MOVABLE BRIDGE SYMPOSIUM

SESSION C - STRUCTURES

November 4, 1985

Tallahassee, Florida

Protection of Steel Members

In Existing Movable Bridge Structures

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Introduction:

In recent years the maintenance painting protection of steel members in existing movable bridge structures has taken on new significance due to rising costs and more governmental regulations. Typical job costs for the complete repainting of a movable bridge structure now range between $1.50 to $2.00 per square foot. With more stringent enforcement of pollution regulations thesecosts could increase to as high as $6.00 per square foot in the not to distant future. How to keep the corrosion protection painting costs of movable bridge structures at a reasonable level through the use of better maintenance practices and better coating materials is the main objective of our presentation today. *Maintenance Spot Repair Painting*

Most of the existing movable bridge structures in Florida have been painted with a coating system that consists of an inorganic zinc primer, a vinyl intermediate coat and a high build vinyl finish coat. When this type of coating system is used in a marine environment localized spot rusting will usually appear within three to five years from the time the coating system was applied.

Spot repair painting of localized rusted areas with paint products requiring minimal surface preparation (power tool cleaning, water blast cleaning, light sandblast cleaning) will extend the life of the existing coating system and reduce overall long-term maintenance costs.

From our field testing experience the coating products that look promising for minimal surface preparation maintenance painting repair work include the following: high build epoxy coatings, urethane coatings, catalyzed vinyl coatings and some of the recently developed non-leaded oil base and water base coatings. The high build epoxy coatings have the advantage of low solvent emission levels and they can provide good dry film thickness (5 to 10 mils) in a single application. Coating products having low dry film build properties (2 to 3 mils) usually require at least a two coat application to be effective. Some of these products can also be used in the repair of corroded galvanized and aluminum surfaces.

For movable bridge structures having existing inorganic zinc primer vinyl finish systems it is recommended that the spot rust repair painting work be accomplished between five and six years from the time of the initial painting. Guideline specifications for the repair painting of a typical bascule bridge structure are outlined below.

1. Cleaning and Painting

All rusted and corroded surfaces, surfaces with lifted paint and surfaces with loose paint shall be cleaned as indicated undersurface preparation requirements. After cleaning, the surfaces shall be painted (brush or spray) with an approved high build aluminum epoxy coating. The aluminum epoxy coating shall be applied in a single application to obtain a dry film thickness ranging between 5 and 8 mils. The aluminum epoxy coating shall not be applied unless the surface temperature is a minimum of 5°F above the Dew Point. Surfaces not painted on the same day the cleaning is accomplished shall be recleaned prior to painting.

During all cleaning and painting operations, the Contractor shall isolate the work area with appropriate containment devices (canvasses, tarpaulins, screens, etc.) in order to prevent any generated debris from causing violations of current State of Florida air and water pollution regulation. The Contractor shall be responsible for the legal disposal of all debris collected by the containment devices.

2. <u>Surface Preparation</u>

a. Structural Steel (Excluding the top flanges of the bridge deck floor beams):

Surfaces shall be cleaned according to the SSPC-SP3-63 Specification (Power Tool Cleaning). Surfaces that are not accessible or practical for Power Tool cleaning shall be sandblast cleaned according to the SSPC-SP7-63 specification (Brush-off Blast Cleaning). If deemed appropriate by both the Engineer and the Contractor, high pressure water blast cleaning and vacuum blast cleaning may be substituted for Power Tool cleaning and sandblast cleaning.

b. Bridge Deck Grating, Sidewalk Grating, Platform Grating and Top Flanges of the Bridge Deck Floor Beams:

Surfaces shall be sandblast cleaned according to the SSPC-SP7-63 Specifications (Brush-off Blast Cleaning). If deemed appropriate by both the Engineer and the Contractor high pressure water blast cleaning may be substituted for sandblast cleaning.

c. Machinery Rooms:

Surfaces shall be cleaned according to the SSPC-SP3-63 Specifications (Power Tool Cleaning)

d. Metal Ladders, Signal Assemblies, Metal Railings:

Surfaces shall be cleaned according to the SSPC-SP3-63 Specifications (Power Tool Cleaning).

Slide Photograph Presentation

Slide No.	Structure and Location
1	Bascule, SR105, Sisters Creek
2	"
3	"
4	"
5	"
6	"
7	"
8	Bascule, SR40, St. Johns River
9	"
10	"
11	"
12	"
13	Swing Span, US17, St. Marys River
14	"
15	"

COMPUTER AIDED STRUCTURAL ANALYSIS OF THE HOPKINS FRAME

INTRODUCTION

This paper describes a computer aided finite element structural analysis of a Hopkins frame on the double leaf bascule Hillsboro Boulevard Bridge located in Deerfield Beach, Florida. Included in this paper are a brief discussion of the development of the finite element computer model, a discussion of the static loads that were used, a discussion of the results of the initial analysis, recommended design changes, a brief discussion of the development to accommodate these design changes, and a discussion of the results of subsequent analyses incorporating the design changes. The analyses were conducted using the SUPERSAP structural analysis package (available from Algor Interactive Systems in Pittsburgh, Pennsylvania), and were conducted on the PRIME 750 computer at the Civil Engineering and Mechanics Department at the University of South Florida in Tampa, Florida.

FINITE ELEMENT DEVELOPMENT

The finite element structural model was developed using rectangular cartesian coordinates referenced to the top left corner of the structure. The X-axis was defined downward; the Y-axis was defined to be from front to back; and the Z-axis was defined to be from left to right. All material properties chosen were those of steel with a modulus of elasticity of 30×10^6 pounds per square inch, Poisson's ratio of 0.3, and a density of 0.283 pounds per cubic inch.

The model was developed with 52 beam elements (capable of axial, shear, torsion, and/or bending), representing the six inch diameter drive shaft, high strength bolts, and six beam elements connected to the drive shaft on the ends of which were the applied static pinion gear loads (these last six beam elements were simply used for convenience to represent and transfer the pinion loads to the drive shaft and into the rest of the structure). There were also 182 six sided, eight noded solid elements which represented the four bearing blocks through which passed the six inch diameter drive shaft. Each of the eight nodes of these solid elements could translate in the X, Y, and Z directions, but could not rotate. Additionally there were 1165 plate elements (which could take membrane stresses and/or bending stresses) which were used to model the four wide flange vertical members, the channel at the top, the two channels with plates on the top near the bottom of the structure on the front and back, stiffeners in between flanges of the wide flange verticals, the four mounting brackets in the front where the bearing blocks were bolted, and the radius arms. Various computer generated drawings are shown in the following figures to illustrate these parts of the structural model. Figure 1 shows the beam elements, each labeled with the structural part they represent, and showing some

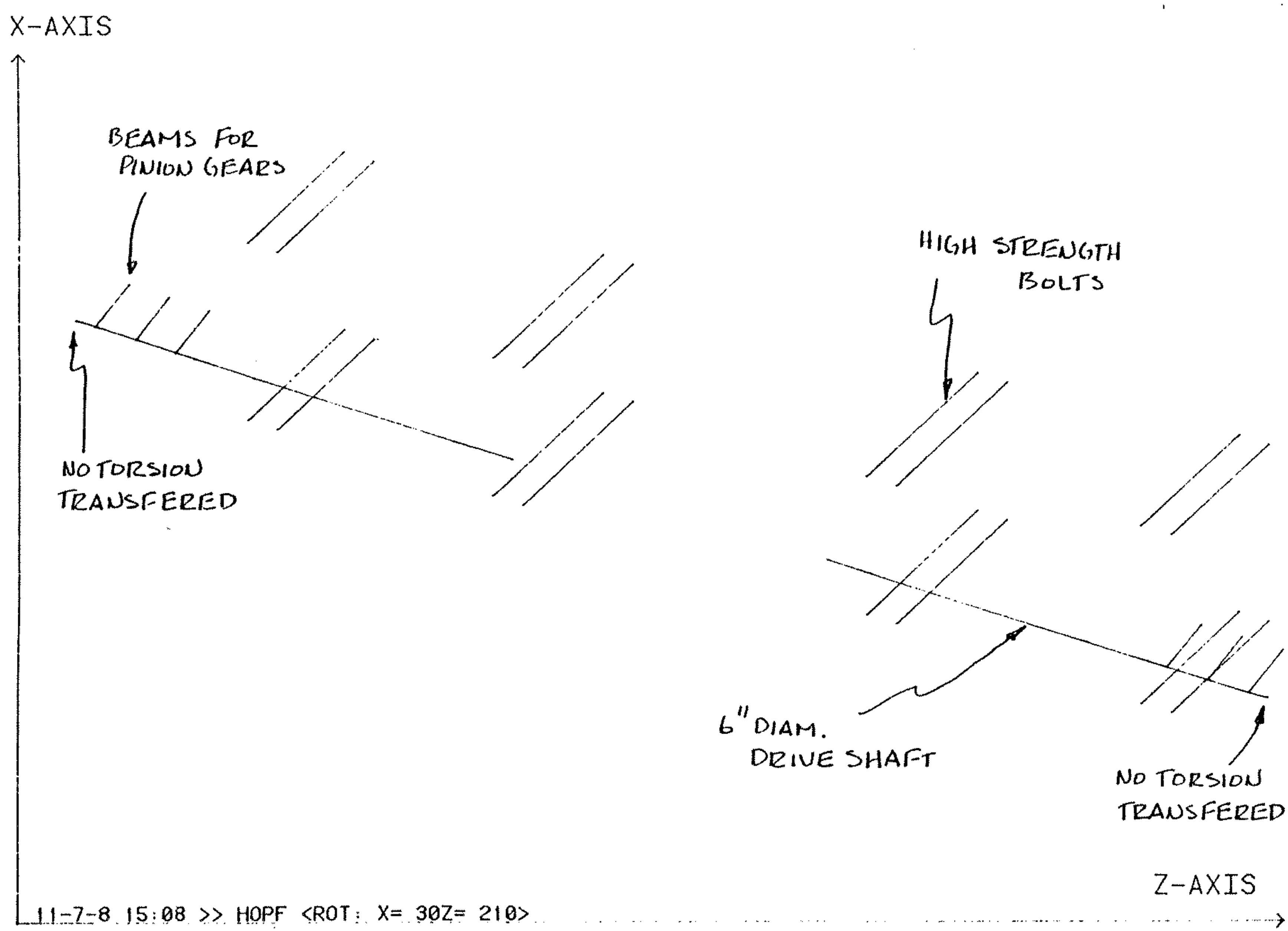

Figure 1. Beam elements in the structural model.

structural constraints that were used. Figure 2 shows the four bearing blocks through which passed the six inch diameter drive shaft. Figure 3 shows just the second of these bearing blocks for clarity. Figure 4 shows the four wide flange vertical elements (numbered) along with points of structural constraint. Figure 5 shows the upper channel member that was welded to the top of the four wide flange vertical members. Figure 6 shows the two channels on front and back of the four wide flange vertical members near the bottom of the frame. Figure 7 shows the four mounting brackets. Figure 8 shows the two radius arms along with points of structural constraint. Figure 9 shows the overall final finite element undeformed structural model that was analyzed under the pinion gear static loads.

The final model consisted of 1605 node points (8050 degrees of freedom), where the bottoms of the first and fourth wide flange vertical members were constrained so that they could not translate in any direction, and could not rotate except around the Z axis (around a line from left to right) as illustrated in Figure 4. Also the back ends of the radius arms, as illustrated in Figure 8, were similarly constrained. Additional constraints were made on the node points at the front of the radius arms as illustrated in Figure 8 so that they could not translate in the Z direction (left or right) or rotate around the Z direction (no torsion in the six inch diameter drive shaft would be transmitted into the radius arms). This model did not include the effect of the load input from the speed reducer or the stiffness afforded by the backing plate. It was included in a subsequent model, but will not be included in this paper.

As mentioned earlier, the pinion gear loads were accommodated by applying three equal loads at the ends of beam elements extending out from the six inch diameter drive shaft on the left and on the right of the frame. These three beam elements defined either edge and the center of the pinion gears. Two analyses were conducted, depending on the orientation of these loads. Loading condition one (representing opening of the spans) is shown in Figure 10. Loading condition two (representing closing of the spans) is shown in Figure 11.

RESULTS OF THE ORIGINAL ANALYSIS

Two loading conditions, as illustrated earlier, were analyzed (loading condition 1, 66780 pounds downward and forward; and loading condition 2, 66780 pounds upward and forward). The deformed structural model for these two loading conditions are illustrated in Figures 12 and 13. Figure 12 shows the deformed structural model from loading condition 1, and Figure 13 shows the deformed structural model from loading condition 2. In each of these two figures the deformations have been amplified by 150 for ease of viewing the deformed shapes. In each figure one will notice that the first

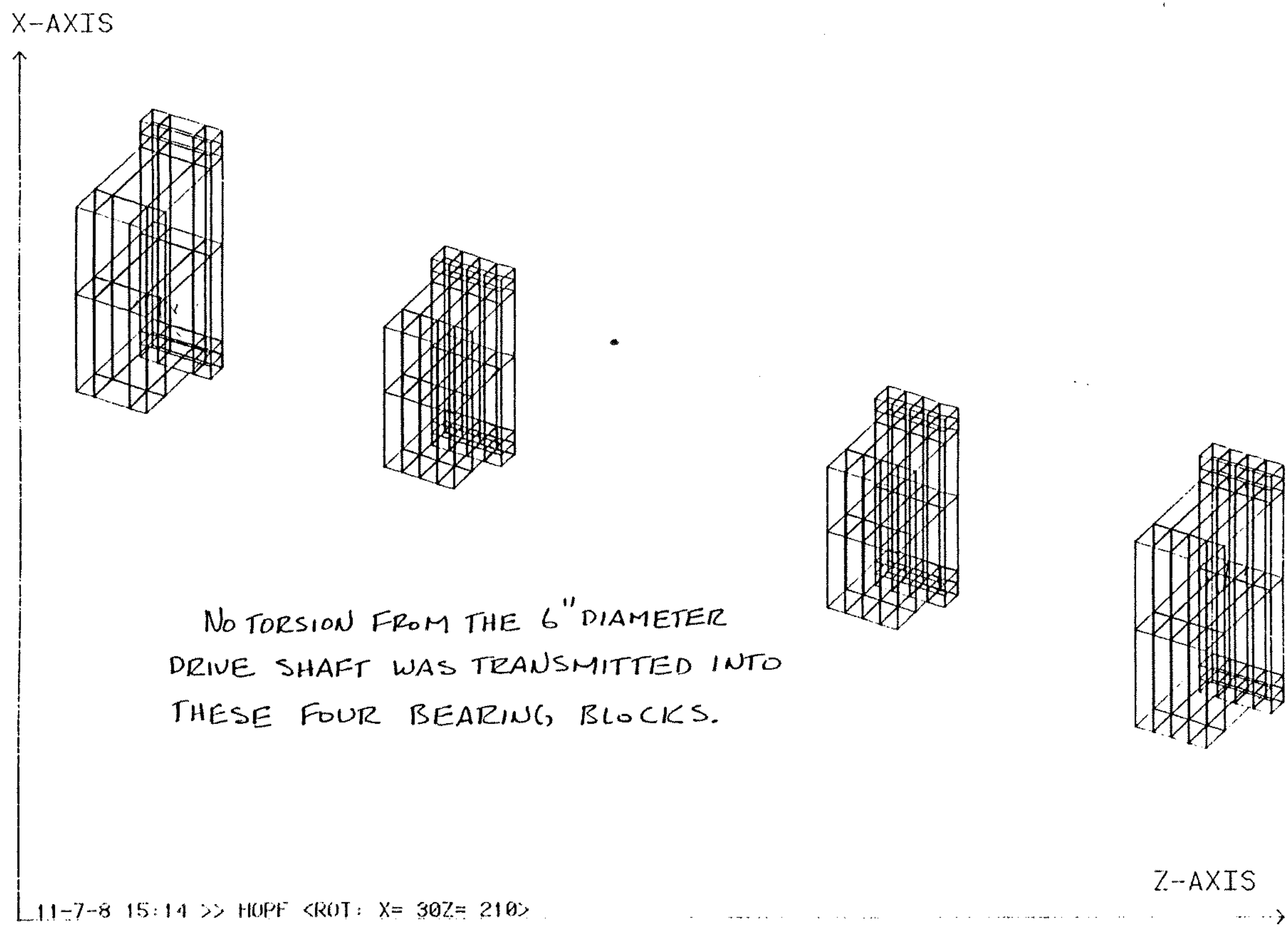

Figure 2. Bearing blocks (solid elements).

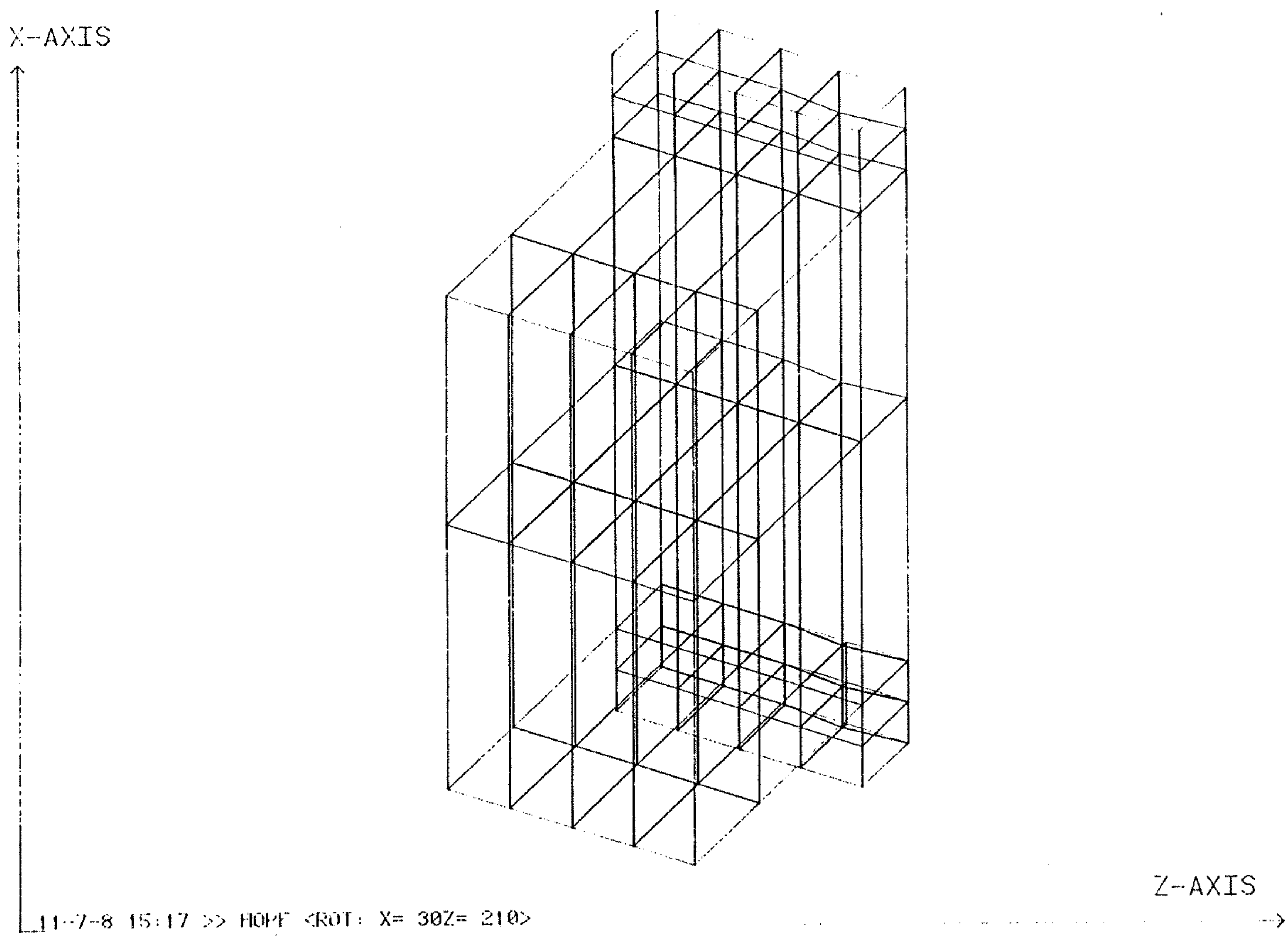

Figure 3. Bearing block number two (solid elements).

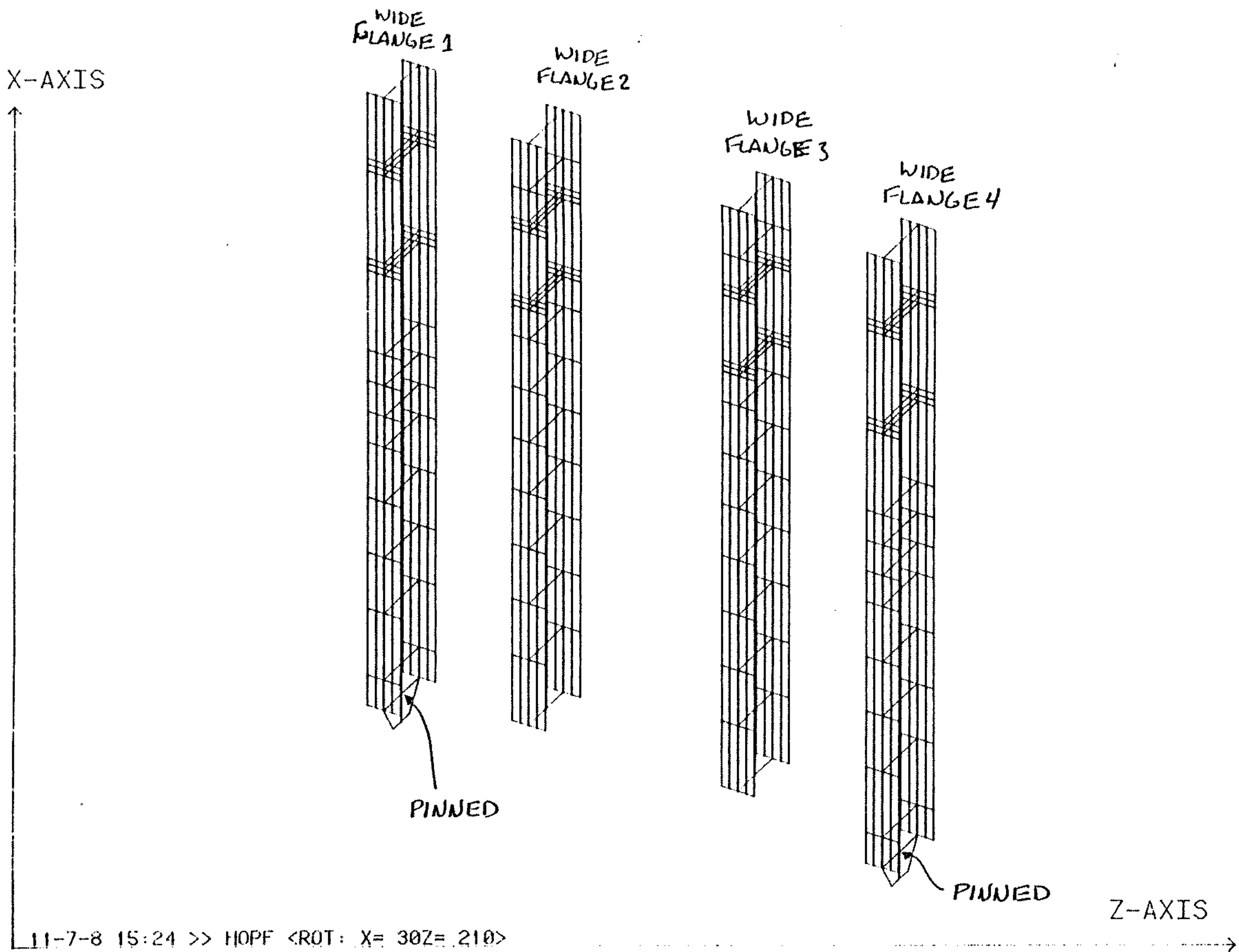

Figure 4. Four wide flange vertical members (plate elements).

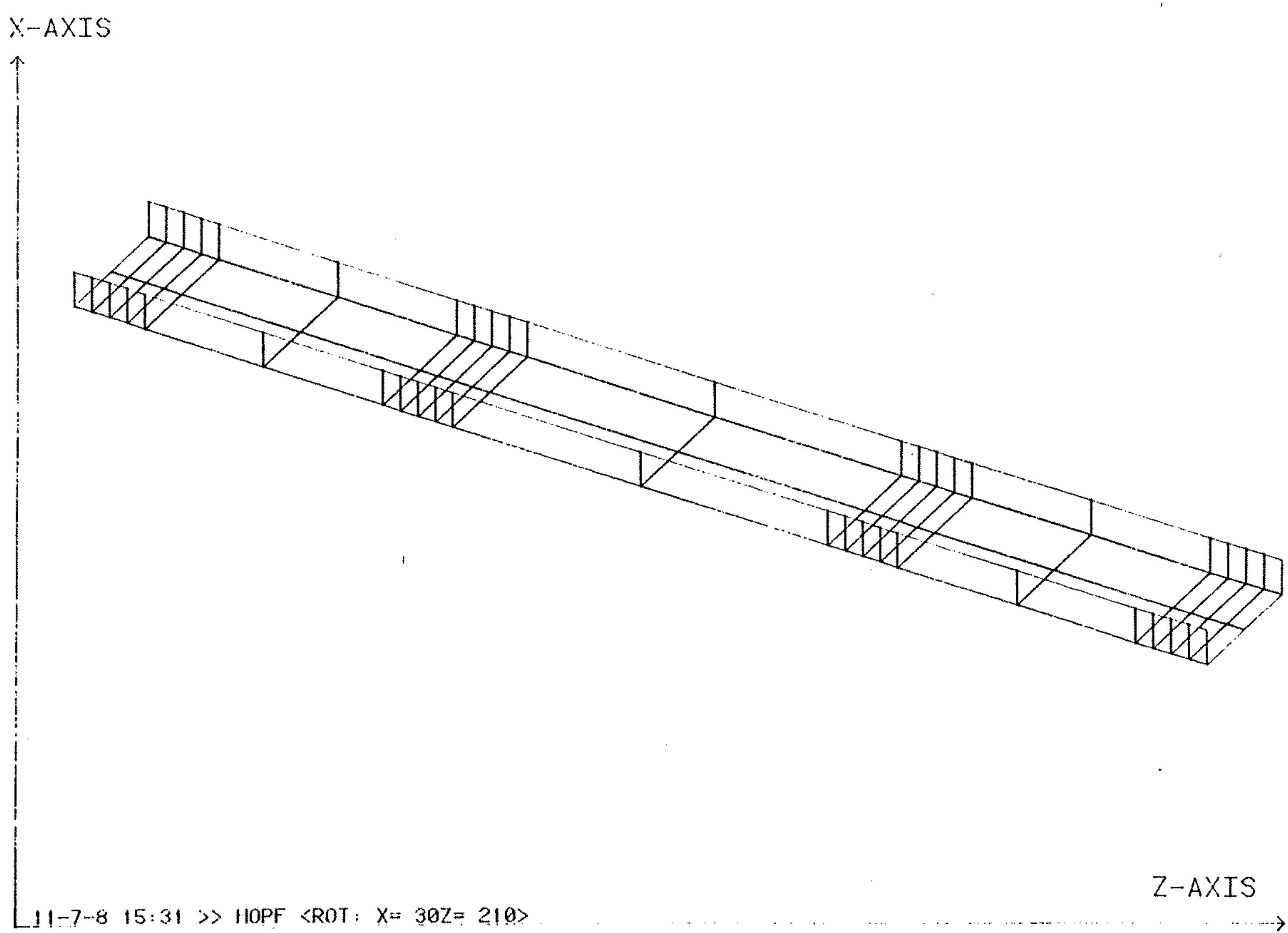

Figure 5. Upper channel member (plate elements).

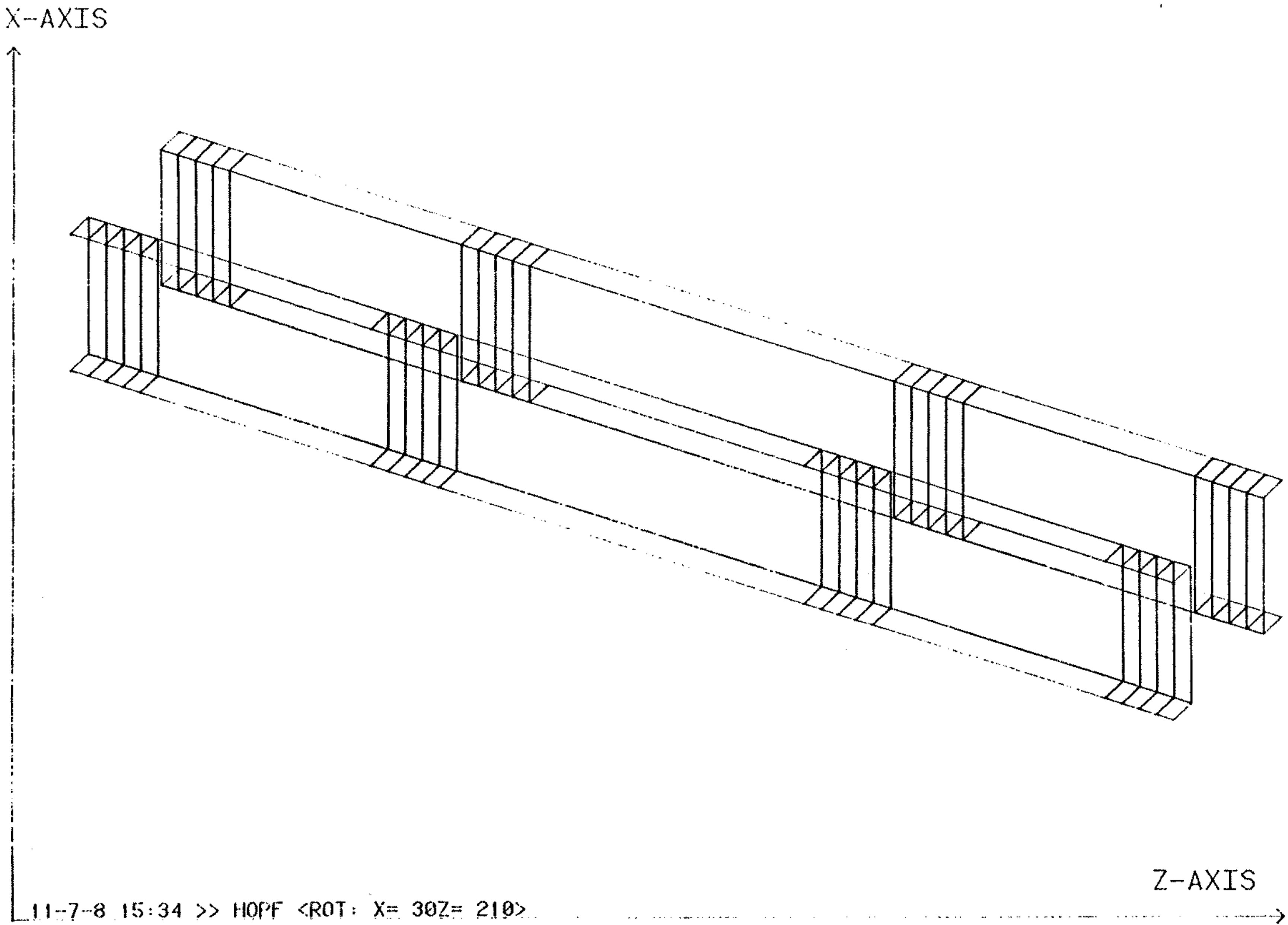

Figure 6. Two lower front and back channel members (plate elements).

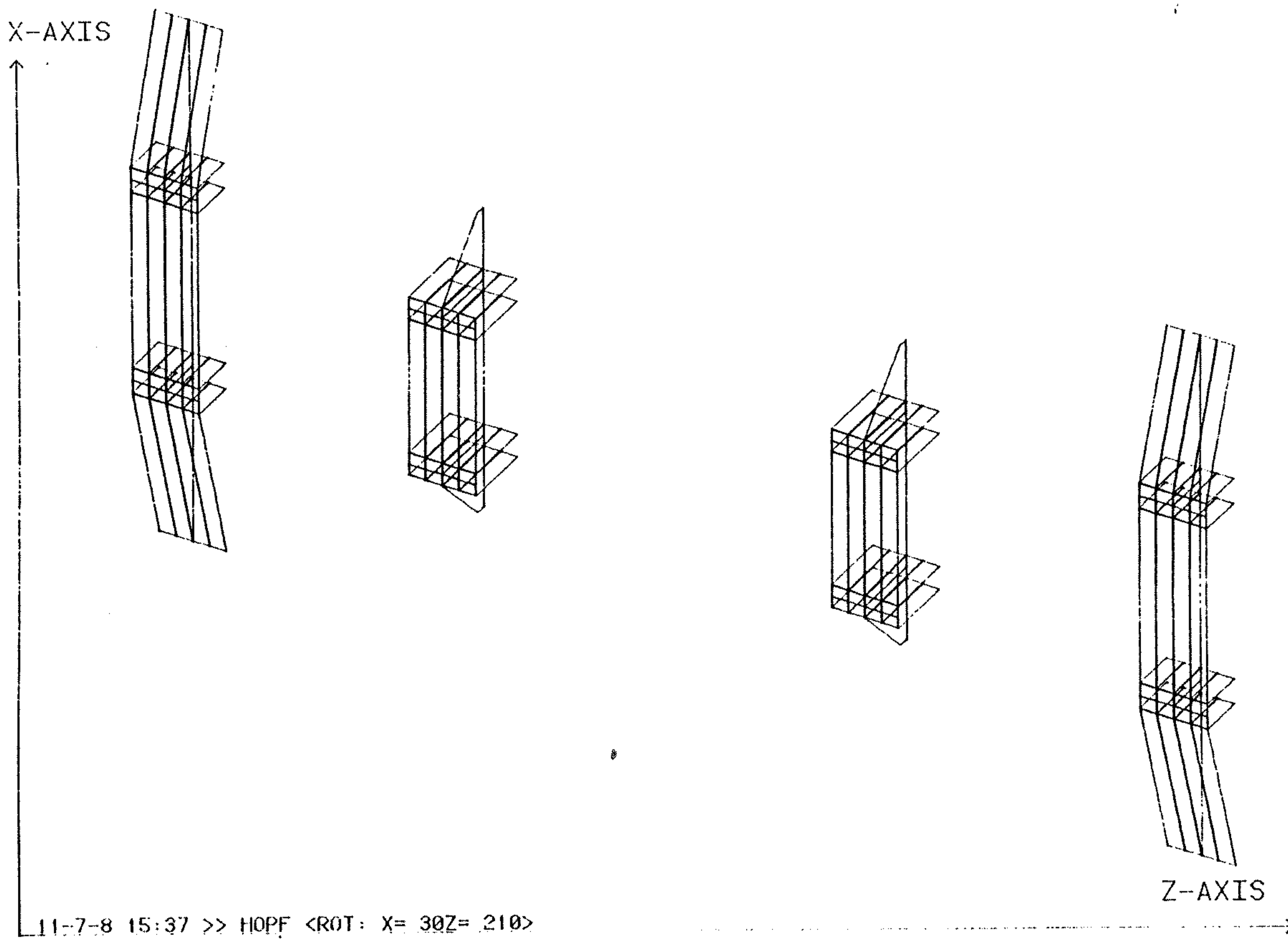

Figure 7. Four mounting brackets (plate elements).

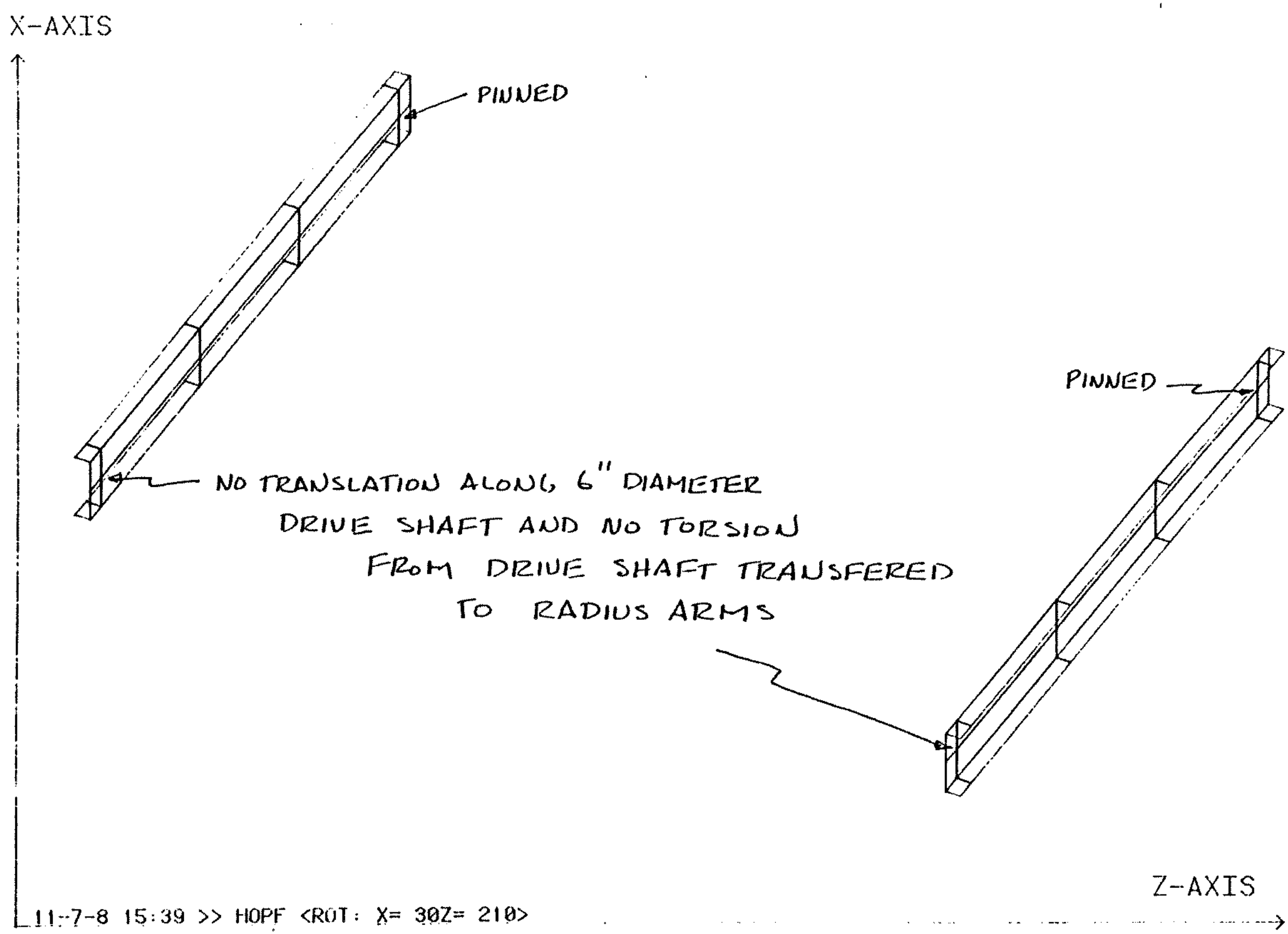

Figure 8. Two radius arms (plate elements).

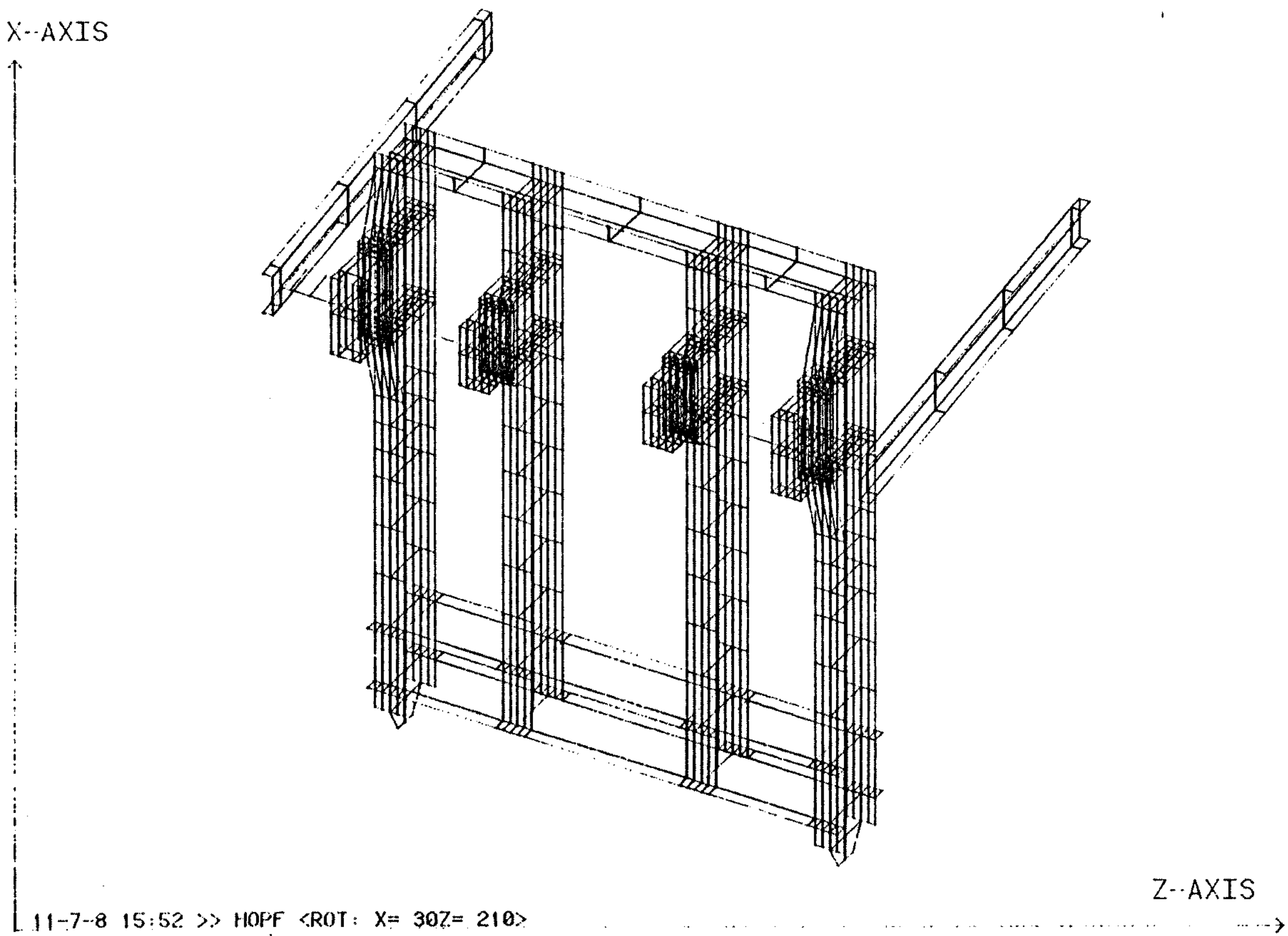

Figure 9. Final overall structural model of Hopkins frame.

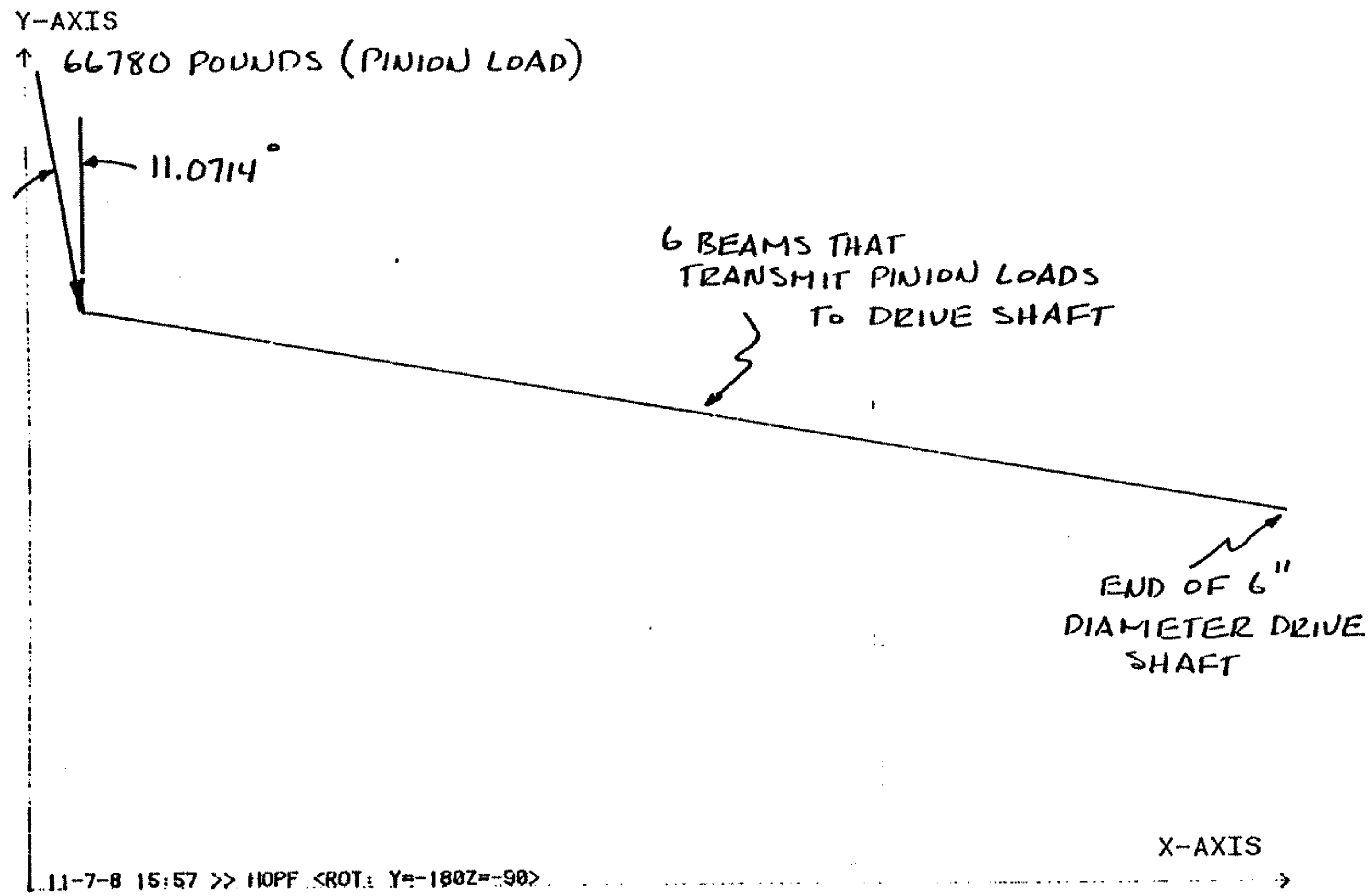

Figure 10. Loading condition 1; 66780 pounds 11.0714° from the vertical.

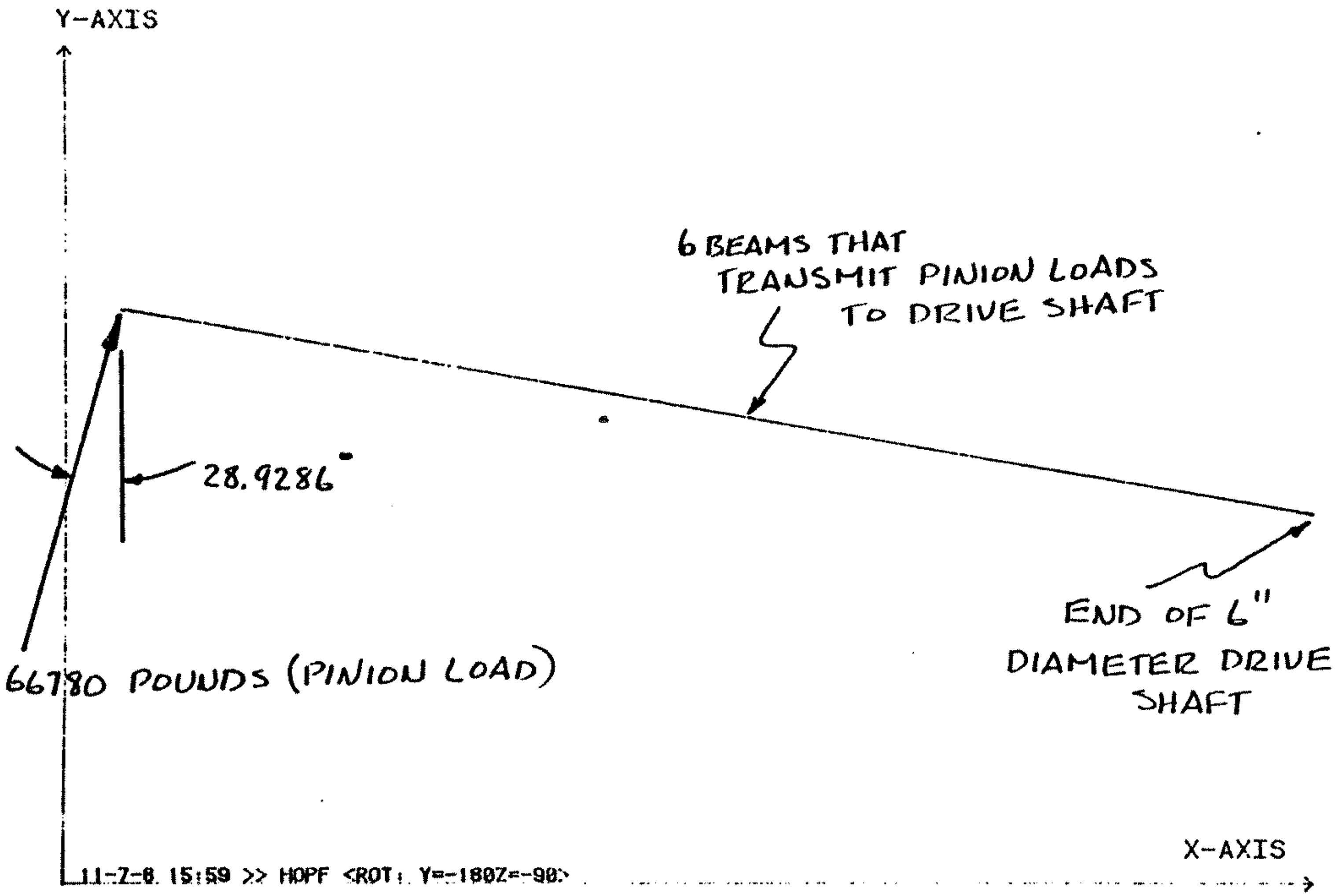

Figure 11. Loading condition 2; 66780 pounds 28.9286° from the vertical.

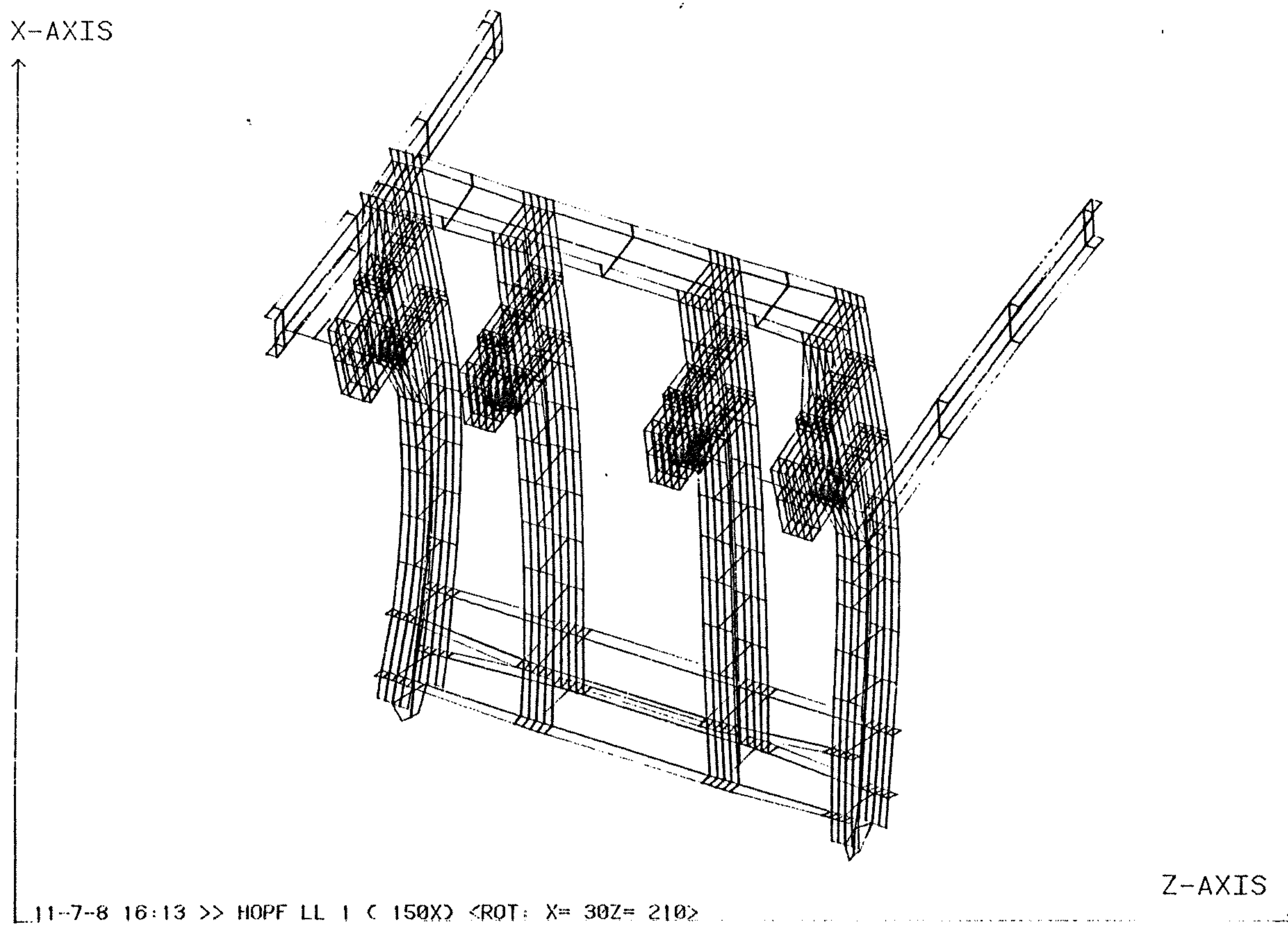

Figure 12. Deformed structural model from loading condition 1, amplified by 150.

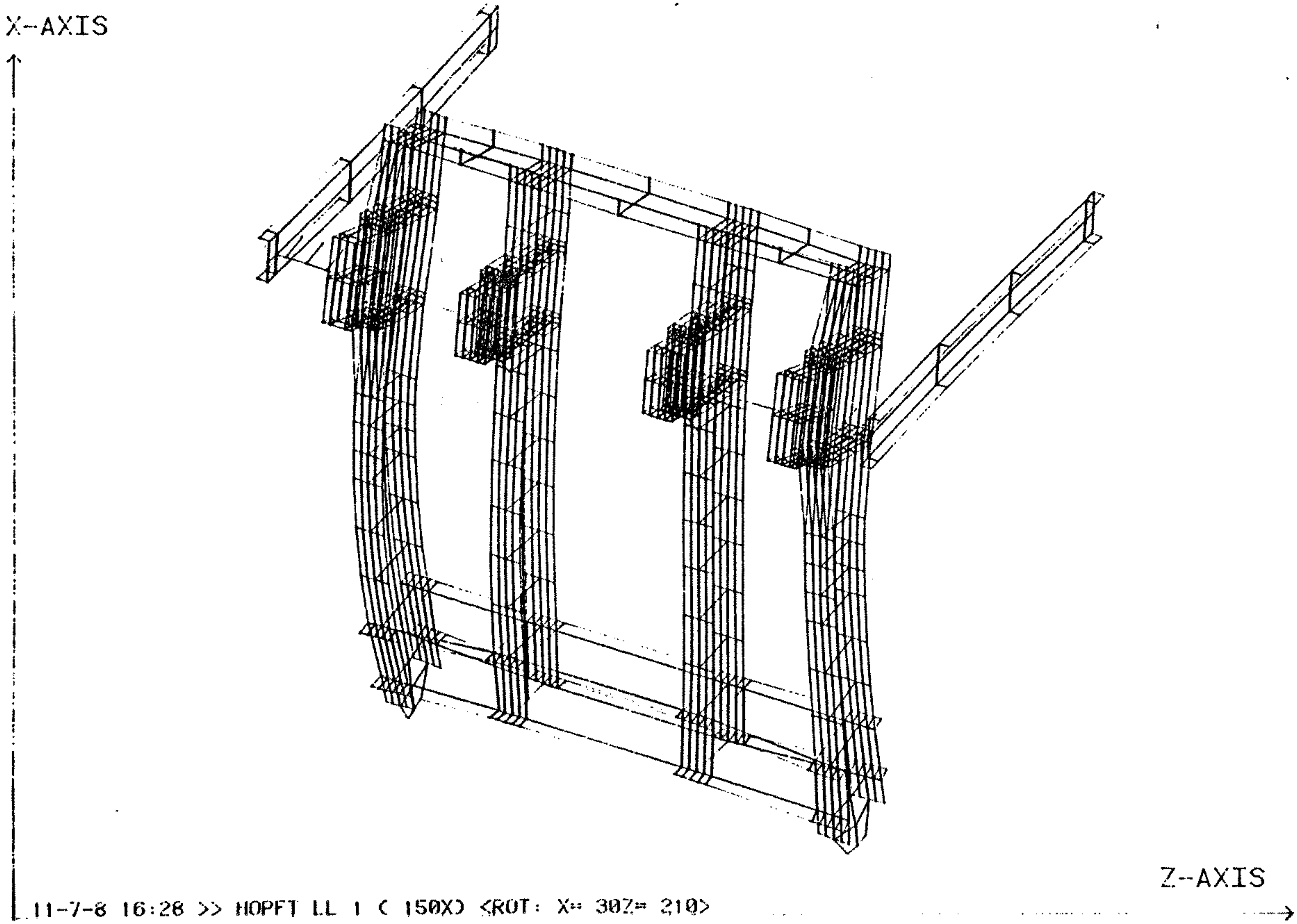

Figure 13. Deformed structural model from loading condition 2, amplified by 150.

and fourth wide flange vertical members are bent about the Z-axis (left to right). The maximum displacement backward of these wide flange vertical members for loading condition 1 was approximately 0.04 inches, while the maximum displacement frontward of these wide flange vertical members for loading condition 2 was approximately 0.06 inches.

The primary area of interest in evaluating the structural response of the Hopkins frame structural model was in the area of maximum stresses. According to the specifications a safety factor of at least three, based on the yield stress, was to be used in evaluating this response. The yield stress in the wide flange vertical members was reported as 36000 pounds per square inch, and the yield stress in the A325 high strength bolts was 81000 pounds per square inch. Table 1 shows a tabulated listing of all the major areas of maximum stress in the plate elements for loading condition 1. Shown are the approximate location of these maximum stresses in the structure, the plate element number in the model, their maximum stress value, and their corresponding safety factor based on the yield strength of this material.

Figure 14 shows plate elements 25 through 40 on the front of wide flange one where the highest plate element stresses were present. This area of the vertical wide flange is just below where the mounting brackets and bearing blocks are bolted to the wide flange. Figure 15 shows the location of highest stresses in the vertical wide flange one plate elements near the clevis connection at the bottom of the web, illustrating plate elements 149 to 152. Figure 16 and 17 show comparable areas of high stress in vertical wide flange four for loading condition 1. Figures 18 and 19 show stress contours drawn for the principal stresses (from Mohr's circle) for the plate elements on the front of the vertical wide flanges one and four respectively, for loading condition 1. These stress contour plots show the location of these highest stress areas that are illustrated in Table 1.

TABLE 1. Maximum stresses from loading condition 1, plate elements.

plate element no.	location	maximum stress (psi)	safety factor
29	front, far left, WF1	17910 (compression)	2.01
32	front, mid left, WF1	18465 (compression)	1.95
35	front, mid right, WF1	18733 (compression)	1.92
38	front, far right, WF1	18852 (compression)	1.91
150	web, WF1, near clevis	22076 (compression)	1.63
569	front, far left, WF4	19080 (compression)	1.89
572	front, mid left,WF4	18920 (compression)	1.90
575	front, mid right, WF4	18653 (compression)	1.93
578	front, far right, WF4	18143 (compression)	1.98
690	web, WF4, near clevis	22167 (compression)	1.62

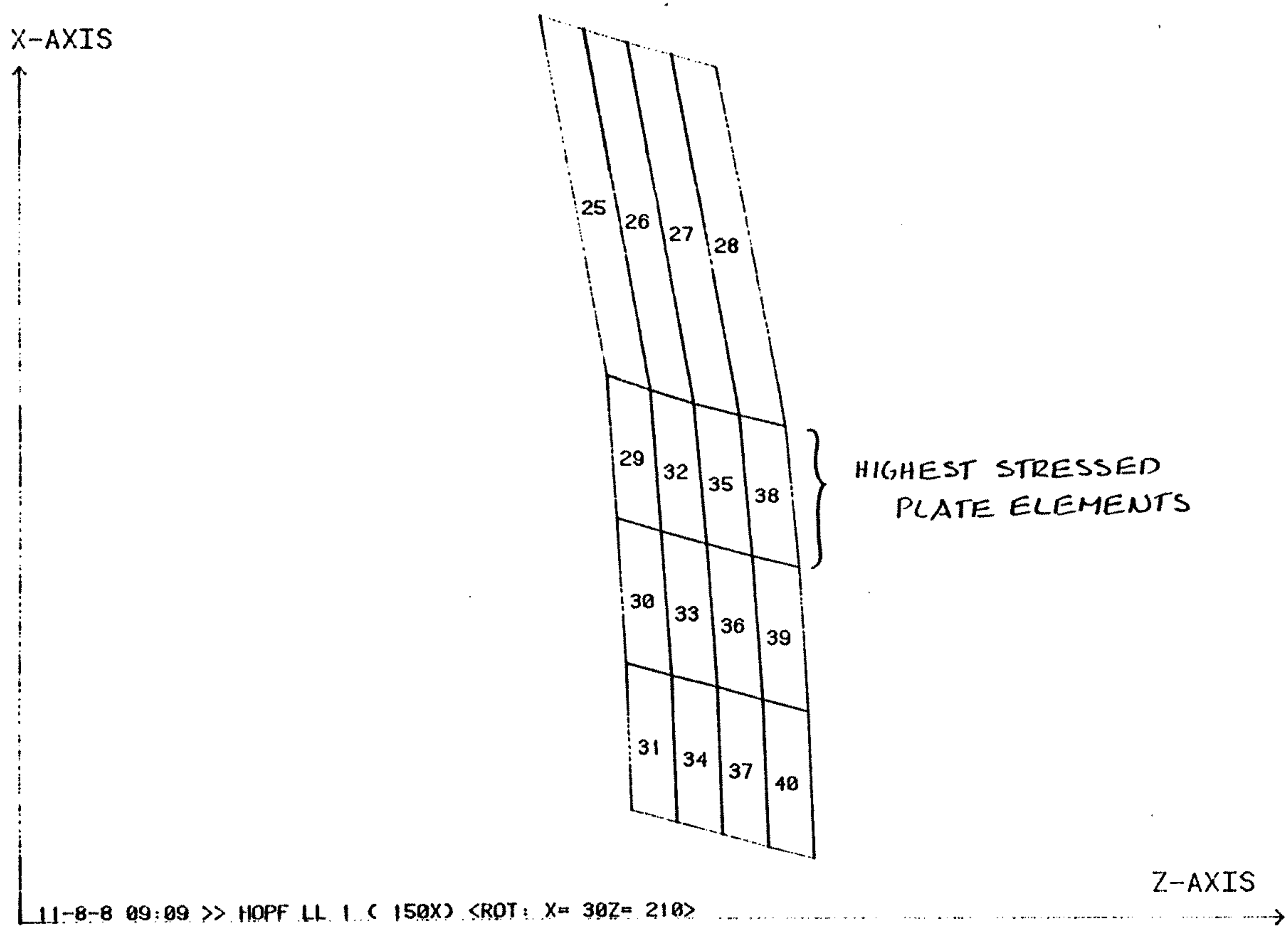

Figure 14. Highest plate element stresses, loading condition 1, elements 25 to 40, front wide flange 1.

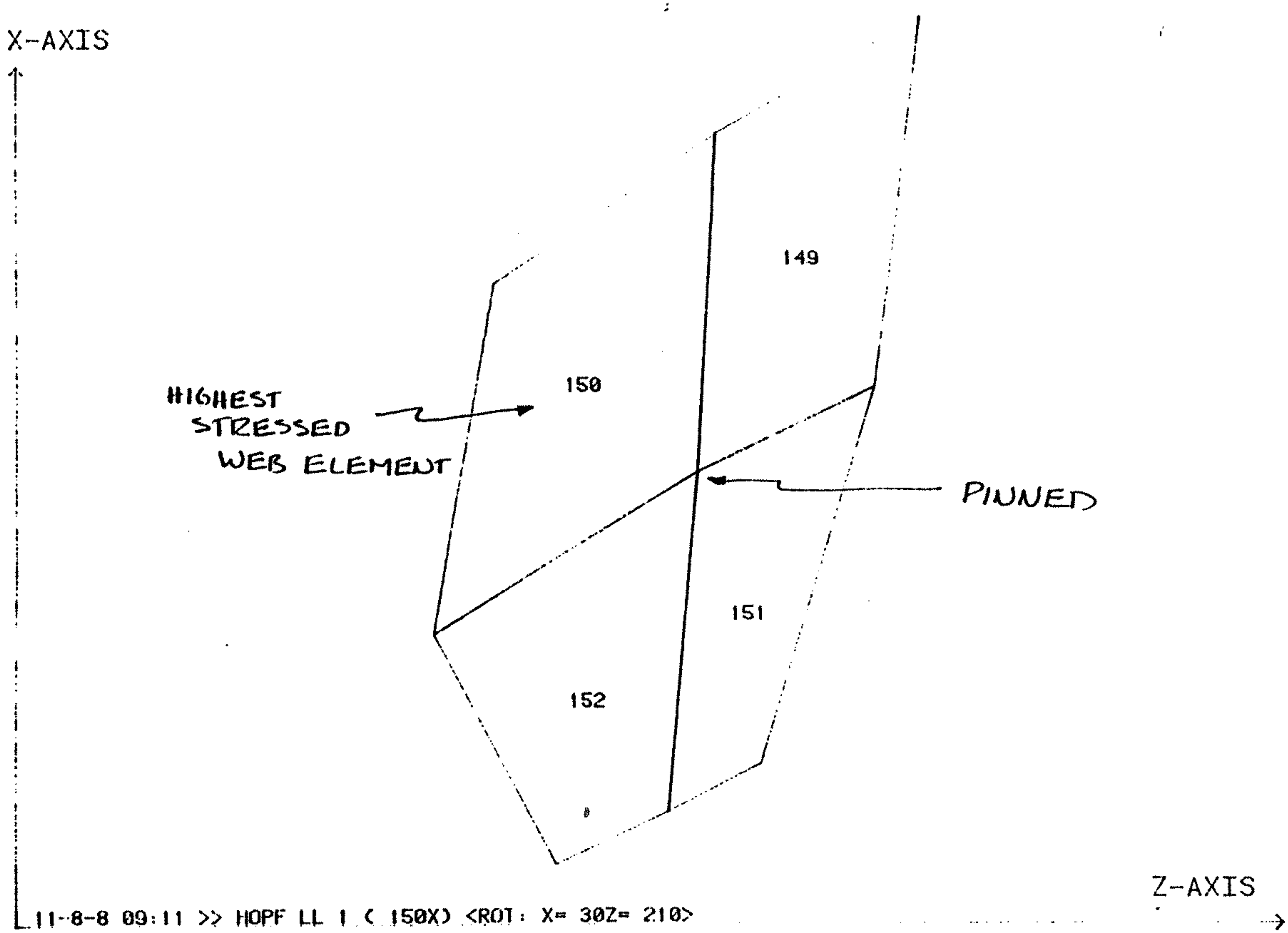

Figure 15. Highest plate element stresses, loading condition 1, elements 149 to 152, bottom web, wide flange 1.

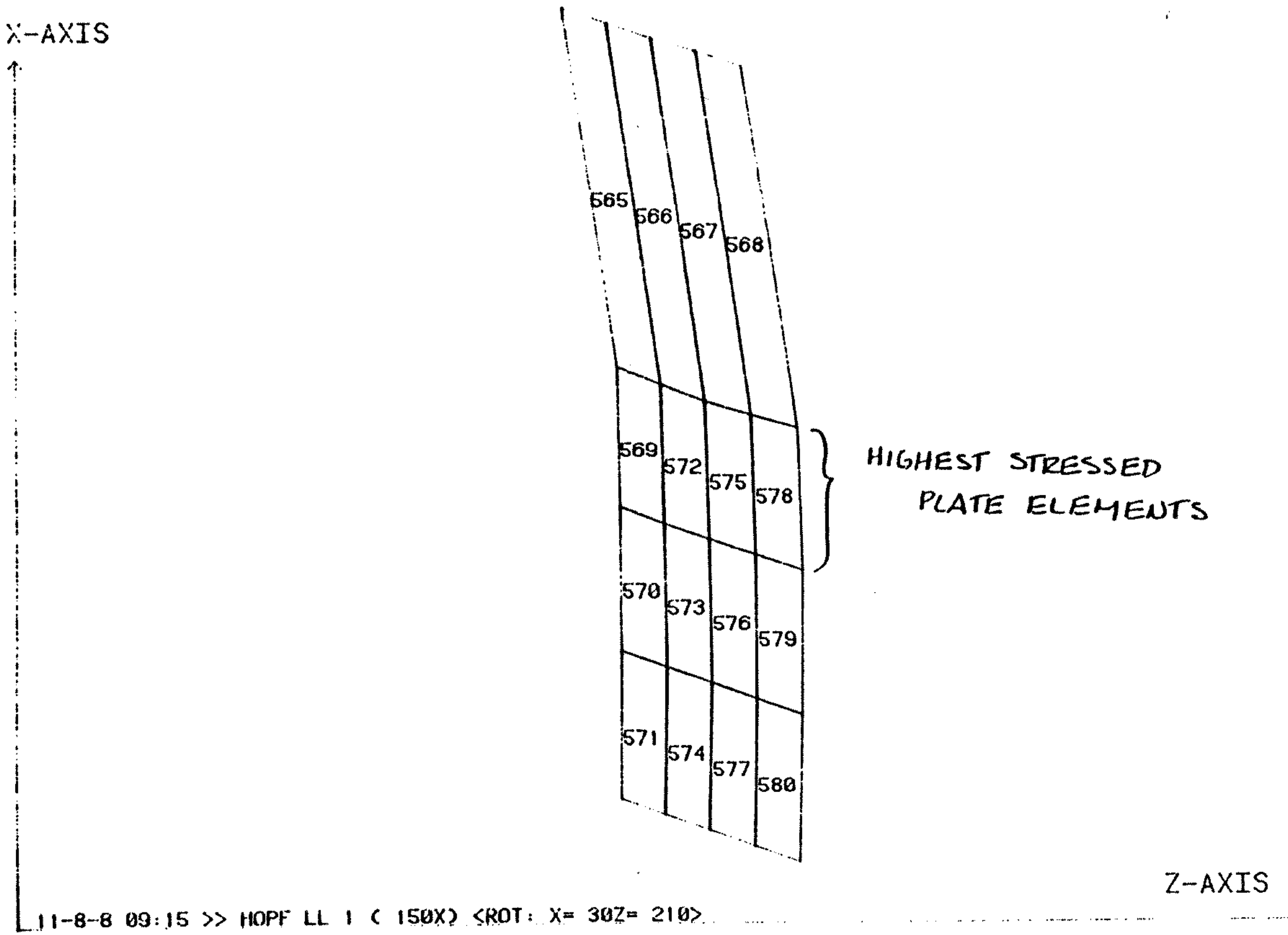

Figure 16. Highest plate element stresses, loading condition 1, elements 565 to 580, front wide flange 4.

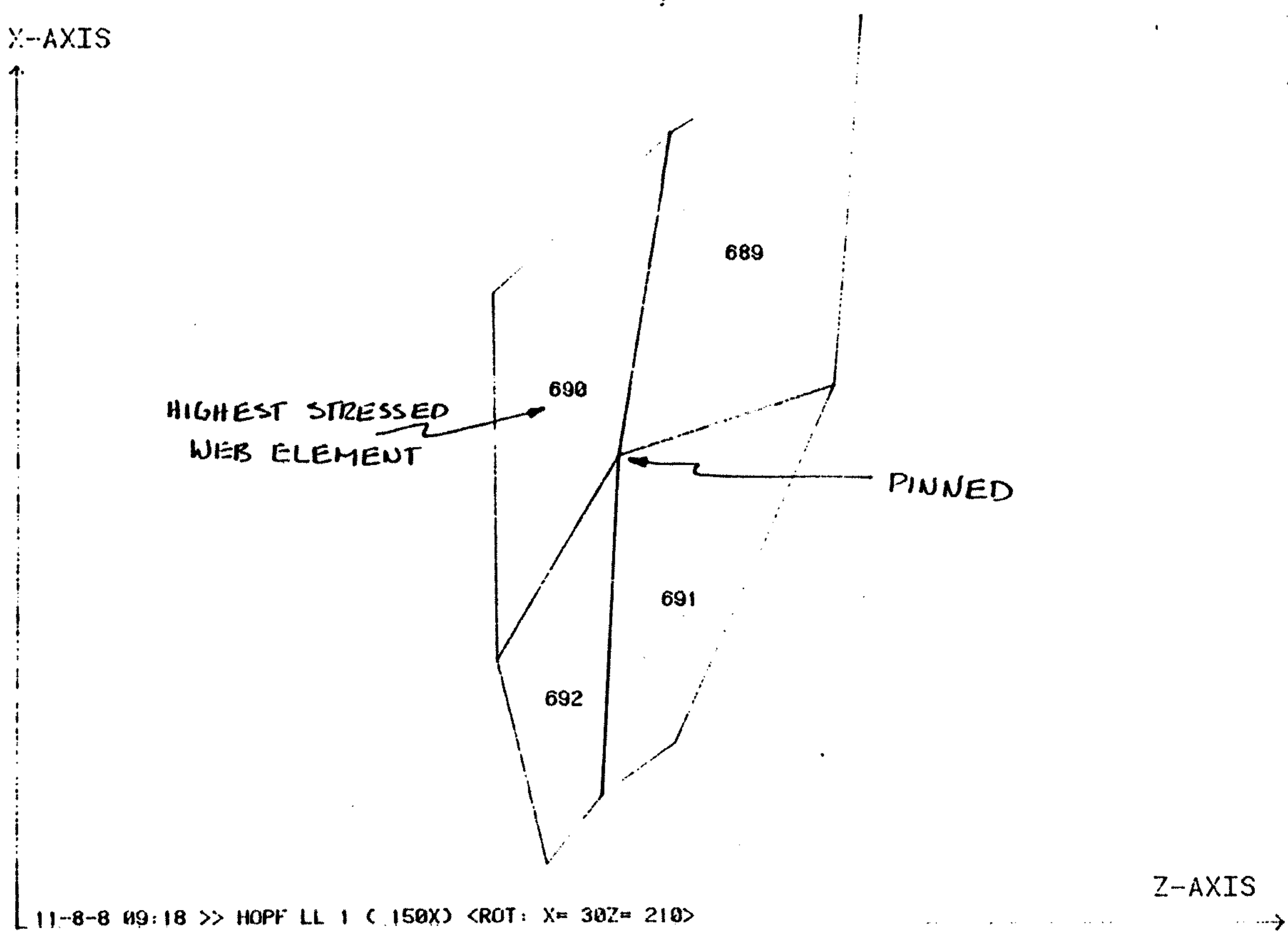

Figure 17. Highest plate element stresses, loading condition 1, elements 689 to 692, bottom web, wide flange 4.

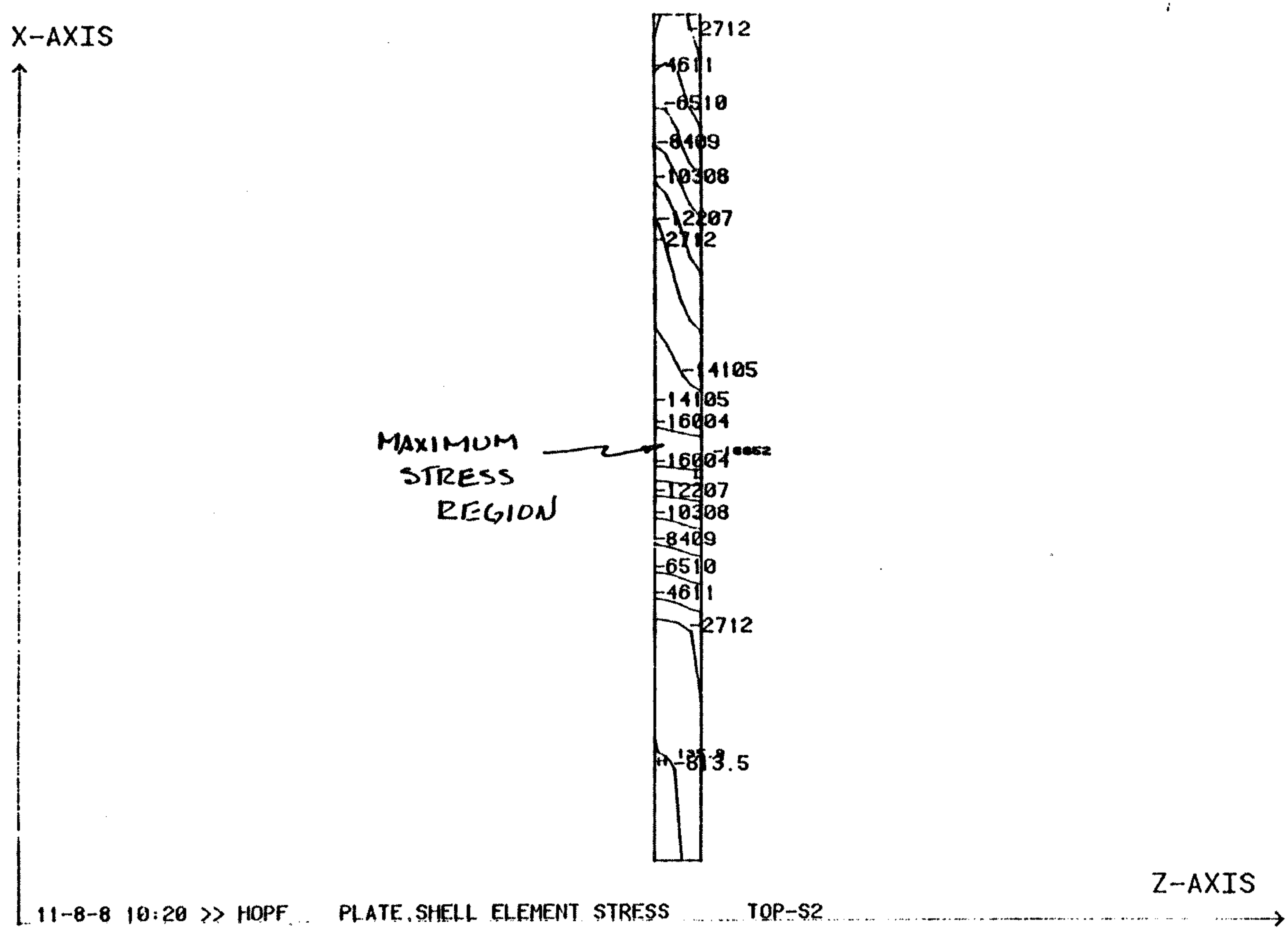

Figure 18. Stress contour plot, front of wide flange 1, loading condition 1.

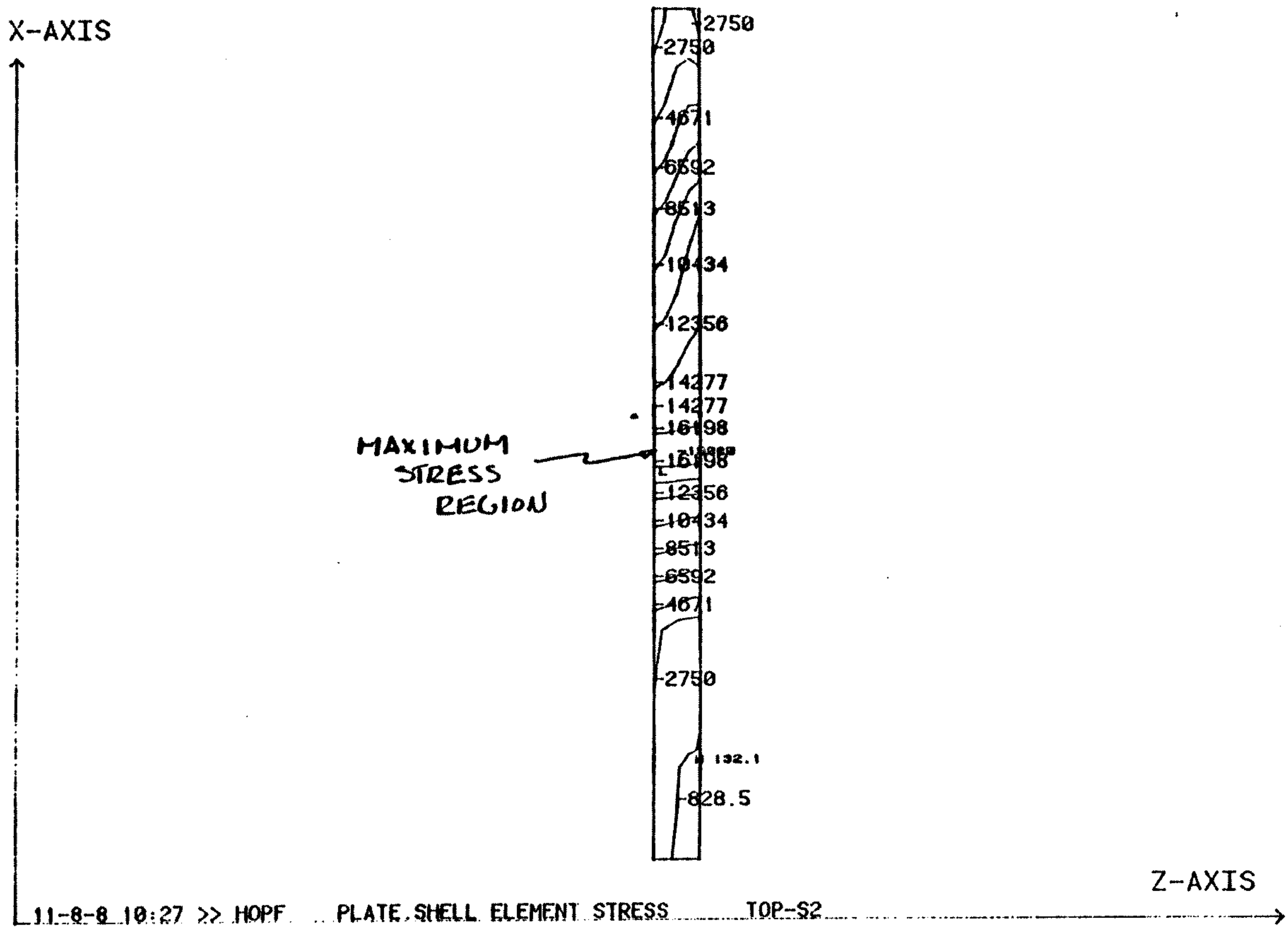

Figure 19. Stress contour plot, front of wide flange 4, loading condition 1.

Table 2 shows a tabulated listing of all the major areas of maximum stress in the beam elements for loading condition 1. Shown are the approximate location of these beam elements in the structure, the beam element number, their maximum stress value, and the corresponding safety factor.

Table 3 shows a tabulated listing of all the major areas of maximum stresses in the plate elements for loading condition 2. Shown are the approximate location of maximum stresses in the structure, the plate element number, their value, and the corresponding safety factor.

TABLE 2. Maximum stresses from loading condition 1, beam elements.

beam element no.	location	maximum stress (psi)	safety factor
15	1.375" diam, WF1	58277	1.39
17	1.375" diam, WF1	43084	1.88
19	1.375" diam, WF1	72574	1.12
21	1.375" diam, WF1	60106	1.35
39	1.375" diam, WF4	54352	1.49
41	1.375" diam, WF4	57855	1.40
43	1.375" diam, WF4	70790	1.14
45	1.375" diam, WF4	69912	1.16

Figures 20 and 21 show stress contours drawn for the principal stresses (from Mohr's circle) for the plate elements on the front of the vertical wide flanges one and four respectively, for loading condition 2. These stress contour plots show the location of these highest stress areas that are illustrated in Table 3.

Table 4 shows a tabulated listing of all the major areas of maximum stresses in the beam elements for loading condition 2. Shown are the approximate location of these beam elements in the structure, the beam element number, their stress value, and the corresponding safety factor.

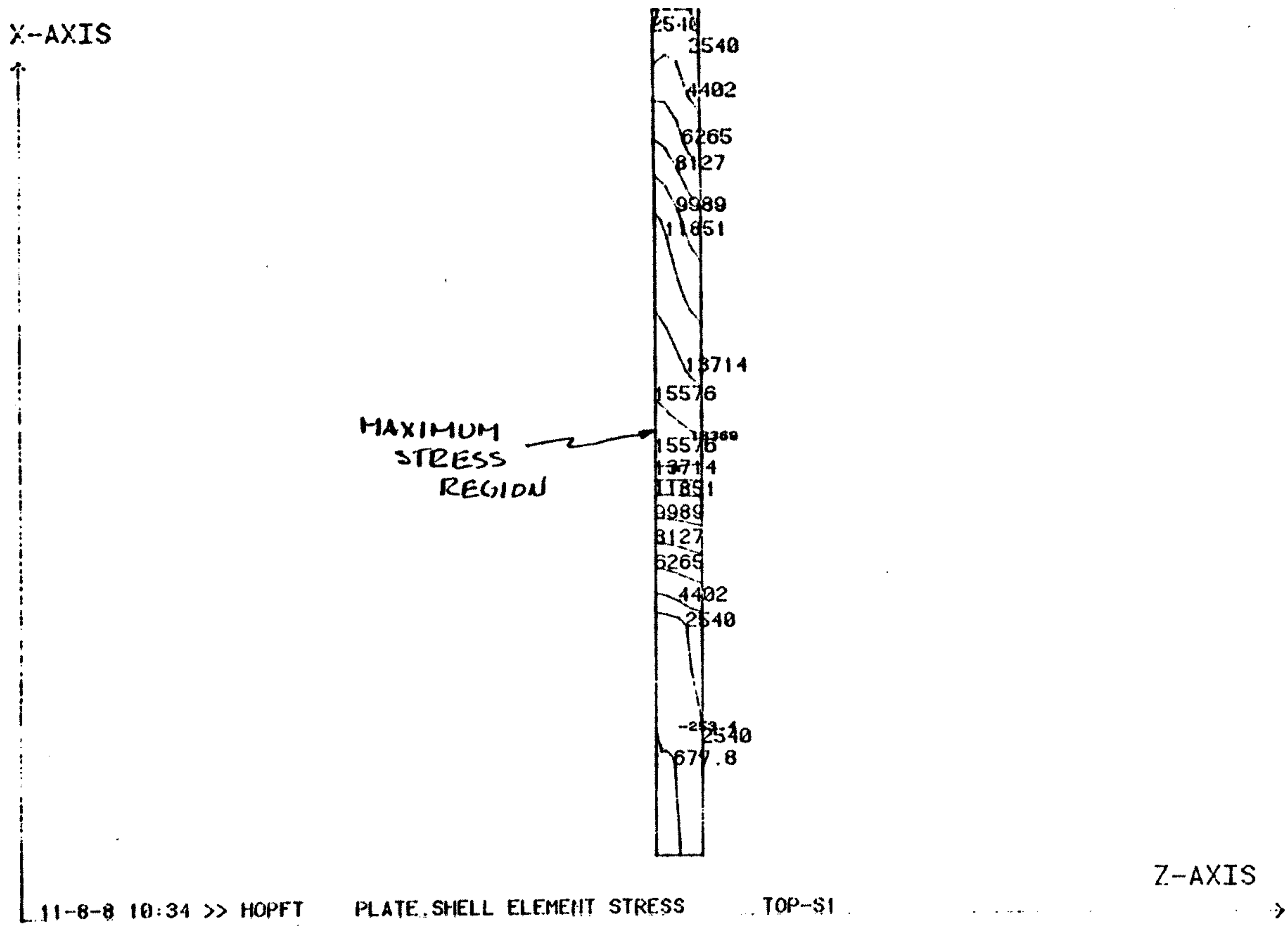

Figure 20. Stress contour plot, front of wide flange 1, loading condition 2.

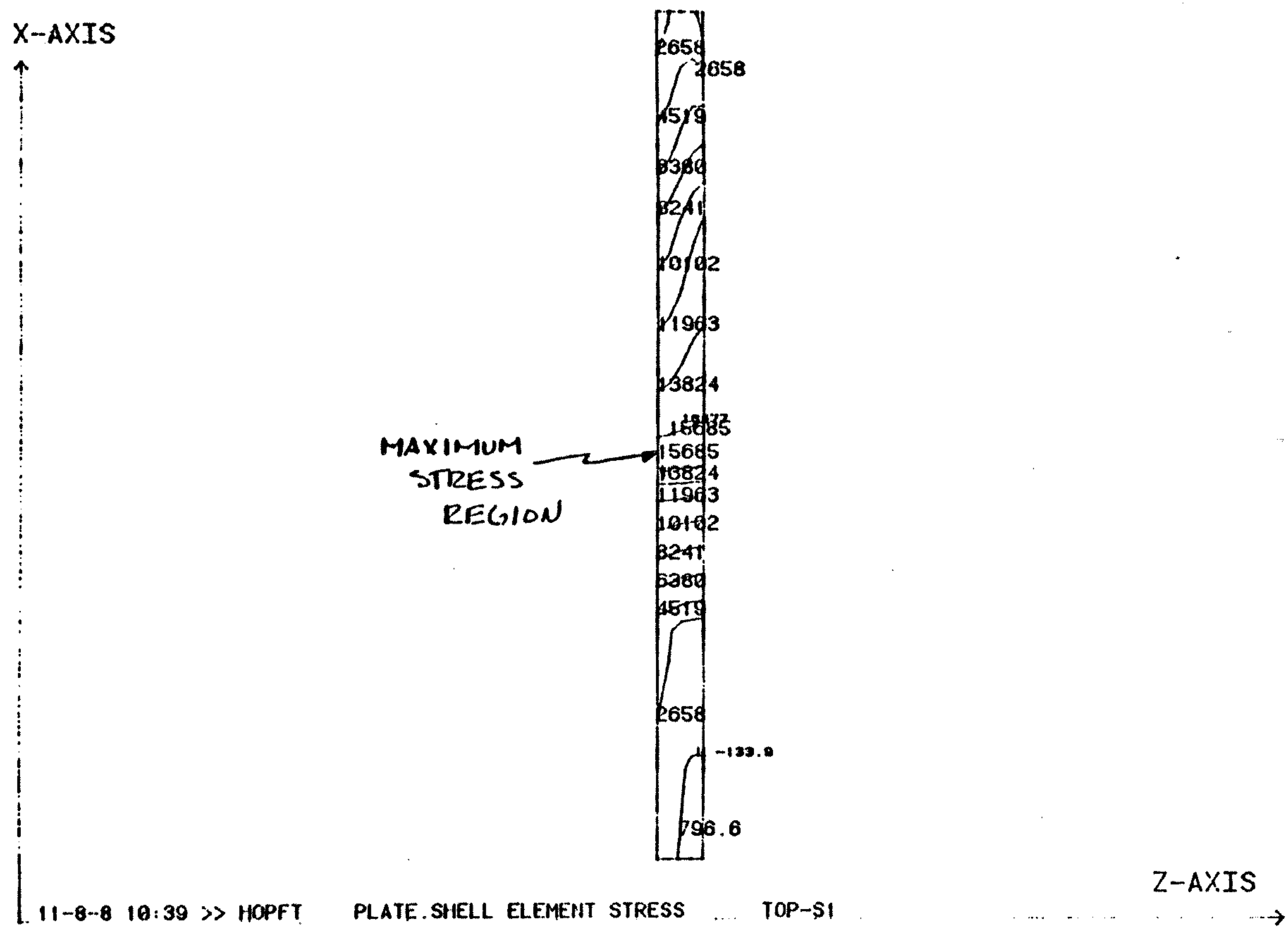

Figure 21. Stress contour plot, front of wide flange 4, loading condition 2.

TABLE 3. Maximum stresses from loading condition 2, plate elements.

plate element no.	location	maximum stress (psi)	safety factor
29	front, far left, WF1	18185 (tension)	1.98
32	front, mid left, WF1	18369 (tension)	1.96
35	front, mid right, WF1	18360 (tension)	1.96
38	front, far right, WF1	18169 (tension)	1.98
150	web, WF1, near clevis	21639 (tension)	1.66
569	front, far left, WF4	18477 (tension)	1.95
572	front, mid left, WF4	18424 (tension)	1.95
575	front, mid right, WF4	18248 (tension)	1.97
578	front, far right, WF4	17849 (tension)	2.02
590	web, WF4, near clevis	21826 (tension)	1.65

TABLE 4. Maximum stresses from loading condition 2, beam elements.

beam element no.	location	maximum stress (psi)	safety factor
15	1.375" diam, WF1	54115	1.50
17	1.375" diam, WF1	41919	1.93
19	1.375" diam, WF1	60531	1.34
21	1.375" diam, WF1	61600	1.31
39	1.375" diam, WF4	52463	1.54
41	1.375" diam, WF4	54453	1.49
43	1.375" diam, WF4	87785	0.92
45	1.375" diam, WF4	85877	0.94

DISCUSSION OF RESULTS OF ORIGINAL ANALYSIS AND DESIGN CHANGES

From the results illustrated in the previous tables and figures for loading condition 1 (opening) and loading condition 2 (closing), there appeared to be three areas of the structure of the Hopkins frame that did not, with the original design, meet the minimum requirement of a safety factor of three based on the yield strength of the material. These three areas were:

1. The front flange area of the vertical wide flange members one and four (far left and far right respectively) just below where the mounting brackets were attached. Safety factors as low as 1.89 were noticed;
2. The bottom web area of the vertical wide flange members one and four near where they were pinned to the clevis support. Safety fators as low as 1.62 were noticed;

3. The 1.375 diameter high strength turned bolts that mount the bearing blocks onto the mounting brackets and vertical wide flange members one and four. Safety factors as low as 0.92 were noticed.

It should be mentioned at this point that the endurance limit (fatigue strength) is usually based on the ultimate strength, not the yield strength. It can therefore be concluded that limiting maximum stress values to 12000 pounds per square inch in every part of the structure other than the bolts will be most conservative.

In order to accommodate safety factors of three based on the yield strength of the material in the problem areas cited above, the following design changes were recommended and an additional analysis was conducted.

1. Change wide flange members one and four from WF12x36 to WF12x65.

2. *Include double plates of approximately one inch thickness on the web of wide flanges one and four near the clevis pin location.*

3. Increase the bearing bolt diameter from 1.375 inches to 2.5 inches, but maintain the same number of bolts (4), and use ASTM 354, Grade BC bolts. These bolts have a yield strength of 109000 pounds per square inch, and an ultimate strength of 125000 pounds per square inch.

RESULTS OF THE SUBSEQUENT ANALYSIS

Tables 5, 6, 7, and 8 show a tabulated listing of all major areas of maximum stress in the plate and beam elements for loading conditions one and two. Shown is a comparison of the results from the original model and the redesigned one.

TABLE 5. Maximum stresses from loading condition one, plate elements.

Plate element no.	Location	Maximum stress(psi) new design	Maximum stress(psi) original	Safety factor new design	Safety factor original
29	front,far left,WF1	9541.	17910.	3.77	2.01
32	front,mid left,WF1	9067.	18465.	3.97	1.95
35	front,mid right,WF1	8286.	18733.	4.34	1.92
38	front,far right,WF1	6941.	18852.	5.19	1.91
150	web,WF1,near clevis	2896.	22076.	12.43	1.63
569	front,far left,WF4	7106.	19080.	5.07	1.89
572	front,mid left,WF4	8431.	18920.	4.27	1.90
575	front,mid right,WF4	9215.	18653.	3.91	1.93
578	front,far right,WF4	9692.	18143.	3.71	1.98
690	web,WF4,near clevis	2905.	22167.	12.39	1.62
1137	radius arm, left	10371.	8420.	3.47	4.28
1154	radius arm, right	10331.	8412.	3.48	4.28

TABLE 6. Maximum stresses from loading condition one, beam elements.

Beam element no.	location	Maximum stress(psi) new design	original	Safety factor new design	original
15	2.5" diam,WF1	21779.	58277.	5.01	1.87
17	2.5" diam,WF1	15193.	43084.	7.18	2.53
19	2.5" diam,WF1	33017.	72574.	3.30	1.51
21	2.5" diam,WF1	29613.	60106.	3.68	1.81
39	2.5" diam,WF4	18584.	54352.	5.87	2.01
41	2.5" diam,WF4	21759.	57855.	5.01	1.88
43	2.5" diam,WF4	29297.	70790.	3.72	1.54
45	2.5" diam,WF4	29340.	69912.	3.71	1.55

TABLE 7. Maximum stresses from loading condition two, plate elements.

Plate element no.	Location	Maximum stress(psi) new design	original	Safety factor new design	original
29	front,far left, WF1	8859.	18185.	4.06	1.98
32	front,mid left, WF1	8329.	18369.	4.32	1.96
35	front,mid right,WF1	7532.	18360.	4.78	1.96
38	front,far right,WF1	6137.	18169.	5.87	1.98
150	web,WF1, near clevis	2274.	21639.	15.83	1.66
569	front,far left, WF4	6479.	18477.	5.56	1.95
572	front,mid left, WF4	7667.	18424.	4.70	1.95
575	front,mid right,WF4	8292.	18248.	4.34	1.97
578	front,far right,WF4	8691.	17849.	4.14	2.02
690	web,WF4, near clevis	2237.	21826.	16.09	1.65

TABLE 8. Maximum stresses from loading condition two, beam elements.

Beam element no.	Location	Maximum stress(psi) new design	original	Safety factor new design	original
15	2.5" diam, WF1	18737.	54115.	5.82	2.02
17	2.5" diam, WF1	14671.	41919.	7.43	2.60
19	2.5" diam, WF1	19145.	60531.	5.69	1.80
21	2.5" diam, WF1	17901.	61600.	6.09	1.77
39	2.5" diam, WF4	16309.	52463.	6.68	2.06
41	2.5" diam, WF4	17576.	54453.	6.20	2.01
43	2.5" diam, WF4	20915.	87785.	5.21	1.24
45	2.5" diam, WF4	25386.	85877.	4.29	1.26

The maximum plate element stresses were illustrated in Tables 5 and 7. These maximum stress levels are principle stresses from Mohr's circle. The design specifications were that these stress levels should not exceed 12000 pounds per square inch (safety factor of three based on the yield strength of 36000 pounds per square inch). All maximum stress levels from Tables 5 and 7 were below this 12000 pound per square inch limitation, thus meeting this requirement. It should however be pointed out that the endurance limit (fatigue strength) is normally calculated as the ultimate strength divided by a factor, not yield strength divided by a factor. The ultimate streength of 36000 psi steel is 58000 pounds per square inch and one normally takes 0.5 of this to determine the endurance strength. Thus the endurance limit for this steel is 29000 pounds per square inch. The maximum stress levels shown in Tables 5 and 7 indicate that with the new design, the maximum stresses are almost one-third of the endurance limit. One can conclude, therefore, that the maximum stress levels for the new design from loading conditions one and two have a safety factor of almost three based on the endurance limit.

The maximum beam element stresses were illustrated in Tables 6 and 8. These maximum stress levels are a combination of axial and bending stresses. The design specifications were that these stress levels should not exceed one-third of the yield stess of 109000 pounds per square inch for A354 bolts, or a stres level of 36333 pounds per square inch where the effect of pretensioning has been included. The maximum stress levels illustrated in Tables 6 and 8 without pretensioning meet this criteria. Further discussion, however, is warranted.

The bolts in question are initially pretensioned with some mean tensile stress. They are then loaded due to alternatively opening and closing the bascule, while always remaining in tension. They never go into compression. They are thus loaded so that they never experience full reverse bending (tension to compression), as most fatigue related failure theories are based. Since there is no reversed bending, then the normal endurance limit (fatigue strength) has no meaning. There is no generally accepted theory for measuring the endurance strength of structures where the stress is not reversed, but experience has offered some guidelines. The most common diagram to consider in these cases is the one shown in Figure 22 below. This fatigue strength

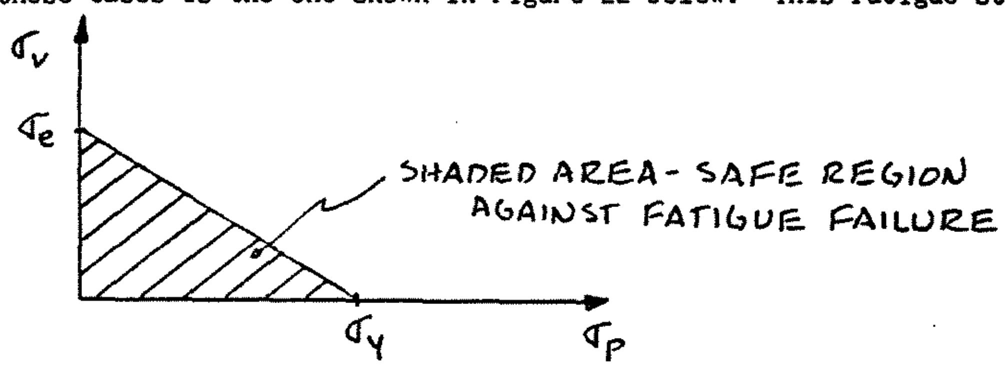

Figure 22. Fatigue diagram for non-reversal stress.

related diagram shows the ordinate as the variable stress σ_v due to alternating loading plotted against the abscissa which is the amount of pretensioning stress σ_p. The shaded area indicates no fatigue problem. Outside the shaded area would indicate that that combination of pretensioning and alternating stress would eventually lead to a fatigue failure, presumably at points of change in section for a bolt since these are areas of stress concentration. The value of σ_e in the figure is the endurance limit, usually for steel one-half the ultimate strength. The value of σ_y in the figure is the yield strength of the material. Obviously from the figure, for a given alternating stress, it may or may not be a problem depending on the amount of pretensioning stress. A convenient equation to describe a safety factor in this situation is given in equation (1) below.

$$\frac{1}{\text{SAFETY FACTOR}} = \frac{\sigma_p}{\sigma_y} + \frac{\sigma_v}{\sigma_e} \qquad (1)$$

This is the description of the safety factor that should be used in evaluating the structural integrity of these bolts, not solely based on the yield strength.

The initial tension in the bolts of course is a function of the torque used in tightening the bolts. In spite of numerous attempts to find an easy way of estimating the initial tension in a bolt, the questions involved have not been completely answered because there are so many variables involved. An experimentally related formula that is most commonly used relating the applied torque to the pretensioning load is shown in equation (2) below.

$$T = C D F_i \qquad (2)$$

In this formula T is the applied tightening torque in inch-pounds, F_i is the initial tensile load in the bolt in pounds, D is the nominal bolt diameter in inches, and C is a constant for a particular set of conditions and ranges from 0.18 for lubricated to 0.20 for unlubricated. The torque, measured by a torque wrench, necessary to induce a certain tension varies principally with the condition of the surfaces in contact between the nut and its seat, with the kind and amount of lubrication of the rubbing surfaces, with the material in contact, and with the slope of the threads. The initial pretensioning stress, σ_p , can then be calculated from equation (3) shown below.

$$\sigma_p = \frac{F_i}{A_s} \qquad (3)$$

The value of A_s in this equation is the stress area of the bolt, based on the root diameter.

As an example of the use of these three equations for this situation, suppose one wants a safety factor of 1.5. The yield strength of the A354 steel is 109000 pounds per square inch, the ultimate strength is 125000 pounds per square inch, and thus the endurance limit is 62500 pounds per square inch (one-half the ultimate). Equation (1) can then be used to calculate the amount of pretensioning stress allowed and still have a factor of safety of 1.5. From Equation (3),

$$\frac{1}{1.5} = \frac{\sigma_p}{109000} + \frac{33017}{62500} \qquad \sigma_p = 15085.\ \text{psi}$$

using a stress area of the bolt of A_s = 3.716 square inches (based on the root diameter), the pretensioning load is calculated as

F_i = (3.716) (15085) = 56056.1 pounds

From equation (2) the amount of applied torque allowed would be

T = (.2) (2.5) (56056.1) = 28028 inch-pound = 2335.7 ft-pound.

This example illustrates that for this structural model, with 2.5 inch diameter bolts, applying a pretensioning torque of 2335.7 foot pounds will produce an initial tensile stress of 15085 pounds per square inch in the bolts, and under loading condition one (opening) will yield a minimum safety factor against fatigue failure of 1.5.

In conclusion, this paper demonstrates how finite element computer aided structural analysis can be formulated for any part of a moveable bridge, in this case the Hopkins frame, and the information and results that can be obtained from this analysis.