ELASTIC-PLASTIC ANALYSES FOR SEISMIC RESERVE CAPACITY IN POWER PLANT BRACED FRAMES

T. A. Nelson
R. C. Murray

MASTER

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University of California, Livermore

LAWRENCE LIVERMORE LABORATORY

University of California, Livermore, California, 94550

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FOREWORD

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ABSTRACT

The Lawrence Livermore Laboratory was asked by the U.S. Nuclear Regulatory Commission to investigate the inelastic behavior of a representative noncategory I structure and determine the amount of reserve seismic capacity that is available beyond elastic design levels.

Elastic and elastic-plastic seismic analyses were conducted on a braced steel frame using eight time-history records. In addition, two spectra were used in elastic analyses only. By comparing elastic limit response with the ultimate capacity, the reserve strength of the frame was determined. In order to ensure operability, a frame model incorporating a piping system was subjected to the above seismic loadings using elastic analyses.

It was found that the piping system components controlled the seismic capacity of the combined structure. The average results show a reserve capacity of 2.6 times the seismic design level.

INTRODUCTION AND SUMMARY

Seismic design and performance criteria for structures have changed as scientific knowledge about earthquake phenomena has grown. As a result, the earthquake-induced forces predicted by present techniques may be significantly higher than those for which power plant structures were designed and built. However, most structural engineers believe that considerable conservatism is present in common elastic design methods. A major portion of this conservatism stems from the fact that structural elements, responding in the inelastic range, dissipate seismic energy and redistribute forces. An inelastic analysis often reveals a reserve capacity in the structure beyond that predicted by an elastic analysis. Hence, a knowledge of the inelastic behavior of typical structural configurations used in operating plants can assist in determining their actual safe seismic capacity.

We analyzed a typical diagonally braced steel frame for reserve capacity. We first performed elastic static and dynamic analyses. The structure was analyzed for dead load, live load, and earthquake loads, represented by an equivalent static lateral seismic load, two response spectra (Housner¹ and Regulatory Guide 1.60²), and a suite of eight earthquake time-history records that represent different site conditions, distances to causative faults, and magnitudes. The static lateral load and Housner spectrum represent vintage design criteria, while the R.G. 1.60 spectrum and the time-history analyses reflect current methodology. Figure 1 illustrates the inputs for our analyses.

Two levels of response can be determined from such elastic analyses. The elastic design level provides a measure of the earthquake's peak ground acceleration (PGA) that results in code design stresses in one or more structural members. We used the AISC allowable buckling stresses as the critical criteria in this study. However, code allowable stresses include a factor of safety, which we removed to determine stresses that correspond to an elastic limit response. We determined both the design-level and elastic-limit responses of the structure for each of the 11 earthquake loadings above.

Seismic input	Vintage design		Current design		Reserve capacity

	Static	Housner spectrum	R.G. 1.60	8 records elastic	8 records elastic-plastic
Frame	X	X	X	X	X
Frame and pipe		X	X	X	

FIG. 1. Seismic inputs for elastic and elastic-plastic analyses of a braced frame and frame-and-piping system.

Next, we used two-dimensional, elastic-plastic methods and the eight time histories to analyze the frame for PGA's corresponding to the ultimate capacity of the frame. We define ultimate capacity as that PGA at which the structure reaches its useful limit of performance. We established a lateral deflection of 1 ft at the top of the structure as the ultimate capacity in this study. Then, we found the reserve capacity of the structure by comparing these ultimate capacities with the design-level responses from the elastic analysis.

Analysis of the frame represents only one aspect of the problem of determining safe seismic capacity. The piping and equipment contained within the structure must function during and after a seismic event. Therefore, a representative piping system was developed that consisted of vertical and horizontal runs of 8- and 10-in. piping, a pump, valves, elbows, tees, and a reducer. A model incorporating the frame and piping system was developed and analyzed using elastic methods and the eight time histories and two spectra. PGA's for operability of each component were calculated based on the limits specified by manufacturers data or the ASME Boiler and Pressure Vessel Code.³

Results of this study will help to generate guidelines for using inelastic response criteria that more accurately assess the actual safe seismic load capacity of operating reactor components and structures. The reserve capacity of this frame or piping system does not necessarily apply directly to others of the same type. Rather, the results and conclusions should serve as a basis for accounting for what appears to be a significant amount of reserve capacity in structural systems.

The study shows that the braced frame alone has a reserve capacity of more than five times the design level. The design level response is controlled by buckling of the bracing, and the ultimate level response is controlled by a 1-ft deflection limitation. Both limiting criteria are based on the mean response to the eight time histories. The analysis of the combined frame-and-piping system indicates that the pump is the controlling element, reducing the total available reserve capacity to 2.6 times the design level. The R.G. 1.60 spectra are the most conservative input for a specified PGA, as is to be expected, since their amplification and broad frequency band are based on response at a mean plus one standard deviation level. The Housner spectrum provides the closest results to the mean response of the eight records. The static lateral load, distributed as specified in the Uniform Building Code, produces the least conservative response compared to the mean response.

SELECTION OF LOADING

Time histories of support motion are used as input for elastic-plastic seismic analyses of a structure. Because dynamic response is sensitive to the frequency content, phasing, magnitude, and duration of the input, we decided to use a suite of actual recorded time histories. A vertical and a horizontal component from each of eight earthquake records were chosen. Table 1 lists the records' magnitudes, epicentral distances, peak accelerations, and site conditions. Plots of the time histories and associated response spectra for the 1952 Santa Barbara Court House earthquake are shown in Fig. 2. Vertical and horizontal R.G. 1.60 spectra are also plotted for comparison. Appendix A contains plots of all eight records.

The time histories that we used are considered current methodology. For comparison, the current Regulatory Guide 1.60 spectrum was also used. To evaluate the difference between current and vintage methodologies, the Housner spectrum (Fig. 3) and an equivalent static lateral load were chosen to represent vintage design methods.

For analyzing the frame, static vertical loadings were defined to represent the dead and live loading. The floor and roof dead loads consisted of the weights of the girders plus a 7.5-in. concrete slab. The live loading included a 30-psf snow load on the roof and a 100-psf equipment load on the floors. These loads were included in the total mass of the structure for seismic analysis.

TABLE 1. Selected earthquake records.

Type	Code	Name	Peak acceleration, g	V H ratio	Richter magnitude	Epicentral distance, km	Site	Comments
Far field	F1	Kern Co. 1952 Santa Barbara Court House	S48E 0.129 V 0.044	0.34	7.6	90		Large earthquake Big spike
	F2	El Centro Site 2/9/56	S90W 0.05 V 0.012	0.24	6.8	126	Deep soil	
	F3	Seattle 4/13/49 Dist. Eng. Office	N88W 0.066 V 0.022	0.33	7.1	60	Soil	Deep focal depth MMVIII intensity Damage
Intermediate distance	I1	Kern Co. 1952 Taft Lincoln School Tunnel	S69E 0.176 V 0.103	0.58	7.6	43	Stiff soil	
	I2	Eureka 12/21/54 Ferndale City Hall	N46W 0.20 V 0.043	0.22	6.5	40	Stiff soil	Relatively large spike
Near field	N1	El Centro 5/18/40 Station 117	S00E 0.34 V 0.20	0.59	6.7	≈ 10	Deep soil	
	N2	Helena, Montana, 10/31/35 Carroll College	S90W 0.146 V 0.089	0.61	6.0	≈ 10	Rock	
	N3	San Fernando 2/9/71 Pacoima Dam	S16E 1.15 V 0.69	0.60	6.6	≈ 10	Rock

FIG. 2. Time-history components and response spectra for earthquake F1.

FIG. 3. Housner spectrum, normalized to 4 g, at 4% damping.

DEVELOPMENT OF STRUCTURAL MODELS

Since the objective of this study was to address generic issues of analysis, the ideal model would have been one that represented all of the structures encountered in an operating power plant. Because such a model does not exist, we modeled a typical structure, a steel frame common to noncategory I structures. The frame chosen was based on a 2-story, 3-bay diagonally braced frame from an operating plant. It represents an interior frame having a bay spacing of 29.3 ft. Connections at the ends of the bracing and beam members are considered to be pinned.

The frame was idealized as an assembly of 20 beam-column elements and 4 truss elements (Fig. 4). The mass contributions from dead and live load were lumped at the nodes as shown. This model was used for elastic and elastic-plastic analyses. Straightforward elastic element formulations in the computer program SAP4⁴ were used for elastic analyses. The program DRAIN-2D⁵ was used to analyze the structure for inelastic behavior. Material characteristics were modeled using the bilinear stress-strain relation shown in Fig. 5. The strain hardening modulus was specified as 0.01% of the elastic modulus to simulate nearly elastic-plastic behavior. The yield stress was taken to be 36 ksi.

Figure 6 shows the behavior of a truss element used to model the bracing in the inelastic analysis. The element yields in tension, with accompanying plastic strain accumulation, and buckles in compression with no residual strain after load removal. Recent work suggests that a more refined model may be desirable for bracing with slenderness ratios between 60 and 200.⁶ A literature review of the behavior of bracing systems is presented in Ref. 7. A typical experimental hysteretic loop of a bracing member with a slenderness ratio KL/r , about the same as those considered in this study (Fig. 7) shows that there can be some energy absorption in the compression region due to the formation of a plastic hinge in the buckled member.⁸ The analytical model used in this study is analogous to the slip model described in Ref. 7, in which energy absorption is provided only in the tension region. The slip

FIG. 4. Model of braced steel frame.

FIG. 5. Bilinear representation of material properties.

FIG. 6. Truss element behavior, showing yield in tension and buckling in compression.

FIG. 7. Typical experimental hysteretic loop (Source: Ref. 8).

model is one of the simplest models for analyzing braced frame behavior. It was considered appropriate because it is readily available and provides a conservative estimate of energy absorption capacity. Additional research is needed to fully understand the behavior of full-size structural bracing.^{7,9}

The beam-column element used to model the other structural members of the frame behaves as shown in Fig. 8. That is,

$$M_{pc} = M_p \quad \text{for } 0 < P/P_y \leq 0.15$$

$$M_{pc} = 1.18 (1 - P/P_y) M_p \quad \text{for } 0.15 < P/P_y \leq 1.0$$

where

M_p = plastic moment capacity

M_{pc} = plastic moment in the presence of axial loads

P = applied load

P_y = axial yield load.

FIG. 8. Yield interaction surface for beam-column element (Source: Ref. 10).

It is assumed that the columns are adequately braced to preclude instability of the members before a plastic hinge forms. Although the beams are not composite with the floor slabs, it is assumed that there is adequate lateral restraint provided to the compression flanges.

The piping system model was developed to investigate the response of a composite frame-and-pipe system, in particular, to determine the effect on the frame of vertical and horizontal runs of various diameter piping and to show the limitations imposed by the operability requirements of such associated equipment as pumps and valves. The pipe layout is shown schematically in Fig. 9. Appendix B lists the coordinates and connectivity information.

The contributing masses were lumped at the nodes shown in Fig. 9. Various configurations were studied to determine the optimum mass points. For the frame alone, an equivalent single-degree-of-freedom system for the simply supported beam members would be to lump 0.49 times the total mass at midspan. Thus, half the beam mass was lumped at a node placed midspan. For the pipe system alone, models with 41, 61, and 79 nodes were subjected to free vibration modal analysis. We decided that the 61-node model would adequately predict the dynamic response of the pipe system. Because the pipe must be supported by the frame, nodes were added to the frame members at the attachment points. We studied the effect on the frame member responses of distributing tributary mass to these added points by conducting free-vibration analyses of different models of isolated frame members (Fig. 10). The resulting frequencies are compared with closed form solutions.¹¹ Table 2 shows the comparisons and the models chosen for analysis.

The frame-and-pipe model was constructed with elements from SAP4. The bracing members were modeled with truss elements, and the remainder of the frame was modeled with beam elements. The pipes were modeled with pipe elements, which are similar to beam elements but allow internal pressure loading. The elbows are modeled with special pipe elements, which are formulated as curved beams.

Flexibility factors allow ovaling effects under induced moment to be included in the analysis. The flexibility factors in SAP4 include the pressure-induced stiffening effects specified in the ASME B&PV code for Class I piping.^{3,4,12}

FIG. 9. Model of the frame-and-pipe system (H indicates hanger locations).

FIG. 10. Beam models and mass distributions considered for use in frame-and-pipe analyses.

TABLE 2. Comparison of frequencies for different structural models.

Beam No.		1a	2a	3	4a	5
Mode 1	"Exact" ω_1	65.40	36.27	36.27	32.68	32.68
	Computed ω_1	68.44	35.99	36.24	33.45	32.40
	% Diff.	+4.6	-0.8	-0.1	+2.4	-0.9
Mode 2	"Exact" ω_2	261.57		145.07	130.73	130.73
	Computed ω_2	316.07		143.99	166.47	135.59
	% Diff.	+20.8		-0.7	+21.3	+3.7
Mode 3	"Exact" ω_3	588.60		326.45	294.14	294.14
	Computed ω_3	471.42		305.72	285.95	323.92
	% Diff.	-19.9		-6.4	-2.8	+10.1

^aModels chosen for analysis.

The weight of the contained water and the pipe insulation were distributed to the nodes on a tributary area basis. Valves were modeled as nodes on the pipe with their associated lumped mass. Valve actuators were modeled as eccentric masses attached by pipe elements to the main pipe run. The pump was modeled with SAP4 boundary elements representing two linear stiffnesses and one rotational stiffness.

STRUCTURAL ANALYSIS

The analyses of the frame and the frame-and-pipe system involved several steps. The frame analysis alone consisted of the following:

- Apply fixed-end forces including the effects of dead and live loads.
- Analyze for static vertical and lateral loads.
- Conduct linearly elastic dynamic analyses using two spectra and eight time histories.
- Combine dead and live load results with seismic results and determine PGA's for the two levels of elastic behavior.
- Conduct inelastic time-history analyses using several different scaling factors times each of the eight time histories and superimpose the dead and live loads.
- Determine PGA's corresponding to the ultimate capacity of the frame.
- Compare elastic-design-level with ultimate-capacity results and calculate reserve capacity due to inelastic response.

Only elastic analyses were conducted for the frame-and-pipe system because the performance criteria for the pipe differed from those for the frame. The pipe was subjected to thermal and pressure loadings in addition to the operating loads applied to the frame. The piping was assumed to have a line pressure of 600 psi and a temperature of 400°F.

The elastic analyses were done using the SAP4 program. The frame static loads (dead load and live load) were considered to be uniformly applied along the length of the members. For the beams, the equivalent fixed-end moments and shears were calculated and applied to the member ends. The dead load of the columns and bracing were accounted for as weight densities, which the program converts to nodal loads. One of the vintage seismic analyses required an equivalent static lateral load as the input. This was provided by proportioning the total weight of the frame to the floor levels based on the Uniform Building Code (UBC) distribution (Fig. 11).¹³ These total lateral floor loads were then distributed to the nodes of the beams comprising the floors. This procedure simulates a 1.0-g horizontal acceleration.

FIG. 11. Equivalent static seismic loads, P_1 , and P_2 , using the Uniform Building Code distribution.

A modal analysis determined the first 10 mode shapes and frequencies of the frame for the elastic response-spectra analyses. The Housner¹ and the R.G. 1.60² response spectra for 4% critical damping were used. For the Housner spectral analysis, the vertical spectrum was defined as 2/3 of the horizontal spectrum. Modal contributions were combined by the square root of the sum-of-the-squares (SRSS) method.

Time-history dynamic analyses were performed using the eight time-history records of Table 1. The vertical component and one horizontal component were input simultaneously. For computer economy, the durations of the records were reduced by adjusting the starting and ending times as shown in Table 3. The direct-integration method was used for the analysis to make results compatible with those of the subsequent inelastic analyses. The technique used for direct integration in SAP4 was the Wilson θ method¹⁴ using a time step of 0.01 s. Rayleigh damping¹⁴ was used for all time-history analyses. The coefficients for Rayleigh damping were calculated so that the equivalent

TABLE 3. Adjusted time-history durations.

Record	Start time, s	End time, s	Adjusted duration, s
F1 - Santa Barbara	5	35	30
F2 - El Centro '56	0	30	30
F3 - Seattle	7.5	37.5	30
I1 - Taft	0	20	20
I2 - Eureka	0	30	30
N1 - El Centro '40	0	30	30
N2 - Helena	0	10	10
N3 - Pacoima Dam	0	13	13

critical damping coefficient equals 4% over a wide band of frequencies. We assumed that

$$c = \alpha m + \beta k$$

where

c = viscous damping coefficient

m = mass

k = stiffness

α = mass-proportional damping coefficient

β = stiffness-proportional damping coefficient.

To get equivalent 4% critical damping ($\lambda = 0.04$), we use the expression

$$\lambda_n = \frac{\alpha T_n}{4\pi} + \frac{\beta \pi}{T_n},$$

where T_n is the n th natural period of the frame. Matching at the first ($T_1 = 0.257$ s) and third ($T_3 = 0.177$ s) natural periods,

$$\alpha = 4\pi \left(\frac{T_1 \lambda_1 - T_3 \lambda_3}{T_1^2 - T_3^2} \right) = 1.158$$

and

$$\beta = \frac{T_1 T_3}{\pi} \left(\frac{T_1 \lambda_3 - T_3 \lambda_1}{T_1^2 - T_3^2} \right) = 0.00134$$

Figure 12 illustrates the resulting fraction of critical damping vs period. The damping is 4% for the first three structure periods, and is between 4% and 5% for periods between the third and eighth mode--sufficient accuracy for the structural modes of interest.

Results from the seismic analyses (seismic vertical load + horizontal load) were combined directly with results from the static analysis (dead load + live load) to obtain a resultant for comparison with limiting criteria, which are discussed in a subsequent section. This comparison was used to compute PGA's at which limiting criteria are first met.

FIG. 12. Resultant Rayleigh damping.

Additional analyses were conducted to determine the ultimate capacity of the frame. The program DRAIN-2D,⁵ which features a constant acceleration step-by-step integration technique, was used to perform the required nonlinear analyses. Although this technique is less accurate than linear-acceleration methods, it is unconditionally stable and excludes artificial damping. Element properties at the beginning of a time step are used to calculate the response during that step. If the element goes into the nonlinear range during a time step, then the force predicted at the end of the step is incorrect. The program corrects this state by applying a corrective load over the next time step. There is no back step or subdivision of a time step that would be a more accurate correction for this phenomenon. The program changes the stiffness of an element at the end of a time step in which its yield status changes. This procedure, in conjunction with the application of a corrective load, should be adequate if the time step of integration is small. However, if the load rate is high, there may be some force overshoot.

The Rayleigh damping used in the elastic analyses was also used in the inelastic analyses. The stiffness-proportional part was based on the original stiffness matrix rather than the current stiffness.

The inelastic analyses required that all the loadings be applied to the structure at the same time. Therefore, the static loads were applied to the structure as an initial condition prior to the application of the seismic loads. Only time-history seismic loadings could be analyzed. Separate analyses were required for each time-history record to determine the ultimate behavior of the frame. In these analyses, the peak acceleration for each record was scaled to progressively larger values (Table 4) until the structure's response indicated that limiting behavior had been reached. The elastic analyses provided a basis for choosing the starting values for the scaled accelerations. Maximum lateral deflection at the top of the frame was plotted vs scaled peak acceleration to determine the limiting acceleration.

The frame-and-pipe system was analyzed elastically in a manner similar to the frame analyses. SAP4 was used with direct integration for the time-history analyses and with modal superposition for the response-spectra analyses. Three static loads were analyzed--dead and live load, pressure load, and

TABLE 4. Record scaling for elastic-plastic analyses.

Record	Scaled peak horizontal acceleration, g	Recorded peak acceleration, g	
		Horizontal	Vertical
F1	1.0	0.129	0.044
	1.5		
	2.0		
	2.4		
	2.5		
F2	0.8	0.05	0.012
	1.5		
	2.0		
	2.5		
F3	0.92	0.066	0.022
	1.5		
	2.0		
	2.5		
I1	0.8	0.176	0.103
	1.2		
	1.35		
	1.5		
	2.0		
	2.5		
I2	1.5	0.20	0.043
	2.0		
	2.5		
N1	0.8	0.34	0.20
	1.5		
	2.0		
	2.5		
N2	0.6	0.146	0.089
	1.5		
	2.0		
	2.5		
N3	1.15	1.15	0.69
	1.5		
	2.0		
	2.5		
	3.0		

thermal load. Piping response was the primary consideration. Operability limits for the pump were based on the resultant nozzle stresses. For the valves, operability was determined on the basis of peak acceleration of the valve body. The resultant accelerations were calculated for the time-history analyses by finding the maximum vector sum of the vertical and horizontal accelerations. For the response-spectra analyses, modal accelerations for each direction were first found by multiplying the relative displacement by the eigenvalue (ω^2). Modal values for each direction were then combined by the SRSS technique. The resultant was the vector sum of the two directional components.

The pipe elements were examined at the stress level using the criteria specified in the ASME code.³ Stress intensification factors were applied to reducers, elbows, and tees. These factors account for the effects of both cross-sectional distortion, such as ovaling effects in elbows, and fatigue considerations.¹²

PGA's for these various criteria were found by scaling up the earthquake portion of the resultants until limiting values were reached. The reserve capacity in the frame alone was determined by comparing the PGA at the elastic design level to the PGA at ultimate capacity. The reserve capacity determined with the inclusion of operability considerations was found by comparing the PGA at the elastic design level for the frame to the controlling PGA for operability of the piping system.

LIMITING CRITERIA

Limiting criteria were established to measure structural performance at various levels of response. The following criteria were considered:

- Elastic design level
- Limit of elastic response
- Ultimate capacity
- Equipment service limits corresponding to upset or emergency operating conditions.

ELASTIC ANALYSES

The performance of the bracing members in compression was the controlling factor for the elastic analyses of the frame. The elastic design level was defined by the AISC¹⁵ specified buckling levels with the appropriate safety factors (Fig. 13). The 33% overstress allowance for earthquake loading was not included. Since bracing members are defined as secondary members, more liberal safety factors are applied when elastic buckling controls.¹⁵ The buckling stresses for the bracing members are shown in Table 5. Note that the limit of elastic response is not reached until the member is subjected to the unfactored buckling stress. Thus, the safety factor provides reserve capacity even within the elastic response range.

After reaching the limit of elastic response, the structural members respond by either buckling or yielding. As shown in Fig. 5, the model is based on the assumption that bracing members display no hysteretic action, i.e., no yielding, on the compression side. Plastic yielding occurs on the tension side. The two-dimensional yield surface for the beam-column members is as shown in Fig. 7. The beam-column values are predicated on the ability of these members to form a plastic hinge without first exhibiting local or global instability. The floor members have full lateral support along the top flange provided by the concrete slabs. It is assumed that the columns have sufficient lateral support and local stability to allow formation of a plastic

hinge. Actually, the intent of AISC specifications¹⁵ is to ensure that sufficient rotation capacity is available to allow the strain to at least reach strain hardening levels.¹⁰ The stress-strain curve for steel is shown in Fig. 14.^{16,17}

FIG. 13. AISC buckling loads and safety factors for A36 steel (Source: Ref. 15).

TABLE 5. Buckling stress, F_A , for bracing members based on simply supported ends.

Frame member	$KL/r_{\min}$	Allowable F_A , ksf	Safety factor	Unfactored F_A , ksf
1 and 3	114.6	1591	1.91	3045
2 and 4	127.4	1375	1.85	2539

FIG. 14. Typical stress-strain curve for structural steel.

ELASTIC-PLASTIC ANALYSES

The limiting criteria are composed of two parts for the elastic-plastic analyses. It was decided that the serviceability of the structure would be limited by excessive deflections. A 1-ft lateral deflection at the roof level of the frame was chosen as a limiting value. The other criterion to consider was the strain capacity of the material. As it turned out, a 1-ft lateral deflection at node 3 (Fig. 4) results in a maximum extension in a bracing member corresponding to a 2% strain. As can be seen in Fig. 14, 2% strain is just slightly beyond the strain hardening point. Thus, 1-ft lateral deflection was also considered to be a reasonable limiting value for strain capacity.

The frame exhibited unusual behavior for certain earthquake loadings. The vertical component caused a plastic hinge to form at node 9 in the center of a floor member. As is shown in the results section, this greatly affected the lateral behavior of the frame. This anomalous behavior seemed to occur when the vertical deflection at node 9 exceeded 1 ft. Therefore, the limiting criteria were adjusted to include a 1-ft limit on vertical deflection as well.

FRAME-AND-PIPE SYSTEM

The emphasis for the frame-and-pipe system was on the operability of the piping system components. The ASME code, Section III, provided the primary criteria for the piping itself. The piping was assumed to be Class 2. It was to survive the earthquake loading at Level C service limit stresses. The pertinent relationship is given in Section NC-3611.2(c) (3) as follows:

$$\frac{P_{\max} D_o}{4 t_n} + (0.75 i) \left(\frac{M_A + M_B}{Z} \right) \leq 1.8 S_h ,$$

where

- $P_{\max}$ = peak pressure, 600 psi
- D_o = O.D. of pipe section
- t_n = nominal pipe wall thickness

- $0.75 i \geq 1.0$ i = stress intensification factor
 M_A = resultant moment loading due to weight
 M_B = resultant moment loading due to earthquake
 Z = section modulus of pipe components calculated per NC-3652.4
 S_h = basic material allowable stress at design temperature
 (400°F) = 15 ksi (SA106B).

The values of i and Z for the piping components were calculated as follows:

	i	Z
10-in. and 8-in. pipe	1.0	$\pi r^2 t_n$

10-in. and 8-in. elbows	$\frac{0.9}{(h_1)^{2/3}}$	$\pi r^2 t_n$
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8 in. x 10 in. x 10 in. tee	$\frac{0.9}{(h_2)^{2/3}}$	$\pi r^2 t_n$
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10-in. x 8-in. reducer	$0.5 + 0.01\alpha \left(\frac{D_2}{t_2} \right)^{1/2}$	≤ 2.0	$\pi r_2^2 t_2$
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where

r = mean radius

R = bend radius

$$h_1 = t_n R / r^2$$

$$h_2 = 4.4 \frac{t_n}{r} \text{ (properties of the straight run)}$$

D_2, t_2, r_2 = diameter, thickness, and radius of small end

α = angle of reducer .

Operability limits for the valves and pump are more difficult to quantify for a generic study. Usually these components would be chosen and certified on the basis of an expected load for a given plant. However, in discussions with the NRC staff, it was decided that reasonably representative values could be devised. Commonly, operability limits for motor operated valves are specified on the basis of peak inertial loading of the valve body. Two common limiting accelerations are 3 g and 5 g. Thus, PGA's were calculated for both values. For the pump, it was decided that the nozzle forces should be limited to a stress that could be resisted without damage requiring repair. This stress corresponds to a Level B service limit as specified in the ASME code. The belief was that the pump should be able to resist nozzle forces capable of resisting at least Level B stresses in the pipe attached to it. The following formula was used:

$$\frac{P_{\max} D_o}{4 t_n} + 0.75 i \left(\frac{M_A + M_B}{Z} \right) \leq 1.2 S_h \quad .$$

RESULTS

BRACED FRAME

Elastic Analyses

The PGA's for factored and unfactored buckling resulting from elastic responses of the braced frame to the different accelerograms are shown in Table 6. The maximum stresses in the lower bracing (member 1 or 3) are also shown. PGA's were found by comparing the stresses due to static and earthquake loads to the limiting values. The factored buckling values reflect the safety factor specified in AISC specifications and indicate design limits. The unfactored buckling values indicate theoretical buckling levels and, thus, incipient inelastic response. Note that there is a rather wide variation in response to the various input records, some of which required very large multiplication factors.

The mean and standard deviation σ of the PGA's for factored buckling for the recorded time histories are shown in Table 7. The near-field records caused the largest elastic response. The Pacoima Dam record, N3, caused lower response because it does not have as high a spectral amplification as the other two near-field records. The records for sites of intermediate distance caused the least elastic excitation. In particular, the Ferndale City Hall record, I2, resulted in the least elastic response, probably because it had a relatively low horizontal spectral acceleration at the first mode of the frame. The frequencies and modal participation factors for the first 10 modes are shown in Table 8, and the mode shapes are shown in Fig. 15.

The mean PGA for all the records turned out to be nearly the same as that for the far-field records, approximately 0.4 g. This was about the same value predicted by the Housner spectrum. The R.G. 1.60 spectra, which are based on a mean-plus-one-standard-deviation level, resulted in the most conservative PGA, 0.23 g. The mean-plus-one-standard-deviation value for the eight

TABLE 6. Peak ground accelerations for buckling of bracing members.

Record	PGA as input, g	Maximum bracing stress ^a due to earthquake, ksf	AISC-factored buckling		Unfactored buckling	
			Record multiplier	PGA, g	Record multiplier	PGA, g
F1	0.129	416	3.61	0.46	7.11	0.90
F2	0.05	188	7.99	0.40	15.73	0.79
F3	0.066	293	5.13	0.34	10.09	0.67
I1	0.176	763	1.97	0.35	3.88	0.68
I2	0.20	450	3.34	0.67	6.57	1.31
N1	0.34	1492	1.01	0.34	1.98	0.67
N2	0.146	814	1.85	0.27	3.63	0.53
N3	1.15	3651	0.41	0.47	0.81	0.93
Housner	0.124	432	3.48	0.43	6.85	0.85
R.G. 160	1.0	6662	0.23	0.23	0.44	0.44
UBC	1.0	2413	0.62	0.62	1.23	1.23

^aIn first-story bracing members.

TABLE 7. Mean values of PGA for elastic response levels.

	Factored buckling		Unfactored buckling	
	Mean PGA, g	σ	Mean PGA, g	σ
Far-field	0.40	0.06	0.79	0.12
Intermediate	0.51	0.23	1.00	0.45
Near-field	0.36	0.10	0.71	0.20
All time histories	0.41	0.12	0.81	0.24
Mean + 1σ	0.29			

TABLE 8. Frequencies and modal participation factors for the first 10 modes of the braced frame.

Mode number	Frequency, Hz	Period, s	Modal participation factor	
			X direction	Y direction
1	3.89	0.257	4.72E+00	3.26E-14
2	5.12	0.195	3.65E-13	1.97E+00
3	5.65	0.177	3.87E-09	-2.47E+00
4	5.66	0.177	-1.24E-01	-1.80E-08
5	5.97	0.166	4.43E-10	-1.03E+00
6	9.60	0.104	-1.82E+00	1.88E-11
7	9.63	0.104	5.87E-03	2.59E-06
8	9.66	0.103	9.54E-10	-2.23E+00
9	16.70	0.060	1.15E-13	3.43E-01
10	18.23	0.054	9.28E-11	1.50E+00

FIG. 15. Mode shapes for the braced frame.

earthquake records was 0.29 g, which is at least in the same vicinity. Note, however, that the R.G. 1.60 spectra are based on a much larger sample of earthquake records.

The PGA's for unfactored buckling were obtained by multiplying the factored values times the safety factor, which for the lower bracing is 1.914 (Table 5). These values were used to determine the starting accelerations for inelastic response analyses. In addition, they indicate a reserve capacity that is available even within the elastic response range of the frame.

Inelastic Analyses

Reserve capacity that results from inelastic response of the frame was evaluated next. Again, the first story bracing members played a major role in the response. Since these members provided the primary lateral restraint to the frame, their yielding provided the major energy absorption mechanism.

The frame was subjected to several different PGA's for each earthquake record. Plots showing maximum displacement of selected nodes vs PGA for the eight time histories are presented in Fig. 16. As explained in the limiting criteria section, a 1-ft horizontal deflection at node 3 was the primary indicator that the capacity of the frame had been reached. However, it was found that the middle, first floor beam was exhibiting inelastic behavior and that this behavior was affecting the overall response of the frame. This effect, which is illustrated in Figs. 16d, f, g, and h, is caused by beam response to the vertical component of the input records as characterized by the vertical displacement at node 9. As expected, the records causing the most significant effect are those with the highest ratio of vertical to horizontal components (Table 1). Therefore, maximum displacement of 1 ft at either node 3 or node 9 was chosen to define the limiting behavior. Table 9 summarizes the PGA's at the ultimate capacity for the frame. The average minimum reserve capacity in the frame alone is 5.4 times the average design value. This reserve capacity is composed of two parts--the elastic safety factor against buckling (1.914) and a factor due to ductile action, which averages 2.8. This factor corresponds to a local ductility of 16 in the lower bracing members.

FIG. 16a. Peak ground acceleration vs maximum displacement for nodes 3 and 9 of the braced frame using time history F1.

FIG. 16b. Peak ground acceleration vs maximum displacement for nodes 3 and 9 of the braced frame using time history F2.

FIG. 16c. Peak ground acceleration vs maximum displacement for nodes 3 and 9 of the braced frame using time history F3.

FIG. 16d. Peak ground acceleration vs maximum displacement for nodes 3 and 9 of the braced frame using time history II.

FIG. 16e. Peak ground acceleration vs maximum displacement for nodes 3 and 9 of the braced frame using time history I2.

FIG. 16f. Peak ground acceleration vs maximum displacement for nodes 3 and 9 of the braced frame using time history N1.

FIG 16g. Peak ground acceleration vs maximum displacement for nodes 3 and 9 of the braced frame using time history N2.

FIG. 16h. Peak ground acceleration vs maximum displacement for nodes 3 and 9 of the braced frame using time history N3.

TABLE 9. PGA's at ultimate capacity and reserve capacities for the frame.

Record	Design ^a PGA, g	Horizontal (Node 3) value controls		Vertical (Node 9) value controls		Minimum reserve capacity ^b
		Ultimate PGA, g	Reserve capacity ^b	Ultimate PGA, g	Reserve capacity ^b	
F1	0.46	2.4	5.2	1.9	4.1	4.1
F2	0.40	1.7	4.2	1.8	4.5	4.2
F3	0.34	1.9	5.6	1.6	4.7	4.7
I1	0.35	3.0	8.6	1.3	3.7	3.7
I2	0.67	2.6	3.9	2.6	3.9	3.9
N1	0.34	3.3	9.7	2.5	7.4	7.4
N2	0.27	---	---	2.8	10.4	10.4
N3	0.47	3.7	7.9	2.2	4.7	4.7
Mean	<u>0.41</u>	<u>2.7</u>	<u>6.4</u>	<u>2.1</u>	<u>5.4</u>	<u>5.4</u>
(σ)	(0.12)	(0.73)	(2.3)	(0.52)	(2.3)	(2.3)

^aAISC factored buckling that would be used for original design.^bReserve capacity = $\frac{\text{ultimate PGA}}{\text{design PGA}}$.

The response of the frame to record N2 is particularly interesting (Figs. 17 to 20). This record had the highest ratio of vertical to horizontal components and had a rather short duration of strong motion. Note (Table 9) that this record had the most effect on the frame for the elastic portion of the response. On the other hand, it had the least effect on the frame for the elastic-plastic response. That a structure's response is greatly reduced by elastic-plastic analysis appears to be a general trait of near-field records.

First, time histories of nodal displacements at the second floor (node 2) and roof (node 3) are shown. Next, the axial force in one of the lower bracing members (member 1) is plotted vs time along with the corresponding yield code. A yield code of zero indicates elastic loading or unloading, while a code of one indicates yielding or buckling of the member. Y indicates yielding, as indicated by the positive member force, and B, or nothing, indicates buckling. Lastly, the history of force vs deflection for this bracing member is shown. The deflection shown in the buckling region does not actually reflect a member shortening, but rather the relative movement of the two nodes at the end of the member. Figure 17 illustrates the response to this record scaled to 0.6-g PGA. At this level the response is essentially elastic. The deflection of nodes 2 and 3 and the lower brace force indicate a response similar to a damped single-degree-of-freedom system subjected to a pulse input.

The input in Fig. 18 has been scaled to 1.5 g. Now, there is significant buckling and yielding in bracing member 1. After a few cycles of yielding in the early part of the record, the bracing has stretched so that it no longer fits into the frame in the zero displacement configuration. That is, the member has experienced an accumulated plastic extension of such a magnitude that it is buckled when the frame returns to a vertical position after removal of the lateral load. The scenario has not greatly changed at 2.0 g. (Fig. 19), but, upon scaling the record to 2.5 g, it appears that the frame is becoming unstable. Figure 20 shows that member 1 buckles twice, yields significantly at the first high peak causing tension, then essentially stays buckled throughout the remainder of the record. At this point, since the columns are continuous to the roof level, they provide the first story lateral restraint, and the second story acts essentially like a truss system. This same general

FIG. 17. Selected responses to record N2 at 0.6 g.

FIG. 18. Selected responses to record N2 at 1.5 g.

FIG. 19. Selected responses to record N2 at 2.0 g.

FIG. 20. Selected responses to record N2 at 2.5 g.

sequence of events is common to all records although there are generally more cycles of yielding for the longer duration earthquakes. Similar plots for all of the records are available on fiche in a pocket inside the back cover of this report, as explained in Appendix C.

The effect of inelastic response of the frame on peak floor accelerations was also investigated. These accelerations, usually in the form of floor spectra, are used to evaluate the seismic loading on equipment attached to the floor. Table 10 indicates some trends in the maximum horizontal accelerations at the floor of the braced frame. These same results are shown graphically in Fig. 21. The results at node 3 show that the amplification (ratio of response/input) decreases or remains approximately the same with increasing inelastic action. However, there is no predictable trend in amplification at node 2.

Figure 21 shows some cases for which the PGA (zero period value) resulted in an increased amplification during inelastic response. However, we did not investigate the effect on the floor spectra at other periods. This raises the problem of the way in which certain equipment is rated for operability in an earthquake environment. For instance, valves had an operability limit specified in terms of PGA regardless of frequency content. Thus, one large spike in the floor response could disqualify the valve, though it may not respond significantly to such an isolated acceleration pulse. It would be preferable to consider the actual limiting mechanism, such as inelastic distortion of the valve stem, and, perhaps, indicate frequency-dependent operability criteria that could be compared with the resultant floor spectra.

The acceleration values in Table 10 and Fig. 21 are not accurate enough to make quantitative conclusions, but are presented to indicate potential trends. They may not represent the actual structural response at high levels of inelastic action because of the nature of the force correction feature in DRAIN. This feature can cause overshoot and oscillations in the calculated accelerations. The accelerations are the most sensitive to these idiosyncrasies, which are related to the time step of integration. When the time step was cut in half, to 0.005 s, the accelerations showed a measurable change while the changes in the nodal displacements were insignificant. The

TABLE 10. Maximum horizontal accelerations at floor levels.

Record	Input acceleration, g	Node 2 (2nd floor)		Node 3 (roof)	
		Response acceleration, g	Ratio of response/ input	Response acceleration, g	Ratio of response/ input
F1	1.0	1.24	1.24	1.75	1.75
	1.5	4.79	3.19	2.61	1.74
	2.0	7.44	3.72	3.95	1.98
	2.5	13.24	5.30	5.18	2.07
F2	0.8	1.04	1.30	1.98	2.48
	1.5	9.56	6.38	3.90	2.60
	2.0	16.48	8.24	4.82	2.41
	2.5	22.35	8.94	4.97	1.99
F3	0.92	3.99	4.34	3.16	3.44
	1.5	12.41	8.27	4.30	2.86
	2.0	12.58	6.29	4.27	2.14
	2.5	7.35	2.94	5.06	2.02
I1	0.8	1.49	1.86	2.34	2.93
	1.2	5.95	4.96	3.44	2.87
	1.35	9.81	7.27	4.38	3.24
	1.5	8.16	5.44	4.26	2.84
	2.0	8.74	4.37	4.53	2.27
	2.5	6.20	2.48	4.13	1.65
I2	1.5	1.89	1.26	2.34	1.55
	2.0	2.55	1.27	2.82	1.41
	2.5	5.86	2.34	4.46	1.78
N1	0.8	2.25	2.82	2.93	3.67
	1.5	10.50	7.00	3.87	2.58
	2.0	8.35	4.18	5.18	2.59
	2.5	17.93	7.17	6.03	2.41
N2	0.6	1.35	2.24	1.98	3.29
	1.5	4.81	3.21	5.40	3.60
	2.0	9.13	4.56	3.98	1.99
	2.5	7.95	3.18	3.17	1.27
N3	1.15	1.36	1.18	2.53	2.20
	1.5	3.11	2.07	3.28	2.19
	2.0	5.75	2.87	3.46	1.73
	2.5	11.03	4.41	4.19	1.68
	3.0	9.77	3.26	4.18	1.39

FIG. 21. Maximum horizontal accelerations on braced-frame nodes 2 and 3 due to the eight time-history records.

credibility of the displacements, which are the basis for the reported reserve capacities, are thus verified. However, the acceleration data may be viewed as merely indicative of potential trends. Further study is needed to evaluate the effect of inelastic response on floor spectra. Although amplification of the input motion is expected to decrease with increasing inelastic action, it may even increase at certain frequencies. Preliminary work in this area is reported in Ref. 19.

The approximate way in which the bracing members were modeled could also affect the results. Figure 22 illustrates the acceleration time histories at nodes 2 and 3 for four input acceleration levels of record F3. Also shown are the corresponding axial force and yield code time histories for member 1. From the figures showing response from successive input levels, one can see that isolated peaks of acceleration begin to appear. The positive peaks coincide with the termination of buckling of member 1, as shown, and the negative peaks coincide with the termination of buckling of member 3. At the termination of buckling, these members experience a rapid take-up or snap as they come under tensile stress. This effect causes the large spikes of acceleration shown in Fig. 22.

This type of behavior seems logical for very slender bracing members in which no significant energy absorption would occur in the post buckling region. However, as the slenderness ratio decreases, the formation of a plastic hinge in a brace after buckling will start to dissipate energy and mitigate somewhat the snapping effects described above.

FIG. 22. Selected responses to record F3 at successively higher input accelerations, showing the effect of take-up on bracing member 1.

FIG. 22. (cont'd.).

FRAME-AND-PIPE SYSTEM

Superposing the piping system model on the frame added mass to the frame system. Table 11 shows the modal frequencies and modal participation factors for the combined frame-and-pipe system. Figure 23 shows the 20 mode shapes.

Table 12 shows PGA's and the corresponding limiting criteria for the frame-and-pipe system components. Mean values are shown in Table 13. The same trends are exhibited for the frame-and-pipe system as for the frame alone.

TABLE 11. First 20 modes of the frame-and-pipe system and modal participation factors.

Mode	Frequency, Hz	Period, s	Modal participation factor	
			X direction	Y direction
1	3.77	0.265	4.83E+00	-3.39E-02
2	5.20	0.192	-1.67E-02	2.42E+00
3	5.65	0.177	1.28E-01	7.36E-01
4	5.66	0.177	-1.02E-02	2.41E+00
5	5.97	0.168	1.35E-03	9.47E-01
6	8.29	0.121	2.96E-01	-1.99E-02
7	9.26	0.108	6.57E-01	1.58E+00
8	9.47	0.106	-5.60E-01	1.70E+00
9	9.57	0.104	1.59E+00	1.79E-01
10	9.96	0.100	2.27E-01	-8.44E-01
11	10.76	0.093	1.15E-01	-9.86E-03
12	14.58	0.069	-1.65E-02	-4.01E-02
13	16.45	0.061	-4.16E-02	1.74E-02
14	17.82	0.056	-3.85E-02	-5.54E-01
15	18.33	0.055	-2.72E-02	4.83E-01
16	18.91	0.053	-7.44E-02	7.01E-03
17	19.21	0.052	4.11E-02	-8.35E-02
18	19.59	0.051	-1.03E-02	2.75E-01
19	20.28	0.049	1.20E-01	-2.55E-01
20	22.07	0.045	-9.07E-03	-1.38E+00

FIG. 23. Mode shapes for the frame-and-pipe system.

FIG. 23 (cont'd.)

FIG. 23 (cont'd.)

FIG. 23 (cont'd.)

FIG. 23 (cont'd.)

FIG. 23 (cont'd.)

FIG. 23 (cont'd.)

FIG. 23 (cont'd.)

FIG. 23 (cont'd.)

FIG. 23 (cont'd.)

TABLE 12. Limiting PGA's, g, for components of the frame-and-pipe system.

Record	Bracing member		Pump ^a	Valve		Tee ^c 8 x 10 x 10 in.	Straight pipe ^c		Reducer ^c 10 x 8 in.	Elbow ^c	
	Factored buckling	Unfactored buckling		3 g ^b	5 g ^b		10 in.	8 in.		8 in.	10 in.
F1	0.42	0.83	1.48	1.48	2.47	1.83	2.35	2.04	2.34	3.28	3.70
F2	0.34	0.67	1.10	1.12	1.88	1.30	1.82	1.61	2.08	3.02	3.03
F3	0.25	0.50	0.78	0.92	1.54	1.12	1.30	2.03	2.32	2.29	3.12
I1	0.30	0.59	0.87	1.00	1.68	1.02	1.44	1.35	1.55	2.00	2.52
I2	0.52	1.03	1.74	1.87	3.12	2.36	2.87	5.97	6.85	5.64	5.06
N1	0.26	0.52	0.82	0.93	1.55	1.15	1.36	2.18	2.50	2.37	2.28
N2	0.22	0.44	0.68	0.78	1.31	1.03	1.13	1.66	1.90	1.80	1.73
N3	0.36	0.70	1.02	0.94	1.57	1.26	1.68	1.58	1.82	2.42	2.65
Housner	0.41	0.81	1.06	0.82	1.35	1.02	1.75	2.06	2.36	2.07	2.69
R.G. 1.60	0.21	0.42	0.56	0.45	0.75	0.56	0.93	1.10	1.26	1.13	1.48

^aLevel B stress limit.^bLimiting acceleration.^cLevel C stress limit.

TABLE 13. Mean values of limiting PGA, g, for components of the frame-and-pipe system.

Input	Bracing members		Pump	Valve		Tee 8 x 10 x 10 in.	Straight pipe		Reducer 10 x 8 in.	Elbow	
				3 g ^a	5 g ^a		10 in.	8 in.		8 in.	10 in.
Far-field	0.34	0.67	1.12	1.17	1.96	1.42	1.82	1.96	2.25	2.86	3.28
Intermediate	0.41	0.81	1.31	1.44	2.40	1.69	2.16	3.66	4.20	3.82	3.79
Near-field	0.28	0.55	0.84	0.88	1.48	1.15	1.39	1.81	2.07	2.20	2.22
All records	0.33	0.66	1.06	1.13	1.89	1.38	1.74	2.33	2.67	2.85	3.01
(σ)	(0.10)	(0.19)	(0.37)	(0.36)	(0.61)	(0.47)	(0.59)	(1.50)	(1.72)	(1.23)	(1.02)
Without record I2	0.31	0.61	0.96	1.02	1.71	1.24	1.58	1.81	2.07	2.45	2.72
(σ)	(0.07)	(0.14)	(0.27)	(0.23)	(0.37)	(0.28)	(0.41)	(0.30)	(0.34)	(0.53)	(0.64)

^aLimiting acceleration values.

The standard deviation of the limiting PGA's becomes quite large for the piping components. This stems, in large part, from the high PGA's in response to record I2. Since response to this record is significantly different from the other time-history responses, we decided to omit record I2 and use the mean of the other seven time histories. This is considered to be the most reasonable option to take since these low responses are likely the result of particular idiosyncrasies of the frame-and-pipe system that we used. For generic results these idiosyncrasies should be bypassed insofar as possible.

The results show that the piping system causes about a 20% lower limiting PGA for bracing member buckling than was the case with the frame alone. This results mainly from an increased inertial load due to the mass of the piping system. Since the assumed live load applied to the frame alone was large enough to include a generic piping system, the results from that analysis were used for the frame response. The frame-and-pipe system analysis will be used for the evaluation of piping components only.

Table 13 shows the limiting PGA's for the various components in the order that they control. The pump is the primary controlling element based on a limiting nozzle load. Next, the valves control based on the inertial load induced by a resultant acceleration. Again, the Housner spectrum predicted results much closer to the mean values than did the R.G. 1.60 spectra. The response to the R.G. 1.60 spectra was more severe than the mean plus standard deviation response from the seven (I2 omitted) time histories.

CONCLUSIONS

A substantial reserve capacity is available in both the braced frame and the attached piping. The pump limits the total capacity of the system. The mean reserve capacities (ratios of the component's capacity to the elastic design level of the frame) available in each component of the system are shown in Table 14. The elastic design level of the frame was taken to be the PGA at the factored buckling level for each earthquake record. The mean values do not include record I2 in the calculation. However, inclusion of I2 would not affect the reserve capacities until the 8-in. straight pipe was the limiting component.

Thus, based on operability considerations, the frame system has a reserve capacity at least 2.6 times its design level when recorded earthquakes are used for analysis. If the frame system were designed using the Housner spectrum, the mean time-history results suggest a reserve capacity of 2.2. If the R.G. 1.60 spectra were the design basis, these analyses predict a reserve capacity of 4.2.

TABLE 14. Reserve capacities (ratios of the component's capacity to the elastic design level of the frame).

Component	Limiting criterion	Mean reserve capacity
Pump	Level B stress limit at nozzle	2.6
Valves	3-g acceleration	2.7
Tee	Level C stress	3.3
Pipe	Level C stress	4.2 to 5.0
Frame	1-ft Deflection	5.4
Reducer and elbows	Level C stress	5.7 to 7.2

Only elastic analyses were needed for operability considerations. Figure 16 shows that the response of the frame is linear in the acceleration range where the pump and valves reach their operability limit. This suggests that elastic analyses of the frame-and-pipe system are adequate to establish operability limits for this model, and that an inelastic response of the combined system is not warranted. This may not be true in general, however.

Conclusions of a generic nature based on the analysis of one frame-and-pipe system and a limited number of earthquake records are tenuous. However, it is reasonable to assume that significant reserve capacity is available beyond that predicted by elastic analysis. This report provides the basis for using reserve capacity in structures and indicates the potential effect of attached equipment. Some of the considerations essential to the use of reserve capacity of nuclear power plant structures are presented in Reference 18.

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RJ/ew

APPENDIX A
Earthquake Records

Record F1--Kern Co., 1952; Santa Barbara Courthouse.

Record F2--E1 Centro site, 2/9/56.

Record F3--Seattle, 4/13/49; Dist. eng. office.

Record I1--Kern Co., 1952; Taft Lincoln School Tunnel.

Record I2--Eureka, 12/21/54; Ferndale City Hall.

Record N1--El Centro, 5/18/40; Station 117.

Record N2--Helena, Montana, 10/31/35; Carroll College.

Record N3--San Fernando, 2/9/71; Pacoima Dam.

APPENDIX B
Frame-and-Pipe Model

Node	Nodal fixity						Coordinates		
	XD	YD	ZD	XR	YR	ZR	X	Y	Z
1	1	1	1	1	1	0	-0.01	0.0	0.0
2	0	0	1	1	1	0	3.500	2.000	0.0
3	0	0	1	1	1	0	2.380	2.000	0.0
4	0	0	1	1	1	0	1.250	2.000	0.0
5	0	0	1	1	1	0	0.0	3.250	0.0
6	0	0	1	1	1	0	0.0	7.688	0.0
7	0	0	1	1	1	0	0.0	12.125	0.0
8	0	0	1	1	1	0	0.0	16.563	0.0
9	0	0	1	1	1	0	0.0	21.000	0.0
10	0	0	1	1	1	0	-0.01	21.000	0.0
11	0	0	1	1	1	0	-0.01	22.000	0.0
12	0	0	1	1	1	0	0.0	26.938	0.0
13	0	0	1	1	1	0	0.0	32.875	0.0
14	0	0	1	1	1	0	0.0	38.813	0.0
15	0	0	1	1	1	0	0.0	44.750	0.0
16	0	0	1	1	1	0	1.250	46.000	0.0
17	0	0	1	1	1	0	-0.01	50.000	0.0
18	0	0	1	1	1	0	3.500	50.000	0.0
19	0	0	1	1	1	0	3.500	46.000	0.0
20	0	0	1	1	1	0	7.625	46.000	0.0
21	0	0	1	1	1	0	11.750	46.000	0.0
22	0	0	1	1	1	0	14.000	50.000	0.0
23	0	0	1	1	1	0	15.875	46.000	0.0
24	0	0	1	1	1	0	20.000	46.000	0.0
25	0	0	1	1	1	0	20.000	50.000	0.0
26	0	0	1	1	1	0	23.375	46.000	0.0
27	0	0	1	1	1	0	27.990	50.000	0.0
28	0	0	1	1	1	0	26.750	46.000	0.0
29	0	0	1	1	1	0	28.000	44.750	0.0
30	0	0	1	1	1	0	28.000	43.500	0.0
31	0	0	1	1	1	0	27.990	43.500	0.0
32	0	0	1	1	1	0	28.000	38.750	0.0
33	0	0	1	1	1	0	28.000	34.000	0.0
34	0	0	1	1	1	0	28.000	29.250	0.0
35	0	0	1	1	1	0	28.000	24.500	0.0
36	0	0	1	1	1	0	27.990	24.500	0.0
37	0	0	1	1	1	0	14.000	22.000	0.0
38	0	0	1	1	1	0	27.990	22.000	0.0
39	0	0	1	1	1	0	28.000	23.240	0.0
40	0	0	1	1	1	0	29.250	21.990	0.0
41	0	0	1	1	1	0	30.500	21.990	0.0
42	0	0	1	1	1	0	30.500	22.000	0.0
43	0	0	1	1	1	0	32.500	21.990	0.0
44	0	0	1	1	1	0	37.000	21.990	0.0
45	0	0	1	1	1	0	41.000	22.000	0.0
46	0	0	1	1	1	0	41.500	21.990	0.0
47	0	0	1	1	1	0	46.000	21.990	0.0

Node	XD	YD	Nodal fixity				X	Coordinates		Z
			ZD	XR	YR	ZR		Y		
48	0	0	1	1	1	0	46.000	22.000	0.0	
49	0	0	1	1	1	0	50.000	21.990	0.0	
50	0	0	1	1	1	0	53.990	22.000	0.0	
51	1	1	1	1	1	0	27.990	0.000	0.0	
52	1	1	1	1	1	0	53.990	0.000	0.0	
53	0	0	1	1	1	0	54.000	27.078	0.0	
54	0	0	1	1	1	0	54.000	32.156	0.0	
55	0	0	1	1	1	0	54.000	37.234	0.0	
56	0	0	1	1	1	0	54.000	42.313	0.0	
57	0	0	1	1	1	0	52.667	42.313	0.0	
58	0	0	1	1	1	0	54.000	45.000	0.0	
59	0	0	1	1	1	0	55.000	46.000	0.0	
60	0	0	1	1	1	0	41.000	50.000	0.0	
61	0	0	1	1	1	0	53.990	50.000	0.0	
62	0	0	1	1	1	0	58.500	46.000	0.0	
63	0	0	1	1	1	0	62.000	46.000	0.0	
64	0	0	1	1	1	0	62.000	50.000	0.0	
65	0	0	1	1	1	0	66.500	46.000	0.0	
66	0	0	1	1	1	0	68.000	50.000	0.0	
67	0	0	1	1	1	0	71.000	46.000	0.0	
68	0	0	1	1	1	0	75.500	46.000	0.0	
69	0	0	1	1	1	0	80.000	46.000	0.0	
70	0	0	1	1	1	0	80.000	50.000	0.0	
71	0	0	1	1	1	0	81.990	50.000	0.0	
72	0	0	1	1	1	0	54.000	21.990	0.0	
73	0	0	1	1	1	0	56.290	21.990	0.0	
74	0	0	1	1	1	0	56.290	22.000	0.0	
75	0	0	1	1	1	0	57.978	21.990	0.0	
76	0	0	1	1	1	0	57.978	23.333	0.0	
77	0	0	1	1	1	0	62.652	21.990	0.0	
78	0	0	1	1	1	0	67.326	21.990	0.0	
79	0	0	1	1	1	0	68.000	22.000	0.0	
80	0	0	1	1	1	0	72.000	21.990	0.0	
81	0	0	1	1	1	0	72.000	22.000	0.0	
82	0	0	1	1	1	0	76.000	21.990	0.0	
83	0	0	1	1	1	0	81.990	22.000	0.0	
84	0	0	1	1	1	0	81.000	21.990	0.0	
85	0	0	1	1	1	0	82.000	20.990	0.0	
86	0	0	1	1	1	0	82.000	16.750	0.0	
87	0	0	1	1	1	0	82.000	12.500	0.0	
88	0	0	1	1	1	0	82.000	8.250	0.0	
89	0	0	1	1	1	0	82.000	4.000	0.0	
90	0	0	1	1	1	0	81.990	4.000	0.0	
91	1	1	1	1	1	0	81.990	0.0000	0.	
92	1	1	1	1	1	1	3.500	2.000	1.00	
93	1	1	1	1	1	1	86.000	54.000	0.0	
94	1	1	1	1	1	1	3.500	4.000	0.0	

Member incidences

Truss

Member No.	Node	
	i	j
1	9	10
2	19	18
3	24	25
4	30	31
5	35	36
6	38	17
7	41	42
8	47	48
9	51	11
10	52	83
11	50	71
12	73	74
13	80	81
14	63	64

Beam

Member No.	Node	
	i	j
1	1	10
2	10	11
3	11	17
4	51	38
5	38	36
6	36	31
7	31	27
8	52	50
9	50	61
10	91	90
11	90	83
12	83	71
13	17	18
14	18	22
15	22	25
16	25	27
17	27	60
18	60	61
19	61	54
20	64	66
21	66	70
22	70	71
23	11	37
24	37	38
25	38	42
26	42	45
27	45	48
28	48	50
29	50	74
30	74	79
31	79	81
32	81	83
33	90	89
34	69	70

Pipe

Member No.		Node i	Node j	Member No.		Node i	Node j
1T		2	3	28T	1.250		11
2T		3	4	29T		40	41
3B		4	5	30T		41	43
	1.250		11	31T		43	44
4T		5	6	32T		44	46
5T		6	7	33T		46	47
6T		7	8	34T		47	49
7T		8	9	35T		49	72
8T		9	12	36T		72	53
9T		12	13	37T		53	54
10T		13	14	38T		54	55
11T		14	15	39T		55	56
12B		15	16	40T		56	57
	1.250		11	41B		56	58
13T		16	19			58	59
14T		19	20	42T	1.000		11
15T		20	21	43T		59	62
16T		21	23	44T		62	63
17T		23	24	45T		63	65
18T		24	26	46T		65	67
19T		26	28	47T		67	68
20B		28	29	48T		68	69
	1.250		11	49T		72	73
21T		29	30	50T		73	75
22T		30	32	51T		75	76
23T		32	33	52T		75	77
24T		33	34	53T		77	78
25T		34	35	54T		78	90
26T		35	39	55T		80	82
27B		39	40	56B		82	84
						84	85
					1.000		11
				57T		85	96
				58T		86	87
				59T		87	88
				60T		88	89

T = Tangent element

B = Bend element

Number below bend element is bend radius.

APPENDIX C
Inelastic Response to Time-History Records

The complete record of the braced frame's inelastic response to the eight time histories (scaled to progressively higher PGA's) is reproduced on 16 fiche. The fiche are provided in a pocket inside the back cover of this report. The reader is referred to the section on inelastic results for a detailed explanation of the response to record N2, which will serve as a guide to the other responses on the fiche. The various responses are located on the 16 fiche as shown in the following table:

Type	Code	Name	Fiche No.
Far field	F1	Kern Co. 1952 Santa Barbara Court House	1, 2
	F2	El Centro Site 2/9/56	5, 6
	F3	Seattle 4/13/49 Dist. Eng. Office	3, 4
Intermediate distance	I1	Kern Co. 1952 Taft Lincoln School Tunnel	7, 8, 9
	I2	Eureka 12/21/54 Ferndale City Hall	9, 10
Near field	N1	El Centro 5/18/40 Station 117	11, 12
	N2	Helena, Montana, 10/31/35 Carroll College	13, 14
	N3	San Fernando 2/9/71 Pacoima Dam	15, 16

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Division of Operating Reactors
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U.S. Nuclear Regulatory Commission
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R. Vollmer
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U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

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Office of Nuclear Reactor Regulation
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Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

T. E. Murley
Division of Reactor Safety Research
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

G. Bagchi (50)
Division of Reactor Safety Research
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

J. Richardson
Division of Reactor Safety Research
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

F. P. Schauer
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

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Office of Standards Development
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

R. P. Gupta
Office of Standards Development
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

C. Y. Liaw
EG&G, Incorporated
2801 Old Crow Canyon Road
San Ramon, CA 94583

S. M. Ma
EG&G, Incorporated
2801 Old Crow Canyon Road
San Ramon, CA 94583

W. Ruvalcaba
EG&G, Incorporated
2801 Old Crow Canyon Road
San Ramon, CA 94583

K. S. Pister
University of California
Civil Engineering Department
Berkeley, CA 94720

N. M. Newmark
Consulting Engineering Services
1211 Civil Engineering Building
Urbana, IL 61801

W. J. Hall
Consulting Engineering Services
1211 Civil Engineering Building
Urbana, IL 61801

R. N. Chittenden
Consulting Engineering Services
1211 Civil Engineering Building
Urbana, IL 61801

C. I. York
Division of Operational and
Environmental Safety
Department of Energy
Germantown, MD 20545

E. K. Loop
Division of Operational and
Environmental Safety
Department of Energy
Germantown, MD 20545

W. J. McCool
Division of Operational and
Environmental Safety
Department of Energy
Germantown, MD 20545

E. K. Collins (2)
EG&G, Incorporated
2801 Old Crow Canyon Road
San Ramon, CA 94583

R. K. Johnson
EG&G, Incorporated
2801 Old Crow Canyon Road
San Ramon, CA 94583

A. M. Davito
EG&G, Incorporated
2801 Old Crow Canyon Road
San Ramon, CA 94583

W. B. Lowenstein
Electric Power Research Institute
3412 Hillview Avenue
P.O. Box 10412
Palo Alto, CA 94303