HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.

169 KINDERKAMACK ROAD, PARK RIDGE, N.J. 07656 • Phone: (201) 391-2115

WATERFORD III SES

ANALYSIS OF CRACKS AND WATER SEEPAGE

IN FOUNDATION MAT

LOUISIANA POWER & LIGHT COMPANY

REPORT NO. 8304-2

OCTOBER 12, 1983

Prepared by: AV du Bouchet
A. V. du Bouchet

Reviewed by: Ahmet Unsal
A. I. Unsal

Approved by: G A Harstead
G. A. Harstead

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1.0 Introduction

This report summarizes a study undertaken by Harstead Engineering Associates (HEA) on behalf of Louisiana Power and Light Company to evaluate the structural adequacy of the Waterford 3 Nuclear Power Island Structure (NPIS) basemat.

The following major evaluation items are addressed in this report:

a) The geometric criteria employed by Ebasco to formulate the finite element model used to evaluate the structural adequacy of the basemat;

b) The magnitudes and distribution of the loads employed by Ebasco to evaluate the structural adequacy of the basemat;

c) A benchmark comparison between an initial HEA test run (in which only the finite element used to model the basemat is revised) against the comparable Ebasco basemat computer analyses;

d) A detailed comparison between a final HEA computer analysis and the corresponding basemat shear and moment capacity.

2.0 Ebasco Basemat Computer Model

The finite element model and the corresponding loads and load combinations generated by Ebasco to perform a structural analysis of the basemat were transmitted by Ebasco to a permanent HEA file (on a computer system operated by United Information Services) on September 1, 1983.

Additional supporting documentation which describes the formulation of the geometry and the loads used to analyze the basemat is found in References 1 - 3.

3.0 Finite Element Model Geometry

3.1 Drawings and Plots

The extent of the finite element model of the basemat and additional structures modeled by Ebasco is detailed on a series of five Ebasco drawings (References 4-8), which show the plan of the basemat, as well as additional plans and elevations of floors and walls located above top of mat.

In order to confirm the modeling shown on these drawings a number of computer plots were executed and are contained in Appendix A: the plan of the basemat at elevation -41.0 ft (one plan with node points numbered, one plan with elements numbered), elevations A through D, and elevations at column lines T2, 1FH and 7FH.

3.2 Basemat Finite Element

The STARDYNE computer code used by Ebasco to perform the structural analysis of the basemat enables the use of two types of triangular plates.

As noted on page M-60 of Reference 9,

> "Two types of triangular plates are available: (1) A linear curvature compatible triangle which simulates thin plate behavior (without consideration of transverse shear effects) for non-sandwich structures (see Reference A (1)) and (2) the "Martin" element which simulates thin or thick plate behavior in a sandwich or homogeneous (solid) structure. The "Martin" plate must be used when transverse shear effects and/or transverse shear stresses are desired. A denser nodal grid mesh is desirable with this element. See Reference A(2), page M-200".

3.3 HEA Benchmark Run

In order to confirm the validity of the basemat internal shears and moments obtained by Ebasco using the thin plate (type 1) element, a benchmark run for the load combination designated as "South to North Design Basis Earthquake" was executed. See Section 4 for a detailed discussion of the loads and load combinations used in the analysis.

This HEA benchmark run contained the following two revisions to the input geometry:

a) the "Martin" plate element was used to model the basemat;

b) the local coordinates of a number of finite elements contained in the basemat were rotated to become parallel to the global coordinate system of the basemat, in order to facilitate interpretation of the output shears and moments.

Page 1-4 of the HEA calculation contained in Appendix B shows a typical basemat element, describes the formulation of the local coordinate system for that element as a function of node point order (connectivity) and defines the global coordinate system used to reference the locations of the node points.

Pages 1-6 through 1-8 of the Appendix B HEA calculation tabulate the identification numbers of the basemat elements having their element coordinate axes rotated parallel to the global coordinate axes, and the respective angles of rotation.

Page 1-5 of the Appendix B HEA calculation defines the positive sense of the membrane forces Fx and Fy, the transverse shears Fxz and Fyz, and the bending moments Mx and My with respect to the element local coordinate axes. Note that the positive sense of the transverse

shears Fxz and Fyz, as shown on page 1-5, is opposite to the definition given on page M-70 of Reference 9.

The letter and two-page attachment contained in Appendix C document the sign convention for the transverse shears used in this report.

3.4 Benchmark Shear and Moment Plots

Appendix D contains three calculation pages excerpted from the Reference 3 Ebasco calculation book.

Page E1 indicates the plan of the basemat at elevation -35.0 ft, and shows section marks A-A and B-B. Pages E4 and E7 show plots of the shear and moment in the basemat along section B-B for the load combination containing the North to South Design Basis Earthquake, for three different soil spring moduli, as well as a plot of the design envelope.

The shears and moments derived from the HEA benchmark run are superimposed on these plots, and are labeled "HEA (Variable Modulus)".

The extraction and interpretation of the shears and moments from the HEA computer run is also detailed in Appendix D. The shear and moment output for the basemat finite elements cut by section B-B are tabulated, the local coordinate system for each of these elements is defined, and either Fxz or Fyz and Mx or My (along with the appropriate sign) is plotted based on the orientation of the local coordinate system and the sign convention defined for the shears and moments.

Note that the rational procedure defined above for the selection of the shears and moments to be plotted does not accord with the procedure used by Ebasco. See pages E36 and E49 of the Reference 3 Ebasco calculation.

There is an excellent correspondence between the plot of the HEA benchmark moments and the plot of the moments (for variable spring modulus) generated by Ebasco, and this was expected.

The correspondence between the equivalent shear plots is not as good, even if the differences in the procedures used by HEA and Ebasco to select the magnitude and sign of the shear to be plotted are taken into account.

The decision was therefore made to retain the "Martin" finite element for the formal structural analysis, which incorporates additional revisions to the model geometry (see Section 3.5) and to the magnitudes and distribution of the input loads (see Section 4.1).

3.5 Simulation of Shield Building Wall

Ebasco simulated the stiffness of the Shield Building wall by a series of 3 ft wide by 40 ft deep beams spanning between (adjacent) node points 21-40.

The moment of inertia of the beam about a horizontal axis ($3 \times 40^3/12$) is 16000 ft^4. The magnitude of the moment of inertia employed in the Ebasco computer analyses was 1600 ft^4.

In order to simulate the Shield Building wall more realistically, a moment of inertia of 250,000 ft^4 was selected, equivalent to a beam 100 ft deep. Review Section A-A of the Reference 10 Ebasco drawing, which shows the cylindrical portion of the Shield Building wall to be 10 ft thick from top of mat (elevation -35.0 ft) to elevation -18.17 ft, and 3 ft thick thereafter to elevation 184.07 ft.

4.0 Loads and Load Combinations

4.1 Dead and Seismic Loads

The dead and seismic loads employed by Ebasco in their computer analyses were reviewed by HEA against the applicable Ebasco calculations (References 1-3, 11).

The total Reactor Building dead loads tabulated by Ebasco on page 39 of Reference 1 were reviewed, as shown on pages 2-1 through 2-3 of the HEA calculation contained in Appendix B.

As shown therein, the total dead weight of the Reactor Building less the weight of the concrete shielding is 108,270 k, which yields a uniformly distributed load of 6.1 ksf over the circular area having a radius of 75.5 ft to centerline of Reactor Building wall, and contained within the ring of basemat node points 21-40.

The magnitude of the distributed dead load acting over the above defined area was therefore revised from 5.3 ksf to 6.1 ksf for the formal computer analysis. An additional 39,680 k was distributed equally to node points 21-40 to account for the dead load of the Reactor Shield Building, which had not been input by Ebasco.

The Design Basis Earthquake (DBE) seismic loads used by Ebasco for the Fuel Handling Building and Auxiliary Building in their computer analyses were reviewed against the maximum accelerations tabulated on page 5 of the Reference 11 Ebasco calculation (FHB mass Nos. 29-32, AB mass Nos. 33-38, from page 2 of that calculation), and were found to be consistent.

The DBE seismic loads used by Ebasco for the Reactor Building Internal Structure in their computer analyses were compared against the base shear and moment tabulated on page 5 of the Reference 11 Ebasco calculation.

As reviewed on page 2-4 of the HEA calculation contained in Appendix B, the base shear used in the Ebasco computer analysis is approximately 70 percent of the base shear tabulated in the referenced Ebasco calculation (not considered critical), while the total moment is substantially higher. The magnitudes and distribution of the total base shear and moment as generated by Ebasco were therefore left unchanged.

Finally, the total base shear and moment used by Ebasco for the Reactor Shield Building in their computer analyses were reviewed against the base shear and moment tabulated on page 5 of the Reference 11 Ebasco calculation, and were found to be consistent, as shown on page 2-6 of the Appendix B HEA calculation.

The decision was made, however, to re-distribute the base shear and moment in accordance with simple beam theory, as shown on pages 2-7 through 2-11 of the Appendix B HEA calculation.

4.2 Load Combinations

Four load combinations were analyzed in the formal HEA computer run:

a) Normal Operating
b) DBE East West Seismic
c) DBE North to South Seismic
d) DBE South to North Seismic

These load combinations contain the component loads and corresponding load factors originally formulated by Ebasco. There were, however, revisions made to the dead and seismic component load files, as noted in Section 4.1.

5.0 HEA Computer Analysis

The formal HEA computer run executed to assess the structural adequacy of the basemat was run with the changes to the input geometry and loads previously noted in Sections 3 and 4.

The primary objective in performing this computer run was to make a direct comparison between the analysis shears and moments derived from this analysis, and the point-to-point shear and moment capacity of the basemat, which is a function of the number, size and placement of the rebar.

5.1 Basemat Shear and Moment Capacity

The reinforcing steel contained in the basemat is detailed on three Ebasco drawings (References 12-14).

As shown on the referenced Ebasco drawings, the top rebar is #11 @ 6 in each way, over the entire mat. The bottom reinforcement varies from zone to zone, both in the E-W and N-S directions. The shear reinforcement is distributed over two broad bands located on either side of the Reactor Building. The distribution of the bottom steel and shear reinforcement is detailed on pages 1-10 through 1-12 of the HEA calculation contained in Appendix B. The point-to-point shear and moment capacities of the basemat are given on pages 1-13 through 1-15 of that calculation.

5.2 Analysis Review Criterion

As detailed on pages 2-12 through 2-14 of the HEA analysis contained in Appendix B, the analysis shears for elements adjacent to basemat node points 21-40, the perimeter of the Reactor Building, are not evaluated in this Report, because the Reactor Building was modeled only in a global, rather than in a detailed, sense. That is, the

dead and seismic loads imposed on the basemat by the Reactor Building and Internal Structure were modeled, and a series of deep beams were modeled (see Section 3.5) to simulate the stiffness of the Reactor Building, but no detailed finite element model of the lower portion of the Reactor Building is contained in the structural model originally formulated by Ebasco.

This assumption is consistent with the assumption made by Ebasco. See Section II, pages 3-4 of the Reference 2 Ebasco calculation book.

5.3 Analysis Shears and Moments

There are approximately 600 basemat elements. For each element there is a corresponding line of output which tabulates the membrane forces, transverse shears and moments. In order to review this output in a systematic manner the following approach is adopted:

a) any shear force greater than 172 k/ft is tabulated, where 172 k is the lesser shear capacity of the basemat;

b) any positive moment greater than 1915 k ft/ft is tabulated, where 1915 k ft/ft is the lesser moment capacity of the top rebar;

c) any negative moment greater than 3643 k ft/ft is tabulated, where 3643 k ft/ft is the least moment capacity of the bottom steel.

This summary is performed for each of the load combinations tabulated in Section 4.2, as shown on pages 3-1 through 3-6 of the Appendix B HEA calculation. Each analysis shear and moment summarized is then evaluated with respect to the shear and moment capacity of the basemat at that location.

As shown on pages 3-1 through 3-6 of the HEA calculation, there were only four entries tabulated for moment greater than the least moment capacity of the bottom steel (elements 192-195, S to N DBE), which were less than the corresponding basemat moment capacities at those locations.

The analysis, therefore, clearly confirms the design adequacy of the basemat with respect to the internal moments generated by the imposed loads.

A total of thirty-five shears are tabulated with magnitudes in excess of the lesser shear capacity of the basemat.

Twelve of these shears act on elements located in areas having shear reinforcement and have magnitudes less than the higher shear capacity of 270 k in these areas (see page 1-13 of the Appendix B HEA calculation).

The remaining twenty-three tabulated shears are evaluated with respect to Equations 11-4 through 11-7 of ACI Standard 318-71, which permit a detailed calculation for the allowable shear stress (instead of the default magnitude specified in Section 11.4.1) for members subjected to axial compression. See pages 3-7 through 3-15 of the HEA calculation.

Of these twenty-three shears all but three satisfy the detailed code requirements: the shears acting on elements 172, 188 and 199 for the load combination which contains the S to N DBE.

The following table summarizes the analysis shears and shear capacities for these three elements.

Element Number	Analysis Shear (k)	Shear Capacity (k)
172	429	420
188	232	204
199	433	422

These magnitudes of analysis shears in excess of shear capacity can be re-evaluated with respect to a more realistic concrete compressive strength. For example, a test report appended to the Reference 16 Ebasco Concrete Masonry Specification entitled "Compressive Strengths of Cores Taken from Placement 499-19 (4-28-76) Lab. Nos. A0826-A0838" by Peabody Testing, summarizes the results

of compressive tests performed on thirteen cores taken sixty-nine days after concrete placement. The compressive strengths varied from a minimum of 4360 psi to a maximum of 5930 psi, with an average compressive strength of 4979 psi.

As the cores were tested on July 6, 1976 (sixty-nine days after date of concrete placement, as previously noted), the average compressive strength can be conservatively employed to re-evaluate the shear capacities of elements 172, 188 and 199.

The following table summarizes the results of the re-analysis.

Element Number	Analysis Shear (k)	Shear Capacity (k)
172	429	456
188	232	223
199	433	459

Only the analysis shear for element 188 exceeds the re-computed shear capacity, by about 4 percent. We judge this to be acceptable, because of the proximity of element 188 both to the Reactor Building and to an adjacent zone of shear reinforcement, and because this slight overstress is localized.

HEA, therefore, additionally confirms the design adequacy of the basemat with respect to the internal shears generated by the imposed loads.

5.4 Plots of Analysis Displacements

Plots of the vertical displacements of the basemat for the four load cases generated are presented in Appendix E.

A number of qualitative observations can be made which confirm the validity of the structural analysis, and provide additional insight into the response of the finite element model to the applied loads.

Note, for example, that the contours of constant vertical displacement are symmetric with respect to the vertical centerline of the Reactor Building for the Normal Operation load combination, while they are skewed for the load combinations which contain the E-W, N to S and S to N Design Basis Earthquakes.

Note also that the internal bending moments and shears are functions of the second and third derivatives of the displacements. Therefore, the moment and shear acting on any element whose normal is tangent to a constant displacement contour will be nominal (zero for a beam), while the moments and shears acting on any element whose normal is perpendicular to a constant displacement contour will be local maxima.

Furthermore, the more closely spaced the contours of constant displacement, the greater the magnitudes of the moments and shears.

It is clear, for example, that the most highly stressed regions within the base mat for the E-W DBE and the N to S DBE (or for the S to N DBE) occur at different locations, as would be expected.

It is finally observed that the southerly region of the basemat underlying the Reactor Auxiliary Building is loaded relatively lightly with respect to the northerly

region of the basemat, which supports the Reactor and Fuel Handling Buildings.

5.5 SRSS of Basemat Shears

To quantify the observation made in Section 5.4 that the most highly stressed regions within the basemat occur at different locations, the square root of the sum of the squares (SRSS) of the shears generated by the E-W and N to S Design Basis Earthquakes were generated, as tabulated on page 3-17 of the Appendix B HEA calculation.

The sample of elements for which this calculation was performed included all elements initially tabulated on pages 3-1 through 3-6 of the HEA calculation, for which the analysis shear exceeded the lesser shear capacity of the basemat.

Excerpted from that tabulation are the six largest SRSS values of the shears Fxz and Fyz, along with a percent increase in the magnitude of the shear, computed with respect to the larger component.

Element Numbers	(Fxz)EW	(Fxz)NS	SRSS	% INC
353	16.7	66.1	68.2	3.1
261	-16.1	45.7	48.5	6.0
150	- 8.4	36.6	37.6	2.6
172	-34.1	13.4	36.6	7.4
188	16.3	31.9	35.8	12.3
404	32.9	9.2	34.2	3.8

Element Numbers	(Fyz)EW	(Fyz)NS	SRSS	% INC
261	-46.1	-24.9	52.4	13.7
230	38.1	21.9	43.9	15.3
391	-39.8	-16.1	42.9	7.9
188	13.5	26.6	29.8	12.1
235	-15.9	-21.1	26.4	25.2
211	9.8	-16.8	19.4	15.8

Recalling that the lesser shear capacity was exceeded for all of the elements tabulated (for at least one load combination), we first note that relatively little shear capacity is employed to resist the internal shears generated by the SRSS response of the Design Basis Earthquakes.

It is finally noted that the percent increases in the SRSS shears calculated with respect to the larger components are nominal.

HEA therefore concludes that a load combination containing the SRSS of the Design Basis Earthquakes will not significantly alter any of the analysis shears and moments, and therefore need not be evaluated.

6.0 Discussion of Results

The basis of our evaluation of the basemat was the finite element analysis of the model and loads as modified by HEA. All the elements are assumed to be homogeneous uncracked concrete as far as stiffness relationships are concerned. For each plate element of the computer model, moments, shears and axial forces are calculated for each plate element. In addition, displacements at each node are calculated for each load combination of interest. Vertical displacements have been plotted and are presented in Appendix E.

The controlling load combinations are:

a) Normal Operating

b) DBE East West Seismic

c) DBE North to South Seismic

d) DBE South to North Seismic

The East-West component of the earthquake was taken in only one direction due to the symmetry of the structure about a north-south axis. On the other hand, the North-South component of the earthquake was taken from North to South and from South to North. This was required due to the lack of symmetry about an east-west axis.

The moments and shears due to the EW and N to S (or S to N) Design Basis Earthquakes are greatest at different locations and, therefore, an SRSS combination would not significantly increase any of the design values. Therefore, this combination was not required for the evaluation of the basemat.

While the perimeter walls and a few major walls were represented in the finite element model, many walls were ignored. This is conservative because such walls would assist the basemat in carrying loads and in redistributing

any moments and shears which might be substantially greater than in surrounding areas.

A review of the results indicates the response of the structural system to the load combinations. Under normal load conditions the maximum displacements occur under the Reactor Building. This is because this region has the softest soil springs and the greatest dead loads. It is very evident for the normal load conditions that the establishment of variable soil springs will result in greater calculated shear and moments as discussed in the Reference 17 HEA Report.

We are in complete agreement with the conservative selection of the variable soil springs based upon the expected soil subgrade modulii.

The seismic loading combinations in general did not control as much as the normal loading conditions due to the fact that the dead load is increased by a factor of 1.5 and 1.1 in normal and seismic load combinations, respectively. Inasmuch as the Waterford 3 plant is in a low seismic site, the dead load is the major load.

Since the basemat is located well below grade, active soil pressures are acting on the perimeter walls during normal conditions. During seismic events passive soil pressures will develop on the face of the perimeter wall which is being pushed into the soil by the earthquake forces.

With the resultant moments, shears and axial forces, the adequacy of the various sections of the mat was investigated to the requirements of ACI 318-71. In general, the stresses due to bending moments are well below the allowables for all of the controlling load combinations. In no case was it found that the reinforcing steel stresses

were close to allowables; in fact, the imposed bending moments were usually only a fraction of the section capacity.

While a similar situation exists for shear, there are a few cases where the applied shear is very close to structural shear capacity as developed by ACI 318-71.

However, the ACI capacity reduction factor, Ø, for calculation of shear capacities is 0.85. This factor is applied during design due to uncertainties in random phenomena which could cause loss in strength. These phenomena cover items such as isolated drops in concrete strength, reinforcing steel, and errors in the placement of reinforcing bars. The basemat was constructed under controlled conditions, and evidence of strengths indicates that values are well above the minimum. For example, the age of the basemat concrete is now almost seven years, so that the concrete strength is considerably above the 28-day strength which was conservatively used in the calculations. Since the mat is twelve feet thick, errors in the placement of steel would have little effect on the section geometry. In fact, the job records indicate no problems in placement of reinforcing bars.

A very strong case, indeed, could be made for justifying substantially increased section shear strengths. However, without taking advantage of this, the results indicate that the basemat has adequate shear strength for the imposed load combinations.

The basemat is very structurally redundant and is very capable of carrying loads well in excess of the applied loading combinations. This is due to the fact that if local capacities were exceeded, there would be a redistribution, rather than a progressive failure.

In fact, there is no structural reason why the basemat could not consist of individual building mats or even spread footings and still function very well. The use of a continuous monolithic mat combined with the very low soil bearing pressures provides the structurally most conservative foundation solution.

Although the top and bottom reinforcing appears quite substantial it is quite low when compared to the concrete area. From a structural point of view, this is very beneficial since the basemat will possess greater ductility. Furthermore, the concrete compressive stresses due to bending will be quite low, probably up to only about 10% of the 28-day compressive strength of the concrete.

6.1 Comparison of Results with Previous Analyses and Design

Due to certain refinements, more complete and consistent results were obtained which were used in this evaluation, as discussed in Section 5.

It must be kept in mind that the top and bottom reinforcing bars were conservatively selected on the basis of primarily manual computations and, therefore, the quantities of top and bottom reinforcing are well in excess of ACI 318-71 requirements. This also applies to shear except for a few isolated elements in which the applied shear did approximate the capacity as required by ACI 318-71.

6.2 Effect of Cracking on Structural Response

Cracking of the type evidenced at the top of the Waterford 3 basemat is expected in reinforced concrete construction, and is assumed in establishing the structural capacity requirements in the ACI 318 Code. The fundamental reason that cracking has little influence on the structural capacity is, of course, that concrete is

not assumed to have tensile strength, the tensile forces being carried by the embedded reinforcing bars. The reinforcing bars are bonded to concrete by adhesion as well as the mechanical interlock of the deformed reinforcing bars themselves. The loss of bond across a very narrow crack is insignificant. If a crack exists where compressive forces will develop, the crack will tend to close up so that the compressive forces are transmitted in bearing in local regions across the crack. Therefore, while the crack may still be in evidence it will have closed sufficiently to transfer the compressive forces.

The effect of the closing of a narrow crack on the total compressive strain of the concrete is negligible for widely spaced cracks.

The completed analyses were performed using currently accepted methods for the design of reinforced concrete structures in nuclear power plants. For a complicated model such as was developed for the Waterford 3 basemat and superstructure, this was a formidable task in itself. Nevertheless, consideration was given to refinements such as incorporation of cracking. While this would have introduced complex nonlinearities into the analysis, its effect upon the structure would have been negligible. Therefore, such exotic approaches were discarded. The general low state of stresses calculated offers further justification for this assessment.

6.3 Conclusions and Recommendations

The results of the finite element analyses indicate that the basemat will be functional for all the required load combinations for the design life of plant. The minor amount of cracking is insignificant in affecting structural response and has no effect whatsoever on the structural integrity of the mat.

The fundamental purpose of a foundation is simply to transmit the dead weight and any other applied loads to competent soil. Of all the foundation systems that are possible from a structural point of view, the continuous monolithic basemat provides the most conservative system. Furthermore, this type of foundation tends to smooth out locally high soil bearing pressures. Such large construction tends to be most susceptible to cracking from benign causes such as shrinkage, differential soil settlement, and temperature changes. However, the evidence of such cracks should not be a cause for concern any more than deliberately constructed expansion joints would have been a cause for concern. The cracks are expected and should not deflect consideration of the inherent superior structural capability of multiple redundant structures such as the Waterford 3 basemat.

The present soil bearing pressures are a small fraction of the soil capability and are in fact less than the pressures existing in the soil prior to the start of excavation and construction. Therefore, the functional requirements of the design have been met.

The major loading on the basemat is, of course, already present, namely, the structural dead load and equipment load. The observations of the basemat covered in the Reference 17 HEA Report provide additional corroboration not usually encountered when a structure is under design.

In closing, the information presented herein as well as in the Reference 17 HEA Report are the result of an examination of all aspects of the design of the basemat. While the seepage of water from the cracks precipitated

the investigation, all aspects of the design were considered, not just that which could be associated with the cracks and seepage. It is our conclusion that the design of the mat is extremely conservative, which, under the circumstances in which the design was carried out, we consider prudent and justifiable. Therefore, we see no need for any remedial measures or the necessity of additional analyses.

REFERENCES

1. Ebasco calculation book entitled "Waterford 3 Base Mat Design - Book 1", dated 06/02/72.
2. Ebasco calculation book entitled "Waterford 3 Base Mat Design - Book 2", dated 07/29/74.
3. Ebasco calculation book entitled "Common Mat Finite Element Analysis, June '81 - In Response to NRC Audit", dated 06/15/81.
4. Ebasco drawing (untitled, undated) showing keyplan of basemat and elevations A-D.
5. Ebasco drawing (untitled, undated) showing partial plans at elevations -4.0ft, +21.0 ft, +46.0 ft, +91.08 ft, and elevations at column lines 1FH, 3FH, 4FH, 5FH, 7FH, T2 and V.
6. Ebasco drawing (untitled, undated) showing a plan at elevation +21.0 ft and twenty-six (26) partial elevations (untitled).
7. Ebasco drawing (untitled, undated) showing plans at elevations -4.0 ft, +35.0 ft, +46.0 ft, +69.0 ft.
8. Ebasco drawing entitled "Common Mat (Finite Element Model)" undated, showing the plan of the basemat at (centerline) elevation -41.0 ft.
9. STARDYNE User Information Manual, dated September, 1980.
10. Ebasco drawing entitled Reactor Building Structural Layout, LOU-1564-G-509, Revision 2, dated 10/28/77.

11. Ebasco calculation entitled Steel Containment Stability, OFS NO 1352.063, DEPT NO 650, Rev. 1, dated 07/28/83.

12. Ebasco drawing entitled Common Foundation Structure Reinforcing Sh. 1, LOU-1564-G-500S01, Rev. 9, dated 12/12/78.

13. Ebasco drawing entitled Common Foundation Structure Reinforcing Sh. 2, LOU-1564-G-500S02, Rev. 2, dated 01/20/75.

14. Ebasco drawing entitled Common Foundation Structure Reinforcing Sh. 3, LOU-1564-G-500S03, Rev. 3, dated 05/09/75.

15. Ebasco calculation entitled Comm Fdn Mat - Moment Capacity, OFS No. 5234.014, Dept. No. 650, Sheet E55, dated 05/11/81.

16. Ebasco Concrete Masonry Specification, PID No. LOU-1564.472, Revision 5, dated 03/11/75.

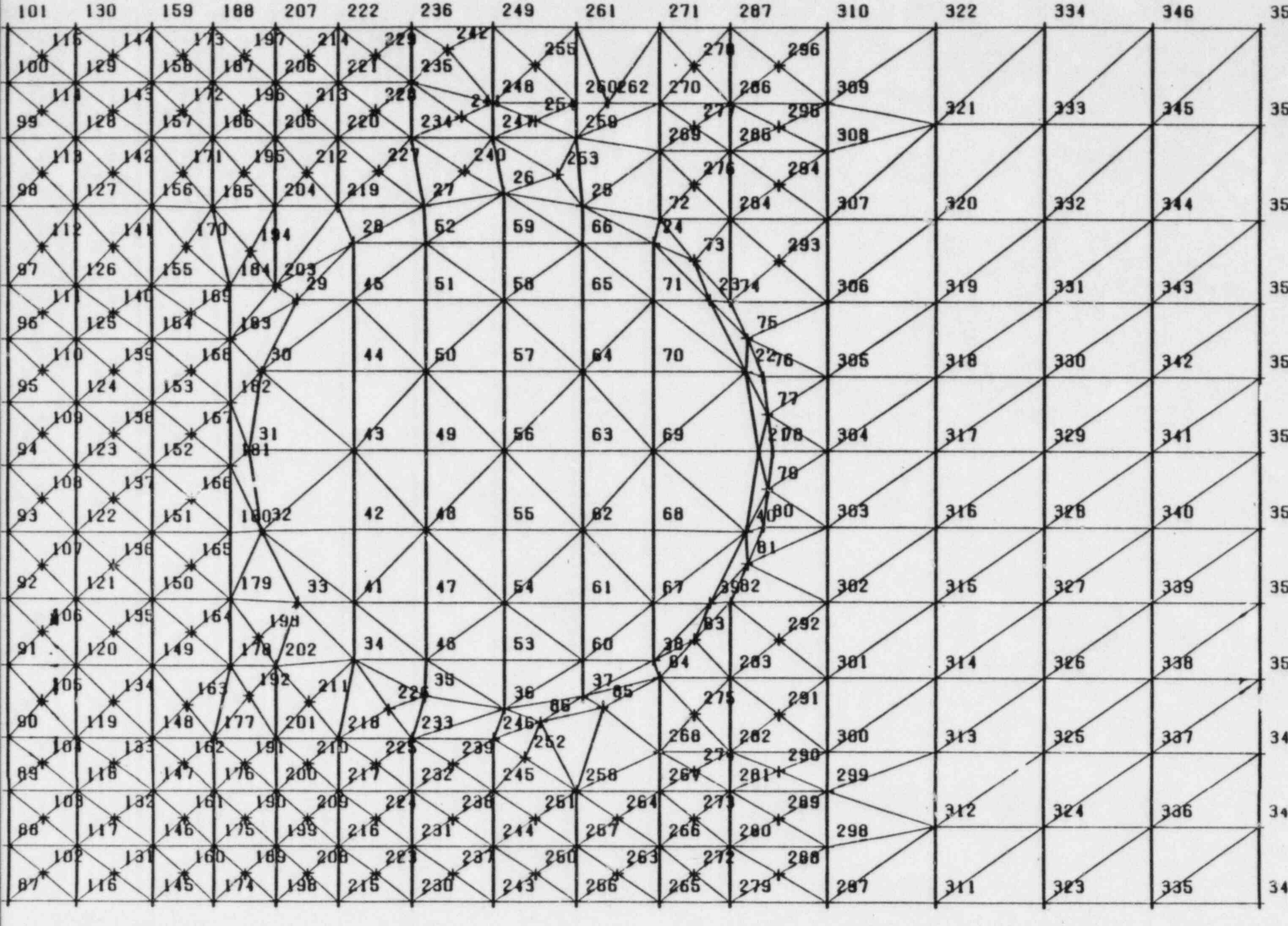
17. HEA Report entitled Waterford III SES Analysis of Cracks and Water Seepage in Foundation Report No. 8304-1, dated 09/19/83.

APPENDIX A

Plots of Plans

and Elevations

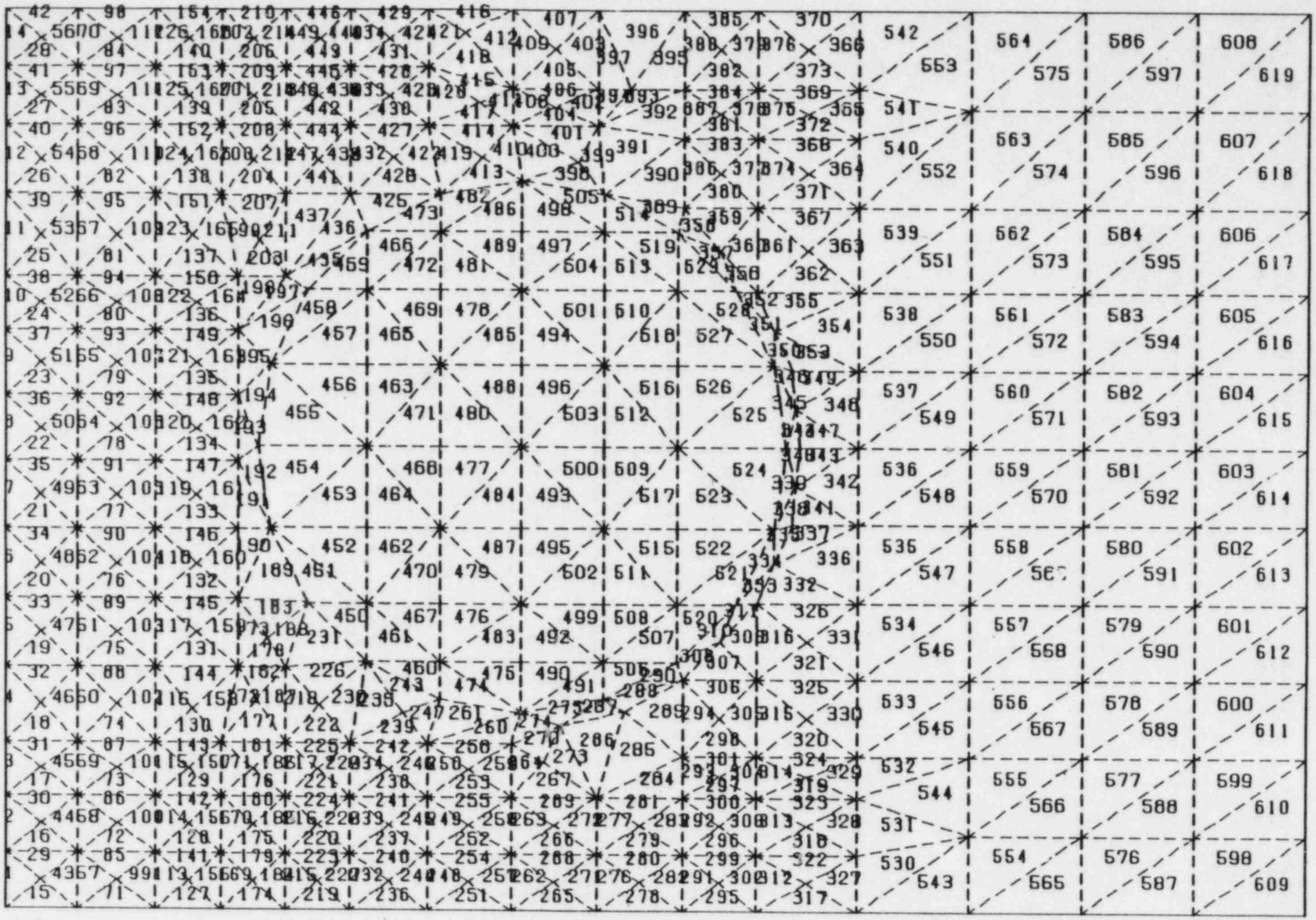
WATERFORD 3 BASEMAT PLAN AT EL -41.0



STARDYNE FINITE ELEMENT MODEL PROJECTION ON X1-X2 PLANE CASE NO. 1

Figure A-1

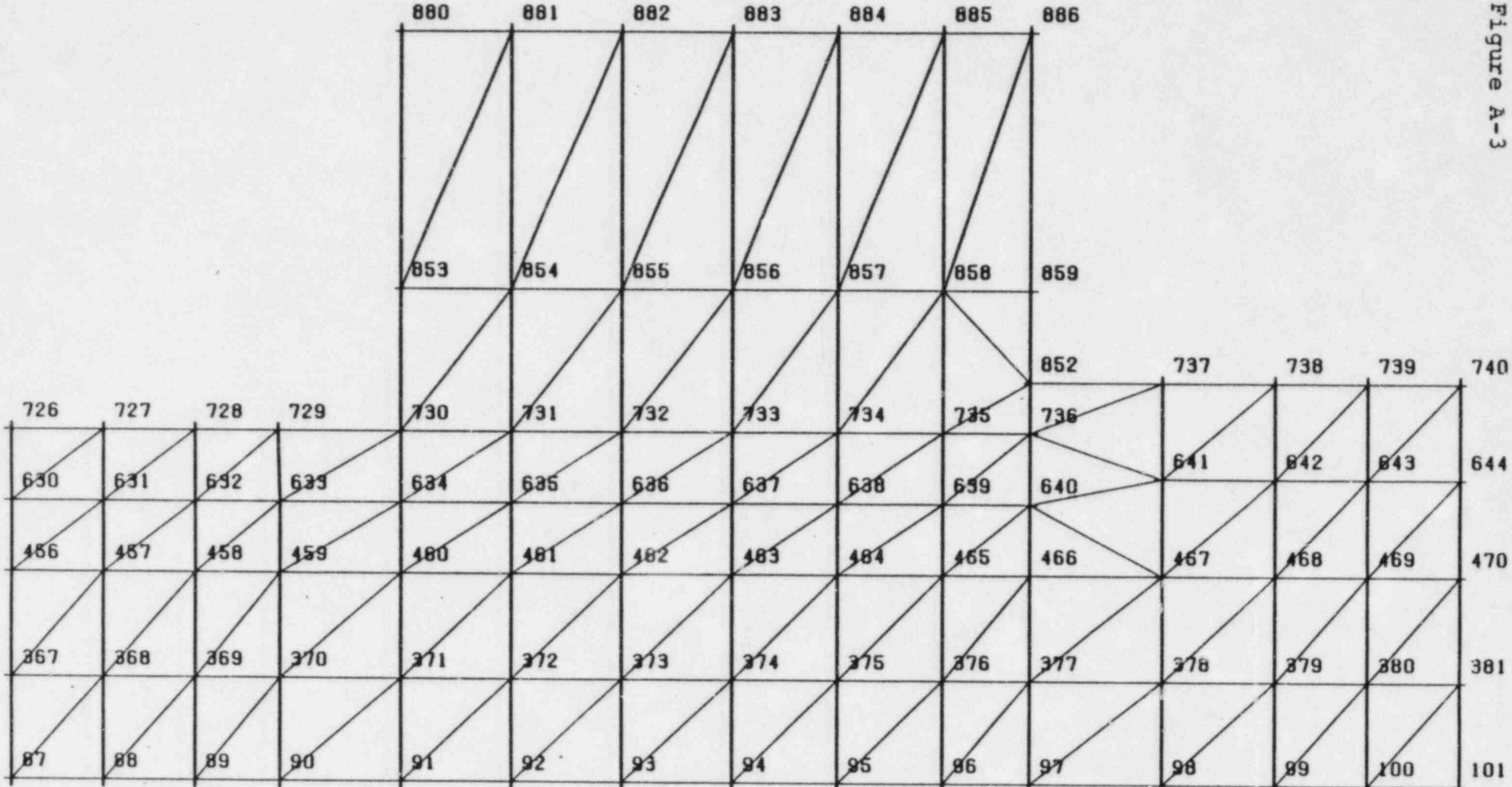
WATERFORD 3 BASEMAT PLAN AT EL -41.0



PLOT OF GEOMETRY PLOT

Figure A-2

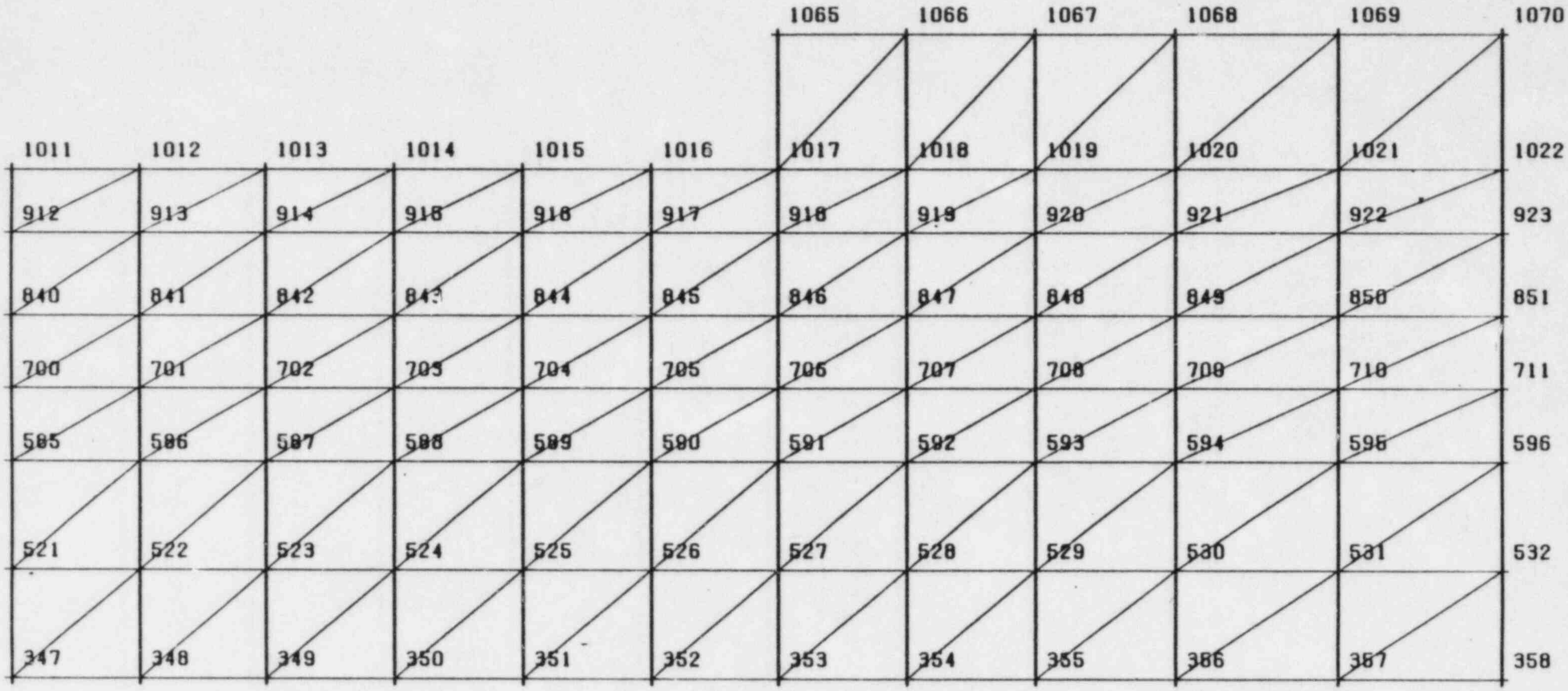
WATERFORD 3 ELEVATION A-A (X=0.0)



STARDYNE FINITE ELEMENT MODEL PROJECTION ON X2-X3 PLANE CASE NO. 1

Figure A-3

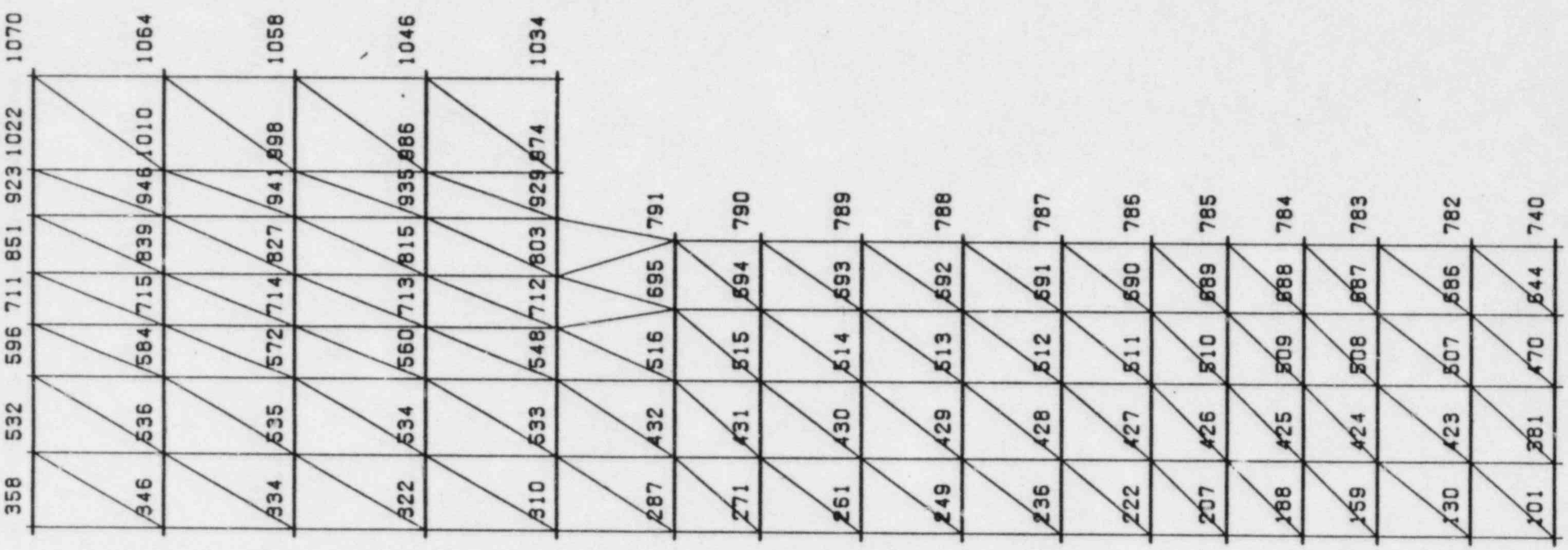
WATERFORD 3 ELEVATION B-B (X=371.5)



STARDYNE FINITE ELEMENT MODEL PROJECTION ON X2-X3 PLANE CASE NO. 2

Figure A-4

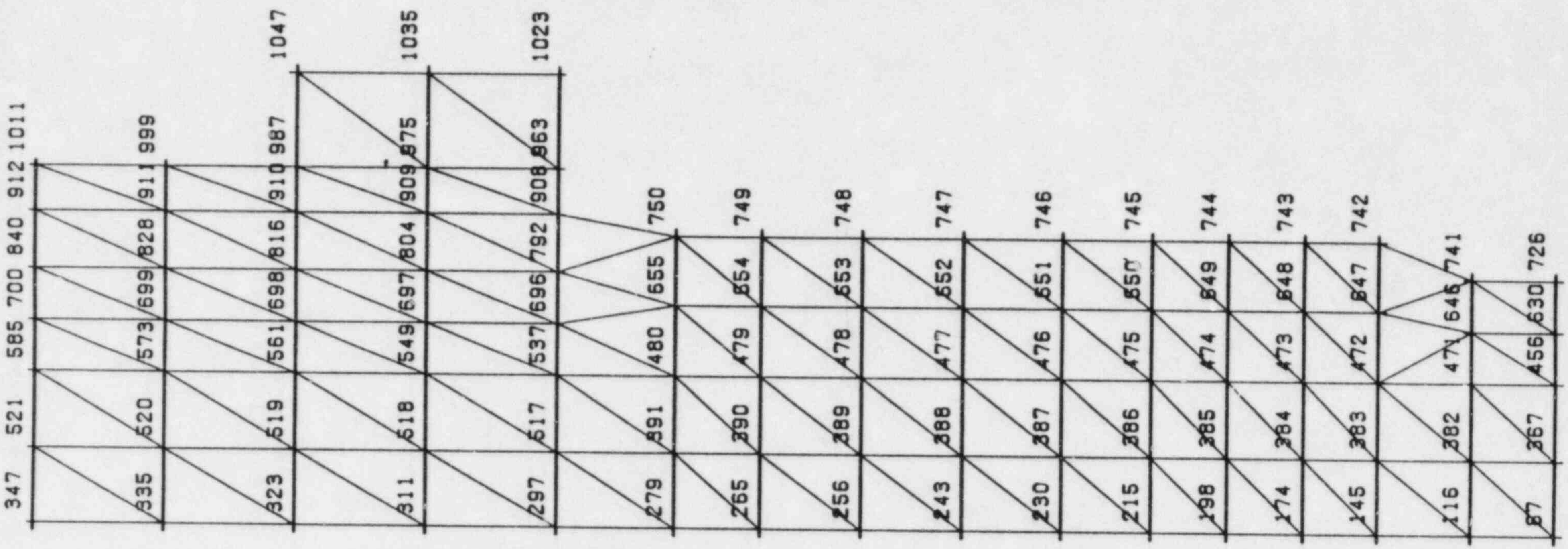
WATERFORD 3 ELEVATION C-C (Y=+256.0)



STARDYNE FINITE ELEMENT MODEL PROJECTION ON X3-X1 PLANE CASE NO. 3

Figure A-5

WATERFORD 3 ELEVATION D-D (Y=0.0)

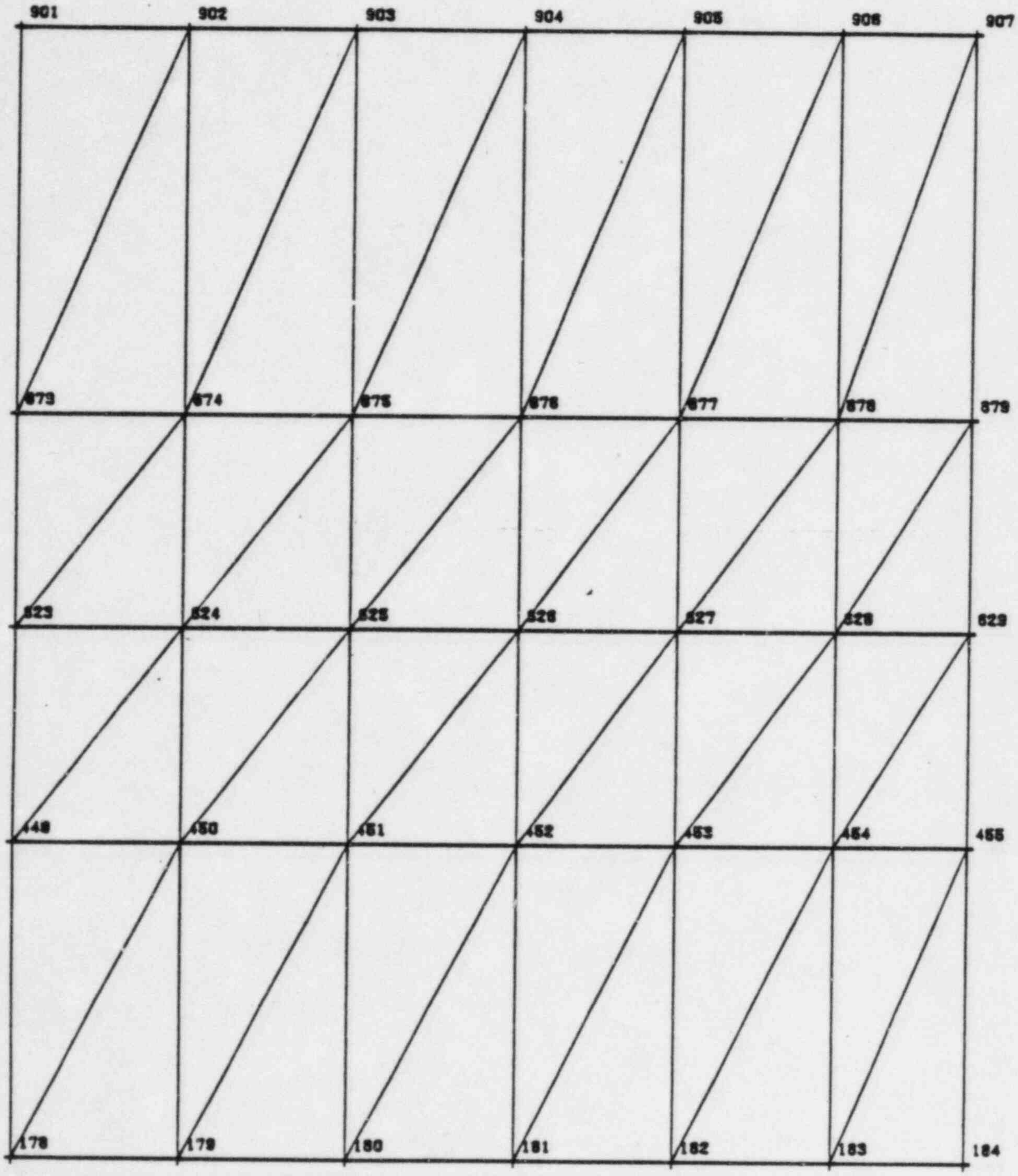


STARDYNE FINITE ELEMENT MODEL PROJECTION ON X3-X1 PLANE CASE NO. 4

Figure A-6

Figure A-7

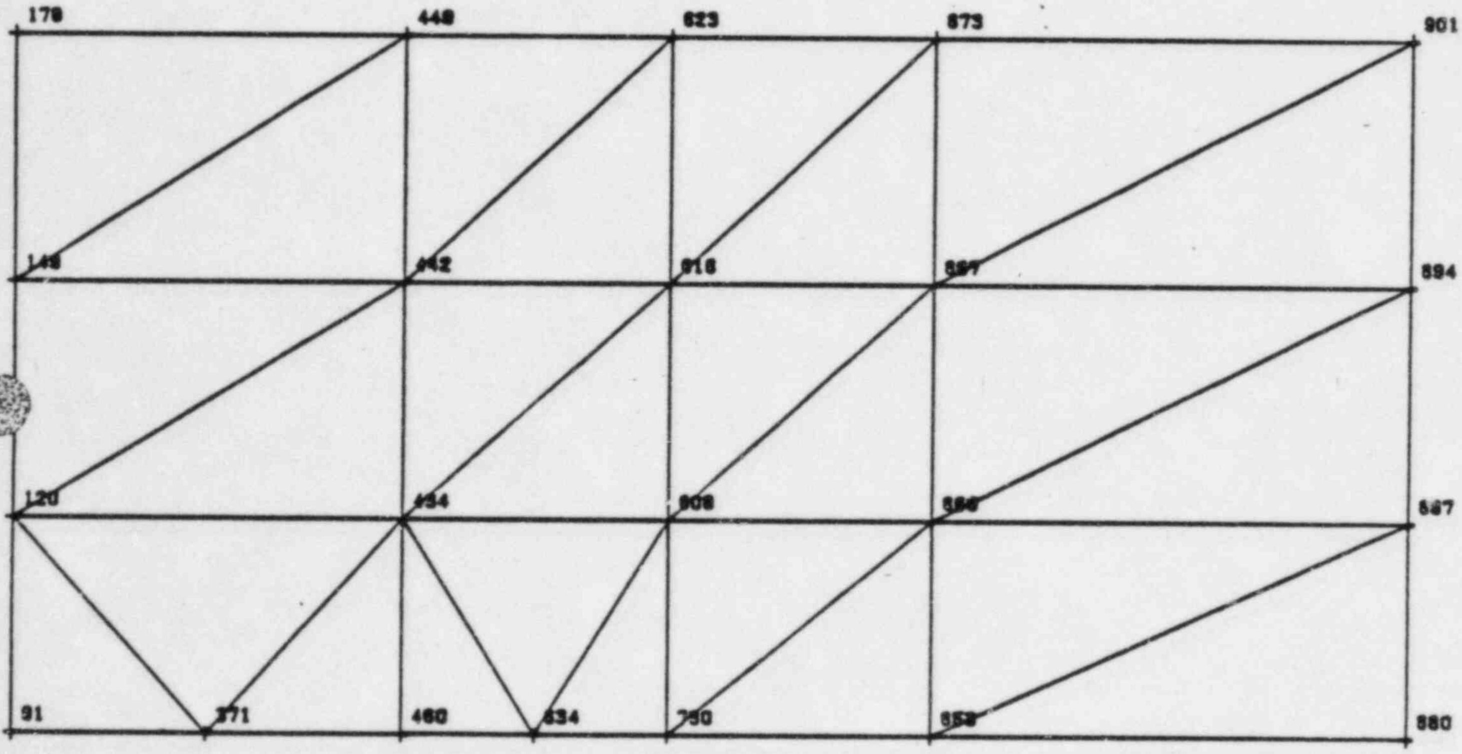
WATERFORD 3 ELEVATION AT COL LINE T2



STARDYNE FINITE ELEMENT MODEL PROJECTION ON X2-X3 PLANE CASE NO. 2

Figure A-8

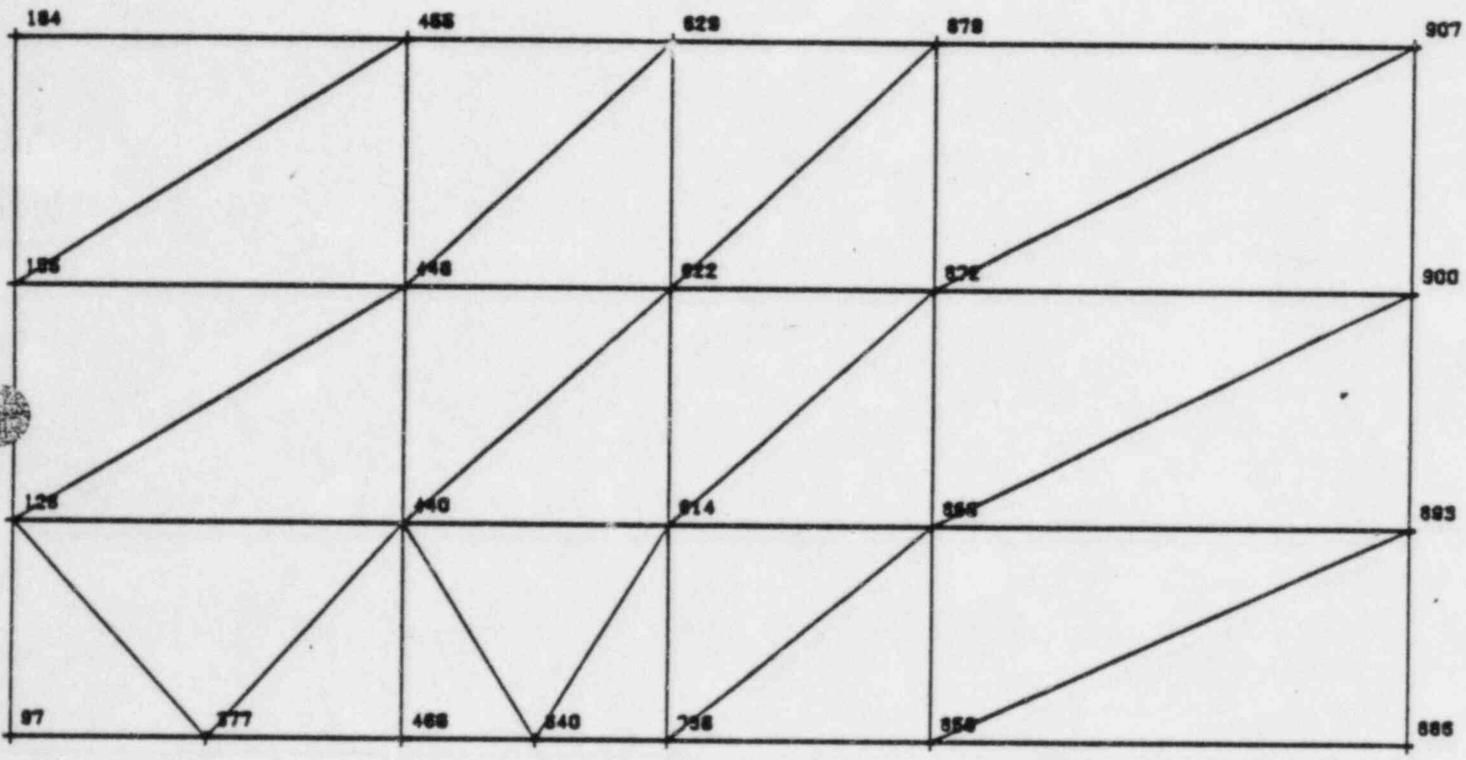
WATERFORD 3 ELEVATION AT COL LINE 1FH



STARDYNE FINITE ELEMENT MODEL PROJECTION ON X3-X1 PLANE CASE NO. 1

Figure A-9

WATERFORD 3 ELEVATION AT COL LINE 7FH



STARDYNE FINITE ELEMENT MODEL PROJECTION ON X3-X1 PLANE CASE NO. 3

APPENDIX B

HEA Calculation

INTRODUCTION

THE PURPOSE OF THIS CALCULATION IS TO PERFORM AN EVALUATION OF THE INTERNAL FORCES AND MOMENTS GENERATED IN THE NPIS BASEMAT DUE TO THE FOLLOWING LOAD COMBINATIONS:

1. NORMAL OPERATING
2. S-N DBE EQ (VARIABLE SPRING)
3. N-S DBE EQ (VARIABLE SPRING)
4. E-W DBE EQ (VARIABLE SPRING)

THE FINITE ELEMENT MODEL OF THE BASEMAT AND ASSOCIATED STRUCTURE, AS WELL AS THE LOADS CONTAINED IN THE ABOVE-DEFINED LOAD COMBINATIONS, WERE PROVIDED BY EBASCO.

TWO CHANGES WERE MADE TO THE CODING OF THE GEOMETRY:

1. THE 'MARTIN' TRIANGULAR ELEMENT WAS EMPLOYED IN THE STRUCTURAL ANALYSES PERFORMED BY HEA.

2. THE LOCAL COORDINATES OF A NUMBER OF FINITE ELEMENTS CONTAINED IN THE BASEMAT WERE ROTATED TO BECOME PARALLEL TO THE GLOBAL COORDINATE SYSTEM OF THE BASEMAT, TO FACILITATE INTERPRETATION OF THE OUTPUT SHEARS AND MOMENTS.

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CLIENT LP&L	
SUBJECT BASEMAT INTERNAL FORCES	CHCKD. BY GH DATE 10/3/83

THE 'MARTIN' TRIANGULAR ELEMENT CAN BE USED TO ADVANTAGE WHEN A THICK PLATE IS BEING ANALYZED OR WHEN TRANSVERSE SHEAR STRESSES ARE REQUIRED IN THE OUTPUT. REFERENCE 'STARDYNE USER INFORMATION MANUAL', SEP/79, PP. M-60, 70 FOR A DEFINITION OF THE 'MARTIN' ELEMENT.

EVALUATION OF INTERNAL SHEARS AND MOMENTS

IN ORDER TO PERFORM A PROPER EVALUATION OF THE BASEMAT SHEARS AND MOMENTS GENERATED BY THE COMPUTER ANALYSIS, IT IS NECESSARY TO STIPULATE:

1. THE DEFINITIONS OF THE LOCAL COORDINATE AXES AND THE ORIENTATIONS AND SIGN CONVENTIONS OF THE LOADS FOR THE TRIANGULAR ELEMENTS USED TO MODEL THE BASEMAT.

2. THE PATTERN OF REBAR (NUMBER AND ORIENTATION) CONTAINED NEAR THE TOP AND BOTTOM OF THE MAT, AS WELL AS THE SHEAR REINFORCEMENT.

3. THE CORRESPONDING MOMENT AND SHEAR CAPACITY OF THE BASEMAT REINFORCEMENT.

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PROJECT	W3
CLIENT	LP&L
SUBJECT	REVIEW OF MODEL & LOADING
PREP. BY GH	DATE 9/19/83
CHCKD. BY AU	DATE 9/19/83

COMPARISON CHECKS ON LOADING
FOR COMPUTER ANALYSIS

TOTAL REACTOR BLDG REF: p39 Ebasco Book 1

	WT
INTERNAL STR	85,500k
MAJOR EQUIP + STL CONT.	18,700k
CONC. SHIELDING	54,340k
REFUELING POOL	4050k
TOTAL	162,610
LESS CONC. SHIELD	54,340
	108270

AREA INSIDE NODES 21 - 40

$$\pi\ (75.5)^2 = 17907.8$$

$$P = 6.05 \approx 6.1 \text{ KSF}$$

CHANGE EBASCO VALUE OF 5.3 KSF TO 6.1 KSF

EBASCO INPUT DID NOT ACCOUNT FOR WT. OF CONCRETE SHIELD

NOTE: VALUE OF SHIELD USED BY HEA IN COMPUTER RUN 38527 + 9692 = 48219k COMPARES WELL WITH 54,340k

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SUBJECT	REVIEW OF MODEL & LOADING	PREP. BY GH	DATE 9/19/83
		CHCKD. BY AU	DATE 9/19/83

TOTAL STRUCTURAL DEAD LOAD

FROM EBASCO COMPUTER RUN

TOTAL DL 513,000^{K}

FROM EBASCO BOOK #1 TOTAL DL 586,081
P. 41 & P. 45 (DL NOT INCL EQUIP)

ADDITIONAL LOAD ADDED BY HEA:

UNIFORM LOAD WITHIN REACTOR BLDG
17907.8 (6.1 - 5.3) 14326

REACTOR SHIELD 1984 (20) 39,680^{K}

TOTAL LOAD ADDED BY HEA 54006

NEW TOTAL IN COMPUTER
513,000 + 54006 567,006^{K}

CORRECTED VALUE OF TOTAL DEAD LOAD OF STRUCTURES COMPARES VERY WELL WITH PREVIOUSLY CALCULATED VALUES

567,006 ≈ 586,081^{K}

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PROJECT W3 CLIENT LP&L SUBJECT REVIEW OF MODEL & LOADING	PREP. BY GH DATE 9/19/83 CHCKD. BY AU DATE 9/19/83

CONC. CYLINDER

Ebasco Calc 3 of 7
Cont Stability

Vol = 256877 C.F

W = 38527^k

W/NODE = 1926^k

CONC DOME

Vol = 64611

W = 9692^k

W/NODE = 485^k

TOTAL ADDED FOR EA. NODE (21-40

W/NODE = 2411^k

SUBTRACT BEAM 3'x 40'

W/NODE = 3(40)(23.7)(.15) = 427

NET ADDITIONAL LOAD NODES 21-40

W/NODE = 1984^k

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PROJECT W3
CLIENT LP&L
SUBJECT REVIEW OF MODEL & LOADING

PREP. BY GH DATE 9/19/83
CHCKD. BY AJ DATE 9/19/83

SEISMIC LOADS - FHB & AUX BLDG

Ebasco Computer Run
FUEL HANDLING BLDG & AUXILIARY BLDG.
FOR OBE Horiz. Acc = 0.075g
TO BE APPLIED TO STRUCTURAL ELEMENTS MODELED.
TO COMPENSATE FOR INCREASING ACCELERATION WITH HEIGHT, THE WEIGHT DENSITY WAS VARIED

FROM 0.1999 TO 0.2748 g
SEE COMPUTER OUTPUT

EFFECTIVE G DISTRIBUTION

$$.075\left(\frac{.1999}{.15}\right) = 0.10\ g$$

$$.075\left(\frac{.2748}{.15}\right) = 0.1374$$

FOR DBE EBASCO DOUBLED THESE VALUE

	COMPUTER VALUES FOR STATIC RUN		DYNAMIC ANALYSIS (FROM EBASCO CONTAINMENT VESSEL STABILITY CALC,
	FHB	AUX BLDG	
MIN.	0.22	0.22	0.2
MAX.	0.27	0.28	0.275

EBASCO PROCEDURE FOR SEISMIC REPRESENTATION
- FOR FUEL HANDLING BLDG & AUX. BLDG IS CONSISTENT WITH G LOADS

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PROJECT W3	
CLIENT LP&L	PREP. BY GH DATE 9/14/83
SUBJECT REVIEW OF MODEL & LOADING	CHCKD. BY AdB DATE 10/07/83

SEISMIC LOADS - REACTOR BLDG. INTERNAL STRUCTURE

TOTAL MOMENT & SHEAR EQUALLY DISTRIBUTED OVER 14 NODES.

COMPARISON OF EBASCO VALUES

		SHEAR K/1	MOMENT 'K/1
STATIC MAT ANALYSIS	OBE	3699	243,620
	DBE	7398 K	487240
DYNAMIC ANALYSIS (CONT. STABILITY ANALYSIS)			
	DBE	10108.	360649

THE BASE SHEAR HAS LITTLE EFFECT ON MAT. HOWEVER THE TOTAL MOMENT USED IN THE STATIC MAT ANALYSIS IS SIGNIFICANTLY GREATER THAN THAT INDICATED FROM DYNAMIC ANALYSIS. AS CONSERVATISM KEEP ALL VALUES AS IS FOR STATIC MAT ANALYSIS

MOMENTS & SHEARS EQUALLY DISTRIBUTED TO 14 NODE

48 - 50
54 - 58
62 - 64
68 - 70

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	C- 21 - 6 (SUBJ. SUBDIV. SHEET)
PROJECT W3	
CLIENT LP&L	PREP. BY GH DATE 9/20/83
SUBJECT REVIEW OF MODEL & LOADING	CHCKD. BY AGB DATE 10/07/83

SEISMIC LOADS - CONCRETE SHIELD STRUCTURE

EBASCO DISTRIBUTED TOTAL MOMENT AND SHEAR EQUALLY TO 20 NODES NODES 21 - 40

		SHEAR K/1	MOMENT 1K/1
STATIC MAT ANALYSIS	OBE	6796	1,060,280
	DBE	13592	2,120,560
DYNAMIC ANALYSIS (CONT. STABILITY ANAL.)	DBE	18894	2,607,574

DISCREPANCY IS PROBABLY DUE TO LOWER CONCRETE FROM INTERNAL STRUCTURE BEING INCLUDED WITH SHIELD BLDG

	SHIELD STR	INTERNAL STR	TOTAL
STATIC MAT ANALYSIS	2120,560	487,240	2,607,800
DYNAMIC	2607,574	360,649	2,968,223

THIS INDICATION OF CONSISTENCY LENDS CREDENCE TO VALUES USED BY EBASCO AS FAR AS TOTAL MOMENTS AND SHEARS

THE EQUAL DISTRIBUTION OF SHEAR AND MOMENTS TO NODES IS REASONABLE FOR THE INTERNAL STRUCTURES

REACTOR CONCRETE SHIELD STRUCTURE

RATHER THAN EQUAL DISTRIBUTION, A DISTRIBUTION CONSISTENT WITH THAT OF CYLINDER WAS ADOPTED BY HEA.

THE STRUCTURAL EFFECT OF THE CONCRETE SHIELD WAS ACCOUNTED FOR BY EBASCO BY BEAMS CONNECTING NODES 21- 40.

WHILE THE BEAM WAS DESCRIBED AS 3'-0" WIDE × 40'-0" DEEP THE COMPUTER INPUT LISTED THE $I = 1600 \text{ FT}^4$ WHICH WAS 10% OF $I_{ACTUAL} = 16{,}000 \text{ FT}^4$

IN CORRECTING THIS SITUATION, HEA SELECTED BEAMS WITH A DEPTH OF 100 FT TO SIMULATE THE STIFFNESS OF 200 FT TALL CYLINDER

$$I = \frac{bd^3}{12} = \frac{3(100)^3}{12} = 250{,}000 \text{ FT}^4$$

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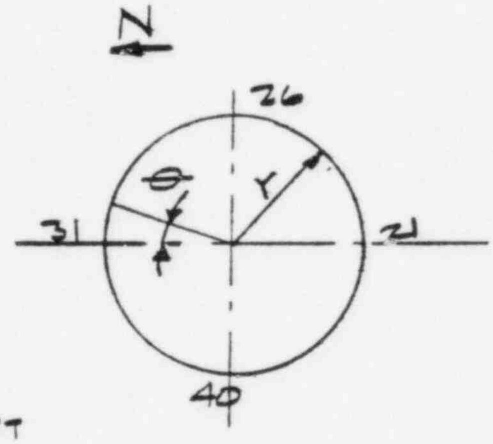
PROJECT W3 CLIENT LP&L SUBJECT REVIEW OF MODEL & LOADING	PREP. BY GH DATE 9/20/83 CHCKD. BY AcB DATE 10/07/83

MOMENT FROM REACTOR SHIELD STRUCTURE RESULTS IN MEMBRANE MERIDIONAL FORCES IN CYLINDER WHICH CAN THEN BE SUMMED AND APPLIED TO NODES 21-40

$r = 75.5$

$$N_\phi = \frac{M}{I} \sin\theta \, r$$



20 SEGMENTS

$$\phi_{SEG} = \frac{360}{20} = 18°$$

$L_{SEGMENT} = 23.7$ FT

$$I_{\substack{NODES \\ 21-40}} = 2\left\{(75.5)^2\left[1 + 2\cos^2 18 + 2\cos^2 36 + 2\cos^2 54 + 2\cos^2 72\right]\right\}$$

$$= 2\,(75.5)^2(5.0) = 57003 \text{ NODE} \cdot FT^2$$

$$N_{NODE} = \frac{M\,(75.5)}{57003} \sin\theta$$

N-S NODE	N_{NODE}
21	M/755
22, 40, 30, 32	M/994
23, 39, 29, 37	M/933
24, 38, 28, 34	M/1284.5
25, 37, 27, 35	M/2443.2

HEA

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C- 2 - 2 - 9
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT REVIEW OF MODEL & LOADING

PREP. BY GH DATE 9/21/83
CHCKD. BY A&E DATE 10/07/83

CHECK - SUMMATION OF COUPLES

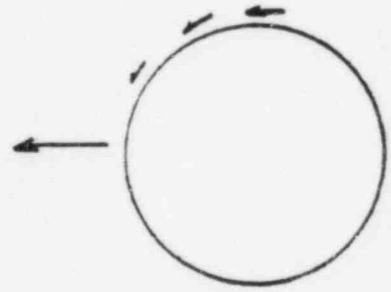
$$M = \frac{M(151)}{755}\left[1 + 2\sin^2 72 + 2\sin^2 54 + 2\sin^2 36 + 2\sin^2 18\right]$$

$$= \frac{M\ (151)(5.0)}{755} = M \qquad \underline{\text{CHECK}}$$

DISTRIBUTION OF SHEAR

SHEAR FLOW

$$\tau = \frac{V_{TOT}}{\pi d} \cos\theta$$



SHEAR @ EA. NODE

$$v = L \frac{V}{\pi r} \cos\theta$$

$$= 23.7 \frac{V}{\pi(75.5)} \cos\theta$$

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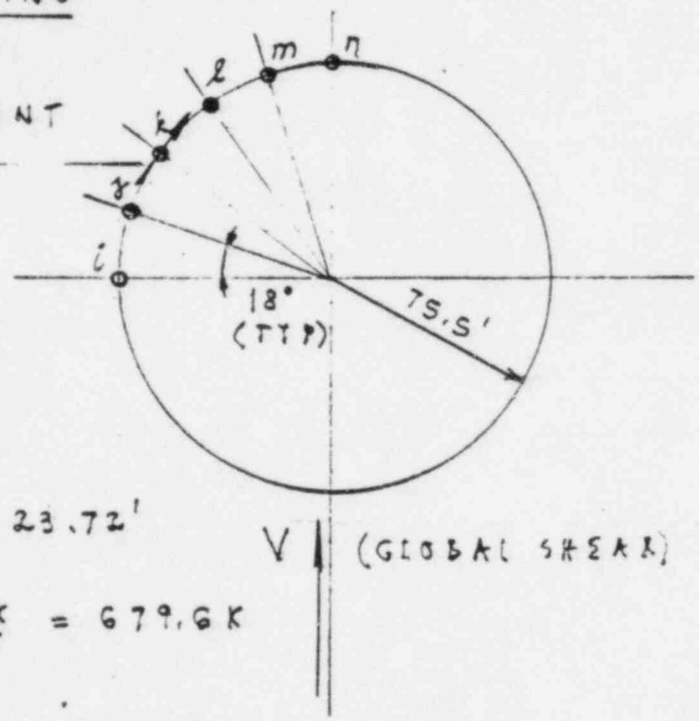
PROJECT N3	PREP. BY AGB DATE 10/06/83
CLIENT LP&L	CHCKD. BY GH DATE 10/7/83
SUBJECT REVIEW OF MODEL & LOADING	

DISTRIBUTION OF SHEAR AT BASE OF REACTOR SHIELD BUILDING



$\frac{1}{4} \times \frac{2\pi \times 75.5}{5} = 23.72'$

$\frac{23.72 \times 6796\,K}{\pi \times 75.5} = 679.6\,K$

			* COMP ∥ TO V:	* COMP ⊥ TO V:
V_i =	679.6 x COS 0° =	679.6 K	679.6	0
V_j =	679.6 x COS 18° =	646.3 K	614.7	199.7
V_k =	679.6 x COS 36° =	549.8 K	444.8	323.2
V_l =	679.6 x COS 54° =	399.5 K	234.8	323.2
V_m =	679.6 x COS 72° =	210.0 K	64.9	199.7
V_n =	679.6 x COS 90° =	0.		

* SPLITTING THESE INTO COMPONENTS:

PARALLEL TO V, $V_i \cos\theta$

PERPENDICULAR TO V, $V_i \sin\theta$

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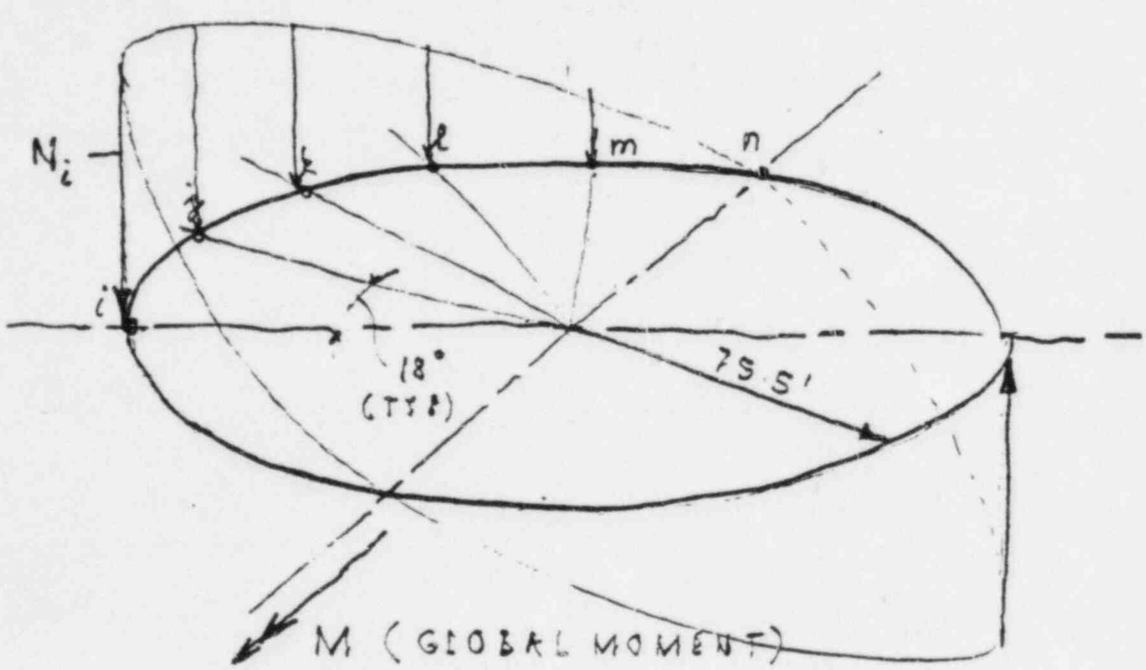
	C- 2 - 2 - 11
	SUBJ. SUBDIV. SHEET
PROJECT W3	PREP. BY AGE DATE 9.26.83
CLIENT LP&L	
SUBJECT REVIEW OF MODEL & LOADING	CHCKD. BY GH DATE 10/7/83

DISTRIBUTION OF MOMENT AT BASE OF REACTOR SHIELD BUILDING



$$N_n = \frac{\overset{1404}{1.06 \times 10^6 \times 75.5}}{57003} \quad \sin 0^\circ = 0$$

$$N_m = 1404 \times \sin 18^\circ = 434\ k$$

$$N_l = 1404 \times \sin 36^\circ = 825\ k$$

$$N_k = 1404 \times \sin 54^\circ = 1136\ k$$

$$N_j = 1404 \times \sin 72^\circ = 1335\ k$$

$$N_i = 1404 \times \sin 90^\circ = 1404\ k$$

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C- 2 - 2 - 12	
SUBJ. SUBDIV. SHEET	
PREP. BY AGE	DATE 09.27.83
CHCKD. BY GH	DATE 10/5/83

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

COMPARISON OF ANALYSIS FORCES AND MOMENTS AND DESIGN ALLOWABLES

REVIEW CRITERIA

THERE ARE APPROX. 600 FINITE ELEMENTS CONTAINED IN THE BASEMAT, WITH AS MANY CORRESPONDING LINES OF OUTPUT. IN ORDER TO REVIEW THIS OUTPUT IN A SYSTEMATIC MANNER, THE FOLLOWING APPROACH IS ADOPTED. FOR A REVIEW OF THE SHEARS FXZ AND FYZ, ANY SHEAR FORCE GREATER THAN THE LESSER SHEAR ALLOWABLE OF 172K IS TABULATED.

FOR A REVIEW OF THE MOMENTS MX AND MY, FIRST RECALL THAT POSITIVE MOMENT GENERATES TENSION IN THE TOP STEEL, WHILE NEGATIVE MOMENT GENERATES TENSION IN THE BOTTOM STEEL.

THEREFORE, ANY POSITIVE MOMENT GREATER THAN 1915 IK/I, THE LESSER MOMENT CAPACITY OF THE TOP STEEL, IS TABULATED, WHILE ANY NEGATIVE MOMENT GREATER THAN 3643 IK/I, THE LEAST MOMENT CAPACITY OF THE BOTTOM STEEL, IS TABULATED.

BASED UPON THE LOCATIONS OF THE FINITE ELEMENTS ON WHICH THESE SHEARS AND MOMENTS ACT, A DIRECT COMPARISON IS THEN MADE BETWEEN THE ANALYSIS SHEARS AND MOMENTS AND THE CORRESPONDING SHEAR AND MOMENT CAPACITY OF THE BASEMAT AT THESE LOCATIONS.

NOTE THAT THE FINITE ELEMENT MODEL AS GENERATED BY EBASCO MODELS THE REACTOR BUILDING IN A GLOBAL SENSE ONLY. A REVIEW OF THE EBASCO

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PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

COMPARISON OF ANALYSIS FORCES AND MOMENTS AND DESIGN ALLOWABLES, CON.

REVIEW CRITERIA

CALCULATION ENTITLED 'COMMON FDN MAT', DATED 07/29/74, CLEARLY ESTABLISHES THIS MODELLING PROCEDURE. SECTION II, 'DESCRIPTION OF THE MODEL', PP. 3-4, NOTES THAT THE REACTOR SHIELD WALL WAS MODELLED BY TWENTY RIGID BEAMS, ON TOP OF WHICH WERE PLACED TWENTY 3 FT. BY 40 FT. DEEP TIE BEAMS. IT IS ALSO NOTED THAT THE WEIGHT OF THE INTERNAL STRUCTURE OF THE REACTOR BUILDING IS EVENLY DISTRIBUTED TO THE BASEMAT ELEMENTS, BUT THAT NO MODEL OF THE INTERNAL STRUCTURE HAD BEEN FORMULATED.

SECTION III, 'LOADING INPUT', PP. 9-10, COMPUTES A CONCENTRATED SHEAR AND MOMENT (TO BE APPLIED TO NODE POINTS 21-40) TO ACCOUNT FOR THE EFFECTS OF EARTHQUAKE ON THE SHIELD BUILDING, AS WELL AS CONCENTRATED SHEARS AND MOMENTS (TO BE APPLIED TO BASEMAT NODE POINTS WITHIN THE SHIELD BUILDING) TO ACCOUNT FOR THE EFFECT OF EARTHQUAKE ON THE INTERNAL STRUCTURE.

THIS MODELLING PROCEDURE WILL YIELD VALID SHEARS AND MOMENTS INDUCED BY THE ABOVE CONCENTRATED LOADS EVERYWHERE IN THE BASEMAT BUT IN THE IMMEDIATE VICINITY OF THE SHIELD BUILDING. THE FOLLOWING CRITERIA ARE THEREFORE EMPLOYED FOR THE ANALYTIC SHEARS AND MOMENTS ACTING ON FINITE ELEMENTS LOCATED NEAR THE SHEAR BUILDING.

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PROJECT W3
CLIENT LP&L
SUBJECT BASE MAT INTERNAL FORCES

PREP. BY GH DATE 10/5/83
CHCKD. BY AdB DATE 10/5/83

NORMAL OPERATION $C = 1.5(D+L'') + 1.8(L+S_1) + 1.05543$

ELEM	FXZ	FYZ	MX	MY	COMMENTS	FX	FY
333	188	-257	-613	-2104	NA		
334	174	193			NA		
335		-238			NA		
337	-173	22	-613	-459	= 172 OK	-187	-130
338		-252			NA		
339		206			NA		
340		-236			NA		
344		-236			NA		
345		206			NA		
346		-254			NA		
350	25	-239	-736	-2207	NA	-135	-185
352	-200	-263			NA		
356		-232			NA		
357		193			NA		
358		-232			NA		
398		195			NA		
399		-316			NA		
400		190			NA		
406	-189				<270 OK		
425		182			NA		
436		181			NA		
351	-167	193			NA		
389		196			NA		

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PROJ. NO. 8304			
C- 2 - 3 - 3			
SUBJ.	SUBDIV.	SHEET	
PREP. BY G H	DATE 10/5/83		
CHCKD. BY AdB	DATE 10/5/83		

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

DBE E W SEISMIC $C = 1.1(D+L^{*}) + 1.0(E1+B+S2)$

ELEM	FXZ	FYZ	MX	MY	COMMENTS	FX	FY
98	-257				< 270 OK		
154	-212				< 270 OK		
172	-270				< 270 OK		
194	-181				NA		
195		-192			NA		
196	-173				NA		
197	-225				NA		
310	417				NA		
334	434				NA		
339	517				NA		
340		-191			NA		
344		-187			NA		
345	433	175			NA		
346		261			NA		
350		-227			NA		
351	176	+186			NA		
352	-250	-303			NA		
353	190	13	-248	-450	> 172 *	-94	-187
356		-240			NA		
357	341	202			NA		
358		259			NA		
389		219			NA		
391		-192			< 270 OK		
398	347	236			NA		
399	148	-392			NA		
400	159	206			NA		
404	367	-29	-451	-1276	> 270 OK SEE P. 11	-108	-362
406	-389	110	-527	-1674	> 270 OK SEE P. 11	-105	-367
415	356	-2	-1197	-1804	> 270 OK SEE P. 12	-186	-321
					* OK BY INSPECTION		

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C- 2 - 3 - 4
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

PREP. BY GH DATE 10/5/83
CHCKD. BY A&B DATE 10/5/83

DBE EW SEISMIC

ELEM	FXZ	FYZ	MX	MY	COMMENTS	FX	FY
430		192			<270 OK ✓		
435	-387	177			NA ✓		
458	215	228			NA ✓		

HEA HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 8304
C- 2 - 3 - 5
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

PREP. BY GH DATE 10/5/83
CHCKD. BY AdB DATE 10/5/83

SOUTH TO NORTH SEISMIC DBE

ELEM	FXZ	FYZ	MX	MY	COMMENTS	FX	FY
134	-210	19	-642	-436	>172 OK SEE P.13	-237	-65
136	-192	-18	-589	-549	>172 OK SEE P.13	-237	-55
146	179	55	-371	-261	>172 OK SEE P.13	-252	-74
150	272	105	-596	-1165	>172 OK SEE P.14	-209	-65
172	429	127	-354	-1348	>270 SAT OK 420 ≈ 429 (P.13)	-118	-334
188	-126	-232	-2105	-1808	>172 204K CAP < 232K (P.14)	-319	-100
190	40	194	-909	-3540	>172 NA	-72	-289
191		-181			NA		
192	-26	242	-924	-4025	NA-SHEAR < 6382-MOM OK		
193		-264		-4330	NA-SHEAR < 6382-MOM OK		
194		193		-3799	NA-SHEAR < 6382-MOM OK		
195		-192		-3703	NA-SHEAR < 6382-MOM OK		
211	276	36	-1166	-1485	>270 *	-119	-331
235	-12	182	-744	-1309	<270 OK		
243	-204	252	-606	-2061	<270 NA	-262	-116
261		221			<270 OK		
274		-173			NA		
275	-455	186			NA		
310	-516				NA		
357	433				NA		
398	290				NA		
413	247				NA		
425		259			NA		
435	326	187			NA		
436		245			NA		
474	219				NA		
482	-221				NA		
199	-433	73	-905	-1562	422K CAP. $\lesssim$ 433 K (P.15)	-134	-346
230		-183			< 270 OK		
288	-315	129	-339	-648	NA	-122	-97
					*OK BY INSPECTION		

HEA HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 8304
C- 2 - 3 - 6
SUBJ. SUBDIV. SHEET

PROJECT WB
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

PREP. BY GH DATE 10/5/83
CHCKD. BY AdB DATE 10/5/83

NORTH TO SOUTH SEISMIC DBE

ELEM	FXZ	FYZ	MX	MY	COMMENTS	FX	FY
172	276	95	-173	-694	>272 *	-140	-188
275	414	211			NA		
287	-160	-214			NA		
288	180	172			NA		
290		260			NA		
308		286			NA		
310	590	185			NA		
311		-284			NA		
333	186	-302			NA		
334	411	225			NA		
335		-307			NA		
337	-258	12			NA		
338		-324			NA		
339		251			NA		
340		-315			NA		
343	-212	-5			NA		
344	5	-315			NA		
345		251			NA		
346		-324			NA		
347	210	-5			NA		
350		-308			NA		
351	-414	226			NA		
352	-202	-310			NA		
353	265	14			NA		
356		-288			NA		
357	-430	191			NA		
358	102	-270			NA		
389		236			NA		
399	150	-339			NA		

* OK BY INSPECT.

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT W3	PROJ. NO. 8304
CLIENT LP&L	C- 2 - 3 - 7 (SUBJ. SUBDIV. SHEET)
SUBJECT BASEMAT INTERNAL FORCES	PREP. BY GH DATE 10/6/83
	CHCKD. BY AGB DATE 10/7/83

NORMAL

ELEM 199

$$V_{MAX} = \sqrt{397^2 + 75^2} = 404\ ^K/_{FT}$$

$P_W = 0.01$

$P_W = \frac{4(4)}{12(133)} = 0.01$ $\tan\phi = \frac{75}{404} \sim 1.0$

E-W Rebar $M = 897$ $\frac{4h-d}{8} = \frac{4(144)-133}{(12)(8)} = 4.61$

$F = -268$

ACI 318-77 (11-7) $M_m = M_u - N_u(4.61) = 897 - 268(4.61) = 0$

(11-6) $V_c = \left(1.9(63.2) + 2500(.01)\frac{404(133)}{0}\right) 12(133) = >>$

(11-8) $V_c = 3.5\sqrt{f'_c}(12)(133)\sqrt{1 + \frac{268}{.5(12)(133)}} = 408$

V_{REBAR} 98

$\phi = 0.85$

$\therefore$ 98 + $\phi \times 408 = 445 > 404\ ^K/_{FT}$ OK

HEA HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8304
	C- 2 - 3 - 9 (SUBJ. SUBDIV. SHEET)
PROJECT W3	PREP. BY GH DATE 10/6/83
CLIENT LP&L	CHCKD. BY AdB DATE 10/7/83
SUBJECT BASEMAT INTERNAL FORCES	

NORMAL

ELEM 172

CHECK EQ 11-7:

$$M_m = M_u - N_u \frac{(4h - d)}{8}$$

$$M_m = 332 \text{ ftK} - \frac{250 (4 \times 144 - 133)}{12 \times 8}$$

$$= -,$$

$\therefore$ USE EQ-11-8:

ACI 318 77

$$(11\text{-}8) \quad V_c = \overbrace{3.5\sqrt{f'_c}\,(12)(133)}^{353.3^K}\sqrt{1 + \frac{250}{.500(12)(133)}} = 405\ ^K/_{FT}$$

V_{REBAR} 98

$$\therefore 98 + \phi(405) = 442\ ^K/_{FT} > 395\ ^K/_{FT} \quad \underline{OK}$$

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	W3	PROJ. NO.	8304
CLIENT	LP&L	C- 2 - 3 - 10	SUBJ. SUBDIV. SHEET
SUBJECT	BASEMAT INTERNAL FORCES	PREP. BY GH	DATE 10-6-83
		CHCKD. BY AdB	DATE 10/6/83

NORMAL

ELEM 295 (cont'd)

(11-8) $V_c = 353.3\sqrt{1 + \frac{76}{.5(12)(133)}} = 370\ ^{k}/_{1}$

$\phi(370) = 314 > 187$ OK

ELEM 188

N-S REBAR: Σ #11@6, #18@12 $\Sigma A_s = 4 \times 1.56 + 4.00 = 10.24\ in^2$

$V_u = 204\ k/1$

$M_u = 1519\ 1k/1$

$N_u = 234\ 1k/1$

$\frac{V_u d}{M_u} = \frac{204 \times (133/12)}{1519} = 1.5 > 1.0$

$\therefore$ USE (11-7): $M_m = 1519 - 234\ \frac{\overset{4.61}{(4 \times 144 - 133)}}{12 \times 8}$

$= 439\ 1k/1$

THEN BY (11-6): $V_c = \left\{ \left(1.9\sqrt{4000} + 2500 \times \overset{\rho}{\frac{10.24}{b_w d}} \cdot \frac{204 \times (133/12)}{439} \right) \right\} \frac{\overset{b_w}{12} \times \overset{d}{133}}{1000}$

$V_c = 192k + 132k = 324k \leq (11-8)$:

(11-8): $353.3\sqrt{1 + \frac{234}{.5(12)133}} = 402k$ OK

$\therefore \phi \times 324k = 275k > 204k$ OK

HEA **HARSTEAD ENGINEERING ASSOCIATES • INC.** 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8302 C- 2 - 3 - 11 SUBJ. SUBDIV. SHEET
PROJECT W3 CLIENT LP&L SUBJECT BASEMAT INTERNAL FORCES	PREP. BY GH DATE 10-6-83 CHKD. BY AdB DATE 10.6.83

DBE EW SEISMIC

EL 404

$p = .005$

$p_w = T \& B$

$V = 367$

$M_x = 451$

$F_x \ 108$

$p_w = \frac{5(1.56)}{12(133)} = .005$

(11-7) $M_m = 451 - 108(4.61) = -$, ∴ USE (11-8).

(11-8) $V_c = 353\sqrt{1 + \frac{108}{.5(12)(133)}} = 376$

98

V_{REBAR}

$98 + \phi(376) = 418 > 367^k$ OK

EL 406

p_w SHOULD INCLUDE BOTT (TOP & BOTT LAYERS)

$p_w = \frac{3(1.56)}{12(133)} = .003$

$V = 389$

$M = 527$

$F = 105$

(11-7) $M_m = 527 - 105(4.6) = 44$

(11-6) $V_c = \left(120.2 + 2500(.003)\left(\frac{389}{44}\right)\left(\frac{133}{12}\right)\right)bd = >>$

98

V_{REBAR}

$98 + \phi(375) = 417 > 389$ OK

$V_c = 353\sqrt{1 + \frac{105}{.5(12)(133)}} = 375$

HEA HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 8304
C- 2 - 3 - 12
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

PREP. BY GH DATE 10-6-83
CHCKD. BY AdB DATE 10/6/83

DBE EW SEISMIC

ELEM 415

$p = .003$

$V = 356$
$M = 1197$
$F = 186$

ACI 318-77

(11-7) $M_m = 1197 - (186)(4.6) = 341.4$

(11-6) $V_c = \left[(120.2 + 2500(.003)\left(\frac{356}{341.4}\right)\left(\frac{133}{12}\right)\right] bd = 330$

V_{REBAR} 98

98 $\phi(330\ 98) = 379 > 356$ OK

$$V_c = 353\sqrt{1 + \frac{186}{.5(12)(133)}} = 392$$

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 8304
C- 2 - 3 - 13
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

PREP. BY GH DATE 10-6-83
CHCKD. BY ADZ DATE 10/6/83

SOUTH TO NORTH

ELEM 134

$V = 210$
$M = 642$
$F = 237$

11-7 $M_m = 642 - 237(4.6) = -$, ∴ USE (11-8)

$V_c = 353\sqrt{1 + \frac{237}{.5(12)(133)}} = 402$

$\phi(402) = 341 > 210$ OK

ELEM 136, 146 OK BY SIMILARITY TO 134

ELEM 172

$V_u = 429$
$M_u = 359$
$N_u = 118$

(11-7) $M_m = 359 - 118(4.6) = -$, ∴ USE (11-8)

$V_c < 353\sqrt{1 + \frac{118}{.5(12)(133)}} = 378$

98

V REBAR

∴ $98 + \phi(378)$ 420 ≈ 429 SAY OK

(2% DIFF.)

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	833?		
C- 2 - 3 - 14	SUBJ.	SUBDIV.	SHEET
PREP. BY	GH	DATE	10-6-83
CHCKD. BY	AdB	DATE	10/6/83

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

SOUTH TO NORTH

ELEM 150

$V_u = 272$
$M_u = 596$
$N_u = 209$

11-7 $M_m = 596 - 209(4.6) = -$, ∴ USE (11-8)

$V_c = 353\sqrt{1 + \frac{209}{.5(12)(133)}} = 397$

$\phi\ 397 = 337 > 292$

ELEM 188

P_w NS = .0064

$V_u = 232$
$M_u = 1808$
$N_u = 100$

$\frac{V_u d}{M_u} = \frac{232 \times 133/12}{1808} = 1.4 > 1.0$

=

$M_m = 1808 - 100(4.6) = 1347$

$V_c = \left(120.2 + 2500\,(.0064\,/\,\frac{232}{1347})\left(\frac{133}{12}\right)\right) bd = 240$

$\phi\,(240) = 204 < 232$

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT W3	PROJ. NO. C- 2 - 3 - 15 (SUBJ. SUBDIV. SHEET)
CLIENT LP&L	PREP. BY GH DATE 10-7-83
SUBJECT BASEMAT INTERNAL FORCES	CHCKD. BY AdB DATE 10/7/83

SOUTH TO NORTH

ELEM 199

$A_s = 4 \times 4 = 16 \text{ in}^2$

$\rho_w = \frac{16}{12 \times 133} = 0.01$

$V_u = 433$

$M_u = 905$

$N_u = 134$

$\frac{V_u d}{M_u} = \frac{433 \times 133/12}{134} > 1.0$

$M_m = 905 - 134(4.6) = 287$

$V_c = \left(120.2 + 2500(0.01)\left(\frac{433}{287}\right)\left(\frac{133}{12}\right)\right) bd = 859 k >>$

V_{REBAR} 98

$98 + \phi\ 381 = 422 < 433$ SAT OK Δ 2.5%

$V_c = 353\sqrt{1 + \frac{134}{5(12)(133)}} = 381$

H E A	HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8304
		C- 2 - 3 - 16 (SUBJ. SUBDIV. SHEET)
PROJECT	W3	PREP. BY GH DATE 10-6-83
CLIENT	LP&L	CHCKD BY ADB DATE 10/7/83
SUBJECT	BASEMAT INTERNAL FORCES	

AXIAL TENSION

ELEM	FX	FY	FXZ	FYZ		
10	+23.5	-82.8	+22.6	+32.3	EW DBE	OK
11	+63.8	-87.5	+72.3	-37.5	"	OK
12	+94.1	-86.2	+25.3	+57.2	"	OK
13	+71.2	-100.4	+16.3	+37.5	"	OK
15	+6.2	-47.8	-8.0	+16.4	"	OK
370	+4.0	-156.7	+53.6	+55.1	"	OK
379	-118.2	+11.8	-51.3	50.0	"	OK
385	+98.7	-131.5	+35.3	+62.7	"	OK
388	-95.2	+5.6	+29.0	+10.2	"	OK
396	+36.6	-198.4	+136.9	-9.7	"	OK
15	+8.1	-53.6	-9.1	+19.8	N TO S	OK
15	+8.3	-80.1	-10.8	+13.2	S TO N	OK
598	+14.7	-72.2	25.5	-43.4	"	OK
608	+71.9	-67.5	-10.8	+36.3	"	OK
607	+10.2	-59.1	+32.7	-5.2	"	OK
609	+608	-35.9	+16.9	-12.1	"	"
610	+19.2	-47.2	-25.5	-12.3	"	OK
619	+14.3	-36.0	-32.7	1.4	"	OK

COMPARE AXIAL FORCES

ELEM	SO. TO NORTH FX	SO. TO NORTH FY	NORTH TO SOUTH FX	NORTH TO SOUTH FY
503	-175.2	-85.7	-210.6	-83.7
496	-175.6	-96.2	-211.2	-98.3
488	-210.9	-102.6	-185.3	-93.6
480	-205.7	-110.7	-178.8	-105.5
477	-206.4	-114.5	-179.2	-107.8
484	-206.9	-111.0	-182.0	-100.4
493	-172.8	-104.8	-208.7	-105.2
500	-176.9	-95.0	-212.0	-91.6
Σ	1530.9		1567.8	OK

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 8304
C- 2 - 3 - 17
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

PREP. BY AdB DATE 10/5/83
CHCKD. BY GH DATE 10/7/83

EL NO	(FXZ) EW	(FXZ) NS	SRSS	% INC.	(FYZ) EW	(FYZ) NS	SRSS	% INC
98	6.5	-5.6	8.6	32.0	6.3	-4.0	7.5	18.5
134	-1.3	18.2	18.2	0.3	1.9	-3.9	4.3	11.2
136	-2.9	4.8	5.6	16.8	-0.5	-4.0	4.0	0.8
146	-5.9	-14.0	15.2	8.5	-3.5	-5.7	6.7	17.3
150	-8.7	36.6	37.6	2.6	3.6	0.8	3.7	2.4
154	0.7	-2.0	2.1	5.9	7.7	-3.0	8.3	7.3
172	-34.1	13.4	36.6	7.4	-2.0	8.0	8.2	3.1
188	16.3	31.9	35.8	12.3	13.5	26.6	29.8	12.1
199	-21.9	-14.8	26.4	20.7	-4.3	13.3	14.0	5.1
211	27.5	-20.2	34.1	24.1	9.8	-16.8	19.4	15.8
230	20.3	13.9	24.6	21.2	38.1	21.9	43.9	15.3
235	-11.9	2.8	12.2	2.7	-15.9	-21.1	26.4	25.2
261	-16.1	45.7	48.5	6.0	-46.1	-24.9	52.4	13.7
353	16.7	66.1	68.2	3.1	1.4	-3.2	3.5	9.2
391	-5.7	4.4	7.2	26.3	-39.8	-16.1	42.9	7.9
404	32.9	9.2	34.2	3.8	18.0	-1.7	18.1	0.4
406	-6.3	-3.7	7.3	16.0	-11.2	0.2	11.2	0.0
415	-13.6	2.3	13.8	1.4	-8.2	0.3	8.2	0.1
430	-8.5	4.7	9.7	14.3	2.7	-2.8	3.9	38.9

APPENDIX C

STARDYNE Shear

Convention for MARTIN Element

Figure C-1

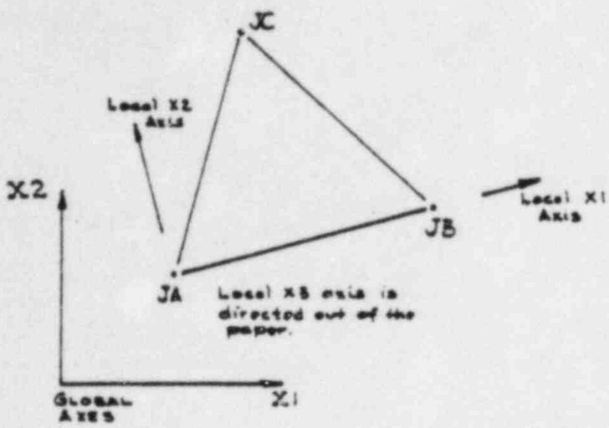


Figure 1 Triangle Local Coordinate System

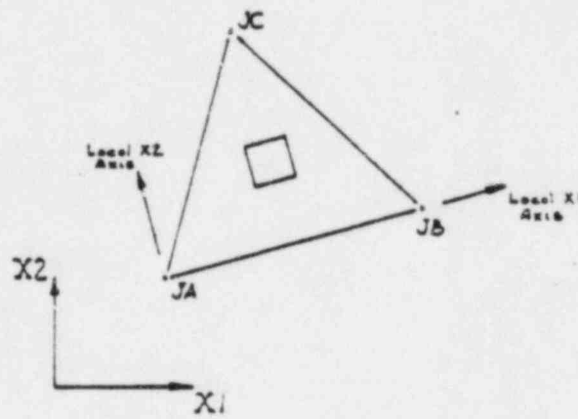


Figure 3 Differential Element for Identifying Stress Components

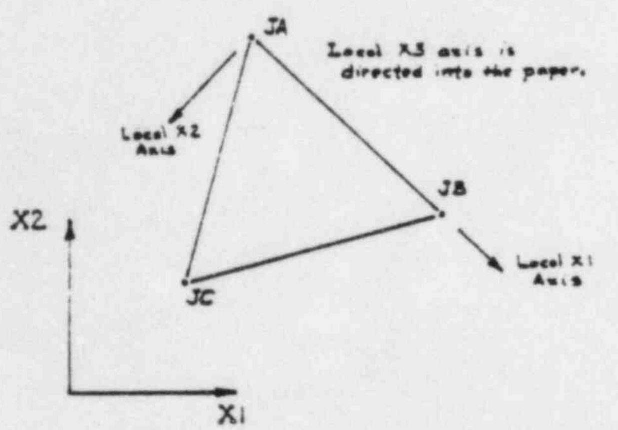


Figure 2 Alternate Local Coordinate System

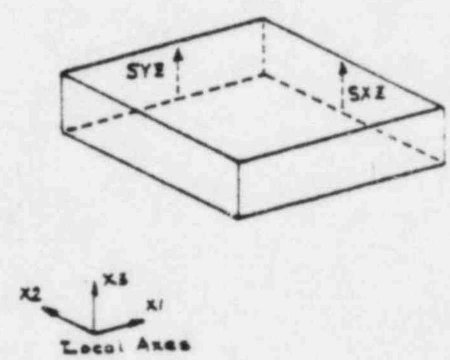
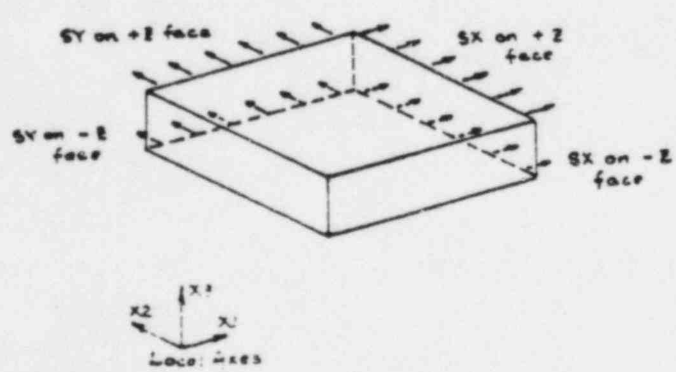


Figure 4 Directions for Transverse Shear Stress



Directions for Direct Stresses

SDC
stem Development Corporation
.0 N. Sepulveda Boulevard, El Segundo, CA 90245, Telephone (213) 615-1188

September 29, 1983

Mr. Andy DuBouchet
Harstead Engineering Associates
169 Kinderkamack Road
Park Ridge, New Jersey 07656

Dear Andy,

As a result of our telephone conversation, last Monday morning, I am sending a description of the stress sign convention for our "thick plate" triangle element.

Please call if you have any additional questions.

Sincerely,

Charles Bell

Charles A. Bell
STARDYNE Division

Enclosure

CAB/mgf

Directions of Stress Components ... Triangle Elements

Ascertaining the directions of the stress components, for the triangle plate element, begins with the orientation of the local coordinate system for each plate element. From the input on the TRIAB cards, node numbers are associated with the designations JA , JB , and JC . The origin of the local system is at node JA, with the local X1 axis directed toward node JB . The local X2 axis is in the direction of node JC , and the local X3 axis completes the right-handed triad. Figure 1 illustrates a possible local coordinate system for a triangle element; Figure 2 shows a second possible local system for the same triangle. The choice of which node is designated JA is left to the user's discretion. An additional option is the AXIS ANGLE input on the TRIAB card; this allows an angle between the JA - JB line and the local X1 axis.

Once a local coordinate system has been selected and identified, a differential element can be located near the triangle center, Figure 3. This element has sides parallel to the local X1 and X2 axes. The output stress values are located on the two faces, of the differential element, that are furthest from the local coordinate origin. Figure 4 shows the directions when the listed transverse shear stresses are positive; a positive stress value means that the stress arrow is in the local X3 axis direction on the indicated face. Figure 5 shows the direction for positive direct stresses. Negative algebraic signs, in the printed output, reverses the sense of the stress arrows in the two figures.

APPENDIX D

Ebasco/HEA

Shear and Moment Plots

EBASCO SERVICES INCORPORATED

NEW YORK

Figure D-1

BY f.a. DATE 4-16-81

CHKD. BY I. CHEN DATE 4/23/81

SHEET E1 OF

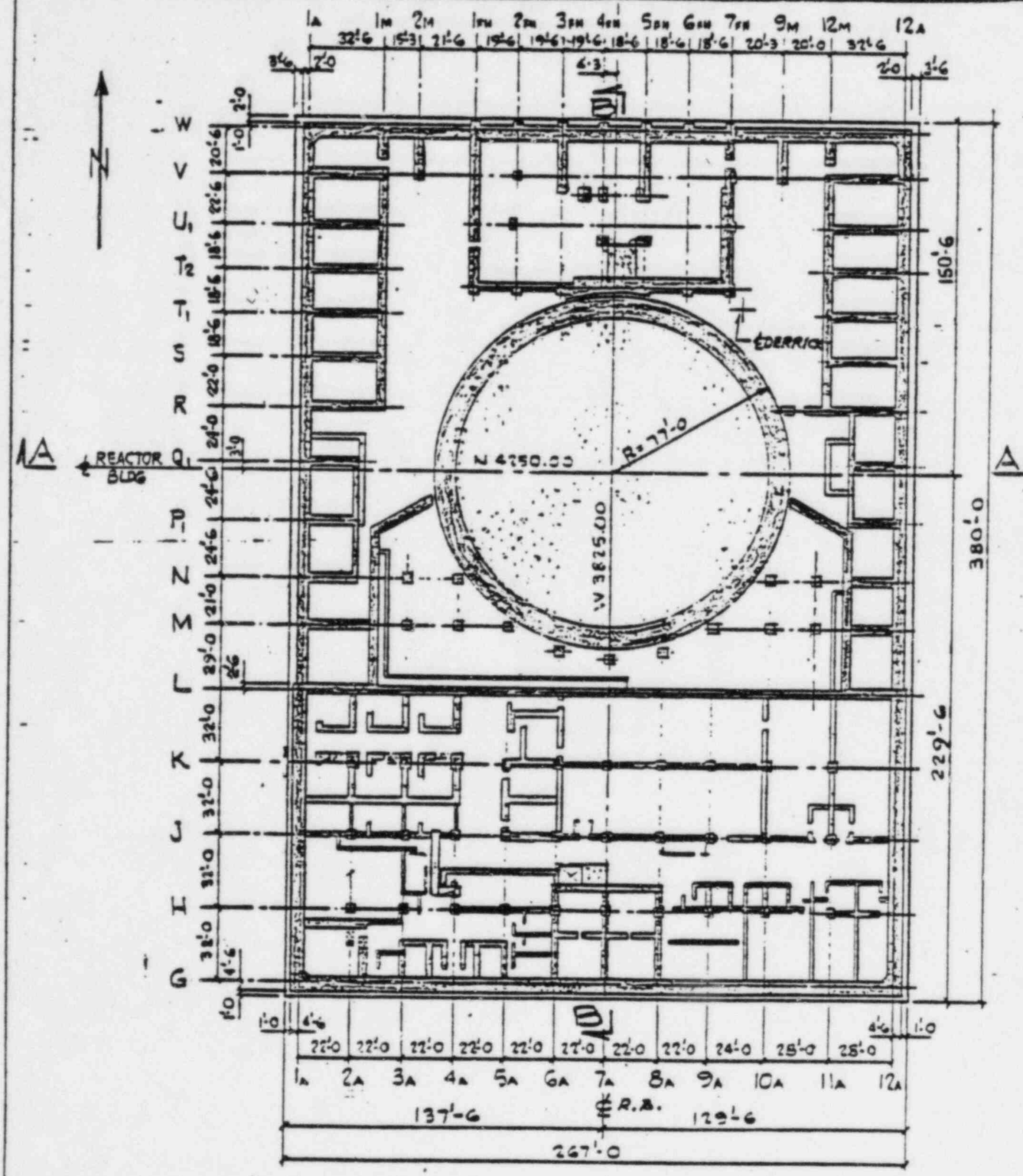
OFS NO. 5234.014 DEPT. NO. 550

CLIENT LOUISIANA POWER & LIGHT CO.

~~WATERFORD STEAM ELECTRIC STATION~~

PROJECT 1977 1165 MW INSTALLATION - UNIT 3

SUBJECT COMMON FOUNDATION MAT - PLAN AT EL 35.00'



PLAN AT EL -35.00'

FORM 881 REV 7-71

Figure D-2

EBASCO SERVICES INCORPORATED

NEW YORK

BY J. CHEN DATE 6/11/81 SHEET 64 OF

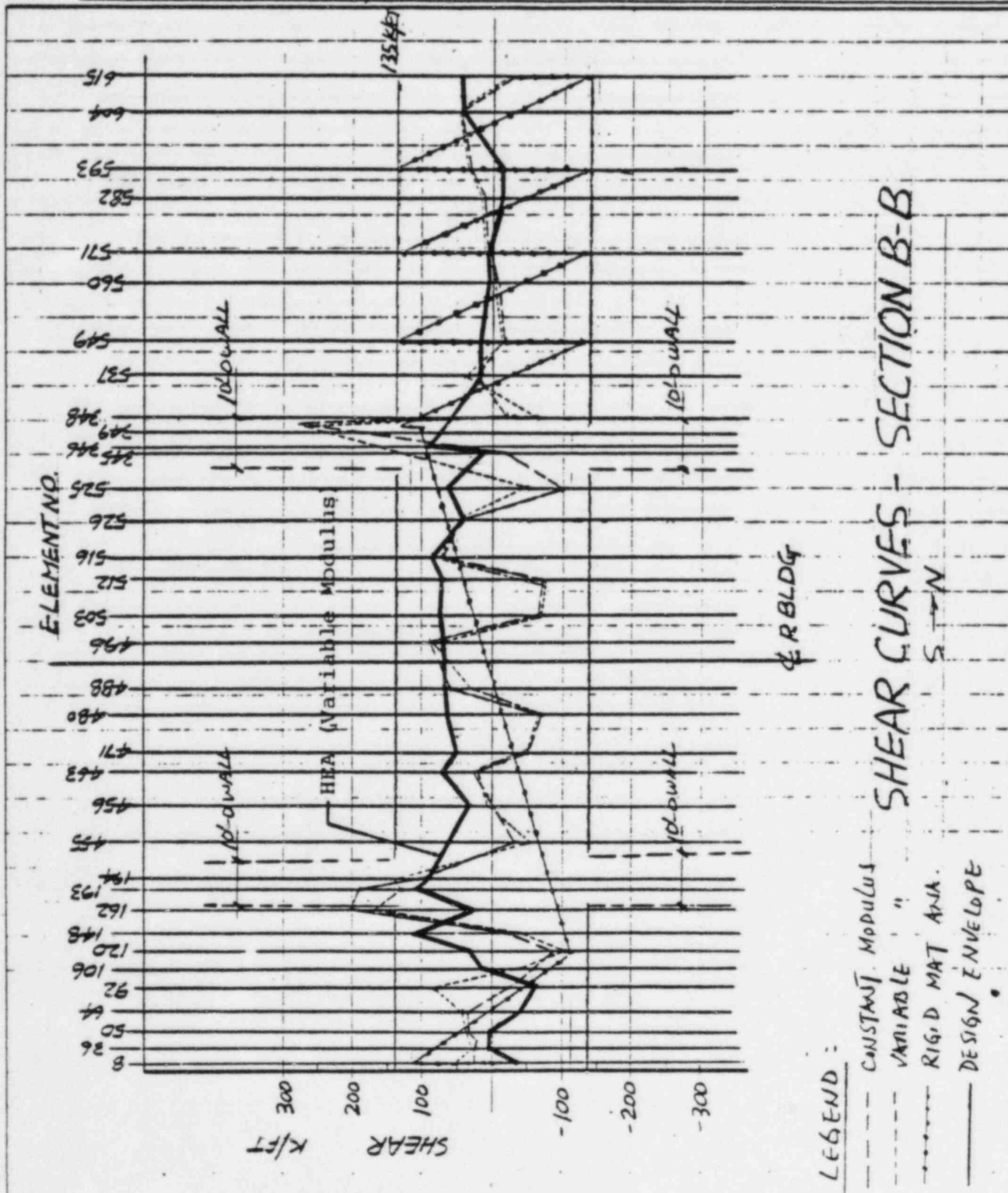
CHKD. BY J. YANG DATE 6/11/81 OFS NO. 5234.014 DEPT. NO. 550

CLIENT LOUISIANA POWER & LIGHT CO.

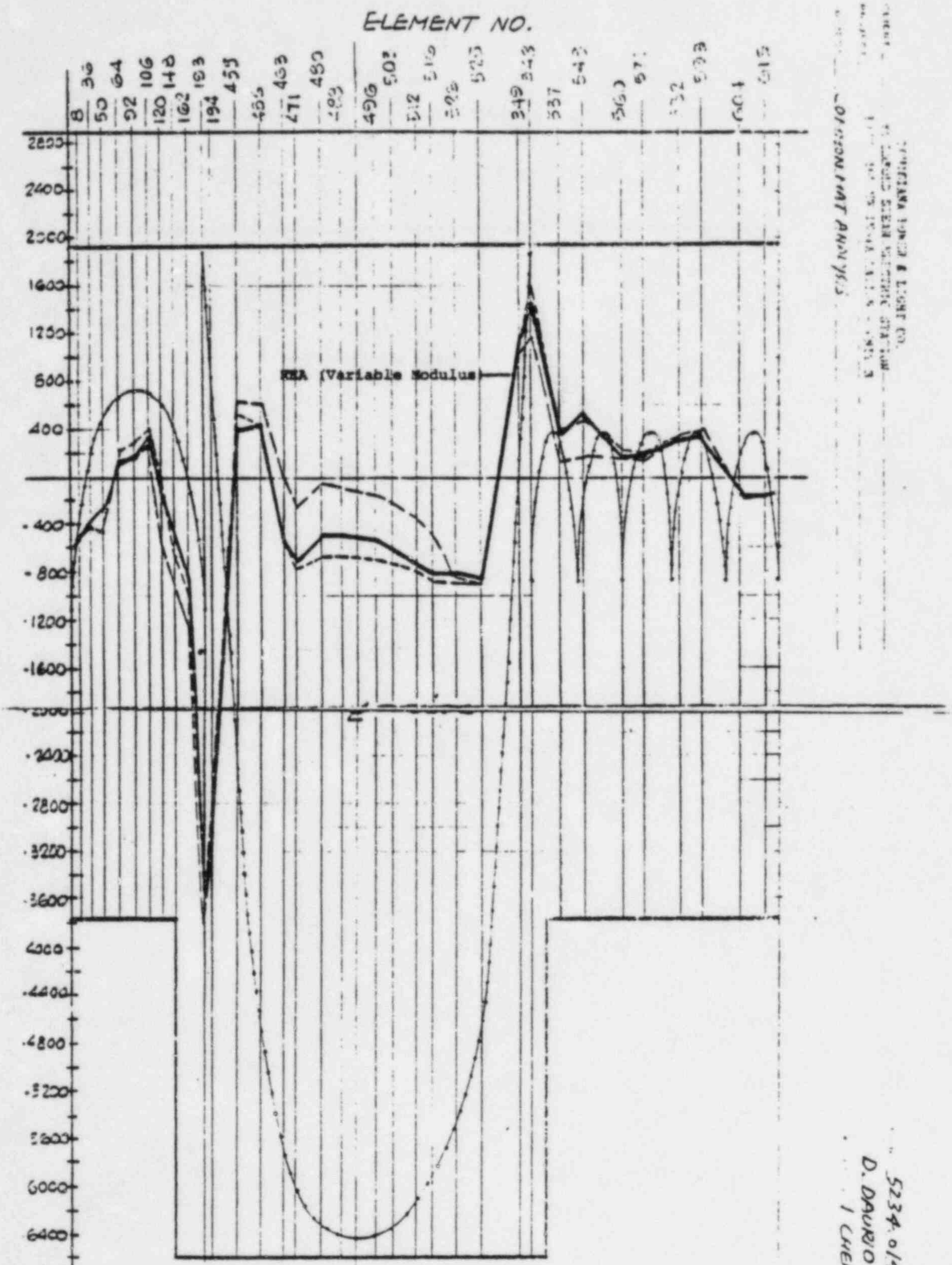
~~WATERFORD STEAM ELECTRIC STATION~~

PROJECT 1977 1165 MW INSTALLATION - UNIT 3

SUBJECT COMMON MAT ANALYSIS



FORM 381 REV 7-71



MOMENT CURVES - SECTION B-B

LEGEND

- CONSTANT MODULUS
- VARIABLE MODULUS
- RIGID MAT ANALYSIS
- DESIGN ENVELOPE

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	8304
C- 2 - 4 - 1	SUBJ. SUBDIV. SHEET
PROJECT	W3
CLIENT	LP&L
SUBJECT	BENCHMARK SHEARS AND MOMENTS
PREP. BY	AGS DATE 10/26/83
CHCKD. BY	GH DATE 10/7/83

DATA FOR THE SHEAR AND MOMENT PLOTS LABELLED 'HEA (VARIABLE MODULUS)' AND SUPERIMPOSED ON PAGES 84 AND 87 OF THE EBASCO CALCULATION BOOK ENTITLED 'WATERFORD 3 BASE MAT DESIGN BOOK L', ARE OBTAINED AS FOLLOWS:

1. SHEARS FXZ, FYZ AND MOMENTS MX, MY ARE TABULATED FOR ALL BASEMAT ELEMENTS CUT BY SECTION B-B AS SHOWN ON PAGE 81 OF THE ABOVE REFERENCED EBASCO CALCULATION BOOK (PAGE 2)

2. THE LOCAL COORDINATE AXES FOR EACH OF THESE ELEMENTS ARE IDENTIFIED (PAGES 4-6)

3. THE SHEARS ARE REVIEWED WITH RESPECT TO THE 'STARDYNE' SIGN CONVENTION. FOR CONSISTENCY IN PLOTTING THE SHEAR, THE SHEAR ACTING ON A VERTICAL FACE WHOSE NORMAL IS ALIGNED ALONG THE GLOBAL −X AXIS (IE NORTH) IS TABULATED. SHEAR ACTING UPWARD DEFINED POSITIVE.

4. THE MOMENTS GENERATING AXIAL STRESS ON THE VERTICAL FACE DEFINED IN (3) ARE ALSO TABULATED.

5. THE FINAL TABULATION OF THE SHEARS (WITH CORRECT SIGN) AND MOMENTS IS GIVEN ON PAGE 3.

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 8304
C- 2 - 4 - 2
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BENCHMARK SHEARS AND MOMENTS

PREP. BY ACB DATE 10.06.83
CHCKD. BY GH DATE 10/7/83

ELEM NO	FXZ	FYZ	MX	MY	ELEM NO	FXZ	FYZ	MX	MY
8	-16.2	(34.3)	-178.4	(-514.6)	593	(11.2)	0.7	(314.5)	17.4
36	(2.7)	26.0	(-377.4)	-217.7	604	(43.6)	-0.7	(-178.0)	-72.3
50	-31.3	(2.6)	-176.6	(-264.6)	615	(-43.4)	0.1	(-180.6)	-55.5
64	29.7	(40.1)	-110.0	(101.4)					
92	(-54.7)	47.7	(188.7)	-218.2					
106	-3.6	(15.7)	-167.1	(332.7)					
120	6.9	(-35.6)	-254.5	(-147.6)					
148	(106.1)	-2.1	(-473.8)	-270.6					
162	14.4	(40.6)	-334.0	(-861.9)					
193	-5.8	(-102.9)	-828.9	(-3581)					
194	-57.7	(86.9)	-677.9	(-3040)					
455	(-59.8)	-5.9	(390.9)	-19.7					
456	(32.3)	2.0	(419.1)	-441.2					
463	(63.9)	2.5	(-575.2)	-620.2					
471	(-51.3)	5.0	(-707.7)	-709.8					
480	(-65.6)	5.0	(-497.5)	-671.6					
488	(70.4)	-2.2	(-493.5)	-697.2					
496	(75.6)	-2.2	(-567.5)	-710.7					
503	(-74.9)	-0.5	(-608.8)	-710.5					
512	(-71.2)	-0.5	(-756.1)	-737.3					
516	(83.4)	-3.3	(-812.1)	-709.6					
526	(46.1)	-3.1	(-809.9)	-709.0					
525	(-58.2)	12.1	(-867.8)	-730.5					
345	-160.3	(-14.3)	-148.5	(2357)					
346	16.1	(93.6)	8.4	(2411)					
349	(69.5)	19.4	(1173)	-156.7					
348	-15.3	(57.6)	187.2	(1413)					
537	(12.5)	2.0	(388.9)	0.8					
549	(-17.5)	-0.2	(491.2)	100.0					
560	(3.6)	0.2	(182.0)	43.8					
571	(-4.6)	0.7	(174.3)	-7.6					
582	(-10.5)	-0.7	(318.8)	18.7					

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

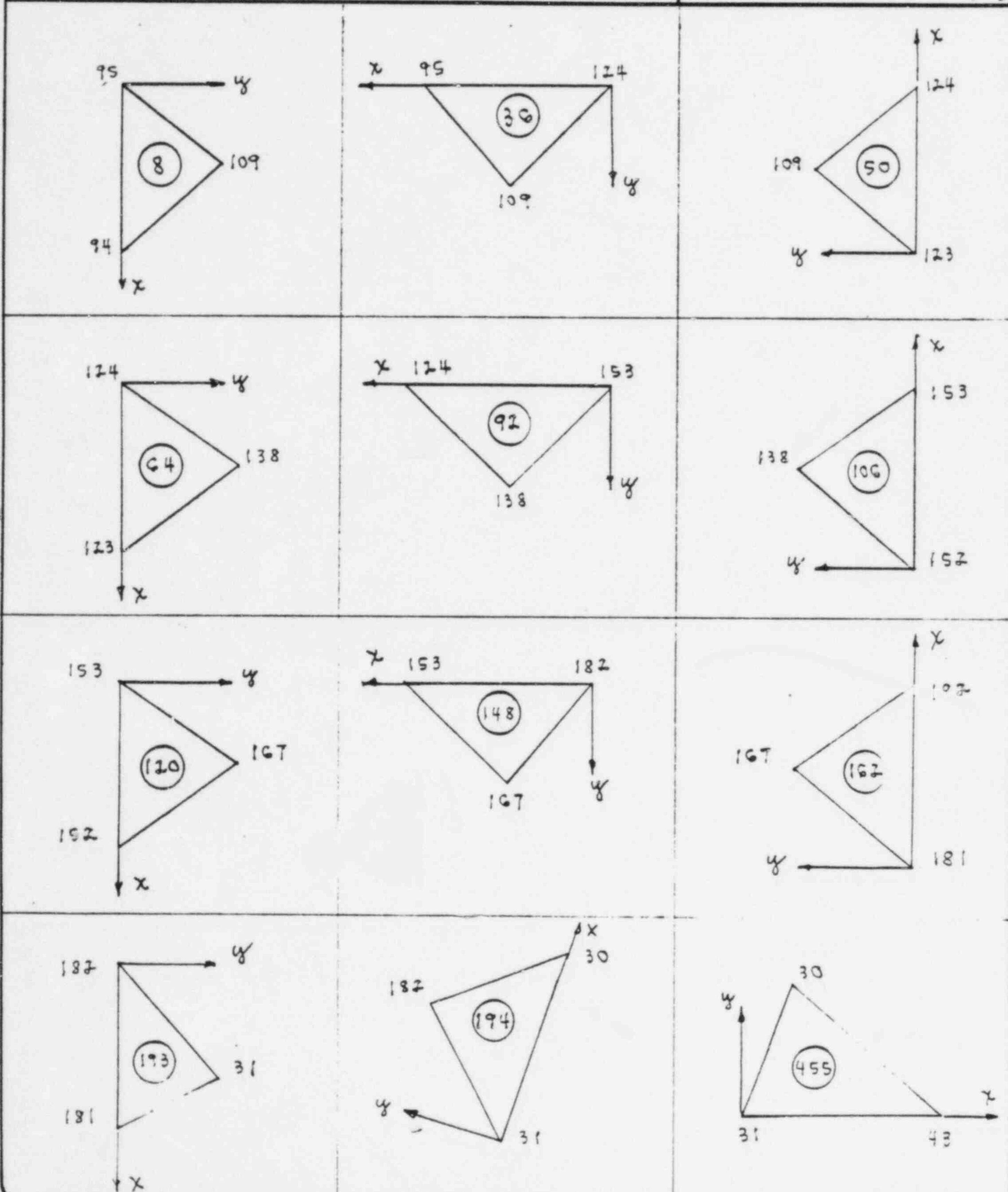
PROJ. NO. 8304
C- 2 - 4 - 3
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BENCHMARK SHEARS AND MOMENTS

PREP. BY AdB DATE 10/06/83
CHCKD. BY GH DATE 10/7/83

ELEMNO	F	M			ELEMNO	F	M		
8	-34.3	-514.6			593	-11.2	314.5		
36	2.7	-377.4			604	43.6	-178.0		
50	2.6	-264.6			615	43.4	-180.6		
64	-40.1	101.4							
92	-54.7	188.7							
106	15.7	332.7							
120	35.6	-147.6							
148	106.1	-473.8							
162	40.6	-861.9							
193	102.9	-3581							
194	86.9	-3040							
455	59.8	390.9							
456	32.3	419.1							
463	63.9	-575.2							
471	51.3	-707.7							
480	65.6	-497.5							
488	70.4	-493.5							
496	75.6	-567.5							
503	74.9	-608.8							
512	71.2	-756.1							
516	83.4	-812.1							
526	46.1	-809.9							
525	58.2	-867.8							
345	14.3	2357							
346	93.6	2411							
349	69.5	1173							
348	57.6	1413							
537	12.5	388.9							
549	17.5	491.2							
560	3.6	182.0							
571	4.6	174.3							
582	-10.5	318.8							

HEA HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8304 C- 2 - 4 - 7 SUBJ. SUBDIV. SHEET
PROJECT W3 CLIENT LP&L SUBJECT BENCHMARK SHEARS AND MOMENTS	PREP. BY AdB DATE 10/06/83 CHCKD. BY GH DATE 10/7/83

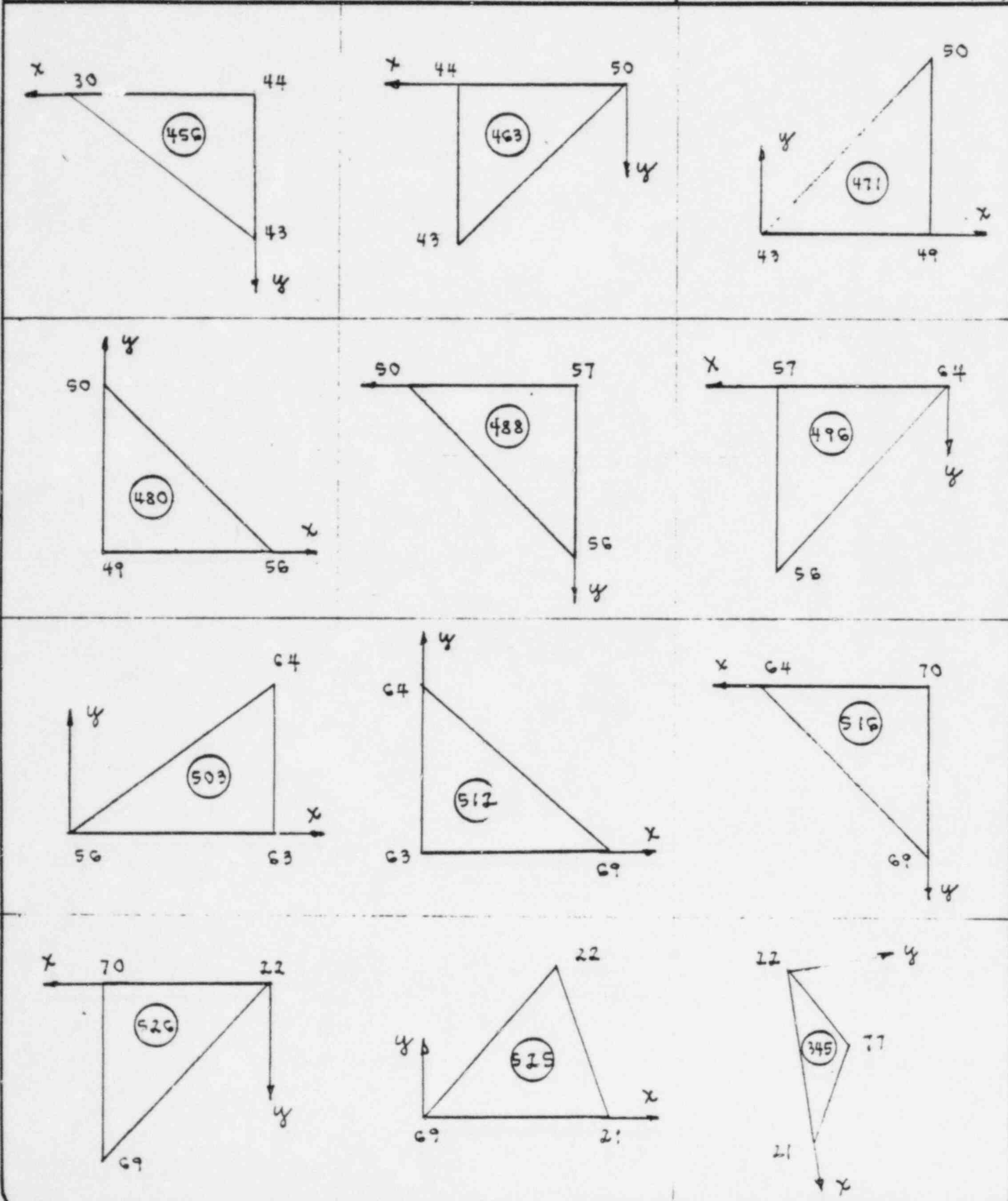


HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	8304
C- 2 - 4 - 5	
SUBJ. SUBDIV. SHEET	
PREP. BY AdB	DATE 10/06/83
CHCKD. BY GH	DATE 10/7/83

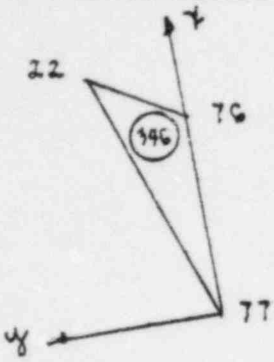
PROJECT W3
CLIENT LP&L
SUBJECT BENCHMARK SHEARS AND MOMENTS

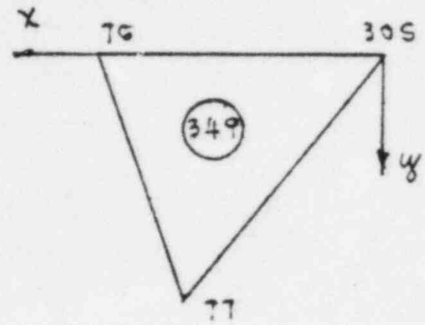


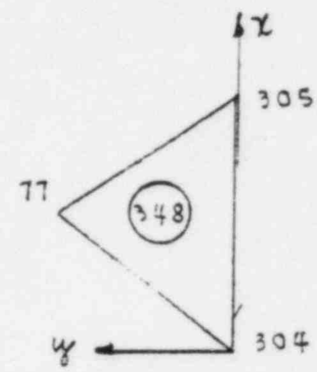
HEA HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

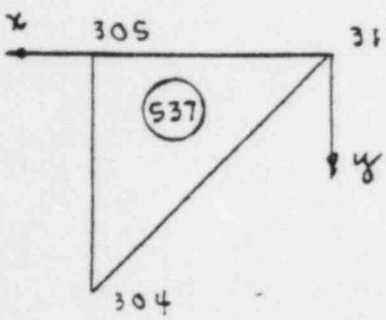
PROJ. NO.	8304
C- 2 - 4 - 6	SUBJ. - SUBDIV. - SHEET
PREP. BY AGB	DATE 10.06.83
CHCKD. BY GH	DATE 10/7/83

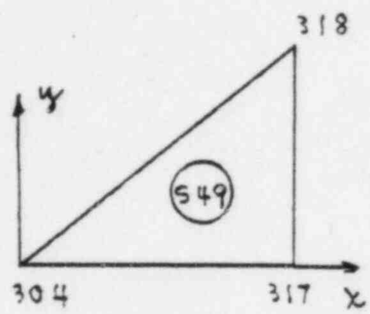
PROJECT W3
CLIENT LP&L
SUBJECT BENCHMARK SHEARS AND MOMENTS

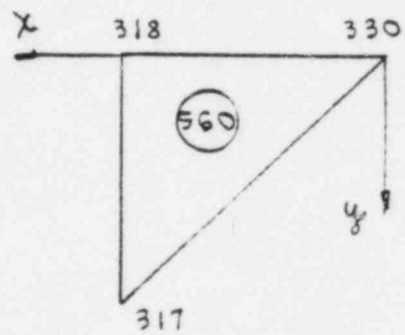


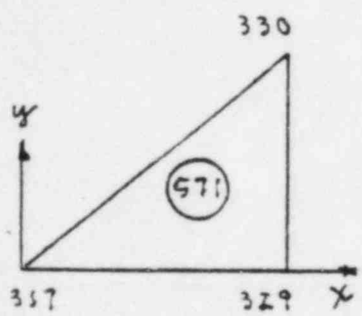


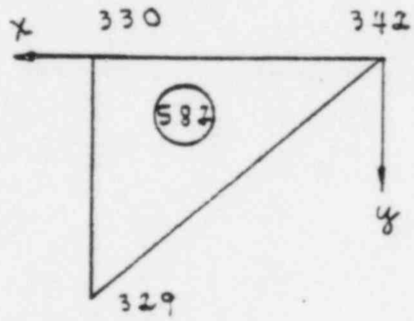


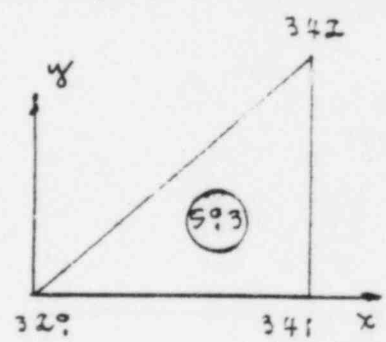


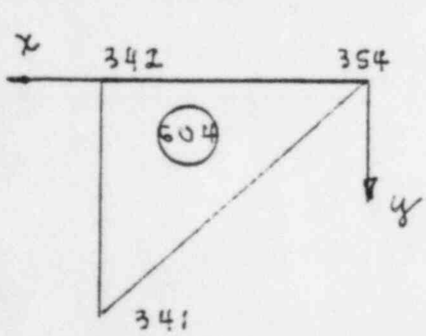


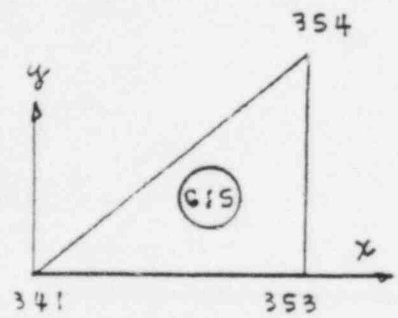










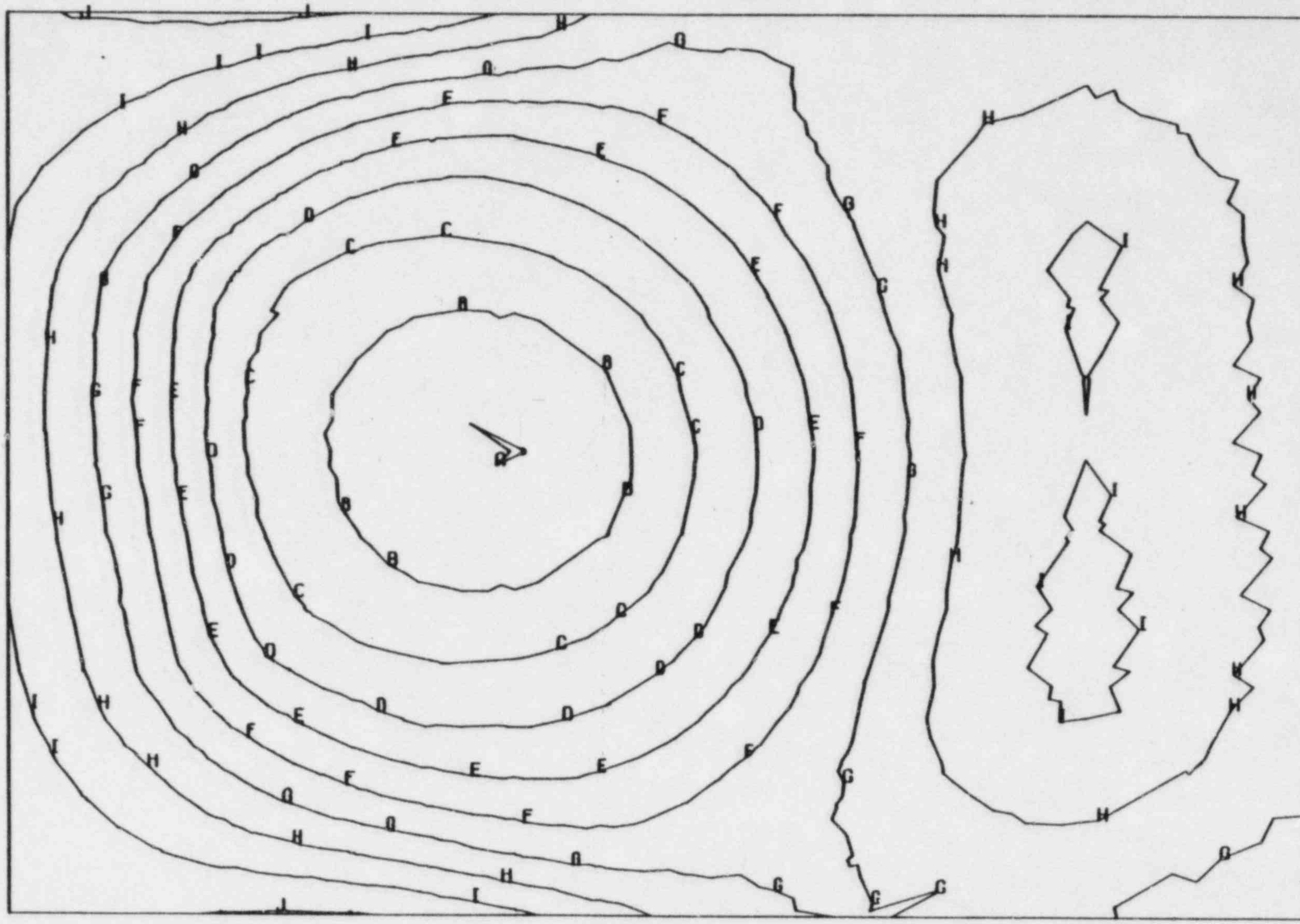


APPENDIX E

Plots of Basemat Displacements

Figure E-1

NORMAL OPERATION C=1.5(D+L1)+1.8(L+S)+1.0B



CONTOUR LEVELS

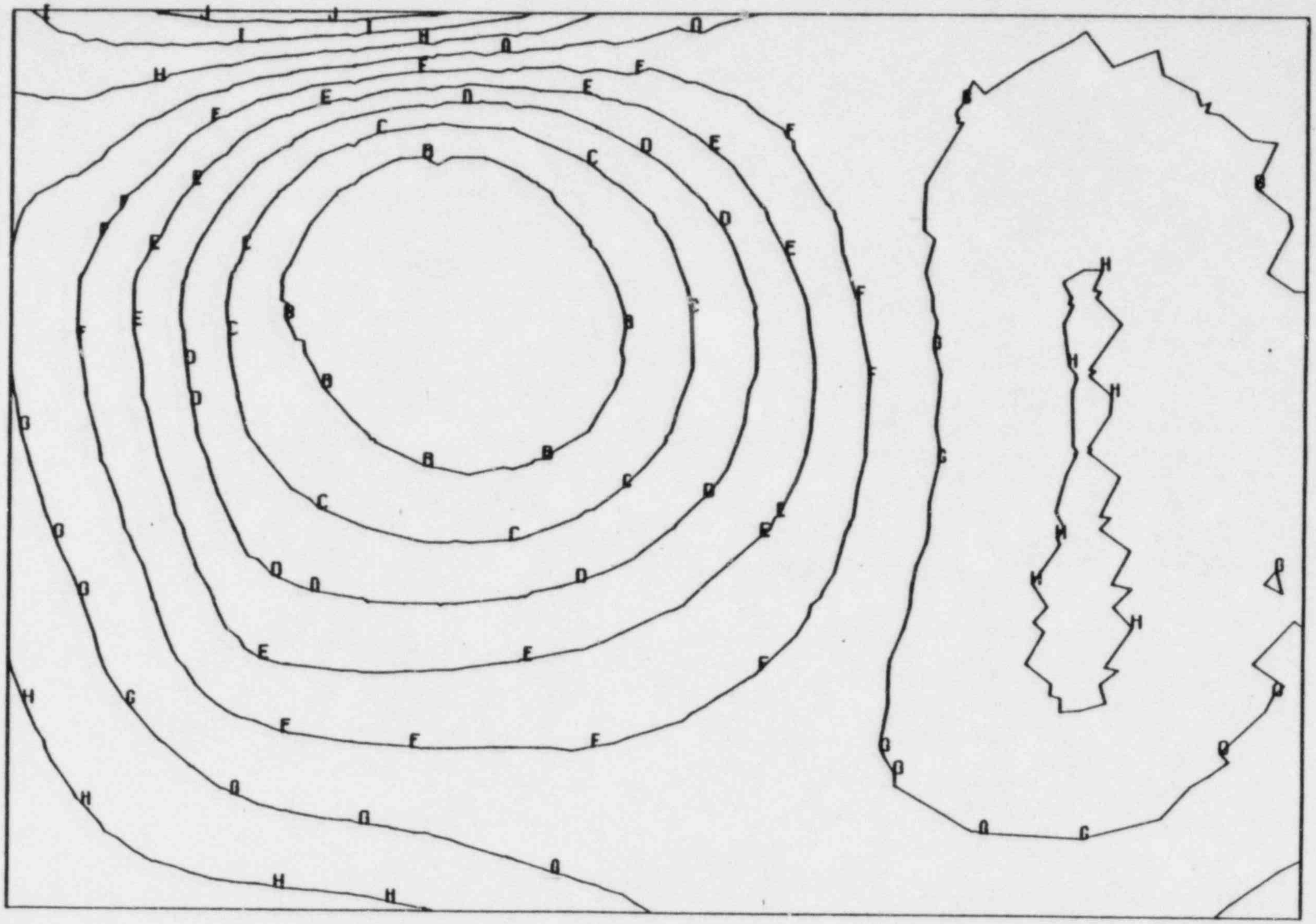
MIN	-.62294E-01
A	-.62294E-01
B	-.56711E-01
C	-.51129E-01
D	-.45546E-01
E	-.39964E-01
F	-.34382E-01
G	-.28799E-01
H	-.23217E-01
I	-.17635E-01
J	-.12052E-01
MAX	-.12052E-01

PLOT OF NORMAL DISPLACEMENT
STAR TAPE4 VECTOR NO 11

SCALE = 43.706

Figure E-2

D.B.E. WEST EAST C=1.1(D+L)+1.0(E+B+S)



CONTOUR LEVELS

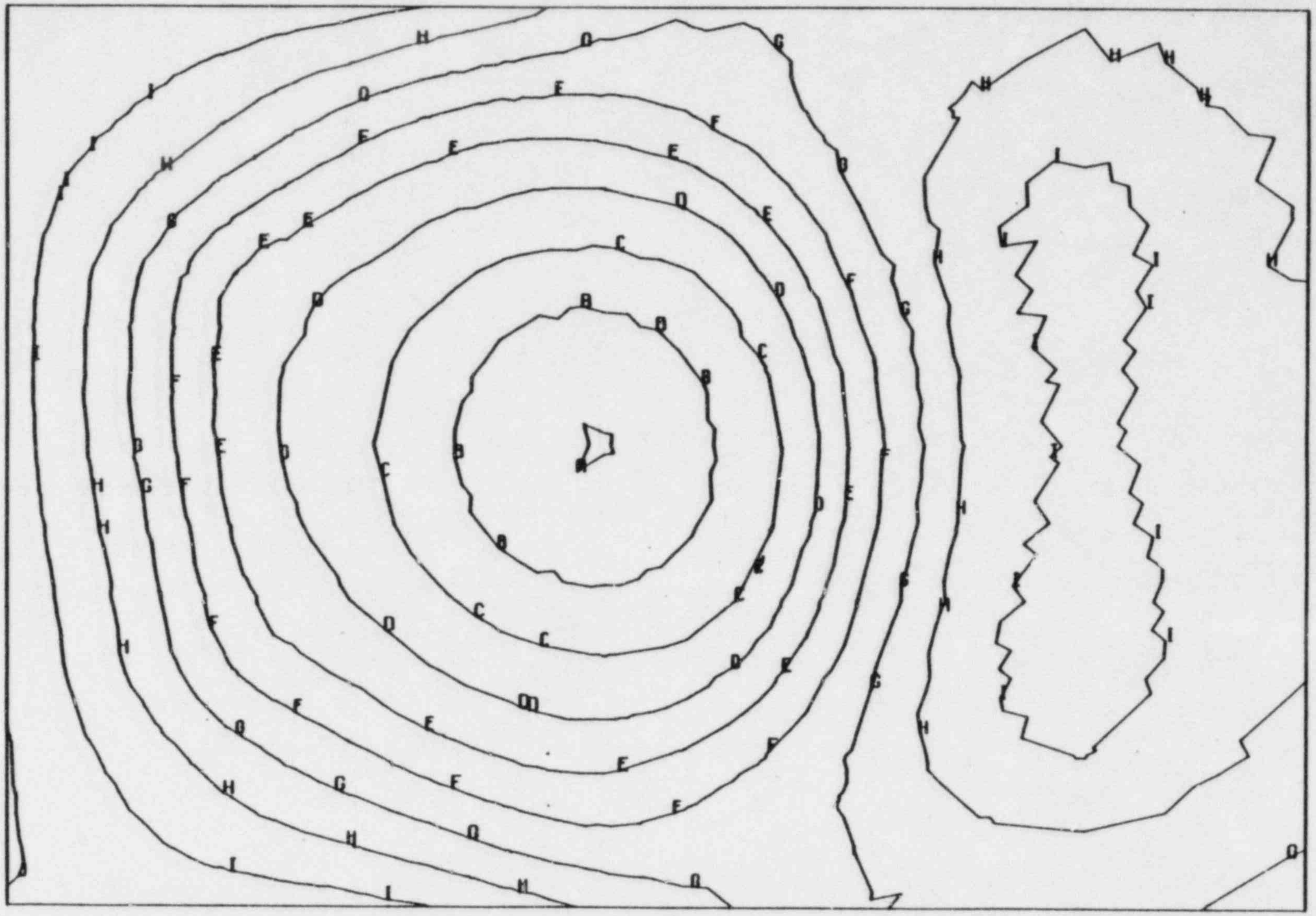
MIN	-.48356E-01
A	-.48356E-01
B	-.42695E-01
C	-.37033E-01
D	-.31372E-01
E	-.25710E-01
F	-.20049E-01
G	-.14388E-01
H	-.87262E-02
I	-.30649E-02
J	.25965E-02
MAX	.25965E-02

PLOT OF NORMAL DISPLACEMENT
STAR TAPE4 VECTOR NO 12

SCALE = 43.706

Figure E-3

D.B.E. NORTH SOUTH C=1.1(D+L)+1.0(E+B+S)



CONTOUR LEVELS

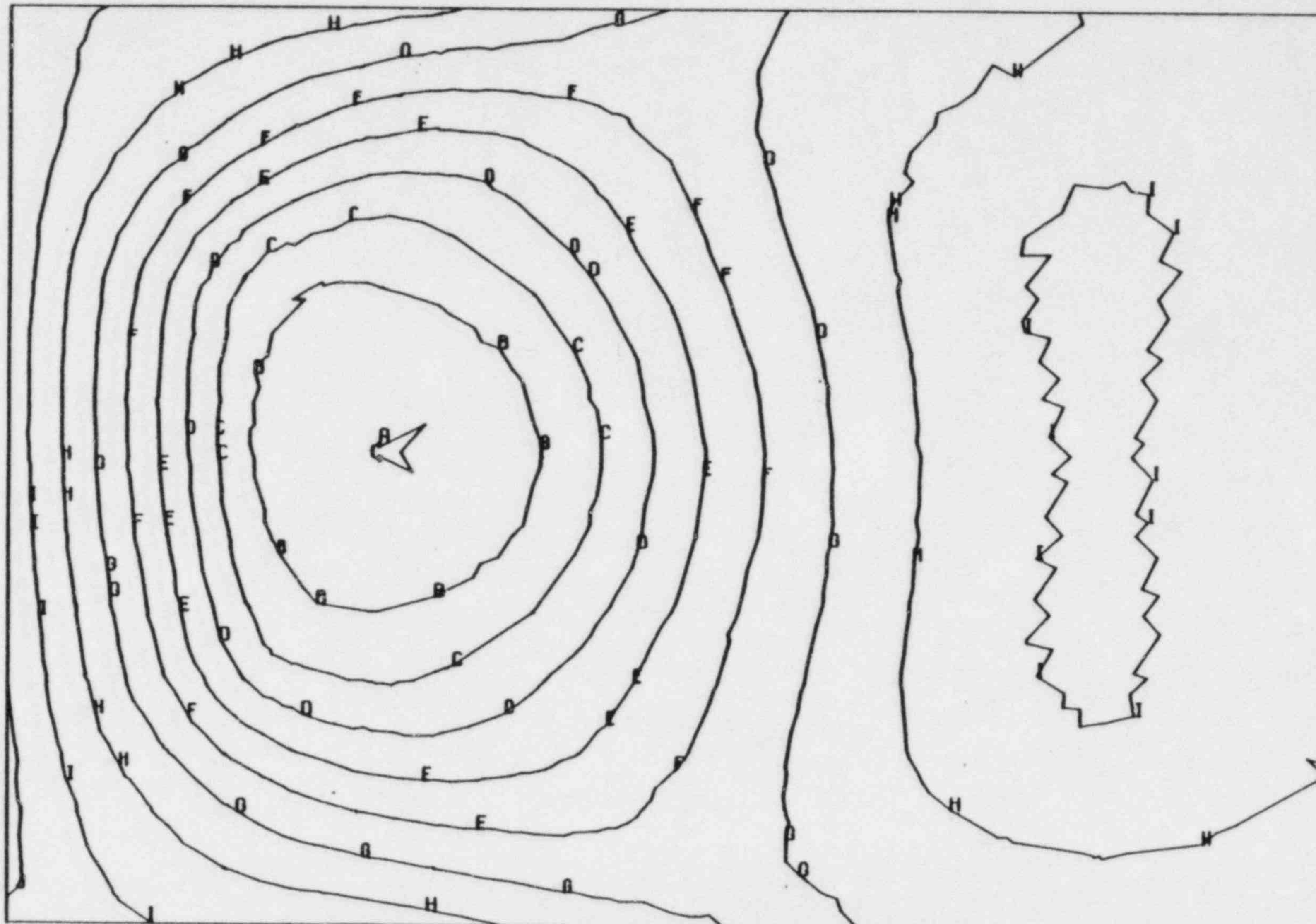
MIN	-.43131E-01
A	-.43131E-01
B	-.39077E-01
C	-.35024E-01
D	-.30971E-01
E	-.26917E-01
F	-.22864E-01
G	-.18811E-01
H	-.14757E-01
I	-.10704E-01
J	-.66505E-02
MAX	-.66505E-02

PLOT OF NORMAL DISPLACEMENT
STAR TAPE4 VECTOR NO 13

SCALE = 43.706

Figure E-4

D.B.E. SOUTH NORTH C=1.1(D+L)+1.0(E+B+S)



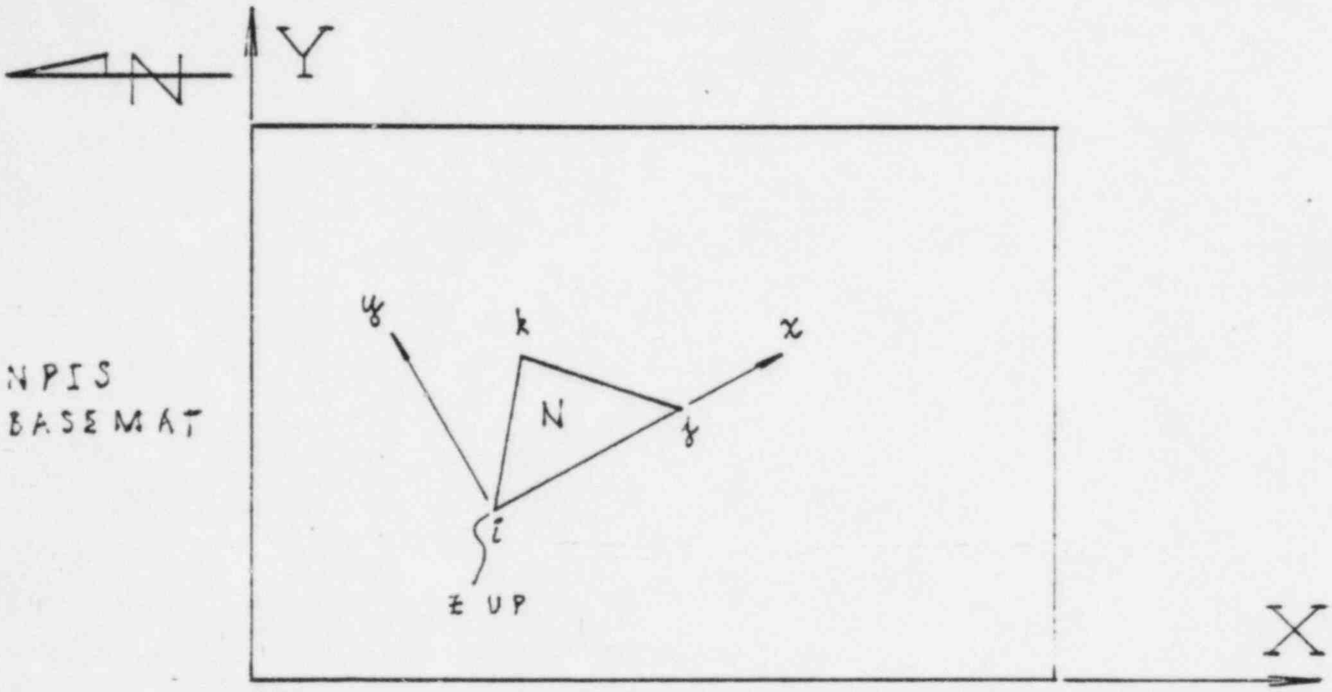
CONTOUR LEVELS

MIN	-.50387E-01
A	-.50387E-01
B	-.45123E-01
C	-.39859E-01
D	-.34595E-01
E	-.29330E-01
F	-.24066E-01
G	-.18802E-01
H	-.13538E-01
I	-.82735E-02
J	-.30093E-02
MAX	-.30093E-02

PLOT OF   NORMAL DISPLACEMENT
STAR TAPE4 VECTOR NO   14

SCALE =   43.706

'MARTIN' TRIANGULAR ELEMENT



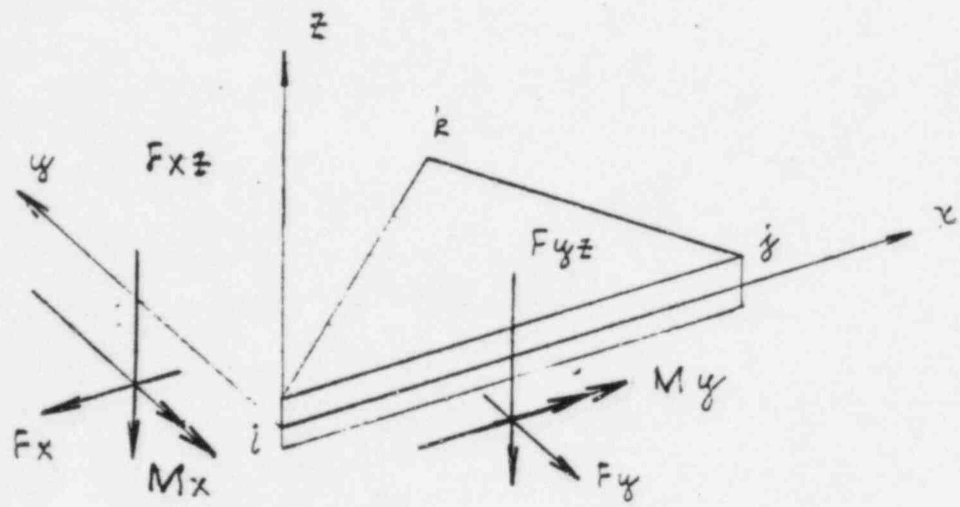
THE ABOVE SKETCH ILLUSTRATES A TYPICAL FINITE ELEMENT CONTAINED IN THE BASEMAT. THE NODE POINTS i, j, k ARE LOCATED BY SPECIFYING THEIR COORDINATE LOCATIONS IN TERMS OF THE GLOBAL COORDINATES X, Y, Z.

THE SEQUENCE IN WHICH THE CONNECTIVITY OF THIS ELEMENT 'N' IS SPECIFIED DETERMINES THE ORIGIN AND ORIENTATION OF THE LOCAL COORDINATE SYSTEM ASSOCIATED WITH THAT ELEMENT. FOR A SEQUENCE 'i', 'j', 'k', THE ORIGIN IS FIXED AT 'i', AND THE LOCAL x AXIS IS THEN ALIGNED ALONG EDGE 'i-j', POSITIVE FROM 'i' TO 'j'. THE y AXIS IS SET PERPENDICULAR TO THE x AXIS IN THE PLANE OF THE ELEMENT, AND THE z AXIS THEN ACTS UPWARD IN ACCORDANCE WITH THE RIGHT-HAND RULE.

HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8304
	C- 2 - 1 - 5 (SUBJ. SUBDIV. SHEET)
PROJECT W3	
CLIENT LP&L	PREP. BY AGL DATE 09.27.83
SUBJECT BASEMAT INTERNAL FORCES	CHCKD. BY GH DATE 10/3/83

'MARTIN' TRIANGULAR ELEMENT, CON.

THE INTERNAL SHEARS AND MOMENTS ARE OUTPUT WITH RESPECT TO THE ELEMENT LOCAL COORDINATE SYSTEM, AS SHOWN IN THE FOLLOWING SKETCH:

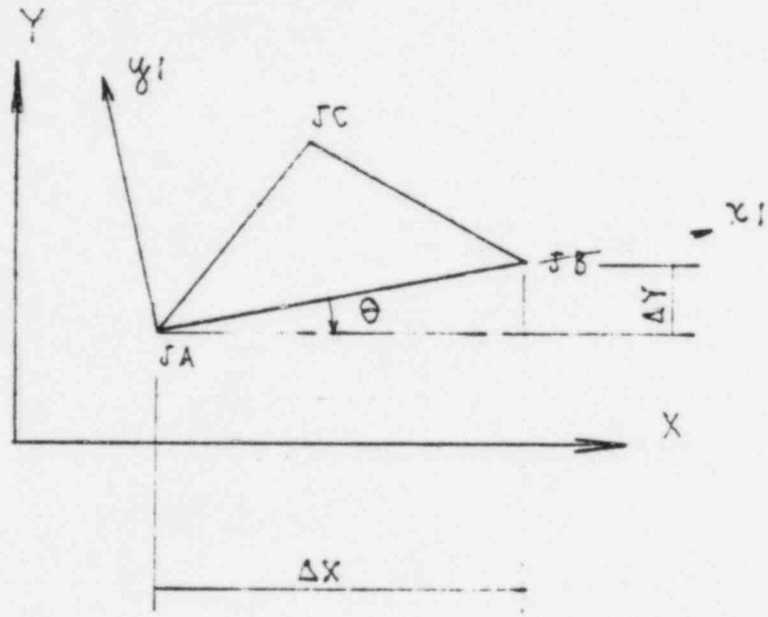


AS DEFINED ABOVE, Mx, My CAUSE TENSION AT THE MAT UPPER SURFACE, SO THAT THE TOP STEEL BECOMES THE TENSION STEEL.

AS NOTED PREVIOUSLY, THE LOCAL COORDINATE AXES OF A NUMBER OF BASEMAT ELEMENTS WERE RE-ORIENTED PARALLEL TO THE GLOBAL X, Y AXES TO SIMPLIFY A CHECK OF THE OUTPUT SHEARS AND MOMENTS. SEE THE FOLLOWING THREE PAGES.

HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8304 C- 2 - 1 - 6 SUBJ. SUBDIV. SHEET
PROJECT W3 CLIENT LP&L SUBJECT BASEMAT INTERNAL FORCES	PREP. BY AGZ DATE 09/23/83 CHCKD. BY GH DATE 10/3/83

COMPUTE 'AXIS ANGLE' FOR ELEMENTS HAVING ELEMENT COORDINATE AXIS X1 SKEW WITH RESPECT TO GLOBAL X AXIS. FOR THE DEFINITION OF 'AXIS ANGLE' REFERENCE THE STARDYNE USER INFORMATION MANUAL, PAGE 31-130, SEP/79.



FROM THE SKETCH,

$$\Delta Y = Y_{JB} - Y_{JA}$$

$$\Delta X = X_{JB} - X_{JA}$$

$$\theta = -\tan^{-1}\left(\frac{\Delta Y}{\Delta X}\right)$$

(θ DEFINED + COUNTER CLOCKWISE, ∴ - SIGN)

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 3304
C- 2 - 1 - 7
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

PREP. BY AGB DATE 09/13/83
CHCKD. BY AV DATE 9/13/83

ELMD	JA	JB	JAX	JAY	JBX	JBY	ΔX	ΔY	θ (DEG.)
183	33	179	86.42	87.62	66.50	88.75	-19.92	1.13	3.25
188	202	33	80.00	69.25	86.42	87.62	6.42	18.37	-70.74
189	179	33	66.50	88.75	86.42	87.62	19.92	-1.13	3.25
~~192~~									
~~195~~									
196	30	29	75.70	155.33	86.42	176.38	10.72	21.05	-63.01
197	183	29	66.50	164.75	86.42	176.38	19.92	11.63	-30.28
226	34	202	103.12	70.92	80.00	69.25	-23.12	-1.67	-4.13
230	218	34	98.50	47.75	103.12	70.92	4.62	23.17	-78.72
231	34	33	103.12	70.92	86.42	87.62	-16.70	16.70	45.00
235	34	218	103.12	70.92	98.50	47.75	-4.62	-23.17	-78.72
245	35	34	124.17	60.20	103.12	70.92	-21.05	10.72	26.99
247	233	35	120.50	47.75	124.17	60.20	3.67	12.45	-73.58
261	36	35	147.50	56.50	124.17	60.20	-23.33	3.70	9.01
270	86	246	158.50	52.70	144.50	47.75	-14.00	-4.95	-19.47
273	258	86	169.00	32.50	158.50	52.70	-10.50	20.20	62.53
274	246	86	144.50	47.75	158.50	52.70	14.00	4.95	19.47
~~278~~									
285	258	268	169.00	32.50	193.50	44.00	24.50	11.50	-25.14
286	85	86	176.80	57.00	158.50	52.70	-18.30	-4.30	-13.22
~~287~~									
~~288~~									
289	84	85	193.50	66.00	176.80	57.00	-16.70	-9.00	-28.32
~~290~~									
~~308~~									
~~310~~									
~~311~~									
~~333~~									
~~334~~									
~~335~~									
~~338~~									
~~337~~									
~~340~~									
~~342~~									

HARSTEAD ENGINEERING ASSOCIATES • INC.	PROJ. NO. 8304
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	C- 2 - 1 - 8
	SUBJ. SUBDIV. SHEET
PROJECT W3	PREP. BY AJ DATE 9/13/83
CLIENT LP&L	CHCKD. BY AdL DATE 09/13/83
SUBJECT BASEMAT INTERNAL FORCES.	

ELNO	JA	JB	JAX	JAY	JBX	JBY	LX	LY	θ
~~345~~									
~~346~~									
~~350~~									
~~351~~									
~~352~~									
~~356~~									
~~357~~									
~~358~~									
~~389~~									
390	25	72	170.8	203.8	193.5	200.0	22.7	-3.8	9.5
391	269	259	193.5	220.0	169.0	223.5	-24.5	3.5	8.1
392	259	269	169.0	223.5	193.5	220.0	24.5	-3.5	8.1
398	26	25	147.5	207.5	170.8	203.8	23.3	-3.7	9.0
399	25	259	170.8	203.8	169.0	223.5	-1.8	19.7	+84.8
400	26	253	147.5	207.5	163.5	213.0	16.0	5.5	-19.0
410	26	247	147.5	207.5	144.5	223.5	-3.0	16.0	79.4
413	27	26	124.2	203.8	147.5	207.5	23.3	3.7	- 9.0
419	234	27	120.5	223.5	124.2	203.8	3.7	-19.7	79.4
422	27	234	124.2	203.8	120.5	223.5	-3.7	19.7	79.4
425	28	27	103.1	193.1	124.2	203.8	21.1	10.7	-26.9
~~435~~									
436	203	28	80.0	180.3	103.1	193.1	23.1	12.8	- 29.0
450	33	34	86.4	87.6	103.1	70.9	16.7	-16.7	45.0
451	32	33	75.7	108.7	86.4	87.6	10.7	-21.1	63.1
~~474~~									
~~482~~									
~~491~~									
530	297	312	243.5	0.	275.5	22.0	32.0	22.0	-34.5
531	298	312	243.5	16.3	275.5	22.0	32.0	5.7	- 10.1
553	309	321	243.5	234.0	275.5	228.0	32.0	-6.0	10.6
401	253	259	163.5	213.0	169.0	223.5	5.5	10.5	-62.4

HEA **HARSTEAD ENGINEERING ASSOCIATES • INC.** 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8304
	C- 2 - 1 - 9 (SUBJ. SUBDIV. SHEET)
PROJECT W3	
CLIENT LP&L	PREP. BY AdB DATE 09/27/83
SUBJECT BASEMAT INTERNAL FORCES.	CHCKD. BY GH DATE 10/3/83

REBAR PATTERN

THE REINFORCING STEEL CONTAINED IN THE BASEMAT IS DETAILED ON THE FOLLOWING EBASCO DRAWINGS:

1. EBASCO DRAWING 'COMMON FOUNDATION STRUCTURE REINFORCING SH. 1', LOU-1564-G-500S01, REV. 9, 12/12/78.
2. EBASCO DRAWING 'COMMON FOUNDATION STRUCTURE REINFORCING SH. 2', LOU-1564-G-500S02, REV. 2, 01/20/75.
3. EBASCO DRAWING 'COMMON FOUNDATION STRUCTURE REINFORCING SH. 3' LOU-1564-G-500S03. REV. 3, 05/09/75

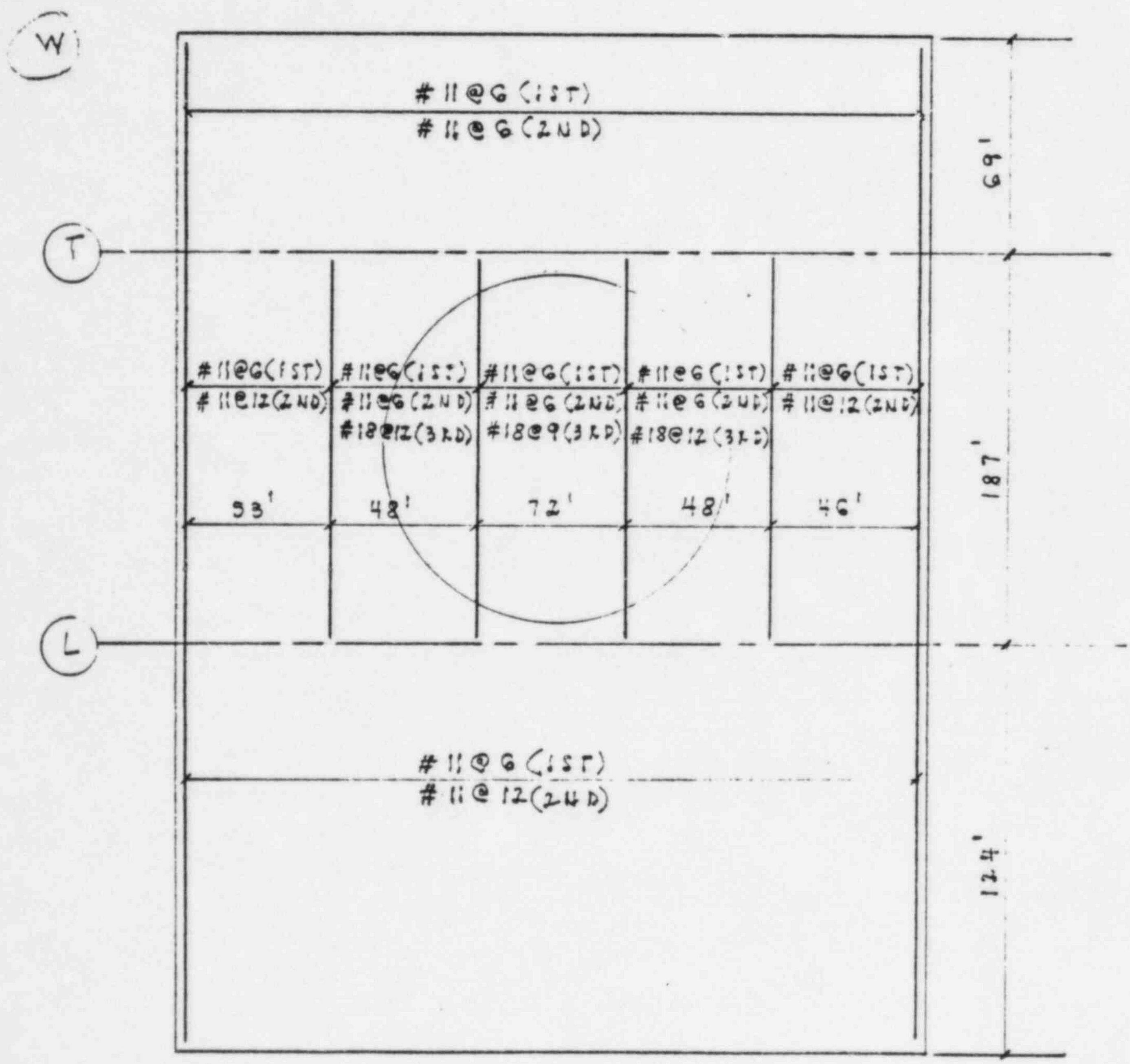
AS SHOWN THEREON, THE TOP STEEL IS #11 @ 6" EACH WAY, OVER THE ENTIRE MAT.

THE BOTTOM STEEL VARIES IN SIZE AND NUMBER WITH RESPECT TO LOCATION AT THE BOTTOM OF THE MAT.

PLOTS OF THE N-S BOTTOM REINFORCEMENT, E-W BOTTOM REINFORCEMENT, AND SHEAR REINFORCEMENT ARE SHOWN ON THE NEXT THREE PAGES.

HEA — HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8307 C- 2 - 1 - 10 SUBJ. SUBDIV. SHEET
PROJECT W3 CLIENT LP&L SUBJECT BASEMAT INTERNAL FORCES	PREP. BY AGL DATE 09/27/83 CHCKD. BY GH DATE 10/3/83

BOTTOM REINFORCEMENT (N-S)



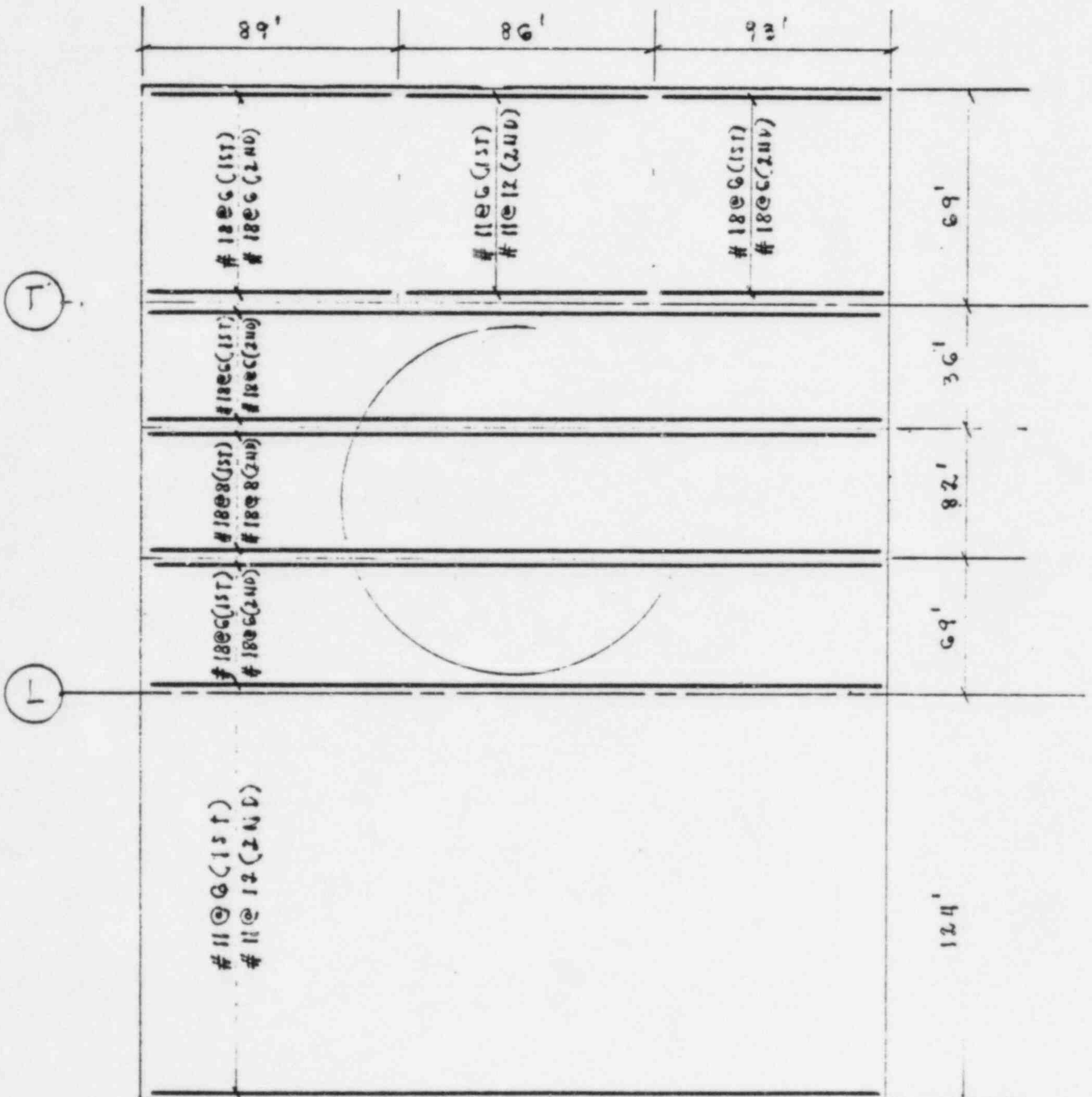
(DIMENSIONS SCALED OFF REBAR DWGS. THIS SKETCH NOT TO SCALE).

HEA
HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	5304
C- 2 - 1 - 11	
SUBJ. SUBDIV. SHEET	
PREP. BY AGS	DATE 09/27/83
CHCKD. BY GH	DATE 10/3/83

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

BOTTOM REINFORCEMENT (E-W)



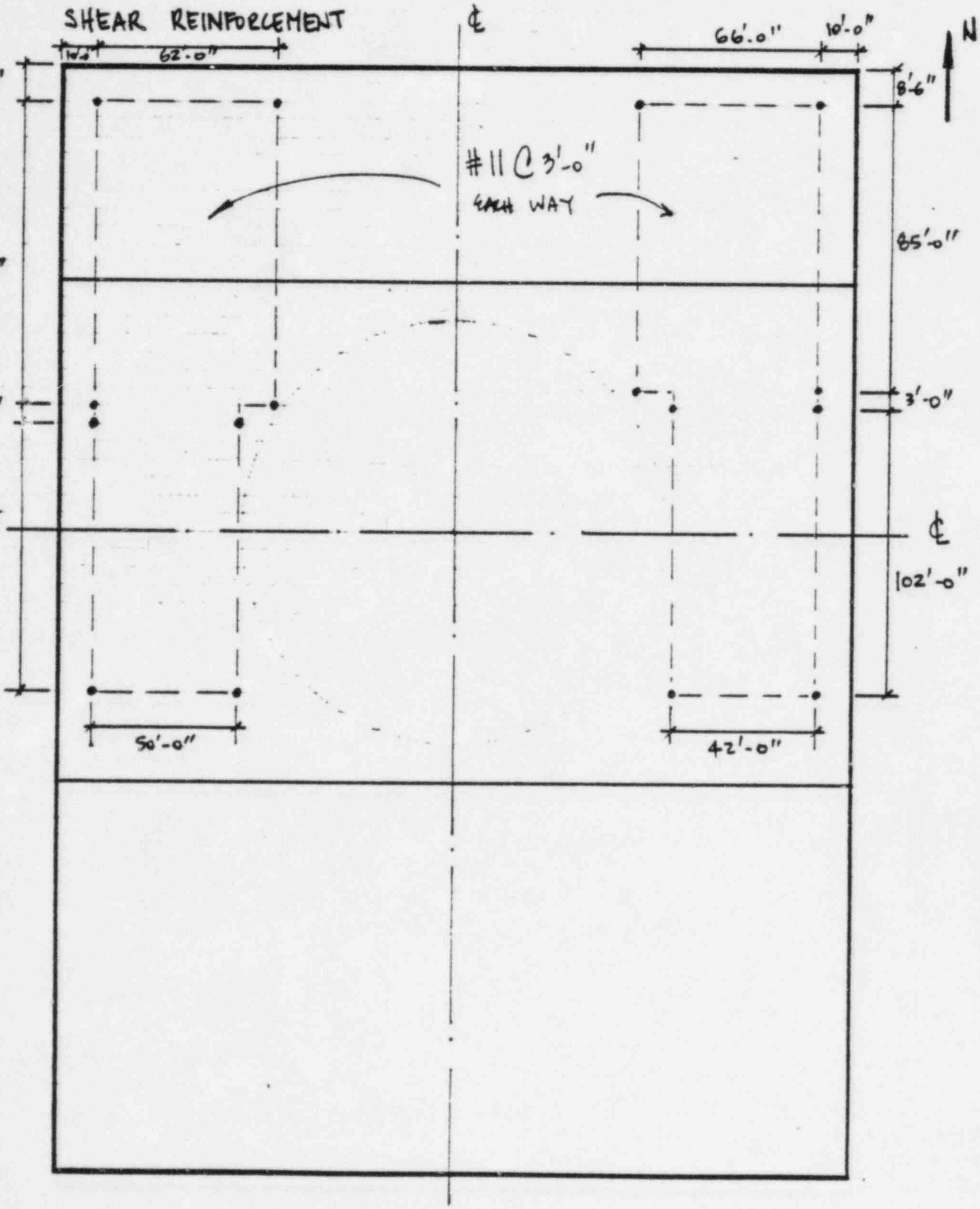
(DIMENSIONS SCALED OFF REBAR DWGS. THIS SKETCH NOT TO SCALE).

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	8304
C- 2 - 1 - 12	SUBJ. - SUBDIV. - SHEET
PROJECT	W3
CLIENT	LP&L
SUBJECT	BASEMAT INTERNAL FORCES
PREP. BY AU	DATE 2/11/83
CHCKD. BY AGB	DATE 08/11/83

SHEAR REINFORCEMENT



REBAR CAPACITY (SHEAR)

WHERE NO SHEAR REINFORCEMENT IS PROVIDED, THE SHEAR CAPACITY OF THE MAT PER FOOT OF WIDTH IS COMPUTED AS FOLLOWS (REFERENCE CHAPTER 11 OF ACI 318-71):

$$v_c = 2\sqrt{f'_c} = 2\sqrt{4000}$$
$$= 127 \text{ PSI} = 0.127 \text{ KSI}$$

$$V_c = \phi\, v_c\, b_w\, d$$
$$= 0.85 \times 0.127 \times 12 \times 133$$
$$= 172 \text{ K/FT}$$

THE ADDITIONAL SHEAR CAPACITY PROVIDED BY #11 @ 3' EACH WAY IS:

$$v_c(\text{reinf.}) = \frac{1.56\text{ IN}^2 \times 60\text{ KSI}}{36\text{ IN} \times 36\text{ IN}}$$
$$= 0.072 \text{ KSI}$$

SO THAT:

$$v_u = 0.127 + 0.072$$
$$= 0.199 \text{ KSI}$$

THEN:

$$V_u = 0.85 \times 0.199 \times 12 \times 133$$
$$= 270 \text{ K}$$

SINCE THE SHEAR REINFORCEMENT IN THE BASEMAT IS ORIENTED VERTICALLY, AND IS SPACED SYMMETRICALLY, THE ABSOLUTE VALUE OF THE SHEAR DERIVED FROM ANALYSIS MAY BE COMPARED DIRECTLY WITH THE SHEAR CAPACITY AT ANY GIVEN LOCATION ON THE BASEMAT.

SHEAR REINF INCREASES CAPACITY IN ONLY ONE DIRECTION

HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 3304
	C- 2 - 1 - 14 (SUBJ. SUBDIV. SHEET)
PROJECT W3	
CLIENT LP&L	PREP. BY AGZ DATE 09.27.83
SUBJECT BASEMAT INTERNAL FORCES	CHCKD. BY GH DATE 10/3/83

REBAR CAPACITY (MOMENT)

THE MOMENT CAPACITIES FOR MOST OF THE REBAR HAVE ALREADY BEEN COMPUTED BY EBASCO. REFERENCE EBASCO CALCULATION ENTITLED 'COMM FDN MAT - MOMENT CAPACITY', OFS NO. 5234.014, DEPT. NO. 650, SHEET E55, 05/11/81.

USING ACI PUBLICATION SP-17(73), FLEXURE 7.6.1, AN ADDITIONAL PAIR OF MOMENT CAPACITIES ARE COMPUTED:

#11 @ 6 N-S (FROM EBASCO CALC.,
#11 @ 6 BOT USE d = 131.03 IN.)
#18 @ 12

$A_s = 4 \times 1.56 + 4.00 = 10.24 \text{ IN}^2$ ✓

$M_u = 4.50 \times 10.24 \times 131.03 - 3.32 \times 10.24^2$
$M_u = 5690 \text{ FT}\cdot\text{K/FT}$

#18 @ 6 E-W (FROM EBASCO CALC.,
#18 @ 6 BOT USE d = 136.37 IN.)

$A_s = 4 \times 4.00 = 16.00 \text{ IN}^2$

$M_u = 4.50 \times 16.00 \times 136.37 - 3.32 \times 16.00^2$
$M_u = 8969 \text{ FT}\cdot\text{K/FT}$

A COMPLETE TABULATION OF THE MOMENT CAPACITIES CALCULATED BOTH BY EBASCO AND HEA IS PROVIDED ON THE NEXT PAGE.

HEA · HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT: W3	PROJ. NO. 3304
CLIENT: LP&L	C- 2 - 1 - 15 (SUBJ. SUBDIV. SHEET)
SUBJECT: BASEMAT INTERNAL FORCES	PREP. BY AGB DATE 09/27/83
	CHCKD. BY GH DATE 10/5/83

REBAR CAPACITY (MOMENT)

REINFORCEMENT	ORIENTATION	LOCATION	MOMENT CAP.
2-#11@6 #18@9	N-S	BOT	6382 K'/'
2-#11@6	N-S	BOT	3643 K'/'
2-#18@8	E-W	BOT	6889 K'/'
#11@6	E-W	TOP	1935 K'/'
#11@6	N-S	TOP	1915 K'/'
#11@6 #11@12	N-S	TOP	2847 K'/'
2-#11@6 #18@12	N-S	BOT	5690 K'/'
2-#18@6	E-W	BOT	8969 K'/'

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	8304
C- 2 - 2 - 14	SUBJ. SUBDIV. SHEET
PREP. BY AGZ	DATE 09.27.83
CHCKD. BY GH	DATE 10/5/83

PROJECT

CLIENT LP&L

SUBJECT BASEMAT INTERNAL FORCES

COMPARISON OF ANALYSIS FORCES AND MOMENTS AND DESIGN ALLOWABLES, CON.

REVIEW CRITERIA

ONLY ANALYSIS SHEARS LOCATED A DISTANCE ±d/2 FROM THE SHIELD BUILDING INNER & OUTER WALL ARE EVALUATED AGAINST DESIGN SHEAR CAPACITY.

FOR SUCH ANALYSIS SHEARS, 'NA', NOT APPLICABLE, IS ENTERED IN THE COMMENTS COLUMN.

HE

HEA HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 3324
C- 2 - 3 - 1
SUBJ. SUBDIV. SHEET

PROJECT W3
CLIENT LP&L
SUBJECT BASEMAT INTERNAL FORCES

PREP. BY GH DATE 10/5/83
CHCKD. BY AdB DATE 10/5/83

COMPARISON OF ANALYSIS FORCES AND MOMENTS AND DESIGN ALLOWABLES, CON.

NORMAL OPERATION $C = 1.5(D+L'') + 1.8(L+S1) + 1.055\,4B$

ELEM. ID	FXZ	FYZ	MX	MY	COMMENTS	FX	FY
172	+376	+120	-332	-993	OK SEE PAGE 8	-250	-255
199	-397	75	-897	-1181	OK SEE PAGE 7	-268	-249
278	-237	8	-157	-513	<270 OK	-205	-96
353	180	24	-546	-459	>172 OK *	-131	-131
352	-200	-263	-635	-2103	NA		
133	-195	28	-197	-102	>172 OK SEE P.9	-172	-120
136	-205	-19	-454	-532	>172 "	-159	-128
146	205	63	-211	-176	>172 "	-177	-143
150	132	115	-467	-1198	>172 "	-132	-138
134	-221	13	-516	-431	>172 "	-163	-115
138	-117	-204	-1636	-1519	>172 OK SEE P.10	-206	-234
192	-28	190	-790	-3376	>172 NA	-149	-192
193	8	-205	-970	-3645	>172 NA	-123	-135
211	236				<270 OK		
243		172			NA		
260		-181			NA		
261		197			NA		
274		-202			NA		
275		279			NA		
287		-237			NA		
288		197			NA		
290		205			NA		
295	-187	63	-226	-189	>172 OK SEE P.9	-76	-93
308	-60	220			NA		
311		-229			NA		

* BY INSPECTION