Enclosure 1

BALANCE OF PLANT STRUCTURES
SEISMIC REEVALUATION PROGRAM

TURBINE BUILDING
AND
TURBINE-GENERATOR PEDESTAL

SAN ONOFRE NUCLEAR GENERATING STATION
UNIT 1

FOREWORD

This report describes the seismic reevaluation of the turbine building and turbine-generator pedestal at San Onofre Unit 1. It describes in detail the stress analysis methods and stress analysis results for typical locations.

The program under which this structure was reevaluated is entitled the Balance of Plant Structures Seismic Reevaluation (BOPSSR) Program. This program is being conducted as part of the Systematic Evaluation Program Topic III-6, Seismic Design Considerations.

The objective of the BOPSSR program is to demonstrate that the expected conditions of stress and deflection induced in the structures as a result of the combined influence of normal operating loads and earthquake loads will not impair an orderly shutdown of the plant following a DBE.

The structures included in the BOPSSR Program are:

- Circulating Water System Intake Structure
- Reactor Auxiliary Building
- Ventilation Equipment Building
- Seawall
- Control and Administration Building
- Turbine Building and Turbine Pedestal
- Fuel Storage Building

The results of the evaluation of the turbine building and turbine-generator pedestal are included in this report.

OUTLINE

1.0 INTRODUCTION

1.1 Organization of the Report

This report is divided into six sections. Section 1 describes the background associated with the Balance of Plant Structures Seismic Reevaluation Program (BOPSSR) and the scope of this report. Section 2 gives the description of the existing turbine building structures and the proposed structural modifications. Section 3 outlines the analytical methods used in the reevaluation process. Section 4 summarizes the results of the structural reevaluation and provides a comparison of the results with the provisions of the BOPSSR criteria. Section 5 summarizes the proposed structural modifications to the Turbine Building. The conclusion of this evaluation is provided in Section 6.

1.2 Background

The San Onofre Unit 1 turbine-generator pedestal is a Seismic Category A structure while the turbine building was originally classified as a Seismic Category B structure. The turbine building currently houses Seismic Category A equipment and therefore has been reevaluated as a Seismic Category A structure. The Turbine Building was originally designed by working stress methods to resist lateral loads equivalent to 0.2g.

Structures and equipment at San Onofre Unit 1 originally designated as Seismic Category A (e.g. turbine-generator pedestal) were designed to withstand a 0.5g Housner Design Basis Earthquake. The plant design was completed in early 1965. The methods of analysis and acceptance criteria were in accordance with accepted practice at that time. The technology of seismic analysis has advanced rapidly in the years since the original design of San Onofre Unit 1 was completed. This advance has been largely in the field of finite-element analysis and numerical methods. During this same period, codes and regulatory practices pertaining to the design of nuclear power plants have also changed. This evolution, while not resulting in a change in the basic concepts of design, has yielded more detailed information concerning the behavior of structures during an earthquake.

San Onofre Nuclear Generating Station Unit 1 (San Onofre Unit 1) was designed before the current technology and codes had fully evolved. In order to obtain an updated understanding of the plant dynamic characteristics and to reflect an increase of the maximum ground acceleration from 0.5g to 0.67g (the design basis for Units 2 and 3), a seismic reevaluation program was initiated to evaluate safety related structures and equipment at San Onofre Unit 1. This program was based upon the use of current analysis methods and acceptance criteria.

The first phase of the seismic reevaluation program began in 1974 with the reevaluation of the NSSS, the concrete reactor building and the steel containment sphere. As a result of this reevaluation, modifications to the NSSS supports were installed in 1976. During this same time two new structures were constructed. These were the sphere enclosure building and the diesel generator building; the former to provide additional biological shielding about the containment structure and the latter to house two new emergency power diesel generators. Both of these structures were designed to the same seismic input levels utilized for Units 2 and 3 (0.67g) and the acceptance criteria were based upon current standards. Therefore, these four structures have been designed or evaluated to criteria equivalent or greater than the BOPSSR criteria and are not included in the seismic reevaluation program.

After the completion of the initial phase of the seismic reevaluation program a "balance of plant" program was begun to reevaluate the remaining safety related structures. This program was suspended in 1978 to allow time for studies of expected site specific ground accelerations and because the NRC staff requested that the seismic reevaluation of San Onofre Unit 1 be performed as part of the Systematic Evaluation Program.

In mid 1980, work was restarted on the Balance of Plant Structures Seismic Reevaluation Program. The scope of this program includes all safety-related structures not previously reevaluated or otherwise qualified. Analysis of the circulating water system intake structure, the reactor auxiliary building, the ventilation equipment building and seawall has been completed and the results reported to the NRC staff by letter dated December 8, 1981.

Analysis of the control and administration building has also been completed and the results reported to the NRC staff by letter dated February 9, 1982.

1.3 Scope

The turbine building complex, which includes the north and south extensions, east and west heater platforms and the turbine-generator pedestal, has been reevaluated as part of the BOPSSR Program and the results are discussed in this report.

The turbine building complex was evaluated for the occurrence of a 0.67g Housner design basis earthquake in combination with normal plant operating loads. This evaluation was based upon the criteria described in Reference 1.

A plot plan of San Onofre Unit 1 which shows the location of the turbine building complex is provided in Figure 1. A description of the structure is provided in the following section.

2.0 DESCRIPTION OF EXISTING STRUCTURES AND MODIFICATIONS

2.1 Turbine Building

The turbine building consists of four individual structural systems which surround the concrete turbine pedestal. These four structural systems are known as the turbine building north and south extensions and the east and west heater platforms (See Figures 2, 3, 4 and 5.

As discussed in Enclosure 2 of Reference 2, the four turbine building steel platforms and extensions will be converted from moment resisting frames to moment resisting braced frames by the addition of seismic bracing. The purpose of this bracing is to reduce the stresses in the structures and to eliminate interactions with adjacent structures by limiting deflections. These new bracing members are shown in Figures 10 thru 15.

The turbine building north extension is a one-story structural steel frame building with two bays in each direction and a mezzanine (see Figure 6). It has approximate plan dimensions of 40 feet by 50 feet with an 8-1/2 inch thick prestressed concrete slab at elevation 42 feet, 0 inch, and a steel grating platform at elevation 30 feet, 0 inch. One and one half inch wide expansion joints are provided at the juncture between the extension and the turbine-generator pedestal (at elevation 42 feet, 0 inch).

The turbine building south extension is a one-story building employing a steel frame system constructed above ground level. The south extension has approximate plan dimensions of 40 feet by 50 feet, with an 8-1/2 inch thick prestressed concrete slab at elevation 42 feet, 0 inches (see figure 7). One and one-half inch wide expansion joints are provided at the junction of the south extension and the turbine-generator pedestal (at elevation 42 feet, 0 inches).

The west heater platform is a one story steel frame with 2 bays in the east-west direction and six bays in the north-south direction. The platform has approximate plan dimensions of 112 feet by 50 feet and supports an 8-1/2 inch thick prestressed concrete slab at elevation 35 feet, 6 inches.

The east heater platform is similar to the west heater platform with one less bay north of column line E, as shown in Figure 8. Deck level framing for the east heater platform and the west heater platform is shown in Figures 8 and 9.

In the north extension six new braces are being installed. The results reported in this evaluation are based upon the preliminary design of these braces. Two of these braces are oriented in the north-south direction and four of the braces are oriented in the east-west direction. Side views of the bays where these braces are located are shown in Figure 10. In the west heater platform framing fourteen new braces are being installed. The results reported in this evaluation are based upon the preliminary design of these braces.

Seven of these braces are oriented in the east-west direction and seven are oriented in the north-south direction. Three of the east-west braces and one of the north-south braces are exterior to the west heater platform structure. Side views of the bays showing brace locations and sizes are shown in Figures 11 and 12.

The installation of the braces in the north and west extensions are scheduled to be completed by June 1, 1982.

The results of the analysis contained in this report are also based upon the addition of eight braces in the south extension framing and fifteen braces in the east heater platform framing. Figures 13 thru 15 show locations and sizes of these braces. These braces will be installed in conjunction with the modifications for the south extension and the east heater platform.

2.2 Turbine-Generator Pedestal

The concrete turbine pedestal is a reinforced concrete space frame (see Figure 5) supported by a 5-foot thick mat foundation. It consists of haunched columns at the four corners of the mat foundation and three haunched intermediate walls. The north columns are 8 feet square; the south columns are 8 feet by 8 feet, 2 inches. Wall thickness varies from 4-1/2 feet to 7 feet. The centerline to centerline distances between columns are 34 feet, 0 inches in the east-west direction and 125 feet, 6 inches in the north-south direction. The operating deck consists of an 8-foot thick center section that supports the turbine-generator, which is accommodated through several large openings in the deck. Two cantilevered wings on the east and west sides are 1-foot, 6 inches thick. The top of the deck is at Elevation 42 feet, while the overall height of the structure is approximately 33 feet, 6 inches from the top of the mat foundation.

2.3 Turbine Gantry Crane

The gantry crane is a large steel structure (weighing approximately 120 tons) that travels back and forth from the turbine building south extension, over the turbine pedestal deck to the turbine building north extension for shipping fuel or other maintenance work. The top-of-rail elevation is 42 feet, 6 inches. The following is a list of other crane data:

Overall height	47 feet, 0 inch
Overall length	82 feet, 0 inch
Wheel base	26 feet, 0 inch
Length between supporting legs	40 feet, 0 inch

The Turbine Gantry Crane is shown in Figure 4.

2.4 Foundations

2.4.1 Turbine Building Foundation

The turbine building foundation consists of individual and combined footings. Footing widths vary from 3 feet to 15 feet while footing thickness varies from 2 feet 6 inches to 15 feet. The elevations of the top of footings vary from (+) 6 feet to (+) 17 feet, 7 inches. Two columns from each of the four extensions are founded on the pedestal basemat. One north extension column has its foundation cast monolithically with the spent fuel pool wall. The existing foundation plan is shown in Figure 16.

As discussed in Enclosure 2 to Reference 2, foundation modifications are being designed to accommodate the increased uplift loads due to the new bracing loads at locations where new bracings are postulated. The foundation modifications generally consist of providing wide grade beams rigidly connecting two or more adjacent individual foundations. Additionally, in some cases the size of an individual foundation is increased by adding a volume of concrete and rigidly connecting the new concrete to the existing footing. Figure 17 shows the plan view of the existing and modified foundations. The analysis and reevaluation results reflect the preliminary design of the foundation modifications. Figure 18 shows typical cross sections thru existing and new foundations.

2.4.2 Masonry Wall Foundations

Reinforced masonry walls are located on the west and south sides of the west heater platform, the north end of the north extension, the west, south, and east sides of the south extension and the south end of the east heater platform. The masonry walls are supported on continuous spread footings. The wall in the north extension is a cantilever wall and is not connected to the structural steel framing. All other masonry walls are connected to the steel framing with out-of-plane only support at the deck level. A typical cross section of a wall in the south extension is shown in Figure 19.

2.4.3 Turbine-Generator Pedestal Foundation

The turbine pedestal foundation is a 147-foot long by approximately 47-foot wide by 5-foot thick reinforced concrete mat at elevation 3 feet, 6 inches (grade is at elevation 14 feet). A 12.5-foot by 20-foot circulating water discharge culvert, built monolithically with the mat, passes through from underneath the mat at the south end. Figure 5 shows plan and sections of the mat foundation.

3.0 ANALYTICAL METHODS

The turbine building complex was evaluated utilizing a three-dimensional finite element model which is described in Section 3.1. The analysis was accomplished utilizing five linear elastic analysis methods: (1) modal analysis, (2) response spectrum analysis, (3) time history analysis, (4) equivalent static analysis, and (5) static analysis.

The modal analysis was used to determine the modal frequencies, mode shapes, modal participation factors, and the composite modal damping. The response spectrum analysis, static analysis, and equivalent static analysis techniques were used to evaluate the overall stability of the structure and to compute stresses.

The time history analysis method was used to calculate in-structure response spectra which were used in subsystem evaluations. The response spectrum method was used to calculate forces, moments and stress resultants in the dynamic structural model. Both analysis techniques utilized eigenvalues and eigenvectors that were calculated by the subspace iteration method. For the response spectrum analysis, modal responses were combined in accordance with the provisions of USNRC Regulatory Guide 1.92. In the time history analysis, modal responses are combined directly by adding the computed responses at each time step. These analysis techniques are described in detail in Reference 1.

The responses for each of the three global axes of the model were computed separately in both the response spectrum and time history analyses. The resulting structural responses due to each of the three components of earthquake motion were then combined utilizing the SRSS method as described in Section 3.7.2.6 of Reference 1. These analyses were performed utilizing the Bechtel Structural Analysis Program (BSAP) computer code. In addition to the BSAP computer code, the SPECTRA computer code was employed to compute response spectra from acceleration time histories. A description of each of these codes, along with information pertaining to the validation and extent of application for each program, is presented in Reference 5.

Masonry wall subsystems that were determined to be capable of inelastic response and for which inelastic deformation was acceptable were evaluated by nonlinear analysis. The time history response technique was used for the nonlinear analysis of the masonry block walls associated with the turbine building. For time history response, the DRAIN 2D computer code was used. A description of this code, along with information pertaining to the validation and extent of application for the program is presented in Reference 6.

The procedure utilized to account for the effects of soil-structure interaction is delineated in Section 3.7.2.4 of Reference 1. The soil medium was represented in the finite element model by including three translational and two rotational linear spring stiffness values and their corresponding damping values. The soil-structure interaction methodology utilized for the reevaluation is described in Reference 3.

Two separate analysis cases of the turbine building complex were considered. The first case considered the gantry crane located on the south end of the north extension and the second case considered the crane on the south end of the south extension. These two cases result in the most severe stress conditions for the turbine complex.

Interaction effects between the turbine building, turbine pedestal and the fuel storage building were considered since they are interconnected at the foundation level.

3.1 Mathematical Model

A three dimensional finite element model of the turbine building complex was developed. This model included the four extensions of the turbine building in conjunction with the turbine-generator pedestal, gantry crane and a lumped mass representation of the spent fuel pool. The model is shown in Figure 21. The model consists of 1183 nodal points; 388 plate elements representing the post-tenisioned concrete decks, the pedestal deck and the pedestal shear walls; 837 beam elements representing the structural steel columns and girders of the four extensions, the reinforced concrete columns and beams of the turbine pedestal, the structural elements of the turbine Gantry Crane, the lumped mass representation of the spent fuel pool and the out-of-plane properties of the masonry walls; 43 truss elements representing the diagonal bracing modifications; 238 boundary elements representing the stiffness characteristics of the soil media; 68 direct links representing the tension-compression only connection between the masonry walls and the steel framing at the deck level; and 115 rigid links representing the rigidity of the pedestal basemat, the rigid connections of the structural steel columns to the pedestal basemat and the rigidity of the massive reinforced concrete connections throughout the turbine-generator pedestal.

The enclosure masonry walls at the periphery of the extensions, although not structural walls, are connected at the deck level to the steel framing with ties. These masonry walls were represented by a grillage of beams having equivalent linear elastic properties, based on an independent nonlinear analysis of the walls. This assured that out-of-plane reaction of the wall due to inertia forces was properly transmitted to the steel framing.

The connections of the pedestal basemat to the columns and walls of the pedestal and to the turbine building steel columns founded on the basemat are essentially rigid. The pedestal basemat itself is considered to be rigid due to its thickness (5 feet). The post-tensioned concrete decks for each of the four extensions are considered to act as rigid diaphragms due to their construction and material properties. All of the above items were modeled in a manner consistent with their rigid characteristics.

Soil-structure interaction effects were considered in the analysis, by representing the soil medium by equivalent spring stiffness values. A set of horizontal and vertical translational springs and a set of rotational spring stiffnesses were attached to the individual columns at the foundation level. The soil stiffness properties were determined based on the elastic half space theory using strain dependent soil properties. Because of its thickness, the basemat of the pedestal was assumed rigid and a single set of soil stiffness values was utilized at the center of the mat.

3.2 Modal Analysis

The modal analysis of the turbine building complex was performed utilizing the BSAP computer code. The details of the three-dimensional finite element model of the structure used for the modal analysis are presented in Section 3.1. A subspace iteration algorithm was used to calculate the first 140 frequencies and mode shapes for the dynamic model. The maximum modal frequency computed was 14.33 Hz, which is within the constant acceleration range of the Housner design response spectra for 7% damping. The calculated modes accounted for 99% of the vertical mass and approximately 90% of the horizontal mass associated with the structure. All mode shapes were plotted and examined to insure that the dominant modes depicted shapes consistent with expected dynamic behavior associated with this structure.

The modal analysis was also utilized to compute composite modal damping values, based upon the strain energy weighting method described in Reference 4. The strain energy weighting method was used to incorporate different damping values associated with various elements of the dynamic model (i.e., concrete, steel, soil). Tables 1 and 1A provide a listing of the characteristics of the dominant frequencies and their composite damping values.

3.3 Response Spectrum Analysis

The response spectrum analysis of the turbine building complex was performed utilizing the BSAP computer code. The mode shapes, frequencies and participation factors, which were computed in the modal analysis as described in Section 3.2, were employed in the response spectrum analysis. The computed composite modal damping ranged from 4.00% to 50.8% of critical damping. The maximum modal damping was conservatively limited to 20% for the response spectrum analysis. Design response spectra curves for 4%, 7%, 10%, 15% and 20% of critical damping were utilized for the analysis. For modes with damping values other than these values, logarithmic interpolation was utilized to compute the actual spectral displacement associated with these modes. The program uses the response spectrum curves that most closely bracket the modal damping ratio for the interpolation.

The resulting structural responses obtained from the response spectrum analysis consist of moments, shears, and forces for the various elements that comprise the finite element model.

3.4 Time History Analysis

The time history analysis of the turbine building complex was performed utilizing the three-dimensional finite element model described in Section 3.1. The analysis was performed using the BSAP computer code. The results from the modal analysis were utilized in the time history analysis. Like the response spectrum analysis, the maximum modal damping for the time history analysis was conservatively limited to 20% of critical damping. The input ground motion for the time history analysis was a free field synthetic time history of 20 seconds duration, digitized at a time interval of 0.01 seconds. The free field time history record was developed in accordance with the provisions of the Standard Review Plan (SRP), subsection 3.7.1. The time history analysis of the turbine building complex was used to develop instructure response spectra and structural displacements.

3.5 Equivalent Static Analysis

The equivalent static analysis method was used for the structural evaluation of various structural elements of the turbine building complex, as illustrated in Section 4. The instructure response spectra developed by the time history analysis were utilized to determine the appropriate acceleration coefficients for the various elements being analyzed. The fundamental frequency of the element being analyzed was computed and its corresponding acceleration coefficient was obtained from the appropriate response spectrum curve. If the computed frequency was within the resonance region of the amplified response spectrum curve, the resulting acceleration coefficient was increased by 50 percent to conservatively account for any increased participation from other modes. The resulting acceleration coefficient was then used to compute the moments, shears, and forces attributed to the seismic loading.

3.6 Static Analysis

The turbine building complex was analyzed for static load conditions using the three-dimensional finite element model of the structure with a fixed base. A detailed description of the finite element model is presented in Section 3.1. The static loads analysis was performed using the BSAP computer code. The static loads include: (1) dead load due to mechanical and electrical subsystems and components and the structure itself, and (2) live loads of 20 psf on the area. The resulting forces, shears, moments and displacements were computed for all elements in the model.

3.7 Masonry Wall Analysis

Masonry wall subsystems in the turbine building were determined to be capable of withstanding substantial inelastic deformations transverse to the plane of the wall without adversely affecting the structural integrity of these walls. This is due to the presence of vertical reinforcement, the location of the walls, and the function of the walls. Therefore, the out-of-plane responses of several masonry wall subsystems were analyzed inelastically and evaluated against appropriate ductility ratios in accordance with the acceptance criteria of

Reference 6. In-plane shears were evaluated separately and the results were considered with the transverse response.

The inelastic analysis of masonry walls was performed by Computech Engineering Services, Inc. An analytical model was developed by Computech based upon the evaluation of data taken from tests of masonry walls. This model was two-dimensional and consisted of planar elements to simulate the concrete block, gap elements at midspan to simulate crack formation, and tension only elements to model the central reinforcement (see Figure 20).

After verification that the modelling assumptions adequately simulated the response of test specimens to cyclic loading, a model was developed for the nonlinear analysis of each masonry wall being evaluated. The nonlinear analysis was performed by the time history direct integration method using the DRAIN 2D Computer Code. The analytical model was excited by several actual earthquake records which were scaled to envelope the 2/3g Housner Response Spectrum.

Detailed descriptions of the modelling procedure, the model verification, evaluation criteria, and the analysis results for out-of-plane loadings are provided in References 6, 7 and 8.

4.0 STRUCTURAL EVALUATION

This section provides the results of the structural evaluation of the four extensions of the turbine building, the turbine generator pedestal and the turbine gantry crane. Unless otherwise specified herein the reevaluation criteria by which the analytical results were reevaluated are given in Section 3.8.4 of Reference 1. In general the basis for criteria governing the stresses within the elastic range as described therein is current day code requirements.

For steel column and beam members if the resultant stresses were in the inelastic range an appropriate inelastic interaction equation was used. In the structural evaluation of steel beam-columns, biaxial bending of the members along with their axial loads was considered. For most of the members the stresses due to applied loads were in the elastic allowable range. When calculated stresses were in the inelastic range, an appropriate plastic interaction equation taking into account biaxial bending was used. The basis for this equation is described in the following paragraphs.

The AISC specification (Ref. 9) does not provide for the ultimate strength of columns in biaxial bending, the Code provisions are for single axis bending only. In this evaluation both the major axis and the minor axis moment were taken into account along with the axial load in determining the ultimate capacity of the steel columns. The interaction equation used was:

$$\frac{P}{Py} + \frac{1}{1.18}\frac{Mx}{Mpx} + \frac{1}{1.67}\frac{My}{Mpy} \leq 1.0 \qquad (4\text{-}1)$$

P = Applied axial load, kips
Py = Yield stress x section area (F_y x A)
Mx = Applied moment, major axis, kip-inches
My = Applied moment, minor axis, kip-inches
Mpx = Plastic moment capacity, major axis, kip-inch (Z_x x F_y)
Mpy = Plastic moment capacity, minor axis, kip-inch (Z_y x F_y)

The first two terms represent the single axis bending as defined by equation 2.4-3 of the AISC specification. The interaction equation relation for axial load and minor axis bending only is:

$$\frac{P}{P_y} + \frac{1}{1.67}\frac{My}{M_{py}} \leq 1 \qquad (4\text{-}2)$$

per Reference 9.

The Structural Stability Research Council's Guide to Stability Deisgn Criteria for Metal Structures, 3rd Ed. (Reference 10) and the latest Canadian Standards Association code, Steel Structures for Buildings - Limit States Design (Reference 11), state that the ultimate design capacity of a column subjected to biaxial beding can be represented by equation (4-1). Therefore this equation was used in evaluating the ultimate strength of the beam-columns when they were stressed inelastically. In the tables wherever equation (4-1) was utilized it is noted with the superscript "b".

The acceptance criteria for concrete structural members include increases in concrete compressive strentgh of 5,000 psi was used for the reevaluation of the pedestal as compared to the original specification which required a minimum 28-day compressive strength of 4,000 psi. The turbine building foundations were reevaluated utilizing a concrete compressive strength of 4,500 psi as compared to the original specified strength of 3,000 pse. There are several factors that indicate the actual overall compressive strength of the insitu concrete is in excess of 5,000 and 4,500 psi respectively. First, the 4,000 and 3,000 psi values are minimum allowables and experience with large volume placements of concrete in this strength range shows that actual test results at 28 days are generally in excess of the required minimum. Secondly, a review of References 12 and 13 indicates a general increase in strength of concret over a time span of 10 years when compared to conventionally controlled cylinders. In some cases the compressive strength more than doubled.

Another factor which predicts increased compressive strength in the in-situ concrete is that Type II Portland cement was specified for the mix. Experience has shown that this cement would be expected to provide a better-than average strength gain after 28 days. The last factor considered was the results of two separate tests conducted in early 1977 on the ractor building concrete inside the San Onofre Unit 1 containment. Five tests using the Windsor Probe showed an average compressive strength of 6,440 psi and seven tests cases were f'c = 3,000 psi. These test results are both based on the manufacturer's calibration curves supplied with the instruments. Since no direct calibration of the test instrument aganst compressive strength specimens is available, these results can only be considered as indicative of the strength of the existing concrete. It should be noted that a suitable strength reduction would be applied to the above values to provide ACI 318 statistical assurance that the concrete meets the strength requirements. In this case (using the Windsor Probe values), the 6,400 psi average strength would be reduced by 550 psi (since the standard deviation is 360 psi), giving a usable f'c = 5,890 psi.

Therefore, taking into account all of the above factors, conservative values of up to 50% more than the original minimal design values for the in-situ concrete compressive strengths were utilized in the structural reevaluation of the turbine pedestal and turbine building foundations.

4.1 North Extension Structural Evaluation

This section summarizes the results of the structural evaluation of the north extension framing. The north extension framing consists of two bays in the east-west direction and two bays in the north-south direction. The north extension operating deck has a 7 foot 6 inch cantilever overhang east of column line 6 and a 4 ft 8 inch overhang west of column line 8. At elevation 30' - 0" there is a mezzanine framing between column lines A and B. The mezzanine provides access to the main steam valves and some controls instrumentation.

There are six new bracing members installed in the north extension. The sizes of these members vary from W12 x 120 to W14 x 330. The bracing members are designed as tension-compression members. By installing these members in both the north-south and east-west directions the existing moment carrying frames are converted to braced moment carrying frames. These stiff members impart significant stiffness to the frames in the vertical and horizontal directions.

Introduction of these members along with the required foundation modifications have changed the fundamental frequency of the north extension from 1.7 Hz to 4.7 Hz for north-south translation. These bracing members carry a significant portion of the lateral load, help reduce lateral displacements and thus reduce the column moments. The locations of the bracing members in the north extension are shown in Figure 22. The sizes of the braces and the stress levels associated with them are tabulated in Table 2. The axial stresses compared with elastic allowable axial stresses indicate a very conservative range in safety factors of 1.34 to 1.51.

The evaluation of the girders and beams is tabulated in Tables 3 and 4. All the girders evaluated have stress levels within the allowable limits. The safety factors which compare actual bending stress vs. allowable bending stress range from 1.05 to 1.80. The crane rail girders have safety factors of approximately 1.5. The stress levels in all the girders and secondary beams indicate that they remain within the elastic range.

The evaluation of the mezzanine beams (see Table 4) shows that all of the beams, except beam NEM-B4, have stresses within the allowable limits. For beam NEM-B4 the factor of safety was 0.80, therefore this member will be strengthened by adding cover plates which will ensure that the stresses will remain within the criteria limits.

The column-girder moment connections were evaluated to determine the stress level in the connection region when subjected to the DBE loads. Specifically the beam web shear, column flange stresses below the connection and the adequacy of the stiffeners for beam web crippling were checked. There are eight moment connections in the north extension. In October, 1981 five of the eight moment connections were modified such that they are now capable of remaining elastic while resisting the full plastic moment capacity of the largest connected member. The locations of these strengthened moment connections are at the top of columns B-6, B-7, B-8, D-6 and D-8. For all the moment connections the column flanges are connected to the underside of the girder with full penetration welds. The web of the column is fillet welded to the underside of the girder. The girder spans continuously over the columns. The stiffeners inside the flanges of the girder, which are located directly in line with the column flanges, are fillet welded to the girder web and flanges.

The results of the moment connection evaluations are tabulated in Table 5. For all the connections the column flange stresses are within the elastic allowable limit of 35.2 ksi. The actual stress values vary from 6.49 ksi to 24.1 ksi. The check on the stiffener area provided versus the area required results in safety factors ranging from 1.15 to 3.47. The beam-web shear stress values vary from 7.81 ksi to 21.0 ksi and have a safety factor range of 1.10 to 4.56 for all connections with the exception of the connection at A-7. The moment connection at column A-7 will be modified during the present outage by adding a web doubler plate, thus ensuring that the resulting stresses will remain within the allowable limits.

The bolted connections primarily use 3/4" ϕ, with some 1" ϕ, ASTM A 325 high strength bolts. All these connections were detailed as shown in Part 4 of the AISC Manual of Steel Construction, Sixth Edition. The results of the adequacy of the bolted connections is tabulated in Table 6. The factor of safety, which compares the applied shear force with the shear capacity of the bolted connection, ranges from 1.64 to 4.24, well within acceptable limits.

The results of the column evaluations are listed in table 7. In this evaluation the column interaction equations given in section 1.6.1 of the AISC manual were used. If the combined stress-factor indicated that the maximum stress on some portion of the section was above the elastic yield then the ultimate strength of the member was compared with the applied forces on the column using equation (4-1). The summary of results indicate the safety factors comparing the ultimate strength versus applied forces vary from 1.06 for column D-8 to 1.49 for column A-6, with the exception of column B-6 whose safety factor was 0.99. The overstressed portion of column B-6 is located at grade and flange cover plates will be added to the column during the present outage. This will ensure that the column stresses will remain within the elastic allowables when subjected to the seismic loads.

In the evaluation of the column anchorages the adequacy of the anchor bolts and the baseplates were examined. If the bending stesses in the base plate exceeded the elastic allowable stresses then the plastic moment capacity of the base plate was compared with the applied moment. The results are tabulated in table 8. All the anchor bolts are made from A193 steel (allowable tension = 101 ksi). The stresses in the anchor bolts vary from 5.3 ksi at B-7 to 92.2 ksi at column A-7, with the exception of D-6 which has a stress of 124 ksi. The anchorage at column D-6 will be strengthened during the present outage by installing rock bolts directly into the pedestal mat.

The base plate evaluations show that the bending stresses in all the base plates are within the criteria allowables, except at A-7 and D-6. At column D-6 the base plate and anchor bolt assembly will be modified as stated above. For the column A-7 base plate the calculated moment exceeds the plastic moment capacity by approximately 10%. However, the base plate is encased to a depth of 2 feet in the concrete column cap (see Figure 40) which has approximate plan dimensions of 4 feet by 3 feet. In computing the baseplate moments the restraint applied by the reinforced concrete column cap in conjunction with the slab at grade and the subsequent reduction in the applied moment on the anchorage was not considered. Because of this conservatism in the analysis it is concluded that the baseplate at column A-7 is adequate.

The existing foundations in the north extension have been modified to resist the increased uplift due to DBE loads. The modifications made are shown in Figure 17. New north-south grade beams were placed between columns A6 and B6 and columns A8 and B8, while an east-west grade beam was placed between columns B6, B7, and B8. Dimensions of the new grade beams are approximately 10'-0" wide and 5'-6" deep. Each grade beam was rigidly connected to the existing footings by drilling and grouting in threaded rock bolts and mechanically connecting them to threaded rebar in the new grade beams. A typical section of the structural connection is shown in Figure 18.

The design of the foundation modifications are such that allowable soil bearing pressures and footing (concrete & reinforced steel) stresses are not exceeded. The resulting soil pressures range from a minimum of 9.7 ksf to a maximum of 10.4 ksf which occures along the new east-west grade beam. The maximum allowable bearing pressure for the soil is 40 ksf, resulting in a factor of safety of 3.85. The analysis of the new foundations revealed that the ultimate design moment of 484 K-FT (obtained in East-West grade beam) was much less than the foundation's moment capacity of 1293 FT-K, resulting in a factor of safety of 2.5.

4.2 West Heater Platform Structural Evaluation

The following section summarizes the results of the structural evaluation of the west heater platform framing. The west heater platform consists of six bays in the north-south direction and two bays in the east-west direction. A small extension to the framing is present at the south-east corner of the structure. The plan view of the west heater platform at the deck level (elev. 35'-6") is shown in Figure 9.

There are fourteen new bracing members installed in the west heater platform framing. The sizes of these members vary from W12 x 79 to W12 x 170. The locations of the braces are shown in Figures 28 and 29. Three of the braces in the east-west direction are located exterior to the west side of the west extension. The bracing members have converted the moment carrying frames into braced moment carrying frames.

By installing the stiff bracing members and the associated foundation modifications the stiffness of the west platform framing has increased significantly in both of the lateral directions. The fundamental frequency of the structure has increased from 1.7 Hz to 4.1 Hz in the east-west direction and from 2.0 Hz to 5.5 Hz in the north-south direction. The bracing carries a significant portion of the lateral load, reducing the lateral displacements and also reducing the column moments. The sizes of the braces and the stress levels associated with them are tabulated in Tables 9 and 10. The axial stresses compared with the elastic allowable axial stresses indicate a very conservative range in safety factors of 1.67 to 3.68.

Girder sizes vary from W24 x 84 to W24 x 120 while typical sizes of secondary beams range from W14 x 30 to W24 x 76. The results of the beam and girder evaluations are tabulated in Tables 11 and 12. The safety factors comparing the actual bending stress with allowable bending stress range from 1.02 to 3.81. The exceptions are beam J12-K12 with a safety factor of 0.93, and beam C11-C13, with a safety factor of 0.97. Cover plates will be added to these two beams to ensure their adequancy.

The column-girder moment connections were evaluated to determine the stress level in the connection region when subjected to DBE loads. Specifically the beam-web shear, column flange stress, and the adequacy of existing beam web stiffeners in preventing web crippling were checked. There are seventeen moment connections in the west heater platform framing. In all connections, the column flanges are connected to the bottom flange of the girder with a full penetration weld. The girder spans continuously over the column. The stiffeners are located inside the flanges of the girders, directly in line with the column flanges to provide a continuation of the flanges. The stiffeners are typically fillet welded to the web and flanges of the girder.

As shown in Tables 13 and 14 the moment connections in the west heater platform have column flange stresses well within the allowable limits of 35.2 ksi. The actual stress values vary from 8.14 ksi to 22.10 ksi. Beam web shear stresses ranged from 6.46 ksi to 18.7 ksi resulting in a safety factor range of 1.23 to 3.56, well within the acceptable limits. The evaluation of beam web crippling indicated that the size of the stiffeners provided are adequate. Calculated safety factors (ratio of area of stiffener provided to area required) ranged from 2.61 to 8.96.

The bolted connections in the west heater platform use 3/4" ASTM A 325 high strength bolts. All connections were detailed as shown in Part 4 of the AISC Manual of Steel Construction, Sixth Edition. The results of the evaluation of the bolted connections are tabulated in Table 15. The factor of safety for these connections, which compares the applied shear force with the shear capacity, range from 1.00 to 1.92 and is acceptable.

There are seventeen major structural columns (sizes vary from W24 x 100 to W24 x 145) and six small size columns (W8 X 31). Evaluation of the columns for the forces obtained from the response spectrum analysis is tabulated in Tables 16 and 17.

The tabulation of the results shows that all the structural columns are acceptable, except column C13. For the columns meeting the criteria the safety factors which compare the combined stress factors with the allowable combined stress factors range from 1.01 for column H9 to 2.50 for column C9. Column C13 is a W8 x 31. This small column does not meet the criteria allowables but will be strengthened by adding a structural tee during the present outage. The installation of this modification will enable the column to remain within its elastic limits when subjected to the design loads.

In the west heater platform, fifteen column anchorages are designed as moment resisting connections. This is achieved by providing stiffeners perpendicular to and on the outside of the column flanges with a horizontal steel plate connecting the tops of the stiffeners. The horizontal plate provides a seat for the anchor bolt nuts. A typical moment resisting column base plate assembly is shown in Figure 40.

In the evaluation of the column anchorages, the adequacy of the anchor bolts and the base plates was examined as well as the stresses in the supporting foundation. In calculating the bending stress in the base plate, if the calculated stress exceeded the elastic limit stress, then the plastic moment capacity of the base plate was compared with the applied moment in the base plate. Results for the column anchorages are tabulated in Tables 18 and 19. Calculated anchor bolt tensile stresses for the A193 steel bolts are all within the elastic allowable of 101 ksi. The tensile stress values for these bolts ranged from 0.9 ksi to 85 ksi.

However, at column anchorages C-11 and C-13 the existing anchor bolts are A 307 material. The tensile stress on the bolts exceeded the allowable stress by a large margin. The base plate anchor bolt assemblies at locations, C9, C11 and C 13, have been modified during the present outage by increasing the base plate area and providing additional anchor bolts of A193 material. The new configuration has stresses within the elastic allowable limits. Base plate bending stresses were all within the elastic allowable of 36 ksi except at column K12. However, at K12 the base plate did not exceed its plastic moment capacity (factor of safety of 1.07) and is therefore acceptable. The allowable bearing stress on the concrete was not exceeded with stresses ranging from 0.36 ksi to 2.63 ksi.

Existing foundations in the west heater platform have been modified to resist the increased uplift loads resulting from the DBE loading. As shown in Figure 17, three types of modifications were made; 1.) increase the size of an existing footing; 2.) provide a grade beam between two existing footings, or 3.) construct a new independent foundation. Column foundations at columns C9 and L9 were increased in size with additional widths and lengths of approximately 3 feet by 8 feet added at C9 and 1 foot by 9 feet at L9 respectively. New north-south grade beams were placed between columns C11 and E11 as well as along column line 13, connecting columns C13, E13, F13, G13, and H13. In addition, at the south end of the west platform grade beams were placed between columns J13 and K13 and K12 and K13. At K13 a 16 foot extension was added extending to the west of column K13.

For the two cases of modifying foundations previously described, (increase in foundation size or addition of a grade beam) a rigid connection was made between the existing footings and the new foundations by drilling and grouting threaded rock bolts in the existing footing and mechanically fastening them to threaded rebar in the new foundations. A typical section showing the structural connection is detailed in Figure 18. Two locations in the west platform required the placement of a new, independent foundation. Seismic bracing extending outside of columns G13, H13, and L10 required new foundations.

The new foundation modifications were designed such that allowable soil bearing pressures as well as foundation (concrete & reinforcing steel) stresses were not exceeded. Resulting soil bearing pressures for the foundation modifications range from 2.1 ksf to 13.1 ksf. The maximum pressure of 13.1 ksf, which occurred at the enlarged footing at column C9, is well within the 40 ksf allowable value, thereby giving a safety factor of 3.0. Ultimate moment capacities of all modified footings were never exceeded. The continuous grade beam from columns C13 to H13 resulted in a moment of 7231 ft-K compared with its ultimate capacity of 8000 ft-K while the grade beam along column Line K reached a moment of 2916 ft-K, which is less than its capacity of 3984 ft-k.

4.3 East Heater Platform Evaluation

In this section the results of the reevaluation of the structural members in the east heater platform are summarized. The deck level framing is shown in Figure 8. In the east platform framing there are 5 bays in the north-south direction and two bays in the east-west direction. There is also a small extension to the framing at the southwest corner of the structure.

In the east heater platform framing fifteen new bracing members have been identified and included in the analysis. The sizes of bracing members vary from W12 x 79 to W12 x 120 as shown in Figures 13 and 14. The bracing members will convert the existing moment carrying frames into brace moment carrying frames. By introducing the bracing members and associated foundation modifications the stiffness of the east heater platform framing will increase significantly in both horizontal directions. The fundamental frequency of the extension will increase from 1.7 Hz to 4.60 Hz in the east-west direction and from 2.0 to 5.8 Hz in the north-south direction.

The stiff bracing members resist a significant part of the total lateral load, reducing the lateral displacements and the column moments. The size of braces, the stresses associated with them and their comparison with elastic allowable axial stresses are shown in Tables 20 and 21. The factors of safety for the bracing members, which compare calculated stresses with elastic allowables, range from 1.32 to 2.60.

The results of the evaluation of the girders and secondary beams are shown in Tables 22 thru 24. All of the girders and beams except the two beams as noted below meet the acceptance criteria. The safety factors which compare the allowable bending stresses with the calculated bending stresses vary from 1.03 to 4.7 for the acceptable beams.

In the north-south direction girders E1-F1 and G2-H2 have bending stresses of 42.5 ksi and 44.9 respectively. The girder between E1 and F1 is a W21 x 55 member and has a simple span of 27.5 feet, a relatively large span. The member between G2 and H2 is a W16 x 36 with a simple span of 8 feet. Both of these will be modified by adding steel coverplates.

The existing column girder moment connections were evaluated to determine the stress level in the connection region when subjected to the DBE loads. Specifically the beam-web shear, column flange stresses and the adequacy of the stiffeners for the beam web crippling were checked. There are sixteen moment connections in the east heater deck framing. For all the connections the column flanges are connected to the underside of the girder with full penetration welds. The girder spans continuously over the columns. The stiffeners inside the web of the girder, which are located directly in line with the column flanges, are fillet welded to the girder web and flanges.

The results of the evaluation of the column girder moment connections are given in Tables 25 and 26. For all the connections the column flange stresses below the connection ranged from 5.27 ksi to a maximum of 19.77 ksi. These are well within the allowable of 35.2 ksi for A36 steel. The beam web shear stresses calculated ranged from 3.42 ksi to 16.94 ksi. These are well within the allowable stress of 23 ksi. The average value of the beam web shear stress was about 12 ksi. The evaluation of the beam web crippling indicated that the size of the stiffeners provided are more than adequate. The safety factors comparing area of stiffener provided versus area required varied from 1.76 to 23.5.

The bolted connections primarily use 3/4"Ø, with some 1"Ø, ASTM A 325 high strength bolts. All these connections were detailed as shown in Part 4 of the AISC Manual of Steel Construction, Sixth Edition. The results of the adequacy of the bolted connections are tabulated in Table 27. The factor of safety, which compares the applied shear force with the shear capacity of the bolted connection, ranges from 1.11 to 3.98, well within acceptable limits.

The East Heater Platform framing has sixteen major structural columns (sizes vary from W24 x 100 to W24 x 130) and four small size columns (W8 x 31). Evaluation of the columns for the forces obtained from the response spectrum analysis is tabulated in Tables 28 and 29.

The tabulation of the results shows that all the structural columns are acceptable, except the two columns, E1, and F2. For the columns meeting the criteria the safety factors which compare the combined stress factors with the allowable combined stress factors range from

1.19 for column H2 to 2.29 for column K1. Column E1 is a W8 x 31 and column F2 is a W24 x 130. These two columns do not meet the criteria allowables but will be strengthened by adding coverplates and structural tees. The installation of these modifications will enable the two columns to remain within their elastic limits when subjected to the design loads.

In the evaluation of the column anchorages the adequacy of the anchor bolts and the base plates were examined. If the bending stresses in the base plate exceeded the elastic allowable stresses then the plastic moment capacity of the base plate was compared with the applied moment. The results are tabulated in Tables 30 and 31. Most of the anchor bolts are made from A193 steel (allowable tension = 101 ksi) and the base plates are A36 steel. The calculated stresses in the anchor bolts vary from 6 ksi to 79.2 ksi and are less than the 101 ksi allowable tensile stress. At column anchorages E1, E3, J2, K1 and L5 the tensile stress exceeds the allowable. The column base anchorage at E3 will be modified by providing a knee brace to the column and a new base plate and anchor bolt assembly. Modifications will also be designed for anchorage at columns E1, J2, K1 and L5. Base plate bending stresses were all within the eleastic allowable of 36 ksi except at columns E1 (123 ksi), E3 (109 ksi) and J2 (78.4 ksi). Base plate modifications at these columns will be provided. The allowable concrete bearing stress was exceeded only at columns E1 (6.59 ksi) and E3 (6.03 ksi). Base plate modification at these columns will result in a reduction of concrete bearing stress to within allowable limits.

Existing foundations in the east heater extension are to be modified such that increased uplift loads from the DBE will be resisted. Figure 17 gives a plan view of the new foundation modifications. New grade beams extending between columns E1, F1, G1, and F2 will be installed resulting in a "T-shaped" combined footing. A similar modification will be done connecting columns H1, J1, K1, and J2. Each portion of the T-shaped footing will be approximately 12' wide and 8' deep. The individual spread footing at column E3 will be increased in size by approximately 8' in length and 4' in width. For both the T-shaped footings and the footing at column E3, rock bolts will be drilled and grouted into the existing foundation and then attached to the new foundation by threaded rebar, thereby providing a rigid connection between the two foundations. A typical section detailing the structural connection is shown in Figure 18. In addition to the previously mentioned modifications, an individual spread footing will be constructed approximately 30 Ft. south of column L5. This footing will be 12' wide by 8' deep and will be be used to provide anchorage for the outside brace extending from column L5.

Foundation modifications were designed such that allowable soil bearing pressures and allowable footing stresses (concrete and reinforcing steel) were not exceeded. Soil pressures in the east extension range form 2.2 ksf to a maximum of 20.1 ksf which occurred in the T-shaped combined footing connecting columns E1, F1, 6±, and F2. The allowable bearing pressure of the soil is approximately 40 ksf giving a factor of safety of 2.

4.4 South Extension Structural Evaluation

This section summarizes the results of the structural evaluation of the south extension framing. In the south extension framing there are two bays in the east-west direction and two bays in the north-south direction. The framing along column lines six and eight is extended south of the extension by extending the crane rail girder one more bay with a contilever (see Figure 7). This extension south of the main structure is used for keeping the gantry crane parked or for unloading the spent fuel cask form the deck level into the transport vehicles.

There will be eight new braces installed in the south extension framing. All the braces will be installed in the frames at the periphery of the south extension. The sizes and locations of the braces are shown in Figure 15. The bracing members will significantly increase the stiffness of the south extension framing. The fundamental frequency of the south extension in the east-west direction will be 8.9 HZ and in the north-south direction 8.3 Hz. The stress levels in the bracing members and the elastic allowable stresses are listed in Table 32. The safety factors vary from 1.87 to 4.14 and are accepatable.

The results of the evaluation of the beams and girders are tabulated in Table 33. The girder sizes vary from W24 x 84 to W36 x 230 for the crane rail girder. Typical sizes of secondary beams are between W16 x 36 to W24 x 68. The overall safety factor comparing combined stress factor with allowable combined stress factor ranges from 1.18 to 5.00. Therefore all the beams and girders located in the south extension satisfy the acceptance criteria.

The existing column girder moment connections were evaluated to determine the stress level in the connection region when subjected to the DBE loads. Specifically the beam-web shear, column flange stresses and the adequacy of the stiffeners for the beam web crippling were checked. There are ten moment connections in the south extension deck framing. For all the connections the column flanges are connected to the underside of the girder with full penetration welds. The girder spans continuously over the columns. The stiffeners inside the web of the girder, which are located directly in line with the column flanges, are fillet welded to the girder web and flanges.

The results of the moment connection evaluations are listed in Table 34. For all of the connections the column flange stresses are well within the allowable limits. The actual stress values vary from 6.89 ksi to 14.0 ksi (stress allowable: 35.2 ksi). The beam web shears range from 5.28 ksi to 10.4 ksi and have a safety factor range of 2.2 to 4.36, well within acceptable limits of the criteria. The evaluation of the beam web crippling indicated that the size of the stiffeners provided are more than adequate. The minimum safety factor comparing area of stiffener provided versus area required was 9.91.

The evaluation of the bolted connections is tabulated in Table 35. The bolted connections were made with 3/4" Ø and 1" Ø bolts, a 325-F type. The safety factors which compare the applied shears versus the shear capacity of the connection varies from 1.57 to 5.73 and are acceptable.

There are ten major structural columns with sizes varying from W24 x 100 to W24 x 130. Table 36 summarizes the results of the column evaluation. All of the columns meet the criteria with safety factors (comparing combined stress factors with allowable combined stress factors (ranging from 1.26 to 1.90.

For the south extension columns there are ten column anchorages that are designed to resist moments at the base of columns. The moment resisting connection consists of stiffeners outside of the column flanges, perpendicular to the column flanges; connecting the tops of the stiffeners with a horizontal steel plate with holes for the anchor bolts. This horizontal plate provides the seat for the nit for the anchor bolt. A typical anchor bolt base plate assembly is shown in Figure 40.

The evaluation of the column anchorages included the adequacy of the anchor bolts and the column base plates. The base plates are A36 steel and the anchor bolts are A193 steel. As shown in Figure 40 all the base plates are encased in the column cap concrete. In calculating the bending stresses in the base plate if the calculated stress exceeded the elastic limit stresses, then the plastic moment capacity of the base plate was compared with the applied moment on the base plate. The calculated results are tabulated in Table 37. These indicate that the anchor bolt tensile stresses are within the elastic allowable stress of 101 ksi (except columns M6, M8, R6 and R8) with the values ranging from 9 ksi to 39.3 ksi. The anchor bolts for columns M6, M8, P6, R6 and R8 had tensile stresses greater than the elastic allowable limits ranging from 148 ksi for P6 to 197 ksi for R8.

The column base plates met the criteria allowables, except for column P6, with base plates at M6 and M8 being the most highly stressed of the base plates that are acceptable. The base plate thickness provided for columns M6 and M8 is acceptable with calculated bending moment being about 50% of the plastic moment capacity of the base plate. For the base plate at column P6 the calculated moment of 45.9 k-in/in exceeds the plastic moment capacity of 40.8 k-in/in by a margin of 13%.

The anchorage of columns M6, M8, P6, R6 and R8 will be strengthened by providing additional anchor bolts and increasing the area of the base plates. By installing these modifications the design safety margins for the anchorages will be restored.

The existing foundations in the south extension of the turbine building are to be modified to resist the increased uplift loads resulting from the DBE. A plan view of the modified foundations is shown in Figure 17. New north-south grade beams will be constructed from columns M6 to N6 and M8 to N8 while a continuous east-west grade beam will be added, connecting columns P6, P7 and P8. The two north-south grade beams will be approximately 12 ft. wide and 8 ft. deep while the continuous footing along column line P will be approximately 10 ft. wide and 6 ft. deep. Each of the three new grade beams will be rigidly attached to the existing foundation that they span between. Rock bolts will be drilled and grouted into the existing foundation and mechanically fastened to threaded rebar in the new grade beams. A typical section detailing the structural connection is shown in Figure 18. In addition to the grade beams, one spread footing will be constructed approximately 20 ft. east of column P6. The new footing which is approximately 12 ft. square and 7 ft. deep will anchor the outside brace extending from column P6.

The new foundation modifications were sized such that allowable soil bearing pressures as well as allowable stresses in the concrete and steel are not exceeded. Resulting soil bearing pressures range from 0.46 ksf to a maximum of 2.2 ksf which occured along the continuous east-west grade beam. The allowable bearing pressure of the soil was not exceeded and the safety factors ranged from 18 to 87.

4.5 Heater Platforms and Extensions Deck Evaluation

The turbine building north and south extensions and east and west heater platforms support 8 1/2" thick reinforced concrete post-tensioned decks, which are welded to the structural steel framing. During construction the decks were post-tensioned by tendons spaced at approximately 2 1/2 feet on center in the north-south and east-west directions. One year after the completion of the post tensioning the decks were attached to the steel framing by welding the girders to insert plates which were keyed into the deck.

Since the deck is post-tensioned and was designed to support heavy construction loads, including a large live load, its moment capacity was found to exceed the calculated moments for the seismic reevaluation criteria. In view of these results it is concluded that the structural integrity of the post-tensioned decks is assured and that the flexural and shear stresses are within the BOPSSR criteria allowable values.

The existing deck slab has steel insert plates with shear connectors embedded in the deck slab. The steel insert plates are welded to the top of the deck steel framing girders. Depending on the orientation of the insert plates, they were considered to offer resistance in either the north-south or east-west direction only. The deck slab to structural steel framing connection was evaluated to examine if it can resist the lateral loads from the deck slab. For the west heater platform the resistance capacity of the connections was 1.03 times the total applied force in the north-south direction and 1.06 times the total applied force in the east-west direction. Even though the total resistance capacity of the connections is greater than the applied forces, the weld size on the connectons will be increased from 5/16" to 7/16" at some insert plate locations and thus a design margin of 1.1 will be established.

For the north extension the connections can resist 70% of the applied load in the north-south direction. In the east-west direction the connection capacity is 1.92 times the applied force. The insert plate connection capacity for the north-south direction will be increased by increasing weld size and by providing some shear connectors to the deck slab to restore the required design margin. In the south extension the resistance capacity of the connections are 1.15 times the total applied forces in the north-south direction and 1.06 times the total resistance capacity of the connections is greater than the applied forces in the east-west direction. The weld size on the connections in the east-west direction will be increased from 5/16" to 7/16" at some insert plate locations and thus a design margin of 1.1 will be established.

In the east heater platform the connections can resist 65% of the total applied load in the north-south direction and 69% of the total applied load in the east-west direction. The insert plate connection capacity for both the north-south and east-west directions will be increased by increasing weld size and by providing some shear connectors to the deck slab to restore the required design margin.

4.6 Masonry Wall Evaluations

Reinforced masonry block walls are located about the periphery of the east and west heater platforms and the south extension. As is indicated in Figure 19, the only connection fo these walls to the structural steel framing is at the top of the wall by a double pin conndtor that transmits only loads normal to the wall. In-plane loads are not transferred to or from the structural steel framing. In the north extension the only masonry wall is located at the north end of the extension between the framing and the steel containment sphere. This wall is a cantilever wall that is not connected to the structural steel framing.

The out-of-plane loadings were evaluated by nonlinear analysis performed by Computech Engineering Service, Inc. (CES). These analyses are described in References 7 and 8. The acceptance criteria for masonry wall ineleastic behavior is given in Reference 6. The only in-plane loads acting on the walls are their own interia effects, which are insignificant when compared to the out-of-plane loading effects. Therefore, as concluded in the CES report the masonry walls are adequate.

The double pin connector that transmits loads normal to the walls to the steel framing system at the deck level was evaluated. The results of the evaluation show that the connections are adequate and the resulting minimum safety factor is 1.1.

4.7 Turbine Gantry Crane Stability Evaluation

The turbine gantry crane (see Figure 4) is not safety related, however its collapse could impact safety related systems. Therefore, the crane's stability was evaluated as part of the BOPSSR program. The criteria utilized for evaluating the gantry crane was essentially the same criteria that was applied to the structural steel portions of the turbine building.

The gantry crane was modeled with and included in the turbine building complex dynamic analyses as described in Section 3.0. The seismic plus static moments, shears, axial loads and torsion applied to the beam elements of the gantry crane resulted in stresses that were less than the yield point of the steel, with one exception. The exception was the lower horizontal member between the vertical legs of the crane. The results indicate a stress that exceeds the material minimum yield point by about 5%. However, this only means at worst that a portion of the member's cross section might enter the plastic realm of behavior and respond as a partially plastic (ductile) member. Therefore it is concluded that the structural members of the turbine gantry crane are adequate to resist the DBE loadings.

The analysis also show that the turbine gantry crane is able to resist the overturning forces associated with the design event. The factor of safety against overturning in the north-south direction is 4.5, while in the east-west direction it is 60 (allowable factor of safety: 1.1). Therefore the turbine gantry crane is adequate to resist the DBE loads.

4.8 Turbine Building Complex Deflection Evaluation

The time history analysis (see Section 3.4) was used to calculate maximum deflections associated with the four extensions of the turbine building and the turbine-generator pedestal. As shown in Figure 46, there is a 1 1/2" gap between the concrete deck of the north extension and the control building on the east side, pedestal on the south side and the spent fuel pool on the west side. Table 38 shows the maximum deflections of the north extension in the east-west and north-south directions. The maximum north-south deflection is 0.747" and the maximum east-west deflection is 0.729 ". The deflections are small at the deck level because of the stiffening effects of the new bracing members in the north-south and east-west directions.

An independent time history analysis of the fuel storage building has shown that the maximum deflection of the spent fuel pool adjecent to the north extension deck slab is 0.46", thus the Square Root of the Sum of the Squares (SRSS) combined deflection is 0.80, much less than the existing seismic gap of 1 1/2". Table 39 shows the SRSS combined deflections in the north-south direction with the pedestal and in the east-west direction with the control building are all less than the seismic gap of 1 1/2". The north-south deflection of the west heater platform adjacent to the 480V Room of the fuel storage building is 0.327". The SRSS combined deflection of the fuel building with the west heater deck is 0.86" less than the seismic gap of 1 1/2".

The analysis results verify that the bracing is adequate to ensure that there will not be any structure-structure interaction (banging) associated with the turbine building complex.

4.9 Turbine-Generator Pedestal Evaluation

The turbine pedestal is a massive reinforced concrete structure consisting of a five foot thick foundation mat, four large columns, two on the north end and two on the south end, and three large walls oriented in the east-west direction.

The dynamic model of the pedestal included 90 beam elements (deck and columns) and 176 plate elements representing the transverse walls. Soil structure interaction stiffnesses were applied to the base The pedestal model was incorporated into the complete turbine pedestal/ turbine building model as indicated in Figure 21. The combined turbine building/turbine pedestal model was evaluated for seismic loadings in three orthogonal directions; north-south, east-west and vertical. The results of these separate analyses were combined by the SRSS technique. In addition, the following loadings were combined with the seismic loads:

- Concrete and machinery dead loads
- Longitudinal force of 10% of the machinery dead load to account for bearing thrust loads
- Cordem vacuum load
- Short circuit torque load

The turbine pedestal was constructed with concrete having a design strength of f'c = 4,000 psi and reinforcement with a yield point of 40,000 psi. In accordance with the discussion in Section 4.0, the concrete was assumed to have increased in strength by 25%, or to 5,000 psi. Strength reduction factors were applied in accordance with ACI 318-77 Section 9.3. These gave allowable shear stresses of 132 psi (without stirrups), an allowable tensile strength of 263 psi, and a compressive strength of 4,590 psi in bending. The slenderness of the beams, columns and walls were such that capacities were not affected.

The evaluation of the beam elements in the deck determined that the computed loads did not exceed allowable capacities for axial forces, shears and moments in two directions and torsion. It was noted that the beam on Line E, between 6 and 8, had less area of stirrups than the minimum required by ACI-318-77, Section N. 5.5.3. However, since the shear stress was only 47% of the allowable shear stress, this was deemed acceptable.

The evaluation of the four columns 6E, 8E, 6M and 8M, determined that they had adequate capacity for the applied shear, torsion, compression and biaxial bending. All reinforcement exceeded the required minimums.

The evaluation of the three walls on lines F, H and K concluded that the walls were adequate for the applied shear, membrane tension in the vertical direction, and the out-of-plane bending for the wall about a horizontal axis. At the top of the walls a 7' wide horizontal strip is designed to resist out-of-plane bending about a vertical axis and was found to be adequate. For the portion of the wall below the top 7', the horizontal longitudinal ties are #4 rebars at 24" on centers on each face of the wall. The vertical reinforcing is #18 rebars at 12" on centers, each face. The horizontal longitudinal ties conform to minimum requirements of ACI 318-77, Section 7.10.5.

If the concrete is assumed not to provide any tension capacity (per ACI 318-77) then the horizontal rebars are shown to be overstressed from 2.9 to 4.5 times their allowable limit. However, the concrete is capable of resisting tensile stresses on the order of 0.4 x f'c or 263 psi in accordance with ACI 349-76, appendix B4.3. Taking this into account the concrete alone can resist 1.3 times the total horizontal tensile stress for wall K, 1.4 times for wall F and wall H. On this basis the walls of the turbine-generator pedestal are concluded to be adequate.

5.0 Summary of Modification

The modifications to the turbine building north and south extensions and east and west heater platforms are described in the previous sections of this report. This section presents a general description of these modifications and a detailed summary of the modifications in each area.

The turbine-generator pedestal and turbine gantry crane were found to meet the requirements of the BOPSSR program without modifications. The north and south turbine building extensions and east and west heater platform, were found to require modification to satisfy the BOPSSR Criteria. The modifications to these structural steel framing systems are intended to limit deflections and stresses. This is accomplished by the addition of diagonal bracing. The diagonal bracing changes the nature of the structures from moment resisting frames to braced moment resisting frames. In so doing stresses are redistributed and deflections are substantially reduced. The new bracing also causes an increase in uplift loads on the existing foundations. Consequently modifications to the turbine building include the addition of more massive foundations. Finally, the new bracing systems also results in a redistribution of loads which in turn required several miscellaneous modifications to increase capacities of existing members. This is accomplished by providing cover plates to some existing structural elements and by providing strengthening to some connections. In summary the modifications to the four areas of the turbine building may be categorized as follows: (1) Structural Steel Bracing, (2) Foundation Modification and (3) Miscellaneous. The following is a summary of modifications to each of the four areas of the turbine building.

5.1 North Extension Modifications

The modifications to the north turbine building extension are currently being installed. The installation is scheduled to be completed by June 1, 1982. Previous modifications to the north turbine building structural framing connections were described in a letter dated September 28, 1981 (supplemented by a letter dated October 6, 1981). The following is a summary of modifications to the north turbine building extension.

STRUCTURAL STEEL BRACING

- o A diagonal brace in the north-south direction from the base of column D-6 to the top of column B-6
- o A structural steel brace in the north-south direction from the base of column B-8 to the top of column A-8
- o An X-brace in the east-west direction between columns B6 and B8 consisting of, diagonals connected at the mezzanine elevation of column b-7.

FOUNDATIONS

- o The addition of a U-shaped foundation which incorporates existing footings and is structurally tied to the foundation which supports columns A-6, A-7, and A-8. The new footing runs from column A-6 to B-6 from B-6 to B-8 and from B-8 to A-8

MISCELLANEOUS

- o A stiffener plate in the connection of column A-8 to girder
- o Additional anchor bolts at the base of column D-6

5.2 West Heater Platform Modifications

The modifications to the west heater platform are currently being installed. The installation is scheduled to be completed by June 1, 1982. The following is a summary of modifications to the west heater platform.

STRUCTURAL STEEL BRACING

- o A diagonal brace in the north-south direction from the top of column C-9 to a concrete foundation located north of column C-9.
- o A diagonal brace in the east-west direction from the base of column C-11 to the top of column C-13.
- o An X-brace in the north-south direction between columns E-13 and F-13.
- o A diagonal brace in the east-west direction from the base of column G-12 to the top of column G-9.
- o A diagonal brace in the east-west direction from the base of column H-12 to the top of column H-9.
- o A diagonal brace in the north-south direction from the base column G-13 to the top of column H-13.
- o A diagonal brace in the north-south direction from the base of column J-13 to the top of column H-13.
- o A diagonal brace in the east-west direction from the top of column G-13 to a concrete pier located west of column G-13.
- o A diagonal brace in the east-west direction from the top of column H-13 to a concrete pier located west of column H-13.
- o A diagonal brace in the east-west direction from the top of column K-13 to a concrete pier located west of column K-13.

- A diagonal brace in the east-west direction from the base of column K-12 to the top of column K-13.

- A diagonal brace in the north-south direction from the base of column K-12 to the top of colum J-12.

- A diagonal brace in the north-south direction from the top of column L-9 to a concrete pier located south of column L-9.

FOUNDATIONS

- The addition of a foundation at column C-9, structurally connecting the existing footing to the new addition.

- The addition of a foundation between columns C-11 and E-11 which incorporates existing footings and is structurally tied to the foundations that support columns C-11 and E-11.

- The addition of a continuous footing which incorporates existing footings and is structurally tied to the foundations which support columns C-13, E-13, F-13, and H-13. The new continuous footing goes from column C-13 to H-13.

- The addition of a combined foundation located west of column line 13 and spanning between columns G-13 and H-13.

- The addition of a T-shaped foundation which incorporates existing footings and is structurally tied to the foundations which support columns J-13, K-13, and K-12. The new footing goes from column K-12 to K-13, from K13 to J-13, and extends west from column K-13.

- The addition of a foundation at column L-9 directly above the existing foundation at column L-9.

- The addition of a foundation located south of column L-9.

MISCELLANEOUS

- Cover plates added to the beam from J12-K12 (Ref. beam No. B20) and the beam from C11-C13 (Ref. beam No. B21).

- Cover plates added at column C-13.

- Additional anchor bolts to the base plates for columns C11 and C13.

- Stiffener plates at the anchorage connection of columns C9, C11, and C13.

- Strengthening the connection between the deck slab and the steel framing by increasing weld sizes and adding through bolts.

5.3 East Heater Platform Modifications

The following is a summary of modifications to the east heater platform structure.

STRUCTURAL STEEL BRACING

- A diagonal brace in the east-west direction from the base of column E-3 to the top of column E-1.
- A diagonal brace in the north-south direction from the base of column E-1 to the top of column F-1.
- An X-brace in the east-west direction between columns F-1 and F-2.
- A diagonal brace in the north-south direction from the base of column F-1 to the top of column G-1.
- An X-brace in the north-south direction between columns F-2 and G-2.
- A diagonal brace in the north-south direction from the base of column G-2 to the top of column H-2.
- A diagonal brace in the east-west direction from the base of colum H-1 to the top of column H-2.
- A K-brace in the east-west direction between column J-1 and J-2.
- A diagonal brace in the north -south direction from the base of colum K-1 to the top of column J-1.
- A diagonal brace in the north-south direction from the base of column K-2 to the top of column J-2.
- A diagonal brace in the north-south direction from the base of column J-1 to the top of column H-1.
- A diagonal brace in the north-south direction from the top of column L-5 to a concrete pier located south of column L-5.

FOUNDATIONS

- The addition of a foundation at column E-3, structurally connecting the existing footing to the new foundation.
- The addition of a T-shaped foundation which incorporates existing footings and is structurally tied to the foundations which support columns E-1, F-1, G-1, and F-2. The new footing goes from column E-1 to F-1, from F-1 to G-1, and from F-1 to F-2.

- The addition of a T-shaped foundation which incorporates existing footings and is structurally tied to the foundations which support columns H-1, J-1, K-1, and J-2. The new footing goes from column H-1 to J-1, from J-1 to K-1, and from J-1 to J-2.

- The addition of a spread foundation located south of column L-5.

MISCELLANEOUS

- Cover plates added to beam from E1-F1 (Ref. Beam No. B1) and beam from G2-H2 (Ref. Beam No. BH13).

- Cover plates added to column E-1.

- Additional anchor bolts to the base plates for columns E-1, E-3, J-2, K-1, and L-5.

- Stiffener plates at the anchorage connection of columns E-1, E-3, J-2, K-1, and L-5.

- Strengthen the connection between the heater deck slab and the steel framing by increasing weld sizes and adding through bolts.

- Strengthen the steel beam supporting the top of the masonry wall between columns L-5 and M-5 by providing intermediate supports.

5.4 South Extension Modifications

The following is a summary of modifications to the south extension.

STRUCTURAL STEEL BRACING

- A diagonal brace in the north-south direction from the base of column M-6 to the top of column N-6.

- A diagonal brace in the north-south direction from the base of column P-6 to the top of column N-6.

- A K-brace in the east-west direction between columns M-6 and M-8.

- A diagonal brace in the north-south direction from the base of column M-8 to the top of column N-8.

- A diagonal brace in the north-south direction from the base of column N-8 to the top of column P-8.

- A diagonal brace in the east-west direction from the base of column P-6 to the top of column P-7.

- A diagonal brace in the east-west direction from the top of column P-6 to a concrete pier located east of column P-6.

FOUNDATIONS

- The addition of a foundation between columns M-6 and N-6 which incorporates existing footings and is structurally tied to the foundations that support columns M-6 and N-6.
- The addition of a foundation between columns M-8 and N-8 which incorporates existing footings and is structurally tied to the foundations that support columns M-8 and N-8.
- The addition of a continuous foundation which incorporates existing footings and is structurally tied to the foundations which support columns P-6, P-7, and P-8. The new foundation goes from columns P-6 and P-8.
- The addition of a spread foundation located east of column P-6.

MISCELLANEOUS

- Additional anchor bolts to the base plates for columns M-6, M-8, P-6, R-6, and R-8.
- Stiffener plates at the anchorage connection of column P-6.
- Strengthen the connection between the deck slab and steel framing by increasing weld sizes and adding through bolts.

6.0 CONCLUSION

This report provides results of the reevaluation of the San Onofre Unit 1 turbine building and turbine-generator pedestal in accordance with the methodology discussed in Section 3.0. A detailed description of the results of the reevaluation is given in Section 4.0. The turbine-generator pedestal and the turbine gantry crane satisfy the BOPSSR criteria and therefore do not require any modifications. Modifications are required for the north and south extensions and the east and west heater platforms. The modifications are required in order to limit deflections and restore design margins to satisfy the BOPSSR criteria. The modifications for each of the turbine building areas are listed in Section 5.0. Upon the completion of installing the modifications the turbine building and turbine-generator pedestal will satisfy the BOPSSR criteria.

7.0 REFERENCES

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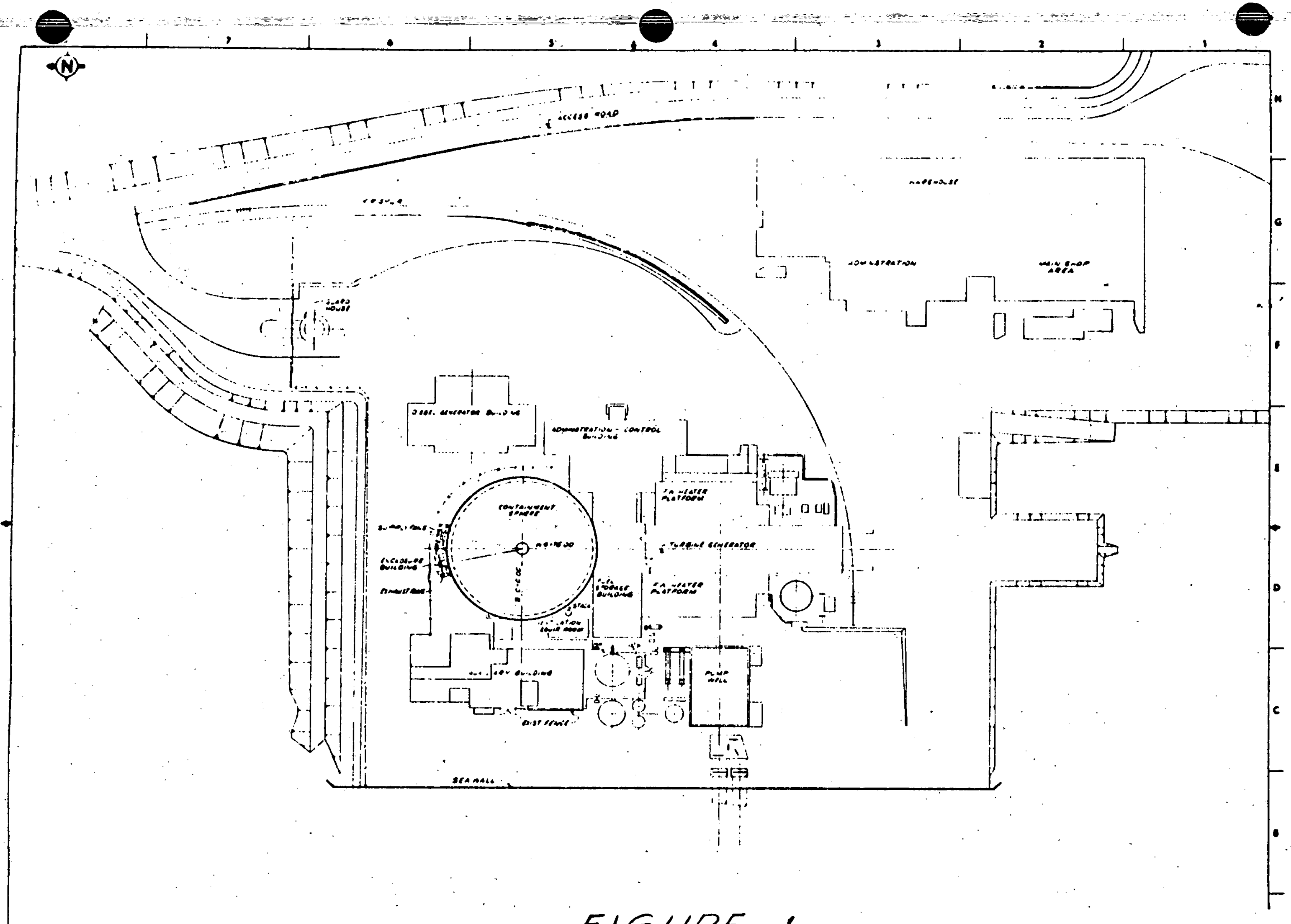

FIGURE 1

GENERAL PLOT PLAN

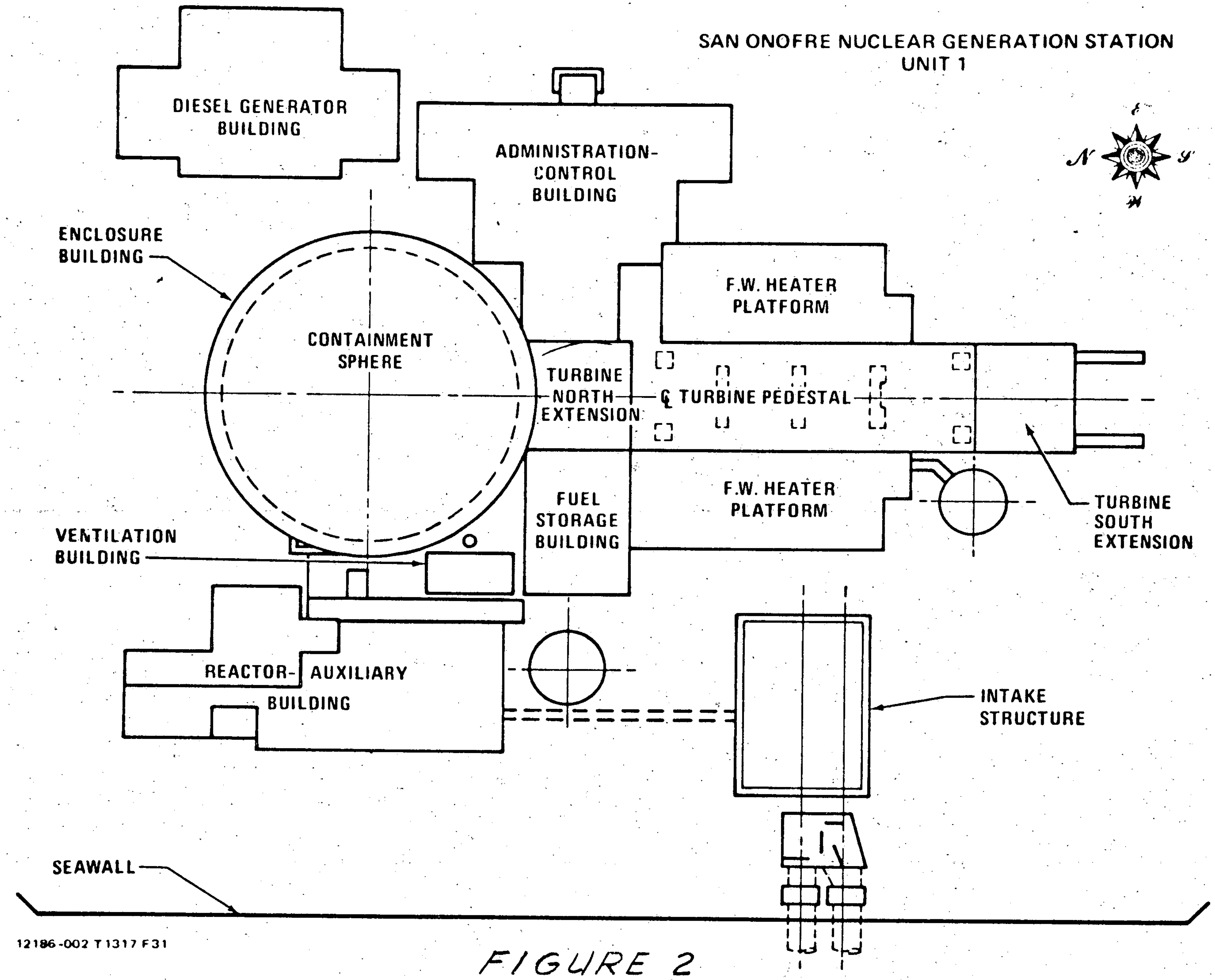

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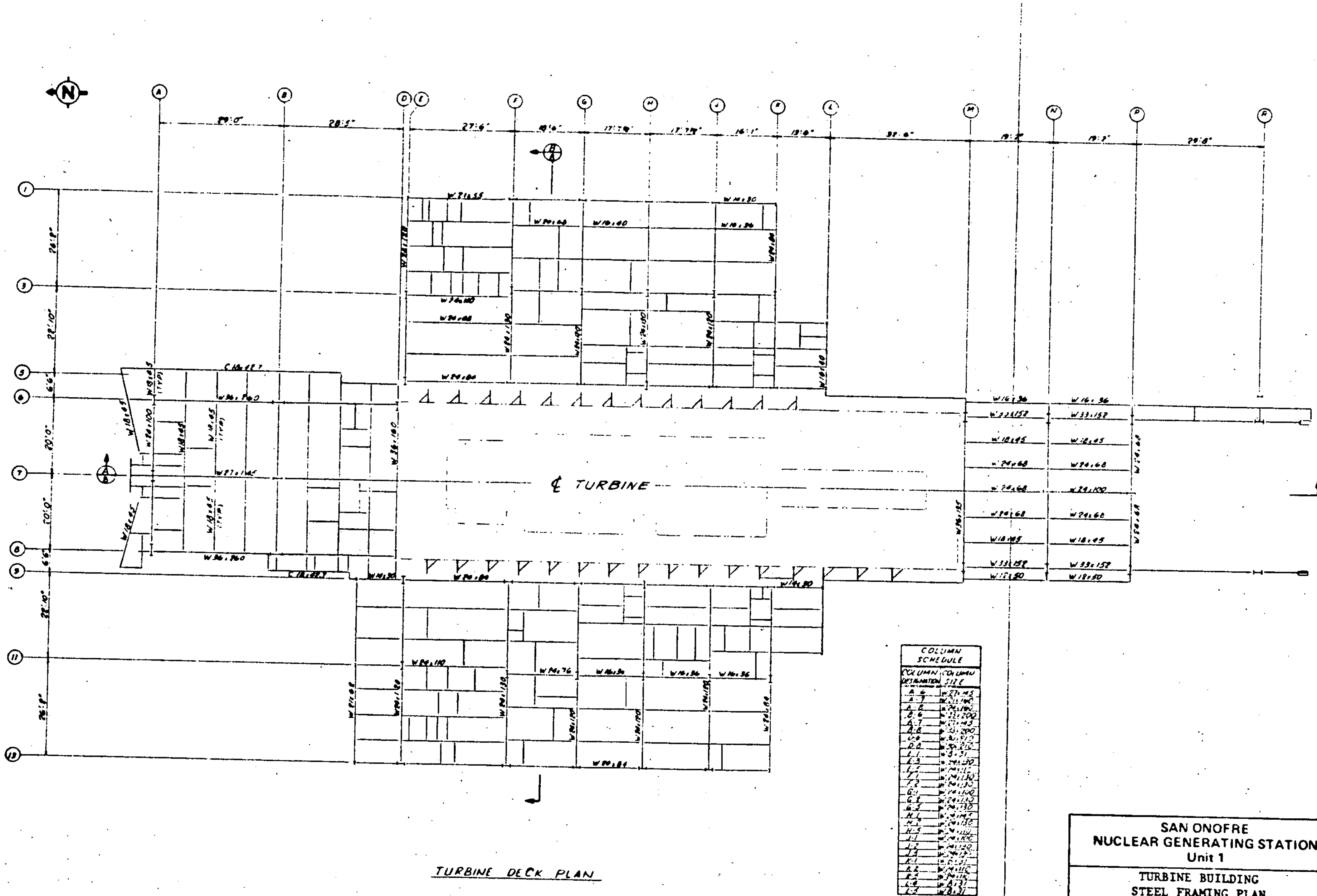
FIGURE 2



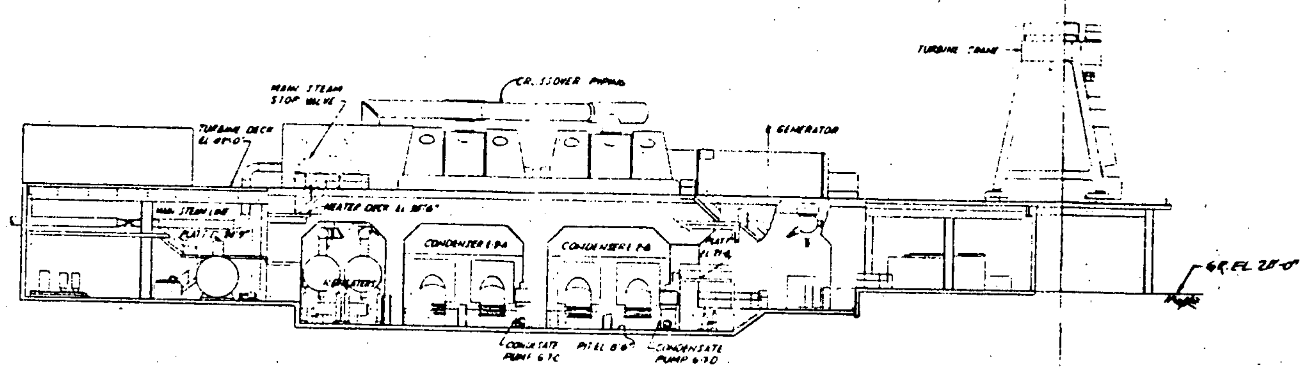
TURBINE DECK PLAN

FIGURE 3

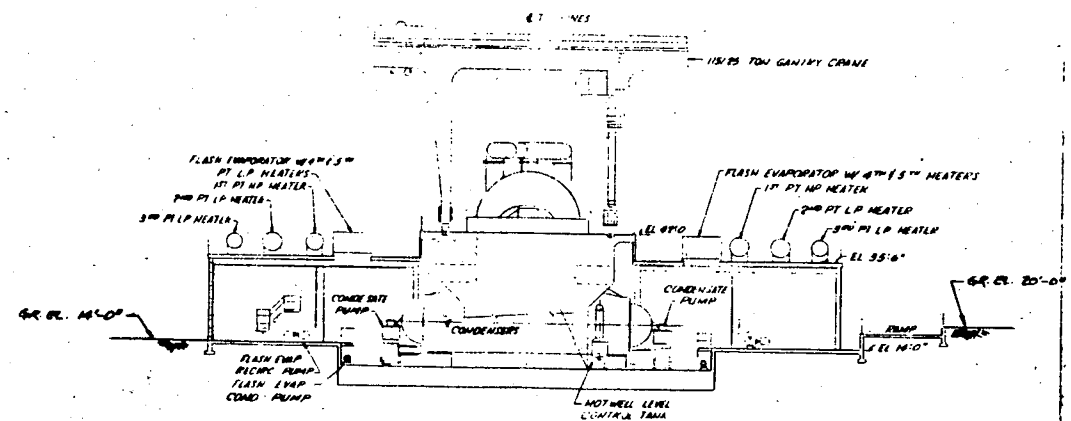
SAN ONOFRE
NUCLEAR GENERATING STATION
Unit 1

TURBINE BUILDING
STEEL FRAMING PLAN

Figure 3



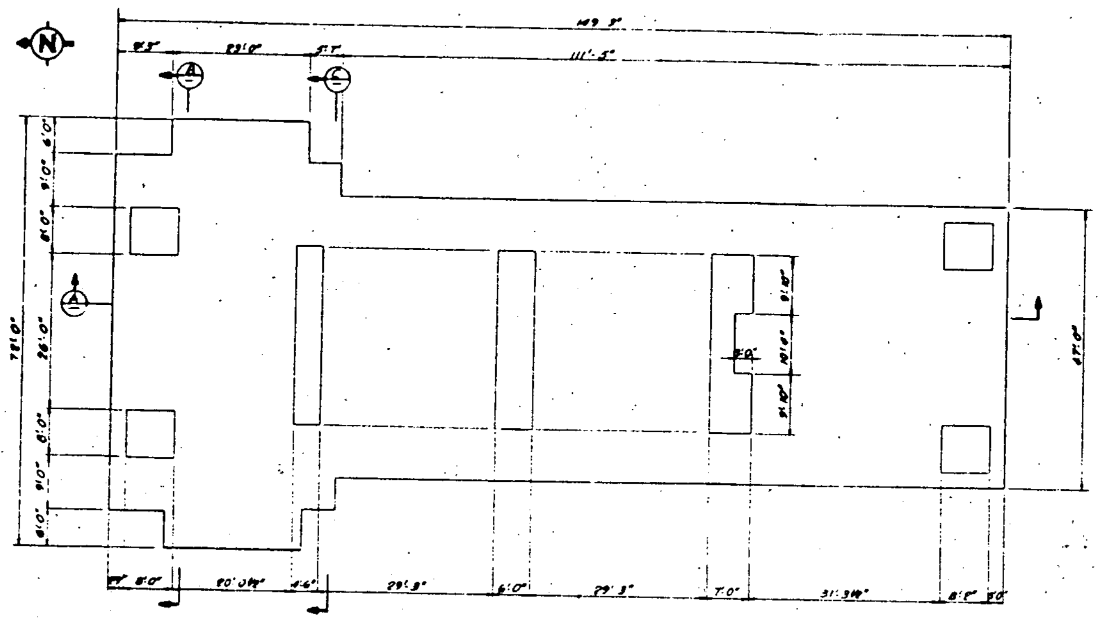
SECTION A



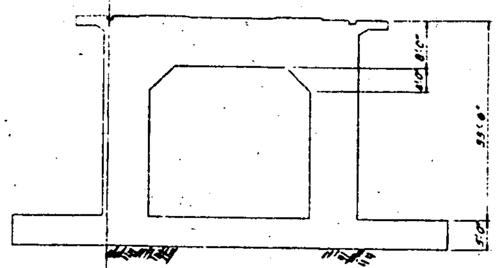
SECTION B

FIGURE 4

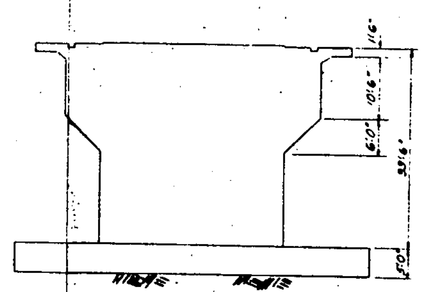
SAN ONOFRE NUCLEAR GENERATING STATION Unit 1
TURBINE BUILDING SECTIONS
Figure 4



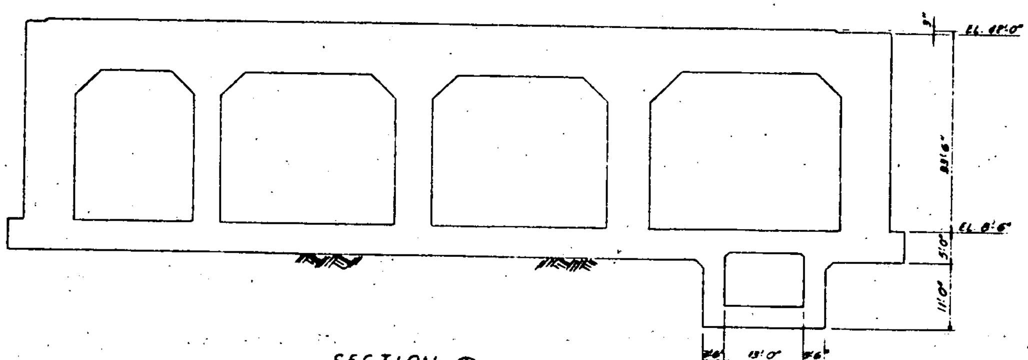
TURBINE-GENERATOR PEDESTAL FOUNDATION MAT-PLAN



SECTION B



SECTION C



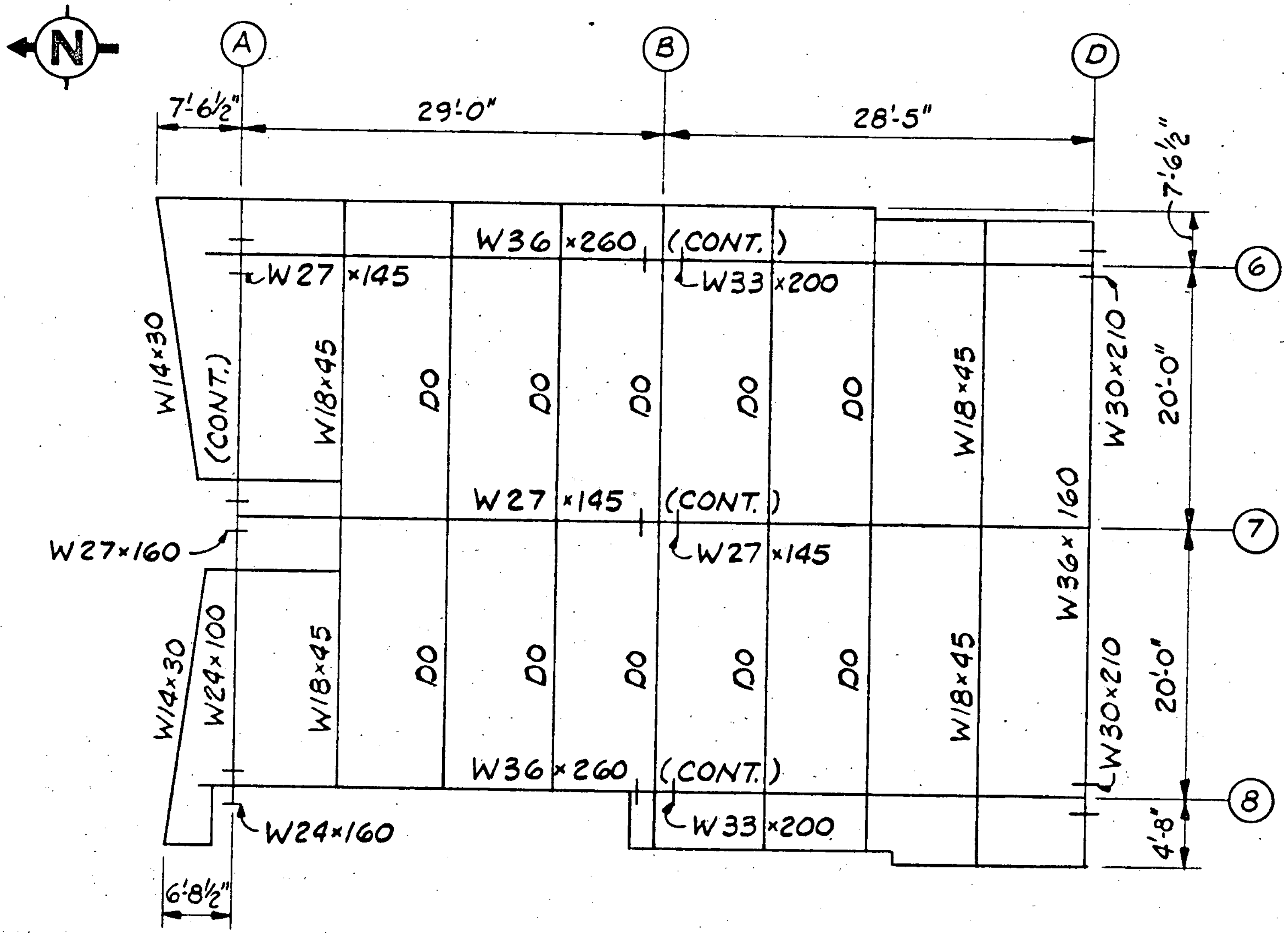
SECTION A

FIGURE 5

SAN ONOFRE
NUCLEAR GENERATING STATION
Unit 1

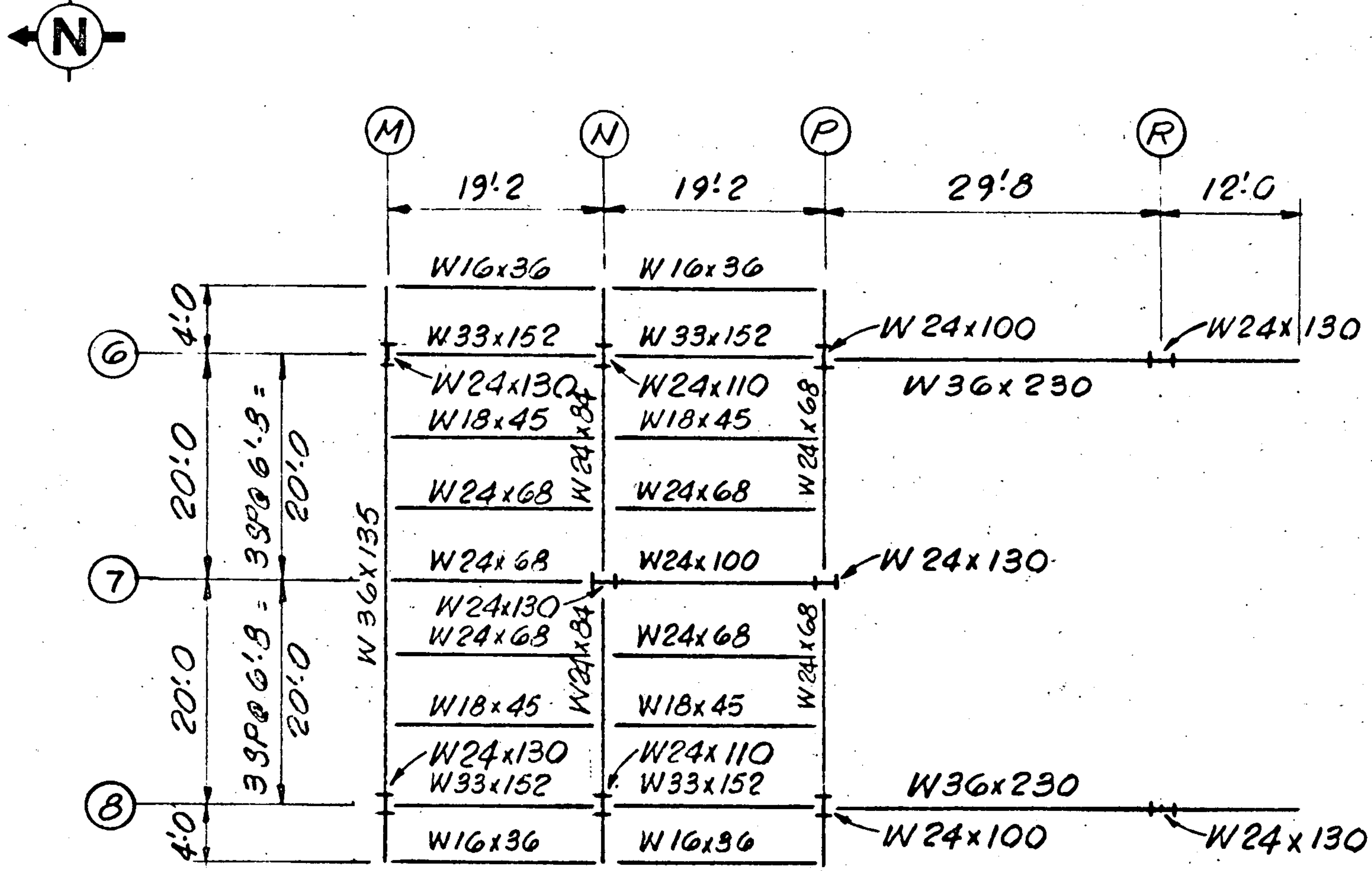
TURBINE PEDESTAL
PLANS & SECTIONS

Figure 5



NORTH EXTENSION
STRUCT. STEEL FRAMING
PLAN @ EL. 42'-0"

FIGURE 6

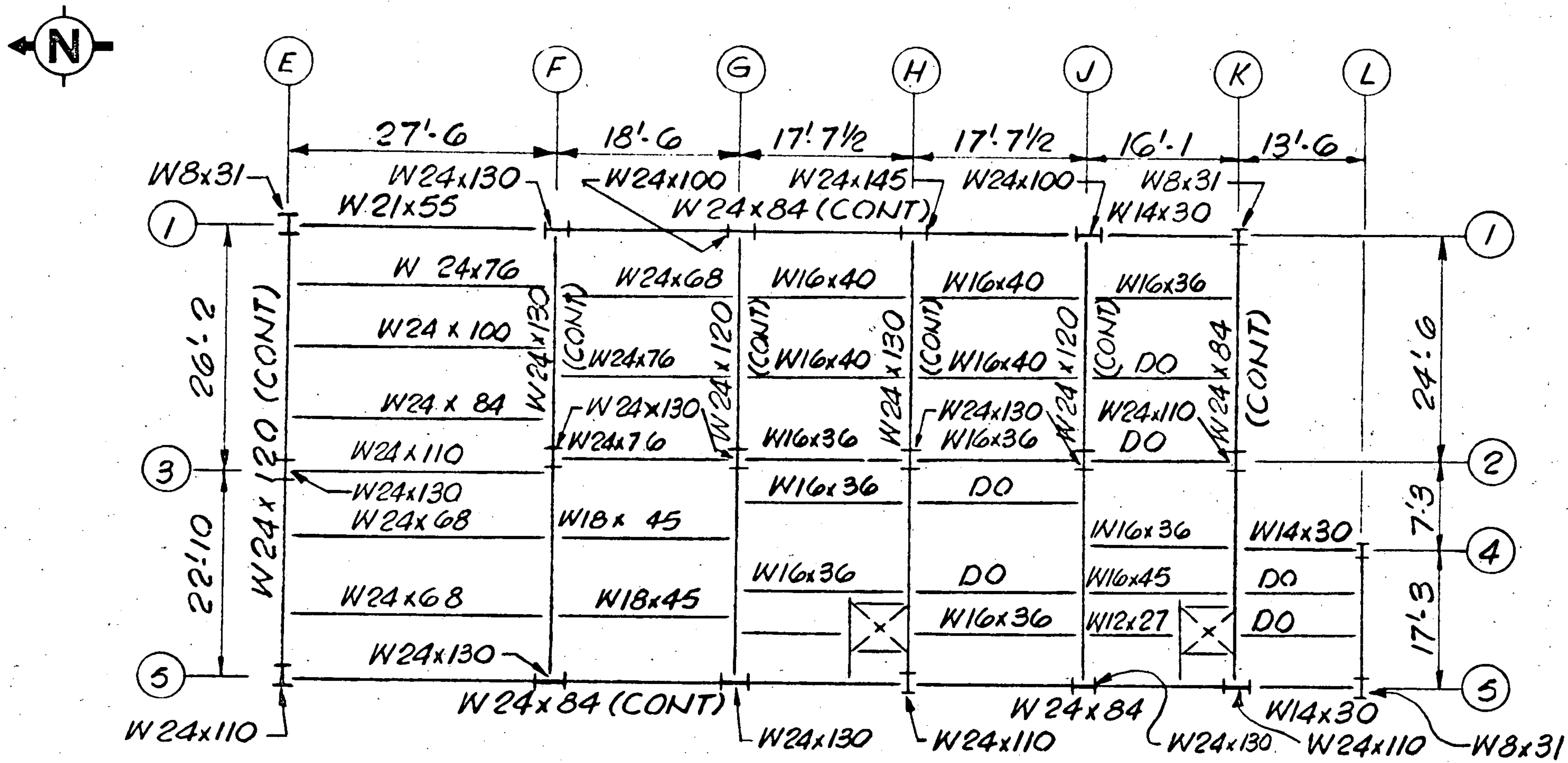


SOUTH EXTENSION

STRUCT STEEL FRAMING

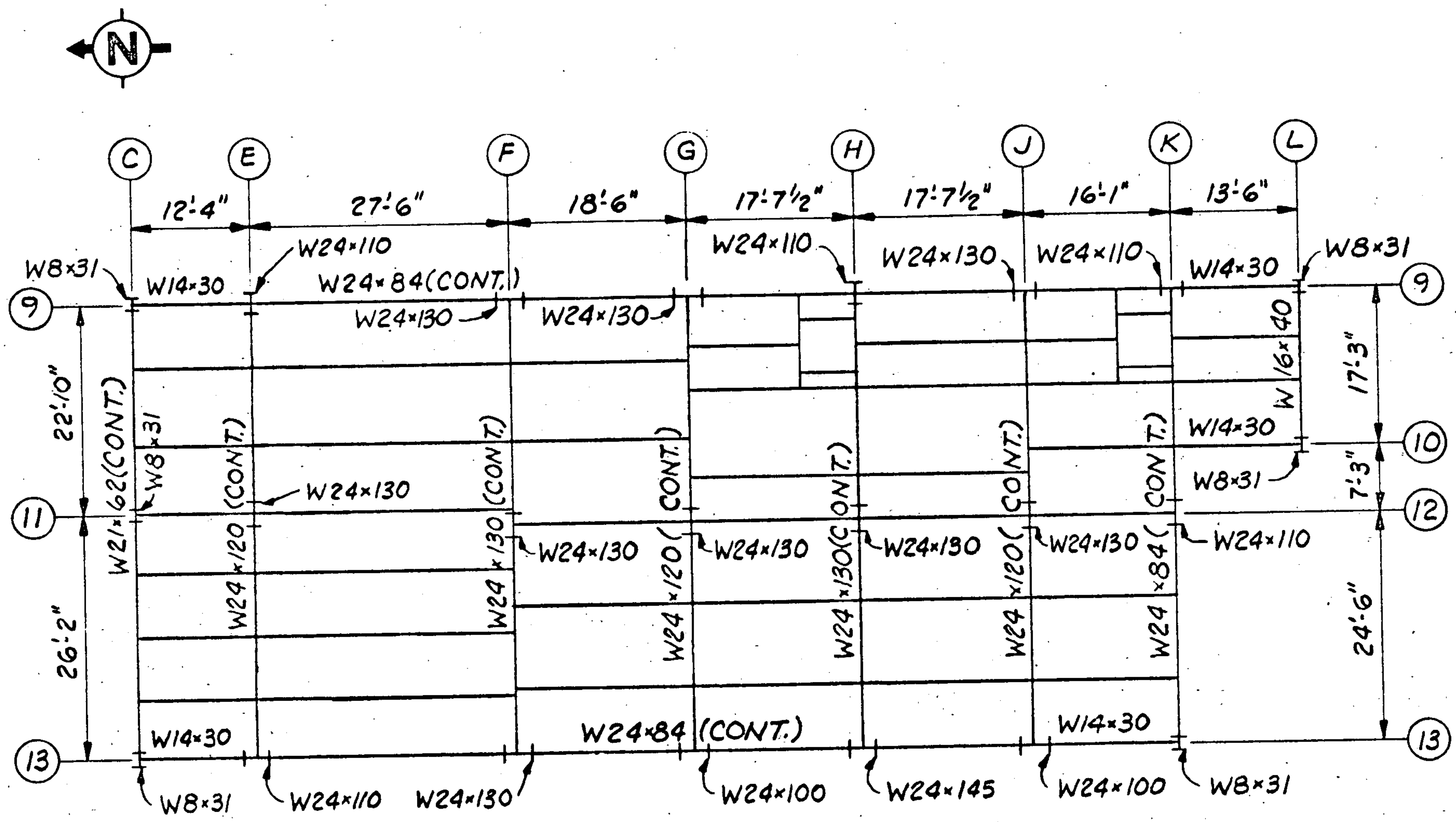
PLAN @ EL 42'-0

FIGURE 7



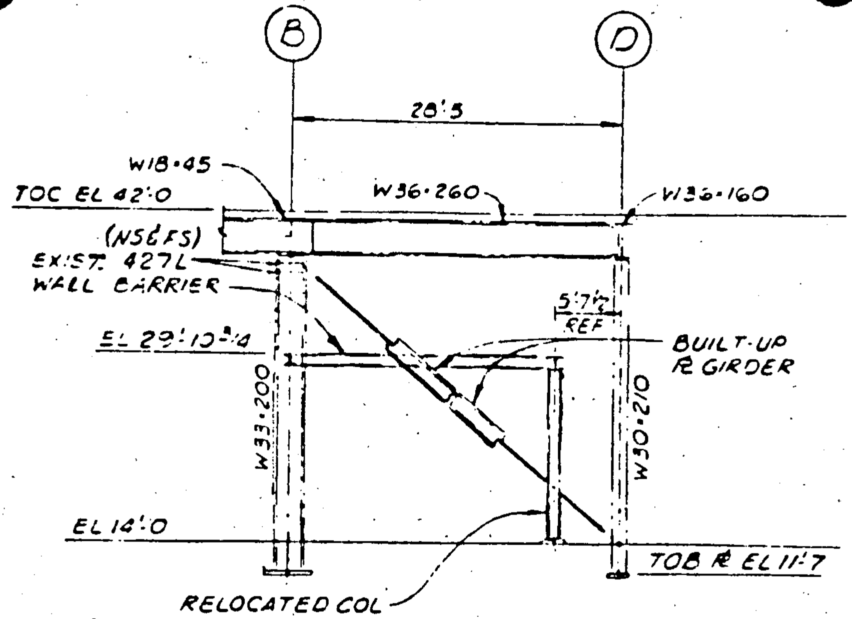
EAST HEATER PLATFORM
STRUCT STEEL FRAMING
PLAN @ EL 35'-6

FIGURE 8

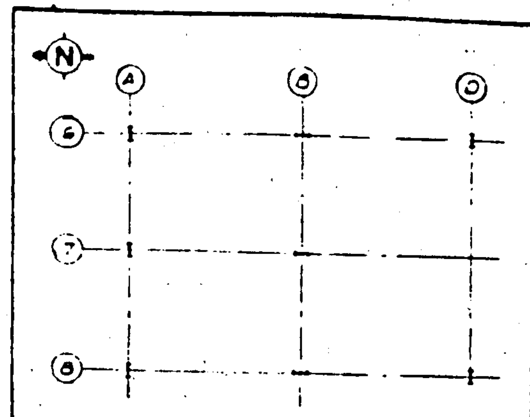


WEST HEATER PLATFORM
STRUCT. STEEL FRAMING
PLAN @ EL. 35'-6"

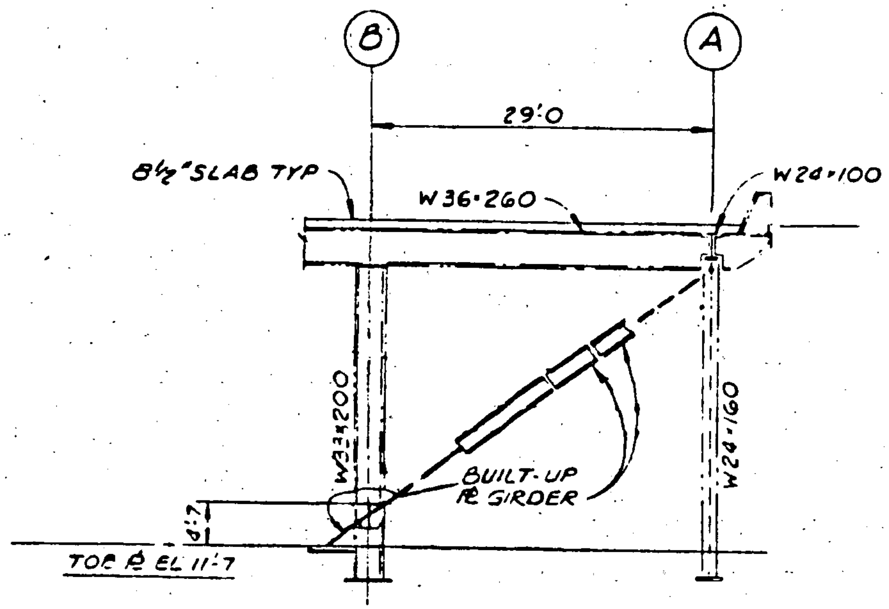
FIGURE 9



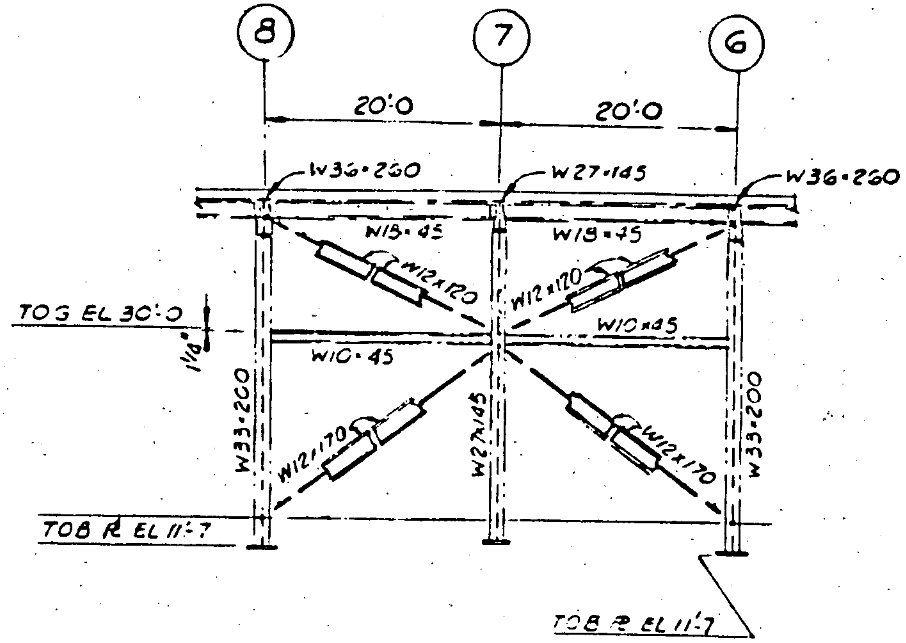
ELEVATION LOOKING EAST
ON COL LINE-6



KEY PLAN - AREA 2
NORTH EXTENSION FRAMING

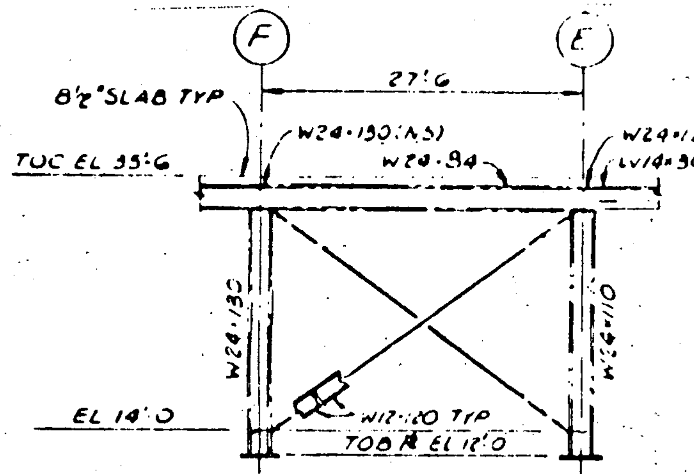


ELEVATION LOOKING WEST
ON COL LINE-8

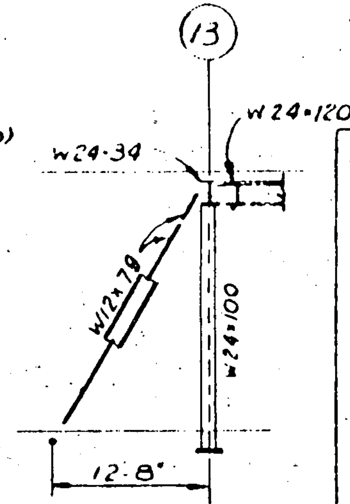


ELEVATION LOOKING NORTH
ON COL LINE-B

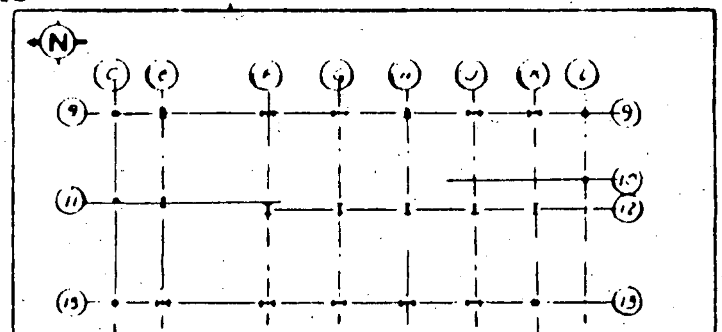
FIGURE 10



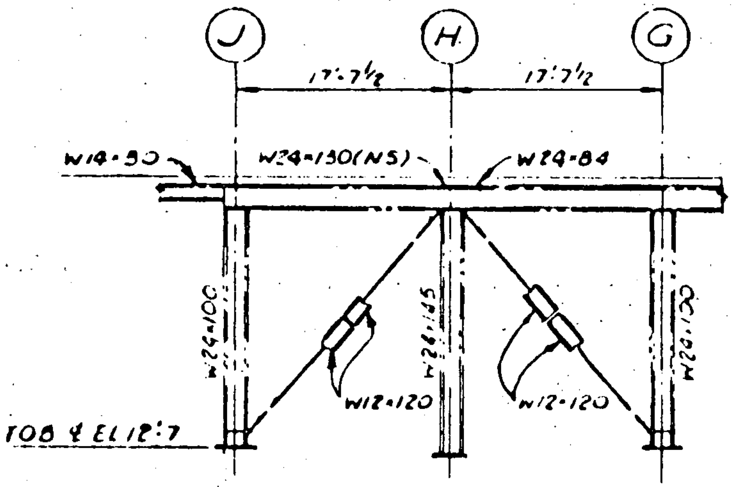
ELEVATION LOOKING WEST
ON COL LINE-13



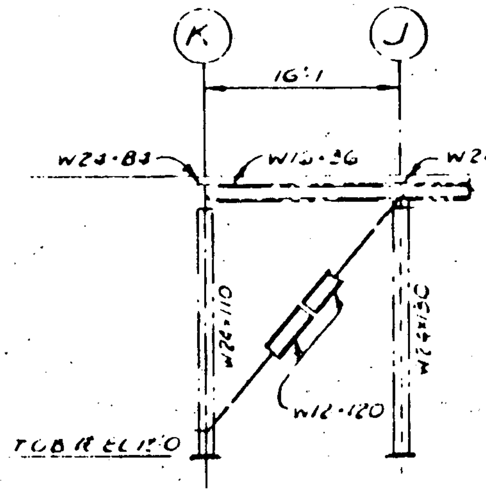
ELEVATION LOOKING NORTH
ON COL LINE-G



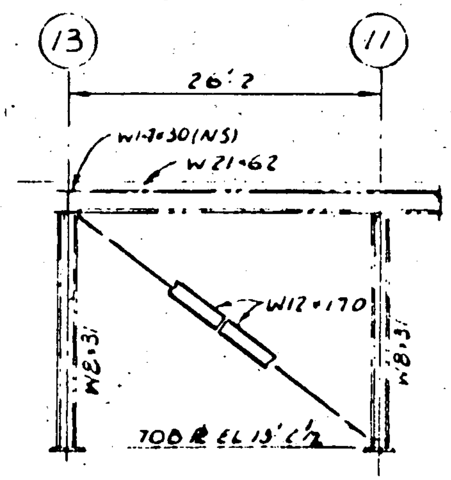
KEY PLAN - AREA 6
WEST HEATER PLATFORM



ELEVATION LOOKING WEST
ON COL LINE-13

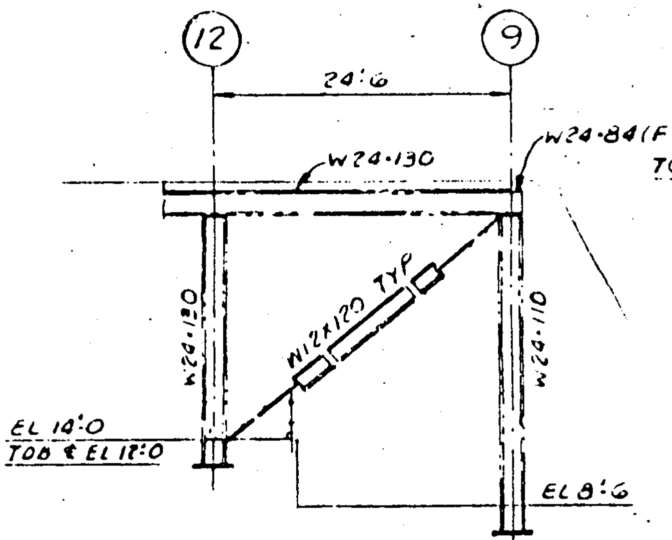


ELEVATION LOOKING WEST
ON COL LINE-12

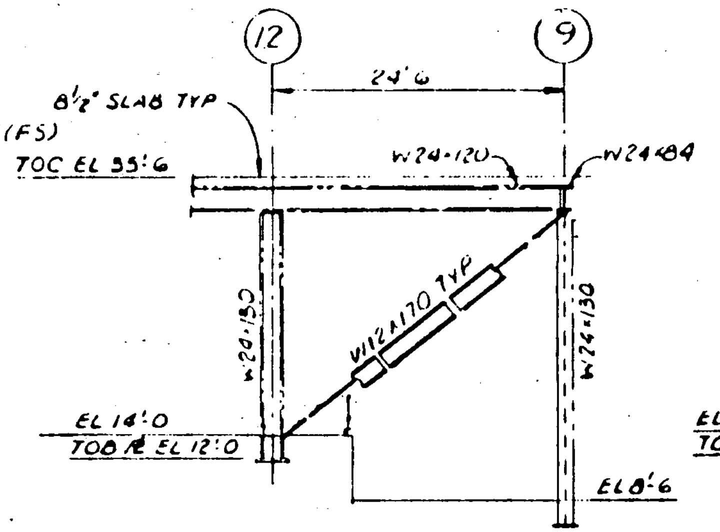


ELEVATION LOOKING NORTH
ON COL LINE-C

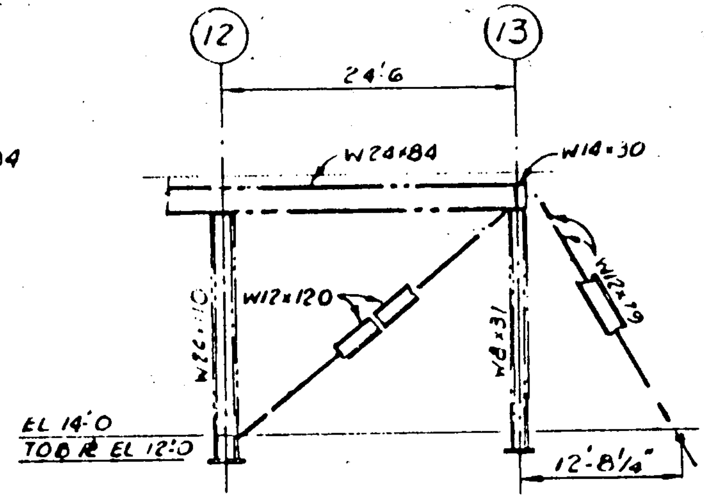
FIGURE 11



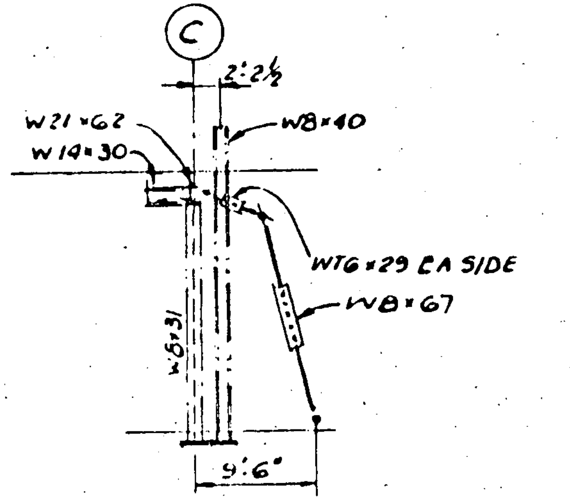
ELEVATION LOOKING NORTH
ON COL LINE-H



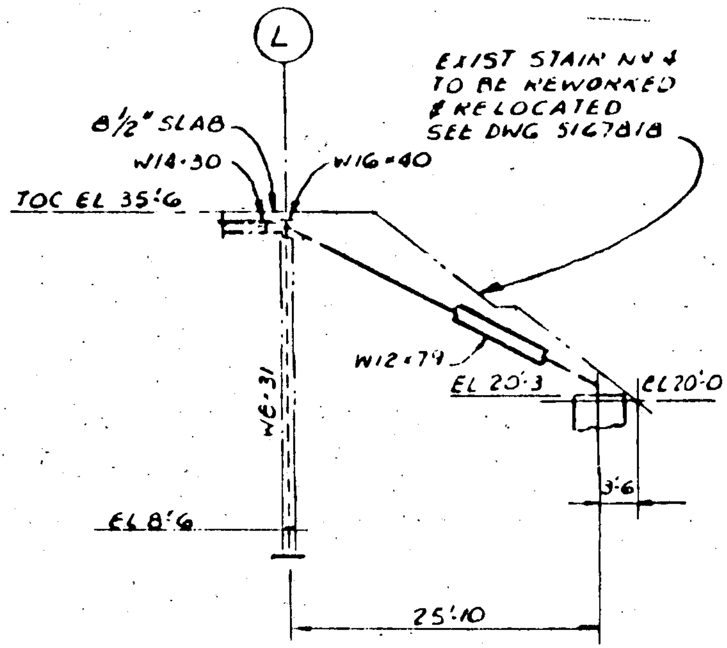
ELEVATION LOOKING NORTH
ON COL LINE-G



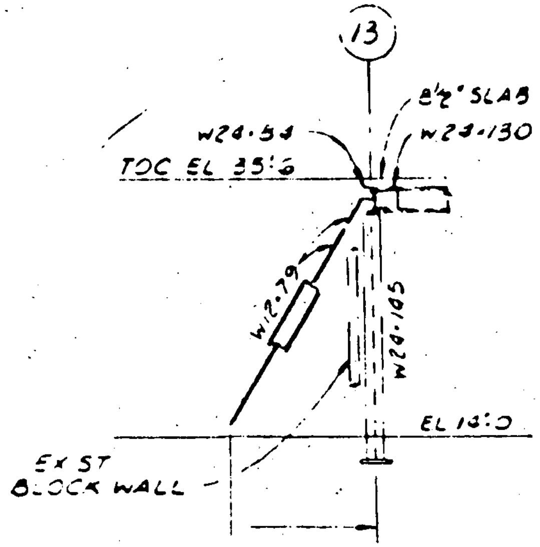
ELEVATION LOOKING SOUTH
ON COL LINE-K



ELEVATION LOOKING WEST
ON COL LINE-9

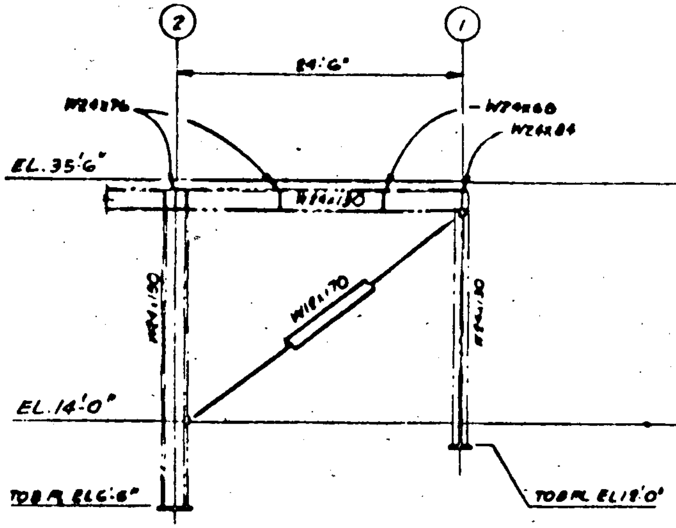


ELEVATION LOOKING EAST
ON COL LINE-9

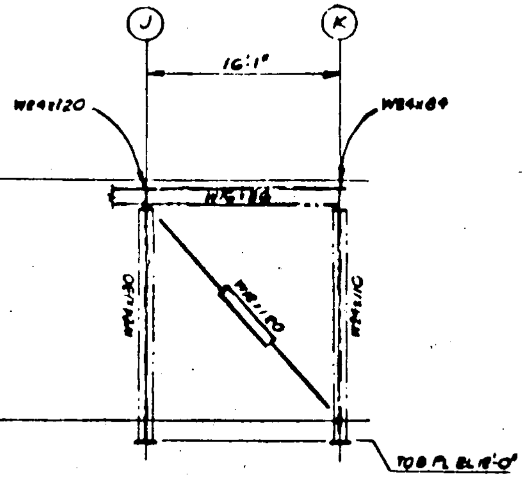


ELEVATION LOOKING NORTH
ON COL LINE-H

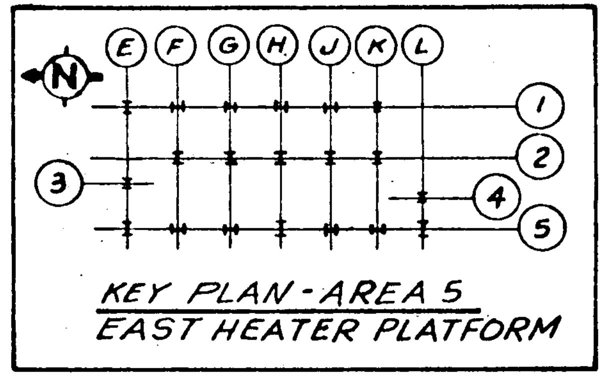
FIGURE 12

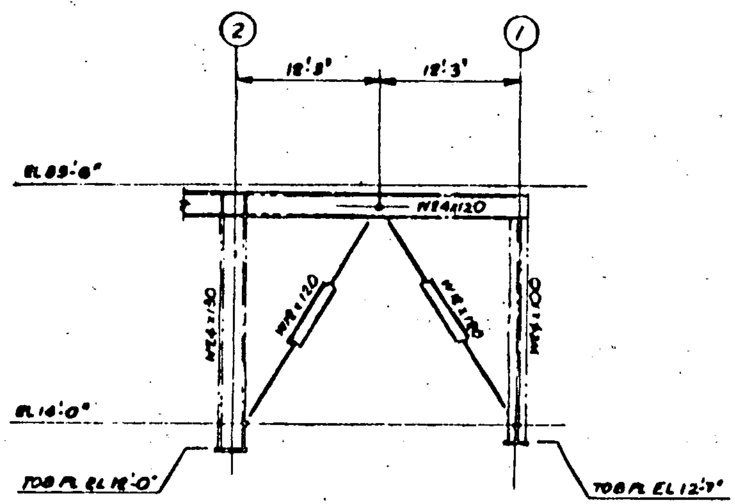


ELEVATION LOOKING NORTH
ON COL 1 AND 2 @ LINE F

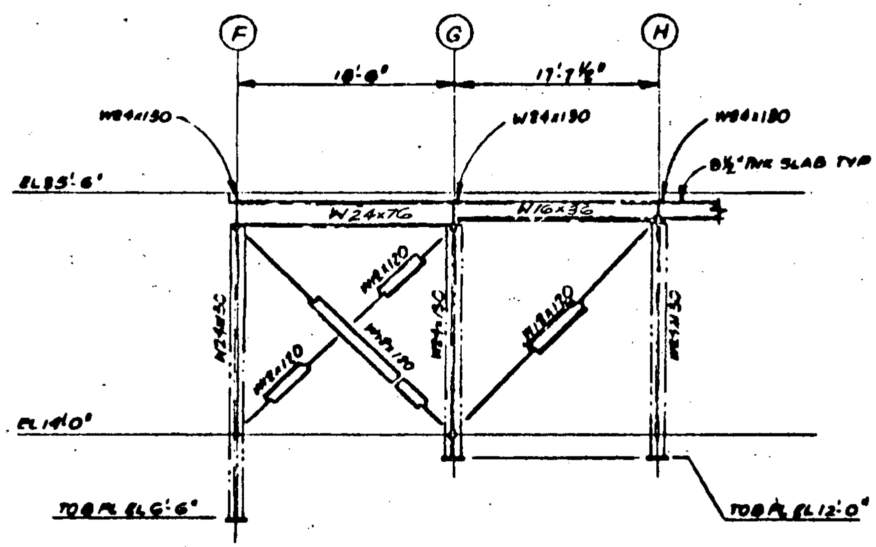


ELEVATION LOOKING EAST
ON COL J AND K @ LINE 2



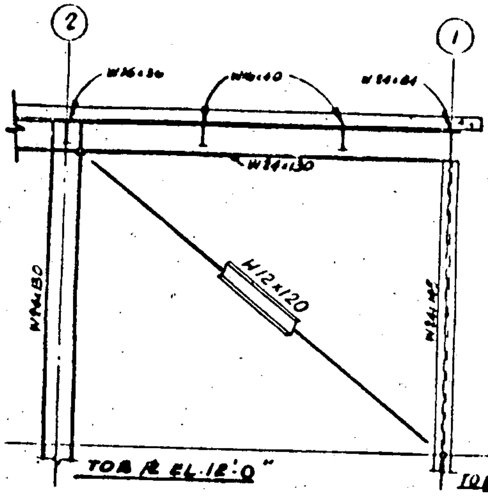


ELEVATION LOOKING NORTH
ON COL 1 AND 2 @ LINE J

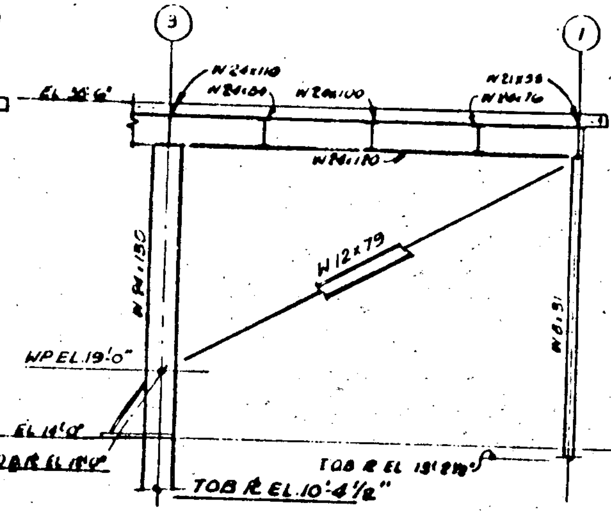


ELEVATION LOOKING EAST
ON COL F THRU H @ LINE 2

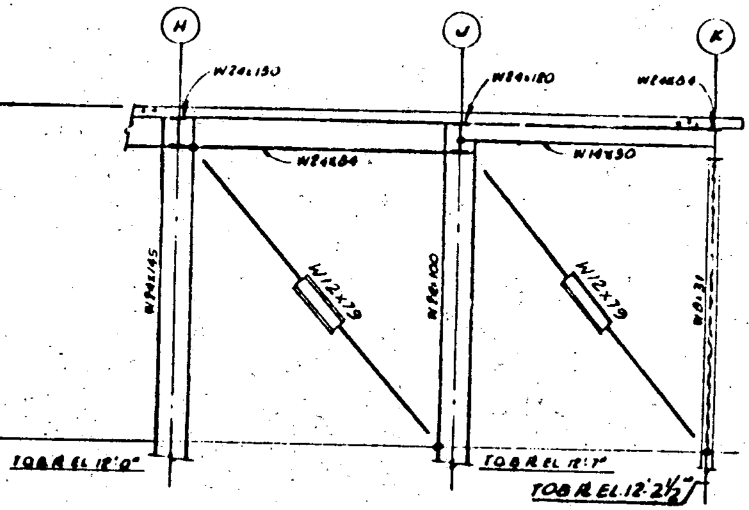
FIGURE 13



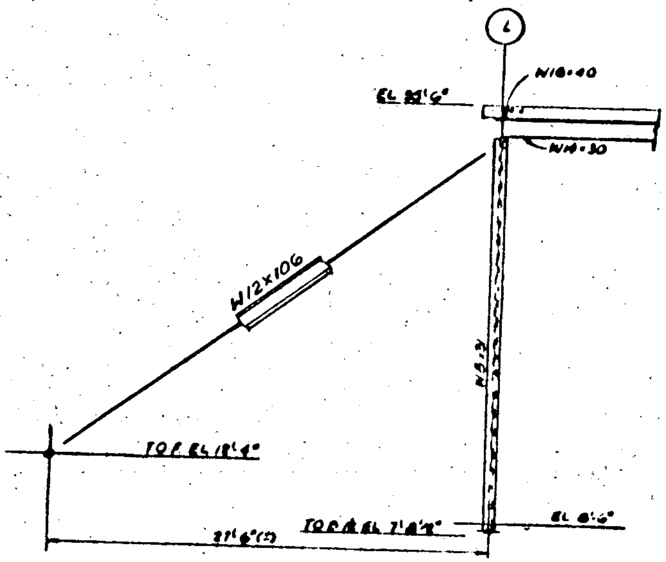
LOOKING NORTH
AREA 5 - EAST HEATER PLATFORM
@ COL LINE "H"



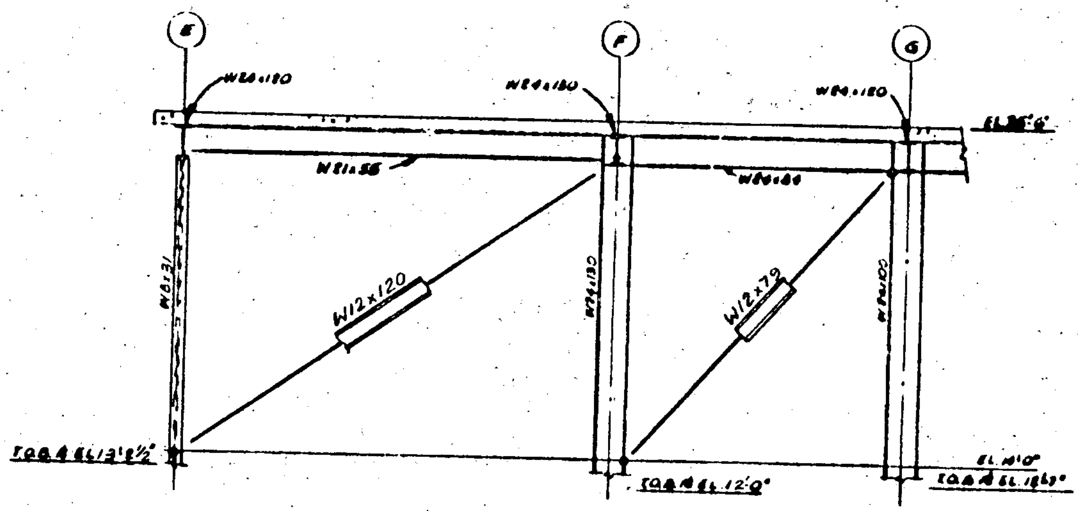
LOOKING NORTH
AREA 5 - EAST HEATER PLATFORM
@ COL LINE "E"



LOOKING EAST
AREA 5 - EAST HEATER PLATFORM
@ COL LINE "4"

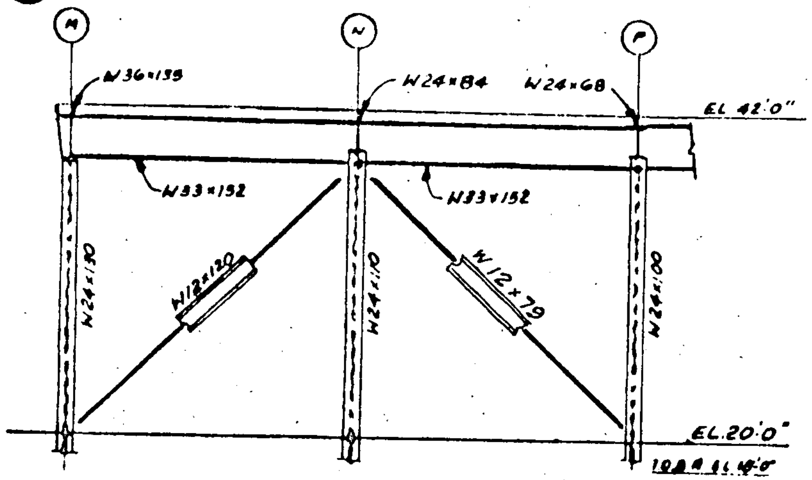


LOOKING WEST
AREA 5 - EAST HEATER PLATFORM
@ COL LINE "5"

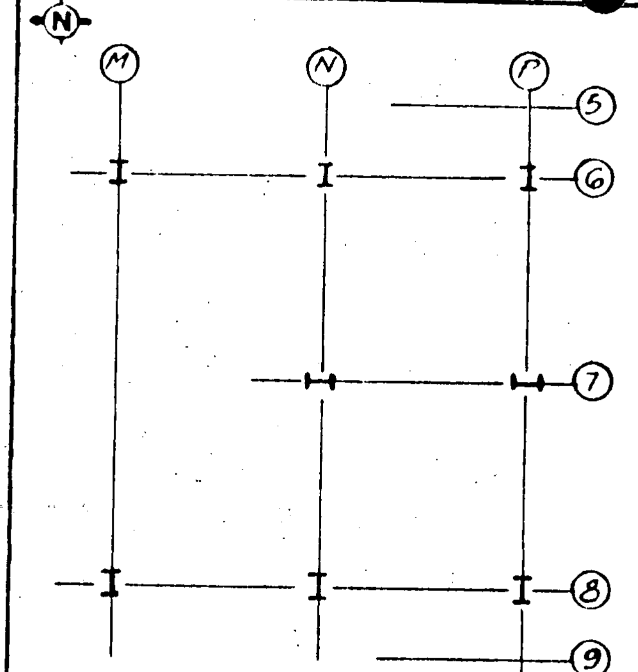


LOOKING EAST
AREA 5 - EAST HEATER PLATFORM
@ COL LINE "1"

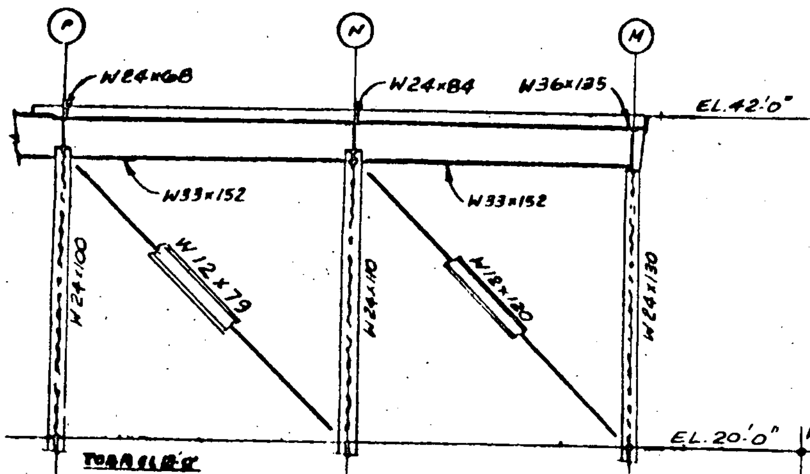
FIGURE 14



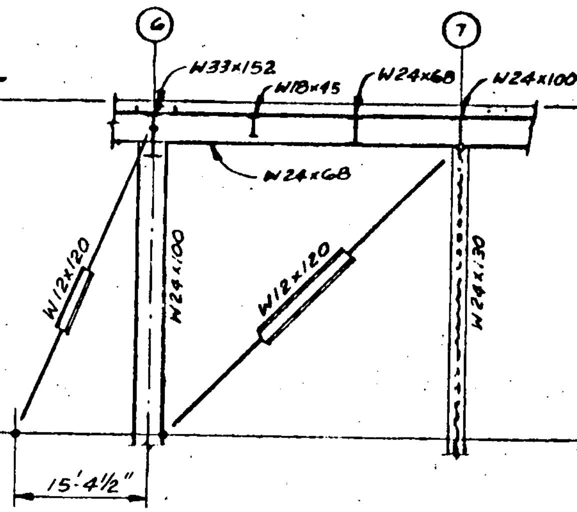
LOOKING EAST
AREA 7 · SOUTH EXTENSION
@ COL LINE "G"



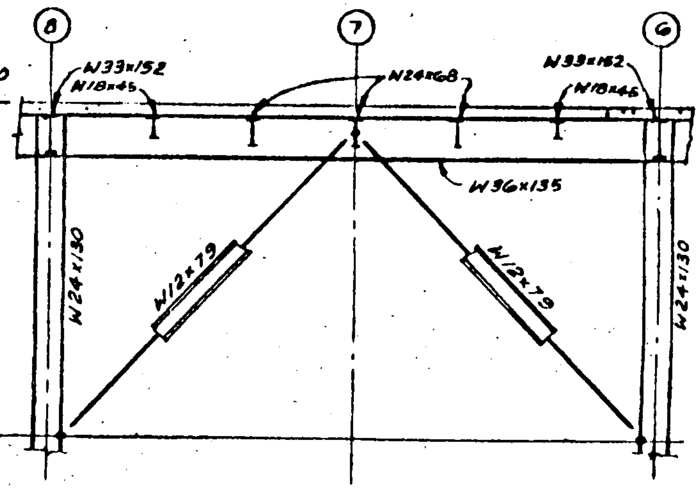
KEY PLAN - AREA 7
SOUTH HEATER PLATFORM



LOOKING WEST
AREA 7 SOUTH EXTENSION
@ COL LINE "8"

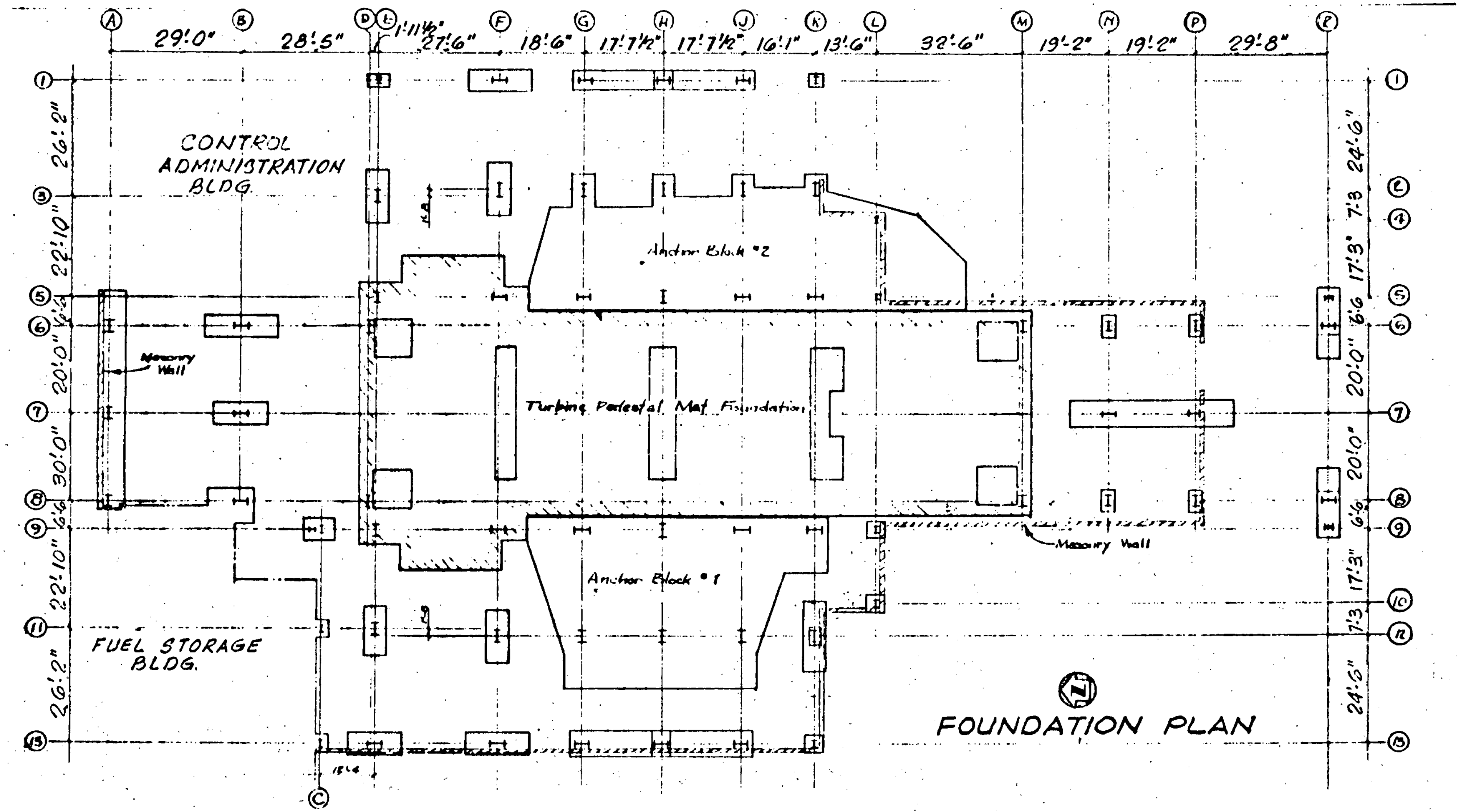


LOOKING SOUTH
AREA 7 SOUTH EXTENSION
@ COL LINE "P"



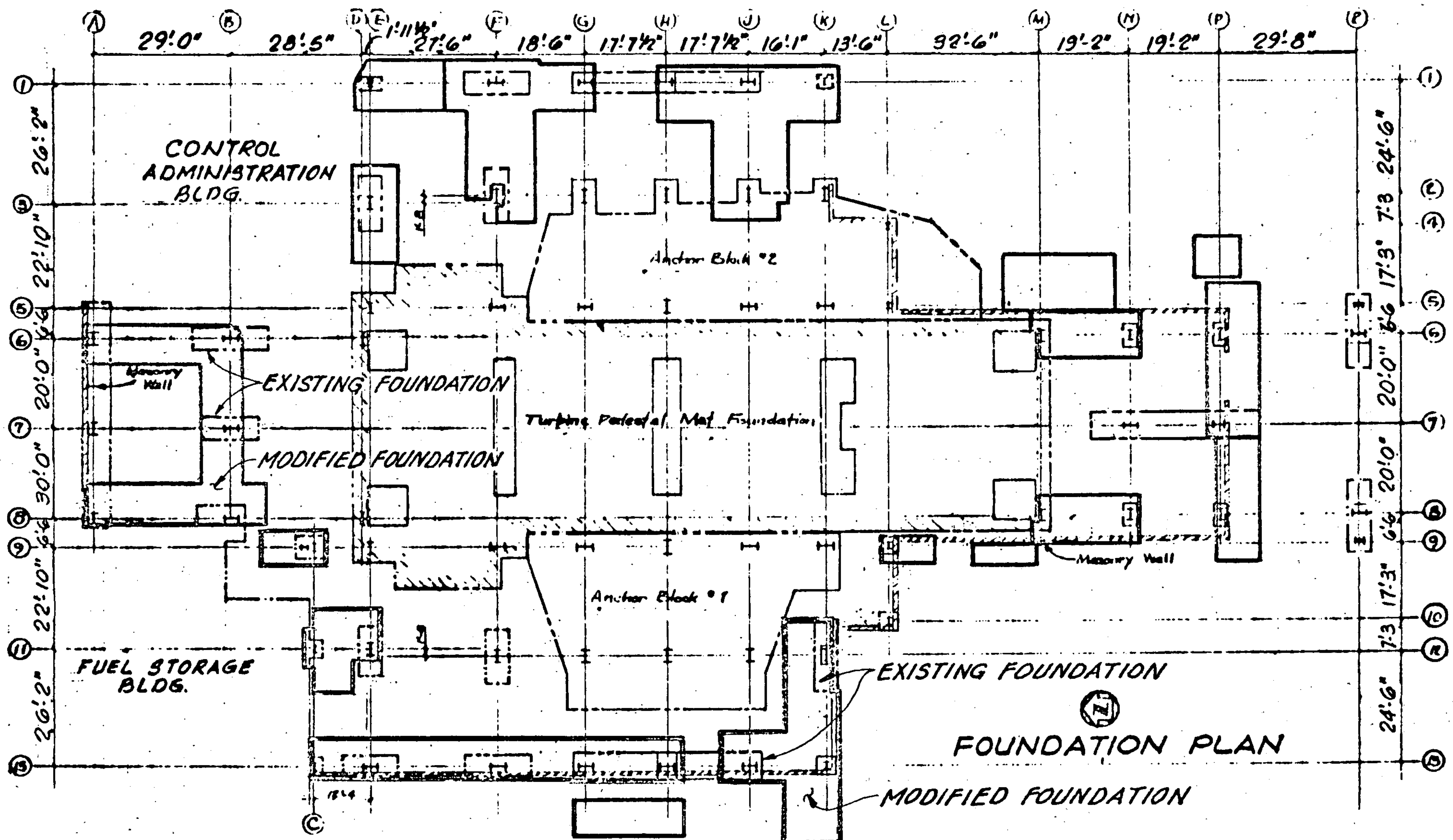
LOOKING NORTH
AREA 7 SOUTH EXTENSION
@ COL LINE "M"

FIGURE 15



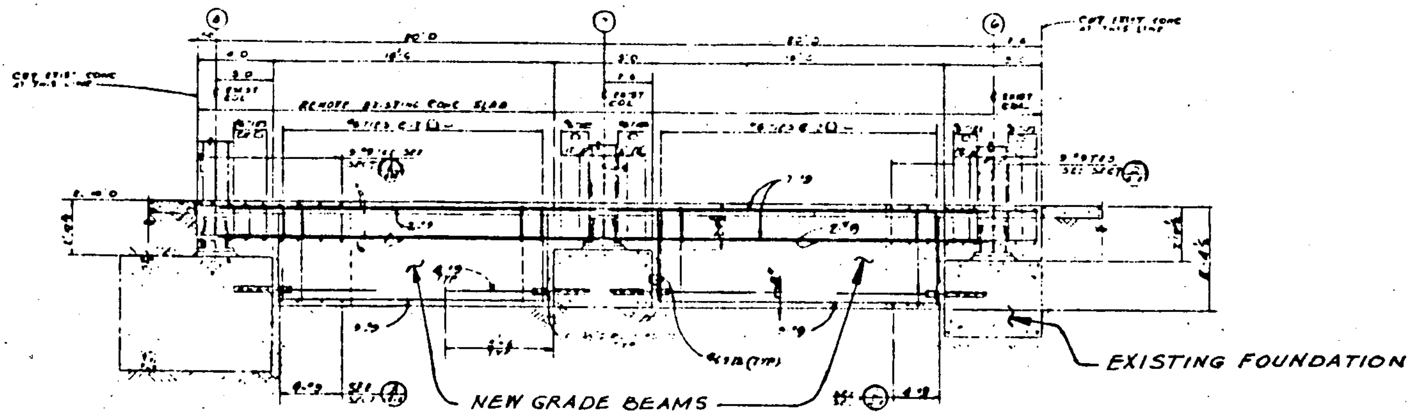
29'0"
28'5"
1'11½"
27'6"
18'6"
17'7½"
17'7½"
16'1"
13'6"
32'6"
19'2"
19'2"
29'8"
26'2"
22'10"
20'0"
30'0"
22'10"
26'2"
24'6"
7'3
17'3"
6'6
20'0"
20'0"
6'6
17'3"
7'3
24'6"
CONTROL
ADMINISTRATION
BLDG.
Anchor Block #2
Masonry Wall
Turbine Pedestal Mat Foundation
Anchor Block #1
FUEL STORAGE
BLDG.
Masonry Wall
FOUNDATION PLAN

FIGURE 16

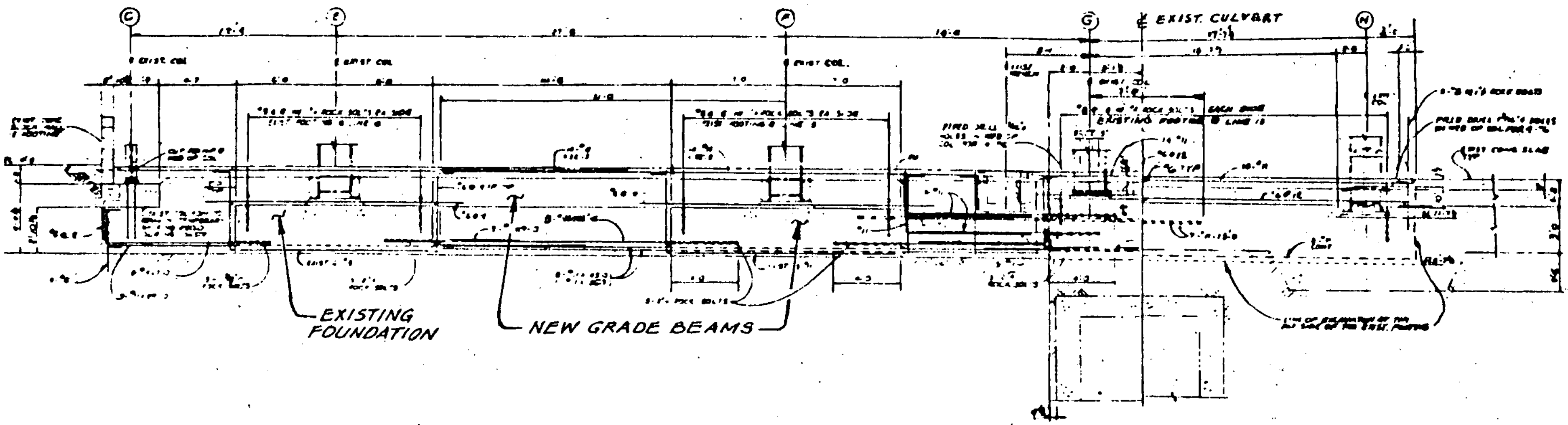


29'0"
28'5"
1'11½"
27'6"
18'6"
17'7½"
17'7½"
16'1"
13'6"
32'6"
19'2"
19'2"
29'8"
26'2"
22'10"
20'0"
30'0"
22'10"
26'2"
24'6"
7'3
17'3"
6'6
20'0"
20'0"
6'6
17'3"
7'3
24'6"
CONTROL ADMINISTRATION BLDG.
Anchor Block #2
EXISTING FOUNDATION
Turbine Pedestal Mat Foundation
MODIFIED FOUNDATION
Anchor Block #1
Masonry Wall
FUEL STORAGE BLDG.
EXISTING FOUNDATION
FOUNDATION PLAN
MODIFIED FOUNDATION

FIGURE 17

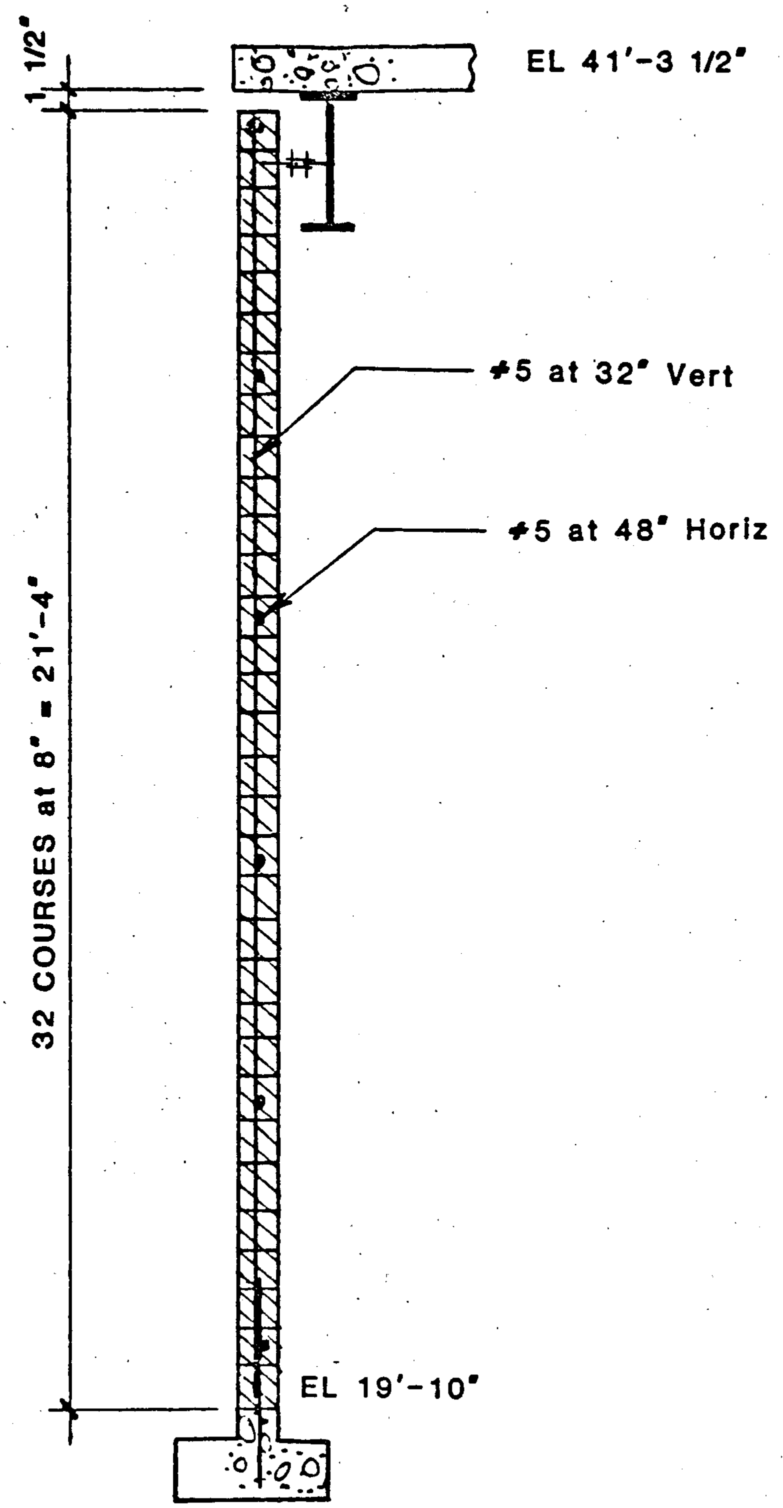


SECTION
LOOKING NORTH



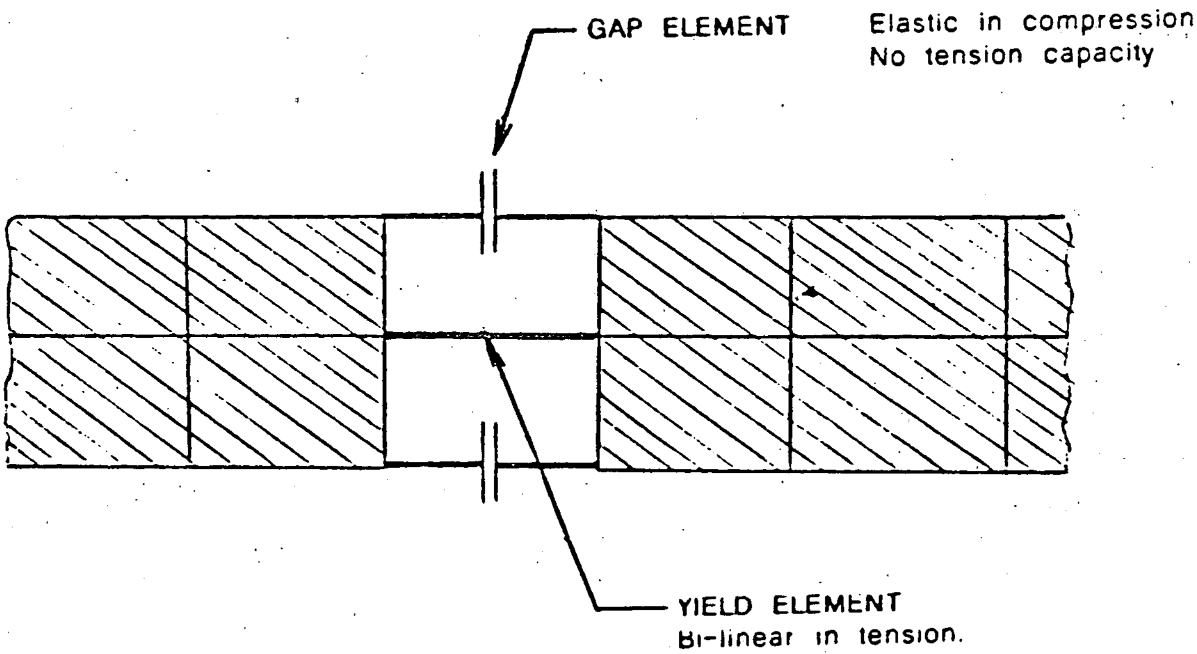
SECTION
LOOKING EAST

FIGURE 8



TYPICAL SECTION THRU MASONRY WALL

FIGURE 19



MODEL FOR CRACKING AND YIELDING JOINT

FIGURE 20

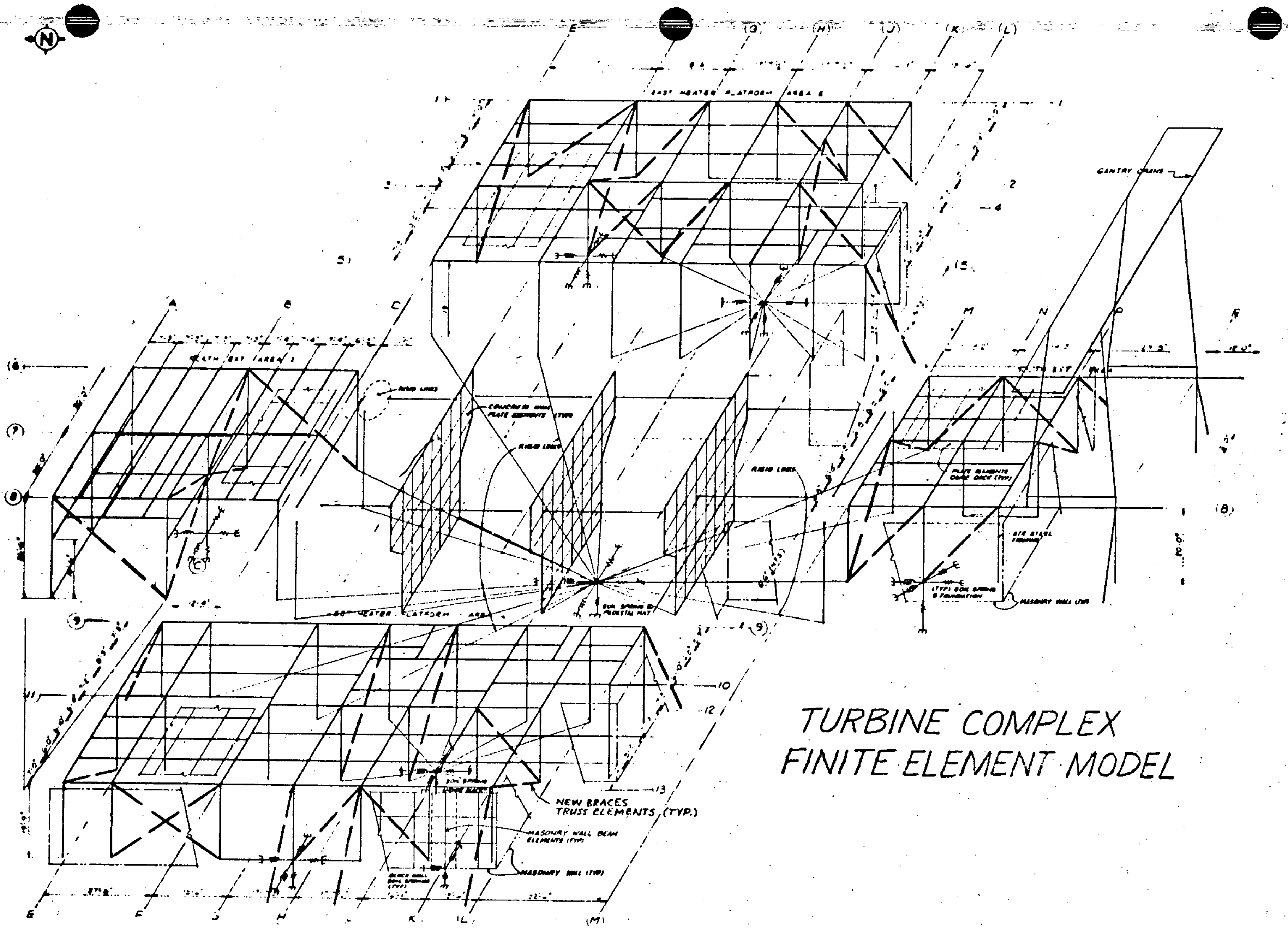
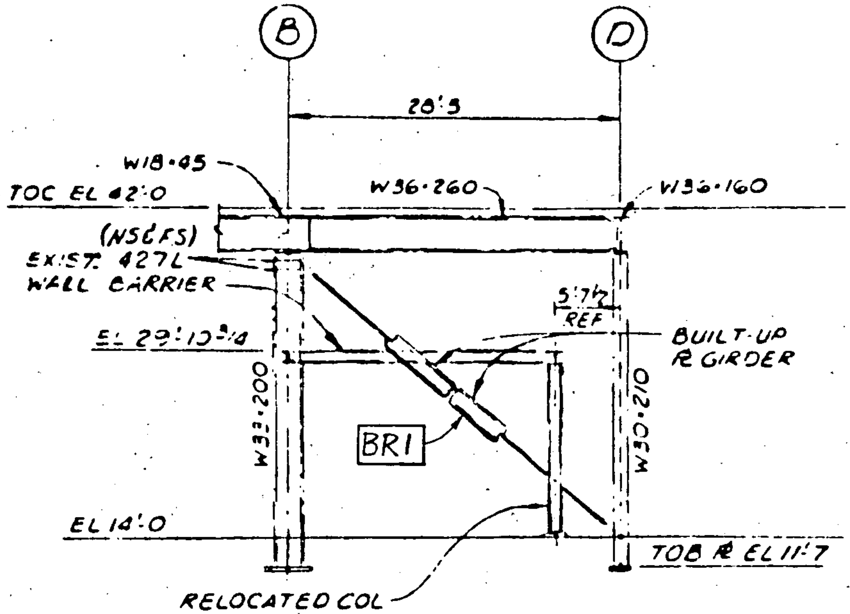
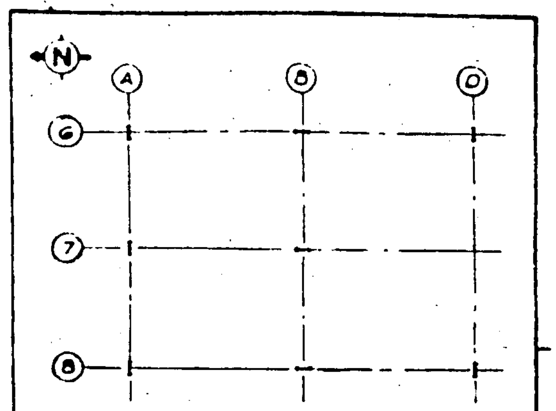

TURBINE COMPLEX
FINITE ELEMENT MODEL
NEW BRACES
TRUSS ELEMENTS (TYP.)
GANTRY CRANE

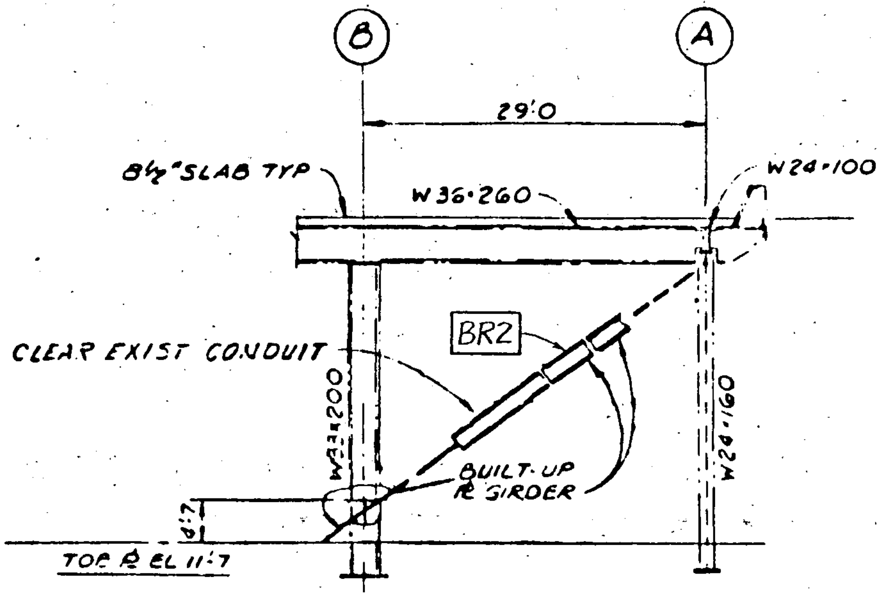
FIGURE 21



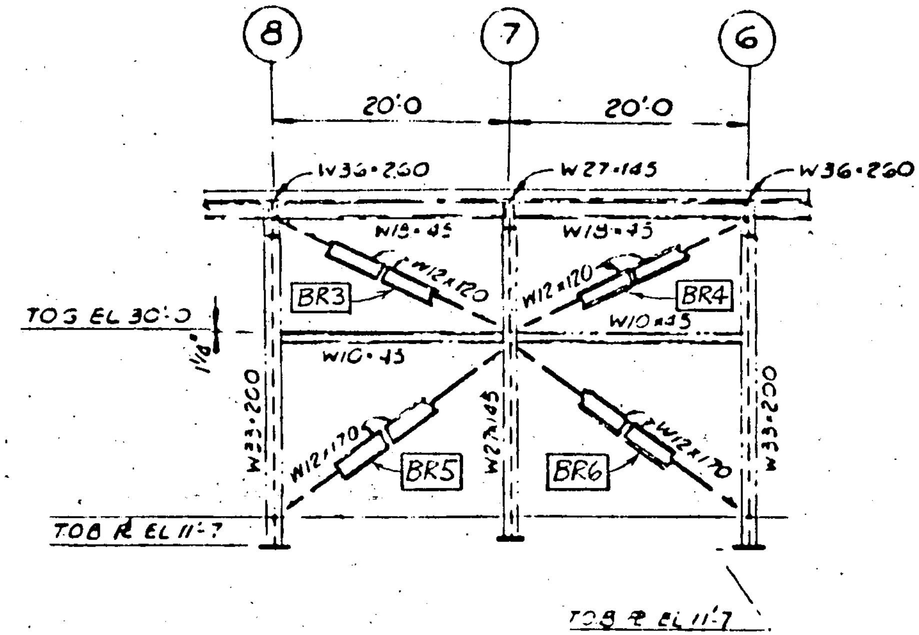
ELEVATION LOOKING EAST
ON COL LINE·6



KEY PLAN - AREA 2
NORTH EXTENSION FRAMING



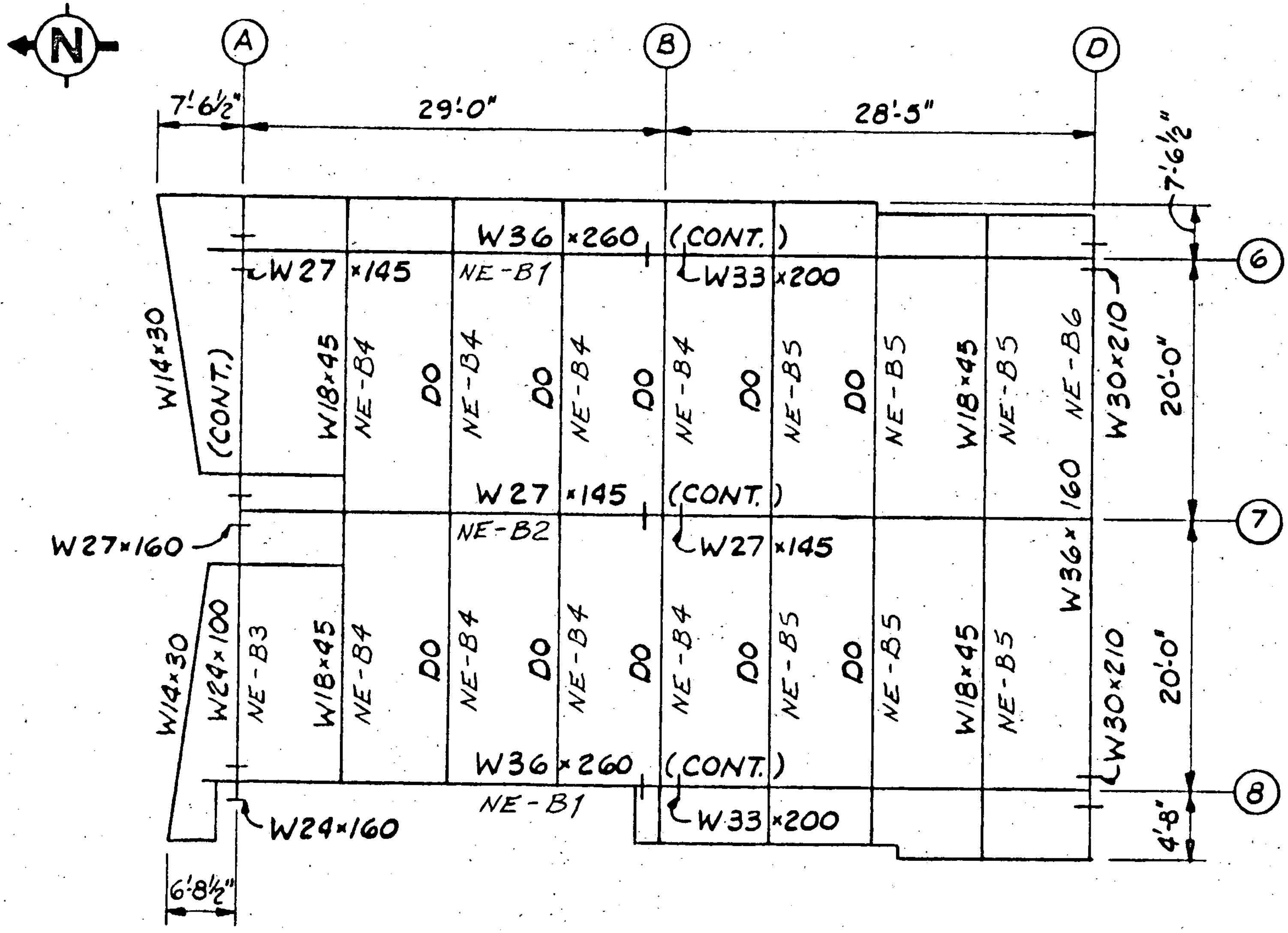
ELEVATION LOOKING WEST
ON COL LINE·8



ELEVATION LOOKING NORTH
ON COL LINE - B

FIGURE 22

REF TABL



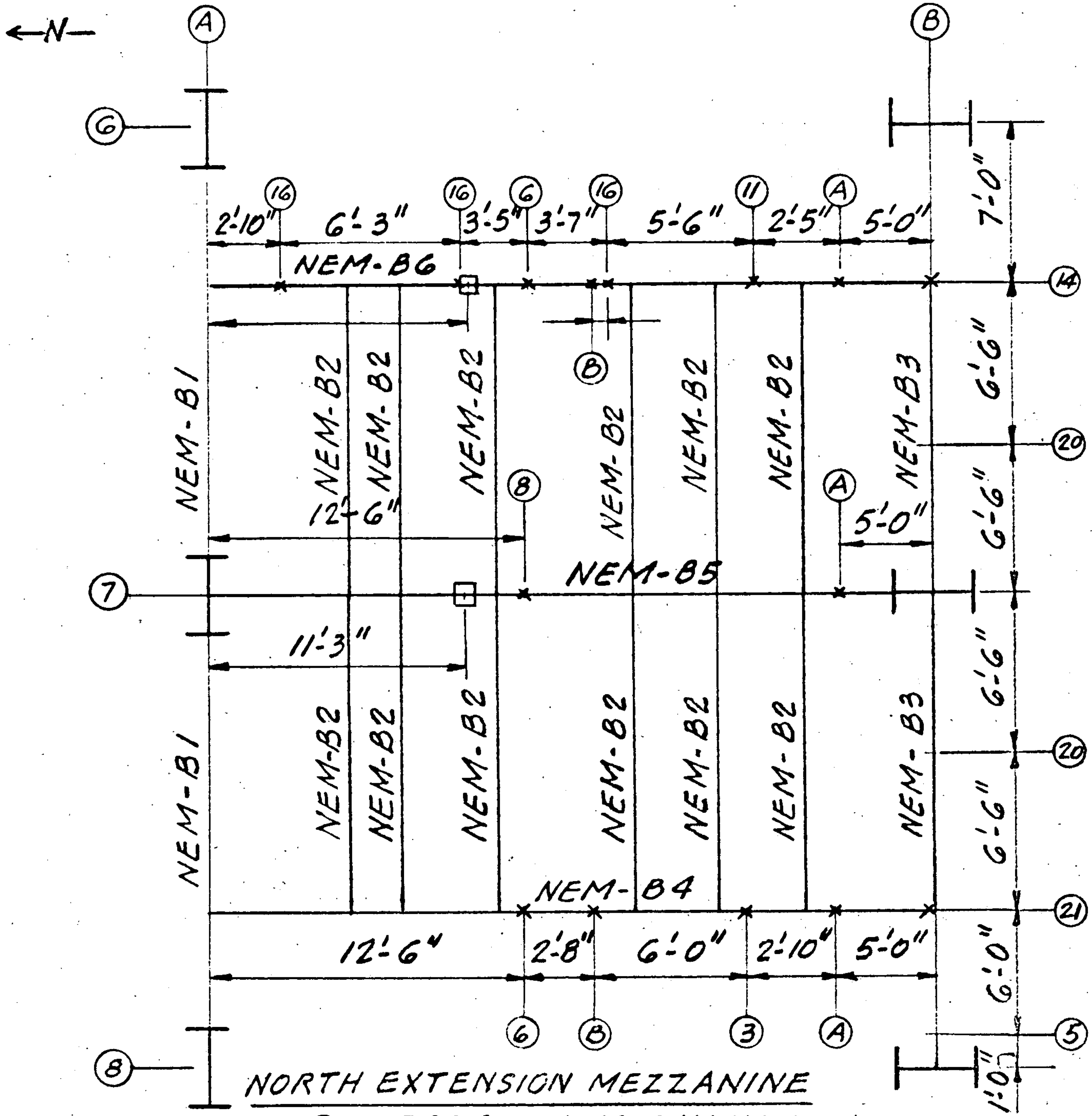
NORTH EXTENSION

GIRDERS & BEAMS EVALUATION

PLAN @ EL. 42'-0"

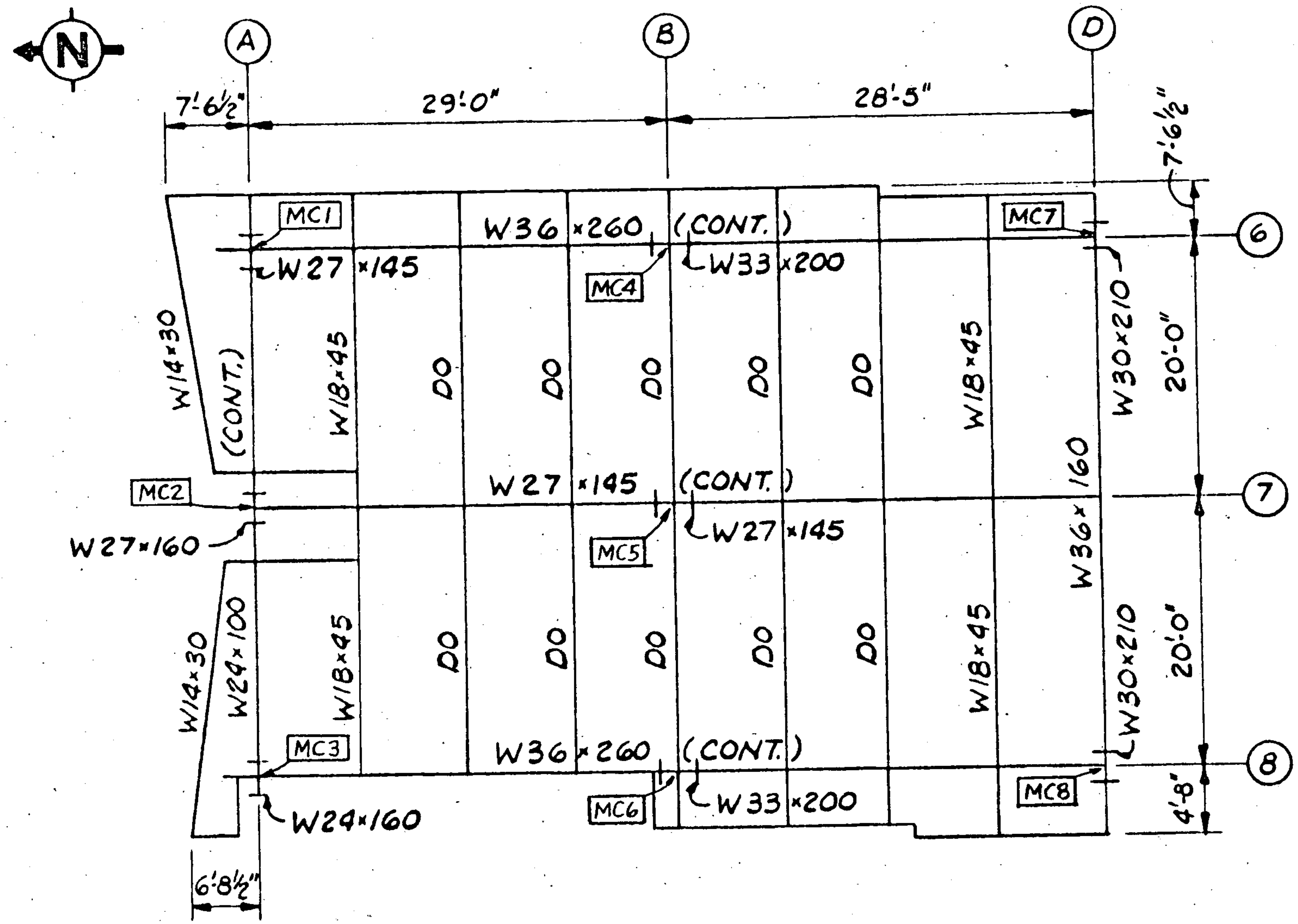
FIGURE 23

REF. TABLE 3



NORTH EXTENSION MEZZANINE
GIRDERS & BEAMS EVALUATION
PLAN @ EL. 30'-0

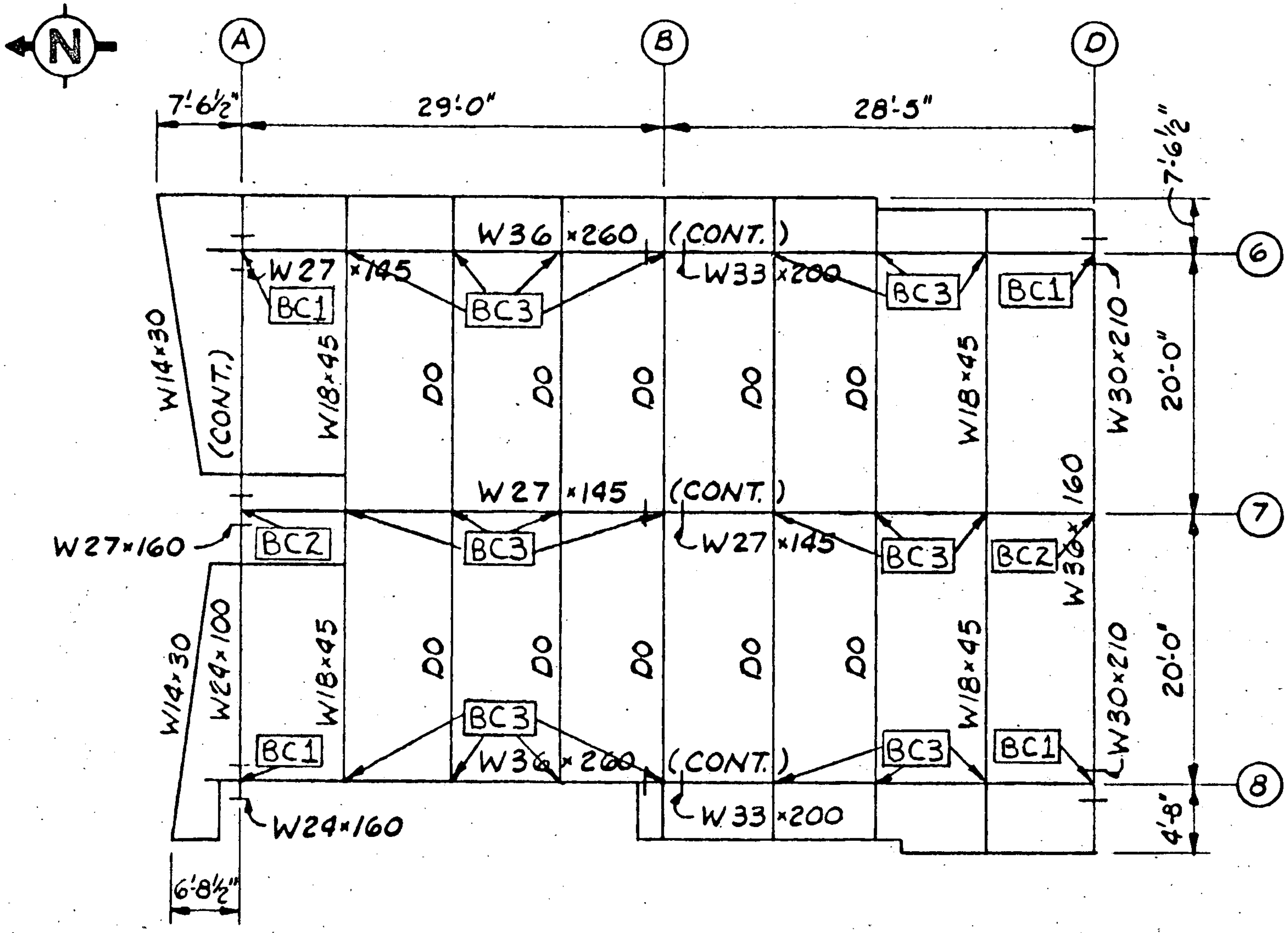
FIGURE 24



NORTH EXTENSION
MOMENT CONNECTION EVALUATION
PLAN @ EL. 42'-0"

FIGURE 25

REF. TABLE 5



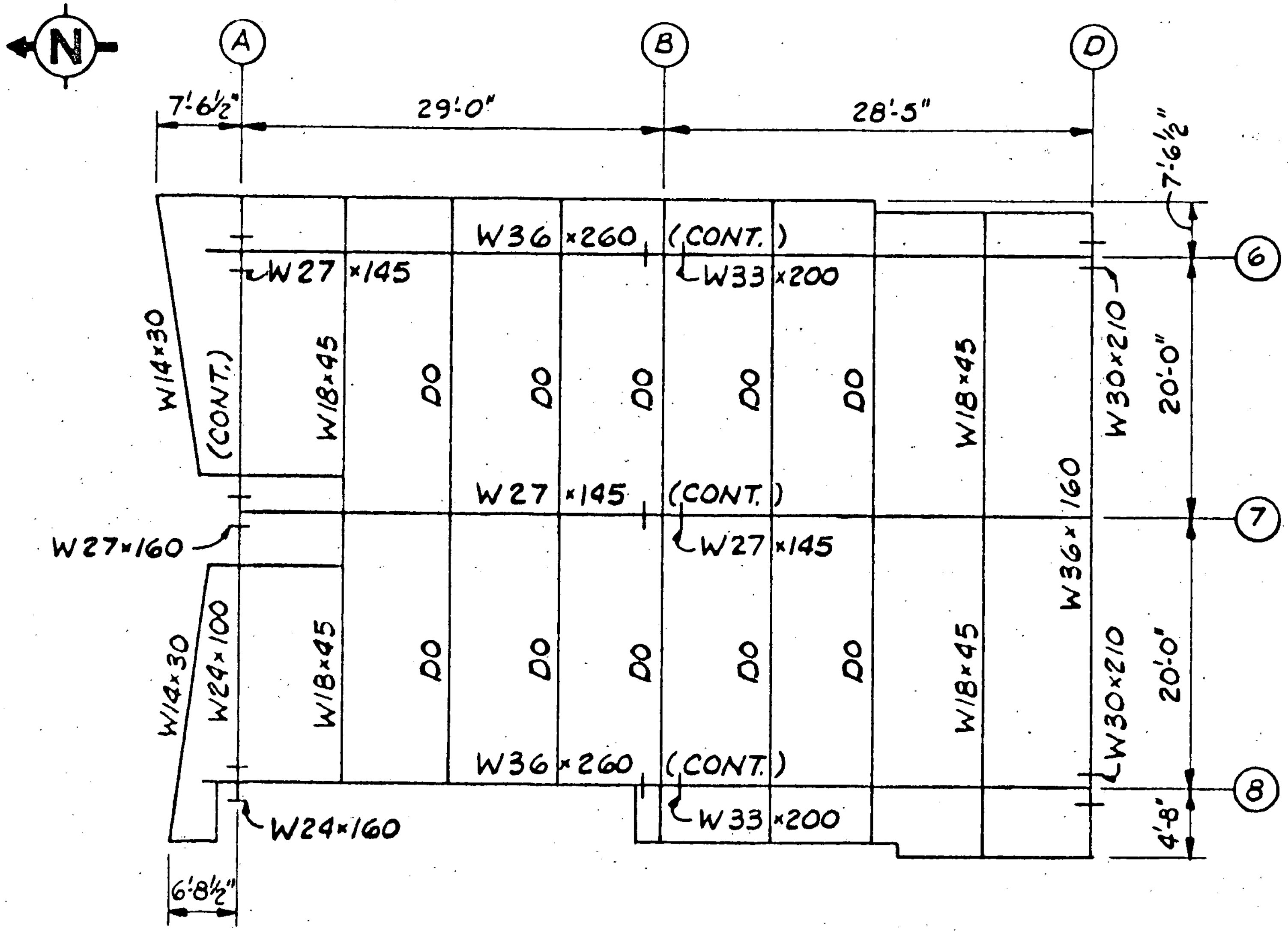
NORTH EXTENSION

BOLTED CONNECTION EVALUATION

PLAN @ EL. 42'-0"

FIGURE 26

REF. TABLE 6



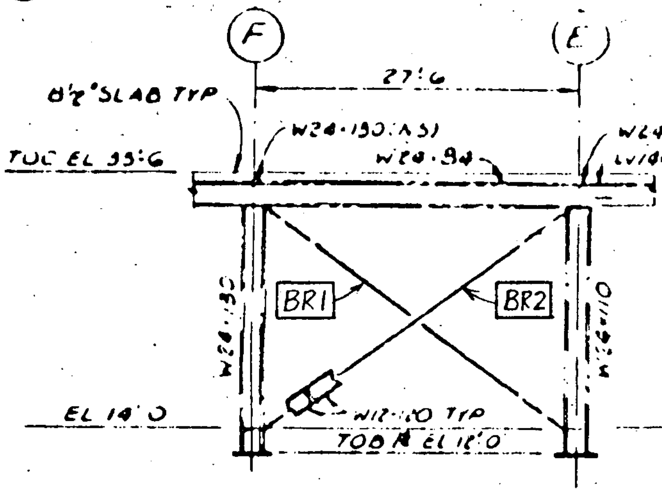
NORTH EXTENSION

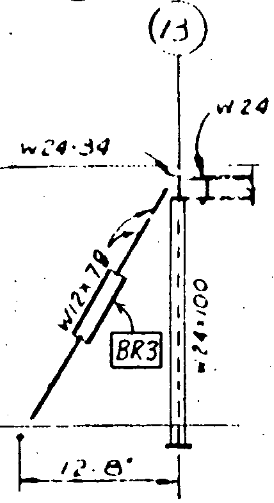
COLUMN ANCHORAGE EVALUATION

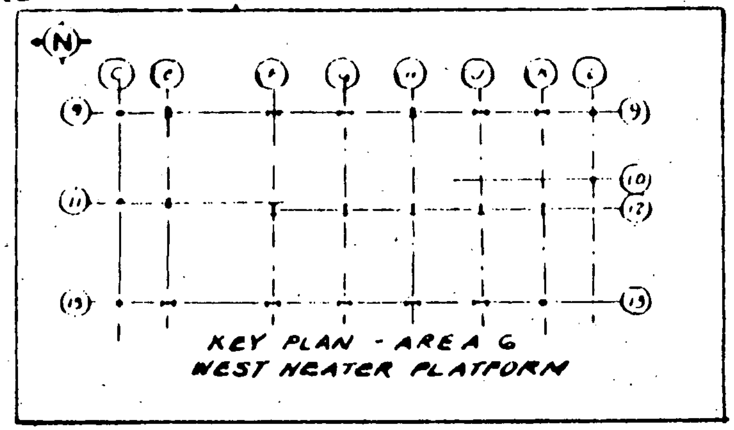
PLAN @ EL. 42'-0"

FIGURE 27

REF. TABLE 7 & 8

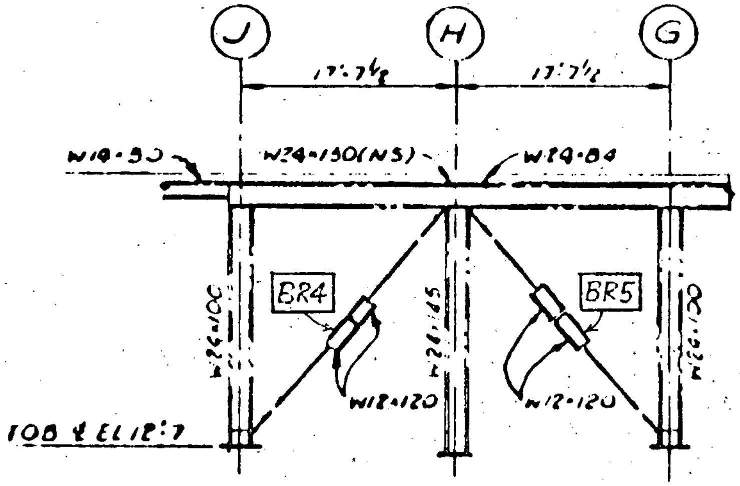


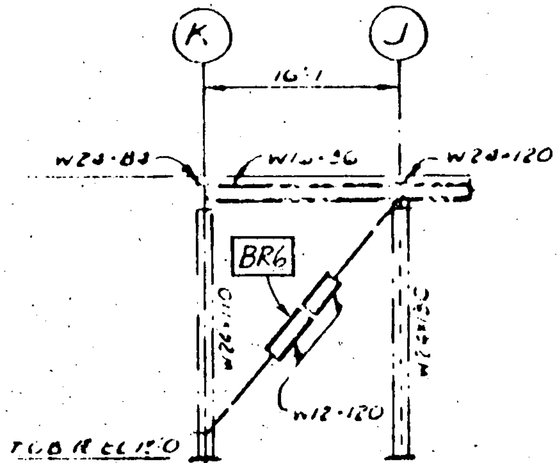


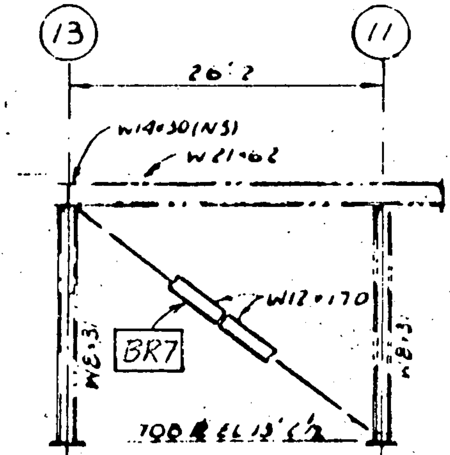


ELEVATION LOOKING WEST
ON COL LINE-13

ELEVATION LOOKING NORTH
ON COL LINE-G







ELEVATION LOOKING WEST
ON COL LINE-13

ELEVATION LOOKING WEST
ON COL LINE-12

ELEVATION LOOKING NORTH
ON COL LINE-C

FIGURE 28

REF. TABLE 9

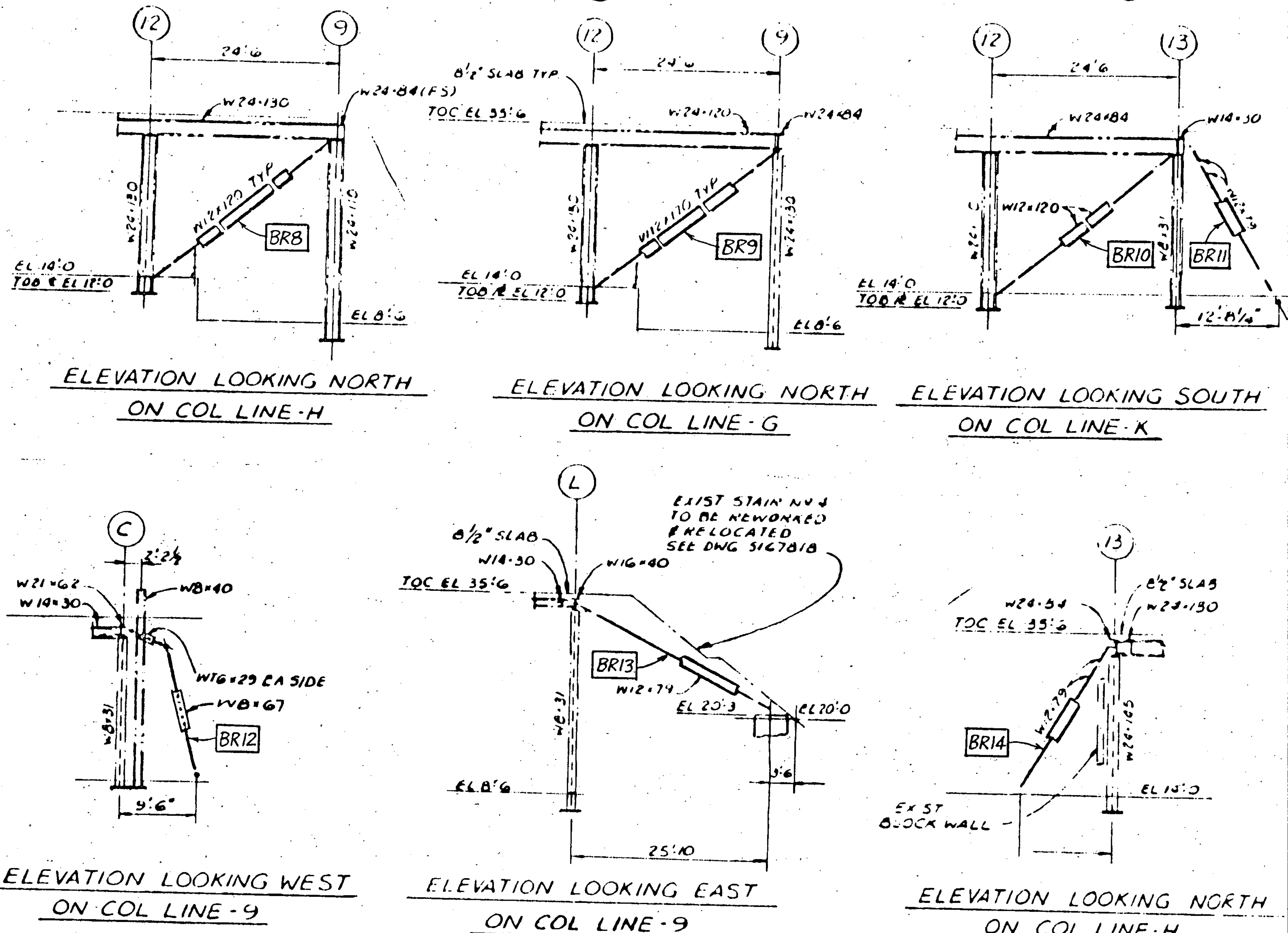
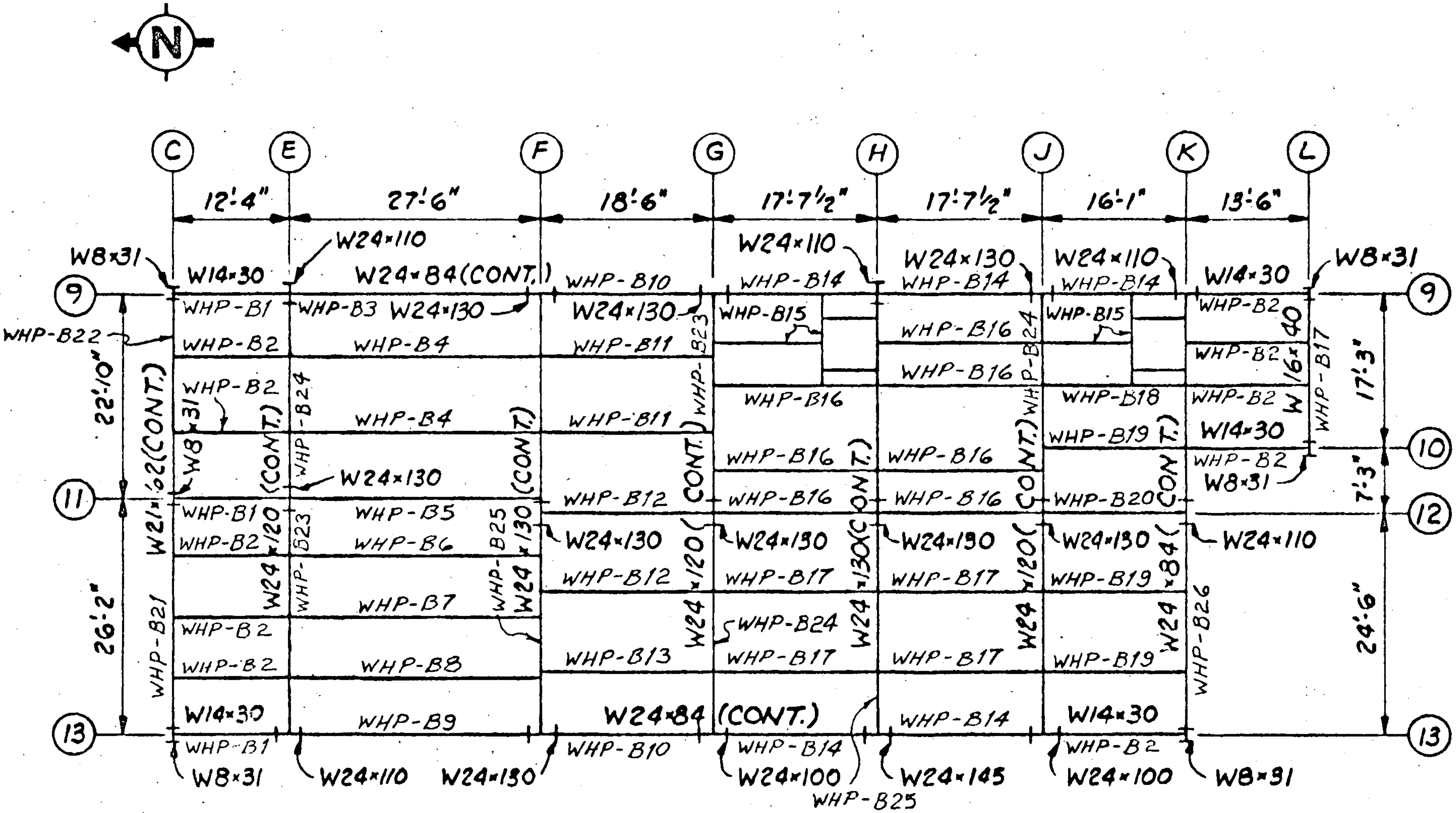


FIGURE 29

REF. TABLE 9.1



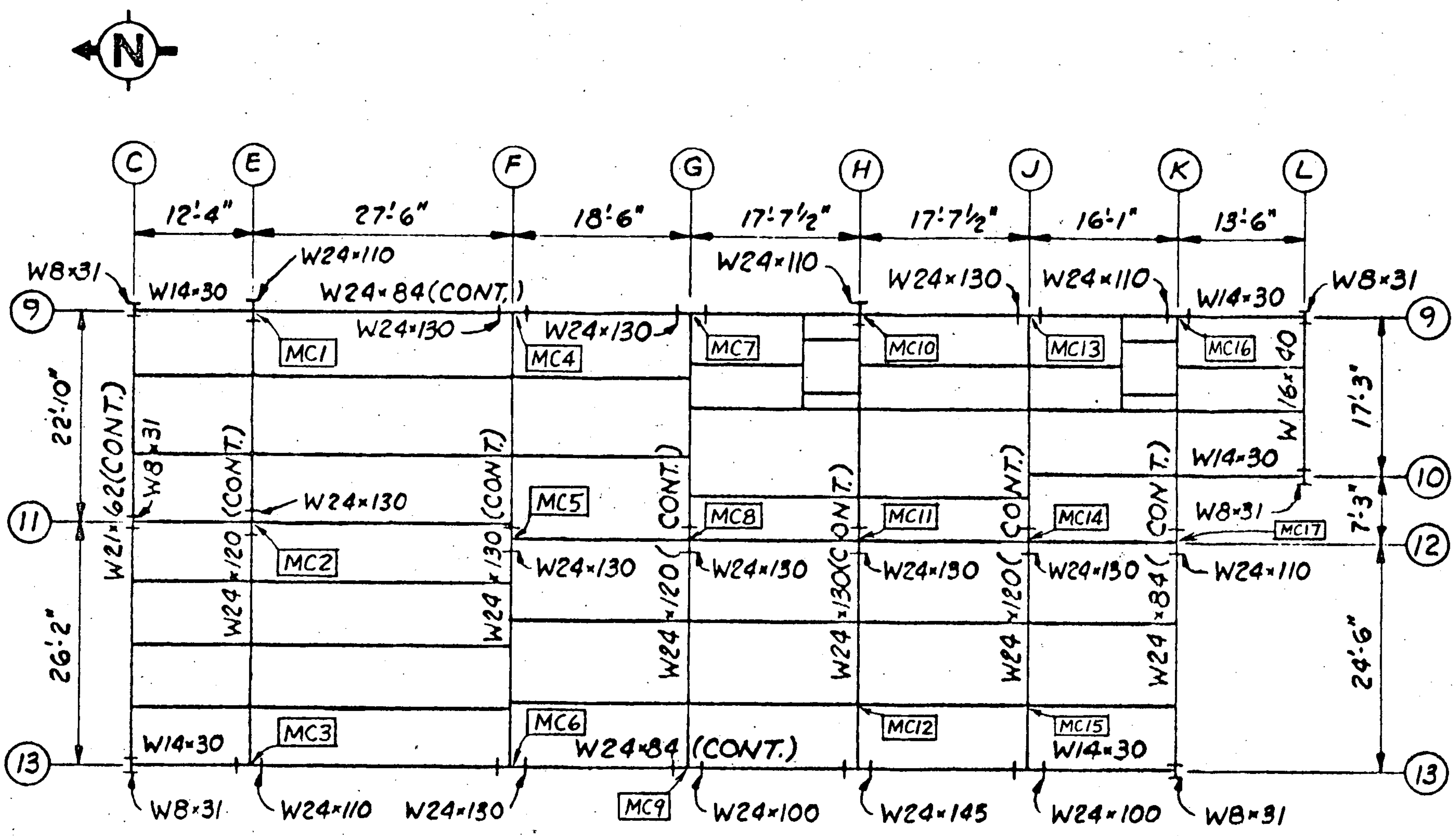
WEST HEATER PLATFORM

GIRDERS & BEAMS EVALUATION

PLAN @ EL. 35'-6"

FIGURE 30

REF. TABLE 11 & 12



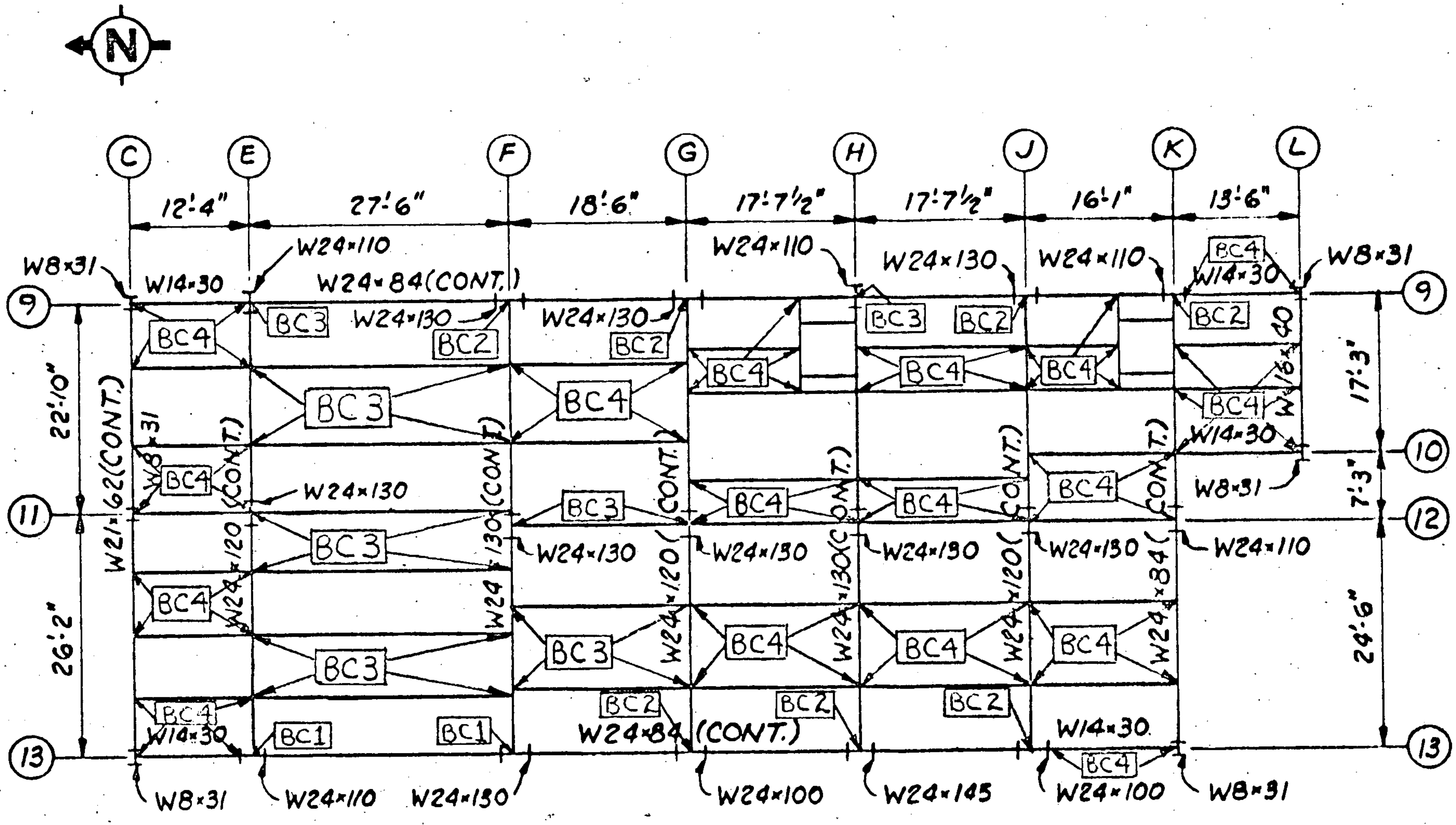
<u>WEST HEATER PLATFORM</u>

<u>MOMENT CONNECTION EVALUATION</u>

<u>PLAN @ EL. 35'-6"</u>

<u>FIGURE 31</u>

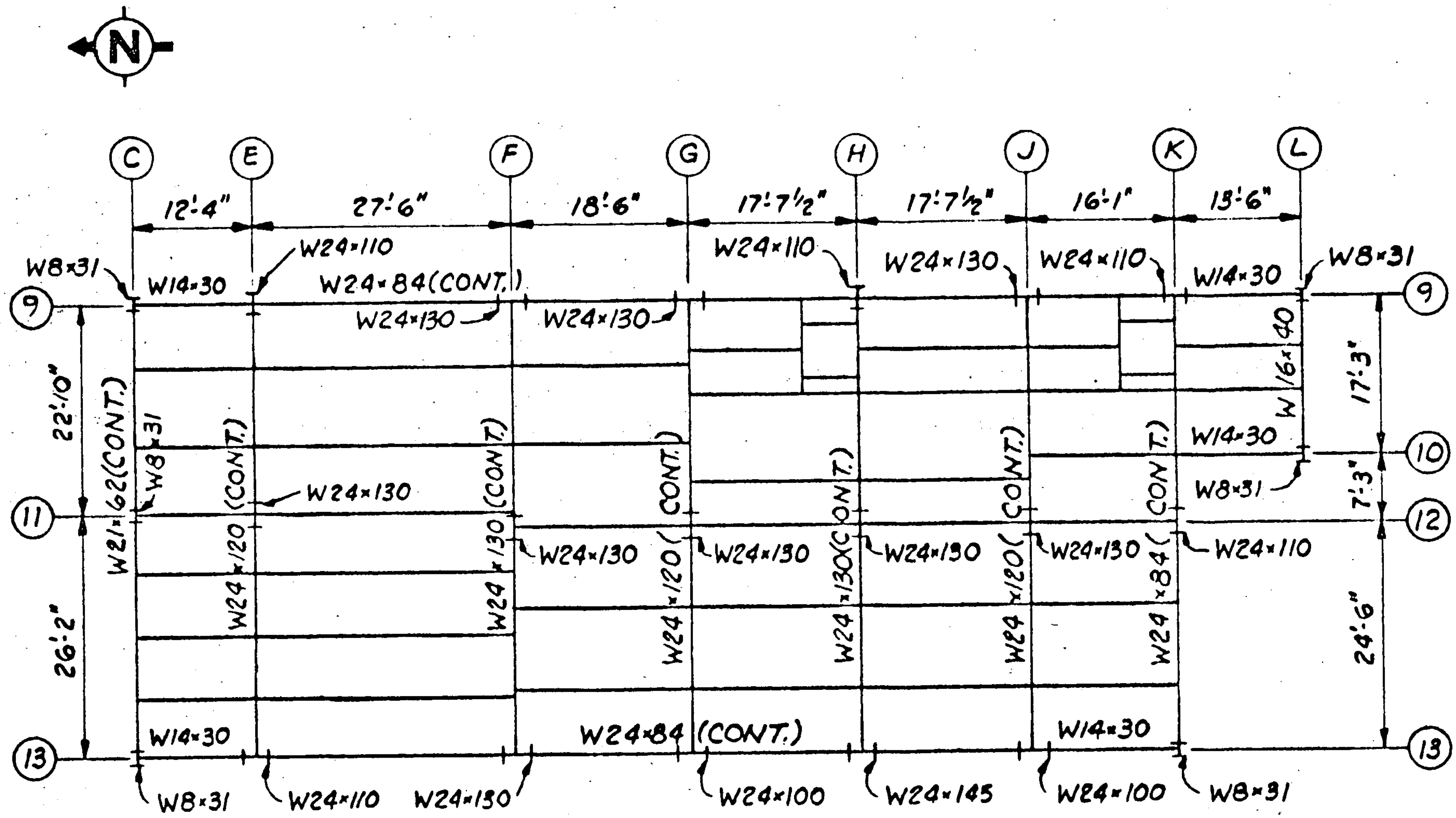
REF. TABLE 13 & 14



WEST HEATER PLATFORM
BOLTED CONNECTION EVALUATION
PLAN @ EL. 35'-6"

FIGURE 32

REF. TABLE 15



WEST HEATER PLATFORM

COLUMN ANCHORAGE EVALUATION

PLAN @ EL. 35'-6"

FIGURE 33

REF. TABLE 16, 17, 18 & 19

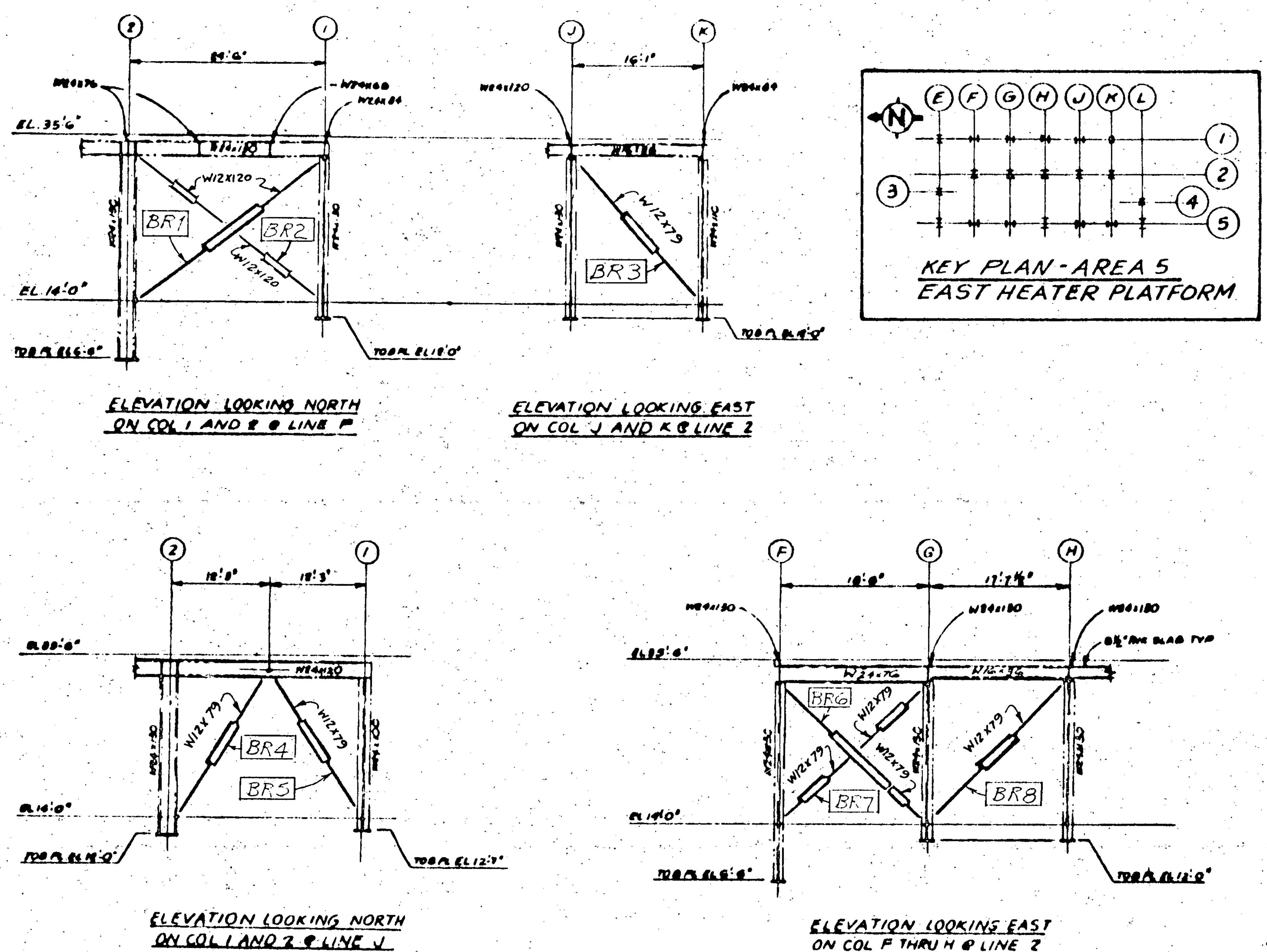


FIGURE 34

REF. TABLE 20

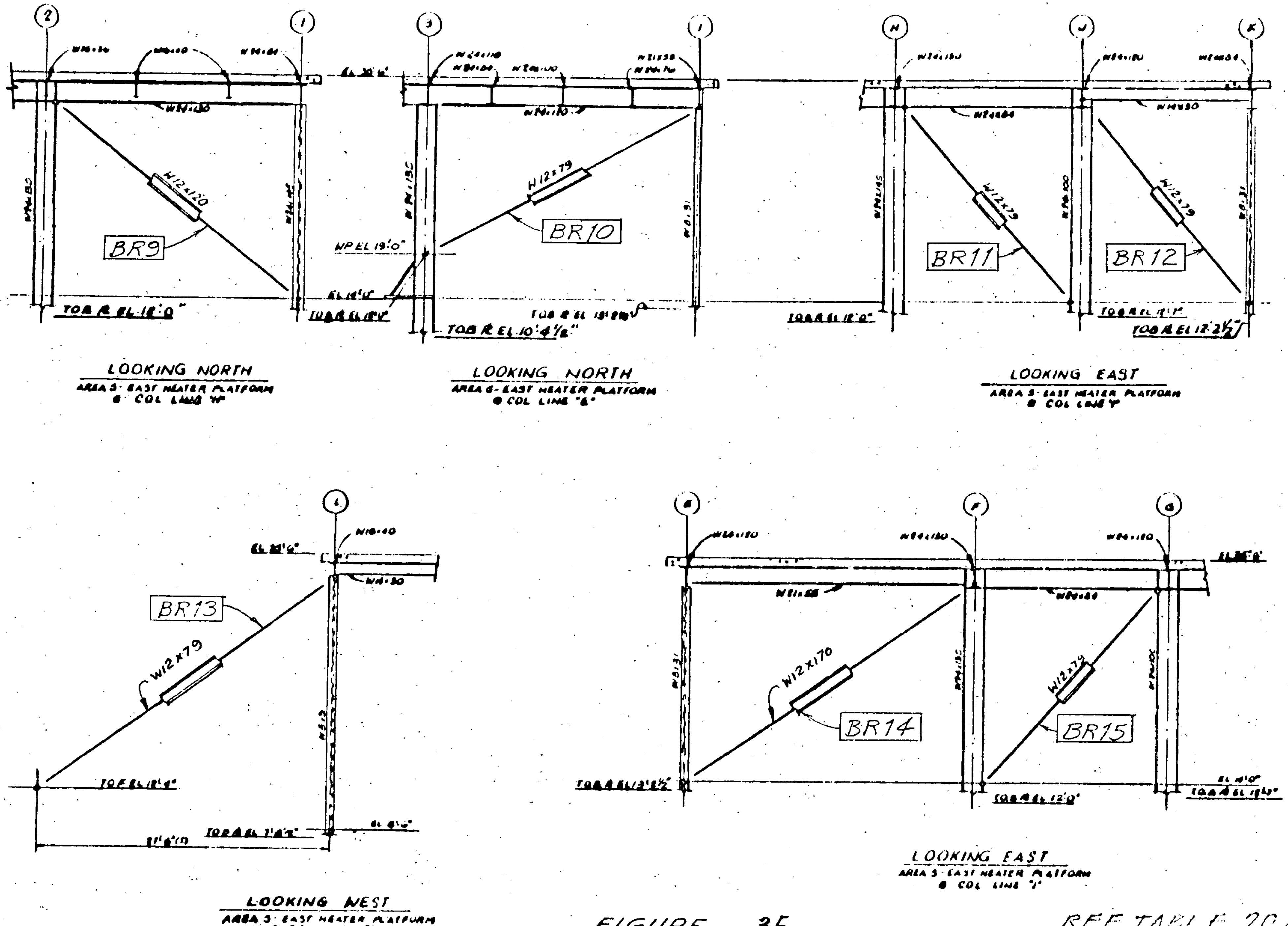
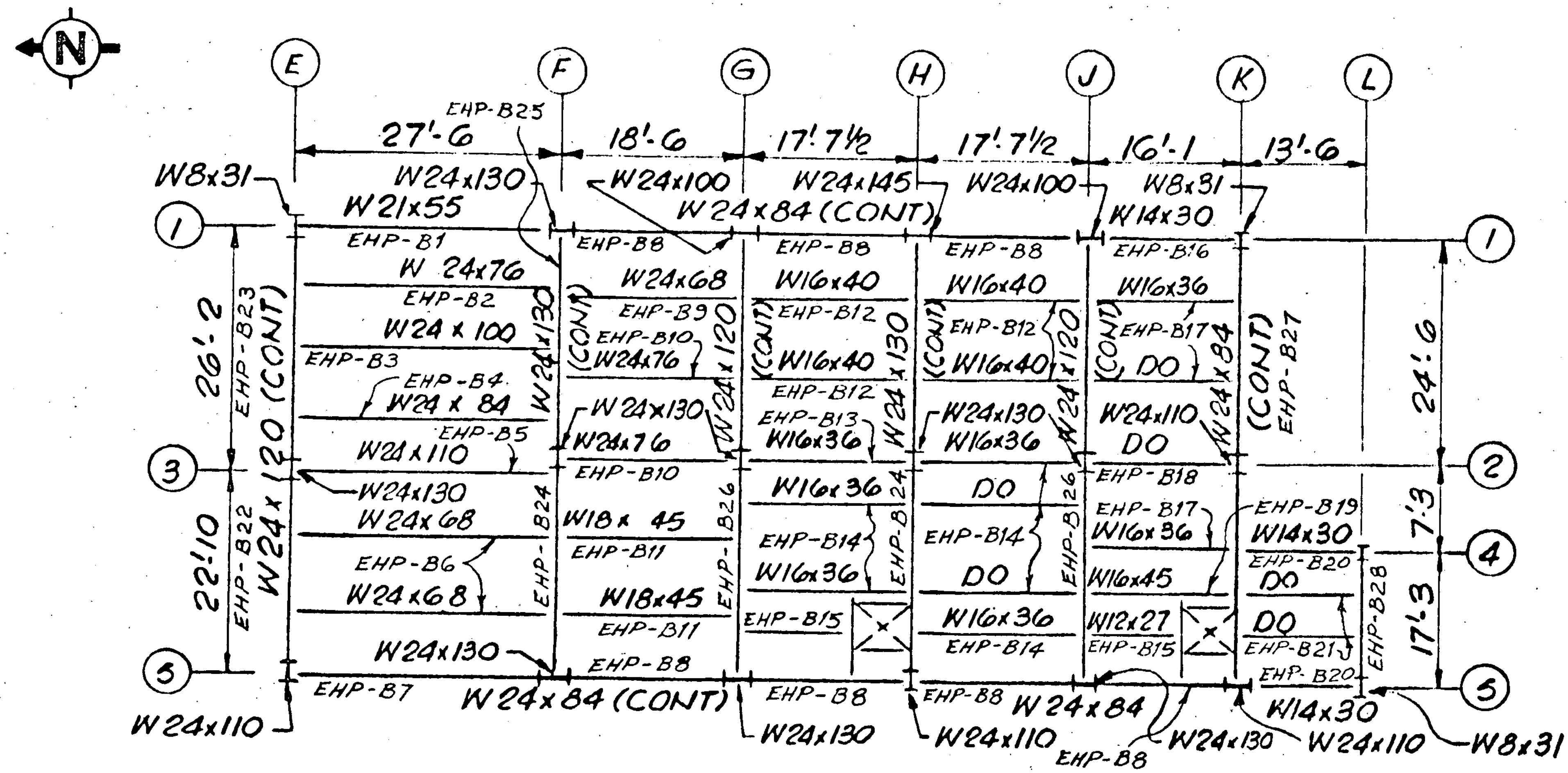


FIGURE 35

REF. TABLE 20 & 21



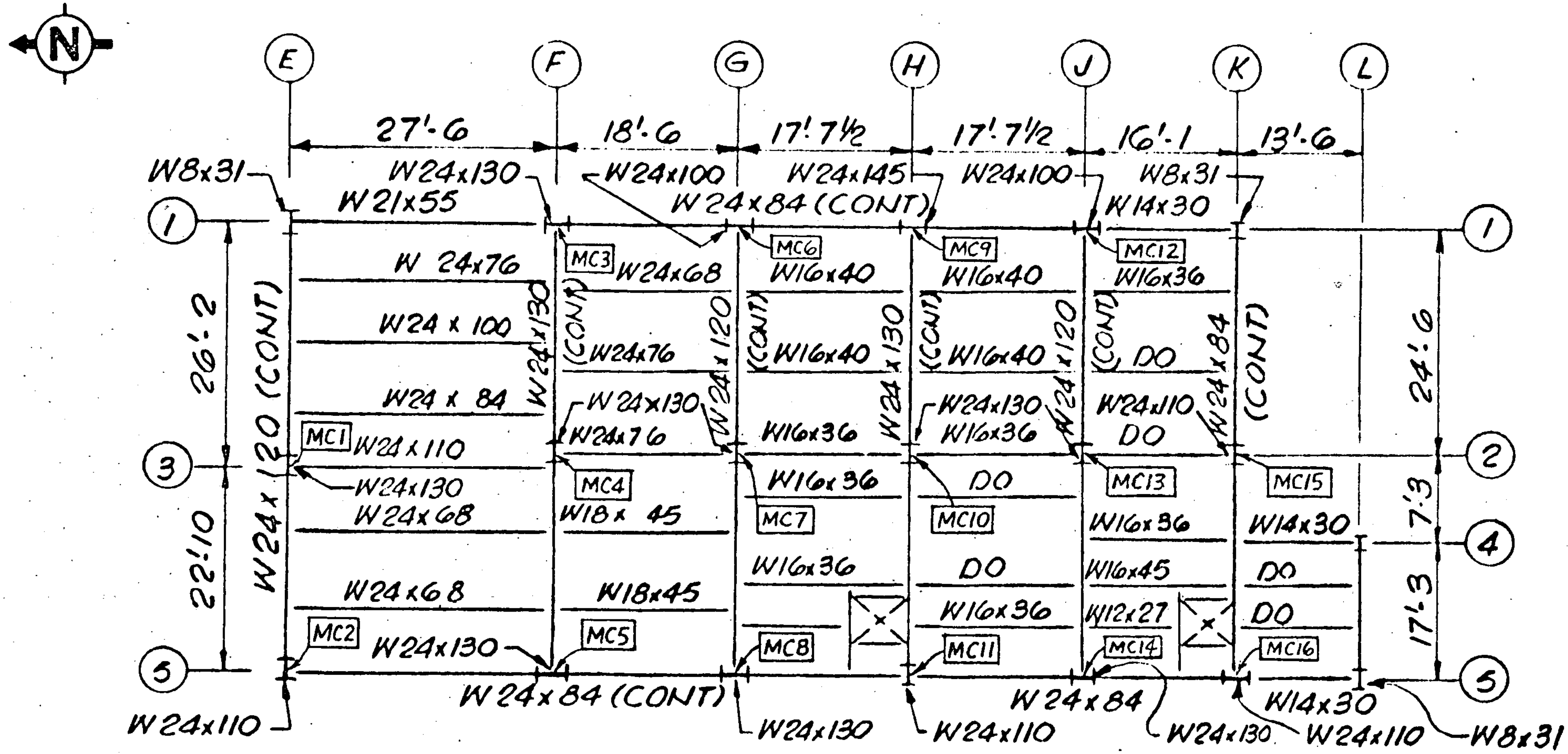
EAST HEATER PLATFORM

GIRDERS & BEAMS EVALUATION

PLAN @ EL 35'-6

FIGURE 36

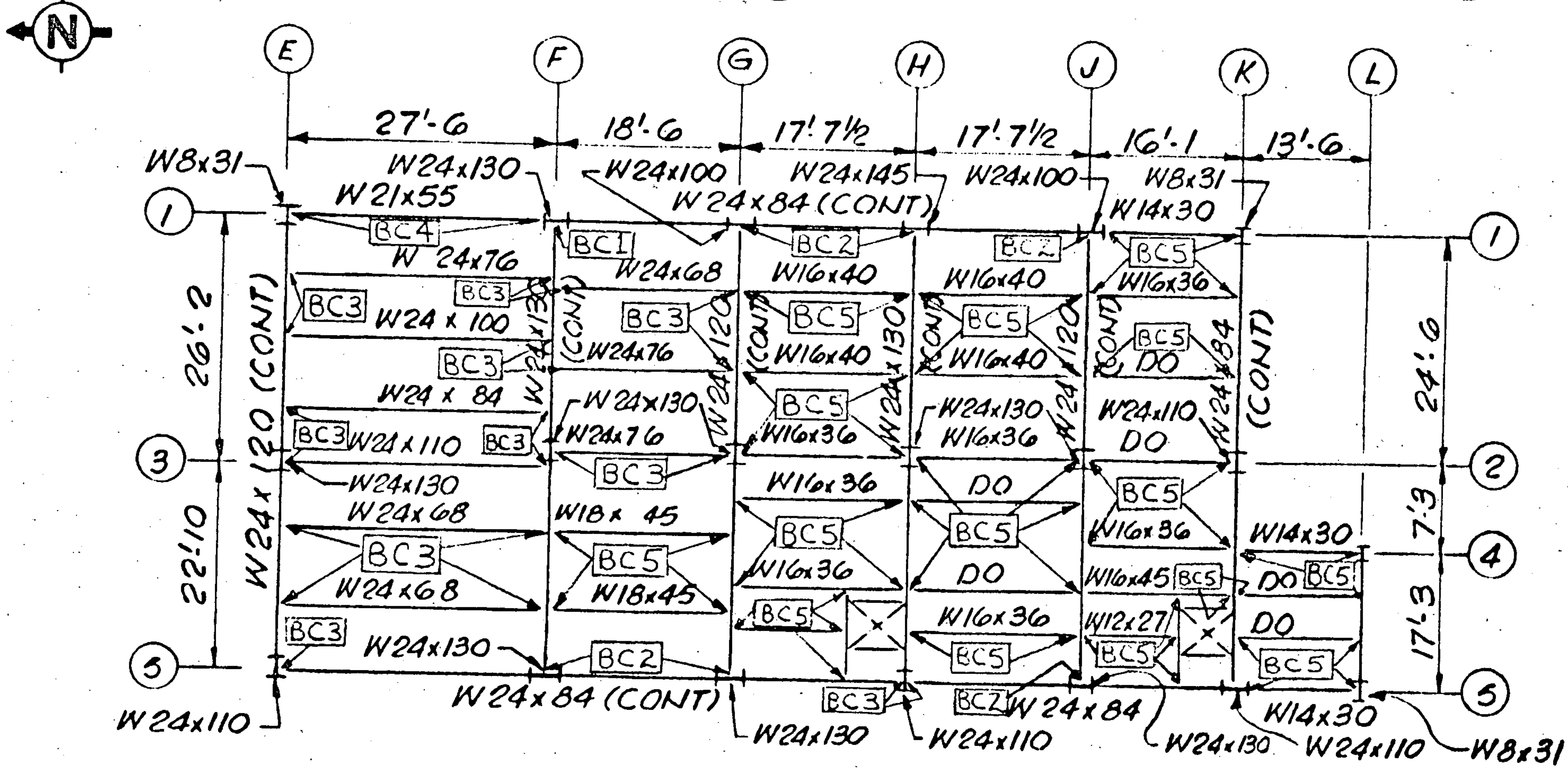
REF. TABLE 22,23&2



EAST HEATER PLATFORM
MOMENT CONNECTION EVALUATION
PLAN @ EL 35'-6

FIGURE 37

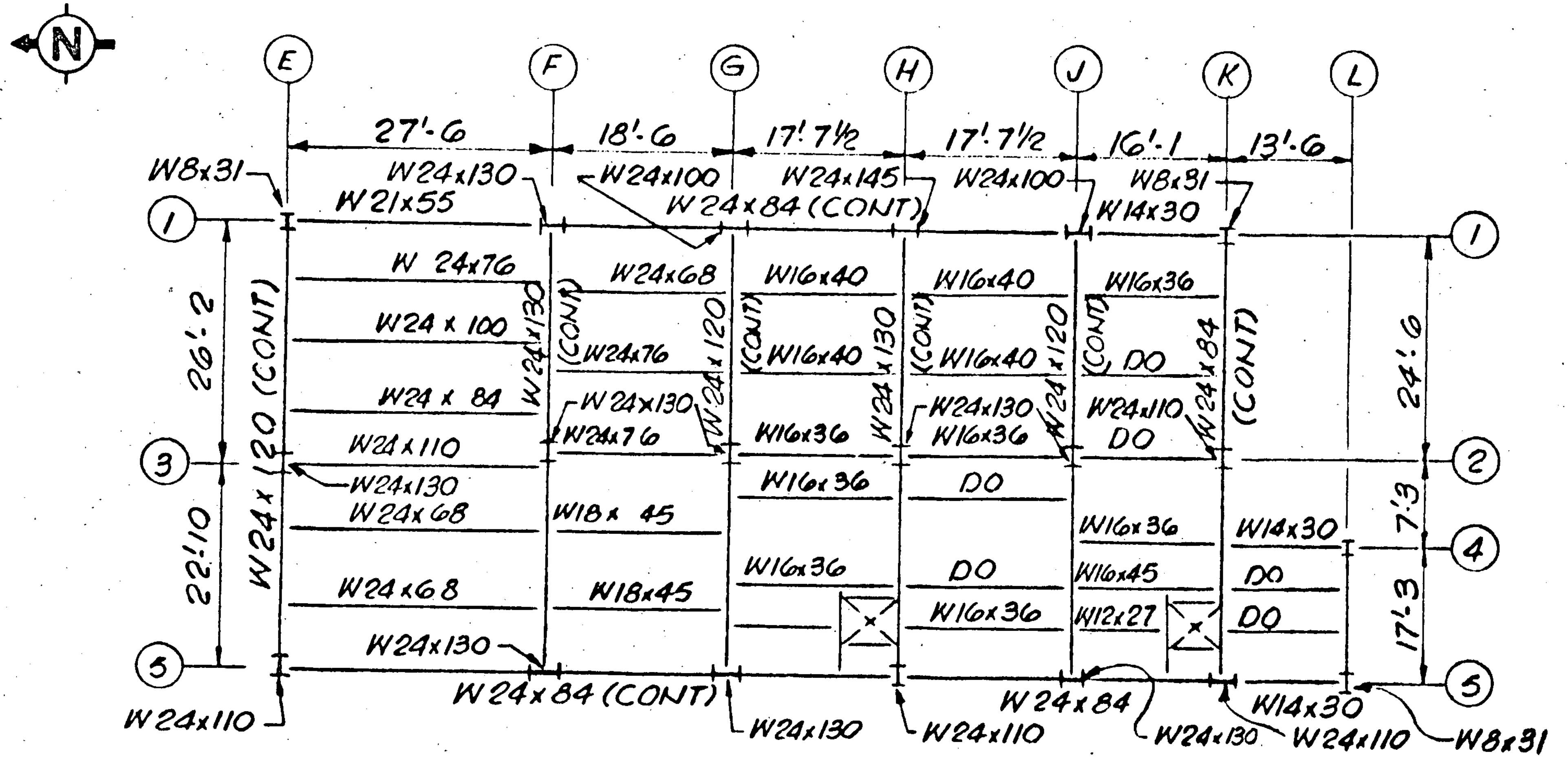
REF. TABLE 25 & 26



EAST HEATER PLATFORM
BOLTED CONNECTION EVALUATION
PLAN @ EL 35'-6

FIGURE 38

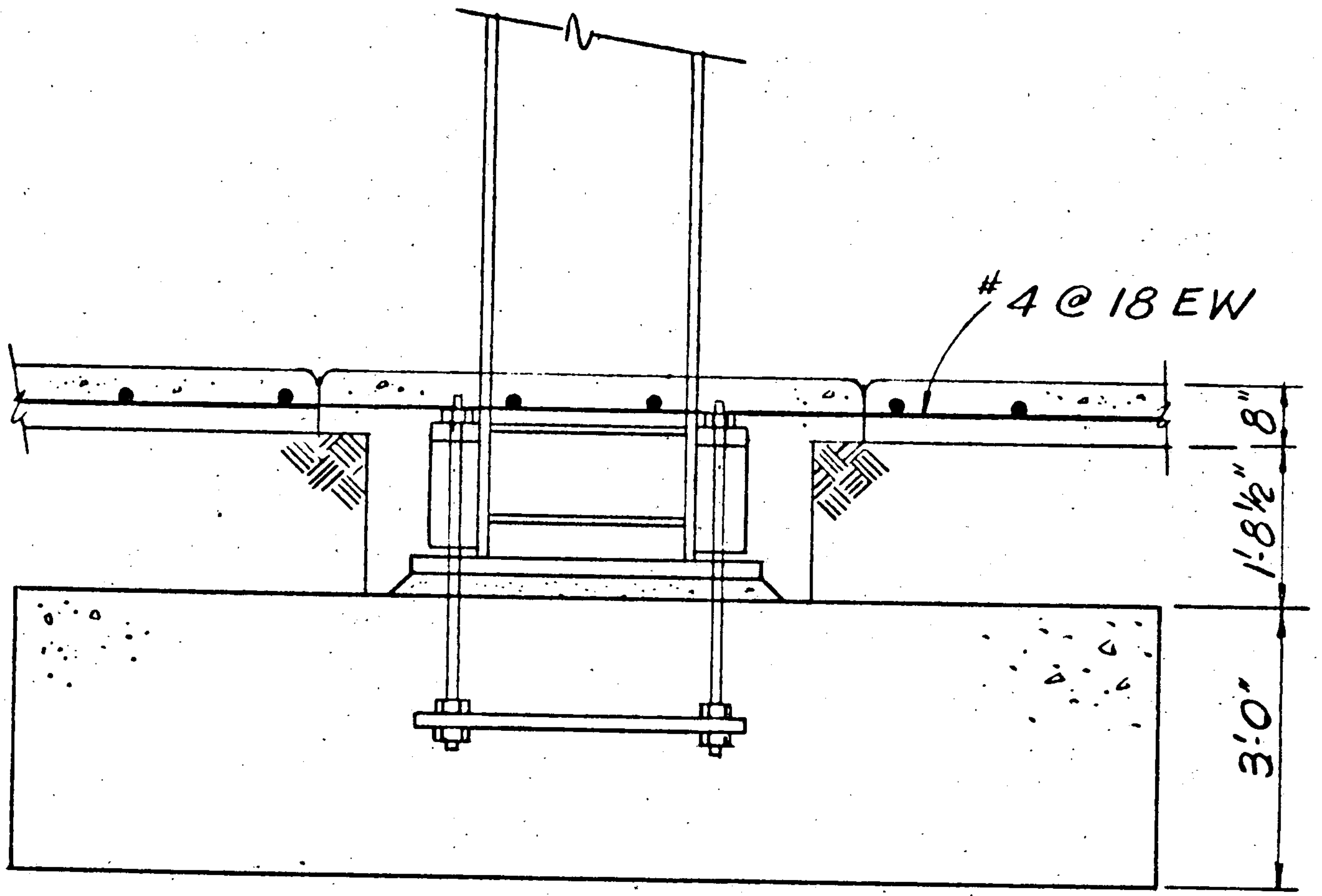
REF. TABLE 27



EAST HEATER PLATFORM
COLUMN ANCHORAGE EVALUATION
PLAN @ EL 35'-6

FIGURE 39

REF. TABLE 28, 29, 30 & 31



TYPICAL COLUMN FOUNDATION CONNECTION

FIGURE 40

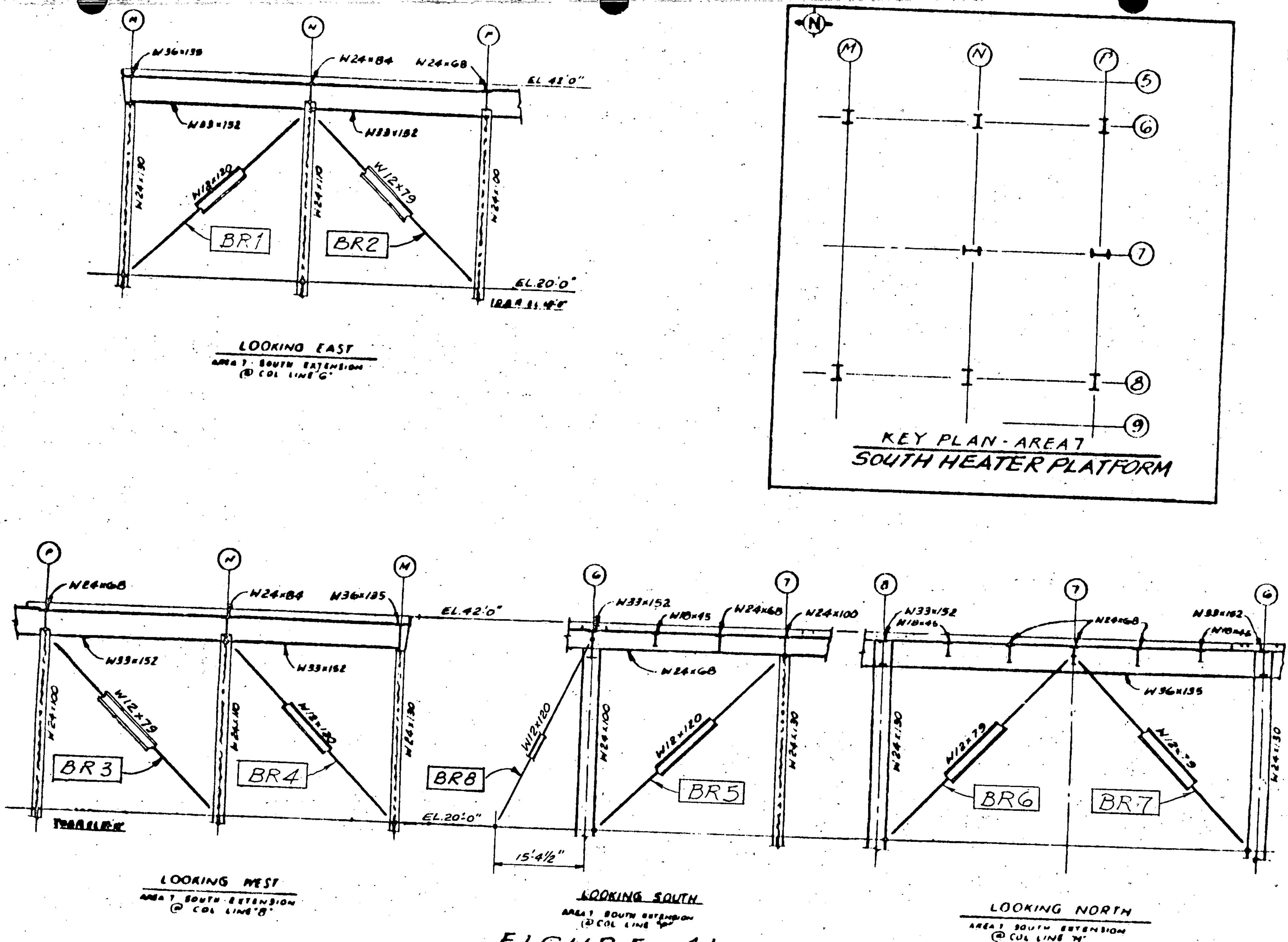
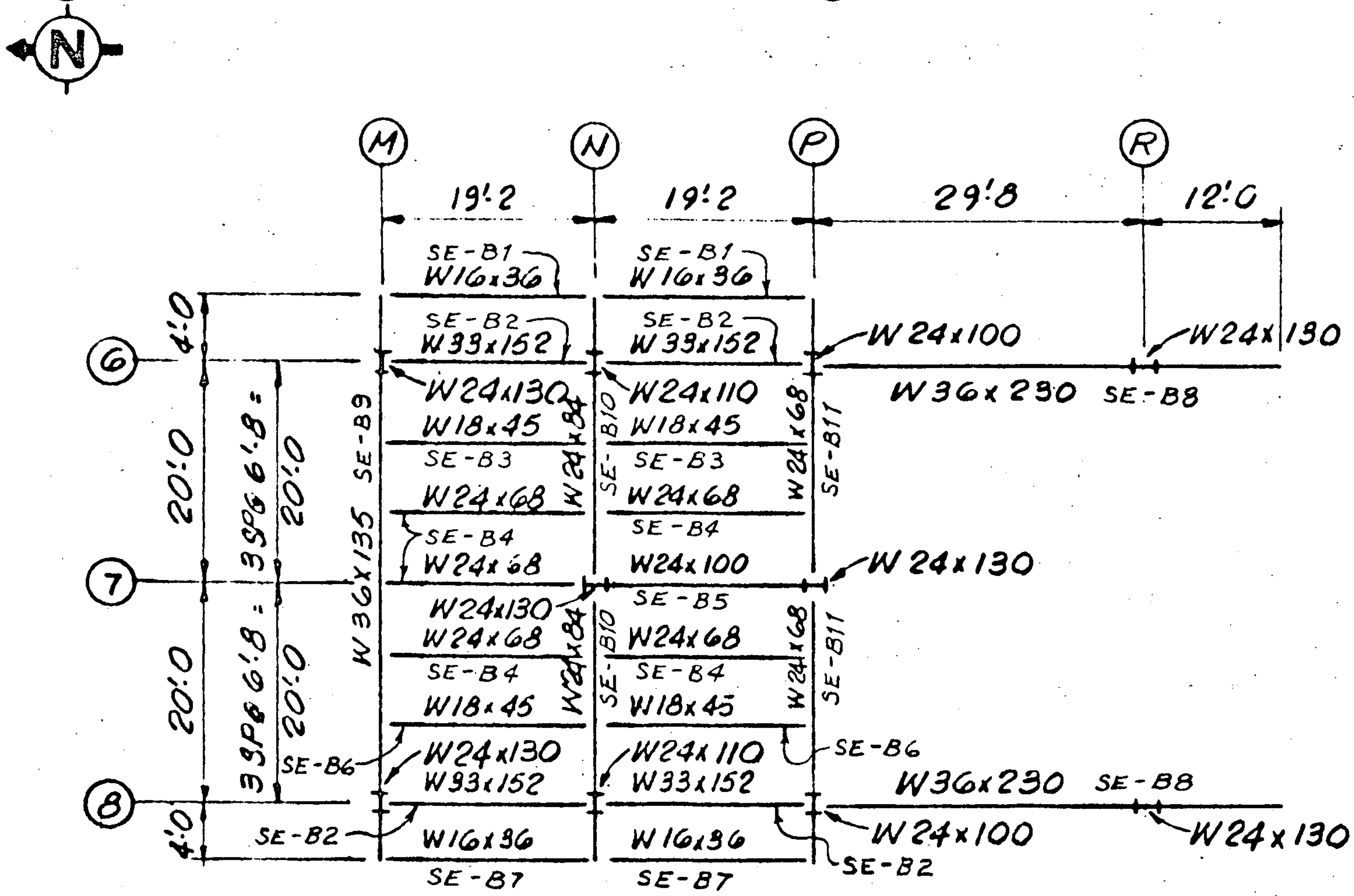


FIGURE 41

REF. TABLE 32



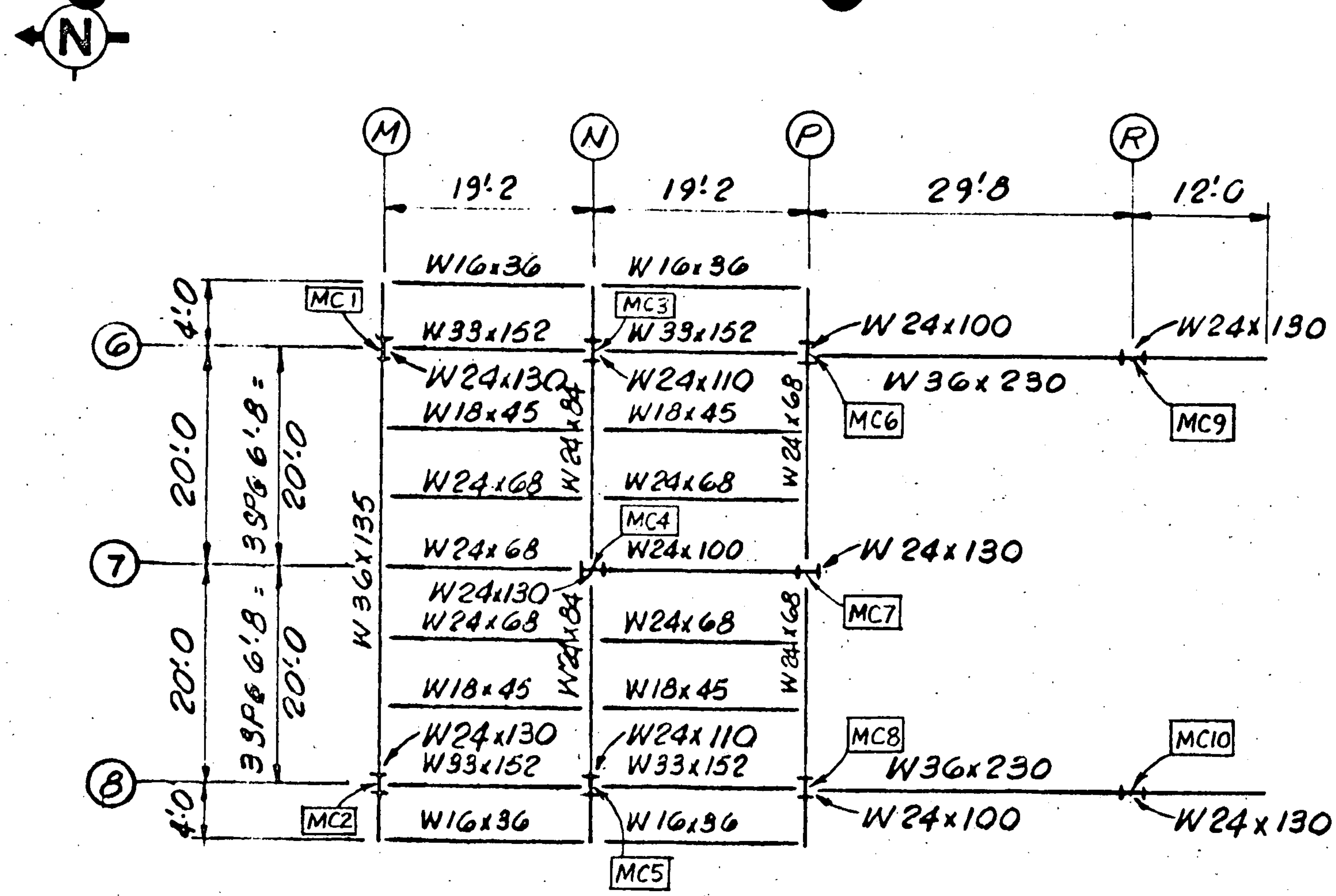
SOUTH EXTENSION

GIRDERS & BEAMS EVALUATION

PLAN @ EL 42'-0

FIGURE 42

REF. TABLE 33



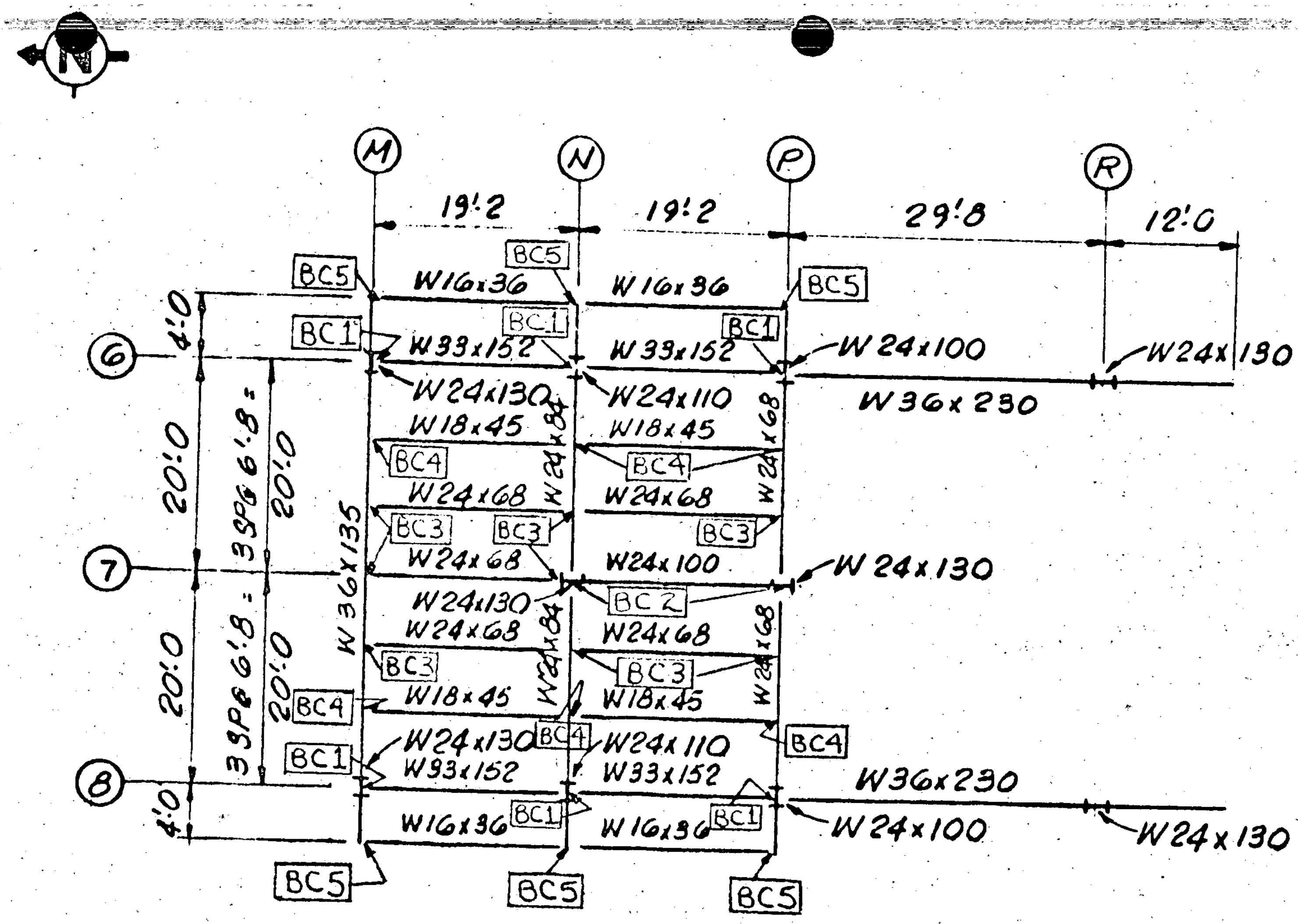
SOUTH EXTENSION

MOMENT CONNECTION EVALUATION

PLAN @ EL 42'-0

FIGURE 43

REF. TABLE 34



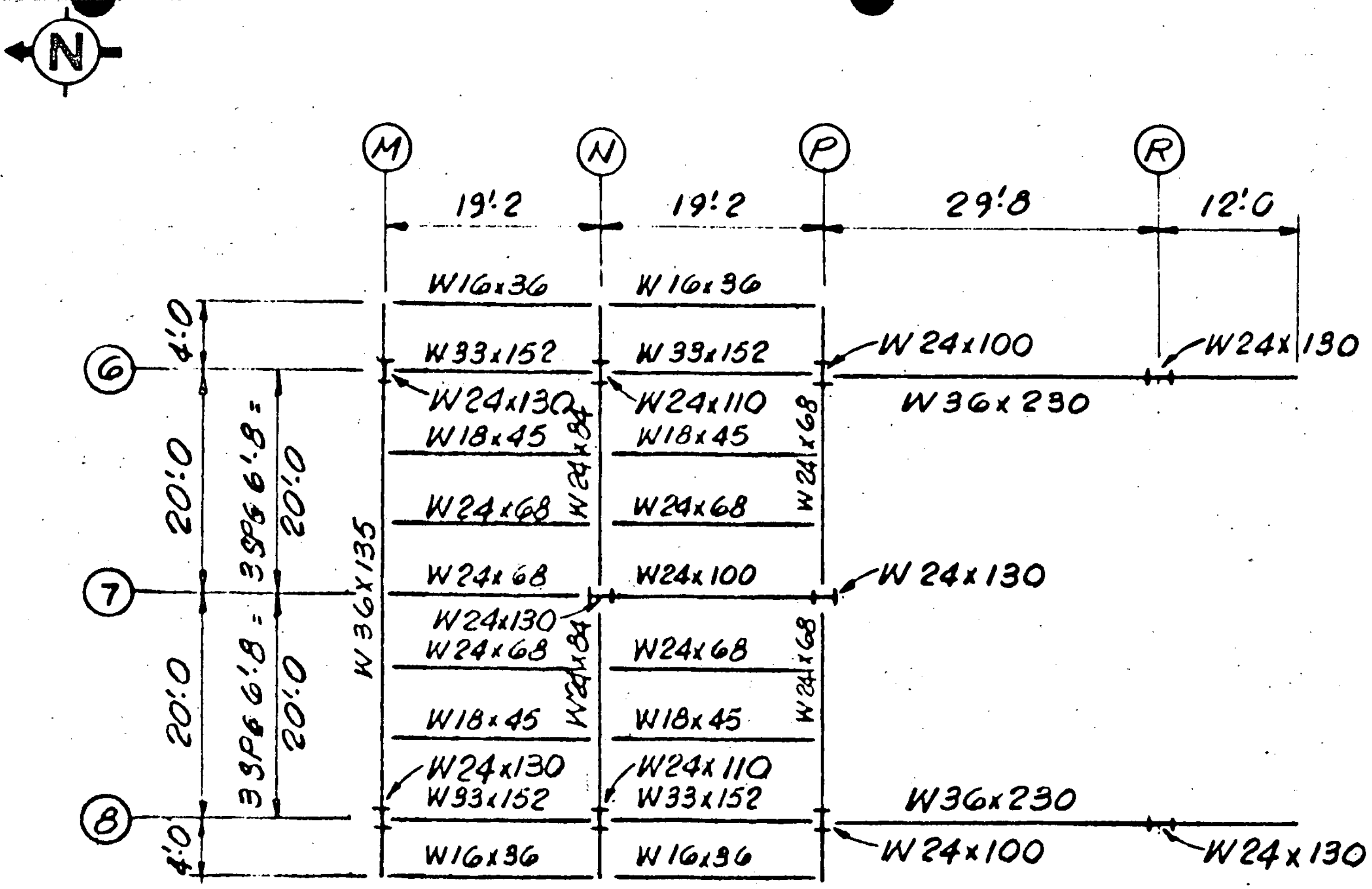
SOUTH EXTENSION

BOLTED CONNECTION EVALUATION

PLAN @ EL 42'-0

FIGURE 44

REF. TABLE 35



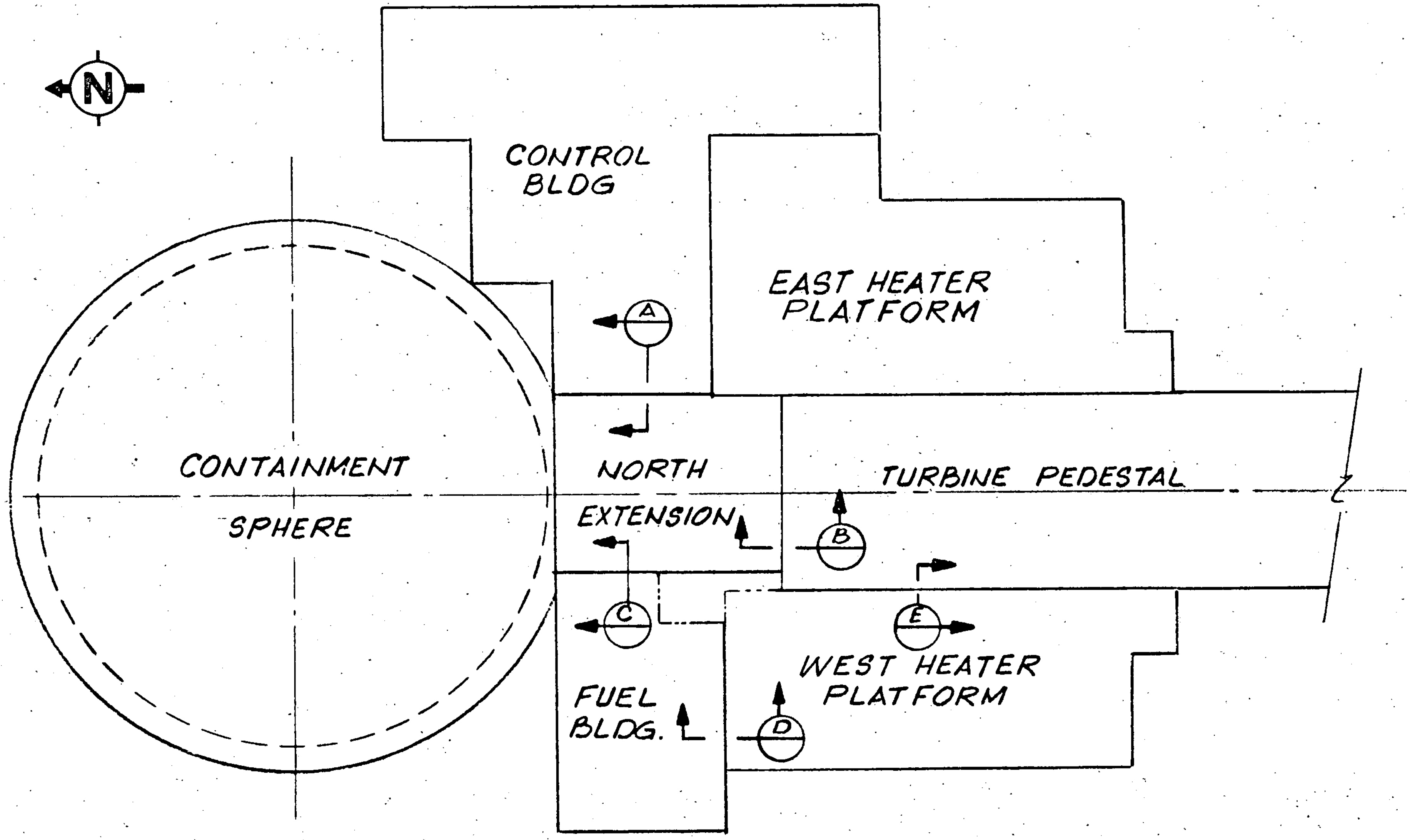
SOUTH EXTENSION

COLUMN ANCHORAGE EVALUATION

PLAN @ EL 42'-0

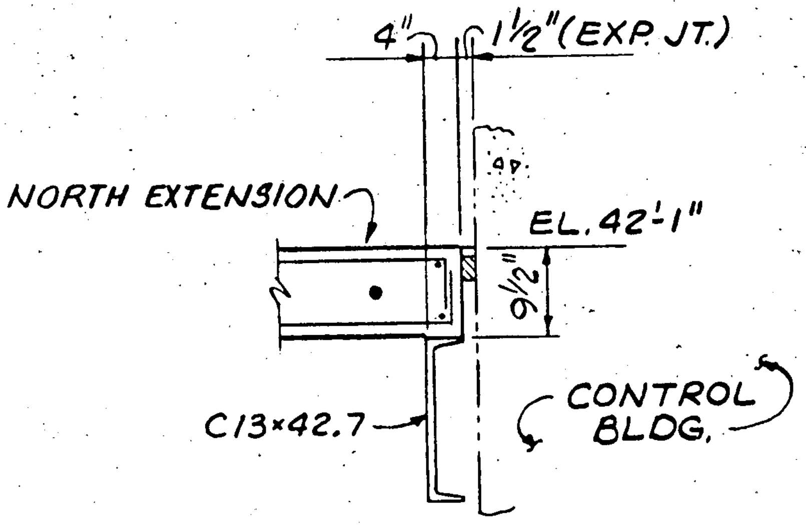
FIGURE 45

REF. TABLE 36 & 37

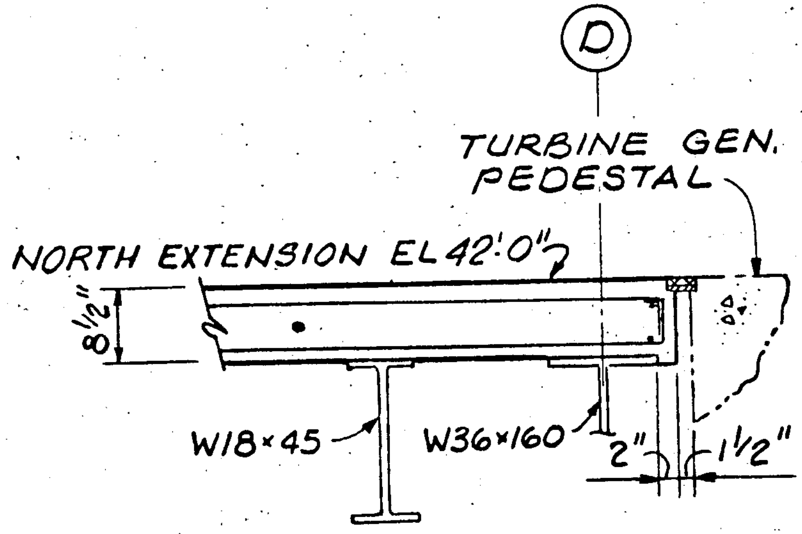


KEY PLAN

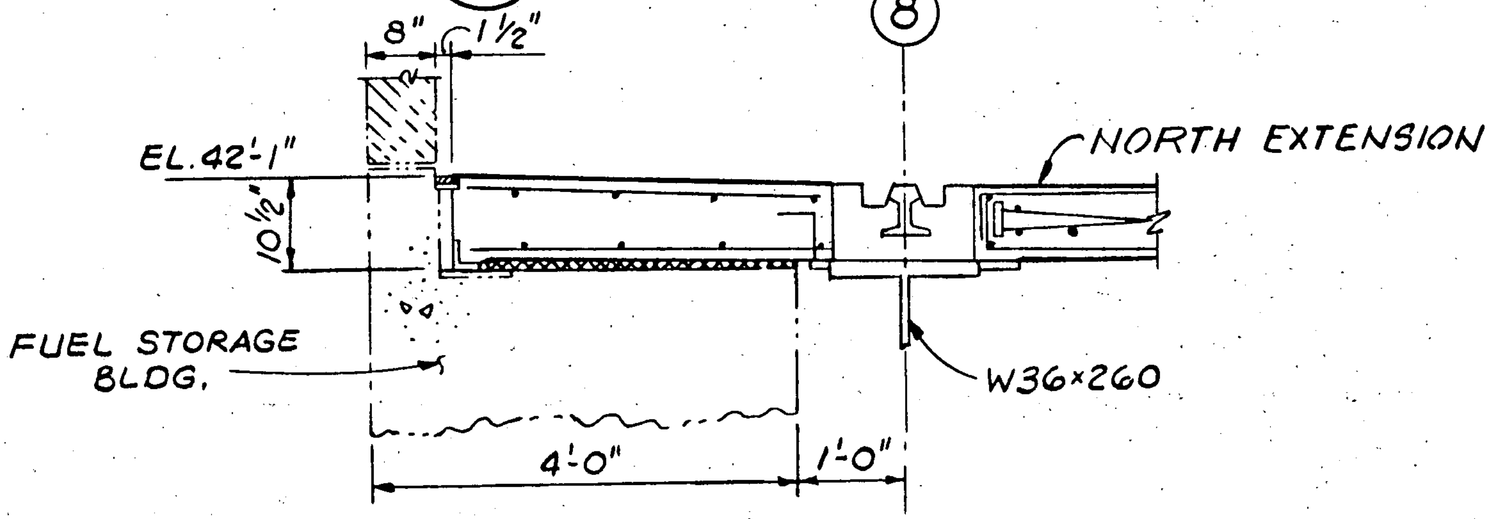
FIGURE 46



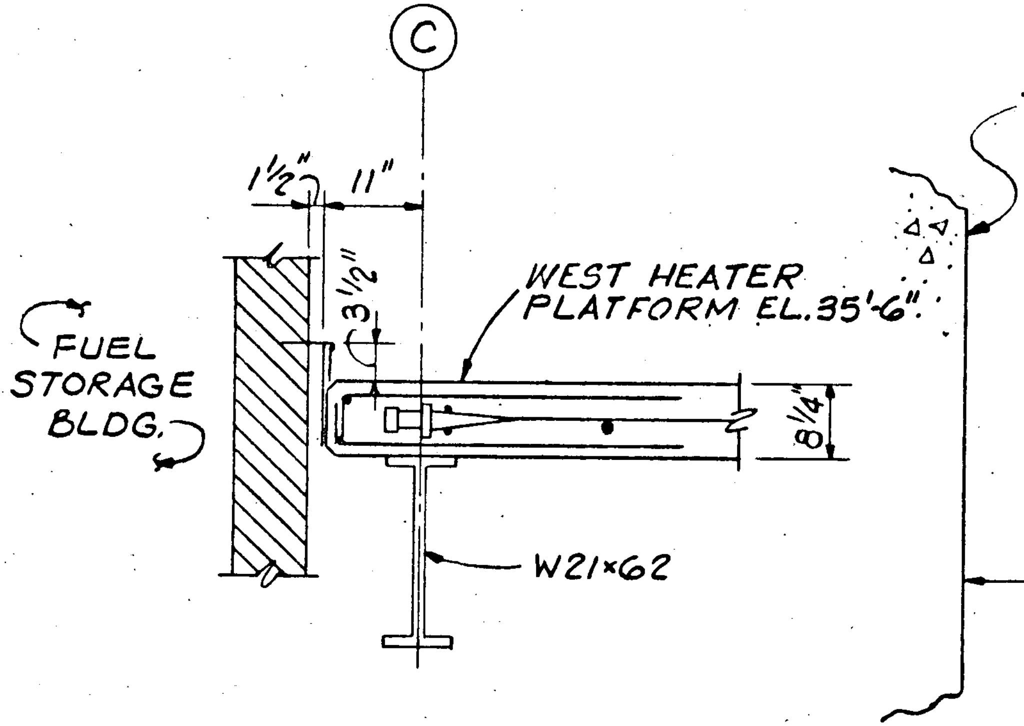
SECTION A
LOOKING NORTH



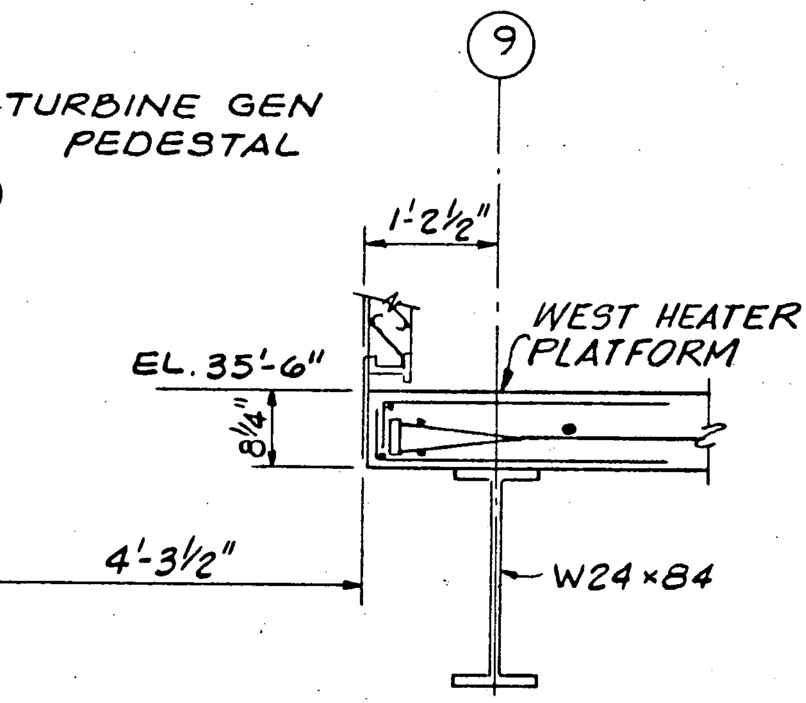
SECTION B
LOOKING EAST



SECTION C
LOOKING NORTH

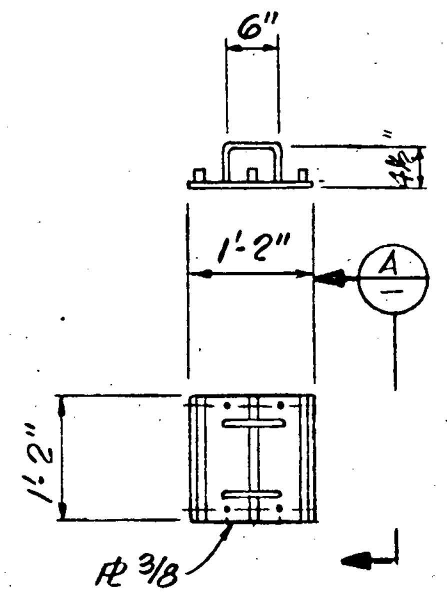


SECTION (D)
LOOKING EAST

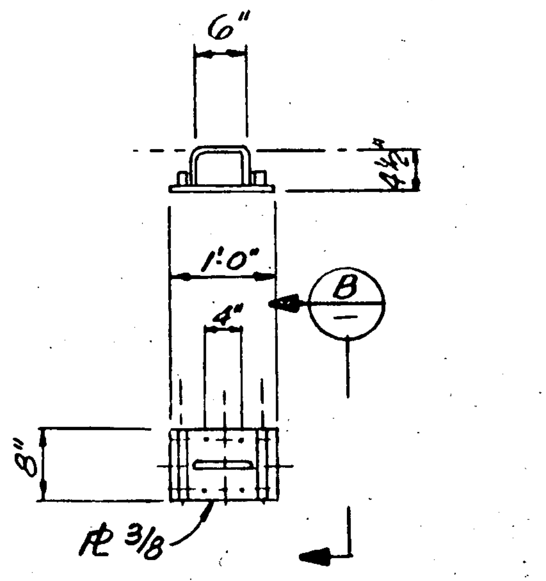


SECTION (E)
LOOKING SOUTH

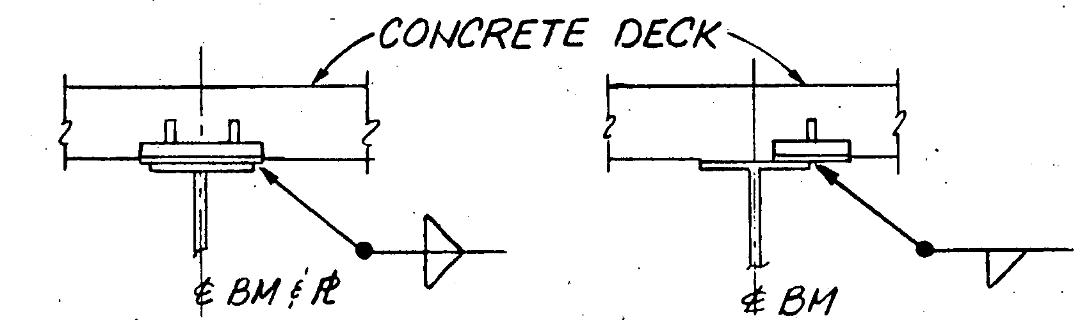
FIGURE 48



DETAIL 1



DETAIL 2



SECTION A

SECTION B

FIGURE 49

TABLE 1
MODAL FREQUENCIES & COMPOSITE MODAL DAMPING

NO.	DESCRIPTION	FREQUENCY Hz	COMPOSITE MODAL DAMPING	REMARKS
1	T/G PEDESTAL EAST-WEST (E-W) ROCKING	3.29	21.3%	
2	T/G PEDESTAL VERTICAL (VERT.) TRANSLATION	3.33	50.5%	
3	T/G PEDESTAL NORTH-SOUTH (N-S) TRANSLATION	3.63	28.7%	
4	NORTH EXTENSION E-W TRANSLATION	4.61	34.8%	
5	NORTH EXTENSION N-S TRANSLATION	4.74	17.2%	
6	EAST HEATER PLATFORM E-W TRANSLATION	4.60	37.9%	
7	EAST HEATER PLATFORM VERT. & N-S TRANSLATION	5.79	28.7%	
8	WEST HEATER PLATFORM N-S TRANSLATION W/E-W TWISTING	4.28	17.3%	
9	WEST HEATER PLATFORM VERT. & N-S TRANSLATION	5.58	23.1%	
10	SOUTH EXTENSION N-S TRANSLATION	8.27	16.0%	
11	SOUTH EXTENSION E-W TRANSLATION	8.89	11.4%	

TURBINE BLDG. COMPLEX MODAL ANALYSIS, GANTRY CRANE @ NORTH EXTENSION

TABLE 1 A
MODAL FREQUENCIES & COMPOSITE MODAL DAMPING

NO.	DESCRIPTION	FREQUENCY Hz	COMPOSITE MODAL DAMPING	REMARKS
1	T/G PEDESTAL EAST-WEST (E-W) ROCKING	3.29	21.4%	
2	T/G PEDESTAL VERTICAL (VERT.) TRANSLATION	3.36	50.8%	
3	T/G PEDESTAL NORTH-SOUTH (N-S) TRANSLATION	3.65	29.5%	
4	NORTH EXTENSION E-W TRANSLATION	4.61	36.2%	
5	NORTH EXTENSION N-S TRANSLATION	5.06	15.6%	
6	EAST HEATER PLATFORM E-W TRANSLATION	4.59	37.6%	
7	EAST HEATER PLATFORM N-S AND VERT. TRANSLATION	5.82	27.9%	
8	WEST HEATER PLATFORM N-S TRANSLATION W/E-W TWISTING	4.14	17.2%	
9	WEST HEATER PLATFORM VERT. & N-S TRANSLATION	5.48	23.2%	
10	SOUTH EXTENSION N-S TRANSLATION	8.26	18.5%	
11	SOUTH EXTENSION E-W TRANSLATION	8.90	7.1%	

TURBINE BUILDING COMPLEX MODAL ANALYSIS, GANTRY CRANE @ SOUTH EXTENSION

NORTH EXTENSION NEW BRACING STRESS EVALUATION

TABLE 2

NO.	DESCRIPTION	REF. FIGURE NO.	SIZE	fa	Fa	SF= Fa/fa	MEETS CRITERIA		REMARKS
							YES	NO	
BR1	B6-D6	22	W12×336 ℄ GIRDER	12.54	18.90	1.51	X		
BR2	A8-B8	22	W12×336 ℄ GIRDER	10.46	14.00	1.34	X		
BR3	B8-B7	22	W12×120	17.54	23.60	1.35	X		
BR4	B6-B7	22	W12×120	17.54	23.60	1.35	X		
BR5	B7-B8	22	W12×170	16.00	21.70	1.36	X		
BR6	B7-B6	22	W12×170	16.00	21.70	1.36	X		

NOTES:

1. fa = CALCULATED AXIAL STRESS, KSI
2. Fa = ALLOWABLE AXIAL STRESS, KSI
3. SF = SAFETY FACTOR FOR AXIAL COMPRESSION

NORTH EXTENSION - GIRDERS & BEAMS EVALUATION

TABLE 3

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR f_a	P_y OR F_a	M_x OR f_{bx}	M_{px} OR F_{bx}	M_y OR f_{by}	M_{py} OR F_{by}	(1) R_c, R_c^b	R_A	$SF = R_A/R_C$	MEETS CRITERIA YES	NO	REMARKS
NE-B1	23	W36X260		1.28	20.4	22.58	24.0	0.32	27.0	1.03	1.60	1.55	x		
NE-B2	23	W27X145		–	–	21.32	24.0	–	–	0.89	1.60	1.80	x		
NE-B3	23	W24X100		1.33	17.94	19.84	24.0	–	–	0.90	1.60	1.78	x		
NE-B4	23	W18X45		–	–	16.51	11.11	–	–	1.49	1.60	1.07	x		
NE-B5	23	W18X45		–	–	16.84	11.11	–	–	1.52	1.60	1.05	x		
NE-B6	23	W36X160		0.72	13.48	26.35	18.80	–	–	1.45	1.60	1.10	x		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. f_a, F_a, F_{bx}, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, P_y, M_{px} ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_c = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_c^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

NORTH EXTENSION MEZZANINE - GIRDERS & BEAMS EVALUATION

TABLE 4

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR fa	Py OR Fa	Mx OR fbx	Mpx OR Fbx	My OR fby	Mpy OR Fby	(1) Rc, Rc^b	RA	SF= RA/Rc	MEETS CRITERIA YES	NO	REMARKS
NEM-B1	24	W10X39		–	–	19.36	22.0	–	–	0.83	1.60	1.82	X		
NEM-B2	24	W8X15		–	–	5.41	12.0	–	–	0.45	1.60	3.56	X		
NEM-B3	24	W10X45		–	–	16.64	22.0	–	–	0.76	1.60	2.11	X		
NEM-B4	24	W10X39	A7.67-B7.67	–	–	45.8	24.0	–	–	1.91	1.60	0.84		X	SEE SECTION 5
NEM-B5	24	W10X60		–	–	9.81	24.0	–	–	0.41	1.60	3.90	X		
NEM-B6	24	W10X39		–	–	16.85	24.0	–	–	0.70	1.60	2.29	X		

NOTATION

1. Rc - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_C = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - F_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_C^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

NORTH EXTENSION - MOMENT CONN. EVALUATION

TABLE 5

IDENTIFICATION	REFERENCE FIGURE NO.	DESCRIPTION	f_f	F_f	SF_f F_f/f_f	V_{BWC}	V_{BWA}	SF_V V_{BWA}/V_{BWC}	A_{BWSC}	A_{BWSR}	SF_A A_{BWSC}/A_{BWSR}	MEETS CRITERIA YES	NO	REMARKS
MC1	25	W27x145 COLUMN W24x100 GIRDER	16.2	35.2	2.17	17.8	23.0	1.29	10.12	2.92	3.47	X		
MC2	25	W27x160 COLUMN W24x100 GIRDER	24.1	35.2	1.46	29.2	23.0	0.79	10.12	7.37	1.37		X	SEE SECTION 5
MC3	25	W24x160 COLUMN W24x100 GIRDER	16.5	35.2	2.13	21.0	23.0	1.10	10.12	4.11	2.46	X		
MC4	25	W33x200 COLUMN W36x260 GIRDER	15.7	35.2	2.24	7.81	23.0	2.94	17.14	6.16	2.78			
MC6	25	W33x200 COLUMN W36x260 GIRDER	15.7	35.2	2.24	7.81	23.0	2.94	17.14	6.16	2.78			
MC5	25	W27x145 COLUMN W27x145 GIRDER	6.49	35.2	5.42	5.04	23.0	4.56	13.37	11.6	1.15			
MC7	25	W30x210 COLUMN W36x160 GIRDER	17.5	35.2	2.01	10.0	23.0	2.30	14.98	12.61	1.19	X		
MC8	25	W30x210 COLUMN W36x160 GIRDER	17.5	35.2	2.01	10.0	23.0	2.30	14.98	12.61	1.19	X		

NOTATION

1. f_f - CALCULATED COLUMN FLANGE STRESS, KSI
2. F_f - ALLOWABLE COLUMN FLANGE STRESS, KSI
3. SF_f - SAFETY FACTOR FOR COLUMN FLANGE
4. V_{BWC} - CALCULATED BEAM WEB SHEAR STRESS, KSI
5. V_{BWA} - ALLOWABLE BEAM WEB SHEAR STRESS, KSI
6. SF_V - SAFETY FACTOR FOR BEAM WEB SHEAR
7. A_{BWSC} - CALCULATED AREA OF BEAM WEB STIFFENER, IN^2
8. A_{BWSR} - REQUIRED AREA OF BEAM WEB STIFFENER, IN^2
9. SF_A - SAFETY FACTOR FOR BEAM WEB STIFFENER

NORTH EXTENSION BOLTED CONNECTION EVALUATION

TABLE 6

IDENTIFICATION	REFERENCE FIGURE NO.	SIZE	V_C	V_A	SF_V V_A/V_C	MEETS CRITERIA YES	NO	REMARKS
BC1	26	7 ROWS	131.0	414.4	3.16	X		
BC2	26	7 ROWS	90.7	148.5	1.64	X		
BC3	26	4 ROWS	20.0	84.8	4.24	X		
BC4	26	2 ROWS	21.8	42.4	1.94	X		

NOTATION

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR FOR SHEAR

NORTH EXTENSION - COLUMN EVALUATION

TABLE 7

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR f_a	P_y OR F_a	M_x OR f_{bx}	M_{px} OR F_{bx}	M_y OR f_{by}	M_{py} OR F_{by}	(1) R_c, R_c^b	R_A	SF = R_A/R_c	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
A6	27	W27x145		102	1537	8322	16308	1018	3514	0.67[b]	1.0	1.49	X		
A7	27	W27x160		149	1696	8428	18036	1632	3924	0.73[b]	1.0	1.37	X		
A8	27	W24x160		513	1696	8063	16740	456	4140	0.78[b]	1.0	1.28	X		
B6	27	W33x200		894	2120	17576	27216	335	5292	1.007[b]	1.0	0.993	X		
B7	27	W27x145		8.70	17.61	17.56	22.0	4.26	27.0	1.40	1.60	1.14	X		
B8	27	W33x200		581	2120	13150	27216	1081	5292	0.81[b]	1.0	1.23	X		
D6	27	W30x210		348	2228	17129	26460	834	5580	0.79[b]	1.0	1.27	X		
D8	27	W30x210		573	2228	16493	26460	1484	5580	0.94[b]	1.0	1.06	X		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. Fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_c = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_c^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

TABLE 8

COLUMN ANCHORAGES — AREA 2

IDENT	REF FIGURE NO.	DESCRIPTION	SIZE	f_t	F_t	SF_t F_t/f_t	f_b	F_b	R_C[7]	R_A[8]	SF_b[9]	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
A-6	27	ANCHOR BOLTS	1¼"φ	45.5	101	2.22								
		BASE PL	2"				39.4	27.0	1.46	1.6	1.10	x		
A-7	27	ANCHOR BOLTS	1½"φ	92.2	101	1.10						x		
		BASE PL	2"				39.5[4]	36.0[5]	—	—	0.91[6]	x		
A-8	27	ANCHOR BOLTS	1¼"φ	66.4	101	1.52						x		
		BASE PL	2"				12.71	27.0	0.47	1.6	3.40	x		
B-6	27	ANCHOR BOLTS	1¾"φ	97.0	101	1.04						x		
		BASE PL	2½"				36.0	27.0	1.33	1.6	1.20	x		
B-7	27	ANCHOR BOLTS	1¼"φ	5.3	101	19.06						x		
		BASE PL	2"				18.3	27.0	0.68	1.6	2.35	x		
B-8	27	ANCHOR BOLTS	1¾"φ	101	101	1.00						x		
		BASE PL	2½"				4.93	27.0	0.18	1.6	8.76	x		SEE SECTION 5
D-6	27	ANCHOR BOLTS	1¾"φ	124.3	101	0.81							x	
		BASE PL	3"				54.1	27.0	2.00	1.6	0.80		x	SEE SECTION 5
D-8	27	ANCHOR BOLTS	1¾"φ	80.3	101	1.26						x		
		BASE PL	3"				42.1	27.0	1.56	1.6	1.03	x		

NOTATION

1. SF_t - SAFETY FACTOR FOR TENSILE LOAD
2. SF_b - SAFETY FACTOR FOR BENDING
3. f_t, F_t, f_b, F_b - SEE AISC STEEL CONSTRUCTION MANUAL 1980, KSI
4. COMPUTED BENDING MOMENT K-IN
5. ALLOWABLE PLAST. MOM = $F_y \times Z$
6. SF_b = M PLASTIC / MC
7. $R_C = f_b / F_b$
8. $R_A = 1.6$
9. $SF_b = R_A / R_C$
10. SHEAR CONE CAPACITY OF BASE PL W/O ANC BOLTS (COL. WELDED TO BASE PL)

WEST EXTENSION NEW BRACING STRESS EVALUATION

TABLE 9

NO.	DESCRIPTION	REF. FIGURE NO.	SIZE	fa	Fa	SF= Fa/fa	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BR1	F13-E13	28	W12×120	5.92	16.10	2.72	X		
BR2	E13-F13	28	W12×120	5.92	16.10	2.72	X		
BR3	G-13	28	W12×79	6.31	23.20	3.68	X		
BR4	H13-J13	28	W12×120	8.54	22.00	2.58	X		
BR5	H13-G13	28	W12×120	8.54	22.00	2.58	X		
BR6	J12-K12	28	W12×120	9.46	21.90	2.32	X		
BR7	C13-C11	28	W12×170	7.00	16.80	2.40	X		
BR8	H9-H12	29	W12×120	6.54	18.10	2.77	X		
BR9	G9-G12	29	W12×170	6.00	18.50	3.08	X		
BR10	K13-K12	29	W12×120	9.08	17.79	1.96	X		
BR11	K13	29	W12×79	9.46	23.20	2.45	X		
BR12	C9	29	W8×58	7.3	12.2	1.67	X		

NOTES:

1. fa = CALCULATED AXIAL STRESS, KSI
2. Fa = ALLOWABLE AXIAL STRESS, KSI
3. SF = SAFETY FACTOR FOR AXIAL COMPRESSION

WEST EXTENSION NEW BRACING STRESS EVALUATION

TABLE 10

NO.	DESCRIPTION	REF. FIGURE NO.	SIZE	fa	Fa	SF = Fa/fa	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BR13	L9	29	W12×79	7.15	17.70	2.48	X		
BR14	H13	29	W12×79	6.31	23.20	3.68	X		

NOTES:

1. fa = CALCULATED AXIAL STRESS, KSI
2. Fa = ALLOWABLE AXIAL STRESS, KSI
3. SF = SAFETY FACTOR FOR AXIAL COMPRESSION

WEST HEATER PLATF. - GIRDERS & BEAMS EVALUATION — TABLE 11

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR f_a	P_Y OR F_a	M_x OR f_{bx}	M_{px} OR F_{bx}	M_Y OR f_{by}	M_{py} OR F_{by}	(1) R_C, R_C^b	R_A	SF = R_A/R_C	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
WHP-B1	30	W14X30		–	–	12.44	15.10	–	–	0.82	1.60	1.95	x		
WHP-B2	30	W14X30		1.29	8.90	13.08	11.58	–	–	1.27	1.60	1.26	x		
WHP-B3	30	W24X84		0.47	5.22	11.10	10.51	–	–	1.15	1.60	1.39	x		
WHP-B4	30	W24X68		–	–	3318.0	5702.4	–	–	0.49^b	1.00	2.04	x		
WHP-B5	30	W24X110		0.14	19.45	18.41	15.47	–	–	1.20	1.60	1.33	x		
WHP-B6	30	W24X84		–	–	4933.2	7257.6	–	–	0.58^b	1.00	1.72	x		
WHP-B7	30	W24X100		–	–	22.18	14.09	–	–	1.57	1.60	1.02	x		
WHP-B8	30	W24X76		8.6	725.8	4516.1	6512.4	–	–	0.60^b	1.00	1.67	x		
WHP-B9	30	W24X84		18.9	800.3	4950.1	7257.6	–	–	0.60^b	1.00	1.67	x		
WHP-B10	30	W24X84		0.68	11.15	18.20	15.62	–	–	1.23	1.60	1.30	x		
WHP-B11	30	W18X45		0.30	15.05	15.42	11.28	–	–	1.39	1.60	1.15	x		
WHP-B12	30	W24x76		–	–	14.90	13.86	–	–	1.08	1.60	1.48	X		
WHP-B13	30	W24x68		0.385	10.60	14.24	11.88	–	–	1.24	1.60	1.29	X		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. — " — KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_C = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_C^b = \frac{P}{P_Y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_Y}{M_{py}}$$

WEST HEATER PLATF.-GIRDERS & BEAMS EVALUATION — TABLE 12

IDEN'T	REF FIGURE NO.	SIZE	DESCRIPTION	P OR f_a	P_y OR F_a	M_x OR f_{bx}	M_{px} OR F_{bx}	M_y OR f_{by}	M_{py} OR F_{by}	(1) R_c, R_c^b	R_A	SF = R_A/R_C	MEETS CRITERIA YES	NO	REMARKS
WHP-B14	30	W24×84		1.39	11.88	8.81	16.40	–	–	0.65	1.60	2.46	X		
WHP-B15	30	W12×27		–	–	8.81	20.87	–	–	0.42	1.60	3.81	X		
WHP-B16	30	W16×36		3.8	343.4	1574.2	2358.7	–	–	0.58^b	1.00	1.72	X		
WHP-B17	30	W16×40		0.19	8.12	16.31	12.50	–	–	1.32	1.60	1.21	X		
WHP-B18	30	W16×45		–	–	9.19	15.28	–	–	0.65	1.60	2.46	X		
WHP-B19	30	W16×36		0.14	9.26	15.98	11.73	–	–	1.38	1.60	1.16	X		
WHP-B20	30	W16×36	J12 - K12	2.4	343.4	2601.1	2073.6	–	–	1.07^b	1.00	0.93		X	SEE SECTION 5
WHP-B21	30	W21×62	C11-C13	42.6	592.9	5258.4	4665.6	–	–	1.03^b	1.00	0.97		X	SEE SECTION 5
WHP-B22	30	W21×62		0.55	17.70	18.81	24.0	–	–	0.81	1.60	1.98	X		
WHP-B23	30	W24×120		22.1	1146.9	10,274.8	10,951.2	17.8	2264.8	0.82^b	1.00	1.22	X		
WHP-B24	30	W24×120		0.21	19.47	28.76	24.0	–	–	1.21	1.60	1.32	X		
WHP-B25	30	W24×130		0.13	20.02	32.16	24.0	0.12	27.0	1.35	1.60	1.19	X		
WHP-B26	30	W24×84		1.60	18.00	28.58	24.0	0.05	27.0	1.28	1.60	1.25	X		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. — " — KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_c = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1-f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1-F_a/F_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_c^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

WEST HEATER PLATF. - MOMENT CONN. EVALUATION

TABLE 13

IDENTIFICATION	REFERENCE FIGURE NO.	DESCRIPTION	f_f	F_f	SF_f F_f/f_f	V_{BWC}	V_{BWA}	SF_V V_{BWA}/V_{BWC}	A_{BWSC}	A_{BWSR}	SF_A A_{BWSC}/A_{BWSR}	MEETS CRITERIA YES	NO	REMARKS
MC1	31	W24x110 COLUMN W24x120 GIRDER	16.40	35.2	2.15	13.2	23.0	1.74	9.86	1.10	8.96	X		
MC2	31	W24x130 COLUMN W24x120 GIRDER	22.10		1.59	18.7		1.23	10.38	3.61	2.88			
MC8	31	W24x130 COLUMN W24x120 GIRDER	22.10		1.59	18.7		1.23	10.38	3.61	2.88			
MC14	31	W24x130 COLUMN W24x120 GIRDER	22.10		1.59	18.7		1.23	10.38	3.61	2.88			
MC3	31	W24x110 COLUMN W24x84 GIRDER	17.90		1.97	13.2		1.74	7.30	2.80	2.61			
MC4	31	W24x130 COLUMN W24x84 GIRDER	12.90		2.73	15.1		1.52	7.69	1.93	3.98			
MC7	31	W24x130 COLUMN W24x84 GIRDER	12.90		2.73	15.1		1.52	7.69	1.93	3.98			
MC13	31	W24x130 COLUMN W24x84 GIRDER	12.90		2.73	15.1		1.52	7.69	1.93	3.98			
MC5	31	W24x130 COLUMN W24x130 GIRDER	16.75		2.10	16.2		1.42	12.00	2.52	4.76			
MC11	31	W24x130 COLUMN W24x130 GIRDER	16.75		2.10	16.2		1.42	12.00	2.52	4.76			
MC6	31	W24x130 COLUMN W24x84 GIRDER	15.30		2.30	13.6		1.69	7.60	3.10	2.45			
MC9	31	W24x100 COLUMN W24x84 GIRDER	9.95		3.54	6.46		3.56		—				
MC10	31	W24x110 COLUMN W24x130 GIRDER	15.56		2.26	12.31		1.87	10.50	2.41	4.36			
MC12	31	W24x145 COLUMN W24x84 GIRDER	8.14	35.2	4.32	6.95	23.0	3.31		—		X		

NOTATION

1. f_f - CALCULATED COLUMN FLANGE STRESS, KSI
2. F_f - ALLOWABLE COLUMN FLANGE STRESS, KSI
3. SF_f - SAFETY FACTOR FOR COLUMN FLANGE
4. V_{BWC} - CALCULATED BEAM WEB SHEAR STRESS, KSI
5. V_{BWA} - ALLOWABLE BEAM WEB SHEAR STRESS, KSI
6. SF_V - SAFETY FACTOR FOR BEAM WEB SHEAR
7. A_{BWSC} - CALCULATED AREA OF BEAM WEB STIFFENER, IN^2
8. A_{BWSR} - REQUIRED AREA OF BEAM WEB STIFFENER, IN^2
9. SF_A - SAFETY FACTOR FOR BEAM WEB STIFFENER

WEST HEATER PLATF. - MOMENT CONN. EVALUATION

TABLE 14

IDENTIFICATION	REFERENCE FIGURE NO.	DESCRIPTION	f_f	F_f	SF_f F_f/f_f	V_{BWC}	V_{BWA}	SF_V V_{BWA}/V_{BWC}	A_{BWSC}	A_{BWSR}	SF_A A_{BWSC}/A_{BWSR}	MEETS CRITERIA		REMARKS
												YES	NO	
MC15	31	W24x100 COLUMN W24x84 GIRDER	9.42	35.2	3.74	6.75	23.0	3.41		—		X		
MC16	31	W24x110 COLUMN W24x84 GIRDER	9.42	35.2	3.74	6.75	23.0	3.41		—		X		
MC17	31	W24x110 COLUMN W24X84 GIRDER	10.50	35.2	3.35	7.55	23.0	3.05		—		X		

NOTATION

1. f_f - CALCULATED COLUMN FLANGE STRESS, KSI
2. F_f - ALLOWABLE COLUMN FLANGE STRESS, KSI
3. SF_f - SAFETY FACTOR FOR COLUMN FLANGE
4. V_{BWC} - CALCULATED BEAM WEB SHEAR STRESS, KSI
5. V_{BWA} - ALLOWABLE BEAM WEB SHEAR STRESS, KSI
6. SF_V - SAFETY FACTOR FOR BEAM WEB SHEAR
7. A_{BWSC} - CALCULATED AREA OF BEAM WEB STIFFENER, IN^2
8. A_{BWSR} - REQUIRED AREA OF BEAM WEB STIFFENER, IN^2
9. SF_A - SAFETY FACTOR FOR BEAM WEB STIFFENER

WEST EXTENSION BOLTED CONNECTION EVALUATION — TABLE 15

IDENTIFICATION	REFERENCE FIGURE NO.	SIZE	V_C	V_A	SF_V V_A/V_C	MEETS CRITERIA		REMARKS
						YES	NO	
BC1	32	6 ROWS	126.6	127.2	1.00	X		
BC2	32	6 ROWS	92.0	127.2	1.38	X		
BC3	32	5 ROWS	55.2	106.0	1.92	X		
BC4	32	3 ROWS	36.7	63.7	1.74	X		

NOTATION

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR FOR SHEAR

WEST HEATER PLATF - COLUMN EVALUATION — TABLE 16

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR fa	Py OR Fa	Mx OR fbx	Mpx OR Fbx	My OR fby	Mpy OR Fby	(1) Rc, Rc^b	RA	SF = RA/Rc	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
C-9	33	W8x31		95	328	103	1094	23	504	0.40 b	1.0	2.50	X		
C-11	33	W8x31		72	328	911	1094	26	504	0.96 b	1.0	1.04	X		
C-13	33	W8X31		214	328	2159	1094	15	504	2.34 b	1.0	0.43		X	SEE SECTION 5
E-9	33	W24x110		2.81	15.21	16.15	22.0	11.13	27.0	1.27	1.60	1.26	X		
E-11	33	W24x130		380	1379	5827	13320	773	3247	0.79 b	1.0	1.27	X		
E-13	33	W24x110		209	1170	2935	11124	976	2290	0.66 b	1.0	1.52	X		
F-9	33	W24x130		3.03	14.34	11.70	21.4	10.10	27.0	1.06	1.60	1.51	X		
F-12	33	W24x130		418	1379	5117	13320	662	3247	0.75 b	1.0	1.33	X		
F-13	33	W24x130		237	1379	3542	13320	1292	3247	0.64 b	1.0	1.56	X		
G-9	33	W24x130		6.40	14.33	9.04	21.43	10.54	27.0	1.32	1.60	1.21	X		
G-12	33	W24x130		7.40	16.55	13.73	22.0	7.53	27.0	1.31	1.60	1.22	X		
G-13	33	W24x100		5.50	15.13	16.26	20.62	9.67	27.0	1.45	1.60	1.10	X		
H-9	33	W24x110		6.90	12.29	13.85	17.51	5.07	27.0	1.59	1.60	1.01	X		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_C = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_C^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

WEST HEATER PLATF.-COLUMN EVALUATION

TABLE 17

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR f_a	P_y OR F_a	M_x OR f_{bx}	M_{px} OR F_{bx}	M_y OR f_{by}	M_{py} OR F_{by}	(1) R_c, R_c^b	R_A	$SF = R_A/R_c$	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
H-12	33	W24x130		7.50	16.55	11.98	22.0	7.25	27.0	1.24	1.60	1.29	X		
H-13	33	W24x145		2.50	16.64	4.91	22.0	9.63	27.0	0.73	1.60	2.19	X		
J-9	33	W24x130		3.04	14.34	6.16	21.45	11.52	27.0	0.89	1.60	1.80	X		
J-12	33	W24x130		10.60	16.55	11.06	22.0	6.61	27.0	1.40	1.60	1.14	X		
J-13	33	W24x100		2.80	15.13	13.40	20.62	6.24	27.0	1.01	1.60	1.58	X		
K-9	33	W24x110		1.71	12.30	7.89	17.50	6.61	27.0	0.83	1.60	1.93	X		
K-12	33	W24x110		3.40	15.20	9.11	22.0	5.33	27.0	0.78	1.60	2.05	X		
K-13	33	W8x31		4.20	11.37	17.05	22.0	—	—	1.13	1.60	1.42	X		
L-9	33	W8x31		10.10	15.69	1.65	22.0	6.47	27.0	1.06	1.60	1.51	X		
L-10	33	W8x31		3.19	15.90	1.66	22.0	3.78	27.0	0.36	1.60	4.44	X		

NOTATION

1. R_C - COMBINED STRESS FACTOR
2. R_A - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. f_a, F_a, F_{bx}, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, P_y, M_{px} ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_c = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_c^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

COLUMN ANCHORAGES — AREA 6

IDENT	REF FIGURE NO.	DESCRIPTION	SIZE	f_t	F_t	SF_t F_t/f_t	f_b	F_b	R_C [7]	R_A [8]	SF_b [9]	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
C-9	33	ANCHOR BOLTS	3/4"φ	35 [10]	92.6 [10]	1.09						X		SEE SECTION 5
		BASE PL	3/4"				29.7	27.0	1.10	1.6	1.46	X		
C-11	33	ANCHOR BOLTS	1"φ	85.0	101	1.19						X		SEE SECTION 5
		BASE PL	3/4"				14.1	24.0	0.59	1.6	2.72	X		
C-13	33	ANCHOR BOLT	3/4"φ	64.9	88.3	1.39						X		SEE SECTION 5
		BASE PL	3/4"				14.1	27.0	0.52	1.6	3.06	X		
E-9	33	ANCHOR BOLTS	1 3/4"φ	31.1	101	3.25						X		
		BASE PL	2 1/4"				35.9	27.0	1.33	1.6	1.20	X		
E-11	33	ANCHOR BOLTS	1 3/4"φ	31.6	101	3.20						X		
		BASE PL	2 1/2"				26.3	27.0	.97	1.6	1.65	X		
E-13	33	ANCHOR BOLTS	1 3/4"φ	26.9	101	3.76						X		
		BASE PL	2 1/4"				27.3	27.0	1.01	1.6	1.58	X		
F-9	33	ANCHOR BOLTS	1 3/4"φ	13.65	101	7.40						X		
		BASE PL	2 1/4"				30.1	27.0	1.12	1.6	1.43	X		
F-12	33	ANCHOR BOLTS	1 3/4"φ	24.2	101	4.17						X		
		BASE PL	2 1/2"				27.6	27.0	1.02	1.6	1.57	X		
F-13	33	ANCHOR BOLTS	1 3/4"φ	14.3	101	7.06						X		
		BASE PL	2 1/4"				10.97	27.0	0.41	1.6	3.90	X		
G-9	33	ANCHOR BOLTS	1 3/4"φ	34.5	101	2.93						X		
		BASE PL	2 1/4"				24.8	27.0	0.92	1.6	1.74	X		
G-12	33	ANCHOR BOLTS	1 3/4"φ	29.3	101	3.45						X		
		BASE PL	2 1/2"				25.4	27.0	0.94	1.6	1.70	X		
G-13	33	ANCHOR BOLTS	1"φ	0.0	32.0	—						X		
		BASE PL	1"				37.2	27.0	1.38	1.6	1.16	X		

NOTATION

1. SF_t - SAFETY FACTOR FOR TENSILE LOAD
2. SF_b - SAFETY FACTOR FOR BENDING
3. f_t, F_t, f_b, F_b - SEE AISC STEEL CONSTRUCTION MANUAL 1980, KSI
4. COMPUTED BENDING MOMENT K-IN
5. ALLOWABLE PLAST. MOM = $F_y \times Z$
6. SF_b = M PLASTIC / MC
7. $R_C = f_b / F_b$
8. $R_A = 1.6$
9. $SF_b = R_A / R_C$
10. SHEAR CONE CAPACITY OF BASE PL W/O ANC BOLTS (COL. WELDED TO BASE PL)

WEST HEATER PLATFORM- COLUMN ANCHORAGE EVALUATION — TABLE 19

COLUMN ANCHORAGES – AREA 6

IDENT	REF FIGURE NO.	DESCRIPTION	SIZE	f_t	F_t	SF_t F_t/f_t	f_b	F_b	R_c[7]	R_A[8]	SF_b[9]	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
H-9	33	ANCHOR BOLTS	1¾"Φ	19.0	101	5.32						X		
		BASE PL	2¼"				14.53	27.0	0.54	1.6	2.98	X		
H-12	33	ANCHOR BOLTS	1¾"Φ	20.5	101	4.93						X		
		BASE PL	2½"				23.2	27.0	0.86	1.6	1.86	X		
H-13	33	ANCHOR BOLTS	2"Φ	0.88	101	114.8						X		
		BASE PL	2½"				0.46	27.0	0.02	1.6	80.0	X		
J-9	33	ANCHOR BOLTS	1¼"Φ	19.92	101	5.07						X		
		BASE PL	2¼"				22.3	27.0	0.83	1.6	1.93	X		
J-12	33	ANCHOR BOLTS	1¼"Φ	13.3	101	7.59						X		
		BASE PL	2½"				21.9	27.0	0.81	1.6	1.98	X		
J-13	33	ANCHOR BOLTS	1"Φ	237[10]	462[10]	1.95						X		
		BASE PL	1"				9.6	27.0	0.36	1.6	4.44	X		
K-9	33	ANCHOR BOLTS	1¾"Φ	7.9	101	12.79						X		
		BASE PL	2¼"				16.2	27.0	0.60	1.6	2.67	X		
K-12	33	ANCHOR BOLTS	1¾"Φ	83	101	1.22						X		
		BASE PL	2¼"				38.3[4]	45.6[5]	—	—	1.19[6]	X		
K-13	33	ANCHOR BOLTS	¾"Φ	13.3	32.0	2.41						X		
		BASE PL	¾"				0.61	27.0	0.02	1.6	80.0	X		
L-9	33	ANCHOR BOLTS	¾"Φ	0.0	32.0	—						X		
		BASE PL	¾"				6.17	27.0	0.23	1.6	7.00	X		
L-10	33	ANCHOR BOLTS	¾"Φ	7.7	32.0	4.17						X		
		BASE PL	¾"				2.71	27.0	0.10	1.6	15.9	X		

NOTATION

1. SF_t - SAFETY FACTOR FOR TENSILE LOAD
2. SF_b - SAFETY FACTOR FOR BENDING
3. f_t, F_t, f_b, F_b - SEE AISC STEEL CONSTRUCTION MANUAL 1980, KSI
4. COMPUTED BENDING MOMENT K-IN
5. ALLOWABLE PLAST. MOM = $F_y \times Z$
6. SF_b = M PLASTIC / MC
7. $R_c = f_b / F_b$
8. $R_A = 1.6$
9. $SF_b = R_A / R_c$
10. SHEAR CONE CAPACITY OF BASE PL W/O ANC BOLTS (COL. WELDED TO BASE PL)

EAST EXTENSION NEW BRACING STRESS EVALUATION

TABLE 20

NO.	DESCRIPTION	REF. FIGURE NO.	SIZE	fa	Fa	SF= Fa/fa	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BR1	F2 - F1	34	W12 X 120	6.27	16.30	2.60	X		
BR2	F2 - F1	34	W12 X 120	7.58	16.30	2.15	X		
BR3	J2 - K2	34	W12 X 79	11.04	20.72	1.88	X		
BR4	J2 - J1.5	34	W12 X 79	15.15	22.46	1.48	X		
BR5	J1.5 - J1	34	W12 X 79	16.72	22.46	1.34	X		
BR6	F2 - G2	34	W12 X 79	8.03	19.42	2.42	X		
BR7	F2 - G2	34	W12 X 79	8.93	19.42	2.18	X		
BR8	G2 - H2	34	W12 X 79	12.34	19.94	1.62	X		
BR9	H1 - H2	35	W12 X 120	9.17	16.30	1.78	X		
BR10	E3 - E1	35	W12 X 79	8.16	14.35	1.76	X		
BR11	H1 - J1	35	W12 X 79	14.53	19.94	1.37	X		
BR12	J1 - K1	35	W12 X 79	11.97	20.72	1.73	X		

NOTES:

1. fa = CALCULATED AXIAL STRESS, KSI
2. Fa = ALLOWABLE AXIAL STRESS, KSI
3. SF = SAFETY FACTOR FOR AXIAL COMPRESSION

EAST EXTENSION NEW BRACING STRESS EVALUATION TABLE 21

NO.	DESCRIPTION	REF. FIGURE NO.	SIZE	fa	Fa	SF = Fa/fa	MEETS CRITERIA		REMARKS
							YES	NO	
BR13	L5	35	W12X79	8.19	12.54	1.53	X		
BR14	E1 - F1	35	W12X170	9.68	16.45	1.70	X		
BR15	F1 - G1	35	W12X79	14.72	79.42	1.32	X		

NOTES:

1. fa = CALCULATED AXIAL STRESS, KSI
2. Fa = ALLOWABLE AXIAL STRESS, KSI
3. SF = SAFETY FACTOR FOR AXIAL COMPRESSION

EAST HEATER PLATF.- GIRDERS & BEAMS EVALUATION TABLE 22

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR fa	Py OR Fa	Mx OR fbx	Mpx OR Fbx	My OR fby	Mpy OR Fby	(1) Rc, R_c^b	RA	SF= RA/Rc	MEETS CRITERIA YES	NO	REMARKS
EHP-B1	36	W21X55	E1 - F1	4.6	524.9	8812.9	4082.4	–	–	1.84^b	1.00	0.54		x	SEE SECTION 5
EHP-B2	36	W24X76		12.8	726	5782	6512	–	–	0.77^b	1.00	1.30	x		
EHP-B3	36	W24X100		0.28	17.82	22.42	22.0	–	–	1.05	1.60	1.52	x		
EHP-B4	36	W24X84		0.23	15.49	23.91	22.0	–	–	1.12	1.60	1.43	x		
EHP-B5	36	W24X110		0.24	16.65	20.10	22.0	–	–	0.94	1.60	1.70	x		
EHP-B6	36	W24X68		2.8	648	3630.4	5702.4	–	–	0.54^b	1.00	1.85	x		
EHP-B7	36	W24X84		0.34	5.22	14.66	10.51	–	–	1.46	1.60	1.10	x		
EHP-B8	36	W24X84		1.47	11.15	15.75	15.62	–	–	1.14	1.60	1.40	x		
EHP-B9	36	W24X68		0.43	15.56	16.00	22.0	–	–	0.77	1.60	2.08	x		
EHP-B10	36	W24X76		0.50	15.77	24.41	21.04	–	–	1.19	1.60	1.34	x		

NOTATION

1. RC- COMBINED STRESS FACTOR
2. RA- ALLOWABLE COMBINED STRESS FACTOR
3. SF- OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC.- SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_c = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_c^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

EAST HEATER PLATE - GIRDERS & BEAMS EVALUATION

TABLE 23

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR fa	Py OR Fa	Mx OR fbx	Mpx OR Fbx	My OR fby	Mpy OR Fby	(1) Rc, R_c^b	RA	SF = RA/Rc	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
EHP-B11	36	W18X45		0.28	12.02	15.47	14.40	–	–	1.10	1.60	1.45	x		
EHP-B12	36	W16X40		0.36	8.12	14.67	12.50	–	–	1.22	1.60	1.31	x		
EHP-B13	36	W16X36	G2-H2	4.0	343.4	2537.1	2073.6	–	–	1.05^b	1.00	0.95		x	SEE SECTION 5
EHP-B14	36	W16X36		2.7	343	1399.7	2074	–	–	0.58^b	1.00	1.72	x		
EHP-B15	36	W12X27		0.31	14.00	7.51	18.70	–	–	0.43	1.60	3.72	x		
EHP-B16	36	W14X30		3.3	286.1	1551.0	1529.3	–	–	0.87^b	1.00	1.15	x		
EHP-B17	36	W16X36		0.60	9.26	13.41	11.73	–	–	1.21	1.60	1.32	x		
EHP-B18	36	W16X36		1.2	343.4	2023.9	2073.6	–	–	0.83^b	1.00	1.19	x		
EHP-B19	36	W16X45		–	–	7.34	22.0	–	–	0.34	1.60	4.71	x		
EHP-B20	36	W14X30		3.20	11.85	8.13	13.79	–	–	0.78	1.60	2.05	x		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_c = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - F_a/F_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_c^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

EAST HEATER PLATF.- GIRDERS & BEAMS EVALUATION

TABLE 24

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR f_a	P_y OR F_a	M_x OR f_{bx}	M_{px} OR F_{bx}	M_y OR f_{by}	M_{py} OR F_{by}	(1) R_C, R_C^b	R_A	SF = R_A/R_C	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
EHP-B21	36	W14X30		—	—	8.39	13.79	—	—	0.61	1.60	2.62	x		
EHP-B22	36	W24X120		0.28	19.42	20.91	24.0	—	—	0.89	1.60	1.80	x		
EHP-B23	36	W24X120		31.0	1147.0	12,297.3	10,951	—	—	0.98^b	1.00	1.02	x		
EHP-B24	36	W24X130		0.92	19.62	22.98	24.0	0.57	27.0	1.07	1.60	1.50	x		
EHP-B25	36	W24X130		14.7	1240.9	13,564.8	11,988	—	—	0.97^b	1.00	1.03	x		
EHP-B26	36	W24X120		1.08	19.37	21.15	24.0	—	—	0.94	1.60	1.70	x		
EHP-B27	36	W24X84		1.21	18.00	20.94	24.0	—	—	0.94	1.60	1.70	x		
EHP-B28	36	W16X40		0.90	18.78	14.84	24.0	—	—	0.67	1.60	2.39	x		

NOTATION

1. R_C - COMBINED STRESS FACTOR
2. R_A - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. f_a, F_a, F_{bx}, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, P_y, M_{px} ETC. — " — KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_C = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_C^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

EAST HEATER PLATF. - MOMENT CONN. EVALUATION

TABLE 25

IDENTIFICATION	REFERENCE FIGURE NO.	DESCRIPTION	f_f	F_f	SF_f F_f/f_f	V_{BWC}	V_{BWA}	SF_V V_{BWA}/V_{BWC}	A_{BWSC}	A_{BWSR}	SF_A A_{BWSC}/A_{BWSR}	MEETS CRITERIA YES	NO	REMARKS
MC1	37	W24x130 COLUMN W24x120 GIRDER	16.52	35.2	2.13	14.95	23.0	1.54	7.57	1.46	5.18	X		
MC7	37	W24x130 COLUMN W24x120 GIRDER	16.52	35.2	2.13	14.95	23.0	1.54	7.57	1.46	5.18			
MC13	37	W24x130 COLUMN W24x120 GIRDER	16.52	35.2	2.13	14.95	23.0	1.54	7.57	1.46	5.18			
MC2	37	W24x110 COLUMN W24x120 GIRDER	15.19		2.32	12.18		1.89	7.53	0.63	11.95			
MC3	37	W24x130 COLUMN W24x84 GIRDER	17.33		2.03	5.17		4.45	11.86	6.73	1.76			
MC4	37	W24x130 COLUMN W24x130 GIRDER	17.50		2.01	16.94		1.36	9.00	2.89	3.11			
MC10	37	W24x130 COLUMN W24x130 GIRDER	17.50		2.01	16.94		1.36	9.00	2.89	3.11			
MC6	37	W24x100 COLUMN W24x84 GIRDER	19.51		1.80	12.67		1.82	5.36	1.01	5.31			
MC9	37	W24x145 COLUMN W24x84 GIRDER	17.80		1.98	15.22		1.51	7.14	1.91	3.74			
MC5	37	W24x130 COLUMN W24x84 GIRDER	19.77		1.78	14.91		1.54	5.36	1.85	2.90			
MC8	37	W24x130 COLUMN W24x84 GIRDER	19.77		1.78	14.91		1.54	5.36	1.85	2.90			
MC14	37	W24x130 COLUMN W24x84 GIRDER	19.77		1.78	14.91		1.54	5.36	1.85	2.90			
MC11	37	W24x110 COLUMN W24x130 GIRDER	14.61		2.41	11.55		1.99	7.53	0.32	23.53			
MC12	37	W24x100 COLUMN W24x84 GIRDER	5.27	35.2	6.68	3.42	23.0	6.73	5.36	0.00	—	X		

NOTATION

1. f_f - CALCULATED COLUMN FLANGE STRESS, KSI
2. F_f - ALLOWABLE COLUMN FLANGE STRESS, KSI
3. SF_f - SAFETY FACTOR FOR COLUMN FLANGE
4. V_{BWC} - CALCULATED BEAM WEB SHEAR STRESS, KSI
5. V_{BWA} - ALLOWABLE BEAM WEB SHEAR STRESS, KSI
6. SF_V - SAFETY FACTOR FOR BEAM WEB SHEAR
7. A_{BWSC} - CALCULATED AREA OF BEAM WEB STIFFENER, IN^2
8. A_{BWSR} - REQUIRED AREA OF BEAM WEB STIFFENER, IN^2
9. SF_A - SAFETY FACTOR FOR BEAM WEB STIFFENER

EAST HEATER PLATF. - MOMENT CONN. EVALUATION

TABLE 26

IDENTIFICATION	REFERENCE FIGURE NO.	DESCRIPTION	f_f	F_f	SF_f F_f/f_f	V_{BWC}	V_{BWA}	SF_V V_{BWA}/V_{BWC}	A_{BWSC}	A_{BWSR}	SF_A A_{BWSC}/A_{BWSR}	MEETS CRITERIA YES	NO	REMARKS
MC15	37	W24x110 COLUMN W24X84 GIRDER	15.72	35.2	2.24	11.27	23.0	2.04	5.36	0.42	12.76	X		
MC16	37	W24x110 COLUMN W24X84 GIRDER	7.30	35.2	4.82	5.23	23.0	4.40	5.36	0.00	—	X		

NOTATION

1. f_f - CALCULATED COLUMN FLANGE STRESS, KSI
2. F_f - ALLOWABLE COLUMN FLANGE STRESS, KSI
3. SF_f - SAFETY FACTOR FOR COLUMN FLANGE
4. V_{BWC} - CALCULATED BEAM WEB SHEAR STRESS, KSI
5. V_{BWA} - ALLOWABLE BEAM WEB SHEAR STRESS, KSI
6. SF_V - SAFETY FACTOR FOR BEAM WEB SHEAR
7. A_{BWSC} - CALCULATED AREA OF BEAM WEB STIFFENER, IN^2
8. A_{BWSR} - REQUIRED AREA OF BEAM WEB STIFFENER, IN^2
9. SF_A - SAFETY FACTOR FOR BEAM WEB STIFFENER

EAST EXTENSION BOLTED CONNECTION EVALUATION — TABLE 27

IDENTIFICATION	REFERENCE FIGURE NO.	SIZE	V_C	V_A	SF_V V_A/V_C	MEETS CRITERIA YES	NO	REMARKS
BC1	38	6 ROWS	100.0	127.2	1.27	X		
BC2	38	6 ROWS	114.6	127.2	1.11	X		
BC3	38	5 ROWS	98.2	106.8	1.09	X		
BC4	38	4 ROWS	79.4	84.8	1.07	X		
BC5	38	3 ROWS	16.0	63.7	3.98	X		

NOTATION
1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR FOR SHEAR

EAST HEATER PLATF.-COLUMN EVALUATION — TABLE 28

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR fa	Py OR Fa	Mx OR fbx	Mpx OR Fbx	My OR fby	Mpy OR Fby	(1) Rc, Rc^b	RA	SF= RA/Rc	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
E-1	39	W8x31		236	616	2108	1649	31	1728	1.48[b]	1.0	0.68		X	SEE SECTION 5
E-3	39	W24x130		277	1379	5224	13320	1275	3247	0.77[b]	1.0	1.30	X		
E-5	39	W24x110		2.61	14.71	16.15	22.0	12.45	27.0	1.33	1.60	1.20	X		
F-1	39	W24x130		474	1379	3603	13320	1316	3247	0.82[b]	1.0	1.22	X		
F-2	39	W24x130		404	1379	4840	13320	2234	3247	1.013[b]	1.0	0.987		X	SEE SECTION 5
F-5	39	W24x130		4.05	13.90	11.72	20.59	13.32	27.0	1.33	1.60	1.20	X		
G-1	39	W24x100		287	1062	3272	10080	436	2059	0.67[b]	1.0	1.49	X		
G-2	39	W24x130		332	1379	2970	13320	2050	3247	0.81[b]	1.0	1.23	X		
G-5	39	W24x130		4.05	13.90	11.72	20.59	13.32	27.0	1.33	1.60	1.20	X		
H-1	39	W24x145		6.53	16.28	11.26	22.0	7.26	27.0	1.15	1.60	1.39	X		
H-2	39	W24x130		370	1379	3444	13320	1889	3247	0.84[b]	1.0	1.19	X		
H-5	39	W24x110		3.65	11.74	13.16	16.83	3.69	27.0	1.16	1.60	1.38	X		
J-1	39	W24x100		251	1062	4823	10080	314	2059	0.73[b]	1.0	1.37	X		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, ksi
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_C = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_C^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

EAST HEATER PLATF. - COLUMN EVALUATION — TABLE 29

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR fa	Py OR Fa	Mx OR fbx	Mpx OR Fbx	My OR fby	Mpy OR Fby	(1) Rc, R_c^b	RA	SF= RA/Rc	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
J-2	39	W24X130		355	1379	7600	13320	338	3247	0.80[b]	1.0	1.25	X		
J-5	39	W24x130		4.05	13.90	11.72	20.59	13.32	27.0	1.33	1.60	1.20	X		
K-1	39	W8X31		3.82	10.55	3.88	22.0	3.32	27.0	0.70	1.60	2.29	X		
K-2	39	W24x110		3.03	13.48	14.05	19.20	5.58	27.0	1.08	1.60	1.48	X		
K-5	39	W24X110		1.91	11.73	6.07	16.83	10.20	27.0	0.85	1.60	1.88	X		
L-4	39	W8X31		3.07	6.38	3.42	16.89	5.29	27.0	1.01	1.60	1.58	X		
L-5	39	W8X31		124	328	75	1094	46	504	0.49[b]	1.0	2.04	X		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_c = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_c^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

COLUMN ANCHORAGES – AREA 5

IDENT	REF FIGURE NO.	DESCRIPTION	SIZE	f_t	F_t	SF_t F_t/f_t	f_b	F_b	R_C [7]	R_A [8]	SF_b [9]	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
E-1	39	ANCHOR BOLTS	3/4"Φ	316	101	0.32							x	SEE SECTION 5
		BASE PL	3/4"				123	27	4.56	1.6	.35		x	
E-3	39	ANCHOR BOLTS	1 3/4"Φ	406	101	0.25							x	SEE SECTION 5
		BASE PL	2 1/2"				109	27	4.04	1.6	.44		x	
E-5	39	ANCHOR BOLTS	1 3/4"Φ	34.1	101	2.96						x		
		BASE PL	2 1/4"				42.2	27	1.56	1.6	1.03	x		
F-1	39	ANCHOR BOLTS	1 3/4"Φ	41.5	101	2.43						x		
		BASE PL	2 1/2"				18.1	27	.67	1.6	2.39	x		
F-2	39	ANCHOR BOLTS	1 3/4"Φ	75.6	101	1.34						x		
		BASE PL	2 1/2"				45.6[4]	56.3[5]	—	—	1.23	x		
F-5	39	ANCHOR BOLTS	1 3/4"Φ	27.7	101	3.65						x		
		BASE PL	2 1/4"				29.0	27	1.07	1.6	1.50	x		
G-1	39	ANCHOR BOLTS	1"Φ	20.7	32	1.55						x		
		BASE PL	1"				35.2	27	1.30	1.6	1.23	x		
G-2	39	ANCHOR BOLTS	1 3/4"Φ	20.1	101	5.02						x		
		BASE PL	2 1/2"				18.2	27	.67	1.6	2.39	x		
G-5	39	ANCHOR BOLTS	1 3/4"Φ	16.9	101	5.98						x		
		BASE PL	2 1/4"				17.8	27	.66	1.6	2.42	x		
H-1	39	ANCHOR BOLTS	2"Φ	35.1	101	2.88						x		
		BASE PL	2 1/2"				39.8	27	1.47	1.6	1.09	x		
H-2	39	ANCHOR BOLTS	1 3/4"Φ	19.8	101	5.10						x		
		BASE PL	2 1/2"				21.1	27	.78	1.6	2.05	x		
H-5	39	ANCHOR BOLTS	1 3/4"Φ	17.6	101	5.74						x		
		BASE PL	2 1/2"				22.2	27	.82	1.6	1.95	x		

NOTATION

1. SF_t - SAFETY FACTOR FOR TENSILE LOAD
2. SF_b - SAFETY FACTOR FOR BENDING
3. f_t, F_t, f_b, F_b - SEE AISC STEEL CONSTRUCTION MANUAL 1980, KSI
4. COMPUTED BENDING MOMENT K-IN
5. ALLOWABLE PLAST. MOM = $F_y \times Z$
6. SF_b = M PLASTIC / MC
7. $R_C = f_b / F_b$
8. $R_A = 1.6$
9. $SF_b = R_A / R_C$
10. SHEAR CONE CAPACITY OF BASE PL W/O ANC BOLTS (COL. WELDED TO BASE PL)

EAST HEATER PLATE - COLUMN ANCHORAGE EVALUATION

TABLE 31

COLUMN ANCHORAGES - AREA 5

IDENT	REF FIGURE NO.	DESCRIPTION	SIZE	f_t	F_t	SF_t F_t/f_t	f_b	F_b	R_C[7]	R_A[8]	SF_b[9]	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
J-1	39	ANCHOR BOLTS	1"Φ	444[10]	462[10]	1.04						X		
		BASE PL	1"				17.0	27.0	.63	1.6	2.54	X		
J-2	39	ANCHOR BOLTS	1¾"Φ	120	101	0.84							X	SEE SECTION 5
		BASE PL	2½"				78.4	27.0	2.90	1.6	0.55		X	
J-5	39	ANCHOR BOLTS	1¾"Φ	6.0	101	16.83						X		
		BASE PL	2¼"				17.7	27.0	.66	1.6	2.42	X		
K-1	39	ANCHOR BOLTS	¾"Φ	155	32.0	0.21							X	SEE SECTION 5
		BASE PL	¾"				15.6	27.0	.58	1.6	2.77	X		
K-2	39	ANCHOR BOLTS	1¾"Φ	60.2	101	1.68						X		
		BASE PL	2¼"				44.0[4]	45.6[5]	—	—	1.04	X		
K-5	39	ANCHOR BOLTS	1¾"Φ	6.1	101	15.56						X		
		BASE PL	2¼"				12.3	27.0	.46	1.6	3.48	X		
L-4	39	ANCHOR BOLTS	¾"Φ	1.8	32.0	17.78						X		
		BASE PL	¾"				5.3	27.0	.20	1.6	108	X		
L-5	39	ANCHOR BOLTS	¾"Φ	79.2	32.0	.40							X	SEE SECTION 5
		BASE PL	¾"				8.2	27.0	.30	1.6	5.33	X		

NOTATION

1. SFt - SAFETY FACTOR FOR TENSILE LOAD
2. SFb - SAFETY FACTOR FOR BENDING
3. ft, Ft, fb, Fb - SEE AISC STEEL CONSTRUCTION MANUAL 1980, KSI
4. COMPUTED BENDING MOMENT K-IN
5. ALLOWABLE PLAST. MOM = $F_y \times Z$
6. SF_b = M PLASTIC / MC
7. $R_C = f_b / F_b$
8. $R_A = 1.6$
9. $SF_b = R_A / R_C$
10. SHEAR CONE CAPACITY OF BASE PL W/O ANC BOLTS (COL. WELDED TO BASE PL)

SOUTH EXTENSION NEW BRACING STRESS EVALUATION

TABLE 32

NO.	DESCRIPTION	REF. FIGURE NO.	SIZE	fa	Fa	SF= Fa/fa	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BR1	M6 - N6	41	W12X120	7.54	20.32	2.69	X		
BR2	N6 - P6	41	W12X79	9.53	19.78	2.08	X		
BR3	P8 - N8	41	W12X79	10.56	19.78	1.87	X		
BR4	N8 - M8	41	W12X120	8.19	20.32	2.48	X		
BR5	P6 - P7	41	W12X120	8.50	19.98	2.35	X		
BR6	M8 - M7	41	W12X79	9.44	19.06	2.02	X		
BR7	M7 - M6	41	W12X79	9.53	19.06	2.00	X		
BR8	P6	41	W12X120	5.38	22.30	4.14	X		

NOTES:

1. fa = CALCULATED AXIAL STRESS, KSI
2. Fa = ALLOWABLE AXIAL STRESS, KSI
3. SF = SAFETY FACTOR FOR AXIAL COMPRESSION

SOUTH EXTENSION - GIRDERS & BEAMS EVALUATION

TABLE 33

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR fa	Py OR Fa	Mx OR fbx	Mpx OR Fbx	My OR fby	Mpy OR Fby	(1) Rc, Rc[b]	RA	SF = RA/Rc	MEETS CRITERIA YES	NO	REMARKS
SE-B1	42	W16×36		–	–	5.08	9.84	–	–	0.52	1.60	3.08	X		
SE-B2	42	W33×152		0.80	13.83	13.48	18.97	–	–	0.77	1.60	2.08	X		
SE-B3	42	W18×45		0.26	7.41	10.04	10.89	–	–	0.96	1.60	1.67	X		
SE-B4	42	W24×68		0.19	9.85	7.53	11.47	–	–	0.68	1.60	2.35	X		
SE-B5	42	W24×100		–	–	10.86	20.22	–	–	0.54	1.60	2.96	X		
SE-B6	42	W18×45		0.23	7.41	10.43	10.89	–	–	0.99	1.60	1.62	X		
SE-B7	42	W16×36 + WT12×50		0.39	13.09	1.65	5.63	–	–	0.32	1.60	5.00	X		
SE-B8	42	W36×230		1.73	19.71	18.30	22.0	7.40	27.0	1.21	1.60	1.32	X		
SE-B9	42	W36×135		1.59	19.69	10.81	24.0	–	–	0.53	1.60	3.02	X		
SE-B10	42	W24×84		–	–	14.86	24.0	–	–	0.62	1.60	2.58	X		
SE-B11	42	W24×68		11.7	648	5572.3	5702.4	–	–	0.85[b]	1.00	1.18	X		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, KSI
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_c = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_c^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

SOUTH EXTENSION - MOMENT CONN. EVALUATION

TABLE 34

IDENTIFICATION	REFERENCE FIGURE NO.	DESCRIPTION	f_f	F_f	SF_f F_f/f_f	V_{BWC}	V_{BWA}	SF_V V_{BWA}/V_{BWC}	A_{BWSC}	A_{BWSR}	SF_A A_{BWSC}/A_{BWSR}	MEETS CRITERIA YES	NO	REMARKS
MC1	43	W24×130 COLUMN W36×135 GIRDER	8.45	35.2	4.17	5.28	23.0	4.36		—		×		
MC2	43	W24×130 COLUMN W36×135 GIRDER	8.45	35.2	4.17	5.28	23.0	4.36		—				
MC3	43	W24×110 COLUMN W24×84 GIRDER	8.83	35.2	3.99	8.45	23.0	2.72		—				
MC5	43	W24×110 COLUMN W24×84 GIRDER	8.83	35.2	3.99	8.45	23.0	2.72		—				
MC6	43	W24×100 COLUMN W24×68 GIRDER	6.89	35.2	5.11	6.92	23.0	3.32		—				
MC8	43	W24×100 COLUMN W24×68 GIRDER	6.89	35.2	5.11	6.92	23.0	3.32		—				
MC4	43	W24×130 COLUMN W24×100 GIRDER	11.90	35.2	2.96	10.4	23.0	2.21		—				
MC7	43	W24×130 COLUMN W24×100 GIRDER	11.90	35.2	2.96	10.4	23.0	2.21		—				
MC9	43	W24×130 COLUMN W36×230 GIRDER	14.0	35.2	2.51	6.81	23.0	3.38	10.5	1.06	9.91	×		
MC10	43	W24×130 COLUMN W36×230 GIRDER	14.0	35.2	2.51	6.81	23.0	3.38	10.5	1.06	9.91	×		

NOTATION

1. f_f - CALCULATED COLUMN FLANGE STRESS, KSI
2. F_f - ALLOWABLE COLUMN FLANGE STRESS, KSI
3. SF_f - SAFETY FACTOR FOR COLUMN FLANGE
4. V_{BWC} - CALCULATED BEAM WEB SHEAR STRESS, KSI
5. V_{BWA} - ALLOWABLE BEAM WEB SHEAR STRESS, KSI
6. SF_V - SAFETY FACTOR FOR BEAM WEB SHEAR
7. A_{BWSC} - CALCULATED AREA OF BEAM WEB STIFFENER, IN^2
8. A_{BWSR} - REQUIRED AREA OF BEAM WEB STIFFENER, IN^2
9. SF_A - SAFETY FACTOR FOR BEAM WEB STIFFENER

SOUTH EXTENSION BOLTED CONNECTION EVALUATION

TABLE 35

IDENTIFICATION	REFERENCE FIGURE NO.	SIZE	V_C	V_A	SF_V V_A/V_C	MEETS CRITERIA YES	NO	REMARKS
BC1	44	7 ROWS	263.3	414.4	1.57	X		
BC2	44	6 ROWS	56.3	212.8	3.78	X		
BC3	44	5 ROWS	36.1	106.0	2.94	X		
BC4	44	4 ROWS	17.1	84.8	4.96	X		
BC5	44	3 ROWS	6.4	36.7	5.73	X		

NOTATION

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR FOR SHEAR

SOUTH EXTENSION-COLUMN EVALUATION TABLE 36

IDEN'T	REF FIGURE NO	SIZE	DESCRIPTION	P OR fa	Py OR Fa	Mx OR fbx	Mpx OR Fbx	My OR fby	Mpy OR Fby	(1) Rc, Rc^b	RA	SF= RA/RC	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
M-6	45	W24x130		1.67	16.14	15.40	22.0	5.17	27.0	1.01	1.60	1.58	X		
M-8	45	W24x130		1.67	16.14	15.40	22.0	5.17	27.0	1.01	1.60	1.58	X		
N-6	45	W24x110		6.32	14.60	8.59	21.3	4.23	27.0	0.98	1.60	1.63	X		
N-7	45	W24x130		4.69	15.99	9.06	22.0	4.74	27.0	0.84	1.60	1.90	X		
N-8	45	W24x110		9.13	14.65	8.79	20.97	4.85	27.0	1.27	1.60	1.26	X		
P-6	45	W24x100		271	1062	3655	10080	193	2059	0.62^b	1.0	1.61	X		
P-7	45	W24x130		6.94	15.98	9.28	22.0	5.85	27.0	1.06	1.60	1.51	X		
P-8	45	W24x100		8.78	14.51	6.99	19.09	5.92	27.0	1.27	1.60	1.26	X		
R-6	45	W24x130		9.43	16.10	12.89	22.0	0.71	27.0	1.15	1.60	1.39	X		
R-8	45	W24x130		9.43	16.10	12.89	22.0	0.71	27.0	1.15	1.60	1.39	X		
R-6, R8 DIAGONALS	45	W14x74		238	785	702	4536	203	1462	0.52^b	1.0	1.92	X		

NOTATION

1. RC - COMBINED STRESS FACTOR
2. RA - ALLOWABLE COMBINED STRESS FACTOR
3. SF - OVERALL SAFETY FACTOR
4. fa, Fa, Fbx, ETC. - SEE AISC ST'L CONSTRUCTION MANUAL 1980, Ksi
   P, Py, Mpx ETC. ——— " ——— KIPS OR KIP-INCHES

FOOT NOTES (DESIGNATED BY SUPERSCRIPTS)

(1) INTERACTION EQS FOR STRUCTURAL STEEL:

$$R_C = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}; \quad R_C^b = \frac{P}{P_y} + \frac{1}{1.18}\frac{M_x}{M_{px}} + \frac{1}{1.67}\frac{M_y}{M_{py}}$$

COLUMN ANCHORAGES — AREA 7

IDENT	REF FIGURE NO.	DESCRIPTION	SIZE	f_t	F_t	SF_t F_t/f_t	f_b	F_b	R_C[7]	R_A[8]	SF_b[9]	MEETS CRITERIA YES	NO	REMARKS
M-6	45	ANCHOR BOLTS	1 1/8"φ	169	101	0.60							x	SEE SECTION 5
		BASE PL	2 1/4"				25.2	27.0	0.93	1.6	1.72	x		
M-8	45	ANCHOR BOLTS	1 1/8"φ	170	101	0.60							x	SEE SECTION 5
		BASE PL	2 1/4"				25.4	27.0	0.94	1.6	1.70	x		
N-6	45	ANCHOR BOLTS	1 3/8"φ	34.6	101	2.92						x		
		BASE PL	2 1/4"				27.3	27.0	1.01	1.6	1.58	x		
N-8	45	ANCHOR BOLTS	1 3/8"φ	9.0	101	11.22						x		
		BASE PL	2 1/4"				27.4	27.0	1.02	1.6	1.57	x		
P-6	45	ANCHOR BOLTS	1 1/4"φ	148	101	0.68							x	SEE SECTION 5
		BASE PL	2 1/4"				54.4	27.0	2.02	1.6	0.79		x	
P-8	45	ANCHOR BOLTS	1 1/4"φ	3.8	101	26.6						x		
		BASE PL	2 1/4"				24.1	27.0	0.89	1.6	1.80	x		
P-7	45	ANCHOR BOLTS	1 3/4"φ	39.3	101	2.57						x		
		BASE PL	2 1/2"				18.4	27.0	0.68	1.6	2.35	x		
N-7	45	ANCHOR BOLTS	1 3/4"φ	—	—	—						x		ADEQUATE BY COMPARISON TO P-7
		BASE PL	2 1/2"				—	—	—	—	—	x		
R-6 (COL)	45	ANCHOR BOLTS	1"φ	180	32	0.18							x	SEE SECTION 5
		BASE PL	1 3/4"				38.4	27.0	1.42	1.6	1.13	x		
R-8 (COL)	45	ANCHOR BOLTS	1"φ	197	32	0.16							x	SEE SECTION 5
		BASE PL	1 3/4"				39.4	27.0	1.46	1.6	1.10	x		

NOTATION

1. SF_t - SAFETY FACTOR FOR TENSILE LOAD
2. SF_b - SAFETY FACTOR FOR BENDING
3. f_t, F_t, f_b, F_b - SEE AISC STEEL CONSTRUCTION MANUAL 1980, KSI
4. COMPUTED BENDING MOMENT K-IN
5. ALLOWABLE PLAST. MOM = $F_y \times Z$
6. SF_b = M PLASTIC / MC
7. $R_C = f_b / F_b$
8. $R_A = 1.6$
9. $SF_b = R_A / R_C$
10. SHEAR CONE CAPACITY OF BASE PL W/O ANC BOLTS (COL. WELDED TO BASE PL)

TURBINE BUILDING COMPLEX DISPLACEMENTS

TABLE 38

LOCATION	DISPLACEMENTS DIRECTION	MAXIMUM DISPLACEMENTS (INCHES)	REMARKS
NORTH EXTENSION	NORTH-SOUTH	0.747"	
	EAST-WEST	0.729"	
EAST HEATER PLATFORM	NORTH-SOUTH	0.223"	
	EAST-WEST	0.285"	
WEST HEATER PLATFORM	NORTH-SOUTH	0.327"	
	EAST-WEST	0.321"	
SOUTH EXTENSION	NORTH-SOUTH	0.384"	
	EAST-WEST	0.232"	
T/G PEDESTAL	NORTH-SOUTH	0.549"	
	EAST-WEST	0.650"	

SRSS COMBINED DISPLACEMENT WITH ADJACENT STRUCTURES

TABLE 39

TURBINE BUILDING EXTENSIONS WITH ADJOINING STRUCTURES	SRSS COMBINED DISPLACEMENT FOR 0.67g HOUSNER (INCHES)	REMARKS
WEST HEATER PLATFORM WITH FUEL STORAGE BUILDING, NORTH - SOUTH	0.856"	
NORTH EXTENSION WITH TURBINE PEDESTAL, NORTH - SOUTH	0.927"	
NORTH EXTENSION WITH FUEL STORAGE BUILDING, EAST - WEST	0.799"	
NORTH EXTENSION WITH CONTROL BUILDING, EAST - WEST	0.393"	
SOUTH EXTENSION WITH TURBINE PEDESTAL, NORTH - SOUTH	0.589"	