)(1/2)(.0856)(21)]$ [COS 21° AND SIN 21°]		-564.6	216.7	17.91			-10112		3881
P_{EARTH_3}	$(750-728.3)^2(.135)(.5)(1/2)(7)-(750-736)^2(.135)(.5)(1/2)(7)+(750-728.3)(728.3-716)(.135)(.5)(7)+(728.3-716)^2(1/2)(.0856)(.5)(7)$		-213.7		19.3			-4124		
P_{WATER_R}	$(718.7-704)^2(1/2)(.0624)(13)$		87.6		-3.1			-272		
$P_{WATER_{OL}}$	$(726.9-710)^2(48)(1/2)(.0624)$		427.7		3.63			1553		
W_{WATER_4}	$(726.9-713)(3)(33)(.0624)$	85.9				1.5	28.5		129	2448
P_{WATER_1}	$(728.3-706)^2(1/2)(.0624)(33)$		-512		1.45			-742		
P_{WATER_2}	$(728.3-707.48)^2(1/2)(.0624)(21)$ [COS 21° AND SIN 21°]		-12.7	4.9	2.42			-31		12
P_{WATER_3}	$(728.3-711)^2(1/2)(.0624)(7)$		-3.8		4.77			-18		

FIG. VI-2 LANDWALL UPPER GATE MONO L-23 (CONTINUED)

SUBJECT

LAND WALL - UPPER GATE MONO L-23

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

ITEM	FACTORS	F_V	F_H	F_D	Arm_Z	Arm_X	Arm_Y	M_{XH}	M_{XV}	M_{YV}
	CONTINUED									
W_{EARTH_1}	[(2.14)(.135)+(4.42)(.135)+(6.7)(.135)+(8.98)(.135)+(11.26)(.135)+(13.54)(.135)+(14.52)(.135)+(1.3)(.0856)+(14)(10)(.135)][33]	901				45.77	44.5		41239	40095
W_{EARTH_2}	(½)(2.01)(8.05)(.135)(21)+(2.99)(8.05)(.135)(21)+(½)(2.99)(14.82)(.0856)(21)+(14)(5)(.135)(21)	329.4				41.4	17.5		13637	5765
W_{EARTH_3}	(½)(2.01)(8.05)(.135)(7)+(2.99)(8.05)(.135)(7)+(½)(2.99)(14.82)(.0856)(7)+(14)(5)(.135)(7)	109.8				37.81	3.5		4152	384
W_{WATER_2}	(½)(2.99)(14.82)(.0624)(21)	29.4				42.62	17.5		1253	515
W_{WATER_1}	(728.3-727)(1)(33)(.0624)	2.7				49.5	44.5		134	120
W_{WATER_3}	(728.3-716)(½)(2.9)(7)(.0624)	8				39.01	3.5		312	28
W_{WATER_c}	(20)(7)(726.9-713)(.0624)+(13.9)(20)(21)(.0624)+(½)(20)(21)(5.04)(.0624)+(2)(9.1)(33)(.0624)(18.9)+(2)(8.83)(.0624)(18.9)	1280.6				26.83	22.33		34358	28596
$GATE_M$								-18		
$GATE_V$		1.2				1.5	45		2	54
$GATE_H$			184		12.5			2300		
$GATE_{D.S.}$				154	12.5					1925
HAWSER			-24		11.7			281		
W_{BLDG}	(0.10)(858)+(0.15)(858)+(0.02)(858)+(0.15)(858)+(92)(1.08)(.128)(14.82)	548.8				28	44.5		15366	24422

WES FORM NO. 1253A

FIG. VI-2 LANDWALL UPPER GATE MONO L-23 (CONTINUED)

SUBJECT

LAND WALL - UPPER GATE MONO L-23

COMPUTED BY | DATE

CHECKED BY | DATE

ITEM	FACTORS	F_V	F_H	F_D	ARM_Z	ARM_X	ARM_Y	M_{XH}*	M_{XV}	M_{YV}
	CONTINUED									
$UPLIFT_1$	$[(0.0869)(10) + \frac{1}{2}(.2059-.0869)(10) + (.9869)(24) + (1.2725-.9869)(24)(\frac{1}{2}) + (.9712)(16) + (1.1117-.9712)(\frac{1}{2})(16)][33]$	-1472.8				21.57	44.5		-32200	-66430
$UPLIFT_2$	$[(.418)(12) + (.583-.418)(\frac{1}{2})(12) + (.864)(26) + (1.216-.864)(\frac{1}{2})(26) + (.994)(6) + (1.216-.994)(\frac{1}{2})(2) + (1.048-.994)(\frac{1}{2})(4)][21]$	-826				25.2	17.5		-20815	-14455
$UPLIFT_3$	$[(.418)(12) + (\frac{1}{2})(.598-.418)(12) + (.723)(23) + (1.068-.723)(\frac{1}{2})(23) + (.848)(5) + (\frac{1}{2})(1.068-.848)(2) + (.893-.848)(\frac{1}{2})(3)][7]$	-218.5				22.66	3.5		-4951	-765
		7080.2	-1323.5	375.6				-25168	166263	245418

NOTE: M_{XH} IS READ: "MOMENT TURNING MONOLITH IN X-DIRECTION (ABOUT Y-AXIS) BY THE HORIZONTAL FORCES"

FIG. VI-2 LANDWALL UPPER GATE MONO L-23 (CONTINUED)

SUBJECT

LAND WALL- UPPER GATE MONO L-23 (NORMAL OPNS)

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

OVERTURNING:

ΣF_y = 7080.2 KIPS

$\Sigma M_{xH} + \Sigma M_{xV}$ = -25168 + 166263 = 141095 FT-KIPS

e_x = 141095 /7080.2 = 19.93 FT (NOT CRITICAL IN OVERTURNING AS LONG AS THE STRUCTURE ACTS AS A MONOLITH AND NOT AS SEPARATE UNITS)

ΣF_v = 7080.2 KIPS

ΣM_{Yv} = 245418 FT-KIPS

e_y = 245418/7080.2 = 34.66 FT

SLIDING:

R= ΣF_v TAN ϕ + SHEAR RESISTANCE DUE TO CONCRETE SECTION

R= (7080.2)(0.63707) + (4)(28)(0.075)

R= 4510.6 + 8.4 = 4519

S_{SF} = 4519/1323.5 = -3.41 (NOT ADEQUATE FOR SLIDING)

BASE PRESSURE:

APPROXIMATE BASE PRESSURE = 28 KSF

VI-21

253A

FIG VI-2 LAND WALL UPPER GATE MONO L-23 (CONTINUED)

SUBJECT: LAND WALL - UPPER GATE MONO L-23 (MAINT COND)

SUBMITTED BY:

CHECKED BY:

DATE:

DATE:

ITEM	FACTORS	F_V	F_H	F_D	ARM_Z	ARM_X	ARM_Y	M_{XH}	M_{XV}	M_{YV}
W_{CONC}		6320.7				17.98	34.62		113646	218823
P_{EARTH_1}			-692		20.2			-13978		
P_{EARTH_2}			-564.6	216.7	17.91			-10112		3881
P_{EARTH_3}			-213.7		19.3			-4124		
P_{WATER_1}			-512		1.45			-742		
P_{WATER_2}			-12.7	4.9	2.42			-31		12
P_{WATER_3}			-3.8		4.77			-18		
W_{EARTH_1}		901				45.77	44.5		41239	40095
W_{EARTH_2}		329.4				41.4	17.5		13637	5765
W_{EARTH_3}		109.8				37.81	3.5		4152	384
W_{WATER_1}		29.4				42.62	17.5		1253	515
W_{WATER_2}		2.7				49.5	44.5		134	120
W_{WATER_3}		8				39.01	3.5		313	28
$GATE_M$								-18		
$GATE_V$		1.2				1.5	45		2	54
W_{BLDG}		548.8				28	44.5		15366	24422
$UPLIFT_1$	[(1/2)(8.28)(.2509) + (24)(.6643) + (1/2)(24)(.7272) + (1/2)(16)(.7891)][33]	-1057				23.69	44.5		-25040	-47037
$UPLIFT_2$	[(1/2)(12)(.2742) + (26)(.3990) + (1/2)(26)(.5839) + (6)(.7790) + (1/2)(2)(.2039) + (1/2)(3)(.1119)][21]	-701.6				28.46	17.5		-19968	-12278
$UPLIFT_3$	[(1/2)(12)(.3048) + (23)(.4296) + (1/2)(23)(.1546) + (5)(.3346) + (1/2)(2)(.2496) + (1/2)(3)(.5577)][7]	-113.7				24.61	3.5		-2798	-398
		6378.7	-1998.8	221.6				-27016	141936	234386

FIG. VI-2 LAND WALL UPPER GATE MONO L-23 (CONTINUED)

SUBJECT: LAND WALL- UPPER GATE MONO L-23 (MAINT CASE)

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

OVERTURNING:

$\Sigma F_V = 6378.7$ KIPS

$\Sigma M_{XH} + \Sigma M_{XV} = -27016 + 141936 = 114920$ FT-KIPS

$e_x = 114920/6378.7 = 18.02$ FT (NOT CRITICAL IN OVERTURNING AS LONG AS THE STRUCTURE ACTS AS A MONOLITH AND NOT AS SEPARATE UNITS)

$\Sigma F_V = 6378.7$ KIPS

$\Sigma M_{YV} = 234386$ FT-KIPS

$e_y = 234386/6378.7 = 36.75$ FT

VI-23

SLIDING:

$R = \Sigma F_V \tan\phi$ + SHEAR RESISTANCE DUE TO CONCRETE SECTIONS

$R = (6378.7)(.63707) + (4)(28)(0.075)$

$R = 4063.7 + 8.4 = 4072.1$

$S_{S-F} = 4072.1/-1998.8 = -2.04$

BASE PRESSURE:

APPROXIMATE BASE PRESSURE = 31 KSF

253A

FIG VI-2 LAND WALL UPPER GATE MONO L-23 (CONTINUED)

SUBJECT:

LAND WALL - LOCK CHAMBER L-32

COMPUTED BY:	DATE:
CHECKED BY:	DATE:

STATION
2+47 B to 2+83 B

LENGTH
36.00 FT

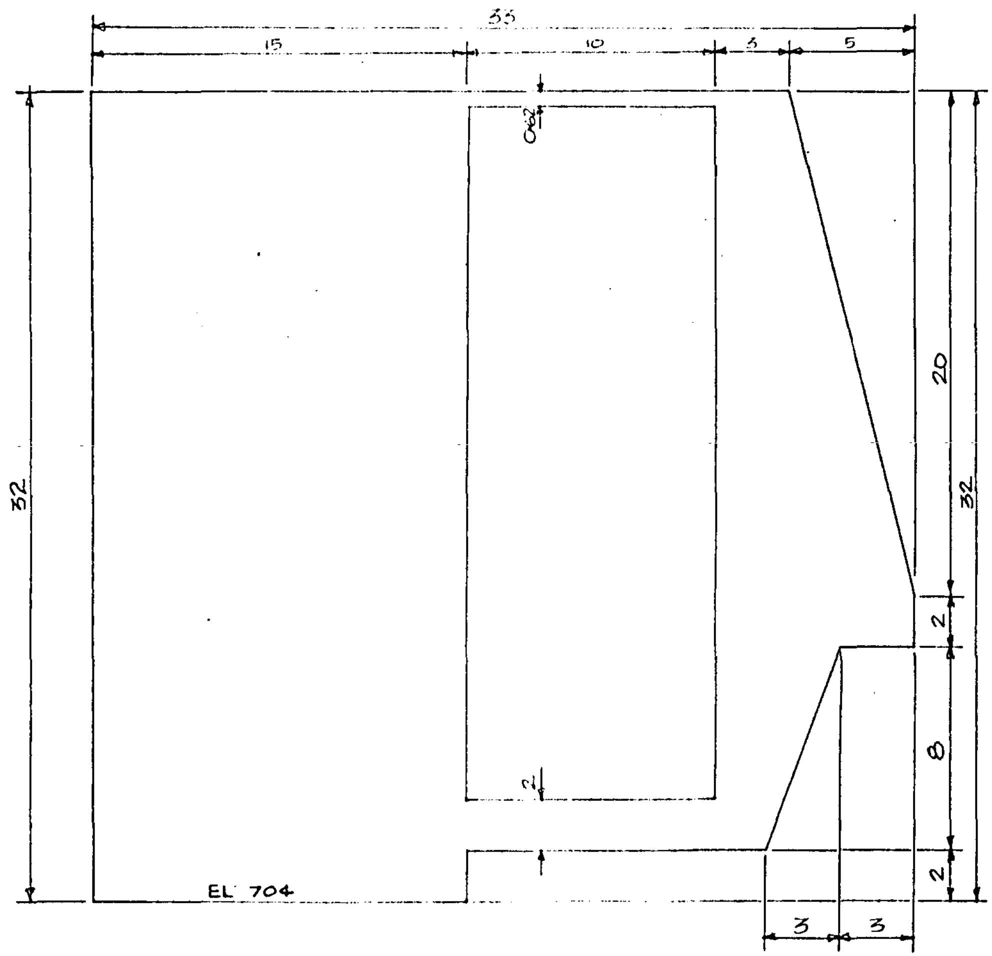


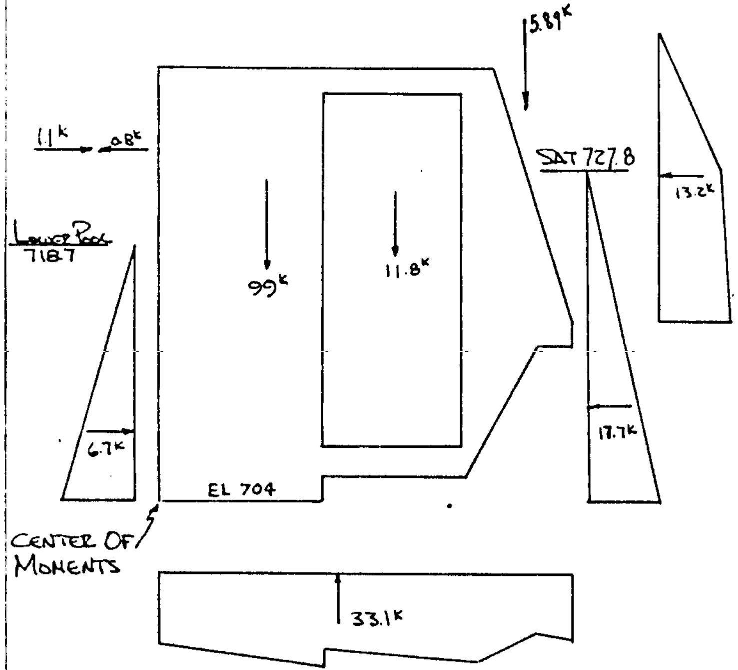
FIGURE VI-3. LAND WALL - LOCK CHAMBER L-32

SUBJECT: LAND WALL - LOCK CHAMBER L-32

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.150][(15)(28)+(3)(20)+(1/2)(5)(20)+(2)(8)+(2)(6)+(1/2)(3)(8)+(2)(27)+(2)(15)+(10)(0.62)][1]	99		12.74	1261
W_{WATER}	(726.9-708)(.0624)(10)(1)	11.8		20	236
W_{earth}		5.89		29.77	175
H_{WATER}	$(727.8-704)^2(.0624)(1/2)(1)$		-17.7	7.9	-140
P_{WATER}	$(718.7-704)^2(.0624)(1/2)(1)$		6.7	4.9	33
UPLIFT		-33.1		15.5	-513
H_{EARTH}	$(736.63-727.8)^2(.135)(1/2)(1)(.5) - (736.63-736.5)^2(.135)(1/2)(1)(.5)+(736.63-727.8)(.135)(727.8-716)(1)(.5)+(727.8-716)^2(.0856)(1/2)(1)(.5)$		-13.2	21.3	-281
HAWSER	(0.8)(1)		-0.8	20	-16
IMPACT	(40/36)(1)		1.1	20	22
	①	83.59	-23.9		777
	NEGLECTING IMPACT ②	83.59	-25		755

$e_1 = 777/83.59 = 9.30'$

$e_2 = 755/83.59 = 9.03'$

CASE I: NORMAL OPERATIONS

ASSUMPTIONS

LOWER POOL (718.7) IN LOCK
UPPER POOL (726.9) IN CULVERT
SATURATION EL 727.8

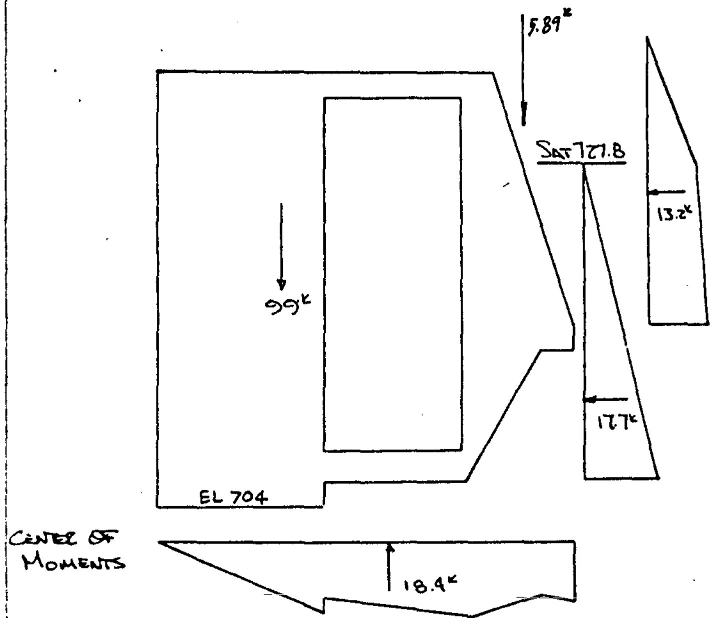
FIGURE VI-3 LAND WALL - LOCK CHAMBER L-32 (CONTINUED)

ENG FORM 1253A

SUBJECT: LAND WALL- LOCK CHAMBER L-32.

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

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		99		12.74	1261
W_{earth}		5.89			175
P_{WATER}			-17.7	7.9	-140
P_{EARTH}			-13.2	21.3	-281
UPLIFT		-18.4		19.7	-362
		86.49	-30.9		653

$$e = 653/86.49 = 7.55$$

CASE II: MAINTENANCE CONDITION

ASSUMPTIONS
- DEWATER LOCK
- DEWATER CULVERT
- SATURATION EL 727.8

FIGURE VI-3 LAND WALL - LOCK CHAMBER L-32 (CONTINUED)

SUBJECT: LAND WALL - LOCK CHAMBER L-32

COMPUTED BY: DATE:

CHECKED BY: DATE:

SLIDING

CASE I: NORMAL OPERATIONS

$R = \Sigma V \tan \phi$

$R = (83.59)(.63707)$

$R = 53.25$

$S_{SF} = 53.25 / -23.9 = -2.23$

INCLUDING IMPACT $S_{SF} = 53.25 / -25 = -2.13$

CASE II: MAINTENANCE CONDITION

$R = (86.49)(.63707)$

$R = 55.10$

$S_{S.F} = 55.10 / -30.9 = 1.78$

FOUNDATION PRESSURE

CASE I: NORMAL OPERATIONS

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

$f = (\frac{2}{3})(\frac{83.59}{9.30}) = 5.99$ K/FT²

NEGLECTING IMPACT $f = (\frac{2}{3})(\frac{83.59}{9.03}) = 6.17$ K/FT²

CASE II: MAINTENANCE CONDITION

$f = (\frac{2}{3})(\frac{86.49}{7.55}) = 7.64$ K/FT²

FIGURE VI-3 LAND WALL - LOCK CHAMBER L-32 (CONTINUED)

SUBJECT: LAND WALL - LOCK CHAMBER L-32 (WITH CONSTRUCTION JOINTS)

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

STATION
2+47B to 2+83B

LENGTH
36.00 FT.

CONSTRUCTION JOINTS

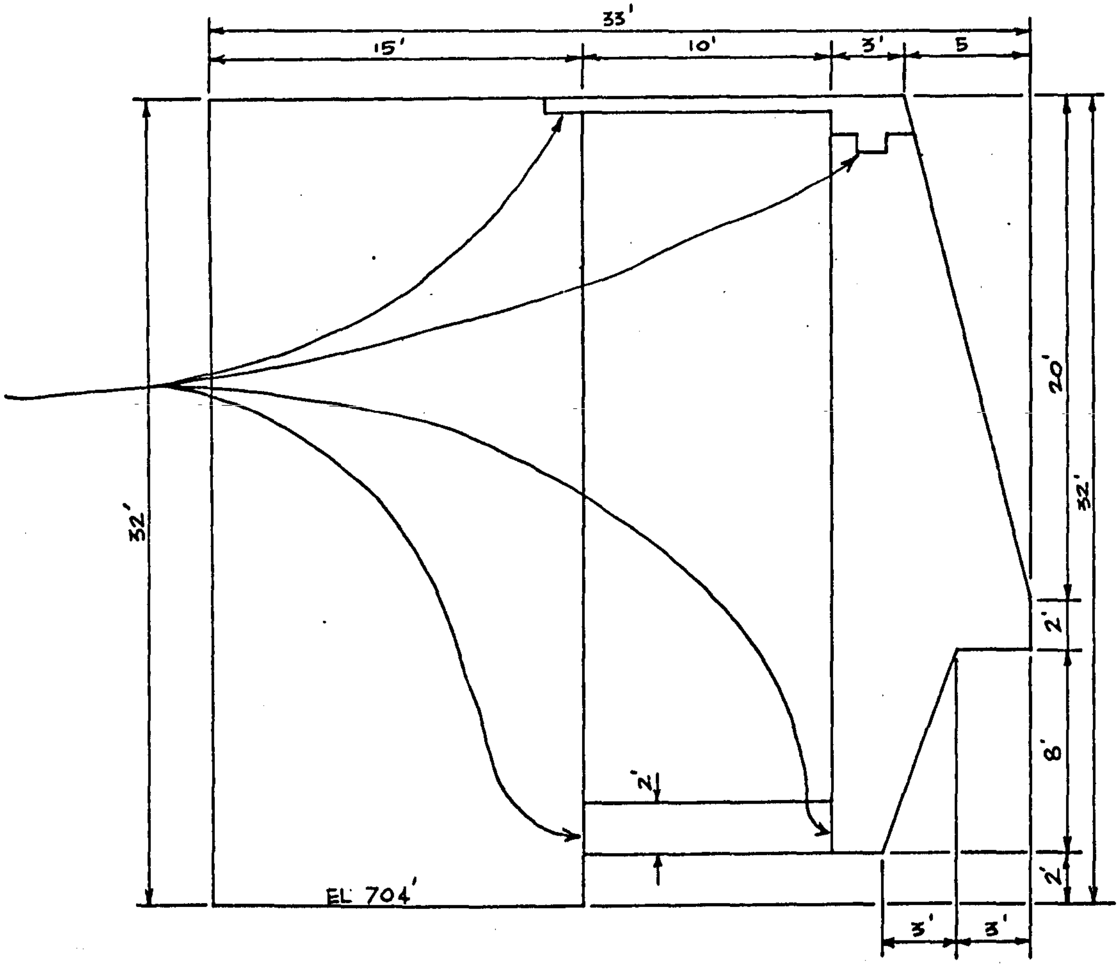


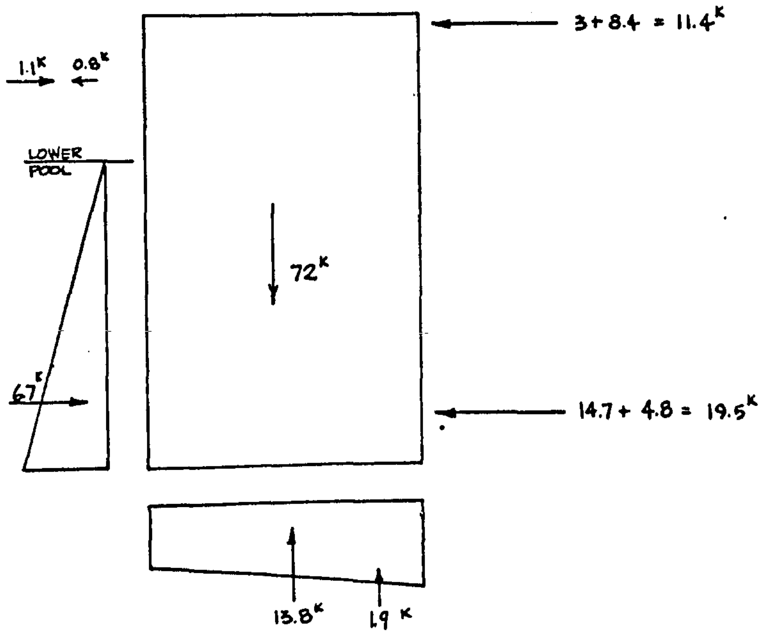
FIGURE VI-4 LAND WALL - LOCK CHAMBER L-32 W/ CONSTRUCTION JOINTS

SUBJECT: LAND WALL LOCK CHAMBER L-32 (W/ CONSTRUCTION JOINTS)

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	Fv	FH	ARM	MOMENT
W_{CONC}	$(15)(32)(0.15)(1)$	72.0		7.5	540
P_{WATER}	$(718.7 - 704)^2(0.0624)(1/2)(1)$		6.7	4.9	33
STRUT LOADS THROUGH TOP SLAB	$(4.9/28.69)(-17.7)$		-3.0	31.69	-95
	$(18.3/28.69)(-13.2)$		-8.4	31.69	-266
STRUT LOADS THROUGH BOTTOM SLAB	$(23.79/28.69)(-17.7)$		-14.7	3.0	-44
	$(10.39/28.69)(-13.2)$		-4.8	3.0	-14
UPLIFT	$-(14.7)(0.0624)(15)(1)$	-13.8		7.5	-103
	$-(4.13)(0.0624)(15)(1)(1/2)$	-1.9		10.0	-19
HAWSER			-0.8	20	-16
IMPACT	$(40/36)$		1.1	20	22
with HAWSER ①		56.3	-25.0		16
with IMPACT ②		56.3	-23.9		54

$$e_1 = \frac{16.0}{56.3} = 0.28$$

$$e_2 = \frac{54.0}{56.3} = 0.96$$

CASE I NORMAL OPERATIONS

LOWER POOL IN LOCK (718.7)

FIGURE VI-4 LAND WALL - LOCK CHAMBER L-32 W/ CONSTRUCTION JOINTS (CONTINUED)

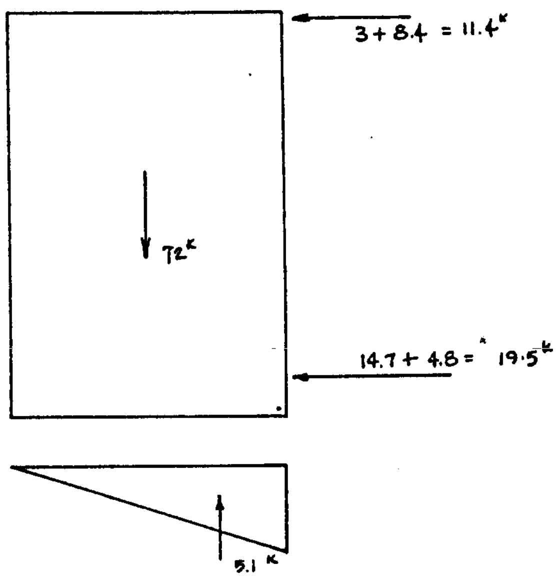
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL LOCK CHAMBER L-32 (W/ CONSTRUCTION JOINTS)

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	Fv	FH	ARM	MOMENT
Wconc	(15)(32)(0.15)(1)	72.0		7.5	540
STRUT LOADS THROUGH TOP SLAB	(4.9/28.69)(-17.7)		-3.0	31.69	-95
	(18.3/28.69)(-13.2)		-8.4	31.69	-266
STRUT LOADS THROUGH BOTTOM SLAB	(23.79/28.69)(-17.7)		-14.7	3.0	- 44
	(10.39/28.69)(-13.2)		- 4.8	3.0	- 14
UPLIFT	(1/2)(23.8)(15/33)(15)(1)(.0624)		- 5.1	10.0	- 51
		72.0	-36.0		70

$$e = \frac{70.0}{72.0} = 0.97$$

CASE II MAINTENANCE CONDITION

LOCK CHAMBER DEWATERED

FIGURE VI-4 LAND WALL - LOCK CHAMBER L-32 W/ CONSTRUCTION JOINTS (CONTINUED)

SUBJECT: LAND WALL - LOCK CHAMBER L-32 (W/ CONSTRUCTION JOINTS)

COMPUTED BY:

CHECKED BY:

DATE:

DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \Sigma V \tan\phi + \cancel{cA}^{\,0}$$

$$R = (56.3)(.63707) = 35.86$$

$$S_{SF} = 35.86 / -25.0 = -1.43$$

CASE II MAINTENANCE CONDITION

$$R = (72.0)(.63707) = 45.87$$

$$S_{SF} = 45.87 / -36. = -1.27$$

FOUNDATION PRESSURE

CASE I NORMAL OPERATION

$$f = \frac{2}{3}\left(\frac{56.3}{0.28}\right) = 134.05 \frac{k}{\square'}$$

$$f = \frac{2}{3}\left(\frac{56.3}{0.96}\right) = 39.10 \frac{k}{\square'}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{2}{3}\left(\frac{72}{0.97}\right) = 49.48 \frac{k}{\square'}$$

FIGURE VI-4 LAND WALL - LOCK CHAMBER L-32 W/ CONSTRUCTION JOINTS (CONTINUED)

SUBJECT:

LAND WALL- UPPER CHAMBER L-37

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

STATION: 4+09B TO 4+49B

LENGTH = 40 FT

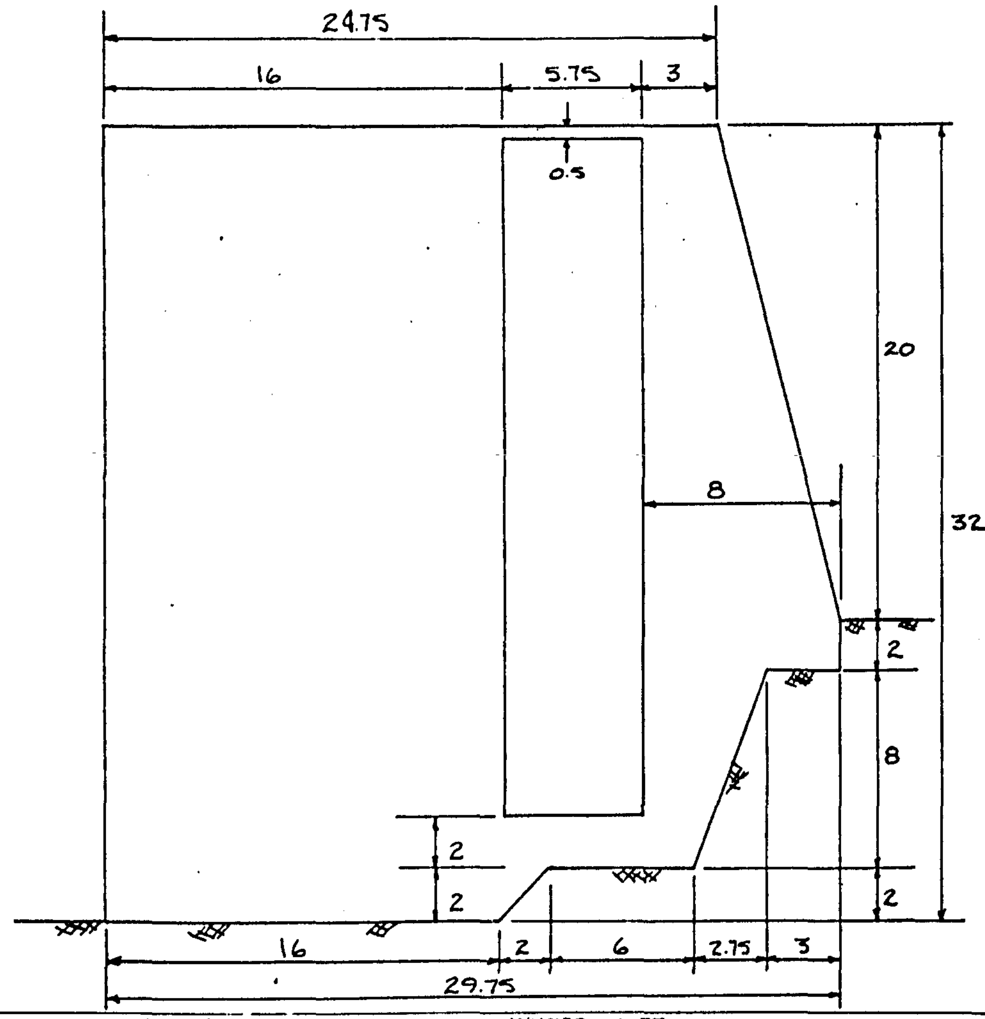


FIGURE VI-5 LAND WALL - UPPER CHAMBER L-37

SUBJECT: LAND WALL - UPPER CHAMBER L-37

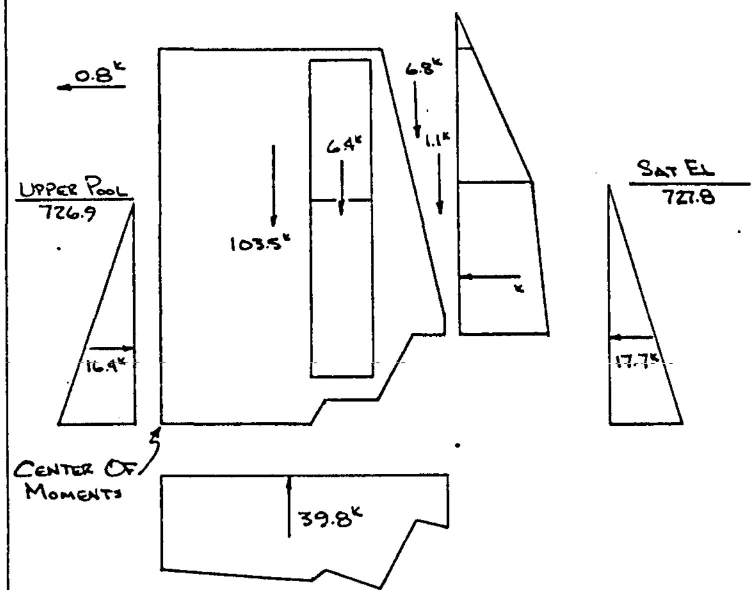
COMPUTED BY:

DATE:

CHECKED BY:

DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.150][(1/2)(20)(5) + (2)(5) + (25)(22) + (1/2)(3)(8) + (8)(24) + (1/2)(2)(2) + (2)(16) - (5.75)(27.5)][1]	103.5		12.16	1259
P_{WATER}	$(726.9-704)^2(1)(.0624)(1/2)$		16.4	7.63	125
P_{WATER}	$(727.8-704)^2(1)(.0624)(1/2)$		-17.7	7.93	-140
P_{EARTH}	$(1/2)(.135)(737.5-727.8)^2(1)(.5) - (1/2)(.135)(737.5-736)^2(1)(5) + (737.3-727.8)(727.8-714)(.135)(1)(.5) + (1/2)(727.8-714)^2(.0856)(1)(.5)$		-16.2	10.58	-171
W_{WATER}	(1/2)(727.8-716)(2.98)(1)(.0624)	1.1		29	32
W_{WATER}	(5.75)(726.9-708)(1)(.0624)	6.8		18.88	128
W_{EARTH}	[(3.99)(736-727.8) + (5)(1/2)(737.5-736)](.135) + (1/2)(727.8-716)(2.98)(.0856)	6.4		28.35	181
UPLIFT	(1/2)(16)(1.429+1.459) + (1/2)(2)(1.459 + 1.336) + (1/2)(6)(1.336 + 1.342) + (1/2)(3)(1.342 + 0.846) + (1/2)(3)(0.846 + 0.861)	-39.8		14.02	-558
HAWSER	(0.8)(1)		-0.8	27.9	-22
		78.0	-18.3		834

$e = 834/78 = 10.69'$

CASE I: NORMAL OPERATIONS

ASSUMPTIONS
SATURATION EL 727.8
UPPER POOL EL 726.9

FIGURE VI-5 LAND WALL - UPPER CHAMBER L-37 (CONTINUED)

VI-33

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - UPPER CHAMBER L-37

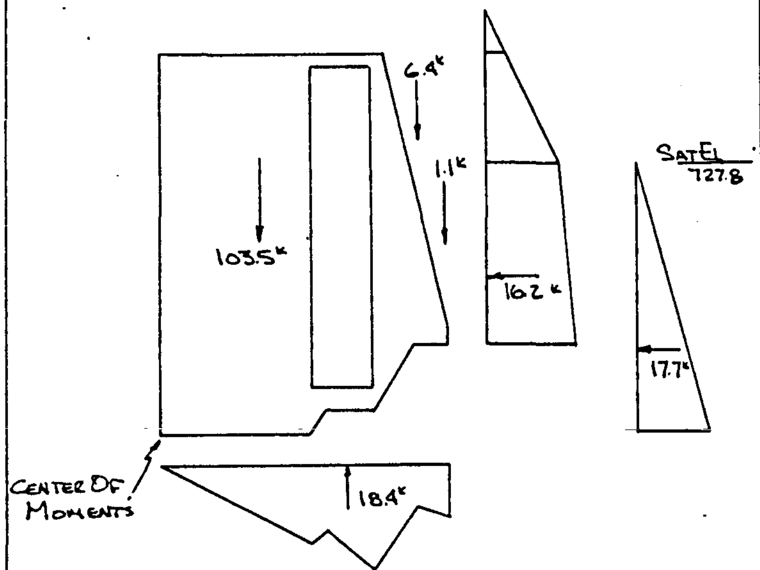
COMPUTED BY:

DATE:

CHECKED BY:

DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		103.5		12.16	1259
P_{WATER}			-17.7	7.93	-140
P_{EARTH}			-16.2	10.58	-171
W_{WATER}		1.1		29.0	32
W_{EARTH}		6.4		28.33	181
UPLIFT	$(\frac{1}{2})(16)(0.791) + (\frac{1}{2})(2)(0.791+0.765) + (\frac{1}{2})(6)(0.765+1.062) + (\frac{1}{2})(3)(1.062+0.712) + (\frac{1}{2})(3)(0.712+0.861)$	-18.4		18.72	-344
		92.6	-33.9		817

$e = 817 / 92.6 = 8.82'$

CASE II: MAINTENANCE CONDITION

ASSUMPTIONS

LOCK CHAMBER DEWATERED

SATURATION EL 727.8

FIGURE VI-5. LAND WALL - UPPER CHAMBER L-37 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - UPPER CHAMBER L-37

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

SLIDING

CASE I: NORMAL OPERATIONS

$$R = \Sigma V \tan\phi$$

$$R = (78)(.63707)$$

$$R = 49.69$$

$$S_{S.F} = 49.69/-18.3 = -2.72$$

CASE II: MAINTENANCE CONDITION

$$R = (92.6)(.63707)$$

$$R = 58.99$$

$$S_{S.F} = 58.99/33.9 = -1.74$$

FOUNDATION PRESSURE

CASE I: NORMAL OPERATIONS

$$f = \frac{78}{29.75} + \frac{(326)(29.75)12}{(2)(29.75)^3}$$

$$= 2.62 + 2.21 = 4.83 \ \frac{k}{ft^2}$$

CASE II: MAINTENANCE CONDITION

$$f = \left(\frac{2}{3}\right)\left(\frac{92.6}{8.82}\right) = 7.00 \ k/FT^2$$

FIGURE VI-5 LAND WALL - UPPER CHAMBER L-37 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT:	COMPUTED BY:	DATE:
LAND WALL - LOCK CHAMBER L42	CHECKED BY:	DATE:

STATION 6+24.5B TO 6+54.5B
LENGTH = 30.0 FT

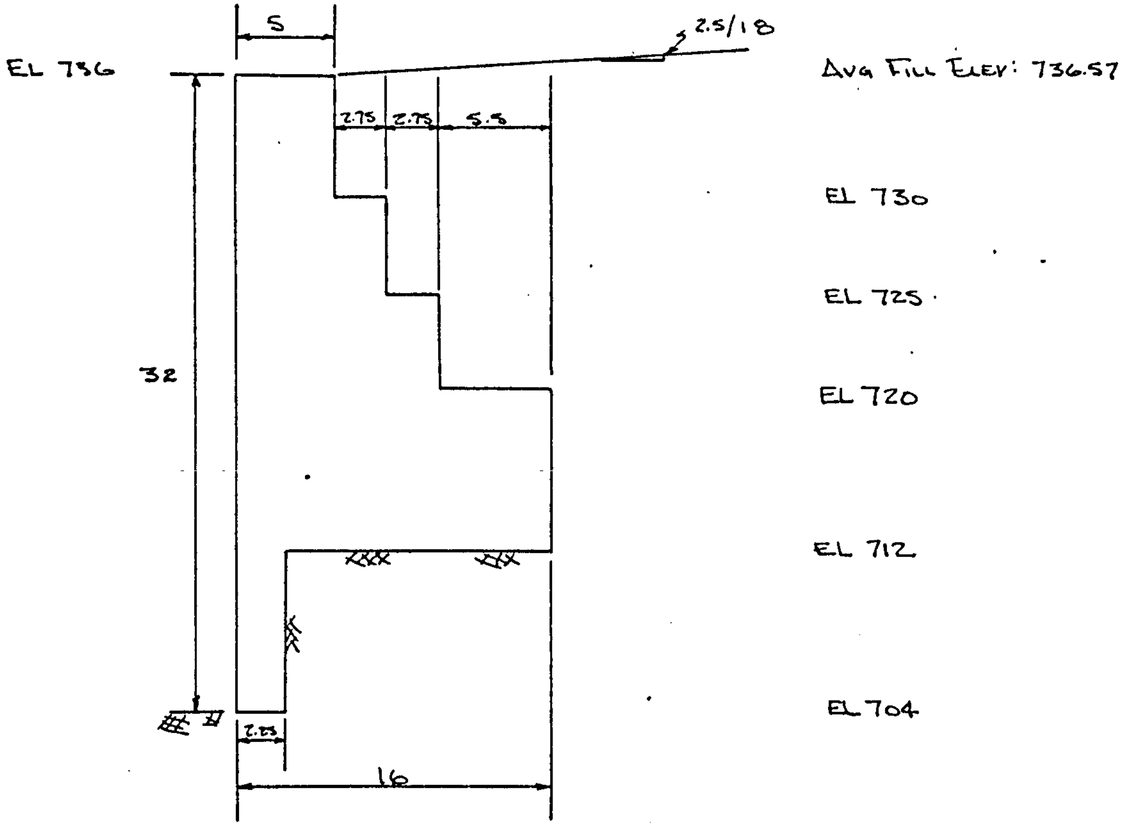
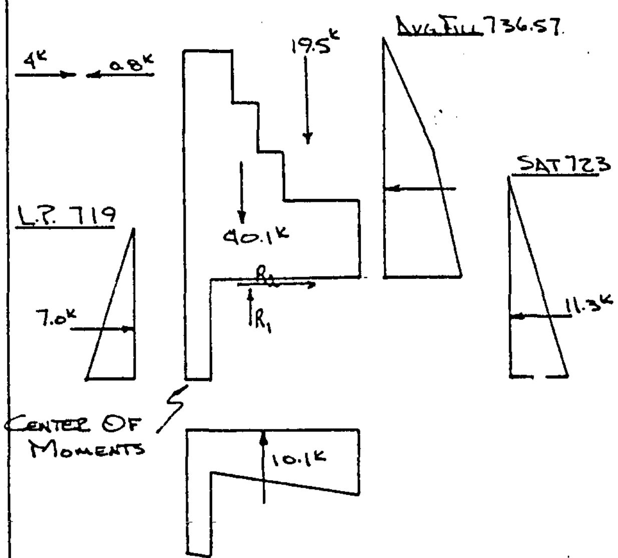


FIG VI-6 LAND WALL - LOCK CHAMBER L-42

SUBJECT:	COMPUTED BY:	DATE:
LAND WALL - LOCK CHAMBER L42	CHECKED BY:	DATE:

NOT TO SCALE:



CASE I: NORMAL OPNS

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[0.150][(2.25)(32)(1)+(2.75)(24)(1)+(2.75)(18)(1)+(2.75)(13)(1)+(5.5)(8)(1)]$	40.1		5.76	231
P_W	$(719-704)^2(.0624)(.5)$		7.0	5.0	35
P_W	$(723-704)^2(.0624)(.5)$		−11.3	6.33	−71
W_E	$[0.135][(2.75)(6.19)(1)+(2.75)(11.57)(1)+(5.5)(14.15)(1)]+(0.148)(5.5)(3)(1)$	19.5		11.54	225
P_E	$(736.57-723)^2(.135)(.5)(.5)+(736.57-723)(.135)(11)(.135)+(723-712)^2(.0856)(.5)(.5)$		−18.9	17.34	−327
UPLIFT	$[.0624][(7.56)(16-2.25)+(11-7.56)(16-2.25)(.5)+(15)(2.25)+(.56)(2.25)(.5)]$	−10.1		7.92	80
HAWSER	24 /30		−0.8	20	−16
IMPACT	4		4	20	80
Reaction (R_1)		−31.48		5.43	−171
Reaction (R_2)			22	8	176
	With Hawser	18.02	−2		2
	With Impact	18.02	1.8		98

$e_1 = 2/18.02 = 0.11$

$e_2 = 98/18.02 = 5.44$

Note: Stability at the upper base section is not critical. 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 the upper section and by assuming R_2 to resist the resultant horizontal force or its maximum value at the section of the upper base.

FIG VI-6 LAND WALL - LOCK CHAMBER L-42 (CONTINUED)

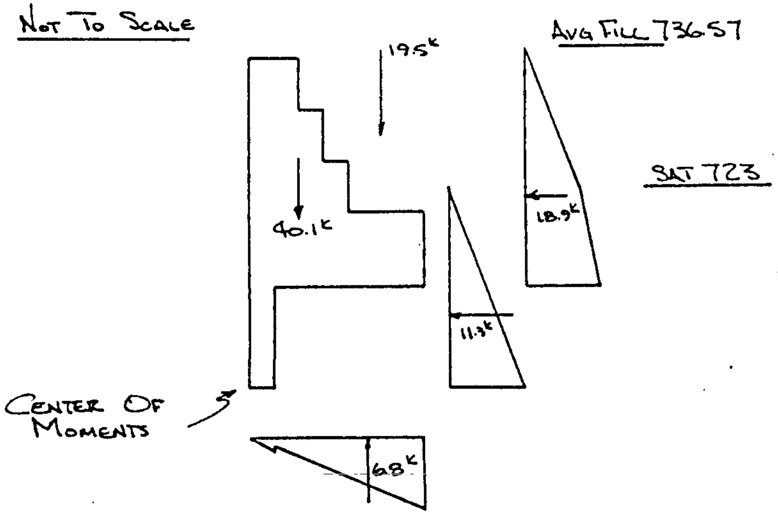
VI-37

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - LOCK CHAMBER L42

COMPUTED BY:
DATE:
CHECKED BY:
DATE:

NOT TO SCALE



CASE II: MAINT COND.

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		40.1		5.67	231
P_W			-11.3	6.33	-71
W_E		19.5		11.54	225
P_E			-18.9	17.34	-327
UPLIFT		6.8		8.38	57
Reaction (R_1)		-34.53		5.66	-195
Reaction (R_2)			20.5	8	164
		18.27	-9.6		-30
	$e = -30/18.27 = -1.64$				

FIG VI-6 LAND WALL - LOCK CHAMBER L-42 (CONTINUED)

SUBJECT: LAND WALL - LOCK CHAMBER L-42 | COMPUTED BY: | DATE: | CHECKED BY: | DATE:

SLIDING

CASE I: NORMAL OPERATIONS

$R = \Sigma V \tan\phi$

$R = (18.02)(.63707) = 11.48$

$S_{S.F} = 11.48 / 2.8 = 4.1$

CASE II: MAINT COND

$R = (18.27)(.63707) = 11.64$

$S_{S.F} = 11.64 / -9.6 = -1.21$

FOUNDATION PRESSURE

CASE I: NORMAL OPERATIONS

$f = (\frac{2}{3})(\frac{?}{e})$

$f = (\frac{2}{3})(\frac{18.02}{0.11}) = 164$ KSF

$f = \frac{18.02}{2.25} + \frac{(98)(4.32)(1.125)(12)}{(1)(2.25)^3}$

$= 8.01 + 502 = 510$ KSF

CASE II: MAINT COND.

Maximum base pressures are very large.

VI-39

WES FORM NO. 1253A DECEMBER 1964

FIG VI-6 LAND WALL - LOCK CHAMBER L-42 (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
LOWER GATE MONO - L46	CHECKED BY:	DATE:

STATION 7+86B TO 8+26B

LENGTH 40.0 FT

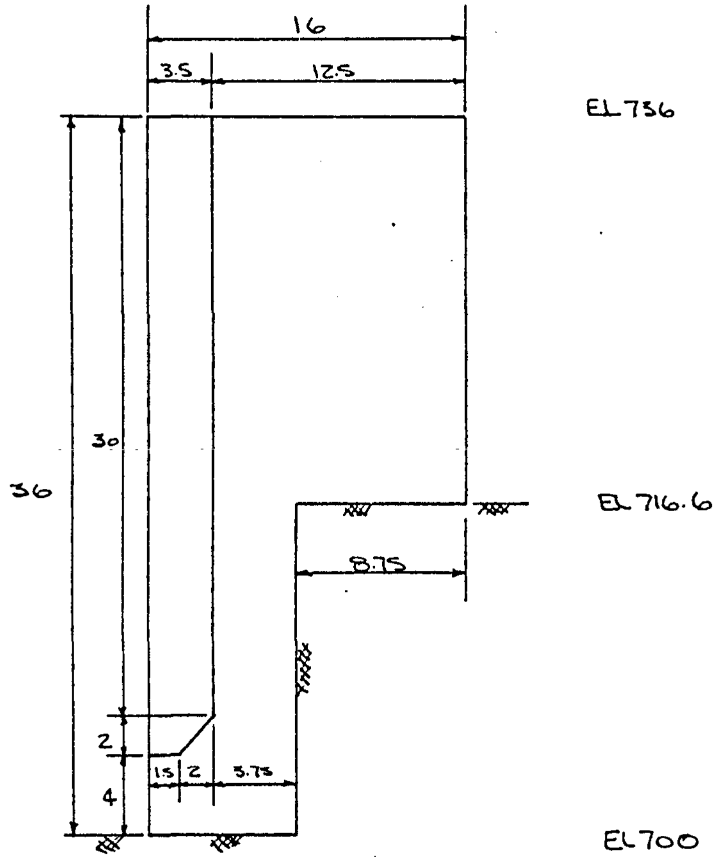


FIG VI-7 LOWER GATE MONO L-46

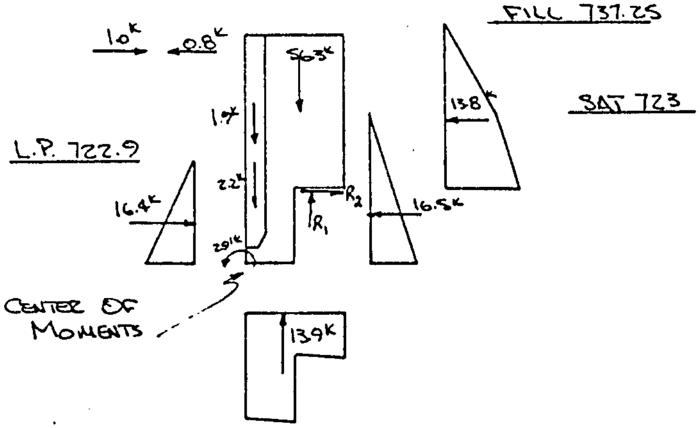
VI-40

SUBJECT: Lower Gate Mono - L46

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

Not To Scale



Case I: Normal Operations

Note: Stability at the upper base section is not critical. 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 the upper section and by assuming R_2 to resist the resultant horizontal force or its maximum value at the section of the upper base.

Item	Factors	Fv	Fh	Arm	Moment
W_{conc}	[0.150][(12.5)(19.4)(1) + (3.5)(30)(1)(.5) + (3.75)(16.6)(1) + (1.5)(2)(1)(.5) + (2)(2)(1/2)(.5)] + (3.5)(4)(1) + (2)(2)(1/2)(1)]	563		7.53	424
W_{water}	[.0624][(16.9)(3.5)(1)(.5) + (1.5)(2)(1)(.5) + (2)(2)(1/2)(.5)]	2.2			3
P_{water}	$(722.9-700)^2(1)(1/2)(.0624)$		16.4	7.6	125
P_{water}	$(23)^2(1)(1/2)(.0624)$		-16.5	7.6	-127
P_{earth}	$(737.25-723)^2(.135)(1/2)(1)(.5) - (737.25-736)^2(.135)(1/2)(1)(.5) + (737.25-723)(723-716.6)(.135)(1)(.5) + (723-716.6)^2(.0756)(1)(.5)(.5)$		-13.8	25.89	-357
Gate	e = 15.75 eV = (74.6)(15.75) = 1175 M_{eV} = 1175/40 = 29.4 V' = 74.6/40 = 1.87	1.9		1.75	-29 3
Uplift	[.0624][(6.37)(16) + (.03)(8.75)(1/2) + (16.53)(7.25) + (0.02)(7.25)]	-13.9		5.61	-78
Hawser			-0.8	27.9	-22
Impact			1	27.9	28
Reaction (R_1)	$\frac{(3.35)(5.66)}{2}$	-9.48		9.14	-87
Reaction (R_2)	(9.48)(0.63707)		6.04	16.6	100
	With Hawser ①	37.02	-8.66		-45
	With Impact ②	37.02	-6.86		5

$e_1 = -45/37.02 = -1.22$

$e_2 = 5/37.02 = 0.14$

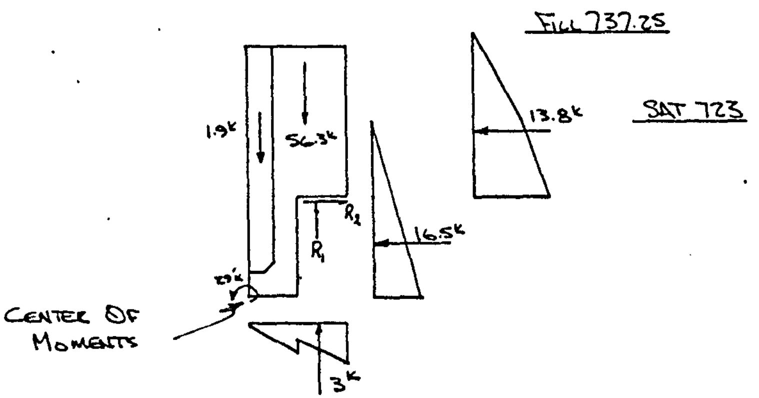
Fig VI-7 Lower Gate Mono L-46 (Continued)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: Lower Gate Mono - L-46

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

Not To Scale



Item	Factors	Fx	Fy	Arm	Moment
W_{conc}		56.3		7.53	424
P_{water}			-16.5	7.6	-127
P_{earth}			-13.8	25.89	-357
Gate	M_{cv}				-29
	V'	1.9		1.75	3
Uplift	[.0624][(.23)(8.75)+(6.17)(8.75)(.5)+(5.12)(7.25)(.5)]	-3.0		9.67	-29
Reaction (R_1)	$\frac{(3.27)(5.54)}{2}$	-9.06		9.1	-82
Reaction (R_2)	(9.06)(0.63707)		5.77	16.6	96
		46.14	-24.53		-101

$e = -101/46.14 = -2.19$

Fig VI-7 Lower Gate Mono L-46 (Continued)

SUBJECT: Lower Gate Mono L46 | COMPUTED BY: | DATE: | CHECKED BY: | DATE:

VI-43

Sliding

Case I: Normal Operations

$R = \sum V \tan\phi$

$R = (37.02)(.63707) = 23.6^{k}$

$S_{S.F} = 23.6 / 8.66 = 2.73$

Case II: Maint Cond

$R = (46.14)(.63707) = 29.39^{k}$

$S_{S.F} = 29.39 / 24.53 = 1.2$

Foundation Pressure

Case I: Normal Operations

Maximum Base Pressures Are Very Large

Case II: Maint Cond

Maximum Base Pressures Are Very Large

WES FORM NO. 1253A DECEMBER 1964

Fig VI-7 Lower Gate Mono L-46 (Continued)

SUBJECT:	COMPUTED BY:	DATE:
Lower Guide Wall - L55	CHECKED BY:	DATE:

Station: 12+00 B to 12+50 B

Length = 50 ft

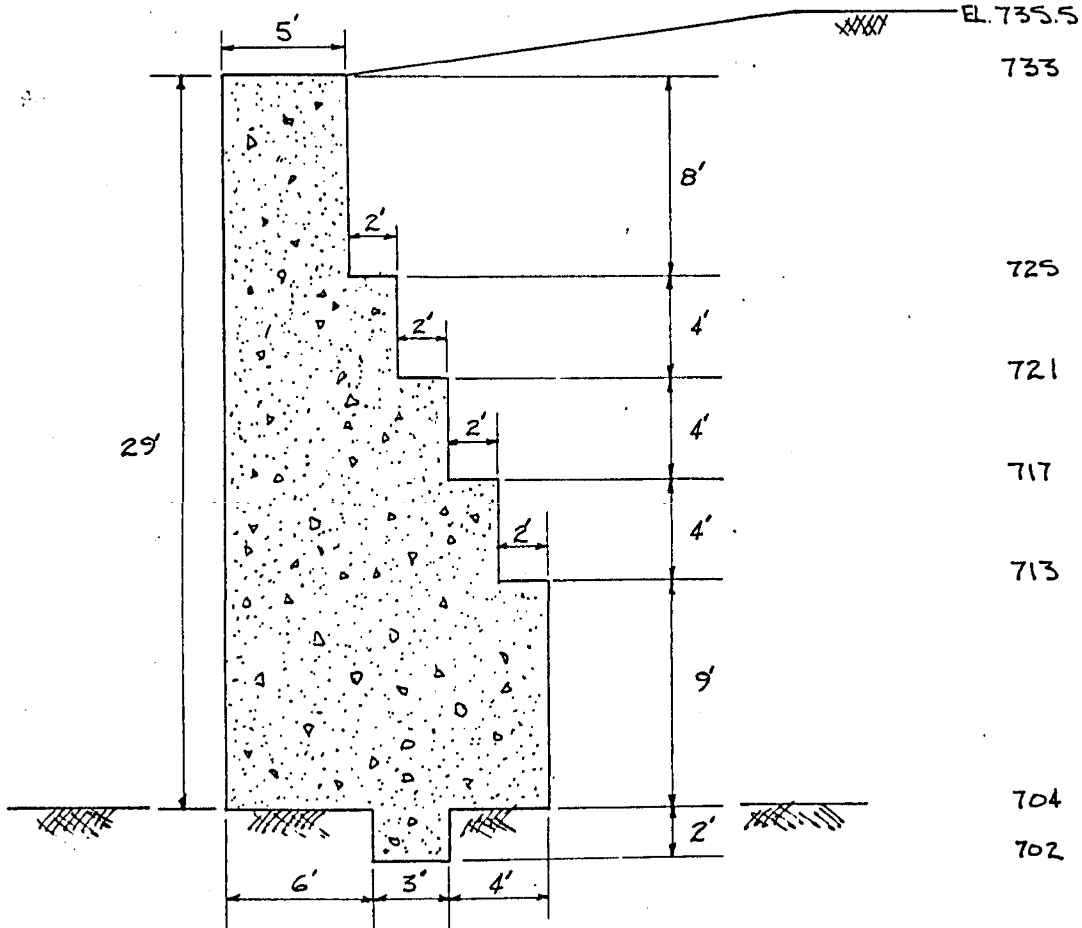


FIGURE VI-8 LAND WALL - LOWER GUIDE WALL L-55

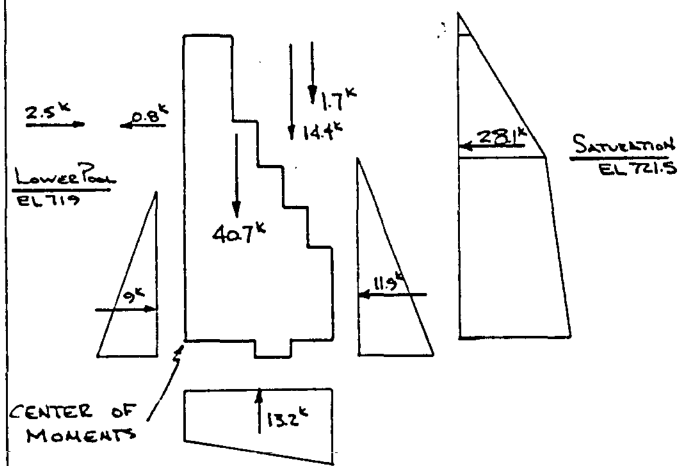
Scale 1:50

SUBJECT: Lower Guide Wall - L55

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

Not to Scale



Item	Factors	F_V	F_H	Dem	Moment
W_{conc}	[0.150][(5)(29)+(2)(21)+(2)(17)+(2)(13)+(2)(9)+(3)(2)][1]	40.7		5.15	210
P_{water}	$(719-702)^2(1)(0.0624)(1/2)$		9	3.67	33
P_{water}	$(721.5-702)^2(1)(.0624)(1/2)$		-11.9	4.5	-54
Hawser	(24)(1/30)		-0.8	20.	-16
Uplift	(15)(.0624)(1)(13)+(1/2)(2.5)(.0624)(13)(1)	-13.2		6.67	-88
P_{earth}	$(1/2)(.135)(734.8-721.5)^2(1)(.5)-(1/2)(.135)(734.8-733)^2(1)(.5)+(734.8-721.5)(721.5-704)(.135)(1)(.5)+(1/2)(721.5-704)^2(.0856)(1)(.5)$		-28.1	13.86	-390
W_{water}	(2)(.0624)(721.5-721)(1)+(2)(.0624)(721.5-717)(1)+(2)(.0624)(721.5-713)(1)	1.7		11.2	19
W_{earth}	(2)(.135)(733.1-723)(1)+(2)(.135)(733.4-721.5)(1)+(2)(.0856)(721.5-721)(1)+(2)(733.7-721.5)(1)(.135)+(2)(.0856)(721.5-717)(1)+(2)(.135)(734-721.5)(1)+(2)(.0856)(721.5-713)(1)	14.4		9.6	138
Impact	(2.5)(1)		2.5	20.	50
	1	43.6	-29.3		-98
	Neglecting Impact 2	43.6	-31.8		-148

Case I_A: Normal Operations

Assumptions:
Lower Pool EL = 719
Saturation EL = 721.5
K_r = 0.5

$e_1 = -98/43.6 = -2.25$ FT

$e_2 = -148/43.6 = -3.39$ FT

FIGURE VI-8 LAND WALL - LOWER GUIDE WALL L-55 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LOWER GUIDE WALL - L55 | COMPUTED BY: | DATE:
CHECKED BY: | DATE:

CASE I_B: NORMAL OPERATIONS

ASSUMPTIONS:
SAME AS CASE I_A EXCEPT K_r = 0.8

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
P_{EARTH}	$(\frac{1}{2})(.135)(734.8-721.5)^2(1)(.8)-(\frac{1}{2})(.135)(734.8-733)^2(1)(.8)+(734.8-721.5)(721.5-704)(.135)(1)(.8)+(\frac{1}{2})(721.5-704)^2(.0856)(1)(.8)$		-45	13.86	-624
	①	43.6	-46.2		-332
	NEGLECTING IMPACT ②	43.6	-48.7		-382

$e_1 = -332/43.6 = -7.61$ FT

$e_2 = -382/43.6 = -8.76$ FT

CASE I_C: NORMAL OPERATIONS

ASSUMPTIONS:
SAME AS CASE I_A EXCEPT K_A = 0.33

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
P_{EARTH}	$(\frac{1}{2})(.135)(734.8-721.5)^2(1)(.33)-(\frac{1}{2})(.135)(734.8-733)^2(1)(.33)+(734.8-721.5)(721.5-704)(.135)(1)(.33)+(\frac{1}{2})(721.5-704)^2(.0856)(1)(.33)$		-18.6	13.86	-258
	①	43.6	-19.8		34
	NEGLECTING IMPACT ②	43.6	-22.3		-16

$e_1 = 34/43.6 = 0.78$ FT

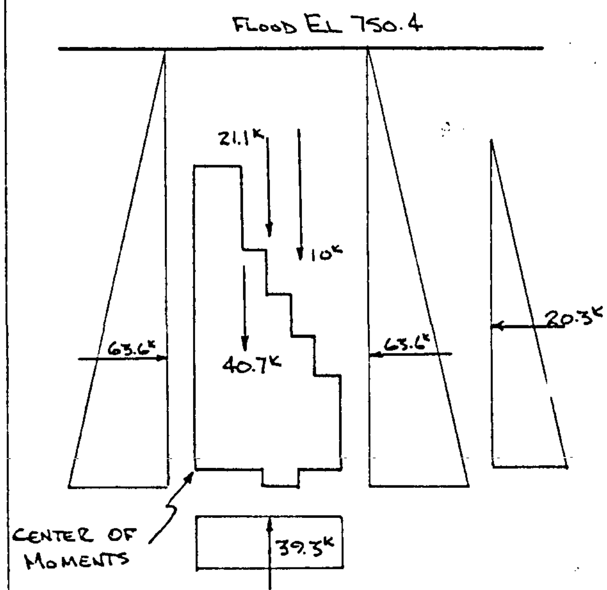
$e_2 = -16/43.6 = -0.37$ FT

FIGURE VI-8 LAND WALL - LOWER GUIDE WALL L-55 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT:	COMPUTED BY:	DATE:
Lower Guide Wall - L55	CHECKED BY:	DATE:

Not To Scale:



Case II: Flood Conditions

Assumptions:
Flood EL 750.4
$K_{r_1} = 0.5$
$K_{r_2} = 0.8$
$K_A = 0.33$

ITEM	FACTORS		F_V	F_H	ARM	MOMENT
W_{CONC}			40.7		5.15	210
P_{WATER}	$(750.4-733)(.0624)(733-702)(1) + (733-702)^2(0.0624)(1)(1/2)$			63.6	13.07	831
P_{WATER}				63.6	13.07	-831
UPLIFT	$(750.4-702)(.0624)(13)(1)$		-39.3		6.5	-255
W_{WATER}	$(5)(.0624)(750.4-733)(1) + (2)(750.4-725)(.0624)(1) + (2)(750.4-721)(.0624)(1) + (2)(750.4-717)(.0624)(1) + (2)(750.4-713)(.0624)(1)$		21.1		7.56	160
W_{EARTH}	$(733.1-725)(2)(.0856)(1) + (733.4-721)(2)(1)(.0856) + (733.7-717)(2)(1)(.0856) + (734-713)(2)(1)(.0856)$		10		9.75	98
P_{EARTH}	$(734.8-704)^2(1/2)(1)(.0856)(.5)$			-20.3	13.86	-281
		1	32.5	-20.3		-68
P_{EARTH}	$(734.8-704)^2(1/2)(1)(.0856)(.8)$			-32.5	13.86	-450
		2	32.5	-32.5		-237
P_{EARTH}	$(734.8-704)^2(1/2)(1)(.0856)(.33)$			-13.4	13.86	-186
		3	32.5	-13.4		27

$e_1 = -68/32.5 = -2.09$ FT

$e_2 = -237/32.5 = -7.29$ FT

$e_3 = 27/32.5 = 0.83$ FT

FIGURE VI-8 LAND WALL - LOWER GUIDE WALL L-55 (CONTINUED)

SUBJECT: LOWER GUIDE WALL - L-55

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

SLIDING

CASE I_A: NORMAL OPERATIONS ($K_r = 0.5$)

$R = \Sigma V \tan\phi + \cancel{sA}^{0} + \text{Key Resistance}$

$= (43.6)(.63707) + (.075)(3)(144)$

$= 27.78 + 32.4 = 60.18^K$

$S_{SF} = 60.18/-29.3 = -2.05$

NEGLECTING IMPACT $S_{SF} = 60.18/-31.8 = -1.89$

CASE I_B: NORMAL OPERATIONS ($K_r = 0.8$)

$S_{S\text{-}F} = 60.18/-46.2 = -1.30$

NEGLECTING IMPACT $S_{S\text{-}F} = 60.18/-48.7 = -1.24$

CASE I_C: NORMAL OPERATIONS ($K_A = 0.33$)

$S_{SF} = 60.18/-19.8 = -3.04$

NEGLECTING IMPACT $S_{SF} = 60.18/-22.3 = -2.70$

CASE II FLOOD CONDITIONS

$R = (32.5)(.63707) + (32.4) = 53.10^K$

$K_r = 0.5$ $S_{S\text{-}F} = 53.10/-20.3 = -2.62$

$K_r = 0.8$ $S_{S\text{-}F} = 53.10/-32.5 = -1.63$

$K_A = 0.33$ $S_{S\text{-}F} = 53.10/-13.4 = -3.96$

FOUNDATION PRESSURE

CASE I_C: NORMAL OPERATIONS ($K_A = 0.33$)

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

$f = (\frac{2}{3})(\frac{43.6}{0.78}) = 37.26\ ^K/_{FT^2}$

CASE II: FLOOD CONDITION ($K_A = 0.33$)

$f = (\frac{2}{3})(\frac{32.5}{0.83}) = 26.10\ ^K/_{FT^2}$

FIGURE VI-8 LAND WALL - LOWER GUIDE WALL L-55

SUBJECT	COMPUTED BY:	DATE:
Lower Guide Wall L-SS	CHECKED BY:	DATE:

Station 12+00B To 12+50B
Length = 50 FT.

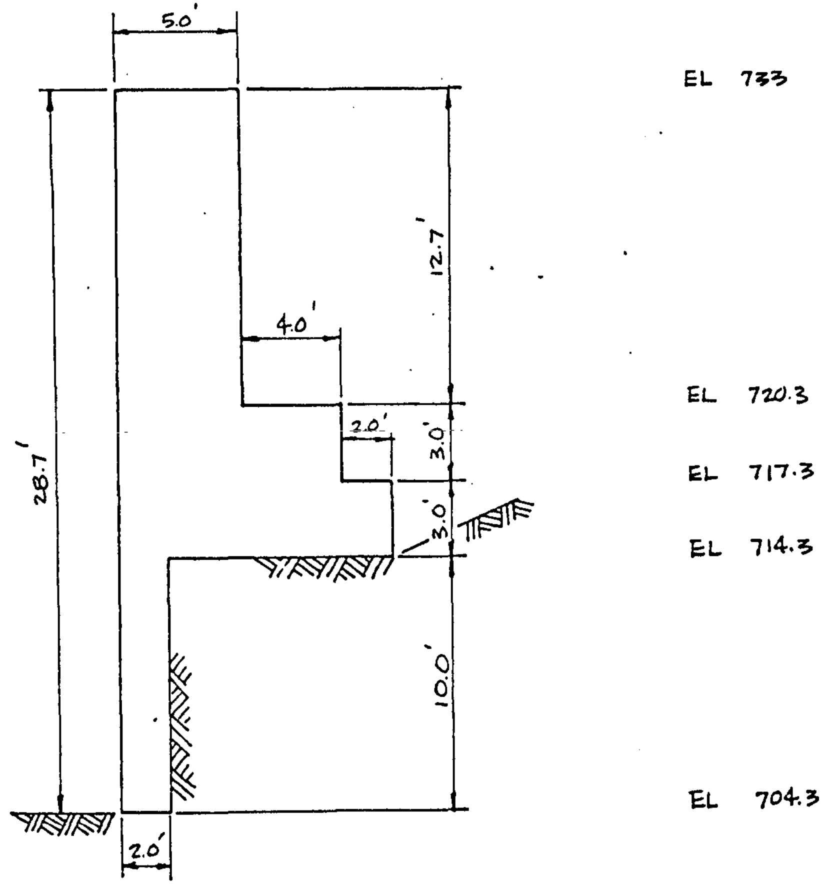


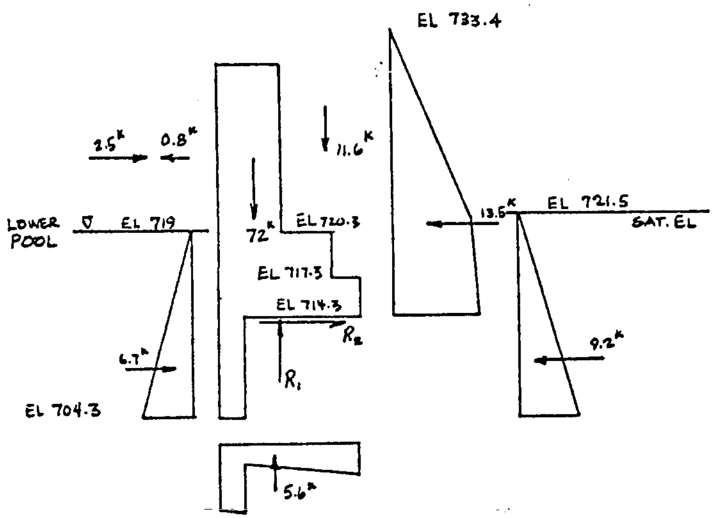
FIG VI-9 Lower Guide Wall L-SS

SUBJECT: Lower Guide Wall L-SS

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$((5)(18.7)+(10)(2)+(4)(6)+(2)(3))(.15)$	21.5		3.35	72.0
$P_{WATER\ LOCK}$	$(719-704.3)^2(.0624)(1/2)$		6.7	4.9	33
$P_{WATER\ LAND}$	$(721.5-704.3)^2(.0624)(1/2)$		-9.2	5.7	-53
P_{EARTH}	$(733.4-721.5)^2(.135)(1/2)(.5)(1)+$ $(733.4-721.5)(721.5-714.3)(.135)(.5)(1)+$ $(721.5-714.3)^2(.0856)(1/2)(.5)(1)$		-11.7	18.6	-218
W_{EARTH}	$((11.8)(4)(.135)+(1.2)(4)(.148))+$ $((12.2)(2)(.135)+(4.2)(2)(.148))$	11.6		8.2	95
UPLIFT	$((6.13)(11)+(1.07)(9)(1/2)+(8.57)(2)+$ $(0.24)(2)(1/2))(.0624)(1)$	-5.6		4.76	-26.6
HAWSER			-0.8	19.7	-16
IMPACT			2.5	19.7	+49
Reaction (R_1)	$\frac{(6.48)(2.56)}{2}$	-8.29		2.85	-24
Reaction (R_2)	$(8.29)(0.63707)$		5.50	10	55
① With Hawser		19.21	-9.5		-81.6
② With Impact		19.21	-6.2		-17.6

CASE I NORMAL OPERATION

LOWER POOL = 719.0
SAT. ELEV = 721.5

$$e_1 = \frac{-82.6}{19.21} = -5.22$$

$$e_2 = \frac{-17.6}{19.21} = -0.92$$

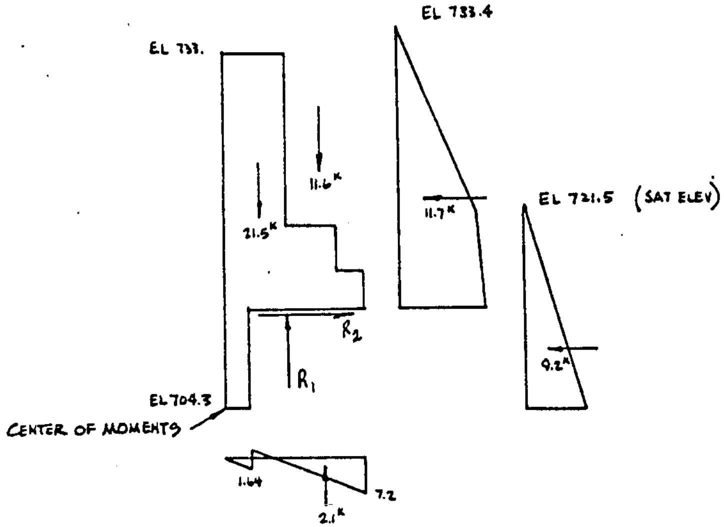
Note: Stability at the upper base section is not critical. The stability of the total monolith should then be considered about the lower base. Reaction R_1 and R_2 must be considered on the total free body. They were calculated assuming no moment in the stem at the upper section and by assuming R_2 to resist the resultant horizontal force at its maximum value at the section of the upper base.

FIG VI-9 LOWER GUIDE WALL L-SS (CONTINUED)

SUBJECT: Lower Guide Wall L-55

COMPUTED BY: | DATE:

CHECKED BY: | DATE:



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		21.5		3.35	72
$P_{WATER\ LAND}$			-9.2	5.7	-53
P_{EARTH}			-11.7	18.6	-218
W_{EARTH}		11.6		8.2	
UPLIFT	$((1.64)(2)(1/2) + (7.2)(8.79)(1/2)).0624$	-2.1		7.74	-16
Reaction (R_1)	$\frac{(6.56)(3.19)}{2}$	-10.5		3.06	-32
Reaction (R_2)	$(0.5)(0.63707)$		6.69	10	67
		20.5	-14.21		-180

$$e = -180/20.5 = -8.78$$

CASE II MAINTENANCE CONDITION

SAT. ELEV = 721.5
LOCK DEWATERED

FIG VI-9 LOWER GUIDE WALL L-55 (continued)

WES FORM NO. 1253A

SUBJECT:

Lower Guide Wall L-55

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

SLIDING

CASE I NORMAL OPERATION

$R = \sum V \tan\phi + \cancel{sA}^{\,0}$

$R = (19.21)(.63707) = 12.24^{k}$

$S_{SF} = 12.24 / -9.5 = -1.29$

CASE II MAINTENANCE CONDITION

$R = (20.5)(.63707) = 13.06$

$S_{SF} = 13.06 / 14.21 = -0.92$

FOUNDATION PRESSURES

Very Large

FIG VI-9 LOWER GUIDEWALL L-55 (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
Middle Wall- Upper Gate Mono M-2	CHECKED BY:	DATE:

Station: 0+03.5A to 0+60.28

Length= 63.7 FT

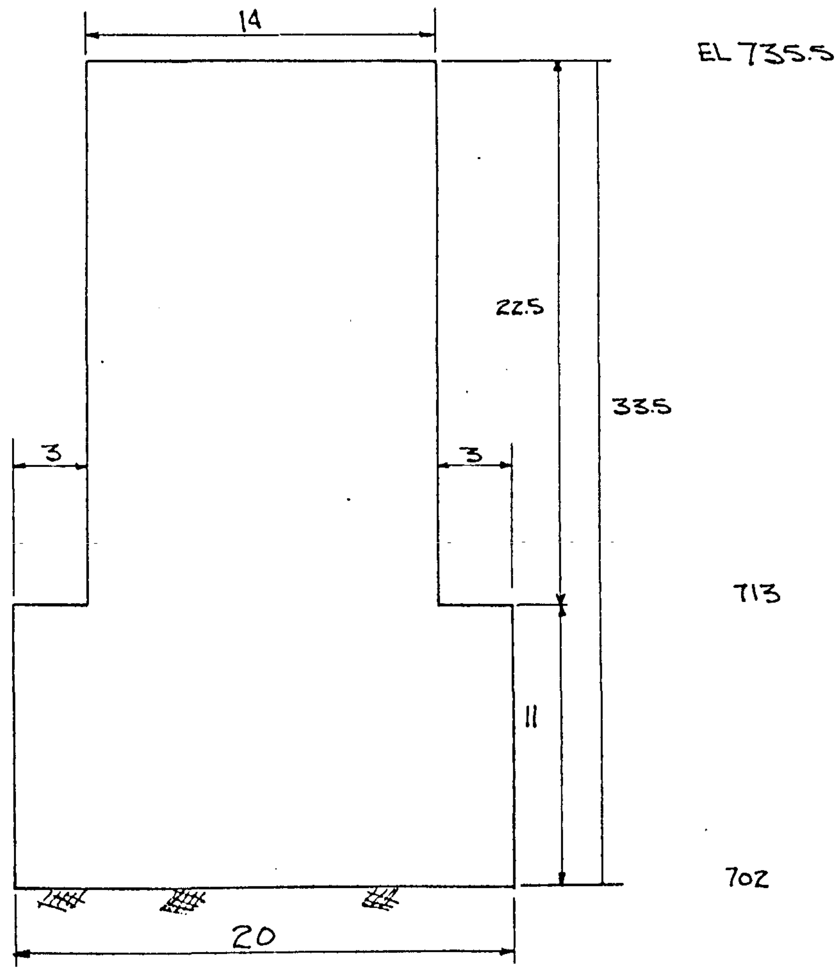
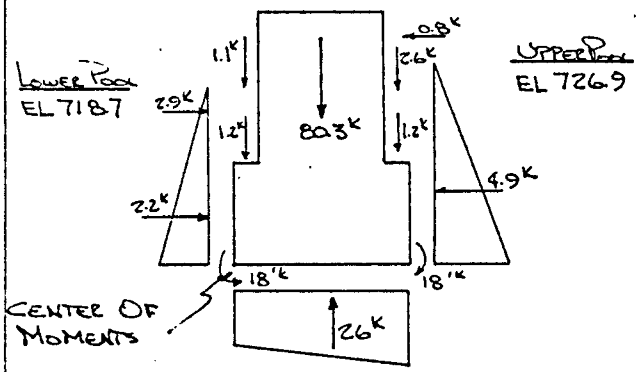


FIGURE VI-10 MIDDLE WALL - UPPER GATE MONO. M-2

Scale 1:50

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - UPPER GATE MONO M-2	CHECKED BY:	DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[0.150][(14)(33.5)+(2)(3)(1)][1]$	80.3		10.0	803
$P_{WATER_{UP}}$	$(1/2)(726.9-702)^2(1)(.0624)(16.25/63.7)$		-4.9	8.3	-41
$P_{WATER_{LP}}$	$(1/2)(718.7-702)^2(1)(.0624)(16.25/63.7)$		2.2	5.57	12
UPLIFT	$(16.7)(20)(1)(.0624)+(1/2)(8.2)(20)(1)(.0624)$	-26		10.66	-277
IMPACT	$(0.8)(1)$		-.8	29.9	-24
$W_{WATER_{LAND}}$	$(3)(13.9)(1)(.0624)$	2.6		18.5	48
$W_{WATER_{RIVER}}$	$(3)(5.7)(1)(.0624)$	1.1		1.5	2
$GATE_L$	$P=0$				
	$T=0$				
	$S=0$				
	$M_{cv}=(74.6)(14+\frac{3}{2})/63.7= \quad 18$	–	–	–	18
	$V= \ 74.6/63.7=1.2$	1.2		18.5	22
$GATE_R$	$P=(1/2)(8.2)^2(1)(.0624)+(8.2)(5.7)(1)(.0624)$				
	$P=5$				
	$T=0.55(5)(56)/(.6428)= \quad 240$				
	$M_{cv}=-18$				-18
	$V=1.2$	1.2		1.5	2
	$S=(240)(.766)= \ 184$				
	$S'=184/63.7= \ 2.9$		2.9	22.25	64
	①	60.4	-0.6		611

CASE I: NORMAL OPERATIONS

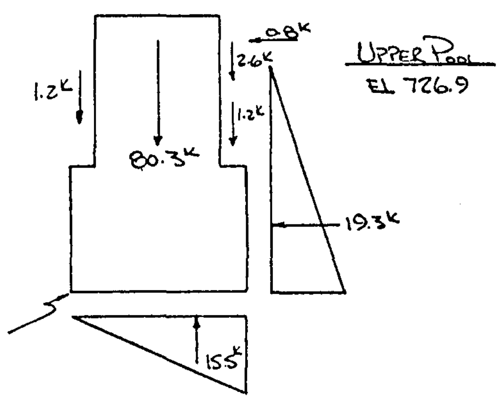
ASSUMPTIONS
UPPER POOL IN LAND LOCK (726.9)
LOWER POOL (718.7) IN RIVER LOCK

NEGLECTING IMPACT ② 60.4 0.2 635

$e_1 = 611/60.4 = 10.12$ FT

$e_2 = 635/60.4 = 10.51$ FT

FIGURE VI-10 MIDDLE WALL - UPPER GATE MONO. M-2 (CONTINUED)



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		80.3		10.0	803
P_{WATER}	$(726.9-702)^2(1/2)(1)(.0624)$		-19.3	8.3	-160
UPLIFT	$(1/2)(24.9)(1)(20)(.0624)$	-15.5		13.33	-207
IMPACT	$(0.8)(1)$		-0.8	29.9	-24
W_{WATER_L}	$(3)(13.9)(1)(.0624)$	2.6		18.5	48
$GATE_L$	P=0 T=0 S=0 M_{CV}= 1156/63.7 = 18 V= 74.6/63.7= 1.2	- 1.2	-	- 18.5	18 22
$GATE_R$	P=0 T=0 S=0 M_{CV}= -18 V= 1.2	- 1.2	-	- 1.5	-18 2
	①	69.8	-20.1		484
	NEGLECTING IMPACT ②	69.8	-19.3		508

CASE II: MAINTENANCE CONDITION

ASSUMPTIONS
DEWATER RIVER LOCK
UPPER POOL (726.9) IN LANDLOCK

$e_1 = 484/69.8 = 6.93$ FT

$e_2 = 508/69.8 = 7.28$ FT

FIGURE VI-10. MIDDLE WALL - UPPER GATE MONO. M-2 (CONTINUED)

SUBJECT: MIDDLE WALL-UPPER GATE MONO M-2

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

SLIDING

CASE I: NORMAL OPERATIONS

$R = \Sigma V \tan\phi$

$R = (60.4)(.63707) = 38.48$

$S_{S.F} = 38.48/-0.6 = -64.13$

NEGLECTING IMPACT $S_{S.F} = 38.48/0.2 = 192.4$

CASE II: MAINTENANCE CONDITION

$R = (69.8)(.63707) = 44.47$

$S_{S.F} = 44.47/-20.1 = -2.17$

NEGLECTING IMPACT $S_{S.F} = 44.47/-19.3 = -2.30$

FOUNDATION PRESSURE

CASE I: NORMAL OPERATIONS

$f = \frac{60.4}{20} + \frac{(7.2)(10)(12)}{(20)^3}$

$f = 3.13 \frac{K}{\square'}$

NEGLECTING IMPACT $f = \frac{60.4}{20} + \frac{(30.8)(10)(12)}{(20)^3} = 3.48 \frac{K}{\square'}$

CASE II: MAINTENANCE CONDITION

$f = \frac{69.8}{20} + \frac{(214)(10)(12)}{(20)^3} = 6.70 \frac{K}{\square'}$

NEGLECTING IMPACT $f = \frac{69.8}{20} + \frac{(190)(10)(12)}{(20)^3} = 6.34 \frac{K}{\square'}$

FIGURE VI-10 MIDDLE WALL - UPPER GATE MONO M-2 (CONTINUED)

SUBJECT: MIDDLE WALL - UPPER CHAMBER M6

COMPUTED BY:

CHECKED BY:

DATE:

DATE:

STATION: 1+46.5B TO 1+76.5B

LENGTH = 30 FT

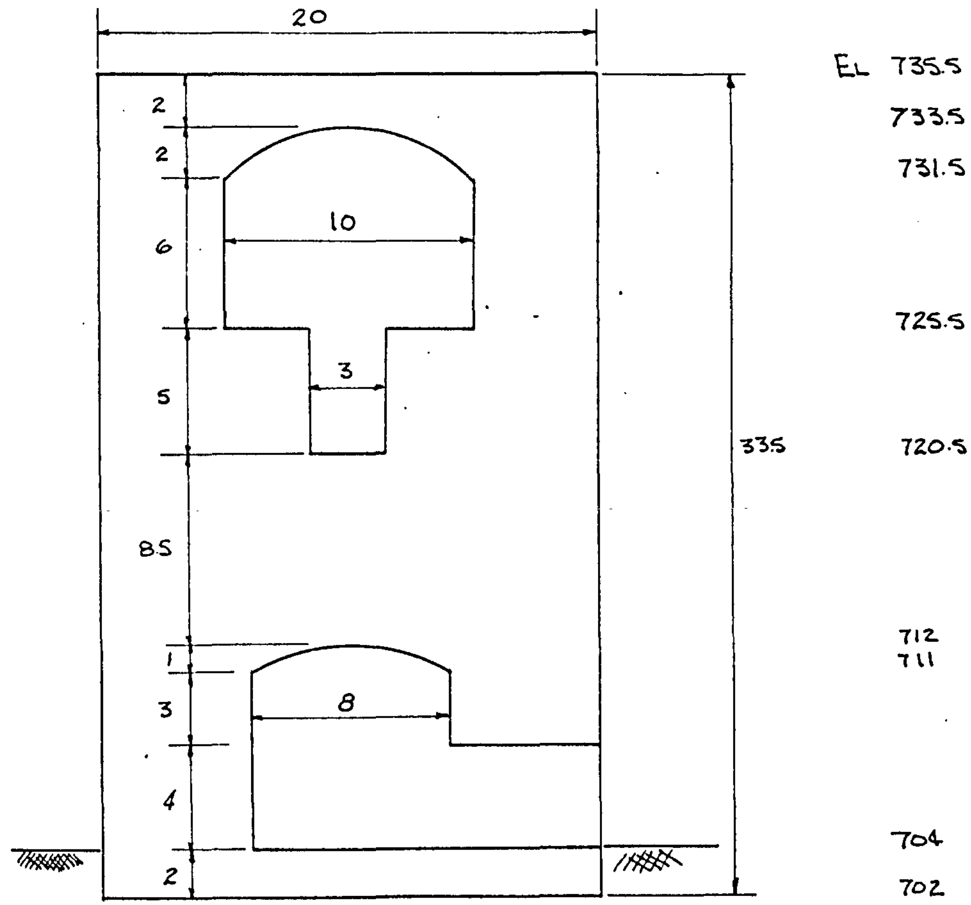


FIGURE VI-11 MIDDLE WALL - UPPER CHAMBER M-6

SCALE 1:50

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: Middle Wall - Upper Chamber M6

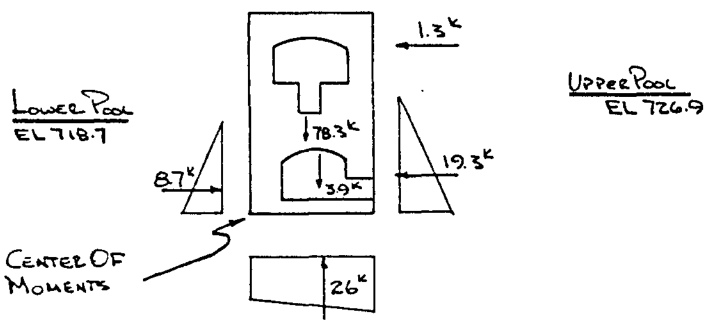
COMPUTED BY:

DATE:

CHECKED BY:

DATE:

Not To Scale



Item	Factors	Fv	FH	Arm	Moment
W_{conc}	[0.150][(20)(33.5)-(3)(5)-(10)(7)-(8)(7.5)-(4)(4)(6)/30][1]	78.3		9.96	780
P_{water}	$(726.9-702)^2(1)(.0624)(1/2)$		-19.3	8.3	-160
P_{water}	$(718.7-702)^2(1)(.0624)(1/2)$		8.7	5.57	48
Uplift	(16.7)(.0624)(1)(20)+(1/2)(8.2)(.0624)(20)(1)	-26		10.66	-279
W_{water}	[(4)(4)(6)][1/30][.0624][1] + (7.5)(8)(1)(.0624)	3.9		10.35	40
Impact	(40/30)(1)		-1.3	29.9	-39
		56.2	-11.9		390

Case I: Normal Operations

Assumptions
Lower Pool (718.7) in River Lock
Upper Pool (726.9) in Land Lock

$e = 390/56.2 = 6.94$ FT

FIGURE VI-11 MIDDLE WALL - UPPER CHAMBER M-6 (CONTINUED)

SUBJECT: MIDDLE WALL - UPPER CHAMBER M6

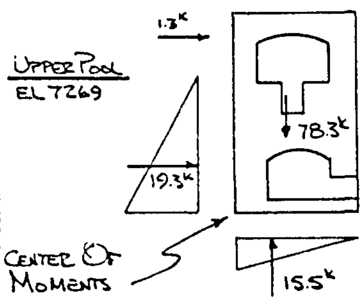
COMPUTED BY:

CHECKED BY:

DATE:

DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		78.3		9.96	780
P_{WATER}	$(1/2)(726.9-702)^2(1)(.0624)$		19.3	8.3	160
UPLIFT	$(1/2)(726.9-702)(20)(1)(.0624)$	-15.5		6.67	-103
IMPACT	$(40/30)(1)$		1.3	29.9	39
		62.8	20.6		876

$e = 876/62.8 = 13.95$ FT

CASE II: MAINTENANCE CONDITION

ASSUMPTIONS:
DEWATER LAND LOCK
UPPER POOL IN RIVER LOCK

FIGURE VI-11 MIDDLE WALL - UPPER CHAMBER M-6 (CONTINUED)

SUBJECT: MIDDLE WALL - UPPER CHAMBER M-6

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

CASE I: NORMAL OPERATIONS

$R = \Sigma V \tan\phi + \cancel{sA}^{\,0}$ Shear Resistance of Concrete Section

$R = (56.2)(.63707) + (.075)(12)(144)$

$R = 35.80 + 129.6 = 165.4$

$S_{S.F} = 165.4 / -11.9 = -13.9$

CASE II: MAINTENANCE CONDITION

$R = (62.8)(.63707) + 120.96$

$R = 40.00 + 120.96 = 160.96$

$S_{S.F} = 160.96 / 20.6 = 7.81$

FOUNDATION PRESSURE

CASE I: NORMAL OPERATIONS

$$f = \frac{56.2}{20} + \frac{(10-6.94)(56.2)(10)(12)}{20^3}$$

$$f = 2.81 + 2.58 = 5.39 \ \frac{K}{\square'}$$

CASE II: MAINTENANCE CONDITION

$$f = \frac{2V}{3e} = \frac{2(62.8)}{3(6.05)} = 6.92 \text{ KSF}$$

FIGURE VI-11 MIDDLE WALL - UPPER CHAMBER M-6 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1966

SUBJECT: MIDDLE WALL - INTERMEDIATE GATE MONO M-15

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

STATION: 4+09.2B TO 4+44.6B

LENGTH = 35.4 FT

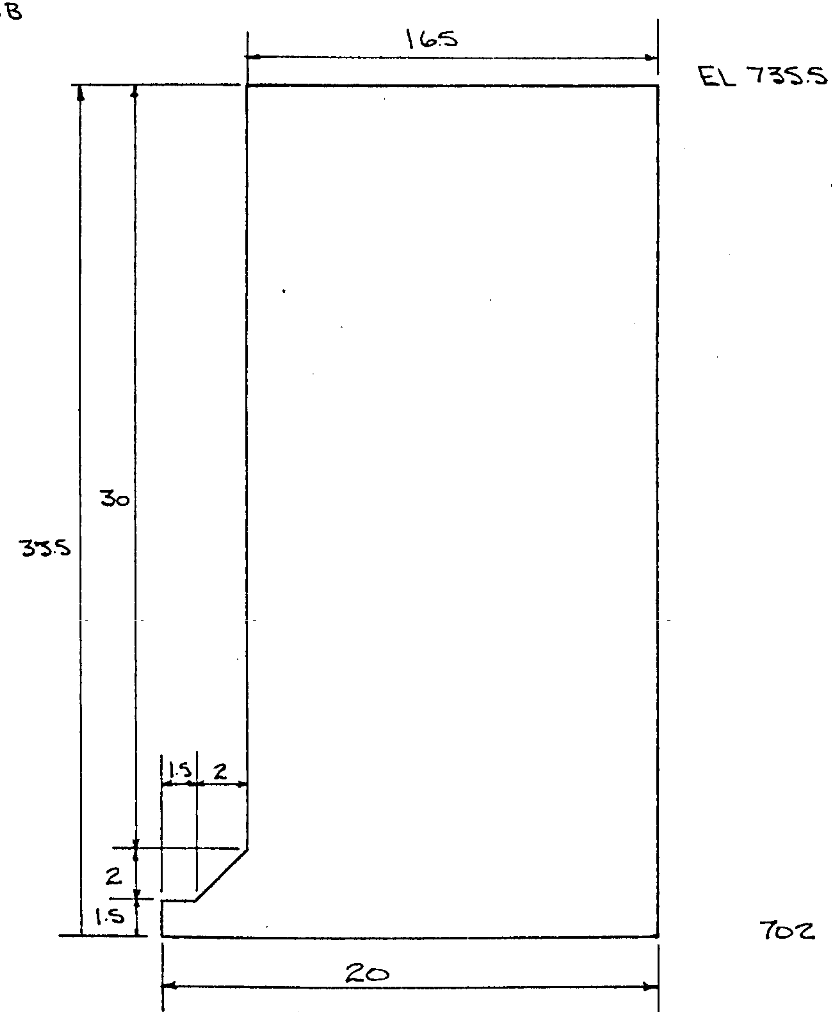


FIGURE VI-12 MIDDLE WALL - INTERMEDIATE GATE MONO. M-15

SCALE 1:50

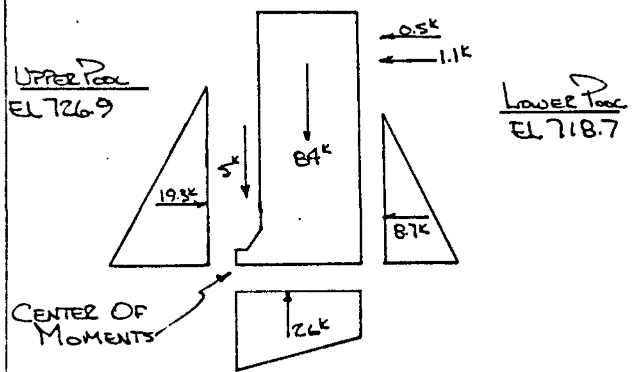
WES FORM NO. 1253A

SUBJECT: MIDDLE WALL- INTERMEDIATE GATE MONO M-15

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

NOT TO SCALE



CASE I: NORMAL OPERATIONS

ASSUMPTIONS
UPPER POOL (726.9) IN RIVER LOCK
LOWER POOL (718.7) IN LAND LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{conc}	$[0.150][(16.5)(33.5)+(3.5)(1.5)+(1/2)(2)(2)][1]$	84		11.62	976
P_{water}	$(1/2)(726.9-702)^2(.0624)(1)$		19.3	8.3	160
P_{water}	$(1/2)(718.7-702)^2(.0624)(1)$		-8.7	5.57	-48
UPLIFT	$(16.7)(20)(1)(.0624)+(1/2)(8.2)(20)(1)(.0624)$	-26		9.33	-243
IMPACT	$(40/35.4)(1)$		-1.1	21.7	-24
W_{water}	$(726.9-703.5)(1)(3.5)(.0624)$	5		1.75	9
P_{wind}	$(0.03)(735.5-718.7)(1)$		-.5	25.1	-13
	①	63	9		817
	Neglecting Impact ②	63	10.1		841

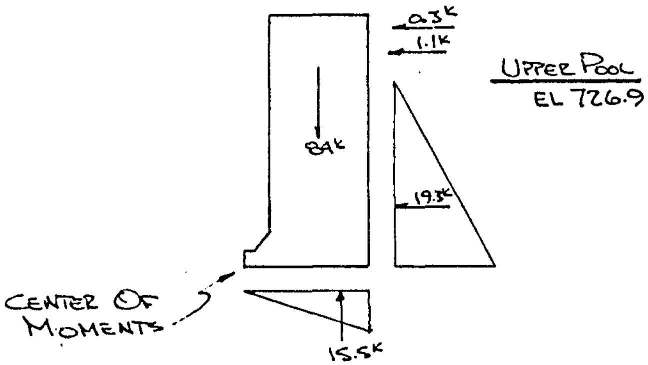
$e_1 = 817/63 = 12.96'$

$e_2 = 841/63 = 13.35'$

FIGURE VI-12, MIDDLE WALL - INTERMEDIATE GATE MONO. M-15 (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - INTERMEDIATE GATE MONO M-15	CHECKED BY:	DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		84		11.62	976
P_{WATER}	$(1/2)(726.9-702)^2(.0624)(1)$		-19.3	8.3	-160
UPLIFT	$(1/2)(24.9)(20)(.0624)(1)$	-15.5		13.33	-207
IMPACT	$(40/35.4)(1)$		-1.1	29.9	-33
P_{WIND}	$(0.03)(735.5-726.9)(1)$		-.3	29.2	-9
		68.5	-20.7		567

$$e = 567/68.5 = 8.28'$$

CASE II: MAINTENANCE CONDITION

ASSUMPTIONS
DEWATERED RIVER LOCK
UPPER POOL (726.9) IN LANDLOCK

FIGURE VI-12 MIDDLE WALL - INTERMEDIATE GATE MONO. M-15 (CONTINUED)

ORD FORM NO. 1253A

SUBJECT: MIDDLE WALL - INTERMEDIATE GATE MONO M-15

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

SLIDING

CASE I: NORMAL OPERATIONS

$R = (63)(.63707) = 40.14$

$S_{S.F} = 40.14/9 = 4.46$

NEGLECTING IMPACT $S_{S.F} = 40.14/10.1 = 3.97$

CASE II: MAINTENANCE CONDITION

$R = (68.5)(.63707) = 43.64$

$S_{S.F} = 43.64/-20.7 = -2.11$

FOUNDATION PRESSURE

CASE I: NORMAL OPERATIONS

$$f = \frac{63}{20} + \frac{(2.96)(63)(7.04)(12)}{(1)(20)^3}$$

$$= 3.15 + 1.97 = 5.12 \frac{k}{\square'}$$

NEGLECTING IMPACT $f = \frac{2(63)}{3\ 6.65} = 6.32 \frac{k}{\square'}$

CASE II: MAINTENANCE CONDITION

$$f = \frac{68.5}{20} + \frac{(68.5)(1.72)(8.28)(12)}{(1)(20)^3}$$

$$= 3.43 + 1.46 = 4.89 \frac{k}{\square'}$$

FIGURE VI-12 MIDDLE WALL - INTERMEDIATE GATE MONO M-15 (CONTINUED)

VI-64

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - INTERMEDIATE GATE MONO M-15	CHECKED BY:	DATE:

SLIDING:

THERE IS A COAL-SHALE INTERFACE 3' BELOW THIS MONOLITH. ASSUME LOWER BOUND VALUES OF $\phi = 15°$ AND A SHEAR STRENGTH OF THE SHALE AS 112 PSI. THE SLIDING IS THEN CALCULATED FOR THE VARIOUS CASE LOADINGS.

CASE I: NORMAL OPERATIONS

$R = (63)\tan 15° + (2)(0.112\ K/IN^2)(144\ IN^2/FT^2)(3\ FT)$

$R = 16.88 + 96.8$

$R = 113.68$

IT CAN BE SEEN FROM FIGURE 6.18 THAT THIS INTERFACE WILL NOT BE CRITICAL FOR SLIDING.

FIG VI-12 MIDDLE WALL - INTERMEDIATE GATE MONO M-15 (CONTINUED)

SUBJECT:

MIDDLE WALL - LOWER CHAMBER M20

COMPUTED BY:

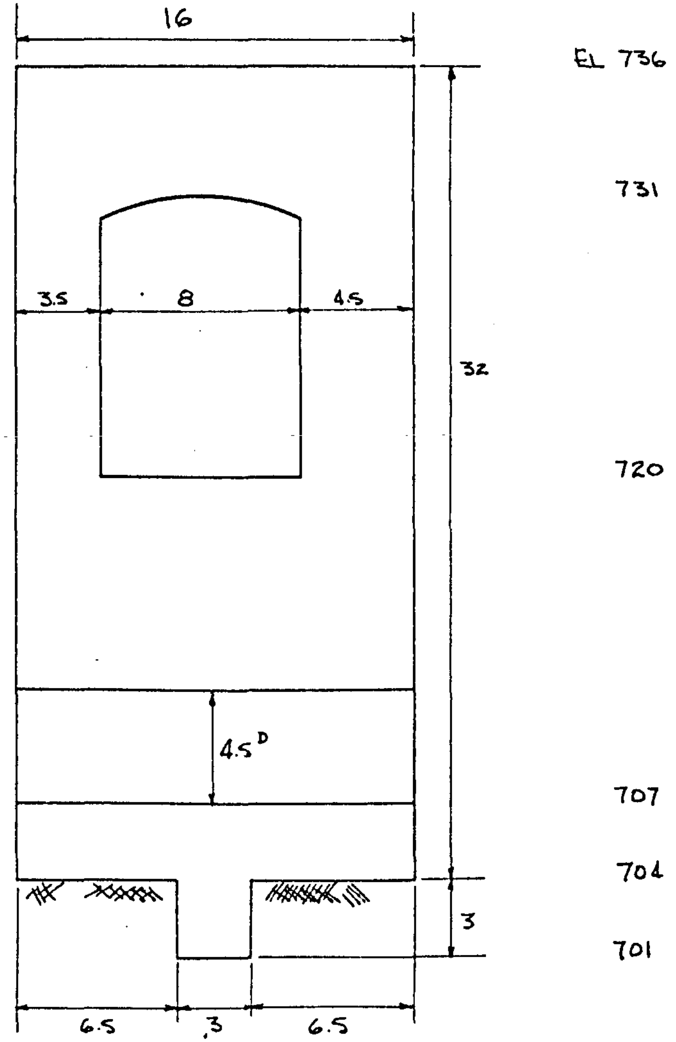
DATE:

CHECKED BY:

DATE:

STATION: 5+90.18 TO 6+33.38

LENGTH = 43.2 FT



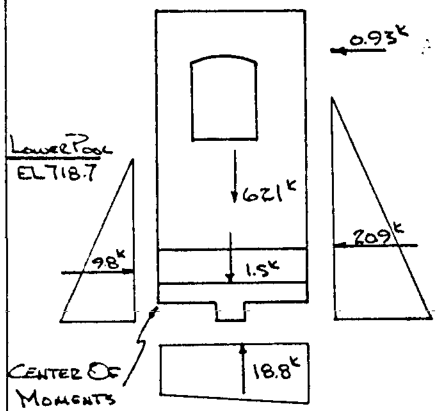
SCALE 1:50

FIGURE VI-13 MIDDLE WALL - LOWER CHAMBER M-20

WES FORM NO. 1253A

SUBJECT:	COMPUTED BY:	DATE:
MIDDLE WALL - LOWER CHAMBER M-20	CHECKED BY:	DATE:

NOT TO SCALE



UPPER POOL
EL 726.9

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[0.150][(16)(32)+(3)(3)-(10.46)(8) - (4)(\pi)(4.5)^2(16)/(43.2)][1]$	62.1		8.1	503
P_{WATER}	$(718.7-701)^2(1)(.0624)(1/2)$		9.8	2.9	28
P_{WATER}	$(726.9-701)^2(1)(.0624)(1/2)$		-20.9	5.63	-118
UPLIFT	$(16)(.0624)(1)(14.7)+(1/2)(8.2)(.0624)(16)(1)$	-18.8		8.58	-161
W_{WATER}	$[(4)(\pi)(\frac{4.5}{2})^2(16)/(43.2)][.0624]$	1.5		8	12
IMPACT	$(.93)(1)(1)$		-.93	28.9	-27
		44.8	-12		237

CASE I: NORMAL OPERATIONS

ASSUMPTIONS
LOWER POOL EL 718.7
UPPER POOL EL 726.9
UPPER TUNNEL EMPTY
LOWER TUNNEL FULL

$e = 237/44.8 = 5.29$ FT

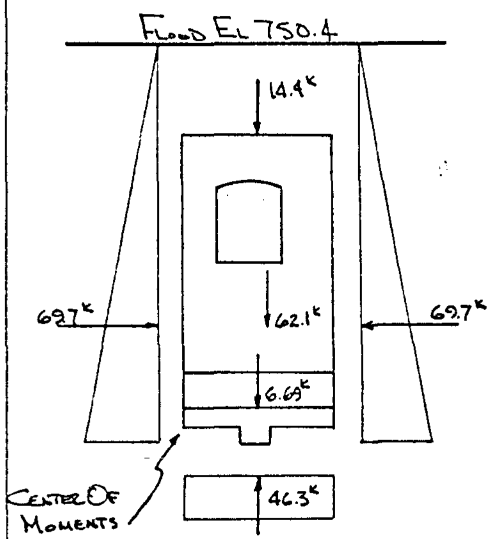
FIGURE VI-13 MIDDLE WALL - LOWER CHAMBER M-20 (CONTINUED)

SUBJECT: MIDDLE WALL - LOWER CHAMBER M 20

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		62.1		8.1	503
P_{WATER}	$(750.4-736)(.0624)(736-701)(1) + (736-701)^2(.0624)(1)(1/2)$		69.7	11.98	835
P_{WATER}			-69.7	11.98	-835
UPLIFT	$(750.4-704)(16)(.0624)(1)$	-46.3		8	-370
W_{WATER}	$(750.4-736)(16)(.0624)(1)$	14.4		8	115
W_{WATER}	$[(10.46)(8)+(4)(\pi)(\frac{4.5}{2})^2(16)/43.21][.0624]$	6.69		7.76	52
		36.9	0		300

$e = 300/36.9 = 8.13$ FT

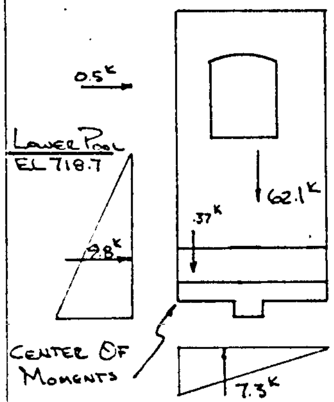
CASE II: FLOOD CONDITIONS

ASSUMPTIONS
FLOOD EL 750.4

FIGURE VI-13 MIDDLE WALL - LOWER CHAMBER M-20 (CONTINUED)

SUBJECT: MIDDLE WALL - LOWER CHAMBER M20	COMPUTED BY:	DATE:
	CHECKED BY:	DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		62.1		8.1	503
P_{WATER}	$(\frac{1}{2})(718.7-701)^2(1)(.0624)$		9.8	2.9	28
UPLIFT	$(16)(.0624)(\frac{1}{2})(14.7)$	-7.3		5.3	-39
P_{WIND}	$(17.3)(1)(.03)$		0.5	23.35	12
W_{WATER}	$[(4)(\pi)(\frac{4.5}{2})^2(16/4)/(43.2)][.0624]$	0.37		2.0	1
		55.2	10.3		505

$e = 505/55.2 = 9.15$ FT

CASE III: MAINTENANCE CONDITION

ASSUMPTIONS:

LOCK DEWATERED
LOWER POOL EL 718.7
WIND ACTING ON RIVER FACE
WATER IN LOWER TUNNEL TO VALVE

FIGURE VI-13 MIDDLE WALL - LOWER CHAMBER M-20 (CONTINUED)

SUBJECT: MIDDLE WALL - LOWER CHAMBER M-20

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

SLIDING

CASE I: NORMAL OPERATIONS

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

$R = (44.8)(.63707) + (.075)(144)(3)$

$R = 28.5 + 32.4 = 60.9^K$

$S_{SF} = 60.9/-12 = -5.1$

CASE II: FLOOD CONDITIONS

$R = (36.9)(.63707) + 32.4$

$R = 23.5 + 32.4 = 55.9^K$

$S_{SF} = 55.9/0 = \infty$

CASE III: MAINTENANCE CONDITION

$R = (55.2)(.63707) + 32.4$

$R = 35.2 + 32.4 = 67.6^K$

$S_{SF} = 67.6/10.3 = 6.56$

FOUNDATION PRESSURE

CASE I: NORMAL OPERATIONS

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

$f = \left(\frac{2}{3}\right)\left(\frac{44.8}{5.29}\right) = 5.65\ K/FT^2$

CASE II: FLOOD CONDITIONS

$f = \frac{36.9}{16} + \frac{(36.9)(0.13)(8)(12)}{16^3}$

$= 2.42\ \frac{K}{\square'}$

CASE III: MAINTENANCE CONDITION

$f = \frac{55.2}{16} + \frac{(55.2)(1.15)(8)(12)}{16^3}$

$= 4.94\ \frac{K}{\square'}$

FIGURE VI-13 MIDDLE WALL - LOWER CHAMBER M-20 (CONTINUED)

SUBJECT

LOWER GATE MONOLITH M-25

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

STATION 7+86.7B to 8+27 B
LENGTH 40.3 FT

MITER GATE AT STATION 8+05.9B

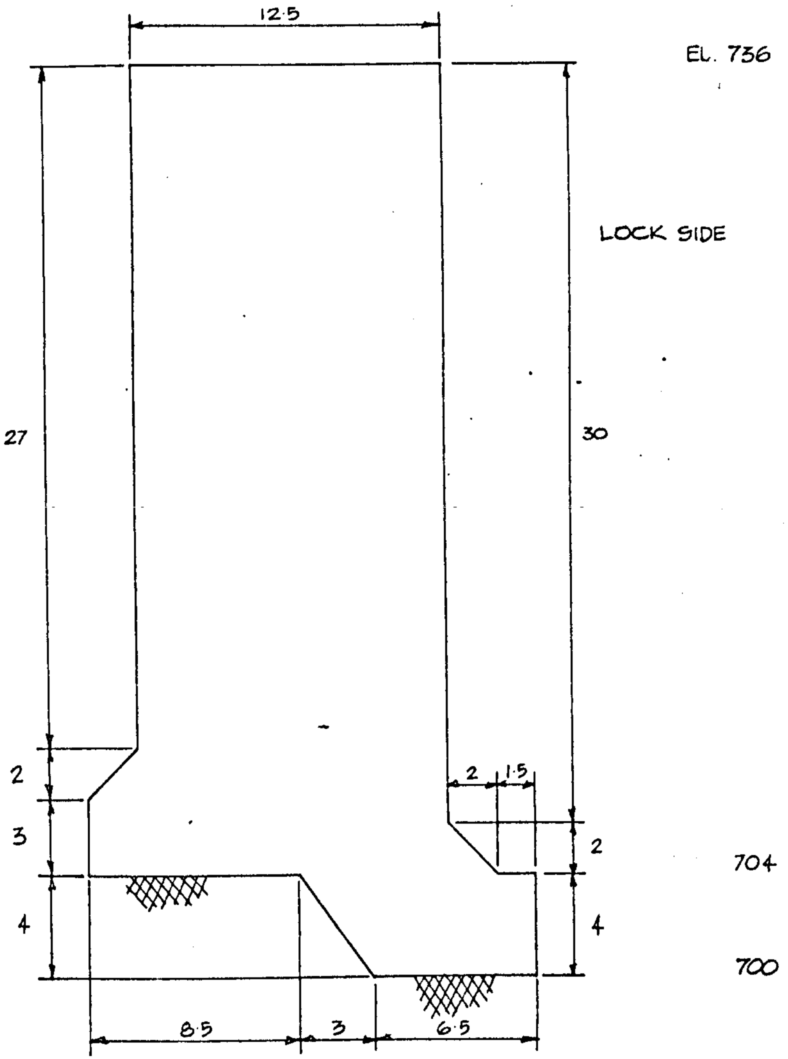
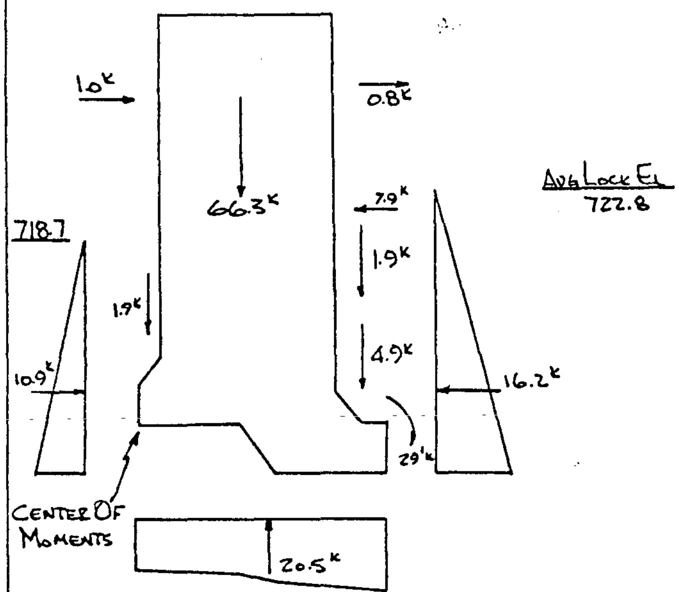


FIGURE VI-14 MIDDLE WALL - LOWER GATE MONO. M-25

WES FORM NO. 1253A DECEMBER 1964

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

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[.150][(12.5)(32)+(6.5)(4)+(2)(1/2)(2)(2) + (3)(2)][1]$	66.3		8.56	568
P_{WATER}	$(718.7-700)^2(.0624)(1/2)(1)$		10.9	2.23	24
P_{WATER}	$(722.8-700)^2(1/2)(.0624)(1)$		-16.2	3.59	-58
W_{WATER}	$(718.7-708)(2.8)(1)(.0624)$	1.9		1.0	2
W_{WATER}	$(722.8-705)(2.8)(1)(.0624) + (722.8-704)(1.5)(1)(.0624)$	4.9		16.25	79
HAWSER	$(0.8)(1)$		0.8	23.8	19
UPLIFT	$(718.7-704)(18)(1)(.0624)+(722.8-714.7)(8)(.0624)(1)$	-20.5		10.2	-209
IMPACT	$(40/40.3)(1)$		1.0	19.7	20
GATE	$P=(8.2)^2(.0624)(1/2)(1)+(8.2)(.0624)(718.7-706)(1) = 8.6$ K/FT^2				
	$T=(0.55)(8.6)(56)/(.6428) = 413.9^K$				
	$S=(413.9)(.766) = 318.7^K$				
	$S' = 318.7/40.3 = 7.9^K$		-7.9	16.0	-126
	$V' = 74.6/40.3 = 1.85^K$	1.9		16.25	31
	$M'_{CV} = (1.85)(15.75) = 29.2$ FT-KIPS	—	—	—	29
	①	54.5	-11.4		379

NEGLECTING IMPACT ② 54.5 -12.4 359

$e_1 = 379/54.5 = 6.95'$

$e_2 = 359/54.5 = 6.59'$

CASE I: NORMAL OPERATIONS

ASSUMPTIONS:
LOWER POOL EL 718.7
PERCENT AVERAGE EL IN LOCK 722.8

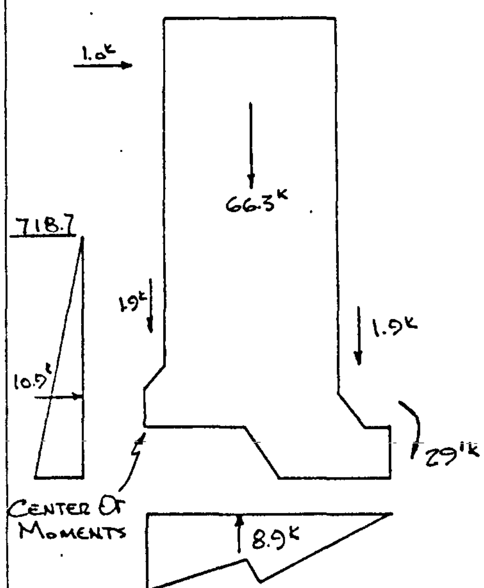
FIGURE VI-14 MIDDLE WALL - LOWER GATE MONO. M-25 (CONTINUED)

SUBJECT: MIDDLE WALL - LOWER GATE MONO - M-25

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		66.3		8.56	568
P_{WATER}			10.9	2.23	24
W_{WATER}		1.9		1.0	2
IMPACT			1.0	19.7	20
UPLIFT	$[(18.7)(1/2)(1)(10)+(12.3)(1/2)(1)(8)][.0624]$	−8.9		6.55	−58
GATE	P=0 T=0 S=0 $V' = 74.6/40.3 = 1.85$ $M_{CV} = 29.2$	 1.9 –	 –	 16.25 –	 31 29
	①	61.2	11.9		616
	NEGLECTING IMPACT ②	61.2	10.9		596

$e_1 = 616/61.2 = 10.07'$

$e_2 = 596/61.2 = 9.74'$

CASE II: MAINTENANCE CONDITION

ASSUMPTIONS
DEWATER LOCK
LOWER POOL EL 718.7

FIGURE VI-14 MIDDLE WALL - LOWER GATE MONO. M-25 (CONTINUED)

WES FORM NO. 1253A

SUBJECT: MIDDLE WALL - LOWER GATE MONO M-25

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

SLIDING

CASE I: NORMAL OPERATIONS

$R = \Sigma V \tan\phi + \overset{0}{\cancel{sA}} + \text{KEY RESISTANCE}$

$R = (54.5)(.63707) + (.075)(6.32)(144)(1)$

$R = 34.72 + 68.2 = 102.9^K$

$S_{S.F} = 102.9/-11.4 = -9.03$

NEGLECTING IMPACT $S_{S.F} = 102.9/-12.4 = -8.30$

CASE II: MAINTENANCE CONDITION

$R = (61.2)(.63707) + 68.2$

$R = 107.2^K$

$S_{S.F} = 107.2/11.9 = 9.00$

NEGLECTING IMPACT $S_{S.F} = 107.2/10.9 = 9.83$

FOUNDATION PRESSURE

CASE I: NORMAL OPERATIONS

$f = \frac{54.5}{18} + \frac{(54.5)(2.05)(9)(12)}{18^3} = 5.10 \frac{K}{\square'}$

$f =$

NEGLECTING IMPACT $f = \frac{54.5}{18} + \frac{(54.5)(2.41)(9)(12)}{18^3} = 5.46 \frac{K}{\square'}$

CASE II: MAINTENANCE CONDITION

$f = \frac{61.2}{18} + \frac{(61.2)(0.07)(9)(12)}{18^3} = 3.48 \frac{K}{\square'}$

NEGLECTING IMPACT $f = \frac{61.2}{18} + \frac{(61.2)(0.74)(9)(12)}{18^3} = 4.24 \frac{K}{\square'}$

FIGURE VI-14 MIDDLE WALL - LOWER GATE MONO M-25 (CONTINUED)

SUBJECT: Upper Guard Wall - R-2 | COMPUTED BY: | DATE:
CHECKED BY: | DATE:

Station: 2+58.6A to 3+09.9A

Length = 51.3 FT

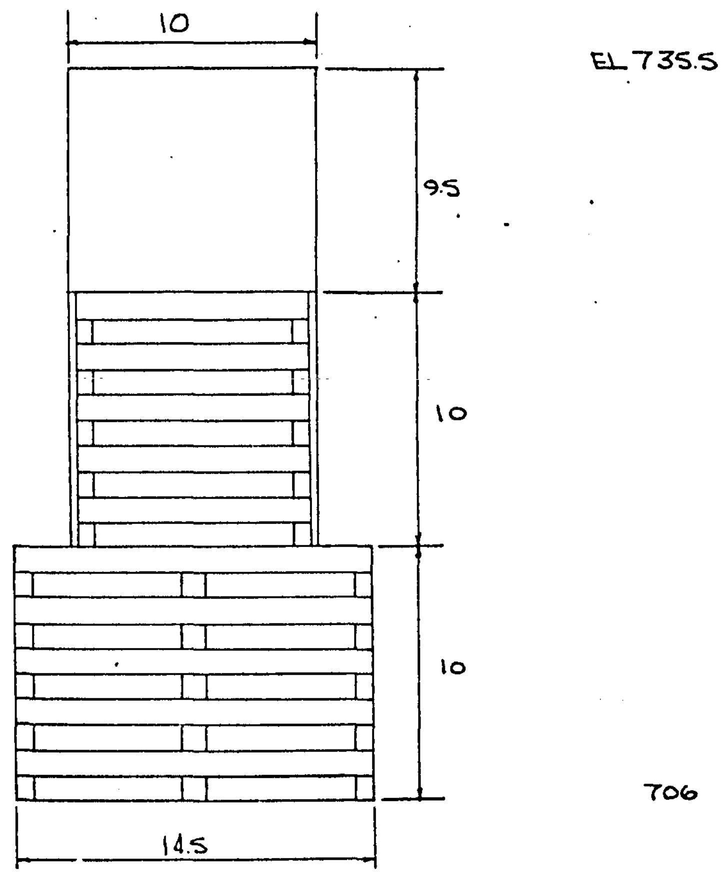


FIGURE VI-15 RIVER WALL - UPPER GUARD WALL R-2

Scale = 1:50

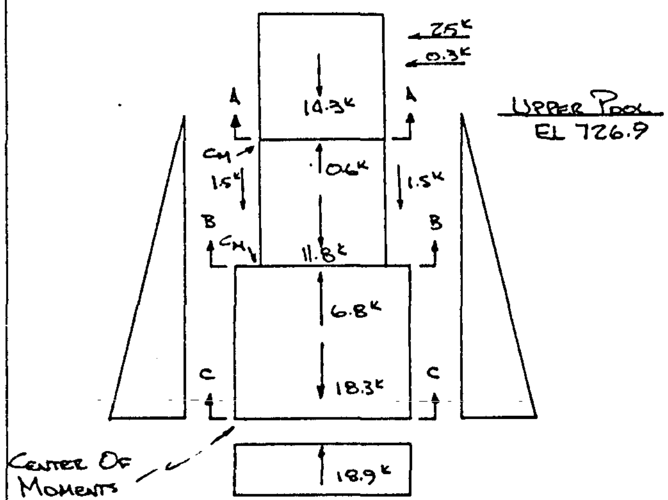
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: UPPER GUARD WALL - R2

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

NOT TO SCALE



CASE I: NORMAL OPERATIONS

$e_1 = 136/28.5 = 4.77'$
$e_2 = 201/28.5 = 7.05'$

ITEM	FACTORS	Fv	FH	ARM	MOMENT
	SECTION A-A:				
W_{CONC}	(.150)(9.5)(1)(10)	14.3		5	72
UPLIFT	(726.9-726)(10)(.0624)(1)	-0.6		5	-3
IMPACT	(2.5)(1)		-2.5	5.9	-15
P_{WIND}	(0.03)(735.5-726.9)(1)		-0.3	5.2	-2
	①	13.7	-2.8		52
	NEGLECTING IMPACT ②	13.7	-0.3		67
	$e_1 = 52/13.7 = 3.80'$ $e_2 = 67/13.7 = 4.89'$				
	SECTION B-B				
W_{CONC}		14.3		5	72
W_{WOOD_1}	[(5)(6)(10)(12)(9.33)/(144)+(2)(5)(10)(12)(51.3)/(144) + (2)(4/12)(10)(51.3)][.110/51.3]	2.2		5	11
W_{ROCK_1}	[10-(2)(10/12)-(2)(4/12)][10][.125](1)	9.6		5	48
UPLIFT	(726.9-716)(.0624)(1)(10)	-6.8		5	-34
IMPACT	(2.5)(1)		-2.5	15.9	-40
P_{WIND}	(0.03)(735.5-726.9)(1)		-0.3	15.2	-5
	①	19.3	-2.8		52
	NEGLECTING IMPACT ②	19.3	-0.3		92
	$e_1 = 52/19.3 = 2.69'$ $e_2 = 92/19.3 = 4.77'$				
	SECTION C-C:				
W_{CONC}		14.3		7.25	104
W_{WOOD_1}		2.2		7.25	16
W_{ROCK_1}		9.6		7.25	70
W_{WOOD_2}	[(5)(6)(10)(12)(14.5)/(144)+((3)(5)(12)(10)/144)((51.3)+(2)(5)(5)(10)(12)/144)(51.3-6(10/12))/5 [.110/51.3]	2.3		7.25	17
W_{ROCK_2}	((14.5-2(10/12))(10)(1)(.125)	16		7.25	116
W_{WATER}	(2.25)(1)(.0624)(726.9-716)	1.5		1.12	2
W_{WATER}		1.5		13.67	21
IMPACT	(2.5)(1)		-2.5	25.9	-65
UPLIFT	(726.9-706)(.0624)(14.5)(1)	-18.9		7.25	-137
P_{WIND}	(0.03)(735.5-726.9)(1)		-0.3	25.2	-8
	①	28.5	-2.8		136
	NEGLECTING IMPACT ②	28.5	-0.3		201

1253A

FIGURE VI-15 RIVER WALL - UPPER GUARD WALL R-2 (CONTINUED)

SUBJECT: Upper Guard Wall - R2

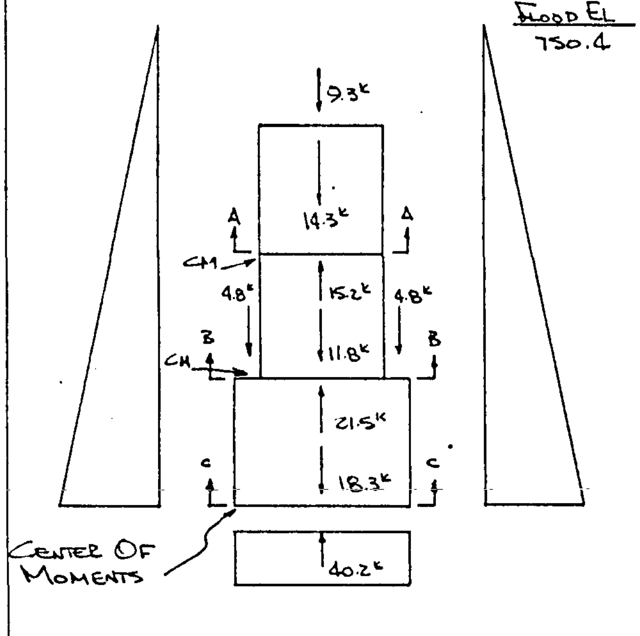
COMPUTED BY:

DATE:

CHECKED BY:

DATE:

NOT TO SCALE



CASE II: FLOOD CONDITION

Item	Factors	Fv	Fh	Arm	Moment
	Section A-A:				
Wconc		14.3		5	72
Wwater	(750.4-735.5)(.0624)(10)(1)	9.3		5	46
Uplift	(750.4-726)(.0624)(10)(1)	-15.2		5	-76
		8.4	0		42
	e = 42/8.4 = 5.0'				
	Section B-B:				
Wconc		14.3		5	72
Wwood1		2.2		5	11
Wrock1		9.6		5	48
Wwater		9.3		5	46
Uplift	(750.4-716)(.0624)(10)(1)	-21.5		5	-108
		13.9	0		69
	e = 7.25'				
	Section C-C:				
Wconc		14.3		7.25	104
Wwood1		2.2		7.25	16
Wrock1		9.6		7.25	70
Wwood2		2.3		7.25	17
Wrock2		16		7.25	116
Wwater		9.3		7.25	67
Wwater	(2.25)(1)(.0624)(750.4-716)	4.8		1.12	5
Uplift	(750.4-706)(.0624)(1)(14.5)	-40.2		7.25	-291
Wwater		4.8		13.67	66
		23.1	0		170

e = 7.25'

FIGURE VI-15 RIVER WALL - UPPER GUARD WALL R-2 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: Upper Guard Wall - R2 | COMPUTED BY: | DATE: | CHECKED BY: | DATE:

Sliding

Case I: Normal Operations

(Section A-A)

$R = \Sigma V \tan \phi$

$R = (13.7)(0.60\ ) = 8.22^k$

$S_{S\text{-}F} = 8.22/-2.8 = -2.94$

Neglecting Impact $S_{S\text{-}F} = 8.22/-0.3 = -27.4$

(Section B-B)

$R = (19.3)(0.543\ .) = 10.48^k$

$S_{S\text{-}F} = 10.48/-2.8 = -3.74$

Neglecting Impact $S_{S\text{-}F} = 10.48/-0.3 = -34.93$

(Section C-C)

$R = (28.5)(0.60\ .) = 17.1^k$

$S_{S\text{-}F} = 17.1/-2.8 = -6.11$

Neglecting Impact $S_{S\text{-}F} = 17.1/-0.3 = 57.0$

Case II: Flood Condition

(Section A-A)

$R = (8.4)(0.60\ .) = 5.04^k$

$S_{S\text{-}F} = 5.04/0 = \infty$

(Section B-B)

$R = (13.9)(0.543\ ) = 7.55^k$

$S_{S\text{-}F} = 7.55/0 = \infty$

(Section C-C)

$R = (23.1)(0.60\ ) = 13.86^k$

$S_{SF} = 13.86/0 = \infty$

Foundation Pressure

Case I: Normal Operations

(Section A-A)

$f = \frac{13.7}{0.9746} + \frac{(13.7)(5-3.8)(5)}{8.122} = 24.18\ ^k/FT^2$

Neglecting Impact $f = \frac{13.7}{0.9746} + \frac{(13.7)(5-4.89)(5)}{8.122} = 14.99\ ^k/FT^2$

(Section B-B)

$f = \frac{19.3}{0.2274} + \frac{(19.3)(5-2.69)(5)}{0.2607} = 939.93\ ^k/FT^2$

Neglecting Impact $f = \frac{19.3}{0.2274} + \frac{(19.3)(5-4.77)(5)}{0.2607} = 170.01\ ^k/FT^2$

(Section C-C)

$f = \frac{28.5}{1.6666} + \frac{(28.5)(7.25-4.77)(7.25)}{77.92} = 23.68\ ^k/FT^2$

Neglecting Impact $f = \frac{28.5}{1.6666} + \frac{(28.5)(7.25-7.06)(7.25)}{77.92} = 17.63\ ^k/FT^2$

Case II: Flood Conditions

(Section A-A)

$f = \frac{8.4}{.9746} = 8.62\ ^k/FT^2$

(Section B-B)

$f = \frac{13.9}{0.2274} = 61.13\ ^k/FT^2$

(Section C-C)

$f = \frac{23.1}{1.6666} = 13.86\ ^k/FT^2$

Figure VI-15 River Wall - Upper Guard Wall R-2 (continued)

SUBJECT:	COMPUTED BY:	DATE:
River Wall - Upper Gate Mono R-12	CHECKED BY:	DATE:

Station: 0+00 to 0+56.2 B
Length = 56.2 FT

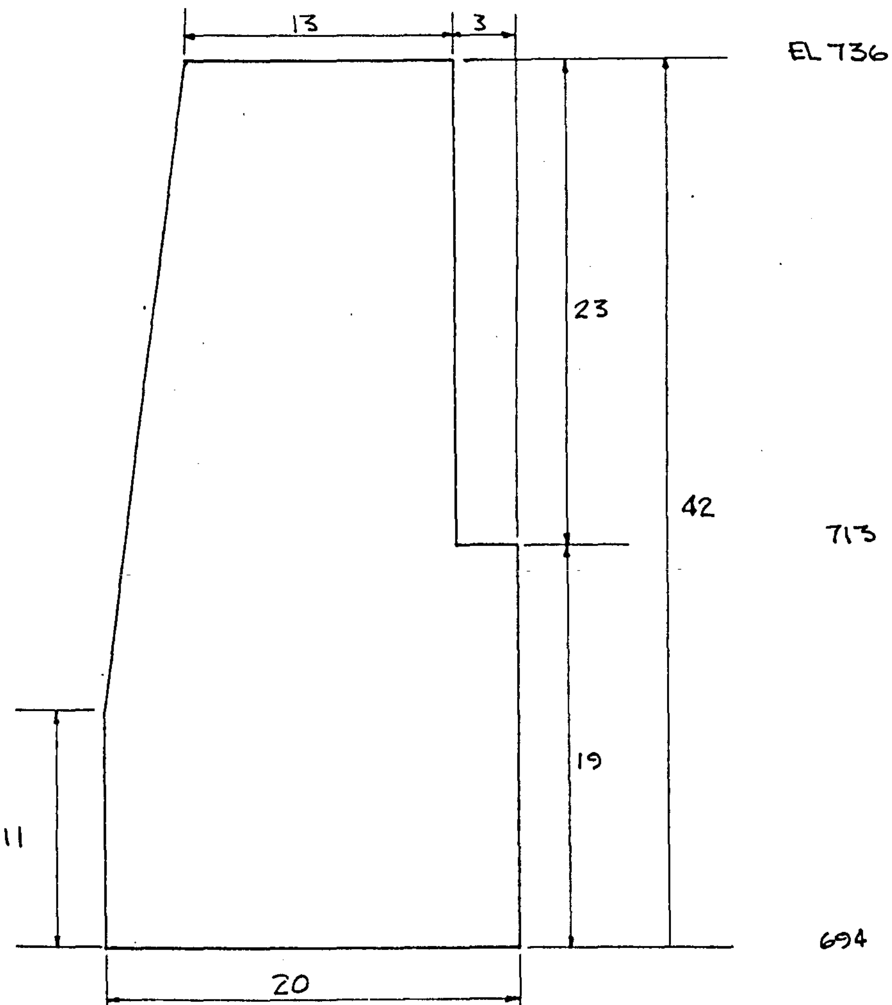


FIGURE VI-16 RIVER WALL - UPPER GATE MONO. R-12

Scale 1:60

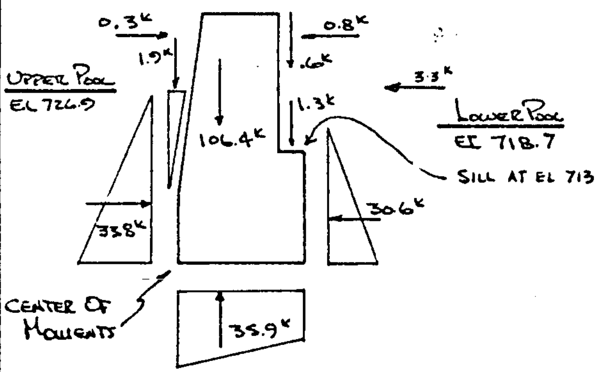
WES FORM NO. 1253A
DECEMBER 1964

SUBJECT: River Wall - Upper Gate Mono R-12

COMPUTED BY: DATE:

CHECKED BY: DATE:

Not to Scale



Item	Factors	Fv	FH	Arm	Moment
W_{conc}	[0.150][(13)(23)+(1/2)(4)(31) +(13)(8)+(3)(19)+(11)(17)][1]	106.4		9.93	1057
P_{water}	$(1/2)(726.9-694)^2(1)(.0624)$		33.8	10.97	371
P_{water}	$(1/2)(726.9-694)^2(.0624)(44/56.2)$ $+(1/2)(718.7-694)^2(.0624)(12.2)/(56.2)$		-30.6	10.59	-324
P_{wind}	(0.03)(736-726.9)(1)		.3	37.45	11
Uplift	(24.7)(20)(1)(.0624)+(1/2)(8.2)(20) (1)(.0624)	-35.9		9.53	-342
W_{water}	(21.9)(1)(1/2)(.0624)(.1290)(21.9)	1.9		0.94	2
W_{water}	(3)(1)(.0624)(718.7-713)(34/56.2)	0.6		18.5	11
Impact	(40/56.2) Use 0.8		-0.8	29.7	-24
Gate	$P=(1/2)(8.2)^2(1)(.0624)+(8.2)(5.7)(1)(.0624)=5^k$				
	T = .055 PW/Sin 2α				
	$= .055(5)(56)/\sin 40 = 239^k$				
	c = 56/4 + 3/2 = 15.5				
	$V = 74.6^k$				
	cV = (74.6)(15.5) = 1156 FT-KIPS				
	M_{cv} = 1156/56.2 = 20.6 FT-KIPS	–	–	–	21
	$V' = 74.6/56.2 = 1.3^k$	1.3		18.5	24
	$S = 239 \cos 40° = 183^k$				
	$S' = 183/56.2 = 3.3^k$		-3.3	30.5	-101
	①	74.3	-0.6		706
	Neglecting Impact ②	74.3	+0.2		730

Case I: Normal Operations

Assumptions:
Lower Pool (718.7) in Lock
Upper Pool EL 726.9

$e_1 = 706/74.3 = 9.50$ FT

$e_2 = 730/74.3 = 9.83$

FIGURE VI-16 RIVER WALL - UPPER GATE MONO. R-12 (CONTINUED)

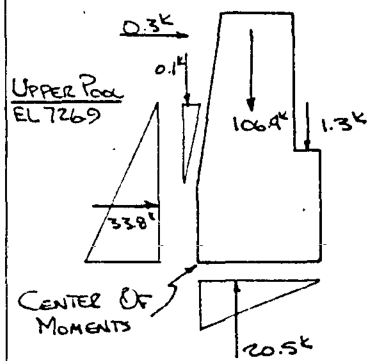
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL - UPPER GATE MONO R-12

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		106.4		9.93	1057
P_{WATER}			33.8	10.97	371
P_{WIND}			0.3	37.45	11
UPLIFT	(½)(726.9-694) (20)(1)(.0624)	-20.5		6.67	-137
W_{WATER}		1.9		0.94	2
GATE	V	1.3		18.5	24
	M_{cV}	—	—	—	21
		89.1	34.1		1349

$e = 1349/89.1 = 15.14'$

CASE II: MAINTENANCE CONDITION

ASSUMPTIONS
DEWATER RIVER LOCK
UPPER POOL EL 726.9

FIGURE VI-16 RIVER WALL - UPPER GATE MONO. R-12 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: River Wall - Upper Gate Mono R-12

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

Sliding

Case I: Normal Operations

$R = \Sigma V \tan\phi$

$R = (74.3)(0.26795) = 19.91^{K}$

$S_{S.F} = 19.91 / 0.6 = 33.18$

Neglecting Impact $S_{S.F} = 19.91 / 0.2 = 99.55$

Case II: Maintenance Condition

$R = (89.1)(0.26795) = 23.87^{K}$

$S_{S.F} = 23.87/34.1 = 0.7$

Bore hole (M1) shows coal at the base of R-12. Lower bound values of $\phi = 15°$ and $c = 0$ were used in obtaining sliding resistance.

Foundation Pressure

Case I: Normal Operations

$f = \frac{74.3}{20} + \frac{(74.3)(10-9.5)(10)(12)}{20^3} = 4.27 \frac{K}{\square'}$

$f = \frac{74.3}{20} + \frac{(74.3)(10-9.83)(10)(12)}{20^3} = 3.90 \frac{K}{\square'}$

Case II: Maintenance Condition

$f = \left(\frac{2}{3}\right)\left(\frac{89.1}{4.86}\right) = 12.22$ K/FT²

Figure VI-16 River Wall - Upper Gate Mono R-12 (continued)

SUBJECT:	COMPUTED BY:	DATE:
River Wall- Upper Chamber R-15	CHECKED BY:	DATE:

Station: 1+17.5 B to 1+47 B

Length = 29.5 ft

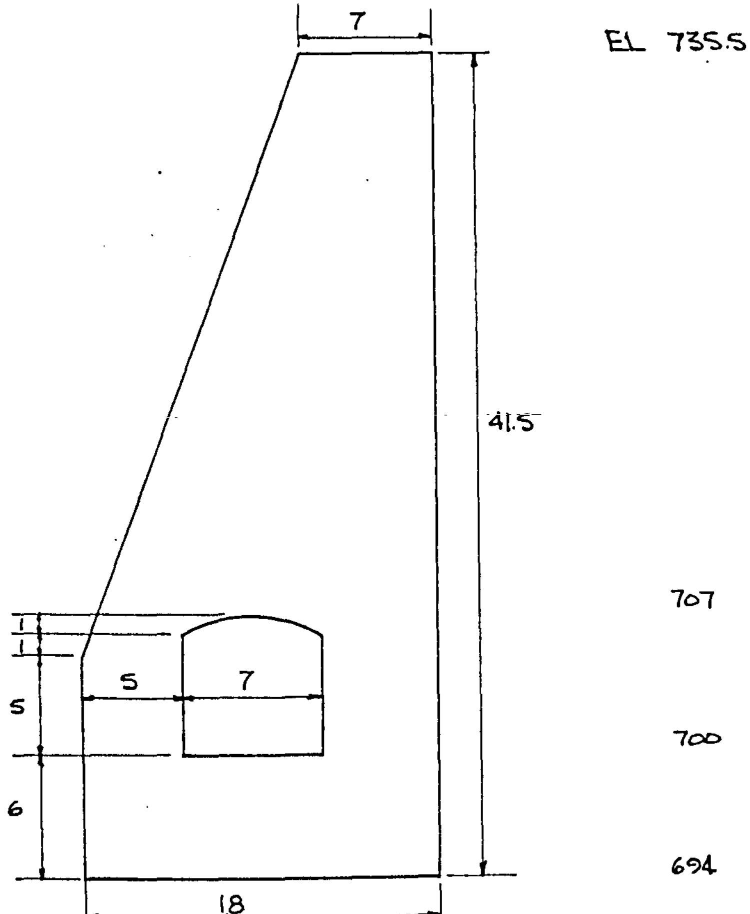


FIGURE VI-17 RIVER WALL - UPPER CHAMBER R-15

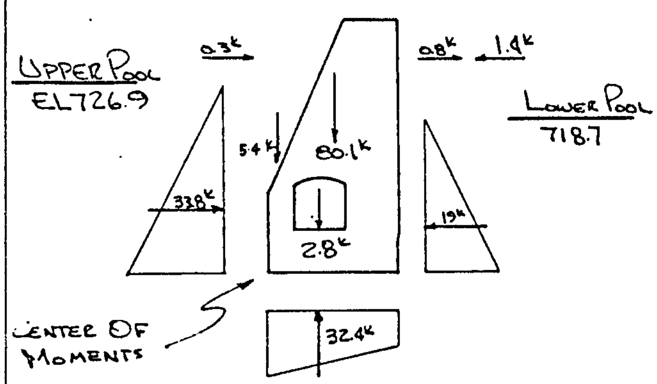
Scale 1:70

WES FORM NO. 1253A

SUBJECT: River Wall - Upper Chamber R-15

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

Not to Scale



Item	Factors	Fv	Fh	Arm	Moment
W_{conc}	$[0.150][(7)(41.5)+(\frac{1}{2})(11)(30.5)+(11)(11)-(6.5)(7)][1]$	80.1		10.72	859
P_{water}	$(718.7-694)^2(\frac{1}{2})(.0624)(1)$		-19	8.23	-156
P_{water}	$(726.9-694)^2(1/2)(.0624)(1)$		33.8	10.97	371
P_{wind}	$(0.03)(735.5-726.9)(1)$		0.3	37.2	11
Uplift	$(24.7)(.0624)(18)(1)+(\frac{1}{2})(8.2)(.0624)(18)(1)$	-32.4		8.57	-278
W_{water}	$(7)(65)(.0624)(1)$	2.8		8.5	24
W_{water}	$(726.9-705)(1)(\frac{1}{2})(.0624)(7.9)$	5.4		2.63	14
Impact	$(40/29.5)(1)$		-1.4	29.7	-42
Hawser	$(0.8)(1)$		0.8	29.7	24
	①	55.9	14.5		827
	Neglecting impact ②	55.9	15.9		869

$e_1 = 827/55.9 = 14.79'$

$e_2 = 869/55.9 = 15.55'$

Case I: Normal Operations

Assumptions
Upper Pool El 726.9
Lower Pool (El 718.7) in Lock

FIGURE VI-17 . RIVER WALL - UPPER CHAMBER R-15 (CONTINUED)

WES FORM NO. 1253A

SUBJECT: River Wall - Upper Chamber R-15

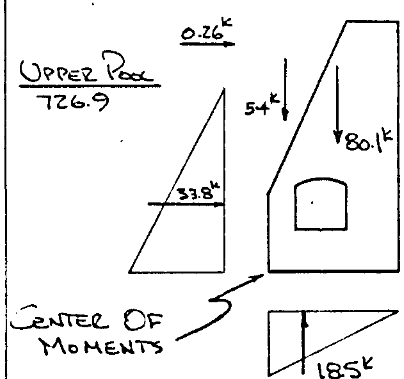
COMPUTED BY:

DATE:

CHECKED BY:

DATE:

Not To Scale



Item	Factors	F_V	F_H	Arm	Moment
W_{conc}		80.1		10.72	859
P_{water}	$(726.9-694)^2(1/2)(.0624)(1)$		33.8	10.97	371
Uplift	$(726.9-694)(1/2)(18)(.0624)(1)$	-18.5		6	-111
P_{wind}	$(0.03)(1)(735.5-726.9)$		0.26	37.2	10
W_{water}	$(726.9-705)(1)(1/2)(.0624)(7.9)$	5.4		2.63	14
		67	34.1		1143

$e = 1143/67 = 17.06$ FT

Case II: Maintenance Condition

Assumptions:
Dewatered River Lock
Upper Pool EL 726.9

FIGURE VI-17 . RIVER WALL - UPPER CHAMBER R-15 (CONTINUED)

WES FORM NO. 1253A

SUBJECT:	COMPUTED BY:	DATE:
River Wall-Upper Chamber R-15	CHECKED BY:	DATE:

Sliding

Case I: Normal Operations

$R = \Sigma V \tan \phi$

$R = (55.9)(.63707)$

$R = 35.61^K$

$S_{S-F} = 35.61/14.5 = 2.46$

Neglecting Impact $S_{S-F} = 35.61/15.9 = 2.24$

Case II: Maintenance Condition

$R = (67)(.63707)$

$R = 42.68^k$

$S_{S-F} = 42.68/34.1 = 1.25$

Foundation Pressure

Case I: Normal Operations

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

$f = \left(\frac{2}{3}\right)\left(\frac{55.9}{3.21}\right) = 11.61\ ^K/_{FT^2}$

Neglecting Impact $f = \left(\frac{2}{3}\right)\left(\frac{55.9}{2.45}\right) = 15.21\ ^K/_{FT^2}$

Case II: Maintenance Condition

$f = \left(\frac{2}{3}\right)\left(\frac{67}{0.94}\right) = 47.52\ ^K/_{FT^2}$

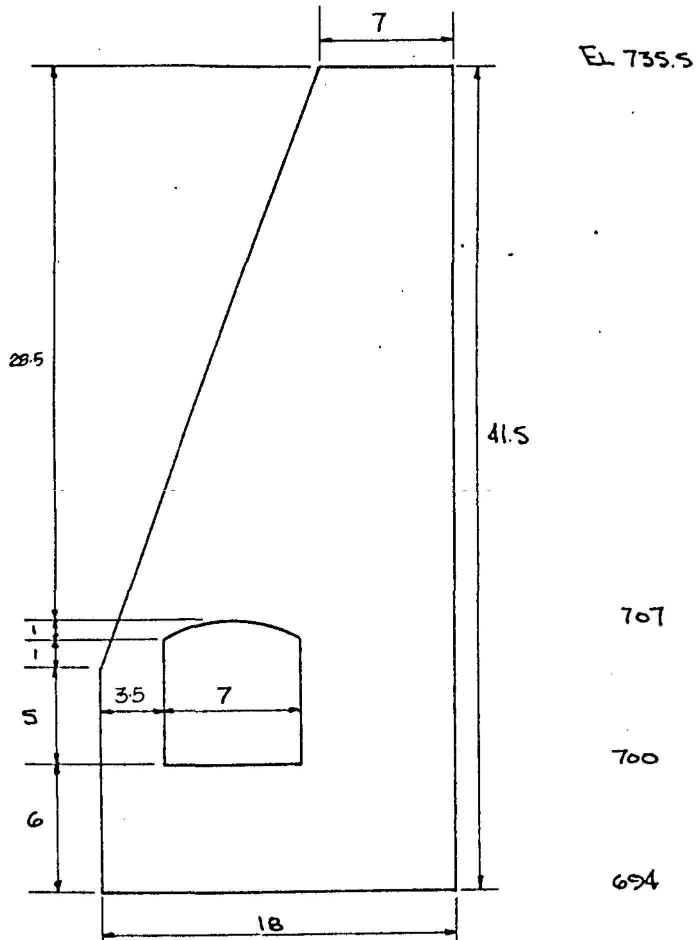
Figure VI-17 River Wall-Upper Chamber R-15 (continued)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT:	COMPUTED BY:	DATE:
RIVER WALL - LOWER CHAMBER R-21	CHECKED BY:	DATE:

STATION: 3+19B TO 3+49.6B

LENGTH = 30.6 FT



WES FORM NO. 1253A DECEMBER 1964

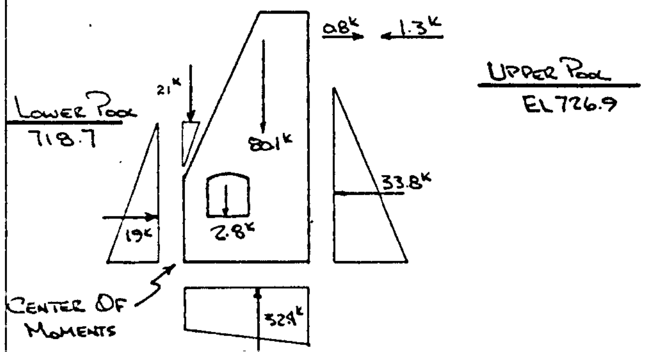
FIGURE VI-18 RIVER WALL - LOWER CHAMBER R-21

SUBJECT: River Wall - Lower Chamber R-21

COMPUTED BY: DATE:

CHECKED BY: DATE:

Not to Scale



Item	Factors	F_V	F_H	Arm	Moment
W_{conc}	[0.150][(7)(41.5)+(½)(11)(30.5) +(11)(11) - (6.5)(7)][1]	80.1		10.85	869
P_{water}	$(726.9-694)^2$(½)(.0624)(1)		-33.8	10.97	-371
P_{water}	$(718.7-694)^2$(½)(.0624)(1)		19	8.23	156
Uplift	(24.7)(.0624)(18)(1)+(½)(8.2) (.0624)(18)(1)	-32.4		9.43	-306
W_{water}	(7)(6.5)(.0624)(1)	2.8		7.0	20
W_{water}	(.0624)(13.7)(1)(½)(.3607)	2.1		1.64	3
Impact	(40/30.6)(1)		-1.3	37.9	-49
Hawser	(0.8)(1)		0.8	37.9	30
	①	52.6	-15.3		352
	NEGLECTING IMPACT ②	52.6	-14		401

Case I: Normal Operations

Assumptions:
Lower Pool EL 718.7
Upper Pool (726.9) in Lock

$e_1 = 352/52.6 = 6.69$ FT

$e_2 = 401/52.6 = 7.62$ FT

FIGURE VI-18 , RIVER WALL - LOWER CHAMBER R-21 (CONTINUED)

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: River Wall- Lower Chamber R- 21

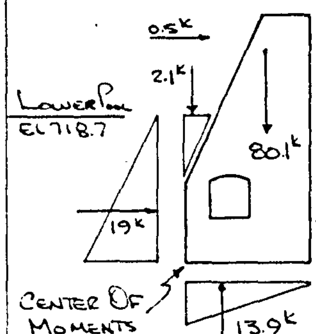
COMPUTED BY:

DATE:

CHECKED BY:

DATE:

Not To Scale



Item	Factors	Fv	FH	Arm	Moment
W_{CONC}		80.1		10.85	869
W_{WATER}		2.1		1.65	3
P_{WATER}	$(718.7-694)^2(\frac{1}{2})(.0624)(1)$		19	8.23	156
Uplift	$(718.7-694)(\frac{1}{2})(18)(.0624)(1)$	-13.9		6	-83
P_{WIND}	$(0.03)(1)(735.5-718.7)$		0.5	33.1	17
		68.3	19.5		962

$e = 962/68.3 = 14.1$ FT

Case II: Maintenance Condition

Assumptions
Dewater River Lock
Lower Pool EL 718.7

FIGURE VI-18 RIVER WALL - LOWER CHAMBER R-21 (CONTINUED)

NCD FORM NO. 1253A

SUBJECT:	COMPUTED BY:	DATE:
River Wall- Lower Chamber R-21	CHECKED BY:	DATE:

Sliding

Case I: Normal Operations

$R = \Sigma V \tan\phi$

$R = (52.6)(.63707)$

$R = 33.51^{K}$

$S_{S\text{-}F} = 33.51/-15.3 = -2.19$

Neglecting Impact $S_{S\text{-}F} = 33.51/-14 = -2.39$

Case II: Maintenance Condition

$R = (68.3)(.63707)$

$R = 43.51$

$S_{S\text{-}F} = 43.51/19.5 = 2.23$

Foundation Pressure

Case I: Normal Operations

$f = \frac{52.6}{18} + \frac{(52.6)(9-6.69)(9)(12)}{18^3} = 5.17\,\frac{K}{D'}$

Neglecting Impact $f = \frac{52.6}{18} + \frac{(52.6)(9-7.62)(9)(12)}{18^3} = 4.27\,\frac{K}{D'}$

Case II: Maintenance Condition

$f = \left(\frac{2}{3}\right)\left(\frac{68.3}{3.9}\right) = 11.68$ K/FT²

Figure VI-18 River Wall- Lower Chamber R-21 (Continued)

SUBJECT: RIVER WALL - LOWER GATE MONO R-23

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

STATION: 3+82B TO 4+70.5 B

LENGTH: 88.5 FT

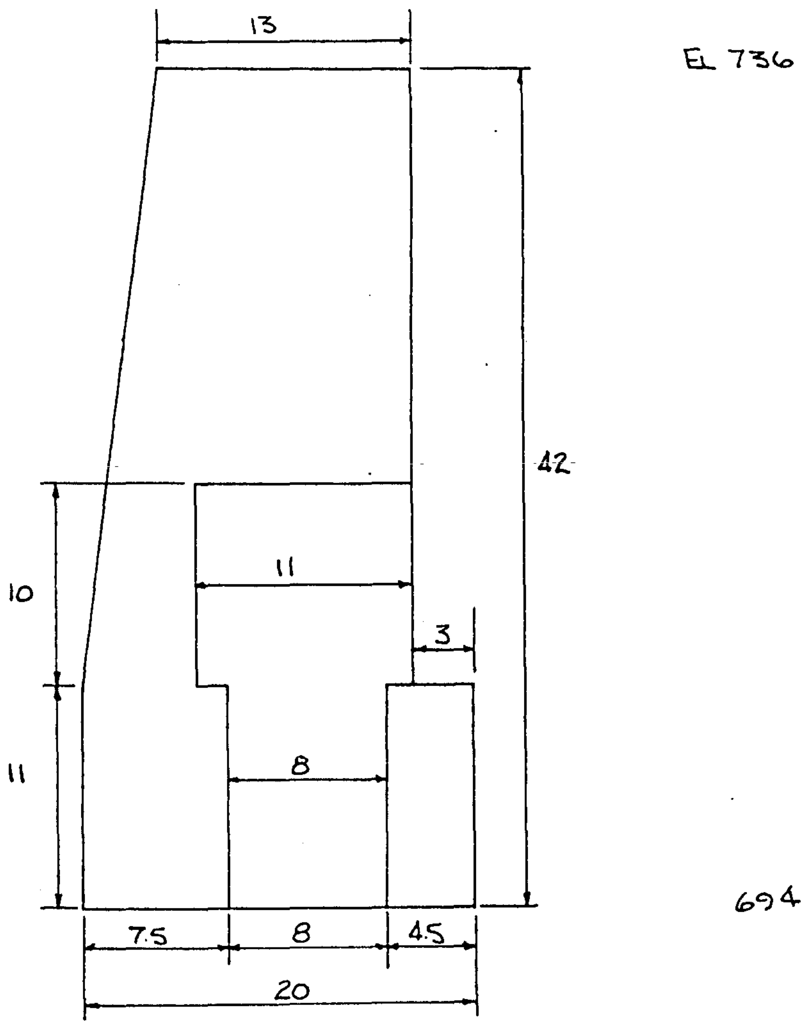


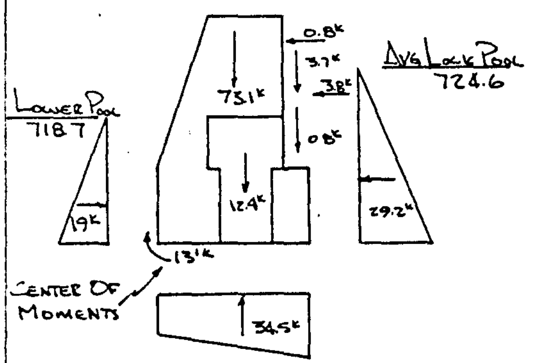
FIGURE VI-19 RIVER WALL - LOWER GATE MONO. R-23

WES FORM NO. 1253A

SUBJECT: River Wall- Lower Gate Mono R-23

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

Not To Scale



Item	Factors	F_V	F_H	Arm	Moment
W_{CONC}	[.150][(13)(31)+(½)(4)(31)+(20)(11) -(11)(10)-(8)(11)][1]	73.1		8.87	648
P_{WATER}	$(\frac{1}{2})(718.7-694)^2(1)(.0624)$		19	8.23	156
P_{WATER}	$(\frac{1}{2})(724.6-694)^2(1)(.0624)$		-29.2	10.2	-298
Uplift	(718.7-694)(20)(1)(.0624)+(724.6-718.7)(20)(½)(1)(.0624)	-34.5		10.35	-357
W_{WATER}	(724.6-705)(1)(3)(.0624)	3.7		18.5	68
W_{WATER}	(11)(10)(1)(.0624)+(8)(11)(1)(.0624)	12.4		11.5	143
W_{WATER}	On River Face Is Negligable				
Impact Gate			-0.8	37.9	-30
	$P=(\frac{1}{2})(8.2)^2(1)(.0624)+(8.2)(13.7)(1)(.0624)$				
	$P=9.1^k$				
	$T=(0.55)(9.1)(56)/.6428=436^k$				
	$S=(436)(.766)=334^k$				
	$V'=74.6/88.5=0.84^k$	0.8		18.5	15
	$M'_{cv}=1156/88.5=13.1^{'k}$	-	-	-	13
	$S'=334/88.5=3.77^k$		-3.8	26.5	-101
①		55.5	-14.8		257
Neglecting Impact ②		55.5	-14		287

$e_1 = 257/55.5 = 4.63'$

$e_2 = 287/55.5 = 5.17'$

Case I: Normal Operation

Assumptions

% Avg Lock Pool El 724.6

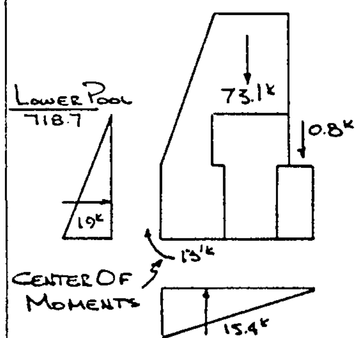
Lower Pool El 718.7

FIGURE VI-19 RIVER WALL - LOWER GATE MONO. R-23 (CONTINUED)

SUBJECT: River Wall - Lower Gate Mono R-23

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

Not To Scale



Item	Factors	Fv	Fh	Arm	Moment
W_{conc}		73.1		8.87	648
P_{water}			19	8.23	156
Uplift	(½)(718.7-694)(20)(1)(.0624)	-15.4		6.67	-103
Gate	P=0				
	T=0				
	S=0				
	V= 0.8	0.8		18.5	15
	M= 13'k	-	-	-	13
		58.5	19		729

$e = 729/58.5 = 12.46'$

Case II: Maintenance Condition

Assumptions
Dewater Lock Chamber
Lower Pool El 718.7

FIGURE VI-19 RIVER WALL - LOWER GATE MONO. R-23 (CONTINUED)

SUBJECT: RIVER WALL - LOWER GATE MONO R-23

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

<u>SLIDING</u>

CASE I: NORMAL OPERATIONS

$R = \Sigma V \tan \phi$

$R = (55.5)(.63707)$

$R = 35.36^{K}$

$S_{S-F} = 35.36/-14.8 = -2.39$

NEGLECTING IMPACT $S_{S-F} = 35.36/-14 = 2.53$

CASE II: MAINTENANCE CONDITION

$R = (58.5)(.63707)$

$R = 37.27^{K}$

$S_{S-F} = 37.27/19 = 1.96$

<u>FOUNDATION PRESSURE</u>

CASE I: NORMAL OPERATIONS

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

$f = \left(\frac{2}{3}\right)\left(\frac{55.5}{4.63}\right) = 7.99\ ^{K}/_{FT^2}$

NEGLECTING IMPACT $f = \left(\frac{2}{3}\right)\left(\frac{55.5}{5.17}\right) = 7.16\ ^{K}/_{FT^2}$

CASE II: MAINTENANCE CONDITION

$f = \frac{58.5}{20} + \frac{(58.5)(2.46)(10)(12)}{20^3} = 5.08 \frac{K}{\square'}$

FIGURE VI-19 RIVER WALL - LOWER GATE MONO R-23 (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
FIXED DAM	CHECKED BY:	DATE:

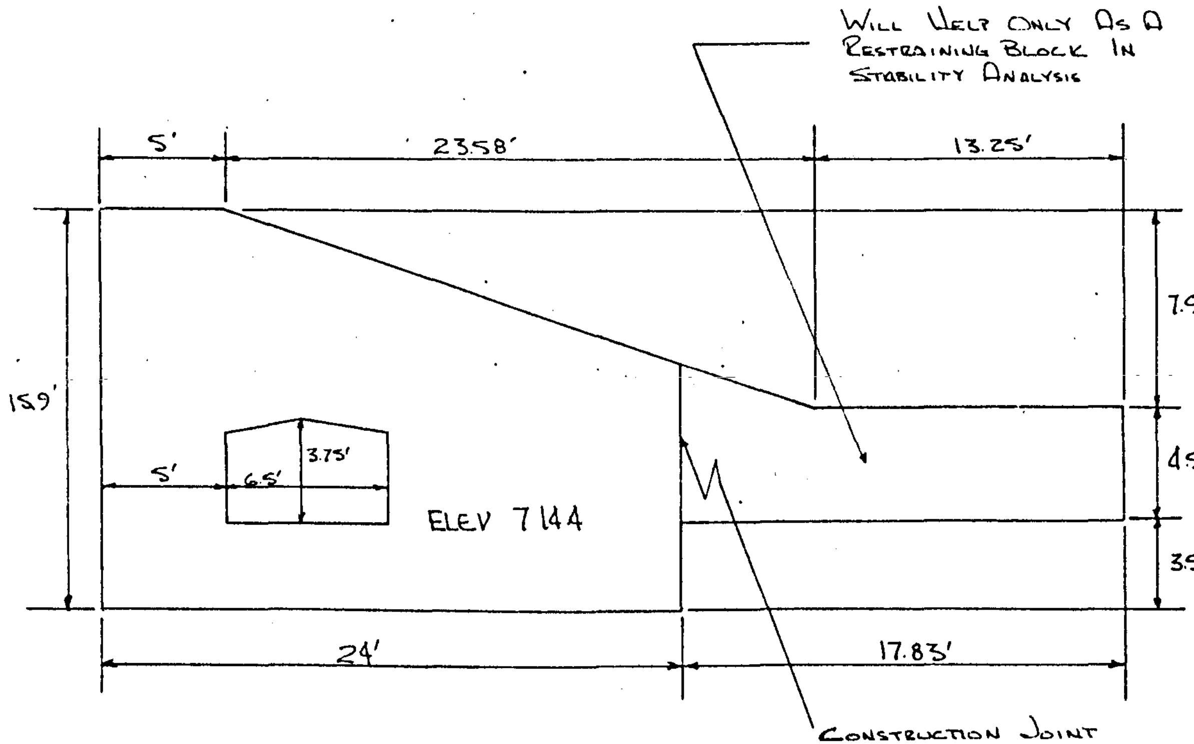
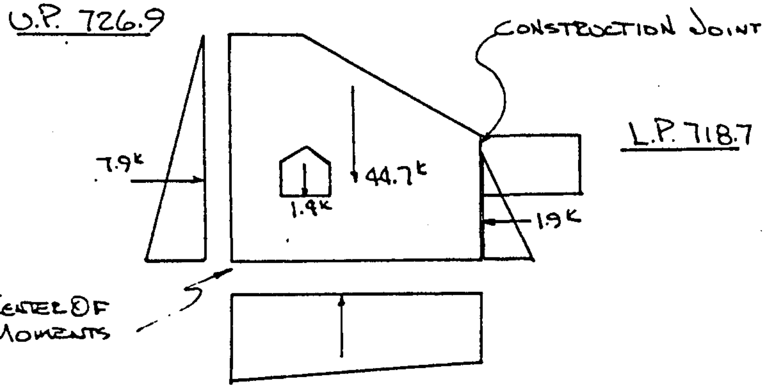


FIG VI-20 FIXED DAM

SCALE 1:50

SUBJECT: Fixed Dam | COMPUTED BY: | DATE:
CHECKED BY: | DATE:

Not To Scale



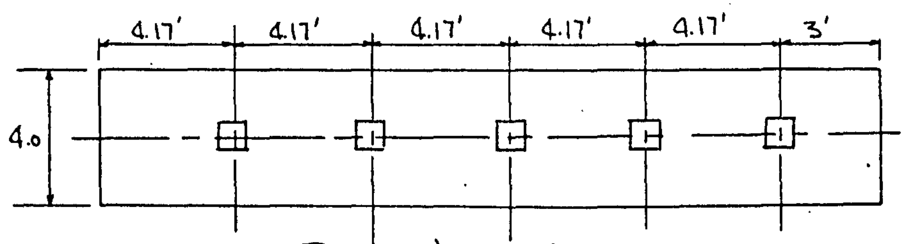
Item	Factors	Fv	Fh	Arm	Moment
W_{conc}	(24)(15.9)(1)(.15) - (6.5)(3.5)(1)(.15) - (19)(6.37)(½)(1)(.15)	44.7		12.44	556
W_{water}	(6.5)(3.5)(1)(.0624)	1.4		8.25	11.6
P_{water}	$(½)(726.9-711)^2(.0624)(1)$		7.9	5.3	41.9
P_{water}	$(½)(718.7-711)^2(.0624)(1)$		-1.9	2.57	-4.9
Uplift	(24)(3.84)(1)(.0624) + (24)(12.06)(1)(.0624)	-23.9		7.41	-177
		22.2	6.0		427.6

$e = 427.6/22.2 = 19.26'$

FIG VI-20 Fixed Dam (Continued)

SUBJECT:	COMPUTED BY:	DATE:
FIXED DAM	CHECKED BY:	DATE:

PILING ARE SPACED ON 4-FOOT CENTERS ALONG THE DAM. THE UPSTREAM ROW OF PILES WILL BE CONSIDERED INEFFECTIVE BECAUSE THEY ARE NOT FULLY EMBEDDED IN THE CONCRETE MONOLITH.



SECTION FOR ANALYSIS

COMPUTE CENTROID OF PILE GROUP FROM LEFT SIDE:

NUMBER OF PILES	ARM	MOMENT
1	4.17	4.17
1	8.34	8.34
1	12.51	12.51
1	16.68	16.68
1	20.85	20.85
5		62.55

CENTROID = 62.55 / 5 = 12.51'

FIG VI-20 FIXED DAM (CONTINUED)

SUBJECT:	COMPUTED BY:	DATE:
Fixed Dam	CHECKED BY:	DATE:

Pile Area = $(\pi)(1)^2/4 = 0.79$ FT^2

$I_{(\text{PILE ROW})} = (2)(0.79)(4.17)^2 + (2)(0.79)(8.34)^2 = 27 + 110 = 137$ FT^4

Solve For Forces In Pile:

Force = $P/A \pm Mc/I$

$$F = \frac{(22.2)(4)}{(5)(.79)} \pm \frac{(22.2)(19.26 - 12.51)(8.34)}{137}$$

$F = 22.5 \pm 9.1$

$F_1 = 31.6$ KSF

$F_2 = 13.4$ KSF

Consider an allowable horizontal force of 6^k/pile

Actual Horizontal Force Per Pile = $(4)(6)/5 = 4.8^k$

Allowable Compressive Force = (300 PSI)(0.79 FT^2)(144 IN^2/FT^2)

= 34 KSF

= 43 ^{OR} k/pile

No support from the foundation rock should be considered because if the rock settles the structure will then be supported entirely by the piling.

FIG VI-20 FIXED DAM (CONTINUED)

VI-98

WES FORM NO. 1253A DECEMBER 1966

SUBJECT: ABUTMENT

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

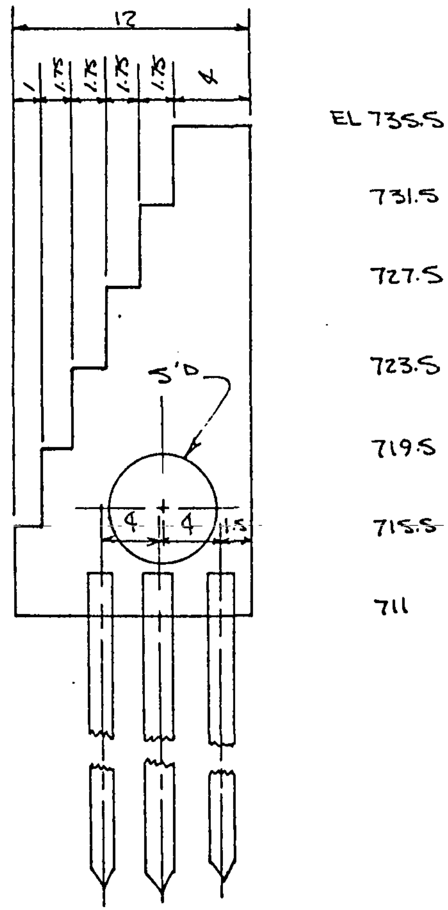


FIG VI-21 ABUTMENT

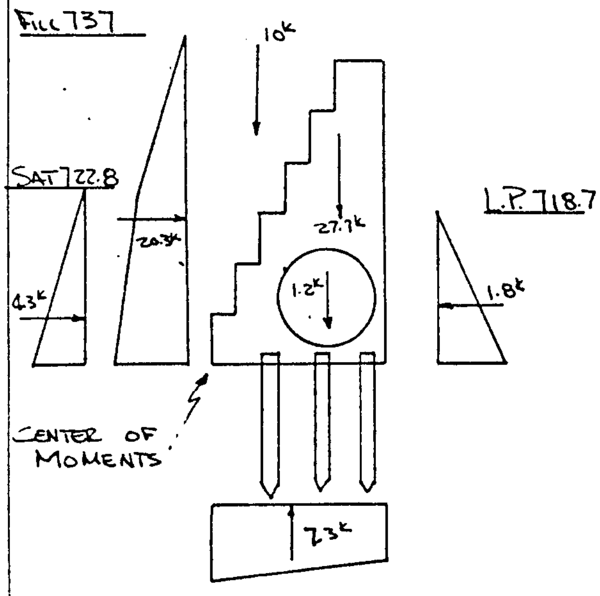
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: Abutment

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

Not To Scale



Item	Factors	ΣV	F_H	Arm	Moment
W_{conc}	$[0.150][(4)(24.5)(1)+(1.75)(1)(20.5)+(1.75)(16.5)(1)+(1.75)(12.5)(1)+(1.75)(8.5)(1)+(1)(1)(4.5)] - \pi(5)^2/4(.15)(1)$	27.7		7.29	202
W_{water} (culvert)	$(\pi)(5)^2/4(1)(.0624)$	1.2		7.5	9
W_{rock}	$[.13][(1)(20)+(1.75)(16)+(1.75)(12)+(1.75)(8)+(1.75)(4)]+(3.3)(1.75)(.015)+(7.3)(1)(.015)$	10		5.4	54
P_{earth}	$(737-722.8)^2(.130)(.5)(1/2) + (737-722.8)(722.8-711)(.130)/2 + (722.8-711)^2(.0826)(.5)(1/2)$		20.3	11.7	238
P_{water}	$(722.8-711)^2(.0624)(1/2)(1)$		4.3	3.93	17
P_{water}	$(718.7-711)^2(.0624)(1/2)(1)$		-1.8	2.6	-5
Uplift	$(7.7)(12)(.0624)+(11.8-7.7)(12)(1/2)(.0624)$	-7.3		10.27	75
		31.6	22.8		440

$e = 440/31.6 = 13.92'$

Fig VI-21 Abutment (continued)

WES FORM NO 1253A DECEMBER 1966

SUBJECT: Abutment | COMPUTED BY: | DATE: | CHECKED BY: | DATE:

Calculate Centroid Of Pile Group:

Number Of Pile	Arm	Moment
1	2.5	2.5
1	6.5	6.5
1	10.5	10.5
3		19.5

Centroid = 19.5/3 = 6.5'

Area Of Pile = 0.79 FT^2

$I_{(PILE ROW)} = (1)(4)^2 + (1)(4)^2 = 32\ FT^4$

Check Horizontal Resistance:

$H_{PER\ PILE} = (22.8)(4)/3 = 30.4$ K/PILE   Exceeds Allowable

Check Allowable Compressive Load On Piles:

$$f = \frac{(31.6)(4)}{0.79} \pm \frac{(13.92-6.5)(4)(4)}{32} = 53 \pm 117$$

$F_1 = 170^k$ (compression)/pile

$F_2 = -64^k$ (tension)/pile

Allowable = $(300)(.79)(3)(144) = 102{,}400^{\#} = 102.4^k$ / pile

The Piles Are Not Adequate In Either Compression or Tension

FIG VI-21 Abutment (continued)

VI-101

WES FORM NO. 1253A

APPENDIX VII

STRESS ANALYSIS

APPENDIX VII

STRESS ANALYSIS

Introduction

Plates which are mentioned (Section VII) but would distract from an efficient review of the stress analysis of selected monoliths at locks 3 are presented in this Appendix.

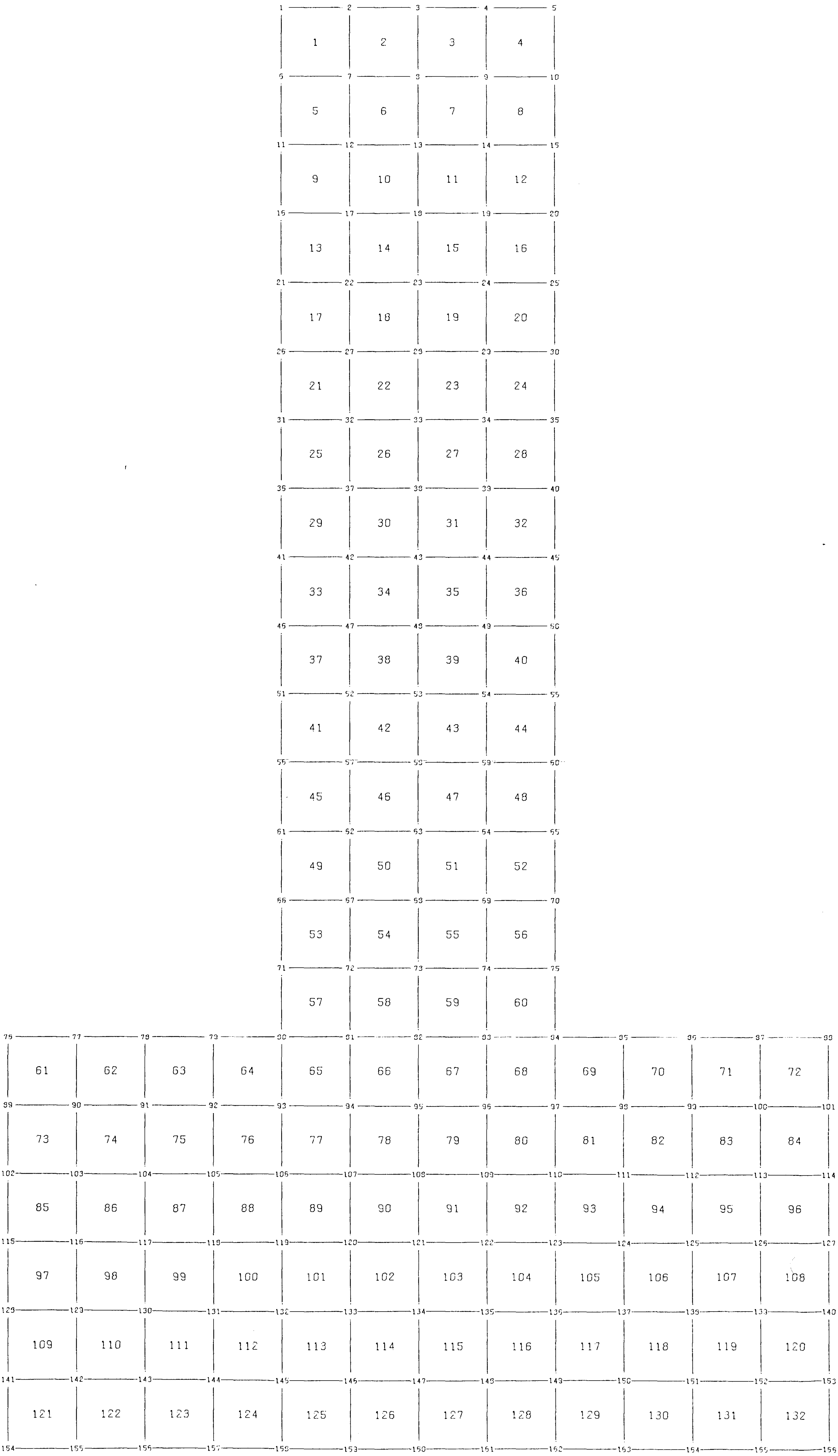


Figure VII.1. Example problem-finite element grid, no uplift. Grid dimensions are 1 X 1 ft

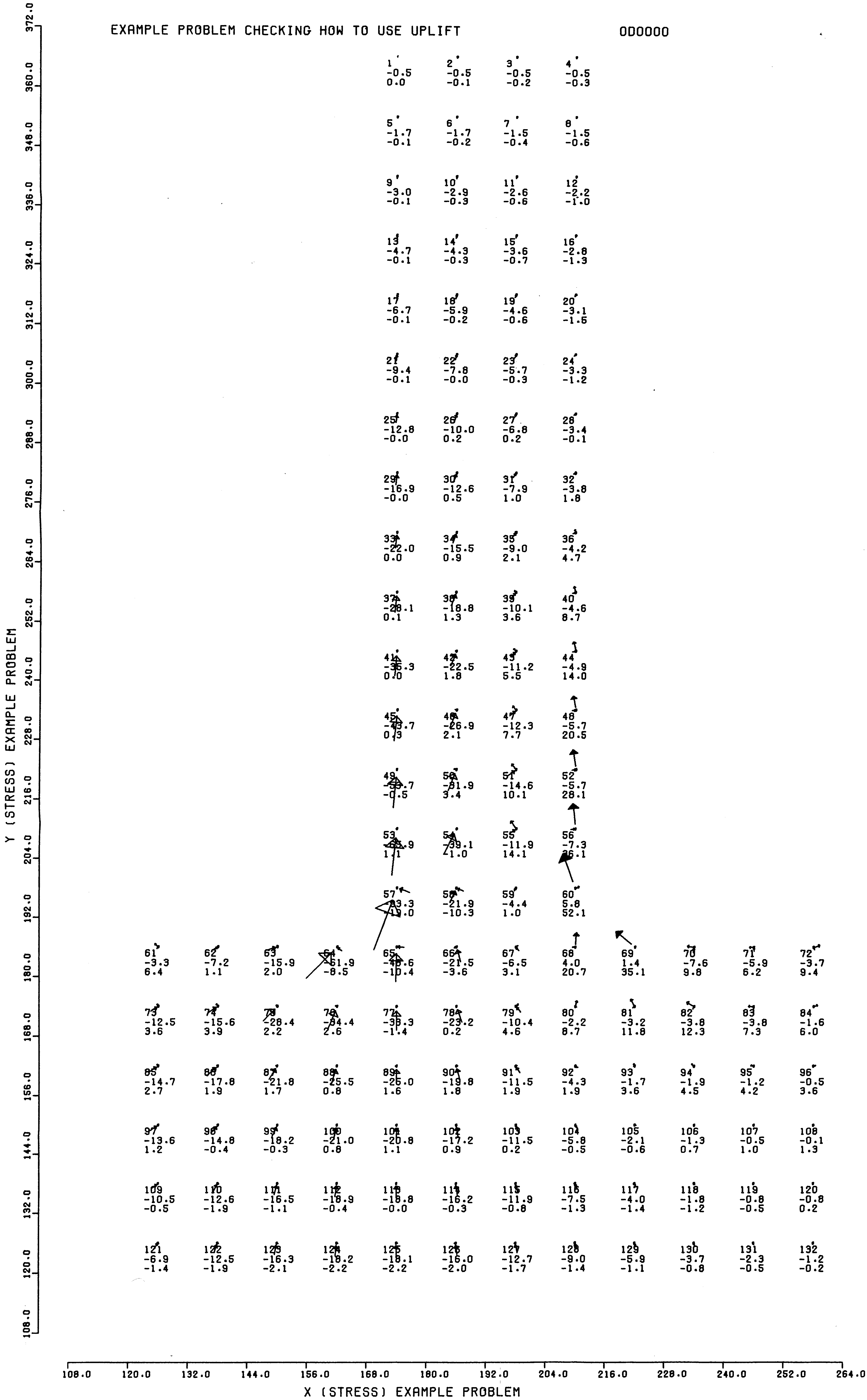


Figure VII.2. Example problem-stress vector plot, no uplift

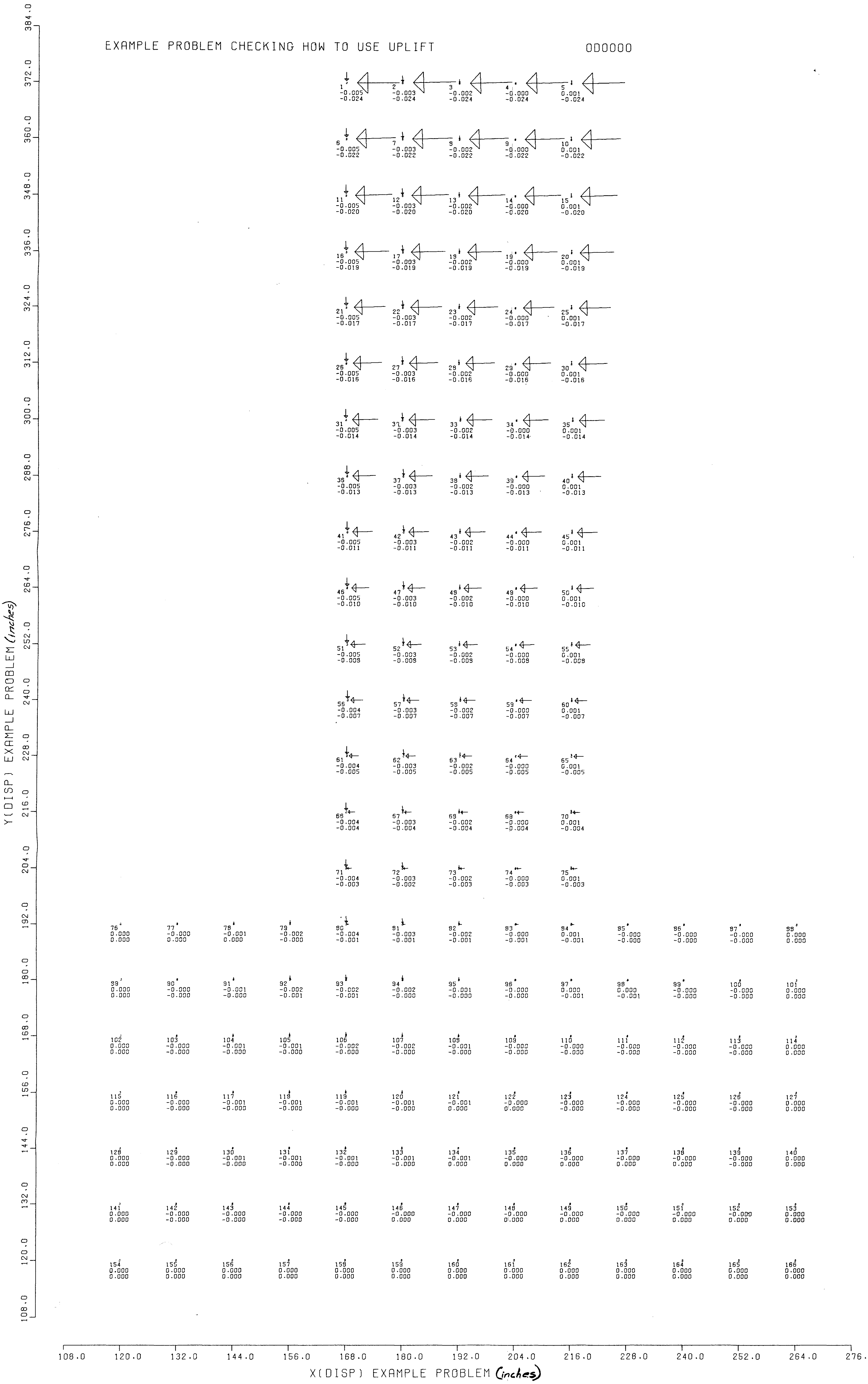


Figure VII.3. Example problem-displacement vector component plot, no uplift

Figure VII.4. Example problem-finite element grid, uplift. Basic grid dimensions are 1' × 1'

Figure VII.4.

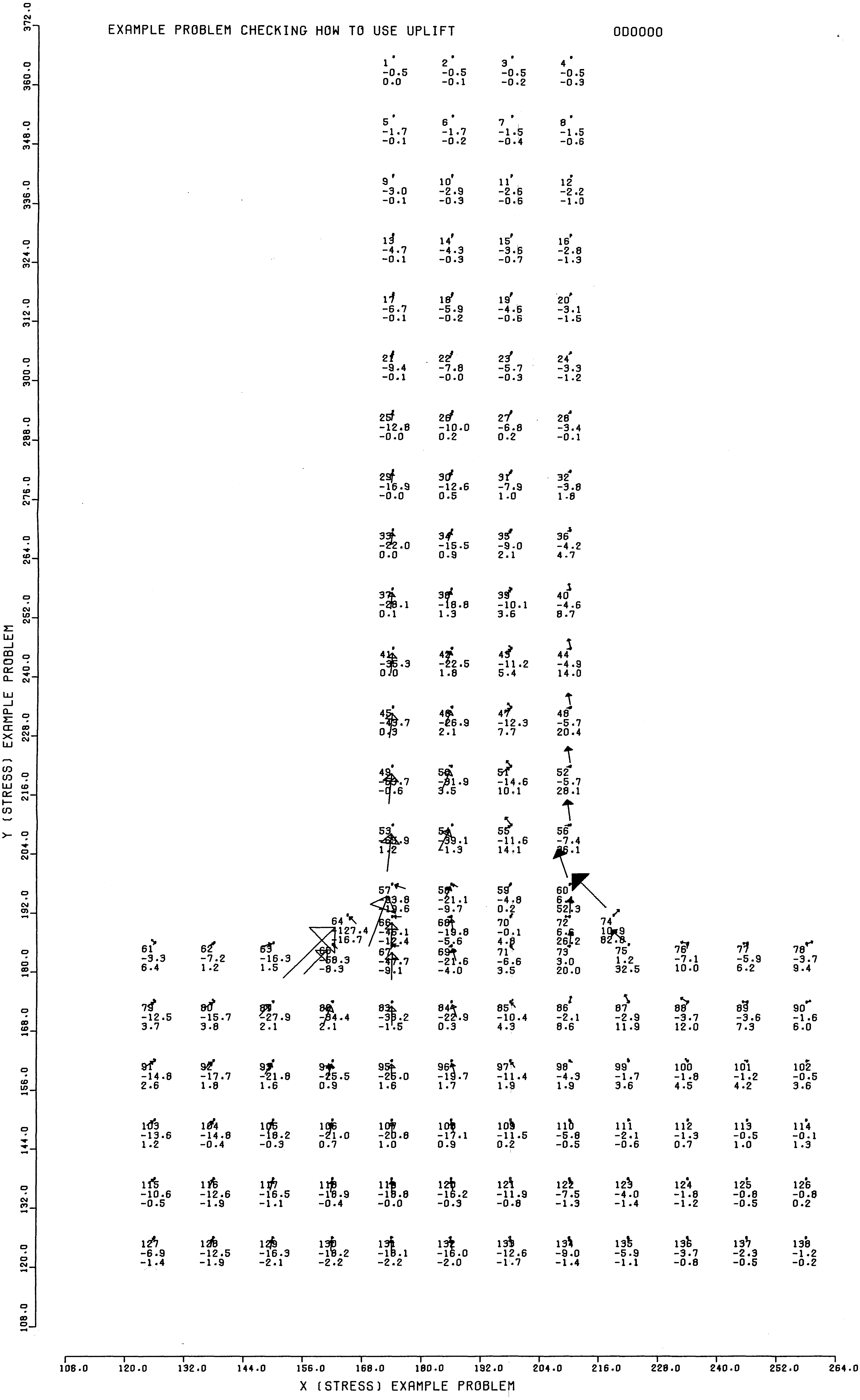


Figure VII.5. Example problem-stress vector plot, uplift

Figure VII.5.

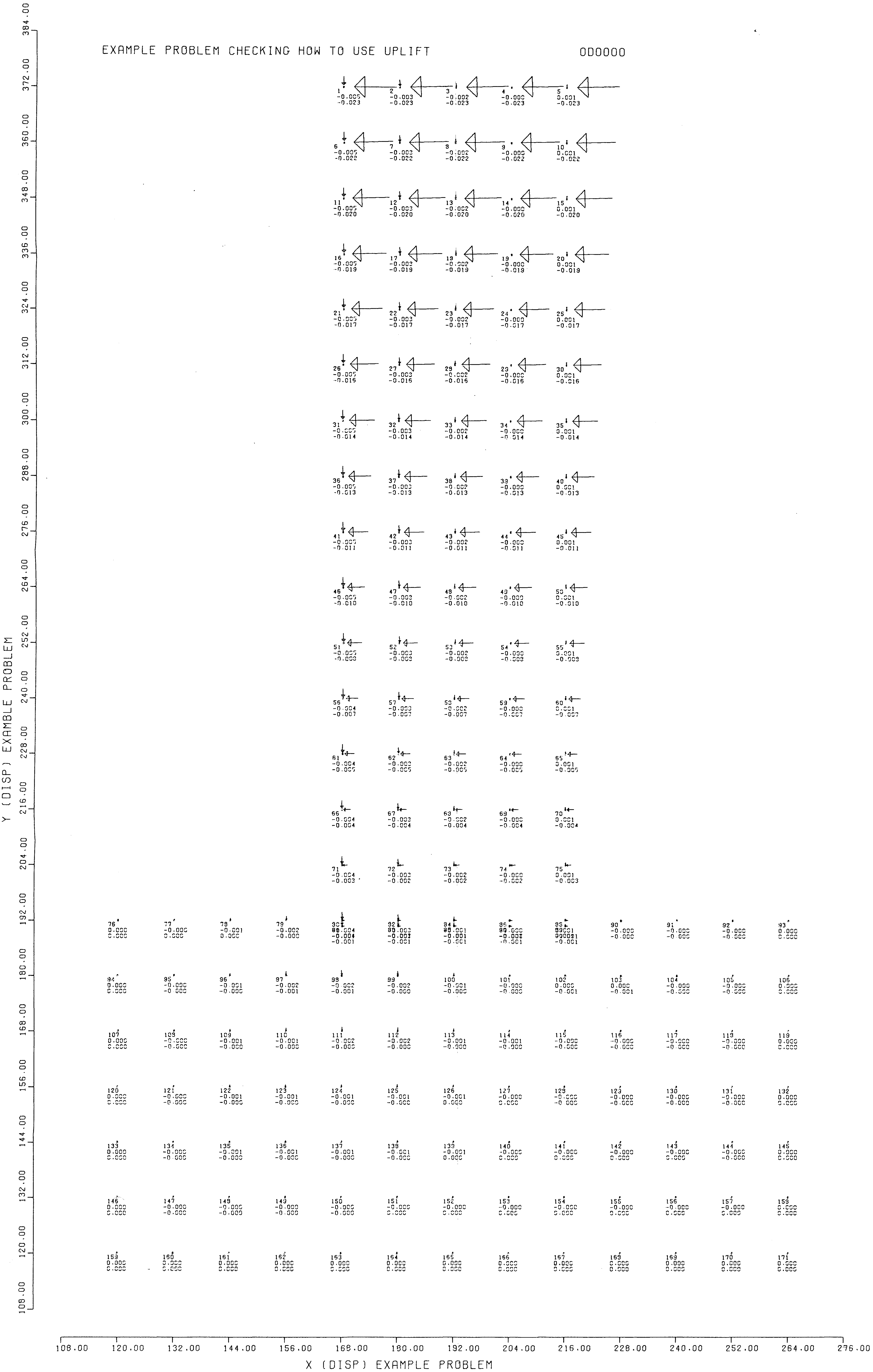


Figure VII.6. Example problem-displacement vector component plot, uplift

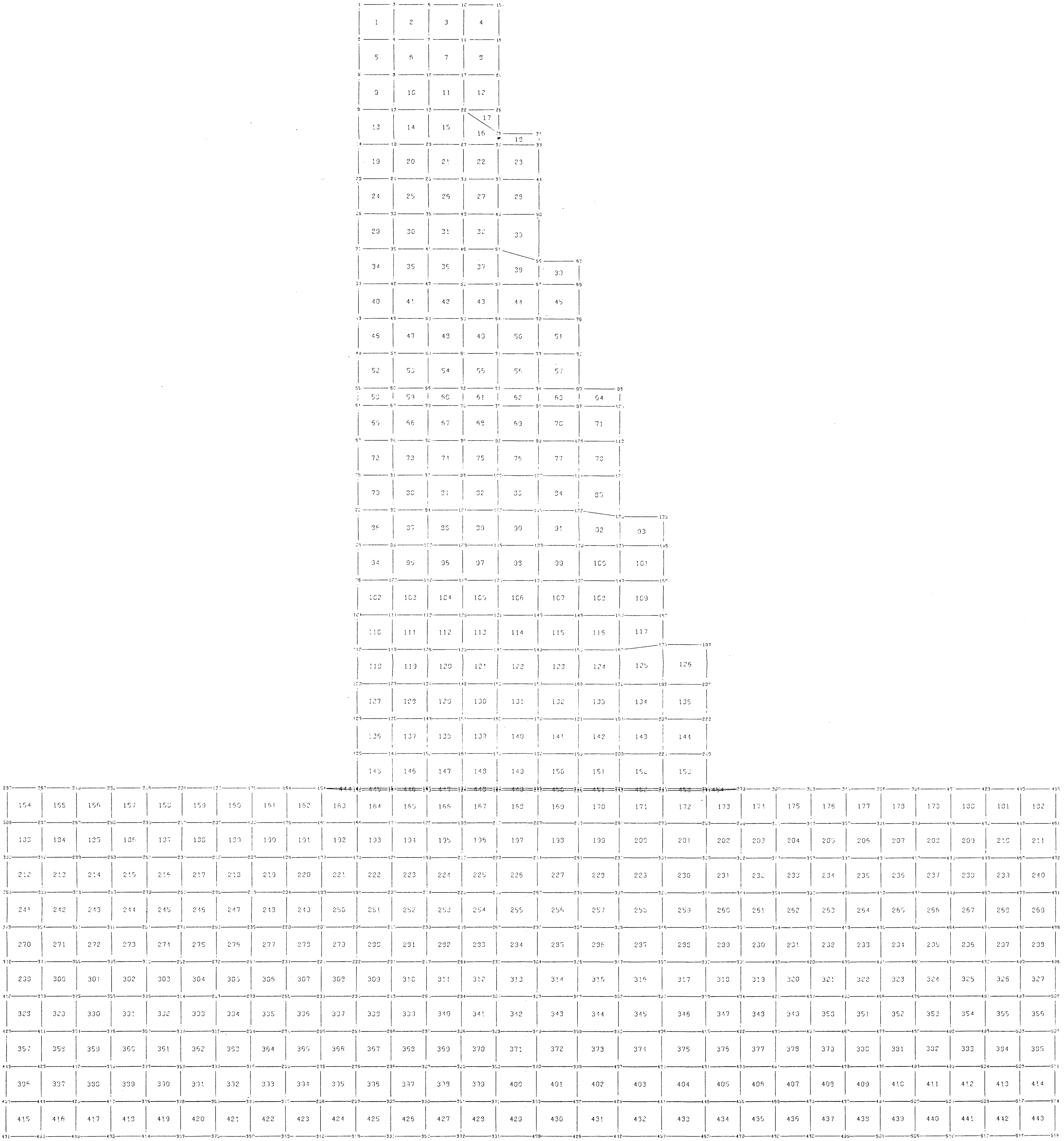


Figure VII.7. Grid-upper guide wall, monolith L-18. Basic grid dimensions are 1' × 1'

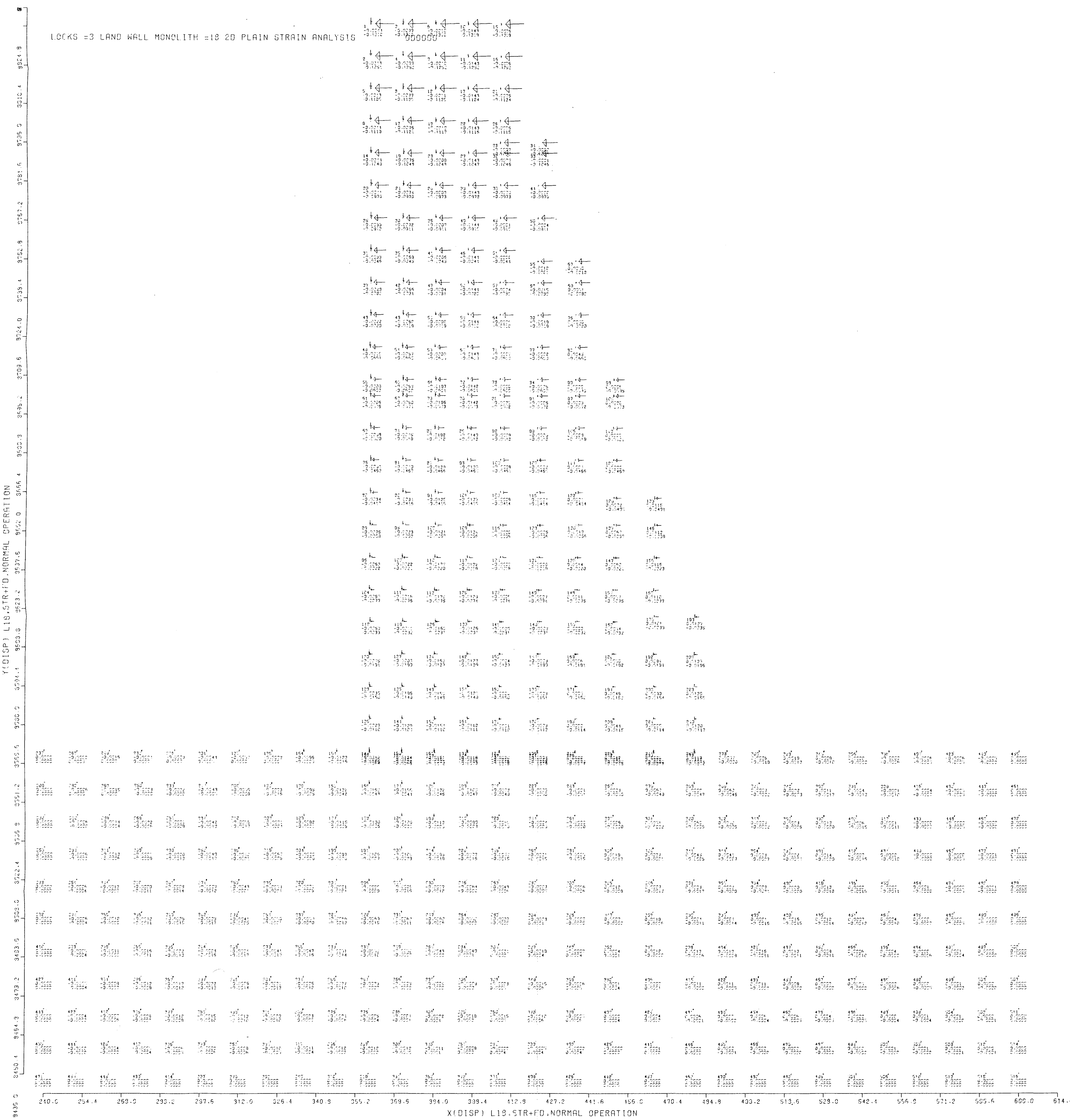


Figure VII.8. Displacement component vector plot; upper guide wall, monolith L-18

Figure VII.8.

Figure VII.9. Grid-landwall, upper gate bay monolith, L-23. Basic grid dimensions are 1' × 1'

Figure VII.9.

Figure VII.10. Displacement vector plot, L-23

Figure VII.10.

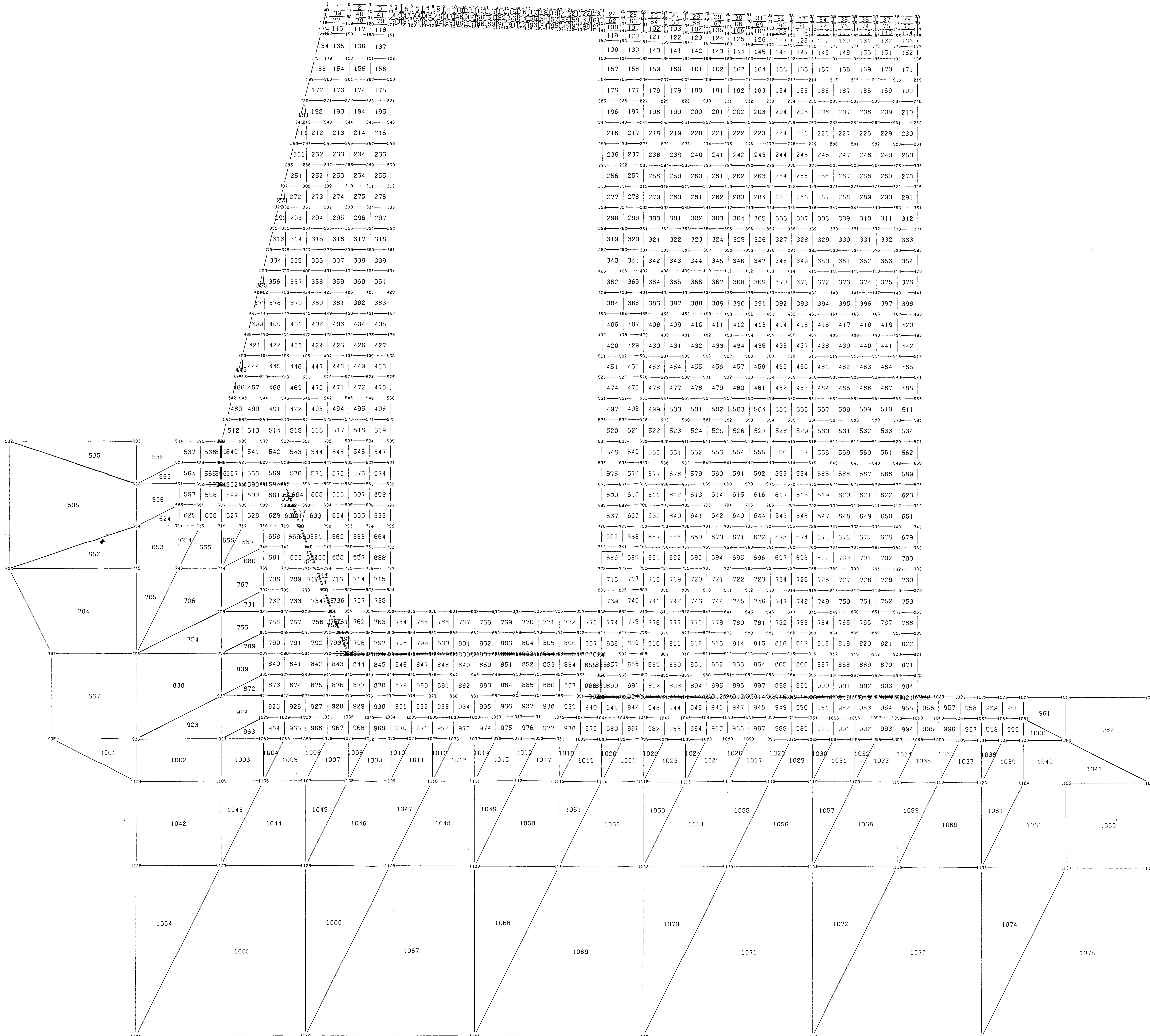


Figure VII.11. Grid-landwall, lock chamber monolith, L-32. Basic grid dimensions are 1' × 1'

Figure VII.11.

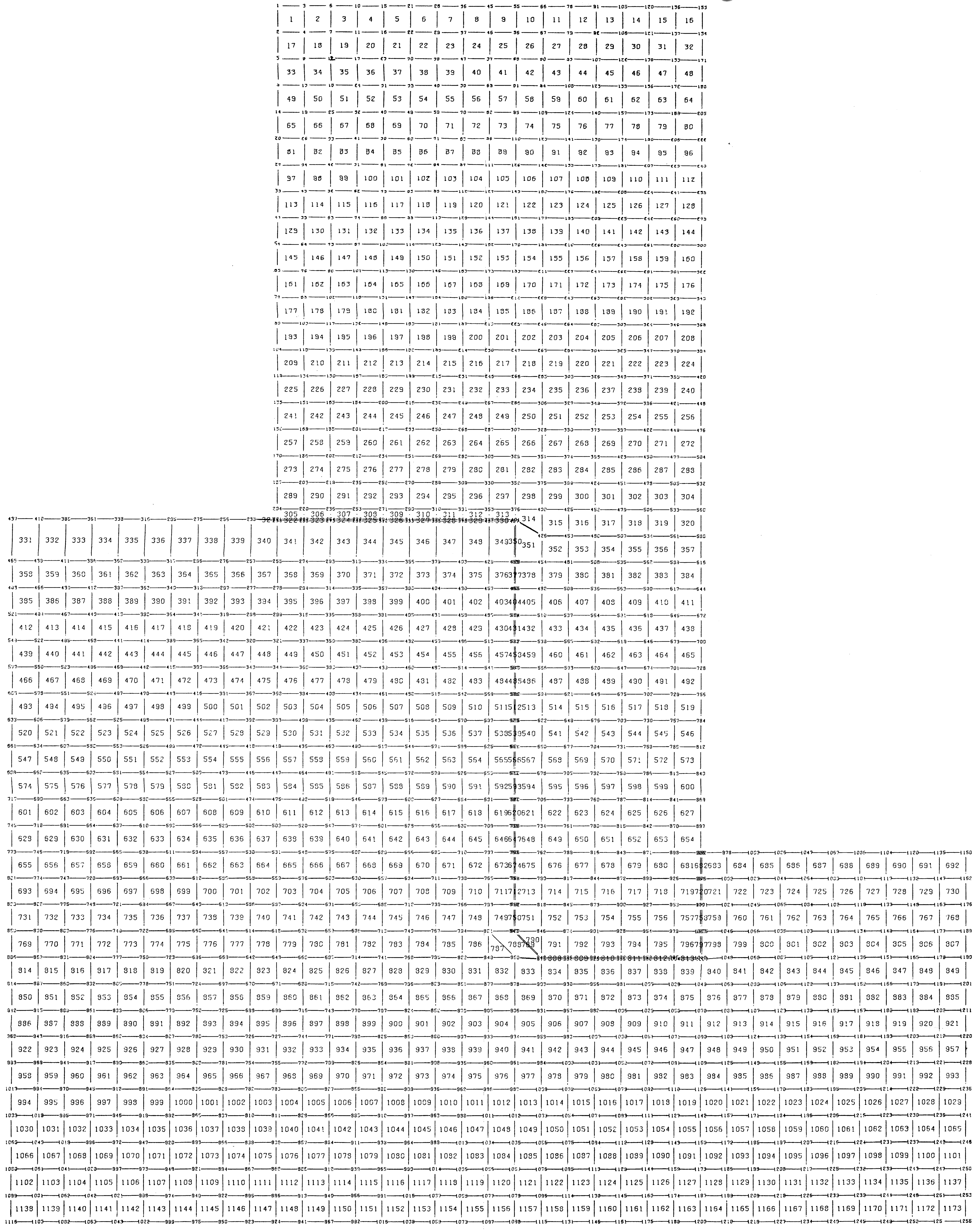


Figure VII.12. Grid-landwall, lower gate bay monolith, L-46. Basic grid dimensions are 1' × 1'

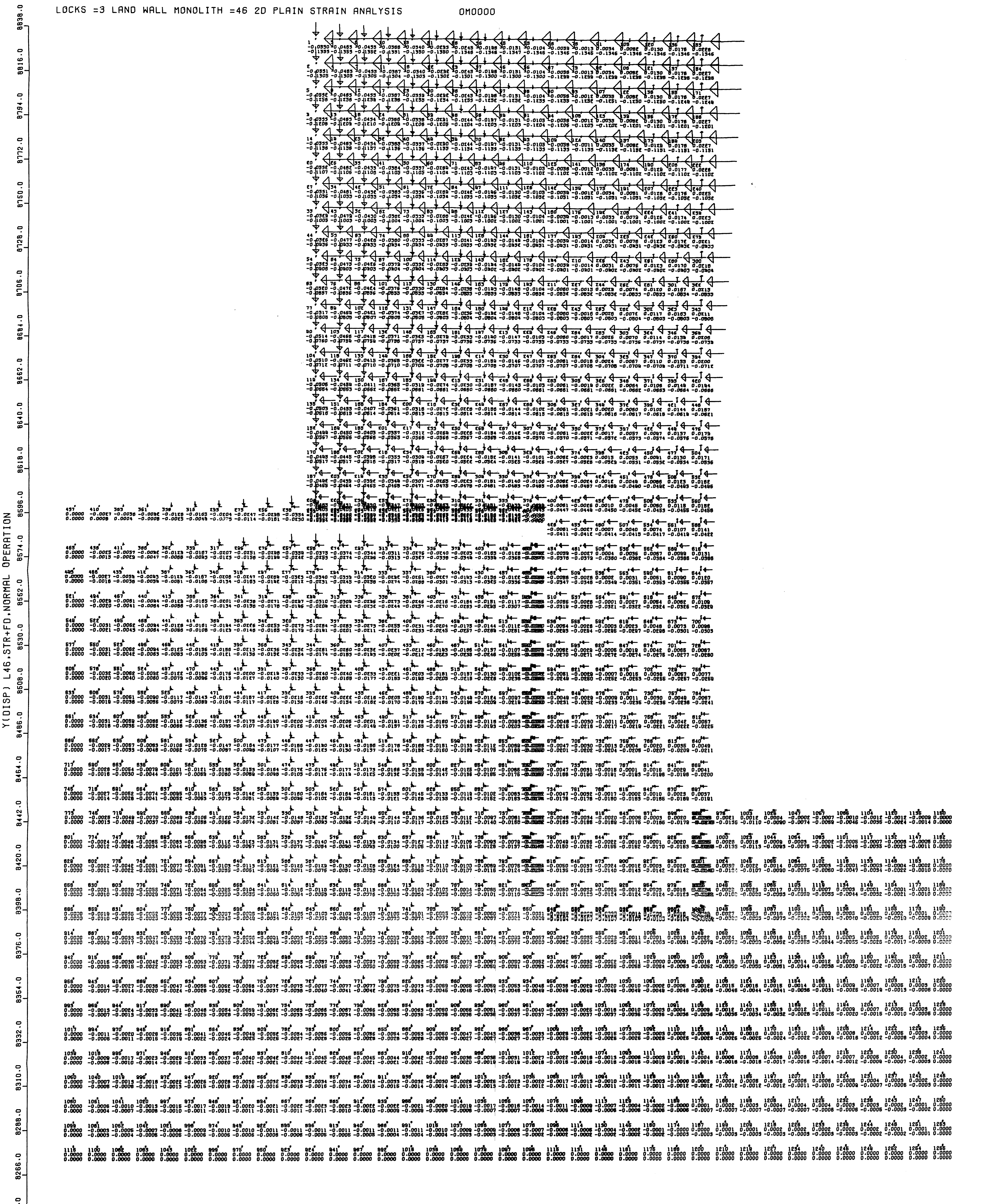


Figure VII.13. Displacement vector component plot, L-46

Figure VII.14. Grid-middle wall, lock chamber monolith, M-6 (structure only). Basic grid dimensions are 1' × 1'

Figure VII.14.

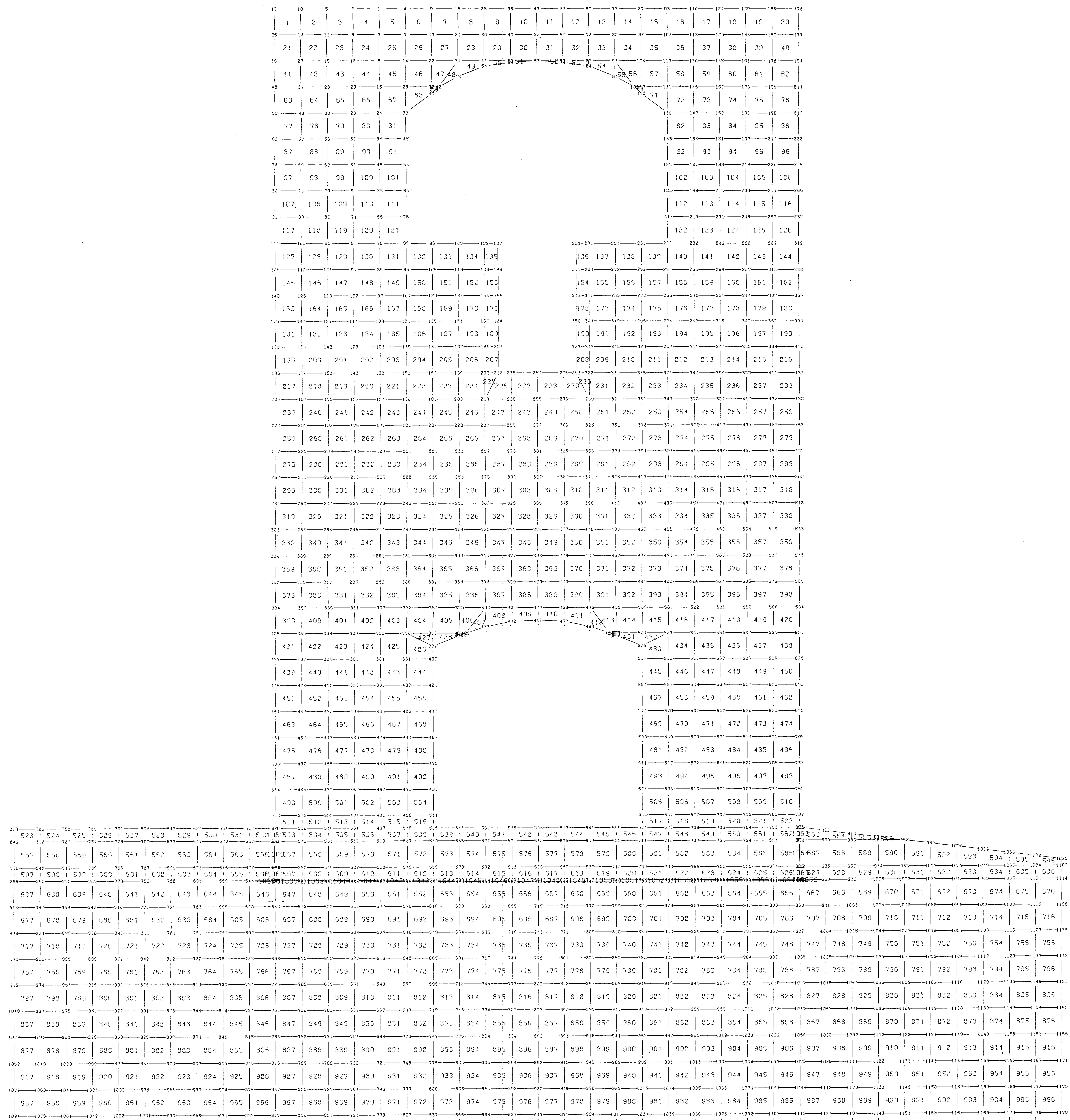


Figure VII.15. Grid-middle wall, lock chamber monolith, M-6 (structure plus first addition). Basic grid divisions are 1' × 1'

Figure VII.15.

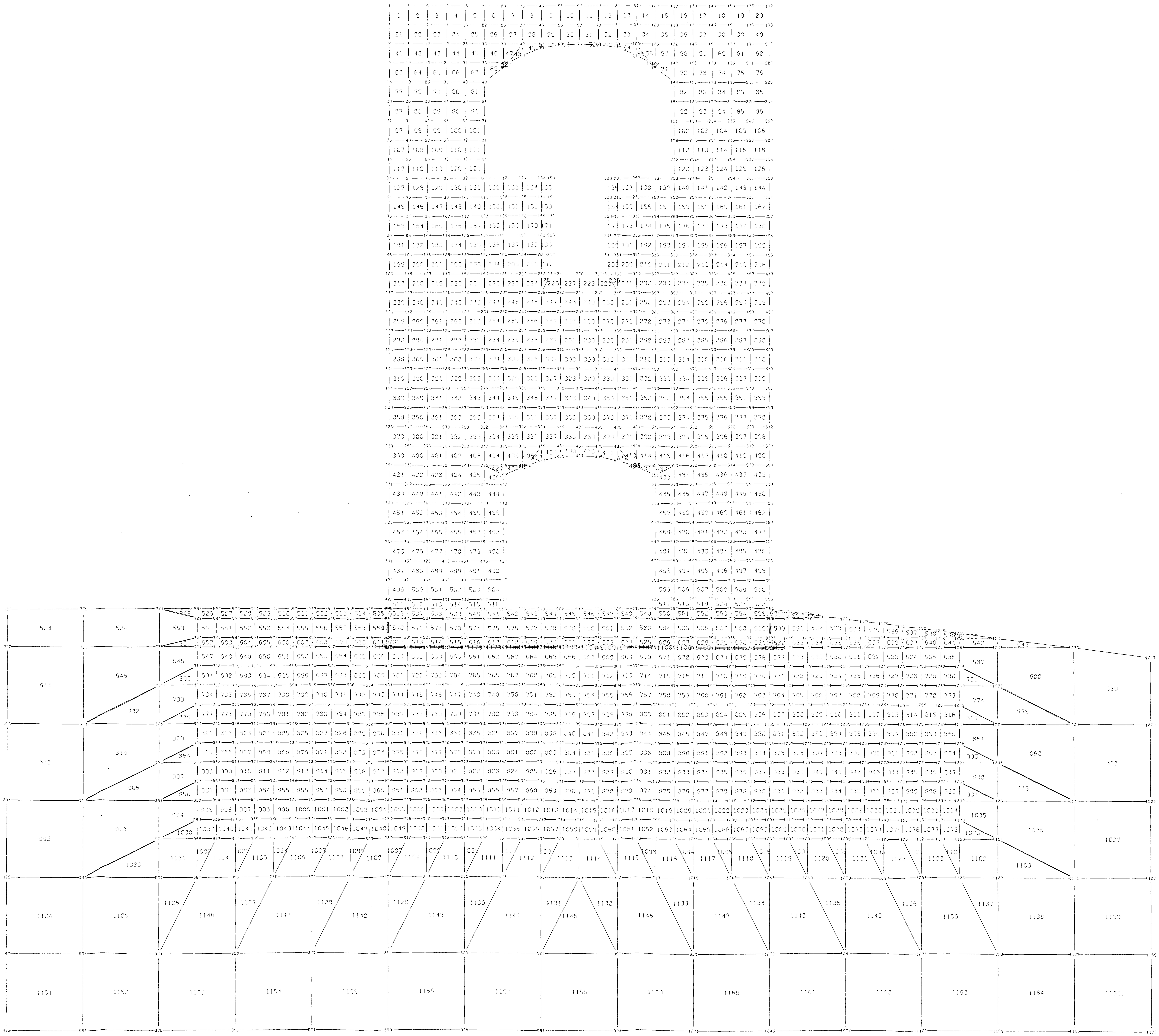


Figure VII.16. Grid-middle wall, lock chamber monolith, M-6 (structure plus first addition plus second addition). Basic grid divisions are 1' × 1'

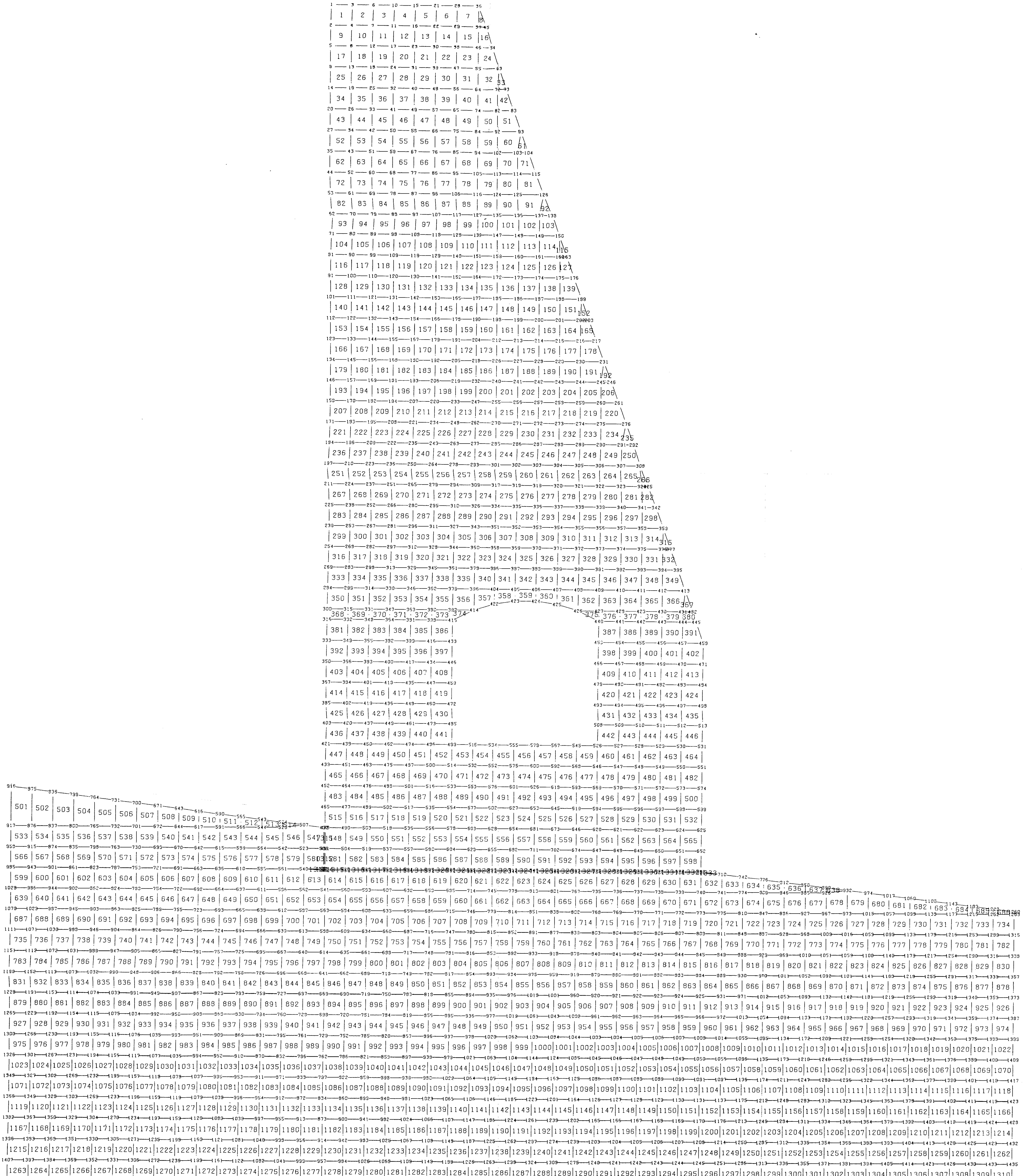


Figure VII.17. Grid-river wall monolith, R-15. Basic grid dimensions are 1' × 1'

Figure VII.17.