Enclosure 1

BALANCE OF PLANT STRUCTURES
SEISMIC REEVALUATION PROGRAM

CONTROL AND ADMINISTRATION BUILDING

San Onofre Nuclear Generating Station
Unit 1

FOREWORD

This report describes the seismic reevaluation of the control and administration building 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 control and administration building are included in this report.

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1.0 INTRODUCTION

1.1 Background

Structures and equipment at San Onofre Unit 1 designated as seismic category A were originally designed to withstand a 0.5g Housner Design Basis Earthquake. The design work 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 anaysis 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.

1.2 Scope

The control and administration building has been reevaluated as part of the BOPSSR Program and the results are discussed in this report.

This structure 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.

1.3 Organization of the Report

This report has been divided into five sections. Section 1 describes the background associated with the BOPSSR Program and the scope of this report. Section 2 includes a description of the control and administration building. Section 3 discusses in detail the various analytical methods utilized in the reevaluation process.

Section 4 describes the results of the evaluation of the structure, including a comparison of the results with the provisions of the BOPSSR criteria. The computed stresses are compared with the criteria allowables and tabulations of these comparisons are included in Section 4. The conclusion of this evaluation is provided in Section 5.

2.0 DESCRIPTION OF STRUCTURE

A plot plan of San Onofre Unit 1 which shows the location of the control and administration building is provided in Figure 1. A description of the building is provided in the following section.

2.1 Control and Administration Building

The control and administration building is a three-story reinforced concrete structure with a single-story administration office building attached to the east side. The three-story portion of the building (housing the control room and cable spreading room) has only a partial second floor slab because structural steel framing (without a slab) is used to support electrical raceways at this floor level in the 4 kV switchgear room. The north and west walls of the control and administration building are 2 feet, 10 inches thick while the remainder of the structural walls vary from 8 inches to 13 inches in thickness. The building has overall plan dimensions of approximately 110 feet wide and 140 feet long and is approximately 36 feet high. The structure is slightly embedded with grade level varying from elevation (+) 14 feet, 0 inches to elevation (+) 19 feet, 9 inches.

The control and administration building foundation consists of reinforced concrete wall footings and individual column footings. The wall footings vary in width from 1 foot, 8 inches to 8 feet, 10 inches and their thickness varies from 1 to 2 feet. The column footings vary in width from 2 feet to 6 feet, 6 inches and their thickness varies from 1 foot to 1 foot, 6 inches.

Refer to Figures 2 through 4 for plans and sections of the control and administration building.

3.0 ANALYTICAL METHODS

The control and administration building was evaluated utilizing a three-dimensional finite element model which is described in Section 3.1. The model included: all interior slabs and roof slabs, exterior and interior reinforced concrete walls, masonry walls, concrete and steel columns, and foundation stiffness and damping. The analysis of the building was accomplished utilizing five linear elastic analysis methods: (1) the eigenvalue-eigenvector computation, (2) modal response spectrum analysis, (3) time history analysis, (4) equivalent static analysis, and (5) static analysis.

The eigenvalue-eigenvector computation was used to determine the modal frequencies, mode shapes, and the modal participation factors. 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 employed to develop instructure reponse spectra at various locations throughout the building.

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 at each foundation nodal point. The soil-structure interaction methodology utilized for the reevaluation is described in Reference 3.

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 directy by adding the computed responses at each time step. These analysis techniques are described in detail in Reference 2.

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 4.

Some structural elements that were determined to be capable of inelastic response, and for which inelastic deformation was acceptable, were evaluated by the inelastic spectrum method. This method is described in the BOPSSR Program criteria and in NUREG/CR-0098. The interrelationships between forces, yield points, displacements, and ductility ratios for the energy balance technique is shown in Figure 3.7-6 of the BOPSSR Criteria.

A detailed description of the finite element model and the analysis methods employed in the structural evaluation of the control and administration building is presented in the following paragraphs.

3.1 Modeling

A three-dimensional finite element model was developed for the control and administration building to analyze the dynamic behavior of this structure. The model was prepared utilizing the BSAP computer code. Figures 5 through 7 show the general characteristics of the finite element model. The model consists of 1291 nodal points; 1353 plate elements representing reinforced concrete slabs and walls; 304 beam elements representing both reinforced concrete and structural steel beams and columns; 103 beam elements representing the out-of-plane properties of the masonry walls; 44 membrane elements representing the in-plane properties of the masonry walls; 13 truss elements representing truss members in the mechanical room; and 543 boundary elements representing the stiffness of the soil media.

The reinforced concrete slabs and walls were modeled with quadrilateral and triangular plate elements. These walls and slabs typically have either principal reinforcement in one direction with mininum perpendicular reinforcement (one way span), or they have principal reinforcement in both perpendicular directions (two way plate action). Because of these conditions, orthotropic properties were used to model one way action and isotropic material properties were used to model two-way action. The input data utilized for the isotropic plate elements were:

modulus of elasticity, E = 3,820 ksi
poisson's ratio, = 0.20
mass density, = 0.004658 k-sec^2/ft^3

Two sets of values for the modulus of elasticity and poisson's ratio are required for the orthotropic elements, one for the strong direction and the other for the weak direction. The modulus of elasticity and poisson's ratio for orthotropic elements in the strong direction are the same as those for the isotropic elements. For the weak direction the following relationship was used in computing the modulus of elasticity and poisson's ratio:

$$\frac{E_x}{E_y} = \frac{\nu_x}{\nu_y} = \frac{I_{cr,x}}{I_{cr,y}}$$

where:

E_x; E_y = Modulus of elasticity for strong and weak direction respectively
ν_x; ν_y = Poisson's ratio for strong and weak axis respectively
$I_{cr,x}$; $I_{cr,y}$ = Cracked section moment of inertia for strong and weak direction respectively

The concrete compressive strength value used in the model included a 50% increase due to the effects of aging. An f'c of 4,500 psi was input to the model as compared to the original design specified 28 day minimum compressive strength of 3,000 psi. A detailed discussion concerning the applicability of this increase is presented in Section 4.

The control and administration building has three walls which are constructed of 8" thick reinforced hollow masonry block. The out-of-plane properties for these walls were modeled with beam elements while the in-plane properties were modeled with membrane elements. The masonry walls were modeled in this manner due to the large differences in stiffness properties associated with in-plane and out-of-plane responses. The wall model consisted of a grid of beam elements and membrane elements. The properties of the beam elements were defined such that the beams would resist all of the resulting out-of-plane forces and shear stresses. This was accomplished by making the in-plane beam properties (area, moment of inertia and polar moment of inertia) very small while using the equivalent cracked moment of inertia and shear area of the masonry wall for the out-of-plane beam properties. Since the modulus of elasticity is frequency dependent, three values were considered. The three values represent the minimum, average, and maximum values associated with masonry block. Those values are:

Em = 800 f'm
Em = 1000 f'm
Em = 1200 f'm

where: f'm = compressive strength of masonry block at 28 days.
Em = modulus of elasticity

The specific value of Em used for the masonry wall beam elements was 1200 f'm because this value resulted in beam frequencies which were closest to the peak of the design spectra. The values used for mass density and poisson's ratio for the beams were computed based on the equivalent solid thickness of the masonry blocks. The properties for the membrane elements, representing the in-plane characteristics of the masonry walls, were established such that the elements would resist the resulting in-plane and normal stresses due to the applied loads. The material properties assigned to the membrane elements were modulus of elasticity, mass density and poisson's ratio. These were incorporated into the analysis in the same way as that described for the beam elements.

The steel truss members in the building were modeled as truss elements with a pin joint at each end of the element. The material properties specified for these elements are modulus of elasticity and mass density. The reinforced concrete and structural steel beams and columns were modeled as beam elements with uniform properties. The material properties for these elements are modulus of elasticity, poisson's ratio and mass density.

3.2 Modal Analysis

The modal analysis of the control and administration building was performed utilizing the BSAP computer code. The details of the three-dimensional finite element model of the structure utilized for the modal analysis are presented in Section 3.1. A subspace iteration algorithm was used to calculate the first 94 frequencies and mode shapes for the dynamic model. The maximum modal frequency computed was 19.26 Hz, which corresponds to the constant acceleration value of the Housner design response spectra for 7% damping. The calculated modes account for 97.2% of the total mass participation associated with the structure. The computed mode shapes were then selectively plotted and examined to insure that the dominant modes depicted motions 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 2. 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). Table 0 provides 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 control and administration building 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 7.03% to 23.3% of critical damping. The maximum modal damping was conservatively limited to 20% for the response spectrum analysis. Design response spectra curves for 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 those 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, forces and modal displacements for the various elements that comprise the finite element model.

3.4 Time History Analysis

The time history analysis of the control and administration building 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 (mode shapes, participation factors, composite modal damping, etc.) from the modal analysis were utlilized 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 control and administration building was used only to develop instructure response spectra.

3.5 Equivalent Static Analysis

The equivalent static analysis method was used for the structural evaluation of various structural elements of the control and administration building, as illustrated in Section 4. The instructure response spectra developed by the time history analysis was utilized to determine the appropriate acceleration coeffiecients 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 control and administration building 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 40 psf due to personnel in the control room area. The resulting forces, shears, moments and displacements were computed for all elements in the model.

4.0 STRUCTURAL EVALUATION

This section provides the results of the evaluation of the control and administration building as part of in the BOPSSR program. Unless otherwise described herein, the specific reevaluation criteria by which the analytical results were evaluated are given in Section 3.8.4 of Reference 1. In general the basis for the criteria governing stresses within the elastic range as described therein, is current day applicable code requirements.

The acceptance criteria for concrete structural members include increases in concrete strength due to the effects of aging. A concrete compressive strenght of 4,500 psi was used for the reevaluation of the control and administration building as compared to the original specification which required a minimum 28-day compressive strength of 3,000 psi. There are several factors that indicate the actual overall compressive strength of the in-situ concrete is well in excess of 4,500 psi. First, the 3,000 psi value is a minimum allowable 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 5 and 6 indicates a general increase in strength of concrete 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 reactor 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 using the Schmidt Hammer averaged 7,290 psi. The concretes in all 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 against 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 a conservative value of 4,500 psi (50% increase in the original minimum design value) for the in-situ concrete compressive strength was utilized in the structural reevaluation of the control and administration building.

The evaluation of the control and administration building is divided into two categories, critical and non-critical portions of the structure. Non-critical portions of the structures are those portions whose response or collapse will not impair the integrity or function of Seismic Category A structures, systems or components. The critical portions of the control and administration building include the three-story reinforced concrete portion comprised of the control room and 4 kV switchgear room and the southern end of the building which houses the safety related station batteries and security batteries. The non-critical portion of the structure includes the remaining portion of the control and administration building. A detailed evaluation of the structural components for both the critical and non-critical portions of the building is given in the following paragraphs.

4.1 Critical Portions Of The Building

The critical portions of the control and administration building were analyzed for the design basis earthquake in accordance with the methodology described in Section 3.0. Results of the structural integrity evaluation and a comparison with the criteria allowables of Reference 1 are presented in Tables 1 through 8. The structural evaluation was performed for the following elements (see Figures 2 through 4).

- Walls surrounding the control room area

- The communictions room and the chemical control room walls; the heating, ventilating, and air conditioning equipment room and chemical laboratory area walls; the classroom and instrument repair area walls

- The battery room walls
- Miscellaneous internal walls
- The supporting slabs for communication equipment; heating, ventilation, and air conditioning equipment; chemical laboratory equipment
- The control room slabs
- The roof slabs for the battery room and the control building
- The structural steel members supporting the battery room roof and the control room slab
- Miscellaneous steel and concrete beams
- Wall footings and isolated column footings

4.1.1 Reinforced Concrete Walls

There are a total of 20 reinforced concrete walls which are part of the critical portion of the control and administration building. The wall thicknesses vary from 8 inches to 2 feet, 10 inches and their heights vary from 12 feet, 6 inches to 45 feet, 6 inches. The computed shears, axial forces and moments were compared with the criteria allowables and the results are summarized in Table 1.

The computed in-plane shear stresses were evaluated in the following manner. First, the shear resultant was checked against the allowable as determined by the following equation from section 11.10.5 of ACI Standard 318-77:

$$V_u = \phi 2 \sqrt{f'_c} \quad \ldots\ldots\ldots\ldots \quad (1)$$

where

V_u = allowable shear (psi)
ϕ = capacity reduction factor
f'_c = concrete compressive strength

If the computed shear exceeded the above allowable, then a more detailed method was utilized. This consisted of verifing that the original condition of the wall rebar at corners and at intersections with cross walls was provided in accordance with current code requirements. Based on this condition being met and in accordance with Section 3.8.4.5 of Reference 1, the computed in-plane shear stress was computed conservatively with the following equation, which includes the effect of reinforcement to concrete section area for added shear capacity (see Reference 7):

$$V_u = 2\sqrt{f'c} + p\,fy \quad (2)$$

where:

p = the ratio of the area of reinforcement to the area of concrete section resisting the shear.

fy = yield strength of reinforcement (psi)

In the following text and in the tables all shear allowables are determined by equation (1) unless otherwise noted.

The north communication room wall (see Figures 2 and 4, wall WC-2), the north control room walls (see Figures 2 and 4, walls WC-5-a and WC-5-b) and the east ventilating equipment room wall (see Figures 2 and 4, wall WC-B) are supported by the communication room slab (El 30' - 1 1/2" and El 32' - 0"), the chemical control room slab (El 42' - 0"), and the roof slab (El 54' - 9 1/2"). The principal reinforcement in these walls is oriented vertically with minimum horizontal reinforcement. To simplify the analysis in a conservative manner, the walls were analyzed as a 1-foot wide, three span, continuous beam. The moments and shears for walls WC-2, WC-5-b, and WC-B were all less than the criteria allowables (safety factors for shear and moment were 2.78 and 4.51 for wall WC-2; 1.68 and 1.96 for wall WC-5-b; and 1.12 and 1.10 for wall WC-B). For wall WC-5-a the computed moment exceeded the design moment by 12 percent. However, the resulting ductility ratio for the wall was 1.14 which is less than the criteria allowable of 3.0.

The west control room wall (see Figures 2 and 4, wall WC-K) is supported by the control room floor slab at El. 42'-0", the control room roof slab at El. 56'-7 1/2" and at its base by a continuous spread footing. The west communication room walls (see Figures 2 and 4, walls WC-F and WC-H) are supported by a continuous spread footing at their base, the communication room floor slab at El. 30'-1 1/2", the floor slab at El. 42'-0", and the roof slab at El. 54'-9 1/2". The principal reinforcement in these walls is oriented vertically with minimum horizontal reinforcement. These walls were analyzed elastically utilizing the finite element model and the response spectrum technique described in Section 3.0. Computed bending moments and axial loads for these walls were compared with the strength design interaction capacity of the wall. The results indicated that 20%, 63% and 48% of the interacion capacity of walls WC-K, WC-F and WC-H, respectively, were utilized to resist the applied loads. Safety factors associated with the in-plane shear ranges from 1.52 to 3.68. Therefore, these walls are adequate.

The south classroom area wall (see Figures 2 and 4, wall WC-8) is divided into two segments. Between column lines 'B' and 'C' the wall has an intermediate support at El. 35'-6" and spans vertically from El. 12" - 0" to 54' - 9 1/2". Between column lines 'C' and 'E' the wall spans vertically from El. 35' - 6" to El. 54' - 9 1/2" with an intermediate support at El. 42' - 0". To simplify the analysis, a one-foot wide strip was analyzed. The computed moment and in-plane shear (see Table 1) for the wall above El. 35' - 6" was less than the criteria allowables. Safety factors for shear and moment were 1.68 and 1.45 respectively. For the portion of the wall below El. 35' - 6", the computed negative moment (6.69 k-ft/ft) was less than the allowable uncracked moment (6.79 k-ft/ft).

The south control room wall (see Figures 2 and 4, wall WC-6) is principally reinforced in the vertical direction with minimum reinforcement in the horizontal direction. It was analyzed as a two span continuous beam that is supported at the top by the control room roof slab at El. 56' - 7 1/2" and fixed at El. 42' - 0" by the control room floor slab and at El 12' - 0" by the wall footing. The maximum negative moment at the footing and at the interior support exceeded the design moment allowables. However, their corresponding ductility ratios of 1.34 and 2.23 are less than the criteria allowable of 3. The computed in-plane shear stresses from the BSAP analysis (26.37 ksf and 26.84 ksf) for the portions of the wall just below and above El. 42' - 0" exceeded the shear values as determined by equation (1) by 38% and 39% respectively. However, when the computed in-plane shear stress was compared with the shear values from equation (2) (35.57 ksf and 27.32 ksf), the computed shear was acceptable.

The north wall of the south stairwell (see Figure 2, wall WC 7.5) is supported by the floor at El. 32' - 0", the slab at El. 42' - 0" and the roof slab at El. 54' - 9 1/2". The principal reinforcement in this wall is oriented vertically with minimum horizontal reinforcement. The wall was analyzed as a 1-foot wide strip, three span, continuous beam. The computed maximum moments (see Table 1) were less than the design allowable moments (safety factor for moment varies from 1.58 to 3.85). The computed in-plane shear stress (BSAP results) between El. 20' - 9" and El. 32' - 0" exceeded the shear value of equation (1) by 34%. However, this shear stress is less than the allowable stress computed based on equation (2) (24.84 ksf versus 31.25 ksf). The computed in-plane shear stress for that portion of the wall between El. 32' - 0" and El. 42' - 0" exceeds the calculated shear values using equations (1) and (2). However, the wall in question represents only 1.11 percent of the total shear capacity in the east-west direction for the structure at this elevation. Considering the worst case, that being that the wall, WC-7.5, were to degrade to a point of contributing zero shear stiffness to the structure, the remaining shear walls are capable of resisting the total calculated shear without exceeding the allowable, as computed by equation (2). The resulting minimum safety factor is 1.83. Because the capacity exists within the structure to accommodate the redistribution of the shear forces, assuming wall WC-7.5 completely degrades at this elevation, and the fact that displacements associated with in-plane shear forces in a reinforced concrete shear resisting structure are small in magnitude, wall WC-7.5 is considered to be acceptable in its in-situ condition in accordance with the general criteria of paragraphs A and B of Section 3.8.4.5 of Reference 1.

The east control room wall (see Figures 2 and 4, walls WC-C-b, WC-C-c and WC-C-d) is divided into three segments. Between column lines '5' and '8' the walls WC-C-b and WC-C-c are supported by the wall's spread footings, interior slabs at El. 32' - 0" and El. 42' - 0", and the roof slab at El. 54' - 9 1/2". Between column lines '8' and '9' the wall WC-C-d is supported by the wall's spread footing at its base and the roof at El. 35' - 6". The principal reinforcement in these walls is oriented vertically with minimum horizontal reinforcement. To simplify the analysis, a one-foot wide strip was analyzed. The computed maximum moments (see Table 1) were less than the design allowable moments for walls WC-C-b an WC-C-c (safety factor for moment is 5.04 and 4.02 respectively). The computed moment for wall WC-C-d exceeded the design moment capacity by 47%. However, the computed ductility ratio is 2.29 which is less than the criteria allowable of 3. The computed in-plane shear for these walls was less than the shear value of equation (1) (minimum safety factor for shear is 1.07) with the exception of wall WC-C-b whose computed in-plane shear exceeded the shear value of equation (1) by 10%. However, this shear stress is less than the allowable stress computed based on equation (2) (18.25 ksf versus 27.36 ksf).

The west wall of the classroom area (see Figure 2, wall WC-E) was analyzed as a one foot wide beam, spanning vertically with one end fixed at the control room roof (El. 54' - 9 1/2") and the other end simply supported at the classroom slab (El. 42' - 0"). The computed maximum moment (2.87 k-ft/ft) was less than the uncracked moment capacity (6.79 k-ft/ft) of the wall (see Table 1). The computed in-plane shear stress exceeded the shear value of equation (1) by 29%. However, the allowable stresses (29.95 ksf) computed based on equation (2) are greater than the in-plane shear stress (23.08 ksf) due to the applied loading.

In conclusion, all the reinforced concrete walls satisfy the the BOPSSR criteria.

4.1.2 Reinforced Concrete Slabs

The floor slabs at El. 32' - 0" and El. 42' - 0" and the control and administration building roof slabs (see Figures 2 and 3) were evaluated in accordance with the methodology described in Section 3.0. Results of the structural integrity evaluation of these slabs are described in the following paragraphs.

The control room roof slab (see Figure 3, slab SC-1); the control and administration building roof slabs surrounding the control room area (see Figure 3, slabs SC-2 and SC-3); the chemical control room slab (see Figure 3, slab SC-5); the communications room slab and the heating-ventilating room slab (see Figure 2, slabs SC-8 and SC-9 respectively) were all analyzed in accordance with section 3.0. The computed bending moments and axial loads for these slabs were compared with the strength design interaction capacity for each slab. The analytical results (see Table 2) indicated that between 68% and 96% of the interaction capacity of each slab was utilized to resist the applied loads.

The classroom slab at El. 32' - 0" (see Figure 2, slab SC-10) is prinicially reinforced in the east-west direction with minimum reinforcement in the north-south direction. The west and east ends of the slab are supported by beam BC-18 and wall WC-C-b respectively. The slab is also supported by the two intermediate beams, BC-17 and BS-25. The floor slab was analyzed as a one foot wide, three span continuous beam. The computed maximum moment (2.92 k-ft/ft) was less than the uncracked moment capacity (3.02 k-ft/ft) of the wall (see Table 2).

The control and administration building roof slab between column lines '6' and '8' and 'B' and 'E' (see Figure 3, slab SC-4) is principally reinforced in the east-west direction with minimum reinforcement in the north-south direction. The west and east ends of the slab are supported by walls WC-E and WC-B respectively. The slab is also supported by two intermediate concrete roof beams (BC-1 and BC-2). The roof slab was analyzed as a one foot wide, two span continuous beam. The resulting maximum slab moment exceeded the design moment capacity by about 11% (see Table 2). However, the slab ductility ratio is less than 2 which is within the allowable of 3 for overall structure ductility. The control and administration building slabs between column lines 'B' and 'C' and column lines '6' and '8' (see Figure 3, slabs SC-6 and SC-7 respectively) have principal reinforcement in the east-west direction with minimum reinforcement in the north-south direction. The west and east ends of the slab (SC-6) are supported by walls WC-C-b and WC-B respectively. Slab SC-7 is supported by walls WC-E and WC-C-c at the west and east ends, respectively, and by an interior concrete floor beam (BC-12). The slabs were analyzed as a one foot wide beam with the applied loadings as described in section 3.0. All computed moments and shears (Table 2) in these slabs were less than the criteria allowables (safety factors for slabs SC-6 and SC-7 were 6.72 and 8.82 due to shear and 1.42 and 1.25 due to moment, respectively).

The control room floor slab (see Figure 3, slab SC-12) has principal reinforcement in the east-west direction with minimum reinforcement in the north-south direction. Four, one foot wide, continuous beams were analyzed which included all of the various edge and interior support conditions, associated with the slab (such as slab to wall connection, interior structural steel supports, floor openings at the control board and console). The analytical results showed that the maximum computed moment (see Table 2) exceeded the design moment capacity by 37%. However, the corresponding slab ductility ratio was less than 2 which is within the allowable of 3 for overall structural ductility.

In conclusion, all the floor and roof slabs satisfy the BOPSSR criteria.

4.1.3 Reinforced Concrete Beams

The concrete beams supporting the second floor slab (see Figure 2, beams BC-17, BC-18 and BC-19), the third floor slab (see Figure 3, beams BC-4, BC-9 through BC-14) and the roof (see Figure 3, beams BC-1 and BC-2) were analyzed in accordance with section 3.0. Computed maximum moments and shears (see Table 3) in the beams were less than the criteria allowables (safety factors for shear ranged from 1.03 to 12.8 and for moment ranged from 1.02 to 10.6), with the exception of beams BC-10, BC-11, and BC-13. Analysis of these beams resulted in moments which slightly exceed the design moment capacity by 7% to 23%. However, the maximum element ductility ratio was less than 2 which is within the allowable of 3 for overall structural ductility. All shears were determined to be less than the allowable values.

The remaining concrete beams (see Figure 2 for beams BC-15, BC-16 and BC-20; Figure 3 for beams BC-3, BC-5 through BC-8) were analyzed utilizing the equivalent static analysis method described in Section 3.0, assuming simply supported end conditions for beams. Computed moments and shears (see Table 3) for beams BC-5 through BC-8, were less than the criteria allowables. The safety factors for these beams ranged from 8.80 to 13.4 due to shear and from 4.99 to 11.4 due to moment. Analysis of beams BC-3, BC-15, BC-16 and BC-20 resulted in moments which exceeded the design moment capacity from 31% to 46%. However, their corresponding ductility ratios were less than 3 which is within the allowable of 3 for overall structural ductility. All shears were found to be less than the allowable values. In conclusion, all of the reinforced concrete beams are found to be adequate.

4.1.4 Structural Steel

The structural steel evaluation included the steel members supporting the battery room roof and the control room floor and the steel columns and their connection details. The tabulated results of the structural steel member evaluations are given in Tables 4 and 5. The results of the evaluations of representative connection details are given in Table 6. The structural steel members were analyzed for the design basis earthquake in accordance with the methodology described in Section 3.0. The computed maximum moments, shears and axial loads (see Table 4) in the beams (see Figures 2 and 3, beams BS-1 through BS-30) were less than the criteria allowables (overall safety factors ranged from 1.24 to 4.76).

The steel columns included in the evaluation are: one tube steel column outside the battery room, the pipe column located in the communication room and three W12 X 65 columns in th switchgear and cable spreading room (see Figure 2, columns CS-1 through CS-5 respectively). These columns were evaluated based on the results of the finite element analysis described in Section 3.0. The calculated maximum compressive stresses of 13.29 ksi and 6.32 ksi for the tube steel column and the pipe column were below the allowable of 13.89 ksi and 16.82 ksi, respectively. The computed maximum moments, shears and axial loads for column CS-3 were less than the criteria allowables. The overall safety factor for the column is 2.46. Comparison of the analytical results showed that the safety factors for columns CS-4 and CS-5 were higher than that for column CS-3. Therefore, all steel columns are adequate.

In addition to the evaluation of the main structural elements of the building, the connection details were also evaluated. Principally, there are three types of steel connections: (1) steel to masonry wall, (2) steel to concrete, and (3) steel to steel. The connection stresses for the masonry wall and the steel beams were compared against the working stress allowables from Table 24-G of the 1979 Edition of te Uniform Building Code. The working stress allowables were increased by a factor of 1.33 in accordance with the provisions of Section 2303 of the Uniform Building Code. The concrete wall and steel beam connection stresses were compared against ultimate strength allowables. The computed stresses for steel-to-steel connections were compared against allowables of the BOPSSR criteria. The response spectrum analysis and the static analysis results, as described in Section 3.0, were utilized for the connection evaluation.

The shears and the axial loads for beam-to-beam connections or beam to column connections are transferred through 3/4" diameter, A325 friction type high strength bolts. For steel beam to concrete wall connections or steel beam to masonry wall connections, the load transfer path is by 3/4" diameter, A307 anchor bolts and/or by direct bearing on top of the wall. The calculated shears and axial loads for the anchor bolts, the bearing plates, the insert plates and the embedded anchors were checked against their allowables. The results showed that the safety factors ranged from 1.19 to 7.95 for shear and 1.05 to 3.72 for axial load (see Table 6).

In conclusion all structural steel elements of the building satisfy the BOPSSR program criteria.

4.1.5 Footings and Foundation Media

The foundation for the control and administration building, which consists of continuous spread footings beneath the walls and individual footings beneath the steel columns, was analyzed conservatively using the maximum computed soil bearing value as a uniform load. The representative results of the footings and the foundation media are given in Table 7. The resulting maximum moment in the wall footing was calculated to be 9.8 k-ft/ft, while the allowable is 26.4 k-ft/ft. Therefore the footing is adequate. The soil bearing pressure beneath the wall footing was found to be acceptable because the maximum pressure was determined to be 8.7 ksf as compared to the allowable of 17.0 ksf from Reference 3. The computed shears, bending moments and axial loads (145 kps, 8.1 k-ft/ft and 13.6 ksf, respectively) for the isolated footings are well within the criteria allowables (see Table 7). In conclusion all the stresses in the footings and the foundation media satisfy the BOPSSR Program criteria.

4.1.6 Reinforced Masonry Block Walls

There are three reinforced masonry walls in the control and administration building. Two are located about the battery room (see Figure 2, walls WM-8-a and WM-A-s) while the third is in the administration area (see Figure 2, wall WM-A-n). The three masonry walls were first analyzed elastically as part of the finite element model, utilizing the response spectrum technique. In cases where the response spectrum analysis indicated inelastic behavior, the walls were evaluated using the energy balance technique described in Section 3.7.2.1 of Reference 1. In addition, the walls were compared with the results from the non-linear analyses performed on similar walls at San Onofre Unit 1.

Each of the three walls has principal reinforcement in the vertical direction, which consists of #4 rebar at 24" spacing. In the horizontal direction the reinforcing consists of Dur-O-Wall truss type S. Each wall spans vertically approximately 14' and is supported at its base by a continuous spread footing. The top of each wall is connected to its respective roof diaphragm in a non-moment resisting manner (see Figure 4).

The analysis results for in-plane shear for walls WM-A-n and WM-A-s are within the criteria allowables giving a minimum safety factor of 1.0 and 1.02 for each of these walls (see Table 8). The computed in-plane shear for wall WM-8-a resulted in a minimum safety factor of 0.96 in relation to the criteira allowables for sliding. However, wall WM-8-a is partially constrained by the reinforced concrete wall it frames into and the sliding criteria itself is conservative, as stated in section 3.2.3 of Reference 8. Therefore, the three reinforced masonry walls are adequate for in-plane shear.

For out-of-plane loading conditions, portions of the three walls exceed the elastic criteria. Wall WM-A-s exceeds the elastic criteria at mid-span height (point of maximum moment) for a region consisting of about 75% of its total length. However, the maximum resulting ductility ratio for this wall is 1.4 (see Table 8) which is less than the criteria allowable of 3. For wall WM-A-n the elastic allowable is exceeded for its entire length at its mid-span height. For this wall the computed ductility ratio is 3.0 or less for 85% of its length with a maximum ductility ratio of 4.0 for the remaining 15%. For the third wall WM-8-a 33% of its length meets the elastic criteria for out-of-plane loading, while the remaining 67% exceeds the elastic criteria with a resulting ductility ratio of 4.1.

Because portions of wall WM-A-n and wall WM-8-a exceeded the ductility of 3.0, these walls were evaluated with respect to similarity to other walls at San Onofre Unit 1, which were evaluated by Computech Engineering Services, Inc. (CES). The criteria and results for this evaluation were submitted in References 8 and 9, respectively. In the CES report, there are two sets of walls which are similar in nature to the masonry walls of the control and administration building. One set is the Group 2 walls of the reactor auxiliary building which span vertically about 15' (as compared to a span of 14' for the walls in question) and have about 30% less reinforcing. The non-linear analysis of this wall resulted in a maximum displacement at mid-span of 2.99" and a local steel ductility ratio of 3.78 as compared to a criteria allowable of 45. Another similar wall exists in the turbine building, wall TB-8, which spans vertically 14' and contains about 10-20 percent more reinforcing than the control and administration building walls. The non-linear analysis of this wall, without attached equipment (same condition as walls WM-A-n and WM-8-a, resulted in a maximum mid-span displacement of less than 2" and a local steel ductility ratio of 3, as compared to an allowable of 45. From these comparisons it is concluded that the three reinforced masonry walls associated with the control and administration building are adequate to resist the design loads imposed upon them.

In conclusion, all of the reinforced masonry walls in the control and administration building are found to satisfy the BOPSSR criteria.

4.2 Non-Critical Portions Of The Building

The non-critical portion of the structure consists of a single-story administration office building located on the east side of the control and administration building. The structural elements were analyzed for the design basis earthquake in accordance with the methodology described in Section 3.0. A comparison with the criteria allowables of Reference 1 is presented in Tables 9 through 12 for the purpose of displaying complete results. However, the acceptance criterion for the non-critical portions of this building (as stated in Reference 1) is that the response or collapse of these structural members will not impair the integrity or function of Seismic Category A structures, systems, or components. The non-critical portions of the building included (see Figure 8) the structural steel beams supporting the administration room slab, two reinforced concrete walls on the north end of the building, and the tube steel columns on the east side of the building. It has been concluded that these structural elements meet the acceptance criteria since there are no safety-related systems in these areas of the building. Evaluation methods and results for the reinforced concrete roof slab and the reinforced masonry block wall located at the north end of the building, which are also non-critical, were described in Sections 4.1.2 and 4.1.6, respectively.

The 13 inch thick reinforced concrete wall (wall NWC-1) has reinforcement in both the vertical and horizontal directions, which consists of #4 rebar at 12" spacing on each face of the wall, while the 8 inch thick wall (wall NWC-C) has central reinforcement which consists of #4 rebar at 12" spacing. Both walls are connected to the administration roof diaphragm at the top and are supported at their base by a continuous wall footing. These walls were analyzed utilizing the equivalent static analysis method described in Section 3.0 and the analytical results are tabulated in Table 9. The analysis results for in-plane shear and moments are within the criteria allowables (the minimum safety factor is 1.06) with the exception of wall NWC-1, whose computed maximum moment exceeded the allowable by 31%. However, the maximum computed ductility ratio for this wall is 1.56 which is less than the criteria allowable of 3.

The structural steel evaluation included all of the steel members supporting the administration roof slab (beams NBS-1 through NBS-20), and three tube steel columns (NCS-1 through NCS-3). The steel beams and the columns were analyzed elastically utilizing the finite element model and the response spectrum technique described in Section 3.0.

The analysis results and their comparison with the criteria allowables (see Table 10) for the steel beams indicated that the computed stresses were less than the criteria allowables (overall safety factors range from 1.01 to 2.42), with the exception of beam NBS-13. The computed moment for beam NBS-13 was found to exceed the ultimate allowable moment by 3%. However, the axial load for this beam is very small in comparison with the allowable (safety factor of 18.31) and the beam possesses excess capacity in bending due to ductile behavior.

Three steel columns (NCS-1 through NCS-3) on the east side of the control and administration building, supporting a portion of the administration roof, were analyzed elastically utilizing the finite element model and the response spectrum technique described in Section 3.0. For all three columns, the compressive stress due to the applied loading exceeded the criteria allowables by 8% to 17% (see Table 11). However, additional capacity exists in the plastic mode of behavior.

In addition to the main structural steel members, the steel to steel connections, the steel to masonry wall connections and the steel and concrete connections were evaluated. The evaluation was based on the computed shear, bending moment and axial loads for each of the attaching structural members utilizing the finite element model and the response spectrum technique described in Section 3.0. Except for the east end connection of beam NBS-17, the minimum safety margin achieved for all of the remaining connections was 1.12 (see Table 12). The computed axial load for the east end connection of beam NBS-17 exceeded the allowable for the bearing plate by 11%. However, additional capacity does exist in the plastic mode of behavior.

5.0 CONCLUSION

This report provides results of the evaluation of the San Onofre Unit 1 control and administration building in accordance with the methodology discussed in Section 3. As described in detail in Section 4, all structural elements of the control and administration building satisfy the BOPSSR criteria. Therefore, no modifications are required for this structure.

6.0 REFERENCES

1. "Balance of Plant Structures Seismic Reevaluation Criteria," San Onofre Nuclear Generating Station, Unit 1, February 17, 1981.

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3. Balance of Plant SONGS Unit 1, "Soil-Structure Interaction Methodology Report," Revision 1, July 20, 1978, Woodward-Clyde Consultants, Orange, California.

4. Final Safety Analysis Report, San Onofre Nuclear Generating Station, Units 2 and 3, Appendix 3C.

5. Washa, George W., and Wendt, Kurt, F. "Fifty Year Properties of Concrete," ACI Journal, January, 1975.

6. Gonnerman, H. F., and Lerch, William, "Changes in Characteristics of Portland Cement as Exhibited by Laboratory Tests Over the Period 1904 to 1950," Portland Cement Association Research Department Bulletin 39.

7. Seismology Committee Structural Engineers Association of California, "Recommended Lateral Force Requirements and Commentary", 1980.

8. "Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 1: General and Criteria," Computech Engineering Services, Inc., Berkeley, California.

9. "Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 3: Masonry Wall Evaluation," Computech Engineering Services, Inc., Berkeley, California.

TABLE O

MODAL FREQUENCIES, COMPOSITE DAMPING AND MODAL MASS FOR MAJOR CONTRIBUTING MODES

MODE NUMBER	FREQUENCY (HERTZ)	COMPOSITE MODAL DAMPING	% OF TOTAL MASS PARTICIPATING NORTH-SOUTH DIRECTION	% OF TOTAL MASS PARTICIPATING EAST-WEST DIRECTION	% OF TOTAL MASS PARTICIPATION VERTICAL DIRECTION
3	3.02	20.0%	30.90	2.00	33.10
7	3.88	14.6%	0.70	44.00	2.70
11	4.84	20.0%	10.10	0.80	7.50
12	5.01	12.9%	20.50	0.40	12.60
13	5.10	12.1%	18.80	0.30	8.60
14	5.29	13.6%	4.20	0.80	1.70
19	6.05	20.0%	4.20	1.60	0.80
25	7.80	12.1%	1.30	23.70	2.30
28	8.36	10.4%	0.00	8.70	1.00

REINFORCED CONCRETE WALLS

IDENTIFICATION	REFERENCE FIG. NO	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	UF	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
WC - B	FIG. 2 & 4	REINFORCED CONCRETE WALL AT EAST SIDE OF CONTROL BLDG. THICKNESS: 9".	14.69	16.42	1.12	-	2.41	2.65	1.10	-	-	X		WALL IS ADEQUATE.
WC - C - a	Fig. 2 & 4	REINFORCED CONCRETE WALL AT EAST END OF CONTROL ROOM, BETWEEN LINES 3 & 5. THICKNESS: 8".	12.93	16.42	1.27	-	1.94	2.35	1.21	-	-	X		WALL IS ADEQUATE.
WC - C - b	Fig. 2 & 4	REINFORCED CONCRETE WALL AT EAST END OF CONTROL ROOM, BETWEEN LINES 5 & 7.5 THICKNESS: 1'-0"	18.25	* 36.92	2.02	-	2.34	11.79	5.04	-	-	X		WALL IS ADEQUATE.
WC - C - c	Fig. 2 & 4	REINFORCED CONCRETE WALL AT EAST END OF CONTROL ROOM, BETWEEN LINES 7.5 & 8 THICKNESS: 1'-0"	15.40	16.42	1.07	—	2.93	11.79	4.02	—	—	X		WALL IS ADEQUATE.

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR STRESS, KIPS/FT^2
4. VA - ALLOWABLE SHEAR STRESS, KIPS/FT^2
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT FEET-KIP/FT.
7. M_A - ALLOWABLE MOMENT FEET-KIP/FT.
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u / \Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. U.F. - UTILITY FACTOR, PERCENT OF THE INTERACTION CAPACITY BEING UTILIZED.

* SEE EQUATION (2) OF SECTION 4.1.1.

REINFORCED CONCRETE WALLS

IDENTIFICATION	REFERENCE FIG. NO	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	UF	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
WC – C – d	FIG. 2 & 4	REINFORCED CONCRETE WALL AT EAST END OF CONTROL ROOM BETWEEN LINES 8 & 9 THICKNESS: 8"	15.26	16.42	1.08	—	4.45	2.35	0.53	2.29	3.0	X		WALL IS ADEQUATE.
WC – D	FIG. 2 & 4	REINFORCED CONCRETE WALL AT WEST SIDE OF NORTH STAIRWELL. THICKNESS: 8"	12.7	16.42	1.28	99.73	—	—	—	—	—	X		WALL IS ADEQUATE.
WC – E	FIG. 3 & 4	REINFORCED CONCRETE WALL AT SOUTH WEST END OF CONTROL BLDG. THICKNESS: 9"	23.08	29.95 *	1.30	—	2.87	6.79 (12)	2.37	—	—	X		WALL IS ADEQUATE
WC – F	FIG. 2 & 4	REINFORCED CONCRETE WALL IN NORTH WEST PART OF CONTROL BLDG. THICKNESS: VARIES FROM 1'-1" to 2'-5½".	4.46	16.42	3.68	63.49	—	—	—	—	—	X		WALL IS ADEQUATE.

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR STRESS, KIPS/FT²
4. V_A - ALLOWABLE SHEAR STRESS, KIPS/FT²
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT FEET-KIP/FT.
7. M_A - ALLOWABLE MOMENT FEET-KIP/FT.
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u / \Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. U.F. - UTILITY FACTOR, PERCENT OF THE INTERACTION CAPACITY BEING UTILIZED.

(12) M_A = ALLOWABLE MOMENT AT CRACKING, FEET-KIP/FT.

* SEE EDQUATION (2) OF SECTION 4.1.1.

TABLE 1 (CONT.)

REINFORCED CONCRETE WALLS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	UF	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
WC — H	FIG. 2 & 4	REINFORCED CONCRETE WALL AT NORTH WEST END OF CONTROL BLDG. THICNKNESS: VARIES FROM 1'-0" to 2'-10"	10.80	16.42	1.52	48.19	—	—	—	—	—	X		WALL IS ADEQUATE.
WC — K	FIG. 2 & 4	REINFORCED CONCRETE WALL AT WEST END OF CONTROL ROOM. THICKNESS: 2'–10"	9.79	16.42	1.68	20.41	—	—	—	—	—	X		WALL IS ADEQUATE.
WC – 2	FIG. 2 & 4	REINFORCED CONCRETE WALL AT NORTH END OF CONTROL BLDG. THICKNESS: VARIES FROM 2'-10" to 3'-2".	5.90	16.42	2.78	—	9.14	41.19	4.51	—	—	X		WALL IS ADEQUATE.
WC – 3	FIG. 2 & 4	REINFORCED CONCRETE WALL AT NORTH SIDE OF NORTH STAIRWELL. THICKNESS: 8".	13.45	16.42	1.22	—	1.05	2.35	2.24	—	—	X		WALL IS ADEQUATE.
WC — 4	Fig. 2 & 4	REINFORCED CONCRETE WALL AT SOUTH SIDE OF NORTH STAIRWELL. THICKNESS: 8"	—	—	—	—	—	—	—	—	—	X		WALL IS ADEQUATE BY COMPARISON TO WC-3.

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, KIPS/FT.^{2}
4. V_A - ALLOWABLE SHEAR, KIPS/FT.^{2}
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP/FT.
7. M_A - ALLOWABLE MOMENT, FEET-KIP/FT.
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u/\Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. U.F. - UTILITY FACTOR, PERCENT OF THE INTERACTION CAPACITY BEING UTILIZED.

REINFORCED CONCRETE WALLS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	UF	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
WC-5-a	FIG. 2 & 4	REINFORCED CONCRETE WALL AT NORTH SIDE OF CONTROL ROOM WEST END. THICKNESS: 2'-10"	7.36	16.42	2.23	—	41.42	36.62	0.88	1.14	3.0	X		WALL IS ADEQUATE.
WC-5-b	FIG. 2 & 4	REINFORCED CONCRETE WALL AT NORTH SIDE OF CONTROL ROOM EAST END THICKNESS: 1'-0"	9.74	16.42	1.69	—	2.81	5.50	1.96	—	—	X		WALL IS ADEQUATE.
WC-6	FIG. 2 & 4	REINFORCED CONCRETE WALL AT SOUTH SIDE OF CONTROL ROOM.												
		BELOW EL. 42'-0" THICKNESS: 1'-1"	26.37	35.57*	1.34	-	16.81	12.95	0.77	1.34	3.0	X		WALL IS ADEQUATE
		ABOVE EL. 42'-0" THICKNESS: 1'-0"	26.84	27.32*	0.61	-	10.33	5.44	0.53	2.23	3.0	X		
WC-7.5	FIG. 2 & 4	REINFORCED CONCRETE WALL AT NORTH SIDE OF SOUTH STAIRWELL THICKNESS: 8" EL. 20'-0" - 32'-0"	24.84	31.25*	1.26	-	1.49	2.35	1.58	-	-	X		WALL IS ADEQUATE.

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, KIPS/FT.²
4. V_A - ALLOWABLE SHEAR, KIPS/FT.²
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP/FT.
7. M_A - ALLOWABLE MOMENT, FEET-KIP/FT.
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u/\Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. U.F. - UTILITY FACTOR, PERCENT OF THE INTERACTION CAPACITY BEING UTILIZED.

* SEE EQUATION (2) OF SECTION 4.1.1.

REINFORCED CONCRETE WALLS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	UF	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	NO	REMARKS
WC-7.5 (Cont.)	FIG. 2 & 4	EL. 32'-0" - El.42'-0"	160.51	16.42	0.10	-	0.61	2.35	3.85	-	-	X		WALL IS ADEQUATE (REFER TO SECTION 4.1.1)
		EL. 42'-0" - El. 54'-2"	4.25	16.42	3.46	-	0.81	2.35	2.90	-	-	X		
WC-8	FIG. 2 & 4	REINFORCED CONCRETE WALL AT SOUTH END OF CONTROL BLDG. ABOVE ELEVATION 37'-8½"	9.75	16.42	1.68	—	4.16	6.04	1.45	—	—	X		WALL IS ADEQUATE.
		BELOW ELEVATION 37'-8½" THICKNESS - 9" NEGATIVE MOMENT	11.89	16.42	1.38	-	6.69	6.79[12]	1.10	2.58	3.0	X		WALL IS ADEQUATE.
WC-9	FIG. 2 & 4	REINFORCED CONCRETE WALL AT SOUTH END OF ADMINISTRATION BLDG. THICKNESS: 1'-1"	3.97	16.42	4.13	—	5.40	6.04	1.12			X		WALL IS ADEQUATE.

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, KIPS/FT.^{2}
4. V_A - ALLOWABLE SHEAR, KIPS/FT.^{2}
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP/FT.
7. M_A - ALLOWABLE MOMENT, FEET-KIP/FT.
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u / \Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. U.F. - UTILITY FACTOR, PERCENT OF THE INTERACTION CAPACITY BEING UTILIZED.

(12) M_A=ALLOWABLE MOMENT AT CRACKING, FEET-KIP/FT.

REINFORCED CONCRETE SLABS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	UF	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
SC - 1	FIG. 3 & 4	REINFORCED CONCRETE SLAB AT EL.56'-7½" THICKNESS: 1' -11"	8.32	16.42	7.97	86.06	203.4	141.2	0.69	1.54	3.0	X		SLAB IS ADEQUATE.
SC - 2	FIG. 3 & 4	REINFORCED CONCRETE SLAB AT EL. 54' -9½". THICKNESS: 1' -11"	3.05	16.42	5.38	94.69	-	-	-	-	-	X		SLAB IS ADEQUATE.
SC - 3	FIG. 3 & 4	REINFORCED CONCRETE SLAB AT EL. 54' -9½". THICKNESS: 7"	3.24	16.42	5.06	74.6	-	-	-	-	-	X		SLAB IS ADEQUATE.
SC - 4	FIG. 3 & 4	REINFORCED CONCRETE SLAB AT EL. 54' -9½" THICKNESS: 7"	2.13	16.42	7.72	-	3.17	2.81	0.89	1.13	3.0	X		SLAB IS ADEQUATE.
SC - 5	FIG. 3 & 4	REINFORCED CONCRETE SLAB AT EL. 42' -0". THICKNESS: 7"	5.98	16.42	2.75	68.3	-	-	-	-	-	X		SLAB IS ADEQUATE.
SC - 6	FIG. 3 & 4	REINFORCED CONCRETE SLAB AT EL. 42'-0". THICKNESS: 7"	2.45	16.42	6.72	-	4.90	6.95	1.42	-	-	X		SLAB IS ADEQUATE.
SC - 7	FIG. 3 & 4	REINFORCED CONCRETE SLAB AT EL. 42' -0". THICKNESS: 7"	1.86	16.42	8.82	-	2.83	3.53	1.25	-	-	X		SLAB IS ADEQUATE.

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, KIPS/FT.^{2}
4. V_A - ALLOWABLE SHEAR, KIPS/FT.^{2}
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP/FT.
7. M_A - ALLOWABLE MOMENT, FEET-KIP/FT.
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u/\Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. U.F. - UTILITY FACTOR, PERCENT OF THE INTERACTION CAPACITY BEING UTILIZED.

TABLE 2

REINFORCED CONCRETE SLABS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	UF	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA		REMARKS
												YES	NO	
SC - 8	FIG. 2 & 4	REINFORCED CONCRETE SLAB AT EL. 30' -1½". THICKNESS: 6"	2.10	16.42	7.82	95.9	-	-	-	-	-	X		SLAB IS ADEQUATE.
SC - 9	FIG. 2 & 4	REINFORCED CONCRETE SLAB AT EL. 32' -0". THICKNESS: 6"	3.48	16.42	4.72	86.19	-	-	-	-	-	X		SLAB IS ADEQUATE.
SC - 10	FIG. 2 & 4	REINFORCED CONCRETE SLAB AT EL. 32' -0". THICKNESS: 6"	2.30	16.42	7.14	-	2.92	(12) 3.02	1.03	-	-	X		SLAB IS ADEQUATE.
SC - 11	FIG. 2 & 4	REINFORCED CONCRETE SLAB AT EL. 35' -0". THICKNESS: 7"	1.12	16.42	14.72	-	2.20	4.11	1.86	-	-	X		SLAB IS ADEQUATE.
SC - 12	FIG. 3 & 4	REINFORCED CONCRETE SLAB AT EL. 42' -0". THICKNESS: 1' -0"	2.93	16.42	5.62	-	10.34	6.47	0.63	1.77	3.0	X		SLAB IS ADEQUATE.

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, KIPS/FT.^{2}
4. V_A - ALLOWABLE SHEAR, KIPS/FT.^{2}
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP/FT.
7. M_A - ALLOWABLE MOMENT, FEET-KIP/FT.
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u / \Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. U.F. - UTILITY FACTOR, PERCENT OF THE INTERACTION CAPACTY BEING UTILIZED.

(12) M_a' = ALLOWABLE MOMENT AT CRACKING, FEET - KIP/FT

REINFORCED CONCRETE BEAMS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	P_C	P_A	$SF_P = P_A/P_C$	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BC-1	FIG. 3	15"x21" CONCRETE BEAM @ EL. 54'-9½"	12.2	32.0	2.62	—	—	—	47.6	48.5	1.02	—	—	X		BEAM IS ADEQUATE
BC-2	FIG. 3	12"x18" CONCRETE BEAM @ EL. 54'-9½"	5.3	21.1	3.98	—	—	—	30.7	80.6	2.63	—	—	X		"
BC-3	FIG. 3	24"x23½" CONCRETE BEAM @ EL. 56'-7½"	111.9	120.3	1.08	—	—	—	1222.0	658.7	0.54	2.22	3.0	X		"
BC-4	FIG. 3	15"x21" CONCRETE BEAM @ EL. 42'-0"	9.8	29.7	3.03	—	—	—	86.2	386.7	4.49	—	—	X		"
BC-5	FIG. 3	12"x18" CONCRETE BEAM @ EL. 40'-0"	1.6	21.5	13.4	—	—	—	5.3	60.3	11.4	—	—	X		"
BC-6	FIG. 3	12"x18" CONCRETE BEAM @ EL.42'-0"	1.6	21.5	13.4	—	—	—	5.3	60.3	11.4	—	—	X		"
BC-7	FIG. 3	12"x18" CONCRETE BEAM @ EL. 42'-0"	2.5	21.5	8.60	—	—	—	8.1	40.4	4.99	—	—	X		"
BC-8	FIG. 3	12"x18" CONCRETE BEAM @ EL. 42'-0"	2.5	21.5	8.60	—	—	—	8.1	40.4	4.99	—	—	X		"
BC-9	FIG. 3	12"x18" CONCRETE BEAM @ EL. 42'-0"	2.6	17.4	6.69	—	—	—	12.7	32.5	2.56	—	—	X		"

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, KIPS
4. V_A - ALLOWABLE SHEAR, KIPS
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP
7. M_A - ALLOWABLE MOMENT, FEET-KIP
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u/\Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. P_C - CALCULATED AXIAL LOAD, KIPS
12. P_A - ALLOWABLE AXIAL LOAD, KIPS
13. SF_P - SAFETY FACTOR FOR AXIAL LOAD

REINFORCED CONCRETE BEAMS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	P_C	P_A	$SF_P = P_A/P_C$	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BC-10	FIG. 3	12"x18" CONCRETE BEAM @ EL. 42'-0"	4.8	21.1	4.40	—	—	—	29.3	27.3	0.93	1.07	3.0	X		BEAM IS ADEQUATE
BC-11	FIG. 3	12"x18" CONCRETE BEAM @ EL. 42'-0"	4.8	21.1	4.40	—	—	—	29.3	27.3	0.93	1.07	3.0	X		"
BC-12	FIG. 3	15"x21" CONCRETE BEAM @ EL. 42'-0"	9.0	29.7	3.30	—	—	—	62.0	353.2	5.70	—	—	X		"
BC-13	FIG. 3	15"x24" CONCRETE BEAM @ EL. 42'-0"	66.9	68.61	1.03	—	—	—	294.4	226.1	0.77	1.35	3.0	X		"
BC-14	FIG. 3	22"x27" CONCRETE BEAM @ EL. 42'-0"	4.5	57.8	12.8	—	—	—	25.3	267.4	10.6	—	—	X		"
BC-15	FIG. 2	12"x18" CONCRETE BEAM @ EL. 32'-2"	5.2	21.5	4.13	—	—	—	27.1	18.56	0.68	1.57	3.0	X		"
BC-16	FIG. 2	12"x18" CONCRETE BEAM @ EL. 32'-2"	—	—	—	—	—	—	—	—	—	—	—	X		ADEQUATE BY COMPARISON TO BC-15.
BC-17	FIG. 2	18"x27" CONCRETE BEAM @ EL. 32'-2"	27.3	49.9	1.83	—	—	—	43.0	64.5	1.50	—	—	X		BEAM IS ADEQUATE
BC-18	FIG. 2	18"x27" CONCRETE BEAM @ EL. 32'-2"	24.4	50.4	2.07	—	—	—	100.1	169.7	1.70	—	—	X		"
BC-19	FIG. 2	18"x27" CONCRETE BEAM @ EL. 32'-2"	11.5	49.9	4.34	—	—	—	36.7	64.4	1.75	—	—	X		"

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, KIPS
4. V_A - ALLOWABLE SHEAR, KIPS
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP
7. M_A - ALLOWABLE MOMENT, FEET-KIP
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u / \Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. P_C - CALCULATED AXIAL LOAD, KIPS
12. P_A - ALLOWABLE AXIAL LOAD, KIPS
13. SF_P - SAFETY FACTOR FOR AXIAL LOAD

TABLE 3 (CONT.)

REINFORCED CONCRETE BEAMS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	P_C	P_A	$SF_P = P_A/P_C$	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BC-20	FIG. 2	12"x15" CONCRETE BEAM @ EL. 32'-2"	18.13	22.64	1.25	—	—	—	47.35	32.49	0.69	1.56	3.0	X		BEAM IS ADEQUATE

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, KIPS
4. V_A - ALLOWABLE SHEAR, KIPS
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP
7. M_A - ALLOWABLE MOMENT, FEET-KIP
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u/\Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. P_C - CALCULATED AXIAL LOAD, KIPS
12. P_A - ALLOWABLE AXIAL LOAD, KIPS
13. SF_P - SAFETY FACTOR FOR AXIAL LOAD

STRUCTURAL STEEL BEAMS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_c	V_a	$SF_r = V_A/V_c$	f_{bx}	F_{bx}	$SF_b = F_{bx}/f_{bx}$	f_a	F_a	$SF_a = F_a/f_a$	R_c (1)	R_A	$SF = R_A/R_c$	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BS-1	FIG. 2	W8X17 COMMUNICATIONS ROOM	.57	42.4	74.4	1.94	15.62	8.05	1.23	5.22	4.24	.41	1.00	2.46	X		BEAM IS ADEQUATE.
BS-2	FIG. 2	W12x14 ADMINISTRATION BLDG. ROOF	8.55	54.9	6.42	13.25	38.4	2.90	2.41	35.2	14.61	.41	1.00	2.44	X		BEAM IS ADEQUATE.
BS-3	FIG. 2	"	4.84	54.9	11.34	10.50	38.4	3.66	1.26	35.2	27.94	.31	1.00	3.24	X		BEAM IS ADEQUATE.
BS-4	FIG. 2	"	2.20	54.9	25.0	5.76	38.4	6.67	1.29	35.2	27.29	0.21	1.00	4.76	X		BEAM IS ADEQUATE.
BS-5	FIG. 2	W12X27 ADMINISTRATION BLDG. ROOF	2.96	66.1	22.3	5.70	13.50	2.38	1.15	35.2	30.61	.46	1.00	2.16	X		BEAM IS ADEQUATE.
BS-6	FIG. 2	"	—	—	—	—	—	—	—	—	—	—	—	—	X		ADEQUATE BY COMPARISON TO BS-5.
BS-7	FIG. 2	W12X27 + WT10.5X22 ON BOT. FLNG.	5.07	149.4	29.5	14.34	38.4	2.68	2.05	35.2	17.2	.47	1.00	2.13	X		BEAM IS ADEQUATE.
BS-8	FIG. 2	W12X27 ADMINISTRATION BLDG. ROOF	—	—	—	—	—	—	—	—	—	—	—	—	X		ADEQUATE BY COMPARISON TO BS-5.
BS-9	FIG. 3	W14X30	5.9	86.2	14.61	6.60	22.9	3.47	0.59	15.33	26.0	0.34	1.00	2.96	X		BEAM IS ADEQUATE.

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_r - SAFETY FACTOR FOR SHEAR
4. SF_b - SAFETY FACTOR MOMENT
5. SF_a - SAFETY FACTOR FOR AXIAL LOAD
6. R_C - COMBINED STRESS FACTOR
7. R_A - ALLOWABLE COMBINED STRESS FACTOR
8. SF - OVERALL SAFETY FACTOR
9. f_a, f_{bx}, f_{by}, ETC. - SEE AISC STEEL CONSTRUCTION MANUAL 1980; KSI

FOOTNOTES: (DESIGNATED BY SUPERSCRIPTS)

1. INTERACTION EQS FOR STRUCTURAL STEEL:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

TABLE 4

STRUCTURAL STEEL BEAMS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_c	V_a	$SF_r = V_A/V_c$	f_{bx}	F_{bx}	$SF_b = F_{bx}/f_{bx}$	f_a	F_a	$SF_a = F_a/f_a$	R_c (1)	R_A	$SF = R_A/R_c$	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BS-10	FIG. 3	W14X30 CONTROL ROOM	—	—	—	—	—	—	—	—	—	—	—	—	X		BEAM IS ADEQUATE BY COMPARISON BS-9.
BS-11	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-12	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-13	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-14	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-15	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-16	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-17	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-18	FIG. 2	W16X36 ADMINISTRATION BLDG. ROOF	13.7	109.2	7.97	21.77	35.2	1.62	1.72	35.2	20.5	.81	1.00	1.24	X		BEAM IS ADEQUATE.
BS-19	FIG. 3	W16X36 CONTROL ROOM	13.7	109.2	7.97	6.92	20.8	3.01	1.55	13.26	8.56	.39	1.00	2.54	X		BEAM IS ADEQUATE.
BS-20	FIG. 3	"	--	--	--	--	--	--	--	--	--	--	--	--	X		BEAM IS ADEQUATE BY COMPARISON TO BS-19.

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_r - SAFETY FACTOR FOR SHEAR
4. SF_b - SAFETY FACTOR MOMENT
5. SF_a - SAFETY FACTOR FOR AXIAL LOAD
6. R_C - COMBINED STRESS FACTOR
7. R_A - ALLOWABLE COMBINED STRESS FACTOR
8. SF - OVERALL SAFETY FACTOR
9. f_a, f_{bx}, f_{by}, ETC. - SEE AISC STEEL CONSTRUCTION MANUAL 1980; KSI

FOOTNOTES: (DESIGNATED BY SUPERSCRIPTS)

1. INTERACTION E_{QS} FOR STRUCTURAL STEEL:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

TABLE 4 (CONT.)

STRUCTURAL STEEL BEAMS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_c	V_a	$SF_r = V_A/V_c$	f_{bx}	F_{bx}	$SF_b = F_{bx}/f_{bx}$	f_a	F_a	$SF_a = F_a/f_a$	R_c [1]	R_A	$SF = R_A/R_c$	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
BS-21	FIG. 3	W16X36 CONTROL ROOM	13.7	109.2	7.97	5.10	25.3	4.96	.45	17.41	38.7	.23	1.00	4.44	X		BEAM IS ADEQUATE.
BS-22	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		BEAM IS ADEQUATE BY COMPARISON TO BS-21.
BS-23	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-24	FIG. 3	W16X64 CONTROL ROOM	9.3	163.3	17.56	7.70	23.63	3.07	1.22	9.16	7.50	.37	1.00	2.71	X		BEAM IS ADEQUATE.
BS-25	FIG. 2	W12X40 CLASSROOM	4.60	80.9	17.59	11.16	23.90	2.14	1.78	11.60	6.52	.57	1.00	1.76	X		BEAM IS ADEQUATE.
BS-26	FIG. 3	W21X68 CONTROL ROOM	41.7	209	5.02	12.87	35.20	2.74	2.96	25.4	8.60	.58	1.00	1.74	X		BEAM IS ADEQUATE.
BS-27	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		BEAM IS ADEQUATE BY COMPARISON TO BS-26.
BS-28	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-29	FIG. 3	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"
BS-30	FIG. 3	W24X76 CONTROL ROOM	23.2	242	10.43	10.95	31.5	2.87	.44	23.6	53.6	.36	1.00	2.75	X		BEAM IS ADEQUATE.

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_r - SAFETY FACTOR FOR SHEAR
4. SF_b - SAFETY FACTOR MOMENT
5. SF_a - SAFETY FACTOR FOR AXIAL LOAD
6. R_C - COMBINED STRESS FACTOR
7. R_A - ALLOWABLE COMBINED STRESS FACTOR
8. SF - OVERALL SAFETY FACTOR
9. f_a, f_{bx}, f_{by}, ETC. - SEE AISC STEEL CONSTRUCTION MANUAL 1980; KSI

FOOTNOTES: (DESIGNATED BY SUPERSCRIPTS)

1. INTERACTION EQS FOR STRUCTURAL STEEL:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

TABLE 4 (CONT.)

STRUCTURAL STEEL COLUMNS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_c	V_a	$SF_r = V_A/V_c$	f_{bx}	F_{bx}	$SF_b = F_{bx}/f_{bx}$	f_a	F_a	$SF_a = F_a/f_a$	R_c (1)	R_A	$SF = R_A/R_c$	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
CS-1	FIG. 2	TS 6X3X½	—	—	—	—	—	—	13.29	13.89	1.05	—	—	—	X		COLUMN IS ADEQUATE.
CS-2	FIG. 2	3" DIA. XS PIPE COLUMN. COMMUNICATIONS ROOM	—	—	—	—	—	—	6.32	16.82	2.66	—	—	—	X		COLUMN IS ADEQUATE.
CS-3	FIG. 2	W12x65 COLUMNS	1.7	108.9	64.1	1.46	35.20	24.11	7.52	23.78	3.16	0.41	1.00	2.46	X		COLUMN IS ADEQUATE.
CS-4	FIG. 2	SWITCHGEAR AND CABLE SPREADING ROOM	—	—	—	—	—	—	—	—	—	—	—	—	X		COLUMN IS ADEQUATE BY COMPARISON WITH CS-3.
CS-5	FIG. 2	"	—	—	—	—	—	—	—	—	—	—	—	—	X		"

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_r - SAFETY FACTOR FOR SHEAR
4. SF_b SAFETY FACTOR MOMENT
5. SF_a - SAFETY FACTOR FOR AXIAL LOAD
6. R_C - COMBINED STRESS FACTOR
7. R_A - ALLOWABLE COMBINED STRESS FACTOR
8. SF - OVERALL SAFETY FACTOR
9. f_a, f_{bx}, f_{by}, ETC. - SEE AISC STEEL CONSTRUCTION MANUAL 1980; KSI

FOOTNOTES: (DESIGNATED BY SUPERSCRIPTS)

1. INTERACTION EQS FOR STRUCTURAL STEEL:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

TABLE 5

CONNECTIONS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	f_b	F_b	$SF_b = F_b/f_b$	P_C	P_A	$SF_P = P_A/P_C$	R_C	R_A	$SF = R_A/R_C$	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
		CLIP ANGLE TO WEB FOR W12X14, W14X30, W16X36, W16X64 AND W21X68 BEAMS.															
		3/4" DIA. A325F BOLTS THROUGH BEAM WEB.	5.13	12.32	2.40	-	-	-	-	-	-	-	-	-	X		CONNECTIONS ARE ADEQUATE
		3/4" DIA. A325F BOLTS THROUGH OUTSTANDING LEGS OF CLIP ANGLE.	5.21	12.32	2.36	-	-	-	8.84	23.88	2.70	-	-	-	X		CONNECTIONS ARE ADEQUATE

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR, FOR SHEAR
4. f_b - CALCULATED BENDING STRESS, KSI
5. F_b - ALLOWABLE BENDING STRESS, KSI
6. SF_b - SAFETY FACTOR FOR BENDING STRESS
7. P_C - CALCULATED AXIAL LOAD, KIPS
8. P_A - ALLOWABLE AXIAL LOAD, KIPS
9. SF_P - SAFETY FACTOR FOR AXIAL LOAD
10. f_b, F_b, KSI, ETC.- SEE AISC CONSTRUCTION MANUAL, 1980.

CONNECTIONS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	f_b	F_b	$SF_b = F_b/f_b$	P_C	P_A	$SF_P = P_A/P_C$	R_C	R_A	$SF = R_A/R_C$	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
		CLIP ANGLE TO INSERT PLATE FOR W12X27 & W16X36															
		3/4" DIA. A325F BOLTS THROUGH BEAM WEB.	3.34	12.32	3.68	-	-	-	-	-	-	-	-	-	X		CONNECTIONS ARE ADEQUATE
		A36 STEEL INSERT PLATE	-	-	-	19.98	43.2	2.16	-	-	-	-	-	-	X		CONNECTIONS ARE ADEQUATE
		3/4 DIA. A307 ANCHORS ON INSERT PLATE	6.46	7.66	1.19	-	-	-	4.91	13.12	2.67	0.94	1.00	1.06	X		CONNECTIONS ARE ADEQUATE
		CLIP ANGLE TO WEB AND CONCRETE FOR W12X40 BEAM.															
		3/4" DIA. A325F BOLTS THROUGH BEAM WEB.	3.59	8.83	2.46	-	-	-	-	-	-	-	-	-	X		CONNECTIONS ARE ADEQUATE
-		3/4" DIA. CONCRETE FASTENERS.	0.77	6.12	7.95	-	-	-	3.51	4.00	1.14	0.79	1.00	1.27	X		CONNECTIONS ARE ADEQUATE

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR, FOR SHEAR
4. f_b - CALCULATED BENDING STRESS, KSI
5. F_b - ALLOWABLE BENDING STRESS, KSI
6. SF_b - SAFETY FACTOR FOR BENDING STRESS
7. P_C - CALCULATED AXIAL LOAD, KIPS
8. P_A - ALLOWABLE AXIAL LOAD, KIPS
9. SF_P - SAFETY FACTOR FOR AXIAL LOAD
10. f_b, F_b, KSI, ETC. - SEE AISC CONSTRUCTION MANUAL, 1980.

CONNECTIONS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	f_b	F_b	$SF_b = F_b/f_b$	P_C	P_A	$SF_P = P_A/P_C$	R_C	R_A	$SF = R_A/R_C$	MEETS CRITERIA		REMARKS
															YES	NO	
		COLUMN CONNECTIONS 3/4" DIA. A307 ANCHOR BOLTS FOR:															
		W12X65 COLUMN	0.87	4.73	5.44	-	-	-	-	-	-	0.06	1.0	16.67	X		CONNECTIONS
		TS 6X3X1/4	-	-	-	-	-	-	22.7	84.4	3.72	-	-	-	X		ARE ADEQUATE
		BASE PLATES FOR:															
		3" DIA. XS PIPE	-	-	-	35.90	43.2	1.20	-	-	-	-	-	-	X		
		W12X65 COLUMN	-	-	-	13.93	43.2	3.10	-	-	-	-	-	-	X		CONNECTIONS
		TS 6X3X1/4	-	-	-	-	-	-	53.68	56.1	1.05	-	-	-	X		ARE ADEQUATE

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR, FOR SHEAR
4. f_b - CALCULATED BENDING STRESS, KSI
5. F_b - ALLOWABLE BENDING STRESS, KSI
6. SF_b - SAFETY FACTOR FOR BENDING STRESS
7. P_C - CALCULATED AXIAL LOAD, KIPS
8. P_A - ALLOWABLE AXIAL LOAD, KIPS
9. SF_P - SAFETY FACTOR FOR AXIAL LOAD
10. f_b, F_b, KSI, ETC.-SEE AISC CONSTRUCTION MANUAL, 1980.

MISCELLANEOUS ELEMENTS

IDENTIFICATION	REFERENCE FIG. NO	DESCRIPTION	V_C	V_A	M_C	M_A	P_C	P_A	MEETS CRITERIA YES	MEETS CRITERIA NO	REMARKS
		MAXIMUM SOIL BEARING PRESSURE	—	—	—	—	8.7	17.0	X		SOIL BEARING PRESSURE IS ACCEPTABLE.
		WALL FOOTINGS	—	—	9.8	26.4	—	—	X		WALL FOOTINGS ARE ADEQUATE.
		ISOLATED COLUMN FOOTINGS	145.0	480.2	8.1	21.0	13.6	17.0	X		SPREAD FOOTINGS ARE ADEQUATE.

NOTATION:

1. V_C CALCULATED SHEAR, KIPS
2. V_m ALLOWABLE SHEAR, KIPS
3. P_C CALCULATED COMPRESSIVE STRESS, KIPS/FT.^{2}
4. P_A ALLOWABLE COMPRESSIVE STRESS, KIPS/FT.^{2}
5. M_C CALCULATED MOMENT, KIP-FT/FT
6. M_A ALLOWABLE MOMENT, KIP-FT/FT

TABLE 7

MASONRY WALLS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	P_C	P_A	$SF_P = P_A/P_C$	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A	MEETS CRITERIA		REMARKS
														YES	NO	
WM-A-n	FIG. 2	NORTH SIDE OF ADMIN. BLDG. FACADE	60	60	1.00	—	—	—	8.36	3.16	0.38	4.0	3.0	X		WALL IS ADEQUATE. BY SIMILARITY (REFER TO SECTION 4.1.6)
WM-A-s	"	SOUTH SIDE OF ADMIN.	59	60	1.02	—	—	—	8.56	6.47	0.76	1.4	3.0	X		WALL IS ADEQUATE.
WM-8-a	"	NORTH SIDE OF BATTERY ROOM	85	81	0.96	—	—	—	20.37	7.54	0.37	4.1	3.0	X		WALL IS ADEQUATE BY SIMILARITY (REFER TO SECTION 4.1.6)

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR, PSI
4. V_A - ALLOWABLE SHEAR, PSI
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP
7. M_A - ALLOWABLE MOMENT, FEET-KIP
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u/\Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. P_C - CALCULATED AXIAL LOAD, KIPS
12. P_A ALLOWABLE AXIAL LOAD, KIPS
13. SF_P SAFETY FACTOR FOR AXIAL LOAD

TABLE 8

REINFORCED CONCRETE WALLS

NON-CRITICAL PORTION
OF THE BUILDING.

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	UF	M_C	M_A	$SF_M = M_A/M_C$	D_C	D_A
NWC-1	FIG. 8	REINFORCED CONCRETE WALL AT NORTH END OF ADIMINISTRATION BLDG. THICKNESS: 1'-1".	7.74	16.42	2.12		8.78	6.04	0.69	1.56	3.0
NWC-C	FIG. 8	REINFORCED CONCRETE WALL AT NORTH-WEST END OF ADMIN. BLDG. THICKNESS: 8"	15.48	16.42	1.06		1.94	2.35	1.21		

NOTATION:

1. Δy - DEFLECTION AT YIELD OF REINFORCEMENT
2. Δu - MAXIMUM DEFLECTION
3. V_C - CALCULATED SHEAR STRESS, KIPS/FT.^{2}
4. V_A - ALLOWABLE SHEAR STRESS, KIPS/FT.^{2}
5. SF_V - SAFETY FACTOR FOR SHEAR
6. M_C - CALCULATED MOMENT, FEET-KIP/FT
7. M_A - ALLOWABLE MOMENT, FEET-KIP/FT
8. SF_m - SAFETY FACTOR FOR MOMENT
9. D_C - CALCULATED DUCTILITY RATIO, $\Delta u / \Delta y$
10. D_A - ALLOWABLE DUCTILITY RATIO
11. U.F. - UTILITY FACTOR, PERCENT OF THE INTERACTION CAPACITY BEING UTILIZED.

STRUCTURAL STEEL BEAMS

NON-CRITICAL PORTION OF THE BUILDING

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_c	V_a	$SF_r = V_A/V_c$	f_{bx}	F_{bx}	$SF_b = F_{bx}/f_{bx}$	f_a	F_a	$SF_a = F_a/f_a$	R_c (1)	R_A	$SF = R_A/R_c$
NBS-1	FIG. 8	W12 X 14 ADMIN. ROOF	2.18	54.9	25.18	13.99	38.4	2.74	1.82	35.2	19.34	0.41	1.0	2.42
NBS-2	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-3	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-4	"	W12 X 22 ADMIN. ROOF	9.85	73.7	7.48	35.0	38.4	1.10	2.59	35.2	13.59	0.99	1.0	1.01
NBS-5	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-6	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-7	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-8	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-9	"		-	-	-	-	-	-	-	-	-	-	-	-

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_r - SAFETY FACTOR FOR SHEAR
4. SF_b - SAFETY FACTOR MOMENT
5. SF_a - SAFETY FACTOR FOR AXIAL LOAD
6. R_C - COMBINED STRESS FACTOR
7. R_A - ALLOWABLE COMBINED STRESS FACTOR
8. SF - OVERALL SAFETY FACTOR
9. f_a, f_{bx}, f_{by}, ETC. - SEE AISC STEEL CONSTRUCTION MANUAL 1980; KSI

FOOTNOTES: (DESIGNATED BY SUPERSCRIPTS)

1. INTERACTION EQS FOR STRUCTURAL STEEL:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

TABLE 10

STRUCTURAL STEEL BEAMS

NON-CRITICAL PORTION OF THE BUILDING

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_c	V_a	$SF_r = V_A/V_c$	f_{bx}	F_{bx}	$SF_b = F_{bx}/f_{bx}$	f_a	F_a	$SF_a = F_a/f_a$	R_c (1)	R_A	$SF = R_A/R_c$
NBS-10	FIG. 8	W12 X 27	3.82	66.1	17.30	17.14	38.4	2.24	2.69	35.2	13.1	0.53	1.0	1.90
NBS-11	"	ADMIN. ROOF	-	-	-	-	-	-	-	-	-	-	-	-
NBS-12	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-13	"		-	-	-	39.75	38.4	0.97	1.92	35.2	18.33	1.13	1.0	0.89
NBS-14	"		-	-	-	20.91	38.4	1.84	1.15	35.2	30.69	0.61	1.0	1.63
NBS-15	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-16	"	W16 X 36 ADMIN. ROOF	7.03	109.2	15.53	10.41	13.82	1.33	1.27	35.2	27.7	0.79	1.0	1.26
NBS-17	"		13.67	109.2	7.99	21.77	35.2	1.62	1.72	35.2	20.5	0.81	1.0	1.24
NBS-18	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-19	"		-	-	-	-	-	-	-	-	-	-	-	-
NBS-20	"		-	-	-	-	-	-	-	-	-	-	-	-

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_r - SAFETY FACTOR FOR SHEAR
4. SF_b - SAFETY FACTOR MOMENT
5. SF_a - SAFETY FACTOR FOR AXIAL LOAD
6. R_C - COMBINED STRESS FACTOR
7. R_A - ALLOWABLE COMBINED STRESS FACTOR
8. SF - OVERALL SAFETY FACTOR
9. f_a, f_{bx}, f_{by}, ETC. - SEE AISC STEEL CONSTRUCTION MANUAL 1980; KSI

FOOTNOTES: (DESIGNATED BY SUPERSCRIPTS)

1. INTERACTION EQs FOR STRUCTURAL STEEL:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}, \text{ OR } \frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

TABLE 10. (CONT.)

NON-CRITICAL PORTION
OF THE BUILDING

STRUCTURAL STEEL COLUMNS

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_c	V_a	$SF_r = V_A/V_c$	f_{bx}	F_{bx}	$SF_b = F_{bx}/f_{bx}$	f_a	F_a	$SF_a = F_a/f_a$	R_c (1)	R_A	$SF = R_A/R_c$
NCS-1	FIG. 10	TS 6"X3"X1/4"				-	-	-	16.34	13.88	0.85	-	-	-
	"	COLUMN												
NCS-2		ADMIN. BLDG	-	-	-	-	-	-	16.83	13.88	0.83	-	-	-
NCS-3	"	FOYER	-	-	-	-	-	-	15.10	13.88	0.92	-	-	-

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_r - SAFETY FACTOR FOR SHEAR
4. SF_b - SAFETY FACTOR MOMENT
5. SF_a - SAFETY FACTOR FOR AXIAL LOAD
6. R_C - COMBINED STRESS FACTOR
7. R_A - ALLOWABLE COMBINED STRESS FACTOR
8. SF - OVERALL SAFETY FACTOR
9. f_a, f_{bx}, f_{by}, ETC. - SEE AISC STEEL CONSTRUCTION MANUAL 1980; KSI

FOOTNOTES: (DESIGNATED BY SUPERSCRIPTS)

1. INTERACTION EQs FOR STRUCTURAL STEEL:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1-\frac{f_a}{F'_{ex}})F_{bx}} + \frac{C_{my} f_{by}}{(1-\frac{f_a}{F'_{ey}})F_{by}}, \text{ OR } \frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}, \text{ OR } \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}$$

CONNECTIONS

NON-CRITICAL PORTION
OF THE BUILDING

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	f_b	F_b	$SF_b = F_b/f_b$	P_C	P_A	$SF_P = P_A/P_C$	R_C	R_A	$SF = R_A/R_C$
		STANDARD CLIP ANGLE CONNECTIONS FOR W12X14, W12X22, W12X27 & W16X36 MEMBERS.												
		3/4" DIA. A325F BOLTS THROUGH OUTSTANDING LEG OF CLIP ANGLE.	5.44	12.32	2.26	-	-	-	-	-	-	-	-	-
		3/4" DIA. A325F BOLTS THROUGH WEB OF BEAM AND CLIP ANGLE	5.91	12.32	2.08	-	-	-	4.19	31.1	7.42	-	-	-

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR, FOR SHEAR
4. f_b - CALCULATED BENDING STRESS, KSI
5. F_b - ALLOWABLE BENDING STRESS, KSI
6. SF_b - SAFETY FACTOR FOR BENDING STRESS
7. P_C - CALCULATED AXIAL LOAD, KIPS
8. P_A - ALLOWABLE AXIAL LOAD, KIPS
9. SF_P - SAFETY FACTOR FOR AXIAL LOAD
10. f_b, F_b, KSI, ETC. - SEE AISC CONSTRUCTION MANUAL, 1980.

TABLE 12

CONNECTIONS

NON-CRITICAL PORTION
OF THE BUILDING

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	f_b	F_b	$SF_b = F_b/f_b$	P_C	P_A	$SF_P = P_A/P_C$	R_C	R_A	$SF = R_A/R_C$
		BEAM TO INSERT PLATE CONNECTION FOR W12X27	2.74	12.32	4.50	6.77	43.2	6.38	3.89	13.12	3.37	0.43	1.00	2.33
		BEAM TO INSERT PLATE CONNECTIONS FOR W16X36	3.47	12.32	3.55	15.36	43.2	2.81	4.87	13.12	2.69	0.82	1.00	1.22
		BEAM POCKET CONNECTION AT EAST END OF NBS-17	1.45	2.32	1.60	—	—	—	6.84	6.08	0.89	1.67	1.0	0.60

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR, FOR SHEAR
4. f_b - CALCULATED BENDING STRESS, KSI
5. F_b - ALLOWABLE BENDING STRESS, KSI
6. SF_b - SAFETY FACTOR FOR BENDING STRESS
7. P_C - CALCULATED AXIAL LOAD, KIPS
8. P_A - ALLOWABLE AXIAL LOAD, KIPS
9. SF_P - SAFETY FACTOR FOR AXIAL LOAD
10. f_b, F_b, KSI, ETC. - SEE AISC CONSTRUCTION MANUAL, 1980.

CONNECTIONS

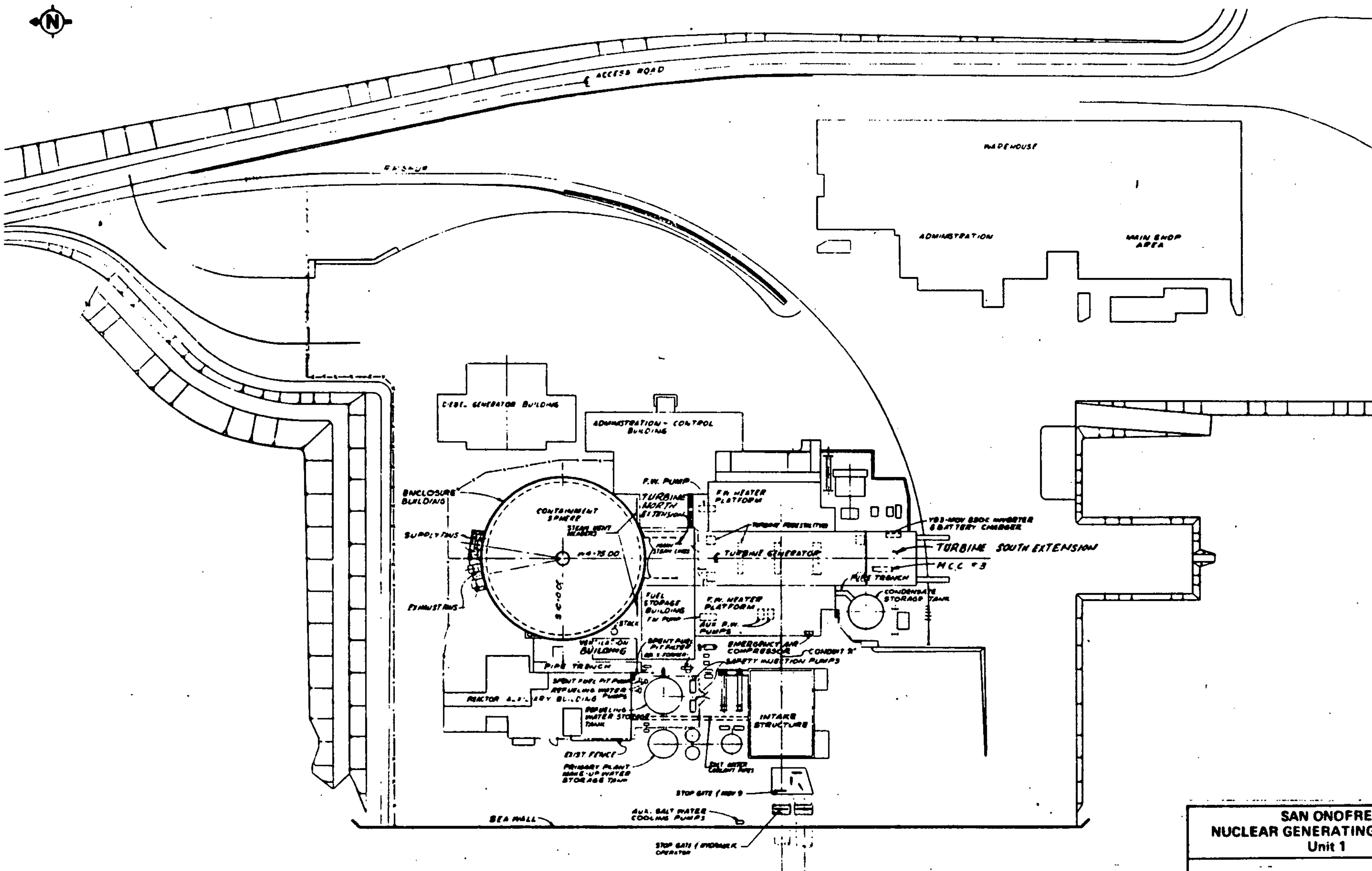
NON-CRITICAL PORTION OF THE BUILDING

IDENTIFICATION	REFERENCE FIG. NO.	DESCRIPTION	V_C	V_A	$SF_V = V_A/V_C$	f_b	F_b	$SF_b = F_b/f_b$	P_C	P_A	$SF_P = P_A/P_C$	R_C	R_A	$SF = R_A/R_C$
		CAP PLATE CONNECTION FOR TS 6X3X1/4 COLUMNS												
		3/4" DIA. A325F BOLTS THROUGH BEARING PLATE.	-	-	-	-	-	-	38.20	62.20	1.63	-	-	-
		COLUMN CAP PLATE	-	-	-	38.70	43.20	1.12	-	-	-	-	-	-
		BASE CONNECTION FOR TS 6X3X1/4 COLUMNS	-	-	-	-	-	-	38.2	84.4	2.22	-	-	-

NOTATION:

1. V_C - CALCULATED SHEAR, KIPS
2. V_A - ALLOWABLE SHEAR, KIPS
3. SF_V - SAFETY FACTOR, FOR SHEAR
4. f_b - CALCULATED BENDING STRESS, KSI
5. F_b - ALLOWABLE BENDING STRESS, KSI
6. SF_b - SAFETY FACTOR FOR BENDING STRESS
7. P_C - CALCULATED AXIAL LOAD KIPS
8. P_A - ALLOWABLE AXIAL LOAD, KIPS
9. SF_P - SAFETY FACTOR FOR AXIAL LOAD
10. f_b, F_b, KSI, ETC. - SEE AISC CONSTRUCTION MANUAL, 1980.

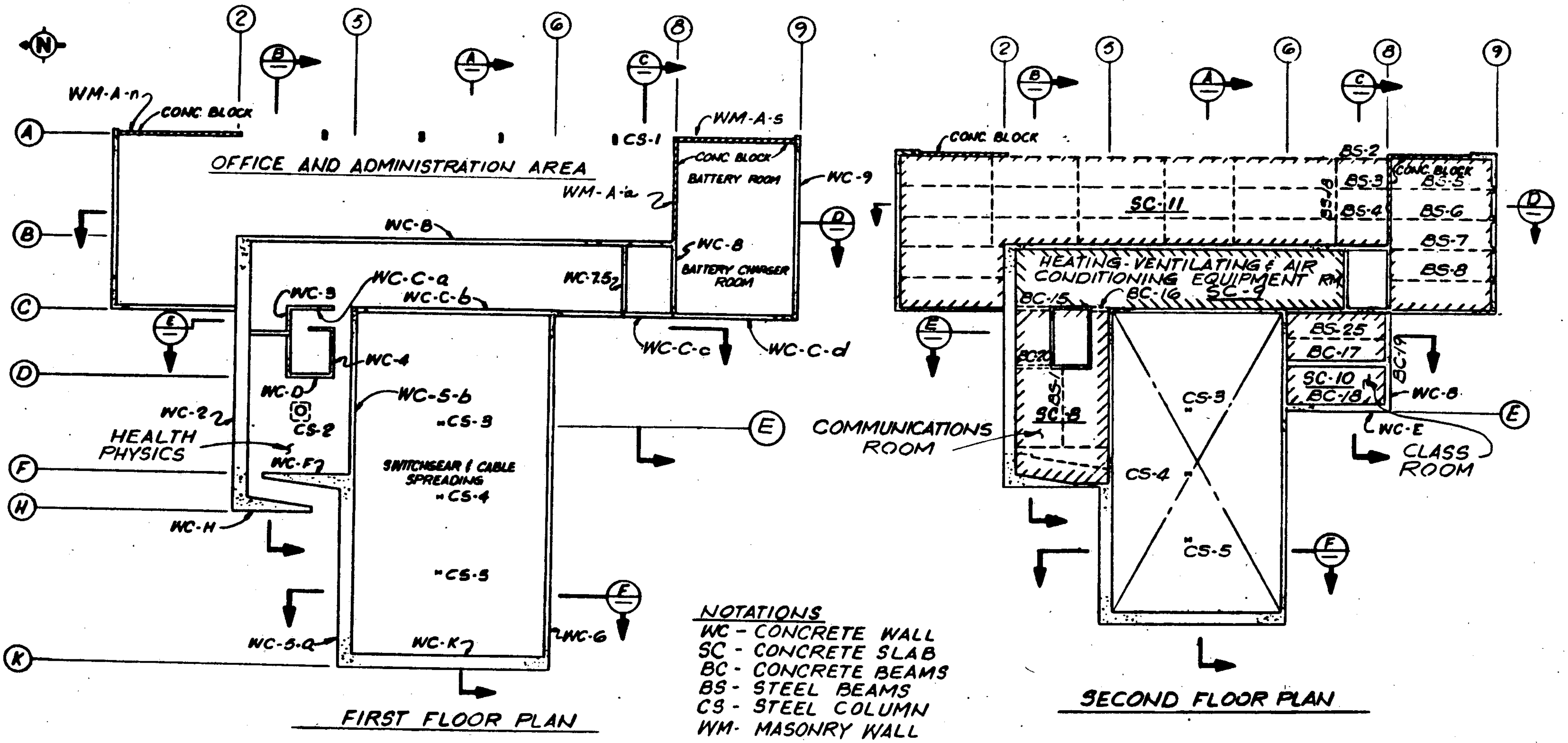
TABLE 12 (CONT.)



SAN ONOFRE NUCLEAR GENERATING STATION Unit 1

GENERAL PLOT PLAN

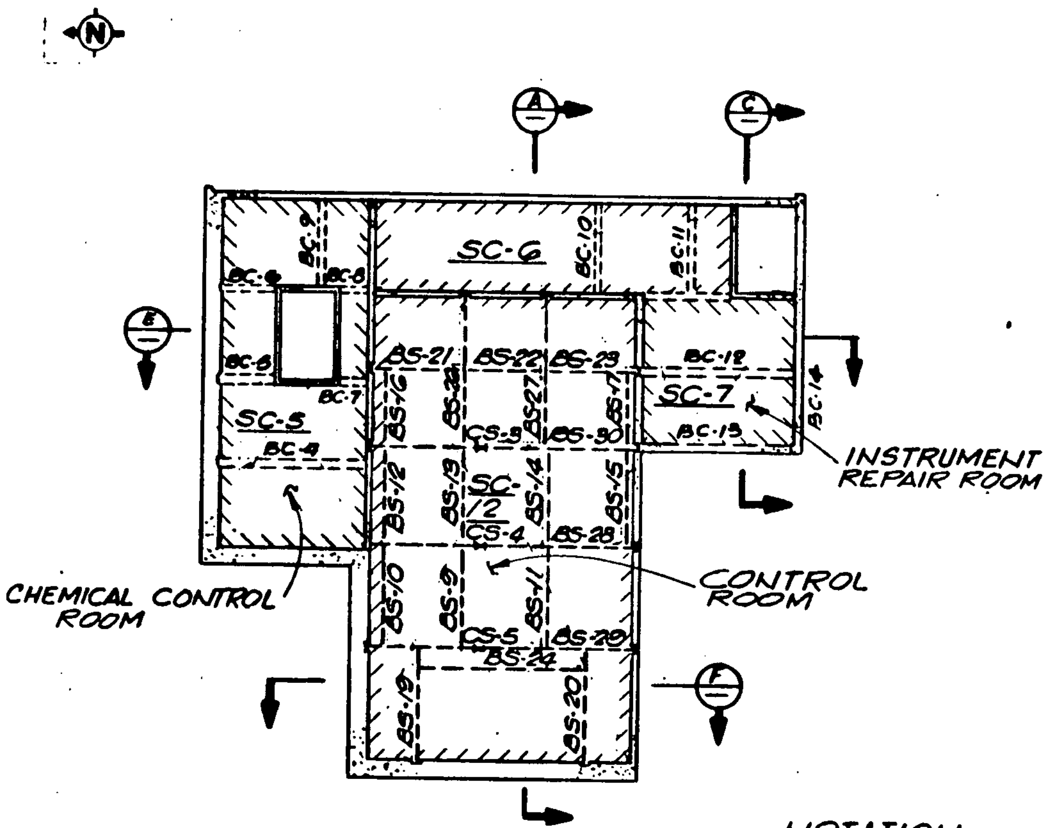
Figure 1



SAN ONOFRE
NUCLEAR GENERATING STATION
Unit 1

CONTROL/ADMINISTRATION PLANS

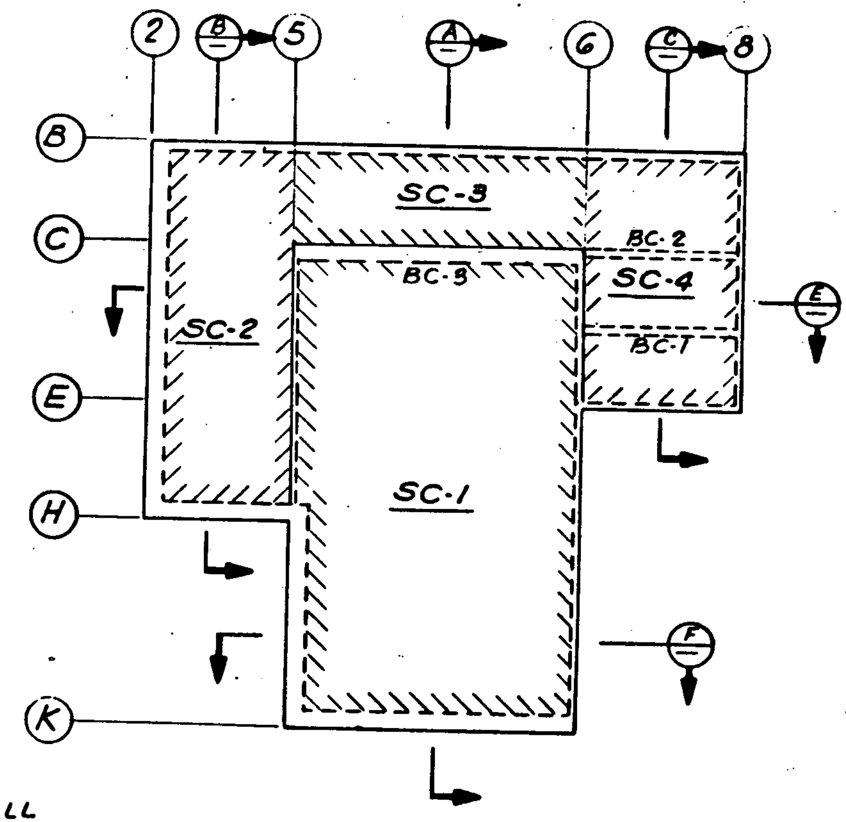
Figure 2



THIRD FLOOR PLAN

NOTATION

WC - CONCRETE WALL
SC - CONCRETE SLAB
BC - CONCRETE BEAMS
BS - STEEL BEAMS
CS - STEEL COLUMN

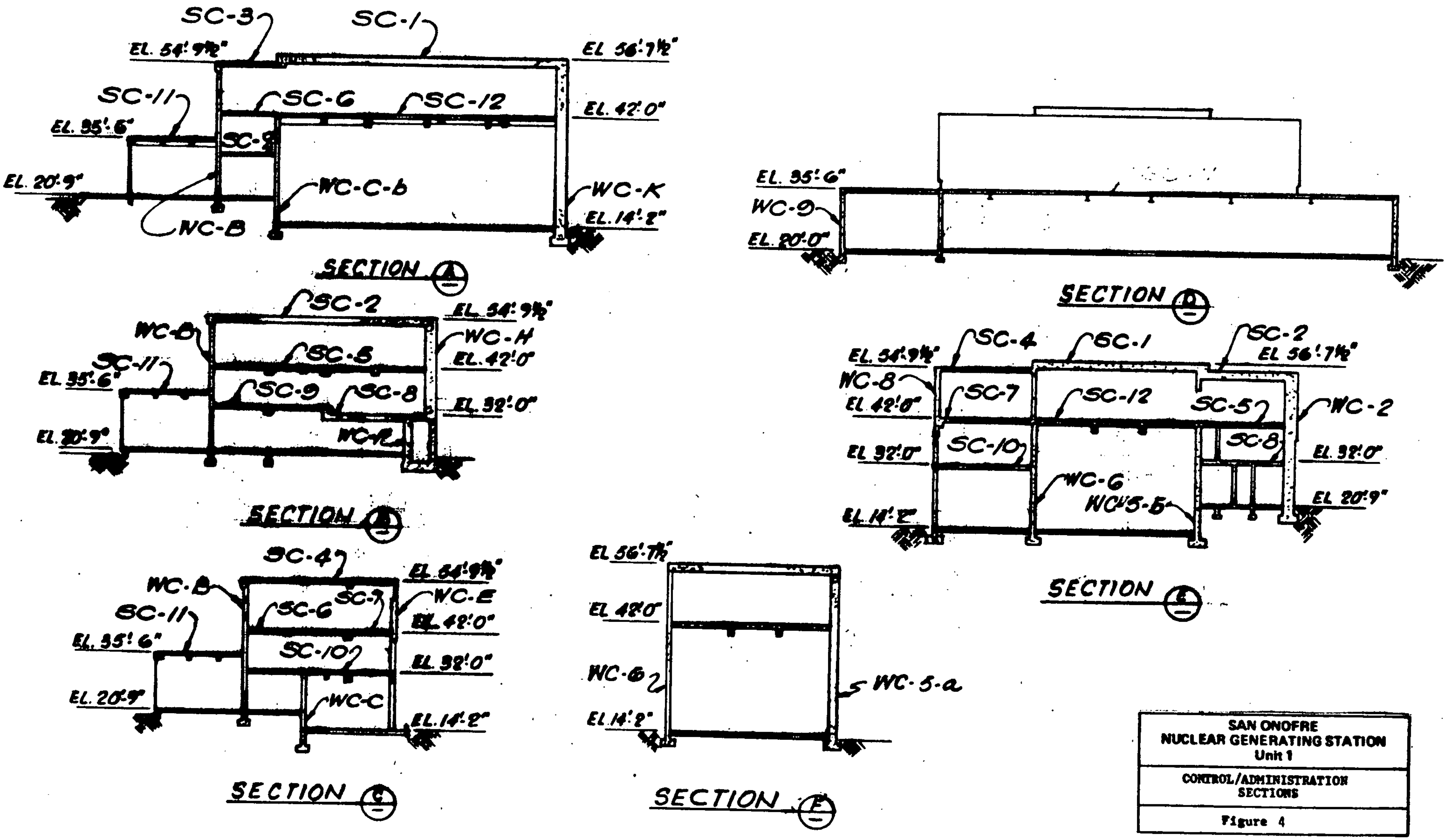


ROOF PLAN

SAN ONOFRE
NUCLEAR GENERATING STATION
Unit 1

CONTROL/ADMINISTRATION
PLANS

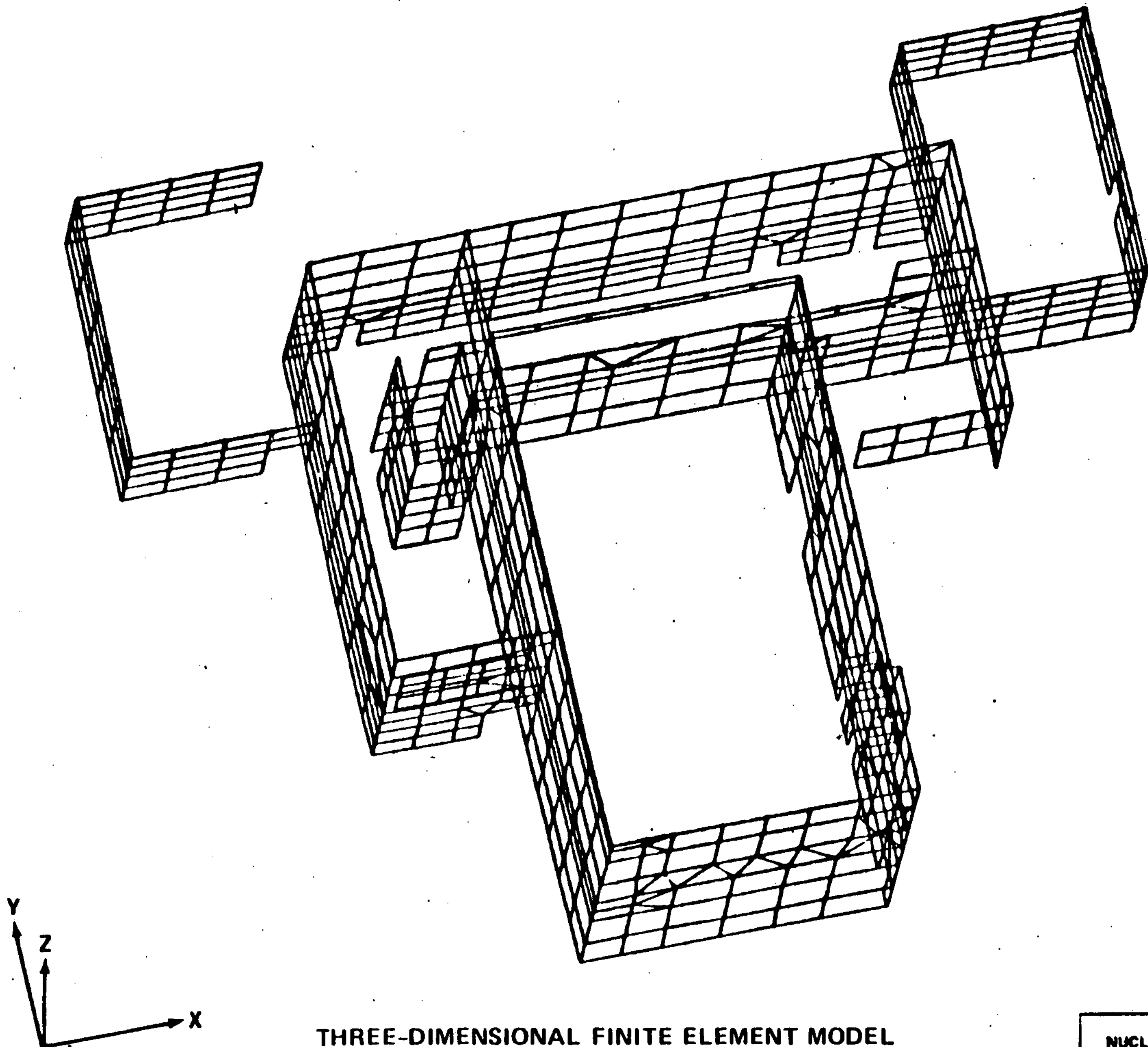
Figure 3



SAN ONOFRE
NUCLEAR GENERATING STATION
Unit 1

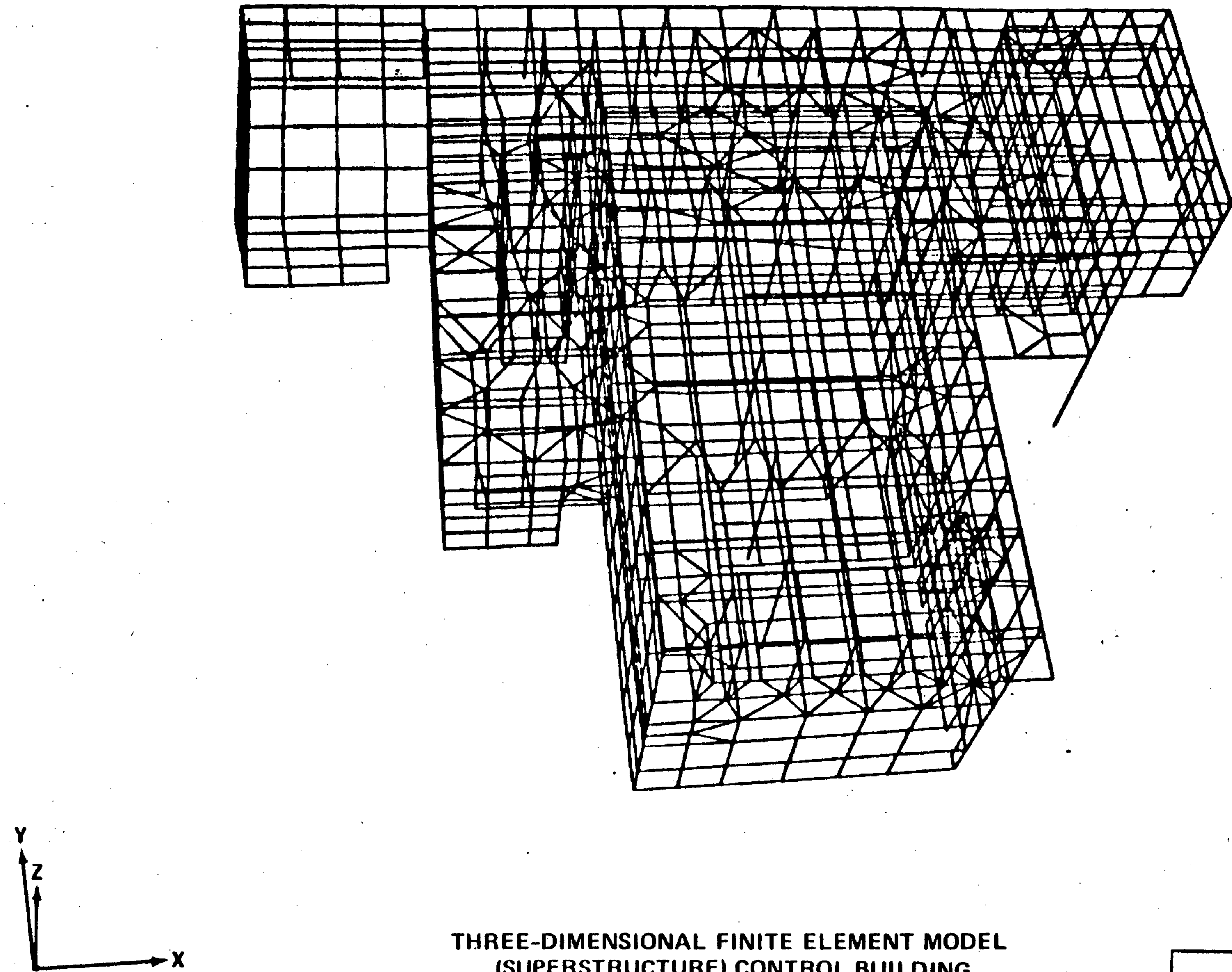
CONTROL/ADMINISTRATION
SECTIONS

Figure 4



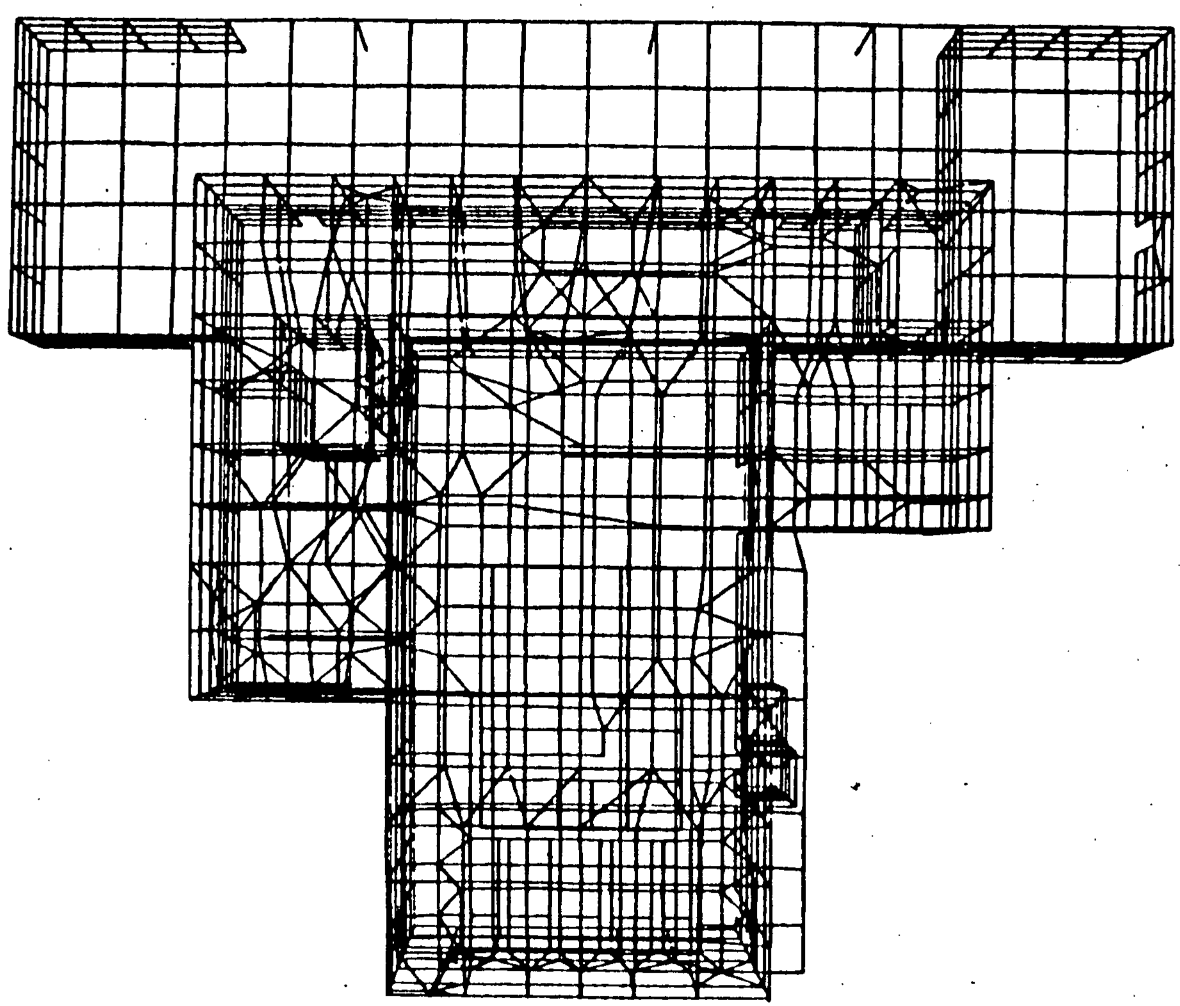
THREE-DIMENSIONAL FINITE ELEMENT MODEL
(WALLS ONLY) CONTROL BUILDING

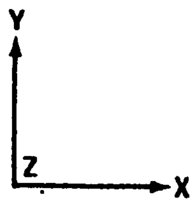
SAN ONOFRE NUCLEAR GENERATING STATION Unit 1
CONTROL/ADMINISTRATION MODEL
Figure 5



THREE-DIMENSIONAL FINITE ELEMENT MODEL
(SUPERSTRUCTURE) CONTROL BUILDING

SAN ONOFRE NUCLEAR GENERATING STATION Unit 1
CONTROL/ADMINISTRATION MODEL
Figure 6



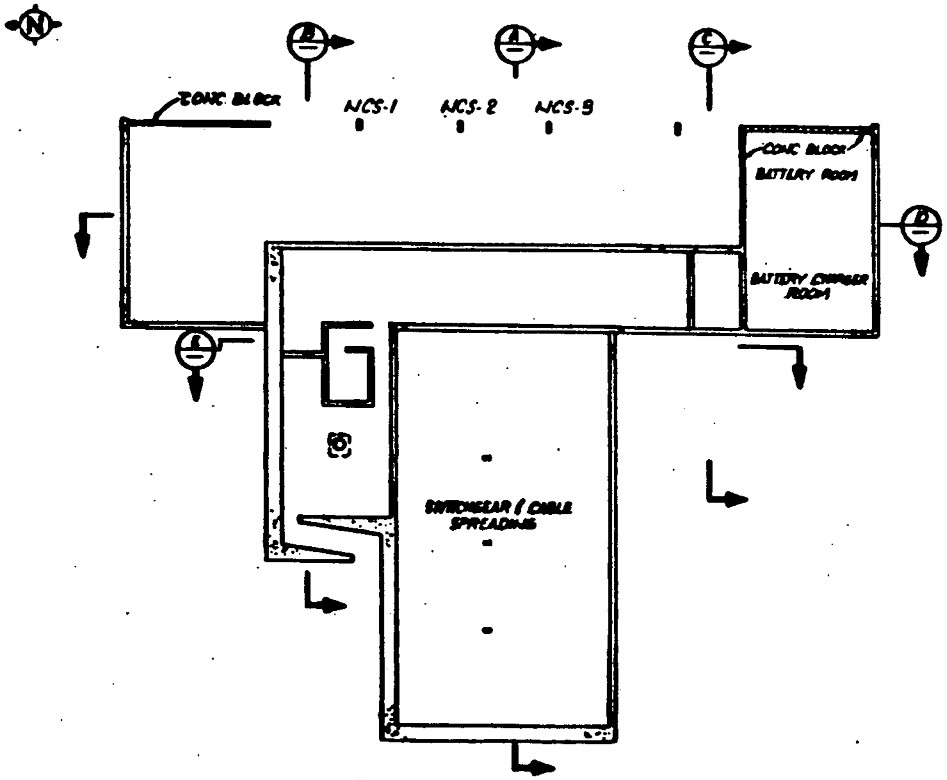


CONTROL BUILDING MODEL

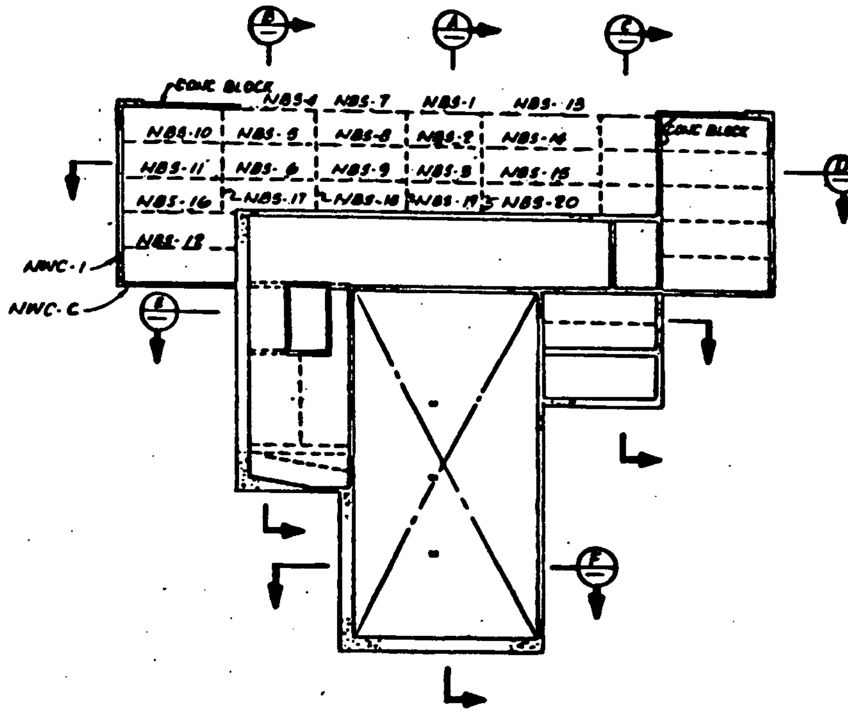
SAN ONOFRE
NUCLEAR GENERATING STATION
Unit 1

CONTROL/ADMINISTRATION
MODEL

Figure 7



FIRST FLOOR PLAN



SECOND FLOOR PLAN

SAN ONOFRE NUCLEAR GENERATING STATION Unit 1
CONTROL/ADMINISTRATION PLANS
Figure 8