Analysis and Design of Multistory Building Using Different Structural Systems

By

Mohamad Radiham bin Mohamed Rahim

Dissertation submitted in partial fulfillment of
the requirements for the
Bachelor of Engineering (Hons)
(Civil Engineering)

DECEMBER 2006

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CERTIFICATION OF APPROVAL

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Approved by,

....
(Professor Waleed A. Thanoon)

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CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

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(MOHAMAD RADIHAM B. MOHAMED RAHIM)

ABSTRACT

Skeleton frame system and load bearing wall system are common structural system being used in high-rise and mid-rise building. These systems may have significant advantages and disadvantages against each other. The decision on using which system may require careful justification in order to build an economical, easy, and fast-to-construct building beside the stability and the durability of the building. In this study, a layout of a multistory building is to be designed using both systems. Both layout systems are then will be analyzed for the same live load as a control factor. The overall cost for the building is mainly depending on the quantity of materials used in the construction. Comparing the quantity of materials used to build the building, the load bearing wall system uses about 60% more concrete and steel reinforcement bars than the frame system. The load bearing wall system also found to require only minimum wall thickness with minimum reinforcement bars in the design. By reversing the calculation to determine the reinforcement for the walls, it is found that the minimum wall thickness with minimum reinforcement can stand the load of 1473.5 kN/ m length of wall while the maximum loads on the wall is only 577.36 kN/m length of wall (about 50% of the capacity). It is concluded that the frame system is suitable for any height and any floor arrangement while the load bearing wall system is more suitable for building with height more than 30 storey with similar floor layout in every floor..

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CHAPTER 1

INTRODUCTION

1.1 PROJECT BACKGROUND

In multi storey building construction, two different structural systems or forms are commonly used. The systems are either frame system or load bearing wall system. In frame system, the load from the upper floor is transmitted through beams and columns to the foundation, while in load bearing wall system, instead of beams and columns, the load from the upper floor is taken by load-bearing walls down to the foundation. The selection of the system to be used for construction of multi-storey building depends on many factors such as the scale of the building, availability of technology, and the condition of the site itself. Feasibility study on both systems will reveal the advantages and disadvantages of those systems, and provide some guidelines in deciding the suitable system for design and construction of mid-rise and high-rise building.

1.2 PROBLEM STATEMENT

Choosing one of the systems to be implemented in a multi storey building will bring some significant impact especially in terms of cost of the building. Theoretically, the frame system provides lighter weight and often more quick to erect compared to the load bearing wall system, in which the walls of the frame system are relieved from load-bearing function. But, in some circumstances, the load bearing wall system which fulfill both load-bearing and enclosing/dividing functions, proven to be cheaper in term of overall construction. Therefore, a feasibility study on both systems is required so that decision on using frame or load bearing wall system in any mid-rise and high rise building is justified.

1.3 OBJECTIVE

The objective of the project is to do a comparative/feasibility study on the use of structural frame (beams-columns) system and the load bearing wall system in mid-rise and high-rise building in term of material quantities and overall weights of the building.

1.4 SCOPE OF WORK

The scopes of work for this project are as the following:

- Analysis of normal vertical (dead load and live load) on the building
- Analysis of lateral load (wind load) on the building
- Design of the building using two different systems
- Compare the both systems in terms of material quantities and overall weights of the building.

CHAPTER 2

LITERATURE REVIEW

2.2 STRUCTURAL SYSTEM IN MULTISTORY BUILDING

In construction of multi storey high-rise building, the common system being used are skeleton framed system or simply framed system, and load bearing wall system.

2.2.1. Frame system

For a multi storey building constructed using frame system, the loads from the upper floor slabs are transferred to the beams of that particular level and then will be taken by columns down to the foundation.

Two types of materials commonly used for this system namely steel and reinforced concrete. For this particular comparison study, only reinforced concrete material is considered. In reinforced concrete construction, two types of construction methods namely cast-in-situ and precast or prefabricate method normally being used.

2.2.2. Load bearing wall system

In load bearing wall system, instead of beams and columns, the loads from the upper floor slabs are taken by the walls down to the foundation. The normal materials being used for this system are reinforced concrete, plain monolithic concrete, bricks, and concrete blocks. Compared to other materials, reinforced concrete walls proved to have the highest load bearing capacity for the same wall thickness. The low load bearing capacity and restriction of using thick wall made the other materials unlikely to be used for high rise building.

2.2.3. Reinforced concrete: frame system versus load bearing wall system

In frame system, the frame (beams and columns) will take all loads, relieving the wall from the load-bearing function. The enclosing function by walls can be made by any other forms more suited to the purpose than the heavy load-bearing walls. Therefore, frame system will provide lighter structure and often more quickly to erect. For this reason, many building use this type of system.

The walls in the load bearing wall system in the other hand, serve both load bearing and enclosing/dividing functions. In some circumstances, this system proved cheaper in terms of overall cost compared to frame structure. The statement may be true for small scaled and domestic buildings. For taller building with suitable plan form such as flat blocks, this type of system can again produce cheaper structure than the frame system.

Study by Stillman and Eastwick-Field proved that load bearing wall system provide much cheaper structure compared to frame system. But, the study was using cavity walls which were using masonry bricks and the structure studied was a two storey building which can be considered a low rise building. From the study, it emphasis that even though frame system proved to be cheaper in terms of materials, the other cost for constructing the building such as the cost for cladding and wall filling should be considered and accounted in overall cost of the building.

2.3 THE EFFECT OF WIND LOADING ON BUILDINGS

In tall buildings, the effect of wind is very significant compared to the low-rise building. One of the effects related to wind and tall buildings is the wind pressure. The wind blowing directly to a structure will be diverted in such way that causes unbalanced pressure around the structure. This unbalance pressure will exert some forces to the structure and cause extra loading.

Uniform Building Code (1994) provide method of estimating wind loading in that it accounts for the effect of gusting and for local extreme pressures over the faces of the building. The method accounts for local differences in exposure between the open countryside and a city center, as well as allowing for vital facilities such as hospitals and fire and police stations, whose safety must be ensured for use after an extreme windstorm.

The design wind pressure is given by the formula:

$$p = C_e C_q q_s I$$

where

C_e is a coefficient to account for the combined effects of height, exposure, and gusting, as defined in *Table 2.1.*

C_q is a coefficient that allows for locally higher pressures for wall and roof elements as compared with average overall pressures used in the design of the primary structure as defined in *Table 2.2.*

q_s is a wind stagnation pressure for minimum basic 50 yr wind speed at height of 30 ft above ground. Where local records indicates greater than basic value of the wind speed, this value should be used instead in determining qs. The value of the qs is obtained using the Bernoulli's equation:

$$q_s = \frac{1}{2} \rho v^2$$

where

ρ is the density of air taken as 0.0765 pcf

v is the velocity of the wind

I is taken as 1.15 for postdisaster buildings and 1.00 for all other buildings.

TABLE 2.11 Combined Height, Exposure and Gust Factor Coefficient C_e (UBC 1994)

Height above average level of adjoining ground (feet) ×304.8 for mm	Exposure D	Exposure C	Exposure B
0–15	1.39	1.06	0.62
20	1.45	1.13	0.67
25	1.50	1.19	0.72
30	1.54	1.23	0.76
40	1.62	1.31	0.84
60	1.73	1.43	0.95
80	1.81	1.53	1.04
100	1.88	1.61	1.13
120	1.93	1.67	1.20
160	2.02	1.79	1.31
200	2.10	1.87	1.42
300	2.23	2.05	1.63
400	2.34	2.19	1.80

Values for intermediate heights above 15 feet (4572 mm) may be interpolated.

Table 2.1: Combine Height, Exposure and Gust Coefficient C_e (UBC 1994)

TABLE 2.12 Pressure Coefficients C_q for Primary Frames and Systems (UBC 1994)

Description	C_q
Method 1 (Normal force method)	
Walls:	
Windward wall	0.8 inward
Leeward wall	0.5 outward
Roofs:	
Wind perpendicular to ridge	
Leeward roof or flat roof	0.7 outward
Windward roof	
less than 2:12 (16.7%)	0.7 owtward
Slope 2:12 (16.7%) to less than 9:12 (75%)	0.9 outward or 0.3 inward
Slope 9:12 (75%) to 12:12 (100%)	0.4 inward
Slope > (12:12 (100%)	0.7 inward
Wind parallel to ridge and flat roofs	0.7 outward
Method 2 (Projected area method)	1.3 horizontal any direction
On vertical projected area	
Structures 40 feet (12,192 mm) or less in height	1.4 horizontal any direction
Structures over 40 feet (12,192 mm) in height	
On horizontal projected area	0.7 upward

Table 2.2: Pressure Coefficient Cq for Primary Frames and Systems (UBC 1994)

In order to resist these forces, stiffeners may be required to be located at certain location of the building. Providing stiffener to resist these forces is called bracing.

2.3.1. Wind bracing for frame system

Providing stiffness or wind bracing for frame system can be achieve by many ways such as:

- The use of deep beams producing very stiff joints with the column
- The use of diagonal bracing in vertical panels of the frame
- Constructing solid walls called shear walls in suitable positions or using stair wells or lift shaft, if these are constructed in monolithic reinforced concrete, running the full height of the building so that they act as stiff, vertical members to which wind loading is transmitted by the floors.

2.3.2. Wind bracing for load bearing wall system

Since no beams are being used in the load bearing wall system, the use of deep beams to provide stiffness for the building is impossible. However, the wind bracing can best be achieved by having shear walls. Since the wall itself is the load bearing wall, designing the wall to be function as shear wall may require slight additional material or may be none at all.

2.4 STRUCTURAL ANALYSIS METHOD

2.4.1. Frame system

Structural analysis is done to determine the loading and its impact to the structural members so that the structural members can be designed accordingly. In frame system, the structural members that carry most of the load are beams and columns and these members are usually constructed continuously. This may result in indeterminate structure in which the reaction forces cannot be determined by simply applied the equilibrium static equation. Many methods are used to analyze the lateral load acted onto the building. These methods are divided into two categories:

- Exact method
- Approximate method

Exact methods

The exact method is the method used to determine the exact reaction forces in indeterminate structure. Some of the methods usually being used are:

Least-work theorems method

This method is developed by Alberto Castigliano which based on "least-work" theorems. His approach was based on analysis of the internal elastic energy stored in various parts of the structure under a loading. The internal work performed can be shown to be the least possible necessary to maintain equilibrium in supporting the loading.

Deflection method

In deflection method, sufficient supports are first conceptually removed to make the structure determinate before critical deflections are calculated. Then, the forces required to push the structure back to its original shape are calculated.

Moment distribution method

The method was introduced by Professor Hardy Cross. The method is based on successive cycles of computation that drew the results nearer to the exact value.

Finite element method

The method uses computer to do the lengthy calculation and analysis of complex structure based on the original approach described earlier. This method however, is quite complicated to bring understanding on the behavior of the indeterminate structure.

Approximate method

The approximate method is the method to approximately determine the reaction forces, shear forces and bending moment forces in indeterminate structure. The method involves sketching the deflected shape of the structure to determine the point of inflection. Number of unknown reactions can be reduced by taking the bending moment at the point of inflection equals to zero.

In analyzing the effect of wind loading of structural frame system in multistory building, the building's frame is assumed to be rigid. Two methods usually being used to analyze the rigid frame for lateral loading under the approximate method:

Portal method

The procedure to estimate the forces in structural member of laterally loaded multistory frames using portal method are based on the following assumptions:

- The shears in interior columns are twice as large as the shear in the exterior columns.
- A point of inflection occurs at the midheight of each column.
- The point of inflection occurs at the midpoint of each beams/girder.

By having all these assumptions, the shear in the beams and columns at the inflection points can be determined knowing that the bending moment at the inflection point equals to zero. The determined shear forces in the inflection point can be used to calculate the moment at beam and column joints since the shear at beams and columns are constant through.

Cantilever method

The procedure to estimate the forces in structural member of laterally loaded multistory frames using cantilever method are based on the following assumptions:

- The building frame act as a cantilever beam where the cross-section of the imaginary beam is composed of the cross-sectional areas of the columns. The horizontal stresses in the columns very linearly from the centroid of the cross section.
- A point of inflection occurs at the midheight of each column.
- The point of inflection occurs at the midpoint of each beams/girder.

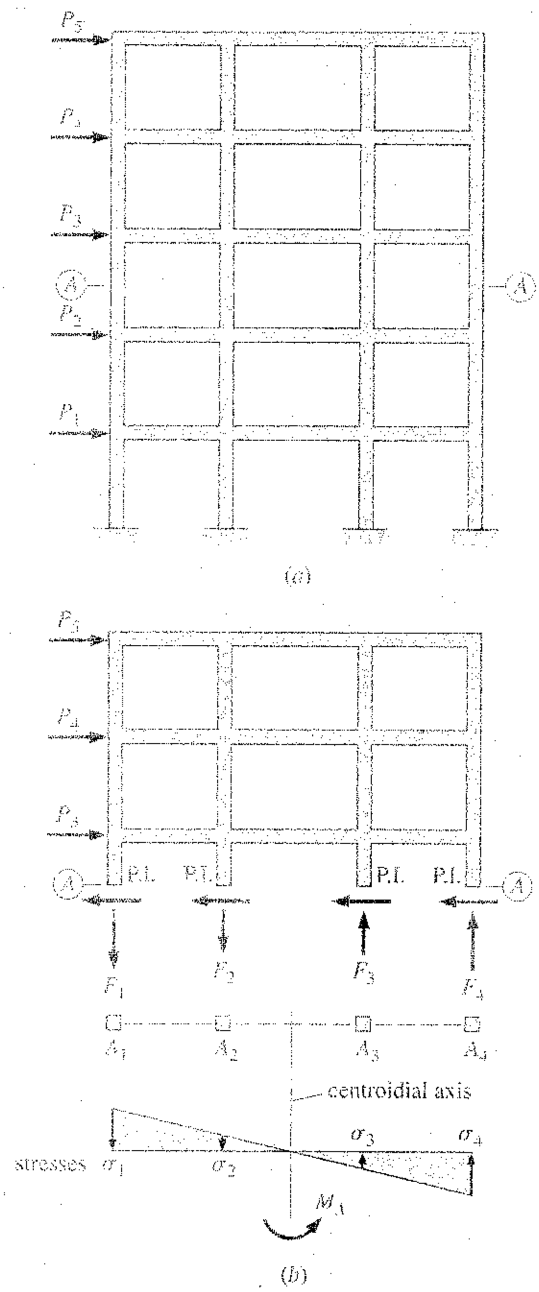


Figure 2.1 Assumption of column stress on column in cantilever method

Again, the assumptions made will simplify the indeterminate structure to determinate structure and the calculation is similar to the portal method.

2.4.2. Load bearing wall system

In load bearing wall system, the main structural member that carries the load is the structural walls. The walls usually can be analyzed using the static's equilibrium equation.

Lateral loading from wind is analyzed to determine the moment at the bottom of the building. The lateral loads from the wind needs to be distributed to the structural wall according to their relative stiffness. For symmetrical wall arrangement building, the distributed force Pi in each wall is then given by

$$Pi = F \times k_i/\Sigma k$$

where

F is the equivalent static force due to wind pressure

k is the relative stiffness of wall

The relative stiffness k, is given by the second moment of area of each wall about its major axis such that

$$k_i \approx h \times b^3$$

where

h is the thickness of the wall and b is the length of the wall.

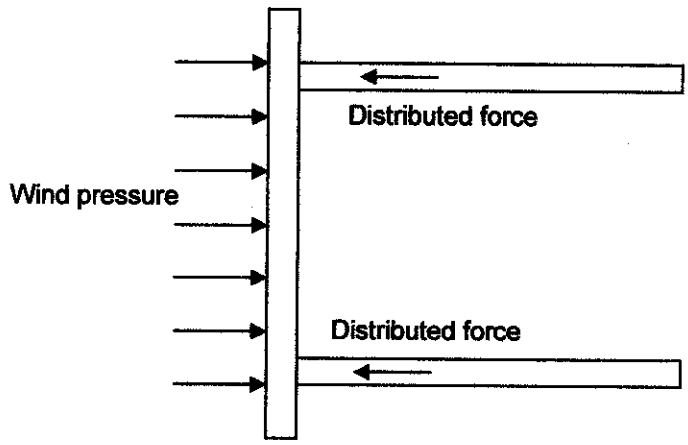
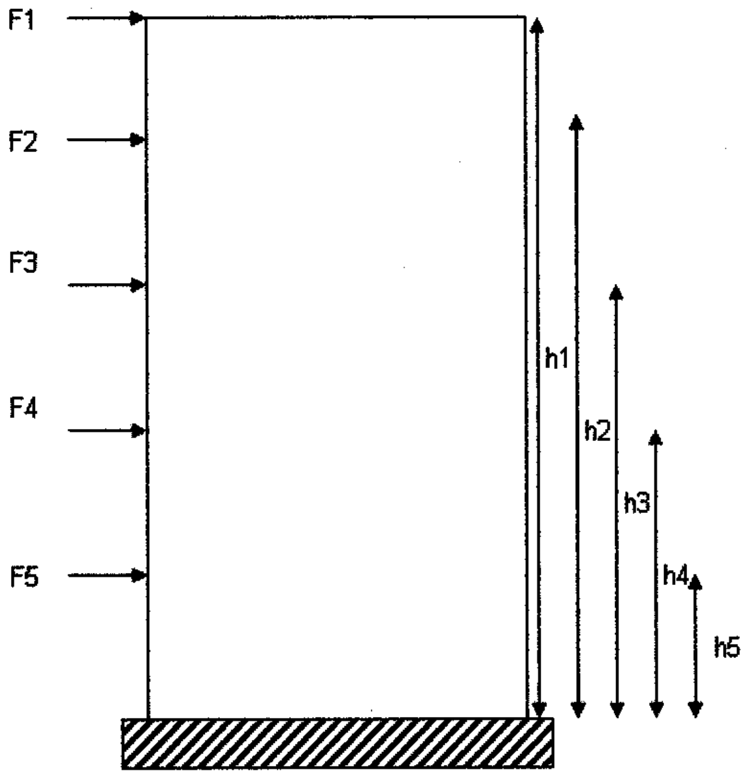


Figure 2.2 Distribution of the force due to wind pressure

The distributed force on the walls tends to bend the wall in the in-plane direction and developed moment at the bottom of the building. The moment developed at the bottom of the wall is then calculated by multiplying the equivalent static force by the distance of where the force is acted on to the bottom of the wall. The *Figure 2.2* explains the calculation to obtain the moment forces at the bottom of the wall.



$M = \Sigma(F_i \times h_i)$

Figure 2.3 Calculation of moment at the bottom of walls due to wind pressure

2.5 HORIZONTAL DEFLECTION

2.5.1. Drift for frame system

Drift is the horizontal deflection of the building due to wind loading. Drift in frame system is determined using the approximate analysis. In this analysis few assumptions are made:

- Point of counterflexure occurs at the mid storey of columns.
- Point of counterfluxure occurs at the mid span of beams/girders

Total drift of the building (top horizontal deflection)is obtained by summing all the storey drifts in the building. The storey drifts are obtained by calculating and summing the three drift components:

Storey drift due to beams/girders flexure. The formula to determine the drift component due to the beams/girders flexure is given by

$$\delta_{ig} = \quad Q_i h_i^2 / 12E\, \Sigma(I_g/L)i$$

where

δ_{ig} is the storey drift due to beams/girders flexure

Q_i is the horizontal forces at storey *i*

h_i is the height of the storey

I_g is the second moment of inertia of the beams

L is the length of the beam

Storey drift due to columns flexure. The formula to determine the drift component due to the column flexure is given by

$$\delta_{ic} = \quad Q_i h_i^2 / 12E\, \Sigma(I_c/h)i$$

where

δ_{ig} is the storey drift due to beams/girders flexure

Q_i is the horizontal forces at storey *i*

h_i is the height of the storey

I_c is the second moment of inertia of the columns

Storey drift due to overall bending. The formula to determine the drift component due to the column flexure is given by

$$\delta_{if} = h_i A_0^i$$

where

h_i is the height of the storey

A_0^i is the area between the base and the mid-height of storey i in the M/EI diagram.

Therefore, total storey drift for frame system is given by the formula

$$\delta_i = \delta_{ig} + \delta_{ic} + \delta_{if}$$

2.5.2. Deflection for load bearing wall system

The defection for the load bearing wall system can be calculated using the same method to calculate the deflection of cantilever beam (fixed at one end, free at another end) due to uniformly distributed load. The formula to calculated the deflection of the load bearing wall building is given by

$$\Delta = wH^4/8EI$$

Where

Δ is the horizontal deflection of the building

w is the uniformly distributed load due to wind

H is the total height of the building

E is the modulus of elasticity of concrete

I is the second moment of inertia of the walls.

CHAPTER 3

METHODOLOGY

3.1 SELECTION OF A MID-RISE BUILDING LAYOUT

A multi storey building layout are selected and designed using two different system; skeleton framed system and load bearing wall system.
The steps to be done for this feasibility study are as the following:

3.2 ANALYSIS OF VERTICAL LOAD

3.2.1. Frame system

The procedures to analyze the vertical load for frame system are as the following:

- The design load of the slabs is determined
- The portion of the slab weight is allocate to the corresponding beams
- The beams' reactions at the column due to the slab loading are determined. The reaction forces will become the axial load on the column.
- The beam is analyzed in term of bending moment and shear forces.

3.2.2. Load bearing wall system

The procedures to analyze the vertical load for load bearing wall system are as the following:

- The design load of the slabs is determined.
- The portion of the slab weight is allocated to the corresponding walls. This loading will be the wall axial load that will be used in designing the walls.

3.3 ANALYSIS OF LATERAL LOAD (WIND LOAD)

In analyzing the wind loading, an appropriate wind velocity profile should be developed. The wind velocity profile can be developed by knowing the average wind speed for every month through the year.

The wind statistic through out the year has been taken and wind velocity profile was developed using the equation given in the Uniform Building Code.

The direction of the wind that has severe impact on the building is determined before the analysis is done.

3.3.1. Frame system

The wind loading analysis for frame system is done using the portal or cantilever method described in the literature review in Chapter 2.

3.3.2. Load bearing wall system

The wind loading analysis for load bearing wall system is done using the relative stiffness force distribution method described in the literature review in Chapter 2

3.4 DETAILED DESIGN OF THE BUILDING USING BOTH FRAME AND LOAD BEARING WALL SYSTEMS

The building will be designed according to British Standard for both systems. The design is based on the vertical and horizontal load analysis.

3.4.1. Frame system

Beam

Moment and shear forces

Moment and shear forces acting on the beam is based on the analysis of the vertical load on the beam.

Preliminary sizing

The size of the beam should satisfy the following conditions:

- $M/bd^2f_{cu} \leq 0.156$ for singly reinforced beam
- $M/bd^2f_{cu} < 10/f_{cu}$ for doubly reinforced beam
- Shear stress, $v < 0.8\sqrt{f_{cu}}$
- Minimum cover for fire resistance

Design for bending

- Calculate $K = M/bd^2f_{cu}$
- Calculate lever arm, $z = d[0.5 + \sqrt{(0.25 - K/0.90}]$
- Calculate area of reinforcement required, $A_s = M/0.95f_yz$
- Check for minimum and maximum reinforcement

Design for shear

The reinforcement for shear, A_s/s_v can be determine by the expression:

$A_s/s_v \geq b(v - v_c)/0.95f_{yv}$

If $v < vc$, beams should be reinforced for nominal link provided that

$A_{sv}/s_v = 0.4b/0.95f_{yv}$

Column

Axial load

The axial load on the column is the sum of the reaction forces of the beams supported by the column. The reaction forces are obtained in the analysis of the beams.

Design for axial load

The area of reinforcement required can be calculated using the equation:

$$N = 0.4f_{cu}A_c + 0.80f_yA_{sc}$$

Links

The link provided must satisfy the following conditions:

- Maximum size = ¼ x size of the largest compression bar but not less than 6 mm.
- Maximum spacing = 12 x size of the smallest compression bar.

3.4.2. Load bearing wall system

Wall sizing

The thickness of the wall should be satisfying the minimum cover for fire resistance. The requirement can be referred to the BS 8110.

Axial load on wall

The axial loads are calculated according to the vertical load analysis discussed before.

Moment at the bottom of wall

The moment at the bottom of the wall is due to the wind loading as being discussed in the wind loading for load bearing wall system section previously.

Vertical reinforcement

Wall subjected to mainly axial load

The design axial load capacity for wall bearing mainly axial load is given by:

$$n_w = 0.35 f_{cu} A_c + 0.67 A_{sc} f_y$$

where

n_w is the axial load capacity per unit length of wall

f_{cu} is the strength of concrete in compression

f_y is the strength of reinforcement steel

A_c is the gross area of concrete in compression

A_{sc} is the gross area of steel reinforcement

Rearrange the equation so that the area of steel reinforcement can be obtained directly

$$A_{sc} = \frac{n_w - 0.35 f_{cu} A_c}{0.67 f_y}$$

Wall subjected to both axial and horizontal load (Design chart)

The reinforcement of the external wall is obtained using the design chart in the following figure. The strength of the concrete used is 30 N/mm^2 and the strength of the reinforcement steel is 460 N/mm^2. By knowing the axial load on the wall, the moment developed at the bottom of the wall, and the dimension of the wall, the area of reinforcement needed can be obtained by calculating the N/bh and M/bh2, and referring to the design chart to obtain the percentage of steel reinforcement required. From the percentage of steel reinforcement required, the actual reinforcement for the wall can be decided.

Minimum reinforcement

The steel reinforcement of the wall should not be less than 0.4% of the gross area of concrete. If the result from the design chart and calculation for axial loaded reinforcement is less than 0.4% of concrete's gross area, the wall should be provided by at least 0.4% steel reinforcement.

Horizontal reinforcement

The area of horizontal reinforcement in walls where the vertical reinforcement resists compression and does not exceed 2% is given in clause 3.12.7.4 as

$f_y = 250$ N/mm^2 0.3% of concrete area

$f_y = 460$ N/mm^2 0.25% of concrete area

3.5 ESTIMATION OF MATERIALS, COST, AND CONSTRUCTION TIME

Material estimation will be done based on the detailing of the structure done from the previous step. Labor forces and construction time will be estimated based on the estimation of materials being used.

3.6 ANALYSIS OF ADVANTAGES AND DISADVANTAGES OF BOTH SYSTEMS

The advantages and the disadvantages of both systems will be analyzed after all the design and estimation of materials are done.

CHAPTER 4

DISCUSSION

This chapter will discuss in details the results of analysis and design of concrete frame system and concrete load bearing wall system followed by the comparison between the two systems. The two systems will be compared in terms of difficulty and complexity of the analysis and design stages, materials used and cost, and other advantages and disadvantages of the systems against each other.

4.1 LAYOUT SELECTION/PREPARATION

The building layout used for this study is a modified 15 storey residential building (*Figure 4.1*). The layout is modified from New Harmony Block, a 60 storey low-rental residential building in Hong Kong. The building will consist of four main wings namely Wing A, Wing B, Wing C, and Wing D. The layout of Wing A is identically the same as Wing C, while Wing B is identically same as Wing D. Each wing has four domestic housing units in every floor. Six elevators are located at the center of the building.

4.1.1 Structural Frame System

For the structural frame system, each floor slabs of every wing will be supported by beams ranging from 2.34 m to 6.29 m span. The loads from these beams are then to be taken by columns down to the foundation. For the core structure, the load from the slabs will be taken by 7.49 m and 6.29 m beams and the load bearing wall of the elevators. The location of all beams and columns for each repeated-layout floor is shown in the *Figure 4.2.*

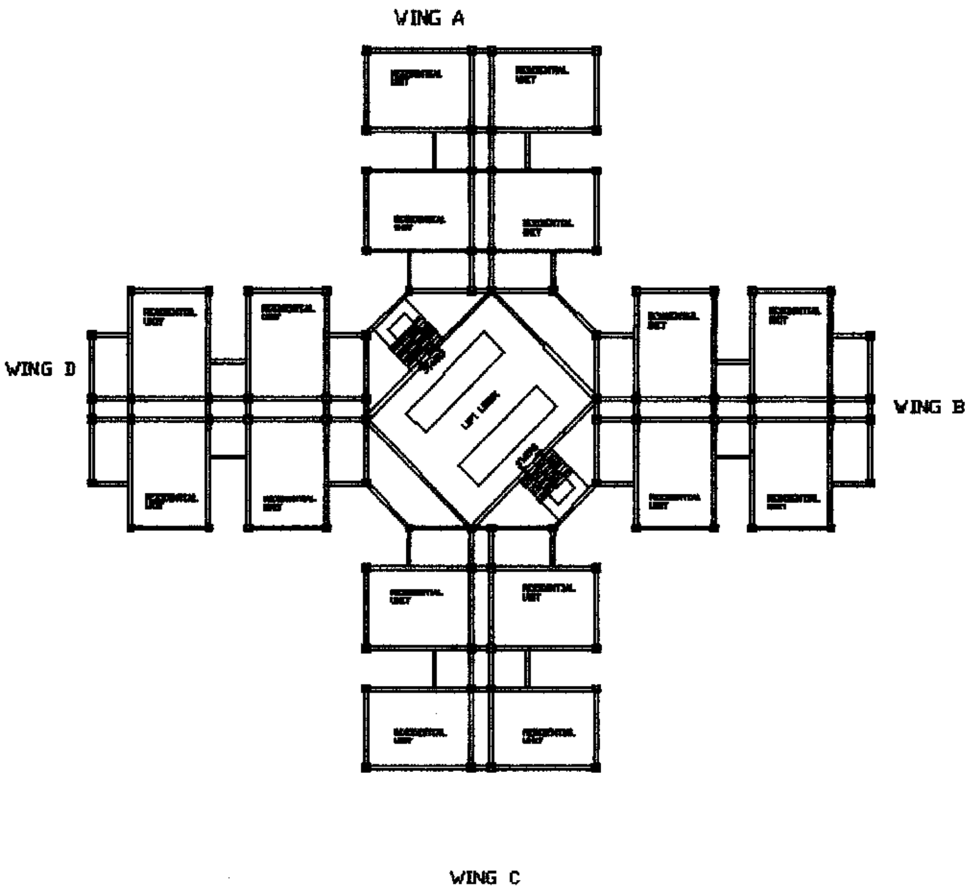


Figure 4.2 Location of beams and column in frame system

4.1.2 Load Bearing Wall System

For the load bearing wall system, each floor slabs of every wing will be supported by the load bearing walls. The arrangement of wall used for the building is cellular wall arrangement. In cellular wall arrangement, both internal and eternal wall are load bearing walls. For the core structure, the load from the slab will be taken by the elevators' walls and the stairs' walls. The location of the load bearing walls is shown in the *Figure 4.3*.

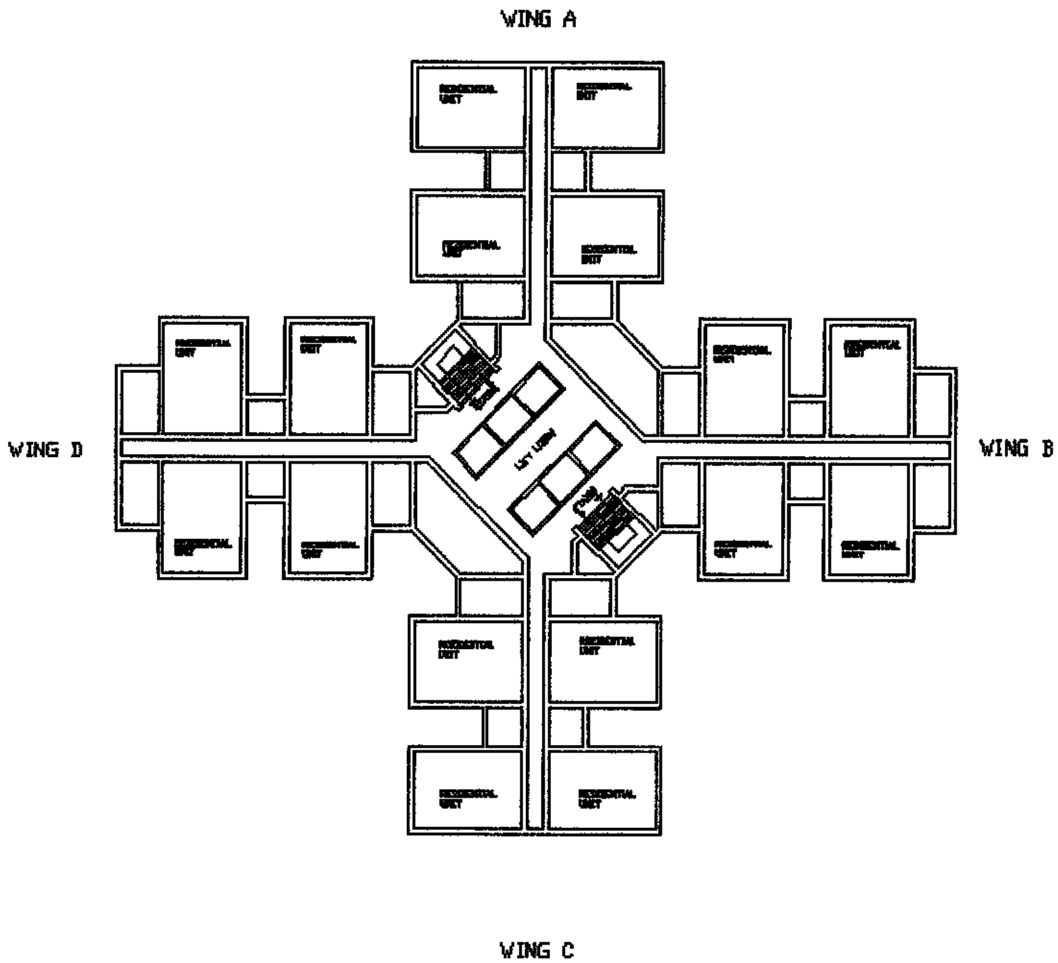


Figure 4.3 Location of load bearing walls in load bearing wall system

4.2 VERTICAL LOADING ANALYSIS

For vertical loading analysis, few assumptions and preliminary assumptions are made to determine the design load of the building. Since the layout plan for both load bearing wall system and frame system are almost the same, the load distribution from the floor to the load-bearing wall is similar to the load distribution from the floor to the beam. The following paragraph will discuss the vertical load distribution of both systems including the assumptions made.

4.2.1 Frame system

The vertical load distribution analysis for the frame system is done based on the following few assumptions and preliminary assumptions:

Slab dead load. The preliminary assumption made to determine this load is the slab thickness. The slab thickness for the building is assumed to be 200 mm thick. Therefore, the slab self weight will be:

$$\text{Slab dead load} = 24\text{kN/m}^3 \times 0.2\text{ m} = 4.8\text{ kN/m}^2$$

Slab live load. For this vertical load analysis, the live load of 3.0 kN/m^2 is used.

Slab design load. The design load for the slab is obtained by combining the slab's dead load and live load as suggested in the BS 8110. The design load for the slab is expressed as:

$$\begin{aligned}\text{Slab design load} &= 1.4(\text{dead load}) + 1.6(\text{live load}) \\ &= 1.4(4.8) + 1.6(3.0) \\ &= 11.52\text{ kN/m}^2\end{aligned}$$

Wall self weight. The preliminary assumptions made to determine the wall self weight are height and thickness of the wall. Since the walls are for dividing and enclosing purposes, brick walls are used. The height of the wall is assumed to be 3.0 m high and the thickness of the wall is assumed to be 150 mm thick. Therefore, the self weight of the wall is:

Wall self weight = 22 kN/cu.m x 3.0 m x 0.15 m

= 9.9 kN/m width

Beam self weight. The assumption required to determine the self weight of the beam is the beam size. The preliminary sizing for the beam is 250 mm x 400 mm. Therefore, beam self weight is:

Beam self weight = 24 kN/cu.m x 0.25 m x 0.4 m

= 2.4 kN/m

The vertical loading analysis for the frame system is done through the following steps:

Slab division. Every slab on the building is divided into several triangles and trapeziums shape as shown in the *Figure 4.4*. These triangles and trapeziums represent the portion of slab's weight need to be carried by the beams. The weight of the slab need to be carried by the beams can be expressed by the following equation:

$$\text{Loading by slab} = \frac{\text{area of triangle/trapezium x } 11.52 \text{ kN/m}^2}{\text{length of wall}}$$

Total load on beams. The beams should be able to carry the loads from the slabs and the wall above together with the self weight of the beams itself. Therefore, the total load on the beams can be expressed by the following equation:

Load on beam = loading by slabs + wall self weight + beam self weight

= loading by slabs + 9.9 kN/m + 2.4 kN/m

= loading by slabs + 12.3 kN/m

Load to column. The load to the column can be determined by calculating the reaction forces of the beams due to the loading on the beams. The load on the column is increased as the floor level is decreased. The lower level column needs to carry the load from the upper column as well as the load from that particular floor.

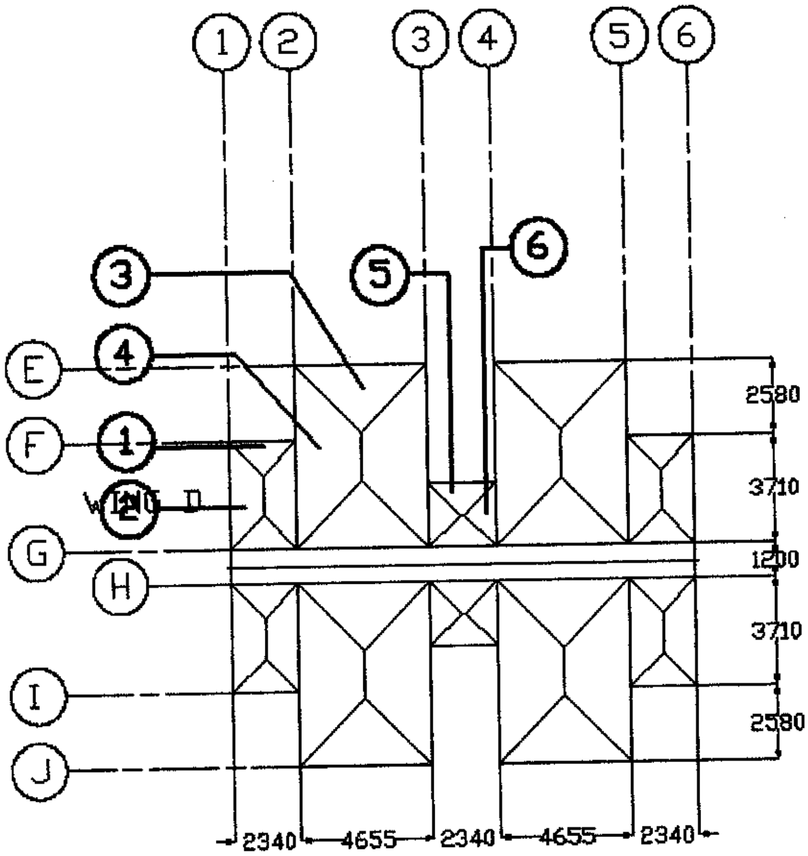


Figure 4.4 Slab division for frame and load bearing wall system

Analysis of beams

The beams in the building can be classified into two categories:

- Simply supported
- Continuous

Simply supported beams. The simply supported beams were easily analyzed by applying the basic static equilibrium equations since they are statically determinate structures. In simply supported beams, the maximum bending moment occurred at the mid span of the beams while the maximum shear occurred at the supports.

Continuous beams. The continuous beams in the other hand are quite difficult to analyze since they are considered statically indeterminate structures. In this project, the method used to analyze the beam is the approximate method. Detailed calculation and analysis are shown in the Appendix B1. From the analysis, it is found that the maximum bending moment occurred at both mid span and supports while the maximum shear occurred at the supports and point loads.

Analysis of column

The axial loads on columns are the support reactions of the beams. The axial load on columns is linearly increasing as the floor level decreasing. The analysis is using full load instead of allowable reduced load as specified in the British Standard as a matter of study and comparing the result to the load bearing wall system. From the analysis, the interior columns are found to have greater axial loads compared to the exterior columns due to the greater portion of slabs area the interior columns have to support.

4.2.2 Load bearing wall system

The vertical load distribution analysis for the load-bearing wall system is done based on the following few assumptions and preliminary assumptions:

Slab dead load. The preliminary assumption made to determine this load is the slab thickness. The slab thickness for the building is assumed to be 200 mm thick. Therefore, the slab self weight will be:

Slab dead load $= 24 kN/m^3 \times 0.2$ m

$= 4.8 kN/m^2$

Slab live load. For this vertical load analysis, the live load of $3.0 kN/m^2$ is used.

Slab design load. The design load for the slab is obtained by combining the slab's dead load and live load as suggested in the BS 8110. The design load for the slab is expressed as:

Slab design load = 1.4(dead load) + 1.6(live load)

= 1.4(4.8) + 1.6(3.0)

$= 11.52 kN/m^2$

Wall self weight. The preliminary assumptions made to determine the wall self weight are height and thickness of the wall. The height of the wall is assumed to be 3.0 m high and the thickness of the wall is assumed to be 160 mm thick. Therefore, the self weight of the wall is:

Wall self weight $= 24 kN/m^3 \times 3.0 m \times 0.16$ m

= 11.52 kN/m width

The vertical loading analysis for the load bearing wall system is done through the following steps:

Slab division. Every slab on the building is divided into several triangles and trapeziums shape as shown in the *Figure 4.4.* These triangles and trapeziums represent the portion of slab's weight need to be carried by the walls. The weight of the slab need to be carried by the walls can be expressed by the following equation:

$$\text{Loading by slab} = \frac{\text{area of triangle/trapezium x 11.52 kN/sq.m}}{\text{length of wall}}$$

Total load on wall. The total load to be carried by the load-bearing wall will be the loads from the adjacent slabs, the wall self weight, and the weight carried by the upper load-bearing walls. The total load on the wall is calculated using the following expression:

Total load on wall = loading by slabs + wall self weight + loads carried by the upper wall

The calculations and summary of the vertical loading analysis for the load bearing wall system are shown in the Appendix B2.

4.3 WIND LOADING

One of the horizontal loads which should be taken into account in designing the load bearing wall is the wind loading. When a structure such as wall is in the path of wind, the wind will be deflected in a way which will cause differential pressure distribution or suction on the structure. This force will cause the undesirable bending to the wall and the wall may fail not because of inadequate resistance of the vertical loading but because of the extreme deflection or bending of the wall.

In analyzing the wind load on the building, a velocity profile and dynamic pressure profile is developed using the equation provided in the methodology in Chapter 3. The wind velocity profile and wind pressure profile is shown in the *Table 4.1*.

Level	Height above ground (m)	Height above ground (ft)	qs	Ce		Cq		Windward pressure (psf)	Leeward action (psf)	Design pressure psf 5+6	Design pressure lbf/in2	Design pressure kPa
				Windward	Leeward	Windward	Leeward	5	6			
15	45.0	147.6	12.54400	1.27	1.59	0.8	-0.5	12.745	9.972	22.717	0.158	1.088
14	42.0	137.8	12.54400	1.25	1.59	0.8	-0.5	12.544	9.972	22.516	0.156	1.078
13	39.0	127.9	12.54400	1.22	1.59	0.8	-0.5	12.243	9.972	22.215	0.154	1.064
12	36.0	118.1	12.54400	1.19	1.59	0.8	-0.5	11.942	9.972	21.914	0.152	1.049
11	33.0	108.2	12.54400	1.16	1.59	0.8	-0.5	11.641	9.972	21.613	0.150	1.035
10	30.0	98.4	12.54400	1.12	1.59	0.8	-0.5	11.239	9.972	21.212	0.147	1.016
9	27.0	88.6	12.54400	1.08	1.59	0.8	-0.5	10.838	9.972	20.810	0.145	0.996
8	24.0	78.7	12.54400	1.04	1.59	0.8	-0.5	10.437	9.972	20.409	0.142	0.977
7	21.0	68.9	12.54400	0.99	1.59	0.8	-0.5	9.935	9.972	19.907	0.138	0.953
6	18.0	59.0	12.54400	0.95	1.59	0.8	-0.5	9.533	9.972	19.506	0.135	0.934
5	15.0	49.2	12.54400	0.89	1.59	0.8	-0.5	8.931	9.972	18.904	0.131	0.905
4	12.0	39.4	12.54400	0.84	1.59	0.8	-0.5	8.430	9.972	18.402	0.128	0.881
3	9.0	29.5	12.54400	0.76	1.59	0.8	-0.5	7.627	9.972	17.599	0.122	0.843
2	6.0	19.7	12.54400	0.67	1.59	0.8	-0.5	6.724	9.972	16.696	0.116	0.799
1	3.0	9.8	12.54400	0.62	1.59	0.8	-0.5	6.222	9.972	16.194	0.112	0.775

Table 4.1 Wind velocity profile

4.3.1 Frame system

The analysis of lateral loading is done using cantilever method. Detailed calculation of the analysis is shown in the Appendix C1. Bending moments developed at the beams' and columns' connections are quite significant especially at the bottom level of the building and these bending moments have to be considered in the design.

From the analysis, it is found that the wind loading also increase the axial load of the column. The increment of the loads will depend on the location of the column. The more exterior the column is, the greater the increment will be.

Maximum deflection of the building for frame system is calculated and the detailed calculation is shown in the Appendix C1. Maximum deflection for the frame systems is 4.49 mm. The deflection is within the drift index which is 1/500 or 90 mm.

4.3.2 Load bearing wall system

Analysis of the wind loading for the building is made by considering the wind blows in four different directions as shown in the *Figure 4.5*. The effect of wind for direction A and C are the same since the building is symmetrical and so the direction B and D. The inclined wind direction should be considered in analysis but since the forces of wind for that direction have to be divided into components and the component forces will definitely be smaller compared to the initial forces, the effect of wind for that particular direction is not considered in the analysis.

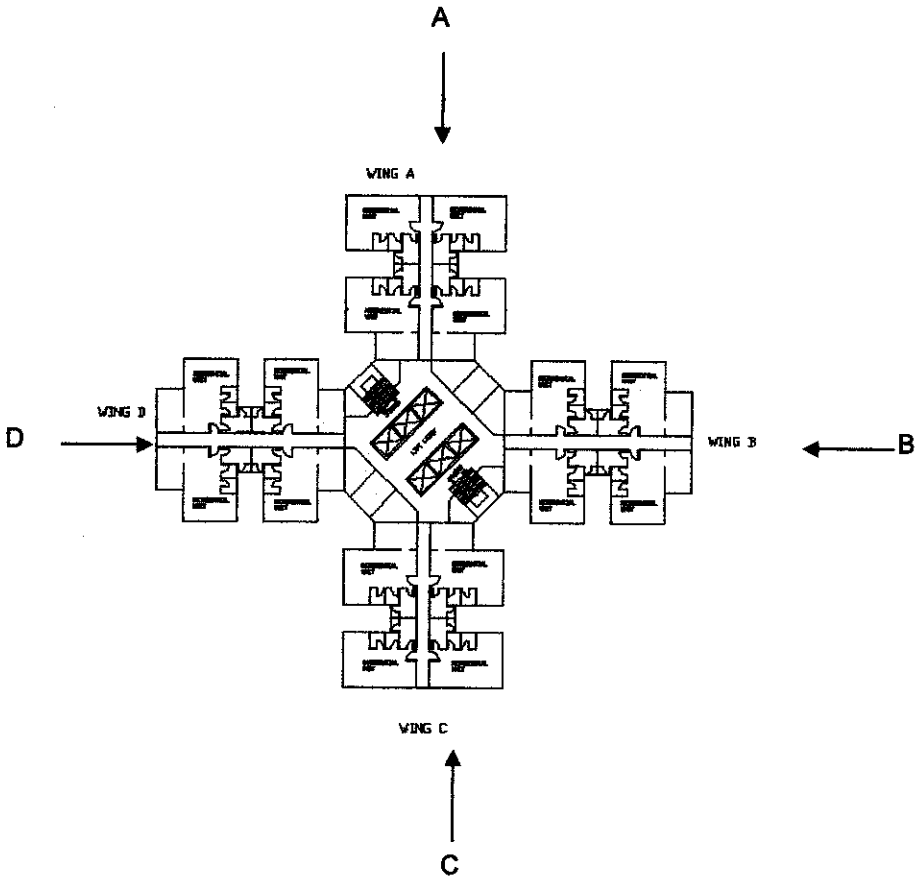


Figure 4.5   *Direction of wind*

Figure 4.6 and *Figure 4.7* show the corresponding walls that resist the wind load for the two different directions.

The magnitude of the wind forces distributed to the walls is according to the relative stiffness of the walls. The longer the wall is, the more forces distributed to it. Detailed calculation of the force distribution on the walls can be referred in the Appendix C2.

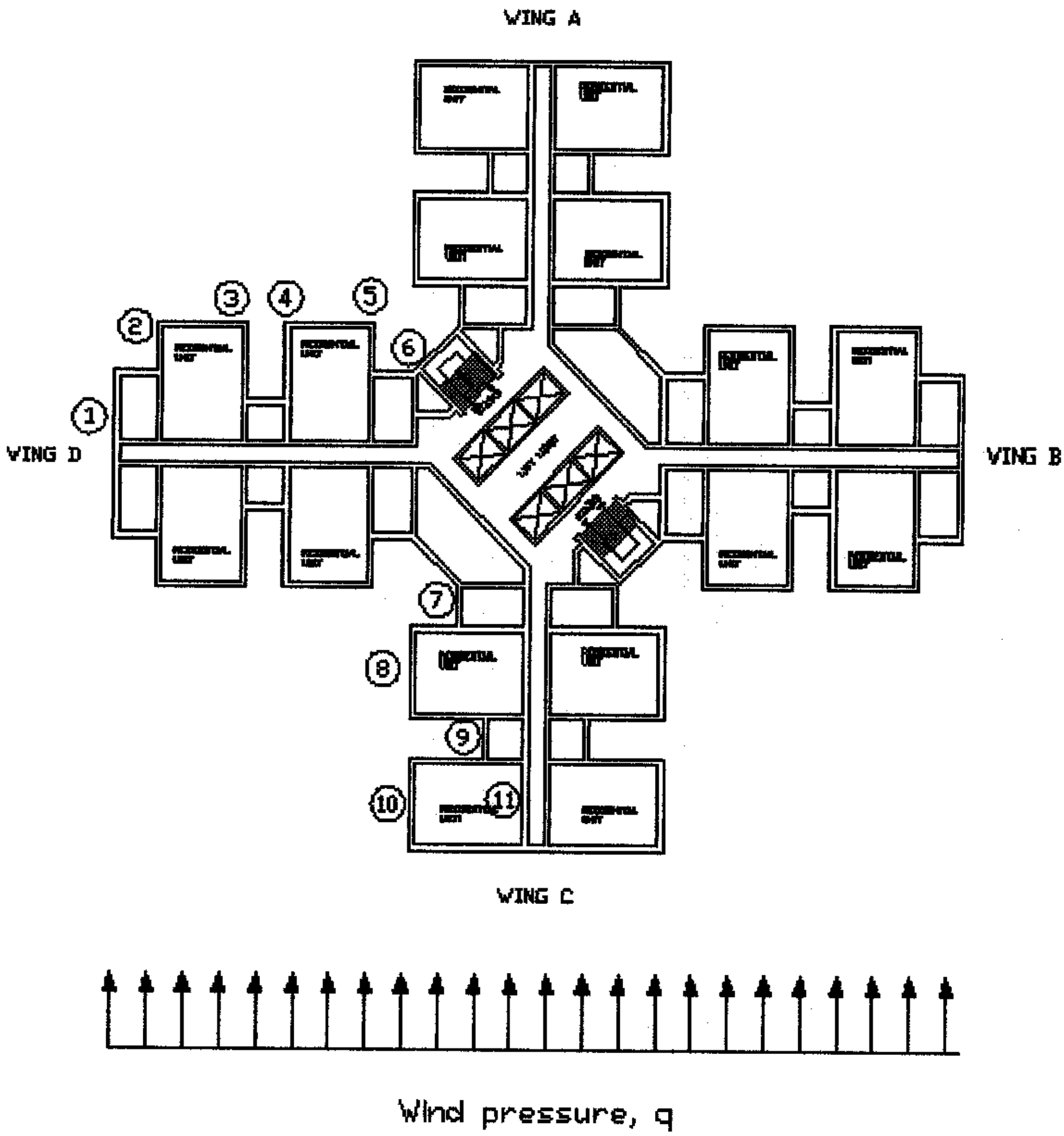


Figure 4.6 The corresponding walls that resist the wind load from the C direction

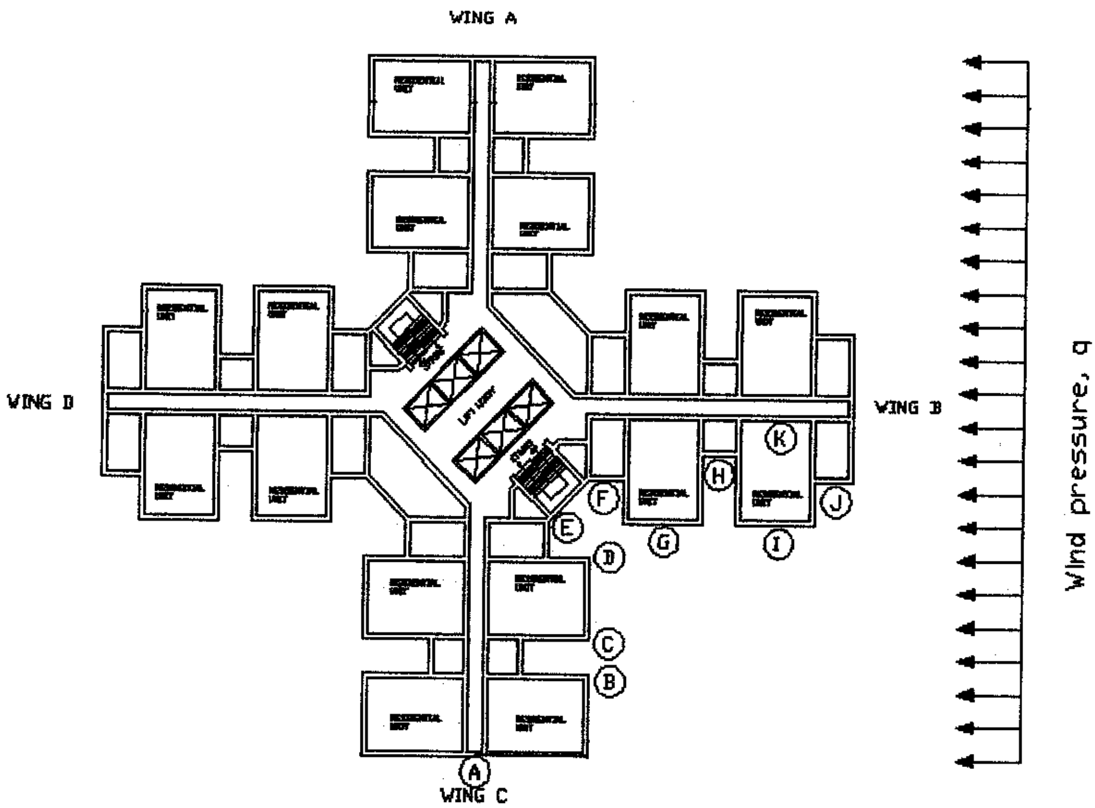


Figure 4.7 The corresponding walls that resist the wind load from the B direction

Maximum deflection of the building for load bearing wall system is calculated and the detailed calculation is shown in the Appendix C2. Maximum deflection for the load bearing wall system is 4.55 mm. The deflection is within the allowable drift index which is 1/500 or 90 mm.

Theoretically, the wall system should be stiffer compared to the frame system. However, in this case, the frame system is a little bit stiffer compared to the load bearing wall system due to stiffer size of beams and columns in the frame system while in wall system, in which the walls are of minimum thickness, the load bearing wall system became more elastic.

In load bearing wall system, the wind analysis is done for two different directions. The wind in direction B results in 2.94 mm deflection which is smaller compared to the deflection resulting by the wind in direction C which is 4.55 mm. The differences are due to the differences in force distribution to the walls and the different relative stiffness of the walls for each direction.

4.4 REINFORCED CONCRETE DESIGN DUE TO VERTICAL LOAD ACCORDING TO BS 8110

4.4.1 Structural frame system

Beam

Preliminary sizing

The size of the beams is tried to be 250 mm x 450 mm. Based on the maximum bending moment calculated in the analysis, the size is adequate.

Material strength

The strength of concrete used in the design, f_{cu}, is 30 N/mm^2 while the strength of the reinforcement steel, f_y is 460 N/mm^2

Design for bending

The beams are designed as a rectangular section. The bending moments in the beams are taken from the vertical and lateral loading analysis of the beam and frame. Detailed calculations on the design are shown in the Appendix D1.

Design for shear

The beams are designed to resist the shear from the vertical loading and the lateral loading. The detailed calculations of the design are shown in the Appendix D1. Most of the beams only required nominal links instead of shear reinforcement due to the small shear in the beams.

Column

Preliminary sizing

The size of the column is 400 mm x 400 mm. This size is adequate to design the column as short column and carry the axial load imposed to it. Size of the column is fixed for all columns to simplify the construction process.

Material strength

The strength of concrete used in the design, f_{cu}, is 30 N/mm^2 while the strength of the reinforcement steel, f_y is 460 N/mm^2

Design for axial loading

Since the columns in the building are classified as short column, the columns are more likely to fail by crushing. Therefore, the main loading for the columns to resist is the axial loading. The bending moment developed by the lateral loading (wind load) is relatively small and can be neglected for design.

The reinforcement for the column also varies and increasing as the floor level decreasing due to the increasing axial load imposed to them.

4.4.2 Load bearing wall system

Wall

Preliminary sizing: Wall thickness

The minimum thickness of load bearing reinforced concrete wall provided that fire resistance of 1.5 hours and the area of steel reinforcement is between 0.4% and 1%, is 140 mm thick. Therefore, the wall thickness of 160 mm should be adequate.

Material strength

The strength of concrete used in the design, f_{cu}, is 30 N/mm^2 while the strength of the reinforcement steel, f_y is 460 N/mm^2

Load combination

In designing the walls, four load combinations were used which are:

- Case 1: 1.4(dead) + 1.6(imposed)
- Case 2: 1.2(dead +imposed + wind)
- Case 3: 1.4(dead + wind)
- Case 4: 1.0(dead) + 1.4(wind)

These combinations were chosen based on the load combination provided by the BS 8110 and being considered in the design to ensure the walls can stand the critical stress developed from different load combinations. The load combination for Case 1 is called design and imposed load combination, for Case 2 is called dead, wind, and imposed load combination, and for Case 3 and Case 4 are called dead and wind load combination.

For Case 1, the walls are mainly subjected to the axial load only. This case is chosen to ensure the walls can stand the maximum dead and imposed load with the given safety factor. The amount of reinforcement required is obtained using the equation discussed in the methodology in Chapter 3.

For Case 2, the combination of dead, imposed and wind load is considered since at the service limit state, all these load can be appeared at the same time and may cause maximum stress to the wall. For Case 3 and Case 4, the imposed load is not being considered to check the severity of the wind load under minimal vertical gravity load. The steel reinforcement required by these load combination is obtained using the design chart modified according to the BS 8110 (*Figure 4.8*).

The detailed calculation and design of the walls for different load combinations is shown in the Appendix D2. The most severe conditions out of three load combinations should be use in the design. Based on the design calculations, all three load combinations results in minimum reinforcement in all walls.

Capacity of the designed walls

Based on the analysis and design calculation of the load bearing wall system, all walls have to be reinforced with minimum reinforcement. By reversing the calculation of determining the reinforcement required, the capacity of the wall is calculated as:

$$
\begin{aligned}
n_W &= 0.35 f_{cu} A_c + 0.67 A_{sc\ fy} \\
&= 0.35(30)(160 \times 1000) + 0.67(670)(460) \\
&= 1473506 \text{ N/ m length} \\
&= 1473.5 \text{ kN/ m length}
\end{aligned}
$$

Comparing to the maximum axial design load of 577.36 kN/m length (in case of ultimate dead and imposed load combination on wall 3 and 9), the wall only carries about 40% of its capacity. Therefore, with the same thickness and reinforcement, the wall may be able to resist the load for 30 storey building rather than only 15 storey building used for this study.

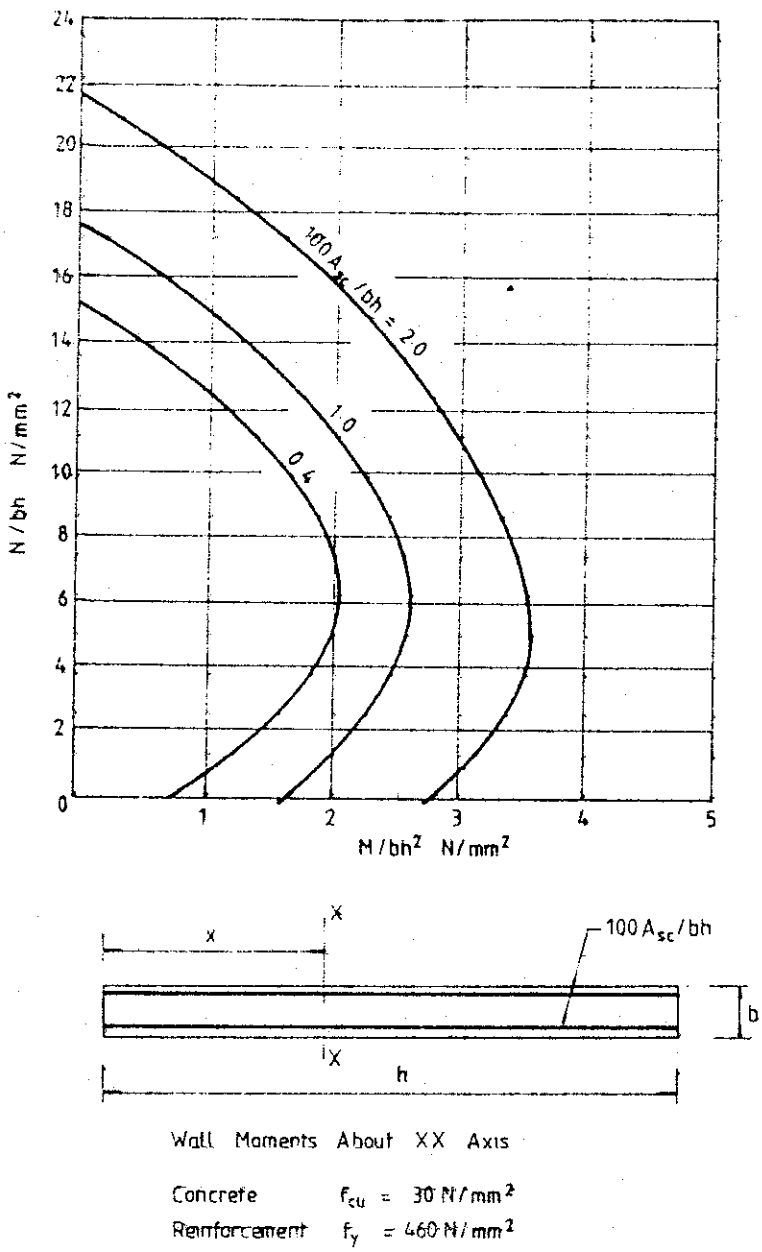


Figure 4.8 Design chart for f_{cu} = 30 N/mm² and f_y = 460 N/mm²

4.5 COMPARISON BETWEEN THE FRAME SYSTEM AND THE LOAD BEARING WALL SYSTEM

4.5.1 Materials quantity

The materials that are compared for their quantity includes:

- Concrete
- Steel reinforcement

These materials are the main contributor of the total cost of the building. The quantity of these materials is on the main structural part of the building. The materials quantity comparison for both frame and load bearing wall systems is summarized in *Table 4.2* and *Figure 4.9.* Detail calculation of determining the quantity of the materials is shown in Appendix E.

	Frame System	Load Bearing Wall System
Concrete (m^3)	2764.525	4459.935
Steel reinforcement (kg)	434030.5	700209.8
Wall (enclosure) (m^2)	21480	-

Table 4.2 Material quantity comparison for frame and load bearing wall system

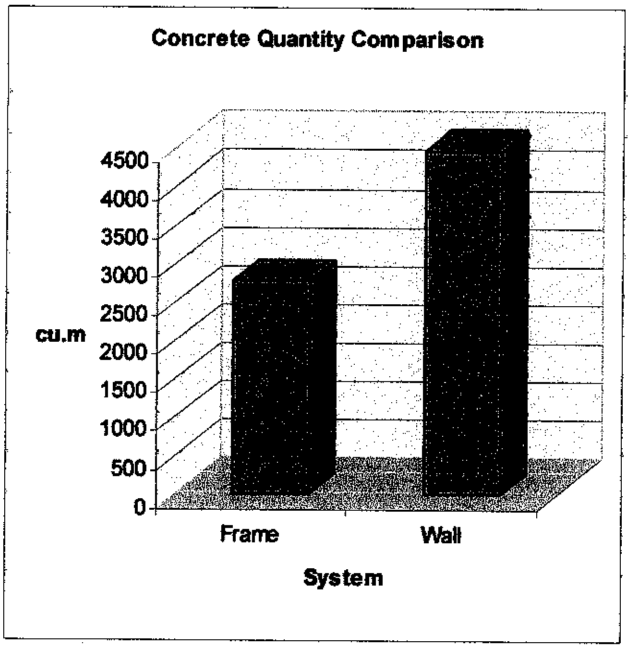


Figure 4.9 *Concrete quantity comparisons between two systems*

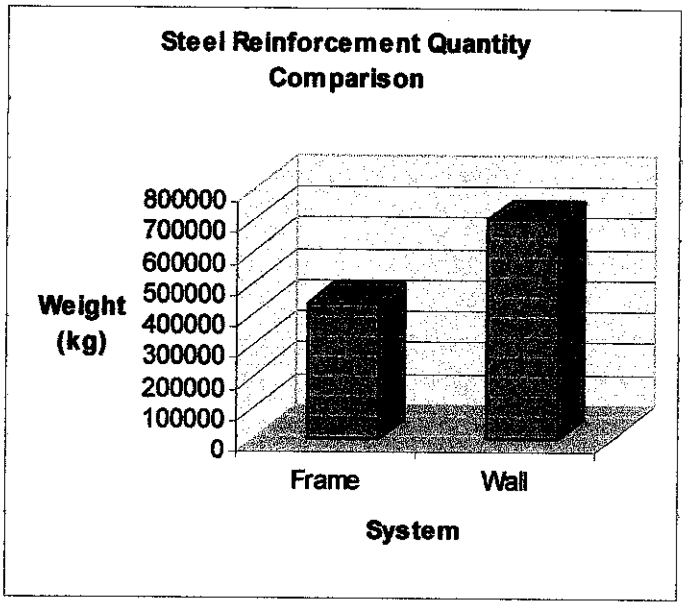


Figure 4.10 *Steel reinforcement quantity comparisons between two systems*

From the *Table 4.2, Figure 4.9,* and *Figure 4.10,*, the frame system has an advantage against the load bearing wall system by having reinforced concrete 61.3% lesser. However, the frame system required additional wall for enclosing and dividing purposes. The quantity of the wall required is 21480 m^2. This additional wall has to be considered in the comparison since the frame system does not have the enclosure wall while in the load bearing system, the enclosure is done by the structural walls.

4.5.2 Total weight of the building

The overall weight of the building is important parameter to be compared between both systems since the parameter's magnitude will significantly affect the foundation design of the building and indirectly the total cost of the building itself. The heavier the building is means that more bearing capacity is required to be provided by the foundation to cater the loads. When the required bearing capacity of the foundation is greater, the cost to construct the foundation will also be greater.

The overall weight of the building constructed using frame system is 149843.6 kN while for load bearing wall system, the overall weight of the building is 119649.4 kN. The total overall weight for frame system is about 20% more than the load bearing wall system. Theoretically, the frame system should be lighter than the load bearing wall system. However, in this case, the frame system is heavier than the load bearing wall system. This is due to the size of the structural wall in the load bearing wall system is almost the same size as the brick wall in the frame system and the unit weight of the brick wall is almost the same as concrete. The extra weight in the frame system is mainly due to the size of columns and beams. Therefore, in this case, the load bearing wall system has an advantage of requiring smaller bearing capacity foundation compared to the frame system.

4.5.3 Construction advantage

Frame system seems to be quicker to be erected due to lesser concrete and reinforcement required. This is because more time is required to shape the larger amount of reinforcement steel, place the greater amount concrete, and curing the massive concrete.

However, the disadvantage of having greater amount of concrete and steel in the load bearing wall system can be minimized by implementing new construction technique such as using prefabricated concrete walls and slabs in the building.

CHAPTER 5

CONCLUSION

Each frame system and load bearing wall system has their own advantages and disadvantages against another. Selecting between the two systems should be made carefully by justifying the following criteria:

- Height of the building
- Layout plan for each floor
- Construction technology availability
- Time limitation to build the building

Selecting the right system can prevent any cost redundancy and time wasting due to overly designed structure or unnecessary complicated analysis.

The frame system is suitable for moderate height building with any floor layout arrangement while the load bearing wall system is more suitable for high rise building with repetitive floor layout arrangement.

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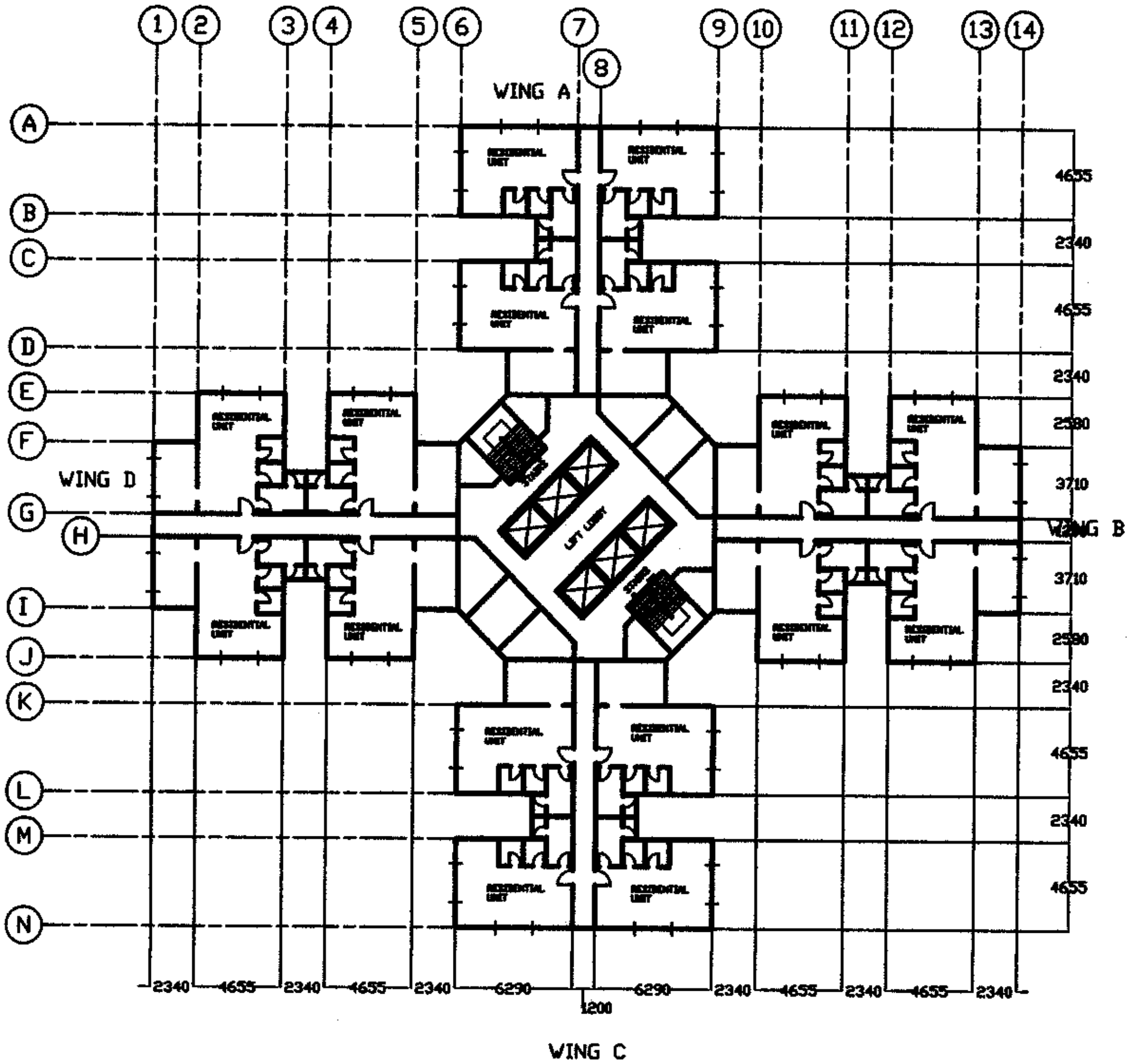
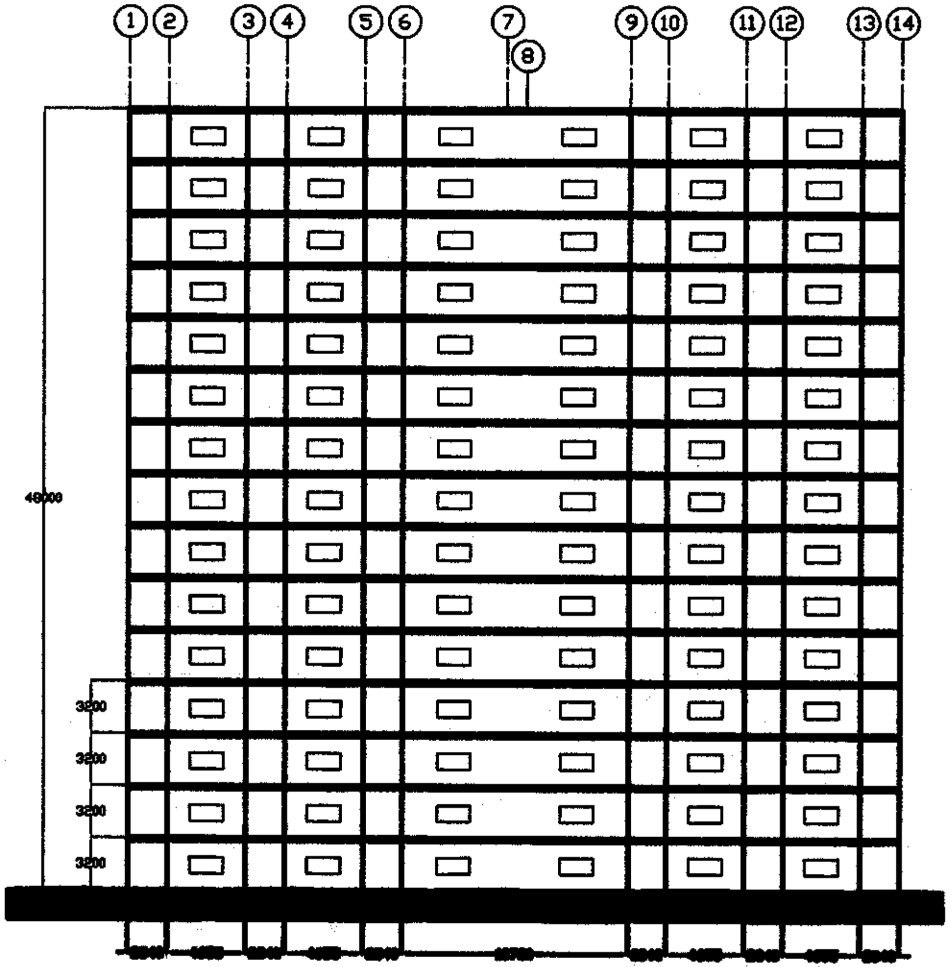
APPENDIX A

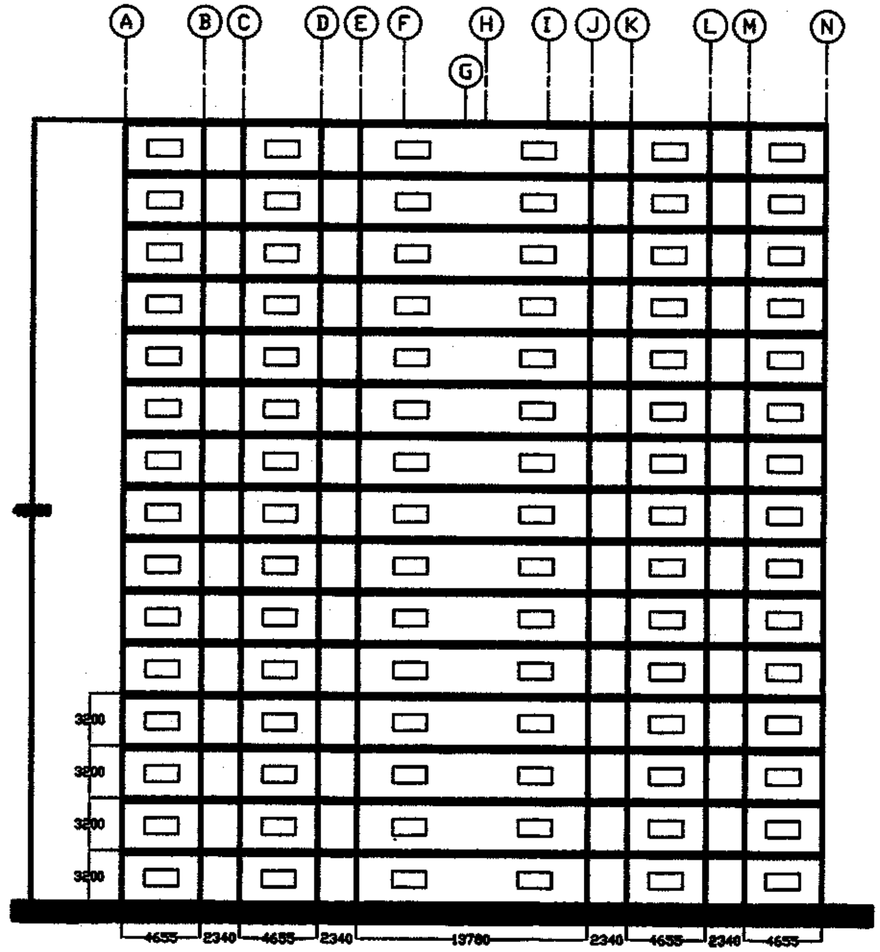
BUILDING LAYOUT PLANS



BASIC LAYOUT



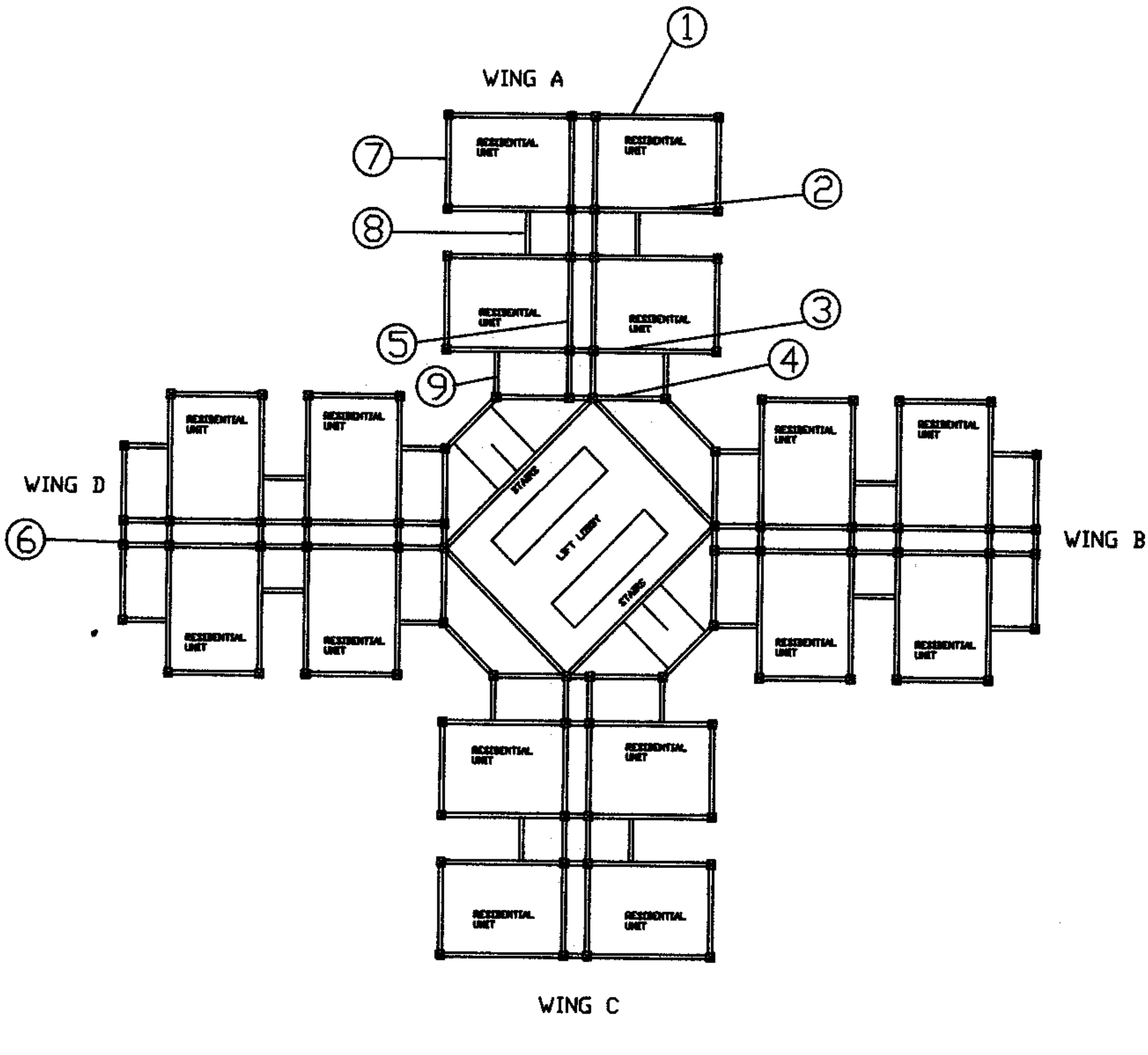
ELEVATION VIEW (A)



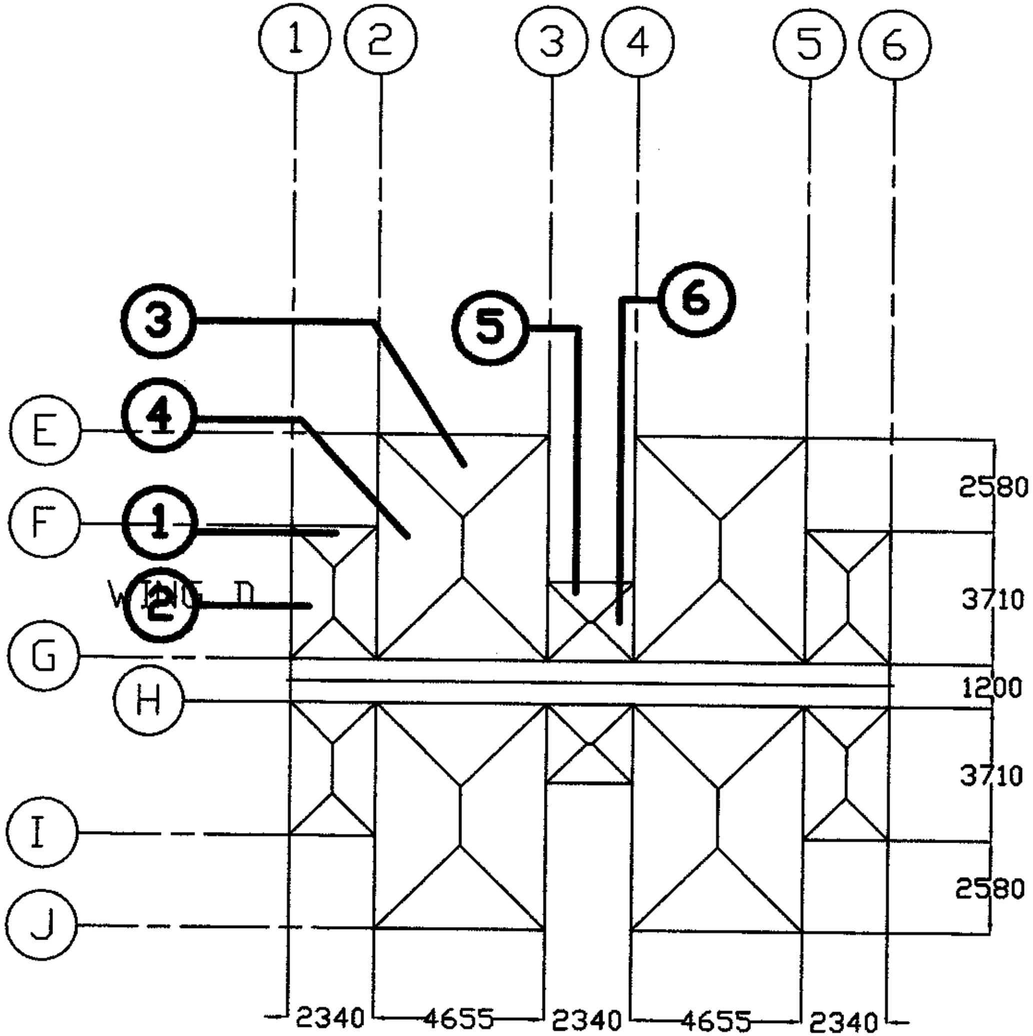
ELEVATION VIEW (B)

APPENDIX B1

VERTICAL LOADING ANALYSIS FOR FRAME SYSTEM



LOCATION OF BEAMS IN FRAME SYSTEM



SLAB DIVISION AT WING D

vertical loading (frame system)

slab division

slab	dimension	area
1	0.5*2.34*1.17	1.37
2	0.5*(1.37+3.71)*1.17	2.97
3	0.5*4.65*2.33	5.42
4	0.5*(1.64+6.29)*2.33	9.24
5	0.5*(0.15+2.34)*1.1	1.37
6	0.5*2.2*1.2	1.32

slab dead load

slab thickness 0.2 m

slab self weight = 24kN/cu.m x 0.2m 4.8 kN/sq.m

slab live load

take slab live load as 3 kN/sq.m

wall dead load (self weight)

wall thickness, d 0.15 m
wall height, h 3 m

wall self weight = 24kN/cu.m x d x h 9.9 kN/m

simply supported beam

size

width 0.25 m

depth 0.4 m

loading

slab dead load 4.8 kN/sq.m

slab live load 3 kN/sq.m

beam self weight 1.8 kN/m

beam	length	slab type	slab area	udl	max shear	max moment
7	4.65	3	5.42	64.93	150.95	175.48
8	2.34	5	1.37	18.30	21.41	12.52
9	2.34	1	1.37	18.29	21.40	12.52

alysis of beam (approximate Method)

beam ①

Udl :

slab ④ $= 9.24\ m^2$

load from slab $= \dfrac{9.24\,[1.4\,(4.8) + 1.6\,(3)]}{6.29}$

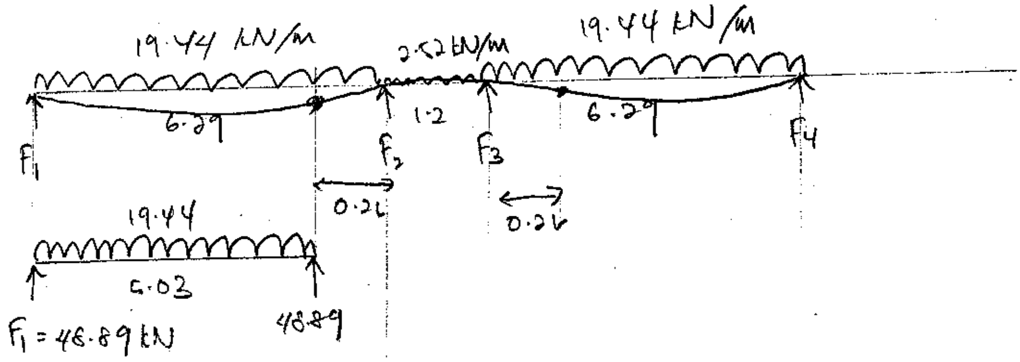
$= 16.92\ kN/m$

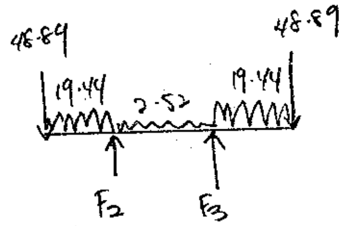
beam self weight $= 1.4\,(1.8)$

$= 2.52\ kN/m$

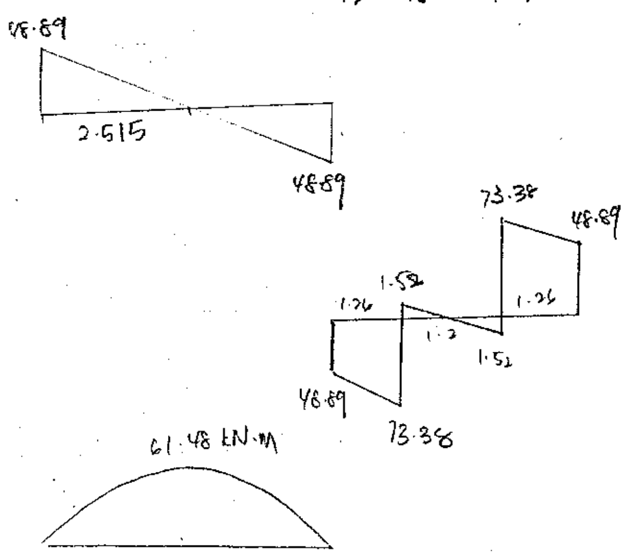
total $= 16.92 + 2.52$

$= \underline{\underline{19.44}}\ kN/m$





$F_2 = F_3 = 74.90\ kN$



beam ②

UdL:

slab ④ = 16.92 kN/m

slab ⑥ = 1.32 m²

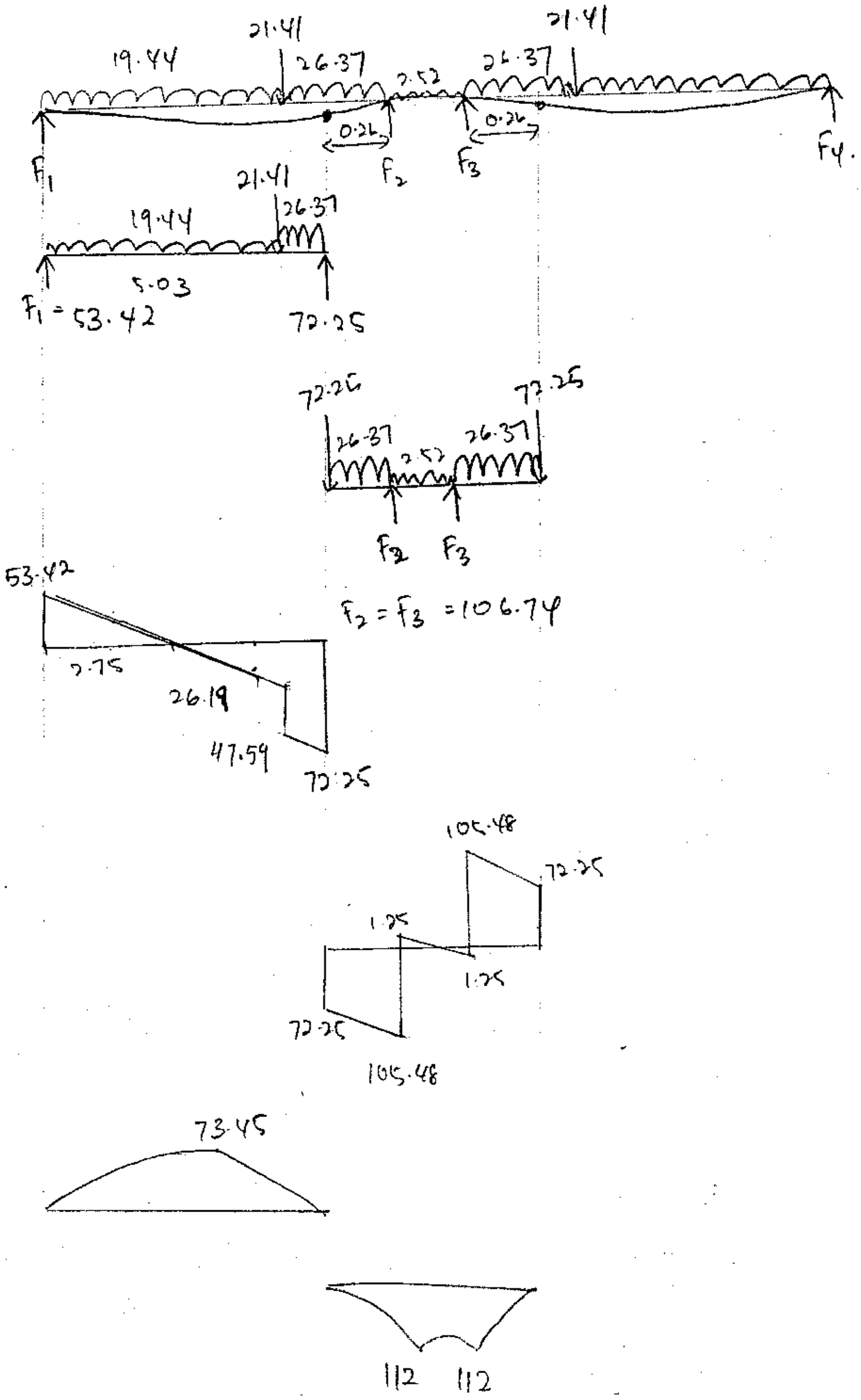
load from slab ⑥ $= \dfrac{1.32\,[1.4(4.8) + 1.6(3)]}{2.195}$

$= 6.93$ kN/m

beam self weight = 2.52 kN/m

total = 16.92 + 6.93 + 2.52

= **26.37 kN/m**



eam ③

UDL :

slab ④ = 16.92 kN/m

slab ② = 2.97 m²

load from slab ② = $\frac{2.97\,[1.4(4.8)+1.6(3)]}{3.71}$

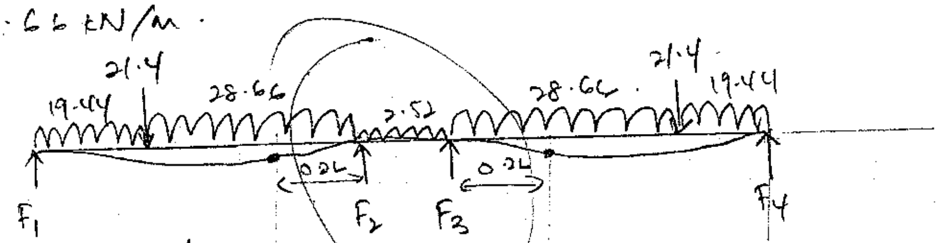
= 9.22 kN/m

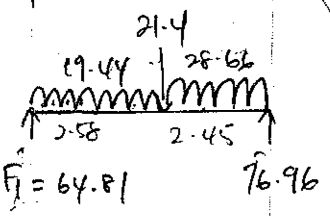
beam self weight = 1.4 (1.8)

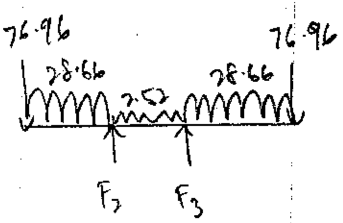
= 2.52 kN/m

total = 16.92 + 9.22 + 2.52

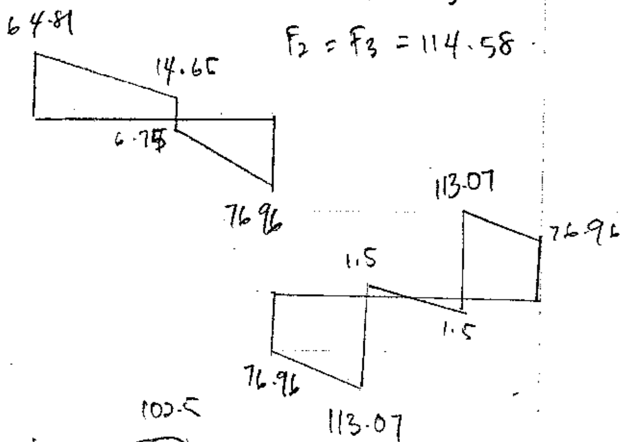
= 28.66 kN/m

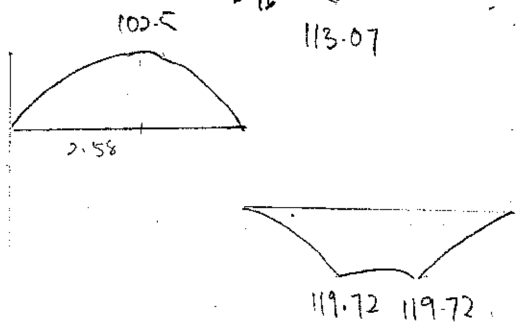






F2 = F3 = 114.58





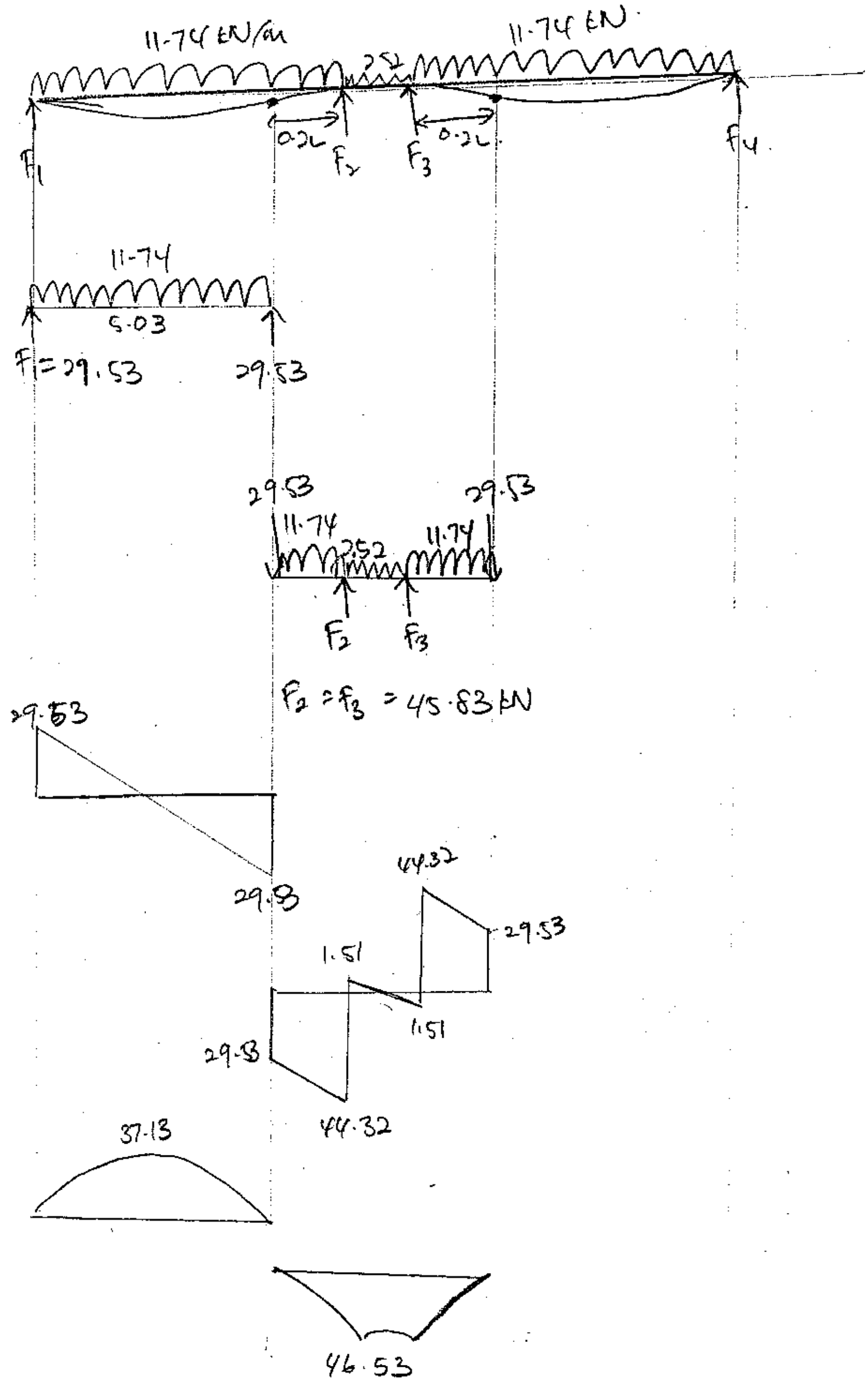
m ④

UDL :

slab ② = 9.22 kN/m

beam self weight = 1.4(1.8) = 2.52 kN/m

total = 11.74 kN/m .



Beam (6)

UDL :

corridor slab $= 1.4(24 \times 0.2 \times 0.6)$

$= 4.03$ kN/m

slab (3) $= \dfrac{5.42\,[1.4(4.8) + 1.6(3)]}{4.65}$

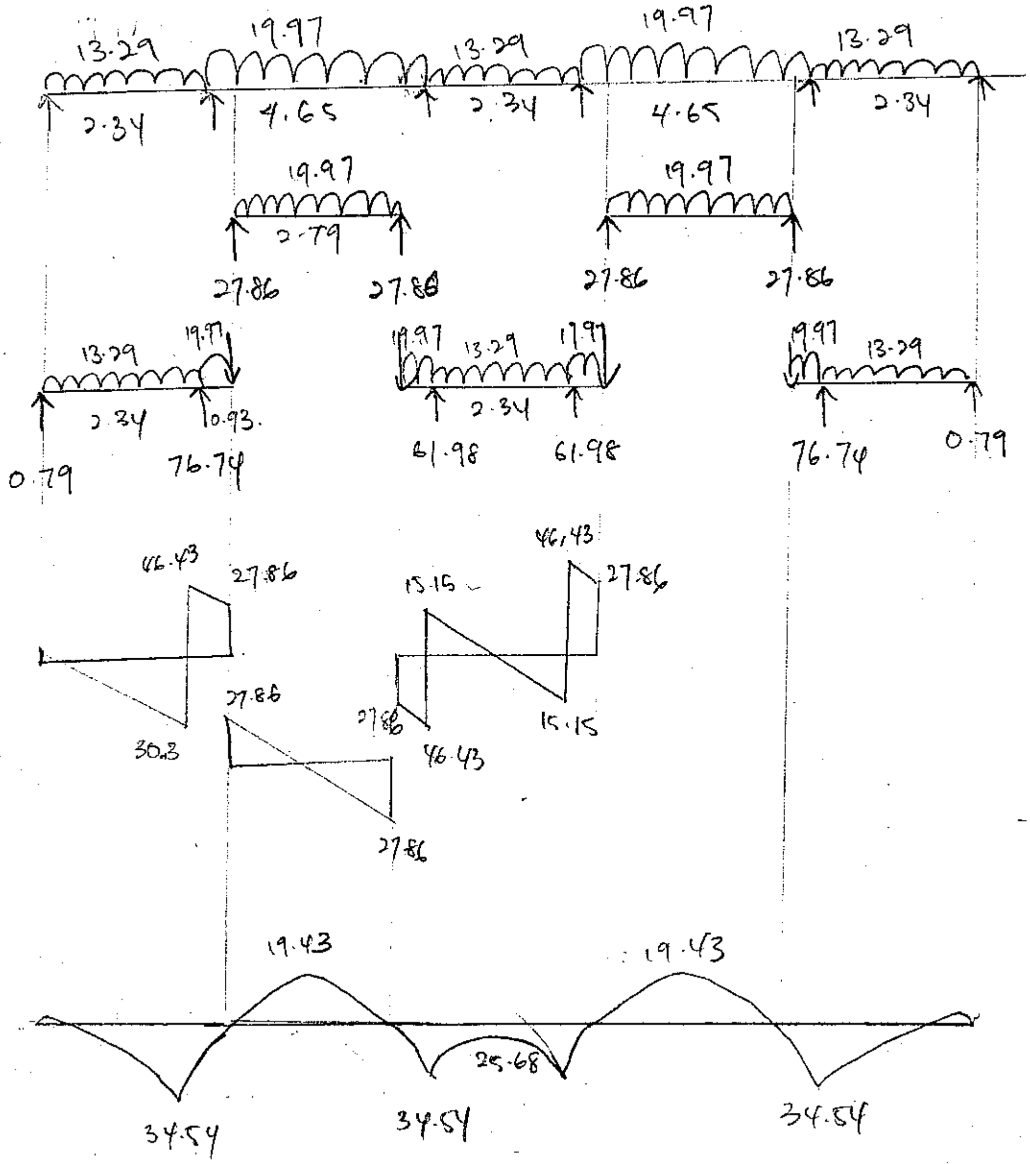
$= 13.42$ kN/m

slab (5) $= \dfrac{1.37\,[1.4(4.8) + 1.6(3)]}{2.34}$

$= 6.74$ kN/m

slab (1) $= 6.74$ kN/m

beam self weight $= 2.52$ kN/m



column

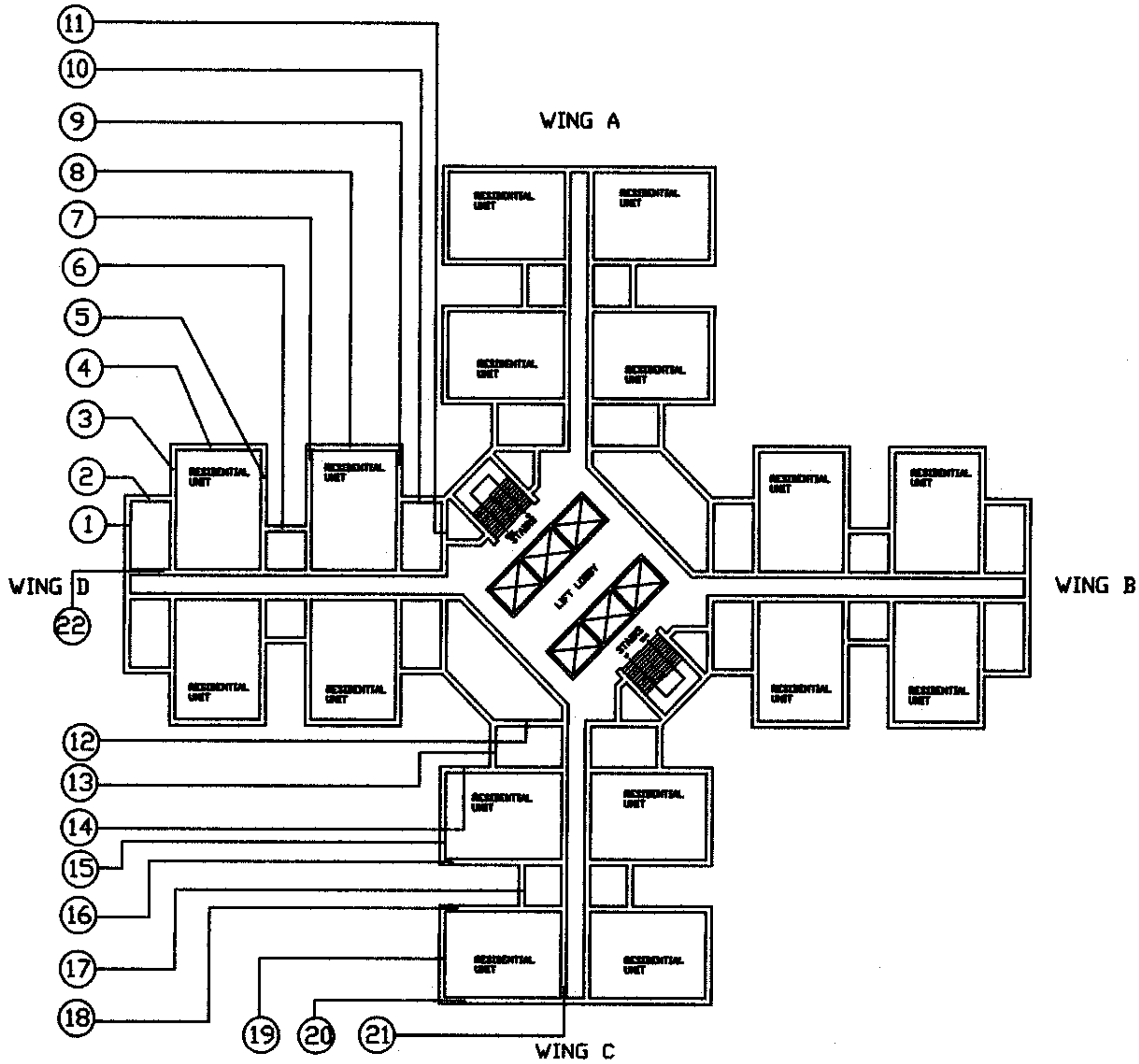
size

width 0.4 m
height 3 m

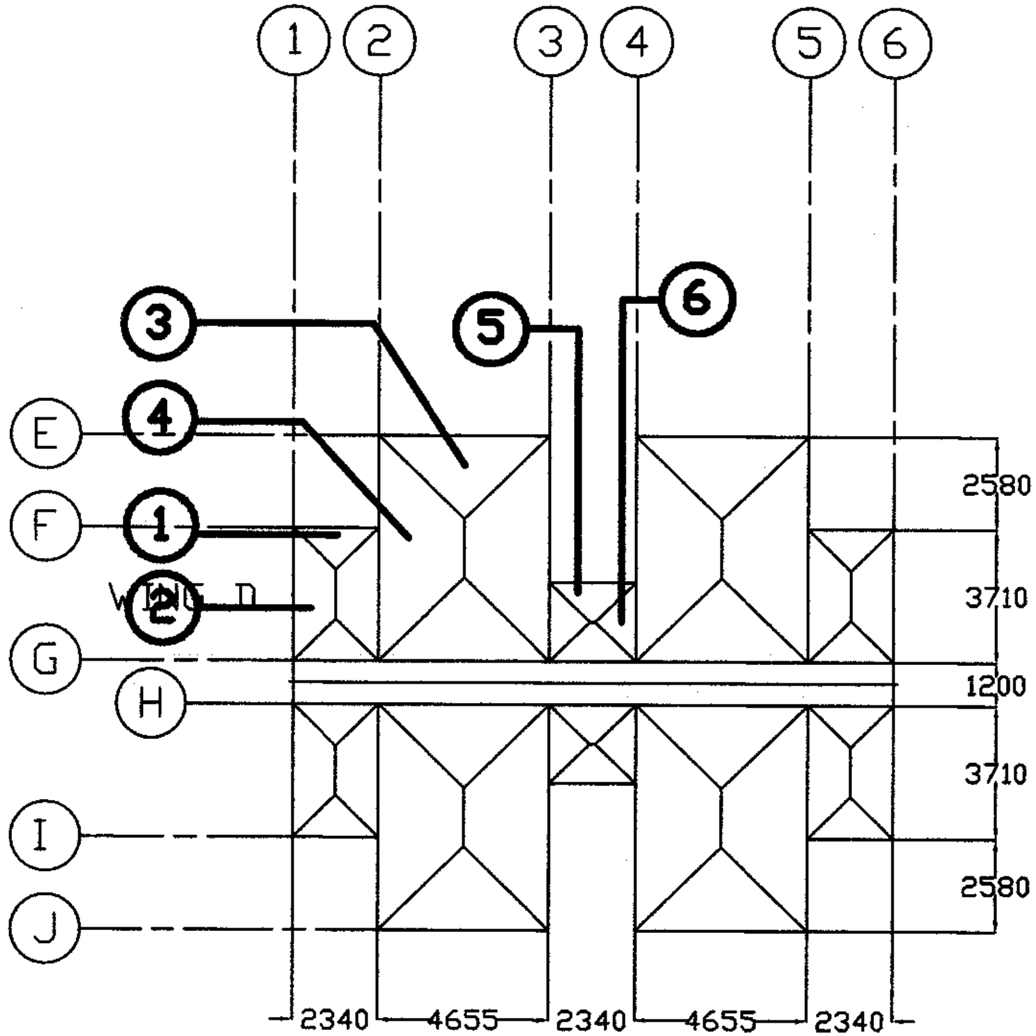
column	loading form beam		total	total x 15
	4	9		
1/F	29.53	21.4	50.93	763.95
	4	6		
1/G	45.83	0.79	46.62	699.3
	3	7		
2/E	64.81	150.95	215.76	3236.4
	3	6		
2/G	113.07	76.74	189.81	2847.15
	2	7		
3/E	53.42	150.95	204.37	3065.55
	2	6		
3/G	105.48	61.98	167.46	2511.9

APPENDIX B2

VERTICAL LOADING ANALYSIS FOR LOAD BEARING WALL SYSTEM



LOCATION OF THE LOAD BEARING WALL IN LOAD BEARING WALL SYSTEM



SLAB DIVISION AT WING D

vertical loading (load bearing wall system)

slab division

slab	dimension	area
1	0.5*2.34*1.17	1.37
2	0.5*(1.37+3.71)*1.17	2.97
3	0.5*4.65*2.33	5.42
4	0.5*(1.64+6.29)*2.33	9.24
5	0.5*(0.15+2.34)*1.1	1.37
6	0.5*2.2*1.2	1.32

slab dead load

slab thickness 0.2 m

slab self weight = 24kN/cu.m x 0.2m 4.8 kN/sq.m

slab live load

take slab live load as 3 kN/sq.m

wall dead load (self weight)

wall thickness, d 0.16 m
wall height, h 3 m

wall self weight = 24kN/cu.m x d x h 11.52 kN/m loading on wall

wall	subjected to slab	length (m)	slb area (sq.m)	dead load (kN/m)	live load (kN/m)
1	2	3.71	2.97	15.36	2.40
3,9	4,2	6.29	12.21	20.84	5.82
5,7	4,6	6.29	10.56	19.58	5.04
11	2	3.71	2.97	15.36	2.40
2,10	1	2.34	1.37	14.33	1.76
4,8	3	4.65	5.42	17.11	3.50
6	5	2.34	1.37	14.33	1.76
22	3,corridor slab	4.65	8.21	19.99	5.30
20	4	6.29	9.24	18.57	4.41

dead load for every floor level (kN/m)

level	wall								
	1	3,9	5,7	11	2,10	4,8	6	22	20
15	15.36	20.84	19.58	15.36	14.33	17.11	14.33	19.99	18.57
14	30.73	41.68	39.15	30.73	28.66	34.22	28.66	39.98	37.14
13	46.09	62.51	58.73	46.09	42.98	51.34	42.99	59.98	55.71
12	61.46	83.35	78.31	61.46	57.31	68.45	57.32	79.97	74.28
11	76.82	104.19	97.89	76.82	71.64	85.56	71.65	99.96	92.85
10	92.19	125.03	117.46	92.19	85.97	102.67	85.98	119.95	111.42
9	107.55	145.86	137.04	107.55	100.30	119.78	100.30	139.94	129.99
8	122.92	166.70	156.62	122.92	114.62	136.90	114.63	159.94	148.56
7	138.28	187.54	176.20	138.28	128.95	154.01	128.96	179.93	167.13
6	153.65	208.38	195.77	153.65	143.28	171.12	143.29	199.92	185.70
5	169.01	229.22	215.35	169.01	157.61	188.23	157.62	219.91	204.27
4	184.38	250.05	234.93	184.38	171.94	205.34	171.95	239.90	222.84
3	199.74	270.89	254.51	199.74	186.26	222.46	186.28	259.90	241.41
2	215.11	291.73	274.08	215.11	200.59	239.57	200.61	279.89	259.98
1	230.47	312.57	293.66	230.47	214.92	256.68	214.94	299.88	278.55

live load for every floor level (kN/m)

level	wall								
	1	3,9	5,7	11	2,10	4,8	6	22	20
15	2.40	5.82	5.04	2.40	1.76	3.50	1.76	5.30	4.41
14	4.81	11.65	10.07	4.81	3.51	6.99	3.51	10.59	8.81
13	7.21	17.47	15.11	7.21	5.27	10.49	5.27	15.89	13.22
12	9.61	23.29	20.14	9.61	7.02	13.98	7.02	21.18	17.63
11	12.02	29.12	25.18	12.02	8.78	17.48	8.78	26.48	22.03
10	14.42	34.94	30.21	14.42	10.53	20.97	10.53	31.77	26.44
9	16.82	40.77	35.25	16.82	12.29	24.47	12.29	37.07	30.84
8	19.22	46.59	40.29	19.22	14.04	27.96	14.05	42.36	35.25
7	21.63	52.41	45.32	21.63	15.80	31.46	15.80	47.66	39.66
6	24.03	58.24	50.36	24.03	17.55	34.95	17.56	52.95	44.06
5	26.43	64.06	55.39	26.43	19.31	38.45	19.31	58.25	48.47
4	28.84	69.88	60.43	28.84	21.06	41.94	21.07	63.54	52.88
3	31.24	75.71	65.47	31.24	22.82	45.44	22.83	68.84	57.28
2	33.64	81.53	70.50	33.64	24.57	48.93	24.58	74.13	61.69
1	36.05	87.35	75.54	36.05	26.33	52.43	26.34	79.43	66.09

1.4 dead load + 1.6 live load

level	wall								
	1	3,9	5,7	11	2,10	4,8	6	22	20
15	25.36	38.49	35.47	25.36	22.87	29.55	22.87	36.46	33.05
14	50.71	76.98	70.93	50.71	45.73	59.10	45.74	72.92	66.10
13	76.07	115.47	106.40	76.07	68.60	88.65	68.61	109.38	99.14
12	101.42	153.96	141.86	101.42	91.47	118.20	91.48	145.84	132.19
11	126.78	192.45	177.33	126.78	114.34	147.74	114.35	182.30	165.24
10	152.13	230.94	212.79	152.13	137.20	177.29	137.22	218.76	198.29
9	177.49	269.44	248.26	177.49	160.07	206.84	160.09	255.23	231.34
8	202.85	307.93	283.72	202.85	182.94	236.39	182.96	291.69	264.38
7	228.20	346.42	319.19	228.20	205.80	265.94	205.83	328.15	297.43
6	253.56	384.91	354.66	253.56	228.67	295.49	228.70	364.61	330.48
5	278.91	423.40	390.12	278.91	251.54	325.04	251.57	401.07	363.53
4	304.27	461.89	425.59	304.27	274.41	354.59	274.44	437.53	396.58
3	329.63	500.38	461.05	329.63	297.27	384.13	297.31	473.99	429.62
2	354.98	538.87	496.52	354.98	320.14	413.68	320.18	510.45	462.67
1	380.34	577.36	531.98	380.34	343.01	443.23	343.05	546.91	495.72

max 577.36 kN/m

APPENDIX C1

WIND LOADING ANALYSIS FOR FRAME SYSTEM

wind pressure profile

Windward Pressure = $q_s C_e C_q I_w$

v	=	70.0	mph
Iw	=	1.0	
qs	=	0.00256*(v^2)	
	=	12.544	psf

exposure type B

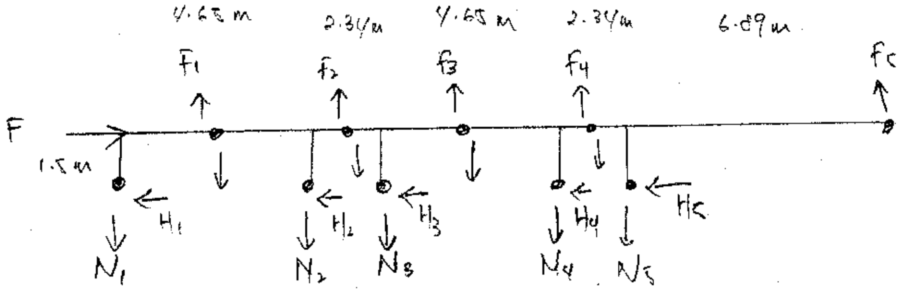
Level	Height above ground (m)	Height above ground (ft)	qs	Ce		Cq		Windward pressure (psf)	Leeward action (psf)	Design pressure psf 5+6	Design pressure lbf/in2	Design pressure kPa
				Windward	Leeward	Windward	Leeward	5	6			
15	45.0	147.6	12.54400	1.27	1.59	0.8	-0.5	12.745	9.972	22.717	0.158	1.088
14	42.0	137.8	12.54400	1.25	1.59	0.8	-0.5	12.544	9.972	22.516	0.156	1.078
13	39.0	127.9	12.54400	1.22	1.59	0.8	-0.5	12.243	9.972	22.215	0.154	1.064
12	36.0	118.1	12.54400	1.19	1.59	0.8	-0.5	11.942	9.972	21.914	0.152	1.049
11	33.0	108.2	12.54400	1.16	1.59	0.8	-0.5	11.641	9.972	21.613	0.150	1.035
10	30.0	98.4	12.54400	1.12	1.59	0.8	-0.5	11.239	9.972	21.212	0.147	1.016
9	27.0	88.6	12.54400	1.08	1.59	0.8	-0.5	10.838	9.972	20.810	0.145	0.996
8	24.0	78.7	12.54400	1.04	1.59	0.8	-0.5	10.437	9.972	20.409	0.142	0.977
7	21.0	68.9	12.54400	0.99	1.59	0.8	-0.5	9.935	9.972	19.907	0.138	0.953
6	18.0	59.0	12.54400	0.95	1.59	0.8	-0.5	9.533	9.972	19.506	0.135	0.934
5	15.0	49.2	12.54400	0.89	1.59	0.8	-0.5	8.931	9.972	18.904	0.131	0.905
4	12.0	39.4	12.54400	0.84	1.59	0.8	-0.5	8.430	9.972	18.402	0.128	0.881
3	9.0	29.5	12.54400	0.76	1.59	0.8	-0.5	7.627	9.972	17.599	0.122	0.843
2	6.0	19.7	12.54400	0.67	1.59	0.8	-0.5	6.724	9.972	16.696	0.116	0.799
1	3.0	9.8	12.54400	0.62	1.59	0.8	-0.5	6.222	9.972	16.194	0.112	0.775

Frame System

Lateral Loading (cantilever method)

Section A - C

roof



$F = q(13.78)(3)$

$= 1.09(13.78)(3)$

$= 44.97$ kN

① Axial load on columns

$N_1 = 3.03P$ $\quad$ $N_3 = 2.01P$ $\quad$ $N_5 = 1.0P$

$N_2 = 2.35P$ $\quad$ $N_4 = 1.34P$

Taking moment at A = 0

$\circlearrowleft \Sigma M_A = 0$: $44.97(1.5) + 2.35P(4.65) + 2.01P(6.99) + 1.34P(11.64)$
$+ 1.0P(13.98) - 1.0P(27.76) - 1.34P(30.1) - 2.01P(34.7$
$- 2.35P(37.09) - 3.03P(41.74)$

$= 67.46 + 2.35P(-32.44) + 2.01P(-27.76) + 1.34P(-18.46)$
$+ 1.0P(-13.78) - 3.03P(41.74)$

$= 67.46 - 297.0202P = 0$

$P = \dfrac{67.46}{297.0202} = 0.2271$

Therefore :

$N_1 = 0.688$ kN $\quad$ $N_3 = 0.456$ kN $\quad$ $N_5 = 0.227$ kN

$N_2 = 0.534$ kN $\quad$ $N_4 = 0.304$ kN

② Vertical shearing force on beam

For each part of subframe, $\Sigma F = 0$

$F_1 = N_1 = 0.688$ kN

$F_2 = N_1 + N_2 = 0.688 + 0.534 = 1.222$ kN

$F_3 = N_1 + N_2 + N_3 = 1.222 + 0.456 = 1.678$ kN

$F_4 = N_1 + N_2 + N_3 + N_4 = 1.678 + 0.304 = 1.983$ kN

$F_5 = N_1 + N_2 + N_3 + N_4 + N_5 = 1.983 + 0.227 = 2.210$ kN

③ Horizontal shearing force on column

Taking moment at counterflexure at each beam $= 0$

$H_1 \times 1.5 - N_1 \times 2.325 = 0$

$H_1 = 1.067$ kN

$(H_1 + H_2) \times 1.5 - N_1 \times 5.82 - N_2 \times 1.17 = 0$

$H_2 = 2.020$ kN

$(H_1 + H_2 + H_3) \times 1.5 - N_1 \times 9.315 - N_2 \times 4.665 - N_3 \times 2.325 = 0$

$H_3 = 3.554$ kN

$(H_1 + H_2 + H_3 + H_4) \times 1.5 - N_1 \times 12.81 - N_2 \times 8.16 - N_3 \times 5.82 - N_4 \times 1.17 = 0$

$H_4 = 4.148$ kN

$(H_1 + H_2 + H_3 + H_4 + H_5) \times 1.5 - N_1 \times 20.87 - N_2 \times 16.22 - N_3 \times 13.88$

$- N_4 \times 9.23 - N_5 \times 6.59 = 0$

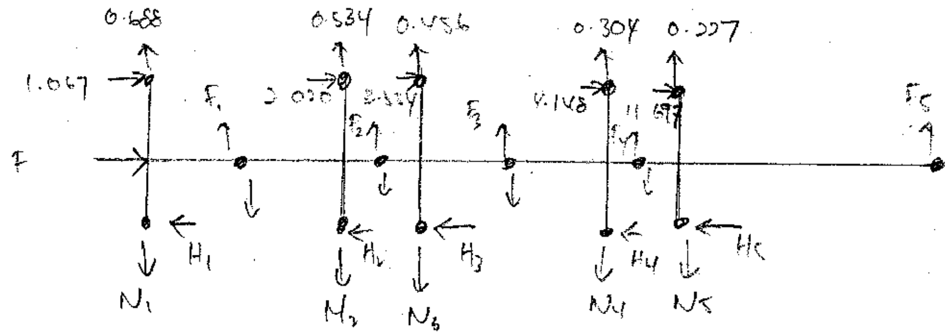
$H_5 = 11.697$ kN

level 14

$F = q(13.78)(3)$

$= 1.08(13.78)(3)$

$= 44.57$ kN



Axial forces on columns.

Taking moment at A = 0

$\circlearrowleft + \sum M_A = 0: \quad 44.97(4.5) + 44.57(1.5) - 297.0202P = 0$

$\therefore P = \dfrac{269.22}{297.0202} = 0.9064$ kN

Therefore: $N_1 = 2.75$ kN $\quad N_3 = 1.82$ kN $\quad N_5 = 0.905$ kN.

$N_2 = 2.13$ kN $\quad N_4 = 1.21$ kN

Vertical shearing forces on beams.

For each part of subframe, $\sum F = 0$

$F_1 = N_1 - 0.688 = 2.746$ kN

$F_2 = N_1 + N_2 - 0.688 - 0.534 = 4.875$ kN

$F_3 = N_1 + N_2 + N_3 - 0.688 - 0.534 - 0.456 = 6.698$ kN

$F_4 = N_1 + N_2 + N_3 + N_4 - 0.688 - 0.534 - 0.456 - 0.304 = 7.913$ kN

$F_5 = N_1 + N_2 + N_3 + N_4 + N_5 - 0.688 - 0.534 - 0.456 - 0.304 - 0.227 = 8.81$

5) Horizontal shearing forces on columns

Taking moment at counterflexures at each beam = 0

$(H_1 + 1.067) \times 1.5 - (N_1 - 0.688) \times 2.325 = 0$

$H_1 = 2.124$ kN

$(H_1 + H_2 + 1.067 + 2.02) \times 1.5 - (N_1 - 0.688) \times 5.82 - (N_2 - 0.534) \times 1.17 = 0$

$H_2 = 4.021$ kN

$(H_1 + H_2 + H_3 + 1.067 + 2.02 + 3.554) \times 1.5 - (N_1 - 0.688) \times 9.315 - (N_2 - 0.534) \times 4$

$- (N_3 - 0.456) \times 2.325 = 0$

$H_3 = 7.077$ kN

$(H_1 + H_2 + H_3 + H_4 + 1.067 + 2.02 + 3.554 + 4.148) \times 1.5 - (N_1 - 0.688) \times 12.81$

$- (N_2 - 0.534) \times 8.16 - (N_3 - 0.456) \times 5.82 - (N_4 - 0.304) \times 1.17 = 0$

$H_4 = 8.259$ kN

$(H_1 + H_2 + H_3 + H_4 + H_5 + 1.067 + 2.02 + 3.554 + 4.148 + 11.697) \times 1.5$

$- (N_1 - 0.688) \times 20.57 - (N_2 - 0.534) \times 16.22 - (N_3 - 0.456) \times 13.88$

$- (N_4 - 0.304) \times 9.23 - (N_5 - 0.227) \times 6.69 = 0$

$H_5 = 23.289$ kN

cantilever method

roof

lateral force		44.97 kN
N1		3.03 P
N2		2.35 P
N3		2.01 P
N4		1.34 P
N5		1 P

axial load on column

taking moment at A =0

P	=	F*(1.5)/297.0202
	=	0.227106 kN
N1	=	0.688 kN
N2	=	0.534 kN
N3	=	0.456 kN
N4	=	0.304 kN
N5	=	0.227 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	0.688 kN
F2	=	1.222 kN
F3	=	1.678 kN
F4	=	1.983 kN
F5	=	2.210 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	1.067 kN
H2	=	2.020 kN
H3	=	3.554 kN
H4	=	4.148 kN
H5	=	11.697 kN

moment at beam support

span1	=	1.600 kN.m
span2	=	1.430 kN.m
span3	=	3.902 kN.m
span4	=	2.320 kN.m

moment at column

column1	=	1.600 kN.m
column2	=	3.029 kN.m
column3	=	5.332 kN.m
column4	=	6.222 kN.m
column5	=	17.545 kN.m

level 14

lateral force		44.57 kN
N1		3.03 P
N2		2.35 P
N3		2.01 P
N4		1.34 P
N5		1 P

axial load on column

taking moment at A =0

P	=	(F*(3.5)+F*(1.5)/297.0202
	=	0.906403 kN
N1	=	2.746401 kN
N2	=	2.130047 kN
N3	=	1.82187 kN
N4	=	1.21458 kN
N5	=	0.906403 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	2.746 kN
F2	=	4.876 kN
F3	=	6.698 kN
F4	=	7.913 kN
F5	=	8.819 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	2.124 kN
H2	=	4.021 kN
H3	=	7.077 kN
H4	=	8.259 kN
H5	=	23.289 kN

moment at beam support

span1	=	6.385 kN.m
span2	=	5.705 kN.m
span3	=	15.574 kN.m
span4	=	9.258 kN.m

moment at column

column1	=	3.186 kN.m
column2	=	6.032 kN.m
column3	=	10.616 kN.m
column4	=	12.388 kN.m
column5	=	34.934 kN.m

level 13

lateral force 43.97 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P = 2.032842 kN

N1	=	6.15951 kN
N2	=	4.777178 kN
N3	=	4.086011 kN
N4	=	2.724008 kN
N5	=	2.032842 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	6.160 kN
F2	=	10.937 kN
F3	=	15.023 kN
F4	=	17.747 kN
F5	=	19.780 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	3.167 kN
H2	=	5.996 kN
H3	=	10.553 kN
H4	=	12.314 kN
H5	=	34.725 kN

moment at beam support

span1	=	14.321 kN.m
span2	=	12.796 kN.m
span3	=	34.928 kN.m
span4	=	20.764 kN.m

moment at column

column1	=	4.750 kN.m
column2	=	8.994 kN.m
column3	=	15.829 kN.m
column4	=	18.472 kN.m
column5	=	52.088 kN.m

level 12

lateral force 43.38 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P = 3.600412 kN

N1	=	10.90925 kN
N2	=	8.460967 kN
N3	=	7.236827 kN
N4	=	4.824552 kN
N5	=	3.600412 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	10.909 kN
F2	=	19.370 kN
F3	=	26.607 kN
F4	=	31.432 kN
F5	=	35.032 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	4.195 kN
H2	=	7.944 kN
H3	=	13.981 kN
H4	=	16.316 kN
H5	=	46.008 kN

moment at beam support

span1	=	25.364 kN.m
span2	=	22.663 kN.m
span3	=	61.861 kN.m
span4	=	36.775 kN.m

moment at column

column1	=	6.293 kN.m
column2	=	11.916 kN.m
column3	=	20.972 kN.m
column4	=	24.473 kN.m
column5	=	69.013 kN.m

level 11

lateral force 42.78 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P	=	5.603104 kN

N1	=	16.9774 kN
N2	=	13.16729 kN
N3	=	11.26224 kN
N4	=	7.508159 kN
N5	=	5.603104 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	16.977 kN
F2	=	30.145 kN
F3	=	41.407 kN
F4	=	48.915 kN
F5	=	54.518 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	5.210 kN
H2	=	9.866 kN
H3	=	17.363 kN
H4	=	20.261 kN
H5	=	57.135 kN

moment at beam support

span1	=	39.472 kN.m
span2	=	35.269 kN.m
span3	=	96.271 kN.m
span4	=	57.231 kN.m

moment at column

column1	=	7.815 kN.m
column2	=	14.798 kN.m
column3	=	26.044 kN.m
column4	=	30.392 kN.m
column5	=	85.703 kN.m

level 10

lateral force 41.99 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P	=	8.033898 kN

N1	=	24.34271 kN
N2	=	18.87966 kN
N3	=	16.14814 kN
N4	=	10.76542 kN
N5	=	8.033898 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	24.343 kN
F2	=	43.222 kN
F3	=	59.371 kN
F4	=	70.136 kN
F5	=	78.170 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	6.206 kN
H2	=	11.751 kN
H3	=	20.681 kN
H4	=	24.134 kN
H5	=	68.057 kN

moment at beam support

span1	=	56.597 kN.m
span2	=	50.570 kN.m
span3	=	138.036 kN.m
span4	=	82.059 kN.m

moment at column

column1	=	9.309 kN.m
column2	=	17.627 kN.m
column3	=	31.022 kN.m
column4	=	36.202 kN.m
column5	=	102.085 kN.m

level 9

lateral force		41.19 kN
N1		3.03 P
N2		2.35 P
N3		2.01 P
N4		1.34 P
N5		1 P

axial load on column

taking moment at A =0

P	=	10.88476 kN
N1	=	32.98084 kN
N2	=	25.5792 kN
N3	=	21.87838 kN
N4	=	14.58558 kN
N5	=	10.88476 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	32.981 kN
F2	=	58.560 kN
F3	=	80.438 kN
F4	=	95.024 kN
F5	=	105.909 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	7.183 kN
H2	=	13.601 kN
H3	=	23.937 kN
H4	=	27.934 kN
H5	=	78.770 kN

moment at beam support

span1	=	76.680 kN.m
span2	=	68.515 kN.m
span3	=	187.019 kN.m
span4	=	111.178 kN.m

moment at column

column1	=	10.775 kN.m
column2	=	20.402 kN.m
column3	=	35.906 kN.m
column4	=	41.900 kN.m
column5	=	118.155 kN.m

level 8

lateral force		40.40 kN
N1		3.03 P
N2		2.35 P
N3		2.01 P
N4		1.34 P
N5		1 P

axial load on column

taking moment at A =0

P	=	14.14767 kN
N1	=	42.86745 kN
N2	=	33.24703 kN
N3	=	28.43683 kN
N4	=	18.95788 kN
N5	=	14.14767 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	42.867 kN
F2	=	76.114 kN
F3	=	104.551 kN
F4	=	123.509 kN
F5	=	137.657 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	8.141 kN
H2	=	15.416 kN
H3	=	27.130 kN
H4	=	31.660 kN
H5	=	89.278 kN

moment at beam support

span1	=	99.667 kN.m
span2	=	89.054 kN.m
span3	=	243.082 kN.m
span4	=	144.506 kN.m

moment at column

column1	=	12.212 kN.m
column2	=	23.123 kN.m
column3	=	40.696 kN.m
column4	=	47.490 kN.m
column5	=	133.917 kN.m

level 7

lateral force	39.41 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P	=	17.81364 kN

N1	=	53.97532 kN
N2	=	41.86205 kN
N3	=	35.80541 kN
N4	=	23.87027 kN
N5	=	17.81364 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	53.975 kN
F2	=	95.837 kN
F3	=	131.643 kN
F4	=	155.513 kN
F5	=	173.327 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	9.076 kN
H2	=	17.185 kN
H3	=	30.245 kN
H4	=	35.295 kN
H5	=	99.528 kN

moment at beam support

span1	=	125.493 kN.m
span2	=	112.130 kN.m
span3	=	306.069 kN.m
span4	=	181.950 kN.m

moment at column

column1	=	13.614 kN.m
column2	=	25.778 kN.m
column3	=	45.368 kN.m
column4	=	52.942 kN.m
column5	=	149.293 kN.m

level 6

lateral force	38.61 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P	=	21.87361 kN

N1	=	66.27705 kN
N2	=	51.40299 kN
N3	=	43.96596 kN
N4	=	29.31064 kN
N5	=	21.87361 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	66.277 kN
F2	=	117.680 kN
F3	=	161.646 kN
F4	=	190.957 kN
F5	=	212.830 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	9.992 kN
H2	=	18.919 kN
H3	=	33.297 kN
H4	=	38.856 kN
H5	=	109.571 kN

moment at beam support

span1	=	154.094 kN.m
span2	=	137.686 kN.m
span3	=	375.827 kN.m
span4	=	223.419 kN.m

moment at column

column1	=	14.988 kN.m
column2	=	28.379 kN.m
column3	=	49.946 kN.m
column4	=	58.284 kN.m
column5	=	164.356 kN.m

level 5

lateral force 37.42 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P = 26.31755 kN

N1	=	79.74219 kN
N2	=	61.84625 kN
N3	=	52.89828 kN
N4	=	35.26552 kN
N5	=	26.31755 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	79.742 kN
F2	=	141.588 kN
F3	=	194.487 kN
F4	=	229.752 kN
F5	=	256.070 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	10.879 kN
H2	=	20.600 kN
H3	=	36.255 kN
H4	=	42.308 kN
H5	=	119.303 kN

moment at beam support

span1	=	185.401 kN.m
span2	=	165.658 kN.m
span3	=	452.182 kN.m
span4	=	268.810 kN.m

moment at column

column1	=	16.319 kN.m
column2	=	30.900 kN.m
column3	=	54.382 kN.m
column4	=	63.461 kN.m
column5	=	178.955 kN.m

level 4

lateral force 36.43 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P = 31.13445 kN

N1	=	94.33738 kN
N2	=	73.16595 kN
N3	=	62.58024 kN
N4	=	41.72016 kN
N5	=	31.13445 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	94.337 kN
F2	=	167.503 kN
F3	=	230.084 kN
F4	=	271.804 kN
F5	=	302.938 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	11.743 kN
H2	=	22.236 kN
H3	=	39.134 kN
H4	=	45.668 kN
H5	=	128.779 kN

moment at beam support

span1	=	219.334 kN.m
span2	=	195.979 kN.m
span3	=	534.944 kN.m
span4	=	318.010 kN.m

moment at column

column1	=	17.615 kN.m
column2	=	33.354 kN.m
column3	=	58.701 kN.m
column4	=	68.502 kN.m
column5	=	193.168 kN.m

level 3

lateral force	34.84 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P	=	36.31127 kN

N1	=	110.0231 kN
N2	=	85.33148 kN
N3	=	72.98565 kN
N4	=	48.6571 kN
N5	=	36.31127 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	110.023 kN
F2	=	195.355 kN
F3	=	268.340 kN
F4	=	316.997 kN
F5	=	353.309 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	12.570 kN
H2	=	23.801 kN
H3	=	41.888 kN
H4	=	48.881 kN
H5	=	137.841 kN

moment at beam support

span1	=	255.804 kN.m
span2	=	228.565 kN.m
span3	=	623.891 kN.m
span4	=	370.887 kN.m

moment at column

column1	=	18.854 kN.m
column2	=	35.701 kN.m
column3	=	62.832 kN.m
column4	=	73.322 kN.m
column5	=	206.761 kN.m

level 2

lateral force	33.05 kN

N1	3.03 P
N2	2.35 P
N3	2.01 P
N4	1.34 P
N5	1 P

axial load on column

taking moment at A =0

P	=	41.83094 kN

N1	=	126.7478 kN
N2	=	98.30272 kN
N3	=	84.0802 kN
N4	=	56.05346 kN
N5	=	41.83094 kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	126.748 kN
F2	=	225.050 kN
F3	=	309.131 kN
F4	=	365.184 kN
F5	=	407.015 kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	13.354 kN
H2	=	25.285 kN
H3	=	44.500 kN
H4	=	51.930 kN
H5	=	146.437 kN

moment at beam support

span1	=	294.689 kN.m
span2	=	263.309 kN.m
span3	=	718.729 kN.m
span4	=	427.265 kN.m

moment at column

column1	=	20.030 kN.m
column2	=	37.928 kN.m
column3	=	66.750 kN.m
column4	=	77.894 kN.m
column5	=	219.655 kN.m

level 1

lateral force	32.06	kN
N1	3.03	P
N2	2.35	P
N3	2.01	P
N4	1.34	P
N5	1	P

axial load on column

taking moment at A =0

P	=	47.66429	kN
N1	=	144.4228	kN
N2	=	112.0111	kN
N3	=	95.80523	kN
N4	=	63.87015	kN
N5	=	47.66429	kN

vertical shear on beam

for each part of subframe, sum(F) = 0

F1	=	144.423	kN
F2	=	256.434	kN
F3	=	352.239	kN
F4	=	416.109	kN
F5	=	463.774	kN

horizontal shearing force at column

taking moment at counterflexure at each beam = 0

H1	=	14.043	kN
H2	=	26.590	kN
H3	=	46.797	kN
H4	=	54.610	kN
H5	=	153.996	kN

moment at beam support

span1	=	335.783	kN.m
span2	=	300.028	kN.m
span3	=	818.956	kN.m
span4	=	486.848	kN.m

moment at column

column1	=	21.064	kN.m
column2	=	39.885	kN.m
column3	=	70.196	kN.m
column4	=	81.915	kN.m
column5	=	230.993	kN.m

drift calculation

1. due to girder/beam

δig = Qihi2/12EΣ(Ig/L)i

beam size

b	=	250	mm
h	=	450	mm

modulus of elasticity

E	=	2.00E+07

moment of inertia

Ig	=	2.278E+10 mm^4	0.022781
	=	0.0227813 m^4	

height of each floor level

hi	=	3	m

lateral force due to wind

Qi	=	q*A
	=	q*6.89*hi

level	q	Qi (kN)	Qi (kN)	L1	L2	L3	L4	L5	L6	L7	L8	L9	Ig/L1	Ig/L2	Ig/L3	Ig/L4	Ig/L5	Ig/L6	Ig/L7	Ig/L8	Ig/L9	Σ(Ig/L)	δig (mm)
15	1.09	44.96729	44.967293	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.0280
14	1.08	44.57001	89.537305	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.0558
13	1.06	43.97409	133.51139	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.0832
12	1.05	43.37817	176.88956	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.1102
11	1.03	42.78225	219.67181	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.1369
10	1.02	41.98768	261.65949	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.1631
9	1.00	41.19312	302.85261	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.1887
8	0.98	40.39856	343.25117	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.2139
7	0.95	39.40535	382.65652	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.2385
6	0.93	38.61079	421.26731	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.2625
5	0.91	37.41895	458.68626	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.2859
4	0.88	36.42574	495.112	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.3086
3	0.84	34.83662	529.94861	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.3303
2	0.80	33.04885	562.99746	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.3509
1	0.78	32.05565	595.05311	4.655	2.34	4.655	2.34	13.78	2.34	4.655	2.34	4.655	0.0049	0.0097	0.0049	0.0097	0.0017	0.0097	0.0049	0.0097	0.0049	0.0602	0.3708
																					Σδig		**3.1274**

2. due to column

δig = Qihi2/12EΣ(Ic/h)i

column size

b	=	500 mm
h	=	500 mm

modulus of elasticity

E	=	2.00E+07

moment of inertia

Ic	=	6.25E+10 mm^4
	=	0.0625 m^4

height of each floor level

hi	=	3 m

lateral force due to wind

Qi	=	q*A
	=	q*13.78*hi

level	q	Qi (kN)	Qi (kN)	Ic/h	nos. column	Σ(Ic/h)i	δig (mm)
15	1.09	44.96729	44.967293	0.0208	10	0.208333	0.0081
14	1.08	44.57001	89.537305	0.0208	10	0.208333	0.0161
13	1.06	43.97409	133.51139	0.0208	10	0.208333	0.0240
12	1.05	43.37817	176.88956	0.0208	10	0.208333	0.0318
11	1.03	42.78225	219.67181	0.0208	10	0.208333	0.0395
10	1.02	41.98768	261.65949	0.0208	10	0.208333	0.0471
9	1.00	41.19312	302.85261	0.0208	10	0.208333	0.0545
8	0.98	40.39856	343.25117	0.0208	10	0.208333	0.0618
7	0.95	39.40535	382.65652	0.0208	10	0.208333	0.0689
6	0.93	38.61079	421.26731	0.0208	10	0.208333	0.0758
5	0.91	37.41895	458.68626	0.0208	10	0.208333	0.0826
4	0.88	36.42574	495.112	0.0208	10	0.208333	0.0891
3	0.84	34.83662	529.94861	0.0208	10	0.208333	0.0954
2	0.80	33.04885	562.99746	0.0208	10	0.208333	0.1013
1	0.78	32.05565	595.05311	0.0208	10	0.208333	0.1071
							0.9033

3. due to overall bending

δif = hiAi

column size

b	=	500 mm
h	=	500 mm

column area

A	=	250000 mm
	=	0.25 sq.m

second moment of inertia

I = Σ(A*c^2)

level	c1	c2	c3	c4	c5	Ic=ΣAc²
15	6.89	9.23	13.86	16.23	20.88	255.963
14	6.89	9.23	13.86	16.23	20.88	255.963
13	6.89	9.23	13.86	16.23	20.88	255.963
12	6.89	9.23	13.86	16.23	20.88	255.963
11	6.89	9.23	13.86	16.23	20.88	255.963
10	6.89	9.23	13.86	16.23	20.88	255.963
9	6.89	9.23	13.86	16.23	20.88	255.963
8	6.89	9.23	13.86	16.23	20.88	255.963
7	6.89	9.23	13.86	16.23	20.88	255.963
6	6.89	9.23	13.86	16.23	20.88	255.963
5	6.89	9.23	13.86	16.23	20.88	255.963
4	6.89	9.23	13.86	16.23	20.88	255.963
3	6.89	9.23	13.86	16.23	20.88	255.963
2	6.89	9.23	13.86	16.23	20.88	255.963
1	6.89	9.23	13.86	16.23	20.88	255.963

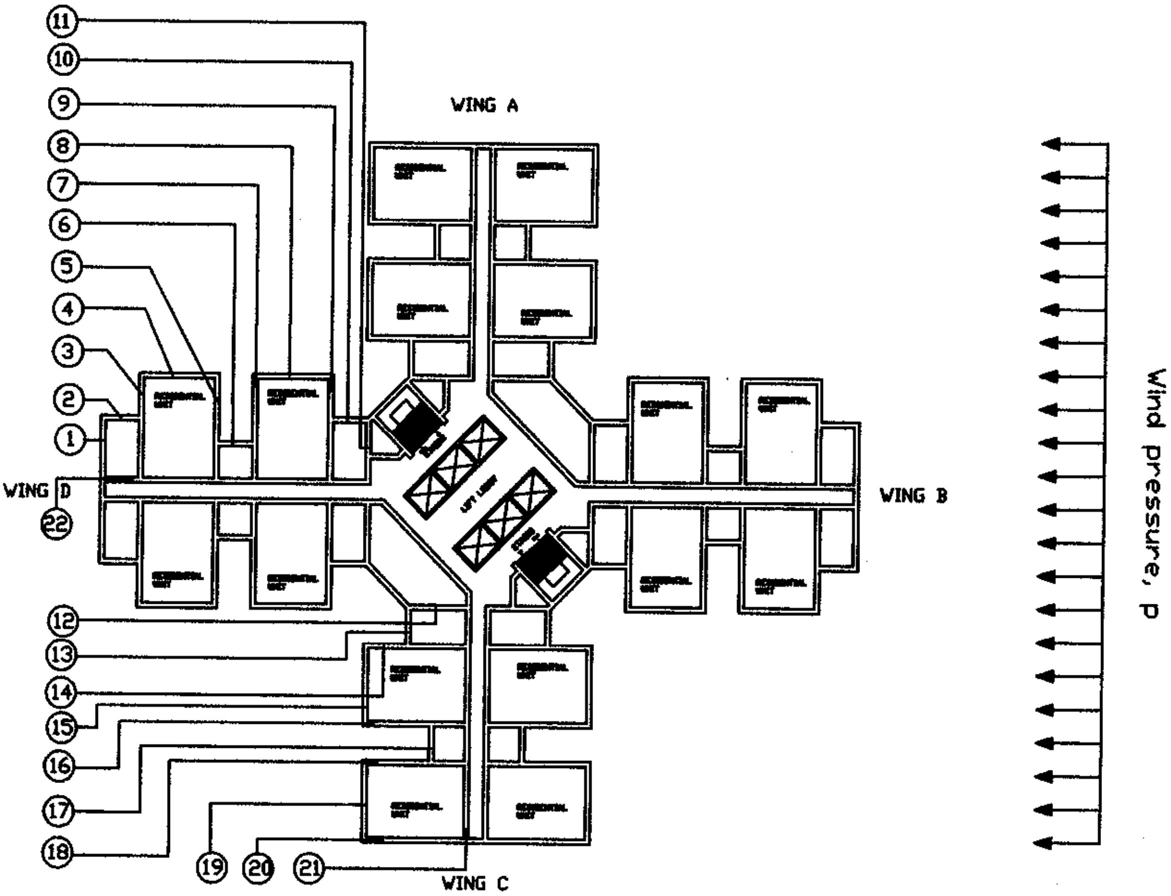
Lev	q	ΣQi (kN)	hi (m)	Mi	Ic	Mi/EIc	θi	δif (mm)
15	1.09	44.96729	3	0.00	255.963	0	7.42E-06	0.0223
14	1.08	44.57001	3	134.90	255.963	2.64E-08	8.02E-06	0.0240
13	1.06	43.97409	3	268.61	255.963	5.25E-08	8.6E-06	0.0258
12	1.05	43.37817	3	400.53	255.963	7.82E-08	9.18E-06	0.0276
11	1.03	42.78225	3	530.67	255.963	1.04E-07	9.76E-06	0.0293
10	1.02	41.98768	3	659.02	255.963	1.29E-07	1.03E-05	0.0310
9	1.00	41.19312	3	784.98	255.963	1.53E-07	1.09E-05	0.0326
8	0.98	40.39856	3	908.56	255.963	1.77E-07	1.14E-05	0.0343
7	0.95	39.40535	3	1029.75	255.963	2.01E-07	1.19E-05	0.0358
6	0.93	38.61079	3	1147.97	255.963	2.24E-07	1.25E-05	0.0374
5	0.91	37.41895	3	1263.80	255.963	2.47E-07	1.3E-05	0.0389
4	0.88	36.42574	3	1376.06	255.963	2.69E-07	1.35E-05	0.0404
3	0.84	34.83662	3	1485.34	255.963	2.9E-07	1.4E-05	0.0419
2	0.80	33.04885	3	1589.85	255.963	3.11E-07	1.44E-05	0.0432
1	0.78	32.05565	3	1688.99		3.3E-07		
								0.4645

4. total drift

level	[illegible] (mm)	[illegible] (mm)	[illegible] (mm)	[illegible] (mm)
15	0.0280	0.0081	0.0223	4.4951
14	0.0558	0.0161	0.0240	4.4367
13	0.0832	0.0240	0.0258	4.3407
12	0.1102	0.0318	0.0276	4.2077
11	0.1369	0.0395	0.0293	4.0380
10	0.1631	0.0471	0.0310	3.8323
9	0.1887	0.0545	0.0326	3.5912
8	0.2139	0.0618	0.0343	3.3153
7	0.2385	0.0689	0.0358	3.0054
6	0.2625	0.0758	0.0374	2.6622
5	0.2859	0.0826	0.0389	2.2864
4	0.3086	0.0891	0.0404	1.8790
3	0.3303	0.0954	0.0419	1.4409
2	0.3509	0.1013	0.0432	0.9734
1	0.3708	0.1071	0.0000	0.4780

APPENDIX C2

WIND LOADING ANALYSIS FOR LOAD BEARING WALL SYSTEM



WIND COMMING FROM B DIRECTION AND THE CORRESPONDING WALLS RESIST THE LOAD

wind load analysis (direction B)

relative stiffness

wall thickness 0.16 m

wall	length	relative stiffness	nos. wall	n x K	
20	13.78	418.6659443	2	837.3319	27.56
18	6.29	39.81731024	4	159.2692	25.16
16	6.29	39.81731024	4	159.2692	25.16
14	6.29	39.81731024	4	159.2692	25.16
12	3.71	8.17036976	4	32.68148	14.84
10	2.34	2.05006464	4	8.200259	9.36
8	4.65	16.08714	4	64.34856	18.6
6	2.34	2.05006464	4	8.200259	9.36
4	4.65	16.08714	4	64.34856	18.6
2	2.34	2.05006464	4	8.200259	9.36
22	16.33	696.7525019	4	2787.01	65.32
sum K				4288.129	248.48

force due to wind pressure

height of each floor 3 m

length longitudinal to the wind direction 41.76 m

area subjected to wind 375.84 sq.m

static force,F q x Area

floor level	q (kN/sq.m)	F (kN)
13-15	1.088	408.817
10-12	1.049	394.370
7-9	0.996	374.505
4-6	0.934	351.028
1-3	0.843	316.715

force distribution

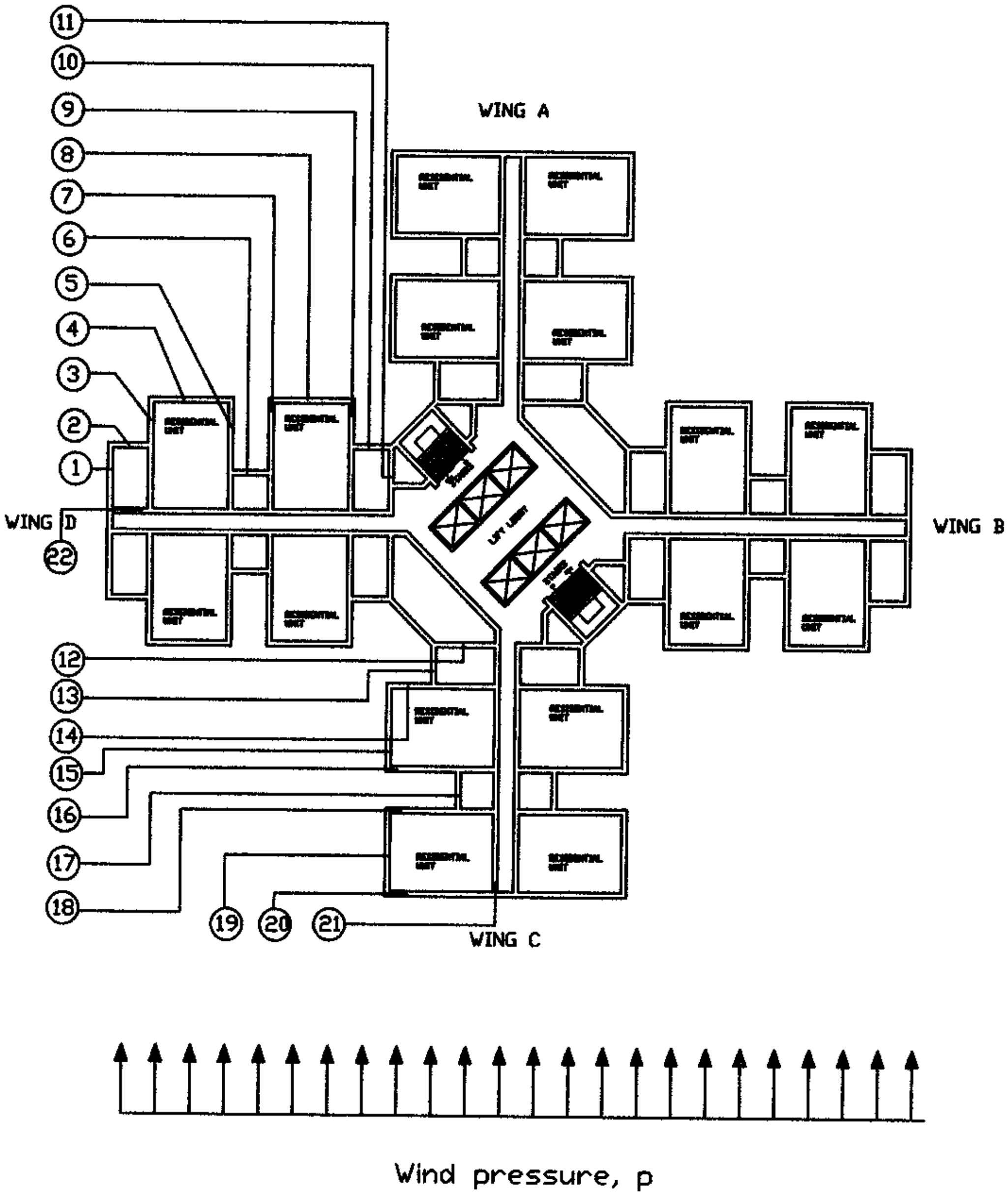
F(distr) = F*K

wall	F(distr) by level (kN)					avg
	13-15	10-12	7-9	4-6	1-3	
20	39.914	38.504	36.564	34.272	30.922	36.035
18	3.796	3.662	3.477	3.259	2.941	3.427
16	3.796	3.662	3.477	3.259	2.941	3.427
14	3.796	3.662	3.477	3.259	2.941	3.427
12	0.779	0.751	0.714	0.669	0.603	0.703
10	0.195	0.189	0.179	0.168	0.151	0.176
8	1.534	1.479	1.405	1.317	1.188	1.385
6	0.195	0.189	0.179	0.168	0.151	0.176
4	1.534	1.479	1.405	1.317	1.188	1.385
2	0.195	0.189	0.179	0.168	0.151	0.176
22	66.426	64.079	60.851	57.036	51.461	59.971

moment at the base

M = F(distr) * distance to base

wall	moment due to force at level					sum
	13-15	10-12	7-9	4-6	1-3	
20	1616.53	1212.87	822.70	462.67	139.15	4253.92
18	153.74	115.35	78.24	44.00	13.23	404.57
16	153.74	115.35	78.24	44.00	13.23	404.57
14	153.74	115.35	78.24	44.00	13.23	404.57
12	31.55	23.67	16.06	9.03	2.72	83.02
10	7.92	5.94	4.03	2.27	0.68	20.83
8	62.11	46.60	31.61	17.78	5.35	163.46
6	7.92	5.94	4.03	2.27	0.68	20.83
4	62.11	46.60	31.61	17.78	5.35	163.46
2	7.92	5.94	4.03	2.27	0.68	20.83
22	2690.26	2018.48	1369.15	769.99	231.58	7079.46



WIND COMMING FROM C DIRECTION AND THE CORRESPONDING WALL RESIST THE LOAD

wind load analysis (direction C)

relative stiffness

wall thickness 0.16 m

wall	length	relative stiffness	nos. wall	n x K
1	8.62	102.4806285	2	204.9613
3	6.29	39.81731024	4	159.2692
5	6.29	39.81731024	4	159.2692
7	6.29	39.81731024	4	159.2692
9	6.29	39.81731024	4	159.2692
11	3.71	8.17036976	4	32.68148
13	2.34	2.05006464	4	8.200259
15	4.65	16.08714	4	64.34856
17	2.34	2.05006464	4	8.200259
19	4.65	16.08714	4	64.34856
21	13.99	438.0998718	4	1752.399
sum K				2772.217

force due to wind pressure

height of each floor 3 m
length longitudinal to the wind direction 41.76 m
area subjected to wind 375.84 sq.m

static force,F q x Area

floor level	q (kN/sq.m)	F (kN)
13-15	1.088	408.817
10-12	1.049	394.370
7-9	0.996	374.505
4-6	0.934	351.028
1-3	0.843	316.715

force distribution

F(distr) = F*K

wall	F (distr) by level (kN)					
	13-15	10-12	7-9	4-6	1-3	average
1	15.113	14.579	13.844	12.976	11.708	13.644
3	5.872	5.664	5.379	5.042	4.549	5.301
5	5.872	5.664	5.379	5.042	4.549	5.301
7	5.872	5.664	5.379	5.042	4.549	5.301
9	5.872	5.664	5.379	5.042	4.549	5.301
11	1.205	1.162	1.104	1.035	0.933	1.088
13	0.302	0.292	0.277	0.260	0.234	0.273
15	2.372	2.289	2.173	2.037	1.838	2.142
17	0.302	0.292	0.277	0.260	0.234	0.273
19	2.372	2.289	2.173	2.037	1.838	2.142
21	64.606	62.323	59.184	55.474	50.051	58.328

moment at the base

M = F(distr) * distance to base

wall	moment due to force at level					
	13-15	10-12	7-9	4-6	1-3	sum
1	612.07	459.23	311.50	175.18	52.69	1610.66
3	237.81	178.43	121.03	68.06	20.47	625.80
5	237.81	178.43	121.03	68.06	20.47	625.80
7	237.81	178.43	121.03	68.06	20.47	625.80
9	237.81	178.43	121.03	68.06	20.47	625.80
11	48.80	36.61	24.83	13.97	4.20	128.41
13	12.24	9.19	6.23	3.50	1.05	32.22
15	96.08	72.09	48.90	27.50	8.27	252.84
17	12.24	9.19	6.23	3.50	1.05	32.22
19	96.08	72.09	48.90	27.50	8.27	252.84
21	2616.56	1963.18	1331.64	748.89	225.23	6885.50

LOAD BEARING WALL SYSTEM

deflection check (due to wind)

wall thickness 0.16 m
total height 45 m

direction b

wall	length (m)	average point load for 3 storey (kN)	udl (kN/m)	I (m4)	Δmax (mm)
20	13.78	36.04	4.004	34.89	2.94
18	6.29	3.43	0.381	3.32	2.94
16	6.29	3.43	0.381	3.32	2.94
14	6.29	3.43	0.381	3.32	2.94
12	3.71	0.70	0.078	0.68	2.94
10	2.34	0.18	0.020	0.17	2.94
8	4.65	1.38	0.154	1.34	2.94
6	2.34	0.18	0.020	0.17	2.94
4	4.65	1.38	0.154	1.34	2.94
2	2.34	0.18	0.020	0.17	2.94
22	16.33	59.97	6.663	58.06	2.94

direction c

wall	length (m)	average point load for 3 storey (kN)	udl (kN/m)	I (m4)	Δmax (mm)
1	8.62	13.64	1.52	8.54	4.55
3	6.29	5.30	0.59	3.32	4.55
5	6.29	5.30	0.59	3.32	4.55
7	6.29	5.30	0.59	3.32	4.55
9	6.29	5.30	0.59	3.32	4.55
11	3.71	1.09	0.12	0.68	4.55
13	2.34	0.27	0.03	0.17	4.55
15	4.65	2.14	0.24	1.34	4.55
17	2.34	0.27	0.03	0.17	4.55
19	4.65	2.14	0.24	1.34	4.55
21	13.99	58.33	6.48	36.51	4.55

maximum allowable deflection 0.09 m
90 mm

APPENDIX D1

DESIGN OF STRUCTURAL FRAME SYSTEM

BEAM DESIGN

desing for bending

b	250	mm
d	400	mm
d'	50	
fcu	30	N/sq.mm
fy	460	N/sq.mm

beam	M (vertical) (kN.m)	M (horizontal) (kN.m)	total moment M (kN.m)	K=M/bd^2fcu	z=d(0.5+sqrt(0.25-K/0.9))	As=M/0.95fyz	provide	size (dia)	As prov
7	175.48		175.48	0.146	318.331	1261.475	3	25	1473
8	12.52		12.52	0.010	395.307	72.493	2	12	226
9	12.52		12.52	0.010	395.309	72.466	2	12	226
			0.00						
1 (outer span)	61.48		61.48	0.051	375.761	374.404	2	16	402
1 (internal support)	77.03		77.03	0.064	369.080	477.593	2	20	628
			0.00						
2 (outer span)	73.45		73.45	0.061	370.641	453.478	2	20	628
2 (internal support)	112		112.00	0.093	352.995	726.053	3	20	942
			0.00						
3 (outer span)	102.5		102.50	0.085	357.527	656.045	3	20	942
3 (internal support)	119.720		119.72	0.100	349.210	784.510	3	20	942
			0.00						
4 (outer span)	37.13	74.36	111.49	0.093	353.242	240.531	2	16	402
4 (internal support)	46.530	131.250	177.78	0.148	316.886	336.008	2	16	402

BEAM DESIGN

design for shear

b	250	mm
d	400	mm
fcu	30	N/sq.mm
fyv	250	N/sq.mm
0.8*sqrt(fcu)	4.3817805	
nominal links, Asv/sv	0.42	
spacing for R6	134.30	mm

beam	V (vertical)	V (wind)	V (total)	v=V/bd	As prov (bending)	100As/bd	vc (table)	Asv/sv	provide	Asv	sv	sv prov
7	150.95		150.95	1.510	1473	1.47	0.76	0.79	6	56.55	71.67	50
8	21.41		21.41	0.214	226	0.23	0.42	-0.22	6	56.55	-260.87	120
9	21.40		21.40	0.214	226	0.23	0.42	-0.22	6	56.55	-260.77	120
1 (outer support)	48.89		48.89	0.489	402	0.40	0.47	0.02	6	56.55	2842.39	120
1 (internal support)	73.38		73.38	0.734	628	0.63	0.57	0.17	6	56.55	327.97	120
2 (outer support)	53.42		53.42	0.534	628	0.63	0.57	-0.04	6	56.55	-1500.59	120
2 (internal support)	72.25		72.25	0.723	942	0.94	0.66	0.07	6	56.55	859.54	120
3 (outer support)	64.81		64.81	0.648	942	0.94	0.66	-0.01	6	56.55	-4514.39	120
3 (internal support)	113.07		113.07	1.131	942	0.94	0.66	0.50	6	56.55	114.13	100
4 (outer support)	29.53	74.33	103.86	0.295	402	0.40	0.47	-0.18	6	56.55	-307.51	120
4 (internal support)	44.32	74.33	118.65	0.443	402	0.40	0.7	-0.27	6	56.55	-209.19	120

column design

concrete fcu 30 N/sq.mm
reinforcement, fy 460 N/sq.mm
d/h 0.8

b 400
h 400

column 1/F

level	N (kN) (from beams)	self weight	N (total)	Asc	provide
15	50.93	0.48	51.41	-5248.85	4 T 12
14	101.86	0.96	102.82	-5104.44	4 T 12
13	152.79	1.44	154.23	-4960.03	4 T 12
12	203.72	1.92	205.64	-4815.62	4 T 12
11	254.65	2.40	257.05	-4671.21	4 T 12
10	305.58	2.88	308.46	-4526.80	4 T 12
9	356.51	3.36	359.87	-4382.39	4 T 12
8	407.44	3.84	411.28	-4237.98	4 T 12
7	458.37	4.32	462.69	-4093.57	4 T 12
6	509.30	4.80	514.10	-3949.16	4 T 12
5	560.23	5.28	565.51	-3804.75	4 T 12
4	611.16	5.76	616.92	-3660.34	4 T 12
3	662.09	6.24	668.33	-3515.93	4 T 12
2	713.02	6.72	719.74	-3371.52	4 T 12
1	763.95	7.20	771.15	-3227.11	4 T 12

column 1/G

level	N (kN)	self weight	N (total)	Asc	provide
15	46.62	0.48	47.10	-5260.96	4 T 12
14	93.24	0.96	94.20	-5128.65	4 T 12
13	139.86	1.44	141.30	-4996.35	4 T 12
12	186.48	1.92	188.40	-4864.04	4 T 12
11	233.10	2.40	235.50	-4731.74	4 T 12
10	279.72	2.88	282.60	-4599.44	4 T 12
9	326.34	3.36	329.70	-4467.13	4 T 12
8	372.96	3.84	376.80	-4334.83	4 T 12
7	419.58	4.32	423.90	-4202.53	4 T 12
6	466.20	4.80	471.00	-4070.22	4 T 12
5	512.82	5.28	518.10	-3937.92	4 T 12
4	559.44	5.76	565.20	-3805.62	4 T 12
3	606.06	6.24	612.30	-3673.31	4 T 12
2	652.68	6.72	659.40	-3541.01	4 T 12
1	699.30	7.20	706.50	-3408.71	4 T 12

column 2/E

level	N (kN)	self weight	N (total)	Asc	provide
15	215.76	0.48	216.24	-4785.84	4 T 12
14	431.52	0.96	432.48	-4178.43	4 T 12
13	647.28	1.44	648.72	-3571.01	4 T 12
12	863.04	1.92	864.96	-2963.60	4 T 12
11	1078.80	2.40	1081.20	-2356.18	4 T 12
10	1294.56	2.88	1297.44	-1748.76	4 T 12
9	1510.32	3.36	1513.68	-1141.35	4 T 12
8	1726.08	3.84	1729.92	-533.93	4 T 12
7	1941.84	4.32	1946.16	73.48	4 T 12
6	2157.60	4.80	2162.40	680.90	4 T 16
5	2373.36	5.28	2378.64	1288.31	8 T 16
4	2589.12	5.76	2594.88	1895.73	8 T 20
3	2804.88	6.24	2811.12	2503.15	8 T 20
2	3020.64	6.72	3027.36	3110.56	8 T 25
1	3236.40	7.20	3243.60	3717.98	8 T 25

column 2/G

level	N (kN)	self weight	N (total)	Asc	provide
15	189.81	0.48	190.29	-4858.74	4 T 12
14	379.62	0.96	380.58	-4324.21	4 T 12
13	569.43	1.44	570.87	-3789.69	4 T 12
12	759.24	1.92	761.16	-3255.17	4 T 12
11	949.05	2.40	951.45	-2720.65	4 T 12
10	1138.86	2.88	1141.74	-2186.12	4 T 12
9	1328.67	3.36	1332.03	-1651.60	4 T 12
8	1518.48	3.84	1522.32	-1117.08	4 T 12
7	1708.29	4.32	1712.61	-582.56	4 T 12
6	1898.10	4.80	1902.90	-48.03	4 T 12
5	2087.91	5.28	2093.19	486.49	8 T 12
4	2277.72	5.76	2283.48	1021.01	8 T 16
3	2467.53	6.24	2473.77	1555.53	8 T 16
2	2657.34	6.72	2664.06	2090.06	8 T 20
1	2847.15	7.20	2854.35	2624.58	8 T 20

column 3/E

level	N (kN)	self weight	N (total)	Asc	provide
15	204.37	0.48	204.85	-4817.84	4 T 12
14	408.74	0.96	409.70	-4242.42	4 T 12
13	613.11	1.44	614.55	-3666.99	4 T 12
12	817.48	1.92	819.40	-3091.57	4 T 12
11	1021.85	2.40	1024.25	-2516.15	4 T 12
10	1226.22	2.88	1229.10	-1940.73	4 T 12
9	1430.59	3.36	1433.95	-1365.31	4 T 12
8	1634.96	3.84	1638.80	-789.89	4 T 12
7	1839.33	4.32	1843.65	-214.47	4 T 12
6	2043.70	4.80	2048.50	360.96	4 T 12
5	2248.07	5.28	2253.35	936.38	8 T 16
4	2452.44	5.76	2458.20	1511.80	8 T 16
3	2656.81	6.24	2663.05	2087.22	8 T 20
2	2861.18	6.72	2867.90	2662.64	8 T 25
1	3065.55	7.20	3072.75	3238.06	8 T 25

column 3/G

level	N (kN)	self weight	N (total)	Asc	provide
15	167.46	0.48	167.94	-4921.52	4 T 12
14	334.92	0.96	335.88	-4449.78	4 T 12
13	502.38	1.44	503.82	-3978.03	4 T 12
12	669.84	1.92	671.76	-3506.29	4 T 12
11	837.30	2.40	839.70	-3034.55	4 T 12
10	1004.76	2.88	1007.64	-2562.81	4 T 12
9	1172.22	3.36	1175.58	-2091.07	4 T 12
8	1339.68	3.84	1343.52	-1619.33	4 T 12
7	1507.14	4.32	1511.46	-1147.58	4 T 12
6	1674.60	4.80	1679.40	-675.84	4 T 12
5	1842.06	5.28	1847.34	-204.10	4 T 12
4	2009.52	5.76	2015.28	267.64	4 T 12
3	2176.98	6.24	2183.22	739.38	8 T 12
2	2344.44	6.72	2351.16	1211.12	8 T 16
1	2511.90	7.20	2519.10	1682.87	8 T 16

APPENDIX D2

DESIGN OF LOAD BEARING WALL SYSTEM

wall design using design chart

concrete fcu 30 N/sq.mm
reinforcement, fy 460 N/sq.mm

load combination

case 1 1.2(dead load + imposed load + wind)

wall	length (m)	thickness (m)	dead load (kN/m)	live load (kN/m)	N (kN)	N/bh (N/sq.mm)	wind (kN.m)	M (kN.m)	M/bh^2 (N/sq.mm)	100Asc/bh
1	8.62	0.16	230.47	36.05	2756.88	2.00	1610.66	1932.79	0.16	0.40
2	2.34	0.16	214.92	26.33	677.42	1.81	20.83	25.00	0.03	0.40
3	6.29	0.16	312.57	87.35	3018.61	3.00	625.80	750.96	0.12	0.40
4	4.65	0.16	256.68	52.43	1724.81	2.32	163.46	196.15	0.06	0.40
5	6.29	0.16	293.66	75.54	2786.70	2.77	625.80	750.96	0.12	0.40
6	2.34	0.16	214.94	26.34	677.50	1.81	20.83	25.00	0.03	0.40
7	6.29	0.16	293.66	75.54	2786.70	2.77	625.80	750.96	0.12	0.40
8	4.65	0.16	256.68	52.43	1724.81	2.32	163.46	196.15	0.06	0.40
9	6.29	0.16	312.57	87.35	3018.61	3.00	625.80	750.96	0.12	0.40
10	2.34	0.16	214.92	26.33	677.42	1.81	20.83	25.00	0.03	0.40
11	3.71	0.16	230.47	36.05	1186.55	2.00	128.41	154.09	0.07	0.40
12	3.71	0.16	230.47	230.47	2052.14	3.46	83.02	99.62	0.05	0.40
13	2.34	0.16	214.92	26.33	677.42	1.81	32.22	38.66	0.04	0.40
14	6.29	0.16	312.57	87.35	3018.61	3.00	404.57	485.48	0.08	0.40
15	4.65	0.16	256.68	52.43	1724.81	2.32	252.84	303.40	0.09	0.40
16	6.29	0.16	293.66	75.54	2786.70	2.77	404.57	485.48	0.08	0.40
17	2.34	0.16	214.94	26.34	677.50	1.81	32.22	38.66	0.04	0.40
18	6.29	0.16	293.66	75.54	2786.70	2.77	404.57	485.48	0.08	0.40
19	4.65	0.16	256.68	52.43	1724.81	2.32	252.84	303.40	0.09	0.40
20	13.78	0.16	278.55	66.09	5699.03	2.58	4253.92	5104.70	0.17	0.40
21	13.99	0.16	299.88	79.43	6367.77	2.84	6885.50	8262.60	0.26	0.40
22	16.33	0.16	299.88	79.43	7432.86	2.84	7079.46	8495.35	0.20	0.40

case 2 1.4(dead load + wind)

wall	length (m)	thickness (m)	dead load (kN/m)	live load (kN/m)	N (kN)	N/bh (N/sq.mm)	wind (kN.m)	M (kN.m)	M/bh^2 (N/sq.mm)	100Asc/bh
1	8.62	0.16	230.47	36.05	2781.36	2.02	1610.66	2254.93	0.19	0.40
2	2.34	0.16	214.92	26.33	704.08	1.88	20.83	29.16	0.03	0.40
3	6.29	0.16	312.57	87.35	2752.47	2.73	625.80	876.12	0.14	0.40
4	4.65	0.16	256.68	52.43	1670.99	2.25	163.46	228.84	0.07	0.40
5	6.29	0.16	293.66	75.54	2585.97	2.57	625.80	876.12	0.14	0.40
6	2.34	0.16	214.94	26.34	704.14	1.88	20.83	29.16	0.03	0.40
7	6.29	0.16	293.66	75.54	2585.97	2.57	625.80	876.12	0.14	0.40
8	4.65	0.16	256.68	52.43	1670.99	2.25	163.46	228.84	0.07	0.40
9	6.29	0.16	312.57	87.35	2752.47	2.73	625.80	876.12	0.14	0.40
10	2.34	0.16	214.92	26.33	704.08	1.88	20.83	29.16	0.03	0.40
11	3.71	0.16	230.47	36.05	1197.08	2.02	128.41	179.78	0.08	0.40
12	3.71	0.16	230.47	230.47	1197.08	2.02	83.02	116.22	0.05	0.40
13	2.34	0.16	214.92	26.33	704.08	1.88	32.22	45.11	0.05	0.40
14	6.29	0.16	312.57	87.35	2752.47	2.73	404.57	566.40	0.09	0.40
15	4.65	0.16	256.68	52.43	1670.99	2.25	252.84	353.97	0.10	0.40
16	6.29	0.16	293.66	75.54	2585.97	2.57	404.57	566.40	0.09	0.40
17	2.34	0.16	214.94	26.34	704.14	1.88	32.22	45.11	0.05	0.40
18	6.29	0.16	293.66	75.54	2585.97	2.57	404.57	566.40	0.09	0.40
19	4.65	0.16	256.68	52.43	1670.99	2.25	252.84	353.97	0.10	0.40
20	13.78	0.16	278.55	66.09	5373.79	2.44	4253.92	5955.49	0.20	0.40
21	13.99	0.16	299.88	79.43	5873.45	2.62	6885.50	9639.70	0.31	0.40
22	16.33	0.16	299.88	79.43	6855.86	2.62	7079.46	9911.25	0.23	0.40

case 3 1.0(dead load) + 1.4(wind)

wall	length (m)	thickness (m)	dead load (kN/m)	live load (kN/m)	N (kN)	N/bh (N/sq.mm)	wind (kN.m)	M (kN.m)	M/bh^2 (N/sq.mm)	100Asc/bh
1	8.62	0.16	230.47	36.05	1986.68	1.44	1610.66	2254.93	0.19	0.40
2	2.34	0.16	214.92	26.33	502.91	1.34	20.83	29.16	0.03	0.40
3	6.29	0.16	312.57	87.35	1966.05	1.95	625.80	876.12	0.14	0.40
4	4.65	0.16	256.68	52.43	1193.56	1.60	163.46	228.84	0.07	0.40
5	6.29	0.16	293.66	75.54	1847.12	1.84	625.80	876.12	0.14	0.40
6	2.34	0.16	214.94	26.34	502.96	1.34	20.83	29.16	0.03	0.40
7	6.29	0.16	293.66	75.54	1847.12	1.84	625.80	876.12	0.14	0.40
8	4.65	0.16	256.68	52.43	1193.56	1.60	163.46	228.84	0.07	0.40
9	6.29	0.16	312.57	87.35	1966.05	1.95	625.80	876.12	0.14	0.40
10	2.34	0.16	214.92	26.33	502.91	1.34	20.83	29.16	0.03	0.40
11	3.71	0.16	230.47	36.05	855.06	1.44	128.41	179.78	0.08	0.40
12	3.71	0.16	230.47	230.47	855.06	1.44	83.02	116.22	0.05	0.40
13	2.34	0.16	214.92	26.33	502.91	1.34	32.22	45.11	0.05	0.40
14	6.29	0.16	312.57	87.35	1966.05	1.95	404.57	566.40	0.09	0.40
15	4.65	0.16	256.68	52.43	1193.56	1.60	252.84	353.97	0.10	0.40
16	6.29	0.16	293.66	75.54	1847.12	1.84	404.57	566.40	0.09	0.40
17	2.34	0.16	214.94	26.34	502.96	1.34	32.22	45.11	0.05	0.40
18	6.29	0.16	293.66	75.54	1847.12	1.84	404.57	566.40	0.09	0.40
19	4.65	0.16	256.68	52.43	1193.56	1.60	252.84	353.97	0.10	0.40
20	13.78	0.16	278.55	66.09	3838.42	1.74	4253.92	5955.49	0.20	0.40
21	13.99	0.16	299.88	79.43	4195.32	1.87	6885.50	9639.70	0.31	0.40
22	16.33	0.16	299.88	79.43	4897.04	1.87	7079.46	9911.25	0.23	0.40

reinforcement summary

wall	length (m)	thickness (m)	case 1	case 2	case 3	max	As req (sq mm)	bar size (mm)	spacing (mm/m)	As prov (sq mm)
1	8.62	0.16	0.4	0.4	0.4	0.4	640	8	150	670
2	2.34	0.16	0.4	0.4	0.4	0.4	640	8	150	670
3	6.29	0.16	0.4	0.4	0.4	0.4	640	8	150	670
4	4.65	0.16	0.4	0.4	0.4	0.4	640	8	150	670
5	6.29	0.16	0.4	0.4	0.4	0.4	640	8	150	670
6	2.34	0.16	0.4	0.4	0.4	0.4	640	8	150	670
7	6.29	0.16	0.4	0.4	0.4	0.4	640	8	150	670
8	4.65	0.16	0.4	0.4	0.4	0.4	640	8	150	670
9	6.29	0.16	0.4	0.4	0.4	0.4	640	8	150	670
10	2.34	0.16	0.4	0.4	0.4	0.4	640	8	150	670
11	3.71	0.16	0.4	0.4	0.4	0.4	640	8	150	670
12	3.71	0.16	0.4	0.4	0.4	0.4	640	8	150	670
13	2.34	0.16	0.4	0.4	0.4	0.4	640	8	150	670
14	6.29	0.16	0.4	0.4	0.4	0.4	640	8	150	670
15	4.65	0.16	0.4	0.4	0.4	0.4	640	8	150	670
16	6.29	0.16	0.4	0.4	0.4	0.4	640	8	150	670
17	2.34	0.16	0.4	0.4	0.4	0.4	640	8	150	670
18	6.29	0.16	0.4	0.4	0.4	0.4	640	8	150	670
19	4.65	0.16	0.4	0.4	0.4	0.4	640	8	150	670
20	13.78	0.16	0.4	0.4	0.4	0.4	640	8	150	670
21	13.99	0.16	0.4	0.4	0.4	0.4	640	8	150	670
22	16.33	0.16	0.4	0.4	0.4	0.4	640	8	150	670

APPENDIX E

QUANTITY ESTIMATION

MATERIAL QUANTITY FOR FRAME SYSTEM (CONCRETE)

slab

slab size			volume	nos	total vol.
length	width	depth			
3.71	2.34	0.2	1.73628	12	20.83536
6.29	4.65	0.2	5.8497	16	93.5952
2.34	2.2	0.2	1.0296	8	8.2368
16.33	1.2	0.2	3.9192	2	7.8384
13.99	1.2	0.2	3.3576	2	6.7152

13.78	13.78	0.2	37.97768	1	37.97768
2.58	2.58	0.2	1.33128	2	2.66256
6.61	1.66	0.2	2.19452	2	4.38904
					30.92608

total 168.147 cu.m

total for 15 storey **840.7352** cu.m

beam

width 0.25 m
depth 0.45 m

	nos.	length	volume
1/F-G	12	3.31	4.469
2/E-G	32	5.89	21.204
1/G-H	22	0.8	1.980
F/1-2	40	2.04	9.180
E/2-3	32	4.255	15.318
E-F/6-6a	4	3.293	1.482
E-H/6-8	2	10.227	2.301
E-G/8-9	2	8.53	1.919

total volume of beam in every level 57.85268

total volume of concrete for beam **867.7901**

column

width 0.5 m
length 0.5 m

nos	length	volume
88	48	1056

1056

TOTAL **2764.525** cu.m

MATERIAL QUANTITY FOR FRAME SYSTEM (BRICK WALL)

walls

thickness	0.15 m
height	3 m

window size

length	2 m
depth	1 m

	nos.	length	window/door	area
1/F-G	4	4.01	1	40.12
2/E-G	32	6.59	1	568.64
6/F-G	8	4.01	0	80.24
1/G-H	4	0.9	0	2.80
F/1-2	20	2.04	0	82.40
E/2-3	16	4.354	1	176.99
G/1-6	4	16.63	2	191.56
7/A-E	4	14.29	2	163.48
E-F/6-6a	4	3.524	0	34.29
E-G/8-9	2	7.734	0	42.40
Stair	4	3.287	0	31.44
Stair2	4	2.136	0	17.63
area of walls at every level				1432.00

TOTAL **21480** s.m

frame system

1 weight due to live load

slab

slab size				
length	width	area	nos	total area
3.71	2.34	8.6814	12	104.1768
6.29	4.65	29.2485	16	467.976
2.34	2.2	5.148	8	41.184
16.33	1.2	19.596	2	39.192
13.99	1.2	16.788	2	33.576
13.78	13.78	189.8884	1	189.8884
2.58	2.58	6.6564	2	13.3128
6.61	1.66	10.9726	2	21.9452
				154.6304

total 840.7352 sq.m

total for 15 storey 4203.676 sq.m

area = 4203.676 sq.m
live load = 3 kN/sq.m

weight due to live load = **12611.03** kN

2. weight due to dead load

total volume of concrete = 2764.525 cu.m
unit weight of concrete = 24 kN/cu.m
weight due to concrete = 66348.61 kN

total area of brick wall = 21480 sq.m
wall thickness = 0.15 m
unit weight of brick = 22 kN/cu.m
weight due to brick wall = 70884 kN

total weight due to dead load = **137232.6** kN

3. total overall weight

total overall weight = **149843.6** kN

MATERIAL QUANTITY FOR LOAD BEARING WALL SYSTEM (CONCRETE)

slab

slab size			volume	nos	total vol.
length	width	depth			
3.71	2.34	0.2	1.73628	12	20.83536
6.29	4.65	0.2	5.8497	16	93.5952
2.34	2.2	0.2	1.0296	8	8.2368
16.33	1.2	0.2	3.9192	2	7.8384
13.99	1.2	0.2	3.3576	2	6.7152

13.78	13.78	0.2	37.97768	1	37.97768
2.58	2.58	0.2	1.33128	2	2.66256
6.61	1.66	0.2	2.19452	2	4.38904
					30.92608

total 168.147 cu.m

total for 15 storey **840.7352** cu.m

wall

thickness 0.16 m
height 3 m

window size
length 2 m
depth 1 m

	nos.	length	window/do	volume
1/F-G	4	4.01	1	6.4192
2/E-G	32	6.59	1	90.9824
6/F-G	8	4.01	0	15.3984
1/G-H	4	0.9	0	1.728
F/1-2	20	2.04	0	19.584
E/2-3	16	4.354	1	28.31872
G/1-6	4	16.63	2	29.3696
7/A-E	4	14.29	2	24.8768
				0
E-F/6-6a	4	3.524	0	6.76608
E-G/8-9	2	7.734	0	7.42464
Stair	4	3.287	0	6.31104
Stair2	4	2.136	0	4.10112

volume of concrete for walls at every level 241.28 cu.m

total volume of concrete for walls **3619.2** cu.m

TOTAL **4459.935** cu.m

load bearing wall system

1 weight due to live load

slab

slab size		area	nos	total area
length	width			
3.71	2.34	8.6814	12	104.1768
6.29	4.65	29.2485	16	467.976
2.34	2.2	5.148	8	41.184
16.33	1.2	19.596	2	39.192
13.99	1.2	16.788	2	33.576
13.78	13.78	189.8884	1	189.8884
2.58	2.58	6.6564	2	13.3128
6.61	1.66	10.9726	2	21.9452
				154.6304

total 840.7352 sq.m

total for 15 storey 4203.676 sq.m

area = 4203.676 sq.m
live load = 3 kN/sq.m

weight due to live load = **12611.03** kN

2. weight due to dead load

total volume of concrete = 4459.935 cu.m
unit weight of concrete = 24 kN/cu.m

weight due to concrete = **107038.4** kN

3. total overall weight

total overall weight = **119649.5** kN