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INVESTIGATION OF THE EFFECT OF TAKING TOGETHER SHEAR AND BENDING DEFORMATIONS OF REINFORCED CONCRETE SHEAR WALLS IN HIGH-RISE BUILDINGS UNDER HORIZONTAL LOADS

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Master's Science

Supervisor

Prof. Dr. Zeki HASGÜR

Istanbul, 2023

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The thesis/dissertation titled **"INVESTIGATION OF THE EFFECT OF TAKING TOGETHER SHEAR AND BENDING DEFORMATIONS OF REINFORCED CONCRETE SHEAR WALLS IN HIGH-RISE BUILDINGS UNDER HORIZONTAL LOADS"** prepared by **SAFA ABDUL KAREEM MOHAMMED** and submitted **14/4/2023** has been **accepted unanimously** for the degree of Master of Science in the Department of Civil Engineering.

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Safa Abdulkareem Mohammed

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ABSTRACT

INVESTIGATION OF THE EFFECT OF TAKING TOGETHER SHEAR AND BENDING DEFORMATIONS OF REINFORCED CONCRETE SHEAR WALLS IN HIGH-RISE BUILDINGS UNDER HORIZONTAL LOADS

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The need for buildings is growing in developing nations as their populations and cities expand. Natural disasters like earthquakes can have a negative impact on human civilization. To quote a popular saying, “earthquakes don't kill people; buildings do.” This is due to the fact that the majority of earthquake fatalities result from the collapse of buildings. Buildings designed to withstand seismic activity, such as by employing core walls and/or shear walls, should have some lateral stiffness to prevent excessive swaying even during moderate earthquakes. In this thesis, the effect of taking both bending and shear deformations in reinforced concrete shear walls under earthquake load is studied and analysed in four different cases of a 20-story high-rise office building in both directions (X and Y) using the response spectrum dynamic analysis method and Erzincan's 1992 N -S spectra. Shear walls with frames and those with core walls will be examined. According to the calculations, as the shear forces, maximum displacement, and maximum moment for the shear wall and core increased, the stiffness of the structure decreased. Data shows that shear deformations, in comparison to bending deformations (without shear deformations), increase time periods.

Keywords: Earthquake, Shear Wall, Deformations, Muto Metod, And Lateral Stiffness.

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1. INTRODUCTION

1.1 PROBLEM STATEMENT

A comparison of a 20-story high-rise office building parameters such as time periods, maximum shear walls moments, maximum frame moments, and maximum drifts is made when taking together shear and bending deformations of reinforced concrete shear walls under horizontal loads such as earthquakes in two directions, X and Y, respectively.

1.2 SCOPE

The major goal of this study is to analyse the construction using the Turkish building code to determine the effect of seismic in both directions, X and Y, respectively.

1.3 GENERAL OVERVIEW

With the increase in the human population and urbanization, the demand for high-rise buildings is increasing in developing countries. Furthermore, the limited availability of land for people to build houses increased the demand for tall buildings.

Earthquakes are a natural hazard that can have a detrimental influence on society.[1] They are ground surface vibrations or oscillations induced by a transient disturbance of the elastic or gravitational equilibrium of the rocks at or beneath the earth's surface. Elastic impulses or waves are produced because of the disturbance and the resulting motions. Natural earthquakes are classified according to the source of the stresses that generate the movement as (tectonic, plutonic or volcanic). [2]

According to many seismologists, “earthquakes don't kill people; buildings do” This is because the majority of earthquake deaths are caused by buildings or other human-made structures collapsing during an earthquake.[3] Therefore, building earthquake-resistant structures—structures that may be vulnerable to damage but are effectively collapse-proof even in the case of the worst earthquake—is a necessary step in any attempts to reduce the impacts of an earthquake. Even when damage cannot be completely avoided, it is workable to minimize it at a relatively low additional cost. Due to their social, economic, and/or financial importance and the potential level of physical risk associated with their destruction, certain structures require higher earthquake resistance than others. [2]

In a tall building in the past decade, performance-based earthquake engineering has evolved into seismic design. Since this breakthrough, structural design engineers may now more easily eliminate the height limitations placed on buildings by prescriptive seismic design codes. There have been major steps over the past decade to create and publish performance-based seismic design guidelines and consensus papers for tall buildings by many organizations, such as the Los Angeles (LATBSDCA), will have access to more specific information on ground motion, design rules, and so on. It allows engineers to create tall buildings and established and published suggested design documents for high-rise structures. Tall Buildings Structural Design Council of Los Angeles (2005, 2008), Structural Engineers Association of Northern California of San Francisco (2007), and CTBUH—Council on Tall Buildings and Urban Habitat (2008). [4]

1.4 SEISMIC RESISTANCE

Engineers endeavour to reinforce the structure and mitigate the effects of earthquake while constructing a seismically building. Since structures are pushed in one direction by the energy released during earthquakes, the technique entails having the building push in the opposing direction. Below are several techniques for making buildings more earthquake-resistant: [5]

1.4.1 Create a Flexible Foundation

Base isolation is a technique used to raise the building's foundation above the ground in order to withstand ground stresses. Base isolation entails erecting a structure on top of flexible steel, rubber, and lead pads. The isolators vibrate when the base shakes during an earthquake, but the building itself doesn't move. As a result, seismic vibrations are successfully absorbed and kept from passing through the building.

Figure 1.1: Flexible Foundation (Retrieved From [05]).

1.4.2 Counter Forces with Damping

a. Vibrational Control Devices

This technique includes inserting dampers between columns and beams at each level of a structure. Each damper comprises of piston heads within a cylinder filled with silicone oil. When there is an earthquake, the structure's vibrational energy is transferred to the pistons, which press on the oil. The power of the vibrations is subsequently dissipated as a result of the energy's transformation into heat.

Figure 1.2: Vibrational Control Devices (Retrieved From [05]).

b. Pendulum Power

Pendulum power, which is mostly employed in skyscrapers, is another typical dampening technique. Engineers do this by suspending a large ball on steel cables that are attached to a hydraulic system at the top of the structure. The ball works as a pendulum and swings in the opposite direction to steady the building when it starts to shake. These characteristics, like damping, are adjusted to coincide with and counteract the movement of the building during an earthquake.

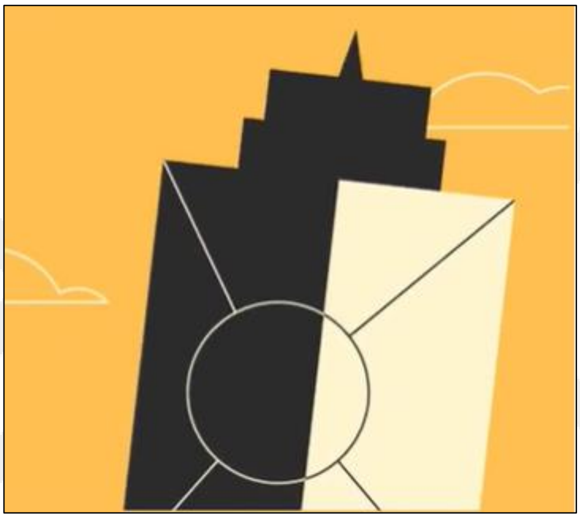


Figure 1.3: Pendulum Power (Retrieved From [05]).

1.4.3 Shield Buildings From Vibrations

Researchers are working with methods that structures may completely deflect and redirect the energy from earthquakes rather than merely counteracting pressures. This invention, known as the "seismic invisibility cloak," entails making a cloak out of plastic and concrete concentric rings and burying them at least three feet below the building's foundation. The ease of passage forces seismic waves to pass through to the outer rings when they enter the rings. They are thus effectively directed away from the structure and dispersed into the earth.

Figure 1.4: Plastic and Concrete Concentric Rings (Retrieved From [05]).

1.4.4 Reinforce the Building's Structure

Buildings must disperse the seismic forces that pass through them in order to prevent collapse. Buildings constructed to resist seismic activity should have some lateral stiffness to prevent excessive swaying during mild earthquakes.[6] Tall structures can have their structural stiffness increased by having a central core, shear walls, a braced frame, tubes, and double tubes. The employment of one or a combination of these methods can significantly boost structural stiffness. A structure's stiffness determines how well it can withstand deformation. The ability of a structure to withstand deformations brought on by applied loads is referred to as structural stiffness.[7]

a. Central Core

Core walls are formed by combining walls and arranging them in the shape of a core in the geometric center of the structure. The foundation is used to build the core wall, which is then elevated up to the structure. Core is utilized to install elevators, stairwells, and step wells, as well as to accommodate services. The core wall is designed to withstand the lateral force imposed on the structure by wind, earthquake, or any other lateral load.[8]

b. Shear Walls

Shear walls are the primary vertical structural components that withstand both gravity and lateral forces. [9] The most frequent loads that shear walls are designed to withstand are wind and seismic loads. Shear wall behavior is affected by the material utilized, wall thickness, wall length, and wall location in the building frame. The height to depth ratio of monolithic shear walls determines whether they are short, squat, or cantilever. The shear walls should be placed at the middle of each side of the building; however, this is rarely possible. Concrete shear walls are among the most dependable types of shear walls. [10]

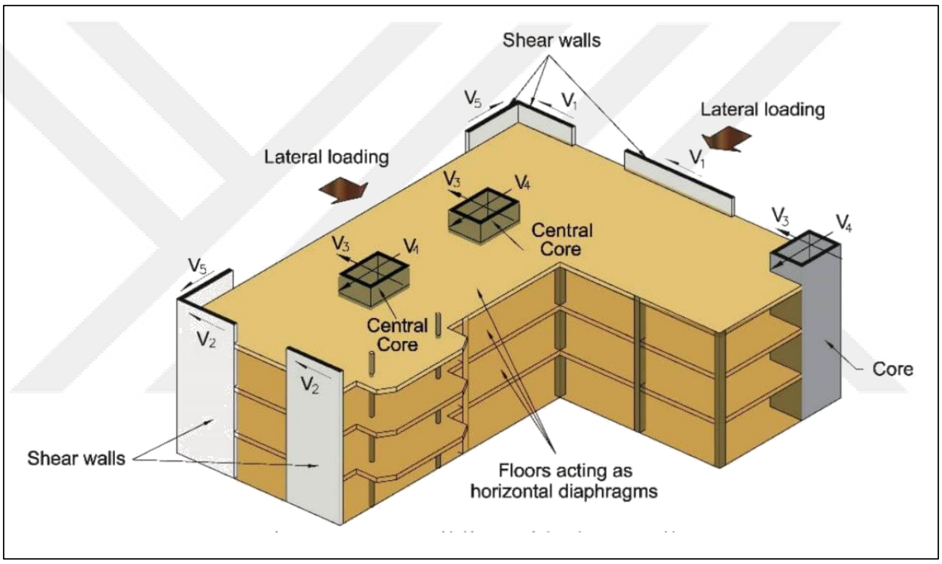


Figure 1.5: Central Core and Shear walls (Retrieved From [Theconstructor.Org]) [10].

The benefits of combining a central core system and a shear wall:[11]

a. Flexural stiffness is significantly increased as compared to the rigid frame and Outrigger system.
b. The most cost-effective solution.
c. Creates a larger, more effective system for decreasing overturning moments in core structures.

c. Braced Frame

A braced frame is a basic construction with bracing that aids in structural rigidity. A braced frame is similar to floor bracing, but it is dependent on the insertion of additional diagonal cross members rather than the rigidity given by the floor system. This system is more difficult to build than the system with floor bracing.

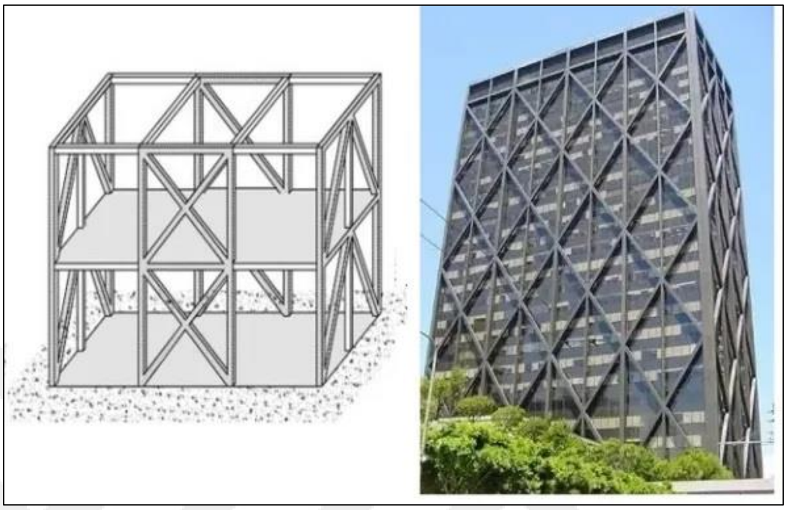


Figure 1.6: Braced Frame (Retrieved From [07]).

d. Tube system

A tube system is composed of two sets of shear walls. The tube system allows the structure to be rigid in all loading directions. The structure would have great torsional resistance. The building's inside is maintained clear. When creating openings in a tube system is required, the openings should be as small as feasible. It functions as a hollow cylinder that is cantilevered perpendicular to the ground, allowing the structure to withstand lateral loads from wind, earthquakes, and other sources.

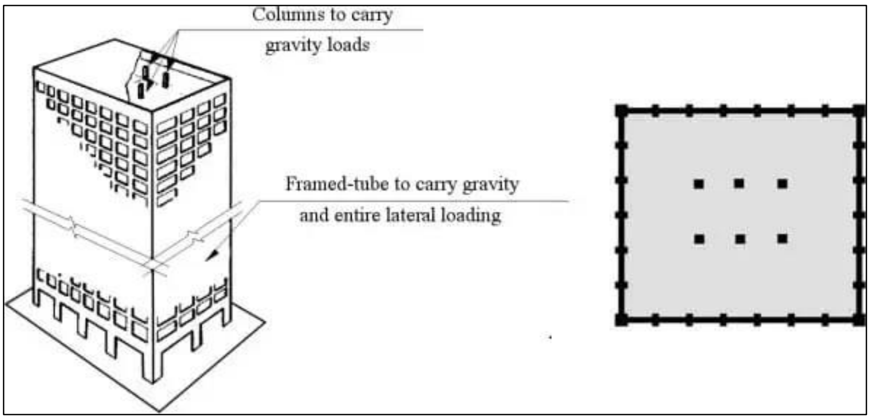


Figure 1.7: Tube System (Retrieved From [07]).

e. Double Tube System

As indicated in the Figure below, a double tube system is a combination of the central core and a tube system. Because of the combination of both methods, the structure is exceptionally rigid. The structure has strong torsional resistance. Window openings in the tube system must be maintained to a minimum. The center core consumes a lot of important space. This method is utilized in high-rise buildings.

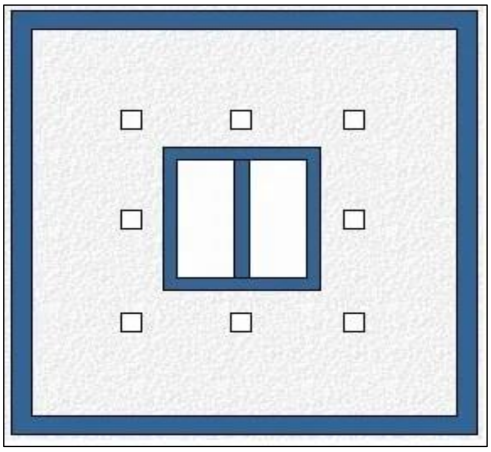


Figure 1.8: Double Tube System (Retrieved From [07]).

2. LITERATURE REVIEW

2.1 DEFINITION OF TALL BUILDINGS

A tall building cannot be defined by its height or number of stories. No one can agree on what qualifies as a "tall structure" or at what specific height, number of floors, or proportion a building may be classified as tall. [12]

According to The Council on Tall Buildings and Urban Habitat's (CTBUH) Criteria for the Defining and Measuring of Tall Buildings, a "tall building" is defined when it's elevated higher than its surroundings, or buildings can appear taller if they are thinner and shorter if they are wider. It also indicates that a building is "supertall" if it is more than 300 meters tall, and it's "mega tall" for a building that is higher than 600 meters. [13]

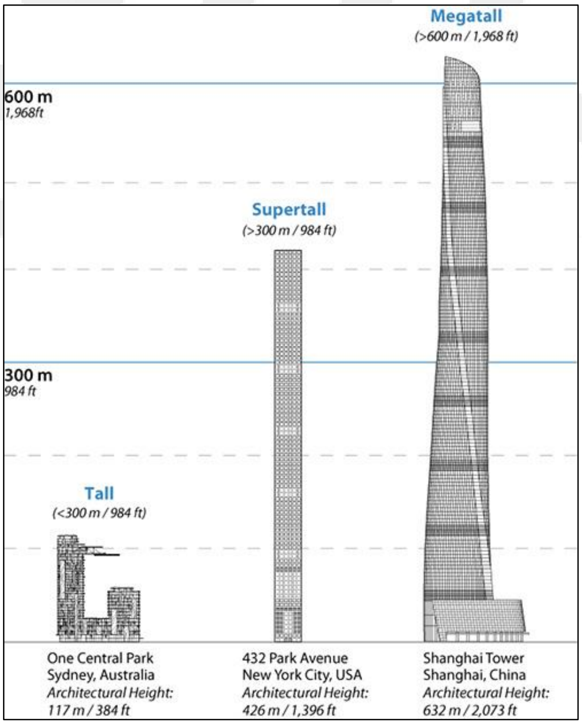


Figure 2.1: Tall, Supertall, and Mega Tall Building (Retrieved From [10]).

"Tall buildings" are defined according to the nature of each country. Therefore, town planning agencies of municipalities, regulatory agencies, standards organizations, and communities frequently develop definitions and guidelines for what high rise buildings are permitted in their control areas.

Case in point, in India by The National Building code of India (2005) defined a tall building as a structure that is more than 16m tall, about 5 stories. [14]

In Canada by the city of Burlington (2017) defined a tall building as a structure that is more than 11 stories. [15]

In London by old oak and park royal development corporation (2021) defined a tall building as a structure that is more than 45 m tall and has 15 stories. [16]

2.2 SHEAR WALL

A shear wall is a structural vertical element that can withstand a combination of shear, moment, and axial force caused by lateral load and gravity load transfer from another structural element. Multi-story buildings are often required to have reinforced concrete walls that contain core shear walls, shear walls, or combine them together.

A structure should be designed with the centroid and mass center of the building in mind. Because the fundamental role of a shear wall is to improve the stiffness for lateral load resistance, its adoption offers a structurally effective method to stiffen a building structural system. [17]

The behavior of a structure during an earthquake is determined by the distribution of weight, stiffness, and strength in both the horizontal and vertical planes of the building.

The structure employs reinforced concrete shear walls to mitigate the effects of earthquakes. This can be used to improve the seismic resistance of a structure. Building structural design for earthquake loading is primarily concerned with structural safety during earthquakes. Especially for tall structures, lateral stiffness to withstand lateral load is critical. The use of shear walls in buildings to achieve stiffness has been shown to be successful and cost efficient. Shear walls are commonly employed in tall buildings to prevent collapse. Shear walls, when placed in opportune places, can constitute an effective lateral force resisting system. [18]

There are several types of shear walls as follows:[19]

a. RC Shear Wall.
b. Plywood shear wall.
c. Mid-ply shear wall.
d. Steel plate shear wall.
e. RC Hollow Concrete Block Masonry Wall.

Shear walls are classified as follows: [19]

a. Simple rectangular types and flanged walls.
b. Coupled shear walls.
c. Rigid frame shear walls.
d. Column-supported shear walls.
e. Core type shear walls.
f. Framed walls with infilled frames.

2.3 RELATED WORKS

K. PRIYANKA et al. (2013): paper on “Dynamic response of high-rise structures under the influence of discrete staggered shear walls” was presented. The impact of various shear wall configurations on the structure is explored, leading to the provision of diagonal and zigzag shear walls in order to evaluate drift in the structure. when regulating the seismic load. The research asserts that the layout of zigzag shear walls is optimal. [20]

P. P. Chandurkar and P. S. Pajgade (2013): paper on “Seismic Analysis of RCC Building with and Without Shear Wall” was presented. The primary goal is to provide a solution for the shear wall position in a multi-story building. The effectiveness of shear walls has been investigated using four different models (framed structure, dual system with shear wall in different positions). They discovered that building a building with a shear wall in a short span at the corner is less expensive than other models. indicated that large shear wall dimensions are ineffective in structures of 10 stories or less than a 10-story building. [21]

R. M. Desai et al. (2016): paper on “Behavior of symmetric and asymmetric structure in high seismic zone” was presented. In order to explore the impact of eccentricity between the center of mass and the center of stiffness, the authors of this work designed three buildings: G+3, G+6, and G+9. They came at the conclusion that symmetrical buildings have a longer time period than asymmetrical buildings using the commercial sap 2000 program to examine low, mid, and high-rise buildings. As height increases, natural time increases. Compared to low and mid-rise buildings, high-rise buildings have a longer time period. The relationship between the time period and the number of storey is extreme. [22]

N. Malviya et al. (2017): a study on the “seismic analysis of high-rise buildings using the IS codes 1893-2002 and 1893-2016” was presented. They used the commercial SAP 2000 software to analyze building that was modelled in accordance with the above two codes. They concluded that new code is more effective than old code, where deflection value decreased in accordance with updated code design. When compared to the old code, shear force and bending moment also display lower values. According to the response spectrum results, new code shows more acceleration versus time. [23]

İ. H. Erkan and T. P. Doğan (2019): paper on “Comparison of equivalent seismic load and response spectrum methods according to TSC 2018 and TSC 2007” was presented. In the analyses performed using the Equivalent Seismic Load Method and Response Spectrum Method specified in TSC 2018 and TSC 2007, the shear force values obtained in the TSC 2018 calculations are lower than the results of the TSC 2007 analyses for both methods, and floor displacement values are higher in TSC 2018 models compared to TSC 2007 results. The fundamental reason for this is the usage of effective stiffness multipliers, which must be determined on the structural system members of buildings to be represented using Design by Strength principles. [24]

S. J. Patel and N. S. Mehta (2020): paper on “Comparison of Shear wall Building and Core wall Building” was presented. In this paper, a ten-story, fifteen-story, and twenty-story structure were examined with different lateral resistances applied with identical structural characteristics and dimensions using the linear static method, and the findings were compared with shear walls, core walls, and concrete frame buildings. As a result of the main difference between the models is the maximum story displacement, or story drift. There is some variety in the outcomes. The core wall in the center has less deformation than other models and is more convenient than the shear wall at the model's edge. [8]

A. Saha (2020): paper on “Seismic Study on Tall Structures with RC Shear Walls: Static and Dynamic Analysis” was presented. In this paper Staad pro was used to analyze a 50-story structure with and without shear walls using equivalent static and response spectrum methods. In compliance with Indian Standard regulations, several loading methods were adopted. Structures with shear walls encounter less moment and lateral force, as well as less base shear, resulting in reduced deflection and a greater moment resisting ability of the corresponding structures. In the case of reinforced concrete constructions without a shear wall, the structure is subjected to greater lateral forces. According to Indian design rules, structural investigations are necessary for safety and serviceability. [25]

3. RESEARCH METHODOLOGY

3.1 CALCULATION OF THE TOTAL WEIGHT OF THE STRUCTURE'S ELEMENTS IN EACH STORY

3.1.1 Slab Weight Due to Self-Weight and Finishing

To obtain the total weight of slabs, the floor slab's due to self-weight and an estimated finishing weight are combined.

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

3.1.2 Columns Weight

The height of the 1st story is 4 m, while the heights of the 2nd to 20th story are 3 m. The weight of the columns on each floor is calculated by v ultiplying the height of the column by the number of columns on the floor and the unit weight of concrete as shown in (Eq 3-2).

$$W_{\text{Column section}} = \text{Column area} \times \text{Height of column} \times \gamma_{concrete} \times \text{Number.} \tag{3.2}$$

Since the column dimensions are changing, there will also be a decrease in the weight of the columns toward the upper levels. Consequently, the clear span of the beams will change.

3.1.3 Beams Weight

Beams' clear spans are variable. Due to the cross-sectional area of the columns, the weight of the beams will increase toward the top floors. We may calculate the weight of the beams by multiplying the clear beam spans by the unit weight of concrete, the cross-sectional area of the beam, and the total number of beams on the floor.

$$W_{\text{Beam Number}} = \text{Beam section Area} \times \text{Length of Span} \times \gamma_{concrete} \times \text{Number.} \tag{3.3}$$

3.1.4 Normal and Core Shear Wall Weight

Determining the weights of the frame shear wall and core on the floors.

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

3.1.5 Exterior Masonry Walls

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

After determining the dead loads acting on the floor, the total floor weight can be computed by applying the following equation:

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

Where:

w: Total story weight.

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 CALCULATION OF DISTRIBUTED LATERAL LOAD (Pi)

Distributed Lateral Load (Pi) can be calculated by using the following equation by assuming base shear force = 1000kN.

$$\mathbf{Pi} = \frac{\mathbf{W_i \times h_{i\,eff}}}{\sum \mathbf{W_i \times h_{i\,eff}}} \times \mathrm{Vt} \qquad (3.7)$$

Where:

W_i: weight of i^{th} story (kN).

$h_{i\,eff}$: effective height of i^{th} story (m).

V_t: Base shear force (kN).

3.3 CALCULATION OF T_i AND M_O.

To found T_i Value:

$$T_i = \sum_j^i \mathbf{P}_i \qquad (3.8)$$

Where:

j: The last story number.

To found M_o Value:

$$M_{o,i} = (T_i \times h_i) + M_{o,i-1} \qquad (3.9)$$

3.4 CALCULATIONS FOR EARTHQUAKE LOAD ACTION ON BOTH DIRECTION

Calculate the Earthquake Load action on structure in both-directions (Y (Case 1), X (Case 2)) with two cases (with considering shear deformation (Case dir.1) and without shear deformation (Case dir.2)) as shown below:

3.4.1 Calculations of Moment of Inertia for Structure's Member (I)

To find moment of inertia (m^4) for rectangular shape. for all member by equation

$$I = \frac{\mathbf{b \times h^3}}{\mathbf{12}} \qquad (3.10)$$

We do expect the structure to behave ductile under the influence of earthquake forces. So, we need to use moment of inertia reduction coefficients for Columns, Beams, and shear wall.

Table 3.2: Moment of Inertia Reduction Coefficients.

Element	Reduction coefficients
Columns	0.70
Beams	0.35
Shear wall	0.50

3.4.2 Calculations of Columns Relative Rigidity (K_c)

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

$$K_c = \frac{\mathbf{I_c}}{\mathbf{h}} \tag{3.11}$$

3.4.3 Calculations of Beams Relative Rigidity (K_b)

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

$$K_b = \frac{\mathbf{I_b}}{\mathbf{L}} \tag{3.12}$$

3.4.4 Calculations Value of K'

For normal and top story:

$$\acute{K} = \frac{\mathbf{k1+k2+k3+k4}}{\mathbf{2\times Kc}} \tag{3.13}$$

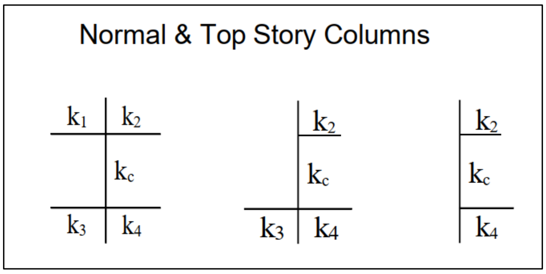


Figure 3.1: Calculation K' (Normal and Top Story).

For bottom story:

$$\acute{K} = \frac{k1+k2}{Kc} \tag{3.14}$$

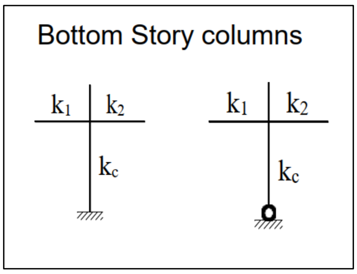


Figure 3.2: Calculation K' (Bottom Story).

3.4.5 Calculations of Rigidity of Columns ($\sum D_c$ and D_c)

Story column:

$$\mathbf{a} = \frac{\acute{K}}{2+\acute{K}} \tag{3.15}$$

Rigid Column-Footing Connection:

$$\mathbf{a} = \frac{0.5+\acute{K}}{2+\acute{K}} \tag{3.16}$$

Pinned Column-Footing Connection:

$$\mathbf{a} = \frac{0.5\times\acute{K}}{1+2\acute{K}} \tag{3.17}$$

By using the following equation to calculate the D_c:

$$D_c = a \times K_c \times 12 \tag{3.18}$$

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

$$\sum D_c = D_{ci} \times \text{Number of columns} \tag{3.19}$$

3.4.6 Calculation of Normal Shear Wall and Core Wall Rectifiers

3.4.6.1 Normal shear walls connect with shear wall

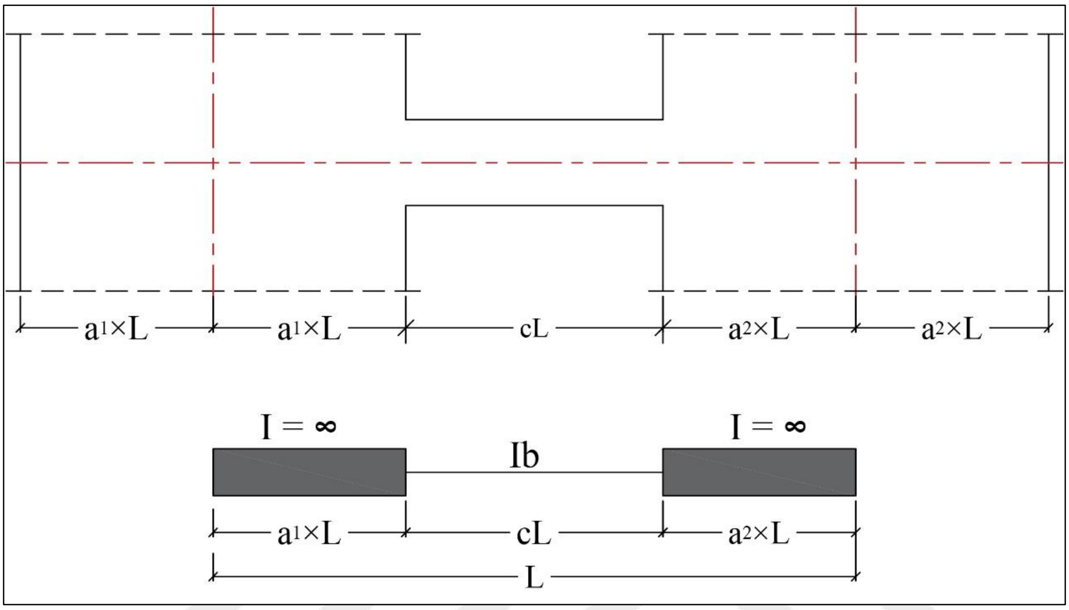


Figure 3.3: Coupled Beams Connected Between Shear Walls.

Shear wall rectifiers (R_A, R_B), These are the moments that occur as a result of the unit rotation of the wall axis. Because the walls are symmetrical therefore ($R_A = R_B$), the equation (3-20) will be utilized for both in case (without shear deformation).

$$R_A = \frac{6 \times E \times I_{cb}}{c^3 \times L} \tag{3.20}$$

Since it is believed that the connecting beams may crack before other elements during an earthquake, the cracked moment of inertia of the connected beams is utilized. The moment of inertia is decreased by half.

$$I_{cb,cr} = I_{cb} \times 0.5 \tag{3.21}$$

Considering the shear deformations in the coupled beams multiply the equation 3-20 with Kβ:

$$R_A = \frac{6 \times E \times I_{cb,cr}}{c^3 \times L} \times K\beta \tag{3.22}$$

Where:

$$K\beta = \frac{1}{1 + 4 \times \beta} \tag{3.23}$$

$$\beta = \frac{3 \times IcouB}{(cl)^2 \times A'} \times \frac{E}{G} \tag{3.24}$$

Where:

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

A'= Shape factor × Area $_{\text{coupled beam}}$

Shape factor of rectangular shape = 5/6

3.4.6.2 Core shear wall and normal shear wall (center) connection

a. Normal shear wall (Center):

For center shear wall it's appropriate to consider these as column widths, which run less than half the length of the shear wall perpendicular to this direction, in case beam stuck in shear wall:

$$b_{col\text{-}w} = B_{\text{(Beam width)}} + 2B_{wall\ \text{(width of shear wall)}} \leq 0.5\ L_w \tag{3.25a}$$

In case beam stuck in shear wall ends:

$$b_{col\text{-}w} = B_{\text{(Beam width)}} + B_{wall\ \text{(width of shear wall)}} \tag{3.25b}$$

Obtain Dc for the considered column with dimension (b_{col} (m) × B_w (m)) by using the procedure of (Muto's method) [26] below:

a. Moment of inertia (I_c).
b. Column Relative Rigidity (K_c).
c. Beam Relative Rigidity (K_b).
d. The Value of (K').
e. The Value of (a).

f. The Column Rigidity (D_{fci}).

g. The Summation of Column Rigidity ($\sum D_{fci}$).

b. Core shear wall connecting with column:

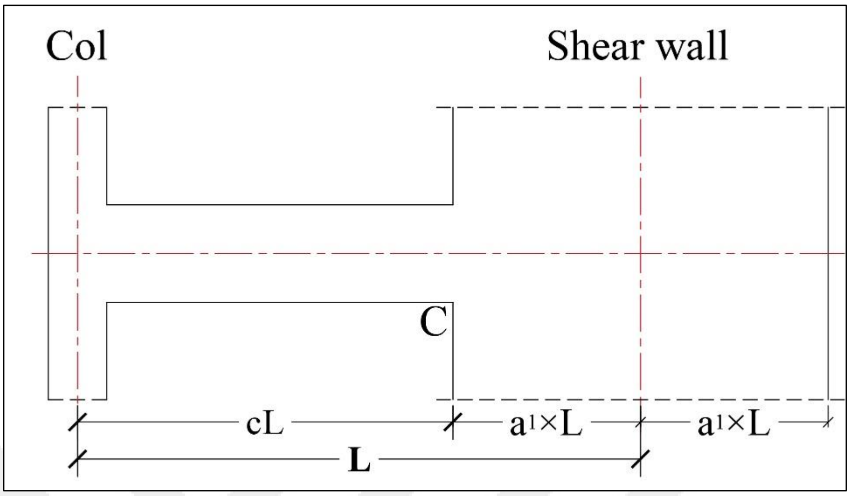


Figure 3.4: Coupled Beams Connected Between Shear Wall and Column.

The rotation of the junction (col) where the column is positioned is critical in this form of connecting beam. The rotation at this point is controlled by the stiffness ratio of the beams and columns that connect. The average stuffiness ratio coefficient, denoted by (r_m), is as follows:

$$\boldsymbol{r_m} = \frac{(r_o + r_u)}{2} \tag{3.26}$$

$$\boldsymbol{r_o} = \frac{K_c}{\sum_{up}(K_c + K_{bfc})} \tag{3.27}$$

$$\boldsymbol{r_u} = \frac{K_c}{\sum_{low}(K_c + K_{bfc})} \tag{3.28}$$

The connecting beam's moment of inertia is increased by the square of the net span coefficient (c); therefore, the beam's fictive rigidity (k_{bf}) is as mentioned below:

$$\boldsymbol{k_{bf}} = \left(\frac{I_{cb}}{c^2}\right) \div \boldsymbol{L_{span}} \tag{3.29}$$

The shear wall rectifier is depending on the r_m coefficient:

$$\text{If } 0 < \mathbf{r_m} < \mathbf{2/3} \rightarrow \mathbf{R_c} = \frac{(\mathbf{3\times E\times I_{b,cr}})}{\mathbf{c^3\times L}} \times [\frac{\mathbf{3}}{\mathbf{2}} \times (3+2 \times c) \times \mathbf{r_m} + 1] \quad (3.30)$$

$$\text{If } \boldsymbol{r_m} > \mathbf{2/3} \quad \rightarrow \mathbf{R_c} = \frac{(\mathbf{6\times E\times I_{b,cr}})}{\mathbf{c^3\times L}} \times [\mathbf{2+c}] \quad (3.31)$$

The equation (3-30 and 3-31) is used to calculate the shear wall rectifier in case (Without shear deformation), Considering the shear deformations in the coupled beams multiply the equation 3-30 or 3-31 (depended on r_m value) with Kβ.

3.4.7 Calculation of Fictive Shear Stiffness D_{FI}

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 (3.32)

Bottom floor: Df1 = Rectifier value × 2

Calculate the Summation of Fictive shear stiffness for shear wall and core in each story:

$$\sum D_{fi} = \text{Fictive shear stiffness} \times \text{number of beams end} \quad (3.33)$$

3.4.8 Calculation of the Summation Rigidity Of Story (∑D)

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:

$$\sum D_i = \sum D_{ci} + \sum D_{fci} + \sum D_{fi} \quad (3.34)$$

3.4.9 Calculations of the Coefficients and Constants of Equation Set

To calculate the coefficients and constants of equation set by using the equation listed below by taking E=1:

$$F_i = \frac{\mathbf{1}}{\sum \mathbf{D}} \tag{3.35}$$

$$f_i = \frac{\boldsymbol{hi}}{\mathbf{6 \times \sum (EIp)i}} \tag{3.36}$$

$$\delta_{i,i-1} = f_{i-1} - F_{i-1} \tag{3.37}$$

$$\delta_{i,i} = 2 \times f_i + 2 \times f_{i-1} + F_i + F_{i-1} \tag{3.38}$$

$$\delta_{i,o} = f_i \, (M_{i+1,o} + 2 \times M_{i,o}) + f_{i-1} \, (M_{i-1,o} + 2 \times M_{i,o}) \tag{3.39}$$

3.4.10 Calculations Of the Matrix Coefficients And Load Vector

From section 3.4.9 above get the value of ($\delta_{i,i-1}$, $\delta_{i,i}$ and $\delta_{i,o}$), and arrange our matrix according to the principle shown in equation (3-42) to find the value of X_i by using the (inverse matrix method) to solve the linear system of equation (3-43):

$$\begin{bmatrix} \delta_{1,1} & \delta_{1,2} & 0 & 0 & 0 & 0 & 0 \\ \delta_{2,1} & \delta_{2,2} & \delta_{2,3} & 0 & 0 & 0 & 0 \\ 0 & \delta_{3,2} & . & .. & 0 & 0 & 0 \\ 0 & 0 & .. & . & .. & 0 & 0 \\ 0 & 0 & 0 & .. & . & \delta_{n-2,n-1} & 0 \\ 0 & 0 & 0 & 0 & \delta_{n-1,n-2} & \delta_{n-1,n-1} & \delta_{n-1,n} \\ 0 & 0 & 0 & 0 & 0 & \delta_{n,n-1} & \delta_{n,n} \end{bmatrix} \times \begin{Bmatrix} X_1 \\ X_2 \\ . \\ X_i \\ . \\ X_{n-1} \\ X_n \end{Bmatrix} = \begin{Bmatrix} -\delta_{1,0} \\ -\delta_{2,0} \\ . \\ -\delta_{i,0} \\ . \\ -\delta_{n-1,0} \\ -\delta_{n,0} \end{Bmatrix} \tag{3.40}$$

$$[\delta] \times \{X\} = \{-\delta_o\} \tag{3.41}$$

$$\{X\} = \{-\delta_o\} \times [\delta]^{-1} \tag{3.42}$$

Where:

$[\delta]$: Coefficients-Band Matrix.

$\{X\}$: Unknown Vector.

$\{\delta_o\}$: Coefficient Vector.

3.4.11 Calculations of the Moment Superposition and Distribution to The Core, Shear walls

The moments in the shear walls may be obtained at the floor slab level by superimposing them with the moments of the isostatic system.

$$M_i = M_{o,i} + X_i \tag{3.43}$$

The moment acting on each shear wall or core wall is distributed according to the stiffness of the wall since the measured moment is the sum of all the moments of all the shear walls on the relevant floor of the building. For instance, the following formula used to calculate the bending moment for shear wall (e) at level (i):

$$\mathbf{M_{e,i}} = \mathbf{M_i} \times \frac{\mathbf{(EIp)_{e,i}}}{\sum \mathbf{(EIp)_i}} \tag{3.44}$$

3.4.12 Calculations of Shear Forces for Columns, shear wall and Core

The total shear force acting on the frames is calculated as the ratio of the story height of the difference between the moments X_{i+1} and X_i in the (i+1) and (i) story.

$$\sum \mathbf{T}_{\boldsymbol{i}} = \frac{\Delta\, \mathbf{X}_{\boldsymbol{i}}}{\boldsymbol{h_i}} = \frac{\mathbf{X}_{\boldsymbol{i+1}} - \mathbf{X}_{\boldsymbol{i}}}{\boldsymbol{h_i}} \tag{3.45}$$

The shear force (T_i) on each element of the floor (i) can be determined using distributing the total shear force according to the ratio of element rigidity ($D_{j,i}/\sum D_i$) value.

$$\mathbf{T_i} = \sum \mathbf{T_i} \times \frac{\mathbf{D_{j,i}}}{\sum \mathbf{D_i}} \tag{3.46}$$

Where:

j: represent to structure element (column, shear wall and core wall).

3.4.13 Calculations Of The Coupled Beam End Moments And Corrected The Shear Wall Moment

The connecting beams end moments (Top moment ($M_{i,o}$) and Bottom moment ($M_{i,u}$)) in floor (i) can be calculate using the equation below depended on the floor position:

Top floor:

$$\mathbf{M_{n,o}} = \frac{2}{3} \times \mathbf{T_{fn}} \times \mathbf{h_n} \tag{3.47}$$

$$\mathbf{M_{n,u}} = \frac{1}{3} \times \mathbf{T_{fn}} \times \mathbf{h_n} \tag{3.48}$$

Intermediate floor and bottom floor:

$$\mathbf{M_{i,o}} = \mathbf{M_{i,u}} + \mathbf{M_{i+1,u}} \tag{3.49}$$

$$\mathbf{M_{i,u}} = \frac{1}{2} \times \mathbf{T_{fi}} \times \mathbf{h_i} \tag{3.50}$$

To correct the shear wall and core wall moment at the top and bottom of the floor (i) by using the equation below:

Corrected moment at top of the floor (i):

$$M_{TOP,i} = \text{Moment of shear wall}_{(i+1)} - \text{coupled beam moment}_{(i)} \tag{3.51}$$

Corrected moment at bottom of the floor (i):

$$M_{BOTTOM,i} = \text{Moment of shear wall}_{(i)} + \text{coupled beam moment}_{(i)} \tag{3.52}$$

3.4.14 Calculation of The First Natural Time-Period ($T_{1,I}$) by using Rayleigh Method

The periods will be in the i-direction since the base shear force operates in the i-direction of the structure. The shear forces operating on the frames and the total ∑D must be used by converting them to their true values. E = 1 was previously used. therefore, (E/h_i^2) values are multiplied by ∑D, using the modulus of elasticity of concrete depended on concrete class from (TS 500).

To calculate the 1st natural time period with Rayleigh method by using the equation listed below:

$$\sum D_{ij} = \frac{E_c}{h^2} \times \sum D_i \tag{3.53}$$

$$\delta_i = \frac{\sum T_i}{\sum D_{i,j}} \tag{3.54}$$

$$d_i = \sum \delta i \tag{3.55}$$

$$M_i = \frac{W_i}{g} \tag{3.56}$$

$$M_i d_i^2 = M_i \times d_i^2 \tag{3.57}$$

$$P_i d_i = P_i \times d_i \tag{3.58}$$

$$\omega_1^2 = \frac{\sum Pidi}{\sum Midi^2} \tag{3.59}$$

$$T_1 = \frac{2\pi}{\omega_1} \tag{3.60}$$

Where:

g = 9.81 (m/sec^2).

3.4.15 Calculation of The Real Base Shear Force

Calculate the real base shear force for structure depended on the Characteristic of seismic that listed below according to (TBC 2007) [27]:

i. Building location in seismic zone.

The seismic zones are the first, second, third, and fourth seismic zones.

ii. Effective Ground Acceleration Coefficient (A_o).

Table 3.3: Effective Ground Acceleration Coefficient (Ao) (TBC 2007- table 2.2).

Seismic Zone	Ao
1	0.40
2	0.30
3	0.20

iii. Building Importance Factor (I) For Office building.

Table 3.4: Building Importance Factor (I) (TBC 2007- table 2.3).

Purpose of Occupancy or Type of Building	**Importance Factor (I)i**
i. 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
ii. 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
iii. Intensively but short-term occupied buildings Sport facilities, cinema, theatre, and concert halls, etc.	1.2
Buildings other than above-defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

iv. Local Site Class.

The Local Site Class are Z1, Z2, Z3 and Z4.

v. Spectrum Coefficient.

The Spectrum Coefficient, $S_{(T)}$:

$$S_{(T)} = 1 + 1.5 \times \frac{T}{T_A} \quad (0 \leq T \leq T_A) \tag{3.61}$$

$$S_{(T)} = 2.5 \quad (T_A < T \leq T_B) \tag{3.62}$$

$$S_{(T)} = 2.5 \times \left(\frac{T_B}{T}\right)^{0.8} \quad (T_B < T) \tag{3.63}$$

Spectrum Characteristic Periods, T_A and T_B are specified in Table 3-5, depending on Local Site Classes.

Table 3.5: Spectrum Characteristic Periods (T_A, T_B) (TBC 2007- Table 2.4).

Local Site Class	**TA (second)**	**TB (second)**
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

vi. Structural Behavior Factors (R).

$$\mathrm{Ra(T)} = 1.5 + (\mathrm{R} - 1.5) \times \frac{T}{T_A} \qquad (0 \leq T \leq T_A) \qquad (3.64)$$

$$\mathbf{Ra(T) = R} \qquad (T_A < T) \qquad (3.65)$$

Table 3-6: Structural Behavior Factors (R) (TBC 2007- Part of Table 2.5).

BUILDING STRUCTURAL SYSTEM For CAST-IN-SITU REINFORCED CONCRETE BUILDINGS	**Nominal Ductility Level**	**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**

To calculate the base shear force by using the equation below:

$$A_{(T)} = A_o \times I \times S_{(T)} \times g \qquad (3.66)$$

Total Equivalent Seismic Load (base shear), V_t, Appling on the building in earthquake direction:

$$V_t = \frac{\sum \mathbf{Mi} \times \boldsymbol{A_{(T)}}}{\boldsymbol{R_{a(T)}}} \qquad (3.67)$$

Base Shear Force In case of using Erzincan 1992 N-S chart:

$$V_{t2} = Vt \times ((Sa/a_{max})/ S_{(T)}) \qquad (3.68)$$

By using Erzincan 1992 N-S chart, intersect the T-period value with Damping ratio (ξ=5%) to obtain the (Sa/a_{max}) value.

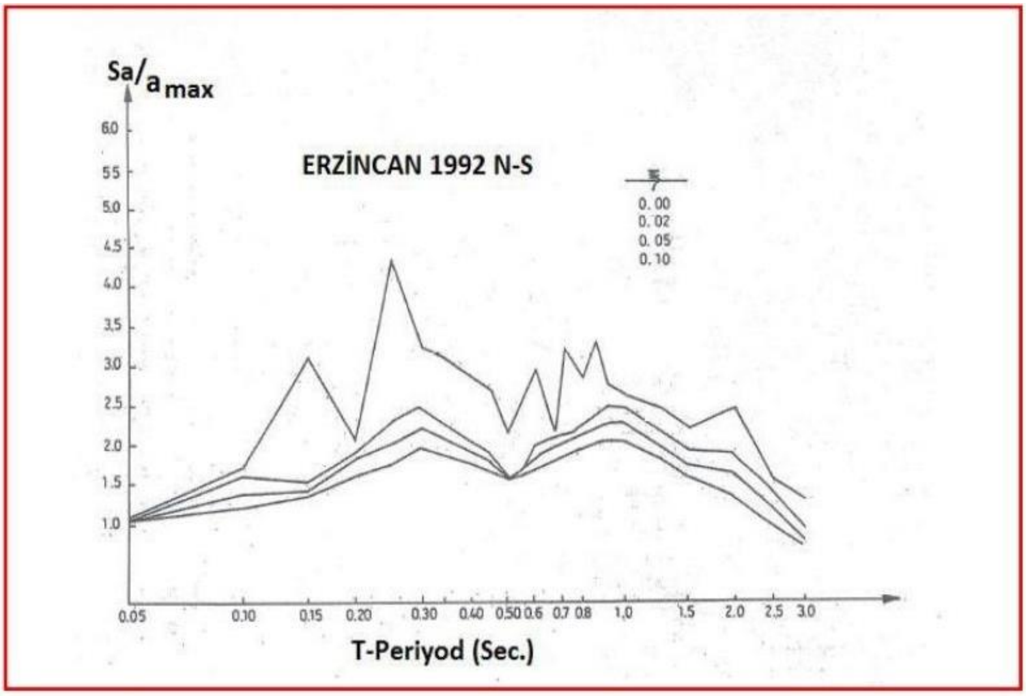


Figure 3.5: Erzincan 1992 N-S Chart.

3.4.16 Correcting The Moment, Shear force, Displacement and Drift

To correct the Values of Moment, Shear force, Displacement and Drift by multiplied its by Vt_2 / Vt_1 ratio for (Case 1.1-With shear deformation) and for (Case 1.2-Without shear deformation) In both-directions.

Check Maximum drift according to (Specification for Buildings to be Built in Seismic Zones 2007 item 2.10.1.3).

$$\text{Maximum drift} = \frac{\boldsymbol{\delta} \times \frac{\mathbf{V_{t2}}}{\mathbf{V_{t1}}} \times \mathbf{R}}{\mathbf{h_i}} \leq 0.02 \tag{3.69}$$

Check Maximum Displacement:

$$\Delta_{\text{max-all}} = 0.002 \times H_B \tag{3.70}$$

Where:

H_B: Building height.

3.4.17 The Second Order Indicator Value $\theta_{II,i}$

To calculate the value of the second-order indicator $\theta_{II,I}$, for each floor facing the earthquake's direction, we use the equation (Eq 3-68) from (TBEC 2018 eq: 4.35).[28]

$$\theta_{II,I} = \frac{\boldsymbol{\delta_i} \times \sum_{k=i}^{n} \mathbf{W_k}}{\mathbf{V_i} \times \mathbf{h_i}} \tag{3.71}$$

Where:

W_k: The floor weights which calculated by sum the weight from the top until the i^{th} floor.

V_i: The real shear forces calculated as the result of the correction.

The $\theta_{II,i}$ values for a 20-story building will be calculate using the Rayleigh method mentioned in the previous section (3.4.14) and checked with the limitation of $\theta_{II,i,max}$ as shown in (Eq 3-69), calculations performed in accordance with the 2018 Turkish Building Earthquake Code.

$$\theta_{II} \leq \mathbf{0.12}\ \frac{D}{C_h \times R} \tag{3.72}$$

Where:

C_h: A coefficient defined depending on the non-linear hysteretic behavior of the structural system, for reinforced concrete buildings C_h=0.5, and C_h=1 for other building types.

a. R: Structural System Behavior Coefficient given in table 3.6.
b. D: Over-Strength Coefficient given in TBEC 2018 table 4.1.

Second order effects are not to be taken into account in the calculation of internal forces based on the design if the maximum value of theta values determined for all floors meets the condition of $\theta_{II,i,max}$ in relation to the value specified in equation (Eq 3-69) from (TBEC eq: 4.36). The local second order effects in the relevant reinforced concrete and steel can therefore be considered in the element design in accordance with the specifications.

3.4.18 Comparing The Result

Compare the result (Maximum Moment (kNm), Shear force for columns (kN), Maximum Deflection Δmax (m), $\sum D$ (dm4/m) and time-period (sec)) of each direction separately.

3.5 LOAD COMBINATIONS

Superpose the total moment of shear wall, the shear force for all stories and displacement of 20^{th} story in Y-dir. with X-dir. for two cases (with shear deformation and without shear deformations) after correcting it by Vt_2/Vt_1 ratio by using load combination mention below:

a. Ex + 0.3 Ey
b. Ex - 0.3 Ey
c. - Ex + 0.3 Ey
d. - Ex - 0.3 Ey
e. 0.3 Ex + Ey
f. 0.3 Ex – Ey
g. - 0.3 Ex + Ey
h. - 0.3 Ex – Ey

4. CASE STUDY ANALYSIS AND RESULTS

4.1 BUILDING CHARACTERISTIC

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

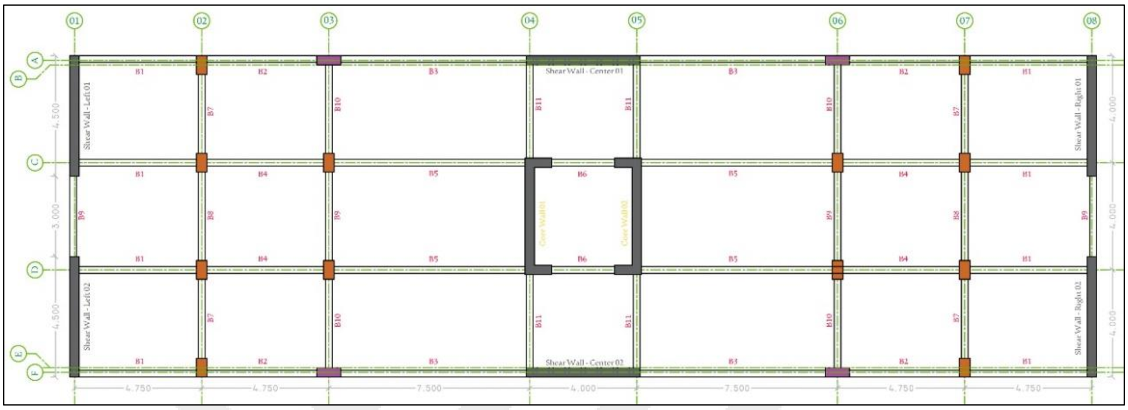


Figure 4.1: Building Plan**.**

a. Building dimensions = 38 m × 12 m

b. Total number of stories = 20 stories

c. Story Height: 1st floor = 4m, 2nd to 20th =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 1st to 5th floor

   iii. C35×70 (cm): in 6th to 10th floor

   iv. C35×60 (cm): in 11st to 20th floor

f. Beams:

   v. Normal Beams: 25 cm × 60 cm

   vi. Coupled Beams: 25 cm × 70 cm

g. Shear Wall width: 35 cm

h. Core Wall width: 35 cm

i. Finishing material thickness = 8 cm

j. Live load for office buildings = 3.5 kN/m2

k. Concrete Strength: C35

l. Modulus of Elasticity for concrete = 33000 N/mm^2

m. Steel Grade: S420

n. γ concrete = 25 kN/m^3

o. γ finishing = 20 kN/m^3

4.2 CALCULATION OF THE TOTAL WEIGHT OF THE STRUCTURE'S ELEMENTS IN EACH STORY

4.2.1 Slab Weight Due To Self-Weight And Finishing

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

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

b. W_{slab} = (Plan area – $SW_{section\ area}$) × Thickness × $\gamma_{concrete}$

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

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

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

4.2.2 Columns Weight

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

a. Weight of Column 90×35 cm.

i. Weight of columns in 1st story.

$$W_{C90\times35} = (0.9\ m \times 0.35\ m) \times 3.85\ m \times 25\ kN/m^3 \times 4 = 121.28\ kN.$$

ii. Weight of columns in 2nd to 20th Story.

$$W_{C90\times35} = (0.9\ m \times 0.35m) \times 2.85\ m \times 25\ kN/m^3 \times 4 = 89.78\ kN.$$

b. Weight of Column 40×70 cm.

i. Weight of columns in 1st story.

$$W_{C40\times70} = (0.4\ m \times 0.7\ m) \times 3.85\ m \times 25\ kN/m^3 \times 12 = 323.4\ kN.$$

ii. Weight of columns in 2^{nd} to 5^{th} Story.

$W_{C40\times70} = (0.4\ m \times 0.7\ m) \times 2.85\ m \times 25\ kN/m^3 \times 12 = 239.4\ kN.$

c. Weight of Column 35×70 cm in 6^{th} to 10^{th} story.

$W_{C35\times70} = (0.7\ m \times 0.35\ m) \times 2.85\ m \times 25\ kN/m^3 \times 12 = 209.48\ kN.$

d. Weight of Column 35×60 cm in 11^{th} to 20^{th} story.

$W_{C35\times60} = (0.6\ m \times 0.35\ m) \times 2.85\ m \times 25\ kN/m^3 \times 12 = 179.55\ kN.$

4.2.3 Beams Weight

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

a. Weight of Beams in 1^{st} to 5^{th} Story.

i. $W_{B1} = (0.25 \times 0.45) \times 4.375 \times 25 \times 8 = 98.44$ kN.
ii. $W_{B2} = (0.25 \times 0.45) \times 4.1 \times 25 \times 4 = 46.13$ kN.
iii. $W_{B3} = (0.25 \times 0.55) \times 6.925 \times 25 \times 4 = 95.22$ kN.
iv. $W_{B4} = (0.25 \times 0.45) \times 4.35 \times 25 \times 4 = 48.94$ kN.
v. $W_{B5} = (0.25 \times 0.45) \times 7.125 \times 25 \times 4 = 80.16$ kN.
vi. $W_{B6} = (0.25 \times 0.55) \times 2.00 \times 25 \times 2 =13.75$ kN.
vii. $W_{B7} = (0.25 \times 0.45) \times 2.95 \times 25 \times 4 = 33.19$ kN.
viii. $W_{B8} = (0.25 \times 0.45) \times 3.3 \times 25 \times 4 = 37.13$ kN.
ix. $W_{B9} = (0.25 \times 0.55) \times 3.0 \times 25 \times 2 = 20.63$ kN.
x. $W_{B10} = (0.25 \times 0.45) \times 3.3 \times 25 \times 4 = 37.13$ kN.
xi. $W_{B11} = (0.25 \times 0.55) \times 3.475 \times 25 \times 4 = 47.78$ kN.
xii. Total Weight of Beams in 1^{st} … 5^{th} Story = 558.47 kN.

b. Weight of Beams in 6^{th} to 10^{th} Story.

i. $W_{B1} = (0.25 \times 0.45) \times 4.4 \times 25 \times 8 = 99.00$ kN.
ii. $W_{B2} = (0.25 \times 0.45) \times 4.125 \times 25 \times 4 = 46.41$ kN.
iii. $W_{B3} = (0.25 \times 0.55) \times 6.925 \times 25 \times 4 = 95.22$ kN.
iv. $W_{B4} = (0.25 \times 0.45) \times 4.4 \times 25 \times 4 = 49.5$ kN.
v. $W_{B5} = (0.25 \times 0.45) \times 7.15 \times 25 \times 4 = 80.44$ kN.

vi. $W_{B6} = (0.25 \times 0.55) \times 2.00 \times 25 \times 2 = 13.75$ kN.

vii. $W_{B7} = (0.25 \times 0.45) \times 2.95 \times 25 \times 4 = 33.19$ kN.

viii. $W_{B8} = (0.25 \times 0.45) \times 3.3 \times 25 \times 4 = 37.13$ kN.

ix. $W_{B9} = (0.25 \times 0.55) \times 3.0 \times 25 \times 2 = 20.63$ kN.

x. $W_{B10} = (0.25 \times 0.45) \times 3.3 \times 25 \times 4 = 37.13$ kN.

xi. $W_{B11} = (0.25 \times 0.55) \times 3.475 \times 25 \times 4 = 47.78$ kN.

xii. Total Weight of Beams in 6^{th} … 10^{th} Story = 560.16 kN.

c. Weight of Beams in 11^{th} to 20^{th} Story.

i. $W_{B1} = (0.25 \times 0.45) \times 4.4 \times 25 \times 8 = 99.22$ kN.

ii. $W_{B2} = (0.25 \times 0.45) \times 4.125 \times 25 \times 4 = 46.41$ kN.

iii. $W_{B3} = (0.25 \times 0.55) \times 6.925 \times 25 \times 4 = 95.22$ kN.

iv. $W_{B4} = (0.25 \times 0.45) \times 4.4 \times 25 \times 4 = 49.5$ kN.

v. $W_{B5} = (0.25 \times 0.45) \times 7.15 \times 25 \times 4 = 80.44$ kN.

vi. $W_{B6} = (0.25 \times 0.55) \times 2.00 \times 25 \times 2 = 13.75$ kN.

vii. $W_{B7} = (0.25 \times 0.45) \times 3.1 \times 25 \times 4 = 34.88$ kN.

viii. $W_{B8} = (0.25 \times 0.45) \times 3.4 \times 25 \times 4 = 38.25$ kN.

ix. $W_{B9} = (0.25 \times 0.55) \times 3.0 \times 25 \times 2 = 20.63$ kN.

x. $W_{B10} = (0.25 \times 0.45) \times 3.35 \times 25 \times 4 = 37.69$ kN.

xi. $W_{B11} = (0.25 \times 0.55) \times 3.475 \times 25 \times 4 = 47.78$ kN.

4.2.4 Normal Shear Wall Weight

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

a. Weight of Right and Left Shear wall.

i. $W_{SW\text{-Right Left in 1st story}} = (4.5 \times 0.35) \times 4 \times 25 \times 4 = 630$ kN.

ii. $W_{SW\text{-Right \& Left in 2nd … 20th story}} = (4.5 \times 0.35) \times 3 \times 25 \times 4 = 472.5$ kN.

b. Weight of Center Shear wall.

i. $W_{SW\text{-center in 1st story}} = (4 \times 0.35) \times 4 \times 25 \times 2 = 280$ kN.

ii. $W_{SW\text{-center in 2nd … 20th story}} = (4 \times 0.35) \times 3 \times 25 \times 2 = 210$ kN.

4.2.5 Core Shear Wall Weight

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

i. $W_{Core,1st\ story} = ((1 \times 4.35) - (0.65 \times 3.3)) \times 4 \times 25 \times 2 = 441$ kN.

ii. $W_{Core,2nd\ to\ 20th\ story} = ((1\times4.35) - (0.65 \times 3.3)) \times 3\times25\times2 = 330.75$ kN.

4.2.6 Exterior Masonry Walls

a. 1st Floor:

$W_1 = (4.375 \times 3.4) \times 2.5 \times 4 = 148.75$ kN.

$W_2 = (4.1 \times 3.4) \times 2.5 \times 4 = 139.40$ kN.

$W_3 = (6.925 \times 3.3) \times 2.5 \times 4 = 228.53$ kN.

$W_4 = (3 \times 3.3) \times 2.5 \times 2 = 49.5$ kN.

Total wall weight: 566.18 kN.

b. 2nd to 5th Floor:

$W_1 = (4.375 \times 2.4) \times 2.5 \times 4 = 105.00$ kN.

$W_2 = (4.1 \times 2.4) \times 2.5 \times 4 = 98.40$ kN.

$W_3 = (6.925 \times 2.3) \times 2.5 \times 4 = 159.28$ kN.

$W_4 = (3 \times 2.3) \times 2.5 \times 2 = 34.50$ kN.

Total wall weight: 397.18 kN.

c. 6th to 20th Floor:

$W_1 = (4.4 \times 2.4) \times 2.5 \times 4 = 105.60$ kN.

$W_2 = (4.125 \times 2.4) \times 2.5 \times 4 = 99.00$ kN.

$W_3 = (6.925 \times 2.3) \times 2.5 \times 4 = 159.28$ kN.

$W_4 = (3 \times 2.3) \times 2.5 \times 2 = 34.50$ kN.

Total wall weight: 398.38 kN.

Table 4.1: Weight Of Structure Element Due To Self-Weight And Finishing Weight.

<table>
<tr><th>Story</th><th>Slabs</th><th>Columns</th><th>Beams</th><th>Wall</th><th>Shear Wall</th><th>Core Wall</th><th>W Total</th></tr>
<tr><td>11 to 20</td><td rowspan="4">2342.64</td><td>269.325</td><td>563.53</td><td>398.38</td><td rowspan="3">682.5</td><td rowspan="3">330.75</td><td>4587.13</td></tr>
<tr><td>6 to 10</td><td>299.25</td><td>560.16</td><td>398.38</td><td>4613.68</td></tr>
<tr><td>2 to 5</td><td>329.175</td><td rowspan="2">558.47</td><td>397.18</td><td>4640.71</td></tr>
<tr><td>1</td><td>444.675</td><td>566.18</td><td>910</td><td>441</td><td>5262.96</td></tr>
</table>

After determining the dead loads acting on the floor, the total floor weight can be computed by applying the equation (3-6):

$w_i = g_i + nq_i$

w_1= 5262.96 + (0.3 × 3.5 × 12 × 38) = 5741.76 kN

$w_{2\text{-}5}$= 4640.71 + (0.3 × 3.5 × 12 × 38) = 5119.51 kN

$w_{6\text{-}10}$= 4613.68 + (0.3 × 3.5 × 12 × 38) = 5123.98 kN

$w_{11\text{-}19}$= 4587.13 + (0.3 × 3.5 × 12 × 38) = 5097.43 kN

w_{20} = 4587.13 kN.

Table 4.2: Total Floor Weights.

Story	Floor Weight Wi (kN)
20	4587.13
11 to 19	5065.93
6 to 10	5092.48
2 to 5	5119.51
1	5741.76

4.3 CALCULATION OF DISTRIBUTED LATERAL LOAD (PI)

Distributed Lateral Load (Pi) can be calculated by using the equation 3-7 by assuming base shear force = 1000kN.

P_{20} = (279814.63/3272131.58) × 1000 = 85.51 kN.

The Pi values are calculated for all floor levels and can be found in table 4-3.

Table 4.3: Distributed Lateral Load Pi.

Story	hi (m)	hi effective (m)	Wi (kN)	Wi × hi Effective (kNm)	Pi (kN)
20	3	61	4587.13	279814.63	85.51
19	3	58	5065.93	293823.65	89.80
18	3	55	5065.93	278625.88	85.15
17	3	52	5065.93	263428.10	80.51
16	3	49	5065.93	248230.33	75.86
15	3	46	5065.93	233032.55	71.22
14	3	43	5065.93	217834.78	66.57
13	3	40	5065.93	202637.00	61.93
12	3	37	5065.93	187439.23	57.28
11	3	34	5065.93	172241.45	52.64
10	3	31	5092.48	157866.73	48.25
9	3	28	5092.48	142589.30	43.58
8	3	25	5092.48	127311.88	38.91
7	3	22	5092.48	112034.45	34.24
6	3	19	5092.48	96757.03	29.57
5	3	16	5119.51	81912.20	25.03
4	3	13	5119.51	66553.66	20.34
3	3	10	5119.51	51195.13	15.65
2	3	7	5119.51	35836.59	10.95
1	4	4	5741.76	22967.05	7.02

4.4 CALCULATION OF T_I AND M_O

To found T_i Value:

Following with these equations

$T_{20} = P_{20} = 85.51$ kN.

$T_{19} = T_{20} + P_{19} = 85.51 + 89.80 = 175.31$ kN.

To found M_o Value:

And also we could continue to following equations

$M_{o,20} = T_{20} \times h_{20} = 85.51 \times 3 = 256.54$ kN.

$M_{o,19} = M_{o,20} + (T_{19} \times h_{19}) = 256.54 + (175.31 \times 3) = 782.47$ kN.

The T_i and M_o values are calculated for all floor levels and can be found in table 4-4.

Table 4.4: Value of T_i and M_o

Story	hi (m)	Pi (kN)	Ti (kN)	Mo (kNm)
20	3	85.51	85.51	256.54
19	3	89.80	175.31	782.47
18	3	85.15	260.46	1563.86
17	3	80.51	340.97	2586.76
16	3	75.86	416.83	3837.25
15	3	71.22	488.05	5301.39
14	3	66.57	554.62	6965.25
13	3	61.93	616.55	8814.90
12	3	57.28	673.83	10836.39
11	3	52.64	726.47	13015.81
10	3	48.25	774.72	15339.96
9	3	43.58	818.29	17794.84
8	3	38.91	857.20	20366.44
7	3	34.24	891.44	23040.76
6	3	29.57	921.01	25803.79
5	3	25.03	946.04	28641.92
4	3	20.34	966.38	31541.07
3	3	15.65	982.03	34487.16
2	3	10.95	992.98	37466.10
1	4	7.02	1000.00	41466.10

4.5 CALCULATIONS FOR EARTHQUAKE LOAD ACTION ON Y-DIRECTION (CASE 1)

4.5.1 Calculations of Moment of Inertia for Structure's Member (I)

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

$I = b \times h^3/12 = 0.25 \times 0.60^3 / 12 = 0.0045\ m^4$.

Total I = 0.0045 × 0.35 (Inertia moment reducing coefficient) = 0.0016 m^4.

Core Shear wall: b_1=1.00 m, h_1=4.35 m, b_2=0.65 m, h_2=3.65.

$I_1 = b_1 \times h_1^3/12 = 1.00 \times 4.35^3 / 12 = 6.86$ m^4.

$I_2 = b_2 \times h_2^3/12 = 0.65 \times 3.65^3 / 12 = 1.95$ m^4.

Total $I = I_1 - I_2 = (6.86 - 1.95) \times 0.50$ (Inertia moment reducing coefficient) = 2.11 m^4.

The Moment of Inertia for Structure’s Member values are calculated and can be found in table 4-5.

Table 4.5: Moment of Inertia of Structure’s Element (I).

Member		**b (m)**	**h (m)**	**bh3/12 (m4)**	**Total I (m4)**
Beam	Normal	0.25	0.6	0.0045	0.0016
	Coupled	0.25	0.7	0.0071	0.0025
	Cracked	0.5 × Ic.b		0.0036	0.0013
Columns	90×35	0.9	0.35	0.0032	0.0023
	40×70	0.4	0.7	0.0114	0.0080
	35×70	0.35	0.7	0.0100	0.0070
	35×60	0.35	0.6	0.0063	0.0044
Normal Shear Wall	Right and left	0.35	4.5	2.6578	1.3289
	Center	4	0.35	0.0143	0.0071
Core Shear Wall	A1	1	4.35	6.8594	2.1127
	A2	0.65	3.65	2.6340	

4.5.2 Calculations of Columns Relative Rigidity (K_c)

$K_{c\ (C90\times35)\ 1st\ Floor} = I_{c,C90\times35} / h_1 = 22.51 / 4 = 5.63$ dm^4/m.

The Kc values are calculated for all columns and can be found in table 4-6.

Table 4.6: Columns Relative Rigidity (Kc).

<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">22.51</td><td rowspan="2">-</td><td>-</td><td>44.10</td><td>7.50</td><td rowspan="2">-</td><td>-</td><td>14.70</td></tr>
<tr><td>6 to 10</td><td>3</td><td>70.03</td><td rowspan="3">-</td><td>7.50</td><td>23.34</td><td rowspan="3">-</td></tr>
<tr><td>2 to 5</td><td>3</td><td rowspan="2">80.03</td><td rowspan="2">-</td><td>7.50</td><td>26.68</td><td rowspan="2">-</td></tr>
<tr><td>1</td><td>4</td><td>5.63</td><td>20.01</td></tr>
</table>

4.5.3 Calculations of Beams Relative Rigidity (K_b)

$K_{b\ (B7)1st\ to\ 10th\ Floor} = I_{b7} / L = 15.75 / 2.95 = 5.34\ dm^4/m$.

The K_b values are calculated for all beams and can be found in table 4-7.

Table 4.7: Beams Relative Rigidity (Kb).

Floor	Ib (dm^4)	Length of span (m)			Kb (dm^4/m)		
		B7	B10	B8	B7	B10	B8
11 to 20	15.75	3.1	3.35	3.4	5.08	4.70	4.63
1 to 10		2.95	3.3	3.3	5.34	4.77	4.77

4.5.4 Calculations Value of K'

K' in 1st Floor (Axes 2,7 Ext) = $K_b / K_c = 5.34 / 20.01 = 0.267$.

K' in 10th Floor (Axes 2,7 Int) = $K_1 + K_2 + K_3 + K_4 / 2 \times Kc$

$$= 5.34 + 4.77 + 5.34 + 4.77 / (2 \times 23.34) = 0.433.$$

The K' values are calculated for all columns and can be found in table 4-8.

Table 4.8: Value of K'.

Floor	K'			
	Axes 2,7 Ext	Axes 3,6 Ext	Axes 2,7 Int	Axes 3,6 Int
12 to 20	0.346	0.627	0.661	0.635
11	0.346	0.631	0.674	0.642
6 to 10	0.229	0.636	0.433	0.409
2 to 5	0.200		0.379	0.358
1	0.267	0.848	0.505	0.477

4.5.5 Calculations of Rigidity of Columns ($\sum D_c$ and D_c)

a Value for C90×35 in 20th Floor Axes 3,6 Ext = K' / (2 + K') = 0.627/ (2+0.627) = 0.239

a Value for C40×70 in 1st Floor Axes 2,7 Ext = (0.5 + K') / (2 + K')

$= (0.5 + 0.267) / (2+0.267) = 0.338$

D_c for C90×35 in 20th Floor Axes 3,6 Ext $= a \times K_c \times 12$

$= 0.239 \times 7.5 \times 12 = 21.479$ dm⁴/m.

D_c for C40×70 in 1st Floor Axes 2,7 Ext $= a \times K_c \times 12$

$= 0.338 \times 20.01 \times 12 = 81.222$ dm⁴/m.

The (a) values and D_c are calculated for all columns and can be found in table 4-9.

Table 4.9: Rigidity of Column and a Value.

Floor	a				Dc (dm⁴/m)			
	Axes 2,7 Ext	Axes 3,6 Ext	Axes 2,7 Int	Axes 3,6 Int	Axes 2,7 Ext	Axes 3,6 Ext	Axes 2,7 Int	Axes 3,6 Int
10 to 20	0.147	0.239	0.248	0.241	25.992	21.479	43.806	42.508
11	0.147	0.240	0.252	0.243	25.992	21.603	44.478	42.873
6 to 10	0.103	0.241	0.178	0.170	28.746	21.726	49.869	47.551
2 to 5	0.091		0.159	0.152	29.120		51.004	48.581
1	0.338	0.473	0.401	0.394	81.222	31.964	96.349	94.707

The summation of D_c in each floor:

20th floor $= 25.992 \times 4 + 21.479 \times 4 + 43.806 \times 4 + 42.508 \times 4$

$= 535.140$ dm⁴/m.

2nd floor $= 29.120 \times 4 + 21.726 \times 4 + 51.004 \times 4 + 48.581 \times 4$

$= 601.728$ dm⁴/m.

The summation of Dc in each floor values are calculated for all stories and can be found in table 4-10.

Table 4.10: Summation of the Rigidity of Columns in Each Floor.

Floor	Dc				No.	∑Dc (dm⁴/m)
	Axes 2,7 Ext	Axes 3,6 Ext	Axes 2,7 Int	Axes 3,6 Int		
12 to 20	25.992	21.479	43.806	42.508	4	535.140
11	25.992	21.603	44.478	42.873		539.784

Table 4.11: Summation of the Rigidity of Columns in Each Floor.

6 to 10	28.746	21.726	49.869	47.551		591.570
2 to 5	29.120		51.004	48.581		601.728
1	81.222	31.964	96.349	94.707		1216.967

4.5.6 Calculation of Normal Shear Wall and Core Wall Rectifiers

4.5.6.1 Normal shear walls connect with shear wall

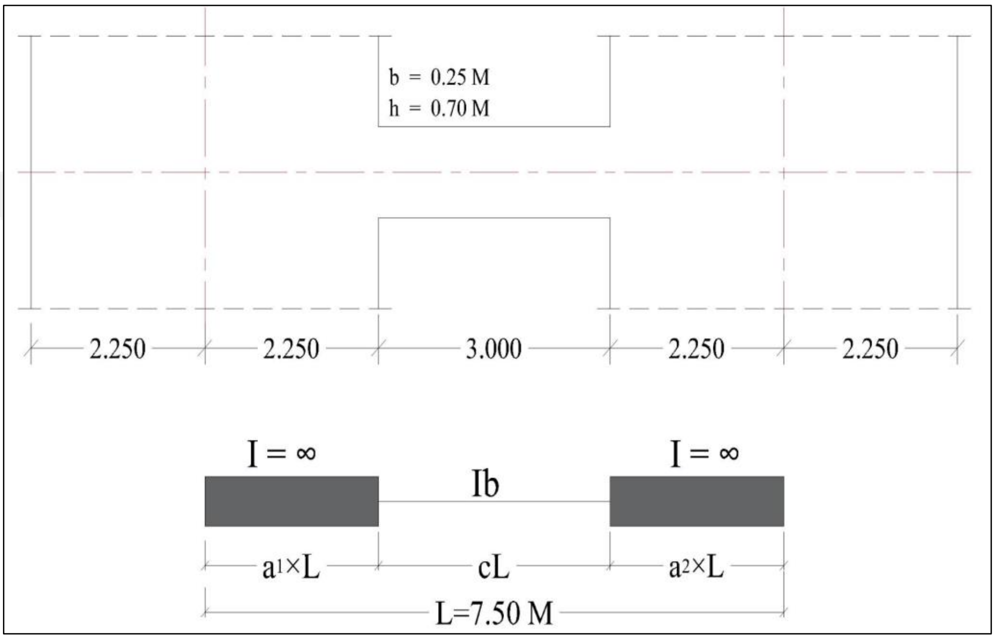


Figure 4.2: Coupled Beams Connected Between Shear Walls in Y-Direction.

From Figure (4.2):

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

b. $cL = 3.00$ m.

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

d. $c = 3.00/7.50 = 0.4$

e. $a = 2.25/7.5 = 0.3$

f. shape factor of rectangular=5/6

g. $A' =$ Shape factor × Area $_{\text{coupled beam}}$ $= 5/6 \times (0.25 \times 0.70) = 0.1458$ m^2

h. $E/G = 2.5$

This data will be used in calculation for Normal shear wall (Right and Left) rectifiers RA and RB in cases (With shear deformations and Without shear deformations).

a. With Shear Deformation (Case 1.1).

$\beta = (3 \times I_{bc} / cl^2 \times A') \times (E/G) = (3 \times 0.0025 / 3.0^2 \times 0.1458) \times 2.5 = 0.0143$

$K\beta = 1/(1+4\times\beta) = 1/ (1+4\times0.0143) = 0.946$

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

$= ((6\times1\times0.0013\times10^4) / (0.4^3\times7.5)) \times 0.946 = 147.862$ dm^4/m.

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

b. Without Shear Deformation (Case 1.2).

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

$= ((6\times1\times0.0013\times10^4) / (0.4^3\times7.5)) =$ **156.315** dm^4/m.

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

4.5.6.2 Core shear wall and normal shear wall (center) connection

a. Normal shear wall (Center):

For center shear wall it's appropriate to consider these as column widths, which run less than half the length of the shear wall perpendicular to this direction.

$b_{col} = B_{\text{(Beam width)}} + 2B_{w\text{ (width of shear wall)}} \leq 0.5\ L_w$
$b_{col} = 0.25 + 2 \times 0.35 \leq 0.5 \times 4$
$b_{col} = 0.95 \leq 2$

Obtain Dc for the considered column with dimension (0.95m × 0.35 m)

i. Moment of inertia (I_c):

$I_c = (b\times h^3 / 12) \times 0.7$

$I_c = (0.95\times0.35^3 / 12) \times 0.7 \times 10^4$

$I_c = 23.76\ dm^4$.

ii. Column Relative Rigidity (K_c):

K_c for 1st floor = $I_c / L = 23.76 / 4 = 5.94$ dm^4/m.

K_c for 2nd – 20th floor = $I_c / L = 23.76 / 3 = 7.92$ dm^4/m.

iii. Beam Relative Rigidity (K_b):

$K_b = I_{b,c} / L_{span} = 0.0025 / 3.475 \times 10^4 = 7.20$ dm^4/m.

iv. The Value of (K'):

K' for 1st floor = $K_b / K_c = 7.20 / 5.94 = 1.21$

K' for 2nd–20th floor = $K_{b1} + K_{b2}/2 \times K_c = 7.2+7.2 / 2\times7.92 = 0.91$

v. The Value of (a):

a for 1st floor = $(0.5 + K') / (2 + K') = (0.5 + 1.21) / (2 + 1.21) = 0.533$

a for 2nd–20th floor = $(K') / (2 + K') = (0.91) / (2 + 0.91) = 0.312$

vi. The Column Rigidity (D_c):

D_c for 1st floor = $K_c \times a \times 12 = 5.94 \times 0.533 \times 12 = 38$ dm^4/m.

D_c for 2nd–20th floor = $K_c \times a \times 12 = 7.92 \times 0.312 \times 12 = 29.70$ dm^4/m.

vii. The Summation of Column Rigidity ($\sum D_c$):

$\sum D_c$ for 1st floor = $38 \times 4 = 151.96$ dm^4/m.

$\sum D_c$ for 2nd–20th floor = $29.7 \times 4 = 118.77$ dm^4/m.

b. Core shear wall connecting with column:

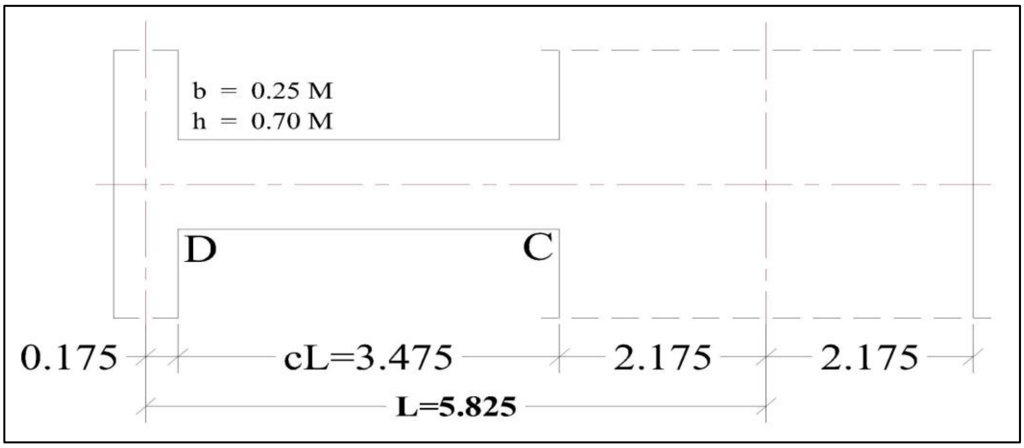


Figure 4.3: Coupled Beams Connected Between Shear Wall and Column in Y-Direction.

From Figure (4-3):

i. $a_1L = 2.175m.$
j. $cL = 3.65$ m.
k. $L = cL + a_1L = 3.65 + 2.175 = 5.825$ m.
l. $c = 3.65/5.825 = 0.627$
m. $a_1 = 2.175/5.825 = 0.373$
n. shape factor of rectangular = 5/6
o. $A' =$ Shape factor × Area $_{\text{coupled beam}}$ $= 5/6 \times (0.25 \times 0.70) = 0.175\ m^2$
p. $E/G = 2.5$
q. $I_{bf,c} = I_{b,c} / c^2 = 0.0025 / 0.627^2 = 0.00637\ m^4$
r. $K_{bf,c} = I_{bf,c} / L = 0.00637 / 3.475 = 0.00183\ m^3$

This data will be used in calculation for Core shear wall rectifiers RC in cases (With shear deformations and Without shear deformations).

$\beta = (3 \times I_{bc} / cl^2 \times A') \times (E/G) = (3\times0.0025 / 3.65^2\times0.1458) \times 2.5 = 0.00965$

$K\beta = 1/(1+4\times\beta) = 1/ (1+4\times0.00965) = 0.946$

EX: rectifiers for story 20th:

$K_c = 0.00131\ m^3$

$K_{c,o} = 0.$

$K_{c,u} = 0.00131\ m^3$

$r_o = Kc / (K_{c,o} + K_{bf,c}) = 0.00131 / (0 + 0.00183) = 0.7147$

$r_u = Kc / (K_{c,u} + K_{bf,c}) = 0.00131 / (0.00131 + 0.00183) = 0.4168$

$r_m = (ro + r_u) / 2 = (0.7147 + 0.4168) / 2 = 0.5658 < 0.67$

i. With Shear Deformation (Case 1.1).

$Rc = (3EI_b / c^3L) [(3/2) (3+2c) r_m +1] \times K\beta$

$= (3\times0.0025/0.627^3 \times 5.825) [(3/2) \times (3+2\times 0.627) \times 0.5658 +1] \times 0.946$

$= 116.18\ dm^4/m.$

The rectifiers of Core shear wall with shear deformation in each floor values are calculated for all stories and can be found in table 4-11.

Table 4.12: Calculation of Core Rectifiers RC- Case (1.1).

Data	**Story no.**			
	20	3 to 19	2	1
Kc	0.00131	0.00131	0.00131	0.00098
$K_{c,o}$	0.00000	0.00131	0.00131	0.00131
$K_{c,u}$	0.00131	0.00131	0.00098	0.00000
$K_{bf,c}$	0.00183	0.00183	0.00183	0.00183
r_o	0.7147	0.4168	0.4168	0.3126
r_u	0.4168	0.4168	0.4653	0.5360
r_m	0.5658 < 0.67	0.4168 < 0.67	0.4410 < 0.67	0.4243 < 0.67
RC	116.18	92.23	96.12	93.43

ii. Without Shear Deformation (Case 1.2).

$Rc = (3EI_b / c^3L) [(3/2) (3+2c) r_m +1]$

$= (3 \times 0.0025/0.627^3 \times 5.825) [(3/2) \times (3+2\times 0.627) \times 0.5658 +1]$ = **120.66** dm^4/m.

The rectifiers of Core shear wall without shear deformation in each floor values are calculated for all stories and can be found in table 4-12.

Table 4.13: Calculation of Core Rectifiers RC - Case (1.2).

Data	**Story no.**			
	20	3 to 19	2	1
Kc	0.00131	0.00131	0.00131	0.00098
$K_{c,o}$	0.00000	0.00131	0.00131	0.00131
$K_{c,u}$	0.00131	0.00131	0.00098	0.00000
$K_{bf,c}$	0.00183	0.00183	0.00183	0.00183
r_o	0.7147	0.4168	0.4168	0.3126
r_u	0.4168	0.4168	0.4653	0.5360
r_m	0.5658 < 0.67	0.4168 < 0.67	0.4410 < 0.67	0.4243 < 0.67
RC	120.66	95.79	99.84	97.04

4.5.7 Calculation of Fictive Shear Stiffness D_{fi}

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

Top floor = D_{f20} = Rectifier value × 1.5 (dm^4/m).

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

Bottom floor = D_{f1} = Rectifier value × 2 (dm^4/m).

4.5.7.1 With shear deformation (case 1.1)

a. Fictive shear stiffness for normal shear wall (right and left):

Top floor = D_{f20} = 147.862 × 1.5 = 221.793 (dm^4/m).

Intermediate floors = D_{f19} to D_{f2} = 147.862 (dm^4/m).

Bottom floor = D_{f1} = 147.862 × 2 = 295.725 (dm^4/m).

b. Fictive shear stiffness for core shear wall:

Top floor = D_{f20} = 116.18 × 1.5 = 174.263 (dm^4/m).

Intermediate floors = D_{f19} to D_{f3} = 92.225 (dm^4/m).

= D_{f2} = 96.123 (dm^4/m).

Bottom floor = D_{f1} = 93.43 × 2 = 186.865 (dm^4/m).

The Summation of Fictive shear stiffness for shear wall and core in each story:

$\sum D_{f,20}$ = 221.793 × 4 + 174.263 × 4 = 887.174 + 697.052 = 1584.225 dm^4/m.

$\sum D_{f,19,3}$ = 147.862 × 4 + 92.225 × 4 = 591.449 + 368.901 = 960.350 dm^4/m.

$\sum D_{f,2}$ = 147.862 × 4 + 96.123 × 4 = 591.449 + 384.493 = 975.943 dm^4/m.

$\sum D_{f,1}$ = 295.725 × 4 + 186.865 × 4 = 1182.898 + 747.460 = 1930.358 dm^4/m.

4.5.7.2 Without shear deformation (case 1.2)

a. Fictive shear stiffness for normal shear wall (right and left):

i. Top floor = D_{f20} = 156.315 × 1.5 = 234.473 (dm^4/m).

ii. Intermediate floors = D_{f19} to D_{f2} = 156.315 (dm^4/m).

iii. Bottom floor = D_{f1} = 156.315 × 2 = 312.630 (dm^4/m).

b. Fictive shear stiffness for core shear wall:

i. Top floor = D_{f20} = 120.66 × 1.5 = 180.993 (dm^4/m).

ii. Intermediate floors = D_{f19} to D_{f3} = 95.79 (dm^4/m) = D_{f2} = 99.84(dm^4/m).

iii. Bottom floor = D_{f1} = 97.04 × 2 = 194.081 (dm^4/m).

The Summation of Fictive shear stiffness for shear wall and core in each story:

i. $\sum D_{f,20}$ = 234.473 × 4 + 180.993 × 4 = 937.891 + 723.971 = 1661.862 dm^4/m.

ii. $\sum D_{f,19,3}$ = 156.315 × 4 + 95.787 × 4 = 625.260 + 383.148 = 1008.408 dm^4/m.

iii. $\sum D_{f,2}$ = 156.315 × 4 + 99.836 × 4 = 625.260 + 399.342 = 1024.602 dm^4/m.

iv. $\sum D_{f,1}$ = 312.630 × 4 + 194.081 × 4 = 1250.521 + 776.326 = 2026.846 dm^4/m

4.5.8 Calculation of the Rigidity of story ($\sum D$)

i. $\sum D_i = D_{ci} + D_{fci} + D_{fi}$

4.5.8.1 With shear deformation (case 1.1)

i. $\sum D_{20}$ = 535.140 + 118.77+ 1584.225 = 2238.134 dm^4/m.

ii. $\sum D_{19}$ = 535.140 + 118.77 + 960.350 = 1614.259 dm^4/m.

iii. $\sum D$ values in each story are calculated for all stories and can be found in table 4-13.

Table 4.14: The Rigidity of Story ($\sum D$) (Case1.1).

<table>
<tr><th rowspan="2">Story</th><th colspan="3">$\sum D_c$ & $\sum D_{fi}$ (dm^4/m)</th><th rowspan="2">$\sum D_{Total}$ (dm^4/m)</th></tr>
<tr><th>Columns</th><th>Considered column</th><th>Coupled beam connection</th></tr>
<tr><td>20</td><td rowspan="2">535.140</td><td rowspan="6">118.77</td><td>1584.225</td><td>2238.134</td></tr>
<tr><td>12 to 19</td><td rowspan="4">960.350</td><td>1614.259</td></tr>
<tr><td>11</td><td>539.784</td><td>1618.902</td></tr>
<tr><td>6 to 10</td><td>591.570</td><td>1670.689</td></tr>
<tr><td>3 to 5</td><td rowspan="2">601.728</td><td>1680.847</td></tr>
<tr><td>2</td><td>975.943</td><td>1696.439</td></tr>
<tr><td>1</td><td>1216.967</td><td>151.96</td><td>1930.358</td><td>3299.280</td></tr>
</table>

4.5.8.2 Without shear deformation (case 1.2)

$\sum D_{20}$ = 535.140 + 118.77 + 1661.862 = 2315.770dm^4/m.

$\sum D_{19}$ = 535.140 + 118.77 + 1008.408 = 1662.317 dm^4/m.

$\sum D$ values in each story are calculated for all stories and can be found in table 4-14.

Table 4.15: The Rigidity of Story (∑D) (Case1.2).

<table>
<tr><th rowspan="2">Story</th><th colspan="3">∑D & ∑Dfi (dm⁴/m)</th><th rowspan="2">∑D Total (dm⁴/m)</th></tr>
<tr><th>Columns</th><th>Considered column</th><th>Coupled beam connection</th></tr>
<tr><td>20</td><td rowspan="2">535.140</td><td rowspan="6">118.77</td><td>1661.862</td><td>2315.770</td></tr>
<tr><td>12 to 19</td><td rowspan="4">1008.408</td><td>1662.317</td></tr>
<tr><td>11</td><td>539.784</td><td>1666.960</td></tr>
<tr><td>6 to 10</td><td>591.570</td><td>1718.747</td></tr>
<tr><td>3 to 5</td><td rowspan="2">601.728</td><td>1728.904</td></tr>
<tr><td>2</td><td>1024.602</td><td>1745.099</td></tr>
<tr><td>1</td><td>1216.967</td><td>151.96</td><td>2026.846</td><td>3395.768</td></tr>
</table>

4.5.9 Calculations of the Coefficients and Constants of Equation Set ($\delta_{i,i-1}$, $\delta_{i,i}$ and $\delta_{i,o}$)

a. IP single Shear wall (Right and Left) = 1.3289 m^4.

b. IP Core =2.1127 m^4.

c. Taking E = 1.

d. $\sum EIP = 1.3289 \times 4 + 2.1127 \times 2 = 9.54\ m^4$.

4.5.9.1 With shear deformation (case 1.1)

a. $F_i = 1/\sum D$

$F_{20} = (1 / 2238.134) \times 10^4 = 4.468\ 1/m^3$.

b. $f_i = h_i/(6\times\sum(EI_p)_i)$.

$f_{20} = 3 / (6 \times 9.54) = 0.0524\ 1/m^3$.

c. $\delta_{i,i-1} = f_{i-1} - F_{i-1}$.

$\delta_{20,19} = f_{19} - F_{19} = 0.0524 - 4.468 = -6.142\ 1/m^3$

d. $\delta_{i,i} = 2\times f_i + 2\times f_{i-1} + F_i + F_{i-1}$.

$\delta_{20,20} = 2\times f_{20} + 2\times f_{19} + F_{20} + F_{19}$

$= 2\times0.0524 + 2\times0.0524 + 4.468 + 6.195$

$= 10.872\ 1/m^3$.

e. $\delta_{i,o} = f_i (M_{i+1,o} + 2 \times M_{i,o}) + f_{i-1} (M_{i-1,o} + 2 \times M_{i,o})$.

$$\delta_{20,o} = f_{20} (M_{21,o} + 2 \times M_{20,o}) + f_{19} (M_{19,o} + 2 \times M_{20,o})$$

$$= 0.0524 \times (0 + 2 \times 256.54) + 0.0524 \times (782.47 + 2 \times 256.54)$$

$$= 94.78 \text{ kNm}$$

The F_i, f_i, $\delta_{i,i-1}$, $\delta_{i,i}$ and $\delta_{i,o}$ values are calculated for all stories and can be found in table 4-15 for (Case 1.1).

Table 4.16: The Coefficients and Constants of Equation Set (δi,i-1 , δi,i and δi,o) (Case 1.1).

Floor	h (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	4.468	0.0524	256.543	-	10.872	94.782
19	3	6.195	0.0524	782.474	-6.1424	12.599	259.421
18	3	6.195	0.0524	1563.859	-6.1424	12.599	504.382
17	3	6.195	0.0524	2586.763	-6.1424	12.599	825.284
16	3	6.195	0.0524	3837.253	-6.1424	12.599	1217.745
15	3	6.195	0.0524	5301.395	-6.1424	12.599	1677.386
14	3	6.195	0.0524	6965.255	-6.1424	12.599	2199.824
13	3	6.195	0.0524	8814.900	-6.1424	12.599	2780.678
12	3	6.195	0.0524	10836.395	-6.1424	12.581	3415.568
11	3	6.177	0.0524	13015.807	-6.1246	12.372	4100.150
10	3	5.986	0.0524	15339.956	-5.9331	12.181	4830.200
9	3	5.986	0.0524	17794.836	-5.9331	12.181	5601.355
8	3	5.986	0.0524	20366.440	-5.9331	12.181	6409.211
7	3	5.986	0.0524	23040.761	-5.9331	12.181	7249.365
6	3	5.986	0.0524	25803.792	-5.9331	12.145	8117.432
5	3	5.949	0.0524	28641.923	-5.8970	12.108	9009.089
4	3	5.949	0.0524	31541.072	-5.8970	12.108	9919.932
3	3	5.949	0.0524	34487.159	-5.8970	12.054	10845.533
2	3	5.895	0.0524	37466.102	-5.8423	9.170	13867.277
1	4	3.031	0.0699	41466.102	-2.9611	3.171	8412.642

4.5.9.2 Without shear deformation (case 1.2)

f. $F_i = 1/\sum D$

$F_{20} = (1 / 2315.77) \times 10^4 = 4.318\ 1/m^3$.

g. $f_i = h_i/(6\times\sum(EI_p)_i)$.

$f_{20} = 3 / (6 \times 9.54) = 0.0524\ 1/m^3$.

h. $\delta_{i,i-1} = f_{i-1} - F_{i-1}$.

$\delta_{20,19} = f_{19} - F_{19} = 0.0524 - 4.318 = -5.963\ 1/m^3$

i. $\delta_{i,i} = 2\times f_i + 2\times f_{i-1} + F_i + F_{i-1}$.

$\delta_{20,20} = 2\times f_{20} + 2\times f_{19} + F_{20} + F_{19}$

$= 2\times0.0524 + 2\times0.0524 + 4.318 + 6.016$

$= 10.544\ 1/m^3$.

j. $\delta_{i,o} = f_i\ (M_{i+1,o} + 2 \times M_{i,o}) + f_{i-1}\ (M_{i-1,o} + 2 \times M_{i,o})$.

$\delta_{20,o} = f_{20}\ (M_{21,o} + 2 \times M_{20,o}) + f_{19}\ (M_{19,o} + 2 \times M_{20,o})$

$= 0.0524 \times (0 + 2 \times 256.54) + 0.0524 \times (782.47 + 2 \times 256.54)$

$= 94.876$ kNm

The F_i, f_i, $\delta_{i,i-1}$, $\delta_{i,i}$ and $\delta_{i,o}$ values are calculated for all stories and can be found in table 4-16 for (Case 1.2).

Table 4.17: The Coefficients and Constants of Equation Set (δi,i-1 , δi,i and δi,o) (Case 1.2).

Floor	h floor (m)	F_i ($1/m^3$)	f_i ($1/m^3$)	M_o (kNm)	$\delta_{i,i-1}$ ($1/m^3$)	$\delta_{i,i}$ ($1/m^3$)	$\delta_{i,o}$ (kNm)
20	3	4.318	0.0524	256.543	-	10.544	94.782
19	3	6.016	0.0524	782.474	-5.9633	12.241	259.421
18	3	6.016	0.0524	1563.859	-5.9633	12.241	504.382
17	3	6.016	0.0524	2586.763	-5.9633	12.241	825.284
16	3	6.016	0.0524	3837.253	-5.9633	12.241	1217.745
15	3	6.016	0.0524	5301.395	-5.9633	12.241	1677.386
14	3	6.016	0.0524	6965.255	-5.9633	12.241	2199.824
13	3	6.016	0.0524	8814.900	-5.9633	12.241	2780.678
12	3	6.016	0.0524	10836.395	-5.9633	12.224	3415.568
11	3	5.999	0.0524	13015.807	-5.9465	12.027	4100.150
10	3	5.818	0.0524	15339.956	-5.7658	11.846	4830.200
9	3	5.818	0.0524	17794.836	-5.7658	11.846	5601.355
8	3	5.818	0.0524	20366.440	-5.7658	11.846	6409.211
7	3	5.818	0.0524	23040.761	-5.7658	11.846	7249.365
6	3	5.818	0.0524	25803.792	-5.7658	11.812	8117.432

Table 4.18: The Coefficients And Constants of Equation Set (δi,i-1 , δi,i and δi,o) (Case1.2).

Floor	h floor (m)	Fi (1/m³)	fi (1/m³)	Mo (kNm)	δi,i-1 (1/m³)	δi,i (1/m³)	δi,o (kNm)
5	3	5.784	0.0524	28641.923	-5.7316	11.778	9009.089
4	3	5.784	0.0524	31541.072	-5.7316	11.778	9919.932
3	3	5.784	0.0524	34487.159	-5.7316	11.724	10845.533
2	3	5.730	0.0524	37466.102	-5.6779	8.920	13867.277
1	4	2.945	0.0699	41466.102	-2.8750	3.085	8412.642

4.5.10 Calculations of the Matrix Coefficients and Load Vector

4.5.10.1 With shear deformation (case 1.1)

i. Matrix[A] × 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
10.87	-6.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X20		-94.78
-6.14	12.60	-6.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X19		-259.42
0.00	-6.14	12.60	-6.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X18		-504.38
0.00	0.00	-6.14	12.60	-6.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X17		-825.28
0.00	0.00	0.00	-6.14	12.60	-6.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X16		-1217.75
0.00	0.00	0.00	0.00	-6.14	12.60	-6.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X15		-1677.39
0.00	0.00	0.00	0.00	0.00	-6.14	12.60	-6.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X14		-2199.82
0.00	0.00	0.00	0.00	0.00	0.00	-6.14	12.60	-6.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X13		-2780.68
0.00	0.00	0.00	0.00	0.00	0.00	0.00	-6.14	12.58	-6.12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X12		-3415.57
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-6.12	12.37	-5.93	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	×	X11	=	-4100.15
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.93	12.18	-5.93	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X10		-4830.20
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.93	12.18	-5.93	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X9		-5601.35
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.93	12.18	-5.93	0.00	0.00	0.00	0.00	0.00	0.00		X8		-6409.21
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.93	12.18	-5.93	0.00	0.00	0.00	0.00	0.00		X7		-7249.36
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.93	12.14	-5.90	0.00	0.00	0.00	0.00		X6		-8117.43
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.90	12.11	-5.90	0.00	0.00	0.00		X5		-9009.09
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.90	12.11	-5.90	0.00	0.00		X4		-9919.93
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.90	12.05	-5.84	0.00		X3		-10845.53
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.84	9.17	-2.96		X2		-13867.28
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-2.96	3.17		X1		-8412.64

ii. Matrix[A] $^{-1}$ × $-\delta_{i,o}$ = 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
0.167	0.134	0.107	0.085	0.068	0.054	0.043	0.035	0.028	0.022	0.018	0.014	0.011	0.009	0.008	0.006	0.005	0.005	0.004	0.004		-94.782		-1351.97
0.134	0.237	0.189	0.151	0.120	0.096	0.077	0.061	0.049	0.040	0.032	0.025	0.020	0.016	0.014	0.011	0.010	0.008	0.008	0.007		-259.421		-2377.64
0.107	0.189	0.281	0.224	0.179	0.143	0.114	0.091	0.073	0.059	0.047	0.038	0.030	0.025	0.020	0.017	0.014	0.012	0.011	0.011		-504.382		-3482.79
0.085	0.151	0.224	0.309	0.247	0.197	0.157	0.126	0.101	0.081	0.065	0.052	0.042	0.034	0.028	0.023	0.020	0.017	0.016	0.015		-825.284		-4684.11
0.068	0.120	0.179	0.247	0.327	0.261	0.209	0.167	0.134	0.107	0.086	0.069	0.055	0.045	0.037	0.030	0.026	0.023	0.021	0.019		-1217.745		-5990.85
0.054	0.096	0.143	0.197	0.261	0.339	0.271	0.216	0.173	0.139	0.111	0.089	0.072	0.058	0.048	0.040	0.034	0.029	0.027	0.025		-1677.386		-7406.01
0.043	0.077	0.114	0.157	0.209	0.271	0.346	0.277	0.222	0.178	0.142	0.114	0.092	0.074	0.061	0.051	0.043	0.038	0.034	0.032		-2199.824		-8927.20
0.035	0.061	0.091	0.126	0.167	0.216	0.277	0.352	0.282	0.226	0.181	0.145	0.117	0.094	0.077	0.064	0.055	0.048	0.044	0.041		-2780.678		-10547.25
0.028	0.049	0.073	0.101	0.134	0.173	0.222	0.282	0.356	0.286	0.228	0.183	0.147	0.119	0.098	0.081	0.069	0.061	0.055	0.052		-3415.568		-12254.50
0.022	0.040	0.059	0.081	0.107	0.139	0.178	0.226	0.286	0.360	0.288	0.231	0.186	0.150	0.123	0.102	0.087	0.076	0.070	0.065	×	-4100.150	=	-14038.16
0.018	0.032	0.047	0.065	0.086	0.111	0.142	0.181	0.228	0.288	0.365	0.292	0.235	0.191	0.156	0.130	0.110	0.097	0.088	0.082		-4830.200		-15932.28
0.014	0.025	0.038	0.052	0.069	0.089	0.114	0.145	0.183	0.231	0.292	0.369	0.297	0.241	0.197	0.164	0.139	0.122	0.111	0.104		-5601.355		-17856.64
0.011	0.020	0.030	0.042	0.055	0.072	0.092	0.117	0.147	0.186	0.235	0.297	0.375	0.304	0.249	0.207	0.176	0.154	0.141	0.131		-6409.211		-19783.24
0.009	0.016	0.025	0.034	0.045	0.058	0.074	0.094	0.119	0.150	0.191	0.241	0.304	0.383	0.313	0.260	0.221	0.194	0.177	0.165		-7249.365		-21678.03
0.008	0.014	0.020	0.028	0.037	0.048	0.061	0.077	0.098	0.123	0.156	0.197	0.249	0.313	0.395	0.328	0.279	0.245	0.223	0.208		-8117.432		-23499.82
0.006	0.011	0.017	0.023	0.030	0.040	0.051	0.064	0.081	0.102	0.130	0.164	0.207	0.260	0.328	0.414	0.352	0.308	0.281	0.263		-9009.089		-25209.26
0.005	0.010	0.014	0.020	0.026	0.034	0.043	0.055	0.069	0.087	0.110	0.139	0.176	0.221	0.279	0.352	0.443	0.388	0.354	0.331		-9919.932		-26735.13
0.005	0.008	0.012	0.017	0.023	0.029	0.038	0.048	0.061	0.076	0.097	0.122	0.154	0.194	0.245	0.308	0.388	0.489	0.446	0.417		-10845.533		-28004.33
0.004	0.008	0.011	0.016	0.021	0.027	0.034	0.044	0.055	0.070	0.088	0.111	0.141	0.177	0.223	0.281	0.354	0.446	0.563	0.526		-13867.277		-28936.21
0.004	0.007	0.011	0.015	0.019	0.025	0.032	0.041	0.052	0.065	0.082	0.104	0.131	0.165	0.208	0.263	0.331	0.417	0.526	0.807		-8412.642		-29676.43

4.5.10.2 Without shear deformation (case 1.2)

i. Matrix[A] × 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
10.54	-5.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X20		-94.78
-5.96	12.24	-5.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X19		-259.42
0.00	-5.96	12.24	-5.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X18		-504.38
0.00	0.00	-5.96	12.24	-5.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X17		-825.28
0.00	0.00	0.00	-5.96	12.24	-5.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X16		-1217.75
0.00	0.00	0.00	0.00	-5.96	12.24	-5.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X15		-1677.39
0.00	0.00	0.00	0.00	0.00	-5.96	12.24	-5.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X14		-2199.82
0.00	0.00	0.00	0.00	0.00	0.00	-5.96	12.24	-5.96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X13		-2780.68
0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.96	12.22	-5.95	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X12		-3415.57
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.95	12.03	-5.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	×	X11	=	-4100.15
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.77	11.85	-5.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X10		-4830.20
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.77	11.85	-5.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X9		-5601.35
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.77	11.85	-5.77	0.00	0.00	0.00	0.00	0.00	0.00		X8		-6409.21
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.77	11.85	-5.77	0.00	0.00	0.00	0.00	0.00		X7		-7249.36
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.77	11.81	-5.73	0.00	0.00	0.00	0.00		X6		-8117.43
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.73	11.78	-5.73	0.00	0.00	0.00		X5		-9009.09
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.73	11.78	-5.73	0.00	0.00		X4		-9919.93
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.73	11.72	-5.68	0.00		X3		-10845.53
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-5.68	8.92	-2.87		X2		-13867.28
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-2.87	3.08		X1		-8412.64

ii. Matrix$[A]^{-1}$ × $-\delta_{i,o}$ = 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
0.172	0.137	0.109	0.087	0.069	0.055	0.044	0.035	0.028	0.022	0.018	0.014	0.011	0.009	0.007	0.006	0.005	0.005	0.004	0.004		-94.782		-1347.26
0.137	0.242	0.193	0.153	0.122	0.097	0.077	0.062	0.049	0.039	0.031	0.025	0.020	0.016	0.013	0.011	0.009	0.008	0.007	0.007		-259.421		-2366.15
0.109	0.193	0.287	0.228	0.182	0.145	0.115	0.092	0.073	0.059	0.047	0.037	0.030	0.024	0.020	0.016	0.014	0.012	0.011	0.010		-504.382		-3466.31
0.087	0.153	0.228	0.315	0.251	0.199	0.159	0.127	0.101	0.081	0.064	0.051	0.041	0.033	0.027	0.022	0.019	0.017	0.015	0.014		-825.284		-4664.66
0.069	0.122	0.182	0.251	0.333	0.265	0.211	0.168	0.134	0.107	0.085	0.068	0.055	0.044	0.036	0.030	0.025	0.022	0.020	0.019		-1217.745		-5970.56
0.055	0.097	0.145	0.199	0.265	0.344	0.274	0.219	0.175	0.140	0.111	0.089	0.071	0.057	0.047	0.039	0.033	0.029	0.026	0.024		-1677.386		-7387.08
0.044	0.077	0.115	0.159	0.211	0.274	0.352	0.281	0.224	0.179	0.143	0.114	0.091	0.074	0.060	0.050	0.042	0.037	0.034	0.031		-2199.824		-8911.81
0.035	0.062	0.092	0.127	0.168	0.219	0.281	0.357	0.285	0.228	0.182	0.145	0.116	0.094	0.077	0.063	0.054	0.047	0.043	0.040		-2780.678		-10537.55
0.028	0.049	0.073	0.101	0.134	0.175	0.224	0.285	0.362	0.289	0.230	0.184	0.147	0.119	0.097	0.080	0.068	0.059	0.054	0.050		-3415.568		-12252.60
0.022	0.039	0.059	0.081	0.107	0.140	0.179	0.228	0.289	0.366	0.291	0.232	0.186	0.150	0.123	0.102	0.086	0.075	0.068	0.064	×	-4100.150	=	-14045.99
0.018	0.031	0.047	0.064	0.085	0.111	0.143	0.182	0.230	0.291	0.370	0.295	0.237	0.191	0.156	0.129	0.109	0.096	0.087	0.081		-4830.200		-15950.45
0.014	0.025	0.037	0.051	0.068	0.089	0.114	0.145	0.184	0.232	0.295	0.374	0.300	0.242	0.198	0.164	0.138	0.121	0.110	0.103		-5601.355		-17887.02
0.011	0.020	0.030	0.041	0.055	0.071	0.091	0.116	0.147	0.186	0.237	0.300	0.380	0.306	0.250	0.207	0.175	0.153	0.139	0.130		-6409.211		-19827.56
0.009	0.016	0.024	0.033	0.044	0.057	0.074	0.094	0.119	0.150	0.191	0.242	0.306	0.387	0.316	0.262	0.222	0.194	0.176	0.164		-7249.365		-21737.78
0.007	0.013	0.020	0.027	0.036	0.047	0.060	0.077	0.097	0.123	0.156	0.198	0.250	0.316	0.399	0.331	0.280	0.245	0.223	0.208		-8117.432		-23576.13
0.006	0.011	0.016	0.022	0.030	0.039	0.050	0.063	0.080	0.102	0.129	0.164	0.207	0.262	0.331	0.418	0.354	0.309	0.282	0.262		-9009.089		-25302.55
0.005	0.009	0.014	0.019	0.025	0.033	0.042	0.054	0.068	0.086	0.109	0.138	0.175	0.222	0.280	0.354	0.448	0.391	0.356	0.332		-9919.932		-26845.23
0.005	0.008	0.012	0.017	0.022	0.029	0.037	0.047	0.059	0.075	0.096	0.121	0.153	0.194	0.245	0.309	0.391	0.494	0.450	0.419		-10845.533		-28129.86
0.004	0.007	0.011	0.015	0.020	0.026	0.034	0.043	0.054	0.068	0.087	0.110	0.139	0.176	0.223	0.282	0.356	0.450	0.570	0.531		-13867.277		-29074.28
0.004	0.007	0.010	0.014	0.019	0.024	0.031	0.040	0.050	0.064	0.081	0.103	0.130	0.164	0.208	0.262	0.332	0.419	0.531	0.819		-8412.642		-29825.78

4.5.11 Calculations of the Moment Superposition and Distribution to The Core, Shear walls

4.5.11.1 With shear deformation (case 1.1)

$Mp_{total} = M_{o,20} + X_{20} = 256.54 + (-1351.97) = -1095.43$ kNm

Moment for Shear wall (Right and Left)

$M_{\text{shear wall (right and left)},20} = M_{\text{Total}} \times (I_{\text{shear wall (right and left)}} / \sum Ip)$

$= -1095.43 \times (1.3289 / 9.54) = -152.57$ kNm

Moment for Core Shear wall

$M_{Core,20} = M_{Total} \times (I_{Core} / \sum Ip)$

$= -1095.43 \times (2.1127 / 9.54) = -242.57$ kNm

The Moment Superposition and Distribution to The Core, Shear walls (Right, Left) values in each story are calculated for all stories and can be found in table 4-17 for (Case1.1).

Table 4.19: Moment superposition and Distribution to the Core, Shear walls (Case 1.1).

Floor	Mo (kNm)	Xi (kNm)	M total Shear wall (kNm)	M single Shear wall (R & L) (kNm)	M Core shear wall (kNm)
20	256.543	-1351.968	-1095.425	-152.574	-242.565
19	782.474	-2377.640	-1595.166	-222.179	-353.224
18	1563.859	-3482.789	-1918.931	-267.274	-424.917
17	2586.763	-4684.109	-2097.346	-292.124	-464.424
16	3837.253	-5990.850	-2153.597	-299.959	-476.880
15	5301.395	-7406.011	-2104.617	-293.137	-466.034
14	6965.255	-8927.205	-1961.950	-273.266	-434.443
13	8814.900	-10547.246	-1732.347	-241.286	-383.601
12	10836.395	-12254.501	-1418.106	-197.518	-314.017
11	13015.807	-14038.160	-1022.353	-142.396	-226.384
10	15339.956	-15932.282	-592.326	-82.501	-131.161
9	17794.836	-17856.640	-61.804	-8.608	-13.686
8	20366.440	-19783.243	583.198	81.229	129.140
7	23040.761	-21678.031	1362.730	189.805	301.755
6	25803.792	-23499.817	2303.975	320.904	510.179
5	28641.923	-25209.262	3432.661	478.111	760.109
4	31541.072	-26735.132	4805.940	669.385	1064.200
3	34487.159	-28004.330	6482.829	902.947	1435.521
2	37466.102	-28936.208	8529.894	1188.068	1888.811
1	41466.102	-29676.429	11789.673	1642.099	2610.638

4.5.11.2 Without shear deformation (case 1.2)

$Mp_{total} = M_{o,20} + X_{20} = 256.54 + (-1347.26) = -1090.71$ kNm

a. Moment for Shear wall (Right and Left)

i. $M_{shear\ wall\ (right\ and\ left),20} = M_{Total} \times (I_{shear\ wall\ (right\ and\ left)} / \sum Ip)$

$$= -1090.71 \times (1.3289 / 9.54) = -151.92 \text{ kNm}$$

b. Moment for Core Shear wall

i. $M_{Core,20} = M_{Total} \times (I_{Core} / \sum Ip)$

$$= -1090.71 \times (2.1127 / 9.54) = -241.52 \text{ kNm}$$

The Moment Superposition and Distribution to The Core, Shear walls (Right, Left) values in each story are calculated for all stories and can be found in table 4-18 for (Case1.2).

Table 4.20: Moment Superposition and Distribution to The Core, Shear Walls (Case 1.2).

Floor	Mo (kNm)	Xi (kNm)	M total Shear wall (kNm)	M single Shear wall (R & L) (kNm)	M Core shear wall (kNm)
20	256.543	-1347.257	-1090.714	-151.918	-241.521
19	782.474	-2366.154	-1583.680	-220.579	-350.681
18	1563.859	-3466.310	-1902.451	-264.979	-421.268
17	2586.763	-4664.655	-2077.892	-289.415	-460.117
16	3837.253	-5970.562	-2133.310	-297.133	-472.388
15	5301.395	-7387.077	-2085.682	-290.500	-461.842
14	6965.255	-8911.809	-1946.554	-271.122	-431.034
13	8814.900	-10537.545	-1722.646	-239.935	-381.453
12	10836.395	-12252.603	-1416.208	-197.253	-313.597
11	13015.807	-14045.985	-1030.178	-143.486	-228.117
10	15339.956	-15950.452	-610.496	-85.032	-135.185
9	17794.836	-17887.023	-92.187	-12.840	-20.413
8	20366.440	-19827.560	538.880	75.057	119.327
7	23040.761	-21737.777	1302.984	181.483	288.525
6	25803.792	-23576.131	2227.661	310.275	493.281
5	28641.923	-25302.554	3339.369	465.117	739.451
4	31541.072	-26845.225	4695.847	654.051	1039.822
3	34487.159	-28129.858	6357.302	885.463	1407.725
2	37466.102	-29074.278	8391.824	1168.837	1858.238
1	41466.102	-29825.783	11640.319	1621.297	2577.566

4.5.12 Calculations of Shear Forces for Columns, shear wall and Core

4.5.12.1 With shear deformation (Case 1.1)

a. Shear Force in each story

$$\sum T_i = \Delta X_i / h_i$$

$= X_{i+1} - X_i / h_i$

$\sum T_{20} = 0 – (-1351.97) / 3 = 450.66$ kN.

$\sum T_{19} = (-1351.97) – (-2377.64) / 3 = 341.89$ kN.

b. Shear Force for columns in each story:

$T_{20,\ \text{Axes 2,7 Ext}} = 450.66 \times (25.992 / 2243.419) = 5.23$ kN.

$T_{19,\ \text{Axes 2,7 Ext}} = 341.89 \times (25.992 / 1619.544) = 5.51$ kN.

c. Shear Force for shear walls in each story:

$T_{f20,\ \text{Right and left}} = 450.66 \times (221.793 / 2243.419) = 44.66$ kN.

$T_{f19,\ \text{Right and left}} = 341.89 \times (147.862 / 1619.544) = 31.32$ kN.

d. Shear Force for core shear walls in each story:

$T_{f20,\ \text{Core}} = 450.66 \times (174.263 / 2243.419) = 35.09$ kN.

$T_{f19,\ \text{Core}} = 341.89 \times (92.225 / 1619.544) = 19.53$ kN.

The Shear Forces for Columns, shear wall and Core wall values in each story are calculated for all stories and can be found in table 4-19 for (Case1.1).

Table 4.21: Shear Forces for Columns, Shear Wall and Core Wall (Case 1.1).

Floor	**h (m)**	**∑T**	**Tcol**				**Tfi shear wall (R & L)**	**Tfi Core**
			Axes 2,7 Ext	**Axes 3,6 Ext**	**Axes 2,7 Int**	**Axes 3,6 Int**		
20	3	450.656	5.234	4.325	8.820	8.559	44.659	35.088
19	3	341.891	5.505	4.549	9.278	9.003	31.316	19.533
18	3	368.383	5.932	4.902	9.997	9.701	33.743	21.046
17	3	400.440	6.448	5.328	10.867	10.545	36.679	22.878
16	3	435.580	7.014	5.796	11.820	11.470	39.898	24.885
15	3	471.721	7.595	6.277	12.801	12.422	43.208	26.950
14	3	507.065	8.165	6.747	13.760	13.352	46.446	28.969
13	3	540.014	8.695	7.185	14.654	14.220	49.464	30.852
12	3	569.085	9.163	7.572	15.443	14.986	52.127	32.513
11	3	594.553	9.546	7.934	16.335	15.745	54.303	33.870
10	3	631.374	10.864	8.211	18.846	17.970	55.879	34.853
9	3	641.453	11.037	8.342	19.147	18.257	56.771	35.409
8	3	642.201	11.050	8.351	19.169	18.278	56.837	35.451
7	3	631.596	10.867	8.214	18.853	17.976	55.899	34.865
6	3	607.262	10.449	7.897	18.126	17.284	53.745	33.522
5	3	569.815	9.872	7.365	17.291	16.469	50.126	31.265
4	3	508.623	8.812	6.574	15.434	14.701	44.743	27.907
3	3	423.066	7.329	5.468	12.838	12.228	37.217	23.213
2	3	310.626	5.332	3.978	9.339	8.895	27.074	17.601
1	4	185.055	4.556	1.793	5.404	5.312	16.587	10.481

4.5.12.2 Without shear deformation (case 1.2)

a. Shear Force in each story

$\sum T_i = \Delta X_i / h_i$

$= Xi_{+1} - X_i / h_i$

$\sum T_{20} = 0 - (-1347.26) / 3 = 449.09$ kN.

$\sum T_{19} = (-1347.26) - (-2366.15) / 3 = 339.63$ kN.

b. Shear Force for columns in each story:

$T_{20,\ \text{Axes 2,7 Ext}} = 449.09 \times (25.992 / 2321.055) = 5.04$ kN.

$T_{19,\ \text{Axes 2,7 Ext}} = 339.63 \times (25.992 / 1667.602) = 5.31$ kN.

c. Shear Force for shear walls in each story:

$T_{f20,\ \text{Right and left}} = 449.09 \times (234.473 / 2321.055) = 45.47$ kN.

$T_{f19,\ \text{Right and left}} = 339.63 \times (156.315 / 1667.602) = 31.94$ kN.

d. Shear Force for core shear walls in each story:

$T_{f20,\ \text{Core}} = 449.09 \times (180.993/ 2321.055) = 35.1$ kN.

$T_{f19,\ \text{Core}} = 339.63 \times (95.787 / 1667.602) = 19.57$ kN.

The Shear Forces for Columns, shear wall and Core wall values in each story are calculated for all stories and can be found in table 4-20 for (Case1.2).

Table 4.22: Shear Forces for Columns, Shear Wall and Core Wall (Case 1.2).

Floor	h (m)	∑T	Tcol				Tfi shear wall (R & L)	Tfi Core
			Axes 2,7 Ext	Axes 3,6 Ext	Axes 2,7 Int	Axes 3,6 Int		
20	3	449.086	5.041	4.165	8.495	8.243	45.470	35.099
19	3	339.632	5.311	4.389	8.950	8.685	31.937	19.570
18	3	366.719	5.734	4.739	9.664	9.378	34.484	21.131
17	3	399.448	6.246	5.161	10.526	10.214	37.562	23.017
16	3	435.302	6.806	5.625	11.471	11.131	40.933	25.083
15	3	472.171	7.383	6.101	12.443	12.074	44.400	27.208
14	3	508.244	7.947	6.567	13.393	12.997	47.792	29.286
13	3	541.912	8.473	7.002	14.281	13.857	50.958	31.226

Table 4.23: Shear Forces for Columns, Shear Wall and Core Wall (Case 1.2).

Floor	h (m)	∑T	Tcol				Tfi shear wall (R & L)	Tfi Core
			Axes 2,7 Ext	Axes 3,6 Ext	Axes 2,7 Int	Axes 3,6 Int		
12	3	571.686	8.939	7.387	15.065	14.619	53.758	32.942
11	3	597.794	9.321	7.747	15.950	15.375	56.057	34.350
10	3	634.822	10.618	8.025	18.419	17.563	57.735	35.379
9	3	645.524	10.797	8.160	18.730	17.859	58.709	35.975
8	3	646.846	10.819	8.177	18.768	17.896	58.829	36.049
7	3	636.739	10.650	8.049	18.475	17.616	57.910	35.486
6	3	612.785	10.249	7.746	17.780	16.953	55.731	34.151
5	3	575.475	9.693	7.232	16.977	16.171	52.030	31.883
4	3	514.224	8.661	6.462	15.170	14.449	46.492	28.490
3	3	428.211	7.212	5.381	12.633	12.033	38.716	23.724
2	3	314.807	5.253	3.919	9.201	8.764	28.198	18.010
1	4	187.876	4.494	1.768	5.331	5.240	17.297	10.738

4.5.13 Calculations of the Coupled Beam End Moments And Corrected The Shear Wall Moment.

4.5.13.1 With shear deformation (case 1.1)

a. Coupled beam end moment for shear wall (right and left):

$M_{20,u} = 1/3 \times 44.66 \times 3 = 44.66$ kNm

$M_{20,o} = 2/3 \times 44.66 \times 3 = 89.32$ kNm

$M_{19,u} = 1/2 \times 31.32 \times 3 = 46.98$ kNm

$M_{19,o} = 44.66 + 46.98 = 91.64$ kNm

b. Coupled beam end moment for Core shear wall:

$M_{20,u} = 1/3 \times 35.09 \times 3 = 35.09$ kNm

$M_{20,o} = 2/3 \times 35.09 \times 3 = 70.18$ kNm

$M_{19,u} = 1/2 \times 19.53 \times 3 = 29.30$ kNm

$M_{19,o} = 35.09 + 29.30 = 64.39$ kNm

The Coupled beam end moment for shear wall and Core wall values in each story are calculated for all stories and can be found in table 4-21 for (Case1.1).

Table 4.24: Coupled Beam End Moments (Case 1.1).

Floor	h (m)	shear wall (R & L)		Core	
		$M_{i,u}$	**$M_{i,o}$**	**$M_{i,u}$**	**$M_{i,o}$**
20	3	44.659	89.318	35.088	70.177
19	3	46.975	91.633	29.299	64.388
18	3	50.615	97.589	31.569	60.869
17	3	55.019	105.634	34.317	65.886
16	3	59.847	114.866	37.328	71.645
15	3	64.813	124.660	40.425	77.753
14	3	69.669	134.482	43.454	83.879
13	3	74.196	143.865	46.278	89.732
12	3	78.190	152.386	48.769	95.047
11	3	81.455	159.645	50.806	99.575
10	3	83.819	165.274	52.280	103.085
9	3	85.157	168.975	53.114	105.394
8	3	85.256	170.412	53.176	106.290
7	3	83.848	169.104	52.298	105.474
6	3	80.617	164.465	50.283	102.581
5	3	75.189	155.807	46.897	97.180
4	3	67.115	142.304	41.861	88.758
3	3	55.825	122.940	34.819	76.680
2	3	40.611	96.436	26.401	61.220
1	4	33.174	73.786	20.962	47.363

c. Corrected shear wall moments at bottom and top (Right and left):

$M_{Bottom,1st} = 1642.1 + 33.17 = 1675.27$ kNm

$M_{Top,1st} = 1188.07 – 33.17 = 1154.89$ kNm

$M_{Bottom,2nd} = 1188.07 + 40.61 = 1228.68$ kNm

$M_{Top,2nd} = 902.95 – 40.61 = 862.34$ kNm

d. Corrected core moments at bottom and top:

$M_{Bottom,1st} = 2610.64 + 20.96 = 2631.60$ kNm

$M_{Top,1st} = 1888.81 – 20.96 = 1867.85$ kNm

$M_{Bottom,2nd} = 1888.81 + 26.40 = 1915.21$ kNm

$M_{Top,2nd} = 1435.52 – 26.40 = 1409.12$ kNm

The corrected normal shear wall and Core wall moments at bottom and top values in each story are calculated for all stories and can be found in table 4-22 for (Case1.1).

Table 4.25: Corrected Shear Wall and Core Moments at Bottom and Top (Case 1.1).

Floor	Shear wall R & L				Core			
	M kNm	$M_{i,u}$	M_1 Bottom	M_2 Upper	M kNm	$M_{i,u}$	M_1 Bottom	M_2 Upper
20	-152.57	89.32 44.66	-107.92	-89.32	-242.56	70.18 35.09	-207.48	-70.18
19	-222.18	46.97	-175.20	-199.55	-353.22	29.30	-323.93	-271.86
18	-267.27	50.61	-216.66	-272.79	-424.92	31.57	-393.35	-384.79
17	-292.12	55.02	-237.11	-322.29	-464.42	34.32	-430.11	-459.23
16	-299.96	59.85	-240.11	-351.97	-476.88	37.33	-439.55	-501.75
15	-293.14	64.81	-228.32	-364.77	-466.03	40.43	-425.61	-517.31
14	-273.27	69.67	-203.60	-362.81	-434.44	43.45	-390.99	-509.49
13	-241.29	74.20	-167.09	-347.46	-383.60	46.28	-337.32	-480.72
12	-197.52	78.19	-119.33	-319.48	-314.02	48.77	-265.25	-432.37
11	-142.40	81.46	-60.94	-278.97	-226.38	50.81	-175.58	-364.82
10	-82.50	83.82	1.32	-226.21	-131.16	52.28	-78.88	-278.66
9	-8.61	85.16	76.55	-167.66	-13.69	53.11	39.43	-184.28
8	81.23	85.26	166.49	-93.86	129.14	53.18	182.32	-66.86
7	189.80	83.85	273.65	-2.62	301.76	52.30	354.05	76.84
6	320.90	80.62	401.52	109.19	510.18	50.28	560.46	251.47
5	478.11	75.19	553.30	245.72	760.11	46.90	807.01	463.28
4	669.39	67.11	736.50	411.00	1064.20	41.86	1106.06	718.25
3	902.95	55.82	958.77	613.56	1435.52	34.82	1470.34	1029.38
2	1188.07	40.61	1228.68	862.34	1888.81	26.40	1915.21	1409.12
1	1642.10	33.17	1675.27	1154.89	2610.64	20.96	2631.60	1867.85

4.5.13.2 Without shear deformation (case 1.2).

e. Coupled beam end moment for shear wall (right and left):

$M_{20,u} = 1/3 \times 45.47 \times 3 = 45.47$ kNm

$M_{20,o} = 2/3 \times 45.47 \times 3 = 90.94$ kNm

$M_{19,u} = 1/2 \times 31.94 \times 3 = 47.91$ kNm

$M_{19,o} = 45.47 + 47.91 = 93.38$ kNm

f. Coupled beam end moment for Core shear wall:

$M_{20,u} = 1/3 \times 35.09 \times 3 = 35.09$ kNm

$M_{20,o} = 2/3 \times 35.09 \times 3 = 70.2$ kNm

$M_{19,u} = 1/2 \times 19.57 \times 3 = 29.36$ kNm

$M_{19,o} = 35.09 + 29.36 = 64.45$ kNm

The Coupled beam end moment for shear wall and Core wall values in each story are calculated for all stories and can be found in table 4-23 for (Case1.2).

Table 4.26: Coupled Beam End Moments (Case 1.2).

Floor	h floor (m)	shear wall (R & L)		Core	
		$M_{i,u}$	$M_{i,o}$	$M_{i,u}$	$M_{i,o}$
20	3	45.470	90.940	35.099	70.198
19	3	47.906	93.376	29.356	64.455
18	3	51.726	99.632	31.697	61.053
17	3	56.343	108.069	34.526	66.223
16	3	61.400	117.743	37.625	72.151
15	3	66.601	128.001	40.812	78.436
14	3	71.689	138.289	43.929	84.741
13	3	76.438	148.126	46.840	90.769
12	3	80.637	157.075	49.413	96.253
11	3	84.085	164.722	51.526	100.939
10	3	86.603	170.688	53.069	104.594
9	3	88.063	174.666	53.963	107.032
8	3	88.243	176.306	54.074	108.037
7	3	86.864	175.108	53.229	107.303
6	3	83.597	170.461	51.226	104.455
5	3	78.045	161.642	47.825	99.051
4	3	69.739	147.784	42.735	90.559
3	3	58.074	127.812	35.586	78.321
2	3	42.298	100.371	27.015	62.601
1	4	34.594	76.891	21.476	48.490

g. Corrected shear wall moments at bottom and top (Right and left):

$M_{Bottom,1st} = 1621.30 + 34.59 = 1655.89$ kNm

$M_{Top,1st} = 1168.84 – 34.59 = 1134.24$ kNm

$M_{Bottom,2nd} = 1168.84 + 42.30 = 1211.13$ kNm

$M_{Top,2nd} = 885.05 – 42.30 = 843.17$ kNm

h. Corrected core moments at bottom and top:

$M_{Bottom,1st} = 2577.57 + 21.48 = 2599.04$ kNm

$M_{Top,1st} = 1858.24 – 21.48 = 1836.76$ kNm

$M_{Bottom,2nd}$ = 1858.24 + 27.01 = 1885.25 kNm

$M_{Top,2nd}$ = 1407.72 – 27.01 = 1380.71 kNm

The corrected normal shear wall and Core wall moments at bottom and top values in each story are calculated for all stories and can be found in table 4-24 for (Case1.2).

Table 4.27: Corrected Shear Wall Moments at Bottom and Top (Case 1.2).

Floor	Shear wall R & L				Core			
	M kNm	**$M_{i,u}$**	**M_1 Bottom**	**M_2 Upper**	**M kNm**	**$M_{i,u}$**	**M_1 Bottom**	**M_2 Upper**
20	-151.92	90.94 45.47	-106.45	-90.94	-241.52	70.20 35.10	-206.42	-70.20
19	-220.58	47.91	-172.67	-199.82	-350.68	29.36	-321.33	-270.88
18	-264.98	51.73	-213.25	-272.31	-421.27	31.70	-389.57	-382.38
17	-289.41	56.34	-233.07	-321.32	-460.12	34.53	-425.59	-455.79
16	-297.13	61.40	-235.73	-350.81	-472.39	37.62	-434.76	-497.74
15	-290.50	66.60	-223.90	-363.73	-461.84	40.81	-421.03	-513.20
14	-271.12	71.69	-199.43	-362.19	-431.03	43.93	-387.10	-505.77
13	-239.94	76.44	-163.50	-347.56	-381.45	46.84	-334.61	-477.87
12	-197.25	80.64	-116.62	-320.57	-313.60	49.41	-264.18	-430.87
11	-143.49	84.09	-59.40	-281.34	-228.12	51.53	-176.59	-365.12
10	-85.03	86.60	1.57	-230.09	-135.18	53.07	-82.12	-281.19
9	-12.84	88.06	75.22	-173.09	-20.41	53.96	33.55	-189.15
8	75.06	88.24	163.30	-101.08	119.33	54.07	173.40	-74.49
7	181.48	86.86	268.35	-11.81	288.53	53.23	341.75	66.10
6	310.28	83.60	393.87	97.89	493.28	51.23	544.51	237.30
5	465.12	78.05	543.16	232.23	739.45	47.82	787.28	445.46
4	654.05	69.74	723.79	395.38	1039.82	42.73	1082.56	696.72
3	885.46	58.07	943.54	595.98	1407.72	35.59	1443.31	1004.24
2	1168.84	42.30	1211.13	843.17	1858.24	27.01	1885.25	1380.71
1	1621.30	34.59	1655.89	1134.24	2577.57	21.48	2599.04	1836.76

4.5.14 Calculation of the First Natural Time-Period (T_{1y}) By Using Rayleigh Method

The periods will be in the y-direction since the base shear force operates in the y-direction of the structure. The shear forces operating on the frames and the total ∑D must be used by converting them to their true values. E = 1 was previously used. Now, (E/h_i^2) values are multiplied by ∑D using E -concrete (33000 N/mm^2).

4.5.14.1 With shear deformation (case 1.1)

$\sum D_{20j} = 2243.42 \times 10^3 \times 33000 \times 10^{-4} / 3^2 = 822587.03$ kN /m.

$\delta_i = (\sum T_i)/ (\sum Dij)$ (m).

$\delta_1 = 185.06/683700.14 = 0.000272$ m.

$\delta_2 = 310.63/623965.38 = 0.000499$ m.

$\delta_3 = 423.07/618248.24 = 0.000686$ m.

$d_i = \sum \delta i$(m).

$d_1 = \delta_0 + \delta_1 = 0.000272$ m

$d_2 = \delta_1 + \delta_2 = d_1 + \delta_2 = 0.000272 + 0.000499 = 0.00077$ m

$d_3 = \delta_1 + \delta_2 + \delta_3 = d_2 + \delta_3 = 0.00077 + 0.000686 = 0.001458$ m

The Stiffness, Displacement and drift values in each story are calculated for all stories and can be found in table 4-25 for (Case1.1).

Table 4-28: Displacement, Drift and Stiffness of Story Values (Case 1.1).

Floor	h (m)	P_i (kN)	∑T (kN)	∑D dm^4/m	$\sum D_{ij}$ kN / m	δ (m)	d_i (m)
20	3	85.51	450.656	2238.134	820649.20	0.000549	0.016047
19	3	89.80	341.891	1614.259	591895.03	0.000578	0.015498
18	3	85.15	368.383	1614.259	591895.03	0.000622	0.014920
17	3	80.51	400.440	1614.259	591895.03	0.000677	0.014298
16	3	75.86	435.580	1614.259	591895.03	0.000736	0.013621

Table 4.29: Summation of the Rigidity of Columns in Each Floor.

15	3	71.22	471.721	1614.259	591895.03	0.000797	0.012885
14	3	66.57	507.065	1614.259	591895.03	0.000857	0.012088
13	3	61.93	540.014	1614.259	591895.03	0.000912	0.011232
12	3	57.28	569.085	1614.259	591895.03	0.000961	0.010319
11	3	52.64	594.553	1618.902	593597.56	0.001002	0.009358
10	3	48.25	631.374	1670.689	612585.91	0.001031	0.008356
9	3	43.58	641.453	1670.689	612585.91	0.001047	0.007325
8	3	38.91	642.201	1670.689	612585.91	0.001048	0.006278
7	3	34.24	631.596	1670.689	612585.91	0.001031	0.005230
6	3	29.57	607.262	1670.689	612585.91	0.000991	0.004199
5	3	25.03	569.815	1680.847	616310.41	0.000925	0.003208
4	3	20.34	508.623	1680.847	616310.41	0.000825	0.002283
3	3	15.65	423.066	1680.847	616310.41	0.000686	0.001458
2	3	10.95	310.626	1696.439	622027.55	0.000499	0.000771
1	4	7.02	185.055	3299.280	680476.41	0.000272	0.000272

$M_i = W_i$ / g (kN/m/sec^2) ; g=9.81 (m/sec^2)

$M_1 = W_1$ / 9.81= 5741.76 / 9.81 = 585.297 (kN/m/sec^2)

$M_2 = W_2$ / 9.81 = 5119.51 / 9.81 = 521.867 (kN/m/sec^2)

$M_i d_i^2 = M_i \times d_i^2$

$M_1 d_1^2 = M_1 \times d_1^2$ = 585.297 × 0.000272^2 = 0.000043 kNm Sec^2

$M_2 d_2^2 = M_2 \times d_2^{2\,=}$ 521.867 × 0.00077^2 = 0.00031 kNm Sec^2

$P_i d_i = P_i \times d_i$

$P_1 d_1 = P_1 \times d_1$ = 7.02 × 0.000272 = 0.001909 kNm

$P_2 d_2 = P_2 \times d_2$ = 10.95 × 0.00077 = 0.008448 kNm

The Mass, $M_i d_i^2$ and $P_i d_i$ values in each story are calculated for all stories and can be found in table 4-26 for (Case1.1).

Table 4.30: Midi and Pidi Value (Case 1.1).

Floor	Wi (kN)	Mi (kN/m/sec2)	Midi² (kNmSec²)	Pidi (kNm)
20	4587.13	467.597	0.120405	1.372229
19	5065.93	516.404	0.124028	1.391619
18	5065.93	516.404	0.114955	1.270454
17	5065.93	516.404	0.105564	1.151051
16	5065.93	516.404	0.095810	1.033320
15	5065.93	516.404	0.085737	0.917646
14	5065.93	516.404	0.075459	0.804744
13	5065.93	516.404	0.065143	0.695546
12	5065.93	516.404	0.054989	0.591118
11	5065.93	516.404	0.045220	0.492579
10	5092.48	519.111	0.036246	0.403146
9	5092.48	519.111	0.027856	0.319219
8	5092.48	519.111	0.020462	0.244275
7	5092.48	519.111	0.014199	0.179068
6	5092.48	519.111	0.009152	0.124162
5	5119.51	521.867	0.005369	0.080297
4	5119.51	521.867	0.002720	0.046436
3	5119.51	521.867	0.001109	0.022808
2	5119.51	521.867	0.000310	0.008448
1	5741.76	585.297	0.000043	0.001909
		10383.551	1.004777	11.150072

$\omega_1^2{}_{(Y)} = \sum P_i d_i / \sum M_i d_i^2 = 11.123 / 0.99995 = 11.12$

$\omega_{1\ (Y)} = 3.34$ rad/sec.

a. $T_{1(Y)} = (2 \times 3.14) / 3.34 = 1.89$ Sec.

4.5.14.2 Without shear deformation (case 1.2)

$\sum D_{20j} = 2321.055 \times 10^3 \times 33000 \times 10^{-4} / 3^2 = 851053.61$ kN /m.

a. $\delta_i = (\sum T_i) / (\sum Dij)$ (m).

b. δ_1 = 187.876/703600.87 = 0.000268 m.

c. δ_2 = 314.807/641807.34 = 0.000492 m.

d. $d_i = \sum \delta i(m)$.

e. $d_1 = \delta_0 + \delta_1 =$ 0.000268 m

f. $d_2 = \delta_1 + \delta_2 = d_1 + \delta_2$ = 0.000268 + 0.000492 = 0.000760 m

The Stiffness, Displacement and drift values in each story are calculated for all stories and can be found in table 4-27 for (Case1.2).

Table 4.31: Displacement, Drift and Stiffness of Story Values (Case 1.2).

Floor	**h (m)**	**Pi (kN)**	**∑T (kN)**	**∑D dm^4/m**	**∑Dij kN / m**	**δ (m)**	**di (m)**
20	3	85.51	449.086	2315.770	849115.78	0.000529	0.015666
19	3	89.80	339.632	1662.317	609516.20	0.000557	0.015137
18	3	85.15	366.719	1662.317	609516.20	0.000602	0.014580
17	3	80.51	399.448	1662.317	609516.20	0.000655	0.013979
16	3	75.86	435.302	1662.317	609516.20	0.000714	0.013323
15	3	71.22	472.171	1662.317	609516.20	0.000775	0.012609
14	3	66.57	508.244	1662.317	609516.20	0.000834	0.011834
13	3	61.93	541.912	1662.317	609516.20	0.000889	0.011000
12	3	57.28	571.686	1662.317	609516.20	0.000938	0.010111
11	3	52.64	597.794	1666.960	611218.73	0.000978	0.009173
10	3	48.25	634.822	1718.747	630207.07	0.001007	0.008195
9	3	43.58	645.524	1718.747	630207.07	0.001024	0.007188
8	3	38.91	646.846	1718.747	630207.07	0.001026	0.006164
7	3	34.24	636.739	1718.747	630207.07	0.001010	0.005137
6	3	29.57	612.785	1718.747	630207.07	0.000972	0.004127
5	3	25.03	575.475	1728.904	633931.58	0.000908	0.003155
4	3	20.34	514.224	1728.904	633931.58	0.000811	0.002247
3	3	15.65	428.211	1728.904	633931.58	0.000675	0.001436
2	3	10.95	314.807	1745.099	639869.51	0.000492	0.000760
1	4	7.02	187.876	3395.768	700377.15	0.000268	0.000268

$M_i = W_i / g$ (kN/m/sec^2) ; g=9.81 (m/sec^2)

$M_1 = W_1$ / 9.81 = 5741.76 / 9.81 = 585.297 (kN/m/sec^2)

$M_id_i^2 = M_i \times d_i^2$

$M_1d_1^2 = M_1 \times d_1^2 = 585.297 \times 0.000268^2 = 0.000042$ kNm Sec²

$P_id_i = P_i \times d_i$

$P_1d_1 = P_1 \times d_1 = 7.02 \times 0.000268 = 0.001883$ kNm

The Mass, $M_id_i^2$ and P_id_i values in each story are calculated for all stories and can be found in table 4-28 for (Case1.2).

Table 4.32: Midi and Pidi Value (Case 1.2).

Floor	Wi (kN)	Mi (kN/m/sec2)	Midi² kNmSec²	Pidi (kNm)
20	4587.13	467.597	0.114763	1.339694
19	5065.93	516.404	0.118329	1.359274
18	5065.93	516.404	0.109778	1.241519
17	5065.93	516.404	0.100905	1.125363
16	5065.93	516.404	0.091665	1.010722
15	5065.93	516.404	0.082101	0.897979
14	5065.93	516.404	0.072323	0.787843
13	5065.93	516.404	0.062490	0.681239
12	5065.93	516.404	0.052797	0.579216
11	5065.93	516.404	0.043457	0.482881
10	5092.48	519.111	0.034866	0.395395
9	5092.48	519.111	0.026822	0.313235
8	5092.48	519.111	0.019722	0.239820
7	5092.48	519.111	0.013701	0.175899
6	5092.48	519.111	0.008842	0.122036
5	5119.51	521.867	0.005194	0.078972
4	5119.51	521.867	0.002635	0.045701
3	5119.51	521.867	0.001076	0.022463
2	5119.51	521.867	0.000302	0.008326
1	5741.76	585.297	0.000042	0.001883
		10383.551	0.961810	10.909460

$\omega_1^2{}_{(Y)} = \sum P_id_i / \sum M_id_i^2 = 10.883 / 0.957 = 11.37$

$\omega_{1\,(Y)} = 3.37$ rad./sec.

$T_{1(Y)} = (2 \times 3.14) / 3.37 = 1.86$ Sec.

4.5.15 Calculation of the real base shear force

Calculate the real base shear force for structure in Second degree earthquake zone with the following Characteristic in Cases (With shear deformation and Without shear deformation):

a. Effective Ground Acceleration Coefficient (A_o) = 0.30
b. Building Importance Factor (I) For Office building = 1
c. Local Site Class = Z2
d. Spectrum Coefficient = (T_A = 0.15 sec), (T_B = 0.40 sec).
e. Structural Behavior Factors (R) = 7.
f. Damping ratio (ξ) = 5%

4.5.15.1 With shear deformation (case 1.1)

By using Erzincan 1992 N-S chart, intersect the T-period value (**1.89**) with Damping ratio (ξ=5%) to obtain the (Sa/a_{max}) value.

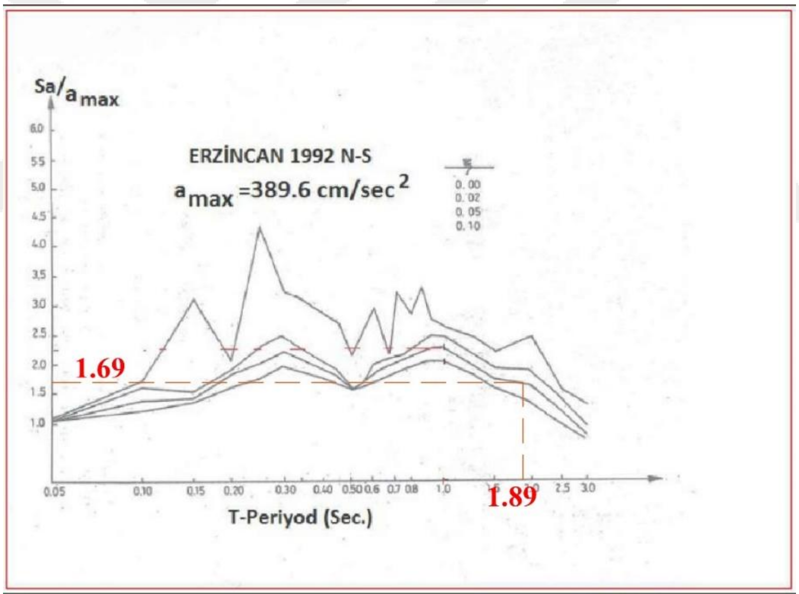


Figure 4.4: Erzincan 1992 N-S Chart (Case1.1).

Sa/a_{max} value = 1.69

$T_{1(y)} = 1.88 > T_B = 0.40$

$S_{(T1, Y)} = 2.5\,(0.4 / 1.89)^{0.8} = 0.724$

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

$A_{(T1, Y)} = 0.30 \times 9.81 \times 0.724 \times 1$

$A_{(T1, Y)} = 2.131$

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

$V_t = 10383.551 \times 2.131 / 7$

Total Equivalent Seismic Load (base shear), V_t , Appling on the building in earthquake direction:

$V_t = 3160.53$ kN.

Base Shear Force in case of using Erzincan 1992 N-S Acceleration Spectrum:

$V_{t2} = Vt \times ((Sa/a_{max}) / S_{(T1, Y)})$

$V_{t2} = 3167.31 \times (1.69/ 0.724)$

$V_{t2} = 7378$ kN.

Vt2 / Vt1 Ratio = 7378/1000 = **7.38**

This ratio will be utilized to rectify the shear forces and moments because the real base shear force is not 1000 kN as assumed.

4.5.15.2 Without shear deformation (case 1.2)

By using Erzincan 1992 N-S chart, intersect the T-period value (1.86) with Damping ratio (ξ=5%) to obtain the (Sa/a_{max}) value.

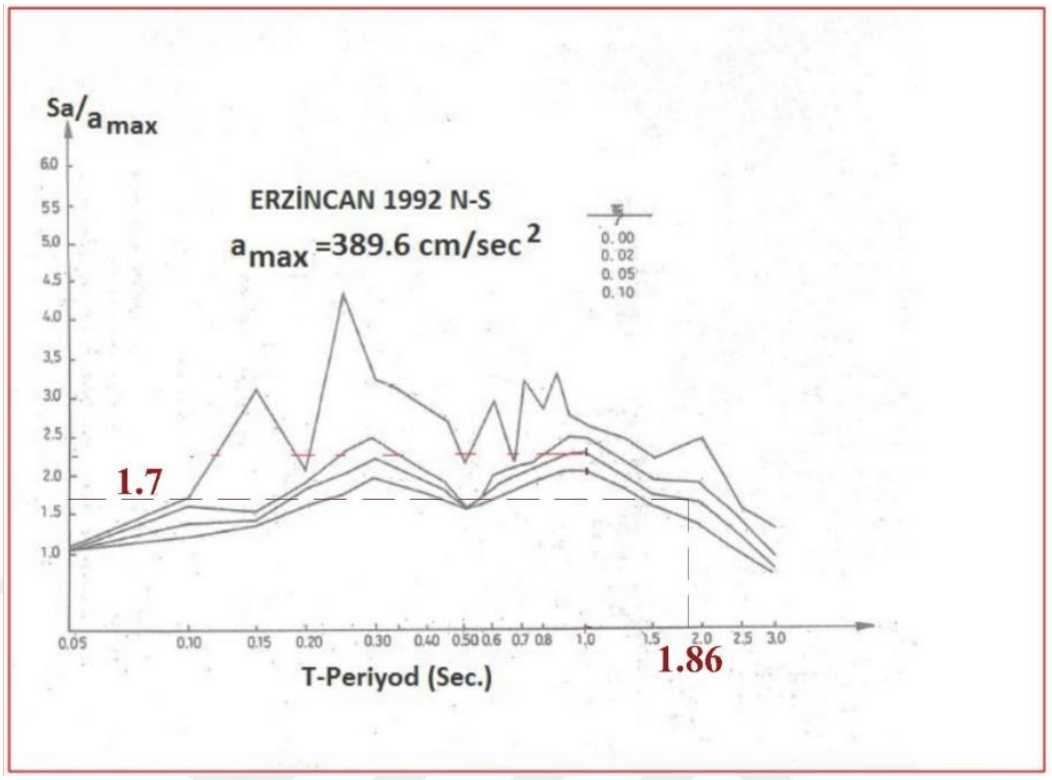


Figure 4.5: Erzincan 1992 N-S chart (Case1.2).

Sa/a_{max} value = 1.7

$T_{1(y)} = 1.86 > T_B = 0.40$

$S_{(T1, Y)} = 2.5\ (0.4 / 1.86)^{0.8} = 0.73$

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

$A_{(T1, Y)} = 0.30 \times 9.81 \times 0.73 \times 1$

$A_{(T1, Y)} = 2.149$

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

$V_t = 10383.551 \times 2.149 / 7$

Total Equivalent Seismic Load (base shear), V_t , Appling on the building in earthquake direction:

$V_t = 3188.25$ kN.

Base Shear Force In case of using Erzincan 1992 N-S Acceleration Spectrum:

$V_{t2} = Vt \times ((Sa/a_{max})/ S_{(T1, Y)})$

$V_{t2} = 3188.25 \times (1.7/ 0.73)$

$V_{t2} = 7421$ kN.

Vt2 / Vt1 Ratio = 7421/1000 = 7.42

This ratio will be utilized to rectify the shear forces and moments because the real base shear force is not 1000 kN as assumed.

4.5.16 Correcting the Moment, Shear Force, Displacement and Drift

To correct the Values of Moment, Shear force, Displacement and Drift by multiplied its by Vt_2 / Vt_1 ratio which is **7.38** for (Case 1.1-With shear deformation) and **7.42** for (Case 1.2-Without shear deformation).

4.5.16.1 With shear deformation (case 1.1)

a. Total moment $_{\text{shear walls 1st story}}$ = 11789.673× 7.38 = 86981.44 kNm

b. Single moment 1st story:

Shear walls R&L = 1642.099× 7.38 = 12115.02 kNm

Core walls = 2610.638× 7.38 = 19260.68 kNm

c. Shear Forces $_{\text{Total 8th}}$ = 642.201 × 7.38 = 4738.01 kN

d. Corrected shear wall moments bottom 1st floor:

Right and left = 1675.273 × 7.38 = 12359.77 kN

Core wall = 2631.601 × 7.38 = 19415.33 kN

e. Maximum drift $_{8th}$ = 0.001048 × 7.38 × 7 / 3 = 0.01805

Check Maximum drift $_{8th}$ according to (Specification for Buildings to be Built in Seismic Zones 2007 item 2.10.1.3).

0.01805 < 0.02 O.K.

f. Maximum Displacement $_{20th}$ = 0.016047 ×10^3 × 7.38 = 118.39 mm

Check Maximum Displacement $_{20th}$ with $\Delta_{\text{max-all}} = 0.002 \times H_B$

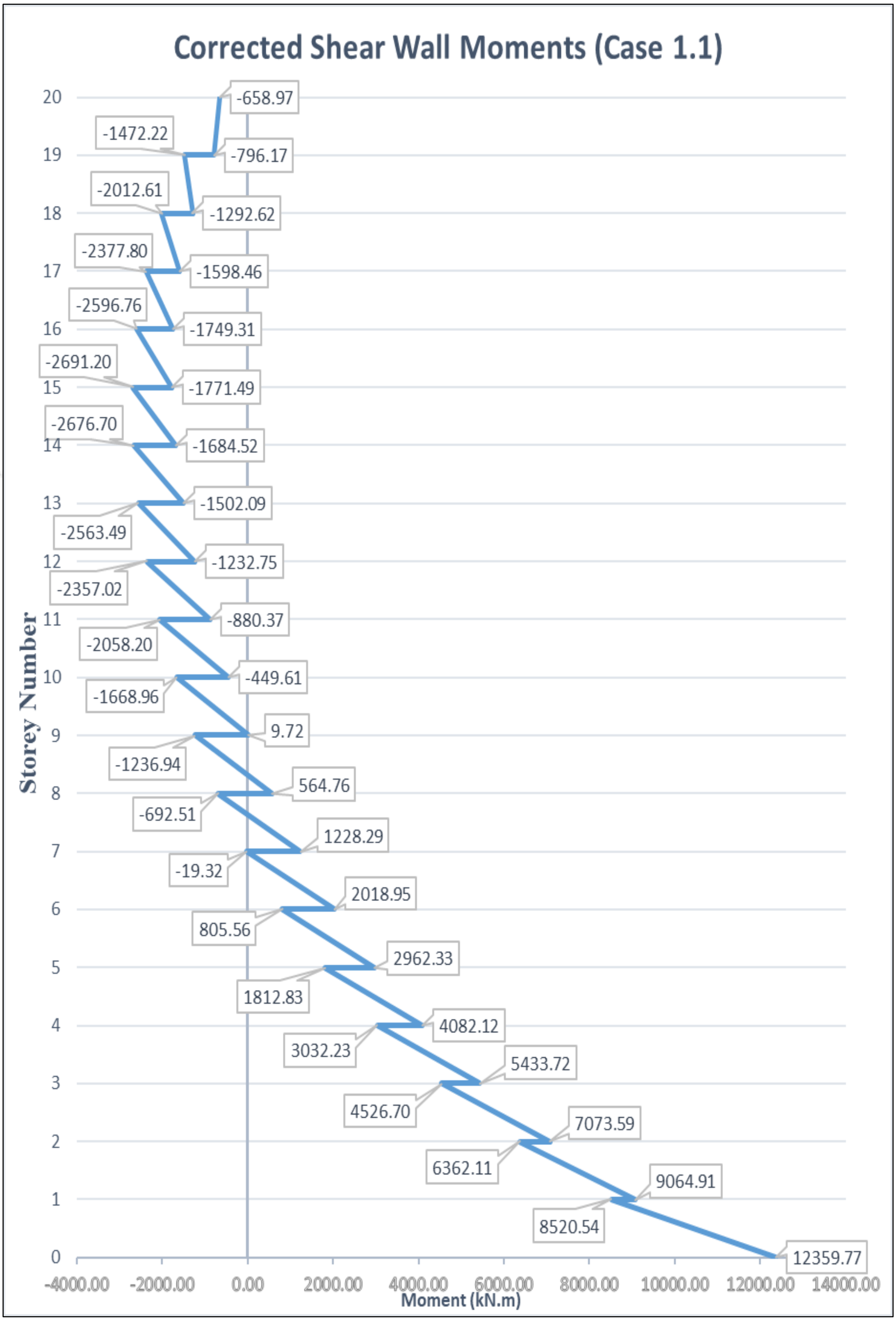
$\Delta_{\text{max-all}} = 0.002 \times 61 = 122$ mm.

118.39 mm < 122 mm O.K.

Corrected shear walls and core moments after multiplying it by the Vt2/Vt1 ratio in each story are calculated for all stories and can be found in table 4-29 and figure (4-6 and 4-7) for (Case1.1).

Table 4.33: Corrected Shear Wall and Core Moments with Vt2/Vt1 Ratio Case (1.1).

Floor	Shear wall R & L		Core	
	M_1 Bottom	M_2 Upper	M_1 Bottom	M_2 Upper
20	-796.17	658.97	-1530.71	517.75
19	-1292.62	-1472.22	-2389.84	-2005.75
18	-1598.46	-2012.61	-2902.03	-2838.92
17	-1749.31	-2377.80	-3173.23	-3388.12
16	-1771.49	-2596.76	-3242.91	-3701.81
15	-1684.52	-2691.20	-3140.04	-3816.56
14	-1502.09	-2676.70	-2884.62	-3758.89
13	-1232.75	-2563.49	-2488.69	-3546.65
12	-880.37	-2357.02	-1956.94	-3189.93
11	-449.61	-2058.20	-1295.38	-2691.58
10	9.72	-1668.96	-581.97	-2055.92
9	564.76	-1236.94	290.90	-1359.54
8	1228.29	-692.51	1345.09	-493.29
7	2018.95	-19.32	2612.12	566.92
6	2962.33	805.56	4134.96	1855.30
5	4082.12	1812.83	5953.90	3417.98
4	5433.72	3032.23	8160.26	5299.07
3	7073.59	4526.70	10847.83	7594.53
2	9064.91	6362.11	14129.99	10396.16
1	12359.77	8520.54	19415.33	13780.55



C

Figure 4.6: Corrected Shear Wall Moments (Case 1.1).

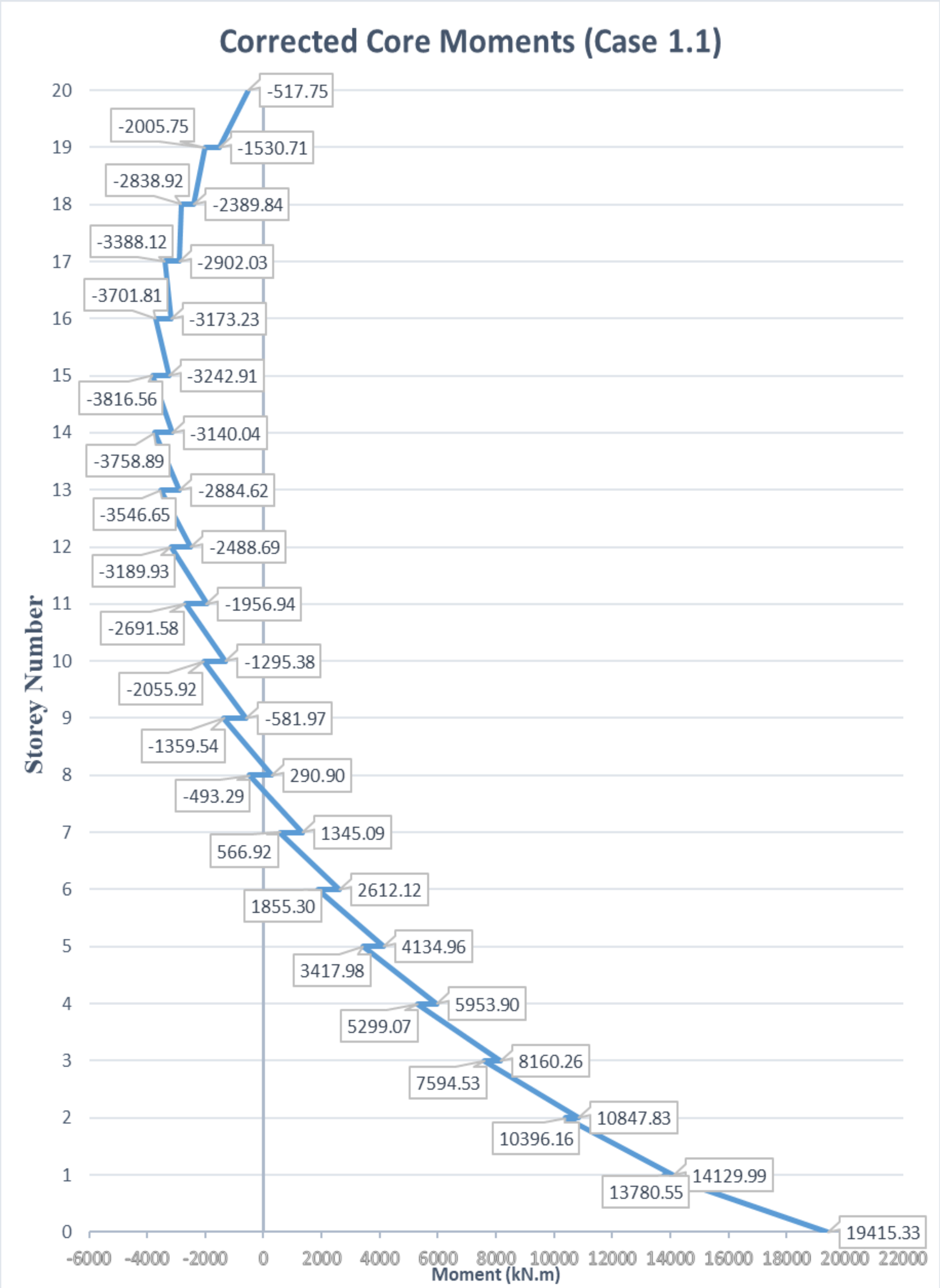


Figure 4.7: Corrected Core Moments (Case 1.1).

4.5.16.2 Without shear deformation (case 1.2)

g. Total moment $_{\text{shear walls 1st story}}$ = 11640.319× 7.42 = 86387.71 kNm

h. Single moment 1st story:

   Shear walls (R&L) = 1621.297 × 7.42 = 12032.33 kNm core walls 1st story = 2577.566 × 7.42 = 19129.20 kNm

i. Shear Forces $_{\text{Total 8th}}$ = 646.846 × 7.42 = 4800.51 kN

j. Corrected shear wall moments bottom 1st floor:

   Right and left= 1655.890 × 7.42 = 12289.06 kNm Core wall = 2599.042 × 7.42 = 19288.58 kNm

k. Maximum drift $_{8th}$ = 0.001026 × 7.42 × 7 / 3 = 0.01777

   Check Maximum drift $_{8th}$ according to (Specification for Buildings to be Built in Seismic Zones 2007 item 2.10.1.3), 0.01777 < 0.02 O.K.

l. Maximum Displacement $_{20th}$ = 0.015666 ×10^3 × 7.42 = 116.27 mm

   Check Maximum Displacement $_{20th}$ with $\Delta_{\text{max-all}}$ = 0.002 × H_B 116.27 mm < 122 mm O.K.

Corrected shear walls and core moments after multiplying it by the Vt2/Vt1 ratio in each story are calculated for all stories and can be found in table 4-30 and figure (4-8 and 4-9) for (Case1.2).

Table 4.34: Corrected Shear Wall and Core Moments with Vt2/Vt1 Ratio Case (1.2).

Floor	Shear wall R & L		Core	
	M_1 Bottom	M_2 Upper	M_1 Bottom	M_2 Upper
20	-789.99	674.91	-1531.95	520.97
19	-1281.48	-1482.97	-2384.69	-2010.29
18	-1582.64	-2020.90	-2891.17	-2837.79
17	-1729.72	-2384.66	-3158.49	-3382.64
16	-1749.48	-2603.55	-3226.56	-3693.95
15	-1661.65	-2699.42	-3124.64	-3808.67
14	-1480.08	-2687.95	-2872.86	-3753.54
13	-1213.38	-2579.38	-2483.31	-3546.50
12	-865.46	-2379.10	-1960.62	-3197.64
11	-440.84	-2087.93	-1310.56	-2709.73
10	11.66	-1707.59	-609.42	-2086.80
9	558.26	-1284.61	248.99	-1403.75
8	1211.92	-750.18	1286.88	-552.80
7	1991.52	-87.63	2536.30	490.54
6	2923.09	726.46	4041.02	1761.10
5	4031.04	1723.47	5842.70	3305.92
4	5371.55	2934.27	8034.10	5170.62
3	7002.38	4423.00	10711.42	7452.85
2	8988.34	6257.49	13991.25	10246.83
1	12289.06	8417.70	19288.58	13631.38

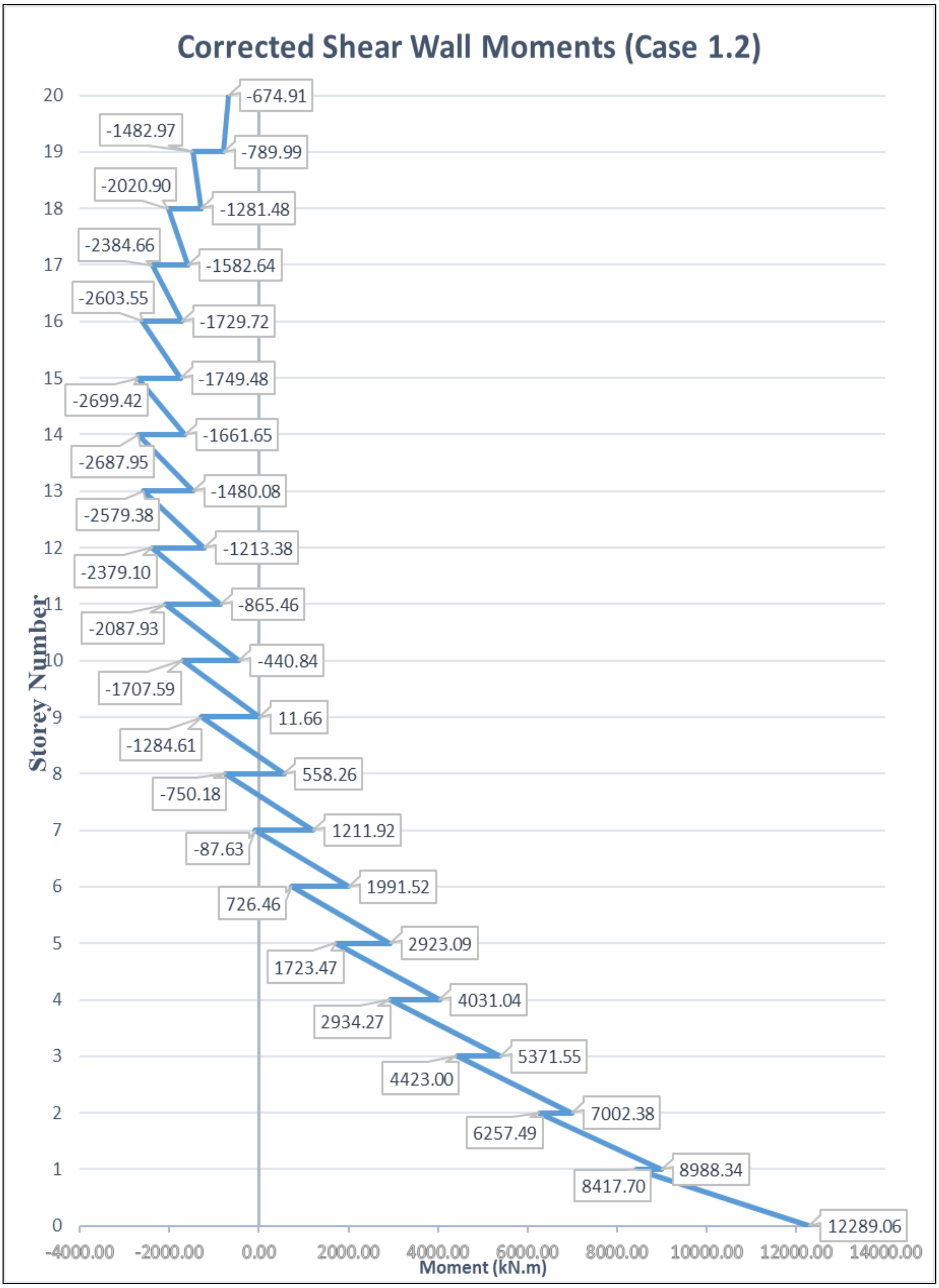


Figure 4.8: Corrected Shear Wall Moments (Case 1.2).

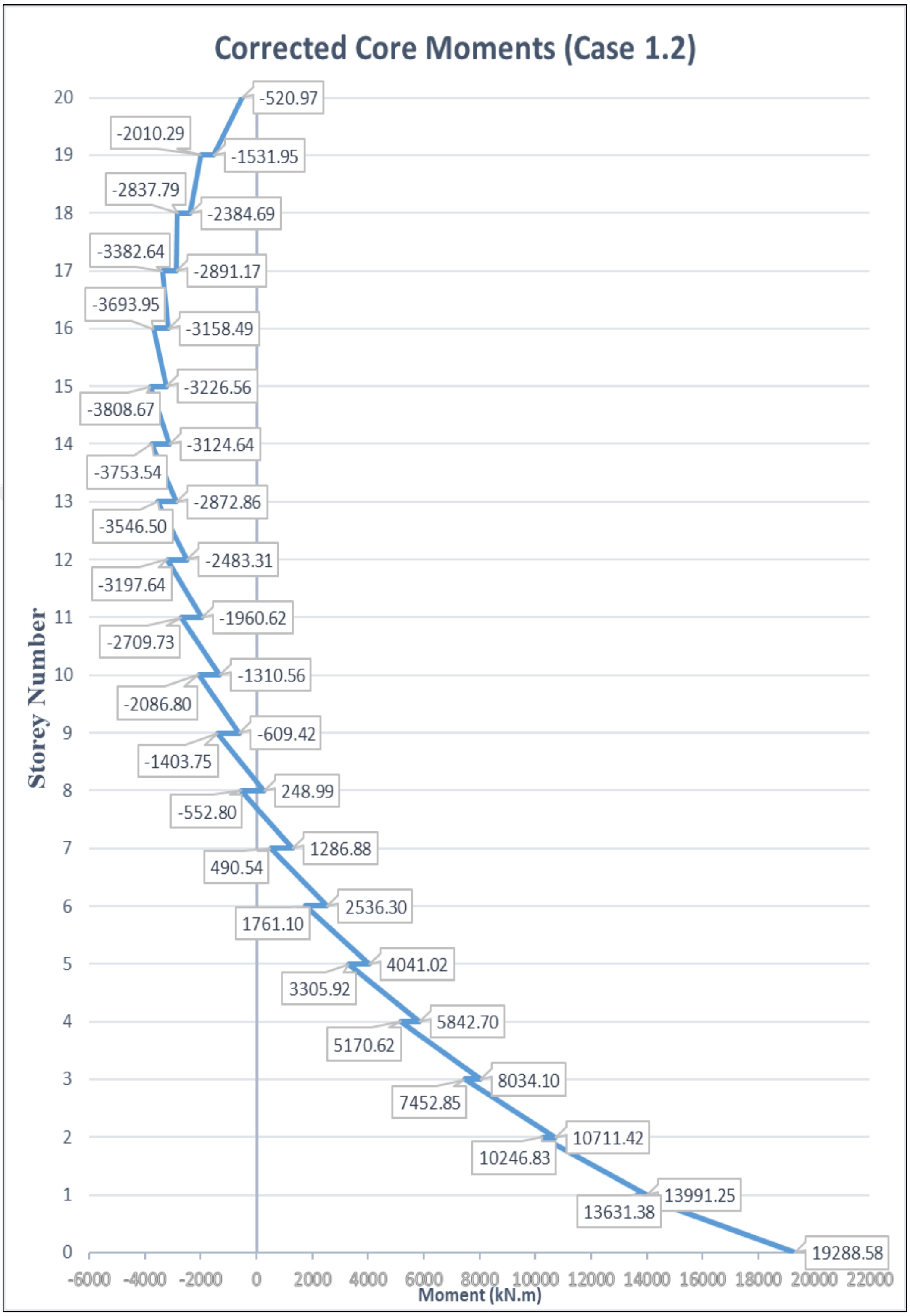


Figure 4.9: Corrected Core Moments (Case 1.2).

4.5.17 The Second Order Indicator Value $\theta_{II,i}$

4.5.17.1 With shear deformation (case 1.1)

Using the equation 3-68 to calculate the $\theta_{II,i}$ and compare with the $\theta_{II,I,max}$ as shown in equation 3-69:

Δ_{ave} : from table 4-25.

$W_{k,i}$: from table 4-26.

V_i: from table 4-4 corrected with Vt_2/Vt_1 for case (1.1).,

$\theta_{II,max}$ = 0.12 × (2.5 / 0.5 × 7) = 0.0857

$\theta_{II,20}$ = 0.00055 × 4587.13 / 630.91 × 3 = 0.0013 ≤ 0.0857

$\theta_{II,19}$ = 0.00058 × 9653.05 / 1293.40 × 3 = 0.0014 ≤ 0.0857

Table 4.35: Second Order Effect Case (1.1).

Story	hi (m)	Δ_{ave}(m)	$W_{k,i}$(kN)	$\sum w_{i,k}$(kN)	V_i(kN)	$\theta_{II,i}$	≤0.12(D/Ch*R)
20	3	0.00055	4587.13	4587.13	630.91	0.0013	0.0857
19	3	0.00058	5065.93	9653.05	1293.40	0.0014	0.0857
18	3	0.00062	5065.93	14718.98	1921.62	0.0016	0.0857
17	3	0.00068	5065.93	19784.90	2515.58	0.0018	0.0857
16	3	0.00074	5065.93	24850.83	3075.27	0.0020	0.0857
15	3	0.00080	5065.93	29916.75	3600.70	0.0022	0.0857
14	3	0.00086	5065.93	34982.68	4091.86	0.0024	0.0857
13	3	0.00091	5065.93	40048.60	4548.75	0.0027	0.0857
12	3	0.00096	5065.93	45114.53	4971.37	0.0029	0.0857
11	3	0.00100	5065.93	50180.45	5359.73	0.0031	0.0857
10	3	0.00103	5092.48	55272.93	5715.68	0.0033	0.0857
9	3	0.00105	5092.48	60365.40	6037.18	0.0035	0.0857
8	3	0.00105	5092.48	65457.88	6324.23	0.0036	0.0857
7	3	0.00103	5092.48	70550.35	6576.84	0.0037	0.0857
6	3	0.00099	5092.48	75642.83	6795.00	0.0037	0.0857
5	3	0.00092	5119.51	80762.34	6979.69	0.0036	0.0857
4	3	0.00083	5119.51	85881.85	7129.75	0.0033	0.0857
3	3	0.00069	5119.51	91001.36	7245.18	0.0029	0.0857
2	3	0.00050	5119.51	96120.88	7325.98	0.0022	0.0857
1	4	0.00027	5741.76	101862.64	7377.77	0.0009	0.0857

4.5.17.2 Without shear deformation (case 1.2)

Using the equation 3-68 to calculate the $\theta_{II,i}$ and compare with the $\theta_{II,I,max}$ as shown in equation 3-69:

Δ_{ave} : from table 4-27.

$W_{k,i}$: from table 4-28.

V_i: from table 4-4 corrected with Vt_2/Vt_1 for case (1.2).

$\theta_{II,max}$ = 0.12 × (2.5 / 0.5 × 7) = 0.0857

$\theta_{II,20}$ = 0.00053 × 4587.13 / 634.64 × 3 = 0.0013 ≤ 0.0857

$\theta_{II,19}$ = 0.00056 × 9653.05 / 1301.05 × 3 = 0.0014 ≤ 0.0857

Table 4.36: Second Order Effect Case (1.2).

Story	hi (m)	Δ_{ave}(m)	$W_{k,i}$(kN)	$\sum w_{i,k}$(kN)	V_i(kN)	$\theta_{II,i}$	≤0.12(D/Ch*R)
20	3	0.00053	4587.13	4587.13	634.64	0.0013	0.0857
19	3	0.00056	5065.93	9653.05	1301.05	0.0014	0.0857
18	3	0.00060	5065.93	14718.98	1932.99	0.0015	0.0857
17	3	0.00066	5065.93	19784.90	2530.47	0.0017	0.0857
16	3	0.00071	5065.93	24850.83	3093.47	0.0019	0.0857
15	3	0.00077	5065.93	29916.75	3622.00	0.0021	0.0857
14	3	0.00083	5065.93	34982.68	4116.07	0.0024	0.0857
13	3	0.00089	5065.93	40048.60	4575.66	0.0026	0.0857
12	3	0.00094	5065.93	45114.53	5000.79	0.0028	0.0857
11	3	0.00098	5065.93	50180.45	5391.44	0.0030	0.0857
10	3	0.00101	5092.48	55272.93	5749.50	0.0032	0.0857
9	3	0.00102	5092.48	60365.40	6072.90	0.0034	0.0857
8	3	0.00103	5092.48	65457.88	6361.65	0.0035	0.0857
7	3	0.00101	5092.48	70550.35	6615.75	0.0036	0.0857
6	3	0.00097	5092.48	75642.83	6835.21	0.0036	0.0857
5	3	0.00091	5119.51	80762.34	7020.99	0.0035	0.0857
4	3	0.00081	5119.51	85881.85	7171.94	0.0032	0.0857
3	3	0.00068	5119.51	91001.36	7288.05	0.0028	0.0857
2	3	0.00049	5119.51	96120.88	7369.33	0.0021	0.0857
1	4	0.00027	5741.76	101862.64	7421.42	0.0009	0.0857

4.5.18 Comparing the Result

Table 4.37: Comparing the Result Between Case 1.1 and Case 1.2 in Y-Dir.

Parameter		Case 1.1.	Case 1.2	Percentage
Maximum Moment (KNm)	Shear Walls Right and left	12115.02	12032.33	**0.68%**
	Core Shear Wall	19260.68	19129.20	**0.68%**
Shear force for columns (kN)	Axes 2,7 Ext	81.52	80.29	**1.51%**
	Axes 3,6 Ext	61.61	60.68	**1.51%**
	Axes 2,7 Int	141.43	139.29	**1.51%**
	Axes 3,6 Int	134.85	132.81	**1.51%**
Maximum Deflection, Δmax (m)		0.0180	0.0178	**1.51%**
∑D (dm4/m)		12137.70	12504.66	**2.93%**
Period, Ty (sec)		1.89	1.86	**1.09%**

4.6 CALCULATIONS FOR EARTHQUAKE LOAD ACTION ON X-DIRECTION (CASE 2)

4.6.1 Calculations of Moment of Inertia for Structure's Member (I)

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

$I = b \times h^3/12 = 0.25 \times 0.60^3 / 12 = 0.0045\ m^4$.

Total I = 0.0045 × 0.35 (Inertia moment reducing coefficient) = 0.0016 m^4.

Core Shear wall: b_1=4.35 m, h_1=1.00 m, b_2=3.65 m, h_2=0.65.

$I_1 = b_1 \times h_1^3/12 = 4.35 \times 1.00^3 / 12 = 0.3625\ m^4$.

$I_2 = b_2 \times h_2^3/12 = 3.65 \times 0.65^3 / 12 = 0.0835\ m^4$.

Total I = I_1-I_2 = (0.3625 – 0.0835) ×0.50 (Inertia moment reducing coefficient) = 0.1395 m^4.

The Moment of Inertia for Structure's Member values are calculated and can be found in table 4-32.

Table 4.38: Moment of Inertia of Structure's Element (I).

Member		b (m)	h (m)	bh3/12 (m4)	Total I (m4)
Beam	Normal	0.25	0.6	0.0045	0.0016
	Coupled	0.25	0.7	0.0071	0.0025
	Cracked	0.5 × Ic.b		0.0036	0.0013
Columns	90×35	0.35	0.9	0.0213	0.0149
	40×70	0.7	0.4	0.0037	0.0026
	35×70	0.7	0.35	0.0025	0.0018
	35×60	0.6	0.35	0.0021	0.0015
Normal Shear Wall	Left and Right	4.5	0.35	0.0161	0.0080
	Center	0.35	4	1.8667	0.9333
Core Shear Wall	A1	4.35	1	0.3625	0.1395
	A2	3.65	0.65	0.0835	

4.6.2 Calculations of Columns Relative Rigidity (K_c)

$K_{c\ (C90\times35)\ 1st\ Floor} = I_{c,C90\times35} / h_1 = 148.84 / 4 = 37.21\ dm^4/m$.

The Kc values are calculated for all columns and can be found in table 4-33.

Table 4.39: Columns Relative Rigidity (Kc).

<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>C90×35</th><th>C40×70</th><th>C35×70</th><th>C35×60</th><th>C90×35</th><th>C40×70</th><th>C35×70</th><th>C35×60</th></tr>
<tr><td>11 to 20</td><td>3</td><td rowspan="4">148.84</td><td rowspan="2">-</td><td>-</td><td>15.01</td><td>49.61</td><td rowspan="2">-</td><td>-</td><td>5.00</td></tr>
<tr><td>6 to 10</td><td>3</td><td>17.51</td><td rowspan="3">-</td><td>49.61</td><td>5.84</td><td rowspan="3">-</td></tr>
<tr><td>2 to 5</td><td>3</td><td rowspan="2">26.13</td><td rowspan="2">-</td><td>49.61</td><td>8.71</td><td rowspan="2">-</td></tr>
<tr><td>1</td><td>4</td><td>37.21</td><td>6.53</td></tr>
</table>

4.6.3 Calculations of Beams Relative Rigidity (K_b)

$K_{b\ (B5)1st\ to\ 5th\ Floor} = I_{b5} / L = 15.75 / 7.125 = 2.211\ dm^4/m$.

The K_b values are calculated for all beams and can be found in table 4-34.

Table 4.40: Beams Relative Rigidity (Kb).

Floor	Ib (dm^4)	Length of span (m)					Kb (dm^4/m)				
		B1	B2	B3	B4	B5	B1	B2	B3	B4	B5
6 to 20	15.75	4.4	4.125	6.925	4.4	7.15	3.580	3.818	2.274	3.580	2.203
1 to 5		4.375	4.1	6.925	4.35	7.125	3.600	3.841	2.274	3.621	2.211

4.6.4 Calculations Value of K'

K' in 1st Floor (Axes 2,7 Ext) = $K_1 + K_3 / K_c = 3.6 + 3.84 / 6.53 = 4.188$.

K' in 10th Floor (Axes 2,7 Int) = $K_1 + K_2 + K_3 + K_4 / 2 \times Kc$

$= 3.58 + 3.58 + 3.58 + 3.58 / (2 \times 5.84) = 1.227$.

The K' values are calculated for all columns and can be found in table 4-35.

Table 4.41: Value of K'.

Floor	K'			
	Axes 2,7 Ext	Axes 3,6 Ext	Axes 2,7 Int	Axes 3,6 Int
11 to 20	1.479	0.123	1.431	1.156
7 to 10	1.268		1.227	0.991
6	1.271		1.232	1.161
2 to 5	0.854		0.829	0.669
1	4.188	0.164	4.154	3.959

4.6.5 Calculations of Rigidity of Columns ($\sum D_c$ and D_c)

a Value for C90×35 in 20th Floor Axes 3,6 Ext = $K' / (2 + K')$

$= 0.123 / (2+0.123) = 0.058$

a Value for C40×70 in 1st Floor Axes 2,7 Ext = $(0.5 + K') / (2 + K')$

$= (0.5 + 4.188) / (2+4.188) = 0.758$

D_c for C90×35 in 20th Floor Axes 3,6 Ext = $a \times K_c \times 12$

$$= 0.058 \times 49.61 \times 12 = 34.441 \text{ dm}^4/\text{m}.$$

D_c for C40×70 in 1st Floor Axes 2,7 Ext = $a \times K_c \times 12$

$$= 0.758 \times 6.53 \times 12 = 58.665 \text{ dm}^4/\text{m}.$$

The (a) values and D_c are calculated for all columns and can be found in table 4-36.

Table 4.42: Rigidity of Column and a Value.

Floor	a				Dc (dm⁴/m)			
	Axes 2,7 Ext	**Axes 3,6 Ext**	**Axes 2,7 Int**	**Axes 3,6 Int**	**Axes 2,7 Ext**	**Axes 3,6 Ext**	**Axes 2,7 Int**	**Axes 3,6 Int**
10 to 20	0.147	0.239	0.248	0.241	25.992	21.479	43.806	42.508
11	0.147	0.240	0.252	0.243	25.992	21.603	44.478	42.873
6 to 10	0.103	0.241	0.178	0.170	28.746	21.726	49.869	47.551
2 to 5	0.091		0.159	0.152	29.120		51.004	48.581
1	0.338	0.473	0.401	0.394	81.222	31.964	96.349	94.707

The summation of D_c in each floor:

$$20^{th} \text{ floor} = 25.517 \times 4 + 34.441 \times 4 + 25.037 \times 4 + 21.986 \times 4$$

$$= 427.926 \text{ dm}^4/\text{m}.$$

$$2^{nd} \text{ floor} = 31.286 \times 4 + 34.441 \times 4 + 30.630 \times 4 + 26.214 \times 4$$

$$= 490.279 \text{ dm}^4/\text{m}.$$

The summation of Dc in each floor values are calculated for all stories and can be found in table 4-37.

Table 4.43: Summation of the Rigidity of Columns in Each floor.

Floor	D For columns				No.	∑D (dm⁴/m)
	Axes 2,7 Ext	**Axes 3,6 Ext**	**Axes 2,7 Int**	**Axes 3,6 Int**		
11 to 20	25.517	34.441	25.037	21.986	4	427.926
7 to 10	23.286	34.441	26.624	23.200		430.203
6	23.328	34.441	26.695	25.719		440.731
2 to 5	31.286	34.441	30.630	26.214		490.279
1	59.395	137.059	59.291	58.665		1257.644

4.6.6 Calculation of Normal Shear Wall and Core Wall Rectifiers

a. Center shear wall connection:

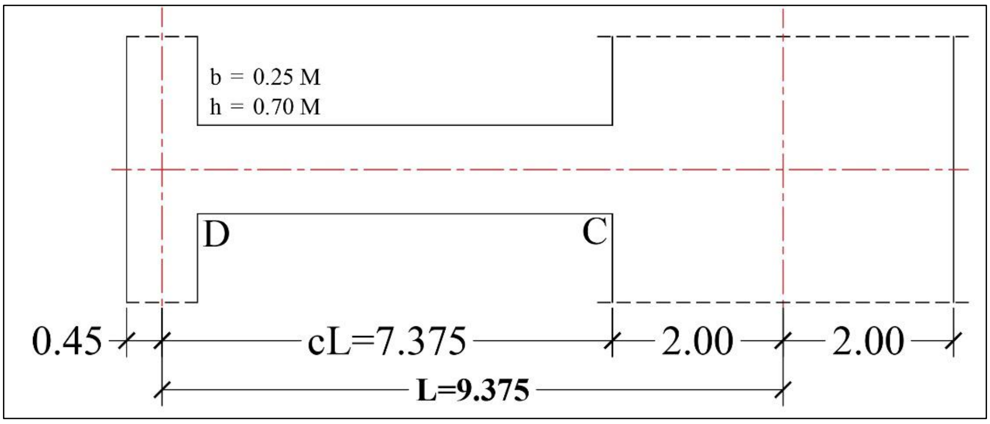


Figure 4.10: Coupled Beams Connected Between Shear Wall and Column in X-Direction.

From Figure (4-10)

s. $a_1L = 2.0$ m.

t. $cL = 7.375$ m.

u. $L = cL + a_1L = 7.375 + 2.0 = 9.375$ m.

v. $c = 7.375/9.375 = 0.787$

w. $a_1 = 2.0/9.375 = 0.213$

x. shape factor of rectangular = 5/6

y. $A' =$ Shape factor × Area $_{\text{coupled beam}} = 5/6 \times (0.25 \times 0.70) = 0.175$ m^2

z. $E/G = 2.5$

aa. $I_{bf,c} = I_{b,c} / c^2 = 0.0025 / 0.787^2 = 0.00404$ m^4

bb. $K_{bf,c} = I_{bf,c} / L = 0.00404 / 9.375 = 0.00058$ m^3

This data will use in calculation for Center shear wall rectifiers RC in cases (With shear deformations and Without shear deformations).

$\beta = (3 \times I_{bc} / cl^2 \times A') \times (E/G) = (3\times0.0025 / 7.375^2\times0.1458) \times 2.5 = 0.00236$

$K\beta = 1/(1+4\times\beta) = 1/ (1+4\times0.00236) = 0.991 \approx 1$

According to the Kβ value, which is approximately equal to 1, this means the results of Case 2.1-With shear deformations are approximately equal to the results of Case 2.2-Without shear deformations. Therefore, I will write the results of Case 2.1.

EX: rectifiers for story 20^{th}:

$K_c = 0.00496\ m^3$

$K_{c,o} = 0.$

$K_{c,u} = 0.005\ m^3$

$r_o = Kc / (K_{c,o} + K_{bf,c}) = 0.00496 / (0 + 0.00058) = 8.501$

$r_u = Kc / (K_{c,u} + K_{bf,c}) = 0.00496 / (0.005 + 0.00058) = 0.895$

$r_m = (ro + r_u) / 2 = (8.501 + 0.895) / 2 = 4.698 > 0.67$

$Rc = (6EI_b / c^3L)\ (2+c) \times K\beta$

$= (6+\times 0.0025/0.787^3 \times 9.375)\ \ \times(2 + 0.787) \times 0.991$

$= 45.38\ dm^4/m.$

The rectifiers of Center shear wall in each floor values are calculated for all stories and can be found in table 4-38.

Table 4.44: Calculation of Core rectifiers RC (Case 2).

Data	Story no.			
	20	3 to 19	2	1
Kc	0.00496	0.00496	0.00496	0.00372
$Kc_{,o}$	0	0.00496	0.00496	0.00496
$Kc_{,u}$	0.00496	0.00496	0.00372	0
ro	8.501	0.895	0.895	0.671
ru	0.895	0.895	1.153	6.376
rm	4.698	0.895	1.024	3.523
RC	45.38			

b. Normal shear wall (Right and left):

For Right and left shear wall it's appropriate to consider these as column widths, which run less than half the length of the shear wall perpendicular to this direction.

$b_{col} = B_{\text{(Beam width)}} + 2B_{W\ \text{(width of shear wall)}} \leq 0.5\ L_w$

$b_{col} = 0.25 + 2 \times 0.35 \leq 0.5 \times 4.5$

$b_{col} = 0.95 \leq 2.25$

Obtain Dc for the considered column with dimension (0.95 m × 0.35 m)

a. Moment of inertia (I_c):

$I_c = (b \times h^3 / 12) \times 0.7$

$I_c = (0.95 \times 0.35^3 / 12) \times 0.7 \times 10^4$

$I_c = 23.76\ dm^4$.

b. Column Relative Rigidity (K_c):

K_c for 1st floor = $I_c / L = 23.76 / 4 = 5.94\ dm^4/m$.

K_c for 2nd – 20th floor = $I_c / L = 23.76 / 3 = 7.92\ dm^4/m$.

c. Beam Relative Rigidity (K_b):

$K_b = I_{b,c} / L_{span} = 0.0025 / 4.375 \times 10^4 = 5.72\ dm^4/m$.

d. The Value of (K'):

K' for 1st floor = $K_b / K_c = 5.72 / 5.94 = 0.96$

K' for 2nd–20th floor = $K_{b1} + K_{b2}/2 \times K_c = 5.72+5.72 / 2 \times 7.92 = 0.72$

e. The Value of (a):

a for 1st floor = $(0.5 + K') / (2 + K') = (0.5 + 0.96) / (2 + 0.96) = 0.494$

a for 2nd–20th floor = $(K') / (2 + K') = (0.72) / (2+0.72) = 0.265$

f. The Column Rigidity (D_c):

D_c for 1st floor = $Kc \times a \times 12 = 5.94 \times 0.494 \times 12 = 35.19\ dm^4/m$.

Dc for 2nd–20th floor = $K_c \times a \times 12 = 7.92 \times 0.265 \times 12 = 25.2\ dm^4/m$.

g. The Summation of Column Rigidity ($\sum D_c$):

$\sum D_c$ for 1st floor = 35.19 × 8 = 281.5 dm4/m.

$\sum Dc$ for 2nd–20th floor = $25.2 \times 8 = 201.63\ dm^4/m$.

4.6.7 Core Shear Wall (End Core)

For end core shear wall, it’s appropriate to consider these as column widths, which run less than half the length of the shear wall perpendicular to this direction.

$b_{col} = B_{\text{(Beam width)}} + B_{w\ \text{(width of shear wall)}}$

$b_{col} = 0.25 + 0.35$

$b_{col} = 0.6$

Obtain Dc for the considered column with dimension (0.35 m × 0.60 m)

a. Moment of inertia (I_c):

$I_c = (b \times h^3 / 12) \times 0.7$

$I_c = (0.35 \times 0.6^3 / 12) \times 0.7 \times 10^4$

$I_c = 44.1\ dm^4$.

b. Column Relative Rigidity (K_c):

K_c for 1st floor $= I_c / L = 44.1 / 4 = 11.03\ dm^4/m$.

K_c for 2nd – 20th floor $= I_c / L = 44.1 / 3 = 14.7\ dm^4/m$.

c. Beam Relative Rigidity (K_b):

$K_{b1} = I_{b,c} / L_{span} = 0.0025 / 7.125 \times 10^4 = 3.51\ dm^4/m$.

$K_b = I_{b,c} / L_{span} = 0.0025 / 2.00 \times 10^4 = 12.505\ dm^4/m$.

d. The Value of (K'):

K' for 1st floor $= K_1 + K_3 / K_c = 3.51+12.505 / 11.03 = 1.45$

K' for 2nd–20th floor $= K_1 + K_2 + K_3 + K_4/2 \times K_c = 3.51+12.505 +3.51+12.505 / 14.7 = 1.09$

e. The Value of (a):

a for 1st floor = (0.5 + K') / (2 + K') = (0.5 + 1.45) / (2 + 1.45) = 0.566

a for 2nd–20th floor = (K') / (2 + K') = (1.09) / (2+1.09) = 0.353

f. The Column Rigidity (D_c):

D_c for 1st floor $= K_c \times a \times 12 = 11.03 \times 0.566 \times 12 = 74.82\ dm^4/m$.

D_c for 2nd–20th floor $= K_c \times a \times 12 = 14.7 \times 0.353 \times 12 = 62.21\ dm^4/m$.

g. The Summation of Column Rigidity ($\sum D_c$):

$\sum D_c$ for 1st floor $= 74.82 \times 4 = 299.3\ dm^4/m$.

$\sum D_c$ for 2nd–20th floor $= 62.21 \times 4 = 248.8\ dm^4/m$.

4.6.8 Calculation of Fictive shear stiffness D_{fi}.

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

   a. Top floor = D_{f20} = Rectifier value × 1.5 (dm^4/m).

   b. Intermediate floors = D_{f19} to D_{f2} = Rectifiers value (dm^4/m).

   c. Bottom floor = D_{f1} = Rectifier value × 2 (dm^4/m).

ii. Fictive shear stiffness for normal shear wall (right and left):

   a. Top floor = D_{f20} = 45.383 × 1.5 = 68.0275 (dm^4/m).

   b. Intermediate floors = D_{f19} to D_{f2} = 45.383 (dm^4/m).

   c. Bottom floor = D_{f1} = 45.383 × 2 = 90.766 (dm^4/m).

iii. The Summation of Fictive shear stiffness for shear wall in each story:

   a. $\sum D_{f,20}$ = 68.075 × 4 = 272.299 dm^4/m.

   b. $\sum D_{f,19,2}$ = 45.383 × 4 = 181.533 dm^4/m.

   c. $\sum D_{f,1}$ = 90.766 × 4 = 363.065 dm^4/m.

4.6.9 Calculation of the Rigidity of story ($\sum D$).

$\sum D_i = D_{ci} + D_{fci} + D_{fi}$

$\sum D_{20}$ = 427.926 + 407.191 + 272.299 = 1107.386 dm^4/m.

$\sum D_{19}$ = 427.926 + 407.191 + 181.533 = 1016.650 dm^4/m.

$\sum D$ values in each story are calculated for all stories and can be found in table 4-39.

Table 4.45: The Rigidity of story ($\sum D$) (Case2).

Story	∑D & ∑Dfi (dm^4/m)			∑D Total (dm^4/m)
	Columns	Considered column	Coupled beam connection	
20	427.926	450.456	272.299	1150.681
11 to 19	427.926	450.456	181.533	1059.915
7 to 10	430.203	450.456	181.533	1062.192
6	440.731	450.456	181.533	1072.720
2 to 5	490.279	450.456	181.533	1122.267
1	1257.644	580.790	363.065	2201.500

4.6.10 Calculations of the Coefficients and Constants of Equation Set (δi, i-1, δi,i and δi,o)

a. IP single Shear wall (Center) = 0.935 m^4.

b. IP single Core = 0.1395 m^4.

c. Taking E = 1.

d. $\sum EIP = 0.935 \times 2 + 0.1395 \times 2 = 2.15\ m^4$.

k. $F_i = 1/\sum D$

$F_{20} = (1 / 1150.68) \times 10^4 = 8.691\ 1/m^3$.

l. $f_i = h_i/(6\times\sum(EI_p)_i)$.

$f_{20} = 3 / (6 \times 2.15) = 0.2330\ 1/m^3$.

m. $\delta_{i,i-1} = f_{i-1} - F_{i-1}$.

$\delta_{20,19} = f_{19} - F_{19} = 0.233 - 8.691 = -9.202\ 1/m^3$

n. $\delta_{i,i} = 2\times f_i + 2\times f_{i-1} + F_i + F_{i-1}$.

$\delta_{20,20} = 2\times f_{20} + 2\times f_{19} + F_{20} + F_{19}$

$= 2\times0.233 + 2\times0.233 + 8.691 + 9.435$

$= 19.057\ 1/m^3$.

Table 4.46: The Coefficients And Constants of Equation Set (δi, i-1 , δi,i and δi,o) (Case 2).

Floor	h_i (m)	F_i ($1/m^3$)	f_i ($1/m^3$)	M_o (kNm)	$\delta_{i,i-1}$ ($1/m^3$)	$\delta_{i,i}$ ($1/m^3$)	$\delta_{i,o}$ (kNm)
20	3	8.691	0.2330	256.543	-	19.057	421.472
19	3	9.435	0.2330	782.474	-9.202	19.802	1153.574
18	3	9.435	0.2330	1563.859	-9.202	19.802	2242.849
17	3	9.435	0.2330	2586.763	-9.202	19.802	3669.814
16	3	9.435	0.2330	3837.253	-9.202	19.802	5414.987
15	3	9.435	0.2330	5301.395	-9.202	19.802	7458.885

Table 4.47: The Coefficients And Constants of Equation Set (δi, i-1 , δi,i and δi,o) (Case 2).

14	3	9.435	0.2330	6965.255	-9.202	19.802	9782.026
13	3	9.435	0.2330	8814.900	-9.202	19.802	12364.929
12	3	9.435	0.2330	10836.395	-9.202	19.802	15188.112
11	3	9.435	0.2330	13015.807	-9.202	19.781	18232.267
10	3	9.414	0.2330	15339.956	-9.181	19.761	21478.600
9	3	9.414	0.2330	17794.836	-9.181	19.761	24907.719
8	3	9.414	0.2330	20366.440	-9.181	19.761	28500.040
7	3	9.414	0.2330	23040.761	-9.181	19.669	32235.978
6	3	9.322	0.2330	25803.792	-9.089	19.165	36096.042
5	3	8.911	0.2330	28641.923	-8.678	18.753	40060.999
4	3	8.911	0.2330	31541.072	-8.678	18.753	44111.273
3	3	8.911	0.2330	34487.159	-8.678	18.753	48227.173
2	3	8.911	0.2330	37466.102	-8.678	14.540	61664.057
1	4	4.542	0.3107	41466.102	-4.232	5.164	37408.759

4.6.11 Calculations of the Matrix Coefficients and Load Vector.

a. Matrix[A] × $X_i = -\delta_{i,o}$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		Xi		δi,o
19.06	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X20		-421.47
-9.20	19.80	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X19		-1153.57
0.00	-9.20	19.80	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X18		-2242.85
0.00	0.00	-9.20	19.80	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X17		-3669.81
0.00	0.00	0.00	-9.20	19.80	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X16		-5414.99
0.00	0.00	0.00	0.00	-9.20	19.80	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X15		-7458.88
0.00	0.00	0.00	0.00	0.00	-9.20	19.80	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X14		-9782.03
0.00	0.00	0.00	0.00	0.00	0.00	-9.20	19.80	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X13		-12364.93
0.00	0.00	0.00	0.00	0.00	0.00	0.00	-9.20	19.80	-9.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X12		-15188.11
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-9.20	19.78	-9.18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	×	X11	=	-18232.27
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-9.18	19.76	-9.18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X10		-21478.60
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-9.18	19.76	-9.18	0.00	0.00	0.00	0.00	0.00	0.00	0.00		X9		-24907.72
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-9.18	19.76	-9.18	0.00	0.00	0.00	0.00	0.00	0.00		X8		-28500.04
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-9.18	19.67	-9.09	0.00	0.00	0.00	0.00	0.00		X7		-32235.98
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-9.09	19.16	-8.68	0.00	0.00	0.00	0.00		X6		-36096.04
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-8.68	18.75	-8.68	0.00	0.00	0.00		X5		-40061.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-8.68	18.75	-8.68	0.00	0.00		X4		-44111.27
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-8.68	18.75	-8.68	0.00		X3		-48227.17
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-8.68	14.54	-4.23		X2		-61664.06
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-4.23	5.16		X1		-37408.76

b. Matrix[A] $^{-1}$ × $-\delta_{i,o} = X_i$

X20	X19	X18	X17	X16	X15	X14	X13	X12	X11	X10	X9	X8	X7	X6	X5	X4	X3	X2	X1		δi,o		Xi
0.078	0.053	0.036	0.024	0.017	0.011	0.008	0.005	0.004	0.002	0.002	0.001	0.001	0.001	0.000	0.000	0.000	0.000	0.000	0.000		-421.472		-818.85
0.053	0.110	0.074	0.051	0.034	0.023	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		-1153.574		-1650.09
0.036	0.074	0.124	0.084	0.057	0.039	0.026	0.018	0.012	0.008	0.006	0.004	0.003	0.002	0.001	0.001	0.001	0.000	0.000	0.000		-2242.849		-2606.70
0.024	0.051	0.084	0.131	0.089	0.060	0.041	0.028	0.019	0.013	0.009	0.006	0.004	0.003	0.002	0.001	0.001	0.001	0.001	0.000		-3669.814		-3715.64
0.017	0.034	0.057	0.089	0.134	0.091	0.062	0.042	0.029	0.019	0.013	0.009	0.006	0.004	0.003	0.002	0.001	0.001	0.001	0.001		-5414.987		-4990.36
0.011	0.023	0.039	0.060	0.091	0.136	0.092	0.063	0.042	0.029	0.020	0.013	0.009	0.006	0.004	0.003	0.002	0.001	0.001	0.001		-7458.885		-6434.88
0.008	0.016	0.026	0.041	0.062	0.092	0.136	0.093	0.063	0.043	0.029	0.020	0.013	0.009	0.006	0.004	0.003	0.002	0.002	0.001		-9782.026		-8046.57
0.005	0.011	0.018	0.028	0.042	0.063	0.093	0.137	0.093	0.063	0.043	0.029	0.020	0.014	0.009	0.006	0.004	0.003	0.003	0.002		-12364.929		-9817.86
0.004	0.007	0.012	0.019	0.029	0.042	0.063	0.093	0.137	0.093	0.063	0.043	0.029	0.020	0.014	0.009	0.007	0.005	0.004	0.003		-15188.112		-11737.20
0.002	0.005	0.008	0.013	0.019	0.029	0.043	0.063	0.093	0.137	0.093	0.063	0.043	0.029	0.020	0.014	0.010	0.007	0.005	0.004	×	-18232.267	=	-13789.42
0.002	0.003	0.006	0.009	0.013	0.020	0.029	0.043	0.063	0.093	0.137	0.093	0.063	0.043	0.030	0.020	0.014	0.010	0.008	0.007		-21478.600		-15960.29
0.001	0.002	0.004	0.006	0.009	0.013	0.020	0.029	0.043	0.063	0.093	0.137	0.094	0.064	0.044	0.030	0.021	0.015	0.012	0.010		-24907.719		-18222.31
0.001	0.002	0.003	0.004	0.006	0.009	0.013	0.020	0.029	0.043	0.063	0.094	0.138	0.094	0.065	0.044	0.031	0.022	0.018	0.014		-28500.040		-20546.46
0.001	0.001	0.002	0.003	0.004	0.006	0.009	0.014	0.020	0.029	0.043	0.064	0.094	0.139	0.095	0.065	0.045	0.033	0.026	0.021		-32235.978		-22895.42
0.000	0.001	0.001	0.002	0.003	0.004	0.006	0.009	0.014	0.020	0.030	0.044	0.065	0.095	0.141	0.096	0.067	0.049	0.038	0.031		-36096.042		-25243.63
0.000	0.000	0.001	0.001	0.002	0.003	0.004	0.006	0.009	0.014	0.020	0.030	0.044	0.065	0.096	0.144	0.101	0.073	0.057	0.047		-40060.999		-27610.94
0.000	0.000	0.001	0.001	0.001	0.002	0.003	0.004	0.007	0.010	0.014	0.021	0.031	0.045	0.067	0.101	0.150	0.109	0.086	0.070		-44111.273		-29810.49
0.000	0.000	0.000	0.001	0.001	0.001	0.002	0.003	0.005	0.007	0.010	0.015	0.022	0.033	0.049	0.073	0.109	0.163	0.128	0.105		-48227.173		-31729.93
0.000	0.000	0.000	0.001	0.001	0.001	0.002	0.003	0.004	0.005	0.008	0.012	0.018	0.026	0.038	0.057	0.086	0.128	0.190	0.156		-61664.057		-33204.22
0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.002	0.003	0.004	0.007	0.010	0.014	0.021	0.031	0.047	0.070	0.105	0.156	0.322		-37408.759		-34454.91

4.6.12 Calculations of the Moment Superposition and Distribution to the Core, Shear Walls

$Mp_{total,20} = M_{o,20} + X_{20}$ = 256.54 + (-818.849) = -562.305 kNm

$Mp_{total,19} = M_{o,19} + X_{19}$ = 783.47 + (-1650.09) = -867.616 kNm

$Mp_{total,18} = M_{o,18} + X_{18}$ = 1563.86 + (-2606.696) = -1042.837 kNm

Moment for Shear wall (Center):

$M_{\text{shear wall (Center)},20} = M_{Ptotal} \times I_{p,\text{single shear}} / \sum IP$

= -562.305 × 0.933 / 2.15 = -244.598 kNm

$M_{\text{shear wall (Center)},19} = M_{Ptotal} \times I_{p,\text{single shear}} / \sum IP$

= -867.616 × 0.933 / 2.15 = -377.406 kNm

Moment for Core:

$M_{\text{shear wall (Center)},20} = M_{Ptotal} \times I_{p,\text{single core}} / \sum IP$

= -562.305 × 0.1395 / 2.15 = -36.555 kNm

$M_{\text{shear wall (Center)},19} = M_{Ptotal} \times I_{p,\text{single core}} / \sum IP$

= -867.616 × 0.1395 / 2.15 = -56.402 kNm

The Moment superposition And Distribution to Shear walls (Center) and Core shear wall values in each story are calculated for all stories and can be found in table 4-41 for (Case2).

Table 4.48: Moment Superposition and Distribution to The Core, Shear Walls (Case 2).

Floor	Mo (kNm)	Xi (kNm)	M total Shear wall (kNm)	M Shear wall (Center) (kNm)	M Core (kNm)
20	256.543	-818.849	-562.305	-244.598	-36.555
19	782.474	-1650.090	-867.616	-377.406	-56.402
18	1563.859	-2606.696	-1042.837	-453.626	-67.793
17	2586.763	-3715.643	-1128.880	-491.054	-73.387
16	3837.253	-4990.359	-1153.106	-501.592	-74.961
15	5301.395	-6434.877	-1133.483	-493.056	-73.686
14	6965.255	-8046.570	-1081.315	-470.363	-70.294
13	8814.900	-9817.861	-1002.962	-436.280	-65.201
12	10836.395	-11737.200	-900.805	-391.843	-58.560
11	13015.807	-13789.416	-773.609	-336.514	-50.291
10	15339.956	-15960.288	-620.331	-269.839	-40.327
9	17794.836	-18222.308	-427.471	-185.946	-27.789
8	20366.440	-20546.462	-180.021	-78.308	-11.703
7	23040.761	-22895.423	145.338	63.221	9.448
6	25803.792	-25243.631	560.161	243.665	36.415
5	28641.923	-27610.938	1030.985	448.470	67.023
4	31541.072	-29810.487	1730.586	752.791	112.502
3	34487.159	-31729.929	2757.231	1199.372	179.243
2	37466.102	-33204.221	4261.881	1853.883	277.058
1	41466.102	-34454.910	7011.193	3049.810	455.786

4.6.13 Calculations of Shear Forces for Columns, Shear Wall and Core

a. Shear Force in each story:

$\sum T_i = \Delta X_i / h_i = X_{i+1} - X_i / h_i$

$\sum T_{20} = 0 – (-818.849) / 3 = 272.95$ kN.

$\sum T_{19} = (-818.849) – (-1650.09) / 3 = 277.08$ kN.

b. Shear Force for columns in each story:

$T_{20,\ \text{Axes 2,7 Ext}} = 272.95 \times (25.517 / 1107.386) = 6.053$ kN.

$T_{19,\ \text{Axes 2,7 Ext}} = 277.08 \times (25.517 / 1016.650) = 6.671$ kN.

c. Shear Force for Core shear walls in each story:

$T_{f20,\ Center}$ = 272.95 × (49.50 / 1107.386) = 16.148 kN.

$T_{f19,\ Center}$ = 277.08 × (49.50 / 1016.650) = 11.864 kN.

d. Shear Force for shear walls in each story:

$T_{f20,\ Center}$ = 272.95 × (68.075 / 1107.386) = 14.756 kN.

$T_{f19,\ Center}$ = 277.08 × (45.383 / 1016.650) = 16.262 kN.

The Shear Forces for Columns, Core and shear wall values in each story are calculated for all stories and can be found in table 4-42 for (Case2).

Table 4.49: Shear Forces for Columns, shear wall and Core Wall (Case 2).

Floor	h floor (m)	∑T	Tcol				Tfi Normal shear wall Center	Tfi Core shear wall
			Axes 2,7 Ext	Axes 3,6 Ext	Axes 2,7 Int	Axes 3,6 Int		
20	3	272.950	6.053	8.869	6.448	5.662	16.148	14.756
19	3	277.080	6.671	9.003	6.545	5.748	11.864	16.262
18	3	318.869	7.677	10.361	7.532	6.614	13.653	18.714
17	3	369.649	8.899	12.011	8.732	7.668	15.828	21.695
16	3	424.905	10.230	13.807	10.037	8.814	18.193	24.938
15	3	481.506	11.592	15.646	11.374	9.988	20.617	28.260
14	3	537.231	12.934	17.457	12.691	11.144	23.003	31.530
13	3	590.431	14.215	19.185	13.947	12.248	25.281	34.652
12	3	639.780	15.403	20.789	15.113	13.271	27.394	37.549
11	3	684.072	16.469	22.228	16.159	14.190	29.290	40.148
10	3	723.624	15.864	23.463	18.138	15.805	30.918	42.378
9	3	754.007	16.530	24.448	18.899	16.469	32.216	44.158
8	3	774.718	16.984	25.120	19.418	16.921	33.101	45.371
7	3	782.987	17.165	25.388	19.626	17.102	33.454	45.855
6	3	782.736	17.022	25.130	19.479	18.767	33.115	45.390
5	3	789.102	21.998	24.216	21.537	18.432	31.910	43.739
4	3	733.183	20.439	22.500	20.010	17.125	29.649	40.640
3	3	639.814	17.836	19.635	17.462	14.945	25.873	35.464
2	3	491.431	13.700	15.081	13.412	11.479	19.873	27.240
1	4	312.672	8.436	19.466	8.421	8.332	12.891	10.627

4.6.14 Calculations of the Coupled Beam End Moments.

i. Coupled beam end moment for shear wall (Center):

$M_{20,u} = 1/3 \times 16.148 \times 3 = 16.148$ kNm

$M_{20,o} = 2/3 \times 16.148 \times 3 = 32.296$ kNm

$M_{19,u} = 1/2 \times 11.864 \times 3 = 17.796$ kNm

j. Coupled beam end moment for Core:

$M_{20,u} = 1/3 \times 14.756 \times 3 = 14.756$ kNm

$M_{20,o} = 2/3 \times 14.756 \times 3 = 29.512$ kNm

$M_{19,u} = 1/2 \times 16.262 \times 3 = 24.393$ kNm

The Coupled beam end moment for shear wall and Core wall values in each story are calculated for all stories and can be found in table 4-43 for (Case2).

Table 4.50: Coupled Beam End Moments (Case 2).

Floor	h_i (m)	Shear wall center		Core	
		$M_{i,u}$	$M_{i,o}$	$M_{i,u}$	$M_{i,o}$
20	3	16.148	32.296	14.756	29.512
19	3	17.796	33.944	24.393	39.148
18	3	20.480	38.276	28.072	52.464
17	3	23.741	44.221	32.542	60.614
16	3	27.290	51.032	37.406	69.948
15	3	30.926	58.216	42.389	79.796
14	3	34.505	65.430	47.295	89.684
13	3	37.921	72.426	51.978	99.273
12	3	41.091	79.012	56.323	108.301
11	3	43.936	85.027	60.222	116.545
10	3	46.376	90.312	63.568	123.790
9	3	48.323	94.700	66.237	129.804
8	3	49.651	97.974	68.056	134.292
7	3	50.181	99.832	68.782	136.838
6	3	49.672	99.853	68.085	136.868
5	3	47.866	97.538	65.609	133.694
4	3	44.474	92.339	60.959	126.568
3	3	38.810	83.284	53.196	114.156
2	3	29.809	68.619	40.859	94.056
1	4	25.783	55.592	21.254	62.113

k. Corrected shear wall moments at bottom and top (Center):

$$M_{Bottom,1st} = 3049.81 + 25.783 = 3075.593 \text{ kNm}$$

$$M_{Top,1st} = 1853.883 – 25.783 = 1828.100 \text{ kNm}$$

$$M_{Bottom,2nd} = 1853.883 + 29.809 = 1883.692 \text{ kNm}$$

$$M_{Top,2nd} = 1199.372 – 29.809 = 1169.563 \text{ kNm}$$

l. Corrected Core moments at bottom and top:

$$M_{Bottom,1st} = 455.786 + 21.254 = 477.039 \text{ kNm}$$

$$M_{Top,1st} = 277.058 – 21.254 = 255.804 \text{ kNm}$$

$$M_{Bottom,2nd} = 277.058 + 40.859 = 317.917 \text{ kNm}$$

$$M_{Top,2nd} = 179.243 – 40.859 = 138.384 \text{ kNm}$$

The corrected normal shear wall and core moments at bottom and top values in each story are calculated for all stories and can be found in table 4-44 for (Case2).

Table 4.51: Corrected Shear Wall Moments at Bottom and Top (Case 2).

Floor	Shear wall-center				Core			
	M kNm	Mi,u	M1 Bottom	M2 Upper	M kNm	Mi,u	M1 Bottom	M2 Upper
20	-244.60	16.15	-228.45	-32.30	-36.55	14.76	-21.80	-29.51
19	-377.41	17.80	-359.61	-262.39	-56.40	24.39	-32.01	-60.95
18	-453.63	20.48	-433.15	-397.89	-67.79	28.07	-39.72	-84.47
17	-491.05	23.74	-467.31	-477.37	-73.39	32.54	-40.84	-100.34
16	-501.59	27.29	-474.30	-518.34	-74.96	37.41	-37.56	-110.79
15	-493.06	30.93	-462.13	-532.52	-73.69	42.39	-31.30	-117.35
14	-470.36	34.50	-435.86	-527.56	-70.29	47.29	-23.00	-120.98
13	-436.28	37.92	-398.36	-508.28	-65.20	51.98	-13.22	-122.27
12	-391.84	41.09	-350.75	-477.37	-58.56	56.32	-2.24	-121.52
11	-336.51	43.94	-292.58	-435.78	-50.29	60.22	9.93	-118.78
10	-269.84	46.38	-223.46	-382.89	-40.33	63.57	23.24	-113.86
9	-185.95	48.32	-137.62	-318.16	-27.79	66.24	38.45	-106.56
8	-78.31	49.65	-28.66	-235.60	-11.70	68.06	56.35	-95.85
7	63.22	50.18	113.40	-128.49	9.45	68.78	78.23	-80.49
6	243.67	49.67	293.34	13.55	36.42	68.09	104.50	-58.64
5	448.47	47.87	496.34	195.80	67.02	65.61	132.63	-29.19
4	752.79	44.47	797.26	404.00	112.50	60.96	173.46	6.06
3	1199.37	38.81	1238.18	713.98	179.24	53.20	232.44	59.31
2	1853.88	29.81	1883.69	1169.56	277.06	40.86	317.92	138.38
1	3049.81	25.78	3075.59	1828.10	455.79	21.25	477.04	255.80

4.6.15 Calculation of the First Natural Time-Period (T_{1x}) By Using Rayleigh Method

The periods will be in the X-direction since the base shear force operates in the X-direction of the structure. The shear forces operating on the frames and the total ∑D must be used by converting them to their true values. E = 1 was previously used. Now, (E/h_i^2) values are multiplied by ∑D using E-concrete (33000 N/mm²).

$\sum D_{20j} = 1150.681 \times 10^3 \times 33000 \times 10^{-4} / 3^2 = 421916.352$ kN /m.

$\delta_i = (\sum T_i)/ (\sum D_{ij})$ in (m).

$\delta_1 = 312.672/454059.347 = 0.000689$ m.

$\delta_2 = 491.431/411497.961 = 0.001294$ m.

$d_i = \sum \delta i$ in (m).

$d_1 = \delta_0 + \delta_1 = 0.000689$ m

$d_2 = \delta_1 + \delta_2 = d_1 + \delta_2 = 0.000689 + 0.001194 = 0.001883$ m

The Stiffness, Displacement and drift values in each story are calculated for all stories and can be found in table 4-45 for (Case 2).

Table 4.52: Displacement, Drift and Stiffness of Story Values (Case 2).

Floor	h floor (m)	Pi (kN)	∑T (kN)	∑D dm⁴/m	∑Dij kN / m	δ (m)	di (m)
20	3	85.51	272.950	1150.681	421916.352	0.000647	0.028692
19	3	89.80	277.080	1059.915	388635.360	0.000713	0.028045
18	3	85.15	318.869	1059.915	388635.360	0.000820	0.027333
17	3	80.51	369.649	1059.915	388635.360	0.000951	0.026512
16	3	75.86	424.905	1059.915	388635.360	0.001093	0.025561
15	3	71.22	481.506	1059.915	388635.360	0.001239	0.024468
14	3	66.57	537.231	1059.915	388635.360	0.001382	0.023229
13	3	61.93	590.431	1059.915	388635.360	0.001519	0.021846
12	3	57.28	639.780	1059.915	388635.360	0.001646	0.020327
11	3	52.64	684.072	1059.915	388635.360	0.001760	0.018681
10	3	48.25	723.624	1062.192	389470.324	0.001858	0.016921
9	3	43.58	754.007	1062.192	389470.324	0.001936	0.015063
8	3	38.91	774.718	1062.192	389470.324	0.001989	0.013127

Table 4.53: Displacement, Drift and Stiffness of story values (Case 2).

7	3	34.24	782.987	1062.192	389470.324	0.002010	0.011137
6	3	29.57	782.736	1072.720	393330.497	0.001990	0.009127
5	3	25.03	789.102	1122.267	411497.961	0.001918	0.007137
4	3	20.34	733.183	1122.267	411497.961	0.001782	0.005219
3	3	15.65	639.814	1122.267	411497.961	0.001555	0.003438
2	3	10.95	491.431	1122.267	411497.961	0.001194	0.001883
1	4	7.02	312.672	2201.500	454059.347	0.000689	0.000689

$M_i = W_i / g$ (kN/m/sec^2) ; g=9.81 (m/sec^2)

$M_1 = W_1 / 9.81 = 5741.76 / 9.81 = 585.297$ (kN/m/sec^2)

$M_2 = W_2 / 9.81 = 5119.51 / 9.81 = 521.867$ (kN/m/sec^2)

$M_i d_i^2 = M_i \times d_i^2$

$M_1 d_1^2 = M_1 \times d_1^2 = 585.297 \times 0.000689^2 = 0.000278$ kNm Sec²

$M_2 d_2^2 = M_2 \times d_2^2 = 521.867 \times 0.001883^2 = 0.001850$ kNm Sec²

$P_i d_i = P_i \times d_i$

$P_1 d_1 = P_1 \times d_1 = 6.98 \times 0.000689 = 0.004833$ kNm

$P_2 d_2 = P_2 \times d_2 = 10.89 \times 0.001883 = 0.020621$ kNm

The Mass, $M_i d_i^2$ and $P_i d_i$ values in each story are calculated for all stories and can be found in table 4-46 for (Case 2).

Table 4.54: Midi and Pidi Value (case 2).

Floor	Wi (kN)	Mi (kN/g)	Midi² kNmSec²	Pidi (kNm)
20	4587.13	467.597	0.384951	2.453616
19	5065.93	516.404	0.406177	2.518366
18	5065.93	516.404	0.385788	2.327397
17	5065.93	516.404	0.362974	2.134393
16	5065.93	516.404	0.337397	1.939100
15	5065.93	516.404	0.309151	1.742515
14	5065.93	516.404	0.278635	1.546392
13	5065.93	516.404	0.246458	1.352897

Table 4.55: Midi and Pidi Value (case 2).

12	5065.93	516.404	0.213372	1.164403
11	5065.93	516.404	0.180211	0.983336
10	5092.48	519.111	0.148625	0.816348
9	5092.48	519.111	0.117777	0.656382
8	5092.48	519.111	0.089447	0.510731
7	5092.48	519.111	0.064392	0.381336
6	5092.48	519.111	0.043244	0.269889
5	5119.51	521.867	0.026583	0.178665
4	5119.51	521.867	0.014217	0.106161
3	5119.51	521.867	0.006167	0.053786
2	5119.51	521.867	0.001850	0.020621
1	5741.76	585.297	0.000278	0.004833
		10383.551	3.617696	21.161168

$\omega_1^2{}_{(X)} = \sum P_i d_i / \sum M_i d_i^2 = 21.1611 / 3.618 = 5.65$

$\omega_{1\ (X)} = 2.4185$ rad/sec.

$T_{1(X)} = (2 \times 3.14) / 2.4185 = 2.60$ Sec.

4.6.16 Calculation of the Real Base Shear Force.

Calculate the real base shear force for structure in Second degree earthquake zone with the following Characteristic in Cases (With shear deformation and Without shear deformation):

a. Effective Ground Acceleration Coefficient (A_o) = 0.30
b. Building Importance Factor (I) For Office building = 1
c. Local Site Class = Z2
d. Spectrum Coefficient = (T_A = 0.15 sec), (T_B = 0.40 sec).
e. Structural Behavior Factors (R) = 7.
f. Damping ratio (ξ) = 5%

4.6.16.1 With shear deformation (case 1.1)

By using Erzincan 1992 N-S chart, intersect the T-period value (**2.60**) with Damping ratio (ξ=5%) to obtain the (Sa/a_{max}) value.

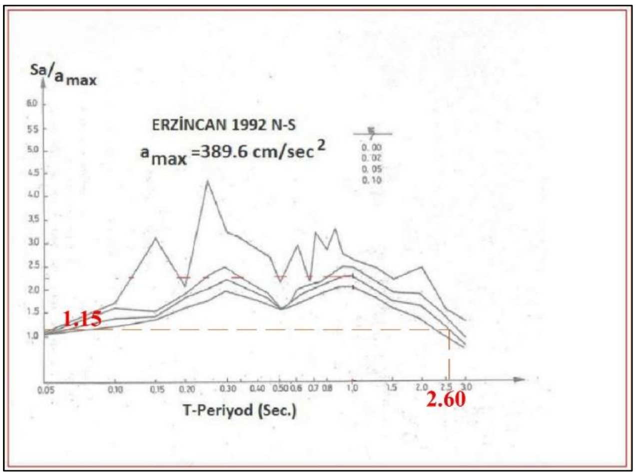


Figure 4.11: Erzincan 1992 N-S Chart (Case2).

Sa/a_{max} value = 1.15

$T_{1(X)} = 2.60 > T_B = 0.40$

$S_{(T1,\,X)} = 2.5\,(0.4 / 2.60)^{0.8} = 0.56$

$A_{(T1,\,X)} = A_o \times g \times S_{(T)} \times I$

$A_{(T1,\,X)} = 0.30 \times 9.81 \times 0.56 \times 1$

$A_{(T1,\,X)} = 1.648$

$V_t = \sum M_i \times A_{(T1,\,X)} / R$

$V_t = 10383.551 \times 1.648 / 7$

Total Equivalent Seismic Load (base shear), V_t, Appling on the building in earthquake direction:

$V_t = 2444$ kN.

Base Shear Force in case of using Erzincan 1992 N-S Acceleration Spectrum:

$V_{t2} = Vt \times ((Sa/a_{max}) / S_{(T1,\,X)})$

$V_{t2} = 2444 \times (1.15 / 0.56)$

$V_{t2} = 5020$ kN.

Vt2 / Vt1 Ratio = 5020/1000 = **5.0**

This ratio will be utilized to rectify the shear forces and moments because the real base shear force is not 1000 kN as assumed.

4.6.17 Correcting the Moment, Shear Force, Displacement and Drift

To correct the Values of Moment, Shear force, Displacement and Drift by multiplied its by Vt_2 / Vt_1 ratio which is 5.0.

m. Total moment $_{\text{shear walls 1st story}}$ = 7011.193 × 5.0 = 35198.80 kNm

n. Single moment 1st story:

Shear walls Center = 3049.810 × 5.0 = 15311.19 kNm

Core walls = 455.786 × 5.0 = 2288.21 kNm

o. Shear Forces $_{\text{Total 5th}}$ = 789.102 × 5.0 = 3961.59 kN

p. Corrected shear wall moments bottom 1st floor:

Center = 3075.593 × 5.0 = 15440.62 kN

Core wall = 477.039 × 5.0 = 2394.92 kN

q. Maximum drift $_{7th}$ = 0.002010 × 5.0 × 7 / 3 = 0.02355

Check Maximum drift 7th according to (Specification for Buildings to be Built in Seismic Zones 2007 item 2.10.1.3).

0.02355 > 0.02 Not O.K.

r. Maximum Displacement $_{20th}$ = 0.02869 ×10^3 × 5.0 = 144 mm

Check Maximum Displacement $_{20th}$ with $\Delta_{\text{max-all}}$ = 0.002 × H_B

$\Delta_{\text{max-all}}$ = 0.002 × 61 = 122 mm.

144 mm > 122 mm Not O.K.

Corrected shear walls and core moments after multiplying it by the Vt2/Vt1 ratio in each story are calculated for all stories and can be found in table 4-47 and figure (4-13 and 4-14) for (Case 2).

Table 4.56: Corrected shear Wall and Core Moments with Vt2/Vt1 Ratio Case (2).

Floor	Shear wall Center		Core	
	M_1 Bottom	M_2 Upper	M_1 Bottom	M_2 Upper
20	-1146.91	-162.14	-109.44	-148.16
19	-1805.38	-1317.32	-160.70	-305.98
18	-2174.55	-1997.53	-199.42	-424.09
17	-2346.08	-2396.56	-205.06	-503.72
16	-2381.17	-2602.28	-188.54	-556.22
15	-2320.06	-2673.43	-157.12	-589.14
14	-2188.17	-2648.55	-115.47	-607.37
13	-1999.91	-2551.78	-66.38	-613.86
12	-1760.91	-2396.58	-11.23	-610.09
11	-1468.85	-2187.77	49.86	-596.33
10	-1121.87	-1922.25	116.68	-571.61
9	-690.92	-1597.29	193.02	-534.99
8	-143.87	-1182.79	282.91	-481.18
7	569.32	-645.06	392.75	-404.07
6	1472.66	68.02	524.63	-294.38
5	2491.79	982.99	665.86	-146.56
4	4002.56	2028.21	870.84	30.44
3	6216.14	3584.45	1166.93	297.74
2	9456.84	5871.64	1596.06	694.74
1	15440.62	9177.74	2394.92	1284.23

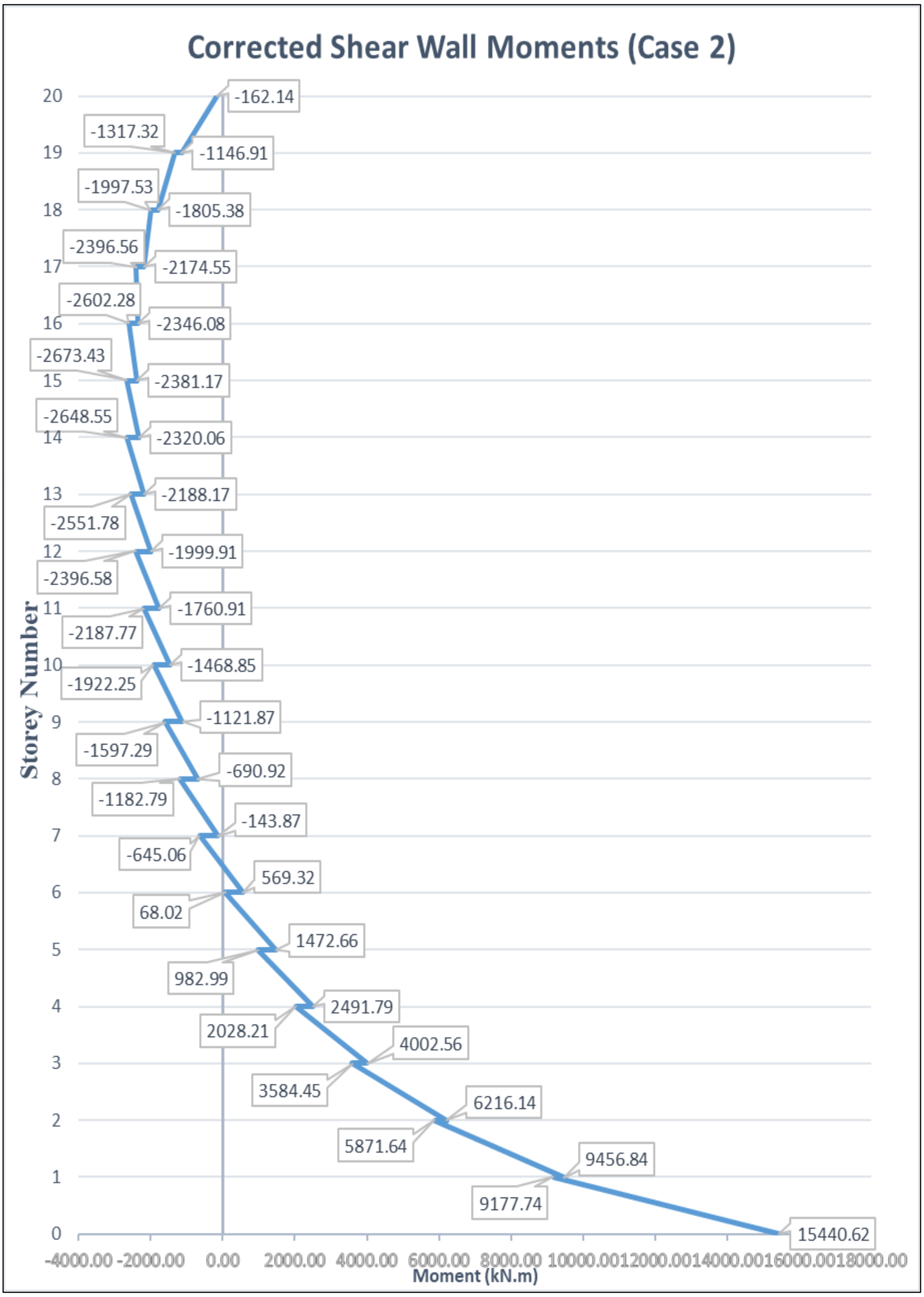


Figure 4.12: Corrected Shear Wall Moments (Case 2).

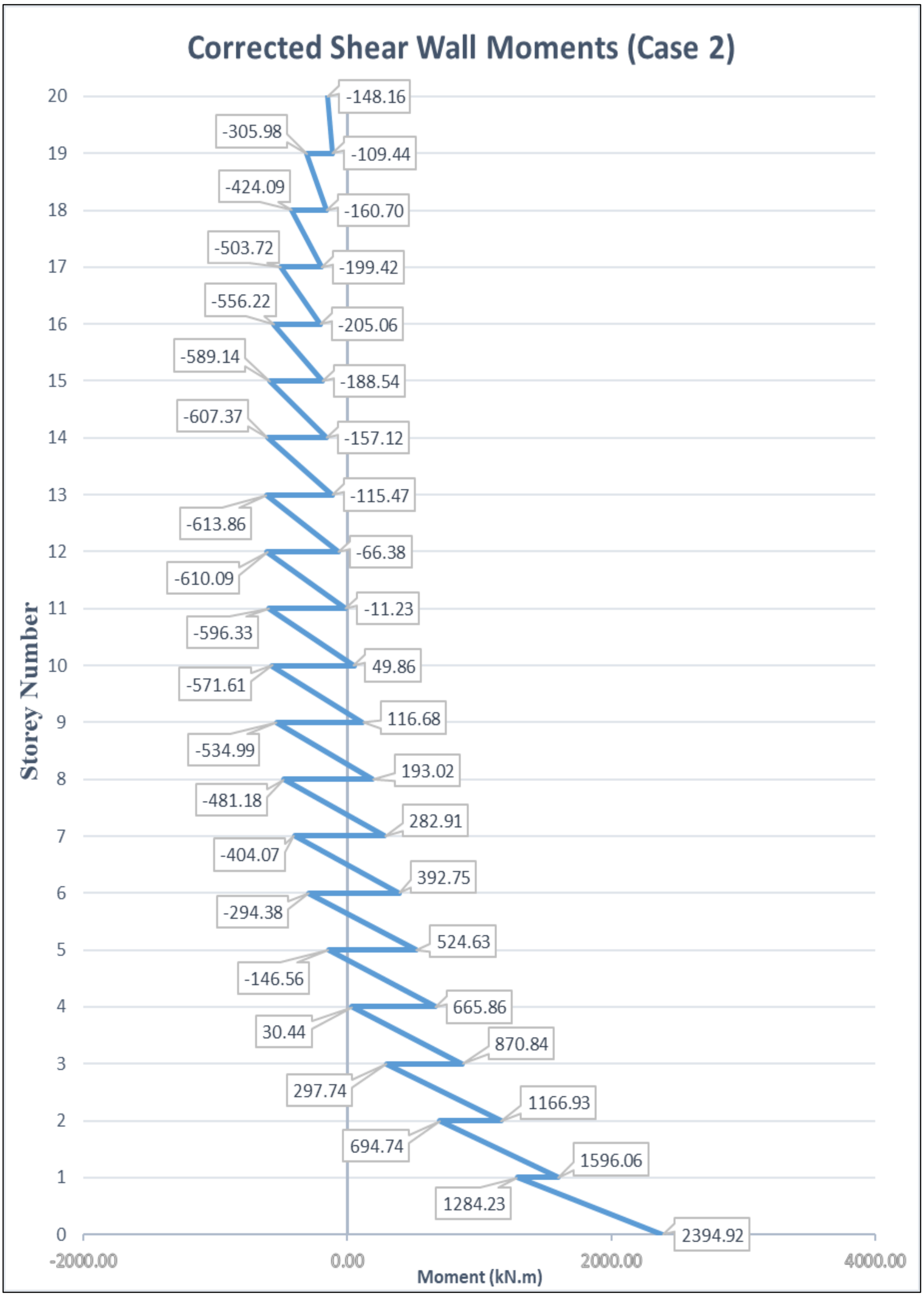


Figure 4.13: Corrected Core Moments (Case 2).

4.6.18 The Second Order Indicator Value $\theta_{\text{II},i}$

Using the equation 3-68 to calculate the $\theta_{\text{II},i}$ and compare with the $\theta_{\text{II},I,max}$ as shown in equation 3-69:

Δ_{ave} : from table 4-47.

$W_{k,i}$: from table 4-48.

V_i: from table 4-4 corrected with Vt_2/Vt_1 for case (2).

$\theta_{\text{II},max} = 0.12 \times (2.5 / 0.5 \times 7) = 0.0857$

$\theta_{\text{II},20} = 0.00065 \times 4587.13 / 429.31 \times 3 = 0.0023 \leq 0.0857$

$\theta_{\text{II},19} = 0.00071 \times 9653.05 / 880.12 \times 3 = 0.0026 \leq 0.0857$

Table 4.57: Second Order Effect Case (1.1).

Story	h(m)	Δave(m)	Wk,i(kN)	∑wi,k(kN)	Vi(kN)	θıı,i	≤0.12(D/Ch*R)
20	3	0.00065	4587.13	4587.13	429.31	0.0023	0.0857
19	3	0.00071	5065.93	9653.05	880.12	0.0026	0.0857
18	3	0.00082	5065.93	14718.98	1307.61	0.0031	0.0857
17	3	0.00095	5065.93	19784.90	1711.79	0.0037	0.0857
16	3	0.00109	5065.93	24850.83	2092.64	0.0043	0.0857
15	3	0.00124	5065.93	29916.75	2450.18	0.0050	0.0857
14	3	0.00138	5065.93	34982.68	2784.40	0.0058	0.0857
13	3	0.00152	5065.93	40048.60	3095.30	0.0066	0.0857
12	3	0.00165	5065.93	45114.53	3382.89	0.0073	0.0857
11	3	0.00176	5065.93	50180.45	3647.15	0.0081	0.0857
10	3	0.00186	5092.48	55272.93	3889.37	0.0088	0.0857
9	3	0.00194	5092.48	60365.40	4108.14	0.0095	0.0857
8	3	0.00199	5092.48	65457.88	4303.47	0.0101	0.0857
7	3	0.00201	5092.48	70550.35	4475.36	0.0106	0.0857
6	3	0.00199	5092.48	75642.83	4623.82	0.0109	0.0857
5	3	0.00192	5119.51	80762.34	4749.49	0.0109	0.0857
4	3	0.00178	5119.51	85881.85	4851.60	0.0105	0.0857
3	3	0.00155	5119.51	91001.36	4930.15	0.0096	0.0857
2	3	0.00119	5119.51	96120.88	4985.13	0.0077	0.0857
1	4	0.00069	5741.76	101862.64	5020.37	0.0035	0.0857

4.7 LOAD COMBINATIONS

Superpose the total moment of shear wall, the shear force for all stories and displacement of 20^{th} storey in Y-dir. with X-dir. for two cases (with shear deformation and without shear deformations) after correcting it by Vt_2/Vt_1 ratio by using load combination mention below:

a. Ex + 0.3 Ey　　b. Ex - 0.3 Ey　　c. - Ex + 0.3 Ey

d. - Ex - 0.3 Ey　　e. 0.3 Ex + Ey　　f. 0.3 Ex – Ey

g. - 0.3 Ex + Ey　　h. - 0.3 Ex – Ey

i. With Shear Deformation.

The total moment of shear walls in X-dir. and Y-dir. case (with shear deformations) as shown in table (4-48) Superpose them using the load combination mentioned above and obtain the result table (4-49).

Table 4.58: The Total Moment of Shear Wall in X-dir. and Y-dir. Case (1).

Storey	M shear wall x	M shear wall y
20	-2822.98	-8081.79
19	-4355.76	-11768.76
18	-5235.43	-14157.42
17	-5667.40	-15473.73
16	-5789.02	-15888.73
15	-5690.51	-15527.37
14	-5428.60	-14474.81
13	-5035.24	-12780.85
12	-4522.38	-10462.45
11	-3883.81	-7542.68
10	-3114.29	-4370.04
9	-2146.06	-455.98
8	-903.77	4302.69
7	729.65	10053.90
6	2812.22	16998.19
5	5175.93	25325.37
4	8688.19	35457.10
3	13842.33	47828.79
2	21396.23	62931.56
1	35198.80	86981.44

Table 459: Load Combinations for the Total Moment of Shear Wall Case (1).

Storey	Load Combinations							
	1	2	3	4	5	6	7	8
20	-5248	-398	398	5248	-8929	-7235	7235	8929
19	-7886	-825	825	7886	-13075	-10462	10462	13075
18	-9483	-988	988	9483	-15728	-12587	12587	15728
17	-10310	-1025	1025	10310	-17174	-13774	13774	17174
16	-10556	-1022	1022	10556	-17625	-14152	14152	17625
15	-10349	-1032	1032	10349	-17235	-13820	13820	17235
14	-9771	-1086	1086	9771	-16103	-12846	12846	16103
13	-8869	-1201	1201	8869	-14291	-11270	11270	14291
12	-7661	-1384	1384	7661	-11819	-9106	9106	11819
11	-6147	-1621	1621	6147	-8708	-6378	6378	8708
10	-4425	-1803	1803	4425	-5304	-3436	3436	5304
9	-2283	-2009	2009	2283	-1100	188	-188	1100
8	387	-2195	2195	-387	4032	4574	-4574	-4032
7	3746	-2287	2287	-3746	10273	9835	-9835	-10273
6	7912	-2287	2287	-7912	17842	16155	-16155	-17842
5	12774	-2422	2422	-12774	26878	23773	-23773	-26878
4	19325	-1949	1949	-19325	38064	32851	-32851	-38064
3	28191	-506	506	-28191	51981	43676	-43676	-51981
2	40276	2517	-2517	-40276	69350	56513	-56513	-69350
1	61293	9104	-9104	-61293	97541	76422	-76422	-97541

The total shear force in X-dir. and Y-dir. for case (with shear deformations) as shown in table (4-50) Superpose them using the load combination mentioned above and obtain the result table (4-51).

Table 4.60: The Total Shear Force in X-Dir. and Y-Dir. Case (1).

Storey	Tx	Ty
20	1370.31	3324.84
19	1391.05	2522.39
18	1600.84	2717.84
17	1855.78	2954.35
16	2133.18	3213.61
15	2417.34	3480.24
14	2697.10	3741.00
13	2964.18	3984.09
12	3211.93	4198.57
11	3434.30	4386.47
10	3632.86	4658.13
9	3785.39	4732.49
8	3889.37	4738.01
7	3930.89	4659.77
6	3929.63	4480.24
5	3961.59	4203.96
4	3680.85	3752.50
3	3212.10	3121.28
2	2467.17	2291.73
1	1569.73	1365.29

Table 4.61: Load Combinations for the Shear Force in X-Dir. and Y-Dir. Case (1).

Storey	Load Combinations							
	1	2	3	4	5	6	7	8
20	2368	373	-373	-2368	3736	2914	-2914	-3736
19	2148	634	-634	-2148	2940	2105	-2105	-2940
18	2416	785	-785	-2416	3198	2238	-2238	-3198
17	2742	969	-969	-2742	3511	2398	-2398	-3511
16	3097	1169	-1169	-3097	3854	2574	-2574	-3854
15	3461	1373	-1373	-3461	4205	2755	-2755	-4205
14	3819	1575	-1575	-3819	4550	2932	-2932	-4550
13	4159	1769	-1769	-4159	4873	3095	-3095	-4873
12	4472	1952	-1952	-4472	5162	3235	-3235	-5162
11	4750	2118	-2118	-4750	5417	3356	-3356	-5417
10	5030	2235	-2235	-5030	5748	3568	-3568	-5748
9	5205	2366	-2366	-5205	5868	3597	-3597	-5868
8	5311	2468	-2468	-5311	5905	3571	-3571	-5905
7	5329	2533	-2533	-5329	5839	3481	-3481	-5839
6	5274	2586	-2586	-5274	5659	3301	-3301	-5659
5	5223	2700	-2700	-5223	5392	3015	-3015	-5392
4	4807	2555	-2555	-4807	4857	2648	-2648	-4857
3	4148	2276	-2276	-4148	4085	2158	-2158	-4085
2	3155	1780	-1780	-3155	3032	1552	-1552	-3032
1	1979	1160	-1160	-1979	1836	894	-894	-1836

The displacement in X-dir. and Y-dir. At story number 20 for case (with shear deformations) as shown in table (4-52) Superpose them using the load combination mentioned above and obtain the result table (4-53).

Storey 20th displacement in X-dir = 144 mm and in Y-dir = 118 mm.

Table 4.62: Displacement in X-Dir. and Y-Dir. Case (1).

Storey	Ex, 0.3 Ey	Ex, - 0.3 Ey	- Ex, 0.3 Ey	- Ex,- 0.3 Ey
20	144 m in X-dir	144 m in X-dir	-144 m in X-dir	-144 m in X-dir
	35.4 m in Y-dir	-35.4 m in Y-dir	35.4 m in Y-dir	-35.4 m in Y-dir
20	118 m in Y-dir	118 m in Y-dir	-118 m in Y-dir	-118 m in Y-dir
	43.2 m in X-dir	-43.2 m in X-dir	43.2 m in X-dir	-43.2 m in X-dir

ii. Without Shear Deformation.

The total moment of shear walls in X-dir. and Y-dir. case (without shear deformations) as shown in table (4-54) Superpose them using the load combination mentioned above and obtain the result table (4-55).

Table 4.63: The Total Moment of Shear Wall in X-Dir. and Y-Dir. Case (2).

Storey	M shear wall x	M shear wall y
20	-2822.98	-8094.65
19	-4355.76	-11753.16
18	-5235.43	-14118.89
17	-5667.40	-15420.91
16	-5789.02	-15832.19
15	-5690.51	-15478.72
14	-5428.60	-14446.20
13	-5035.24	-12784.48
12	-4522.38	-10510.28
11	-3883.81	-7645.39
10	-3114.29	-4530.74
9	-2146.06	-684.15
8	-903.77	3999.26
7	729.65	9669.99
6	2812.22	16532.41
5	5175.93	24782.86
4	8688.19	34849.86
3	13842.33	47180.21
2	21396.23	62279.26
1	35198.80	86387.71

Table 4.64: Load Combinations for The Total Moment of Shear Wall Case (2).

Storey	Load Combinations							
	1	2	3	4	5	6	7	8
20	-5251	-395	395	5251	-8942	-7248	7248	8942
19	-7882	-830	830	7882	-13060	-10446	10446	13060
18	-9471	-1000	1000	9471	-15690	-12548	12548	15690
17	-10294	-1041	1041	10294	-17121	-13721	13721	17121
16	-10539	-1039	1039	10539	-17569	-14095	14095	17569
15	-10334	-1047	1047	10334	-17186	-13772	13772	17186
14	-9762	-1095	1095	9762	-16075	-12818	12818	16075
13	-8871	-1200	1200	8871	-14295	-11274	11274	14295
12	-7675	-1369	1369	7675	-11867	-9154	9154	11867
11	-6177	-1590	1590	6177	-8811	-6480	6480	8811
10	-4474	-1755	1755	4474	-5465	-3596	3596	5465
9	-2351	-1941	1941	2351	-1328	-40	40	1328
8	296	-2104	2104	-296	3728	4270	-4270	-3728
7	3631	-2171	2171	-3631	9889	9451	-9451	-9889
6	7772	-2148	2148	-7772	17376	15689	-15689	-17376
5	12611	-2259	2259	-12611	26336	23230	-23230	-26336
4	19143	-1767	1767	-19143	37456	32243	-32243	-37456
3	27996	-312	312	-27996	51333	43028	-43028	-51333
2	40080	2712	-2712	-40080	68698	55860	-55860	-68698
1	61115	9282	-9282	-61115	96947	75828	-75828	-96947

The total shear force in X-dir. and Y-dir. for case (without shear deformations) as shown in table (4-56) Superpose them using the load combination mentioned above and obtain the result table (4-57).

Table 4.65: The Total Shear Force in X-Dir. and Y-Dir. Case (2).

Storey	Tx	Ty
20	1370.31	3332.85
19	1391.05	2520.55
18	1600.84	2721.57
17	1855.78	2964.47
16	2133.18	3230.56
15	2417.34	3504.18
14	2697.10	3771.89
13	2964.18	4021.76
12	3211.93	4242.72
11	3434.30	4436.48
10	3632.86	4711.28
9	3785.39	4790.70
8	3889.37	4800.51
7	3930.89	4725.51

Table 4.66: The Total Shear Force in X-Dir. and Y-Dir. Case (2)

5	3961.59	4270.84
4	3680.85	3816.27
3	3212.10	3177.93
2	2467.17	2336.31
1	1569.73	1394.31

Table 4.67: Load Combinations for the shear force in X-dir. and Y-dir. Case (2).

Storey	**Load Combinations**							
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
20	2370	370	-370	-2370	3744	2922	-2922	-3744
19	2147	635	-635	-2147	2938	2103	-2103	-2938
18	2417	784	-784	-2417	3202	2241	-2241	-3202
17	2745	966	-966	-2745	3521	2408	-2408	-3521
16	3102	1164	-1164	-3102	3871	2591	-2591	-3871
15	3469	1366	-1366	-3469	4229	2779	-2779	-4229
14	3829	1566	-1566	-3829	4581	2963	-2963	-4581
13	4171	1758	-1758	-4171	4911	3133	-3133	-4911
12	4485	1939	-1939	-4485	5206	3279	-3279	-5206
11	4765	2103	-2103	-4765	5467	3406	-3406	-5467
10	5046	2219	-2219	-5046	5801	3621	-3621	-5801
9	5223	2348	-2348	-5223	5926	3655	-3655	-5926
8	5330	2449	-2449	-5330	5967	3634	-3634	-5967
7	5349	2513	-2513	-5349	5905	3546	-3546	-5905
6	5294	2565	-2565	-5294	5727	3369	-3369	-5727
5	5243	2680	-2680	-5243	5459	3082	-3082	-5459
4	4826	2536	-2536	-4826	4921	2712	-2712	-4921
3	4165	2259	-2259	-4165	4142	2214	-2214	-4142
2	3168	1766	-1766	-3168	3076	1596	-1596	-3076
1	1988	1151	-1151	-1988	1865	923	-923	-1865

The displacement in X-dir. and Y-dir. At story number 20 for case (without shear deformations) as shown in table (4-58) Superpose them using the load combination mention above and obtain the result table (4-59).

Storey 20th displacement in X-dir = 144 mm.

Storey 20th displacement in Y-dir = 116 mm.

Table 4.68: Displacement in X-Dir. and Y-Dir. Case (2).

Storey	Ex, 0.3 Ey	Ex, - 0.3 Ey	- Ex, 0.3 Ey	- Ex,- 0.3 Ey
20	144 m in X-dir	144 m in X-dir	-144 m in X-dir	-144 m in X-dir
	34.8 m in Y-dir	-34.8 m in Y-dir	34.8 m in Y-dir	-34.8 m in Y-dir
20	116 m in Y-dir	116 m in Y-dir	-116 m in Y-dir	-116 m in Y-dir
	43.2 m in X-dir	-43.2 m in X-dir	43.2 m in X-dir	-43.2 m in X-dir

5. CONCLUSION

The purpose of this research was to examine the effects of horizontal loads on shear and bending deformations (without shear deformation) in reinforced concrete shear walls in both directions (X and Y). With the help of the Response Spectrum Technique, an adapted dynamic method of analysis, we were able to solve the case study of a 20-story high-rise office structure and obtain essential comparing parameters, including shear wall moments, column moments, displacement of the building, a D-value, and fundamental time-periods.

a. In Y-direction:

  i. The parameter values were predicted to increase when shear deformations were included, as opposed to without shear deformations. Due to the decreasing coefficient "Kβ" in the wall rectifiers, this was to be expected. As a result, the stiffness of the structure decreased by around 3% as shear forces and maximum displacement increased by 1.5% and the maximum moment for the shear wall and core increased by 0.68 percent. Shear deformations were shown to increase periods by 1.1% compared to without shear deformations.

  ii. When shear deformation is considered, the real base shear force acting on the bottom of the structure will reduce by 1%, according to the values of (7378 kN with shear deformation and 7421 kN without shear deformation).

  iii. We observe that the values of the maximum store drift ratio at the 8th floor are 0.018 and 0.0177 for cases (with and without shear deformation), respectively less than 0.02 from TEC-2007.

  iv. We observe that the maximum displacement of the structure are 118 mm and 116 mm for cases (with and without shear deformation), respectively less than the displacement limitation ($\Delta_{\text{max-all}} = 0.002H_B$) from TEC-2007.

b. In X-direction:

  i. According to the Kβ value, which is approximately equal to 1, this means the results of cases with and without shear deformations are approximately equal.

ii. We observe that the value of the maximum store drift ratio at the 7th floor is 0.0235 more than 0.02 from TEC-2007.

iii. We observe that the maximum displacement of the structure is 144 mm more than the displacement limitation 122 mm from TEC-2007 ($\Delta_{max\text{-}all} = 0.002H_B$).

c. In general:

i. When comparing shear rigidity in the X-direction to that in the Y-direction with considering shear deformation, we observe that the shear rigidity of structures decreases by roughly 64% to 67%. Therefore, the X-direction relative displacement of the structure exceeds the Y-direction value.

ii. The X-direction time-period of the building (2.60 sec) is greater than the Y-direction value (1.89 sec) when considering the shear deformation; therefore, stiffer buildings have smaller natural periods. The Y direction is the more rigid one.

iii. When comparing the $\theta_{II,i}$ value with $\theta_{II,I,max}$ in both direction for both cases (with and without shear deformation) we observe that the value of second order indicator is less than $\leq 0.12(D/C_h{*}R)$.therefore, the second order effects are not to be taken into account in the calculation of internal forces based on the design.

iv. When we superimpose the total moment of the shear wall and the shear force for all stories in both directions with and without shear deformation, we observe that the most effective load combination in the structure is (0.3 Ex + Ey).

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