COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
VENTURA, CALIFORNIA

MATILIJA DAM

STRESS INVESTIGATIONS

AUGUST 1972

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213-302

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January 24, 1973

Mr. A.P. Stokes, Director
Department of Public Works
County of Ventura
597 East Main Street
Ventura, California 93001

Attention: Mr. Gordon Marsh
Project Manager

Gentlemen:

We are pleased to submit twenty-five (25) copies of our report, entitled:

MATILIJA DAM
STRESS INVESTIGATIONS.

The report presents the results of investigations conducted to evaluate the stress conditions of Matilija Dam under various loadings. The studies were conducted using the latest analysis techniques, and the results provide a comprehensive and realistic representation of the structural action of the dam.

We wish to express our appreciation for the assistance and cooperation received from your staff during the course of the studies and thank you for giving us the opportunity to assist you on this important investigation.

Very truly yours,

G.S. Sarkaria

G.S. Sarkaria
Vice President - Engineering

GSS/EBK/lt

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CONTENTS

PLATES

FIGURES

FIGURES

EXHIBITS

EXHIBITS

EXHIBITS

APPENDIXES

SUMMARY

The investigations presented in this report were conducted to: 1) determine the state of stresses in the existing Matilija Dam; 2) evaluate the structural stability and adequacy of the footbridge; and 3) verify the reliability of the abutment deformation measuring instruments. Stress studies were made for both static and dynamic loading conditions using the three-dimensional finite element method of structural analysis. Analyses criteria were established by extensive review, interpretation and correlation studies of the test data and field measurements available in the reports supplied by the County of Ventura, Department of Public Works.

The analyses for static loading determined stresses throughout the structure due to the normal and maximum water and existing silt load, and combination of normal water and silt load stresses with those due to temperature drop and effective chemical expansion. A range of incremental stresses was established for the anticipated future silt level increase and for theoretical and effective chemical expansion caused by alkali-aggregate reaction in concrete. Ranges of displacements and stresses that would result in the event of yield of the right abutment at lower level were also determined. Results of all these studies are presented in Exhibits IV-1 through IV-13.

The following stress studies were made:

STUDY I - Static 3-D FEM, Reservoir El. 1095, Silt El. 1040

STUDY II - Static 3-D FEM, Reservoir El. 1113.7, Silt El. 1040

STUDY III - Static 3-D FEM, Reservoir El. 1095, Silt El. 1040, Maximum Temperature Drop

STUDY IV - Static 3-D FEM, Incremental Stresses for Chemical Expansion

STUDY V - Static 3-D FEM, Reservoir El. 1095, Silt El. 1040, Assumed Effective Chemical Expansion

STUDY VI - Static 3-D FEM, Incremental Stresses for Increase in Silt Level from El. 1040 to El. 1069 and Incremental Stresses for Abutment Deformation.

Dynamic response of the dam with full pool at El. 1095, was analyzed for two seismic ground motions: one (Study I-EQ1) representing an earthquake of Richter magnitude 8+ originating on the San Andreas fault and the other (Study I-EQ2) an event of Richter magnitude 6-1/2 to 7 occurring on the Santa Ynez fault. The results of these analyses are presented on Exhibits V-4 through V-12.

The principal results of these investigations are summarized as follows:

- Based on observed deformations and deflections, the average effective modulus of elasticity of the concrete in the major portion of the dam is in the order of 2,000,000 psi. For the normal water and silt load, the maximum normal stresses were 693 psi compression and -47 psi tension, and the maximum principal stresses were 732 psi compression and -51 psi tension in the arch. For the maximum reservoir water surface level and existing silt level, the maximum normal stresses were 855 psi compression and -110 psi tension. The preceding stress results are for static behavior of the dam.

Normal arch stresses for the normal and maximum water and silt load are generally compressive throughout the dam, except for an isolated zone near the base, and in the most part are less than 600 psi. For the case of maximum water load, a zone of cantilever tension covering almost one-half of the arch and for full height, is indicated at the upstream face, with tension averaging -65 psi and reaching a maximum of -110 psi near the base.

Incremental stresses due to temperature drop, when superimposed on normal load stresses, introduce tension zones in both arch and cantilever stresses near the abutments at the upstream face.

- The spectral acceleration response associated with the fundamental period of vibration of the dam for specified damping ratio is close to the maximum value likely to be produced by the ground motions representing the two design earthquakes. High dynamic compressive and tensile stresses with maximum values of ±1002 psi are indicated in the dam when it is subjected to the earthquake loadings. Maximum combined stresses (static plus dynamic) are compression of 1323 psi and tension of -681 psi.

- Correlation of computed and field-measured values indicates that chemical expansion due to alkali-aggregate reaction is confined to the concrete above El. 1095 in the shoulders flanking the existing spillway section. Chemical expansion in the upper concrete, above El. 1095, caused excessive cracking, rendering the concrete structurally ineffective. Cracking of the upper concrete due to chemical expansion relieved the high resultant stresses without significant transfer of stresses to lower concrete, which remained intact. The effect of incremental stresses due to assumed effective chemical expansion, on the normal load stresses was to reduce arch compression and tension at the downstream face and increase cantilever tension at the downstream near the abutments, the resulting combined maximum cantilever tension being -375 psi.

- Questionable deformation values indicated by meter DH-3R were not caused by the differential yield of the lower right abutment, but by the erratic functioning of this meter after December 1969 due to solidification of its flexible bellows by dried mud.

- Dam blocks "L" and "M", which support the footbridge near the left abutment, are badly cracked and deteriorated. While they and the footbridge are stable under maximum reservoir load (El.1113.7) or for horizontal acceleration up to 0.1g applied at El.1095, the footbridge would be unsafe under the high seismic accelerations that would occur at this level due to the ground motions generated by the design earthquakes.

RECOMMENDATIONS

For further evaluation of the structural behavior and physical quality of concrete in Matilija Dam, it is recommended that the following monitoring and testing program be undertaken in practical sequential stages:

- A system of targets suitably spaced, should be installed on the crest, on the downstream face of the dam and at both abutments as part of the long-term program of monitoring the movements of the structure. These targets should be surveyed at quarterly intervals during the year by precise triangulation. Readings should also be taken at times of significant variations in reservoir water surface level and ambient temperature conditions. Data obtained from target surveys should be correlated with data from foundation deformation meters and marker plate surveys.

- All the meters of the present foundation deformation monitoring system should be checked and those found to be malfunctioning, defective or inoperable should be repaired or replaced by new ones.

- The present fortnightly schedule of reading the deformation meters should be continued. However, readings should also be taken for the high and low reservoir levels, with weekly readings for the duration of the full reservoir.

- Periodic detailed visual examination and mapping of cracking in both faces of the dam and at the spillway crest and sides should be undertaken.

- The present footbridge, supported by the cracked and deteriorated concrete above El.1095 at the left of the spillway, should be replaced by a steel-truss bridge similar to the central spans. The new pier required for support should be founded on the sound concrete below El.1095. The condition of the concrete supporting the footbridge on the right side should be carefully examined to determine whether any cracks have developed that could effect the stability at this section.

- Comprehensive tests should be conducted on concrete cores secured from different locations in the dam to determine the present structural properties of the concrete in the existing dam.

- Petrographic examination of portions of concrete cores should be performed to determine the presence of or potential for alkali-aggregate reaction and chemical expansion.

- Thermometers should be embedded at suitable locations in the core drill holes before these are grouted, to monitor the temperature conditions in the dam concrete, and ambient temperatures concurrently with measurements of the movements of the structure.

- Sonic testing of concrete in the dam for evaluation of its in situ quality should be undertaken if core drilling reveals presence of poor quality concrete.

CHAPTER I

INTRODUCTION

1.1 GENERAL

Matilija Dam, which was constructed in 1946-47, is a thin concrete arch structure located on Matilija Creek near Wheeler Springs in Ventura County, California. It is owned by the County of Ventura, Department of Public Works, and operated by the Casitas Municipal Water District.

The dam, with a crest length of 620 ft, was originally constructed with a central spillway crest at El. 1125 and average height of 190 feet above lowest foundation block at El. 935. The thickness of the arch at the crown varies from 8 feet at El. 1123 to 35 feet at the base, El. 960. Gravity thrust blocks are located on both abutments, the left thrust block being larger than the right. A special structural feature of this dam is the slip joint at El. 960 between the arch and foundation block. This joint was provided to prevent transmission of shear loads to the fractured and faulted foundation material underlying the dam. The joint was constructed by covering the top of the foundation block with graphite mortar and 1/8-inch-thick Johns-Manville asbestos sheet.

Profile of the damsite is unsymmetrical. The right abutment below El. 1100 has a steep slope averaging about 24° from the vertical, while the left abutment has a nearly uniform slope of about 35° from the vertical. In general, the right abutment is composed of sound sandstone with interbeds of weak shale nearly normal to the thrust of the dam. A major shale bed about 3 feet wide, discovered in the abutment, was excavated and backfilled with concrete during construction to strengthen the foundation. The left abutment is composed of fractured beds of sandstone and shale, generally at an acute angle to the resultant thrust of the dam. The channel section rock reportedly consists of crushed sandstone, shale and, probably, fault gouge.

The first detailed review of the safety and stability of the Matilija Dam was conducted by Bechtel Corporation, San Francisco. The results are presented in the report, "Review of Matilija Dam," dated February 1965. In 1965 the dam was modified, to improve its factor of safety, by removing the significantly decomposed and reactive concrete in the dam between Sta. 1 + 75 and Sta. 4 + 55, down to El. 1095. The new spillway crest was set at El. 1095, thus lowering the normal full pool level from El. 1125 to El. 1095. The maximum reservoir level for passing the probable maximum flood of 70,000 cfs is El. 1113.7.

A system of abutment deformation monitoring devices was installed in 1965, and a program of regular surveillance was started, as recommended in the report. Observations of the deflections of the dam from survey markers on the dam, started at quarterly intervals in 1950, have been continued. A second review of Matilija Dam was made by Bechtel Corporation in 1967 and presented in the report, "Review of Matilija Dam," dated August 1967.

1.2 SCOPE OF INVESTIGATIONS

This report presents results of investigations conducted to evaluate stress conditions in Matilija Dam in July-August 1972. The studies included the following:

(A) Static Stress Analyses - Including three dimensional stress analyses for dead load, water load, silt load, temperature load and chemical expansion using the finite element method.

(B) Dynamic Stress Analyses - The dynamic response of the structure to representative ground motions using the three dimensional finite element method.

(C) Review of the structural adequacy and safety of the crest foot bridge and piers.

(D) Review of field data to determine reliability of the foundation deformation meters.

1.3 ACKNOWLEDGMENTS

This work was performed under the general supervision of Mr. S. Gordon Marsh, Project Manager for the County of Ventura, Department of Public Works, Ventura, as conveyed in the contract dated May 8, 1972, with the International Engineering Company, Inc.

Valuable assistance was rendered by Dr. G.W. Housner, California Institute of Technology, Pasadena, who provided recommendations for the seismic ground motion criteria used in the dynamic analyses. Dr. Roy W. Carlson provided valuable assistance in checking the physical conditions of the instruments and made recommendations for their future maintenance.

Static and dynamic stress analyses and review of the instrument data were performed by IECO engineers H.E. Jackson and R.P. Sharma, and studies of the foot bridge were made by T.L. Kardos. The investigation was under the direct supervision of E.B. Kollgaard, IECO Project Manager, under the general supervision of D. Zayakov, Chief Civil Engineer, and G.S. Sarkaria, Vice President - Engineering.

CHAPTER II
STRESS ANALYSIS PROGRAM

2.1 GENERAL

During the past few decades the trial load method has been the most widely used technique for three-dimensional stress analysis of arch dams. However, this method is being superseded by the recently developed three-dimensional finite element method (3-D FEM), which offers greater flexibility.

The finite element method is a powerful tool for the analysis of various problems in structural and continuum mechanics. It has great advantage over other numerical methods in handling structures with irregular geometry and variations in material properties. Thermal, mechanical, pressure and inertia loadings can be accommodated, and various boundary conditions can be applied. Development of the method and its successful application to various engineering problems has been extensively reported in published literature and is therefore not presented here.

The reliability and accuracy of the 3-D FEM was verified by comparing the results of studies of two arch dams, recently made by IECO using both the trial load method and the 3-D FEM. The results obtained from the two methods agree closely. The finite element method was ideally suited for the stress analyses of Matilija Dam, since several preliminary studies using different material properties and loading conditions were required to develop realistic criteria for the final analysis.

2.2 3-D FEM PROGRAM

All static stress studies for Matilija Dam were made using the finite element structural analysis computer program "3D-SAP". This program is a modified version of "SAP", a general-purpose structural analysis program developed at the University of California by Professor E.L. Wilson. It is used specifically for the solution of problems in three-dimensional structural systems. The program used for the Matilija Dam stress studies performs static linear elastic analysis of arbitrary three-dimensional solid structures using eight-node hexahedron or brick-shaped elements, subjected to concentrated or distributed (pressure) loadings, thermal expansion, and static gravity forces. The displacements and stresses are determined with respect to a global coordinate system.

A post-processing program developed by IECO computes normal and principal stresses on the upstream and downstream faces of the dam using the global stresses computed by 3D-SAP. The computer work associated with the extensive data preparation and card punching for this program was performed on a UNIVAC 1108 computer. The 3D-SAP and post processing programs were executed on a CDC 6600 computer. Plots were made by a CALCOMP pen plotter from data generated by the computer.

2.3 GEOMETRY OF DAM AND 3-D FEM MESH

Data for the geometry and details of the layout of the dam, thrust blocks and topography of the foundation and abutment area were obtained from drawings furnished by the County and relevant drawings in the reports, "Review of Matilija Dam," dated February 1965 and August 1967. General arrangement plan and profile of the dam and appurtenant features are shown on Exhibit No. II-1.

Finite element idealization of the structure was based on realistic representation of the geometry and special structural features. Regular eight-node hexahedron (brick-shaped) elements were used to model the dam and foundation. A sufficient portion of the foundation and abutment rock was included for adequate representation of the interaction between the structure and its foundation. The thickness of the dam was modeled by elements one row thick. The dam-foundation contact was modeled in a stair-step configuration using the skewed hexahedron elements to more closely represent the actual foundation and abutment excavation profile.

To preserve reasonable proportions of length to width to height in the elements, the dam was divided into rows of bricks, 20 to 30 feet high and 30 to 40 feet long in the direction of the arch. The same general pattern is projected into the abutment rock, with elements added upstream and downstream to adequately model the foundation and abutment topography. Reduction of arch thickness at the contraction joints as described in the report Structural Analysis of Matilija Dam dated March 1965 by the MacNeal-Schwendler Corporation, was not modeled since such reduction was not considered necessary.

The horizontal slip joint at El.960, between the arch and the foundation, was represented by a physical separation of the elements on either side of the joint. The actual structural effect of the slip joint was approximated by applying constraints on the nodal points along the slip joint, which allowed the elements above the joint to have free movement in the horizontal plane, while restraining their movement in the vertical plane. The weight of the dam above the joint was applied on the base as equivalent concentrated nodal point loads in the downward direction, while frictional forces acting along the joint were represented by horizontal nodal point loads in upstream and downstream radial directions on nodal points above and below the joint, respectively.

The mathematical model used in these stress studies consisted of a mesh of 610 elements (111 of which represented arch dam concrete) connected by 1112 nodal points. This layout required the computer to solve 2666 equations with a "bandwidth" of 195.

The element layout along a developed profile of the dam, including foundation and abutment elements, is shown on Exhibit II-2. Zones of different physical properties, in both rock and concrete, are also indicated. Cross section reference numbers refer to generally radial vertical sections, on which the nodal points were sequentially numbered in order to minimize the bandwidth. Exhibits II-3 through II-5 show plan views of the element layout at various horizontal lifts, viewed from the top surface of each tier of elements. The concrete and abutment element numbering sequence, cross section numbers and nodal point boundary constraints are also shown. Exhibits II-6 through II-8 are isometric views plotted by computer of the 3-D mesh, showing the element assembly in the right and left sides of the concrete arch and the crown cantilever section, respectively. Mesh plotting is a quick, economical method of verifying the complex 3-D geometric input data and provides useful visual demonstration of the element assembly. The elements in the foundation and abutment rock have been omitted for clarity.

CHAPTER III
CRITERIA DEVELOPMENT

3.1 PROCEDURE

Development of criteria for the stress investigations of Matilija Dam involved preliminary studies usually not required in performance of stress analyses for an arch dam. Although only limited test data and information regarding the physical properties of the concrete in the dam and the foundation and abutment rock were available, reasonable property values could normally have been selected for an arch dam stress analysis and representative results obtained. For Matilija Dam, however, the unknown influence of chemical expansion of the concrete immensely complicated the problem. Not only were the rates of actual expansion unknown, but it was also unresolved whether the expansion had occurred only in the obviously afflicted region above El.1095 or, to a lesser degree, throughout the dam.

It thus became mandatory to isolate the effects of various loadings to first confirm the preliminary assumptions as to the elastic properties of both concrete and foundation rock. Once this was accomplished, it was possible to determine, from analyses of the structure's response, the theoretical chemical expansion that had taken place and reach a qualitative judgment as to the effective expansion.

Field data, including periodic survey measurements of deflection plates installed on the dam and regular readings of abutment deformation meters, were supplied by the County. Periods of record were carefully selected so that the incremental measured deflection and deformation values resulting from identifiable loading conditions could be compared with computed values from the mathematical FEM model subjected to these same conditions.

The resulting displacements of the nodal points of the 3-D FEM mesh for a particular loading case are printed out as three components in global directions by the computer. Data reduction computer programs were written and used to first interpolate between points on the mesh to match the location of particular meters or plates and then to resolve the data into either meter direction or actual radial and tangential directions of plate deflections.

The procedure described was carried out in logical sequence for the step-by-step determination of criteria for the moduli of rock and concrete and parameters to represent the effects of the chemical expansion of the concrete, as described in Sections 3.5 and 3.6.

3.2 STATIC LOADING CONDITIONS

With the spillway crest lowered to El. 1095, the controlling reservoir water surface levels for stress analysis are full reservoir (El. 1095) and maximum reservoir (El. 1113.7). Survey data supplied by the County show the present silt level to be about El. 1040. Future silt level has been estimated to rise to El. 1069. In addition to the water and silt load, the present and future structure loadings include those due to temperature changes and the chemical expansions of concrete due to alkali-aggregate reactivity.

Equivalent concentrated loads applied at the nodal points along the slip joint at El. 960 to represent the frictional forces were calculated using a static coefficient of friction of 0.15. These concentrated loads were applied in an upstream radial direction for nodal points on the arch dam, and equal and opposite for the foundation block.

To provide a comprehensive picture of the static stress conditions presently prevailing and likely to prevail in the structure due to various probable combinations of loadings in its service operation, loading combinations used in these studies were as follows:

- Full Reservoir W.S. El. 1095, Silt El. 1040 and Dead Load.

- Maximum Reservoir W.S. El. 1113.7, Silt El. 1040, Dead Load.

- Full Reservoir W.S. El. 1095, Silt El. 1040, Maximum Temperature Drop.

- Full Reservoir W.S. El. 1095, Silt El. 1040 and Assumed Effective Chemical Expansion.

In the finite element stress analyses these loads were applied as follows:

- Hydrostatic pressure due to reservoir level under investigation applied to dam and to faces of foundation elements exposed to reservoir.

- Equivalent hydrostatic pressure due to silt accumulation level under investigation applied to dam and to faces of foundation elements affected.

- Gravity load of dam.

- Temperature change as shown in Section 3.4 for each element assumed at its centroid.

- Chemical expansion applied as equivalent coefficient of thermal expansion for 1° temperature rise.

3.3 GENERAL CRITERIA

The following general criteria were used in these stress studies:

- Unit weight of water, 62.5 lb per cu ft.
- Unit weight of concrete, 145 lb per cu ft.

- Equivalent hydrostatic pressure of silt, 20 lb per cu ft.
- Poisson's ratio of concrete, 0.20.
- Poisson's ratio of rock, 0.25.
- Coefficient of thermal expansion, 5.6×10^{-6} per $1^{o}F$.

3.4 TEMPERATURE CHANGE

Grouting of the contraction joints in the dam is reported to have been performed during December 1947 and January 1948, following completion of concrete placement operations, but actual data on temperatures in the dam are not available. Data on maximum temperature drop from the grouting temperature are given in the Bechtel report, "Review of Matilija Dam," dated February 1965. Maximum temperature drops, based on these data interpolated to the centerline of each element, are:

Elevation (feet)	Temperature Drop (Degrees F)
1110.0	- 12.5
1087.5	- 14.2
1070.0	- 14.6
1047.5	- 14.5
1022.5	- 13.0
997.5	- 12.6
972.5	- 11.0
948.0	- 10.0

3.5 MODULI ASSUMPTIONS

Preliminary values for the modulus of elasticity of the foundation and abutment rock were estimated from data on tests on rock cores reported in the Bechtel report of February 1965. Eight rock cores were obtained from holes drilled for the foundation deformation meters, four from each abutment.

Preliminary values of modulus of elasticity for the concrete were selected after reviewing the concrete test data included in the Bechtel reports. By far, the majority of the data concerns the deteriorated concrete above El. 1095, and only a limited amount is available for concrete below El. 1095, which represents a major part of the structure.

Study of the deformation meter records (Exhibit III-2) indicated that most of the middle and lower meters behave elastically in their response to reservoir level changes. In January, 1969, the reservoir filled rapidly from a low level of El. 1036 to the spillway lip, El. 1095, in less than 7 days. Readings of the deformation meters were made on January 15 with the Res. W.S. El. 1036.37 and again on January 22 at Res. W.S. El. 1095.80. Since air temperatures at the time of the readings were in the same range, being 50 and 51°F, respectively, the incremental deformation data between these dates represent, as nearly as possible, the elastic response of the dam and foundation to a reservoir rise of over 59 feet.

The mathematical model was subjected to the same loading conditions, excluding temperature change, which is not applicable, and the computed incremental displacements were resolved into comparable deformation in the directions of the meter to confirm the selection of abutment rock moduli values. Two trials were required to obtain reasonable agreement between the computed values and actual measured values, as shown on Exhibit III-4.

To confirm the assumptions regarding the modulus of elasticity of the concrete throughout the main body of the dam, it was necessary to compare incremental deflection of the crest. The most representative survey plate is Plate No. 21 on the spillway crest at El. 1095, close to the crown cantilever. Surveys of the deflection plates are taken at approximately quarterly intervals during the year. Fortunately, this interval spanned the time of the reservoir rise cited above--one reading was taken on January 3, with the reservoir at El. 1034.44, and the next was taken on April 8, with the reservoir at El. 1088.48. After adjusting the preliminary values of concrete moduli, the computed incremental deflection of Plate No. 21 agreed closely with the measured values (0.601 computed, 0.65 measured), as plotted on Exhibit III-3. Movements of other plates on the dam showed considerable variation from the theoretical computed deflection. However, these plates are located at higher elevations on the portions of the dam flanking the spillway. Movements of plates in these areas are not only influenced to a greater extent by temperature variation, but are also strongly affected by chemical expansion of the concrete. These factors are significant over the three-month interval between survey readings.

Final criteria for average effective moduli of elasticity of the concrete and rock were:

- Modulus of elasticity of intact concrete, 2,000,000 psi.
- Modulus of elasticity of deteriorated concrete (between Sections 17 and 20 above El. 1095), 25,000 psi.
- Modulus of elasticity of left abutment rock, 1,000,000 psi.
- Modulus of elasticity of right abutment rock, 1,500,000 psi.
- Modulus of elasticity of foundation rock in the channel section, 500,000 psi.

3.6 CHEMICAL EXPANSION ASSUMPTIONS

A review of the deflection records of Plate No. 21, installed in May 1967 on the lowered spillway crest at El. 1095 near the center of the dam, did not reveal any discernible upstream permanent deflection during the last 5 years, as would have been expected if chemical expansion had taken place in the concrete below El. 1095. The results of 3-D FEM analyses for deflections due to chemical expansion of the concrete above El. 1095, which are discussed in the following paragraphs, show that while plates installed on these higher concrete blocks have moved as much as 3 inches upstream, the computed and measured values for Plate No. 21 (0.013 inch and 0.08 inch, respectively) are very small. The evidence, therefore, confirms the conclusion that the chemical expansion to date has been confined to the concrete above El. 1095.

It follows that expansion prior to 1965 was also confined to this region, primarily to the portion of badly deteriorated concrete that was removed when the spillway crest was lowered. Stresses that may have been distributed to the sound concrete by expansion before 1965 are assumed to have been relieved when the poor concrete in the spillway was removed.

Studies for determination of the chemical expansion of the concrete above El. 1095 are based primarily on the recorded movements of Plates No.3, 7 and 22 for the upper right side and Plates No. 5, 18 and 19 for the upper left side of the dam. Movements of these plates from 1965 to 1972 are shown on Exhibit III-1.

The displacements due to the chemical expansion during the period 1966-1969 were obtained from the measurements made on October 3, 1966, and October 14, 1969. Values for the period 1969-1971 were obtained from the measurements made on October 14, 1969, and October 6, 1971. The missing reading for October 6, 1971, for Plate No. 22 was approximated by using the average cyclic movement. The reservoir water surface levels and the ambient temperature conditions that prevailed on these dates are:

Reading Date	Reservoir W.S. El.	Air Temp. at Reading Time (^{o}F)	Water Temp. (^{o}F)
October 3, 1966	1054.58	60	68
October 14, 1969	1068.04	77	63
October 6, 1971	1071.83	98	65

The effect of chemical expansion on the stresses in the dam was included in the analysis by expressing the measured displacements attributable to chemical expansion in terms of equivalent coefficients of thermal expansion for one degree temperature rise. The criteria values representing the theoretical chemical expansion were estimated by trial so that the computed incremental displacements due to the chemical expansion were in reasonable agreement with the actually measured values for the selected time intervals. The computed values include the effects of slight difference in the reservoir loading prevailing on the three selected dates. All the three selected reading dates were in the month of October representing almost the same time in the yearly temperature cycle. It was therefore, assumed that the measured incremental displacement values were not significantly affected by differences in the ambient temperature since the temperature conditions in most of the interior of the dam would generally be the same during the same month in successive years.

The theoretical criteria values overestimate the actual chemical expansion because the measured plate displacements included not only the elastic structural displacements but also the crack openings in the deteriorated concrete. These values were reduced by judgment to obtain the 'effective' criteria values, which are believed to represent more realistically the actual chemical expansion effects in the concrete.

The criteria values, expressed as the equivalent coefficients of thermal expansion for one degree temperature rise, used in the stress studies are given below.

A. <u>Theoretical values</u>:

Period from October 1969 to October 1971

- Expansion coefficient for upper left abutment, 0.0016.
- Expansion coefficient for upper right abutment, 0.0007.

Period from October 1966 to October 1969

- Expansion coefficient for upper left abutment, 0.0012.
- Expansion coefficient for upper right abutment, 0.0010.

B. <u>Effective values:</u>

Period from October 1966 to October 1971

- Expansion coefficient for upper left abutment, 0.0009.
- Expansion coefficient for upper right abutment, 0.0005.

CHAPTER IV
STATIC STRESS ANALYSES

4.1 GENERAL

Several three-dimensional finite element static stress analyses were completed to determine the state of stress in the dam for different loading conditions using the criteria and assumptions discussed in Chapter III. Stresses were determined for the normal full-pool loading condition, maximum reservoir water surface elevation, and including combinations with the stresses caused by maximum temperature drop and assumed chemical expansion. Incremental stresses resulting from increase in the silt level and those from possible yield of the right abutment were also computed.

Results of stress studies are presented graphically in terms of normal arch and cantilever stresses at both faces of the dam. Principal stress values at the upstream and downstream faces are shown for the study made for the full-reservoir loading condition to give an indication of the magnitude of principal stresses, compared to corresponding normal stress values and the direction and pattern of the principal stresses. Maximum values of the stresses reported in the subsequent sections are those occurring in the main body of the arch dam. Stresses for the thrust blocks and foundation block are also shown on the exhibits. Tension areas indicated by each study are identified on the exhibits by distinctive shading. Maximum stress values are also indicated for reference.

4.2 STRESS STUDY I - FULL RESERVOIR AND EXISTING SILT LEVEL

The objective of this study was to determine the state of stresses in the dam with the reservoir at the level of the spillway lip (El. 1095) for the normal loading and the existing silt level (El. 1040). Results of this study are summarized on Exhibits IV-1 through IV-3. Horizontal normal arch (thrust) stresses at the upstream and downstream faces are shown on

Exhibit IV-1. Vertical normal cantilever stresses at the upstream and downstream faces are shown on Exhibit IV-2. Principal stresses at the upstream and downstream faces are shown graphically on Exhibit IV-3.

Maximum values of the stresses in the arch dam under Study I loading conditions are as follows:

Arch Stress		
Upstream face:	Compression	554 psi
	Tension	-38 psi
Downstream face:	Compression	693 psi
	Tension	-
Cantilever Stress		
Upstream face:	Compression	147 psi
	Tension	-47 psi
Downstream face:	Compression	152 psi
	Tension	-45 psi
Principal Stresses		
Upstream face:	Compression	554 psi
	Tension	-51 psi
Downstream face:	Compression	732 psi
	Tension	-46 psi

Zones of significant arch compression occur near the lower half of the dam in the central portion on the upstream face and near the abutments on the downstream face. Minor arch tension occurs at the upstream face near the top of the right abutment at El. 1131.5. Cantilever stresses are generally low with maximum compression at the upstream face at about El. 1070 near the right half of the dam and maximum tension near the left abutment at El. 997.5. On the downstream face, maximum cantilever compression occurs near the right abutment at El. 972.5, and maximum tension occurs near the center at El. 1070.

4.3 STRESS STUDY II - MAXIMUM RESERVOIR AND EXISTING SILT LEVEL

The maximum reservoir water surface for passage of the probable maximum flood flow of 70,000 CFS is reported as El. 1113.7 in the report, "Review of Matilija Dam," dated 1967. Study II was made to predict the stress conditions in the dam for this water loading condition and silt level at El. 1040. Horizontal normal arch stresses for Study II at the upstream and downstream faces are shown on Exhibit IV-4. Exhibit IV-5 shows the vertical normal cantilever stresses at the upstream and downstream faces.

Maximum stress values for this study are as follows:

<u>Arch Stress</u>		
Upstream face:	Compression	660 psi
	Tension	-26 psi
Downstream face:	Compression	855 psi
	Tension	-
<u>Cantilever Stress</u>		
Upstream face:	Compression	119 psi
	Tension	-110 psi
Downstream face:	Compression	235 psi
	Tension	-56 psi

Maximum arch compression on the upstream face occurs near the crown at El. 972.5, while tension occurs near the top of the right abutment at El. 1131.5. On the downstream face, maximum arch compression occurs at El. 972.5 near the right abutment. No arch tension occurs on the downstream face. Maximum cantilever compressive stress on the upstream face occurs at El. 1087.5 near the right abutment, while tension occurs at El. 972.5 near the left abutment. On the downstream face, maximum cantilever compression occurs at El. 997.5 near the left abutment, while maximum tension occurs at El. 1110.0 near the right abutment. Stress distribution patterns and locations where maximum arch and cantilever stresses occur are similar to those for Study I, except that the magnitudes of corresponding stresses are about 10 to 20% higher because of the increased water load.

4.4 STRESS STUDY III - FULL RESERVOIR, EXISTING SILT LEVEL WITH MAXIMUM TEMPERATURE DROP

The purpose of Study III was essentially to evaluate the effect of the maximum temperature drop shown on page III-4 on the stress conditions in the structure. Normal arch stresses at both faces are summarized on Exhibit IV-6, and normal cantilever stresses at the upstream and downstream faces are shown on Exhibit IV-7.

Maximum stress values for this study are as follows:

Arch Stress

Upstream face:	Compression	623 psi
	Tension	-113 psi
Downstream face:	Compression	844 psi
	Tension	-64 psi

Cantilever Stress

Upstream face:	Compression	118 psi
	Tension	-105 psi
Downstream face:	Compression	176 psi
	Tension	-189 psi

Maximum arch compression occurs near the crown on the upstream face and near the right abutment at El. 972.5 on the downstream face. Maximum arch tension occurs on the upstream face near the right abutment at El. 997.5 and on the downstream face at the left abutment at El. 1047.5. Maximum cantilever compression occurs at El. 1070 near the right third of the arch on the upstream face and at El. 972.5 close to the slip joint near the left abutment. Maximum cantilever tension occurs at El. 972.5 at the left abutment on the upstream face and at El. 1047.5 at the left abutment on the downstream face. Since the effects of drop in temperature of the dam are essentially similar to those of reservoir pressure, locations where maximum stresses occur for this loading condition are the same as those in Studies I and II.

The stress results indicate that the maximum temperature drop would cause a 15 to 20% increase over Study I values for arch compression in the central region on the upstream face and near the abutments on the downstream face. Arch tension occurs near the abutments on the upstream face. Maximum temperature drop causes development of arch tension areas on the downstream face, with a maximum value of about 64 psi in the arch dam. Cantilever compression is lower on the upstream face and somewhat higher on the downstream face than in Study I. Cantilever tension is significantly higher on both the upstream and downstream faces, compared with the values found in Study I. On the upstream face, tension zones with tension values ranging from about 50 to 105 occur in the lower half, extending from each abutment for about one-third the length of the arch. On the downstream face, a continuous zone of tension occurs in the upper part of the arch, extending from the right abutment for two-thirds the length of

the arch at that level. Tension values range from -40 to -100 psi. The maximum tensile value of -189 psi, however, occurs near the left abutment at El. 1047.5.

4.5 STRESS STUDY IV - INCREMENTAL STRESSES FOR CHEMICAL EXPANSION

This study was made to estimate the stresses indicated in the dam due to chemical expansion of the concrete, caused by the alkali-aggregate reaction that has occurred in the upper part of the dam. The effect of chemical expansion on stresses was included in the stress analysis by expressing the chemical expansion assumed to have occurred during a selected time interval in terms of the equivalent coefficient of thermal expansion. The values of the theoretical equivalent coefficients of thermal expansion used as criteria for chemical expansion in the analysis were estimated as described in Chapter III. The results of this study are summarized on Exhibits IV-8 and IV-9.

The upper values of stress shown on these exhibits represent the incremental stresses, assuming a rate of expansion that theoretically matches the deflection of the dam recorded from 1965 to 1972. The lower values are based on an assumed effective rate of expansion, which could have induced stress in the arch dam below El. 1095. Maximum incremental stress values for both cases are:

		Theoretical Assumption (psi)	Effective Assumption (psi)
Arch Stress			
Upstream face:	Compression	3681	1183
	Tension	-1124	- 331
Downstream face:	Compression	2736	805
	Tension	-1977	- 635

			Theoretical Assumption (psi)	Effective Assumption (psi)
Cantilever Stress				
	Upstream face:	Compression	1352	435
		Tension	- 309	-100
	Downstream face:	Compression	513	165
		Tension	-1167	-343

The summary clearly indicates that the theoretical expansion rate produces an unrealistic picture of the resulting stresses. This is true for two reasons: first, the excessive tensile stresses in the area between the upper expanding portion of the dam and the main body below El. 1095 are relieved by cracking in the upper part of the dam, and second, the opening of these cracks accounts for a portion of the measured deflections and, consequently, results in overestimation of the theoretical expansion rate. The assumed effective expansion rate was selected to take these indeterminate factors into account, and the results give an indication of the range of stresses that have probably been distributed to the dam, due to the continued expansion of the concrete in the upper blocks. It can be seen that, except in the boundary region between the upper blocks and main arch, arch and cantilever stress values throughout the dam are generally low.

Extrapolation of expansion rates for Study IV to reflect future chemical expansion of the dam is not meaningful for the following reasons:

- As long as the expansion is confined to the concrete in the lifts above El. 1095, the stresses induced in the main portion of the arch dam depend on the tensile stresses built up at the interface between these areas. When the tensile stress exceeds the concrete strength, the crack that develops relieves the stress transmitted to the dam. Since the tensile stress values with the effective expansion assumed to have already occurred are close to the ultimate tensile strength of concrete, further stress buildup in the portion of the dam below El. 1095 is unlikely.

- It is not possible to predict whether chemical expansion will begin in the body of the dam below El. 1095.

4.6 STRESS STUDY V - FULL RESERVOIR AND SILT WITH ASSUMED EFFECTIVE CHEMICAL EXPANSION

The incremental stresses due to chemical expansion, as determined by Study IV, were added to the stresses due to full reservoir and existing silt levels, and the combined stresses are presented as Study V. Normal arch stresses on the upstream and downstream faces are shown on Exhibit IV-10, while cantilever stresses on both faces are shown on Exhibit IV-11. Maximum stress values are indicated by arrow heads, and tension areas are shaded.

Maximum values of normal stresses in the arch dam are:

<u>Arch Stress</u>

Upstream face:	Compression	1223 psi
	Tension	-254 psi
Downstream face:	Compression	1011 psi
	Tension	-591 psi

<u>Cantilever Stress</u>

Upstream face:	Compression	503 psi
	Tension	- 77 psi
Downstream face:	Compression	194 psi
	Tension	-375 psi

Arch stresses are mainly compressive throughout the dam, ranging from about 100 to 513 psi, with isolated maximum values located above the spillway crest at El. 1131.5 on the upstream face and El. 1110 on the downstream face near the right abutment. Local arch tensions are indicated near the abutments with stress values ranging from -114 to -266 psi, with maximum values at El. 1087.5 near the left abutment. Cantilever compressions range from 5 to 503 psi on the upstream face and from 10 to 153 psi on the downstream face. Maximum values on both faces occur at El. 1110 at the right abutment. On the downstream face, a tension zone is indicated, which generally extends from the right abutment to the left abutment above El. 1022.5 with tensions ranging from -9 to -375 psi, with a maximum at El. 1070 near the right abutment. No significant cantilever tensions are indicated on the upstream face.

The zones of high stresses, compression as well as tension, in the arch and the thrust blocks occur above El. 1085 near the interface with or in the areas of alkali-aggregate reactivity. Consequently, they are not considered representative of stress conditions in most of the dam below that elevation.

4.7 STRESS STUDY VI - MISCELLANEOUS INCREMENTAL STRESS CONDITIONS

A. <u>Incremental Stresses for Future Increased Silt Load</u> - The silt loading assumed for the analyses in Studies I, II, III and V is the existing level, El. 1040. It is anticipated that the silt level may rise to about El. 1069 in the future. Therefore, this study was conducted to indicate the order of incremental stresses that would result from the anticipated increase in silt level. The incremental stresses for arch and cantilever stresses on both face are shown as the upper values on Exhibits IV-12 and IV-13.

Maximum incremental arch compression stress is 67 psi. It occurs near the right abutment at El. 972.5 on the downstream face, while the maximum arch tension increment amounts to -3 psi. Maximum cantilever stress increases are small, the compression value being about 15 psi on the upstream face in the upper part of the dam, and the tension value being about -17 psi near the left abutment at El. 972.5.

B. Incremental Stress for Possible Abutment Deformation - The purpose of this study was to determine the order of displacements and incremental stresses that would result from assumed yielding of a portion of the lower right abutment. This information was used primarily to check the validity of measurements of an apparent deformation of the lower right abutment, which were indicated by foundation deformation meter DH-3R, as discussed below and in Chapter VII. The results of the study also provide a useful indication of the incremental stresses likely to develop if yield were actually to occur.

The deformation meter indicated that a compression of 0.1371 inches occurred between October 1969 and October 1971. Stresses resulting from such deformation, assuming that the indicated measurement was valid, could not be directly computed by the computer program. Therefore, a trial analysis run was made assuming a greatly reduced value for modulus of elasticity (300,000 psi) for the lower right abutment zone between the dam and the backfilled shear zone shown on Exhibit II-1. For this condition the computed displacement in the vicinity of the meter location, resolved in the direction of meter alignment, was 0.0854 inches, and the resulting incremental stress values are shown as the lower numbers on Exhibits IV-12 and IV-13.

Since the study showed deformations inconsistent with measurements at other foundation deformation meter locations, it was concluded that abutment yield is not responsible for the questionable readings produced by meter DH-3R. This is discussed in detail in Chapter VII. The stress results are of interest, however, in indicating the response of the dam in the event of small differential yield of the abutment. The results clearly indicate transfer of dam load from the assumed weak zone to the more competent foundations. This results in concentration of stresses in the dam near the interface of the zones. However, the effect of the weak zone is quite local and confined generally to the right half of the dam predominantly near the abutment area. The maximum incremental arch compression of 127 psi occurs at the right abutment at El. 1047.5, and maximum tension of -70 psi occurs at El. 997.5; both are on the downstream face. The incremental cantilever stresses are low, with maximum values of about 40 psi.

CHAPTER V

DYNAMIC STRESS ANALYSIS

5.1 GENERAL

Matilija Dam lies in a seismically active area. It is located within 2 miles of the Santa Ynez fault, 5 miles of the Santa Ana fault and 25 miles of the San Andreas fault. No earthquake epicenters have been reported in the immediate vicinity, and none with a magnitude greater than 4 on Richter Scale have been reported within 5 miles of the dam. However, earthquakes having a magnitude of 6+ on the Richter Scale and having epicenters within 20 to 30 miles of the dam have been reported.

Because of the location of the dam in relation to these seismically active faults, stress analyses were made using the 3-D FEM mathematical model to determine the dynamic response of the dam to ground motions during earthquakes originating on these faults.

5.2 THREE-DIMENSIONAL FINITE ELEMENT DYNAMIC PROGRAM

The computer program used for the dynamic analyses of the dam described in this chapter was developed at the University of California under the direction of Professors E.L. Wilson and R.W. Clough. It performs linear elastic dynamic analysis of three-dimensional structures. The 3-D FEM program is organized to determine the natural frequencies of vibrations and related vibration mode shapes by solving the generalized eigenvalue problems using subspace iteration. The program derives and uses the consistent-mass matrix approach as compared to the lumped-mass approach for the mass of the structure.

Water pressure during earthquake has significant influence on the vibration characteristics of a dam and acts on the dam as an external force. The magnitude of the influence of the hydrodynamic pressure can be evaluated realistically only by conducting field vibration tests on the dam

for different reservoir water levels and measuring the resulting change in the vibration characteristics.

The hydrodynamic effects of the reservoir are approximated by computing an effective water mass which moves horizontally with the dam, in accordance with the Westergaard equation, derived by neglecting the compressibility of water. This mass is proportionately distributed and applied as a lumped mass to the adjacent upstream nodal points. It is recognized that the lumped-mass approach used in this program has limitations since it cannot realistically represent the actual hydrodynamic effects due to the three-dimensional dynamic interaction between dam and reservoir, including the effects of the compressibility of water. Research is currently in progress to develop mathematical relationships to model the actual three-dimensional hydrodynamic effects for use in the computer program. However, no operational 3-D FEM program for dynamic analysis with such capability is currently available.

The first step in predicting the dynamic response of a dam is to determine the mode shapes and the natural frequencies for undamped free vibrations of the structure, using an appropriate number of vibration modes. Dynamic response analyses are then performed for specified ground motion accelerograms, using the computed frequencies and the mode shapes and applicable values of the damping factor. The response of the dam for each mode is computed at each time interval used to specify the input ground motion. The total response at any time is obtained by superimposing the modal responses occurring at that time. Accelerograms representing the two horizontal components and one vertical component of the selected earthquake are read in, and displacement and stress histories due to this excitation are computed for selected nodal points and elements. Time-displacement and time-stress histories may then be plotted by the computer.

5.3 EVALUATION OF SEISMIC PARAMETERS FOR ANALYSIS

For these analyses, earthquake accelerograms representing the maximum credible ground shaking that might occur at the site of Matilija Dam were recommended by Dr. G.W. Housner, who served as consultant to IECO. His

recommendations are contained in two letter reports (dated June 2, 1972, and June 22, 1972), which are included as appendixes to this report. The two seismic ground motions recommended by Dr. Housner are designated as Design Earthquakes No.1 and 2; they represent, respectively, the following seismic events:

1) A major earthquake on the San Andreas Fault of magnitude 8+ on the Richter scale (Figure V-1) with epicenter approximately 25 miles from the dam.

2) An earthquake similar to the 1940 El Centro earthquake (magnitude 6.5 to 7.0 on the Richter scale) occurring on the Santa Ynez Fault with epicenter close to the damsite.

The analyses included the accelerograms of the two horizontal and one vertical component of the recommended events with the amplitudes and durations scaled according to Dr. Housner's recommendations. Accelerograms for three components of the Design Earthquakes No.1 and 2 are shown on Exhibits V-2 and V-3, respectively.

Response spectra for the ground motions of Design Earthquake No.1 are shown on Figures V-2 through V-5 and those for Design Earthquake No.2 on Figures V-6 and V-7.

Earthquake response spectra provide a direct measure of the maximum response of any single-degree-of-freedom system to the earthquake motion they represent; the response being in the form of velocity, acceleration or displacements of the system. These consist of a set of curves, each curve representing relationship between the response quantity and the natural periods of vibration of the system for specified damping ratio. Figures V-2 and V-4 represent the velocity (wSd) response spectra while V-3 and V-5 represent the combined earthquake response (tripartite logarithmic plot) spectra for the three response quantities namely acceleration, velocity, and displacement for the upstream-downstream and

cross-canyon components respectively of the Design Earthquake No.1. Figure V-6 and V-7 represent velocity and combined response spectra respectively for the Design Earthquake No.2. The period of vibration for which the acceleration response value for a specified damping ratio is a maximum is designated as the predominant period.

5.4 DYNAMIC 3-D FEM ANALYSIS CRITERIA

The finite element mesh layout used for the static analyses described in Chapter II was used for the dynamic analyses, with minor revisions to input data to make them compatible with the dynamic program input. The slip joint was represented by a row of elements of low-strength material to model the dynamic response of the slip joint more accurately. General criteria and material properties were assumed to be the same as those used in the static stress studies, except that the modulus of elasticity of concrete was increased from 2,000,000 to 3,000,000 psi to reflect the increased resistance of the concrete when subjected to instantaneous transient loadings. Additional criteria used in the dynamic analyses are:

Damping - Inherent damping of vibrations by the dam has a significant effect on the dynamic response of the structure. The effects of damping are included in the dynamic analysis by using a damping constant, which expresses the ratio of the actual to the critical damping as a percent. The damping present in the structure is different for each mode of vibration, depending on the order of displacement. The actual value of this damping constant can be determined by forced vibration tests on the dam itself. Extensive field vibration tests on various arch dams have been carried out in the United States and Japan. Also, several vibration studies have been made on arch dam models. A review of available data on such field and model tests shows that equivalent damping of about 5% of the critical reasonably represents the damping in thin-arch dams. For these dynamic analyses a damping constant of 5% of the critical damping was selected.

Modes - During earthquakes, arch dams vibrate in translational and torsional modes because of the random nature of seismic ground motion. All the mode shapes and natural frequencies of free vibrations of an arch dam can be obtained from the complete solution of the eigenvalue problem, which would, however, entail considerable computer effort and cost. Mode shapes associated with lower frequencies of vibration contribute most to the total response of the structure. The first six modes of vibration were considered adequately representative for these dynamic studies of Matilija Dam.

Time Interval - Response histories of the dam were computed for time intervals of 0.025 and 0.02 seconds for Design Earthquakes No.1 and 2, respectively. These values correspond to the time intervals for the ordinates of the respective accelerograms used as input. For satisfactory accuracy of results, an interval of 0.10 seconds was selected for printing and plotting the deflection and stress response histories.

Length of Record - Inspection of the design earthquake accelerograms indicated that maximum response values would be obtained if only the portion of the accelerogram record for the significant motion were used, making it unnecessary to use the entire record. Lengths of record selected for the analyses were 60 seconds for Design Earthquake No.1 and 30 seconds for Design Earthquake No.2.

5.5 DYNAMIC RESPONSE ANALYSIS RESULTS

Results of the dynamic response analyses of Matilija Dam, expressed in terms of mode shapes and frequencies of vibrations, time-displacement and time-stress histories, are given below:

Mode Shapes and Frequencies - The six mode shapes for the six related vibration frequencies plotted by the computer are shown on Exhibit V-1. The natural frequencies of vibration, expressed in cycles per second, and corresponding periods of vibration in seconds are also shown on

Exhibit V-1. The mode shapes are plotted isometrically to present the integrated relative deformed shape of the total structure in various modes. The mode shapes and frequencies are computed for full reservoir W.S. El.1095. The symmetrical and unsymmetrical mode shapes occur alternately, the first mode being symmetrical, corresponding to the fundamental (lowest) frequency of vibration. The frequencies of vibration range from 3.096 cycles per second for the first mode to 6.26 cycles per second for the sixth mode; the corresponding periods of vibration are 0.323 seconds and 0.16 seconds, respectively. The fundamental frequency of vibration of Matilija Dam is within the general frequency range of thin-arch dams. It should be noted that the mode shapes shown on Exhibit V-1 indicate only the relative deformed configurations of the structure for the indicated modes and do not represent the magnitude of the actual displacement contributions of each mode to the total displacement response of the structure.

Reference to response spectra shown on Figures V-3, V-5 and V-7, would indicate that for the fundamental period of the dam, namely 0.323 seconds, acceleration response values for 5 percent damping ratio are close to the maximum values likely to be produced by the ground motions representing the two design earthquakes.

The node point nearest the left abutment on the top arch in the sketches represents the junction of the arch with the thrust block. This point experienced a much larger movement than the corresponding point on the right abutment where the dam is much less flexible.

Time-Displacement Histories - Dynamic time-displacement histories of selected nodal points were computed at specified time intervals for the first 60 seconds of the record for Design Earthquake No. 1 and 30 seconds for Design Earthquake No. 2. Total dynamic displacements of the nodal points were computed in the three orthogonal global coordinate-axes directions, representing the upstream-downstream, cross-canyon and vertical displacements of the structure. Time-displacement histories for the up-

stream-downstream directions for both Design Earthquake No. 1 and Design Earthquake No. 2, as plotted by the computer, are shown on Exhibit V-4 for nodal point 461--the downstream crest point of the crown cantilever (Cross Section Reference No. 12). The computer plot of the time-displacements for the first 40 seconds of Design Earthquake No. 1 (the significant portion of the 60-second period studied) is reproduced on Exhibit V-4. The maximum deflection of the crest of the crown cantilever for Design Earthquake No. 1 is 1.7 inches and occurs at 31.8 seconds of the record, while for Design Earthquake No. 2 the maximum deflection is 1.2 inches and occurs at 3.7 seconds of the record. An indication of the magnitude of the contributions of modal displacements to the total displacement response can be obtained by inspection of the maximum displacement response in the upstream-downstream direction for each mode. These maximum responses do not necessarily occur at the same instant. The maximum modal responses of node point 461 for both design earthquakes are:

	Maximum Response Displacement (inches)	
Mode No.	Design Earthquake No. 1	Design Earthquake No. 2
1	1.455	0.972
2	0.386	0.328
3	0.018	0.015
4	0.209	0.247
5	0.011	0.010
6	0.190	0.197

This table shows that for points near the center of the arch the fundamental mode contributes most of the total response displacement. For points on the crest near the quarter points, the 2nd, 3rd and 4th modes are the most significant.

Time-Stress Histories - Arch and cantilever dynamic stresses were computed at specified time intervals at the upstream and downstream faces

of the selected elements. The selected elements represent locations where significant stresses were indicated by the static stress analyses and regions where the structure was expected to experience maximum response. These elements are located at arch levels, representing locations near the crest (El. 1087.5), mid-height (El. 1047.5), and the base (El. 972.5), and at three cantilever sections located near the left mid-half, center and right mid-half of the dam. The location of the selected element centerlines both in the three arch levels and in the cantilever sections designated as right, center and left cantilevers, are shown on Exhibits V-7 through V-12, which graphically present the dynamic and combined (static plus dynamic) arch and cantilever stresses at selected instants of time.

Time-stress history plots for Design Earthquakes No. 1 and 2 are shown on Exhibits V-5 and V-6, respectively, for the arch stress at the upstream and downstream faces of element No. 240, located at the central cantilever at El. 1087.5. Since the maximum response values for Design Earthquake No. 1 occur within the first 40 seconds of the response, stress plots are shown only for that duration.

Dynamic arch and cantilever stresses at the upstream and downstream faces of the selected elements for three instants of the record are shown on Exhibits V-7 through V-9 for Study I-EQ1 and on Exhibits V-10 through V-12 for Study I-EQ2. The three instants of the earthquake record were at 23.0, 31.8 and 32.0 seconds for Design Earthquake No. 1 and at 2.6, 4.3 and 9.0 seconds for Design Earthquake No. 2. These instants, which were selected after a close study of the time-stress histories of the selected elements, represent instants when maximum stress values occurred.

Maximum values of the dynamic arch and cantilever stresses indicated at the three selected instants for both design earthquakes are tabulated below.

	Design Earthquake No. 1 Time of Occurrence (seconds)			Design Earthquake No. 2 Time of Occurrence (seconds)		
	23.0	31.8	32.0	2.6	4.3	9.0
Arch Stress (in psi)						
Upstream face						
Compression	871	391	500	309	311	619
Tension	- 33	-1002	-731	-684	-712	-155
Downstream face						
Compression	521	103	245	114	213	316
Tension	- 82	- 905	-576	-549	-231	-157
Cantilver Stress (in psi)						
Upstream face						
Compression	341	351	300	290	212	164
Tension	-188	-	-218	-104	-111	-234
Downstream face						
Compression	210	13	85	41	120	124
Tension	-155	- 258	-291	-254	-120	-119

The dynamic arch and cantilever stresses tabulated above are the maximum instantaneous stresses likely to develop in the structure at the indicated time, when the dam would be subjected to the design earthquake ground motions. Higher maximum response stresses are indicated for Design Earthquake No. 1 than for No. 2. The envelope (maximum range of stress found in the dam for record studied) arch dynamic stresses are 871 psi compression and -1002 psi tension for Design Earthquake No. 1 and 619 psi compression and -712 psi tension for Design Earthquake No. 2. The envelope cantilever dynamic stresses are 351 psi compression and -291 psi

tension for Design Earthquake No. 1 and 290 psi compression and -254 psi tension for Design Earthquake No. 2.

The maximum arch stress values for Design Earthquake No. 1 are about 40 to 60% higher than for No. 2. Cantilever compressions are about 20% higher for No. 1 than for No. 2, while tensions are approximately of the same magnitude for both. Maximum arch stresses are found at Element No. 240 for all cases, while the locations of maximum cantilever stress vary (Elements 245, 173 and 310).

The combined (static plus dynamic) arch stresses corresponding to the envelope values are 1192 psi compression and -681 psi tension, while combined cantilever stresses are 416 psi compression and -276 psi tension for Design Earthquake No. 1. Because of the reversible character of the dynamic stresses, it would be possible to obtain a dynamic arch compression of 1002 psi, corresponding to the maximum tension value found for Study I-EQ1. The maximum combined arch stress considering the reversible character of the oscillating dynamic stress would therefore be 1323 psi compression and -681 psi tension, while the combined cantilever stresses would be 416 psi compression and -295 psi tension for Design Earthquake No. 1.

These results demonstrate conclusively that the ground motion generated by an earthquake of magnitude 8+ originating on the San Andreas fault would affect the dam more severely than the 1940 El Centro earthquake.

5.6 EVALUATION OF DYNAMIC STRESSES

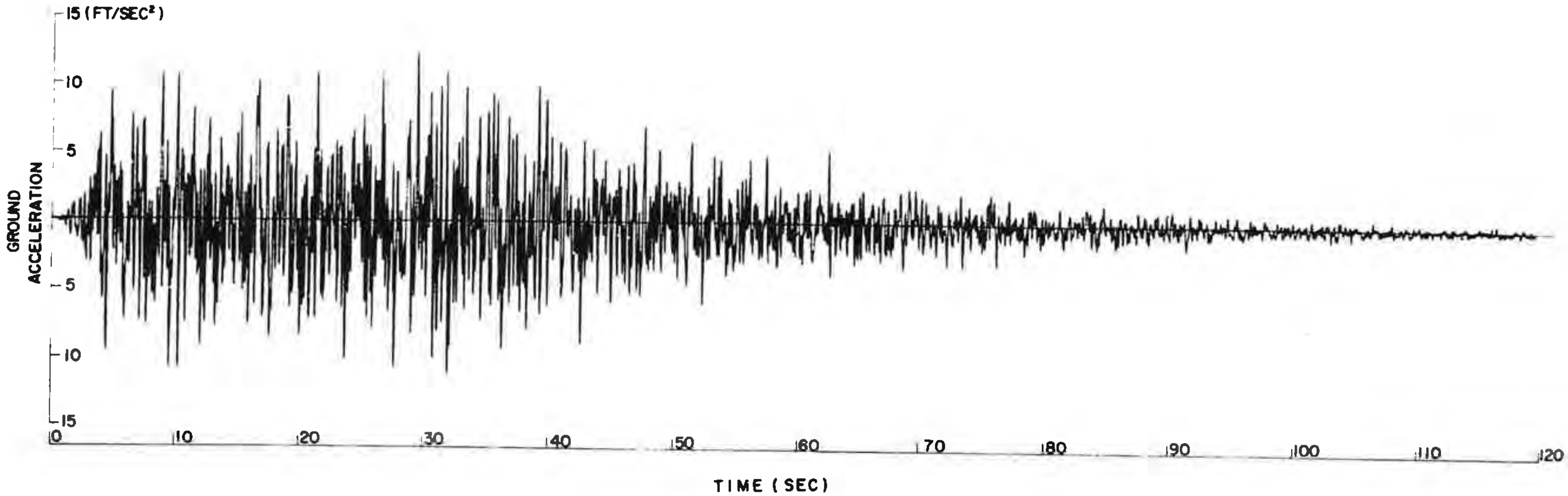
Stresses computed in arch dams under dynamic response to strong earthquake ground motions are invariably higher than those for the sustained or static loading condition. When evaluating the overall stability and safety of an arch dam for the maximum credible ground shaking likely to occur at the site, the following should be considered:

- For dynamic loading of a rapdily oscillating nature, the resistance of concrete to failure is significantly greater than its ultimate strength under a sustained static loading. Testing of concrete specimens under impact loading has indicated that the dynamic strength could be as much as 1.85 times the static strength, depending on the quality of the concrete and the rate of loading.

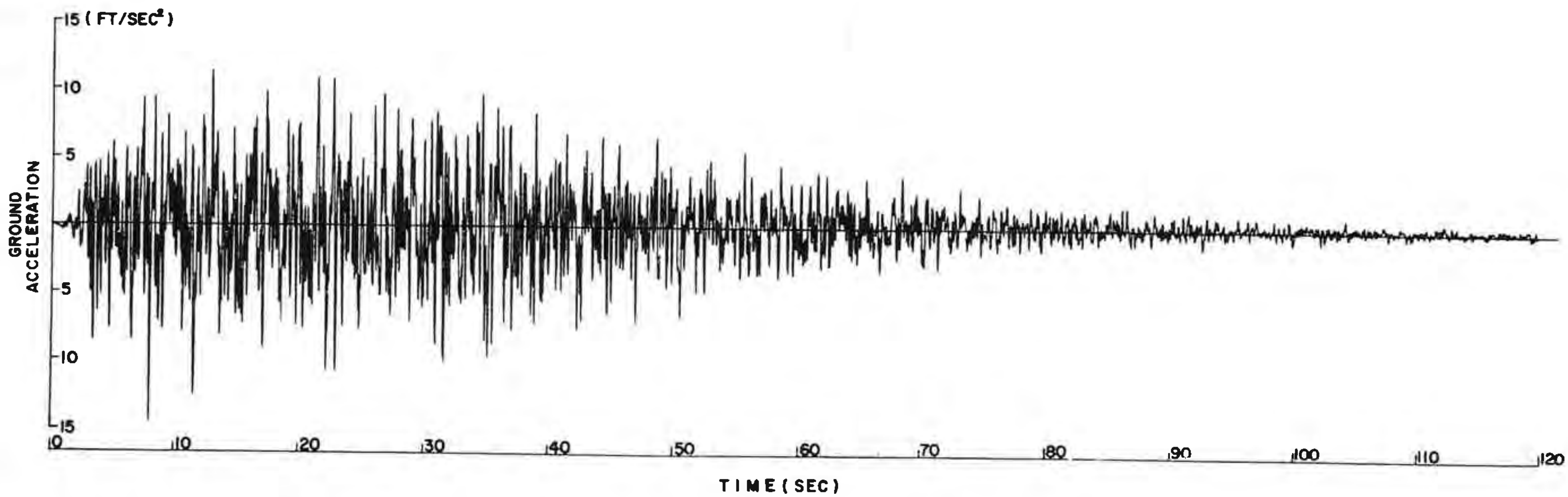
- The most significant arch stresses are indicated in the upper central part of a dam. If tensile stresses in the dam exceed the dynamic tensile strength of the concrete, this transient tension would be relieved by a slight momentary opening of the vertical contraction joints in the dam. These openings would close as the structure went into the opposite phase of the oscillating cycle and would be permanently closed when shaking ceased and the structure regained its equilibrium position.

- If compressive stresses reach the yield point in any part of the structure, plastic deformation and friction will dissipate considerable energy, increasing the damping effect and offsetting further increases in stress.

- If extensive joint openings due to tensile stresses should occur, cantilever oscillation will greatly change the dynamic characteristics of the dam. The friction between adjacent cantilevers would considerably dampen the vibrations.



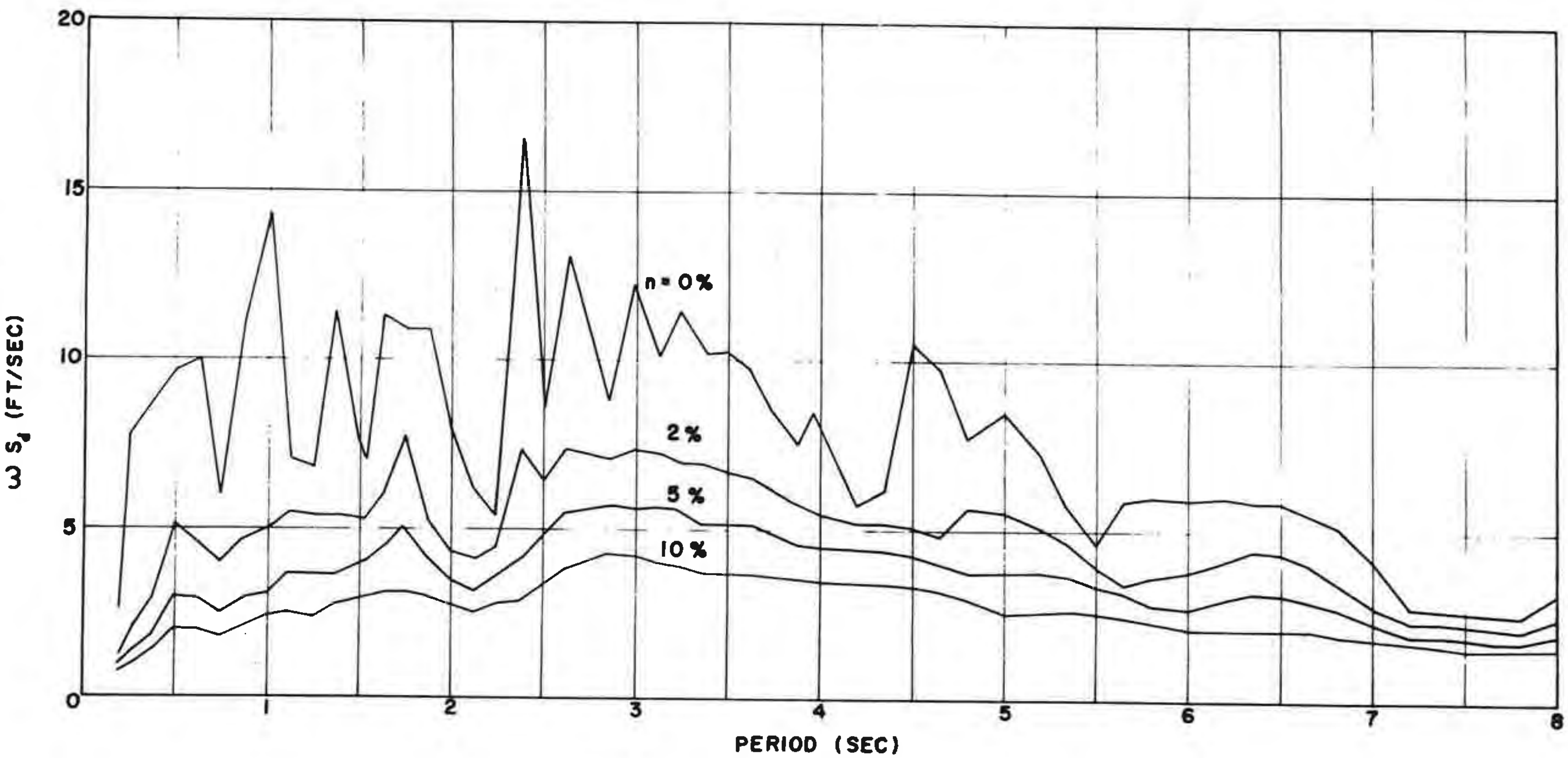
SIMULATED EARTHQUAKE MOTION BASIS FOR DESIGN EARTHQUAKE 1
Acceleration for Cross-Canyon Component



SIMULATED EARTHQUAKE MOTION BASIS FOR DESIGN EARTHQUAKE 1
Acceleration for Upstream-Downstream Component

FIGURE V-1

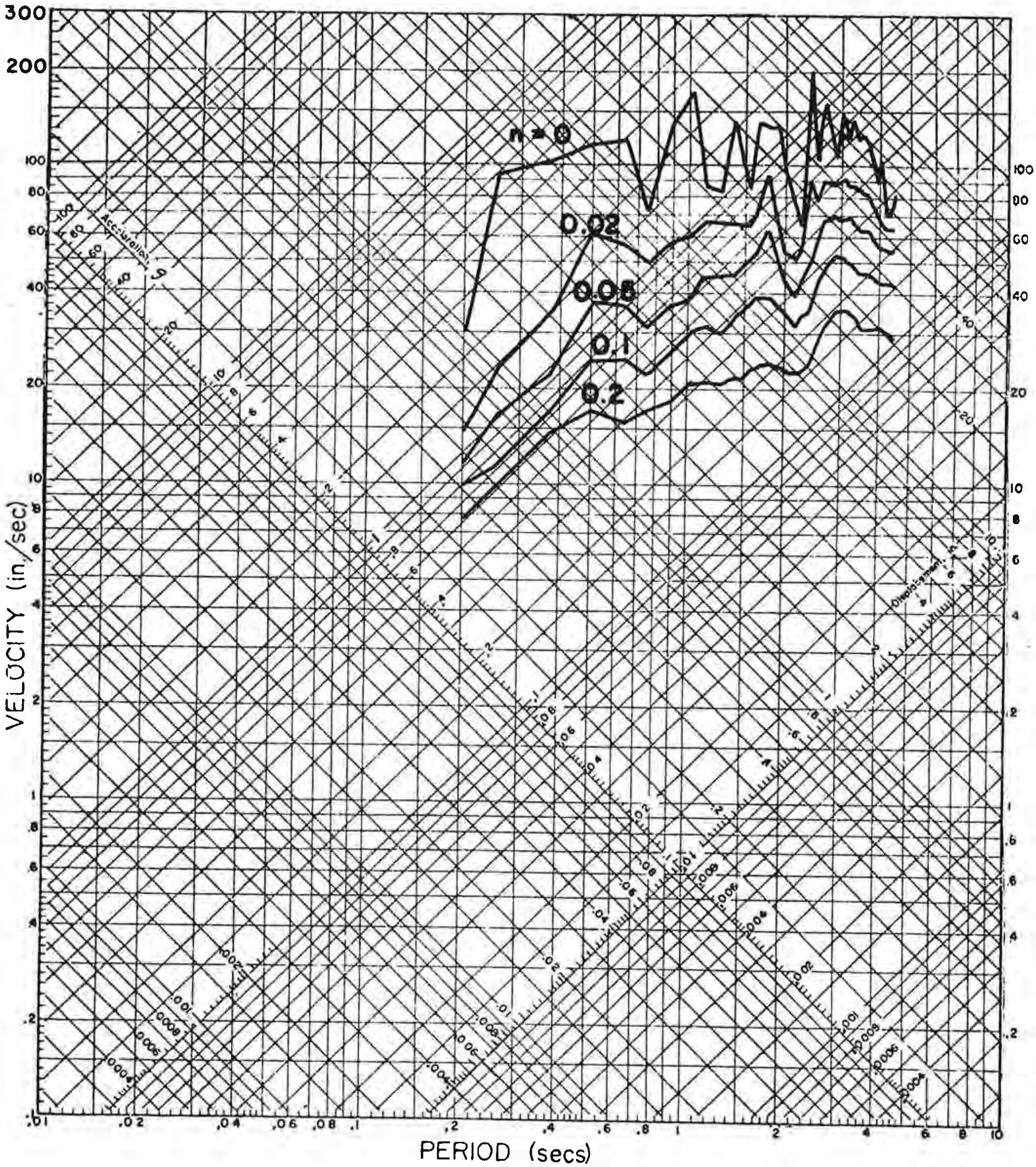
DESIGN EARTHQUAKE 1



Spectrum for Upstream-Downstream Component

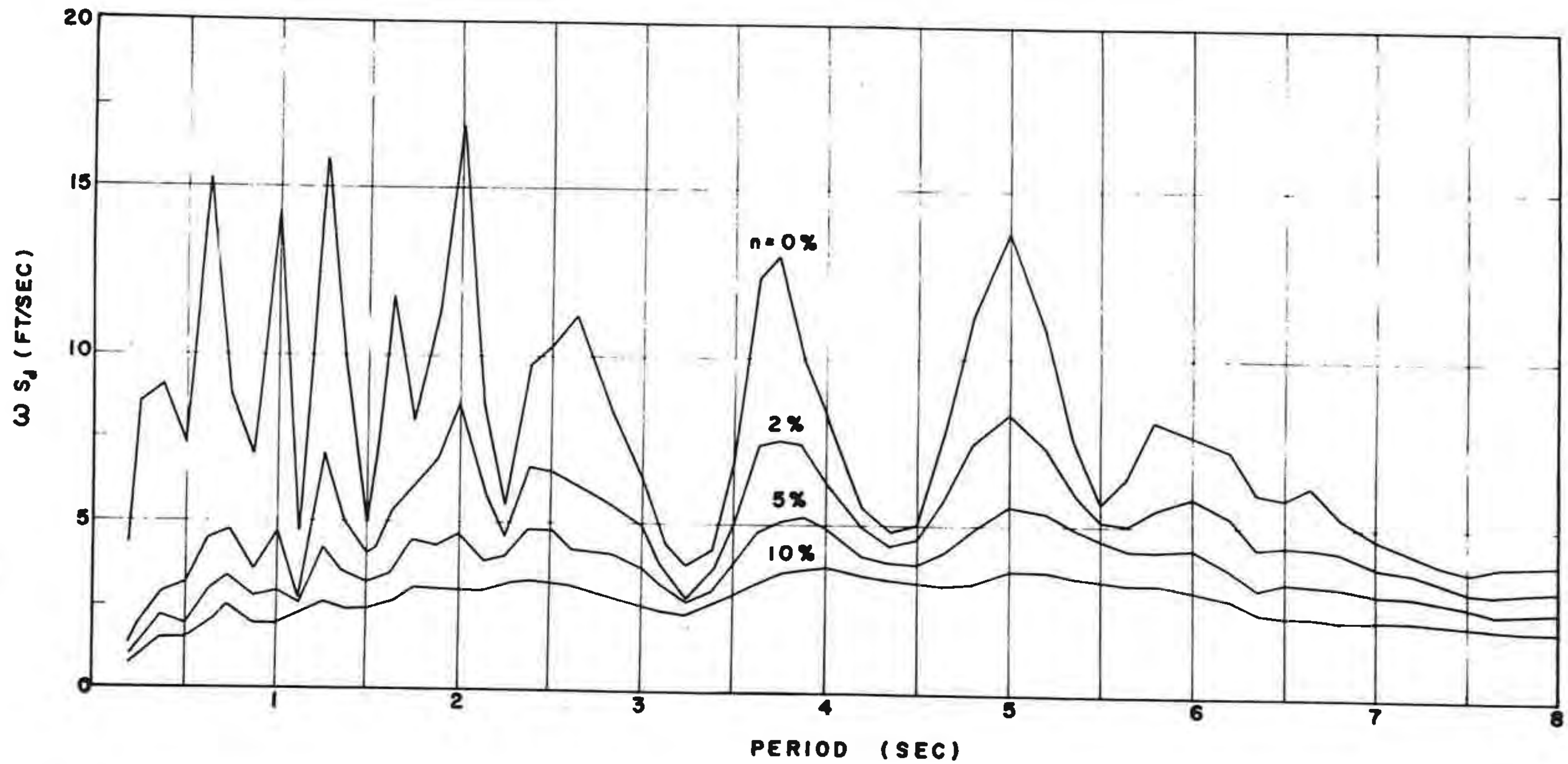
FIGURE V-2

DESIGN EARTHQUAKE 1



Tripartite Logarithmic Plot of
Spectra for Upstream-Downstream Component

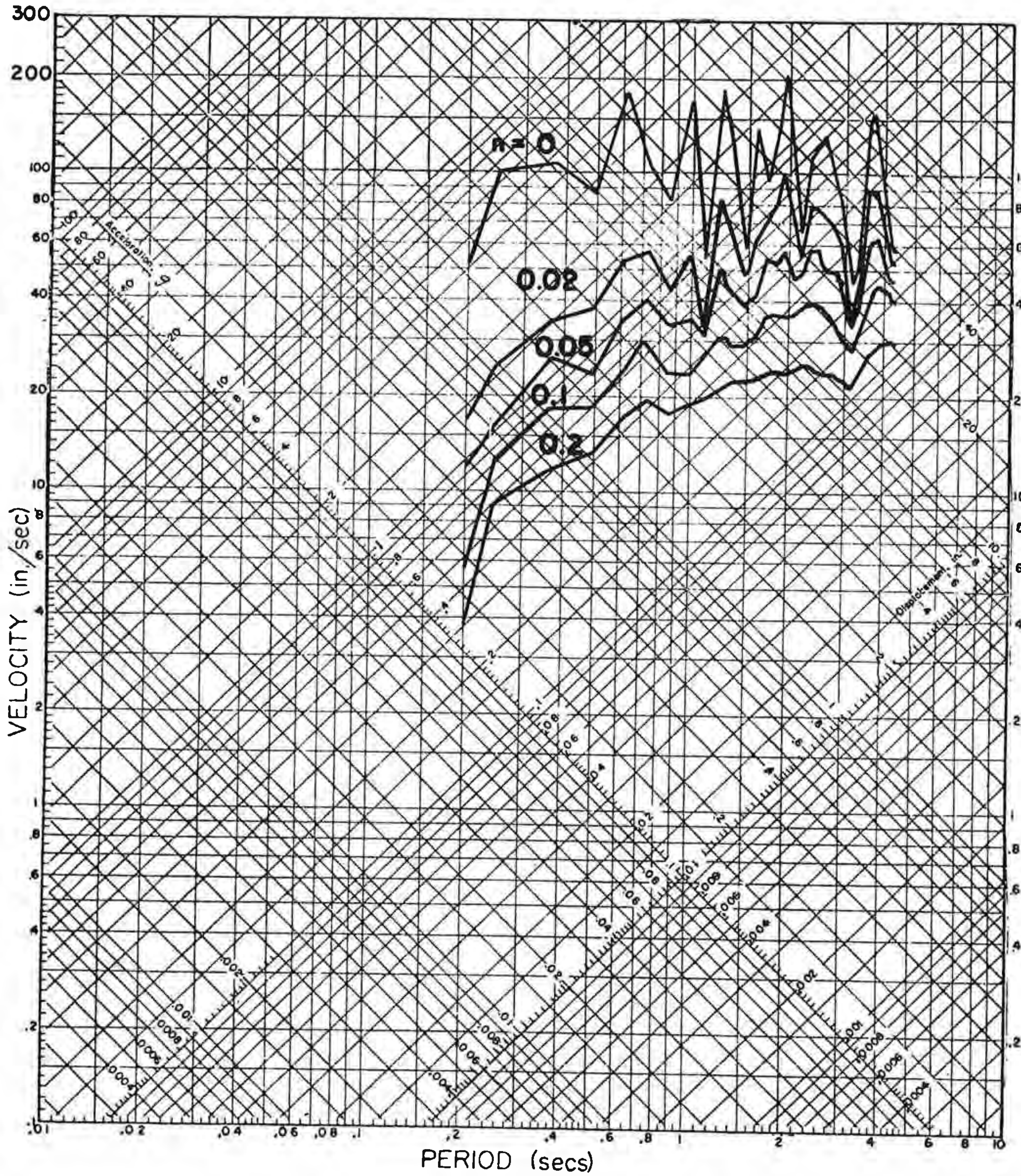
DESIGN EARTHQUAKE 1



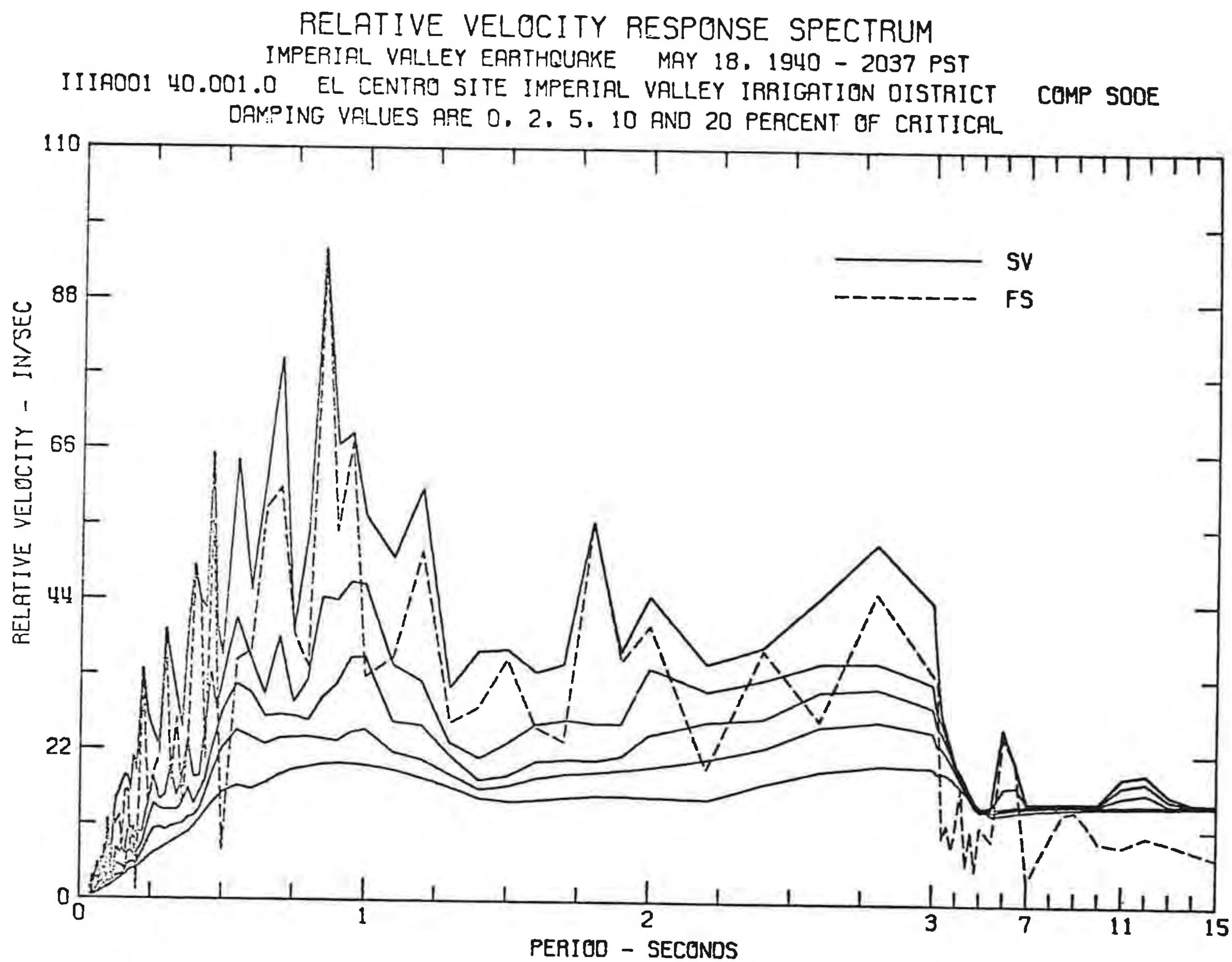
Spectrum for Cross-Canyon Component

FIGURE V -4

DESIGN EARTHQUAKE 1



Tripartite Logarithmic Plot of
Spectra for Cross-Canyon Component



RELATIVE VELOCITY RESPONSE SPECTRUM
IMPERIAL VALLEY EARTHQUAKE MAY 18, 1940 - 2037 PST
IIIA001 40.001.0 EL CENTRO SITE IMPERIAL VALLEY IRRIGATION DISTRICT COMP SOOE
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL
SV
FS
RELATIVE VELOCITY - IN/SEC
PERIOD - SECONDS
0
22
44
66
88
110
0
1
2
3
7
11
15

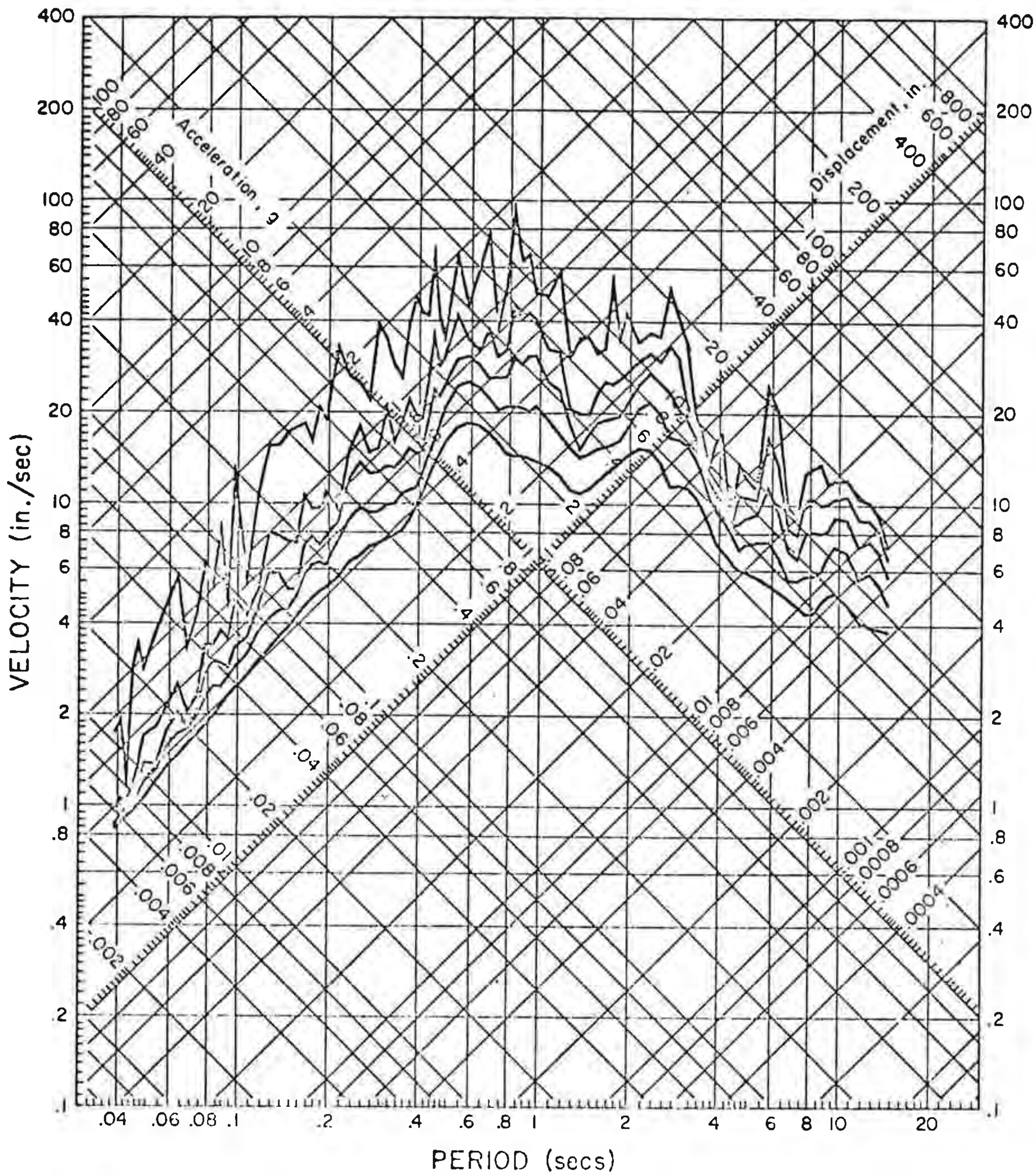
FIGURE V -6

RESPONSE SPECTRUM

IMPERIAL VALLEY EARTHQUAKE MAY 18, 1940 - 2037 PST

IIIA001 40.001.0 EL CENTRO SITE IMPERIAL VALLEY IRRIGATION DISTRICT COMP SOOE

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL



CHAPTER VI

EVALUATION OF CREST FOOTBRIDGE STABILITY

6.1 GENERAL

At the crest of the dam, service access is provided by a concrete footbridge, following the centerline of the top arch. When the crest of the central spillway was lowered, the corresponding portion of the footbridge was also removed and replace by a two-span, light, steel truss structure. Continued chemical expansion of the dam concrete above El. 1095 adjoining the left side of the lowered spillway section, severely cracked even in 1965, has caused considerable displacement at the top of the remaining piers and made bearing conditions inadequate for the bridge superstructure. Furthermore, the expansion cracks extending through the entire depth of Blocks "L" and "M" have reduced the safety of the bridge structure to an unacceptable level in the case of an earthquake or high flood loading. The cracked condition of the concrete is shown in Plate VI-1.

6.2 STABILITY DETERMINATION

The analysis was based on the assumption of a continuous crack at El. 1095.0 in Block "L" and at El. 1110.0 in Block "M". The structure above these elevations is safe in the case of a high flood with water level at El. 1113.7. The safety factor against overturning is 2.5 for this case. Considering earthquake loading, the structure could stand horizontal acceleration up to 0.1g applied at El. 1095.0. The safety factor against overturning for this case is slightly less than 2.0. However, acceleration of more than 0.1g at the spillway level would result from a considerably smaller value of ground motion acceleration at the foundation of the dam. This fact, coupled with the high probability of small earthquakes at the site, clearly indicates the necessity of some safety measures if service access is to be maintained.

6.3 ALTERNATIVE REHABILITATION SCHEMES

The three alternatives described below were considered as possible means of rehabilitation.

A. The first alternative involves removal of deteriorated concrete at the top of Blocks "L", "M" and, possibly, "N". This would also mean the removal of Piers 2, 3 and 4, the construction of a new pier at the location of present Pier 4 and the replacement of the bridge structure between Piers 1 and 4. Because of the high costs of concrete removal, the total construction costs are expected to be well above $100,000, making this alternative questionable from the economic point of view. Furthermore, removal of the horizontal restraint from the remaining blocks would undoubtedly accelerate the deterioration of these blocks due to chemical expansion. In addition, the removal of the concrete in Blocks "M" and "N" would also introduce the danger of erosion damage in the event of high floods because of the weak downstream apron in this region.

B. The second alternative involves replacement of the bridge structure between Piers 2 and 4 by a one-span, light, steel truss, removal of Piers 3 and 4 and partial removal of Block "L" from El. 1095.0 to accommodate the construction of a new pier at the location of present Pier 4. The construction costs of this alternative would be about 50% lower than in the case of Alternative A, avoiding at the same time its technical disadvantages. It must be emphasized, however, that this solution does not exclude the possibility of earthquake damage in the dam. It only ensures the safety of the bridge, by eliminating its dependence on the stability of the deteriorated blocks.

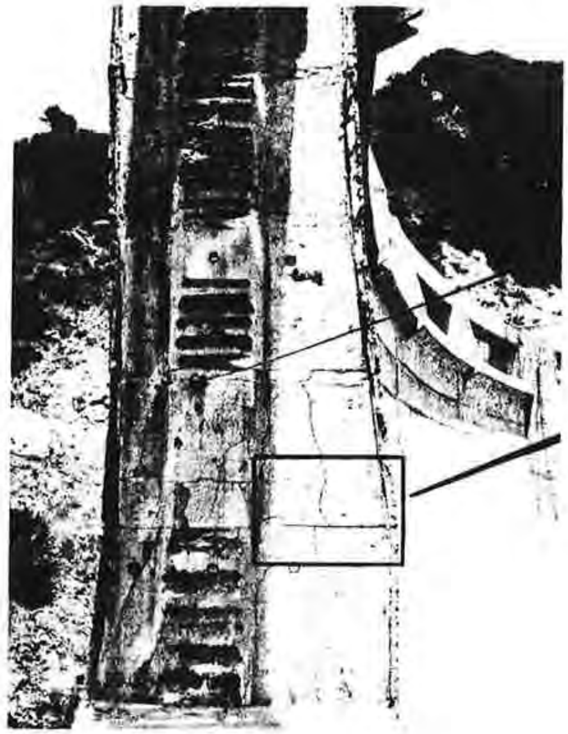
C. The last alternative studied includes provisions to make the existing structure safer against earthquake loadings by anchoring the top of Blocks "L" and "M" into the sound concrete. The construction costs of this alternative would be about the same as or slightly lower than those of Alternative B. The use of prestressed anchors would not be effective because of the extensive cracking in the concrete. One disadvantage of this

solution is that it does not stop the crest movement caused by the expansion of the concrete and thus could result in problems regarding the bearing conditions at the bridge supports.

6.4 RECOMMENDATION

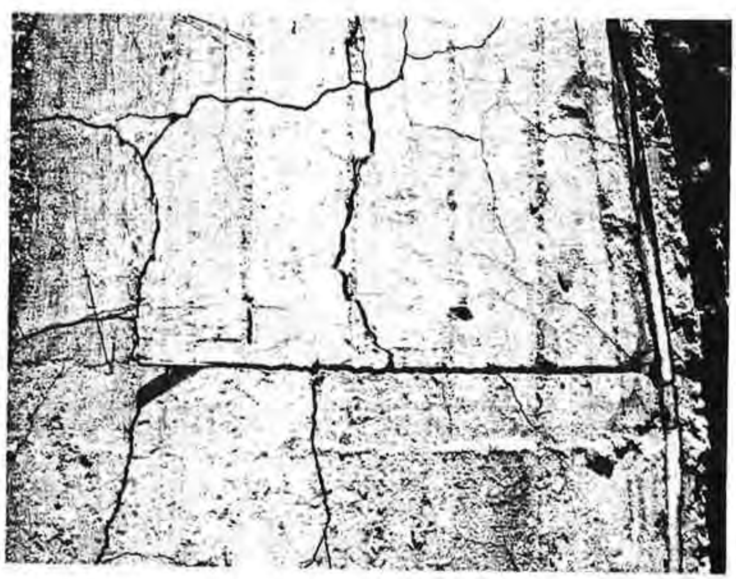
Based on the above explanations, we recommend Alternative B for rehabilitation of the footbridge. This alternative would involve removal of about 300 cu yd of concrete and construction of a light, steel-truss footbridge with a span of 92 feet similar to the existing structure over the lowered spillway section. The safety of the new bridge could be further improved by replacing three spans (between Piers 1 and 4) instead of two (between Piers 2 to 4) of the existing bridge. The additional costs due to the 42-foot increase in the span of the new bridge may well be justified, since the effects of expansion of the concrete in Block "N" would be excluded, and Pier 1, being further from the affected area, could provide safe support for the structure.

Cracking can be seen in the concrete above El. 1095 adjacent to the right side of the present spillway notch, although it is not as extensive there as on the left side. If deterioration of this concrete continues, rehabilitation of this section of the footbridge, similar to that recommended for the left side, will eventually be necessary to ensure continued safe service access.



See Detail Below

Deterioration of Concrete Above El. 1095 at Sta 1 + 75 Due to Alkali-Aggregate Reaction



Detail of Open Cracks Shown in Top Photo. Note Penny in Crack for Scale.

CHAPTER VII

PERFORMANCE AND RELIABILITY OF FOUNDATION DEFORMATION METERS

7.1 GENERAL

Eight Carlson-type foundation deformation meters were installed in 1965, four in each abutment, to monitor the abutment deformations under service loading conditions. Locations of these meters are shown on Exhibit II-1. Details of installation of these meters are described in the Bechtel report "Review of Matilija Dam", dated February 1965. Since January, 1971 the meters have been read at fortnightly intervals. All the meters have been operable except Meter 1-L which ceased functioning in July 1972.

The abutment deformations indicated by these meters have generally been consistent and as normally expected, except for readings from Meter 3-R. In December 1969, this meter indicated a change in trend and the indicated foundation yield has since gradually increased in magnitude.

The indicated large change in abutment deformation caused considerable concern about the competence of the abutment and condition of the dam, since Meter 3-R is located in the lower right abutment region between the dam and the major shear zone which had been partially excavated and backfilled with concrete during construction. It was not known if the indicated large deformations actually resulted from physical movement of the right abutment or were due to possible erratic functioning of the meter. No such sudden and significantly large deformation changes were indicated by any of the other meters. Investigations were conducted to evaluate the performance and reliability of these foundation deformation meters. This was done by careful detailed review of the observed data, theoretically determined abutment displacements and in-situ and laboratory physical checks of selected meters. Dr. Roy W. Carlson, Consultant, inspected these meters in the field and conducted a physical check of the calibra-

tion and performance of selected meters in his laboratory, Carlson Instruments, Campbell, California. His report dated September 1, 1972 is included in the appendixes to this report.

7.2 CONSISTENCY OF DATA

A. Review of Observed Data - It was recognized that consistency of trends in the variation of deformations indicated by the meters for corresponding load changes would provide some indications of the performance reliability of the meters. To evaluate consistency of observed data, the time-deformation histories indicated by all the meters for the entire period of observations and the corresponding reservoir water surface levels were plotted graphically as shown on Exhibit III-2.

The following conclusions were drawn from the detailed review of the observed time-deformation histories:

Deformations indicated by Meters 3-R, 3-L, 2-R and 2-L consistently reflect elastic behavior of the respective abutments in response to the related changes in reservoir water level. This is markedly evident when reservoir levels changed in short periods of time during which the effects of other loads; namely, temperature change, chemical expansion, etc., on the deformations would be relatively insignificant. Peaks in the deformation plots correspond with the peaks in reservoir water level at the same time (Exhibit III-2).

The measured deformations indicate that all meters, except Meter 3-R, have functioned consistently since installation. Some unsteadiness characterized the response of the Meter 3-L in the initial stages after which it has continued to indicate consistent response.

The deformations indicated by Meters 1-L and 1-R do not show close correlation and easily discernible movement in response to sudden reservoir load changes as the lower meters do. This is due to the fact that both

these meters, namely DH-1R (El. 1113) and DH-1L (El. 1112.5) are located above the normal maximum pool El. 1095 outside the region of influence of the main body of the dam, and are thus less responsive to reservoir load. They are however, more susceptible to temperature change, since the relatively thin concrete dam in this region is exposed to air on both sides. The deformation plots show the response of these meters to be generally consistent with the annual cyclic temperature variation; expansion of the dam during warm months causing inward abutment movement is indicated by compressive deformation of the meters, with reversed movement indicated for cold months.

The plots for these two meters show that their readings were also affected by the continuing expansion of the concrete flanking the present spillway above El. 1095. This is indicated by the progressive increase in the amplitude of the abutment yield indicated by these meters in successive years, the increase being significantly greater for Meter DH-1L than for Meter DH-1R. This is consistent with the field measurements which show greater expansion of the left side concrete and is evidenced by the extensive cracking of the concrete in this region. Deformation plots of other meters do not exhibit any such obvious long-term movements which could be attributed to the effects of chemical expansion of concrete in regions of their locations. This observation also generally confirms that expansion due to alkali-aggregate reaction is confined to the concrete above El. 1095. The plots of Meters DH-2aR and DH-2aL exhibit consistent response of these meters to load changes even though the small magnitudes of indicated deformations make it difficult to identify the response peaks corresponding to the reservoir level peaks. The small response values are due to the fact that these two meters are oriented at 45^{o} to the direction of arch thrust.

Meter DH-3R had been indicating consistent response until December 1969 when it started showing the considerably large than normal abutment yield which apparently increased progressively in subsequent years. Corresponding large increases have not been shown by Meter 3-L which is located in the left abutment at the same level as Meter 3R, nor have any of the re-

maining meters indicated proportionately large deformations. A total of 0.1371 inches of abutment yield was indicated by Meter 3-R in the two years from October 1969 to October 1971. The trend of progressive increase in the abutment yield indicated by this meter from 1969 thru 1971, seems to have ceased in 1972. Notwithstanding the apparent large abutment yield over the 2 year period indicated by Meter 3-R, the change in response of this meter to rapid rise of the reservoir water level was found to be almost equally linearly elastic both before and after December 1969, as shown on Figure VII-1. This does not seem to be the case for rapid drop of the reservoir level where readings before 1969 appear to agree with the values for corresponding reservoir rise while those taken after 1969 show less deformation change.

Review of the field readings of this meter showed that all the observed values have been well within the useful elastic range of the meter supplied by the manufacturer.

B. <u>3-D FEM Analyses Results</u> - Consistency of the response of the meters was also verified by comparing the measured deformation changes with those theoretically computed by the 3-D FEM analyses for applicable load changes. These values were computed using assumed values for the modulus of elasticity of rock and concrete. The measured incremental deformations due to particular rise in the reservoir water surface agreed reasonably close with the computed values, attesting to the validity of the observed performance of the meters.

As described in Chapter III chemical expansion due to alkali-aggregate reaction was found to be confined to the concrete in the upper regions above El. 1095, where it caused significant measurable movement. However, this expansion caused only very minor movements at lower regions, so small as to be indiscernible in the plots of deformations indicated by the meters in these regions. This insignificant effect of chemical expansion on the deformations at lower regions was generally confirmed by the values theoretically computed by 3-D FEM analyses described in Chapters III and IV.

The large abutment yield indicated by Meter 3-R since December 1969 has been shown to be inconsistent with the related reservoir load changes. The possibility that the indicated deformation might be due to chemical expansion caused by the onset in 1969 of alkali-aggregate reaction in the lower intact concrete was investigated and found untenable. 3-D FEM analyses were made using reasonable assumed values to represent chemical expansion in the right half of intact concrete in the region of Meter 3-R. The resulting displacement values in the region of this meter indicated that the chemical expansion would cause pulling away of the downstream arch concrete from the right abutment, which is contrary to the trend of abutment yield, indicated by Meter 3-R. This confirms that chemical expansion to date has been confined to concrete above El. 1095 and that the indicated large abutment yield does not reflect the response of Meter 3-R to the structural load effects.

Another possible explanation for abutment yield indicated by Meter 3-R is actual yielding of a portion of the lower right abutment. To examine this possibility, a 3-D FEM analysis was made assuming a greatly reduced value for modulus of elasticity (300,000 psi) in the lower right abutment zone, in order to model the yield response of the abutment due either to structural deformation or slip. For this assumed condition the computed displacement in the vicinity of this meter location resolved in the direction of meter alignment, was 0.0854 inches. This computed compression of the right abutment, however, is accompanied by a corresponding movement out of the left abutment because of the slip joint at El. 960. Meter 3-L located in the left abutment at lower level does not indicate such movement out of the left abutment.

No discernible permanent abutment yielding over the two year period is indicated by either Meter 2-R or survey marker No. 8 which are located on the right abutment just about 67 feet above Meter 3-R. No apparent cracks indicative of the effects of chemical expansion due to alkali-aggregate reaction are visible in this lower region nor can any cracks which might be due to differential abutment yielding be seen in the concrete structure.

Since the studies show deformations inconsistent with measurements at other meter locations, it is concluded that the neither water load nor chemical expansion nor abutment yielding is responsible for the questionable readings indicated by Meter 3-R. It is therefore concluded that these readings are erroneous and result from either some physical malfunctioning of the meter or some deterioration in the physical condition of the meter and the fixtures.

7.3 PHYSICAL CHECK OF METERS

In order to ascertain whether or not the large abutment yield indicated by Meter 3-R was possibly due to any physical defects in the meter itself or its mountings, inspection of the in-situ physical conditions of the deformation meters was made on August 2, 1972 by Dr. Roy W. Carlson, Consultant who visited Matilija Dam accompanied by R. P. Sharma of IECO. Deformation readings of all the meters were taken before making the physical check.

All the meters indicated readings consistent with the earlier trends, except that Meter 1-L did not show any reading. Physical check of the meter after it was removed from the mounting indicated that meter had an open circuit with one conductor separated from the terminal.

Meter 3-R and its mountings, accessible by boat, were inspected in its in-situ condition before the meter was dismantled from its mounting. No sign of any distress or damage to the installation was visible. Inspection of the meter when recovered from its mounting indicated that its flexible bellows section has been filled up with dried mud. Both Meters 3-R and 1-L recovered on August 2, 1972 were personally carried by Dr. Carlson while three more were removed later and shipped to his laboratory. Calibrations and performance testing of the five meters were conducted by Dr. Carlson and the results summarized in his letter dated September 1, 1972, which is included in the appendixes to this report.

Physical check of the recovered meters in the laboratory showed that all meters were in excellent condition internally and it was only severe exposure conditions which caused some lack in perfect performance of the meters after over six years. Inspection in the laboratory revealed that dried mud in the flexible bellows sections of Meter 3-R restrained the free movement of the flexible section of the meter.

This restraint is probably responsible for the inconsistent large and progressively increasing compression of the right abutment indicated by this meter since December 1969. The restraint did not seem to significantly affect the response of the meter under compressive loads corresponding to increase in the reservoir water levels, but was considerably effective in preventing elastic recovery of such induced compression, during the subsequent falling of the reservoir level. This resulted in residual compression at the end of each successive loading and unloading cycle corresponding to rising and falling reservoir water level, and annual temperature cycles. Successive accumulation of such residual compression caused indications of the progressively increasing yield of the right abutment by this meter.

Dr. Carlson recommended replacement of the five recovered meters by new ones with improved ceramic-insulated terminals because the old ones have some surface deterioration. Extra care in installing the new ones is advised to prevent access of mud to the bellows. This protection can be provided by wrapping the meter with soft fabric, taped at the ends. A flexible schedule of reading these meters was recommended by Dr. Carlson in his report.

7.4 CONCLUSIONS

Conclusions from detailed review of the observed data, theoretically determined deformations and physical check of meters are as follows:

- All meters were in excellent condition internally except for Meter 1-L which had developed an open circuit.

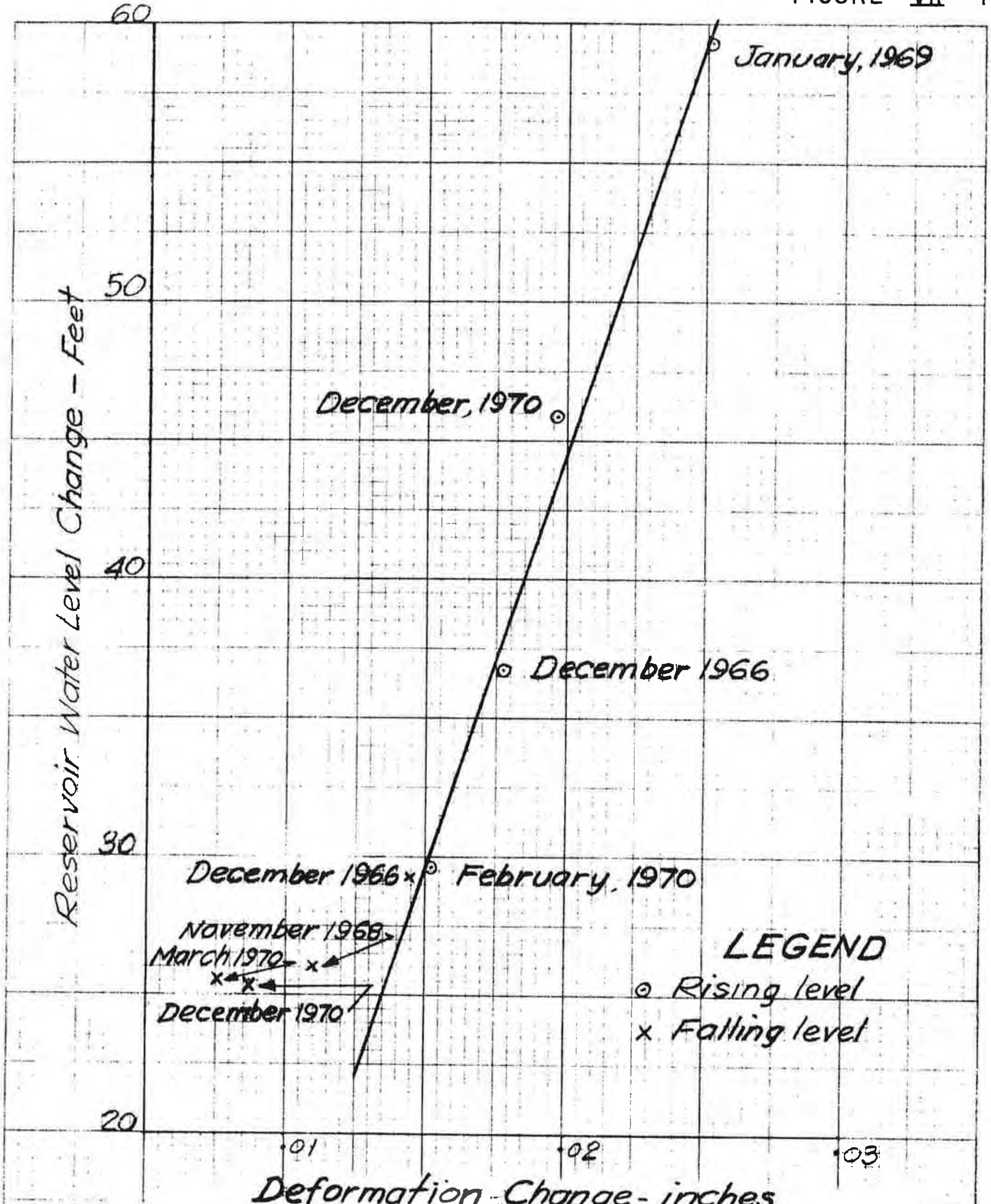
- All meters indicated consistent response to the operating load changes except for Meter 3-R which has shown erratically increasing abutment yield since the end of 1969.

- Meters at upper locations are susceptible to significant temperature effects due to their exposed location, and are less responsive to reservoir level changes.

- The questionable abutment yield indicated by Meter 3-R since December 1969, is not consistent with either the readings of the other meters or deformations determined analytically for corresponding conditions. The inconsistent deformation readings were evidently caused by restraint and solidification of the flexible bellows by dried mud.

- Meters at lower locations, (namely Meters 3-R and 3-L), which have been subjected to submergence for long periods did not have adequate protection.

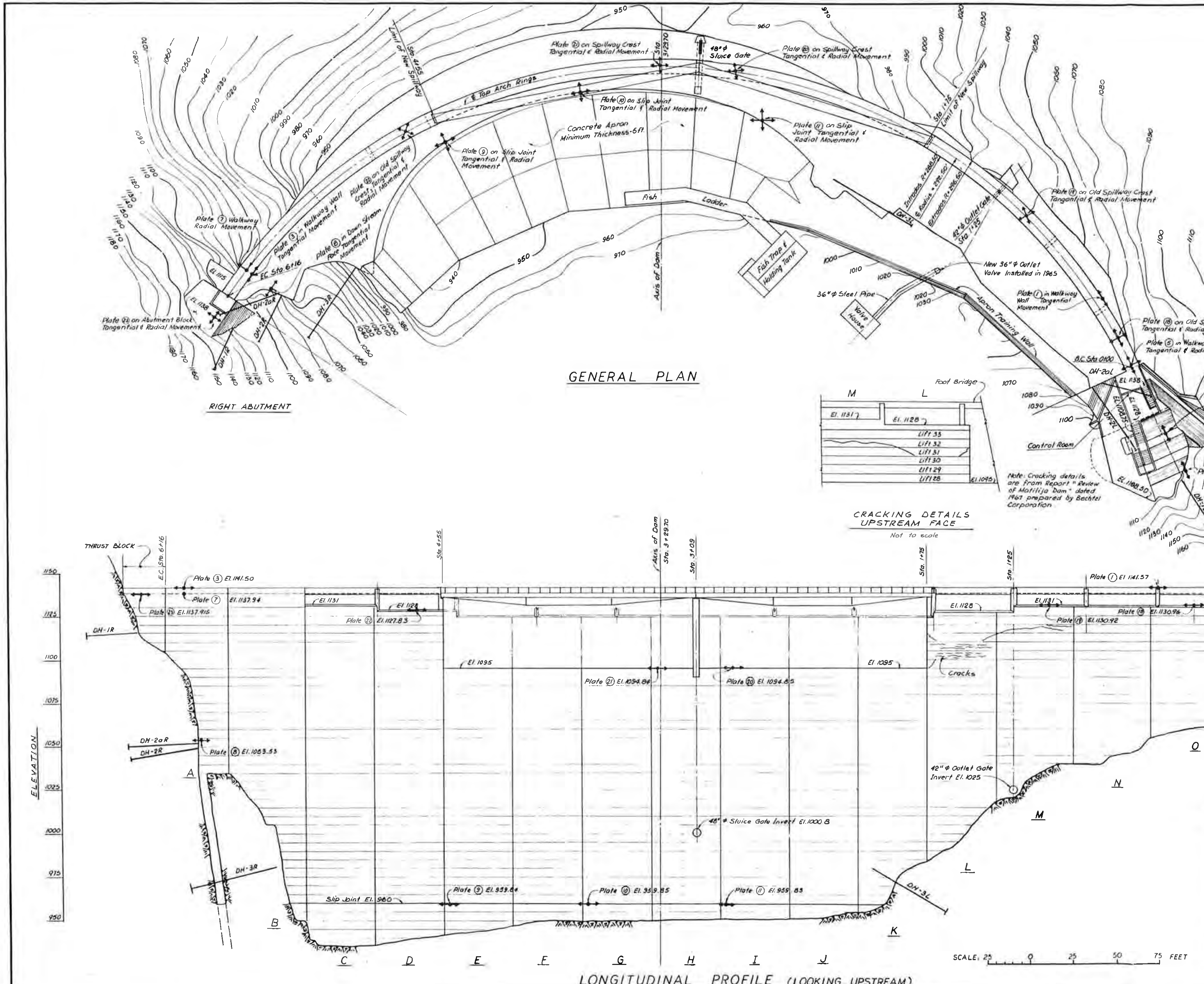
FIGURE VII - I



MATILIJA DAM

METER 3-R

Water Level Change VS Deformation Change



COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

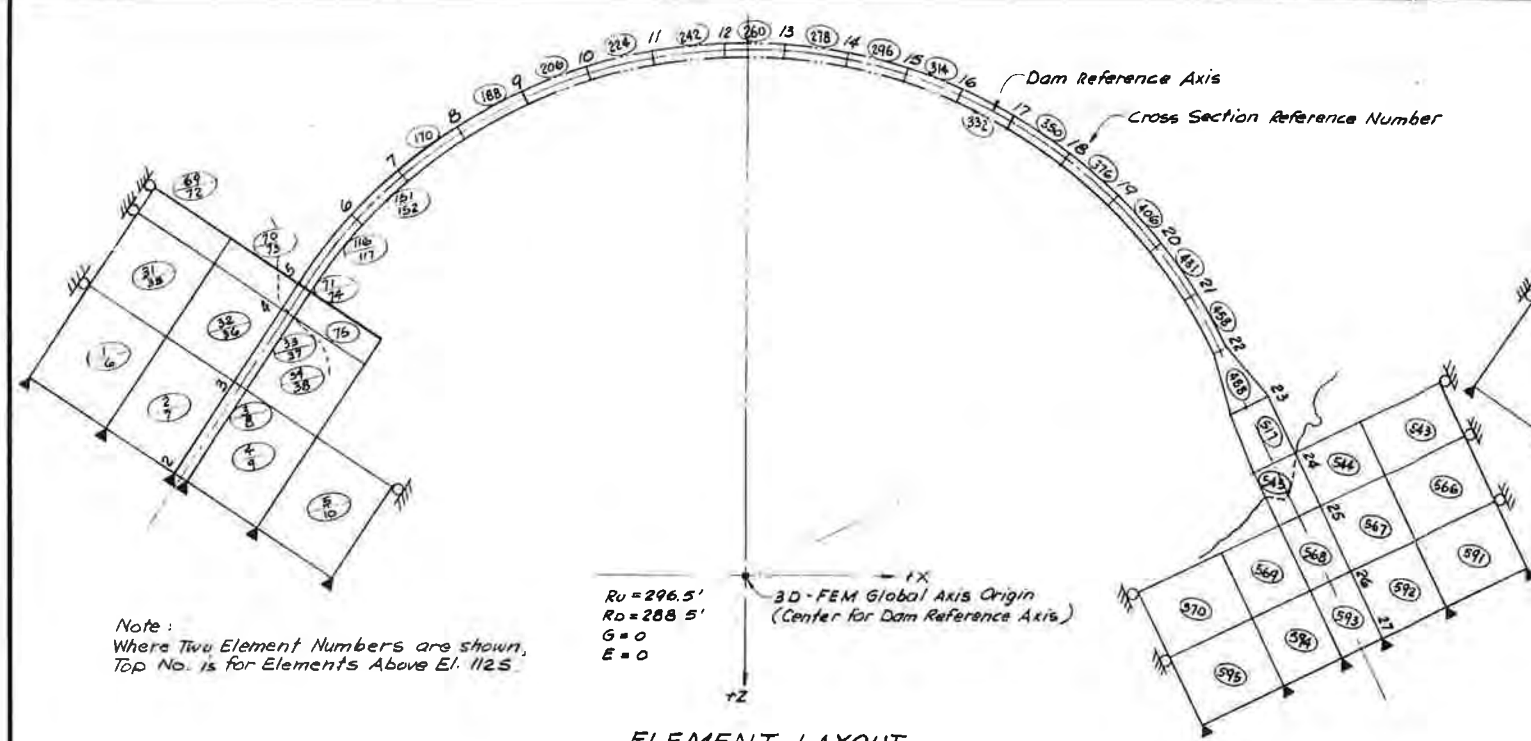
MATILIJA DAM

GENERAL ARRANGEMENT PLAN AND PROFILE

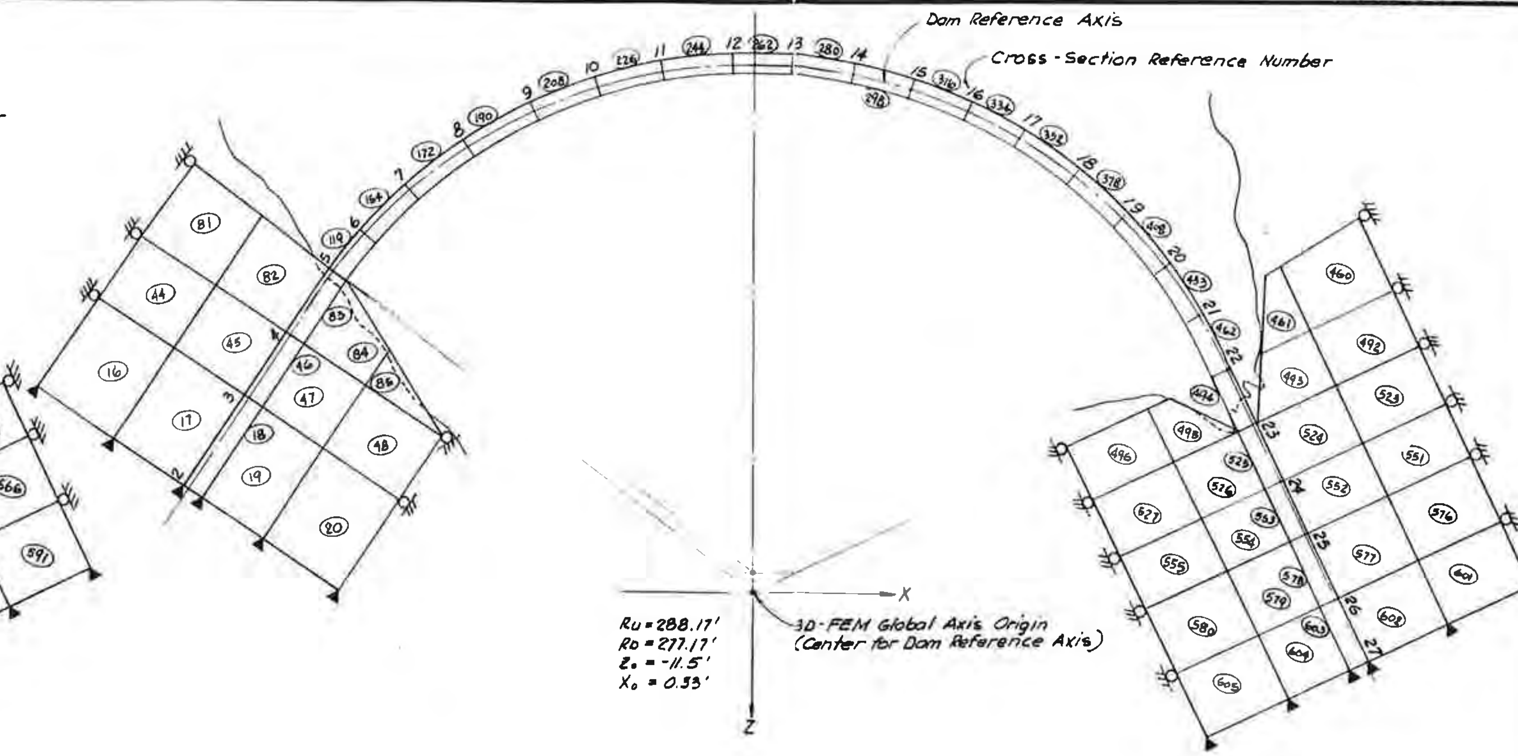
INTERNATIONAL ENGINEERING COMPANY, INC.

DESIGNED IECO — CHKD — SUBMITTED
DRAWN — INSP — RECOMMENDED — APPROVED

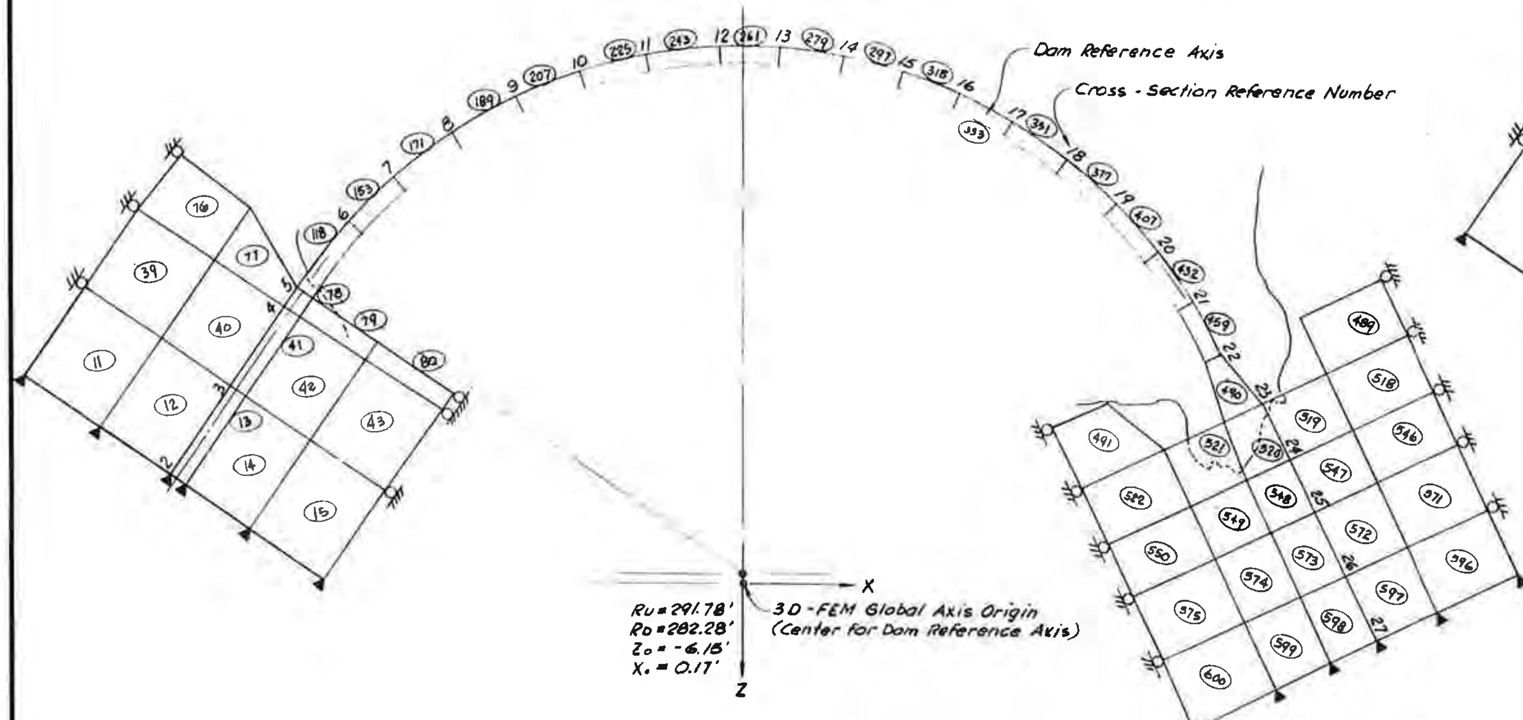
AUGUST 3, 1972 — HR-11-001



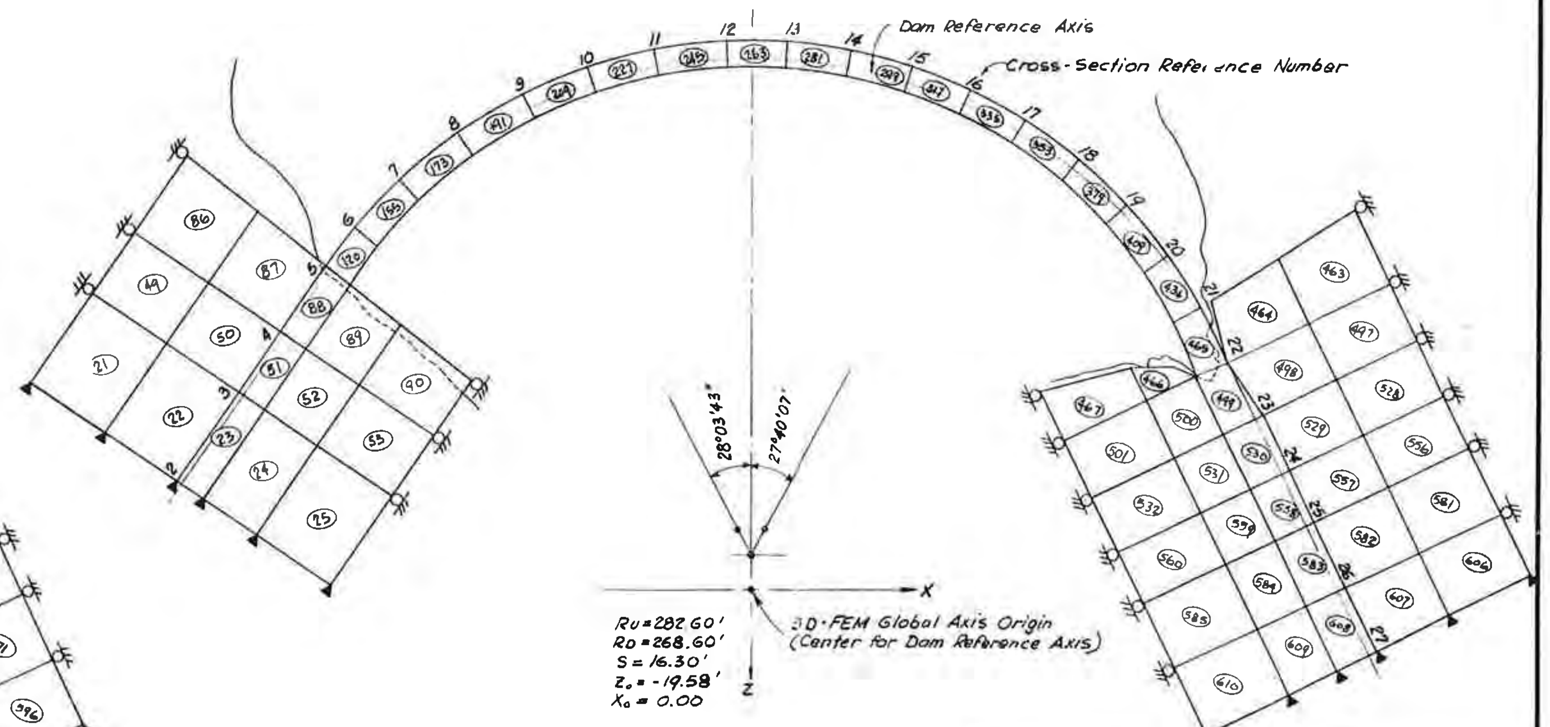
ELEMENT LAYOUT
HORIZONTAL LIFT EL. 1125



ELEMENT LAYOUT
HORIZONTAL LIFT EL. 1080



ELEMENT LAYOUT
HORIZONTAL LIFT EL. 1095



ELEMENT LAYOUT
HORIZONTAL LIFT EL. 1060

LEGEND

Locked Boundary Condition
Elements Number
Restricted (One Direction) Boundary Condition

NOTE

Horizontal Lift is the Top Surface of Elements shown

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM

3-D FEM STRESS ANALYSIS

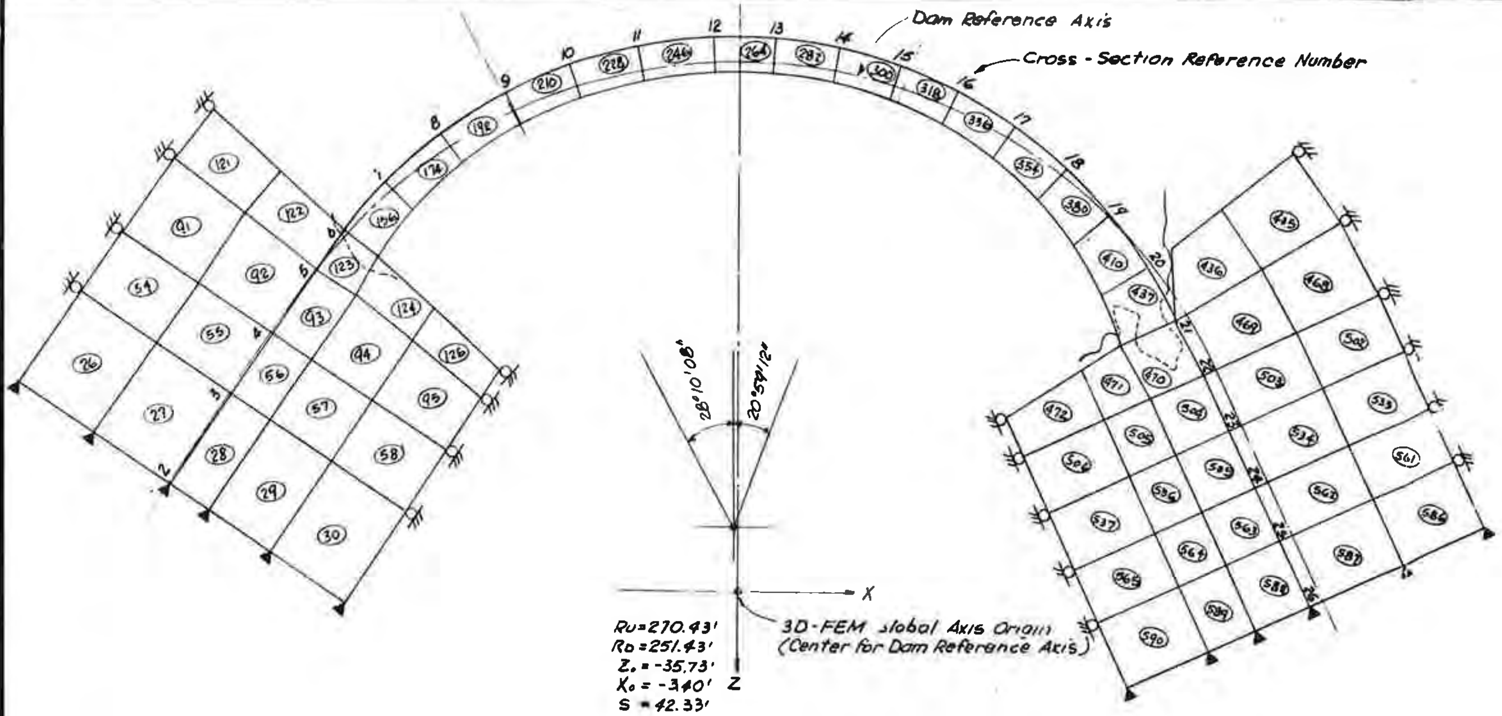
MESH LAYOUT Sheet 2 of 4

INTERNATIONAL ENGINEERING COMPANY, INC.

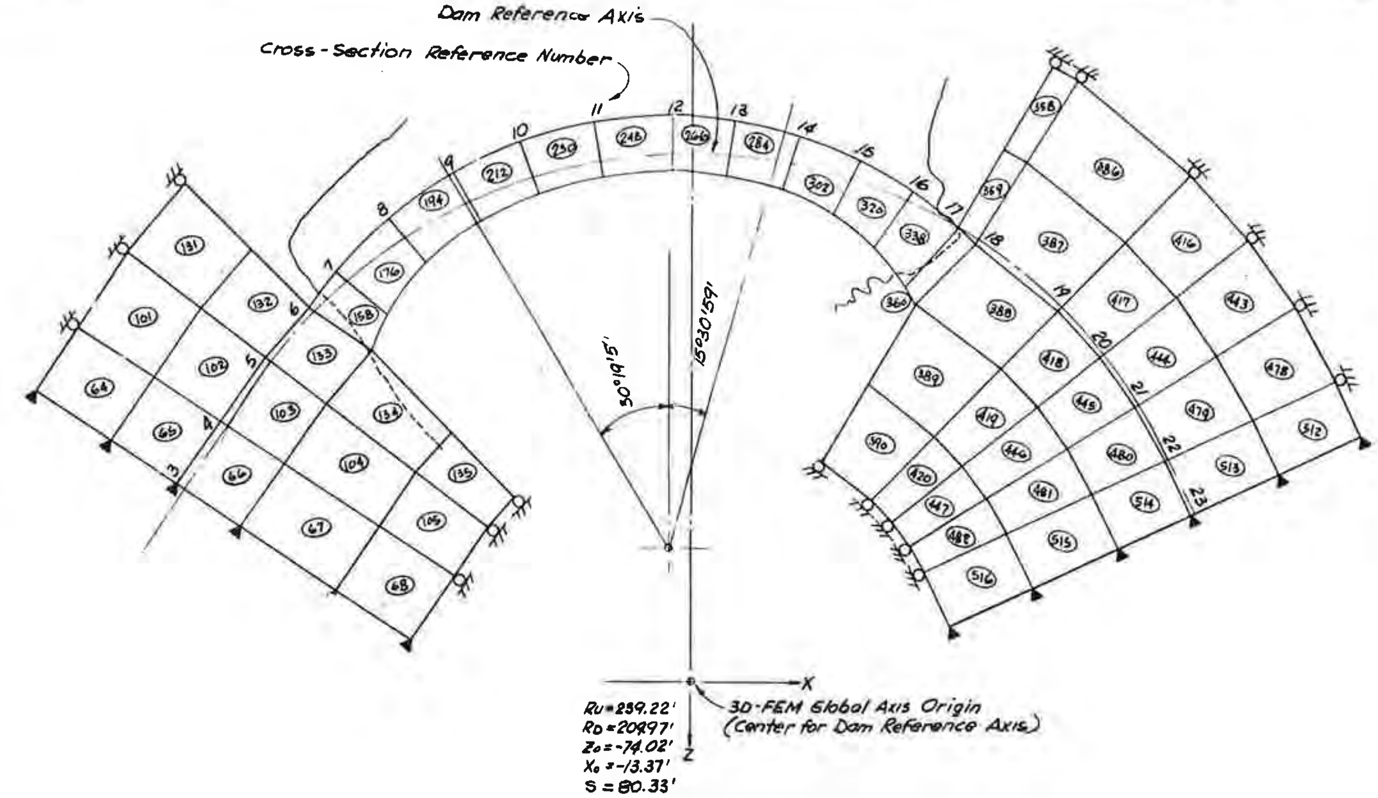
DESIGNED HEJ — CHKD. HEJ — SUBMITTED [signature]
DRAWN A.M.W. — INSP. — RECOMMENDED [signature]
APPROVED [signature]

SAN FRANCISCO, CAL.
DATE AUGUST 3, 1972

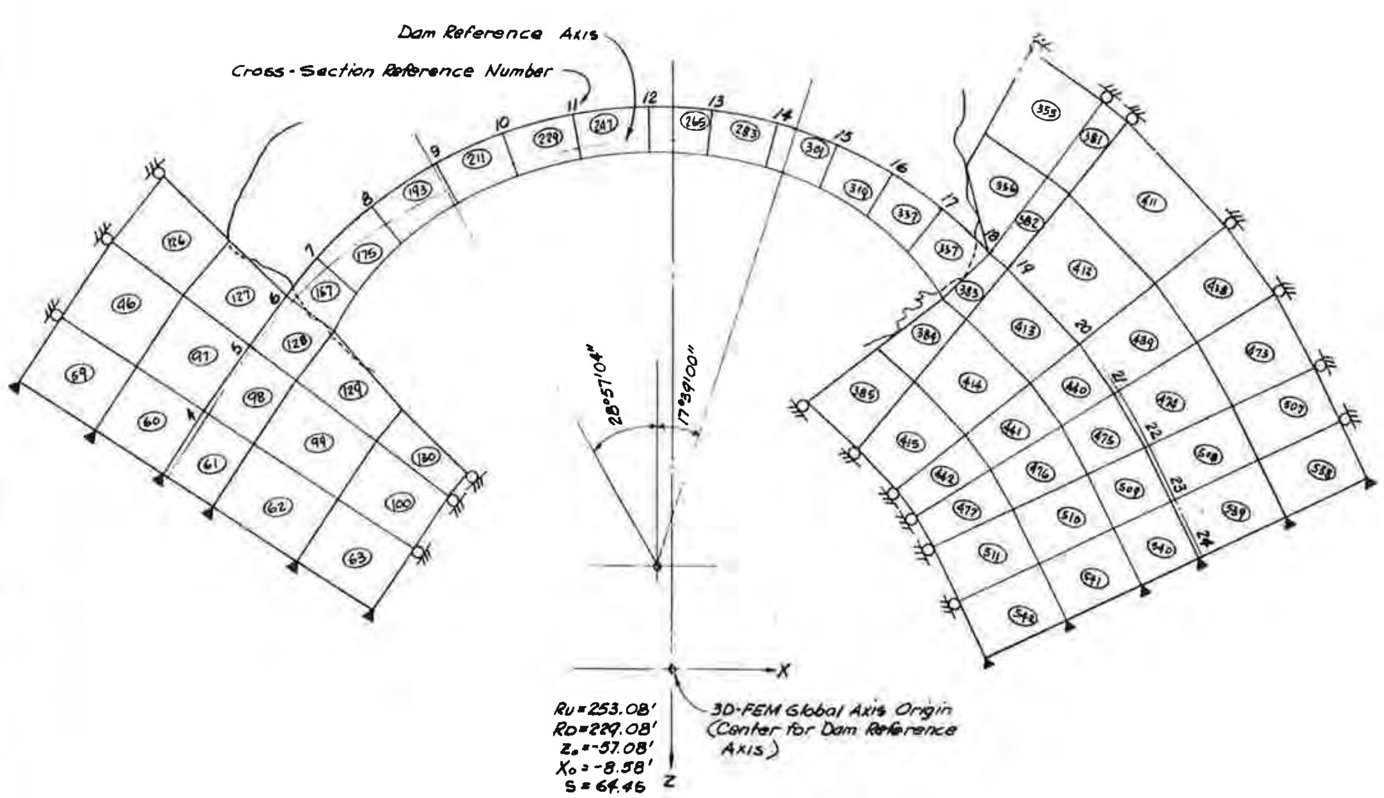
HR-11-003



ELEMENT LAYOUT
HORIZONTAL LIFT EL.1035



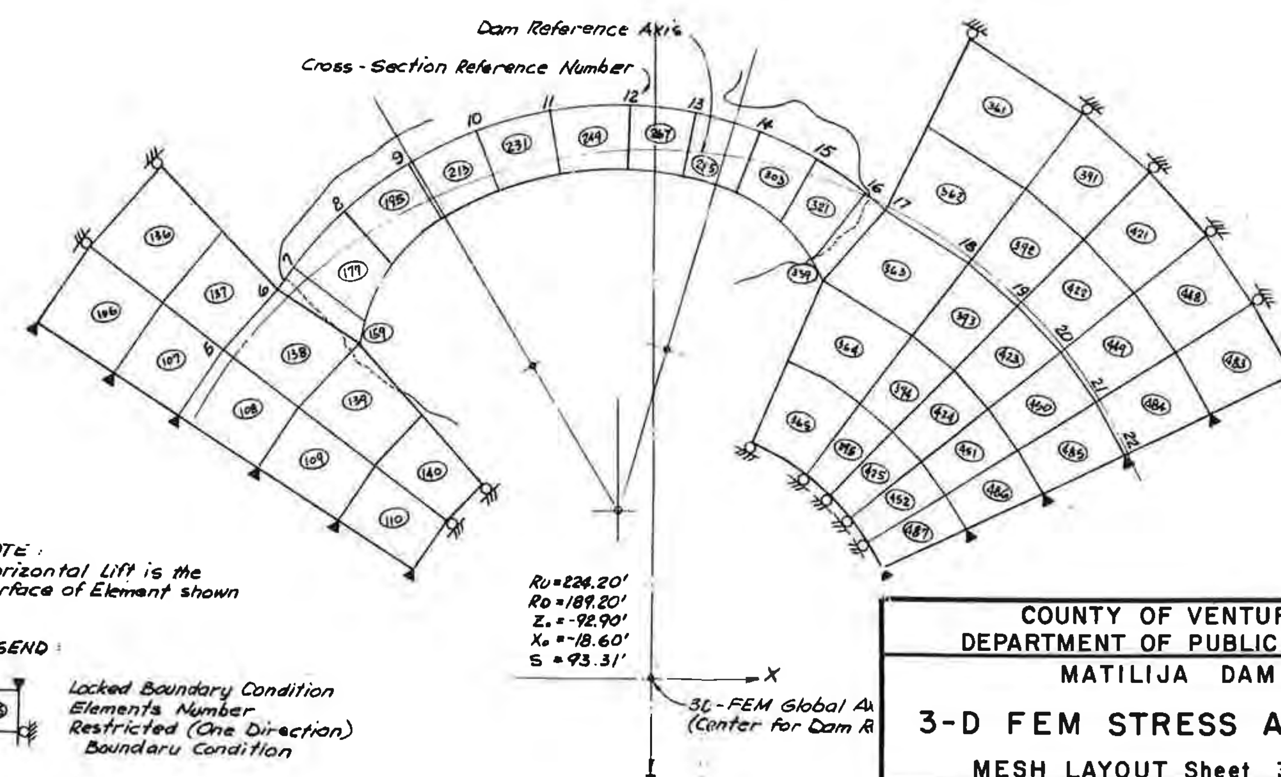
ELEMENT LAYOUT
HORIZONTAL LIFT EL.985



ELEMENT LAYOUT
HORIZONTAL LIFT EL.1010

NOTE:
Horizontal Lift is the surface of Element shown

LEGEND:
Locked Boundary Condition
Elements Number
Restricted (One Direction) Boundary Condition



ELEMENT LAYOUT
HORIZONTAL LIFT EL. 960

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM

3-D FEM STRESS ANALYSIS

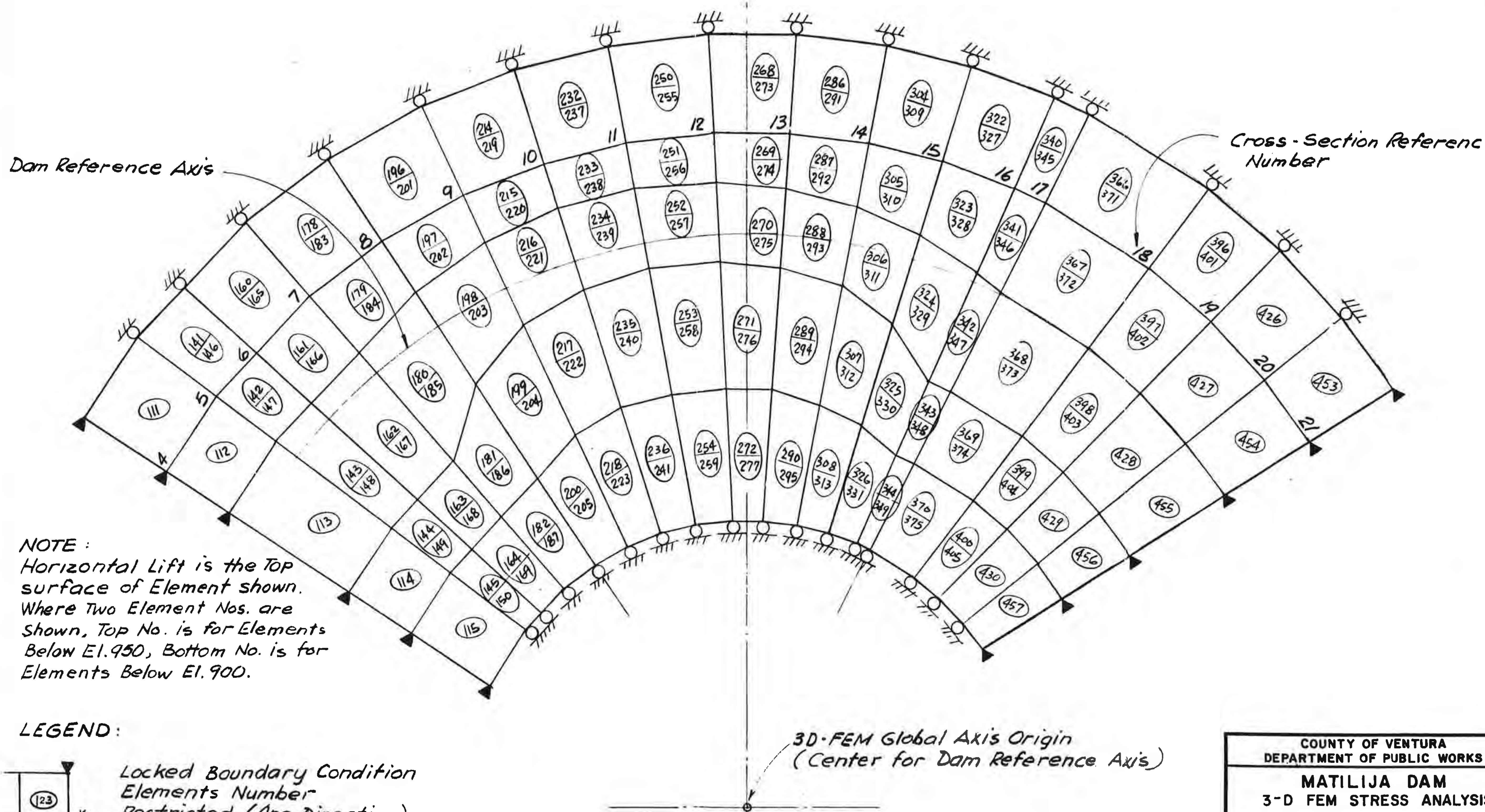
MESH LAYOUT Sheet 3 of 4

INTERNATIONAL ENGINEERING COMPANY, INC.

DESIGNED HEJ CHKD HEJ SUBMITTED
DRAWN A.M.W. INSP. RECOMMENDED
APPROVED

SAN FRANCISCO, CAL.
DATE AUGUST 3, 1972

HR-11-004



NOTE:
Horizontal Lift is the Top surface of Element shown. Where Two Element Nos. are Shown, Top No. is for Elements Below El. 950, Bottom No. is for Elements Below El. 900.

LEGEND:

Locked Boundary Condition
Elements Number
Restricted (One Direction) Boundary Condition

FOUNDATION SECTION
HORIZONTAL LIFTS AT EL. 950 & 900

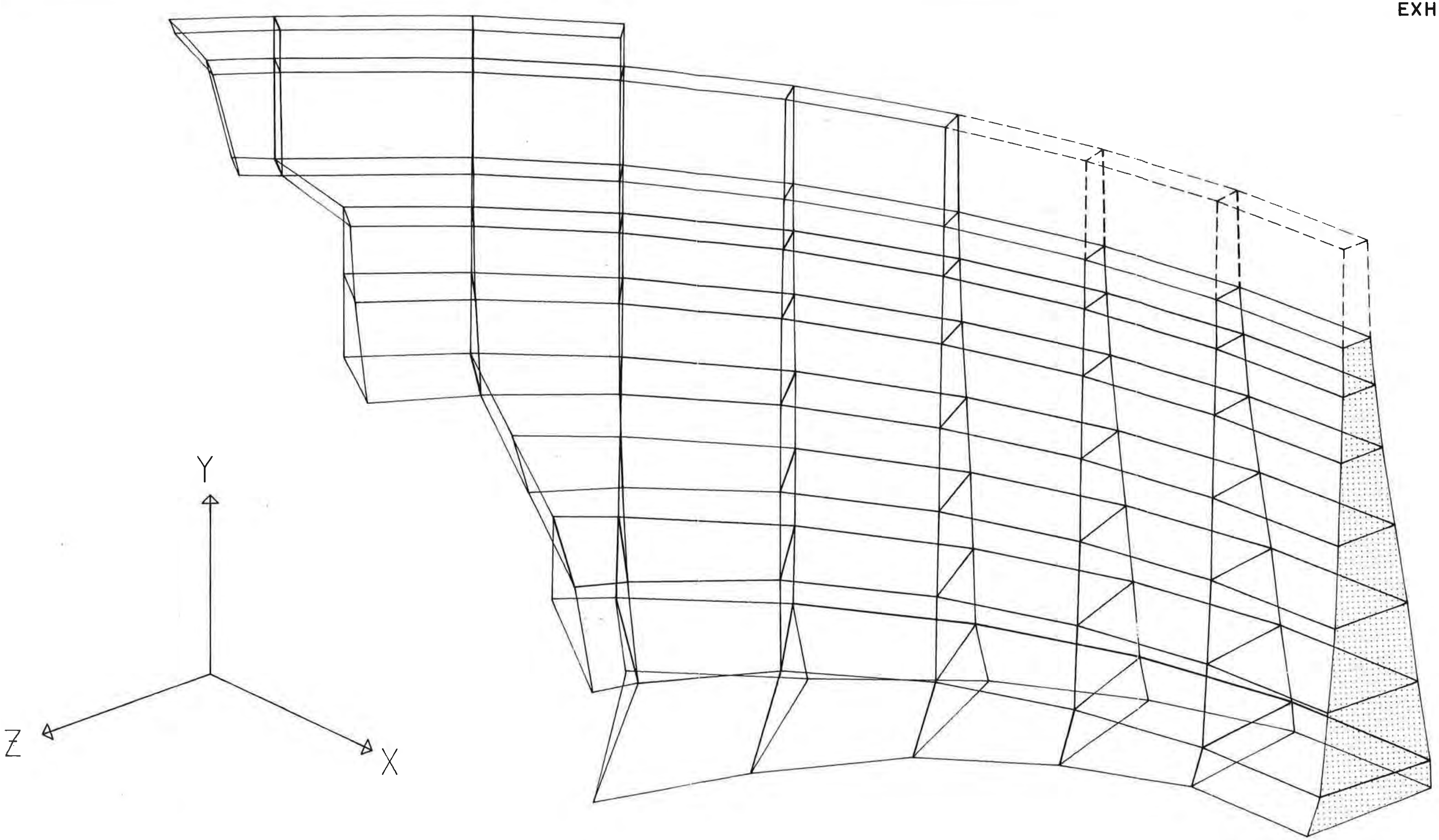
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM
3-D FEM STRESS ANALYSIS
MESH LAYOUT Sheet 4 of 4

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

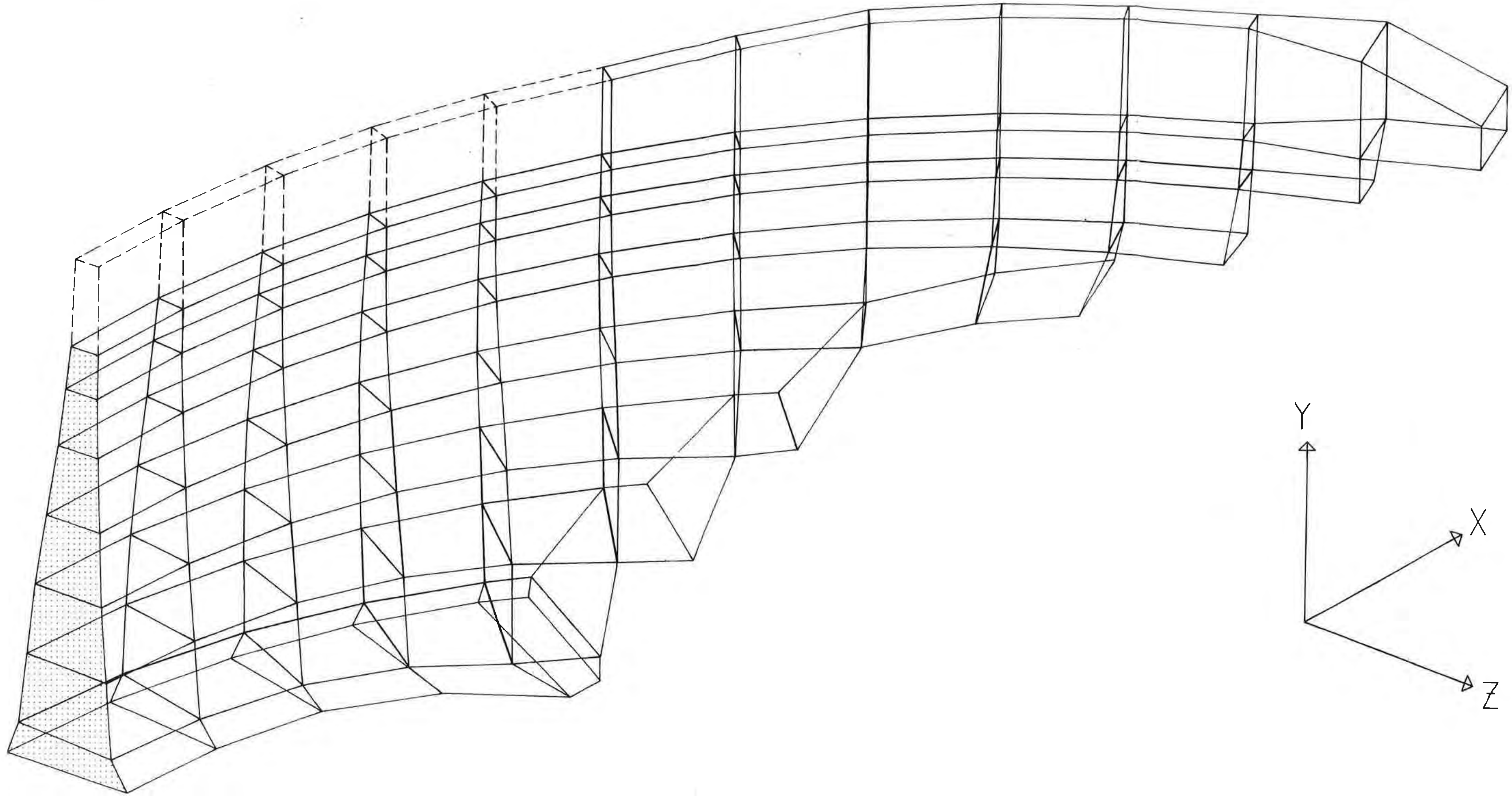
DR. A.M.W. CK. DEJ RECOMMENDED EBK. OL APPROVED [signature]

DATE: AUGUST 9, 1972 HR-11-005

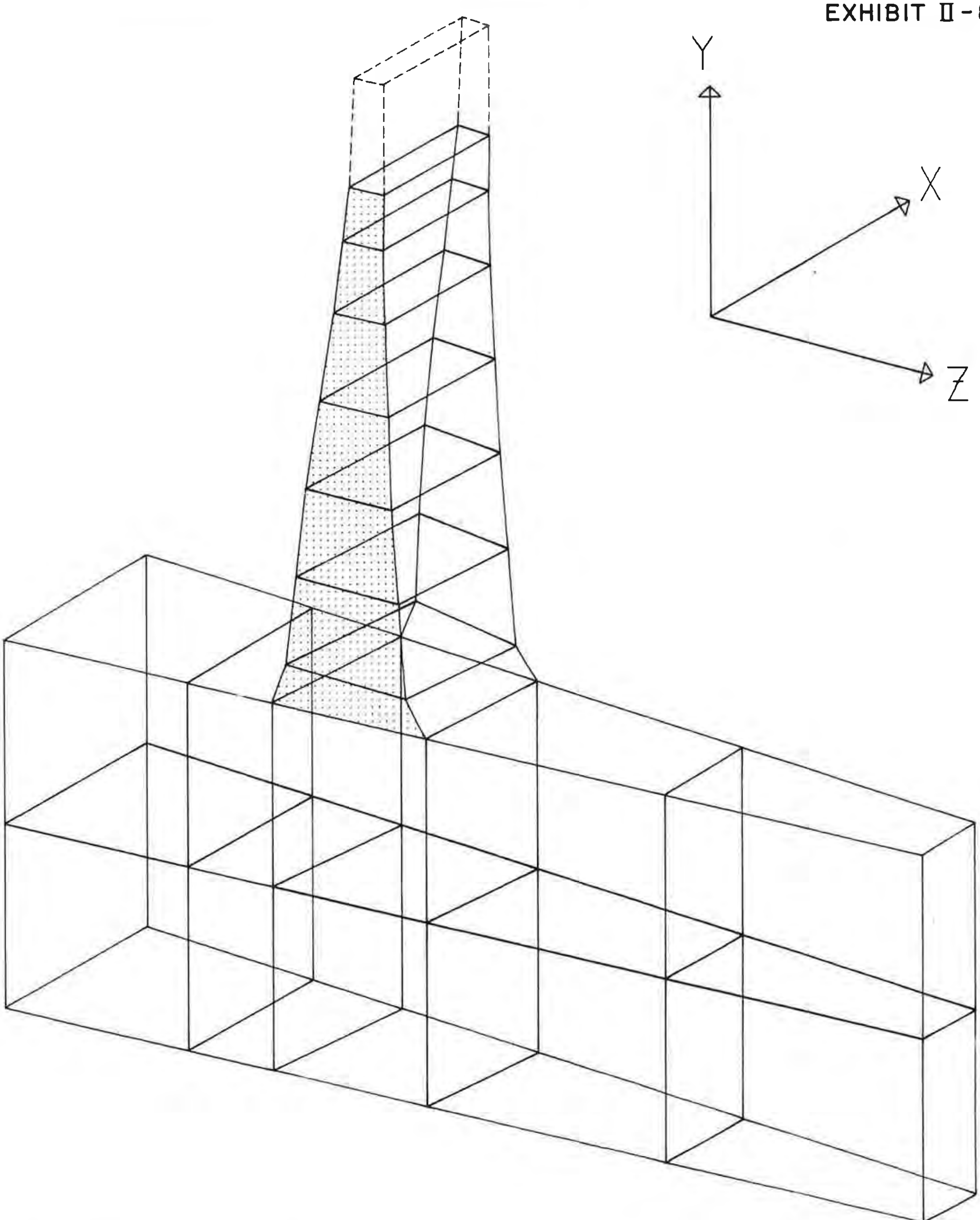


MATILIJA DAM 3D-SAP
CONCRETE ARCH - RIGHT SIDE

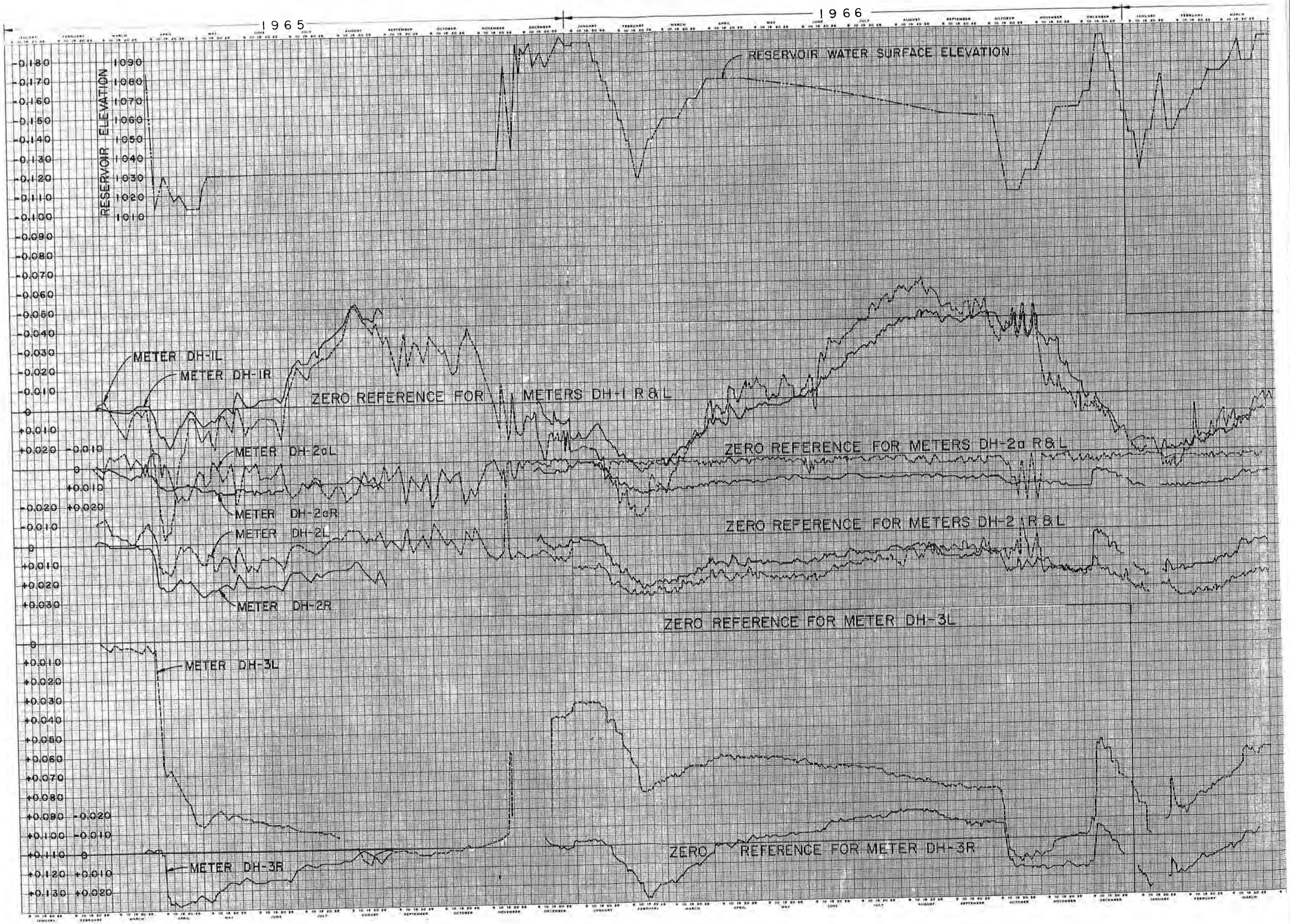
IECO INTERNATIONAL ENGINEERING COMPANY, INC.
220 MONTGOMERY ST · SAN FRANCISCO · CALIFORNIA



MATILIJA DAM 3D-SAP
CONCRETE ARCH - LEFT SIDE

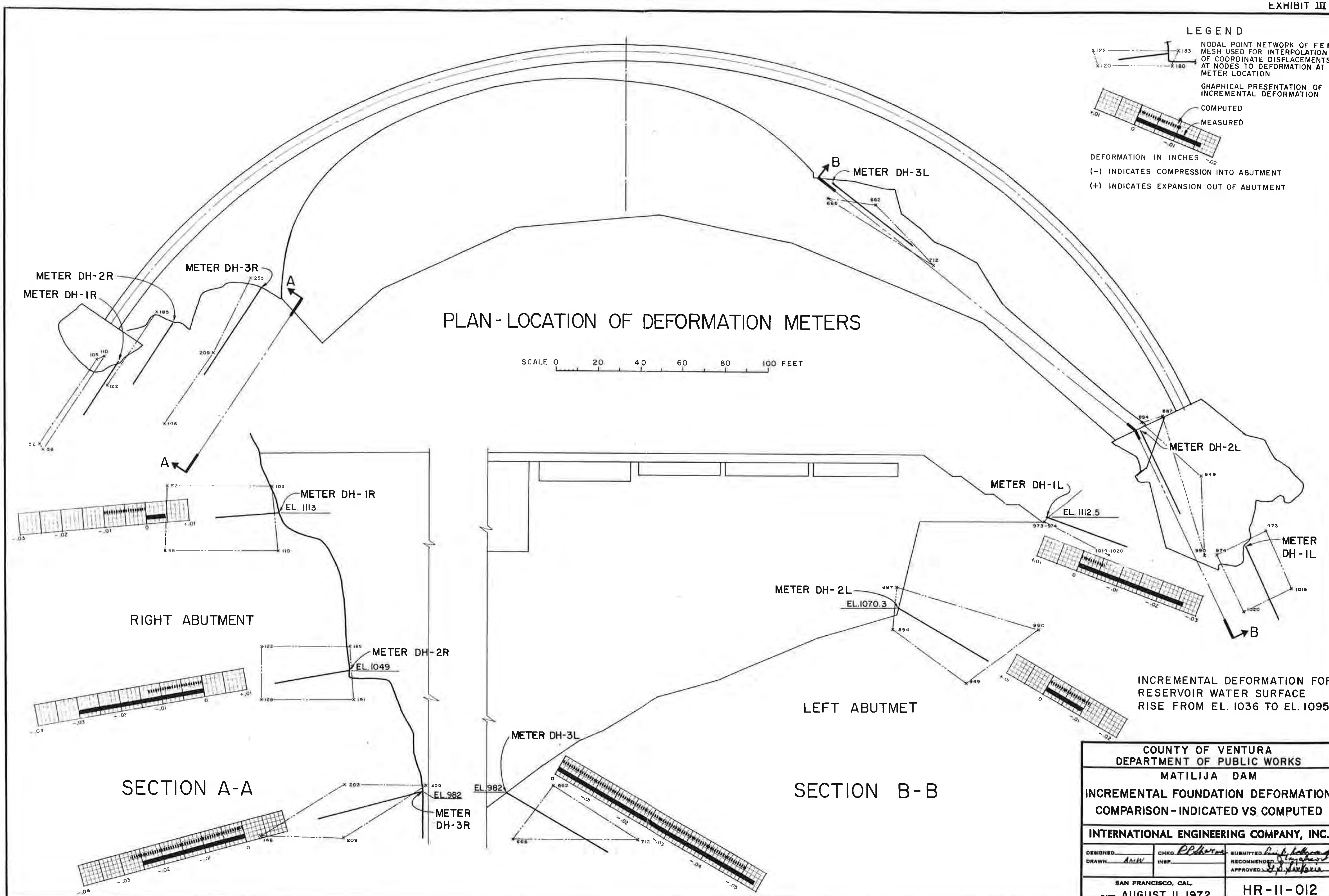


MATILIJA DAM 3D-SAP
CROWN CANTILEVER



1965
1966
RESERVOIR WATER SURFACE ELEVATION
RESERVOIR ELEVATION
METER DH-IL
METER DH-IR
ZERO REFERENCE FOR METERS DH-I R & L
METER DH-2aL
METER DH-2aR
ZERO REFERENCE FOR METERS DH-2a R & L
METER DH-2L
METER DH-2R
ZERO REFERENCE FOR METERS DH-2 R & L
ZERO REFERENCE FOR METER DH-3L
METER DH-3L
METER DH-3R
ZERO REFERENCE FOR METER DH-3R

EXHIBIT III-4



COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM
INCREMENTAL FOUNDATION DEFORMATION
COMPARISON - INDICATED VS COMPUTED

INTERNATIONAL ENGINEERING COMPANY, INC.

DESIGNED
DRAWN AM/W
CHKD R.P. Sharma
INSP.
SUBMITTED [signature]
RECOMMENDED [signature]
APPROVED [signature]

SAN FRANCISCO, CAL.
DATE AUGUST 11, 1972

HR-11-012

UPSTREAM ARCH (σ_{xu}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

CROSS-SECTION REFERENCE NUMBER

ELEVATION	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1138																					
1131.5		+40	+154	+-38																	
1110.0		+46	+153	+22	+9	+20									+3	+4	+4	+9	+40	+42	+28
1087.5			+177	+76	+77	+144	+280	+312	+321	+287	+228	+170	+133	+122	+125	+118	+114	+72	+67	+53	
1070.0			+173	+102	+112	+193	+295	+368	+370	+332	+260	+187	+139	+121	+117	+126	+112	+100	+89		
1047.5			+130	+94	+107	+205	+333	+418	+430	+383	+289	+190	+128	+104	+108	+108	+134	+144			
1022.5				+52	+70	+196	+360	+465	+484	+430	+313	+182	+102	+92	+98	+106					
997.5				+-20	+35	+186	+376	+497	+522	+465	+332	+186	+89	+68	+95						
972.5				+9	+19	+165	+384	+524	+554	+490	+342	+188	+136	+83							
948.0					+-35	+31	+24	+22	+27	+11	+-27	+-34	+-74	+-150							

RIGHT ABUTMENT

LEFT ABUTMENT

LOADING CONDITIONS:

RES. W. S. EL. 1095

SILT EL. 1040

DEAD LOAD

DOWNSTREAM ARCH (σ_{xd}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

CROSS-SECTION REFERENCE NUMBER

ELEVATION	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1138																					
1131.5		+1	+27	+158																	
1110.0		+16	+60	+206	+200	+92									+3	+5	+2	+29	+12	+-5	+3
1087.5			+109	+228	+240	+214	+203	+126	+123	+143	+177	+206	+216	+208	+186	+152	+149	+90	+44	+34	
1070.0			+158	+239	+284	+264	+192	+135	+118	+148	+196	+233	+244	+228	+189	+161	+133	+93	+60		
1047.5			+166	+289	+353	+311	+208	+124	+107	+148	+221	+280	+297	+268	+219	+158	+130	+103			
1022.5				+376	+428	+372	+225	+113	+92	+149	+257	+350	+369	+348	+252	+170					
997.5				+478	+527	+445	+238	+95	+69	+144	+300	+450	+476	+390	+352						
972.5				+678	+693	+522	+246	+74	+38	+128	+339	+573	+636	+572							
948.0					+-22	+0	+25	+27	+27	+8	+-23	+-42	+-50	+-140							

RIGHT ABUTMENT

LEFT ABUTMENT

LEGEND

MAX. COMPRESSION

500 INDICATED STRESS VALUE

-100 TENSION ZONE

MAX. TENSION

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

ALL STRESSES IN PSI

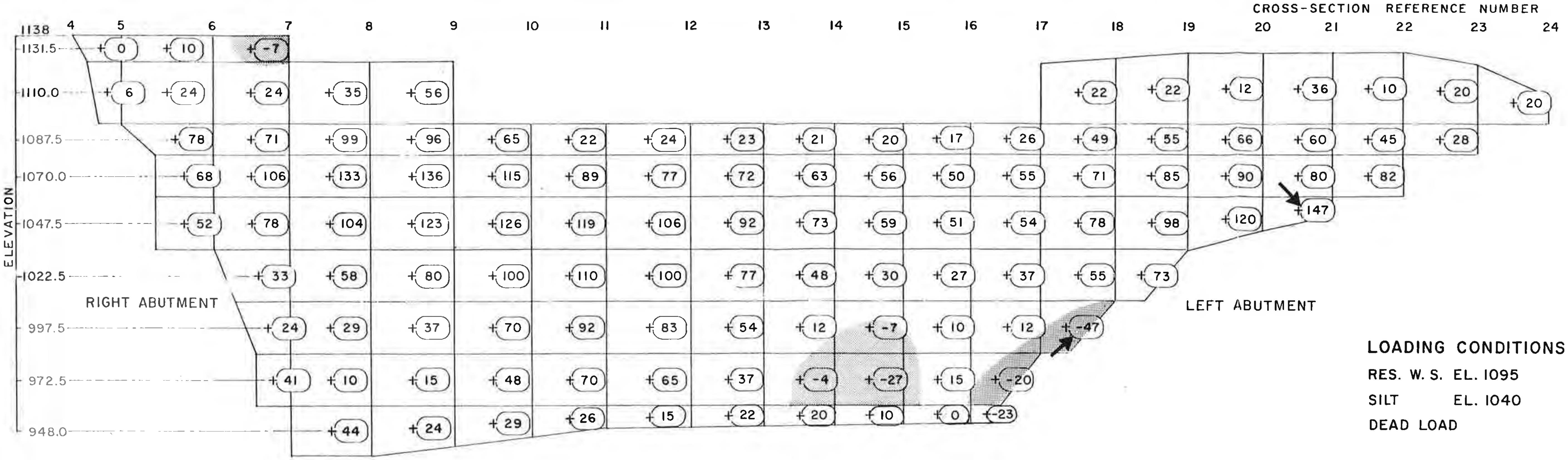
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY I
Summary Sheet 1 of 3

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EW | RECOMMENDED C.B.K. | APPROVED G. S. Sarkaria
CK. R.P.S.

DATE: AUGUST 3, 1972 | HR-11-013



LOADING CONDITIONS:

RES. W. S. EL. 1095

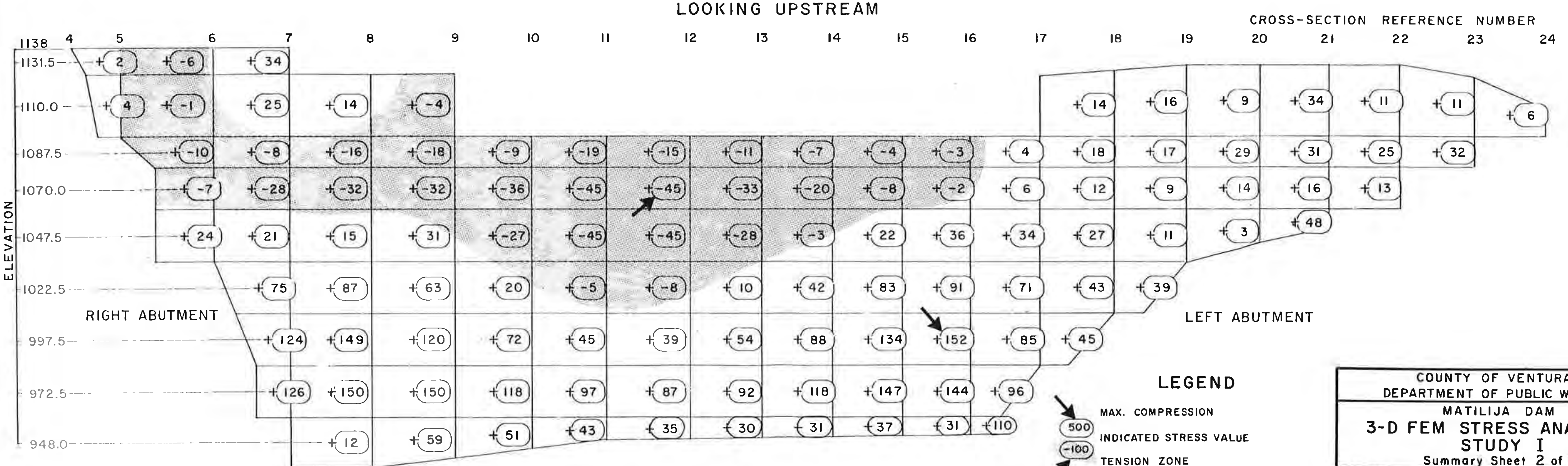
SILT EL. 1040

DEAD LOAD

UPSTREAM CANTILEVER (σ_{zu}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM



DOWNSTREAM CANTILEVER (σ_{zD}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

MAX. COMPRESSION

500 INDICATED STRESS VALUE

-100 TENSION ZONE

MAX. TENSION

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

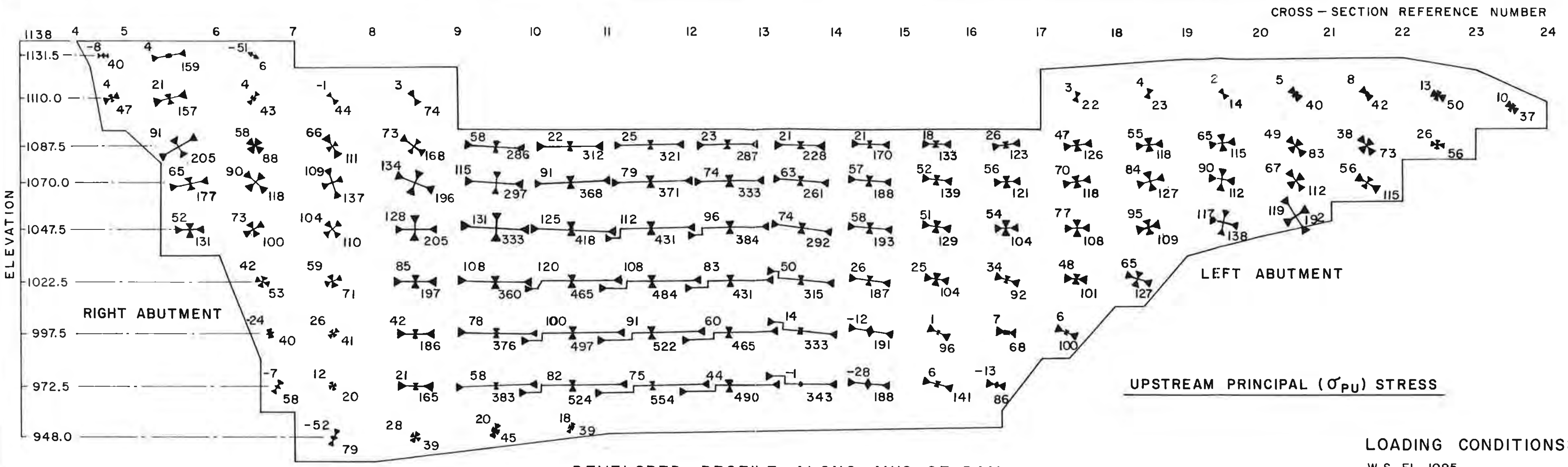
ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY I
Summary Sheet 2 of 3

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

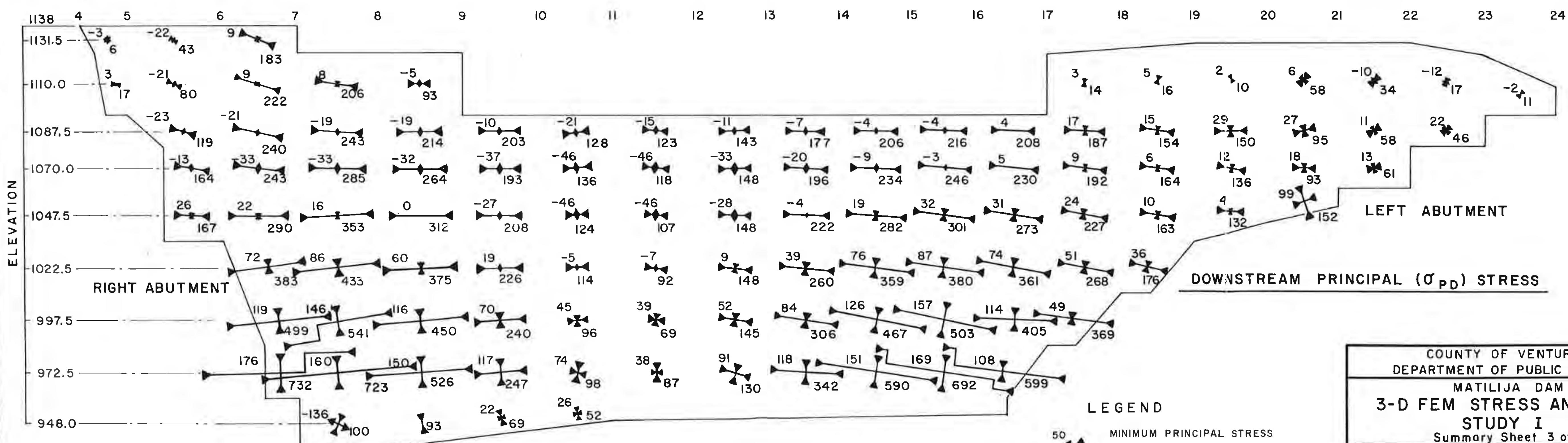
DR. EW	RECOMMENDED	APPROVED
CK. R.P.S.	C.B.K. [illegible]	G. S. Sarkaria
DATE: AUGUST 3, 1972	HR-11-014	



DEVELOPED PROFILE ALONG AXIS OF DAM
LOOKING UPSTREAM

LOADING CONDITIONS:

W.S EL. 1095
SILT EL. 1040
DEAD LOAD

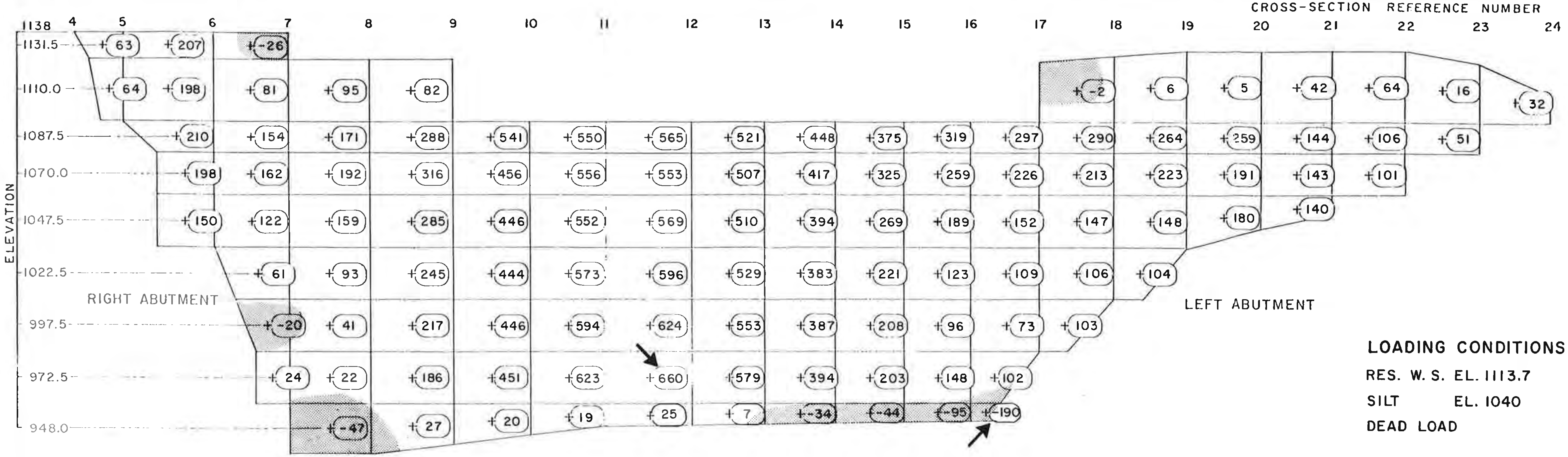


DEVELOPED PROFILE ALONG AXIS OF DAM
LOOKING UPSTREAM

LEGEND

50 MINIMUM PRINCIPAL STRESS
162 MAXIMUM PRINCIPAL STRESS
COMPRESSION (+) OR WITHOUT SIGN
TENSION (-)
ALL STRESSES IN PSI

COUNTY OF VENTURA DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM **3-D FEM STRESS ANALYSIS STUDY I** Summary Sheet 3 of 3
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.
DR. GN / CK. R.P.S. — RECOMMENDED: C.B.K. — APPROVED: S. S. SavKoor
DATE: AUGUST 3, 1972 — HR-11-015



LOADING CONDITIONS:

RES. W. S. EL. 1113.7

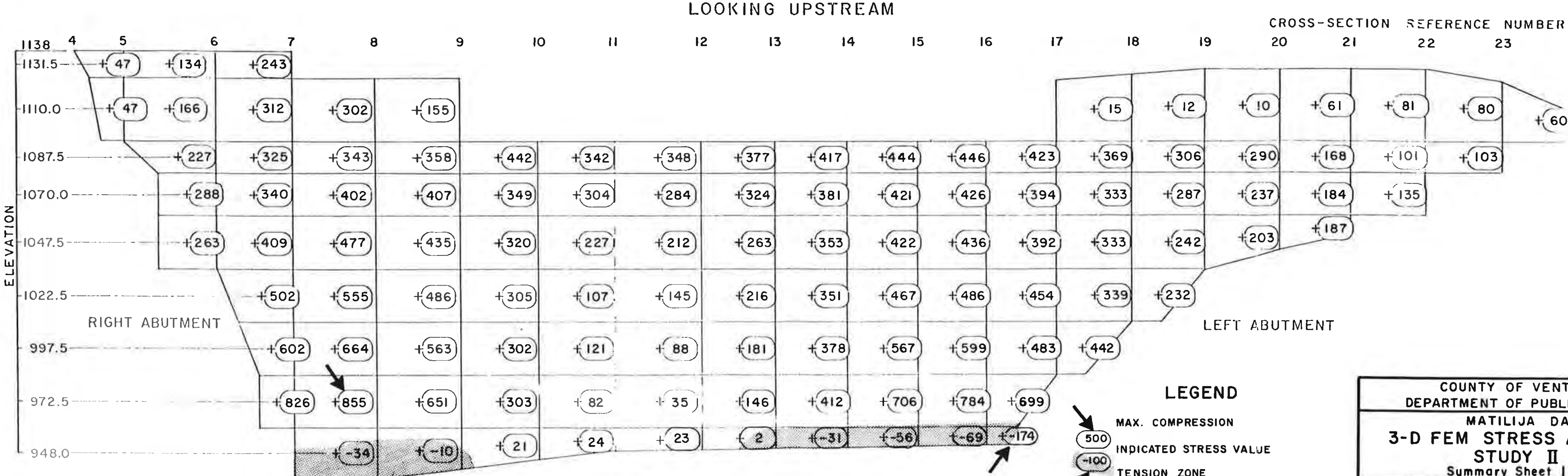
SILT EL. 1040

DEAD LOAD

UPSTREAM ARCH (σ_{xu}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM



DOWNSTREAM ARCH (σ_{xD}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

MAX. COMPRESSION

500 INDICATED STRESS VALUE

-100 TENSION ZONE

MAX. TENSION

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

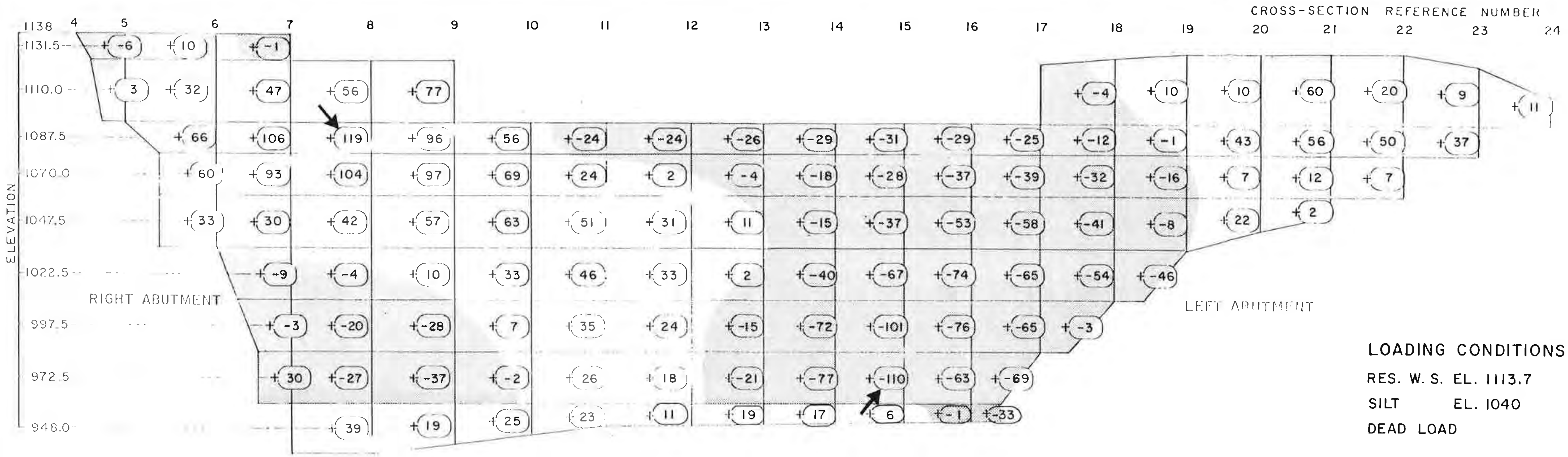
MATILIJA DAM

3-D FEM STRESS ANALYSIS
STUDY II
Summary Sheet 1 of 2

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EW / CK. R.P.S.	RECOMMENDED E.B.K.	APPROVED [signature]
DATE: AUGUST 3, 1972	HR-11-016	

EXHIBIT IV - 5



LOADING CONDITIONS:

RES. W. S. EL. 1113.7

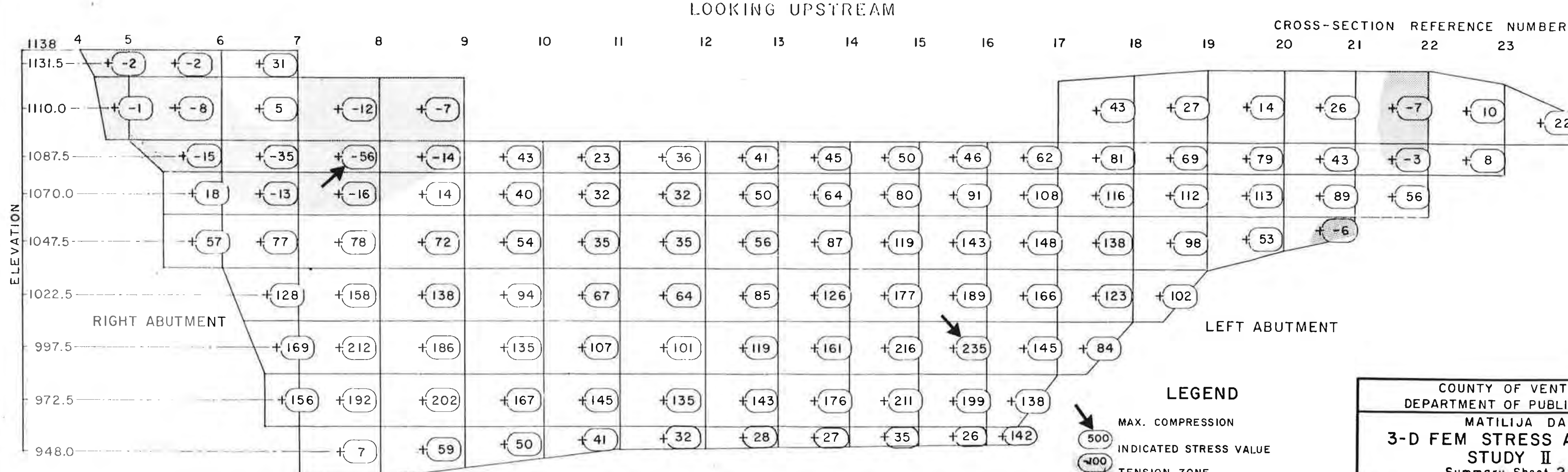
SILT EL. 1040

DEAD LOAD

UPSTREAM CANTILEVER (σ_{zu}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM



DOWNSTREAM CANTILEVER (σ_{zD}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

MAX. COMPRESSION

500 INDICATED STRESS VALUE

-100 TENSION ZONE

MAX. TENSION

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

ALL STRESSES IN PSI

COUNTY OF VENTURA DEPARTMENT OF PUBLIC WORKS		
MATILIJA DAM 3-D FEM STRESS ANALYSIS STUDY II Summary Sheet 2 of 2		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. E W CK. R.P.S.	RECOMMENDED C.M.K. DL	APPROVED G. S. Sarkaria
DATE: AUGUST 3, 1972	HR-11-017	

EXHIBIT IV - 6

CROSS-SECTION REFERENCE NUMBER

ELEVATION	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1138																				
1131.5	-164	14	-37																	
1110.0	-132	16	2	-11	1									3	4	3	-19	-72	-45	-121
1087.5		46	17	24	114	307	342	348	318	242	175	129	116	104	86	38	-31	-87	-138	
1070.0		62	13	42	155	276	388	386	353	257	169	106	79	61	61	39	-26	-109		
1047.5		-16	-14	24	155	325	433	445	399	275	151	66	32	16	31	14	-64			
1022.5			-58	-21	142	356	488	509	452	298	129	21	-2	-21	0					
997.5			-113	-53	137	383	536	565	501	327	135	4	-45	-30						
972.5			-69	-60	123	411	588	623	549	355	143	53	-59							
948.0				-126	-55	-94	-116	-123	-137	-172	-181	-223	-202							

RIGHT ABUTMENT

LEFT ABUTMENT

LOADING CONDITIONS:

RES. W. S. EL. 1095
SILT EL. 1040
DEAD LOAD
MAX. TEMPERATURE DROP

UPSTREAM ARCH (σ_{xu}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM
LOOKING UPSTREAM

CROSS-SECTION REFERENCE NUMBER

ELEVATION	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1138																				
1131.5	-192	-23	136																	
1110.0	-162	-6	159	199	124									3	5	-47	-13	-22	-22	-59
1087.5		50	153	214	217	188	104	106	125	185	230	250	237	190	94	-18	-46	-39	-97	
1070.0		122	164	266	249	156	88	77	106	182	234	252	222	166	64	4	-38	-59		
1047.5		69	256	350	292	156	52	40	84	188	266	287	242	178	61	42	-64			
1022.5			395	455	365	167	27	5	68	213	336	364	337	233	62					
997.5			554	602	466	185	3	-29	54	257	456	518	425	334						
972.5			796	844	581	205	-16	-62	36	304	624	764	680							
948.0				-119	-130	-115	-126	-138	-159	-188	-224	-195	-172							

RIGHT ABUTMENT

LEFT ABUTMENT

DOWNSTREAM ARCH (σ_{xd}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM
LOOKING UPSTREAM

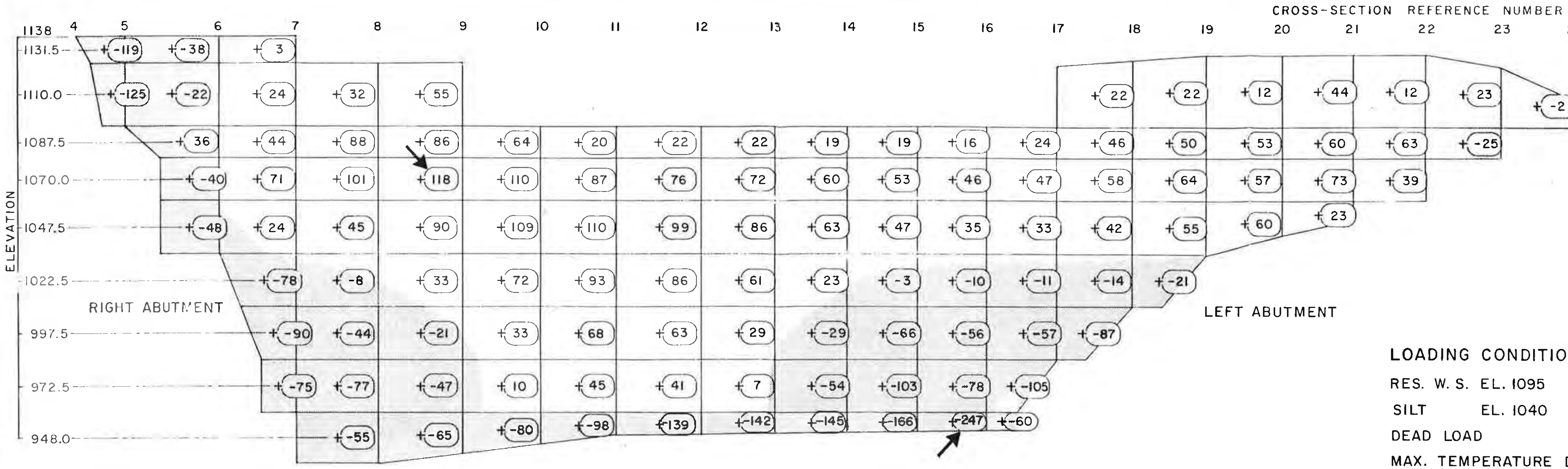
LEGEND

MAX. COMPRESSION
500 INDICATED STRESS VALUE
-100 TENSION ZONE
MAX. TENSION
COMPRESSION (+) OR WITHOUT SIGN
TENSION (-)
ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY III
Summary Sheet 1 of 2
INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. E W	RECOMMENDED	APPROVED
CK. R.P.S.	E.U.K.	G. S. Sarkaria

DATE: AUGUST 3, 1972 | HR-11-018



UPSTREAM CANTILEVER (σ_{zu}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

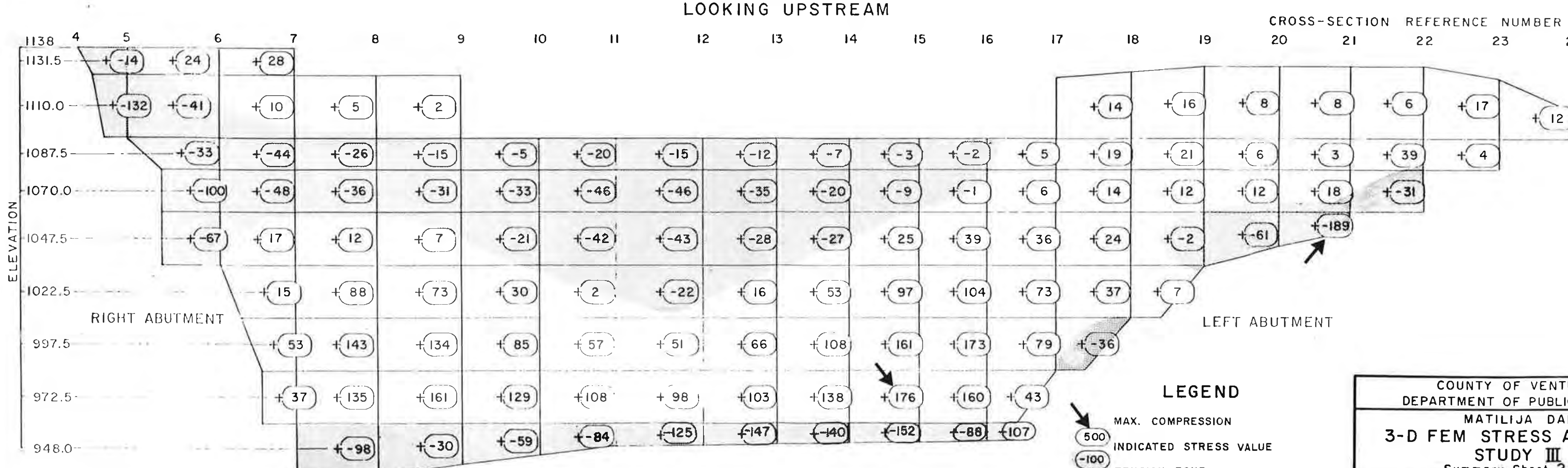
LOADING CONDITIONS:

RES. W. S. EL. 1095

SILT EL. 1040

DEAD LOAD

MAX. TEMPERATURE DROP



DOWNSTREAM CANTILEVER (σ_{zd}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

MAX. COMPRESSION

500 INDICATED STRESS VALUE

-100 TENSION ZONE

MAX. TENSION

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM

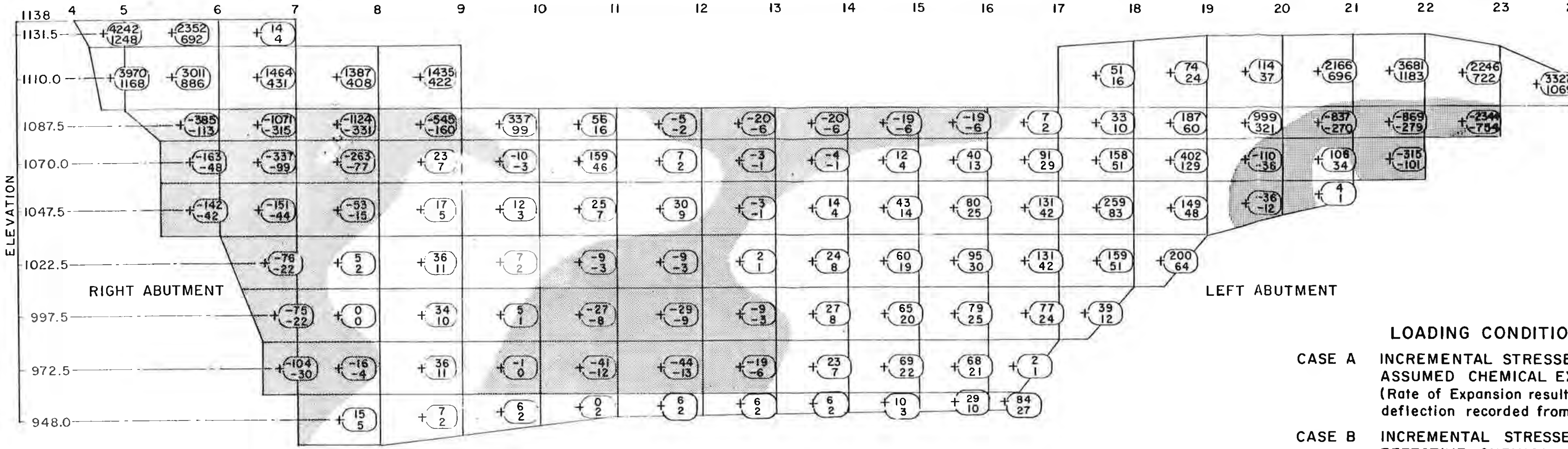
3-D FEM STRESS ANALYSIS STUDY III

Summary Sheet 2 of 2

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. E W — CK. R.P.S. — RECOMMENDED C.W.K. — APPROVED G.S. Sarkaria

DATE: AUGUST 3, 1972 — HR-11-019



UPSTREAM ARCH (σ_{xu}) STRESSES

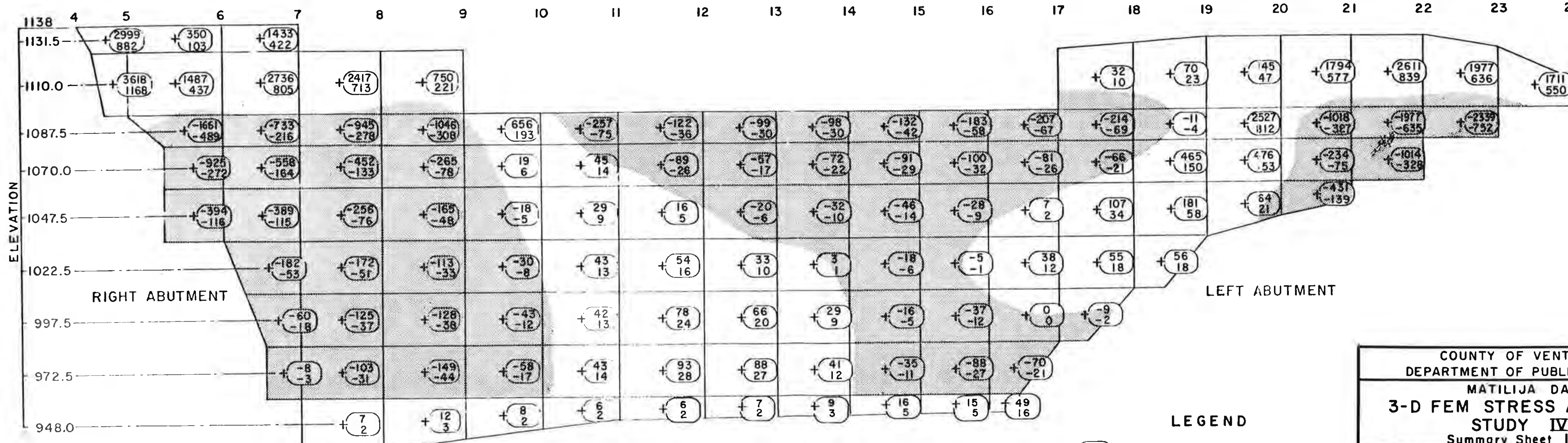
DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LOADING CONDITIONS:

CASE A INCREMENTAL STRESSES FOR ASSUMED CHEMICAL EXPANSION (Rate of Expansion resulting in dam deflection recorded from 1965 to 1972)

CASE B INCREMENTAL STRESSES FOR EFFECTIVE CHEMICAL EXPANSION 1965 TO 1972



DOWNSTREAM ARCH (σ_{xD}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

25 STRESS VALUE CASE A
30 STRESS VALUE CASE B

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM

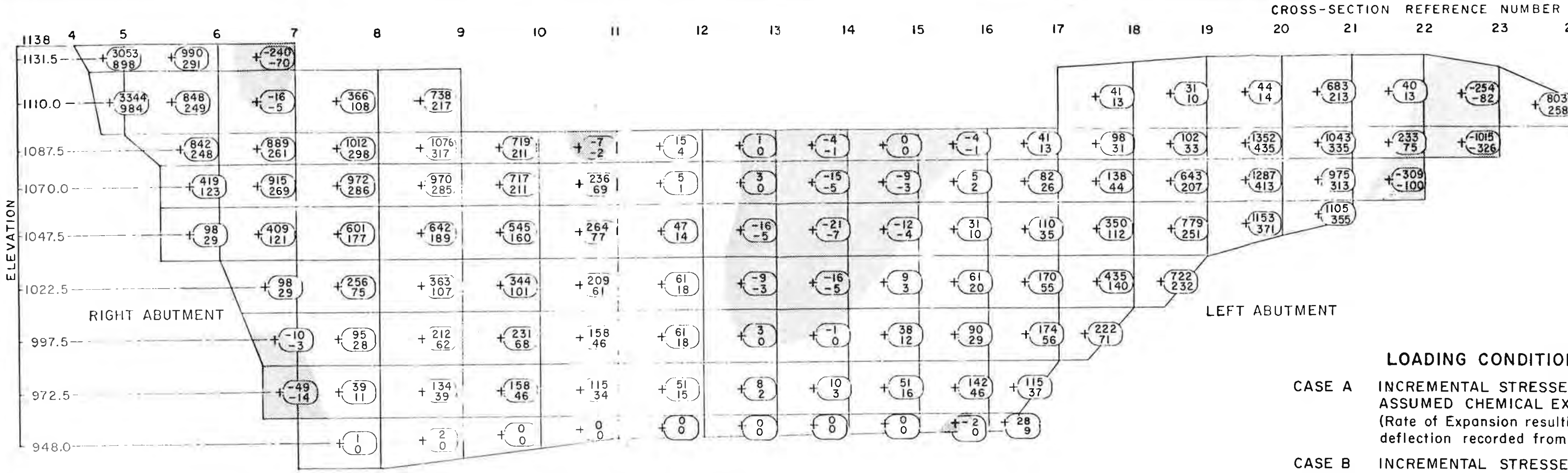
3-D FEM STRESS ANALYSIS STUDY IV

Summary Sheet 1 of 2

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EW RECOMMENDED APPROVED
CK. R.P.S C.O.K. D.L. G.S. Sarkaria

DATE: AUGUST 3, 1972 HR-11-020



UPSTREAM CANTILEVER (σ_{zu}) STRESSES

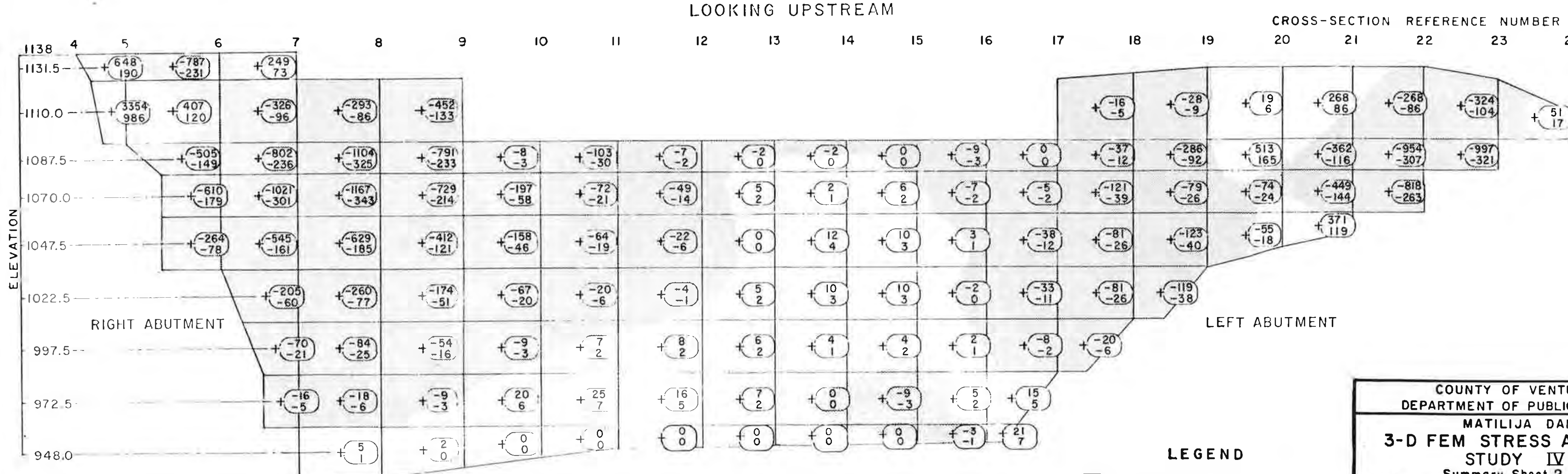
DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LOADING CONDITIONS:

CASE A INCREMENTAL STRESSES FOR ASSUMED CHEMICAL EXPANSION (Rate of Expansion resulting in dam deflection recorded from 1965 to 1972)

CASE B INCREMENTAL STRESSES FOR EFFECTIVE CHEMICAL EXPANSION 1965 TO 1972



DOWNSTREAM CANTILEVER (σ_{zd}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

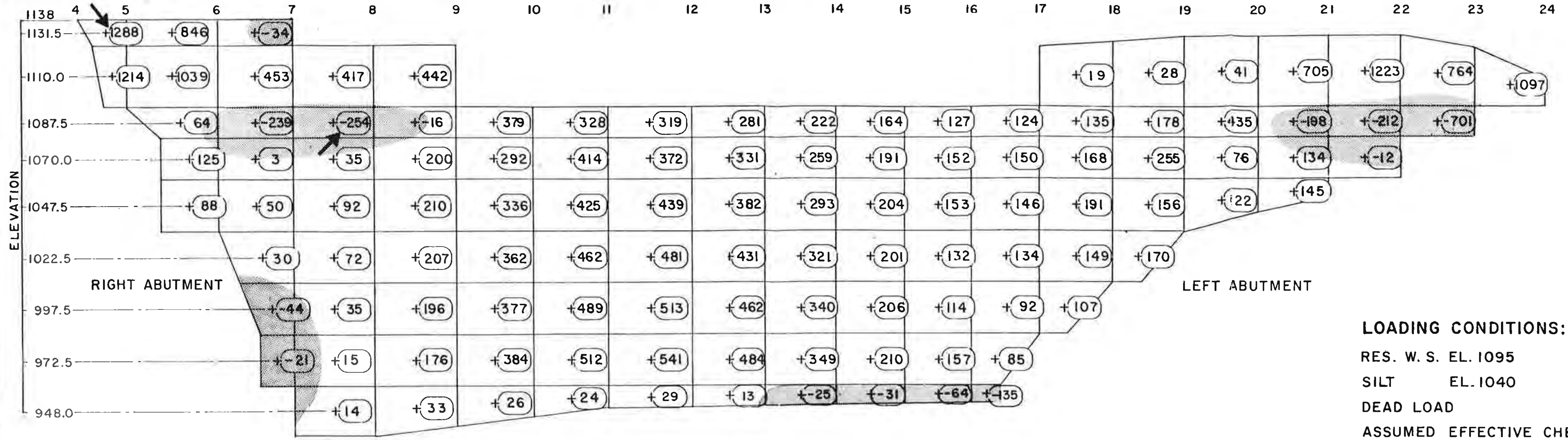
25 STRESS VALUE CASE A
30 STRESS VALUE CASE B

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

ALL STRESSES IN PSI

COUNTY OF VENTURA DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM 3-D FEM STRESS ANALYSIS STUDY IV Summary Sheet 2 of 2
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.
DR. EW / CK. RPS. — RECOMMENDED: E.B.K. T.L. — APPROVED: S.S. Sarkaria
DATE: AUGUST 3, 1972 — HR-11-021



UPSTREAM ARCH (σ_{XU}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

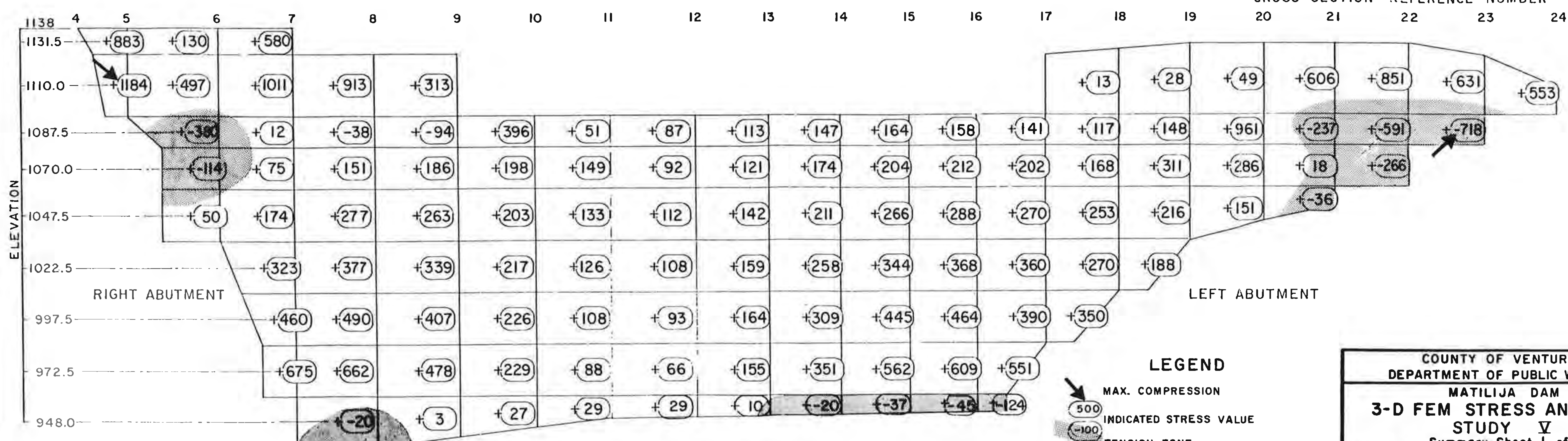
LOADING CONDITIONS:

RES. W. S. EL. 1095

SILT EL. 1040

DEAD LOAD

ASSUMED EFFECTIVE CHEMICAL EXPANSION



DOWNSTREAM ARCH (σ_{XD}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

MAX. COMPRESSION

500 INDICATED STRESS VALUE

-100 TENSION ZONE

MAX. TENSION

COMPRESSION (+) OR WITHOUT SIGN

TENSION (–)

ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

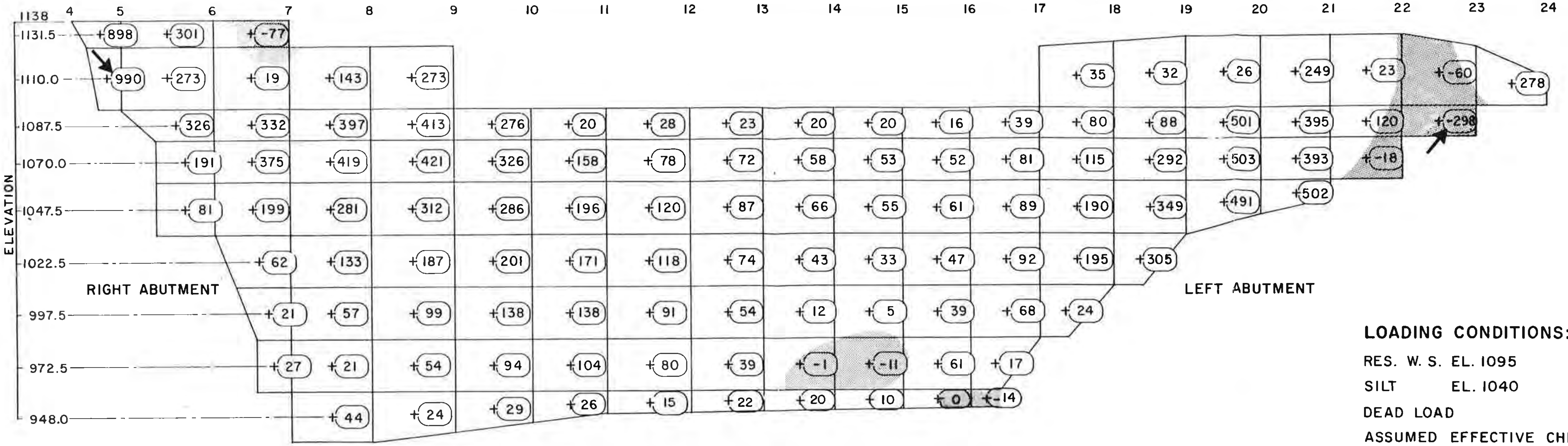
MATILIJA DAM

3-D FEM STRESS ANALYSIS STUDY V

Summary Sheet 1 of 2

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. E.W. / CK. R.P.S.	RECOMMENDED	APPROVED
DATE: AUGUST 3, 1972	HR-11-022	



UPSTREAM CANTILEVER (σ_{zu}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

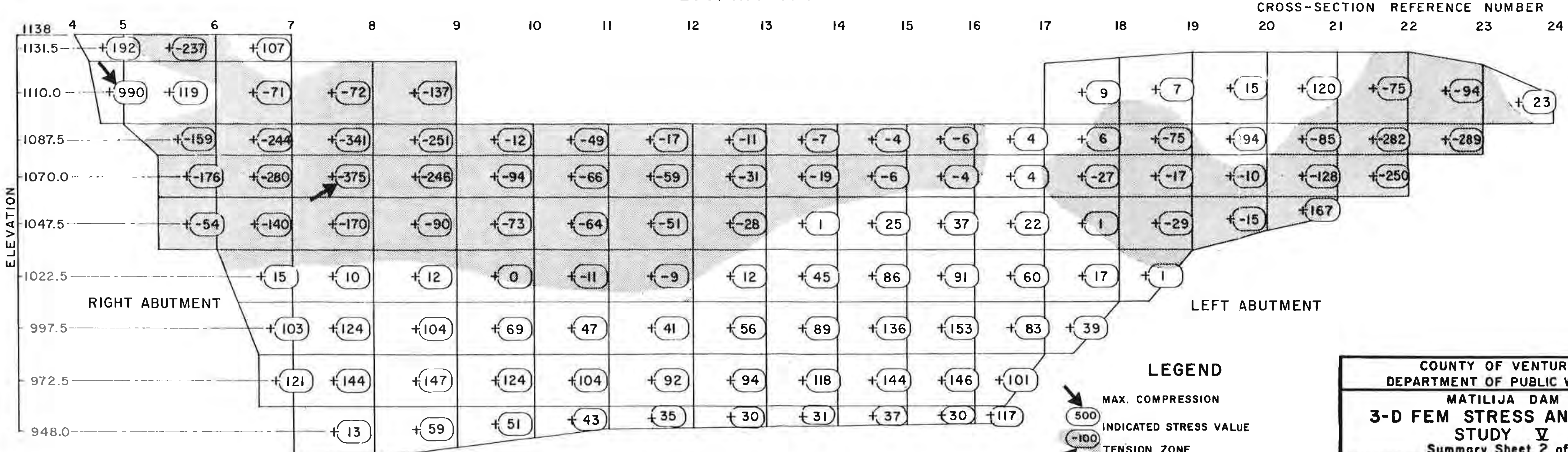
LOADING CONDITIONS:

RES. W. S. EL. 1095

SILT EL. 1040

DEAD LOAD

ASSUMED EFFECTIVE CHEMICAL EXPANSION



DOWNSTREAM CANTILEVER (σ_{zD}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

MAX. COMPRESSION

500 INDICATED STRESS VALUE

-100 TENSION ZONE

MAX. TENSION

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

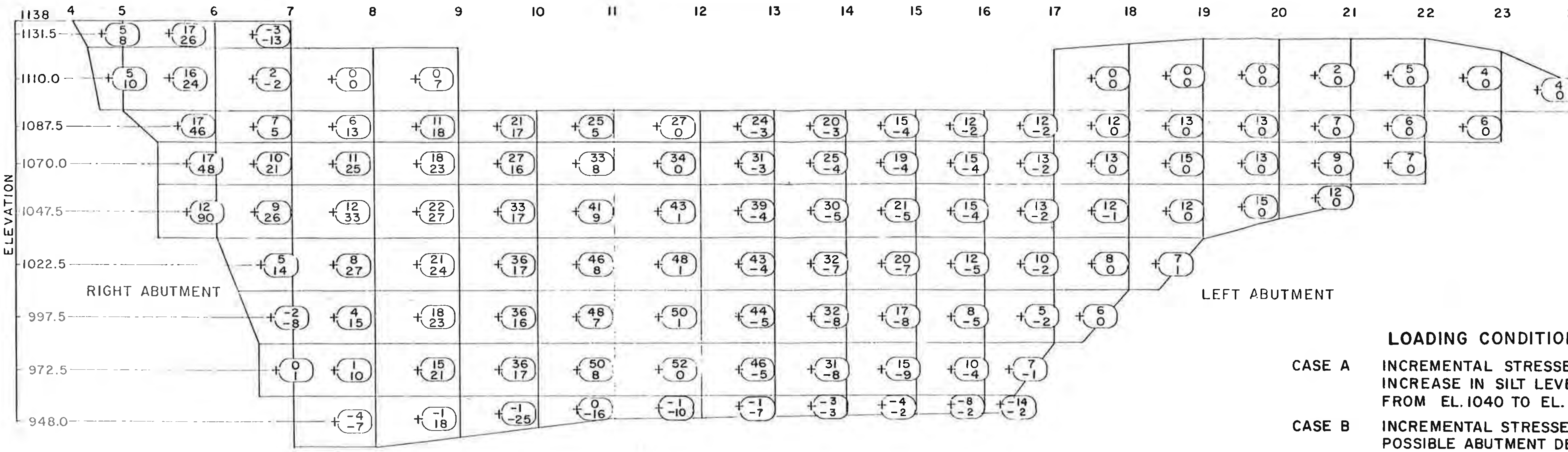
MATILIJA DAM

3-D FEM STRESS ANALYSIS STUDY V

Summary Sheet 2 of 2

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EW	RECOMMENDED	APPROVED
CK. RPS	E.B.K. DL	G.S. Sarkaria
DATE: AUGUST 3, 1972	HR-11-023	



UPSTREAM ARCH (σ_{XU}) STRESSES

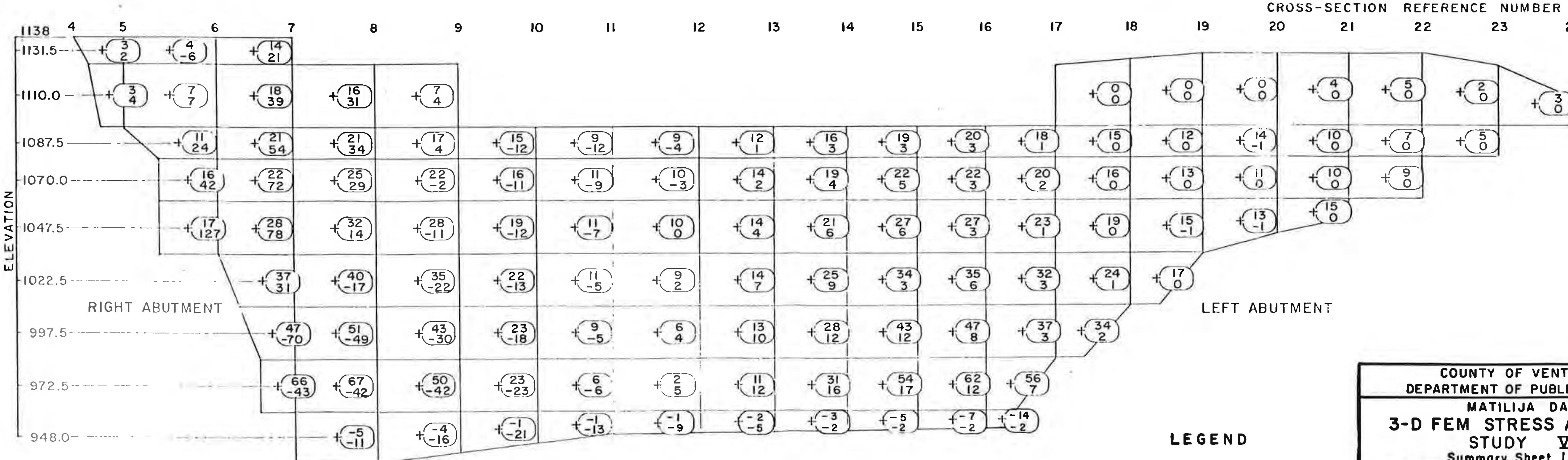
DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LOADING CONDITIONS:

CASE A — INCREMENTAL STRESSES FOR INCREASE IN SILT LEVEL FROM EL. 1040 TO EL. 1069

CASE B — INCREMENTAL STRESSES FOR POSSIBLE ABUTMENT DEFORMATION IN LOWER RIGHT REGION (0.085 INCH COMPRESSION - METER DH-3R)



DOWNSTREAM ARCH (σ_{XD}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

25 STRESS VALUE CASE A
30 STRESS VALUE CASE B

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

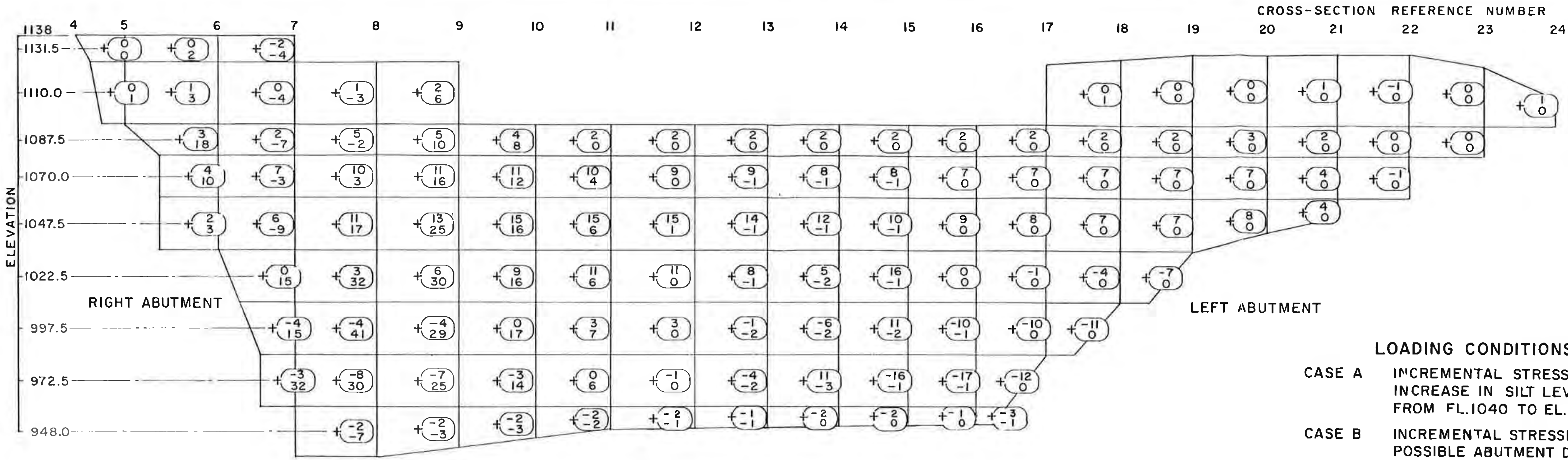
ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY VI
Summary Sheet 1 of 2

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EW CK. R.P.S.	RECOMMENDED C.S.K.	APPROVED G.S. Sarkaria
DATE: AUGUST 3, 1972	HR - 11 - 024	



LOADING CONDITIONS:

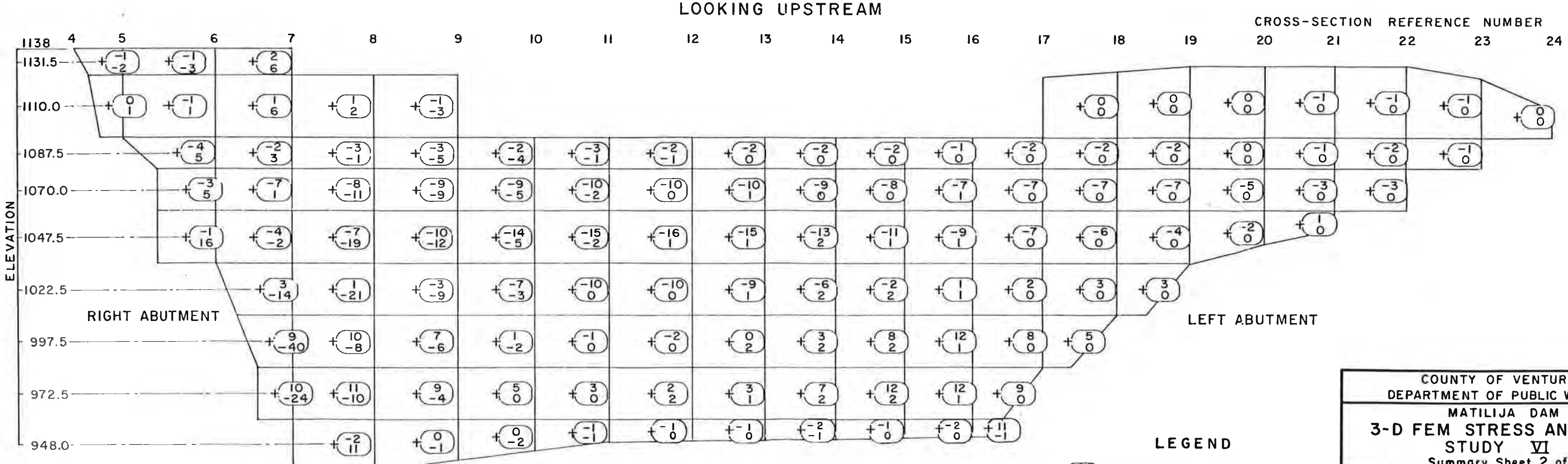
CASE A INCREMENTAL STRESSES FOR INCREASE IN SILT LEVEL FROM EL.1040 TO EL. 1069

CASE B INCREMENTAL STRESSES FOR POSSIBLE ABUTMENT DEFORMATION IN LOWER RIGHT REGION (0.085 INCH COMPRESSION - METER DH-3R)

UPSTREAM CANTILEVER (σ_{zu}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM



DOWNSTREAM CANTILEVER (σ_{zD}) STRESSES

DEVELOPED PROFILE ALONG AXIS OF DAM

LOOKING UPSTREAM

LEGEND

(25) STRESS VALUE CASE A
(30) STRESS VALUE CASE B

COMPRESSION (+) OR WITHOUT SIGN

TENSION (-)

ALL STRESSES IN PSI

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM

3-D FEM STRESS ANALYSIS
STUDY VI

Summary Sheet 2 of 2

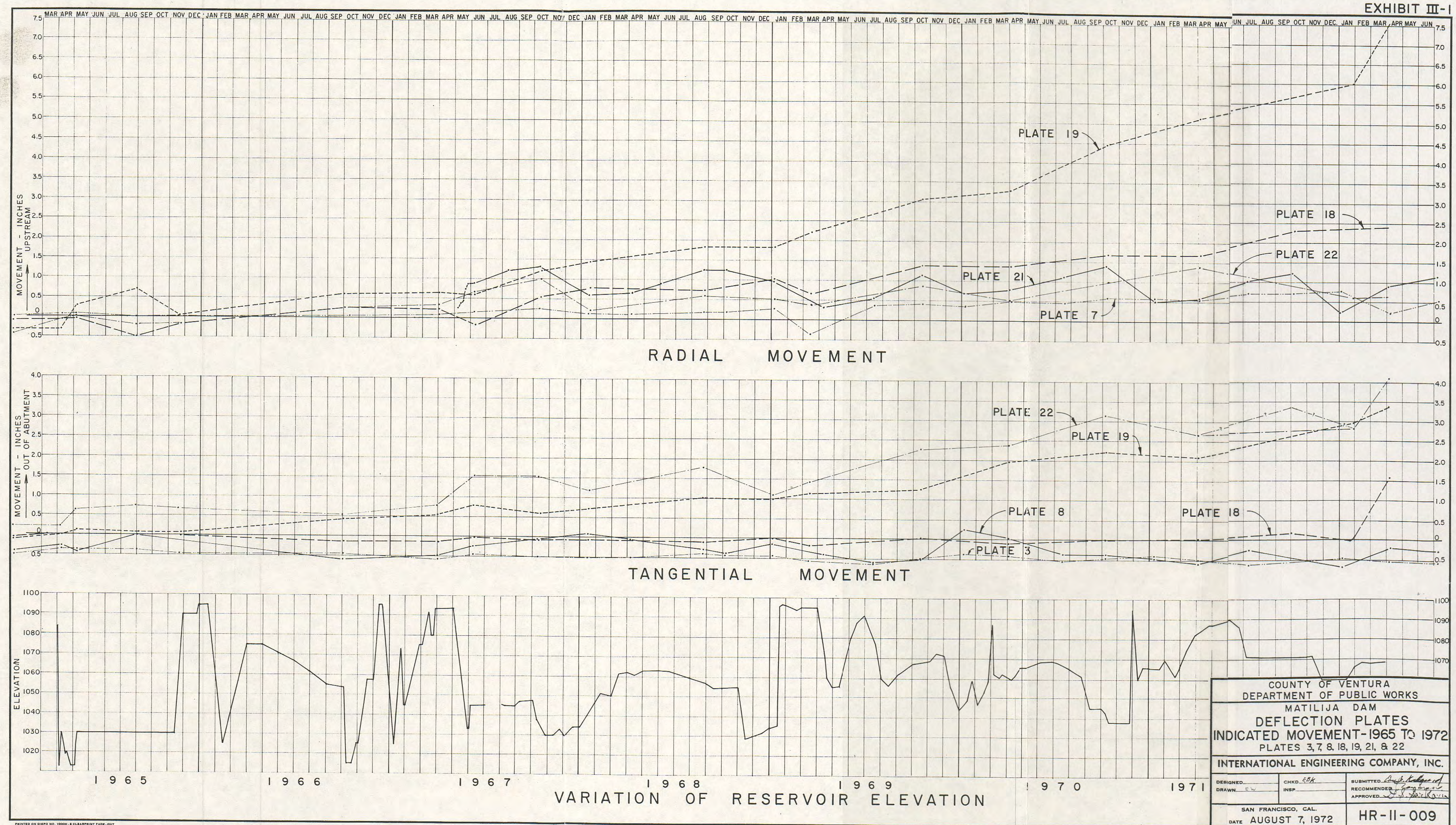
INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EW CK. R.P.S.	RECOMMENDED C.B.K.	APPROVED S.S. Sarkaria
DATE: AUGUST 3, 1972	HR-11-025	

EXHIBIT II-2

Sta.6+16
Sta.3+09
Sta.0+00
2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27
1138
1125
1095
1080
1060
1035
1010
985
960
936
900
850
ELEVATION - FEET
Sliding Joint
LEGEND
Concrete E = 2,000,000 psi *
Concrete E = 25,000 psi *
Rock Mass E = 1,000,000 psi
Rock Mass E = 500,000 psi
Rock Mass E = 1,500,000 psi
Rock Mass E = 1,500,000 psi
* For Static Stress Analysis
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM
3-D FEM STRESS ANALYSIS
MESH LAYOUT Sheet 1 of 4
INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.
DR. A.M.W.
CK. RPS
RECOMMENDED
APPROVED
DATE: AUGUST 9, 1972
HR-11-002

EXHIBIT III-1



COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM
DEFLECTION PLATES
INDICATED MOVEMENT-1965 TO 1972
PLATES 3, 7, 8, 18, 19, 21, & 22

INTERNATIONAL ENGINEERING COMPANY, INC.

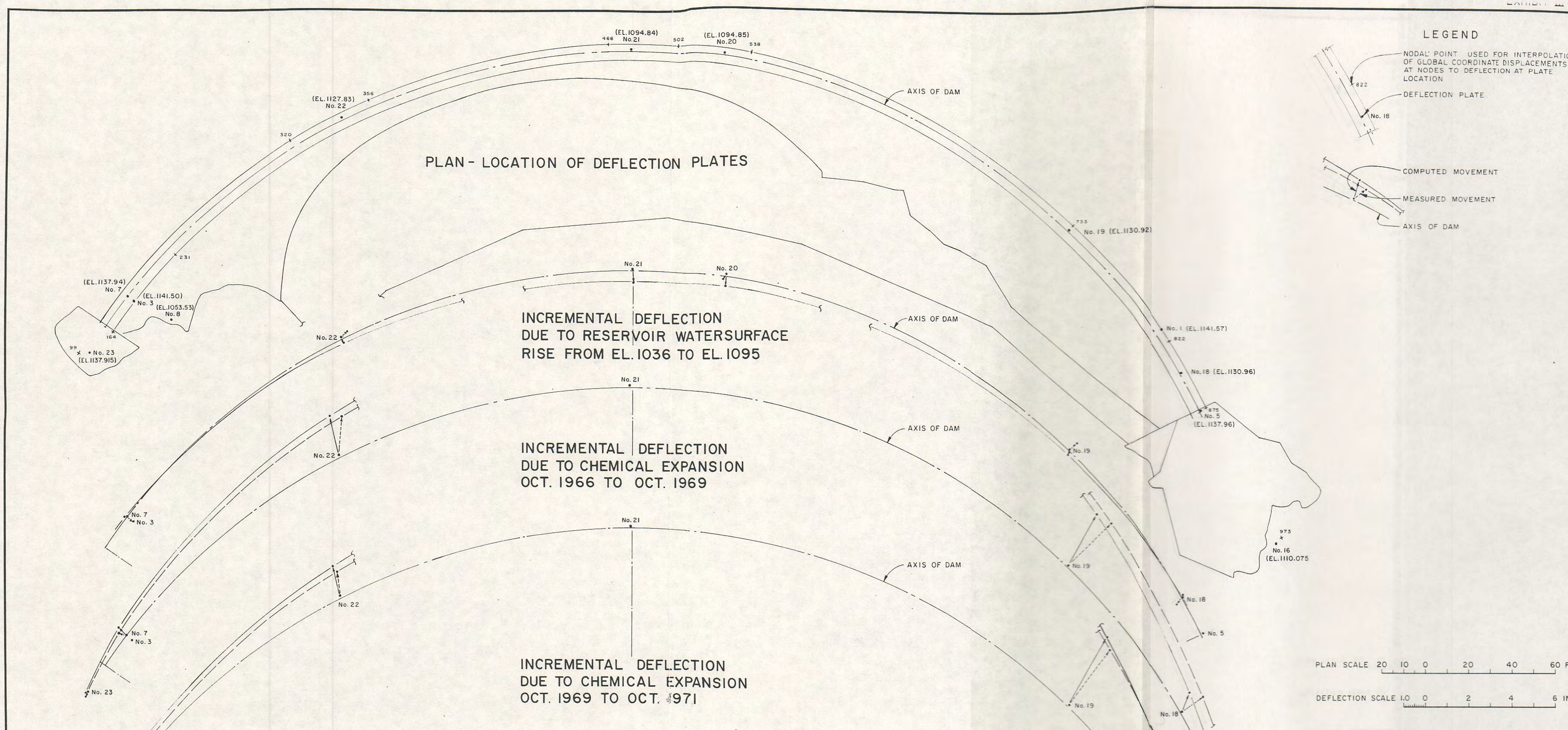
DESIGNED
DRAWN EW
CHKD EBK
INSP
SUBMITTED
RECOMMENDED
APPROVED

SAN FRANCISCO, CAL.
DATE AUGUST 7, 1972

HR-11-009

INDICATED DEFORMATION METER MOVEMENT - 1965 to 1972

HR-II-010



COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

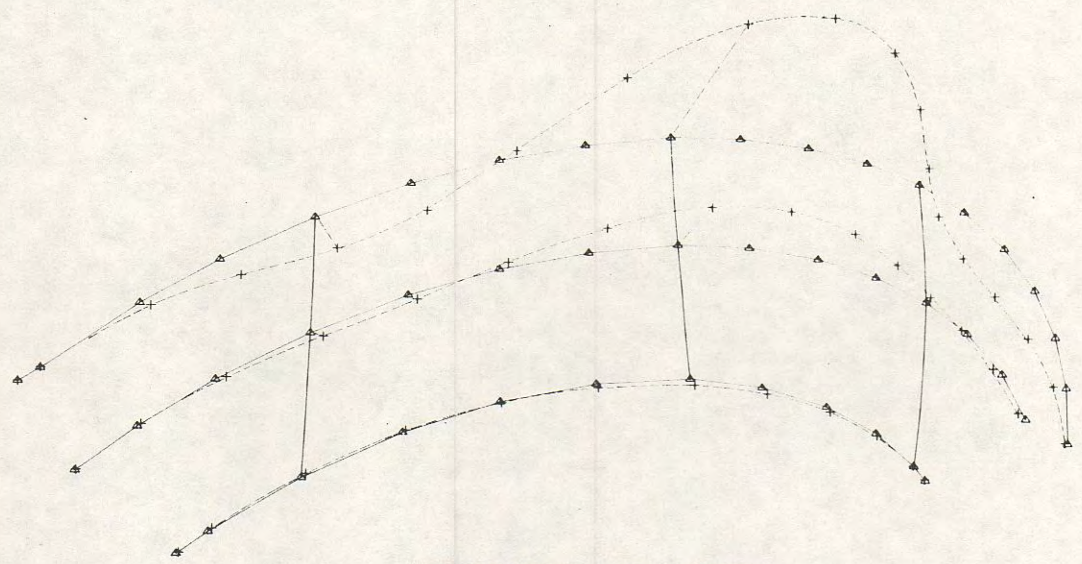
MATILIJA DAM

INCREMENTAL DAM DEFLECTION

COMPARISON - INDICATED VS COMPUTED

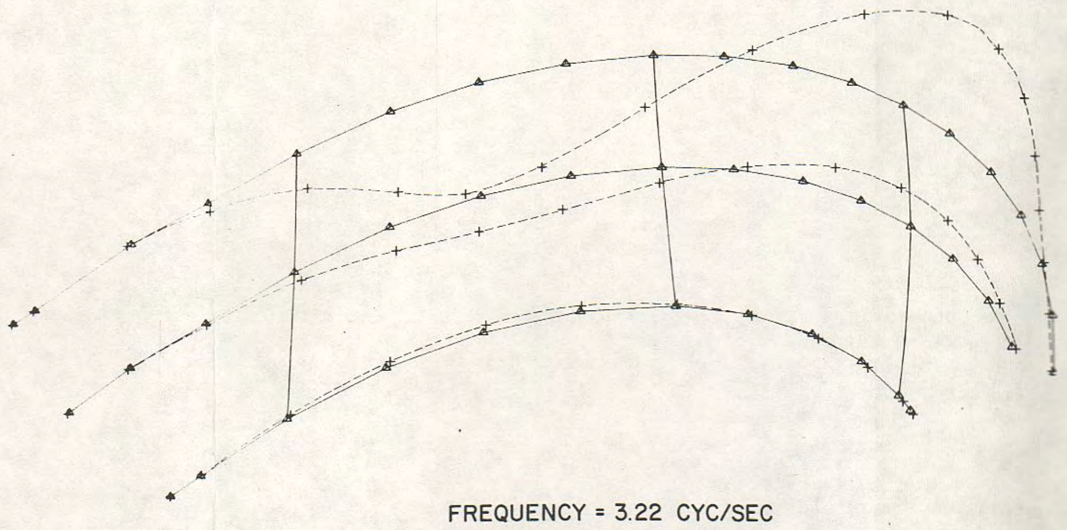
INTERNATIONAL ENGINEERING COMPANY, INC.

DESIGNED ______ DRAWN AO EW	CHKD. R.P.Lha... INSP.	SUBMITTED [signature] RECOMMENDED [signature] APPROVED [signature]
SAN FRANCISCO, CAL. DATE AUGUST 11, 1972		HR-11-011



FREQUENCY = 3.094 CYC/SEC
PERIOD = 0.323 SEC

MATILIJA DAM FIRST MODE OF VIBRATION

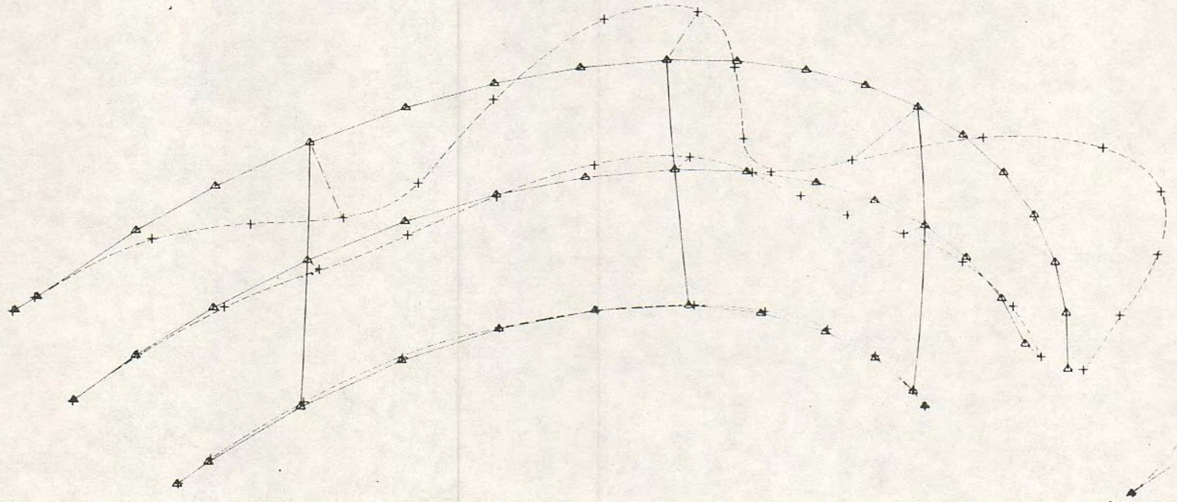


FREQUENCY = 3.22 CYC/SEC
PERIOD = 0.311 SEC

MATILIJA DAM SECOND MODE OF VIBRATION

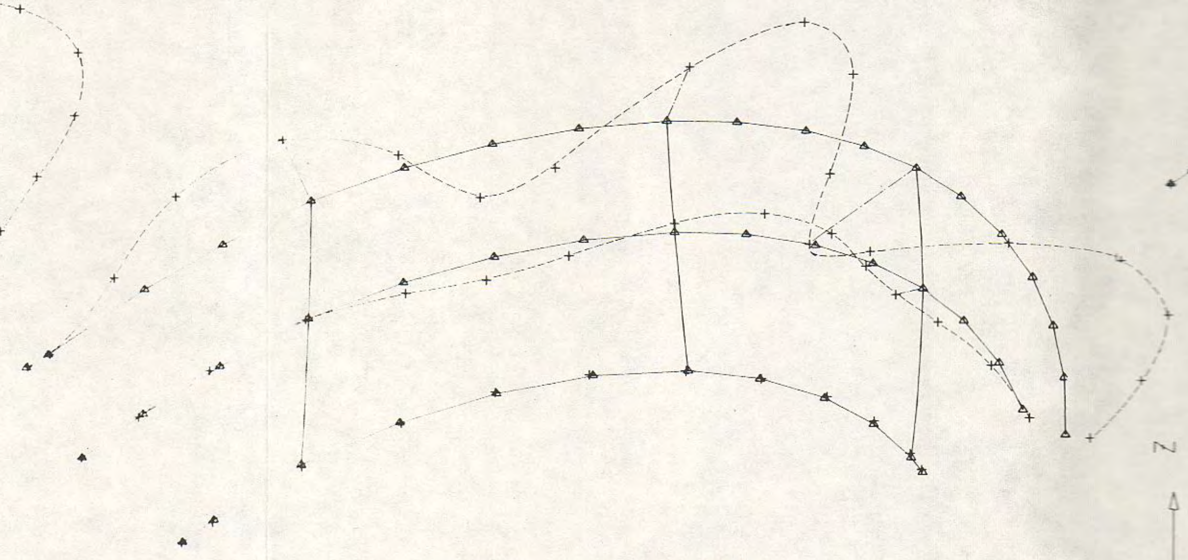
FREQUENCY = 4.17 CYC/SEC
PERIOD = 0.24 SEC

MATILIJA DAM THIRD MODE OF VIBRATION



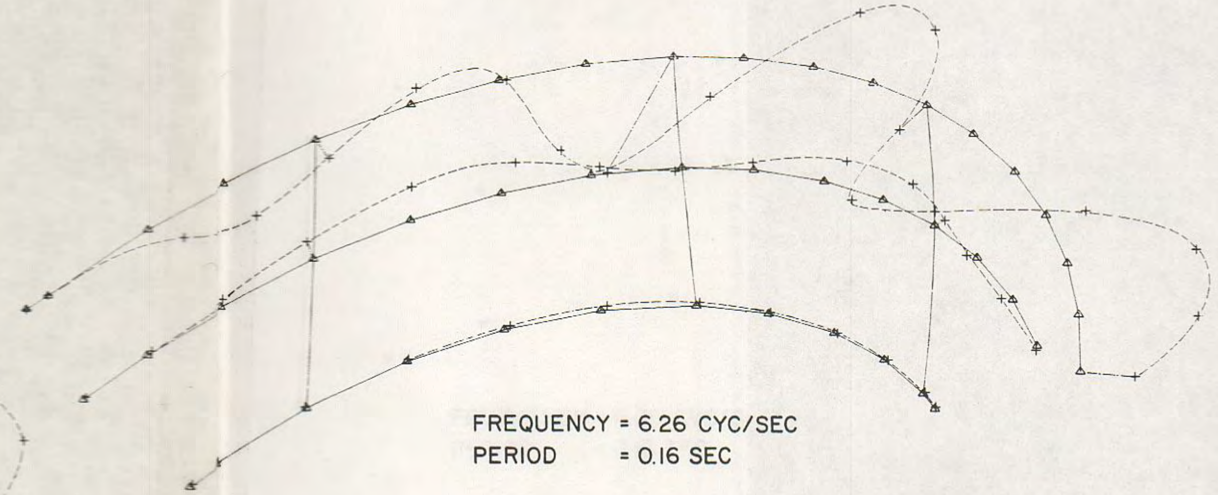
FREQUENCY = 4.313 CYC/SEC
PERIOD = 0.232 SEC

MATILIJA DAM FORTH MODE OF VIBRATION



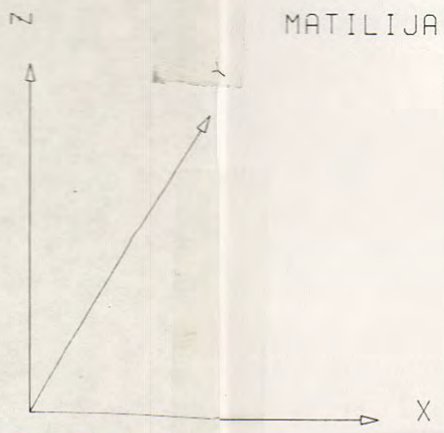
FREQUENCY = 5.22 CYC/SEC
PERIOD = 0.191 SEC

MATILIJA DAM FIFTH MODE OF VIBRATION



FREQUENCY = 6.26 CYC/SEC
PERIOD = 0.16 SEC

MATILIJA DAM SIXTH MODE OF VIBRATION



COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM
DYNAMIC RESPONSE
MODE SHAPES AND FREQUENCIES
RESERVOIR ELEVATION 1095

INTERNATIONAL ENGINEERING COMPANY, INC.

DESIGNED
DRAWN
CHKD
INSP
SUBMITTED
RECOMMENDED
APPROVED

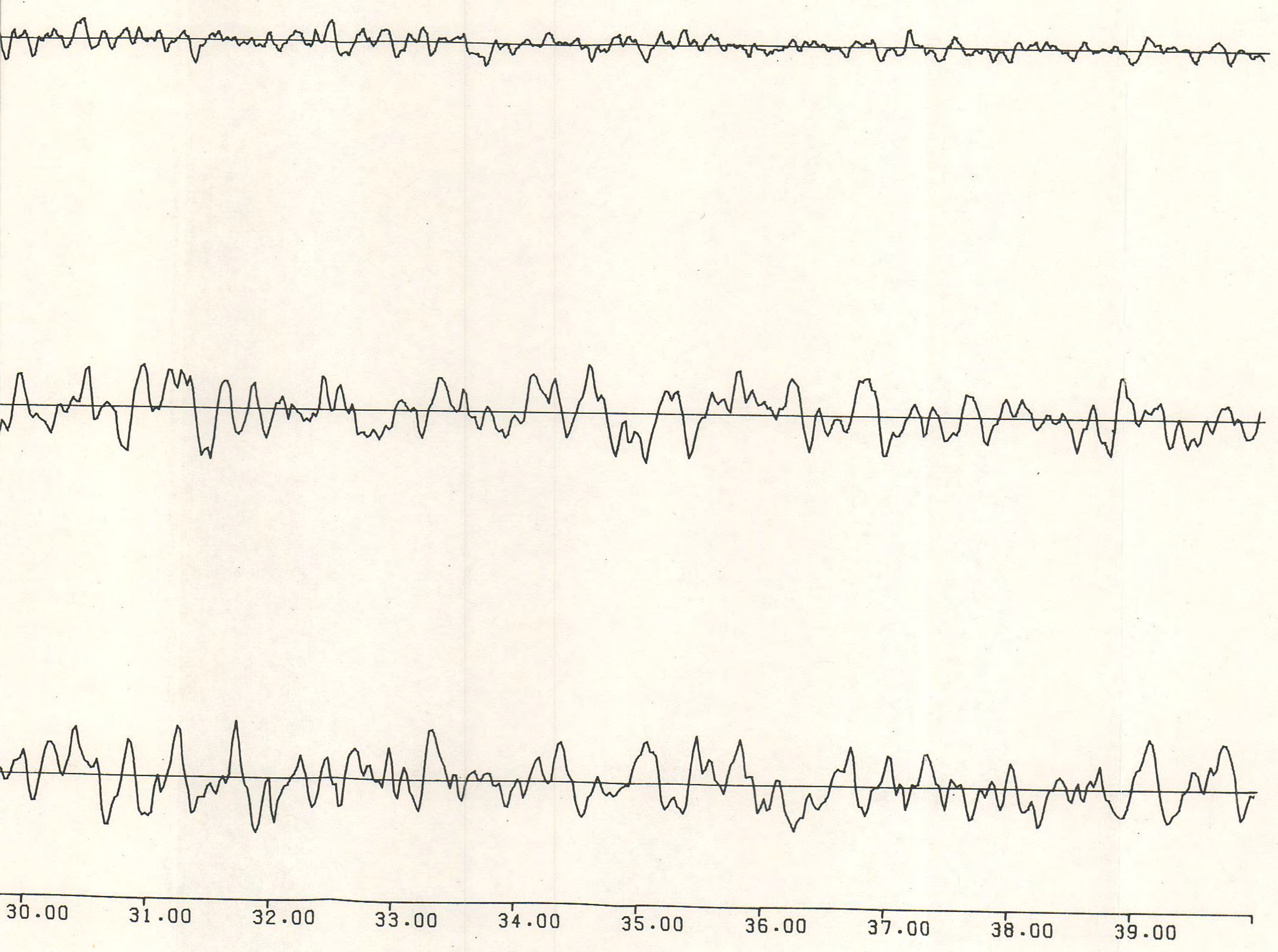
AUGUST 3, 1972

HR-II-026

SAN ANDREAS FAULT EQ., MAG. 8+
(HOUSNER A-I
SCALED TO 2/3 AMPLITUDE)

SAN ANDREAS FAULT EQ., MAG. 8+
(HOUSNER A-1
SCALED TO 2/3 AMPLITUDE)

EXHIBIT V - 2



DESIGN EARTHQUAKE I

HR - 11 - 027

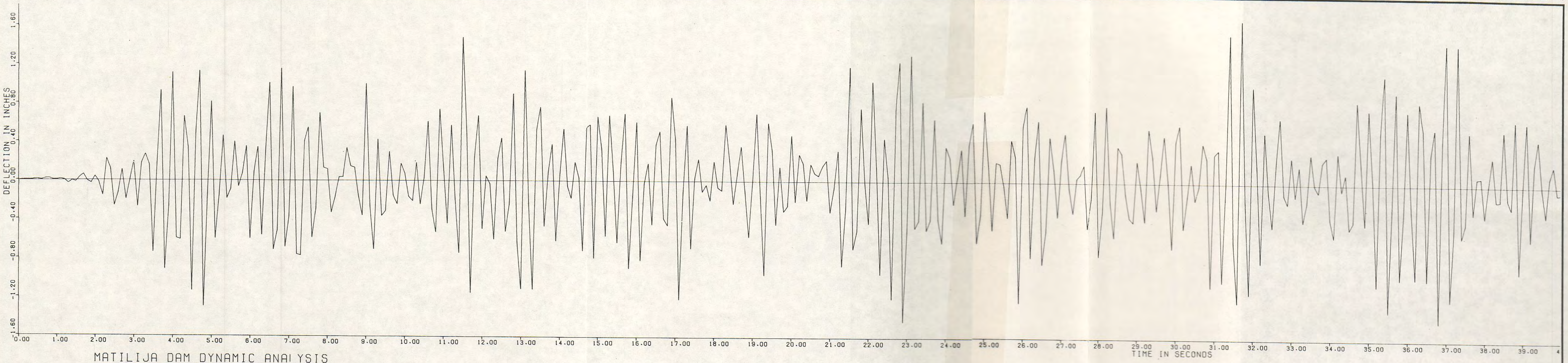
EL CENTRO EARTHQUAKE

SCALED TO 1.25 AMPLITUDE

DESIGN EARTHQUAKE 2

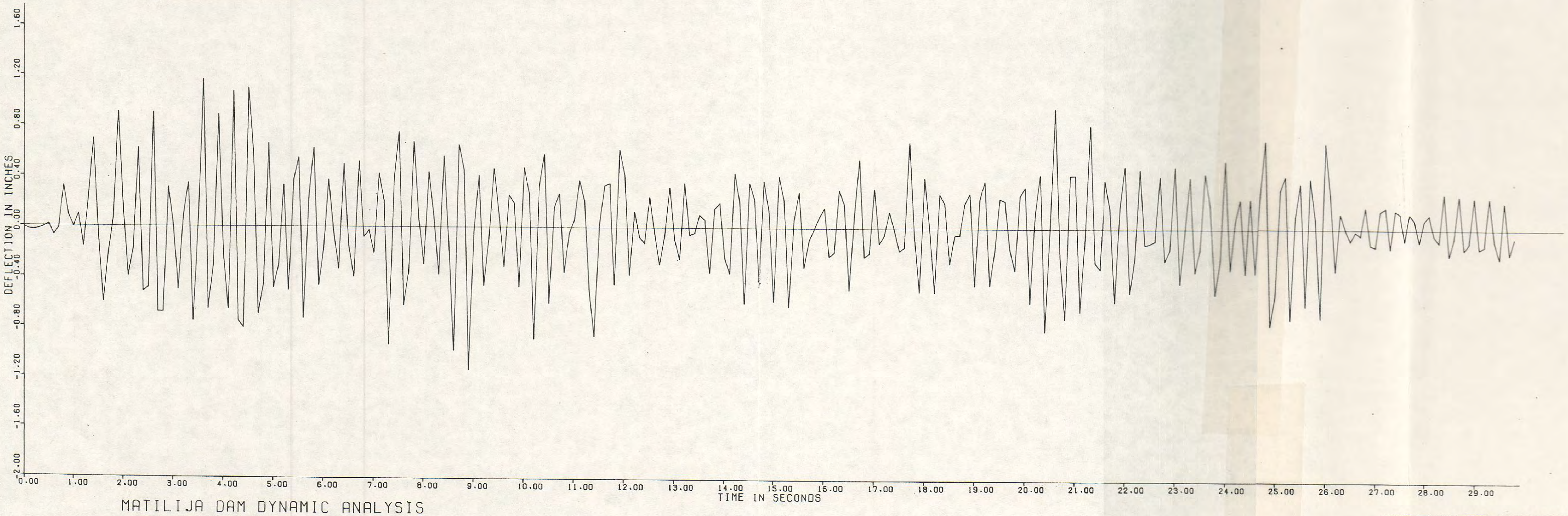
HR - 11 - 030

EXHIBIT V - 4



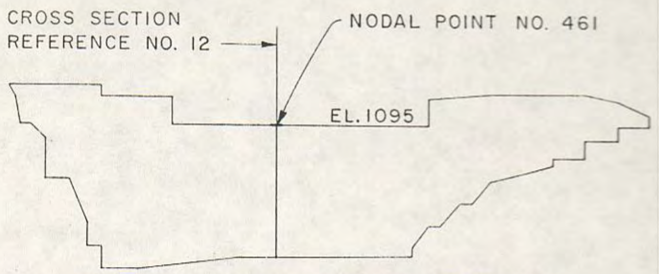
MATILIJA DAM DYNAMIC ANALYSIS

DEFLECTION HISTORY NODAL POINT NO 461.



MATILIJA DAM DYNAMIC ANALYSIS

DEFLECTION HISTORY NODAL POINT NO 461.



LOCATION KEY - LOOKING UPSTREAM

NOTE: (-) INDICATES DOWNSTREAM DEFLECTION

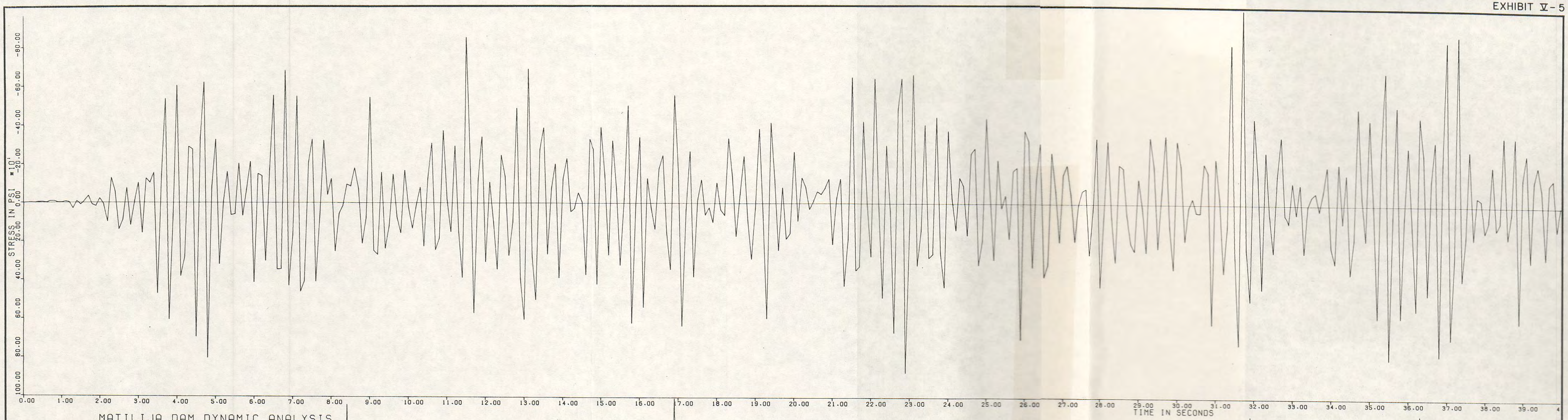
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM
3-D FEM STRESS ANALYSIS
D/S CREST AT CROWN CANTILEVER
DEFLECTION HISTORY

INTERNATIONAL ENGINEERING COMPANY, INC.

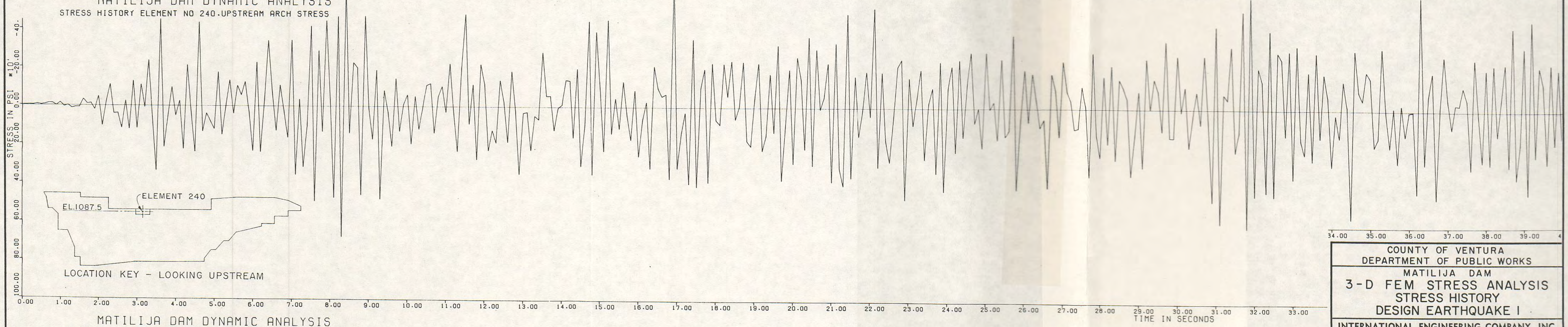
DESIGNED. DRAWN. E.W.	CHKD. INSP.	SUBMITTED RECOMMENDED APPROVED

SAN FRANCISCO, CAL.
DATE AUGUST 14, 1972

HR-II-029



MATILIJA DAM DYNAMIC ANALYSIS
STRESS HISTORY ELEMENT NO 240.UPSTREAM ARCH STRESS



MATILIJA DAM DYNAMIC ANALYSIS
STRESS HISTORY ELEMENT NO 240.DOWNSTREAM ARCH STRESS

NOTE: (-) INDICATES TENSILE STRESS

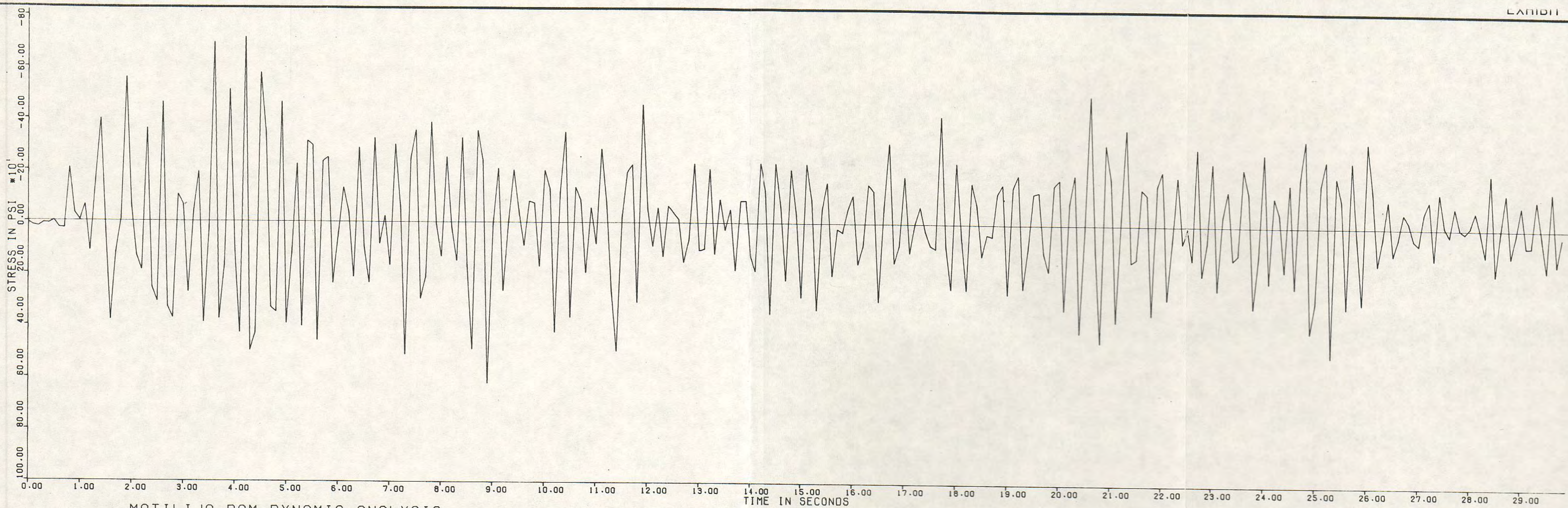
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM
3-D FEM STRESS ANALYSIS
STRESS HISTORY
DESIGN EARTHQUAKE I

INTERNATIONAL ENGINEERING COMPANY, INC.

DESIGNED DRAWN E.W.	CHKD. R.P. [signature] INSP.	SUBMITTED [signature] RECOMMENDED [signature] APPROVED [signature]

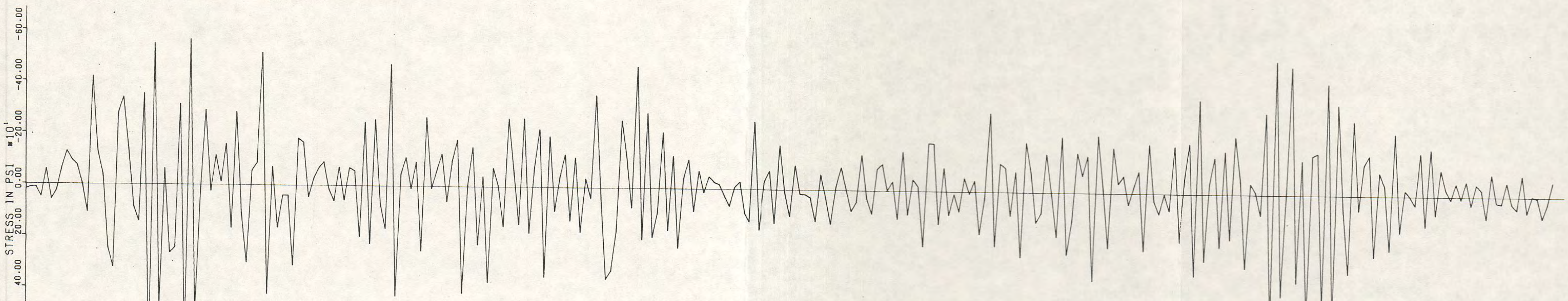
SAN FRANCISCO, CAL.
DATE AUGUST 14, 1972

HR-II-030



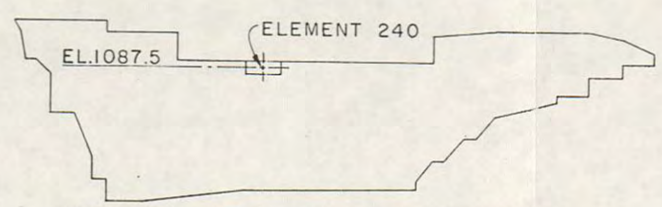
MATILIJA DAM DYNAMIC ANALYSIS

STRESS HISTORY ELEMENT NO 240.UPSTREAM ARCH STRESS



MATILIJA DAM DYNAMIC ANALYSIS

STRESS HISTORY ELEMENT NO 240.DOWNSTREAM ARCH STRESS



LOCATION KEY – LOOKING UPSTREAM

NOTE: (–) INDICATES TENSILE STRESS

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM
3-D FEM STRESS ANALYSIS
STRESS HISTORY
DESIGN EARTHQUAKE 2

INTERNATIONAL ENGINEERING COMPANY, INC.

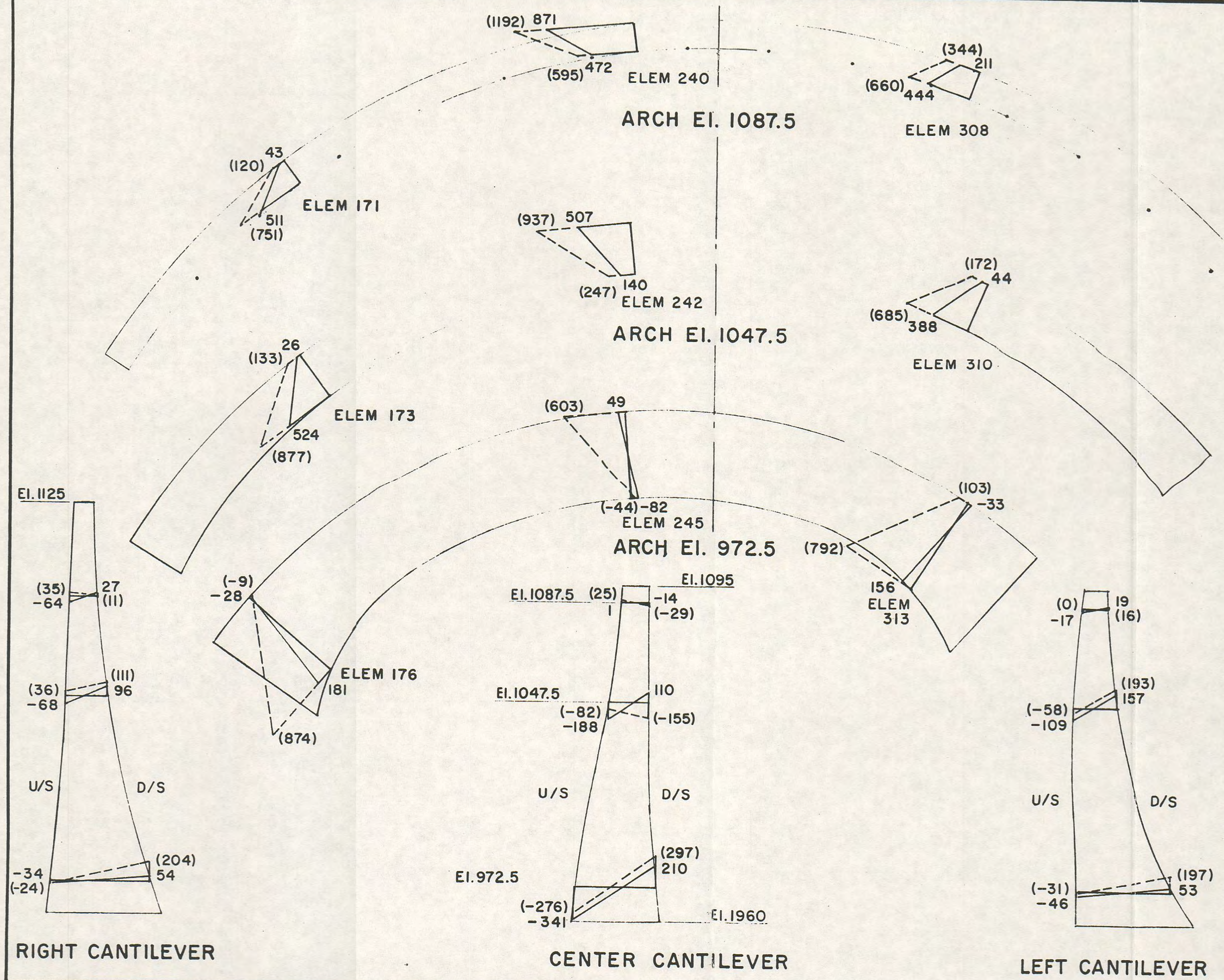
DESIGNED ____ CHKD R.P. Sharma SUBMITTED [signature]
DRAWN E.W. INSP. ____ RECOMMENDED [signature]
APPROVED [signature]

SAN FRANCISCO, CAL.
DATE AUGUST 14, 1972

HR-11-031

STUDY I-EQI

SUMMARY OF ARCH AND CANTILEVER STRESSES AT 23.0 SECOND INTERVAL OF DESIGN EARTHQUAKE I



LEGEND

DYNAMIC STRESS DIAGRAM

COMBINED STRESS DIAGRAM

DYNAMIC VALUE	100
COMBINED VALUE	(150)

STATIC LOADING CONDITIONS

RES. W. S. EL. 1095.0

SILT ELEVATION 1040.0

ALL STRESSES IN PSI, TENSION (-)

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

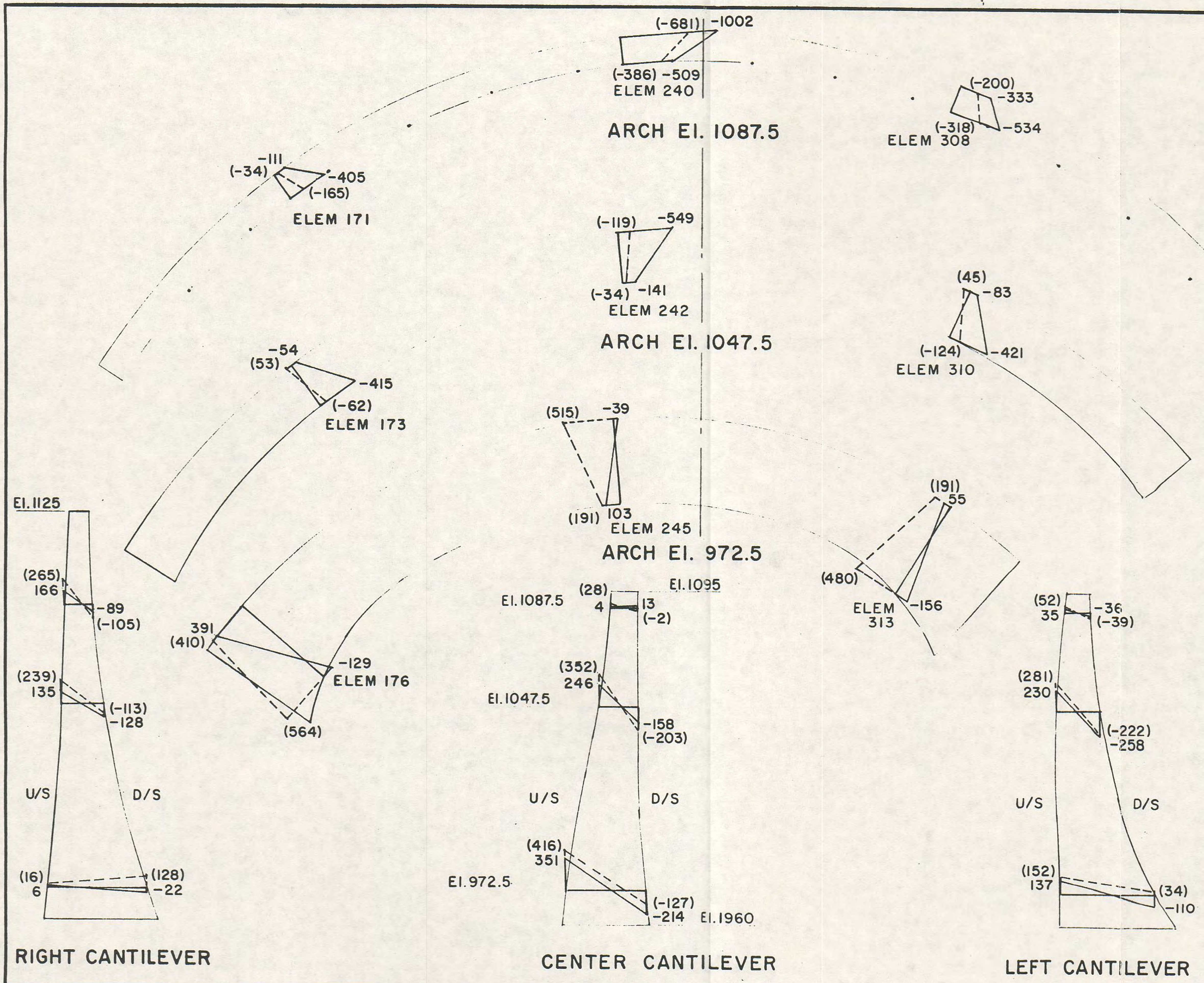
MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY I-EQI
Summary Sheet 1 of 3

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EBK	RECOMMENDED	APPROVED
CK.	EBK, DL	[illegible signature]

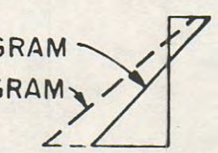
DATE: AUGUST 7, 1972 | HR-11-032

STUDY I-EQI
SUMMARY OF ARCH AND CANTILEVER STRESSES AT 31.8 SECOND INTERVAL OF DESIGN EARTHQUAKE I



LEGEND

DYNAMIC STRESS DIAGRAM
COMBINED STRESS DIAGRAM



DYNAMIC VALUE 100
COMBINED VALUE (150)

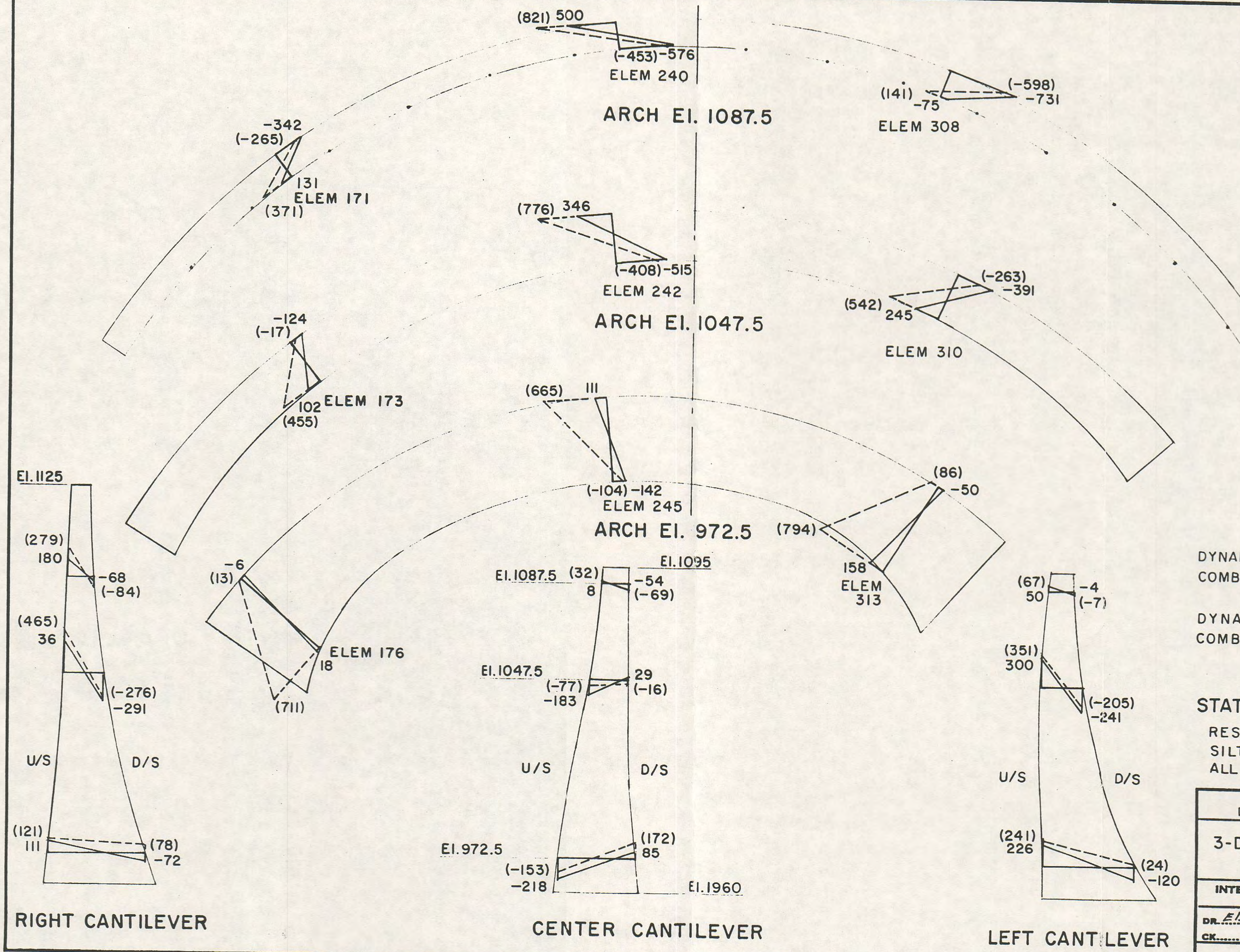
STATIC LOADING CONDITIONS

RES. W. S. EL. 1095.0
SILT ELEVATION 1040.0
ALL STRESSES IN PSI, TENSION (-)

COUNTY OF VENTURA DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM 3-D FEM STRESS ANALYSIS STUDY I-EQI Summary Sheet 2 of 3
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.
DR. EBK CK. / RECOMMENDED EBK. 72 / APPROVED [signature]
DATE: AUGUST 7, 1972 / HR-11-033

STUDY I-EQI

SUMMARY OF ARCH AND CANTILEVER STRESSES AT 32.0 SECOND INTERVAL OF DESIGN EARTHQUAKE I



LEGEND

DYNAMIC STRESS DIAGRAM

COMBINED STRESS DIAGRAM

DYNAMIC VALUE 100

COMBINED VALUE (150)

STATIC LOADING CONDITIONS

RES. W. S. EL. 1095.0

SILT ELEVATION 1040.0

ALL STRESSES IN PSI, TENSION (-)

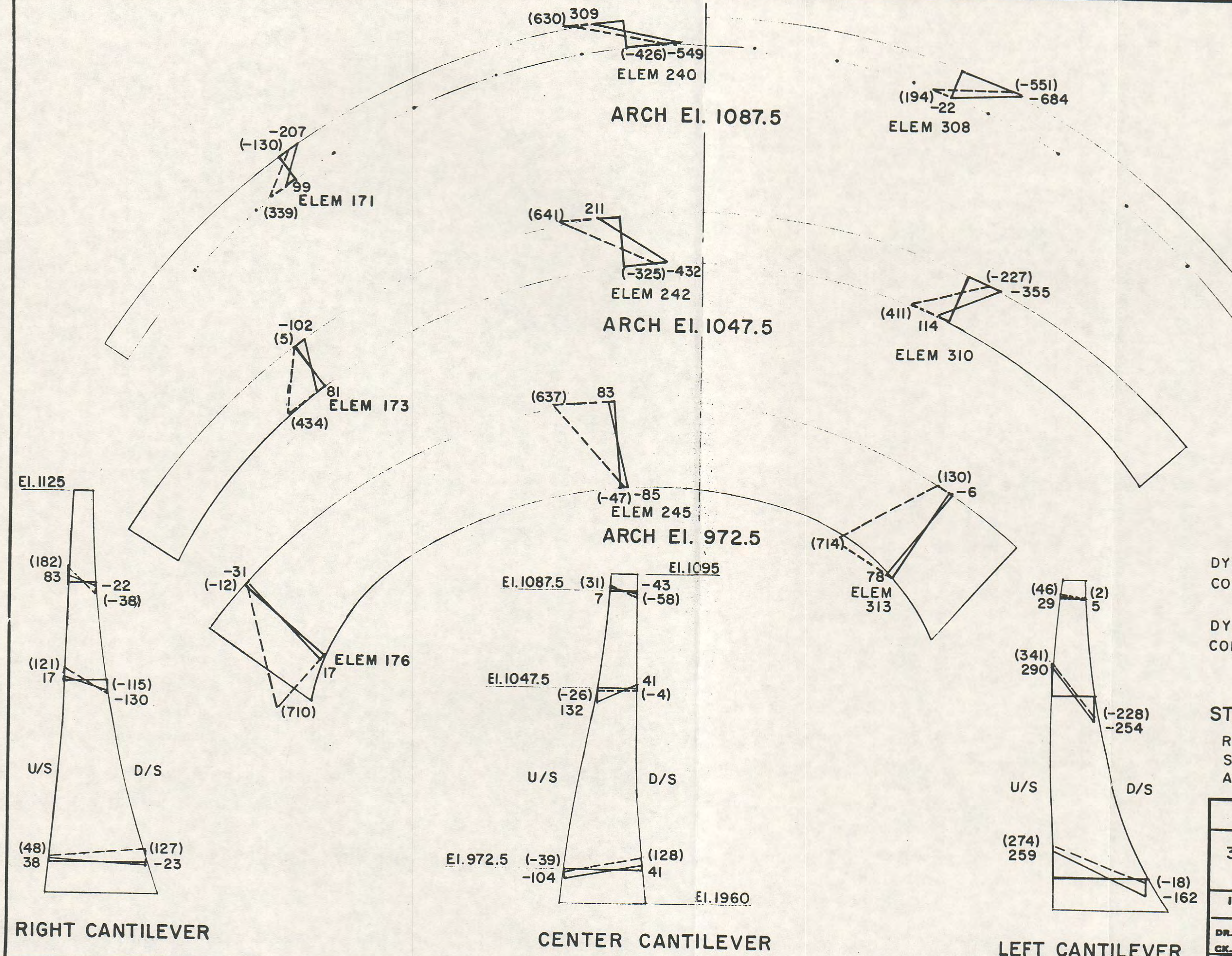
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY I-EQI
Summary Sheet 3 of 3

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EBK	RECOMMENDED	APPROVED
CK.	EBK, DL	[illegible]
DATE: AUGUST 7, 1972	HR - 11 - 034	

STUDY I-EQ 2
SUMMARY OF ARCH AND CANTILEVER STRESSES AT 2.6 SECOND INTERVAL OF DESIGN EARTHQUAKE 2



LEGEND

DYNAMIC STRESS DIAGRAM
COMBINED STRESS DIAGRAM

DYNAMIC VALUE	100
COMBINED VALUE	(150)

STATIC LOADING CONDITIONS

RES. W. S. EL.	1095.0
SILT ELEVATION	1040.0

ALL STRESSES IN PSI, TENSION (-)

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS

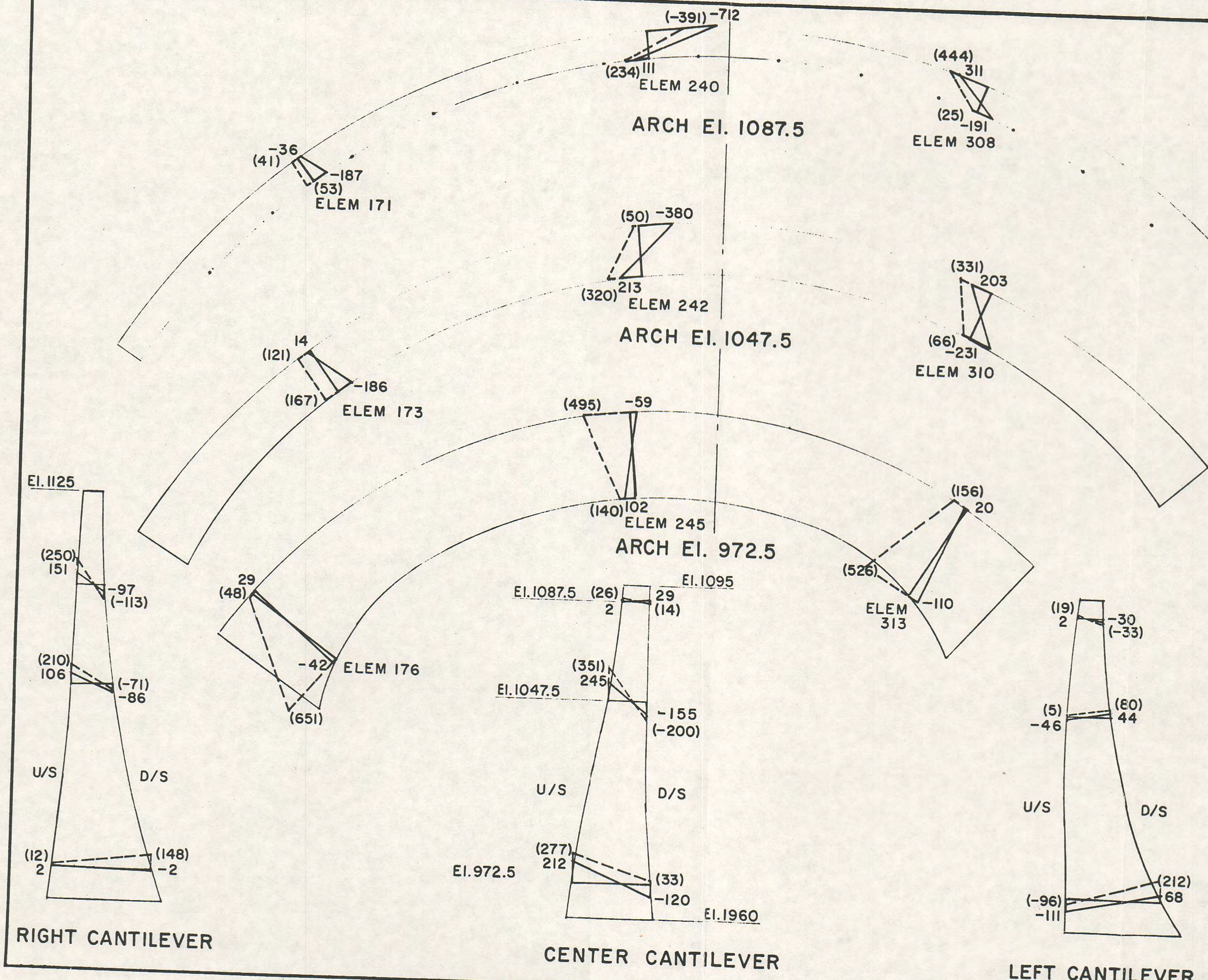
MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY I-EQ2
Summary Sheet 1 of 3

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

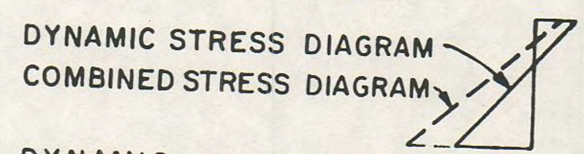
DR. EBK	RECOMMENDED	APPROVED
CK.	EBK. DZ.	[illegible]
DATE: AUGUST 7, 1972	HR-11-035	

EXHIBIT V - 11

STUDY I-EQ2
SUMMARY OF ARCH AND CANTILEVER STRESSES AT 4.3 SECOND INTERVAL OF DESIGN EARTHQUAKE 2



LEGEND



DYNAMIC VALUE 100
COMBINED VALUE (150)

STATIC LOADING CONDITIONS

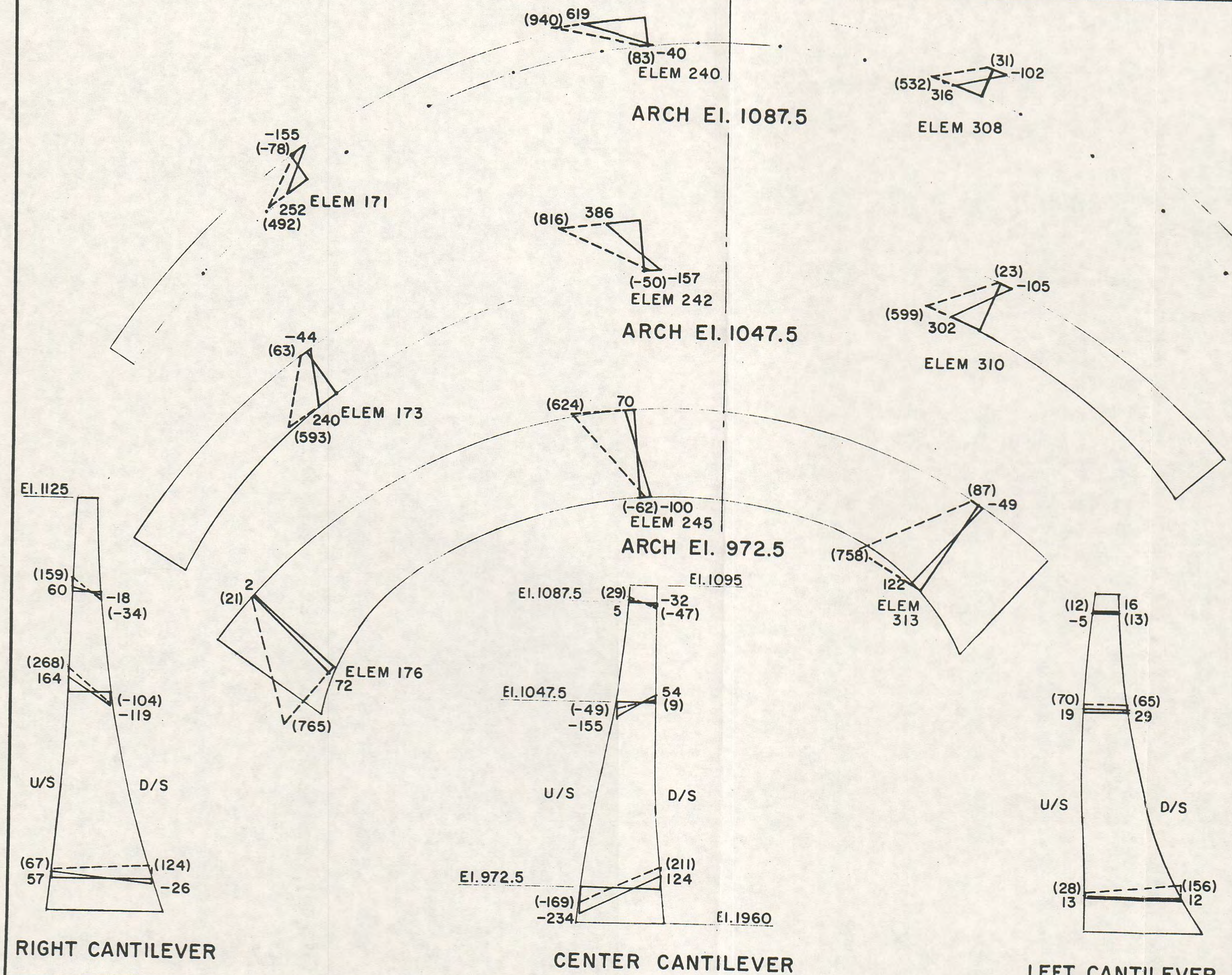
RES. W. S. EL. 1095.0
SILT ELEVATION 1040.0
ALL STRESSES IN PSI, TENSION (-)

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY I-EQ 2
Summary Sheet 2 of 3

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EBK	RECOMMENDED	APPROVED
CK.	EBK. DL	[illegible signature]
DATE: AUGUST 7, 1972	HR - 11 - 036	

STUDY I-EQ2
SUMMARY OF ARCH AND CANTILEVER STRESSES AT 9.0 SECOND INTERVAL OF DESIGN EARTHQUAKE 2



LEGEND
DYNAMIC STRESS DIAGRAM
COMBINED STRESS DIAGRAM

DYNAMIC VALUE 100
COMBINED VALUE (150)

STATIC LOADING CONDITIONS
RES. W.S. EL. 1095.0
SILT ELEVATION 1040.0
ALL STRESSES IN PSI, TENSION (-)

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
MATILIJA DAM
3-D FEM STRESS ANALYSIS
STUDY I-EQ2
Summary Sheet 3 of 3

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. EBK	RECOMMENDED E.B.K. DL	APPROVED L. J. Sarkaria
CK.		
DATE: AUGUST 7, 1972	HR - II - 037	

APPENDIXES

G. W. HOUSNER
1201 East California Blvd.
Pasadena, California 91109
2 June 1972

Mr. Eric B. Kollgaard
International Engineering Company
220 Montgomery Street
San Francisco, California 94104

Subject: Matilija Dam, Seismic Criteria

Dear Mr. Kollgaard:

This is in reply to your letter of 25 May regarding the stress investigation of Matilija Dam. The dam, I note, is approximately 25 miles from the San Andreas fault, hence, one of the seismic criteria would be a magnitude 8+ earthquake on the San Andreas. The corresponding ground shaking on firm rock at the site that I would recommend for stress analysis is the same as was recommended for Pacoima Dam, namely, the 50%g maximum horizontal ground acceleration with duration of approximately 60 seconds, reduced by the factor 0.67 (this gives 33%g max acceleration). You have a set of punch cards for this ground acceleration.

I note that both the Santa Ynez and the Santa Ana faults are within a few miles of the site. I cannot make recommendations for seismic design requirements for earthquakes originating on these faults as I do not have the requisite geological and seismological information. I would need to know the following:

a) What are the largest earthquakes likely to occur on these faults (or any other faults) in the vicinity of the site?

b) How are these faults located and oriented with respect to the site?

To provide the requisite information there should be geological investigation of the faults in the general vicinity of the site. Perhaps this has already been done. If not, you could phone me and I could give you more specific recommendations on what is needed.

Yours truly,

George W. Housner

GEORGE W. HOUSNER

G. W. HOUSNER
1201 East California Blvd.
Pasadena, California 91109

22 June 1972

Mr. E. B. Kollgaard
International Engineering Company
220 Montgomery Street
San Francisco, California 94104

Subject: Recommended accelerograms for Matilija Dam

Dear Mr. Kollgaard:

Mr. Wade of your organization has given me the following geologic information with reference to Matilija Dam:

a) The surface trace of the Santa Ynez Fault is approximately 2 miles from the site.

b) Although essentially vertical near the surface, the fault plane appears to incline toward the site with increasing depth and at considerable depth may underly the site.

c) The most recent movements on the fault are judged to have been essentially strike-slip.

d) The Santa Ynez fault extends approximately 25 miles east of the site and dies out before intersecting the San Gabriel fault.

e) The fault extends some 40 to 50 miles west of the site.

f) The 1925 Santa Barbara earthquake probably originated about 10 miles west of Santa Barbara City, perhaps on the Santa Ynez fault. It had a magnitude of 6. 3.

It is my opinion that the largest credible earthquake that might occur on the eastern portion of the Santa Ynez fault would have a magnitude in the range of 6. 5 to 7. 0. This would make it similar to the 1940 El Centro earthquake. The ground motion of the El Centro earthquake was recorded in El Centro, approximately 5 miles from the causative fault (strike-slip).

The geology of the Matilija dam site differs from that at El Centro, and the Santa Ynez fault is closer to the site than the Imperial fault is to El Centro. A conservative approach would dictate a somewhat stronger maximum credible ground shaking at Matilija than was recorded at El Centro. Accordingly, I recommend for purposes of checking the seismic safety of the Matilija dam, ground shaking 1.25 times as intense as was recorded at El Centro in 1940. This would correspond to the maximum credible ground shaking at the site, originating on the

ROY W. CARLSON
CIVIL ENGINEER
55 MARYLAND AVENUE
BERKELEY, CALIFORNIA
94707

1 September 1972

Mr. Eric Collgaard
Project Manager
International Engineering Company
220 Montgomery Street
San Francisco 94104

Dear Mr. Collgaard:

On August 2, I inspected the Matilija Dam in Ventura County for the express purpose of inspecting the instrumentation of the abutments. I was accompanied by Mr. Ram Sharma of your office.

In 1965, eight eight joint meters were installed to measure abutment yielding, four in each abutment. The scheme was to drill holes into the abutments, install a pipe in each hole which could be firmly grouted at its deep end and attached to the concrete of the dam at its exposed end. The joint meter was inserted at the dam end, in series with the pipe in such a way that if the dam were to push into the abutment the amount of the movement could be measured by the meter. All of the joint meters showed reasonable movements, fluctuating with the seasons until recently. One of the meters developed an open circuit, while another gave erratic readings.

Two of the joint meters were recovered during the 2 August inspection, and three more were recovered later and shipped to the laboratory of Carlson Instruments in Campbell, California. The inspection of the instruments showed that one had its flexible-bellows section solidified with dried mud so that its free movement was restrained. This was the one which had shown erratic readings. The one with open circuit had one conductor separated from the terminal. However, all were in excellent condition internally, it was only severe conditions externally which had caused some lack of perfect operation after seven years.

It was recommended that the five joint meters be replaced with new ones, even though they could have been reclaimed. The main reason for advising new meters was that the new ones have ceramic-insulated terminals and the old ones have some surface deterioration. But when the new meters are installed, extra care should be taken to prevent mud from access to the bellows. This can be done by wrapping with soft fabric, taped at the ends.

The frequency of reading the joint meters should vary with the conditions. During filling of the reservoir or when the reservoir is full, readings once weekly are suggested.

Very truly yours,

Roy W. Carlson