Determination of Behavior of Coupled Shear Walls Subjected to Horizontal Forces through Nonlinear Static Methods

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ABSTRACT

Coupled shear walls are one of the systems commonly used in medium and highrise structures to resist lateral forces. Yet these systems should not collapse or be induced severe damage during earthquake actions. For this reason, coupled shear walls must have high strength, high ductility, high energy absorption capacity and high shear stiffness to limit lateral deformations. The recent advances in structural engineering have increased the interest in performance based design. In the study herein, hence, the performance based design of a coupled shear wall system has been carried out. The design has later been checked against nonlinear time history analysis and the design performed has been confirmed to be quite safe. In the second stage of the study, the horizontal capacity of couple shear walls is predicted by the pushover analyses. Though these procedures have been used for different types of structures, they have not been employed for coupled shear walls. The procedures employed are conventional pushover (deformation and forced based), force based adaptive pushover, and deformation based adaptive pushover. The capacity curves obtained through these procedures have been compared with the one determined through Incremental Dynamic Analysis. The evaluation shows that it is almost unlikely to determine the capacity curve of coupled shear walls by the nonlinear static analyses. Nonetheless, the displacement based adaptive pushover analyses has been able to predict the base shear capacity and capture the displacement profile of the system up to a certain level in the nonlinear region.

INTRODUCTION

The efficacy of structural walls in building systems have been recognized for a long time. With a suitable arrangement of wall elements in the plan of a structure, not only these elements can be employed as a very effective load carrying system but also the other functional needs of a structure can be satisfied. It is generally required that spaces be formed systematically in wall elements for elevator, window, door, etc., openings. If these open spaces can be created regularly

and reasonably, it is well possible to obtain structures that have a increased energy absorption capacity and high ductility [1].
Much attention has been paid to performance-based seismic design in earthquake engineering research in the last 20 years [2]. This new method requires designing a building for several expected performance levels associated with different earthquake hazard levels. An important step in performance-based design is to estimate the nonlinear seismic response of buildings. There are two procedures [3]: nonlinear time history analysis and simplified nonlinear analysis (commonly referred to as pushover analysis). The nonlinear time history analysis can provide more realistic results for a given earthquake ground motion. However, such analytical results tend to be highly sensitive to the earthquake input. Thus, the results commonly need to be interpreted statistically. Pushover analysis is not as complicated as nonlinear time history analysis, and can use response spectrum as demand diagram to estimate the seismic response of structures Therefore, it is generally recommended in performance-based design [4].

The design of a 12-story structure with coupled shear walls is carried out in the first part of the study herein. The fundamental steps for such a design are supplied in [5]. Additional equations necessary for a complete design is derived in this study. The building has been tested using a nonlinear dynamic analysis in SeismoStruct program [6]. Having confirmed the performance of the designed structure, the capacity of the coupled shear wall building under horizontal loadings is determined by nonlinear dynamic and static procedures. While the classical pushover analysis, forced-based adaptive pushover analysis and displacement-based pushover analysis has been resorted to as nonlinear static procedures, incremental dynamic analysis is used as dynamic method. Seven artificial ground acceleration records are used as input excitations to analysis methods. The study concludes with a discussion about whether the response of couple shear wall building under horizontal excitations can be captured by nonlinear static procedures.

Displacement-Based Design of Coupled Walls

The elevation of a coupled wall building is shown in Fig. 1(a). Coupled walls typically occur in the configuration suggested in this figure where two symmetrically opposed channel-shaped walls enclosing service facilities such as elevators, stairs and toilets are linked at floor levels by beams. Typically these beams have low aspect ratios L_{CB}/h_{CB}. Hence, they are susceptible to high shear effects which must be carefully considered in the design. The design process proceeds in the following steps:

Step 1: Selection of structural geometry.

The number of stories is 12 and floor height is 3.2 m. Thus, the total height of the building is H_n=12×3.2=38.4 m. Other geometrical and material characteristics are given in the following table.

Wall properties	***Coupling beam properties***	***Material properties***

Wall flange length l_W	5 m	Beam length L_{CB}	1.8 m	f_{ck}	25 MPa
Wall thickness	250 mm	Beam depth h_{CB}	800 mm	f_y	450 MPa
Storey height	3.2 m			f_u/f_y	1.2

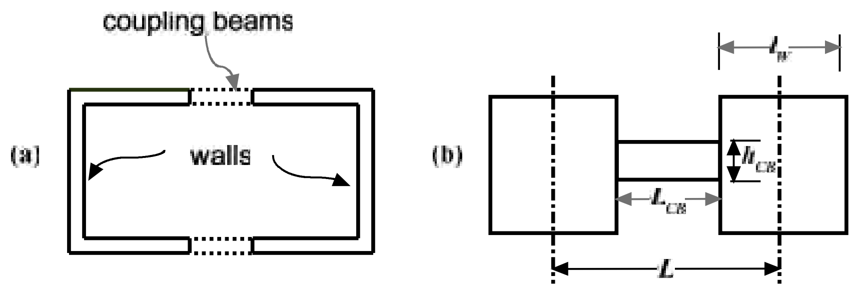

Figure 1a) Coupled structural walls, b) Coupling beams.

Step 2. Chose proportion of overturning moment (OTM) carried by coupling beams.
The degree of coupling between connecting beams and wall, and its influence on response is normally quantified by the ratio of the base moment $M_{CB,B}$ provided by the coupling beams to the total M_{OTM} [7]. Hence,

$$\beta_{CB} = M_{CB,B} / M_{OTM} = \left(\sum_{i=1}^{n} V_{CB,i} \right) L / M_{OTM} \tag{1}$$

where $V_{CB,i}$ are the seismic shear forces in the coupling beams, L is the distance between wall centerlines. The value of β_{CB} is normally in the range of 0.25 to 0.75 and will be chosen such that wall reinforcement ratios are within acceptable bounds. Another consideration is that the value of β_{CB} should not be so large that the shear forces in the coupling beams induce a tension force at the wall base that exceeds the gravity load. In this study, a β_{CB} value of 0.60 is selected.

Step 3. Determine the height of moment contraflexure, H_{CF}.
H_{CF} defines the height at which the drift will be a maximum, since the moment reversal occurring above this point reduces the drift in the upper stories. The building is a regular building, from [5], $H_{CF} = 0.52H_n = 0.52 \times 38.4 = 20.0$ m.

Step 4. Determine the effective height, H_e.
The building is a regular building and, thus, from [5], $H_e = 0.7H_n = 0.7 \times 38.4 = 26.9$ m.

Step 5. Calculate wall yield displacement.
The average yield curvature:

$$\phi_{y,W} = 1.75\varepsilon_y / l_W = 1.75 \times 0.002475 / 5 = 0.000866 \text{ 1/m} \tag{2}$$

where $\varepsilon_y = f_y / E$ (steel yield strain) and $f_y = 1.1 \times 450 = 490$ MPa is the expected steel yield strength. The wall displacement at yield is:

$$\Delta_y = C_4 \phi_{yW} H_n^2 = 0.14 \times 0.000866 \times 38.4^2 = 0.179 \text{ m} \tag{3}$$

In this equation, C_4=0.14 is taken from [5].

Step 6. Calculate yield drift of coupling beams at the contraflexure height.
Either conventional reinforcement or diagonal reinforcement may be used for the design of coupling beams [7]. In this study, coupling beams with conventional reinforcement will be considered. In order to compute the yield drift of coupling beams, strain penetration length and a flexibility coefficient to consider the shear deformations are needed. With a selection of 28 mm diameter bar for coupling beams, strain penetration length:

$$L_{SP} = 0.022 f_{ye} d_{bl} = 0.022 \times 490 \times 0.028 = 0.305 \text{ m} . \tag{4}$$

where f_{ye} and d_{bl} are the yield strength of the reinforcing steel and its diameter, respectively. The flexibility coefficient:

$$F_V = 3(h_{CB} / L_{CB})^2 = 3 \times \left(0.8/1.8\right)^2 = 0.5926 \tag{5}$$

The beam yield curvature is again from [5]:

$$\phi_{y,CB} = 1.7\varepsilon_y / h_{CB} = 1.7 \times 0.002475 / 0.8 = 0.00526 \text{ 1/m} \tag{6}$$

The yield drift of conventionally reinforced coupling beam is then:

$$\theta_{CB,y} = 0.5\phi_{y,CB}\left(0.5L_{CB} + L_{SP}\right)(1 + F_V) = 0.5 \times 0.00526 \times \left(0.5 \times 1.8 + 0.305\right)\left(1 + 0.5926\right) = 0.00504 \text{ rad} \tag{7}$$

Steps 7 and 8. Determine the design displacement.
The wall design displacement may be governed by wall-base material strains, or by wall drift at height H_{CF}. It may also be limited by the material strains in the coupling beams. All three options will need to be examined.

a) Wall-base material strains:
Assuming a limit-state strain of ε_{su}=0.10, the limit-state yield curvature for damage control region:

$$\phi_{dc} = 0.0072 / l_W = 0.0072 / 5 = 0.0144 \text{ 1/m} \tag{8}$$

If 20 mm diameter reinforcing steel is assumed for the walls, strain penetration length for such a reinforcing bar:

$$L_{SP} = 0.022 f_{ye} d_{bl} = 0.022 \times 490 \times 0.020 = 0.218 \text{ m} \tag{9}$$

and plastic hinge length:

$$L_P = 0.2(f_u / f_y - 1)H_{CF} + 0.1 l_W + L_{SP} = 0.04 \times 20 + 0.1 \times 5 + 0.218 = 1.52 \text{ m} \tag{10}$$

Wall-base limit displacement:

$$\Delta_{D\varepsilon} = \Delta_y + (\phi_{ls} - \phi_y) L_P H_e = 0.179 + (0.0144 - 0.00866) \times 1.52 \times 26.9 = 0.723 \text{ m} \tag{11}$$

b) Wall drift limit at H_{CF}:
The maximum wall drift will occur at the contraflexure height H_{CF}. In this case, the design displacement

$$\Delta_{D\theta} = \Delta_y + (\theta_C - 0.5\phi_{yW} H_{CF}) H_e = 0.179 + (0.02 - 0.5 \times 0.00866 \times 20) \times 26.9 = 0.482 \text{ m} \tag{12}$$

c) Coupling beam drift limit:
The standard equations for plastic rotation may be modified and used to predict the coupling beam rotation limit. The plastic hinge length is given by $L_P = k \cdot L_C + L_{SP} \geq 2L_{SP}$. In this equation, L_C is the length from the critical section to the point of contraflexure in the beam. For coupling beams, the low length/depth aspect ratio invariably results in the strain penetration limit governing. That is, L_P=2L_{SP}. Assuming the concrete is well-confined by transverse reinforcement, which will also be necessary to restrain the compression bars from buckling, the tensile strain limit will govern the plastic rotation capacity. Again making an assumption that the distance from the centroid of tensile reinforcement to neutral axis is 0.75h_{CB}, the limit-state rotation capacity of the coupling beam for damage control will be

$$\theta_{CB,ls} \approx \frac{0.6\varepsilon_{su} \times 2L_{SP}}{0.75 L_{CB}} = \frac{0.6 \times 0.1 \times 2 \times 0.305}{0.75 \times 0.8} = 0.061 \text{ rad} \tag{13}$$

and the corresponding critical wall rotation, at H_{CF} will be:

$$\theta_{W,CB} = \frac{\theta_{CB,ls}}{1 + l_W / L_{CB}} = \frac{0.061}{1 + 5/1.8} = 0.0159 \tag{14}$$

Because this value is less than the code drift limit (0.0159<0.02) the case stated by (c) limits the design. Hence, the design displacement:

$$\Delta_{D\theta} = \Delta_y + (0.0159 - 0.5\phi_{yW} H_{CF}) H_e = 0.179 + (0.0159 - 0.5 \times 0.00866 \times 20) \times 26.9 = 0.374 \text{ m} \tag{15}$$

Step 9. Calculate the wall and coupling beam average displacement ductility demands

Wall: $\mu_W = \dfrac{0.374}{0.179} = 2.09$

Coupling beams: $\mu_{CB} = \dfrac{0.0159}{0.00504} = 3.15$,

This is the peak ductility demand, applying at the contraflexure height. The average ductility demand is taken as $0.67\mu_{CB} = 2.11$.

Step 10. Calculate the system equivalent viscous damping.

The damping associated with wall and coupling-beam action is [5]:

$$\xi_W = 0.05 + 0.444\left(\frac{\mu - 1}{\mu\pi}\right) = 0.05 + 0.444\left(\frac{2.09 - 1}{2.09\pi}\right) = 0.124 \qquad (16)$$

$$\xi_{CB} = 0.05 + 0.565\left(\frac{\mu - 1}{\mu\pi}\right) = 0.05 + 0.565\left(\frac{2.11 - 1}{2.11\pi}\right) = 0.145 \qquad (17)$$

The equivalent viscous damping for the whole system is

$$\xi_{sys} = (1 - \beta_{CB})\xi_W + \beta_{CB}\xi_{CB} = (1 - 0.6)\times 0.124 + 0.6\times 0.145 = 0.1377 \qquad (18)$$

Step 11. Determine the required base shear force and overturning moment by direct displacement based design principles.

For an equivalent viscous damping ratio of 0.1377, the displacement spectrum reduction factor:

$$R_\xi = \left(\frac{0.07}{0.02 + \xi_{sys}}\right)^{0.5} = \left(\frac{0.07}{0.02 + 0.1377}\right)^{0.5} = 0.666 \qquad (19)$$

For ξ=0.1377, corner period displacement:

$$\Delta_{T_c = 5\ \text{san},\ \xi_{sys} = 0.1377} = 1\ \text{m} \times R_\xi = 0.666\ \text{m} \qquad (20)$$

The system effective period:

$$T_e = T_c \frac{\Delta_d}{\Delta_{T_c = 5\ \text{san},\ \xi_{sys} = 0.1377}} = 5.0\frac{0.374}{0.666} = 2.81\ \text{san} \qquad (21)$$

Effective mass is determined using

$$m_e = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} \qquad (22)$$

In this equation, m_i is the mass of the *i*th floor, Δ_i is the displacement of *i*th floor and Δ_d is design displacement. It is therefore necessary to compute the design deformation of each floor in order to determine the effective mass. These values can be computed in a way similar to that of system design displacement. The yield displacement of each floor needs to be calculated first and then the plastic displacement displacements of each storey need to be determined. Summing these two quantities yields the storey displacements. Storey yield displacements:

$$\Delta_y = \left[\frac{0.4015}{1-\beta_{CB}} \left| \left(H_i - 0.263 H_n \right) \right| - \left(\frac{\beta_{CB}}{1-\beta_{CB}} \right) \left| \left(H_i \left(\frac{0.5}{n} + 0.5 \right) - H_n \left(\frac{1}{4n} + \frac{1}{6} \right) \right) \right| \right] \phi_{yW} H_n \tag{23}$$

Table 1 shows the yield and design displacements of each floor. The building effective mass is:

$$m_e = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} = \frac{m}{\Delta_d} \sum_{i=1}^{n} \Delta_i = \frac{5/g}{0.374} 3.659 = 48.7 \text{ MN}/g \tag{24}$$

Table 1. Computation of storey design displacements.

Storey	$\boldsymbol{H_i}$ **(m)**	$\boldsymbol{\Delta_{yi}}$ **(m)**	$\boldsymbol{\Delta_{pi}}$ **(m)**	$\boldsymbol{\Delta_{di}}$ **(m)**
12	38.4	0.2663	0.2780	0.544
11	35.2	0.2459	0.2548	0.501
10	32	0.2256	0.2317	0.457
9	28.8	0.2052	0.2085	0.414
8	25.6	0.1849	0.1853	0.370
7	22.4	0.1645	0.1622	0.327
6	19.2	0.1442	0.1390	0.283
5	16	0.1238	0.1158	0.240
4	12.8	0.1035	0.0927	0.196
3	9.6	0.0831	0.0695	0.153
2	6.4	0.0627	0.0463	0.109
1	3.2	0.0424	0.0232	0.066
Total				**3.659**

Effective stiffness:

$$K_e = m_e \left(\frac{2\pi}{T_e} \right)^2 = 48.7 \left(\frac{2\pi}{2.69} \right)^2 = 243.49/g \ \text{ MN/m} = 24.82 \text{ MN/m} \tag{25}$$

Design base shear:

$$V_{\text{base}} = K_e \times \Delta_d = 9.28 \ \text{ MN} = 9280 \ \text{ kN} \tag{26}$$

The base shear force is distributed to the floor levels in proportion to the product of mass and displacement as:

$$F_i = V_{\text{base}} \frac{m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{27}$$

From the floor level forces, storey shear forces and moments are calculated. The procedure is summarized in the following table. Design base overturning moment (M_{OTM}) is computed to be 243.5 MN.m.

Table 2. Story shear forces and base overturning moment.

Storey	H_i (m)	F_i (MN)	V_i (MN)	$M_{OTM,i}$ (MN.m)
12	38.4	1.380	1.380	0.000
11	35.2	1.270	2.650	4.417
10	32	1.160	3.810	12.898
9	28.8	1.049	4.859	25.089
8	25.6	0.939	5.798	40.638
7	22.4	0.828	6.626	59.191
6	19.2	0.718	7.344	80.395
5	16	0.608	7.952	103.897
4	12.8	0.497	8.450	129.344
3	9.6	0.387	8.837	156.382
2	6.4	0.277	9.113	184.659
1	3.2	0.166	9.279	213.821
Total			V_{base}=9.279	M_{OTM} = 243.515

Step 12. Calculate the seismic shear to be carried by each coupling beam.

With β_{CB}=0.6, the shear force to be carried by each beam is given by, noting there are two coupling beams at each level,

$$V_{CB,i} = \frac{\beta_{CB} M_{OTM}}{2nL} = \frac{0.6 \times 243.5}{2 \times 12 \times (5 + 1.8)} = 0.895 \text{ MN} = 895 \text{ kN} \tag{28}$$

Step 13. Calculate the required strength for each coupling beam and design the beams.

By demanding that the strength of the coupling beams at each floor is the same, the required strength:

$$M_{CB,i} = 0.5 V_{CB,i} L_{CB} = 0.5 \times 895 \times 1.8 = 805.5 \text{ kN} \cdot \text{m} \tag{29}$$

This strength can be provided by 4Φ20 bars.

Step 14. Design the wall-base flexural reinforcement.

The amount of overturning moment acting on each wall:

$$M_{W,OTM} = (1 - \beta_{CB}) M_{OTM} / 4 = 24.35 \text{ MN} \cdot \text{m} \tag{30}$$

The axial force in each wall is N_{ave}= 5750 kN. Therefore,

$$\text{Axial load ratio } = \frac{N_{ave}}{f_{ck} A_g} = \frac{5750}{32500 \times 0.25 \times 5} = 0.142 \qquad (31)$$

$$\text{Dimensionless moment} = \frac{M_{W,OTM}}{f_{ye} b_W l_W^2} 1000 = \frac{24350}{495000 \times 0.25 \times 5^2} 1000 = 7.6 \qquad (32)$$

Based on the above design values, a steel ratio of ρ=0.01 is needed. This can be provided by 20Φ28 bars.

Checking the displacement based design of coupled wall system

The response of the 12-storey coupled shear wall building designed above will be determined under earthquake excitations. The structure is modeled using SeismoStruct program. It is a fiber-element based program for seismic analysis of framed structures. The program is capable of predicting the large displacement behavior and the collapse load of framed structural configurations under static or dynamic loading, accounting for geometric nonlinearities and material inelasticity. The nonlinear response of coupled wall system is determined using a time history analysis. For this purpose, seven artificially generated earthquake acceleration records have been used. The behavior of the structure under each of these records has been computed and, then, the average of the maximum story displacements and story drifts are determined. The average values are consequently compared with those obtained in the design process of the building. Figure 2 shows the computed design displacements and the mean story displacements determined from nonlinear time history analyses. It is observed that the design of the building seems satisfactory. Figure 3 also indicates that the average peak floor drifts are lower than design values.

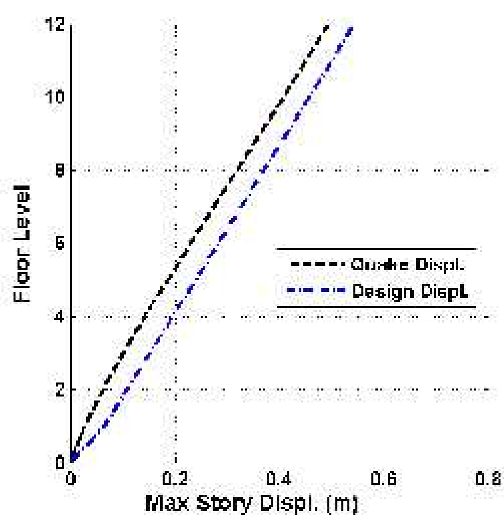

Figure 2 Maximum story displacements

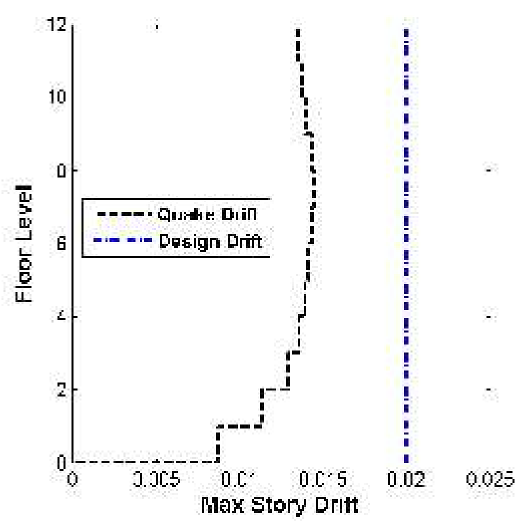

Figure 3 Maximum story drifts obtained in

obtained in the dynamic analyses and design process.	the dynamic analyses and design process.

Determination of the Capacity Curve of Coupled Wall Building by Pushover Analyses

The applicability of pushover analysis procedures in determining the seismic response of coupled shear walls to earthquake forces can be figured by comparing the capacity curve obtained from the nonlinear static procedures with the “exact” curve computed from the nonlinear time history analysis. To this end, it is first necessary to obtain the true capacity curve. Incremental dynamic analysis method [8] is used for this purpose.

Incremental Dynamic Analysis - IDA

Seven generated earthquake acceleration records that have been used previously will also be employed to compute the structural capacity curve. They are scaled such that the responses of the structure in both elastic and damaged states are captured. In other words, the magnitude of earthquake intensity can be arranged by scaling the records. For each earthquake time history, about 20 scaling levels are utilized. Hence, the nonlinear dynamic analysis of the structural model under the ground motion records is performed 7x20=140 times. From an IDA curve, some statistical quantities can be extracted given the structural model and a statistical population of records. Because the capacity curve is determined in this study only, the base shear force and top story displacement are needed to be computed. Various alternatives have been presented in literature to obtain the shear force-top displacement couple: 1) maximum top displacement and shear force at the time instant corresponding to maximum displacement, 2) maximum top displacement and maximum base shear, and 3) maximum base shear and top displacement at the time instant corresponding to maximum base shear. This study considers the first two options. Figures 4 and 5 show the capacity of the coupled wall building under each earthquake. The curve of the former figure is obtained for the maximum top displacement vs. the corresponding base shear, and the latter for the maximum top displacement vs. maximum base shear. The average of these curves that will be used in comparison with the result of the IDA is shown in Figure 6. On the same figure, the design base shear value (9280/2=4640 kN) is plotted.

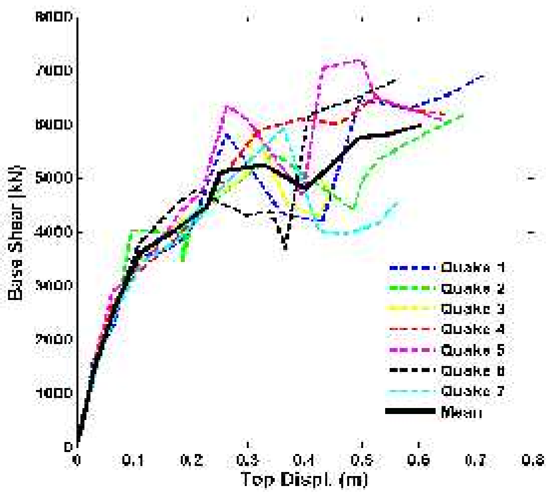

Figure 4 The capacity curve of the structure under each earthquake excitation (Max. top displacement vs. the corresponding base shear)

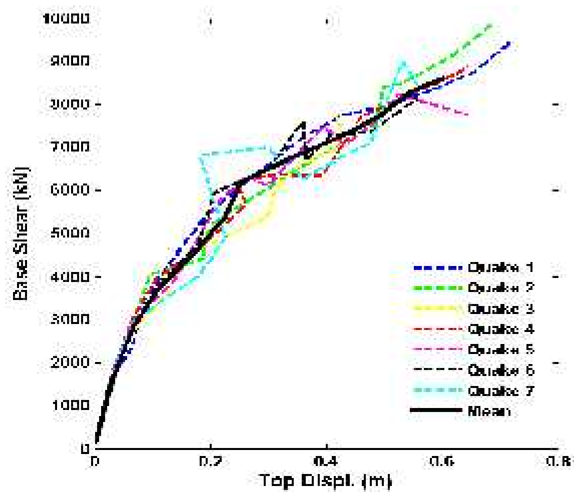

Figure 5 The capacity curve of the structure under each earthquake excitation (Max. top displacement vs. max. base shear)

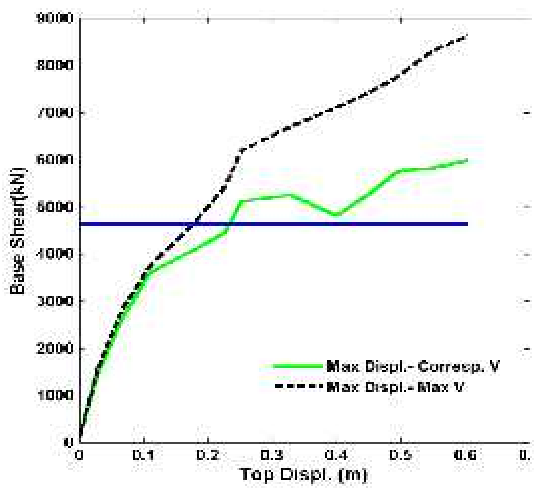

Figure 6 The average capacity curves and the design base shear value.

Conventional Pushover Analyses - CP

Conventional pushover analysis methods consist of pushing a given structure under monotonically increasing lateral load pattern in one direction. Both the force distribution and target displacement are based on the assumptions that the response is controlled by the first mode of vibration and the mode shape remains unchanged until incipient collapse occurs. Two lateral load patterns, that is, the first mode proportional and the uniform, are recommended to approximately bound the likely distribution of the inertia forces in the elastic and inelastic range, respectively [9].

A number of studies [10] raise doubts on the effectiveness of these conventional force-based pushover methods in estimating the seismic demand throughout the full deformation range: 1) inaccurate prediction of deformations when higher modes are important and/or the structure is highly pushed into its nonlinear post-yield range, 2) inaccurate prediction of local damage concentrations, responsible for changing the modal response, 3) inability of reproducing peculiar dynamic effects, neglecting sources of energy dissipation such as kinetic energy, viscous damping, and duration effects, 4) difficulty in incorporating three-dimensional and cyclic earthquake loading effects.

Due to the type of structure under investigation, it is expected that these kinds of conventional pushover procedures will not produce sound results as they do for regular buildings. The following figures present a comparison of capacity curves that have been obtained using force-based and displacement-based conventional pushover analyses with the “exact” capacity curve. Note that especially the difference between the exact curve and that of displacement-based pushover analysis is quite big.

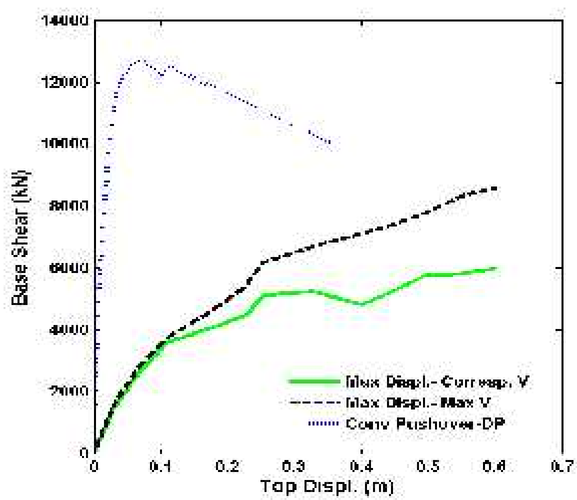

Figure 7 Capacity curves from displacement-based conventional pushover and IDA.

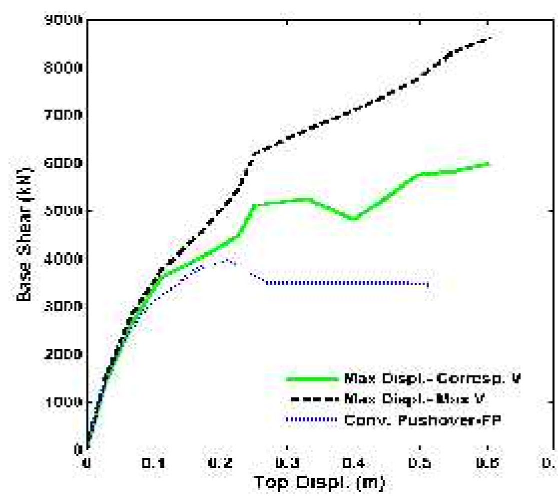

Figure 8 Capacity curves from force-based conventional pushover and IDA.

Force-based Adaptive Pushover Analysis - FAP

To overcome the shortcomings of the conventional pushover analysis procedures, a number of researchers have proposed the so-called adaptive pushover methods. Amongst them is the procedure established by [11] and later modified and developed by [12]. The procedure is single-run, fully adaptive, multi-modal and accounts for system degradation and period elongation by updating the force distribution at every step of the analysis. The implementation of the procedure can be structured in four main stages; 1) definition of nominal load vector and inertia mass, 2) computation of load factor, (3) calculation of normalized scaling vector and 4) update of loading force vector. Whilst the first step is carried out only once, at the start of the analysis, its three remaining counterparts are repeated at every equilibrium stage of the nonlinear static analysis method. The dynamic properties of the structure are determined by means of eigenvalue analyses that consider the instantaneous structural stiffness state, at each analysis step. Site or record specific spectral shapes can also be explicitly considered in the scaling of forces, so as to account for the dynamic amplification that the expected ground motion might have on the different vibration modes of the structure. 7 previously used artificial earthquake records are also utilized in this method to determine the pushover curve. The capacity curve is shown in Figure 9. The figure also presents the curve of IDA procedure and the design base shear quantity. It is observed that the FAP method, yielding reasonable results for moment-dominant normal and regular buildings, does not produce a curve close to the exact capacity curve of coupled shear wall building. Neither the design base shear nor the deformation characteristics of the building could be estimated with this method.

Displacement-based Adaptive Pushover Analysis - DAP

In this nonlinear static procedure, the loading is applied to the structure in terms of displacements instead of forces. The capacity curve obtained from this method is given in Figure 10. The average curve is the statistical mean of capacity curves determined according to the seven scaled artificial records. It is seen from the figure that the DAP accurately estimates the displacement profile of the structure up to the design base shear. After that point on, the building assumes no further displacements and the execution of the analysis is stopped by the program.

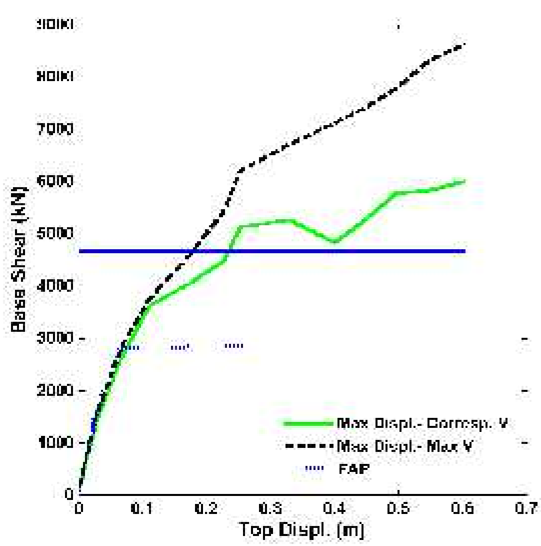

Figure 9 Capacity curves determined by FAP and IDA.

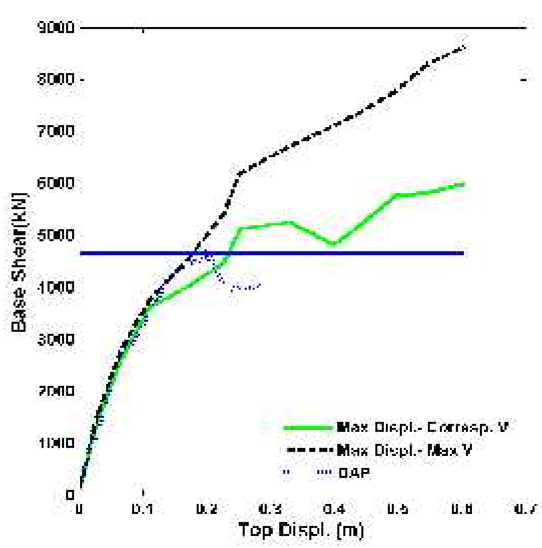

Figure 10 Capacity curves determined by DAP and IDA

conclusion

In the study herein the displacement-based design of coupled shear wall building is carried out and the prediction of horizontal load capacity of the building is made by nonlinear static procedures. First, the design of the coupled wall structure is carried out using the principles of recently popular performance based design. The aim of the displacement based design is to design a structure so that it can achieve a specified deformation state under the design-level earthquake, rather than achieve a displacement that is less than a specified displacement limit. In this regard, the deformation limit states for the wall and coupling beam elements are defined in terms of material strain limits and code specified drift limits. The yield and plastic displacements of the building is computed and then the design displacement of the structure is found. From that point on, the structure is transformed into a single degree of freedom system and the corresponding structural period and stiffness are determined. Later, the design base shear and overturning moment of the building are computed. The design overturning moment is distributed between the wall and coupling beam elements based on the designer's choice, not dictated by the initial stiffness of coupling beams and walls. Depending on the element forces and moments, the reinforcing of the members is performed and consequently the design process is ended. The mentioned process is explained by a step by step fashion on a chosen example building. The safety of the design process is tested by carrying out a nonlinear dynamic analysis of the building subjected to artificially generated earthquake time histories.

The second part of the study examines the applicability of the nonlinear static procedures in determining the horizontal loading capacity of the designed building. First of all, the "exact" capacity of the building is obtained by incremental dynamic analysis. In computing the capacity curve, two alternatives are considered: the maximum displacement vs. the corresponding base shear and the maximum displacement vs. the maximum base shear, which may not necessarily take place at the same time instants. The same artificial excitation records that have been used in checking the design process are also utilized in the nonlinear dynamic analysis. The intensity scale of the earthquake records are changed from a low value (corresponding to the elastic region) to high value (corresponding to highly nonlinear region). This curve of base shear response vs. top displacement has been compared with those determined from the nonlinear static procedures. The comparison of the curve determined using the displacement-based conventional pushover analysis with the "exact" capacity shows that this method is not able to predict the capacity of the coupled shear wall building. Similarly, force-based conventional pushover analysis could not estimate the capacity curve. However, it is seen that the results of force-based conventional pushover analysis have become better. Afterwards, the adaptive pushover analyses have been tried. The force-based adaptive pushover analysis yielded reasonable results when the earthquake intensity is low, however, as the intensity level is increased, the results started to diverge from the exact ones. That is, the procedure could not provide the accurate design displacement and design base shear of the building. Lastly, the resulting capacity curve of displacement-based adaptive pushover analysis has been compared with the exact capacity curve. It is seen that the last procedure could reasonably follow the exact capacity curve up to a point where the top displacement corresponds to the design base shear. From this point on, the analysis could not assume further displacement and analysis ends. Therefore, the procedure is not able to estimate the remaining part of the exact capacity curve, which corresponds to displacements obtained under the earthquakes of high intensity levels.

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