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COMPARISON BETWEEN THE DIRECT DISPLACEMENT BASED DESIGN AND THE FORCE BASED DESIGN METHODS IN REINFORCED CONCRETE FRAMED STRUCTURES

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ABSTRACT

Force-based design (FBD), of reinforced concrete buildings subjected to seismic loads, suffers from many problems such as the assumed stiffness of the different structural elements, inappropriate R-factor, and others. These problems resulted in the need for an alternative design approach, which lead to the introduction of the performance based design (PBD) [1]. The Direct Displacement-Based Design (DDBD) which is the subject of this paper is based on (PBD). This method is characterized by its clear procedures which can be applied to different types of structures. The (DDBD) gives accurate solution for a certain category of structures since the damage of different structural elements is related to the maximum strain at these elements. The objective of this paper is to apply (DDBD) on different reinforced concrete frame buildings. The base shear force calculated by (DDBD) are compared with those calculated by (FBD) that is defined in the Euro-Code (EC8). Three computer programs have been used in analysis of the studied buildings to perform a pushover analysis and to get the displaced shape corresponding to the drift limit at the first floor. The results of the analysis show that the (DDBD) is more suitable for moment resisting frame type buildings with number of storeys more than 8 subjected ground accelerations exceeding 0.5 g.

KEYWORDS: Base shear force, FBD, DDBD, pushover analysis, displacement, drift, displaced shape, moment resisting frame buildings.

1. INTRODUCTION

1.1 Background:

The concept of response spectrum, where a spectrum of responses is plotted for a very wide range of single degree of freedom periods, was introduced in 1990. After calculating of the structure periods, these spectrum graphs are used to deduce the expected response of the structure under the effect of earthquakes.

The base shear force is an estimate of the maximum expected total lateral force that may occur due to seismic ground motion at the base of a structure. Calculations of the base shear (**V**) depend on many factors such as the soil conditions at the building site, the epicentral distance, the probability of occurrence of the earthquake, and the fundamental (natural) period of vibration of the structure.

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1.2 Force-Based Design, (FBD):

The force-based design, FBD, procedure is based on calculating the base shear force resulting from the earthquake dynamic motion using the acceleration response spectrum and the expected elastic period of the building. In this procedure the static loads are applied on a structure with magnitudes and directions that closely approximate the effects of dynamic loading caused by earthquakes. Concentrated lateral forces due to dynamic loading tend occur at each floor in buildings, where concentration of mass exists. Furthermore, concentrated lateral forces tend to follow the fundamental mode shape of the building, in other words it is larger at higher elevations in a structure. Thus, the greatest lateral displacements and the largest lateral forces often occur at the top level of a structure. These effects are modeled in *equivalent static lateral force* procedures of most design codes by placing a force at each story level in the structure, which is directly proportional with the height [1].

1.3 Displacement-Based Design, (DBD):

This approach uses the displacement response spectrum as a basis for calculating the base shear force. It also depends on studying the building considering its inelastic phase. This paper presents the fundamentals of the new seismic design method known as Direct Displacement-Based Design (DDBD) [2]. It is considered as one of the simplest design approaches for analysis of the multi-degree of freedom structures. In this method, the structure is characterized by the secant stiffness and equivalent damping of an equivalent single degree of freedom structure. This design is based on achieving a specified displacement limit state, defined either by material strain limits, or non-structural drift limits obtained from design codes under the design level seismic intensity. The characterization of the structure using the substitute structure avoids many problems inherent in force-based design, (FBD), where initial stiffness is used to determine an elastic period which is a drawback that is present in most of the building codes [3].

1.4 Scope of Work:

Reinforced concrete moment resisting frames are used as a part of seismic force-resisting systems. Beams, columns, and beam-column joints in moment resisting frame buildings are proportioned and detailed so as to resist flexural, axial, and shearing actions that result from seismic forces together with gravity loads. In this paper, analyses for six moment-resisting frame buildings with different heights have been applied–without any shear walls.

2. DIRECT DISPLACEMENT-BASED DESIGN, DDBD METHOD:

To start performing DDBD, the design drift of the building, θ_d, is determined according to the type of the building and its performance level [3].

The drift slope is defined by the following equation

$$\theta_d = \frac{\text{Inter-storey displacement}}{\text{Storey height}} \tag{1}$$

In this method, the structure is characterized by the secant stiffness and the damping at maximum displacement. The calculated base shear is applied to the structure and the assumed level of damping is checked , the design forces are then adjusted, if necessary. It is usually not necessary to adjust the forces as the adjustments are generally not significant [4]. When DDBD is performed, the secant stiffness, K_e , is used at maximum displacement, $\Delta_{\max}$, at the level of equivalent viscous damping. On the other hand, in FBD, the elastic stiffness and elastic damping (ξ =5%) are used.

A single degree of freedom (SDOF) representation is used in DDBD, which was developed by Shibata and Sozen [5]. In this representation the characteristics of the substitute structure are used. These characteristics are the secant stiffness at maximum displacement, equivalent viscous damping, effective mass, and the effective height as shown in Fig. 1.

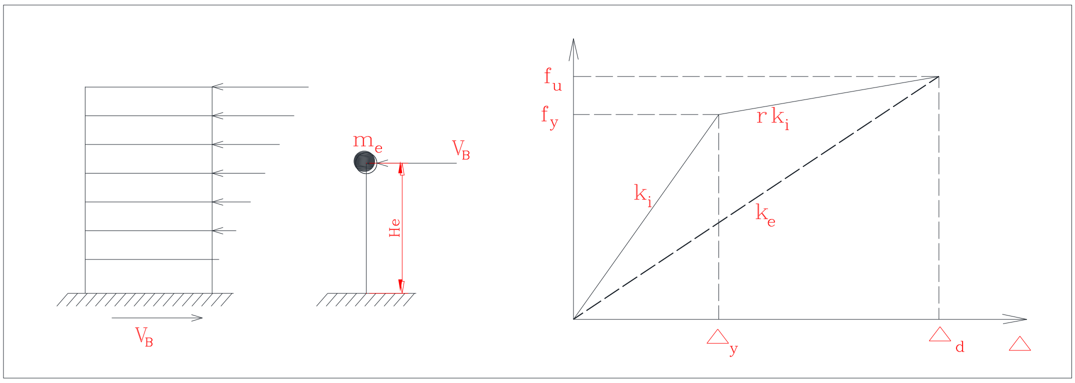

Fig. 1. Substitute structure representation for multi-degree of freedom frame and the force displacement relation up to failure (Elastic and inelastic stages) [6]

A full description of the steps needed for applying the DDBD method in the design of reinforced concrete moment resisting frame buildings [7] follows:

Step1 Determine Displacement shapes:

The following equation is presented by Calvi and Sullivan [8] to estimate the inelastic displaced shape of the building.

$$\Delta_i = \omega_\theta \theta_d h_i . \frac{(4H_n - h_i)}{(4H_n - h_1)} \tag{2}$$

where: Δ_i: Displacement at level i

ω_θ :Drift reduction factor to include allowance for higher mode amplification of drift by reducing the design floor displacement

H_n ,h_i: ; are the total building height, and height of floor i.

Step 2 Determine the design displacement, Δ_d:

Design displacement for the substitute structure

$$\Delta_d = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i} \tag{3}$$

where: m_i , Δ_i Mass and displacement at significant mass locations

Step 3 Calculate the effective height

$$H_e = \frac{\sum m_i \Delta_i H_i}{\sum m_i \Delta_i} \tag{4}$$

Step 4 Calculate yield drift slope θ_y:

$$\theta_y = 0.5 \varepsilon_y \frac{l_b}{h_b} \tag{5}$$

where l_b is the beam span between column centerline

h_b is the overall beam depth

ε_y is the yield strain of flexure reinforcement

Step 5 Calculate yield displacement , Δ_y :

$$\Delta_y = \theta_y \cdot H_e \tag{6}$$

$$\Delta_y = 0.5 \varepsilon_y \frac{l_b}{h_b} H_e \tag{7}$$

Step 6 Calculate the displacement ductility, μ :

$$\mu = \frac{\Delta_d}{\Delta_y} \tag{8}$$

Step 7 Estimate the equivalent viscous damping:
The formula of the equivalent viscous damping is deduced from best fittings of certain experiments. Through this study, the following formula was adopted.

$$\xi = 0.05 + 0.565\frac{(\mu - 1)}{\mu\pi} \tag{9}$$

Step 8 Plot the elastic displacement response spectrum for (ξ =0.05)
Firstly, the acceleration response spectrum is plotted from the code (according to type of soil and peak ground acceleration), then the displacement response spectrum is deduced using the following formula.

$$\Delta_{T,5} = \frac{T^2}{4\pi^2} a_{T,5} \tag{10}$$

where: $\Delta_{T,5}$: Response displacement at ξ = 0.05 (5%)
$a_{T,5}$: Response acceleration at ξ = 0.05 (5%)

Step 9 Plot the displacement response spectrum for ($\xi = \xi_d$)
A damping modifier R_ξ is applied to the displacement spectrum obtained in the previous step to obtain the displacement spectrum at different levels of damping. The folowing eaqaution is the damping modifier R_ξ suggested by the Euro Code EC8 in 2003.

$$R_\xi = \left(\frac{0.10}{0.05 + \xi}\right)^{0.5} \tag{11}$$

Typical shapes for the displacement response spectrum at different damping ratios are shown in Fig. 2. From this figure, it can be found that when the damping increases, the corresponding displacements decrease.

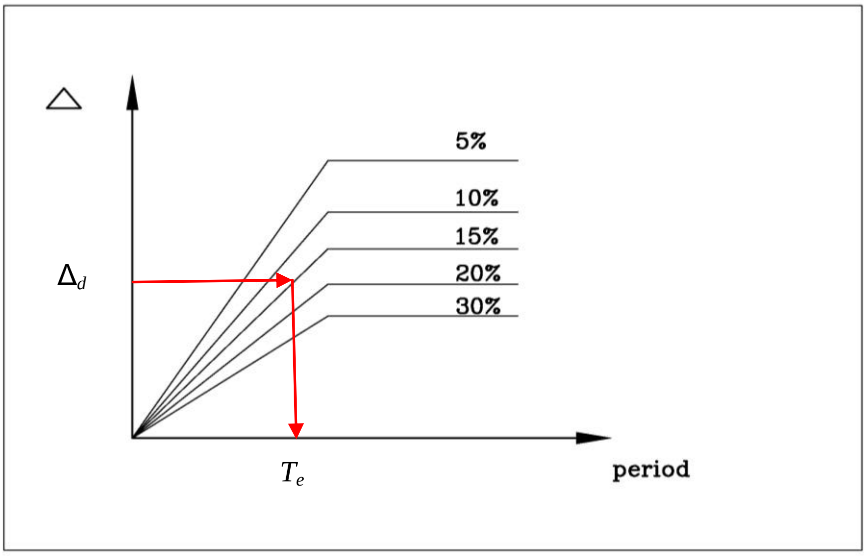

Fig. 2. Displacement response spectrums for different effective damping ratio [9]

Step 10 Calculate the effective period, T_e:
The effective period can be obtained from the displacement response spectrum (using the design displacement) (calculated from step 2)

Step 11 Calculate the effective mass, m_e (mass of the substitute structure):

$$m_e = \frac{\sum m_i \Delta_i}{\Delta_D} = \frac{\left(\sum m_i \Delta_i\right)^2}{\sum m_i \Delta_i^{\,2}} \tag{12}$$

Step 12 Calculate the effective stiffness of the building:

$$k_e = \frac{4\pi^2 m_e}{T_e^{\,2}} \tag{13}$$

Step 13 Calculate the design base shear force:

$$V_b = k_e \cdot \Delta_d \tag{15}$$

Step 14 Distribute the base shear force at different levels of the building using the following equation:

$$F_i = V \frac{m_i h_i}{\sum m_i h_i} \tag{16}$$

Step 15 Calculate the straining actions:
Using any finite element program same as (SAP) ,the building can be modeled then the forces are assigned at each floor level. Finally the corresponding straining actions and design moments at plastic hinge regions can be calculated.
Step 16 Design the structural elements and calculate the displacements at each level using the designed member dimensions.
Step 17 Compare the displacements with those assumed in step 1
If the calculated displacements are equal to those assumed in step 1, the design of the structural elements can be completed and the reinforcement can be designed. If the calculated displacements are not equal to those assumed in step 1 go back to step 2 and repeat the remaining steps using the displacements calculated in this step.

3. NUMERICAL ANALYSIS

The proposed equation for the inelastic displaced shape for any moment resisting frame building that is given by Eq. (2) is examined in this section.
2, 4, 8, 12, 16, and 20-storey frame building have been modeled to ensure that Eq. (2) gives reasonable values for the displacement at each floor. For this purpose three computer programs, specialized in the analysis of 2 and 3-dimensional frame building, were used applying the inelastic concepts. The computer programs are 2D-IDARC, SAP2000, and SeismoStruct.
These computer programs provide the ability to perform pushover analysis. In this type of analysis certain lateral loads are applied at each floor such that they are incrementally increased until the drift at first floor reach the required maximum drift specified by the code limits.
To perform accurate pushover analysis, the cross sections have been designed for the frame such that all beams are designed to resist the bending moments at their ends due to the earthquake loading in addition to the bending moment at mid span due to the vertical loads. After all columns are designed to resist the vertical loads in addition to the moments due to earthquake loading, these sections are assigned to the different elements of the frame building in the computer models.
A constant rectangular cross section with width and thickness equal to 400 mm and 900 mm, respectively, is used for all beams. The designs of the beam cross sections is limited to the determination of the reinforcing steel.
A constant square concrete cross section with side length equal to 800mm is used for all columns. The design of column cross sections is limited to the determination of the reinforcing steel using the interaction diagrams corresponding to the used steel reinforcement yield stress. The steel bars are assumed uniformly distributed in all columns.

4. RESULTS

4.1. Comparison between the Inelastic displaced shape of the studied buildings:

Three methods have been used to plot the inelastic displaced shape of the studied buildings so as to compare it with the one deduced from the DDBD procedure.

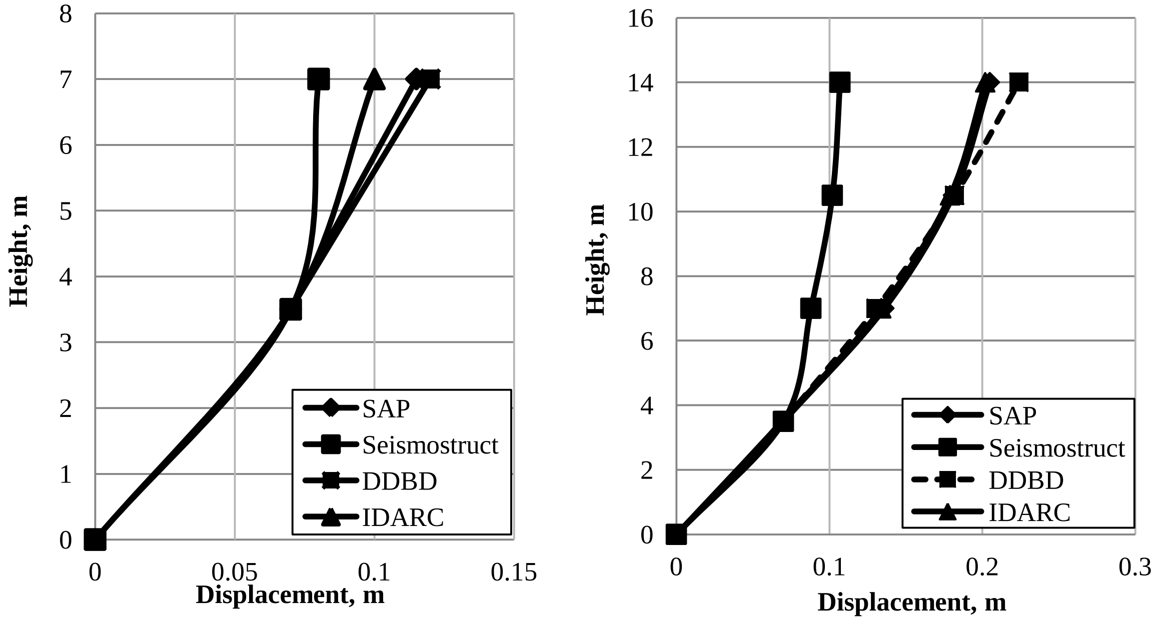

Fig. 3. Inelastic displaced shape for 2 and 4-storey buildings

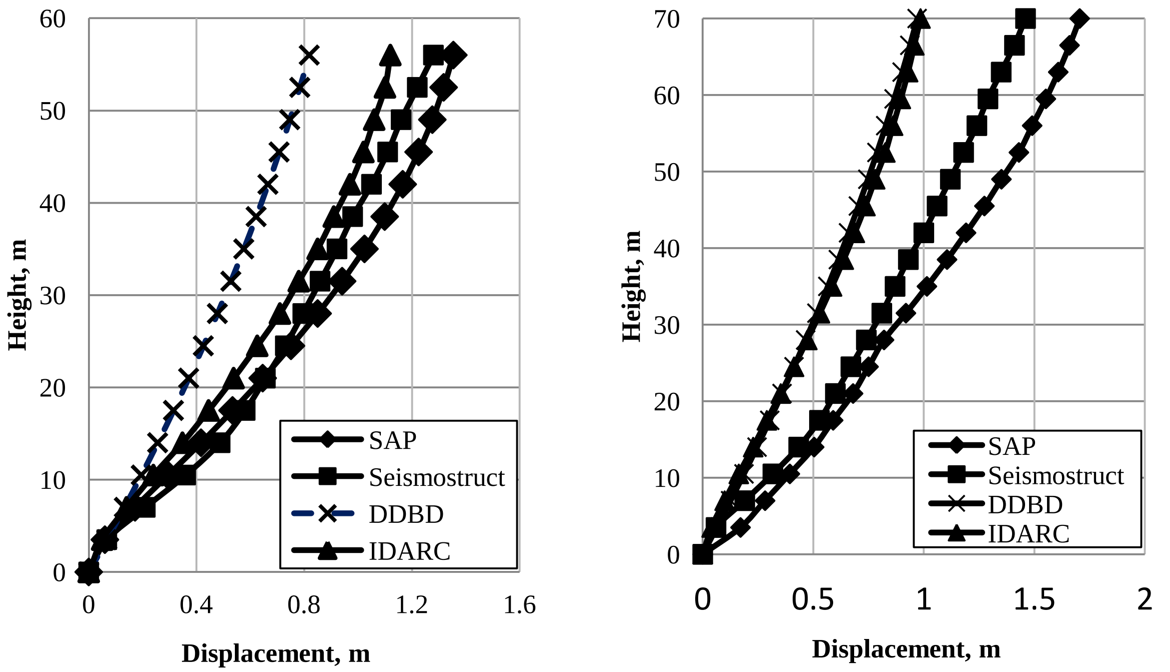

Fig. 4. Inelastic displaced shape for 8 and 12-storey buildings

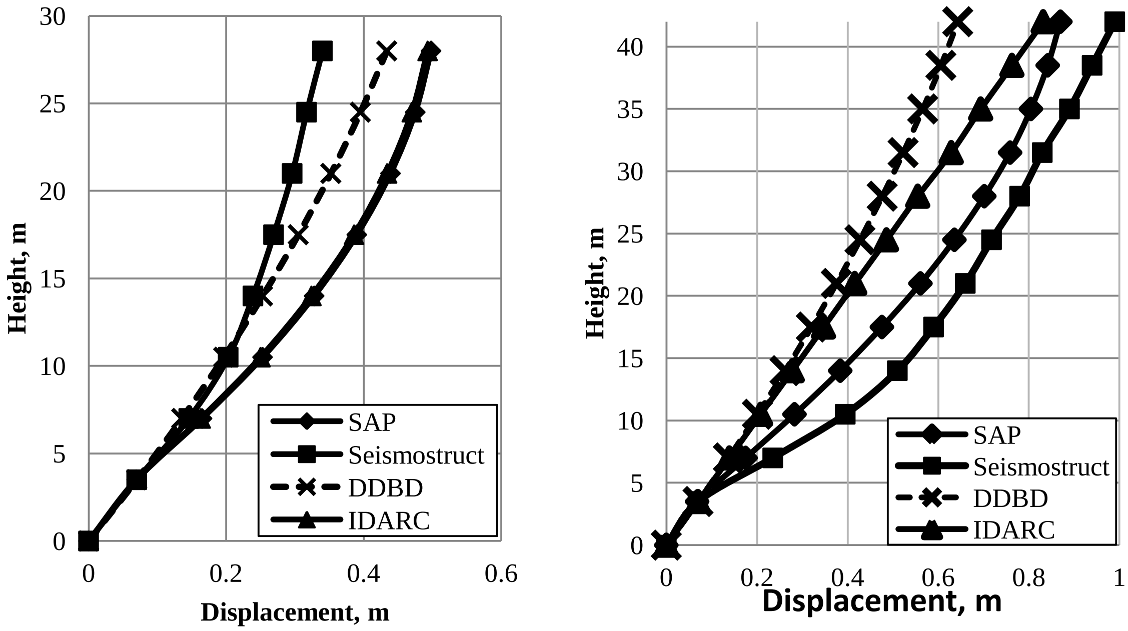

Fig. 5. Inelastic displaced shape for 16 and 20-storey buildings

From previous plots, the following can be concluded:

1. . For the 2, 4, and 8 storey buildings, the inelastic displaced shape by DDBD procedure is larger than the displaced shape from the SeismoStruct program. So the DDBD will give small base shear force which confirms that the DDBD procedure is not suitable for buildings with small height.
2 For the 12, 16, and 20-storey buildings, the inelastic displaced shape from the DDBD has lower displacements than those obtained by all other programs. Referring to the DDBD procedure, it was found that when the displacements in the displaced shape decrease, the corresponding base shear increases. It can therefore be inferred that the displaced shape by the DDBD procedure is the most conservative as it gives the largest base shear force.
   For the 12, 16, and 20-storey buildings, the ratio between the displacements by DDBD procedure and the pushover analysis by SeismoStruct are approximately from 0.57.
3. The SAP program and the SeismoStruct program give approximately near values for displacements for all buildings (The differences between them don't exceed 0.18 m, approximately). This gives indication that these programs provide suitable and logical values for the displacements of the displaced shape. **SAP** does not take into account the stiffness degradation (decrease) at each step- the assigned values of reduced stiffnesses are used throughout the whole steps- when the pushover loads are applied. **Seismostruct** takes this phenomenon into consideration such that as the pushover load increases, the stiffness of the beams and columns decrease [10]. Additionally, SAP deals with linear stress-strain curve while SeismoStruct uses non-linear stress-strain curves. These differences may interpret the difference in the displaced shape between the two programs
   The **IDARC** program takes the effect of high stiffness of the beam column connection, so beam and column elements can include a rigid length zone to simulate the increase in their stiffness at this zone. The effect of ridge zone is not considered in the SAP and SeismoStruct models. This may interpret the difference of the displaced shape between the models by these programs. The difference in displacements is about 15% in the 12, and 16 storey building, while it reaches to 35% for the 20 storey building.

4.2 Comparison between the base shear calculated by FBD and the base shear by DDBD at different ground accelerations:

The base shear calculated by FBD according to the Euro code (EC8) is compared with that calculated using the DDBD for different building heights. The analysis was carried on 2,4,8,12,16, and 20 storey buildings. The main supporting elements at these buildings were reinforced concrete frames (i.e. the seismic forces are resisted mainly by means of columns and beams with their rigid connections).

The ratio between the base shear calculated from DDBD, $V_{B,\ DDBD}$ to the base shear calculated by FBD, $V_{B,\ FBD}$ has been plotted. A reference line was also plotted at each graph at which $V_{B,DDBD}= V_{B,\ FBD}$ so as to simplify the comparison process and to clarify the ranges of acceleration at which the two forces are equal, and the other ranges at which the two forces are unequal..

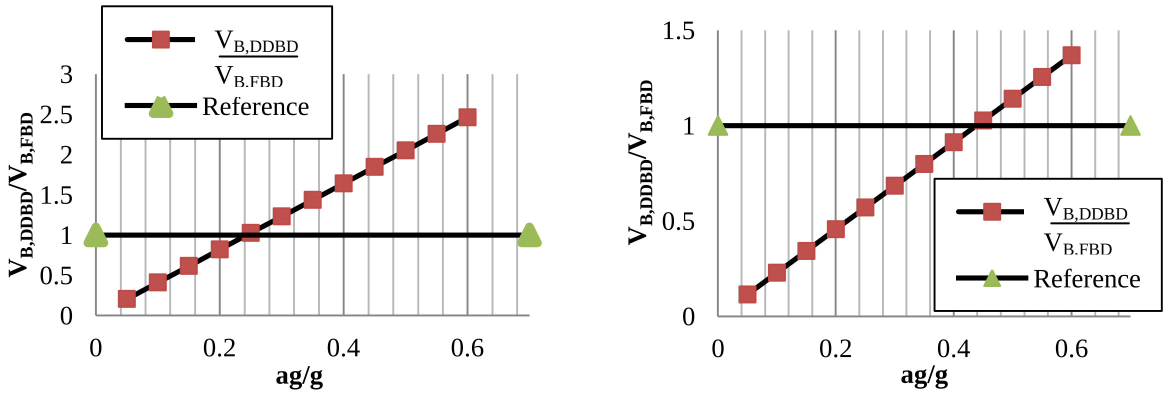

Fig. 6. The ratio between the base shear calculated by DDBD and FBD for 2and 4 -storey buildings for different values of ground accelerations

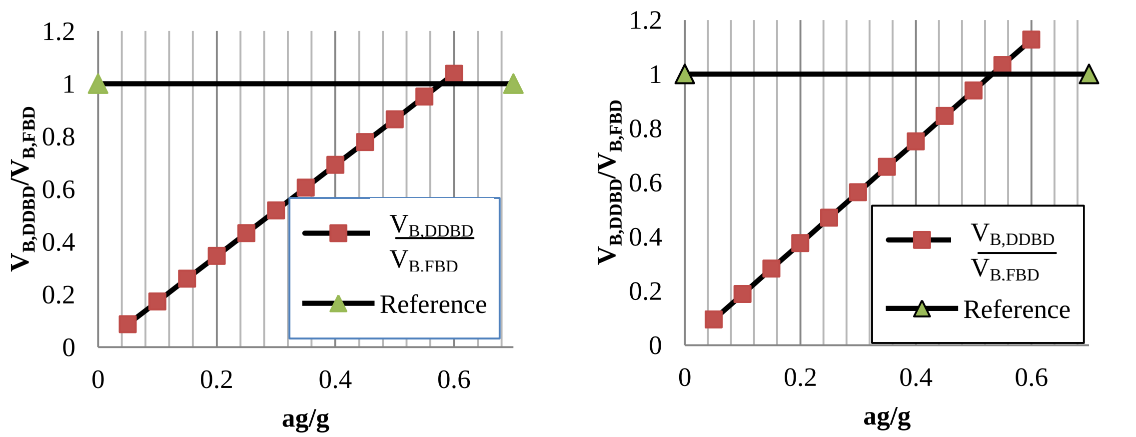

Fig. 7. The ratio between the base shear calculated by DDBD and FBD for 8 and 12-storey buildings for different values of ground accelerations

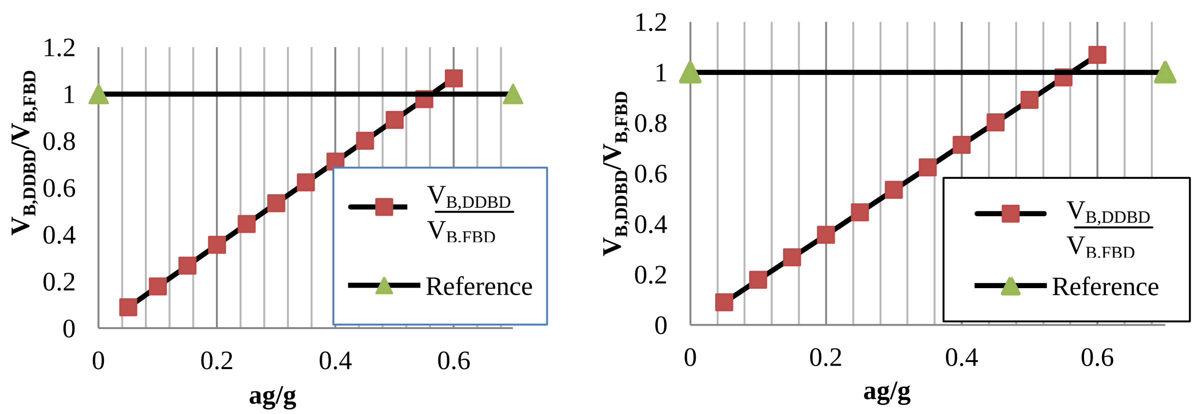

Fig. 8. The ratio between the base shear calculated by DDBD and FBD for 16 and 20-storey buildings for different values of ground accelerations

From the previously outlined graphs, the following results can be concluded:

1. As the value of the ground acceleration increases, the ratio between $V_{B, DDBD}$ and $V_{B, FBD}$ increases and this increase was found to be linear. The ratio between the two forces is 0.1 at ground acceleration 0.05g, while this ratio increases to 1.1 at ground acceleration 0.6g.
2. For very low values of ground acceleration (0.05g), the $V_{B, DDBD}$ is lower than the $V_{B, FBD}$. This trend is observed to be valid until 0.55g.
3. For very large values of the ground acceleration (0.6g), the $V_{B, DDBD}$ is higher than the $V_{B, FBD}$. This trend is observed to be valid after 0.55g.
4. For 2, and 4 Storey buildings , At intermediate values of the ground acceleration, the $V_{B,DDBD}$ is approximately equal to the $V_{B,FBD}$ and these values are different from a building to the other such that:

   For 2-storey building $V_{B, DDBD} = V_{B, FBD}$ at ground acceleration =0.25g
   For 4-storey building $V_{B, DDBD} = V_{B, FBD}$ at ground acceleration =0.45g
5. For 8, 12, 16, and 20 Storey buildings , the $V_{B,DDBD}$ is approximately equal to the $V_{B,FBD}$ at high values of the ground accelerations but these values are different from a building to the other such that:

   For 8-storey building $V_{B, DDBD} = V_{B, FBD}$ at ground acceleration =0.57g
   For 12-storey building $V_{B, DDBD} = V_{B, FBD}$ at ground acceleration =0.55g
   For 16-storey building $V_{B, DDBD} = V_{B, FBD}$ at ground acceleration =0.55g
   For 20-storey building $V_{B, DDBD} = V_{B, FBD}$ at ground acceleration =0.58g
6. The ground acceleration in the Euro code (EC8) ranges between 0.1g to 0.25g. At 0.1g, the ratio between $V_{B, DDBD}$ and $V_{B, FBD}$ is approximately 0.2 ,but at 0.25g, the ratio between $V_{B, DDBD}$ and $V_{B, FBD}$ is approximately 0.5. This result shows how the FBD –adopted by the Euro code (EC8) - gives large values of the base shear force compared with those calculated by DDBD.
7. In this research, similar results were obtained for the studied buildings at ground acceleration of 0.6g (by Pettinga, J.D., and Priestley M.J.N [9]). This indicated that the procedure of DDBD is correctly applied at this research. Based on these results, DDBD was applied at different ranges of ground acceleration so as to measure the applicability of this procedure at the range of ground accelerations in the Euro code (EC8).

4.3 Comparison between the base shear calculated by (FBD) and the base shear by (DDBD) at constant ground accelerations:

The base shear force has been calculated for the studied buildings at ground acceleration =0.3g using different approaches which are:

1. DDBD procedure.

Table 1. Summary of calculations by DDBD procedure

Number		(m)	(m)		(KN.sec2/m)	(m)		(sec)	(KN/m)	(KN)
of floors	θ_d	Δ_d	Δ_y	μ_y	m_e	H_e	ζ_{eff}	T_e	K_e	V_b
2	0.02	0.102	0.035	2.911	209.5	5.71	16.81	1.34	4597	466
4	0.02	0.174	0.063	2.776	391.65	10.23	16.5	2.27	2985	518
8	0.02	0.321	0.118	2.718	747.3	19.31	16.37	4.19	1676	537
12	0.02	0.469	0.174	2.701	1100.78	28.41	16.33	6.12	1158	543
16	0.02	0.235	0.229	2.584	1453.67	37.51	16.02	7.68	972	576
20	0.02	0.222	0.285	2.451	1806.32	46.61	15.65	8.97	886	618

2. FBD procedure (from the Euro code (EC8))
3. The period from DDBD procedure and using it with the elastic acceleration response spectrum from the Euro code (EC8).
4. The period from the modal analysis in the SAP model (with gross (elastic) moment of inertia for the cross sections) and using inelastic acceleration response spectrum from the Euro code (EC8) (i.e. using the acceleration response spectrum with R-factor equals 5).

5. The period from the modal analysis in the SAP model (with reduced moment of inertia for the cross sections) and using inelastic acceleration response spectrum from the Euro code (EC8) (i.e. using the acceleration response spectrum with R-factor equals 5).

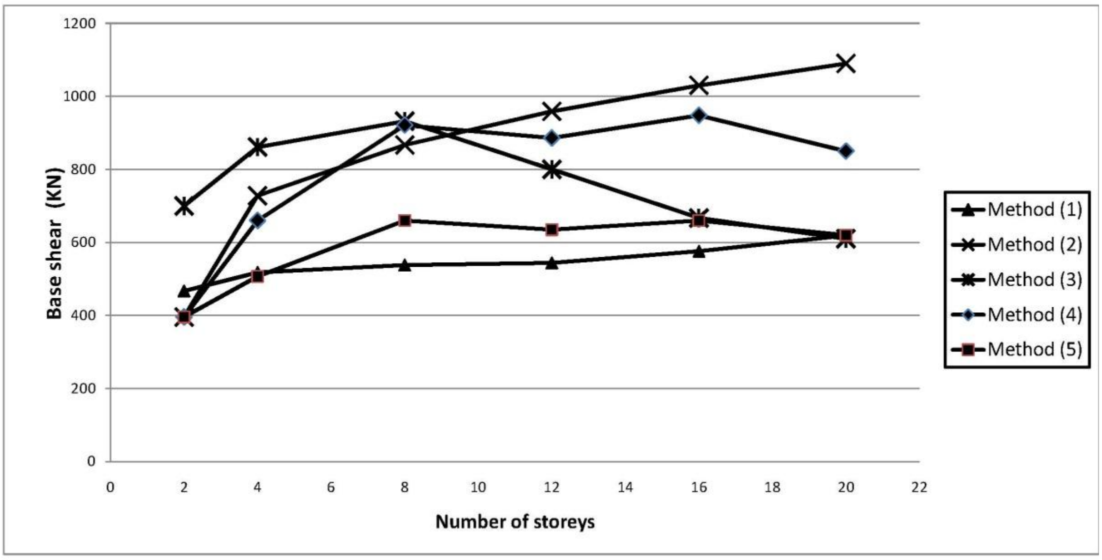

Fig. 9. Calculation of the base shear for the studied frames at constant ground acceleration

From the graph, the following results can be derived:

1. Method (1) gives the least value of the base shear force for all buildings except for the 2, and 4-storey buildings. There is a small increase such that for the 2-storey building , the base shear by method (1) gives 467 KN while methods (2), (4), and(5) give 396 KN
2. Method (2) gives the maximum value for the base shear force for the 12, 16, and 20 storey buildings. This shows that the FBD gives larger values than any other method by DDBD.
3. Method (3) gives the maximum value for the base shear force for 2, 4, and 8 storey buildings .This is a logical result as the period by DDBD is always larger than the period from the Euro code (EC8). Consequently the acceleration will be larger than the acceleration by FBD for ($T<T_B$) which results in large value of the base shear.
4. The base shear forces given by method (1) and method (5) are approximately the same. The ratio between the two forces ranges from 0 to 15%. This infers that the DDBD procedure can be represented by using the period from the SAP model with reduced moment of inertia of the element cross sections and using inelastic acceleration response spectrum.
5. For the 8, 12, 16, and 20-storey buildings, the ratio between the base shear force calculated by method (2) to the base shear force calculated by method (1) is approximately $\frac{1090}{619} \approx 1.8$ .This interprets that the base shear force by FBD is approximately twice the base shear force by DDBD at a ground acceleration of 0.3g.

4.5 Comparison between the stiffness of the studied frame buildings:

The stiffness representing the whole building (effective stiffness) has been calculated for each building using three different methods which are:

a. Using pushover analysis from SAP program.
b. Using pushover analysis from SeismoStruct program.
c. Using DDBD procedure.

4.5.1 Using pushover analysis from SAP program:

The SAP model has been used for each building to calculate the effective stiffness by means of calculating the base shear and getting the maximum displacement at top floor using Eq. (15).

4.5.2 Using pushover analysis from SeismoStruct program:
The base shear at this program can be found directly as any certain output of the analysis, and then using the displacement at top of the building the effective stiffness can be obtained for any building.

$$K = \frac{V_B}{\Delta} \tag{16}$$

4.5.3 Using DDBD procedure:
This procedure calculates the effective stiffness simply using the Eq. (12).

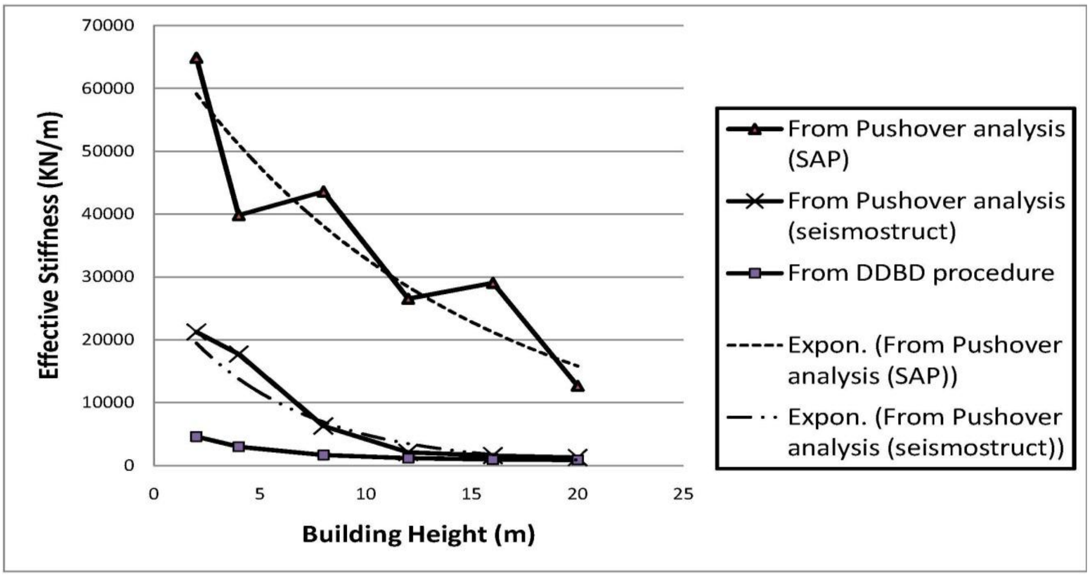

Fig. 10. Stiffness of studied buildings from different methods

From the Fig. 10, the following can be deduced:

- The stiffness calculated by pushover analysis by SAP program is larger than the stiffness calculated by pushover analysis using SeismoStruct program. Such that, For 2, 4,8,12,16,20 studied frame buildings, the ratio between the stiffness from SAP to the stiffness from SeismoStruct are 3, 3.5, 4, 6, 7, 8.5 respectively This result can be interpreted by referring to the stiffness of the cross sections of each model in the two programs. The SAP program doesn't take into account the stiffness degradation (decrease) at each step when the pushover loads are increased. While the SeismoStruct program takes this phenomenon into consideration such that as pushover load increases, beam and column stiffnesses decrease. This asserts that the stiffness by SAP program is more than the stiffness from SeismoStruct model.
- The stiffness by DDBD is approximately equal to the stiffness by SeismoStruct program for the 12, 16, and 20-storey building. This shows that the DDBD procedure is suitable for buildings which have number of storeys larger than 8.
- For buildings with number of storey less than 8, the stiffness from DDBD is dramatically less than the stiffness from the SeismoStruct. This shows that this procedure may not be suitable to be used for buildings with this number of storeys .It is found that for 2-storey building the ratio between the stiffness by DDBD and the stiffness by SeismoStruct program approximately equals to $\frac{4597}{21250} \approx 0.22$
- Due to high variability in the line representing both the stiffness form the SAP program and the stiffness from the SeismoStruct program, an exponential trend line have been added so as to simplify the comparison processes as shown in Fig. 10. From this figure, it can be found that the ratio between stiffness by SAP program to stiffness by SeismoStruct program is $\approx \frac{60000}{20000} \approx 3.0$ and $\approx \frac{16000}{2000} \approx 8$ for the 2-Story building and 20- storey building respectively.

5. CONCLUSIONS:

From analysis and results of this research the following conclusions can be derived:

1. For low values of seismic intensity(less than 0.5g) for buildings with number of storeys more than 8 storeys-the base shear given by FBD using the Euro code (EC8) for loads is higher than the base shear given by (DDBD). The ratio between the two forces is 0.1 at ground acceleration 0.05g, while this ratio increases to 1.1 at ground acceleration 0.6g. This ratio is approximately linearly varying between 0.05g and 0.6g.
2. For high values of seismic intensity (approximately 0.55g)-for buildings more than 8 storeys- the base shear given by (FBD) using the Euro code (EC8) for loads is approximately equal to the base shear given by direct displacement based design.
3. For values of ground acceleration higher than 0.55g, with buildings having more than 8 storeys, it is found that the base shear by (DDBD) is higher than the base shear by (FBD) this ratio is 1.1 at ground acceleration 0.6g., so it is recommended to design the building on the largest value of the base shear to ensure the safety of these tall buildings.
4. The base shear force by (DDBD) gives values near the values of base shear force by (FBD) but using the elastic period for the building with reduced moment of inertia of the cross sections and using the inelastic acceleration response spectrum. The ratio between the two forces ranges from 0 to 15%
5. The period given by (FBD) proposed by the Euro code (EC8) is less than the period given by (DDBD) because the period given by (FBD) represents the building at its elastic stage while the period given by (DDBD) represents the building at its inelastic phase. Such that, the ratio between the periods is 0.4 at 8 storey building, but this ratio increases to 1.3 at 20 storey building.
6. When applying the Euro code (EC8), at ground acceleration of 0.1g, the ratio between $V_{B,\,DDBD}$ and $V_{B,\,FBD}$ is approximately 0.2 ,but at at ground acceleration of 0.25g, the ratio between $V_{B,\,DDBD}$ and $V_{B,\,FBD}$ is approximately 0.5. This indicates that the Euro code (EC8) gives large values of the base shear force compared with those calculated by DDBD.
7.. The DDBD procedure can be represented by using the period from the SAP model with reduced moment of inertia of the element cross sections - using the beams and columns stiffness reduction factors- and using inelastic acceleration response spectrum The ratio between the two forces ranges from 0 to 15%

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