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ANALYSIS PROCEDURES FOR PERFORMANCED BASED DESIGN

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SUMMARY

This paper evaluates the two nonlinear procedures used for Performance Based Design, the nonlinear static procedure (pushover) and the nonlinear dynamic procedure (time history). Two buildings damaged during the Northridge and Kobe earthquakes were used as case studies.

The evaluation showed that the pushover procedure could provide a reasonable estimate of maximum response provided the hysteretic damping was conservatively calculated. The pushover procedure is less intensive in terms of computer resources than the time history method but it does require extensive calculations to account for directional effects and mass eccentricities. The paper concludes that the earthquake engineering profession should be making efforts to move toward the time history procedure for performance based design.

INTRODUCTION

Prescriptive codes such as the UBC [1] provide structural design forces and detailing requirements. Provided these requirements are met, it is assumed that the seismic performance will be adequate. With performance based design, the actual damage to the building is evaluated at particular levels of ground shaking and the extent of damage is measured against prescribed acceptance criteria. This approach provides detailed information to the owner on expected damage for particular levels of earthquake, forming a logical and rational basis for decision making based on the cost effectiveness of particular structural systems.

To date, developments in implementing performance based design have been aimed at the strengthening of existing buildings [2, 3]. However, a logical extension will be to the design of new buildings. As this occurs, structural engineers will require more sophisticated analytical tools than are generally used for designs complying to current codes. This paper examines the applications of these analysis tools.

ANALYSIS PROCEDURES

There is a hierarchy of four levels of structural analysis appropriate for the evaluation of existing buildings [2]. Each higher level procedure provides a more accurate model of the actual performance of a building subjected to earthquake loads, but requires greater effort in terms of data preparation time and computational effort. The two most basic procedures, the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP), are mainly suitable for buildings which respond primarily in the elastic range. The Nonlinear Static Procedure (NSP) can evaluate buildings loaded beyond the elastic range but does not fully capture the dynamics of response, especially higher mode effects. The Nonlinear Dynamic Procedure (NDP) is the most complete form of analysis, modeling both dynamic effects and inelastic response. However, it is sensitive to modeling and ground motion assumptions.

The more advanced the form of analysis, the more realistic is the evaluation of demands on individual components that are loaded significantly beyond the elastic range. Because of this capability, the nonlinear procedures have less inherent safety margin and generally require less remedial work than the more simplified methods.

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The cost of obtaining the benefits of the most efficient and cost effective remedial work is the increased complexity of analysis and evaluation to ensure that the sensitivity of the procedure to modeling and ground motion is properly addressed.

Implementation of performance based design almost always requires assessment of performance beyond the elastic range and so procedures will tend to require the NSP and NDP as means of compliance. This paper evaluates these two procedures by examining two case studies.

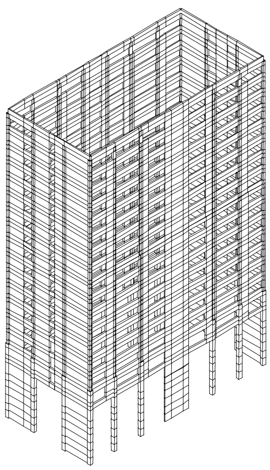
SHEAR WALL BUILDING

Building Description

The Californian example building is located in Santa Monica, California. The structure is 17 stories above ground level with a three storey basement, as shown in Figure 1-a. It is rectangular in plan, 38 m in the East West direction and 15 m in the North South direction. The structural system is a flat slab with reinforced concrete shear walls and columns.

The structure was damaged in the 1994 Northridge earthquake with cracking in wall piers, boundary elements and coupling beams. The damage was repairable.

An acceleration time history was recorded during the 1994 Northridge earthquake at Santa Monica City Hall, 0.9 miles from the site. The spectrum from this time history is shown in Figure 1-b.



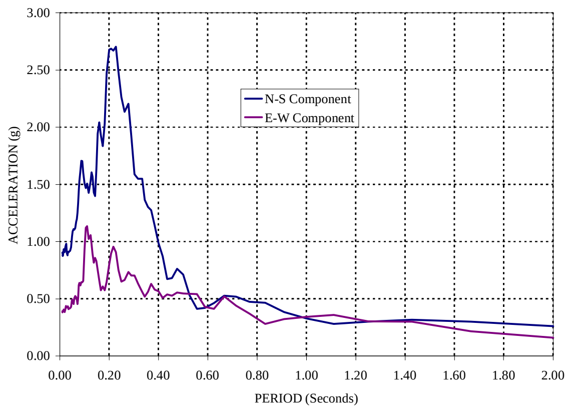


Figure 1

(a) Californian Example Structure **(b) 5% Damped Spectrum of Santa Monica City Hall Record**

Structural Model

A model of the building was developed using the ANSR-II computer program [4]. The shear walls and coupling beams were modelled as 4 node plane stress elements with a degrading strength and stiffness hysteresis function, as shown in Figure 2. The wall boundary elements and columns were modelled as flexural members with a moment-axial load interaction yield function.

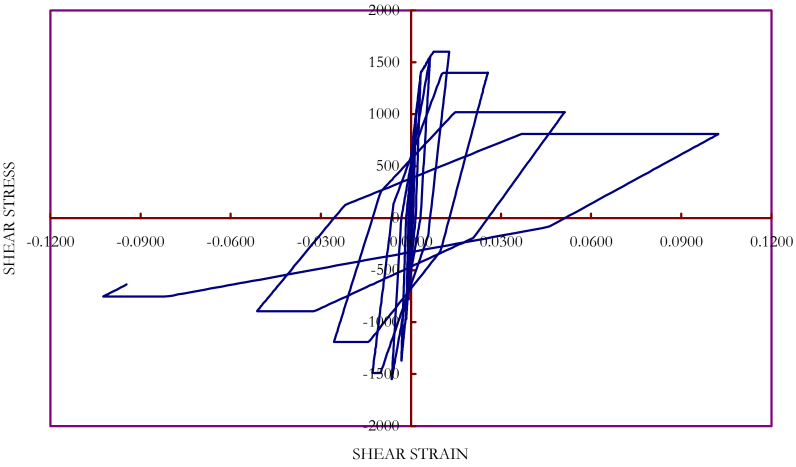


Figure 2 : Degradation of Wall and Coupling Beam Elements

Nonlinear Dynamic Procedure

The NDP procedure is based on a step by step integration of the equations of motion with the stiffness matrix updated as the member state changes. The model was evaluated using 30 seconds of the Santa Monica City Hall record at a time step of 0.01 seconds. Damage was evaluated by examining the maximum shear stresses in the wall and coupling beam elements and maximum plastic deformations in the flexural elements.

The analysis showed damage to the East elevation coupling beams and to the piers, coupling beams and columns on the North and South elevations. The extent of damage was not such as to pose a collapse possibility but was such as to preclude immediate occupancy. The predicted damage generally agreed in location and magnitude to that recorded after the earthquake although there were differences in specific damage locations. Figure 3 compares damage patterns for the South elevation.

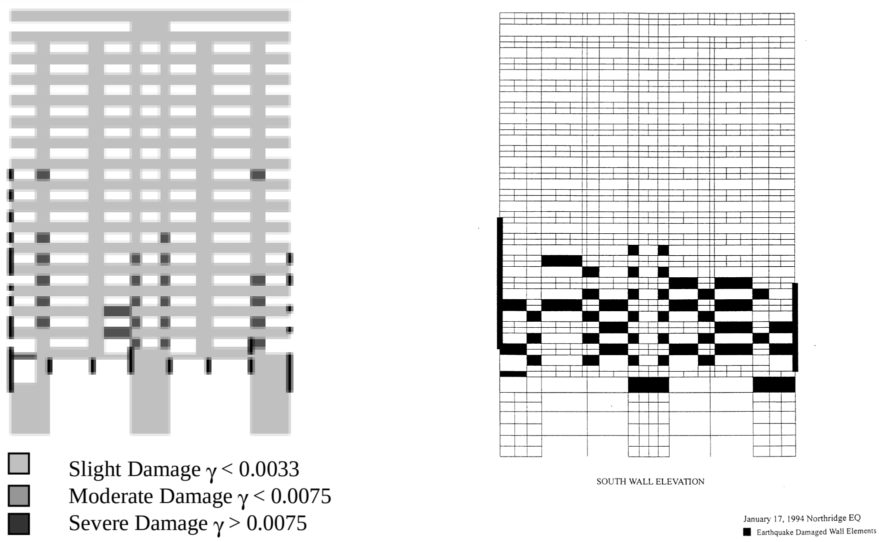


Figure 3: South Elevation, Predicted and Recorded Damage

(a) Predicted Damage **(b) Recorded Damage**

Once the model was validated using the Northridge earthquake motion, the performance of the building was evaluated using a ground motion equivalent to UBC seismic loads for Z = 0.4, soil profile type SB and near fault factors of 1.0. These parameters were selected for the purpose of comparison; the near fault factors would actually be greater than 1 for this site. The results from this analysis were used to compare with results from the NSP.

Nonlinear Static Procedure

The NSP was based on the methods set out in Reference [3]. This is based on the Capacity Spectrum Method (CSM), which uses the intersection of the capacity (pushover) curve and a reduced response spectrum to estimate maximum displacement.

The NSP was applied using the same structural model as was used for the NDP. A load vector based on the mode shapes was applied incrementally with each step equivalent to a base shear of 0.05% of the building weight. The analysis was continued until a mechanism formed, at which point equilibrium with applied loads could not be established and the analysis terminated.

The pushover curve generated was then used to develop the capacity spectrum assuming Structural Behaviour Types A, B, and C [3], representing increasing amounts of degradation. Examples of the Capacity Spectra for Structural Behaviour Type C are given in Figure 4.

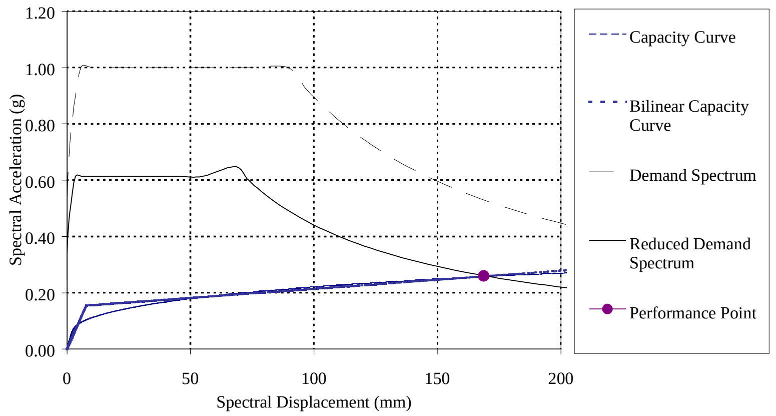


Figure 4 : Capacity Spectrum Based on UBC Record

Comparison of Procedures

Damage is related to maximum structural displacement. Table 1 compares the maximum roof displacements predicted by the NDP and NSP evaluations. The NSP consistently underestimates Northridge Earthquake displacements. For UBC loads, the type C hysteresis provides a reasonable estimate.

Table 1

	Northridge Earthquake		UBC Compatible Earthquake	
	E-W Direction	N-S Direction	E-W Direction	N-S Direction
Time History Analysis	98	184	162	248
Type A Structure	45	87	110	145
Type B Structure	51	127	134	171
Type C Structure	84	154	175	221

CONCRETE FRAME BUILDING

Building Description

The second example building, the Jeunesse Rokko building, was located in Kobe, Japan. The 9 storey structure, shown in Figure 5-a, suffered moderate damage in the 1995 Hyogoken Nanbu Earthquake and was subsequently demolished because it was considered too difficult to repair in a satisfactory manor. The building consists of 6 bays at 5.5m spacing in the longitudinal (North South) direction, and 1 bay of 6m spacing in the transverse (East West) direction.

An acceleration time history of the Hyogoken Nanbu earthquake was recorded at the Fukiai gas station. Figure 5-b shows the 5% damped response spectrum for the two horizontal components of the record.

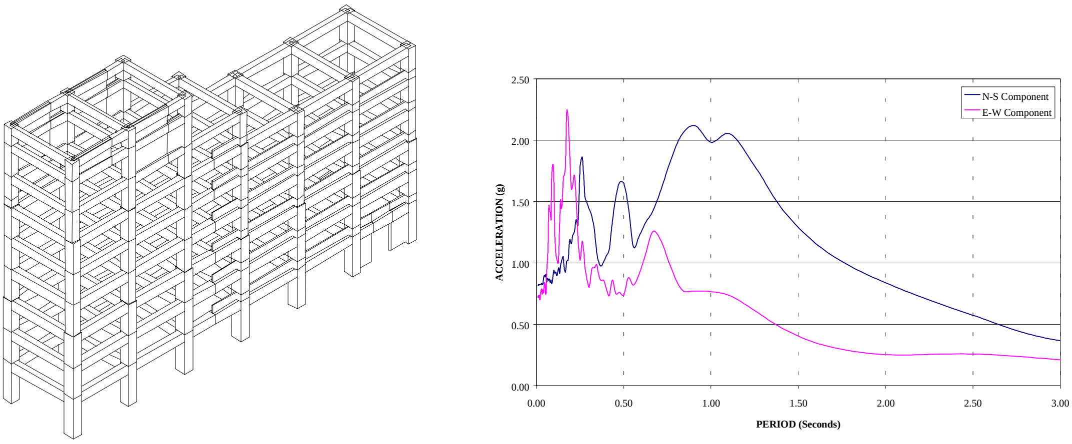


Figure 5

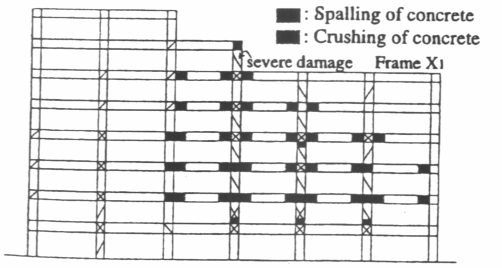
(a) Jeunesse Rokko Building **(b) 5% Damped Spectrum of Fukiai Gas Station Record**

Structural Model

The building was modelled in ANSRII using standard frame elements with bilinear hysteresis relationships as described above The structure had a number of non-structural walls and spandrels. These elements sustained significant damage during the Kobe earthquake, indicating that they did influence the response of the structure. Because of this, they were included in the structural model using the degrading wall element described earlier.

Nonlinear Dynamic Procedure

The model was subjected to 20 seconds of the Fukuai earthquake record at a timestep of 0.02 seconds. Patterns and magnitudes of plastic rotations predicted by the analysis were compared to those observed in the actual structure. Damage to spandrel and wall elements was also correlated with observed damage.



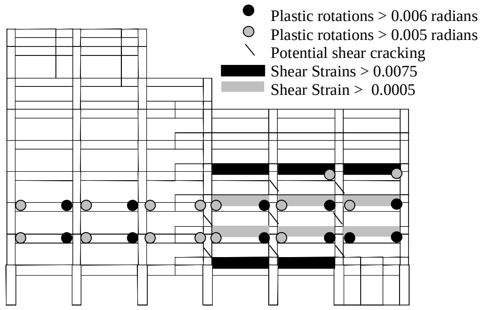


Figure 6: West Elevation, Recorded and Predicted Damage

(a) Recorded Damage **(b) Predicted Damage**

Figure 6 shows damage reported by Matsumori, Taizo and Otani [6] and damage predicted by the NDP for the western most frame. The analysis generally showed good agreement with actual damage, although the magnitude of predicted damage was considerably less than what occurred. Incomplete data used in preparation of the model is likely to be a major cause of these discrepencies.

Nonlinear Static Procedure

The CSM described above was used to relate the capacity curve of the Jeunesse Rokko building in the North South direction to the N30W component of the Fukiai acceleration record and for UBC motion. The results assuming a Type A structure, for UBC loads are illustrated in Figure 7.

Table 2 compares the results of the NSP and NDP analyses for the Jeunesse Rokko building. To allow direct comparison between the two methods, NDP analyses were conducted for each earthquake direction separately. As for the Californian example, the NSP tended to underestimate the response of the Jeunesse Rokko Building, even if a Type C structure was assumed.

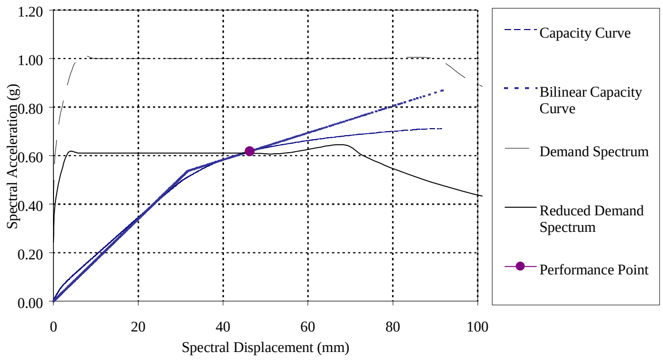


Figure 7 : Capacity Spectrum Based on UBC Record

Table 2

	Hygoken Nanbu Earthquake		UBC Compatible Earthquake	
	E-W Direction	N-S Direction	E-W Direction	N-S Direction
Time History Analysis	166	99	132	77
Type A Structure	67	50	77	45
Type B Structure	76	>92*	83	52
Type C Structure	143	>92*	100	77

*Exceeded predicted collapse displacement

EVALUATION OF ANALYSIS PROCEDURES

Assessment of Performance

The structural model of the building is the same for both the NSP and NDP. The difference is in the load functions and evaluation of performance. A major component of the NSP is the incorporation of the hysteretic damping provided by yielding structural elements. For example, for the Santa Monica building under UBC loads in the E-W direction, the added damping reduces the displacement from 300 mm to 175 mm (compared to 162 mm for the NDP). It is obvious that an accurate estimation of damping is important for an accurate estimate of performance.

The evaluations performed here suggest that the procedure from [3] tends to over-estimate the amount of hysteretic damping. Figure 8 shows the overall hysteresis for the Santa Monica building in the E-W direction, generated by applying a cyclic displacement at roof level. This hysteresis would be classified as Structural Behaviour Type A or B as the structural elements have similar performance to a new building. However, this classification would under-estimate building response.

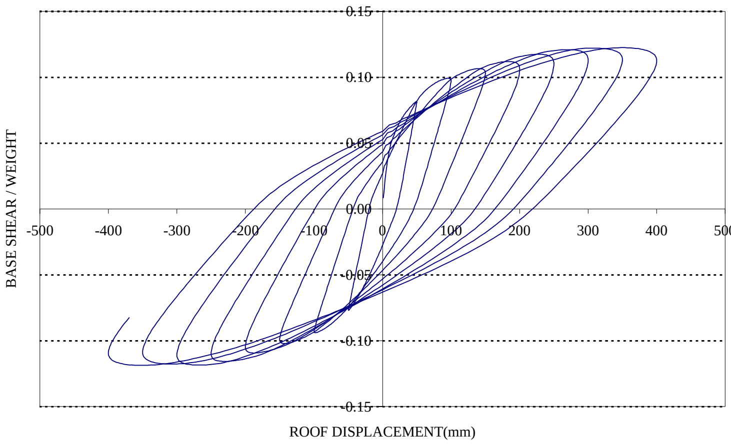


Figure 8 : Santa Monica Building Hysteresis

Level of Effort

As noted above, the structural model is the same for both the NSP and the NDP. The difference between the two procedures is in the specification of loads and in the evaluation of performance. For these evaluations, the NDP was more intensive in terms of computer time but the NSP required a lot more effort in evaluating results.

The NDP can be implemented by enveloping the results of multiple analyses (for example, varying earthquake records, earthquake orientation and eccentricity of the mass). This can be performed in a single, automated procedure. The NDP also allows earthquake components to be applied concurrently.

The NSP cannot simply be automated as a complete procedure although individual steps can be automated. For example, for each mass eccentricity and each axis, a separate eigensolution is required to define the load vector. Each capacity curve must then be solved to obtain the performance level. If the pushover curve is developed incrementally using a linear elastic program then the process is very time consuming.

If an automated procedure is used to generate the pushover curve, using a nonlinear analysis program such as ANSR-II, a further problem arises because the required performance level is not known at the time the pushover curve is generated. Unless a complete set of results is saved at every point on the pushover curve, the analysis must be repeated to obtain detailed results at the performance point.

Impediments restricting adoption of the NDP include the complexity, the sensitivity to modeling and ground motion assumptions. However, compared to the NSP, the only added complexity is in considering the changing dynamic characteristics of the structure as they influence response. The sensitivity to ground motion can be included in the evaluation by including multiple time history files. The available database of recorded ground motions is such that appropriate motions for most soil conditions and fault proximity can be obtained.

Advances in computer hardware favours procedures which are based on automated procedures, such as the NDP, rather than procedures which require extensive manual intervention, such as the NSP. Figure 9 shows how the computer time required to evaluate a 35 story perimeter frame building has reduced over the last decade. Such an improvement makes it practical to evaluate multiple earthquake time history analyses on design office hardware.

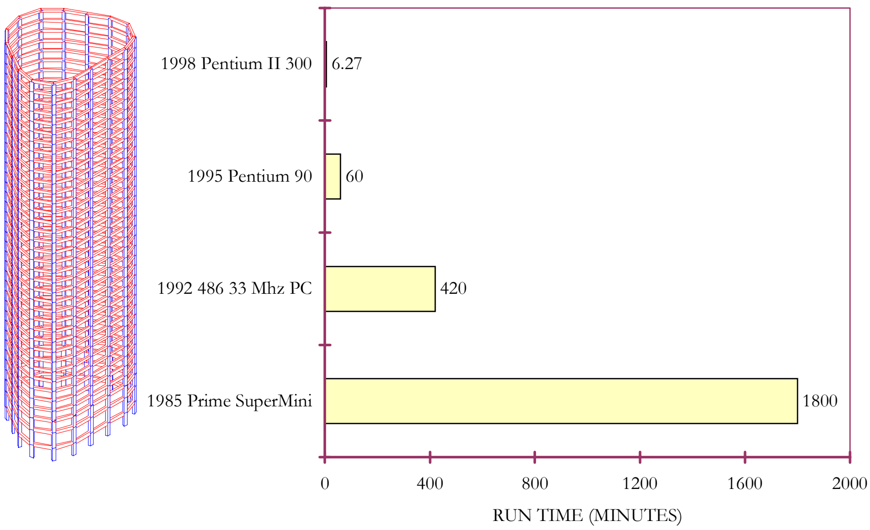


Figure 9 : Computer Resources

CONCLUSIONS

The comparison of the NSP and NDP has shown that the static procedure can provide a reasonable estimate of the performance of buildings under severe earthquake loads. However, the procedures for calculating equivalent viscous damping tend to estimate at the upper end of the range and so should be applied conservatively.

Although NSP is the procedure currently recommended for performance based design, we consider that the earthquake engineering profession should be making efforts to move towards the NDP. This more accurately accounts for the dynamic characteristics of the building and requires less effort, although more demanding on computer resources.

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