DETERMINATION OF EQUIVALENT SDOF CHARACTERISTICS OF 3D DUAL RC STRUCTURES

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ABSTRACT :

The recent drive for the use of a single-degree-of-freedom representation in displacement-based design and assessment of 3D reinforced concrete (RC) structures has significantly increased the demand for the determination of equivalent SDOF characteristics of such buildings. Nonlinear static analyses are frequently used to describe the response of a structure with reduced computational effort with respect to nonlinear dynamic analyses. The response parameters of interest include the mechanical SDOF characteristics such as yield period, deformed shape etc. However, the inherent irregularities and uncertainties of existing RC buildings render the SDOF representation rather difficult and more demanding. The main focus of this study is on existing dual (frame-wall) structures; 4 case study RC buildings from the existing Turkish building stock have been modelled in 3D using a fibre-based finite elements software. Displacement-based adaptive pushover (DAP) analyses have been conducted in both directions of the buildings. The DAP capacity curves have been used to extract the yield periods, deformed shapes, and effective heights of the case study buildings in order to define the SDOF characteristics of dual structures for use in displacement-based assessment.

KEYWORDS: Dual structures, reinforced concrete, SDOF characteristics, period of vibration

1. INTRODUCTION

Seismic design code provisions are traditionally force-based, whilst more recent proposals are moving towards the displacement-based design and assessment of structures. Displacement-based assessment and design provisions are currently available for regular, conventional buildings (e.g. Priestley, 1997; Priestley *et al*., 2007). The use of current displacement-based design and assessment techniques requires an accurate estimate of the single-degree-of-freedom (SDOF) characteristics of buildings. The SDOF representation of a MDOF system is a complex issue with many inherent uncertainties. In particular, for what concerns the assessment of the existing building stock, common irregularities, uncertainties in the use and quality of materials, lack of knowledge on dynamic characteristics, uncertainties related to infill walls and failure mechanisms render the use of SDOF characteristics even more complex.

In this study, an effort to define the SDOF characteristics of 3D dual structures is presented through the use of Displacement-Based Adaptive Pushover (DAP) analyses (Antoniou and Pinho, 2004). Attention is paid to the determination of the characteristics related to the yield period, deformed shape, and effective height. Four reinforced concrete (RC) case study buildings with RC walls with 4, 5, 6 and 8-storeys, which have been taken from the existing Turkish building stock, have been used for the analyses. These case studies are intended to be representative of the existing mid-rise RC buildings in Turkey. All case study buildings are selected from real buildings existing in the Turkish building stock in the Northern Marmara Region. Detailed drawings and additional information about these case studies can be found in Vuran (2007), whilst structural plan drawings of the case studies are shown in Figure 1.

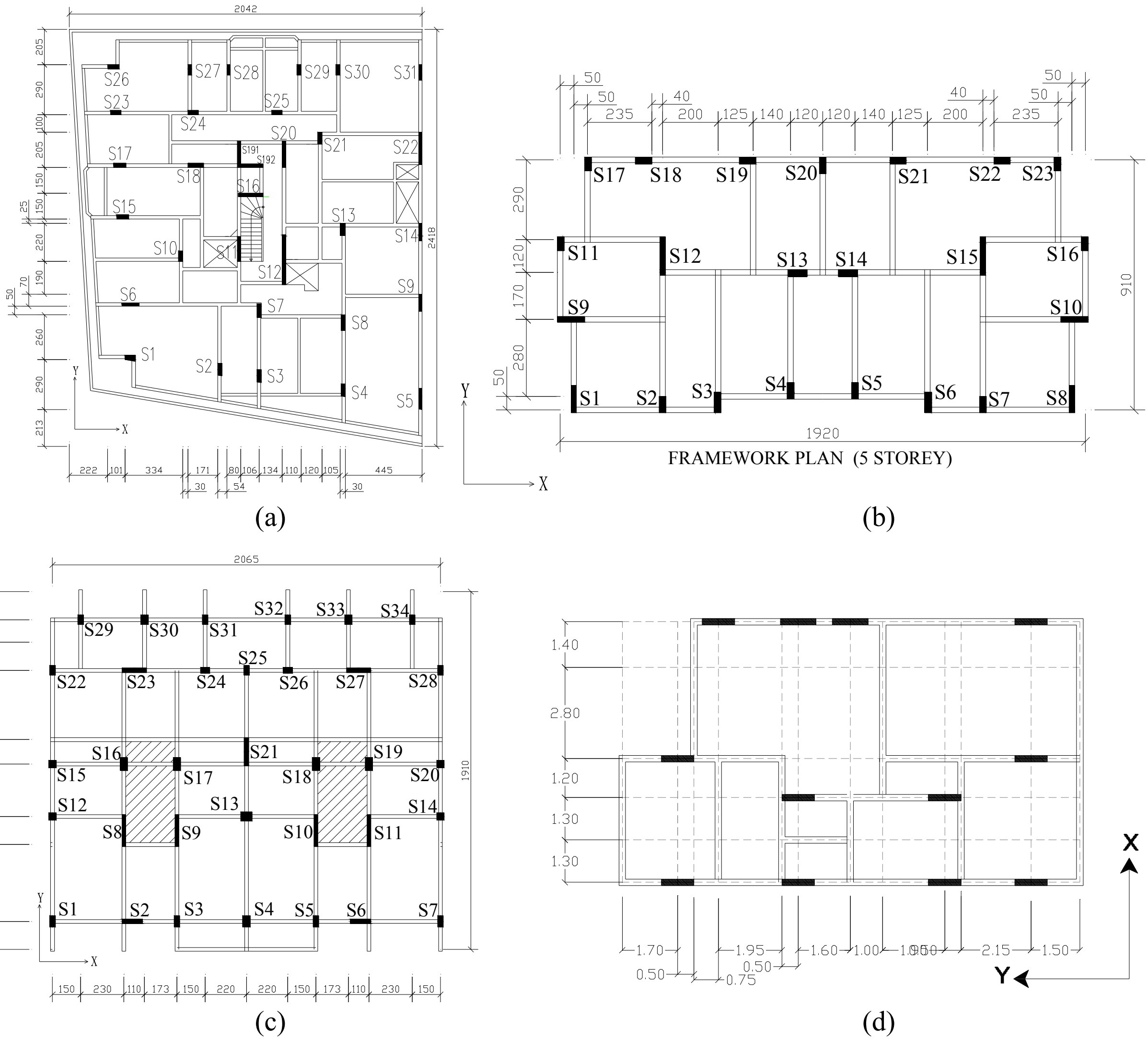


Figure 1 Storey plans of case study buildings (a) 4-storey, (b) 5-storey, (c) 6-storey and (d) 8-storey buildings

The construction of structural walls (commonly referred to as shear walls) is currently being used extensively as a retrofitting scheme for RC frame buildings in Turkey, leading to a dual system of frame and walls. Dual system buildings started to be used more widely after 1998 due to the new specifications of the Turkish Earthquake Code of 1998. Although it is possible to find dual system buildings which were constructed before 1998, the performance of the structural RC walls is questionable since they were not necessarily constructed to contribute to the lateral load resistance of the building. Instead, they were mostly constructed as core walls round the stairs or elevator, or both, and they were often not effectively connected with the existing frame.

2. MODELLING AND ANALYSES

The fibre-element finite elements analysis program SeismoStruct (SeismoSoft, 2008) has been utilized to run all of the adaptive pushover analyses herein. The material types of the case study buildings were taken from the drawings, whilst the average material properties to be used in the analyses were based on a study of Turkish material properties by Bal *et al*. (2008). The average unconfined concrete strength has been taken as 16.7 MPa whilst the average yield strength of the S220 rebars is assumed to be 371 MPa. The confinement factor for the concrete has been taken as 1.1 for sections with largely spaced stirrups and 1.2 for sections with closely spaced stirrups. The average mass of the 4-storey building per square-metre is 1.52 tonnes, while the same parameter is 1.38 tonnes for the 5-storey building, 1.37 tonnes for the 6 storey building and 0.70 tonnes for the 8-storey building.

The philosophy of the Displacement-Based Adaptive Pushover (DAP) is that displacements, rather than forces, are applied at each analysis step. The eigenvalue problem is solved at each step of the analysis based on the corresponding structural stiffness and by setting a modal combination rule, the higher mode effects can be taken into account to update the profile of applied displacements. A spectrum scaling of the modes is also possible, and advised, in DAP analyses by introducing a displacement spectrum which is used to amplify the considered modes of vibration. In the current study, the displacement spectra from 8 different spectrum-compatible records have been used for spectrum scaling. The same 8 records have also been used in Incremental Dynamic Analyses, IDA, (see e.g. Vamvatsikos and Cornell, 2002) which have also been applied to the case study frames. These IDA analyses can be used in the verification of pushover analyses (e.g. Gupta & Kunnath, 2000; Elnashai, 2001), and some initial studies to verify DAP using a large number of records have already been carried out elsewhere (see Ferracuti *et al*., 2007; Vuran, 2007). An example application to the 4-storey case study is shown in Figure 2 (x direction) and Figure 3 (y direction) where the DAP curves on these plots for each of the records using in the spectral scaling are plotted together with the IDA results. The results show that some of the inherent variability in the response, which is observed in the nonlinear dynamic analyses, is captured by the spectrum-scaled DAP analyses, with closer results being observed in the y direction of the building. The aim of this work is not to verify the DAP algorithm with IDA analyses, which as mentioned previously is being carried out in other research efforts. In the current application, the use of DAP to define the single-degree-of-freedom characteristics is adopted, whilst future research will look at the influence of the variability in the response of the building under dynamic loading to these characteristics.

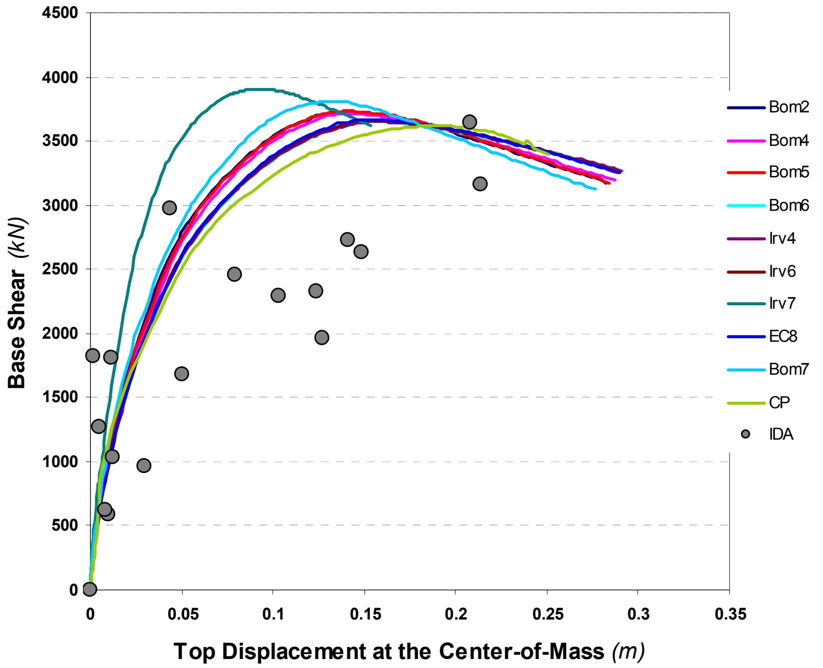


Figure 2 DAP versus IDA graphs for the 4-storey building in the x direction (see Vuran (2007) for a description of the records used in IDA and spectrum-scaled DAP)

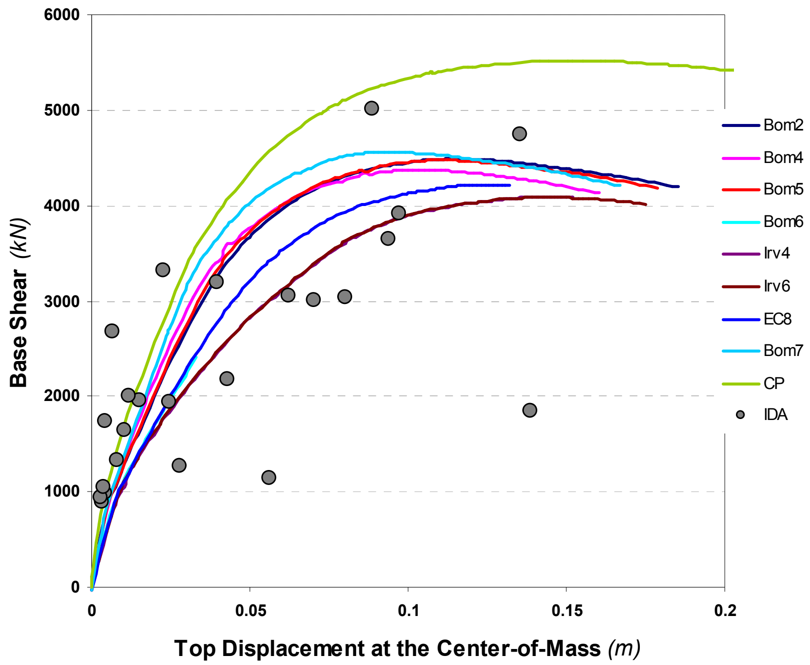


Figure 3 Example DAP versus IDA graphs for 4-storey building in the y direction (see Vuran (2007) for a description of the records used in IDA and spectrum scaled DAP)

3. SDOF CHARACTERISTICS

3.1 Deformed Shape and Effective Height

The deformed shape is an important parameter since most of the SDOF characteristics (i.e. effective height, effective mass and design displacement) are derived from the deformed shape of the building. The deformed shapes for frames and dual systems suggested by Priestley *et al*. (2007) have been used herein. For regular frame buildings, the following equations, though approximate, have been shown to be adequate to represent the deformed shape for Displacement-Based Design purposes (Pettinga and Priestley, 2005):

$$\text{for} \quad n \le 4: \quad \delta_I = H_i/H_n \qquad \text{and} \qquad \text{for} \quad n>4 \quad \delta_I = (4/3)(H_i/H_n)(1-H_i/(4H_n)) \tag{3.1}$$

where H_i and H_n are the heights of level *i*, and the roof level, *n*, respectively. The displacement profile of dual systems is rather more complex and given by a set of equations proposed by Sullivan *et al*. (2006). It is noted that the equations given below provide the displacement profile, not the deformed shape; however, the deformed shape can easily be found by normalizing the displacement values of each floor to the maximum displacement. The yield displacement profile is shown in Eqn. (3.2) and the design displacement profile in Eqn. (3.3):

$$\text{for} \quad H_i \le H_{CF} \quad \Delta_{yi}=\phi_{yW}(H_i^2/2-H_i^3/(6H_{CF})) \quad \text{and} \quad \text{for} \quad H_i>H_{CF} \quad \Delta_{yi}=\phi_{yw}(H_{CF}H_i/2-H_{CF}^2/6) \tag{3.2}$$

$$\Delta_{Di}=\Delta_{yi}+(\phi_{ls}-\phi_{yW})L_pH_i \tag{3.3}$$

where L_p is the plastic hinge length, given in Eqn. (3.4), ϕ_{yw} is the yield curvature of the wall, which is given in Eqn. (3.5) and ϕ_{ls} is the curvature for the damage control limit state, which is presented in Eqn. (3.6).

$$L_p=kH_{CF}+0.1l_w+L_{sp} \tag{3.4}$$

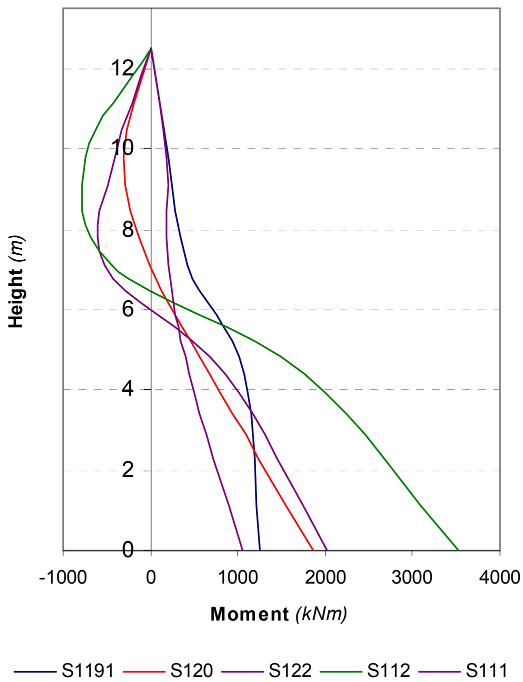
$$\phi_{yW} = 2.10\ \varepsilon_y/l_w \tag{3.5}$$

$$\phi_{ls} = 0.072\ /\ l_w \tag{3.6}$$

where k is $0.2(f_u/f_y-1) \leq 0.08$, H_{CF} is the height of contraflexure, as described below, ε_y is the yield strain of the steel, l_w is the wall length, and L_{sp} is the strain penetration length given in Eqn. (3.7), f_u is the ultimate and f_y the yield strength of the steel material (in MPa) and d_{bl} is the diameter of the vertical rebars:

$$L_{sp}=0.0022 f_y d_{bl} \tag{3.7}$$

The point of contraflexure is essentially the point where the moments of the wall change sign. The point of contraflexure and the displacement profiles for dual systems are often based on the properties of a single wall. However, the case studies used in this research work have different sized walls which contribute by different amounts to the response of the system. A weighted average of the moment ratio of each wall to the overturning moment has been followed herein. The moment distributions of two of the case study buildings along their height are shown in Figure 4. It can be seen from this figure that some walls do not exhibit a change in sign of the moment distribution and thus the height of contraflexure of these walls has been assumed to be equal to the height of the building, where the moments are zero. Using similar analyses for all of the buildings, the average contraflexure heights have been calculated for each building by averaging the contraflexure heights of walls weighted by their moment contribution. The ratio of contraflexure height to the total building height has thus been calculated and is listed in Table 1.



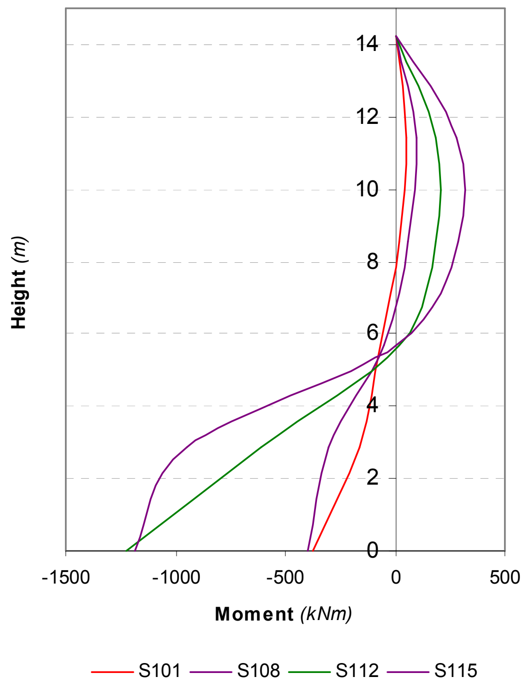


Figure 4 Wall moment distributions along the height for 4-storey (left) and 5-storey (right) buildings in their y-directions *(moment values are considered at the bottom sections of walls)*

The deformed shapes of the structures have been plotted according to the mean results of all DAP analyses and the DDBD suggestions by Priestley *et al*. (2007), as described above in equations (3.1) to (3.7). The obtained deformed shapes and comparisons can be seen from Figure 5. The ratio of the wall moments to the total overturning moment in each direction of each building at the failure step has also been calculated and listed in Table 1. It can be seen from Figure 5 that the 4-storey building in the y direction, which has the highest ratio of

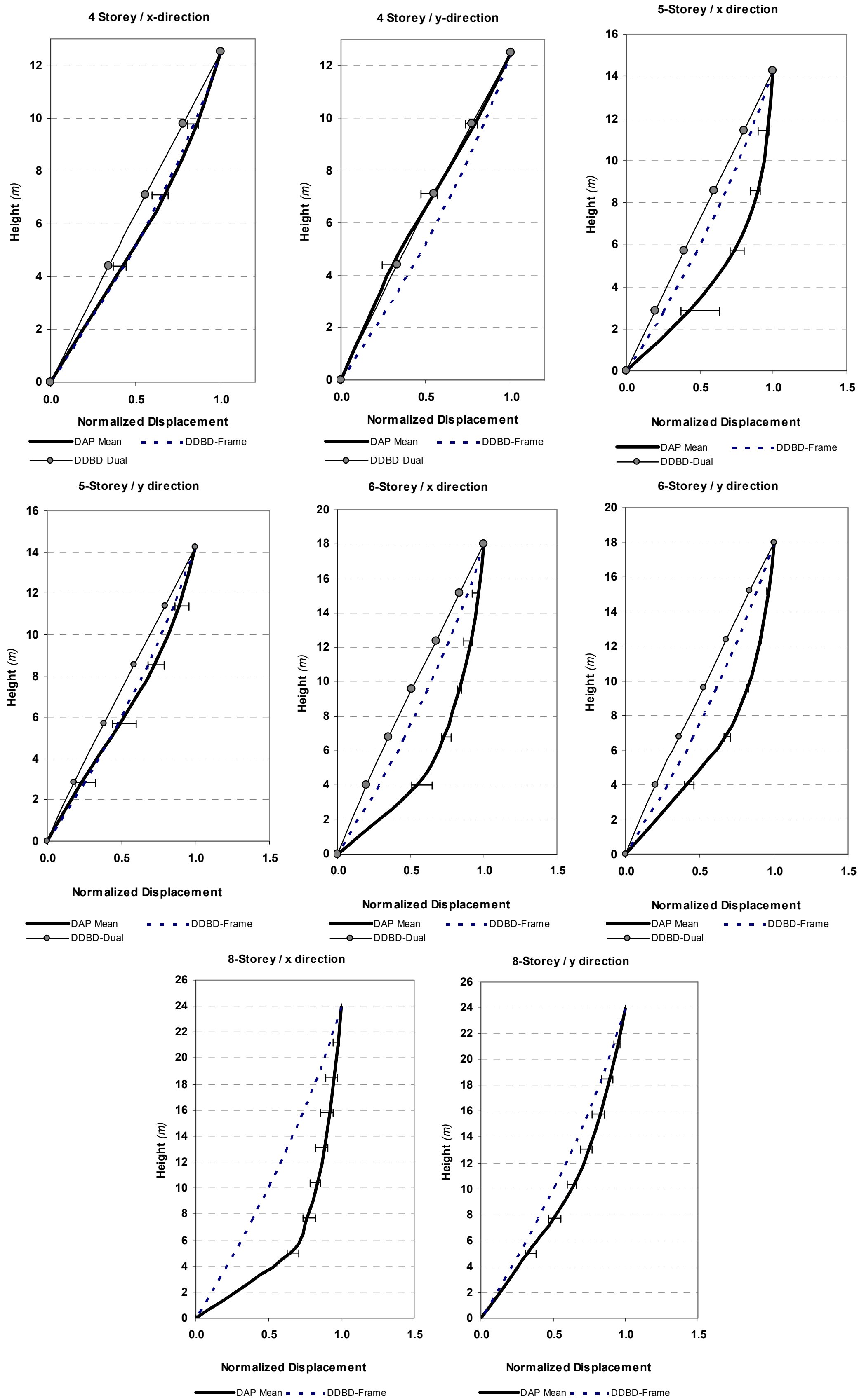


Figure 5 Deformed shapes of all structures in both directions and comparison with DDBD equations

moment contribution of the walls, has the closest deformed shape to the deformed shape suggested for the frame-wall dual system. The 5, 6 and 8 storey buildings develop a storey mechanism in the first floors in the x-direction and thus they do not follow the displacement profile suggested by DDBD procedure since this displacement profile is based on the assumption that the building will develop a beam-sway (or global) mechanism. The ratio of the effective height to the total building height has also been calculated by using the deformed shapes. These ratios can also be found in Table 1. It is observed that the buildings which develop a storey mechanism (or so called "column-sway mechanism") in the first floors have a decreased H_{eff}/H_n ratio which converges to 0.50, which is the value suggested by Priestley *et al*. (2007) for column-sway buildings.

Table 1 Weighted average wall contraflexure heights, effective heights and wall contributions for each model

Building / Direction	H_{CF}/H_n	H_{eff}/H_n	$\Sigma M_{wall}/\Sigma M_{OVR}$ (2)
4-storey-x	0.38	0.65	0.071
4-storey-y	0.64	0.68	0.255
5-storey-x	0.40	0.61	0.055
5-storey-y	0.12	0.65	0.156
6-storey-x	0.85	0.52	0.081
6-storey-y	0.52	0.53	0.098
8-storey-x	0.00 (1)	0.58	0.000
8-storey-y	1.00	0.63	1.000

[1] 8-storey building exhibits a pure frame behaviour in x direction
[2] M_{OVR}=Total overturning moment

3.2 Period-Height Relationship

The yield period of the buildings may be calculated by reducing the whole structure to a SDOF system which has an effective mass and an effective height, whilst the yield stiffness is the ratio of the base shear to the yield displacement. A comprehensive study on the period-height relationships for existing frame structures is given in Crowley and Pinho (2004). The relationship between the yield period of the case study structures used herein and their total height is rather complicated since all structures do not exhibit a frame-wall combined behaviour in all directions. As can be seen from Table 1, the moment contribution of the walls decreases to around 5% for some structures, which means that the structures behave similarly to frame structures. Considering the deformed shapes given in Figure 5 and the wall moment contributions, the case study structures have been divided into 3 groups of behaviour: "frame behaviour" which consists of the x directions of the 4, 5 and 8 storey buildings, "dual behaviour" where the y directions of the 4 and 5 storey structures and both directions of the 6 storey building are considered, and "wall behaviour" considering the y direction of the 8 storey building, since this structure consists of only walls in the y-direction. The yield periods of vibration of the buildings in each group have been calculated and plotted versus their total heights in Figure 6.

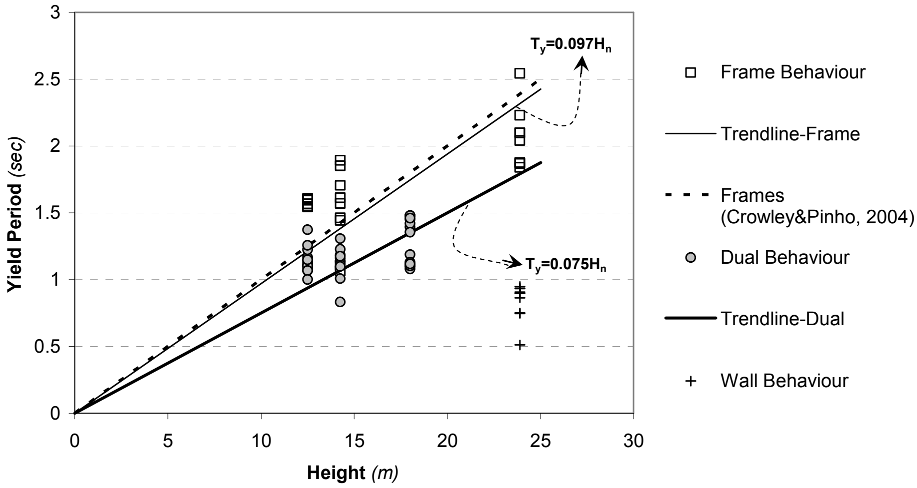


Figure 6 Yield Period – Height relationships

The best-fit trend-line of the frame group resulted in the period-height formula of 0.097H where H is the total building height in metres; this relationship agrees with the relationship proposed by Crowley and Pinho (2004), which is 0.1H. The best-fit trend-line for the dual structures has the equation 0.075H, again with H the height in metres. As expected, for a given height of the structure, the dual systems have a lower yield period due to the increased stiffness of the walls. As expected, the wall structures, which in this case is just the 8-storey building in the y direction, have a much lower yield period of vibration.

4. CONCLUSIONS

Displacement-based Adaptive Pushover (DAP) has been applied to four case study dual frames from the Turkish building stock. The resulting capacity curves have been used to obtain the SDOF characteristics of these types of buildings for use in displacement-based assessment of existing buildings. The deformed profiles have been compared with existing equations for frames and dual systems, and it has been observed that when the contribution of the walls to the total resisting moment of the building was low, the profile matched the frame equation, whilst the dual structure profile was matched for the case where the walls had a high contribution to the total capacity. In some cases, the buildings formed a storey mechanism and thus the deformed profiles did not match the existing equations, which have been derived for the design of new buildings with global mechanisms. The equation for the period of vibration of dual buildings derived herein was found to be 0.075H, where H is the height in metres of the building. Future research will look at the effect of torsional response on the SDOF characteristics, the possibility of a shear-sway mechanism and the variability in the SDOF characteristics.

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