EVALUATION OF STEEL-SIDED PORTION OF REACTOR AND TURBINE BUILDINGS FOR BLAST LOAD FROM THE HYDROGEN WATER CHEMISTRY FACILITY

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1. INTRODUCTION

The purpose of this report is to evaluate the steel-sided portions of the reactor and turbine buildings for the loads generated by a postulated explosion of the liquid or gaseous hydrogen stored at the hydrogen water chemistry (HWC) facility located east of the cooling towers.

The liquid hydrogen storage tank and gaseous hydrogen receiver bank of the HWC facility are remotely located to lessen the impact of a hydrogen explosion on the safety-related structures. The reinforced concrete walls of the safety-related structures are not damaged by the postulated blast. This result is based on review of the EPRI Guidelines for Permanent BWR Hydrogen Water Chemistry Installations (Reference 1). The guidelines, however, do not cover the effect of a blast on steel framed structures.

A structural analysis is performed for the reactor building enclosure to provide assurance against its collapse onto a safety-related structure, system, or component due to a hydrogen blast. The turbine building is analyzed by similarity.

2. CONCLUSIONS

It is concluded that the reactor building enclosures will satisfy the evaluation criteria limits due to the postulated hydrogen blast without modification. Local areas of the structure will be stressed beyond the elastic limit and may suffer minor permanent deformations. Similarly, the analysis indicates that the turbine building has sufficient strength and ductility to resist the postulated blast load. Therefore, no modification is required for the turbine building with local areas stressed beyond the elastic limit but within the evaluation criteria limit.

In summary the structural integrity of both the turbine and reactor building enclosures is maintained during the postulated hydrogen explosion. The HWC blast loading does not impact the safety function of plant structures, systems, or components.

3. DESCRIPTION OF STRUCTURES AND BLAST LOADING

3.1 General Arrangement

The reactor building consists of a reinforced concrete shear wall structure supported on a basemat and topped with an enclosure building. The enclosure building is a structural steel, braced frame structure enclosed with common metal siding supported by wind girts attached to the structural steel. The enclosure building also supports the runway girders for the reactor bridge crane.

The turbine building is similar to the reactor building in that it is a concrete shear wall structure sitting on a basemat shared by the turbine pedestal and topped with an enclosure building. The enclosure building is similar in width to the reactor building but it is about 7 ft-4 in. higher and about twice as long. The structural members are of similar sizes and arrangement. The turbine building enclosure also supports runway girders for the turbine bridge crane. The dynamic characteristics of the two structures are considered to be similar.

As seen in Figure 1, the hydrogen tank is located at approximate site coordinate N9800, E6380. The section of the reactor building nearest to the tank is the southeast corner located at coordinate N9933, E5067. The distance from the tank to the structure is approximately 1325 feet. The distance from the tank to the southeast corner of the turbine building is approximately 1260 feet.

The hydrogen water chemistry facility holds 9000 gallons of liquid hydrogen and 39,000 standard cubic feet of gaseous hydrogen. An explosion of the liquid and gaseous hydrogen is equivalent to 12,330 lbs and 1060 lbs of TNT respectively. The facility also includes a liquid oxygen storage tank but the effects from an oxygen explosion represent a lesser hazard as discussed in Reference 1.

3.2 Characteristics of Blast

The primary result of a hydrogen explosion is the formation of a shock wave composed of a high-pressure shock front which expands radially outward from the center of the detonation. The intensity of the pressure decays as a function of distance and time. The magnitude and distribution of the blast load on the structure are a function of the following factors:

A. Explosive properties of the material (hydrogen),

B. amplification of the blast pressure due to its reflection off the ground,

C. distance of the explosion to the impacted structure,

D. angle of incidence of the blast pressure on the structure,

E. interaction of the blast pressure with the structure.

Army publication TM5-1300 (Reference 2) provides a procedure to calculate the blast magnitude and duration. The blast is categorized as an "unconfined surface-burst load" with the shock wave amplified due to ground reflections. The characteristics of the shock wave are summarized in Table 1. The incident shock front strikes the wall and causes the pressure to rise immediately from zero to the reflected pressure (p_r). Typical pressure-duration relationship is shown in Figure 2.

As the wave travels along the building, its roof, and both the north and south walls are subjected to the overpressure plus drag.

Table 1

CHARACTERISTICS OF SHOCK WAVE

Peak Incident Pressure (psi)	Peak Normal Reflected Pressure (psi)	Arrival Time (sec)	Duration (sec)	Wavelength (feet)
0.82	1.55	1.04	0.104	106

(Peak pressure on north face, south face and roof = 0.56 psi.)

4. METHOD OF ANALYSIS

4.1 Dynamic Analysis

The blast's shock wave is an impulse load with a relatively short duration of approximately one tenth of a second. An elastic analysis using dynamic load factors as presented by Biggs (Reference 3) is used to determine the behavior of the structure when impacted by the blast. To compute the overall dynamic load factor the structure is considered an undamped, linearly elastic, one degree-of-freedom system in the direction of loading. As seen in Figure 2, the blast load is treated as a triangular load pulse. A dynamic load factor (DLF) is determined for elements subjected to the triangular pulse. The DLF is dependent on the ratio of the load duration (t_o) to the period of the structure (T).

To determine the period of the overall structure, a free vibration analysis is performed for the reactor structure using a three-dimensional finite element model. The primary steel members are modeled with BEAM and TRUSS elements. The wall and roof panels are assumed to be rigid and are not modeled. The base, which represents the concrete structure, consists of a stick model composed of BEAM elements. The BOUNDARY elements represent the soil supporting the building. The reactor enclosure structure model and its deflected shape are shown in Figures 3 and 4.

To consider the roof or wall flexibility, separate frequency analyses are performed for the different walls and roof panels.

Based on these and the free vibration analyses, the overall system frequencies are determined. It was shown that the load transmitted to the frame from the girts was limited by the capacity of the girts, in the east wall.

For the turbine building analysis, the structural steel framing frequency obtained from the reactor enclosure building analysis were used rather than performing a separate modal analysis for the turbine building. Considering the similarities between the two buildings, this is a reasonable simplifying assumption.

Taking into account the system frequencies, the load magnitude and duration, and the load limit on the girts of east bay, the loading on the main structural frame members are determined using the elastic or inelastic dynamic analysis method given in Reference 3.

The dynamic characteristics of the reactor enclosure building frame is shown in Table 2. The dynamic characteristics of the turbine building taking into account wall/roof panel flexibilities, are shown in Table 3.

Results of the analyses considering the system frequencies and appropriate dynamic load factors are given in the following section.

Table 2

DYNAMIC CHARACTERISTICS OF REACTOR BUILDING FRAME

Direction	Fundamental Period (Seconds)	Dynamic Load Factor
E - W	0.29	0.95
N - S	0.41	0.70
Vertical	0.31	0.93

TABLE 3

DYNAMIC CHARACTERISTICS OF THE TURBINE BUILDING
INCLUDING WALL/ROOF FLEXIBILITIES

Direction	Fundamental Period (Seconds)	Dynamic Load Factor
East Wall	0.34	0.82
North/South Wall	0.44	0.64
Roof	0.33	0.83

4.2 Design Pressure

The design pressure is developed by combining the effective shock wave pressure acting on the building frame and the wall/roof panels. The effective shock wave pressure is determined by factoring the peak shock wave pressure by the corresponding dynamic load factors. The design pressures are summarized in Table 4. The equivalent design pressure is obtained per the following:

$$P_{eg} = \frac{P_{fr} A_{fr} + P_p A_p}{A_{fr} + A_p}$$

where

P_{eg} = Equivalent design pressure

P_{fr} = Effective shock wave pressure acting on the frame

A_{fr} = Area of the frame subject to P_{fr}

P_p = Effective shock wave pressure acting on the wall/roof panel

A_p = Area of the wall/roof panel subject to Pp

For comparison, the existing design windward pressure on the walls and the downward live load on the roof are also included in Table 4.

TABLE 4

DESIGN PRESSURE FOR THE TURBINE BUILDING

Direction	Equivalent Design Pressure	Existing Design Loads
East Wall	0.54* (78 psf)	28 to 40 psf Windward wind pressure
North/South Wall	0.36 (52 psf)	28 to 40 psf windward wind pressure
Roof	0.46 (66 psf)	50 psf live load

* Note: The dynamic pressure on the east wall is limited by the yielding capacity of the girls. For the analysis of the frame, this pressure is increased by 25% to account for possible higher yield strength.

Comparing the design pressures with the existing design loads on the building, the following conclusions can be reached:

For all walls and the roof, the design pressure from the blast are higher than the existing design pressure. Evaluation of the wall and roof structural components are required. (See Section 6 for results)

4.3 Loading Combination

The loading combination, which is in accordance with USNRC Regulatory Guide 1.91, used to check the structure and its components is:

$$C = D + L + T_o + R_o + B$$

where,

C = combined load effect

D = dead load

L = live load

T_o = thermal load during normal operation or shutdown

R_o = pipe reaction during normal operation or shutdown

B = blast load effects.

For the analysis of the reactor and turbine buildings, T_o and R_o are considered to equal zero.

5. ACCEPTANCE CRITERIA

For the purpose of this analysis, the structures are considered Seismic Class II/I structures designed to UBC criteria but assured against collapse due to extreme seismic or wind conditions.

The structures may undergo inelastic deformations as a consequence of the blast load. The calculated ductility of the individual structural elements must be such that they will be capable of absorbing the blast loads and prevent collapse of the structures. The allowable ductility factors are given in Table 5. Structural members are acceptable when the calculated ductility is less than the allowable value.

Table 5

ALLOWABLE MEMBER DUCTILITIES

MEMBER	μ_{ma}
Tension	10.0
Compression (Main Column)	See Footnote *
Compression (Braces)	6.0
Tension - Flexure	8.0
Flexure ($P/P_y < 0.15$)	10.0

Note: * For $Kl/r \geq 120$, $\mu = 1.0$ and for $Kl/r \leq 30$, $\mu = 4.0$.

A linear interpolation should be used between these limits.

For flexural members member ductility, μ_{mr} is calculated by:

$$\mu_{mr} = M/Mp$$

where,

M = equivalent moment calculated from computed angle of rotation = θEI

Mp = plastic moment capacity

Columns are checked in accordance with AISC Specification Part 2 (Reference 5), using an effective length factor (K) equal to, or greater than unity. In addition, the total axial force (P) must satisfy the following requirements:

$$P < 0.5P_y \quad \text{and} \quad P \leq 0.6P_{cr}$$

where,

P_y = $A_g f_y$ = axial yield load capacity

P_{cr} = $1.7A_g F_a$

A_g = gross cross-sectional area

f_y = yield strength

F_a = allowable compressive stress as determined in Part I of the AISC Specification.

The steel frame of both buildings is fabricated from ASTM A36 steel with a minimum yield strength of 36,000 psi.

For combined axial and flexural stresses on short axially loaded members, μ_{mr} is calculated by:

$$\mu_{mr} = P/P_y + M/M_p$$

where,

P, P_y, M and M_p are as defined previously.

For intermediate and long columns, the ductility ratios are based on the following interaction equation. This equation takes into account the secondary bending moments due to the lateral deflection of the columns, i.e., the P-Δ effect:

$$\mu_{mr} = P/P_u + C_m M/M_p(1 - P/P_e)$$

where,

P_u = ultimate axial capacity as provided by AISC, excluding the factor of safety

M_p = $0.9F_yZ$ for weak axis bending

= $[1.07 - (L/r_y)f_y/3160]M_p$ for strong axis bending

C_m = moment magnification factor due to secondary bending moments from the midspan deflection (P-Δ)

= $1 + \psi P/P_e$ where,

$\psi = \pi^2\delta_o EI/M_o L^2 - 1$

P_e = $12\pi^2 E/23(Kl_b/r_b)^2$

δ_o = maximum deflection

M = as defined previously

Connections are checked to provide assurance that they have sufficient strength to develop the ductility of the structural member. For members which are stressed beyond their elastic limit, the connection strength must be greater than the reaction necessary to develop the plastic capacity of the member. In general the connections use 7/8 in. diameter ASTM A325 high strength bolts. The capacity of the connection is determined by methods consistent with the AISC Specification.

The siding and roof decking is checked to determine if these elements remain intact during the blast. The roof is a commercial ribbed sheet metal decking with roofing materials covering it. The siding is a composite section made of a thin gage sheet metal interior and a ribbed aluminum exterior. The two panels are bolted together with sheet metal screws. Vertically, the siding of the reactor building is composed of two sections with a lap seam at mid-height. The allowable stresses of the siding is based on the design strength of the material as provided by the manufacturer, i.e., 20,000 psi for steel and 10,300 psi for aluminum. The maximum strength is based on the ultimate membrane strength which the siding and decking can resist. The connection of the siding to the girts is subjected only to shear due to membrane reactions since the siding spans across four girts and the pressure causes the siding to bear against the girts.

6. SUMMARY OF RESULTS

6.1 Reactor Enclosure Building

Minor overstressing is caused by the impact of the shock wave on the reactor enclosure structure from the postulated explosion of the 9000-gallon hydrogen tank. The brunt of the shock wave is taken by the easterly bay of the building. Figures 3 and 4 show the deformed shape of the structure.

The siding experiences deformations but has sufficient strength so that the panels do not fail. The wind girts supporting the siding in the east wall have sufficient capacity to withstand the blast load and remain within the allowable ductility limits. The wind girts in the other walls will remain within elastic limits.

The two corner columns and three wind columns on the east face experience the highest stresses due to the blast. As the wave passes by the structure the corner columns will be in a combined axial and biaxial bending state. The bridge crane is supported on its own columns which are adjacent and connected to the corner columns. The crane columns experience some lateral load transfer through the shear plates connecting them to the corner columns. The three interior wind columns of the east face see the most critical stress condition of all the columns since they have the longest unsupported length. These columns exceed their elastic limit under the blast loading but remain within the allowable ductility range. Thus the structural integrity of the structure is unimpaired.

Other members, specifically in the lower chord of the roof truss in the proximity of the east face, experience high compression, but no member fails or causes failure of the structure. All other structural components remain within the allowable working stress limits of the AISC.

For the reactor building enclosure, Table 6 summarizes the ductilities of the critical members.

The connections were evaluated and it was found that all the connections are within the allowable load limits.

6.2 Turbine Building Enclosure

The turbine building enclosure is analyzed as described above. The turbine building enclosure also experiences minor yielding due to the postulated explosion.

The columns of the turbine building are the same size as those of the reactor enclosure but are approximately 7 ft-4 in. longer. Due to their longer length, the applied moments increase and thus the required ductility increases.

The required ductilities for the governing structural elements of the turbine building are shown in Table 6.

As can be seen from this table, all required ductilities (ductility demand) are less than the allowable values. Therefore, it is concluded that the turbine building will maintain its structural integrity during a postulated blast.

TABLE 6

TURBINE BUILDING

REQUIRED MEMBER DUCTILITIES

Member Description	Ductility Radio	
	Required	Allowable
East Wall		
Girts	3.7	10
Corner Columns	1.1	10
Interior Columns	4.0	10
North-South Wall. Girts/Columns	remains elastic	-
Roof-Purlins	remains elastic	-

7. REFERENCES

1. EPRI, Guidelines for Permanent BWR Hydrogen Water Chemistry Installation, 1987 Revision.

2. "Structures to Resist the Effects of Accidental Explosions", Department of the Army Technical Manual TM5-1300, June 1969.

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4. USNRC Regulatory Guide 1.91, "Evaluation of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants (For Comments)", Revision 1, February 1978.

5. AISC Manual of Steel Construction, Eighth Edition, American Institute of Steel Construction, Inc.

6. Bechtel Design Guide C-2.33, "Simplified Inelastic Seismic Design of Nonsafety-Related Structures", Revision 2, July 1980.

7. Johnston, Bruce G.(editor), Guide to Stability Design Criteria for Metal Structures, Third Edition, 1976.

8. Monticello N.G.S. Calculations 245-C-7, 245-C-8, 245-C-9, and 245-C-11 "Evaluation of Steel-Sided Structures for Blast Loading (HWC)".

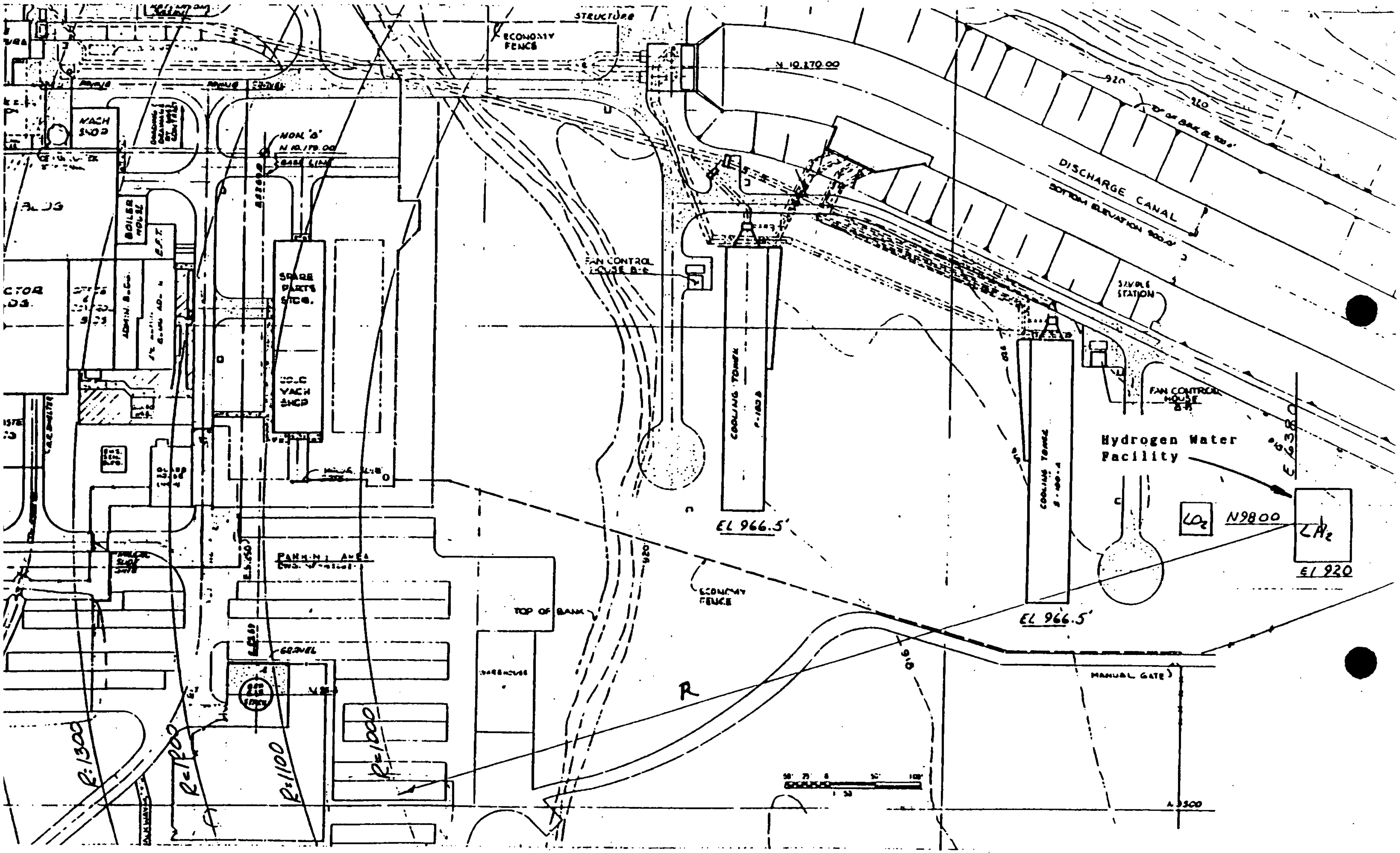

Figure 1

MONTICELLO N.G.P.
PLOT PLAN

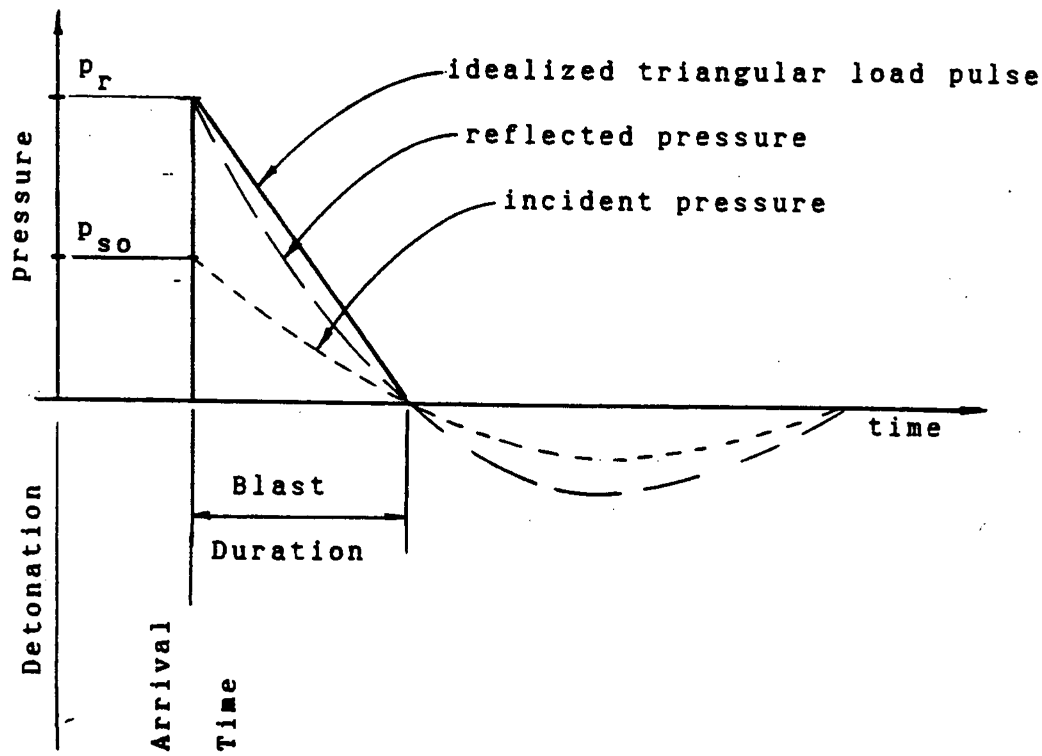

Figure 2
MONITCELLO N.G.P. BLAST LOAD DIAGRAM

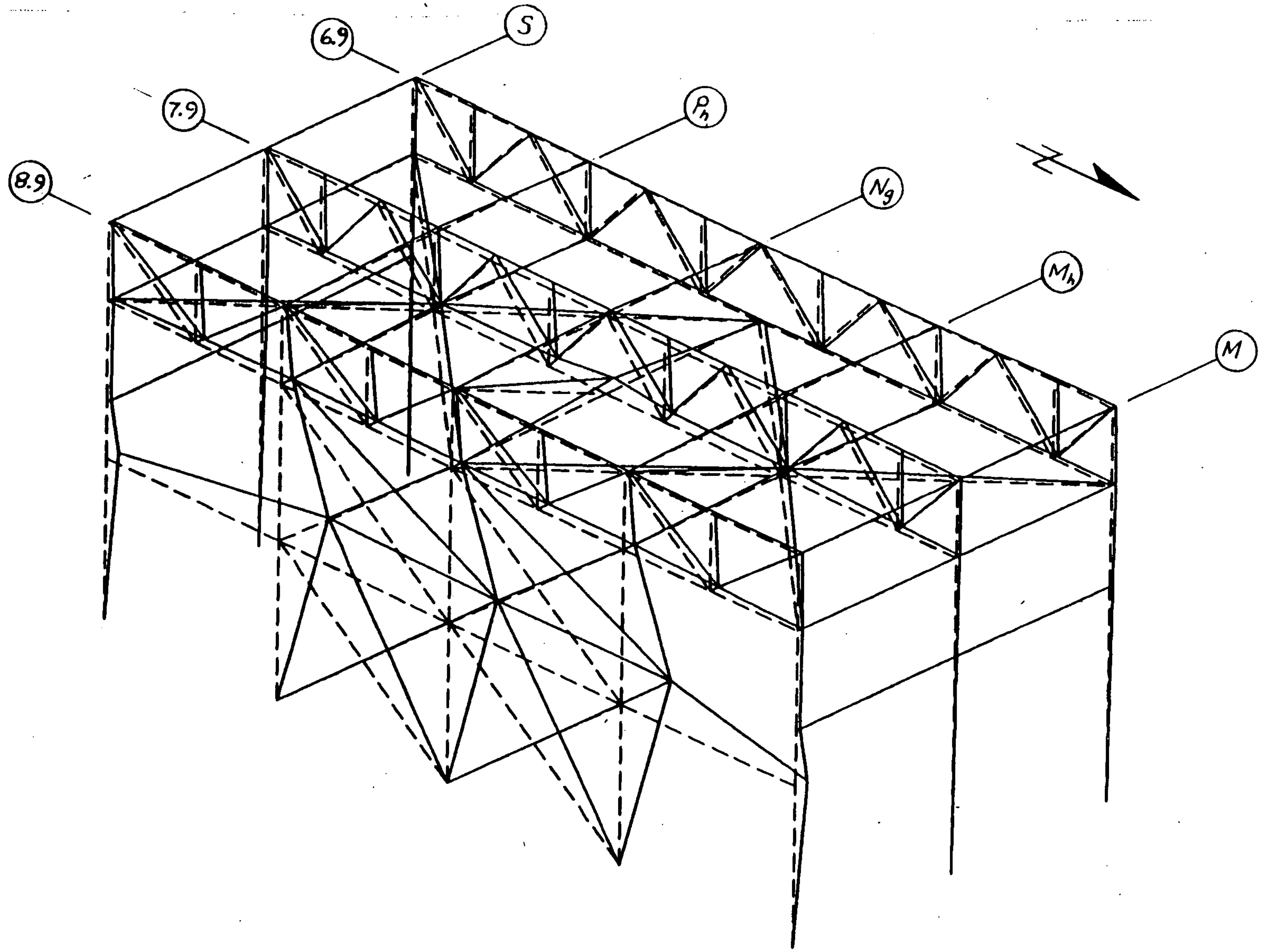

Figure 3

**MONTICELLO N.G.P.
REACTOR ENCLOSURE BUILDING
DEFORMED SHAPE**

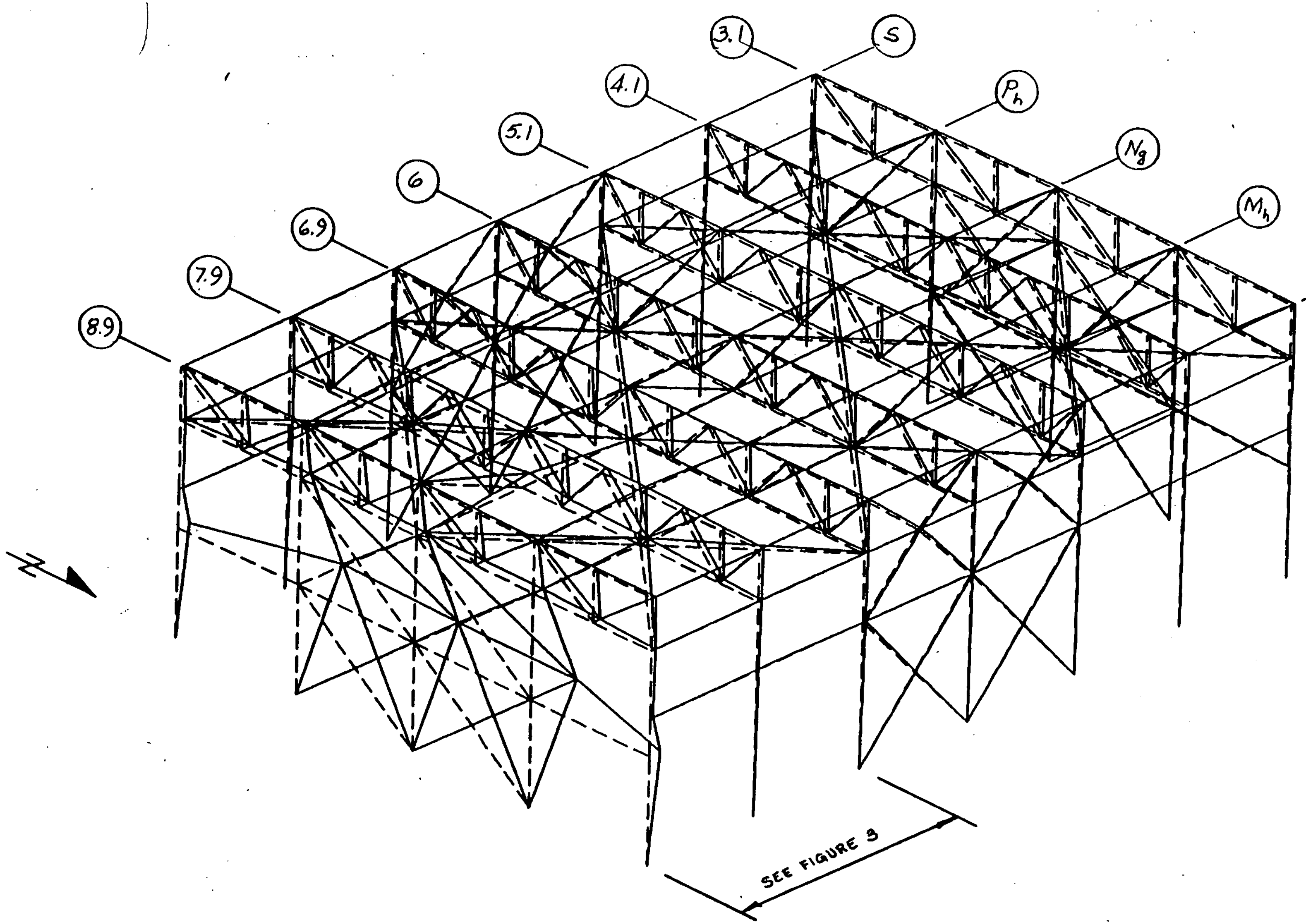

Figure 4

MONTICELLO N.G.P.
REACTOR ENCLOSURE BUILDING
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