EVALUATION OF NONLINEAR STATIC PROCEDURES FOR SEISMIC DESIGN OF BUILDINGS

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ABSTRACT

This paper examined the effectiveness of nonlinear static procedures for seismic response analysis of buildings. Nonlinear static procedures are recommended by FEMA 273 document in assessing the seismic performance of buildings for a given earthquake hazard representation. Three nonlinear static procedures specified in FEMA 273 are evaluated for their ability to predict deformation demands in terms of inter-story drifts and potential failure mechanisms. Two steel and two reinforced concrete buildings were used to evaluate the procedures. Strong-motion records during the Northridge earthquake are available for these buildings. The study has shown that nonlinear static procedures are not effective in predicting inter-story drift demands compared to nonlinear dynamic procedures. Nonlinear static procedures were not able to capture yielding of columns in the upper levels of a building. This inability can be a significant source of concern in identifying local upper story failure mechanisms.

KEYWORDS: dynamic analysis; earthquake engineering; nonlinear static procedures; performance design; story drift.

1.0 INTRODUCTION

The American Society of Civil Engineers (ASCE) is in the process of producing an U.S. standard for seismic rehabilitation existing buildings. It is based on Guidelines for Seismic Rehabilitation of Buildings (FEMA 273) which was published in 1997 by the U.S. Federal Emergency Management Agency. FEMA 273 consists of three basic parts: (a) definition of performance objectives; (b) demand prediction using four alternative analysis procedures; and (c) acceptance criteria using force and/or deformation limits which are meant to satisfy the desired performance objective.

FEMA-273 suggests four different analytical methods to estimate seismic demands: (i) linear static procedure (LSP); (ii) linear dynamic procedure (LDP); (iii) nonlinear static procedure (NSP); and (iv) nonlinear dynamic procedure (NDP). Given the limitations of linear methods and the complexity of nonlinear time-history analyses, engineers favor NSP as the preferred method of analysis.

Following the analysis of a building, the safety and integrity of the structural system is assessed using acceptance criteria. For linear procedures acceptance criteria are based on demand-to-capacity ratios, and for nonlinear procedures, they are based on deformation demands.

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This paper examined the ability of the FEMA 273 nonlinear static procedures to predict deformation demands in terms of inter-story drift and potential failure mechanisms in the system.

1.1 Nonlinear Static Procedures for Seismic Demand Estimation

There are several procedures that can be adopted for conducting a nonlinear static analysis. While the fundamental procedure for the step-by-step analysis is essentially the same, the different procedures vary mostly in the form of lateral force distribution to be applied to the structural model in each step of the analysis. FEMA-273 recommends the following three procedures:

i) Inverted Triangular Pattern (FEMA-1)
A lateral load pattern represented by the following FEMA-273 equation:

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V$$

where: $F_x =$ lateral load at floor level $x$

- $w_{x,i} =$ weight at floor level $x,i$
- $h_{x,i} =$ height from base to floor level $x,i$
- $k = 1.0$ for $T < 0.5$ seconds;
- $k = 2.0$ for $T > 2.5$ seconds, with linear interpolation for intermediate values

- $V =$ total lateral load (base shear) to be applied to the building

This load pattern results in an inverted triangular distribution across the height of the building and is normally valid when more than 75% of the mass participates in the fundamental mode of vibration.

ii) Uniform Load Pattern (FEMA-2)
A uniform load pattern based on lateral forces that are proportional to the total mass at each floor level.

$$F_x = \frac{w_x V}{\sum_{i=1}^{n} w_i}$$

This pattern is expected to simulate story shears.

iii) Modal Load Pattern (FEMA-3)
A lateral load pattern proportional to the story inertia forces consistent with the story shear distribution calculated by a combination of modal responses.

Each of the above procedures will be evaluated using four sample structures: 6-story steel frame building; 13-story steel frame building, 7-story concrete building, and 20-story concrete building. These buildings have strong-motion records from the Northridge earthquake. The strong-motion records at the roof level were used to calibrate the building models.

1.2 Benchmark Response

Time-history analyses are performed on the four buildings, and the results are used as benchmark values to which the results of each of the nonlinear static analyses are compared. The target displacements are based on either the actual measured building response or a displacement consistent with the guidelines of FEMA-273. The following aspects of the building response behavior are examined:
1) The ability of each nonlinear static procedure to predict the interstory drift demands.
2) The ability of each nonlinear static procedure to identify the distribution of yielding (plastic hinge formation) in the structure leading to a local or global failure mechanism.

2.0 MODELING AND CALIBRATION OF BUILDINGS

2.1 6-Story Steel Building

This building, located in the Los Angeles area, was designed in 1976 based on the 1973 UBC requirements. The primary lateral load resisting system is moment frames around the perimeter of the building. Since the structural system is symmetrical, only one of the perimeter frames was considered in the evaluation. The interior frames were designed as gravity frames and consist of simple shear connections only. The plan view of the building and the elevation of a typical frame used in the analysis are shown in Figure 1.

The building was instrumented with a total of 13 strong-motion sensors at the ground, 2nd, 3rd and roof levels.

The building performed well in the Northridge earthquake with no visible signs of damage. Recorded data indicate an essentially elastic response. Minimal calibration was required to obtain a good match of the computed response with the observed roof response (Figure 2).

2.2 13-Story Steel Building

A 13-story steel moment-resisting frame building (13 floors above ground plus one-story basement) was designed based on the 1973 UBC and built in 1975. The layout of the perimeter frames and a typical elevation of one of these perimeter frames are shown in Figure 3. The overall building plan dimensions are 160 x 160 ft (48.7 x 48.7 m). The typical floor system consists of about 2.5 in (64 mm) of concrete fill over 3-inch (76-mm) steel decking.

The instrumentation was located in the basement and on the sixth and twelfth floors. Recorded accelerations at the basement indicate that the building experienced a PGA of 0.41g in the N-S direction and 0.32g in the E-W direction. Approximately 12% of the connections of the west perimeter (N-S direction) frame fractured during the earthquake. Connections of the remaining three sides fractured less than 6%.

Since the building was symmetrical in both directions, only one frame was analyzed. Gross section properties were used to compute the initial stiffness values. Each perimeter frame was assumed to carry 50% of the building mass in each direction.

The building response in the East-West direction is shown in Figure 4. This indicates that the building stiffness and mass modeling are reasonably accurate.

2.3 7-Story RC Building

A seven-story nonductile reinforced concrete building was designed and constructed during the mid-1960s in accordance with the 1964 Los Angeles City building code. The building sustained damage in both the 1971 San Fernando and the 1994 Northridge earthquake. The building suffered relatively minor damage in the 1971 event but was heavily damaged during the 1994 event.
This building has a rectangular plan (Figure 5) with overall dimensions of approximately 63 ft (19 m) in the north-south (transverse) direction and 150 ft (46 m) in the east-west (longitudinal) direction. The total height of the building is 66 ft (20 m). The floor system consists of reinforced concrete flat slabs. Perimeter spandrel beam-column frames resist primarily the lateral load in each direction. The interior slab-column frames are also expected to carry a significant portion of the lateral load.

The building was repaired after the 1971 event. During the 1994 event, damage was primarily confined to the longitudinal perimeter frames with the most severe damage between the fourth and fifth floors of the south perimeter frame in the form of shear failure of the columns immediately below the fifth floor spandrel beam. Many beam-column joints suffered minor to moderate shear cracks below the fifth floor level. Concrete spalling and hairline flexural cracks were observed in several spandrel beams. In the transverse direction, the damage was limited to minor flexural cracks in the beams in the end bay.

The two interior frames and the two exterior frames were considered to be identical for purposes of the modeling. The response of the calibrated building model to the input ground motion is shown in Figure 6. The comparison indicates that the resulting model is adequate for the analytical studies.

There are twenty stories above ground and a basement. The typical story height is 8.75 ft (2.7 m). The bottom two and top two levels have different heights as indicated in the figure. This building experienced a PGA of 0.33g at the basement level during the 1994 Northridge earthquake.

The gravity load resisting system of the building consists of concrete slab supported on concrete beams and columns. The primary lateral force resisting system consists of moment-resisting frames with strong shear walls in the basement only. Beams are provided between all column lines except between lines 2 and 3 for intermediate frames (also shown in the figure). Post-earthquake survey of the building indicated that the building suffered significant non-structural and content damage. However, there was no evidence of any significant structural damage at any level.

Two typical frames were considered in the modeling of the building. Eight frames with three continuous beams across all three spans and seven frames with the discontinuous beams were modeled separately. The interior slab between the discontinuous beams was assumed to remain elastic. A total of 19 column types and 19 beam types were used to construct the building model. A fictitious stiff column was added at the basement level to simulate the effect of the rigid walls at this level. Since the recorded ground motion was at the basement level, it was considered essential to model the basement level.

Using gross section inertia and uncracked concrete stiffness, a fundamental building period of 2.46 seconds was obtained. Since this value is in good agreement to the recorded data, no additional tuning of the model was required. Figure 8 shows a plot.
of the time-history response of the roof, which indicates that the building model is adequate.

3.0 EVALUATION OF NONLINEAR STATIC PROCEDURES FOR SEISMIC DEMAND ESTIMATION

3.1 Analysis of 6-Story Steel Building

As indicated previously, this building did not sustain damage during the Northridge earthquake. Instrument records and the calibrated response indicate that the structural response was essentially elastic. To examine the effectiveness of nonlinear static procedures to estimate seismic demands, particularly in the inelastic phase of the response, the recorded base motion was scaled uniformly to achieve a roof displacement in the yielded state of the system (the “target” displacement). This approach allowed a better comparison of the different methods of analyses.

The target roof displacement was determined according to FEMA-273. The computed target roof displacement was 18 inches (450 mm). The scale factor for the recorded base motion was established through trial-and-error until the computed roof displacement matched the target displacement of 18 inches.

The displacement profile at maximum roof displacement is shown in Figure 9. It is observed that all pushover procedures predict a similar profile. In comparison with nonlinear dynamic procedure, the displacements are over-predicted.

The drift profiles over the height of the building are shown in Figure 10. The drift at the first level is over-estimated by all nonlinear static procedures but under-estimated at all other levels except the second level. It should be noted that the maximum inter-story drift in the case of the time-history response could occur at different times.

A comparison of the hinge patterns at the target displacement is shown in Figure 11. The plots show the distribution of beam and column yielding for each of the analyses using different shape of lateral load distribution. The yielding patterns are consistent with the story drift profiles.

3.2 Analysis of 13-Story Steel Building

This building experienced widespread connection damage during the Northridge earthquake. Since this implies inelastic action, it was reasonable to select the recorded roof displacement as the target displacement for the comparative study.

Figure 12 compares the displacement profile of the building at the target displacement for the different nonlinear static procedures. Figure 13 compares the maximum inter-story drift along the building height.

While the displacement profiles for all methods look very similar, the predicted maximum inter-story drift varies considerably for the nonlinear static procedures when compared to the dynamic response indicating the presence of higher modes in the response that cannot be captured by the static methods.

The locations of plastic hinges predicted by the different analytical methods are shown in Figure 14. Since FEMA-1, FEMA-2 and FEMA-3 resulted in almost the same hinge configuration, only one of three patterns is shown.
3.3 Analysis of 7-Story Concrete Building

This building experienced inelastic behavior and significant structural damage during the Northridge earthquake. Therefore, it was not necessary to establish a projected target displacement for the response analyses. The actual displacement obtained from the nonlinear time-history analysis was used as the target displacement. The correlation of the time-history procedure to the observed response was presented in the earlier section.

The displacement profiles using the different analytical approaches are shown in Figure 15. The corresponding drift values are shown in Figure 16. The overall trends in both displacement and drift are similar. The nonlinear static procedures tend to underestimate drift in the upper levels. As seen in Figure 17, the plastic hinge formation in the upper levels is not estimated adequately by the nonlinear static procedures.

3.4 Analysis of 20-Story Concrete Building

This building did not experience damage during the Northridge earthquake. Instrument records and the calibrated response indicate that the structural response was essentially elastic. As was in the 6-story steel building (Section 3.1), the recorded base motion was scaled uniformly to achieve a roof displacement in the yielded state of the system. The displacement profiles at the target roof displacement are shown in Figure 18.

The drift profiles predicted by different procedures along the height of the building are compared in Figure 19. It is seen that all nonlinear static procedures underestimate the drift at almost all story level above the third floor. Figure 20 presents a summary of the plastic hinge patterns resulting from each of the different analytical procedures. The locations of plastic hinges are consistent with the inter-story drift profiles.

4.0 CONCLUSIONS

This paper examined the effectiveness of nonlinear static procedures for analysis of inelastic response of buildings. Specifically, the FEMA 273 procedures are evaluated to see whether nonlinear static procedures can predict deformation demands in terms of inter-story drift and potential failure mechanisms in the system.

The evaluation was carried out using four buildings for which instrumented data were available:

1) Six-Story Steel Moment-Frame Building,
2) Thirteen-Story Steel Moment-Resisting Frame Building,
3) Seven-Story Reinforced Concrete Moment Frame Building, and
4) Twenty-Story Concrete Moment Frame Building.

A frame model of each of the above buildings was first calibrated against observed instrument data. Then, each of the building models was analyzed using a detailed nonlinear time-history analysis followed by a series of nonlinear static pushover procedures. They were:

1) A lateral load pattern represented by an inverted triangular load (FEMA-1).
2) A uniform load pattern based on lateral forces that are proportional to the total mass at each floor level (FEMA-2).
3) A lateral load pattern proportional to the story inertia forces consistent with the
story shear distribution calculated by a combination of modal responses using a response spectrum analysis (FEMA-3)

The following conclusions are drawn:

1. Nonlinear static procedures are generally not effective in predicting inter-story drift demands compared to nonlinear dynamic procedures. Drifts are generally under-estimated at upper levels and sometimes over-estimated at lower levels.

2. The peak displacement profiles predicted by both nonlinear static and nonlinear dynamic procedures are in agreements. This suggests that the estimation of the displacement profile at the peak roof displacement by nonlinear static procedures is reasonable so long as inter-story drifts at the lower levels are reasonably estimated.

3. Nonlinear static methods did not capture yielding of columns at the upper levels. This inability can be a significant source of concern in identifying local upper story mechanisms.

REFERENCE

TYPICAL FRAMING FLOOR PLAN

Figure 1  Plan and Elevation of 6-Story Steel Building
Figure 2  Recorded vs. Computed Response of 6-Story Steel Building

Figure 3  Plan and Elevation of Perimeter Frame of 13-Story Steel Building
Figure 4  Recorded vs. Computed Response of 13 Story Steel Building

Figure 5  Typical Floor Plan and Elevation of 7-Story RC Building
Figure 6  Recorded vs. Computed Response of 7-Story RC Building

Figure 7  Plan and Elevation of 20-Story RC Building
Figure 8  Recorded vs. Computed Response of 20-Story RC Building

Figure 9  Displacement Profiles of 6 Story Steel Building

Figure 10  Inter-story Drift Profiles of 6 Story Steel Building
Figure 11  Hinging Patterns of 6-Story Steel Building at Target Displacement

Figure 12  Displacement Profiles of Different NSPs vs.NDP of 13-Story Steel Building
Figure 13 Inter-story Drift Profiles of 13-Story Steel Building

Figure 14 Hinging Patterns of 13-Story Steel Building
Figure 15 Displacement Profiles of 7-Story Concrete Building

Figure 16 Inter-story Drift Profiles of 7-Story Concrete Building

Figure 17 Hinging Patterns of 7-Story Concrete Building
Figure 18  Displacement Profiles of 20-Story Concrete Building

Figure 19  Inter-story Drift Profiles of 20-Story Concrete Building
Figure 20  Hinging Patterns of 20-Story Concrete Building

(a) Nonlinear Dynamic Procedure
(b) Nonlinear Static Procedure
FEMA 1, FEMA 2, FEMA 3

Figure 20  Hinging Patterns of 20-Story Concrete Building