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TECHNICAL REPORT K-80-4

DOCUMENTATION OF FINITE ELEMENT ANALYSES

Report 2

ANCHORED WALL MONOLITH, BAY SPRINGS LOCK

by

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December 1980

Report 2 of a Series

A report under the Computer-Aided Structural Engineering (CASE) Project

DEPARTMENT OF THE ARMY
OFFICE OF THE CHIEF OF ENGINEERS
WASHINGTON, D.C. 20314

REPLY TO ATTENTION OF:

DAEN-CWE-DS 15 January 1981

SUBJECT: Technical Report K-80-4, Documentation of Finite Element Analyses, Anchored Wall Monolith, Bay Springs Lock, Report 2

All Corps Elements with Civil Works Responsibilities

1. The subject report provides an example of a finite element analysis documentation which satisfies the documentation guidelines contained in Engineer Technical Letter (ETL) 1110-2-254, dated 31 December 1980. The report shows how a good analysis and documentation effort can satisfy the guidelines contained in ETL 1110-2-254 even though the documentation does not follow the exact organization of the documentation in the guidelines.

2. Examples of finite element analysis documentations which do follow the organization of the documentation contained in ETL 1110-2-254 are provided in Report 1 of this series and in Inclosure 3 of the ETL.

3. The documentation guidance was developed to expedite the review of finite element analyses and to help insure that all applicable factors are considered in the problem idealization, analysis, and the interpretation of the output.

FOR THE CHIEF OF ENGINEERS:

Lloyd A. Duscha

LLOYD A. DUSCHA
Chief, Engineering Division
Directorate of Civil Works

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Tennessee-Tombigbee Waterway

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

Bay Springs Lock and Dam, located in northeast Mississippi, is the northernmost of the system of locks and dams on the Tennessee-Tombigbee Waterway. The navigation lock will be formed by a combination of anchored chambered walls and gravity-type gate blocks, with a reinforced concrete floor strut. The "anchored wall" monoliths differ from conventional gravity-type monoliths in that posttensioned tendons engaging rock account in part for the stability of the walls.

The anchored wall, with its concrete-foundation-tendon (Continued)

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20. ABSTRACT (Continued)

interaction characteristics, poses a formidable challenge to analysis by classical methods, making it an ideal problem for solution by a computerized, numerical analysis technique. Due to the complex geometry of the structure, multiple materials, posttensioning of the tendons, and the attendant boundary conditions, the finite element method was selected as the only viable analysis technique for determining the state of stress in the anchored wall monolith and its surrounding foundation.

The ANSYS FEM program was used, primarily to take advantage of its nonlinear boundary element which was used to approximate the interaction at material interfaces. Solutions for the various load cases were displayed graphically in numerous pressure, stress, and deflected shape diagrams.

Results confirm the assumptions of the rigid body stability analysis program used to size the tendons of the anchored wall monolith and indicate a safe design with regard to the established criteria.

Preface

This report documents a finite element analysis of the Bay Springs Lock Anchored Wall Monolith performed by the U. S. Army Engineer District, Nashville. The documentation effort was sponsored through funds provided to the U. S. Army Engineer Waterways Experiment Station (WES) by the Office, Chief of Engineers (OCE), under the Computer-Aided Structural Engineering (CASE) Project.

This report was written before the release of ETL 1110-2-254, dated 31 December 1980, subject "Finite Element Analysis Interpretation and Documentation Guidelines," so its organization does not reflect the guidelines presented in the ETL. The ETL was written by OCE with the assistance of members of the CASE Task Group on Finite Element Analysis. The report is being published to show how a good analysis and documentation effort can satisfy the ETL guidelines even though it may not follow a precise organizational pattern. An example of a documentation effort that strictly follows the guidelines in the ETL is provided in Report 1 of this series.

The CASE task group members reviewed this report. Members of the task group were:

P. Thomas McGee, Nashville District (Chairman)
Richard Flauaus, St. Louis District
Richard Huff, Kansas City District
David Raisanen, North Pacific Division
Byron E. Bircher, Kansas City District (Observer)
Robert B. Felder, Mobile District (Observer)
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The finite element analysis and the compilation of this report were performed by Mr. McGee under the supervision of Mr. Barney Johnson, former Chief, Structures Section, and Mr. Herman Gray, Chief, Design Branch, Nashville District.

OCE point of contact for this work was Mr. Guthrie, Structures Branch, Civil Works Directorate. Dr. Radhakrishnan, Special Technical

Assistant, Automatic Data Processing (ADP) Center, WES, is CASE Project Manager. The work was monitored by him under the general supervision of Mr. Donald L. Neumann, Chief, ADP Center.

Director of WES during the publication of this report was COL N. P. Conover, CE. Technical Director was Mr. F. R. Brown.

Contents

Conversion Factors, Inch-Pound to Metric Units of Measurement

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	2.54	centimetres
kips (1000 lb force)	4.448222	kilonewtons
kips (force) per square foot	47.880263	kilopascals
kips (force) per foot	14.593904	kilonewtons per metre

DOCUMENTATION OF FINITE ELEMENT ANALYSES

ANCHORED WALL MONOLITH, BAY SPRINGS LOCK

Key to Using this Report

1. One of the purposes of publishing this report is to show how a good analysis and documentation effort can satisfy the requirements of Engineer Technical Letter (ETL) 1110-2-254, "Finite Element Analysis Interpretation and Documentation Guidelines," even though it may not follow the precise organizational pattern called for in the guidelines. To achieve this purpose, the following key lists the numbers of paragraphs, figures, etc., in this report which contain the information called for in the ETL checklist:

Checklist Item	Where Documented
A.1.	Paragraph 2
A.2.	Paragraph 4
A.3.	Paragraph 4
B.1.	Paragraph 5
B.2.	Paragraph 6
B.3.	Paragraph 5
C.	Paragraph 7
D.1.	Paragraph 12
D.2.	Paragraph 9
D.3.	Paragraphs 11 and 13
D.4.	NA
D.5.	Paragraphs 17 and 18
D.6.	None
D.7.	NA
E.1.	Paragraphs 19-34
E.2.	Appendix A
E.3.	Appendix A
E.4. to E.6.	NA
F.1.	NA
F.2.	Figures 1-8
F.3.	None
G.1.	Paragraph 21
G.2.	Paragraphs 19-34
G.3.	Paragraphs 19-38
G.4.	Paragraph 38

(Continued)

Checklist Item	Where Documented
H.1.	Paragraph 35
H.2.	None
H.3.	Paragraph 37

Description of Structure

2. Bay Springs Lock and Dam, located in northeast Mississippi, is the northernmost of the system of locks and dams on the Tennessee-Tombigbee Waterway. It is located at the southern end of the Divide Section of the waterway and will create a pool extending through the Divide Cut to the Yellow Creek embayment of Pickwick Lake (Tennessee River).

3. The navigation lock will be located near the left end of the dam and will be perpendicular to the axis of the dam. The lock will have nominal chamber dimensions of 110 ft* wide by 600 ft long with a lift, based on normal upper and lower pools, of 84 ft. The lock chamber will be formed by a combination of anchored chamber walls and gravity-type gate blocks, with a reinforced concrete floor strut.

4. Of particular interest for the purpose of this discussion are the 24 "anchored wall" monoliths forming the center portion of the lock chamber. These monoliths differ from the conventional gravity-type monoliths in that tendons engaging existing rock account in part for the stability of the walls. The configuration of the walls and the number of anchors (tendons) necessary to provide stability vary from monolith to monolith. The number and locations of the anchors in each lock monolith were established by a stability analysis computer program developed by the Nashville District's Design Branch.** To insure that the assumptions for design which were incorporated into the stability program were

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* A table of factors for converting inch-pound units of measurement to metric (SI) units is presented on page 4.

** For detailed information on the stability analysis program for anchored lock wall monoliths, and other considerations related to the design of the anchors, contact the Civil/Structural Section, Nashville District, ORNED-D.

valid, it was decided that a typical anchored wall monolith design would be checked using the finite element method.

5. The wall section chosen for investigation by the finite element method is shown in Plate 1. The monolith is 131 ft high with a horizontal base width of 38.25 ft at el 293.0.* The 4 vertical on 1 horizontal backslope extends to the top of rock creating a large horizontal shelf from which the relatively thin "stem" portion of the lock wall projects to el 424.0. A reinforced concrete floor strut anchored to the rock foundation provides stability against sliding toward the center line. Two rows of three prestressed tendons (six tendons total) are provided for the stability of the 41-ft-long monolith against overturning toward the center line of the lock. Each row consists of a central tendon flanked on each side by outer tendons at a spacing of 13 ft. The rock comprising the structure's foundation is stratified.

Loading Cases

6. Five loading cases were considered in the analysis of the anchored wall monolith (see Plate 2). A description of each load case follows:

a. Case 1:

Chamber pool at el 330.0.
Line pull of 100 kips per monolith at el 340.0.
Top of rockfill at el 409.5, stiffness $K = 0.4$.
Groundwater at el 372.0.
Full uplift.
Tendons stressed at 694 kips per tendon.

b. Case 1 NT. Same as case 1 above except with no tendons.

c. Case 2:

Chamber pool at el 414.0.
Top of rockfill at el 409.5, $K = 0.4$.
Groundwater at el 330.0.

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* Elevations (el) cited herein are in feet referenced to mean sea level.

Full uplift.

Tendons stressed at 694 kips per tendon.

d. Case 2 NT. Same as Case 2 above except with no tendons.

e. Construction condition:

No rockfill placed.

No water in chamber.

No uplift.

Tendons stressed st 694 kips per tendon.

Results of the anlyses of these loading cases are discussed in following sections.

Finite Element Model

7. Two main reasons dictated the use of the finite element method in the analysis of the anchored wall monolith. First, the anchored wall with its concrete-foundation-tendon interaction characteristics poses a formidable challenge to analysis by classical methods. Second, alternative methods of analysis such as scale model testing require such operations as casting, machining, placing strain gages, loading, and interpretation of the strain gage data and other data observed. Much care must be exercised in all of these steps to obtain valid results. Often, therefore, approximate analytical methods offer the potential of "reasonable results" in a shorter period of time and at far less expense.

8. One such approximate analytical technique for the analysis of elastic bodies that has developed significantly in recent years with the advancement of computer technology, and that has come to the forefront of structural engineering as an analytical tool, is the finite element method. General-purpose finite element computer programs are readily available to the engineer and offer a relatively easy way of analyzing complex structures.

9. The finite element method was selected as the only viable analysis technique for determining the state of stress in the anchored wall monolith and its surrounding foundation. The finite element method can handle the complex geometry of the structure, hydrostatic pressure

and applied loads, the posttensioning of the tendons, multiple materials, and the attendant boundary conditions. Several commercially available finite element programs (frequently called codes) were considered, each having its own capabilities, limitations, and accessibilities. It was decided that, for the anchored wall problem, the ANSYS finite element code included several capabilities which made it attractive for use in this investigation. Using known physical and material properties for the elements comprising the structure, special boundary elements contained in the ANSYS formulation can be employed to approximate the interaction at material interfaces. The advantage of using this type of element is that the structure and the stratified rock foundation can be modeled with the only rigid restraint boundary conditions imposed some distance away from expected areas of stress concentrations. This method of modeling is thought to be less of an approximation for this class of problem than the alternative technique of modeling the concrete structure only and applying assumed boundary constraints and/or linear elastic springs to represent the effect of the foundation. Also, by modeling the foundation with finite elements, the stresses within the foundation are yielded for examination. For use in modeling the tendons are the ANSYS two-dimensional spar elements which may be "prestressed" by specifying a coefficient of thermal expansion for the element and assigning a negative temperature to the end nodes. These same elements were also used to model the grouted portion of the anchors. Plane strain quadrilateral elements were used in the modeling of the concrete structure and the rock foundation.

10. A description of the ANSYS general-purpose structural analysis program is given in the "ANSYS Engineering Analysis System User's Guide," by DeSalvo and Swanson. The following is a brief description of the element types used in modeling the anchored wall.

STIF42, two-dimensional
isoparametric solid element

11. This element, a quadrilateral described by four nodal points with two translational degrees of freedom per node, is used to model the solid portion of the structure; namely, the concrete and rock. For the

anchored wall application, the element was used as a linear plane strain element including incompatible displacement shapes; thus, the element is formulated such that values of stress and strain are calculated at Gaussian integration points located in a three by three lattice within the element. Although the stresses at the integration points are available, only the stresses at the centroid of each element were requested to avoid excess and cumbersome computer printout. This is justified from the standpoint that all stresses are taken into account in the post-processing step of producing stress contour plots, with the results being far more easily visualized. These plots are included in this report in Appendix A.

12. Several considerations prompted the use of a plane strain, two-dimensional idealization:

a. The stability program used to design the wall anchors analyzes a 1-ft strip, and the primary reason for doing the finite element analysis is to check the design yielded by the stability program.

b. The tendons on monolith R-22 are spaced uniformly and symmetrically making them readily adaptable to plane strain modeling.

c. Loading on monolith R-22 is symmetrical.

d. The Nashville District has prior experience modeling conventional lock wall monoliths in plane strain.

e. Time constraints and the cost of obtaining the analyses make a plane strain approach more attractive than the alternative, full three-dimensional modeling.

STIF1, two-dimensional spar element

13. This element is used in modeling the tendons, both the stressed length and the grounted zone. The element is described by two nodes, with two translational degrees of freedom at each node. It is capable of uniaxial tension or compression (bending is not considered). Only the elements representing the stressed (nongrouted) length of the tendons are loaded by assigning a negative temperature at the end nodes. The elements making up the zones of anchorage are assigned a coefficient of thermal expansion equal to zero and are therefore unaffected by the nodal temperatures.

14. Since the structure is analyzed in plane strain, the properties of the STIF1 spar elements must be selected so that the effect of the tendons on the behavior of the model is commensurate with that of the STIF42 solid elements. In this instance, behavioral compatibility is achieved by assigning a cross-sectional area to the STIF1 elements equal to the sum of the actual tendon areas in a particular row of tendons dividied by the 41-ft length of the monolith (see Table 1).

STIF12, two-
dimensional interface element

15. Used to describe the interface between two surface planes, this nonlinear element is capable of allowing gapping or relative slipping of the surfaces it represents. The element is formulated so that, in the direction normal to the surfaces, only compression is supported, and, in the tangential direction, Coulomb frictional shear is developed. The element is described by two nodes, which may be coincident, and an angle θ measured from the global X-axis which establishes the orientation of the interface plane. A value of stiffness K is assigned to the interface element, the magnitude of which is determined by the stiffness $(AE)/L$ of adjacent quadrilateral elements. An initial displacement interference (or an initial gap) is specified for the first iteration. Only one material property is designated, the coefficient of friction μ at the interface.

16. After the first iteration, the force-deflection characteristics of the interface element can vary with the status of the element. For example, if an interface is open, no stiffness is associated with this element for this iteration. A compressive normal force ($F_n < 0$) dictates that the interface is closed and the stiffness K is used in the normal direction as a linear spring. In the tangential (sliding) direction with $F_n < 0$, there are two possibilities for the force-deflection relationship. If $F_n < 0$ and the sliding frictional force $F_s \leq \mu|F_n|$, there is no sliding at the interface (frictional resistance is not overcome), and the stiffness K is used in the tangential direction in a linear spring relationship. However, if $F_n < 0$ and $F_s > \mu|F_n|$, frictional resistance has been overcome and sliding occurs.

In this case, there is no stiffness associated with this element in the tangential direction for this iteration, but the constant force $\mu|F_n|$ is used to oppose motion.

17. A view of the finite element model is shown in Plate 3. The number in the center of each quadrilateral element identifies the element by its material type. Material properties associated with the element material identifiers are outlined in Table 1. The material properties of the rock strata were obtained from "Bay Springs Lock Design Memorandum No. N-12, Appendix IV, Geotechnical Analysis and Test Data." In addition to the labeled quadrilateral elements, the physical interfaces described by the ANSYS boundary interface elements are denoted in Plate 3 by a heavy line and the notation "interface plane."

18. Also shown are the spar elements forming the anchorage system of the monolith. The stressed length portions of the tendons are drawn as lines connecting a concrete nodal point with a rock nodal point and are labeled "anchors." The grouted portions of the tendons are shown as a heavier line connecting rock nodal points and are denoted "zone of anchorage" in Plate 3.

Finite Element Analysis Results

19. Solutions for the finite element model and load cases described above were obtained by running the ANSYS program on the CDC equipment of Boeing Computer Service. Graphical results were reviewed interactively on a Tektronix 4014 terminal, and, if the plot was satisfactory, the plotting information was then transferred to magnetic tape and plotted on the CALCOMP equipment in the Nashville District's ADP center. These plots were reproduced and are included in Appendix A. The plots in Appendix A are assigned plate numbers so that the loading condition (as defined above and shown in Plate 2) is identified in addition to the sequence number of the plot within that load case. For example, Plate 1-3 shows the third plot for load case 1. Further identification for each plot is contained in the title located at the bottom of the page.

20. Deflected shape plots, computer-generated plots showing the

finite element predicted displacements, are included in Appendix A for the various load cases. Plates 1-1, 1 NT-1, 2-1, 2 NT-1, and C-1 depict the deformed shape of the entire model under the prescribed loads, while Plates 1-6, 1 NT-6, 2-6, 2 NT-6, and C-6 detail the appearance of the concrete lock wall structure under load. Study of these plots shows that the finite element analysis predicts a maximum deflection (vertical) of just under 1.1 in. for all load cases except the construction condition. These deflections seem to compare favorably with the calculated settlement of approximately 1.97 in. predicted by geotechnical analyses. The maximum settlement for the construction condition is 0.65 in. This smaller settlement is due to the fact that for the construction condition the chamber is empty and no backfill is in place, and therefore the gravity load on the structure and foundation is less than that for the other cases.

21. It is interesting to compare the deflected shapes for the various load cases to get a feel for the effect of the tendons on the behavior of the structure. Plates 1-1 and 1 NT-1 serve to illustrate this point. In Plate 1-1 (lower pool in chamber with the tendons stressed), it can be seen that the structure settles evenly into the foundation, maintaining contact with the 4V on 1H backslope. Because of a slight clockwise rotation of the base and bending in the stem due to the backfill and high water table, there is very little horizontal deflection. Therefore, if the maximum vertical deflection of 1.06 in. can be attributed primarily to the effect of gravity which is always present (which seems to be a fair assumption considering that the maximum vertical deflection for case 2 with a full chamber pool and tendons stressed is 1.067 in. (see Plate 2-1)), the FEM analysis predicts that for case 1 there is very little movement of the wall.

22. Plate 1 NT-1 shows the deflected shape for case 1 NT (the same loads as case 1 except without tendons). Here one immediately notices that the structure loses contact with the 4V on 1H backslope above the H_e member bedding plane. Also, there is now a counterclockwise rotation at the base which, combined with the bending in the stem, produces a net horizontal deflection toward the center line of the lock

of 0.41 in. It should be mentioned that the nature of this analysis (full gravity "turn-on") predicts a settlement at the top of the lock wall approximately equal to the settlement at the base. In actuality, a large portion of the settlement at the base will occur gradually as the structure is brought up in lifts, and the top of the lock will be poured at its proper elevation by compensating the height of one or more of the lifts.

23. It can be seen that the effect of the tendons on the lock wall for case 1 is to make the structure "better behaved" by eliminating the gapping on the backface and minimizing the horizontal movement of the wall, especially in the upper stem area. Although the finite element analysis of case 1 NT indicates the structure is stable, the tendency for the interface between the structure and the rock backface to open is not desirable; indeed, it is unacceptable in view that the application of the stressed tendons eliminates this occurrence.

24. A comparison of Plates 2-1 and 2 NT-1 (i.e., upper pool in chamber, with and without tendons, respectively), shows very little difference in the appearance of the two deflected shapes. For that matter, there is very little difference in the state of stress between the two cases, as can be determined by comparing the stress contour plots for the two load cases. The reason for the similarity between the two chamber-full cases is that the net horizontal load is directed toward the rock backface, thereby minimizing the effect of tendons on the behavior of the structure.

25. An interesting phenomenon predicted by the finite element analysis, which is readily seen in the deflected shape plots, is the thrusting upward of the H_e foundation member near the center of the lock chamber. This is a Poisson's effect caused by the weight of the structure acting down on the H_e material which has a relatively high Poisson's ratio of 0.41 (Table 1). Although this effect is present in each load case, it is most prominent in the construction condition (Plate C-1) which has no water in the lock chamber to counteract the upheaval. Study of Plates C-1, 1-1, and 2-1 reveals the relative movement of the lock floor foundation and strut as the lock is filled. In

Plate 1-1, the chamber pool is at el 330.0 (lower pool) and the net deflection of the strut is still above its nonloaded position, although the weight of the water in the lock chamber has pushed it lower than in the construction condition. With the water in the chamber at the normal upper pool level (el 414.0) the Poisson's effect in the rock is still in evidence (see Plate 2-1), though diminished, and the net vertical deflection of the strut is below the nonloaded position (dotted line in Plate 2-1). Neglecting the differences in lateral load (due to line pull and water table), the numerical difference between the position of the top of the strut at the center line of the lock chamber in Plates 1-1 and 2-1 represents the maximum fluctuation (0.53 in.) of the struct/foundation as the lock is emptied and filled. Similarly, the maximum horizontal fluctuation at the top of the lock wall due to lockages at normal upper and lower pools is 0.075 in. as predicted by the finite element analysis (case 1 and case 2). The maximum predicted horizontal fluctuation at the top of the lock for cases 1 NT and 2 NT (no tendons) is 0.59 in.

26. An area of concern in the anchored lock wall scheme was meeting the base pressure criteria. Compared to the base width of a typical gravity-type lock wall monolith, the 38.25 ft of base afforded by the anchored wall is quite small, thereby creating the potential for excessively high base pressures. Since the finite element analysis determines the stresses at the centroid of each element, a picture of the base pressures yielded by a finite element run can be visualized.

27. There are several approaches that may be taken in determining pressures at the base. Figures 1-3 illustrate the range of pressures by the various methods. The base pressure diagrams in Figures 1 and 2 were obtained using the same basic approach, but considering two separate materials. The ANSYS finite element program has, as a part of the total package, extensive postprocessing flexibility. For example, in the investigation of base pressures acting on the 38.25-ft horizontal base, the component of stress is considered which best describes the base pressure condition, which in this case is the Y-normal stress. However, if the value of the Y-normal stress at the centroid of each adjacent element

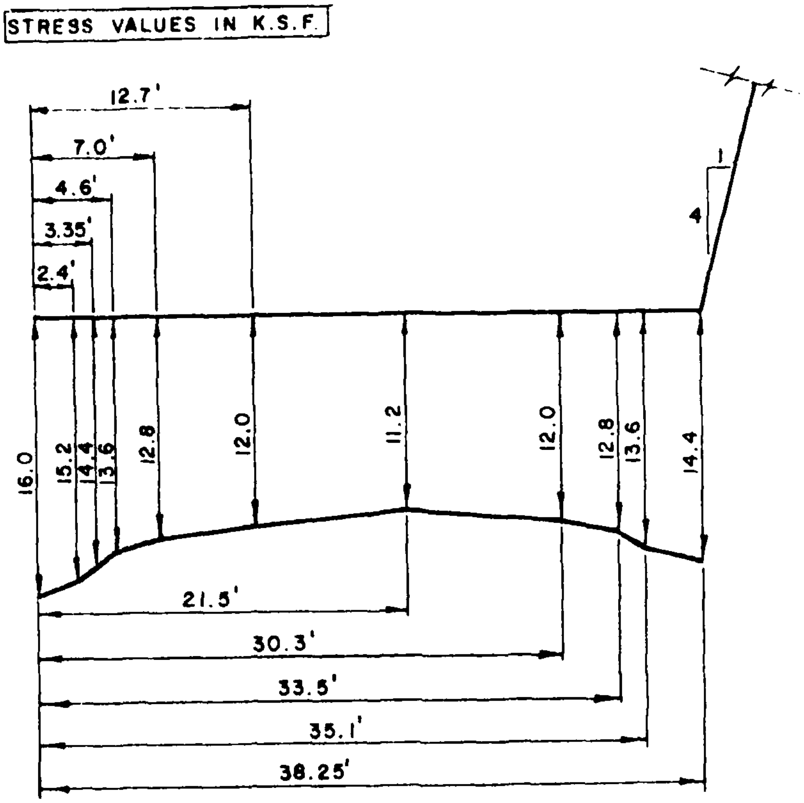


Figure 1. Foundation base pressure diagram for load case 1 (H_e member)

along the base is used to plot the base pressure diagram, only a vague approximation of the pressures is achieved because the value of the stress is valid only at the centroid of the element. A closer approximatior could possibly be obtained by considering the Y-normal stresses a few elements away from the element adjacent to the base and extrapolating a stress by some means (french curve, linear interpolation, etc.) to the baseline. But this method is subjective (how many elements away from the base should be considered?) and rather unscientific. Fortunately, the ANSYS finite element program has a stress contouring algorithm built into it which automatically takes into account not only the centroidal stresses, but also the stresses at integration points of all elements within a specified range (a default range was used) to determine the contour pattern and the contour values. These contour lines, which

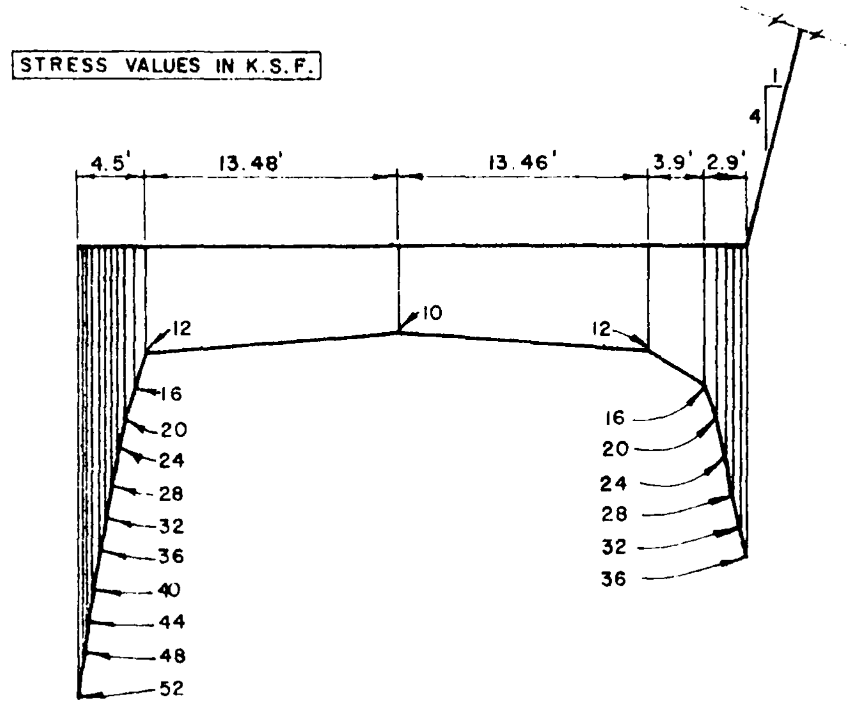


Figure 2. Structure base pressure diagram
for load case 1 (concrete)

extend to the boundary of the model, can then be used to plot a stress diagram. A diagram of the Y-normal stresses at the base can be plotted by scaling the value of each contour at the position it contacts the base.

28. The accuracy of this type of diagram is roughly the same as the stress interval between contours. Figure 2, for example, is a base pressure diagram obtained from the Y-normal stress contour plot shown in Plate 1-8 of Appendix A. The interval between stress contours in this case is 4 ksf. Therefore, on the base pressure diagram in Figure 2, a value of 20 ksf may be plotted where contour number 10 meets the base, but the actual value of the Y-normal stress at that point is somewhere within the range of the next higher and lower contour values, or 20 ksf + 4 ksf . In areas of high stress concentrations, such as on the extreme left and right sides of the base in Figure 2 and Plate 1-8, the assumption of a linear variance between contours becomes less approximate.

29. As mentioned above, the base pressure diagrams in Figures 1

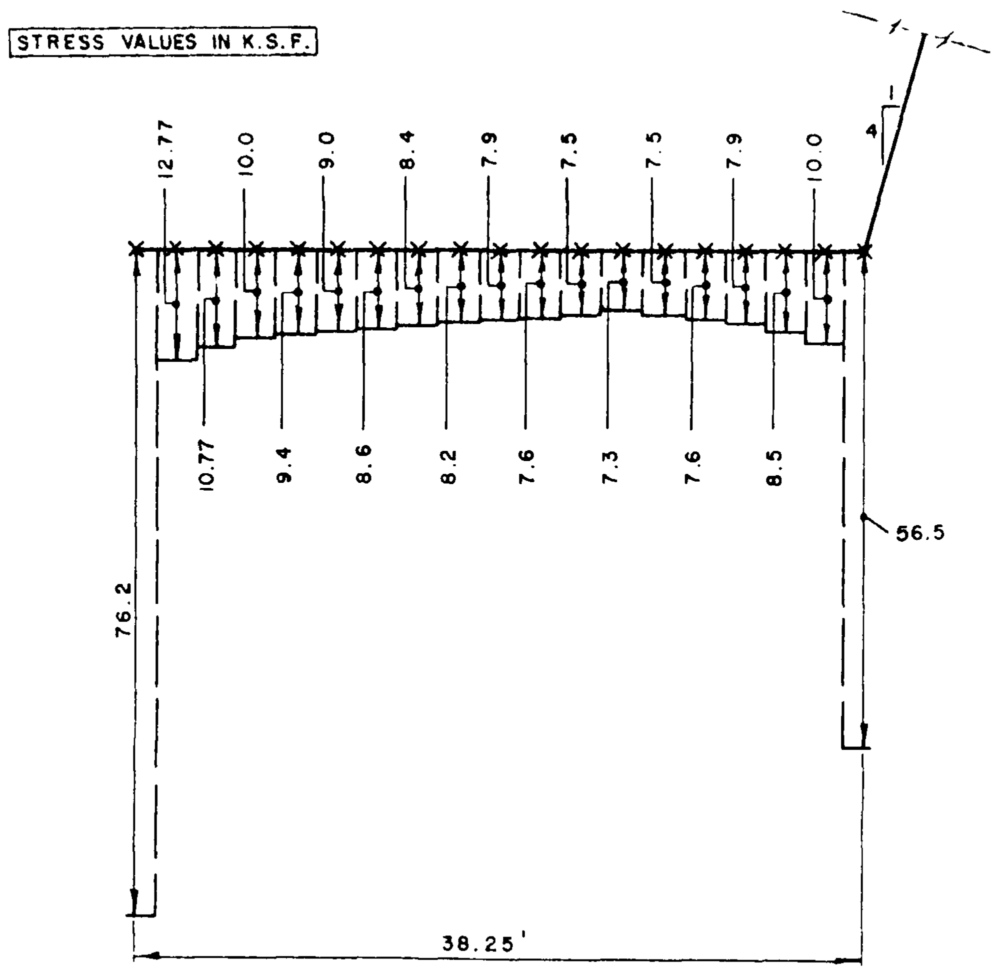


Figure 3. Boundary spring forces distributed across the base

and 2 were both drawn using the stress contouring approach. The difference in appearance between the two diagrams is due to the way in which the ANSYS program determined the contours. The pressure diagram in Figure 2 utilizes stress contours which were developed considering only the stress of elements comprising the concrete lock wall structure, while the diagram in Figure 1 was obtained by contouring only the Y-normal stresses in the H_e member. The obvious question arises: "Since the laws of elasticity state that at a closed interface between two materials the state of stress in the two materials must be identical, why are the two base pressure diagrams different?"

30. To answer this question, one must consider again the means by which the diagrams were obtained. True, the state of stress at the interface is in reality identical in the concrete structure and the rock foundation, and therefore there is only one "true" base pressure diagram. However, it must be kept in mind that a few inches away from the baseline interface, the stresses in the concrete can indeed be quite different from the corresponding stresses in the rock an equal distance away from the base. Plate 1-8 shows that there is a high concentration of Y-normal stress in the concrete structure in the area of the base. These concentrations are due to the presence of the 14- by 14-ft culvert (whose relatively flexible, 5-ft-deep floor slab carries proportionately less of the foundation reaction than the massive sections of the monolith) and the chamber-side toe, which has the highest concentration of Y-normal stress. And these concentrations of high stress, away from the base, nevertheless contribute an effect on the base pressure through the ANSYS contouring algorithm mentioned above (see Figure 1).

31. The stresses in the H_e rock foundation, on the other hand, are unaffected by such unusual geometry as the culvert (rectangular void) and the toe (edge effects). Therefore, when the H_e member Y-normal stresses are contoured (these contour plots are not included in this report), the variance of the stresses along the base is more gradual and the concentrations of Y-normal stress near the ends of the base are not as high as those derived from the concrete elements (see Figure 2). Clearly the "true base pressure" diagram must be bracketed by the two diagrams in Figures 1 and 2, the former being the lower and the latter, the upper bound. More likely than not, the actual base pressures will be closer to the diagrams obtained from the foundation element stresses, since it seems unlikely that the internal stress concentrations of the concrete monolith could actually affect the stress in the rock at the interface. And it is, after all, the rock, and not the concrete, which is the cause of concern in regard to meeting the base pressure requirements. For the sake of comparison, then, only the base pressure diagrams obtained by contouring the Y-normal stress of the H_e foundation

elements are included herein for load cases other than case 1.

32. As expected, the base pressure diagrams for case 2 and case 2 NT (upper pool in chamber) are practically identical, the effect of the tendons on the structural behavior being minimal (see Figures 4 and 5). The finite element analyses indicate that the allowable maximum base pressure of 21 ksf is not exceeded in the rock for these loading conditions.

33. By comparing the base pressure diagrams for case 1 and case 1 NT (Figures 1 and 6, respectively), the effect of the tendons on the minimum chamber pool case can be determined. In this instance, the total base pressure diagrams for the two loading conditions, again, show that in neither case is the maximum allowable pressure for the rock exceeded. However, the tendons do serve to reduce the maximum pressure

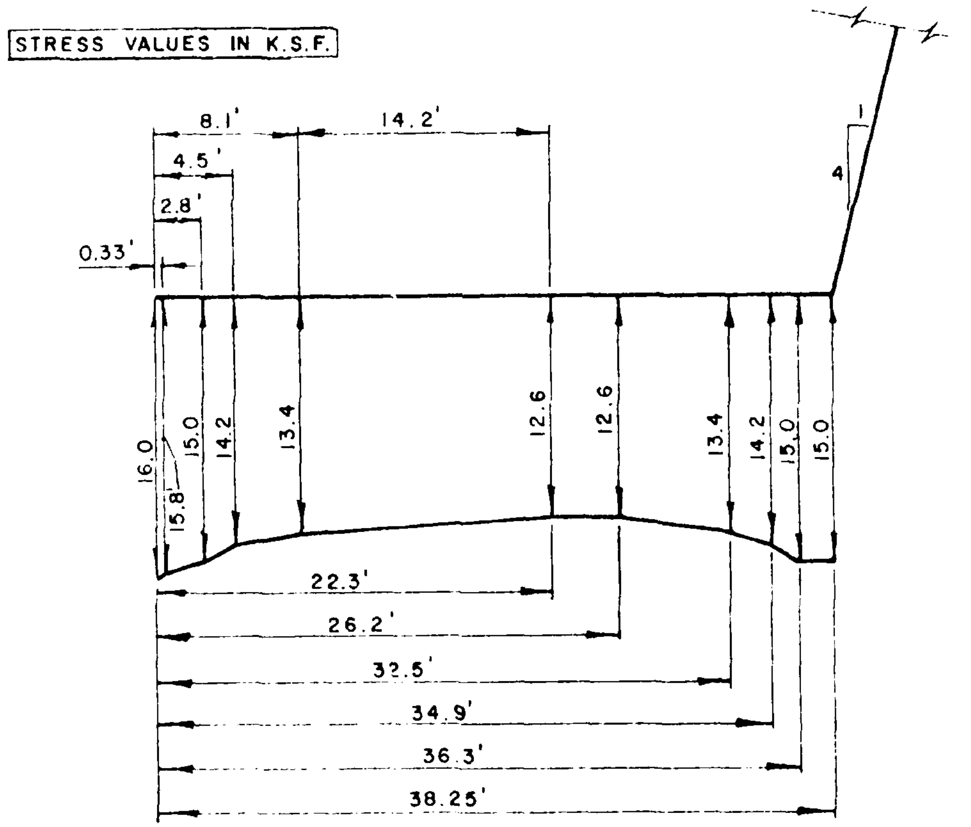


Figure 4. Foundation base pressure diagram for load case 2 (H_e member)

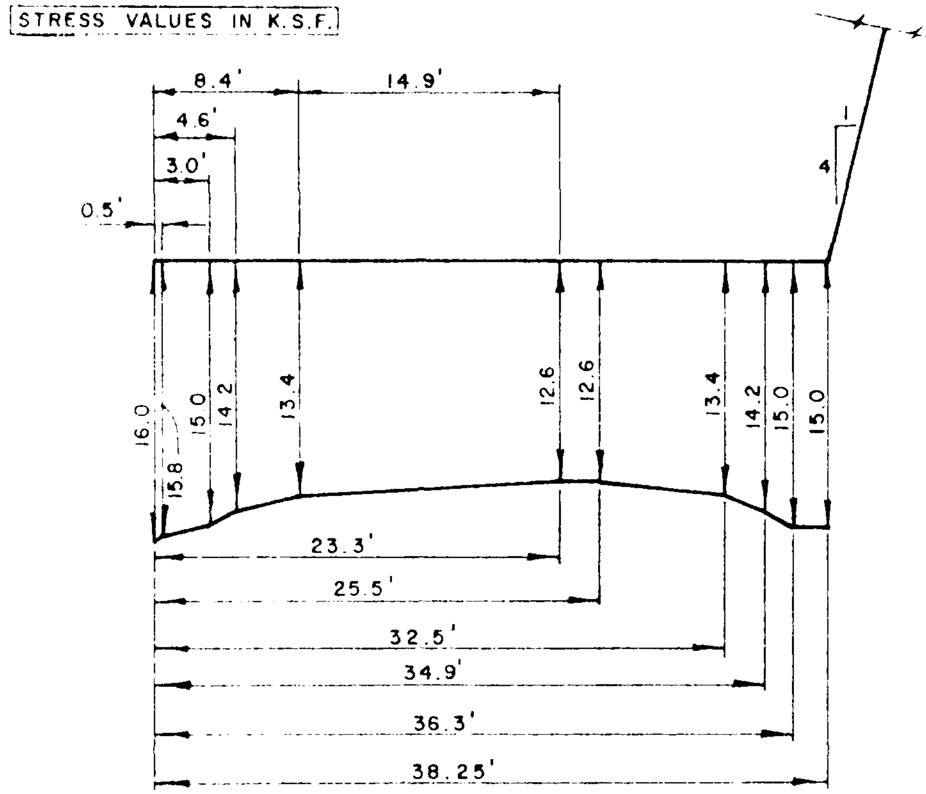


Figure 5. Foundation base pressure diagram for load case 2 NT (H_e member)

acting under the toe of the structure (16 ksf + 0.8 ksf versus 18 ksf + 1.0 ksf). Another effect of the tendons is to reduce the force taken by the strut. The finite element results indicate that with the tendons stressed the force felt by the strut is 226 kips, as opposed to a 286-kip strut force with no tendons. The strut was designed using a 311-kips/ft force.

34. Figures 7 and 8 are diagrams of the X-normal stresses in the rock at the centroid of the elements along the 4V on 1H backslope. These diagrams illustrate the increase of compressive stresses along the cut, especially in the H_a and H_b stratum, with the tendons stressed. This effect correlates directly with the gapping observed in Plate 1 NT-1 with the tendons unstressed. Apparently, then, the finite element analyses indicate that the presence of the stressed tendons not only

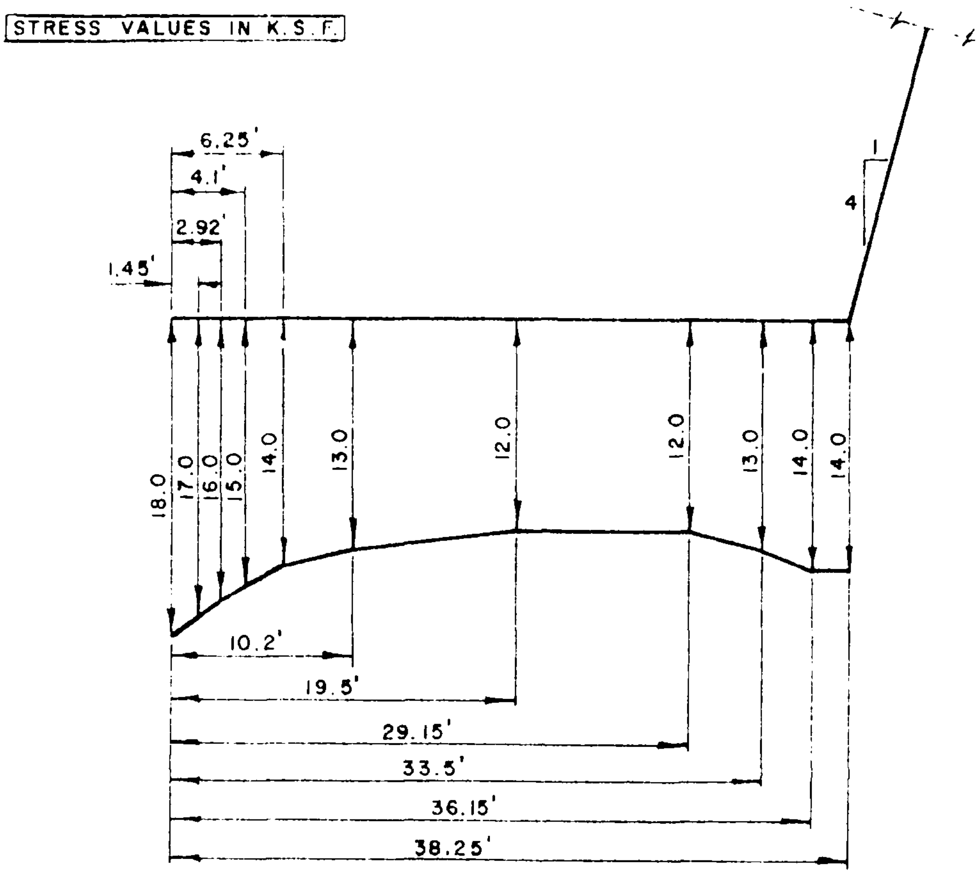


Figure 6. Foundation base pressure diagram for load case 1 NT (H_e member)

eliminates the gapping tendency at the 4V on 1H backslope, but also produces a slight increase in the X-normal compressive forces along that face. Therefore, the finite element results show that, from the standpoint of controlling the tendency for a gap to occur at the backface between the concrete monolith and the rock, the size and number of the tendons used and the tendon forces are adequate.

Summary and Conclusions

35. The finite element analysis results indicate a "better behaved" structure with respect to deflections when the effects of the stressed

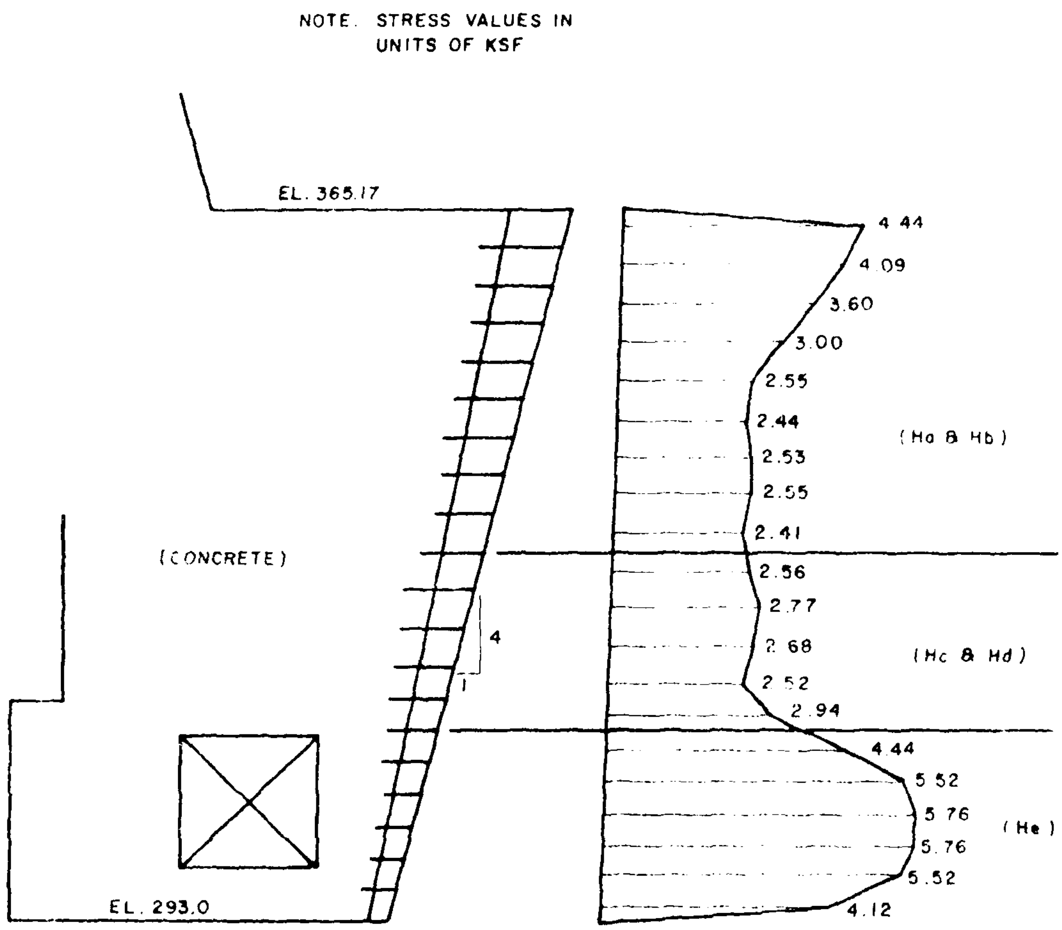


Figure 7. Compressive stresses in rock backslope, X-normal slope, load case 1 (tendons stressed)

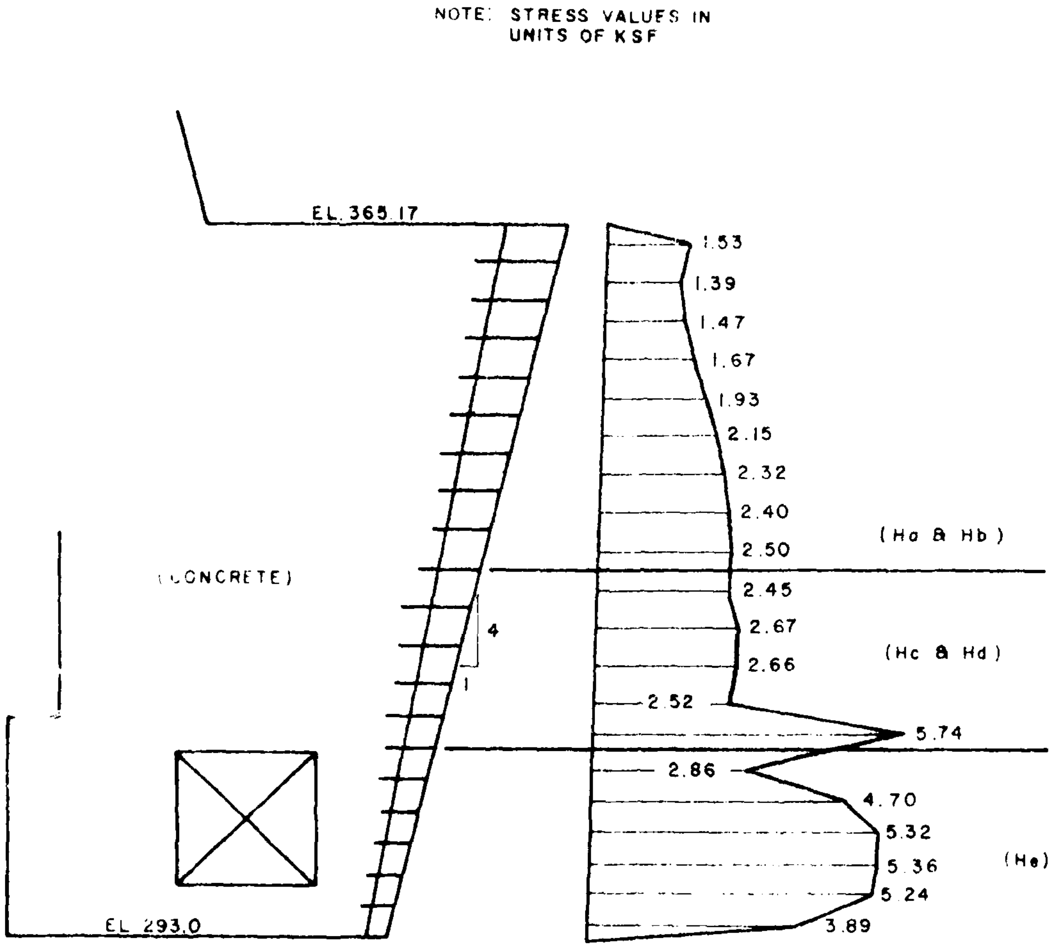


Figure 8. Compressive stresses in rock backslope, X-normal slope, load case 1 NT (no tendons)

tendons are included. A gap is formed at the 4V on 1H concrete-rock interface for the lower chamber pool case with no tendons. The application of the tendons, in this case, eliminates the gap and induces a small additional compressive force in the rock at the backface. The maximum lateral fluctuation at the top of the lock between high and low chamber pools is less with tendons than without (0.075 in. as opposed to 0.59 in.).

36. Allowable base pressure criteria were not exceeded by any of the load cases included in this study based upon total base pressure diagrams obtained from contour plots of the Y-normal stresses in the H_e foundation member. These diagrams were used to avoid the influence of internal stress concentrations in the concrete monolith upon the base pressures. The effect of the tendons was most pronounced in the lower chamber pool case in which the maximum base pressure under the toe was reduced from 18 ksf $\pm$ 1.0 ksf to 16 ksf $\pm$ 0.8 ksf .

37. Based on the results of information provided by the finite element analyses of the load cases included in this study of monolith R-22, it can be concluded that the effects of the tendons on the behavior of the structure are beneficial, and that the design assumptions used in the rigid body stability analysis to determine the number and position of the tendons were valid and sufficiently conservative to produce a safe design.

Postscript

38. To enable the reader of this report to intelligently weigh the information provided by this finite element study, a brief mention of some of the ANSYS characteristics is in order:

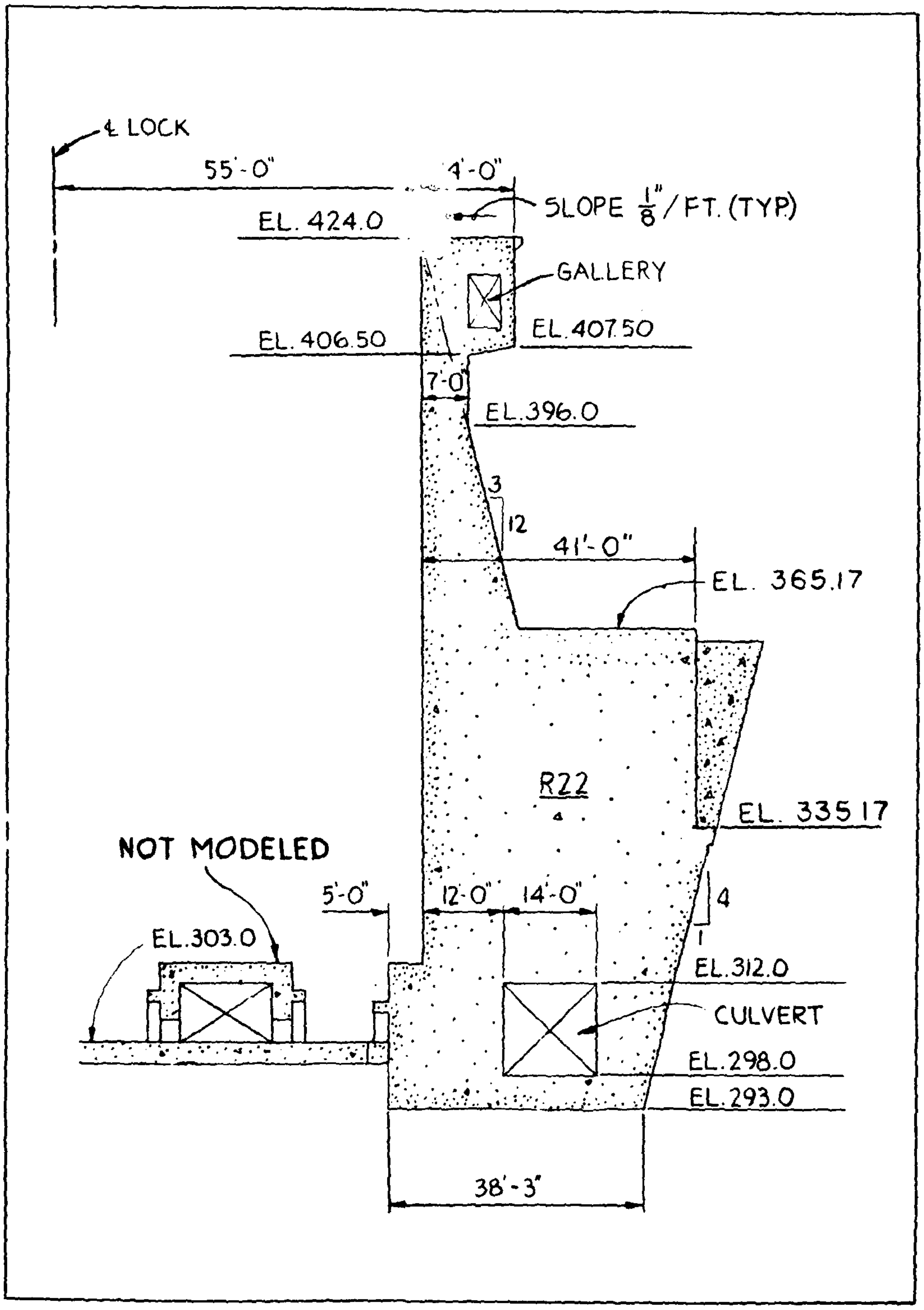
a. Use of the ANSYS interface boundary element (STIF12) makes the final solution "path dependent." That is, the sequence of convergence of the STIF12 elements affects the final results. The nature by which the loads are applied is one factor which determines the path of convergence of the nonlinear boundary element. In this analysis, gravity was "turned on" on the first iteration and the external pressure loads were applied in two load steps of four iterations (maximum) each. It is therefore conceivable that if gravity, for example, had been applied gradually over several iterations the STIF12 convergence path would be altered and the final solution subsequently would be affected.

b. In the same vein, the nature of convergence of the ANSYS STIF12 element must be considered. It is possible to obtain a converged analysis using these elements without ensuring a "static equilibrium" solution. For example, if a STIF12 element is sliding (STAT = ± 2) for two consecutive iterations and the status of every other element also remains unchanged for these iterations, the ANSYS program now interprets this condition as convergence. (A telephone communication with Swanson Analysis System Co., suggested that the ANSYS programmers are investigating this problem). This condition exists on the backface (4V on 1H) boundary between the concrete and rock in nine STIF12 elements in the converged ANSYS solution for case 1. However, case 1 is the loading condition in which it was shown that an effect of the tendons was to keep the 4V on 1H interface closed while inducing a small compressive stress into the rock (it was further concluded that, in view of the small magnitude of the induced compressive stress, the tendons were adequately designed, but not oversized). The sliding status of these nine elements tends to reinforce these hypotheses. What is apparently happening at the backface is that the interface is kept closed by the tendons, but the normal compressive force is so small that not enough frictional resistance can be developed to resist sliding before the ANSYS convergence criteria are met and the iterative analysis is stopped. If this is true, then the magnitude of the forces involved is small and, therefore, it is reasonable to assume that the effect of this convergence discrepancy on the final solution for this load case would be slight.

TABLE I
MATERIAL AND PHYSICAL PROPERTIES

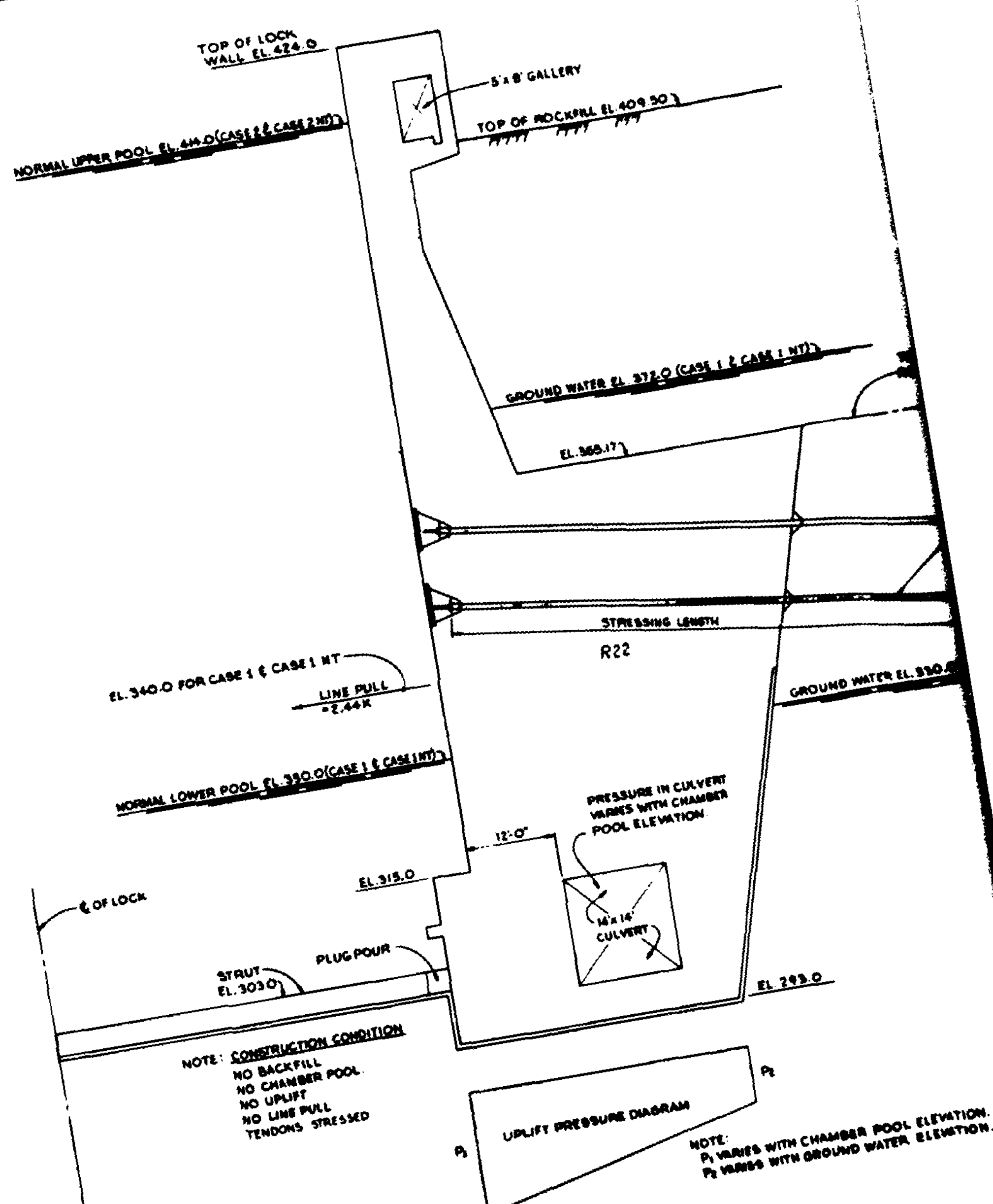
	MATERIAL NUMBER	MOD. OF ELASTICITY, E (KSI)	UNIT WEIGHT, γ (KSF)	POISSONS RATIO, ν	COEFF. OF THERMAL EXPANSION, α	COEFF. OF FRICTION, μ	AREA (FT^2/FT)
SOLID ELEMENTS	1 CONCRETE	3000	0.15	0.25	0	N/A	N/A
	2 (H_a & H_b)	1000 *	0.13	0.20	0	N/A	"
	3 (H_c & H_d)	270 *	0	0.20	0	N/A	"
	4 (H_e)	27 *	0	0.41	0	N/A	"
INTERFACE ELEMENTS	5 (1 — 2)	N/A	N/A	N/A	N/A	0.727	"
	6 (1 — 3)	"	"	"	"	0.466	"
	7 (1 — 4)	"	"	"	"	0.466	"
	8 (1 — 4)	"	"	"	"	0.466	"
	9 (1 — 4)	"	"	"	"	0.466	"
	10 (1 — 1)	"	"	"	"	0.727	"
	11 (2 — 3)	"	"	"	"	0.989	"
	12 (3 — 4)	"	"	"	"	0.319	"
TENDON ELEMENTS	13 (STRESSED)	30000	0	N/A	1.0	N/A	0.00217
	14 (GROUTED)	30000	0	N/A	0	N/A	0.00217

* FOUNDATION MODULUS

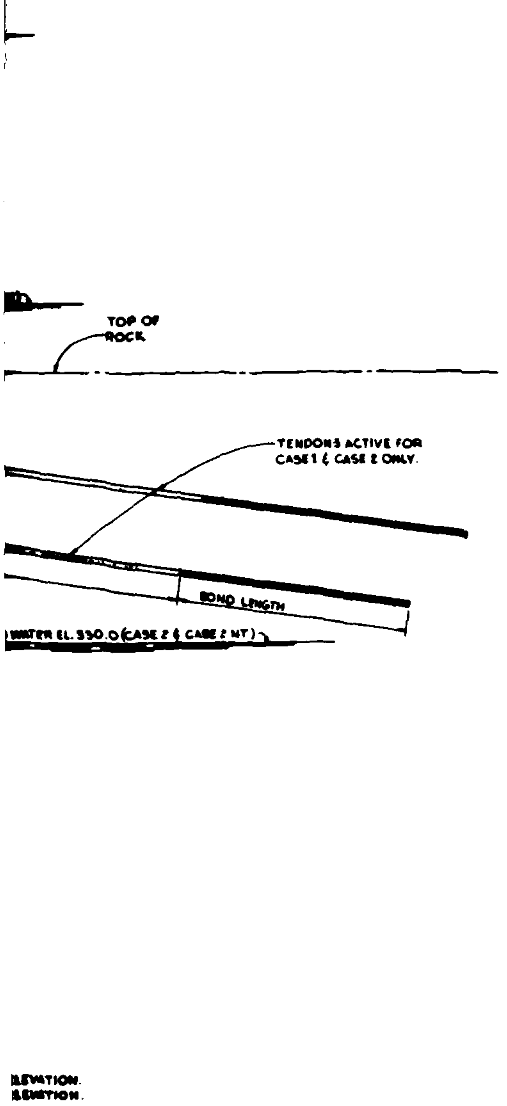


℄ LOCK
55'-0"
4'-0"
SLOPE 1/8"/FT. (TYP.)
EL. 424.0
GALLERY
EL. 407.50
EL. 406.50
7'-0"
EL. 396.0
3
12
41'-0"
EL. 365.17
R22
EL. 335.17
NOT MODELED
5'-0"
12'-0"
14'-0"
4
1
EL. 303.0
EL. 312.0
CULVERT
EL. 298.0
EL. 293.0
38'-3"

PLATE 1



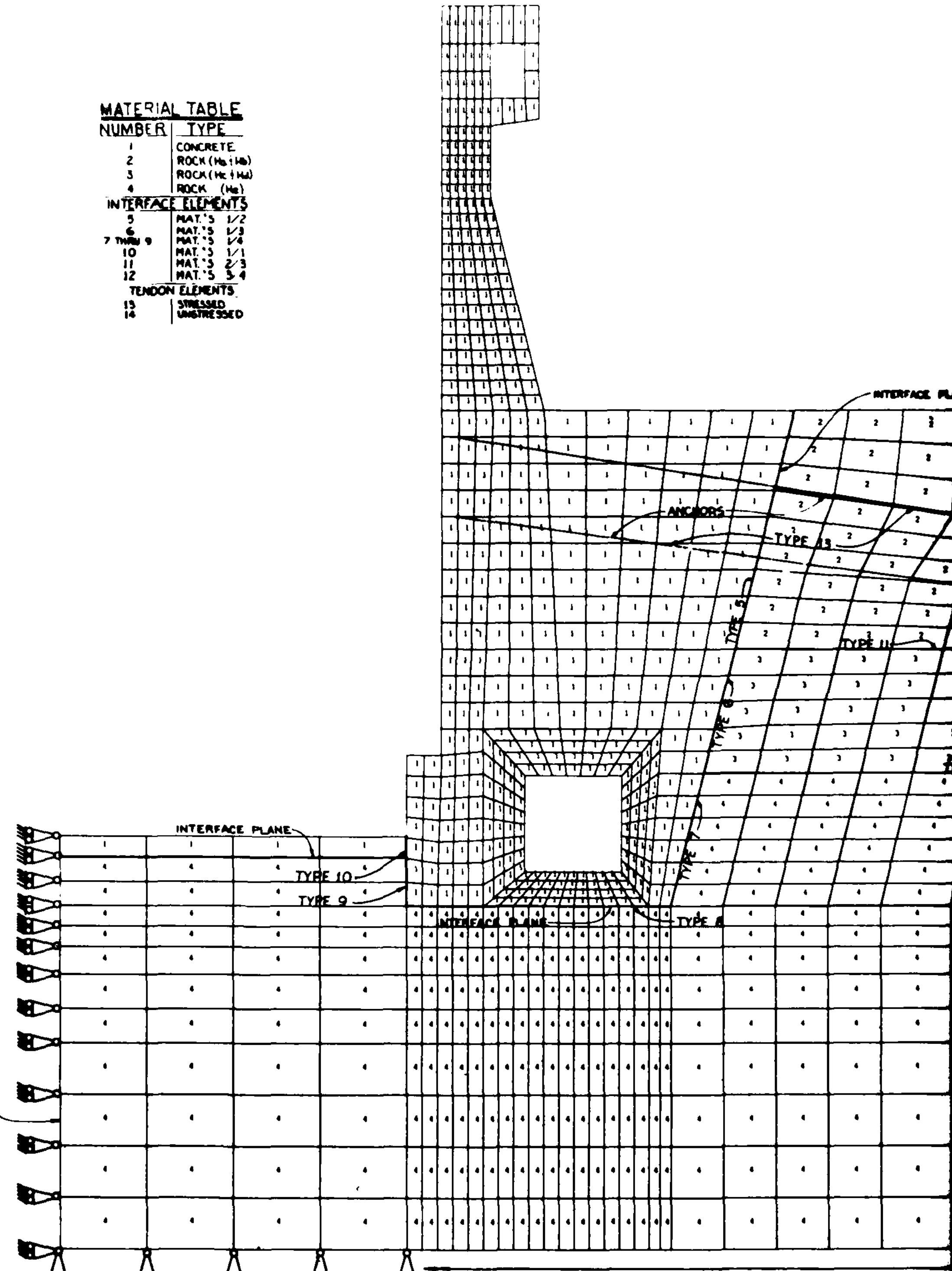
FINITE ELEMENT LOAD CASES



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

MATERIAL TABLE

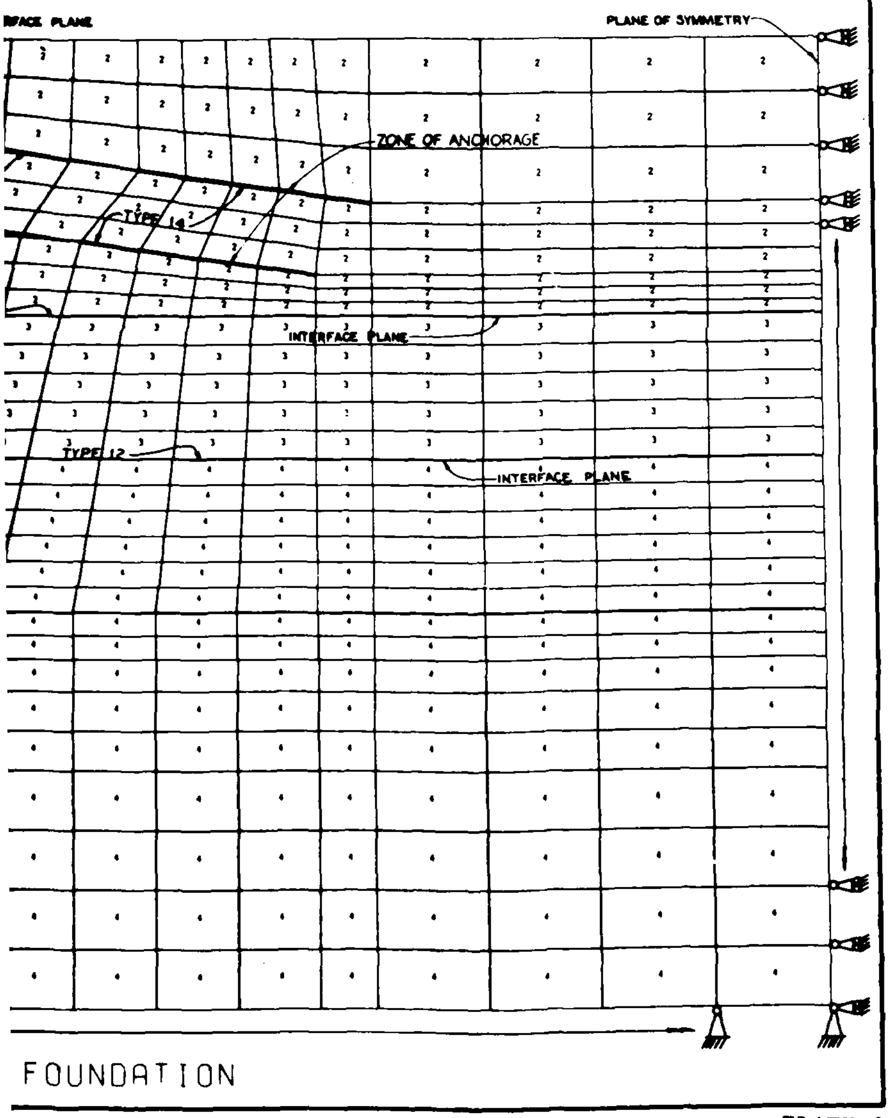
NUMBER	TYPE
1	CONCRETE
2	ROCK (Ha & Hb)
3	ROCK (Hc & Hd)
4	ROCK (He)
INTERFACE ELEMENTS	
5	MAT.'S 1/2
6	MAT.'S 1/3
7 THRU 9	MAT.'S 1/4
10	MAT.'S 1/1
11	MAT.'S 2/3
12	MAT.'S 3/4
TENDON ELEMENTS	
13	STRESSED
14	UNSTRESSED



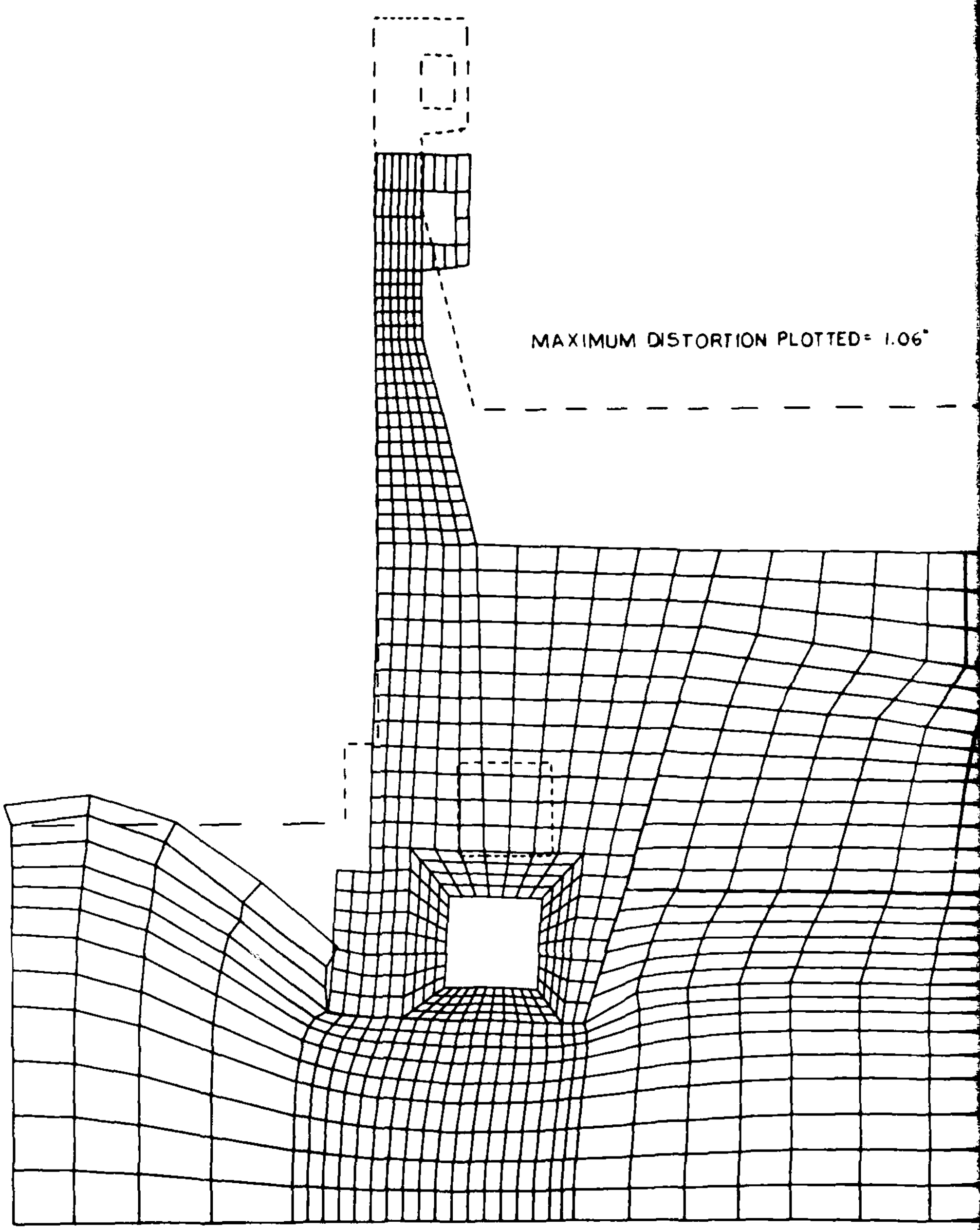
BAY SPRINGS LOCK MONOLITH R-22 --- F.E.M. MODEL INCL. FO

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

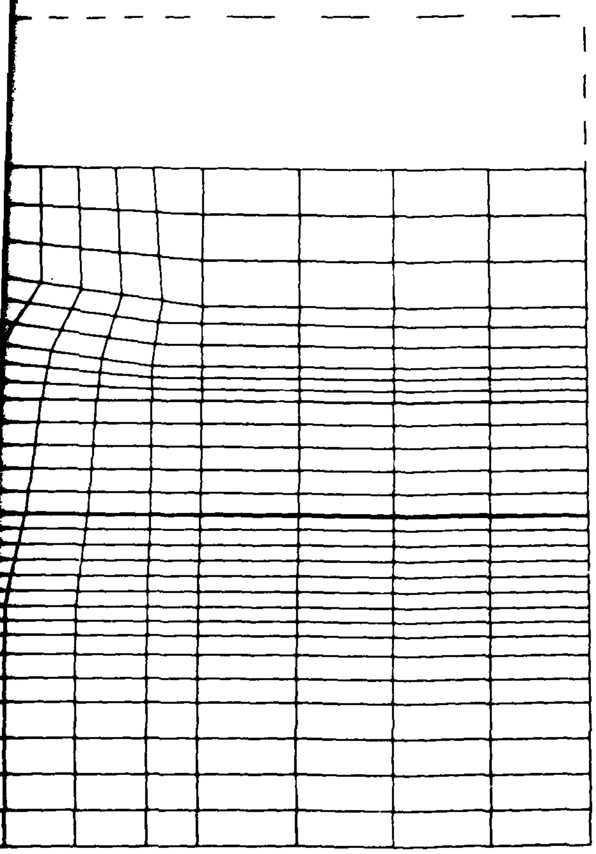
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD



Appendix A: Graphical Results--Computer-Generated Distorted Shapes and Stress Contour Mapping



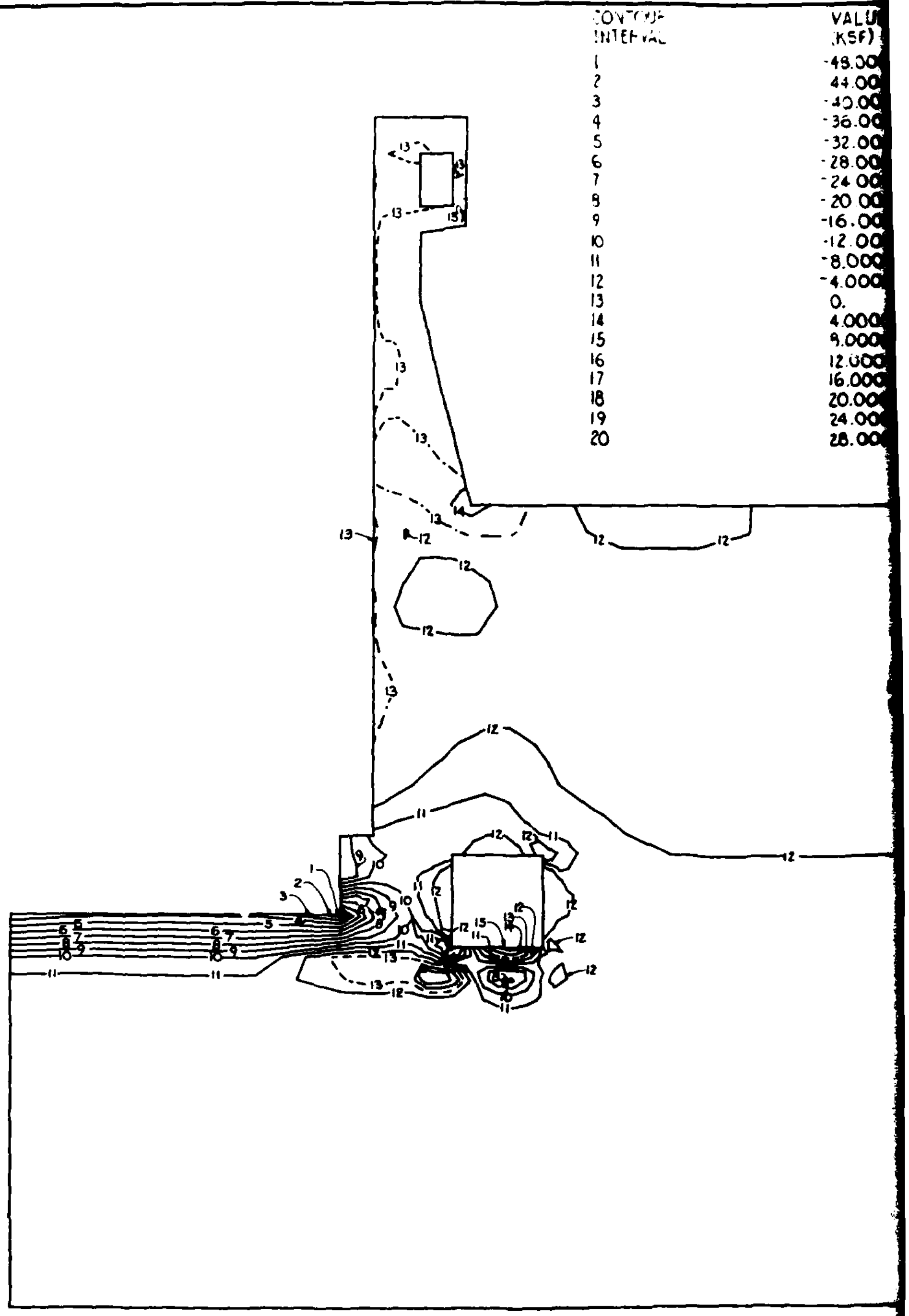
LOWER POOL CASE 1 --- TENDONS STRESSED --- DISTORTED SHAPE



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

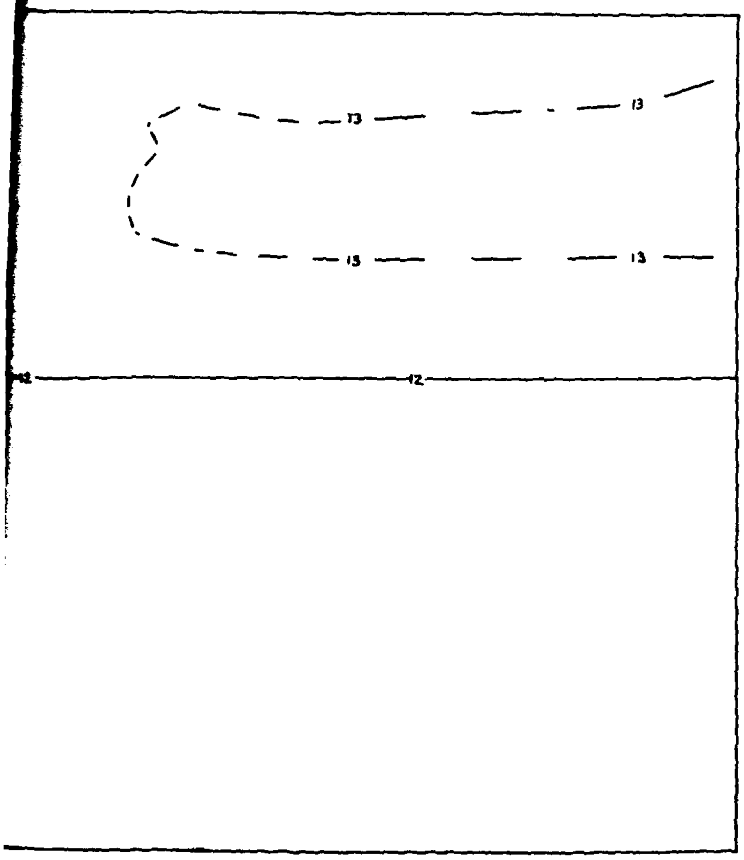
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1-1



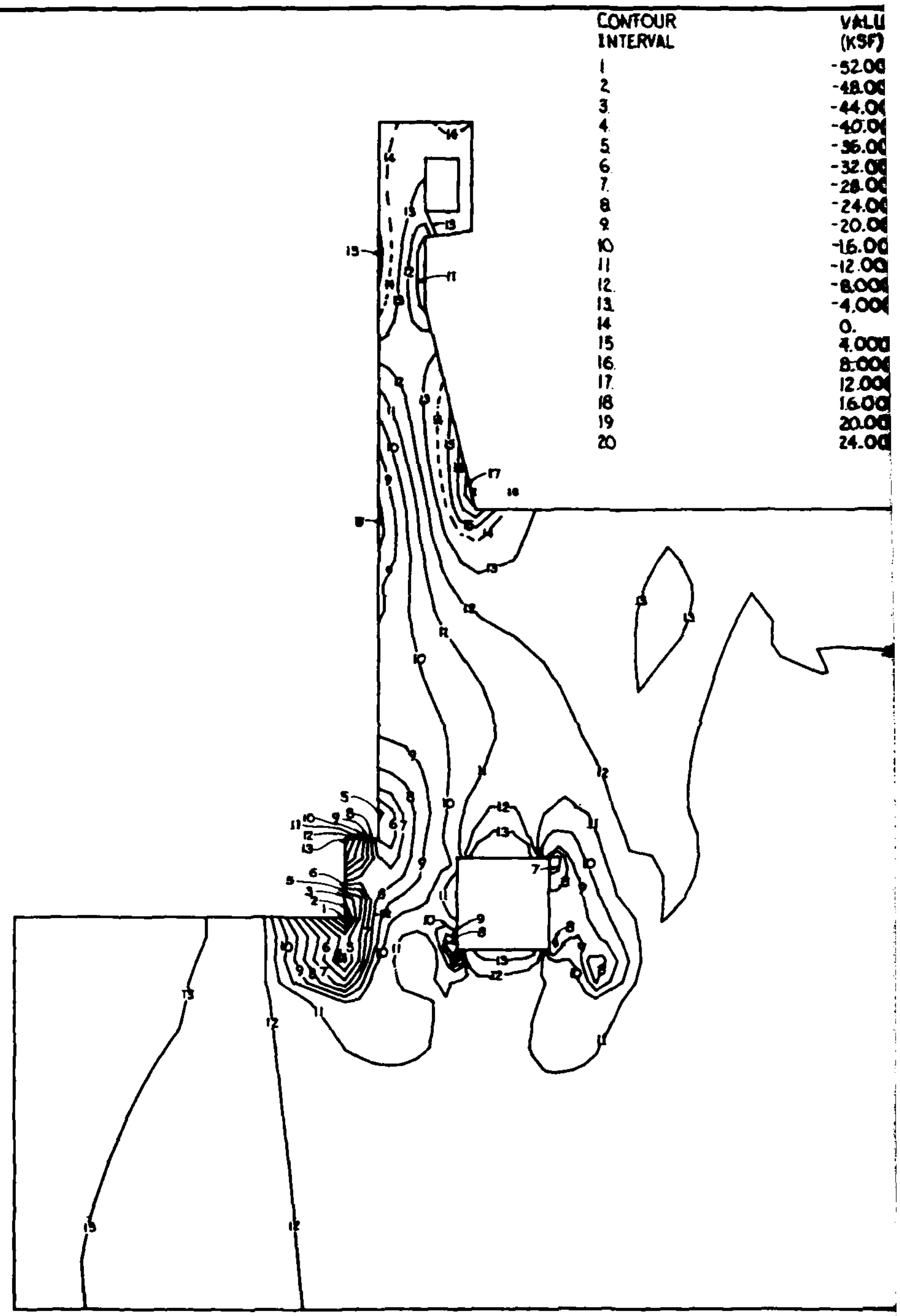
LOWER POOL CASE 1 --- TENDONS STRESSED --- SIGMA X

VALUE
(KSF)
-48.000
-44.000
-40.000
-36.000
-32.000
-28.000
-24.000
-20.000
-16.000
-12.000
-8.0000
-4.0000
0.
4.0000
8.0000
12.000
16.000
20.000
24.000
28.000

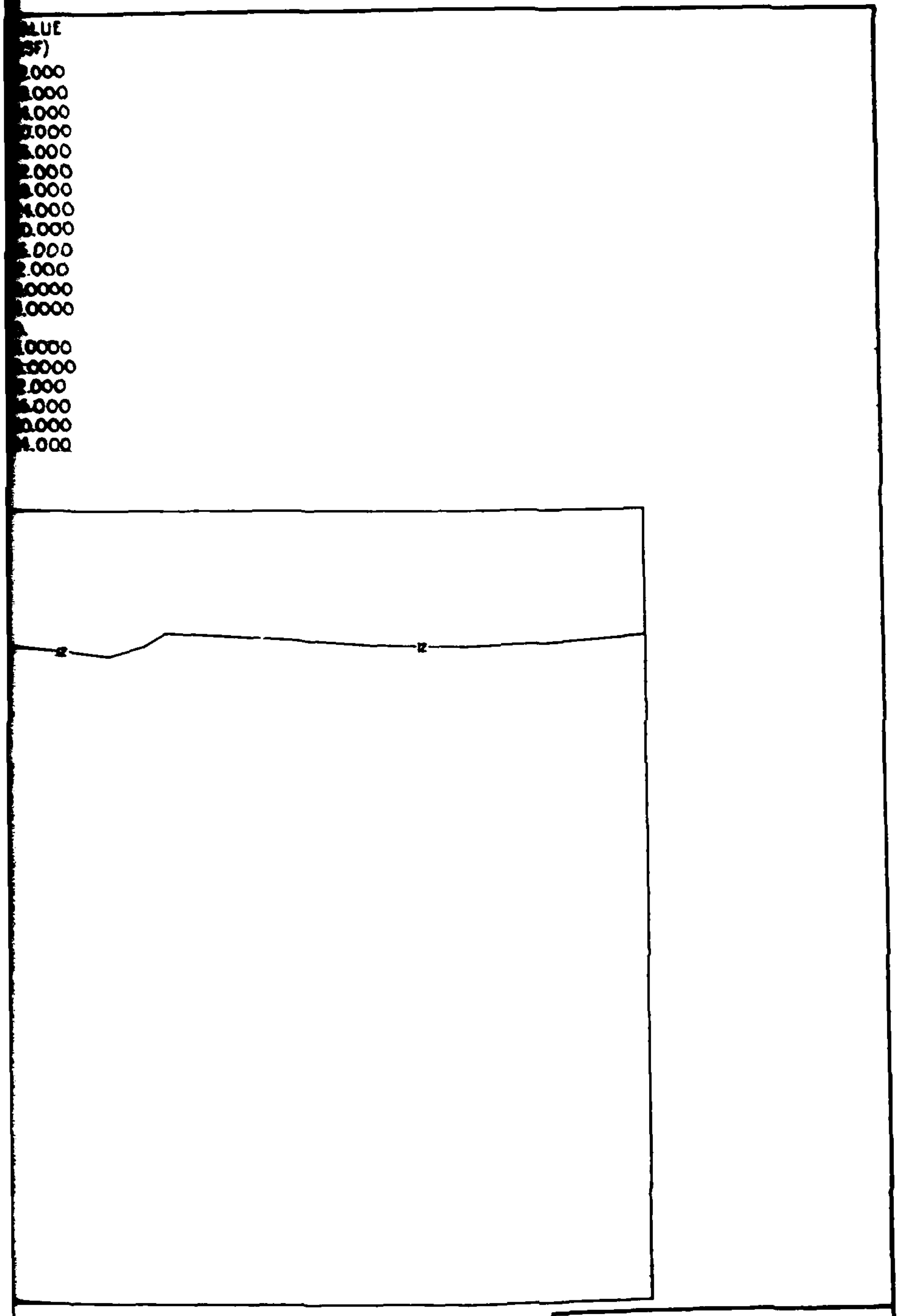


TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

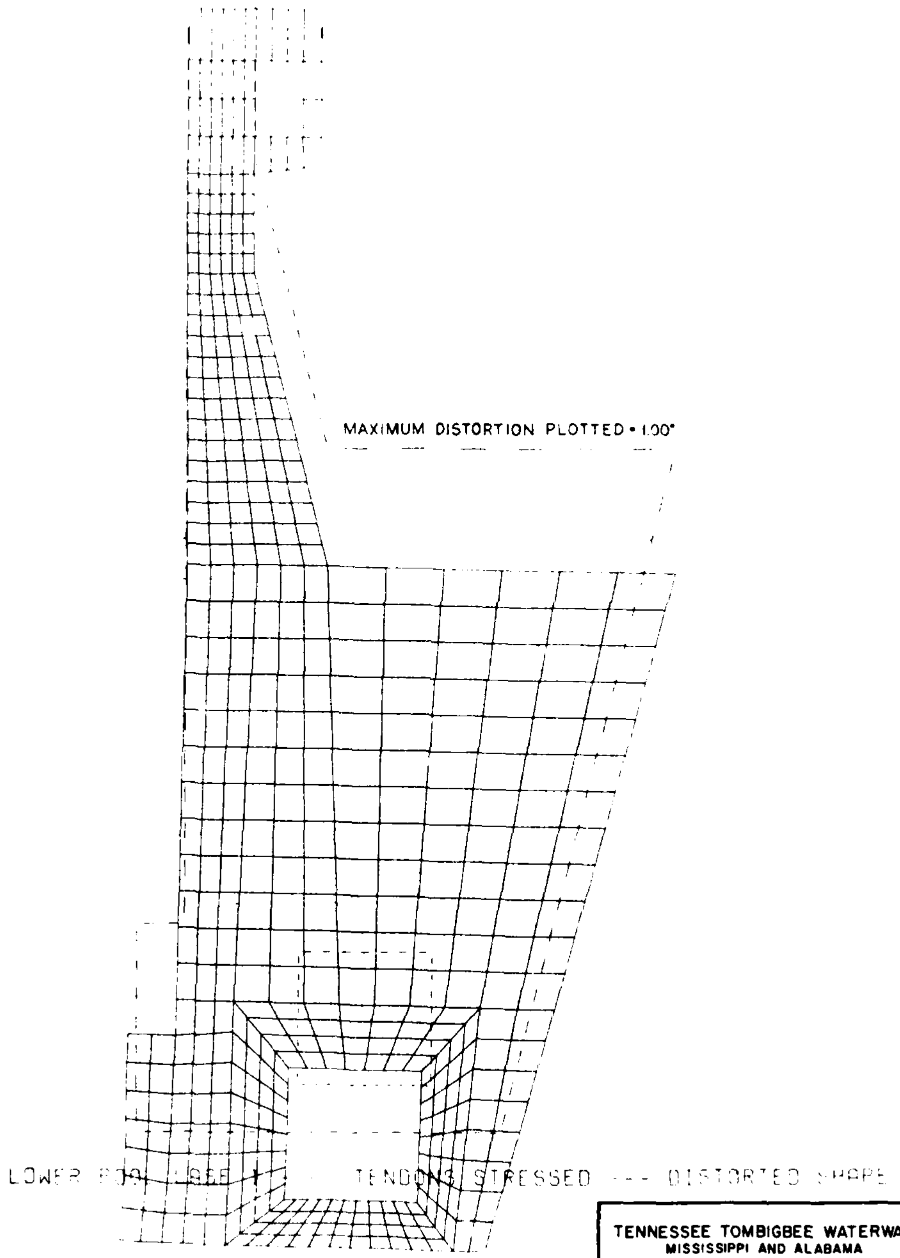
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD



LOWER POOL CASE 1 --- TENDONS STRESSED --- SIGMA Y



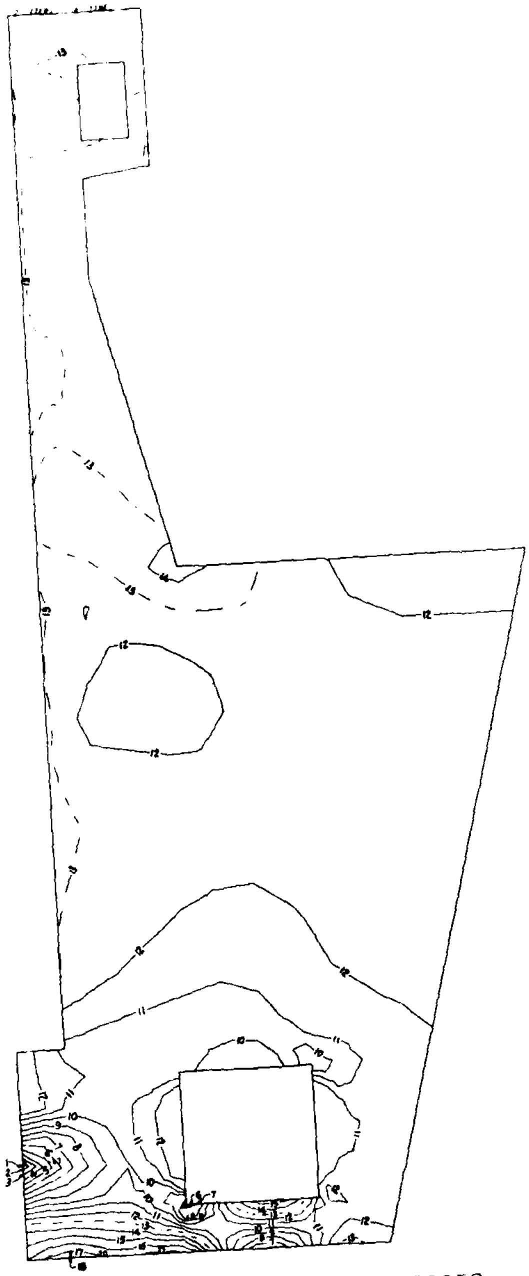
TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22

STRESS ANALYSIS
BY FINITE ELEMENT METHOD



CONTOUR INTERVAL	VALUE (KSF)
1.	-48.000
2.	-44.000
3	-40.000
4.	-36.000
5.	-32.000
6.	-28.000
7.	-24.000
8.	-20.000
9.	-16.000
10.	-12.000
11.	-8.0000
12.	-4.0000
13.	0.
14.	4.0000
15.	8.0000
16.	12.000
17.	16.000
18.	20.000
19	24.000
20.	28.000

LOWER POOL CASE 1 --- TENDONS STRESSED --- SIGMA X

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
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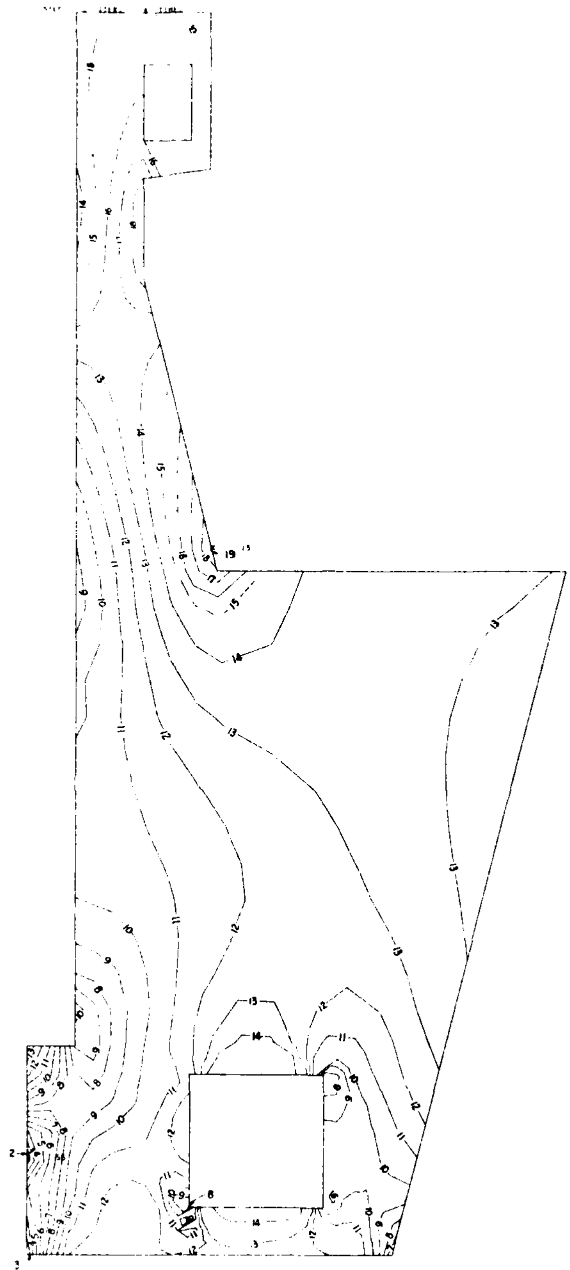
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1-7

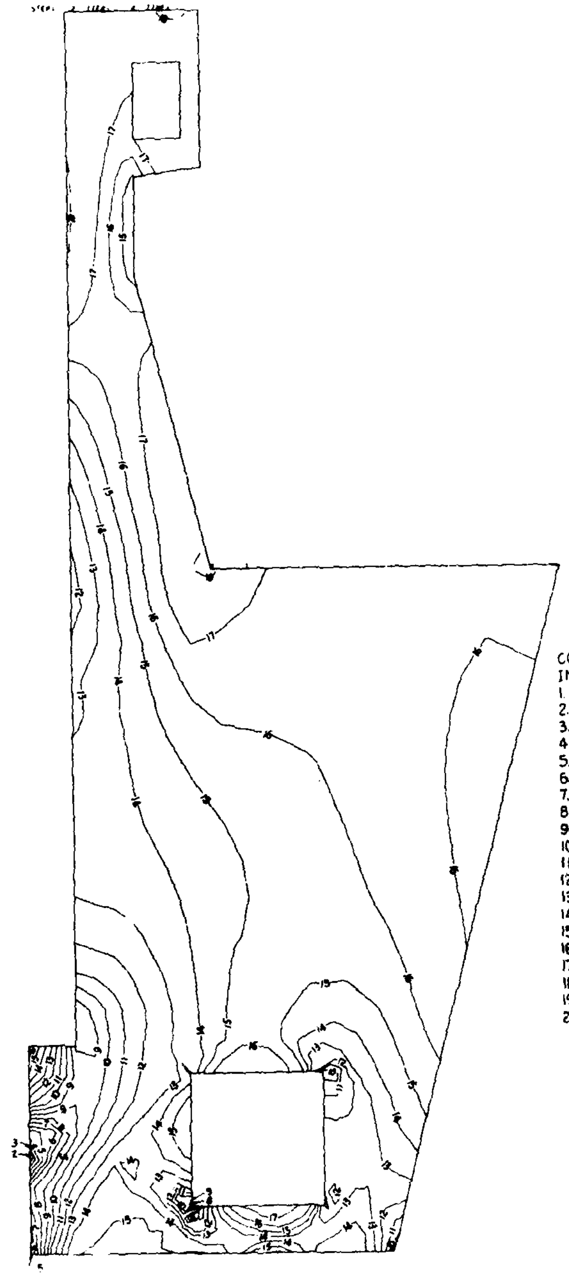


CONTOUR INTERVAL	VALUE (KSF)
1	-56.000
2.	-52.000
3	-48.000
4.	-44.000
5.	-40.000
6	-36.000
7.	-32.000
8.	-28.000
9.	-24.000
10	-20.000
11.	-16.000
12	-12.000
13	-8.0000
14	-4.0000
15	0.
16	4.0000
17.	8.0000
18	12.000
19	16.000
20.	20.000

[illegible] CASE I --- TENDONS STRESSED --- SIGMA Y

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

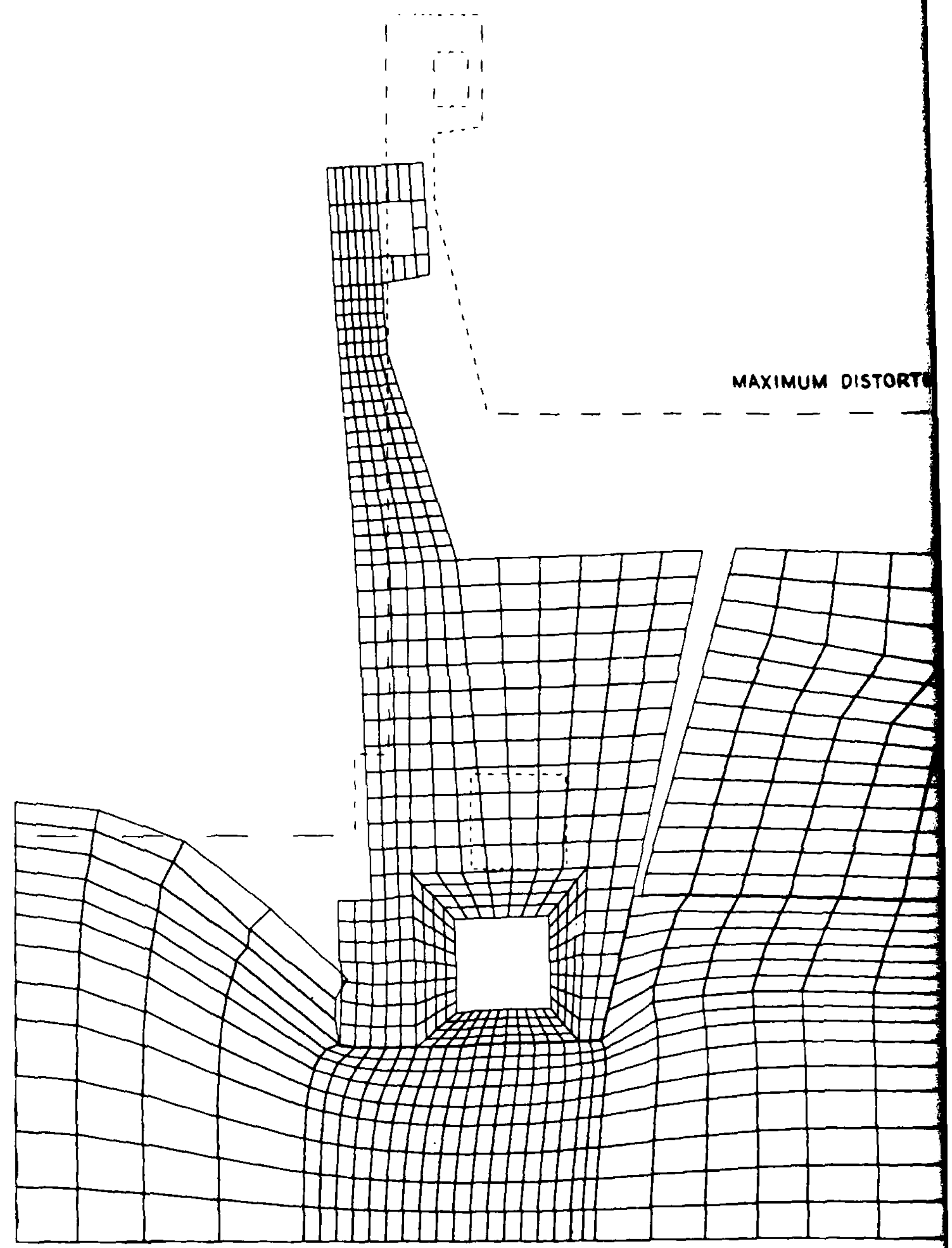


CONTOUR INTERVAL	VALUE (KSF)
1.	-68.000
2.	-64.000
3.	-60.000
4	-56.000
5.	-52.000
6.	-48.000
7.	-44.000
8.	-40.000
9.	-36.000
10.	-32.000
11.	-28.000
12.	-24.000
13.	-20.000
14.	-16.000
15.	-12.000
16.	-8.0000
17.	-4.0000
18.	0.
19.	4.0000
20.	8.0000

LOWER POOL CASE 1 --- TENDONS STRESSED --- SIGMA MIN.

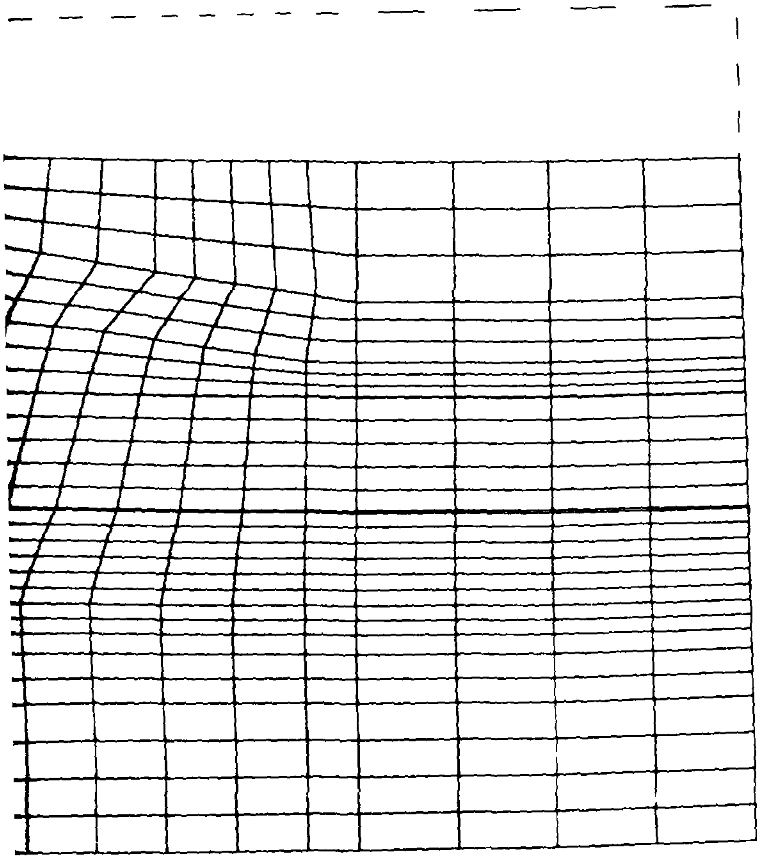
TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1-11



OWER POOL CASE 1 --- NO TENDONS STRESSED --- DISTORTED SH

IUM DISTORTION PLOTTED = 1.05"

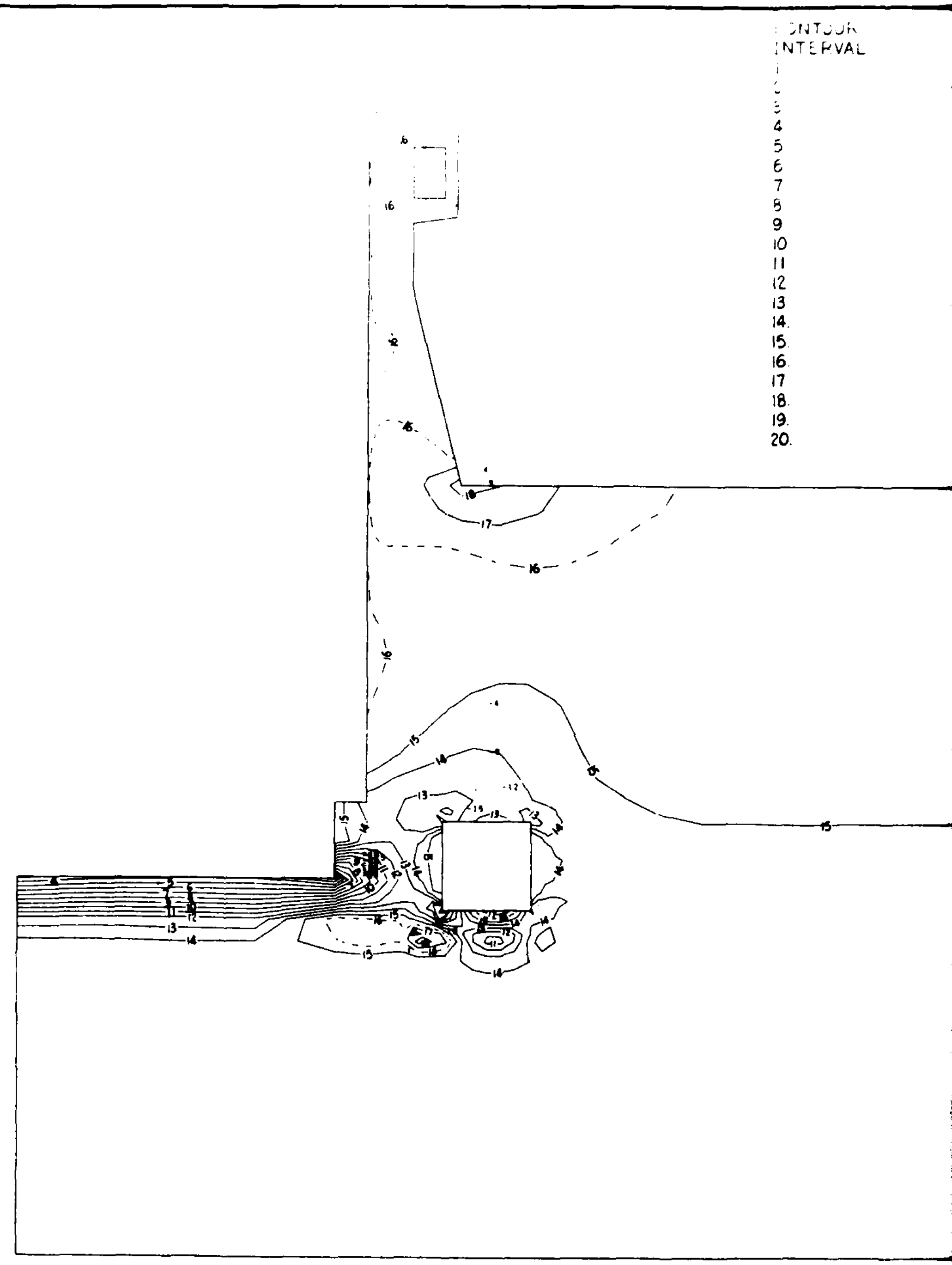


ORTED SHAPE

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1NT-1

2



LOWER POOL CASE 1 --- NO TENDONS STRESSED --- SIGMA X

VALUE
(KSF)
-60.000
-56.000
-52.000
-48.000
-44.000
-40.000
-36.000
-32.000
-28.000
-24.000
-20.000
-16.000
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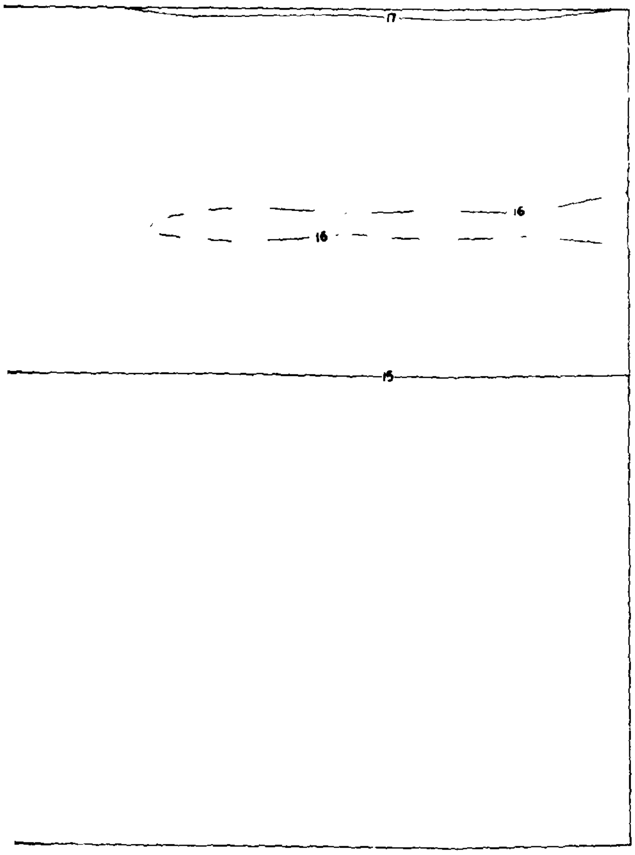
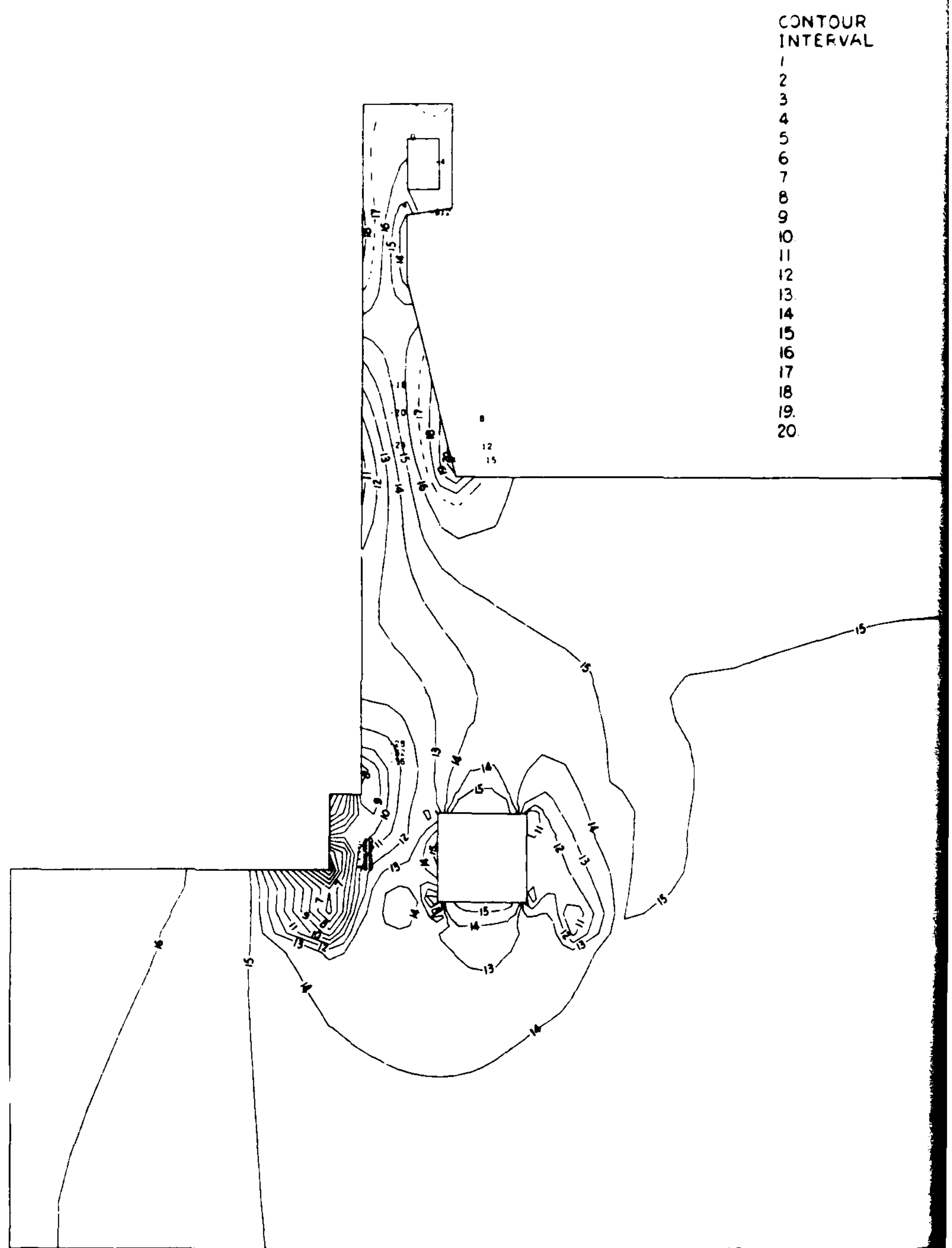
-8.0000
-4.0000
0.
4.0000
8.0000
12.000
16.000

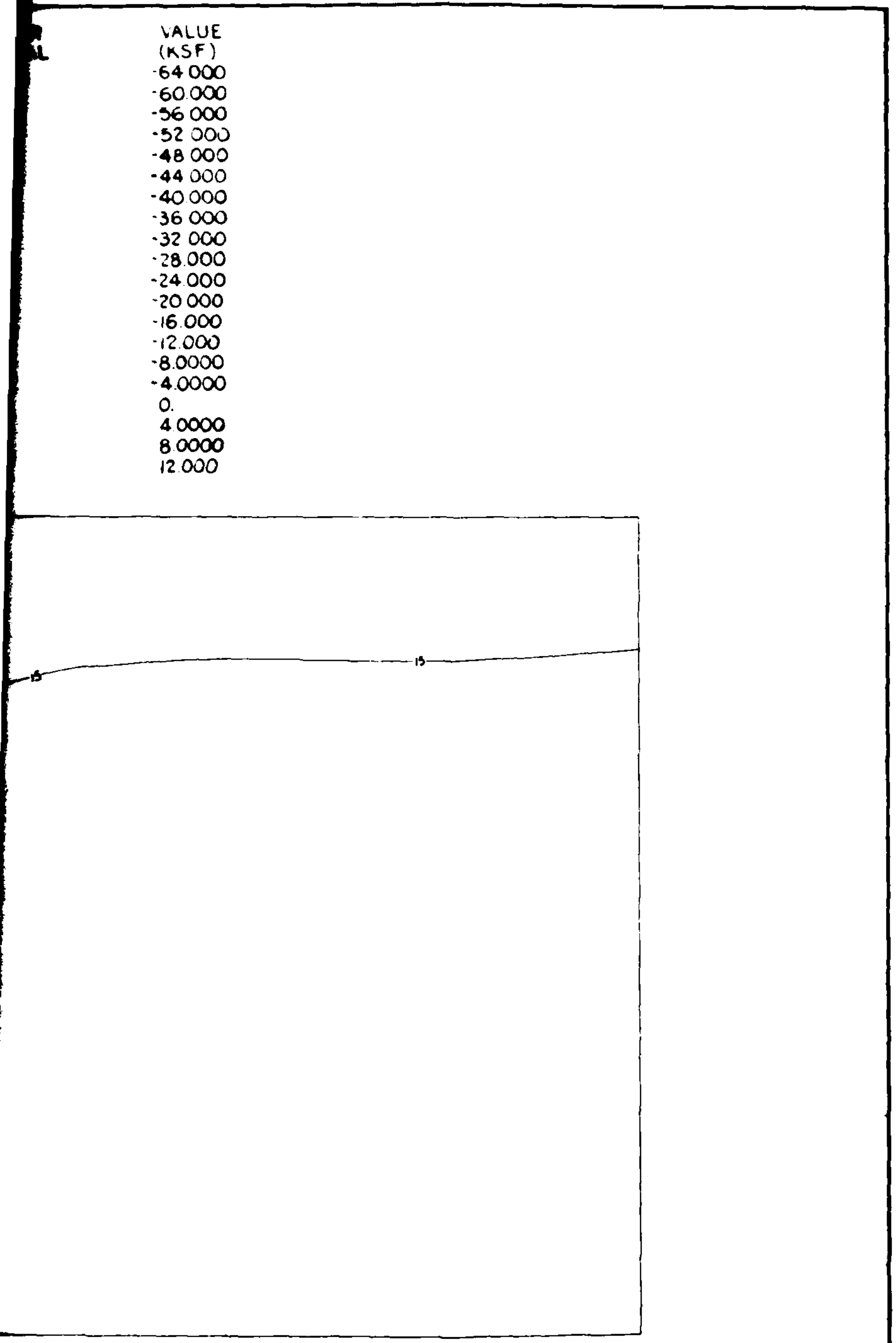


TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1NT-2



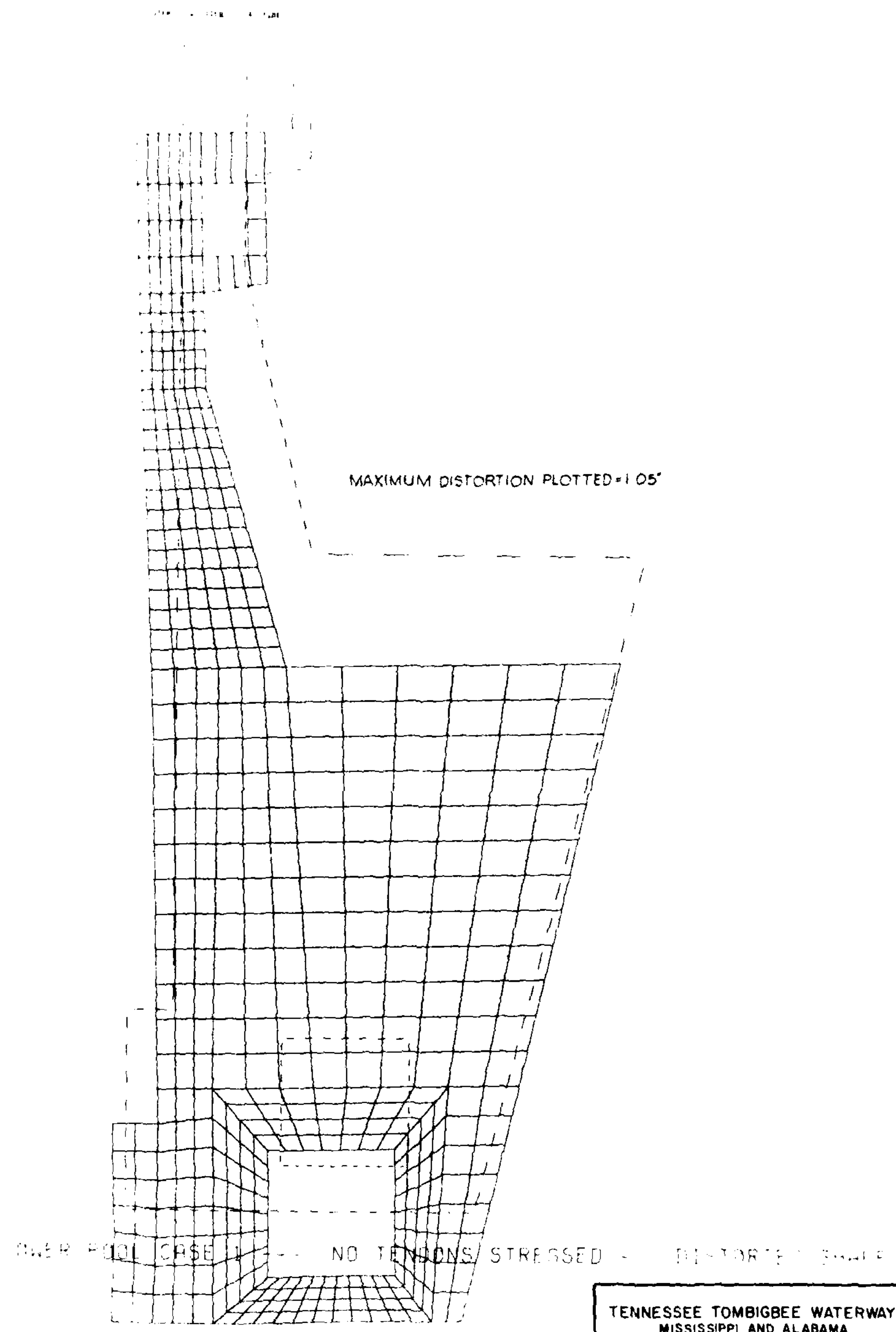
LOWER POOL CASE 1 --- NO TENDONS STRESSED --- SIGMA Y



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

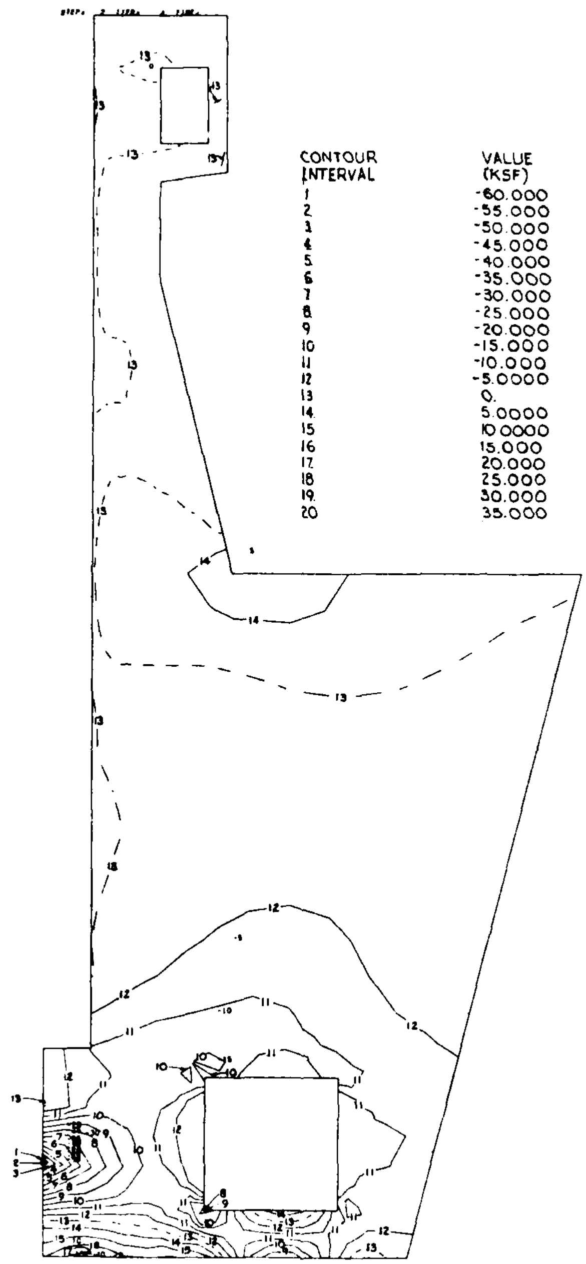
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1NT-3



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

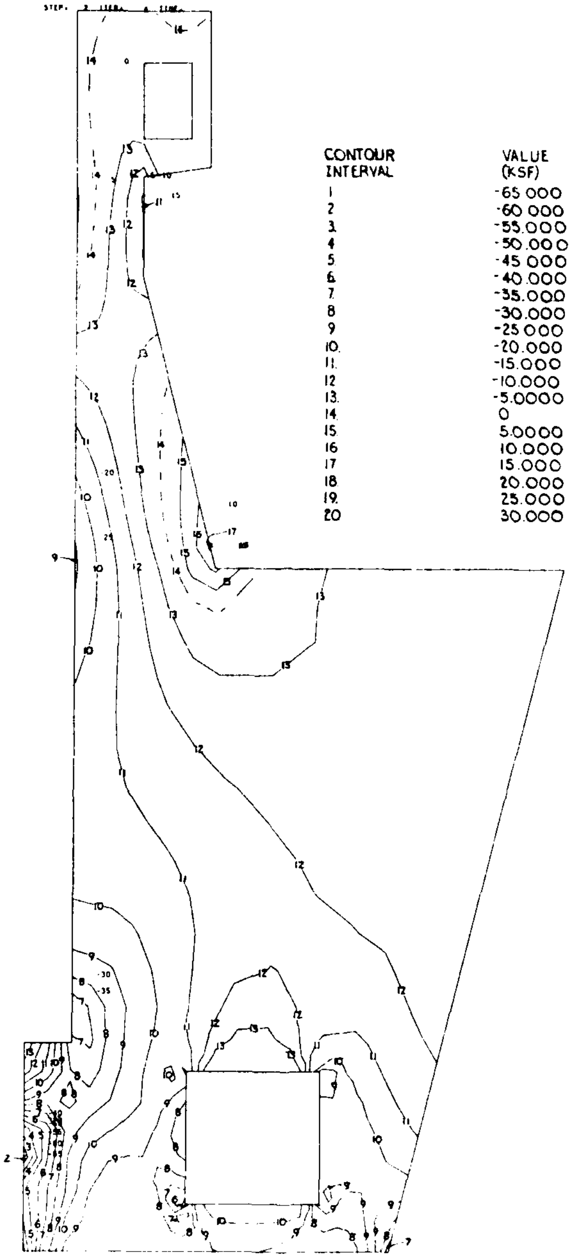


OWER POOL CASE 1 --- NO TENDONS STRESSED --- SIGMA Y

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1NT-7



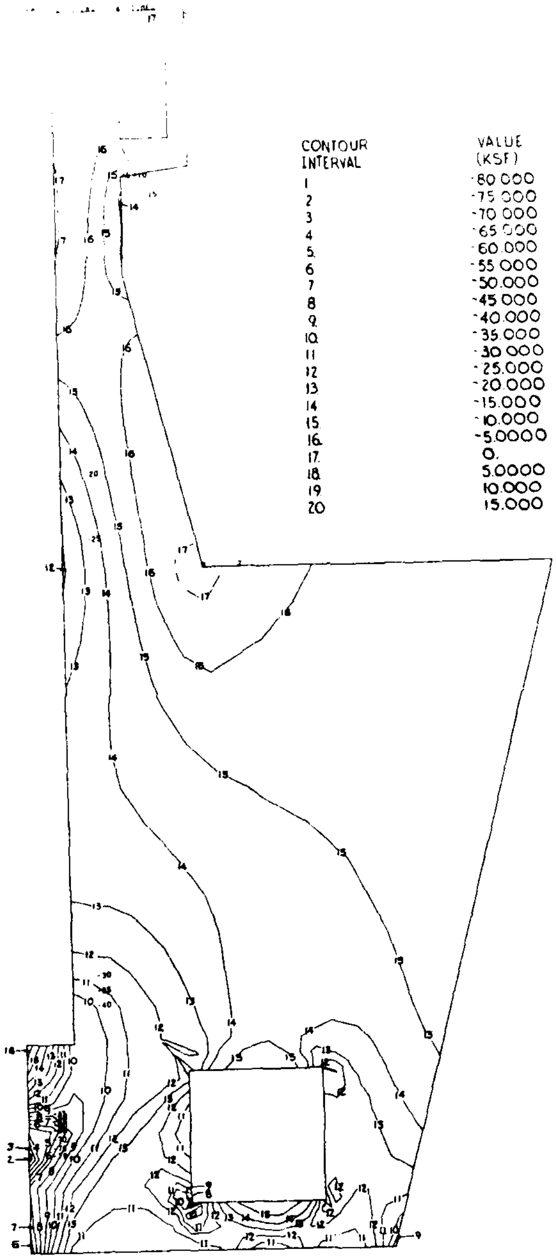
CONTOUR INTERVAL	VALUE (KSF)
1	-65.000
2	-60.000
3	-55.000
4	-50.000
5	-45.000
6	-40.000
7	-35.000
8	-30.000
9	-25.000
10	-20.000
11	-15.000
12	-10.000
13	-5.0000
14	0
15	5.0000
16	10.000
17	15.000
18	20.000
19	25.000
20	30.000

[illegible] CASE 1 NO TENDONS STRESSED [illegible]

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1NT-8

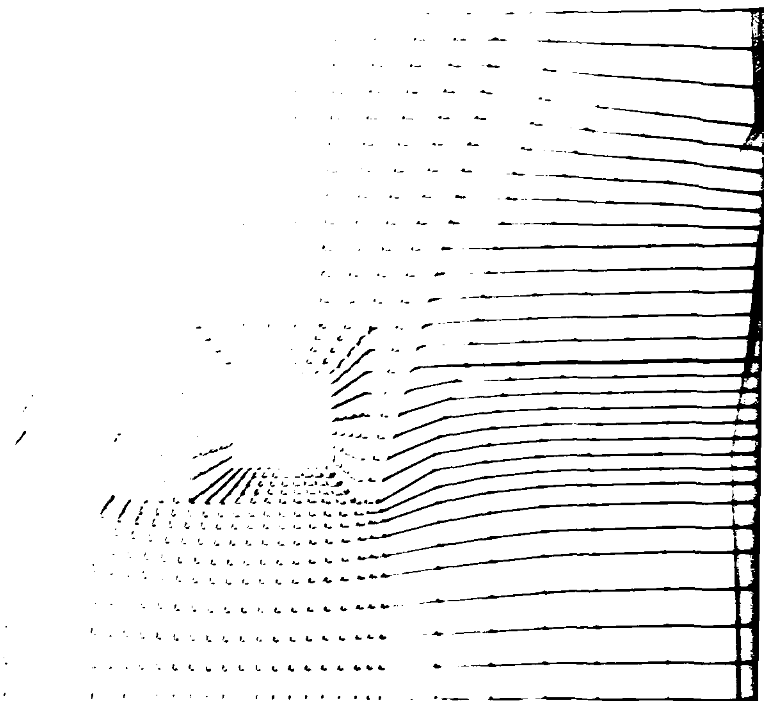


CONTOUR INTERVAL	VALUE (KSF)
1	-80.000
2	-75.000
3	-70.000
4	-65.000
5	-60.000
6	-55.000
7	-50.000
8	-45.000
9	-40.000
10	-35.000
11	-30.000
12	-25.000
13	-20.000
14	-15.000
15	-10.000
16	-5.0000
17	0.
18	5.0000
19	10.000
20	15.000

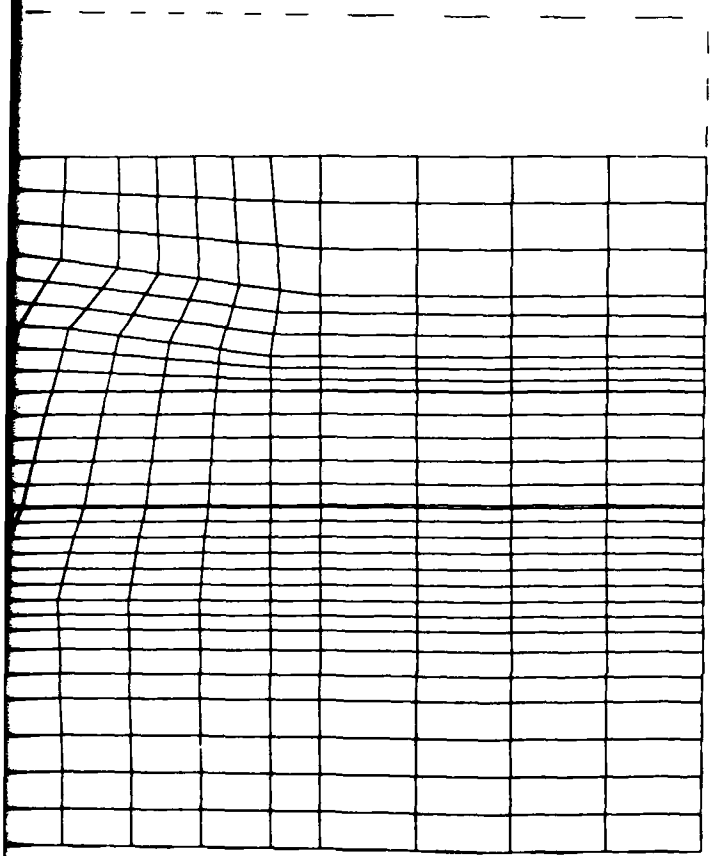
LOWER POOL CASE 1 --- NO TENDONS STRESSED --- SIGMA MIN

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 1NT-11



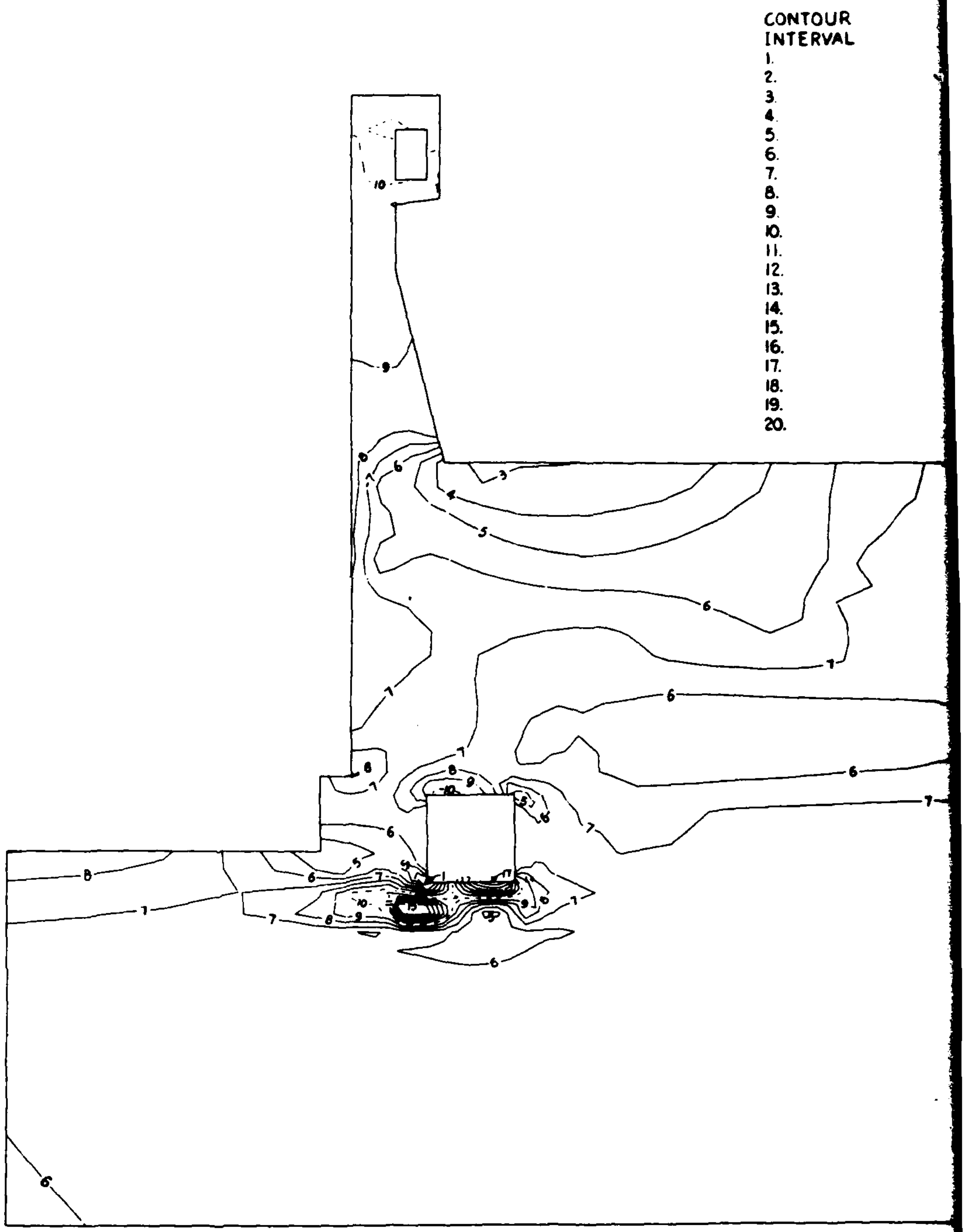
TORTION PLOTTED = 1.067



SHAPE

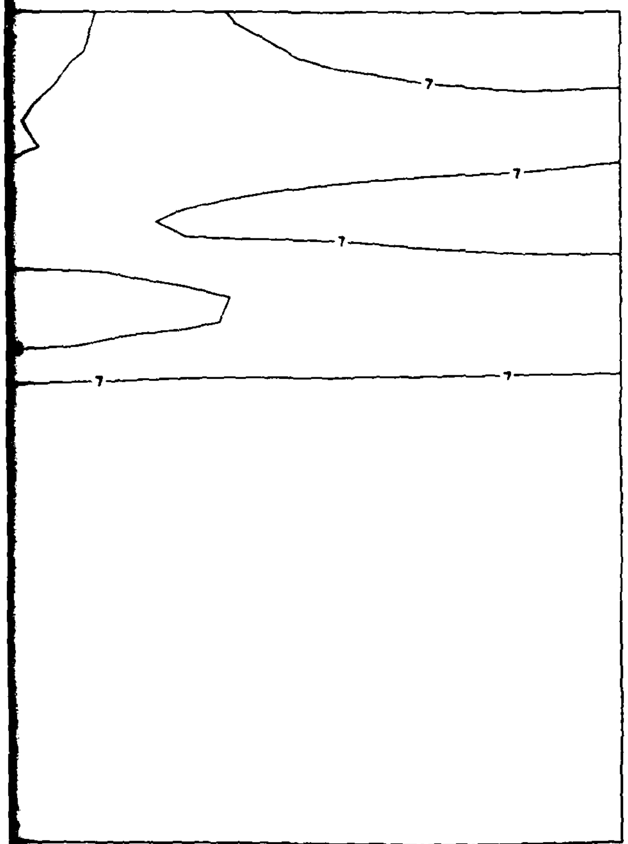
TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2-1



UPPER POOL CASE 2 --- TENDONS STRESSED --- SIGMA X

VALUE
(KSF)
-18.000
-16.000
-14.000
-12.000
-10.000
-8.0000
-6.0000
-4.0000
-2.0000
0.
2.0000
4.0000
6.0000
8.0000
10.000
12.000
14.000
16.000
18.000
20.000

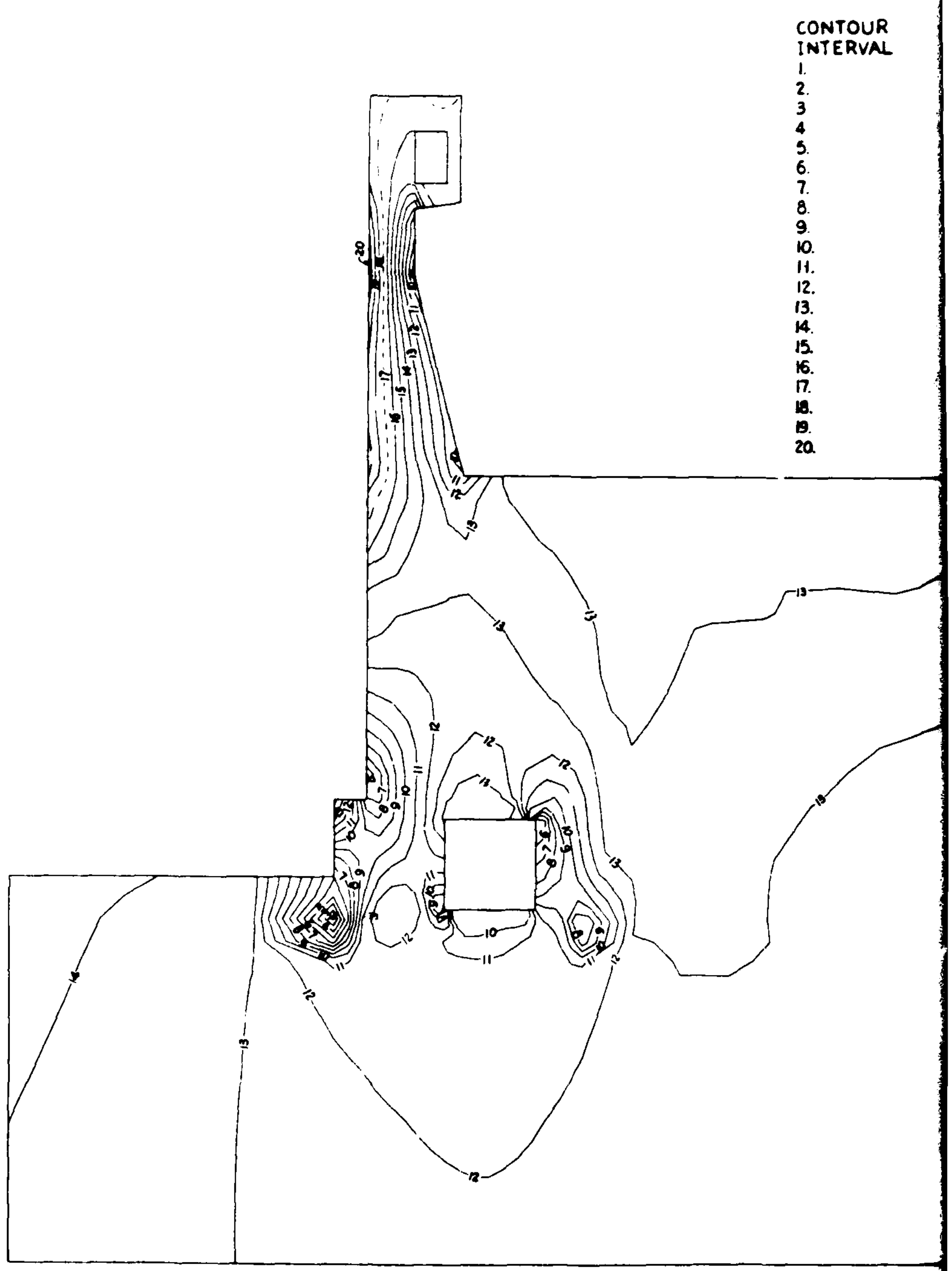


TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

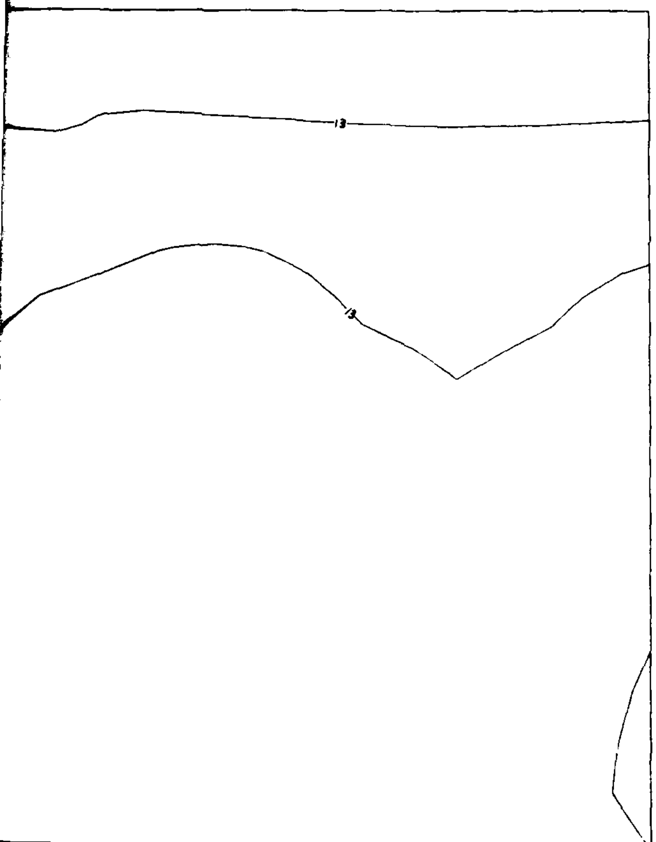
PLATE 2-2

1 . 2



UPPER POOL CASE 2 --- TENDONS STRESSED --- SIGMA Y

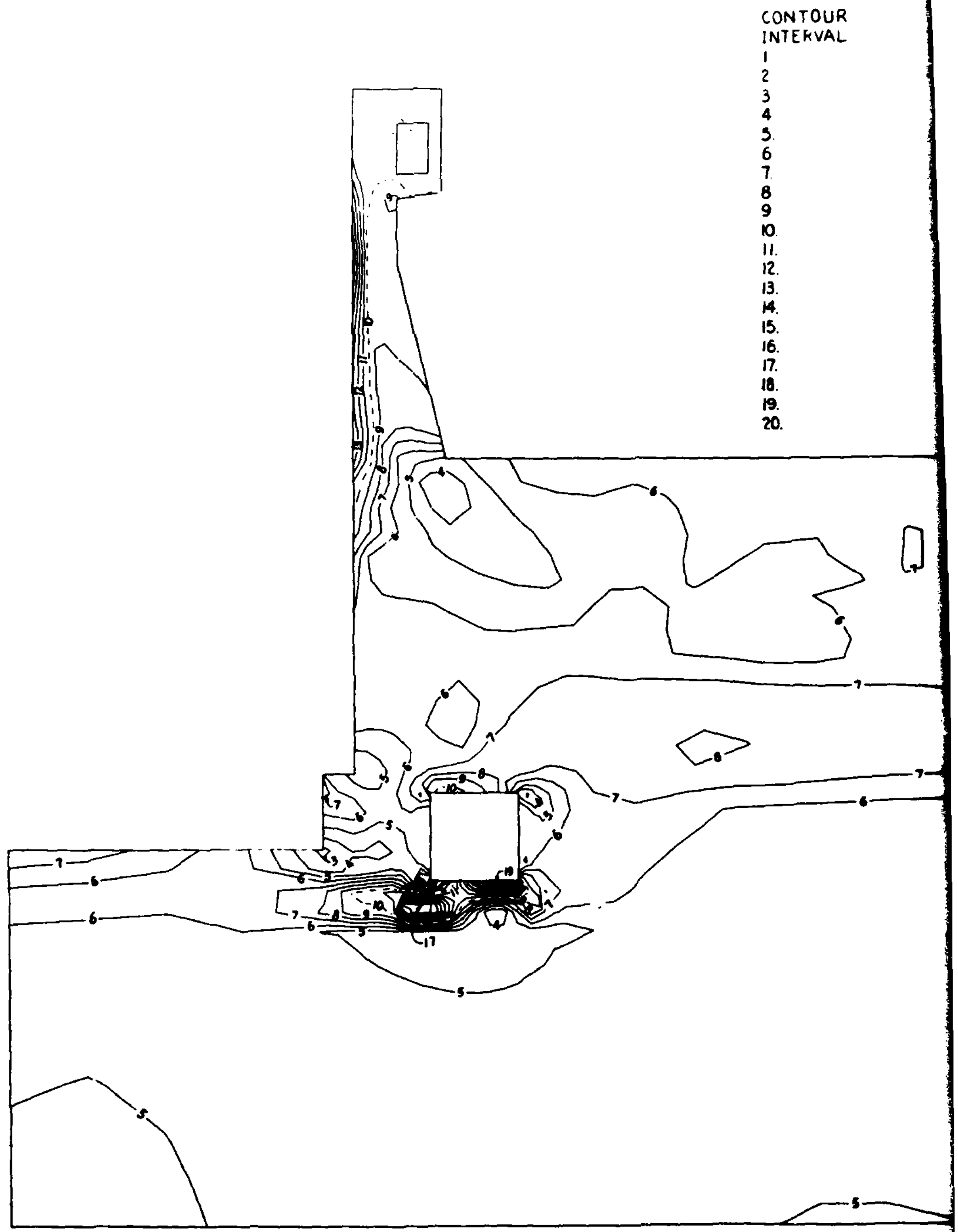
VALUE (KSF)
-40.000
-37.500
-35.000
-32.500
-30.000
-27.500
-25.000
-22.500
-20.000
-17.500
-15.000
-12.500
-10.000
-7.5000
-5.0000
-2.5000
0.
2.5000
5.0000
7.5000



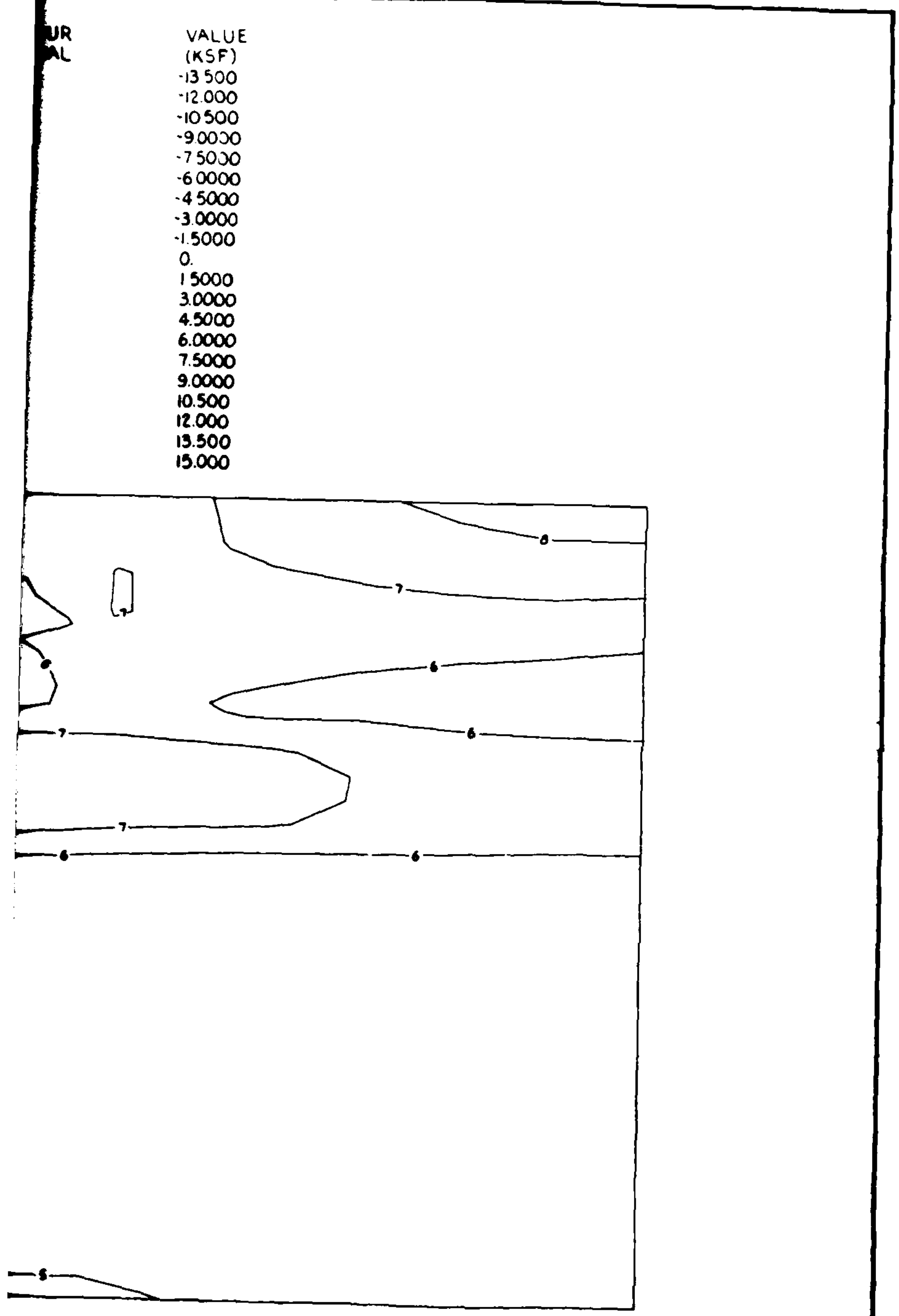
TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2-3



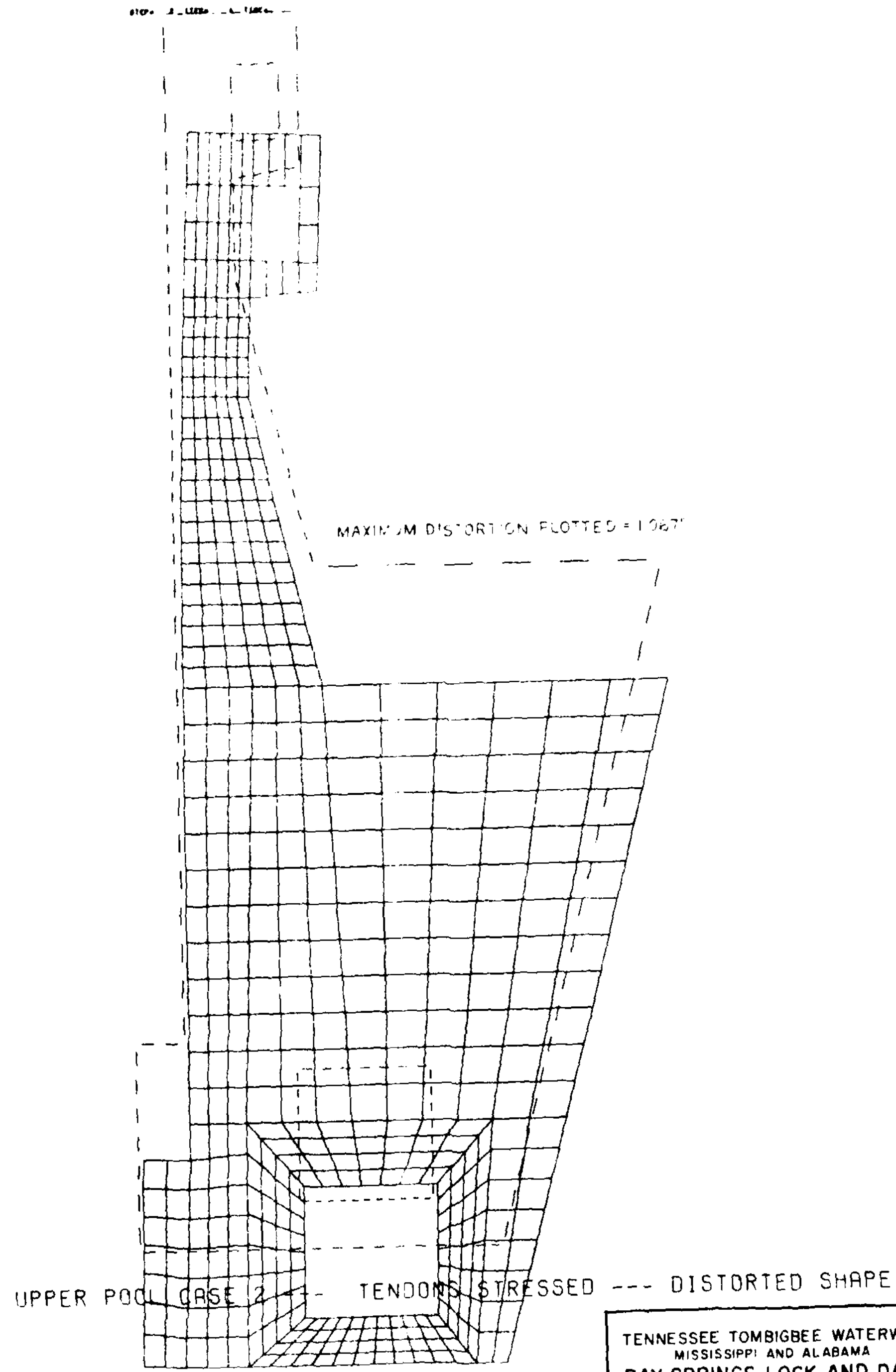
UPPER POOL CASE 2 --- TENDONS STRESSED --- SIGMA MAX.



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

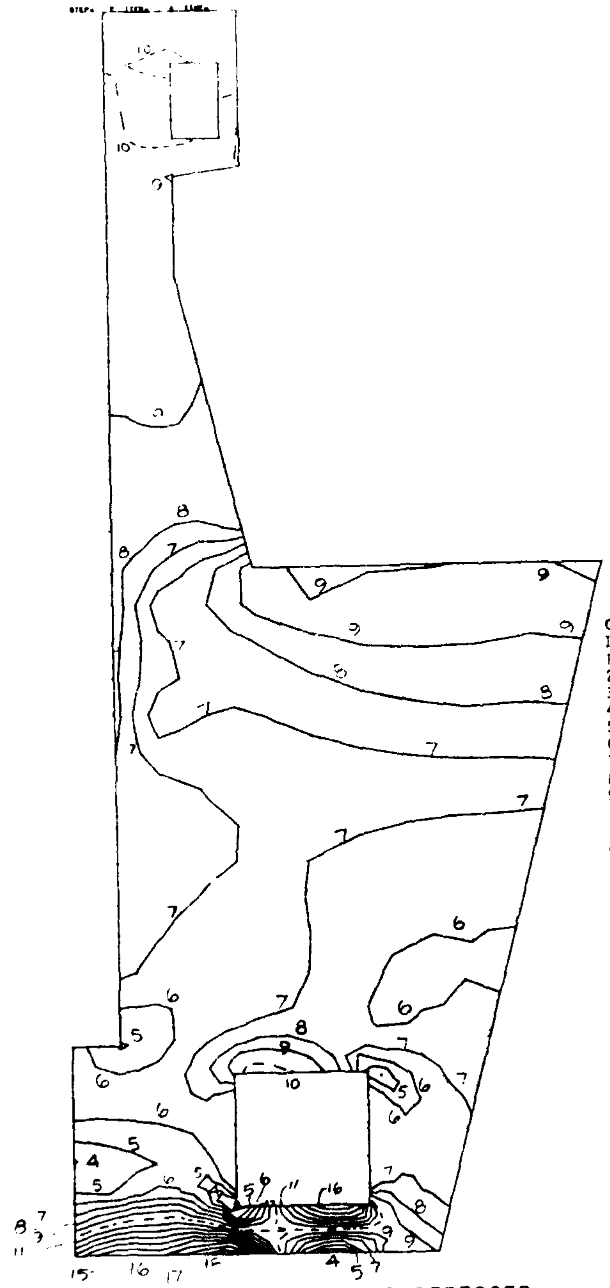
PLATE 2-4



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2-6



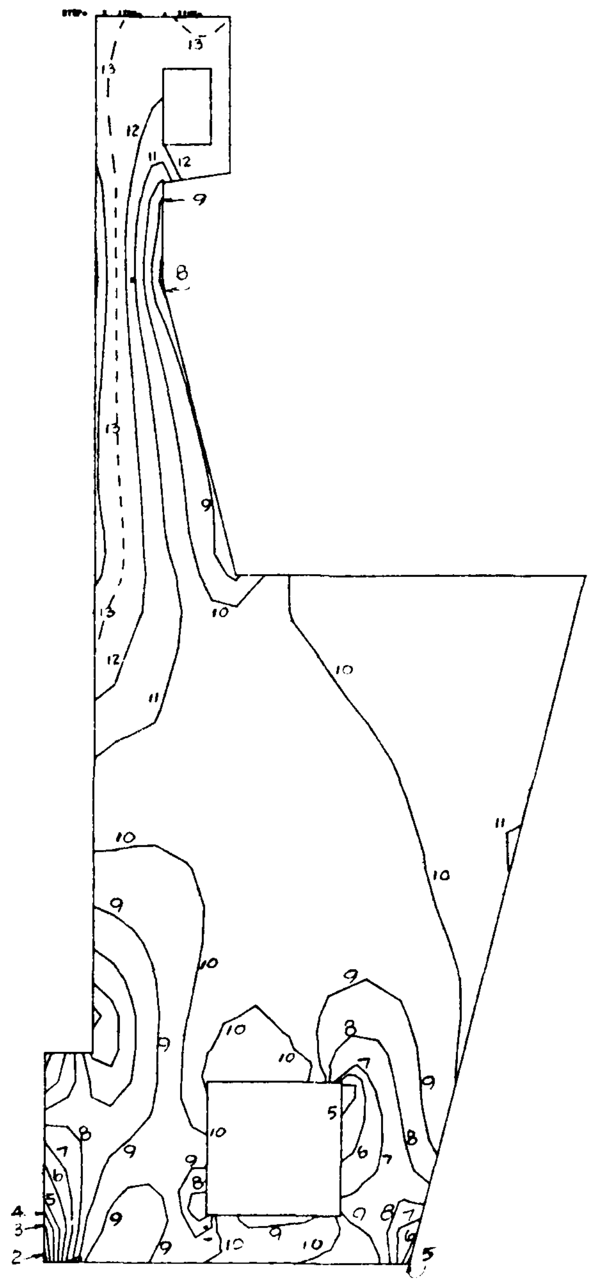
CONTOUR INTERVAL	VALUE (KSF)
1.	-18.000
2	-16.000
3.	-14.000
4	-12.000
5	-10.000
6	-8.000
7.	-6.000
8	-4.000
9.	-2.000
10.	0.
11	2.000
12	4.000
13	6.000
14	8.000
15.	10.000
16.	12.000
17	14.000
18.	16.000
19.	18.000
20.	20.000

UPPER POOL CASE 2 --- TENDONS STRESSED --- SIGMA X

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2-7

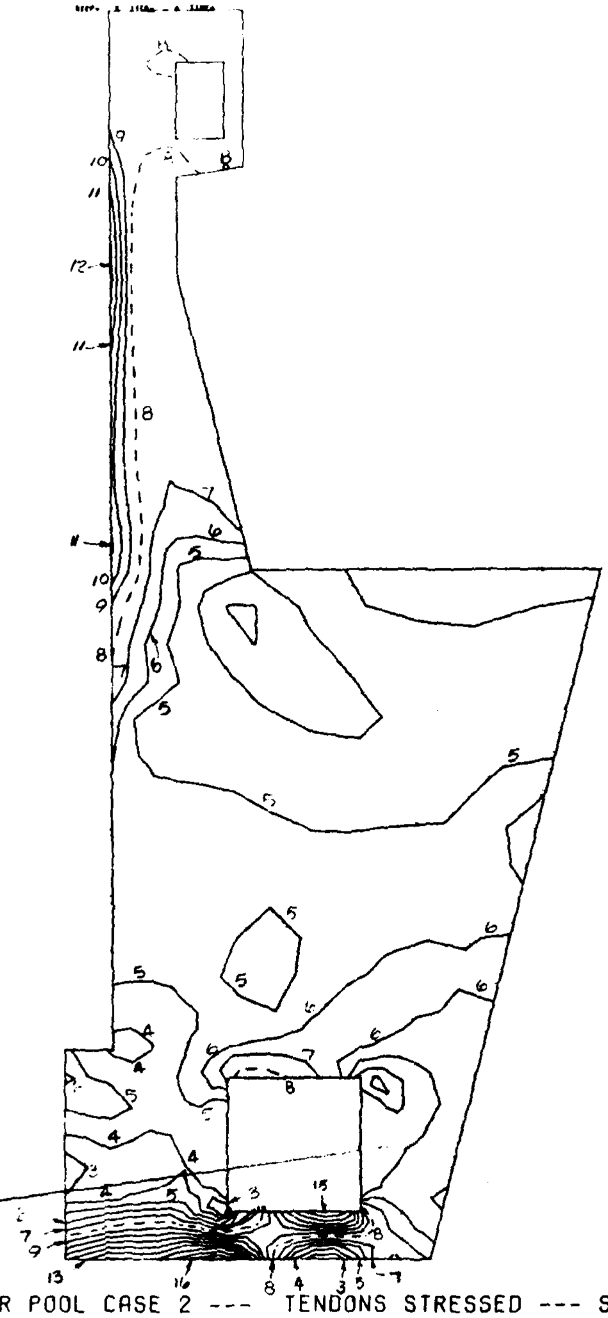


CONTOUR INTERVAL	VALUE (KSF)
1.	-48.000
2.	-44.000
3	-40.000
4.	-36.000
5	-32.000
6	-28.000
7.	-24.000
8.	-20.000
9	-16.000
10.	-12.000
11	-8.000
12	-4.000
13.	0.
14.	4.000
15.	8.000
16.	12.000
17	16.000
18.	20.000
19	24.000
20.	28.000

UPPER POOL CASE 2 --- TENDONS STRESSED --- SIGMA Y

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD



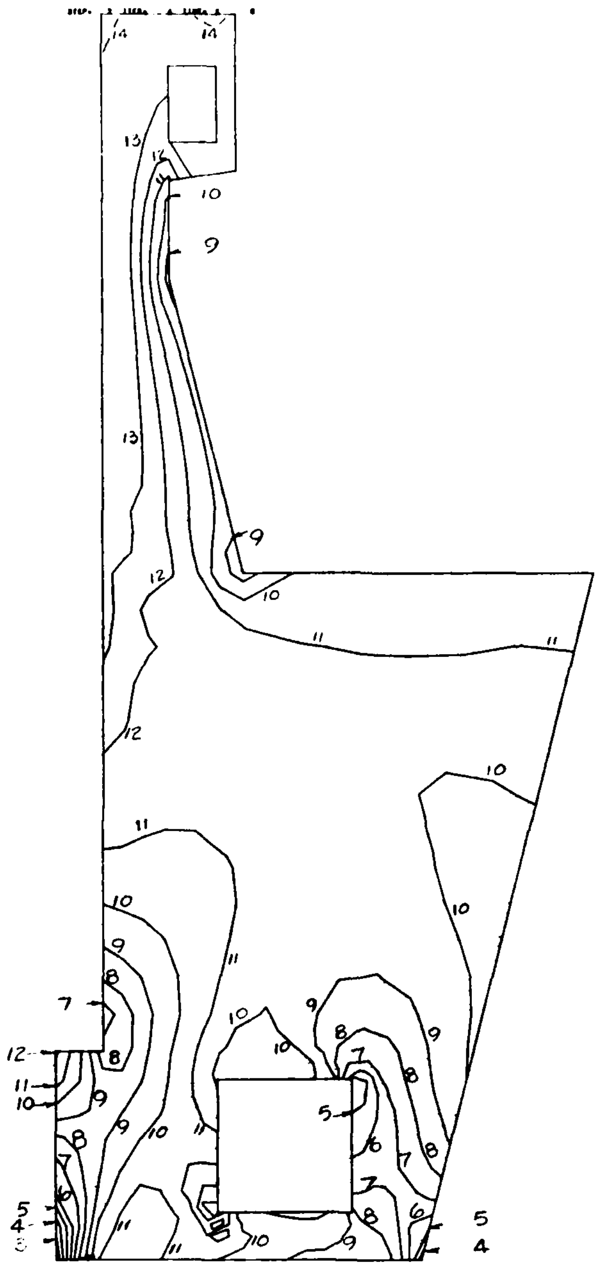
CONTOUR INTERVAL	VALVE (KSF)
1.	-14.000
2.	-12.000
3.	-10.000
4.	-8.000
5.	-6.000
6.	-4.000
7.	-2.000
8.	-0.
9.	2.000
10.	4.000
11.	6.000
12.	8.000
13.	10.000
14.	12.000
15.	14.000
16.	16.000
17.	18.000
18.	20.000
19.	22.000
20.	24.000

UPPER POOL CASE 2 --- TENDONS STRESSED --- SIGMA MAX.

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2-11

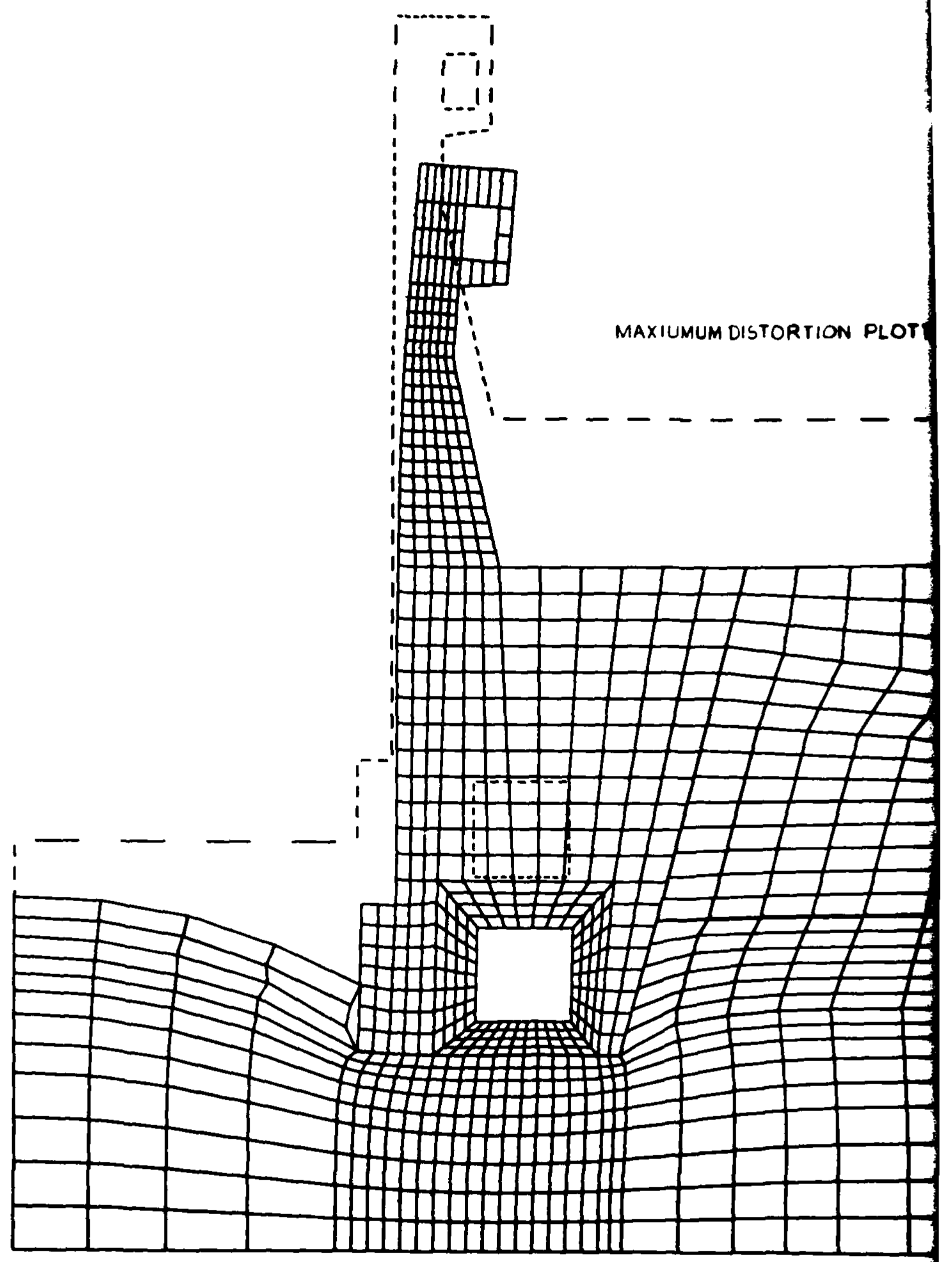


CONTOUR INTERVAL	VALUE (KSF)
1	-52.000
2.	-48.000
3.	-44.000
4.	-40.000
5.	-36.000
6	-32.000
7.	-28.000
8.	-24.000
9.	-20.000
10.	-16.000
11.	-12.000
12.	-8.000
13.	-4.000
14.	0.
15.	4.000
16.	8.000
17.	12.000
18.	16.000
19.	20.000
20.	24.000

UPPER POOL CASE 2 --- TENDONS STRESSED --- SIGMA MIN.

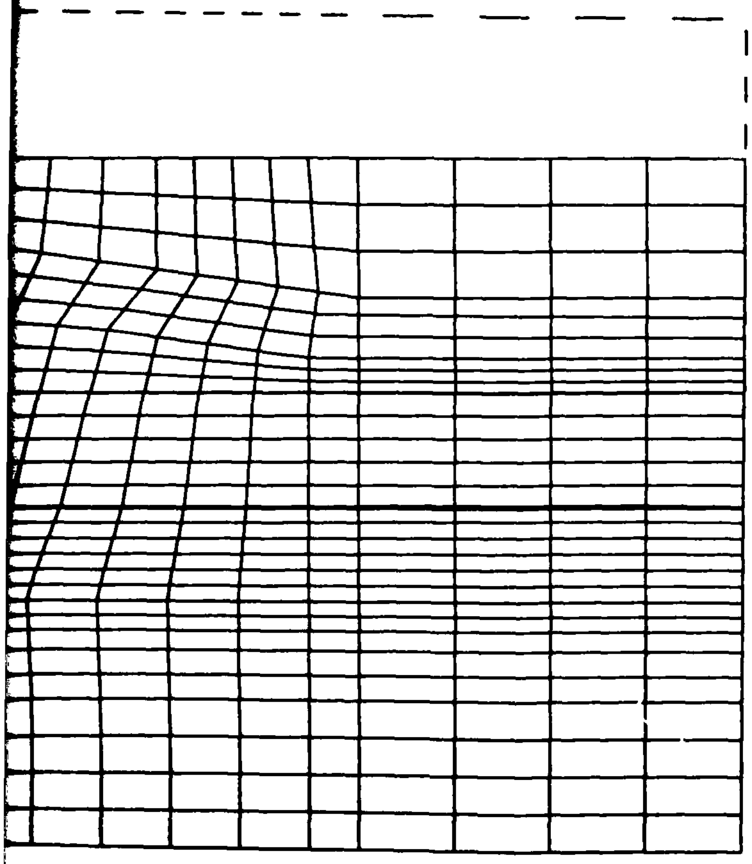
TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD



UPPER POOL CASE 2 --- NO TENDONS STRESSED --- DISTORTED SHAPE

ON PLOTTED=1.07"



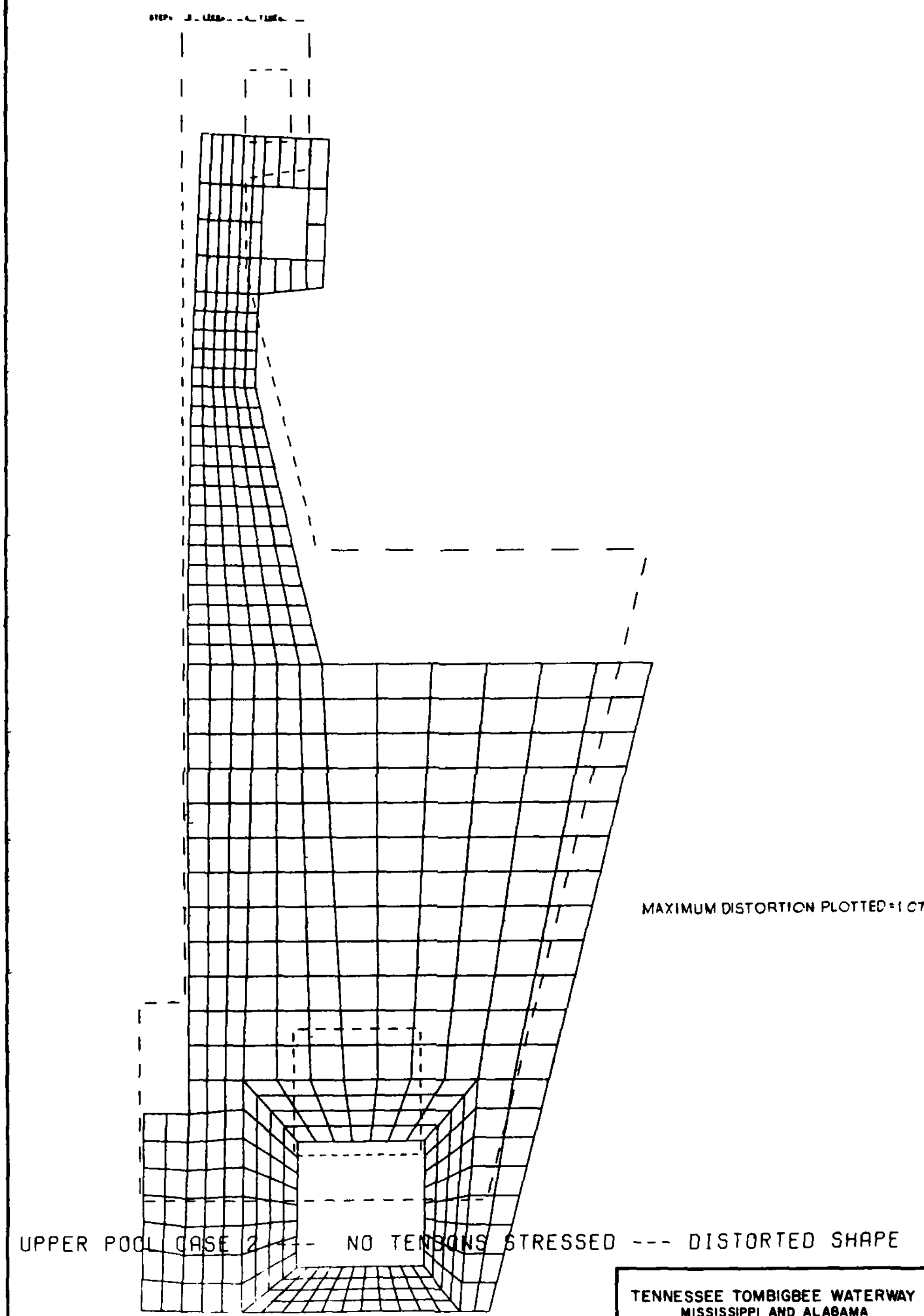
SHAPE

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2NT-1

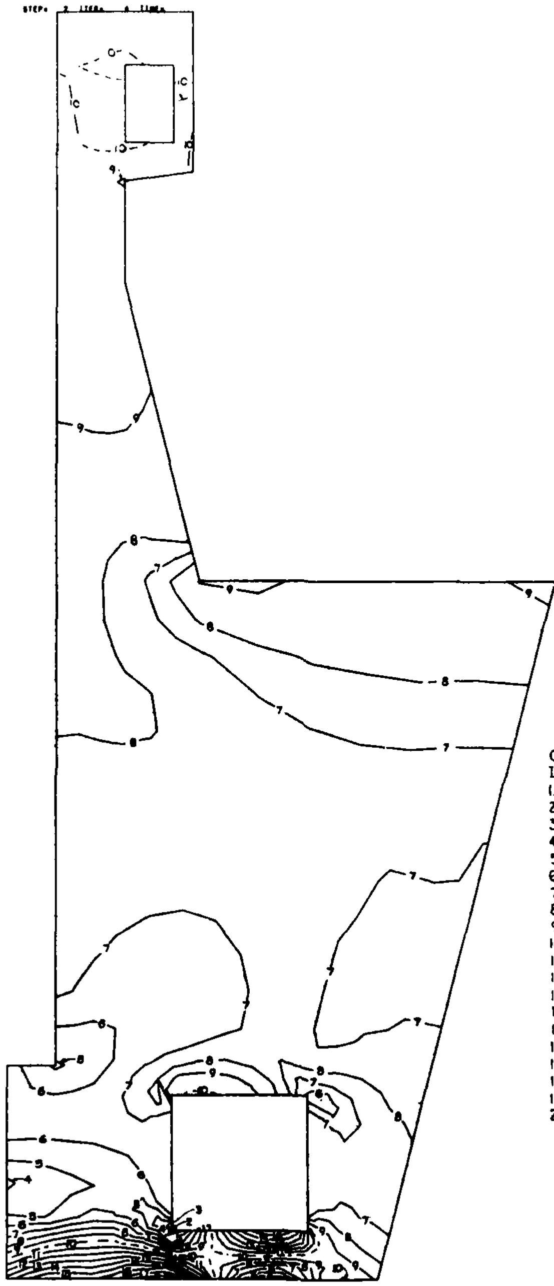
1 , 2



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2NT-6

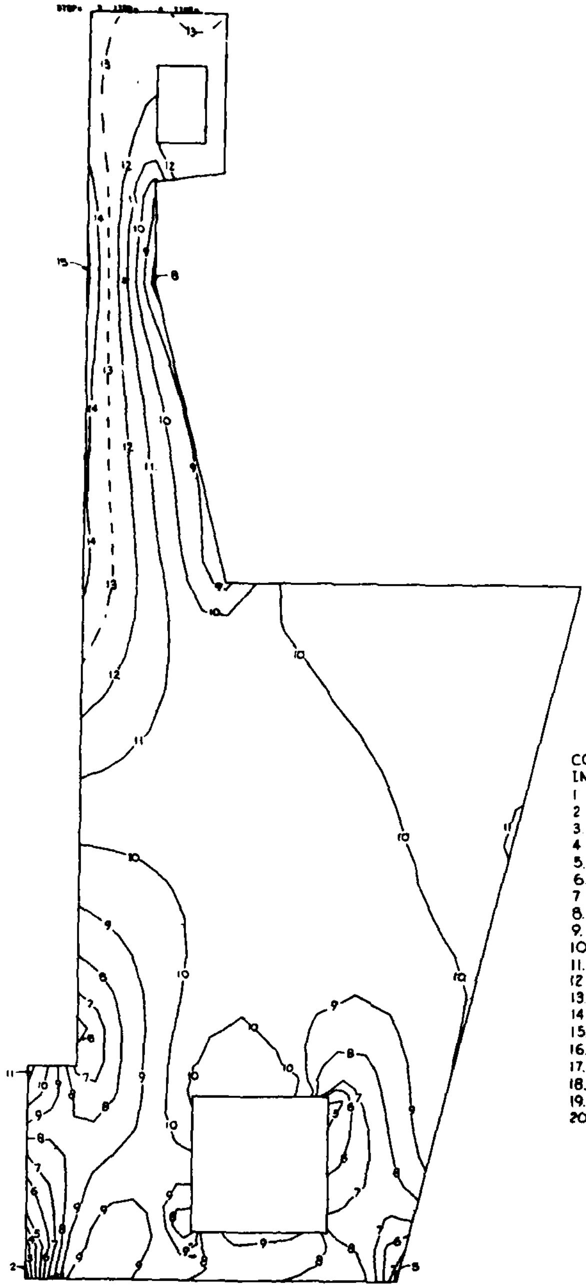


CONTOUR INTERVAL	VALUE (KSF)
1.	-18.000
2	-16.000
3.	-14.000
4.	-12.000
5.	-10.000
6.	-8.0000
7.	-6.0000
8.	-4.0000
9.	-2.0000
10.	0.
11.	-2.0000
12	4.0000
13.	6.0000
14.	8.0000
15.	10.000
16.	12.000
17.	14.000
18	16.000
19	18.000
20.	20.000

UPPER POOL CASE 2 --- NO TENDONS STRESSED --- SIGMA X

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2NT-7

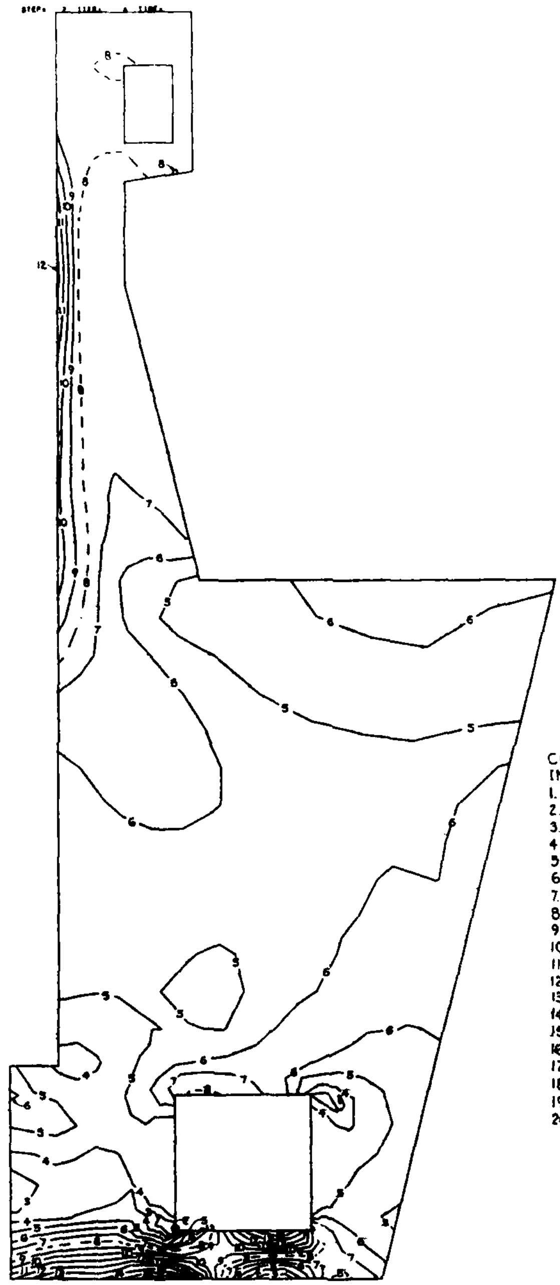


CONTOUR INTERVAL	VALUE (KSF)
1	-48.000
2	-44.000
3	-40.000
4	-36.000
5.	-32.000
6.	-28.000
7	-24.000
8.	-20.000
9.	-16.000
10.	-12.000
11.	-8.0000
12	-4.0000
13	0.
14	4.0000
15.	8.0000
16.	12.000
17.	16.000
18.	20.000
19.	24.000
20.	28.000

UPPER POOL CASE 2 --- NO TENDONS STRESSED --- SIGMA Y

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2NT-8

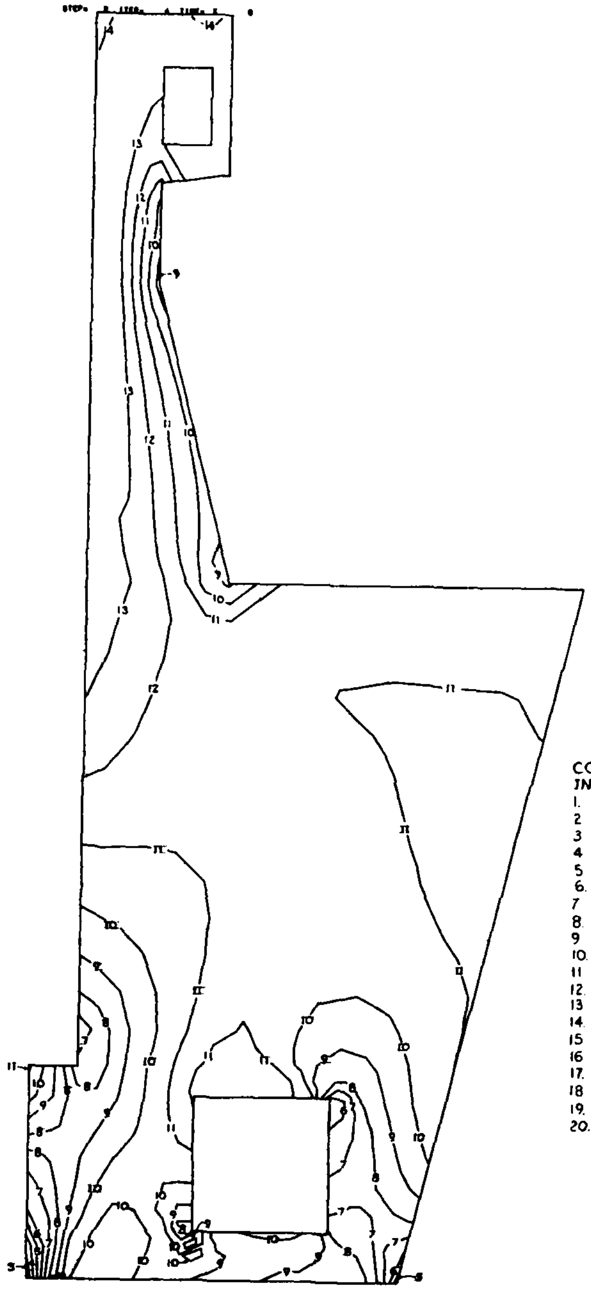


CONTOUR INTERVAL	VALUE (KSF)
1.	-14.000
2.	-12.000
3.	-10.000
4	-8.0000
5	-6.0000
6.	-4.0000
7.	-2.0000
8.	0.
9	2.0000
10.	4.0000
11.	6.0000
12	8.0000
13.	10.000
14	12.000
15	14.000
16	16.000
17	18.000
18.	20.000
19.	22.000
20.	24.000

UPPER POOL CASE 2 --- NO TENDONS STRESSED --- SIGMA MAX.

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE 2NT-10

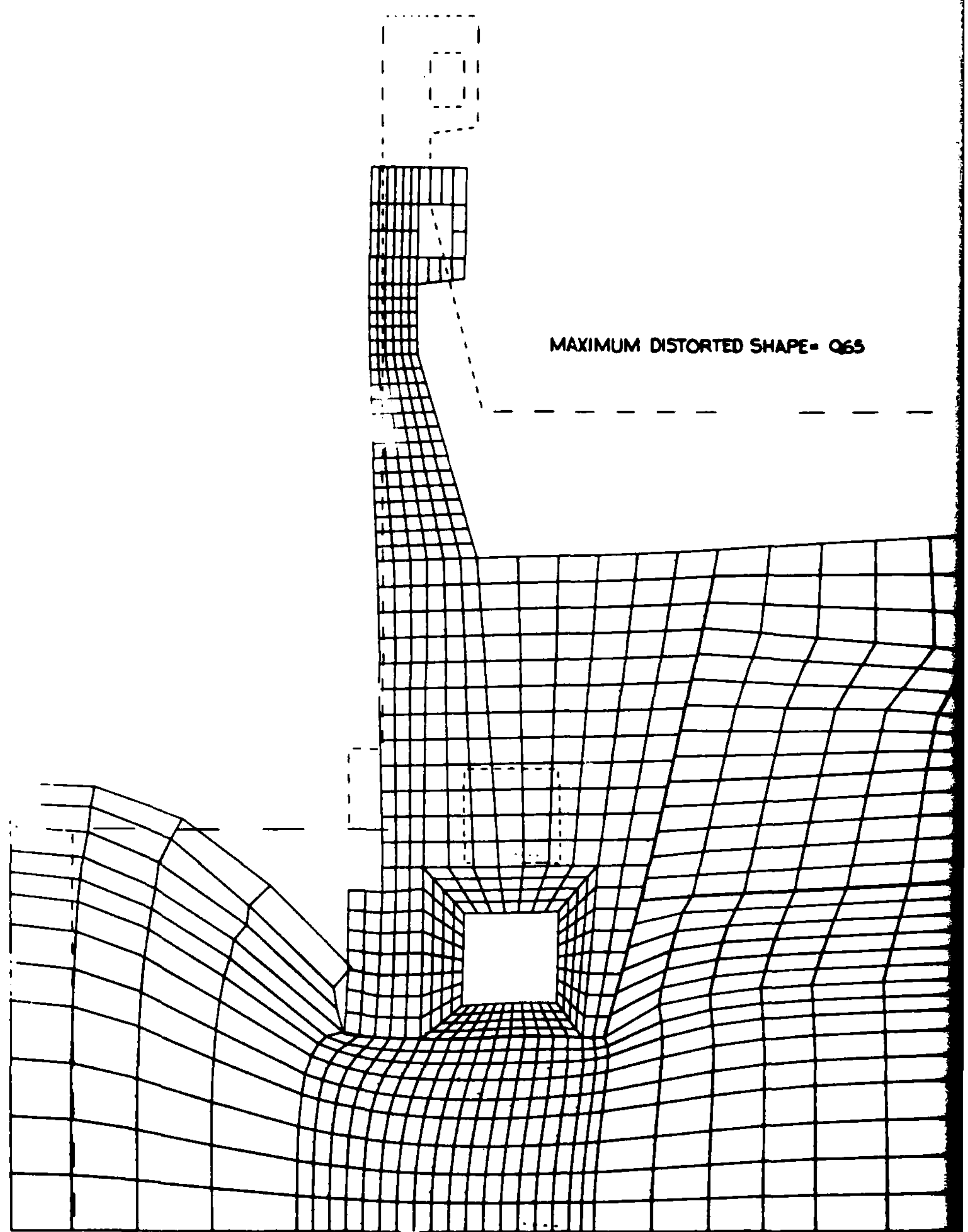


CONTOUR INTERVAL	VALUE (KSF)
1.	-52.000
2	-48.000
3	-44.000
4	-40.000
5	-36.000
6.	-32.000
7	-28.000
8.	-24.000
9	-20.000
10.	-16.000
11	-12.000
12.	-8.0000
13	-4.0000
14	0.
15	4.000C
16	8.0000
17.	12.000
18	16.000
19.	20.000
20.	24.000

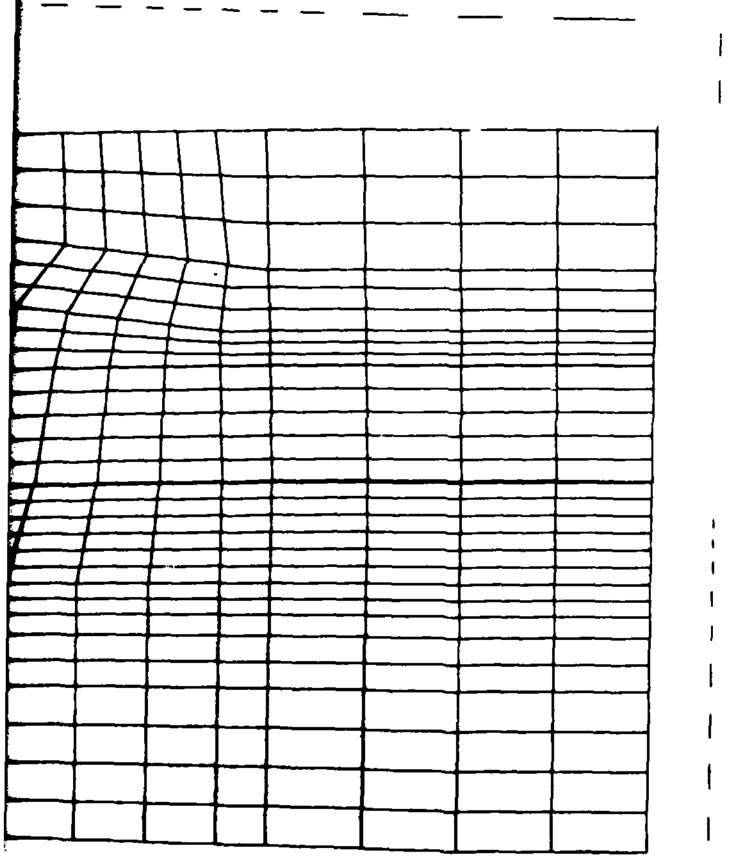
UPPER POOL CASE 2 --- NO TENDONS STRESSED --- SIGMA MIN.

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD



CONSTRUCTION CONDITION --- TENDONS STRESSED --- DISTORTED SH

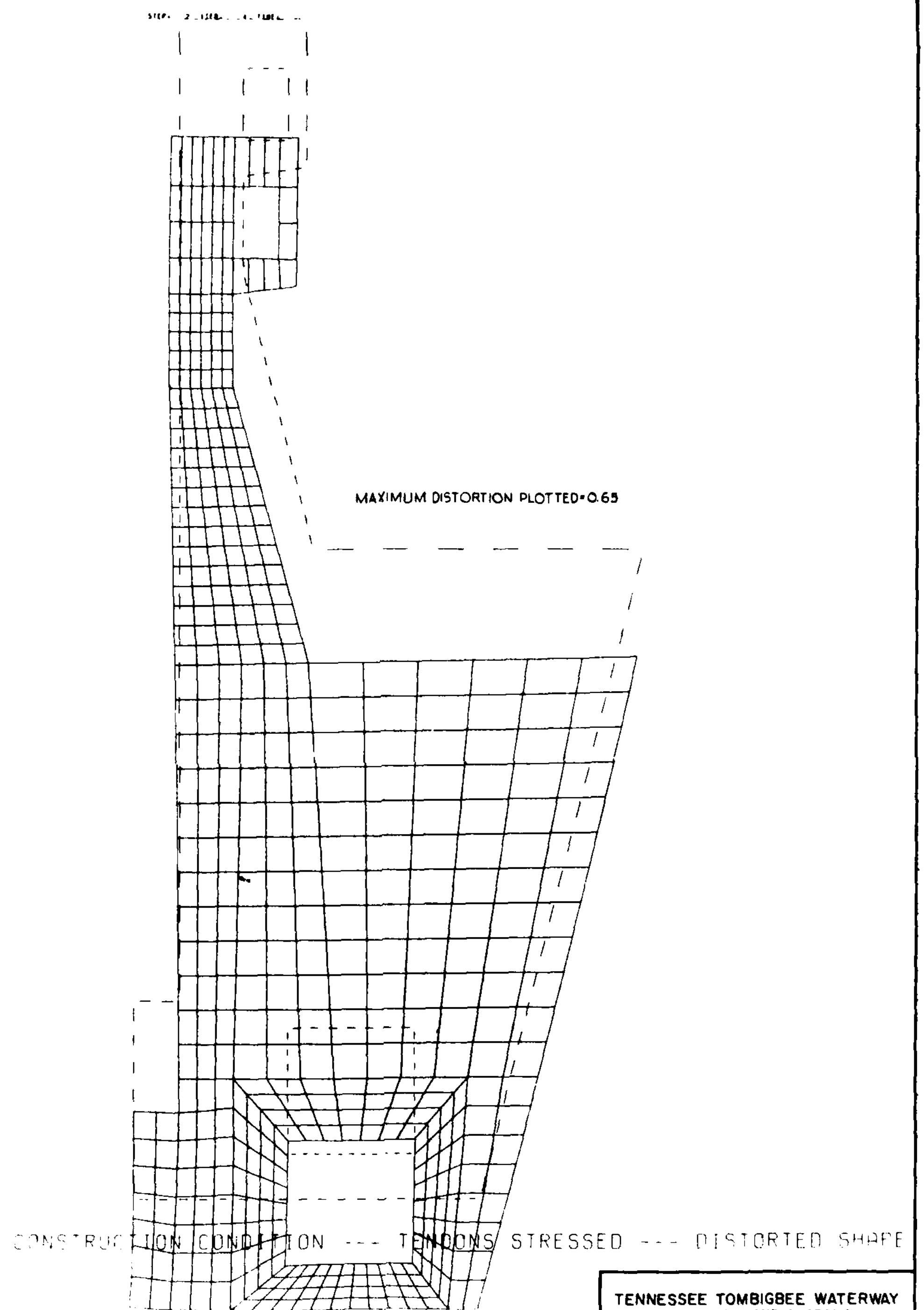


SHAPE

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM
MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE C-1

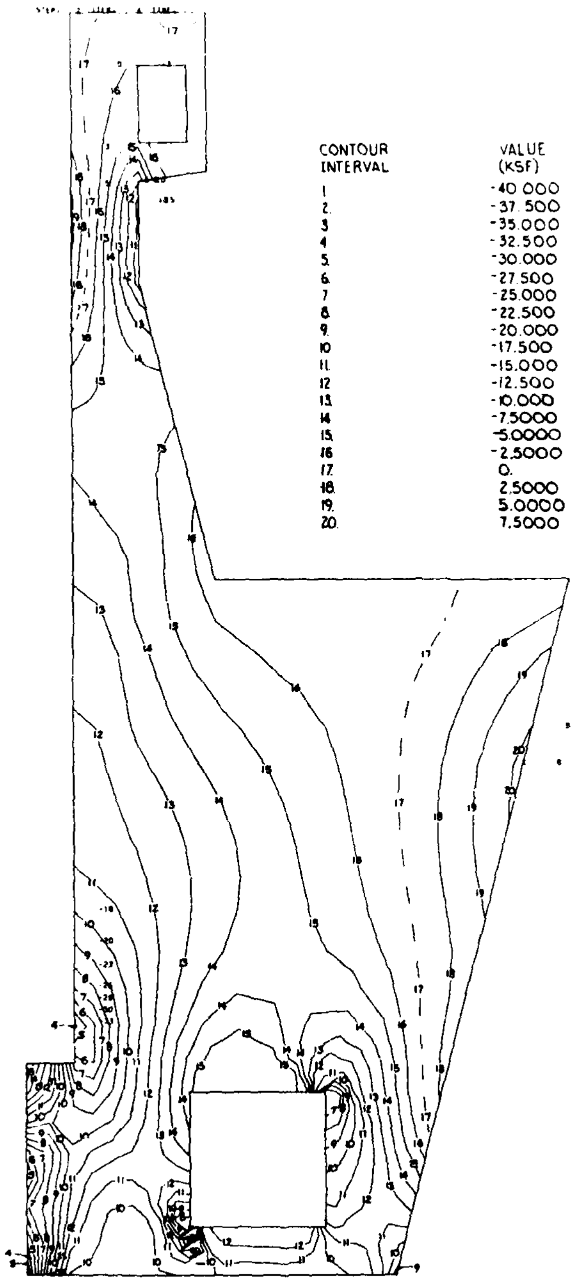
2



TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE C-6



CONSTRUCTION CONDITION --- TENDONS STRESSED --- SIGMA Y

TENNESSEE TOMBIGBEE WATERWAY
MISSISSIPPI AND ALABAMA
BAY SPRINGS LOCK AND DAM

MONOLITH R22
STRESS ANALYSIS
BY FINITE ELEMENT METHOD

PLATE C-7

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

McGee, P Thomas
Documentation of finite element analyses; Report 2: Anchored wall monolith, Bay Springs Lock / by P. Thomas McGee, U.S. Army Engineer District, Nashville, Nashville, Tenn. (Automatic Data Processing Center. U.S. Army Engineer Waterways Experiment Station) ; prepared for Office, Chief of Engineers, U.S. Army -- Vicksburg, Miss. : U.S. Army Engineer Waterways Experiment Station ; Springfield, Va. : available from NTIS, 1980.
26, [6] p., [26] leaves of plates : ill. ; 27 cm. -- (Technical report / U.S. Army Engineer Waterways Experiment Station ; K-80-4, Report 2)
Cover title.
"December 1980."
"A report under the Computer-Aided Structural Engineering (CASE) project."

1. Bay Springs Lock and Dam. 2. Finite element method. 3. Locks (Waterways). 4. Navigation dams. 5. Tennessee-Tombigbee Waterway. I. United States. Army. Corps of Engineers. Office of the Chief of Engineers. II. United

McGee, P Thomas
Documentation of finite element analyses...
1980. (Card 2)

States. Army Engineer Waterways Experiment Station. Automatic Data Processing Center. III. Title. IV. Series: Technical report (United States. Army Engineer Waterways Experiment Station) ; K-80-4, Report 2.
TA7.W34 no.K-80-4 Report 2

WATERWAYS EXPERIMENT STATION REPORTS PUBLISHED UNDER THE COMPUTER-AIDED STRUCTURAL ENGINEERING (CASE) PROJECT

	Title	Date
Technical Report K-78-1	List of Computer Programs for Computer-Aided Structural Engineering	Feb 1978
Instruction Report O-79-2	User's Guide: Computer Program With Interactive Graphics for Analysis of Plane Frame Structures (CFRAME)	Mar 1979
Technical Report K-80-1	Survey of Bridge-Oriented Design Software	Jan 1980
Technical Report K-80-2	Evaluation of Computer Programs for the Design/Analysis of Highway and Railway Bridges	Jan 1980
Instruction Report K-80-1	User's Guide: Computer Program for Design/Review of Curvilinear Conduits/Culverts (CURCON)	Feb 1980
Instruction Report K-80-3	A Three-Dimensional Finite Element Data Edit Program	Mar 1980
Instruction Report K-80-4	A Three-Dimensional Stability Analysis/Design Program (3DSAD)	
	Report 1: General Geometry Module	Jun 1980
Instruction Report K-80-6	Basic User's Guide: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Dec 1980
Instruction Report K-80-7	User's Reference Manual: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Dec 1980
Technical Report K-80-4	Documentation of Finite Element Analyses	
	Report 1: Longview Outlet Works Conduit	Dec 1980
	Report 2: Anchored Wall Monolith, Bay Springs Lock	Dec 1980
Technical Report K-80-5	Basic Pile Group Behavior	Dec 1980

DATE

LMED

8