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Performance Based Seismic Qualification of Reinforced Concrete Nuclear Materials Processing Facilities

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PERFORMANCE BASED SEISMIC QUALIFICATION OF REINFORCED CONCRETE NUCLEAR MATERIALS PROCESSING FACILITIES

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ABSTRACT

A seismic qualification of a reinforced concrete nuclear materials processing facility using performance based acceptance criteria is presented. Performance goals are defined in terms of a minimum annual seismic failure frequency. Pushover analyses are used to determine the building's ultimate capacity and relate the capacity to roof drift and joint rotation. Nonlinear dynamic analyses are used to quantify the building's drift using a suite of ground motion intensities representing varying soil conditions and levels of seismic hazard.

A correlation between joint rotation and building drift to damage state is developed from experimental data. The damage state and seismic hazard are convolved to determine annual seismic failure frequency. The results of this rigorous approach is compared to those using equivalent force methods and pushover techniques recommended by ATC-19 and FEMA-273.

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Introduction

An engineer faced with performing a seismic evaluation of existing reinforced concrete buildings, particularly older Department of Energy (DOE) facilities built before 1970, often has to deal with construction details that are not as rugged as those required by current design codes. Examples include beams detailed only for gravity loads when subjected to load reversal, reinforcing steel development lengths that are inadequate to fully develop the moment capacity in framed structures, and reinforcing details that may not allow ductile behavior under load reversal. Inadequate concrete cover or ties that do not adequately confine the longitudinal reinforcement may exacerbate this problem.

Methodologies used to evaluate older structures that have deficient seismic design with respect to current codes vary from traditional deterministic methods to non-linear analyses coupled with probabilistic techniques. Several alternative evaluation methodologies exist that meet the requirements of the DOE Acceptance Criteria, DOE Standard 1020 (DOE, 1994) to demonstrate the seismic capability of a facility. The engineer can apply non-linear analysis coupled with probabilistic techniques if deterministic evaluations prove inadequate to demonstrate seismic capability. On the other hand, if the outcome of a cost benefit analysis suggests design modifications to strengthen a facility, then the engineer should apply traditional conservative deterministic methodology to design upgrades.

An evaluation methodology that addresses the deficiencies found in older DOE reinforced-concrete facilities is presented in this paper. Using this methodology the authors have shown that the nuclear material processing canyon facilities (canyons) at the Savannah River Site are capable of surviving the postulated design basis earthquake (DBE).

Acceptance Criteria

DOE Standard 1020 sets acceptance criteria for DOE facilities based on performance goals which for this structure is an annual failure frequency of 2×10^{-4}. The standard allows considerable latitude in evaluation methodology as long as the specified performance goals for a given category of structures, systems and components (SSC) are met. This latitude consists essentially of three approaches to meet the performance goals for the SSC.

- For a hazard probability specified for the facility use, apply conservative deterministic evaluation techniques based on national consensus standards as supplemented by DOE Standard 1020 requirements for the facility performance goal.
- Achieve less than a 10% probability of unacceptable performance of an SSC subjected to a scaled design basis earthquake (SDBE) that is 50% larger than the DBE.
- Demonstrate acceptable structural behavior by showing that the building annual probability of seismic failure is less than the facility performance goal.

A key factor in the probabilistic approach is the specification of an acceptance criteria, stated in probabilistic terms, that conforms to the performance goals and associated conservatism in DOE Standard 1020. In this paper, significant structural degradation in a concrete joint under cyclic loading is associated with a 50% probability of failure. Identifying a joint failure, as such, does not imply the failure of the building frame. In fact, substantial conservatism exists since a failure of the frame cannot occur until a sufficient number of joints sustain sufficient damage to cause a collapse mechanism. Using this definition of unacceptable structural behavior the evaluation methodology leads to the calculation of the conditional probability of seismic failure. This conditional failure probability is measured against the acceptance criteria in DOE Standard 1020.

Analysis Methodology

There are numerous options available to the engineer to evaluate older existing facilities for seismic loads. The choice should consider the in-situ building condition, the level of seismic load, the effects of foundation embedment, the functional requirements, the level of acceptable damage, the confinement requirements, and the performance category of the facility. This paper concentrates on the more rigorous approach to demonstrate the capability of a facility to withstand a postulated seismic event. However, the engineer should consider alternative and simpler approaches first.

The approach presented applies probabilistic evaluation techniques coupled with nonlinear dynamic analysis. The approach uses in-situ material properties along with knowledge gained through testing of reinforced concrete available in the technical literature. The objective of the probabilistic approach is to calculate an annual probability of seismic failure (fragility) for the facility and compare with acceptable levels as specified in DOE Standard 1020. A process to achieve this objective has been implemented by the authors at the Savannah River Site for the evaluation of its material processing canyon facilities and consists of the following steps:

1. Establish a suite of DBE time histories for the seismic evaluation of dynamic models of the facility structure. A sufficient number of time histories are required (usually 10-30) to calculate the mean annual probability of failure, whose median response spectrum closely matches the DBE response spectrum. These time histories account for variations in the geomorphology at the site.

2. Identify the lateral and vertical load paths of the structure to aid in preparing adequate analytical models of the structure.

3. Identify the structural details of the facility that influence the capacity of the facility. These include the reinforcing steel details, embedment lengths, confinement detailing, concrete strength, and concrete condition.

4. Perform pushover analyses that include the non-linear behavior of the building. This analysis provides a distribution and magnitude of the bending, shear and axial forces, joint rotations and associated drift of the structure under a monotonically increasing static lateral load.

5. Develop probabilities of failure of the critical details identified in Step 3 and relate to building lateral drift. For the analysis of the SRS canyon structures a probability of failure as a function of joint rotation associated with bond slip is used. Drift from the pushover analysis associated with a 50% probability of failure of the first critical joint is identified that would lead to a collapse mechanism of the building frame.

6. Develop simple dynamic models of the building structure. The detailed non-linear finite element models used in the pushover analysis are reduced in dynamic degrees of freedom such that the important modes of the structure are retained. These models are used to calculate building drift for the suite of time histories. The use of simpler non-linear models has the advantage of reducing computational time considering that numerous time history analyses are to be made.

7. Perform time history analysis of the simplified model. Compare the time history response for a sampling of time histories with the detailed model to assure that the simple models provide sufficiently accurate results. Multiple analyses with each of the time histories are made to determine the sensitivity of displacement to variability in the seismic level, soils, concrete material strength, soil stiffness and structural stiffness.

8. Using the results of the pushover and time history analysis a building fragility curve representing the conditional probability of failure of the building is developed. The fragility provides a relationship between the probability of failure of the structure and the input ground motion. The fragility estimate is developed for the median capacity of the canyon structure and includes the variabilities due to response, strength, capacity, and ground motion correlation.

9. Convolving the building fragility curve with the seismic hazard curve for the site provides the annual probability of seismic failure. Comparing the annual probability of seismic failure with the acceptance performance goal from DOE Standard 1020 demonstrates that the building is adequate for its specific performance category.

Facility Description

The Canyon facilities were designed and constructed in the early 1950s of reinforced-concrete. They are 66 ft high by 122 ft wide (Figure 1) and consist of eighteen segments, typically 43 ft long. The segments are separated from each other by one half inch expansion joints. The exterior walls range from 2.5 to 4.8 ft thick and support a haunched roof slab that is from 2.5 to 3.5 ft thick. A frame structure is contained within the building. The lower portion consists of continuous walls and discrete columns while the upper portions contain continuous walls. The specified design strength of the concrete was 2,500 psi and Grade-40 reinforcing steel was used. The structure is supported on a 5-foot-thick reinforced-concrete foundation mat. Two-story reinforced-concrete penthouses were constructed at a later date over portions of each building. The total mass of a typical segment is 771 k-s^2 /ft (24,800 kips) with the vertical distribution of mass shown in Figure 1.

Primary longitudinal (N-S) stiffness against seismic loading comes from the 4 ft thick shear walls while the transverse (E-W) stiffness is provided by frame action of the reinforced-concrete walls.

The original design was based on the Uniform Building Code (UBC, 1946) with a 1951 Addenda and on the American Concrete Institute (ACI) Code ACI 318-47. The design focused on gravity loading with only a nominal seismic lateral load applied statically to the building structure. The exterior walls were designed to resist an external blast pressure resulting in heavier reinforcement on the inside face of the walls than the outside face. Many of the embedment and splice lengths of the reinforcing steel do not satisfy current ACI specifications with some embedment lengths as small as 25 percent of that required by the current code.

A typical joint that connects the exterior canyon roof to the exterior wall is shown in Figure 2. This joint was designed for gravity loads and the bottom slab reinforcing is not fully anchored in the wall. Seismic loads will cause load reversal, putting the bottom bars in tension and the capacity of this underdeveloped bar is reduced due to bond slip. This joint is 43 feet long and the geometry of the joint constrains the concrete around the reinforcing bar.

Behavior of Partially Developed Constrained Joints

Rotations are imposed on the joints when a frame drifts during seismic loads. Joints that do not meet the ACI 318-95 development length requirements may not develop their full yield moment. In this evaluation, the ACI bond stress is assumed to limit the bending capacity ϕ Mn, of a partially developed joint, by the ratio of actual development length, l, to the ACI development length, l_d. The amount of bar slip at the reduced moment ϕ Mn l/l_d, is determined by the rotation imposed on the joint and ultimately by the lateral drift of the frame as shown in Figure 3. Failure is defined as the inability of a joint to resist the moment ϕ Mn l/l_d, at a given drift.

Typical confined bar pullout test results (Eligehausen, 1983), are shown in Figure 4. Monotonic test indicate that for low magnitudes of bond slip the bars have a much larger capacity than the ACI 318-95 code allows and that the code bond capacity corresponds to the capacity at large deformations. Cyclic tests indicate that the capacity degrades with an increasing number of cycles and the degradation is more pronounced when cycled with larger ranges of slip. After 10 cycles, the code capacity is obtained when loading beyond the maximum post bond slip. This test data was reviewed and judged that 13% and 75% of the specimen would fail to maintain the ACI bond capacity at peak slip ranges of 0.2 in and 0.36 in respectively. A log normal probability distribution is fit to these two failure estimates as shown in Figure 5.

Test of full scale joints with partially developed bars, shown in Figure 6 (Beres, 1992, Aycardi, 1992), demonstrate that the joint is capable of resisting considerable rotation at the reduced bending moment ϕ Mn l/l_d. Joint rotations for this data are converted to bar slip, ranked, and used to validate the bar slip probability of failure in Figure 5. Note that the bar slip in Figure 3 is the product of joint rotation and the distance between the neutral axis to the reinforcing, d. A joint rotation of 0.02 radians of the 24 inch deep test specimen, in Figure 6, causes the same bond slip as a 0.01 radian rotation on a section that is 48 inches thick. Thus, allowable joint rotation limits should be viewed with caution when evaluating thick sections.

Static Pushover Analysis

In the east-west direction, the canyon structure consists of a moment resisting lateral frame with a rigid penthouse structure. A static pushover analysis of the lateral load resisting frame is performed to determine the building's lateral load capacity and the relationship between displacement and joint rotation.

Nonlinear springs are located at critical joints at the ends of elastic beam elements to represent both yielding of the concrete members and bond slip due to inadequate development, as shown in Figure 7. A trilinear curve is used at each joint to represent the initial elastic, cracked, and yield stiffness. The members' stiffness are developed from the moment curvature diagram. In partially developed joints, the members' capacity is truncated at ϕ Mn l/l_d.

A typical monotonic load-deformation curve or backbone curve for the structure is shown in Figure 8. The structure remains elastic below a base shear of 1300 kips. Above this load, individual joints crack and yield at different load levels which gradually soften the overall structure. A plastic collapse mechanism is nearly formed after six inches of displacement. For displacements beyond this point, the slight increase in capacity due to strain hardening is nearly offset by the increasing P-Δ forces. The ultimate capacity is $\approx$2800 kips or 11% of the structure's weight.

Rotations corresponding to each increment of roof drift are converted into components of bending rotation and bond slip. Bond slip failure probabilities from Figure 5 are assigned to each joint based on the computed bar slip. Bending failure probabilities are also assigned using a similar relationship and combined with the bond slip failure probability. The building is assumed to fail when any critical member fails or when global instability is detected.

The probability of joint failure, shown in Figure 9, is an envelope of the individual joint failure probabilities for the critical members. For roof displacements less than 4 inches (0.5% drift) the probability of failure is negligible while a roof displacement of 5.8 inches (0.8% drift) corresponds to a 50% probability of failure. The probability of failure for the canyon building is dominated by a joint on the exterior frame that fails by bond slip.

Nonlinear Dynamic Analysis

A fragility analysis for a reinforced-concrete structure subjected to strong earthquake motions requires realistic conceptual structural models that consider changes in stiffness and material properties, the variability of seismic source and in-situ soil conditions. To adequately address these parameters, numerous non-linear time history analyses are performed, and in this study over two hundred analyses are required. Since non-linear time history analyses are numerically intensive, reduced dynamic models are used to economically perform the analyses.

Ground Motion

The seismic hazard and geotechnical analyses include probabilistic determination of bedrock outcrop motion ground motion with a 2,000 year return period and a ground motion with a 10,000 year return period. These motions are in the form of 5 percent damped, horizontal component uniform hazard response spectra.

Site response analyses were used to develop a suite of 96 time histories from which 11 records were selected for each earthquake level for input to the structural analyses. The median response spectra of these time histories are shown in Figure 10 for the 2,000-year and the 10,000-year events.

Elastic Modeling

The MDOF finite element model of the structure developed in this study was reduced to a single-degree-of-freedom (SDOF) model using a Guyan reduction procedure. The elastic natural period of the structure is on the order of 0.8 to 1.0 seconds. Elastic, dynamic time history analyses of both the MDOF and SDOF model are performed to insure that the MDOF and SDOF model produce comparable roof drifts. The results of the comparison of roof drifts indicate that the structure's dynamic response is dominated by its first fundamental mode, and the SDOF model adequately reproduce the response. Further study of displacements and rotations throughout the MDOF structural model indicated in-phase behavior, confirming that accurate prediction of roof drifts also predicts joint rotations. This is critical in the analysis since the joint rotations are essential to establish failure probabilities.

Non-Linear Analyses

Having demonstrated the linear, elastic correlation between roof drifts of the two building models, the elastic behavior of the SDOF model is replaced with a Takeda hysteresis model (Takeda, 1970) to compute non-linear time history responses. The Takeda model represents the experimentally observed behavior of reinforced concrete beams subject to cyclic loads, and is shown in Figure 11. The objective is to demonstrate that the SDOF model using the Takeda concrete model, based solely on the roof displacement of the pushover model, predicts non-linear roof drift and joint rotations.

The Takeda spring is based on the backbone curve generated from a MDOF pushover model. The maximum displacement from the SDOF model is used to determine joint rotations at that same displacement in the MDOF pushover model. The time history response of the MDOF pushover model, modified to include Takeda springs at critical joints, is computed. Comparison of roof drifts and joint rotations between the two approaches indicate that, on the average, the SDOF model predicts displacements 20% larger than the MDOF model. Study of the MDOF responses indicates that the non-linear behavior increases the tendency of the structure to respond in a fundamental mode and further supports the adequacy of the SDOF model.

Fragility Analysis

The acceptance criteria in DOE Standard 1020 uses probabilistic performance goals. To compare the results of the structural analysis described above to probabilistic criteria, a fragility analysis is used to develop the median seismic capacity and the associated variability. The median capacity is defined as the lateral drift that results in a 50% probability of structural failure. The median capacity and the associated variability are assigned a log-normal distribution to provide a cumulative probability distribution, or fragility curve, for the structure.

The variability in a fragility represents a measure of both the uncertainty, or lack of knowledge of the structural behavior, and randomness in the earthquake motion. The variability is the logarithmic standard deviation on the distribution used in a fragility analysis. The sources of variability evaluated are: (1) input time-histories; (2) stiffness uncertainty; (3) material strength; (4) the correlation between ground motion and drift; and (5) drift capacity.

The variability in the input time-histories and the building stiffness are combined into the variability in response. The variation in the drift values accounts for both input time history randomness and uncertainty in the structural stiffness. The variability associated with the response of the structure, β_R, is calculated by fitting the drift response from the nonlinear SDOF analysis to a log-normal distribution and is β_R=0.28.

Variability in material properties, β_S, is determined by performing analyses at both 95% exceedance strength and median concrete strength. The effect on the building drift at the different capacity levels is negligible and the strength variability is β_S=0.0.

Seismic fragility for a structure is correlated to a ground motion parameter, β_{EQ}, representing the magnitude of the earthquake. For large earthquakes, the non-linear structure responds at very low frequencies of 0.35 to 0.9 Hz and spectral displacement is the ground motion parameter that provides the best correlation between building drift and input motion at this frequency range. Based on judgment a variability of β_{EQ}=0.05 is estimated for the ground motion correlation.

Drift capacity variability, β_C, is determined by fitting the failure data from the static pushover analysis to a log-normal distribution. Fitting the joint rotation data to a log-normal distribution provides a variability of β_C=0.17. The median capacity (50 percent probability of failure) shown in Figure 8 is 5.8 inches.

The variabilities for each individual source are summarized in Table 1. These variabilities were combined using the square root of the sum of the squares to determine a composite variability, β=0.29. The median capacity and the composite variability are used to define the fragility curve as shown in Figure 12.

Table 1 Fragility parameters

	Logarithmic Standard Deviations				
Median Capacity, S_D (in.)	Response β_R	Strength β_S	Equation β_{EQ}	Capacity β_C	Composite β
5.8	0.23	0.00	0.05	0.17	0.29

The second criteria in the DOE 1020 Standard for showing acceptable performance of the structure is to demonstrate that a less than 10% conditional probability of failure exists at 1.5 times the design basis earthquake. The drift from the design basis event was determined to be 2.1 inches, therefore the factored drift is 3.2 inches. The conditional probability of failure determined from the fragility curve for a 3.2 in drift is one percent, which is much lower than the 10% acceptance probability for this criteria.

The third criteria for acceptable performance allows demonstration of acceptable structural behavior by meeting a performance goal. For this facility the acceptable annual probability of failure is less than 2×10^{-4}. The annual probability of failure is obtained by convolving the fragility curve with the mean site hazard curve to determine an estimate of the mean annual probability of failure. The calculated annual probability of seismic failure is determined to be 1.8×10^{-4} which is less than the performance goal of 2.0×10^{-4}.

For the SRS canyon structures, both probabilistic approaches allowed predict that the building seismic capacity meets the performance goal.

Comparison With Alternate Evaluation Methodologies

FEMA-273 / 274 Evaluation Methodology

The rigorously computed drifts discussed above are compared to drifts estimated using the methodology described in the ballot version of FEMA 274. This method consists of developing a pushover curve describing the displacement of the roof of the structure with increasing load while accommodating non-linear behavior of the structural components. This pushover curve is described in terms of spectral acceleration vs. roof drift and is then overlain on a 10% damped spectral acceleration vs. spectral displacement plot of the median site response spectra as shown in Figure 13. The predicted demand for the 2,000 year return period earthquake and conservative material properties is 2.2 in as compared to the 2.1 in drift calculated by nonlinear analysis. For the more demanding earthquake with a 10,000 year return period, the predicted drift is 6.7 in which is consistent with the drift of 6.1 in calculated by the nonlinear analysis.

Acceptance criteria for plastic rotation angle, radians, are proposed in the ballot version of FEMA 273 for beams and columns for three performance levels; Immediate Occupancy, Life Safety, and Collapse Prevention. These criteria provide limiting allowable plastic rotations for collapse prevention of 0.015 radians for concrete beams and 0.005 radians for columns controlled by flexure. Beam joint rotations predicted at 5.8 in roof displacement, corresponding to 50% probability of failure, range from 0.009 to 0.013 radians while column joints are on the order of 0.003 to 0.008 radians. These predicted rotations compare well with the Collapse Prevention acceptance criteria proposed in FEMA 273.

ATC 19 Computed Structural Response Modification Factor

Building codes typically use a design base shear at allowable stress levels that is computed using an expected design spectral value divided by an R factor (1994 UBC) of the form: ZICW /R.

ATC-19, provides a formulation in which R is expressed as the product of a period dependent strength factor, R_s, a period dependent ductility factor, R_u, and a redundancy factor, R_r.

The canyon structures were originally designed for a base shear of 846 kips. The displacement associated with the low failure probability 2000-year evaluation basis earthquake is 2.2 in corresponding to a base shear of 1400 kips. The increase in base shear capacity reflects the increased capacity developed in the original design to carry non seismic loads. The strength factor is estimated as 1400/846=1.65.

The ductility factor is approximated based on the equal-energy method and assumes that the yield force is equal to the maximum base shear. This approximation neglects the reserve capacity available at larger drifts and results in a lower estimate of the R factor. The maximum displacement using these data is 2.2 in and the yield displacement is 0.75 in. The displacement ductility ratio is calculated as $\mu = \Delta m/\Delta y = 2.9$. For a fundamental period of approximately 1 second, $R \approx \mu$, and the ductility factor is equal to 2.9. Combining the strength factor, Rs, the ductility factor, Ru, with the redundancy factor, R_R=0.71 (suggested by ATC-19 Table 4.3, for 2 lines of vertical seismic framing), results in an estimate of $R = 1.65 \times 2.9 \times 0.71 = 3.4$. This expected R is between the ratio of the elastic demand to the elastically computed base shear capacity R=2.2 and the ratio of the peak spectral demand to elastic capacity, R=4.7, shown in Figure 14.

Conclusions

A methodology to predict structural behavior in older DOE reinforced concrete facilities was implemented and results compared to the simpler methods proposed by FEMA 274 and ATC-19. These methodologies can be used to demonstrate seismic capability consistent with the performance goals stated in DOE Standard 1020 when traditional deterministic methods indicate deficient seismic designs. Adaptations of the methodology can be used to meet performance goals developed for other nuclear and non-nuclear buildings.

Key to the development of realistic estimates of the ultimate lateral load resisting capacities for existing structures is the understanding of the behavior of reinforced concrete joints with partially developed bars. These joints do not necessarily demonstrate brittle behavior and the limited rotation capability of partially developed joints can contribute significantly to the overall ductility of the reinforced concrete frame. Notwithstanding the ductility available when estimating the limit state, significant damage in the structure is expected to occur, thus new construction should fully develop bars.

The canyon structures have an elastic period on the order of 0.8 to 1.0 second, which increases further as the lateral load increases. This high period structure is displacement controlled thus computed drifts are relatively insensitive to the initial structural stiffness and capacity. The drifts are a function of the low frequency displacements in the ground motions.

Acknowledgments

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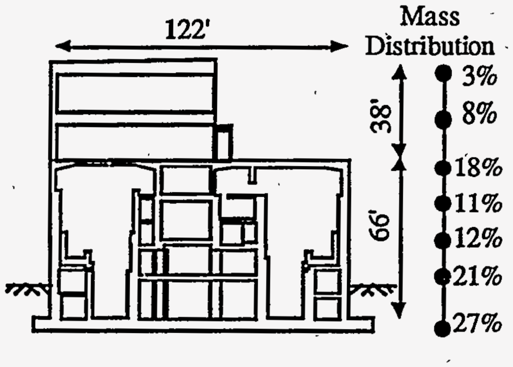

Figure 1. Frame cross section and vertical mass distribution

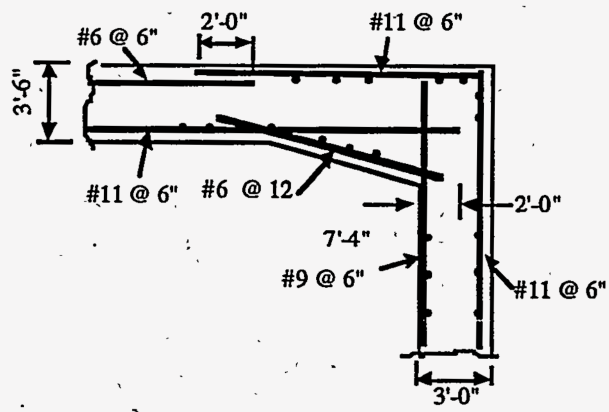

Figure 2. Typical joint detail

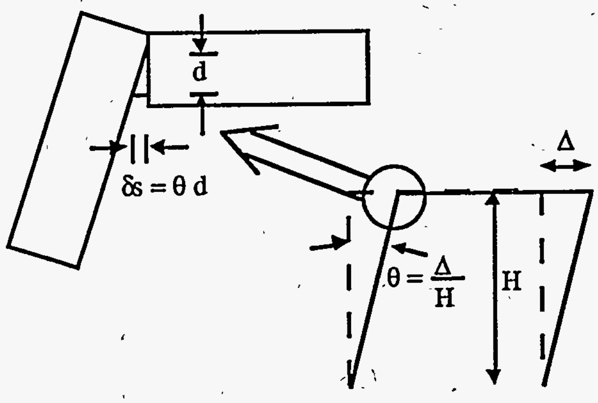

Figure 3. Relationship between bar slip and drift

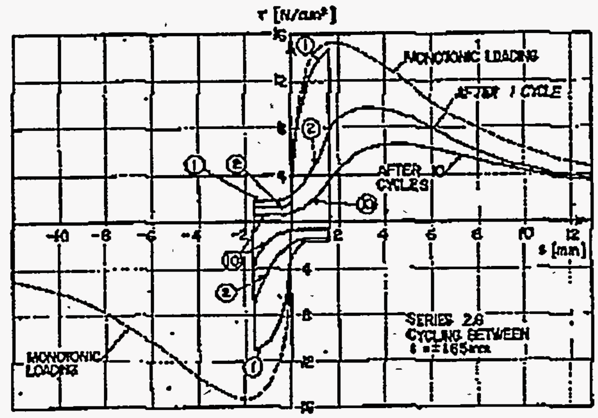

Figure 4. Bar pullout test (Eligehausen, 1983)

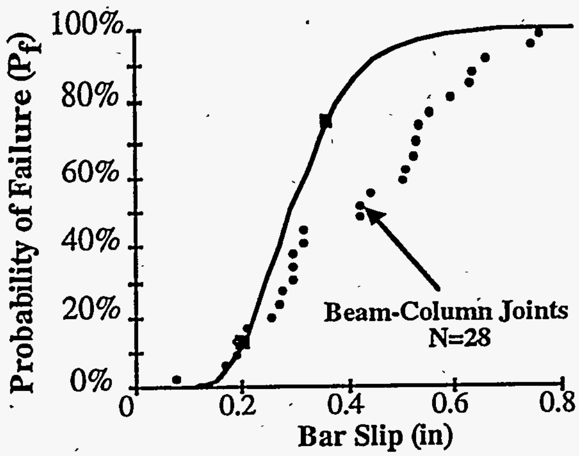

Figure 5. Probability of failure versus bar slip for confined joints

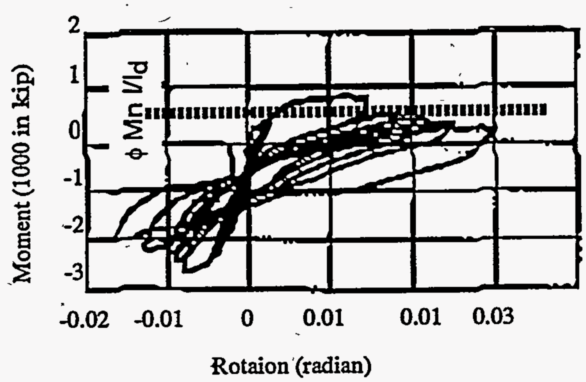

Figure 6. Full scale joint test (Beres, 1992)

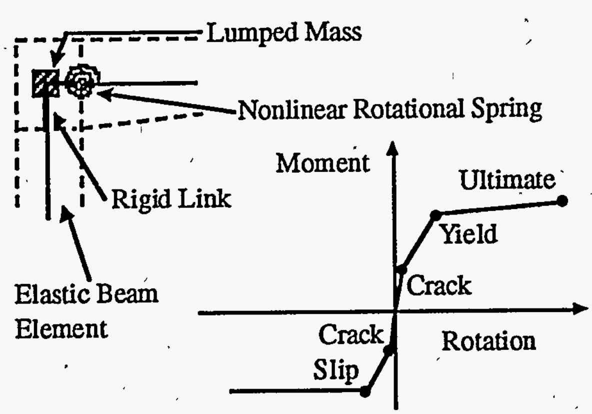

Figure 7. Nonlinear joint spring

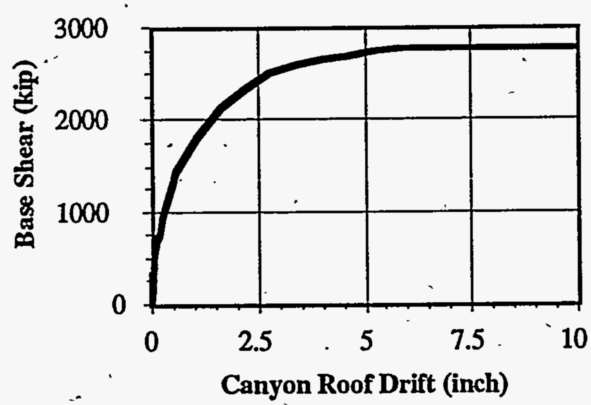

Figure 8. Backbone curve.

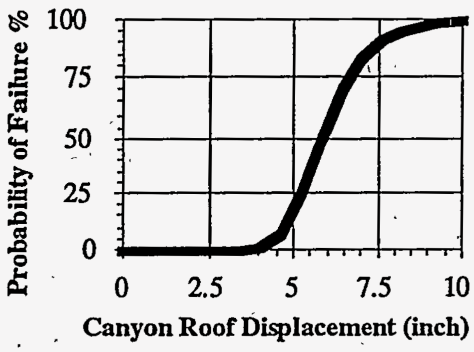

Figure 9. Probability of failure versus roof displacement

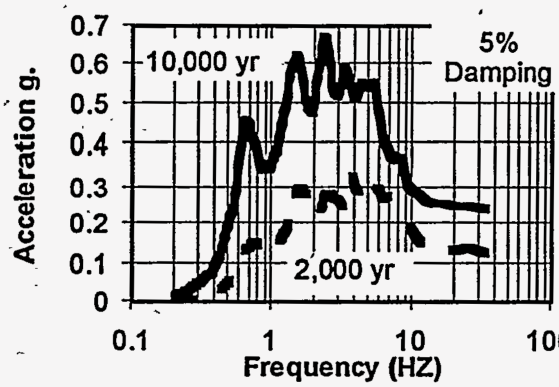

Figure 10. Mean site specific spectra

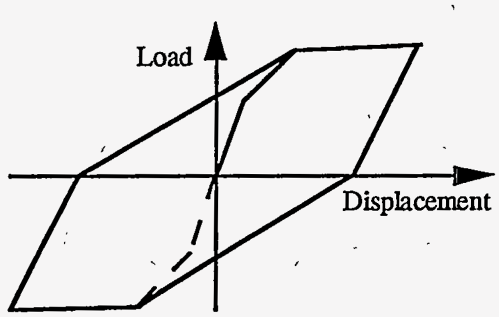

Figure 11. Takeda hysteresis model

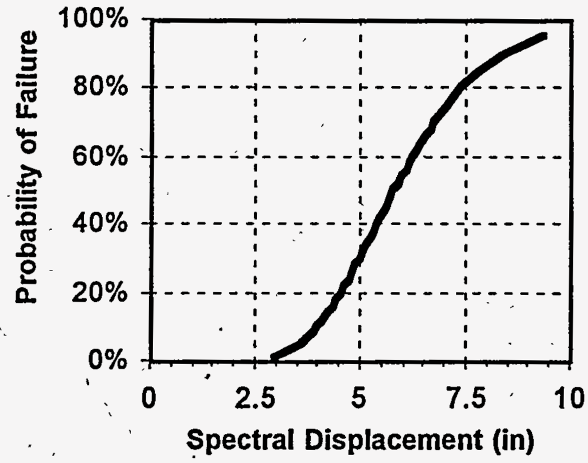

Figure 12. Building fragility curve

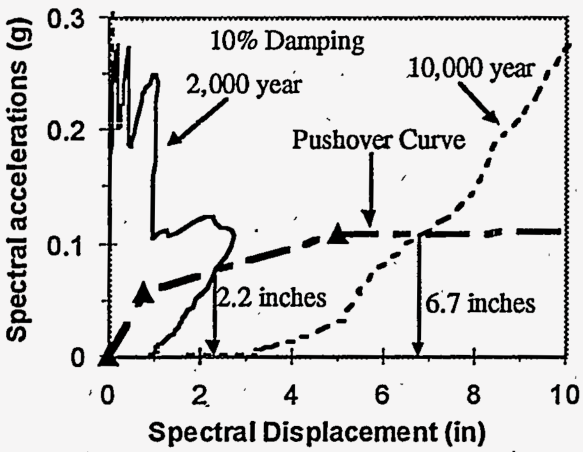

Figure 13. Spectral acceleration versus spectral displacement

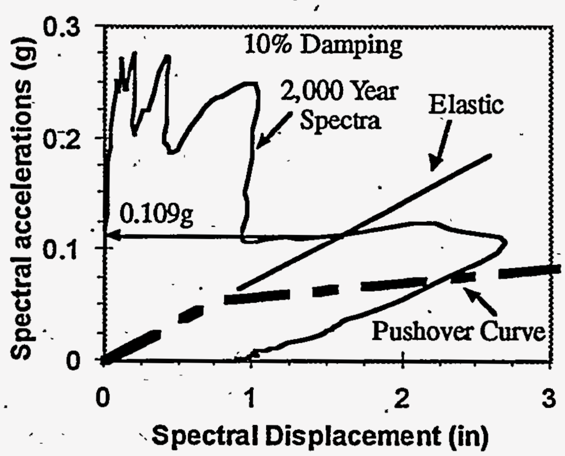

Figure 14. Elastic versus nonlinear response, 2000 year spectra