Displacement Based Seismic Design of Torsionally Unbalanced Shear Wall Structures

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SUMMARY:
The present study proposes a simple displacement based design method for torsionally unbalanced shear wall buildings in which the global displacements are limited to control the level of structural and non-structural damage, to keep the ductility demands on structural elements to within their ductility capacity, and to prevent second order instability. The seismic hazard is represented by a design inelastic response spectrum corresponding to the ductility demand derived from the global values of acceptable ultimate and yield displacements. At the stage of preliminary design, the values of such displacements are estimated using empirical expressions. In subsequent iterations, pushover and moment-curvature analyses are carried out to refine these estimates. On convergence, a multi-modal pushover analysis is carried out to account for higher modes effect. For the design of a multistory building the latter is represented by an equivalent 2-degree-of-freedom system. The proposed procedure is applied to the design of a 12-storey asymmetric shear wall building.

Keywords: Performance based design, torsionally unbalanced, shear wall buildings, pushover analysis

1. INTRODUCTION

A method of design that seeks to achieve but not exceed a limiting value of the lateral displacement in a structure subjected to seismic ground motion is referred to as displacement based seismic design (DBSD) and is an essential component of performance based design. This is mainly because the ability of a structural system to withstand strong ground motion depends on its ductility and its capacity to dissipate energy and the demand on these capacities is related to the displacements and drifts experienced by the system.Various DBSD procedures have been reported in the literature for buildings that are generally regular in plan and elevation. However, limited number of studies exist on the DBSD of torsionally unbalanced buildings. In such buildings the coupling of the translational and torsional response to strong motions causes the displacements in the lateral resisting elements to increase significantly.

The challenge in DBSD for torsionally unbalanced shear wall structures is to estimate the displacement demand on different shear walls in the plan. In order to estimate such displacement demands, one needs to know the displacement shape or shapes in which the building vibrates. Because of the nonlinear behavior such shapes change continually with time. In spite of this the use of a set of lower order elastic mode shapes presents a reasonable approach to the estimation of displacements in the elements of the structure. In the present study the first or in some cases the first two elastic mode shapes of the building are used to estimate the displacements in the various lateral load-resisting structural elements of the building.

2. DISPLACEMENT BASED SEISMIC DESIGN

A displacement based seismic design method for shear wall buildings has been presented by Humar (2008). Fazileh (2011) has proposed an extension to the method so as to incorporate the rotational effect due to asymmetry in plan of such buildings. These modifications focus on obtaining approximate estimates of the yield and acceptable ultimate displacement on different structural elements and the equivalent values of these displacements at the center of mass assuming that the building vibrates predominantly in the first or the second elastic mode shape. The proposed method is applied in the present study to the seismic design of buildings that are unsymmetrical about the Y axis but symmetric about the X axis. Figure 2.1 shows the plan view of such a building. The earthquake ground motion is assumed to act in the Y direction

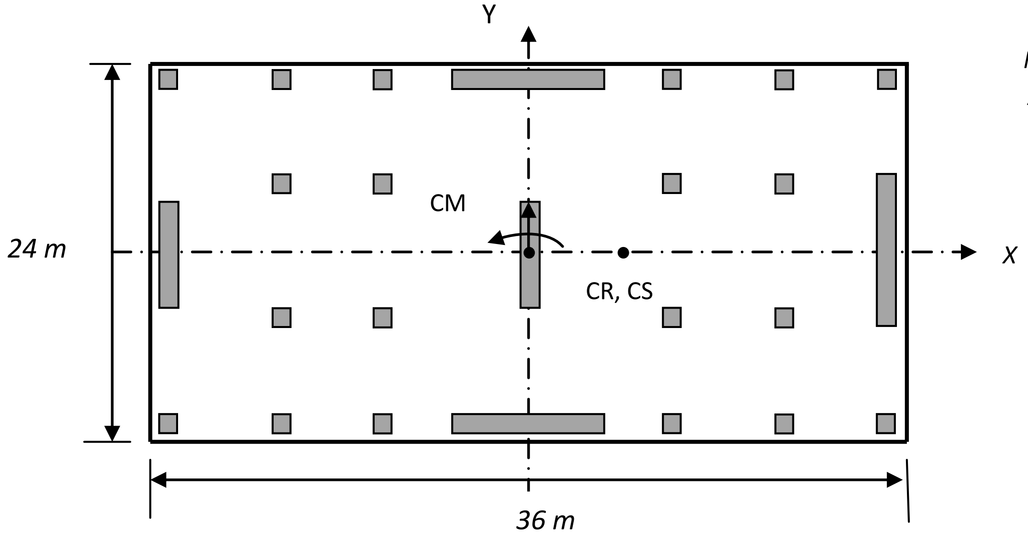

Figure 2.1 Plan view of a torsionally unbalanced building

Figure 2.2 shows the deformation shapes for the first two modes of a single-story building with torsional unsymmetry. For a multi-story building the figure represents the translational and rotational components at the roof level. The angle of twist ψ shown in the figure is defined as being equal to the torsional component of the elastic mode shape when it is normalised by its translational component. The first mode shape deforms in such a way that the flexible side of the building has a larger displacement than the stiff side, while in the second mode the displacement at the stiff edge is greater than that at the flexible side.

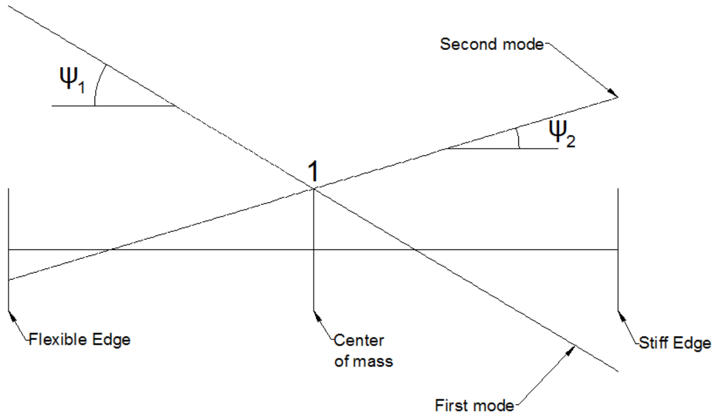

Figure 2.2 . Normalised deformed shape in first and second mode shapes

In the design of a particular building whether the first mode shape or both the first and the second mode shapes govern the design depends on the nature of torsional unbalance. Thus a building is referred to as torsionally stiff if the period of its first torsion dominant mode is considerably shorter than that of its first translation dominant mode, which therefore becomes the first mode of the building. Only the first mode needs to be considered in the design of such a building. A building is said to be torsionally similar when the translation and torsion dominant modes have period that are close. A torsionally flexible building is one in which the first mode is torsion dominant and the period of the next mode which is translation dominant is considerably shorter. In the design of the torsionally similar and torsionally flexible buildings both the first and the second modes need to be considered.

2.1 Equivalent yield and ultimate displacements

The yield and ultimate displacements at the roof level of of individual shear walls in an asymmetric plan building can be estimated using empirical relations given below (Paulay 2002).

$$\phi_y = 2.0\varepsilon_y / l_w \qquad \Delta_y = \phi_y H^2 / 3 \tag{2.1}$$

$$\theta_{pd} = 0.025 - \theta_y = 0.025 - \phi_y H/2 \qquad \theta_{p\mu} = (\phi_u - \phi_y) \times L_p \tag{2.2}$$

$$\Delta_u = \Delta_y + (H - L_p/2) \times \theta_p \tag{2.3}$$

where ϕ_y is the yield curvature, ε_y is the steel strain at yield, l_w is the length of the wall, Δ_y is the displacement at the top of the wall at yield, H is the height of the wall, θ_{pd} is the plastic rotation capacity as governed by the code-specified interstory drift of 0.025, $\theta_{p\mu}$ is the plastic rotation as governed by the ductility capacity of the wall, ϕ_u is the acceptable ultimate curvature in the concrete section, L_p is the length of plastic hinge in the wall taken as being equal *to* $l_w/2$, θ_p is either θ_{pd} or $\theta_{p\mu}$, and Δ_u is the acceptable ultimate displacement at the top of the wall.

Since the lateral and torsional responses are coupled in an unsymmetrical plan building, the yield displacements of individual walls are reached at different displacements at the center of mass. The displacement at the center of mass at the instant plane *i* yields, Δ^*_{yi}, is obtained from

$$\Delta^*_{yi} = \frac{\Delta_{yi}}{1 + x_i \psi} \tag{2.4}$$

where, Δ_{yi} is the yield displacement of plane *i*, and x_i is the distance of plane *i*, from the center of mass. Based on the proportion of the base shear that is assigned to the individual walls, V_i, the global yield displacement can be estimated as follows:

$$\Delta_y = \frac{\sum V_i}{\sum (V_i / \Delta^*_{yi})} \tag{2.5}$$

This yield displacement estimate is used in the preliminary design. During subsequent iterations the global yield displacement is obtained from a pushover analysis.

The acceptable ultimate displacement in a building is assumed to be the minimum of the lateral displacements under which (1) the compressive strain in any concrete shear walls reaches a specified limit, for example, 0.004, (2) the maximum drift ratio along the periphery of the building reaches the code specified limit of, say 0.025, and (3) the reduction in shear capacity of the system owing to P–Δ effect is about 5% of the maximum shear capacity. The first of the three criteria defines the ductility capacity of the wall, while the last one is specified to guard against the possibility of instability in the system. At the preliminary design stage only the second limit can be obtained; the other two limits can be evaluated when a pushover analysis is carried out.

Corresponding to Δ_{ui}, the ultimate displacement of wall *i*, the displacement at the center of mass is given by:

$$\Delta^*_{ui} = \frac{\Delta_{ui}}{1 + x_i \psi} \tag{2.6}$$

The acceptable ultimate displacement of the system is the minimum of all Δ^*_{ui} .

In torsionally unbalanced buildings, the maximum interstory drift occurs at the edge of the building. For minimizing the damage to non-structural components this drift should be limited to the value prescribed in the codes. In those cases where a lateral resisting element is located at the edge of the building, the drifts can be estimated using empirical relations given in Eqns. 2.1 through 2.3. However, if the lateral resisting elements are not located at the boundary of the building, the maximum acceptable displacement to prevent the drifts from exceeding the drift limit is calculated using the mode shapes at the stage of preliminary design, and from a pushover analysis in the subsequent iterations.

At the stage of preliminary design, mode shapes are obtained from the relative stiffnesses of the resisting elements. The drift ratio at *n*th story, θ_n is then obtained by taking the difference between the displacements of the *n*th and the (n-1)th story and dividing it by the height of *n*th storey, h_n giving

$$\theta_n = \frac{(\phi_y^n + x\ \phi_\theta^n) - (\phi_y^{n-1} + x\ \phi_\theta^{n-1})}{h_n} \tag{2.7}$$

where x is the distance of the edge of the building from the center of mass, ϕ_y^n is translational component of the mode shape at *n*th storey, and ϕ_θ^n is the rotational component of the mode shape at n^{th} storey. The ultimate displacement at the center of mass that would produce a drift ratio of 0.025, is then calculated from:

$$\Delta_u = \frac{0.025}{\theta_n}\phi_y^n \tag{2.8}$$

The ultimate displacement at which the drift at the flexible edge will reach its acceptable limit is calculated using the first mode shape, while the ultimate displacement at which the drift at the stiff edge will reach its acceptable limit is calculated using the second mode shape. In subsequent iterations where pushover analyses are carried out, the drifts are recorded for both the flexible and the stiff edges of the building. The equivalent ultimate displacement at center of mass is the minimum displacement when the recorded drift in any plane reaches the code specified drift limit.

2.2 Equivalent single-degree-of-freedom system

To proceed with the DBSD of a multi-story unsymmetrical building it needs to be represented by an equivalent two-degree-of-freedom (2DOF) system. This is accomplished by using the first or the second mode shape and the following expressions

$$\Gamma_n = \frac{\boldsymbol{\varphi}_{yn}^T \mathbf{m} \boldsymbol{\varphi}_{yn}}{\boldsymbol{\varphi}_n^T \mathbf{M} \boldsymbol{\varphi}_n} \tag{2.9}$$

$$M_n^* = \frac{\left(\boldsymbol{\varphi}_{yn}^T \mathbf{M} \boldsymbol{\varphi}_{yn}\right)^2}{\boldsymbol{\varphi}_n^T \mathbf{M} \boldsymbol{\varphi}_n} \tag{2.10}$$

$$I_{On}^* = \Gamma_n \boldsymbol{\varphi}_{\theta n}^T \mathbf{I}_o \mathbf{1} \tag{2.11}$$

where $\boldsymbol{\varphi}_{yn}$ is the translational component of $\boldsymbol{\varphi}_n$, the *n*th mode, $\boldsymbol{\varphi}_{\theta n}$ the corresponding rotational component, **M** is the mass matrix comprised of the submatrices of story masses **m** and the story moments of inertia $\mathbf{I}_o$, **1** is a vector with 1s along Y degrees of freedom and zeroes elsewhere, Γ_n is the modal participation factor, M_n^* the modal mass for the nth mode, and I_{On}^* is the modal static response for base torque. The yield and ultimate target displacements for the equivalent SDOF system are given by

$$\delta_y = \Delta_y / \Gamma_1 \quad \text{and} \quad \delta_u = \Delta_u / \Gamma_1 \tag{2.12}$$

2.3 Inelastic demand spectrum

The inelastic demand spectrum for a given ductility provides the value of spectral acceleration S_{ay}, such that when the structure having the corresponding yield strength is subjected to the design earthquake the ductility demand is equal to the specified value of ductility μ. The ratio of the elastic spectral acceleration S_a to S_{ay} is denoted by R_y. The construction of inelastic demand spectrum from the known elastic design spectrum requires the definition of a relationship between R_y, μ, and the period T. A number of empirical relationships between the three are available in the literature; of these the following by Krawinkler and Nasser (1992) is used in the present work.

$$R_y = [c(\mu - 1) + 1]^{1/c}, \qquad c = \frac{T^a}{1 + T^a} + \frac{b}{T} \tag{2.13}$$

and parameters a and b depend on the post-yield stiffness. In the present work it is assumed that the force-displacement relationship is elasto-plastic for which a = 1 and b = 0.42.

2.4 Base shear and base torque

Having determined the ultimate displacement of the 2DOF system for the design mode(s), the corresponding spectral acceleration (A_y) is obtained from the inelastic acceleration displacement response spectrum for the ductility demand calculated from the ultimate and yield displacements. The base shear (V_b) and the base torque (T_b) are then given by

$$V_b = M^* A_y, \qquad T_b = I_O^* A_y \tag{2.14}$$

The design base shear is distributed among the resisting planes in proportions determined by the designer. The resulting actions of these lateral forces and the accompanying gravitational loads are used to design the resisting planes along the axis of unymmetry. The base torque on the other hand is assumed to be resisted by orthogonal planes. The demand on orthogonal planes is calculated from

$$T_{orth} = T_{bn} + T_{asym} \tag{2.15}$$

where, T_{orth} is the total torque to be resisted by orthogonal planes, and T_{asym} is the torque arising from asymmetry created when assigning the base shear to resisting planes.

2.5 Moment-curvature analysis

For carrying out modal analysis and push over analyses, as required in subsequent iterations of design, more refined estimates are needed for the yield and ultimate curvatures of the walls, the strengths of the walls, and the effective moments of inertia of the walls. This is accomplished by carrying out moment-curvature analyses of the wall sections. Such analyses are performed using the material characteristics and strain compatibility condition. The resulting moment-curvature relationship is idealized by a bilinear curve from which estimates of the curvatures, strengths, and effective moments of inertia of the walls are obtained.

2.6 Modal Push over analyses

Once a preliminary design has been obtained a push over analysis for a force distribution proportional to the selected mode shape is carried out to obtain a better estimate of the global yield displacement as well as an estimate of the limiting ultimate displacement to preclude the chance of instability under P-Delta effect. If these values are close to those in the previous iteration the design can be assumed to have converged otherwise a new iteration is carried out.

The lateral forces to be used in the push over analysis are given by

$$\boldsymbol{s}_n = \begin{Bmatrix} \boldsymbol{s}_{yn} \\ \boldsymbol{s}_{\theta n} \end{Bmatrix} = \boldsymbol{\Gamma}_n \begin{Bmatrix} \mathbf{m}\,\phi_{yn} \\ \mathbf{I}_O \phi_{\theta n} \end{Bmatrix} \tag{2.16}$$

where, $\boldsymbol{s}_{yn}$ is a Nx1 vector containing the lateral forces along the Y axis at each floor level, and $\boldsymbol{s}_{\theta n}$ is a Nx1 vector containing the rotational torques at each floor level. These forces and torques are applied at the center of mass and displacement control is also applied at the center of mass.

2.6 Multi-mode push over analysis

On convergence, the DBSD procedure based on the first mode or the first two modes provides good estimates of the design moments and drifts. However, the shear forces are not accurate, and the contribution of the higher modes must be considered for obtaining better estimates of such forces. In many cases, the multi-mode push over analysis (MPA) proposed by Chopra and Goel (2002, 2004) provides a simple and reasonably accurate method of considering the contribution of higher modes. The MPA procedure uses push over analyses based on the first few mode shapes and combines the modal responses so obtained assuming that they are uncoupled. Often, the higher modes respond in an elastic manner, so that the response of higher modes may be obtained from an elastic response spectrum.

Pushover analysis is carried out during each iteration of design using the updated section properties. The analysis provides a pushover curve showing the relationship between the total base shear and the roof displacement at the center of mass. The global yield displacement and the ultimate displacement up to which instability will not occur in the structure are determined from the pushover curve. The pushover data also provides the ultimate displacement at which the maximum drift still remains within the code prescribed drift limit. Also provided are the interstory drift ratios at the boundaries of the structure at each step of the pushover analysis, from which one can obtain the displacement at the center of mass when the first recorded drift ratio reaches the code drift limit of 0.025.

The results of pushover analysis for the design mode(s) when pushed up to the corresponding ultimate displacement obtained in the last iteration of the design are combined with the results of the higher mode pushover analysis in which the structure is pushed up to the demand calculated from the following expression.

$$D_n = \Gamma_n \frac{T_n^2}{4\pi^2} A_n \tag{2.18}$$

where, D_n is the target displacement for *n*th mode, and An is the spectral acceleration corresponding to T_n , the period of the *n*th mode. Equation 2.17 is based on the assumption that the structure remains elastic in the higher mode pushover analyses. If this assumption is found not to be valid, an iterative process is used to find the performance point from inelastic capacity demand curves.

The results obtained from the individual modes can be combined using either the SRSS rule or the CQC rule. The CQC rule is more reliable when periods of the different modes are close to each other. Although it is apparent that this combination would provide only an approximate estimate of the coupled response of the higher modes, particularly when the structure has been strained into the inelastic range, comparisons of the results from a nonlinear dynamic analysis with those of multi-mode pushover analysis have shown that the multi-mode pushover analysis can provide an acceptable estimate of the seismic demand on asymmetric plan building as well.

3. PROPOSED DBSD PROCEDURE

The design procedure for torsionally unbalanced system with unsymmetry along one axis starts with the design of the planes along the axis of symmetry. Because of the symmetry no torsion is induced by the lateral vibration along that axis. The planes can therefore be designed by using a procedure similar to that proposed by Humar (2008). The structure is next designed along the axis of unymmetry. During iterations for the design of planes along that axis, the demand on planes in the orthogonal direction due to rotation of the floors is calculated. If the capacities of the designed orthogonal planes are less than the calculated demands on them, such planes are re-designed. The iterations continue until convergence is achieved in the design of planes along the axis of asymmetry. The steps in the DBSD of planes along the axis of asymmetry are illustrated through an example presented in the next section.

4. CASE STUDY

The proposed DBSD procedure is applied to the design of a 12-storey torsionally unbalanced building whose plan layout is shown in Fig. 2.1. The floor dimension is 36 meter along the X axis, and 24 meter along the Y axis. The first story is 4.85 m in height and the rest are 3.65 m high. The structural framing consists of 200 mm thick RC flat slab on 500 mm by 500 mm columns. Lateral resistance is provided by two 6.0 m by 0.4 m walls in the X direction and three walls in the Y direction. Of the Y direction walls the west and center walls are each 5.0 m by 0.4 m, while the east wall is 7.0 m by 0.4 m. Floor dead load is 5.8 kPa, the live load is 2.4 kPa, and the snow load is taken as 2.2 kPa. The ultimate strength of concrete is 30 MPa and the yield strength of steel 400 MPa. The modulus of elasticity of concrete is taken as 24,500 MPa. The building is located in the city of Vancouver and the design spectrum is obtained from the uniform hazard spectrum for 2% chance of exceedance in 50 years for Vancouver, as provided in NBCC 2005 (Canadian Commission 2005).

The inertial masses and mass moments of inertia are calculated for the dead plus 25% of snow load, while the gravity loads are obtained for the load combination D+0.L+0.25S using the tributary areas for individual walls. Following results are obtained: first floor mass m = 675.3 tonne and mass moment of inertia I_0 = 105,347.0 tonne.m^2; typical floor m = 650.9 and I_0 = 101,546.0; roof m = 629.3 and I_0 =981,741.0; gravity load at base for 7-m wall 6506.3 kN; for 5-m edge wall 5688.4 kN, for 5-m center wall 9281.4 kN.

Step 1: Determine the governing mode shape(s)

The design of planes parallel to the axis of symmetry is not described here. For design of the planes parallel to the axis of unsymmetry it is assumed that base shear in the Y direction is distributed in proportion to the wall length, so that 30% is assigned to each of the 5-m walls and 40% is assigned to the 7-m wall. The relative stiffnesses of the three walls will be proportional to the square of the wall lengths. For preliminary design the same assumption is made for the 6-m wall. An eigenvalue analysis of the building based on these relative stiffnesses reveals that the deformed configuration based on the first mode shape imposes larger relative displacement on both the flexible side wall and the stiff side. Thus, only the first mode needs to be considered in the design. The angle of twist for this mode is found to be ψ_1 = -0.021.

Step 2: Calculate the global yield and ultimate displacements from those for individual walls

Preliminary estimates of the yield and ultimate displacements of individual walls are obtained from Eqns. 2.1 through 2.3 and are reported in Table 4.1. Also shown there are the equivalent values of the two sets of displacements at the centre of mass (CM) as obtained from Eqns. 2.4 and 2.6. The global yield displacement at the CM is calculated from Eqn. 2.5 and is found to be 0.509 m. The global ultimate displacement is the minimum of the equivalent ultimate displacements for the three walls, namely 0.614 m. The corresponding ductility demand is μ = 0.614/0.509 = 1.207

Table 4.1: Ultimate and yield displacements of walls and equivalent values at the mass center

Parameter	5-m Flexible	5-m Center	7-m Stiff
ϕ_y 1/m	8E-04	8E-04	5.71E-4
Δ_y m	0.540	0.540	0.385
Δ_u m	0.846	0.846	0.911
Δ^*_y m	0.392	0.540	0.620
Δ^*_u m	0.614	0.846	1.465

Step 3: Calculate the dynamic properties of the design mode(s) and obtain equivalent displacements
The dynamic parameters for the first mode are calculated from Eqns. 2.9 through 2.11 and are shown in Table 4.3 in the row marked preliminary. The yield and ultimate displacements of the equivalent 2DOF system are 0.509/1.488 = 0.342 m and 0.614/1.488 = 0.413 m.

Step 4: Construct the inelastic demand spectrum and obtain the base shear and base torques
The inelastic demand spectrum corresponding to the ductility of 1.207 is constructed and expressed in the acceleration-displacement format. The spectrum is entered with the displacement of 0.413 m and gives A_y = 0.0647 g. The base shear and base torque are now obtained from Eqn. 2.14 and are V_b = 3250.8 kN and T_b = 10659.1 kNm.

Step 5: Distribute the base shear and base torque among individual walls and design the wall sections
The base shear is assigned to the individual walls in the proportions assumed earlier. The shears are distributed along the height according to the mode shape used and the moments at the bases of the walls are determined. The wall sections are designed for the combined effect of the base moments and the axial gravity loads. Properties of the designed sections are obtained from moment-curvature analysis.

The total torque resulting from the calculated base torque and asymmetry of the resisting shears in the walls must be resisted by the orthogonal walls. The total torque is 10659.1 + (0.4-0.3) × 3250.8 × 18 = 16510.0 kNm. The resultant shear in each orthogonal wall is 106510/24 = 687.9 kN. This shear is distributed across the height of the wall. The distributed forces cause a base moment of 23317.8 kN. The orthogonal walls are checked to verify whether they have adequate capacity to resist this moment and, if necessary, their sections redesigned.

Step 6: Carry out a pushover analysis using the base shear and torque determined in Steps 4 and 5
A pushover analysis is carried out for the story force and torque distributions obtained in Step 5. Figure 4.1 shows the resulting pushover curves with and without the P-Δ effect. The global yield displacement is found to be 0.445 m. The 95% of the maximum base shear is reached at a roof displacement of 0.549 m, which becomes the acceptable ultimate displacement to preclude instability. The push over data provides a value of 0.557 m for the ultimate displacement at CM when the drift at the flexible edge plane reaches 2.5%,

Step 7: Using the section properties determined in Step 5 calculate new modal properties
The revised modal properties are determined using the effective section moments of inertia obtained in Step 5; they are shown in Table 4.3 under the title 1st iteration. The angle of twist ψ_1 = -0.0243.

Step 8: Using the results of Steps 5, 6, and 7 find acceptable ultimate displacement
Using the new value of ψ_1 and the section properties determined in Step 5 the values of the yield displacements and acceptable ultimate displacements as governed by the ductility capacity as well as the corresponding displacements at the CM are calculated. The computations are shown in Table 4.2.

The global yield displacement as computed from Eqn. 2.5 is 0.459 m as compared to the value of 0.445 m obtained from the pushover analysis. On comparing the global displacements, namely the one controlled by drift, 0.557 m, that controlled by instability, 0.549 m, and that controlled by ductility capacity 0.673, the limit corresponding to instability is seen to govern.

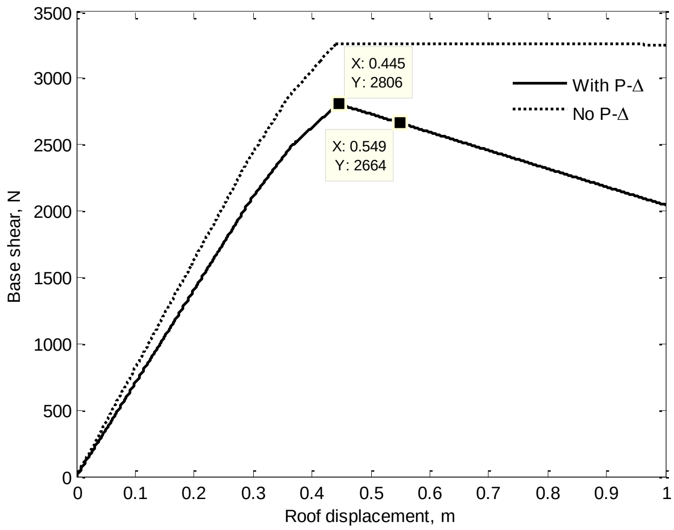

Figure 4.1 First mode pushover curves

Table 4.2: Wall yield and ultimate displacements as governed by ductility capacity

Parameter	5-m Flexible	5-m Center	7-m Stiff
ϕ_y 1/m	7.14E-04	7.47E-04	4.85E-04
ϕ_u 1/m	5.15E-03	3.65E-03	4.39E-03
Δ_y m	0.482	0.504	0.327
Δ_u m	0.967	0.822	0.918
Δ^*_y m	0.335	0.504	0.582
Δ^*_u m	0.673	0.822	1.632

Step 9: Using the governing value of acceptable ultimate displacement find shear and torque

Corresponding to the global yield displacement value of 0.445 m and the global ultimate displacement of 0.549 m the equivalent values for the 2DOF system are determined and are shown in Table 4.3. The ductility is now found to be 0.403/0.326 = 1.234. The inelastic spectrum for this ductility is entered with the ultimate displacement of 0.326 m to provide Ay = 0.0646g. The new base shear and base torques are calculated and are also shown in Table 4.3

Step 10: Repeat Steps 5 through 9 until convergence

Starting from the new values of base shear and base torque Steps 5 through 9 are repeated. The new base shear demand of 2550.8 compared with the capacity of 2586 kN determined from the last pushover analysis. The process is therefore taken to have converged.

Table 4.3: Details of the equivalent 2DOF systems for the first mode

	Γ	M*	I_0^*	δ_y (m)	δ_u (m)	μ	V_b (kN)	T_b (kN.m)
Preliminary	1.488	5118.5	-164644	0.342	0.413	1.207	3250.8	-10659.1
1st iteration	1.363	4677.7	-174019	0.326	0.403	1.023	2964.4	-11241.6
2nd Iteration	1.370	4701.9	-169507	0.325	0.434	1.335	2550.8	-9373.7

After the convergence of the base shear, a multi-mode pushover analysis is carried out to investigate the contribution of higher modes to the design parameters. The first 6 modes that account for 92.2% of the modal mass are included in the analysis. The first mode pushover is carried out to the target displacement of 0.594 m determined during the last iteration. The response in the higher is found to be elastic and their target displacements are determined from Eqn. 2.17. The results are presented in Table 4.3 and 4.4. As can be seen in Table 4.4, the higher modes have significant effect on the estimates of base shear, but the effect on drifts, displacements and rotation of the plan is negligible.

Table 4.4: Dynamic parameters in multi-mode pushover analyses

Dynamic Parameters	First mode	Second mode	Third mode	Fourth mode	Five mode	Sixth mode
M^*/M_{total}	0.6020	0.0521	0.1841	0.0159	0.0624	0.0054
Γ	1.3702	0.1186	0.6624	0.0574	0.3377	0.0292
T (sec)	4.22	2.74	0.67	0.44	0.24	0.16
Sa (g)	0.0553	0.1466	0.5990	0.8083	1.0511	1.1000
D_{target} (m)	0.594	0.0325	0.0446	0.0022	0.0051	0.0002

Table 4.5: Modal response parameters and their combination

	Base shear (kN)	Roof displacement at C.M (m)	Rotation at roof level	Maximum drift (%)
1st mode	2,586.9	0.594	0.0112	2.2677
2nd mode	565.4	0.0325	0.0093	0.4191
3rd mode	7,558.2	0.0446	0.0010	0.7187
4th mode	972.3	0.0022	0.0006	0.1052
5th mode	6,172.9	0.0051	1.17×10^{-4}	0.1930
6th mode	489.31	0.0002	5.13×10^{-5}	0.0178
SRSS	8,008.6	0.59656	0.01459	2.4155
CQC	10,286	0.59826	0.01496	2.4486

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When both the first and the second modes govern design, the above steps need to be supplemented with additional analysis. The computations in Steps 1 through 4 and Steps 6 through 9 must be carried out for both the first and the second mode. From the set of values of base shear and base torque determined in Step 4 the more critical combination will govern the design of individual walls. If the walls are not located at the edges of the building Step 2 should include computation of acceptable ultimate displacement using Eqn. 2.7 to ensure that drifts at the edges do not exceed the code-prescribed value. In similar manner in the analysis referred to in Step 6 the pushover data should be used to determine the acceptable ultimate displacement for limiting the edge drifts.

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