ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

Assessment of Direct Displacement Based Seismic Design Philosophy for Reinforced Concrete Frame Buildings.

A Thesis Submitted to the Graduate School of the Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structural Engineering)

By

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Advisor: ADIL ZEKARIA (Dr.-Ing)

November, 2011

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DECLARATION

I, the undersigned, declare that this thesis is my work and all sources of materials used for the thesis has been duly acknowledged.

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a_g	Ground acceleration
C_s	Soil correction
DDBD	Direct displacement based design
E	Elastic modulus of elasticity
FBD	Force based design
h_b	Section depth
H_e	Effective height
I	Second moment of inertia
K_e	Secant stiffness
K_i	Initial elastic stiffness
L_b	Span length
L_p	Plastic hinge length
MDOF	Multi degree of freedom
n	Number of stories
PGA	Peak ground acceleration
PGD	Peak ground displacement
SDOF	Single degree of freedom
S_e	Acceleration response spectrum
T	Structural period
T_c	Corner period
T_e	Effective period
V_{base}	Base shear
δ	Inelastic mode shape
Δ_d	Design displacement
$\varepsilon_{c,ls}$	Limit state strain for concrete
$\varepsilon_{s,ls}$	Limit state strain for steel
μ	Ductility
ξ	Damping
ξ_{el}	Elastic viscous damping
ξ_{eq}	Equivalent viscous damping
ξ_{hyst}	Hysteretic viscous damping
$\Phi_{ls,c}$	Limit state curvature on concrete
$\Phi_{ls,s}$	Limit state curvature on steel

Abstract

Structural displacements and member deformations do not enjoy a primary role in the current force-based seismic load resistance design. Their absolute magnitude is of interest only for aspects considered of secondary importance for seismic performance and safety. In the main phases of current force-based design, namely that of member dimensioning for given strength demands and that of detailing, structural displacements and member deformations enter in an average sense and indirectly. Recent years have seen, however, an increased interest in absolute magnitude of displacement and deformations as the basis of seismic design. The main reason for this is the recent recognition that displacement rather than strength, demands and capacities is what determines seismic performance and safety. Since an earthquake is dynamic action, it represents for a structure a demand to withstand certain displacement and deformations, but not specific forces. In this work the recent displacement based design philosophy is assessed, examined and compared with the current traditional force-based design philosophy as applied for reinforced concrete frame buildings by taking representative frame buildings without structural wall to be analyzed using both methods so that the limitations and suitability of the recent method can be drawn as well as the parameters to be considered is examined by taking span length, story height, beam depth and number of stories as a parameter. The parametric study shows that this new design philosophy accounts parameters those affect the response of the structure directly in transparent way. Specially, the span length and beam depth have significant influence on the resulting strength requirement. It is also simple for practical application in addition to its basic on convincing and logical theoretical back ground. In this thesis work, a lot of effort have been done to introduce the world wide trend of shifting the design philosophy towards displacement-based which and also help further researchers to insight the detail researchable areas those guide for the revision of our seismic design building standard codes.

Key words: direct displacement based design, design displacement, substitute structure, equivalent damping.

CHAPTER ONE

1 Introduction

1.1 Background

Earthquake induces forces and displacements in structures. For elastic systems these are directly related by the system stiffens, but for structures responding inelastically, the relationship is complex, being dependant on both the current displacement and the history of displacement during the seismic response. Traditionally, seismic structural design has been based primarily on forces. The reason for this is largely historical, and related to how we design for actions such as dead and live load. For such cases we know that force considerations are critical; if the strength of the design structure does not exceed the applied load, the failure will occur.

However, it has been recognized for some considerable time that strength has less importance when considering seismic actions. We regularly design structures for less than calculated elastic force levels, because we understand that the well designed structures posses ductility, and can deform inelastically to the required deformations imposed by the earthquake without loss of strength. This implies damage, but not collapse. Since design level of earthquake are by definition rare events, we accept the possibility of damage under the design earthquake as economically acceptable, and benefit economically from the reduced construction costs associated with the reduced design force level. In general, for inelastic systems, the strength is less important than the displacement. It would thus seem more logical to use displacement as basis for design. For elastic systems; it is exactly equivalent to use either displacement or force as fundamental design quantity.

Seismic design is currently based on force (and hence acceleration) rather than displacement, which is based on strength consideration. As the importance of displacement has come to be better appreciated, the approach has been to attempt to modify the existing force-based approach to include consideration of displacement, rather than to rework the procedure on more rational basis. In the last decade, however, researchers started pointing out this inconsistency, proposing displacement-based approaches for seismic load design. With the aim of providing imposed reliability in the engineering posses by more directly relating computed response and expected structural performance. [4]

The concept of seismic design based on limit displacement has been going credence over the past years, as it has become appreciated that the structural damage can be directly related to strain (and hence by integration to displacement), and non-structural damage, in buildings at least, can be related to drift. The inverse relationship between damage potential and strength, long held to be self evident, has proven to be illusory. [2]

As seismic damage is directly as well as closely related to displacement or deformation, research works highlight the importance of employing displacement as performance quantifier. Since the maximum displacement associated with particular performance or damage level is used as the starting design variable, displacement-based seismic design approach has been recognized as the most promising as well as effective tool for performance-based design to some extent performance-based design and displacement based design have been used interchangeably.[5]

In general, displacement-based seismic design is defined broadly as a seismic design method in which displacement-related quantities are used directly to judge performance acceptability, in contrast with force-based procedures in which the acceptability of a structure is judged on the basis of force-based quantities.

The main objective of this work is to asses, summarize, critically review and compare the new displacement-based design approach with the current traditional force-base design approach, in addition to showing the trend of variation with specific parametric consideration. Comparisons of the approaches, however does not imply the production of a scale of merits, but rather to assess the relative ease or difficulty with which the design methods can be applied and any apparent limitations the methods may have. In addition, it aims to compare the required strength for each method for each parametric case study.

1.2 Statement of the problem

Traditional seismic design methods are generally adapted from techniques assumed to be adequate for gravity load design (design approaches that emphasize strength instead of deformation). The design is based on an instantaneous snapshot of a dynamic event (elastic deformation) in which response at the time of maximum base shear is considered to be adequate. The member sizes are calculated via the distribution of this maximum base shear (elastic strength demand) along the height of the structure. During this process, the structural periods are calculated using section properties assuming that elastic (shorter) period will usually result in higher design forces. In brief, traditional methods do not explicitly consider the duration effects of ground motions and hysteretic behavior of structural members, which in turn affect the overall force and deformation patterns on the structure. As yet, seismic design codes are developed to accomplish the above effects implicitly by checking deformation limits (even extending to inelastic range) after sizing the structural members from the calculated elastic forces. However, there is a lack of consensus among different seismic codes about these empirical formulations for converting the calculated elastic displacements to inelastic ones. The above discrepancies of traditional seismic design methods are well known and it is recognized that this method do not provide a structure that responds and performs well in seismic events in addition to its illogical basis of the philosophy. However the lack of alternative design approaches has made them essential for daily design practice.

Here in this thesis work it is kept in mind that to assess and examine whether displacement-based seismic design is logical, simpler for practical application, safe and economical method of design philosophy as compared with the traditional force-based design philosophy having the mentioned problems in it.

1.3 Objectives

General objective:

The objective of this thesis is therefore to introduce a theoretical background, its compliance to the real world and to assess suitability of the displacement-based design philosophy procedure for reinforced concrete frame buildings. In doing so, the result gives structural engineers options of analysis and design approaches capable of application to reinforced frame buildings.

Specific objectives

- To know and understand the design procedure of frames on the basis of displacement.
- To see the advantage and limitation of displacement based design
- To compare it with the current force- based design philosophy and show the trend.
- To show others the researchable area about the flourishing design concept.
- To increase the awareness of seismic design of structures on the basis of displacement.
- To serve as an input for the coming code revisions.

1.4 Methodology

The complete work of this thesis is divided in to three stages. The first part involves Detail literature review of previous works, journals and books has been be conducted for understanding of the basic concepts theories and procedures of the new design philosophy in general and its application for reinforced concrete frame building in particular. The second stage involves performing a parametric study on analysis and design of reinforced concrete frame building using displacement based and current traditional force based analysis and design procedures for parameters of number of stories, story height, bay width and depth of the beam variations. This is in order to see the procedure, their suitability, to assess the relative ease or difficulty with which the design methods can be applied and any apparent limitations the methods have as well as to compare the required strength for each method and the performance of the methods in terms of predicted ductility or drift values. The final stage is comparing and comments the two applied methods by their suitability, strength requirement and ease of application and noticeable significant variation.

The work covered by this thesis is limited only for reinforced concrete frame buildings without structural wall. It does not also consider damping equipment effects for seismic load resistance.

CHAPTER TWO

2. General Introduction for Direct Displacement Based Design

2.1 Historical consideration

The reason that seismic design is currently based on force (and hence acceleration) rather than displacement, is largely on historical considerations. Prior to the 1930's, few structures were specifically designed for seismic actions. In the 1920's and the early 1930's several major earthquakes occurred. It was noticed that had been designed for lateral wind force performed better in these earthquakes than those without lateral force design. As a consequence, design codes started to specify that structures in seismic regions be designed for lateral inertial forces. Typically, a value of about 10% of the building weight, regardless of the building period, applied as a vertically distributed lateral force vector, proportional to the mass vector, was specified.

During the 1940's and 1950's, significance of structural dynamics become better understood, leading by 1960's; to period- dependent design lateral force level in most seismic design codes. Relations between ductility and for force reduction factor and others such as equal energy approximation were developed as a basis for determining the appropriate design lateral force levels.

During the 1970's and 1980's much research effort was directed to determining the available ductility capacity of different structural systems. Ductility consideration became a fundamental part of design. This may be seen as the first departure from force as the basis for design. Required strength was determined from a force-reduction factor that reflects the perceived ductility capacity of structural system and material to be chosen for design. Nevertheless, the design process was still carried out in terms of required strength, and displacement capacity, if directly checked at all, was the final stage of the design. Also during this era the concept of "capacity design" was introduced.

In the 1990's, further emphasis on displacement considerations and capacity design become widely used for seismic design of structures and the concept of "performance-based seismic design", based largely on displacement considerations become the subject of intense research attention.[4] It may be seen from this brief description of the history of seismic design, that initially design was purely based on strength, or force, considerations using assumed rather than valid estimates of elastic stiffness. As the importance of displacement has come to be better appreciated in recent years, the approach has been to attempt to modify the existing force-based approach to include consideration of displacement, rather than to rework the procedure to be based on a more rational displacement basis. The question of why force-based earthquake design procedures become prevalent while displacement as the governing parameter become regretted to a secondary (and still not well understood role), probably rests with the traditional engineering quest of providing strength as the only of safety.[6]

2.2 Force based seismic design

Although current force based design is considerably improved compared with procedures used in earlier years, there are many fundamental problems with the procedure, particularly when applied to reinforced concrete structures.[4] In order to examine theses problems it is first necessary to briefly review the force-based design procedure, as applied in modern seismic design codes. The sequence of operations required in force-based seismic design is summarized in Fig.2.1

1. The structural geometry, including member sizes estimated.
2. Member elastic stiffnesses are estimated, based on the preliminary estimate of member size. Some codes account softening of reinforced concrete members due to expected cracking by reducing section stiffness while others use gross sectional stiffness.
3. Based on the assumed member stiffness, the fundamental period (equivalent lateral approach) or periods (multiple-mode dynamic analysis) are calculated.
4. The design base shear for the structure corresponding to elastic response with no allowance for ductility is calculated.
5. The appropriate force reduction factor corresponding to the assessed ductility capacity of the structural system and material is selected. Generally the reduction factor is specified in the code not a design choice.
6. The shear force is then distributed to different part of the structure to provide the vector applied seismic forces. For building structures, the distribution is typically proportional to the product of the height and mass at different levels. The total seismic force is distributed between different lateral force resisting elements in proportion to their elastic stiffness.

7. The structure is then analyzed under the vector of lateral seismic design forces, and the required moment capacity at potential location of inelastic action is determined.
8. Structural design of the member sections at plastic hinge locations is carried out, and displacements under the seismic action are estimated.
9. The displacements are checked with the code specified displacement limits.
10. If the calculated displacements exceeded the code limits, redesign is required. This is normally affected by increasing member sizes, to increase member stiffness.
11. If the calculated displacements are satisfactory, the final step of the design is to determine the required strength of actions.

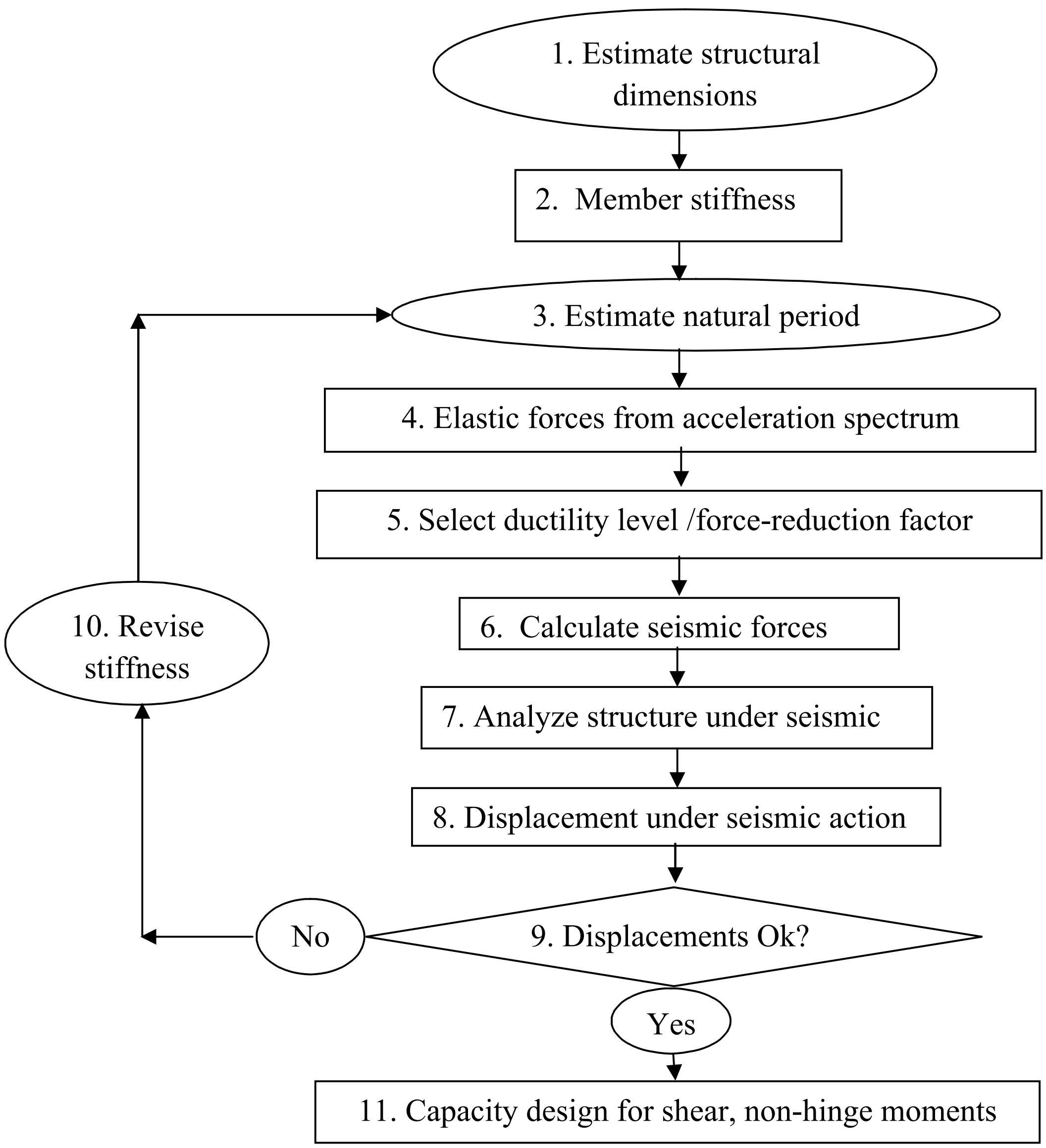


Fig 2.1 sequence of operations for force – based design [4]

2.3 Problems with for force-based seismic design

2.3.1 Inter dependency of strength and stiffness.

A fundamental problem with force-based design particularly when applied to reinforced concrete structures is the selection of appropriate member stiffness. Assumptions must be made about member sizes before the design seismic forces are determined. These forces are then distributed between members in proportion to their assumed stiffness. Clearly if the member sizes are modified from the initial assumption, then the calculated design forces will no longer be valid, and recalculation, though rarely carried out, is theoretically required.

With regard to reinforced concrete structures, a more important consideration is the way in which individual member stiffness is calculated. The stiffness of a components or element is sometimes based on the gross-section stiffness and sometimes on reduced stiffness to represent the influence of cracking. Regardless of what assumption is made, the member stiffness is traditionally assumed to be independent of strength, for a given member section. Detailed analysis and experimental evidence show that the assumption is invalid, in that stiffness is essentially proportional to strength.[4]

As a consequence of these findings it is not possible to perform an accurate analysis of either the elastic structural periods, nor of the elastic distribution of required strength throughout the structure, until the member strength have been determined. Since the required member strength are the end product of force-based design, the implication is that successive iteration must be carried out before an adequate elastic characterization of the structure is obtained. Although this iteration is simple, it is rarely performed by designers and does not solve additional problems associated with initial stiffness representation.

The assumption that the elastic characteristics of the structure are best indicator of inelastic performance, as implied by force based design is in itself clearly of doubtful validity. With reinforced concrete structures the initial elastic stiffness will never be valid after yield occurs, since stiffness degrades due to crushing of concrete. It would seem obvious that structural characteristics that represented performance at maximum response might be better predictor of performance at maximum response than the initial values of stiffness and damping. [3]

2.3.2 Period calculation

As discussed in the previous section, considerable variation in calculated periods can result as consequence of different assumptions for member stiffness. When the height dependent equations common in several codes are considered it is shown that significant variation are observed due to their different design assumptions.[4]

It is often stated that it is conservative, and hence safe, to use artificially low periods in seismic design. However, as has been already discussed, strength is less of an issue in seismic design than is displacement capacity. Calculated displacement demand based on an artificially low period will also be low, and therefore non-conservative.

2.3.3 Structural Ductility capacity and force reduction factors

It has long been realized that the equal displacement approximation is inappropriate for both very short-period and very long period structures and is also of doubtful validity for medium period structures when hysteric character of the inelastic system deviates significantly from elasto-plastic. Furthermore, there has been difficulty in reaching consensus within the research community as to the appropriate definition of yield and ultimate displacements.[4]

A key principle of force based design, as currently practiced, is that unique ductility capacities, and hence unique force-reduction factors can be assigned to different structural systems.[3] However, it is noted that different codes will provide different force reductions for identical systems and materials. So, it can be seen that the validity of this principle to be inappropriate. And the concept of uniform displacement ductility capacity and hence of a constant force reduction factor is inappropriate.

2.3.4 Relation between strength and ductility demand

A common assumption in force-based design is that increasing the strength of a structure (by reducing the force-reduction factor) improves its safety. Using the common force-based assumption that stiffness is independent of strength, for a given section, it is seen that increasing the strength reduces the ductility demand, since the final displacement remains the same. It has already been noted, that this assumption is not valid. However, we continue, as it is essential to the argument that increasing strength reduces damage.

2.3.5 Structures with dual (elastic and inelastic) load paths.

A more serious deficiency of force-based design is apparent in structures which possess more than one seismic load path, one of which remains elastic while the others respond inelastically at the design earthquake level. It is probable that the maximum response displacement will differ significantly from the initial elastic estimate, since at maximum displacement, the effective damping of the system will less than expected, as hysteretic damping is only associated with load path of combined response.

2.3.6 Relation between elastic and inelastic displacement demand

Force-based design requires assumption to be made when determining the maximum displacement response. The most common assumption is the equal-displacement approximation, which states that the displacement of the inelastic system is the same as that of equivalent system with the same elastic stiffness, and unlimited strength. Force-based seismic design does not normally take account of the different hysteretic characteristics of different materials and structural systems. Thus the fact that seismic structures, is not directly considered, though different force-reduction factors may be assigned to different materials.

In general it is seen some of the problems associated with the force-based design. These can be summarized as:

- Force-based design relies on estimate of initial stiffness to determine the period and distribution of design force between structural elements. Since the stiffness is dependent on the strength of the elements, this cannot be known until the design process is complete.
- Allocating seismic force between elements based on initial stiffness is illogical for many structures, because it incorrectly assumes that the different elements can be forced to yield simultaneously.
- Force-based design is based on the assumption that unique force-reduction factors (based on ductility capacity) are appropriate for a given structural type and material. This is demonstrably invalid.

So we need a design method that equips with a mechanism to address those listed deficiencies of the current conventional force-based design method.

2.4 Development of displacement–based design methods.

2.4.1 Force based/ displacement checked

Deficiencies inherent in the force-based system of seismic design, some of which have been outline in the preceding sections, have been recognized for some time, as the importance of deformation, rather than strength, in assessing seismic performance has come to be better recognized. Consequently a number of new design methods, or improvements to existing methods, have recently developed. Initially the approaches were designed to fit within, and improve, existing force-based design. These can be characterized as force-based/ displacement checked where enhanced emphasis is placed on realistic determination of displacement demand for structures designed to force- based procedures.

Such methods include the adoption of more realistic member stiffness for deformation (if not required strength) determination, and possibly use of inelastic time- history analysis or pushover analysis, to determine peak deformation and drift demand. In the event that displacements exceed the code- specified limits, redesign is required. In general, no attempt is made to achieve a uniform risk of damage, or of collapse for structures design to this approach.

There are, however, problems associated with this approach. Although the yield displacements of the lateral-force resisting elements may be known at the start of the procedure, the equivalent system yield displacement will not be known until the distribution of strength between elements is decided. The approach relies on assumptions about the equivalence between elastic and ductile displacements may be invalid and considerable experience is required of the designer.[4]

2.4.2 Deformation calculation based design

A more refined version of the force-based /displacement-checked approach relates the detailing of critical section (in particular details of transverse reinforcement for reinforced concrete members) to the local deformation demand and may hence be termed deformation calculation based design. Strength is related to a force-based design procedure, with specified force reduction factors. Local deformation demands typically in the form of member end rotations or curvatures are determined by state-of-the-art analytical tools, such as inelastic pushover analyses or inelastic time-history analyses. Transverse reinforcement details are then determined from state-or-the-art relationships between transverse reinforcement details and local deformation demand.

The structure is initially designed for strength to requirements of direct combination of gravity load plus a serviceability level of seismic force, using elastic analysis methods. The designed structure is then analyzed using advanced techniques such as inelastic time-history analysis or inelastic pushover analysis to determine the required transverse reinforcement details.

2.4.3 Deformation-specification based design

Recently a number of design approaches have been developed where the aim is to design structures so that they achieve a specified deformation state under the design-level earthquake, rather than achieve a displacement that is less than a specified displacement limit. These approaches appear more philosophically satisfying than those of the preceding two sections. This is because damage can be directly related to deformation. Hence designing structures to achieve a specified displacement limit implies designing for a specified risk of damage, which is compatible with the concept of uniform risk applied to determining the design level of seismic excitation. It thus means that different structures designed to this approach will (ideally) have the same risk of damage, rather than the variable risk associated with current design approaches.

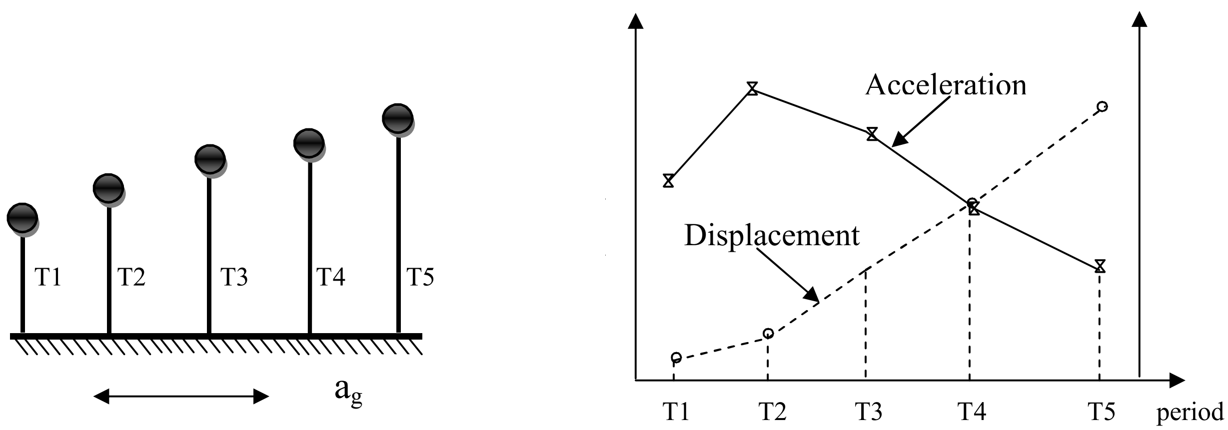
Different procedures have been developed to achieve this aim. The most basic division between them is on the basis of stiffness characterization for design. Some methods uses adopt the initial pre-yield elastic stiffness as in conventional force-based design. Generally some iteration is required, modifying initial stiffness and strength, to achieve the desired displacement.

The second approach utilizes the secant stiffness to maximum displacement, based on the substitute structure characterization and an equivalent elastic representation of hysteretic damping at maximum response. Generally these methods require little or no iteration to design structure to archive the specified displacement, and are hence known as direct displacement-based design.

2.5 Seismic input for displacement – based design, Response spectrum

Our understanding of the response of structures to earthquakes, and our design methodologies, either force-based or displacement-based, are critically dependent on recordings of strong ground motion by accelerographs. The fundamental information extracted from accelograms for use in design is typically expressed in the form of response spectra, which represent the peak response of single-degree-of freedom oscillators of different periods of vibration to accelogram. The quantities most commonly represented in response spectra are absolute acceleration (with respect to "at rest" conditions), and relative displacement (with respect to instantaneous ground displacement), though relative velocity response spectra are also sometimes computed.

Normally response spectra provide information on peak elastic response for a specified elastic damping ratio (typically 5%) and are plotted against the elastic period. It is, however, also possible to plot inelastic spectra related to displacement ductility levels. In this case the period may represent the initial elastic period, or the effective period at peak displacement demand, related to effective stiffness.



(a) SDOF Oscillators

(b) Elastic Response Spectrum

Fig. 2.2 formation of response spectra

2.5.1 Design elastic spectra

a) Elastic acceleration spectra: until recently, design spectra for seismic design of structure were typically specified in design codes as a special shape related to soil conditions, modified by design PGA, reflection the assessed seismicity of the region where the structure was to be built. Typically only acceleration spectra were provided and mapping of the variation of PGA with location. This is the case with many seismic design codes. However this is typically provided only for acceleration response spectra; displacement response is not available in many design codes. A typical acceleration response spectrum is shown below in Fig 2.4.The spectrum rises from PGA at T=0 to a maximum value at period T_A. The plateau typically has a response acceleration of 2.5 times the PGA. The acceleration plateau continues to a period of $T_{B,}$ the value of which depends on the ground conditions in the near-surface layers. For periods greater than T_B the response acceleration reduces, typically proportional to T up to Tc and T^2 beyond

Tc. The general form of the elastic acceleration spectrum can be defined by the following equations:

$0<T<T_A$: $Se(T) = PGA(1+(C_A-1)T/T_A)$ (2.1a)

$T_B<T<T_A$: $Se(T) = C_A.PGA$ (2.1b)

$T_B <T<T_C$: $Se(T) = C_A.PGA.(T_B/T)$ (2.1c)

$T>TC$: $Se(T) = C_A.PGA.(T_BT_C/T^2)$ (2.1d)

Where S_e is the spectral acceleration and C_A is the multiplier to the PGA to obtain the peak response acceleration. Figure 2.5 has been developed from these equations with the parameters for soil B of EBCS-8, T_A=0.15sec, T_b=0.6sec, C_A=2.5.

b) Elastic displacement spectra**:** Although many codes still do not define design displacement spectra, they are becoming more common. Ideally these should be developed separately, though using the same data, from acceleration spectra. However, most code-based design displacement spectra are generated from the acceleration spectra assuming that the peak response is governed by the equations of steady-state sinusoidal response.

A more general form of the elastic displacement response spectrum is defined by euro code EC8 and is shown in fig.2.3. This shows the linear displacement increase up to the corner period Tc, with a subsequent plateau of displacement up to period T_D, followed by decrease in displacement up to period T_E at which stage the response displacement has decreased to the peak ground displacement (PGD).

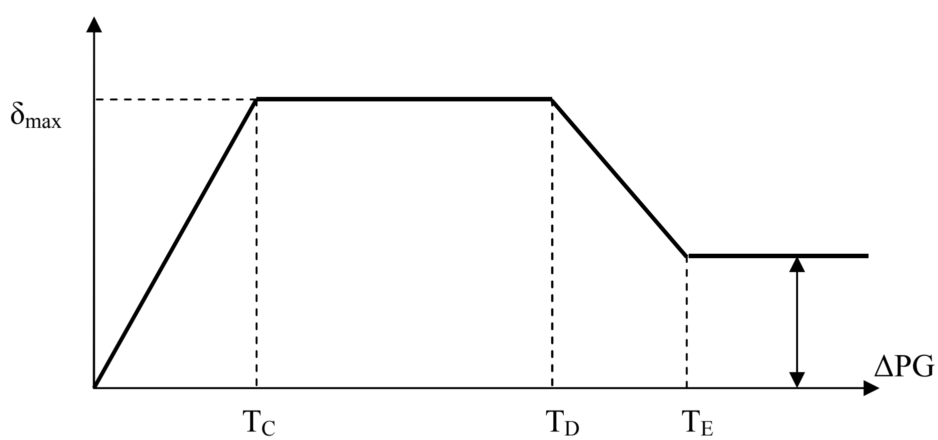


Fig 2.3 Elastic displacement response spectra from EC 8

Displacement-based seismic design using a secant stiffness representation of structural response requires a modification to the elastic displacement response spectrum to account for ductile response. The influence of ductility can be represented either by equivalent viscous damping or directly by inelastic displacement spectra for different ductility levels. The use of spectra modified by different levels of damping requires relationships between ductility and damping to be developed for different structural hysteric characteristics.

Since the equivalent properties of the substitute structure (which will be discussed in detail latter on) are elastic, a set of elastic displacement response spectra can be used for design. Therefore, the substitute structure approach allows an inelastic system to be designed and analyzed by using elastic displacement spectra. [1, 9]

The displacement spectrum is used to directly evaluate the displacement demand of the equivalent substitute structure. The elastic displacement response spectrum, $\Delta_{(T)}$, shall be obtained by direct transformation of the elastic acceleration response spectrum $Se_{(T)}$ using the following expression

$$\Delta_{(T)} = Se_{(T)}(-)^2 g \qquad (2.2)$$

However, for direct displacement based design, it may be necessary to adjust the displacement spectra for such use. Code based design spectra are normally developed for 5% damping, hence to modify the spectra for damping levels ξ_d other than 5%, the following equation from EC8 is used to give a corrected value of displacement demand.

$$\Delta(T,\xi) = \Delta(T,5) \;\text{——}\; ½ \qquad (2.3)$$

Where ξ_d is the design damping value

The response displacement resulting should be also modified for other than firm ground firm ground as suggested by [4] the following corrections are recommended.

Rock	Cs=0.7
Firm ground	Cs=1.0
Intermediate soil	Cs=1.4
Very soft soil	Cs=1.8

As per EBCS 8, the elastic acceleration response spectrum for the soil type B is shown as below:

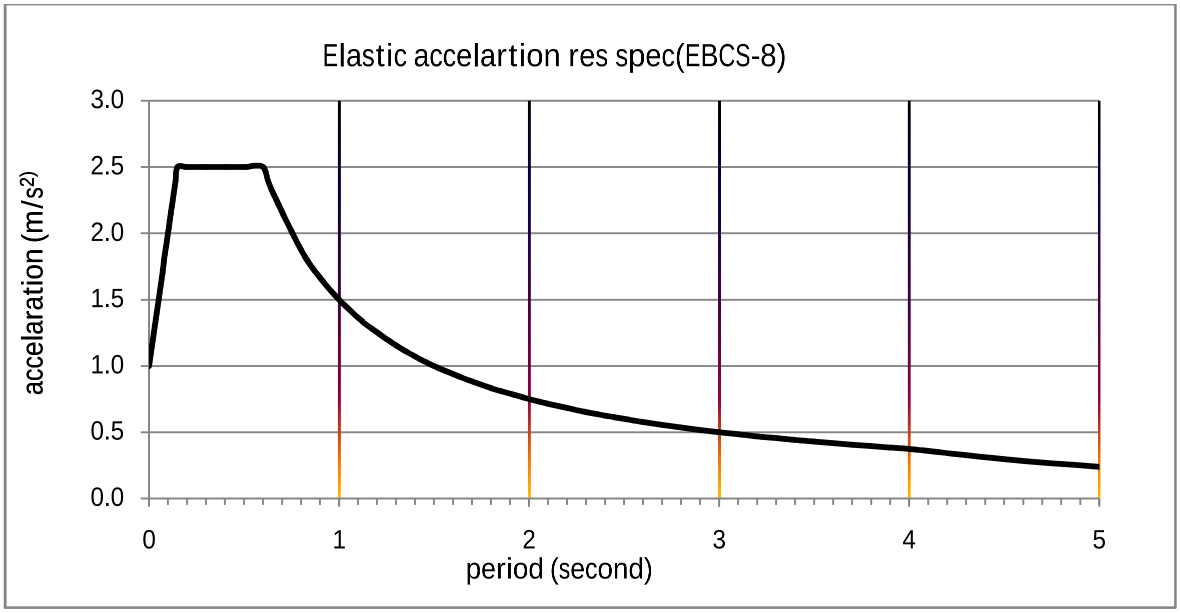


Fig 2.4 elastic acceleration response spectrum(soil-B)

The corresponding elastic displacement spectra for 5% damping from the show elastic acceleration response spectrum can be extracted as described previously.

For this thesis the value of corner period of T_C is taken as 4 second as recommended and the value of T_B is taken as 0.5. taking soil class B. So that the corresponding displacement spectral value at the corner period is given as [4] by:

$$\Delta c = Ca.T_B.Tc.g/4\pi^2$$

Where Ca is the multiplying coefficient of 2.5

$$= 2.5x0.5x4x9.81/4\pi^2$$

$$= 1.244m$$

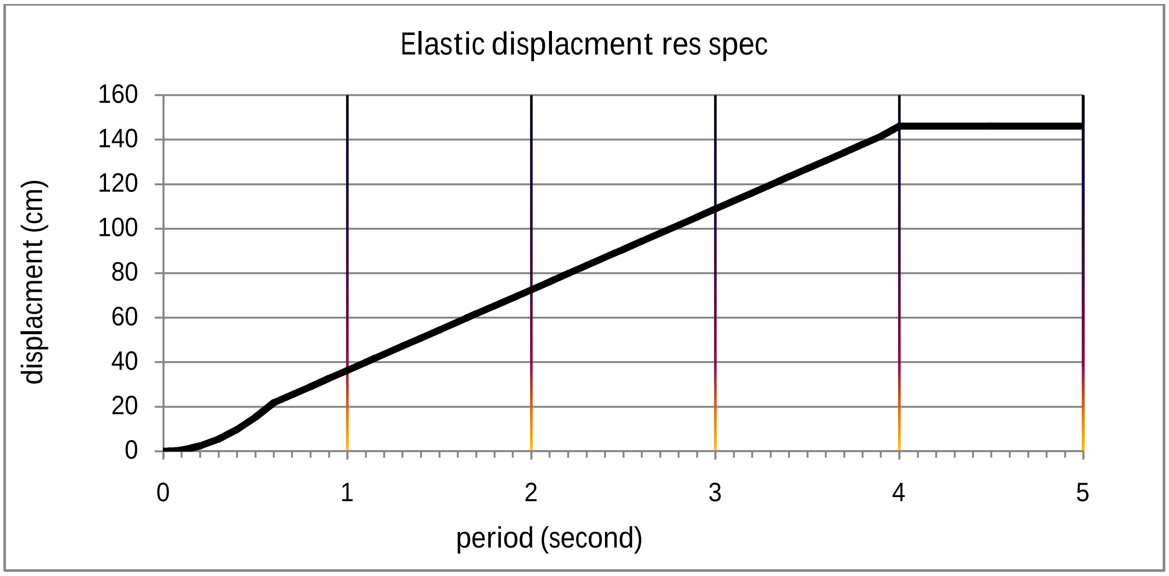


Fig 2.5 elastic displacement response spectrum for 5% damping

Using the procedure used to find the displacement response spectrum for different damping ratios, to modify the spectra for damping levels ξ_d other than 5%, the resulting displacement response spectrum is shown below. But it is still the response is related with the elastic period of the structure.

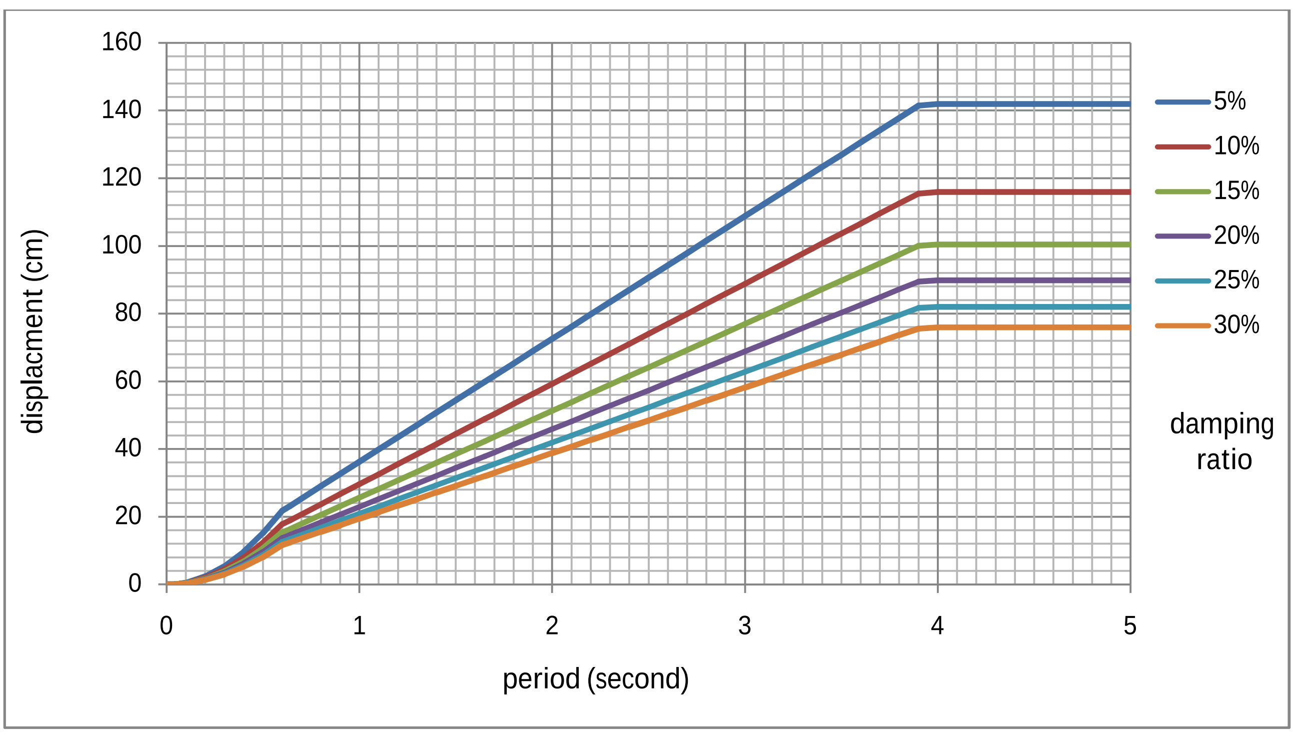


Fig 2.6 Elastic displacement response spectrum for different damping ratios

CHAPTER THREE

3. Fundamental Considerations in Direct Displacement-Based Design

3.1 Introduction

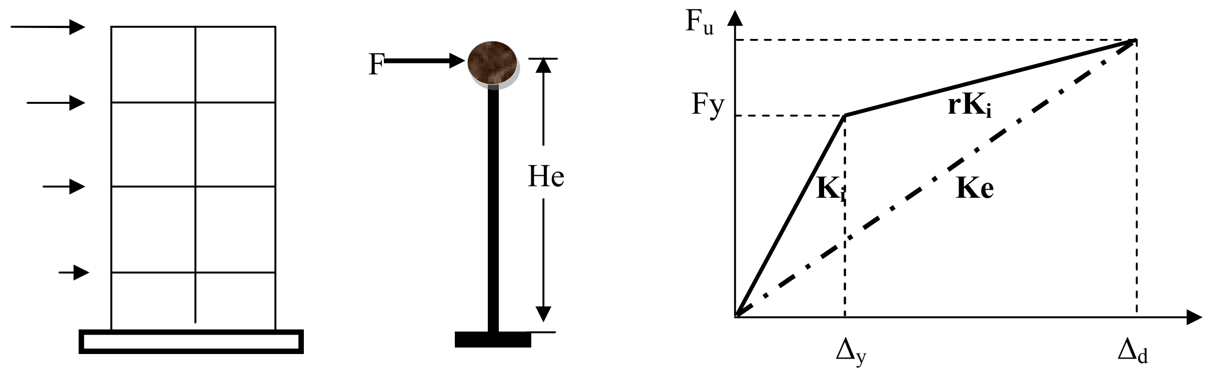
The design procedure known as direct displacement based design (DDBD) has been developed over the past ten years with the aim of mitigating the deficiencies in current force-based design, discussed previously. The fundamental difference from force-based design is that DDBD characterizes the structure to be designed by a single degree of freedom representation of performance at peak displacement response, rather than by its initial elastic characteristics. This is based on the substitute structure approach. The substitute structure approach is a procedure where an inelastic system is modeled as equivalent elastic system that has properties of equivalent stiffness K_{eq}; and equivalent damping ξ_{eq} to the inelastic system. [1, 8]

The fundamental philosophy behind the design approach is to design a structure which would achieve, rather than bounded by, a given performance limit state under seismic intensity. This would result in essentially uniform risk structures. The design procedures determine the strength required at designated plastic hinge locations to achieve the design aims in terms of defined displacement objectives. It must then be combined with capacity design procedures to ensure that plastic hinges occur only where intended.

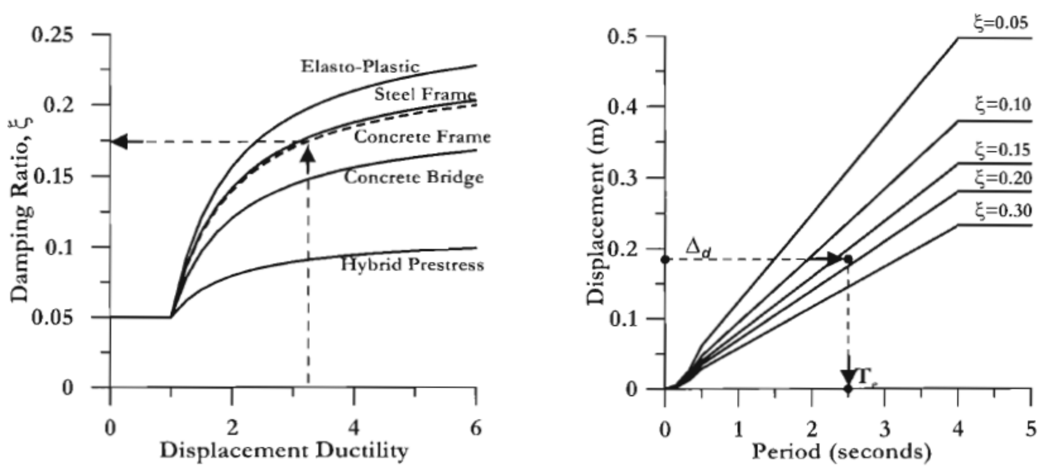
3.2 Basic formulation of the method

The design method is illustrated with reference to fig 3.1, which considers a SDOF representation of frame building (fig 3.1 (a)), though the basic fundamentals apply to all structural types. The bi-linear envelop of the lateral force-displacement response of the SDOF representation is shown in fig 3.1(b). An initial elastic stiffness $\boldsymbol{K_i}$ is followed by a post yield stiffness of $\boldsymbol{rK_i}$.

While force-based seismic design characterizes a structure in terms of elastic, pre-yield, properties (initial stiffness $\boldsymbol{K_i}$, elastic damping), DDBD characterizes the structure by secant stiffness $\boldsymbol{K_e}$ at maximum displacement Δ_d (fig.3.1(b)), and a level of equivalent viscous damping $\boldsymbol{\xi}$**,** representative of the combined elastic damping and hysteretic energy absorbed during inelastic response. Thus, as shown in fig 3.1(c), for a given level of ductile demand, a structural steel frame building with compact members will be assigned a higher level of equivalent viscous damping than a reinforced concrete bridge designed for the same level of ductility demand, as a consequence of fatter hysteresis loops.

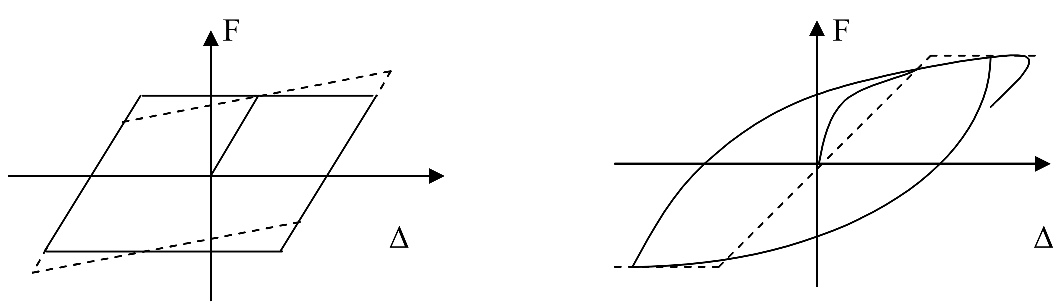


(a) SDOF simulation (b) Effective stiffness Ke



(c) Equivalent damping vs. ductility **(d) Design Displacement Spectra**

Fig. 3.1 Fundamentals of Direct Displacement- Based Design



(a) Idealized steel frame response (b) reinforced concrete frame response

Fig 3.2 Structural Force-Displacement Hysteresis Response [4]

With the design displacement at maximum response determined and the corresponding damping estimated from the exculpated ductility demand as will be discussed, the effective period $\boldsymbol{T_e}$ at maximum displacement response, measured at the effective height ***He*** can be read from a set of displacement spectrum for different levels of damping, as shown in example of fig 3.1(d). The effective stiffness ***Ke*** of the equivalent SDOF system at maximum displacement can be found by inverting the normal equation for period of SDOF oscillator, given by equation 3.1 to provide equation 3.2.

$$T = 2\pi \quad \tag{3.1}$$

$$K_e = \frac{\quad^{2}}{\quad^{2}} \tag{3.2}$$

Where $\boldsymbol{m_e}$ is the effective mass of the structure participating in the fundamental mode of vibration. The design lateral force, which is also the design base shear force, is thus.

$$F = \boldsymbol{V_{base}} = \boldsymbol{K_e \Delta_d} \tag{3.3}$$

The design concept is thus very simple. Such complexity that exists relates to determination of the "substitute structure" characteristics, the determination of the design displacement, and design displacement spectra. Careful consideration is however also necessary for the distribution of the design base shear force $\boldsymbol{V_{base}}$ to the different discredited mass locations, and for the analysis of the structure under the distributed seismic force.

The formulation of DDBD described above has the merit of characterizing the effects of ductility on seismic demand that is independent of the hysteretic characteristics, since the damping /ductility relationships are separately generated for different hysteretic rules. It is comparatively straightforward to generate the influence of different levels of damping on displacement response spectra.

It is also possible ,however, to combine the damping/ ductility relationship for a specific hysteresis rule with the seismic displacement spectral demand in a single inelastic displacement spectra set, where the different curves directly relate to displacement ductility demand as show in the figure below.

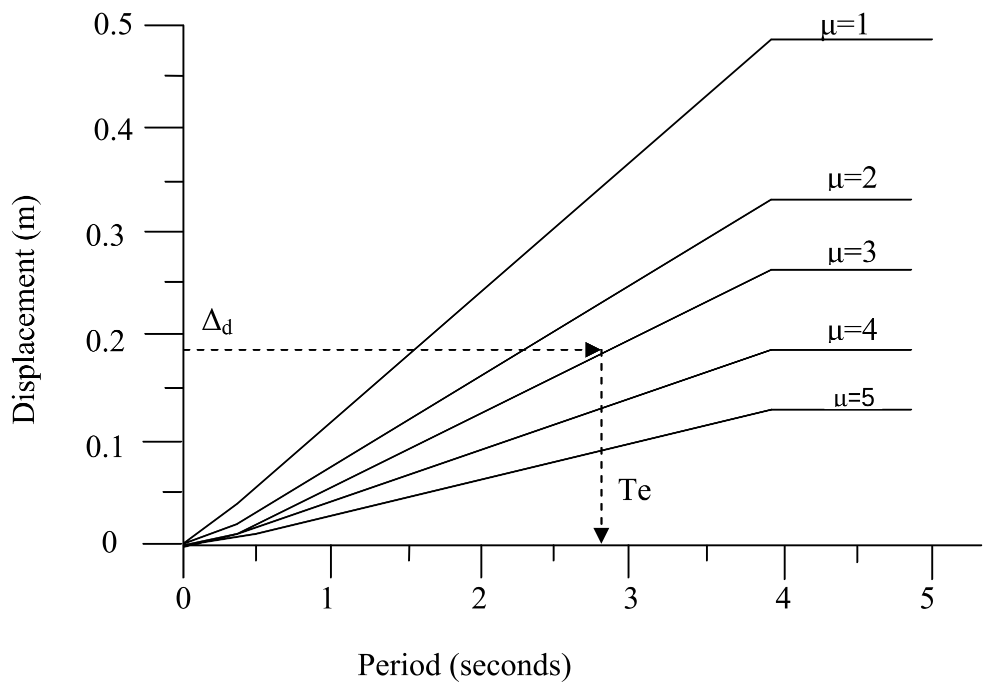


Fig 3.3 Inelastic displacement spectra set related to effective period.

With the seismic demand characterized in this fashion, the design procedure is slightly simplified, as one step in the process is removed. The inelastic displacement spectra set is entered with the design displacement and design effective period is read off for level of the design displacement ductility. Although this is slightly simplified procedure, it requires that inelastic displacement spectra be generated for different hysteresis rules for each new seismic intensity considered.

3.3 Design limit states and performance levels

In recent years there has been increased interest in defining seismic performance objectives for structures. This has been defined as the "coupling of expected performance levels with expected levels of seismic ground motion." Four performance levels of seismic excitation are designated as:

- Level 1: *fully operational*. Facility continues in operation with negligible damage.
- Level 2: *operational*. Facility continues in operation with minor damage and minor disruption in non-essential services.
- Level 3*: life safe*. Life safety is essentially protected, damage is moderate to extensive.
- Level 4: *near collapse*. Life safety is at risk, damage is sever, structural collapse is prevented

The relationship between these performance levels and earthquake design levels is summarized in fig.3.4. In the figure the line “basic objective” identifies a series of performance levels for normal structures. The line “essential objective” and “safety critical objective” relates performance level to seismic intensity for two structural classes of increased importance, such as lifeline structures and hospitals.

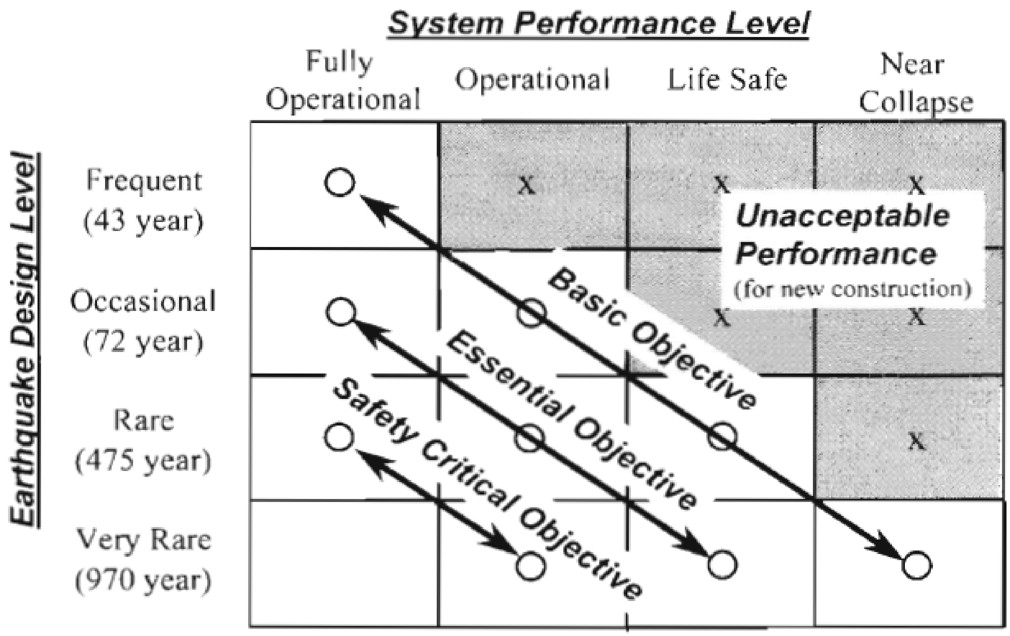


Fig. 3.4 Relationship between earthquake design level and performance level [3]

Although this approach is useful conceptually, it can be argued that it requires some modification, and that it provides an incomplete description of performance. The performance levels do not include a “damage control” performance level, which is clearly of economic importance. The performance level implicit in most current seismic design codes is, in fact, a damage- control performance level. In order to better understand the relationship between structural response levels and performance levels, it is instructive to consider the relationship between member and structural limit state.

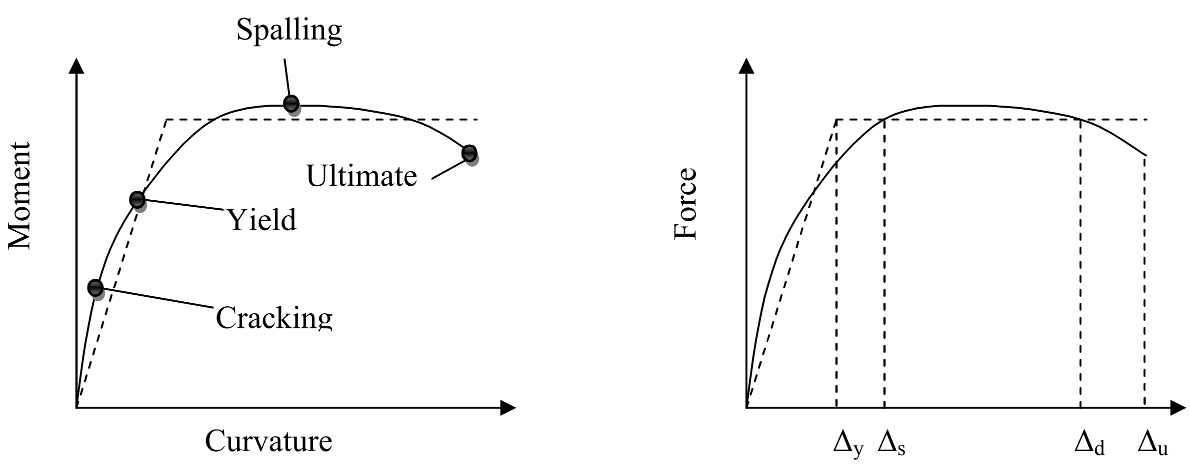
3.3.1 Section limit state

(a) ***Cracking limit state***: for concrete members the onset of cracking generally marks and point for a significant change in stiffness. The limit state is important for members that are expected to respond essentially elastically to the design-level earthquake.

(b)First-yield limit state: significant change in stiffness of concrete occurs at the onset of yield in extreme tension reinforcement.

(c) Spalling limit state: with concrete sections, the onset of spalling of the concrete cover may be significant limit state, particularly for unconfined sections or sections subjected to high level of axial compression

(d) Buckling limit state: with reinforced concrete members, an initiation of buckling of longitudinal reinforcement is significant limit state. Beyond this limit state remedial action will often require removal and replacement of the member.



(a) Section limit state (b) structural limit state

Fig 3.5 Member and structural design limits.

(e) Ultimate limit state: definition of ultimate limit state for members is somewhat subjective. It is sometimes taken to correspond to a critical physical event, such as fracture of confinement reinforcement in a potential plastic hinge zone of a concrete member. Another common definition relates to specified strength drop (20% is often used) from the maximum attained (or sometimes from design) strength.[4]

3.3.2. Structural limit state

(a) Serviceability limit state: this approach corresponds to the “fully functional” seismic performance level. No significant remedial action should be needed for a structure that corresponds at this limit state. With concrete structures no spalling of concrete should occur and though yield of reinforcement should be acceptable at this limit state. Potential for non-structural damage must be also considered when determining whether or not serviceability limit state has been exceeded. Ideally, non-structural elements, such as partition walls and glazing, should be designed so that no damage will occur to them before the structure archives the strain limits corresponding to the serviceability limit state.

(b) Damage-control limit state: at this limit state a certain amount of repairable damage is acceptable, but the cost should be significantly less than the cost of replacement. Damage to concrete buildings may include spalling of concrete cover and formation of wide residual flexural cracks. Fracture of transverse or longitudinal reinforcement, or buckling of longitudinal reinforcement should not occur, and the core concrete in plastic hinge region should not need replacement. Again non structural limits must be considered to keep damage to an acceptable level. It is difficult to avoid excessive damage when the drift level exceeds about 0.025, and hence it is common for building design codes to specify the drift to limits of 0.02 to 0.025.[4,14]

(c) Survival limit state: it is important that a reserve of capacity exists above that corresponding to the damage-control limit state, to ensure that during the strongest ground shaking considered feasible for the site, collapse of the structure should not take place. Protection against loss of life is the prime concern here, and must be accorded high priority in the overall seismic design philosophy. Extensive damage may have to be acceptable, to the extent that it may not economically or technically feasible to repair the structure after earthquake. This limit state is represented by ultimate displacement, $\Delta_{u.}$

3.4 Single Degree of Freedom

3.4.1 Design displacement for a SDOF structure

The design displacement will depend on the limit state being considered, and whether structural or non-structural considerations are more critical. For any given limit state structural performance will be governed by limiting material strains, since damage is strain-related for structural elements. Damage to non-structural elements can be generally considered drift-related.

It is possible to compute the design displacement from strain limits. Consider the vertical cantilever structure of fig 3.6(a), the most realistic structure conforming to the assumption of a SDOF approximation is a regular bridge under transverse excitation. Two possible reinforced sections, one circular and one rectangular are shown 3.6(b). The strain profile at maximum displacement response is shown together with the sections. Maximum compression strain ε_c and reinforcement tensile strain ε_s are developed. The limit-state strains are $\varepsilon_{c,ls}$ and $\varepsilon_{s,ls}$ for concrete compression and steel tension respectively. These will not generally occur simultaneously in the same section, since the neutral axis depth c is fixed by the reinforcement ratio, and the axial load on the section.[4]consequently there are two possible limit state curvatures, based on concrete compression and the reinforcement tension respectively ;

$$\text{Ø}_{ls,c} = \varepsilon_{c,ls}/c \qquad \text{(concrete compression)} \qquad (3.4a)$$

$$\text{Ø}_{ls,s} = \varepsilon_{s,ls}/(d-c) \qquad \text{(reinforcement tension)} \qquad (3.4b)$$

The lesser of $\text{Ø}_{ls,c}$ and $\text{Ø}_{ls,s}$, will govern the structural design. The design displacement can now be estimated as in [4] from

$$\Delta_{d,ls} = \Delta_y + \Delta_p = \text{Ø}_y(H+L_{sp})^2/3 + (\text{Ø}_{ls} - \text{Ø}_y)\, L_p H \qquad (3.5)$$

Where Ø_{ls} is the lesser of $\text{Ø}_{ls,c}$ and $\text{Ø}_{ls,s}$, Δ_y is the yield displacement. H is the column height, L_{sp} is the effective additional height representing strain penetration effects and L_p is the plastic hinge length.If the limit state has a code specified, non structural drift limit θ_c the displacement given by must be checked against.

$$\Delta_{d\theta} = \theta_c H \qquad (3.6)$$

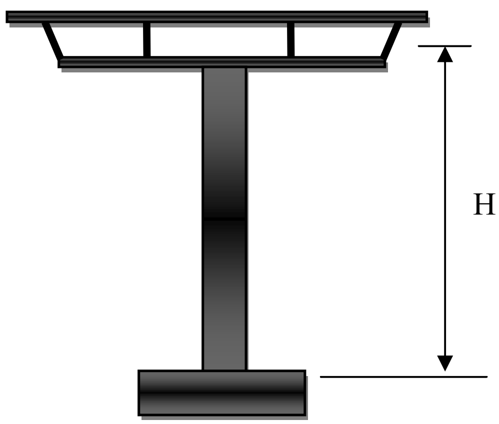
The lesser of the displacement given Eqs. (3.5) and (3.6) is the design displacement.

The yield displacement and the yield curvature which must be known. From analytical results the yield curvature is essentially independent of reinforcement content and axial load level and is a function of yield strain and section depth alone. The following equations for yield curvature of section shapes provide adequate approximation.

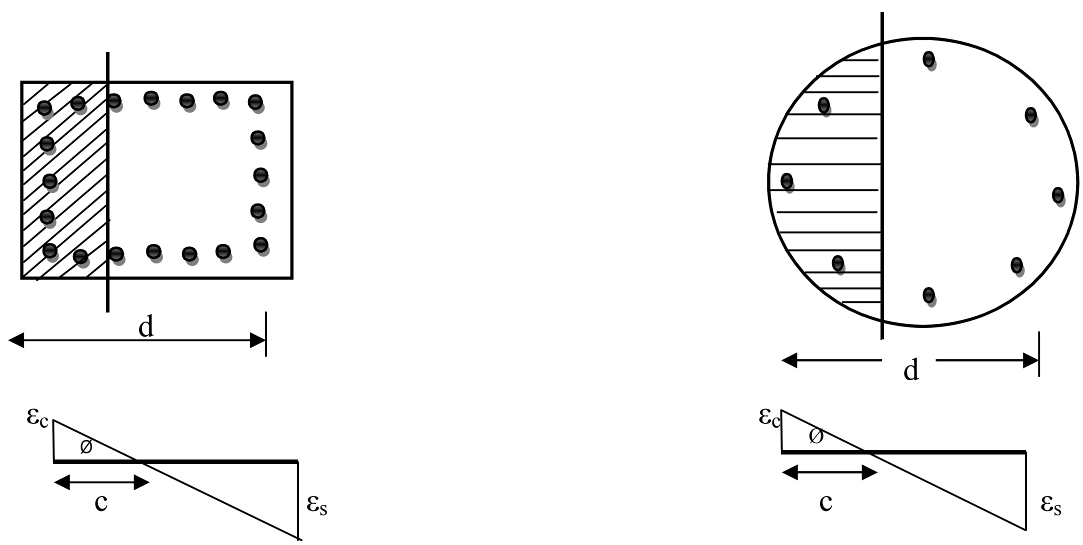
Circular concrete column $$\text{Ø}_y = 0.5\ \varepsilon_y L_b/h_b \qquad (3.7)$$

Rectangular concrete column. $$\text{Ø}_y = 0.65\ \varepsilon_y L_b/h_b \qquad (3.8)$$

Where L_b is the beam span, and h_b is the section depth.



(a) Cantilever bridge column



(b) Column sections and limit state strains

Fig. 3.6 Curvatures corresponding to limit strains for bridge pier.

3.4.2 Equivalent viscous damping

The design procedure requires relationships between displacement ductility and equivalent viscous damping. The damping is the sum of elastic and hysteretic damping:

$$\xi_{eq} = \xi_{el} + \xi_{hyst} \tag{3.9}$$

Where the hysteretic damping ξ_{hyst} depends on hysteretic rule appropriate for structure being designed. Normally the elastic damping ratio for concrete structures is taken as 0.05.

For concrete frame buildings considering its hysteretic rule, representing the response of ductile reinforced concrete frame buildings can be expressed as:

$$\xi_{eq} = 0.05 + 0.056(\frac{}{}) \tag{3.10}$$

It was mentioned that inelastic displacement spectra set could be developed for different levels of development ductility. One possible way is that to generate inelastic spectra set directly from the data used to generate the damping ductility relationships.

3.4.3 Design base shear equation

It will be clear that the approach described above can be simplified to single design equation, once the design displacement and damping have been determined. As it can be noted, the displacement spectra are in many cases linear with effective period. Let $\Delta_{c,5}$ be the displacement at the corner period T_c for the design displacement of Δ_d and design damping ξ, the effective period is,

$$T_e = T_c \frac{}{,} . (\frac{.}{.})^{\alpha} \tag{3.11}$$

Where $\alpha = 0.5$. The effective stiffness at peak response is thus.

$$Ke = \frac{2}{2} . \frac{2\ ,}{2} (\frac{.}{.})^{2\alpha} \tag{3.12}$$

$$V_{base} = K_e\Delta_d = \frac{2}{2} . \frac{2\ ,}{} (\frac{.}{.})^{2\alpha} \tag{3.13}$$

3.5 Multi- degree of freedom structures

For multi- degree-of-freedom (MDOF) structures the initial part of the design process requires the determination of the characteristics of the equivalent SDOF substitute structure. The required characteristics are the equivalent mass, the design displacement, and effective damping. When these have been determined, the design base shear for the substitute structure can be determined. The base shear is then distributed between the mass elements of the real structure as inertia forces, and the structure analyzed under these force determine the design moments at locations of potential plastic hinges.

3.5.1 Design Displacement

The characteristic design displacement of the substitute structure depends on the limit state displacement or drift of the most critical member of the real structure, and an assumed displacement shape of the structure. This displacement shape is that which corresponds to the inelastic first mode at the design level of seismic excitation. Thus the changes to the elastic first mode shape resulting from local changes to member stiffness caused by inelastic action in plastic hinges taken in to account at the beginning of the design. Representing the displacement by the inelastic rather than the elastic first-mode shape is consistent with characterizing the structure by secant stiffness to maximum response. The design displacement is thus given by:

$$\Delta_d = \sum \quad (\qquad) / \sum \quad (\qquad) \qquad (3.14)$$

Where m_i and Δ_i are the masses and displacements of the n significant mass locations respectively. For multi-story buildings, these will normally be at the n floors of the building.

With a knowledge of displacement of the critical element and the design displacement shape, displacements of the individual masses are given by:

$$\Delta_i = \delta_i \; — \qquad (3.15)$$

Where δ_i is the inelastic mode shape and Δ_c is the design displacement at the critical mass,c, and δ_c is the value of the mode shape at mass c. The displacement shape for frame building is adequately approximated for design purpose.

For $n \leq 4$: $\delta_i = H_i/H_n$ (3.16)

For n>4: $\delta_i = -.\ —\quad 1 - —$ (3.17)

Where H_i and H_n are the heights of level i. and the roof (level n) respectively.

From consideration of mass participating in the first inelastic mode of vibration, the effective system mass for the substitute structure is:

$m_e = \sum \quad (\qquad)/$ (3.18)

The equivalent viscous damping of the system depends on the structural system and ductility demand. This requires determination of the displacement ductility demand of the substitute structure. For frames it is adequate to assume that the yield drift is constant with height and hence the yield displacement is

$\Delta_y = \theta_y.H_e$ (3.19)

Where He, the effective height is given by

$He = \sum \quad (\qquad)/\sum \quad (\qquad)$ (3.20)

And the design ductility factor, for use is then

$\mu = \Delta_d/\Delta_y$ (3.21)

3.5.2 Distribution of design base shear force

The principles outlined in the previous sections enable the design base shear to be established for a MDOF system. This base shear force must be distributed as design forces to the various discritized masses of the structure, in order that the design moments for potential plastic hinges can be established. Assuming essentially sinusoidal response at peak response, the base shear should be distributed in proportion to mass and displacement at the discredited mass locations. Thus the design force at mass is:

$F_i = V_{base}(m_i\Delta_i)/\sum$ (3.22)

Similarity with force based design for multi-story buildings will immediately be apparent. The difference is that the design inelastic displacement profile, rather than a height proportional displacement is used.

3.5.3 Analysis of structure under design forces.

Analysis of the structure under the lateral force vector to determine the design moments at potential hinge locations is analytically straightforward, but nevertheless needs some conceptual consideration. In order to be compatible with the substitute structure concept that forms the basis of DDBD, member stiffness should be representative of effective secant stiffness at design displacement. Weak beam/ strong-column frame designs, beam members will be subjected to inelastic actions, and the appropriate beam stiffness will be:

$$(EI)_{beam} = E_c I_{cr}/\mu_b \qquad (3.23)$$

Where $E_c I_{cr}$ is the cracked-section stiffness, μb is the excepected beam displacement ductility demand. Analyses have shown that the member forces are not particularly sensitive to the level of stiffness assumed, and thus it is acceptable to assume that μb= μs, the frame design ductility. Since the column will be protected against inelastic action by capacity design procedures, their stiffness should be taken $E_c I_c$ with no reduction for ductility [3, 11].

3.6. Frame Buildings

This part builds on the basic material provided previously relating to the fundamentals of direct displacement-based seismic design (DDBD), to provide complete procedure for seismic design of buildings whose primary lateral force resisting systems is composed of reinforced concrete frames. The relevant equations are first summarized.

3.6.1 Review of basic DDBD procedures for frame buildings.

The first stage of the design process is the representation of multi-degree-of-freedom (MDOF) structure by equivalent single-degree-of-freedom (SDOF) structure modeling the first inelastic mode of response. The following were developed previously and summarized hereunder.

(a) *Design story displacements:* the design floor displacement of the frame are related to a normalized inelastic mode shape $\boldsymbol{\delta_i}$, where i=1 to n are the storys, and to the displacement Δ_c of critical story by the relationship

$$\Delta_i = \boldsymbol{\delta_i}\left(\frac{\Delta}{\delta}\right) \qquad (3.25)$$

Where the normalized inelastic mode shape depends on the height, $\boldsymbol{H_i}$, and roof height $\boldsymbol{H_n}$ according to the following relationships:

For $n \leq 4$: $\quad \delta_i = —$ (3.26 a)

For $n > 4$: $\quad \delta_i = - (—)\ (1-—)$ (3.26 b)

(b) *Equivalent SDOF design displacement:* the equivalent design displacement is related to the story displacements by the relationship:

$\Delta_d = \sum \quad (\qquad) / \sum \quad (\qquad)$ (3.27)

Where mi is the mass at height Hi associated with displacement Δ_i.

(c) *Equivalent mass:* the equivalent SDOF mass $\boldsymbol{m_e}$ is given by:

$\boldsymbol{m_e} = \sum \quad (\qquad) /$ (3.28)

(d) *Effective height:* the effective height He of the SDOF given by:

$He = \sum \quad (\qquad) / \sum \quad (\qquad)$ (3.29)

(e) ***Design displacement ductility:*** the SDOF design displacement ductility factor is related to equivalent yield displacement Δ_y by:

$$\mu = \text{—} \qquad (3.30)$$

The yield drift of a story in frame depended on geometry, and was independent of strength. This can be expressed as:

$$\theta_y = 0.5\varepsilon_y.\text{—} \qquad (3.31)$$

Where L_b and h_b are the beam span between column centerlines, and overall beam depth respectively, and ε_y is the yield strength of the flexural reinforcement. It is generally sufficiently accurate to assume a linear yield displacement profile for the purpose of estimating ductility demand, and hence the yield displacement is given by:

$$\Delta_y = \theta_y.He \qquad (3.32)$$

(f) ***Equivalent viscous damping:*** The equivalent viscous damping of the substitute structure for frames can be related to the design displacement ductility demand for reinforced concrete frames as:

$$\xi_{eq} = 0.05 + 0.565.\ \text{—} \qquad (3.33)$$

(g) ***Effective period of substitute structure:*** The effective period at peak displacement response is found from displacement spectra, entering with design displacement and determining the period, ***Te***, corresponding to the calculated equivalent viscous damping.

(h) ***Effective stiffness of substitute structure:*** the effective stiffness at maximum displacement response of the substitute structure, F/Δ_d is given by:

$$K_e = \frac{\;^{2}}{\;^{2}} \qquad (3.34)$$

(i) ***Design base shear force:*** the design base shear force for MDOF structure is found from substitute structure; since the SDOF substitute structure is elastic, it can be calculated as:

$$F = V_{base} = K_e\,\Delta_d \qquad (3.35)$$

The base shear from Eq.(3.35) is distributed to floor levels in proportion to the product of mass and displacement as:

$$F_i = V_{base}(mi\Delta i)/\sum(\quad) \qquad (3.36)$$

The building is then analyzed under the force vector represented by Eq(3.36) to determine the required flexural strength at plastic hinge locations.

Analysis have indicated that control of higher mode drifts is critically affected by the design displacement profile (and hence the design drift associated with inelastic first mode), and the vertical distribution of the base shear force resulting from DDBD process. Since these are inter-related, they are considered together. A modified form of Eq.(3.36) was used to distribute the base shear, where 10% of the base shear force was allocated to the roof level, and the remaining 90% was distributed in accordance with Eq.(3.36). The revised equation is thus:

$$F_i=F_t+0.9V_b\ (m_i\Delta_i)/\sum \tag{3.37}$$

Where F_t =0.1V_b at roof level, and F_t = 0 at all other levels. This is similar to the approach adopted in several seismic design codes.

On the basis of analysis results it is recommended that drift reduction factor be in the design for taller buildings. On the basis of analysis reported in *dynamic behavior of reinforced concrete frame buildings by petting and pristely* as quoted in [4], it is recommended that the buildings be designed to peak displacement of :

$$\Delta_{i,\omega}=\omega_\theta\Delta_i \tag{3.38a}$$

Where:

$$\omega_\theta = 1.15- 0.0034\ H_n\leq 1(H_n \text{ in m}) \tag{3.38b}$$

Therefore, to allow for drift amplification resulting from higher mode response, it is recommended that the frame buildings be designed for the reduced displacement defined by Eq.(3.38) and that the vertical distribution of base shear force conform with Eq(3.37)

3.6.2 Structural analysis under lateral force vector

Conventional frame analysis method, considering relative stiffness of members, for determining the required moment capacities at potential hinge locations for frames designed by DDBD is adopted. To be consistent with the principle of DDBD the frame structure analyzed should represent the relative stiffness of members at the peak displacement. Thus beams, which are expected to sustain ductility demands, should have their stiffness reduced from the elastic cracked-section stiffness in proportion to the excepted member displacement ductility demand. For frame members of normal proportions, it will be adequate to reduce elastic stiffness of all beam members by the system displacement ductility level μ_Δ. However, as the member ductility reduces with height, this will overestimate the relative stiffness of beams at lower levels and underestimate relative stiffness at higher levels. An improved solution will result if the member ductilities at different levels are proportional to the drift demands. Thus the member ductility at the first floor beams will be taken as 1.33 μ_Δ and at the roof level, as 0.667 μ_Δ.

The design philosophy of weak beak/strong column will require that the columns between the first floor and roof remain essentially elastic. Hence, the stiffness of these columns should be modeled by cracked-section stiffness, without any reduction for ductility. Since our design criterion is that column hinges do not form at the underside of first level beams, it would appear logical to design in such a way that the point of contra flexure in the column occurs approximately at 60% of the storey height. The design moment at the column base will thus be 0.6VcH1 where Vc is the column shear and H1 is the story height to the center of the first floor beam. Assuming that the column flexural strength is kept constant up the height of the first floor columns, this strength margin provides adequate protection against a soft-story mechanism forming under hire mode response as in [4].

The desired column-base moment capacities can then be defined before the structural analysis for required flexural strength of beam plastic hinges. The structural analysis is then proceeds with the first-story columns modeled as having cracked-section stiffness, and pinned-base conditions. The base moments are then applied as force to the column-base hinges, in addition to the applied lateral forces. The results of the analysis will then defined the required beam moments to satisfy the lateral force distribution, and preferred column-base moments. Typically, commercially available frame analysis computer program can be used for analysis. [3]

CHAPTER FOUR

4. Parametric Study

4.1. Description of the Parametric Study

In the previous chapters, it has been reviewed about the theoretical background information and the fundamental principles and systematically presented for the purpose of using it in this work and also for future reference about direct displacement based seismic design philosophy. In this chapter, parametric studies are carried out to demonstrate numerically the procedures as applied to reinforced concrete frame buildings and to investigate the effect of certain selected parameters how they influence for the variation of results as compared with that of traditional method of force based design philosophy and it also used for the observation of the trend in addition to their consideration in both methods.

Among the parameters, an effort is made to study the effect of the number of stories, the story height, the span length and depth of the beam which affect the response of the structure. With in each case the results are computed and compared with the force based method results especially that of base shear and distributed story forces are used for comparison in addition to checking the serviceability requirement for different cross sections by their drift values. For each parametric studies frame buildings of number of stories from four to ten stories are considered.

First, the procedure employed in direct displacement based design philosophy is demonstrated numerically in detail showing all the steps by taking four stories and two bays reinforced concrete frame. And the same frame is analyzed using the force based method for comparison purpose. This will be followed by the case studies considering different parameters listed previously.

For the force based design philosophy method equivalent statically method is adopted after implementing response spectrum method of analysis, which was found that in all cases it gives lesser value as compared with the equivalent static method. It such case it is recommended to scale up the results to that of equivalent static results. In addition all frames taken for this study are regular. Which is allowed to use equivalent statically method of analysis for such regular frames in design codes.

4.2. Numerical Demonstration Example

For demonstration purpose, at it is described in the previous section, the frame shown below is taken and analyised it using both methods showing all the steps employed step by step.

Frame geometry and properties of materials assumed for this taken frame:

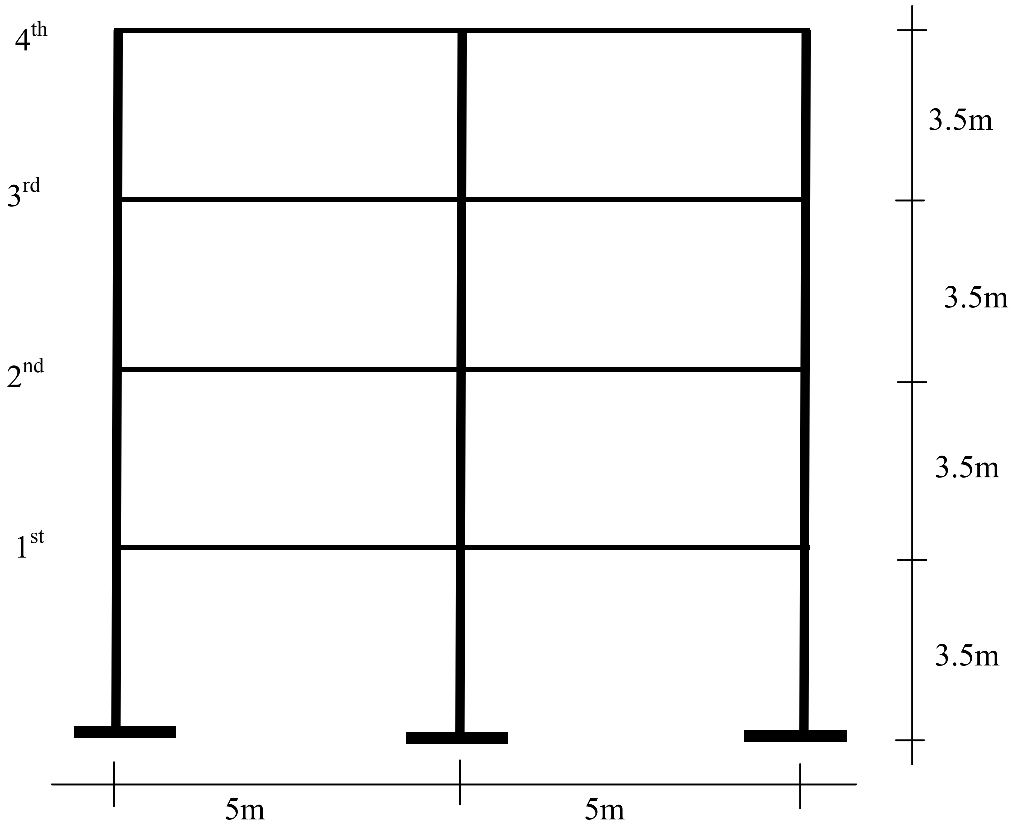


Fig 4.1 reinforced concrete frame

Properties of materials:

Concrete grade C-25

Steel grade S-400

Mass at each story level 30 ton

Founded on Soil class B

Step by step analysis procedure of the frame using DDBD method :

To determine the system ductility, and hence equivalent viscous damping, it is first necessary to determine the substitute structure displacement and effective height.

a) Design story displacement :

$\Delta_i = \delta i(—)$

For n=4, the normalized inelastic mode shape δi, is given by:

$\delta i = —$

⇨ $\delta 1 = — = — = 0.25$

$\delta 2 = — = — = 0.5$

$\delta 3 = — = — = 0.75$

$\delta 4 = — = — = 1$

⇨ $\Delta c = \Delta 1 = 0.02x3.5$ (taking inter-story drift limit of 0.02)

$= 0.07m$

⇨ $\Delta i = \delta i(—)$

$= \delta i(—)$

$= \delta i(0.28)$

Higher mode effects consideration on design story displacements,

$\omega_\theta = 1.15 - (0.0034xHn) \leq 1$

$= 1.15 - (0.0034x14) \leq 1$

$= 1.1024 \leq 1$

Take $\omega_{\theta =} 1$

⇨ $\Delta 1 = 0.28x0.25x1 = 0.07$

$\Delta 2 = 0.28x0.5x1 = 0.14$

$\Delta 3 = 0.28x0.75x1 = 0.21$

$\Delta 4 = 0.28x1x1 = 0.28$

b) Equivalent SDOF design displacement

$$\Delta_d = \frac{\Sigma \quad {}^2}{\Sigma}$$

$$= \frac{(\ .\ )\quad(\ .\ )\quad(\ .\ )\quad(\ .\ )^2}{.\quad .\quad .\quad .}$$

$= \boldsymbol{0.21m}$

c) Equivalent mass: Me

$$Me = \frac{\Sigma_{=1}^{4}}{}$$

$$= \frac{(\ .\quad .\quad .\quad .\ )}{.} \text{ x } 30kkg$$

$= \boldsymbol{100.0kkg}$

d) Effective height, He

$$He = \frac{\Sigma}{\Sigma}$$

$$= \frac{(\ .\quad .\ )\ (\ .\quad)\ (\ .\quad .\ )\ (\ .\quad)}{(\ .\quad .\quad .\quad .\ )}$$

$= \boldsymbol{10.5m}$

e) Design displacement ductility

$\mu = \frac{}{}$,

$\Delta y = \theta y x He$

Where $\theta y = 0.5\varepsilon y\ Lb/hb$

$= 0.5(\frac{.}{,})\frac{}{.}$, taking 50cm beam depth.

$= 0.011$

⇨ $\Delta y = 0.011 \text{ x } 10.5$

$= 0.1155$

⇨ $\mu = \frac{.}{.}$

$= \boldsymbol{1.818}$

f) Equivalent viscous damping ,ξ_{eq}

$\xi_{eq} = 0.05 + 0.565(\frac{}{})$

$= 0.05 + 0.565(\frac{.}{.})$

= 0.131

= ***13.10%***

g) Effective period of substitute structure

For: Δd = 0.21m (lesser of 0.21 and 0.015 x 14= 0.21m)

ξ_{eq} = 13.10%

$Te = T_c \frac{}{,} .(\frac{.}{.})^{0.5}$

For soil class B and 0.3g peak ground acceleration,

$\Delta_{c,5}$=1.244 x1.4 x 0.3= 0.5225m

$Te = 4x \frac{.}{.} x(\frac{. \quad .}{.})^{0.5}$

= ***2.36sec***

h) Effective stiffness of substitute structure

$Ke = \frac{^2}{^2}$

$= \frac{^2}{. \;\; ^2}$

= ***707.13kN/m***

i) Design base shear force ,

F= V_{base} = KeΔd

= 707.13kN/m x 0.21m

= ***148.47kN***

j) Distribution of the base shear to story levels:

$Fi = Ft + 0.9\ Vb \frac{}{\Sigma}$

$= Ft + 0.9 \times 148.47 \frac{}{\Sigma}$

⇨ F1= 0.9 x 148.47($\frac{.}{}$) = ***13.36kN***

F2 = 0.9 x 148.47($\frac{.}{}$) = ***26.73kN***

F3= 0.9 x 148.47 $\frac{.}{}$ =***40.09kN***

F4 = 0.1x (148.47) + 0.9 x 148.47($\frac{.}{}$) = ***68.31kN***

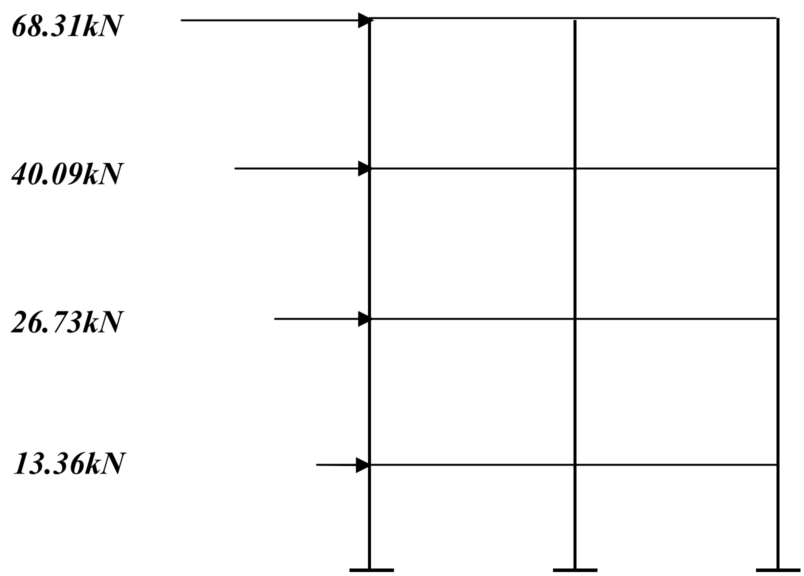


Fig 4.2. Distribution of the base shear to story levels

The same structural frame following the force based design method as per EBCS 8 recommendation is done as follows:

Using equivalent statics method:

Base shear F_b,

$$F_b = S_d\,(T_1)\,W$$

The fundamental period, T1 is given by:

$$T_1 = C_1 H^{3/4}$$

H =14m for this building

C_1= 0.075, for RC moment resisting frames

$$= 0.075(14)^{3/4}$$

$$= 0.543 \text{ sec}$$

Then, base shear force

$$F_b = S_d\,(T_1)\,W$$

Where Sd=design spectrum ordinate at period T1

$$S_d(T_1)=\alpha\beta\gamma$$

α is the ratio of design bed rock acceleration to the acceleration due to gravity and is given by

$\alpha_o I$, where α_o is the bedrock acceleration ratio for the site taken as 0.3and important factor of 1 is taken so it becomes 0.3.

β is the design response factor for the site and is given by:

$$\beta = 1.2S/T^{2/3}$$

Where S is site coefficient for soil characteristics= 1.2 for soil type B.

$$= 1.2x1.2/0.543^{2/3}$$

$$= 2.164$$

γ is the behavior factor to account for energy dissipation capacity and is given by:

$$\gamma = \gamma_o k_D k_R K_w$$

$$= 0.2x1.5x1x1 \;\; = 0.3 \leq 0.70 \text{ 0k!}$$

Then, $S_d(T_1)= \alpha\beta\gamma$

$$= 0.3x2.164\,x0.3$$

$$=0.1944$$

Then, the baseshear can be calculated using

$$Fb = S_d(T_1)xW$$

$= 0.1944 \times 120 \times 9.81$

$= \boldsymbol{229kN}$

Distribution of the base shear to different stories

$F_i = (F_b - F_t) \frac{}{\Sigma}$

$F_t = 0.07 T_1 F_b$

$= 0.07(0.543)(229)$

$= 8.70$ kN

⇨ $F_i = (229 - 8.7) \frac{}{\Sigma}$

$F_1 = 22.06$kN

$F_2 = 44.11$kN

$F_3 = 66.17$kN

$F_4 = 8.7 + 88.22 = 96.92$kN

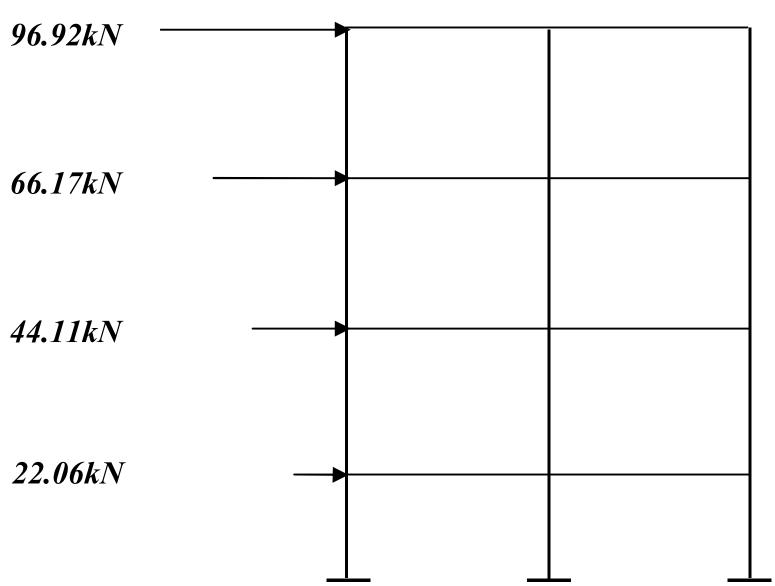


Fig 4.3. Distribution of the base shear to story level

4.3. Cases Considered In This Study

Once the procedure and the stepwise method of analysis is demonstrated as shown in the previous section, for the subsequent parametric case studies the same procedure is applied for about a total of 105 reinforced concrete frames for different cases and the resulting values are only presented here under which is followed by discussion at the end of each case.

Case-1:

The first case considers the variation of span length as a parameter for different stories. The span length of 4m, 5m, 6m and 7m is taken for seven different frames ranging from four to ten stories. In this case, the beam depth and story height is taken as constant which are 0.5m and 3.5 m respectively for all frames. The structure is also assumed to be founded on soil class B as per EBCS-7 soil classification. The result of both the direct displacement based approach and that of force based approach is computed and is shown as below:

Results for 4m span length

	4 STORIES			**5 STORIES**			**6 STORIES**			**7 STORIES**		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	131.3	229.3	74.6	185.2	256.3	38.4	187.4	280.8	49.9	188.8	303.3	60.6
2	119.5	207.2	73.4	172.3	240.0	39.3	178.0	268.1	50.6	181.7	293.1	61.3
3	95.8	163.1	70.2	147.8	207.4	40.3	160.0	242.8	51.7	168.0	272.7	62.3
4	60.4	96.9	60.5	113.1	158.4	40.1	134.3	204.7	52.4	148.2	242.1	63.3
5				69.5	93.1	33.9	101.6	154.0	51.5	122.9	201.2	63.8
6							62.8	90.6	44.1	92.5	150.2	62.4
7										57.7	89.0	54

	8 STORIES			**9 STORIES**			**10 STORIES**		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	189.9	324.2	70.8	197.4	343.9	74.2	240.9	362.5	50.5
2	184.3	315.8	71.4	192.8	336.8	74.7	236.2	356.4	50.9
3	173.5	298.9	72.3	183.7	322.6	75.6	227.2	344.2	51.5
4	157.8	273.7	73.4	170.6	301.3	76.6	213.9	326.0	52.4
5	137.6	239.9	74.4	153.5	272.8	77.7	196.7	301.6	53.3
6	113.3	197.8	74.6	132.9	237.3	78.6	175.8	271.1	54.2
7	85.1	147.2	72.9	108.9	194.6	78.7	151.4	234.6	54.9
8	53.6	88.2	64.5	81.9	144.9	76.9	123.8	191.9	55.0
9				52.1	88.0	68.9	93.3	143.2	53.5
10							59.9	88.3	47.4

Table 4.1 Story shear forces and their variation resulting from both methods for 4m span length

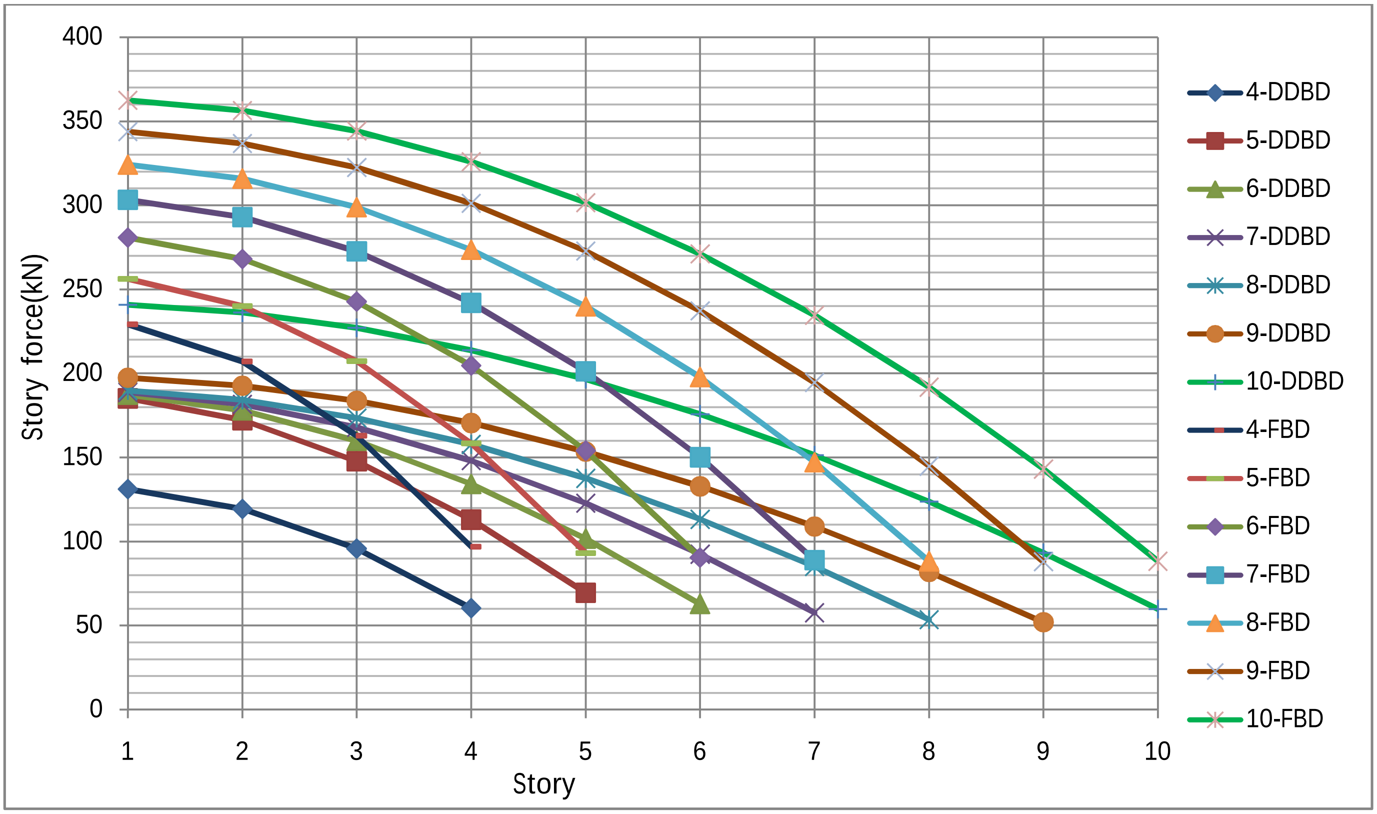


Fig 4.4. Story forces and their variation resulting from both methods for 4m span length.

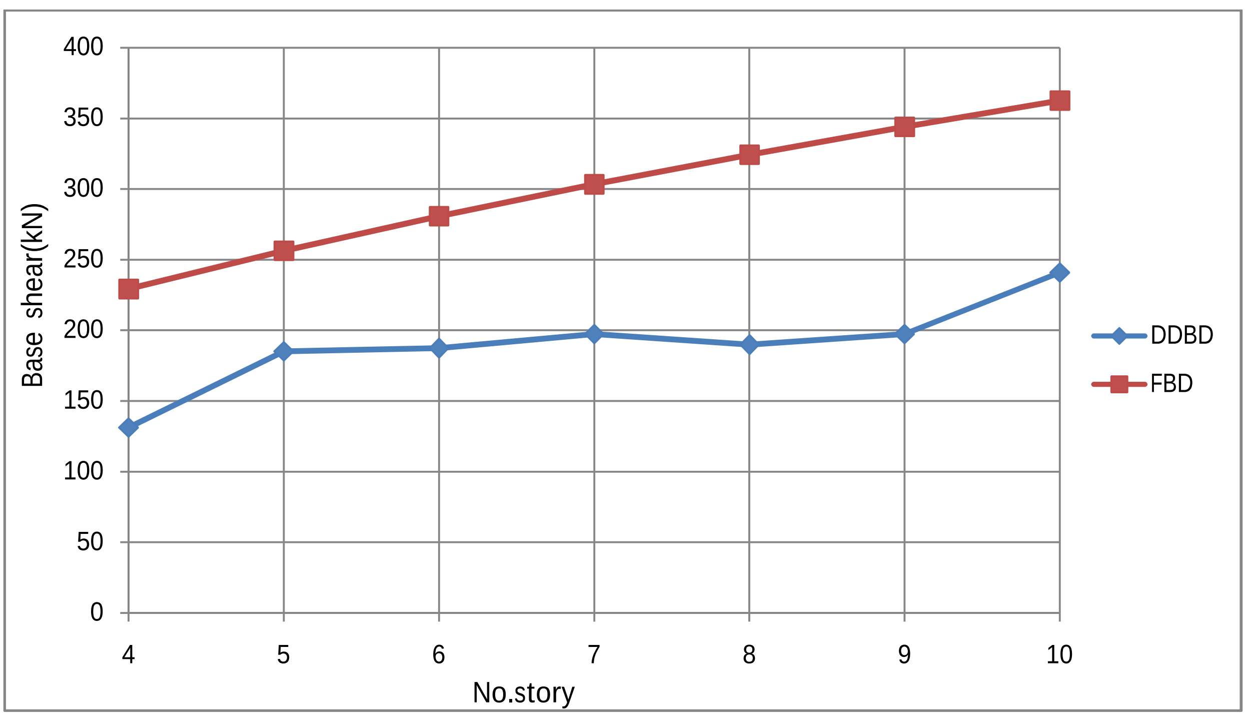


Fig 4.5. Base shear forces and their variation resulting from both methods for 4m span length.

Results for 5m span length

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	148.5	229.3	54.4	218.2	256.3	17.5	221.1	280.8	27.0	223.0	303.3	36.0
2	135.1	207.2	53.3	202.9	240.0	18.3	210.0	268.1	27.7	214.6	293.1	36.6
3	108.4	163.1	50.5	174.1	207.4	19.1	188.8	242.8	28.6	198.4	272.7	37.4
4	68.3	96.9	41.9	133.2	158.4	18.9	158.4	204.7	29.2	175.0	242.1	38.3
5				81.9	93.1	13.7	119.9	154.0	28.4	145.1	201.2	38.7
6							74.1	90.6	22.1	109.3	150.2	37.5
7										68.1	89.0	31

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	224.5	324.2	44.4	197.4	225.5	52.5	240.9	362.5	50.5
2	217.9	315.8	45.0	192.8	220.2	52.9	236.2	356.4	50.9
3	205.1	298.9	45.8	183.7	209.9	53.7	227.2	344.2	51.5
4	186.5	273.7	46.7	170.6	194.8	54.6	213.9	326.0	52.4
5	162.7	239.9	47.5	153.5	175.4	55.6	196.7	301.6	53.3
6	133.9	197.8	47.7	132.9	151.8	56.3	175.8	271.1	54.2
7	100.7	147.2	46.2	108.9	124.4	56.4	151.4	234.6	54.9
8	63.4	88.2	39.1	81.9	93.6	54.8	123.8	191.9	55.0
9				52.1	59.5	47.9	93.3	143.2	53.5
10							59.9	88.3	47.4

Table 4.2 Story shear forces and their variation resulting from both methods for 5m span length

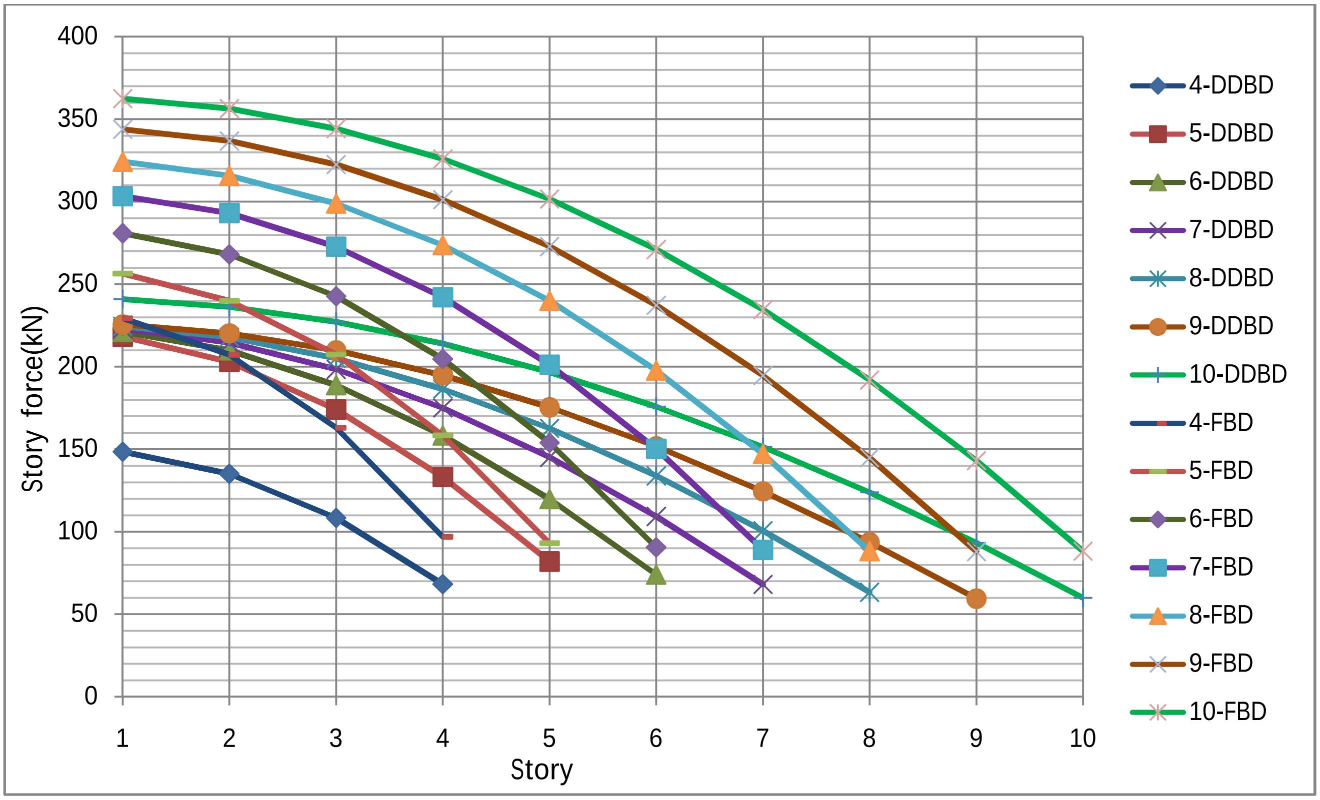


Fig 4.6. Story forces and their variation resulting from both methods for 5m span length.

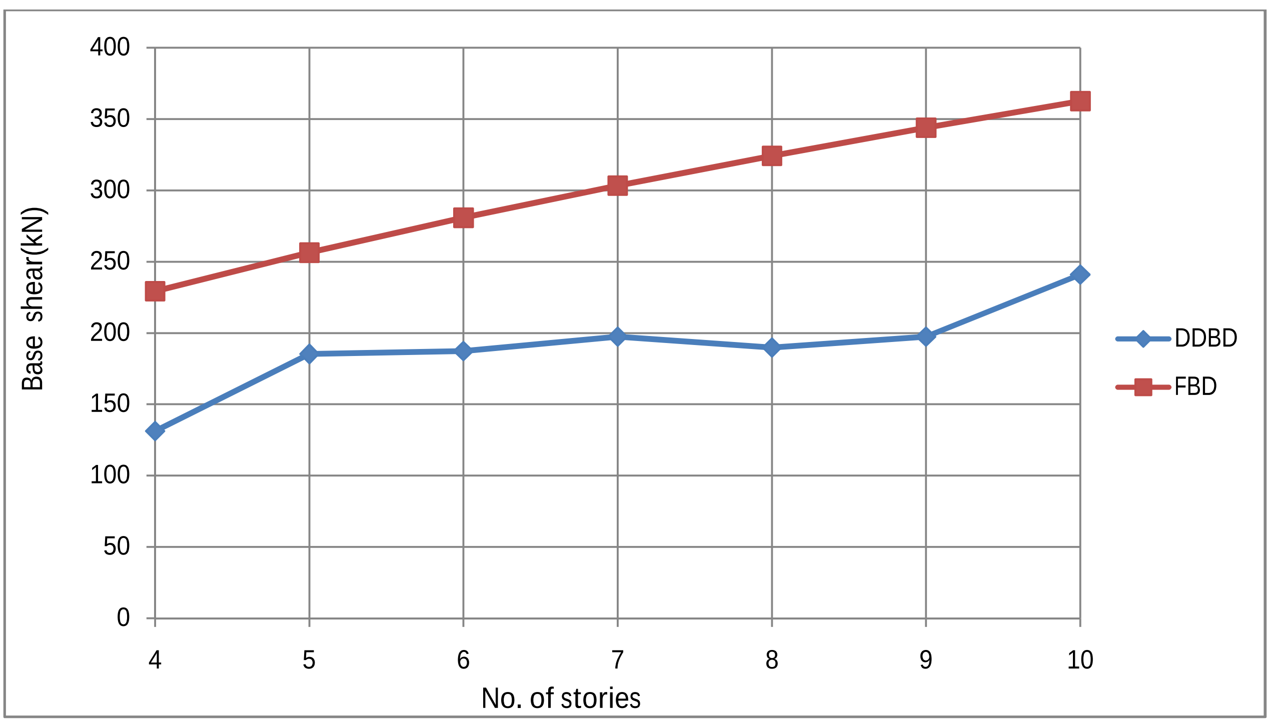


Fig 4.7. Base shear forces and their variation resulting from both methods for 5m span length.

Results for 6m span length

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDB**	**FDB**	**% DIF**	**DDB**	**FDB**	**% DIF**	**DDB**	**FDB**	**% DIF**
1	170.9	229.3	34.2	265.4	256.3	-3.4	269.6	280.8	4.2	272.4	303.3	11.3
2	155.5	207.2	33.2	246.9	240.0	-2.8	256.1	268.1	4.7	262.1	293.1	11.8
3	124.8	163.1	30.7	211.8	207.4	-2.1	230.2	242.8	5.5	242.3	272.7	12.5
4	78.6	96.9	23.3	162.1	158.4	-2.3	193.2	204.7	6.0	213.8	242.1	13.2
5				99.7	93.1	-6.6	146.2	154.0	5.3	177.2	201.2	13.5
6							90.4	90.6	0.2	133.5	150.2	12.6
7										83.2	89.0	7

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	274.5	324.2	18.1	276.0	343.9	24.6	277.3	362.5	30.7
2	266.4	315.8	18.5	269.5	336.8	25.0	271.9	356.4	31.1
3	250.8	298.9	19.2	256.9	322.6	25.6	261.5	344.2	31.7
4	228.1	273.7	20.0	238.5	301.3	26.3	246.2	326.0	32.4
5	198.9	239.9	20.6	214.6	272.8	27.1	226.4	301.6	33.2
6	163.7	197.8	20.8	185.8	237.3	27.7	202.3	271.1	34.0
7	123.1	147.2	19.6	152.3	194.6	27.8	174.3	234.6	34.6
8	77.5	88.2	13.8	114.5	144.9	26.5	142.5	191.9	34.6
9				72.8	88.0	20.8	107.3	143.2	33.4
10							69.0	88.3	28.0

Table 4.3 Story shear forces and their variation resulting from both methods for 6m span length

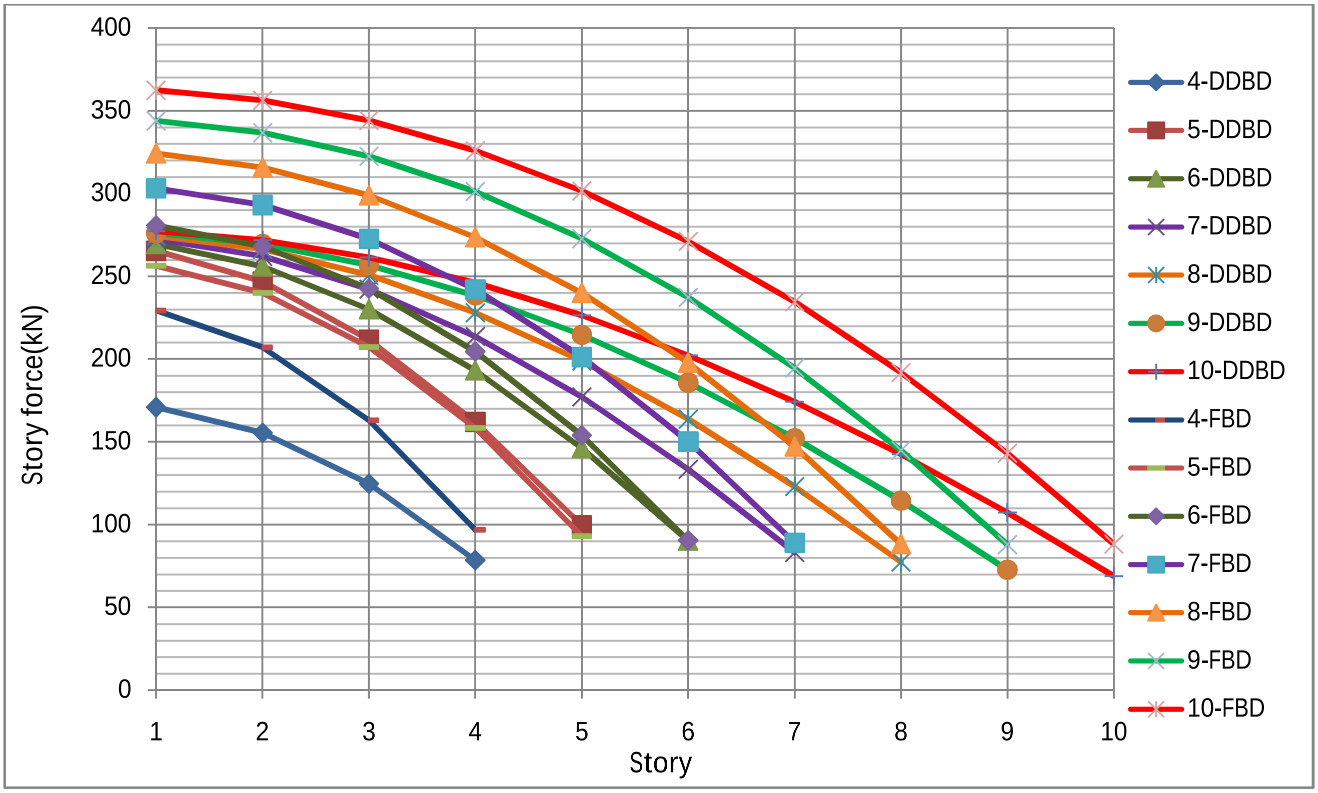


Fig 4.8. Story forces and their variation resulting from both methods for 6m span length.

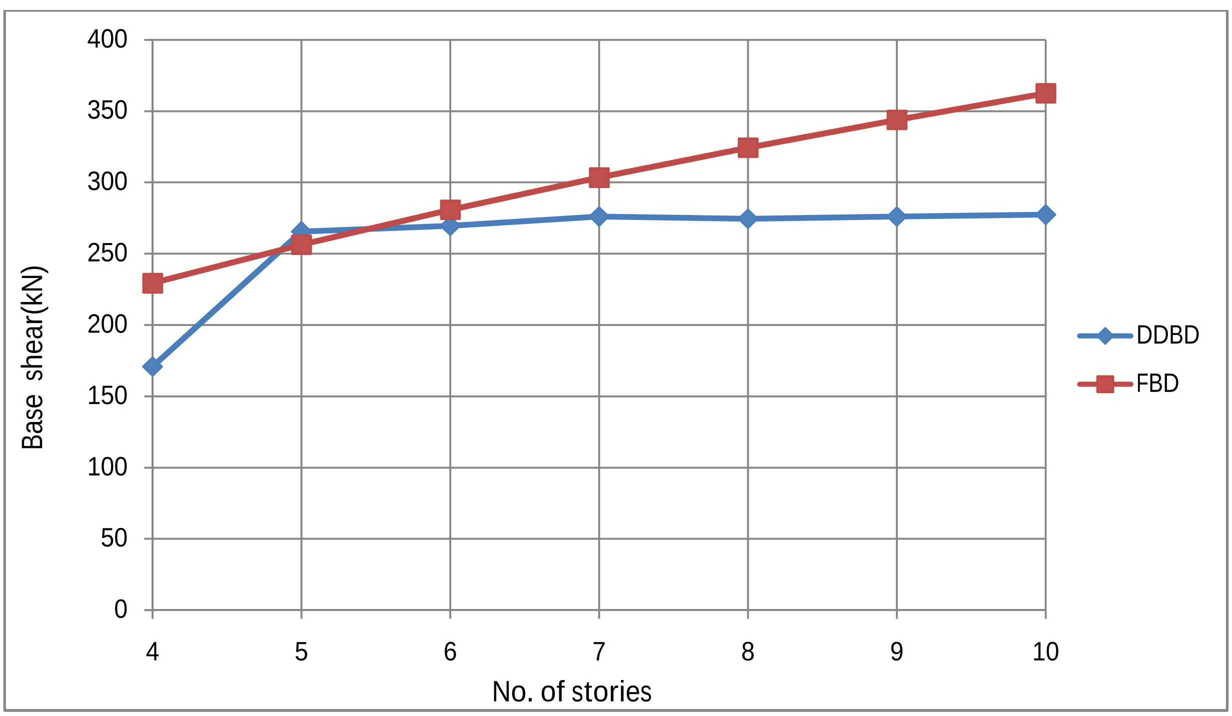


Fig 4.9. Base shear forces and their variation resulting from both methods for 6m span length.

Results for 7m span length

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	201.3	229.3	13.9	338.8	256.3	-24.3	345.3	280.8	-18.7	349.8	303.3	-13.3
2	183.2	207.2	13.1	315.1	240.0	-23.8	328.0	268.1	-18.3	336.6	293.1	-12.9
3	146.9	163.1	11.0	270.3	207.4	-23.3	294.9	242.8	-17.7	311.2	272.7	-12.4
4	92.6	96.9	4.7	206.9	158.4	-23.4	247.5	204.7	-17.3	274.5	242.1	-11.8
5				127.2	93.1	-26.8	187.3	154.0	-17.8	227.6	201.2	-11.6
6							115.8	90.6	-21.8	171.4	150.2	-12.4
7										106.8	89.0	-17

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	353.2	324.2	-8.2	355.7	343.9	-3.3	357.7	362.5	1.3
2	342.8	315.8	-7.9	347.3	336.8	-3.0	350.8	356.4	1.6
3	322.6	298.9	-7.3	331.0	322.6	-2.5	337.3	344.2	2.1
4	293.5	273.7	-6.8	307.2	301.3	-2.0	317.6	326.0	2.6
5	255.9	239.9	-6.2	276.6	272.8	-1.4	292.1	301.6	3.3
6	210.7	197.8	-6.1	239.4	237.3	-0.9	261.0	271.1	3.9
7	158.4	147.2	-7.1	196.2	194.6	-0.8	224.8	234.6	4.3
8	99.7	88.2	-11.6	147.6	144.9	-1.8	183.9	191.9	4.4
9				93.8	88.0	-6.2	138.5	143.2	3.4
10							89.0	88.3	-0.7

Table 4.4 Story shear forces and their variation resulting from both methods for 7m span length

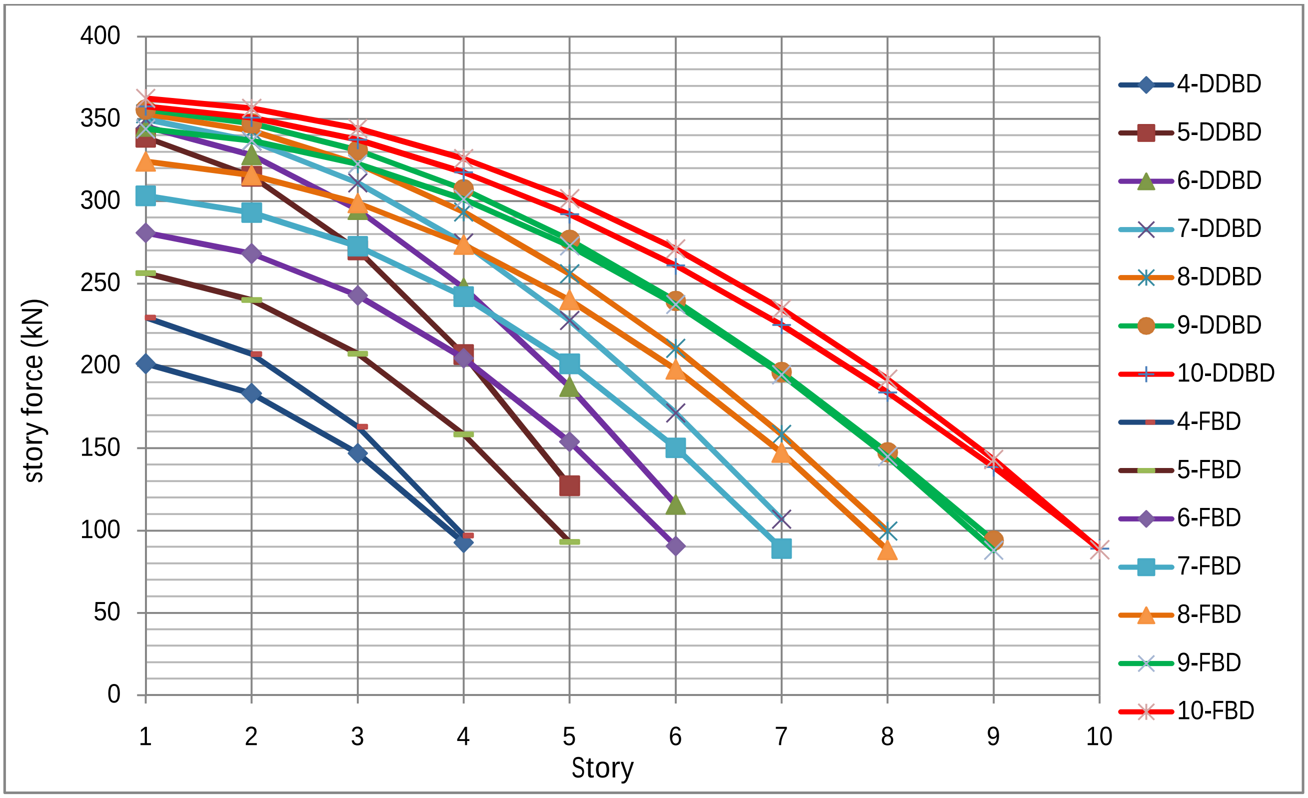


Fig 4.10. Story forces and their variation resulting from both methods for 7m span length.

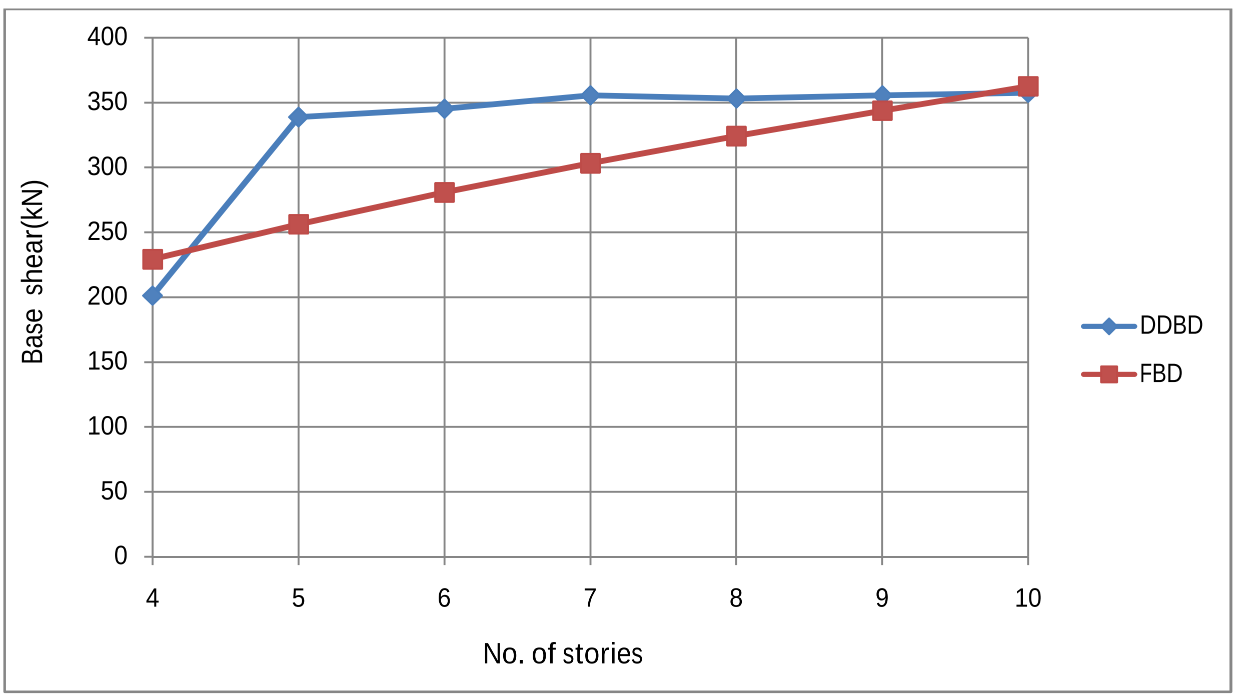


Fig 4.11. Base shear forces and their variation resulting from both methods for 7m span length

When the above results are summarized by comparing the values only in terms of base shear the following results will be the out puts

	4m span			**5m span**			**6m span**			**7m span**		
Story	**DDBD**	**FBD**	**%DIF**	**DDBD**	**FBD**	**%DIF**	**DDBD**	**FBD**	**%DIF**	**DDBD**	**FBD**	**%DIFF**
4	131.2	229.2	74.6	148.5	229.2	54.3	170.90	229.2	34.1	201.2	229.2	13.9
5	185.2	256.3	38.4	218.1	256.3	17.4	265.42	256.3	-3.4	338.7	256.3	-24.3
6	187.3	280.8	49.8	221.0	280.8	27.0	269.56	280.8	4.1	345.3	280.8	-18.6
7	197.4	303.3	53.6	225.5	303.3	34.4	276.05	303.3	9.8	355.6	303.3	-14.7
8	189.8	324.2	70.7	224.4	324.2	44.4	274.48	324.2	18.1	353.1	324.2	-8.1
9	197.4	343.9	74.1	225.5	343.9	52.4	276.05	343.9	24.5	355.6	343.9	-3.3
10	240.9	362.5	50.5	240.9	362.5	50.4	277.27	362.5	30.7	357.6	362.5	1.4
Aver.			59%			40%			17%			-8%

Table 4.5 Variation of base shear forces for different span lengths

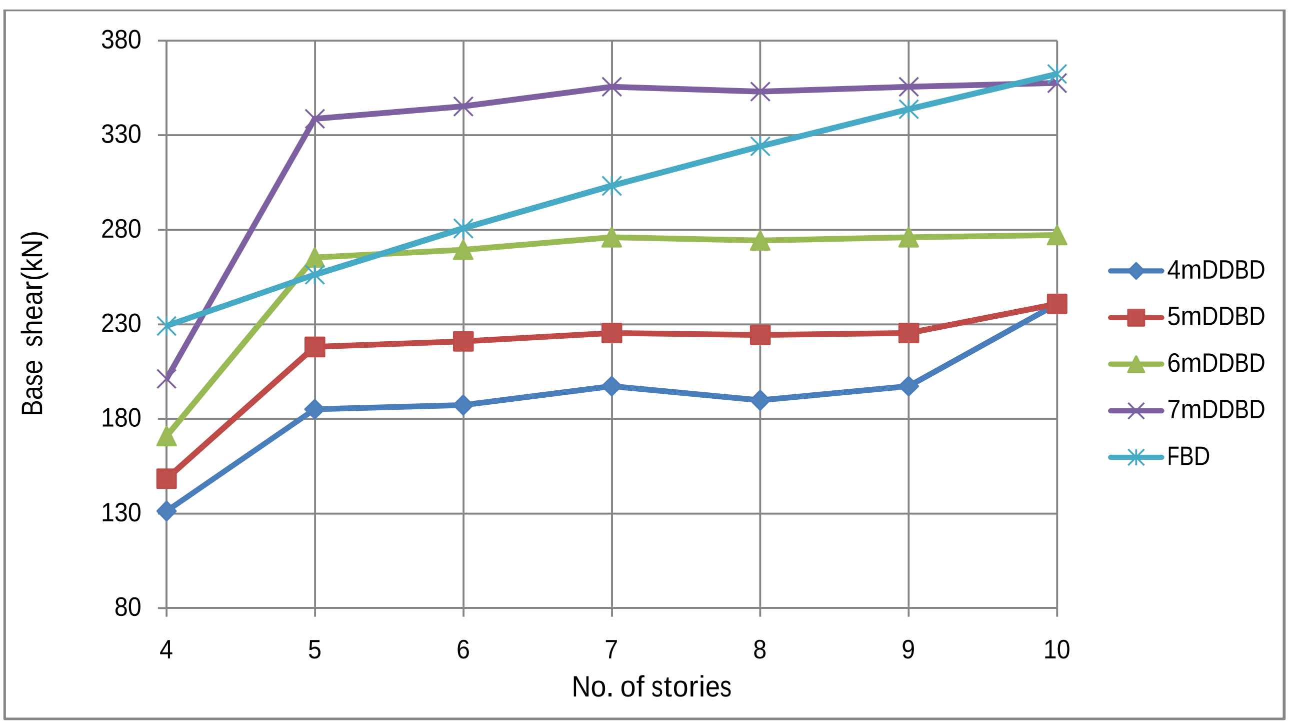


Fig 4.12 Variation of shear forces for different span lengths

From the results shown above both from the table and the figure for different span lengths, it is observed clearly that the base shear demand increases as the span of the beam increases for direct displacement based method of analysis. For DDBD resulting values variation as compared with the force-based approach increases in an average from 59% for 4m span frame having larger capacity of damping so that lesser strength demand to -8% for 7m span length, which have lower damping capacity. The trend can be seen easily from 4m to7mspan from the table shown above. This is because that the damping capacity decreases with larger span. This phenomenon cannot be observed in force based method of analysis. The value of shear force simply increases linearly with number of stories regardless of span length.

The result also shows us that the results of force based design approach are unnecessarily and more importantly unsafely conservative for those frames up to 6m spans except for five story frame. Whereas, the resulting values are not safe being less than the value required for spans of 7m. This is due to dramatical decrement of damping and a need of more strength to overcome such problem.

From this observation we can appreciate the rationality and the transparency of the direct displacement based design approach which considers the span length to damping effect and corresponding strength requirement in a convincing manner. Whereas the force based method do not account such condition directly.

Case-2:

The second case considers the variation of story height for different stories similarly as considered in the previous case. The span length of the beam is taken as constant of 5m for frames ranging from four to ten stories. The beam depth is also taken as constant of 0.5m. The structure is assumed to be founded on soil class B as per EBCS-7 soil classification. The result of both the direct displacement based approach and that of force based approach is calculated and documented as shown below. In order to avoid repetitions, graphical representations are presented here under. The tabular forms of the results are documented as an annex.

Results for Story height of 2.8m

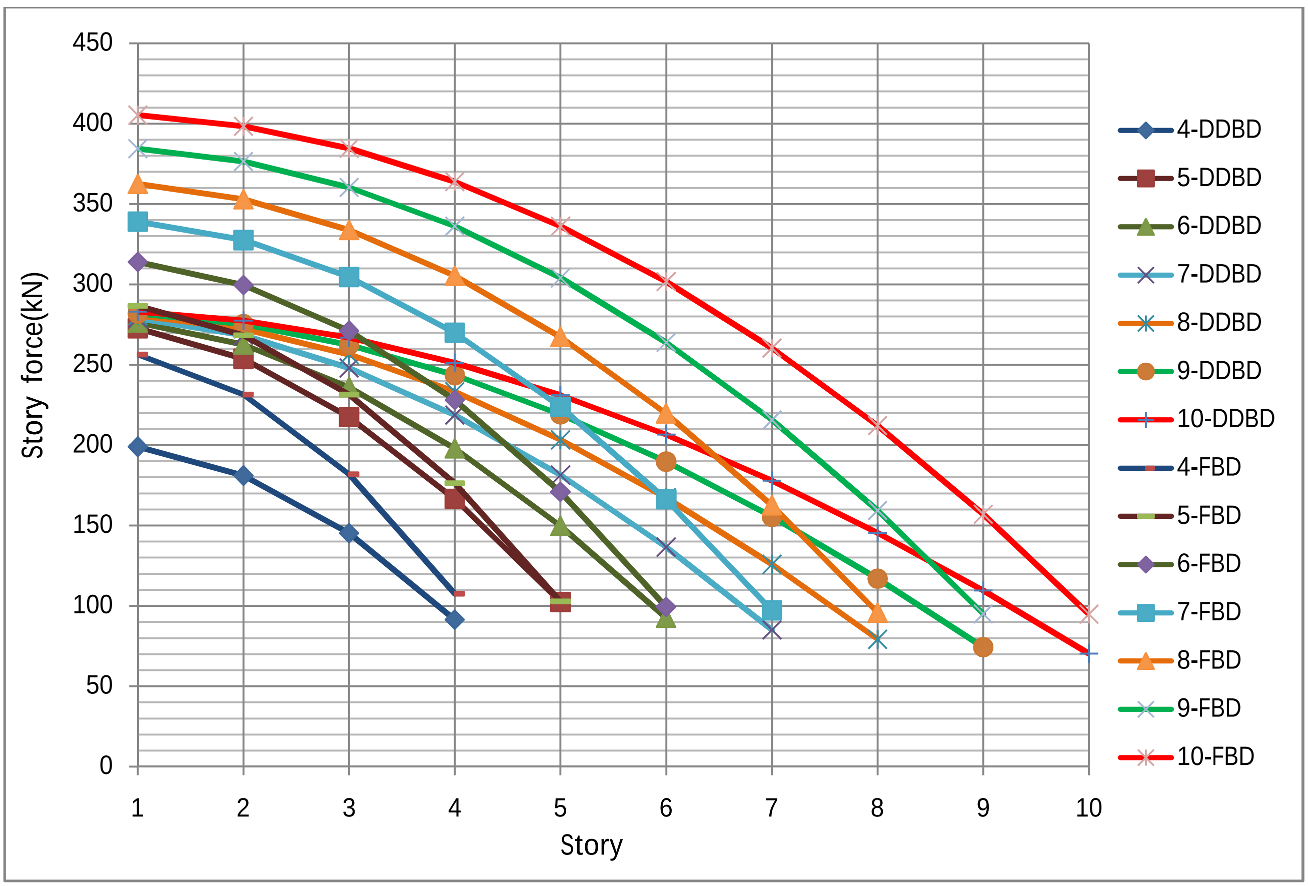


Fig 4.13. Story forces variation resulting from both methods for 2.8m story height.

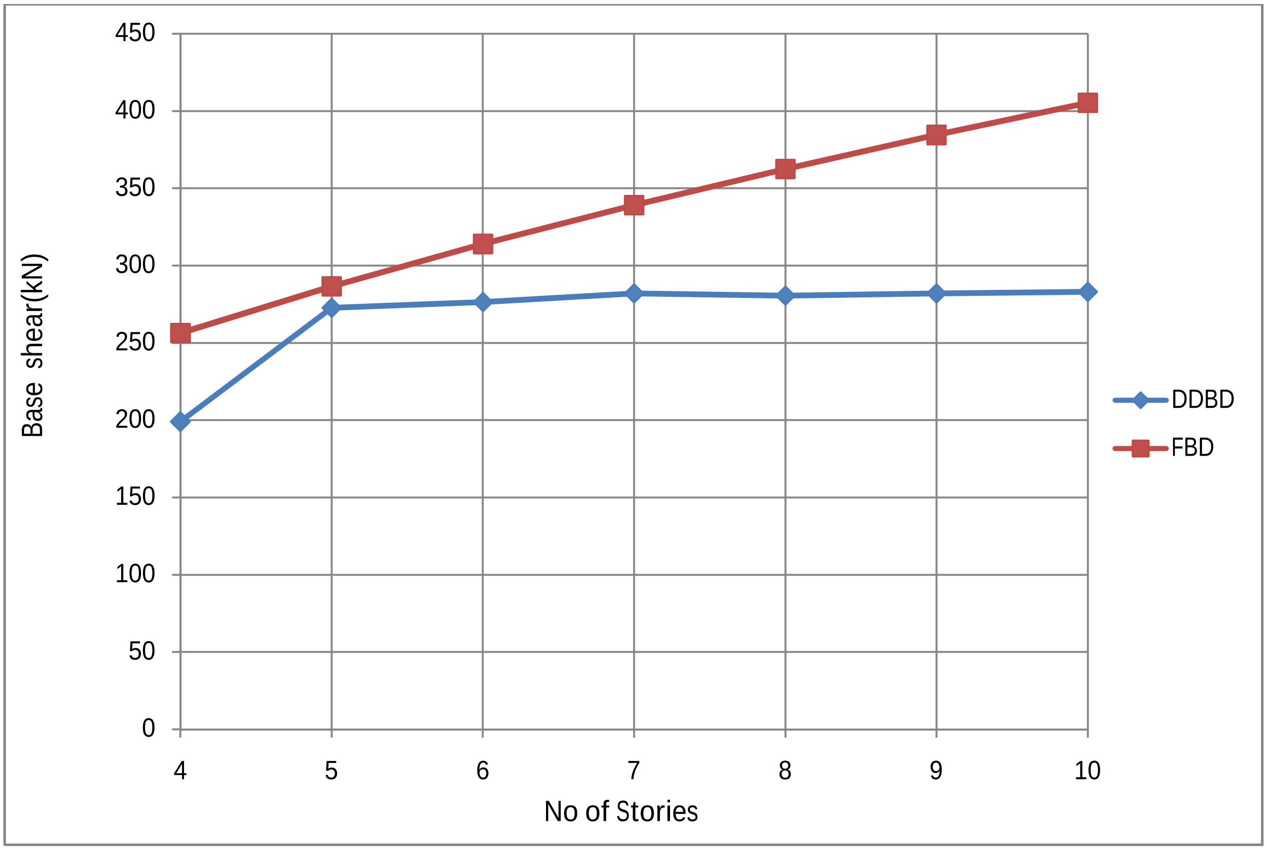


Fig 4.14. Base shear force variation resulting from both methods for 2.8m story height.

Results for Story height of 3m

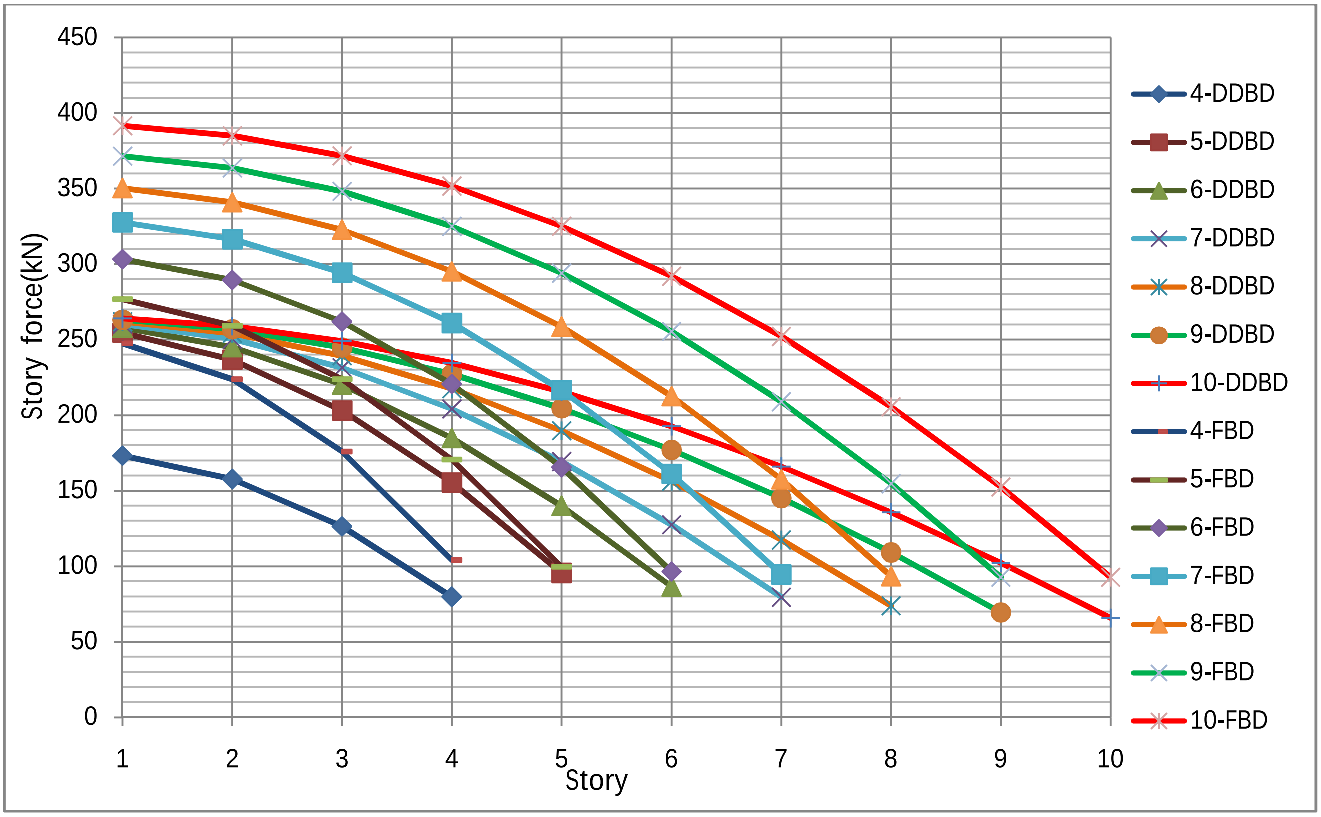


Fig 4.15. Story forces variation resulting from both methods for 3m story height.

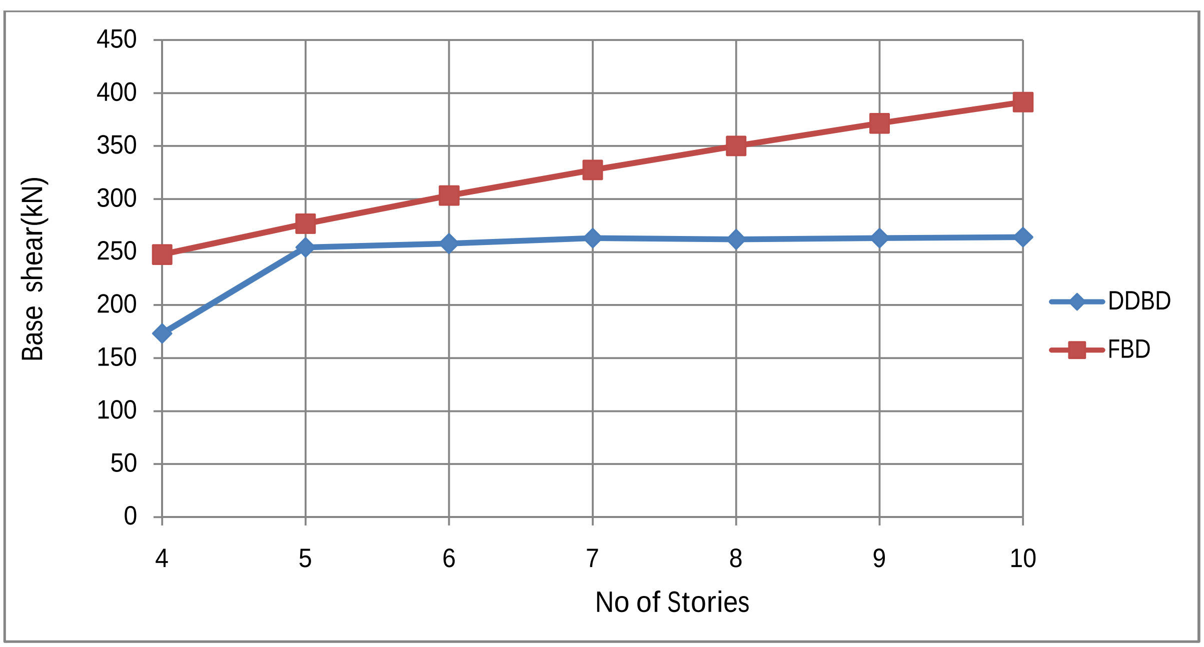


Fig 4.16. Base shear force variation resulting from both methods for 3m story height

Results for Story height of 3.2m

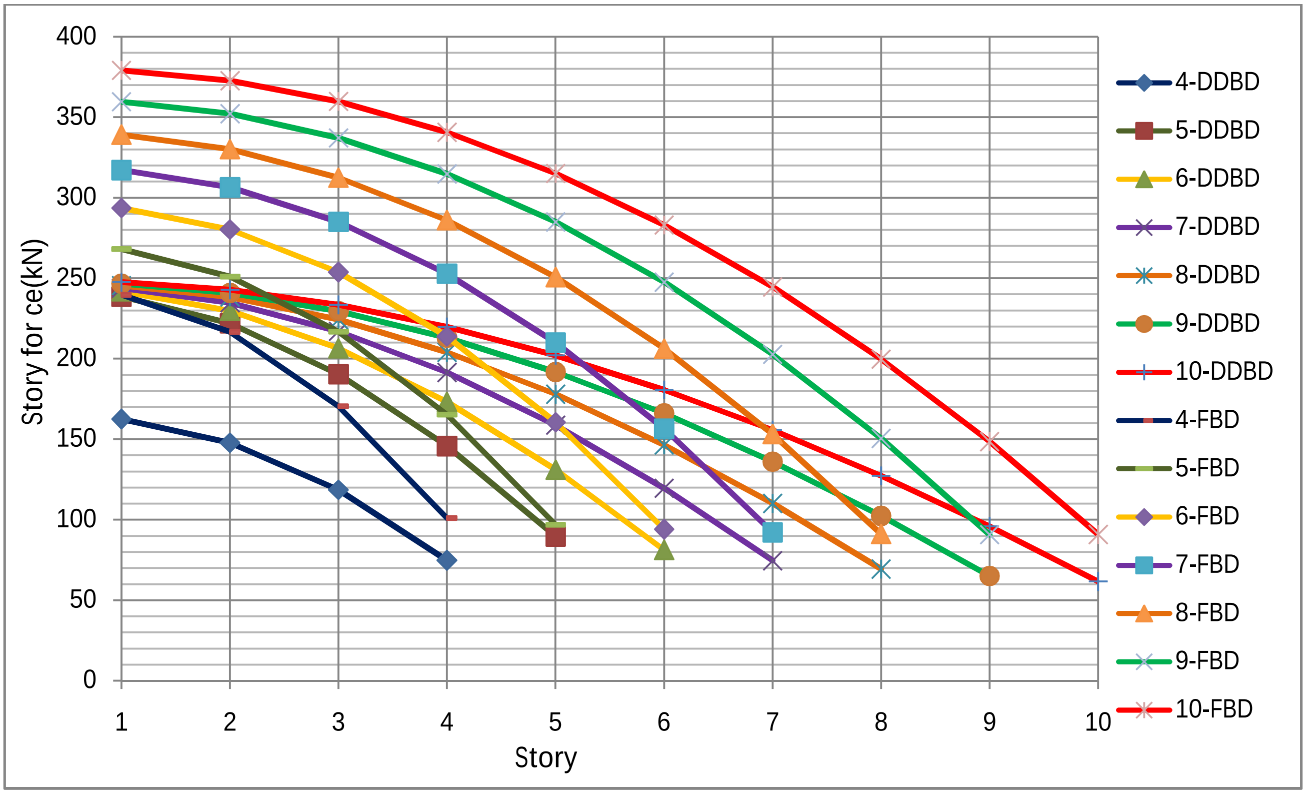


Fig 4.17. Story forces variation resulting from both methods for 3.2m story height.

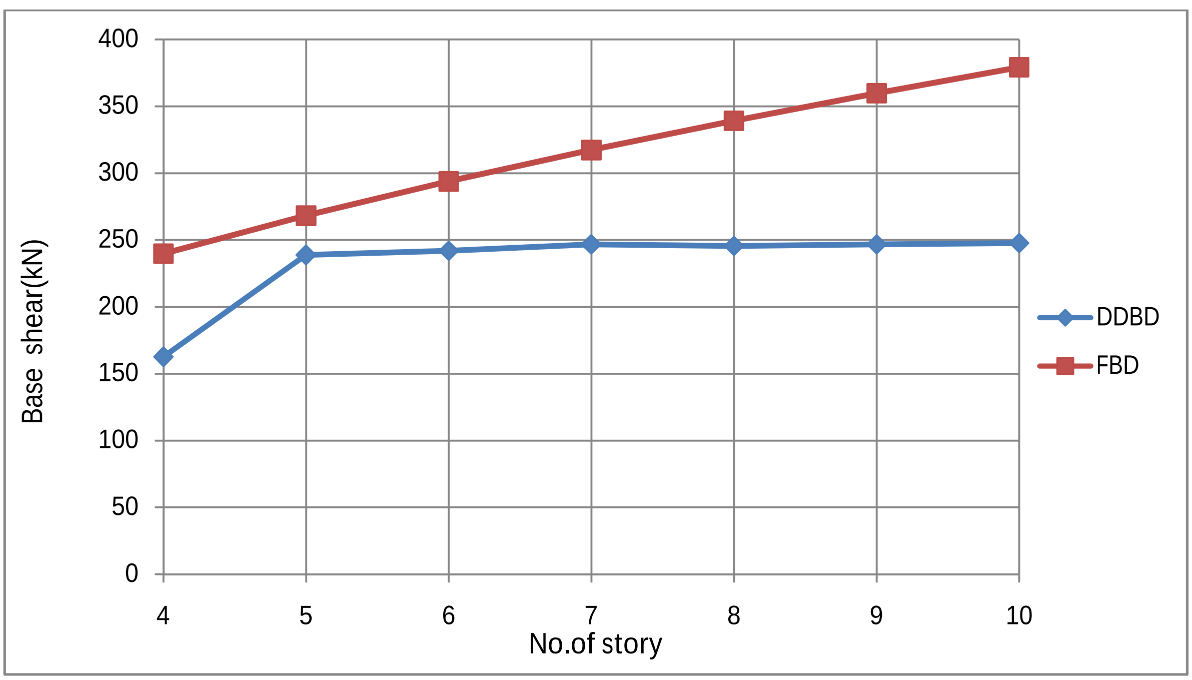


Fig 4.18. Base shear force variation resulting from both methods for 3.2m story height.

Results for Story height of 3.8m

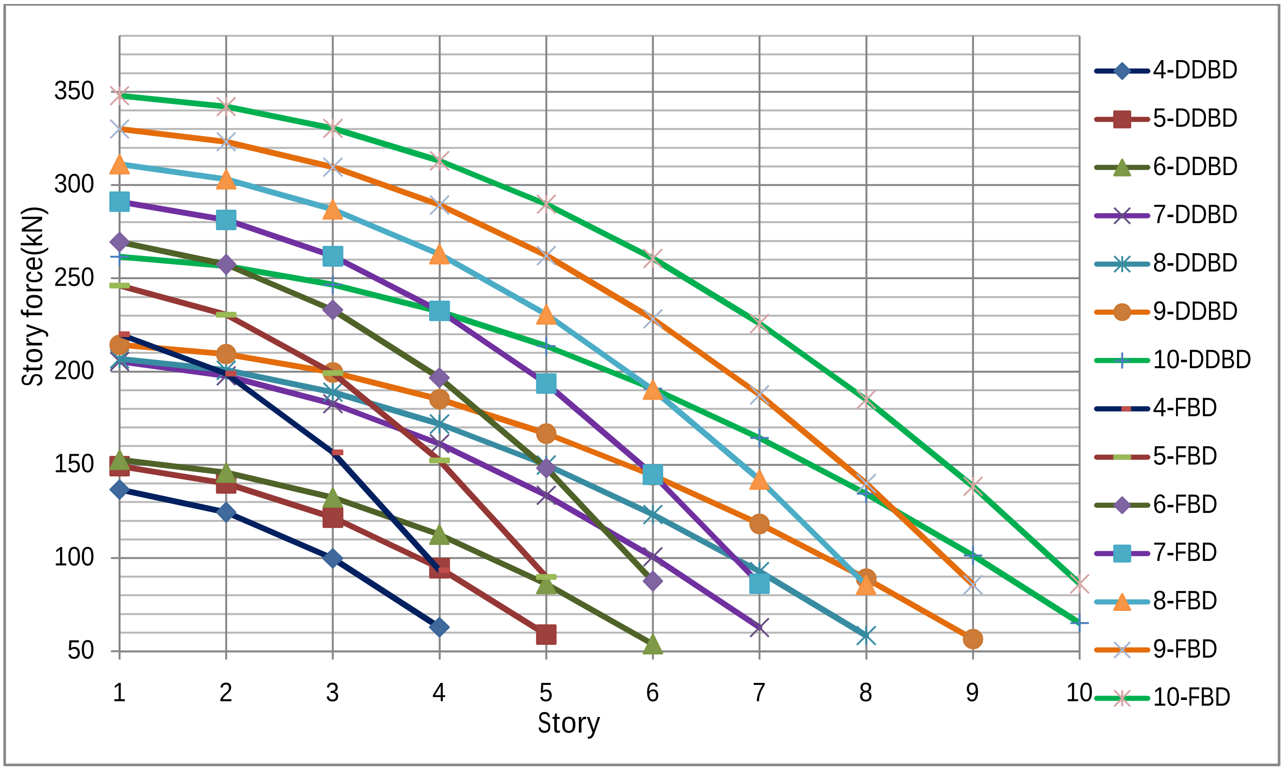


Fig 4.19. Story forces variation resulting from both methods for 3.8m story height

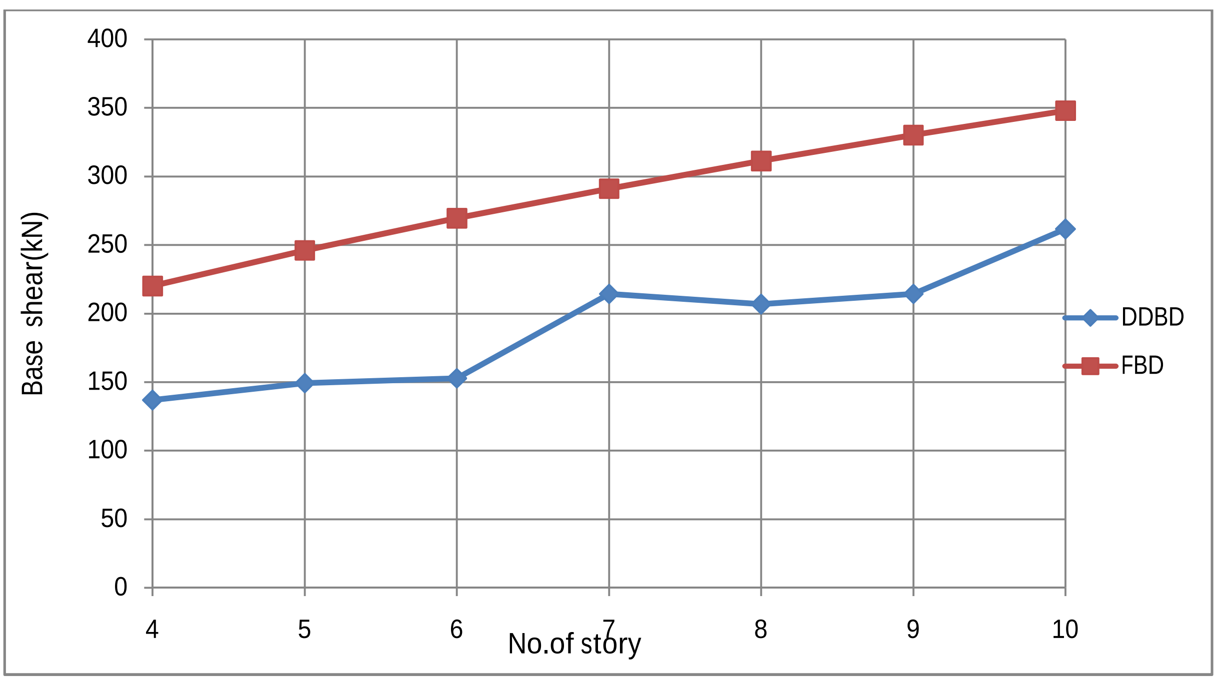


Fig 4.20. Base shear force variation resulting from both methods for 3.8m story height.

Results for Story height of 4m

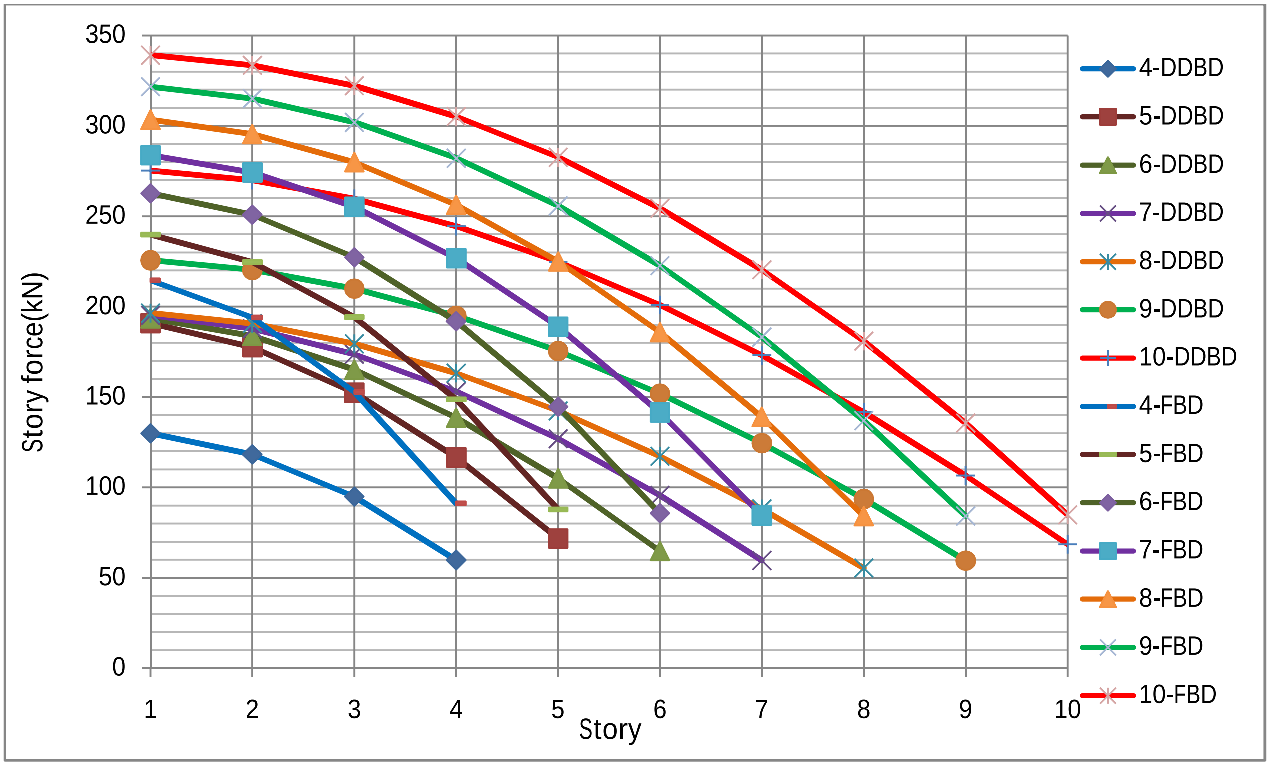


Fig 4.21. Story shear forces variation resulting from both methods for 4m story height

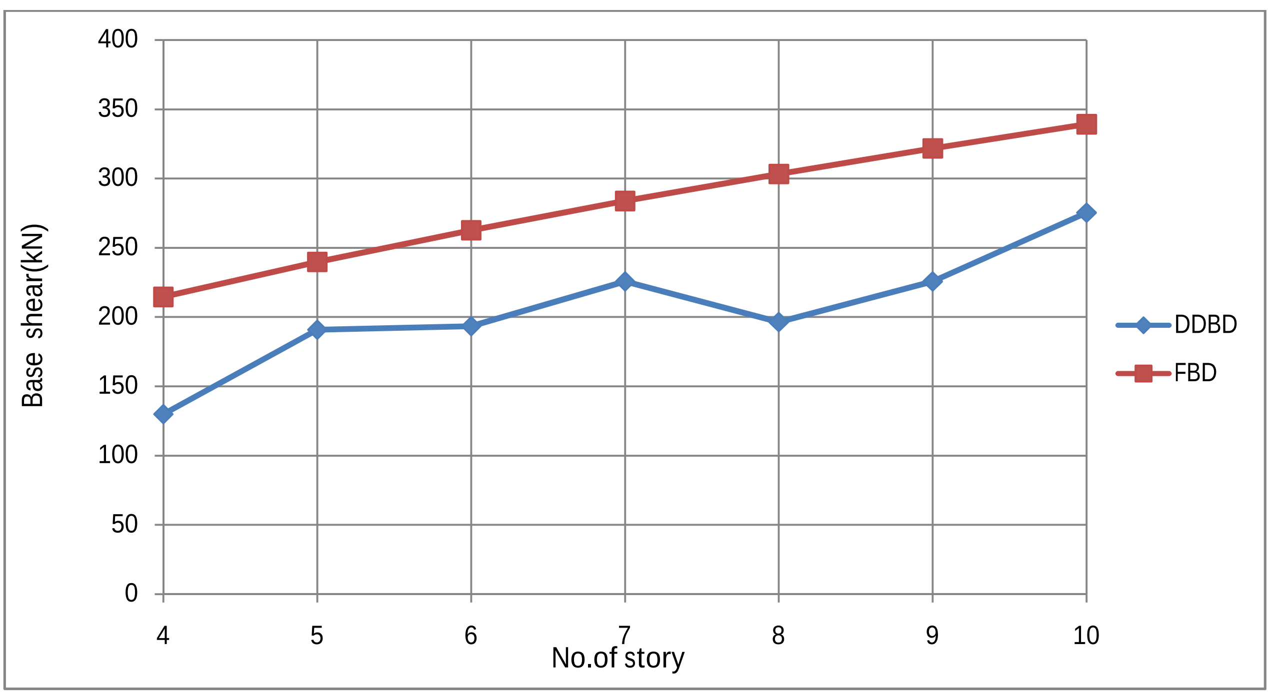


Fig 4.22. Base shear force variation resulting from both methods for 4m story height.

The above results can be summarized by comparing the values only in terms of base shear values. The comparison is tabulated and shown in graphical format as shown below.

Story	2.8m height			3m height			3.2m height		
	DDBD	FBD	%DIFF	DDBD	FBD	%DIFF	DDBD	FBD	%DIFF
4	198.9	256.3	28.9	173.2	247.6	42.9	162.4	239.8	47.6
5	272.7	286.6	5.1	254.5	276.9	8.8	238.6	268.1	12.3
6	276.3	313.9	13.6	257.9	303.3	17.6	241.8	293.7	21.5
7	281.9	339.10	20.3	263.1	327.6	24.5	246.7	317.2	28.6
8	280.6	362.5	29.2	261.9	350.2	33.7	245.5	339.1	38.1
9	281.9	384.5	36.4	263.1	371.5	41.2	246.7	359.7	45.8
10	283.0	405.3	43.2	264.1	391.6	48.3	247.6	379.1	53.1
Average			25.2			31.0			35.3

Story	3.5m height			3.8m height			4m height		
	DDBD	FBD	%DIFF	DDBD	FBD	%DIFF	DDBD	FBD	%DIFF
4	148.5	229.3	54.4	136.8	220.0	60.9	129.9	214.5	65.1
5	218.2	256.3	17.5	149.3	246.0	64.8	190.9	239.8	25.6
6	221.1	280.8	27.0	152.7	269.5	76.5	193.4	262.7	35.8
7	225.5	303.3	34.5	214.4	291.1	35.8	225.6	283.7	25.7
8	224.5	324.2	44.4	206.8	311.2	50.5	196.4	303.3	54.4
9	225.5	343.9	52.5	214.4	330.1	54.0	225.6	321.7	42.6
10	240.9	362.5	50.5	261.5	347.9	33.0	275.3	339.1	23.2
Average			40.1			53.6			38.9

Table 4.6 Variation of base shear forces for different story height

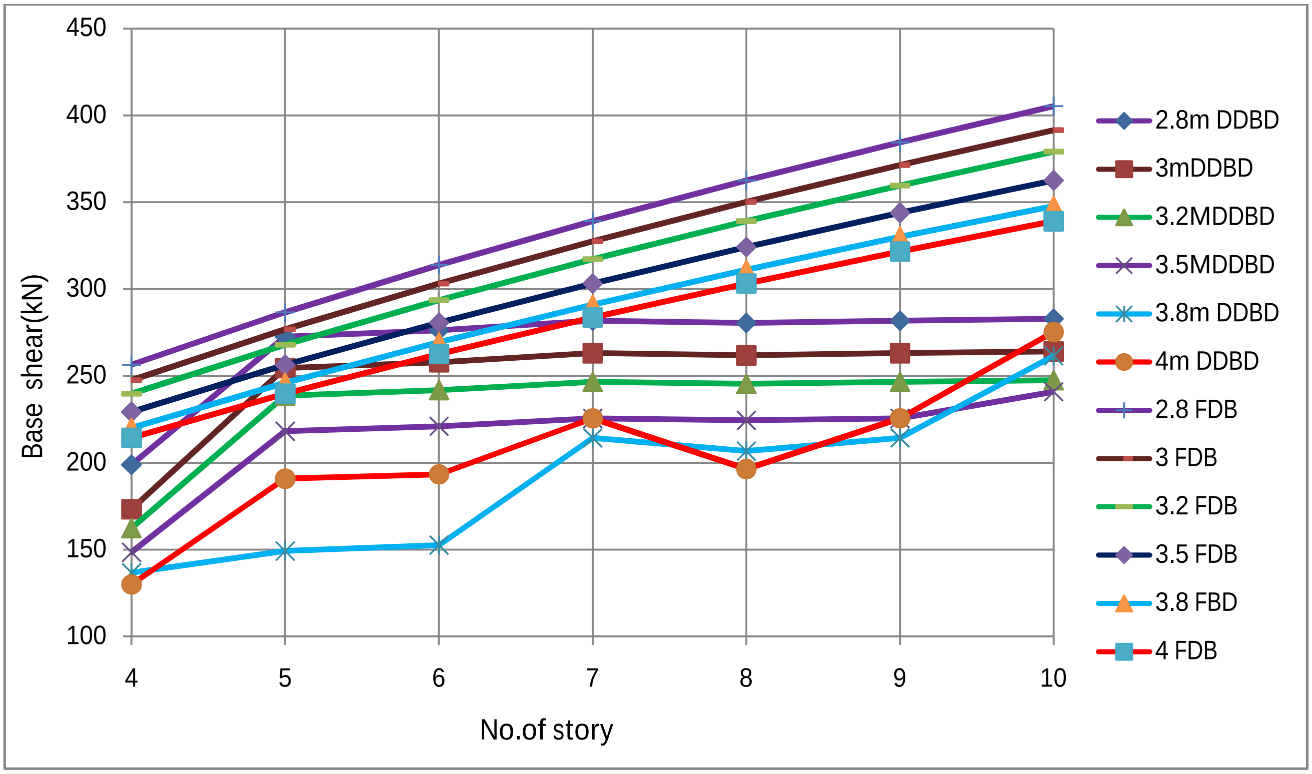


Fig 4.23 Variation of shear forces for different story heights for both methods

Similarly, from the results shown above both from the table and the figure it is observed clearly that the base shear demand increases as the story height decrease for both methods. This is because that the damping capacity decreases with smaller story height. But the trend of variation of strength demand is not linear as that of the force method results for direct displacement based design case.

The variation of the strength demand resulting from displacement based approach and force-based approach becomes more significant as the story height increases. Ranging from 25.2% for 2.8m story height to 53.6% for 3.8m in average and comes down for 4m. Clearly from this the story height of the building really affect the damping effect of the structure so that the corresponding strength requirement too. This phenomenon is accounted in direct displacement based approach in a more transparent way than the force based approach. The result also shows us that the results of force based design approach are still unnecessarily and more importantly unsafely conservative for most frames.

Case-3:

The third case considers the variation of beam depth for different stories. The span length of 5m is taken for four to ten stories and the story height of 3.5 m is taken as constant for all frames. The structure is also assumed to be founded on soil class B as in the previous case. The result is calculated and shown as below in graphical form only the table format is also documented as an annex:

Results for Beam depth of 0.4m

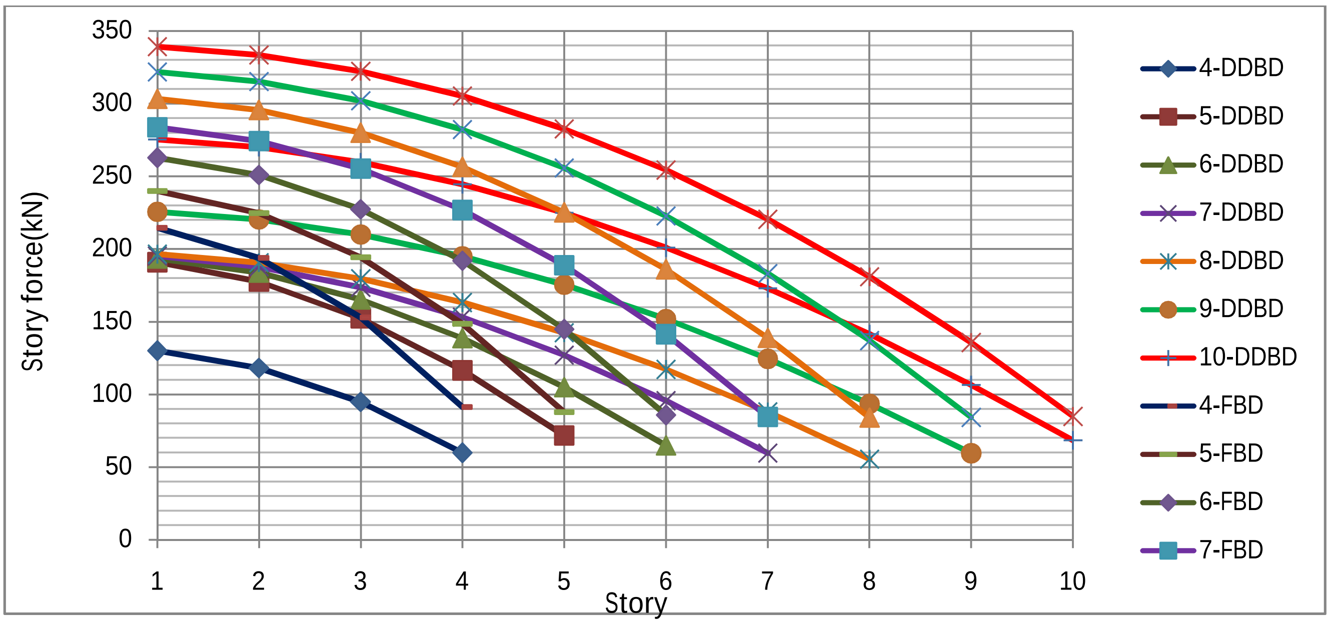


Fig 4.24. Story shear forces variation resulting from both methods for 0.4m beam depth

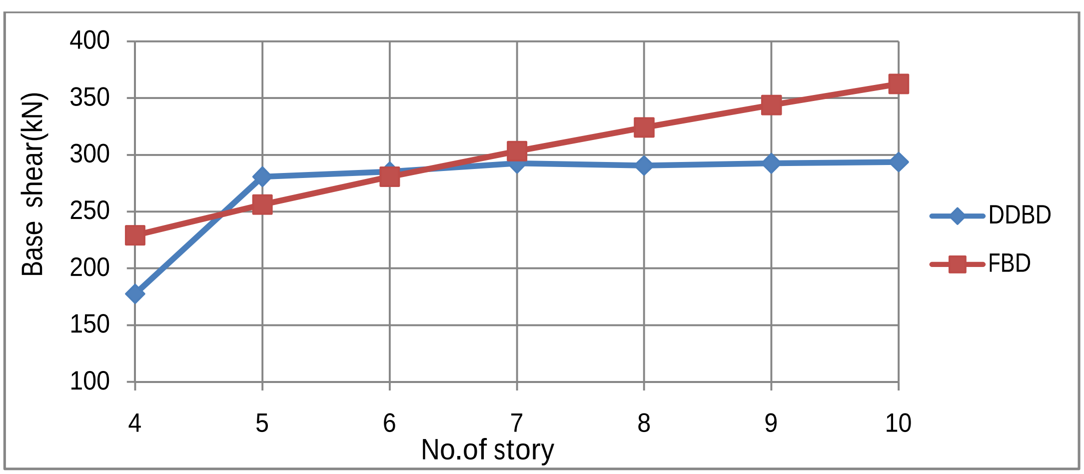


Fig 4.25. Base shear force variation resulting from both methods for 0.4m beam depth.

Results for Beam depth of 0.5m

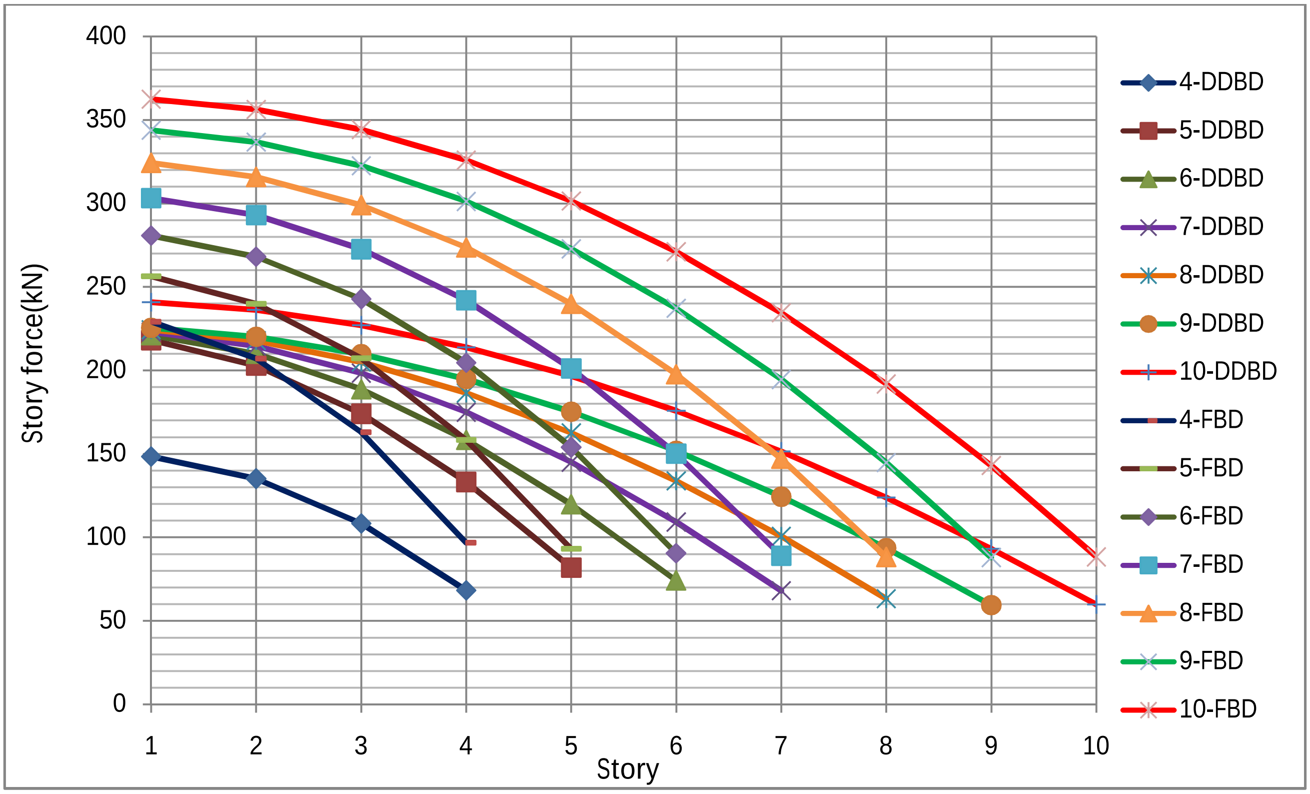


Fig 4.26. Story forces variation resulting from both methods for 0.5m beam

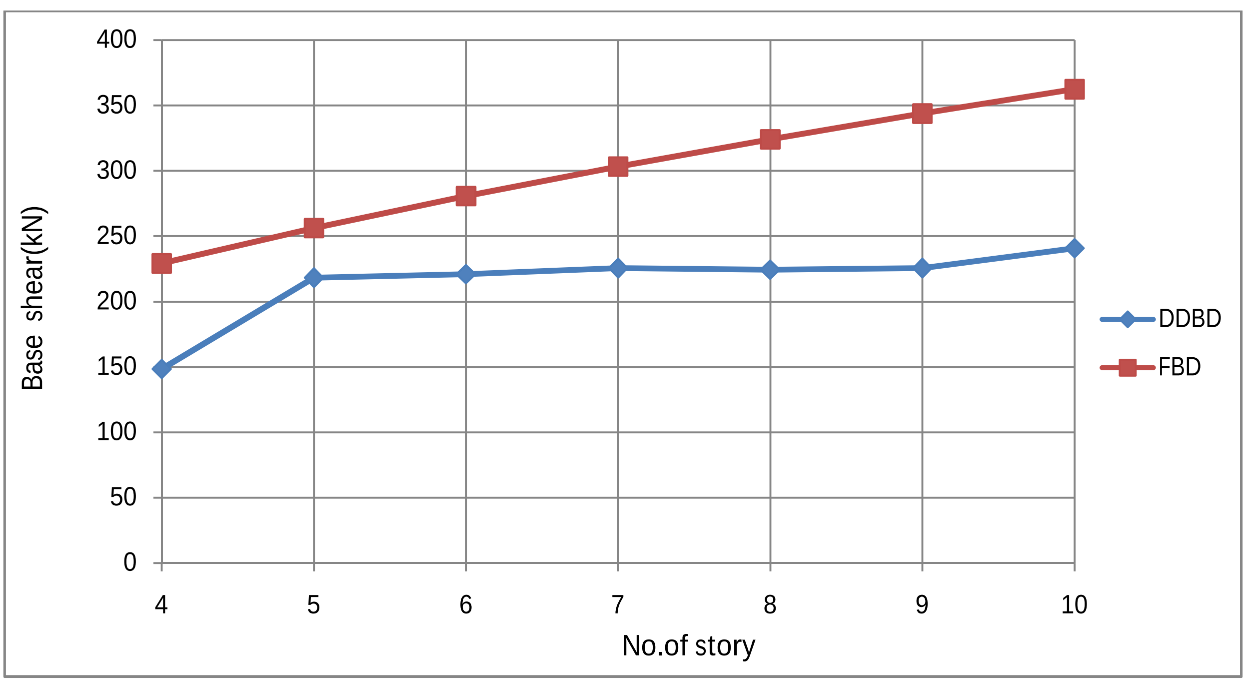


Fig 4.27. Base shear force variation resulting from both methods for 0.5m beam depth.

Results for Beam depth of 0.6m

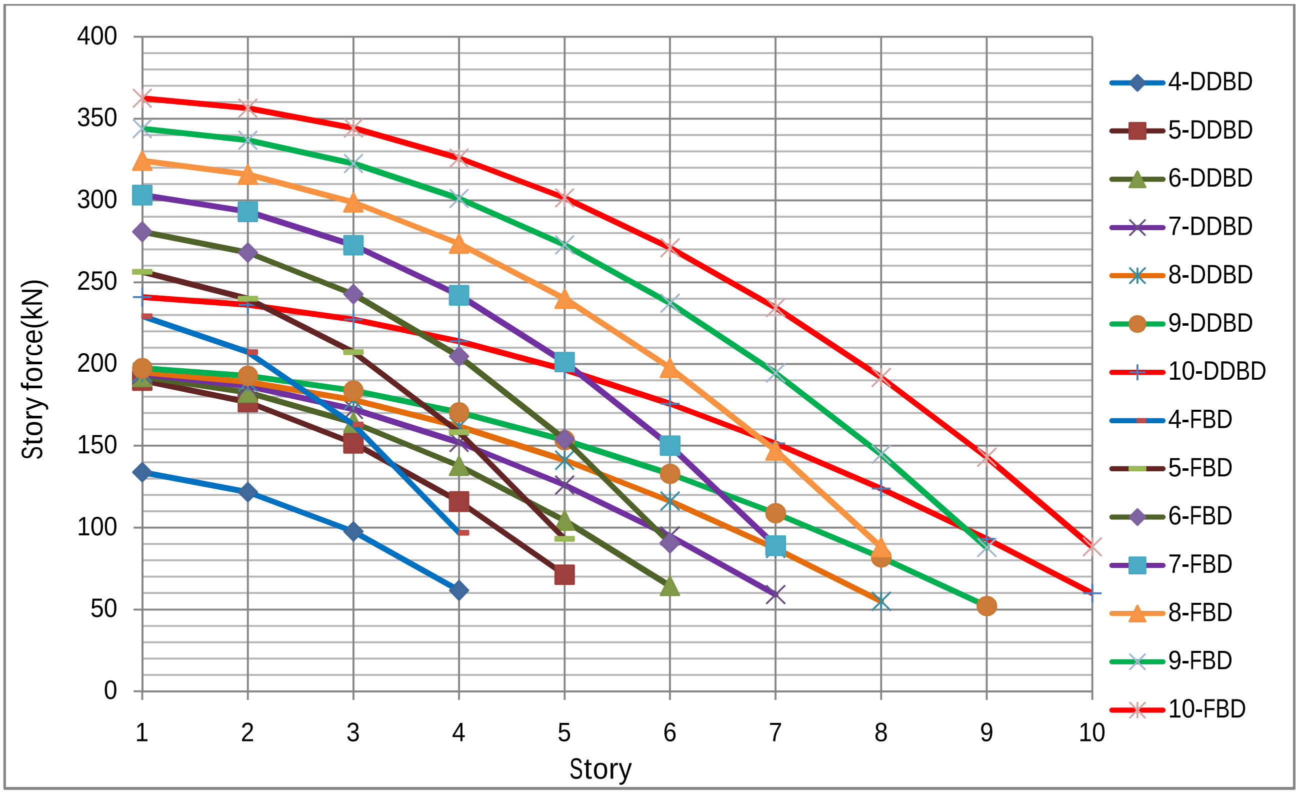


Fig 4.28. Story forces variation resulting from both methods for 0.6m beam

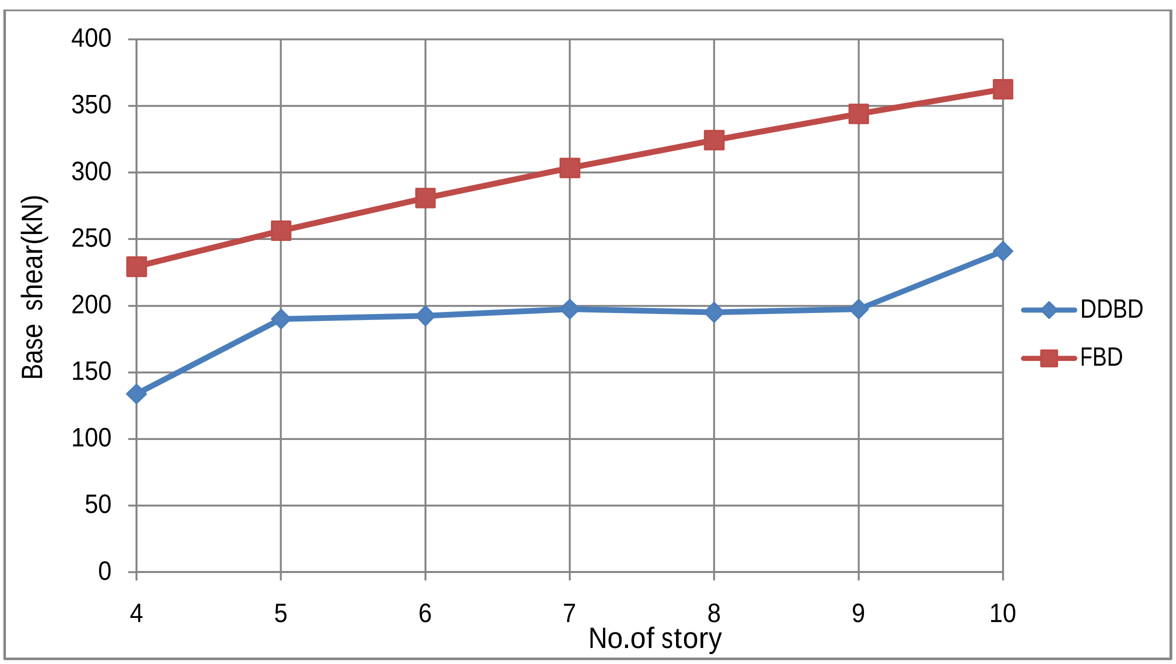


Fig 4.29 Base shear force variation resulting from both methods for 0.6m beam depth

Results for Beam depth of 0.7m

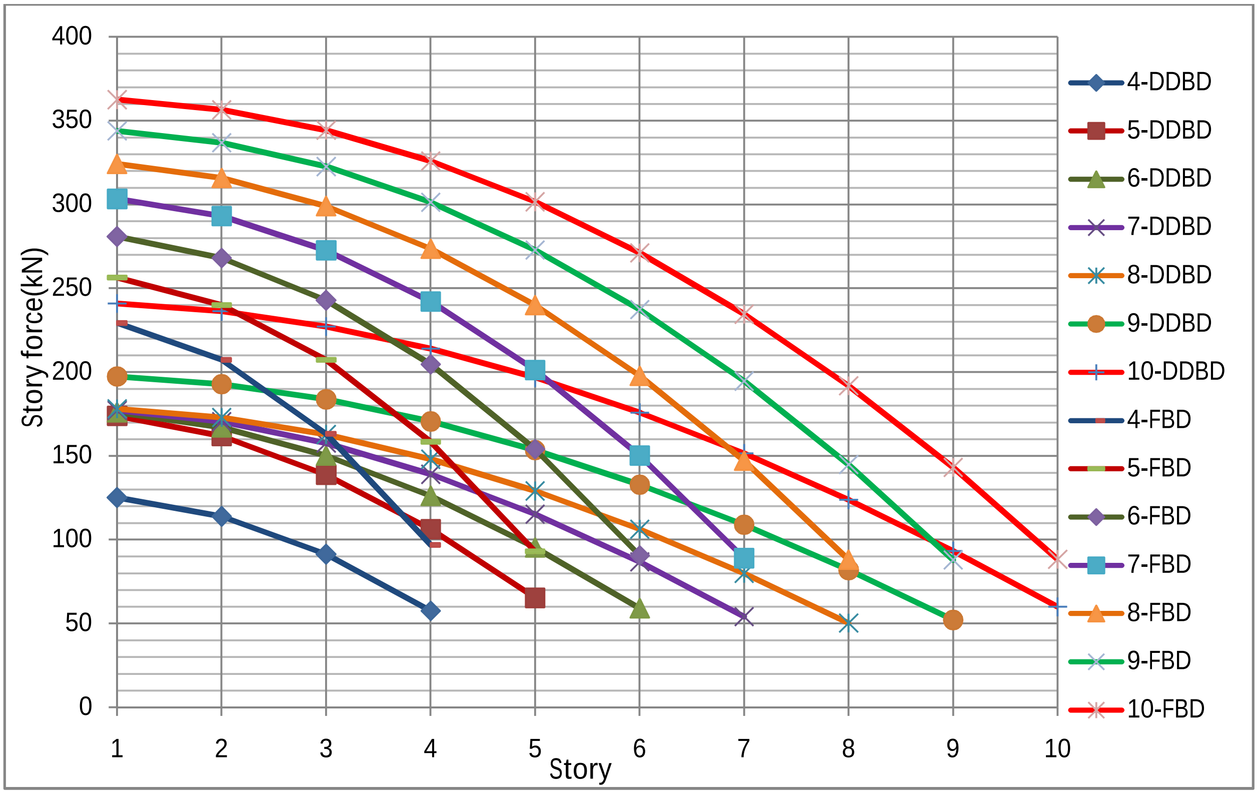


Fig 4.30. Story shear forces variation resulting from both methods for 0.7m beam

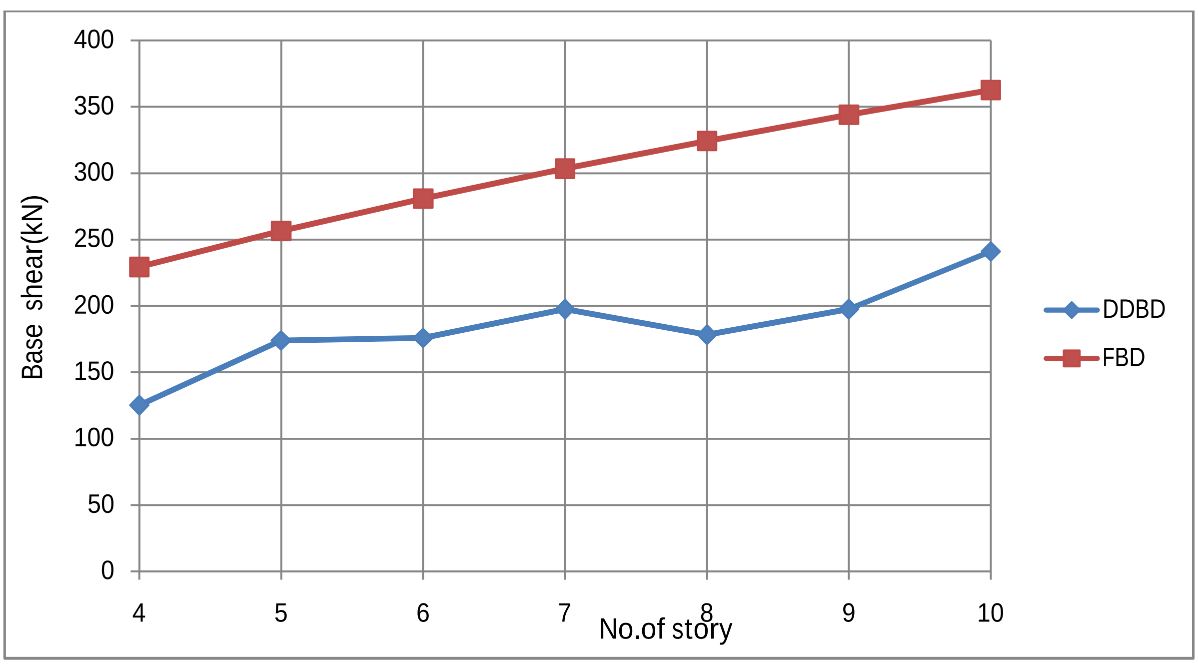


Fig 4.31 Base shear force variation resulting from both methods for 0.7m beam depth

Results for Beam depth of 0.8m

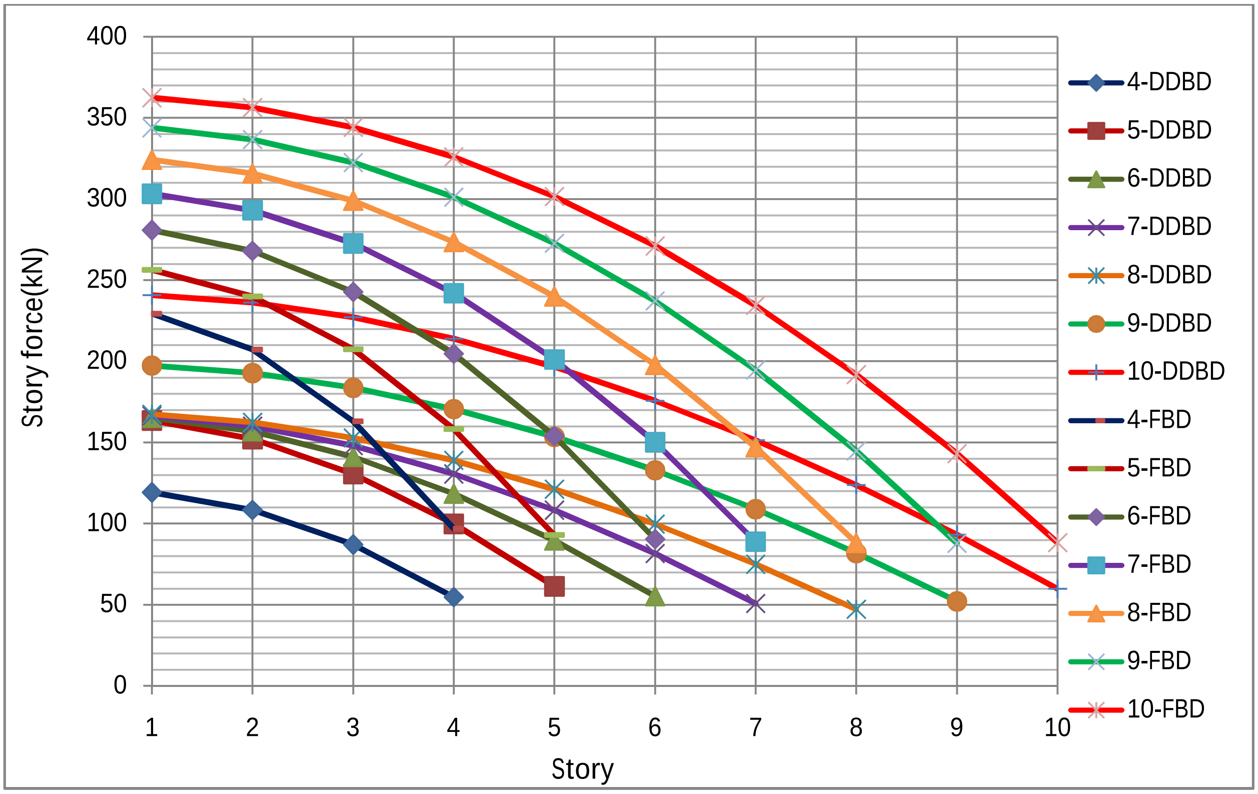


Fig 4.32. Story forces variation resulting from both methods for 0.8m beam

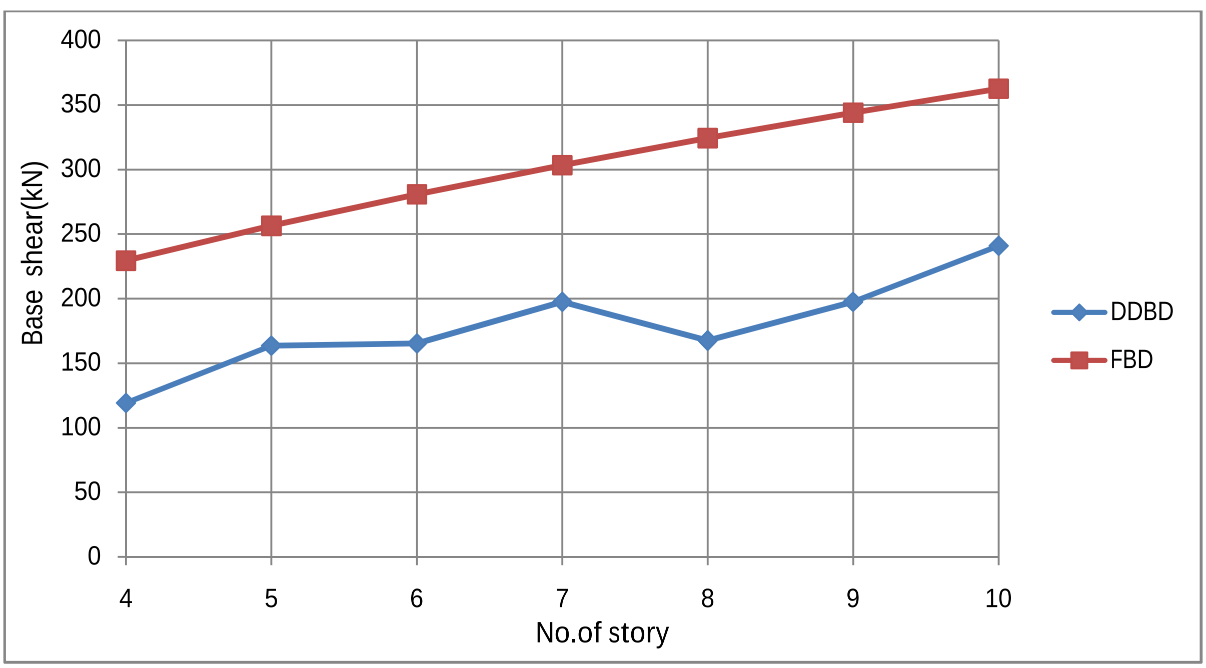


Fig 4.33 Base shear force variation resulting from both methods for 0.8m beam depth

The above results can be summarized by comparing the values only in terms of base shear values. The comparison is tabulated and shown in graphical format as shown below.

	0.4m depth			0.5m depth			0.6m depth		
Story	DDBD	FBD	%DIFF	DDBD	FBD	%DIFF	DDBD	FBD	%DIFF
4	177.6	229.3	29.1	148.5	229.3	54.4	133.9	229.3	71.3
5	280.6	256.3	-8.7	218.2	256.3	17.5	190.0	256.3	34.9
6	285.2	280.8	-1.5	221.1	280.8	27.0	192.3	280.8	46.1
7	292.4	303.30	3.7	225.5	303.3	34.5	197.4	303.3	53.6
8	290.7	324.2	11.5	224.5	324.2	44.4	194.9	324.2	66.4
9	292.4	343.9	17.6	225.5	343.9	52.5	197.4	343.9	74.2
10	293.8	362.5	23.4	240.9	362.5	50.5	240.9	362.5	50.5
Average			10.7%			40.1%			56.7%

	0.7m depth			0.8m depth		
Story	DDBD	FBD	%DIFF	DDBD	FBD	%DIFF
4	125.1	229.3	83.3	119.2	229.3	92.3
5	173.9	256.3	47.4	163.6	256.3	56.7
6	175.9	280.8	59.7	165.3	280.8	69.9
7	197.4	303.30	53.6	197.4	303.3	53.6
8	178.1	324.2	82.0	167.3	324.2	93.8
9	197.4	343.9	74.2	197.4	343.9	74.2
10	240.9	362.5	50.5	240.9	362.5	50.5
Average			64.4%			70.1%

Table 4.7 Variation of shear forces for different beam depths

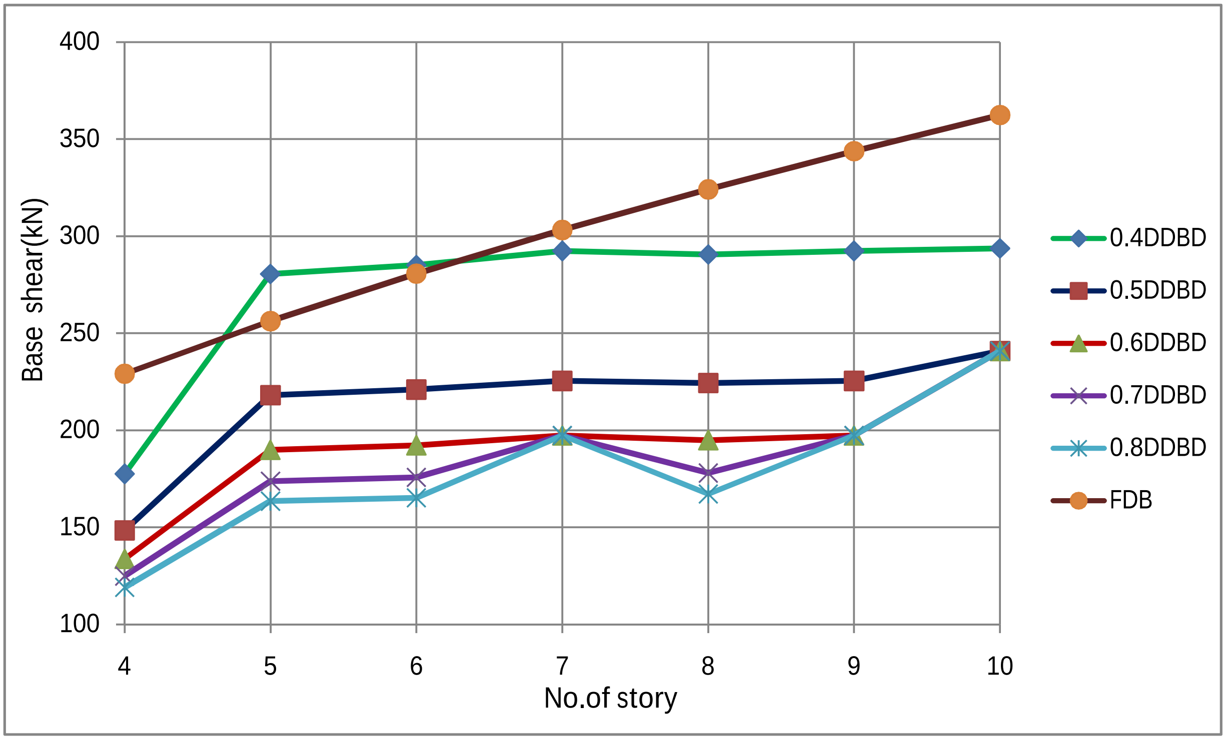


Fig 4.34 Variation of shear forces for different beam depths

The result shown above for different beam depths shows that the value of the beam depth is highly appreciated and significantly affect the damping of the structure so that the required strength is different correspondingly when we use direct displacement based design philosophy. This consideration is not apparent when force based philosophy is applied. The variation of results become more significant as the depth of the beam used increases. The variation increases from 10.7% for 0.4m depth to 70.1% when 0.8m depth of beam is selected in average.

Using deeper beam depths reduces the strength requirement for lateral load resisting system if direct displacement based method is used whereas for the force method, since this phenomenon is not consider directly the strength requirement is conservative in unsafe manner for majority cases and lesser strength than required is resulted for beam depth of 0.4m and five story frame. This gives an option for a designer to reduce the required strength for lateral load resisting system by increasing the beam depth. In this case also the transparency and direct accountancy of parameters to be considered is clear for direct displacement based design philosophy.

4.4 Checking for the Serviceability requirement.

In order to check the serviceability requirement of the code using the analysis method recommended by the two approaches, four frames are taken among the frames considered previously in the first case. Frames of 4m and 7m span with four story and nine story reinforced concrete frames are taken to see the serviceability requirement of the code. The results for different column cross-sections using both methods of DDBD and FBD are compared.

From the results, for the 4m span frames a larger column cross sectional dimension demands in the case of force based design approach method as compared with that of direct displacement based approach to satisfy the serviceability requirement of the code recommendation. The condition described above is reversed when a span of 7m is taken. In this case the drift values of DDBD approach is larger than the FBD approach and it needs larger cross sectional dimension.

This noticeable difference is visible due two reasons. The first one is the difference in the procedures of the methods to account different parameters as it is discussed in the previous parametric studies and their results. Specially, in this case the span length is considered directly in the case of DDBD approach and influences significantly the response of the structure but, in the case of FBD approach this parameter is not considered directly. The other important reason is that the variation of analysis recommendations in consideration of cross-section property modification. In the case of DDBD approach, in order to insure weak beam- strong column principle of capacity design approach the cracked section stiffness is further reduced by system ductility which results higher drift value. But for the case of FBD approach cracked section stiffness without any further reduction is used.

4.5 Summary of results and discussion.

The results obtained from the parametric studies are presented in the previous sections both in the form of graph and in tabular format. Here, the results are summarized once again and discussed briefly. To start from the first case of variation of frame span length, which considers spans of 4m, 5m, 6m and 7m of stories ranging from 4 to 10, the result shows us that the base shear increases as the span of the beam increases for DDBD approach. When it is compared with FBD results, for shorter spans the FBD results of base shear is greater than that of DDBD results. Whereas, for 7m span the result is reversed and the DDBD base shear is greater than that of the FBD. This similar result is also obtained when the frame is checked for serviceability requirement. Due to the approaches difference in parameters consideration and analysis procedure recommendation differences, for the shorter spans a sectional requirement of FBD approach is greater than that of DDBD whereas the result is reversed for 7m long span.

One can sense that, as the span length increases the frame damping capacity decreases, which means that the required strength should be increased. This logic is entertained in the DDBD approach which seems to be more convenient and rational than that of FBD approach as there is no means of accounting this parameter directly.

From the second case of story height variation, the base shear increases as the story height decreases for both methods. The trend of variation for FBD with the story height is linear but not for DDBD case. The difference of results of the two cases is significant for longer story height which results a difference of about 53.6% in average for 3.8m story height.

From the third case of beam depth variation, the value of beam depth affects the damping of the structure significantly so that the required strength is different correspondingly when we use DDBD philosophy. But this parameter is not considered when FBD approach is followed so that it gives similar value for different beam depths without consideration of its effect. The difference increases from 10.7% for 0.4m depth of beam to 70.1% when 0.8m depth of beam is selected in average. From this result using deeper beams reduces the strength requirement of the lateral load resisting system when DDBD method is used. Whereas, for the FBD method the resulting strength requirement is unnecessarily conservative in unsafe manner for majority of cases, since this parameter is not considered.

In all of the parameters considered in this study, they affect the damping of the structure so that the resulting base shear is different. These parameters are considered in the transparent and rational way in the DDBD approach. Especially the span length and the beam depth parameters are very crucial to influence the structural responses. In contrary, they are not considered for FBD approach directly so that the rationality and its logicality becomes in question.

Inelastic seismic design achieves economy because the system is designed for a reduced base shear. However, ductility is required to ensure that this inelastic response is obtained and, as well, it is implicit that the structural and non-structural damage consistent with the response must be accepted. In the force-based design method displacements are treated in a somewhat cursory manner and are checked at the end of the design process only. There appears to be a lack of concern about the implied inelastic displacements both non-structural and structural elements may be deemed unsatisfactory if they deform excessively under earthquakes. At the ultimate limit states, the deformations are likely to contribute to the instability of the structure and, as well, the damage, perhaps implying that the building is partially or completely non-functional or even beyond repair, may also be considered unacceptable. These limit states are governed by deformations. Furthermore, it may be easier to define failure of a structural element as a limit on deformation rather than as a limit on the force. Therefore, it seems rational to examine a seismic design method wherein displacements are considered at the start of the design process with attention focused on deformations to provide a structure that meets the requirements for the several limit states. A serviceability limit state on deformations could be applied under moderate earthquakes that are likely to occur relatively frequently in the life of the structure (by imposing drift limits so that non-structural damage is limited or does not occur). To prevent collapse in a major earthquake, the ductility demand on the structural elements and the overall deformation of the structure would have to be controlled. It is suggested this can be achieved more rationally with a displacement-based rather than force-based design method.

CHAPTER FIVE

5. Conclusion and Recommendation

5.1 Conclusions

In this thesis work the conceptual basis of the force-based design method currently used in seismic codes is reviewed and its limitations are discussed. An alternative method that uses displacements as the basis for the design procedure is then presented. Its conceptual basis for elastic and inelastic seismic design and its application to single-degree-of-freedom and multi-degree-of-freedom structures is assessed. By focusing especial consideration for reinforced concrete frames, its procedure of application is studied and shown by demonstrating with an example. Once this new design philosophy conceptual back ground and its procedure is understood, further study is continued on the parametric studies by taking frame span length, story height and beam depth as a parameter to see their influence on the structural responses and for comparison of the results with the current trend of force-based design philosophy.

From philosophy point of view, it is found that this new method is based on the fundamental response of the structure, which determines its performance that is displacement is used as a basis instead of strength as in the case of current tradition of force-based design philosophy. So that, the method is based on logical theoretical background in addition to being convincing due to its transparency. It is observed that the displacement-based design method is considered to offer the following advantages over the force-based design method.

1. Displacements play a major role at the preliminary design stage itself resulting in good control on displacements over the entire design process. Target displacement criteria are selected for the serviceability and ultimate limit states and thus damage control is achieved directly.
2. The strength and stiffness of the lateral load resisting system are chosen to satisfy the desired deformation criteria.
3. Empirical equations for estimating the fundamental period of the structure for preliminary design of the lateral load resisting system are not required as in the case of force-based approach.
4. The empirical and somewhat arbitrary force modification factor, used in the force- based design method, is not needed.

5. It directly addresses the inelastic nature of a structure during an earthquake.
6. Address service and ultimate limit states using the same design procedure.
7. Parameters those contribute for the damping of the structures are considered directly.

From the parametric study, it is found that the current force based design approach do not account the parameters directly how they influence the damping capacity of the structure. For example by varying the span length of the frame, what is observer from the resulting values is that for shorter span lengths FBD results of base shear is greater than DDBD values where as for longer span the result is reversed. The same observation is also noticed in terms of their cross-sectional requirement checked by their serviceability limit code recommendation. Similarly, from the variation of beam depths, due to its consideration in DDBD approach, the resulting responses are very different when they are compared with FBD approach. Lesser results are recorded for DDBD approach when the beam depth increases, but not for FBD approach. Because of this, in many cases unnecessary and unsafe conservatism is observed like for shorter span length, deeper beam and longer story heights. Those increase the damping capacity so that it reduces the required strength. In other cases, there are also situations where necessary strength is not provided and makes the structure unsafe. This is due to that the necessary parameters are not accounted, instead a single reduction factor is provided for different structures in the code for the current force-based design philosophy.

It can be concluded that by using the substitute structure approach, the ultimate displacement of buildings can be well estimated by the displacement-based design procedure. Strength and stiffness are a result of the design procedure and are dependant of the on the target displacement. The method provides a rational seismic design procedure that is compatible with the philosophy that structures are designed to undergo plastic deformation in large earthquake while satisfying service criteria in small earthquake.

In general, the displacement based design philosophy is transparent, it is simple for practical application, relays on convincing and logical theoretical back ground as well as it is effective means of controlling seismic response and accounting parameters those affect the structural responses.

5.2. Recommendation

A mentality setup of structural engineers, to get safer structure make it stronger, is getting deteriorated due to recent recognition of structural responses for seismic attack in which structural performance depends on displacement capacity rather than its strength. So that the performance based design can be employed through the direct displacement based seismic design for new structures instead of its restriction to check it for designed and constructed buildings as is done currently. So from the observation of this work, it is recommended to introduce and increase the awareness of the structural engineers in order to provide economical and safer structure. This can be achieved by including this new design philosophy in the curriculum of the course for structural engineers and in the seismic design codes as this is the current trend in the rest of the world.

In this study, an effort is done to be introduced about the new design philosophy and study about limited parameters those affect the response and its comparison with the current method for reinforced concrete frames. As it is new in its kind and it is the current issue of the world community, it is open for future intense works those helps to widen the horizon of our understanding for designing of structures for seismic load and for improvement of our day to day practical designing trend to be founded from logical and convincing theoretical back ground. Some of the recommended working areas are listed below:

- The structural system considered in this work is reinforced concrete frame building only. So it is possible to see the effect in detail for other structural systems such as dual wall frame buildings, steel and timber structures.
- Away from buildings, showing how the design philosophy is applicable for bridge structure can be taken as another concern.
- Computation done for this thesis work is performed using hand calculation except excel program is developed. Most of the available commercial soft wares for analysis purpose are developed based on the current trend of seismic design. So that response spectrums are developed for acceleration not for displacement. This gap can be filled in future works by developing a program for analysis of structures using displacement response spectrum.

Annex

Tabular results for case-2

Results of 2.8m story height

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIF**
1	198.9	256.3	28.9	272.7	286.6	5.1	276.3	313.9	13.6	278.8	339.1	21.6
2	181.0	231.5	27.9	253.7	268.2	5.7	262.5	299.6	14.2	268.3	327.6	22.1
3	145.2	181.9	25.3	217.6	231.4	6.4	236.0	271.0	14.9	248.0	304.5	22.8
4	91.5	107.5	17.5	166.5	176.3	5.9	198.1	228.2	15.2	218.8	270.0	23.4
5				102.4	102.8	0.4	149.9	171.0	14.1	181.4	223.9	23.4
6							92.7	99.5	7.3	136.6	166.3	21.8
7										85.2	97.2	14

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	280.6	362.5	29.2	281.9	384.5	36.4	283.0	405.3	43.2
2	272.3	353.0	29.6	275.3	376.5	36.8	277.5	398.4	43.6
3	256.4	333.9	30.3	262.4	360.4	37.4	266.8	384.6	44.1
4	233.2	305.4	31.0	243.5	336.3	38.1	251.3	363.9	44.8
5	203.3	267.3	31.4	219.2	304.1	38.7	231.1	336.3	45.6
6	167.4	219.6	31.2	189.8	263.9	39.1	206.5	301.8	46.2
7	125.8	162.5	29.1	155.5	215.7	38.7	177.9	260.4	46.4
8	79.2	95.8	21.0	117.0	159.4	36.3	145.5	212.2	45.8
9				74.4	95.1	27.8	109.5	157.0	43.3
10							70.4	94.9	34.8

Table app.1 Variation of story shear forces for 2.8m story height

Results of 3m story height

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDB**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDB**	**FDB**	**% DIF**
1	173.2	247.6	42.9	254.5	276.9	8.8	257.9	303.3	17.6	260.2	327.6	25.9
2	157.7	223.7	41.9	236.8	259.2	9.5	245.0	289.5	18.2	250.4	316.5	26.4
3	126.5	175.9	39.1	203.1	223.7	10.1	220.3	262.0	18.9	231.5	294.3	27.1
4	79.7	104.1	30.6	155.4	170.6	9.7	184.9	220.6	19.3	204.2	261.0	27.8
5				95.6	99.7	4.3	139.9	165.5	18.3	169.3	216.6	27.9
6							86.5	96.6	11.7	127.5	161.1	26.4
7										79.5	94.6	19

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	261.9	350.2	33.7	263.1	371.5	41.2	264.1	391.6	48.3
2	254.2	341.0	34.2	256.9	363.7	41.6	259.0	384.9	48.6
3	239.3	322.7	34.9	244.9	348.2	42.2	249.1	371.6	49.2
4	217.6	295.2	35.6	227.3	325.0	43.0	234.5	351.7	50.0
5	189.8	258.5	36.2	204.6	294.0	43.7	215.7	325.2	50.8
6	156.2	212.6	36.1	177.1	255.3	44.2	192.7	292.0	51.5
7	117.4	157.6	34.2	145.2	208.9	43.9	166.0	252.1	51.9
8	73.9	93.3	26.3	109.2	154.7	41.7	135.8	205.6	51.5
9				69.4	92.8	33.6	102.2	152.5	49.2
10							65.7	92.8	41.2

Table app.2 Variation of story shear forces for 3m story heights

Results of 3.2m story height

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	162.4	239.8	47.6	238.6	268.1	12.3	241.8	293.7	21.5	244.0	317.2	30.0
2	147.8	216.7	46.6	222.0	251.0	13.1	229.7	280.4	22.1	234.8	306.5	30.6
3	118.6	170.4	43.7	190.4	216.7	13.8	206.5	253.7	22.9	217.0	285.0	31.3
4	74.7	101.0	35.2	145.7	165.4	13.5	173.3	213.8	23.4	191.5	252.9	32.1
5				89.6	96.9	8.1	131.1	160.6	22.4	158.7	210.0	32.3
6							81.1	94.0	15.9	119.5	156.5	30.9
7										74.5	92.2	24

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	245.5	339.1	38.1	246.7	359.7	45.8	247.6	379.1	53.1
2	238.3	330.2	38.6	240.9	352.2	46.2	242.8	372.7	53.5
3	224.3	312.5	39.3	229.6	337.3	46.9	233.5	359.9	54.1
4	204.0	286.0	40.2	213.1	314.8	47.7	219.9	340.7	55.0
5	177.9	250.5	40.8	191.8	285.0	48.6	202.2	315.1	55.8
6	146.5	206.2	40.8	166.0	247.6	49.1	180.7	283.0	56.6
7	110.1	153.1	39.1	136.1	202.8	49.0	155.6	244.6	57.1
8	69.3	91.1	31.5	102.3	150.5	47.0	127.3	199.7	56.9
9				65.1	90.7	39.4	95.8	148.5	54.9
10							61.6	90.8	47.5

Table app.3   Variation of story shear forces for 3.2m story heights

Results of 3.8m story height

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	136.8	220.0	60.9	149.3	246.0	64.8	152.7	269.5	76.5	205.4	291.1	41.7
2	124.5	198.9	59.8	140.0	230.4	64.6	145.9	257.4	76.4	197.7	281.3	42.3
3	99.8	156.7	56.9	121.7	199.2	63.7	132.4	233.1	76.1	182.8	261.8	43.3
4	62.9	93.3	48.4	94.6	152.3	61.0	112.4	196.7	75.0	161.2	232.5	44.2
5				58.9	89.8	52.5	86.1	148.2	72.1	133.7	193.5	44.8
6							53.7	87.5	63.1	100.6	144.7	43.8
7										62.7	86.2	37

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	206.8	311.2	50.5	214.4	330.1	54.0	261.5	347.9	33.0
2	200.7	303.1	51.1	209.3	323.3	54.4	256.5	342.1	33.4
3	188.9	287.0	51.9	199.5	309.7	55.2	246.6	330.5	34.0
4	171.8	262.8	53.0	185.2	289.3	56.2	232.2	313.0	34.8
5	149.8	230.6	53.9	166.7	262.2	57.3	213.6	289.7	35.7
6	123.3	190.3	54.3	144.3	228.2	58.2	190.9	260.6	36.6
7	92.7	142.0	53.2	118.3	187.5	58.5	164.4	225.7	37.3
8	58.4	85.6	46.7	88.9	139.9	57.4	134.4	185.0	37.6
9				56.6	85.6	51.4	101.2	138.5	36.8
10							65.1	86.1	32.4

Table app.4   Variation of story shear forces for 3.8m story heights

Results of 4m story height

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	129.9	214.5	65.1	190.9	239.8	25.6	193.4	262.7	35.8	195.2	283.7	45.4
2	118.2	193.9	64.0	177.6	224.6	26.5	183.7	250.9	36.5	187.8	274.2	46.0
3	94.9	152.8	61.1	152.3	194.2	27.5	165.2	227.3	37.6	173.6	255.3	47.0
4	59.8	91.2	52.6	116.6	148.6	27.5	138.6	191.9	38.4	153.2	226.8	48.1
5				71.7	87.9	22.6	104.9	144.7	37.9	127.0	188.9	48.7
6							64.9	85.7	32.2	95.6	141.4	47.9
7										59.6	84.5	42

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	196.4	303.3	54.4	225.6	321.7	42.6	275.3	339.1	23.2
2	190.6	295.5	55.0	220.3	315.1	43.0	270.0	333.4	23.5
3	179.4	279.8	55.9	210.0	301.9	43.8	259.6	322.1	24.1
4	163.2	256.3	57.0	194.9	282.1	44.7	244.5	305.2	24.8
5	142.3	225.0	58.1	175.4	255.7	45.8	224.8	282.6	25.7
6	117.2	185.9	58.6	151.9	222.7	46.7	200.9	254.3	26.6
7	88.1	138.9	57.7	124.5	183.2	47.1	173.1	220.4	27.4
8	55.4	84.1	51.6	93.6	137.0	46.3	141.5	180.9	27.8
9				59.5	84.2	41.4	106.6	135.7	27.3
10							68.5	84.8	23.9

Table app.5 Variation of story shear forces for 4m story heights

Tabular results for case-3

Results of 0.4m beam depth

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	177.6	229.3	29.1	280.6	256.3	-8.7	285.2	280.8	-1.5	288.4	303.3	5.2
2	161.6	207.2	28.2	261.0	240.0	-8.1	270.9	268.1	-1.0	277.5	293.1	5.6
3	129.7	163.1	25.8	223.9	207.4	-7.4	243.6	242.8	-0.3	256.5	272.7	6.3
4	81.7	96.9	18.7	171.3	158.4	-7.6	204.4	204.7	0.1	226.3	242.1	7.0
5				105.4	93.1	-11.6	154.7	154.0	-0.5	187.6	201.2	7.3
6							95.6	90.6	-5.3	141.3	150.2	6.3
7										88.1	89.0	1

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	290.7	324.2	11.5	292.4	343.9	17.6	293.8	362.5	23.4
2	282.1	315.8	11.9	285.5	336.8	18.0	288.1	356.4	23.7
3	265.6	298.9	12.6	272.1	322.6	18.5	277.0	344.2	24.3
4	241.5	273.7	13.3	252.6	301.3	19.3	260.9	326.0	25.0
5	210.6	239.9	13.9	227.4	272.8	20.0	239.9	301.6	25.7
6	173.4	197.8	14.1	196.8	237.3	20.6	214.4	271.1	26.5
7	130.3	147.2	12.9	161.3	194.6	20.6	184.7	234.6	27.0
8	82.0	88.2	7.5	121.3	144.9	19.4	151.0	191.9	27.1
9				77.1	88.0	14.1	113.7	143.2	25.9
10							73.1	88.3	20.8

Table app.6 Variation of story shear forces for o.4m beam depth

Results of 0.5m beam depth

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	148.5	229.3	54.4	218.2	256.3	17.5	221.1	280.8	27.0	223.0	303.3	36.0
2	135.1	207.2	53.3	202.9	240.0	18.3	210.0	268.1	27.7	214.6	293.1	36.6
3	108.4	163.1	50.5	174.1	207.4	19.1	188.8	242.8	28.6	198.4	272.7	37.4
4	68.3	96.9	41.9	133.2	158.4	18.9	158.4	204.7	29.2	175.0	242.1	38.3
5				81.9	93.1	13.7	119.9	154.0	28.4	145.1	201.2	38.7
6							74.1	90.6	22.1	109.3	150.2	37.5
7										68.1	89.0	31

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	224.5	324.2	44.4	225.5	343.9	52.5	240.9	362.5	50.5
2	217.9	315.8	45.0	220.2	336.8	52.9	236.2	356.4	50.9
3	205.1	298.9	45.8	209.9	322.6	53.7	227.2	344.2	51.5
4	186.5	273.7	46.7	194.8	301.3	54.6	213.9	326.0	52.4
5	162.7	239.9	47.5	175.4	272.8	55.6	196.7	301.6	53.3
6	133.9	197.8	47.7	151.8	237.3	56.3	175.8	271.1	54.2
7	100.7	147.2	46.2	124.4	194.6	56.4	151.4	234.6	54.9
8	63.4	88.2	39.1	93.6	144.9	54.8	123.8	191.9	55.0
9				59.5	88.0	47.9	93.3	143.2	53.5
10							59.9	88.3	47.4

Table app.7 Variation of story shear forces for 0.5m beam depth

Results of 0.6m beam depth

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	133.9	229.3	71.3	190.0	256.3	34.9	192.3	280.8	46.1	193.8	303.3	56.5
2	121.8	207.2	70.1	176.7	240.0	35.8	182.6	268.1	46.8	186.5	293.1	57.2
3	97.7	163.1	66.9	151.6	207.4	36.8	164.2	242.8	47.9	172.4	272.7	58.2
4	61.6	96.9	57.4	116.0	158.4	36.5	137.8	204.7	48.6	152.1	242.1	59.2
5				71.3	93.1	30.5	104.3	154.0	47.7	126.1	201.2	59.6
6							64.5	90.6	40.5	94.9	150.2	58.2
7										59.2	89.0	50

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	194.9	324.2	66.4	197.4	343.9	74.2	240.9	362.5	50.5
2	189.2	315.8	67.0	192.8	336.8	74.7	236.2	356.4	50.9
3	178.1	298.9	67.9	183.7	322.6	75.6	227.2	344.2	51.5
4	162.0	273.7	69.0	170.6	301.3	76.6	213.9	326.0	52.4
5	141.2	239.9	69.9	153.5	272.8	77.7	196.7	301.6	53.3
6	116.3	197.8	70.1	132.9	237.3	78.6	175.8	271.1	54.2
7	87.4	147.2	68.4	108.9	194.6	78.7	151.4	234.6	54.9
8	55.0	88.2	60.3	81.9	144.9	76.9	123.8	191.9	55.0
9				52.1	88.0	68.9	93.3	143.2	53.5
10							59.9	88.3	47.4

Table app.8 Variation of story shear forces for 0.6m beam depth

Results of 0.7m beam depth

	4 STORIES			**5 STORIES**			**6 STORIES**			**7 STORIES**		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	125.1	229.3	83.3	173.9	256.3	47.4	175.9	280.8	59.7	177.2	303.3	71.2
2	113.8	207.2	82.1	161.8	240.0	48.3	167.1	268.1	60.5	170.5	293.1	71.9
3	91.3	163.1	78.6	138.8	207.4	49.4	150.2	242.8	61.6	157.6	272.7	73.0
4	57.5	96.9	68.5	106.2	158.4	49.1	126.1	204.7	62.4	139.1	242.1	74.1
5				65.3	93.1	42.6	95.4	154.0	61.4	115.3	201.2	74.6
6							59.0	90.6	53.5	86.8	150.2	73.0
7										54.1	89.0	64

	8 STORIES			**9 STORIES**			**10 STORIES**		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	178.1	324.2	82.0	197.4	343.9	74.2	240.9	362.5	50.5
2	172.9	315.8	82.7	192.8	336.8	74.7	236.2	356.4	50.9
3	162.7	298.9	83.7	183.7	322.6	75.6	227.2	344.2	51.5
4	148.0	273.7	84.9	170.6	301.3	76.6	213.9	326.0	52.4
5	129.1	239.9	85.9	153.5	272.8	77.7	196.7	301.6	53.3
6	106.3	197.8	86.1	132.9	237.3	78.6	175.8	271.1	54.2
7	79.9	147.2	84.3	108.9	194.6	78.7	151.4	234.6	54.9
8	50.3	88.2	75.4	81.9	144.9	76.9	123.8	191.9	55.0
9				52.1	88.0	68.9	93.3	143.2	53.5
10							59.9	88.3	47.4

Table app.9 Variation of story shear forces for 0.6m beam depth

Results of 0.8m beam depth

	4 STORIES			5 STORIES			6 STORIES			7 STORIES		
Story	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIF**	**DDBD**	**FDB**	**% DIF**
1	119.2	229.3	92.3	163.6	256.3	56.7	165.3	280.8	69.9	166.5	303.3	82.2
2	108.5	207.2	91.0	152.2	240.0	57.7	157.0	268.1	70.7	160.2	293.1	82.9
3	87.0	163.1	87.4	130.5	207.4	58.9	141.2	242.8	71.9	148.1	272.7	84.1
4	54.8	96.9	76.8	99.9	158.4	58.6	118.5	204.7	72.8	130.7	242.1	85.3
5				61.4	93.1	51.6	89.7	154.0	71.7	108.3	201.2	85.8
6							55.4	90.6	63.3	81.6	150.2	84.2
7										50.9	89.0	75

	8 STORIES			9 STORIES			10 STORIES		
Story	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**	**DDBD**	**FDB**	**% DIFF**
1	167.3	324.2	93.8	197.4	343.9	74.2	240.9	362.5	50.5
2	162.4	315.8	94.5	192.8	336.8	74.7	236.2	356.4	50.9
3	152.9	298.9	95.6	183.7	322.6	75.6	227.2	344.2	51.5
4	139.0	273.7	96.8	170.6	301.3	76.6	213.9	326.0	52.4
5	121.3	239.9	97.9	153.5	272.8	77.7	196.7	301.6	53.3
6	99.8	197.8	98.2	132.9	237.3	78.6	175.8	271.1	54.2
7	75.0	147.2	96.2	108.9	194.6	78.7	151.4	234.6	54.9
8	47.2	88.2	86.7	81.9	144.9	76.9	123.8	191.9	55.0
9				52.1	88.0	68.9	93.3	143.2	53.5
10							59.9	88.3	47.4

Table app.9 Variation of story shear forces for 0.6m beam depth

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