Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1097

EVALUATION OF PERFORMANCE-BASED ANALYSIS AND DESIGN METHODS FOR ASYMMETRICAL SHEAR WALL BUILDINGS

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ABSTRACT

The current building codes for seismic design specify minimum levels of strength and stiffness, and give details for elements based on capacity design approach. While these prescriptive force-based design methods intend to provide a certain levels of performance under one hazard level, the actual performance of the building is not evaluated as part of the code process. Therefore, the buildings designed based on these codes are bounded by a structural limit and their performance can be better or worse than the minimum anticipated by the code. Performance-based design (PBD) methods, on the other hand, explicitly evaluate the building performance subjected to a certain hazard. Using a nonlinear static analysis (pushover analysis) or a nonlinear dynamic analysis (time-history analysis), PBD allows the design of buildings with irregular structural systems that do not satisfy the regulations of the current prescriptive codes. Several PBD approaches exist, however, for the time being, there is no agreement in the scientific and engineering communities as to the most appropriate method. This paper aims to examine the available pushover analysis method for asymmetrical shear wall buildings with a torsional fundamental mode. Pushover analysis is not directly applicable to buildings which have their predominant displacement mode as torsion. Forty-storey building located in the Canadian city of Montreal is considered and analyzed using both pushover and time-history analysis for Life Safety performance level. The effective cracked stiffness and the nonlinear material modeling of the structural elements are based on sectional analysis taking into account the confinement provided by the transverse steel reinforcement, while the acceptance criteria and the deformation capacities of these components are evaluated using ASCE 41-06 and ATC 40 documents. Nonlinear computer program PERFORM-3D with inelastic fiber section for shear wall elements is used to model and analyze the structures. Moreover, a time-history analysis is performed to evaluate the suitability of the pushover analysis in estimating the seismic response. It is shown that torsional and higher mode effects have a significant influence on the behavior of asymmetrical shear wall buildings with torsional first mode. Unless these effects are addressed and resolved, the nonlinear time-history analysis should be used as the PBD method for asymmetrical shear wall buildings.

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Introduction

Current prescriptive force-based methods are unable to properly assess the performance of structures for multi-level performance objectives. Structural displacements, which can be directly related to damage potential through material strains (structural damage) and drifts (non-structural damage), are checked using coarse and unreliable methods at the end of the design process. At best, this provides designs that satisfy damage-control criteria, but with widely variable risk levels (Priestley et al. 2007).

Performance-based design (PBD) is a term widely popular and used by the structural and seismic research community, and is being accepted as a rational approach for design of important and iconic structures. The evolution of international standards and codes and the complexity of projects across almost all sectors point towards more rigorous assessment of the lateral load resisting and gravity systems. The required structural assessment of the structural systems for practically all types of buildings and especially tall and iconic structures will very soon require nonlinear static (pushover analysis) and nonlinear dynamic (time-history) analyses. These analyses methodologies provide better tools towards the so called performance based design (PBD) method. In the traditional force-based design method only one hazard level is taken into account to design structures without any real consideration of risk of damage. In order to make the building performance transparent to the buildings owners, different seismic hazard levels should be accounted for. Moreover, the structure is designed to achieve, rather than be bounded by, a structural limit state under a specified hazard level. The relationship between the hazard level and the performance level of a building is demonstrated in Fig. 1. By using the PBD method one can predict how the structure behaves after different hazard levels. The performance levels were set by different standards or reports (ASCE 41-06, SEAOC 1995) as: "Fully Operational" (FO), "Immediate Occupancy" (IO), "Life Safety" (LS), or "Collapse Prevention" (CP) as shown in Fig. 1. Thus, using the PBD methodology, building structures can be designed with a realistic understanding of the risk of life, occupancy and economic loss that may result during future events. The PBD process is performed by creating a building analytical model followed by simulating the performance of the design for various seismic events. The design of the structure can be adjusted until the projected risks of loss are deemed acceptable, given the cost of achieving this performance.

One advantage of PBD over conventional methods is that the strict new code rules limiting architectural expression (e.g., discontinuity in capacity – weak storey) could be by-passed if the performance-based design is utilized. Other advantages of PBD over conventional methods can be postulated as:

- Multi level seismic hazards are considered with an emphasis on the transparency of performance objective.
- Building performance is guaranteed through limited inelastic deformation in addition to strength and ductility.
- Inelastic deformations are rationally obtained.
- The building meets the prescribed performance objectives reliably with accepted criteria.

The pushover analysis purpose is to assess the performance of a structural system by estimating its strength and deformation demands in design earthquakes and comparing these demands to estimated capacities at the required performance levels (Krawinkler and Seneviratna 1998).

		Building Performance Level			
		Fully Operationa l	Immediate Occupancy	Life Safety	Collapse Preventio n
Earthquake Hazard Level	Frequent (50% in 50 years)	O	X	X	X
	Occasiona l (20% in 50 years)	O	O	X	X
	Rare (10% in 50 years)	O	O	O	X
	Very Rare (2% in 50 years)	O	O	O	O

Basic Objective

Essential

Safety Critical Objective

Figure 1. Performance objective – relationship between the hazard and the performance levels (O = acceptable performance, X = unacceptable performance) (SEAOC 1995)

The evaluation is based on an assessment of important performance parameters, including global drift, interstory drift, and element ductility demands and corresponding force demands for protected elements and actions. It has been shown that this method gives good estimates of the maximum seismic response for structures with dominant fundamental translational mode shape (Fajfar and Fischinger 1988). One of the unresolved pushover issues that need to be addressed is the incorporation of torsional effects. Torsional effects are not incorporated properly in the pushover method since this method is based on a single-degree-of-freedom (SDOF) displacement demand (Krawinkler and Seneviratna, 1998). This study aims to examine the pushover analysis results of a multi-storey building with a torsional fundamental mode.

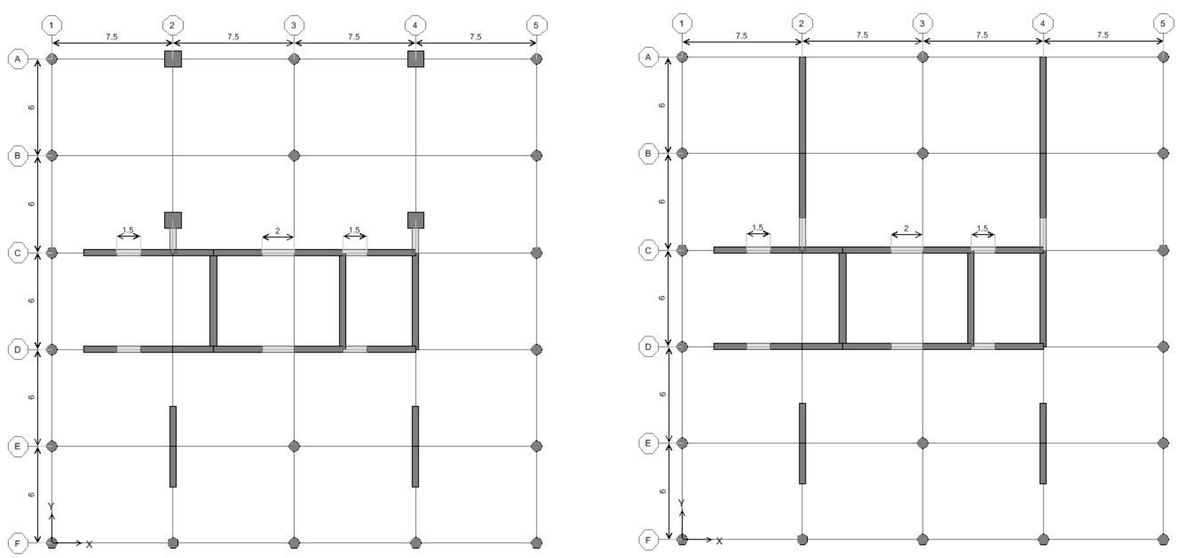
Case Study

Fig. 2 shows the ground floor and the typical structural floor plans of the building under study. This building is a 40-storey reinforced concrete residential building located in the Canadian city of Montreal and founded on soft rock. The overall dimensions of the building are: East-West (E-W) = 30 m (4 bays of 7.5 m), North-South (N-S) = 30 m (5 bays of 6.0 m), and the height is 120 m (3.0 m of floor-to-floor height). The concrete strength is $f_c' = 40\,\text{MPa}$ and steel reinforcement yield strength is $f_y = 420\,\text{MPa}$. Each floor consists of a 250 mm thick flat plate. The walls thicknesses are 400 mm, 350 mm, and 300 mm for storeys 0-15, 16-30, and 31-40, respectively, and the coupling beams sizes are 400 mm (width) x 425 mm (depth), 350x425 mm, and 300x425 mm, for storeys 0-15, 16-30, and 31-40, respectively. The diameter of the reinforced concrete (RC) circular columns is 650 mm. At ground floor level, the north walls of the N-S direction discontinue and are replaced by four 1000x1000 mm RC columns (Fig. 2a). Thus, the building is considered irregular type 6 (discontinuity in capacity – weak storey) and is not permitted according to NBCC (2005). The gravity loads applied to each floor consist of the concrete self weight, superimposed dead load of 1.2 kPa, and live load of 1.9 kPa. The mass of the structure is based on 100% of the self weight, 80% of the superimposed dead load, and 30 % of the live load and is lumped at each floor level, as it is assumed that the floors of the building act as rigid diaphragms. It should be also noted that the mass at the 20th floor was assigned as three times the typical floor mass.

The computer program PERFORM-3D (CSI 2008) is used to conduct the nonlinear static (pushover) and time-history dynamic analyses. The first mode of the structure is torsional and the period is 6.41 Sec. Modal properties of the first three modes are given in Table 1. Note that these properties are based on the effective stiffness values given in Table 2.

Table 1. Dynamic Characteristics of the structure

Mode	**1**	**2**	**3**
Period (sec)	6.41	4.86	4.6
Principal direction	Torsional	Translational 25.7° from NS	Translational 24.2° from EW
Mass participation factor	-	0.53 (NS) and 0.12 (EW)	0.55 (EW) and 0.19 (NS)



(a) (b)

Figure 2. (a) Ground floor and (b) typical floor plan of the building

Material Modeling

One of the main tasks in the performance-based seismic method is to define material and structural component nonlinear force-deformation relationships taking into account the effects of stiffness and strength degradation. In this study, the inelastic behavior of the structural components is modeled using the techniques for 3D inelastic seismic analysis provided by the structural analysis program PERFORM-3D (CSI 2008).

Shear Walls

The shear wall elements are modeled using a shear wall component that is defined by separately specifying two properties: shear and axial-bending. The shear in the walls is modeled using an elastic material. A strength capacity is assigned to verify that the shear forces are smaller than the capacity of the wall at the demand displacement. This is a force-controlled behavior with no allowed inelastic deformations. The axial-bending behavior of a shear wall is modeled using an inelastic fiber cross section. Each wall is defined using eight RC fibers. The axial-bending behavior of each fiber is controlled by the confined concrete stress-strain model and the longitudinal reinforcement ratio.

Table 2. Effective stiffness values

Component	**Shear rigidity***	**Flexural rigidity**
Walls		
In-plane	$0.4E_cA_g$	$0.5E_cI_g$ *
Out-of-plane	$0.4E_cA_g$	$0.05E_cI_g$
Coupling beams	$0.4E_cA_g$	$0.29\text{-}0.43E_cI_g$ **
Columns	$0.4E_cA_g$	$0.7E_cI_g$ **

* Based on ASCE 41-06 (2007) and ATC-40 (1996).
** Derived from analytical moment-curvature curves (MNPHI 2001).

The concrete compressive behavior confined by transverse steel reinforcement is modeled using Legeron and Paultre confinement model (2003) (simplified to a trilinear stress-strain relationship in order to be used in PERFORM-3D). For the calculated confined concrete strength, f'_{cc}, the ultimate confined axial strain is derived based on the following equation (Paulay and Priestley 1992, Priestley et al. 2007):

$$\varepsilon_{cu} = 0.004 + 1.4\frac{\rho_v f_{yh}\varepsilon_{su}}{f'_{cc}} \tag{1}$$

where ρ_v is the volumetric transverse reinforcement ratio, f_{yh} is the transverse reinforcement yield strength (=420MPa), ε_{su} is the steel strain at maximum tensile stress (from the stress-strain steel curve taken as 0.08). Tension strength is neglected in the analysis. The cyclic behavior of the compressive confined concrete is modeled by introducing energy degradation factors to fit the experimental behavior and commonly used analytical models (e.g., Mander et al. 1988, Konstantinidis et al. 2007). The stress-strain behavior of the longitudinal steel reinforcement is defined by an elastic perfectly plastic (EPP) curve with modulus of elasticity E_s=200000 MPa, f_y=420 MPa, and ultimate steel strain of 0.08. The Bauschinger effect of the reinforcement cyclic behavior is taken into account by using energy degradation factors. It should be noted that according to FEMA 440A the influence of the energy degradation is not a determining factor for a nonlinear response of an RC component. The wall component acceptance criteria for different performance levels are taken from ASCE 41-06 standard while the concrete compressive strain capacities are based on Priestley et al. (2007).

Columns and Coupling Beams

In PERFORM-3D there are a number of ways to model inelastic RC beams and columns behavior. At one extreme are finite element models using fiber sections and at the other extreme are chord rotation models that consider the member as a whole, and essentially require specifying the relationship between end moment and end rotation. In this study, the latter model is used to simulate the inelastic behavior of coupling beams and columns. This model consists of a linearly elastic element with one equivalent inelastic rotational spring at each end. The inelastic deformations of the modeled member are lumped at these two inelastic springs. One advantage for using this model is that ASCE 41-06 specifies its end rotation capacities. Moment-curvature behavior is derived from a sectional analysis program MNPHi (2001) by taking into account concrete core confinement (Legeron and Paultre 2003 model) and tension stiffening (Vecchio and Collins 1986). The flexural hysteretic behavior of the elements is based on the modified Takeda model proposed by Otani and Sozen (1972). This is implemented in PERFORM-3D by scaling energy degradation factors to approximate the modified Takeda model. Axial force – biaxial bending interaction is taken into account in the RC column modeling. The beams and columns acceptance criteria for different performance levels are based on ASCE 41-06.

Performance Assessment

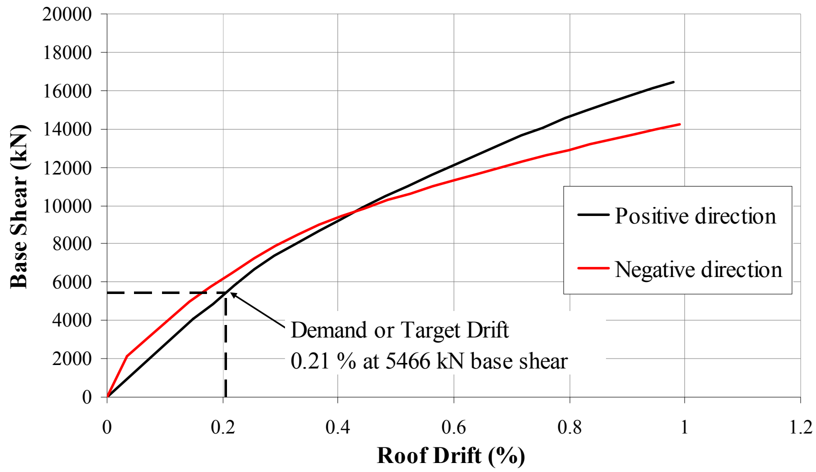
Pushover Analysis

One of the challenging tasks in pushover static analysis is the determination of the lateral load distribution over the structure height. The load should be in proportion to the distribution of inertia forces in the plane of each floor diaphragm. The distribution of the inertia forces will vary with the severity of the earthquake and is not only dependent on mass distribution but also on the effective stiffness which is a time-dependent variable during a time-history seismic event. The extremes of this distribution depend on the earthquake intensity and the degree of nonlinear response of the structure. In the current study, the commonly used load cases based on the first mode shape of each principle direction (mode shapes 2 and 3) are used. These load cases are applied twice: in the positive and negative directions of the first translational mode shapes.

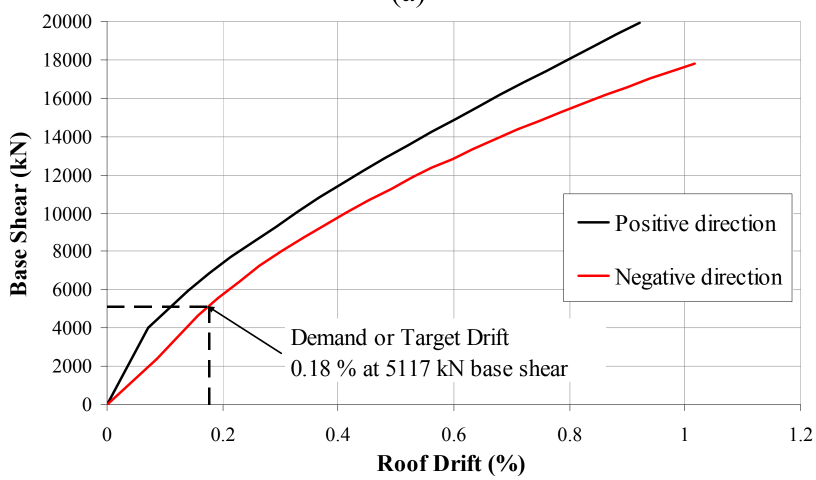
Fig. 3 shows the positive and negative base shear vs. roof drift curves of each principle direction. Fig. 3 also shows the target roof drift at each direction. This drift is derived based on the capacity spectrum method (ATC-40) using family of response spectra constructed from the Montreal response spectrum (NBCC 2005) for different damping ratios. Table 3 presents the target drift and its corresponding base shear for each direction. At Life Safety (LS) performance level, Fig. 4a shows the usage ratio (= ratio between the demand and the limit state capacity) of the structural components at the target drift level. It is shown in the figure that none of the structural components reached its capacity limit (i.e., usage ratio equal to 1.0).

Time-History Analysis

Eight simulated ground motion time histories were used for the inelastic time-history dynamic analysis (Boivin 2006, Boivin and Paultre 2010). Table 4 provides their characteristics.



(a)



(b)

Figure 3. Base shear vs. drift of (a) the positive and the negative mode 2-based (NS) lateral load case, and (b) the positive and the negative mode 3-based (EW) lateral load case

Table 3. Pushover and dynamic analyses results

Analysis	**Roof Drift (%)**	**Base Shear (kN)**	**Capacity reached?**
Pushover - NS	0.21	5466	No
Pushover - EW	0.18	5117	No
Dynamic analysis (Mean results)	0.09	-	No

The records are based on the simulated ground-motion time histories for eastern Canadian locations produced by Atkinson (1999) compatible with the NBCC 2005 uniform hazard spectra. The Montreal uniform hazard spectrum can be matched using the simulated records for eastern Canadian earthquakes of M=6.0 at R=30 km (short periods events) and M=7.0 at R=70 km (long period events) and by scaling the acceleration values with 0.85 and 0.90 factors, respectively (Atkinson 1999). The analysis is performed using step-by-step integration through time, using the constant average acceleration method (also known as the trapezoidal rule) and by using 5% modal damping ratio. Table 3 presents the mean value of the maximum roof drift from all eight ground motion time-histories. Fig. 4b shows the mean usage ratio of the structural components at the end of the analysis.

Table 4. Characteristics of the NBCC 2005 compatible time-history records for Montreal

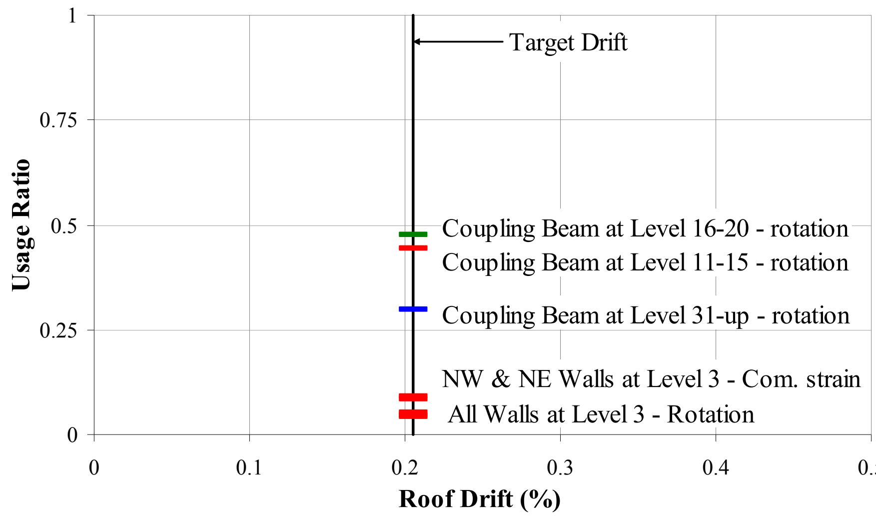
Record	**M***	**R* (km)**	**Duration (sec)**	**Scaling Factor**	**Tuned PGA (g)**
MTL1-M6	6.0	30	8.88	0.85	0.37
MTL2-M6					0.44
MTL3-M6					0.4
MTL4-M6					0.37
MTL1-M7	7.0	70	24.07	0.90	0.27
MTL2-M7					0.26
MTL3-M7					0.31
MTL4-M7					0.26

* M = Moment magnitude; R=epicentral distance

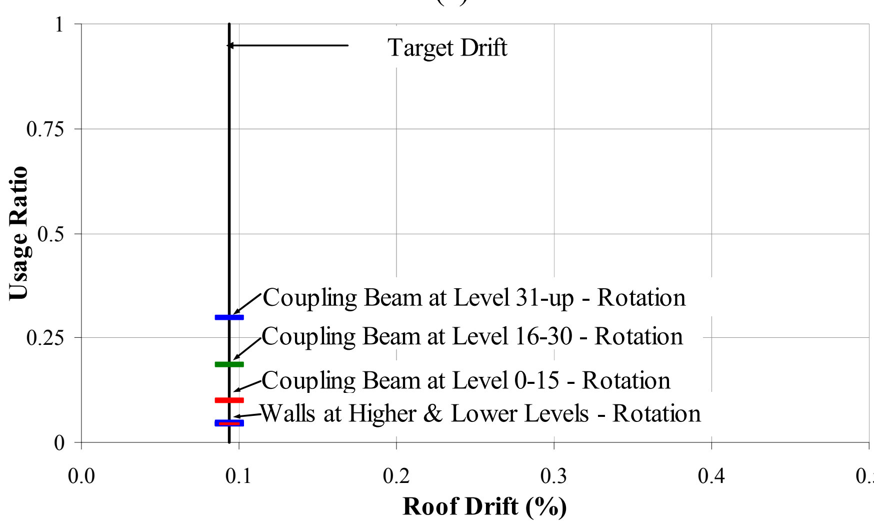
The figure shows that, similar to the pushover analysis, none of the structural components reached its capacity limit. However, the figure also shows that for larger drifts the limit state capacity of the coupling beams is reached initially at higher storey levels (level 31 and above). This result is inconsistent with the pushover results (Fig. 4a), which show that the coupling beams reach their limit state capacity at first at the mid-height levels and the shear walls limit state capacity is reached at the lower levels (level 3 and below). These behavior discrepancies can be attributed to the torsional as well as the higher mode effects. The dynamic time-history analysis also indicates that about 60% of the dissipated energy was obtained at higher storey levels (by coupling beams at level 31 and above) while about 10% of the dissipated energy was obtained at lower storey levels (by coupling beams at level 15 and below). Thus, for a structure with components having disproportionate hysteretic energy dissipation capacities, such as walls and coupling beams in this case, nonlinear static procedures, which use stiffness degradation in a global sense, are not suitable for component design.

Conclusions

This paper discusses the seismic analysis and design of a 40-storey building in a high seismic zone of Montreal. The building represents a typical /North American residential building with lateral-load resisting system being discontinued at podium levels resulting in a ‘soft storey’. In the height of the soft storey, the coupled shear-walls are supported on columns to accommodate lobbies and vast circulation spaces. The paper discusses the inapplicability of the quasi-static provisions of NBCC 2005 as the ‘soft storey’ leads to building lateral displacement profiles being inconsistent with the fundamental mode displacement patterns. The building is analyzed using the pushover analysis of ATC40 and the nonlinear time-history analysis. The former uses the response spectra of NBCC2005 for Montreal and the latter is based on area-specific time-histories scaled linearly to the above-referred response spectra. The limitations of the static nonlinear procedure for a building with predominant torsional modes are discussed along with the higher mode effects that the nonlinear static procedure fails to capture.



(a)



(b)

Figure 4. Usage ratio at target drift for Life Safety performance level (a) Pushover analysis (NS direction) and (b) Dynamic time-history analysis

Besides, for a structure with components having disproportionate hysteretic energy dissipation capacities, such as walls and coupling beams in this case, nonlinear static procedures, which use stiffness degradation in a global sense, are not suitable for component design.

Acknowledgments

The authors gratefully acknowledge Dr. Tibor Kokai for initiating the concept of this article and for his encouragement at the preliminary phase of this study. The authors also gratefully acknowledge Professor Patrick Paultre for kindly providing the computer program MNPHi (2001) for the study purpose. The authors also wish to thank Professor Patrick Paultre and Mr. Yannick Boivin for kindly providing the simulated ground motion time histories.

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