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COMPARISON OF TWO STATICALLY SOLUTION METHODS OF SPECIAL FACTORY FRAME

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Master of Science

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COMPARISON OF TWO STATICALLY SOLUTION METHODS OF SPECIAL FACTORY FRAME

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ABSTRACT

COMPARISON OF TWO STATICALLY SOLUTION METHODS OF SPECIAL FACTORY FRAME

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The manual methods for structural analysis entail a lot of complexity and repetitive computations. From the standpoint of structure design, the bending moment is a critical metric. To compute the bending moment, various approaches were used in this study, which including Moment Distribution, moment redistribution, Flexibility matrix method and the ETABS software. Moment Distribution, moment redistribution where used in the analysis of beams whereas Flexibility matrix method and ETABS where used in the analysis on the frames. The results show that the bending moment produced using both approaches is almost identical. Because the number of checks that were carried in the Flexibility matrix method is not significantly high, the findings produced using this method are highly precise.

KEYWORDS: Flexibility Matrix Method, ETABS, Moment Distribution

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CHAPTER ONE

1.1 INTRODUCTION

A structure can be said to be a system of two or more connected parts used to support a load. It is also an assemblage of basic components connected to each other to serve the purpose of carrying the loads developing due to the self-weight and super-imposed loads safely without causing any serviceability failure on itself or to the foundation. [1]. Structures are constructed to specifically support loads (floors or decks) and their main function can be to give protection against the environmental constrain. Important examples related to civil engineering include buildings, tunnels, bridges, and dams; and in other section of engineering, motor vehicles frames and aircraft frames, tanks, floating vessels, mechanical frames, and electrical infrastructures are important.[1].

Structural analysis and design is the anticipation or prognostication of performance of a said structure under a given load or an external load such as supports movement or thermal changes. Structural analysis and design is the organised investigation of the stability, strength and rigidity of structures as mention in Anil et al.[2]. The fundamental goal of structural analysis and design is to create a structure that can sustain all applied loads for the duration of its existence. If a structure is not properly analysed and designed or the loads applied exceed the design load, the structure may fail to perform its intended purpose, which may lead to collapse. A well analysed and designed structure greatly minimizes the possibility of failures Sharma et al.[3]. The designer or engineer must take into account safety, esthetics, and serviceability while designing a structure for a certain function, such as public usage. and also taking into consideration of costs and environmental factors Reddy et al.[4]. To design a structure in a rational manner, it is important to determine the loads acting on each structural member. These loads acting on the structural members will establish the amount of stress and their internal force (resultant force) at a given section of the structure as seen in Hibbeler [1]. The load acting on structural member as usually govern by design codes. These stresses or resultant forces have to be within desired limits in order to maintain safety and to avoid excessive deformations. To determine the stresses and resultant forces, the geometrical shape and material properties must be determined. Once a structure is proposed, a preliminary design of the structure is done, the structure must then be analyzed and design to ensure that it has its required capacity and strength Vinod et al.[5]. To analyze a structure properly. To analyze a structure properly, certain judgement is made on how the members are supported and connected together. The forces in the members and their displacements are calculated using structural analysis theory, and the design loadings are estimated using various design codes according on the country and local standards. Vinod et al.[5].

1.2 AIMS AND OBJECTIVES

The aim of this thesis is to provide an insight on the different method of analysing a structure using software (ETABS) and analysing manually using (Force Method). This is specially investigated when comparing vertical and horizontal (wind) loading with different modelling techniques and how the shear force can be determined with ETABS using displacement method. Furthermore, in comparison, the precision of analytical results is calculated manually.

The objectives of this thesis are focused on analysing a three bay frame using two different and comparing the results. This thesis is a comparative analysis of Force Method and ETABS. The fundamental objectives are;

a. To firmly understand the formal methods and modern methods of analysis, Force Method and ETABS
b. Performance analysis of the frames using Force Method as well as ETABS under vertical loadings (Dead load) and horizontal (Wind load).
c. Carryout an investigative analysis to compare the results gotten from both and determine their advantages and disadvantages.

1.3 SCOPE

In recent study, frames are analyzed and designed by using manual and software methods to understand the advantages of each and compare their accuracy. Force Method is considered to be among the simplest and accurate methods of manual methods structural analysis and therefore has been chosen to represent the manual methods. ETABS has a growing popularity and reputation among the software method

1.4 LIMTATIONS

The analysis of a three bay frames consists of many steps and factors, and to evaluate all these steps are beyond the scope this Master's thesis. For concrete, thermal variations, shrinkage and creep effects are not analyzed, concrete is taken as not cracked. Design of element cross-sections were not carried out. Acceleration of frame were not done. Hence, no time-history analysis was performed.

1.5 OUTLINE OF THE THESIS

CHAPTER ONE	General introduction to the Master's thesis and the summary of aims and objectives, scope and limitation
CHAPTER TWO	Shows the information gotten from the literature review study and history as well as the commonly analysis methods used in the analysis of structures
CHAPTER THREE	Methodology of the thesis
CHAPTER FOUR	Load transformation on longitudinal beams and analysis of beams
CHAPTER FIVE	Load transformation on frames and analysis
CHAPTER SIX	Conclusion and Recommendations

CHAPTER TWO

2.1 LITERATURE REVIEW

A considerable number of research has been carried out by various scholars on comparing of between manual analysis and software analysis.

Pidurkar et al. (2016), concluded in their paper "Comparative study of beam by analytical method and ETABS software" that the hogging and sagging moments computed using the analytical technique for the considered beam are quite similar to those estimated using the ETABS program.

Mittal et al. (2016), carried out a research "A Comparison of the Analysis and Design Results of 4 Story Using STAAD Pro and ETABS Software" and concluded ETABS gave a minimal forces compared to STAAD PRO[6].

Hiwase et al. (2018). in their paper title "Comparison between Manual and Software Approach towards design of structural element" showed the difference in analyzing using Manual Approach and Software Approach in separate ways, and found out there was a minor difference in the manual analysis results and the Software approach.

Furthermore, Vinod et.al. (2020). wrote a paper titled "Comparison between Analysis of a Portal Frame by ETABS and Moment Distribution Moment Software Approach" described the basic concepts and difference between ETABS and Moment Distribution Method. Which concluded that the results from ETABS and Moment Distribution Method are nearly the same and suggested that ETABS be an option instead of Moment Distribution Method as it is faster, easier and minimize error.

Structures have an important role to play in economic development of any Nation, which can be expensive when not analyzed and design properly. Therefore, it is important to have a suitable method of analysis and design to achieve economical and strength of a structure.

2.2 STRUCTURE

In analyzing a structure, it is necessary to know the type of structure it is. Different structures require different methods to be analyzed. For instance, determinate structures can be analyzed completely by static equilibrium only, while structures that are said to be indeterminate can be analyzed by using compatibility and equilibrium static relations in order to determine the internal forces. In reality, structures must be stable. Which implies that structure after disturbance can be able to recover static equilibrium. Hibbeler [1].

A structure is designed for resultants of bending moment, shear force, deflection, torsional and axial stresses. When these resultants stresses are determined at their respective critical sections then the plotting of the bending moment, shear force and deflection for the structural element can be said to be *analysis*. Dimension evaluation of theses stress components and proportion is said to be design. Hibbeler[1].

2.2.1 Structural Element

Identifying various structural element, their function and forms by a structural engineer is crucial. Structural element come in different forms, shapes and sizes with the primary objective to withstand the load subjected to it. The shapes and size usually selected to support the load carrying functions, in other instances the sizes and shapes are dependent on the architectural considerations.

SLABS

Slabs are common structural element made of plate element taking form of a floor or roof in modern buildings which generally supports uniformly distributed load. Slabs can be simply supported, continuously supported over one or more spans and also can be cantilever free at one end and supported at the other end Kassimali [7].

BEAMS

Beams are horizontal structural element used to transfer load from the slab to the columns or support vertical loads or bridge a gap between two points, they are often described by their support conditions. Beams are generally designed withstand bending moments, short beams that carry large loads may quite have large internal shear force this force is then used in the design of the beam Kassimali [7]. Steel or aluminum beams cross section are more efficient, the cross section is efficient when shaped as I-shape or U-shaped, the forces are generated at the top and bottom flanges of the beam and form a couple to resist the applied moment and the web is effective in resisting shear force.

Concrete beams are usually rectangular in shape which is constructed easily in practice. Concrete have weak tensile resistance that is why it is reinforced with steel which is excellent in tensile resistance

Timber beams are sawn from wood piece or laminated. Laminated beams are made from solid section of timber which are tighten together using high tensile glue.

COLUMNS

Column are vertical structural member capable of resisting axial load compressive load. Tube and wide-flange cross section are usually steel columns, Rectangular, square, and circular shaped columns are usually concrete and reinforced with steel when weak in tension. Columns are sometimes subjected to axial and bending moments Kassimali [7].

TIE RODS

These are structural elements under tensile force and are also called bracing struts, the elements are slender and chosen from rods, channel, and angles due to the nature of the load as Hibbeler [1] shown.

CABLES AND ARCHES

Cables are made of steel and flexibly to bear loads in axial tension by taking the U-shaped or funicular shape, cables are mostly used to support bridges, stadium roofs or mall roof. when used as support they of greater advantage than beam or truss especially for larger spans because they are tension members, cables are stable and wont suddenly collapse like in truss or beams. Beam requires increase in costs, construction and depth as the span increases. When cables are used instead it is only confined by their sag, weight or anchorage.

Arches attain its strength in compression because of its opposite curvature to that of cable, arches have to be rigid to maintain its shape or status. They used in bridge structures, masonry works, domes or enclosed large areas, stadiums or sports are as mentioned in Hibbeler [1].

FRAMES

Frames are the composition of beams and columns coupled or riveted together either by pin, hinge or fixed connection to support axial, shear and bending moment loadings. Frames can be a two or three dimension, load subjected to the frame make it members to bend. With rigid connections the structure can be said to be “indeterminate” in analysis. The strength of the frames is determined from the moment interaction between the columns and beam connections Hibbeler [1].

SURFACE STRUCTURE

Surface structure is a material with small thickness with a very flexible material and can take the form of an air-inflated structure. Surface structure can also consist of rigid material like reinforced concrete. They can be shaped, folded plates, cylinder, paraboloids or hyperbolic and can be referred to as *shells structure*, these structures are like cables or arches because they support load primarily in compression and tension with small bending. These structures are very difficult to analyzed because of its three-dimension geometry of its surface.

2.3 LOADS

When the cross section requirements of the structural members are determined, it remains imperative to also define the loads the structure must withstand, it is the estimation of the loads that will subjected to the structure will be selected for design. For instance, tall buildings have to withstand large horizontal loadings due to wind or earthquake, then shear walls and tube frame system are selected. When structural members are determined, the design of those element begin to support the intended load to carry Kassimali [7].

The loads on a structure are generally defined by a specified code, the structural engineer or designer reference the two types of codes: the general building codes for minimum design load on structures and minimum standards for construction. The design gives detailed technical standard which are used to determine the requirement for the real structural design Kassimali [7].

2.3.1 Dead Loads

Dead loads are the self-weight of the various structural elements and the self-weight of any of the structural element are permanent, therefore the dead load of a building include the weight of beams, columns, slabs, roofing, windows. Acceptable structural dead load is determined by simple formulas depending on the size and weight of similar structure. By experience one can assume the weight the of a structural member. Table shows various weight of structural material Hibbeler [1].

Table 2.1: Materials and their unit weight

MATERIAL	UNIT WEIGHT (kN/m^3)
Aluminum	26.7
Concrete	23.6
Steel	77.3
Wood	5.8
Masonry	21.2

2.3.2 Live Loads

Live loads changes depending on the magnitude and location, these load are mobile or movable loads subjected to the structure and because of their nature they are precisely determined easily.

Example of such loads are moving load and natural forces, the live load specified in codes are known from studying the nature of their effects on an existing structure. Normally, these loads are extra precautions against undue deflection or unpredictable overload. Table 2.3 shows minimum live loads as seen in Hibbeler[1].

Table 2.2: Live load and their values of occupancy

OCCUPANCY	Live load (kN/m^2)	Live load (kN)
All uses self-contained	1.5	2.0
Bedrooms and hostels	1.5	2.0
Hotels, motels, hospitals wards	2.0	2.0
Family dwelling units	2.5	2.0
Offices general use	2.5	2.7
Assembly halls, theatres, concert halls, places of worship	5.0	3.6
Shopping malls	4.0	3.6
Storage areas	2.4 per meter height	7.0
Warehouses for mobile stacking	4.8	7.0
Vehicle weight ≤30kN	25	10.0

2.3.3 Wind Loads

While buildings obstruct the flow of wind, the kinetic energy of the wind is converted to potential energy of pressure, which induces a wind loading. The force of wind acting on a structure depends on the density and velocity of the air, the incidence angle of the wind, the shape, size and stiffness of the buildings, the surface roughness of the structure. For design intent wind loading can be analyzed by static approach or dynamic.

In the static approach, the varying pressure induced by a continuously blowing wind is estimated by an average velocity pressure that is subjected to the structure. The pressure is determined by a kinetic energy Hibbeler [1].

$$Q = \frac{1}{2}\rho v^2 \qquad (1)$$

Design standard codes provide equations for adjustment according of the importance of the structure, the location of the terrain, height are considered.

In the design of wind pressure for an enclosed building the selection depends on the Flexibility and height of the structure and if the design is for a wind force resisting system or for the structural members. For instance, the wind pressure on an enclosed structure of any height is defined using a two-termed equation ensuing from external and internal pressures. Tables 2.4 and 2.4.1 below shows the wind pressure coefficients Hibberler[1].

Table 2.3: Wall pressure coefficient

Surface	L/B	C_f	For use
Windward	All values	0.8	q
Leeward	0 -1	-0.5	q
	2	-0.3	q
	≥ 4	-0.2	q
Sidewall	All values	-0.7	q

Table 2.4: Wind speed and suction with respect to height

Height from ground (m)	Wind speed (m/s)	Suction q (kN/m^2)
0-8	28	0.5
9-20	36	0.8
21- 100	42	1.1
>100	46	1.3

For tall buildings or structures having a shape or location that are wind sensitive, it is encouraged to use a dynamic approach to define the wind loadings. The methods for doing this are outlined in various structural design standard, wind tunnel testing are performed on a model of the building to stimulate the natural environment and its surrounding, the effect of the wind pressure on the structure can be defined from pressure traducers attached to the model. Although a model with stiffness characteristics and proper scale to the structure the dynamic deflections can be defined.

The difference between the pressures on the opposing surfaces, taking into consideration their signs, is the net pressure on a wall, roof, or element. Suction directed away from the surface is considered negative, whereas pressure directed towards the surface is considered positive. Figure 2.0 shows several examples.

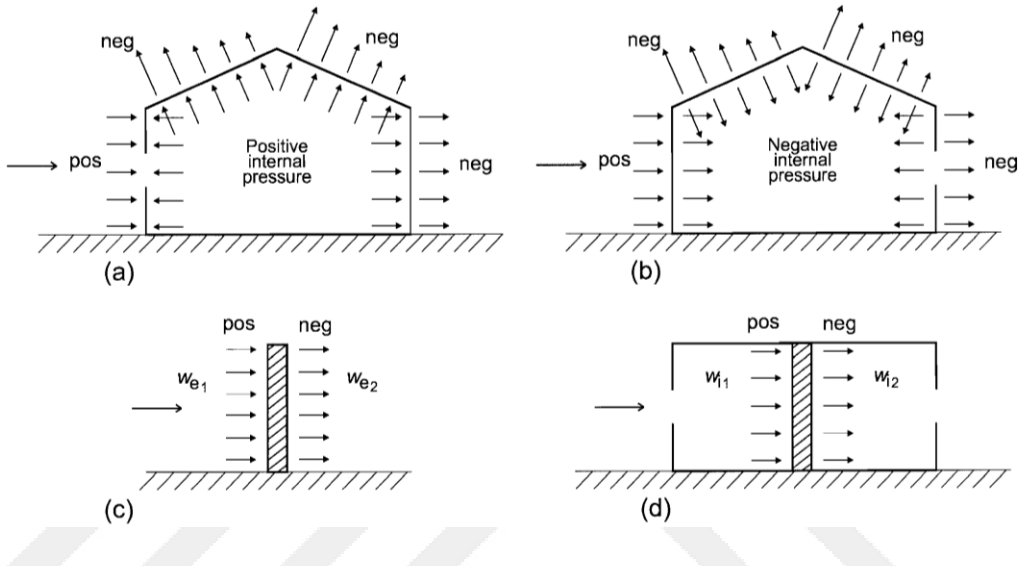


Figure 2.0: Wind pressure on surfaces

2.4 STRUCTURAL DESIGN

In designing a structure, it is crucial to consider the material used and the load uncertainties. The uncertainties can be the variation of material properties, parallax error during construction, stress in materials, corrosion of materials(steel) and unforeseen circumstances.

2.4.1 Methods of Structural Design

The methods of structural design include in the following:

Allowable stress design: In this design method the load uncertainties and material are combined into a single safety factor. The type of loads mentioned earlier can act coincidentally on a structure, but rarely that the highest of all those load can act all at once or the same time. For instance, the unfavorable wind load and earthquake loads can rarely act both at the same time on a structure. So in *allowable stress design* the elastic stress of the material must not be greater than the allowable stress for every load combination. Below are some load combinations defined in TS-500 Turkish standard:

Table 2.5: Load Combination in Ts-500 Turkish Standard

Load Combination	As in Ts-500 Turkish Standard
Vertical Loadings Combination	$F_d = 1.4\ G_k + 1.6\ Q_k$
	$F_d = 1.0\ G_k + 1.2\ Q_k + 1.2T_k$
Earthquake Loadings Combination	$F_d = 1.0\ G_k + 1.0\ Q_k + 1.0E_k$
	$F_d = 0.9\ G_k + 1.2T_k$
Wind Loadings Combination	$F_d = 1.0\ G_k + 1.3\ Q_k + 1.3W_k$
	$F_d = 0.9\ G_k + 1.3W_k$
Earth Pressure Loading Combination	$F_d = 1.4\ G_k + 1.6\ Q_k + 1.6\ H_k$
	$F_d = 0.9\ G_k + 1.6\ H_k$

Where;

Design Load, F_d

Dead load, G_k

Live load, Q_k

Temperature Variation, T_k

Wind Load, W_k

Earth Pressure Load, H_k

These load combinations are to provide the most unfavorable and realistic loads acting on the structure.

Load resistance factor design: Uncertainties are now considered by using probability theory. Material uncertainties are now separated from load uncertainties, in accounting for these uncertainties this method uses the load combination and load factors. For different loads, the increment factor will be different (Probabilistic Approach).

$$F_d = \gamma_F * F_k \qquad (2)$$

Where;

Design Load, F_d

Load Coefficient, γ_F

Characteristics Load, F_k

2.5 STRUCTURAL ANALYSIS

The aim of structural analysis in structures is to determine the distribution of moments and internal forces over the structure and know the design criteria at all sections. The cross-sections are popularly idealized by regarding the structure whether it is made up of two-dimensional elements or linear elements. When the preliminary design of the is done, the structure can then be analyzed to determine the bending moments, shear force, and requirements to maintain its strength and rigidity. For idealization to be made, how the structural member is connected or supported needs to be defined, the loadings on the structure are defined by design codes and local specifications. The displacements and forces in the structural element can be determined using the theory of structure analysis.

Structural system and its loading system can be complicated, in order to make the analysis easier, certain assumption can be made such as strength of the materials, cross-section of the structural members, nature of the design load and their distributions, how they are connected at joints and support conditions. This make the structural analysis easier and less complex to some extent.

2.5.1 Methods of Structural Analysis

There are two methods of analyzing statically indeterminate structures are discussed below

2.5.1.1 Force method

Force Method was firstly developed by James Clerk Maxwell in 1864, and later Otto Mohr and Heinrich Muller-Breslau, the Force Method which is also known as Flexibility matrix method was among the early methods of analyzing statically indeterminate structures. This method is based on compatibility equations. In this method the equations are to satisfy the compatibility and force-displacement requirement for a given structure for the redundant forces, when the redundant forces are determined the rest of the reaction forces of the structure can be determined by satisfying the compatibility equations[1].

2.5.1.2 Displacement method

Displacement method is like the opposite of Force Method; Displacement matrix method is also known as Stiffness matrix method. In this method load-displacement relation is first written for the structural members and then satisfy the equilibrium equation. Here by using the load-displacement relation the unknown displacements are written then the equations will be solved to know the displacements. The loads are determined using compatibility and load-displacement equations after the displacements are known. Below are some displacement methods discussed[1].

2.5.1.2.1 Slope deflection method

The slope deflection method was first introduced by G. A. Maney in 1915, as a common method for analyzing indeterminate frames and beams. In this method all the joints are assumed to be rigid (angles between the structural element don't change in a loaded frame). When the frames and beams deflect the rigid joints are assumed to rotate as a whole. Also deformation because of shear stresses axial stresses are considered to be neglected. This method is not commonly used nowadays since the introduction of Moment Distribution in 1930 due to its simplicity, understanding and scientific approach[Hibbeler 1].

2.5.2.2 Moment distribution method

Moment Distribution method also known as *cross method*. The method was introduced by Hardy Cross in 1930, which is generally used for analysis of any type of indeterminate structures. In cross method sequential estimation is done to a certain degree of accuracy, the method starts by releasing and not releasing each joint in sequence, then the internal joint moments are then distributed and balanced till the joints rotate to the final positions. The process of calculation is repetitive and easy to apply as mention in Hibbeler[1].

2.5.2.3 Kani's method

Kani's method was developed by a German, Gasper Kani in 1947 which he named the method after him. This method is an indirect extension of slope deflection method. It is an effective method because of distribution of moments. The method gives iteration by using the structural analysis of slope deflection. The technique lowers the number of simultaneous equations, thus the equation must be equal to the translator number of displacements in the Moment Distribution method, while in Kani's method, the equation must be equal to the translator number of displacements. the number of equations needs to be zero. This method is said to be a simplification of Moment Distribution method where the sway problems are calculated in a tabulated form and shear coefficient are determined Hibberler[1].

CHAPTER THREE

3.1 METHOLOGY

As explained in the previous chapter, there are two common methods of analyzing indeterminate structures which include:

- Force Method which is also called Flexibility matrix method, method of consistent deformation or compatibility methods
- Displacement Method which also called stiffness or equilibrium method

Each of these method include a *particular solution* combination which gotten by turning the structure to a statically determinate structure, and the modification effect of a complimentary solution is assessed. In Force Method, unknown forces are taken into account in the behavior of the structure, whereas in displacement method the unknown displacement are taken into account in the behavior of the structure. Both the methods of analysis include reducing the structure to a determine structure. In Flexibility matrix method, the solution requires the removal of all redundant forces, whereas the displacement method includes restraining the structural joints against displacement Hiwase [8].

3.2 FORCE METHOD

When using Flexibility matrix method, the solution depends on the degree of redundancy. A structure that is must be statically determinate for it to be analyzed by Force Method. The selection of the redundant forces to be removed depends on the geometrical foam, stability of the structure and ease of analysis. The degree of redundancy of a structure can be determined by using the equation:

$$D = (3m + r) - 3j \qquad (3)$$

Where:

D = Degree of redundancy

m = number of member

r = number of reaction forces

j = number of joints

Another relation can be used to determine the degree of redundancy:

$$D = r - 3 - s \qquad (4)$$

Where:

D = Degree of redundancy

s = number of special conditions such as an internal hinge

r = number of reaction forces

When the selection of an easy system is done, then the next step to follow is to replace all the redundant support with a unit load (1kN) and draw their bending moment one after the other. This step involves calculating the deflection of each respective redundant force and applied load differently from force-perpendicular relationship (moment). Without knowing the magnitude of the redundant force the deflection caused by the redundant forces cannot be determined. So by applying a unit load (1kN) the same direction as the redundant force the deflection can be determined. In the analysis of elasticity, the principle of superposition is accepted, by multiplying the unknown redundant with the deflection from unit load the deflections due to redundant force can be obtained, then the total deflection due to applied loading is calculated by application of the superposition principle which is necessary that it is compatible with the boundary condition Hiwase [8].

A series of simultaneous equations are generated for more than one redundant forces with the redundant forces as unknowns and the force coefficients as coefficient of the equations, and the simultaneous equations can be solved using matrix. The force coefficients are also known as *influence coefficients*. The number of equation depends on the number of unknown redundant forces Hiwase [8]. And the series of simultaneous are called canonical equations and are shown in the form below.

$$\delta_{11}X_1 + \delta_{12}X_2 + ... + \delta_{1n}X_n + \Delta_1 P = 0$$

$$\delta_{21}X_1 + \delta_{22}X_2 + ... + \delta_{2n}X_n + \Delta_2 P = 0$$

$$\vdots \qquad\qquad \vdots \qquad\qquad \vdots \qquad\qquad (5)$$

$$\delta_{n1}X_1 + \delta_{n2}X_2 + \ldots + \delta_{nn}X_n + \Delta_n P = 0$$

Where;

$\delta_{i,i}$ = deformation at point *i* due to due unit load at point *j*

X_1 = redundant force at point *i*

$\Delta_i P$ = deformation at point *i* due to externally applied load

$$\delta_{i,i} = \int_0^l \frac{M_i m_j dx}{EI} \qquad (6)$$

Where;

E = modulus of elasticity

I = moment of inertia

m_0 = moment due to the external load applied

m_1 = moment due to the applied unit load

3.2.1 Vereshchagin's Rule

The simplest method of determining the influence coefficient is the graphical method, it is easier than the Mohr's integral and it established by the Vereshchagin's rule. One must be very good at drawing bending moment diagrams together with the right sign conventions before the method can be of use to them. it based on combining the bending moment diagram of both the externally applied load and the redundant loads. Moreover, one must draw the bending moment diagram of every externally applied and redundant load. Below is explained how it is done Hiwase [8].

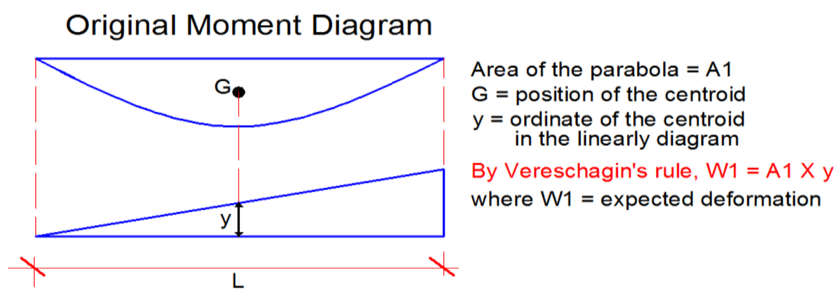


Figure 3.0: Moment diagram due to applied unit load

By combing the two shapes in the figure above by using the Vereshchagin's rule, by assuming the top moment diagram is from a simply supported beam that uniformly loaded and drawn due to the internal stresses, and the bottom moment diagram is gotten from due to a unit load subjected from the left side.

The maximum moment of a simply support beam that uniformly loaded occurs at the mid-span is $wl^2/8 = Mc$, w is the external load applied and l is the length of the span Hiwase [8].

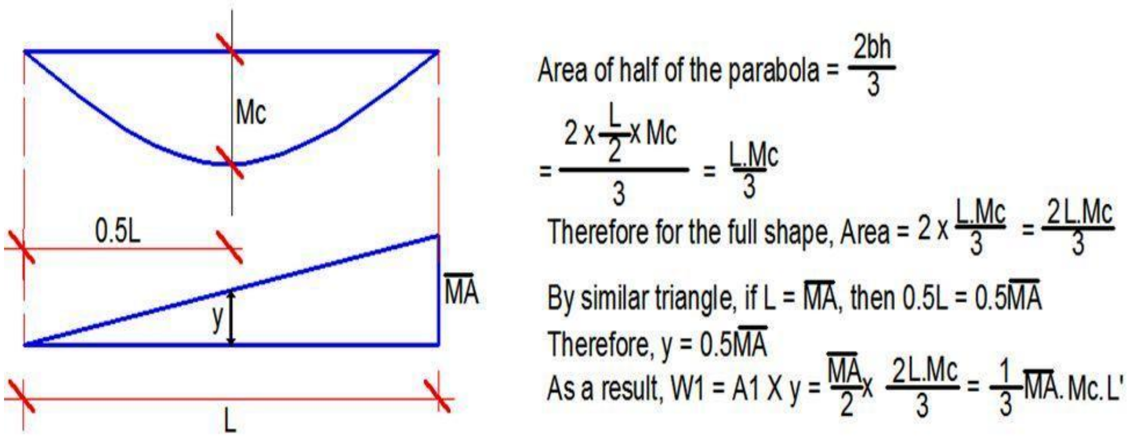


Figure 3.1: Bending moment diagram of a simply supported beam

In the linear diagram the "maximum moment" is at the right support, and value is equal to the section span since load is a unit load (1kN). $L = M_A$. The value of y in the triangle will be the ordinate of the linear diagram that corresponds to the centroid of the original diagram,

when $x = l/2$

As a result, the effect coefficient for the two forms above can be calculated as follows:

$$\delta_{11}X = 1/3 \times M_A \times M_C \times L \qquad (7)$$

Table now contains lists of equations that we will use to merge the diagrams that we will develop during the analysis. They're all derived in the same way as the one above.

There is no difficulty in the combination of diagrams, anyone can be added to another and take into account that they must be of the same length. Otherwise, you should separate the figures at their points of contra flexure, or any other convenient point. Also, in moment diagrams such as those drawn when a cantilever with an evenly distributed weight is propped at the free end, it's best to divide them into two diagrams as illustrated below, paying close attention to the sign conventions. You'll completely get why such a divide is conceivable, when you use the integration approach to solve this Hiwase [8].

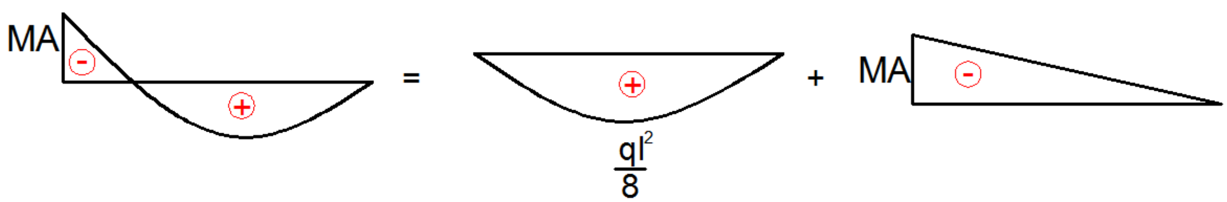


Figure 3.2: Bending moment diagram of a propped canteliver beam

Table 3.1: Product integrals

M \ M_M	b, l	b, l	b, l *	b_1, b_2, l *	αl, βl, b, l *
a, l	abl	$\frac{1}{2}abl$	$\frac{1}{2}abl$	$\frac{al}{2}(b_1+b_2)$	$\frac{1}{2}abl$
a_1, a_2, l	$\frac{bl}{2}(a_1+a_2)$	$\frac{bl}{6}(a_1+2a_2)$	$\frac{bl}{6}(2a_1+a_2)$	$\frac{1}{6}(2a_1b_1+a_1b_2+a_2b_1+2a_2b_2)$	$\frac{bl}{6}[(1+\beta)a_1+(1+\alpha)a_2]$
αl, βl, a, l	$\frac{1}{2}abl$	$\frac{abl}{6}(1+\alpha)$	$\frac{abl}{6}(1+\beta)$	$\frac{al}{6}[(1+\beta)b_1+(1+\alpha)b_2]$	$\frac{1}{3}abl$
a, l	$\frac{2}{3}abl$	$\frac{1}{3}abl$	$\frac{1}{3}abl$	$\frac{1}{3}al(b_1+b_2)$	$\frac{abl}{3}(1+\alpha\beta)$
tangent, a, l	$\frac{1}{3}abl$	$\frac{1}{4}abl$	$\frac{1}{12}abl$	$\frac{al}{12}(b_1+3b_2)$	$\frac{abl}{12}(1+\alpha+\alpha^2)$
tangent, a, l	$\frac{2}{3}abl$	$\frac{5}{12}abl$	$\frac{1}{4}abl$	$\frac{al}{12}(3b_1+5b_2)$	$\frac{abl}{12}(5-\beta-\beta^2)$

3.3 DISPLACEMENT METHOD: MOMENT DISTRIBUTION METHOD

Once certain elastic constants have been found, the moment-distribution technique is a displacement method of analysis that is simple to use. The Moment Distribution is a way of making repeated approximations to any desired level of precision. The approach starts by assuming that each joint in a structure is fixed. The internal moments at the joints are then "distributed" and balanced until the joints have rotated to their final or almost final positions, which is accomplished by releasing and locking each joint in turn. This method of computation will be discovered to be both repetitious and simple to use. However, before discussing Moment Distribution strategies, several definitions and ideas must be provided.

3.3.1 Sign Convention

Sign conventions are established in the figure 3.4 as Moments acting on the member that are clockwise are positive, and moments acting on the member that are counterclockwise are negative.

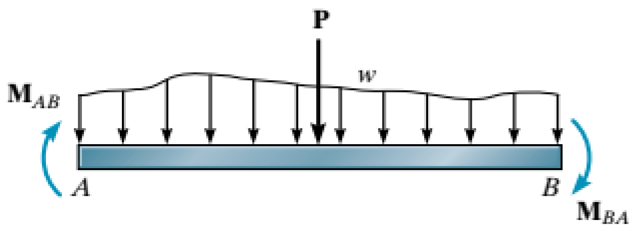


Figure 3.3: Moments at beam ends

3.3.2 Fixed End Moment

Fixed-end moments are the moments at a loaded member's "walls" or fixed joints. Depending on the type of loading on the member, these moments can be calculated using the table on the inside back cover Hibberler [1].

3.3.3 Stiffness Factor of a Member

Consider the pinned-at-one-end, fixed-at-the-other beam in Figure 3.5. When the moment M is applied, the end A rotates by an angle θ_A. The conjugate-beam technique is used to transform M to θ_A. when the far end is fixed the member stiffness factor is a calculated as shown in equation and and may be defined as the amount of moment M necessary to rotate the beam's end A by one rotational degree ($\theta_A = 1$ rad) Kassimali [7].

$$K = \frac{4EI}{L} \qquad (8)$$

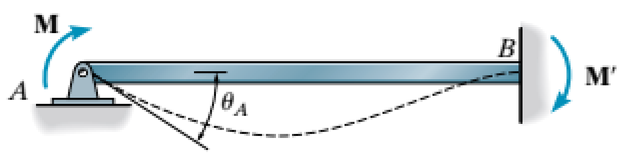


Figure 3.4: A beam pinned at point A and fixed at point B

3.3.4 Joint Stiffness Factor

If multiple members are fixedly attached to a joint and each of their far ends are fixed, the total stiffness factor at the joint is equal to the sum of the member stiffness factors at the joint, $K_T = \sum K$, according to the principle of superposition Kassimali [7].

3.3.5 Distribution Factor

When a moment M is applied to a fixed connected joint, each of the connecting members will contribute a fraction of the resisting moment required to achieve moment equilibrium at the joint. The distribution factor is the percentage of the total resisting moment provided by the member (DF). Assume the joint is fixed and connected to n members to get its value. If a moment M rotates the joint by u, then each member *i* rotates by the same amount Kassimali [7]. If the *i*th member's stiffness factor is K_i, the moment supplied by the member is

$$M_i = K_i\theta. \qquad (9a)$$

Because equilibrium is required is

$$M = M_1 + M_2 + \dots + Mn = K_1\theta + K_2\theta + \dots + Kn\theta = \theta \sum Ki, \qquad (9b)$$

then the ith member's distribution factor is

$$DF_i = \frac{Mi}{M} = \frac{KI\theta}{\theta \sum Ki} \qquad (9c)$$

When the common term u is removed, the distribution factor for a member is equal to the member's stiffness factor divided by the total stiffness factor for the joint; in other words, in general, the distribution factor for a member is equal to the member's stiffness factor divided by the total stiffness factor for the joint Kassimali [7].

$$DF = \frac{K}{\sum K} \qquad (10)$$

3.3.6 Carry-over Factor of a Member

At the wall, the moment M at the pin produces a moment M = 1/2M. The carry-over factor is the percentage of M that is transferred from the pin to the wall. As a result, the carry-over factor for a beam with the far end fixed is + 1/2. Both moments act in the same direction, as shown by the plus sign Hibbeler [1].

3.4 EXTENDED THREE-DIMENSIONAL ANALYSIS OF BUILDING SYSTEM (ETABS).

"ETABS-Extended three dimensional analysis of Building Systems" is a Computers and Structures Inc. product. It's a type of engineering software that's utilized in the construction industry. It includes sophisticated structure analysis and design software tailored to multi-story building systems. Modeling tools and templates, code-based load prescriptions, analysis methodologies, and solution approaches are all part of the integrated system. It can handle the most complicated and massive building models and setups [10]. The ETABS program includes CAD-like drawing tools with a grid representation and an object-based interface.

In the building, design, and modeling industries, ETABS software has the following impacts;

a. It's a type of building software. It tests the load-bearing capability of building structures and analyzes and assesses seismic performance.
b. You may see and edit the analytical model with high accuracy with this program. At every grid line, plans and elevation views are automatically created.
c. Concrete shear walls and concrete moment frames are analyzed using the ETABS program. It's well-known for its static and dynamic analyses of multi-story frame and shear wall structures.
d. It is one of the most widely used civil design tools in the construction business, and it helps structural engineers work more efficiently. It also saves time and money by avoiding the use of general-purpose software.
e. The ETABS input, output, and numerical solution approaches are specifically intended to take use of the unique physical and numerical features of building type structures. As a result, data preparation, output interpretation, and overall execution are all sped up with this analysis and design tool.

3.5 BENEFITS OF THE ETABS SOFTWARE

ETABS Software has been acknowledged as the industry-standard software for building design analysis for almost 30 years. The following are some of the benefits of using ETABS software:

a. It shows the model in 3D axonometric view, plan view, elevation view, elevation development view, and a user-defined custom view.
b. It allows you to enter cross-sections of any shape and material graphically (Section Designer).
c. It can copy and paste a model's geometry into and out of spreadsheets.
d. It's compatible with EC – Praxis 3J for steel connection analysis and design.
e. Engineers' modeling experience is enhanced by ETABS' built-in drawing and drafting tools. In reality, the program includes numerous standard industry shortcuts and controls.
f. ETABS program now provides solutions to certain difficult issues such as Panel Zone Deformations, Diaphragm Shear Stresses, and Construction Sequence Loading.
g. The ultimate SAP Fire 64-bit solver allows complicated models to be evaluated in seconds and supports non-linear modeling approaches such as construction sequencing.
h. Comprehensive reports are supplied for all analysis and design outputs, which may be customized to fit your needs. Schematic construction drawings including frame plans, schedules, minute detailing, and cross-sections may be generated for both concrete and steel buildings.
i. Steel and concrete frames are designed with automatic optimization, and capacity tests for steel connections and foundation plates are incorporated.

CHAPTER FOUR

4.1 LOAD TRANSFORMATION AND BEAM ANALYSIS

The factory is a 60-meter-wide 30-meter-long single-story reinforced concrete factory structure, consisting of 3-span, 20-meter-wide spanning with fixed and pinned -supports columns, 6 broken-roof frames with horizontally 6-meter span, with a roof slab thickness of the roof slab will be 15 cm and some openings for ventilation and natural light with the dimension 5x6 mxm, these will be covered with glass. The factory is analyzed as 2D and divided into two as external frames and internal frames.

The two frames have the same properties, dimensions, compressive strength of concrete except loadings. The external loading supports the exterior beams and exterior slabs. Whereas the internal frames support the interior beams and interior slabs.

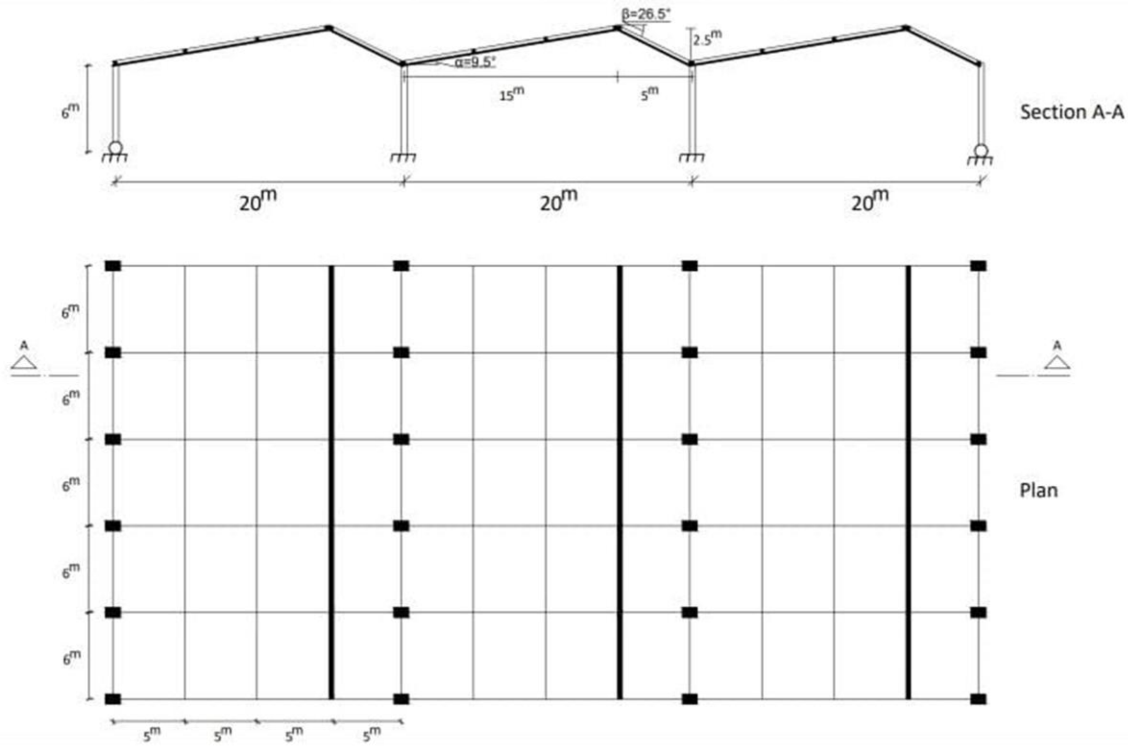


Figure 4.0: The case study

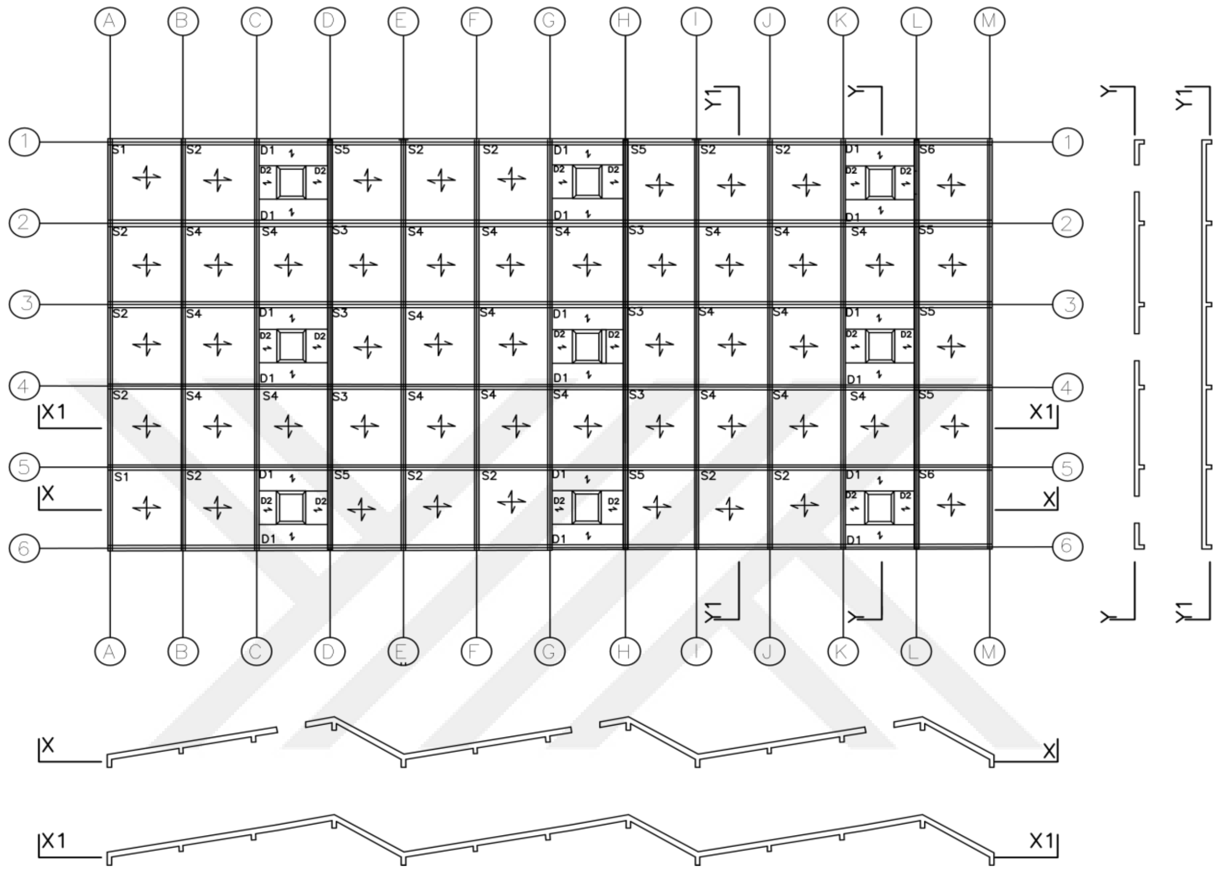


Figure 4.1: Slab layouts, opennings and sections

4.2 LOADINGS AND DIMENSIONS

Slab thickness = 150mm

Slab openings = 1.69m by 2m

Slab length = 6m

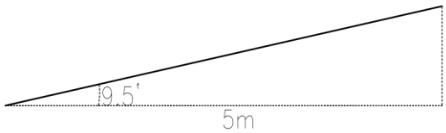
Slab live load = 1.5kN/m

Slab Dead load = 0.15 * 25 = 3.75kN/m

C30/420

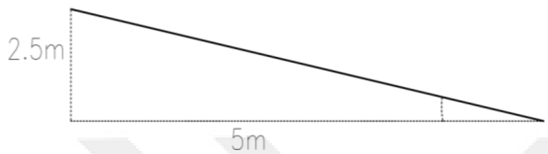
Slab breath will change due to the inclination, therefore the breath is calculated as follow:

For S1, S2, S4, D1 and D



Hypotenuse = $\frac{5}{\cos 9.5}$ = 5.07m

For S3, S5 and S6



$\sqrt{5^2 + 2.5^2}$= 5.59m

4.2.1 Determination of Design Load Due to Vertical Loadings

The design load on slabs:

$$1.4G_K + 1.6Q_K \qquad (11)$$

Design load = 1.2(3.75) + 1.6 (1.5) = 7.65 kN/m^2

Table 4.1: Slab long span and short span ratio

Slab name	LY/LX	m
S1	6/5.07	1.18
S2	6/5.07	1.18
S3	6/5.59	1.07
S4	6/5.07	1.18
S5	6/5.59	1.07
S6	6/5.60	1.07

4.3 TRANSFORMATION OF LOADS ON THE LONGITUDINAL BEAMS

4.3.1 External Beam Analysis

Beam dimension 30cm by 60cm

Design load for Slab S1 = 7.65kN/m

Design load for Slab S2 = 7.65kN/m

Depth of beam = 0.6 – 0.15 = 0.45

Beam self-weight = 25 * 0.3 * 0.45 = 3.38kN/m^2

Live load = 1.2kN/m^2

m =1.18

4.3.2 Transformation of Trapezoidal Load From Slab Longitudinal Span

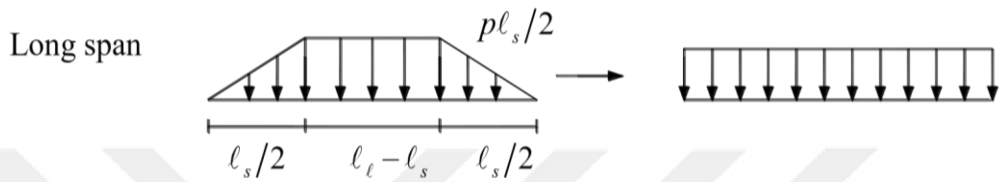


Figure 4.2: Load transformation from longitudinal slab span

$$\frac{p\,\ell_s}{3}\left(\frac{3}{2}-\frac{1}{2m^2}\right) \qquad (12)$$

$\frac{7.65*5}{3} * (\frac{3}{2} - \frac{1}{2(1.18)^2})$ = 14.54kN/m^2

Total G_k = 14.54 + 3.38 = 17.92kN/m^2

4.3.3 Design Load on Beam

Design load 1.4G_k +1.6Q_k

Design dead load = 1.4 (17.72) = 25.09kN/m

Total design load on Beam = 1.4(17.92) +1.6(1.2) = 27.01kN/m

4.4 BEAM ANALYSIS USING CROSS - METHOD (BEAM A-A)

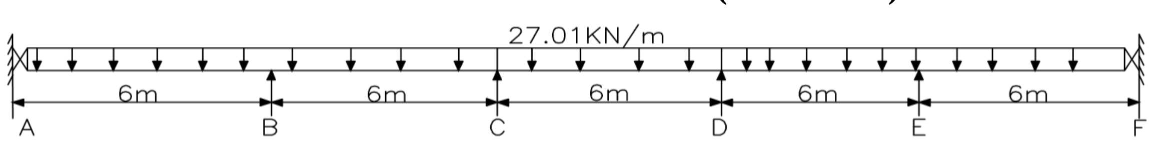


Figure 4.3: Beam A-A with symmetrical loadings

Fixed end moment

$M_{AB} = M_{BC} = M_{CD} = M_{DE} = M_{EF} = \frac{wl^2}{12} = \frac{27.01*6^2}{12}$ = 81.03

$M_{BA} = M_{CB}$ = -81.03, $M_{DC} = M_{ED}$ = -81.03, M_{FE} = (Due symmetry of the beam)

Beam rigidity

For exterior span

S = 0.8L = 0.8 * 6 = 4.8m (reducing the beam rigidity)

b= $b_w + \frac{lp}{10}$s (unsymmetrical span)

b= 30 + 0.1s = 30 + 0.1 (480) = 78 cm

h = 60 cm, h_f = 15cm, b_w = 30 cm

b_w/b = 0.384, h_f/h = 15/60 = 0.25

From Moment of inertia for flanged section table μ = 20.9

Therefore moment of inertia = $\frac{7.8*6^3}{20.9}$ = 80.6dm^4

For interior span (reference beam)

S = 0.6L = 0.6 * 6 = 3.6m

b = 30 +0.1(360) = 66cm

h = 60 cm, h_f = 15cm, b_w = 30 cm

b_w/b = 0.29, h_f/h = 15/60 = 0.25

From Moment of inertia for flanged section table μ = 24.0

Therefore moment of inertia = $\frac{6.6*6^3}{24}$ = 59.4dm^4

$\frac{80.6}{59.4}$ = 1.36

Distribution coefficient

$\underline{m}_{a1}$ = 0.8 * 1.36 = 1.09, $m_{a2} = \frac{4*1.36}{6}$ =0.91

$1.09 + 0.91 = 2$, $r_{a1} = \frac{1.09}{2} = 0.545$, $r_{a2} = \frac{0.91}{2} = 0.455$

$m_{ba} = \frac{1.36}{6} = 0.227$ $m_{bc} = \frac{1}{6} = 0.1667$

$0.227 + 0.1667 = 0.394$ $r_{ba} = \frac{0.227}{0.394} = 0.576$, $r_{bc} = \frac{0.1667}{0.394} = 0.423$

$m_{cb} = \frac{1}{6} = 0.1667$, $m_{cd} = \frac{2*1}{6} = 0.333$ (multiplied by 2 due to symmetrical span)

$0.1667 + 0.333 = 0.4997$

$r_{cb} = \frac{0.1667}{0.4997} = 0.333$, $r_{cd} = \frac{0.1667}{0.4997} = 0.667$

Table 4.2: Moment Distribution table for support moment

SUPPORT	A		B		C	
DF	0.545	0.455	0.576	0.423	0.333	0.667
FEM	-	81.03	-81.03	81.03	-81.03	81.03
BAL 1	-44.19	-36.87	0.00	0.00	0.00	0.00
CoF 1	-	0.00	-18.43	0.00	0.00	0.00
BAL 2	-	0.00	10.62	7.80	0.00	0.00
CoF 2	-	5.31	0.00	0.00	3.90	0.00
BAL 3	-2.90	-2.42	0.00	0.00	-1.30	-2.60
CoF 3	-	0.00	-1.21	-0.65	0.00	0.00
BAL 4	0.00	0.00	1.07	0.79	0.00	0.00
TOTM	**-47.09**	**47.05**	**-88.40**	**88.97**	**-78.10**	**78.43**

$M_{max} = \frac{27.01 * 6^2}{8} = 121.545$

$121.545 – 47.09 = 74.46$

$121.545 – 88.4 = 33.15$

$\frac{74.46+33.15}{2} = 53.82$

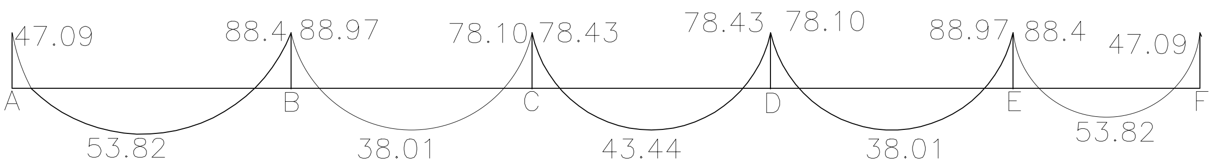


Figure 4.4: Bending moment (kNm) for beam A-A with symmetrical loadings

4.4.1 Reaction Forces

Taking moment at E

$(R_F * 6) – 27.01 *6 *3 = 0$

$R_F = 81.03kN = R_A$ (Due to symmetry of the section)

Taking moment at D

$(81.03 *12) – (27.01 * 6* 9) + (R_E * 6) – (27.01 * 6 *3) = 0$

$R_E =162.06$ kN $= R_B = R_C = R_D = R_E$ (Due to symmetry of the section)

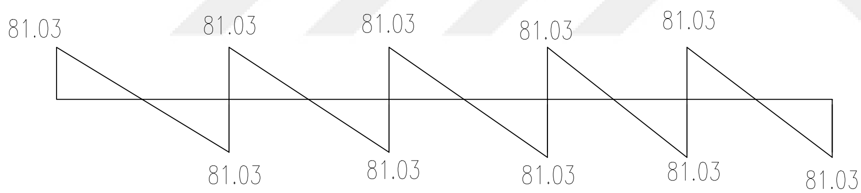


Figure 4.5: Shear force (kN) diagram of beam A-A

Due to the slabs having the same design load Beam A-A and Beam M-M will have the same design load therefore their analysis results will also be the same.

4.5 INTERNAL BEAM ANALYSIS FOR (BEAM B-B)

4.5.1 Beam Loadings and Dimensions

Beam dimension 30cm by 60cm

Design load for Slab S2 = 7.65kN/m

Design load for Slab S4 = 7.65kN/m

Beam self-weight = 25 * 0.3 * 0.45 = 3.38kN/m

Live load = 1.2kN/m

m = 1.18

Load transformation from S2 = $\frac{7.65*5}{3} * (\frac{3}{2} - \frac{1}{2(1.18)^2})$ = 14.54kN/m^2

Load transformation from S4 = $\frac{7.65*5}{3} * (\frac{3}{2} - \frac{1}{2(1.18)^2})$ = 14.54kN/m^2

G_k = (14.54 + 14.54) + 3.38 = 32.46

Total design load on Beam = 1.4(32.46) +1.6(1.2) = 47.36kN/m^2

Design dead load = 1.4 (32.46) = 45.44kN/m^2

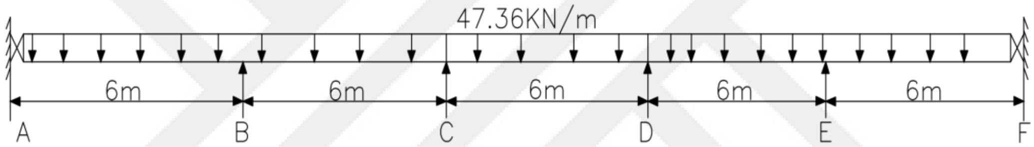


Figure 4.6: Beam B-B with symmetrical loadings

Fixed end moment

$M_{AB} = M_{BC} = M_{CD} = M_{DE} = M_{EF} = \frac{wl^2}{12} = \frac{47.36*6^2}{12}$ = 142.08(Due symmetry of the beam)

$M_{BA} = M_{CB}$ = -142.08, $M_{DC} = M_{ED}$ = -142.08, M_{FE} = (Due symmetry of the beam)

Beam rigidity

For exterior span

S = 0.8L = 0.8 * 12 = 9.6m

b= 30 + 0.2s = 30 + 0.2 (960) = 222cm (Symmetrical section)

h = 60 cm, h_f = 15cm, b_w = 30 cm

b_w/b = 0.14, h_f/h = 15/60 = 0.25

From Moment of inertia for flanged section table μ = 38.5

Therefore moment of inertia = $\frac{2.22*6^3}{38.5}$ = 12.46dm^4

For interior span

S = 0.6L = 0.6 * 12 = 7.2m

b = 30 +0.2(720) = 174cm

h = 60 cm, h_f = 15cm, b_w = 30 cm

b_w/b = 0.17, h_f/h = 15/60 = 0.25

From Moment of inertia for flanged section table μ = 35.3

Therefore moment of inertia = $\frac{1.74*6^3}{35.3}$ = 10.65dm^4

$\frac{12.46}{10.65}$ = 1.17

Distribution coefficient

m_{a1} = 0.8 * 1.17 = 0.94, $m_{a2} = \frac{4*1.17}{6}$ =0.78

0.94 + 0.78 = 1.72, $\frac{0.94}{1.72}$ = 0.547, $\frac{0.78}{1.72}$ = 0.454

$m_{ba} = \frac{1.17}{6}$ = 0.195 $m_{bc} = \frac{1}{6}$ = 0.1667

0.195 + 0.1667 = 0.362 $r_{ba} = \frac{0.195}{0.362}$ = 0.539, $r_{bc} = \frac{0.1667}{0.362}$ = 0.461

$m_{cb} = \frac{1}{6}$ = 0.1667, $m_{cd} = \frac{2*1}{6}$ = 0.333

0.1667 + 0.333 = 0.4997

$r_{cb} = \frac{0.1667}{0.4997}$ = 0.333, $r_{cd} = \frac{0.1667}{0.4997}$ = 0.667

Table 4.3: Moment Distribution table for support moment of symmetrical loadings

SUPPORT	A		B		C	
DF	0.547	0.454	0.539	0.461	0.333	0.667
FEM	-	142.08	-142.08	142.08	-142.08	142.08
BAL 1	-77.72	-64.50	0.00	0.00	0.00	0.00
CoF 1	-	0.00	-32.25	0.00	0.00	0.00
BAL 2	-	0.00	17.38	14.87	0.00	0.00
CoF 2	-	8.69	0.00	0.00	7.43	0.00
BAL 3	-4.75	-3.95	0.00	0.00	-2.48	-4.96
CoF 3	-	0.00	-1.97	-1.24	0.00	0.00
BAL 4	0.00	0.00	1.73	1.48	0.00	0.00
TOTM	**-82.47**	**82.32**	**-156.65**	**157.19**	**-136.797**	**137.12**

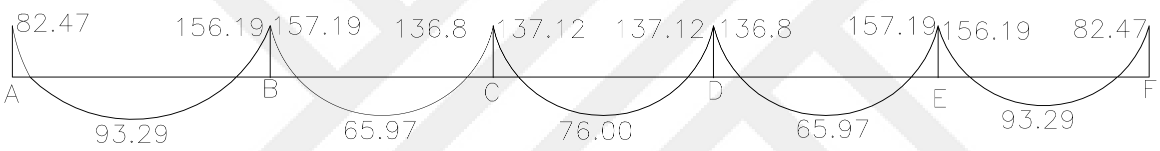


Figure 4.7: Bending moment (kNm) diagram of beam B-B with symmetrical loadings

4.5.2 Reaction Forces

Taking moment at E

$(R_F * 6) – 47.36 *6*3 = 0$

R_F =142.08kN = R_A (Due to symmetry of the section)

Taking moment at D

$(142.08 *12) – (47.36 * 6* 9) + (R_E * 6) – (47.36 * 6 *3) = 0$

R_E =284.16 kN = R_B = R_C = R_D = R_E (Due to symmetry of the section)

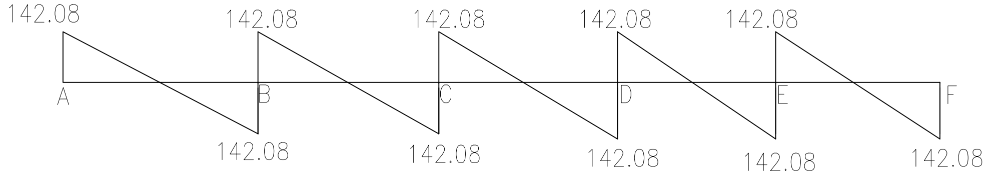


Figure 4.8: Shear force (kN) diagram of beam B-B with symmetrical loadings

Use Beam B-B analysis for beam E-E, F-F, I-I, J-J

4.6 ANALYSIS OF BEAM C - C

Beam dimension 30cm by 45cm

$M_2 = 1.18$, $m_4 = 1.18$,

4.6.1 Load Transformation

Load transformation from S2 = $\frac{7.65*5}{3} * (\frac{3}{2} - \frac{1}{2(1.18)^2}) = 14.54kN/m^2$

Load transformation from S4 = $\frac{7.65*5}{3} * (\frac{3}{2} - \frac{1}{2(1.18)^2}) = 14.54kN/m^2$

Load transformation from cantilever slab D1 = 7.65 * 1.69 = $12.92kN/m^2$

Beam self-weight = 25 * 0.3 * 0.45 = 3.38kN/m

G_{K1} = S2 + S4 = 14.54 + 14.54 + 3.38 = $32.46kN/m^2$

G_{K2} = S2 + D1 = 14.54 +12.92 +3.38 = $30.84kN/m^2$

Live load = 1.2kN/m

Total design load exterior span= 1.4(32.46) +1.6 (1. 2) = 47.36kN/m

Total design load interior span = 1.4(30.84) + 1.6 (1.2) = 45.1kN/m

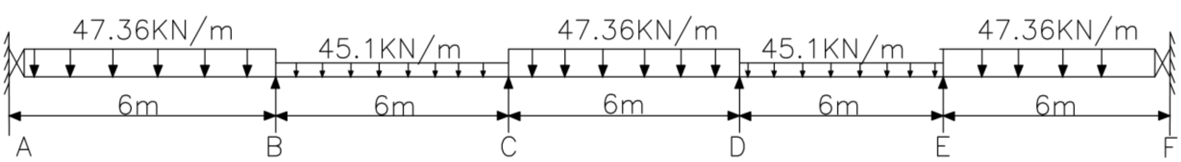


Figure 4.9: Beam C-C with unsymmetrical loading

Fixed end moment

$M_{AB} = \frac{wl^2}{12} = \frac{47.36*6^2}{12} = 142.08$, $M_{BA} = -142.08$, $M_{BC} = \frac{45.1*6^2}{12} = = 135.3$, $M_{CB} = -135.3$

$M_{CD} = = 142.08$, $M_{CB} = -142.08$, $M_{DE} = 135.3$, $M_{ED} = -135.3$

For exterior span

$S = 0.8L = 0.8 * 12 = 9.6m$

$b = 30 + 0.2s = 30 + 0.2\ (960) = 222cm$ (Symmetrical section)

$h = 60$ cm, $h_f = 15cm$, $b_w = 30$ cm

$b_w/b = 0.14$, $h_f/h = 15/60 = 0.25$

From Moment of inertia for flanged section table $\mu = 38.5$

Therefore moment of inertia = $\frac{2.22*6^3}{38.5} = 12.46dm^4$

For interior span

$S = 0.6L = 0.6 * 7.69 = 4.614m$

$b = 30 + 0.2(461.4) = 122.8cm$

$h = 60$ cm, $h_f = 15cm$, $b_w = 30$ cm

$b_w/b = 0.244$, $h_f/h = 15/60 = 0.25$

From Moment of inertia for flanged section table $\mu = 27.8$

Therefore moment of inertia = $\frac{1.228*6^3}{27.8} = 9.54dm^4$

$\frac{38.5}{27.8} = 1.39$

Distribution coefficient

$\underline{m}_{a1} = 0.8 * 1.39 = 1.112$, $m_{a2} = \frac{4*1.39}{6} = 0.93$

$1.112 + 0.93 = 2.042$, $\frac{1.112}{2.042} = 0.545$, $\frac{0.93}{2.042} = 0.455$

$m_{ba} = \frac{1.112}{6} = 0.185$ $m_{bc} = \frac{1}{6} = 0.1667$

$0.185 + 0.1667 = 0.352$ $r_{ba} = \frac{0.185}{0.352} = 0.525$, $r_{bc} = \frac{0.1667}{0.352} = 0.474$

$m_{cb} = \frac{1}{6} = 0.1667, \qquad m_{cd} = \frac{1}{6} = 0.1667$

$0.1667 + 0.1667 = 0.3334$

$r_{cb} = \frac{0.1667}{0.3334} = 0.5$, $r_{cd} = \frac{0.1667}{0.3334} = 0.5$

Table 4.4: Moment Distribution table for support moment of case 3

SPAN	A		B		C		D		E		F	
DF	0.545	0.455	0.525	0.474	0.500	0.500	0.500	0.500	0.474	0.525	0.455	0.545
FEM	**0.00**	**142.08**	**-142.08**	**135.30**	**-135.30**	**142.08**	**-142.08**	**135.30**	**-135.30**	**142.08**	**-142.08**	
BAL 1	-77.43	-64.65	3.56	3.21	-3.39	-3.39	3.39	3.39	-3.21	-3.56	64.65	77.43
CoF 1	0.00	1.78	-32.32	-1.70	1.61	1.70	-1.70	-1.61	1.70	32.32	-1.78	
BAL 2	-0.97	-0.81	17.86	16.12	-1.65	-1.65	1.65	1.65	-16.12	-17.86	0.81	0.97
CoF 2	0.00	8.93	-0.40	-0.83	8.06	0.83	-0.83	-8.06	0.83	0.40	-8.93	
BAL 3	-4.87	-4.06	0.65	0.58	-4.44	-4.44	4.44	4.44	-0.58	-0.65	4.06	4.87
CoF 3	0.00	0.32	-2.03	-2.22	0.29	2.22	-2.22	-0.29	2.22	2.03	-0.32	
BAL 4	-0.18	-0.15	2.23	2.02	-1.26	-1.26	1.26	1.26	-2.02	-2.23	0.15	0.18
CoF 4	0.00	1.12	-0.07	-0.63	1.01	0.63	-0.63	-1.01	0.63	0.07	-1.12	
BAL 5	-0.61	-0.51	0.37	0.33	-0.82	-0.82	0.82	0.82	-0.33	-0.37	0.51	0.61
CoF 5	0.00	0.18	-0.25	-0.41	0.17	0.41	-0.41	-0.17	0.41	0.25	-0.18	
BAL 6	-0.10	-0.08	0.35	0.31	-0.29	-0.29	0.29	0.29	-0.31	-0.35	0.10	0.10
CoF 6	0.00	0.17	-0.04	-0.14	0.16	0.14	-0.14	-0.16	0.14	0.05	-0.17	
BAL 7	-0.09	-0.08	0.10	0.09	-0.15	-0.15	0.15	0.15	-0.09	-0.10	0.08	0.09
CoF 7	0.00	0.05	-0.04	-0.08	0.04	0.08	-0.08	-0.05	0.08	0.04	-0.05	
TOTM	-84.25	84.30	-152.14	151.97	-135.96	136.08	-136.08	135.96	-151.98	152.14	-84.28	84.25

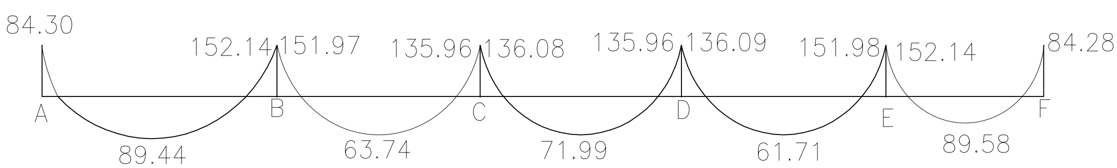


Figure 4.10: Beam C-C bending moment (kNm) diagram with unsymmetrical loadings

4.6.2 Reaction Forces

Taking moment at E

$(R_F * 6) – 47.36 * 6 * 3 = 0$

$R_F = 142.08kN = R_A$

Taking moment at D

$(142.08 * 12) – (47.36 * 6 * 9) + (R_E * 6) – (45.1 * 6 * 3) = 0$

$R_E = 277.38kN = R_B$

Taking moment at C

(142.08 * 18) – (47.36 * 6 * 15) + (277.38 * 12) – (45.1 * 6 * 9) + (RD * 6) – (47.36 * 6 * 3) = 0

$R_C = R_D = 280.98$kN

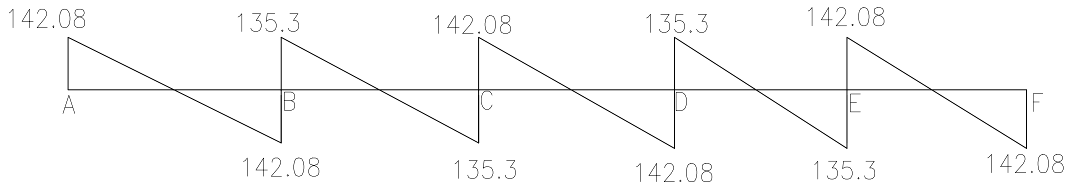


Figure 4.11: Beam C-C shear force (kN) diagram

Use Beam C-C analysis for beam D-D, G-G, H-H, K-K, L-L

CHAPTER FIVE

5.1 LOAD TRANSFORMATION ON FRAMES AND ANALYSIS

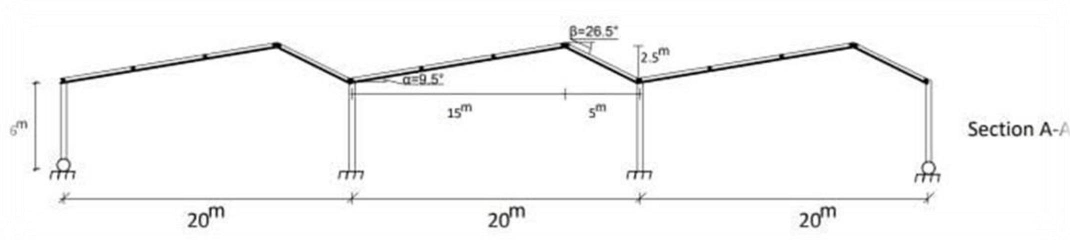


Figure 5.0: The bay frame (case study)

5.1.2 Frame data

Beam dimensions 45cm by 125cm

Column dimension 50cm by 25cm

Live load = 1.2kN/m`

Frame beam depth = 1.25 – 0.15 = 1.10cm

Beam self-weight = 0.45 * 1.25 * 25 = 12.38kN/m

5.1.3 Load Transformation from slabs

Short span

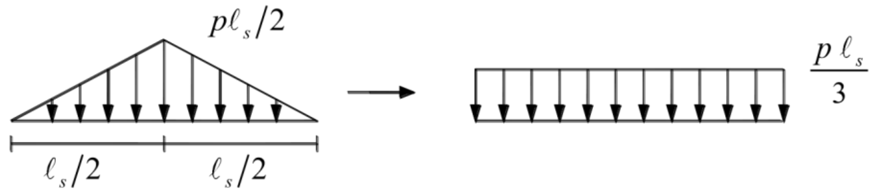


Figure 5.1: Load transformation from the short span of the slab

Design load of slab = 7.65kN/m

$\frac{7.65*5.07}{3}$ = 12.93kN/m

$\frac{7.65*5.59}{3}$ = 14.26kN/m

Load from D2 slab = 7.65 * 2 = 15.3kN/m^2

5.2 DESIGN LOAD FOR EXTERNAL FRAMES

$G_{k1} = 12.93 + 12.38 = 25.31kN/m^2$

$G_{k2} = 14.26 + 12.38 = 26.64kN/m^2$

Design load on = 1.4 (25.31) + 1.6 (1.2) = 37.35kN/m

Design load on CD, FG, IJ= 1.4 (26.64) + 1.6 (1.2) = 39.22kN/m

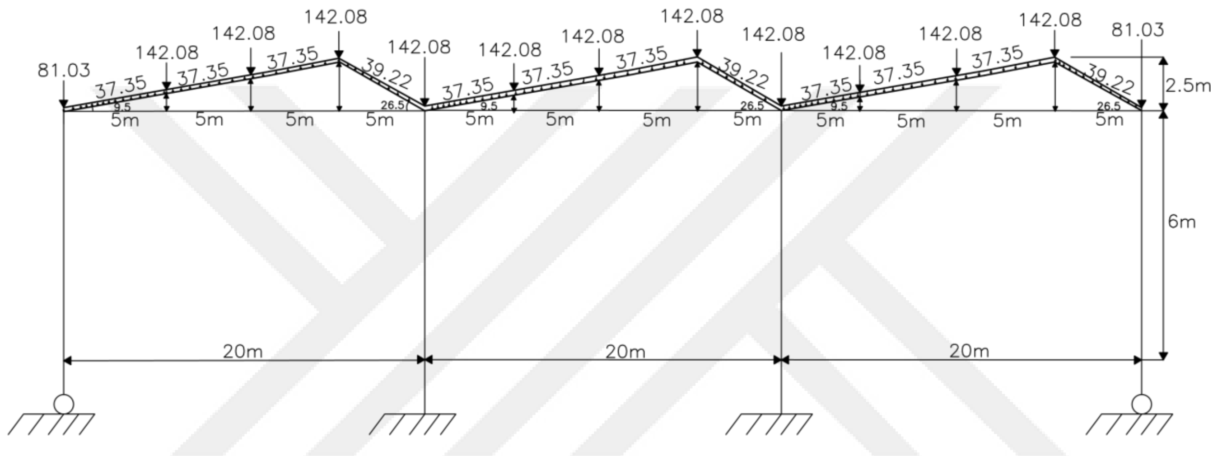


Figure 5.2: Showing loads transformed on external frames.

5.3 DESIGN LOAD FOR INTERNAL FRAMES

$G_{k1} = 12.93 + 12.93 +12.38 = 38.24kN/m^2$

$G_{k2} = 12.93 + 15.3 + 12.38 = 40.43kN/m^2$

$G_{k3} = 14.26 + 14.26 + 12.38 = 42.78 kN/m^2$

Design load = 1.4 (38.24) + 1.6 (1.2) = 55.46kN/m

Design load = 1.4 (40.43) + 1.6 (1.2) = 58.52kN/m

Design load on CD, FG, IJ = 1.4 (42.78) + 1.6 (1.2) = 61.81kN/m

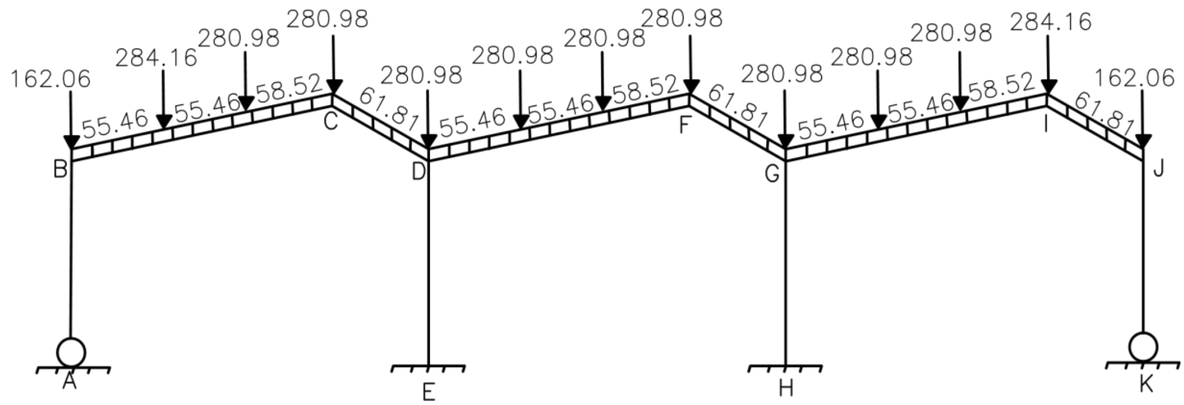


Figure 5.3: Showing loads transformed on internal frames.

5.4 ANALYSIS OF INTERNAL FRAME

5.4.1 Frame Data

$|AB| = |DE| = |GH| = |JK| = 6\text{m}$

$|BC| = |DF| = |GI| = 15/\cos 9.5 = 15.21\text{m}$

$|CD| = |FG| = |IJ| = 5 / \cos 26.5 = 5.59\text{m}$

Moment of Inertia for Columns

$I = \frac{bd^3}{12} = \frac{500*250^3}{12} = 651.04 * 10^6 \text{mm}^4$

$E = 300\text{kN/mm}^2$

$EI = 195312 \text{ kN/m}^2$

Moment of Inertia for Beams

$I = \frac{bd^3}{12} = \frac{450*1250^3}{12} = 73242.2 * 10^6 \text{mm}^4$

$E = 300\text{kN/mm}^2$

$EI = 21972660\text{kN/m}^2$

5.4.2 Determination of Degree of Indeterminacy

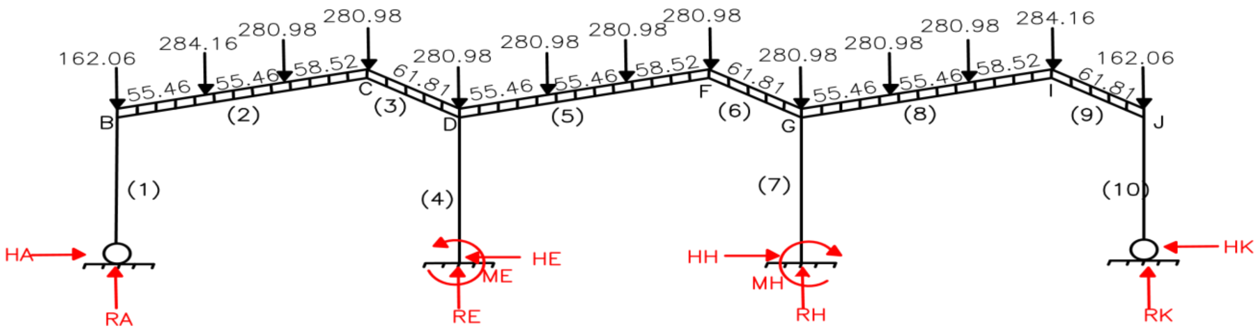


Figure 5.4: Internal frame with redundant forces

Number of members (m) = 10

Number of joints(j) = 11

Number of reaction (r) = 10

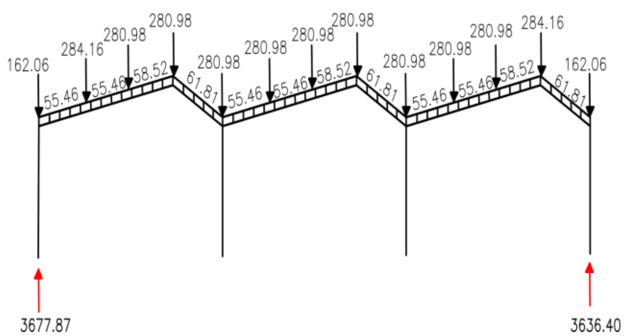
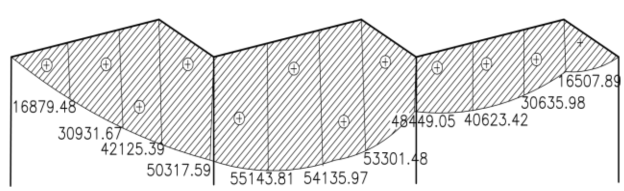
Degree of indeterminacy = 3(m) + r -3(j)

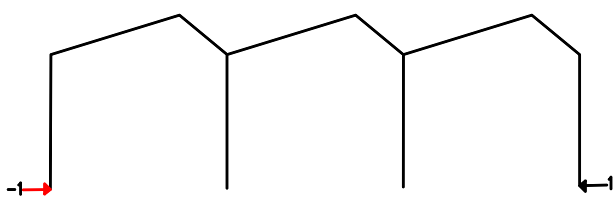
Degree of indeterminacy = 3(10) + 10 - 3(11) = 7

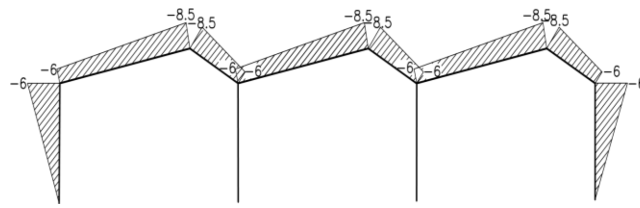
H_k, R_h, H_h, M_h, R_e, M_e, and H_e are taken as redundant forces

5.4.3 Analysis of Frame

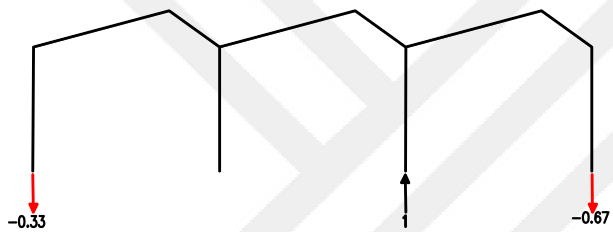
Table 5.1: Load cases and respective bending moment diagram

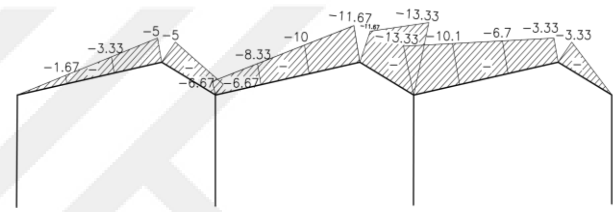
Load Case and Reactions	Bending moment Diagram
	 m_0


−1
−1

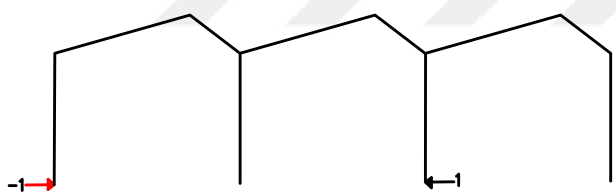

−8.5
−8.5
−6
−6
−6
−6
−8.5
−8.5
−6
−6
−8.5
−8.5
−6
−6

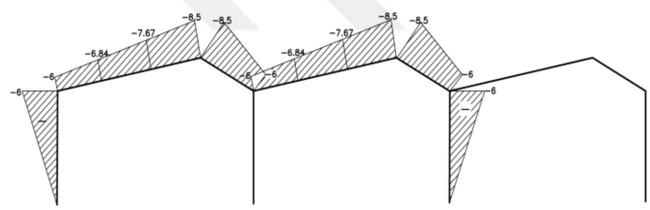
M$_1$


−0.33
1
−0.67

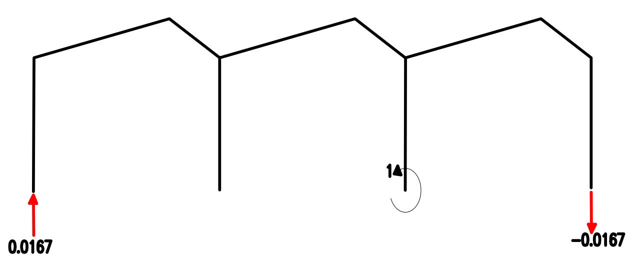

−1.67
−3.33
−5
−5
−6.67
−6.67
−8.33
−10
−11.67
−13.33
−13.33
−10.1
−6.7
−3.33
−3.33

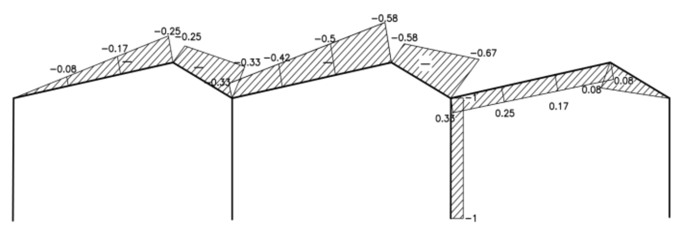
M$_2$


−1
−1

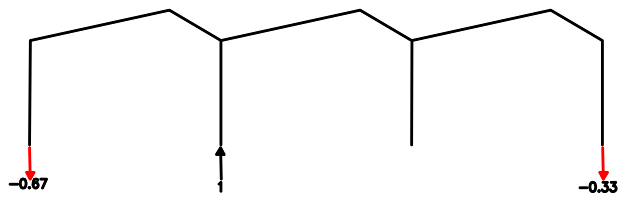
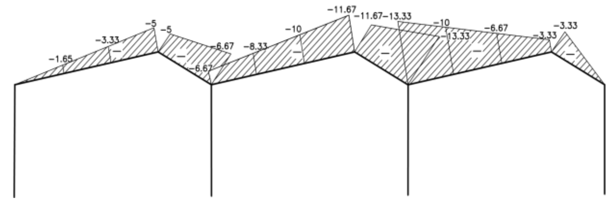
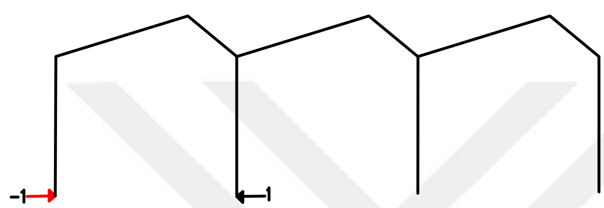
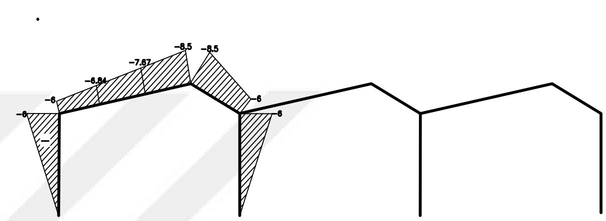
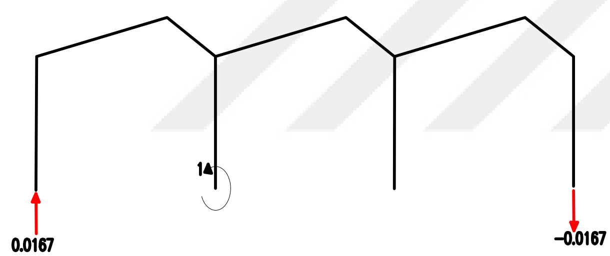
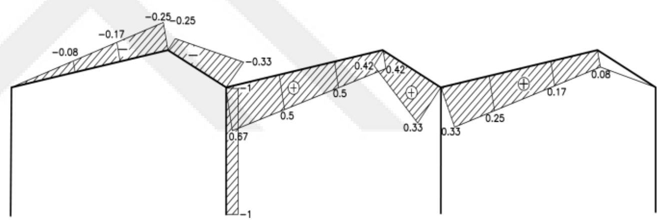

−6
−6
−6.84
−7.67
−8.5
−8.5
−6
−6
−6.84
−7.67
−8.5
−8.5
−6
−6

M$_3$


0.0167
1
−0.0167


−0.08
−0.17
−0.25
−0.25
−0.33
−0.33
−0.42
−0.5
−0.58
−0.58
−0.67
0.33
0.25
0.17
0.08
0.08
−1

M$_4$

	 M5
	 M6
	 M7

For this case the compatibility equations are as follows:

$$\begin{bmatrix} \Delta 10 \\ \Delta 20 \\ \Delta 30 \\ \Delta 40 \\ \Delta 50 \\ \Delta 60 \\ \Delta 70 \end{bmatrix} + \begin{bmatrix} f11 & f21 & f31 & f41 & f51 & f61 & f71 \\ f12 & f22 & f32 & f42 & f52 & f62 & f72 \\ f13 & f23 & f33 & f43 & f53 & f63 & f73 \\ f14 & f24 & f34 & f44 & f54 & f64 & f74 \\ f15 & f25 & f35 & f45 & f55 & f65 & f75 \\ f16 & f26 & f36 & f46 & f56 & f66 & f76 \\ f17 & f27 & f37 & f47 & f57 & f67 & f77 \end{bmatrix} * \begin{bmatrix} R1 \\ R2 \\ R3 \\ R4 \\ R5 \\ R6 \\ R7 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

By using integral table to multiply the bending moment diagrams to obtain the coefficient matrix and constant

$f_{11} = \int_0^L \frac{m_1{}^2 dx}{EI} = 0.0007904$ $f_{22} = \int_0^L \frac{m_2{}^2 dx}{EI} = 0.0001148$, $f_{33} = \int_0^L \frac{m_3^2 dx}{EI} = 0.0008384$

$f_{44} = \int_0^L \frac{m_4{}^2 dx}{EI} = 0.00003099$ $f_{55} = \int_0^L \frac{m_5{}^2 dx}{EI} = 0.0001687$ $f_{66} = \int_0^L \frac{m_6{}^2 dx}{EI} = 0.0007882$

$f_{77} = \int_0^L \frac{m_7{}^2 dx}{EI} = 0.0000307$ $f_{12} = f_{21} = \int_0^L \frac{m_1 m_2 dx}{EI} = 0.0001162$, $f_{13} = f_{31} = \int_0^L \frac{m_1 m_3 dx}{EI} = 0.0004694$

$f_{14} = f_{41} = \int_0^L \frac{m_1 m_4 dx}{EI} = 0.00000485$ $f_{15} = f_{51} = \int_0^L \frac{m_1 m_5 dx}{EI} = 0.0022874$

$f_{16} = f_{16} = \int_0^L \frac{m_1 m_6 dx}{EI} = 0.0004192$ $f_{17} = f_{71} = \int_0^L \frac{m_1 m_7 dx}{EI} = -0.000003502$

$f_{23} = f_{32} = \int_0^L \frac{m_3 m_2 dx}{EI} = 0.0000875$ $f_{24} = f_{42} = \int_0^L \frac{m_4 m_2 dx}{EI} = 0.0000037458$

$f_{25} = f_{52} = \int_0^L \frac{m_2 m_5 dx}{EI} = 0.0001326$ $f_{26} = f_{62} = \int_0^L \frac{m_2 m_6 dx}{EI} = 0.0000236$

$f_{27} = f_{72} = \int_0^L \frac{m_2 m_7 dx}{EI} = -0.000005082$

$f_{34} = f_{43} = \int_0^L \frac{m_3 m_4 dx}{EI} = 0.0000922$ $f_{35} = f_{53} = \int_0^L \frac{m_3 m_5 dx}{EI} = 0.00116$

$f_{36} = f_{63} = \int_0^L \frac{m_3 m_6 dx}{EI} = 0.0004192$ $f_{37} = f_{73} = \int_0^L \frac{m_3 m_7 dx}{EI} = -0.00000233$

$f_{45} = f_{54} = \int_0^L \frac{m_4 m_5 dx}{EI} = 0.00000545$ $f_{46} = f_{64} = \int_0^L \frac{m_4 m_6 dx}{EI} = 0.000001173$

$f_{47} = f_{74} = \int_0^L \frac{m_4 m_7 dx}{EI} = -0.0000002003$ $f_{56} = f_{65} = \int_0^L \frac{m_5 m_6 dx}{EI} = 0.0000471$

$f_{57} = f_{75} = \int_0^L \frac{m_5 m_7 dx}{EI} = -0.000004149$ $f_{67} = f_{76} = \int_0^L \frac{m_6 m_7 dx}{EI} = 0.00009337$

$\Delta_{10} = \int_0^L \frac{m_1 m_0 dx}{EI} = -6.73$ $\Delta_{20} = \int_0^L \frac{m_2 m_0 dx}{EI} = -0.66$ $\Delta_{30} = \int_0^L \frac{m3 m_0 dx}{EI} = -0.54$

$\Delta_{40} = \int_0^L \frac{m_4 m_0 dx}{EI}, = -0.03$ $\Delta_{50} = \int_0^L \frac{m5\ m_0 dx}{EI} = -0.76$ $\Delta_{60} = \int_0^L \frac{m6\ m_0 dx}{EI} = -0.18$

$\Delta_{70} = \int_0^L \frac{m7\ m_0 dx}{EI} = 0.03$

Coefficient matrix

$$\begin{bmatrix} -6.84 \\ -0.63 \\ -0.51 \\ -0.02 \\ -0.70 \\ -0.17 \\ -0.02 \end{bmatrix} + \begin{bmatrix} 0.0007904 & 0.0001162 & 0.0004694 & 0.00000485 & 0.0022874 & 0.0004192 & -0.000003502 \\ 0.0001162 & 0.0001148 & 0.0000875 & 0.0000037458 & 0.0001326 & 0.0000236 & -0.000005082 \\ 0.0004694 & 0.0000875 & 0.0008384 & 0.0000922 & 0.00116 & 0.0004192 & -0.00000233 \\ 0.00000485 & 0.0000037458 & 0.0000922 & 0.00003099 & 0.00000545 & 0.000001173 & -0.0000002003 \\ 0.0022874 & 0.0001326 & 0.00116 & 0.00000545 & 0.0001687 & 0.0000471 & -0.000004149 \\ 0.0004192 & 0.0000236 & 0.0004192 & 0.000001173 & 0.0000471 & 0.0007882 & 0.00009337 \\ -0.000003502 & -0.000005082 & -0.00000233 & -0.0000002003 & -0.000004149 & 0.00009337 & 0.00000485 \end{bmatrix} \begin{bmatrix} R1 \\ R2 \\ R3 \\ R4 \\ R5 \\ R6 \\ R7 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

Matrix inverse

$$\begin{bmatrix} -3.27 & -533.89 & 1.17 & -19.98 & 467.47 & -12.21 & 11.52 \\ -533.91 & 10402.19 & 1845.022 & 4281.74 & -200.5 & 1218.25 & -2185.35 \\ 1.25 & -1846.291 & 5123.5 & -14755.6 & -1540.32 & -4120.05 & 12495.86 \\ -20.38 & 4289.64 & -14757.03 & 74897.58 & 441.48 & 11846.833 & -35888.4 \\ 467.47 & -200.4 & -154.38 & 441.73 & -74.46 & -245.26 & 747.34 \\ -12.21 & 1218.98 & -4119.84 & 11844.96 & -245.31 & 5370.50 & -16401.54 \\ 11.52 & -2185.35 & 12495.86 & -35888.4 & 747.34 & -16401.54 & 82927.7 \end{bmatrix} * \begin{bmatrix} 6.84 \\ 0.63 \\ 0.51 \\ 0.02 \\ 0.70 \\ 0.17 \\ -0.02 \end{bmatrix} = \begin{bmatrix} -30.13 \\ 2538.91 \\ 13.35 \\ -24.91 \\ 2540.62 \\ -10.01 \\ -16.93 \end{bmatrix} \begin{bmatrix} -30.13 \\ 2538.91 \\ 13.35 \\ -24.91 \\ 2540.62 \\ -10.01 \\ -16.93 \end{bmatrix} = \begin{bmatrix} R1 \\ R2 \\ R3 \\ R4 \\ R5 \\ R6 \\ R7 \end{bmatrix} = \begin{bmatrix} Hk \\ Rh \\ Hh \\ Mh \\ Re \\ He \\ Me \end{bmatrix}$$

Check;

$$\begin{bmatrix} -6.84 \\ -0.63 \\ -0.51 \\ -0.02 \\ -0.70 \\ -0.17 \\ 0.02 \end{bmatrix} + \begin{bmatrix} 0.0007904 & 0.0001162 & 0.0004694 & 0.00000485 & 0.0022874 & 0.0004192 & -0.000003502 \\ 0.0001162 & 0.0001148 & 0.0000875 & 0.0000037458 & 0.0001326 & 0.0000236 & -0.000005082 \\ 0.0004694 & 0.0000875 & 0.0008384 & 0.0000922 & 0.00116 & 0.0004192 & -0.00000233 \\ 0.00000485 & 0.0000037458 & 0.0000922 & 0.00003099 & 0.00000545 & 0.000001173 & -0.0000002003 \\ 0.0022874 & 0.0001326 & 0.00116 & 0.00000545 & 0.0001687 & 0.0000471 & -0.000004149 \\ 0.0004192 & 0.0000236 & 0.0004192 & 0.000001173 & 0.0000471 & 0.0007882 & 0.00009337 \\ -0.000003502 & -0.000005082 & -0.00000233 & -0.0000002003 & -0.000004149 & 0.00009337 & 0.00000485 \end{bmatrix} \begin{bmatrix} -30.13 \\ 2538.91 \\ 13.35 \\ -24.91 \\ 2540.62 \\ -10.01 \\ -16.93 \end{bmatrix} = \begin{bmatrix} 0.00739 \\ 0.0010085 \\ 0.00384 \\ 0.00 \\ 0.01591 \\ 0.00326 \\ 0.00 \end{bmatrix}$$

By obtaining these moment and reaction forces the structure is now determinate and can now be analysed by law of static or progress using the Force Method so that bending moment can and shear force can be drawn

Horizontal forces; 0

-30.13 + 13.25 -10.01 + H_a = 0

H_a = 26.89kN

Taking moment at E; 0

R_a * 20 -162.07 *20 – (55.46 * 10.14 * 15) – (284.16 * 15) – (58.52 * 5.07 *7.5) –(280.98 * 5)-(61.81 *5.59*2.5) – (280.92 * 10) – 13.12 = 0

Ra = 950.2 kN

Taking moment at H; 0

R_j * 20 – (162.07 * 20) – (61.81 * 5.59 * 17.5) – (284.16 * 15) – (58.52 * 5.07 * 12.5) – (280.98 * 10) – (55.46 * 10.14 * 5) – (280.98 * 5) + 22.80 = 0

R_j = 1003.32kN

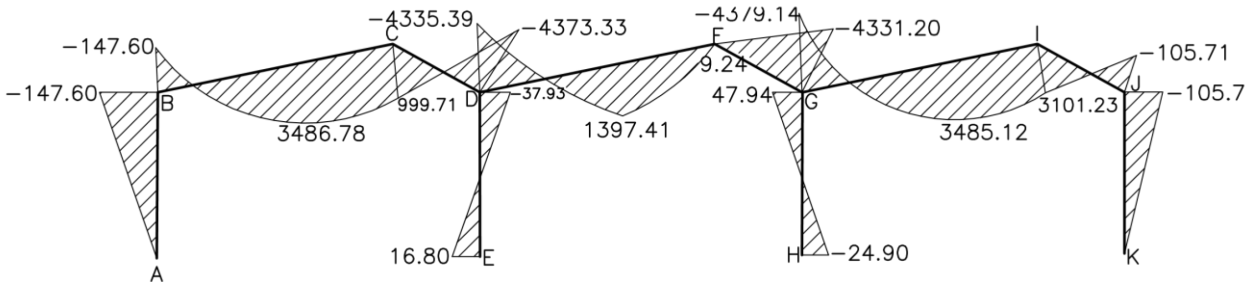


Figure 5.5: Bending moment (kNm) diagram of internal frame due to vertical loadings

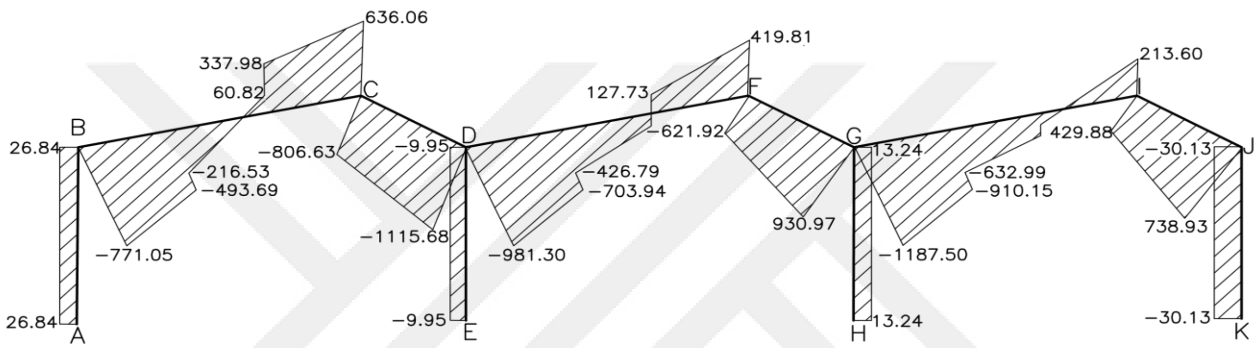


Figure 5.6: Shear force (kN) diagram internal of frame due to vertical loadings

5.5 ANALYSIS FOR EXTERNAL FRAME

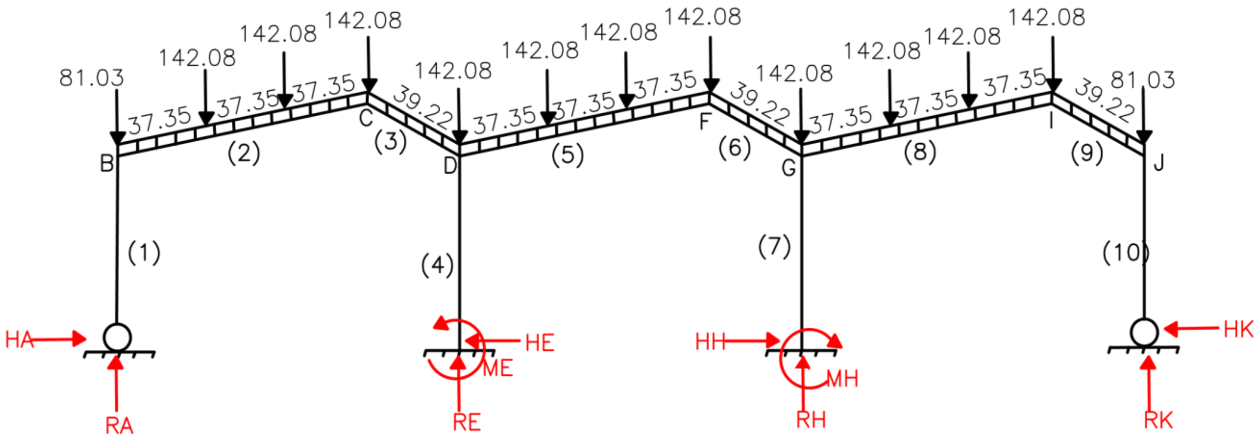


Figure 5.7: External frame with redundant forces

Number of members (m) = 10

Number of joints(j) = 11

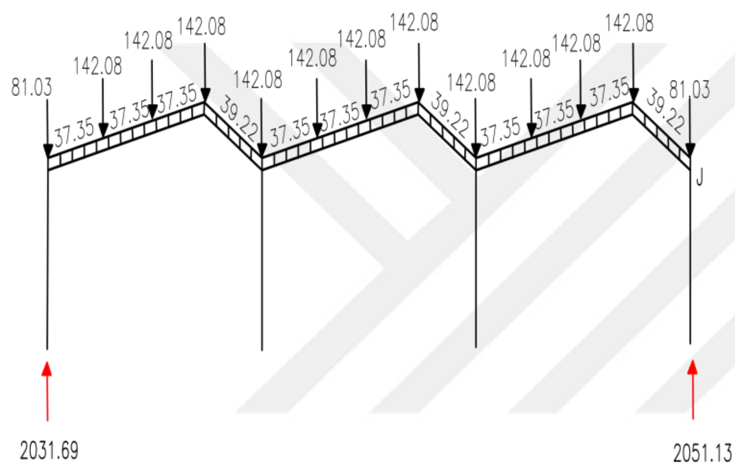
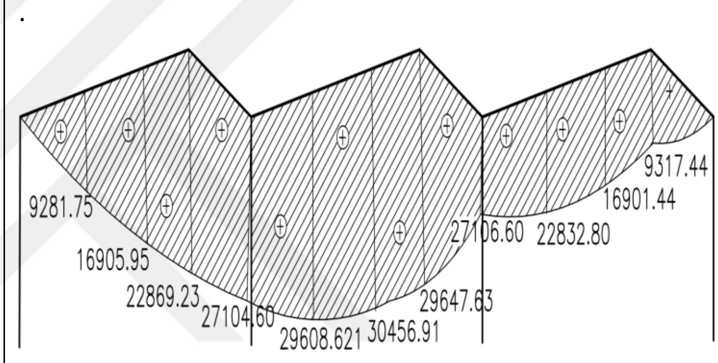
Number of reaction (r) = 10

Degree of indeterminacy = 3(m) + r -3(j)

Degree of indeterminacy = 3(10) + 10 -3(11) = 7

H_k, R_h, H_h, M_h, R_e, M_e, and H_e are taken as redundant forces

Table 5.2: Load cases and respective bending moment diagram

Load cases and reaction	Bending moment diagram
	 M_0

The external frame will have the coefficient matrix due to having the same unit loadings but will have different constants.

$$\begin{bmatrix}-3.54\\-0.36\\-0.29\\-0.01\\-0.4\\-0.09\\0.02\end{bmatrix}+\begin{bmatrix}0.0007904 & 0.0001162 & 0.0004694 & 0.00000485 & 0.0022874 & 0.0004192 & -0.000003502\\0.0001162 & 0.0001148 & 0.0000875 & 0.0000037458 & 0.0001326 & 0.0000236 & -0.000005082\\0.0004694 & 0.0000875 & 0.0008384 & 0.0000922 & 0.00116 & 0.0004192 & -0.00000233\\0.00000485 & 0.0000037458 & 0.0000922 & 0.00003099 & 0.00000545 & 0.000001173 & -0.0000002003\\0.0022874 & 0.0001326 & 0.00116 & 0.00000545 & 0.0001687 & 0.0000471 & -0.000004149\\0.0004192 & 0.0000236 & 0.0004192 & 0.000001173 & 0.0000471 & 0.0007882 & 0.00009337\\-0.000003502 & -0.000005082 & -0.00000233 & -0.0000002003 & -0.000004149 & 0.00009337 & 0.00000485\end{bmatrix}\begin{bmatrix}R1\\R2\\R3\\R4\\R5\\R6\\R7\end{bmatrix}=\begin{bmatrix}0\\0\\0\\0\\0\\0\\0\end{bmatrix}$$

$$\begin{bmatrix}-3.27 & -533.89 & 1.17 & -19.98 & 467.47 & -12.21 & 11.52\\-533.91 & 10402.19 & 1845.022 & 4281.74 & -200.5 & 1218.25 & -2185.35\\1.25 & -1846.291 & 5123.5 & -14755.6 & -1540.32 & -4120.05 & 12495.86\\-20.38 & 4289.64 & -14757.03 & 74897.58 & 441.48 & 11846.833 & -35888.4\\467.47 & -200.4 & -154.38 & 441.73 & -74.46 & -245.26 & 747.34\\-12.21 & 1218.98 & -4119.84 & 11844.96 & -245.31 & 5370.50 & -16401.54\\11.52 & -2185.35 & 12495.86 & -35888.4 & 747.34 & -16401.54 & 82927.7\end{bmatrix}*\begin{bmatrix}3.54\\0.36\\0.29\\0.01\\0.4\\0.09\\-0.02\end{bmatrix}=\begin{bmatrix}-13.07\\1479.51\\5.92\\-12.11\\1480.26\\-4.57\\8.57\end{bmatrix}\begin{bmatrix}-13.07\\1479.51\\5.92\\-12.11\\1480.26\\-4.57\\8.57\end{bmatrix}=\begin{bmatrix}R1\\R2\\R3\\R4\\R5\\R6\\R7\end{bmatrix}=\begin{bmatrix}Hk\\Rh\\Hh\\Mh\\Re\\He\\Me\end{bmatrix}$$

Check:

$$\begin{bmatrix}-3.54\\-0.36\\-0.29\\-0.01\\-0.4\\-0.09\\0.02\end{bmatrix}+\begin{bmatrix}0.0007904 & 0.0001162 & 0.0004694 & 0.00000485 & 0.0022874 & 0.0004192 & -0.000003502\\0.0001162 & 0.0001148 & 0.0000875 & 0.0000037458 & 0.0001326 & 0.0000236 & -0.000005082\\0.0004694 & 0.0000875 & 0.0008384 & 0.0000922 & 0.00116 & 0.0004192 & -0.00000233\\0.00000485 & 0.0000037458 & 0.0000922 & 0.00003099 & 0.00000545 & 0.000001173 & -0.0000002003\\0.0022874 & 0.0001326 & 0.00116 & 0.00000545 & 0.0001687 & 0.0000471 & -0.000004149\\0.0004192 & 0.0000236 & 0.0004192 & 0.000001173 & 0.0000471 & 0.0007882 & 0.00009337\\-0.000003502 & -0.000005082 & -0.00000233 & -0.0000002003 & -0.000004149 & 0.00009337 & 0.00000485\end{bmatrix}\begin{bmatrix}-13.07\\1479.51\\5.92\\-12.11\\1480.26\\-4.57\\8.57\end{bmatrix}=\begin{bmatrix}0.0021\\0.004\\0.014\\0.00495\\-0.001545\\0.0140\\0.00667\end{bmatrix}$$

By obtaining these moment and reaction forces the structure is now determinate and can now be analysed by law of static or progress using the Force Method so that bending moment can and shear force can be drawn

Horizontal forces; 0

$-13.07 + 5.92 - 4.56 + H_a = 0$

$H_a = 11.71$kN

Taking moment at E; 0

$(R_a * 20) – (81.03 * 20) – (37.35 * 15.21 * 12.5) – (142.08 * 15) – (142.08 * 10) - (142.08 * 5) - (39.22 * 5.59 * 2.5) – 13.54 = 0$

Ra = 551.99kN

Taking moment at H; 0

$(R_j * 20) – (81.03 * 20) – (39.22 * 5.59 * 17.5) (142.08 * 15) – (142.08 * 10) – (142.08 * 5) - (37.35 * 15.21 * 7.5) + 19.03 = 0$

Rj = 574.85kN

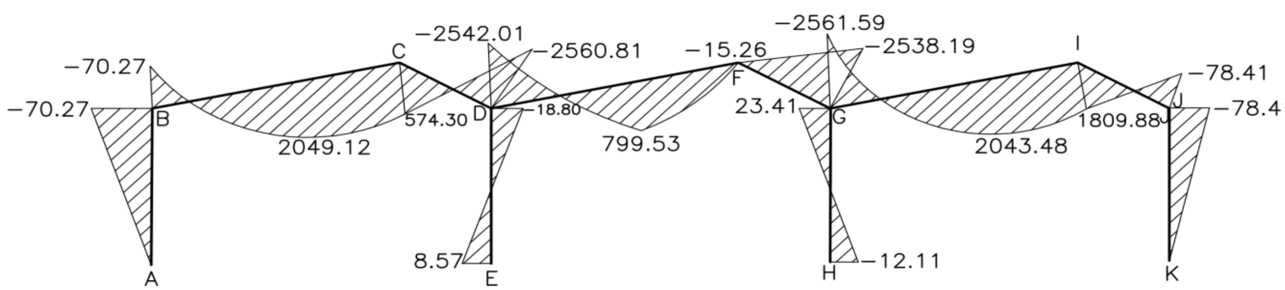


Figure 5.8: Bending moment (kNm) diagram of external frame due to vertical loadings

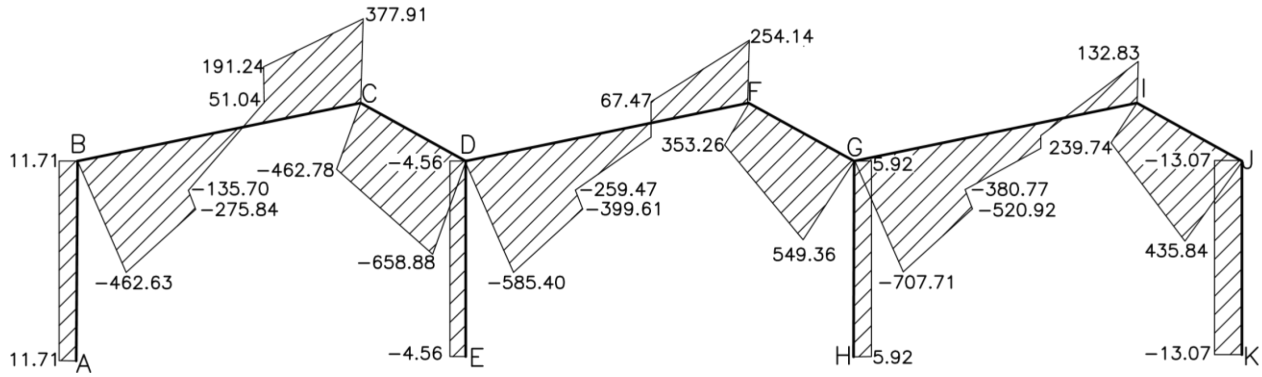


Figure 5.9: Shear force (kN) diagram of external frame due to vertical loadings

5.6 ANALYSIS DUE WIND LOADINGS

According to TS-498 wind force is determined by

$W = c_f * q$

Where $c_{f} =$ aerodynamic load factor

q = suction pressure

Table 5.3: Wind speed and suction according to height (TS-498(Turkish standard))

Height from ground (m)	Wind speed (m/s)	Suction q (kN/m^2)
0-8	28	0.5
9-20	36	0.8
21- 100	42	1.1
>100	46	1.3

Table 5.4: Coefficient factor according surface L/B ratio (TS-498(Turkish standard))

Surface	L/B	C_f	For use
Windward	All values	0.8	q
Leeward	0 -1	-0.5	q
	2	-0.3	q
	≥ 4	-0.2	q
Sidewall	All values	-0.7	q

Taking L = 6m (distance between two frames

5.6.1 Determination of Wind Load Acting from the Left Side to the Right Side

Calculation of wind load on member AB in the windward direction;

$q = 0.5\ kN/m^2$

$c_f = 0.8$

$w = 0.8 * 0.5 = 0.4 kN/m^2$

Calculation of wind load on member BC in the sidewall direction;

$q = 0.5\ kN/m^2$

$c_f = -0.7$

$w = -0.7 * 0.5 = -0.35 kN/m^2$

Calculation of wind load on member CD in the leeward direction;

$q = 0.5\ kN/m^2$

$c_f = -0.6$

$w = -0.6 * 0.5 = -0.3 kN/m^2$

Calculation of wind load on member DE in the leeward direction;

$q = 0.5\ kN/m^2$

$L/B = 20/6 = 3.33$

$c_f = -0.3$

$w = -0.3 * 0.5 = -0.15 kN/m^2$

Calculation of wind load on member DF in the leeward direction;

$q = 0.5\ kN/m^2$

$c_f = -0.7$

$w = -0.7 * 0.5 = -0.35 kN/m^2$

Calculation of wind load on member FG in the leeward direction;

q= 0.5 KN/m^2

c_f = -0.6

w = -0.6 * 0.5 = -0.3kN/m^2

Calculation of wind load on member GH in the leeward direction;

q= 0.5 KN/m^2

L/B = 40/6 = 6.67

c_f = -0.2

w = -0.2 * 0.5 = -0.1kN/m^2

Calculation of wind load on member GI in the leeward direction;

q= 0.5 KN/m^2

c_f = -0.7

w = -0.7 * 0.5 = -0.35kN/m^2

Calculation of wind load on member IJ in the leeward direction;

q= 0.5 KN/m^2

c_f = -0.7

w = -0.7 * 0.5 = -0.35kN/m^2

Calculation of wind load on member JK in the leeward direction;

q= 0.5 kN/m^2

L/B = 60/6 = 10

c_f = -0.2

w = -0.2 * 0.5 = -0.1kN/m^2

5.6.2 Analysis of Wind Load Acting from the Left Side to the Right Side

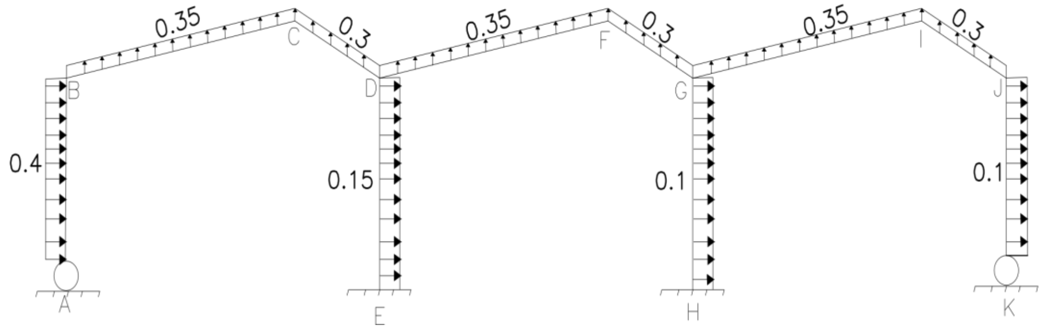


Figure 5.10: The frame subjected to wind loads acting from left side to right side

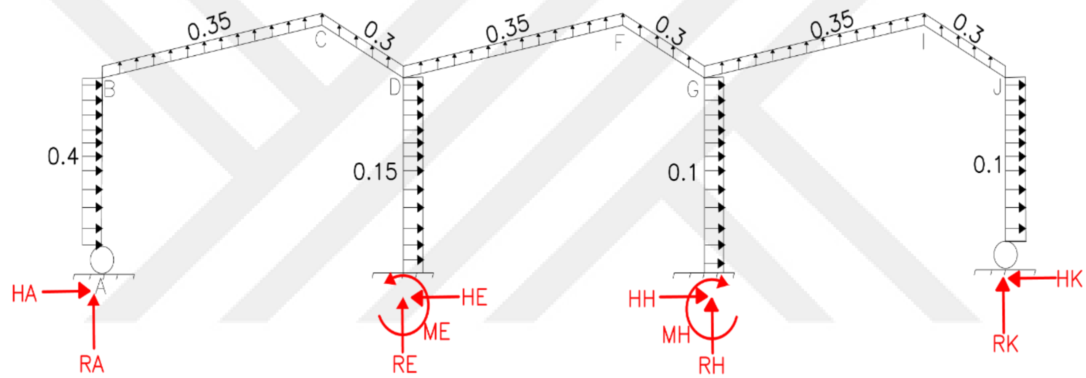


Figure 5.11: The frame subjected to wind loads acting from left side to right side with redundant forces

Number of members (m) = 10

Number of joints(j) = 11

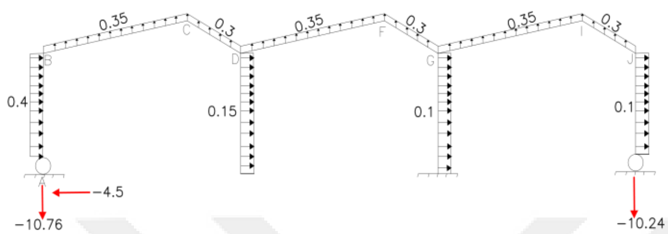
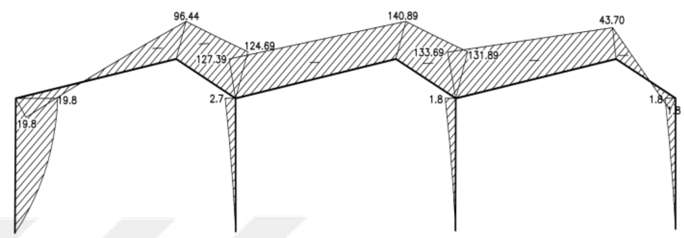
Number of reaction (r) = 10

Degree of indeterminacy = 3(m) + r -3(j)

Degree of indeterminacy = 3(10) + 10 -3(11) = 7

H_k, R_h, H_h, M_h, R_e, M_e, and H_e are taken as redundant forces

Table 5.5: Load cases and respective bending moment diagram

Load case and reaction	Bending moment diagram
	

$$\begin{bmatrix} -3.27 & -533.89 & 1.17 & -19.98 & 467.47 & -12.21 & 11.52 \\ -533.91 & 10402.19 & 1845.022 & 4281.74 & -200.5 & 1218.25 & -2185.35 \\ 1.25 & -1846.291 & 5123.5 & -14755.6 & -1540.32 & -4120.05 & 12495.86 \\ -20.38 & 4289.64 & -14757.03 & 74897.58 & 441.48 & 11846.833 & -35888.4 \\ 467.47 & -200.4 & -154.38 & 441.73 & -74.46 & -245.26 & 747.34 \\ -12.21 & 1218.98 & -4119.84 & 11844.96 & -245.31 & 5370.50 & -16401.54 \\ 11.52 & -2185.35 & 12495.86 & -35888.4 & 747.34 & -16401.54 & 82927.7 \end{bmatrix} * \begin{bmatrix} -0.02 \\ -0.002 \\ -0.003 \\ -9.64E-05 \\ -0.0032 \\ -0.002 \\ 3.97E-05 \end{bmatrix} = \begin{bmatrix} -0.32 \\ -7.28 \\ -1.12 \\ 2.55 \\ -7.63 \\ -1.18 \\ 2.54 \end{bmatrix} \begin{bmatrix} -0.32 \\ -7.28 \\ -1.12 \\ 2.55 \\ -7.63 \\ -1.18 \\ 2.54 \end{bmatrix} = \begin{bmatrix} R1 \\ R2 \\ R3 \\ R4 \\ R5 \\ R6 \\ R7 \end{bmatrix} = \begin{bmatrix} Hk \\ Rh \\ Hh \\ Mh \\ Re \\ He \\ Me \end{bmatrix}$$

Check:

$$\begin{bmatrix} 0.01956 \\ 0.00201 \\ 0.002877 \\ 9.64E-05 \\ 0.003167 \\ 0.001825 \\ -3.97E-05 \end{bmatrix} + \begin{bmatrix} 0.0007904 & 0.0001162 & 0.0004694 & 0.00000485 & 0.0022874 & 0.0004192 & -0.000003502 \\ 0.0001162 & 0.0001148 & 0.0000875 & 0.0000037458 & 0.0001326 & 0.0000236 & -0.000005082 \\ 0.0004694 & 0.0000875 & 0.0008384 & 0.0000922 & 0.00116 & 0.0004192 & -0.00000233 \\ 0.00000485 & 0.0000037458 & 0.0000922 & 0.00003099 & 0.00000545 & 0.000001173 & -0.0000002003 \\ 0.0022874 & 0.0001326 & 0.00116 & 0.00000545 & 0.0001687 & 0.0000471 & -0.000004149 \\ 0.0004192 & 0.0000236 & 0.0004192 & 0.000001173 & 0.0000471 & 0.0007882 & 0.00009337 \\ -0.000003502 & -0.000005082 & -0.00000233 & -0.0000002003 & -0.000004149 & 0.00009337 & 0.00000485 \end{bmatrix} \begin{bmatrix} -0.32 \\ -7.28 \\ -1.12 \\ 2.55 \\ -7.63 \\ -1.18 \\ 2.54 \end{bmatrix} = \begin{bmatrix} 0.00 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

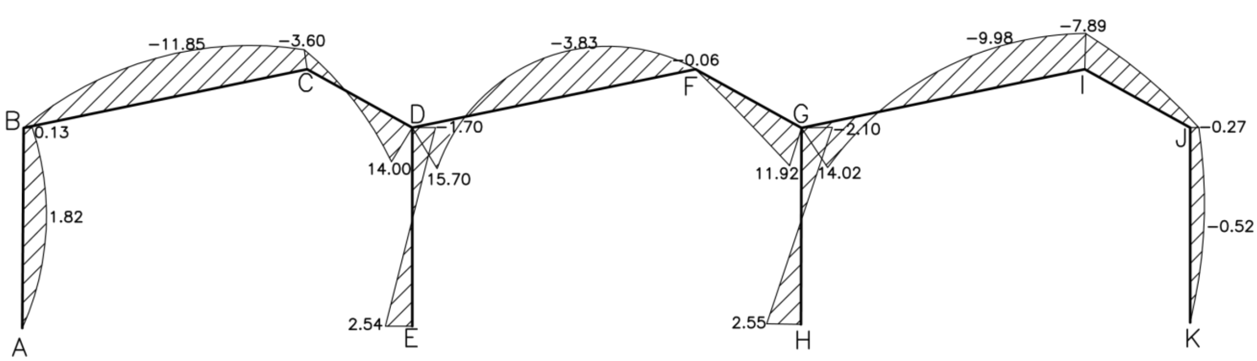


Figure 5.12: Bending moment (kNm) diagram due to wind loads acting from left side to right side

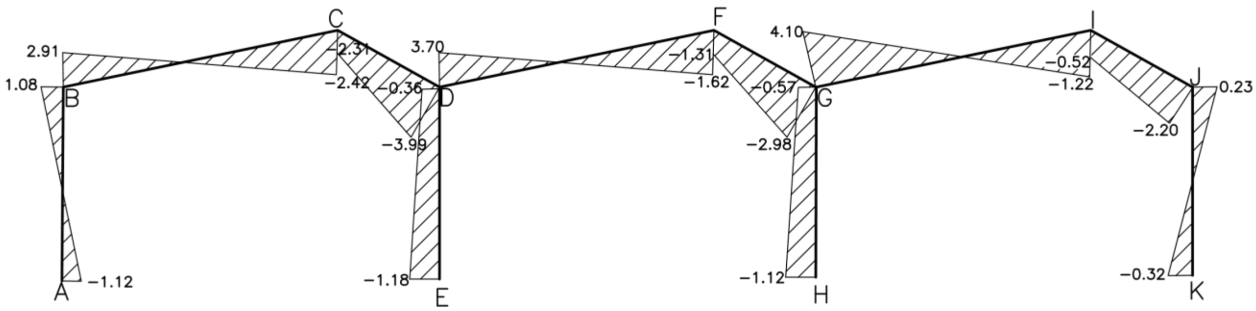


Figure 5.13: Shear force (kN) diagram due to wind loads acting from left side to right side

5.7 ANALYSIS OF WIND LOADS ACTING FROM RIGHT SIDE TO THE LEFT SIDE

5.7.1 Determination of Wind Load Acting from the Right Side to the Left Side

Calculation of wind load on member JK in the windward direction;

q= 0.5 kN/m^2

c_f = 0.8

w = 0.8 * 0.5 = 0.4kN/m^2

Calculation of wind load on member IJ in the sidewall direction;

q= 0.5 kN/m^2

c_f = -0.7

w = -0.7 * 0.5 = -0.35kN/m^2

Calculation of wind load on member GI in the leeward direction;

q= 0.5 kN/m^2

c_f = -0.6

w = -0.6 * 0.5 = -0.3kN/m^2

Calculation of wind load on member GH in the leeward direction;

q= 0.5kN/m^2

c_f = -0.3

$w = -0.3 * 0.5 = -0.15kN/m^2$

Calculation of wind load on member FG in the leeward direction;

$q= 0.5kN/m^2$

$c_f = -0.7$

$w = -0.7 * 0.5 = -0.35kN/m^2$

Calculation of wind load on member DF in the leeward direction;

$q= 0.5kN/m^2$

$c_f = -0.6$

$w = -0.6 * 0.5 = -0.3kN/m^2$

Calculation of wind load on member DE in the leeward direction;

$q= 0.5kN/m^2$

$c_f = -0.2$

$w = -0.2 * 0.5 = -0.1kN/m^2$

Calculation of wind load on member CD in the leeward direction;

$q= 0.5kN/m^2$

$c_f = -0.7$

$w = -0.7 * 0.5 = -0.35kN/m^2$

Calculation of wind load on member BC in the sidewall direction;

$q= 0.5kN/m^2$

$c_f = -0.6$

$w = -0.6 * 0.5 = -0.3kN/m^2$

Calculation of wind load on member AB in the windward direction;

$q= 0.5kN/m^2$

$c_f = -0.2$

$w = -0.2 * 0.5 = -0.1 kN/m^2$

5.7.2 Analysis of Wind Load Acting from the Right Side to the Left Side

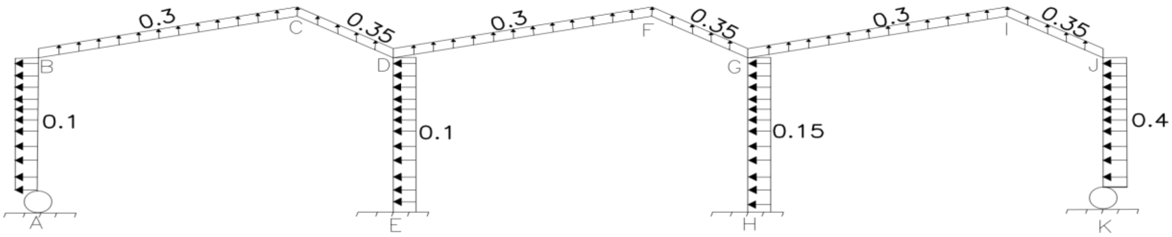


Figure 5.14: The frame subjected to wind loads acting from right side to left side

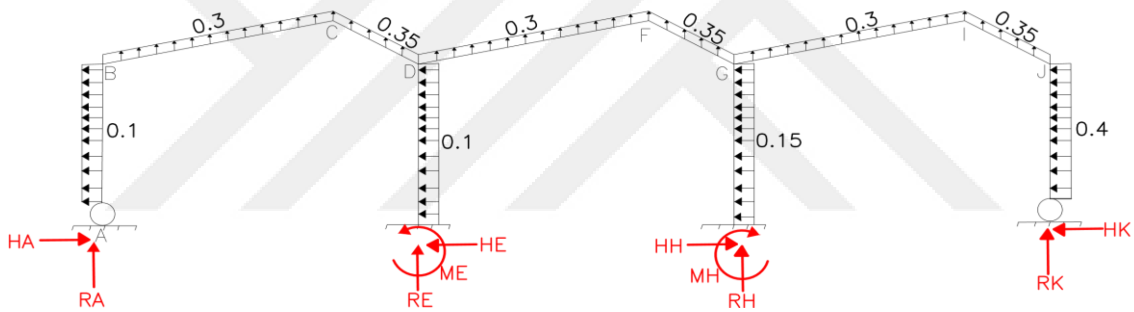


Figure 5.15: The frame subjected to wind loads acting from right side to left side with their redundant forces

Number of members (m) = 10

Number of joints(j) = 11

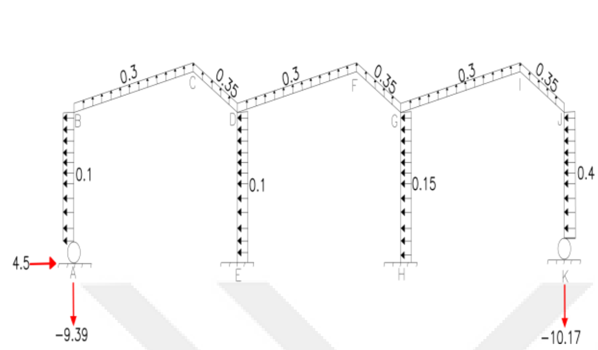
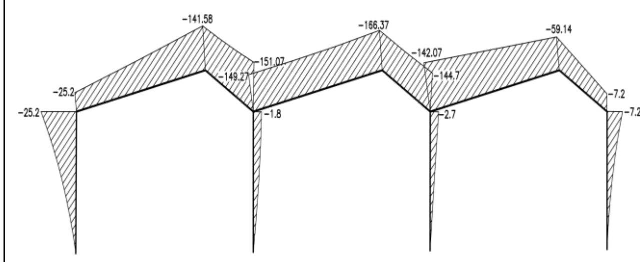
Number of reaction (r) = 10

Degree of indeterminacy = 3(m) + r -3(j)

Degree of indeterminacy = 3(10) + 10 -3(11) = 7

H_k, R_h, H_h, M_h, R_e, M_e, and H_e are taken as redundant forces

Table 5.6: Load cases and respective bending moment diagram

Load Case and Reaction	Bending moment
	

$$\begin{bmatrix} 0.0146 \\ 0.0015 \\ -0.0006 \\ 3.36\text{E}-05 \\ -0.001 \\ -0.0021 \\ -0.0003 \end{bmatrix} + \begin{bmatrix} 0.0007904 & 0.0001162 & 0.0004694 & 0.00000485 & 0.0022874 & 0.0004192 & -0.000003502 \\ 0.0001162 & 0.0001148 & 0.0000875 & 0.0000037458 & 0.0001326 & 0.0000236 & -0.000005082 \\ 0.0004694 & 0.0000875 & 0.0008384 & 0.0000922 & 0.00116 & 0.0004192 & -0.00000233 \\ 0.00000485 & 0.0000037458 & 0.0000922 & 0.00003099 & 0.00000545 & 0.000001173 & -0.0000002003 \\ 0.0022874 & 0.0001326 & 0.00116 & 0.00000545 & 0.0001687 & 0.0000471 & -0.000004149 \\ 0.0004192 & 0.0000236 & 0.0004192 & 0.000001173 & 0.0000471 & 0.0007882 & 0.00009337 \\ -0.000003502 & -0.000005082 & -0.00000233 & -0.0000002003 & -0.000004149 & 0.00009337 & 0.00000485 \end{bmatrix} \begin{bmatrix} R1 \\ R2 \\ R3 \\ R4 \\ R5 \\ R6 \\ R7 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

$$\begin{bmatrix} -3.27 & -533.89 & 1.17 & -19.98 & 467.47 & -12.21 & 11.52 \\ -533.91 & 10402.19 & 1845.022 & 4281.74 & -200.5 & 1218.25 & -2185.35 \\ 1.25 & -1846.291 & 5123.5 & -14755.6 & -1540.32 & -4120.05 & 12495.86 \\ -20.38 & 4289.64 & -14757.03 & 74897.58 & 441.48 & 11846.833 & -35888.4 \\ 467.47 & -200.4 & -154.38 & 441.73 & -74.46 & -245.26 & 747.34 \\ -12.21 & 1218.98 & -4119.84 & 11844.96 & -245.31 & 5370.50 & -16401.54 \\ 11.52 & -2185.35 & 12495.86 & -35888.4 & 747.34 & -16401.54 & 82927.7 \end{bmatrix} * \begin{bmatrix} -0.0146 \\ -0.0015 \\ 0.0006 \\ -3.36\text{E}-05 \\ 0.001 \\ 0.0021 \\ 0.0003 \end{bmatrix} = \begin{bmatrix} 1.29 \\ -7.39 \\ 1.44 \\ -3.47 \\ -6.98 \\ 1.38 \\ 3.50 \end{bmatrix} \begin{bmatrix} 1.29 \\ -7.39 \\ 1.44 \\ -3.47 \\ -6.98 \\ 1.38 \\ 3.50 \end{bmatrix} = \begin{bmatrix} R1 \\ R2 \\ R3 \\ R4 \\ R5 \\ R6 \\ R7 \end{bmatrix} = \begin{bmatrix} Hk \\ Rh \\ Hh \\ Mh \\ Re \\ He \\ Me \end{bmatrix}$$

Check:

$$\begin{bmatrix} 0.0146 \\ 0.0015 \\ -0.0006 \\ 3.36\text{E}-05 \\ -0.001 \\ -0.0021 \\ -0.0003 \end{bmatrix} + \begin{bmatrix} 0.0007904 & 0.0001162 & 0.0004694 & 0.00000485 & 0.0022874 & 0.0004192 & -0.000003502 \\ 0.0001162 & 0.0001148 & 0.0000875 & 0.0000037458 & 0.0001326 & 0.0000236 & -0.000005082 \\ 0.0004694 & 0.0000875 & 0.0008384 & 0.0000922 & 0.00116 & 0.0004192 & -0.00000233 \\ 0.00000485 & 0.0000037458 & 0.0000922 & 0.00003099 & 0.00000545 & 0.000001173 & -0.0000002003 \\ 0.0022874 & 0.0001326 & 0.00116 & 0.00000545 & 0.0001687 & 0.0000471 & -0.000004149 \\ 0.0004192 & 0.0000236 & 0.0004192 & 0.000001173 & 0.0000471 & 0.0007882 & 0.00009337 \\ -0.000003502 & -0.000005082 & -0.00000233 & -0.0000002003 & -0.000004149 & 0.00009337 & 0.00000485 \end{bmatrix} \begin{bmatrix} 1.29 \\ -7.39 \\ 1.44 \\ -3.47 \\ -6.98 \\ 1.38 \\ 3.50 \end{bmatrix} = \begin{bmatrix} -2.56E-05 \\ -3.29E-06 \\ -1.33E-05 \\ -3.98E-07 \\ -4.7E-05 \\ -1.04E-05 \\ -8.5E-08 \end{bmatrix}$$

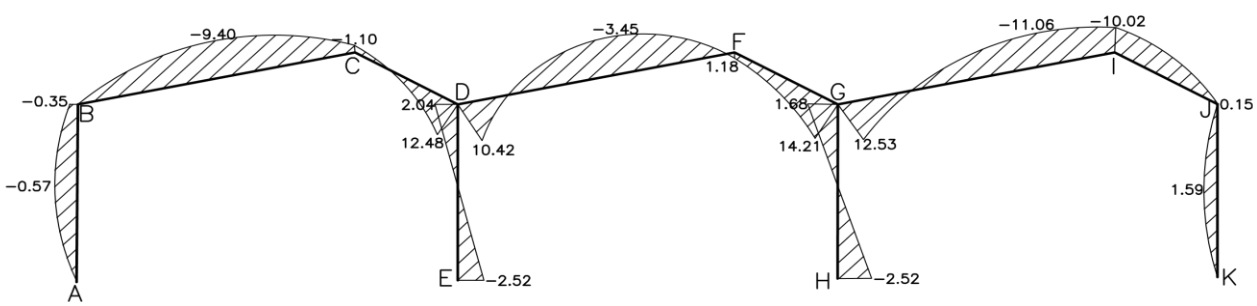


Figure 5.16: Bending moment (kNm) diagram due to wind loads acting from right side to the left side

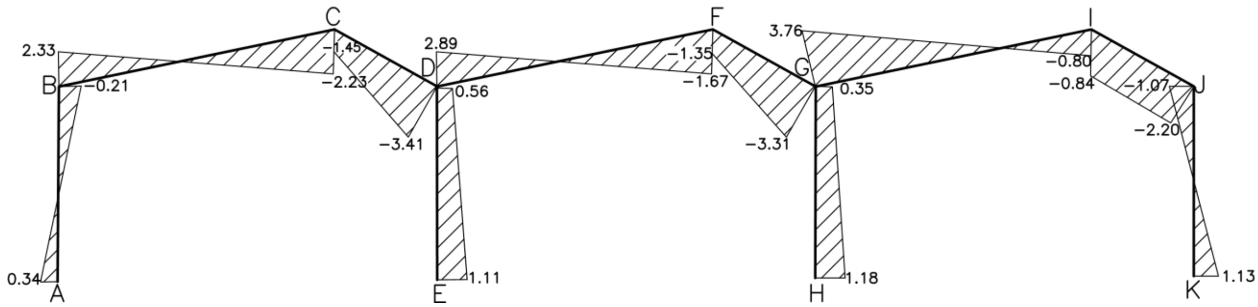


Figure 5.17: Shear force (kN) diagram due to wind loads acting from right side to the left side

The beam analysis was done by using Moment Distribution method (cross-method), and the frames were analyzed using Force Method (Flexibility matrix) and the matrix multiplication was done by using Microsoft (EXCEL) program.

5.8 ANALYSIS OF FRAMES USING ETABS

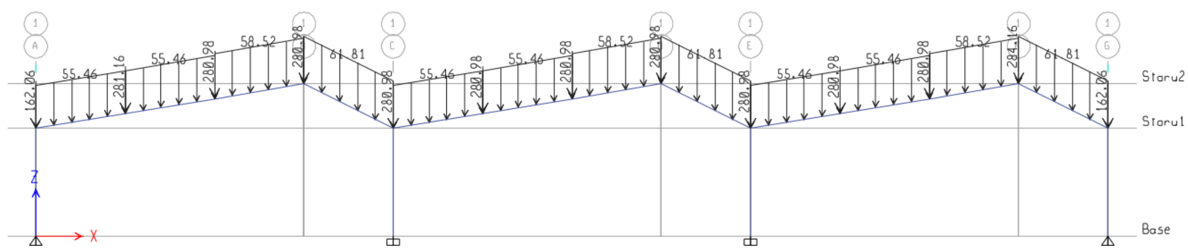


Figure 5.18: Internal frame subjected to vertical loadings using ETABS

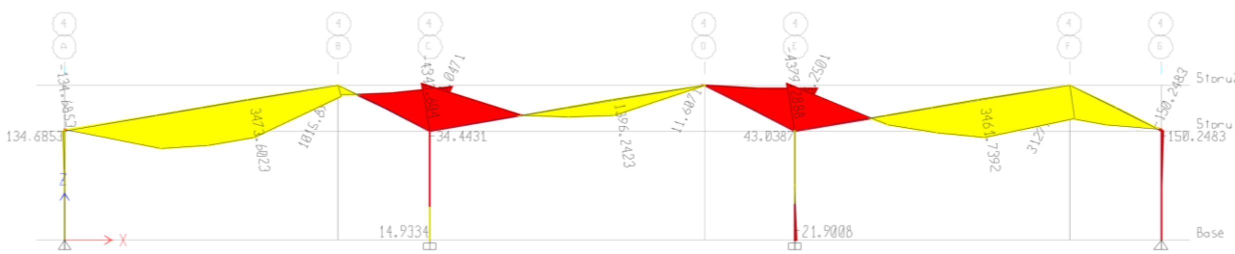


Figure 5.19: Internal frame bending moment (kNm)diagram due to vertical loadings using ETABS

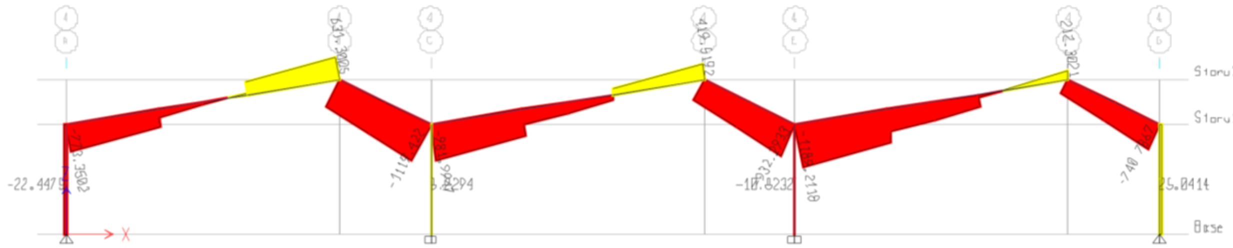


Figure 5.20: Internal frame shear force (kN) diagram due to vertical loadings using

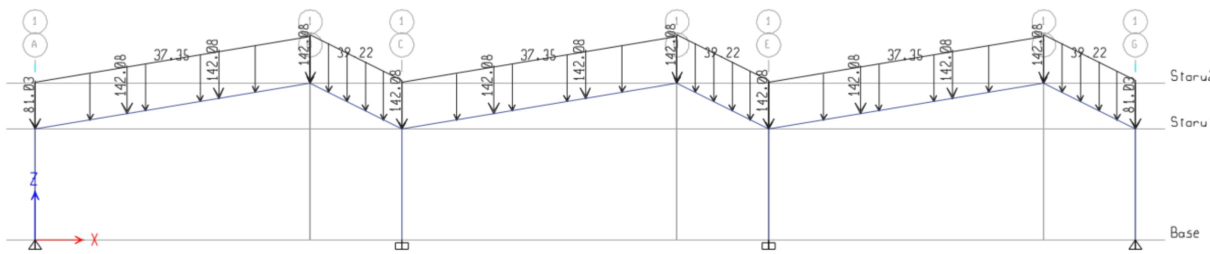


Figure 5.21: External frame subjected to vertical loadings using ETABS

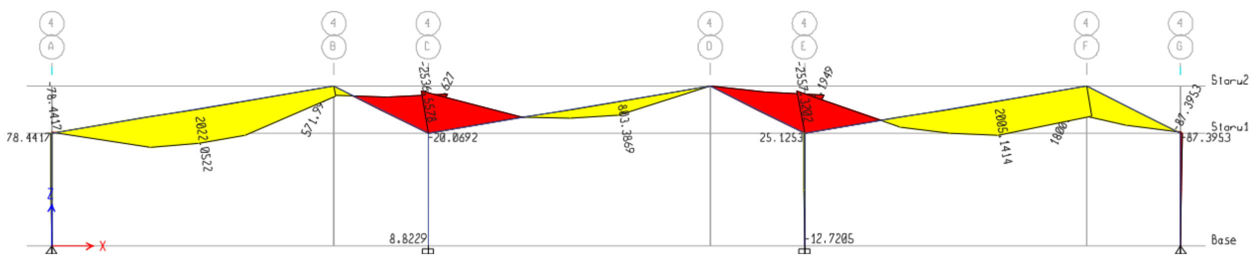


Figure 5.22: External frame bending moment (kNm) diagram due to vertical loadings using ETABS

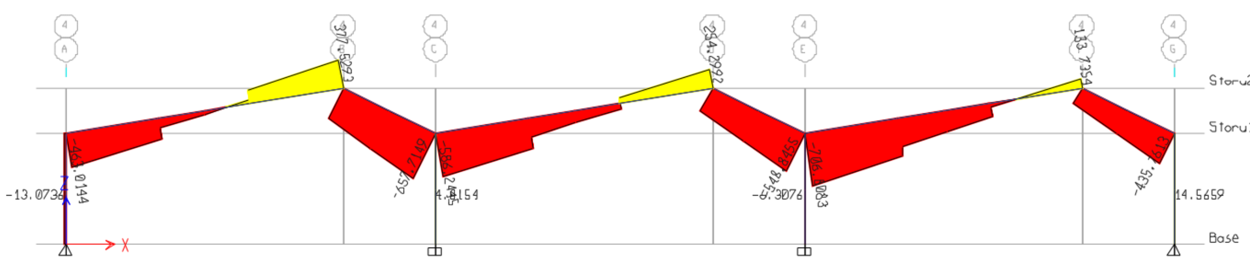


Figure 5.23: External frame shear force (kN) diagram due to vertical loadings using ETABS

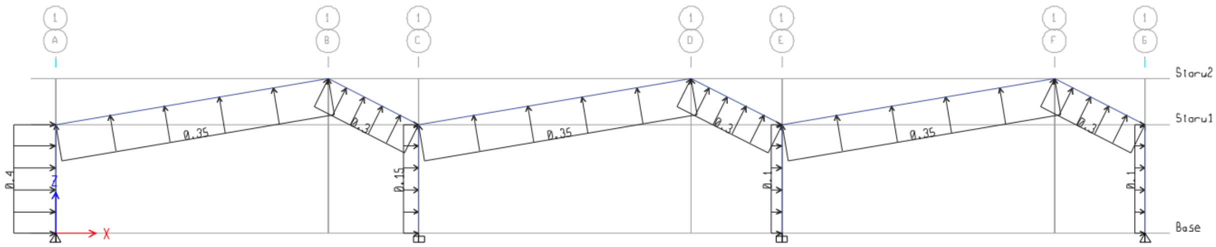


Figure 5.24: Frame subjected to wind loadings acting from left side to the right side using ETABS

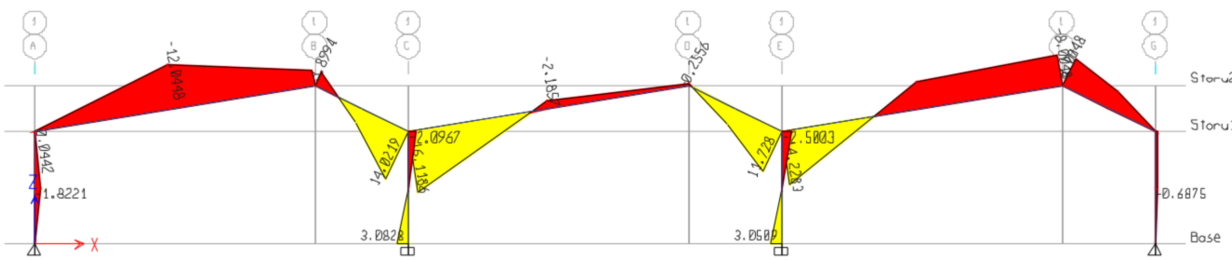


Figure 5.25: Frame bending moment (kNm) diagram due to wind loadings acting from left side to the right side using ETABS

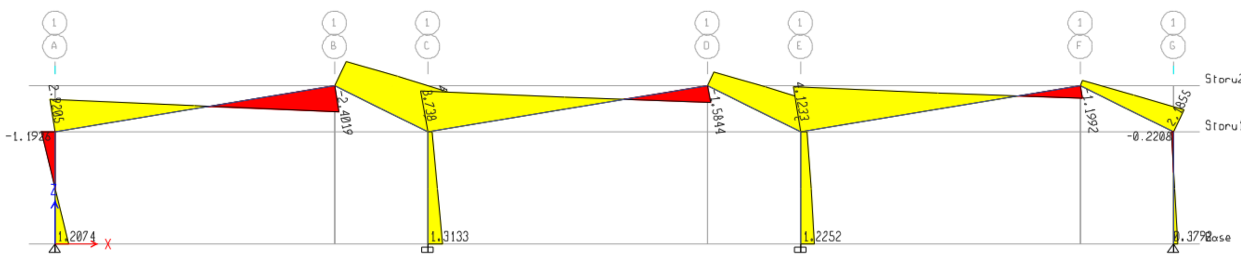


Figure 5.26: Frame shear force (kN) diagram due to wind loadings acting from left side to the right side using ETABS

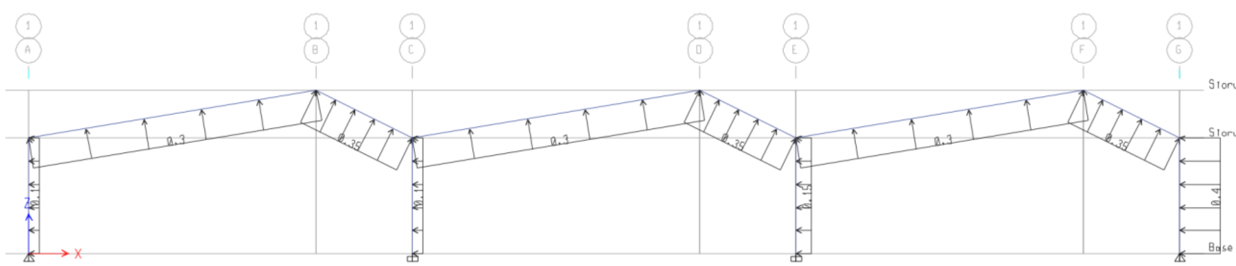


Figure 5.27: Frame subjected to wind loadings acting from right side to the left side using ETABS

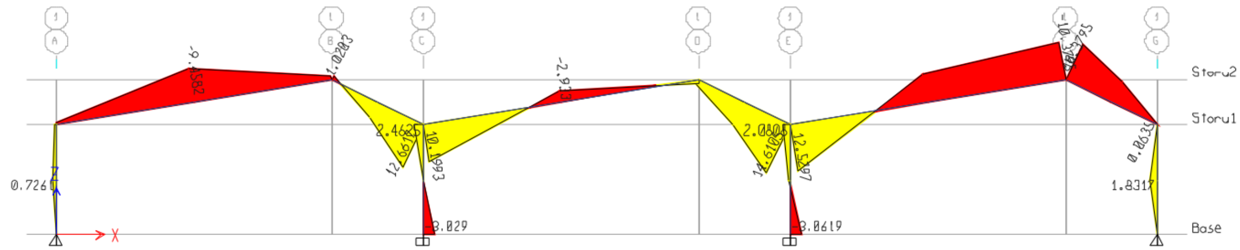


Figure 5.28: Frame bending moment (kNm) diagram due to wind loadings acting from right side to the left side using ETABS

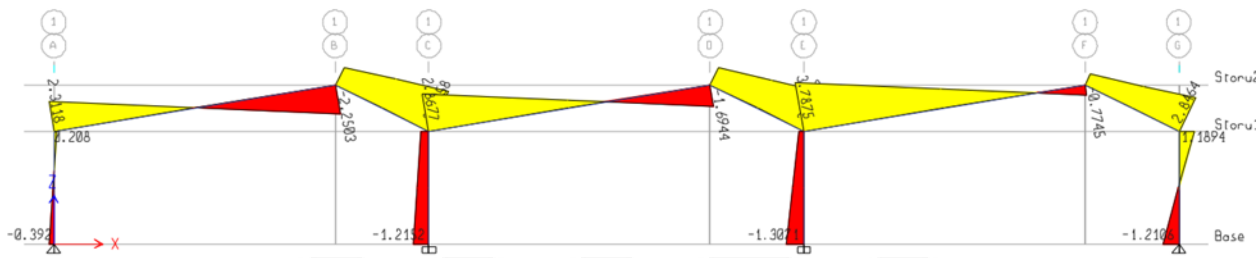


Figure 5.29: Frame shear force (kN) diagram due to wind loadings acting from right side to the left side using ETABS

Table 5.7: Comparison of joint Bending moment results under vertical loadings between Force Method results and ETABS results for internal frame.

Member	Joint	Force Method joint moments(kNm)	ETABS joint moments(kNm)
AB	A	0	0
BA	B	-147.92	-134.67
BC	B	-147.92	-134.67
CB	C	1000.58	1015.89
DC	D	-4377.18	-4378.04
DE	D	-38.13	-34.44
ED	E	16.93	14.93
DF	D	-4339.04	-4343.6
FG	F	-9	-11.61
GF	G	-4330.3	-4336.25
GH	H	47.97	43.04
HG	G	-24.91	-21.9
GI	I	-4378.27	-4379.29
IG	I	3101.42	3127.054
JI	J	-165.74	-150.25
JK	J	-165.74	-150.25
KJ	K	0	0

From the table above it was found out that the minimum percentage difference is -0.02% which is at joint I on member GI, a maximum percentage difference of 10% at joint B the joint moment of the frame has an average percentage difference of 4.3%.

Table 5.8: Comparison of span Bending moment results under vertical loadings between Force Method results and ETABS results for internal frame.

Member	Force Method mid-span Moment(kNm)	ETABS mid-span moment (kNm)
BC	3486.78	3450.93
DG	840.44	844.059
GI	2367.03	2367.47

Table 5.8 shows the mid span moment difference which has a minimum difference of -0.02% which is at member DG and a maximum of 1.04% at member BC. The spans have an averege perctega difference of 0.12%

Table 5.9: Comparison of shear force results under vertical loadings between Force Method results and ETABS results for internal frame.

Member	Joint	Force Method shear force(kN)	ETABS shear force(kN)
AB	A	26.89	22.45
BA	B	26.89	22.45
BC	B	-773.21	-773.35
CB	C	631.48	631.30
CD	C	-806.63	-810.32
DC	D	-1116.53	-1119.42
DE	D	-10.01	-8.23
ED	E	-10.01	-8.23
DF	D	-981.52	-981.99
FD	F	419.58	419.52
FG	F	-621.71	-623.24
GF	G	930.76	932.29
GH	G	13.25	10.82
HG	H	13.25	10.82
GI	G	-1187.5	-1189.21
IG	I	213.65	212.02
IJ	I	429.92	431.74
JI	J	738.93	740.79
JK	J	-30.13	-25.04
KJ	K	-30.13	-25.04

Shear Force differences are shown on table which illustrates the minimum Percentage difference is -0.25% at joint D on member DC with a maximum difference of 22.48% at joint G on member GH, the exerts an average percentage difference of 8.3%

Table 5.10: Comparison joint Bending moment results under vertical loadings between Force Method results and ETABS results for external frame

Member	Joint	Force Method joint moments(kNm)	ETABS joint moments(kNm)
AB	A	0	0
BA	B	-70.27	-78.44
BC	B	-70.27	-78.44
CB	C	574.3	571.99
DC	D	-2560.81	-2556.63
DE	D	-18.8	-20.06
ED	E	8.57	8.82
DF	D	-2542.01	-2536.56
FG	F	-15.26	-12.19
GF	G	-2538.19	-2532.19
GH	H	23.41	25.12
HG	G	-12.11	-12.72
GI	I	-2561.59	-2557.32
IG	I	1809.88	1800.46
JI	J	-78.41	-87.39
JK	J	-78.41	-87.39
KJ	K	0	0

From the table above it was found out that the minimum percentage difference is -10.4% which is at joint B on member BA ad BC, a maximum percentage difference of 25% at joint F the joint moment of the frame has an average percentage difference of -2.3%.

Table 5.11: Comparison of span Bending moment results under vertical loadings between Force Method results and ETABS results for external frame.

Member	Force Method mid-span Moment (kNm)	ETABS mid-span moment (kNm)
BC	2027.93	2021.74
DG	496.63	496.45
GI	1399.43	1401.88

Table shows the Minimum Percentage difference of -0.17% at member GI ad a maximum percntahe difference of 0.31% at member BC with average percetage difference of 0.05%

Table 5.12: Comparison of shear force results under vertical loadings between Force Method results and ETABS results for external frame.

Member	Joint	Shear force (kN)	ETABS (kN)
AB	A	11.71	13.07
BA	B	11.71	13.07
BC	B	-462.63	-463.01
CB	C	377.91	377.52
CD	C	-462.78	-461.61
DC	D	-658.88	-657.72
DE	D	-4.56	-4.1854
ED	E	-4.56	-4.1854
DF	D	-585.4	-586.24
FD	F	254.14	254.29
FG	F	353.26	352.74
GF	G	549.36	548.85
GH	G	5.92	6.31
HG	H	5.92	6.31
GI	G	-707.71	-706.81
IG	I	132.83	133.74
IJ	I	239.74	239.95
JI	J	435.84	435.76
JK	J	-13.07	-14.47
KJ	K	-13.07	-14.47

Table shows a minimum percentage shear force difference of -10.41% at member AB at joint A and B, the average percantage shear force difference -1.74% and the maximum percentage is 8.95% which is at member DE in joints D and E.

Table 5.13: Comparison joint Bending moment results between Force Method results and ETABS results for wind load acting on the frame from left to right

Member	Joint	Force Method joint moment (kNm)	ETABS joint moment (kNm)
AB	A	0	0
BA	B	0.13	0.04
BC	B	0.13	0.04
CB	C	-3.6	-3.89
DC	D	14	14.02
DE	D	-1.7	-2.09
ED	E	2.54	3.08
DF	D	15.7	16.11
FG	F	-0.06	-0.25
GF	G	11.92	11.72
GH	H	-2.1	-2.50
HG	G	2.55	3.05
GI	I	14.02	14.22
IG	I	-7.89	-8.00
JI	J	-0.27	-0.48
JK	J	-0.27	-0.48
KJ	K	0	0

Table 5.13 show the minimum percentage difference at joint D on member DC of -0.0014%, average percentage difference of 0.14% and the maximum difference occurs at B with percentage difference of 2.25%.

Table 5.14: Comparison joint Bending moment results between Force Method results and ETABS results for wind load acting on the frame from left to right

Member	Force Method mid-span Moment(kNm)	ETABS mid-span moment(kNm)
AB	-1.82	-1.82
BC	-11.85	-12.05
DE	0	0
DG	-2.3	-2.19
GH	0	0
GI	-7.05	-6.89

JK	-0.52	-0.68

Table 5.14 show the minimum percentage difference at member bc of -0.2%, average percentage difference of -0.11% and the maximum difference occurs at member GI and JK with percentage difference of -0.16%.

Table 5.15: Comparison joint Shear force results between Force Method results and ETABS results for wind load acting on the frame from left to right

Member	Joint	Shear force (kN)	ETABS shear force (kN)
AB	A	-1.12	-1.20
BA	B	1.08	1.19
BC	B	2.91	2.92
CB	C	-2.42	-2.40
CD	C	-2.31	-2.37
DC	D	-3.99	-4.04
DE	D	-0.36	-0.41
ED	E	-1.18	-1.31
DF	D	3.7	3.73
FD	F	-1.62	-1.58
FG	F	-1.31	-1.32
GF	G	-2.98	-2.98
GH	G	-0.57	-0.62
HG	H	-1.12	-1.22
GI	G	4.1	4.12
IG	I	-1.22	-1.19
IJ	I	-0.52	-0.51
JI	J	-2.2	-2.18
JK	J	0.23	0.22
KJ	K	-0.32	-0.38

Table 5.15 show the minimum shear force percentage difference of -15.7% at joint K on member KJ the farme has an average shear force percentage difference of -3.14% and a maximum shear force at 4.55% at joint J on member JK.

Table 5.16: Comparison joint Bending moment results between Force Method results and ETABS results for wind load acting on the frame from right to left

Member	Joint	Force Method joint moment(kNm)	ETABS joint moment (kNm)
AB	A	0	0
BA	B	-0.35	-0.55
BC	B	-0.35	-0.55
CB	C	-1.1	-1.02
DC	D	12.48	12.66
DE	D	2.04	2.46
ED	E	-2.52	-3.02
DF	D	10.42	10.19
FG	F	1.18	1.27
GF	G	14.21	14.61
GH	H	1.68	2.08
HG	G	-2.52	-3.06
GI	I	12.53	12.52
IG	I	-10.02	-10.38
JI	J	0.15	0.06
JK	J	0.15	0.06
KJ	K	0	0

The Minimum Percentage difference is shown in table 5.16 to be -36.36% which is at joint B on member BA and BC, the frame has an average difference of 10.15% and maximum pecentage difference of 150% which is at the face side where the wind acts/

Table 5.17: Comparison of mid-span Bending moment results between Force Method results and ETABS results for wind load acting on the frame from left to right

Member	Force Method mid-span Moment (kNm)	ETABS mid-span moment (kNm)
AB	0.57	0.72
BC	-9.4	-9.49
DE	0	0
DG	-2.87	-2.93
GH	0	0
GI	-7.42	-7.60

JK	1.60	1.83

Table 5.17 show the minimum percentage difference at member JK of -12.5%, average percentage difference of -6.2% and the maximum difference occurs at member JK with percentage difference of -12%.

Table 5.18: Comparison joint Shear force results between Force Method results and ETABS results for wind load acting on the frame from right to left

Member	Joint	Shear force(kN)	ETABS shear force (kN)
AB	A	0.34	0.39
BA	B	0.21	0.21
BC	B	2.33	2.31
CB	C	-2.23	-2.25
CD	C	-1.43	-1.46
DC	D	-3.41	-3.42
DE	D	0.56	0.61
ED	E	1.11	1.21
DF	D	2.89	2.86
FD	F	-1.67	-1.69
FG	F	-1.35	-1.43
GF	G	-3.3	-3.36
GH	G	0.35	0.40
HG	H	1.18	1.30
GI	G	3.76	3.78
IG	I	-0.8	-0.77
IJ	I	-0.84	-0.89
JI	J	-2.2	-2.84
JK	J	-1.07	-1.18
KJ	K	1.13	1.21

The Minimum Percentage difference is shown in table 5.18 to be -22.53% which is at joint J on member JI, the frame has an average difference of -5.01% and maximum percentage difference of 1503.89% which is at the face side where is wind acts.

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

6.1 CONCLUSION

Following the discussion and observation of the outcomes, some of the findings are summarized. Based on the Force method and ETABS program solutions behavior of RC frames have come to some significant findings:

a. It was also discovered that the results of the FORCE METHOD and those of the ETABS analysis differed somewhat, with the latter's results being more accurate.
b. The results of bending moment obtained under vertical loadings from ETABS are slightly greater.
c. The shear force results have a maximum difference of ±4kN under vertical loadings
d. In the analysis of the frames under wind load there was a considerable difference in bending moment at the face members (refer table 5.13 and 5.15)
e. Naturally, the analysis using the ETABS program will be much easier than the manual analysis using the Force method, even if Excel spreadsheets are also used.
f. It is better to analyze the problem using the ETABS approach since the FORCE METHOD requires calculating a number of simultaneous equations, making it time consuming to solve the problem using these methods.
g. In contrast to the FORCE METHOD of analysis or any other current approach, an approximate solution of any sort of frame, extremely close to the precise answer but without solving simultaneous equations, is sufficient.

6.2 RECOMMENDATION

This study shows the numerical differences between force method of analysis of indeterminate structures and the Extended Three-Dimensional Analysis Building System (ETABS) and recommend the following:

a. Before analysing any structure using computer programs be conversant with the formal methods of analysis so as to know when the program is wrong.
b. In the analysis and design phase, ETABS software is an important tool. Last-minute adjustments in load or design can be incorporated into current plans fast and precisely without causing many design faults, which gives it.
c. In the force method, one can easily add not only the effect of bending deformations, but also the shear force and deformation of normal forces to the equations. This work is not easy in displacement methods.

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