A DISPLACEMENT-BASED ANALYSIS AND DESIGN PROCEDURE FOR STRUCTURAL WALLS.

H.G. URREGO[1] and R.L. BONETT[2]

[1] *AREA Ingenieros Consultores. Colombia*
Professor, Dept. of Civil Engineering, University EAFIT, Medelĺin, Colombia
[2] *Professor, Dept. of Civil Engineering, University of Medellin, Colombia*
Email: hurrego@une.net.co, rbonett@udem.edu.co

ABSTRACT :

In this paper, a procedure for displacement-based design is presented. The method assumes a geometrical section of known flexural reinforcement at the critical section. Using concrete and reinforcing steel properties and axial forces, the moment capacity is obtained by equilibrium of the section. The curvature ductility and the displacement corresponding to the first yield of the extreme bar in tension and ultimate deformation of the concrete are calculated from the strain profile of the section. The design spectrum in ADSR format is used to determine the ductility demand and it is compared with the capacity of the structure. Thus, it is possible to define whether is necessary to change the initial reinforcement proposed until structural capacity is equal or bigger than the seismic demand. When this iterative procedure concludes, the section is satisfactorily designed because the analyses, as well as the design, are made simultaneously. Finally, the maximum displacement and curvature ductility are calculated. The method has been applied to a 15-storey cantilever structural wall building. The results indicate that the procedure is able to satisfy the design objectives and fulfill a non-linear deformation pattern.

KEYWORDS: Performance-based design, displacement based design method, displacement ductility demand and capacity, structural walls, RC buildings.

1. INTRODUCTION

During the past decade, there has been considerable advances in the definition of a general framework for performance-based seismic design, in terms of seismic demands, admissible levels of damage (structural and not structural) and in the definition of the performance objectives to guarantee the stability, operation of the structure and the safety and integrity of its occupants (SEAOC, 1999). Displacement-Based Design methods, DBD, began in the early 1990s as a means to design structures with the ability to predict more narrowly structural damage states. In recent years, a variety of these methods have been developed (Moehle, 1992; Kowalsky et. al, 1994; Medhekar and Kennedy, 2000; Priestley and Kowalsky, 2000; Xue, 2001; Panagiotakos and Fardis, 2001; Kowalsky, 2002; Sullivan et.al., 2006; Restrepo and Preti, 2006; Panagiotou, 2008). Nevertheless, the majority of the proposed methods present the following limitations: 1) they use the concept of lateral equivalent stiffness, not taking into account the different stages of the materials behavior, which does not permit to verify the fulfillment of the performance objectives, 2) they do not take into account the dynamic effects of higher modes on the flexural and shear design, 3) they do not consider the kinematic overstrength effect and 4) most of them are devised just for single degree of freedom systems. In this work, a design procedure is proposed for reinforced concrete structural walls, which allows incorporating the capacity of the section among the variables and considers on an explicit way the effect of higher modes on the flexural and shear design. The proposed method has been applied to a 15-storey building located in the city of Los Angeles (California). The obtained results show that the proposed procedure allows the fulfillment of the proposed performance objectives.

2. DISPLACEMENT-BASED DESIGN (DBD)

During the last years, the conceptual framework of Performance-Based Design, PBD, has been largely developed. The definition of the performance design objectives in terms of expected levels of damage resulting from expected levels of earthquake ground motions has been proposed. Displacement-based design methods are recognized as excellent options to use within a PBD due to the ability to predict structural damage states. An interesting number of DBD approaches have been developed over the last decades. A state-of-the-art report was issued on this topic (Fib, 2003). Normally, these DBD procedures do not consider the behavior of the section element when it responses to external loads in the variables that are include in the structural analysis. These methods suppose that ductility and plastic rotation can be imposed to the structure and with these values, the displacement and forces demanded can be obtained. Thus, the required reinforcement is calculated. Nevertheless, it is necessary to confront if the section has the ductility and rotation that were supposed. In addition, the dynamic effects of higher modes are not considered explicitly.

3. DBD PROCEDURE

The procedure here presented has been developed for Multi-Degree-of-Freedom Systems adopting the idea of an equivalent three degree-of-freedom system, which allows including the effect of higher modes. Only the flexural effect is considered in order to determine the displacements due to seismic loads. It does not consider the effects due to the rotational inertia of the mass supported by the structural system. The proposed DBD procedure can be described in a global way by the following steps:

- Definition of the performance objectives in terms of the seismic demand and the performance levels.
- Geometric pre-dimensioning of the structural walls, assumption of an initial reinforcement ratio, the longitudinal and the transversal reinforcement, and the confinement in the zones anticipated as regions of higher compression stress.
- Calculation of the flexural capacity of the wall (bilinear representation of the moment – curvature diagram) considering the axial level force. Definition of the plastic length in such a way that a certain displacement ductility should be ensured.
- Generation of a three DOF system to consider the effect of higher modes.
- Calculation of the displacement ductility capacity of the wall.
- Determination of the displacement ductility demand for the required seismic hazard level.
- Definition of the final reinforcement and geometry to satisfy the demand needed for the performance levels, considering the effect of higher modes.

3.1 Performance Objectives, Earthquake Design Levels and Performance Levels.

In this work, three performance objectives are considered: operational, life-safety and near collapse, which are associated with three Earthquake Design Levels: Service Earthquake (SE), Design Earthquake (DE), and the Maximum Earthquake (ME), respectively, represented in this case for an elastic response spectrum. Initially, in order to control the structural damage, the structural parameters have been defined which are shown in Table 1. The adopted values have been chosen based on the information given by the Committee Vision 2000, experimental test and the experience of the authors in the post-earthquake evaluation of structures and in moderate and high seismic hazard zones.

Table1. Relation of Performance Objectives, Earthquake Design Levels, and structural control parameters.

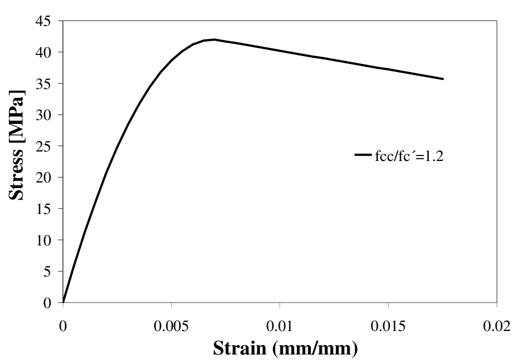
Performance Level	Earthquake Design Level	Inter-story drift	Confined concrete strain, ε_c	Steel reinforcement strain, ε_s
Operational	Service Earthquake	< 0.5 %.	$\varepsilon_c < 0.004$	$\varepsilon_s \leq \varepsilon_y$
Life-Safety	Design Earthquake	< 1.5 %.	$\varepsilon_c < 0.015$	$\varepsilon_s < 0.010$
Near to collapse	Maximum Earthquake	< 2.5 %.	$\varepsilon_c < 0.015$	$0.010 < \varepsilon_s < 0.08$

3.2 Pre-dimensioning and Assumption of the Reinforcement Ratios.

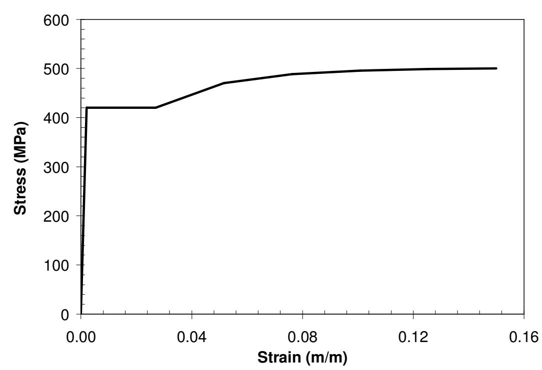
The proposed method supposes an initial geometry of the transverse section of the wall, using the following criteria: a) the height-to-length ratio (H/L_w) must be less or equal to 4.0, b) the storey height-to-wall width (h_n/b_w) must be less or equal than 25 and, c) and ultimate axial force P_u, less than $0.2 \cdot f_c^{'} \cdot A_g$, to guarantee displacement ductility greater than 3. Thus, the fulfillment of the performance objectives can be attained. The method begins from supposing 1% and 0.3% of longitudinal and transversal reinforcement ratios, respectively. The confining effect on the stress-strain relationship of the concrete is considered, but only it is taken into account to increase the ultimate compression strain. The increase in the compression strength is not considered because it can generate a spalling of the unconfined concrete reducing the original section.

3.3 Wall Capacity and Plastic Hinge Length

Starting from the geometry, the moment capacity of the section is evaluated through the moment – curvature diagram. A bilinear representation is adopted which is defined by two states: 1) the first yielding of the steel tension and 2) the maximum compression strain of the confined concrete. The stress-strain curves used in order to calculate the moment – curvature diagram are shown in Figure 1.



Concrete stress-strain curve



Steel Reinforcement

Figure 1. Stress-Strain curves.

Initially, the plastic hinge length is defined as 10 % of the wall height. Nevertheless, this condition must be verified and fixed for the three performance objectives, guaranteeing for every element that the reinforcement does not reach the yield point on the rest of its length.

3.4 *Equivalent three Degrees of Freedom System. Higher Modes.*

For a system where the mass can be considered as uniformly distributed along the element height, this procedure proposes to concentrate the masses in four points distributed at equal distance along the element height (see Fig. 2). Thus, a MDOF system can be transformed into an equivalent three DOF system. When the periods and modal combination are calculated using this simplification, the error is less than 10% and 5%, respectively. Therefore, the proposed simplification is considered as proper to be adopted in the DBD procedure. In order to simplify the modal analysis, a linear distribution is proposed for the first mode and a tri linear distribution for the second and third modes (see Figure 2). From these distributions, the lateral forces are determined for each mode, as a proportion to the masses and the components of the modal shape in the four points defined along the height. By equilibrium, the maximum values of acceleration and displacement for each mode are obtained.

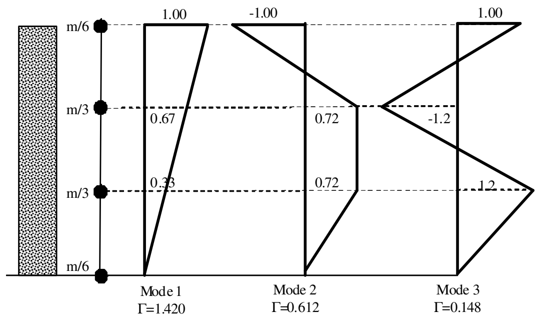


Figure 2. Scheme of the equivalent three DOF system and proposed simplified modal shapes.

Subsequently, the corresponding values for the first mode are shown in the following equations:

$$L_n = \sum m_j \Phi_{j,n} \quad n = 1,3\ ; j = 1,4 \quad ; \quad L_1 = 1.0 \cdot \frac{m}{6} + \frac{2}{3} \cdot \frac{m}{3} + \frac{1}{3} \cdot \frac{m}{3} = \frac{1}{2} m \tag{1}$$

$$M_n = \sum m_j \left(\Phi_{j,n}\right)^2 \quad n = 1,3\ ; j = 1,4 \quad ; \quad M_1 = 1.0^2 \cdot \frac{m}{6} + \left(\frac{2}{3}\right)^2 \cdot \frac{m}{3} + \left(\frac{1}{3}\right)^2 \cdot \frac{m}{3} = \frac{19}{54} m \tag{2}$$

$$\Gamma_n = \frac{L_n}{M_n} \quad ; \quad \Gamma_1 = \frac{L_1}{M_1} = \frac{54}{38} = 1.42 \tag{3}$$

Where m is the total mass of the element and Γ_1 is the modal participation factor of the first mode of vibration. The shear force V_{b1} and the moment M_{b1} at the base are obtained by equilibrium and they are related to the first yield of the tension reinforcement:

$$V_{b1} = \frac{1}{2} \Gamma_1 \cdot m \cdot S_{ay} \quad ; \quad M_{b1} = M_y = \frac{19}{54} \Gamma_1 \cdot m \cdot S_{ay} \cdot H \tag{4}$$

$$S_{ay} = \frac{54}{19} \frac{M_y}{\Gamma_1 \cdot m \cdot H} \tag{5}$$

Where S_{ay} is the pseudo-acceleration, termed here acceleration, associated with the first yield of the reinforcement, which represents the demand of acceleration as well as the capacity of the inelastic system, and M_y is the yielding moment.

$$\Delta_{1,\max} = \frac{145}{513} \varphi \cdot H^2 \approx \frac{\varphi H^2}{3.54} \quad \text{and} \quad \varphi \le \varphi_y \tag{6}$$

Modal participation factors and the obtained periods for the equivalent 3 DOF system are shown in Table 2.

Table 2. Modal participation factors and periods of the equivalent three DOF system.

Mode	Modal Participation factor, Γ	Period, T_i (s)	Participation mass
First	1.420	$T_1 = T_y = 2\pi \sqrt{\Delta_y / (S_{ay} \Gamma_1)}$	0.710 **m**
Second	0.612	$T2 = Ty / 6$	0.192 **m**
Third	0.148	$T3 = Ty / 16$	0.024 **m**

3.5 *Displacement Ductility capacity for the Wall*.

The initial displacement ductility capacity, μ_Δ, is obtained as the relation between the maximum displacement (Δ_u) and the yield displacement (Δ_y). Both displacements are obtained from the curvatures distribution along height, by means of the following expressions:

$$\Delta_y = \frac{145}{543}\varphi_y \cdot H^2 \quad ; \quad \Delta_p = L_p \cdot (\varphi_u - \varphi_y) \cdot \left(H - \frac{L_p}{2} \right) \tag{7}$$

$$\mu_\Delta = 1 + \frac{\Delta_p}{\Delta_y} \tag{8}$$

3.6 ***Displacement Ductility Demand.***

The displacement ductility demand is obtained in terms of the reduction factor, R_μ, due to the hysteretic energy dissipation of ductile structures, the characteristic period of the ground motion, T_C, and the period of each mode, when the moment in the base overcomes the yielding moment, T_i. Equations (9) and (10) are used to calculate the displacement ductility demand, μ_d and they represent a simplified version of the formula proposed by Vidic et al., (1994). For medium and long periods, Equation 10.a applies the equal displacement rule and, for short periods, the constant acceleration zone, equation 10.b. is used.

$$\begin{aligned} \mu_d &= R_\mu & T_i &\geq T_C \\ \mu_d &= (R_\mu - 1)\frac{T_C}{T_i} + 1 & T_i &< T_C \end{aligned} \tag{9}$$

$$R_\mu = \frac{S_{ae}(T_i)}{S_{ay}} \tag{10}$$

3.7 ***Comparison between the capacity and the demand.***

It is necessary to compare the capacity and the demand in terms of displacement ductility, maximum inter-story storey drift, and strains in the confined concrete and longitudinal steel reinforcing (see Table 2). Thus:

If μ_d is less than μ_c, the design is satisfactory. Nevertheless, in order to optimize the design, the reinforcement can be reduced and the plastic hinge length must be fixed.

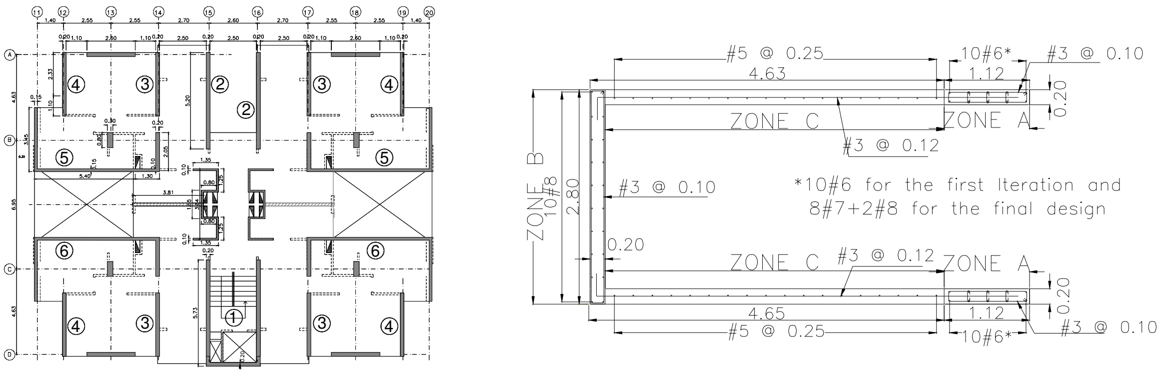
- If the μ_d is bigger than μ_c, it is necessary to increase the plastic hinge length.
- If the maximum inter-storey drift value for the three performance levels is not fulfilled, it is possible to choose one of the following alternatives or a combination of these: 1) modify the geometry, 2) modify the conventional reinforcement and/or 3) increase the compression of the element using post-tensioned reinforcement.

3.8 Definition of the final reinforcement.

The forces associated with each mode are defined considering the ductility demand for each one. In general, for the second and third mode displacement ductility equal to 1.0 is used. Nevertheless, when the bending moment in the base for some of two modes reaches the yielding point, it is necessary to apply the same procedure described in the section 3.8. The final forces that the structure must resist are obtained as a combination of the three modes of vibration, through the method of the square root of the sum of the squares (SRSS). Then, for these forces, the capacity of the wall must be verified in order to fulfillment the performance objectives.

4. EXAMPLE OF DESIGN.

The DBD procedure explained in the previous section was applied to a structural wall that belongs to a 15-storey building shown in Figure 3a. The building has a height of 37.50 m and a floor area of 340 m^2 per level. The performance objectives are shown in table 2. The Figure 3b shows a detail of the wall 1, which was designed using this procedure.



a) Plan View of 15-story building b) Plant view of structural wall

Figure 3. Detail of the building and of the structural designed wall.

4.1. Design parameters for the building

The concrete nominal compressive strength is 35 MPa and the nominal yield strength of the steel is 420 MPa with an over strength of 20 % for both. The initial array of the reinforcement appears in the Figure 3b. The stress-strain relationships are shown in Figure 1. The maximum compression strain of the concrete is limited to 0.010. A dead load of 4.8 kN/m^2, a live load of 2.0 kN/m^2, and the next combination: 0.75 x [1.4 x Dead Load + 1.7 x Live Load] were used. The tributary area of the wall is 48 m^2, which implies a maximum axial load of 7935 kN. The mass of the wall is 580.8 kg. Initially L_p was 2.5 m (Inter storey height).

4.2. Capacity of the wall

Table 3 shows the bilinear representation of the moment-curvature. From these values, the displacements are obtained for both states, defining the plastic hinge length as the equivalent to one storey height of the building (L_p = 2.5 m). The maximum shear strength of the wall is equal to 3594 kN. In figure 3.b is shown the initial reinforcement and the reinforcement for the second iteration. In the third one, L_p was modified to 0.50 m.

Table 3. Moment capacity of two zones of the wall.

Zone	ϕ_y	ϕ_u	M_y	M_u	Δ_y	Δ_u	μ_Δ
A	0.506 x 10^{-3} rad/m	29.650 x 10^{-3} rad/m	38611 kN-m	43548 kN-m	0.201 m	0.744m	3.70
B	0.625 x 10^{-3} rad/m	10.170 x 10^{-3} rad/m	42456 kN-m	50695 kN-m	0.248 m	0.688 m	2.77

4.3. Seismic demand

The Design Earthquake, DE, corresponds to the one defined by the UBC-97 for a soil type SB, a seismic source type B, a nearby field, with a seismic acceleration and velocity coefficient C_a and C_v equal to 0.4. For the SE and ME earthquake levels the design spectra were factored by 0.5 and 1.5 respectively (see Fig. 4).

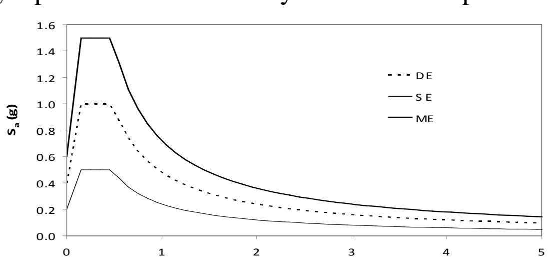


Figure 4. Design acceleration response spectra.

4.4. Equivalent three DOF System and Ductility Demand.

Table 4 shows the elastic spectral acceleration for the zone *A* and the mass for each mode. The spectral acceleration associated at the yield limit, S_{ay} , is equal to 0.392 g and it is obtained by mean of equation (5). The displacement ductility demand, μ_d , is equal to 1.21 and 1.82 associated to DE and ME respectively.

Table 4. Parameters of the equivalent three DOF system (zone A).

			S_{ae} (g)		
Mode	m_i (% m)	T_i (s)	DE	SE	ME
1	71.0	1.25	0.355	0.190	0.355
2	19.2	0.21	1.000	0.500	1.500
3	2.4	0.08	0.900	0.450	1.350

4.5. Capacity vs. Demand

The results obtained for three earthquake design levels and the structural control parameters are shown in the table 5. The square root of the sum of the squares (SRSS) has been used to combine the effect of higher modes.

Table 5. Values of capacity, demand and structural control parameters.

Earthquake Level	ϕ_{max} (rad/m)	M_{max} (kN-m)	M_i (kN-m)	V_i (kN)	δ (%)	μ_d
DE	1.370×10^{-3}	38756	39843[1]	1844	0.7	1.07
SE	0.346×10^{-3}	26401	26433	1133	0.4	1.00
ME	4.150×10^{-3}	39726	41334[2]	2231	1.4	1.61

1. The demand overcomes 2.8 % of the flexural capacity. The reinforcement might be accepted.
2. The demand overcomes 4.0 % of the flexural capacity. The reinforcement might be accepted.

4.6. Definitive Detailing

Since the requirements are not fulfilled in terms of strength and inter-story drifts, it is necessary to increase the longitudinal reinforcement. In this case, the option is to increase the tension reinforcement in zone A to 82 cm^2. The detailed procedure in the previous sections is redone, fulfilling in this way with all the requirements. In Figure 3.b a detail of the wall appears with the final array of the reinforcement in both zones.

5. DISCUSSION AND CONCLUSIONS

The mathematical model, based on four masses, allows to represent the nonlinear behavior of a building configured by resistant cantilever walls and makes the design, based on objectives, practically for any height. In general, the participation of the third mode can be omitted. The plastic length is a variable to be considered and it has to be defined in the design to determine, in a correct way, the ductility capacity. It is not correct to define the plastic length as a function of the wall length. It is not considered in this paper a detailed design of the plastic zone. An optimal reinforcement design needs to bear in mind the section behavior. Supposing the curvature or the displacement ductility requires design verification. As it is shown in this work, it is an advantage to begin the design with a reinforcement ratio. In this paper, a procedure is shown, step by step, to make an optimal design. For a more rigorous design, when the conditions are required, a tri linear Moment-Curvature diagram has to be used, which includes the concrete tensile capacity. When deformations over 0.004 are considered, it is necessary to make an allowance for the compression bars buckling.

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