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COMPARISON OF THE EFFECTS OF THE POSITION OF SHEAR WALLS IN HIGH- RISE BUILDINGS ON THE HORIZONTAL STIFFNESS OF THE BUILDING UNDER THE EFFECT OF HORIZONTAL WIND LOADS

Sinan HAMDI

Master's Science

Supervisor

Prof. Dr. Zeki HASGÜR

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Sinan Raad Ahmed HAMDI

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The thesis dissertation titled "Comparison of the effects of the position of shear walls in high- rise buildings on the horizontal stiffness of the building under the effect of horizontal wind loads" prepared by SINAN RAAD AHMED HAMDI and submitted 10/3/2023 has been **accepted unanimously** for the degree of Master of Science in the Department of Civil Engineering.

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Prof. Dr. Zeki HASGÜR

Supervisor

Thesis Defense Committee Members:

Academic Title - First/Last Name	Department, University	___________________
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ABSTRACT

COMPARISON OF THE EFFECTS OF THE POSITION OF SHEAR WALLS IN HIGH- RISE BUILDINGS ON THE HORIZONTAL STIFFNESS OF THE BUILDING UNDER THE EFFECT OF HORIZONTAL WIND LOADS

HAMDI, Sinan

M.Sc.Civil Engineering, Altınbaş University,

Supervisor: Prof. Dr. Zeki HASGÜR

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In developing countries, the demand for high-rise buildings is increasing due to urbanization and population growth. Also, there is more demand for "tall buildings" as there is less land for buildings. It is imperative for these buildings to withstand lateral loads and prevent excessive displacement and swaying of the building. Buildings must be strong enough to withstand vertical forces and rigid enough to withstand lateral forces. In making a building resistant to lateral loads, the primary consideration is to ensure the location, dimensions, and symmetry of the shear wall, and to construct shear walls around the building and around a central core. The aim of this study is to analyse the behaviour of a 20-storey hotel building under the effect of equivalent earthquake force according to the Turkish Building Earthquake Code of 2018, by using the Muto method in the analysis of the shear-frame interaction with Group Loads, by using the "Force Method", and to analyses the results in Eurocode at 30 m/sec. To compare the effects of wind force obtained using (EN 1991-1-4) The results found that the forces that have the greatest impact on the building are the seismic effects due to the seismic activity in the area where the building will be built.

Keywords: Wind Load, Earthquake, Shear Wall, Stiffness, and Muto Method.

TABLE OF CONTENTS

Pages

LIST OF TABLES

Pages

LIST OF FIGURES

Pages

1. INTRODUCTION

1.1 GENERAL

In developing nations, demand for high-rise structures is rising because of urbanization and population growth. In addition, the demand for "Tall Building" had risen due to the restricted amount of land available for housing construction. [1]

According to Bungale S. Taranath (2010), the term "tall building" does not depend on the specific height of the building or number of the stories to classify the structure as a tall building or otherwise. [2] The Council on Tall Buildings and Urban Habitat (CTBUH) developed the international standards for measuring and defining tall buildings with height less than 300 meters as "tall", "super-tall" with height more than 300 meters, and "mega tall" with height more than 600 meters.[3]

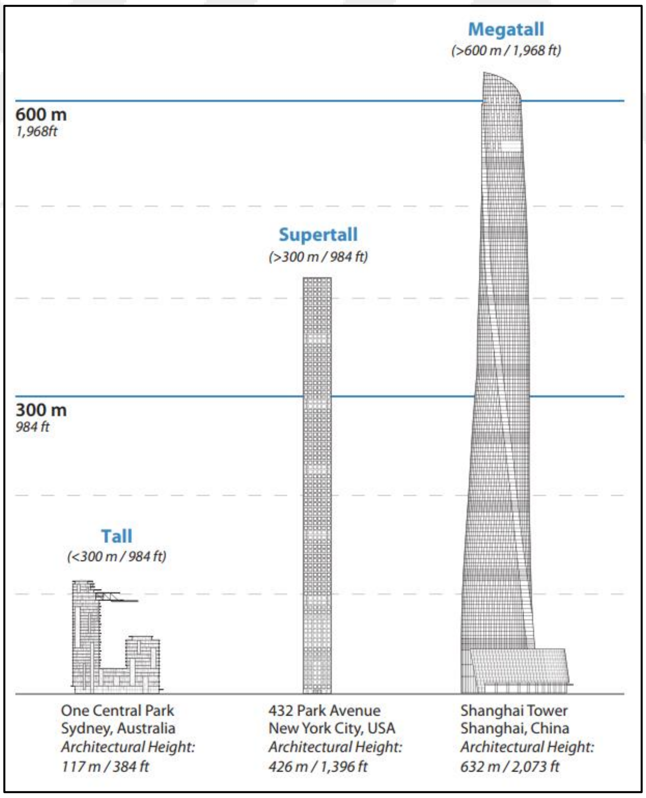


Figure 1.1: Height Criteria for Tall Buildings as Defined by the CTBUH [3].

The construction of the Home Insurance Building in Chicago heralded the beginning of a new period for tall structures. The “skeleton frame” method was utilized in the construction of the first skyscraper, which was a 12-story office building designed by William Le Jenney and built in 1885. The structure stood until 1931 before being dismantled.

Figure 1.2: Home Insurance Building (En. Wikipedia.Org).

Because of the skeleton structure, a substantial amount of masonry mass was not required to sustain the tall buildings. In addition to that, it made it possible to have a basic shear wall system and internal daylighting ,[4], [5] which was followed sometime later by the development of bending moments and shear forces in the frame parts and joints. Because of the hard beam–column connections, a moment frame is incapable of moving laterally without bending either the beams or the columns, depending on the geometry of the connection; this approach is referred to as a “Moment-resisting frame” .[6] These two methods make it feasible for structures to be both higher and lighter.

Supporting gravity loads is the primary objective of all different kinds of structural models that are employed in the construction of structures. The most common types of loads that are caused by gravity's impacts are the dead load, the living load, the rain load, and the snow load. In addition to being subjected to vertical loads, buildings are also sensitive to lateral forces, which may be caused by things like earthquakes and winds. The effects of lateral load on high-rise structures are highly considerable and increase rapidly with height. These buildings were confronted with a new challenge to resist lateral load and minimize the swaying of the building. Therefore, it is necessary for buildings to have sufficient strength to withstand vertical forces in addition to the necessary stiffness to resist lateral loads [5], [7], [8].

1.2 LATERAL FORCE-RESISTING SYSTEMS

There are a multitude of structural solutions available in both steel and concrete to meet the various architectural requirements. Due to the low cost of computers and software, structures are analyzed on a large-scale using computers today. The proper design of a structure requires an understanding of the structural system's behavior. Furthermore, the structural system's behavior must be understood for correct design. Even with the most advanced technologies, a designer who rushes through the modeling process is more likely to produce mistakes in the resulting design than not. The executing engineer also needs to comprehend the behavior so that he is aware of the crucial actions in the structure and may take additional safety measures during the building process. [9]

Fazlur Khan created "Heights for Structural Systems" diagrams in 1969 to categorize structural systems for tall buildings based on height, taking efficiency into account. [10]

The structural systems of tall buildings may be categorized into two separate groups: interior structure such as: (Moment-resisting frames, shear walls and core-supported outrigger structure) and exterior structure such as: (Super frame, Exoskeleton, Diagrid, Space truss and Tube structure). The distribution of major lateral load-resisting system components across the building determines this categorization. [11]

Moment-resisting frames and shear walls are interior structural lateral load-resisting solutions. These systems are generally planar constructions in two main orthogonal directions and may interact as a combined system. The core-supported outrigger structure. extensively utilized for supertall structures, is another essential technology in this group [11] These are vertical components that transmit lateral loads, such as wind and seismic, to the foundation. They prevent a building from collapsing. [12]

I will discuss below some of the most important methods that contribute to resisting the lateral forces of structure:

1.2.1 Moment-Resisting Frames

Rigid frame action, or the generation of bending moments and shear forces in the frame components and joints, is the primary mechanism for providing resistance to lateral forces. A moment frame cannot move laterally without bending its beams or columns, depending on the geometry of the connection, due to the rigidity of the beam-column connections. In this case, the lateral stiffness and strength of the whole frame come mostly from the bending rigidity and strength of the frame components. [6]

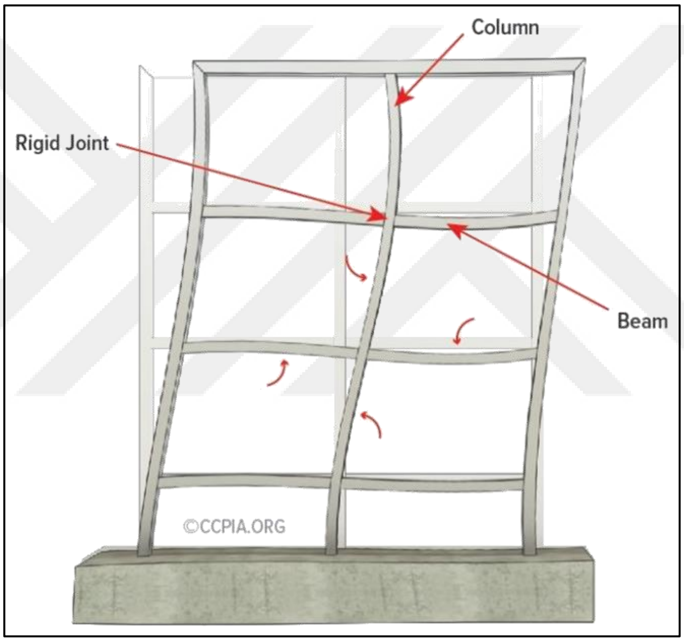


Figure 1.3: Moment-Resisting Frames (CCPIA.ORG).

According to (Seismic Evaluation for Building Handbook, 1988) Identifying the building's frame type is unnecessary. Ductile moment frames must meet the following minimum requirements:

a. They are strong enough to withstand earthquakes.
b. They are stiff enough to stop movement between floors.
c. Beam-column joints are flexible enough to handle the rotations they go through.
d. Plastic hinges can form between elements.

e. The strong column/weak beam theory predicts that beams will generate hinges before thecolumns in various points around the building.

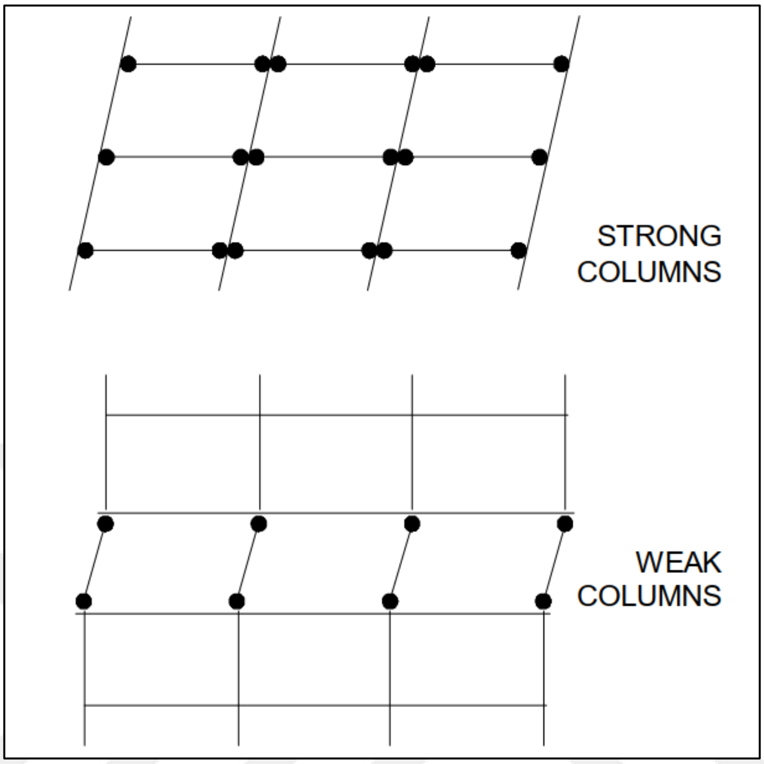


Figure 1.4: Formation of Plastic Hinge [12].

1.2.2 Shear Wall

Shear walls are the primary vertical structural components that must deal with both gravity and lateral force. The number of stories, age, and need for thermal insulation all play a role in determining the optimal wall thickness, which can range anywhere from 40 mm to 500 mm. Moreover, the behavior is affected by factors such as the material, wall Thickness, wall length, and wall location in the building frame. Typically, these walls continue up the entire height of the structure. [13], [14]

For structures to minimize the impacts of twist, shear walls should be placed in symmetrical positions in the floor design. Placement symmetry might be along one or both of the plan's axes. The location of shear walls around the building's external perimeter is optimal; this configuration improves the structure's resistance to twisting forces. [15]

It is necessary to take this into consideration when constructing shear walls; they are built parallel to the direction of the lateral forces acting on the building. Therefore, the shear walls can perform their purpose in resisting these lateral forces, and the wall develops in-plane shear and flexural forces. Otherwise, the shear wall has little contribution to resisting lateral loads on the building, and the wall is subjected to out-of-plane forces that tend to detach it from the rest of the building's structure. [12]

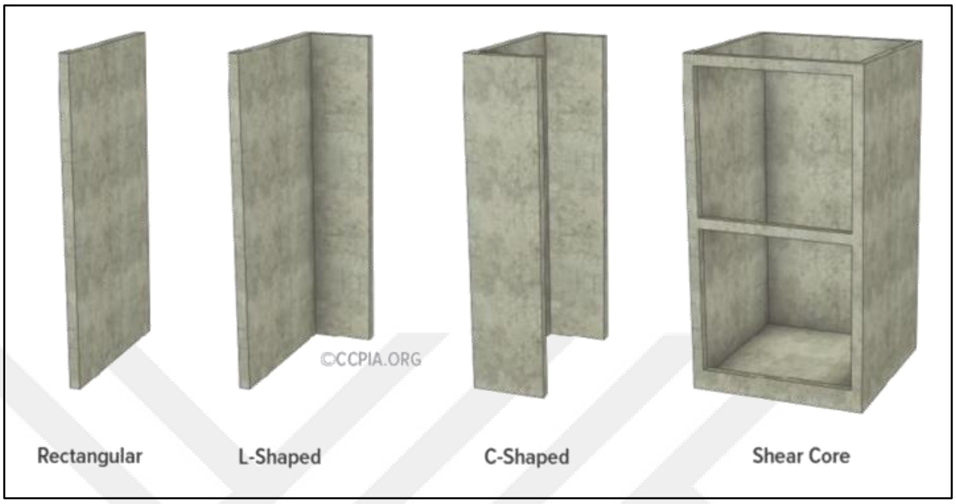


Figure 1.5: Shear wall (CCPIA.ORG).

1.2.3 Braced Frames

In steel construction, braced frames are popular. Steel beams and cables arranged in a diagonal and/or triangular pattern are used to counteract lateral loads. A structure's strength can come from either vertical or horizontal bracing. Lateral forces are transferred to the ground by vertical bracing between the structural columns. Lateral forces are transmitted from horizontal bracing at each story or the roof to the vertical bracing, and from there to the foundation. However, the floor system often serves as a sufficient diaphragm, therefore steel bracing is unnecessary. Multiple-story structures between three and five stories in height can benefit from braced frames. Concentric and eccentric bracing are the two most common types of braced frames as shown in figure (1.5). [12], [16] To create a truss, the triangular parts of a concentrically braced frame are usually joined at the ends of other framing components (joints). The X-brace, the chevron brace (an inverted V-brace), and the single diagonal brace are all frequent arrangements. The diagonal braces used in eccentrically braced systems have one or both of their ends placed at a location that is not in the geometric center of the part they are bracing. The "structural fuse region," which is the space between the offset bracing, is meant to absorb and disperse a great deal of earthquake-induced force. [16]

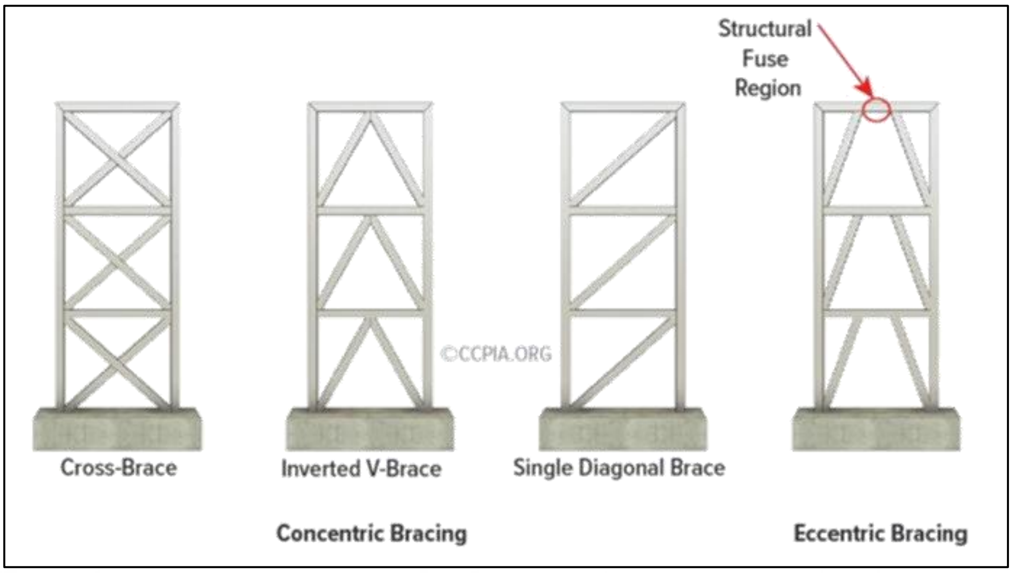


Figure 1.6: Concentric and Eccentric Bracing [16].

1.3 OBJECTIVE OF THE WORK

The purpose of this work is to investigate and analyze the behavior of the 20-storey hotel building by using the turkish code and create a comparison between the equivalent earthquake force and the real earthquake force with the wind force that obtained by using the eurocode (EN 1991-1-4).

Following this introductory chapter, chapter two tries to review the literature related to the subject.

Chapter three includes the methodology and procedure to calculate the D-value using muto method and obtain the wind force acting on a building by using the eurocode (EN 1991-1-4) [17] and then try to distribute it to the floor level using the "Equivalent Seismic Load method" mentioned in the Turkish code (TBC 2007) and obtain the real base shear force that results from the earthquake, study the impact of these forces on the building behavior sequentially, and compare them.

Chapter four includes a case study with analysis and results by using the excel microsoft office software. tables and figures have been made to show the results.
Finally, conclusions and recommendations have been summarized in chapter five.

2. REVIEW OF LITERATURE

As a building's height increases, the importance of its structural stiffness grows, creating new challenges in the race to new heights in design. Because lateral loads are so preponderant, buildings keep becoming taller and taller, subjecting themselves to all sorts of new loading effects and values. Every designer has the challenge of providing sufficient protection from the impacts of lateral loads such as wind loads and seismic forces. a resistance to lateral forces that is strong and stable.

A. Rahman, S. F. Fancy, and S. A. Bobby, (2012): “Analysis of drift due to wind loads and earthquake loads on tall structures by programming language c” was presented in a paper. (C++) programming was utilized to examine the drift caused by earthquakes and winds on tall buildings and compare it. He used three different forms of tall buildings: rigid frames, coupled shear walls, and wall frames. To ensure the safety of people, equipment, and the building itself, designers must take into account the building's height, weight, and stability. Limit stresses provide sufficient strength, while drift limitations in the 0.002H to 0.001H range maintain optimal serviceability. On the other hand, a significant degree of protection against buckling and P-delta effects is what ensures stability. At last, “every tall structure should include the drift due to earthquake load as well as wind load”. [18]

M.D. KEVADKAR, P.B. KODAG (2013): “Lateral Load Analysis of R.C.C. Building” was presented in a paper. In high seismic zones, the lateral load system was calculated using E-TABS software, and the three models (framed, braced, and shear wall) were analyzed. increased structural stiffness and the maximum inter-story drift, lateral displacement, and demand capacity of R.C.C buildings are found to be reduced using the X-bracing of steel system as compared to the shear wall system. [19]

Varsha R. Harne (2014): “Comparative Study of Strength of RC Shear Wall at Different Locations on Multi-Story Residential Building” was presented in a paper. STAAD Pro was used to calculate the earthquake load for a six-story R.C.C. structure, and then four models (framed, L-shaped shear walls at corner of building, I-shaped shear walls at center of building, and I-shaped shear wall along periphery of building) were studied to find out how a change in the location of the shear walls would affect the strength of the building. He observed that the structure with an L type shear wall was more efficient than all other types of shear walls.[20]

Anuj Chandiwala (2012): “Earthquake Analysis of Building Configuration with Different Position of Shear Wall” was presented in a paper. Using SAP2000 software, a moment is created at a specific column by considering a variety of lateral load resistant structural systems, floor numbers, and earthquake shear wall locations. The best result is obtained from the shear wall located at the end of the "L" section. Furthermore, “the end portion of the flange always oscillates more during earthquakes,” which is the main cause. Here, the shear walls acts as a direct wall against its terminal oscillation, reducing the building's bending moment. [21]

Shruti Badami, M. R. Suresh (2014): “A Study on Behavior of Structural Systems for Tall Buildings Subjected to Lateral Loads” was presented in a paper. ETABS software is used to study how different structural systems, like a "Rigid Frame," "Shear Wall/Central Core," "Wall Frame Interaction," and "Outrigger," respond to the forces of gravity and wind. This makes it possible to choose the best building structure for a certain building height. The period, storey displacement, drift, lateral displacement, base shear values, and core moments all affect the structural effectiveness. The most important things observed about the Shear wall and central core system were that it was the most cost-effective method, that the overturning moment in the core structures was reduced, and that, compared to other systems, its flexural stiffness was much higher. [22]

U. Waseem and N. Aadil, (2014): “Analysis of Lateral Forces on A Multi-Storied Frame Structure Using D-Value Method and Its Comparison with Finite Element Method” was presented in a paper. The D-value method and ETABS software were compared to design a multi-storied moment resistant frame structure in Karachi. The design moments are the most important thing to the designers, so they compare the lateral force moments on each joint. The D-value method is easy to use and is only 20% different from the finite element method (ETABS) for analyzing lateral forces on superstructures. For analyzing or confirming moments from CAD software, the D-value method is suggested, but the comparison data may be different for structures that aren't moment-resistant frame structures. [23]

A. M. Aly and S. Abburu, (2015): “On the design of high-rise buildings for multihazard: Fundamental differences between wind and earthquake demand” was presented in a paper.

Using MATLAB software, this study dynamically analyzed two high-rise structures (one 54 stories tall and one 76 stories tall), and the results showed that earthquake loads ignite higher modes that result in less inter-story drift but greater accelerations for shorter periods. Concerns about convenience and safety may arise from wind-induced accelerations, and problems with security may result from large inter-story drifts. Additionally, the findings suggest that tall, narrow structures with wind designs may be resistant to mild earthquake loads, at least in terms of the primary force resisting system. Furthermore, due to increased floor acceleration, non-structural components may pose a considerable proportion of a structure's loss hazard. As a result, it is suggested that tall structures employ suitable damping or control mechanisms as a means of protection against the effects of many hazards. [24]

Anupam Rajmani, Prof Priyabrata Guha (2015): "Analysis of Wind & Earthquake Load For Different Shapes of High-Rise Building" was presented in a paper. this study looks at four different types of building shapes: circular, rectangular, square, and triangular. This is because the shape of a building and changes to its architecture are very important and effective ways to reduce movement caused by wind and earthquakes by changing how air flows around a building. It has been shown that rectangular buildings, especially those between 30 and 45 stories tall, can withstand the strongest earthquakes and winds with the least amount of damage. [25]

M. Aejaz et al., (2015): "Study on Effect of Wind Load And Earthquake Load On Multistorey R.C. Framed Buildings" was presented in a paper. Analyzing and designing for lateral loads (wind load and earthquake loads) on medium and tall buildings with ETABS-2013. Case studies include a 16-story and 31-story symmetrical building with and without shear walls. The influence of shear walls has been included into values of the forces, shear, drift of storey, and displacements. Forces on a structure must be defined for it to be designed to withstand wind and earthquake loads. When shear walls are placed at the building's corners, the values of story drift are seen to decrease. A building's design should take into account the essential forces of wind and earthquake from opposite directions. [26]

N. Uysal, P. Usta, and Ö. Bozdağ, (2022): "Structural Resistance of a Reinforced Concrete Building Under Earthquake and Wind Loads in Isparta and Burdur Region" was presented in a paper. The implications of the irregularities were studied in accordance with the 2018

Turkish Building Earthquake Code (TBEC). The effects of earthquake and wind loads on a reference building with a non-uniform setback in the vertical direction were examined. The structural analysis was performed using Etabs. Seismic analysis was performed using the response spectrum method, while wind load analysis was performed using TS498. Therefore, it appears that earthquake- and wind-resistant reinforced concrete structures are secure. However, it is important to keep in mind that nonstructural systems may represent a high percentage of loss exposure of buildings to earthquakes and winds, even if story drift ratios in buildings may be relatively small with no significant apparent issues for the main force resisting system of the structure. This means that designing with wind load in mind is essential, and that regular design procedures that account for wind and seismic loads may greatly help mitigate their effects. [27]

3. THE METHADOLOGY OF RESEARCH

3.1 TOTAL WEIGHT OF THE STRUCTURE'S ELEMENTS

To calculate the total weight of the structure, we need to calculate the weight of each element of the building as shown below:

3.1.1 Slab Weight

Obtain the total floor slab's weight due to the self-weight combined with finishing weight.

$$W_{element} = \text{element area} \times \text{Thickness} \times \gamma_{\text{of element}} \tag{3.1}$$

3.1.2 Beams Weight

Calculate the beam's weight, considering that the lengths of the beams do not change, and the beams are split according to the sectional area after subtracting the columns from the calculations.

$$W_{\text{Beam Number}} = \text{Beam section Area} \times \text{Length of Span} \times \gamma_{concrete} \times \text{Number} \tag{3.2}$$

3.1.3 Columns Weight

Calculate the column's weight in each story by multiplying the column cross area by the height of the column subtracting the beam's depth by the number of columns on the floor and the unit weight of concrete.

$$W_{\text{Column section}} = \text{Column area} \times \text{Height}_{\text{(column-Beam)}} \times \text{Number} \times \gamma_{concrete} \tag{3.3}$$

3.1.4 Shear Wall and Core Weight

Calculate the shear and core wall weight in each story by multiplying the shear wall area the height of the story by unit weight of concrete by number of shear wall or core wall.

$$W_{\text{SW-Position}} = \text{Shear Wall area} \times \text{Height of story} \times \gamma_{concrete} \times \text{Number} \tag{3.4}$$

3.1.5 Exterior Masonry Walls Weight

Calculate the exterior masonry walls weight on each story by multiplying the length of wall span by the height of the wall by unit weight of concrete by number.

The weights of the brick walls will only be computed for the external walls because there are no architectural drawings to depend on.

$$W_{EMW} = \text{Length of span} \times (h_{\text{of story}} - h_{\text{of beam}}) \times \gamma_{masonry} \times \text{Number} \quad (3.5)$$

After determining the dead loads acting on the floor for all elements of the structure from the above steps, the total floor weight can be computed by applying the equation (3-6) by using the typical live Load values for hotel building is 2 kN/m^2 (TBEC 2007):

$$w_i = g_i + nq_i \quad (3.6)$$

Where:

w: Total weight of story.

g: Dead Load.

q: Live Load.

n: Live Load Participation Factor, can be found in table (3-1) below:

Table 3.1: Live Load Participation Factor (N).

Purpose of Occupancy of Building	n
Depot, warehouse, etc.	0.80
School, dormitory, sport facility, cinema, theatre, concert hall, car park, restaurant, shop, etc.	0.60
Residence, office, hotel, hospital, etc.	0.30

3.2 ∑D OF STORY IN BOTH DIRECTIONS

By using Muto method, [28] the procedure below will used to calculate the ∑D for structure in both direction (Y and X), respectively:

3.2.1 Calculation of Moment of Inertia of Structure Member

Moment of inertia for all rectangular shape structure member can be calculated by equation:

$$I = \frac{b \times h^3}{12} \tag{3.7}$$

3.2.2 Rigidity of Columns (Dci)

a. Calculation of stiffness of columns (K_c).

To calculate K_c (dm^4/m) by using the following equation:

$$K_c = \frac{I_c}{h} \tag{3.8}$$

b. Calculation of stiffness of beams (K_b).

To calculate K_b (dm^4/m) by using the following equation:

$$K_b = \frac{I_b}{L} \tag{3.9}$$

c. Calculation of K' and a value.

K' value depended on the beam's rigidity connected to the column and passion of column:

For normal and top story:

$$\acute{K} = \frac{k1+k2+k3+k4}{2 \times Kc} \tag{3.10}$$

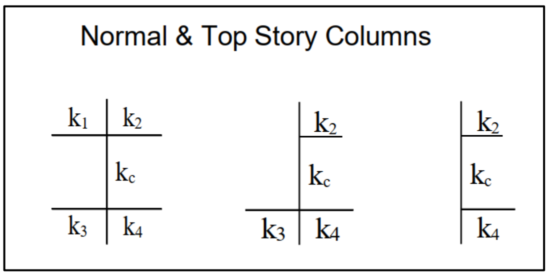


Figure 3.1: Calculation K' For Normal and Top Story.

For bottom story:

$$Ḱ = \frac{k1+k2}{Kc} \tag{3.11}$$

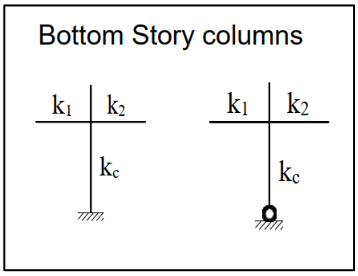


Figure 3.2: Calculation K' for Bottom Story.

The (a) value depended on the footing connection type:

Story column:

$$\mathbf{a} = \frac{Ḱ}{2+Ḱ} \tag{3.12}$$

Rigid column-footing connection:

$$\mathbf{a} = \frac{0.5+Ḱ}{2+Ḱ} \tag{3.13}$$

Pinned column-footing connection:

$$\mathbf{a} = \frac{0.5\times Ḱ}{1+2Ḱ} \tag{3.14}$$

d. Calculation of rigidity of columns (D_c).

By using the following equation to calculate the D_c:

$$D_c = a \times K_c \times 12 \tag{3.15}$$

By using the following equation to calculate the $\sum D_c$:

$$\sum D_c = D_{ci} \times \text{Number of columns} \tag{3.16}$$

3.2.3 Fictive Shear Stiffness for The Coupled Beam (DFi)

To calculate the D_{fi} value for end of coupled beam perpendicular (out of plan) to lateral load direction acting on the building direction it should be taken into consideration as column widths.

$$Lu \geq \max\,[0.2\ L_w\ ,2\ B_w] \tag{3.17}$$

Obtain D_c for the considered column with dimension (L_u (m) × B_w (m)) by using muto method, from equation (3.7) to (3.16).

To calculate the D_{fi} value for end of coupled beam parallel (in plan) to lateral load direction acting on the building depended on type of connection:

a. A coupled beam connected between two shear walls.

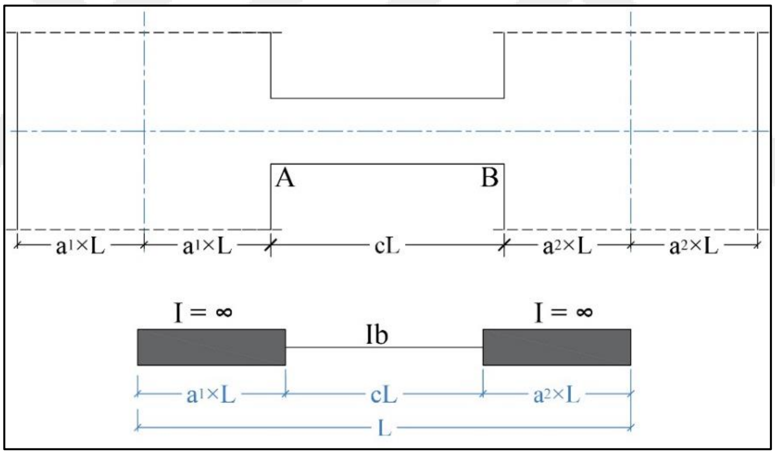


Figure 3.3: Coupled Beam Connected Between Two Shear Walls.

Shear wall rectifiers (R_A, R_B), these are the moments that occur as a result of the unit rotation of the wall axis. $R_A = R_B$ because the walls are symmetrical**:**

$$R_A = \frac{6 \times E \times I_{cb}}{c^3 \times L} \tag{3.18}$$

With the assumption that the connecting beams may crack before other elements during an earthquake, the moment of inertia of a cracked section is reduced**:**

$$I_{cb,cr} = I_{cb} \times 0.5 \tag{3.19}$$

Taking into account the shear deformations in the coupled beams, multiply the equation 3-16 by Kβ.

The shear deformations in the coupled beams (Kβ):

$$K\beta = \frac{\mathbf{1}}{\mathbf{1+4\times\beta}} \tag{3.20}$$

$$\beta = \frac{\mathbf{3\times IcouB}}{\mathbf{(cl)^2\times A'}} \times \frac{\mathbf{E}}{\mathbf{G}} \tag{3.21}$$

Where:

E/G = 2.5 (According to TS 500).

A': the modified beam cross section area = shape factor × area coupled beam

Shape factor of rectangular shape = 5/6

b. Coupled beam connected between shear wall and column.

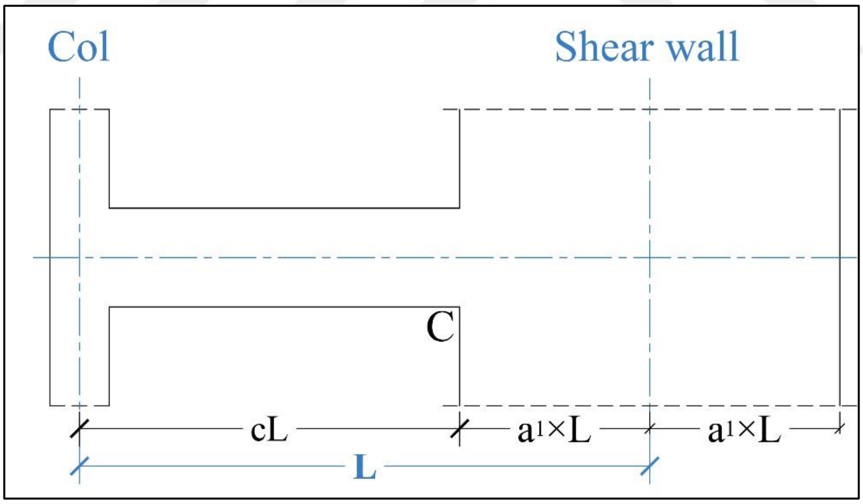


Figure 0.4: Coupled Beam Connected Between Shear Wall and Column.

In this type of connecting beam, the rotation of the junction (col) where the column is positioned is crucial. The stiffness ratio of the beams and columns that join controls rotation at this location. The average stuffiness ratio coefficient is indicated by (rm):

$$\mathbf{r_m} = \frac{(\mathbf{r_o}+\mathbf{r_u})}{\mathbf{2}} \tag{3.22}$$

$$\mathbf{r_o} = \frac{\mathbf{K_c}}{\sum_{\mathbf{up}}(\mathbf{K_c}+\mathbf{K_b})} \tag{3.23}$$

$$\mathbf{r_u} = \frac{\mathbf{K_c}}{\sum_{\mathbf{low}}(\mathbf{K_c}+\mathbf{K_b})} \tag{3.24}$$

The moment of inertia of the connecting beam is raised by the square of the net span coefficient (c); consequently, the beam's fictive stiffness (k_{bf}) is as follows:

$$\boldsymbol{k_{bf}} = \frac{\left(\frac{I_{cb}}{c^2}\right)}{L_{span}} \tag{3.25}$$

The shear wall rectifier is depending on the r_m coefficient:

$$\text{If } 0 < \mathbf{r_m} < \mathbf{2/3} \rightarrow \ \mathbf{R_c} = \frac{(\mathbf{3\times E\times I_{b,cr}})}{\mathbf{c^3\times L}} \times [\tfrac{\mathbf{3}}{\mathbf{2}} \times (3+2 \times c) \times \ \mathbf{r_m} + 1] \tag{3.26}$$

$$\text{If } \ \boldsymbol{r_m} > \mathbf{2/3} \quad \rightarrow \ \mathbf{R_c} = \frac{(\mathbf{6\times E\times I_{b,cr}})}{\mathbf{c^3\times L}} \times [\mathbf{2+c}] \tag{3.27}$$

Taking into account the shear deformations in the coupled beams multiply the equation 3.30 or 3.31 (depended on r_m value) with Kβ.

Fictive Shear stiffness D_{fi} (dm^4/m) is calculated as follows for one connecting beam end:

Top floor: D_{fn} = Rectifier value × 1.5

Intermediate floors: D_{fi} = Rectifiers value (3.28)

Bottom floor: Df1 = Rectifier value × 2

Calculate the summation of fictive shear stiffness for shear wall and core in each story:

$\sum D_{fi}$ = Fictive shear stiffness × number of beams end (3.29)

3.2.3.1 Summation of rigidity for each floor

The summation of rigidity of story is combined of rigidity of column, rigidity of considered column and fictive shear stiffness of coupled beam as follow:

$$\sum D_i = \sum D_{ci} + \sum D_{fci} + \sum D_{fi\,(\text{Shear wall connection})} + \sum D_{fi\,(\text{Core connection})} \quad (3.30)$$

3.2.3.2 The rigidity of story ∑Dij (Kn/M)

The shear forces acting on the frames and the total D must be employed by converting them to their real values. E = 1 was previously used. As a result, (E/hi2) values are multiplied by D, with the modulus of elasticity of concrete varying depending on concrete class (TS 500).

$$\sum D_{ij} = \frac{\mathbf{E_c}}{\mathbf{h^2}} \times \sum D_i \quad (3.31)$$

3.3 CALCULATION OF WIND FORCE (EUROCODE DISTRIBUTED) AND IMPACT ON STRUCTURE (CASE 1)

Calculate the wind force acting in building surface according to (EN 1991-1-4 (2005) (English): Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions) in both direction (Y and X), respectively.

3.3.1 Calculation of Wind Forces on the Structure.

3.3.1.1 Terrain categories and parameters.

By assuming the terrain category and obtain the terrain parameters from table below:

Table 3.2: Terrain Categories and Terrain Parameters (Table 4.1 in EN 1991-1-4).

Terrain category	Z_o m	Z_{min} m
0 Sea or coastal area exposed to the open sea	0.003	1
I Lakes or flat and horizontal area with negligible vegetation and without obstacles.	0.01	1
II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights.	0.05	2
III Area with regular cover of vegetation or buildings or with isolated obstacles with separation of maximum 20 obstacles heights. (Such as villages, suburban terrain, permanent forest)	0.3	5
IV Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15m.	1.0	10

3.3.1.2 The mean wind–variation with height (Vm(Z))

The terrain roughness and orography and the basic wind velocity (V_b) are reliable values to obtain the mean wind velocity ($V_{m(z)}$) at a height z above the terrain:

$$V_{m(Z)} = Cr_{(Z)} \times Co_{(Z)} \times V_b \qquad (3.32)$$

Where:

$V_{m(Z)}$: in (m/sec).

$Cr_{(Z)}$: The roughness factor.

$C_{o(Z)}$: The orography factor, taken as 1.0 (The building construction in flat surface)

V_b: The basic wind velocity.

Terrain roughness:

Equation (3-33) shows the recommended method for calculating the roughness factor at height z, which is based on a logarithmic velocity profile:

$$C_{r(z)} = Kr \times Ln\left(\frac{z}{zo}\right) \quad \text{for} \quad z_{min} \leq z \leq z_{max} \qquad (3.33)$$

$$C_{r(z)} = C_r(z_{min}) \quad \text{for} \quad z \leq z_{min}$$

Where:

K_r: Terrain factor depending on the roughness length Z_o.

$$K_r = 0.19 \times \left(\frac{z_o}{z_{o,II}}\right)^{0.07} \qquad (3.34)$$

3.3.1.3 The wind turbulence (Iv(Z))

The wind turbulence (Iv(z)):

$$I_{v(z)} = \frac{\sigma_v}{V_{m(z)}} \quad \text{for} \quad z_{min} \leq z \leq z_{max} \qquad (3.35)$$

$$I_{v(z)} = I_v(z_{min}) \quad \text{for} \quad z \leq z_{min}$$

Where:

σ_v = The standard deviation of the turbulence, may be calculate using the following equation:

$$\sigma_v = K_r \times V_b \times k_I \quad (3.36)$$

Where:

k_I : The turbulence factor, and recommended value is 1.

3.3.1.4 The peak velocity pressure

The peak velocity pressure qp(z) at height z should be calculated in (kN/m^2) unit, taking into account mean and short-term velocity changes:

$$q_{p(z)} = [1+ (7\times I_{v(z)})] \times (\frac{1}{2}) \times \rho \times V_{m(z)}{}^2 \quad (3.37)$$

Where:

ρ: The air density, the recommended value is 0.00125 KNm/s^2

3.3.1.5 Wind forces on surface

The wind force F_w acting on a structure may be simply calculated using equation (3-38):

$$F_w = c_s c_d \times c_f \times A_{ref} \times q_{p(ze)} \quad (3.38)$$

Where:

$c_s c_d$: The structural factor.

c_f: The force coefficient for the structure.

A_{ref}: The reference area of the structure.

Calculation of the Force coefficient (c_f) and structural factor ($c_s c_d$):

a. Force coefficient (c_f).

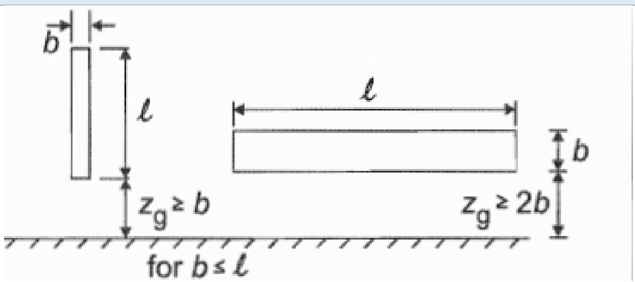
Force coefficient depended on the Effective slenderness, Reduction factor for rounded corners and End effect factor, and can be calculated by using the equation (3-39):

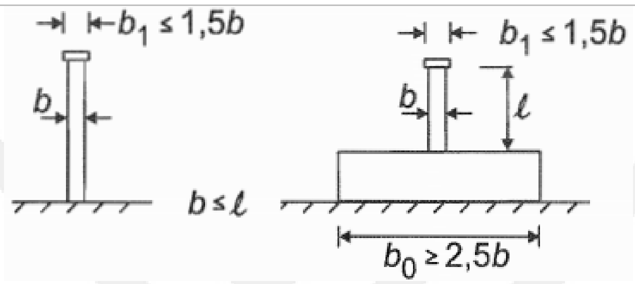
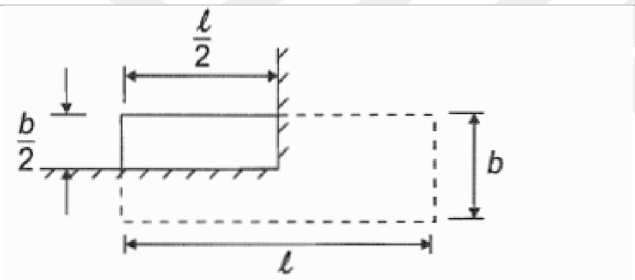
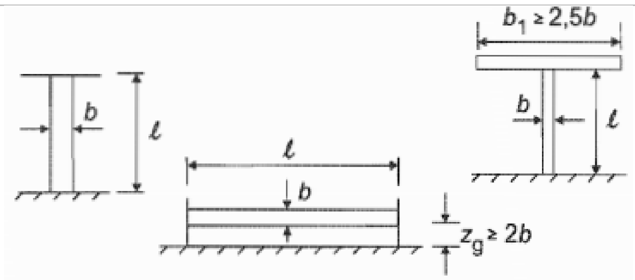
$$c_f = c_{f,0} \times \psi_r \times \psi_\lambda \quad (3.39)$$

a. Effective slenderness (λ).

To obtain the Effective slenderness (λ) value by using the table (3-3) depended on the position and height of the structure:

Table 3.3: Recommended Values of Effective Slenderness (λ) (EN 1991-4-1 Table 7.16).

<table>
<tr><th>No.</th><th>Position of the structure,
wind normal to the plane of the page</th><th>Effective slenderness λ</th></tr>
<tr><td>1.</td><td></td><td rowspan="3">For polygonal, rectangular and sharp edged sections and lattice structures:
for l ≥ 50 m, λ =1,4 l/b or λ=70, whichever is smaller.
for l <15 m, λ =2 l/b or λ = 70, whichever is smaller.
For circular cylinders:
for l ≥ 50 m, λ =0,7 l/b or λ =70, whichever is smaller.
for l <15 m, λ = l/b or λ =70, whichever is smaller.

For intermediate values of l, linear interpolation should be used</td></tr>
<tr><td>2.</td><td></td></tr>
<tr><td>3.</td><td></td></tr>
<tr><td>4.</td><td></td><td>for l ≥ 50 m, λ =0,7 l/b or λ =70, whichever is larger.
for l <15 m, λ = l/b or λ =70, whichever is larger.
For intermediate values of l, linear interpolation should be used</td></tr>
</table>

b. End effect factor (ψλ).

The value of the end effect factor ψλ is determined from (EN1991-1-4 Figure 7.36) as a function of the slenderness λ and solidity ratio φ.

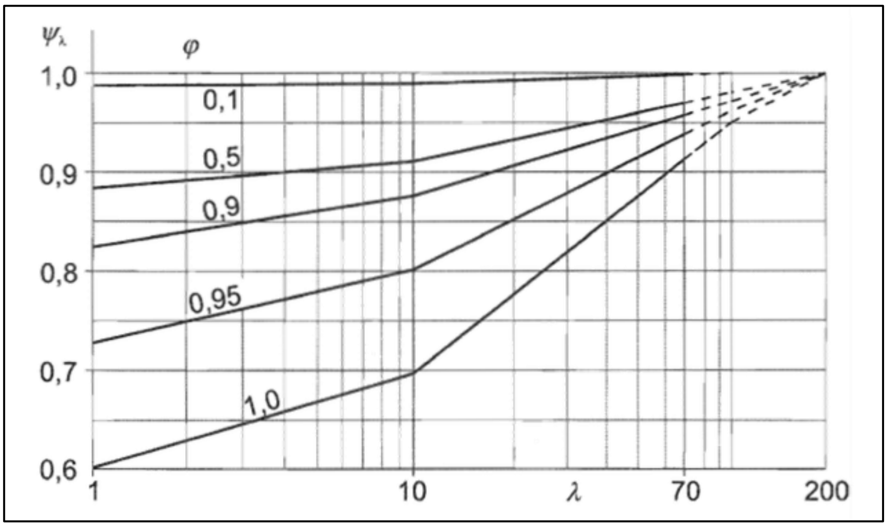


Figure 3.5: Indicative Values of The End-Effect Factor ($\psi\lambda$).

c. Force coefficient without free-end flow ($c_{f,0}$).

The value of $c_{f,0}$ is determined according to EN1991-1-4 Figure 7.23 for the ratio of d/b values.

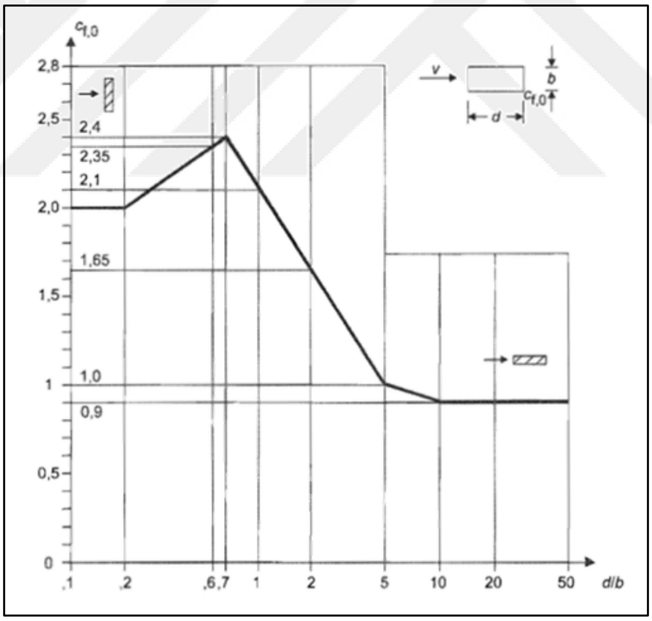


Figure 3.6: Force Coefficient ($C_{f,0}$) Without Free-End Flow of Rectangular Sections.

d. Reduction factor for rounded corners (ψ_r).

The value of ψr is calculated in accordance with EN1991-1-4 -7.6 (NOTE 1).

For cross-sections with sharp corners (r = 0.0 m) there is no reduction, $\psi_r = 1$.

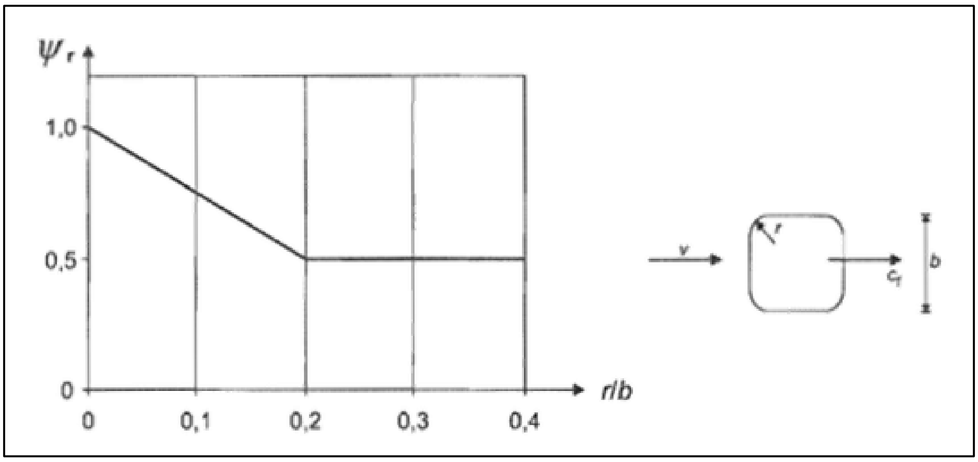


Figure 3.7: Reduction Factor Ψ_r for a Square Cross-Section.

b. Structural factor ($c_s c_d$).

By using the procedure below to obtain the structural factor ($c_s c_d$):

$Z_{(s)}$ for vertical structures such as buildings (Case 1) shown in figure (3-6).

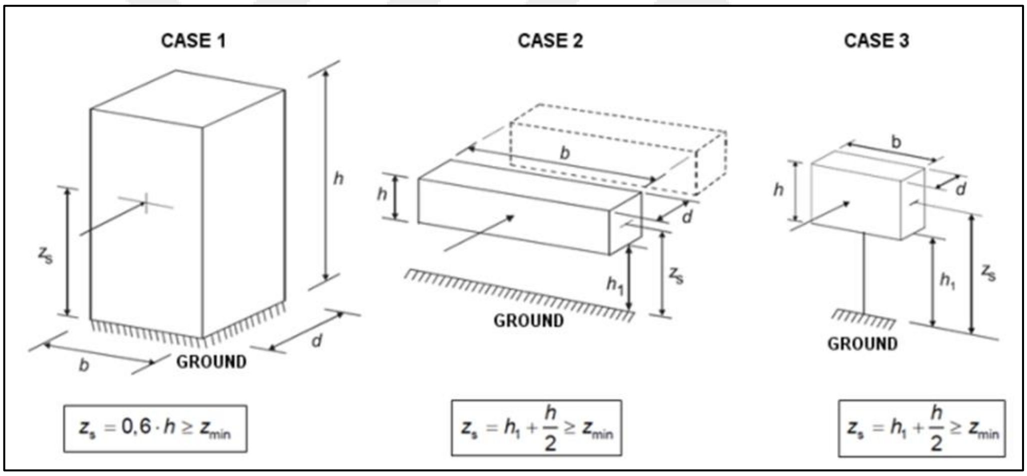


Figure 3.8: General Shapes of Structures Covered by The Design Procedure.

e. The turbulent length scale ($L_{(z)}$):

The average gust size for natural winds is represented by the turbulent length scale L(z). The turbulent length scale may be determined at heights z less than 200 m using the equation (3-40):

$$L_{(z)} = \boldsymbol{L_t} \times \left(\frac{\boldsymbol{z}}{\boldsymbol{z_t}}\right)^{\alpha} \quad \text{for} \quad z \geq z_{min} \tag{3.40}$$

$$L_{(z)} = L_{(z_{min})} \quad \text{for} \quad z < z_{min}$$

Where:

L_t: The reference length scale = 300 m.

Z_t: The reference height = 200 m.

$\alpha = 0.67 + 0.05 \times \text{Ln}\,(z_o)$

f. The fundamental flexural frequency (n_1).

Buildings with many stories and a height of more than 50 m can have their fundamental flexural frequency (n_1), estimated using equation (3-41):

$$n_1 = \frac{\mathbf{46}}{\mathbf{h}} \quad \text{in Hz} \tag{3.41}$$

g. The non-dimensional power spectral density function ($S_{L(zs,n)}$).

$$S_{L(z,n)} = \frac{\mathbf{6.8 \times f_{L(z,n)}}}{\left(\mathbf{1+10.2 \times f_{L(z,n)}}\right)^{\frac{5}{3}}} \tag{3.42}$$

Where:

$f_{L(z,n)}$: A non-dimensional frequency, calculate using equation below:

$$f_{L(z,n)} = \frac{\boldsymbol{n \times L_{(z)}}}{\boldsymbol{V_{m(z)}}} \tag{3.43}$$

h. The Background factor (B^2):

Calculating the background factor that accounts for the pressure on the structure's surface is not fully correlating, by using equation (3.44):

$$B^2 = \frac{\mathbf{1}}{\mathbf{1 + 0.9} \times \left(\frac{\boldsymbol{b+h}}{\boldsymbol{L_{(z)}}}\right)^{\mathbf{0.63}}} \tag{3.44}$$

i. The resonance response factor (R^2):

The resonance response factor R^2, which allows for turbulence in resonance with the structure's proposed vibration mode, should be calculated using equation (3-45):

$$R^2 = \frac{\boldsymbol{\pi^2}}{\mathbf{2} \times \delta} \times \mathbf{SL(z,n)} \times \mathbf{R_h}(\eta_\mathbf{h}) \times \mathbf{R_b}(\eta_\mathbf{b}) \tag{3.45}$$

a. Aerodynamic admittance functions (for 1st mode shape):

$$R_h = \frac{\mathbf{1}}{\eta\mathbf{h}} - \frac{\mathbf{1}}{\mathbf{2} \times \eta\mathbf{h^2}} \times \left(\mathbf{1} - \mathbf{e}^{-\mathbf{2}\times\eta\mathbf{h}}\right) \tag{3.46}$$

$$R_b = \frac{1}{\eta b} - \frac{1}{2 \times \eta b^2} \times \left(1 - e^{-2\times\eta b}\right)$$

$$\text{With:} \quad \eta_h = \frac{4.6 \times h}{L_{(z)}} \times fL(z,n) \quad \text{and} \quad \eta_b = \frac{4.6 \times b}{L_{(z)}} \times fL(z,n)$$

b. Logarithmic decrement of damping (δ):

$$\delta = \delta_s + \delta_a + \delta_d \tag{3.47}$$

Where:

δ_s: The logarithmic decrement of structural damping, can be obtained by table 3-4 as a part of (Table F.2 in EN 1991-1-4).

Table 3.4: Approximate Values of δ_s, (Part of Table F.2 in EN 1991-1-4).

Structural type	structural damping δ_s
Reinforced concrete buildings	0.10
Steel buildings	0.05
Mixed structures concrete + steel	0.08
Reinforced concrete towers and chimneys	0.03

δd: The logarithmic decrement of damping due to special devices.

δa: The logarithmic decrement of aerodynamic damping for the fundamental mode, can be calculated by equation (3.48):

$$\delta_a = \frac{\boldsymbol{Cf} \times \boldsymbol{\rho} \times \boldsymbol{b} \times \boldsymbol{V_{m(zs)}}}{\mathbf{2} \times \boldsymbol{n_1} \times \text{ɱ}\boldsymbol{e}} \tag{3.48}$$

Where:

C_f: The force coefficient.

$ɱ_e$: The equivalent mass per unit area of the structure which for rectangular areas, can be calculated as shown below:

$ɱ_e$ may be "estimated for cantilevered structures with varied mass distributions by the average value of m across the top third of the structure, h_3". (Item F.4.(2) EN 1991-1-4)

j. The up-crossing frequency (v):

$$v = \boldsymbol{n_1} \times \left(\frac{\boldsymbol{R^2}}{\boldsymbol{R^2+B^2}}\right)^{\mathbf{0.5}} \geq 0.08 \text{ Hz} \tag{3.49}$$

c. The Peak factor (K_p):

$$K_p = \text{Max}\left(\left(\sqrt{\mathbf{2} \times \boldsymbol{Ln}\,(\boldsymbol{v.T})} + \frac{\mathbf{0.6}}{\sqrt{\mathbf{2} \times \boldsymbol{Ln}\,(\boldsymbol{v.T})}}\right), 3\right) \tag{3.50}$$

Where:

T: The averaging time for the mean wind velocity, T = 600 sec.

k. The turbulence intensity $I_{v(z)}$ at height $Z_{(s)}$.

To calculate the turbulence intensity $I_{v(z)}$ at height $Z_{(s)}$ by using the equation (3.35).

And Finally:

The size factor (C_s):

$$C_s = \frac{\mathbf{1+7\times I_{v(zs)} \times \sqrt{B^2}}}{\mathbf{1+7\times I_{v(zs)}}} \tag{3.51}$$

The dynamic factor (C_d):

$$C_d = \frac{\mathbf{1+2 \times k_p \times I_{v(zs)} \times \sqrt{B^2+R^2}}}{\mathbf{1+7\times I_{v(zs)} \times \sqrt{B^2}}} \tag{3.52}$$

To calculate the wind force acting on the structure in i^{th} floor, needs to define the reference area of the structure, and equation (3-38) will be as shown below:

$$F_{w,i} = c_s c_d \times c_f \times (b \times ((\tfrac{\mathbf{1}}{\mathbf{2}}\, h_i \times q_{p(ze)}) + (\tfrac{\mathbf{1}}{\mathbf{2}}\, h_{i+1} \times q_{p(ze)}))) \tag{3.53}$$

3.3.2 Shear Force (Ti) and Cantilever Moments (Mo)

To obtain T_i Value (kN):

$$T_i = \sum_{\mathbf{n}}^{\mathbf{i}} \mathbf{P_i} \tag{3.54}$$

To obtain M_o Value (kNm):

$$M_{o,i} = T_i \times h_i + M_{o,i+1} \tag{3.55}$$

3.3.3 The Coefficients and Constants of Equation Set

To obtain the coefficients and constants of an equation, use the following equation with E=1:

$$F_i = \frac{1}{\sum D} \quad \text{in } (1/m^3) \tag{3.56}$$

$$\sum(EI_p)_i = Ip \text{ (shear wall)} \times \text{Number} \tag{3.57}$$

$$f_i = \frac{hi}{6\times\sum(EIp)i} \quad \text{in } (1/m^3) \tag{3.58}$$

$$\delta_{i,i-1} = f_{i-1} - F_{i-1} \quad \text{in } (1/m^3) \tag{3.59}$$

$$\delta_{i,i} = 2\times f_i + 2\times f_{i-1} + F_i + F_{i-1} \quad \text{in } (1/m^3) \tag{3.60}$$

$$\delta_{i,o} = f_i\,(M_{i+1,o} + 2 \times M_{i,o}) + f_{i-1}\,(M_{i-1,o} + 2 \times M_{i,o}) \quad \text{in (kNm)} \tag{3.61}$$

3.3.4 Calculations of Matrix Coefficients & Load Vector

After calculated the values of ($\delta_{i,i-1}$, $\delta_{i,i}$ and $\delta_{i,o}$) in previous section, need to arrange the coefficients as a matrix according to the principle shown in figure (3-9) to solve the linear system of equation (3.62) to find the value of X_i by using the (inverse matrix method) equation (3.63):

$$
\begin{array}{ccccccccccc}
\boldsymbol{\delta_{1,1}} & \boldsymbol{\delta_{1,2}} & \mathbf{0} & \mathbf{0} & \mathbf{0} & \mathbf{0} & \mathbf{0} & & \mathbf{X_1} & & -\boldsymbol{\delta_{1,0}} \\
\boldsymbol{\delta_{2,1}} & \boldsymbol{\delta_{2,2}} & \boldsymbol{\delta_{2,3}} & \mathbf{0} & \mathbf{0} & \mathbf{0} & \mathbf{0} & & \mathbf{X_2} & & -\boldsymbol{\delta_{2,0}} \\
\mathbf{0} & \boldsymbol{\delta_{3,2}} & . & .. & \mathbf{0} & \mathbf{0} & \mathbf{0} & & . & & . \\
\mathbf{0} & \mathbf{0} & .. & . & .. & \mathbf{0} & \mathbf{0} & \times & \mathbf{X_i} & = & -\boldsymbol{\delta_{i,0}} \\
\mathbf{0} & \mathbf{0} & \mathbf{0} & .. & . & \boldsymbol{\delta_{n-2,n-1}} & \mathbf{0} & & . & & . \\
\mathbf{0} & \mathbf{0} & \mathbf{0} & \mathbf{0} & \boldsymbol{\delta_{n-1,n-2}} & \boldsymbol{\delta_{n-1,n-1}} & \boldsymbol{\delta_{n-1,n}} & & \mathbf{X_{n-1}} & & -\boldsymbol{\delta_{n-1,0}} \\
\mathbf{0} & \mathbf{0} & \mathbf{0} & \mathbf{0} & \mathbf{0} & \boldsymbol{\delta_{n,n-1}} & \boldsymbol{\delta_{n,n}} & & \mathbf{X_n} & & -\boldsymbol{\delta_{n,0}}
\end{array}
$$

Figure 3.9: Principle of The Coefficient's Arrangement in Matrix.

$$[\delta] \times \{X\} = \{-\delta_o\} \tag{3.62}$$

$$\{X\} = \{-\delta_o\} \times [\delta]^{-1} \tag{3.63}$$

Where:

[δ]: Coefficients-Band Matrix.

{X}: Unknown Vector.

{δ_o}: Coefficient Vector.

3.3.5 Calculations of Moment Superposition and Distribution it

In the previous section the (X_i) unknowns were obtained as a moment at the slab level. Furthermore, the total of all wall moments is superposed with the isostatic system moment as follows:

$$M_i = M_{i,o} + X_i \tag{3.64}$$

Because the measured moment is the summation of all the moments of all the shear walls on the relevant floor of the building, the moment acting on each shear wall or core wall is distributed according to the stiffness of the wall. For example, consider the following formula for calculating the bending moment of shear wall (a) at level (i):

$$\mathbf{M_{a,i}} = \mathbf{M_i} \times \frac{(\mathbf{EIp})_{\mathbf{a,i}}}{\sum(\mathbf{EIp})_{\mathbf{i}}} \tag{3.65}$$

3.3.6 Calculations of Shear Forces

The ratio of the difference between the (X) moments in the (i+1) and (i) story to the story height gives the total shear force acting on the frames:

$$\sum T_i = \frac{\Delta X}{h_i} = \frac{X_{i+1} - X_i}{h_i} \tag{3.66}$$

The shear force (T_i) in the i^{th} floor for each structure element can be obtain by distributing the total shear force ($\sum T_i$) in the i^{th} floor according to the structure element fictitious shear stiffness:

$$T = \sum \mathbf{T} \times \left(\frac{\mathbf{D}_i}{\sum \mathbf{D}_i}\right) \tag{3.67}$$

3.3.7 Calculations of Coupled Beam End Moments

The coupled beam end moments (Top moment ($M_{i,o}$) and Bottom moment ($M_{i,u}$)) in the i^{th} floor can be calculate according to the position of the floor by using the following equation:

Top floor:

$$\mathbf{M_{n,u}} = \frac{1}{3} \times \mathbf{T_n} \times \mathbf{h_n} \tag{3.68}$$

$$\mathbf{M_{n,o}} = \frac{2}{3} \times \mathbf{T_n} \times \mathbf{h_n} \tag{3.69}$$

Intermediate floor and bottom floor:

$$\mathbf{M_{i,u}} = \frac{1}{2} \times \mathbf{T_i} \times \mathbf{h_i} \tag{3.70}$$

$$\mathbf{M_{i,o}} = \mathbf{M_{i,u}} + \mathbf{M_{i+1,u}} \tag{3.71}$$

3.3.8 Calculation for Corrected Shear Wall and Core Moments at Bottom and Top

To correct the bottom and top moment of the shear and core wall at of the i^{th} floor by using the following equation:

Corrected moment at bottom of the i^{th} floor:

$$M_{BOTTOM,i} = \text{Moment of wall element}_{(i)} + \text{coupled beam moment}_{(i)} \tag{3.72}$$

Corrected moment at top of the i^{th} floor:

$$M_{TOP,i} = \text{Moment of wall element}_{(i+1)} - \text{coupled beam moment}_{(i)} \qquad (3.73)$$

3.3.9 Calculation of Displacement and Drift of Story

The drift of the story can be obtained by dividing the sheer force of the i^{th} story by the sum of its shear rigidity, as shown in equation (3-74):

$$\delta_i = \frac{\sum \mathbf{T_i}}{\sum \mathbf{D_{i,j}}} \qquad (3.74)$$

and the displacement of the i^{th} story is the summation of the story drift from the 1^{st} story to the i^{th} story:

$$d_i = \sum \delta i \qquad (3.75)$$

Now, after completing the calculation of story drift and displacement needs to check the result of the building's displacement and the maximum story drift with the limitation mention in Specification for Buildings to be Built in Seismic Zones 2007:

$$\Delta_{max} = 0.002 \times H_{Building} \qquad (3.76)$$

$$\text{The drift ratio} = \frac{\boldsymbol{\delta_{max}}}{\mathbf{h_i}} < 0.02 \qquad (3.77)$$

3.4 WIND FORCE (TURKISH CODE DISTRIBUTED) AND IMPACT ON STRUCTURE (CASE 2)

The calculation of the base shear force acting on the building surface is completed according to (EN 1991-1-4) in Section 3.3.1. Consider the base shear force as the total equivalent seismic load and distribute it to the structure floor using Turkish code by the "Equivalent Seismic Load Method" as shown in Section 3.4.1 below and study the impact on the structure as shown in the previous sections (3.3.2 to 3.3.9) in both direction (Y and X), respectively.

3.4.1 Distributed Lateral Load

The total equivalent seismic load is the sum of the equivalent seismic loads acting at each story and the additional equivalent seismic load (ΔF_n), acting at the n^{th} story of the building, as shown in equation (3-78) and figure (3-8):

$$V_t = \Delta F_n + \sum_{i=1}^{n} F_i \tag{3.78}$$

Where:

n: number of stories.

$\Delta F_n = 0.0075 \times n \times V_t$

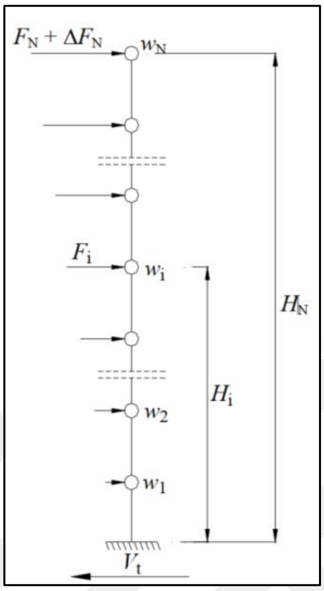


Figure 3.10: The Total Equivalent Seismic Load (TBC 2007).

The total equivalent seismic load (without ΔF_n) must be distributed to stories of the building by using the equation (3.79):

$$F_i = (\mathbf{V_t} - \Delta\mathbf{F_n}) \times \left(\frac{\mathbf{W_i} \times \mathbf{H_i}}{\sum_{\mathbf{j=1}}^{\mathbf{n}}(\mathbf{W_j} \times \mathbf{H_j})}\right) \qquad (3.79)$$

The equivalent seismic loads are calculated for each story with additional force on the n^{th} floor; these values are used for studying the effect of the load on the building by using the procedure mentioned in previous sections (3.3.2 to 3.3.9).

3.4.2 Acceleration Values of Equivalent Earthquake Force to Wind Force

The "Equivalent seismic load method" was used to distribute the base shear force that was calculated based on the Eurocode to study the effect of earthquakes on the building with a force equivalent to the wind force affecting the building. comparison of the moment, shear force, story drift, and displacement between the two cases (case 1: wind load impact and case 2: impact of seismic force equivalent to the wind force).

To calculate the acceleration values of equivalent earthquake force to wind force, first, we need to obtain the fundamental period of the building by using the Rayleigh Method:

$$T_1 = \mathbf{2\pi} \times \sqrt[2]{\frac{\sum_{\mathbf{i=1}}^{\mathbf{n}} \mathbf{M_i} \times \mathbf{d_i}^{\mathbf{2}}}{\sum_{\mathbf{i=1}}^{\mathbf{n}} \mathbf{P_i} \times \mathbf{d_i}}} \quad \text{in sec} \qquad (3.80)$$

Where:

Mi = wi / g   in kN.sec2/m
di: the ith floor displacement.
Pi: The equivalent seismic loads on ith floor.
Second: obtain the seismic characteristic according to (TEBC 2007):

a. Building location in seismic zone.
The seismic zones are the first, second, third, and fourth seismic zones, as shown on turkey's seismic zoning map.

b. Effective ground acceleration coefficient (A_o).

Table 3.5: Effective Ground Acceleration Coefficient (Ao) (TBC 2007- table 2.2).

Seismic Zone	Ao
1	0.40
2	0.30
3	0.20
4	0.10

c. Building importance factor (i) for hotel building.

Table 3.6: Building Importance Factor (I) (TBC 2007- Table 2.3).

Purpose of Occupancy or Type of Building	Importance Factor (I)
1. Buildings to be utilized after the earthquake and buildings containing hazardous materials: a) Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, firefighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations). b) Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
2. Intensively and long-term occupied buildings and buildings preserving valuable goods: a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. b) Museums.	1.4
3. Intensively but short-term occupied buildings Sport facilities, cinema, theatre and concert halls, etc.	1.2
4. Other buildings: Buildings other than above-defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

a. Local site class.

The local site class are Z1, Z2, Z3 and Z4.

b. Spectrum coefficient.

The spectrum coefficient, $S_{(T)}$:

$$S_{(T)} = 1 + 1.5 \times \frac{T}{T_A} \quad (0 \leq T \leq T_A)$$

$$S_{(T)} = 2.5 \qquad (T_A < T \leq T_B) \tag{3.81}$$

$$S_{(T)} = 2.5 \times \left(\frac{T_B}{T}\right)^{0.8} \qquad (T_B < T)$$

Table 3.7: Spectrum Characteristic Periods (T_A, T_B) (TBC 2007- table 2.4).

Local Site Class	TA sec	TB sec
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

a. Structural Behavior Factors (R).

Table 3.8: Structural Behavior Factors (R) (TBC 2007- Part of Table 2.5).

BUILDING STRUCTURAL SYSTEM For cast-in-situ reinforced concrete buildings.	Systems of Nominal Ductility Level	Systems of High Ductility Level
(1.1) Buildings in which seismic loads are fully resisted by frames..	4	8
(1.2) Buildings in which seismic loads are fully resisted by coupled structural walls........................	4	7
(1.3) Buildings in which seismic loads are fully resisted by solid structural walls............................	4	6
(1.4) Buildings in which seismic loads are jointly resisted by frames and solid and/or coupled structural walls.......................................	4	7

$$Ra(T) = 1.5 + (R - 1.5) \times \frac{T}{T_A} \qquad (0 \leq T \leq T_A)$$

$$Ra(T) = R \quad (T_A < T) \tag{3.82}$$

Third: Acceleration values of equivalent earthquake force to wind force can be obtained by using the equation (3.83):

Real base shear force in case of using erzincan 1992 N-S chart calculated by multiply the ratio of Sa/a_{max} over the spectrum coefficient with the base shear force:

$$V_{t2} = Vt \times ((Sa/a_{max})/ S_{(T)}) \tag{3.83}$$

Where:

V_t: Total Equivalent Seismic Load, Appling on the building in earthquake direction =

S_a/a_{max}: This value can be obtained by using ERZICAN 1992 N-S chart figure (3.11), intersect the time-period value with damping ratio.

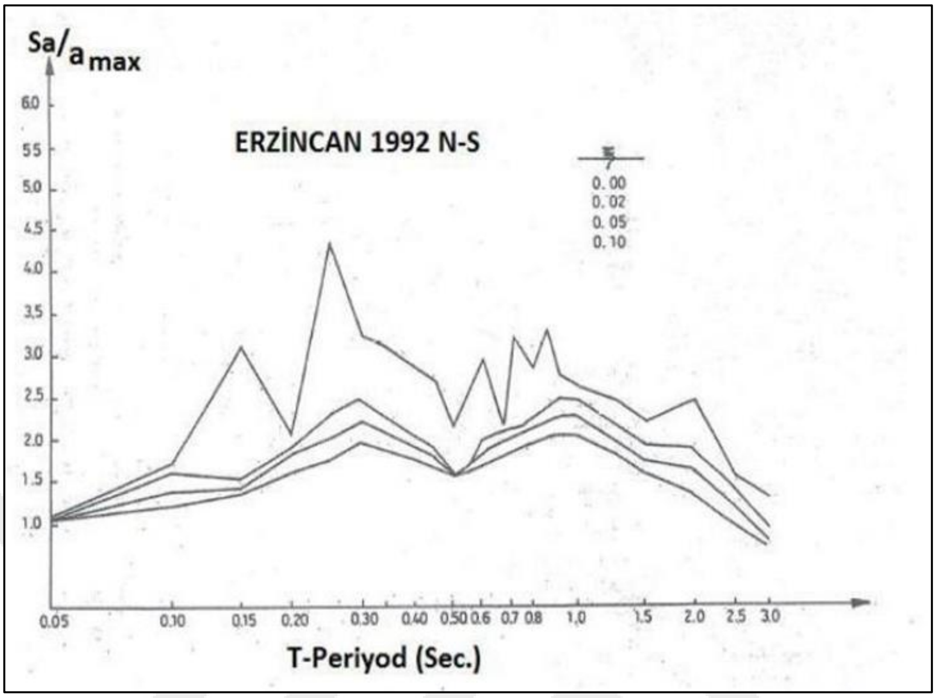


Figure 3.11: ERZICAN 1992 N-S Chart.

Compare the acceleration values of equivalent earthquake force to wind force with the real value of the effective ground acceleration coefficient (A_o), which depends on the seismic zone. and obtain the real base shear force acting on a structure due to the earthquake force according to the seismic characteristics mentioned in (item: second) above by using the equation (3.83).

Therefore, calculate the ratio of the real base shear force obtained in the case of a real earthquake over the base shear force obtained in the case of earthquake force equivalent to wind force and correct the results of moment, shear force, story drift, and displacement for the case of earthquake force equivalent to wind force. Considered, the acceptable store drift is given as a function of the building's behavior coefficients (R).

In this way, we may evaluate the wind force as a reference for the difference with the equivalent earthquake force and the real earthquake force.

4. CASE STUDY: ANALYSIS AND RESULTS.

4.1 BUILDING CHARACTERISTIC

Given the following floor plan and data of the planned high-rise hotel building in accordance with TS500:

Figure 4.1: 1st to 5th Floor Plan.

Figure 4.2: 6th to 10th Floor Plan.

Figure 4.3: 11th to 20th Floor Plan.

a. Building dimensions: 38 m × 12 m

b. Total number of stories = 20 stories

c. Story height: 1^{st} floor = 4m, 2^{nd} to 20^{th} =3m

d. Slabs thickness: h = 15 cm

e. Columns:

i. C90×35 (cm): Dimension will be constant through all stories
ii. C40×70 (cm): In 1^{st} to 5^{th} floor
iii. C35×70 (cm): In 6^{th} to 10^{th} floor
iv. C35×60 (cm): In 11^{st} to 20^{th} floor

f. Axes definition:

v. Axes 2,7 Ext: C1
vi. Axes 3,6 Ext: C2
vii. Axes 2,7 Int: C3
viii. Axes 3,6 Int: C4

g. Beams:

ix. Normal beams: 25 cm × 60 cm.
x. Coupled beams: 25 cm × 70 cm.

h. Shear wall and core wall width: 35 cm.

i. Concrete strength: c35

j. Modulus of elasticity for concrete = 33000 n/mm^2

k. Steel grade: s420

l. γ oncrete = 25 kn/m^3

m.γ finishing = 20 kn/m^3

n. Thickness of Finishing = 8 cm.

4.2 TOTAL WEIGHT OF THE STRUCTURE’S ELEMENTS

4.2.1 Calculation of Slab Weight

i. $W_{finishing}$ = Finishing area × Thickness × $\gamma_{finishing}$

$W_{finishing}$ = 426 m^2 × 0.08 m × 20 kN/m^3 = 681.6 kN.

ii. W_{slab} = (Plan area – S_W) × Thickness × $\gamma_{concrete}$

W_{slab} = ((12 m × 38 m) - 13.055 m^2) × 0.15 m × 25 kN/m^3 = 1661.044 kN.

iii. Total Weight $_{slab+finishing}$ = W_{slab} + $W_{finishing}$

Total Weight $_{slab+finishing}$ = 1661.044 kN + 681.6 kN = 2342.644 kN.

4.2.2 Calculation of Beams Weight

W $_{Beam}$ = Beam section Area × Length of Span × $\gamma_{concrete}$ × Number.

a. Beams in X-dir.

i. Beam weight = (0.25 × 0.45) × 9.775 × 25 × 8 = 219.94 kN.
ii. Beam weight = (0.25 × 0.55) × 6.925 × 25 × 4 = 95.22 kN.
iii. Beam weight = (0.25 × 0.45) × 9.525 × 25 × 4 = 107.16 kN.
iv. Beam weight = (0.25 × 0.55) × 7.125 × 25 × 4 = 97.97 kN.
v. Beam weight = (0.25 × 0.55) × 2 × 25 × 2 = 13.75 kN.
i. Total weight = 534.03 kN.

b. Beams in Y-dir.

vi. Beam weight = (0.25 × 0.45) × 12.00 × 25 × 4 = 135.00 kN.
vii. Beam weight = (0.25 × 0.55) × 3.475 × 25 × 4 = 47.78 kN.
viii. Beam weight = (0.25 × 0.45) × 3.00 × 25 × 2 = 20.63 kN.
ii. Total weight = 203.41 kN.

c. Summation of total weight for each floor = 737.44 kN.

4.2.3 Calculation of Columns Weight

$W_{Column\ section}$ = Column area × Height of column × $\gamma_{concrete}$ × Number.

iv. Weight of Column 90×35 cm.

a. Weight of columns in 1^{st} story.

$W_{C90\times35}$ = (0.9 m × 0.35 m) × 3.4 m× 25 kN/m^3 × 4 = 107.1 kN.

b. Weight of columns in 2^{nd} to 20^{th} Story.

$W_{C90\times35}$ = (0.9 m × 0.35m) × 2.4 m× 25 kN/m^3 × 4 = 75.6 kN.

v. Weight of Column 40×70 cm.

a. Weight of columns in 1st story.

$W_{C40\times70}$ = (0.4 m × 0.7 m) × 3.4 m×25 kN/m^3 × 12 = 285.6 kN.

b. Weight of columns in 2nd to 5th Story.

$W_{C40\times70}$ = (0.4 m × 0.7 m) × 2.4 m×25 kN/m^3 × 12 = 201.6 kN.

vi. Weight of Column 35×70 cm in 6th to 10th story.

$W_{C35\times70}$ = (0.7 m × 0.35 m) × 2.4 m×25 kN/m^3 × 12 = 176.4 kN.

vii. Weight of Column 35×60 cm in 11th to 20th story.

$W_{C35\times60}$ = (0.6 m × 0.35 m) × 2.4 m×25 kN/m^3 × 12 = 151.2 kN.

4.2.4 Calculation of Shear Wall and Core Weight.

$W_{SW\text{-}Position}$ = Shear Wall area × Height of story × $\gamma_{concrete}$ × Number.

W_{Core} = Core Shear Wall area × Height of story × $\gamma_{concrete}$ × Number.

a. 1st floor.

i. $W_{SW\text{-}Right\ \&\ Left}$ = (4.5 × 0.35) × 4 × 25 × 4 = 630 kN.
ii. $W_{SW\text{-}center}$ = (4 × 0.35) × 4 × 25 × 2 = 280 kN.
iii. W_{Core} = ((1 × 4.35) -(0.65 × 3.3)) × 4 × 25 × 2 = 441 kN.

b. 2nd to 20th floor.

iv. $W_{SW\text{-}Right\ \&\ Left}$ = (4.5 × 0.35) × 3 × 25 × 4 = 472.5 kN.
v. $W_{SW\text{-}center}$ = (4 × 0.35) × 3 × 25 × 2 = 210 kN.
vi. W_{Core} = ((1×4.35) -(0.65 × 3.3)) ×3×25×2 = 330.75 kN.

4.2.5 Calculation of Exterior Masonry Walls Weight

a.1st Floor:

i. W_1 = (4.375 × 3.4) × 2.5 × 4 = 148.75 kN.
ii. W_2 = (4.1 × 3.4) × 2.5 × 4 = 139.40 kN.

iii. $W_3 = (6.925 \times 3.3) \times 2.5 \times 4 = 228.53$ kN.
iv. $W_4 = (3 \times 3.3) \times 2.5 \times 2 = 49.5$ kN.
v. Total weight: 566.18 kN.

b. 2nd to 5th Floor:

i. $W_1 = (4.375 \times 2.4) \times 2.5 \times 4 = 105.00$ kN.
ii. $W_2 = (4.1 \times 2.4) \times 2.5 \times 4 = 98.40$ kN.
iii. $W_3 = (6.925 \times 2.3) \times 2.5 \times 4 = 159.28$ kN.
iv. $W_4 = (3 \times 2.3) \times 2.5 \times 2 = 34.50$ kN.
v. Total weight: 397.18 kN.

c. 6th to 20th Floor:

i. $W_1 = (4.4 \times 2.4) \times 2.5 \times 4 = 105.60$ kN.
ii. $W_2 = (4.125 \times 2.4) \times 2.5 \times 4 = 99.00$ kN.
iii. $W_3 = (6.925 \times 2.3) \times 2.5 \times 4 = 159.28$ kN.
iv. $W_4 = (3 \times 2.3) \times 2.5 \times 2 = 34.50$ kN.
v. Total weight: 398.38 kN.

Collecting data of weight for all elements of the structure from the above steps for each floor, as shown in the table 4-1 below:

Table 4.1: Weight for All Elements of The Structure Due to Self-Weight.

Story	Structure's Element					Total Weight
	Slabs	Columns	Beams	Wall	Shear Wall	
11 to 20	2342.64	226.80	737.44	398.38	1013.25	4718.51
6 to 10	2342.64	252.00	737.44	398.38	1013.25	4743.71
2 to 5	2342.64	277.20	737.44	397.18	1013.25	4767.71
1	2342.64	392.70	737.44	566.18	1351.00	5389.96

* All Unit in kN.

After determining the dead loads acting on the floor, the total floor weight can be computed by applying the equation (3.6) by using the typical live Load values for hotel building is 2 kN/m^2 according to TEC 2007:

W_1=5389.96 + (0.3 × 2 × 12 × 38) = 5663.56 kN

$W_{2\text{-}5}$= 4767.71 + (0.3 × 2 × 12 × 38) = 5041.31 kN

$W_{6\text{-}10}$ = 4743.74 + (0.3 × 2 × 12 × 38) = 5017.31 kN

$W_{11\text{-}19}$ = 4718.51 + (0.3 × 2 × 12 × 38) = 4992.11 kN

W_{20} = 4718.51 kN.

Table 4.2: Weight of Each Floor.

Story	Floor Weight Wi (kN)
20	4718.51
19	4992.11
18	4992.11
17	4992.11
16	4992.11
15	4992.11
14	4992.11
13	4992.11
12	4992.11
11	4992.11
10	5017.31
9	5017.31
8	5017.31
7	5017.31
6	5017.31
5	5041.31
4	5041.31
3	5041.31
2	5041.31
1	5663.56

4.3 ∑D OF STORY IN BOTH DIRECTIONS

4.3.1 Calculation in Y-Dir

4.3.1.1 Calculation of moment of nertia of structure member.

Moment of inertia for all rectangular shape structure member can be calculated by using the equation (3-7).

Examples for calculations:

Column C90×35: b=0.90 m, h=0.35 m.

$I = b\times h3/12 = 0.90 \times 0.353 / 12 = 0.0032$ m4.

Normal Beams: b=0.25 m, h=0.60 m.

$I = b\times h3/12 = 0.25 \times 0.603 / 12 = 0.0045$ m4.

Shear wall (right and left): b=0.35 m, h=4.50 m.

$I = b\times h3/12 = 0.35 \times 4.503 / 12 = 2.66$ m4.

Core Shear wall: b1=1.00 m, h1=4.35 m, b2=0.65 m, h2=3.30.

$I1 = b1\times h13/12 = 1.00 \times 4.353 / 12 = 6.86$ m4.

$I2 = b2\times h23/12 = 0.65 \times 3.303 / 12 = 1.95$ m4.

Pure $I = I_1 - I_2 = (6.8594 - 1.9466) = 4.23\ m^4$.

Table 4.3: The Moment of Inertia (I) Values for Each Structural Member (Y-Dir.).

Member		b (m)	h (m)	$bh^3/12$ (m^4)	Total I (m^4)
Columns	90×35	0.9	0.35	0.0032	0.0032
	40×70	0.4	0.7	0.0114	0.0114
	35×70	0.35	0.7	0.0100	0.0100
	35×60	0.35	0.6	0.0063	0.0063
Beam	Normal	0.25	0.6	0.0045	0.0045
	Coupled	0.25	0.7	0.0071	0.0071
	Cracked	0.5 × Ic.b		0.0036	0.0036
Normal Shear Wall	Right and left	0.35	4.5	2.6578	2.6578
	Center	4	0.35	0.0143	0.0143
Core Shear Wall	A1	1	4.35	6.8594	4.2254
	A2	0.65	3.65	2.6340	

4.3.1.2 Rigidity of columns (Dci)

a. Calculation of stiffness of columns (K_c).

Obtain the stiffness of columns by using the equation (3-8):

Examples for calculations:

$K_{C90\times35,1st}$ = 32.16 / 4 = 8.04 dm⁴/m.

$K_{C40\times70,2nd}$ = 114.33 / 3 = 38.11 dm⁴/m.

Table 4.4: The Stiffness of Columns (Kc) Values for Each Column (Y-Dir.).

<table>
<tr><th rowspan="2">Floor</th><th rowspan="2">hi
m</th><th colspan="4">Ic (dm⁴)</th><th colspan="4">Kc (dm⁴/m)</th></tr>
<tr><th>C 90×35</th><th>C 40×70</th><th>C 35×70</th><th>C 35×60</th><th>C 90×35</th><th>C 40×70</th><th>C 35×70</th><th>C 35×60</th></tr>
<tr><td>11 to 20</td><td>3</td><td rowspan="4">32.16</td><td rowspan="2">-</td><td>-</td><td>63.00</td><td>10.72</td><td rowspan="2">-</td><td>-</td><td>21.00</td></tr>
<tr><td>6 to 10</td><td>3</td><td>100.04</td><td rowspan="3">-</td><td>10.72</td><td>33.35</td><td rowspan="3">-</td></tr>
<tr><td>2 to 5</td><td>3</td><td rowspan="2">114.33</td><td rowspan="2">-</td><td>10.72</td><td>38.11</td><td rowspan="2">-</td></tr>
<tr><td>1</td><td>4</td><td>8.04</td><td>28.58</td></tr>
</table>

b. Calculation of stiffness of beams (Kb).

Obtain the stiffness of columns by using the equation (3-9):

Examples for calculations:

Kb = Ib / L

Kb7,1st = 45 / 2.95 = 15.25 dm⁴/m.

Kb10,11st = 45 / 3.35 = 13.43 dm⁴/m.

Table 4.5: The Stiffness of Beams (Kb) Values for Each Beam (Y-Dir.).

<table>
<tr><th rowspan="2">Floor</th><th rowspan="2">I_b (dm⁴)</th><th colspan="3">Length of span (m)</th><th colspan="3">K_b (dm⁴/m)</th></tr>
<tr><th>B7</th><th>B10</th><th>B8</th><th>B7</th><th>B10</th><th>B8</th></tr>
<tr><td>11 to 20</td><td rowspan="2">45.00</td><td>3.1</td><td>3.35</td><td>3.4</td><td>14.52</td><td>13.43</td><td>13.24</td></tr>
<tr><td>1 to 10</td><td>2.95</td><td>3.3</td><td>3.3</td><td>15.25</td><td>13.64</td><td>13.64</td></tr>
</table>

c. Calculation of K' and a value.

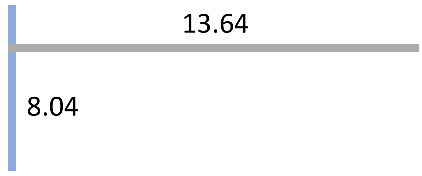
Obtain K' value by using the equation (3-10) for normal and top story and equation (3-11) for bottom story.

Examples for calculations:

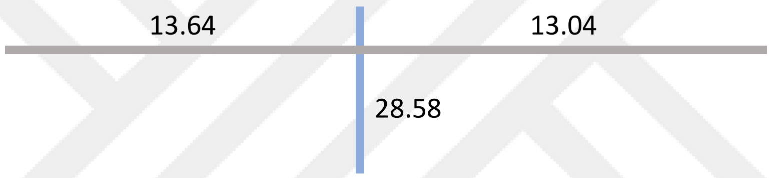
Ḱ C3, 20th = (14.52+13.24+14.52+13.24) / (2×21) = 1.321

14.52	13.24
14.52	21 13.24

$\acute{K}_{C2,\ 1st} = (13.64) / (8.04) = 1.696$



$\acute{K}_{C4,\ 1st} = (13.64+13.04) / (28.58) = 0.954$



Obtain (a) value by using the equation (3-12) for story column and equation (3-13) for Rigid column.

Examples for calculations of (a):

Story column C2,20th: a = 1.253 / (2+1.253) = 0.385

Rigid column C2,1st: a = (0.5+1.696) / (2+1.696) = 0.594

The Ḱ and (a) values for each floor are obtained and found in table 4.6.

Table 4.6: The Ḱ and (A) Values for Each Floor (Y-Dir.).

Floor	K'				a			
	C1	C2	C3	C4	C1	C2	C3	C4
12 to 20	0.691	1.253	1.321	1.270	0.257	0.385	0.398	0.388
11	0.691	1.263	1.349	1.284	0.257	0.387	0.403	0.391
6 to 10	0.457	1.272	0.866	0.818	0.186	0.389	0.302	0.290
2 to 5	0.400		0.758	0.716	0.167		0.275	0.264
1	0.534	1.696	1.011	0.954	0.408	0.594	0.502	0.492

c. Calculation of rigidity of columns (Dc).

Obtain the rigidity of columns by using the equation (3-15):

Examples for calculations of (Dc):

$D_{C2,20th} = 12 \times 0.385 \times 10.72 = 49.55$ dm^4/m

$D_{C2,1th} = 12 \times 0.594 \times 8.04 = 57.32$ dm^4/m

Examples for calculations of (∑Dc):

$\sum D_{C2,20th} = 49.55 \times 4 = 198.2$ dm^4/m

$\sum D_{C2,1st} = 57.32 \times 4 = 229.28$ dm^4/m

Examples for calculations of (∑Dci):

$\sum D_{c,20th} = 258.9 + 198.2 + 401.04 + 391.47 = 1249.62$ dm^4/m

$\sum D_{c,1st} = 559.74 + 229.28 + 688.45 + 675.35 = 2152.82$ dm^4/m

Table 4.7: The Rigidity and Summation of Rigidity of Columns Values (Y-Dir.).

<table>
<tr><th rowspan="2">Floor</th><th colspan="4">Dc (dm⁴/m)</th><th colspan="4">∑Dc (dm⁴/m)</th><th rowspan="2">∑Dc (dm⁴/m)</th></tr>
<tr><th>C1</th><th>C2</th><th>C3</th><th>C4</th><th>C1</th><th>C2</th><th>C3</th><th>C4</th></tr>
<tr><td>12 to 20</td><td>64.73</td><td>49.55</td><td>100.26</td><td>97.87</td><td>258.90</td><td>198.20</td><td>401.04</td><td>391.47</td><td>1249.62</td></tr>
<tr><td>11</td><td>64.73</td><td>49.78</td><td>101.49</td><td>98.54</td><td>258.90</td><td>199.12</td><td>405.96</td><td>394.17</td><td>1258.15</td></tr>
<tr><td>6 to 10</td><td>74.49</td><td rowspan="2">50.01</td><td>120.95</td><td>116.14</td><td>297.95</td><td rowspan="2">200.03</td><td>483.80</td><td>464.57</td><td>1446.36</td></tr>
<tr><td>2 to 5</td><td>76.26</td><td>125.70</td><td>120.52</td><td>305.05</td><td>502.80</td><td>482.06</td><td>1489.94</td></tr>
<tr><td>1</td><td>139.94</td><td>57.32</td><td>172.11</td><td>168.84</td><td>559.74</td><td>229.28</td><td>688.45</td><td>675.35</td><td>2152.82</td></tr>
</table>

4.3.1.3 Fictive shear stiffness for the coupled beam (dfi)

a. Coupled beam connected to shear wall (Center).

Center share wall are Perpendicular to the force action in Y-direction it should be taken into consideration as column widths.

$Lu \geq \max [0.2\ Lw\ ,\ 2\ Bw] = [0.2 \times 4,\ 2 \times 0.35] = [0.8,\ 0.7] = 0.8$ m

Obtain Dc for the considered column with dimension (0.8 m × 0.35 m) by using muto method the equation from (3-7) to (3-16).

Calculations 20th to 2nd floor:

$Ic = (0.8 \times 0.353) \times 104 /12 = 28.58$ dm4

$Kc = 28.58 / 3 = 9.53$ dm4/m

$Kb = 0.0045 \times 104/ 3.475 = 20.56$ dm4/m

$K' = (20.56 + 20.56) / 2 \times 9.53 = 2.16$

$a = 2.16 / (2 + 2.16) = 0.519$

$Dc = 9.53 \times 0.519 \times 12 = 59.34$ dm4/m

Calculations 1st floor:

$Kc = 28.58 / 4 = 7.15$ dm4/m

$K' = (20.56) / 9.53 = 2.88$

$a = (0.5 + 2.88) / (2 + 2.88) = 0.692$

$Dc = 7.15 \times 0.692 \times 12 = 59.34$ dm4/m

Calculations of ($\sum Dci$):

$\sum Dc20th = 59.34 \times 4 = 237.37\ dm^4/m$

$\sum Dc1th = 59.38 \times 4 = 237.52\ dm^4/m$

b. Coupled beam connected between shear wall (right and left).

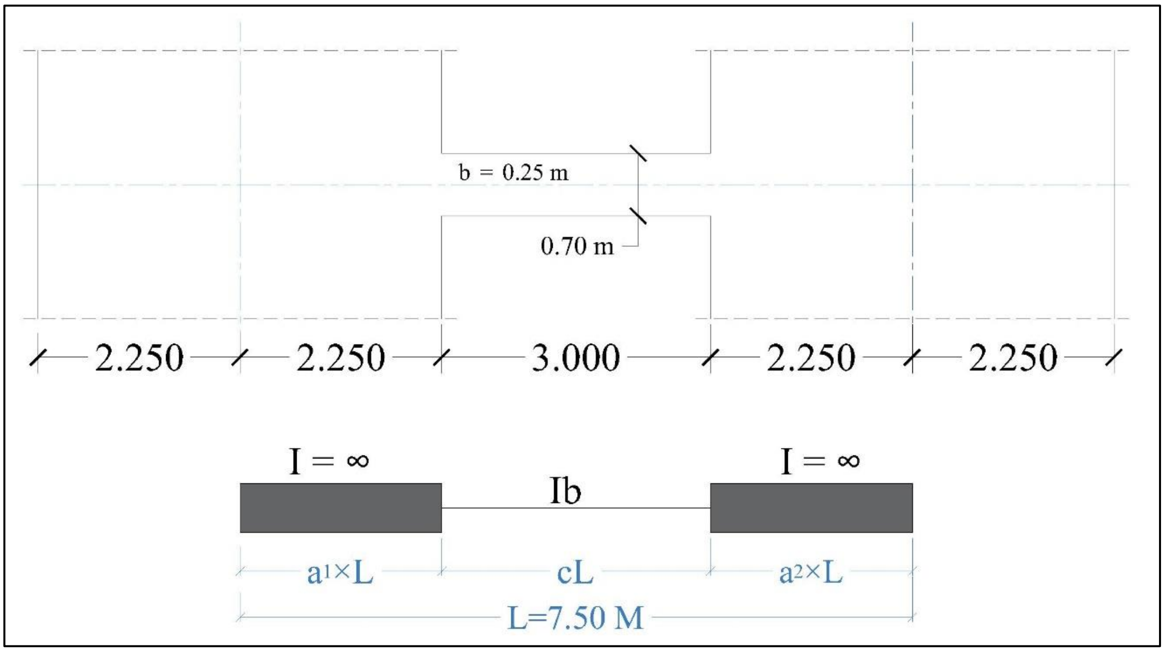


Figure 4.4: Coupled Beams Connected Between Two Shear Walls (Right and Left) (Y-Dir.).

From figure (4.4):

i. $a_1L = a_2L = aL = 2.25$ m.

ii. $cL = 3.00$ m.

iii. $L = cL + (2 \times aL) = 3.00 + (2 \times 2.25) = 7.50$ m.

iv. $c = 3.00/7.50 = 0.4$

v. $a = 2.25/7.5 = 0.3$

vi. shape factor of rectangular = 5/6

vii. $A' =$ Shape factor × Area $_{c,b}$ = $5/6 \times (0.25 \times 0.70) = 0.146\ m^2$

viii. $E/G = 2.5$ taking $E = 1$.

$\beta = (3 \times I_{bc} / cl^2 \times A') \times (E/G) = (3 \times 0.0071 / 3.0^2 \times 0.146) \times 2.5 = 0.041$

$K\beta = 1/(1+4\times\beta) = 1/(1+4\times0.041) = 0.86$

Rectifier value with considering the shear deformation:

$R_A=R_B = ((6 \times E \times I_{bc,cr}) / (c^3 \times L)) \times K\beta$

$= ((6 \times 1 \times 0.0036 \times 10^4) / (0.4^3 \times 7.5)) \times 0.86 = 383.91$ dm^4/m.

The connecting beams are identical in size. For all floors, therefor the $R_A=R_B$ values are the same.

Fictive Shear stiffness D_{fi} is calculated as follows for one connecting beam end:

Top floor = $D_{f20} = 383.91 \times 1.5 = 575.864$ (dm^4/m).

Intermediate floors = D_{f19} to $D_{f2} = 383.91$ (dm^4/m).

Bottom floor = $D_{f1} = 383.91 \times 2 = 767.82$ (dm^4/m).

∑Dfi for each floor:

Top floor = Df20 = 575.864 × 4 = 2303.46(dm^4/m).

Intermediate floors = Df19 to Df2 = 383.91 × 4 = 1535.64 (dm^4/m).

Bottom floor = Df1 = 767.82 × 4 = 3071.28 (dm^4/m).

a. Coupled beam connected between Core Shear wall and column.

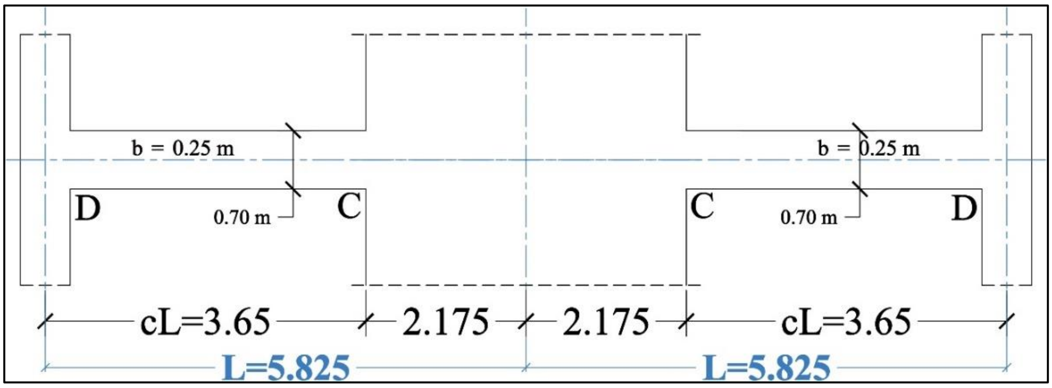


Figure 4.5: Coupled Beams Connected Between Core Wall and Column (Y-Dir.).

From figure (4.5):

ix. a1L = 2.175m.

x. cL = 3.65 m.

xi. L = cL + a1L = 3.65 + 2.175 = 5.825 m.

xii. $c = 3.65/5.825 = 0.627$

xiii. $a1 = 2.175/5.825 = 0.373$

xiv. shape factor of rectangular = 5/6

xv. A'= Shape factor × Area c,b = 5/6 × (0.25 × 0.70) = 0.175 m2

xvi. E/G = 2.5, taking E = 1.

$I_{bf,c} = I_{b,c} / c2 = 0.0071 / 0.6272 = 0.0182$ m4

$K_{bf,c} = I_{bf,c} / L = 0.0182 / 3.475 = 0.00524$ m3

$\beta = (3 \times I_{bc} / cl2 \times A') \times (E/G) = (3\times0.0071/ 3.652\times0.1458) \times 2.5 = 0.0276$

$K\beta = 1/(1+4\times\beta) = 1/ (1+4\times0.0276) = 0.90$

Examples for calculations of rectifiers for story 20th:

$K_c = 0.00095$ m3

$K_{c,o} = 0.$

$K_{c,u} = 0.00095$ m3

$r_o = K_c / (K_{c,o} + K_{bf,c}) = 0.00095 / (0 + 0.00524) = 0.1819$

$r_u = K_c / (K_{c,u} + K_{bf,c}) = 0.00095 / (0.00095 + 0.00524) = 0.1539$

$r_m = (r_o + r_u) / 2 = (0.1819 + 0.1539) / 2 = 0.168 < 0.67$

Rectifier value with considering the shear deformation:

$R_c = (3EI_{b,c} / c3L) [(3/2) (3+2c) r_m +1] \times K\beta$

$= (3\times0.0036\times104/0.6273 \times 5.825) [(3/2) \times (3+2\times 0.627) \times 0.168+1] \times 0.9 = 139.52$ dm4/m.

Table 4.8: The Rectifiers of the Core Shear Wall Connection in Each Story in (Y-Dir).

Story no.	Kc	$Kc_{,o}$	$Kc_{,u}$	$K_{bf,c}$	ro	ru	rm	RC
20	0.00095	0.00000	0.00095	0.0052	0.182	0.154	0.168	139.52
3 to 19	0.00095	0.00095	0.00095	0.0052	0.154	0.154	0.154	133.51
2	0.00095	0.00095	0.00071	0.0052	0.154	0.160	0.157	134.83
1	0.00071	0.00095	0.00000	0.0052	0.115	0.136	0.126	121.48

Fictive Shear stiffness Dfi is calculated as follows for one connecting beam end:

Top floor = Df20 = 139.52 × 1.5 = 209.29 (dm^4/m).

Intermediate floors = Df19 to Df3 = 133.51 (dm^4/m).

= Df2 = 134.83 (dm^4/m).

Bottom floor = Df1 = 121.48 × 2 = 242.97 (dm^4/m).

∑Dfi for each floor:

Top floor = Df20 = 209.29 × 4 = 837.14 (dm^4/m).

Intermediate floors = Df19 to Df3 = 133.51 × 4 = 534.03 (dm^4/m).

= Df2 = 134.83 × 4 = 539.32 (dm^4/m).

Bottom floor = Df1 = 971.86 × 4 = 971.86 (dm^4/m).

Summation of rigidity for each floor.

Examples for calculations of summation of rigidity for 20th story:

∑D20 = 1249.615 + 237.37 + 2303.456 + 837.141 = 4627.583 dm^4/m.

Table 4.9: Summation of Rigidity for Each Floor in (Y-Dir.).

Story	∑Di & ∑Dfi (dm^4/m)				∑Di Total (dm^4/m)
	Columns	Col - Shear	connection with shear wall right and left	connection with core	
20	1249.615	237.37	2303.456	837.141	4627.583
19	1249.615	237.37	1535.64	534.027	3556.650
18	1249.615	237.37	1535.64	534.027	3556.650
17	1249.615	237.37	1535.64	534.027	3556.650
16	1249.615	237.37	1535.64	534.027	3556.650
15	1249.615	237.37	1535.64	534.027	3556.650
14	1249.615	237.37	1535.64	534.027	3556.650
13	1249.615	237.37	1535.64	534.027	3556.650
12	1249.615	237.37	1535.64	534.027	3556.650

Table 4.9: Summation of Rigidity for Each Floor in (Y-Dir).

"Table Continued"

Story	∑Di & ∑Dfi (dm⁴/m)				∑Di Total (dm⁴/m)
	Columns	Col - Shear	connection with shear wall right and left	connection with core	
11	1258.154	237.37	1535.64	534.027	3565.189
10	1446.361	237.37	1535.64	534.027	3753.395
9	1446.361	237.37	1535.64	534.027	3753.395
8	1446.361	237.37	1535.64	534.027	3753.395
7	1446.361	237.37	1535.64	534.027	3753.395
6	1446.361	237.37	1535.64	534.027	3753.395
5	1489.944	237.37	1535.64	534.027	3796.979
4	1489.944	237.37	1535.64	534.027	3796.979
3	1489.944	237.37	1535.64	534.027	3796.979
2	1489.944	237.37	1535.64	539.321	3802.273
1	2152.824	237.52	3071.275	971.861	6433.480

4.3.1.4 Shear rigidity of story ∑Dij (Kn/M)

We considered the value of E=1, then (E/hi2) values are multiplied by ∑D using E - concrete (33000 N/mm2).

∑D20j = (33000 × 103 × 4627.583 × 10-4) / 32 = 1696780.37 kN/m.

Table 4.10: Shear Rigidity of Story in (Y-Dir.).

Floor	hi (m)	∑Di dm⁴/m	∑Dij kN / m
20	3	4627.583	1696780.37
19	3	3556.650	1304104.92
18	3	3556.650	1304104.92
17	3	3556.650	1304104.92
16	3	3556.650	1304104.92
15	3	3556.650	1304104.92
14	3	3556.650	1304104.92
13	3	3556.650	1304104.92
12	3	3556.650	1304104.92

Table 4.10: Shear Rigidity of Story in (Y-Dir.).

"Table continued"

Floor	hi (m)	∑Di dm^4/m	∑Dij kN / m
11	3	3565.189	1307235.85
10	3	3753.395	1376244.96
9	3	3753.395	1376244.96
8	3	3753.395	1376244.96
7	3	3753.395	1376244.96
6	3	3753.395	1376244.96
5	3	3796.979	1392225.50
4	3	3796.979	1392225.50
3	3	3796.979	1392225.50
2	3	3802.273	1394166.79
1	4	6433.480	1326905.29

4.3.2 Calculation in X-Dir

4.3.2.1 Calculation of moment of inertia of structure member

Moment of inertia for all rectangular shape structure member can be calculated by using the equation (3.7).

Examples for calculations:

Column C90×35: b=0.35 m, h=0.90 m.

I = b×h3/12 = 0.35 × 0.903 / 12 = 0.0213 m4.

Core Shear wall: b1=4.35 m, h1=1.00 m, b2=3.65 m, h2=0.65.

I1 = b1×h13/12 = 4.35 × 1.003 / 12 = 0.363 m4.

I2 = b2×h23/12 = 3.65 × 0.653 / 12 = 0.084 m4.

Pure I = I1 - I2 = (0.363 – 0.084) = 0.279 m4.

Table 4.11: The Moment of Inertia (I) Values for Each Structural Member (X-Dir.).

<table>
<tr><th colspan="2">Member</th><th>b
(m)</th><th>h
(m)</th><th>bh3/12
(m4)</th><th>Total I
(m4)</th></tr>
<tr><td rowspan="4">Columns</td><td>90×35</td><td>0.35</td><td>0.9</td><td>0.0213</td><td>0.0213</td></tr>
<tr><td>40×70</td><td>0.7</td><td>0.4</td><td>0.0037</td><td>0.0037</td></tr>
<tr><td>35×70</td><td>0.7</td><td>0.35</td><td>0.0025</td><td>0.0025</td></tr>
<tr><td>35×60</td><td>0.6</td><td>0.35</td><td>0.0021</td><td>0.0021</td></tr>
<tr><td rowspan="3">Beam</td><td>Normal</td><td>0.25</td><td>0.6</td><td>0.0045</td><td>0.0045</td></tr>
<tr><td>Coupled</td><td>0.25</td><td>0.7</td><td>0.0071</td><td>0.0071</td></tr>
<tr><td>Cracked</td><td colspan="2">0.5 × Ic.b</td><td>0.0036</td><td>0.0036</td></tr>
<tr><td rowspan="2">Normal Shear Wall</td><td>Left and Right</td><td>4.5</td><td>0.35</td><td>0.0161</td><td>0.0161</td></tr>
<tr><td>Center</td><td>0.35</td><td>4</td><td>1.8667</td><td>1.8667</td></tr>
<tr><td rowspan="2">Core Shear Wall</td><td>A1</td><td>4.35</td><td>1</td><td>0.3625</td><td rowspan="2">0.2790</td></tr>
<tr><td>A2</td><td>3.65</td><td>0.65</td><td>0.0835</td></tr>
</table>

4.3.2.2 Rigidity of columns (Dci)

a. Calculation of Stiffness of Columns (Kc).

Obtain the stiffness of columns by using the equation (3-8):

Examples for calculations:

KC90×35,1st = 212.63 / 4 = 53.16 dm⁴/m.

KC40×70,2nd = 37.33 / 3 = 12.44 dm⁴/m.

Table 4.12: The Stiffness of Columns (Kc) Values for Each Column (X-Dir.).

<table>
<tr><th rowspan="2">Floor</th><th rowspan="2">h_i
m</th><th colspan="4">I_c (dm^4)</th><th colspan="4">K_c (dm^4/m)</th></tr>
<tr><th>$C90_{\times 35}$</th><th>$C40_{\times 70}$</th><th>$C35_{\times 70}$</th><th>$C35_{\times 60}$</th><th>$C90_{\times 35}$</th><th>$C40_{\times 70}$</th><th>$C35_{\times 70}$</th><th>$C35_{\times 60}$</th></tr>
<tr><td>11 to 20</td><td>3</td><td rowspan="4">212.63</td><td rowspan="2">-</td><td>-</td><td>21.44</td><td>70.88</td><td rowspan="2">-</td><td>-</td><td>7.15</td></tr>
<tr><td>6 to 10</td><td>3</td><td>25.01</td><td rowspan="3">-</td><td>70.88</td><td>8.34</td><td rowspan="3">-</td></tr>
<tr><td>2 to 5</td><td>3</td><td rowspan="2">37.33</td><td rowspan="2">-</td><td>70.88</td><td>12.44</td><td rowspan="2">-</td></tr>
<tr><td>1</td><td>4</td><td>53.16</td><td>9.33</td></tr>
</table>

b. Calculation of stiffness of beams (Kb).

Obtain the stiffness of columns by using the equation (3-9):

Examples for calculations:

Kb = Ib / Lspan

I normal beam = 45 dm4 and I coupled beam = 71.46 dm4

Kb1,1st = 45 / 4.38 = 10.29 dm^4/m.

Kb5,11st = 45 / 7.15 = 6.29 dm^4/m.

Kb3,6th = 71.46 / 6.93 = 10.32 dm^4/m.

Table 4.13: The Stiffness of Beams (Kb) Values for Each Beam (X-Dir.).

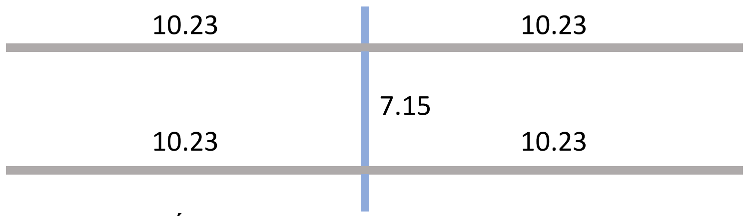
Floor	Length of span (m)					K_b (dm^4/m)				
	B1	B2	B3	B4	B5	B1	B2	B3	B4	B5
6 to 20	4.40	4.13	6.93	4.40	7.15	10.23	10.91	10.32	10.23	6.29
1 to 5	4.38	4.10	6.93	4.35	7.13	10.29	10.98	10.32	10.34	6.32

c. Calculation of K' and a value.

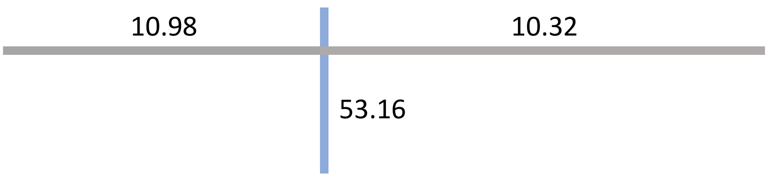
Obtain K' value by using the equation (3.10) for normal and top story and equation (3.11) for bottom story.

Examples for calculations:

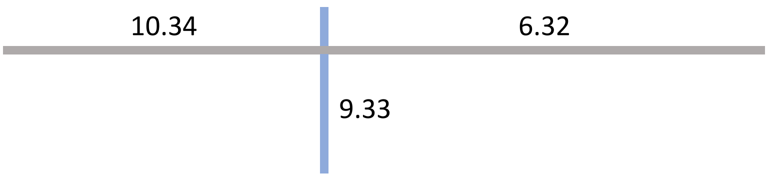
$$\acute{K}\ C3,\ 20th = (10.23+10.23+10.23+10.23) / (2\times7.15) = 2.862$$



$$\acute{K}_{C2,\ 1st} = (10.98+10.32) / (53.16) = 0.401$$



$$\acute{K}_{C4,\ 1st} = (10.34+6.32) / (9.33) = 11.02$$



Obtain (a) value by using the equation (3.12) for story column and equation (3.13) for Rigid column.

Examples for calculations of (a):

Story column C2,20th: a = 0.30 / (2+0.30) = 0.13

Rigid column C2,1st: a = (0.5+0.401) / (2+0.401) = 0.375

The Ḱ and (a) values for each floor are obtained and found in table 4.14.

Table 4.14: The Ḱ and (a) Values for Each Floor (X-Dir.).

<table>
<tr><th rowspan="2">Floor</th><th colspan="4">K'</th><th colspan="4">a</th></tr>
<tr><th>C1</th><th>C2</th><th>C3</th><th>C4</th><th>C1</th><th>C2</th><th>C3</th><th>C4</th></tr>
<tr><td>11 to 20</td><td>2.958</td><td rowspan="4">0.300</td><td>2.862</td><td>2.312</td><td>0.597</td><td rowspan="2">0.130</td><td>0.589</td><td>0.536</td></tr>
<tr><td>7 to 10</td><td>2.535</td><td>2.454</td><td>1.982</td><td>0.559</td><td>0.551</td><td>0.498</td></tr>
<tr><td>6</td><td>2.543</td><td>2.464</td><td>2.322</td><td>0.560</td><td>0.130</td><td>0.552</td><td>0.537</td></tr>
<tr><td>2 to 5</td><td>1.708</td><td>1.658</td><td>1.339</td><td>0.461</td><td>0.130</td><td>0.453</td><td>0.401</td></tr>
<tr><td>1</td><td>11.462</td><td>0.401</td><td>11.394</td><td>11.022</td><td>0.889</td><td>0.375</td><td>0.888</td><td>0.885</td></tr>
</table>

d. Calculation of rigidity of columns (Dc).
Obtain the rigidity of columns by using the equation (3.15):

Examples for calculations of (Dc):

D C2,20th = 12 × 0.13 × 70.88 = 110.78 dm^4/m

D C2,1st = 12 × 0.375 × 53.16 = 239.30 dm^4/m

Examples for calculations of (∑Dc):

∑D C2,20th = 110.78 × 4 = 443.113 dm^4/m

∑D C2,1st = 239.30 × 4 = 957.212 dm^4/m

Examples for calculations of (∑Dci):

∑Dc,20th = 204.63 + 443.11 + 201.92 + 183.91 = 1033.57 dm^4/m

∑Dc,1st = 191.74 + 443.11 + 220.46 + 199.16 = 1054.48 dm^4/m

Table 4.15: The Rigidity and Summation of Rigidity of Columns Values (X-Dir.).

Floor	Dc (dm⁴/m)				∑Dc (dm⁴/m)				∑Dci (dm⁴/m)
	C1	C2	C3	C4	C1	C2	C3	C4	
11 to 20	51.16	110.78	50.48	45.98	204.63	443.11	201.92	183.91	1033.57
7 to 10	47.94		55.11	49.79	191.74		220.46	199.16	1054.48
6	48.00	110.78	55.22	53.74	191.99	443.11	220.88	214.98	1070.97
2 to 5	68.80	110.78	67.68	59.88	275.19	443.11	270.73	239.52	1228.55
1	99.52	239.30	99.46	99.10	398.08	957.21	397.83	396.39	2149.51

4.3.2 Fictive Shear Stiffness for the Coupled Beam (DFI).

a. Shear wall (Center) connection.

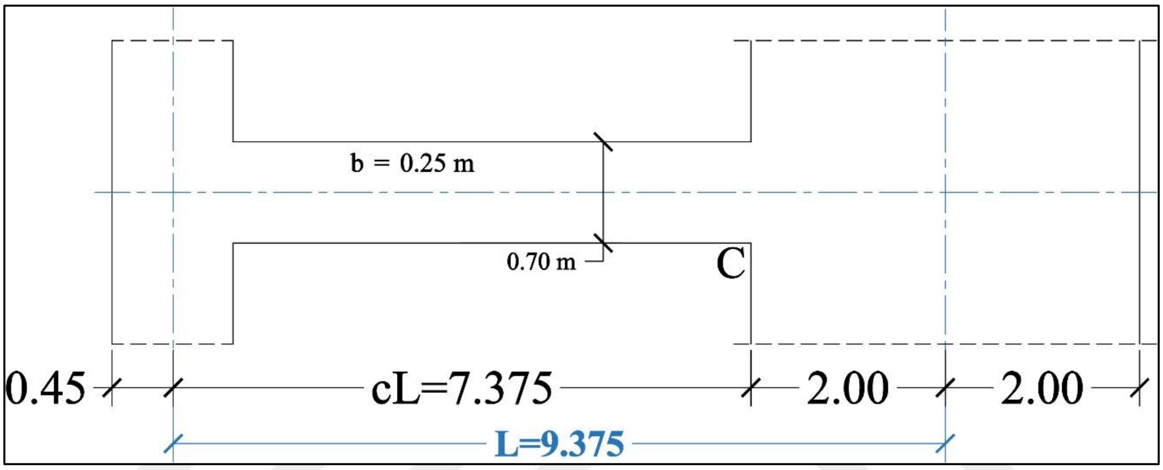


Figure 4.6: Coupled Beams Connected Between Center Shear Wall and Column (X-Dir.).

From Figure (4-6):

xvii. a1L = 2.00 m.

xviii. cL = 7.375 m.

xix. L = cL + a1L = 7.375 + 2.00 = 9.375 m.

xx. c = 7.375/9.375 = 0.787

xxi. a1 = 2.00/9.375 = 0.231

xxii. shape factor of rectangular = 5/6

xxiii. A'= Shape factor × Area c,b = 5/6 × (0.25 × 0.70) = 0.175 m2

xxiv. E/G = 2.5 taking E = 1.

xxv. Ibf,c = Ib,c / c2 = 0.0071 / 0.7872 = 0.01155 m4

xxvi. Kbf,c = Ibf,c / L = 0.01155 / 6.925 = 0.00167 m3

β = (3 × Ibc / cl2×A') × (E/G) = (3×0.0071/ 7.3752×0.1458) × 2.5 = 0.00676

Kβ = 1/(1+4×β) = 1/ (1+4×0.00676) = 0.97

Examples for calculations of rectifiers for story 20th:

Kc = 0.0071 m3

Kc,o = 0.

Kc,u = 0.0071 m3

ro = Kc / (Kc,o + Kbf,c) = 0.0071 / (0 + 0.00167) = 4.251

ru= Kc / (Kc,u + Kbf,c) = 0.0071 / (0.0071 + 0.00167) = 0.81

rm= (ro + ru) / 2 = (4.251 + 0.81) / 2 = 2.53 > 0.67

Table 4.16: The (RM) Value for all Stories for Shear Wall (Center) Connection (X-Dir.).

Story	Kc	$Kc_{,o}$	$Kc_{,u}$	$K_{bf,c}$	ro	ru	rm
20	0.00709	0.00000	0.00709	0.00167	4.251	0.810	2.530
3 to 19	0.00709	0.00709	0.00709	0.00167	0.810	0.810	0.810
2	0.00709	0.00709	0.00532	0.00167	0.810	1.015	0.912
1	0.00532	0.00709	0.00000	0.00167	0.607	3.188	1.898

From the table 4.16 above we notice the rm value for all stories more than 0.67, therefore the rectifier value will be the same for all stories.

Rectifier value with considering the shear deformation:

Rc = (6EIb / c3L)× [2 + c] × Kβ

= (6×0.0036×104/0.7873 × 9.375)× [2 + 0.787] × 0.97 = 127.45 dm4/m.

Fictive shear stiffness Dfi is calculated as follows for one connecting beam end:

Top floor = Df20 = 127.45 × 1.5 = 191.17 (dm^4/m).

Intermediate floors = Df19 to Df3 = 127.45 (dm^4/m).

= Df2 = 127.45 (dm^4/m).

Bottom floor = Df1 = 127.45 × 2 = 254.90 (dm^4/m).

∑Dfi for each floor:

Top floor, Df20 = 191.17 × 4 = 764.69 (dm^4/m).

Intermediate floors, Df19 to Df3 = 127.45 × 4 = 509.79 (dm^4/m).

Df2 = 127.45 × 4 = 509.79 (dm^4/m).

Bottom floor, Df1 = 254.90 × 4 = 1019.59 (dm^4/m).

b. Shear wall (Right and Left) connection.
Right and left share wall are Perpendicular to the force action in X-direction it should be taken into consideration as column widths.

Lu ≥ max [0.2 Lw , 2 Bw] = [0.2 × 4.35, 2 × 0.35] = [0.9, 0.7] = 0.9m

Obtain Dc for the considered column with dimension (0.9m × 0.35m) by using Muto method using the equation from (3.7) to (3.16).

Calculations 20th to 2nd floor:
Ic = (0.9 × 0.353) × 104 /12 = 32.16 dm4

Kc = 32.16 / 3 = 10.72 dm4/m

Kb = 0.0045 × 104/ 4.375 = 16.33 dm4/m

K’ = (16.33 + 16.33) / 2 × 10.72 = 1.52

a = 1.52 / (2 + 1.52) = 0.432

Dc = 10.72 × 0.432 × 12 = 55.62 dm4/m

Calculations 1st floor:
Kc = 32.16 / 4 = 8.04 dm4/m

K’ = (16.33) / 8.04 = 2.03

a = (0.5 + 2.03) / (2 + 2.03) = 0.628

Dc = 8.04 × 0.628 × 12 = 60.58 dm4/m

Calculations of (∑Dci):
∑Dc20th = 55.62 × 8 = 444.97 dm^4/m

∑Dc1th = 60.58 × 8 = 484.62 dm^4/m

c. Core Shear wall connection.
For core shear wall connection (end of core) it’s required to convert it to equivalent rectangle by using the moment of inertia.

Icore = 0.279 m4, b = 4.35 m

I = (b ×h3) / 12

0.279 = (4.35 ×h3) / 12

h = 0.92 m

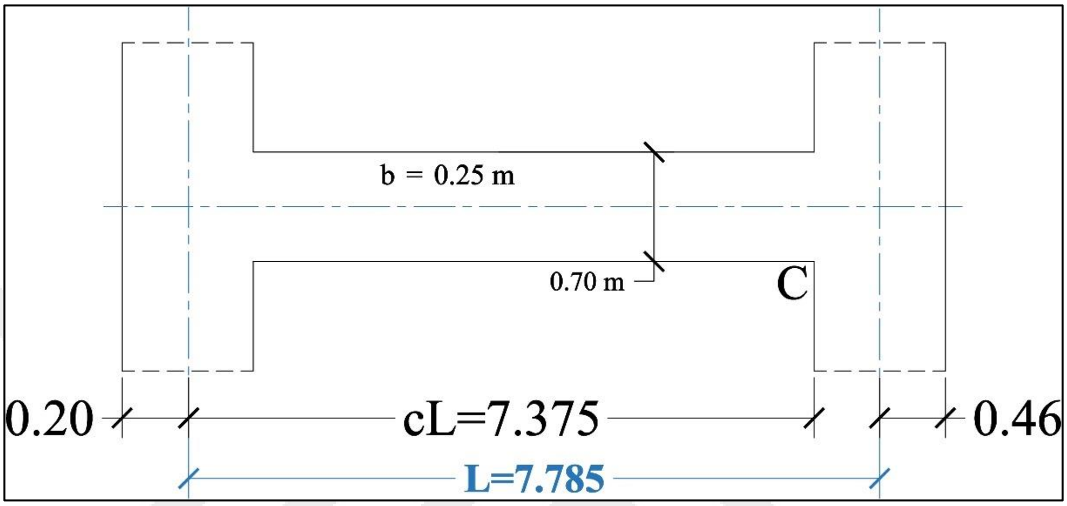


Figure 4.7: Coupled Beams Connected Between Equivalent Core Wall and Column (X-Dir.).

From figure (4.7):

xxvii. a1L = 0.46 m.

xxviii. cL = 7.325 m.

xxix. L = cL + a1L = 7.325 + 0.46 = 7.785 m.

xxx. c = 7.325/7.785 = 0.941

xxxi. a1 = 0.46/7.785 = 0.059

xxxii. shape factor of rectangular = 5/6

xxxiii. A'= Shape factor × Area c,b = 5/6 × (0.25 × 0.70) = 0.175 m2

xxxiv. E/G = 2.5 taking E = 1.

Ibf,c = Ib,c / c2 = 0.0071 / 0.9412 = 0.0081 m4

Kbf,c = Ibf,c / L = 0.00807 / 7.125 = 0.00113 m3

β = (3 × Ibc / cl2×A') × (E/G) = (3×0.0071/ 7.3252×0.1458) × 2.5

= 0.0069

Kβ = 1/(1+4×β) = 1/ (1+4×0.0069) = 0.97

Examples for calculations of rectifiers for story 20th:

Kc = 0.00071 m3

Kc,o = 0.

Kc,u = 0.00071 m3

ro = Kc / (Kc,o + Kbf,c) = 0.00071 / (0 + 0.00113) = 0.631

ru= Kc / (Kc,u + Kbf,c) = 0.00071 / (0.00071 + 0.00113) = 0.387

rm= (ro + ru) / 2 = (0.631 + 0.387) / 2 = 0.509 < 0.67

Table 4.17: The (RM) and Rectifier Value for Each Storey for Core Connection in (X-Dir.).

Story no.	Kc	$Kc_{,o}$	$Kc_{,u}$	$K_{bf,c}$	ro	ru	rm	RC
20	0.00071	0	0.00071	0.00113	0.631	0.387	0.509	76.03
12 - 19	0.00071	0.00071	0.00071	0.00113	0.387	0.387	0.387	61.66
11	0.00071	0.00071	0.00083	0.00113	0.387	0.363	0.375	60.28
10	0.00083	0.00071	0.00083	0.00113	0.451	0.424	0.438	67.64
7 - 9	0.00083	0.00083	0.00083	0.00113	0.424	0.424	0.424	66.03
6	0.00083	0.00083	0.00124	0.00113	0.424	0.351	0.387	61.72
5	0.00124	0.00083	0.00124	0.00113	0.633	0.523	0.578	84.20
4 - 3	0.00124	0.00124	0.00124	0.00113	0.523	0.523	0.523	77.76
2	0.00124	0.00124	0.00093	0.00113	0.523	0.602	0.563	82.40
1	0.00093	0.00124	0.00000	0.00113	0.393	0.824	0.608	87.74

From the table 4.17 above we notice the average stiffness ratio coefficient (r_m) value for all stories less than 0.67, therefore the rectifier value will be depending on the (r_m) coefficient for each storey and the rectifier value for all stories can be found in table 4.17 above.

Rectifier value with considering the shear deformation:

$Rc = (3EI_{b,c} / c^3L)\ [(3/2)\ (3+2c)\ r_m +1] \times K\beta$

$= (3\times0.0036\times10^4/0.94^3 \times 7.785)\ [(3/2) \times (3+2\times 0.94) \times 0.509+1] \times 0.97$

$= 76.03\ dm^4/m.$

Fictive Shear stiffness D_{fi} is calculated as follows for one connecting beam end:

Top floor = $D_{f20} = Rc_{20} \times 1.5$

Intermediate floors = D_{f19} to D_{f2} = Rc_i

Bottom floor = D_{f1} = $Rc_1 \times 2$

Examples for calculations of ∑Dfi for 20th floor:

$\sum D_{f20} = 114.042 \times 4 = 456.166$ dm⁴/m.

Table 4.18: The D_{fi} and $\sum D_{fi}$ Values for Core Connection in (X-Dir.).

Story no.	D_{fi} (dm^4/m)	$\sum D_{fi}$ (dm^4/m)
20	114.042	456.166
12 - 19	61.656	246.623
11	60.276	241.104
10	67.641	270.563
7 - 9	66.031	264.123
6	61.716	246.864
5	84.198	336.791
4 - 3	77.757	311.028
2	82.400	329.599
1	175.488	701.951

4.3.2.3 Summation of rigidity for each floor

Examples for calculations of summation of rigidity for 20th story:

$\sum D20 = 1033.57 + 444.97 + 456.17 + 764.69 = 2699.402$ dm^4/m.

Table 4.19: Summation of Rigidity for Each Floor in (X-Dir.).

Story	∑Di & ∑Dfi (dm^4/m)				$\sum Di_{Total}$ (dm^4/m)
	Columns	Col - Shear Right and left	connection with core	connection with Center Shear wall	
20	1033.573	444.97	456.17	764.690	2699.402
19	1033.573	444.97	246.62	509.793	2234.963
18	1033.573	444.97	246.62	509.793	2234.963
17	1033.573	444.97	246.62	509.793	2234.963
16	1033.573	444.97	246.62	509.793	2234.963
15	1033.573	444.97	246.62	509.793	2234.963
14	1033.573	444.97	246.62	509.793	2234.963
13	1033.573	444.97	246.62	509.793	2234.963
12	1033.573	444.97	246.62	509.793	2234.963
11	1033.573	444.97	241.10	509.793	2229.443

4.3.2.4 Shear rigidity of story ∑Dij (Kn/M)

We considered the value of E=1, then (E/hi2) values are multiplied by ∑D using E - concrete (33000 N/mm2).

∑D20j = (33000 × 2699.402 × 10-4) / 32 = 989780.67 kN/m.

Table 4.20: Shear Rigidity of Story in (X-Dir.).

Floor	h_i (m)	$\sum D_i$ dm^4/m	$\sum D_{ij}$ kN / m
20	3	2699.402	989780.67
19	3	2234.963	819486.29
18	3	2234.963	819486.29
17	3	2234.963	819486.29
16	3	2234.963	819486.29
15	3	2234.963	819486.29
14	3	2234.963	819486.29
13	3	2234.963	819486.29
12	3	2234.963	819486.29
11	3	2229.443	817462.52
10	3	2279.806	835928.78
9	3	2273.366	833567.70
8	3	2273.366	833567.70
7	3	2273.366	833567.70
6	3	2272.598	833285.84
5	3	2520.106	924038.91
4	3	2494.343	914592.61
3	3	2494.343	914592.61
2	3	2512.915	921402.13
1	4	4355.674	898357.75

4.4 CALCULATION OF WIND FORCE (EUROCODE) AND IMPACT ON STRUCTURE IN Y- DIRECTION (CASE 1.1)

Calculate the wind force acting in building surface according to (EN 1991-1-4 (2005) (English): Eurocode 1: Actions on structures - Part 1.4: General actions - Wind actions).

4.4.1 Calculation of Wind Forces on the Structure

4.4.1.1 Terrain categories and parameters

Terrain category = 4

z_O = 1 m.

z_{min} = 10 m.

z_{oII} = 0.05 m.

z_{max} = 200 m.

From table 4.1 in EN 1991-1-4

According to terrain category.

4.4.1.2 Calculation of the mean wind–variation with height ($V_{m(z)}$).

$V_{m(Z)} = Cr_{(Z)} \times Co_{(Z)} \times V_b$ in (m/sec)

V_b = 30 m/s.

$C_{o(Z)}$ = 1. (The building construction in flat surface)

$C_{r(Z)} = Kr \times Ln\ (z/z_o)$ for $z_{min} \leq z \leq z_{max}$

$C_{r(Z)} = Kr \times Ln\ (z_{min}/z_o)$ for $z \leq z_{min}$

$K_r = 0.19 \times (z_o/z_{oII})^{0.07} = 0.19 \times (1/0.05)^{0.07} = 0.234$

Examples for calculations of Cr(Z):

a. For (z ≤ zmin) use Cr(Zmin) = Kr × Ln (zmin/zo).

Cr (4,7,10) = 0.234 × Ln (10/1) = 0.540 (4 ≤ 10), (7 ≤ 10) and (10 ≤ 10).

b. For (zmin ≤ z ≤ zmax) use Cr(Z) = Kr × Ln (z/zo).

Cr (13) = 0.234 × Ln (13/1) = 0.601 (10 ≤ 13 ≤ 200).

Cr (61) = 0.234 × Ln (61/1) = 0.963 (10 ≤ 61 ≤ 200).

Examples for calculations of Vm(Z):

Vm(Z) = Cr(Z) × Co(Z) × Vb

Vm(4) = 0.540 × 1 × 30 = 16.19 m/s.

Vm(13) = 0.601 × 1 × 30 = 18.03 m/s.

Vm(61) = 0.963 × 1 × 30 = 28.90 m/s.

Table 4.21: The Roughness Factor ($Cr_{(Z)}$) and The Mean Wind Velocity ($Vm_{(Z)}$) Values.

Story	Z(m)	Cr(z)	Vm(z)
1	4	0.540	16.19
2	7	0.540	16.19
3	10	0.540	16.19
4	13	0.601	18.03
5	16	0.650	19.49
6	19	0.690	20.70
7	22	0.724	21.73
8	25	0.754	22.63
9	28	0.781	23.42
10	31	0.805	24.14
11	34	0.826	24.79
12	37	0.846	25.38
13	40	0.864	25.93
14	43	0.881	26.44
15	46	0.897	26.91
16	49	0.912	27.36
17	52	0.926	27.78
18	55	0.939	28.17
19	58	0.951	28.54
20	61	0.963	28.90

4.4.1.2 Calculation of the wind turbulence ($I_{v(z)}$).

$I_{v(z)} = \sigma_v / V_{m(z)}$

$\sigma_v = Kr \times V_b \times k_I$, k_I = the recommended value is 1.

$\sigma_v = 0.234 \times 30 \times 1 = 7.030$

Examples for calculations:

Iv (4) = 7.030 / 16.19 = 0.434

Iv (13) = 7.030 / 18.03 = 0.390

Iv (61) = 7.030 / 28.90 = 0.243

Table 4.22: The Turbulence Intensity ($Iv_{(Z)}$) Values.

Story	Z(m)	Iv(z)
1	4	0.434
2	7	0.434
3	10	0.434
4	13	0.390
5	16	0.361
6	19	0.340
7	22	0.324
8	25	0.311
9	28	0.300
10	31	0.291
11	34	0.284
12	37	0.277
13	40	0.271
14	43	0.266
15	46	0.261
16	49	0.257
17	52	0.253
18	55	0.250
19	58	0.246
20	61	0.243

4.4.1.3 Calculation of the peak velocity pressure

$q_{p(z)}$ (kN/m^2) = $[1+(7\times I_{v(z)})] \times (1/2) \times \rho \times V_{m(z)}{}^2$

ρ = 0.00125 KNm/s2

Vm(z) and Iv(z): from table 4-31 and 4.32 respectively.

Examples for calculations:

qp(4) = [1+(7×0.434)] × (1/2) × 0.00125 × (16.19)2 = 0.662 kN/m2

qp(13) = [1+(7×0.390)] × (1/2) × 0.00125 × (18.03)2 = 0.758 kN/m2

$q_{p(61)} = [1+(7\times 0.243)] \times (1/2) \times 0.00125 \times (28.90)^2 = 1.411$ kN/m^2

Table 4.23: The Peak Velocity Pressure (qp(z)) Values.

Story	Z(m)	qp(z)
1	4	0.662
2	7	0.662
3	10	0.662
4	13	0.758
5	16	0.837
6	19	0.904
7	22	0.963
8	25	1.016
9	28	1.063
10	31	1.107
11	34	1.147
12	37	1.183
13	40	1.218
14	43	1.250
15	46	1.281
16	49	1.309
17	52	1.337
18	55	1.362
19	58	1.387
20	61	1.411

4.4.1.4 Calculation of wind forces on surface

$F_w = c_s c_d \times c_f \times A_{ref} \times q_{p(ze)}$

First, obtain the Force coefficient (cf) and structural factor (cscd).

a. Force coefficient (cf).

cf = cf,0 $\times$ ψr $\times$ ψλ

cf = 2.146 $\times$ 1 $\times$ 0.635 (Data calculation below)

cf = 1.363.

Data calculation of force coefficient:

l.Effective slenderness (λ).

For elements with rectangular cross-section and length l ≥ 50 m the effective slenderness λ is equal to:

λ = λ50 = min (1.4 × l / b, 70)

= min (1.4 × 61 m / 38, 70) = 2.247

m. End effect factor ($\psi\lambda$).

The value of the end effect factor $\psi\lambda$ is determined from EN1991-1-4 Figure 7.36 as a function of the slenderness λ.

solidity ratio φ = 1.000 and $\psi\lambda$ = 0.635.

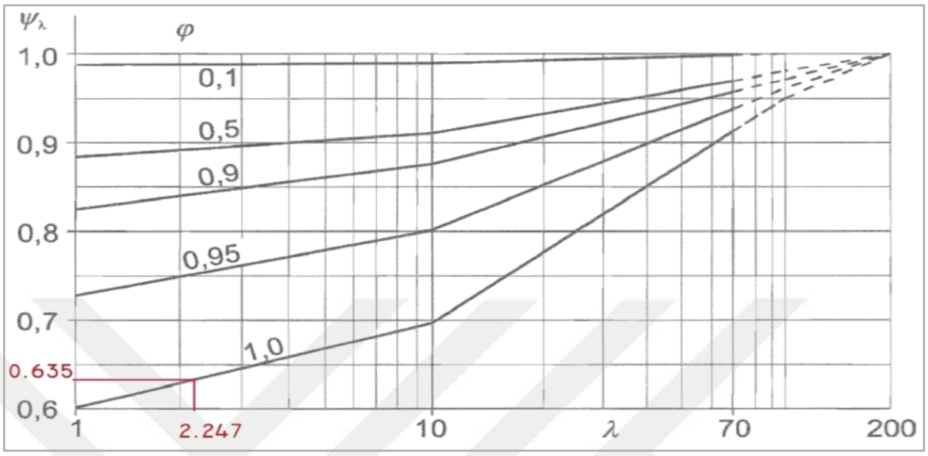


Figure 4.8: End Effect Factor in (Y-Dir).

n. Force coefficient without free-end flow ($c_{f,0}$).

The value of cf,0 is determined according to EN1991-1-4 Figure 7.23 for the values of d= 12 m, b= 38 m, d/b= 0.316.

cf,0 = 2.146.

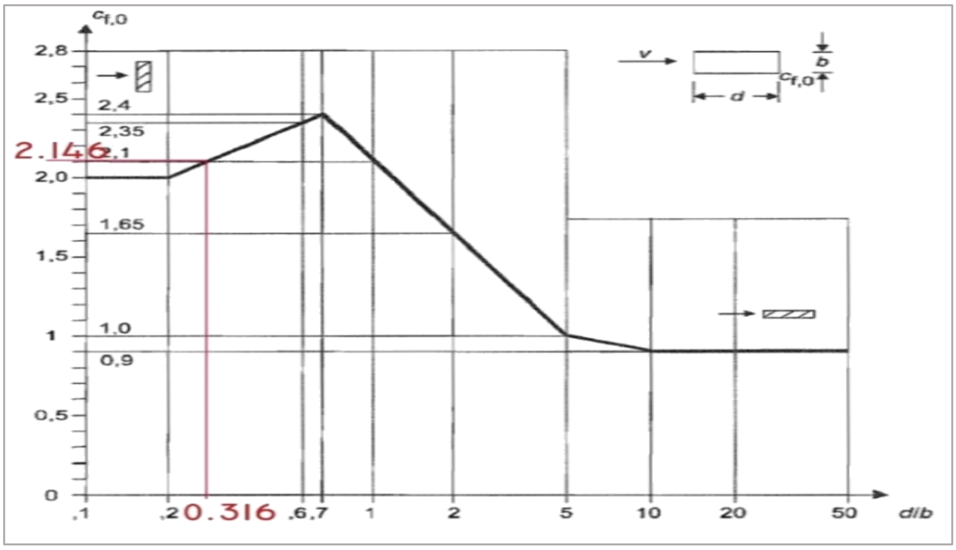


Figure 4.9: Force Coefficient Without Free-End Flow in (Y-Dir.).

o. Reduction factor for rounded corners (ψ_r).

The value of ψr is calculated in accordance with EN1991-1-4 -7.6(1).

For cross-sections with sharp corners (r = 0.0 m) there is no reduction, $\psi_r = 1$.

a. Structural factor (c_sc_d).

The size factor (C_s): (Data calculation below)

$C_s = 1 + 7 \times 0.278 \times (0.5217)^{0.5} / 1 + 7 \times 0.278 = 0.817$

The dynamic factor (C_d): (Data calculation below)

$C_d = 1 + 2 \times 3.351 \times 0.278 \times (0.0626 + 0.5217)^{0.5} / 1 + 7 \times 0.278 \times (0.5217)^{0.5} = 1.008$

$C_s \times C_d = 0.817 \times 1.008 = 0.82$

Data calculation of structural factor (cscd):

By using the procedure mention in item (3.3.1.5.II)

$Z_{(s)}$ for vertical structures such as buildings (Case 1).

$Z_{(s)} = 0.6 \times 61 = 36.6 \geq 10$

$Cr_{(z)} = 0.234 \times Ln\ (36.6/1) = 0.844$

$V_{m(z)} = 0.844 \times 30 = 25.31$ m/sec

The fundamental flexural frequency (n_1) = 46/h = 46/62 = 0.75 Hz

p. The turbulent length scale ($L_{(z)}$):

$\alpha = 0.67 + 0.05 \times Ln\ (1) = 0.67$

$L_{(Zs)} = 300 \times (36.6/200)^{0.67} = 96.153$

q. The non-dimensional power spectral density function ($S_{L(zs,n)}$).

$f_{L(zs,n)} = 0.75 \times 36.6 / 25.31 = 2.87$

$S_{L(zs,n)} = 6.8 \times 2.87 / (1+10.2 \times 2.87)^{(5/3)} = 0.066$

r.The Background factor (B^2):

$B^2 = 1 / 1 + 0.9 \times (38 + 61 / 36.6)^{0.63} = 0.5217$

s. The resonance response factor (R^2):

$R^2 = (3.14^2 / 2 \times 0.1022) \times (0.066) \times (0.1125) \times (0.1736) = 0.0626$

d. Aerodynamic admittance functions (for 1st mode shape):

$\eta_h = (4.6 \times 61 / 96.15) \times 2.87 = 8.3611$

$\eta_b = (4.6 \times 38 / 96.15) \times 2.87 = 5.2085$

$R_h = (1/8.3611) - (1 / 2 \times 8.3611^2) \times (1 - e^{-2\times 8.3611}) = 0.1125$

R_b = (1/5.2085) - (1 / 2 × 5.2085^2) × (1 – $e^{-2\times5.2085}$) = 0.1736

e. Logarithmic decrement of damping (δ):

$\delta = \delta_s + \delta_{a} + \delta_d$

δ_s = 0.1 for reinforced concrete buildings (Table F.2 EN 1991-1-4)

δ_d = 0, because there are no special dissipative devices are added to the structure.

$\delta_a = C_f \times \rho \times b \times V_{m(zs)} / 2 \times n_1 \times ɱ_e$ = 0.0022

C_f = 1.363, ρ = 0.00125 KNm/sec^2 , b = 38m, $V_{m(zs)}$ = 25.31 m/sec

n_1 = 0.75 Hz and $ɱ_e$ = 504.90 kN/g Per area. (Data calculation below)

Data calculation of ɱe:

For cantilevered structures with a varying mass distribution, me may be approximated by the average value of m over the upper third of the structure h_3. (Item F.4.(2) EN 1991-1-4)

h = 61m.

h/3 = 20.33m.

h- h/3 = 40.667m, we get 40, it's over the 13th floor.

δ = 0.1 + 0.0022 + 0 = 0.1022

t. The up-crossing frequency (v):

v = 0.75 × (0.0626 / 0.0626 + 0.5217)0.5 = 0.247 Hz > 0.08 Hz

T = 600 sec, vT = 148.067

u. The peak factor (Kp):

Kp = (2 × Ln (148.067))0.5 + 0.6 / (2 × Ln (148.067))0.5 = 3.351 > 3

v. The turbulence intensity Iv(z) at height Z(s).

Z(s) = 36.6m, σv = 7.03 and Vm(zs) = 25.31 m/sec

Iv(zs) = 7.03 / 25.31 = 0.278

Examples for calculations of the wind force (Fw) acting on a structure:

Fw = cscd × cf × Aref × qp(ze)

Fw (4) = 0.82 × 1.363 × ((38 × 2 × 0.662) + (38 × 1.5 × 0.662)) = 98.68 kN.

Fw (13) = 0.82 ×1.363 × ((38 × 1.5 × 0.758) + (38 × 1.5 × 0.837)) = 101.94 kN.

Fw (34) = 0.82 × 1.363 × ((38 × 1.5 × 1.218) + (38 × 1.5 × 1.25)) = 148.94 kN.

Fw (61) = 0.82 × 1.363 × 38 × 1.5 × 1.411 = 90.18 kN.

The wind pressure acting on the external surfaces (We) and the wind force (Fw) acting on a structure values in Y-direction are calculated for all height and can be found in table 4.24, Figure 4.10 shown the wind pressure acting on the external surfaces (We) and Figure 4.11 shown the wind force acting on the external surfaces (Fw).

Table 4.24: The External Surfaces (We) and the Wind Force (FW) Acting on a Structure in (Y-Dir.).

$Z_{(m)}$	Story	hi	$qp_{(z)}$	h_{ref}	We	Fw
0 - 4	1	4	0.662	2	0.74	98.68
4 - 7	2	3	0.662	1.5	0.74	84.58
7 - 10	3	3	0.662	1.5	0.74	90.73
10 - 13	4	3	0.758	1.5	0.85	101.94
13 - 16	5	3	0.837	1.5	0.94	111.31
16 - 19	6	3	0.904	1.5	1.01	119.40
19 - 22	7	3	0.963	1.5	1.08	126.53
22 - 25	8	3	1.016	1.5	1.14	132.92
25 - 28	9	3	1.063	1.5	1.19	138.72
28 - 31	10	3	1.107	1.5	1.24	144.03
31 - 34	11	3	1.147	1.5	1.29	148.94
34 - 37	12	3	1.183	1.5	1.33	153.50
37 - 40	13	3	1.218	1.5	1.37	157.77
40 - 43	14	3	1.250	1.5	1.40	161.77
43 - 46	15	3	1.281	1.5	1.44	165.55
46 - 49	16	3	1.309	1.5	1.47	169.13
49 - 52	17	3	1.337	1.5	1.50	172.53
52 - 55	18	3	1.362	1.5	1.53	175.76
55 - 58	19	3	1.387	1.5	1.56	178.85

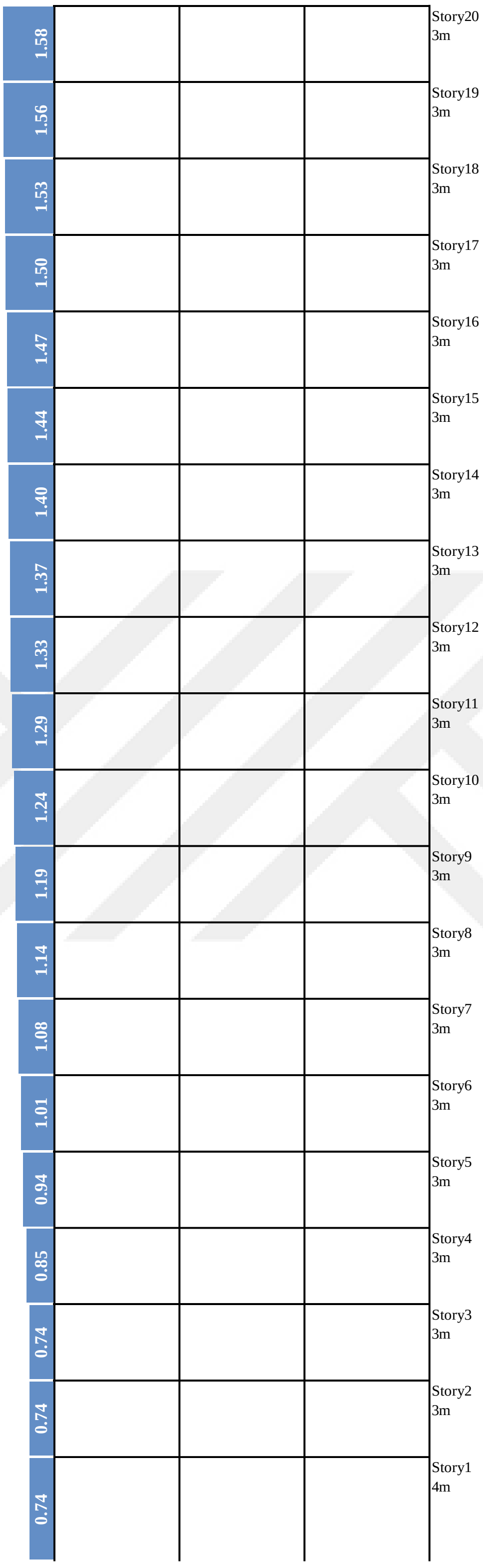


Figure 4.10: The Wind Pressure Acting on the External Surfaces (We) in (Case 1.1).

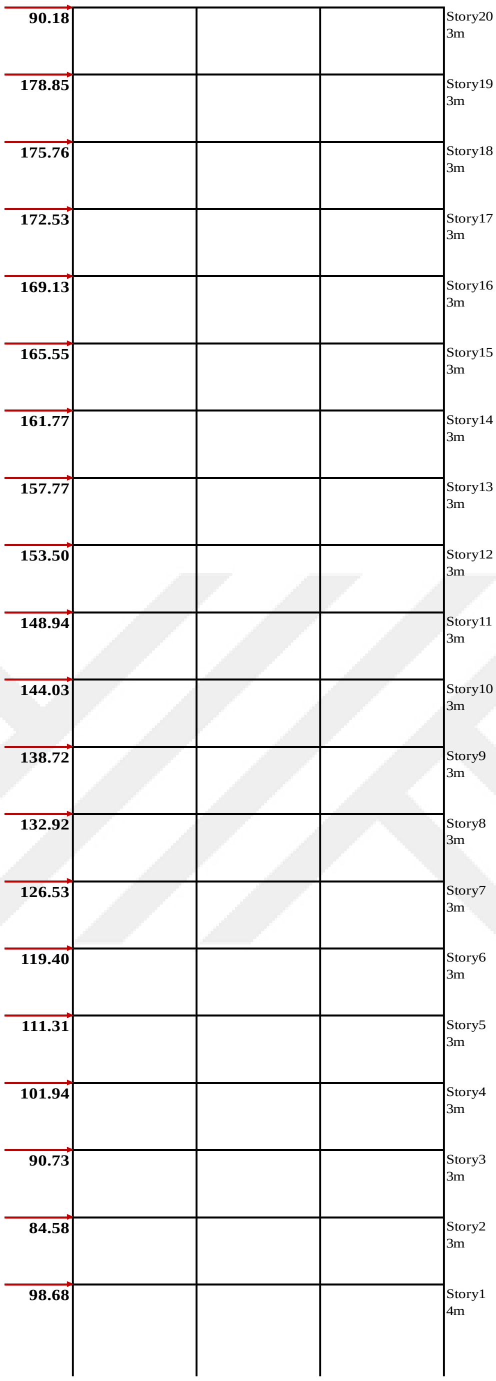


Figure 4.11: The Wind Force Acting on the External Surfaces (FW) in (Case 1.1).

4.4.2 Calculation of Shear Force (T_i) and Cantilever Moments (M_o)

Examples for calculations:

$Ti = \sum Pi$

T20 = P20 = 90.18 kN.

$T19 = T20 + P19 = 90.18 + 178.85 = 269.04$ kN.

$M_{o,i} = T_i \times h_i + M_{o,i+1}$

$Mo20 = T20 \times h20 = 90.18 \times 3 = 270.55$ kNm

$Mo19 = T19 \times h19 + Mo20 = 178.85 \times 3 + 270.55 = 1077.65$ kNm

Table 4.25: Shear Force (T_i) and Cantilever Moments (M_o) Calculation in (Case 1.1).

Story	h_i (m)	P_i (kN)	T_i (kN)	M_o (kNm)
19	3	178.85	269.04	1077.65
18	3	175.76	444.80	2412.04
17	3	172.53	617.32	4264.02
16	3	169.13	786.45	6623.37
15	3	165.55	952.00	9479.38
14	3	161.77	1113.77	12820.70
13	3	157.77	1271.54	16635.31
12	3	153.50	1425.04	20910.43
11	3	148.94	1573.98	25632.36
10	3	144.03	1718.01	30786.40
9	3	138.72	1856.73	36356.59
8	3	132.92	1989.65	42325.55
7	3	126.53	2116.18	48674.11
6	3	119.40	2235.58	55380.85
5	3	111.31	2346.89	62421.52
4	3	101.94	2448.83	69768.01
3	3	90.73	2539.56	77386.69
2	3	84.58	2624.14	85259.11
1	4	98.68	2722.82	96150.41

4.4.3 Calculations of Coefficients and Constants of Equation Set

Examples for calculations:

c. $F_i = 1/\sum D$

$F20 = (1 / 8627.58) \times 104 = 2.161$ 1/m3.

d. $\sum(EIp)_i$ = ip (shear wall right and left) × number + ip (shear wall center) × number + ip (core) × number

$\sum(EIp)i = (2.658 \times 4) + (0.014 \times 2) + (4.225 \times 2) = 19.11\ m^4$.

e. $fi = hi/(6\times\sum(EIp)i)$.

f20 = 3 / (6× 19.11) = 0.0262 1/m3.

f.Mo from table above.

Mo,20 = 270.55 kNm

g. δi,i-1 = fi-1 – Fi-1 .

δ20,19 = f19 – F19 = 0.0262 – 2.812 = -2.785 1/m3

h. δi,i = 2×fi + 2×fi-1 + Fi + Fi-1 .

δ20,20 = 2×f20 + 2×f19 + F20 + F19 = 2×0.0262 + 2×0.0262 + 2.161 + 2.812

= 5.077 1/m3.

i.δi,o = fi (Mi+1,o + 2 × Mi,o) - fi-1 (Mi-1,o + 2 × Mi,o).

δ20,o = f20 (M21,o + 2 × M20,o) - f19 (M19,o + 2 × M20,o)

= 0.0262 × (0 + 2 × 270.55) – 0.0262 × (1077.65 + 2 × 270.55)

= 56.51 kNm

The Fi, fi, δi,i-1 , δi,i and δi,o values are calculated for all story and can be found in table 4-26.

Table 4.26: The Fi, Fi, Δi,I-1 , Δi,I and Δi,O Values in (Case 1.1).

Floor	hi (m)	Fi ($1/m^3$)	fi ($1/m^3$)	Mo (kNm)	$\delta i_{,i-1}$ ($1/m^3$)	$\delta i_{,i}$ ($1/m^3$)	$\delta i_{,o}$ (kNm)
10	3	2.664	0.0262	30786.40	-2.638	5.433	4843.74
9	3	2.664	0.0262	36356.59	-2.638	5.433	5717.69
8	3	2.664	0.0262	42325.55	-2.638	5.433	6654.20
7	3	2.664	0.0262	48674.11	-2.638	5.433	7650.24
6	3	2.664	0.0262	55380.85	-2.638	5.403	8702.43
5	3	2.634	0.0262	62421.52	-2.608	5.372	9806.94
4	3	2.634	0.0262	69768.01	-2.608	5.372	10959.31
3	3	2.634	0.0262	77386.69	-2.608	5.368	12154.81
2	3	2.630	0.0262	85259.11	-2.604	4.306	15788.61
1	4	1.554	0.0349	96150.41	-1.519	1.624	9682.53

4.4.4. Calculations of Matrix Coefficients & Load Vector

a. [δ] × {Xi} = -{δi,o}

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		Xi		δi,o
5.077	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X20		-56.51
-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X19		-182.97
0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X18		-392.18
0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X17		-682.64
0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X16		-1052.73
0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X15		-1500.77
0.000	0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X14		-2024.98
0.000	0.000	0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X13		-2623.46
0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.785	5.721	-2.779	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X12		-3294.21
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.779	5.574	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	×	X11	=	-4035.08
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X10		-4843.74
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X9		-5717.69
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000		X8		-6654.20
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000		X7		-7650.24
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.403	-2.608	0.000	0.000	0.000	0.000		X6		-8702.43
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.608	5.372	-2.608	0.000	0.000	0.000		X5		-9806.94
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.608	5.372	-2.608	0.000	0.000		X4		-10959.31
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.608	5.368	-2.604	0.000		X3		-12154.81
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.604	4.306	-1.519		X2		-15788.61
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.519	1.624		X1		-9682.53

b. $[\delta]^{-1} \times -\{\delta_{i,o}\} = \{X_i\}$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		δi,o		Xi
0.347	0.274	0.216	0.171	0.135	0.107	0.084	0.067	0.053	0.042	0.033	0.026	0.021	0.017	0.013	0.011	0.009	0.008	0.007	0.007		-56.51		-2429.76
0.274	0.500	0.394	0.311	0.246	0.194	0.154	0.122	0.096	0.077	0.060	0.048	0.038	0.030	0.024	0.020	0.017	0.014	0.013	0.012		-182.97		-4408.58
0.216	0.394	0.595	0.470	0.371	0.293	0.232	0.183	0.145	0.115	0.091	0.072	0.057	0.045	0.037	0.030	0.025	0.022	0.019	0.018		-392.18		-6570.17
0.171	0.311	0.470	0.654	0.517	0.408	0.323	0.255	0.202	0.161	0.127	0.100	0.079	0.063	0.051	0.042	0.035	0.030	0.027	0.025		-682.64		-8961.23
0.135	0.246	0.371	0.517	0.691	0.546	0.432	0.342	0.271	0.215	0.170	0.134	0.106	0.085	0.068	0.056	0.047	0.040	0.036	0.034		-1052.73		-11612.25
0.107	0.194	0.293	0.408	0.546	0.715	0.565	0.447	0.355	0.282	0.222	0.175	0.139	0.111	0.089	0.073	0.061	0.053	0.048	0.044		-1500.77		-14539.76
0.084	0.154	0.232	0.323	0.432	0.565	0.731	0.578	0.459	0.364	0.287	0.226	0.179	0.143	0.115	0.094	0.079	0.068	0.061	0.057		-2024.98		-17747.90
0.067	0.122	0.183	0.255	0.342	0.447	0.578	0.742	0.588	0.467	0.368	0.290	0.230	0.184	0.148	0.121	0.101	0.087	0.079	0.074		-2623.46		-21229.28
0.053	0.096	0.145	0.202	0.271	0.355	0.459	0.588	0.751	0.597	0.470	0.371	0.294	0.235	0.189	0.154	0.129	0.112	0.101	0.094		-3294.21		-24965.23
0.042	0.077	0.115	0.161	0.215	0.282	0.364	0.467	0.597	0.760	0.599	0.472	0.374	0.299	0.241	0.197	0.164	0.142	0.128	0.120	×	-4035.08	=	-28935.09
0.033	0.060	0.091	0.127	0.170	0.222	0.287	0.368	0.470	0.599	0.770	0.607	0.481	0.384	0.310	0.253	0.211	0.183	0.165	0.154		-4843.74		-33308.85
0.026	0.048	0.072	0.100	0.134	0.175	0.226	0.290	0.371	0.472	0.607	0.779	0.617	0.492	0.397	0.324	0.271	0.234	0.211	0.198		-5717.69		-37828.58
0.021	0.038	0.057	0.079	0.106	0.139	0.179	0.230	0.294	0.374	0.481	0.617	0.790	0.630	0.508	0.415	0.347	0.300	0.270	0.253		-6654.20		-42431.95
0.017	0.030	0.045	0.063	0.085	0.111	0.143	0.184	0.235	0.299	0.384	0.492	0.630	0.805	0.649	0.530	0.443	0.383	0.346	0.323		-7650.24		-47037.88
0.013	0.024	0.037	0.051	0.068	0.089	0.115	0.148	0.189	0.241	0.310	0.397	0.508	0.649	0.829	0.677	0.566	0.489	0.441	0.413		-8702.43		-51542.89
0.011	0.020	0.030	0.042	0.056	0.073	0.094	0.121	0.154	0.197	0.253	0.324	0.415	0.530	0.677	0.866	0.724	0.626	0.565	0.528		-9806.94		-55866.34
0.009	0.017	0.025	0.035	0.047	0.061	0.079	0.101	0.129	0.164	0.211	0.271	0.347	0.443	0.566	0.724	0.926	0.800	0.722	0.675		-10959.31		-59792.07
0.008	0.014	0.022	0.030	0.040	0.053	0.068	0.087	0.112	0.142	0.183	0.234	0.300	0.383	0.489	0.626	0.800	1.022	0.923	0.863		-12154.81		-63114.49
0.007	0.013	0.019	0.027	0.036	0.048	0.061	0.079	0.101	0.128	0.165	0.211	0.270	0.346	0.441	0.565	0.722	0.923	1.179	1.103		-15788.61		-65578.60
0.007	0.012	0.018	0.025	0.034	0.044	0.057	0.074	0.094	0.120	0.154	0.198	0.253	0.323	0.413	0.528	0.675	0.863	1.103	1.648		-9682.53		-67314.60

The X_i values are calculated for all the story and can be found in table 4-27.

Table 4.27: The X_i Values Calculated for (Case 1.1).

Floor	Xi (kNm)
1	-2429.760
2	-4408.580
3	-6570.168
4	-8961.233
5	-11612.251
6	-14539.762
7	-17747.901

4.4.5 Calculations of Moment Superposition and Distribution it

Examples for calculations:

j. Mo from table above.

$M_{o,20}$ = 270.55 kNm

k. Xi from table above.

X_{20} = -2429.76 kNm

l. Mptotal = ($M_{i,o}$ + X_i).

Mptotal = ($M_{o,20}$ + X_{20}) = 270.55 + (-2429.76) = -2159.21 kNm.

m. IP single Shear wall- right and left = 2.658 m^4.

n. IP single Shear wall- center = 0.0143 m^4.

o. IP core Shear wall = 4.225 m^4.

p. ∑IP = 19.11 m^4.

q. M single shear wall = mptotal × (ip single shear wall/∑IP) .

M single shear wall (right and left),20 = -2159.21 × (2.658 / 19.11)

= -300.29 kNm.

M single shear wall (center),20 = -2159.21 × (0.0143 / 19.11)

= -1.61 kNm

r. M core shear wall = Mptotal × (Ip core shear wall/∑IP).

M core shear wall = -2159.21 × (4.225 / 19.11)

= -477.41 kNm

The total moment in shear wall and moment in single shear wall values are calculated for all story and can be found in table 4.28.

Table 4.28: The Total Moment in Shear Wall and Moment in Single Shear Wall Values in (Case1.1).

Floor				M single Shear wall		
	M_o (kNm)	X_i (kNm)	M_{Ptotal} (kNm)	(Right and Left) (kNm)	(Center) (kNm)	M_{Core} (kNm)
20	270.55	-2429.76	-2159.21	-300.29	-1.61	-477.41
19	1077.65	-4408.58	-3330.93	-463.25	-2.49	-736.48
18	2412.04	-6570.17	-4158.12	-578.29	-3.11	-919.37
17	4264.02	-8961.23	-4697.22	-653.26	-3.51	-1038.57
16	6623.37	-11612.25	-4988.88	-693.83	-3.73	-1103.06
15	9479.38	-14539.76	-5060.39	-703.77	-3.78	-1118.87
14	12820.70	-17747.90	-4927.21	-685.25	-3.68	-1089.42
13	16635.31	-21229.28	-4593.96	-638.90	-3.44	-1015.74
12	20910.43	-24965.23	-4054.80	-563.92	-3.03	-896.53
11	25632.36	-28935.09	-3302.73	-459.33	-2.47	-730.24
10	30786.40	-33308.85	-2522.45	-350.81	-1.89	-557.72
9	36356.59	-37828.58	-1471.99	-204.72	-1.10	-325.46
8	42325.55	-42431.95	-106.39	-14.80	-0.08	-23.52
7	48674.11	-47037.88	1636.23	227.56	1.22	361.78
6	55380.85	-51542.89	3837.96	533.76	2.87	848.58
5	62421.52	-55866.34	6555.19	911.66	4.90	1449.37
4	69768.01	-59792.07	9975.94	1387.40	7.46	2205.71
3	77386.69	-63114.49	14272.20	1984.90	10.67	3155.63
2	85259.11	-65578.60	19680.52	2737.06	14.72	4351.42
1	96150.41	-67314.60	28835.81	4010.33	21.56	6375.69

4.4.6 Calculations of Shear Forces

Examples for calculations:

s. $\sum Ti = \Delta X/hi = (Xi+1-Xi)/hi$

$\sum T20 = (X21-X20)/h20 = (0 – (-2429.76))/3 = 809.92$ kN. t. $Tcol = \sum T \times (Dc/\sum D)$

Tcol (C1),20 = 809.92 × (64.73 / 4627.58) = 11.33 kN. Tcol (C2),20 = 809.92 × (49.55 / 4627.58) = 8.67 kN.

Tcol (C3),20 = 809.92 × (100.26 / 4627.58) = 17.55 kN.

Tcol (C4),20 = 809.92 × (97.87 / 4627.58) = 17.13 kN.

Tshear w (R&L),20 = 809.92 × (575.86 / 4627.58) = 403.15 kN.

Tshear w (center),20 = 809.92 × (59.34 / 4627.58) = 41.54 kN.

Tcore,20 = 809.92 × (209.29 / 4627.58) = 146.52 kN.

Table 4.29: Shear Force Calculation in (Case 1.1).

Floor	∑T	T_{col}				T_{fi} Normal shear wall		T_{fi} Core
		C1	C2	C3	C4	Right and Left	Center	
16	883.67	16.08	12.31	24.91	24.32	381.54	58.98	132.68
15	975.84	17.76	13.59	27.51	26.85	421.33	65.13	146.52
14	1069.38	19.46	14.90	30.15	29.43	461.72	71.37	160.57
13	1160.46	21.12	16.17	32.71	31.93	501.05	77.45	174.24
12	1245.32	22.66	17.35	35.11	34.27	537.68	83.11	186.98
11	1323.29	24.02	18.48	37.67	36.58	569.98	88.10	198.21
10	1457.92	28.93	28.93	46.98	45.11	596.48	92.20	207.43
9	1506.58	29.90	29.90	48.55	46.62	616.39	95.28	214.35
8	1534.46	30.45	30.45	49.45	47.48	627.80	97.04	218.32
7	1535.31	30.47	30.47	49.47	47.51	628.15	97.10	218.44
6	1501.67	29.80	29.80	48.39	46.47	614.38	94.97	213.66
5	1441.15	28.95	28.27	47.71	45.74	582.85	90.09	202.69
4	1308.58	26.28	25.67	43.32	41.53	529.24	81.81	184.05
3	1107.47	22.24	21.73	36.66	35.15	447.90	69.23	155.76
2	821.37	16.47	16.09	27.15	26.03	331.73	51.28	116.50
1	434.00	9.44	9.44	11.61	11.39	207.19	16.02	65.56

4.4.7 Calculations of Coupled Beam End Moments

Examples for calculations:

u. For normal shear walls (right and left):

M20,u = 1/3 × 403.15 × 3 = 403.15 kKNm.

M20,o = 2/3 × 403.15 × 3 = 806.30 kNm.

M19,u = 1/2 × 284.8 × 3 = 427.19 kNm.

$M_{19,o}$ = 403.15 + 427.19 = 830.34 kNm.

Table 4.30: The Coupled Beam End Moments Values in (Case 1.1).

Floor	h_i (m)	Shear wall Right and Left		Core	
		$M_{i,u}$	$M_{i,o}$	$M_{i,u}$	$M_{i,o}$
19	3	427.19	830.34	148.56	295.08
18	3	466.65	893.84	162.28	310.84
17	3	516.19	982.84	179.51	341.79
16	3	572.31	1088.50	199.02	378.53
15	3	632.00	1204.31	219.78	418.80
14	3	692.58	1324.58	240.85	460.63
13	3	751.57	1444.15	261.36	502.21
12	3	806.53	1558.09	280.47	541.84
11	3	854.97	1661.50	297.32	577.80
10	3	894.72	1749.70	311.15	608.47
9	3	924.59	1819.31	321.53	632.68
8	3	941.69	1866.28	327.48	649.01
7	3	942.22	1883.91	327.66	655.14
6	3	921.58	1863.79	320.48	648.15
5	3	874.28	1795.85	304.04	624.52
4	3	793.86	1668.13	276.07	580.10
3	3	671.85	1465.71	233.64	509.71
2	3	497.59	1169.45	174.76	408.40
1	4	414.37	911.97	131.12	305.88

4.4.8 Calculation for Corrected Shear Wall and Core Moments at Bottom and Top

Examples for calculations shear wall:

a. Moment of shear wall (right and left) for 1st floor:

M1= 4010.33 + 414.37 = 4424.70 kNm

M2= 2737.06 – 414.37 = 2322.68 kNm

Moment of shear wall (right and left) for 2nd floor:

M1= 2737.06 + 497.59 = 3234.65 kNm

M2= 1984.9 – 497.59 = 1487.30 kNm

Table 4.31: The Corrected Shear Wall Moments at Bottom and Top Values in (Case 1.1).

Floor	Shear wall (Right and left) (kNm)			
	M single	$M_{i,u}$	M_1 Bottom	M_2 Upper
20	-300.29	403.15	102.86	-806.30
19	-463.25	427.19	-36.05	-727.48
18	-578.29	466.65	-111.64	-929.90
17	-653.26	516.19	-137.07	-1094.48
16	-693.83	572.31	-121.52	-1225.57
15	-703.77	632.00	-71.77	-1325.82
14	-685.25	692.58	7.33	-1396.35
13	-638.90	751.57	112.66	-1436.82
12	-563.92	806.53	242.61	-1445.43
11	-459.33	854.97	395.65	-1418.89
10	-350.81	894.72	543.92	-1354.05
9	-204.72	924.59	719.87	-1275.39
8	-14.80	941.69	926.90	-1146.41
7	227.56	942.22	1169.78	-957.02
6	533.76	921.58	1455.34	-694.02
5	911.66	874.28	1785.94	-340.52
4	1387.40	793.86	2181.25	117.80
3	1984.90	671.85	2656.75	715.54
2	2737.06	497.59	3234.65	1487.30
1	4010.33	414.37	4424.70	2322.68

Examples for calculations of core wall:

b. Moment of core shear wall for 1st floor:

M1= 6375.69 + 131.12 = 6506.81 kNm

M2= 4351.42 – 131.12 = 4220.30 kNm

c. Moment of core shear wall for 2nd floor:

M1= 4351.42 + 174.76 = 4526.18 kNm

M2= 3155.63 – 174.76 = 2980.87 kNm

Table 4.32: The Corrected Core Wall Moments at Bottom and Top Values in (Case 1.1).

Floor	Core Wall (kNm)			
	M single	$M_{i,u}$	M_1 Bottom	M_2 Upper
20	-477.41	146.52	-330.89	-293.03
19	-736.48	148.56	-587.92	-625.97
18	-919.37	162.28	-757.09	-898.76
17	-1038.57	179.51	-859.06	-1098.88
16	-1103.06	199.02	-904.03	-1237.59
15	-1118.87	219.78	-899.09	-1322.84
14	-1089.42	240.85	-848.57	-1359.72
13	-1015.74	261.36	-754.38	-1350.78
12	-896.53	280.47	-616.05	-1296.21
11	-730.24	297.32	-432.92	-1193.85
10	-557.72	311.15	-246.58	-1041.39
9	-325.46	321.53	-3.93	-879.25
8	-23.52	327.48	303.96	-652.94
7	361.78	327.66	689.44	-351.19
6	848.58	320.48	1169.07	41.29
5	1449.37	304.04	1753.41	544.55
4	2205.71	276.07	2481.78	1173.30
3	3155.63	233.64	3389.27	1972.07
2	4351.42	174.76	4526.18	2980.87
1	6375.69	131.12	6506.81	4220.30

4.4.9 Calculation of Displacement and Drift of Story

Examples for calculations:

a. $\sum T$, $\sum D_{ij}$ from table above.

$\sum T20$ = 809.92 kN.

$\sum D20j$ = = 1696780.37 kN/m

- $\delta i = \sum T / \sum Dij$ (m).

$\delta 20$ = 809.92 / 1696780.37 = 0.000477 m.

- $di = \sum \delta i$ (m). $d20 = \delta 20 + d19$ = 0.000477 + 0.0.015935 = 0.01641 m.

Table 4.33: The Displacement and Drift Values in (Case 1.1).

Floor	h_i (m)	$\sum T$ (kN)	$\sum D_{ij}$ kN / m	δ_i (m)	d_i (m)
17	3	797.02	1304104.92	0.000611	0.014877
16	3	883.67	1304104.92	0.000678	0.014265
15	3	975.84	1304104.92	0.000748	0.013588
14	3	1069.38	1304104.92	0.000820	0.012840
13	3	1160.46	1304104.92	0.000890	0.012020
12	3	1245.32	1304104.92	0.000955	0.011130
11	3	1323.29	1307235.85	0.001012	0.010175
10	3	1457.92	1376244.96	0.001059	0.009162
9	3	1506.58	1376244.96	0.001095	0.008103
8	3	1534.46	1376244.96	0.001115	0.007008
7	3	1535.31	1376244.96	0.001116	0.005893
6	3	1501.67	1376244.96	0.001091	0.004778
5	3	1441.15	1392225.50	0.001035	0.003687
4	3	1308.58	1392225.50	0.000940	0.002652
3	3	1107.47	1392225.50	0.000795	0.001712
2	3	821.37	1394166.79	0.000589	0.000916
1	4	434.00	1326905.29	0.000327	0.000327

Check the result Case (1.1):

The Muto method is used to calculate the sway of a framed building based on lateral wind forces:

Δ max = 0.000327+ 0.000589 + ……… + 0.000506 + 0.000477 = 0.01641 m

= 0.01641 m × 103 = 16.4 mm

The recommended maximum sway is Δ max-all = 0.002 × H building

Δ max-all = 0.002 × H = 0.002 × (61 m) × 103 = 122 mm > 16.4 mm. (O.K.)

The drift ratio = 0.001116 / 3 = 0.000372 < 0.02 (O.K.)

4.5 CALCULATION OF WIND FORCE (EQUIVALENT SEISMIC LOAD DISTRIBUTED) AND IMPACT ON STRUCTURE IN Y-DIR (CASE 1.2)

Calculate the Base shear force acting in building surface is complete according to (EN 1991-1-4) in item (This:4.4.1), distributed it to the structure floor using “Equivalent Seismic Load Method” and study the impact on the structure, as following:

4.5.1 Distributed Lateral Load

Base shear (V_t) = 2723 kN

N = 20 storey

$\Delta F_n = 0.0075 \times N \times V_t$

$\Delta F_n = 0.0075 \times 20 \times 2723 = 408.45$ kN

$\sum F_i = V_t - \Delta F_n = 2723 - 408.45 = 2314.55$ kN

By using the equation (3-79) to distribute the load:

$Fi = (Wi \times Hi / \sum Wj \times Hj) \times \sum Fi$

$F_{20} = (W_{20} \times H_{20} / \sum W_{20} \times H_{20}) \times \sum F_{20}$

$F20 = (4718.51 \times 61 / 3236278.46) \times 2314.55 = 205.85$ kN

$Fi,total = \Delta Fn + Fi$

$F20,total = \Delta F20 + F20$

$F20,total = 408.45 + 205.85 = 614.30$ kN

$F19{:}1,total = Fi$

Table 4.34: External Force Acting in Each Floor Disturbed by Using ESLM (Case 1.2).

Story	h_i	H_i	w_i	$w_i \times H_i$	$\frac{w_i \times H_i}{\sum w_j \times H_j}$	ΔF_n	F_i	$F_{i,total}$
20	3	61	4718.51	287828.88	0.089	408.45	205.852	614.30
19	3	58	4992.11	289542.16	0.089	-	207.077	207.08
18	3	55	4992.11	274565.84	0.085	-	196.366	196.37
17	3	52	4992.11	259589.53	0.080	-	185.656	185.66
16	3	49	4992.11	244613.21	0.076	-	174.945	174.94
15	3	46	4992.11	229636.89	0.071	-	164.234	164.23
14	3	43	4992.11	214660.57	0.066	-	153.523	153.52
13	3	40	4992.11	199684.25	0.062	-	142.812	142.81
12	3	37	4992.11	184707.93	0.057	-	132.101	132.10
11	3	34	4992.11	169731.61	0.052	-	121.390	121.39
10	3	31	5017.31	155536.49	0.048	-	111.238	111.24
9	3	28	5017.31	140484.58	0.043	-	100.473	100.47
8	3	25	5017.31	125432.66	0.039	-	89.708	89.71
7	3	22	5017.31	110380.74	0.034	-	78.943	78.94
6	3	19	5017.31	95328.82	0.029	-	68.178	68.18
5	3	16	5041.31	80660.90	0.025	-	57.688	57.69
4	3	13	5041.31	65536.98	0.020	-	46.871	46.87
3	3	10	5041.31	50413.06	0.016	-	36.055	36.05
2	3	7	5041.31	35289.14	0.011	-	25.238	25.24
1	4	4	5663.56	22654.23	0.007	-	16.202	16.20
Summation				3236278.46	1.000	408.450	2314.55	2723.00

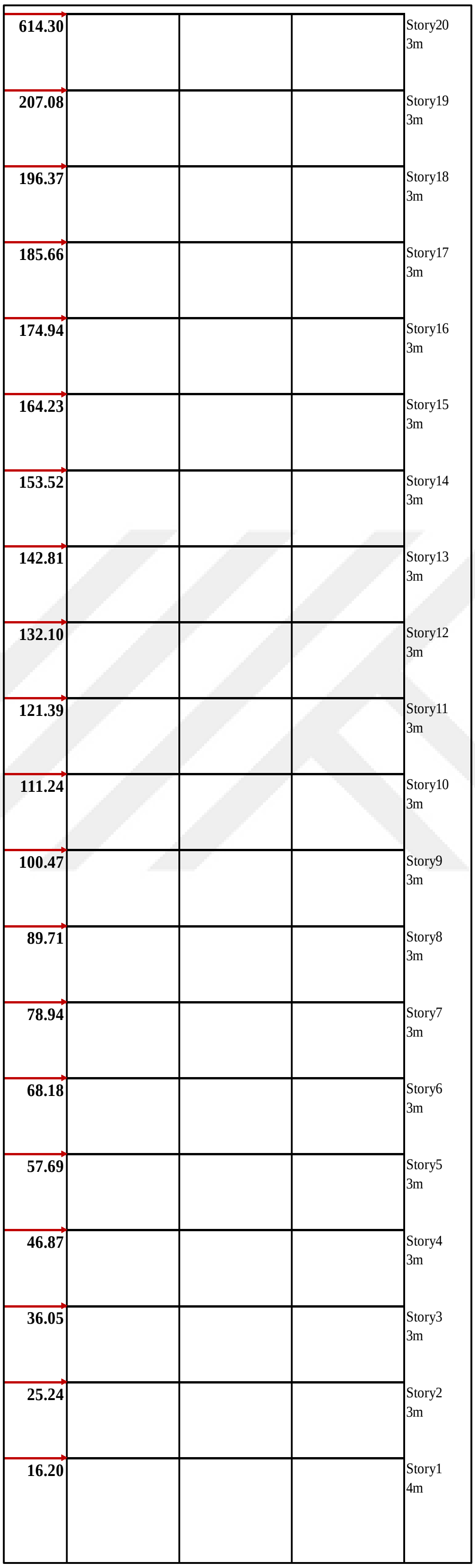


Figure 4.12: The Seismic Force Acting on Each Floor (F_i) in (Case 2.1).

4.5.2 Calculation of Shear Force (Ti) and Cantilever Moments (Mo)

Table 4.35: Shear Force (T_i) and Cantilever Moments (M_o) Calculation in (Case 1.2).

Story	h_i (m)	P_i (kN)	T_i (kN)	M_o (kNm)
20	3	614.30	614.30	1842.91
19	3	207.08	821.38	4307.04
18	3	196.37	1017.75	7360.28
17	3	185.66	1203.40	10970.48
16	3	174.94	1378.35	15105.52
15	3	164.23	1542.58	19733.26
14	3	153.52	1696.10	24821.57
13	3	142.81	1838.91	30338.31
12	3	132.10	1971.02	36251.36
11	3	121.39	2092.41	42528.57
10	3	111.24	2203.64	49139.50
9	3	100.47	2304.12	56051.85
8	3	89.71	2393.82	63233.33
7	3	78.94	2472.77	70651.63
6	3	68.18	2540.95	78274.47
5	3	57.69	2598.63	86070.37
4	3	46.87	2645.50	94006.88
3	3	36.05	2681.56	102051.56
2	3	25.24	2706.80	110171.95
1	4	16.20	2723.00	121063.95

4.5.3 Calculations of Coefficients and Constants of Equation Set

Table 4.36: The Fi, fi, δi,i-1 , δi,i and δi,o Values in (Case 1.2).

Floor	h_i (m)	F_i (1/m³)	f_i (1/m³)	M_o (kNm)	$\delta_{i,i-1}$ (1/m³)	$\delta_{i,i}$ (1/m³)	$\delta_{i,o}$ (kNm)
20	3	2.161	0.0262	1842.906	-	5.077	305.553
19	3	2.812	0.0262	4307.044	-2.7855	5.728	691.533
18	3	2.812	0.0262	7360.281	-2.7855	5.728	1169.989
17	3	2.812	0.0262	10970.484	-2.7855	5.728	1735.879
16	3	2.812	0.0262	15105.522	-2.7855	5.728	2384.156
15	3	2.812	0.0262	19733.261	-2.7855	5.728	3109.778
14	3	2.812	0.0262	24821.568	-2.7855	5.728	3907.701
13	3	2.812	0.0262	30338.311	-2.7855	5.728	4772.878
12	3	2.812	0.0262	36251.357	-2.7855	5.721	5700.268

Table 4.36: The Fi, fi, δi,i-1 , δi,i and δi,o Values in (Case 1.2).

"Table continued"

Floor	h_i (m)	F_i (1/m³)	f_i (1/m³)	M_o (kNm)	$\delta_{i,i-1}$ (1/m³)	$\delta_{i,i}$ (1/m³)	$\delta_{i,o}$ (kNm)
9	3	2.664	0.0262	56051.853	-2.6381	5.433	8806.064
8	3	2.664	0.0262	63233.326	-2.6381	5.433	9932.568
7	3	2.664	0.0262	70651.629	-2.6381	5.433	11096.248
6	3	2.664	0.0262	78274.465	-2.6381	5.403	12292.058
5	3	2.634	0.0262	86070.366	-2.6075	5.372	13515.010
4	3	2.634	0.0262	94006.880	-2.6075	5.372	14760.036
3	3	2.634	0.0262	102051.558	-2.6075	5.368	16022.041
2	3	2.630	0.0262	110171.952	-2.6038	4.306	20344.771
1	4	1.554	0.0349	121063.952	-1.5195	1.624	12289.790

4.5.4 Calculations of Matrix Coefficients and Load Vector.

a. $[\delta] \times \{X_i\} = -\{\delta_{i,o}\}$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		Xi		δi,o
5.077	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X20		-305.55
-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X19		-691.53
0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X18		-1169.99
0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X17		-1735.88
0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X16		-2384.16
0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X15		-3109.78
0.000	0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X14		-3907.70
0.000	0.000	0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X13		-4772.88
0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.785	5.721	-2.779	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X12		-5700.27
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.779	5.574	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X11		-6684.87
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	×	X10	=	-7721.81
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X9		-8806.06
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000		X8		-9932.57
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000		X7		-11096.25
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.403	-2.608	0.000	0.000	0.000	0.000		X6		-12292.06
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.608	5.372	-2.608	0.000	0.000	0.000		X5		-13515.01
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.608	5.372	-2.608	0.000	0.000		X4		-14760.04
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.608	5.368	-2.604	0.000		X3		-16022.04
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.604	4.306	-1.519		X2		-20344.77
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.519	1.624		X1		-12289.79

b. $[\delta]^{-1} \times -\{\delta_{i,o}\} = \{X_i\}$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		δi,o		Xi
0.347	0.274	0.216	0.171	0.135	0.107	0.084	0.067	0.053	0.042	0.033	0.026	0.021	0.017	0.013	0.011	0.009	0.008	0.007	0.007		-305.55		-4398.83
0.274	0.500	0.394	0.311	0.246	0.194	0.154	0.122	0.096	0.077	0.060	0.048	0.038	0.030	0.024	0.020	0.017	0.014	0.013	0.012		-691.53		-7908.32
0.216	0.394	0.595	0.470	0.371	0.293	0.232	0.183	0.145	0.115	0.091	0.072	0.057	0.045	0.037	0.030	0.025	0.022	0.019	0.018		-1169.99		-11615.23
0.171	0.311	0.470	0.654	0.517	0.408	0.323	0.255	0.202	0.161	0.127	0.100	0.079	0.063	0.051	0.042	0.035	0.030	0.027	0.025		-1735.88		-15556.70
0.135	0.246	0.371	0.517	0.691	0.546	0.432	0.342	0.271	0.215	0.170	0.134	0.106	0.085	0.068	0.056	0.047	0.040	0.036	0.034		-2384.16		-19751.71
0.107	0.194	0.293	0.408	0.546	0.715	0.565	0.447	0.355	0.282	0.222	0.175	0.139	0.111	0.089	0.073	0.061	0.053	0.048	0.044		-3109.78		-24203.93
0.084	0.154	0.232	0.323	0.432	0.565	0.731	0.578	0.459	0.364	0.287	0.226	0.179	0.143	0.115	0.094	0.079	0.068	0.061	0.057		-3907.70		-28903.78
0.067	0.122	0.183	0.255	0.342	0.447	0.578	0.742	0.588	0.467	0.368	0.290	0.230	0.184	0.148	0.121	0.101	0.087	0.079	0.074		-4772.88		-33829.67
0.053	0.096	0.145	0.202	0.271	0.355	0.459	0.588	0.751	0.597	0.470	0.371	0.294	0.235	0.189	0.154	0.129	0.112	0.101	0.094		-5700.27		-38948.59
0.042	0.077	0.115	0.161	0.215	0.282	0.364	0.467	0.597	0.760	0.599	0.472	0.374	0.299	0.241	0.197	0.164	0.142	0.128	0.120	×	-6684.87	=	-44228.87
0.033	0.060	0.091	0.127	0.170	0.222	0.287	0.368	0.470	0.599	0.770	0.607	0.481	0.384	0.310	0.253	0.211	0.183	0.165	0.154		-7721.81		-49888.52
0.026	0.048	0.072	0.100	0.134	0.175	0.226	0.290	0.371	0.472	0.607	0.779	0.617	0.492	0.397	0.324	0.271	0.234	0.211	0.198		-8806.06		-55589.76
0.021	0.038	0.057	0.079	0.106	0.139	0.179	0.230	0.294	0.374	0.481	0.617	0.790	0.630	0.508	0.415	0.347	0.300	0.270	0.253		-9932.57		-61260.83
0.017	0.030	0.045	0.063	0.085	0.111	0.143	0.184	0.235	0.299	0.384	0.492	0.630	0.805	0.649	0.530	0.443	0.383	0.346	0.323		-11096.25		-66812.18
0.013	0.024	0.037	0.051	0.068	0.089	0.115	0.148	0.189	0.241	0.310	0.397	0.508	0.649	0.829	0.677	0.566	0.489	0.441	0.413		-12292.06		-72133.03
0.011	0.020	0.030	0.042	0.056	0.073	0.094	0.121	0.154	0.197	0.253	0.324	0.415	0.530	0.677	0.866	0.724	0.626	0.565	0.528		-13515.01		-77144.82
0.009	0.017	0.025	0.035	0.047	0.061	0.079	0.101	0.129	0.164	0.211	0.271	0.347	0.443	0.566	0.724	0.926	0.800	0.722	0.675		-14760.04		-81617.85
0.008	0.014	0.022	0.030	0.040	0.053	0.068	0.087	0.112	0.142	0.183	0.234	0.300	0.383	0.489	0.626	0.800	1.022	0.923	0.863		-16022.04		-85343.93
0.007	0.013	0.019	0.027	0.036	0.048	0.061	0.079	0.101	0.128	0.165	0.211	0.270	0.346	0.441	0.565	0.722	0.923	1.179	1.103		-20344.77		-88067.24
0.007	0.012	0.018	0.025	0.034	0.044	0.057	0.074	0.094	0.120	0.154	0.198	0.253	0.323	0.413	0.528	0.675	0.863	1.103	1.648		-12289.79		-89959.48

The X_i values are calculated for all story and can be found in table 4.37.

Table 4.37: The Xi Values Calculated for (Case 1.2).

Floor	X_i (kNm)
20	-4398.834
19	-7908.321
18	-11615.230
17	-15556.702
16	-19751.709
15	-24203.932
14	-28903.782
13	-33829.668
12	-38948.593
11	-44228.870
10	-49888.524
9	-55589.760
8	-61260.831
7	-66812.178
6	-72133.030
5	-77144.817
4	-81617.847
3	-85343.934
2	-88067.241
1	-89959.480

4.5.5 Calculations of Moment Superposition and Distribution it.

Table 4.38: The Total Moment in Shear Wall and Moment in Single Shear Wall Values in (Case1.2).

Floor	M_o (kNm)	X_i (kNm)	M total Shear wall (kNm)	M single Shear wall		M Core shear wall (kNm)
				(Right and Left) (kNm)	(Center) (kNm)	
9	56051.853	-55589.760	462.092	64.265	0.346	102.170
8	63233.326	-61260.831	1972.495	274.324	1.475	436.125
7	70651.629	-66812.178	3839.450	533.970	2.871	848.914
6	78274.465	-72133.030	6141.435	854.117	4.593	1357.890
5	86070.366	-77144.817	8925.549	1241.316	6.675	1973.467
4	94006.880	-81617.847	12389.033	1722.999	9.265	2739.254
3	102051.558	-85343.934	16707.624	2323.605	12.495	3694.108
2	110171.952	-88067.241	22104.711	3074.202	16.531	4887.420
1	121063.952	-89959.480	31104.472	4325.840	23.261	6877.296

4.5.6 Calculations of Shear Forces

Table 4.39: Shear Force Calculation in (Case 1.2).

Floor	∑T	Tcol				$T_{\text{Normal shear wall}}$		T Core
		C1	C2	C3	C4	Right and Left	Center	
20	1466.278	20.509	15.700	31.768	31.010	729.864	75.212	265.253
19	1169.829	21.289	16.297	32.977	32.190	505.091	78.074	175.648
18	1235.636	22.487	17.214	34.832	34.001	533.505	82.466	185.529
17	1313.824	23.910	18.303	37.036	36.152	567.264	87.684	197.269
16	1398.336	25.448	19.481	39.419	38.478	603.753	93.325	209.958
15	1484.074	27.008	20.675	41.836	40.837	640.772	99.047	222.832
14	1566.617	28.510	21.825	44.163	43.108	676.410	104.556	235.226
13	1641.962	29.881	22.875	46.287	45.181	708.942	109.584	246.539
12	1706.308	31.052	23.771	48.100	46.952	736.724	113.879	256.200
11	1760.092	31.954	24.575	50.105	48.650	758.126	117.187	263.643
10	1886.552	37.440	37.440	60.793	58.376	771.850	119.308	268.415
9	1900.412	37.715	37.715	61.239	58.805	777.521	120.185	270.387
8	1890.357	37.515	37.515	60.915	58.494	773.407	119.549	268.957
7	1850.449	36.723	36.723	59.629	57.259	757.080	117.025	263.279
6	1773.617	35.199	35.199	57.154	54.882	725.645	112.166	252.347
5	1670.595	33.554	32.773	55.305	53.024	675.650	104.438	234.961
4	1491.010	29.947	29.250	49.360	47.324	603.019	93.211	209.703
3	1242.029	24.946	24.366	41.118	39.422	502.322	77.646	174.685
2	907.769	18.207	17.784	30.010	28.772	366.624	56.671	128.760
1	473.060	10.290	10.290	12.656	12.415	225.834	17.465	71.462

4.5.7 Calculations of Coupled Beam End Moments

Table 4.40: The Coupled Beam End Moments Values in (Case 1.2).

Floor	h_i (m)	Shear wall Right and Left		Core	
		$M_{i,u}$	$M_{i,o}$	$M_{i,u}$	$M_{i,o}$
20	3	729.864	1459.729	265.253	530.507
19	3	757.637	1487.501	263.473	528.726
18	3	800.257	1557.894	278.294	541.767
17	3	850.895	1651.152	295.904	574.198
16	3	905.629	1756.524	314.938	610.842
15	3	961.157	1866.786	334.248	649.186
14	3	1014.616	1975.773	352.839	687.087
13	3	1063.413	2078.029	369.808	722.647
12	3	1105.087	2168.500	384.300	754.108

4.5.8 Calculation for Corrected Shear Wall and Core Moments at Bottom and Top.

Table 4.41: The Corrected Shear Wall Moments at Bottom and Top Values in (Case 1.2).

Floor	Shear wall (Right and left) (kNm)			
	M single	$M_{i,u}$	M_1 Bottom	M_2 Upper
20	-355.46	729.86	374.40	-1459.73
19	-500.85	757.64	256.79	-1113.10
18	-591.75	800.26	208.50	-1301.10
17	-637.83	850.90	213.07	-1442.65
16	-646.17	905.63	259.46	-1543.46
15	-621.76	961.16	339.40	-1607.32
14	-567.73	1014.62	446.88	-1636.37
13	-485.56	1063.41	577.85	-1631.14
12	-375.12	1105.09	729.97	-1590.65
11	-236.47	1137.19	900.72	-1512.31
10	-104.17	1157.78	1053.61	-1394.24
9	64.27	1166.28	1230.55	-1270.45
8	274.32	1160.11	1434.43	-1095.85
7	533.97	1135.62	1669.59	-861.30
6	854.12	1088.47	1942.58	-554.50
5	1241.32	1013.48	2254.79	-159.36
4	1723.00	904.53	2627.53	336.79
3	2323.60	753.48	3077.09	969.52
2	3074.20	549.94	3624.14	1773.67
1	4325.84	451.67	4777.51	2622.53

Table 4.42: The Corrected Core Wall Moments at Bottom and Top Values in (Case 1.2).

Floor	Core Wall (kNm)			
	M single	$M_{i,u}$	M_1 Bottom	M_2 Upper
20	-565.12	265.25	-299.87	-530.51
19	-796.25	263.47	-532.78	-828.60
18	-940.78	278.29	-662.49	-1074.55
17	-1014.03	295.90	-718.12	-1236.69
16	-1027.29	314.94	-712.35	-1328.96
15	-988.48	334.25	-654.23	-1361.53
14	-902.59	352.84	-549.75	-1341.32
13	-771.95	369.81	-402.14	-1272.40
12	-596.37	384.30	-212.07	-1156.25
11	-375.94	395.46	19.52	-991.83
10	-165.61	402.62	237.01	-778.56
9	102.17	405.58	507.75	-571.19
8	436.12	403.44	839.56	-301.27
7	848.91	394.92	1243.83	41.21
6	1357.89	378.52	1736.41	470.39
5	1973.47	352.44	2325.91	1005.45
4	2739.25	314.56	3053.81	1658.91
3	3694.11	262.03	3956.14	2477.23
Floor	Core Wall (kNm)			
	M single	$M_{i,u}$	M_1 Bottom	M_2 Upper
2	4887.42	193.14	5080.56	3500.97
1	6877.30	142.92	7020.22	4744.50

4.5.9 Calculation of Displacement, Drift of Story

Table 4.43: The Displacement and Drift Values in (Case 1.2).

Floor	h_i (m)	$\sum T$ (kN)	$\sum D_{ij}$ kN / m	δ_i (m)	d_i (m)
9	3	1900.41	1376244.96	0.001381	0.009558
8	3	1890.36	1376244.96	0.001374	0.008178
7	3	1850.45	1376244.96	0.001345	0.006804
6	3	1773.62	1376244.96	0.001289	0.005459
5	3	1670.60	1392225.50	0.001200	0.004171
4	3	1491.01	1392225.50	0.001071	0.002971
3	3	1242.03	1392225.50	0.000892	0.001900
2	3	907.77	1394166.79	0.000651	0.001008
1	4	473.06	1326905.29	0.000357	0.000357

Check the result Case (1.2):

The Muto method is used to calculate the sway of a framed building (di) based on lateral wind forces:

$\Delta max = 0.000357 + 0.000651 + \ldots\ldots\ldots + 0.000897 + 0.000864 = 0.02197$ m

$= 0.02197 \text{ m} \times 103 = 22.00$ mm

The recommended maximum sway is Δ max-all = 0.002 × H Building

Δ max-all = 0.002 × H = 0.002 × (61 m) × 103 = 122 mm > 22.0 mm. (O.K.)

The drift ratio = 0.001381 × 7 / 3 = 0.00322 < 0.02 (O.K.)

4.5.10 Acceleration Values of Equivalent Earthquake Force to Wind Force

The method "Equivalent Seismic Load Method" was used to distribute the base shear force that was calculated based on (Eurocode) to study the effect of earthquakes on the building with a force equivalent to the wind force affecting the building. Below is a comparison between the two cases:

Table 4.44: Comparison Between the Two Cases in (Y-dir.).

Item	Unit	Value	
		Eurocode	ESLM
$M_{o,20}$	kNm	96150.41	121063.95
X_{20}	kNm	-67314.60	-89959.48
$Mp_{,20}$	kNm	28835.81	31104.47
$M1_{SW\text{-}R\&L,1}$	kNm	4424.70	4777.51
$M1_{CW,1}$	kNm	6506.81	7020.22
$\sum T_{7,9}$	kN	1535.31	1900.41
Ratio $\delta_{7,9}$	-	0.000372	0.00322
d_{20}	mm	16.41	21.97

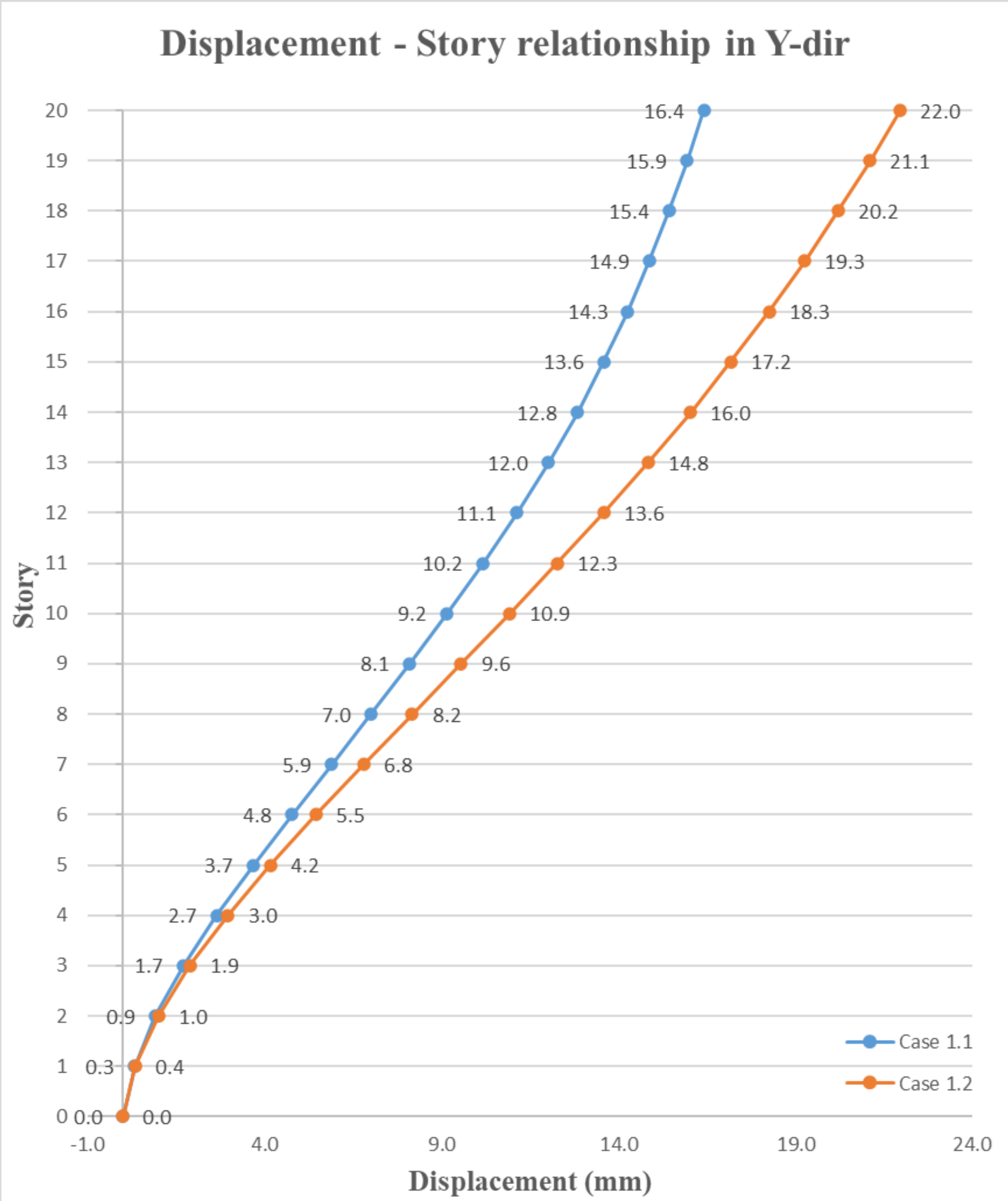


Figure 4.13: Displacement - Story Relationship in (Y-Dir.).

First, need to calculate of the Fundamental Period of the Building

by Rayleigh Method in Y-direction:

Examples for calculations:

M1 = w1 / 9.81 = 5663.56 / 9.81 = 577.3 (kN/m/sec2)

M1d12 = M1 × d12 = 577.56 × 0.0003612 = 0.000075 kNm Sec^2

P1d1 = P1 × d1 = 16.20 × 0.000361 = 0.00585 kNm

The Mass, Midi2 and Pidi values in each story are calculated for all stories and can be found in table 4.45 for (Case1.2).

Table 4.45: The Mass, $M_i d_i^2$ and Pidi Values for Case (1.2).

Floor	w_i (kN)	M_i (kN/m/sec2)	$M_i d_i^2$ kNm.Sec²	$P_i d_i$ (KNm)
20	4718.51	481.0	0.232181	13.49669
19	4992.11	508.9	0.226701	4.37070
18	4992.11	508.9	0.207840	3.96848
17	4992.11	508.9	0.188809	3.57611
16	4992.11	508.9	0.169575	3.19355
15	4992.11	508.9	0.150239	2.82193
14	4992.11	508.9	0.130997	2.46318
13	4992.11	508.9	0.112115	2.11977
12	4992.11	508.9	0.093901	1.79446
11	4992.11	508.9	0.076683	1.49014
10	5017.31	511.4	0.061091	1.21574
9	5017.31	511.4	0.046727	0.96036
8	5017.31	511.4	0.034201	0.73359
7	5017.31	511.4	0.023677	0.53712
6	5017.31	511.4	0.015244	0.37221
5	5041.31	513.9	0.008939	0.24060
4	5041.31	513.9	0.004535	0.13924
3	5041.31	513.9	0.001855	0.06850
2	5041.31	513.9	0.000522	0.02543
1	5663.56	577.3	0.000073	0.00578
			1.79	43.59

$\omega 1^2$ (Y) = ∑Pidi / ∑Midi² = 43.59 / 1.79 = 24.41

ω1 (Y) = 4.94 rad/sec.

T1(Y) = (2 × 3.14) / 4.94 = 1.27 Sec.

The acceleration values of equivalent earthquake force to wind force:

Seismic characteristic:

a. Assume local site class = z2

b. Spectrum coefficient = (ta = 0.15 sec), (tb = 0.40 sec).

c. Effective ground acceleration coefficient (ao) = 0.30

d. Structural behavior factors (r) = 7.

e. Building importance factor (i) for hotel building = 1

f. Damping ratio (ξ) = 5%

By using ERZICAN 1992 N-S chart, intersect the T-period value (1.27) with Damping ratio (ξ=5%), the (Sa/amax) value = 2.02

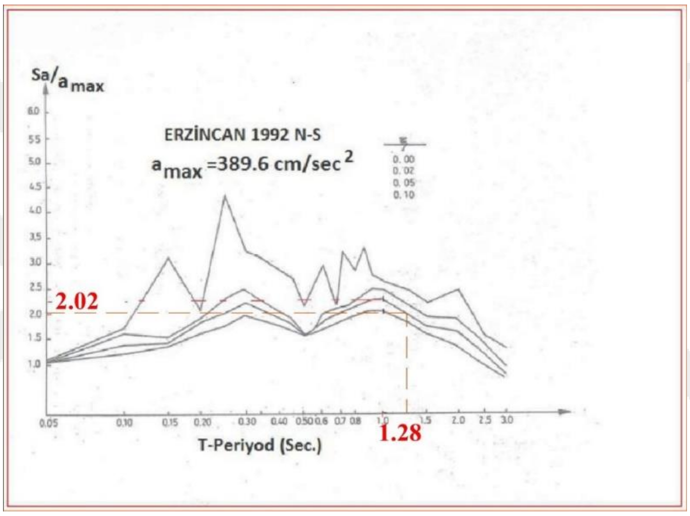


Figure 4.14: Erzican 1992 N-S Chart in Case (1.2).

Spectrum coefficient $S_{(T)} = 2.5 \times (0.4/1.27)^{0.8} = 0.991$

$A_{(T1(Y))} = A_o \times S_{(T)} \times I \times 9.81$

$V_t = \sum m_i \times A_{(T1(Y))} / R$

$V_{t2} = Vt \times (S_a/a_{max})/S_{(T)}$

$2723 = Vt \times 2.04$

$V_t = 2723 / 2.04 = 1332.72$ kN

$1332.72 = 10251.05 \times A_{(T1(Y))} / 7$

$A_{(T1(Y))} = 1332.72 \times 7 / 10251.05 = 0.91$

$0.91 = A_o \times 0.989 \times 1 \times 9.81$

$A_o = 0.989 / (0.91 \times 9.81) = 0.09$

Therefore, the acceleration values of equivalent earthquake force to wind force is (0.09), But we assumed the seismic zone is second so effective ground acceleration coefficient (Ao) = 0.3, we need to obtain the real base shear force acting on structure due to the earthquake force:

$A(T1(Y)) = Ao \times S(T) \times I \times 9.81$

$= 0.3 \times 0.989 \times 1 \times 9.81 = 2.91$

$Vt = \sum mi \times A(T1(Y)) / R$

$= 10251.05 \times 2.91 / 7$

= 4260.92 kN (Total equivalent seismic load)

$Vt2 = Vt \times (Sa/amax)/S(T)$

$= 4260.92 \times 2.04$

= 8705.86 kN (In case of using Erzincan 1992 N-S chart)

Vt2 / Vt1 ratio = 8705.86 / 2723 = 3.2

Displacement 20th = 22.0 × 3.2 = 70.24 mm < 122 mm (O.K.)

The difference between the wind force and the earthquake force (Y-dir.):

Percentage (Equivalent earthquake/wind) = d20 case 1.2 / d20 case 1.1

= 22.09/16.51 = 134%

Percentage (earthquake/wind) = (d20 case 1.2 × Vt2 / Vt1 ratio) / d20 case 1.1

= 70.62/16.51 = 428%

the force of the earthquake (in both cases) is stronger than the wind force and its effect on the displacement by (134%, 428%) respectively, therefore the effect of the earthquake is stronger and more dangerous on the building.

4.6 CALCULATION OF WIND FORCE (WIND PROFILE DISTRIBUTED) AND IMPACT ON STRUCTURE IN X-DIRECTION (CASE 2.1)

Calculate the wind force acting in building surface according to (EN 1991-1-4 (2005) (English): Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions).

4.6.1 Calculation of Wind Forces on the Structure

4.6.1.1 Terrain categories and parameters.

Calculation shown in item (This:4.5.1.1).

4.6.1.2 Calculation of the mean wind–variation with height (vm(z)).

Calculation shown in item (This:4.5.1.2).

4.6.1.3 Calculation of the wind turbulence (iv(z)).

Calculation shown in item (This:4.5.1.3).

4.6.1.4 Calculation of the peak velocity pressure.

Calculation shown in item (This:4.5.1.4).

4.6.1.5 Calculation of wind forces on surface.

$F_w = c_s c_d \times c_f \times A_{ref} \times q_{p(ze)}$

First, obtain the force coefficient (cf) and structural factor (cscd).

a. Force coefficient (cf).

$cf = cf,0 \times \psi r \times \psi\lambda$

$cf = 1.324 \times 1 \times 0.685$ $cf = 0.91$. (Data calculation below)

Data calculation of force coefficient:

w. Effective slenderness (λ).

For elements with rectangular cross-section and length $l \geq 50$ m the effective slenderness λ is equal to:

$\lambda = \lambda 50 = \min (1.4 \times l / b, 70) \quad = \min (1.4 \times 61\ m / 12, 70) = 7.117$

x. End effect factor ($\psi\lambda$).

The value of the end effect factor $\psi\lambda$ is determined from EN1991-1-4 Figure 7.36 as a function of the slenderness λ.

solidity ratio $\varphi = 1.000$ and $\psi\lambda = 0.685$.

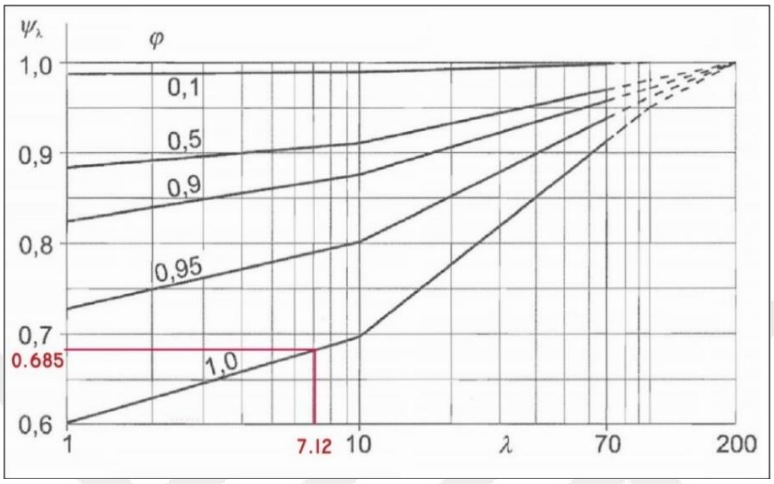


Figure 4.15: End Effect Factor in (X-dir.).

y. Force coefficient without free-end flow (cf,0).

The value of cf,0 is determined according to EN1991-1.4 figure 7.23 for the values of b= 12 m, d= 38 m, d/b= 3.1667.

cf,0 = 1.324.

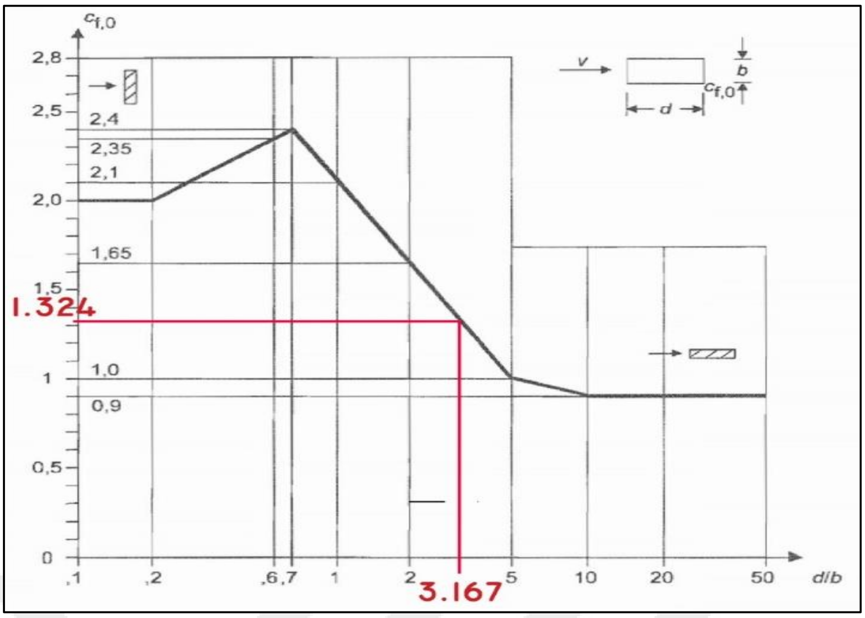


Figure 4.16: Force Coefficient Without Free-End Flow in (X-Dir.).

z. Reduction factor for rounded corners (ψr).

The value of ψr is calculated in accordance with EN1991-1-4 -7.6(1).

For cross-sections with sharp corners (r = 0.0 m) there is no reduction, $\psi r = 1$.

a. Structural factor (cscd).

The size factor (Cs): (Data calculation below)

$Cs = 1 + 7 \times 0.278 \times (0.569)0.5 / 1 + 7 \times 0.278 = 0.838$

The dynamic factor (Cd): (Data calculation below)

$Cd = 1 + 2 \times 3.455 \times 0.278 \times (0.158 + 0.569)0.5 / 1 + 7 \times 0.278 \times (0.569)0.5 = 1.069$

$Cs \times Cd = 0.838 \times 1.069 = 0.90$

Data calculation of Structural factor (cscd):

By using the procedure mention in item (3.3.1.5.II)

$Z(s) = 0.6 \times 61 = 36.6 \geq 10$

$Cr(z) = 0.234 \times Ln\ (36.6/1) = 0.844$

$Vm(z) = 0.844 \times 30 = 25.31$ m/sec

The fundamental flexural frequency (n1) = 46/h = 46/62 = 0.75 Hz

aa. The turbulent length scale (L(z)):

$\alpha = 0.67 + 0.05 \times Ln\ (1) = 0.67$

$L(zs) = 300 \times (36.6/200)0.67 = 96.153$

bb. The non-dimensional power spectral density function (SL(zs,n)).

$fL(zs,n) = 0.75 \times 36.6 / 25.31 = 2.87$

$SL(zs,n) = 6.8 \times 2.87 / (1+10.2 \times 2.87)(5/3) = 0.066$

cc. The Background factor (B2):

$B2 = 1 / 1 + 0.9 \times (12 + 61 / 36.6)0.63 = 0.569$

dd. The resonance response factor (R2):

$R2 = (3.142/ 2 \times 0.1005) \times (0.066) \times (0.1125) \times (0.430) = 0.158$

f.Aerodynamic admittance functions (for 1st mode shape):

$\eta h = (4.6 \times 61 / 96.15) \times 2.87 = 8.36$

$\eta b = (4.6 \times 12 / 96.15) \times 2.87 = 1.65$

$Rh = (1/8.36) - (1 / 2 \times 8.362) \times (1 – e-2 \times 8.36) = 0.113$

$Rb = (1/1.65) - (1 / 2 \times 1.652) \times (1 – e-2 \times 1. 65) = 0.430$

g. Logarithmic decrement of damping (δ):

$\delta = \delta s + \delta a + \delta d = 0.1005$

δs = 0.1 for reinforced concrete buildings (Table F.2 EN 1991-1-4)

$\delta d = 0.$

$\delta a = 0.91 \times 0.00125 \times 12 \times 25.31 / 2 \times 0.75 \times 504.90 = 0.0005$

ee. The up-crossing frequency (v):

$v = 0.75 \times (0.158 / 0.158 + 0.569)0.5 = 0.351$ Hz > 0.08 Hz

T = 600 sec.

vT = 210.72

ff. The Peak factor (Kp):

$Kp = (2 \times Ln (210.72))0.5 + 0.6 / (2 \times Ln (210.72))0.5 = 3.46 > 3$

gg. The turbulence intensity Iv(z) at height Z(s).

Z(s) = 36.6m

$\sigma v = 7.03$

Vm(zs) = 25.31 m/sec

$Iv(zs) = 7.03 / 25.31 = 0.278$

Examples for calculations of the wind force (Fw) acting on a structure:

$Fw = cscd \times cf \times Aref \times qp(ze)$

$Fw(4) = 0.90 \times 0.91 \times ((12 \times 2 \times 0.662) + (12 \times 1.5 \times 0.662)) = 22.56$ kN.

$Fw(13) = 0.90 \times 0.91 \times ((12 \times 1.5 \times 0.758) + (12 \times 1.5 \times 0.837)) = 23.31$ kN.

$Fw(34) = 0.90 \times 0.91 \times ((12 \times 1.5 \times 1.218) + (12 \times 1.5 \times 1.25)) = 34.06$ kN.

$Fw(61) = 0.90 \times 0.91 \times 12 \times 1.5 \times 1.411 = 20.62$ kN.

The wind pressure acting on the external surfaces (We) and the wind force (Fw) acting on a structure values in X-direction are calculated for all height and can be found in table 4-46, Figure 4-16 shown the wind pressure acting on the external surfaces (We) and Figure 4.17 shown the wind force acting on the external surfaces (Fw).

Table 4.46: The External Surfaces (We) and the Wind Force (Fw) Acting on a Structure in (X-dir.).

$Z_{(m)}$	Story	h_i	$q_{p(z)}$	h_{ref}	W_e	F_w
0 - 4	1	4	0.662	2	0.54	22.56
4 - 7	2	3	0.662	1.5	0.54	19.34
7 - 10	3	3	0.662	1.5	0.54	20.75
10 - 13	4	3	0.758	1.5	0.62	23.31
13 - 16	5	3	0.837	1.5	0.68	25.45
16 - 19	6	3	0.904	1.5	0.73	27.30
19 - 22	7	3	0.963	1.5	0.78	28.93
22 - 25	8	3	1.016	1.5	0.83	30.39
25 - 28	9	3	1.063	1.5	0.86	31.72
28 - 31	10	3	1.107	1.5	0.90	32.93
31 - 34	11	3	1.147	1.5	0.93	34.06
34 - 37	12	3	1.183	1.5	0.96	35.10
37 - 40	13	3	1.218	1.5	0.99	36.07
40 - 43	14	3	1.250	1.5	1.02	36.99
43 - 46	15	3	1.281	1.5	1.04	37.85
46 - 49	16	3	1.309	1.5	1.06	38.67
49 - 52	17	3	1.337	1.5	1.09	39.45

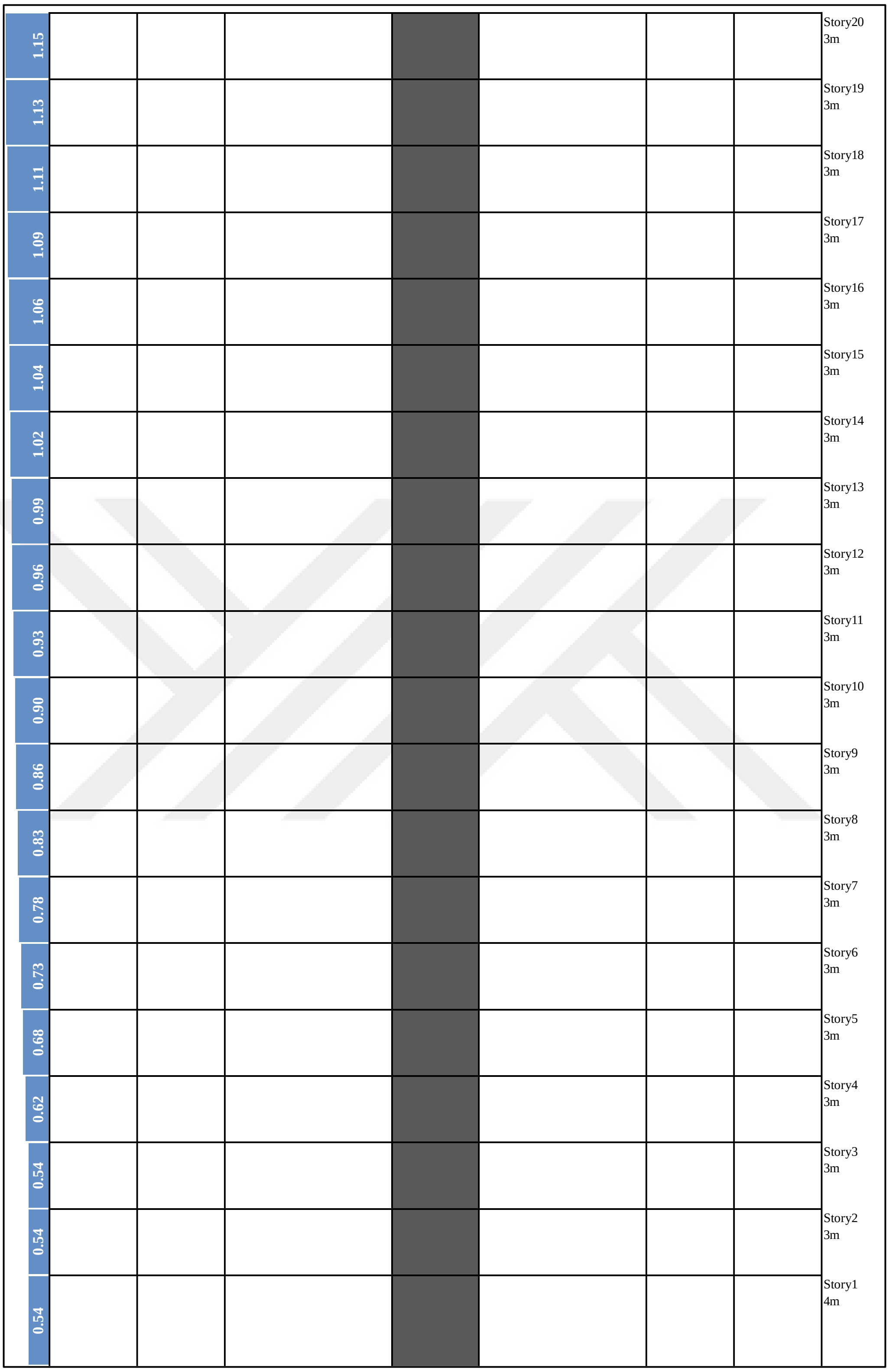


Figure 4.17: The Wind Pressure Acting on the External Surfaces (We) in (Case 2.1).

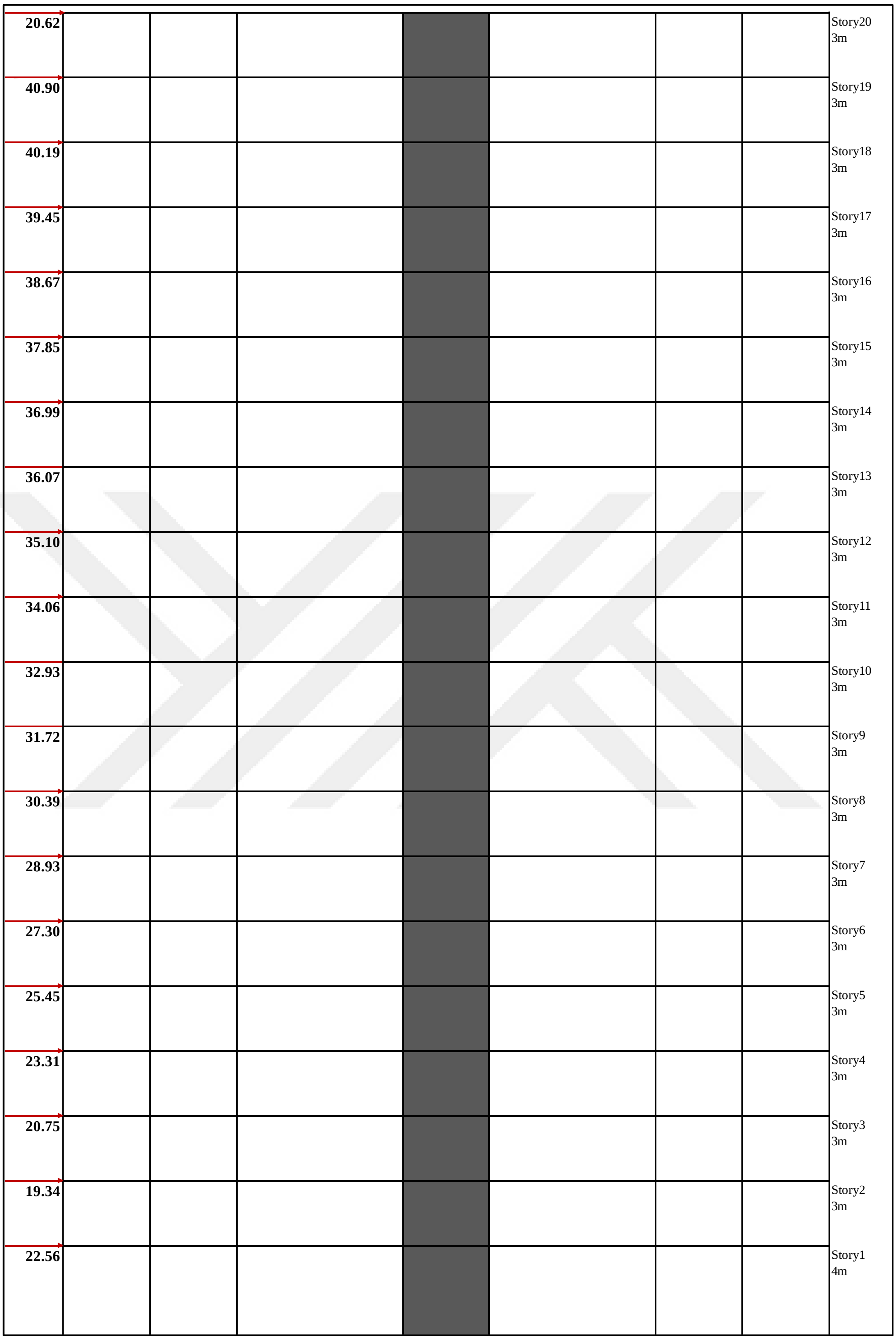


Figure 4.18: The Wind Force Acting on the External Surfaces (Fw) in (Case 2.1).

4.6.2 Calculation of Shear Force (Ti) and Cantilever Moments (Mo)

Examples for calculations:

Ti = ∑Pi

T20 = P20 = 20.62 kN.

T19 = T20 + P19 = 20.62 + 40.90 = 61.52 kN.

Mo,i = Ti × hi + Mo,i+1

Mo20 = T20 × h20 = 20.62 × 3 = 61.86 kNm

Mo19 = T19 × h19 + Mo20 = 61.52 × 3 + 61.86 = 246.42 kNm

Table 4.47: Shear Force (Ti) and Cantilever Moments (Mo) Calculation in (Case 2.1).

Story	h_i (m)	P_i (kN)	T_i (kN)	M_o (kNm)
19	3	20.62	20.62	61.86
18	3	40.90	61.52	246.42
17	3	40.19	101.71	551.54
16	3	39.45	141.16	975.02
15	3	38.67	179.83	1514.51
14	3	37.85	217.69	2167.57
13	3	36.99	254.68	2931.60
12	3	36.07	290.75	3803.85
11	3	35.10	325.85	4781.41
10	3	34.06	359.91	5861.13
9	3	32.93	392.84	7039.66
8	3	31.72	424.56	8313.35
7	3	30.39	454.96	9678.22
6	3	28.93	483.89	11129.89
5	3	27.30	511.19	12663.46
4	3	25.45	536.64	14273.39
3	3	23.31	559.95	15953.25
2	3	20.75	580.70	17695.35
1	3	19.34	600.04	19495.47

4.6.3 Calculations of Coefficients and Constants of Equation Set

Examples for calculations:

v. $F_i = 1/\sum D$

$F_{20} = (1 / 2699.402) \times 10^4 = 3.705\ 1/m^3$.

w. $\sum(EI_p)_i$ = Ip (shear wall $_{\text{right and left}}$) × Number + Ip (shear wall $_{\text{center}}$) × Number + Ip (core) × Number

$\sum(EI_p)_i = (0.0161 \times 4) + (1.867 \times 2) + (0.279 \times 2) = 4.36\ m^4$.

x. $F_i = h_i/(6\times\sum(EI_p)_i)$.

$F_{20} = 3 / (6\times 4.36) = 0.1148\ 1/m^3$.

y. M_o from table above.

$M_{o,20} = 61.86$ KNm

z. $\delta_{i,i-1} = f_{i-1} - F_{i-1}$.

$\delta_{20,19} = f_{19} - F_{19} = 0.1148 - 4.474 = -4.3596\ 1/m^3$

aa. $\delta_{i,i} = 2\times f_i + 2\times f_{i-1} + F_i + F_{i-1}$.

$\delta_{20,20} = 2\times f_{20} + 2\times f_{19} + F_{20} + F_{19} = 2\times 0.1148 + 2\times 0.1148 + 3.705 + 4.474$

$= 8.638\ 1/m^3$.

bb. $\delta_{i,o} = f_i\,(M_{i+1,o} + 2 \times M_{i,o}) - f_{i-1}\,(M_{i-1,o} + 2 \times M_{i,o})$.

$\delta_{20,o} = f_{20}\,(M_{21,o} + 2 \times M_{20,o}) - f_{19}\,(M_{19,o} + 2 \times M_{20,o})$

$= 0.1148 \times (0 + 2 \times 61.86) - 0.1148 \times (246.42 + 2 \times 61.86)$

$= 56.69$ KNm

The F_i, f_i, $\delta_{i,i-1}$, $\delta_{i,i}$ and $\delta_{i,o}$ values are calculated for all story and can be found in table 4-48.

Table 4.48: The Fi, fi, δi,i-1 , δi,i and δi,o Values in (Case 2.1).

Floor	hi (m)	Fi (1/m³)	fi (1/m³)	Mo (kNm)	$\delta i_{,i-1}$ (1/m³)	$\delta i_{,i}$ (1/m³)	$\delta i_{,o}$ (kNm)
20	3	3.705	0.1148	61.86	-	8.638	56.694
19	3	4.474	0.1148	246.42	-4.3596	9.408	183.566
18	3	4.474	0.1148	551.54	-4.3596	9.408	393.472
17	3	4.474	0.1148	975.02	-4.3596	9.408	684.881
16	3	4.474	0.1148	1514.51	-4.3596	9.408	1056.187
15	3	4.474	0.1148	2167.57	-4.3596	9.408	1505.697
14	3	4.474	0.1148	2931.60	-4.3596	9.408	2031.624
13	3	4.474	0.1148	3803.85	-4.3596	9.408	2632.073
12	3	4.474	0.1148	4781.41	-4.3596	9.419	3305.025
11	3	4.485	0.1148	5861.13	-4.3706	9.331	4048.322
10	3	4.386	0.1148	7039.66	-4.2715	9.244	4859.640
9	3	4.399	0.1148	8313.35	-4.2840	9.257	5736.464
8	3	4.399	0.1148	9678.22	-4.2840	9.257	6676.045
7	3	4.399	0.1148	11129.89	-4.2840	9.258	7675.351
6	3	4.400	0.1148	12663.46	-4.2855	8.828	8730.995
5	3	3.968	0.1148	14273.39	-3.8533	8.436	9839.131
4	3	4.009	0.1148	15953.25	-3.8943	8.477	10995.286
3	3	4.009	0.1148	17695.35	-3.8943	8.448	12194.709
2	3	3.979	0.1148	19495.47	-3.8646	6.811	15840.445
1	4	2.296	0.1531	21985.89	-2.1428	2.602	9714.315

4.6.4 Calculations of Matrix Coefficients & Load Vector

I. $[\delta] \times \{X_i\} = -\{\delta_{i,o}\}$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		Xi		δi,o
8.638	-4.360	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X20		-61.04
-4.360	9.408	-4.360	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X19		-197.64
0.000	-4.360	9.408	-4.360	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X18		-423.64
0.000	0.000	-4.360	9.408	-4.360	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X17		-737.39
0.000	0.000	0.000	-4.360	9.408	-4.360	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X16		-1137.16
0.000	0.000	0.000	0.000	-4.360	9.408	-4.360	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X15		-1621.13
0.000	0.000	0.000	0.000	0.000	-4.360	9.408	-4.360	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X14		-2187.38
0.000	0.000	0.000	0.000	0.000	0.000	-4.360	9.408	-4.360	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X13		-2833.86
0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.360	9.419	-4.371	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X12		-3558.40
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.371	9.331	-4.272	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	×	X11	=	-4358.68
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.272	9.244	-4.284	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X10		-5232.20
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.284	9.257	-4.284	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X9		-6176.24
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.284	9.257	-4.284	0.000	0.000	0.000	0.000	0.000	0.000		X8		-7187.86
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.284	9.258	-4.285	0.000	0.000	0.000	0.000	0.000		X7		-8263.77
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-4.285	8.828	-3.853	0.000	0.000	0.000	0.000		X6		-9400.35
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-3.853	8.436	-3.894	0.000	0.000	0.000		X5		-10593.44
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-3.894	8.477	-3.894	0.000	0.000		X4		-11838.23
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-3.894	8.448	-3.865	0.000		X3		-13129.60
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-3.865	6.811	-2.143		X2		-17054.84
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.143	2.602		X1		-10459.05

II. $[\delta]^{-1} \times -\{\delta_{i,o}\} = \{X_i\}$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		δi,o		Xi
0.175	0.118	0.080	0.054	0.036	0.024	0.016	0.011	0.007	0.005	0.003	0.002	0.002	0.001	0.001	0.000	0.000	0.000	0.000	0.000		-61.04		-378.28
0.118	0.234	0.158	0.106	0.072	0.048	0.033	0.022	0.015	0.010	0.007	0.005	0.003	0.002	0.001	0.001	0.001	0.000	0.000	0.000		-197.64		-735.53
0.080	0.158	0.261	0.176	0.118	0.080	0.054	0.036	0.024	0.017	0.011	0.007	0.005	0.003	0.002	0.002	0.001	0.001	0.001	0.000		-423.64		-1163.65
0.054	0.106	0.176	0.273	0.184	0.124	0.084	0.056	0.038	0.026	0.017	0.012	0.008	0.005	0.004	0.002	0.002	0.001	0.001	0.001		-737.39		-1678.44
0.036	0.072	0.118	0.184	0.278	0.188	0.126	0.085	0.057	0.039	0.026	0.018	0.012	0.008	0.005	0.004	0.002	0.002	0.001	0.001		-1137.16		-2289.27
0.024	0.048	0.080	0.124	0.188	0.281	0.189	0.128	0.086	0.058	0.039	0.026	0.018	0.012	0.008	0.005	0.004	0.003	0.002	0.002		-1621.13		-3000.93
0.016	0.033	0.054	0.084	0.126	0.189	0.282	0.190	0.128	0.087	0.058	0.039	0.026	0.018	0.012	0.008	0.006	0.004	0.003	0.002		-2187.38		-3814.87
0.011	0.022	0.036	0.056	0.085	0.128	0.190	0.283	0.191	0.129	0.087	0.058	0.039	0.027	0.018	0.012	0.008	0.006	0.004	0.004		-2833.86		-4729.77
0.007	0.015	0.024	0.038	0.057	0.086	0.128	0.191	0.283	0.192	0.129	0.087	0.058	0.040	0.027	0.018	0.012	0.009	0.007	0.005		-3558.40		-5741.91
0.005	0.010	0.017	0.026	0.039	0.058	0.087	0.129	0.192	0.284	0.191	0.128	0.087	0.059	0.040	0.027	0.018	0.013	0.010	0.008	×	-4358.68	=	-6842.19
0.003	0.007	0.011	0.017	0.026	0.039	0.058	0.087	0.129	0.191	0.285	0.192	0.129	0.088	0.060	0.040	0.027	0.019	0.015	0.012		-5232.20		-8050.87
0.002	0.005	0.007	0.012	0.018	0.026	0.039	0.058	0.087	0.128	0.192	0.286	0.193	0.131	0.089	0.060	0.041	0.029	0.022	0.018		-6176.24		-9329.11
0.002	0.003	0.005	0.008	0.012	0.018	0.026	0.039	0.058	0.087	0.129	0.193	0.287	0.195	0.133	0.089	0.061	0.043	0.033	0.027		-7187.86		-10665.56
0.001	0.002	0.003	0.005	0.008	0.012	0.018	0.027	0.040	0.059	0.088	0.131	0.195	0.290	0.199	0.132	0.090	0.064	0.049	0.040		-8263.77		-12038.95
0.001	0.001	0.002	0.004	0.005	0.008	0.012	0.018	0.027	0.040	0.060	0.089	0.133	0.199	0.296	0.197	0.134	0.095	0.073	0.060		-9400.35		-13418.48
0.000	0.001	0.002	0.002	0.004	0.005	0.008	0.012	0.018	0.027	0.040	0.060	0.089	0.132	0.197	0.304	0.207	0.147	0.113	0.093		-10593.44		-14911.70
0.000	0.001	0.001	0.002	0.002	0.004	0.006	0.008	0.012	0.018	0.027	0.041	0.061	0.090	0.134	0.207	0.316	0.224	0.172	0.142		-11838.23		-16306.34
0.000	0.000	0.001	0.001	0.002	0.003	0.004	0.006	0.009	0.013	0.019	0.029	0.043	0.064	0.095	0.147	0.224	0.341	0.262	0.215		-13129.60		-17545.14
0.000	0.000	0.001	0.001	0.001	0.002	0.003	0.004	0.007	0.010	0.015	0.022	0.033	0.049	0.073	0.113	0.172	0.262	0.398	0.328		-17054.84		-18523.03
0.000	0.000	0.000	0.001	0.001	0.002	0.002	0.004	0.005	0.008	0.012	0.018	0.027	0.040	0.060	0.093	0.142	0.215	0.328	0.655		-10459.05		-19273.86

The X_i values are calculated for all story and can be found in table 4-49.

Table 4.49: The Xi Values Calculated For (Case 2.1).

Floor	Xi (kNm)
20	-351.346
19	-683.155
18	-1080.790
17	-1558.926
16	-2126.259
15	-2787.253
14	-3543.230
13	-4392.989
12	-5333.054
11	-6354.988
10	-7477.604
9	-8664.826
8	-9906.114
7	-11181.717
6	-12463.014
5	-13849.908
4	-15145.244
3	-16295.835
2	-17204.098
1	-17901.461

4.6.5 Calculations of Moment Superposition and Distribution it

Examples for calculations:

cc. M_o from table above.

$M_{o,20}$ = 61.86 KNm

dd. X_i from table above.

X_{20} = -351.35 KNm

ee. $Mp_{total} = (M_{i,o} + X_i)$.

$Mp_{total} = (M_{o,20} + X_{20}) = 61.86 + (-351.35) = -289.48$ KNm.

ff. $I_{P\ \text{single Shear wall- right and left}} = 0.0161\ m^4$.

gg. $I_{P\ \text{single Shear wall- center}} = 1.867\ m^4$.

hh. I_P core Shear wall = 0.279 m^4.

ii. $\sum I_P = 4.36\ m^4$.

jj. $M_{\text{Single Shear wall}} = Mp_{total} \times (Ip_{\text{single shear wall}}/\sum I_P)$.

$M_{\text{Single Shear wall (right and left)},20} = -289.48 \times (0.0161 / 4.36)$

$= -1.07$ kNm.

$M_{\text{Single Shear wall (center)},20} = -289.48 \times (1.867 / 4.36)$

$= -124.06$ kNm.

kk. $M_{\text{Core Shear wall}} = Mp_{total} \times (Ip_{\text{Core shear wall}}/\sum I_P)$.

$M_{\text{Core Shear wall}} = -289.48 \times (0.279 / 4.36)$

$= -18.54$ kNm.

The Total Moment in shear wall and moment in single shear wall values are calculated for all story and can be found in table 4.50.

Table 4.50: The Total Moment in Shear Wall and Moment in Single Shear Wall Values in (Case2.1).

Floor	M_o (kNm)	X_i (kNm)	M total Shear wall (kNm)	M single Shear wall		M Core (kNm)
				Right and Left (kNm)	Center (kNm)	
20	61.86	-351.35	-289.48	-1.07	-124.06	-18.54
19	246.42	-683.16	-436.74	-1.61	-187.17	-27.97
18	551.54	-1080.79	-529.25	-1.95	-226.82	-33.90
17	975.02	-1558.93	-583.91	-2.16	-250.25	-37.40
16	1514.51	-2126.26	-611.75	-2.26	-262.18	-39.18
15	2167.57	-2787.25	-619.69	-2.29	-265.58	-39.69
14	2931.60	-3543.23	-611.63	-2.26	-262.13	-39.17
13	3803.85	-4392.99	-589.14	-2.17	-252.49	-37.73
12	4781.41	-5333.05	-551.65	-2.04	-236.42	-35.33
11	5861.13	-6354.99	-493.86	-1.82	-211.65	-31.63
10	7039.66	-7477.60	-437.94	-1.62	-187.69	-28.05
9	8313.35	-8664.83	-351.48	-1.30	-150.63	-22.51
8	9678.22	-9906.11	-227.89	-0.84	-97.67	-14.60
7	11129.89	-11181.72	-51.83	-0.19	-22.21	-3.32
6	12663.46	-12463.01	200.45	0.74	85.91	12.84
5	14273.39	-13849.91	423.49	1.56	181.49	27.12
4	15953.25	-15145.24	808.01	2.98	346.29	51.75
3	17695.35	-16295.84	1399.51	5.17	599.79	89.64
2	19495.47	-17204.10	2291.37	8.46	982.01	146.76
1	21985.89	-17901.46	4084.43	15.08	1750.46	261.60

4.6.6 Calculations of Shear Forces

Examples of calculations:

ll. $\sum Ti = \Delta X/hi = (Xi+1-Xi)/hi$

$\sum T20 = (X21-X20)/h20 = (0 - (-289.48))/3 = 117.12$ kN.

mm. $Tcol = \sum T \times (Dc/\sum D)$

Tcol (C1),20 = 117.12 × (51.16 / 2699.402) = 2.22 kN.

Tcol (C2),20 = 117.12 × (110.78 / 2699.402) = 4.81 kN.

Tcol (C3),20 = 117.12 × (50.48 / 2699.402) = 2.19 kN.

Tcol (C4),20 = 117.12 × (45.98 / 2699.402) = 2.00 kN.

Tshear w (R&L),20 = 117.12 × (55.62 / 2699.402) = 2.41 kN.

Tshear w (center),20 = 117.12 × (191.172 / 2699.402) = 4.29 kN.

Tcore,20 = 117.12 × (114.042 / 2699.402) = 4.95 kN.

Table 4.51: Shear Force Calculation in (Case 2.1).

Floor	∑T	T_{col}				T_{fi} Normal shear wall		T_{fi} Core
		C1	C2	C3	C4	Right and Left	Center	
20	117.12	2.22	4.81	2.19	1.99	2.41	8.29	4.95
19	110.60	2.53	5.48	2.50	2.28	2.75	6.31	3.05
18	132.55	3.03	6.57	2.99	2.73	3.30	7.56	3.66
17	159.38	3.65	7.90	3.60	3.28	3.97	9.09	4.40
16	189.11	4.33	9.37	4.27	3.89	4.71	10.78	5.22
15	220.33	5.04	10.92	4.98	4.53	5.48	12.56	6.08
14	251.99	5.77	12.49	5.69	5.18	6.27	14.37	6.95
13	283.25	6.48	14.04	6.40	5.83	7.05	16.15	7.81
12	313.35	7.17	15.53	7.08	6.45	7.80	17.87	8.64
11	340.64	7.82	16.93	7.71	7.02	8.50	19.47	9.21
10	374.21	7.87	18.18	9.05	8.17	9.13	20.92	11.10
9	395.74	8.34	19.28	9.59	8.67	9.68	22.19	11.49
8	413.76	8.72	20.16	10.03	9.06	10.12	23.20	12.02
7	425.20	8.97	20.72	10.31	9.31	10.40	23.84	12.35
6	427.10	9.02	20.82	10.38	10.10	10.45	23.95	11.60
5	462.30	12.62	20.32	12.42	10.98	10.20	23.38	15.45
4	431.78	11.91	19.18	11.72	10.37	9.63	22.06	13.46
3	383.53	10.58	17.03	10.41	9.21	8.55	19.60	11.96

4.6.7 Calculations of Coupled Beam End Moments.

Examples for calculations:

nn. For normal shear walls (Center):

M20,u = 1/3 × 8.29 × 3 = 8.29 kNm.

M20,o = 2/3 × 8.29 × 3 = 16.59 kNm.

M19,u = 1/2 × 6.31 × 3 = 9.46 kNm.

M19,o = 8.29+ 9.46 = 17.75 kNm.

Table 4.52: The Coupled Beam End Moments Values in (Case 2.1).

Floor	h i (m)	Shear wall Center		Core	
		Mi,u	Mi,o	Mi,u	Mi,o
19	3	9.461	17.755	4.577	9.525
18	3	11.338	20.798	5.485	10.062
17	3	13.633	24.970	6.595	12.080
16	3	16.176	29.809	7.826	14.421
15	3	18.847	35.023	9.117	16.943
14	3	21.555	40.401	10.428	19.545
13	3	24.229	45.783	11.721	22.149
12	3	26.803	51.032	12.967	24.688
11	3	29.210	56.013	13.815	26.781
10	3	31.379	60.589	16.654	30.468
9	3	33.279	64.658	17.242	33.895
8	3	34.794	68.073	18.027	35.269
7	3	35.756	70.550	18.525	36.552
6	3	35.928	71.684	17.398	35.923
5	3	35.069	70.997	23.168	40.566
4	3	33.093	68.162	20.190	43.358
3	3	29.395	62.487	17.934	38.124
2	3	23.032	52.427	14.891	32.825
1	4	20.405	43.437	14.048	28.939

4.6.8 Calculation for Corrected Shear Wall and Core Moments at Bottom and Top.

Examples for calculations:

Moment of shear wall (Center) for 1st floor:

M1= 1750.46 + 20.41 = 1770.86 kNm

M2= 982.01 – 20.41 = 961.61 kNm

Moment of shear wall (Center) for 2nd floor:

M1= 982.01 + 23.03 = 1005.04 kNm

M2= 599.79 – 23.03 = 576.76 kNm

Table 4.53: The Corrected Shear Wall Moments at Bottom and Top Values in (Case 2.1).

Floor	Shear wall (Center) (kNm)			
	M single	Mi,u	M1 Bottom	M2 Upper
18	-226.82	11.34	-215.48	-198.51
17	-250.25	13.63	-236.61	-240.45
16	-262.18	16.18	-246.00	-266.42
15	-265.58	18.85	-246.73	-281.02
14	-262.13	21.55	-240.57	-287.13
13	-252.49	24.23	-228.26	-286.35
12	-236.42	26.80	-209.61	-279.29
11	-211.65	29.21	-182.44	-265.63
10	-187.69	31.38	-156.31	-243.03
9	-150.63	33.28	-117.35	-220.97
8	-97.67	34.79	-62.87	-185.43
7	-22.21	35.76	13.54	-133.42
6	85.91	35.93	121.83	-58.14
5	181.49	35.07	216.56	50.84
4	346.29	33.09	379.38	148.40
3	599.79	29.39	629.18	316.89
2	982.01	23.03	1005.04	576.76
1	1750.46	20.41	1770.86	961.61

Examples for calculations:

d. Moment of core shear wall for 1st floor:

M1= 261.60 + 14.05 = 275.65 kNm

M2= 146.76 – 14.05 = 132.71 kNm

e. Moment of core shear wall for 2nd floor:

M1= 146.76 + 14.89 = 161.65 kNm

M2= 89.64 – 14.89 = 74.75 kNm

Table 4.54: The Corrected Core Wall Moments at Bottom and Top Values in (Case 2.1).

Floor	Core (kNm)			
	M_{single}	$M_{i,u}$	M1 Bottom	M2 Upper
20	-18.54	4.95	-13.59	-9.90
19	-27.97	4.58	-23.40	-23.12
18	-33.90	5.48	-28.41	-33.46
17	-37.40	6.60	-30.80	-40.49
16	-39.18	7.83	-31.36	-45.22
15	-39.69	9.12	-30.57	-48.30
14	-39.17	10.43	-28.75	-50.12
13	-37.73	11.72	-26.01	-50.90
12	-35.33	12.97	-22.37	-50.70
11	-31.63	13.81	-17.82	-49.15
10	-28.05	16.65	-11.40	-48.28
9	-22.51	17.24	-5.27	-45.29
8	-14.60	18.03	3.43	-40.54
7	-3.32	18.53	15.21	-33.12
6	12.84	17.40	30.24	-20.72
5	27.12	23.17	50.29	-10.33
4	51.75	20.19	71.94	6.93
3	89.64	17.93	107.57	33.82
2	146.76	14.89	161.65	74.75
1	261.60	14.05	275.65	132.71

4.6.9 Calculation of Displacement, Drift of Story

Examples for calculations:

b. $\sum T$, $\sum Dij$ from table above.

$\sum T20 = 117.12$ kN.

$\sum D20j = = 989480.67$ kN/m

▪ $\delta i = \sum T / \sum Dij$ (m).

$\delta 20 = 117.12 / 989480.67 = 0.000118$ m.

▪ $di = \sum \delta i$ (m).

$d20 = \delta 20 + d19 = 0.000118 + 0.006799 = 0.006917$ m.

Table 4.55: The Displacement and Drift Values in (Case 2.1).

Floor	hi (m)	∑T (KN)	∑Dij KN / m	δi (m)	di (m)
18	3	132.545	819486.29	0.000162	0.006664
17	3	159.379	819486.29	0.000194	0.006502
16	3	189.111	819486.29	0.000231	0.006308
15	3	220.331	819486.29	0.000269	0.006077
14	3	251.992	819486.29	0.000308	0.005808
13	3	283.253	819486.29	0.000346	0.005501
12	3	313.355	819486.29	0.000382	0.005155
11	3	340.644	817462.52	0.000417	0.004773
10	3	374.205	835928.78	0.000448	0.004356
9	3	395.741	833567.70	0.000475	0.003908
8	3	413.763	833567.70	0.000496	0.003433
7	3	425.201	833567.70	0.000510	0.002937
6	3	427.099	833285.84	0.000513	0.002427
5	3	462.298	924038.91	0.000500	0.001914
4	3	431.779	914592.61	0.000472	0.001414
3	3	383.530	914592.61	0.000419	0.000942
2	3	302.754	921402.13	0.000329	0.000523
1	4	174.341	898357.75	0.000194	0.000194

Check the result Case (2.1):

The Muto method is used to calculate the sway of a framed building based on lateral wind forces:

Δ max = 0.000194 + 0.000329 + ……… + 0.000135 + 0.000118 = 0.006917 m

= 0.006917 m × 103 = 6.92 mm

The recommended maximum sway is Δ max-all = 0.002 × H building

Δ max-all = 0.002 × H = 0.002 × (61 m) × 103 = 122 mm > 6.92 mm (O.K.)

That mean the frame sways just slightly; the bending stiffness of the framed structure is significantly higher.

Maximum drift 7th = 0.000552

The drift ratio = 0.000513 / 3 = 0.000171 < 0.002 (O.K.) according to TBC 2007.

4.7 CALCULATION OF WIND FORCE (EQUIVALENT SEISMIC LOAD DISTRIBUTED) AND IMPACT ON STRUCTURE IN X-DIR (CASE 2.2)

Calculate the Base shear force acting in building surface is complete according to (EN 1991-1-4) in item (This:4.6.1), distributed it to the structure floor using “Equivalent Seismic Load Method” and study the impact on the structure, as following:

4.7.1 Distributed Lateral Load

Base shear (Vt) = 623 kN

ΔFn = 0.0075 × N × Vt

ΔFn = 0.0075 × 20 × 623 = 93.45 kN

∑Fi = Vt - ΔFn = 623 – 93.45 = 529.55 kN

By using the equation (3-79) to distribute the load:

Fi = (Wi × Hi / ∑Wj × Hj) **×** ∑Fi

F20 = (W20 × H20 / ∑W20 × H20) **×** ∑F20

F20 = (4718.51 × 61 / 3236278.46) **×** 529.55 = 47.10 kN

Fi,total = ΔFn + Fi

F20,total = ΔF20 + F20

F20,total = 93.45 + 47.10 = 140.55 kN

Table 4.56: External Force Acting in Each Floor Disturbed by 1st Mode Shape (Case 2.2).

Story	h_i	H_i	w_i	$w_i \times H_i$	$\frac{w_i \times H_i}{\sum w_j \times H_j}$	ΔF_n	F_i	$F_{i,total}$
8	3	25	5017.31	125432.66	0.039	-	20.52	20.52
7	3	22	5017.31	110380.74	0.034	-	18.06	18.06
6	3	19	5017.31	95328.82	0.029	-	15.60	15.60
5	3	16	5041.31	80660.90	0.025	-	13.20	13.20
4	3	13	5041.31	65536.98	0.020	-	10.72	10.72
3	3	10	5041.31	50413.06	0.016	-	8.25	8.25
2	3	7	5041.31	35289.14	0.011	-	5.77	5.77
1	4	4	5663.56	22654.23	0.007	-	3.71	3.71
Summation				3236278.46	1.00	93.45	529.55	623.00

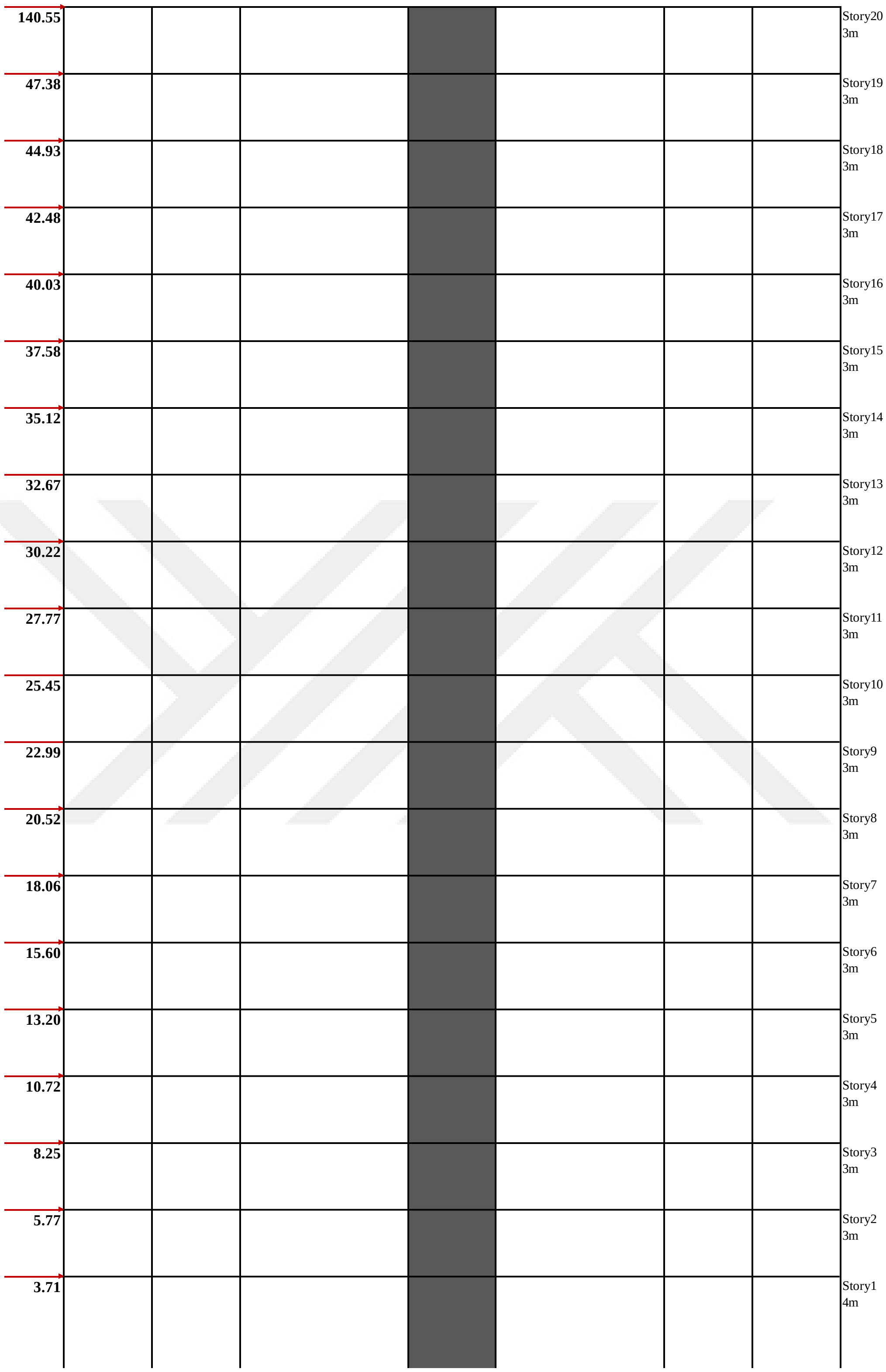


Figure 4.19: The Seismic Force Acting on Each Floor (F_i) in (Case 2.2).

4.7.2 Calculation of Shear Force (Ti) and Cantilever Moments (Mo)

Table 4.57: Shear Force (Ti) and Cantilever Moments (Mo) Calculation in (Case 2.2).

Story	hi (m)	Pi (kN)	Ti (kN)	Mo (kNm)
20	3	140.55	140.55	421.64
19	3	47.38	187.92	985.42
18	3	44.93	232.85	1683.97
17	3	42.48	275.33	2509.96
16	3	40.03	315.35	3456.02
15	3	37.58	352.93	4514.81
14	3	35.12	388.05	5678.97
13	3	32.67	420.73	6941.16
12	3	30.22	450.95	8294.01
11	3	27.77	478.73	9730.19
10	3	25.45	504.18	11242.71
9	3	22.99	527.16	12824.20
8	3	20.52	547.69	14467.26
7	3	18.06	565.75	16164.51
6	3	15.60	581.35	17908.55
5	3	13.20	594.55	19692.19
4	3	10.72	605.27	21508.00
3	3	8.25	613.52	23348.56
2	3	5.77	619.29	25206.44
1	4	3.71	623.00	27698.44

4.7.3 Calculations of Coefficients and Constants of Equation Set

Table 4.58: The Fi, fi, δi,i-1 , δi,i and δi,o Values in (Case 2.2).

Floor	h_i (m)	F_i (1/m³)	f_i (1/m³)	M_o (kNm)	$\delta_{i,i-1}$ (1/m³)	$\delta_{i,i}$ (1/m³)	$\delta_{i,o}$ (kNm)
20	3	3.705	0.1148	421.642	-	8.638	306.731
19	3	4.474	0.1148	985.416	-4.3596	9.408	694.199
18	3	4.474	0.1148	1683.972	-4.3596	9.408	1174.500
17	3	4.474	0.1148	2509.957	-4.3596	9.408	1742.570
16	3	4.474	0.1148	3456.019	-4.3596	9.408	2393.347
15	3	4.474	0.1148	4514.808	-4.3596	9.408	3121.766
14	3	4.474	0.1148	5678.970	-4.3596	9.408	3922.764
13	3	4.474	0.1148	6941.156	-4.3596	9.408	4791.277
12	3	4.474	0.1148	8294.012	-4.3596	9.419	5722.242
11	3	4.485	0.1148	9730.188	-4.3706	9.331	6710.639
10	3	4.386	0.1148	11242.714	-4.2715	9.244	7751.575

Table 4.58: The Fi, fi, δi,i-1 , δi,i and δi,o Values in (Case 2.2).

"Table continued"

Floor	h_i (m)	F_i (1/m³)	f_i (1/m³)	M_o (kNm)	$\delta_{i,i-1}$ (1/m³)	$\delta_{i,i}$ (1/m³)	$\delta_{i,o}$ (kNm)
9	3	4.399	0.1148	12824.203	-4.2840	9.257	8840.011
8	3	4.399	0.1148	14467.265	-4.2840	9.257	9970.857
7	3	4.399	0.1148	16164.511	-4.2840	9.258	11139.023
6	3	4.400	0.1148	17908.554	-4.2855	8.828	12339.443
5	3	3.968	0.1148	19692.192	-3.8533	8.436	13567.109
4	3	4.009	0.1148	21508.001	-3.8943	8.477	14816.934
3	3	4.009	0.1148	23348.557	-3.8943	8.448	16083.804
2	3	3.979	0.1148	25206.436	-3.8646	6.811	20423.198
1	4	2.296	0.1531	27698.436	-2.1428	2.602	12337.165

4.7.4 Calculations of Matrix Coefficients and Load Vector

a. $[\delta] \times \{X_i\} = -\{\delta_{i,o}\}$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		Xi		δi,o
5.077	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X20		-290.22
-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X19		-650.81
0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X18		-1094.97
0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X17		-1621.19
0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X16		-2227.50
0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X15		-2911.39
0.000	0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X14		-3669.94
0.000	0.000	0.000	0.000	0.000	0.000	-2.785	5.728	-2.785	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X13		-4499.74
0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.785	5.721	-2.779	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X12		-5396.97
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.779	5.574	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	×	X11	=	-6357.44
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X10		-7376.77
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000	0.000		X9		-8450.26
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000	0.000		X8		-9572.84
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.433	-2.638	0.000	0.000	0.000	0.000	0.000		X7		-10739.20
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.638	5.549	-2.754	0.000	0.000	0.000	0.000		X6		-11943.77
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.754	5.665	-2.754	0.000	0.000	0.000		X5		-13180.81
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.754	5.665	-2.754	0.000	0.000		X4		-14444.11
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.754	5.661	-2.750	0.000		X3		-15727.24
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-2.750	4.453	-1.519		X2		-20030.03
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-1.519	1.624		X1		-12121.68

b. $[\delta]^{-1} \times -\{\delta_{i,o}\} = \{X_i\}$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		δi,o		Xi
0.347	0.274	0.216	0.171	0.135	0.107	0.084	0.067	0.053	0.042	0.033	0.026	0.021	0.016	0.013	0.011	0.009	0.008	0.007	0.007		-290.22		-4189.13
0.274	0.500	0.394	0.311	0.246	0.194	0.154	0.122	0.096	0.077	0.060	0.048	0.038	0.030	0.024	0.020	0.017	0.015	0.013	0.012		-650.81		-7531.59
0.216	0.394	0.595	0.470	0.371	0.293	0.232	0.183	0.145	0.115	0.091	0.072	0.057	0.045	0.036	0.030	0.025	0.022	0.020	0.019		-1094.97		-11064.86
0.171	0.311	0.470	0.654	0.517	0.408	0.323	0.255	0.202	0.161	0.127	0.100	0.079	0.063	0.051	0.042	0.035	0.031	0.028	0.026		-1621.19		-14828.61
0.135	0.246	0.371	0.517	0.691	0.546	0.432	0.342	0.271	0.215	0.169	0.134	0.106	0.084	0.068	0.056	0.047	0.041	0.037	0.035		-2227.50		-18846.04
0.107	0.194	0.293	0.408	0.546	0.715	0.565	0.447	0.355	0.282	0.222	0.175	0.138	0.110	0.089	0.073	0.061	0.053	0.049	0.045		-2911.39		-23125.88
0.084	0.154	0.232	0.323	0.432	0.565	0.731	0.578	0.458	0.364	0.287	0.226	0.179	0.143	0.115	0.094	0.079	0.069	0.063	0.059		-3669.94		-27663.82
0.067	0.122	0.183	0.255	0.342	0.447	0.578	0.742	0.588	0.467	0.368	0.290	0.230	0.183	0.147	0.121	0.102	0.089	0.080	0.075		-4499.74		-32443.26
0.053	0.096	0.145	0.202	0.271	0.355	0.458	0.588	0.751	0.596	0.469	0.370	0.293	0.233	0.188	0.154	0.130	0.113	0.103	0.096		-5396.97		-37435.68
0.042	0.077	0.115	0.161	0.215	0.282	0.364	0.467	0.596	0.760	0.598	0.472	0.373	0.297	0.239	0.197	0.166	0.144	0.131	0.122	×	-6357.44	=	-42612.81
0.033	0.060	0.091	0.127	0.169	0.222	0.287	0.368	0.469	0.598	0.769	0.606	0.480	0.382	0.307	0.253	0.213	0.185	0.168	0.157		-7376.77		-48191.78
0.026	0.048	0.072	0.100	0.134	0.175	0.226	0.290	0.370	0.472	0.606	0.777	0.615	0.490	0.394	0.324	0.273	0.238	0.216	0.202		-8450.26		-53842.16
0.021	0.038	0.057	0.079	0.106	0.138	0.179	0.230	0.293	0.373	0.480	0.615	0.787	0.627	0.504	0.415	0.350	0.304	0.276	0.258		-9572.84		-59493.25
0.016	0.030	0.045	0.063	0.084	0.110	0.143	0.183	0.233	0.297	0.382	0.490	0.627	0.801	0.644	0.530	0.447	0.389	0.353	0.330		-10739.20		-65055.80
0.013	0.024	0.036	0.051	0.068	0.089	0.115	0.147	0.188	0.239	0.307	0.394	0.504	0.644	0.823	0.677	0.570	0.496	0.450	0.421		-11943.77		-70418.68
0.011	0.020	0.030	0.042	0.056	0.073	0.094	0.121	0.154	0.197	0.253	0.324	0.415	0.530	0.677	0.856	0.721	0.628	0.569	0.533		-13180.81		-75232.94
0.009	0.017	0.025	0.035	0.047	0.061	0.079	0.102	0.130	0.166	0.213	0.273	0.350	0.447	0.570	0.721	0.914	0.795	0.721	0.675		-14444.11		-79549.47
0.008	0.015	0.022	0.031	0.041	0.053	0.069	0.089	0.113	0.144	0.185	0.238	0.304	0.389	0.496	0.628	0.795	1.007	0.914	0.855		-15727.24		-83155.60
0.007	0.013	0.020	0.028	0.037	0.049	0.063	0.080	0.103	0.131	0.168	0.216	0.276	0.353	0.450	0.569	0.721	0.914	1.159	1.084		-20030.03		-85794.87
0.007	0.012	0.019	0.026	0.035	0.045	0.059	0.075	0.096	0.122	0.157	0.202	0.258	0.330	0.421	0.533	0.675	0.855	1.084	1.630		-12121.68		-87730.02

The X_i values are calculated for all the story and can be found in table 4-59.

Table 4.59: The Xi Values Calculated for (Case 2.2).

Floor	Xi (kNm)
20	-781.778
19	-1478.663
18	-2249.928
17	-3107.254
16	-4055.786
15	-5096.107
14	-6225.494
13	-7438.645
12	-8728.008
11	-10080.309
10	-11518.382
9	-12994.759
8	-14496.907
7	-16002.369
6	-17479.995
5	-19045.560
4	-20479.331
3	-21730.435
2	-22702.228
1	-23437.337

4.7.5 Calculations of Moment Superposition and Distribution it

Table 4.60: The Total Moment in Shear Wall and Moment in Single Shear Wall Values in (Case2.2).

Floor	Mo (kNm)	Xi (kNm)	M total Shear wall (kNm)	M single Shear wall		M Core (kNm)
				(Right and Left) (kNm)	(Center) (kNm)	
20	421.642	-781.778	-360.137	-1.329	-154.343	-23.066
19	985.416	-1478.663	-493.247	-1.821	-211.390	-31.592
18	1683.972	-2249.928	-565.957	-2.089	-242.551	-36.249
17	2509.957	-3107.254	-597.297	-2.205	-255.983	-38.256
16	3456.019	-4055.786	-599.767	-2.214	-257.041	-38.414
15	4514.808	-5096.107	-581.299	-2.146	-249.127	-37.231
14	5678.970	-6225.494	-546.523	-2.017	-234.223	-35.004
13	6941.156	-7438.645	-497.489	-1.836	-213.208	-31.863
12	8294.012	-8728.008	-433.996	-1.602	-185.997	-27.797
11	9730.188	-10080.309	-350.121	-1.292	-150.051	-22.425

Table 4.61: The Total Moment in Shear Wall and Moment in Single Shear Wall Values in (Case2.2).

Floor	Mo (kNm)	Xi (kNm)	M total Shear wall (kNm)	M single Shear wall		M Core (kNm)
				(Right and Left) (kNm)	(Center) (kNm)	
9	12824.203	-12994.759	-170.556	-0.630	-73.095	-10.924
8	14467.265	-14496.907	-29.642	-0.109	-12.704	-1.899
7	16164.511	-16002.369	162.142	0.599	69.489	10.385
6	17908.554	-17479.995	428.559	1.582	183.667	27.449
5	19692.192	-19045.560	646.632	2.387	277.126	41.416
4	21508.001	-20479.331	1028.670	3.797	440.856	65.885
3	23348.557	-21730.435	1618.122	5.973	693.477	103.638
2	25206.436	-22702.228	2504.209	9.244	1073.226	160.391
1	27698.436	-23437.337	4261.099	15.729	1826.174	272.917

4.7.6 Calculations of Shear Forces

Table 4.62: Shear Force Calculation in (Case 2.2).

Floor	∑T	Tcol				Tfi Normal shear wall		Tfi Core
		C1	C2	C3	C4	Right and Left	Center	
20	260.593	4.939	10.694	4.873	4.438	5.370	18.455	11.009
19	232.295	5.317	11.514	5.247	4.779	5.781	13.247	6.408
18	257.088	5.885	12.743	5.807	5.289	6.398	14.660	7.092
17	285.775	6.541	14.165	6.455	5.879	7.112	16.296	7.884
16	316.177	7.237	15.672	7.141	6.504	7.869	18.030	8.722
15	346.774	7.938	17.188	7.832	7.134	8.630	19.775	9.566
14	376.462	8.617	18.660	8.503	7.744	9.369	21.468	10.385
13	404.384	9.256	20.044	9.134	8.319	10.064	23.060	11.156
12	429.788	9.838	21.303	9.707	8.841	10.696	24.509	11.857
11	450.767	10.344	22.398	10.206	9.296	11.246	25.769	12.187
10	479.358	10.079	23.293	11.589	10.469	11.695	26.798	14.222
9	492.125	10.377	23.981	11.931	10.778	12.041	27.589	14.294
8	500.716	10.558	24.399	12.139	10.967	12.251	28.071	14.544
7	501.821	10.581	24.453	12.166	10.991	12.278	28.133	14.576
6	492.542	10.399	24.009	11.968	11.648	12.055	27.622	13.376
5	521.855	14.246	22.940	14.015	12.400	11.518	26.392	17.435
4	477.924	13.182	21.225	12.968	11.473	10.657	24.419	14.898
3	417.035	11.502	18.521	11.316	10.011	9.299	21.308	13.000
2	323.931	8.868	14.280	8.725	7.719	7.170	16.429	10.622
1	183.777	4.199	10.097	4.196	4.181	2.556	10.755	7.404

4.7.7 Calculations of Coupled Beam End Moments

Table 4.63: The Coupled Beam End Moments Values in (Case 2.2).

Floor	h i (m)	Shear wall Center		Core	
		Mi,u	Mi,o	Mi,u	Mi,o
20	3	18.455	36.911	11.009	22.019
19	3	19.870	38.325	9.612	20.622
18	3	21.991	41.860	10.638	20.251
17	3	24.444	46.435	11.826	22.464
16	3	27.045	51.489	13.084	24.909
15	3	29.662	56.707	14.350	27.433
14	3	32.202	61.864	15.578	29.928
13	3	34.590	66.791	16.734	32.312
12	3	36.763	71.353	17.785	34.518
11	3	38.653	75.416	18.281	36.065
10	3	40.196	78.849	21.333	39.614
9	3	41.384	81.580	21.441	42.774
8	3	42.106	83.490	21.815	43.256
7	3	42.199	84.306	21.863	43.679
6	3	41.433	83.632	20.064	41.927
5	3	39.587	81.020	26.153	46.217
4	3	36.629	76.217	22.348	48.501
3	3	31.963	68.592	19.501	41.848
2	3	24.643	56.606	15.933	35.433
1	4	21.510	46.153	14.809	30.741

4.7.8 Calculation for Corrected Shear Wall and Core Moments at Bottom and Top

Table 4.64: The Corrected Shear Wall Moments at Bottom and Top Values in (Case 2.2).

Floor	Shear wall (Center) (kNm)			
	M single	$M_{i,u}$	M_1 Bottom	M_2 Upper
20	-154.34	18.46	-135.89	-36.91
19	-211.39	19.87	-191.52	-174.21
18	-242.55	21.99	-220.56	-233.38
17	-255.98	24.44	-231.54	-267.00
16	-257.04	27.04	-230.00	-283.03
Floor	Shear wall (Center) (kNm)			
	M single	$M_{i,u}$	M_1 Bottom	M_2 Upper
15	-249.13	29.66	-219.46	-286.70
14	-234.22	32.20	-202.02	-281.33
13	-213.21	34.59	-178.62	-268.81
12	-186.00	36.76	-149.23	-249.97
11	-150.05	38.65	-111.40	-224.65
10	-118.14	40.20	-77.95	-190.25
9	-73.09	41.38	-31.71	-159.53
8	-12.70	42.11	29.40	-115.20
7	69.49	42.20	111.69	-54.90
6	183.67	41.43	225.10	28.06
5	277.13	39.59	316.71	144.08
4	440.86	36.63	477.48	240.50
3	693.48	31.96	725.44	408.89
2	1073.23	24.64	1097.87	668.83
1	1826.17	21.51	1847.68	1051.72

Table 4.65: The Corrected Core Wall Moments at Bottom and Top Values in (Case 2.2).

Floor	Core (kNm)			
	M single	$M_{i,u}$	M_1 Bottom	M_2 Upper
20	-23.07	11.01	-12.06	-22.02
19	-31.59	9.61	-21.98	-32.68
18	-36.25	10.64	-25.61	-42.23
17	-38.26	11.83	-26.43	-48.07
16	-38.41	13.08	-25.33	-51.34
15	-37.23	14.35	-22.88	-52.76
14	-35.00	15.58	-19.43	-52.81
13	-31.86	16.73	-15.13	-51.74
12	-27.80	17.78	-10.01	-49.65
11	-22.42	18.28	-4.14	-46.08
10	-17.66	21.33	3.68	-43.76
9	-10.92	21.44	10.52	-39.10
8	-1.90	21.82	19.92	-32.74
7	10.38	21.86	32.25	-23.76
6	27.45	20.06	47.51	-9.68
5	41.42	26.15	67.57	1.30
4	65.88	22.35	88.23	19.07
3	103.64	19.50	123.14	46.38
2	160.39	15.93	176.32	87.71
1	272.92	14.81	287.73	145.58

4.7.9 Calculation of Displacement, Drift of Story

Table 4.66: The Displacement and Drift Values in (Case 2.2).

Floor	hi (m)	∑T (KN)	∑Dij KN / m	δi (m)	di (m)
20	3	260.593	989780.67	0.000263	0.009104
19	3	232.295	819486.29	0.000283	0.008841
18	3	257.088	819486.29	0.000314	0.008557
17	3	285.775	819486.29	0.000349	0.008243
16	3	316.177	819486.29	0.000386	0.007895
15	3	346.774	819486.29	0.000423	0.007509
14	3	376.462	819486.29	0.000459	0.007086
13	3	404.384	819486.29	0.000493	0.006626
12	3	429.788	819486.29	0.000524	0.006133
11	3	450.767	817462.52	0.000551	0.005608
10	3	479.358	835928.78	0.000573	0.005057
9	3	492.125	833567.70	0.000590	0.004484
8	3	500.716	833567.70	0.000601	0.003893
7	3	501.821	833567.70	0.000602	0.003293
6	3	492.542	833285.84	0.000591	0.002691
5	3	521.855	924038.91	0.000565	0.002099
4	3	477.924	914592.61	0.000523	0.001535
3	3	417.035	914592.61	0.000456	0.001012
2	3	323.931	921402.13	0.000352	0.000556
1	4	183.777	898357.75	0.000205	0.000205

Check the result Case (2.2):

The Muto method is used to calculate the sway of a framed building (di) based on lateral wind forces:

Δ max = 0.000205 + 0.000352 + ……… + 0.000283 + 0.000263 = 0.009104 m

= 0.009104 m × 103 = 9.1 mm

The recommended maximum sway is Δ max-all = 0.002 × H building

Δ max-all = 0.002 × H = 0.002 × (61 m) × 103 = 122 mm > 9. 1 mm. (O.K.)

The drift ratio = 0.000602 × 7/ 3 = 0.0014 < 0.02 (O.K.)

4.7.10 Acceleration Values of Equivalent Earthquake Force to Wind Force

The method "equivalent seismic load method" was used to distribute the base shear force that was calculated based on (Eurocode) in X-direction to study the effect of earthquakes on the building with a force equivalent to the wind force affecting the building. Below is a comparison between the two cases:

Table 4.67: Comparison Between the Two Cases in (X-Dir).

Item	Unit	Value	
		Eurocode	ESLM
$M_{o,20}$	kNm	21985.89	27698.436
X20	kNm	-17901.46	-23437.337
$M_{p,20}$	kNm	4084.43	4261.0993
$M1_{SW\text{-}C,1}$	kNm	1770.86	1847.68
$M1_{CW,1}$	kNm	275.65	287.73
$\sum T_5$	kN	462.30	521.8548
Ratio $\delta_{6,7}$	-	0.000513	0.0014
d_{20}	mm	6.92	9.10

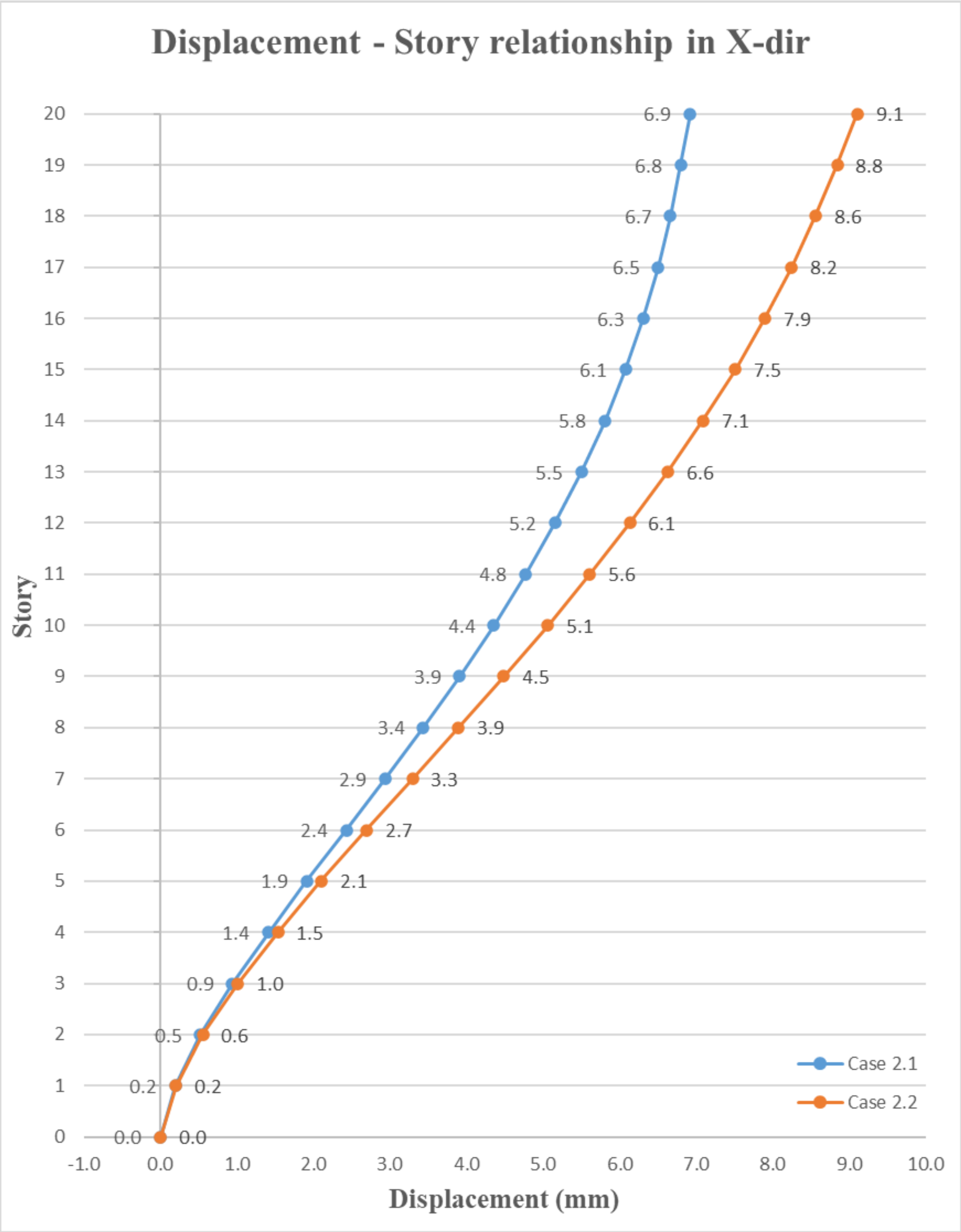


Figure 4.20: Displacement - Story Relationship in (X-Dir).

First, need to calculate of the fundamental period of the building.

by rayleigh method in x-direction:

examples for calculations:

$m_1 = w_1 / 9.81 \quad = 5663.56 / 9.81 = 577.3$ (kn/m/sec^2)

$m_1d_1^2 = m_1 \times d_1^2 = 577.56 \times 0.000361^2 = 0.000075$ knm sec²

$p_1d_1 = p_1 \times d_1 = 16.20 \times 0.000361 = 0.00585$ knm

The mass, midi2 and pidi values in each story are calculated for all stories and can be found in table 4-67 for (case 2.2).

Table 4.68: The Mass, Midi² and Pidi Values for Case (2.2).

Floor	w_i (KN)	M_i (KN/m/sec²)	$M_id_i^2$ kNm.Sec²	P_id_i (KNm)
17	4992.11	508.9	0.03458	0.350
16	4992.11	508.9	0.03172	0.316
15	4992.11	508.9	0.02869	0.282
14	4992.11	508.9	0.02555	0.249
13	4992.11	508.9	0.02234	0.217
12	4992.11	508.9	0.01914	0.185
11	4992.11	508.9	0.01601	0.156
10	5017.31	511.4	0.01308	0.129
9	5017.31	511.4	0.01028	0.103
8	5017.31	511.4	0.00775	0.080
7	5017.31	511.4	0.00554	0.059
6	5017.31	511.4	0.00370	0.042
5	5041.31	513.9	0.00227	0.028
4	5041.31	513.9	0.00121	0.016
3	5041.31	513.9	0.00053	0.008
2	5041.31	513.9	0.00016	0.003
1	5663.56	577.3	0.00002	0.001
			0.339	4.307

$\omega_1^2{}_{(Y)} = \sum P_id_i / \sum M_id_i^2 = 4.307 / 0.339 = 12.69$

$\omega_{1\,(Y)} = 3.56$ rad/sec.

T1(Y) = (2 × 3.14) / 3.56 = 1.76 Sec.

The acceleration values of equivalent earthquake force to wind force:

Seismic characteristic:

a. Assume Local Site Class = Z2

b. Spectrum Coefficient = (TA = 0.15 sec), (TB = 0.40 sec).

c. Effective Ground Acceleration Coefficient (Ao) = 0.30

d. Structural Behavior Factors (R) = 7.

e. Building Importance Factor (I) For Hotel building = 1

f. Damping ratio (ξ) = 5%

By using ERZICAN 1992 N-S chart, intersect the T-period value (1.76) with Damping ratio (ξ=5%), the (Sa/amax) value = 1.72

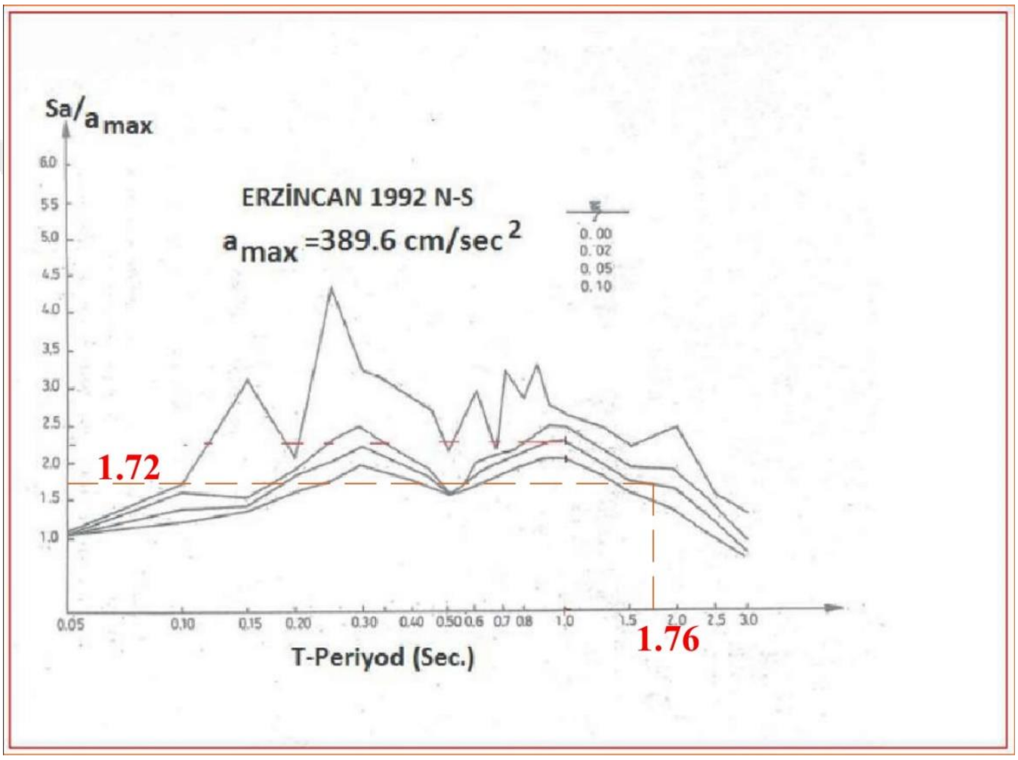


Figure 4.21: Erzican 1992 N-S Chart in Case (2.2).

Spectrum coefficient $S_{(T)} = 2.5 \times (0.4/1.72)^{0.8} = 0.763$

$A_{(T1(Y))} = A_o \times S_{(T)} \times I \times 9.81$

$V_t = \sum m_i \times A_{(T1(Y))} / R$

$V_{t2} = Vt \times (S_a/a_{max})/S_{(T)}$

$623 = Vt \times 2.254$

$V_t = 623 / 2.254 = 276.398$ kN

$276.398 = 10251.05 \times A_{(T1(Y))} / 7$

$A_{(T1(Y))} = 276.398 \times 7 / 10251.05 = 0.189$

$0.189 = A_o \times 0.763 \times 1 \times 9.81$

$A_o = 0.189 / (0.763 \times 9.81) = 0.025$

So, the acceleration values of equivalent earthquake force to wind force is (0.025), but we assumed the seismic zone is second so effective ground acceleration coefficient (Ao) = 0.3, we need to obtain the real base shear force acting on structure due to the earthquake force:

A(T1(Y)) = Ao × S(T) × I × 9.81

= 0.3 × 0.763 × 1 × 9.81

= 2.25

Vt = ∑mi × A(T1(Y)) / R

= 10251.05 × 2.25 / 7

= 3288.79 kN (total equivalent seismic load)

Vt2 = Vt × (Sa/a max)/S(T)

= 3288.79 × 2.254

= 7412.91 kN (In case of using Erzincan 1992 N-S chart)

Vt2 / Vt1 ratio = 7412.91 / 623 = 11.9

Displacement 20th = 9.1 × 11.9 = 108.33 mm < 122 mm (O.K.)

The difference between the wind force and the earthquake force in (X-dir.):

Percentage (Equivalent earthquake/wind) = d20 case 2.2 / d20 case 2.1

= 9.1/6.92 = 132%

Percentage (Earthquake/wind) = (d20 case 2.2 × Vt2 / Vt1 ratio) / d20 case 2.1

= 108.33/6.92 = 1566%

the force of the earthquake (in both cases) is stronger than the wind force and its effect on the displacement by (132%, 1566%) respectively, therefore the effect of the earthquake is stronger and more dangerous on the building.

5. CONCLUSION.

a. Shear rigidity: It has been observed that shear values in the Y direction are higher as a percentage than those in the X direction (58% to 68%).

b. The Forces: The difference in the distribution of wind loads from seismic loads in both directions (X and Y) depends on many factors, where wind loads depend on the building geometry and the turbulence environment, and the resonant response depends on the structure’s dynamic properties: mass, stiffness, and damping. [29] while seismic loads depends on the Earthquake's acceleration, duration and period.[30]

c. Wind Force: It has been discovered that the base shear force in the Y direction is 77% greater than in the X direction.

d. Earthquake Force: It has been discovered that the base shear force in the Y direction is 15% greater than in the X direction.

e. Wind Force and Earthquake Force: The ratios of wind force over the real earthquake in the Y and X directions are (3.2) and (11.9), respectively.

f. The Displacement: The value of displacement resulting from seismic loads equivalent to wind loads is greater than the value of displacement resulting from wind loads, as shown in figures (4.13) and (4.20); therefore, the value of displacement resulting from real seismic loads will certainly be greater than those resulting from wind loads.

g. The effect of the wind force on the building displacement in two directions (X and Y) is 6.9 mm and 16.4 mm, respectively.

h. The effect of the equivalent earthquake force on the building displacement in two directions (X and Y) is 9.1 mm and 22 mm, respectively.

i. The effect of the real earthquake force on the building displacement in two directions (X and Y) is 108.33 mm and 70.24 mm, respectively.

j. The values of displacement of the building in both directions (X and Y) are within the displacement limitation mentioned in the Turkish code, which must not exceed 1/500 of the height of the building.

k. Storey drift ratio: As shown in tables 4.44 and 4-66, the value of storey drift ratio resulting from seismic loads equivalent to wind loads is greater than the value resulting from wind loads; thus, the value of storey drift ratio resulting from real seismic loads will almost certainly be greater than those resulting from wind loads. When the acceptable store drift is taken into account, it is given as a function of the building's behavior coefficients (R) and corrected with V_{t2}/V_{t1}. In general, the X direction value is greater than the Y direction value.

l. Moment: In buildings subjected to lateral loads (wind and earthquake), the value of moments in the Y direction is greater than the value of moments in the X direction.

m. Time-period: The value in the X direction is greater than the value in the Y direction, stiffer buildings have a smaller natural period.[31]

From what was mentioned above, it is clear to us that earthquake loads are the dominant forces on the building more than wind loads; therefore, consideration must be taken when designing the building to ensure that it is strong enough to resist earthquake loads, which will also include wind resistance.

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APPENDIX A

EXCEL SHEET FOR NUMERICAL CALCULATIONS

The numerical analysis for this thesis was completed in **Excel**. In accordance with the defined methodology procedure, the sheets provide the full solution methodology as calculated and numbered tables.

Calculation Y-dir.xlsx

Calculation X-dir.xlsx