

EARTHQUAKE ENGINEERING PRACTICE

ASCE-41 and FEMA-351 Evaluation of E-Defense Collapse Test

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A welded steel moment-frame building is used to assess performance-based engineering guidelines. The full-scale four-story building was shaken to collapse on the E-Defense shake table in Japan. The collapse mode was a side-sway mechanism in the first story, which occurred in spite of a strong-column and weak-beam design. Computer analyses were conducted to simulate the building response during the experiment. The building was then evaluated using the *Seismic Rehabilitation of Existing Buildings (ASCE-41)* and *Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings (FEMA-351)* for the collapse prevention performance level via linear and nonlinear procedures. The guidelines had mixed results regarding the characterization of collapse, and no single approach was superior. They mostly erred on the safe side by predicting collapse at shaking intensities less than that in the experiment. Recommendations are made for guideline improvements. [DOI: 10.1193/1.3224159]

INTRODUCTION

Performance-based engineering (PBE) is evolving as the preferred approach to earthquake design, especially for building rehabilitation. By assessing specific performance objectives, PBE provides an important link between design, risk management, regulation, and policy. Implicit is the notion that probable seismic response can be predicted.

Guidelines such as the ASCE standard *Seismic Rehabilitation of Existing Buildings (ASCE-41 2006)* and the FEMA report *Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings (FEMA-351 2000)* are now available and accepted by some jurisdictions. These represent the state-of-the-art in PBE, but incorporate expert opinion to bridge gaps where definitive research is lacking. Benchmarking PBE against actual building response is essential to assess its efficacy and to help guide its future development.

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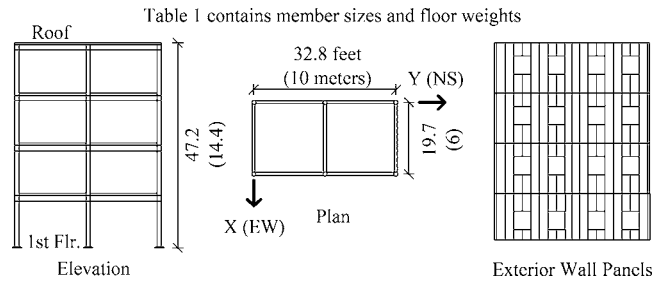


Figure 1. Elements of case study building: Steel moment-frames on each column row and autoclaved lightweight concrete panels for nonstructural exterior wall cladding system.

The welded steel moment-frame (WSMF) is a relatively young structural system, and much is yet to be revealed about its behavior during actual earthquakes—recall the surprise connection fractures caused by the 1994 Northridge earthquake. PBE guidelines cover WSMFs, but their effectiveness for collapse prevention evaluation cannot be judged via post-quake damage surveys because collapse of modern steel buildings has yet to be documented. That leaves comparison to shake table tests as one of the best ways currently available to assess PBE for this class of buildings.

This paper presents a case study of a full-scale, four-story WSMF office building shaken to collapse as part of an E-Defense steel building research project (Kasai et al. 2007, Yamada et al. 2008, Suita et al. 2008, Matsuoka et al. 2008). An objective is to assess how well current United States PBE guidelines characterize an actual collapse.

CASE STUDY BUILDING

The building (Figure 1) was tested in September 2007 on the E-Defense shake table located in Miki, Japan. Story heights and bay widths were typical full-scale dimensions, but the numbers of bays were fewer than usual in order to optimize the specimen in relation to the shake table capabilities. Typical Japanese design and construction practice was followed, namely, the use of tubular columns and welded steel moment-frames on each column row (Figure 2). Enhanced welded connection details were used to reduce the possibility of brittle fracture (Figure 3). The exterior-wall cladding consisted of autoclaved lightweight concrete (ALC) panels designed to be nonstructural relative to the moment-frames (Figure 4). Table 1 contains pertinent information, and additional descriptions can be found elsewhere (Yamada et al. 2008).

E-DEFENSE EXPERIMENT

SHAKE TABLE EXCITATIONS

The experiment had repeated applications (tests) of the same wave forms with increasing amplitude scale factors: $SF=0.05, 0.2, 0.4, 0.6,$ and 1.0 . The motions were those recorded at the JR Takatori train station in Kobe, Japan during the 1995 Hyogo-Ken Nambu earthquake (M_w 6.9). This is a severe excitation recorded in close proximity to the faulting.

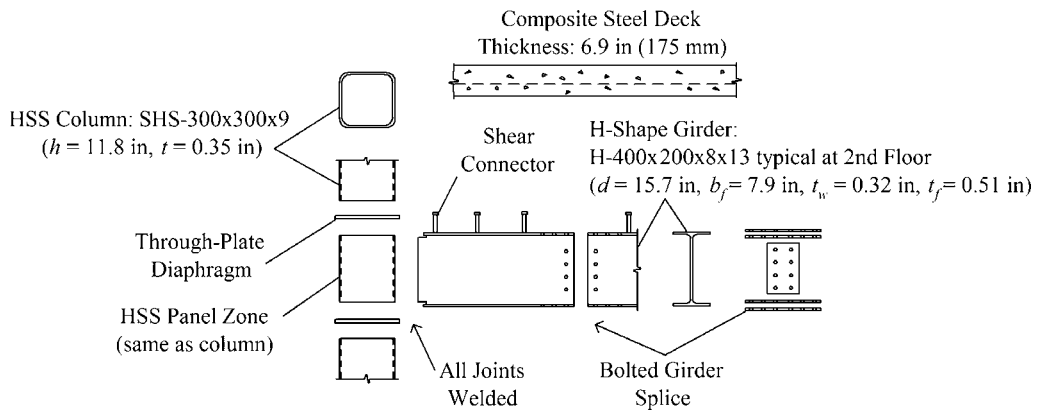


Figure 2. Exploded view of typical girder-to-column framing with shop-welded components that were erected via site-bolting (Table 1 contains additional member descriptions).

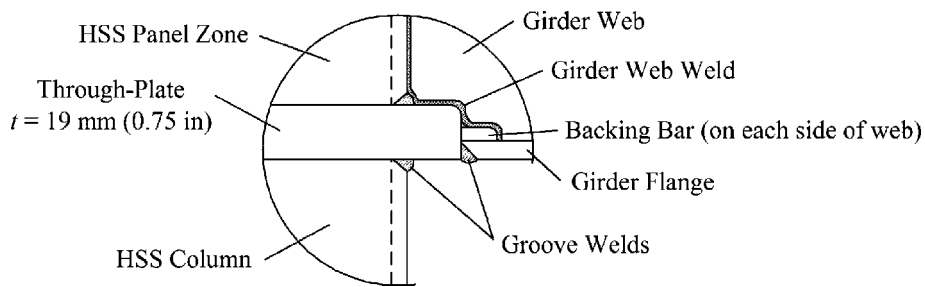


Figure 3. Typical joint detail at bottom flange with enhanced details reflecting post-Kobe earthquake experience, notably having no through-web access hole.

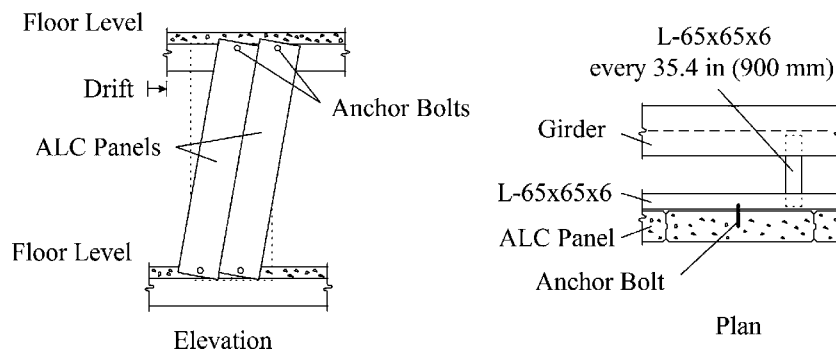


Figure 4. Nonstructural exterior wall autoclaved lightweight concrete (ALC) panel attachment to moment-frames (for angle $L-65 \times 65 \times 6$: leg length=2.6 in and thickness=0.24 in).

Table 1. Building properties

Attribute	Comment
Design Basis	<i>Japan Standard Building Law</i> . Designed for two earthquake levels. Level 1 had a base shear coefficient=0.2 and required a peak story drift ratio less than 0.005. Level 2 required demonstration of lateral stability with a base shear coefficient=0.3.
SCWB	Strong-column and weak-beam (SCWB) design at floors 2 through 4. The SCWB ratio was >1 for each floor and in each building principal direction according to: $\frac{\sum M_{pc}}{\sum \min(1.5M_{pb}, 1.3M_{pp})}$, where M_{pc} , M_{pb} and M_{pp} are the plastic moment strengths of the column, beam and panel zone, respectively. Panel zones governed over beams in the calculation (weaker versus beams).
Girders	Hot-rolled wide-flange girders varying from H-346 \times 174 \times 6 \times 9 (depth $d=13.6$ in, width $b_f=6.9$ in, web $t_w=0.24$ in, flange $t_f=0.35$ in) at roof level to H-400 \times 200 \times 8 \times 13 ($d=15.7$ in, $b_f=7.9$ in, $t_w=0.32$ in, $t_f=0.51$ in) at second floor. SN400B steel. Nominal and expected (as tested) $F_y=34$ ksi and $F_{ye}=45$ ksi (235 MPa and 313 MPa), respectively.
Girder ratios	$\frac{b_f}{2t_f} \approx 6 \rightarrow 10$, compact per Japan code and <i>AISC</i> (2005): $0.38 \sqrt{\frac{E}{F_y}} = 11$
Columns	Cold-formed steel hollow structural section (HSS), SHS-300 \times 300 \times 9 ($h=11.8$ in, $t=0.35$ in). BCR295 steel. Nominal and expected (as-tested 0.2% offset rule) $F_y=43$ ksi and $F_{ye}=48$ ksi (295 MPa and 331 MPa), respectively.
Column ratios	$\frac{b}{t} = \frac{h-3t}{t} = \frac{300-27}{9} = 30.3 \approx 30$, borderline compact by Japan code and <i>AISC</i> : $1.12 \sqrt{\frac{E}{F_y}} = 29$. Noncompact by <i>ASCE-41</i> and <i>AISC Seismic</i> (2005): $\frac{190}{\sqrt{F_{ye}}} = 27$ and $0.64 \sqrt{\frac{E}{F_y}} = 17$, respectively
Column bases	Columns welded to baseplates bolted to fixtures attached to shake table. Connection strength was greater than plastic moment of column. Baseplate provided moderate base fixity having rotational stiffness $\sim 6EI/L$ of first-story column.
Roof and floors	Composite steel deck with normal weight concrete. Thickness=6.9 inch (175 mm).
Exterior walls	Autoclaved lightweight concrete (ALC) panels. Typical panel dimensions: 11.5 feet (3,500 mm) tall by 2 feet (600 mm) wide by 5 inches (125 mm) thick.
Interior walls	Partitions located on the second to fourth floors consisted of drywall gypsum boards attached to metal-stud framing. Ceiling consisted of gypsum board attached to metal framing. Both seismic and non-seismic connection details were used to compare performance. Seismic details had clearances to mitigate the effect of structure drifts on partitions.
Weights	Actual weights having equivalent lumped floor weights of 186 psf (8.92 kN/m ²) at roof and 140 psf (6.7 kN/m ²) at floors. Included were collapse safety-catch steel frame systems located on each floor.

As shown in Figure 5, the spectra mostly envelopes a Maximum Considered Earthquake (MCE) that is the usual basis of collapse safety checks in the western United States.

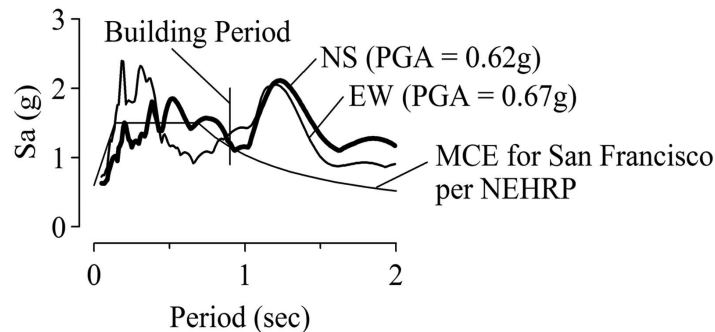


Figure 5. Takatori earthquake spectra at 5% damping.

The building period of ~ 0.9 sec indicated in Figure 5 is the apparent fundamental value in both principal directions during the $SF=0.6$ test. Ambient vibration measurements of the pre-experiment pristine building indicate periods of ~ 0.7 sec. The building may be especially vulnerable to the Takatori motions because the period lies in the spectral “valley”—the shaking intensity could increase as the period lengthens from yielding.

EXPERIMENT SUMMARY

The building survived shaking having $SF \leq 0.6$ and collapsed at $SF=1.0$ via a mechanism in the first story (Table 2). The behavior may be explained by the following sequence of events:

1. Yielding initially occurred in panel zones and columns at the baseplates.
2. As the strength of the panel zones increased via strain hardening, the strength of the columns effectively decreased due to biaxial bending and local buckling. The girders remained elastic due to their overstrength caused by actual yield stress being much greater than the nominal design value and slab-girder composite action.
3. Deformations then coalesced in the columns in response to increasing story drifts, and yielding and local buckling occurred at both the top and bottom of the columns in the first story. A story mechanism was created, and the building collapsed.

The building may be considered to have met its performance goal because it withstood shaking much greater than the Level 2 design basis. See Suita et al. (2008) for additional description of the experiment.

DISPLACEMENT RESULTS

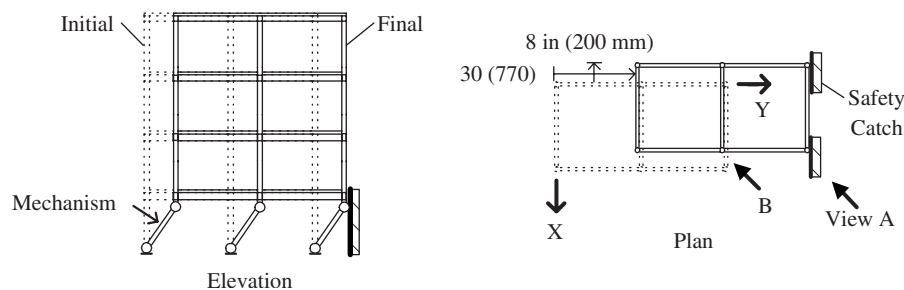
The side-sway mechanism had the primary direction of collapse in the Y-direction (Figures 6 and 7), and severe local buckling occurred at top and bottom of the columns in the first story (Figure 8). The exterior wall cladding was omitted on one side (Figure

Table 2. Experiment summary

Scale Factor (SF)	Building Response
0.05	No yielding. Small amplitude linear-elastic behavior.
0.2	No yielding. Peak drift ratio of ~ 0.005 . Equivalent to a Level 1 loading condition customarily taken as motions with a peak ground velocity of 10 in/sec (25 cm/sec).
0.4	Slight yielding. Peak drift ratio of ~ 0.01 and peak plastic rotations of ~ 0.005 rad. Equivalent to a Level 2 loading condition (20 in/sec (50 cm/sec)). <ul style="list-style-type: none"> • Yielding in columns at all baseplates • Yielding in panel zones at second floor
0.6	Yielding. Peak drift ratio of ~ 0.02 and peak plastic rotations of ~ 0.01 rad. <ul style="list-style-type: none"> • Yielding in columns at all baseplates • Slight local buckling of mid-side columns at baseplates • Yielding in panel zones at second floor and in middle columns at third floor • Peak base shear attained ~ 0.7 times building weight • Some damage to ALC panels (corner cracking) and interior partitions
1.0	Collapse. Peak drift ratio of 0.19 as building was restrained by safety-catch systems. <ul style="list-style-type: none"> • Mechanism in first story with severe local buckling in columns • No fracture of moment-frame connection welds • Damage to ALC panels in first story, including 3 falling off and numerous fractured connections resulting with hanging panels

7), but this did not cause a lateral twisting collapse. The floor loads were relatively heavy so that the mass eccentricity from the cladding was minimal, being only $\sim 3.5\%$ of the X-direction plan dimension. Good simulation of the experiment was achieved using a planar computer model (having no twisting degree-of-freedom), thus somewhat confirming that torsion did not play a major role in the collapse.

Figure 9 shows the first story displacement trajectories, and Figure 10 shows the displacement time histories. Large oscillations were oriented diagonally in plan. The displacement patterns for $SF \leq 0.6$ were similar indicating a quasi-linear-elastic behavior, but

**Figure 6.** Building collapse mechanism (no scale).

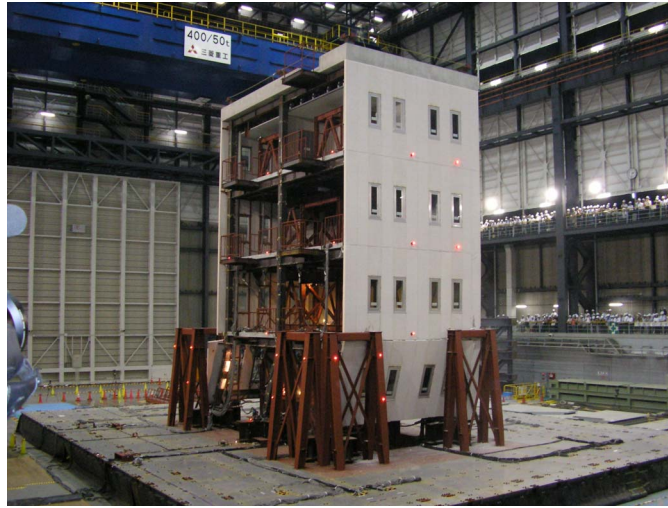


Figure 7. Building post-collapse configuration (from View A depicted in Figure 6). Safety-catch braced-frame systems were located at the corners of building. Note tilting of wall panels in first story.

the pattern started to change late in the $SF=0.6$ test. For $SF=1.0$, the response was markedly different leading up to collapse. The motions were still diagonal, but the oscillatory signature and peak amplitude trends were very different, indicating a softening of the structure caused by only a few large displacement excursions.

Figures 11 and 12 illustrate collapse during the $SF=1.0$ test. Deformations progressively concentrated in the first story as it softened, and as they increased unbounded in the first story, they decreased in the other stories. Collapse was sudden, occurring within a four-second duration of intense shaking. Some progressive oscillatory one-direction offsets (ratcheting) was evident.

EFFECT OF NONSTRUCTURAL COMPONENTS

Figure 13 shows peak story shears from the $SF=0.6$ test (Suita et al. 2008). The difference between the total building and moment-frame shears may be attributed mostly to the nonstructural elements. The figure indicates they resisted $\sim 9\%$ of the shear at the base and $\sim 15\%$ in the third story. Their role in the collapse behavior is difficult to ascertain because of their complex interaction with the moment-frames (Figure 4) and the fact that such components have deteriorating pinched hysteretic loop behavior and brittle failure modes when their connections to the frame fracture. For the $SF=1.0$ test, most of the ALC panels in the first story suffered major damage, with three falling off and many hanging (Matsuoka et al. 2008). It is noteworthy that good simulation of the experiment was achieved using a computer model based only on the moment-frames, suggesting that nonstructural elements had negligible influence on the collapse.



Figure 8. Severe local buckling of HSS column at baseplate, with drift ratio of 0.19 (from View B depicted in Figure 6).

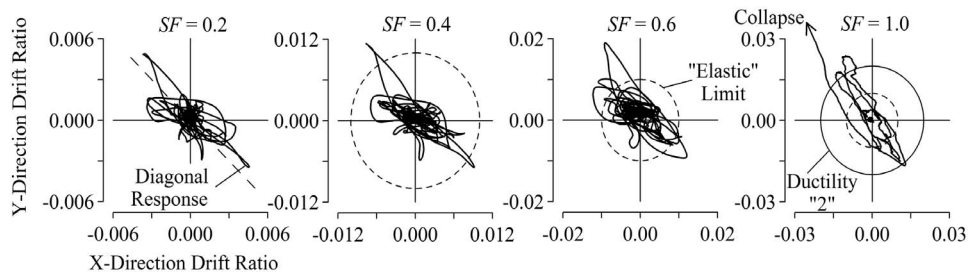


Figure 9. Plan view of the first-story motion ($SF=0.2, 0.4, 0.6,$ and 1.0). Circles at 0.01 indicate the drift customarily associated with the initiation of yielding in steel moment-frame buildings.

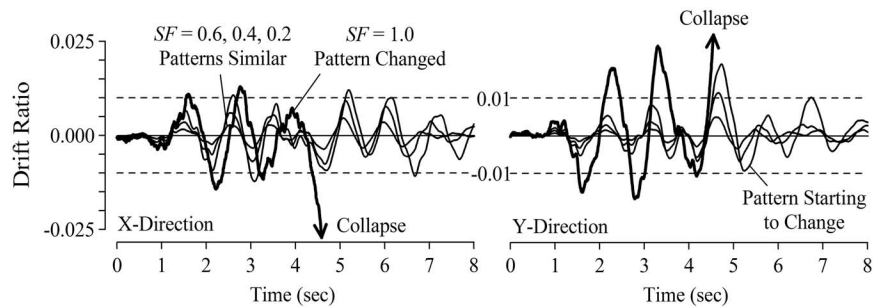


Figure 10. First-story drift-ratio time histories ($SF=0.2, 0.4, 0.6,$ and 1.0 results are superimposed to have the same starting time).

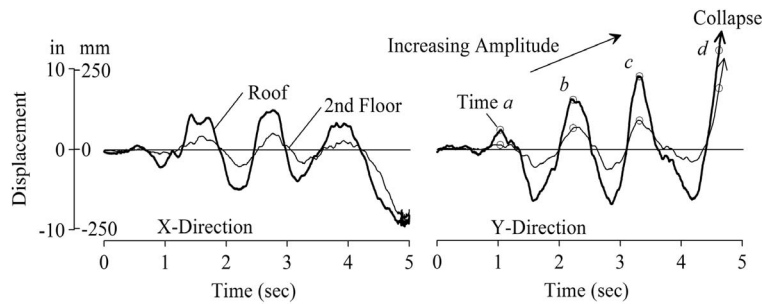


Figure 11. Displacement time history leading to collapse during $SF=1.0$.

STRONG-COLUMN AND WEAK-BEAM DESIGN

The building was designed to have strong-column and weak-beam (SCWB) framing, yet a story-mechanism collapse occurred. The SCWB check (Table 1) differs from customary United States methods, but the idea of balancing member moment capacities is similar. The check did not preclude a story-mechanism because of the following:

- *Column Biaxial Bending.* The dominant motions were oriented along the plan diagonal resulting with large concurrent bending moments in both column principal directions. This has the effect of reducing the strength of the columns, which was not considered, since the check was applied separately in each building principal direction.
- *Column Local Buckling.* The columns suffered strength deterioration from local buckling starting in the $SF=0.6$ test. The check used plastic flexural strengths.
- *Girder Overstrength.* The girders' as-tested yield stress was much larger than nominal value (by 32%), and slab-girder composite action was not included in the check. Hence, the girders were much stronger than the column and panel zone strengths used in the check.

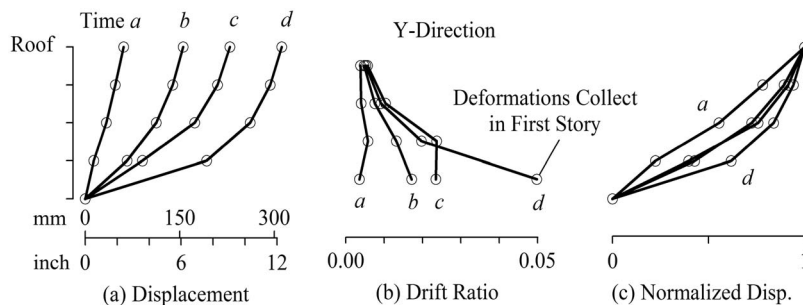


Figure 12. “Snapshots” of displacement and drift patterns at times a to d , per Figure 11. Normalized displacements are set with roof displacement equal to 1.0.

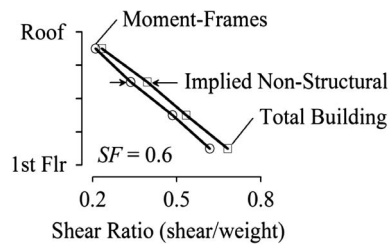


Figure 13. Envelopes of Y-direction peak story shears for total building and moment-frames.

SEAOC (1999) recognized deficiencies of customary SCWB checks and has suggested an alternative method in their Blue Book commentary. Column moments above the floor level under consideration are omitted from the proposed calculation.

COMPUTER SIMULATION OF EXPERIMENT

The minimum intensity causing collapse under a *single* application of the scaled Takatori record was needed to benchmark the PBE evaluations. The authors formulated a two-dimensional planar model having the best predictions in the category of 2-D Analysis by Practicing Engineers for an E-Defense blind analysis contest (*EERI Newsletter* 2008). This model was used to estimate the smallest earthquake scale factor (SF) causing collapse.

The model was essentially the same as the Y-direction model used in the PBE evaluation below, except for the nonlinear rotational springs representing the column flexure behaviors. They were based on component test data rather than *ASCE-41* guidelines (Figure 14). A key assumption was the reduction of column flexural strength to account for biaxial bending due to multidirectional earthquake excitation (Figure 14c).

The blind analysis had excellent agreement with the test oscillatory signature, instant in time of collapse, and pattern of yielding (Figures 15 and 16). However, the analysis had the exterior girders yielding at the second floor, whereas the test had panel zone yielding. This may be due to girder overstrength from composite action not accounted for in the analysis.

Figure 17 shows the incremental dynamic analysis (IDA) graph. The graph closely agrees with the test results (solid diamonds) confirming the adequacy of the computer model. It is also close to analyses having pre-existing damage from prior runs with $SF=0.4$ and 0.6 (hollow diamonds) indicating prior yielding had negligible effect on collapse response. The blind analysis model suggested that a single application of the Takatori record scaled to ~ 0.8 is close to the lower bound intensity just sufficient enough to cause collapse.

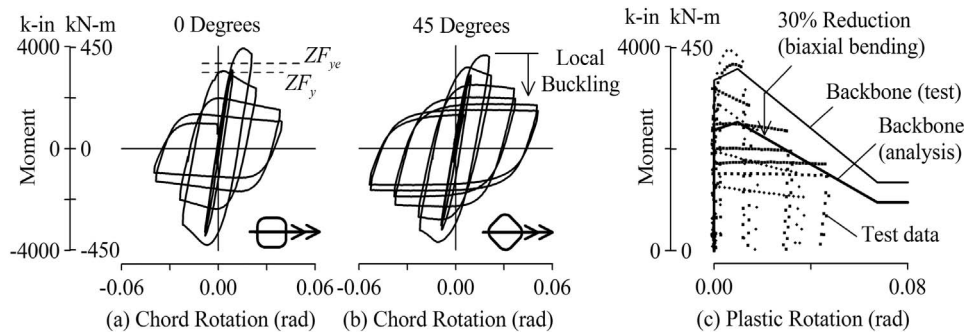


Figure 14. Column component flexure tests and backbone curves: (a) and (b) component test results for cantilever HSS column specimens (length=56.8 in [1468 mm], and axial load ratio, $P/P_y=0.15$), and (c) derivation of backbone model based on test data.

PBE GUIDELINES

The building was evaluated⁴ by *ASCE-41* and *FEMA-351*, and they are quite different (Table 3). *ASCE-41* is a deterministic approach that computes individual member demand-to-capacity ratios. For linear analysis, the checks use ductility m -factors intended to relate member elastic forces to inelastic deformations. For nonlinear analysis, *ASCE-41* provides criteria for member and connection inelastic deformations. *FEMA-351* computes confidence estimates for particular building failure modes.

The guidelines are very detailed (*ASCE-41* is 400 pages and *FEMA-351* is 200 pages) with prescriptive code-type command language (e.g., shall), which some engineers contend can inhibit good practice (Searer et al. 2008). This study strictly followed the guidelines as close as possible, but some compromises had to be made in order to apply them to the experiment. The main ones follow:

Single Set of Earthquake Records. The evaluation was based on the single set of records used in the experiment, whereas the guidelines stipulate use of multiple records. It would be ideal to have multiple collapse tests using the same building specimen type, but it is clear that such an ambitious experiment is cost prohibitive and probably will never be performed. Therefore, this exception was unavoidable, and it should be recognized that if other earthquake records were used, then different outcomes could result.

Regarding *FEMA-351*, the evaluation was based on a 50% confidence level to represent a median value for comparison to the experiment. *FEMA-351 Appendix A* was used to account for the deterministic earthquake input in the experiment. Reducing the demand uncertainty has the effect of increasing the capacity at a given confidence level.

⁴ Using *ASCE-41* for seismic evaluation is a Tier 3 approach per ASCE standard *Seismic Evaluation of Existing Buildings (ASCE-31 2003)*. However, the demand reduction factor of 0.75 was not used here because it appears to be intended to reduce conservatism in *design* versus *evaluation* earthquake motions (“mean-plus-one” standard deviation aspects) that are not relevant to the experiment having a given deterministic input.

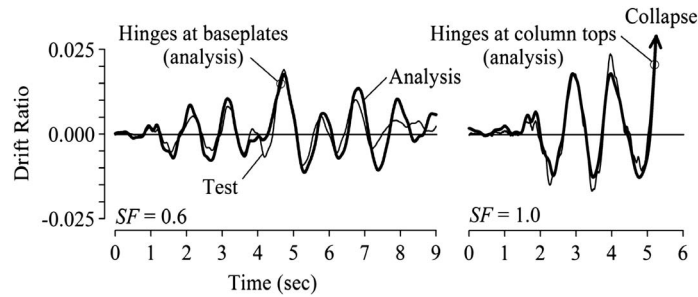


Figure 15. Time histories of first-story drift ratios comparing test and blind analysis.

Moment Connection Type. The connections were typical for Japan but are not common in the United States, so the guidelines did not specifically address them. They were assumed to be equivalent to an *improved WUF-welded web* type, i.e., a fully welded connection. This is a rugged connection having a large collapse prevention m -factor (5.3) and drift ratio capacity ($\theta_V=0.064$). It turns out that connections did not govern in *ASCE-41* but did control in *FEMA-351* (floor vertical collapse check).

COMPUTER MODELS

Practical application of PBE requires use of computer-based analysis. Both linear and nonlinear computer models were created using guideline criteria. Modeling and scaling of results were based on *ASCE-41* (*FEMA-351* refers to *FEMA-273* for modeling, which can be considered as superseded by *ASCE-41*). For linear analysis, a center-line model was used with girder properties based on the bare steel sections. For nonlinear analysis, a more detailed model having panel zones and slab-girder composite action was used. Nonstructural components were not modeled.

Separate planar models representing the moment-frames in each building principal direction were created (Figure 18). Results from the two directions were combined via the 30%-combination rule per *ASCE-41*. Member properties were well known so the knowledge factor (κ) used to scale *ASCE-41* capacities was set to unity. Three different procedures were applied: linear dynamic (response spectrum), nonlinear dynamic (time history), and nonlinear static (push over).

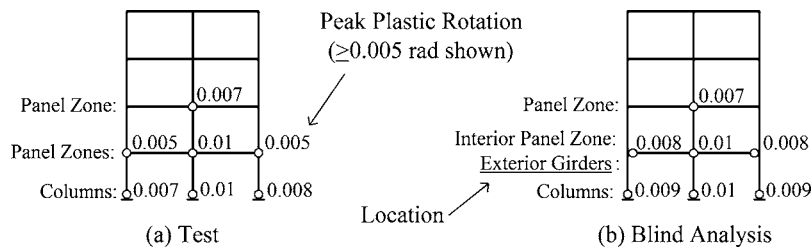


Figure 16. Distribution of yielding in test and blind analysis ($SF=0.6$).

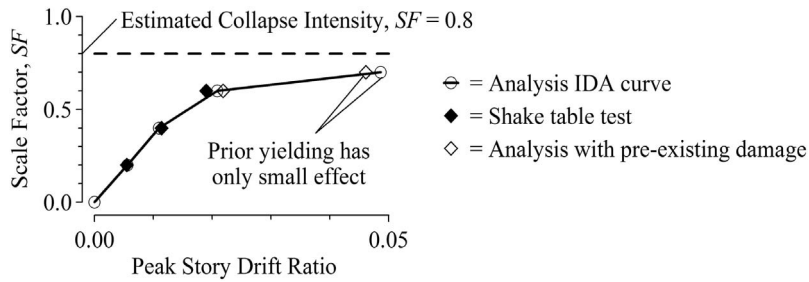


Figure 17. Blind analysis model incremental dynamic analysis (IDA) graph. Each symbol represents the peak-story drift ratio from shaking by the Takatori record, scaled by the particular scale factor.

LINEAR DYNAMIC PROCEDURE (LDP) MODELS

Girders and column members were modeled with standard linear-elastic beam-column line elements. Nodes were located at the intersection of the column and girder centerlines (Figure 19b). The baseplates were idealized as rotational springs having stiffness as estimated by Tada et al. (2007). The effects of slab-girder composite action were ignored, unlike the nonlinear models below. The fundamental periods of 1.1 sec and 1.0 sec in the X- and Y-directions, respectively, were greater than the measured periods from the test.

Dynamic analysis was conducted by the response spectrum method using 5% damping. *ASCE-41* displacement modification factors, relating linear-elastic to inelastic response, were equal to unity: inelastic displacement (C_1) and hysteretic behavior (C_2).

NONLINEAR DYNAMIC (NDP) AND STATIC (NSP) PROCEDURES MODELS

The models were like the linear versions, but with the inclusion of panel zones (Figure 19c). Inelastic actions were accounted for by nonlinear rotational springs connecting the members to the panel zones and baseplates. The spring properties were based on *ASCE-41* parameters (Figure 20a). Degrading strength and stiffness hysteretic behavior

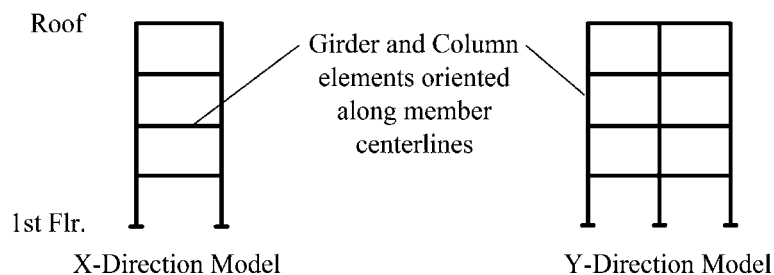


Figure 18. Computer models of case study building.

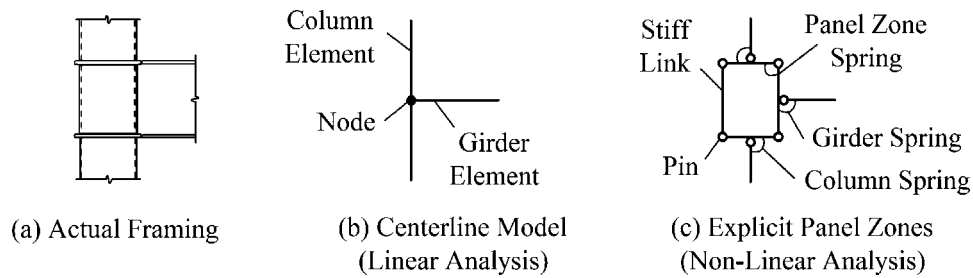


Figure 19. Computer modeling of girder-to-column framing.

(Figure 20b) was used for connections, girders, and columns, and a full-loop trilinear hysteretic behavior (Figure 20c) was used for panel zones, since they do not deteriorate per *ASCE-41*. P-delta effects were accounted for on the element level by geometric stiffness based on a truss bar analogy using axial force.

Composite action was assumed to increase girder stiffness, but not strength (girder moment of inertias were scaled by 1.5). The girders were relatively shallow versus the slab thickness. The periods in the X- and Y-directions were 0.98 sec and 0.91 sec, respectively. These are in better agreement with the test values versus those from the linear models because of the larger stiffness resulting from member clear spans and composite action.

NDP analysis was carried out using the time-history method. Raleigh damping of 3% was used in addition to the damping from inelastic member actions. NSP analysis used a lateral load pattern based on the fundamental mode shape.

For column flexure, the parameters (strength, a -value, b -value, CP acceptance criteria) depend on the axial force in the member. This was accounted for by first performing an analysis using assumed axial forces and then performing another analysis using adjusted parameters based on the peak axial forces from the first analysis.

It was found that the columns in the first story controlled the collapse behavior, in part because they were considered noncompact (Table 1). Per *ASCE-41*, the column plastic rotation at the onset of strength deterioration was very small (a -value ~ 0.002 rad), and once yielding occurred in the first story, a collapse mechanism formed almost immediately as the moment capacity dropped off quite rapidly (b -value ~ 0.003 rad). This is discussed below.

COMPUTER SOFTWARE

It was intended that several analysis programs would be used to contrast the use of different software packages: *OpenSees* (2007), *Perform-3D* (Powell 2007), and *PC-ANSR* (Maison 1992). Unfortunately, project schedule and budget constraints precluded this exercise. However, quality assurance analyses were run to test features relevant to modeling the building. It was found that all three packages were capable of producing similar results, and any one of them could have been employed. *OpenSees* was used for

Table 3. Overview of *ASCE-41* and *FEMA-351* procedures for steel moment-frame buildings

<i>ASCE-41</i>	<i>FEMA-351</i>
Originated from: <i>FEMA-356</i> (2000) and <i>FEMA-273</i> (1997).	Originated from: SAC Joint Venture Project pursuant to 1994 Northridge earthquake.
Performance levels: Criteria for immediate occupancy (IO), life safety (LS), and collapse prevention (CP).	Performance levels: Criteria for IO and CP.
Checks members: Connections, girders, columns, panel zones.	Checks failure modes: Side-sway collapse (drift check), floor vertical collapse (drift), column buckling (force), column splice fracture (force).
Collapse-safe when: Member demands are less than capacities via member-by-member demand-to-capacity ratio pass-fail type checks.	Collapse-safe when: Acceptable confidence achieved for precluding the failure modes.
Analysis procedures: Choice of linear static (LSP), linear dynamic (LDP), nonlinear static (NSP), and nonlinear dynamic (NDP).	Analysis procedures: Choice of LSP, LDP, NSP, and NDP.
<p>Acceptance criteria (for deformation-controlled actions): For <i>linear</i> analysis: $D/mC \leq 1$, where D=demand force from analysis factored to account for inelastic actions, C=capacity force per guideline rules, and m=ductility, m-factor depending on particular member properties and performance level (force is used as a proxy for deformations). For <i>nonlinear</i> analysis: $D/C \leq 1$, where D=demand deformation from analysis, and C=capacity deformation depending on particular member properties and performance level. If demand-to-capacity ratios for all members < 1, then the probable performance is deemed acceptable.</p>	<p>Acceptance criteria:</p> <ol style="list-style-type: none"> 1. Compute $\lambda = \gamma\gamma_a D / \phi C$, where D=demand (force or drift) from <i>linear</i> or <i>nonlinear</i> analysis, C=capacity per guideline rules, and γ, γ_a, ϕ=parameters to account for bias and uncertainties in computation of D and C. 2. Given λ computed above and another uncertainty parameter β_{UT}, the confidence level is read from tables. <p>If the computed confidence levels for precluding all failure modes exceed recommended values, then the probable performance is deemed acceptable.</p>
No checking of drifts.	No checking of member ductilities.

all nonlinear analyses primarily because it was used first. For linear analysis, the *SUPER-ETABS* program (Maison and Neuss 1983) was used. It must be stressed that the quality of structural analysis depends more on the skill of the engineer and less on the particular software package.

EVALUATION BY *ASCE-41* AND *FEMA-351*

The building was evaluated for the collapse prevention (CP) performance level. The smallest scale factors (SF) causing the building to fail the CP criteria were taken as the

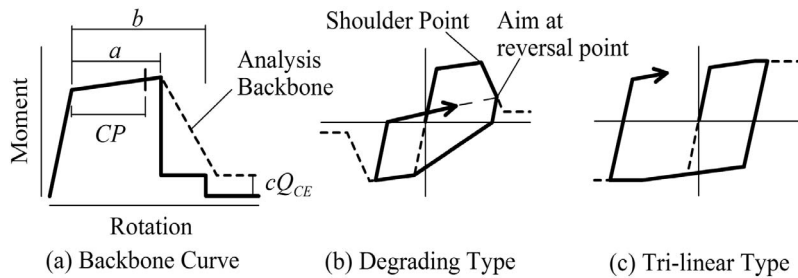


Figure 20. Computer modeling of nonlinear flexure behaviors.

“collapse” intensities for comparison with the experiment. How well the guidelines estimated the earthquake intensity causing collapse was quantified by computing a “safety” margin (M) defined as

$$M = SF_{test}/SF \quad (1)$$

where SF_{test} = earthquake scale factor causing collapse in the experiment, taken as 0.8, and SF = scale factor at which the building failed the CP criteria. When margins were greater than unity, this implied the guidelines were *conservative* because they predicted collapse at earthquake intensities smaller than the actual value.

LINEAR DYNAMIC PROCEDURE (LDP)

Figure 21 shows envelopes of peak displacements and drifts at two response levels: slight yielding ($SF=0.4$) and during collapse in the test ($SF=1.0$). Analysis overestimated the tests because the models were more flexible than the actual building. At $SF=0.4$, analysis had peak drifts in the middle stories whereas those from the test diminished with height. At $SF=1.0$, the collapse mechanism in the first story was clearly evident

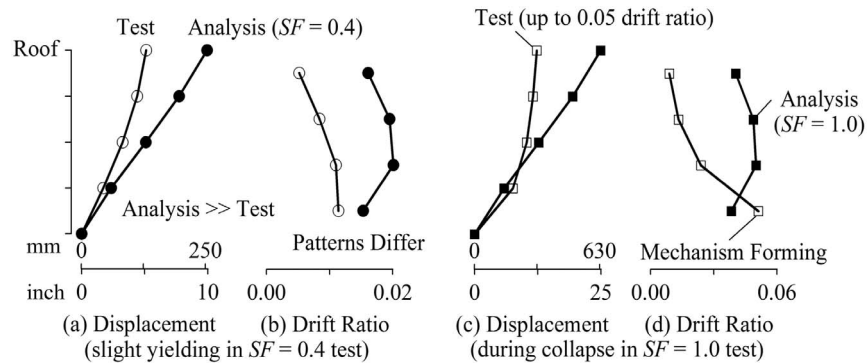


Figure 21. Comparison of peak displacements and drifts from LDP analyses and tests.

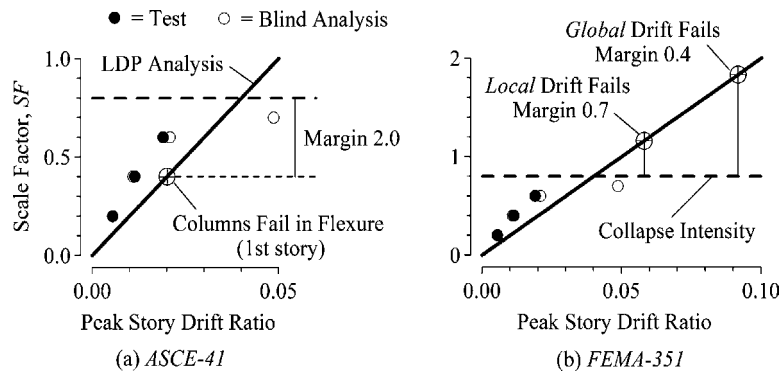


Figure 22. ASCE-41 and FEMA-351 evaluation by linear dynamic procedure (LDP).

in the test, and linear analysis could not capture such a localized nonlinear displacement response. The largest drift from analysis would incorrectly suggest the second or third story as key whereas the actual collapse mechanism was in the first story.

Figure 22 summarizes the PBE guideline evaluations. The LDP analysis plots as a straight line because the computer model is linear, and thus peak drifts are proportional to scale factor (SF). The peak drifts from the experiment (solid dots) and the blind analysis model (hollow dots) are also shown.

For ASCE-41, the columns in the first story failed the CP acceptance criteria at $SF=0.4$, translating to a conservative safety margin of two (Figure 22a). The columns failed due to excessive flexure, as in the test. Table 4 contains the governing CP criteria. Connections, girders, and panel zones were within their acceptance criteria at $SF=0.4$ meaning they did not govern.

For FEMA-351, the *local* drift check controlled at $SF=1.1$ resulting with an unconservative margin of 0.7 (Figure 22b, and Table 4). This is for the failure mode where the moment connections lose their ability to resist gravity loads leading to vertical collapse of the floor system. This did not occur in the test. The *global* drift check corresponds to a lateral sway-type collapse like that in the test, and it had an unconservative margin of 0.4. Column compressive buckling checks failed at larger scale factors indicating this failure mode did not govern.

NONLINEAR DYNAMIC PROCEDURE (NDP)

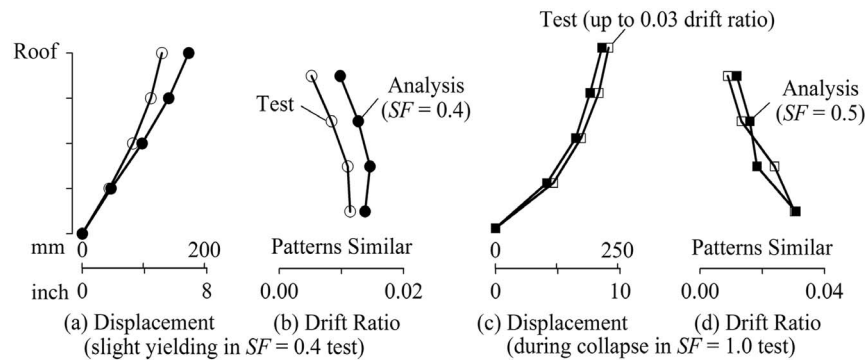
Figure 23 shows envelopes of peak deformations at slight yielding and during collapse in the test. Analysis displacement patterns were similar to those from the test. Note the change when going from slight yielding to collapse; displacements increased in the first story as the collapse mechanism forms. Because nonlinear analysis captured this effect, its displacements were in much better agreement to the test versus those from linear analysis.

Figure 24a summarizes the ASCE-41 evaluation depicted in an incremental dynamic

Table 4. Governing conditions in PBE evaluations

Method	Governing Condition
LDP <i>ASCE-41</i>	At $SF=0.4$, column demand-to-capacity ratios in first story attained limiting criterion: $0.2 \leq \frac{P_{UF}}{P_{CL}} \leq 0.5 \text{ and } \frac{P_{UF}}{P_{CL}} + \frac{8}{9} \left[\frac{M_x}{m_x M_{CE_x}} + \frac{M_y}{m_y M_{CE_y}} \right] \approx 1.0$ where P_{UF} =axial force based on limit-state analysis, M_x, M_y =bending moments from analysis, P_{CL} =lower bound compressive strength, M_{CE_x}, M_{CE_y} =expected bending strengths, and m_x, m_y = m -factors=1.5 for CP evaluation.
LDP <i>FEMA-351</i>	At $SF=1.1$, drift ratio in second story attained limiting criterion for floor vertical collapse at 50% confidence. $D = \frac{\phi C \lambda}{\gamma \gamma_a}$ where, D =drift ratio, $C=0.064$ drift capacity for the connections, ϕ, γ, γ_a =parameters to account for bias and uncertainties in computation of D and C , and λ =index parameter for 50% confidence.
NDP <i>ASCE-41</i>	At $SF=0.4$, columns in the first story attained limiting criterion: column plastic rotations=CP acceptance criterion= $0.8\theta_y \sim 0.002$ rad.
NDP <i>FEMA-351</i>	For $SF > 0.5$, drift ratios in first story increased abruptly and exceeded floor vertical collapse and side-sway criteria.

analysis (IDA) format. The IDA curve flattened out for $SF > 0.5$ reflecting collapse of the analysis model at a smaller scale factor than the actual collapse intensity. For $SF \leq 0.4$, the IDA curve agreed favorably with the peak drifts from the test indicating that the elastic properties of the building were reasonably captured. Columns in the first story were correctly identified as being the weak link in the building (Table 4). They failed the CP acceptance criterion at a $SF=0.4$, resulting in a conservative margin of two. Some connections, girders,

**Figure 23.** Comparison of peak displacements and drifts from NDP analyses and tests (analysis at $SF=0.5$ was incipient to collapse).

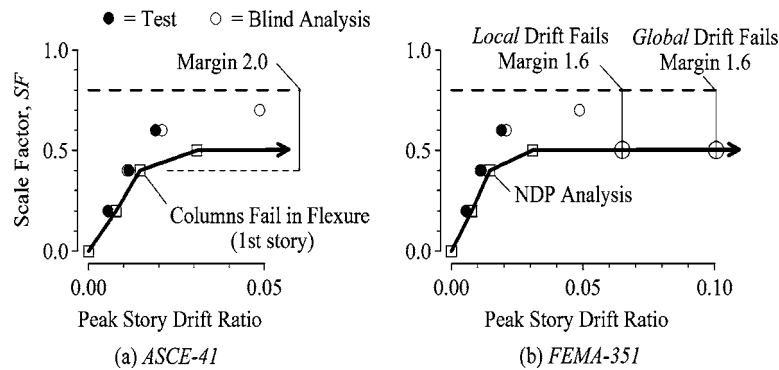


Figure 24. ASCE-41 and FEMA-351 evaluation by nonlinear dynamic procedure (NDP).

and panel zones experienced yielding, but their plastic rotations were within their acceptance criteria at $SF=0.4$, thus indicating they did not govern.

Figure 24b summarizes the FEMA-351 evaluation. The building became unstable (collapsing) at drifts well below the capacities defined by the *local* and *global* CP criteria (Table 4). Although the acceptance criterion for the *local* drift check was less than that for the *global* check, both of these failure modes had the same conservative margin of 1.6 because the CP drifts were on the IDA plateau. The column compressive forces were within the acceptance criteria indicating that column buckling did not govern.

NONLINEAR STATIC PROCEDURE (NSP)

The pushover graphs exhibited sudden strength degradation, with collapse occurring when the first story columns lost their flexural strength (plastic rotations exceeding shoulder point *a*-values). ASCE-41 does not allow NSP when the strength ratio $R > R_{max}$, as was the case here. Nevertheless, evaluation was continued to see the outcome. Taking the target displacement (δ_t) as the instant when the first story columns exceeded their acceptance criteria, a spectral acceleration was back-calculated using ASCE-41 equations. This acceleration corresponded to the NS Takatori spectrum when scaled by $SF \sim 0.5$. This was similar to the scale factor causing collapse via nonlinear time history analysis and hence the conclusions are the same as those for the NDP above.

DISCUSSION

The exercise revealed several points that warrant comment as follows:

Panel Zone and Composite Action Modeling. ASCE-41 has general discussion about panel zones but is silent on slab-girder composite action, and hence there is latitude regarding modeling these aspects. This study used a simple linear model (no panel zones or composite action) and a more detailed nonlinear model (explicit panel zones and composite action increasing girder stiffness). The linear model was too flexible and the

Table 5. Comparison of nonlinear modeling parameters for columns

Situation	Shoulder Point <i>a</i> -value (rad)	Failure Point <i>b</i> -value (rad)	Residual Strength <i>c</i> -factor
Columns in lower stories of case study building:			
Noncompact and Moderate Force, $0.2 < P/P_{CL} < 0.5$	~0.002	~0.003	0.2
Columns in upper stories of case study building:			
Noncompact and Low Force, $P/P_{CL} < 0.2$	~0.01	~0.015	0.2
If columns were:			
Compact and Moderate Force	~0.01	~0.02	0.2
If columns were:			
Compact and Low Force	~0.02	~0.024	0.6
Successful Blind Analysis Model	0.01	0.06	0.4

P =axial force from analysis, and P_{CL} =lower bound compressive strength.
 When Noncompact and Moderate Force: $a = 1\theta_y$, $b = 1.5\theta_y$, where $\theta_y = \frac{ZF_{ye}L}{6EI} \left(1 - \frac{P}{P_{ye}}\right)$

nonlinear model had a better fit to the test periods and peak displacements. Accounting for these stiffness qualities therefore was appropriate.

The nonlinear model did not account for increased girder strength from composite action and this may be the reason for girder yielding that was not observed in the test (Figure 16). The effects were ignored because it was arguably considered conservative, which may not always be the case in view of above.

Another aspect with panel zone and composite action modeling is the distribution of yielding among the components. Gupta and Krawinkler (1999) found that such computer modeling features (that can be difficult to define accurately) influence computed member plastic rotations. Concentration of plastic deformations can occur in either the girders or panel zones or shared between these elements and possibly also with the columns, depending on the model formulation. *ASCE-41* uses plastic rotations for acceptance criteria when using nonlinear analysis and this is a potential drawback.

Column Modeling and Evaluation. The first story columns controlled the building collapse behavior, and how they were handled by the guidelines was crucial. The column section was borderline compact per Japan code and *AISC* rules, but noncompact per *ASCE-41* and *AISC Seismic* (Table 1). The compactness aspect had a large impact.

Table 5 illustrates the effect of column section compactness and axial force on the flexure backbone curve. There are huge reductions when going from compact to noncompact and low to moderate force. Per *ASCE-41*, the lower story columns (noncompact and moderate force) had very little ductility, essentially being type 3 force-controlled members. However, *ASCE-41* stipulates that column flexural behaviors are deformation-controlled when $P/P_{CL} < 0.5$, and hence they were evaluated as such.

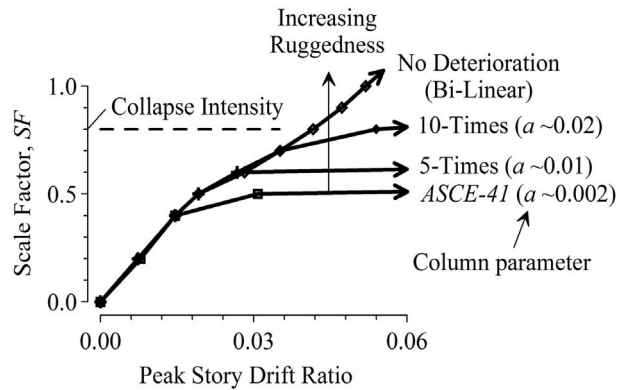


Figure 25. Incremental dynamic analysis (IDA) showing influence of different shoulder point a -values on collapse ruggedness.

Also listed are the parameters based on component tests used in the blind analysis model that successfully simulated the building response, and these are much larger than the *ASCE-41* values. While it is understandable that *ASCE-41* values may have been set low in the absence of test data, unfortunately the guideline offers no comment to alert the engineer on whether particular provisions were set conservative as was apparently the case here.

The magnitude of the backbone curve a -value has a large effect on the building collapse behavior. Figure 25 shows the IDA graphs from models that vary the shoulder point a -value of the columns (the rate at which the columns lose strength after a -value exceedance was held constant). With no strength deterioration, the building is robust, having no collapse at $SF=1.5$. With a -values set at 10-times the *ASCE-41* values, analysis agrees with the actual collapse intensity. While it is clear this is an important parameter, other factors are likely to be influential as well, such as member strength and the rate at which strength deteriorates.

Floor Vertical Collapse. The floor vertical collapse failure mode governed in the *FEMA-351* evaluations (local drift check). However, no distress in the girder-to-column connections was observed in the experiment. *FEMA-351* uses drift as a proxy for connection demand that was not the case in the experiment; distortions occurred mostly in the panel zones and columns. Using drift to check for connection damage was inappropriate for this case.

Moreover, floor vertical collapse, resulting from moment connection fracture, is a failure mode that has yet to be documented in lab tests, computer simulations, or post-earthquake surveys. It is relevant to note that this check did not have a strong consensus among the SAC Joint Venture researchers (Lee and Foutch 2004); some wanted a high confidence against its occurrence, while others wanted to omit the check altogether.

Guideline Interpretations. There were situations where judgments had to be made about the intent of various provisions and how to comply with them. While example applications were undoubtedly part of the guideline development, they were not available (referenced) to clarify guideline use. Example cases are essential to promote uniformity in guideline application. In addition, the guidelines seemed to imply that linear procedures were intentionally biased to be conservative versus nonlinear procedures. Although this may have been intended, it was not specifically stated in the guidelines nor was this apparent in the case study.

FINDINGS

This study is the first of its kind and it must be emphasized that it dealt with only one collapse experiment, and it is therefore prudent *not* to over-generalize the findings. Strictly speaking, the results are for the particular experiment, and caution must be exercised when applying them elsewhere. A summary of key points follow.

Shake Table Test Results. The building achieved its design objective by withstanding shaking intensities much greater than that required by the Japan building code (design basis corresponded to $SF=0.4$). For $SF \leq 0.6$, the behavior was fairly linear-elastic with the building responding in a desirable side-sway manner having yielding in panel zones and at the column bases. At $SF=1.0$, the dynamic response changed markedly due to inelastic actions and the building then entered into a sequence of lengthened period oscillations having progressively increasing amplitudes leading to collapse via a mechanism in the first story.

The story-mechanism occurred in spite of a strong-column and weak-beam design because of factors not accounted for in customary calculations: large concurrent bending moments in both column principal directions (biaxial bending), column local buckling, and beam overstrength.

Test Simulation by Computer Analysis. State-of-practice nonlinear analysis was able to simulate the experiment in terms of yielding pattern, peak displacements and instant in time of collapse (blind analysis modeling). The computer model did not include torsion or nonstructural components. Successful analysis used specific data about hysteretic behavior from component tests that are not often available when creating computer models in engineering practice. The *ASCE-41* modeling parameters, which would otherwise be used, were much smaller (more conservative) than those from component tests and using these led to less accurate results.

The deformation at which strength deterioration occurred in the columns was an important factor in the building collapse behavior (a -values). This and member strength were probably the two most important parameters governing the building collapse ruggedness. Analysis also suggested that yielding in shaking runs prior to the final $SF=1.0$

collapse run had a negligible effect on the collapse behavior (pre-existing damage did not appreciably weaken the building).

PBE Performance Assessment. *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. In terms of safety margins, *FEMA-351* predicted collapse more accurately than *ASCE-41*, but it incorrectly identified floor vertical collapse as controlling.

ASCE-41 linear (LDP) and nonlinear (NDP and NSP) procedures had the same margins because the CP acceptance criteria was exceeded when the building was essentially linear-elastic. This had a relatively large safety margin of about two, highlighting the fact that the linear procedure *m*-factors, and the nonlinear deformation parameters (*a*-values and plastic deformation acceptance criteria) were too small (over-conservative) for this case.

The *FEMA-351* linear procedure (LDP) 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 CP evaluation was not well suited for this type of behavior. For the nonlinear procedure (NDP), the building was unstable (collapsing) at drifts well below the *local* and *global* CP criteria at 50% confidence.

The highly detailed *ASCE-41* and *FEMA-351* provisions gave the impression that the results should be quite accurate. However, the case study revealed that the predicted collapse intensity was as much as a factor of two from the actual value.

ASCE-41 was suitable as a *design tool* for building rehabilitation because although it was 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. Its process of member-by-member checking is pragmatic within a design context. *FEMA-351* is appealing as a building *performance predictor* because it provides probability estimates for specific failure modes, but it does not have member-acceptance criteria.

RECOMMENDATIONS

The following needs became apparent during the course of the exercise. It must be recognized they are from the study of a *single* collapse experiment, and hence may not be universal. With this caveat, they are offered here for consideration by practicing engineers and researchers:

1. Some *ASCE-41* component-modeling parameters (*a*, *b*, and *c*) and acceptance criteria (*m*-factors and plastic deformations) appear to be too small (conservative), and hence require improved calibration to better simulate actual behavior. A recent supplement to *ASCE-41* recognizes this for concrete structures (Elwood et al. 2007), and a similar update ought to be created for steel moment-frames.
2. *ASCE-41* should adopt a quality ranking system to indicate how well component-modeling parameters values (*a*, *b*, and *c*) and acceptance criteria

(*m*-factors and plastic deformations) 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 *Low*, meaning values are based mostly on expert opinion and thus may be conservative. Engineers could then make better-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. 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, potentially leading to failure of the floor system.
4. Use of linear procedures for 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. Better collapse prevention performance measures should be sought. *ASCE-41* and *FEMA-351* have a fundamental weakness when evaluating for collapse prevention. Both use finite peak deformation as basis of judgment: component ductility in *ASCE-41* and story drift in *FEMA-351*. The case study demonstrated when the shaking intensity is close to that causing collapse, the key peak deformations can be sensitive to small changes in intensity. Reliably estimating peak deformations in this intensity range is difficult. In addition, subtle differences in the way certain aspects—e.g., panel zones or composite action—are modeled can influence the distribution of plastic deformations among the components, thereby injecting unpredictability in local ductility estimates. Thus, basing collapse prevention acceptance criteria on finite peak deformations is open to question. Global behavior parameters are better, and a performance measure based on ground shaking *intensity* is described in the next point.
6. For collapse prevention evaluation, future generations of PBE ought to consider the use of a *safety margin* informed by incremental dynamic analysis (IDA) as the measure (Figure 26). The acceptance criterion is that the safety margin must be greater than unity ($SI_c/SI_d > 1$). The shaking intensity causing collapse (instability) is more meaningful than various finite deformations just on the threshold of collapse and is thus preferred. It also avoids use of local member ductility as acceptance criteria since these can be difficult to accurately estimate due to computer modeling limitations. Such performance measures are the subject of ongoing research (Deierlein et al. 2008).
7. Finally, guideline writers are encouraged to publish more example applications illustrating intended provision use, and such examples should be referenced in guideline commentary. This would go a long way toward making guidelines better understood and embraced by practicing engineers.

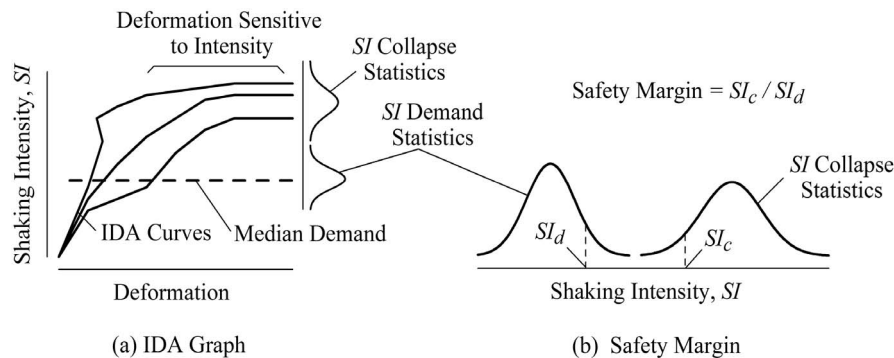


Figure 26. A collapse prevention evaluation approach based on shaking intensity (shaking intensity could be the scale factor applied to a suite of earthquake records, and deformation could be roof displacement or peak story drift).

CLOSING THOUGHTS

Accurate characterization of collapse within the context of PBE is akin to the search for the Holy Grail—perhaps a noble, but highly elusive, goal. The case study provides a sobering illustration on some of the challenges. A relatively simple lab specimen building was evaluated by current PBE guidelines, yet the results had considerable variation with some procedures:

- Being unconservative in terms of safety margin
- Identifying incorrect failure mode
- Having complicated methods, such that engineers could miss the forest for the trees by being consumed with calculation minutia
- Producing an illusion of accuracy, given that PBE calculations for shaking intensities causing rejection of collapse prevention acceptance criteria varied by a factor of two, versus the actual intensity causing collapse

Despite these shortcomings, the bottom-line conclusions from using the guidelines were generally conservative, so “appropriate” outcomes—in terms of providing for public safety—was mostly achieved. PBE is an evolving science, and engineers should recognize that current guidelines are probably more *conservative* than *accurate*, and they should not yet be treated as building-code dogma. Case studies, like that presented in this paper, are essential for gauging the guidelines against reality, even if they can only provide fleeting glimpses, rather than definitive judgments.

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