EVALUATION OF ANALYSIS PROCEDURES FOR PERFORMANCE-BASED SEISMIC DESIGN OF BUILDINGS

H S LEW[1] And Sashi K KUNNATH[2]

SUMMARY

A critical issue in seismic design is how to account for energy dissipation through inelastic deformation of structural components and the use of control devices. A nonlinear analysis accounts for inelastic deformation capacity of components through the modeling of force-deformation characteristics. Current building codes, however, permit linear elastic analysis to predict the structural response and estimate seismic demands. Since the actual response of most structures under the design earthquake loading results in inelastic behavior, linear elastic procedure are clearly inaccurate and inadequate. The recent introduction of a performance-based design philosophy in FEMA-273 (1997) for seismic rehabilitation of buildings in the United States marks a significant departure from tradition seismic design. Inherent in the new guidelines are four different procedures for estimating seismic demand in building structures. They are 1) linear static, 2) linear dynamic, 3) nonlinear static and 4) nonlinear dynamic procedures. In an attempt to validate the different analytical methods, case studies of several existing buildings were carried out. A typical subset of analyses for a 7-story reinforced concrete building is presented in this paper. It is shown that each of the different methods results in vastly different responses and that the acceptance criteria specified in FEMA-273 can be inconsistent in establishing a reliable basis for assessing the adequacy of a building design.

INTRODUCTION

A conceptual framework for performance-based design of structures is presented in the NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA, 1997). Inherent in the FEMA-273 procedure are three basic components: (1) definition of a performance objective, categorized in the guidelines by four levels: Operational, Immediate Occupancy, Life Safety and Collapse Prevention; (2) demand prediction using four alternative analysis procedures; and (3) acceptance criteria using force and/or deformation limits which are related to the performance objectives set forth in step (1). The estimation of seismic demands can be accomplished by four alternative analysis procedures. The computed demands are then compared to so-called "acceptance criteria" which establish the adequacy of the design. Acceptance criteria for linear procedures are based on force estimates while those for nonlinear procedures are based on deformation estimates. The objective of this study is to evaluate the different procedures and identify any shortcomings, if any, in the proposed guidelines.

EVALUATION METHODOLOGY

The evaluation of the four analysis methods recommended in FEMA-273 will be carried out on an existing 7-story concrete frame building that was instrumented prior to the 1971 San Fernando earthquake. The advantage in selecting this building for the evaluation study is the fact that the building model can be calibrated against observed data. The overall evaluation will comprise the following tasks:

1. Develop and validate a two-dimensional model of the building.
2. Select an appropriate set of ground motions to characterize the earthquake loading at the site.

[1] *Building and Fire Research Lab, National Institute of Standards and Technology, Gaithersburg, MD 20899-8610*
[2] *Department of Civil and Environmental Engineering, University of Central Florida, Orlando, FL 32816-2450*

3. Analyze the building model using each of the four analysis procedures.
4. Compare the computed demands with the acceptance criteria specified in FEMA-273.
5. Identify the differences in the various analysis procedures and their impact on seismic design.

DESCRIPTION AND MODELING OF BUILDING

The seven-story Holiday Inn building considered in the present evaluation has a rectangular plan (Figure 1) with overall dimensions of approximately 62'8" in the north-south (transverse) direction and 150' in the east-west (longitudinal) direction. The total height of the building is 65'-8 1/2" with variable story heights (13'-6" for the first floor, 8'-8 1/2" for second through 6th floor and 8'-8" for the seventh floor). The sub-structural system for the building consists of pile foundations. All pile caps are connected by a grid of tie beams and grade beams. For the superstructure, the floor system consists of RC flat slabs and perimeter beams supported by concrete columns. The RC flat slab is 10 inches thick at the second floor, 8.5 inches thick at the third to the seventh level and 8 inches thick at the roof level. Material properties specified were: 5 ksi concrete for the columns on the ground through the 2nd floor, 4 ksi concrete for the remaining columns and the 2nd floor beams, and 3 ksi for all remaining concrete. 40 ksi reinforcing steel was specified for beams and slabs while 60 ksi steel was used in the columns. In general, interior partitions are gypsum wall-board on metal studs. The north side of the building has four bays of brick masonry infill between the ground and second floor. Nominal expansion joints separate the walls from the columns and spandrel beams. These walls are not designed as part of the lateral resisting system but do contribute to the stiffness of the building in the longitudinal (EW) direction. The lateral load in each direction is resisted primarily by perimeter spandrel beam-column frames. The interior slab-column frames are also expected to carry a significant portion of the lateral load.

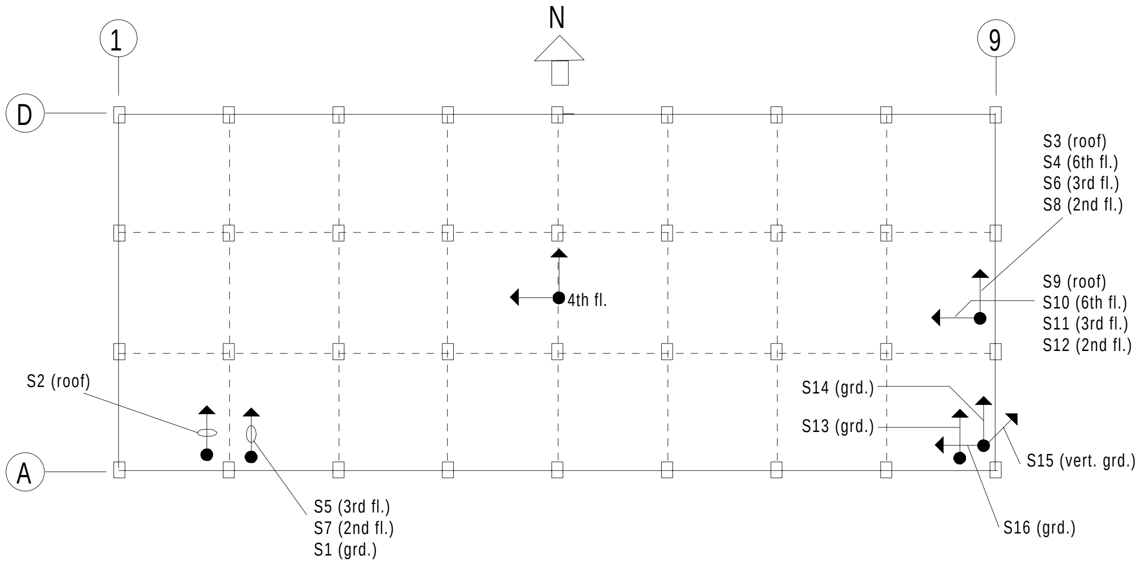


Figure 1. Typical Floor Framing Plan and Location of Strong Motion Sensors

The building was equipped with 16 CSMIP (California Strong Motion Instrumentation Program) sensors as shown in Figure 1. Ten of these sensors recorded the building motion in the north-south (transverse) direction, five recorded the east-west response and one sensor recorded the vertical acceleration.

Building Model for Evaluation Studies

An IDASS (Kunnath, 1995) building model of the Holiday Inn building was developed considering both the interior and exterior frames. The two interior frames and the two exterior frames were considered to be identical

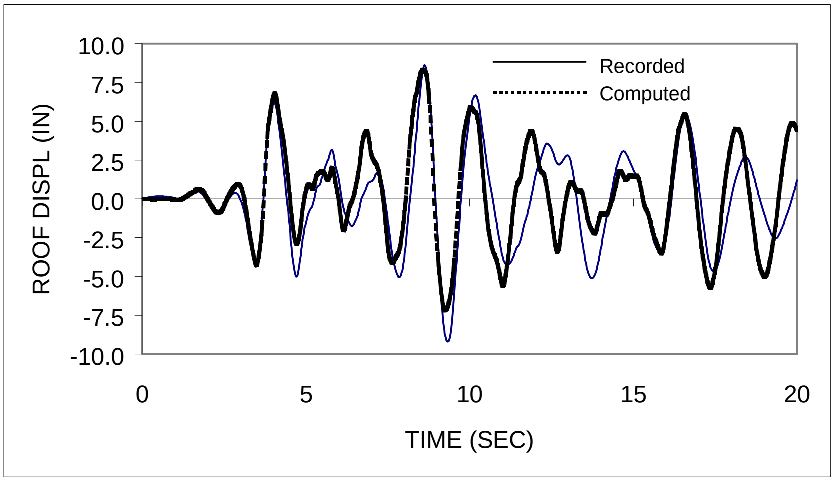


Figure 2. Computed vs. Observed Response of the Holiday Inn Building

for purposes of the modeling. The presence of the infills in the north perimeter frame were not modeled since they were assumed to contribute primarily to the initial stiffness of the frame. A total of 20 separate column types and 15 different beam types were used to model the 7-story frame structure. Slab steel was included when computing the negative capacity of the T-beam sections. A 30% increase in the specified nominal material properties was assumed in developing the moment-curvature data for the elements. A separate IDASS preprocessor which is capable of generating moment-curvature envelopes using a fiber model algorithm and considers the effect of confining steel using Mander's confinement model was used to generate the trilinear moment-curvature envelopes for each element type. The building was assumed to be fixed at the base. The floor weights of the building were estimated as: 1341 kips at the roof level, 1751 kips at the second level and 1381 kips at all other levels.

Validation of Building Model

The initial stiffness values of beam and column elements were tuned to match the initial period and elastic phase of the recorded roof response. The building suffered minor damage in the 1971 San Fernando earthquake. Recorded data indicates that the fundamental period of the building prior to Northridge to be in the range of 1.2 - 1.4 seconds. The fundamental period using gross section properties was 0.8 seconds. In order to match the initial elastic response of the structure, the initial stiffness values were scaled uniformly as follows: column EI values were scaled to 0.4 times the original values while beam EI values were reduced to 0.33 times the gross uncracked quantities. These reductions are quite reasonable for reinforced concrete structures following distributed cracking due to dead and live loads and any minor damage resulting from previous seismic events. No additional calibration of mass or element capacity was required. The response of the calibrated building model to the input ground motion is shown in Figure 2. Tuning of the building model to achieve a reasonable representation of mass and stiffness was considered crucial to the overall objective of this research.

SELECTION OF GROUND MOTIONS

The nonlinear response of structures is strongly influenced by ground motion characteristics such as magnitude, frequency content, strong motion duration, site conditions, etc. FEMA-273 provides a basis for selecting ground motions based on different hazard levels. A probabilistic approach is used to define these hazard levels wherein earthquakes with a certain probability of exceedance (corresponding to some mean return period) are used.

As part of the SAC (Structural Engineers Association of California, SEAOC; Applied Technology Council, ATC; and California Universities for Research in Earthquake Engineering, CUREe) Phase 2 Steel Project, a set of accelerograms have been developed for use in various structural investigations (SAC Draft Report, 1997). Suites of time histories for 2%, 10% and 50% probability of exceedance in 50 years (referred to as 2%/50, 10%/50, and 50%/50, respectively) for three locations in the United States and for firm soil conditions have been developed. These SAC acceleration time-histories have been derived from historical recordings as well as physical simulations. These have been modified so that their mean response spectra matches the target 1997 NEHRP (National Earthquake Hazard Reduction Program) design spectrum modified for soil category SD. In order to preserve the original frequency contents and phasing of the ground motions, the shapes of the response spectra of the original time-histories are not altered. The accelerations are amplitude scaled by a factor that minimized the weighted sum of the squared error between the target spectrum values and average of the spectra of two horizontal components at four discrete periods of 0.3, 1.0, 2.0, and 4.0 seconds.

For the present study, only the 10%/50 records will be considered in the dynamic time-history evaluations. Figure 3 shows the mean spectra of all records used in the evaluation. Also shown in the figure is the NEHRP spectra for comparison.

Selection of Strong Motion Duration

Since detailed time-history evaluations are time-consuming, it was considered prudent to use only the strong motion duration of each earthquake. In order to determine the strong motion component of each record, the method proposed by Trifunac and Brady (1975) is used. In this method, the monotonic function, E(T), which is the integral of the square of the acceleration time history, is plotted as ordinate with time, T, as abscissa. The maximum value of E(T), Em, at the end of the record is a measure of the maximum energy imparted by the earthquake. The strong motion duration is defined as the time taken to grow E(T) from 5% to 95% of Em.

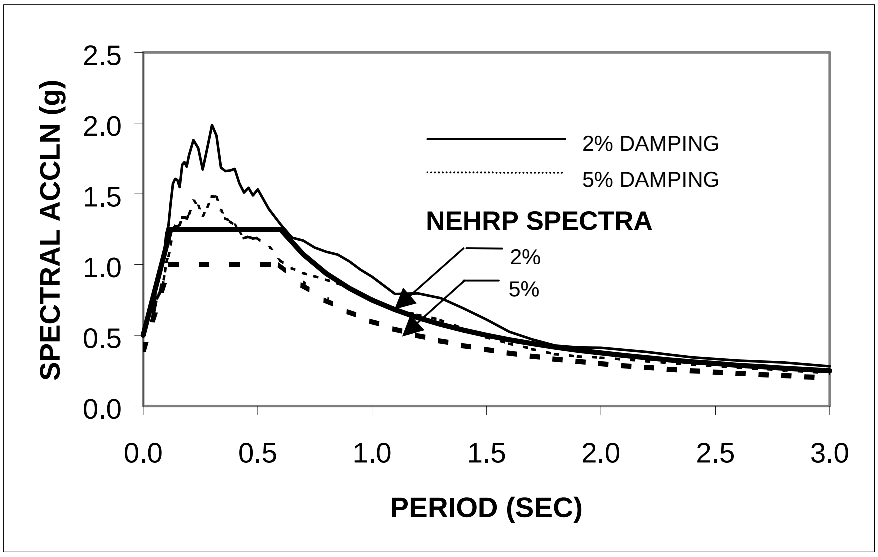


Figure 3. Site Specific Design Spectra Corresponding to 10% Probability of Exceedance in 50 Years.

SUMMARY OF ANALYTICAL STUDY

Linear Static Procedure (LSP)

The lateral forces applied to the building were computed using Equation (3-7) in FEMA-273 where the period-dependent factor k was calculated as 1.28. The fundamental period of the building based on an eigenvalue analysis was determined to be 1.26 secs. The lateral forces are shown in Table 1. The calibrated building model was subjected to the lateral forces shown in Table 1. The resulting forces were used to compute the allowable "m" factors for each element in the building. In calculating the "m" factors, the maximum axial and shear forces in a given story level were used. The axial stress ratios were less than 0.4 for all columns. Shear stress values exceeded 6.0 in many cases. The resulting acceptance criteria for columns and beams were determined in accordance with the provisions of Table 6-10 and 6-11 of FEMA-273. A comparison of the demand-to-capacity (DCR) ratios for columns and beams by story level are shown in Figure 4. In developing the DCRs, the maximum value of the DCR for an element in the story level was used. This is consistent with FEMA-273 recommendations. The acceptance criteria specified in FEMA-273 are based on performance objectives. In the plots developed for the present evaluation study, two essential performance levels are considered: Life Safety (LS) and Collapse Prevention (CP).

Table 1. Lateral Forces Used for Linear Static Procedure

Story	Weight	Height	W * H ^k	Fx	Fx
7	402	66	129559	350	382
6	414	57	109675	296	323
5	414	48	87266	236	257
4	414	40	66324	179	195
3	414	31	47071	127	139
2	414	22	29796	80	88
1	524	14	19019	51	56

Note: W = Story weight (in kips)
H = Story height (in feet)
k = 1.28 (see 3.3.1, FEMA-273)
Fx = Lateral force applied at floor level 'x'

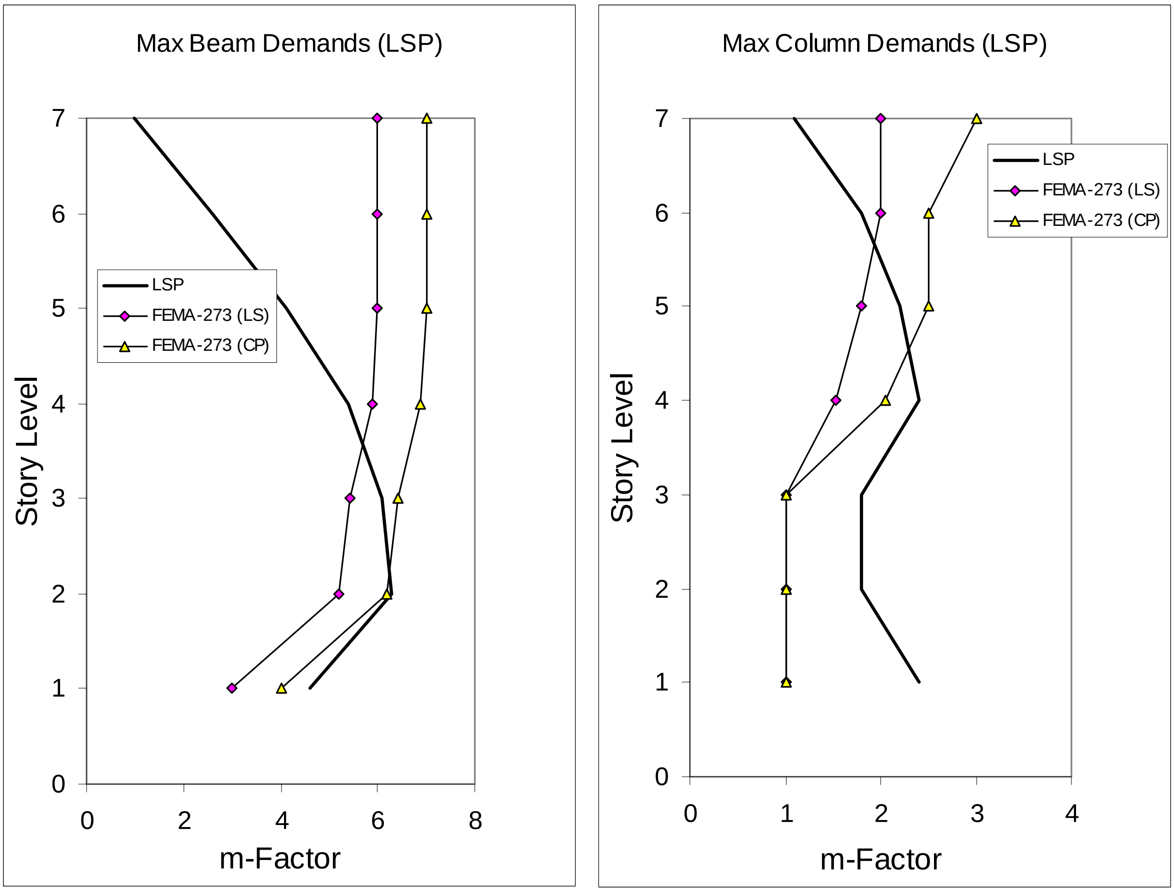


Figure 4. Comparison of Element Demands using LSP with Acceptance Criteria
Notation: LS = Life Safety ; CP = Collapse Prevention)

Linear Dynamic Procedure (LDP)

The SAC ground motions were used in all dynamic simulations. As indicated earlier, only the strong motion duration of each record was used in the analyses. Twenty records, which represented an event with a 10%

probability of being exceeded in 50 years, were used in this phase of the evaluation. For each run, the maximum effects for a particular component (beam and column) in a story level were recorded. The mean and maximum component forces for the 20 simulations were computed. These element forces were converted into "m" factors and compared with acceptance criteria. The results are displayed in Figure 5.

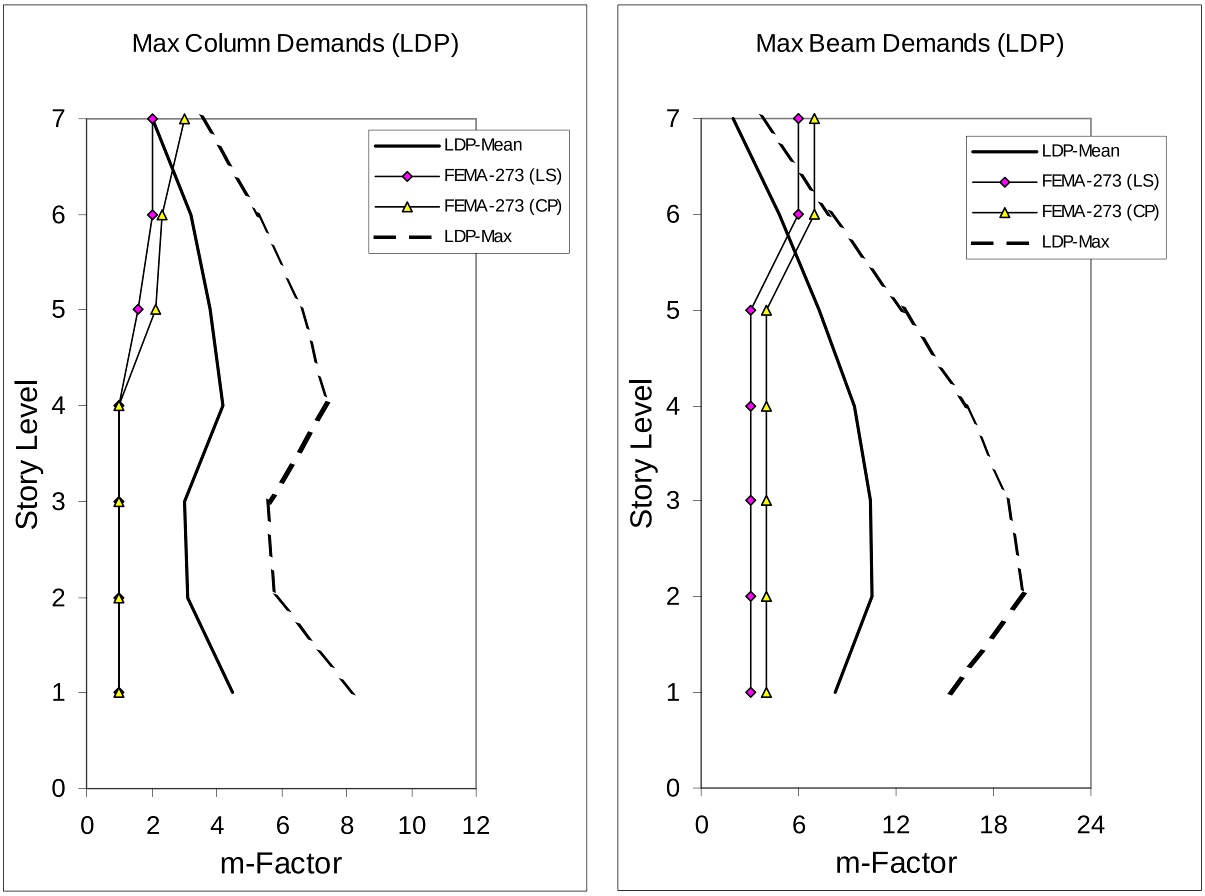


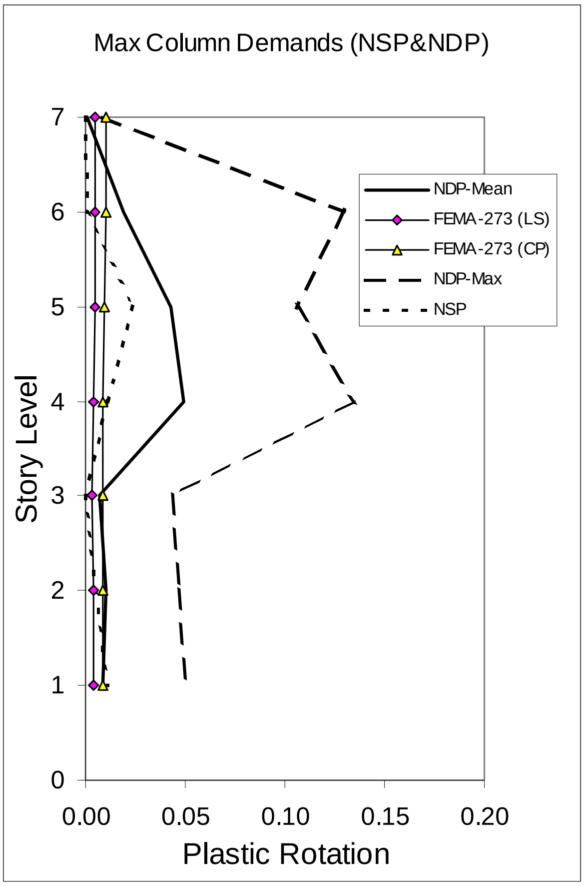
Figure 5. Comparison of Element Demands using LDP

Nonlinear Static Procedure (NSP)

The first of the two nonlinear procedures recommended in FEMA-273 is the so-called pushover analysis. The load pattern to be used in the pushover analysis can either be a triangular distribution based on forces computed in the linear static procedure or a uniform distribution that is expected to represent a post-yield scenario. In the present evaluation, the triangular pattern was used. Member plastic rotations were calculated from chord rotations of the members as suggested in FEMA-273. The imposed demands (plastic rotations) are compared to the acceptance criteria as a function of story level in Figure 6. The demand values shown correspond to the maximum plastic rotation experienced by an element (beam or column) at the given story level.

Nonlinear Dynamic Analyses (NDP)

The building model was subjected to the same 20 seismic inputs used for LDP. The elements were allowed to respond inelastically using the multi-parameter hysteresis model available in IDASS with nominal degrading characteristics (typical of well-detailed sections). The resulting average force demands were used to develop acceptance criteria for the individual elements. It was found that the average peak forces were similar to those obtained in the nonlinear static procedure (this is to be expected for yielding systems). The demands are compared to FEMA-273 allowable levels in Figure 6. Note that both nonlinear demands (NSP and NDP) are superimposed on the same plots since the acceptance criteria were the same for both methods.



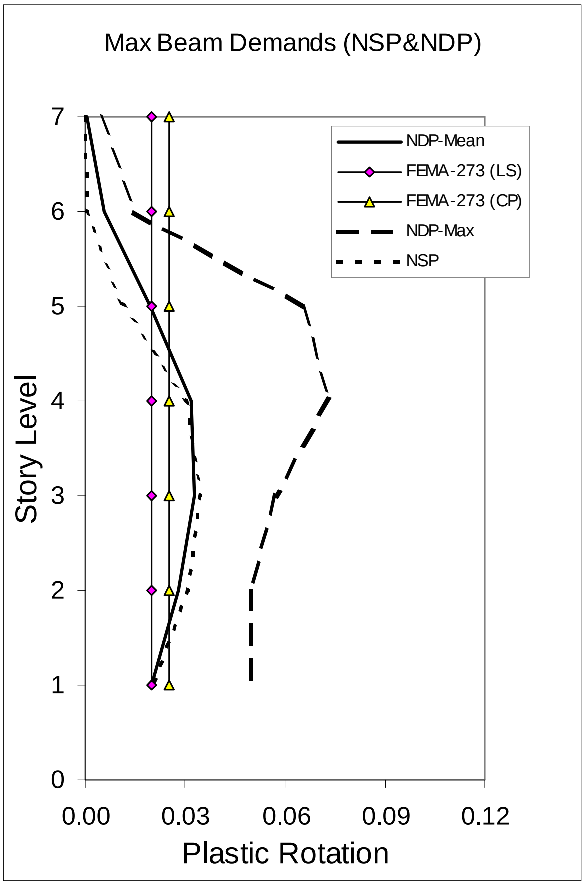


Figure 6. Comparison of Element Demands using Nonlinear Procedures (NSP and NDP) with FEMA-273 Acceptance Criteria

SUMMARY OF FINDINGS

Results of the evaluation using different analysis methods indicate:

- The columns below the 5th level were found to be deficient while almost all beams (with the exception of a few beams in the first story) in the structure were found to possess adequate capacity when using LSP.
- Almost all columns and beams were found to be deficient when using LDP, the only exceptions being the columns and beams at the top story level.
- The demand estimates using NSP were generally similar to the mean estimates of NDP.
- Mean column demands using NDP exceeded the acceptance criteria for life-safety and collapse prevention in most levels. The average beam demands were unacceptable in levels two through four only.

The most significant aspect of the above findings is that each procedure identifies a different weakness in the system. The results presented here are for a single building only. Several buildings were analyzed in a similar fashion and the observed inconsistencies were common to all systems. The following additional observations summarize the overall study:

- The linear static procedure generally results in the lowest demands which translates into a higher degree of acceptance (i.e., it is more likely to pass FEMA-273 acceptance criteria using LSP than any of the other methods).

- While both dynamic methods generally exceeded FEMA-273 acceptance criteria, the distribution of the demands were different (for example, the largest beam and column demands using LDP were in the lowest story levels while peak demands using NDP were in the mid-story elements).

- Current acceptance criteria are based on the maximum forces or deformations of a single element in a story level. While deformation demands are generally reflective of story drift demands, this is often not the case. Hence, the use of local maximum demands as the basis of design acceptance needs to be re-examined.

REFERENCES

FEMA-273 (1997) NEHRP Guidelines for Seismic Rehabilitation of Buildings. Building Seismic Safety Council, FEMA, Washington, D.C.

Kunnath, S.K. (1995). Enhancements to Program IDARC: Modeling Inelastic Behavior of Welded Connections in Steel Moment-Resisting Frames. Report No. NIST GCR 95-673, National Institute of Standards and Technology, Gaithersburg, Maryland.

SAC, Draft Report on Ground Motions (1997), available from http:// quiver. eerc. berkeley.edu: 8080/studies/system/draftreport.html.

Trifunac, M.D. and Brady, A.G. (1975). “A study of the duration of strong earthquake ground motion”. Bulletin of the Seismological Society of America, Vol. 65, 1975.