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Article

Quantification of Different Sources of Over-Strength in Seismic Design of a Reinforced Concrete Tall Building

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Abstract. An over-strength factor in seismic design plays an important role in computing actual forces in a structural member designed to remain elastic. However, sources contributing to this over-strength have not yet been systematically quantified for tall buildings. This paper aims to investigate the contribution from different sources of the over-strength factor in a reinforced concrete (RC) tall building. The effect of how floor slabs are modeled in nonlinear structural models to compute lateral load capacity of the building is also investigated. 39-story RC building subjected to earthquake ground motions in Bangkok was first designed according to the current building codes. Then, pushover analysis was conducted to compute lateral load capacity of the building with three different specified strengths: design strength (with ϕ factor), nominal strength (without ϕ factor), and actual strength (with material over-strength). It was found that modeling floor slabs by elastic shell elements in nonlinear structural model should not be used in computing the ultimate lateral load capacity of the building because the contribution from slab-column framing action is unrealistically large at large roof displacement. When floor slabs are modelled by inelastic effective beam width approach, slab-column framing action contributes about 60% of the ultimate lateral load capacity of the building. The building has an overall lateral over-strength factor of 3.36 to 3.71. The over-strength factor arising from design process is 2.12 to 2.42 in which the contributions from strength reduction factor, material over-strength, and other sources involving the design requirements are about 1.10, 1.17, and 1.77, respectively. The over-strength factor arising from redundancy due to the redistribution of internal forces is about 1.55 and the contribution from steel strain hardening to the over-strength factor is relatively small.

Keywords: Over-strength factor, tall RC shear wall building, seismic design, pushover analysis.

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1. Introduction

Majority of buildings with shear wall structures have a good performance with apparently little to no damage after an earthquake event, even though actual seismic demands from the earthquake exceed their design values [1, 2]. This is due to the reserved strength or over-strength inherent in the design which prevents collapse of buildings. The over-strength factor is defined as the ratio of actual ultimate lateral strength to code-based design lateral force. Both ASCE 7-16 [3] and NBCC [4] provisions recognize the presence of significant over-strength in a building. The over-strength factor is explicitly shown in NBCC [4] as an over-strength related force modification factor (R_0) but is implicitly considered in ASCE 7-16 [3] with the use of response modification factor (R) representing both ductility and over-strength factors. The over-strength factor plays an important role in amplifying seismic forces to get forces for use in design of critical structural members designed to remain essentially elastic such as collectors in a diaphragm, transfer beams, discontinuous system, and elements supporting discontinuous frames or walls [3]. The over-strength factor resulted from a flexural strength design of beams and columns is also used to determine shear forces for use in design of beams and columns in the moment resisting frame [5-8]. The over-strength factor is also related to shear-amplification problem in shear walls as it can increase shear forces of RC shear walls [9-11]. Therefore, accurate estimation of the over-strength factor is important to compute the design forces of those structural elements.

Over-strength in a building is resulted from many sources which were comprehensively described in many previous studies [12-15]. Different sources contributing to the over-strength factor can be classified into two groups [3, 12]. The first group is related to design process whose contributions are: material over-strength (actual material strengths are higher than nominal strengths specified in design), strength reduction factor (ϕ), multiple load cases and load combinations, conservativeness in the design selection (selecting sections or specifying reinforcements that exceeds the required design), design controlled by code minimum requirement, and design controlled by drift rather than strength (usually for the case of flexible structural system). The second group is related to redundancy (internal forces redistribution among structural members after occurrence of yielding) and from steel strain-hardening. The over-strength in buildings has been determined by several researchers using nonlinear static pushover analysis [15-19]. For RC buildings with medium periods designed and detailed to EC8 [7], a minimum over-strength factor of 2 is suggested [15]. For RC frame buildings, the over-strength factor decreases with increasing number of stories such that the over-strength factor arising from design is 2.7 to 3.3 for a single story building and 1.3 for 10-story building, and the over-strength involving structural redundancy beyond yielding does not significantly change with number of stories and is approximately 1.2 to 1.6 [16]. The over-strength factors in RC shear wall buildings have been found to be larger than those given in the prescriptive code, i.e., ASCE 7-16 (Ω_0=2.5) [17, 19]. Mwafy [17] conducted a study using five RC wall buildings ranging from 20 to 60 stories and indicated that over-strength factors slightly decrease with increasing building height and generally vary from 2.5 to 3.5. This finding corresponds to the case of RC wall buildings where earthquake loads govern the design. The over-strength factors of 3.4 to 4.2 were reported from a study of a 17-story RC building with stiff RC shear walls designed with minimum reinforcement for shear and flexure [19]. The over-strength factor variably depends on design situation and it would not be generally true to say that the over-strength factor in the code is conservative or non-conservative. The over-strength factor in the code is simply an approximate value, which is neither an upper bound nor a lower bound value according to ASCE 7-16 [3]. To obtain a more realistic over-strength factor and to realize relative importance of each source, contribution to the over-strength factor due to each source should be quantified. However, these sources have not yet been systematically quantified for tall buildings.

In a common seismic design practice, strength and stiffness of a gravity load resisting system might be neglected; however, including them in the analysis would be helpful to assess their influence on the overall system behavior [20]. In an RC shear wall building where structural walls resist most of the total lateral force, slab-column frames could be modelled with hinge connections not transferring moment between slabs and columns so that all seismic forces are carried by the walls [21]. Gravity columns and floor slabs might be conservatively excluded in a nonlinear structural model for conducting a nonlinear static or dynamic analysis of a building. However, contribution from gravity slabs should be taken into account as it may result in greater strain demands, greater shear demands, different flexural reinforcement pattern, and better prediction of yielding locations for providing ductile detailing of RC walls in a building [22]. Moreover, including floor slabs in a structural model has significant influence on ultimate lateral capacity and dynamic behavior of the building. The slab-column gravity load framing action in steel frame buildings of 2 to 20 stories with perimeter seismic moment resisting frames can increase a total lateral strength by 50% compared to the case excluding slabs and columns [23]. The floor slabs contribute more than 15% of the total lateral stiffness in a 40-story RC tall building [24] and explicit modeling of floor slabs in a 50-story RC tall building helps decrease story drifts by about 22% compared to the case excluding floor slabs [25]. However, the effects of floor slabs contributing to the ultimate lateral capacity of tall buildings have not yet been studied.

This study aims to quantify different sources contributing to the over-strength factor of a 39-story RC tall building. The effects of including floor slabs in a structural model to compute the lateral load capacity of a tall building using elastic and inelastic slab models are investigated. The findings of this study quantitatively reveal the relative importance of sources contributing to the over-strength, which may be used to develop an approach or an empirical formula to compute more realistic over-strength factors in a building than approximate values in the code. The over-strength factor is necessary in calculating design force demands of members intended not to experience any yielding, e.g., shear in vertical members or transfer beams. This study also demonstrates problems in using an elastic slab model in computing ultimate lateral load capacity of a tall building.

2. Methodology

The procedure to conduct this research is the followings:

1) Apply gravity loads, wind loads per Bangkok Building Control Law [26], and earthquake loads; and use the response spectrum analysis (RSA) procedure according to ASCE 7-10 [27] to compute design demands.
2) Design structural members according to ACI 318M-14 [5] specifications considering applicable load combinations. These tasks were implemented in ETABS software [28]. The outcome is a typical structure designed to current standards and code of practice.
3) Develop a nonlinear structural model in PERFORM-3D [29] with strengths of structural members designed in step (2).
4) Conduct pushover analysis using PERFORM-3D [29] to investigate effects of floor slabs on ultimate lateral load capacity of the building considering elastic and inelastic slab models.
5) Conduct pushover analysis to compute base shear-roof displacement relationship also known as lateral force resisting capacity curve and identify the first-yield lateral strength and the ultimate lateral strength of the building with three different specified strengths: (1) design strength (with ϕ factor), (2) nominal strength (without ϕ factor), and (3) actual strength (with material over-strength).
6) Classify the over-strength factor into two groups: (1) the over-strength factor arising from design process and (2) the over-strength factor arising from redundancy and steel strain-hardening.
7) Quantify different sources contributing to the over-strength factor: strength reduction factor (ϕ), material over-strength, steel strain-hardening, redundancy, and other factors involving the design requirements.

3. Description of the Building

A 39-story RC existing tall building in Bangkok was used. This building has a tower surrounded by a podium at the first-eight stories (Fig. 1), which was selected to represent a typical style of numerous tall buildings. The primary lateral force resisting system of the building consists of two RC core walls and the gravity load carrying system is RC columns and RC flat slabs. The typical floor plans and three-dimensional model of the building are shown in Fig. 1. For this building, RC core walls in each story, except at the top floor, resist more than 75% of the story shear force (Fig. 2). The percentages of story shear forces carried by RC walls and columns in each floor shown in Fig. 2 were computed using RSA procedure. The lateral force resisting system was considered to be special RC shear wall whose design factors according to ASCE 7-10 [27] are: $R = 6, C_d = 5, \Omega_0 = 2.5$, where R is the response modification factor; C_d is the deflection amplification factor; and Ω_0 is the over-strength factor. The building was in occupancy type III in which the importance factor is $I = 1.25$. This building was assigned to seismic design category *D*.

The building was modelled using the as-built drawings of the existing building. The concrete cross section sizes of structural members were kept the same as in existing building but the steel reinforcements were re-designed according to ACI 318M-14 [5] considering all load combinations including gravity load, wind load using the Bangkok Building Control Law [26], and earthquake load using the RSA procedure in ASCE 7-10 [27]. The design live loads and super-imposed dead loads at each floor level are shown in Table 1, and design wind pressures from Bangkok Building Control Law [26] are shown in Table 2. The elastic spectral acceleration representing the earthquake load for downtown Bangkok is described in Section 4.

Three-dimensional linear structural model considering cracked cross-sections of structural members was used for analysis and design of the structural system. The effective stiffness values of concrete columns, beams, slabs and walls are presented in Table 3. The floor diaphragms were assumed to be rigid. Accidental eccentricity of 0.05 times dimension of the structure perpendicular to the direction of applied earthquake load was used to account for accidental torsional effect as required by the code. Seismic weight was computed from all dead loads. The foundation was assumed to be fixed support.

It was found that the gravity load governs the design of RC columns and flat slabs, while the wind load results in larger force demands in RC core walls than the earthquake load, but requires vertical steel reinforcement less than code minimum reinforcement; hence, the minimum reinforcement of 0.25% was used for RC core walls in this building (Fig. 3). The design of this shear wall system was carried out such that each RC core wall is considered as a single core-wall section. The internal forces of this core-wall section were compared with P-M interaction strength of this core-wall section with the specified uniform reinforcement ratio (Fig. 3a). Similarly, for shear strength design, shear forces due to wind and earthquake loads are less than design shear strength associated with minimum shear reinforcement of 0.25% as shown in Fig. 3b. The concrete cross sections of structural members and the basic characteristics of the building are summarized in Tables 4 and 5, respectively. The mode shapes and natural periods of the first mode corresponding to translation in X- and Y-directions and torsion of the building are presented in Table 6. These natural periods were computed from the elastic structural model with reduced stiffness of structural members as shown in Table 3.

4. Design Base Shear from RSA Procedure

The building is assumed to be located on a soft-soil site in downtown Bangkok. The spectral acceleration values corresponding to the design basis earthquake (DBE) level taken as 2/3 of the maximum considered earthquake (MCE) level which is 2% probability of exceedance in 50 years, were taken from Thailand seismic design standard, DPT 1301/1302-61 [30]. For this tall building, damping ratio of 2.5% was used as recommended by PEER [31]. The elastic spectral acceleration for 2.5% damping ratio is shown in Fig. 4.

The earthquake load was computed using the RSA procedure in ASCE 7-10 [27], which requires that design shear force from RSA be scaled to have the modal base shear (V_t) not less than 85% of the base shear (V) computed from the equivalent lateral force (ELF) procedure. Such requirement results in a scaling factor (*SF*) which is computed by $0.85V/V_t$ and must be not less than 1. The RSA method was conducted on a three-dimensional linear structural model considering cracked cross sections of structural members with effective stiffness values shown in Table 3. The fundamental period in the RSA is longer than the upper-limited period computed using code-empirical formula according to DPT 1301/1302-61 [30]; hence, the latter was used in the ELF procedure as shown in Table 7. The code-upper limited period (T_a) is the product of coefficient of upper limit period of 1.5 and 0.02 times the total height of the building ($T_a = 1.5 \times 0.02 \times H = 3.76\text{sec}$). The fundamental period, design base shear force computed from the RSA and ELF, and the required scaling factor are summarized in Table 7. The RSA modal base shear is less than 85% of the ELF base shear; therefore, the design base shear of 1.43% of the seismic weight ($0.85V$ from the ELF) was used in the calculation of the over-strength factor (Section 8) for both X- and Y-directions of the building.

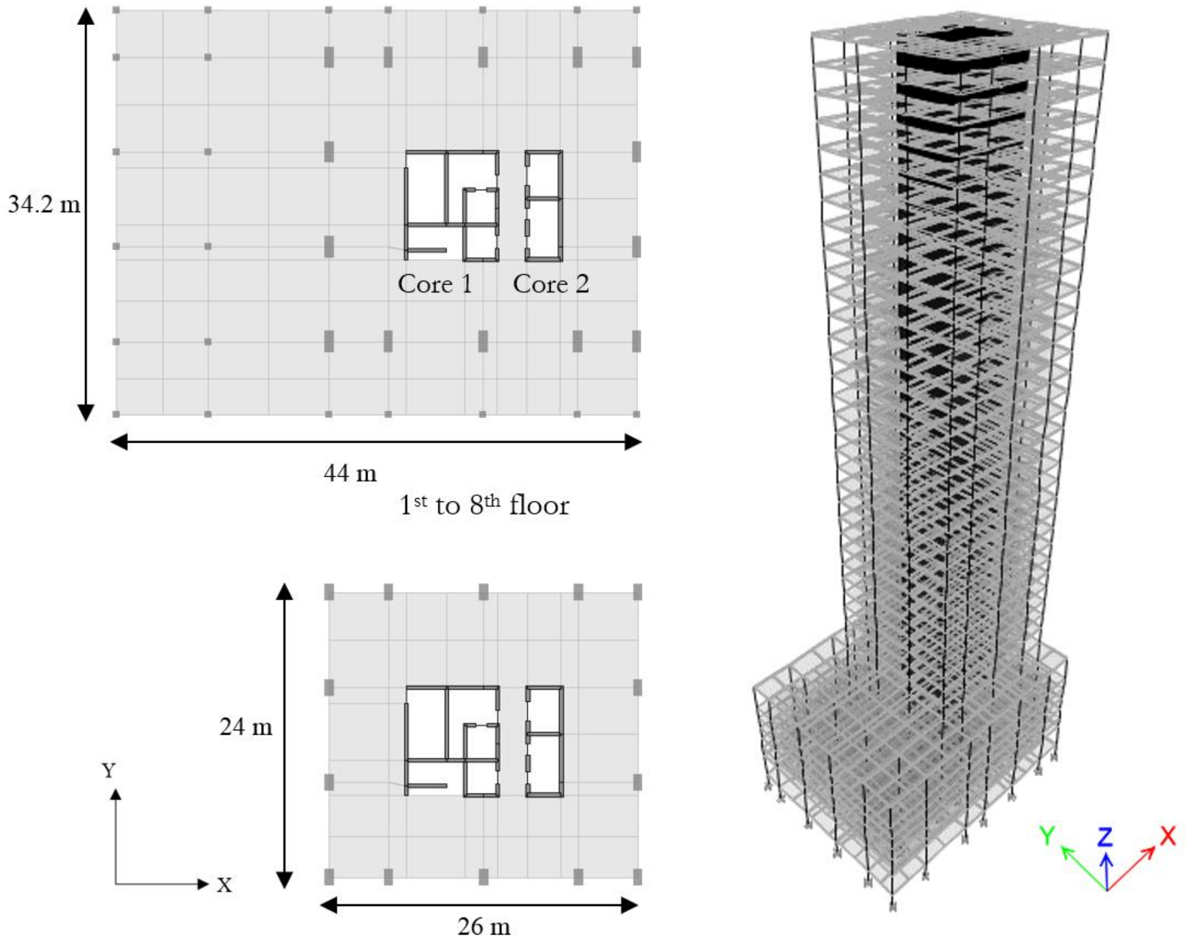


Fig. 1. Floor plans and three-dimensional model of the 39-story case-study building.

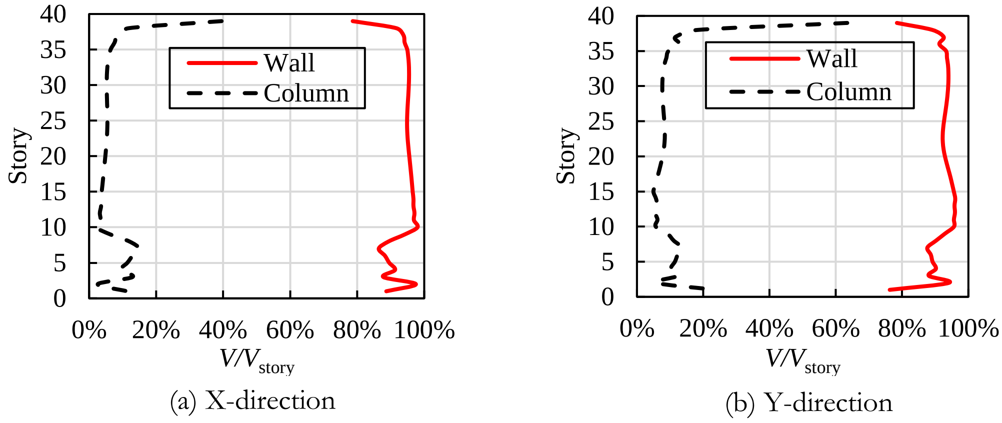


Fig. 2. Percentages of story shear force carried by RC walls and columns determined from RSA procedure: (a) X-direction; (b) Y-direction.

Table 1. Super-imposed dead load (SDL) and live load (LL).

Story	SDL (kPa)	LL (kPa)
9^{th}-39^{th}	2.5	2
1^{st}-8^{th}	2.5	3

Table 2. Design wind pressure from Bangkok Building Control Law [26].

Height (m)	Design wind pressure (kPa)
H>80	2.0
40<H≤80	1.6
20<H≤40	1.2
10<H≤20	0.8
H≤10	0.5

Table 3. Effective stiffness of concrete structural members.

Element	Effective stiffness	
	Moment of inertia	Cross-sectional area
Wall	0.70 I_g	1.0 A_g
Column	0.70 I_g	1.0 A_g
Beam	0.35 I_g	1.0 A_g
Slab	0.50 I_g	1.0 A_g

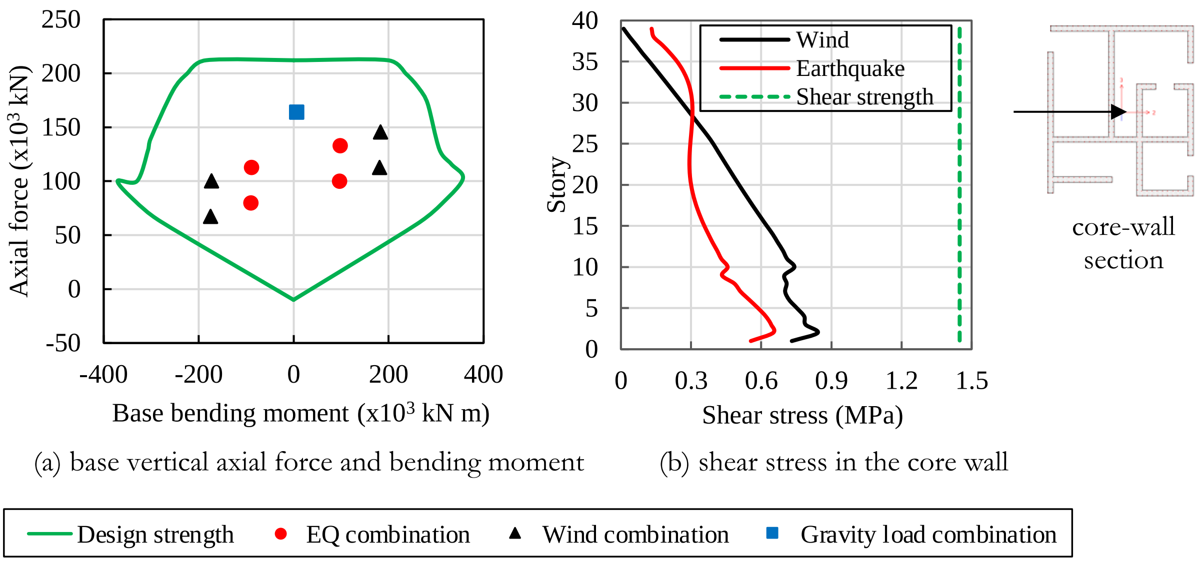


Fig. 3. Design strength and internal forces of a core-wall section: (a) base vertical axial force and bending moment; (b) shear stress in the core wall. The design strength in (a) and (b) is associated with minimum reinforcement. The arrow indicates the direction of lateral loads.

Table 4. Concrete cross-section sizes of structural members of the case-study building.

Story	Wall thickness (m)		Column size (m x m)		Slab thickness (m)
	Core 1	Core 2	Podium	Tower	
36^{th}-39^{th}	0.30	0.35		0.80 x 0.80	0.25
15^{th}-35^{th}	0.30	0.35		0.80 x 1.20	0.25
12^{th}-14^{th}	0.30	0.35		0.80 x 1.40	0.25
9^{th}-11^{th}	0.30	0.35		0.80 x 1.60	0.25
1^{st}-8^{th}	0.30	0.35	0.60 x 0.60	0.80 x 1.80	0.30

Table 5. Basic characteristics of the case-study building.

No. of stories	39
Total height (m)	125.5
Podium height (m)	26.5
Typical story height (m)	3.2
Maximum height-to-width ratio in X-direction	3.6
Maximum height-to-width ratio in Y-direction	2.8
RC wall section area / floor area at the base (%)	1.5
RC column section area / floor area at the base (%)	1.3
Maximum longitudinal reinforcement ratio in RC wall (%)	0.25
Maximum axial load ratio in RC wall $(P / A_g f_c^{'})^{*}$	0.21
Nominal compressive strength of concrete for all members (MPa)	32
Nominal yield strength of longitudinal steel reinforcement (MPa)	400

* P = axial load; A_g = gross section area of wall; $f_c^{'}$ = compressive strength of concrete

Table 6. Three-dimensional mode shapes and natural periods of the first-three modes of the building.

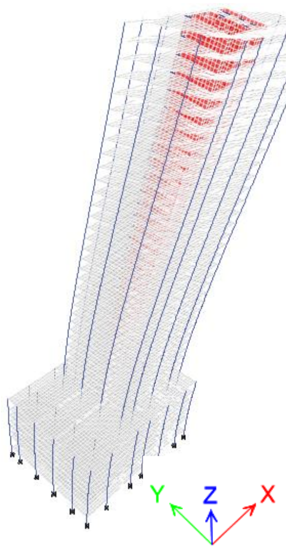
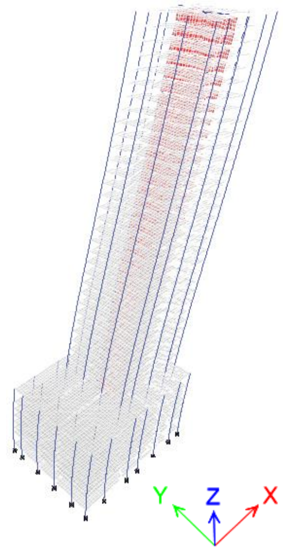
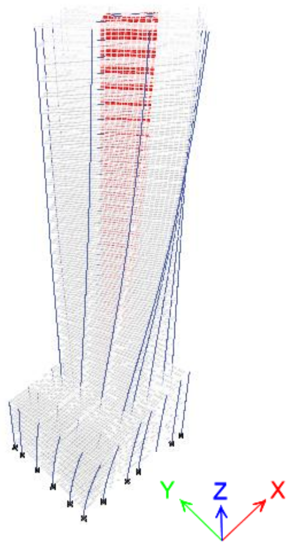
Mode shapes and natural periods		
Mode 1 Y-Translation (T = 5.12 sec)	Mode 2 X-Translation (T = 4.54 sec)	Mode 3 Z-Rotation (Torsion) (T = 2.98 sec)
		

Table 7. Fundamental period, base shear force from the RSA and ELF, and required scaling factor.

Analysis method	Fundamental period (sec)		Base shear normalized by weight (V_b / W)		Scaling factor (SF)	
	X-direction	Y-direction	X-direction	Y-direction	X-direction	Y-direction
RSA (V_t)	4.54	5.12	1.30%	0.91%	1.10	1.57
ELF (0.85V)	3.76	3.76	1.43%	1.43%	-	-

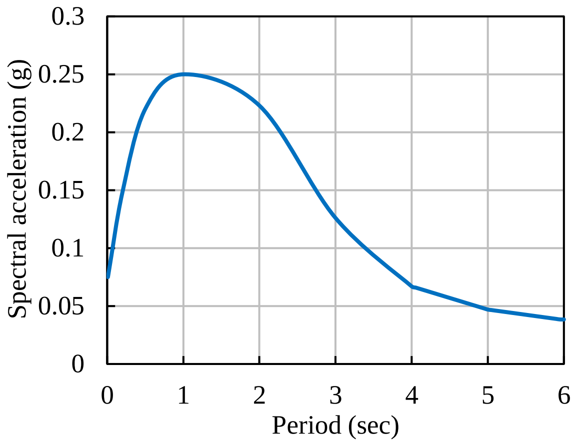


Fig. 4. Elastic spectral acceleration for 2.5% damping ratio for downtown Bangkok [30].

5. Nonlinear Structural Model

Nonlinear structural model was developed in PERFORM-3D program [29]. The RC walls were modelled using nonlinear fiber elements throughout the height of the walls. The concrete stress-strain relationship proposed by Mander et al. [32] was adopted such that it was characterized by a tri-linear relationship in PERFORM-3D. A bilinear inelastic steel model proposed by Menegotto and Pinto [33] was used. Expected material strengths with a material over-strength of concrete and steel equals to 1.25 were used [34]. This material over-strength was taken from Research and Consultancy Institute of Thammasat University [34], where material over-strength factors were obtained from experimental test using local material properties in Thailand. The RC columns were modelled by a linear elastic element with nonlinear plastic hinge zones at both ends which were modelled by nonlinear fiber elements similar to those used for the RC walls. The plastic hinge length of columns was assumed equal to 0.5 times the smaller cross-sectional dimension of the column [35].

The RC coupling beams were modelled by nonlinear plastic hinge elements (zero-length plastic hinge) at both ends with a linear elastic element at the middle portion. The RC slabs were modelled using the effective beam width approach according to ASCE 41-13 [36]. In this approach, the effective width of the slabs computed using equation provided by Hwang and Moehle [37] with the effective stiffness value of 50% times the gross stiffness value was used to model nonlinear behavior of the slabs according to ASCE 41-13 [36]. This effective beam width model consists of an equivalent linear elastic beam and nonlinear plastic hinge elements at both ends. The nonlinear plastic hinge of the coupling beams and slabs was represented by a tri-linear moment-hinge relationship whose plastic rotation modelling parameters were taken from ASCE 41-13 [36] (Table 10.19 for coupling beams and Table 10.15 for slabs). The yielding and ultimate bending moment strengths of the coupling beams and slabs corresponding to effective beam width were determined from its design flexural reinforcements and concrete cross sections. The moment-hinge rotation relationships used for coupling beams and slabs are shown in Fig. 5a and 5b, respectively, where bending moment strengths are normalized by yield bending moment. For the purpose of comparison of the effect of elastic slabs and inelastic slabs on the lateral capacity of the building, nonlinear structural model with assumed elastic slabs was also developed, where slabs were modelled using elastic shell element with reduced flexural stiffness as shown in Table 3. Joints between structural members were considered to be rigid connections. All nodes on each floor were constrained to behave as an in-plane rigid floor diaphragm. Foundation was assumed to be fixed support.

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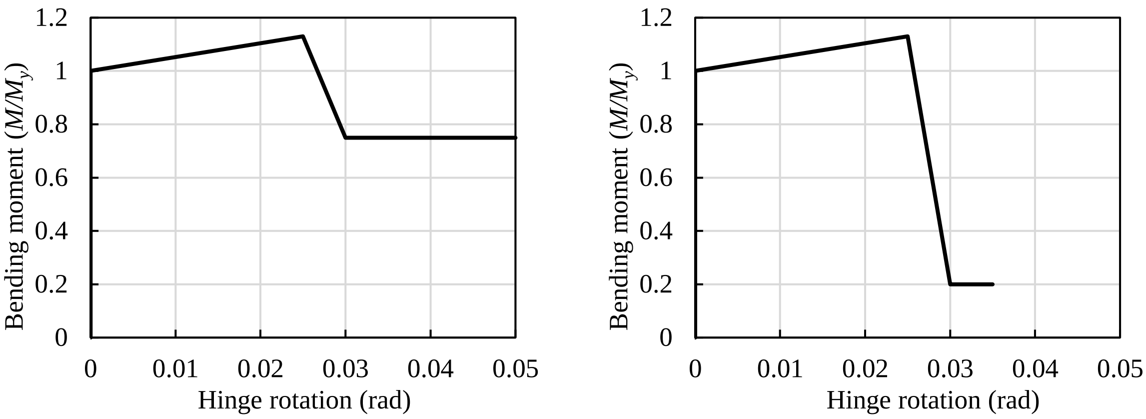

(a) Coupling beams (b) Slabs using effective beam width

Fig. 5. Tri-linear moment-hinge rotation relationship: (a) coupling beams; (b) slabs using effective beam width model.

6. Nonlinear Pushover Analysis

Nonlinear pushover analysis was conducted using PERFORM-3D [29]. Three lateral force patterns were considered for pushover analysis: (1) uniform force distribution, (2) square root of the sum of the squares (SRSS) force distribution, and (3) first-mode shape force distribution. The force pattern resulting in the least lateral capacity was chosen for computing the over-strength factor [7]. The gravity load of all dead loads plus 25 percent of all live loads was applied before pushover analysis with the specified lateral forces.

The pushover curves obtained from analysis using the three specified lateral force patterns are compared in Fig. 6. The pushover curve is presented in terms of base shear normalized by the seismic weight (V_b / W) and roof drift (roof displacement normalized by the total building height). It was found that using the uniform force distribution provides the largest lateral capacity of the building followed by the SRSS force distribution, while using the first-mode shape force distribution results in the least lateral capacity of the building; therefore, it was used to compute the lateral capacity of the building in the following sections.

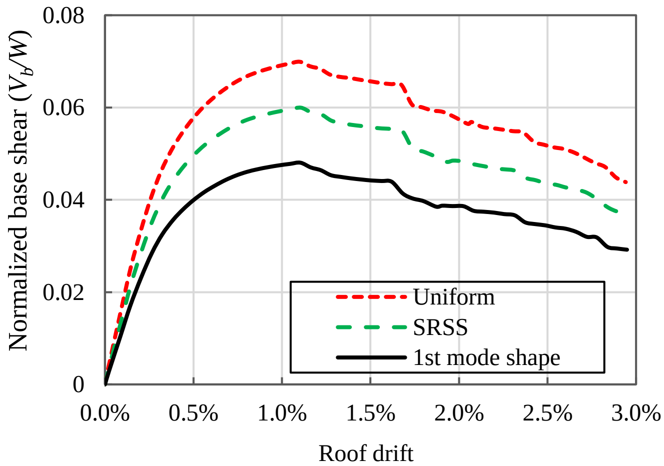

Fig. 6. Base shear-roof displacement relationship computed from pushover analysis using three lateral force patterns in X-direction of the building.

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7. Effects of Floor Slabs on Lateral Load Capacity of the Building

To investigate the contribution of floor slabs on the lateral load capacity of the building, pushover analysis was conducted on nonlinear structural models with and without including floor slabs in the structural model. For the structural model without floor slabs, all nodes in each floor are constrained to behave like an in-plane rigid floor diaphragm. By this assumption, each node of column and wall will behave like horizontal rigid floor even though the floor slabs are excluded in the analytical model. This case is defined as the structural model without slab bending stiffness. For the structural model with floor slabs, an inelastic slab model using inelastic effective beam width approach (Section 5), and elastic slab models using shell elements and elastic effective beam width approach were used to investigate the effects of elastic and inelastic slabs on the lateral load capacity of the building. The first-mode shape lateral force pattern was used to carry out pushover analysis. The gravity load of all dead loads plus 25 percent of all live loads was applied before pushover analysis.

The base shear-roof displacement relationship also known as lateral force resisting capacity curve computed from structural model: (1) without slab bending stiffness, (2) with inelastic slabs using inelastic effective beam width, (3) with elastic slabs using elastic effective beam width, and (4) with elastic shell slabs, are compared in Fig. 7. It was found that both elastic slab models (3) and (4) provide similar capacity curve, which is significantly larger than the capacity curve from the case with (2) inelastic slab at large roof drift. The building strength drop cannot be realized because of the significant contribution from the elastic slabs, which is clearly seen by subtracting the capacity curves (3) or (4) with (1) as shown in Fig. 7 as (3)-(1) or (4)-(1). Therefore, elastic slab model should not be used to compute the ultimate capacity of the building.

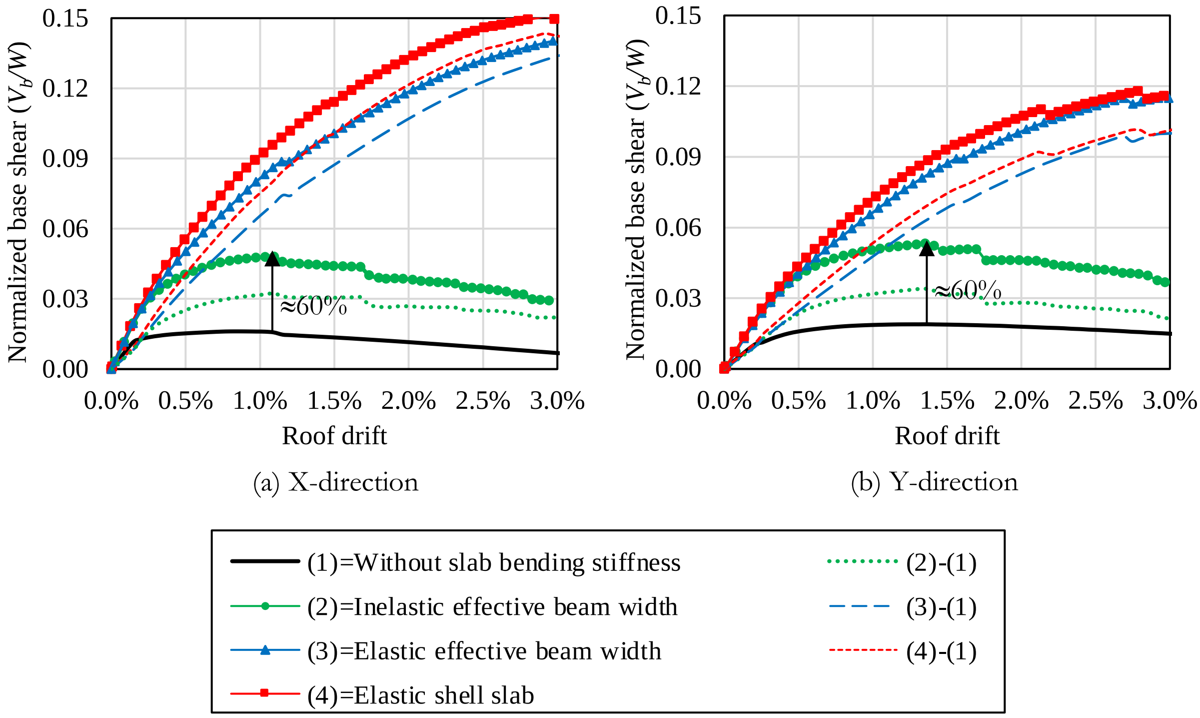


Fig. 7. Base shear-roof displacement relationship computed from pushover analysis using structural models: (1) without slab bending stiffness; (2) with inelastic slab; (3) and (4) with elastic slab: (a) X-direction; and (b) Y-direction.

The inelastic slab model using inelastic effective beam width approach (2) can capture the strength drop and provides more realistic capacity curve than the elastic slab model. In the elastic behavior range (roof drift<0.5%), the inelastic effective beam width model provides very similar capacity curve as the elastic shell slab model. The slabs experience inelasticity (first rotational hinge) at about 0.5% roof drift. When floor slabs are modelled by inelastic effective beam width approach in the nonlinear structural model, the framing action between columns and slabs contributes about 60% of the ultimate lateral load capacity of the building, which can be investigated by subtracting the capacity curve (2) with (1) as indicated in Fig. 7

as (2)-(1). This is the effect of including floor slabs in nonlinear structural model to compute ultimate lateral load capacity of the building. This significant contribution from slab-column frame is not realizable using elastic model at small roof drift where columns resist less than 25% of story shear force. Contribution from slab-column frame becomes more significant at large roof drift in nonlinear models.

8. Over-Strength Factor of the Building

The total over-strength factor (Ω) is classified into two types as shown in Fig. 8. The first type is the over-strength arising from design process or the so-called first-yield over-strength factor (Ω_1) which is the ratio between the first-yield lateral strength (V_{fy}) and the design base shear force (V_d), as shown in Eq. (1). The second type is the over-strength arising from redundancy (internal force redistribution) and steel strain hardening (Ω_2) which is the ratio between the ultimate lateral strength (V_{ult}) and the first-yield lateral strength, as shown in Eq. (2). The total over-strength factor is the ratio between the ultimate lateral strength and the design base shear force, as shown in Eq. (3).

$$\Omega_1 = \frac{V_{fy}}{V_d} \quad (1)$$

$$\Omega_2 = \frac{V_{ult}}{V_{fy}} \quad (2)$$

$$\Omega = \Omega_1 \times \Omega_2 = \frac{V_{ult}}{V_d} \quad (3)$$

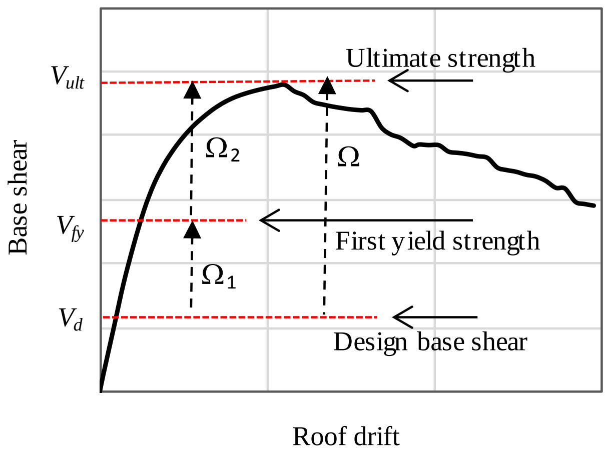


Fig. 8. Lateral load capacity curve and over-strength factor of the building.

To investigate the different sources of the over-strength factor, pushover analysis was conducted to compute the lateral capacity curves of the building with three different specified strengths: (1) design strength; (2) nominal strength; and (3) actual strength. Here, the actual strength refers to the nominal strength that considers material over-strength; the nominal strength is computed using the nominal material strength; and the design strength is the nominal strength multiplied with the strength reduction factor (ϕ) according to ACI 318M-14 [5]. The material over-strength of concrete and steel of 1.25 was used according to Research and Consultancy Institute of Thammasat University [34]. The strength reduction factors according to ACI 318M-14 vary from 0.65 to 0.9 depending on the force action. For flexural action, it is equal to 0.9. As the lateral capacity curve from pushover analysis is controlled by flexural failure mode for this tall building, the strength reduction factor for both steel and concrete materials was taken as 0.9. The strength reduction factor for the bending moment strength of hinges in all beams and slabs was 0.9. The material stress-strain relationship corresponding to the three specified strengths of concrete and steel is shown in Figs. 9a and 9b, respectively. The moment-hinge relationship corresponding to the three specified strengths of slabs is shown in Fig. 10 in which the presented moment strengths are normalized by nominal yielding moment strength. The first-mode shape lateral force pattern was used to carry out pushover analysis.

The sources contributing to the first-yield over-strength can be quantified as the followings:

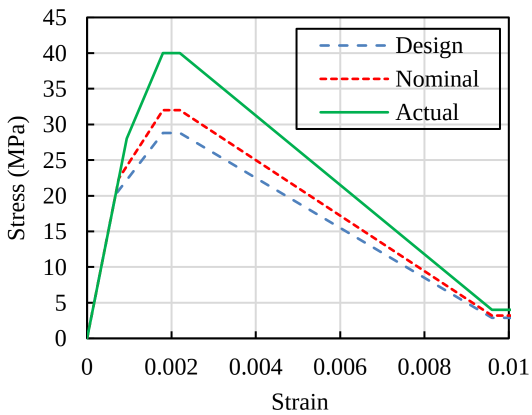
(a) Material over-strength: the over-strength factor due to material over-strength is the ratio between the first-yield strength of the actual-strength pushover curve and that of the nominal-strength pushover curve.

(b) Strength reduction factor: the over-strength factor due to strength reduction factor is the ratio between the first-yield strength of the nominal-strength pushover curve and that of the design-strength pushover curve.

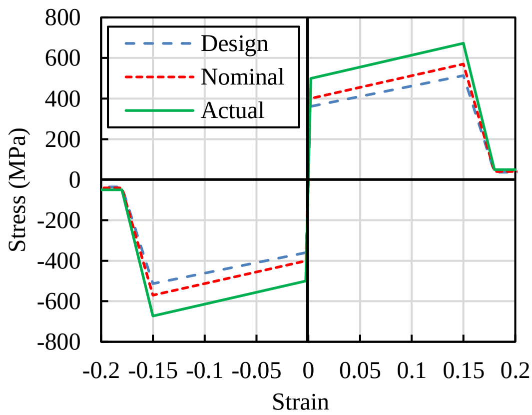
(c) Other factors involving the design requirements (multiple load cases and load combinations, conservativeness in the design selection, design controlled by code minimum requirement, design controlled by drift rather than strength): the over-strength factor resulted from those factors is the ratio between the first-yield strength of the design-strength pushover curve and the design base shear force computed in Section 4.

The lateral load capacity curves corresponding to each of the specified strength level (design, nominal, and actual strength) along with the progressive inelasticity from pushover analysis are shown in Fig. 8. The first-yield strength and the ultimate strength can be identified for each of the capacity curve. It was found that the plastic hinges of the slabs start to form first at roof drifts lower than 0.5% followed by yielding of walls at roof drifts of 0.7% to 0.9%, and then by yielding of columns at roof drifts of 0.8% to 1.0%, and after that the ultimate strengths of the slabs are attained causing the drop of overall lateral strength at roof drifts of 1.1% to 1.4%.

The building possesses an overall over-strength factor of 3.36 and 3.71 in X- and Y-directions, respectively (Fig. 11). The first-yield over-strength factor (Ω_1) is 2.12 to 2.42 and the over-strength arising from the redundancy and steel strain hardening (Ω_2) is 1.53 to 1.59. For the first-yield over-strength (Ω_1), the factors contributing from material over-strength, strength reduction factor, and other factors involving the design requirements are on average 1.17, 1.10, and 1.77, respectively. The factors involving the design requirements are more significant than other sources because code minimum requirement criterion governs the design of RC core walls which are the primary lateral force resisting system in this building. The over-strength factor after the first yielding occurs (Ω_2) came mainly from the redundancy, redistribution of internal forces among structural members, which is about 1.56 on average. The contribution of steel strain hardening to the over-strength factor Ω_2 is very small because the ultimate lateral capacity of the building is controlled by failure of slabs which occurs at roof drift about 1.1% to 1.4%. The steel tensile strains in RC walls at the ultimate lateral capacity are only about 0.01 at which the steel strength is about 1.02 times the corresponding steel yield strength.



(a) Concrete



(b) Steel

Fig. 9. Material stress-strain relationship corresponding to design strength, nominal strength and actual strength of: (a) concrete; and (b) steel.

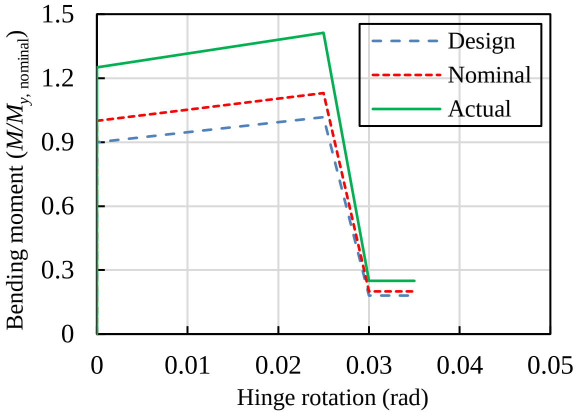


Fig. 10. Tri-linear moment-hinge relationship corresponding to design strength, nominal strength and actual strength of slabs using effective beam width model.

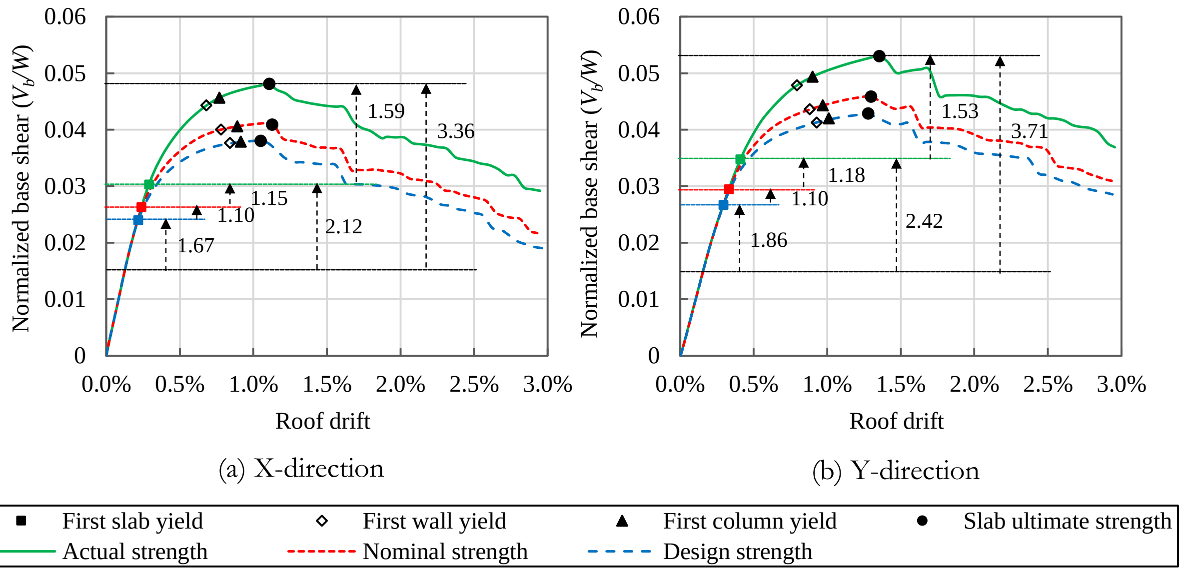


Fig. 11. Different sources contributing to the over-strength factor in: (a) X-direction; and (b) Y-direction. Number next to the arrow indicates the ratio between the upper and lower values.

9. Conclusions

The effects of how floor slabs are modelled in nonlinear structural model to compute the lateral load capacity of the building and the quantification of different sources contributing to the over-strength factor in a 39-story RC tall building have been investigated. The significant findings are summarized as the followings:

1) The elastic slab model should not be used to determine ultimate lateral capacity of the building because the contribution from slab-column framing action is unrealistically large at large roof displacement; therefore, global structural strength drop cannot be realized.
2) The inelastic slab model using effective beam width approach according to ASCE 41-13 provides more realistic lateral load capacity curve than the elastic slab model and could be used to determine ultimate lateral capacity of the building. When floor slabs are modelled by inelastic effective beam width approach in the nonlinear structural model, the slab-column framing action contributes about 60% of the total ultimate lateral load capacity of the building.
3) The case-study tall building designed according to the current design codes possesses an overall lateral over-strength factor of 3.36 to 3.71 which is larger than that specified in ASCE 7-10 (Ω_0=2.5).

4) The over-strength arising from the design process is about 2.12 to 2.42 in which the contributions from strength reduction factor, material over-strength, and other factors involving the design requirements are on average 1.10, 1.17 and 1.77, respectively. The over-strength arising from redundancy due to redistribution of internal forces after yielding is about 1.56 on average, while the contribution from the steel strain hardening to the over-strength is small (less than 5%).

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