ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

ASSESSMENT AND APPLICATION OF DIRECT DISPLACEMENT BASED DESIGN FOR DUAL FRAME–WALL REINFORCED CONCRETE BUILDINGS

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Structural Engineering.

By: Eskedar Kassahun

Advisor: Dr.Ing. Adil Zekaria

June, 2016

Addis Ababa, Ethiopia

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Declaration

I, the undersigned, declare that this thesis work, to the best of my knowledge and belief, is my original work, has not been presented for a degree in this or any other universities, and all sources of materials used for the thesis work have been fully acknowledged.

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Abstract

Assessment and Application of Direct Displacement Based Design for Frame –Wall Reinforced Concrete Buildings

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Addis Ababa University, 2016

In this thesis work Direct Displacement Based Design method is assessed in dual frame-wall reinforced concrete building. Traditional forced based design method compared with this method applying to different case studies. The effect of frame shear ratio, base shear shared by the frame, when DDBD design method is applied is studied in detail by taking frame base shear ratio 25%, 40% and 60%, in terms of base shear and displacement of the building.

Also the effect of presence and absence of connected link beam which directly has an effect on moment profile of the wall is studied. The effectiveness of the method has been tested by designing a set of case studies. Dual frame-wall building with story 4, 8, 12, 16 and 20 is taken as case studies.

From the study the base shear calculated using DDBD is less than FBD in all case studies considered. In low rise building, with storey 4 in this study, the base shear increases as frame shear ratio increase. However in building with higher stories the base shear decrease as frame shear ratio increases. Similarly the storey shear obtained from DDBD is less than FBD in all case studies. Considering building with and without link beam the moment profile with link beam has low moment than that beam without link beam in all case studies.

Acknowledgment

Primary I would like to thank from the bottom of my heart the almighty God who permits all to happen in his way and his mother, Saint Mary, who is always with me in each step of my work.

I wish to express my sincere appreciation to my advisor Dr. Ing.Adil Zekaria for his support, guidance and insight through the study. I am sure this work would have not been possible without his help.

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Chapter one

1. Introduction

1.1. Background

Viewed through the historical prism of the past 100 years, seismic structural design can be seen to have been in constant evolution much more so than design for other load cases or actions such as gravity, wind, traffic etc. Initially, following structural damage in the seminal earthquakes of the early 20thcentury, seismic attack was perceived in terms of simple mass proportional lateral forces, resisted by elastic structural action. In the 1940's and 50's the influence of structural period in modifying the intensity of the inertia forces started to be incorporated into structural design, but structural analysis was still based on elastic structural response. Ductility considerations were introduced in the 1960's and 70's as a consequence of the experimental and empirical evidence that well-detailed structures could survive levels of ground shaking capable of inducing inertia forces many times larger than those predicted by elastic analysis. Predicted performance came to be assessed by ultimate strength considerations, using force levels reduced from the elastic values by somewhat arbitrary force-reduction factors, that differed markedly between the design codes of different seismically-active countries. Gradually this lead to a further realization, in the 1980's and 90's that strength was important, but only in that it helped to reduce displacements or strains, which can be directly related to damage potential, and that the proper definition of structural vulnerability should hence be related to deformations, not strength.**[1]**

Most structural damage during earthquakes is caused by the failure of the surrounding soil or from strong shaking. Damage also results from surface ruptures, the failure of nearby lifelines, or the collapse of more vulnerable structures. We consider these affects secondary because they are not always present during an earthquake. Most engineered structures are designed only to prevent collapse. This is not only to save money, but also because as a structure becomes stronger it attracts larger forces. Thus, most structures are designed to have sufficient ductility to survive an earthquake. This means that elements will yield and deform but they will be strong in shear and continue to support their load during and after the earthquake. During large earthquakes the ground is jerked back and forth, causing damage to the element whose capacity is furthest below the earthquake demand. **[2]**

Nowadays there are four design philosophy types: Strength design philosophy, Capacity design philosophy, Performance based design philosophy and displacement based design philosophy (structures should be designed to achieve a specified performance level, defined by drift limits, under a specified level of seismic intensity)

Among these methods Direct Displacement-Based Design (DDBD) which has been developed over the past ten years with the aim of mitigating the deficiencies in current force-based design.

The fundamental difference from force-based design is that DDBD characterizes the structure to be designed by a single-degree-of-freedom (SDOF) representation of performance at peak displacement response, rather than by its initial elastic characteristics.**[3]**

Most building codes propose a simplified method called the equivalent lateral force (ELF) procedure or the multi-mode response spectrum method to compute design forces which assume the dynamic forces developed in a structure during an earth quake are proportional to the maximum ground acceleration and the modal characteristics of the structure. These forces are approximated as a set of equivalent lateral forces which are distributed over the height of the structure. However, the equivalent lateral force method is based on a number of assumptions which are true for regular structures “structures with uniform distribution of stiffness, strength, and mass over the height”. **[4]**

1.2. Objective

The main objective of this thesis work is to asses and apply direct displacement based design method (DDBD) for dual frame-wall reinforced concrete buildings and give brief design procedures of the method . In addition comparing this method with traditional force based design method then put final conclusion which method is more appropriate in designing of such buildings for seismic.

Specific objective of this study includes:

- introduce the newly recommended design philosophy, DDBD
- Putting this method as one method of design for seismic design of a building.
- Study the merits and also the limitation of the method

1.3. Scope of the project

This work is limited to only dual frame-wall reinforced concrete buildings in which seismic force is resisted by both frame and wall. And also the case studies considered here are regular buildings both in elevation and plan.

1.4. Organization of the study

This study is organized in to five chapters. The first chapter introduces the background, objectives and the scope of the study. The procedure of direct displacement based design for frame-wall reinforced concrete building is briefly covered in chapter three of this study. Chapter four describes application of the method for frame-wall reinforced concrete building by taking different case studies, buildings with storey 4, 8, 12, 16 and 20, is discussed and also in this chapter the result obtained from the case studies and the corresponding discussion is also covered. Finally chapter five contains the conclusion and recommendations of this study.

Chapter two

2. Literature review

2.1. General

Severity of ground shaking at a given location during an earthquake can be minor, moderate and strong. Relatively speaking, minor shaking occurs frequently; moderate shaking occasionally and strong shaking rarely. For instance, on average annually about 800 earthquakes of magnitude 5.0-5.9 occur in the world while the number is only about 18 for magnitude range 7.0-7.9. So, should we design and construct a building to resist that rare earthquake shaking that may come only once in 500 years or even once in 2000 years at the chosen project site, even though the life of the building itself may be only 50 or 100 years? Since it costs money to provide additional earthquake safety in buildings, a conflict arises: Should we do away with the design of buildings for earthquake effects? Or should we design the buildings to be “earthquake proof” wherein there is no damage during the strong but rare earthquake shaking? Clearly, the former approach can lead to a major disaster, and the second approach is too expensive. Hence, the design philosophy should lie somewhere in between these two extremes. **[5]**

Earthquake Design Philosophy

The earthquake design philosophy may be summarized as follows: (Figure 2.1):

(a) Under minor but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damaged; however building parts that do not carry load may sustain repairable damage.

(b) Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake; and

(c) Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the building should not collapse.

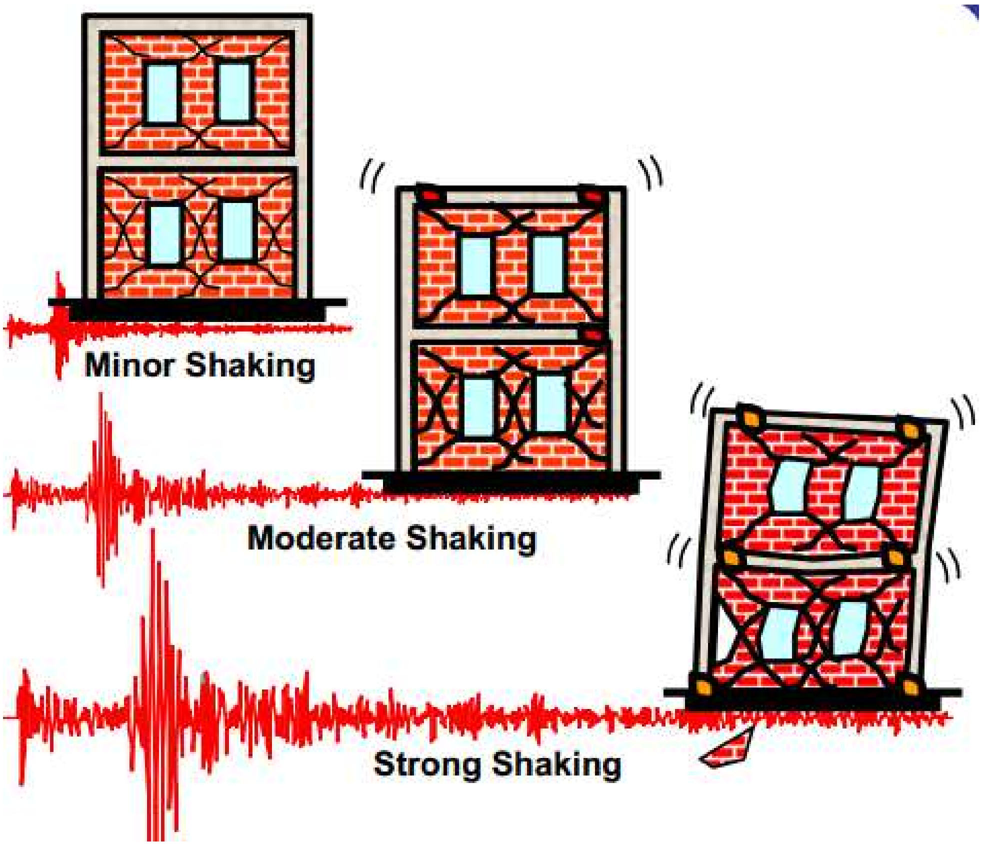


Fig.2.1: Performance objectives under different intensities of earthquake shaking

Thus, after minor shaking, the building will be fully operational within a short time and the repair costs will be small. And, after moderate shaking, the building will be operational once the repair and strengthening of the damaged main members is completed. But, after a strong earthquake, the building may become worthless for further use, but will stand so that people can be evacuated and property recovered.

The consequences of damage have to be kept in view in the design philosophy. For example, important buildings, like hospitals and fire stations, play a critical role in post-earthquake activities and must remain functional immediately after the earthquake. These structures must sustain very little damage and should be designed for a higher level of earthquake protection. Collapse of dams during earthquakes can cause flooding in the downstream reaches, which itself can be a secondary disaster. **[5]**

2.2. Evolution of Seismic Design

The concept of seismic design started in early 20th century. Discussions on deficiencies of structural systems and the resulting damage due to the 1906 San Francisco earthquake can be found in abundance in the literature. Since those days, people in seismically active countries like USA (especially the west coast), New Zealand, and Japan have been working towards forming a robust earthquake resistant design. The first active step in mitigating seismic risk was taken by the Seismological Society of America in 1910, when it identified three earthquake-related issues requiring further investigation: phenomenon of earthquakes (when, where and how they occur), the resulting ground motions, and their effect on structures. The seismic performance of then-existing structural forms had been perceived to be weak. Records show that structural engineering communities throughout the world had understood that earthquakes expose structures to lateral forces that are different from the vertical gravity loads and structures need to be specially designed to withstand earthquake induced ground shaking. A review of historical seismic design codes of different countries reveals that the definition of seismic safety has undergone gradual changes towards making it more concise, specific and performance-based. To accommodate these sophistications, several important concepts have evolved through the years. Through all these revisions of seismic design philosophies, the underlying design concept of "capacity greater than demand" has remained pivotal. Nevertheless, the meaning of the general terms "capacity" and "demand" has been interpreted differently at different stages of this journey. **[6]**

2.2.1. Strength based design

Until the 1960s, seismic design provisions were largely based on "induced stress less than allowable stress" criterion. The induced stresses were calculated by applying lateral seismic design forces which were taken as a fraction of the weight of the structure and the structure was designed such that the stresses induced by the design seismic forces when combined with gravity loads were less than the allowable stress levels. This was the "working stress method" applied in seismic design. In seismic design a truly elastic design approach is difficult to correlate with expected structural response. After all, by definition, a design earthquake is an ultimate-strength event. From the 1970s onwards, the concept of "ultimate strength design" started to appear in the seismic design codes.

This change also brought the need to take inelastic behavior into account; mainly to conduct nonlinear analysis to calculate the ultimate strength of a member. The ultimate strength based seismic design basically involved calculating the design strengths and comparing them against factored seismic design actions. **[6]**

2.2.2. Multi-objective prescriptive design

When the ultimate strength design method was being commonly used in seismic design, earthquake engineers realized that just ensuring that a designed building does not fail in an ULS earthquake is not enough and the building also needs to respond to smaller and more frequent earthquakes without causing any significant discomfort to its occupants. This led to the use of limit state design where both the serviceability and ultimate limit states would need to be satisfied. The serviceability criteria required buildings to sustain no or minimum damage (loosely referred to as remaining elastic) in frequent earthquakes (typically with 50% probability of exceedance in 50 years) and the ULS required the building not to collapse (to ensure life safety) in a design level earthquake (5% probability of exceedance in 50 years). This was a significant advancement as for the first time a building needed to satisfy more than one performance criteria. This marked also the beginning of multi objective performance based seismic design, where multiple performance criteria corresponding to different levels of earthquakes (usually specified in terms of their probability of occurrence) are checked in a precise and quantitative manner.

Structural Engineers Association of California (SEAOC) seismic design manual stated that the lateral force requirements are to produce structures that should be able to resist: a small earthquake with no damage, medium earthquake with some nonstructural and contents damage but no significant structural damage, and the largest earthquake predicted at the site with significant damage of structural components but without structural collapse. Design of structures following today's design standards, although having many different forms and equations, generally still follow the same philosophy presented in the SEAOC document mentioned above. One of the features of these guidelines is that the demand and capacity are not concisely defined; vague and subjective terms such as “moderate”, “one or more times”, “limited damage” are used. Three levels of performance against three different levels of earthquake are required, but only the largest earthquake intensity (i.e. major) is quantified as 10% probability of exceedance in 50 years. The ambiguity of the definitions can lead to wide variations in the interpretation of the code. **[6]**

2.2.3. Performance based seismic design

Until late in the 20thcentury, all design codes had prescriptive guidelines to achieve serviceability and safety. In doing so, the codes specified a common value of response parameter that the designed structures shall not exceed in limit state events. The concept of performance based design evolved when designers started realizing that such a prescriptive design was not always the most appropriate method. Different structures have different performance requirements and it is not appropriate that the same prescriptive criteria be used for designing different structures. In performance based design, the aim is to satisfy the performance requirements of a structure rather than to ensure that the response is within a prescribed limit. The performance requirements are structure specific; for a residential building severe damage in an extreme event is permitted where as any damage in a hospital or an emergency facility (even in an extreme event) is required to be minor so that the functionality of such important facilities are not interrupted after an earthquake.

Currently, many seismic design codes require structures to satisfy more than one seismic performance requirement. In such a multi-level seismic performance based design concept, in addition to verifying the prevention of collapse in an extreme earthquake, structural performances in smaller levels of earthquakes also need to be checked. Typically, required performances against three different seismic hazard levels are specified in modern performance based seismic design codes for buildings. The three seismic hazards are generally categorized as frequent earthquakes (usually with 100 years return period; 50% probability of exceedance in 50 years), design basis earthquake (DBE) with 475 years return period (i.e. 10% probability of exceedance in 50 years), and maximum considered earthquake (MCE) with 2475 years return period (i.e. 2% probability of exceedance in 50 years). The actual earthquake intensities corresponding to these hazard levels depend on the seismicity of the location of interest. **[6]**

2.3. Force Based Design (FBD)

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 or reinforced masonry structures. In order to examine these problems, it is first necessary to briefly review the force based design procedure, as currently applied in modern seismic design codes.

The sequence of operations required in force-based seismic design is summarized in Fig below. **[3]**

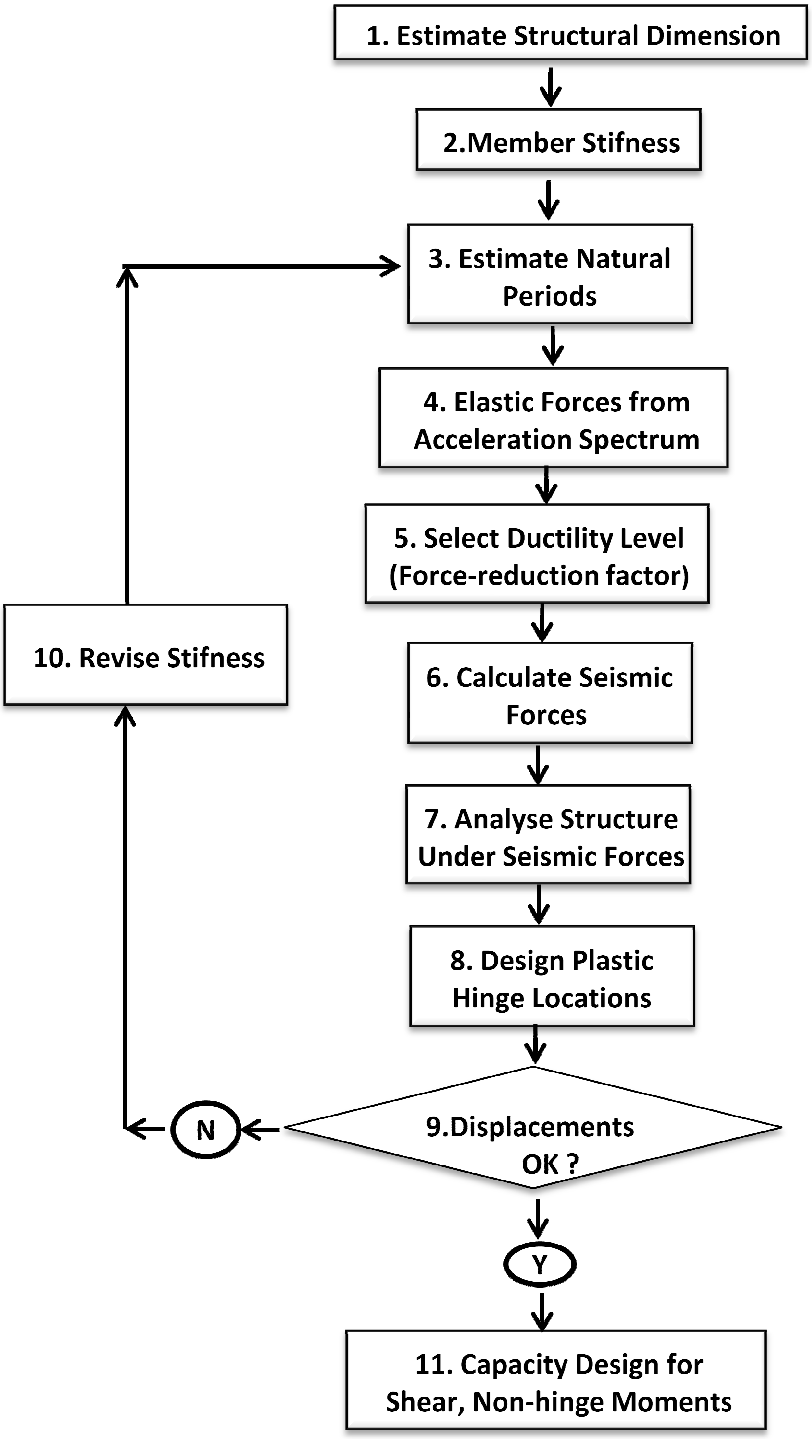


Fig. 2.2: Sequence of Operations for Force- Based Design Method

1. The structural geometry, including member sizes is estimated. In many cases the geometry may be dictated by non-seismic load considerations.
2. Member elastic stiffnesses are estimated, based on preliminary estimates of member size.
3. Based on the assumed member stiffnesses, the fundamental period or periods (multi-mode dynamic analysis) are calculated. The fundamental period is given by:

$$T = 2\pi\sqrt{\frac{m_e}{K}} \qquad (2.1)$$

Where, $\mathbf{m_e}$ is the effective seismic mass (normally taken as the total mass) and **K** is member stiffness

In some building codes a height-dependent fundamental period is specified, independent of member stiffness, mass distribution, or structural geometry. The typical form of this is given in Eq.(2.2), as:

$$T = C_1 (H_n)^{0.75} \qquad (2.2)$$

Where, $\mathbf{C_j}$ depends on the structural system, and $\mathbf{H_n}$ is the building height.

4. The design base shear $V_{base\cdot E}$ for the structure corresponding to elastic response with no allowance for ductility is given by an equation of the form

$$V_{base,E} = C_T I (g m_e)$$

(2.3)

Where, $\mathbf{C_t}$ is the basic seismic coefficient dependent on seismic intensity, soil conditions and period **T,** and **I** is an importance factor reflecting different levels of acceptable risk for different structures, and g is the acceleration of gravity.

5. The appropriate force-reduction factor $\mathbf{R_\mu}$ corresponding to the assessed ductility capacity of the structural system and material is selected. Generally $\mathbf{R_\mu}$ is specified by the design code and is not a design choice, though the designer may elect to use a lesser value than the code specified one.
6. The design base shear force is then found from

$$V_{base,E} = \frac{V_{base,E}}{R_\mu}$$

(2.4)

The base shear force is then distributed to different parts of the structure to provide the vector of applied seismic forces.

7. The structure is then analyzed under the vector of lateral seismic design forces, and the required moment capacities at potential locations of inelastic action (plastic hinges) is determined. The final design values will depend on the member stiffness.
8. Structural design of the member sections at plastic hinge locations is carried out, and the displacements under the seismic action are estimated.
9. The displacements are compared with code-specified displacement limits.
10. If the calculated displacements exceed the code limits, redesign is required. This is normally effected by increasing member sizes, to increase member stiffness.
11. If the displacements are satisfactory, the final step of the design is to determine the required strength of actions and members that are not subject to plastic hinging. The process known as *capacity design ensures* that the dependable strength in shear, and the moment capacity of sections where plastic hinging must not occur, exceed the maximum possible input corresponding to maximum feasible strength of the potential plastic hinges. Most codes include a prescriptive simplified capacity design approach. **[3]**

2.3.1. Problems with Force Based Design

1) Interdependency of Strength and Stiffness

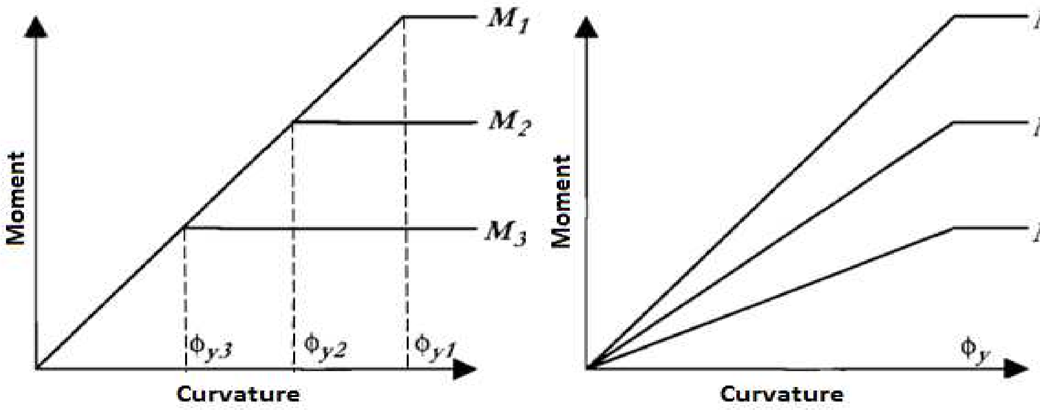
A fundamental problem with force-based design, particularly when applied to reinforced concrete and reinforced masonry structures is the selection of appropriate member stiffness. Assumptions must be made about member sizes before the design seismic forces are determined. These forces are then distributed between members in proportion to their assumed stiffness. Clearly if 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 reinforced concrete and reinforced masonry, a more important consideration is the way in which individual member stiffness is calculated. The stiffness of a component or element is sometimes based on the gross-section stiffness, and sometimes on a reduced stiffness to represent the influence of cracking. A common assumption is 50% of the gross section stiffness, though some codes specify stiffnesses that depend on member type and axial force.

Regardless of what assumption is made, the member stiffness is traditionally assumed – to be independent of strength, for a given member section. To examine this assumption, consider the flexural rigidity which can be adequately estimated from the moment- curvature relationship in accordance with the beam equation: **[3]**

$$EI = \frac{M_N}{\phi_y} \tag{2.5}$$

Where $\mathbf{M_N}$ is the nominal moment capacity, and Φ_y is the yield curvature based on the equivalent bi-linear representation of the moment-curvature curve. The assumption of constant member stiffness implies that the yield curvature is directly proportional to flexural strength, as shown in Fig. 2.3(a). Detailed analyses, and experimental evidence show that this assumption is invalid, in that stiffness is essentially proportional to length, and the yield curvature is essentially independent of strength, for a given section as shown in Fig. 2.3(b).



(a) Design Assumption
(Constant stiffness)

(b)Realistic Conditions
(Constant yield curvature)

Fig.2.3: Influence of strength on Moment-Curvature Relationship

As a consequence of these findings it is not possible to perform an accurate analysis of neither the elastic structural periods, nor of the elastic distribution of required strength throughout the structure, until the member strengths have been determined. Since the required member strengths 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 presentation. **[3]**

2) Period Calculation

Considerable variation in calculated periods can result as a consequence of different assumptions for member stiffness. When the height-dependent equations common in several codes are considered, the potential variations are exacerbated.

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.

3) 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 the hysteretic character of the inelastic system deviates significantly from elasto-plastic. Further, there has been difficulty in reaching consensus within the research community as to the appropriate definition of yield and ultimate displacements.

Clearly, with a wide choice of limit displacements, there has been considerable variation in the assessed experimental displacement ductility capacity of structures. This variation is assessed ductility capacity has, not surprisingly, been expressed in the codified force-reduction factors of different countries.

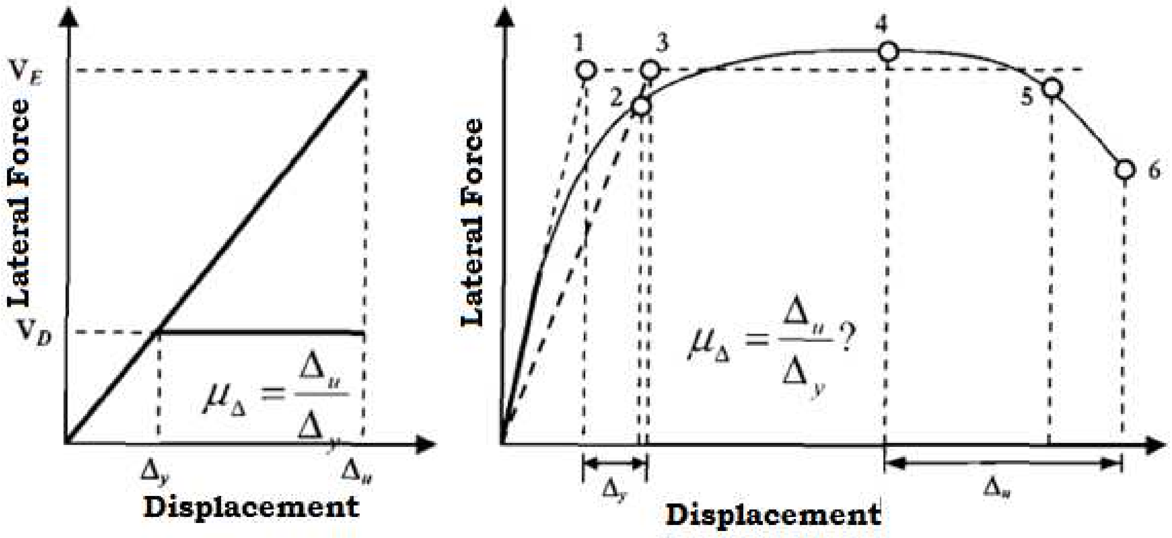
In the United States of America, force- reduction factors as high as 8.0 are permitted for reinforced concrete frames. In other countries, notably Japan and Central America, maximum force-reduction factors of about 3.0 apply for frames.

With such a wide diversity of opinion as to the appropriate level of force- reduction factor, the conclusion is inescapable that the absolute value of the strength is of relatively minor importance. **[3]**

4) Ductility of Structural Systems

A key tenet 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. Thus force-reduction factors of 6 and 4 might be assigned to reinforced concrete frame and wall structures respectively, and concrete bridges might be assigned a value of 3. Note, however, that we have already established that different codes will provide different force-reduction factors for identical systems and materials.

With reference to Fig.2.4, the yield displacement is taken to be defined by point 3, and the ultimate displacement by the lesser of displacement at point 6 or point 5,where point 5 is defined by a strength drop of 20% from the peak strength obtained.



(a) Equal Displacement Approximation

(b)Definition of Yield and Ultimate Displacement

Fig. 2.4: Defining Ductility Capacity

This assumes a bi-linear approximation to force-displacement (and to moment-curvature) response and enables direct relationships to be established between the displacement ductility and force-reduction factors. The choice of a yield displacement based on secant stiffness through the first-yield point is also based on rational considerations. A reinforced concrete structure loaded to first yield, unloaded and then reloaded, will exhibit essentially linear unloading and reloading, along the line defined by point 3. Thus once cracking occurs, the line from the origin to point 3 provided the best estimate of elastic stiffness at levels close to yield.

It is noted that for design purposes, a maximum displacement for the damage-control limit state should be reduced from the expected ultimate, or collapse displacement by a displacement-reduction factor of approximate ϕ_Δ =0.67. **[3]**

5) Relationship 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 essentially constant (the “equal displacement” approximation is assumed), while the yield displacement increases.

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. The reduction in ductility demand results in the potential for damage also being decreased, since structures are perceived to have a definable ductility demand, and the lower the ratio of ductility demand to ductility capacity, the higher is the safety. We have already identified three flaws in this reasoning:

1) Stiffness is not independent of strength;
2) The “equal displacement”, approximation is not valid; and
3) It is not possible to define a unique ductility capacity for a structural type.

More importantly, we note that the displacement capacity displays the opposite trend from that expected by the force-based argument: that is, the displacement capacity decreases as the strength increases. At a reinforcement ratio of 0.5% it is 31% higher than the reference value, while at 4% reinforcement ratio the displacement Thus, if the “equal displacement” approach was valid, we have decreased the safety by increasing the strength, and we would be better off by reducing the strength.

The elastic stiffness increases with strength, the elastic period reduces, and the displacement demand is thus also reduced. If we assume that the structural periods for all the different strength levels lie on the constant-velocity slope of the acceleration spectrum , then since the period is proportional to the inverse of the square root of the stiffness (Eq.2.1), the displacement **demand** will also be related to $\mathbf{1/k^{0.5}}$ We can then relate the ratio of displacement demand to displacement capacity, and compare with the reference value.

It will be seen that taking realistic assessment of stiffness into account, the displacement demand/capacity ratio is insensitive to the strength, with the ratio only reducing from 1.25 to 0.92 as the strength ratio increases by 400% (corresponding to the full range of reinforcement content). Clearly the reasoning behind the strength/safety argument is invalid. **[3]**

6) Structural Wall Buildings with Unequal Wall Lengths

Force-based design to requirements of existing codes will require the assumption that the design lateral forces be allocated to the walls in proportion to their elastic stiffness, with the underlying assumption that the walls will be subjected to the same displacement ductility demand. Hence the force-reduction factor is assumed to be independent of the structural configuration. The yield curvature for a given section is essentially constant, regardless of strength.

A more rational decision would be to design the walls for equal flexural reinforcement ratios, which would result in strengths proportional to the square of wall length. the code force-reduction factor for the structure will not take cognizance of the fact that the different walls must have different displacement ductility demands in the design earthquake.

7) 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.

A common example is dual wall/frame buildings (see Fig2.5). If the seismic force is distributed between the frame and the wall in proportion to their elastic stiffness, the load-carrying capacity of the frame will be unnecessarily discounted. The yield displacement of the frame will inevitably be several times larger than that of the wall, so the proportion of seismic force carried by the frame at maximum response will be larger than at first yield of the wall.

Note that the interaction between the frame and wall due to resolving the incompatibilities between their natural vertical displacement profiles will also be modified by inelastic action, and bear little resemblance to the elastic predictions. **[3]**

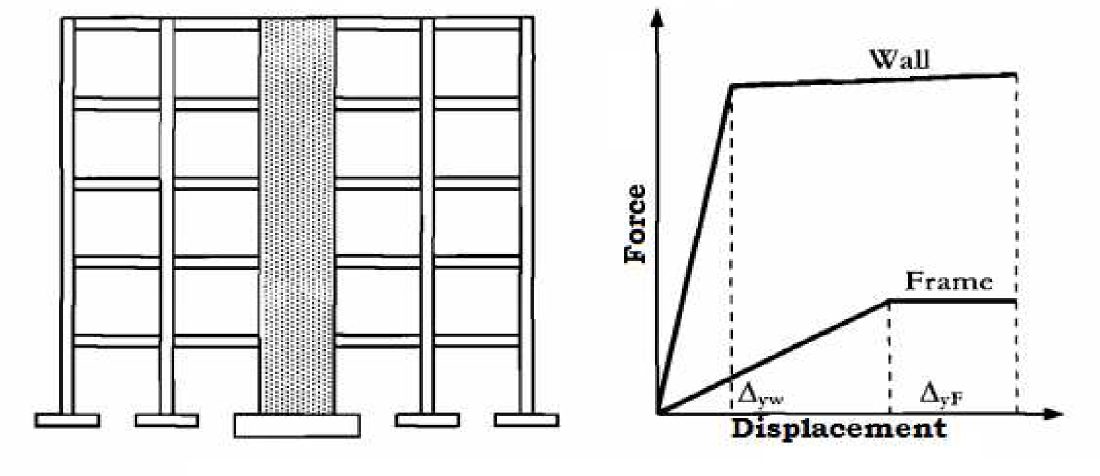


Fig.2.5: Dual Wall/Frame Building

8) Relationship between Elastic and Inelastic Displacement Demand

Force-based design requires assumptions 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 the equivalent system with the same elastic stiffness, and unlimited strength.

The equal displacement approximation is known to be non-conservative for short- period structures. As a consequence, some design codes, notably in Central and South American, and some Asian countries, apply the equal-energy approximation when determining peak displacements. The equal energy approach equates the energy absorbed by the inelastic system, on a monotonic displacement to peak response, to the energy absorbed by the equivalent elastic system with same initial stiffness.

Where codes employ inelastic design spectra design is based on specified ductility, rather than force-reduction factor, and the design spectral accelerations for short-period structures are adjusted to correct for displacement amplification.

In the United States, where until recently the dominant building code for seismic regions has been the UBC , design displacements were estimated as

$$\Delta_{\max,duct} = \Delta_y \cdot \left(\frac{3R}{8}\right) \tag{2.6}$$

Where Δ_y is the yield displacement corresponding to the reduced design forces, found from structural analysis. Since the structure is designed for a force-reduction factor of R, this would appear to imply that the displacement ductility is and the displacement of the ductile system is 3/8 of the equivalent elastic system.

However, the apparent reason behind this seemingly unconservative result is that the actual force-reduction factor was substantially lower than the design force-reduction factor, as a consequence of the design period being pegged to an unrealistic height- dependent equation. **[3]**

Generally:

- Force-based design relies on estimates of initial stiffness to determine the period and the distribution of design forces between different 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 (even if accurately known) 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.

Despite these criticisms it should be emphasized that current force-based seismic design, when combined with capacity design principles and careful detailing, generally produces safe and satisfactory designs. However, the degree of protection provided against damage under a given seismic intensity is very non-uniform from structure to structure. Thus, the concept of “uniform risk” which is implicit in the formulation of current seismic design intensity, has not been continued into the structural design. **[3]**

2.4. Direct Displacement Based Design (DDBD)

Nowadays methodologies based on forces rather than displacements, are still the most widespread in various design codes and most used in design offices to estimate the response of structures subjected to seismic action. During the 1990’s as a result of the growing interest for methods based on displacements, in particular for what regards RC structures, as they are felt more appropriate and able to overcome inherent deficiencies of traditional force-based methodologies, several displacement based seismic design methodologies emerged. One of the new seismic design methodologies was the Direct Displacement-Based Design (DDBD) developed on the base of Priestley's works. **[7]**

The use of displacement-based design is becoming accepted as the logical direction for seismic design practice. Among all displacement-based design methods only few ones can be used in codes for designing purpose. The particular form known as Direct Displacement based Design (DDBD) has been developed over the past 10 years with the aim of mitigating the deficiencies in current force-based design method. **[8]**

The fundamental difference from force-based design is that DDBD characterizes the structure to be designed by a single-degree-of-freedom (SDOF) representation of performance at peak displacement response, rather than by its initial elastic characteristics. This is based on the Substitute Structure approach. The fundamental philosophy behind the design approach is to design a structure which would achieve, rather than be bounded by, a given performance limit state under a given seismic intensity. This would result in essentially uniform-risk structures, which is philosophically compatible with the uniform-risk seismic spectra incorporated in design codes. The design procedure determines 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, and that non-ductile modes of inelastic deformation do not develop. These capacity design procedures must be calibrated to the displacement- based design approach.**[3]**

Direct Displacement-Based Design (DDBD) has now been developed to a point that engineers can start to take advantage of it in practice. Not only does Direct Displacement Based Design overcome the fundamental drawbacks of force-based design, but the methodology also offers designers a rational means of designing structural systems that may not be classified with in a traditional design code approach. **[9]**

It is widely recognized that the traditional Force-Based Design (FBD) approach cannot provide the appropriate means for implementing concepts of Performance-based Earthquake Engineering. Performance levels, indeed, are described in terms of displacements, as damage is better correlated to displacements rather than forces. As a consequence, new design approaches, based on displacements, have been recently implemented. One of such approach is the Direct Displacement-Based Design (DDBD). The fundamental goal of DDBD is to obtain a structure which will reach a target displacement profile when subjected to earthquakes consistent with a given reference response spectrum. **[10]**

2.4.1. Development of Displacement-Based Design Methods

a) Force-Based/Displacement Checked

A number of new design methods, or improvements to existing methods, have been 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 stiffnesses for deformation (if not for 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. Many modern codes require some version of this approach. Several recent design approaches have used this approach. In general, no attempt is made to achieve uniform risk of damage, or of collapse for structures designed to this approach.

Paulay has suggested that the deficiencies noted in previous sections can be eliminated within a force-based design approach. yield displacement can be determined from section and structure geometry without a prior knowledge of strength. Displacement demand, Δ_d , at least for frame buildings will normally be governed by code drift limits and the building geometry. The yield strength **V** is assumed, and hence the initial stiffness **K = V/Ay** is calculated. The elastic period is calculated from Eq. 2.1, and the elastic displacement demand will be calculated. This is compared with the code drift limit, and the strength adjusted incrementally until the elastic displacement equals the drift limit. Strength is then distributed between the different lateral-force resisting elements based on experience, rather than on elastic stiffness. This has been termed a displacement focused force-based 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 (e.g. the equal displacement approximation), and considerable experience is required of the designer. The procedure is suitable for those well versed in seismic design, but ill-suited for codification. a design approach based directly on displacements is simpler, better suited to codification and does not require assumptions to be made about elastic/inelastic displacement equivalence. **[3]**

b) Deformation-Calculation Based Design

A more refined version of the force-based/displacement-checked approach relates the detailing of critical sections (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.

Many additional variants of the approach have recently been developed. In the variant suggested by Panagiatokos and Fardis 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. **[3]**

c) 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. 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. Using state-of-the-art detailing/deformation relationships, structures with uniform risk of collapse, as well as of damage can theoretically be achieved. 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 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. These approaches also rely on existing relationships between elastic and inelastic displacement, such as the equal-displacement, or equal-energy approximations

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 a structure to achieve the specified displacement, and are hence known as ***Direct Displacement-Based Design*** **(DDBD)** methods. **[3]**

The different stiffness assumptions of the two approaches are illustrated for a typical maximum hysteretic force-displacement response in Fig. 2.6, where $\mathbf{K_i}$ and $\mathbf{K_s}$ are the initial and secant stiffness to maximum response respectively.

one of the principal problems with force-based seismic design is that reliance on initial stiffness results in illogical force distribution between different structural elements. The way in which hysteretic energy dissipation is handled also varies between the methods.

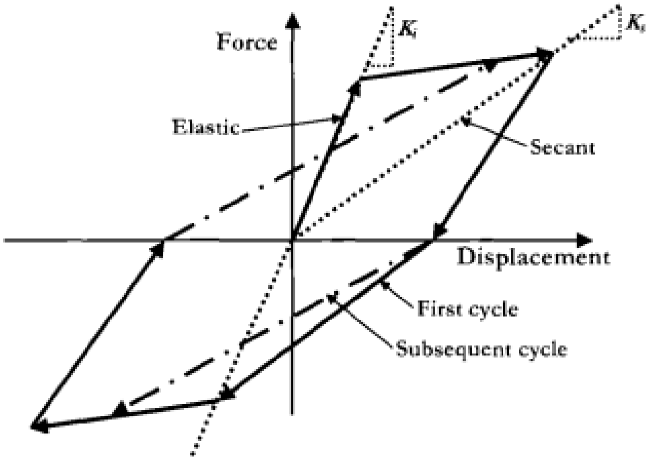


Fig. 2.6: Initial and Secant Stiffness Characterization of Hysteretic Response

Alternatively, simplified relationships between force-reduction factor and ductility that vary between equal-displacement at long periods, and equal energy at short periods are directly generated. **[3]**

d) Choice of Design Approach

Comprehensive presentation and comparison of different displacement-based designs methods is available in two recent documents. Apparent in these and other recent documents is a plethora of different nomenclature to describe the new design processes. This includes the use of "*Displacement-Based Design*", "*limit-states Design*", "*Performance Based Design*" and "*Consequence Based Design*" amongst others. In our view, all attempt generally the same goal: that of providing satisfactory displacement solutions to seismic design problems.

Direct Displacement-Based Design approach is the most intellectually satisfying, and best equipped to address the deficiencies of conventional force-based design. This approach has also been developed in rather more complete form than other methods, and has been applied to a wider category of structures. Finally, we claim that the method is simpler to apply, and better suited to incorporation in design codes.

Because of the simplicity of generation, and wider applicability, representation of hysteretic energy absorption by equivalent viscous damping will be preferred to the use of inelastic spectra. **[3]**

Chapter Three

3. DDBD of Dual Frame-Wall Reinforced Concrete Buildings

3.1 Introduction

A frame-wall system is a structural system that uses both frames and walls to resist the earth quake induced actions in the structure. The seismic behavior of a frame-wall structure considerably different from that of a wall structure and a frame structure. The wall structures have flexural behavior like a cantilevered beam and they can control the drifts at lower floor levels, while pure frame structures restrained formation at upper floors. On combining these two systems a very efficient and economical system for resisting the earth quake forces is obtained. **[11]**

Frame-wall structures are advantageous seismic-resistant structural systems, for they combine the benefits of each of their components. While frames tend to concentrate deformations at the lower storeys, walls deflect more at their top, and hence their working together leads to better controlled displacements along the whole height of the structure. **[12]**

In fact, they combine the structural advantages of frames and walls. One of these advantages is that walls provide good lateral stiffness to help control displacements over lower storeys and resist the seismic load. Even more, due to the intrinsic characteristics of functionality and service, layouts of buildings are usually required to include walls to form stair wells and lift shafts, being then convenient to use them also as earthquake resistant members.

Frames offer additional energy dissipation and are particularly effective in controlling the deformations of upper storeys. Additionally, thanks to the interaction between frames and walls, smaller shapes can be used for steel beams and columns in dual systems than in bare moment resisting frames, with consequent economic savings. Despite the fact that significant research efforts have been focused on the experimental and analytical performance of frame-wall systems, current seismic provisions include rather limited design guidelines for those structures.

A general drawback of current seismic design methodologies is that they are force-based, implying that they incorporate irrational design decisions and do not effectively control damage, as well documented by Priestly current design methods is that floor diaphragms impose displacement compatibility between frames and walls. An arbitrary assignment of ductility factors by means of force reduction factors (as suggested by the codes) does not satisfy the displacement compatibility requirement. Actually, to achieve the same displacement, walls of typical dual systems are likely to undergo a much larger ductility demand than frames because of their smaller value of yield displacement. **[13]**

Dual structures in general, and frame-wall systems in particular, are especially well suited to be designed with the Direct Displacement-Based Design method, given the known difficulties in accounting for the different deformation and ductility capacities within force-based methodologies **[12]**

An innovative seismic design procedure developed for frame wall structures. In dual frames, the system has wall dominating behavior in the lower story levels and frame dominating behavior in the upper storey. DDBD procedure developed for the moment resisting frames can be used for the design of wall-frame structures. But the most important step of the DDBD is to define the displacement profile for the dual frames. **[14]**

3.2 DDBD Procedure for Frame-wall Buildings

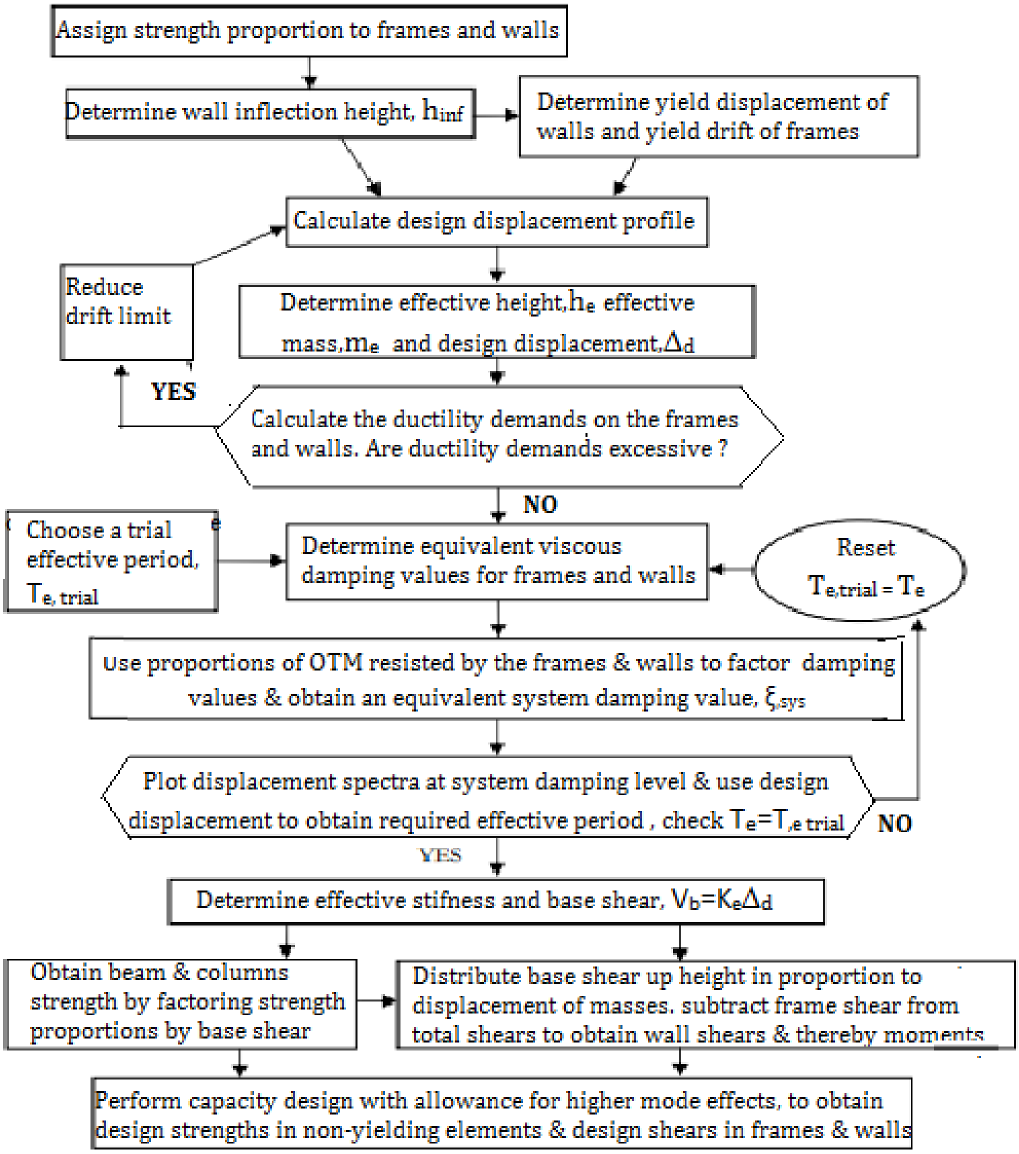
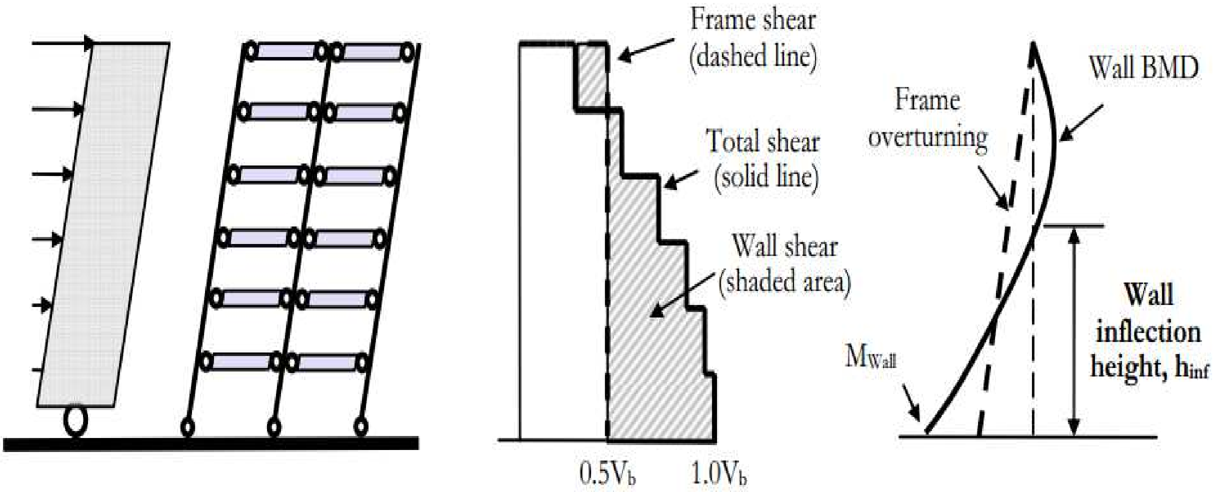


Fig.3.1. Flowchart of recommended design procedure for frames-wall structures

Step 1: Assignment of strength proportions to establish the wall inflection height

In order to develop a sufficiently accurate SDOF representation of the frame-wall structure, strength proportions are assigned at the very start of the design procedure. This involves setting the proportion of base shear or overturning resistance offered by the frames and walls, in addition to the relative strength distribution of yielding elements (beams and ground storey columns) within the frames. As mentioned above, by assigning these strength proportions the shear and moment profile in the walls can be established and this then enables determination of the inflection height. Figure 3.2 locates the inflection height for a frame-wall structure in which the frames and walls resist the total base shear in equal proportions and the frames provide a constant shear resistance over their height. The inflection height is of particular interest as it will be used to form the design displacement profile. **[15]**



(a)Beam-sway mechanism (b) Shears (c) Moments

Fig.3.2. Use of frame-wall strength proportions to locate inflection height in walls

Note that the proportions of strength assigned at this stage of the design process are related to the forces expected at formation of a 1st mode plastic mechanism. They should not be confused with the proportions of force that are expected to develop at maximum response. The maximum forces are affected by over strength and higher mode effects and are established following DBD as part of a capacity design procedure.

The storey shear above the base of the walls cannot be obtained directly from the design base shear since the walls remain elastic above the ground storey and upper storey shears will depend on the proportion of shear carried by the frames. As such, wall shears are obtained as the difference between the total shear and the frame shear as shown in Eq. (3.1). Recall that the frame storey shear can be determined since it is dependent only on the strength of the beams up the building height.

$$\frac{V_{i,wall}}{V_b} = \frac{V_{i,total}}{V_b} - \frac{V_{i,frame}}{V_b} \tag{3.1}$$

Where **V_b** is the total base shear, **V_i**, wall is the wall shear at level **i**, **$V_{i,total}$** is the total shear at level **i**, and **$V_{i,frame}$** is the frame shear at level **i**. For the purpose of establishing the inflection height, a triangular distribution of the fundamental mode inertia forces up the height of the structure is assumed. This approximation enables the total storey shear to be obtained as a function of the base shear as shown in Eq. (3.2)

$$\frac{V_{i,total}}{V_b} = 1 - \frac{i(i-1)}{n(n+1)} \tag{3.2}$$

Where **$V_{i,total}$** is the total shear at level **i, V_b** is the total base shear, and **n** is the total number of storeys in the building.

As Eq. (3.2) provides the distribution of total storey shear up the building height, the only unknown of Eq. (3.1) is the frame storey shear distribution. To obtain this shear proportion, the relative strength distribution of yielding elements within the frames is used. Although the designer is free to choose any strength distribution they prefer, it is proposed that the use of beams of equal strength up the height of the structure is advantageous for design and construction. Assuming that beam moments are carried equally by columns above and below a beam-column joint, the frame storey shear is obtained as a function of the beam strength using Eq. (3.3).

$$V_{i,frame} = \frac{(\sum M_{b,i} + \sum M_{b,i-1})}{2(h_i - h_{i-1})} = \frac{\sum M_{b,i}}{h_{col}} \tag{3.3}$$

Where **$V_{i,frame}$** is the frame shear at level **i, $M_{b,i}$** are the beam strengths at level **i**, and **h_{col}**, is the inter-storey height.

Although the beam strengths are not actually known to begin with, Eq. (3.3) is useful as it indicates that provided beams of equal strength are to be used then the frame storey shear is constant up the building height. Consequently, if 40% of the base shear is being carried by the frames, this 40% V_b will be carried up the entire height of the frame. As such, the shear proportion carried by the frame can be substituted into Eq.(3.1) and the wall shears and bending calculated, all as a function of the design base shear.

A perfectly constant shear up the height of the frame requires that the sums of the base column strengths and roof beam strengths are both equal to half the sum of the intermediate level beam strengths. If roof level beams are assigned strength equal to those on other stories, then the frame shear at roof level should be considered to be 50% greater than that at other levels. Larger base column strengths will also imply larger ground storey shears, with the column inflection height shifting above 0.5hcol.

The storey shear and consequently the moment in the walls are used to establish the inflection height in the walls, $\mathbf{h_{inf}}$, where the moment and curvature is zero. This inflection height will be used to find the displacements of the structure at yield of the walls and to develop the design displacement profile, as detailed in the next subsections.

Other important design quantities that should be obtained from the strength assignments are the proportion of overturning resisted by the frames and walls respectively. The proportions of overturning can be obtained directly from the shear profile up the height of the structures. These overturning proportions are used later in the design procedure for definition of the system damping and for adjustment of the design drift to allow for higher modes.

Step 2: Moment Profiles for Frames and Walls

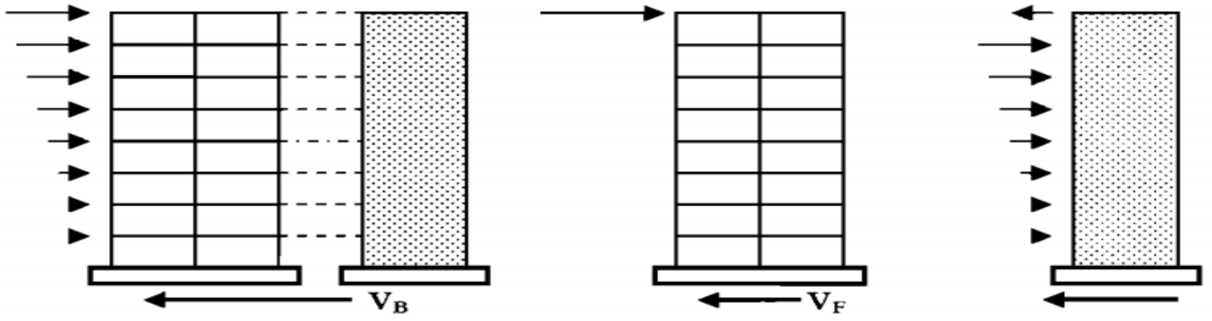
Total overturning moments resulting from the lateral forces are shown in Figure (3.3(g)), together with the vertical distribution of overturning moment for the frames. Although these are schematic, they have been based on an eight-storey structure with uniform storey heights, and a chosen frame shear ratio of $\boldsymbol{\beta_F}$ = 0.35 The vertical distribution of wall moments, shown in Figure (3.3(i)). is found by subtracting the linear distribution of frame moments from the total. For this case (and most cases) this implies a wall contraflexure point at a height $\mathbf{H_{cf}}$, as indicated in Figure (3.3(i)). This contraflexure height is an important parameter in determining the wall design displacements. **[3]**

It is useful at this stage to consider the distribution of moments induced in the frame by $\mathbf{V_F}$, as illustrated in Figure (3.4) Here we assume that all frames in a given direction are identical, and the calculations relate to the combined strength of all frames. We also assume a point of contraflexure at mid-column heights at each storey. Initially we assume that all storey heights are equal at $\mathbf{H_s}$- In each storey the sum of the column shears is: **[3]**

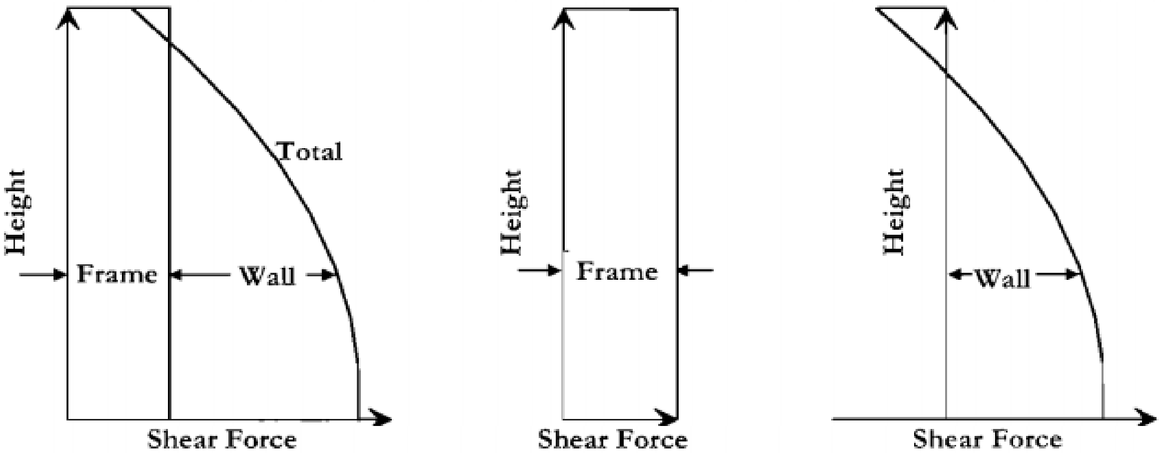
$$\sum V_C = V_1 + V_2 + V_3 = V_F \tag{3.4}$$

Consideration of moment equilibrium at the beam/column joint centres requires for constant storey height $\mathbf{H_s}$ that the sum of all beam-end moments at all levels except roof level, measured at the column centrelines must be: **[3]**

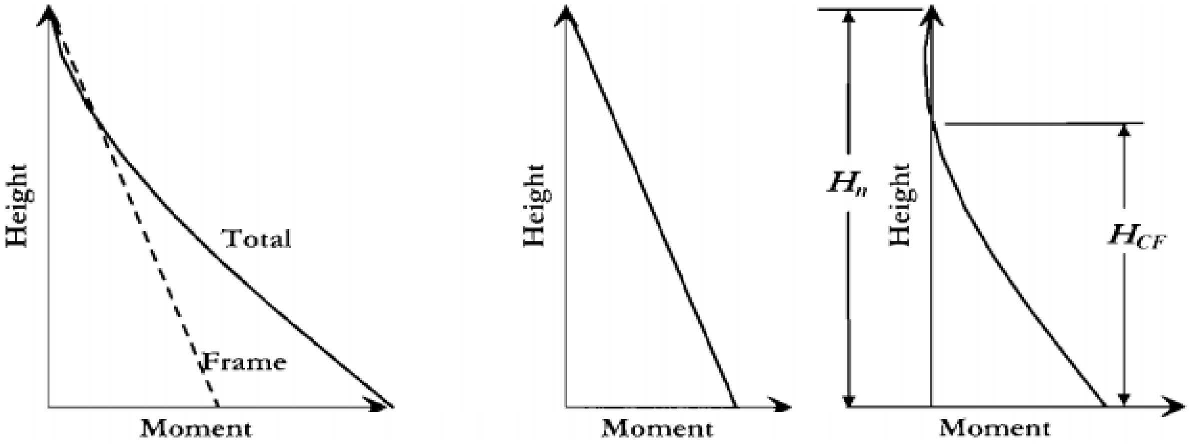
$$\sum M_{bi} = M_{i,1} + M_{i,2} + M_{i,3} + M_{i,4} = \sum V_C H_S \tag{3.5}$$



(a) Total Forces **(b) Frame Forces** **(C) Wall Forces**



(d) Total Shears **(e) Frame Shears** **(f) Wall Shears**



(g) Total Moments **(h) Frame Moments** **(i) Wall Moments**

Fig. 3.3 Distributions of Lateral Forces and OTM in a Dual Wall-Frame Building-without Link Beams

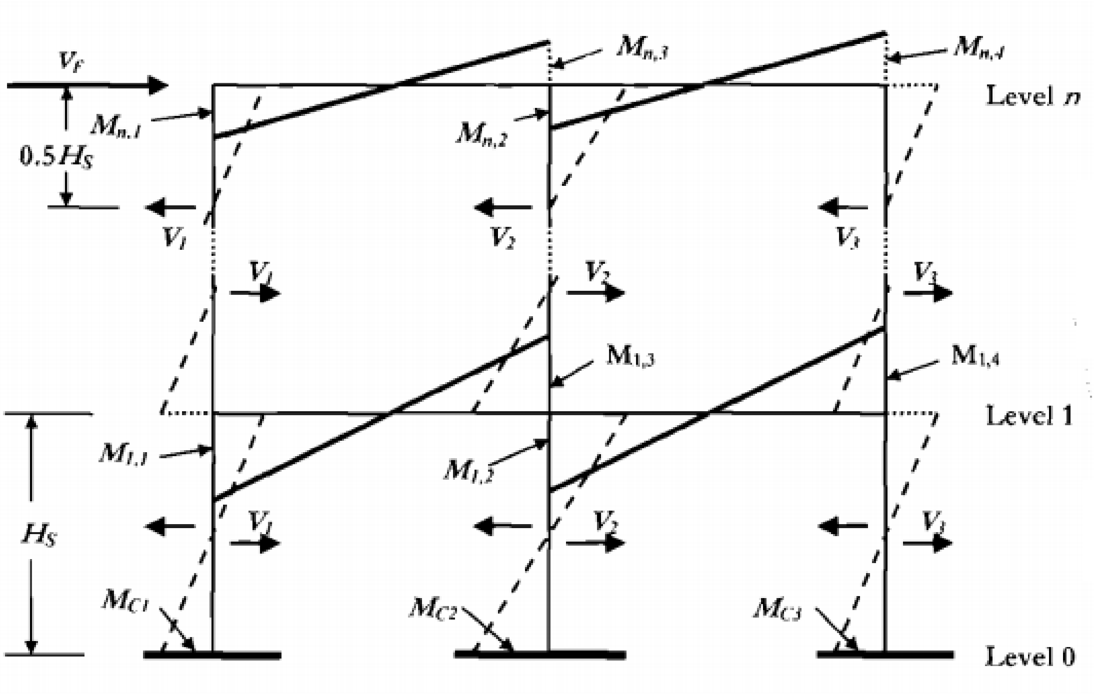


Fig. 3.4. Frame Moments and Shears for Constant Frame Shear, V_F

At roof level, the sum of beam moments should ideally be half that given by Eq. (3.5) since the moment input from the columns at the joint centres is 50% of that at other levels. If this suggestion is not adopted, the shear carried by the column in the top storey will be greater than in other storeys, unless the column flexural strength is reduced to provide a column hinge at the appropriate moment capacity. However, some excess strength of the roof level beams, especially in buildings taller than (say) 10 storeys, is unlikely to adversely affect performance. **[3]**

Column base moments are governed by

$$\sum M_C = \frac{\sum V_C H_S}{2} = 0.5 V_F H_S \quad and \quad \sum M_{Ci} = 0.5 V_i H_S \tag{3.6}$$

The strength defined by Eq. (3.6) is less than that recommended for column bases in reference **[3]** in Section 5.5.1. This is necessary to maintain the required uniformity of frame shear with height. The reason for selecting a higher base moment capacity for pure frame structures was to provide protection against a soft-storey mechanism developing in the ground floor columns. With a dual wall-frame building, the stiffness of the wall above the base plastic hinge provides adequate protection against such a soft-storey mechanism, and hence the lower moment, defined by Eq. (3.6) is acceptable. **[3]**

Note that the designer may select the way in which the total shear force **V_F** is distributed between the different columns in recognition of the different axial forces in the columns, and to optimize beam flexural design. If the storey heights vary up the height of the building, the modifications to beam design moments is obvious, substituting **$0.5(H_i + H_{i+1})$** for **H_s** in Eq. (3.5) where **H_i** is the height of the storey below the beam considered. Beam moments at roof level, and column base moments are found using the adjacent storey height in the appropriate equation. **[3]**

Step 3: Yield deformations of the walls and frames

As the walls tend to control the response of frame-wall structures, the wall yield curvature and displacements at yield are important for the development of the design displacement profile. The frame yield displacement, or yield storey drift, is also important to the design process as it is used to provide an indication of the energy absorbed through hysteretic response of the frame. The yield curvature of the walls, **ϕ_{yWall},** is firstly obtained using Eq. (3.7) **[15]**

$$\phi_{y,wall} = \frac{2\varepsilon_y}{L_w} \tag{3.7}$$

Where **ε_y** is the yield strain of the longitudinal reinforcement in the wall and **L_w** is the wall length.

The displacement profile of the structure at yield of the wall, **$\Delta_{i,y}$,** can then be established using the wall yield curvature, inflection height and storey height in accordance with Eq. (3.8a) and Eq. (3.8b).

$$\Delta_{iy} = \frac{\phi_{y,wall} h_{\text{inf}} h_i}{2} - \frac{\phi_{y,wall} h_{\text{inf}}^2}{6} \qquad for\ h_i \geq h_{\text{inf}} \tag{3.8a}$$

$$\Delta_{iy} = \frac{\phi_{y,wall} h_i^2}{2} - \frac{\phi_{y,wall} h_i^3}{6 h_{\text{inf}}} \qquad for\ h_i < h_{\text{inf}} \tag{3.8b}$$

The frame yield drift, **$\theta_{y,frame}$**, used to estimate the ductility and equivalent viscous damping of the frames, is obtained in accordance with Eq. (3.9)

$$\theta_{y,frame} = \frac{0.5 l_b \varepsilon_y}{h_b} \tag{3.9}$$

where **l_b** is the average beam length, **ε_y** is the yield strain of beam longitudinal reinforcement and h_b is the average depth of the beams at the level of interest.

Step 4: Design displacement profile and equivalent SDOF characteristics

The design displacement profile is developed using the various values obtained in the preceding subsections, together with the design storey drift, as shown in Eq. (3.10).

$$\Delta_i = \Delta_{iy} + \left[\theta_d - \frac{\phi_{y,wall} h_{\inf}}{2}\right] h_i \tag{3.10}$$

Where Δ_i is the design displacement for level **i,** $\Delta_{i,y}$ is the displacement of level i at yield of the walls, **θ_d** is the design storey drift, **$\phi_{y,wall}$** is the yield curvature of the walls, **h_{inf}** is the inflection height, and **h_i** is the height at level **i**.

Note that the design storey drift can be initially taken as the code limit for non-structural damage, reduced to allow for higher mode effects in accordance with Eq. (3.11).

$$\theta_d = \theta_{d,\lim it}\left[1 - \frac{(N-5)}{100}\left(\frac{M_{OT,frame}}{M_{OT,total}} + 0.25\right)\right] \le \theta_{d,\lim it} \tag{3.11}$$

Where **N** is the number of stories, **$M_{OT,frame}$** is the overturning resistance of the frame and **$M_{OT,total}$** is the total overturning resistance of the structure.

As mentioned earlier, the ratio of frame to total overturning resistance can be obtained in terms of the base shear using the strength assignments made at the start of the design procedure. The design drift given by Eq. (3.11) may be reduced further if it is found that inelastic demands on the structure are likely to be excessive. Alternatively, the critical value of storey drift can be determined before the design displacement profile is developed. With knowledge of the displacement profile at maximum response; Δ_i, the seismic masses; **m_i**, and storey heights; **h_i**, the equivalent SDOF design displacement; Δ_d, effective mass; **m_e**, and effective height; **h_e**, can be calculated as shown in Eq. (3.12) to Eq. (3.14) respectively. **[3]**

$$\Delta_d = \frac{\sum_{i=1}^{n}(m_i \Delta_i^2)}{\sum_{i=1}^{n}(m_i \Delta_i)} \tag{3.12}$$

$$m_e = \sum_{i=1}^{n} \frac{m_{i_i}\Delta_i}{\Delta_d} \tag{3.13}$$

$$H_e = \frac{\sum_{i=1}^{n}(m_i\Delta_i H_i)}{\sum_{i=1}^{n}(m_i\Delta_i)} \tag{3.14}$$

Step 5: Design ductility values, effective period and equivalent viscous damping

The only other substitute structure characteristic required for Direct DBD is the equivalent viscous damping. This is a function of ductility and the effective period. The ductility demands on the walls for use within this equivalent viscous damping approach should be calculated using displacement at the effective height. The wall ductility demand, **μ_{wall},** is therefore simply the design displacement divided by the yield displacement of the walls at the effective height, as shown in Eq. (3.15).

$$\mu_{wall} = \frac{\Delta_d}{\Delta_{he,y}} \tag{3.15}$$

Where $\Delta_{\mathbf{d}}$ is the design displacement (from Eq. (3.12)) and $\Delta_{\mathbf{he,y}}$ is the yield displacement of the wall at the effective height (obtained substituting the effective height into the appropriate version of Eq. (3.8)). The displacement ductility demands on the frames at each level up the height of the structure can be obtained using the storey drifts as shown in Eq. (16).

$$\mu_{frame,i} = \left(\frac{\Delta_i - \Delta_{i-1}}{h_i - h_{i-1}}\right)\frac{1}{\theta_{y,frame}} \tag{3.16}$$

Where $\Delta_{\mathbf{i}}$, $\Delta_{\mathbf{i-1}}$, $\mathbf{h_i}$, and $\mathbf{h_{i-1}}$,are the displacements and heights at level **i** and level **i-1** respectively, $\boldsymbol{\mu}_{\mathbf{frame},i}$ is the frame ductility at level **i,** and $\boldsymbol{\theta}_{\mathbf{y,frame}}$ is the yield drift of the frame (from Eq.(3.9)). When beams of equal strength are used up the height of the structure, the ductility obtained from Eq.(3.16) for each storey can be averaged to give the frame displacement ductility demand.

Before proceeding with calculations of the equivalent viscous damping, it is necessary to check that the ductility demands are sustainable. Ductility demands on frames are typically not critical as the walls tend to have smaller yield curvatures and yield displacements. For frame-wall structures in which frames are parallel to walls, ductility demands will be fairly low and can typically be detailed for relatively easily. However, when link-beams connect between frames and walls then these link-beams are likely to be subject to higher curvatures than other beams and should be checked separately. Although the wall displacement ductility demand indicated by Eq. (3.15) is appropriate for estimation of the equivalent viscous damping, it is not a good representation of the inelastic deformation that the walls must undergo. A more appropriate parameter is the wall curvature ductility, $\boldsymbol{\mu_{\phi,wall}}$, which can be obtained in accordance with Eq. (3.17).

$$\mu_{\phi,wall} = 1 + \frac{1}{L_p \phi_{y,wall}} \left(\theta_d - \frac{\phi_{y,wall} h_{\text{inf}}}{2} \right) \tag{3.17}$$

Where $\mathbf{L_p}$ is the wall plastic hinge length, $\boldsymbol{\theta_d}$ is the design storey drift, $\boldsymbol{\phi_{y,Wall}}$ is the yield curvature of the walls and $\mathbf{h_{inf}}$ is the inflection height.

Note that because the curvature ductility demand is a function of the inflection height and not the total height, inelastic deformation demands in walls of frame-wall structures will typically be larger than those in plane wall structures. The wall plastic hinge lengths to be used within Eq. (3.17) are taken as the minimum of Eq. (3.18a) and Eq. (3.18b).

$$L_P = 0.022 f_y d_b + 0.054 h_{\text{inf}} \tag{3.18a}$$

$$L_P = 0.2 L_w + 0.03 h_{\text{inf}} \tag{3.18b}$$

Where $\mathbf{f_y}$ is the yield stress and $\mathbf{d_b}$ the diameter of the longitudinal reinforcement in the wall, $\mathbf{L_w}$ is the wall length and $\mathbf{h_{inf}}$ is the inflection height.

The curvature ductility capacity of a RC wall will depend on the strain limits selected for the concrete in compression ($\boldsymbol{\varepsilon_c}$) and longitudinal reinforcement in tension ($\boldsymbol{\varepsilon_s}$). For reasonably conservative values of $\varepsilon_c = 0.018$ and $\varepsilon_s = 0.06$,. The ultimate curvature of reinforced concrete walls is well represented by Eq. (3.19).

$$\phi_u = \frac{0.072}{L_w} \tag{3.19}$$

Where **ϕ_u** is the ultimate curvature and **L_w** is the wall length. This equation was shown to be representative of ultimate curvature over a range of axial load ratios and longitudinal reinforcement contents. Combining Eq. (3.19) and Eq. (3.7), it is found that the curvature ductility capacity is approximately equal to $0.036/\varepsilon_y$. If the checks on ductility indicate that the inelastic deformations associated with the design drift will be excessive then the design drift must be reduced and the design displacement profile re-computed as discussed in the previous sub-section. If the ductility demands are sustainable then the next step in the design procedure is to compute equivalent viscous damping values. Recent work recommends that the equivalent viscous damping be computed as a function of the effective period. As this is unknown at the start of the design process, a trial value can be used and an iterative design process adopted. A reasonable estimate for the trial value of the effective period can be obtained from Eq. (3.20).

$$T_{e,trial} = \frac{N}{6}\sqrt{\mu_{sys}} \tag{3.20}$$

Where **N** is the total number of stories and **μ_{sys}** is the system ductility. Eq.(3.20) is similar in form to a code based equation that uses the height or number of storeys to estimate the initial period. The ductility term accounts for the difference between the initial and effective periods, neglecting the effect of strain hardening. Given the approximate nature of Eq.(3.20) trial effective period values may be some 30% different than the final effective period, however by using such a trial value, it will be found that convergence is attained within one or, at most, two iterations. Having set the trial effective period and established expected ductility values, the frame and wall equivalent viscous damping components are calculated using Eq. (3.21) and Eq. (3.22) respectively. **[15]**

$$\zeta_{hyst,wall} = \frac{95}{\pi}\left(1-\frac{1}{\mu_{wall}^{0.5}}-0.1r\mu_{wall}\right)\left(1+\frac{1}{(T_{e,trial}+0.85)^4}\right)\frac{1}{1.30} \tag{3.21}$$

$$\zeta_{hyst,frame} = \frac{120}{\pi}\left(1-\frac{1}{\mu_{frame}^{0.5}}-0.1r\mu_{frame}\right)\left(1+\frac{1}{(T_{e,trial}+0.85)^{4}}\right)\frac{1}{1.30} \tag{3.22}$$

Where **r** is the post-elastic stiffness coefficient, typically taken as 0.05 for new RC structures. Note that by considering the influence of the period on the damping values, it could be argued that the period-dependence of the damping values can be neglected when effective periods are greater than 1.0s, which is usually the case for frame-wall structures. The equivalent viscous damping for the frames and walls is obtained adding the elastic and hysteretic components together and then a value of damping for the equivalent SDOF system is determined using Eq. (3.23).

$$\zeta_{SDOF} = \frac{M_{wall}\cdot\zeta_{wall} + M_{OT,frame}\cdot\zeta_{frame}}{M_{wall} + M_{OT,frame}} \tag{3.23}$$

Where $\mathbf{M_{OT,frame}}$ is the overturning resistance of the frames and $\mathbf{M_{wall}}$ is the overturning resistance (flexural strength) of the walls. At this point of the design process, all of the substitute structure characteristics have been established and as such, the displacement spectrum is developed at the design level of damping. This can be done using a damping dependent scaling factor appropriate for the seismological characteristics of the design region. The Eurocode 8 **[16]** recommends that the $\boldsymbol{\eta}$ value obtained from Eq. (3.24) be used to scale the elastic spectrum to the damping level of interest.

$$\eta = \sqrt{\frac{10}{(5+\zeta_{SDOF})}} \geq 0.55 \tag{3.24}$$

Where $\boldsymbol{\xi_{SDOF}}$ is the equivalent viscous damping of the system as given by Eq. (3.23). The design displacement is then used to read off (or interpolate between known points) the required effective period, **Te**, as shown in Fig. 3.5.

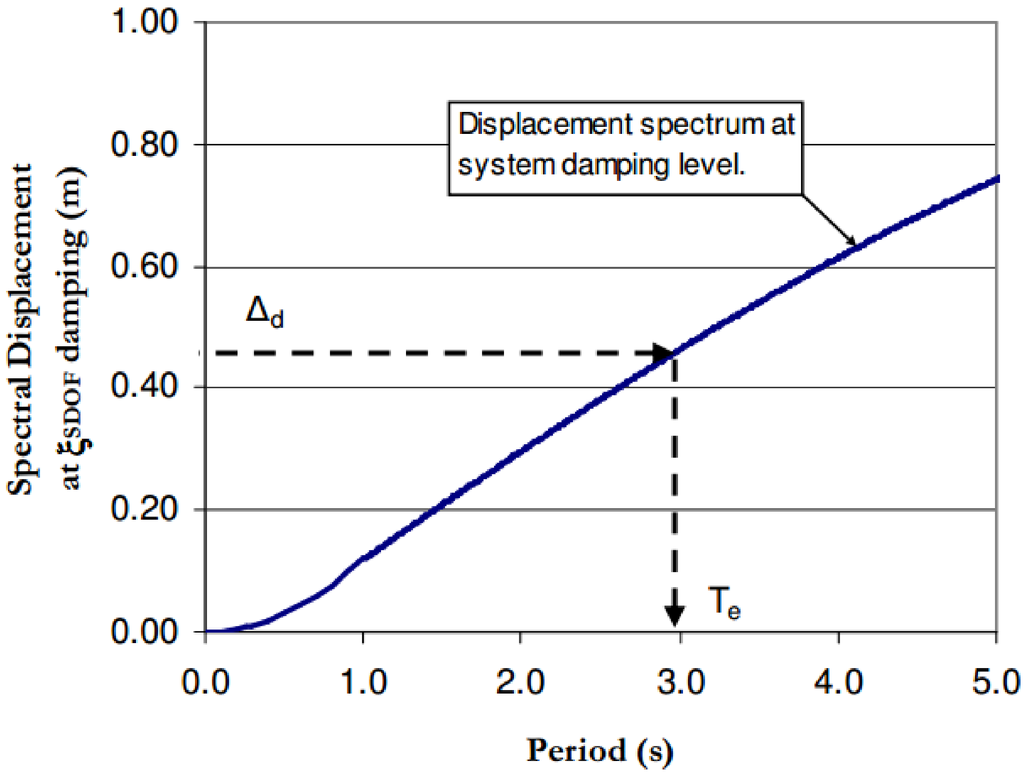


Fig. 3.5 Direct displacement based design to obtain the required effective period

The effective period obtained from the Direct DBD process illustrated in Fig.3.5 is then compared to the trial effective period value. If the period values do not match, then the period obtained from Fig.3.5 replaces the trial period and the design step is repeated. When effective periods finally match, the designer is in a position to determine the effective stiffness and design base shear as outlined next.

Step 6: Determining the design base shear and member strengths

With the effective period established, the effective stiffness, **K_e,** is determined in accordance with Eq. (3.25).

$$K_e = 4\pi^2 \frac{m_e}{T_e^2} \tag{3.25}$$

Where, **m_e** is the effective mass (from Eq. (3.13)) and **T_e** is the effective period. This effective stiffness is then multiplied by the design displacement, **Δ_d**, to obtain the base shear, **V_b**, as shown by Eq. (3.26).

$$V_b = K_e \Delta_d \tag{3.26}$$

Individual member strengths are then determined maintaining the strength proportions assigned at the start of the design process. Note however, that rather than use a triangular lateral force distribution, better results are obtained distributing the base shear up the height of the structure according to Eq. (3.27).

$$F_i = \frac{m_i \Delta_i}{\sum_{i=1}^{N} m_i \Delta_i} V_b \tag{3.27}$$

Where **F_i** is the portion of base shear applied at level **i**, **m_i** is the mass at level **i**, and **Δ_i** the displacement at level **i**. This then completes the DBD process. It is evident that there are several steps to the design procedure, however, the process is simple and does provide excellent control of displacements and storey drifts.**[15]**

3.3 Design recommendations for frame-wall structures with link-beams

Frame-wall structures with link-beams possess peculiar characteristics that must be allowed for in design. One of the first adjustments that must be made when link-beams exist, is to alter the wall moment profile associated with the 1st mode wall shears to account for the moments transferred from the link-beams. Having decided on the strength assignments for the frame-wall system, the beam strengths can be established as a fraction of the total design base shear. The sum of the beam strengths, **ΣM_b**, at a given level**, i,** is given by Eq. (3.28).

$$\sum M_{bi} = \frac{V_{i,frame} h_{col}}{\left(1 + \frac{d_{col}}{L_b}\right)} \tag{3.28}$$

Where, $\mathbf{V_{i,frame}}$ is the frame shear (known as a fraction of the total design base shear), $\mathbf{h_{col}}$ is the storey height at level **i**, $\mathbf{d_{col}}$ is the depth of the columns and $\mathbf{L_b}$ is the beam length (between column faces). The beam strengths in this equation refer to the strength at the face of the columns which have been projected to the column centerlines using the $\mathbf{d_{col}}$ on $\mathbf{L_b}$ ratio. For simplicity, these case studies neglect the effects of beam-column joints and assume that the beam strengths develop at the column centerlines.

This simplification implies that the d_{col} on L_b term drops out of Eq. (3.28). The strength of a single beam is obtained using Eq.(3.29), in which the sum of the beam moments on the floor are divided by the number of beam ends, $\mathbf{n_{bj}}$**,** that connect to beam-column joints. As the frame shear used in Eq. (3.28) is equal to the sum of the column shears, the number of beam ends that connect to the walls should not be included within $\mathbf{n_{bj}}$**.**

$$M_b = \frac{\sum M_{bi}}{n_{bj}} \tag{3.29}$$

Since the link-beams will develop the same strength as given by Eq. (3.28) at the edge of the wall, the moment transferred to the centre of the walls can be obtained from the beam moments and geometry as shown in Fig. 3.6. Substituting Eq. (3.28) into Eq. (3.29) and using the geometry and beam bending moment diagram presented in Fig. 3.6, Eq. (3.30) is obtained for the moment transferred from a link-beam to the wall centerline.

$$M_{b,wall} = V_{i,frame}\left(1+\frac{L_w}{L_b}\right)\frac{h_{col}}{n_{bj}\left(1+\frac{d_{col}}{L_b}\right)} \tag{3.30}$$

Where $\mathbf{L_w}$ is the wall length and $\mathbf{n_{bj}}$ is the number of beam ends connecting to beam column joints per link-beam.

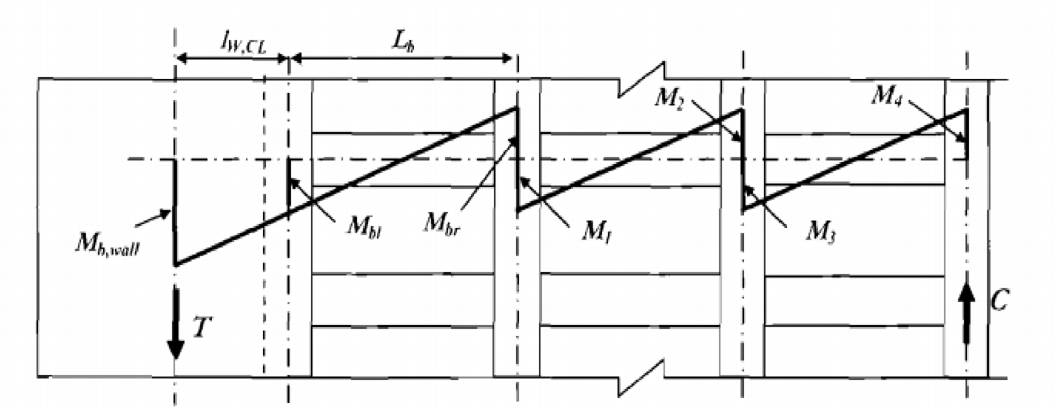


Fig.3.6. Wall moment increment from the link

The moments transferred to the wall from the link-beams are used to adjust the moment using this approach, the moment profile in the walls is known as a proportion of the design base shear. This then allows the inflection height to be determined and the design can proceed as normal. Another stage in the design process in which the inclusion of link-beams needs to be accounted for is in determination of the frame displacement ductility. As mentioned earlier, link-beams undergo larger plastic rotations than other beams at the same level. In order to estimate the ductility demands on the link beams it is worth reviewing how the ductility demands on a standard RC frame are established.

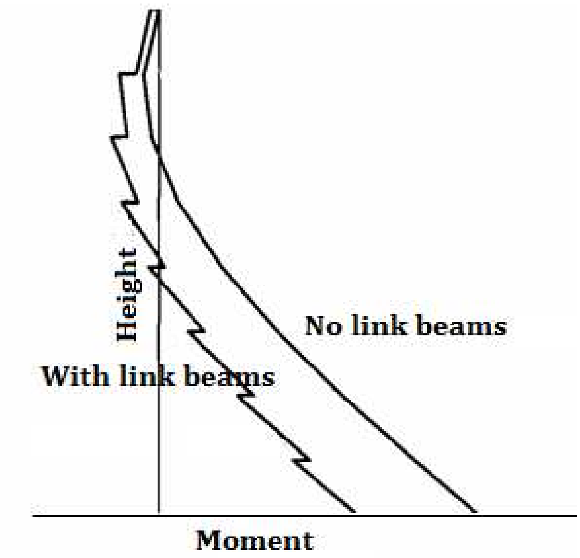


Fig.3.7. Influence of Link Beams on Wall Moment Profiles

For a standard beam-column sub-assemblage, the yield drift for design is obtained using Eq. (3.31).This is an approximate expression developed by assuming that the columns and joints add respectively an additional 40% and 25% of the displacement associated with the beams yielding in flexure, to the storey deformation. It also assumes that member shear deformations add a further 10% to the yield drift.

$$\theta_{y,beam} = 0.5\varepsilon_y \left(\frac{l_b}{h_b} \right) \tag{3.31}$$

Where **ε_y** is the yield strain of the longitudinal reinforcement in the beams, **b** is the depth of the beams and **l_b** is the beam length. For a beam-wall assemblage it could be assumed that the "column" and joint deformation contributions can be neglected. This would imply that the factor of 0.5 in the yield drift equation of Eq. (3.31) reduces to 0.31. As a link-beam is supported at one end by a stiff wall and at the other end by a column, it is apparent that an average factor of 0.4 can be used to approximate the yield drift of a link-beam, **θ_y,** link, as shown in Eq. (3.32).

$$\theta_{y,link} = 0.4\varepsilon_y \left(\frac{l_b}{h_b} \right) \tag{3.32}$$

The displacement ductility demands on the link-beams and other bays of the frame can be obtained using Eq. (3.31) and Eq. (3.32) respectively, together with the storey drift associated with the design displacement profile. A weighted average ductility value, **$\mu_{frame,i}$,** for each floor is then obtained in proportion to the number of link-beams, as shown in Eq. (3.33).

$$\mu_{frame,i} = \frac{\frac{\theta_{D,i}}{\theta_{y,link}} n_{link} + \frac{\theta_{D,i}}{\theta_{y,beam}} \left(n_b - n_{link} \right)}{n_b} \tag{3.33}$$

Where **$\theta_{D,i}$** is the storey drift associated with the design displacement profile at level **i**, **n_{link}** is the number of link-beams in the storey, and **n_b** is the total number of beams on the storey. Eq. (3.33) is valid when beams have equal length and strength. If this is not the case, it would be more appropriate to factor the ductility demands by the beam shears. Having determined the frame ductility, the design proceeds as normal with the equivalent viscous damping determined in the same manner as for the standard frame-wall structures. **[15]**

Chapter Four

4. Application of DDBD for Dual Frame-Wall Reinforced Concrete Buildings – case studies

4.1 Introduction

In order to assess the performance of the design process outlined in the previous chapter, it is implemented in the design of a series of reinforced concrete frame-wall buildings. The building is to be constructed in a region of moderate seismicity as an ordinary building with class of importance **II,** peak ground acceleration, PGA, of **0.3g** and the site is classified as soft soil. The seismic action was defined according to Eurocode 8 **[16]** and elastic acceleration response spectrum from the national annex. The elastic displacement spectrum S_{De} used for DDBD, shown in Fig. 4.1 below, is the one defined in Eurocode 8 by Eq.(4.1) The assumed corner period is 5 sec and the design drift limit, $\boldsymbol{\theta_c}$**,** is taken 0.02 as suggested by Priestly. **[3]**

$$S_{De}(T) = S_a(T)\left[\frac{T}{2\pi}\right]^2 \qquad (4.1)$$

Fig. 4.1: Design Displacement Spectrum

4.2 Problem Definition

For all of the buildings studied here, the height of first storey is **3.5m**; the other stories are each **3m** in height. There are five equal 6-meter bays along the East-West direction, and three equal 6-meter bays along the North-South direction. The lateral resistance in the North-South direction is provided by two shear walls and frames as shown in Figure 4.2. Dimensions of the slab, walls, columns and beams for all buildings are shown in Table 4.1. In addition to the self-weight of the beams and the slab, due to floor finishing and partitions a distributed dead load of 3kN/m^2, as well as an imposed live load with nominal value of 2kN/m^2 is considered. The material used in this study is concrete with grade C25 and the steel with yield strength of 400MPa.

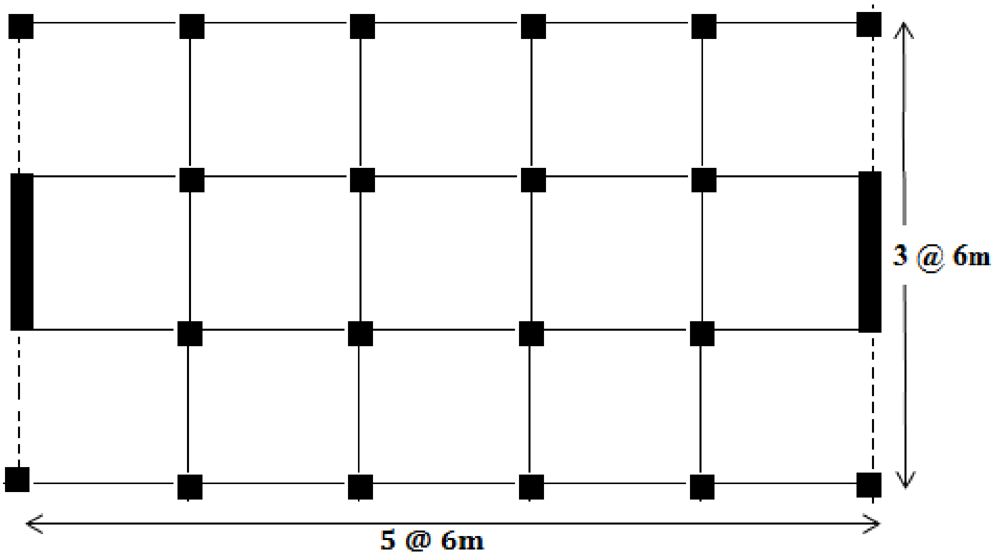


Fig. 4.2 Plan view and dimensions of the case studies -without link beams

In this chapter 8 storey frame-wall reinforced concrete building will be discussed in detail following step of DDBD design method. However buildings with stories 4, 12, 16 and 20 will be done following similar procedure and application and the results will be attached on appendix of this paper.

Table 4.1. Characteristics of the Frame-Wall Buildings

		4 storey	**8 storey**	**12 storey**	**16 storey**	**20 storey**
Slab (mm)	Thickness	150	150	150	150	150
Beams except top floor (mm)	Width	250	300	300	300	300
	Depth	300	400	500	500	500
Top floor beams (mm)	Width	250	250	250	250	250
	Depth	250	250	250	250	250
Interior columns (mm)	Width	400	500	600	700	750
	Depth	400	500	600	700	750
Exterior columns (mm)	Width	400	500	600	700	750
	Depth	400	500	600	700	750
Walls – with link beam (mm)	Length	6000	6000	6000	6000	6000
	Thickness	200	250	300	300	350
Walls- without link beam (mm)	Length	6000	10000	6000	6000	6000
	Thickness	200	250	300	300	350

4.3DDBD of 8 Storey Building

Step by step procedure of DDBD will be applied for a building with storey 8 from now ongoing. For this specific building in addition to the above described loads the lumped mass for each story is **290, 385** and **400** tonne top storey, first storey and other typical stories respectively.

Step 1: Assignment of strength proportions: Allocate 25**%** of the base shear to the frames: β_F = 0.25. To ensure beam strength is approximately 50% of that at the lower levels, the size for top floor is reduced from the rest of the floors (it is taken **250 x 250** mm). Hence the frame storey shear will be constant up the height of the building and the internal columns will carry twice the moment and shear of the external columns. [3]

Step 2: Wall Contraflexure (Inflection) Height

Here to calculate contraflexure point displacement vector is assumed to be linear with height. All necceray calculation to calculate contraflexure height (H_{CF}) is presented in table 4.2 below.

The vertical profile of wall moments calculated in column 10 of table 4.2 is calculated from the relationship:

$$M_i = M_{i+1} + V_{i+1}[H_{i+1} - H_i] \tag{4.2}$$

Table 4.2 Preliminary Calculations to Determine H_{cF} _without link beam

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall**
8	24.5	290	7105	0.174	0.174	0	0.25	-0.076	0
7	21.5	385	8277.5	0.203	0.377	0.522	0.25	0.127	**-0.228**
6	18.5	385	7122.5	0.174	0.551	1.652	0.25	0.301	**0.152**
5	15.5	385	5967.5	0.146	0.697	3.305	0.25	0.447	1.055
4	12.5	385	4812.5	0.118	0.815	5.396	0.25	0.565	2.396
3	9.5	385	3657.5	0.09	0.904	7.841	0.25	0.654	4.091
2	6.5	385	2502.5	0.061	0.966	10.554	0.25	0.716	6.054
1	3.5	400	1400	0.034	1	13.451	0.25	0.75	8.201
0	0	0	0	0	1	**16.95**	0.25	0.75	**10.826**
sum		**3000**	**40,845**	1					

From table 4.2 wall contraflexure point is between level 6 and 7 and its value can be calculated by interpolating linearly.

Thus, **H_{cf} = 19. 7m**

Step 3: Wall Yield Displacement

It is recommended that flexural strength reduction factors not be used when designing locations of intended plastic hinging. So based on this suggestion the expected yield strength of reinforcing steel is: **[3]**

$$f_{ye} = 1.1 f_y \tag{4.3}$$

Thus in this case:

f_{ye}= 1.1*400Mpa = 440Mpa

Hence the yield strain of the flexural reinforcement will be

$$\varepsilon_y = \frac{1.1 f_y}{E} = \frac{440MPa}{200{,}000MPa} = 0.0022$$

The yield curvature for the wall is estimated from Eq.(3.7):

$$\phi_y = \frac{2\varepsilon_y}{l_w}$$

$$\phi_y = \frac{2*0.0022}{6} = 7.33*10^{-4}/m$$

From Eq.3.8a & 3.8b described above:

$$For \quad H_i < 19.7m \quad \Delta_{yi} = \phi_{yw}\left[\frac{H_i^2}{2} - \frac{H_i^3}{6H_{CF}}\right] = 7.33*10^{-4}\left[\frac{H_i^2}{2} - \frac{H_i^3}{6H_{CF}}\right]$$

$$For \quad H_i > 19.7m \quad \Delta_{yi} = \phi_{yw}\left[\frac{H_{CF}H_i}{2} - \frac{H_{CF}^2}{6}\right] = 7.33*10^{-4}\left[\frac{H_{CF}H_i}{2} - \frac{H_{CF}^2}{6}\right]$$

So the corresponding vertical profile of yield displacements from the above equations is presented in table 4.3

Step 4: Design Displacement Profile

Here material strain has to be considered first then check whether drift governs.

i. Wall material strains:

The suggested limit-state curvature is given by: **[3]**

$$\phi_{ls} - l_w = 1.2\varepsilon_{s,ls} \tag{4.4}$$

Where **ϕ_{ls}** is limit curvature, **l_w** – length of the wall and **$\varepsilon_{s,ls}$** – steel limit state, strain range between $0.01 \le \varepsilon_{s,ls} \le 0.08$

Taking $\varepsilon_{s,ls} = 0.05$

$$\phi_{ls} = \frac{1.2\varepsilon_{s,ls}}{l_w} \qquad \phi_{ls} = \frac{1.2 * 0.05}{6m} \quad = 0.01/m$$

The recommended form of the equation for plastic hinge length for walls is: **[3]**

$$L_P = KH_{CF} + 0.1l_w + L_{sp} \tag{4.5}$$

Where, according to reference **[3]**

$$k = 0.2(\frac{f_u}{f_y} - 1) \le 0.08 \tag{4.6}$$

And

$$L_{sp} = 0.022 f_{ye} d_{bl} \tag{4.7}$$

From the above equations the corresponding values will be:

$$k = 0.2(1.3 - 1) \le 0.08 \quad = 0.06 \le 0.08$$

$$L_{sp} = 0.022 * 440 * 20 \quad = 193.6mm$$

Taking bar with diameter 20

Thus the plastic hinge length is:

$$L_P = 0.06*19.7 + 0.1*10m + 0.1936$$

$$= 2.376m$$

Hence check if drift limit at **H$_{CF}$** is exceeded from: **[3]**

$$\theta_{CF} = \phi_{yw}\frac{H_{CF}}{2} + (\phi_{ls} - \phi_{yw})L_p$$

$$= 7.33*10^{-4}*\frac{19.7}{2} + (0.01 - 7.33*10^{-4})*2.376 = 0.02923$$

This exceeds the drift limit of 0.02, hence code drift govern the wall design.

ii. Drift limits:

According to reference **[3]** for building with number of stories greater than 10 correction for drift application is necessary, however here the building is with story number 8 so this consideration is not necessary (it is negligible).

Thus the design profile is given by Eq. (3.10)

$$\Delta_{di} = \Delta_{yi} + (\theta_c - \phi_{yw}\frac{H_{CF}}{2})H_i \ = \Delta_{yi} + (0.02 - 7.33*10^{-4}*\frac{19.7}{2})H_i$$

$$\Delta_{di} = \Delta_{yi} + 0.013H_i$$

The design displacement profile for each story is calculated and presented in table (4.3)

Step 5: Design SDOF Displacement

The design displacement (generalized displacement coordinate) is thus given by Eq.(3.12):

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

All the necceray calculation to compute design displacement is presented in table (4.3). Hence:

$$\Delta_d = \frac{209.15}{694.27} \quad = 0.3013m$$

Step 6: Effective Height

Effective height of the substitute structure is given by: Eq. (3.14):

$$H_e = \frac{\sum_{i=1}^{n}(m_i \Delta_i H_i)}{\sum_{i=1}^{n}(m_i \Delta_i)}$$

We can find all the necceray values to calculate effective height from table (4.3)

$$Thus \qquad H_e = \frac{12{,}020.95}{694.27} = 17.31m$$

It has around ***78%*** of total height of the building.

Table 4.3 Design Displacement Information

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**
8	24.5	290	0.130	0.443	56.80	128.35	3144.51
7	21.5	385	0.108	0.383	56.35	147.29	3166.78
6	18.5	385	0.086	0.323	40.06	124.19	2297.55
5	15.5	385	0.065	0.263	26.64	101.27	1569.66
4	12.5	385	0.045	0.205	16.16	78.88	986.04
3	9.5	385	0.028	0.149	8.57	57.43	545.54
2	6.5	385	0.014	0.097	3.61	37.28	242.34
1	3.5	400	0.004	0.049	0.96	19.58	68.53
0	0	0	0	0.000	0.00	0.00	0.00
sum		**3000**			**209.15**	**694.27**	**12,020.95**

Step 7: Equivalent Damping

To obtain equivalent damping displacement ductility demands of walls and frames be evaluated first. Thus

i. **Walls**:

Yield displacement of SDOF found by substituting the value of **H_e** in Eq. (3.8a)

Since, $H_e < H_{CF}$ (17. 31 > 19.7)

$$\Delta_{yw} = \phi_{yw}\left[\frac{H_e{}^2}{2} - \frac{H_e^3}{6H_{CF}}\right]$$

$$\Delta_{yw} = 7.33*10^{-4}\left[\frac{17.31^2}{2} - \frac{17.31^3}{6*19.7}\right]$$

$$= 0.0777m$$

Ductility demands of wall obtained from Eq. (3.15)

$$\mu_w = \frac{\Delta_d}{\Delta_{yw}} = \frac{0.3013}{0.0777} = 3.88$$

Damping of the wall can be calculated as provided from reference **[3]**

$$\zeta_w = 0.05 + 0.444\left[\frac{\mu - 1}{\mu\pi}\right] \tag{4.8}$$

$$\zeta_w = 0.05 + 0.444\left[\frac{3.88 - 1}{3.88\pi}\right] = 0.1549 \approx 15.49\%$$

ii. **Frames:**

Yield drift of reinforced concrete frame is given by Eq. (3.9)

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

Where **ε_y**=0.0019, **L_b**=6m and **h_b** = 0.55m

$$\theta_y = \frac{0.5*0.0019*6}{0.55} = 0.0104$$

Thus frame ductility demand can be calculated from reference **[3]**

$$\mu_F = \frac{\Delta_d}{\theta_{yF} H_e} = \frac{0.3013}{0.0104*17.31} = 1.673$$

Damping of reinforced concrete frame is given by reference **[3]**

$$\zeta_{eq} = 0.05 + 0.565\left[\frac{\mu - 1}{\mu\pi}\right] \tag{4.9}$$

$$\zeta_{eq} = 0.05 + 0.565\left[\frac{1.673 - 1}{1.673}\right]$$

$$\zeta_{eq} = 0.1223$$

Then, equivalent elastic damping of the system can be calculated from Eq. (3.23)

$$\zeta_{sys} = \frac{\zeta_w M_{OTM,w} + \zeta_F M_{OTM,F}}{M_{OTM}}$$

The values of OTM is obtained from table (4.2)

$$\zeta_{sys} = \frac{0.1549*10.83 + 0.1223*6.13}{16.95}$$

$$= 0.1432 = 14.32\%$$

Step 8- Base Shear Force:

i. Effective Period:

To calculate effective period first elastic acceleration spectrum is calculated using the general equations defined by priestly. Then design displacement spectra are generated from the acceleration spectra assuming that the peak response is governed by the equations of steady-state sinusoidal response. Thus the relationship between displacement and acceleration can be expressed: **[3]**

$$\Delta_{(T)} = \left[\frac{T^2}{4\pi^2} S_{A(T)} g\right] \tag{4.10}$$

Finally the effective period for the required damping calculated fro the relation defined in Eurocode8 **[16]:**

$$\Delta_{(\zeta)} = \Delta_{5\%}\left[\frac{10}{5+\zeta}\right]^{\frac{1}{2}} \tag{4.11}$$

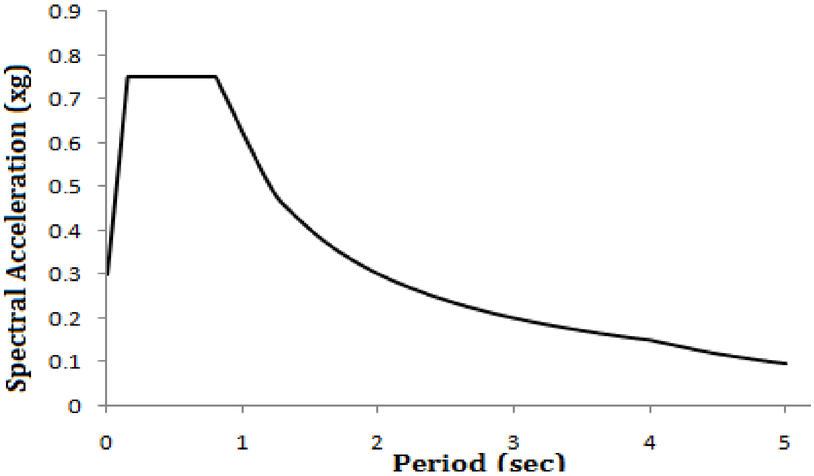


Fig.4.3 Acceleration Response Spectrum

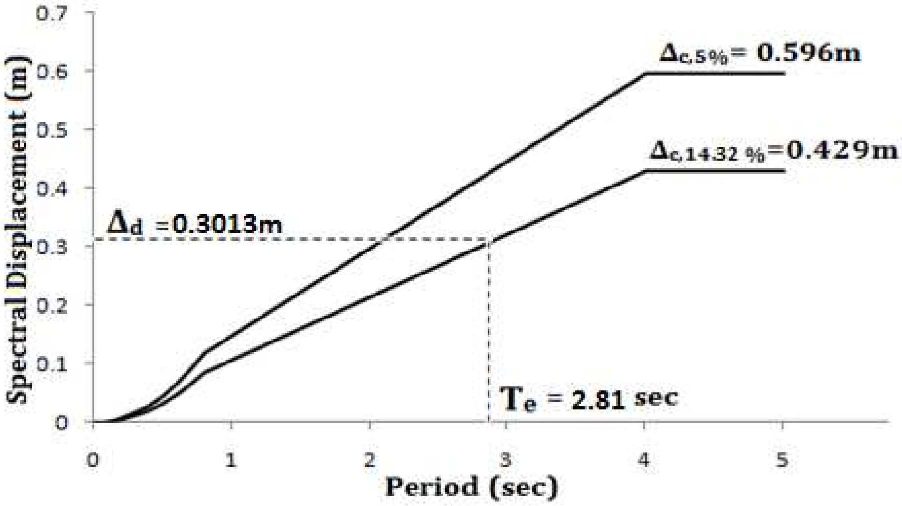


Fig. 4.4 Displacement Response Spectrum

From the equations and the graphs above the effective period of the system is **2.81 sec**.

ii. Effective Mass:

From consideration of the mass participating in the first inelastic mode of vibration, the effective system mass for the substitute structure is given by Eq. (3.13)

$$m_e = \sum_{i=1}^{n} \frac{m_{i_i} \Delta_i}{\Delta_d}$$

Taking the corresponding values from table (4.3)

$$m_e = \frac{694.27}{0.3013} = 2304.63\ tonne$$

iii. Effective Stiffness:

The effective stiffness **K_e** of the equivalent SDOF system at maximum displacement can be found by Eq. (3.25):

$$K_e = \frac{4\pi^2 m_e}{T_e^2}$$

Substituting the values

$$= \frac{4\pi^2 * 2304.63}{2.81^2} = 11.52 MN/m$$

iv. Base Shear:

The design lateral force, which is also the design base shear force, is given by Eq. (3.26):

$$V_{base} = K_e \Delta_D$$

Substituting the corresponding values

$$= 11.52 * 0.3013 = 3.471 MN$$

It is **11.79%** of total weight

Step 9: Wall Base Flexural Design:

The total wall base moment from will be, taking from table (4.2)

$$M_{w,base} = 10.83 V_{Base}$$

$$M_{w,base} = 10.83 * 3.471m \ = 37.58 MNm$$

This total wall base is shared between the two end walls which resulting in a design moment of **18.79MNm** per wall.

Step 10: Frame Beam Flexural Design

Except at the top level in each level the design is based on the assumption of equal beam strength.

Thus the total shear force to be carried by the frame is:

0.25 * 8.376 = 2.71

Then the shear force per frame is 2.71/3 = 0.902MN

From Eq.(3.5), with all beam plastic hinge having equal strength there are six potential plastic hinges per frame.

$$M_{bi} = V_F H_S / 6 = 0.902 * 3 / 6 = 0.451 KNm$$

Step 11: Column base

The design shear force for the columns will be:

Outer columns (**C_1** & **C_4**) - 0.902/6 = 0.15 MN

Inner columns (**C_2** & **C_3**) - 0.902/3 = 0.30 MN

Moment capacity of columns at the base will be

$$V_{Col} H_{01} - 0.5 \sum M_{bi} \tag{4.12}$$

Outer columns

$$\begin{aligned} M_{CB} &= 3.25m*0.15-0.5*0.451 \\ &= 0.263MNm \end{aligned}$$

Inner columns

$$\begin{aligned} M_{CB} &= 2*0.263 \\ &= 0.526MNm \end{aligned}$$

Step 12: Capacity design for walls

The system ductility can be found from the base shear forces weighted by ductility demand:

$$\mu_{sys} = \frac{\mu_w V_{w,Base} + \mu_{F,Base}}{V_{Base}}$$

$$= \frac{3.88*0.75+1.673*0.25}{1} = 3.33$$

i. Mid-height moment

With an elastic fundamental period of T_i

$$T_i \approx \frac{T_e}{\sqrt{\mu}} \qquad T_i = \frac{2.81}{\sqrt{3.33}} = 1.54$$

From the following equation found from reference **[3]** with $\phi^0=1$

$$C_{1,T} = 0.4 + 0.075 T_i \left[\frac{\mu}{\phi^0} - 1 \right] \geq 0.4 \tag{4.13}$$

$$\begin{aligned} &= 0.4 + 0.075*1.545\left[\frac{3.33}{1} - 1\right] \geq 0.4 \\ &= 0.697 > 0.4 \end{aligned}$$

Thus $M_{0.5Hn} = 0.697M_{Wb}$

ii. Wall shear force

As suggested in reference **[3]** taking the over strength factor $\phi^0 = 1.25$ so using Eq.(7.18) from reference **[3]**

$$\omega_v = 1 + \frac{\mu_{sys}}{\phi^0} C_{2,T} \tag{4.14}$$

$$Where\, C_{2,T} = 0.4 + 0.2(T_i - 0.5) \leq 1.15 \quad = 0.609 \leq 1.15$$

$$\omega_v = 1 + \frac{3.33}{1.25} * 0.609 = 2.23$$

The over strength shear demand on each of the two wall is found from reference **[3]** Eq.(6.49)

$$V^0_{base} = \phi^0 \omega_v V_{w,base} \tag{4.15}$$

$$V^0_{base} = 1.25 * 2.23 * (0.75 * 3.471) = 7.256 MN$$

According to Eq.(7.19) of reference **[3]**

$$V^0_n = 0.4 V^0_B \tag{4.16}$$

$$V^0_n = 0.4 * 7.256 = 2.9 MN$$

Step 13: Capacity design for walls

The over strength requirement are based on the beam strength, not the columns strength from reference **[3]** section 7.3.2, Eqs. (7.16) and (7.17) require that the design moments and shears for the columns (except for the column base hinges) be designed for the moments and shears resulting from the design forces amplified by a factor of 1.3ϕ° = 1.3x1.25=1.625. here the column base has a moment demand that is already 48% higher than at other levels to provide the required shear force in the ground floor which is close to the capacity design enhancement factor, implying that the same column size could be used up the height of the building

4.4 DDBD of 8 Storey Building connected with link beams

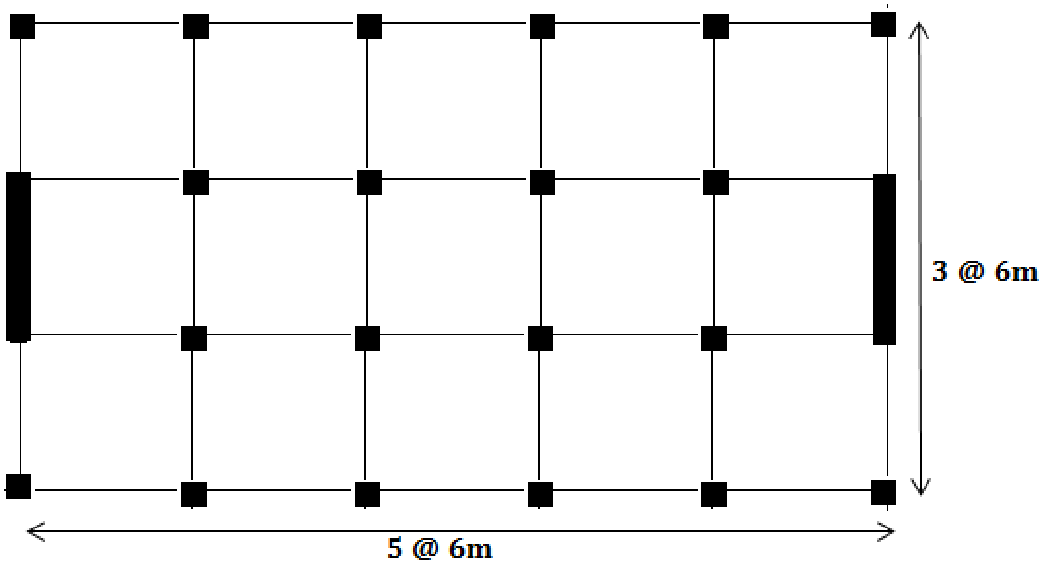


Fig. 4.5 Plan view and dimensions of the case studies - with link beams

As mentioned earlier the two main consideration here with link is: to alter the wall moment profile to account for the moments transferred from the link-beams and determination of the frame displacement ductility.

So the procedure done for frame-wall building without link beam will be repeated here except for the two considerations mentioned above.

- **Wall moment profile when frames and wall are connected by link beams**

From equation (3.30) moment transferred from a link-beam to the wall can be calculated. The values are tabulated in the table 4.4 below. Allocation of base shear for frame taken is **0.25**.

- **Wall Contraflexure Height**

From table 4.4 below wall contraflexure point is between level **5** and **6** and its value can be calculated by interpolating linearly.

Thus, $\mathbf{H_{cf}}$ = **<u>17.17</u>m**

Table 4.4 Preliminary Calculations to Determine H_{cF} –with link beams

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall**
8	24.5	290	7105	0.174	0.174	0.000	0.25	-0.076	0.000
7	21.5	385	8277.5	0.203	0.377	0.522	0.25	0.127	**-0.228**
6	18.5	385	7122.5	0.174	0.551	1.652	0.25	0.301	**0.152**
5	15.5	385	5967.5	0.146	0.697	3.305	0.25	0.447	1.055
4	12.5	385	4812.5	0.118	0.815	5.396	0.25	0.565	2.396
3	9.5	385	3657.5	0.090	0.904	7.841	0.25	0.654	4.091
2	6.5	385	2502.5	0.061	0.966	10.554	0.25	0.716	6.054
1	3.5	400	1400	0.034	1.000	13.451	0.25	0.750	8.201
0	0	0	0	0.000	1.000	**16.95**	0.25	0.750	**10.826**
sum		3000	40845	1.000					

- **Design SDOF Displacement**

Since the contraflexure height is changed the design profile and SDOF displacement also changed. The information need to calculate design displacement of frame-wall building with link beam is mentioned in table 4.5 below.

Table 4.5 Design displacement information **– with link beams**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**
8	24.5	290	0.118	0.454	59.764913	131.6504	3225.43452
7	21.5	385	0.099	0.394	59.755806	151.6772	3261.06066
6	18.5	385	0.080	0.334	42.940537	128.5772	2378.67894
5	15.5	385	0.062	0.274	28.897268	105.4772	1634.89722
4	12.5	385	0.043	0.214	17.625999	82.37724	1029.7155
3	9.5	385	0.024	0.154	9.1267304	59.27724	563.133781
2	6.5	385	0.005	0.094	3.3994616	36.17724	235.152061
1	3.5	400	-0.014	0.034	0.461499	13.58674	47.5536006
0	0	0	-0.036	0	0	0	0
sum		**3000**			**221.97**	**708.80**	**12,375.63**

So the design displacement will be: - Eq. (3.12)

$$\Delta_d = \frac{221.97}{713.9} \quad = 0.313m$$

- **Effective Height**

Effective height of the substitute structure will be: - Eq. (3.14):

$$H_e = \frac{12{,}375.63}{713.9} = 17.46m$$

- **Equivalent Damping**

- **Walls**:

$$\Delta_{yw} = 0.074m$$

$$\mu_w = 4.238$$

$$\zeta_w = 0.158 \approx 15.8\%$$

iii. Frames:

As described earlier the inclusion of link-beams needs to be accounted for in the determination of the frame displacement ductility.

From equations (3.31) and (3.32)

$$\theta_{y,beam} = 0.5\varepsilon_y \left(\frac{l_b}{h_b} \right) = \frac{0.5 * 0.0019 * 6}{0.55} = 0.0104$$

$$\theta_{y,link} = 0.4\varepsilon_y \left(\frac{l_b}{h_b} \right) = \frac{0.4 * 0.0019 * 6}{0.55} = 0.0083$$

The average ductility value of the frame will be from equation (3.33) by substituting the corresponding values.

$$\mu_{frame,i} = 1.731$$

Damping of reinforced concrete frame will be:

$$\zeta_{eq} = 0.05 + 0.565\left[\frac{1.731-1}{1.731\pi}\right] = 0.1260 \approx 12.6\%$$

Thus, equivalent elastic damping of the system can be calculated as:

$$\zeta_{sys} = \frac{0.158*8.42+0.126*8.53}{16.95}$$

$$= 0.1420 = 14.2\%$$

- **Base Shear Force**:

- **Effective Period:**

Following the same procedure to calculate the effective period of frame-wall building without link beams the effective period e is **3.14 sec.**

- **Effective Mass:**

Using Eq.(3.13) effective mass is:

$$m_e = 2263.34 tonne$$

- **Effective Stiffness**:

Using Eq.(3.25):

$$K_e = 10.19 MN/m$$

- **Base Shear**:

From Eq. (3.26):

$$V_{base} = 3.19 MN$$

- **Wall Base Flexural Design:**

The total wall base moment from will be, taking from table (4.2)

$$M_{w,base} = 8.42 * 3.19$$

$$M_{w,base} = 26.85 MNm$$

After this all the procedure used in frame-wall building without link beam is repeated here for frame-wall buildings connected with link beams.

4.5 Result and Discussion

4.5.1 Discussion on Comparison of FBD and DDBD

The theory and application of direct-displacement method is discussed clearly in the previous chapters. Here this method and force based method is compared based on storey's shear and base shear obtained from both methods taking different case studies, frame-wall building with stories, 4, 8, 12, 16 and 20.

For DDBD method the base shear shared by frame (frame shear ratio) is considered by taking three values 25%, 40% and 60%. The results obtained from each case study are given and discussed below.

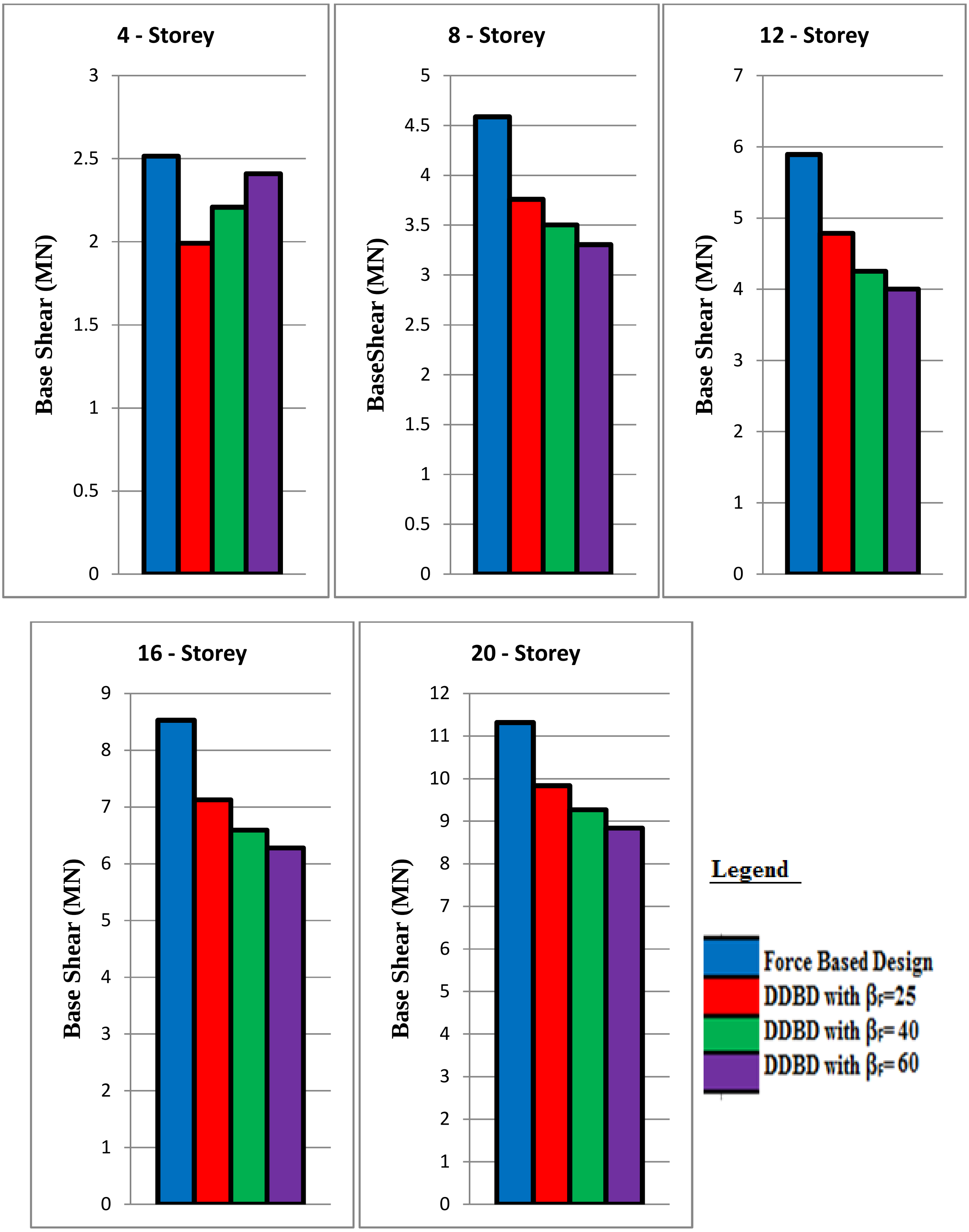


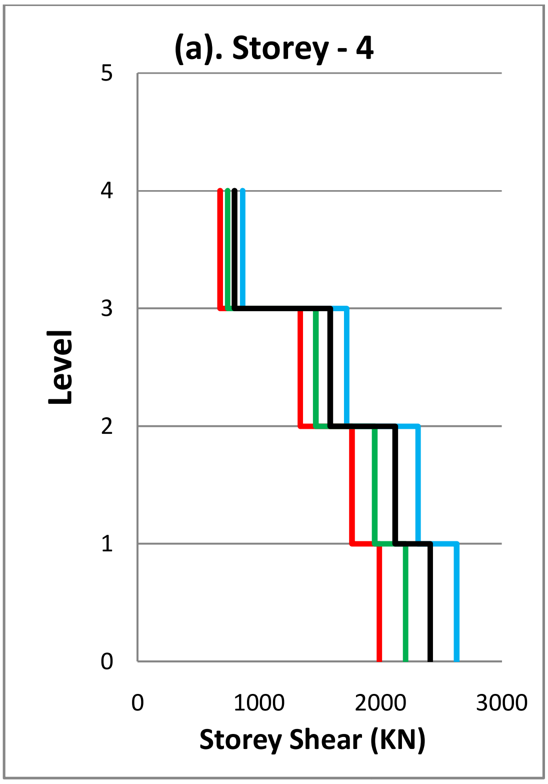
Fig. 4.6 Base Shear Variation of FBD and DDBD

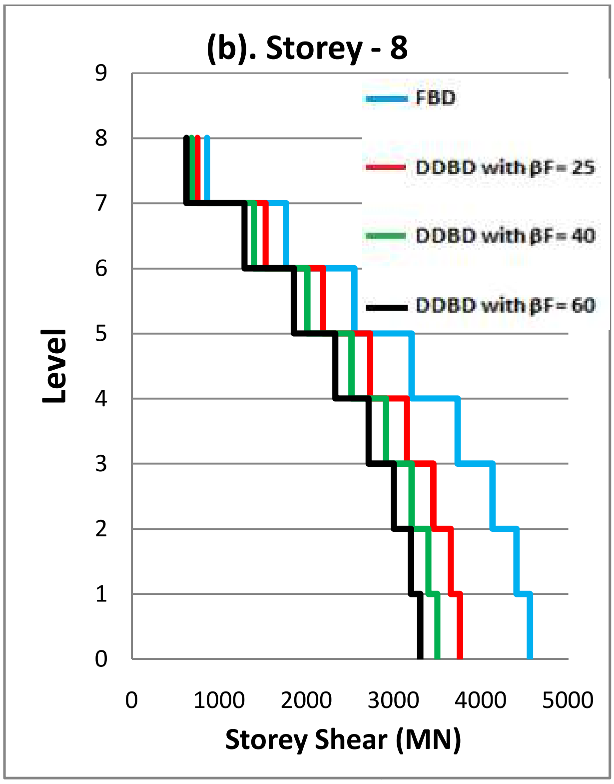
From the above results of case studies one can conclude that the base shear obtained from traditional force based method is greater than that of done by Direct Displacement Based Design method.

Looking the results obtained from DDBD with different frame shear ratio. When a building assume high frame shear ratio (β_F =60% in this case) the base shear obtained approached FBD method in a building with storey 4. However in case of building with stories 8, 12, 16 and 20 the result obtained assigning frame shear ratio 25% (which is the smaller value in this case studies) is greater than results from other DDBD but less than that of FBD.

Thus from the case studies considered above though the values of base shear is different in each three cases of DDBD (with β_F =25%, 40% and 60%) it can be concluded that using DDBD method reduce the base shear though the same structure is designed using both methods.

From the case studies above varying frame shear ratio in DDBD vary the value of base shear. In case of buildings with storey 4 base shear with β_F =60%, is somehow large which indicates the base shear carried by wall is large but the revere works in case of buildings with stories 8, 12, 16 and 20.





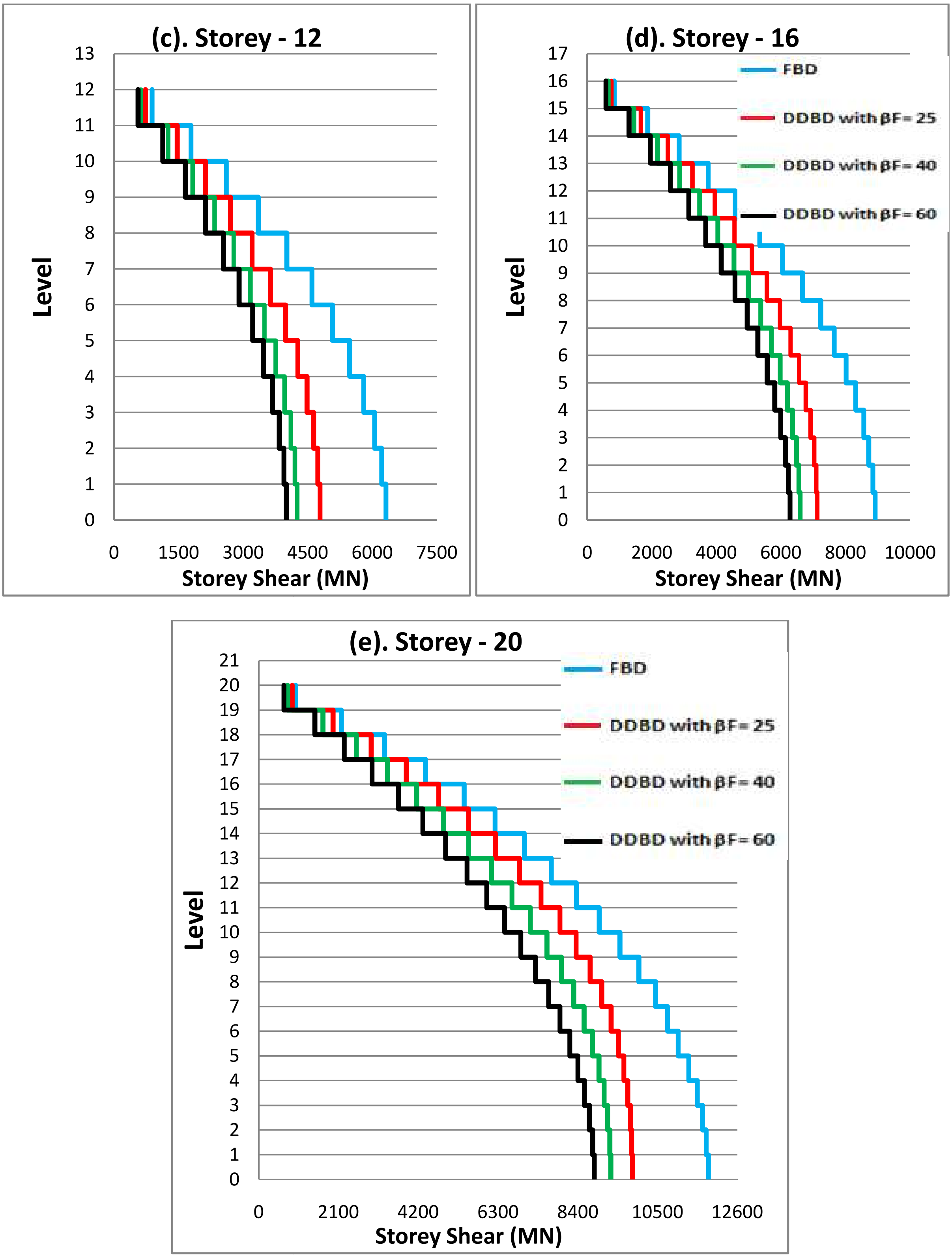


Fig.4.7 (a-e) Storey Shear variation of FBD and DDBD

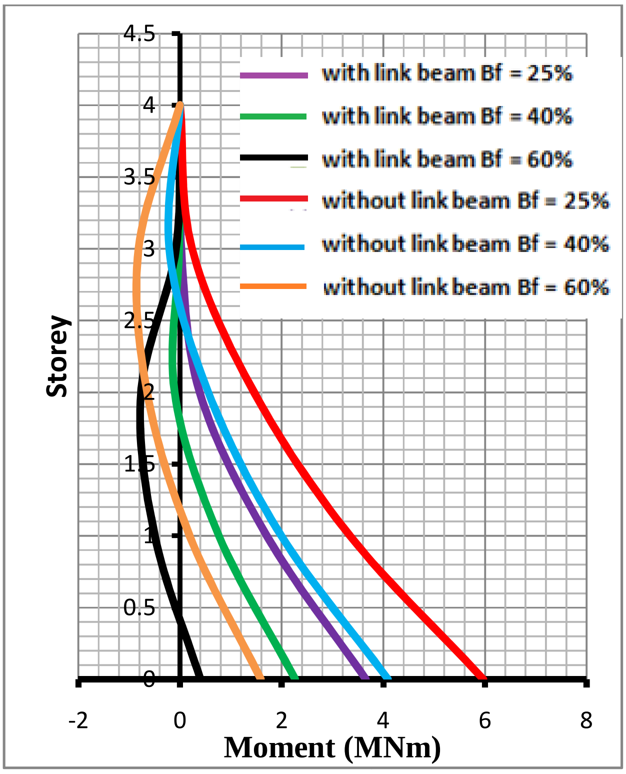
The storey shears in each level of each case study are described in the figures (4.7a) to (4.7e) above. Similar to the base shear the forces in each storey calculated using DDBD is less than the forces calculated using force based method.

For a building with storey 4 the storey shear increases as the frame shear ratio assigned increases with a value of 6% from 25 to 40 and 7% from 40 to 60. This indicates that more shears is resisted by frame in a building with low storey probably shear wall may not be required.

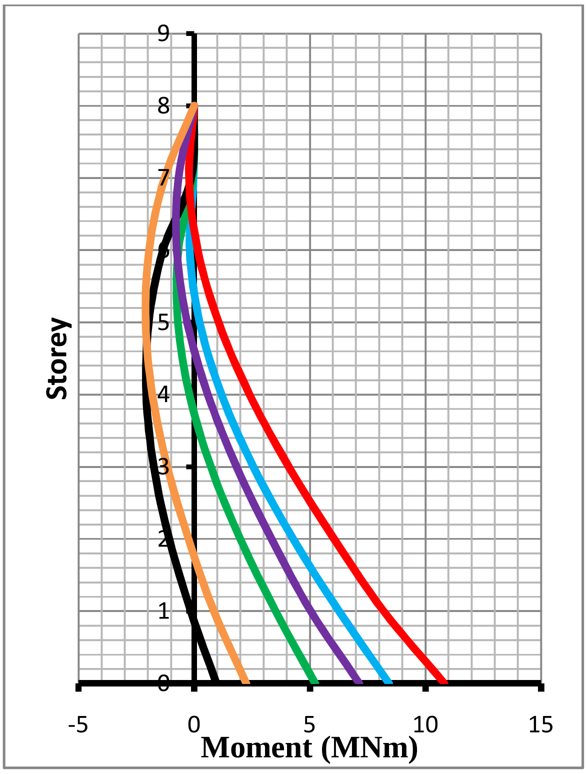
For a building with stories 8, 12, 16 and 20 unlike with storey 4 the storey shear increases as the frame shear ratio decreases from 60 to 25.

4.5.2 Discussion on Moment profile of stories – with connected link beam and without link beams

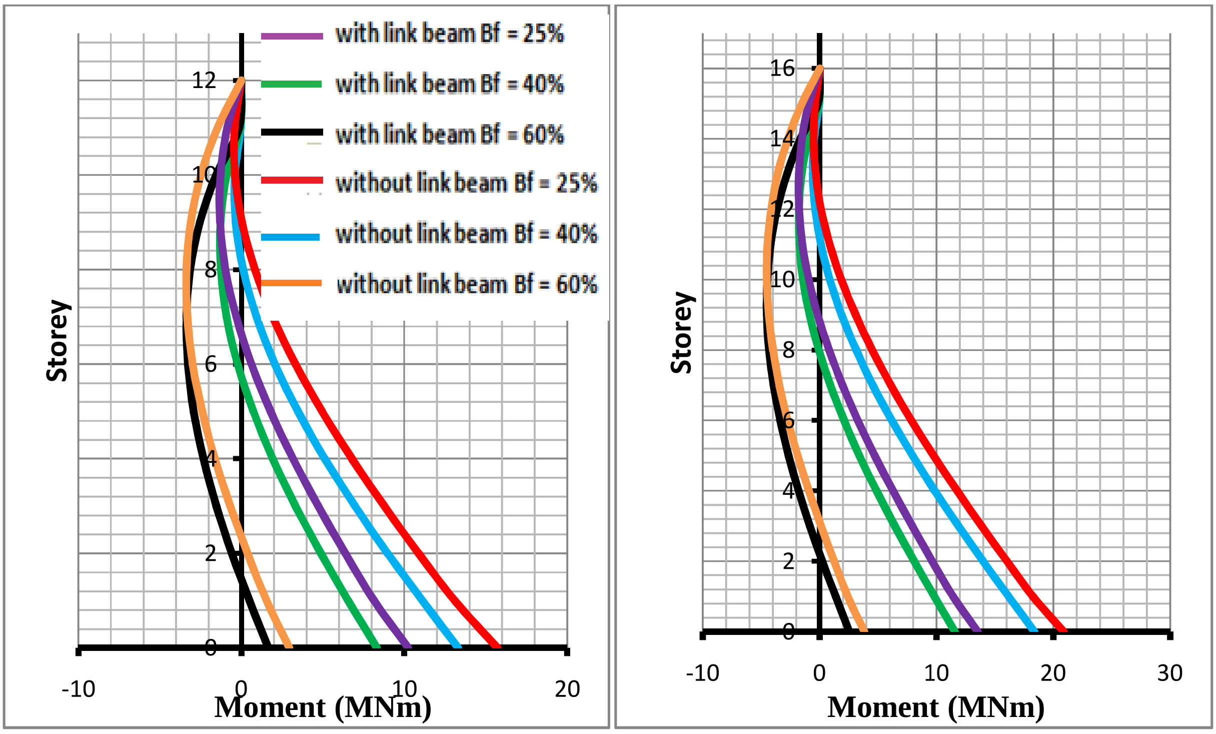
As discussed in the previous chapters' dual Frame-Wall building can be designed with or without link beams which connect the wall with frame. In the case studies above it is shown in both cases, with and without link beam, for all story types. The result is described using graphs as shown in the figures below.



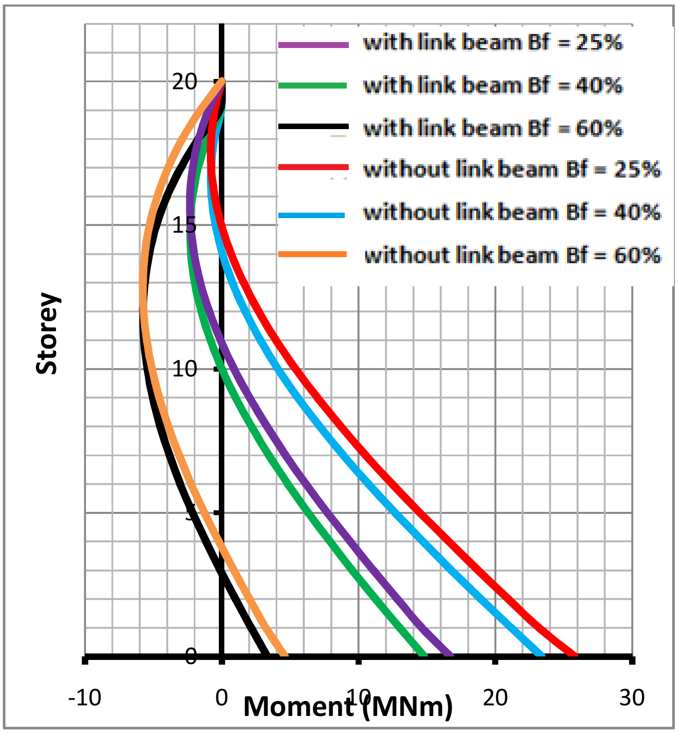
(a) 4 stories



(d) 8 stories



(b) **12 stories** **(b) 16 stories**



(e) 20 stories

Fig. 4.8(a-e) Moment profile variation of buildings with and without link beams

From the above graphs one general idea about the moment profile of the each building with different stories can be seen, which is the moment profile with connected link beam, is smaller than that of without link beams for each frame shear ratio cases.

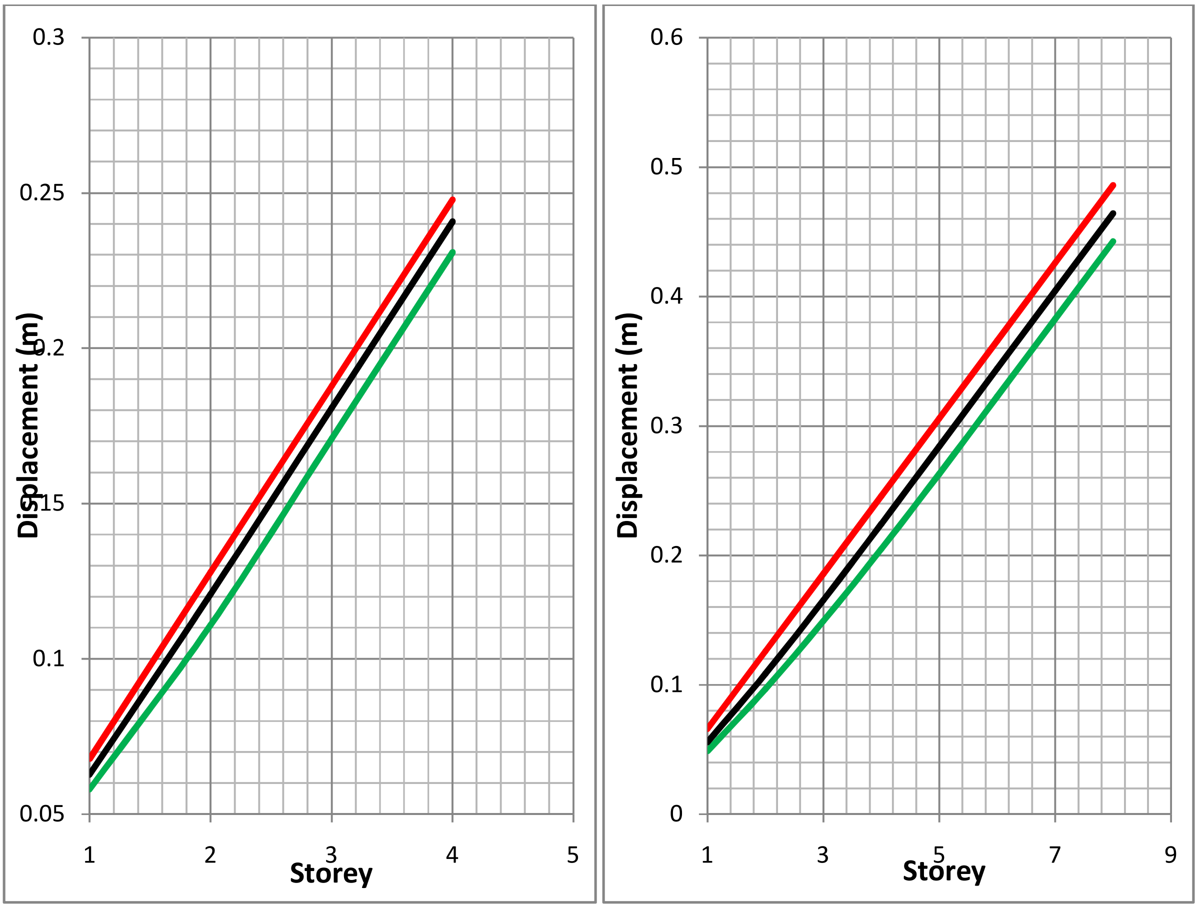
Specifically speaking in both cases, with and without link beam, as frame shear ratio increases the value of wall moment also increases up to the contraflexure point, from the top to the bottom, after that the value of wall moment decline as frame shear ratio increase.

Base overturning moment of the wall carried is less in case of 60% base shear shared by the frame as compared to the other two cases (25% & 40%) in both cases with and without link beams.

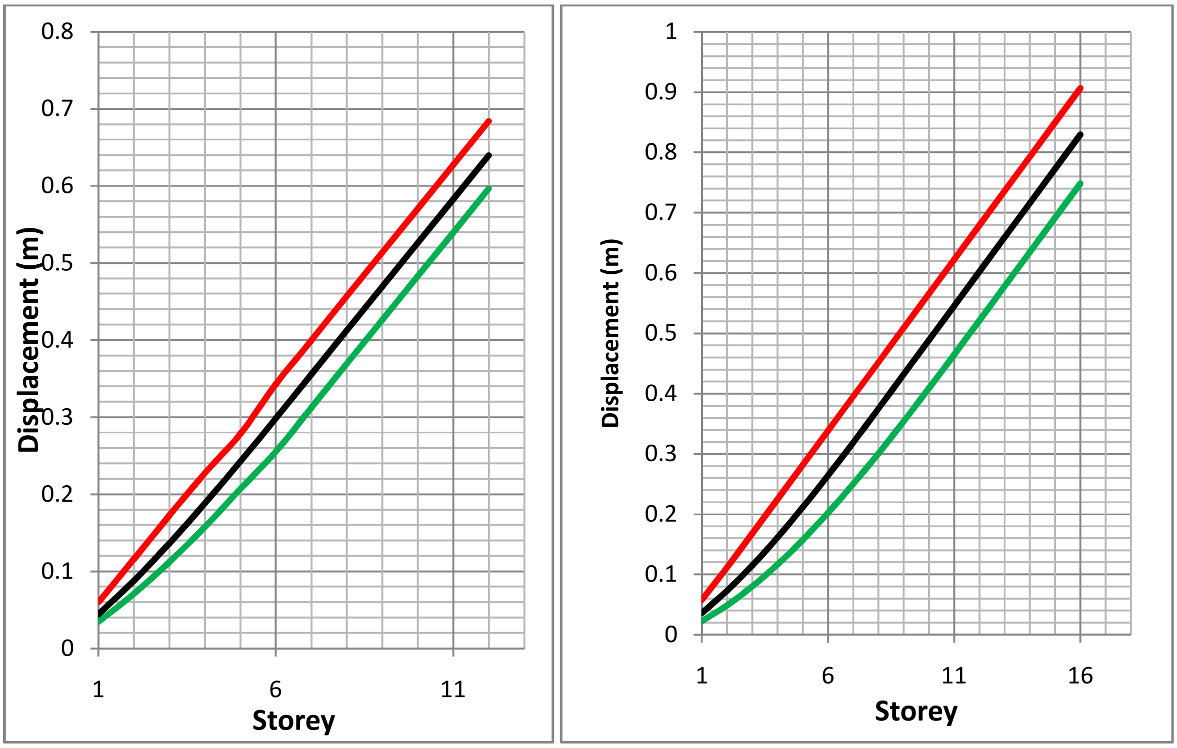
However, taking the same base shear ratio of the frame overturning moment of the wall without link beam has large OTM than that of connected with link beams.

4.5.3 Storey Displacement from DDBD – with and without link beams

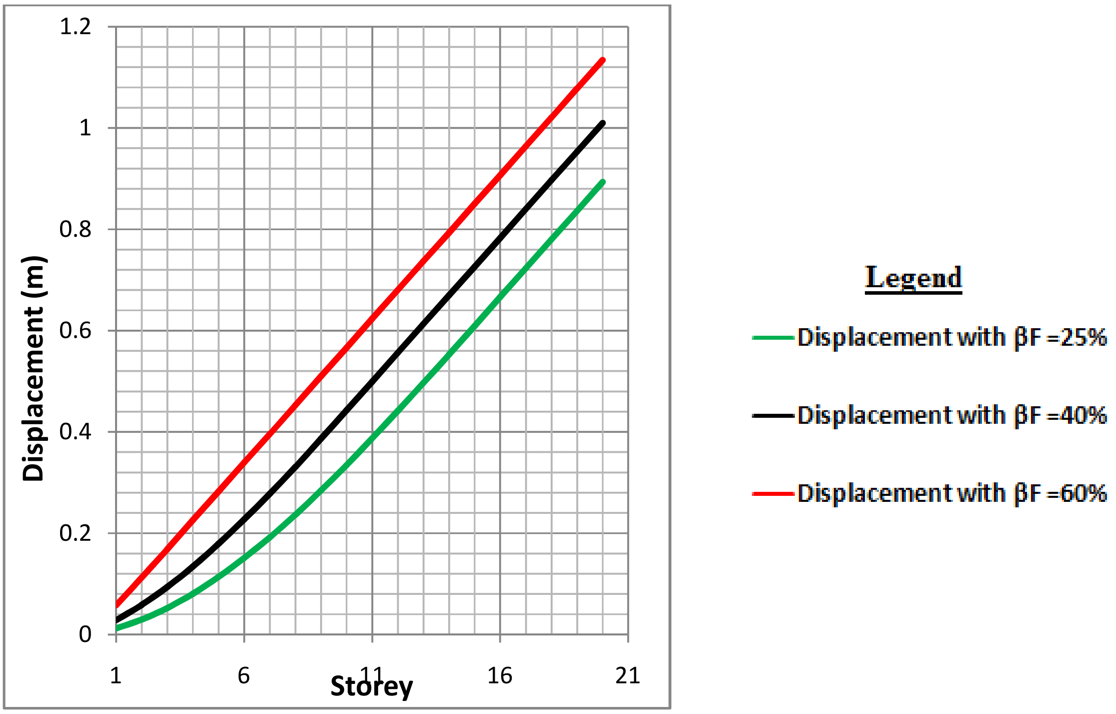
In this section DDBD method is applied for the mentioned numerical case studies to assess the effect of different frame shear ratio, 25%, 40% and 60%, on storey displacement. In addition, the effect of providing connected link beam to the wall and frame-wall without connected link beam for each case is also discussed.



(a) 4 stories **(b) 8 stories**



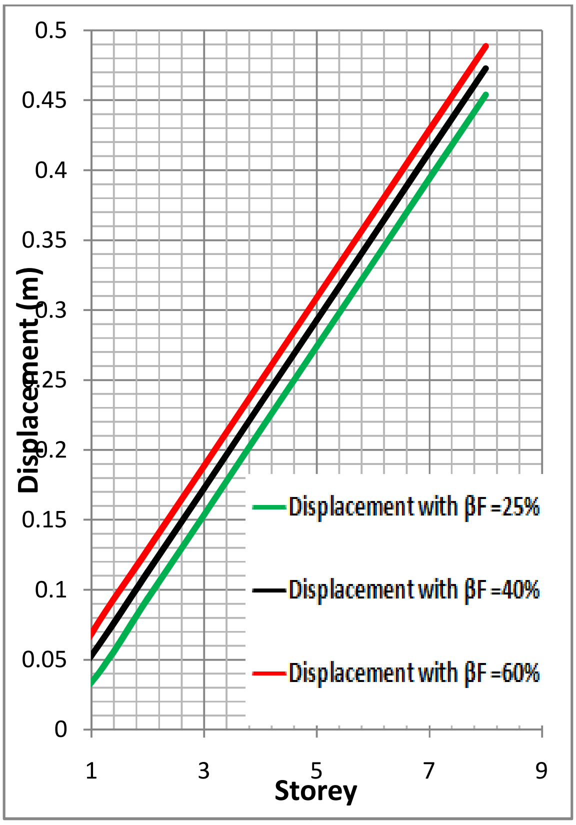
(a) **12 stories** **(d) 16 stories**



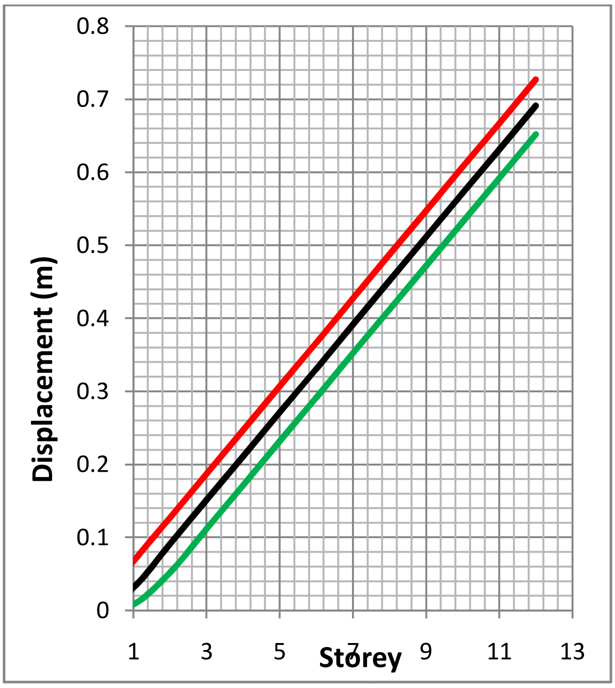
(e) 20 stories

Fig.4.9 (a-e) Stories displacement result from DDBD- without connected link beams

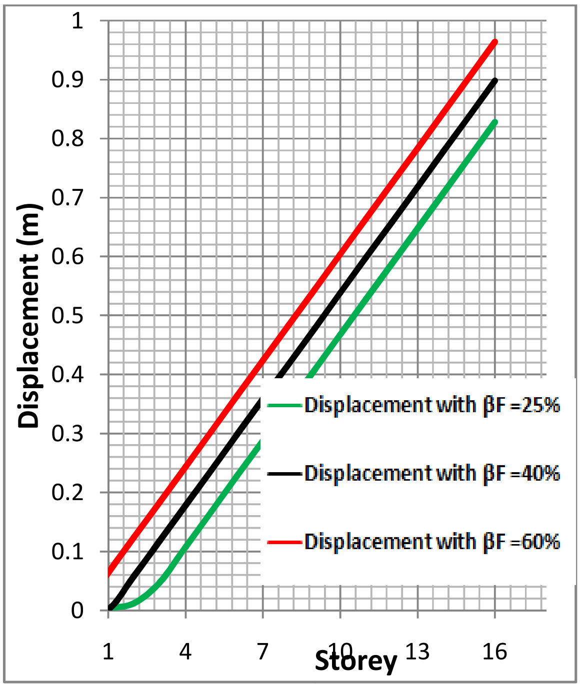
(a) 4 stories



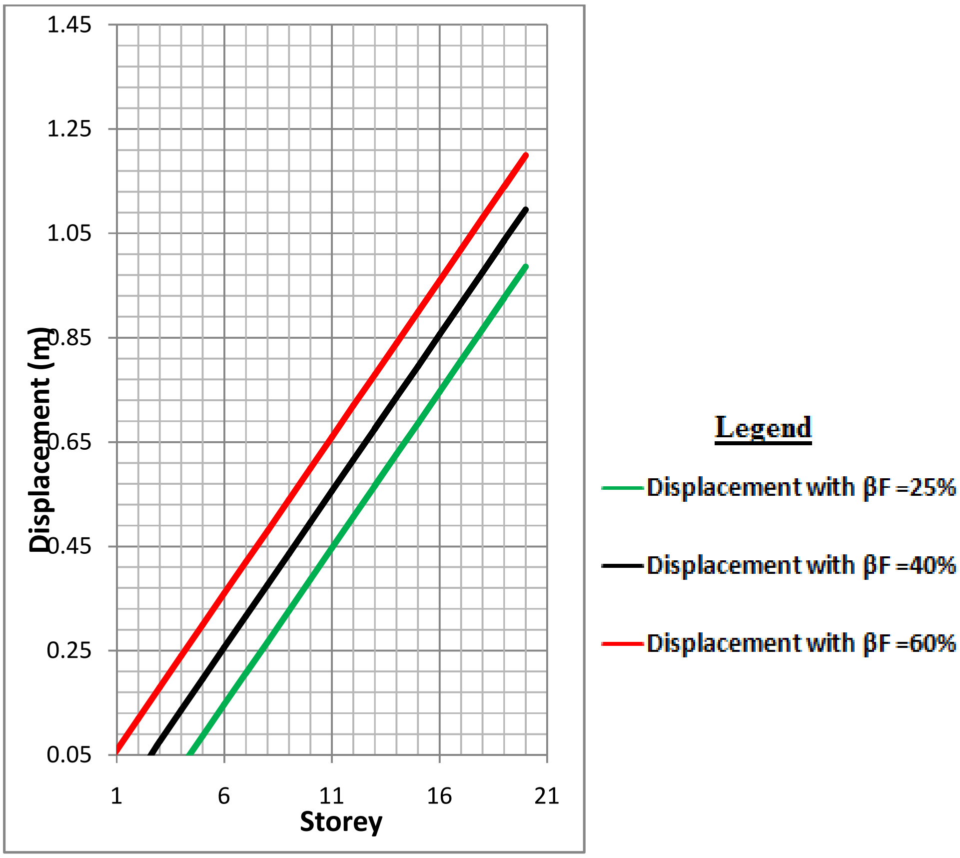
(b) 8 stories



(a) 12 stories



(d) 16 stories



d) 20 Stories

Fig.4.10 (a-e) Stories displacement result from DDBD - with connected link beams

From the above figures, graphs showing displacement of stories with different frame shear ratio, as frame shear ratio increases the displacement of the story also increases. And this increment is significantly seen in buildings with high stories than in lower stories, in these case of studies the displacement is significantly increases in case of a building with 20 Stories than in buildings which has small stories. This is true in both cases with and without connected link beam. This indicates that as the base shear carried by wall is large the storey displacement decreases.

In case of buildings without connected link beam the value of displacement with different frame shear ratio shows significant difference unlike buildings with connected link beam. In general, in addition to providing shear wall to the building sharing much base shear to the wall can reduce storey's displacement.

Chapter Five

5. Conclusion and Recommendation

4.1. 5.1 Conclusion

In this thesis work general theoretical background of DDBD method and its advantage over force based design is covered in detail. The drawback of traditional force based method is also briefly discussed. The detail procedure and application of DDBD for dual frame wall building is mentioned. And finally the design method is verified using different case studies. Buildings with storey 4, 8, 12, 16 and 20 with and without link beam are considered.

The proposed DDBD procedure for frame-wall building is quite straight forward and easy to implement though further refinement would be needed before these procedures are ready for use in any design based on our code.

In this study it is observed that an initial strength assignment that assumes the frames will carry a large fraction of the lateral load is likely to result in a low inflection height. Having a low inflection height implies that the design displacement and damping is maximized and the minimum possible base shear is obtained. However, a low inflection height will impose large curvature ductility demands on the walls, and if these are excessive then the design storey drift should be reduced or a larger strength proportion assigned to the walls. By assuming a large proportion of the shear is given to the walls then the opposite will occur, the consequences being that the design may not be very efficient, with heavily reinforced walls and poorly utilized beams and columns possessing only minimum reinforcement contents.

Direct displacement-based seismic design is a simple procedure for determining the required base shear strength to ensure that a structure responds at the design drift limit. And from the case studies it is observed that base shear obtained from DDBD is less than values those from traditional force based method. This indicates that using DDBD method is economical than FBD.

Use of direct displacement-based design will result in more consistent designs than force-based design criteria, and will generally result in reduced design forces.

Generally from the study the following points can be concluded:

- Comparing with force based method applying DDBD method for design of buildings is preferable because building failure is more related to deformation than force.
- The proportions of total shear resisted by the frames and the walls can be assigned arbitrarily to begin with and adjusted according to the required strength.
- DDBD method is more preferable method in terms of ductility, which is basic and important parameters in seismic design, through its full application in a way it is directly

used in analysis rather than being used indirectly by behavior factors in force based method. And this method is more economical as compared with force based method.

5.2 Recommendation

From the work above it is recommended structural designer can use this method since it is easily applicable. And also in addition to those stated method in our code it is advantages to introduce DDBD for seismic design as an option.

In this work only dual frame-wall reinforced concrete buildings are assessed though there are different buildings in different way of categories or aspects.

Related to the present work the following issues are the areas for further study.

- Studies may be carried out to verify that the proposed method could be extended to the design of structures having walls with a cross-section other than rectangular and/or core walls.
- Application of the method for 3D analysis
- The application of this method for steel structures

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Appendix - A
Different Calculation of DDBD for 4 storey Building

Table A.1 Preliminary calculations of 4 storey to determine **H_{cF} –** for **β_F =25%**

Level	Height, H_i (m)	Mass,m_i (tonne)	m_iH_i	F_i (relative)	V_{Ti} (relative)	$M_{OTM,i}$ (relative)	$V_{F,i}$ frame	$V_{w,i}$ wall	$M_{w,i}$ wall (without link beam)	$M_{w,i}$ wall (with link beam)
4	12.5	270	3375	0.329	0.329	0.000	0.25	0.079	0.000	0
3	9.5	352	3344	0.326	0.656	0.988	0.25	0.406	0.238	0.023
2	6.5	352	2288	0.223	0.879	2.954	0.25	0.629	1.454	0.357
1	3.5	355	1242.5	0.121	1.000	5.591	0.25	0.750	3.341	1.639
0	0	0	0	0.000	1.000	9.091	0.25	0.750	5.966	3.560
sum		1329	10249.5	1.000						

Table A.2 Design displacement information of 4 story **– without link beam** and **β_F =25%**

Level	Height, H_i (m)	Mass,m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
4	12.5	270	0.038	0.231	14.40	62.34	779.30	**0.166**	9.20	1098.46	12.01	1.99
3	9.5	352	0.024	0.171	10.28	60.16	571.50					
2	6.5	352	0.011	0.111	4.33	39.04	253.75					
1	3.5	355	0.004	0.058	1.20	20.60	72.10					
0	0	0	0.000	0.000	0.00	0.00	0.00					
Sum		**1329**			**30.20**	**182.14**	**1,676.64**					

Table A.3 Design displacement information of 4 story **– with link beam** and **β_F =25%**

Level	Height, H_i (m)	Mass,m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
4	12.5	270	0.035	0.233	14.61	62.81	785.16	0.168	9.265662	1,083.15	11.07	1.86
3	9.5	352	0.022	0.173	10.49	60.77	577.30					
2	6.5	352	0.010	0.113	4.47	39.65	257.72					
1	3.5	355	-0.003	0.053	0.98	18.69	65.40					
0	0	0	-0.017	0.000	0.00	0.00	0.00					
sum		**1329**			**30.55**	**181.92**	**1685.58**					

Table A.4 Preliminary calculations of 4 storey to determine **H_{cF} –** for **β_F =40%**

Level	Height, H_i (m)	Mass,m_i (tonne)	m_iH_i	F_i (relative)	V_{Ti} (relative)	$M_{OTM,i}$ (relative)	$V_{F,i}$ frame	$V_{w,i}$ wall	$M_{w,i}$ wall (without link beam)	$M_{w,i}$ wall (with link beam)
4	12.5	270	3375	0.329	0.329	0.000	0.4	-0.071	0.000	0
3	9.5	352	3344	0.326	0.656	0.988	0.4	0.256	-0.212	-0.020
2	6.5	352	2288	0.223	0.879	2.954	0.4	0.479	0.554	-0.137
1	3.5	355	1242.5	0.121	1.000	5.591	0.4	0.600	1.991	0.695
0	0	0	0	0.000	1.000	9.091	0.4	0.600	4.091	2.167
Sum		**1329**	**10249.5**	**1.000**						**4.267**

Table A.5 Design displacement information of 4 story **– without link beam** and **β_F =40%**

Level	Height, H_i (m)	Mass,m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
4	12.5	270	0.031	0.241	15.66	65.02	812.74	**0.1742**	9.158	1110.671	12.67	2.21
3	9.5	352	0.021	0.181	11.51	63.65	604.64					
2	6.5	352	0.011	0.121	5.14	42.53	276.42					
1	3.5	355	0.004	0.063	1.40	22.28	77.98					
0	0	0	0.000	0.000	0.00	0.00	0.00					
sum		**1329**			**33.70**	**193.47**	**1,771.78**					

Table A.6 Design displacement information of 4 story **– with link beam** and **β_F =40%**

Level	Height, H_i (m)	Mass,m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
4	12.5	270	0.023	0.246	16.29	66.31	828.87	0.178	9.13	1117.34	11.46	2.04
3	9.5	352	0.017	0.186	12.12	65.33	620.62					
2	6.5	352	0.010	0.126	5.55	44.21	287.36					
1	3.5	355	0.003	0.066	1.53	23.29	81.50					
0	0	0	-0.004	0.000	0.00	0.00	0.00					
sum		**1329**			**35.49**	**199.13**	**1818.35**					

Table A.7 Preliminary calculations of 4 storey to determine **H_{cF}** – for **β_F =60%**

Level	Height, H_i (m)	Mass, m_i (tonne)	m_iH_i	F_i (relative)	V_{Ti} (relative)	$M_{OTM,i}$ (relative)	$V_{F,i}$ frame	$V_{w,i}$ wall	$M_{w,i}$ wall (without link beam)	$M_{w,i}$ wall (with link beam)
4	12.5	270	3375	0.329	0.329	0.000	0.6	-0.27	0.00	0.00
3	9.5	352	3344	0.326	0.656	0.988	0.6	0.06	-0.81	-0.08
2	6.5	352	2288	0.223	0.879	2.954	0.6	0.28	-0.65	-0.80
1	3.5	355	1242.5	0.121	1.000	5.591	0.6	0.40	0.19	-0.56
0	0	0	0	0.000	1.000	9.091	0.6	0.40	1.59	0.31
Sum		**1329**	**10249.5**	**1.000**						

Table A.8 Design displacement information of 4 story **– without link beam** and **β_F =60%**

Level	Height, H_i (m)	Mass, m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
4	12.5	270	0.017	0.248	16.59	66.92	836.53	**0.1801**	9.11	1122.65	13.38	2.41
3	9.5	352	0.012	0.188	12.42	66.13	628.20					
2	6.5	352	0.008	0.128	5.75	45.01	292.54					
1	3.5	355	0.003	0.068	1.64	24.09	84.33					
0	0	0	0.000	0.000	0.00	0.00	0.00					
sum		1329			36.40	202.15	1,841.60					

Table A.9 Design displacement information of 4 story **– with link beam** and **β_F =60%**

Level	Height, H_i (m)	Mass, m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
4	12.5	270	0.005	0.250	16.85	67.45	843.12	0.18	9.09	1127.04	12.05	2.17
3	9.5	352	0.004	0.190	12.68	66.81	634.73					
2	6.5	352	0.003	0.130	5.93	45.69	297.01					
1	3.5	355	0.001	0.070	1.73	24.78	86.74					
0	0	0	0.000	0.000	0.00	0.00	0.00					
Sum		1329			37.19	204.74	1861.61					

Appendix - B
Different Calculation of DDBD for 8 storey Building

Table B.1 Preliminary calculations of 8 storey to determine **H_{cF} –** for **β_F =40%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall (without link beam)**	**$M_{w,i}$ wall (with link beam)**
8	24.5	290	7105	0.174	0.174	0.00	0.4	-0.226	0.000	0
7	21.5	385	8277.5	0.203	0.377	0.52	0.4	-0.023	-0.678	-0.066
6	18.5	385	7122.5	0.174	0.551	1.65	0.4	0.151	-0.748	-0.685
5	15.5	385	5967.5	0.146	0.697	3.30	0.4	0.297	-0.295	-0.704
4	12.5	385	4812.5	0.118	0.815	5.39	0.4	0.415	0.596	-0.208
3	9.5	385	3657.5	0.090	0.904	7.84	0.4	0.504	1.841	0.717
2	6.5	385	2502.5	0.061	0.966	10.55	0.4	0.566	3.354	1.99
1	3.5	400	1400	0.034	1.000	13.45	0.4	0.600	5.051	3.51
0	0	0	0	0.000	1.000	16.95	0.4	0.600	7.151	5.22
sum		**3000**	**40845**	**1.000**						

Table B.2 Design displacement information of 8 story **– without link beam** and **β_F =40%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
8	24.5	290	0.105	0.464	62.51	134.64	3298.73	**0.319**	17.19	2341.76	10.99	3.50
7	21.5	385	0.089	0.404	62.93	155.65	3346.45					
6	18.5	385	0.073	0.344	45.63	132.55	2452.15					
5	15.5	385	0.057	0.284	31.11	109.45	1696.41					
4	12.5	385	0.041	0.224	19.38	86.37	1079.69					
3	9.5	385	0.026	0.165	10.52	63.66	604.73					
2	6.5	385	0.013	0.109	4.54	41.81	271.79					
1	3.5	400	0.004	0.056	1.23	22.21	77.72					
0	0	0	0	0.000	0.00	0.00	0.00					
sum		3000			237.86	746.33	12,827.65					

Table B.3 Design displacement information of 8 story **– with link beam** and **β_F =40%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
8	24.5	290	0.089	0.473	64.8	137.1	3360.0	0.326	17.17	2345.6	9.72	3.17
7	21.5	385	0.076	0.413	65.6	158.9	3417.8					
6	18.5	385	0.063	0.353	47.9	135.8	2513.6					
5	15.5	385	0.050	0.293	33.0	112.7	1747.9					
4	12.5	385	0.037	0.233	20.8	89.6	1120.8					
3	9.5	385	0.024	0.173	11.5	66.5	632.4					
2	6.5	385	0.011	0.113	4.9	43.4	282.5					
1	3.5	400	-0.002	0.053	1.1	21.1	74.0					
0	0	0		0	0	0	0					
sum		**3000**			**249.90**	**765.63**	**13,149.2**					

Table B.4 Preliminary calculations of 8 storey to determine **H_{cF}** – for **β_F =60%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall (without link beam)**	**$M_{w,i}$ wall (with link beam)**
8	24.5	290	7105	0.174	0.174	0.00	0.6	-0.426	0.000	0
7	21.5	385	8277.5	0.203	0.377	0.52	0.6	-0.223	-1.278	-0.125
6	18.5	385	7122.5	0.174	0.551	1.65	0.6	-0.049	-1.948	-1.34
5	15.5	385	5967.5	0.146	0.697	3.30	0.6	0.097	-2.095	-1.96
4	12.5	385	4812.5	0.118	0.815	5.39	0.6	0.215	-1.804	-2.06
3	9.5	385	3657.5	0.090	0.904	7.84	0.6	0.304	-1.159	-1.74
2	6.5	385	2502.5	0.061	0.966	10.55	0.6	0.366	-0.246	-1.07
1	3.5	400	1400	0.034	1.000	13.45	0.6	0.400	0.851	-0.138
0	0	0	0	0.000	1.000	16.95	0.6	0.400	2.251	0.968
Sum		3000	40845	1.000						

Table B.5 Design displacement information of 8 story **– without link beam** and **β_F =60%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
8	24.5	290	0.048	0.486	68.4	140.9	3451.9	**0.336**	17.00	2394.87	9.84	3.30
7	21.5	385	0.042	0.426	69.8	163.9	3524.9					
6	18.5	385	0.035	0.366	51.5	140.8	2605.7					
5	15.5	385	0.029	0.306	36.0	117.7	1825.1					
4	12.5	385	0.023	0.246	23.2	94.6	1183.1					
3	9.5	385	0.016	0.186	13.3	71.5	679.7					
2	6.5	385	0.010	0.126	6.1	48.4	314.9					
1	3.5	400	0.003	0.066	1.7	26.3	92.1					
0	0	0	0.004	0.004	0.0	0.0	0.00					
sum		3000			270.22	804.45	13,677.8					

Table B.6 Design displacement information of 8 story **– with link beam** and **β_F =60%**

Level	Height, H_i (m)	Mass, m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
8	24.5	290	0.026	0.489	69.30	141.77	3473.31	0.338	16.97	2405.56	8.90	3.01
7	21.5	385	0.023	0.429	70.81	165.11	3549.84					
6	18.5	385	0.020	0.369	52.38	142.01	2627.17					
5	15.5	385	0.016	0.309	36.73	118.91	1843.09					
4	12.5	385	0.013	0.249	23.84	95.81	1197.61					
3	9.5	385	0.010	0.189	13.73	72.71	690.74					
2	6.5	385	0.006	0.129	6.39	49.61	322.46					
1	3.5	400	0.003	0.069	1.90	27.54	96.40					
0	0	0		0	0	0	0					
sum		**3000**			**275.08**	**813.46**	**13,800.61**					

Appendix - C
Different Calculation of DDBD for 12 storey Building

Table C.1 Preliminary calculations of 12 storey to determine $\mathbf{H_{cF}}$ – for $\mathbf{\beta_F}$ **=25%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall (without link beam)**	**$M_{w,i}$ wall (with link beam)**
12	36.5	300	10950	0.11	0.11	0.00	0.25	-0.14	0.00	0
11	33.5	420	14070	0.15	0.26	0.34	0.25	0.01	-0.41	-0.04
10	30.5	420	12810	0.13	0.39	1.12	0.25	0.14	-0.38	-0.41
9	27.5	420	11550	0.12	0.51	2.29	0.25	0.26	0.04	-0.34
8	24.5	420	10290	0.11	0.62	3.83	0.25	0.37	0.83	0.12
7	21.5	420	9030	0.09	0.71	5.69	0.25	0.46	1.94	0.94
6	18.5	420	7770	0.08	0.79	7.82	0.25	0.54	3.32	2.07
5	15.5	420	6510	0.07	0.86	10.20	0.25	0.61	4.95	3.48
4	12.5	420	5250	0.05	0.91	12.78	0.25	0.66	6.78	5.13
3	9.5	420	3990	0.04	0.96	15.52	0.25	0.71	8.77	6.97
2	6.5	420	2730	0.03	0.98	18.39	0.25	0.73	10.89	8.98
1	3.5	440	1540	0.02	1.00	21.34	0.25	0.75	13.09	11.10
0	0	0	0	0.00	1.000	24.84	0.25	0.75	15.72	13.31
Sum		4940	96490	1.000						

Table C.2 Design displacement information of 12 story **– without link beam** and $\mathbf{\beta_F}$ **=25%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
12	36.5	300	0.28	0.596	106.7	178.9	6530.4	**0.396**	25.73	3600.11	12.08	4.79
11	33.5	420	0.25	0.540	122.3	226.6	7592.1					
10	30.5	420	0.22	0.483	97.9	202.8	6184.7					
9	27.5	420	0.19	0.426	76.2	178.9	4920.5					
8	24.5	420	0.16	0.369	57.3	155.1	3799.3					
7	21.5	420	0.13	0.312	41.0	131.2	2821.3					
6	18.5	420	0.09	0.256	27.4	107.4	1986.4					
5	15.5	420	0.07	0.207	18.0	87.0	1348.0					
4	12.5	420	0.05	0.158	10.5	66.3	828.7					
3	9.5	420	0.03	0.112	5.3	47.2	448.0					
2	6.5	420	0.01	0.071	2.1	29.8	193.9					
1	3.5	440	0.004	0.035	0.5	15.3	53.7					
0	0	0	0	0.000	0.0	0.0	0.0					
Sum		4940			565.2	1,426.5	36,706.9					

Table C.3 Design displacement information of 12 story **– with link beam** and **β_F =25%**

Level	Height, H_i (m)	Mass,m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
12	36.5	300	0.26	0.65	127.5	195.6	7137.6	0.445	26.17	3467.24	9.91	4.41
11	33.5	420	0.23	0.59	147.1	248.6	8327.1					
10	30.5	420	0.20	0.53	118.8	223.4	6812.8					
9	27.5	420	0.18	0.47	93.5	198.2	5449.7					
8	24.5	420	0.15	0.41	71.2	173.0	4237.8					
7	21.5	420	0.12	0.35	52.0	147.8	3177.1					
6	18.5	420	0.09	0.29	35.8	122.6	2267.6					
5	15.5	420	0.07	0.23	22.6	97.4	1509.3					
4	12.5	420	0.04	0.17	12.4	72.2	902.1					
3	9.5	420	0.01	0.11	5.3	47.0	446.2					
2	6.5	420	-0.02	0.05	1.1	21.8	141.5					
1	3.5	440	-0.05	0.01	0.0	3.6	12.6					
0	0	0		0.000	0	0	0					
sum		**4940**			**687.26**	**1,543.67**	**40,396.22**					

Table C.4 Preliminary calculations of 12 storey to determine **H_{cF} –** for **β_F =40%**

Level	Height, H_i (m)	Mass,m_i (tonne)	m_iH_i	F_i (relative)	V_{Ti} (relative)	$M_{OTM,i}$ (relative)	$V_{F,i}$ frame	$V_{w,i}$ wall	$M_{w,i}$ wall (without link beam)	$M_{w,i}$ wall (with link beam)
12	36.5	300	10950	0.11	0.11	0.00	0.4	-0.29	0.00	0.00
11	33.5	420	14070	0.15	0.26	0.34	0.4	-0.14	-0.86	-0.08
10	30.5	420	12810	0.13	0.39	1.12	0.4	-0.01	-1.28	-0.90
9	27.5	420	11550	0.12	0.51	2.29	0.4	0.11	-1.31	-1.28
8	24.5	420	10290	0.11	0.62	3.83	0.4	0.22	-0.97	-1.27
7	21.5	420	9030	0.09	0.71	5.69	0.4	0.31	-0.31	-0.91
6	18.5	420	7770	0.08	0.79	7.82	0.4	0.39	0.62	-0.22
5	15.5	420	6510	0.07	0.86	10.20	0.4	0.46	1.80	0.74
4	12.5	420	5250	0.05	0.91	12.78	0.4	0.51	3.18	1.93
3	9.5	420	3990	0.04	0.96	15.52	0.4	0.56	4.72	3.33
2	6.5	420	2730	0.03	0.98	18.39	0.4	0.58	6.39	4.88
1	3.5	440	1540	0.02	1.00	21.34	0.4	0.60	8.14	6.56
0	0	0	0	0.00	1.00	24.84	0.4	0.60	10.24	8.32
sum		4940	96490	1.00						

Table C.5 Design displacement information of 12 story **– without link beam** and **β_F =40%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
12	36.5	300	0.22	0.64	122.7	191.9	7003.8	**0.431**	25.39	3711.14	9.88	4.26
11	33.5	420	0.20	0.58	142.7	244.8	8200.4					
10	30.5	420	0.18	0.53	116.2	220.9	6738.5					
9	27.5	420	0.16	0.47	92.5	197.1	5419.8					
8	24.5	420	0.13	0.41	71.5	173.2	4244.2					
7	21.5	420	0.11	0.36	53.1	149.4	3211.7					
6	18.5	420	0.09	0.30	37.5	125.5	2322.3					
5	15.5	420	0.07	0.24	24.8	102.0	1580.8					
4	12.5	420	0.05	0.19	14.9	79.1	988.8					
3	9.5	420	0.03	0.14	7.8	57.3	544.4					
2	6.5	420	0.01	0.09	3.3	37.0	240.4					
1	3.5	440	0.00	0.04	0.9	19.4	68.1					
0	0	0	0.00	0.00	0.0	0.0	0.0					
Sum		4940			687.8	1597.7	40563.2					

Table C.6 Design displacement information of 12 story **– with link beam** and **β_F =40%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
12	36.5	300	0.20	0.69	143.36	207.38	7569.40	0.470	25.43	3700.64	8.57	4.03
11	33.5	420	0.18	0.63	167.37	265.13	8881.96					
10	30.5	420	0.16	0.57	137.07	239.93	7317.96					
9	27.5	420	0.14	0.51	109.79	214.73	5905.16					
8	24.5	420	0.12	0.45	85.53	189.53	4643.56					
7	21.5	420	0.10	0.39	64.30	164.33	3533.16					
6	18.5	420	0.08	0.33	46.09	139.13	2573.96					
5	15.5	420	0.06	0.27	30.91	113.93	1765.96					
4	12.5	420	0.04	0.21	18.75	88.73	1109.16					
3	9.5	420	0.02	0.15	9.61	63.53	603.56					
2	6.5	420	0.00	0.09	3.50	38.33	249.16					
1	3.5	440	-0.02	0.03	0.43	13.76	48.15					
0	0	0		0.00	0.00	0.00	0.00					
sum		**4940**			**816.69**	**1738.47**	**44201.17**					

Table C.7 Preliminary calculations of 12 storey to determine **H_{cF} –** for **β_F =60%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall (without link beam)**	**$M_{w,i}$ wall (with link beam)**
12	36.5	300	10950	0.11	0.11	0.00	0.6	-0.49	0.00	0.00
11	33.5	420	14070	0.15	0.26	0.34	0.6	-0.34	-1.46	-0.14
10	30.5	420	12810	0.13	0.39	1.12	0.6	-0.21	-2.48	-1.56
9	27.5	420	11550	0.12	0.51	2.29	0.6	-0.09	-3.11	-2.54
8	24.5	420	10290	0.11	0.62	3.83	0.6	0.02	-3.37	-3.13
7	21.5	420	9030	0.09	0.71	5.69	0.6	0.11	-3.31	-3.36
6	18.5	420	7770	0.08	0.79	7.82	0.6	0.19	-2.98	-3.28
5	15.5	420	6510	0.07	0.86	10.20	0.6	0.26	-2.40	-2.92
4	12.5	420	5250	0.05	0.91	12.78	0.6	0.31	-1.62	-2.33
3	9.5	420	3990	0.04	0.96	15.52	0.6	0.36	-0.68	-1.53
2	6.5	420	2730	0.03	0.98	18.39	0.6	0.38	0.39	-0.57
1	3.5	440	1540	0.02	1.00	21.34	0.6	0.40	1.54	0.50
0	0	0	0	0.00	1.00	24.84	0.6	0.40	2.94	1.66
sum		4940	96490	1.000						

Table C.8 Design displacement information of 12 story **– without link beam** and **β_F =60%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
12	36.5	300	0.09	0.68	140.3	205.2	7488.5	**0.465**	24.96	3846.32	8.61	4.00
11	33.5	420	0.09	0.63	165.2	263.4	8823.2					
10	30.5	420	0.08	0.57	136.6	239.5	7305.6					
9	27.5	420	0.07	0.51	110.8	215.7	5931.1					
8	24.5	420	0.06	0.46	87.6	191.8	4699.7					
7	21.5	420	0.05	0.40	67.2	168.0	3611.5					
6	18.5	420	0.04	0.34	49.5	144.1	2666.3					
5	15.5	420	0.03	0.28	32.6	116.9	1812.4					
4	12.5	420	0.03	0.23	21.8	95.6	1195.3					
3	9.5	420	0.02	0.17	12.5	72.5	689.0					
2	6.5	420	0.01	0.12	5.7	48.7	316.7					
1	3.5	440	0.00	0.06	1.6	26.5	92.9					
0	0	0	0.00	0.00	0.0	0.0	0.0					
sum		4940			831.18	1,788.01	44,632.20					

Table C.9 Design displacement information of 12 story **– with link beam** and **β_F =60%**

Level	Height, H_i (m)	Mass,m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
12	36.5	300	0.06	0.73	158.6	218.1	7961.4	0.495	24.88	3871.73	7.86	3.89
11	33.5	420	0.06	0.67	186.9	280.2	9385.6					
10	30.5	420	0.05	0.61	154.8	255.0	7776.5					
9	27.5	420	0.05	0.55	125.7	229.8	6318.6					
8	24.5	420	0.04	0.49	99.6	204.6	5011.9					
7	21.5	420	0.04	0.43	76.6	179.4	3856.4					
6	18.5	420	0.03	0.37	56.6	154.2	2852.1					
5	15.5	420	0.02	0.31	39.6	129.0	1999.0					
4	12.5	420	0.02	0.25	25.6	103.8	1297.1					
3	9.5	420	0.01	0.19	14.7	78.6	746.4					
2	6.5	420	0.01	0.13	6.8	53.4	346.9					
1	3.5	440	0.00	0.07	2.0	29.5	103.3					
0	0	0		0.00	0.0	0.0	0.0					
Sum		**4940**			**947.5**	**1915.3**	**47655.3**					

Appendix - D

Different Calculation of DDBD for 16 storey Building

Table D.1 Preliminary calculations of 16 storey to determine **H_{cF} –** for **β_F =25%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall (without link beam)**	**$M_{w,i}$ wall (with link beam)**
16	48.5	350	16975	0.09	0.09	**0.00**	0.25	-0.16	0	**0**
15	45.5	460	20930	0.11	0.20	0.61	0.25	-0.05	-0.48	-0.05
14	42.5	460	19550	0.11	0.31	1.22	0.25	0.06	-0.62	-0.49
13	39.5	460	18170	0.10	0.41	2.15	0.25	0.16	-0.44	-0.60
12	36.5	460	16790	0.09	0.50	3.37	0.25	0.25	0.03	-0.39
11	33.5	460	15410	0.08	0.58	4.86	0.25	0.33	0.77	0.10
10	30.5	460	14030	0.08	0.65	6.60	0.25	0.40	1.76	0.87
9	27.5	460	12650	0.07	0.72	8.56	0.25	0.47	2.97	1.88
8	24.5	460	11270	0.06	0.78	10.73	0.25	0.53	4.39	3.11
7	21.5	460	9890	0.05	0.84	13.08	0.25	0.59	5.99	4.55
6	18.5	460	8510	0.05	0.88	15.59	0.25	0.63	7.75	6.16
5	15.5	460	7130	0.04	0.92	18.23	0.25	0.67	9.65	7.94
4	12.5	460	5750	0.03	0.95	21.00	0.25	0.70	11.66	9.84
3	9.5	460	4370	0.02	0.97	23.85	0.25	0.72	13.76	11.86
2	6.5	460	2990	0.02	0.99	26.77	0.25	0.74	15.94	13.98
1	3.5	480	1680	0.01	1.00	29.75	0.25	0.75	18.16	16.15
0	0	0	0	0.00	1.00	33.25	0.25	0.75	20.79	18.38
Sum		7270	186095	1.00						

Table D.2 Design displacement information of 16 story **– without link beam** and **β_F =25%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
16	48.5	350	0.49	0.75	196.0	261.9	12701.7	**0.493**	34.66	5041.24	14.46	7.13
15	45.5	460	0.45	0.69	219.9	318.1	14472.5					
14	42.5	460	0.41	0.63	185.3	292.0	12408.1					
13	39.5	460	0.37	0.58	153.6	265.8	10500.4					
12	36.5	460	0.33	0.52	124.9	239.7	8749.4					
11	33.5	460	0.29	0.46	99.2	213.7	7157.9					
10	30.5	460	0.25	0.41	76.8	187.9	5731.9					
9	27.5	460	0.21	0.35	57.6	162.7	4475.6					
8	24.5	460	0.17	0.30	41.6	138.4	3389.9					
7	21.5	460	0.14	0.25	28.8	115.0	2472.9					
6	18.5	460	0.10	0.20	18.8	93.0	1719.7					
5	15.5	460	0.08	0.16	11.4	72.4	1122.6					
4	12.5	460	0.05	0.12	6.3	53.7	670.8					
3	9.5	460	0.03	0.08	3.0	36.9	350.8					
2	6.5	460	0.01	0.05	1.1	22.4	145.9					
1	3.5	480	0.00	0.02	0.2	10.9	38.2					
0	0	0	0.00	0.00	0.0	0.0	0.0					
sum		7270			1224.5	2484.5	86108.2					

Table D.3 Design displacement information of 16 story **– with link beam** and **β_F =25%**

Level	Height, H_i (m)	Mass, m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
16	48.5	350	0.46	0.83	239.8	289.7	14050.5	0.572	35.73	4696.63	12.11	6.93
15	45.5	460	0.43	0.77	271.1	353.1	16068.3					
14	42.5	460	0.39	0.71	230.4	325.5	13835.8					
13	39.5	460	0.35	0.65	193.0	297.9	11769.0					
12	36.5	460	0.31	0.59	158.9	270.3	9867.7					
11	33.5	460	0.28	0.53	128.1	242.7	8132.1					
10	30.5	460	0.24	0.47	100.6	215.1	6562.0					
9	27.5	460	0.20	0.41	76.5	187.5	5157.6					
8	24.5	460	0.16	0.35	55.6	159.9	3918.7					
7	21.5	460	0.13	0.29	38.1	132.3	2845.5					
6	18.5	460	0.09	0.23	23.9	104.7	1937.8					
5	15.5	460	0.05	0.17	12.9	77.1	1195.8					
4	12.5	460	0.01	0.11	5.3	49.5	619.4					
3	9.5	460	-0.02	0.05	1.0	21.9	208.5					
2	6.5	460	-0.06	-0.01	0.1	-5.7	-36.7					
1	3.5	480	-0.10	-0.07	2.5	-34.7	-121.4					
0	0	0		0.00	0.0	0.0	0.0					
sum		**7270**			**1537.8**	**2687.5**	**96010.9**					

Table D.4 Preliminary calculations of 16 storey to determine **H_{cF} –** for **β_F =40%**

Level	Height, H_i (m)	Mass, m_i (tonne)	m_iH_i	F_i (relative)	V_{Ti} (relative)	$M_{OTM,i}$ (relative)	$V_{F,i}$ frame	$V_{w,i}$ wall	$M_{w,i}$ wall (without link beam)	$M_{w,i}$ wall (with link beam)
16	48.5	350	16975	0.09	0.09	0.00	0.4	-0.309	**0**	**0**
15	45.5	460	20930	0.11	0.20	0.61	0.4	-0.196	-0.93	-0.09
14	42.5	460	19550	0.11	0.31	1.22	0.4	-0.091	-1.52	-0.98
13	39.5	460	18170	0.10	0.41	2.15	0.4	0.006	-1.79	-1.54
12	36.5	460	16790	0.09	0.50	3.37	0.4	0.097	-1.77	-1.79
11	33.5	460	15410	0.08	0.58	4.86	0.4	0.179	-1.48	-1.74
10	30.5	460	14030	0.08	0.65	6.60	0.4	0.255	-0.94	-1.43
9	27.5	460	12650	0.07	0.72	8.56	0.4	0.323	-0.18	-0.87
8	24.5	460	11270	0.06	0.78	10.73	0.4	0.383	0.79	-0.08
7	21.5	460	9890	0.05	0.84	13.08	0.4	0.436	1.94	0.90
6	18.5	460	8510	0.05	0.88	15.59	0.4	0.482	3.25	2.07
5	15.5	460	7130	0.04	0.92	18.23	0.4	0.521	4.70	3.39
4	12.5	460	5750	0.03	0.95	21.00	0.4	0.551	6.26	4.85
3	9.5	460	4370	0.02	0.97	23.85	0.4	0.575	7.91	6.42
2	6.5	460	2990	0.02	0.99	26.77	0.4	0.591	9.64	8.08
1	3.5	480	1680	0.01	1.00	29.75	0.4	0.600	11.41	9.81
0	0	0	0	0.00	1.00	33.25	0.4	0.600	13.51	11.59
Sum		7270	186095	1.00						

Table D.5 Design displacement information of 16 story **– without link beam** and **β_F =40%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
16	48.5	350	0.39	0.83	240.7	290.3	14077.6	**0.555**	33.91	5305.47	11.87	6.59
15	45.5	460	0.36	0.77	274.5	355.4	16169.0					
14	42.5	460	0.33	0.72	235.6	329.2	13992.7					
13	39.5	460	0.30	0.66	199.7	303.1	11973.1					
12	36.5	460	0.27	0.60	166.8	277.0	10110.3					
11	33.5	460	0.24	0.55	136.8	250.9	8404.2					
10	30.5	460	0.21	0.49	109.8	224.7	6854.8					
9	27.5	460	0.18	0.43	85.8	198.6	5462.2					
8	24.5	460	0.15	0.38	64.7	172.5	4227.1					
7	21.5	460	0.12	0.32	46.8	146.7	3154.4					
6	18.5	460	0.10	0.26	32.1	121.5	2248.0					
5	15.5	460	0.07	0.21	20.6	97.3	1507.6					
4	12.5	460	0.05	0.16	12.0	74.3	928.8					
3	9.5	460	0.03	0.12	6.1	53.0	503.2					
2	6.5	460	0.01	0.07	2.5	33.6	218.4					
1	3.5	480	0.00	0.04	0.6	17.3	60.4					
0	0	0	0.00	0.00	0.0	0.0	0.0					
sum		7270			1635.2	2945.4	99891.8					

Table D.6 Design displacement information of 16 story **– with link beam** and **β_F =40%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
16	48.5	350	0.36	0.90	282.3	314.3	15245.9	0.61	34.10	5243.07	10.46	6.38
15	45.5	460	0.33	0.84	323.1	385.5	17542.2					
14	42.5	460	0.31	0.78	278.5	357.9	15212.6					
13	39.5	460	0.28	0.72	237.2	330.3	13048.6					
12	36.5	460	0.25	0.66	199.2	302.7	11050.1					
11	33.5	460	0.23	0.60	164.6	275.1	9217.3					
10	30.5	460	0.20	0.54	133.2	247.5	7550.1					
9	27.5	460	0.17	0.48	105.2	219.9	6048.4					
8	24.5	460	0.15	0.42	80.4	192.3	4712.4					
7	21.5	460	0.12	0.36	59.0	164.7	3542.0					
6	18.5	460	0.09	0.30	40.9	137.1	2537.2					
5	15.5	460	0.07	0.24	26.1	109.5	1697.9					
4	12.5	460	0.04	0.18	14.6	81.9	1024.3					
3	9.5	460	0.01	0.12	6.4	54.3	516.3					
2	6.5	460	-0.01	0.06	1.6	26.7	173.8					
1	3.5	480	-0.04	0.00	0.0	-0.9	-3.1					
0	0	0		0.00	0.0	0.0	0.0					
Sum		**7270**			**1952.4**	**3199.5**	**109116.4**					

Table D.7 Preliminary calculations of 16 storey to determine **H_{cF} –** for **β_F =60%**

Level	Height, H_i (m)	Mass,m_i (tonne)	m_iH_i	F_i (relative)	V_{Ti} (relative)	$M_{OTM,i}$ (relative)	$V_{F,i}$ frame	$V_{w,i}$ wall	$M_{w,i}$ wall (without link beam)	$M_{w,i}$ wall (with link beam)
16	48.5	350	16975	0.09	**0.09**	**0.00**	0.6	-0.51	**0.00**	**0.00**
15	45.5	460	20930	0.11	0.20	0.27	0.6	-0.40	-1.53	-0.15
14	42.5	460	19550	0.11	0.31	0.88	0.6	-0.29	-2.72	-1.64
13	39.5	460	18170	0.10	0.41	1.81	0.6	-0.19	-3.59	-2.80
12	36.5	460	16790	0.09	0.50	3.03	0.6	-0.10	-4.17	-3.65
11	33.5	460	15410	0.08	0.58	4.52	0.6	-0.02	-4.48	-4.20
10	30.5	460	14030	0.08	0.65	6.26	0.6	0.05	-4.54	-4.49
9	27.5	460	12650	0.07	0.72	8.22	0.6	0.12	-4.38	-4.53
8	24.5	460	11270	0.06	0.78	10.39	0.6	0.18	-4.01	-4.34
7	21.5	460	9890	0.05	0.84	12.74	0.6	0.24	-3.46	-3.96
6	18.5	460	8510	0.05	0.88	15.25	0.6	0.28	-2.75	-3.39
5	15.5	460	7130	0.04	0.92	17.90	0.6	0.32	-1.90	-2.67
4	12.5	460	5750	0.03	0.95	20.66	0.6	0.35	-0.94	-1.81
3	9.5	460	4370	0.02	0.97	23.51	0.6	0.37	0.11	-0.84
2	6.5	460	2990	0.02	0.99	26.44	0.6	0.39	1.24	0.22
1	3.5	480	1680	0.01	1.00	29.41	0.6	0.40	2.41	1.35
0	0	0	0	0.00	1.00	32.91	0.6	0.40	3.81	2.53
Sum		7270	186095	1.000						

Table D.8 Design displacement information of 16 story **– without link beam** and **β_F =60%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
16	48.5	350	0.16	0.91	287.48	317.20	15384.40	**0.614**	33.08	5596.8	10.22	6.28
15	45.5	460	0.15	0.85	331.97	390.77	17780.22					
14	42.5	460	0.14	0.79	289.07	364.65	15497.67					
13	39.5	460	0.13	0.74	249.13	338.53	13371.87					
12	36.5	460	0.12	0.68	212.17	312.41	11402.80					
11	33.5	460	0.11	0.62	178.17	286.28	9590.47					
10	30.5	460	0.10	0.57	147.14	260.16	7934.87					
9	27.5	460	0.09	0.51	119.07	234.04	6436.01					
8	24.5	460	0.08	0.45	93.97	207.91	5093.89					
7	21.5	460	0.07	0.40	71.84	181.79	3908.51					
6	18.5	460	0.05	0.34	52.68	155.67	2879.86					
5	15.5	460	0.04	0.28	36.48	129.55	2007.95					
4	12.5	460	0.03	0.22	23.25	103.42	1292.78					
3	9.5	460	0.02	0.17	12.99	77.30	734.35					
2	6.5	460	0.01	0.11	5.74	51.39	334.01					
1	3.5	480	0.00	0.06	1.59	27.65	96.78					
0	0	0	0.00	0.00	0.00	0.00	0.00					
sum		7270			2112.7	3438.7	113746.4					

Table D.9 Design displacement information of 16 story **– with link beam** and **β_F =60%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
16	48.5	350	0.12	0.96	325.1	337.3	16360.3	0.654	33.00	5623.71	9.18	6.01
15	45.5	460	0.11	0.90	375.7	415.7	18916.3					
14	42.5	460	0.10	0.84	327.5	388.1	16496.0					
13	39.5	460	0.10	0.78	282.6	360.5	14241.4					
12	36.5	460	0.09	0.72	241.0	332.9	12152.4					
11	33.5	460	0.08	0.66	202.7	305.3	10229.0					
10	30.5	460	0.07	0.60	167.7	277.7	8471.1					
9	27.5	460	0.07	0.54	136.0	250.1	6878.9					
8	24.5	460	0.06	0.48	107.7	222.5	5452.3					
7	21.5	460	0.05	0.42	82.6	194.9	4191.3					
6	18.5	460	0.04	0.36	60.9	167.3	3095.8					
5	15.5	460	0.03	0.30	42.5	139.7	2166.0					
4	12.5	460	0.03	0.24	27.3	112.1	1401.8					
3	9.5	460	0.02	0.18	15.5	84.5	803.2					
2	6.5	460	0.01	0.12	7.0	56.9	370.1					
1	3.5	480	0.00	0.06	2.0	30.6	107.2					
0	0	0		0.00	0.0	0.0	0.0					
sum		**7270**			**2403.8**	**3676.7**	**121333.0**					

Appendix - E

Different Calculation of DDBD for 20 storey Building

Table E.1 Preliminary calculations of 20 storey to determine $\mathbf{H_{cF}}$ – for $\boldsymbol{\beta_F}$ **=25%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall (without link beam)**	**$M_{w,i}$ wall (with link beam)**
20	60.5	400	24200	0.07	0.07	0.00	0.25	-0.176	**0**	**0**
19	57.5	520	29900	0.09	0.17	0.50	0.25	-0.084	-0.53	-0.05
18	54.5	520	28340	0.09	0.25	1.00	0.25	0.003	-0.78	-0.55
17	51.5	520	26780	0.08	0.34	1.76	0.25	0.085	-0.77	-0.78
16	48.5	520	25220	0.08	0.41	2.76	0.25	0.163	-0.51	-0.74
15	45.5	520	23660	0.07	0.49	4.00	0.25	0.235	-0.02	-0.47
14	42.5	520	22100	0.07	0.55	5.46	0.25	0.303	0.68	0.04
13	39.5	520	20540	0.06	0.62	7.12	0.25	0.366	1.59	0.77
12	36.5	520	18980	0.06	0.67	8.97	0.25	0.425	2.69	1.70
11	33.5	520	17420	0.05	0.73	10.99	0.25	0.478	3.97	2.82
10	30.5	520	15860	0.05	0.78	13.18	0.25	0.527	5.40	4.11
9	27.5	520	14300	0.04	0.82	15.51	0.25	0.571	6.98	5.55
8	24.5	520	12740	0.04	0.86	17.97	0.25	0.610	8.69	7.15
7	21.5	520	11180	0.03	0.89	20.55	0.25	0.644	10.52	8.87
6	18.5	520	9620	0.03	0.92	23.23	0.25	0.674	12.46	10.71
5	15.5	520	8060	0.02	0.95	26.00	0.25	0.699	14.48	12.65
4	12.5	520	6500	0.02	0.97	28.85	0.25	0.719	16.57	14.68
3	9.5	520	4940	0.02	0.98	31.75	0.25	0.734	18.73	16.78
2	6.5	520	3380	0.01	0.99	34.71	0.25	0.744	20.93	18.94
1	3.5	550	1925	0.01	1.00	37.69	0.25	0.750	23.16	21.15
0	0	0	0	0.00	1.00	41.19	0.25	0.750	25.79	23.38
sum		10310	325645	1.00						

Table E.2 Design displacement information of 20 story **– without link beam** and **β_F =25%**

Level	Height, H_i (m)	Mass,m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
20	60.5	400	0.75	0.89	319.24	357.35	21619.4	**0.585**	43.808	6,810.41	16.8	9.83
19	57.5	520	0.70	0.83	363.93	435.02	25013.62					
18	54.5	520	0.65	0.78	316.20	405.49	22099.16					
17	51.5	520	0.60	0.72	271.82	375.96	19361.89					
16	48.5	520	0.55	0.66	230.79	346.43	16801.80					
15	45.5	520	0.50	0.60	193.12	316.90	14418.89					
14	42.5	520	0.45	0.55	158.85	287.40	12214.61					
13	39.5	520	0.40	0.49	128.13	258.13	10195.95					
12	36.5	520	0.35	0.44	101.11	229.29	8369.23					
11	33.5	520	0.31	0.38	77.80	201.13	6738.02					
10	30.5	520	0.26	0.33	58.14	173.88	5303.20					
9	27.5	520	0.22	0.28	41.98	147.74	4062.91					
8	24.5	520	0.18	0.23	29.08	122.96	3012.56					
7	21.5	520	0.14	0.19	19.14	99.76	2144.86					
6	18.5	520	0.11	0.15	11.81	78.37	1449.79					
5	15.5	520	0.07	0.11	6.70	59.01	914.60					
4	12.5	520	0.05	0.08	3.38	41.91	523.82					
3	9.5	520	0.03	0.05	1.43	27.29	259.27					
2	6.5	520	0.01	0.03	0.46	15.39	100.05					
1	3.5	550	0.00	0.01	0.08	6.80	23.82					
0	0	0	0	0.00	0.00	0.00	0.00					
sum		10310			2,333.17	3,986.21	174,627.47					

Table E.3 Design displacement information of 20 story **– with link beam** and **β_F =25%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
20	60.5	400	0.73	0.99	389.27	394.6	23873.4	0.697	46.02	6039.22	13.78	19.61
19	57.5	520	0.68	0.93	446.37	481.8	27702.4					
18	54.5	520	0.63	0.87	390.43	450.6	24556.7					
17	51.5	520	0.58	0.81	338.23	419.4	21598.1					
16	48.5	520	0.54	0.75	289.78	388.2	18826.8					
15	45.5	520	0.49	0.69	245.07	357.0	16242.6					
14	42.5	520	0.44	0.63	204.10	325.8	13845.7					
13	39.5	520	0.40	0.57	166.88	294.6	11636.0					
12	36.5	520	0.35	0.51	133.40	263.4	9613.4					
11	33.5	520	0.30	0.45	103.67	232.2	7778.1					
10	30.5	520	0.25	0.39	77.68	201.0	6129.9					
9	27.5	520	0.21	0.33	55.43	169.8	4669.0					
8	24.5	520	0.16	0.27	36.93	138.6	3395.2					
7	21.5	520	0.11	0.21	22.17	107.4	2308.7					
6	18.5	520	0.07	0.15	11.16	76.2	1409.4					
5	15.5	520	0.02	0.09	3.89	45.0	697.2					
4	12.5	520	-0.03	0.03	0.37	13.8	172.3					
3	9.5	520	-0.07	-0.03	0.58	-17.4	-165.5					
2	6.5	520	-0.12	-0.09	4.55	-48.6	-316.0					
1	3.5	550	-0.17	-0.15	12.96	-84.4	-295.5					
0	0	0		0.00	0.00	0.0	0.0					
sum		**6270**			**2932.93**	**4208.6**	**193678.1**					

Table E.4 Preliminary calculations of 20 storey to determine $\mathbf{H_{cF}}$ – for $\boldsymbol{\beta_F}$ **=40%**

Level	**Height, H_i (m)**	**Mass,m_i (tonne)**	**m_iH_i**	**F_i (relative)**	**V_{Ti} (relative)**	**$M_{OTM,i}$ (relative)**	**$V_{F,i}$ frame**	**$V_{w,i}$ wall**	**$M_{w,i}$ wall (without link beam)**	**$M_{w,i}$ wall (with link beam)**
20	60.5	400	24200	0.074	0.074	**0**	0.4	-0.33	**0**	**0**
19	57.5	520	29900	0.09	0.17	0.22	0.4	-0.23	-0.98	-0.10
18	54.5	520	28340	0.09	0.25	0.72	0.4	-0.15	-1.68	-1.05
17	51.5	520	26780	0.08	0.34	1.48	0.4	-0.06	-2.12	-1.72
16	48.5	520	25220	0.08	0.41	2.49	0.4	0.01	-2.31	-2.14
15	45.5	520	23660	0.07	0.49	3.73	0.4	0.09	-2.27	-2.31
14	42.5	520	22100	0.07	0.55	5.18	0.4	0.15	-2.02	-2.25
13	39.5	520	20540	0.06	0.62	6.84	0.4	0.22	-1.56	-1.97
12	36.5	520	18980	0.06	0.67	8.69	0.4	0.27	-0.91	-1.49
11	33.5	520	17420	0.05	0.73	10.72	0.4	0.33	-0.08	-0.83
10	30.5	520	15860	0.05	0.78	12.90	0.4	0.38	0.90	0.01
9	27.5	520	14300	0.04	0.82	15.23	0.4	0.42	2.03	1.01
8	24.5	520	12740	0.04	0.86	17.69	0.4	0.46	3.29	2.15
7	21.5	520	11180	0.03	0.89	20.27	0.4	0.49	4.67	3.43
6	18.5	520	9620	0.03	0.92	22.96	0.4	0.52	6.16	4.82
5	15.5	520	8060	0.02	0.95	25.73	0.4	0.55	7.73	6.31
4	12.5	520	6500	0.02	0.97	28.57	0.4	0.57	9.37	7.89
3	9.5	520	4940	0.02	0.98	31.48	0.4	0.58	11.08	9.54
2	6.5	520	3380	0.01	0.99	34.43	0.4	0.59	12.83	11.25
1	3.5	550	1925	0.01	1.00	37.41	0.4	0.60	14.61	13.00
0	0	0	0	0.00	1.00	40.91	0.4	0.60	16.71	14.79
sum		10310	325645	1.00						

Table E.5 Design displacement information of 20 story **– without link beam** and **β_F =40%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
20	60.5	400	0.60	1.01	408.2	404.1	24446.2	**0.672**	42.53	7323.37	13.78	9.27
19	57.5	520	0.57	0.95	472.7	495.8	28506.2					
18	54.5	520	0.53	0.90	418.0	466.2	25409.6					
17	51.5	520	0.49	0.84	366.7	436.7	22490.1					
16	48.5	520	0.46	0.78	318.8	407.2	19747.8					
15	45.5	520	0.42	0.73	274.3	377.6	17182.6					
14	42.5	520	0.38	0.67	233.0	348.1	14794.7					
13	39.5	520	0.35	0.61	195.2	318.6	12583.9					
12	36.5	520	0.31	0.56	160.7	289.0	10550.3					
11	33.5	520	0.27	0.50	129.5	259.5	8693.9					
10	30.5	520	0.24	0.44	101.8	230.0	7015.9					
9	27.5	520	0.20	0.39	77.6	200.8	5522.6					
8	24.5	520	0.17	0.33	57.0	172.2	4219.1					
7	21.5	520	0.13	0.28	40.2	144.5	3106.6					
6	18.5	520	0.10	0.23	26.8	118.0	2182.9					
5	15.5	520	0.07	0.18	16.6	93.0	1441.8					
4	12.5	520	0.05	0.13	9.4	69.9	873.4					
3	9.5	520	0.03	0.09	4.6	48.9	464.2					
2	6.5	520	0.01	0.06	1.8	30.3	197.0					
1	3.5	550	0.00	0.03	0.4	15.4	53.8					
0	0	0	0.00	0.00	0.0	0.0	0.0					
sum		10310			3313.2	4925.8	209482.4					

Table E.6 Design displacement information of 20 story **– with link beam** and **β_F =40%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
20	60.5	400	0.56	1.10	480.48	438.4	26522.9	0.745	42.97	7161.27	11.97	8.92
19	57.5	520	0.53	1.04	558.10	538.7	30976.0					
18	54.5	520	0.50	0.98	495.33	507.5	27659.5					
17	51.5	520	0.46	0.92	436.30	476.3	24530.1					
16	48.5	520	0.43	0.86	381.01	445.1	21588.0					
15	45.5	520	0.40	0.80	329.47	413.9	18833.1					
14	42.5	520	0.36	0.74	281.67	382.7	16265.3					
13	39.5	520	0.33	0.68	237.62	351.5	13884.8					
12	36.5	520	0.29	0.62	197.31	320.3	11691.4					
11	33.5	520	0.26	0.56	160.74	289.1	9685.3					
10	30.5	520	0.23	0.50	127.92	257.9	7866.4					
9	27.5	520	0.19	0.44	98.84	226.7	6234.6					
8	24.5	520	0.16	0.38	73.51	195.5	4790.1					
7	21.5	520	0.13	0.32	51.92	164.3	3532.7					
6	18.5	520	0.09	0.26	34.08	133.1	2462.6					
5	15.5	520	0.06	0.20	19.97	101.9	1579.7					
4	12.5	520	0.03	0.14	9.62	70.7	883.9					
3	9.5	520	-0.01	0.08	3.00	39.5	375.4					
2	6.5	520	-0.04	0.02	0.13	8.3	54.0					
1	3.5	550	-0.07	0.01	1.07	24.2	84.7					
0	0	0		0.000	0	0	0					
sum		**10310**			**3,978.1**	**5,337.4**	**229,331.1**					

Table E.7 Preliminary calculations of 20 storey to determine H_{cF} – for β_F **=60%**

Level	Height, H_i (m)	Mass,m_i (tonne)	m_iH_i	F_i (relative)	V_{Ti} (relative)	$M_{OTM,i}$ (relative)	$V_{F,i}$ frame	$V_{w,i}$ wall	$M_{w,i}$ wall (without link beam)	$M_{w,i}$ wall (with link beam)
20	60.5	400	24200	0.07	**0.07**	**0.00**	0.6	-0.53	**0.00**	0.00
19	57.5	520	29900	0.09	0.17	0.22	0.6	-0.43	-1.58	-0.15
18	54.5	520	28340	0.09	0.25	0.72	0.6	-0.35	-2.88	-1.70
17	51.5	520	26780	0.08	0.34	1.48	0.6	-0.26	-3.92	-2.98
16	48.5	520	25220	0.08	0.41	2.49	0.6	-0.19	-4.71	-4.00
15	45.5	520	23660	0.07	0.49	3.73	0.6	-0.11	-5.27	-4.77
14	42.5	520	22100	0.07	0.55	5.18	0.6	-0.05	-5.62	-5.31
13	39.5	520	20540	0.06	0.62	6.84	0.6	0.02	-5.76	-5.63
12	36.5	520	18980	0.06	0.67	8.69	0.6	0.07	-5.71	-5.75
11	33.5	520	17420	0.05	0.73	10.72	0.6	0.13	-5.48	-5.69
10	30.5	520	15860	0.05	0.78	12.90	0.6	0.18	-5.10	-5.45
9	27.5	520	14300	0.04	0.82	15.23	0.6	0.22	-4.57	-5.05
8	24.5	520	12740	0.04	0.86	17.69	0.6	0.26	-3.91	-4.50
7	21.5	520	11180	0.03	0.89	20.27	0.6	0.29	-3.13	-3.83
6	18.5	520	9620	0.03	0.92	22.96	0.6	0.32	-2.24	-3.04
5	15.5	520	8060	0.02	0.95	25.73	0.6	0.35	-1.27	-2.15
4	12.5	520	6500	0.02	0.97	28.57	0.6	0.37	-0.23	-1.17
3	9.5	520	4940	0.02	0.98	31.48	0.6	0.38	0.88	-0.12
2	6.5	520	3380	0.01	0.99	34.43	0.6	0.39	2.03	0.99
1	3.5	550	1925	0.01	1.00	37.41	0.6	0.40	3.21	2.15
0	0	0	0	0.00	1.00	40.91	0.6	0.40	4.61	3.33
sum		10310	325645	1.00						

Table E.8 Design displacement information of 20 story **– without link beam** and **β_F =60%**

Level	Height, H_i (m)	Mass, m_i (tonne)	Δ_{yi} (m)	Δ_{Di} (m)	$m_i\Delta^2_{Di}$	$m_i\Delta_{Di}$	$m_i\Delta_{Di}H_i$	Δ_D (m)	H_e (m)	m_e (tonne)	k_e (MN/m)	V_{base} (MN)
20	60.5	400	0.20	1.13	514.6	453.7	27447.9	**0.767**	41.08	7895.39	11.53	8.84
19	57.5	520	0.19	1.08	603.6	560.3	32214.9					
18	54.5	520	0.18	1.02	541.7	530.7	28924.7					
17	51.5	520	0.17	0.96	483.1	501.2	25811.7					
16	48.5	520	0.16	0.91	427.8	471.7	22875.9					
15	45.5	520	0.15	0.85	375.9	442.1	20117.3					
14	42.5	520	0.14	0.79	327.4	412.6	17535.9					
13	39.5	520	0.13	0.74	282.2	383.1	15131.6					
12	36.5	520	0.12	0.68	240.4	353.5	12904.5					
11	33.5	520	0.11	0.62	201.9	324.0	10854.6					
10	30.5	520	0.10	0.57	166.8	294.5	8981.9					
9	27.5	520	0.08	0.51	135.0	265.0	7286.3					
8	24.5	520	0.07	0.45	106.6	235.4	5768.0					
7	21.5	520	0.06	0.40	81.5	205.9	4426.8					
6	18.5	520	0.05	0.34	59.8	176.4	3262.8					
5	15.5	520	0.04	0.28	41.5	146.8	2276.0					
4	12.5	520	0.03	0.23	26.5	117.3	1466.3					
3	9.5	520	0.02	0.17	14.8	87.8	833.9					
2	6.5	520	0.01	0.11	6.6	58.4	379.8					
1	3.5	550	0.00	0.06	1.9	31.9	111.7					
0	0	0	0.00	0.00	0.0	0.0	0.0					
sum		10310			4639.5	6052.3	248612.4					

Table E.9 Design displacement information of 20 story **– with link beam** and **β_F =60%**

Level	**Height, H_i (m)**	**Mass, m_i (tonne)**	**Δ_{yi} (m)**	**Δ_{Di} (m)**	**$m_i\Delta^2_{Di}$**	**$m_i\Delta_{Di}$**	**$m_i\Delta_{Di}H_i$**	**Δ_D (m)**	**H_e (m)**	**m_e (tonne)**	**k_e (MN/m)**	**V_{base} (MN)**
20	60.5	400	0.19	1.20	575.7	479.9	29032.8	0.811	41.07	7899.36	10.55	8.56
19	57.5	520	0.18	1.14	675.4	592.6	34077.0					
18	54.5	520	0.17	1.08	606.2	561.4	30598.7					
17	51.5	520	0.16	1.02	540.7	530.2	27307.6					
16	48.5	520	0.15	0.96	478.9	499.0	24203.6					
15	45.5	520	0.14	0.90	420.9	467.8	21286.9					
14	42.5	520	0.13	0.84	366.7	436.6	18557.4					
13	39.5	520	0.12	0.78	316.1	405.4	16015.0					
12	36.5	520	0.11	0.72	269.3	374.2	13659.9					
11	33.5	520	0.10	0.66	226.3	343.0	11492.0					
10	30.5	520	0.09	0.60	187.0	311.8	9511.3					
9	27.5	520	0.08	0.54	151.5	280.6	7717.7					
8	24.5	520	0.07	0.48	119.7	249.4	6111.4					
7	21.5	520	0.06	0.42	91.6	218.2	4692.3					
6	18.5	520	0.05	0.36	67.3	187.0	3460.3					
5	15.5	520	0.04	0.30	46.7	155.8	2415.6					
4	12.5	520	0.03	0.24	29.9	124.6	1558.1					
3	9.5	520	0.02	0.18	16.8	93.4	887.7					
2	6.5	520	0.01	0.12	7.5	62.2	404.6					
1	3.5	550	0.00	0.06	2.0	32.8	114.9					
0	0	0		0.00	0.0	0.0	0.0					
sum		**10310**			**5196.1**	**6406.7**	**263104.8**					

Appendix - F
Force Based Method Calculation

Storey	**H_i (m)**	**G+4**		**G+8**		**G+12**		**G+16**		**G+20**	
		m_i (tonne)	**F_i (KN)**	**m_i (tonne)**	**F_i (KN)**	**m_i (tonne)**	**F_i (KN)**	**m_i (tonne)**	**F_i (KN)**	**m_i (tonne)**	**F_i (KN)**
1	3.5	355	318.56	400	153.66	440	95.24	480	71.45	550	55.29
2	6.5	352	586.60	385	274.68	420	168.84	460	127.16	520	97.08
3	9.5	352	857.34	385	401.45	420	246.77	460	154.88	520	141.89
4	12.5	270	865.29	385	528.22	420	324.70	460	244.55	520	224.04
5	15.5			385	654.99	420	402.62	460	303.24	520	277.81
6	18.5			385	781.77	420	480.55	460	361.93	520	276.31
7	21.5			385	908.54	420	579.16	460	420.62	520	321.12
8	24.5			320	860.52	420	659.98	460	559.20	520	439.11
9	27.5					420	740.79	460	627.67	520	496.99
10	30.5					420	821.60	460	696.14	520	546.65
11	33.5					420	902.42	460	764.61	520	600.42
12	36.5					380	889.59	460	833.09	520	659.64
13	39.5							460	901.56	520	713.86
14	42.5							460	970.03	520	768.08
15	45.5							460	1038.51	520	815.50
16	48.5							350	842.27	520	1014.14
17	51.5									520	1076.87
18	54.5									520	1139.60
19	57.5									520	1202.34
20	60.5									400	973.13
	m (Sum)	**1329**		**3030**		**5020**		**7270**		**10310**	
	F_b (MN)	2.52		4.59		5.59		8.53		11.32	
	F_t (MN)	0.97		1.77		2.276		3.29		4.36	