



DISPLACEMENT PROFILE FOR DISPLACEMENT BASED DESIGN OF DUAL FRAME SYSTEMS

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ABSTRACT

Direct Displacement Based Design (DDBD) method designs reinforced concrete structures on the basis of seismic induced displacement demand rather than force demands. DDBD has been developed for several types of structures in the last decade. In this study, displacement profile for wall-frame type structures to be used in the DDBD of dual frame is investigated. An iterative two phase method that uses DDBD in the first phase and nonlinear time history analysis in the second phase is proposed for determining the displacement profiles. Displacement profiles for six, eight and twelve story four span dual-frame type structures are determined. The effects of the number of the bay and the base shear ratio carried by the wall are also investigated. A new displacement profile function is proposed for the DDBD of dual frames.

Keywords: Direct Displacement Based Design, Displacement Profile, Dual Frame

INTRODUCTION

The use of displacement based design as a seismic design became important in the last decade. By using these new procedures the designers control the response of structures in earthquake instead of traditional force based design methods. Direct displacement based design (DDBD) is a particular form of displacement based design. DDBD has been developed over the last decade.

DDBD procedure is first developed for the design of bridge columns that can be easily modeled as a single degree of freedom system. (Kowalsky et al., 1994) Then the procedure is extended for the design of multi degree of freedom bridge structures (Calvi and Kingsley, 1995), moment resisting frame structures. (Leoding, 1998; Priestley et al., 1996) and cantilever wall structures (Kowalsky, 2001). The DDBD procedure of precast and prestressed concrete buildings is also developed (Priestley, 2002). An innovative seismic design procedure developed for frame wall structures (Sullivan et al., 2005).

The use of frame and walls in a structural system has lots of advantages. In dual frames, the system has wall dominating behavior in the lower story levels and frame dominating behavior in the upper storey. DDBD procedure developed for the moment resisting frames can be used for the design of wall frame structures. But the most important step of the DDBD is to define the displacement profile for the dual frames. So that the displacement profile of dual frames are investigated by using nonlinear time history analysis in this study. The changes in the displacement profile with the number of spans, number of story and the base shear ratio that the wall carries are investigated.

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FUNDAMENTALS OF DIRECT DISPLACEMENT BASED DESIGN

In DDBD the multi degree of freedom system is represented by an single degree of freedom system as shown in Fig. 1-a. Force based seismic design characterizes a structure in terms of elastic, pre-yield properties (initial stiffness, elastic damping), DDBD characterizes a structure by secant stiffness K_{ef} at maximum displacement as shown in Fig. 1-b and a level of viscous damping and hysteretic energy absorbed during elastic response (Priestley, 2003). This is based on Substitute structure approach (Gulkan and Sozen, 1974 and Shibata and Sozen, 1976).

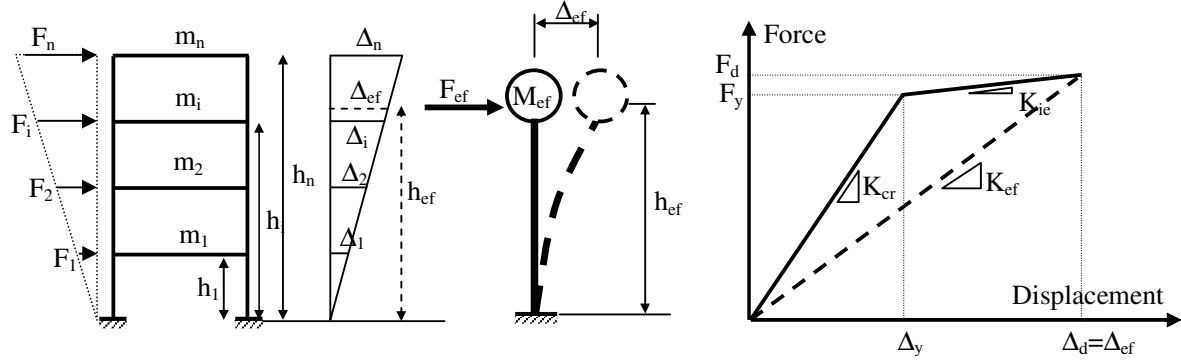


Figure 1. (a) Equivalent single degree of freedom system (b) Effective Stiffness

When the displacement profile as shown in Fig 1-a is known then the design displacement Δ_d , can be obtained by using Eq. 1

$$\Delta_d = \frac{\sum_{i=1}^n m_i \Delta_i^2}{\sum_{i=1}^n m_i \Delta_i} \quad (1)$$

where n is the number of story, m_i is the story mass and Δ_i is the story displacement obtained by the displacement profile.

In DDBD, the nonlinear behavior is represented by an equivalent viscous damping (ξ_{eq}). Using equivalent value of viscous damping representing both elastic and hysteretic energy dissipation, it is possible to solve a simple nonlinear system instead of a nonlinear system. The effective damping depends on the structural system and displacement factor. There are several equivalent approaches. For frame structures Eq. 2 and for wall structures Eq. 3 is selected (Priestley, 2001)

$$\xi_{frame} = 5 + \frac{125 (1 - \mu^{-0.5})}{\pi} \quad (2)$$

$$\xi_{wall} = 5 + \frac{100 (1 - \mu^{-0.5})}{\pi} \quad (3)$$

where ξ_{frame} and ξ_{wall} is equivalent viscous damping of frame and wall respectively and μ is the ductility factor. For the DDBD of dual frames a decision will be require as to the proportion of total base shear to be carried by the frame and by the wall.

The equivalent viscous damping for the dual frames can be defined by the Eq. 4 depending on the base shear ratios of the wall and frame (Priestley, 2003).

$$\xi_{eq} = \frac{V_{wall} \xi_{wall} + V_{frame} \xi_{frame}}{V_{base}} \quad (4)$$

where V_{wall} is the base shear carried by the wall, V_{frame} is the base shear carried by the frame and V_{base} is the total base shear carried by the dual frame.

The effective period (T_{ef}) of the substitute structure can be obtained from displacement a spectrum which is reduced for the determined damping ratio as shown in Fig. 2.

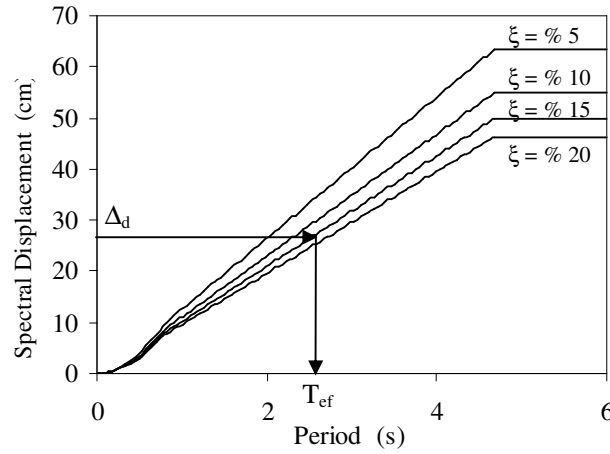


Figure 2. Obtaining effective period

Effective stiffness (K_{eff}) of the equivalent single degree of freedom system at maximum displacement can be found using the equivalent period with Eq. 5

$$K_{ef} = \frac{4\pi^2 M_{ef}}{T_{ef}^2} \quad (5)$$

where M_{ef} is the effective mass of the SDOF system and can be obtained using the displacement profile as in Eq. 6

$$M_{ef} = \frac{\sum_{i=1}^n (m_i \Delta_i)}{\Delta_d} \quad (6)$$

As shown in Fig. 1-b, the base shear of the equivalent SDOF system is given by Eq. 7

$$V_b = K_{ef} \Delta_d \quad (7)$$

With the base shear and the story displacements calculated the story forces can be found by Eq. 8

$$F_i = V_b \frac{m_i \Delta_i}{\sum_{i=1}^n (m_i \Delta_i)} \quad (8)$$

The story forces are proportional to story displacements. The force profile for the system has the same shape as the maximum displacement profile.

Analysis of the Structure under Design Forces

In the study the structure will be analyzed under the force vector obtained. In order to be compatible with the substitute structure concept, member stiffness should be representative of secant stiffness at design displacement response for frame and dual frame system building, beam members will be subjected to inelastic actions and the appropriate stiffness can be given by Eq. 9 (Priestley, 2003).

$$I_b = I_{cr} / \mu \quad (9)$$

where I_{cr} is the cracked section stiffness. The columns and the wall will be protected against inelastic actions so that their stiffness can be taken without reduction of ductility. However the wall stiffness will need to be reduced over the lower levels in proportion to expected ductility demand. Plastic hinge will be expected at the base level of the ground floor columns and wall. A modification to the column stiffness must be made for columns ground floor. The most appropriate way to model this in an elastic analysis is to place a hinge at the base level and apply a base resisting moment as shown in Fig. 3. These moments are called prefixed moments.

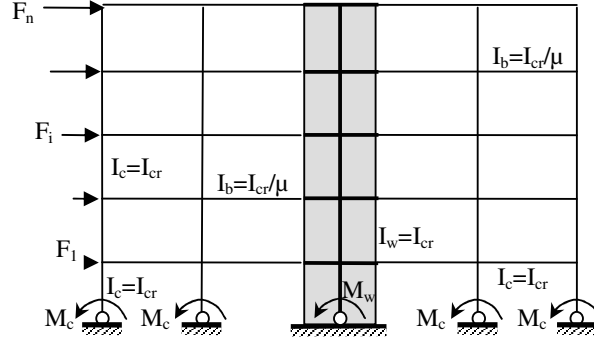


Figure 3. Member stiffness for structural analysis

For the DDBD of dual frames a decision will be required to the proportion of total base shear to be carried by the columns and by the wall. After that prefixed M_c column moment and M_w wall moment can be determined using the base shear of column.

Member Stiffness Ratio

A set of dual frame structure is analyzed under lateral force with different column and wall stiffness ratio to determine the base shear ratio carried by the wall. The beam cracked moment of inertia can be assumed to be equal to the column cracked moment of inertia. The beam stiffness is reduced by the expected ductility to represent the substitute structure at maximum displacement as given in Eq. 9. The change of the base shear ratio with the element stiffness ratio is given by Eq. 10

$$\frac{V_{wall}}{V_{base}} = 0.1466 \ln \left(\frac{I_w}{\sum I_c} \right) + 0.3759 \quad (10)$$

where I_w is the stiffness of the wall, $\sum I_c$ is the total of the column stiffness.

Prefixed Base Moments of Column and Wall Elements

Before a complete structural analysis is done, the base moments must be determined. Column moment (M_c) is a designer choice. But can be calculated using the base shear of the column and the height of

the point of contraflexure which is between %55 and %65 of the column height. With a point of contra flexure chosen %60 of the column height, column moment (M_c) can be given in Eq 11.

$$M_c = V_c (0,6h_1) \quad (11)$$

where V_c is the column base shear force, h_1 is the height of the first floor.

Using the same set of structures the change of the ground floor base moments with stiffness ratios of wall and columns are investigated. A base moment change with the stiffness ratio is given in Eq .12

$$\frac{M_w}{\sum M_c} = 0.631 \left(\frac{I_w}{\sum I_c} \right)^{0.96} \quad (12)$$

where M_w is the prefixed base moment, $\sum M_c$ is the total base moment of columns as given in Fig. 3.

DETERMINATION OF DISPLACEMENT PROFILE OF DUAL FRAME

Displacement profile is an important step of the DDBD because most of the properties of the equivalent linear system is obtained using displacement profile. When a frame structure is subjected to earthquake, the displaced shape is concave but the displacement shape of a cantilever wall is convex as seen in Fig. 4. The displaced shape of the dual frame contains both frame and wall displacement response. In the lower storey, it acts like a cantilever wall but in the upper storey acts like a frame so that the displacement profile must be defined.

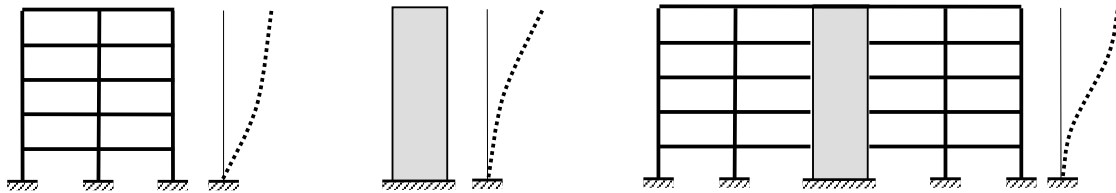


Figure 4. Displacement shape of frame, cantilever wall and dual frame structures

An iterative two phase analysis is used to obtain displacement profile (Yavas,2004). In the first phase the dual frame is designed by the DDBD whose principles are given before, for a pre defined displacement profile. In the second phase, the designed structure is analyzed by using time history analysis for using 10 different artificial records. Displacement profile is obtained by the mean value of the displacement envelopes for each of time history analyses results by means of the artificial record. The obtained displacement profile is compared by the pre defined displacement profile. If those displacement profiles do not match each other, the structure is designed with DDBD. Two phase analysis is repeated until the displacement profiles match each other.

Nonlinear Time History Analysis

Nonlinear time history (NTH) is used in the second phase for displacement profile. Newmark-Beta numerical integration method is used for the NTH analyzes. In NTH analyzes, it's assumed that the plastic deformations are concentrated in a region called plastic sections and the response is linear elastic except for plastic sections. The moment-plastic rotation response of the column and beam elements assumed rigid plastic with strain hardening, but the wall elements moment-plastic rotation response is assumed rigid perfectly plastic. RAM Perform 2D (Ramint, 2004) software is used for NTH analyzes.

In the study ten different artificial records are generated for NTH analyzes. Five of the records are generated using SIMQKE (Carr, 2001) the other five artificial records are generated using TARSCTHS (Papageorgiou et al., 1999). All motions were scaled to the displacement response spectrum of zone 4 earthquake in soil type D, according to SEAOC (Tentative, 1999) to include a constant displacement region Displacement spectra of each artificial record and the target displacement spectra are shown in Fig. 5

above a specified period. The peak ground accelerations of the generated records are given in Table 1.

Table 1. PGA of the generated artificial records

SIMQKE		TARSCTHS	
Record Name	PGA	Record Name	PGA
YK-1	0.4	YP-YK-1	0.5970
YK-2	0.4	YP-YK-2	0.5893
YK-3	0.4	YP-YK-3	0.45361
YK-4	0.4	YP-YK-4	0.5104
YK-5	0.4	YP-YK-5	0.4340

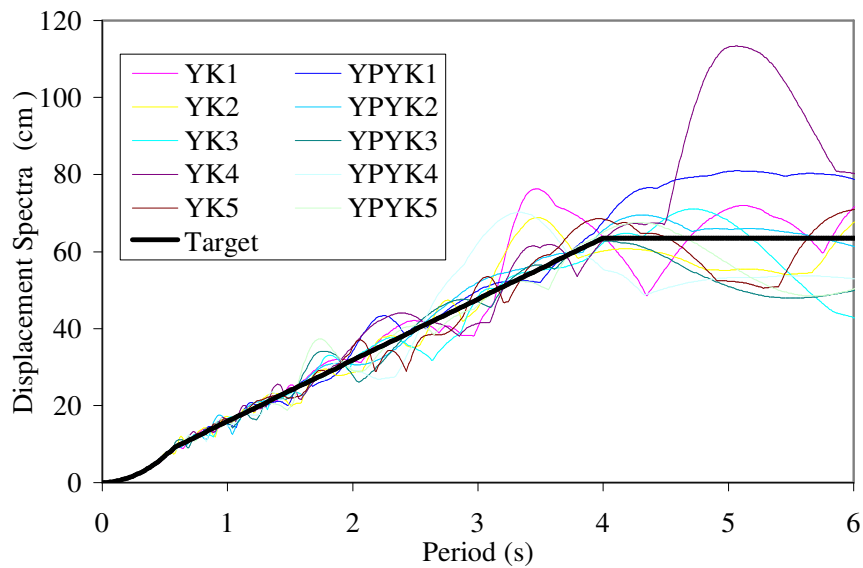


Figure 5. Target and artificial records displacement spectra

NUMERICAL EXAMPLES

Six story, nine story and twelve story structures are analyzed to determine the displacement profile as shown in Fig. 6. A story mass of 89700 kg is selected to represent a typical mass that a frame of the given dimensions would carry. The structures are detailed in Fig. 6.

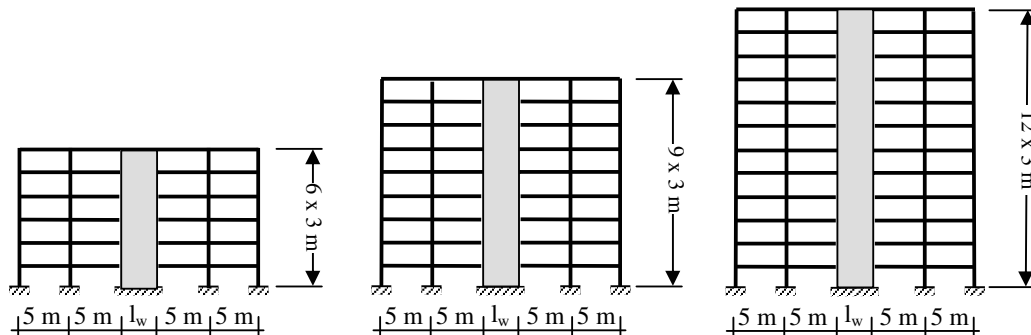


Figure 6. Structures analyzed

It's assumed that the wall carries the %50 of the base shear. Displacement profile of the six story structure is determined using iterative two phase analyses. The displacement profile is converged in the fourth iteration step. The displacement profile in different iteration steps and the displacement envelopes of NHT analyzes results of each artificial record in the last iteration is given in Fig 7.

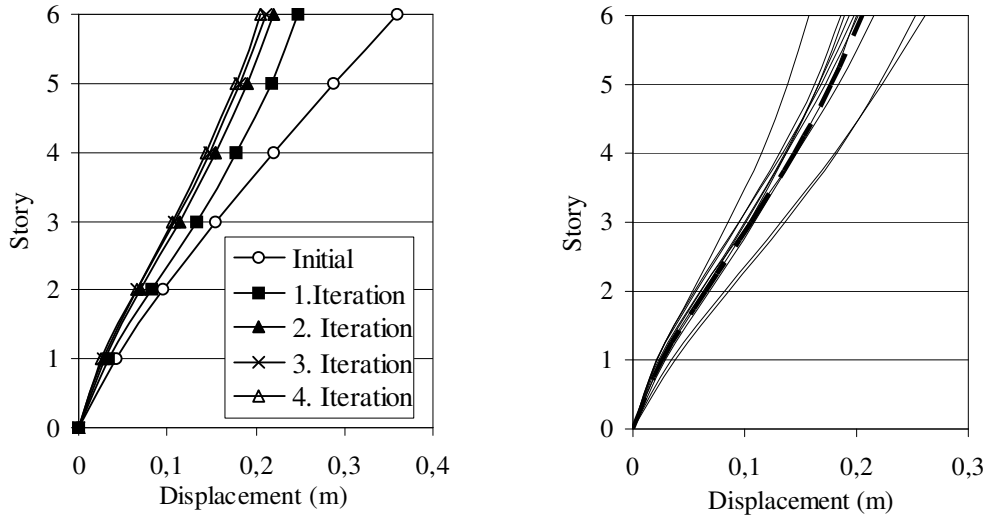


Figure 7. Iterations for displacement profile of six story dual frame and the displacement envelope of NHT analyzes results of each artificial record in the last iteration

Fig. 8 and Fig. 9 show the displacement profiles of the iterations and the envelope displacements of the last iteration from nonlinear time history analyzes for nine and twelve story dual frames, respectively.

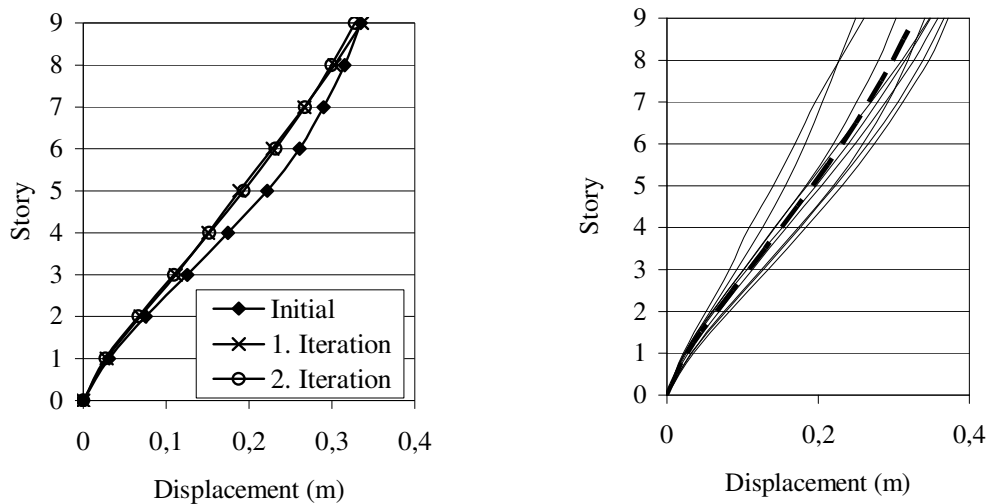


Figure 8. Iterations for displacement profile of nine story dual frame and the displacement envelope of NHT analyzes results of each artificial record in the last iteration

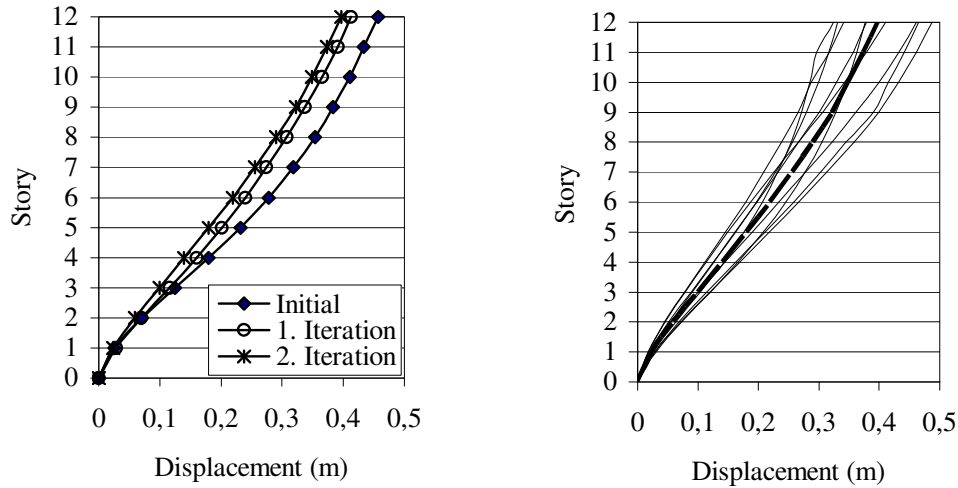


Figure 9. Iterations for displacement profile of twelve story dual frame and the displacement envelope of NHT analyzes results of each artificial record in the last iteration

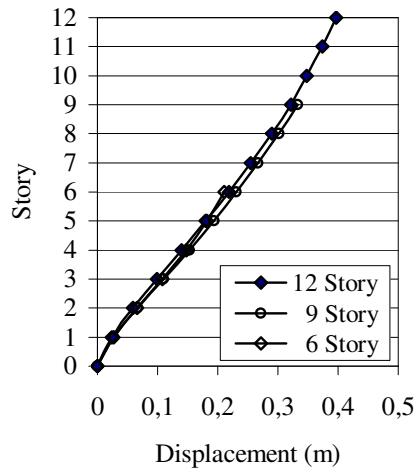


Figure 10. Displacement profiles of six, nine and twelve story dual frames

Fig 10 shows that the displacement profiles of six, eight and twelve story dual frames can be expressed with one function. Displacement profiles of six, nine and twelve story dual frames are shown in Fig 10. The function of displacement profile obtained from the results of nonlinear time history analysis is given in Eq. 13.

$$\Delta_i = (0,0004 n^{-1,7}) h_i [h_i + (2,5 n - 6,7)] [h_i - (4,2 n + 18)] \quad (13)$$

where n is the number of story, h_i is the height of i 'th story.

Effect Of Base Shear Ratio Carried By The Wall To Displacement Profile.

Displacement profiles of six, nine and twelve story dual frames are determined for the base shear ratio carried by the wall is %50. Dual frames are designed for different base shear ratio carried by wall using the displacement profiles obtained. Using nonlinear time history analyses the displacement profiles are obtained for six, nine and twelve story frames that the wall carries %40 and %60 base shear of the system and given in Fig. 11. The variation of the base shear ratio of the wall is not change the displacement profile as shown in Fig.11.

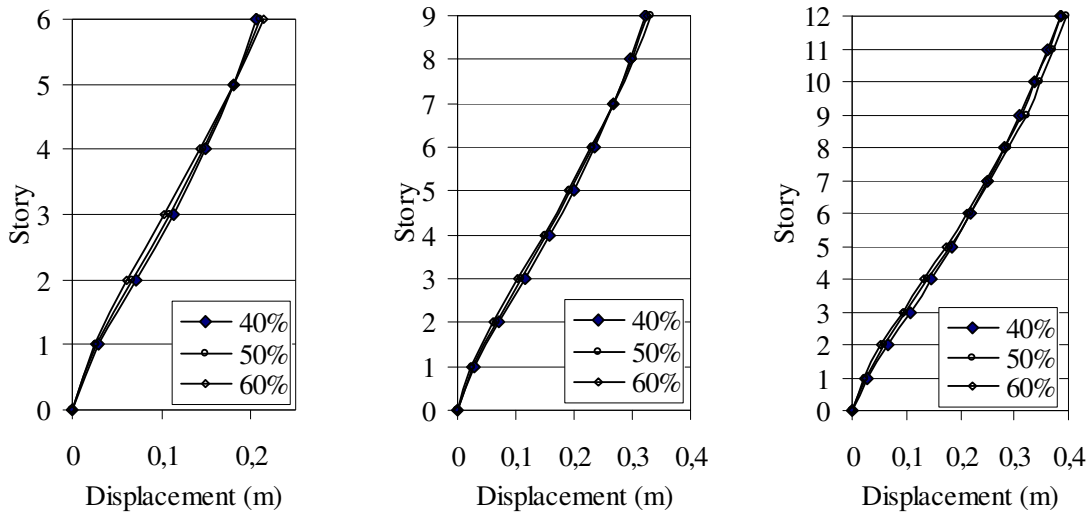


Figure 11. Displacement profile of six, nine and twelve story dual frames for different base shear ratio

Effect of Number of Bays to Displacement Profile

The displacement profiles of the dual frames are obtained for the four bay dual frames. Analyzes are repeated for six story, six and eight bay dual frames. It's determined that the displacement profiles do not change with the number of bay of dual frames as shown in Fig 12.

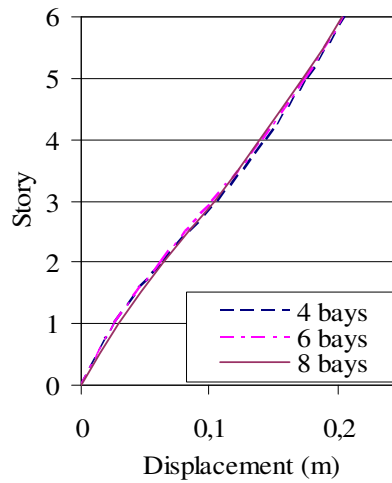


Figure 12. Displacement profiles for six story dual frames of four, six and eight bay

CONCLUSIONS

DDBD procedure developed for the moment resisting frames can be used for the design of wall frame structures. However the most important step of the DDBD is to define the displacement profile for the dual frames. In this study an iterative two phase analysis is used to determine the displacement profile. The displacement function is obtained four, six, eight and twelve story which has dual frames four bays. Effect of number of bays to displacement profile is investigated with six story, four, six and eight bay dual frames. Also, the effect of base shear ratio carried by the wall is investigated and it's found that the number of bays and the base shear ratio carried by the wall don not change the displacement profile. A new displacement profile function which is given in Eq. 13 is obtained.

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