13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 254

SEISMIC PERFORMANCE-BASED EVALUATION OF NUCLEAR FACILITY STRUCTURES

Nebojsa ORBOVIC[1], Bozidar STOJADINOVIC[2], Marc BOUCHON[3], Tim WILEY[4]

SUMMARY

Periodic evaluation of structures in nuclear facilities is imperative to insure such structures remain safe. Severe failure consequence associated with nuclear facility structures preclude direct application of seismic performance-based evaluation procedures developed for existing conventional structures. A link between FEMA-356 evaluation acceptance criteria in terms of DEO-1020 risk reduction factors is presented. Using this link, acceptance criteria for conventional structures coupled with probabilistic evaluation of seismic hazard and improved capacity-side fragility of structural components may form an acceptable framework for performance-based evaluation of existing nuclear facility structures. Use of such framework is illustrated on a reinforced-concrete industrial building.

INTRODUCTION

Periodic evaluation and upgrading of structures in nuclear facilities must be done to insure they remain safe during and continue to function after a possible earthquake. Probabilistic nature of earthquake hazard to nuclear facilities is well understood. An elaborate set of performance objectives for design of new nuclear facility structures has also been developed. However, there is little guidance for practicing engineer on how to evaluate the performance of an existing structure in a nuclear facility. Furthermore, severe consequence associated with some nuclear facility structural failures preclude direct application of seismic performance-based evaluation procedures developed for existing conventional structures.

A review of seismic performance-based procedures used in the United States to evaluate the existing conventional and nuclear facility reinforced concrete structures is presented in this paper. The probability-of-failure basis for design and evaluation of nuclear structures is presented first. Then, procedures documented in DOE-1020-2002 [4] standard for nuclear structures, in FEMA-356 [5] pre-standard for

[1] Senior Engineer, Institut de Radioprotection et de Sûreté Nucléaire (IRSN) B.P. 17 – 92262 FONTENAY AUX ROSES CEDEX - FRANCE IRSN
[2] Associate Professor, Department of Civil and Environmental Engineering, University of California, Berkeley, Berkeley, CA 94720-1710, USA. e-mail: boza@ce.berkeley.edu
[3] Senior Engineer, Institut de Radioprotection et de Sûreté Nucléaire (IRSN) B.P. 17 – 92262 FONTENAY AUX ROSES CEDEX - FRANCE IRSN
[4] Engineer, Degekolb Engineers, Oakland, CA, USA

conventional structures, and in FEMA-350 [6] recommendations for new steel structures, are compared to a comprehensive performance-based evaluation methodology developed in NSF's PEER Center. This comparison demonstrates the common probabilistic basis for these procedures. Furthermore, a relation between the procedures is established: this relation is used to link the deterministic acceptance criteria in FEMA-356 to the risk reduction goals stated in DOE-1020.

This review shows that acceptance criteria defined FEMA-356 for conventional structures, coupled with a customary site-specific probabilistic evaluation of seismic hazard and an improved evaluation of capacity side fragility of structural components may be an acceptable framework for developing a performance-based evaluation procedure for existing nuclear facility structures. A possible application of such framework is illustrated using a reinforced-concrete industrial building example. Ongoing work is focused on the development and implementation of such new seismic evaluation framework for nuclear facility structures.

GENERAL PROBABILISTIC PERFORMANCE-BASED EVALUATION FRAMEWORK

Probabilistic performance-based seismic evaluation framework, developed by the PEER Center [2], is a general framework based on acceptance criteria formulated by stating an acceptable level of probability of exceeding a performance limit state for a given structure in its seismic risk environment. The probability of exceeding a limit state is computed using a total probability integral equation:

$$P(DV) = \int_{dm}\int_{edp}\int_{im} P(DV|DM)\,dP(DM|EDP)\,d\,P(EDP|IM)\,d\,|dH(im)| \qquad [1]$$

The following symbols are used:

- *DV* – decision variable, used to quantify the performance limit state. The limit state notion is not restricted: it may be associated with structural response, non-structural damage, loss of function or monetary costs;
- *DM* – damage measure, used to quantify the ability of a structure sustain a limit state in engineering terms. In conventional design, a damage measure is associated with the capacity (*C*) of the structure or structural elements;
- *EDP* – engineering demand parameter, used to quantify the threat of an earthquake to the structure in engineering terms. In conventional design, an engineering demand parameter is simply the seismic demand (*D*); and
- *IM* – intensity measure, used to quantify the intensity of ground motion. Spectral acceleration at the fundamental period of the structure ($S_a(T_1)$ or simply s_a) is the primary intensity measure used in PEER Center research.

Evaluation of the total probability of exceeding a limit state using the PEER framework equation involves evaluation of randomness associated with ground motions and uncertainty associated with methods of evaluating intensity measures, engineering design parameters, damage measures and decision variables. Therefore, the probability of exceeding a limit state cannot be established with absolute certainty: instead, a finite amount of confidence is associated with the computed probability of exceeding a limit state.

FEMA-350 confidence-based design

Recent intensive research to develop design provisions for new steel moment frame buildings after the fully restrained moment connection failures during the 1994 Northridge earthquake yielded a number of important innovations [6]. Confidence-based design provisions, developed by Cornell and coworkers [3], [8] and [13], represent the first code-of-practice application of probabilistic performance-based seismic design for conventional structures. These provisions are derived from a truncated version of the general probabilistic performance-based design framework equation. Namely, the decision variable step is omitted to evaluate the probability that a performance limit state is not met by a direct comparison of demand and capacity:

$$P_F = P(C \le D) = \int_d \int_{s_a} P(C \le D) P(D \ge d \,|IM \ge s_a) |dH(s_a)| \quad [2]$$

The variables in this equation are: P_F – probability of failure (not meeting the performance limit); C – capacity; D – demand; and $H(s_a)$ – seismic hazard measured in terms of annual probability of exceeding of a value of ground motion spectral acceleration at the fundamental period of the structure are treated as random variables with log-normal distributions. Log-normal distribution is chosen because it realistically represents the uncertainty in determining the random variable and also enables a closed form evaluation of the total probability integral [2]. Therefore, the ground motion hazard is described by its median value $\hat{H}(s_a)$ and dispersion β_H; assuming a log-normal distribution, the mean estimate of ground motion hazard is:

$$\bar{H}(s_a) = \hat{H}(s_a) \exp(\frac{1}{2}\beta_{\mathrm{H}}^2) \quad [3]$$

The mean estimate of ground motion hazard, which implicitly includes its variability, is chosen because US Geological Service provides mean hazard data for spectral acceleration and velocity for the territory of United States. The spectral acceleration hazard corresponding to a probability P_H of exceeding a value of s_a can be approximated using:

$$P_H = H(s_a^{P_H}) = k_0 (s_a^{P_H})^{-k} \quad [4]$$

implying that the relation is locally linear on a log-log plot. Typical values of the slope k are between 1 and 4 [7].

Similarly, distribution of demand is log-normal with a median value of $\hat{D}$ and dispersions $\beta_{DT} = \sqrt{\beta_{DU}^2 + \beta_{DR}^2}$ equal to the geometric mean of the dispersion due to the uncertainty in determining demand and ground motion record-to-record randomness. Moreover, log-normal distribution of capacity is defined by a median value of $\hat{C}$ and dispersions $\beta_{CT} = \sqrt{\beta_{CU}^2 + \beta_{CR}^2}$. Both demand and capacity are expressed using the same engineering quantity, such as drift or shear force. The relation between the hazard intensity measure and the demand (or capacity) engineering quantity is established by a statistical fit of the data generated by a number of nonlinear time-history analysis runs using a representative set of ground motions. Two procedures, the bin approach [10] and incremental dynamic analysis [12] may be used to compute such statistical fits and express them in linear log-log form:

$$\hat{D} = a(S_a^{P_F})^b; \quad \hat{C} = a(S_a^{\hat{C}})^b \quad [5]$$

Note that the value of $b = 1$ corresponds to the equal displacement rule which applies for a significant majority of regular structures.

Using the properties of the standard normal distribution function and integrating equation [2] in closed form [Cornell et.al, 2002], the mean value of probability of failure is:

$$\bar{P}_F = \bar{H}(s_a^{\hat{C}})\exp[\frac{1}{2}\frac{k^2}{b^2}(\beta_{DU}^2 + \beta_{DR}^2 + \beta_{CU}^2 + \beta_{CR}^2)] \qquad [6]$$

The log-normal distribution of P_F is defined by median and dispersion values as follows:

$$\hat{P}_F = \bar{H}(s_a^{\hat{C}})\exp[\frac{1}{2}\frac{k^2}{b^2}(\beta_{DR}^2 + \beta_{CR}^2)];\ \beta_{P_F} = \sqrt{\frac{k^2}{b^2}(\beta_{DU}^2 + \beta_{CU}^2)} \qquad [7]$$

Thus, the failure probability level associated with an *x*% confidence level is:

$$P_F^x = \hat{P}_F \exp(K_{x,}\beta_{P_F}) \qquad [8]$$

where K_x is the standardized Guassian variable associated with probability x of not being exceeded.

Setting the mean probability of failure equal to the probability of exceeding a hazard level ($\bar{P}_F = P_H$, implying a risk reduction ratio equal to 1.0) in Equation [6], this equation can be re-arranged into a familiar load and resistance factor design format:

$$\phi\hat{C} \geq \gamma\hat{D}^{P_H} \qquad [9]$$

where the capacity reduction factor and the load factor are:

$$\phi = \exp(-\frac{1}{2}\frac{k}{b}\beta_{CT}^2); \quad \gamma = \exp(\frac{1}{2}\frac{k}{b}\beta_{DT}^2) \qquad [10]$$

The acceptance criterion associated with the probability of failure P_H is formulated in terms of a confidence ratio:

$$\lambda_{con} = \frac{\gamma\hat{D}^{P_H}}{\phi\hat{C}} = \exp(-K_x\beta_{UT} + \frac{1}{2}\frac{k}{b}\beta_{UT}^2) \qquad [11]$$

where the total uncertainty $\beta_{UT} = \sqrt{\beta_{DU}^2 + \beta_{CU}^2}$. Confidence ratio enables direct computation of the confidence level associated with the probability of failure P_H as tabulated in [Yun et.al, 2002]. For example, assuming a total uncertainty $\beta_{UT} = 0.3$, a factored demand equal to reduced capacity ($\lambda_{con} = 1$) gives approximately 70% confidence that the probability of failure is equal to the probability of exceeding a given hazard level (risk reduction ratio equal to 1.0) for the hazard curve slope $k = 3$ characteristic for the US West Coast seismic risk environment.

For conventional structures considered in FEMA-350, hazard levels described by 2% and 10% recurrence probabilities in any 50-year period are associated with the collapse prevention and immediate occupancy limit states. Furthermore, two confidence levels are associated with satisfying the performance goal (having the probability of failure less or equal the probability of seismic hazard): 90% confidence is required for the performance of the structure as a system, while 50% confidence is required for structural element performance.

DOE-1020 risk reduction factor design

Probability-based design and evaluation of nuclear facility structures has been used since the 1980's. The high levels of risk have forced engineers to use the best available tools to realistically evaluate the probability of failure, and yet put them in a practical design provision form. US Department of Energy nuclear facilities are designed following the DOE-STD-1020 Code [4]. The acceptance criteria used in this code have been developed by Kennedy and coworkers [7]. They are based on evaluating the probability of unacceptable performance using, also, a truncated form of the general probabilistic performance-based design framework equation. Namely, the decision variable step is omitted and the evaluation of capacity-demand comparison is folded into a fragility function, the conditional probability of failure given a ground motion intensity level, to evaluate the probability that a performance limit state is not met:

$$P_F = P(C \le D) = \int_{s_a} P(F\,|s_a)\,|dH(s_a)| \qquad [12]$$

The principal difference between the conventional and nuclear facility structures is that the more stringent performance criteria for nuclear structures (PC-3 and PC-4) are accepted if they have a probability of failure P_F that is reduced with respect to the probability of seismic hazard they are exposed to P_H. Note that in FEMA-350 the probability of failure is equal to the probability of seismic hazard occurrence, meaning that the structural code provisions are risk-neutral. The amount of risk reduction is quantified using a risk reduction ratio:

$$R_R = \frac{P_H}{P_F} \qquad [13]$$

DOE-STD-1020 performance criteria PC-1 and PC-2 adopt design provisions for conventional structures and do not have a specified risk reduction ratio (although a risk reduction ratio of approximately 1.0 is implied in [7]), while performance criteria PC-3 and PC-4 have risk reduction ratios of 10 and 20, respectively.

Relation between confidence ratio and risk reduction ratio

Given the common root of the acceptance criteria in FEMA-350 and DOE-STD-1020, the confidence ratio and the risk reduction ratio can be related. Using the mean value of seismic hazard (equation 4) and a median value of probability of failure (equation 7) in equation 13:

$$R_R = \frac{1}{\lambda_{con}^{k/b}} \exp\left(\frac{1}{2}\frac{k^2}{b^2}\beta_{UT}^2\right) \qquad [14]$$

Using this equation, the risk reduction factor implied by the FEMA-350 acceptance criteria for evaluation of conventional structures can be computed. Assuming that equal displacement rule applies (b=1), Table 1 shows values of risk reduction ratios computed with respect to the probability of seismic hazard P_H using equation 14 for different values of seismic hazard parameter *k*, and total uncertainty β_{UT}. Five levels of confidence that the performance is satisfactory are considered even though the FEMA-350 acceptance criteria consider only two (50% and 90%). Table 1 in [13], which lists the values of the confidence ratio λ_{con} as a function of the desired confidence level, seismic hazard parameter k, and total uncertainty β_{UT}, was used. The risk reduction factors implied by the FEMA-350 confidence level for satisfying the immediate occupancy performance objective (50%) and collapse prevention performance objective (90%) on the US West Coast (k is between 3 and 4) are approximately 1 and 4, respectively, assuming a total uncertainty of 0.3. These limit states correspond roughly to DOE-STD-1020 limit states PC-1 and PC-2 for nuclear facility structures, with the implied risk reduction ratio of 1.0, according to [Kennedy and Short, 1994]. Note also that achieving a very high confidence level of 99% with linearly increasing total uncertainty requires an exponentially higher risk reduction ratio.

Table 1. Risk reduction ratios for FEMA-350 conventional structure evaluation.

R_R	Conf.=50%	Conf.=70%	Conf.=90%	Conf.=95%	Conf.=99%
k	$\beta_{UT} = 0.2$				
1	1.00	1.11	1.29	1.40	1.59
2	1.00	1.23	1.65	1.93	2.56
3	1.01	1.35	2.17	2.73	3.98
4	1.01	1.49	2.77	3.72	6.44
k	$\beta_{UT} = 0.3$				
1	1.00	1.18	1.47	1.63	1.97
2	1.00	1.38	2.19	2.07	4.11
3	1.01	1.59	3.16	4.37	8.10
4	1.00	1.90	4.54	7.23	15.85
k	$\beta_{UT} = 0.4$				
1	1.00	1.23	1.67	1.93	2.52
2	1.01	1.53	2.81	3.70	6.51
3	1.00	1.88	4.68	7.15	16.44
4	1.00	2.29	7.96	14.15	42.30

DOE-STD-1020 performance categories PC-3 and PC-4 imply risk reduction ratios of 10 and 20, meaning that the probability of failure is 10 or 20 times less likely than the probability of seismic hazard P_H. Achieving such risk reduction factors using the FEMA-350 framework is possible by computing the required confidence ratio λ_{con}. Using Equation 14:

$$\lambda_{con} = \frac{1}{R_R^{b/k}} \exp\left(\frac{1}{2}\frac{k}{b}\beta_{UT}^2\right) \qquad [15]$$

Required confidence levels are listed in Table 2, using the same assumptions used to compute the values in Table 1. Note that the required confidence levels to achieve a desired risk reduction are rather high: this is consistent with the level of safety expected for nuclear facility structures required to satisfy performance criteria PC-3 and PC-4. In terms of the FEMA-350 acceptance criteria for conventional structures, this means that the median capacity has to be significantly larger than the median demand.

Table 2. Confidence levels corresponding to DOE-STD-1020 risk reduction ratios for PC-3 and PC-4 performance criteria.

Conf.	$R_R = 10$	$R_R = 20$	$R_R = 10$	$R_R = 20$	$R_R = 10$	$R_R = 20$
k	$\beta_{UT} = 0.2$		$\beta_{UT} = 0.3$		$\beta_{UT} = 0.4$	
1	>99%	>99%	>99%	>99%	>99%	>99%
2	>99%	>99%	>99%	>99%	>99%	>99%
3	>99%	>99%	>99%	>99%	~97.5%	>99%
4	>99%	>99%	~97.5%	>99%	~92.5%	~97.5%

EVALUATION OF DEMAND AND CAPACITY IN FEMA-356

Demand is usually evaluated using a mathematical model of the structure and one or more of four types of analysis procedures: linear static, linear dynamic, non-linear static and non-linear dynamic. Modeling requirements are often specified in support documents such as FEMA-356 [5] for conventional structures and ASCE-4-98 [1] for nuclear facility structures. The principal outcome of demand evaluation are estimates of seismic demand, associated with a seismic hazard with a probability P_H , for the structural elements that form the structure. Assuming that non-linear dynamic analysis produces accurate results, factors expressing the bias (overshoot or undershoot) of other demand analysis procedures with respect to non-linear dynamic analysis can be computed. A FEMA-350 γ_1 bias factor has been determined for regular steel moment frames. FEMA-356 factors C_1, C_2, C_3 and J in equation 3-19 may also be interpreted as bias factors, but not without further study.

Capacity is usually evaluated using physical models of structural elements and different types of experimental procedures: static monotonic testing, quasi-static cyclic testing, pseudo-dynamic testing, and shaking-table dynamic testing. Capacities are, sometimes, evaluated using calibrated analytical models made using sophisticated finite element software. The principal outcomes of capacity evaluation are code equations that prescribe a way to compute the capacity of structural elements. Capacity data for evaluation of conventional structures has been collected in the FEMA-356 pre-standard. The data is presented in terms of load-response envelopes for common structural elements. Capacity of each structural element is associated with one of three types of response envelopes, shown in Figure 1. The shape of the response envelope, defined by the yield strength of a structural element Q_y and values a, b and c, is specified for each structural element addressed in FEMA-356. The yield strength values for structural elements are specified either as expected (Q_{CE}) or lower-bound (Q_{CL}) strength values at the deformation level under consideration. All response envelope values have a strong foundation in the observed behavior of structural elements either in experiments or during earthquakes, but they have not been statistically rendered. Thus, it is not possible to determine the relation between the expected Q_{CE} or lower-bound Q_{CL} and median C capacities without further study.

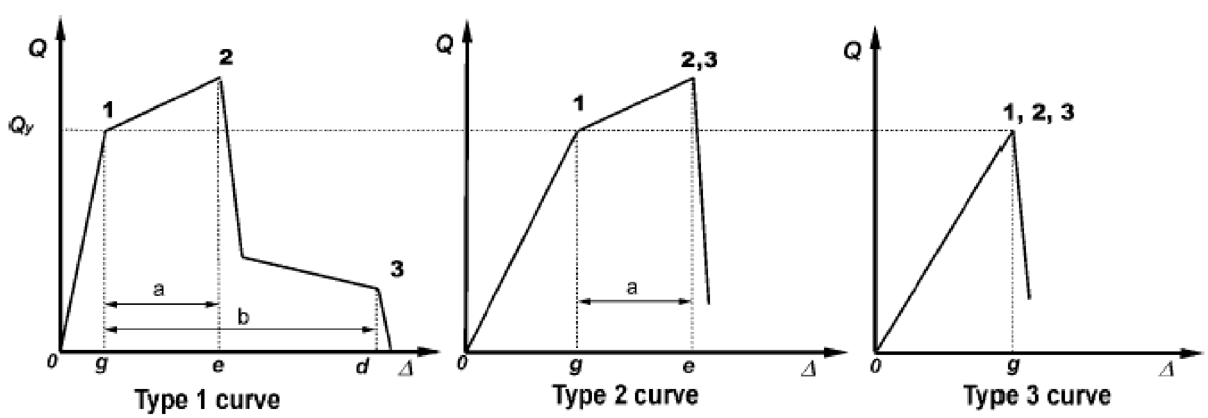


Figure 1 FEMA-356 response envelope types [5].

Acceptance Criteria

FEMA-356 defines three limit states: Immediate Occupancy, Life Safety and Collapse. Capacity associated with each limit state is defined by specifying limiting values on the structural element load-response envelope. Therefore, a demand-capacity comparison to determine acceptance is conducted at the level of structural elements. Format of the acceptance criteria for a limit state depends on the type of action on the structural element under consideration and the type of demand analysis used to compute demand. In the following, value of the knowledge factor κ is set to 1.

Non-linear Demand Procedures

Acceptance of displacement-controlled actions is evaluated by comparing computed deformation demand to deformation capacity limits specified for each limit state. Force-controlled actions are acceptable if the lower-bound capacity of a structural element is larger than maximum demand computed using a combination of gravity and unreduced earthquake loading.

Linear Demand Procedures

Force-controlled actions are acceptable if the lower-bound capacity of a structural element Q_{CL} is larger than the force-controlled design action due to a combination of gravity and earthquake loads Q_{UF}. Deformation-controlled actions are acceptable if the expected capacity of a structural element Q_{CE} multiplied by an element demand modifier factor m, which accounts for expected ductility of the structural element, is larger than the deformation-controlled design action due to a combination of gravity and earthquake loads Q_{UD}, or:

$$mQ_{CE} \geq Q_{UD} = Q_G \pm Q_E \text{ i.e. } Q_{CE} \geq \frac{Q_G \pm Q_E}{m} \qquad [16]$$

A parallel between FEMA-356 and DOE-STD-1020 can be draw at this point. Acceptance criteria in DOE-STD-1020 are based on a demand-capacity comparison made assuming the structure is slightly non-linear. This state corresponds to FEMA-356 acceptance criteria for deformation-controlled actions determined using linear analysis. Capacities C_C in DOE-STD-1020 are computed using ultimate limit state design provisions. As such, they correspond to the expected capacity values Q_{CE} in FEMA-356. Total inelastic demand in FEMA-356 is computed as the combination of non-seismic loads coincident with design-basis seismic and seismic loads reduces using the inelastic energy absorption factor F_μ and a performance category scale factor SF (equal to 0.9 for PC-3 and 1.2 for PC-4). Acceptance criteria, formulated at the structural element level, are:

$$C_C \geq D_{NS} + \frac{D_S}{\frac{F_\mu}{SF}} \qquad [17]$$

Therefore, in case when non-seismic loads are small compared to seismic loads and when the elastic seismic demand is computed at the probability of hazard P_H, the energy absorption factor F_μ is analogous to the FEMA-356 element demand modifier factor m.

ACCEPTANCE CRITERIA AND RISK REDUCTION RATIO

FEMA-350 equation 11 is a ratio of median demand, computed for a hazard with a probability P_H, and median capacity. The demand factor γ accounts for uncertainty in determining demand using different demand analysis methods and earthquake record-to-record randomness in response. The capacity factor ϕ in equation 11 adjusts for uncertainty in determining the capacity using a method for capacity analysis and material and geometry randomness. While FEMA-356 demand and capacity are not computed at the median level, they can still be used in equation 11. Thus, FEMA-356 acceptance criteria can be re-cast in the form of a ratio:

$$\lambda = \frac{D}{C} \qquad [18]$$

and used to compute the risk reduction ratio using equation 14 as follows:

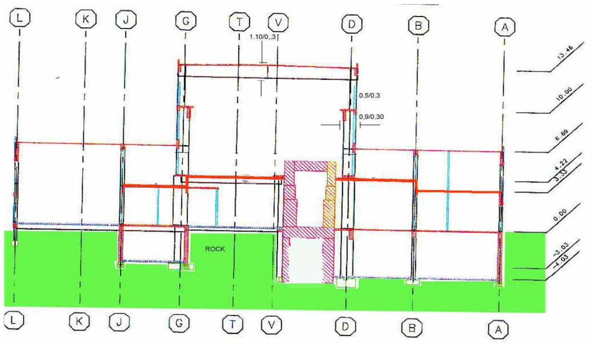
$$R_R = \frac{1}{\lambda^{k/b}} \exp\left(\frac{1}{2}\frac{k^2}{b^2}\beta_{UT}^2\right) \qquad [19]$$

The slope of the seismic hazard curve slope k, the equal-displacement rule factor b, and the total uncertainty β_{UT} have the same meaning as discussed above. In the end, using this approach, it is possible to compute the risk reduction ratio implied by FEMA-356, and provide a link to DOE-STD-1020. While successful pilot applications of conventional FEMA-273 and FEMA-356 provisions for evaluation of nuclear facility structures have been made, examples are presented in [9] and [11], the link between acceptance criteria in FEMA-356 and DOE-STD-1020 is crucial. Achieving a level of risk reduction with respect to the probability of hazard is at the core of DOE-STD-1020. Providing a method to compute a level or risk reduction implied by FEMA-356 structural evaluation provisions opens the door to proper calibration and rational use of these provisions for nuclear facilities.

TEST-BED STRUCTURE

The test-bed building, shown in Figure 1 is a 1960's vintage reinforced concrete moment-frame structure with masonry infills. This structure has a number of irregularities compared to regular conventional structures considered in FEMA-356. Some of them are: 1) the stiff and strong interior cell is separated from the frame by 2 cm gaps; 2) transverse frames are vertically irregular; 3) horizontal diaphragms at 3.33m and 4.22m levels are not continuous; and 4) distribution of diaphragm masses and location of the cell results in torsional irregularity. Detailing of the structural elements, typical of the 1960's practice, is another challenge. It is well known that such detailing, characterized by markedly less transverse reinforcement than required today, may result in the response of structural elements that is not as ductile as expected from new structures designed today.

The seismic environment of the test-bed building site is typical for low-to-moderate seismically active regions in Europe. An acceleration spectrum used in this evaluation is shown in Figure 2. This spectrum is site-specific, but is not a uniform-hazard spectrum. Therefore, the level of seismic hazard associated with this spectrum is not specified. It is interesting to note that ground motions that the 1994 Northridge ground motion recorded at Arleta fits this spectrum fairly well when scaled to match it at the fundamental period of the structure.



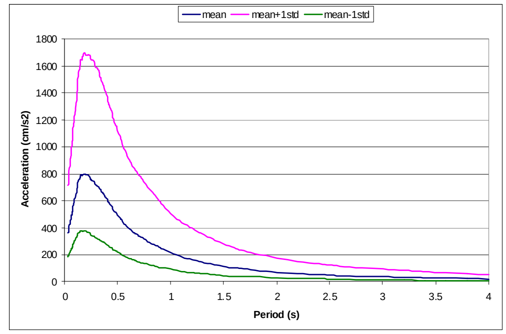
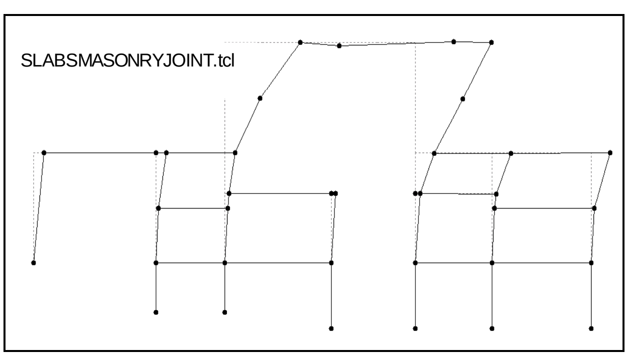


Figure 2 Typical elevation of the test-bed building and the site-specific acceleration spectra used in the evaluation (mean and plus-minus one standard deviation).

MODELING AND EVALUATION

The test-bed structure was modeled using OpenSees, a software framework under development by PEER (http://opensees.berkeley.edu). OpenSees is intended for performance-based seismic evaluation and supports linear and nonlinear, static and dynamic analysis procedures. Beam and column elements of the structure were modeled using a fiber cross-section flexibility-based beam-column element. The fiber cross sections allowed for accurate modeling of the flexural strength. The shear strength was modeled using FEMA-356 equation that accounts for shear strength degradation at elevated displacement ductility levels. Horizontal restraint provided by the floor and roof slabs was modeled using displacement slaving. Vertical strength and stiffness of the masonry walls was modeled using linear spring elements attached to the appropriate beam nodes. Finally, the knee joint connections at the high-bay roof were modeled using non-linear rotational springs. The force-deformation envelope for the knee joint was a FEMA-356 Type 2 envelope, with the strength computed counting only the concrete contribution taken at $0.33\sqrt{f_c^{'}}$ MPa, according to FEMA-356. A number of two- and three-dimensional models of the structure were made to investigate the effect of the stated irregularities. In particular, 2, the transverse frame model was used to evaluate the effect of the cell on the response, while a three-dimensional model was used to evaluate the effect of longitudinal motion and torsion. The fundamental modes of the models had periods of approximately 1 second (Figure 3).



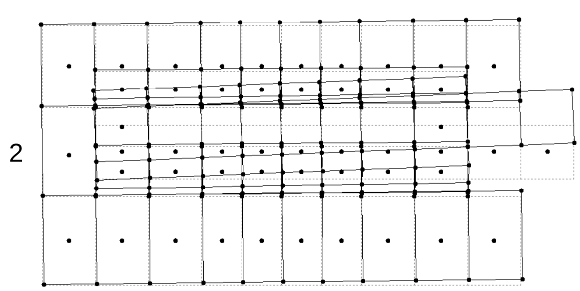
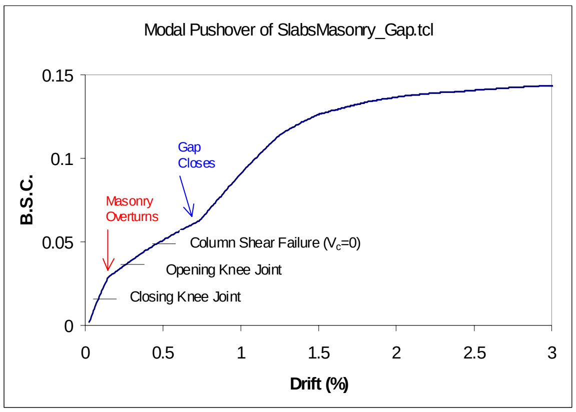


Figure 3 Fundamental modes of the two- and three-dimenasional models of the test-bed structure.

STATIC ANALYSIS

Non-linear static pushover analyses were conducted using the two-dimensional model. Different lateral force distribution patterns as well as different models of the gap between the cell and the frame, the masonry infill, and the knee joint were investigated. Response of progressively more complex models was converging. More important, when capacities of the critical components of the transverse frame (shear in knee joint and shear in the high-bay column) are plotted on a pushover curve, it was revealed that the frame has a relatively small capacity compared to the approximately 0.2 base shear coefficient demand computed according to FEMA-356 (Figure 4). A similar result is obtained by Incremental Dynamic Analysis (IDA) procedure [12]. In this procedure, non-linear time-history dynamic analyses are conducted repeatedly while steadily increasing the magnitude of the chosen ground motion. The resulting force-deformation response represents the peak points achieved during each analysis combined into one curve. In the IDA example shown in Figure 3, the knee joint clearly limits the capacity of the frame.



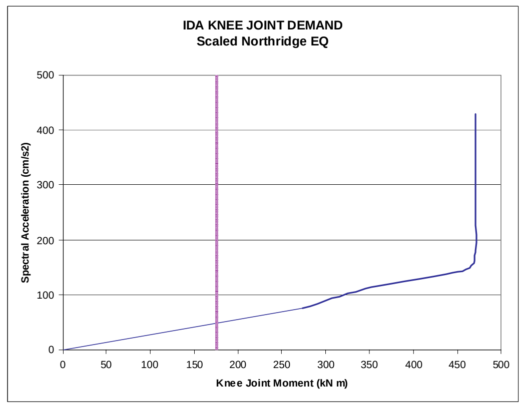


Figure 4 Static and dynamic (IDA) pushover responses.

DYNAMIC ANALYSIS

Non-linear time-history dynamic analyses were conducted using both two- and three-dimensional models and several US and European ground motions. A typical cyclic response of the knee joint (Figure 4) shows that significant deterioration of its strength may be expected: this confirms the findings of the static analyses. However, a three-dimensional analysis revealed additional properties of the test-bed structure. The response of three transverse frames located at each end (frames 4 and 18) middle of the cell (frame 11) shows the effect of torsion of the structure as well as the effect of gap closure and pounding between the frames and the structure (Figure 5). Even though the response of the outer frames is amplified by approximately 20%, the amplification is limited by the size of the gap between the frames and the cell. Nevertheless, the high-frequency response induced by the pounding is not desirable. Furthermore, the response of the structure in the longitudinal direction reveals the weakness of frame columns in this direction due to their elongated rectangular cross-section. The three-dimensional analysis demonstrates a marked potential for soft-story formation between the ground level and the horizontal slab system at elevations 3.33m and 4.22m.

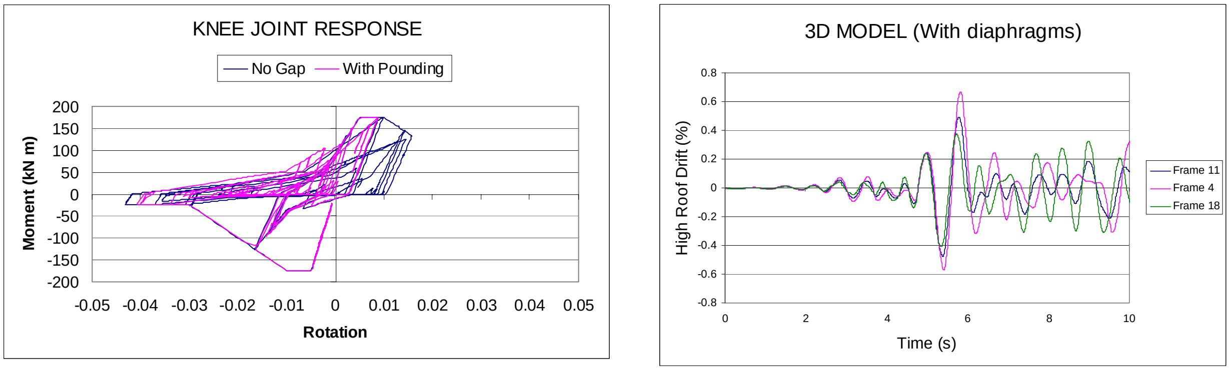


Figure 5. Sample results of dynamic analyses.

CONCLUSION

The test-bed structure example demonstrates that FEMA-356 procedures for evaluation of conventional buildings to evaluate a nuclear facility structure can be done. However, to form a rational basis for use of FEMA-356 to evaluate a nuclear facility, a level of risk reduction afforded by FEMA-356 provisions with respect to the probability of hazard must be established. This is because the DOE-STD-1020 nuclear facility design provisions are developed such that a prescribed level of risk reduction is achieved for the performance category of interest. Therefore, rational use of FEMA-356 provisions for evaluation of existing nuclear facilities must provide the risk reduction factor information.

A method for establishing the risk reduction level implied by FEMA-356 provisions is presented in this paper. This method was developed by considering a general formulation of probabilistic performance-based acceptance criteria based on the equation to compute the total probability of failure. Starting with such probability-based formulation two acceptance criteria used in US seismic evaluation practice were derived: 1) a confidence-ratio based acceptance criteria used in FEMA-350 for conventional structures; and 2) a risk reduction ratio based acceptance criteria used in DOE-STD-1020 for nuclear facility structures. Then, an analytical relation between these two acceptance criteria was derived and tables for conversion from one to the other criteria were presented. Finally, a comparison between DOE-STD-1020 and FEMA-356 acceptance criteria is used to propose a way to compute the risk reduction ration afforded by FEMA-356 provisions.

However, much work remains to be done. First, a statistical rendering of FEMA-356 demand and capacity analysis methods must be done to establish the relation between the pre-standard values and the actual median values. Such statistical analysis will enable rational calibration of the FEMA-356 provisions to a desired level of risk reduction. Second, the test-bed structure demonstrates the difficulties in applying FEMA-356 provisions to structures that are irregular and have structural components with limited ductility. Improvements in FEMA-356 analysis and modeling procedures to enable demand analysis for irregular structures, as well as ways to evaluate capacity of components with limited or no displacement ductility, should be developed and implemented. Nevertheless, these obstacles are not insurmountable: evaluation of existing nuclear facilities using performance-based criteria, similar to those used in FEMA-356, will enable more rational evaluation and substantial savings in upgrading costs while still maintaining a satisfactory level of safety of nuclear facility structures.

ACKNOWLEDGEMENTS

The results presented in this paper are a part of the research project "Evaluation of US Performance-Based Design Procedures for Nuclear Facilities" funded by the Institute for Radioprotection and Nuclear Safety (IRSN), Paris, France under contract number 30000364-30000365 with the University of California, Berkeley, Department of Civil and Environmental Engineering.

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