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PBEE EVALUATION OF E-DEFENSE COLLAPSE EXPERIMENT

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ABSTRACT :

A full-scale four-story steel building tested on the E-Defense shake table was used as a case study to evaluate how well performance-based earthquake engineering (PBEE) guidelines characterize collapse. The study had mixed results with the guidelines mostly erring on the safe-side by prediction collapse at shaking intensities less than that in the experiment. Recommendations are made regarding guideline use and development.

KEYWORDS: Collapse, steel moment frame, performance based design, shake table test

1. INTRODUCTION

Performance-based earthquake engineering (PBEE) is evolving as the preferred way for design of the built environment, especially for rehabilitation of existing buildings. PBEE Guidelines such as the ASCE standard: *Seismic Rehabilitation of Existing Buildings* (*ASCE-41* 2006) and the FEMA recommended criteria: *Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings* (*FEMA-351* 2000) are now available, and have been accepted by some US building officials. This paper evaluates such guidelines by application to a building that was shaken to collapse with the objective to assess how well PBEE predicts an actual collapse.

2. CASE STUDY BUILDING

The full-scale four-story steel moment-frame building (Figures 1 and 2) was tested in September 2007 on the E-Defense shake table located in Miki, Japan. It reflects typical Japanese design and construction practice. Detailed descriptions of the building can be found elsewhere (Yamada *et al.* 2008).

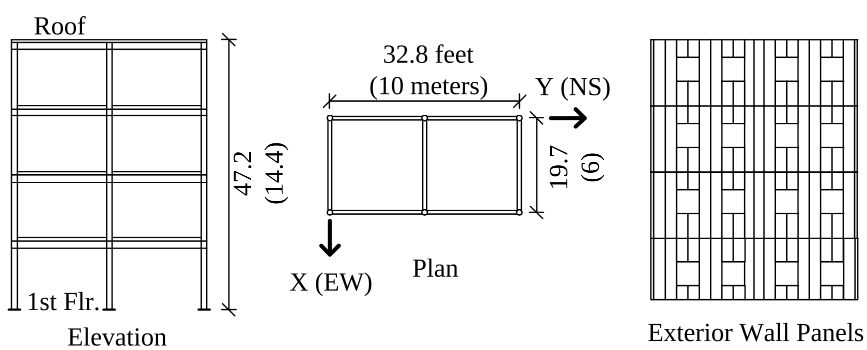

Figure 1 Case study building.

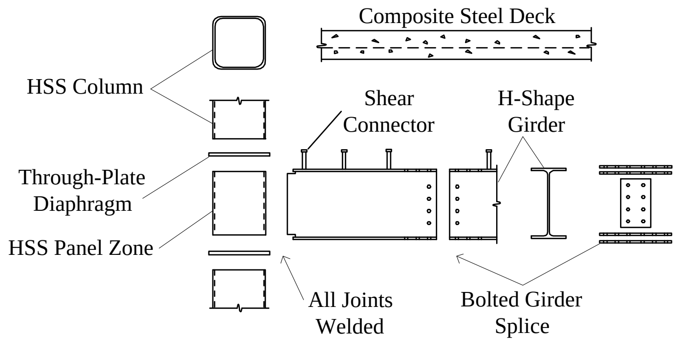

Figure 2 Girder-to-column framing (exploded view).

3. E-DEFENSE EXPERIMENT

The building was subjected to a series of shakes (tests) under input records with increasing amplitude scale factors: *SF* = 0.05, 0.2, 0.4, 0.6, and 1.0. The input motions were the JR Takatori train station accelerations as recorded in Kobe, Japan during the 1995 Hyogo-Ken Nambu (M_w 6.9) earthquake. As shown in Figure 3, for periods greater than 1 second, the spectra exceeds a Maximum Considered Earthquake (MCE) that is the usual basis of collapse safety checks in the western United States. See Suita *et al.* (2008) for detailed description of the experiment.

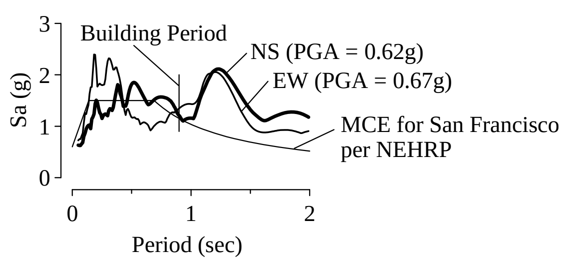

Figure 3 Takatori earthquake spectra at 5% damping.

3.1. Experiment Results

Figure 4 shows time histories of the first story displacements expressed as story drift ratios from the tests with *SF* = 0.2, 0.4, 0.6, and 1.0, respectively. For tests with scale factors, *SF* $\leq$ 0.6, the building responded in a quasi-linear-elastic manner with similar displacement patterns and amplitudes roughly proportional to scale factor. At *SF* = 1.0, the displacement pattern is markedly different due to inelastic behaviors, and collapse occurred with a side-sway mechanism in the first story having the primary direction of collapse in the Y-Direction (Figures 5 and 6). Using refined computer analyses, the authors have estimated *SF* = 0.8 as the minimum intensity that would cause collapse under a *single* application of the scaled Takatori record. This is used as a benchmark by which to compare to estimates made using PBEE guidelines.

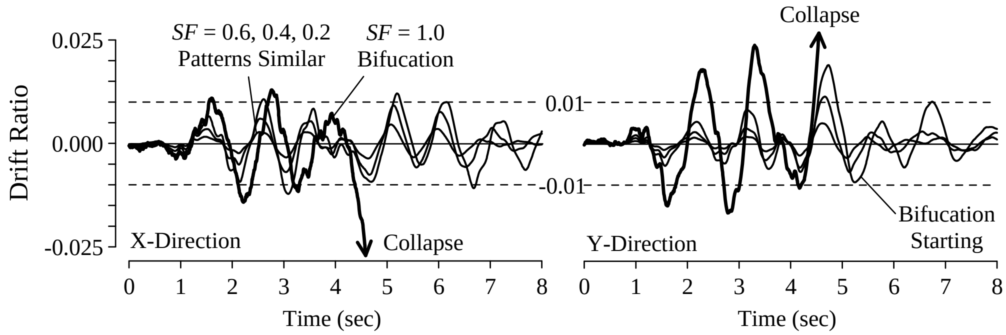

Figure 4 First floor time history responses (four test results are superimposed for comparison).

Figure 5 Building in post-collapse configuration (from View A in Figure 6).

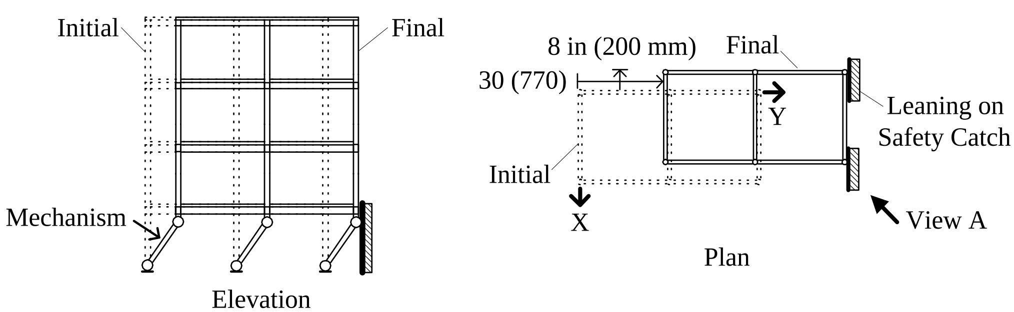

Figure 6 Building collapse mechanism (no scale).

4. PBEE EVALUATION

Both *ASCE-41* and *FEMA-351* were used, and they have very different evaluation approaches. *ASCE-41* is a deterministic approach that evaluates individual member demand-to-capacity ratios. For linear analysis methods, the checks are made using ductility *m*-factors, which are intended to relate member elastic forces to inelastic deformations. For non-linear methods, *ASCE-41* provides criteria for member and connection inelastic deformations. *FEMA-351* defines probabilistic confidence (probability) estimates of demands and capacities for particular building failure mechanisms. For both methods, *FEMA-351* evaluates four failure modes: side-sway collapse (global drift check), floor vertical collapse (local drift), column buckling (axial force), and column splice fracture (axial force). The building was evaluated for the Collapse Prevention (CP) performance level using both linear and non-linear analysis methods. The smallest scale factors (*SF*) causing the building to fail the CP criteria were taken as the "collapse" intensities for comparison with the experiment.

4.1. Computer Analysis Models

Both linear and non-linear computer models were formulated based on the criteria presented in the guidelines. Separate planar models representing the moment frames in each building principal direction were used (Figure 7). Results from analyses in the two directions were combined using the "30 percent" combination rule per *ASCE-41*. The non-linear models had member inelastic actions based on *ASCE-41* parameters (Figure 8): plastic rotation defining the onset of moment strength degradation (*a*-value); plastic rotation defining the onset of complete strength loss (*b*-value); and residual strength ratio (*c*-value). Three different evaluation procedures were used: linear dynamic (response spectrum), non-linear dynamic (time history) and non-linear static (push-over).

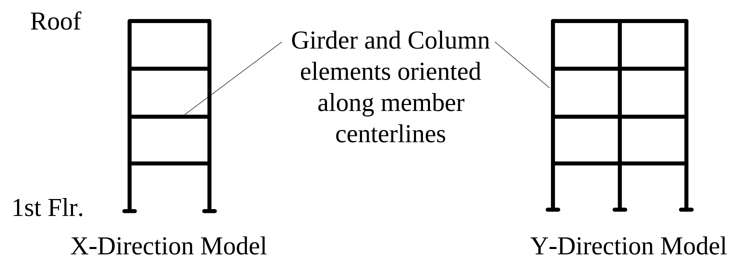

Figure 7 Building computer models.

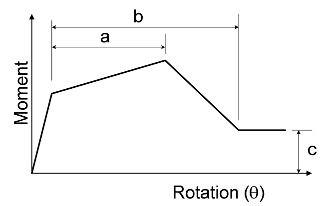

Figure 8 *ASCE-41* component modeling parameters (*a*, *b*, and *c*-values).

4.1. Linear Dynamic Procedure (LDP) Evaluation

How well the PBEE methods predict the earthquake intensity causing collapse was quantified by computing "safety" margins defined as: $M = SF_{test} / SF$, where $SF_{test} = 0.8$ = scale factor causing collapse inferred from experiment and *SF* = scale factor associated with CP criteria from the guidelines. When $M > 1$, the guidelines are *conservative*, that is, they predicted collapse at earthquake intensities smaller than actual value inferred from experiment.

Figure 9 summarizes the evaluation using the LDP. Superimposed on the plots are the drifts from the experiment, refined analysis, LDP analysis, and the points where the PBEE CP limit states are reached. The *ASCE-41* check indicated that the columns in the first story fail the CP acceptance criteria at $SF = 0.4$ translating to a conservative $M = 2$. *ASCE-41* specifies *m*-factors of 1.5 for the first story columns based on the section slenderness and axial load ratio. The columns reached their limiting values mainly due to moments, indicating failure due to excessive flexure as observed in the experiment. The connections, girders, and panel zones all failed their acceptance criteria at larger scale factors meaning they did not govern. For *FEMA-351*, the *local* drift check controlled and produced an unconservative $M = 0.7$. However, this is for the case where the moment connections loose their ability to resist gravity loads leading to local vertical collapse of the floor system. This failure mode did not occur in the test. The *global* drift check corresponds to a lateral sway-type collapse like that in the test, and it also has an unconservative $M = 0.6$. Column compressive buckling checks fail at larger scale factors indicating this failure mode does not govern.

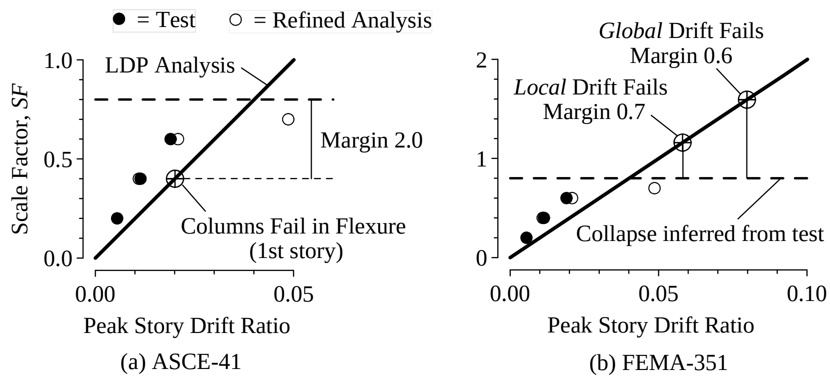

Figure 9 Linear dynamic procedure (LDP) evaluation results.

4.2. Non-Linear Dynamic Procedure (NDP) Evaluation

Figure 10 summarizes the NDP evaluation as depicted in an incremental dynamic analysis (IDA) format. The IDA curve plateaus (corresponding to building collapse) when $SF > 0.5$ (Analysis X), which is below the estimated intensity causing collapse as well as peak drifts from the test and refined analysis. For *ASCE-41*, the columns in the first story were correctly identified as being the weak link in the building. They failed the CP acceptance criterion at a $SF = 0.4$ resulting in a conservative $M = 2$. Some connections, girders, and panel zones experienced yielding but their plastic rotations were within their acceptance criteria at this scale factor thus indicating they did not govern. For *FEMA-351*, the acceptance criterion for the *local* drift check is less than that for the *global* check, however both of these collapse mechanisms have the same conservative $M = 1.6$ because of the CP drifts lie on the IDA plateau. Thus, the *FEMA-351* limits are essentially governed by the member strength and rotation capacity (*a*, *b*, *c*-values) assumptions in the non-linear analysis model. The column compressive forces are within the acceptance criteria indicating that column buckling did not govern. Both *ASCE-41* and *FEMA-351* conservative margins are a consequence of column modeling parameters indicated in the guidelines. Per *ASCE-41*, the column plastic rotation at the onset of strength degradation is very small (*a*-value ~0.002 rad), and once yielding occurs in the first story, a collapse mechanism forms almost immediately as the moment capacity drops off quite rapidly (*b*-value ~0.003 rad). This is why *ASCE-41* LDP and NDP agreed with both having $M = 2$. Non-linear static (NSP) pushover analysis was also performed and the results were essentially the same as that from NDP due to the limited rotation capacity of the columns.

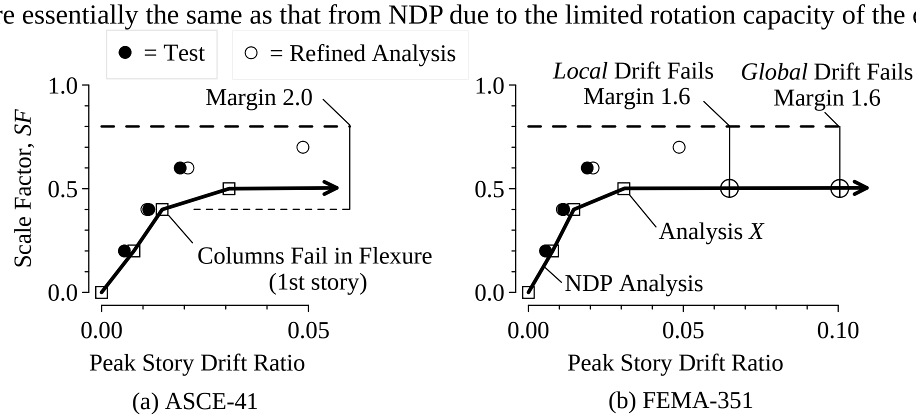

Figure 10 Non-linear dynamic (NDP) evaluation results.

5. FINDINGS

A summary of key points follow with many of these expected to be relevant for other buildings having similar conditions.

Shake Table Test Results. The building achieved its design objective by withstanding shaking intensities much greater than that required by the Japanese building code (code design basis corresponds to *SF* ~ 0.4). Collapse occurred due to a side-sway mechanism in the first story with hinging of columns at the top and bottom. Yielding occurred in other members but these did not govern the collapse. The story mechanism occurred in spite of a strong-column and weak-beam design because of factors not accounted for in customary calculations. The main factor probably was large concurrent bending moments in both column principal directions (bi-axial bending). Up to a certain shaking intensity ($SF \leq 0.6$), the behavior was fairly linear. At an intensity SF = 1.0, the dynamic response changed markedly due to yielding and the building then entered into a collapse sequence of lengthened period oscillatory motions having progressively increasing amplitudes up to collapse.

Performance Assessment. State-of-practice non-linear analysis was able to closely simulate the experiment, *e.g.*, in terms of peak displacements and instant in time of collapse (*i.e.*, Blind Analysis Contest modeling, see Ohsaki *et al.* 2008). However, this does not mean it will consistently lead to accurate results. Successful analysis used specific data about component hysteretic behavior from tests that are not often available when creating computer models in practice. The *ASCE-41* modeling parameters, which would otherwise be used in engineering practice, were much smaller than those from component tests.

The analyses suggest that yielding in tests prior to the final test causing collapse had negligible effect on the collapse behavior (pre-existing damage did not appreciably weaken the building). The deformation at which strength degradation occurs in the columns was an important factor in the building collapse behavior (*a*-values). This and member strength were the two most important parameters governing the building collapse ruggedness.

The two PBEE guidelines and evaluation approaches (*ASCE-41* and *FEMA-351*) had mixed results regarding characterization of collapse and neither approach was clearly superior. Collapse evaluation using the guidelines was mostly conservative — erring on the safe-side. Hence, for other buildings having similar conditions that pass the criteria, it is very likely they are collapse-safe. On the other hand, if they exceed the criteria by only modest amounts, it is likely they still have some margin of safety against collapse. In terms of safety margins, *FEMA-351* predicted collapse more accurately than *ASCE-41*. However, *ASCE-41* correctly identified the columns in the first story as being the weak link in the building, whereas *FEMA-351* incorrectly identified floor local collapse as controlling.

The *ASCE-41* linear and non-linear procedures had the same margins because the collapse prevention acceptance criteria was exceeded when the building was essentially linear-elastic. This had a relatively large margin of about two, highlighting the fact that the linear procedure *m*-factors, and the non-linear deformation parameters (*a*-values and plastic deformation acceptance criteria) were too small.

The *FEMA-351* linear procedure produced unconservative margins in part because linear analysis could not capture the inelastic concentration of drift in the first story occurring incipient to collapse. Hence, use of linear procedures for collapse prevention evaluation is not appropriate for this type of structure. The *FEMA-351* non-linear procedure confidence level (probability) estimations became meaningless once the analysis approached a collapse mechanism because the drifts became very sensitive to shaking intensity (Figure 10b). The "allowable" drifts for various confidence levels (20%, 50%, etc.) all had the same margin. This is a weakness of using story drift ratio as the primary collapse performance measure.

The guidelines, especially *ASCE-41*, had many intricate provisions requiring detailed calculations implying that the results were quite accurate. However, the case study revealed that the predicted intensity causing collapse can be as much as a factor of about two from the estimated actual value. *ASCE-41* is well suited as a design tool for building rehabilitation, since while not especially accurate in predicting the intensity causing collapse; it did identify the weak link in the building thereby targeting the right members for upgrading. Also, the process of member-by-member checking is pragmatic within a design context. On the other hand, while *FEMA-351* is appealing as a building performance predictor, since it provides probability estimates for specific failure modes, it does not have specific member acceptance criteria.

6. RECOMMENDATIONS

The following needs became apparent during the course of the case study exercise. These are offered here for consideration by practicing engineers and researchers.

1. Many of the *ASCE-41* component modeling parameters (*a, b, c*-values) and acceptance criteria (*m*-factors and plastic deformations) are too small, and hence require improved calibration to better simulate actual behavior. Recent updates to *ASCE-41* recognize this for concrete structures (Elwood, *et al*. 2007), and similar updates ought to be created for steel moment-frames.
2. *ASCE-41* should adopt a quality ranking system to indicate how well component modeling parameters and acceptance criteria are defined. It would be ideal if the existing tables containing the numerical values had a column with *High* and *Low* designations for each component: High meaning values are supported by consensus research and tests, and Low meaning values are based mostly on judgment and hence may be conservative. Engineers could then make more informed decisions when applying the guidelines.
3. The check for vertical collapse of floor systems (local drift check) in *FEMA-351* needs to be improved, since the current criterion (based on peak drift), does not distinguish between cases where drifts are excessive due to a column hinging story mechanism versus excessive deformations in the girder-to-column connections leading to vertical failure of floor system. The local drift check controlled in the case study evaluation.
4. Use of *linear* procedures for *FEMA-351* collapse prevention performance evaluation is discouraged. Inelastic deformations dominate behavior incipient to collapse and they coalesce in certain stories to form the failure mechanism. It is unlikely that any set of linear analysis modification factors can reliably capture these effects.
5. *ASCE-41* and *FEMA-351* have a fundamental weakness when evaluating for collapse prevention. Both use peak deformation as basis of judgment: component ductility in *ASCE-41* and story drift in *FEMA-351*. The case study revealed that when the shaking intensity is close to that causing collapse, the key peak deformations were very sensitive to small changes in intensity. Reliably estimating peak deformations in this intensity range is problematic, and their use as measures of performance is questionable. A performance measure based on ground shaking *intensity*, is preferred (see next point).
6. For collapse prevention evaluation, future generations of PBEE method ought to consider use of *safety margin* informed by incremental dynamic analysis (IDA) as the measure (Figure 11). The acceptance criterion is that the safety margin must be greater than unity ($SI_c / SI_d > 1$). Estimation of intensity causing collapse has less variability than the peak deformations incipient to collapse, and thus is a better measure.

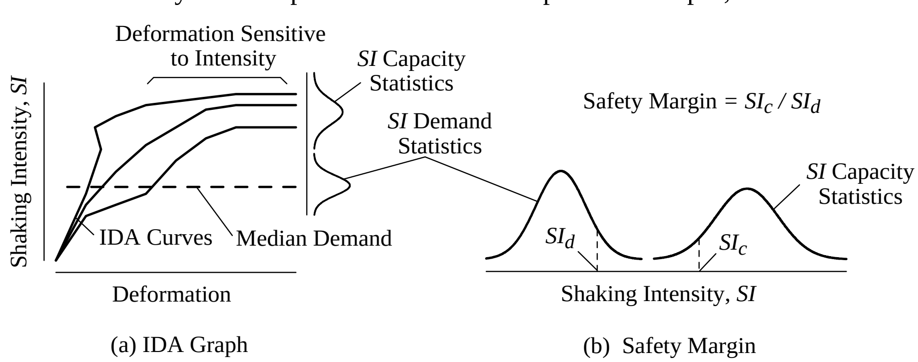

Figure 11 A collapse evaluation approach based on shaking intensity.

4. CONCLUSION

Accurate characterization of collapse within the context of PBEE is an important, but highly elusive goal. This case study provides a sobering illustration on some of the challenges. A relatively simple lab specimen building was evaluated by different PBEE approaches, yet the results had considerable variation. Despite this shortcoming, the bottom-line conclusions using PBEE evaluations were generally conservative so "appropriate" outcomes in terms of providing for public safety was mostly achieved. PBEE is an evolving science, and engineers should recognize that current guidelines are probably more *conservative* than *accurate* and they must not be treated as building code dogma. Case studies like that herein are essential to anchor the application of PBEE against reality, even if they can only provide fleeting glimpses and not definitive judgments.

5. ACKNOWLEDGEMENTS

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