

Chapter 9

Developments in Seismic Design of Tall Buildings: Preliminary Design of Coupled Core Wall Systems

M. Nuray Aydınoglu and Eren Vuran

Abstract Performance-based seismic engineering has brought new dimensions to tall building design, leading to a major transformation from the *prescriptive/linear* strength-based approach to the explicit *non-prescriptive/nonlinear* deformation-based design approach. In this context, current tall building seismic design practice is based on a well-established design methodology, which starts with a preliminary design followed by two *performance evaluation* stages. In this methodology, preliminary design represents the critical phase of the tall building design where all structural elements have to be preliminarily proportioned and reinforced for the subsequent performance evaluation stages. However, there are several problems inherent in the existing preliminary design practice. Preliminary design based on linear analysis could lead to unacceptable sizing and reinforcing of the main structural elements of tall buildings. In particular, linear preliminary design procedures applied to coupled core wall systems would most likely lead to an overdesign of coupling beams with inappropriate and heavily congested reinforcement requirements. In addition, linear analysis with reduced seismic loads may result in under-designed wall elements especially in terms of their shear strength. Simple procedures based on first principles have been developed to estimate base overturning moment capacity, total coupling shear capacity and overall ductility demand of the coupled core wall systems, which can be efficiently used in the preliminary seismic design of tall buildings.

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9.1 Introduction

Tall building seismic design has evolved during the last decade to become a major area of application of performance-based earthquake engineering. This development has opened a new door to structural design engineers who were struggling to overcome the structural restrictions imposed on tall buildings by traditional prescriptive seismic design codes. In a broader sense, performance-based earthquake engineering has brought new dimensions to tall building design, leading to a major transformation from the linear strength-based design to a nonlinear deformation-based design practice. In line with this development, special seismic design recommendations/guidelines and consensus documents for tall buildings based on performance-based design principles have been developed and published in the last decade by several institutions. In this respect starting from 2005, Los Angeles Tall Buildings Structural Design Council has published and continuously updated a series of *consensus documents* (LATBSDC 2005, 2008, 2011, 2013, 2014), reflecting the progress achieved in the state of practice of performance-based seismic design of tall buildings. In 2007 Structural Engineers Association of Northern California – SEAONC Tall Buildings Task Group (2007) published its first recommendations on tall building seismic design, which is adopted in 2008 and later updated by San Francisco Department of Building Inspection (2014). On the other hand Council on Tall Buildings and Urban Habitat published in 2008 its design recommendations prepared by Seismic Working Group (CTBUH 2008). As a parallel development, a draft version of a tall building design code was prepared in 2008 for the Istanbul Metropolitan Municipality by the *Kandilli Observatory and Earthquake Research Institute* (IMM 2008; Aydınoğlu 2011) at the time when tall building construction started booming. In the meantime Pacific Earthquake Engineering Research Center (PEER) conducted a multi-year collaborative effort, called Tall Buildings Initiative (TBI), to develop more comprehensive performance-based seismic design guidelines for tall buildings (PEER/TBI 2010) along with a supporting document on modeling and acceptance criteria for nonlinear response (PEER/ATC 2010).

Current tall building seismic design guidelines/consensus documents (PEER/TBI 2010; SFDBI 2014; LATBSDC 2014) are all based on the same design methodology, starting with a preliminary design followed by two *performance evaluation* stages. In the preliminary design, tall building structural system is preliminarily proportioned and reinforced on the basis of linear analyses and *capacity design principles*. San Francisco practice (SFDBI 2014) treats the preliminary design as a *code-level evaluation* stage where selected *prescriptive* provisions including minimum base-shear requirement of the San Francisco Building Code are applied while a number of exceptions are allowed, such as removal of force amplification (over-strength) and reliability/redundancy factors, etc. Thus, SFDBI (2014) formally applies a three-stage procedure, while other guidelines (PEER/TBI 2010; LATBSDC 2014) do not formally define the preliminary design as a design

stage and insist on a *non-prescriptive* two-stage scheme by completely eliminating the prescriptive code provisions.

The two-stage *performance evaluation* following the preliminary design includes a *serviceability evaluation* stage under the so-called *service earthquake* and a *collapse level evaluation* stage under the so-called *maximum credible earthquake*, corresponding to 43 and 2,475 year return periods, respectively. The damping is considered 2.5 % in both stages.

The *serviceability evaluation* stage requires the tall building structural system remains essentially elastic (or nearly elastic with almost negligible nonlinear behavior) under frequently occurring small earthquakes.

On the other hand *collapse level evaluation* considers the worst-case scenario, where the structure is evaluated under the *maximum credible earthquake* with a performance objective aiming at a *reasonably low risk of partial or total collapse*, which corresponds to an acceptable level of damage in terms of ductile response quantities while keeping all other brittle response quantities, e.g., internal forces below their strength capacities, thus preserving the gravity load carrying capacity of the structural system.

Preliminary design represents the critical phase of the tall building design where all structural elements need to be preliminarily proportioned and reinforced for the subsequent performance evaluation stages. Here the problem lies with the fact that designer has no reliable analysis tools at this phase other than linear response analysis and application of capacity design principles, which in fact may not provide a guarantee for an acceptable nonlinear response under the *maximum credible earthquake*. It means that the preliminary design may need to be revised according to the results of the nonlinear performance evaluation. In other words, the so-called *performance evaluation stage* should not be considered only as an evaluation stage, but at the same time as a *design improvement stage*.

In this contribution particular emphasis will be given to the preliminary design of *coupled core wall systems*, which are the most commonly used tall building structural systems for seismic resistance. In an attempt to search for alternate preliminary design procedures, attention will be focused on a recently developed simple and novel capacity estimation procedure as well as a ductility demand estimation procedure (Vuran 2014; Vuran and Aydınoglu 2015). In addition, *shear amplification* and *shear migration* effects will be considered during the preliminary design stage, which are relatively lesser-known but very significant effects governing the core wall seismic design.

9.2 Preliminary Design Issues

Preliminary design stage needs to be given a special emphasis for the development of a suitable tall building structural system later to be evaluated/designed on performance basis through nonlinear seismic analysis.

In this respect, LATBSDC (2014) considers the preliminary design stage as merely equivalent to the application of *capacity design rules* while SFDBI (2014) applies the *prescriptive* provision of *minimum base shear* strength requirement. On the other hand TBI (PEER/TBI 2010) treats the preliminary design issue in a more detailed fashion, additionally including recommendations on system configuration, wind effects, limiting building deformations, setbacks and offsets, diaphragm demands, outrigger elements, etc.

Capacity design rules are intended to insure that “*structural system for the building has well defined inelastic behavior where nonlinear actions and members are clearly defined and all other members are stronger than the elements designed to experience nonlinear behavior.*” Detailed lists are provided in both TBI (PEER/TBI 2010) and LATBSDC (2014) to identify the “*zones and actions commonly designated for nonlinear behavior.*”

When applying capacity design principles, it is stated in LATBSDC (2014) that “*linear analysis may be used to determine the required strength of the yielding actions.*”. This recommendation is problematic in the sense that linear analysis cannot correctly estimate the internal force redistribution in real response due to nonlinear behavior, in particular for coupled core wall systems. On the other hand *capacity protected actions* such as shears in beams and columns may be estimated by capacity design principles to an acceptable accuracy, but shears in walls could be grossly underestimated. In this respect, a frequently encountered example is the preliminary design of coupled core wall systems.

Core walls with peripheral columns represent the most common structural system of tall buildings. Frames with down stand beams are rarely used and in many cases, even completely eliminated leading to flat plate systems. Thus, the so-called *dual systems* with moment-resisting frames (back-up systems) are practically discarded. A number of engineers who faithfully provided the back-up systems in all their past prescriptive code applications appear to be hesitant in accepting this new situation. However it can be argued that properly designed coupled walls with sufficiently stiff and strong coupling beams effectively provide a similar back-up action expected from the moment resisting frames of dual systems with cantilever walls.

Engineers often experience difficulty in preliminary sizing of coupled core wall systems. Reliable practical analysis tools that would help consider the nonlinear seismic behavior of wall piers and coupling beams as well as their combined effect in seismic response of coupled wall systems are not available. Both coupled walls and coupling beams generally undergo significant nonlinear response and coupling beams experience excessive plastic deformations throughout the height of the building. The nonlinear behavior of wall pieces is significantly influenced by the stiffness and strength of coupling beams.

In the current practice, linear analysis is being employed inevitably in the preliminary design stage to identify the stiffness and strength of coupled wall components and their distribution. Such a procedure would most likely lead to an overdesign of coupling beams with inappropriate and probably heavily congested reinforcement requirements. On the other hand, a preliminary design based on a

linear analysis with reduced seismic loads may result in under-designed wall elements especially in terms of their shear strength (Aydinoğlu 2014).

In an attempt to avoid the inappropriate use of linear analysis in the preliminary design stage, employment of multi-mode pushover analysis has been proposed by Aydinoğlu (2014). Based on Incremental Response Spectrum Analysis – IRSA Method (Aydinoğlu 2003, 2004), multi-mode pushover analysis has proven to be a useful tool in preliminary proportioning of coupled core wall systems. In the following, even simpler but very efficient capacity and demand estimation tools are presented, which were developed only recently (Vuran 2014; Vuran and Aydinoğlu 2015).

9.3 Capacity and Ductility Demand Estimation Tools for Preliminary Design of Coupled Core Wall Systems

A simple, *strength-of-materials approach* is developed to estimate the *base overturning moment capacity* and *total coupling shear capacity* of a typical coupled core wall system starting from *first principles*. Based on estimated overturning moment capacity, the simple approach is further extended to estimate the overall *ductility demand* of the coupled core wall system utilizing a novel modification of the pushover concept (Vuran 2014; Vuran and Aydinoğlu 2015).

9.3.1 A Capacity Estimation Tool for Coupled Core Walls

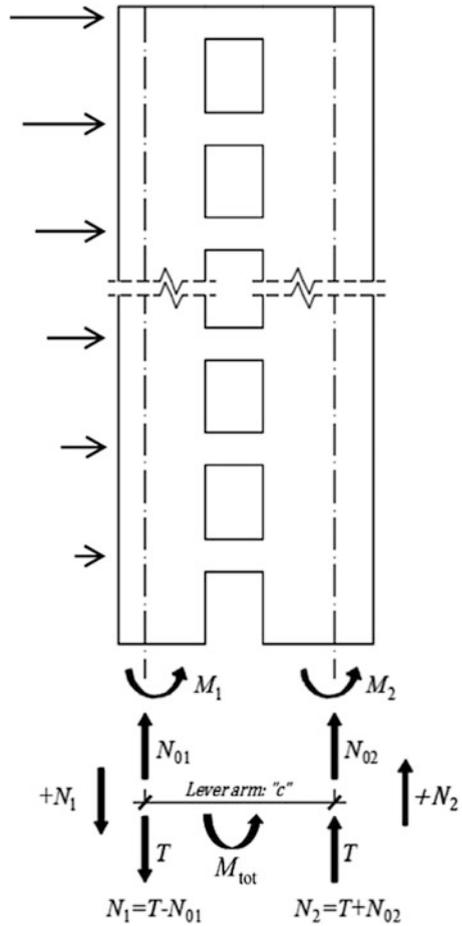
It is assumed that the coupled core wall system shown in Fig. 9.1 responds to earthquake action on its own as the main structural system without stiffness and strength contribution of any other structural element. Actually this is the case in most of tall buildings with core wall at the centre and gravity frames along the periphery.

Using simple equilibrium equations, individual wall axial reaction forces at the base can be expressed as

$$N_1 = -N_{01} + T \quad ; \quad N_2 = N_{02} + T \quad (9.1)$$

where N_1 is considered positive in tension and N_2 positive in compression as indicated in Fig. 9.1, representing the axial force reactions of the so-called *tension wall* and *compression wall*, respectively. N_{01} and N_{02} represent gravity axial loads of walls and T refers to the so-called *total coupling shear* representing the sum of shear forces developed in coupling beams throughout the building. The sense of earthquake direction is assumed from left to right. If opposite, then subscripts 1 and 2 should be interchanged.

Fig. 9.1 Base reactions and coupling shear forces acting on coupled wall system



The base section of the coupled core wall system is the most critical section controlling the nonlinear behaviour of the entire structure. Total base overturning moment reaction of the coupled wall system can be expressed by the following equilibrium equation:

$$M_{tot} = M_1 + M_2 + Tc \tag{9.2}$$

where M_1 and M_2 represent the bending moments of the tension and compression walls, respectively, and c refers to lever arm between the centroids of walls.

The contribution of the force couple, Tc , in total base overturning moment is traditionally represented by *degree of coupling parameter*, A , as follows:

$$A = \frac{Tc}{M_{\text{tot}}} = \frac{Tc}{M_1 + M_2 + Tc} \quad (9.3)$$

The reaction forces and the degree of coupling parameter given above are traditionally evaluated as *demand quantities* obtained from the linear analysis of a given system under a given earthquake action (Paulay and Priestley 1992). However, here they are considered to represent the corresponding *strength capacities*. The ultimate capacity term that would control the coupled wall design is the *total base overturning moment capacity* defined by Eq. (9.2).

It is clear that maximizing the force couple, i.e. the *total coupling shear*, corresponds to maximizing the overturning moment capacity. However, inspection of Eq. (9.1) suggests that *total coupling shear* T should not be increased arbitrarily, as it would lead to increasing tension strains in the tension wall, i.e., spreading of the yielding from the base to the upper parts and hence larger concrete cracking along the wall. At the same time it would lead to increasing compression strains in the compression wall, even it could cause non-ductile compression failure if compressive axial force N_2 exceeds the balance point of axial force-moment interaction. Moreover increased coupling shear would result in reinforcement congestion and construction difficulties in coupling beams.

Thus, it is imperative that a reasonable compromise should be achieved between the strength capacities of individual walls and the coupling beams and such a “*balanced solution*” has to be worked out during the preliminary design stage. This observation has motivated the development of a *capacity estimation procedure* for the initial sizing of the individual walls and the coupling beams in the preliminary design stage.

It has been shown by Vuran and Aydinoglu (2015) that *total coupling shear capacity* and consequently *total base overturning moment capacity* of a coupled core wall system is essentially controlled by three independent parameters:

- (a) Normalized gravity load of the tension wall: $n_{01} = N_{01}/(A_{c1}f_{ce})$
- (b) Mechanical reinforcement ratio of the tension wall: $\rho_{m1} = (A_{s1}/A_{c1})(f_{ye}/f_{ce})$
- (c) Relative yield parameter of the tension wall, β_1 , which represents the ratio of the axial load reaction N_1 to its full yield strength in tension, N_{Y1} :

$$N_1 = \beta_1 N_{Y1}$$

where f_{ce} and f_{ye} denote the “*expected compressive strength*” of concrete and “*expected yield strength*” of reinforcing steel with A_{c1} and A_{s1} representing the corresponding areas in the tension wall and ρ_{m1} is the mechanical reinforcement ratio.

Utilizing the first expression in Eq. (9.1), normalized total coupling shear can be expressed as

$$n_{T1} = \frac{T}{A_{c1}f_{ce}} = n_{01} + \beta_1 \rho_{m1} \quad (9.4)$$

from which application range of the relative yield parameter β_1 can be defined as

$$-\frac{n_{01}}{\rho_{m1}} \leq \beta_1 \leq 1 \quad (9.5)$$

This relationship suggests that the limiting condition $\beta_1 = 1$ corresponds to the largest attainable axial tension force in the tension wall (strain-hardening is neglected for the sake of simplicity) and hence greatest coupling shear according to Eq. (9.1). On the other hand $\beta_1 = -n_{01}/\rho_{m1}$ corresponds to the other limiting condition leading to zero coupling shear, i.e., $n_{T1} = 0$ in Eq. (9.4), which corresponds to the degeneration of the coupled wall system into two individual cantilever walls with axial force reactions equal to their gravity loads only, i.e., $-n_{0i}$.

By appropriate selection of the independent parameters defined above, total coupling shear capacity can be readily estimated from Eq. (9.4), and total base overturning moment capacity can be calculated from Eq. (9.2) by adding bending moment capacities of individual walls, namely M_1 and M_2 . Implementation details are given in Vuran and Aydinoglu (2015).

Note that although above-described capacity estimation procedure is given for a simple coupled wall system shown in Fig. 9.1, it can be extended to more complex systems by appropriate applications of equilibrium equations.

9.3.2 A Ductility Demand Estimation Tool for Coupled Core Walls

Following the estimation of total base overturning moment capacity of the coupled core wall system, it needs to be checked whether it is sufficient for the purpose of preliminary design. This is achieved by evaluating the overall ductility demand, μ , of the system under maximum credible earthquake (MCE) through a novel application of an alternate pushover concept developed, the details of which can also be found in Vuran and Aydinoglu (2015). As an end product, ductility demand, μ , is estimated as

$$\mu = S_{ac}(T_1) \frac{m_{o1}^*}{M_{tot}} \quad (9.6)$$

where $S_{ac}(T_1)$ refers to first-mode spectral pseudo-acceleration of the MCE level earthquake and m_{o1}^* represents the “participating modal mass for the base overturning moment” of the first (dominant) mode, which can be calculated as

$$m_{o1}^* = L_{o1}^* \frac{L_{x1}^*}{M_1^*} \tag{9.7}$$

The parameters of the above equation are defined as

$$L_{o1}^* = \mathbf{h}_o^T \mathbf{M} \Phi_1 \quad ; \quad L_{x1}^* = \mathbf{i}_x^T \mathbf{M} \Phi_1 \quad ; \quad M_1^* = \Phi_1^T \mathbf{M} \Phi_1 \tag{9.8}$$

where \mathbf{M} represents the mass matrix and Φ_1 denotes the first (dominant) mode shape vector. \mathbf{i}_x refers to a vector whose elements are unity for degrees of freedom in x earthquake direction while others are zero. \mathbf{h}_o is a similar vector whose nonzero elements are the story elevations each measured from the base level.

If ductility demand calculated from Eq. (9.6) falls below an acceptable value, the preliminary design may be deemed to be successfully completed. For a satisfactory seismic performance under MCE level earthquake, results of the nonlinear response history analyses (Vuran 2014) have suggested that overall ductility demand of a typical coupled core wall system should be approximately bounded by the limits of $2.5 \leq \mu \leq 3.5$.

If the ductility demand is found acceptable, nonlinear performance evaluation stage can be initiated based on reinforcements calculated for the individual walls and the coupling beams, the latter of which is selected on the basis of coupling shear capacity estimated by Eq. (9.4).

A preliminary estimation may also be made for the base shear demands of tension and compression walls by amplifying the first-mode base shear, which can be approximately calculated in terms of M_{tot} . Based on nonlinear response history analysis performed for symmetrical coupled core wall systems (Vuran 2014), base shear demand for each wall individual may be estimated for preliminary design purpose as (Vuran and Aydınoğlu 2015)

$$V_{base} \cong \frac{M_{tot}}{0.7H} \alpha_{vH} \alpha_{vM} \tag{9.9}$$

where H represents the total building height, α_{vH} is the dynamic shear amplification factor accounting for *higher mode effects* and α_{vM} denotes the dynamic shear amplification factor representing *shear migration* from the yielding tension wall to the compression wall at sections near the base. Recommended dynamic shear amplification factors for preliminary design are:

$$\alpha_{vH} \cong 1.5 \quad ; \quad \alpha_{vM} \cong 2 \tag{9.10}$$

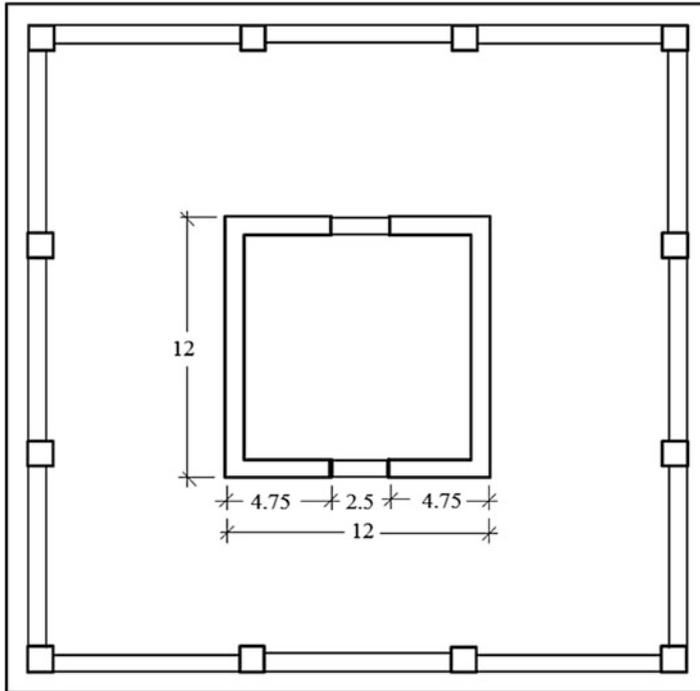


Fig. 9.2 Tall building floor plan with coupled core wall system CW12

9.4 Evaluation of Capacity and Ductility Demand Estimation Tools for Preliminary Design of Coupled Core Wall Systems

In order to evaluate the effects of three independent parameters controlling the capacity of the coupled core wall system, a parametric study is performed (Vuran and Aydinoglu 2015).

Several tall buildings with a central core wall system and gravity columns are designed, ranging from 25 to 50 stories. All cores are of square hollow sections in plan with openings only in one direction spanned by coupling beams with a constant depth/span ratio of $\frac{1}{2}$, thus forming a symmetrical coupled core wall system in that direction. Outer plan dimensions of square cores are selected as 10, 12, 14 and 16 m.

For space limitations, only 12 m² symmetrical core wall system, called CW12 is evaluated here, as shown in Fig. 9.2. Details of the dimensions and loading combinations of the other core wall systems can be found in Vuran (2014).

CW12 has two types with 30 and 40 stories. In 30 story building wall thicknesses are 0.75 m at 1st–10th stories, 0.60 m at 11th–20th stories and 0.45 m at 21st–30th stories. Same wall thicknesses are applied to 40 story building at 1st–15th stories, 16–30th stories and 31st–40th stories, respectively.

Table 9.1 Variation of n_T and μ with respect to n_0 , ρ_m and β for CW12

ρ_m	β	30 story building				40 story building			
		$T_1 = 3.3 s$				$T_1 = 5.7 s$			
		$n_0 = 0.075$		$n_0 = 0.125$		$n_0 = 0.175$		$n_0 = 0.225$	
		n_T	μ	n_T	μ	n_T	μ	n_T	μ
ρ_{mI}	- 1.0	0.041	7.1	0.091	4.3	0.141	4.2	0.191	3.3
	- 0.5	0.058	6.0	0.108	3.9	0.158	3.9	0.208	3.1
	0	0.075	5.3	0.125	3.6	0.175	3.7	0.225	3.0
	0.5	0.092	4.7	0.142	3.3	0.192	3.5	0.242	2.8
	1.0	0.110	4.2	0.160	3.1	0.210	3.3	0.260	2.7
$2\rho_{mI}$	- 1.0	0.006	8.9	0.056	4.8	0.106	4.6	0.156	3.5
	- 0.5	0.041	6.1	0.091	3.9	0.141	3.9	0.191	3.1
	0	0.075	4.6	0.125	3.3	0.175	3.5	0.225	2.8
	0.5	0.110	3.8	0.160	2.9	0.210	3.1	0.260	2.6
	1.0	0.144	3.2	0.194	2.5	0.244	2.8	0.294	2.4
$3\rho_{mI}$	- 1.0	-	-	0.022	5.6	0.072	5.1	0.122	3.7
	- 0.5	0.023	6.2	0.073	3.9	0.123	4.0	0.173	3.1
	0	0.075	4.2	0.125	3.0	0.175	3.3	0.225	2.7
	0.5	0.127	3.2	0.177	2.5	0.227	2.9	0.277	2.4
	1.0	0.179	2.6	0.229	2.2	0.279	2.5	0.329	2.2

$$f_{ce} = 65 \text{ Mpa}; f_{ye} = 491.4 \text{ Mpa}; \rho_I = 0.00457; \rho_{mI} = (f_{ye}/f_{ce}) \rho_I = 0.0345$$

For each building type, two sets of wall gravity loading were considered by changing the number and distribution of gravity columns and hence tributary floor areas of cores. Total floor masses were kept unchanged. This has been deliberately arranged such that normalized wall gravity loads are specified as 0.075 and 0.125 at the base level of the 30 story building and, 0.175 and 0.225 for the 40 story building. Thus for each building type, only one linear dynamic model is defined based on the linear stiffness characteristics, while two different nonlinear dynamic models are defined based on different strength characteristics due to different gravity loading applied to the core walls. Masses are the same in both linear and nonlinear models, which are all developed in accordance with rigid diaphragm assumption. First-mode natural vibration periods of 30 story and 40 story buildings are calculated as 3.3 and 5.7 s, respectively.

Walls are reinforced according to the requirements of the Turkish Seismic Design Code. Minimum wall total reinforcement ratio is designated as ρ_I . Table 9.1 summarizes the results in terms of *total normalized coupling shear*, n_T , versus *ductility demand*, μ , calculated for a typical MCE level earthquake (see Fig. 9.3 for pseudo-acceleration spectrum) for four levels of *normalized wall gravity load*, n_0 , three levels of wall reinforcement ratio, ρ , and five levels of relative yield factor, β , of the tension wall (wall numbers as subscripts are dropped due to symmetrical system considered). *Expected material strengths* are used as indicated at the footer of Table 9.1.

The results given in Table 9.1 are also displayed in Figs. 9.4, 9.5, 9.6, and 9.7 where acceptable range for the ductility demand ($2.5 \leq \mu \leq 3.5$) is indicated.

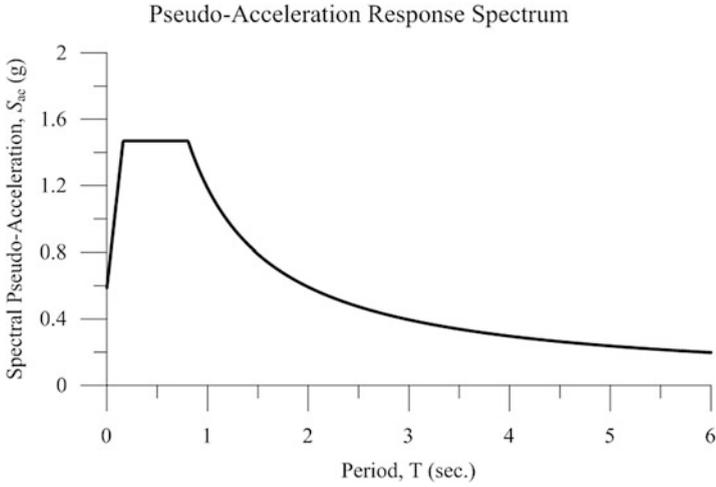


Fig. 9.3 Pseudo-acceleration spectrum for a typical MCE level earthquake

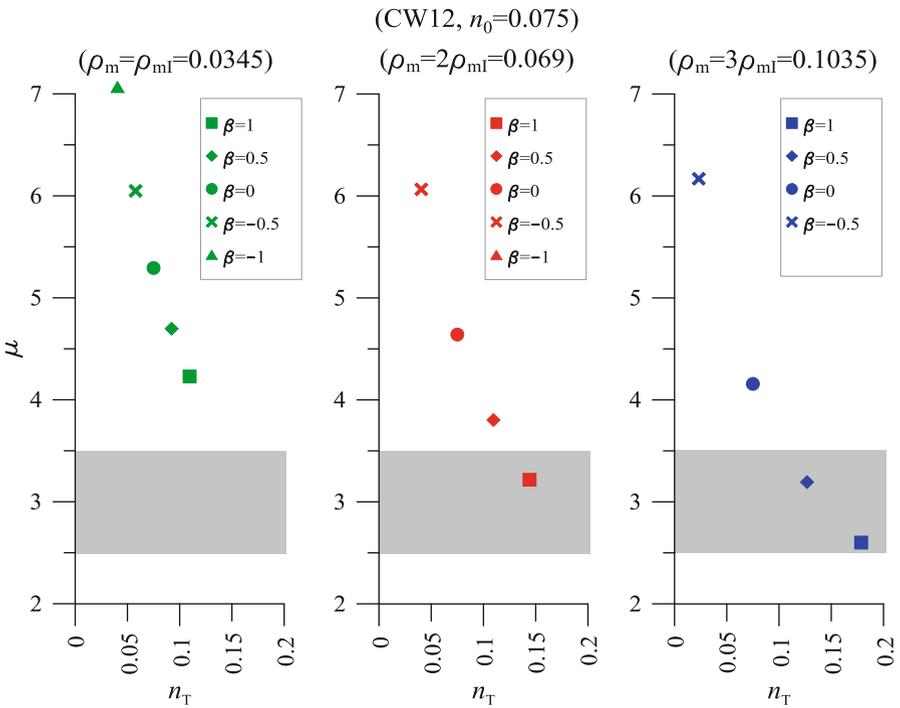


Fig. 9.4 Ductility demand vs total coupling shear for various combinations of wall mechanical reinforcement ratio and relative yield parameter (CW12, $n_0=0.075$)

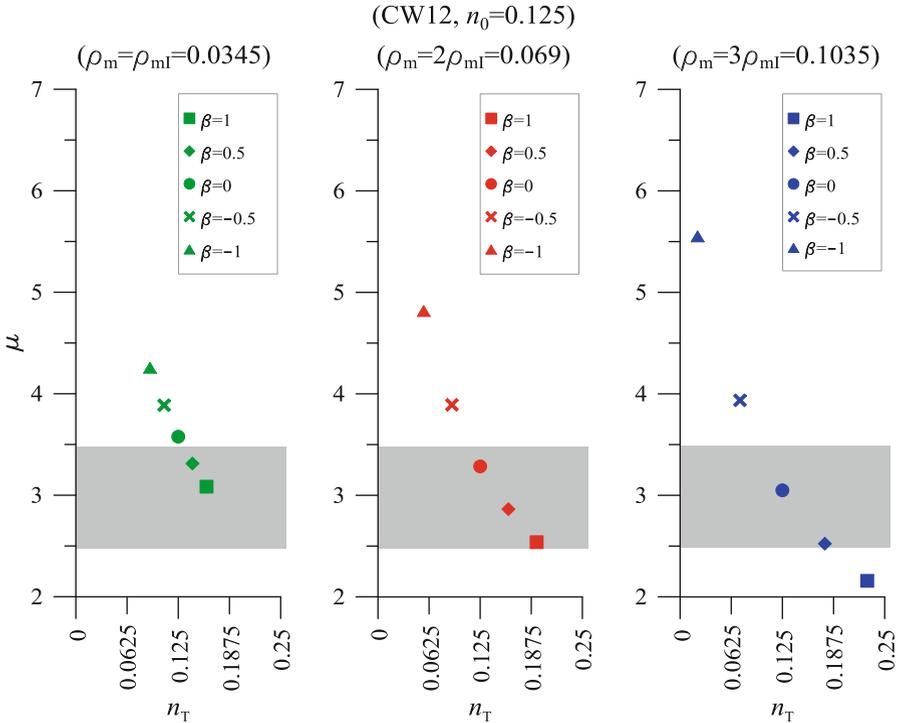


Fig. 9.5 Ductility demand vs total coupling shear for various combinations of wall mechanical reinforcement ratio and relative yield parameter (CW12, $n_0=0.125$)

Total base overturning capacities obtained by the proposed procedure have been confirmed by nonlinear response history analysis performed for a typical parameter set under Chi-chi earthquake (record no: TCU065), whose response spectrum matches well with the typical MCE level spectrum shown in Fig. 9.3. Nonlinear analysis results are shown in Fig. 9.8 in terms of peak base overturning moment normalized with respect to that estimated by the proposed simple procedure versus ductility demand. Acceptable range for the ductility demand ($2.5 \leq \mu \leq 3.5$) is also indicated on the figure.

Following conclusions may be drawn from Table 9.1 and Figs. 9.4, 9.5, 9.6, 9.7, and 9.8.

- (a) As long as concrete crushing is avoided in the compression wall, higher values of wall gravity loads n_0 are beneficial in \supset shaped walls. The outcome would be a direct increase in base overturning moment capacity and decrease in overall ductility demand.
- (b) Contribution of $\beta\rho_m$ to total coupling shear capacity (see Eq. (9.4)) is more pronounced for lower n_0 levels. For higher values of n_0 , contribution of $\beta\rho_m$ remains limited.

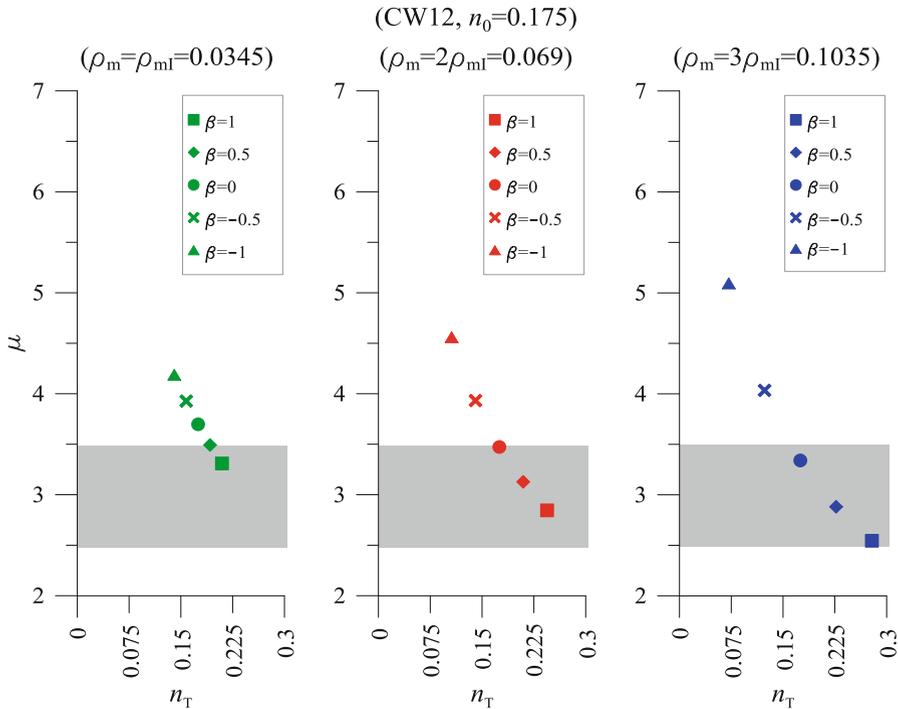


Fig. 9.6 Ductility demand vs total coupling shear for various combinations of wall mechanical reinforcement ratio and relative yield parameter (CW12, $n_0 = 0.175$)

(c) Results show that reinforcement ratio ρ_m as well as relative yield parameter β of the tension wall cannot be selected arbitrarily, as only certain combinations of those parameters would result in acceptable ductility demand levels. Ease of implementation of the proposed simple capacity and ductility demand estimation tools allows the designer to play with the independent parameters to reach an acceptable design configuration with a minimum effort. Implementation details are given in Vuran and Aydınoglu (2015).

9.5 Concluding Remarks

The following remarks can be made to conclude this contribution:

- (a) Preliminary design based on linear analysis may lead to unacceptable sizing and reinforcing of the main structural elements of tall buildings.
- (b) In particular, linear preliminary design procedures applied to coupled core wall systems would most likely lead to an overdesign of coupling beams with inappropriate and heavily congested reinforcement requirements. On the

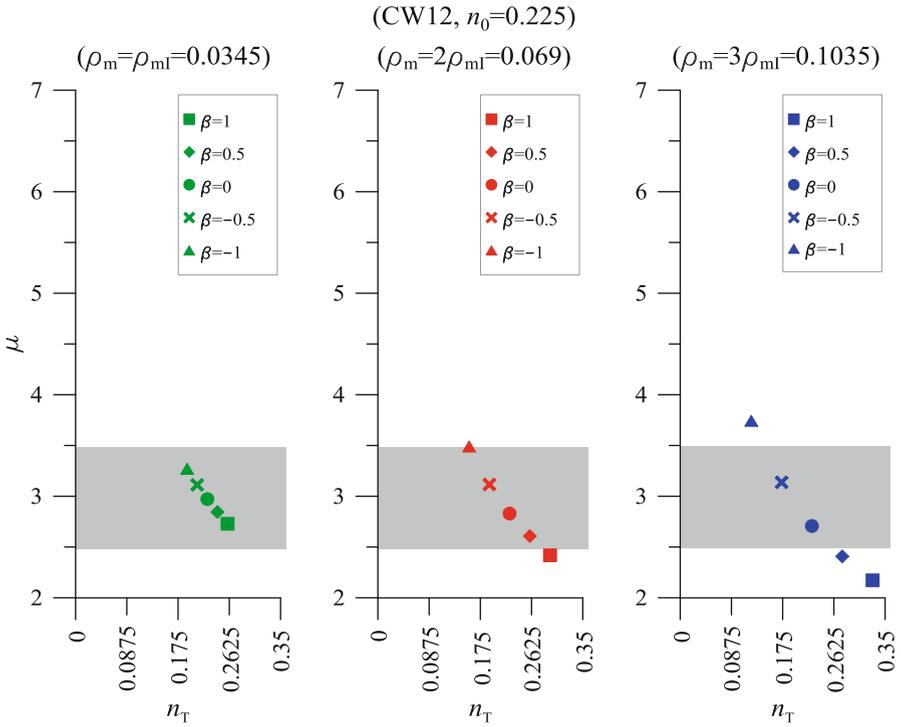


Fig. 9.7 Ductility demand vs total coupling shear for various combinations of wall mechanical reinforcement ratio and relative yield parameter (CW12, $n_0=0.225$)

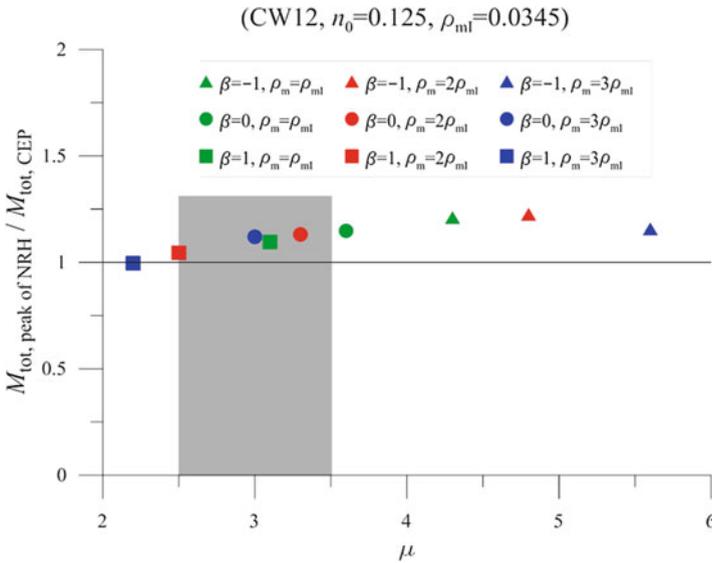


Fig. 9.8 Total base overturning moment capacity obtained from NRHA divided by the same from proposed procedure versus ductility demand (CW12, $n_0=0.125$, Chi-chi earthquake – record no: TCU065)

contrary, linear analysis with reduced seismic loads may result in under-designed wall elements especially in terms of their shear strength.

- (c) Total coupling shear capacity and total base overturning moment capacity of a coupled core wall system can be successfully estimated in the preliminary design stage by a simple procedure, which starts from the “*first principles*” based on limit equilibrium conditions.
- (d) In order to assess the adequacy of total base overturning moment capacity, overall ductility demand of the coupled core wall system can be estimated again by a simple procedure based on an alternate implementation of the pushover concept.
- (e) Since capacity and ductility demand estimation procedures are very easy to implement and not time consuming, several trials can be made during the preliminary design stage by playing with the independent variables to reach an acceptable ductility level.
- (f) A reasonable estimate of the base shear can also be made considering significant amplifications due to higher mode effects and shear migration from the tension wall to the compression wall.

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