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EVALUATION OF THE LEVEL OF SEISMIC PROTECTION OF AN 85-STOREY CONCRETE SHEAR WALL BUILDING

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ABSTRACT

This paper presents the results of an investigation into the seismic level of protection afforded to reinforced concrete shear wall systems. The vulnerability and damage potential of an 85-storey building consisting of coupled and non-coupled shear walls as lateral force resisting systems, is evaluated. The structure is located in a moderate seismic zone, and designed in accordance with the ACI-318. Elastic analysis is performed using three-dimensional shell element models for lateral loading considering the effects of torsion. Element design specifications are used to create moment-curvature relation to describe the members (beam and wall) deformation characteristics. These characteristics are incorporated into the nonlinear pushover analysis. The Target Displacement Method (FEMA-356) is employed. A modal pushover analysis is used to consider the effect of higher modes. The level of protection investigation illustrates that the coupled and non-coupled shear wall systems exhibit excellent performance following excitations. Maximum inter-storey drift and element damage levels are within the acceptable limits for life-safe performance.

Introduction

The seismic level of protection of building structures is concerned with the performance of structures during seismic shaking and has been the source of major changes to code provisions on earthquake resistant design. Although current building codes govern seismic design of tall buildings, the provisions do not address many critical behaviors of these structures. Many of these tall buildings use high-performance materials and framing systems that are not commonly used for building construction or that fall outside the height limits of current buildingcodes. In many cases, provisions of governing building codes are found to be restrictive and force the designer to do a more detailed analysis.

One of the important classes of structural systems commonly used to resist lateral loads, due to both wind and earthquakes, in tall (taller than 20 stories or 70 meters) building structures are reinforced concrete shear walls. Lateral deflections and inter-storey drifts are easy to control

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because of the high in-plane stiffness of such systems; it is also relatively easy to provide adequate reinforcement to achieve the necessary strength. Shear wall systems may be composed of parallel cantilever-type walls, with or without coupling beams or slabs at each floor; another common shear wall configuration is the core wall assembly which also functions to contain the elevator shafts, stairwells, and service areas.

Studies over the past few decades have provided extensive information on the response of reinforced concrete shear wall systems and system components (e.g., Paulay and Priestley 1992; Papanikolaou et al. 1992; Fintel 1995). These investigations concentrated on studying the effects of detailing of specific lateral load resisting elements. The designs of reinforced concrete coupled shear wall systems have also been studied (e.g., Chaallal et al. 1996; Paulay and Priestley 1992; Stonehouse, Heidelbrecht and Kianoush 1999). Performance investigations involving shear wall systems have been initiated in the United States (Munshi and Ghosh 1998).

There has been limited investigation on the performance of tall shear wall systems designed according to governing code when these are subjected to the kinds of seismic ground motions expected in moderate seismic zones. The performance expectations, which are required for both functionality and life-safety are related to structural response parameters such as deflection, inter-storey drift, and damage.

This paper is concerned with the performance of an 85-storey reinforced concrete building in which the lateral load is resisted entirely by central core wall assembly and shear walls linked by coupling beams and slabs in both orthogonal directions.

The behaviour of the orthogonal shear wall systems is determined using static pushover analysis. Performance is evaluated by examining response parameters such as inter-storey drift and element curvatures (wall and coupling beam) in relation to the performance criteria recommended by the FEMA 356.

Building configuration and loading

Description of Building Configuration

The 85-storey structure used for this investigation is similar in configuration to an actual building designed in a moderate seismic zone. The plan of the structural element of half of a typical floor of this building is shown in Fig. 1. The *x* and *y* directions are referred to as long and short direction in plan, In the long direction the core walls and shear walls, acting as vertical cantilevers, resist lateral loads. In the short direction the core walls and the shear walls are coupled by connecting beams. The storey height is about 3.3 m, resulting in a total building height of 300 m. The dimensions of the wall cross sections change over the height of the building.

All other dimensions remain constant throughout. The material properties for the walls and coupling beams are constant throughout the height of the structure: (i) reinforcing steel yield strength, f_y=460 MPa; (ii) reinforcing steel modulus. E_s=200000 MPa; *(iii)* concrete compressive strength, f'_c=85 MPa; and *(iv)* concrete elastic modulus, E_c=49000 MPa.

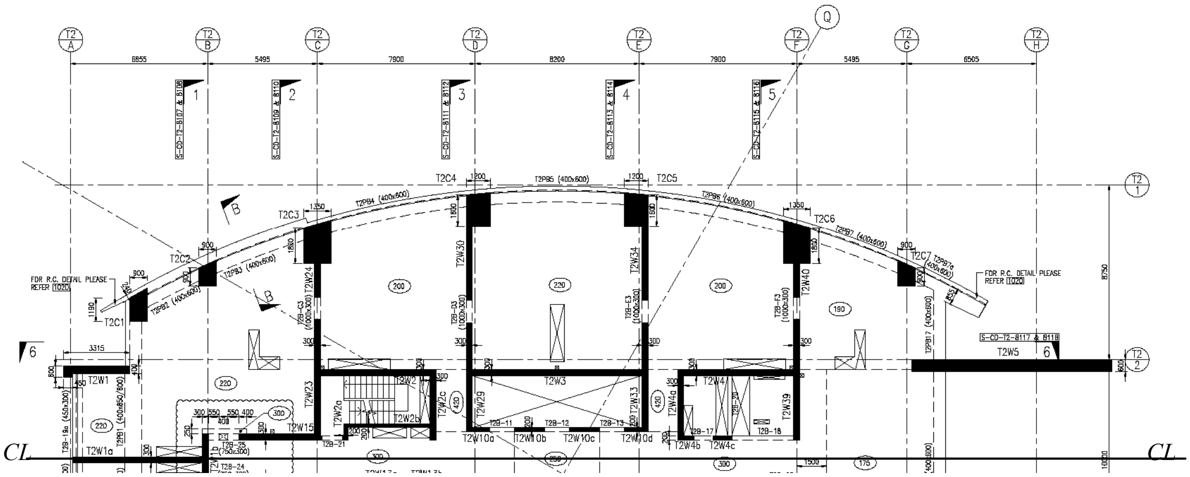


Figure 1 – Half of a Typical Floor Plan of Building

Lateral loading

The Uniform Building Code 1997 was used in this study to determine the loading on the building. Accordingly, the seismic base shear *V* is given by:

$$0.11C_a IW \leq V = \frac{C_v I}{RT} W \leq \frac{2.5 C_a I}{R} W \qquad [1]$$

where C_a and C_v are seismic coefficients defined by soil profile type (S) and seismic zone factor (z). This building is laid on a very dense soil, type Sc, and a moderate seismic hazard region (2A) with seismic zone factor (z) equal to 0.15. Based on table 16-Q and 16-R of the UBC 1997, C_a and C_v are 0.18 and 0.25, respectively. *R* is the force modification factor which is 5.5 in both directions. The building is intended for apartment and office occupancy and is therefore of normal importance, *I* = 1.0. The UBC 1997 specifies that structural period, can be determined using the formula:

$$T = 0.0488(h_n)^{3/4} \qquad [2]$$

where h_n is the building height. In the long *(x)* direction, *T* is 7.27 s; for the short direction, *T* is 7.90 s. *W* is the dead load of the entire building plus 25% of snow load and the weight of permanent equipments. The dead load (W) of the building is 1,655 MN.

The calculated base shears is 32,772 kN in each direction. UBC 1997 requires that the lateral loading be distributed over the building height as follows:

$$F_i = \frac{(V - F_t) W_i h_i}{\sum_{r=1}^{n} W_r h_r} \quad [3]$$

where F_i is the lateral force applied at level *i;* $F_t = 0.07TV$ but less than 0.25 V; W_r and W_i are portions of W at levels *r* and i respectively; and h_r and h_i are the heights above the base to levels *r* and i respectively.

Provisions of UBC 1997 require that where diaphragms are not flexible, the mass at each level be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration; and the effect of this displacement on the story shear distribution shall be considered. The preliminary result of elastic analysis shows that the maximum drift in each floor is less than two times average story drift, so we consider the diaphragm as rigid.

Analysis and design

Elastic Modeling and Analysis

Initial elastic analysis of the shear wall structure is performed in order to determine the member seismic design forces. The structural analysis program ETABS (Wilson and Habibullah 1992), is used to perform three-dimensional analyses. The walls are modeled as shell elements which combine both membrane and plate bending deformations using four-node elements. The coupling beams are modeled as beam elements. The effects of flexure, axial, and shear deformations are included in the analysis.

Stiffness reductions are necessary to consider the effect of cracking in the concrete and subsequent loss of stiffness in order to provide a more accurate estimation of lateral deflections and force distribution. The ACI-318 provides estimates for the effective stiffness properties of structural elements. The bending, axial, and shear stiffnesses of the walls are each reduced to 70% of the uncracked values, while the bending stiffness of the coupling beams is reduced to 35% of the uncracked values.

UBC 1997 limits the maximum drift for the building with T > 0.7 s to 2% of the storey height. Story drifts is computed using the Maximum Inelastic Response Displacement, Δ_M. The maximum computed elastic drifts (Δ_S) are 0. 351% and 0. 431% in the long and short directions, respectively, using the effective member stiffness parameters described previously. Therefore the maximum inelastic response drift (Δ_M), are 1.35% and 1.66% in the long and short directions, respectively, which are less than the UBC 1997 limit.

Inelastic Modeling and Analysis

In order to perform the static pushover analysis, moment curvature characteristics need to be determined for the structural elements. The Section Designer (SD) of ETABS program is used to analyze the reinforced concrete sections. The program uses the strain compatibility approach and can predict the moment curvature response under a given axial load condition. The effect of confinement, due to the presence of closely spaced transverse reinforcement is considered in the stress-strain relationship of concrete in the analysis.

The inelastic dynamic analysis of reinforced concrete building structures program ETABS, is used to calculate the nonlinear static responses of the structure to pushover loading. This program contains the relevant modeling features and capabilities necessary for proper analysis.

Second-order effects are also calculated by the ETABS program. Both the wall and beam elements are defined using the same basic frame element, and the building is simply modeled as a series of 3D frames linked by rigid horizontal diaphragms. The basic element accounts for flexural, shear, and axial deformations. Rigid end conditions are applied to the coupling beams in order to account for the shear wall widths.

The nonlinear behaviour of the structural elements is modeled using plastic hinges located at critical sections of the elements. The plastic hinges properties are developed by moment-curvature of elements and FEMA 356 recommendations. Nonlinear Static Procedure (NSP) or Pushover analysis consists of applying a monotonically increasing static lateral load. Loading is steadily increased either to a predetermined level of base shear or to the collapse of the structure (formation of a mechanism through plastic hinging), whichever occurs first. Pushover analysis is an efficient tool used to analyze the behaviour of structures that respond primarily in the first mode, highlighting the sequence of element cracking and yielding as a function of the level of base shear. However, the seismic responses of tall structures, such as the building considered in this paper, are dominated by second and higher modes. Consequently, the results of a pushover analysis cannot be expected to provide a realistic simulation of the distribution of internal deformation.

In this building which higher mode effect is significant, the NSP is permitted if a Linear Dynamic Procedure (LDP) is also performed to supplement the NSP. The LDP is performed using the response spectrum method to calculate peak modal responses for sufficient modes to capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building. In this building, 20-modes were selected to capture the 90% of the participating mass in each of the two orthogonal principal horizontal directions.

NSP can be classified into three major groups based on the type of lateral load patterns applied to the structural model during analysis (Kalkan E. and Kunnath S., 2007): invariant single load vectors (FEMA-356), invariant multi-mode vectors, and adaptive load vectors.

Two sets of lateral load distributions are recommended in FEMA-356 for nonlinear static analysis. The first set consists of a vertical distribution proportional to (a) pseudo lateral load (this pattern becomes an inverted triangle for systems with fundamental period $T1 < 0.5$ s); (b) elastic first mode shape; (c) story shear distribution computed via response spectrum analysis. The second set encompasses mass proportional uniform load pattern and adaptive load patterns. FEMA-356 recommends that at least one load pattern from each set be used to obtain the response envelope. Therefore, in this study, story shear distribution computed via response spectrum analysis and a modified modal pushover analysis , are employed. The modified modal pushover analysis use the mode shapes of the effective modes as the load patern for pushover analysis. The result of pushover analysis for each mode, is incorporated into modal pushover result of the system based on mass participation of modes The results presented in this paper represent the envelope of the two distributions, i.e., the modified modal pushover and the story shear load pattern based on spectrum analysis.

Performance criteria

The recommendations of the FEMA-356 include specific performance levels for seismic design according to the categories: fully operational, operational, life-safe, near collapse, and collapse. Of particular interest to this study are the operational, life-safe, and near collapse performance levels. The FEMA also suggests that each of these performance levels be associated with a different level of seismic hazard. For normal structures, life-safe performance should be expected for normal code design level ground motions (e.g., 10% in 50-year probability of exceedance) while near collapse performance is expected for lower probability ground motions (e.g., 10% in 100 years) and operational performance is expected for higher probability seismic activity (e.g., 50% in 50 years).

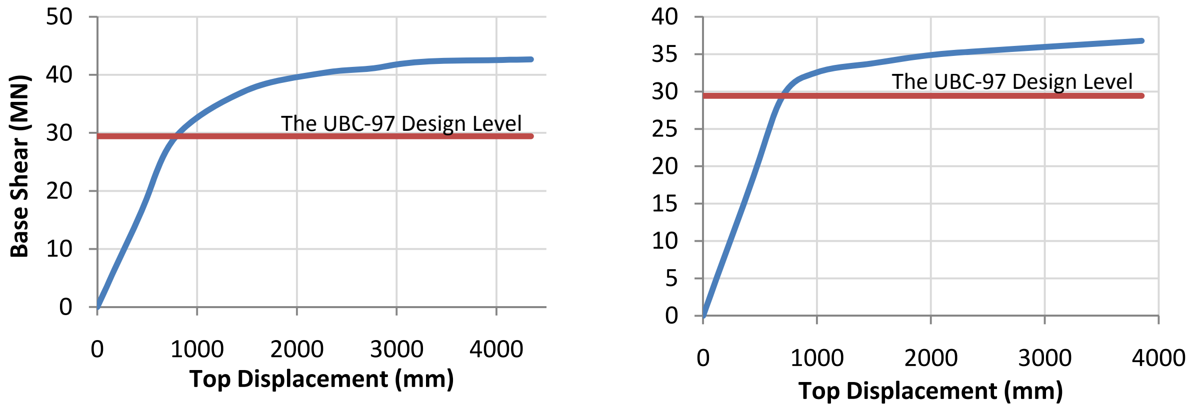
Maximum drift level is of particular interest to engineers, since it is a parameter easily obtained from analysis. The maximum permissible drifts are 0.5%, 1.5%, and 2.5% for the operational, life-safe, and near collapse performance levels respectively. Since the UBC 1997 limits drift to 2% for design level excitations (probability of exceedance of 10% in 50 years), one would expect structures meeting the UBC design criteria to have damage somewhere between the life-safe and near collapse performance levels when subjected to motions at the design level.

In the evaluation of coupling beam and wall element performances, element damage is related to the maximum absolute curvature. Specific curvature values that indicate both cracking and yielding in the walls and beams are determined in the analysis. Beyond these values, a maximum value of curvature that indicates complete loss of strength and stiffness is defined. Based on previous research (Paulay and Priestly 1992), it is conservatively estimated that the ultimate curvature ductility factor is 10 for both the coupling beams and walls. Consequently, the maximum calculated curvatures can be related to the descriptions of shear wall and coupling beam damage.

Results and Discussion

Short direction response

Due to the unsymmetrical wall arrangements, analysis is conducted in both the positive and negative directions. However, since the response in each direction follows almost the same trend, only the response in the positive direction is considered here. The force-displacement relationship for the response of the shear wall system in the coupled direction is shown in Fig. 2-a.



(a) – Coupled Direction (b) – Non-Coupled Direction

Figure 2– Pushover Curves

Most of the coupling beams yield prior to first yielding in the walls. This behaviour confirms the successful application of the capacity design method in restricting inelastic deformation to the coupling beams until the maximum drift approaches the near collapse performance level. Inelastic behaviour begins when the first coupling beam yields at a load that is slightly below the design base shear and the maximum drift is just over 1 %. The beams yield in a gradual progression with increasing deformation and load; the maximum drift is over 2% when 90% of the beams have yielded.

Table 1. Summary of Pushover Analyses:
Damage State of Lateral Force Resisting Elements at selected drifts

Drift Level	**Expected Performance of FEMA 356**	**Performance determined from pushover analyses**		
		Coupled direction		**Noncoupled direction**
		Walls	**Coupling beams**	**Walls**
0.5%	Minor hairline cracking of shear walls, coupling beams	Completely elastic response, stiffness maintained	Minor cracking through bottom third of building	Minor cracking in wall 2, while remaining elements are in the elastic region
1.5%	Limited bar buckling, some crush and flexural cracking, coupling beams – extensive shear and flexural cracking, concrete remains in place	Maximum wall curvature 0.61 of yielding, some cracking has occurred mostly at the base	Significant cracking, yielding, peak ductility – some strength remains	Onset of shear wall yielding, extensive cracking
2.5%	Major flexural and shear cracks, extensive crushing, coupling beams shattered, virtually disintegrated	Wall yielding just barely begins, remaining base elements have cracked	Severe coupling beam damage, little reserve strength, stiffness	Significant yielding at the base of the wall

At the 0.5% drift level, there is practically no structural damage. Cracking has occurred in about one-half of the coupling beams, but none of the walls has cracked. The structure at this stage is still responding with its initial linear elastic stiffness, which is confirmed by the pushover curves shown in Fig. 2-a. The damage is equal to or less than that associated with the FEMA Operational Performance Level (Table 1).

At the 1.5% drift level, the coupling beams suffer significant damage. Beams in the bottom 2/3 to 3/4 of the structure yield and most of the remaining beams crack. The maximum curvature ductility is 5.5. Some of the walls crack for about the bottom 1/3 of the height of the structure. The degree of wall cracking, even at the base of the walls is quite limited at this drift level. There is some loss of stiffness when the deformation reaches 1.5% drift, but that this deformation is not much past the original elastic stiffness line.

Interpreting the above in light of the descriptions in Table 1 indicates that the calculated damage is consistent with the expected "moderate damage" at the FEMA life-safe performance level including extensive cracking in the coupling and some reduction in lateral stiffness.

At 2.5% drift level, almost all of the coupling beams yield. Approximately one-half of the coupling beams have deformed beyond the ultimate beam curvature capacity, indicating that these beams can be expected to have essentially no reserve strength. This damage is consistent with the description given in Table 1 for the FEMA near collapse performance level, in which severe coupling beam damage virtually disintegrated. The wall cracking is extensive in the bottom stories and extends to approximately 1/3 of the height of the structure in some of the walls. The maximum damage is consistent with that given in Table 1 for the state corresponding to near collapse.

Table 1 summarizes the state of damage at each of the three drift levels in relation to the performance expected by the FEMA-356 criteria. Based on this summary and the foregoing discussion of damage, the performance of the structure in the coupled direction at various drift levels is at least as good as, and sometimes better than, that expected using the FEMA-356 descriptions.

Long direction response

The wall system in the non-coupled direction relies completely on the shear walls to resist all lateral loads. Using the capacity design approach, the base of each shear wall is designed with a potential plastic hinge zone, detailed to deform inelastically prior to yielding the wall elements above the base.

Figure 2-b shows the base shear versus top displacement relationship for the non-coupled direction as well as the design level base shear. At the design base shear, the maximum drift is approximately 0.75%, which is less than one half of that estimated during the design process. As discussed in the section on long direction response it is clear that the level of cracking observed during the pushover analysis is substantially less than that estimated in the design drift calculations.

At the 0.5% drift level, the system is already resisting lateral forces approaching the UBC design base shear. The wall elements behave elastically. Maximum wall curvature is limited to 32% of the yield curvature. This response meets the permissible damage levels defined by FEMA 356 (Table 1).

At the 1.5% drift level, Additional cracking of the shear wall elements continues as the lateral forces are increased. The base and first storey of the core walls are subjected to the curvature demands which are just barely beyond yielding (curvature ductility factor of 1.02). Cracking occurs in all the walls from the base to about 70% of the height of the building. The post-cracking stiffness of the wall elements is very close to the uncracked stiffness. Therefore a significant change in the force displacement relationship does not occur until all three walls yield at the base (approximately 1.6% drift). This level of performance is slightly better than that expected at 1.5% drift, for which it is expected that the lateral stiffness would be reduced (Table 1).

At 2.5% drift level, yielding of all three walls is excessive, with maximum curvature ductilities about 10 for the walls. Significant yielding occurs from the base to just above the first floor, which is well within the plastic hinge region (extending 5 storeys above the base) assumed during the design. The capacity design approach is effective in limiting yielding to the plastic hinge region. The maximum curvatures mentioned above indicate that the walls are severely damaged at the base and are near collapse. These damage levels correspond to the FEMA 356 permissible "near collapse" damage levels.

Table 1 also includes the performance of the shear wall system in the noncoupled direction at the selected drift levels. It is clear from this table and the foregoing discussion that the shear wall system meets or exceeds all of the FEMA expected performance levels at each of the three drift levels.

Conclusion

The results of the static pushover analysis shows that the building system responds effectively in resisting moderate levels of lateral loading. The systems in both directions can be expected to perform at the operational level or better when excitation is at the design level. Since the pushover analysis does not replicate the actual dynamic relationship between drift and element damage, further investigation using inelastic dynamic analysis is required to identify both element damage and drift during various levels of dynamic excitation.

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