UNIVERSITI
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To investigate the effective structural system for high-rise building design of height between 200m to 250m and subjected to wind conditions in KL city centre

by

Lim Ming Seong

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

December 2004

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CERTIFICATION OF APPROVAL

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Approved by,

AP Nasir Shafiq

UNIVERSITI TEKNOLOGI PETRONAS
TRONOH, PERAK
DEC 2004

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 acknowledgement, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

Limming Seong

LIM MING SEONG

ABSTRACT

The design and construction techniques of tall building structure have been evolved and improving throughout the past. In today's modern world, improvement on construction materials, development of new structural systems and the usage of structural software have all significantly contributed to the effectiveness of tall building design.

This analytical study was carried out to investigate the influence of various parameters on the design of tall building structures of 200 to 250 m height. The influencing parameters include the effects of various concrete grades, such as grade 50, 60 and 80 as well as effectiveness of the structural system based on rigid frame system and the shear wall system. The analysis was carried out using structural software STAAD.PRO 2002.

Based on analytical and structural design results as well as obtained from computer analysis, higher concrete grade resulted in appreciable reduction in the vertical member sizes such as column dimensions and the shear wall thickness. Although analytical values of maximum drift at the roof level was obtained within permissible limit for rigid frame, however shear wall structural system has shown a lower value of the maximum drift at the roof level. In studying the shear wall system, three different configurations and arrangement of walls were considered, which also affected the maximum drift.

ACKNOWLEDGEMENT

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The growth of modern tall building construction, began in the 1880s, which was been largely for commercial and residential purposes. Tall commercial buildings are primarily a response to the demand by business activities and further to the need of more hotel accommodations due to tourism activities.

Significant advances have occurred from time to time with the advent of a new material, construction facilities and level of services. Multistory buildings were a feature in the ancient Rome: four storey wooden tenement buildings of post and lintel construction, were common. Those built after the great fire of Nero, however, used the new brick and concrete materials in the form of arch and barrel vault structures. Through the following centuries, the two basic construction materials were timber and masonry. With the rapidly increasing number of masonry high-rise building in North America toward the end of the nineteenth century, the limits of this form of construction became apparent in 1891 in the 16-story Monadnock Building in Chicago. It was the last building in the city for which massive load-bearing masonry walls were employed.

A major technical innovation had occurred in the middle of nineteenth century: the development of higher strength and structurally more efficient materials such as wrought iron and subsequently steel. The new materials allowed the development of lightweight skeletal structures, permitting building of greater height and with larger interior open spaces and windows. The first high-rise building totally supported by a metal frame was the 11-story home Insurance Building built in Chicago in 1883, followed by the construction of steel frame 9-story Rand-McNally Building built in 1989. Two years later, in the same city, diagonal bracings were introduced in the façade frames of the 20-story Masonic Temple to form vertical trusses, the forerunner of modern shear wall and braced frame construction. It was by then appreciated that at that height wind forces were an important design consideration. Improved design methods and construction techniques allowed the maximum height of steel-frame structures to increase steadily, reaching the height of 60 stories with the construction of the Woolworth Building in New York in 1913. This golden age of American skyscraper construction culminated in 1931 in its crowning glory, the Empire State Building, whose 102-story braced steel frame reached a height of 1250 ft (381m).

Reinforced concrete construction have been introduced for multistory buildings at the end of World War I. Progress in reinforced concrete was slow and intermittent, at the time the steel framed Empire State Building was completed, the tallest concrete building, the Exchange Building in Seattle, has attained a height of only 23 stories. Rather than bringing significant increase in height, however, these modern developments comprised new structural systems, improved material qualities and services, and better design and construction techniques. In 1973, the twin towers of the 110-story, 1350 ft (412 m) high World Trade Center in New York was completed followed by the 1450 ft (442 m) high bundled-tube Sears Tower in Chicago in 1974.
Since then, different structural systems have gradually evolved for residential and office buildings, reflecting their differing functional requirements.

1.2 PROBLEM STATEMENT

In business-oriented metropolis like Kuala Lumpur, construction of high-rise buildings of 200m to 250m height is very common. In order to meet the architectural and client requirements various consultant opted the structural system according to their experience and approach. This study was focused on the proposal of the most effective structural system for 200m to 250m high buildings. The main parameters as considered were stability, human comfort, robustness, space available, speed of construction and cost effectiveness. The building was supposed to be subjected to moderate wind pressure.

1.3 OBJECTIVE AND SCOPE OF STUDY

The main objectives of this study were:

1) To determine the effective and robust structural system for 200 to 250 m high buildings, which must satisfactorily comply with the requirement of maximum drift and human comfort.
2) To meet the above objective, the following variables were included in the computational based analytical study
    i) Effectiveness of structural system in between a rigid frame structure and shear wall structure
    ii) Influence of various concrete grades on each of the respective structural systems

CHAPTER 2

LITERATURE REVIEW

2.1 DEFINITION & CLASSIFICATION OF TALL BUILDING

The Council of Tall Buildings & Urban Habitat [1] definition of a tall building defines the unique nature of the high-rise project: "A building whose height creates different conditions in the design, construction, and use than those exist in common buildings of a certain region or period." For the practicing structural engineer, cataloging the structural systems for tall buildings has historically recognized the primary importance of the system to resist lateral loads.

In 1965 Fazlur Khan [1] recognized a hierarchy of system forms could roughly be categorized with respect to relative effectiveness in resisting lateral loads (Figure 2.1). At one end of the spectrum are the moment resisting frames, which are efficient for buildings in the range of 20 to 30 stories; at the other end is the generation of tubular systems with high cantilever efficiency. With the endpoints defined, other systems were placed with the idea that the application of any particular form is economically only over a limited range of building heights. The system charts were updated periodically as new systems were developed and improvement in materials and analysis techniques evolved.

In 1984, the Council of Tall Buildings & Urban Habitat [1] attempted to develop a rigorous methodology for the cataloging of tall buildings with respect to their structural systems. The classification scheme involves four distinct levels of framing-oriented division: primary framing system, bracing subsystem, floor framing, and configuration and load transfer. These levels are further broken down into subgroups and discrete systems (Figure 2.2). This format allows for the consistent and specific identification and documentation of tall buildings and their systems, the overriding goal being to achieve a comprehensive worldwide survey of the performance of buildings in the high-rise environment.

Figure 2.1: Comparison of Structural Systems

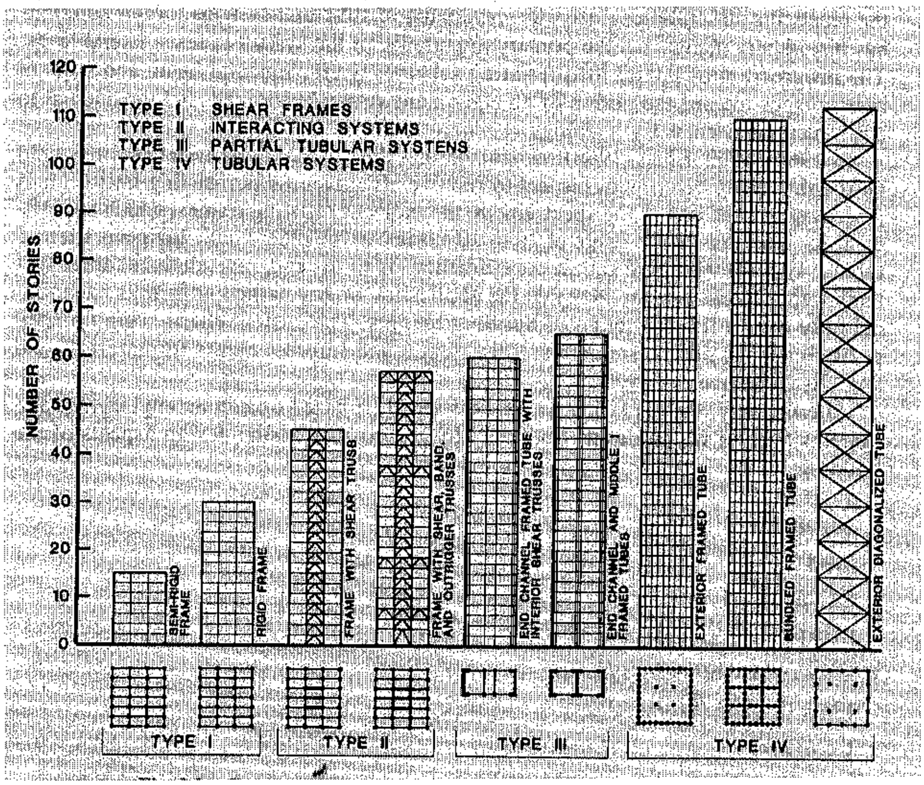
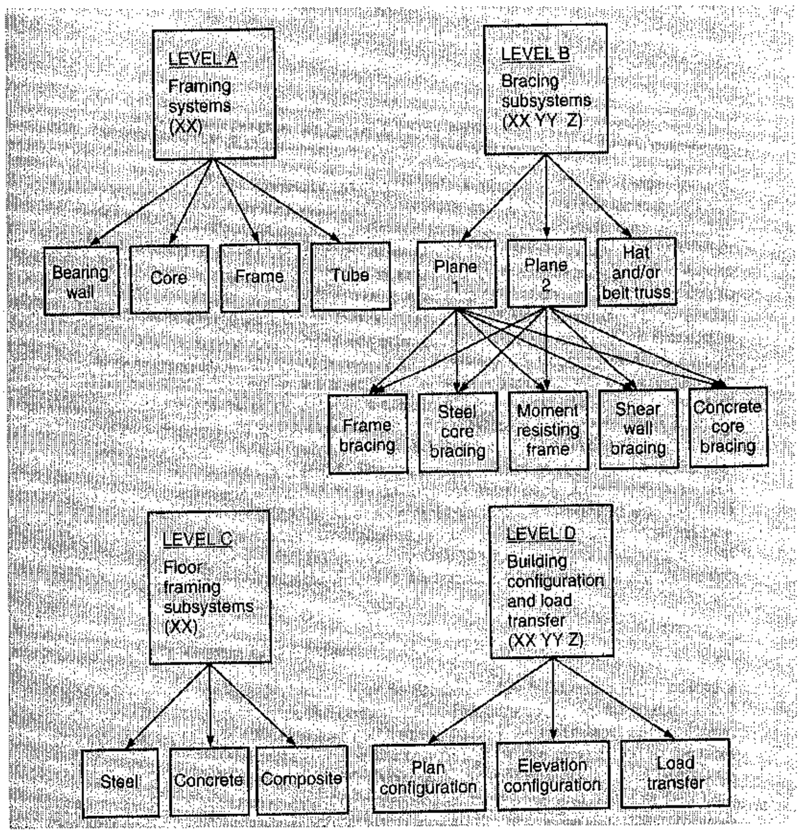


Figure 2.2: Classification of Structural Systems



2.2 DESIGN CRITERIA

2.2.1 Strength and Stability [2]

For the ultimate limit state, the prime design requirement is that the building structure should have adequate strength to resist, and to remain stable under, the worst probable load actions that may occur during the lifetime of the building, including the period of construction.

This requires an analysis of the forces and stresses that will occur in the members as a result of the most critical possible load combinations, including the augmented moments that may arise from second-order additional deflections (P-Delta effects). An adequate reserve of strength, using prescribed load factors, must be present. Particular attention must be paid to critical members, whose failure could prove catastrophic in initiating a progressive collapse of part of or the entire building. Any additional stresses caused by restrained differential movements due to creep, shrinkage, 'or temperature must be included.

In addition, a check must be made on the most fundamental condition of equilibrium, to establish that the applied lateral forces will not cause the entire building to topple as a rigid body about one edge of the base. Taking moments about that edge, the resisting moment of the dead weight of the building must be greater than the overturning moment for stability by an acceptable factor of safety.

2.2.2 Stiffness and Drift Limitations [2]

The provision of adequate stiffness, particularly lateral stiffness, is a major consideration in the design of a tall building for several important reasons. As far as the ultimate limit state is concerned, lateral deflections must be limited to prevent second-order P-Delta effects due to gravity loading being of such a magnitude as to precipitate collapse. In terms of the serviceability limit states, deflections must first be maintained at a sufficiently low level to allow the proper functioning of nonstructural components such as elevators and doors; second, to avoid distress in the structure, to prevent excessive cracking and consequent loss of stiffness, and to avoid any redistribution of load to non-load-bearing partitions, infills, cladding, or glazing; and third, the structure must be sufficiently stiff to prevent dynamic motions becoming large enough to cause discomfort to occupants, prevent delicate work being undertaken, or affect sensitive equipment. In fact, it is in the particular need for concern for the provision of lateral stiffness that the design of a high-rise building largely departs from that of a low-rise building.

One simple parameter that affords an estimate of the lateral stiffness of a building is the drift index, defined as the ratio of the maximum deflection at the top of the building to the total height. In addition, the corresponding value for a single, story height, the interstory drift index, gives a measure of possible localized excessive deformation. The control of lateral deflections is of particular importance, for modem buildings in which the traditional reserves of stiffness due to heavy internal partitions and outer cladding have largely disappeared. It must be stressed, however, that even if the drift index is kept within traditionally accepted limits, such as 1/500, it does not necessarily follow that the dynamic comfort criteria will also be satisfactory. Problems may arise, for example, if there is coupling between bending and torsional oscillations that leads to unacceptable complex motions of accelerations. In addition to static deflection calculations, the question of the dynamic response, involving the lateral acceleration, amplitude, and period of oscillation, may also have to be considered.

The establishment of a drift index limit is a major design decision, but, unfortunately, there are no unambiguous or widely accepted values, or even, in some of the National Codes concerned, any firm guidance. The designer is then faced with having to decide on an appropriate value. The figure adopted will reflect the building usage, the type of design criterion employed (for example, working or ultimate load conditions), the form of construction, the materials employed, including any substantial infills or claddings, the wind loads considered, and, in particular, past experience of similar buildings that have performed satisfactorily.

Design drift index limits that have been used in different countries range from 0.001 to 0.005. To put this in perspective, a maximum horizontal top deflection of between 0.1 and 0.5 m (6 to 20 in.) would be allowed in a 33-story, 100-m (330-ft.) high building, or, alternatively, a relative deflection of 3 to 15 mm (0. 12 to 0.6 in.) over a story height of 3 m (10 ft). Generally, lower values should be used for hotels or apartment buildings than for office buildings, since noise and movement tend to be more disturbing in the former. Consideration may be given to whether the stiffening effects of any internal partitions, infills, or claddings are included in the deflection calculations.

The consideration of this limit state requires an accurate estimate of the lateral deflections that occur, and involves an assessment of the stiffness of cracked members, the effects of shrinkage and creep and any redistribution of forces that may result, and of any rotational foundation movement. In the design process, the stiffness of joints, particularly in precast or prefabricated structures, must be given special attention to develop adequate lateral stiffness of the structure and to prevent any possible progressive failure. The possibility of torsional deformations must not be overlooked.

In practice, non-load-bearing infills, partitions, external wall panels, and window glazing should be designed with sufficient clearance or with flexible supports to accommodate the calculated movements.

Sound engineering judgment is required when deciding on the drift index limit to be imposed. However, for conventional structures, the preferred acceptable range is 0.0015 to 0.003 (that is, approximately 1/650 to 1/350), and sufficient stiffness must be provided to ensure that the top deflection does not exceed this value under extreme load conditions. As the height of the building increases, drift index coefficients should be decreased to the lower end of the range to keep the top story deflection to a suitably low level. Succeeding chapters describe how deflections may be computed.

The drift criteria apply essentially to quasistatic conditons. When extreme force conditions are possible, or where problems involving vortex shedding or other unusual phenomena may occur, a more sophisticated approach involving a dynamic analysis may be required.

If excessive, the drift of a, structure can be reduced by changing the geometric configuration to alter the mode of lateral load resistance, increasing the bending stiffness of the horizontal members, adding additional stiffness by the inclusion of stiffer wall or core members, achieving stiffer connections, and even by sloping the exterior columns. In extreme circumstances, it may be necessary to add dampers, which may be of the passive or active type.

2.2.3 Human Comfort Criteria [2]

If a tall flexible structure is subjected to lateral or torsional deflections under the action of fluctuating wind loads, the resulting oscillatory movements can induce a wide range of responses in the building's occupants, ranging from mild discomfort to acute nausea. Motions that have psychological or physiological effects on the occupants may thus result in an otherwise acceptable structure becoming an undesirable or even unrentable building.

There are as yet no universally accepted international standards for comfort criteria, although they are under consideration, and engineers must base their design criteria on an assessment of published data. It is generally agreed that acceleration is the predominant parameter in determining human response to vibration, but other factors such as period, amplitude, body orientation, visual and acoustic cues, and even past experience can be influential. Threshold curves are available that give various limits for human behavior, ranging from motion perception through work difficulty to ambulatory limits, in terms of acceleration and period. A dynamic analysis is then required to allow the predicted response of the building to be compared with the threshold limits.

2.3 STRUCTURAL FORM

2.3.1 Rigid-Frame Structures [2]

Rigid-frame structures consist of columns and girders joined by moment-resistant connections. The lateral stiffness of a rigid frame bent depends on the bending stiffness of the columns, girders, and connections in the plane of the bent (Appendix: Figure 4). The rigid frame's principal advantage is its open rectangular arrangement, which allows freedom of planning and easy fitting of doors and windows.

Rigid-frame construction is ideally suited for reinforced concrete joints. The sizes of the columns and girders at any level of a rigid frame are directly influenced by the magnitude of the external shear at that level, and they therefore increase toward the base. Consequently, the design of the floor framing cannot be repetitive as it is in some braced frames. A further result is that sometimes it is not possible in the lowest stories to accommodate the required depth of girder within the normal ceiling space.

Gravity loading also is resisted by the rigid-frame action. Negative moments are induced in the girders adjacent to the columns causing the mid-span positive moments to be significantly less than in a simply supported span. In structures in which gravity loads dictate the design, economies in member sizes that arise from this effect tend to be offset by the higher cost of the rigid joints.

2.3.2 Shear Wall Structures [2]

Concrete or masonry continuous vertical walls may served both architecturally as partitions and structurally to carry gravity and lateral loading. Their very high in-plane stiffness and strength makes them ideally suited for bracing tall buildings. In a shear wall structure, such walls are entirely responsible for the lateral load resistance of the building. They act as vertical cantilevers in the form of separate planar walls, and as nonplanar assemblies of connected walls around elevator, stair, and service shafts.

In contrast to rigid frames, the shear walls' solid form tends to restrict planning where open internal spaces are required. They are well suited, however, to hotels and residential buildings where the floor-by-floor repetitive planning allows the walls to be vertically continuous and where they serve simultaneously as excellent acoustic and fire insulators between rooms and apartments.

If, in low-to-medium-rise buildings, shear walls are combined with frames, it is reasonable to assume that the shear walls attract all the lateral loading so that the frame may be designed for only gravity loading. It is especially important in shear wall structures to try to plan the wall layout so that the lateral load tensile stresses are suppressed by the gravity load stresses. This allows them to be designed to have only the minimum reinforcement. Shear wall structures have been shown to perform well in earthquakes, for which case ductility becomes an important consideration in their design.

CHAPTER 3

ANALYTICAL AND SOFTWARE MODELLING

3.1 RIGID FRAME STRUCTURE

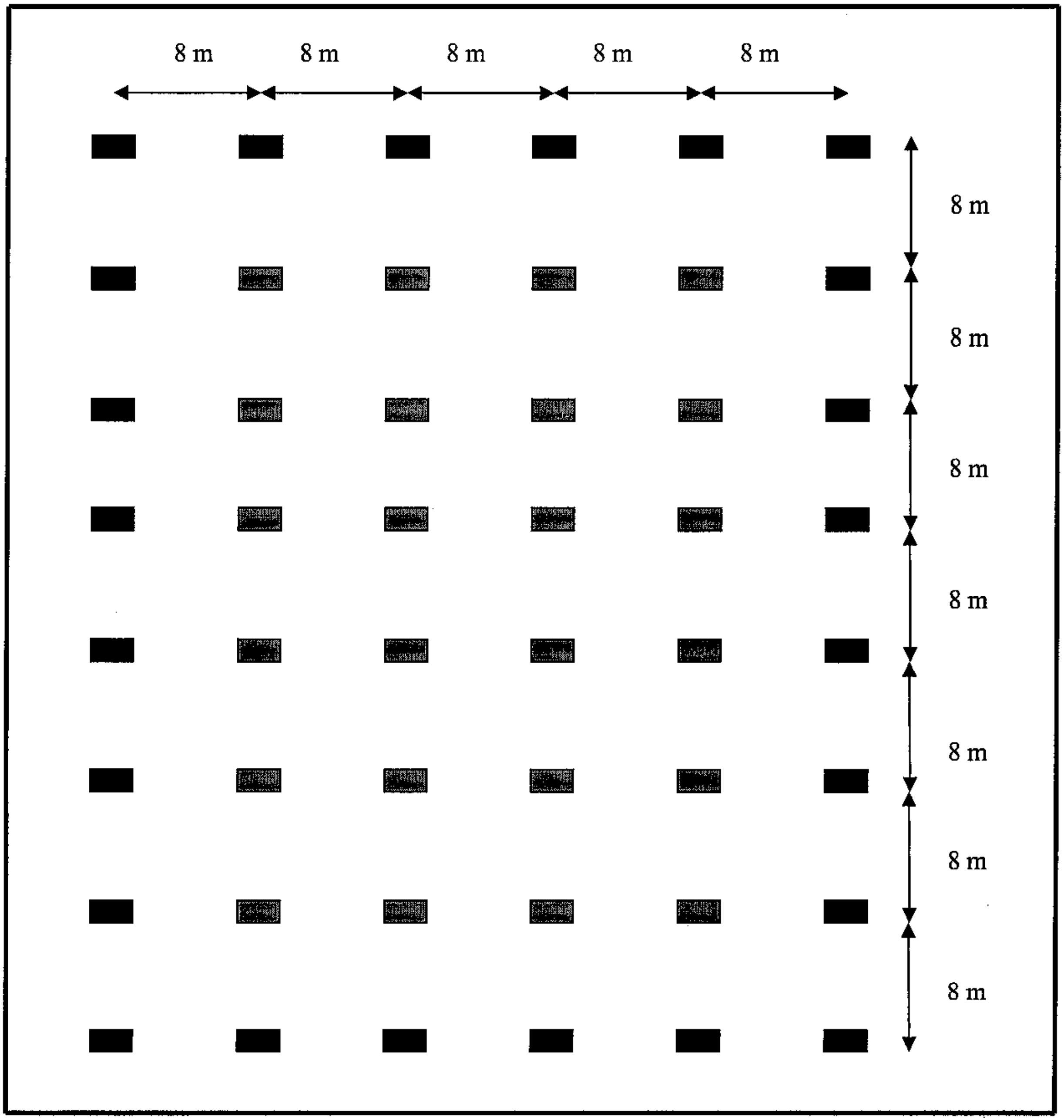


Figure 3.1: Typical Layout Plan and Column Position For Rigid Frame Structure

--- External Column
--- Internal Column

3.1.1 Description

Figure 1 shows a typical floor layout plan of a rigid frame structure that was chosen in modeling one of the structural system. The shape of the external column would be a rectangular (1 width: 2 length) while the shape of the internal column would be a square. All columns are spaced 8 m apart. The structure would be 70 storeys tall; each storey is 3m, which would add up to a total height of 210 m. The structure consist no beams, as rigid slab is used. The thickness of the slab will be 250 mm and concrete grade 35 will be used for the slab.

3.1.2 Procedure

The structure would be modeled as 3 dimensional structure (space analysis) using Staad.Pro 2002 software. There analysis of the structure is includes:

- To find the optimum column sizes of the rigid frame structure for three different concrete grades, namely Grade 50, 60, and 80.
- To find the horizontal displacements and maximum drift of the rigid frame structure for concrete Grade 50, 60 and 80. For this analysis, column sizes are fixed (using optimum column sizes for concrete Grade 50).
- To find the quantity of steel reinforcement to be used in the rigid frame structure for concrete Grade 50, 60 and 80. For this analysis, column sizes are also fixed.

The analysis results above would be compared and discussed. However, it is important to note that the results obtained would be theoretical, based on the analysis of the Staad.Pro 2002 structural software.

3.2 SHEAR WALL STRUCTURE

Type 1

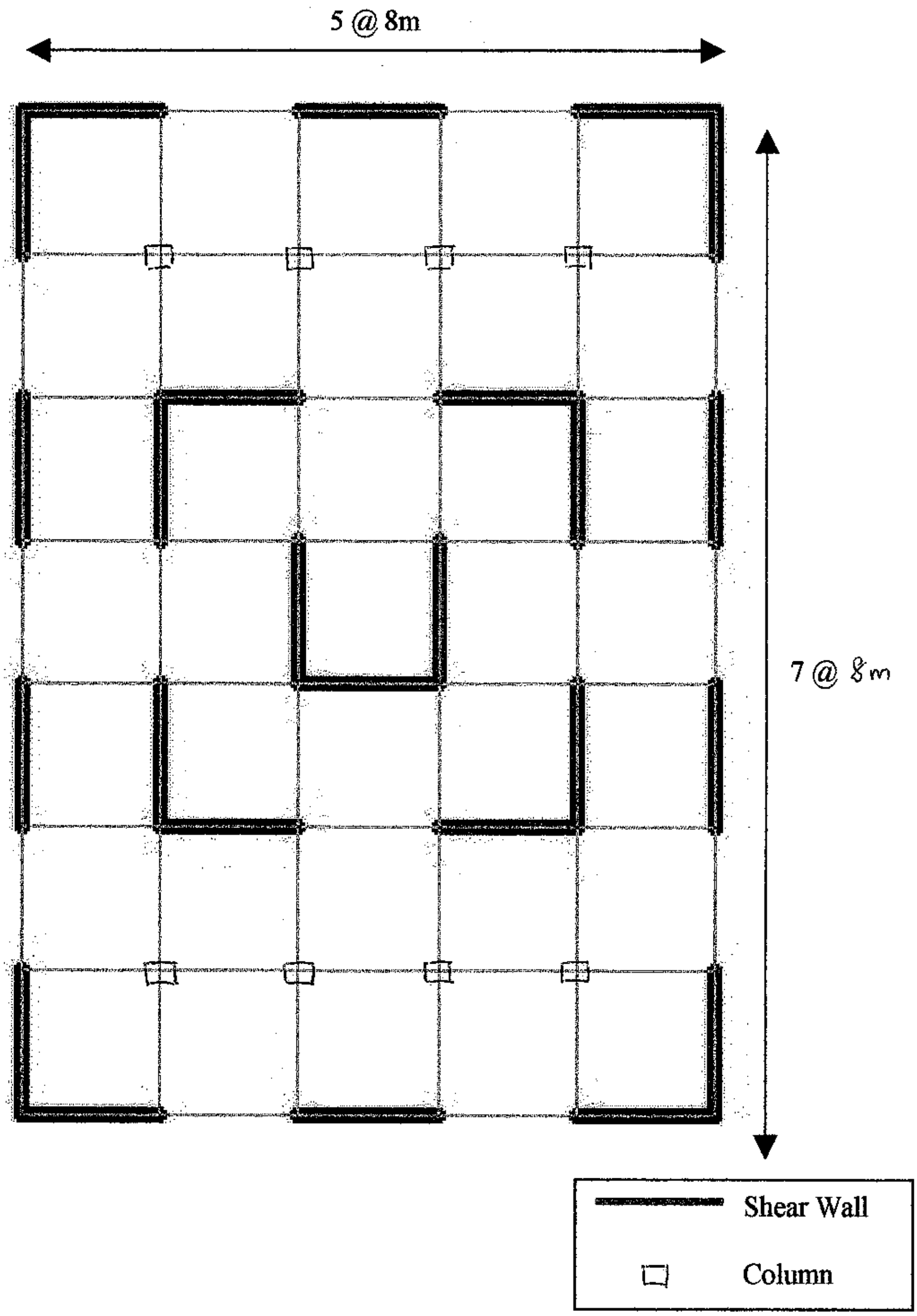


Figure 3.2: Typical Layout Plan for Shear Wall Type 1.

Type 2

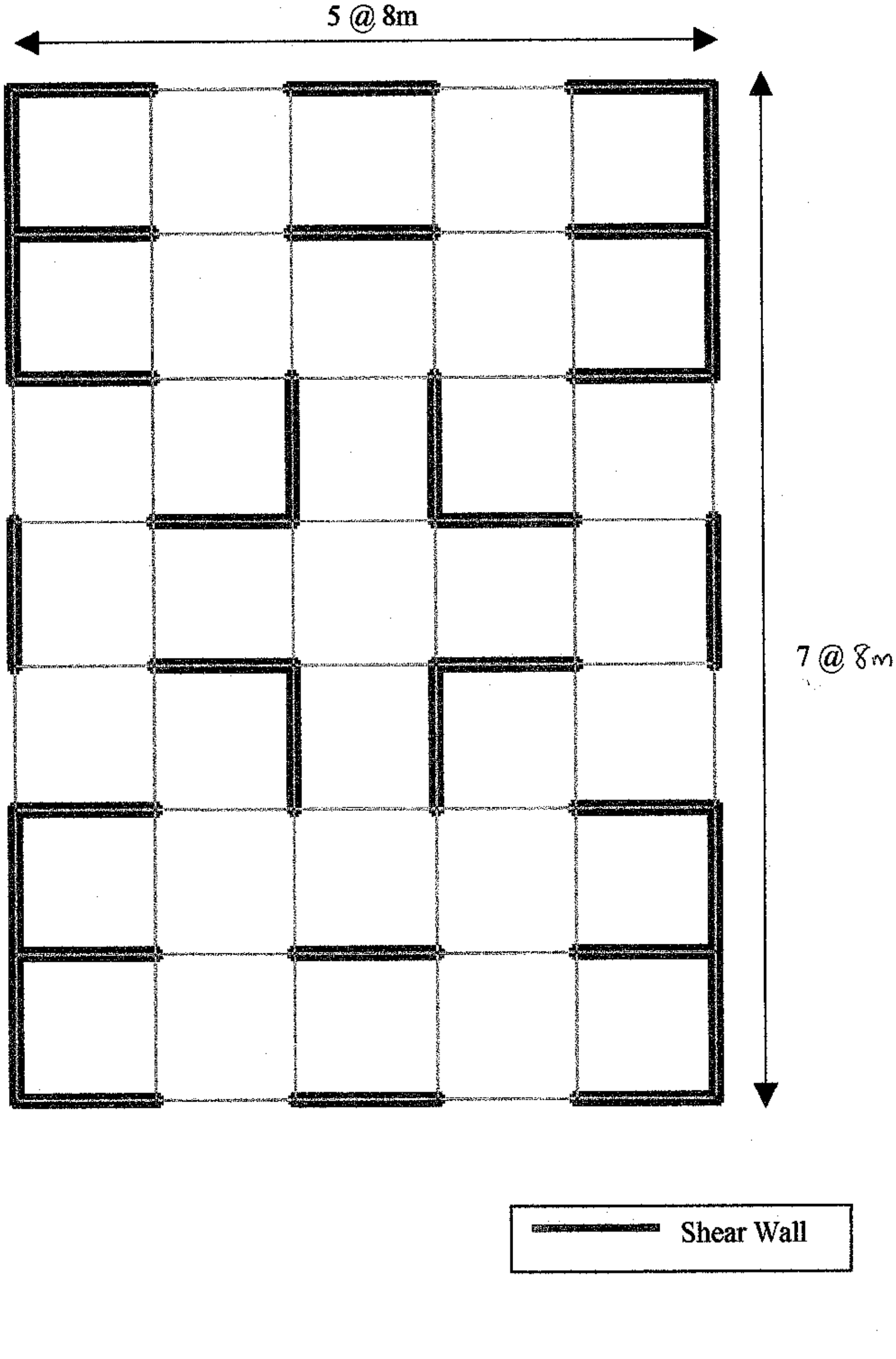


Figure 3.3: Typical Layout Plan for Shear Wall Type 2.

Type 3

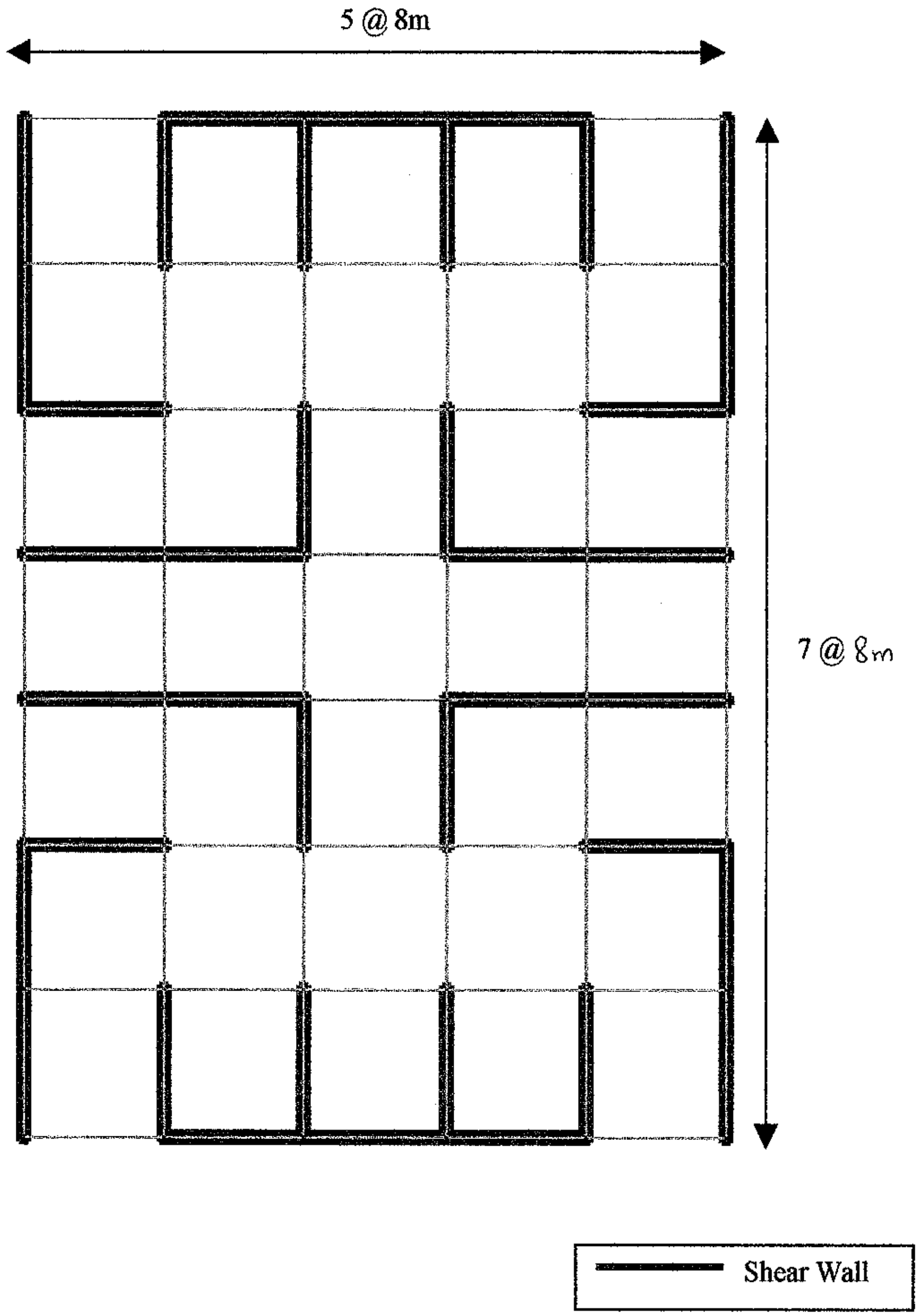


Figure 3.4: Typical Layout Plan for Shear Wall Type 3.

3.2.1 Description

There are three types of shear wall structures to be modeled in this project. Figure 2, 3 and 4 showed the typical floor layout plan of these shear wall structures that will be used for modeling in this project. The structure would be 70 storeys tall; each storey is 3m, which would add up to a total height of 210 m. The structure consist no beams, as rigid slab is used. The thickness of the slab will be 250 mm and concrete grade 35 will be used for the slab.

The thickness of shear wall and size of columns (for structure 1 only) used for the structure are shown at the table below.

Height of structure (m)	Thickness of shear wall (mm)	Sizes of column (mm)
180 - 210	200	200 x 200
150 – 180	250	250 x 250
120 – 150	300	300 x 300
90 – 120	350	350 x 350
60 – 90	400	400 x 400
30 - 60	450	450 x 450
0 - 30	500	500 x 500

Table 3.1: Thickness of shear wall and sizes of column with relative to the height of the shear wall structure

3.2.2 Procedure

The structure would be modeled as 3 dimensional structure (space analysis) using Staad.Pro 2002 software. There analysis of the structure is includes:

- To find the horizontal displacements and maximum drift of the shear wall structure for concrete Grade 50, 60 and 80. For this analysis, the shear wall thickness and column sizes (for structure Type 1) are fixed.

The analysis results for the three types of shear wall structure will be compared and discussed. However, it is important to note that the results obtained would be theoretical, based on the analysis of the Staad.Pro 2002 structural software.

3.3 DESIGN INFORMATION AND ASSUMPTIONS

3.3.1 Design Standards and Codes of Practice

In the design of structure, the following Codes of Practice provide the guide:

- BS 8110 : Part 1: 1997 – Structural use of concrete
- BS 6399 : Part 2: 1997 – Wind loads

3.3.2 Material Properties

The following strengths and properties will be used for design:

Concrete

Density	: 24 kN/m^3
Poisson ratio	: 0.17
Young's Modulus	: 29.5 kN/mm^2 (Grade 35)
	34 kN/mm^2 (Grade 50)
	36 kN/mm^2 (Grade 60)
	40 kN/mm^2 (Grade 80)

Reinforcement

High Tensile Deformed Type 2 : 460N/mm^2

3.3.3 Base Support

The base support used for the structure is fixed support.

3.3.4 Dead Load and Imposed Load

Dead Load used for the structure consists of:

- Self-weight of the concrete structure
- Floor finishing load = 1.7 kN/m^2

Imposed Load used for the structure is 3.0 kN/m^2

3.3.5 Wind Load

The wind load is calculated based on BS 6399 : Part 2: 1997:

$$\boxed{q_i = 0.613\ V_s^2}$$

Where;

q_i is the dynamic pressure in N/m^2

V_s is the design wind speed in m/s

The design wind speed , V_s can be calculated using the formula:

$$\boxed{V_s = S_1\ S_2\ S_3\ V_b}$$

Where:

V_b is basic wind speed of a given site in m/s

S_1 is multiplying factor relating to topology

S_2 is multiplying factor relating to height above ground and wind braking

S_3 is multiplying factor related to life of structure

In this case the chosen site will be Kuala Lumpur, and therefore the basic wind speed is assumed to be 33 m/s. The value of S_1 and S_3 will be taken as unity (recommended for general use). Value S_2 can be found in Table 1 (Appendix), and the topographical factor chosen was 4 (city centres and other environments with large and frequent obstructions)

Below are the results of the wind calculation:

Height, h	S_2	Vs	q_i (N/m^2)	Height, h	S_2	Vs	q_i (N/m^2)
210.0000	1.1900	39.2700	945.3275	97.5000	1.0650	35.1450	757.1598
208.5000	1.1885	39.2205	942.9458	94.5000	1.0590	34.9470	748.6525
205.5000	1.1855	39.1215	938.1915	91.5000	1.0530	34.7490	740.1932
202.5000	1.1825	39.0225	933.4491	88.5000	1.0470	34.5510	731.7820
199.5000	1.1795	38.9235	928.7188	85.5000	1.0410	34.3530	723.4188
196.5000	1.1765	38.8245	924.0005	82.5000	1.0350	34.1550	715.1037
193.5000	1.1735	38.7255	919.2942	79.5000	1.0288	33.9488	706.4933
190.5000	1.1705	38.6265	914.6000	76.5000	1.0213	33.7013	696.2296
187.5000	1.1675	38.5275	909.9177	73.5000	1.0138	33.4538	686.0410
184.5000	1.1645	38.4285	905.2475	70.5000	1.0063	33.2063	675.9275
181.5000	1.1615	38.3295	900.5893	67.5000	0.9988	32.9588	665.8892
178.5000	1.1585	38.2305	895.9431	64.5000	0.9913	32.7113	655.9259
175.5000	1.1555	38.1315	891.3089	61.5000	0.9838	32.4638	646.0377
172.5000	1.1525	38.0325	886.6868	58.5000	0.9740	32.1420	633.2953
169.5000	1.1495	37.9335	882.0766	55.5000	0.9620	31.7460	617.7866
166.5000	1.1465	37.8345	877.4785	52.5000	0.9500	31.3500	602.4702
163.5000	1.1435	37.7355	872.8924	49.5000	0.9375	30.9375	586.7200
160.5000	1.1405	37.6365	868.3183	46.5000	0.9225	30.4425	568.0952
157.5000	1.1375	37.5375	863.7562	43.5000	0.9075	29.9475	549.7707
154.5000	1.1345	37.4385	859.2061	40.5000	0.8925	29.4525	531.7467
151.5000	1.1315	37.3395	854.6681	37.5000	0.8650	28.5450	499.4828
148.5000	1.1285	37.2405	850.1420	34.5000	0.8350	27.5550	465.4374
145.5000	1.1255	37.1415	845.6280	31.5000	0.8050	26.5650	432.5936
142.5000	1.1225	37.0425	841.1260	28.5000	0.7765	25.6245	402.5050
139.5000	1.1195	36.9435	836.6360	25.5000	0.7495	24.7335	375.0003
136.5000	1.1165	36.8445	832.1580	22.5000	0.7225	23.8425	348.4689
133.5000	1.1135	36.7455	827.6921	19.5000	0.6940	22.9020	321.5195
130.5000	1.1105	36.6465	823.2381	16.5000	0.6580	21.7140	289.0281
127.5000	1.1075	36.5475	818.7962	13.5000	0.6220	20.5260	258.2671
124.5000	1.1045	36.4485	814.3663	10.5000	0.5860	19.3380	229.2364
121.5000	1.1015	36.3495	809.9484	7.5000	0.5400	17.8200	194.6596
118.5000	1.0978	36.2258	804.4429	4.5000	0.4920	16.2360	161.5915
115.5000	1.0933	36.0773	797.8612				
112.5000	1.0888	35.9288	791.3064				
109.5000	1.0843	35.7803	784.7787				
106.5000	1.0798	35.6318	778.2780				
103.5000	1.0753	35.4833	771.8044				
100.5000	1.0708	35.3348	765.3578				

3.3.6 Load Combinations

There are three type of load combinations used for the analysis of the structure:

- 1.4 x Dead Load + 1.6 x Live Load
- 1.2 x Dead Load + 1.2 x Live Load + 1.2 x Wind Load
- 1.0 x Dead Load + 1.4 x Wind Load

3.4 SOFTWARE

The only software which was used for computational based analytical study is STAAD.PRO 2002.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 EFFECTS OF CONCRETE GRADE ON OPTIMUM COLUMN SIZES FOR RIGID FRAME STRUCTURE

From the results as obtained, the optimum column sizes for the rigid frame structure using concrete grade 50, 60, and 80 are shown in Table 4.1, 4.2 and 4.3.

From these tables, it can be seen that the concrete grade 60 reduced the column sizes to 100 mm for each side when compared with the column sizes as obtained using grade 50 concrete, whereas the external column remained the same. The use of grade 80 concrete further optimized the column sizes in comparison to concrete grade 50. At the lowest one quarter height of the building, a reduction of 200 mm on each side was obtained and for rest of the upper height, a reduction of 150 mm was obtained.

From the results and above discussion, it is very clear that the higher concrete grades largely influence the optimization of internal columns compared to the optimization of external column. External columns for a rigid frame structure played a vital role in resisting lateral loads exerted on the structure, and this might be the reason why reduction in concrete grade has little impact on the external column sizes.

*Table 4.1: Column sizes for Rigid Frame Structure using **Grade 50 Concrete***

Height (m)	Column Sizes (mm)	
	External (1 width : 2 length)	Internal (square)
180 - 210	400 x 200	500 x 500
150 – 180	500 x 250	600 x 600
120 – 150	600 x 300	700 x 700
90 – 120	700 x 350	800 x 800
60 – 90	800 x 400	900 x 900
30 – 60	900 x 450	1000 x 1000
0 – 30	1000 x 500	1100 x 1100

*Table 4.2: Column sizes for Rigid Frame Structure using **Grade 60 Concrete***

Height (m)	Column Sizes (mm)	
	External (1 width : 2 length)	Internal (square)
180 – 210	400 x 200	400 x 400
150 – 180	500 x 250	500 x 500
120 – 150	600 x 300	600 x 600
90 – 120	700 x 350	700 x 700
60 – 90	800 x 400	800 x 800
30 – 60	900 x 450	900 x 900
0 – 30	1000 x 500	1000 x 1000

*Table 4.3: Column sizes for Rigid Frame Structure using **Grade 80 Concrete***

Height (m)	Column Sizes (mm)	
	External (1 width : 2 length)	Internal (square)
180 – 210	400 x 200	350 x 350
150 – 180	500 x 250	450 x 450
120 – 150	600 x 300	550 x 550
90 – 120	700 x 350	650 x 650
60 – 90	800 x 400	700 x 700
30 – 60	900 x 450	800 x 800
0 – 30	900 x 450	900 x 900

4.2 EFFECTS OF CONCRETE GRADE ON DRIFT FOR RIGID FRAME STRUCTURE

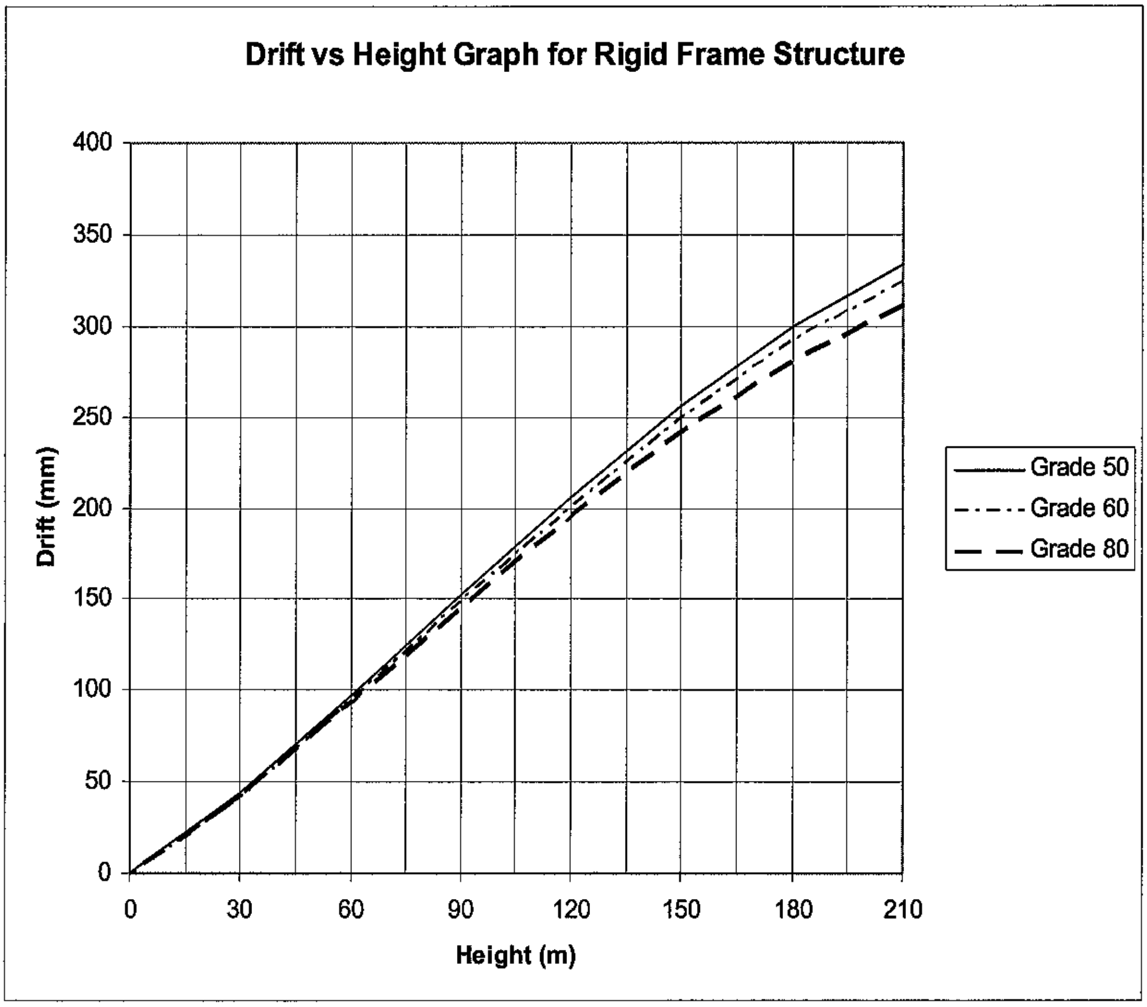
Table 4.4 shows the results of drift for rigid frame structure modeled using concrete grade 50, 60 and 80 respectively. The column sizes used for these modeling are fixed, as per column sizes stated in Table 4.1. Figure 4.1 displayed these results in a graphical from, for clearer view and understanding.

The drift index calculated for the rigid frame structure is 0.00159, 0.00155 and 0.00148 for concrete grade 50, 60 and 80 respectively. The allowable drift index used in different countries ranged from 0.001 to 0.005 and the drift index for the rigid frame structure lies in the upper range of the allowable drift index. This might suggest that rigid frame structure is weak in controlling drift for tall building. From figure 4.1, we can also see that the difference in drift for rigid frame structure of grade 50, 60 and 80 are quite small, therefore we can safely assumed that increase in concrete grades have little impact on the drift of the rigid frame structure.

Table 4.4: Drift for Rigid Frame Structure Using Concrete Grade 50, 60 and 80.

Height (m)	Drift (mm)		
	Grade 50	Grade 60	Grade 80
210	333.929	325.178	311.301
180	299.984	293.185	281.548
150	255.559	250.419	241.607
120	205.567	201.956	195.756
90	152.277	149.986	146.034
60	97.600	96.356	94.194
30	43.647	43.132	42.216

Figure 4.1: Graph Showing the Relationship between Drift vs Height of Rigid Frame Structure Using Different Grades of Concrete.



4.3 EFFECTS OF CONCRETE GRADE ON THE REINFORCEMENT STEEL USED FOR RIGID FRAME STRUCTURE

Table 4.5 shows the estimated reinforcement steel used in the rigid frame structure for concrete grade 50, 60 and 80. The column sizes used for these modeling are fixed, as per column sizes stated in Table 4.1. From table 4.5, we can see that there is a significantly large reduction in reinforcement steel when concrete grade increased from grade 50 to grade 60. Increased in concrete grade from grade 60 to 80 somehow shows a much lesser reduction in reinforcement steel compared to the previous case.

Table 4.5: Quantity of Reinforcement Steel used for Rigid Frame Structure

Concrete Grade	Total Reinforcement Steel Used (Newton)
80	2,167,857
60	2,867,534
50	7,228,911

4.4 EFFECTS OF SHEAR WALL ARRANGEMENT ON DRIFT

Table 4.6, 4.7 and 4.8 shows the result of drift of shear wall structure of different types namely Type 1, Type 2 and Type 3 each modeled with concrete grade 50, 60 and 80. Figure 4.2, 4.3 and 4.4 displayed the results in graphical form.

The calculated drift index for the shear wall structure is as shown in Table 4.9. From the table, we can see that drift index for the shear wall structure for all type and concrete grade are much lesser than the allowable range of 0.001 to 0.005. This might an indication that shear wall are efficient in resisting lateral loads on tall buildings.

From figure 4.2, 4.3 and 4.4, we can see from the graph that for a shear wall structure of a particular concrete grade, different type of arrangement of shear wall structure does have a huge effect on the drift of the structure. It is clear from the graph that the drift of shear wall structure type 3 is the lowest, following by type 2 and type 1.

Table 4.6: Drift for Shear Wall Structure of Different Type Using Concrete Grade 50

Height (m)	Drift (mm)		
	Type 1	Type 2	Type 3
210	168.330	114.237	49.204
180	146.698	100.421	41.159
150	121.405	83.839	32.628
120	92.671	64.467	23.858
90	62.357	43.578	15.397
60	33.460	23.374	7.976
30	10.420	7.212	2.492

Table 4.7: Drift for Shear Wall Structure of Different Type Using Concrete Grade 60

Height (m)	Drift (mm)		
	Type 1	Type 2	Type 3
210	163.807	111.498	47.320
180	142.608	97.884	39.548
150	117.900	81.611	31.328
120	89.893	62.660	22.887
90	60.400	42.278	14.754
60	32.345	22.623	7.634
30	10.440	6.960	2.382

Table 4.8: Drift for Shear Wall Structure of Different Type Using Concrete Grade 80

Height (m)	Drift (mm)		
	Type 1	Type 2	Type 3
210	155.811	106.628	44.035
180	135.374	93.368	36.752
150	111.697	77.642	29.063
120	84.974	59.440	21.195
90	56.937	39.966	13.637
60	30.377	21.293	7.040
30	9.384	6.515	2.191

Table 4.9: Drift Index for Shear Wall Structure

Concrete Grade	Drift Index		
	Type 1	Type 2	Type 3
50	0.00080	0.00054	0.00023
60	0.00078	0.00053	0.00023
80	0.00074	0.00051	0.00021

Figure 4.2: Graph Showing the Relationship between Drift vs Height of Shear Wall Structure using Concrete Grade 50 for 3 different types of Structure.

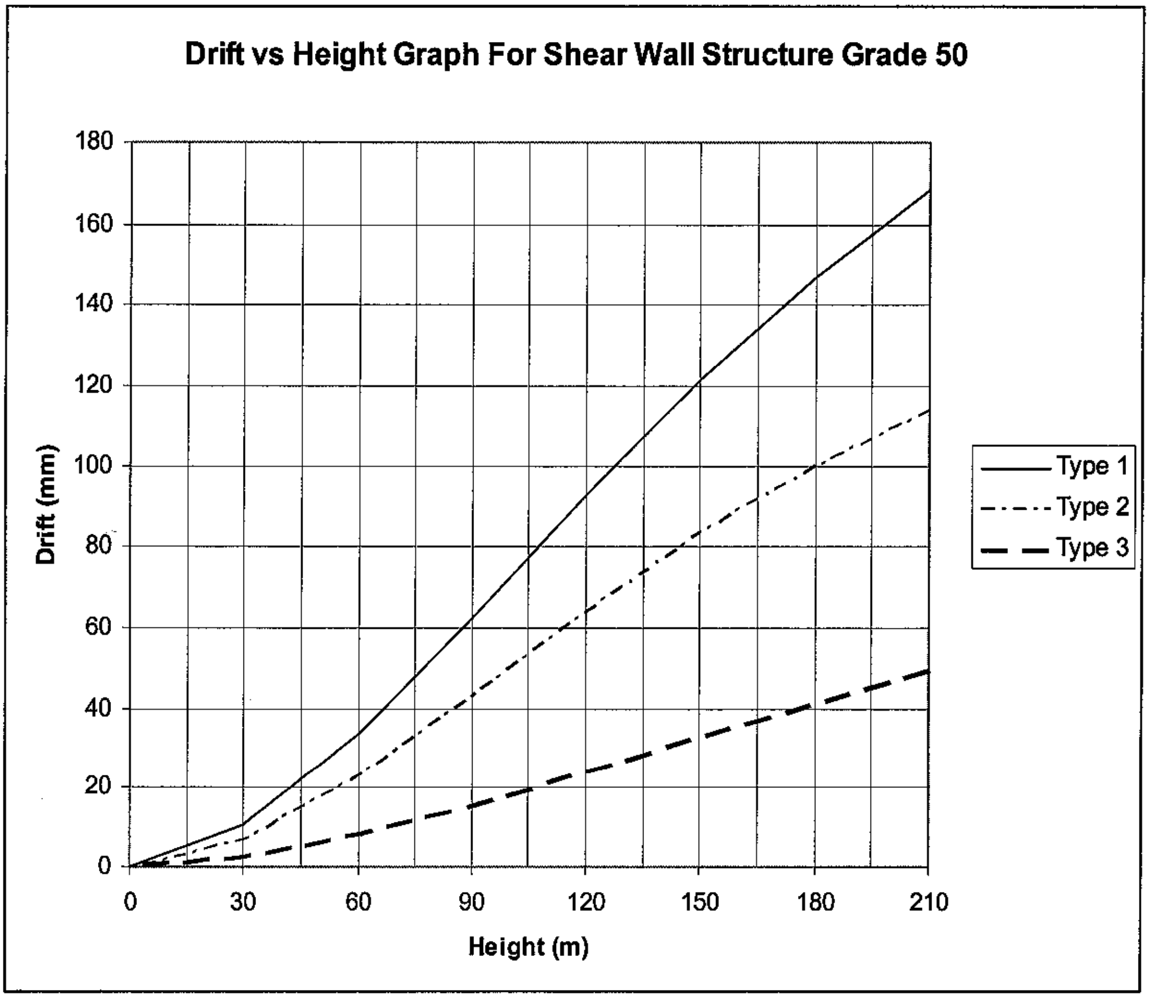


Figure 4.3: Graph Showing the Relationship between Drift vs Height of Shear Wall Structure using Concrete Grade 60 for 3 different types of Structure.

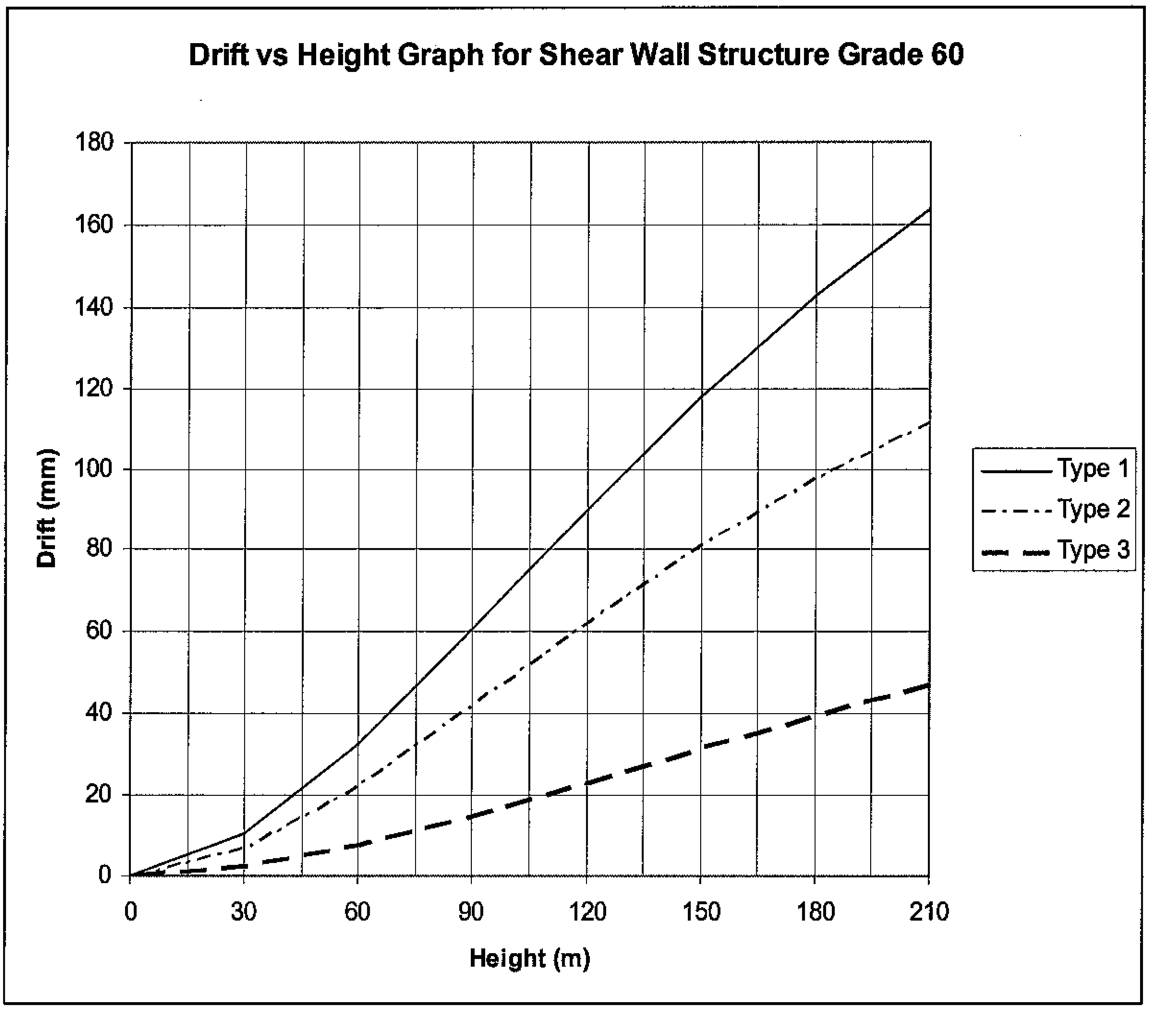
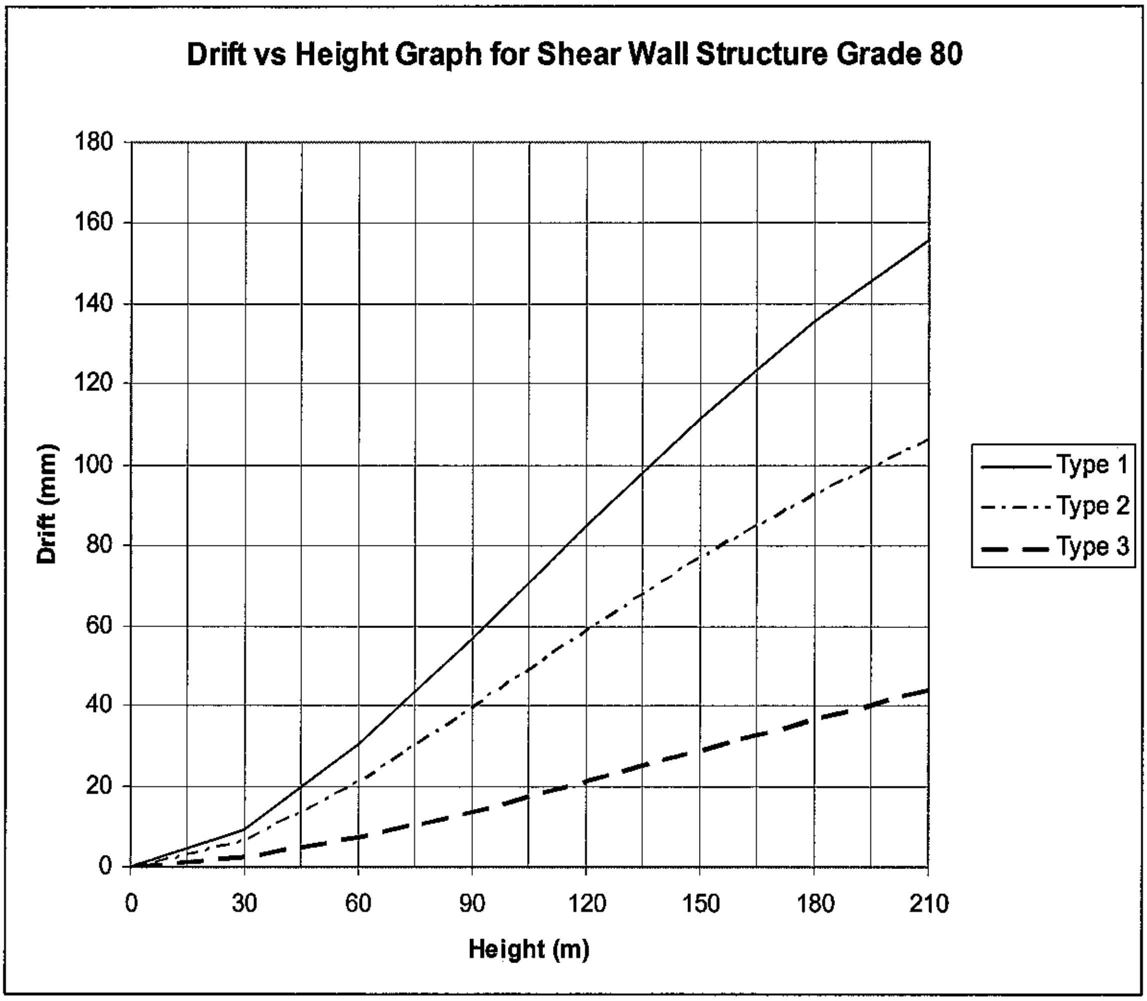


Figure 4.4: Graph Showing the Relationship between Drift vs Height of Shear Wall Structure using Concrete Grade 80 for 3 different types of Structure.



4.5 EFFECTS OF CONCRETE GRADE ON DRIFT FOR SHEAR WALL STRUCTURE

Figure 4.5, 4.6 and 4.7 shows the drift of a particular type of shear wall structure using different types of concrete grade. The purpose is to find out the effect of concrete grades on a particular type of shear wall structure.

From figure 4.5, 4.6, and 4.7, the graph shows that the increase in concrete grades for a particular type of structure actually only slightly reduce the drift. The similar case was also encountered for the rigid frame structure, as discussed earlier.

Figure 4.5: Graph Showing the Relationship between Drift vs Height of Shear Wall Structure Type 1 Using Different Grades of Concrete.

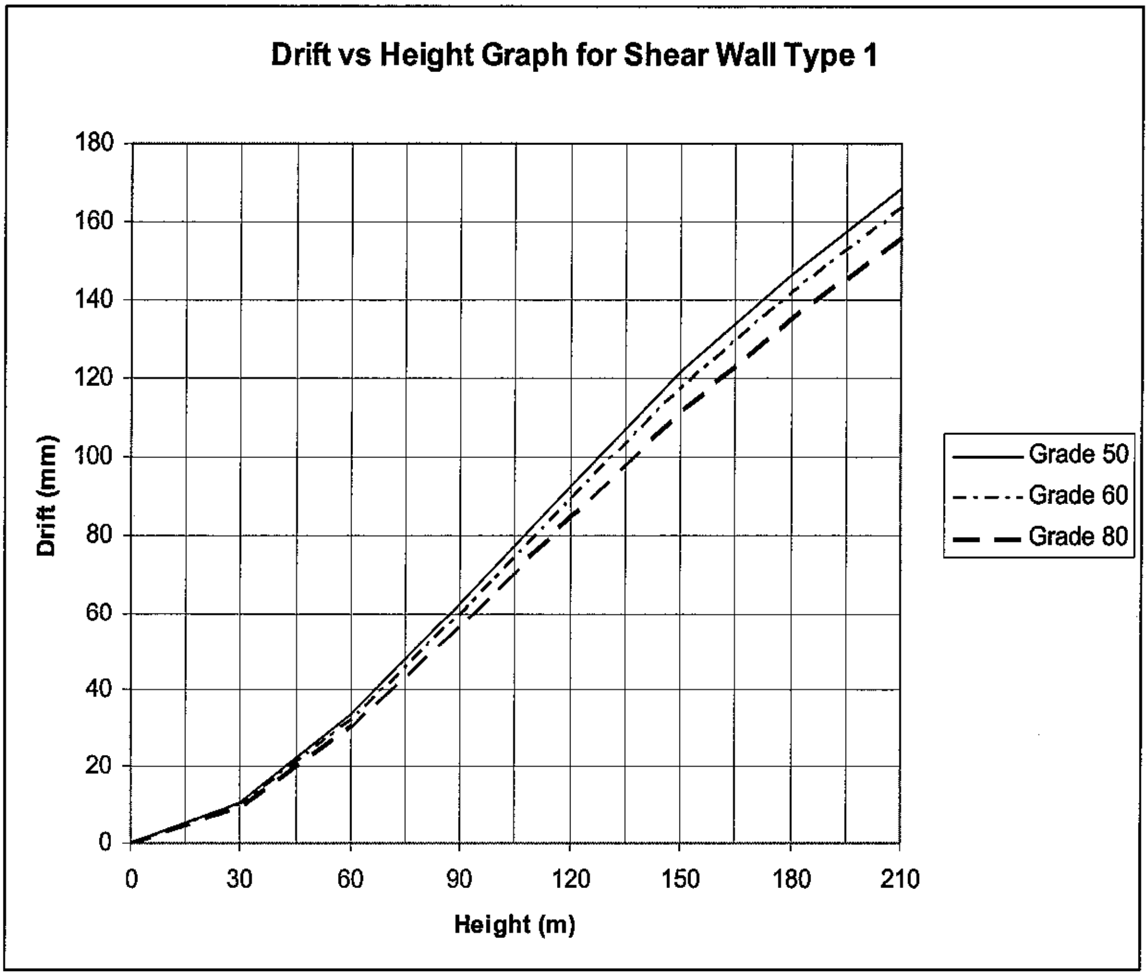


Figure 4.6: Graph Showing the Relationship between Drift vs Height of Shear Wall Structure Type 2 Using Different Grades of Concrete.

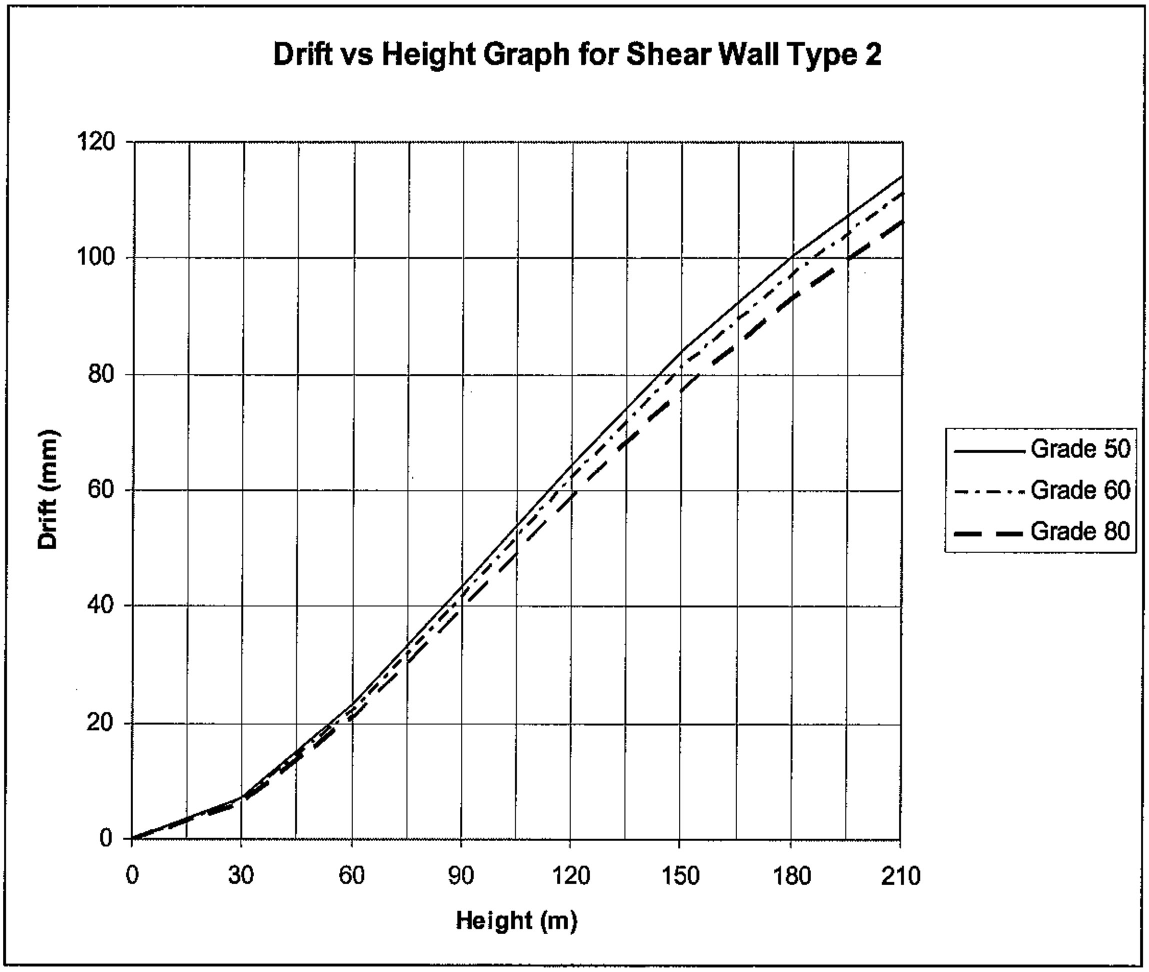
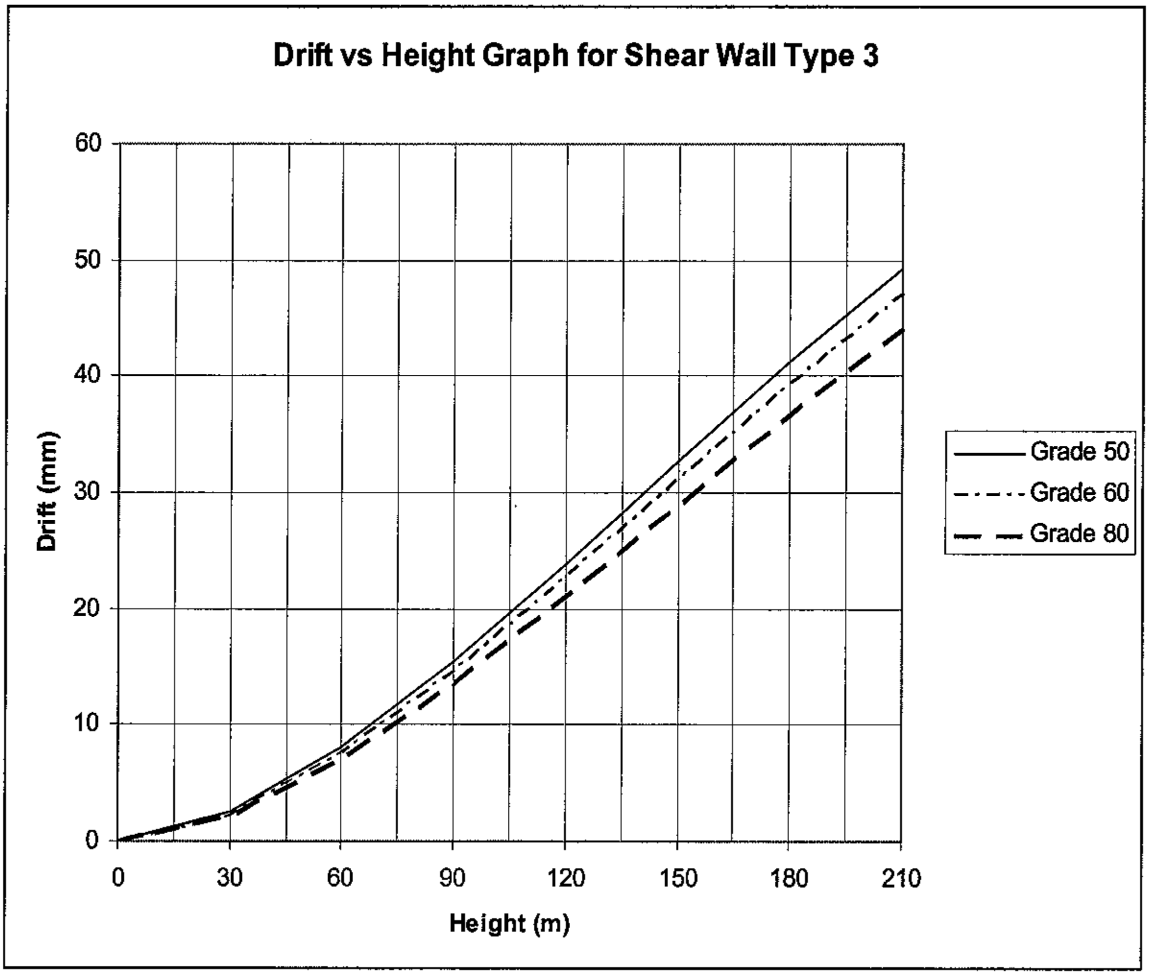


Figure 4.7: Graph Showing the Relationship between Drift vs Height of Shear Wall Structure Type 3 Using Different Grades of Concrete.



4.6 COMPARISON OF RIGID FRAME & SHEAR WALL STRUCTURAL SYSTEM FOR 210 M TALL BUILDING IN TERMS OF DRIFT

Basically from the results of the modeling, it is quite obvious that the shear wall structure is more effective in resisting lateral loads. The drift results obtained for the shear wall structure in general is much smaller than the one obtained from the rigid frame structure.

4.7 COMPARISON OF OTHER ASPECTS OF RIGID FRAME AND SHEAR WALL STRUCTURAL SYSTEM

4.7.1 Availability and Flexibility of Space

The shear walls act as structural member. These walls are permanent and therefore it does not have the flexibility for the walls position to be altered in the future. On the other hand, the rigid frame structure brick masonry partition walls which may offer flexibility to any changes in the future. However, in the taller rigid frame structure, columns are usually extruded out from the walls, which might lead to ineffective use of the floor space. This does significantly affect the sellable floor area of a building which has discussed later on.

4.7.2 Speed of Construction

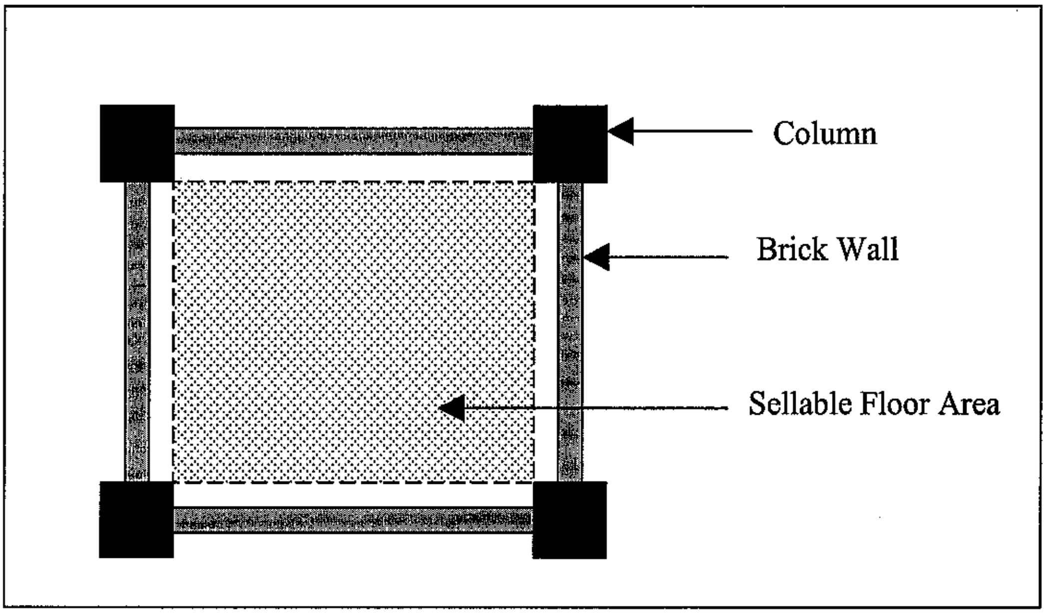
In terms of time for construction, the shear wall structure actually has advantages over the rigid frame structure. The construction of rigid frame structure always involves brick masonry work, which is one of the time consuming activity. However for the shear wall structure, there is actually minimum brick wall work involved as most of the walls of the building are shear walls. These shear walls are concrete walls cast on site using reusable standard size steel formwork, which actually saves a lot of time compared to the conventional brick wall systems.

4.7.2 Cost

As discussed earlier, more time is saved for the construction of shear wall structure if compared to the rigid frame structure. In construction time, is equivalent to money and therefore by saving time, a significant amount of cost is actually saved. Cost of labor of brick wall work can also be reduced in this case.

For the rigid frame structure, the total sellable floor area of the building is actually reduced as an effect from the extruded column from the wall. The sellable floor area of a building which has columns is actually the area calculated starting from the edge of the column (see Figure 4.8). Due to this reason, rigid frame structure which usually has large sizes of columns if compared to the shear wall structure with adequate thin shear walls and therefore, most of the time the total sellable floor area for rigid frame structure is lesser. This will definitely affect the profit which can be gained from the building.

Figure 4.8: Picture Showing Sellable Floor Area



CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

From the computational based analysis, results and discussions following conclusions are made:

- Concrete grade significantly affect internal column sizes than the external column sizes for rigid frame structure
- Higher grade concrete can reasonably reduce the amount of steel reinforcement in rigid frame structure.
- Total drift and storey drift is not significantly affected by the concrete grade.
- Shear wall arrangement greatly influenced the total drift of a shear wall structure.
- For a building height between 200 – 250 m which is subjected to moderate wind pressure, shear wall structure is more effective system as compare to rigid frame structural system.

5.2 RECOMMENDATION

The recommendations derived from this project include:

- The investigation shall be done using more advanced software on tall buildings such as E-Tabs as many limitations are encountered in using Staad.Pro2002 software for tall building modeling purposes.
- As this project only touches the very surface of tall building design, more detail and specific investigations of tall buildings can be built from here.
- Civil students of UTP shall be exposed to structural software at earlier stage to prepare them better for their future final year projects.

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2. Bryan Stafford Smith and Alex Coull (1991). Tall Building Structures – Analysis and Design. John Wiley & Sons.
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4. British Standard, Structural Use Of Concrete – Part 1. Code of Practice For Design And Construction.
5. Research Engineers International. Staad.Pro2002 User's Manual.

APPENDIX I

(Example of Staad.Pro 2002 Input Codes)

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 07-Aug-04
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
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4745 2764 2772 3390 3389; 4746 2772 2771 3391 3390; 4747 2771 2770 3392 3391;
4748 2770 2754 2755 3392; 4749 2746 2763 2764 2747; 4750 2763 2773 2772 2764;
4751 2773 2776 2771 2772; 4752 2776 2769 2770 2771; 4753 2769 2753 2754 2770;
4754 2745 2762 2763 2746; 4755 2762 2774 2773 2763; 4756 2774 2775 2776 2773;
4757 2775 2768 2769 2776; 4758 2768 2752 2753 2769; 4759 2744 2761 2762 2745;
4760 2761 2765 2774 2762; 4761 2765 2766 2775 2774; 4762 2766 2767 2768 2775;
4763 2767 2751 2752 2768; 4764 2743 3385 2761 2744; 4765 3385 3386 2765 2761;
4766 3386 3387 2766 2765; 4767 3387 3388 2767 2766; 4768 3388 2750 2751 2767;
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4772 2780 2781 3396 3395; 4773 2781 2782 2790 3396; 4774 2788 3397 2797 2789;
4775 3397 3398 2798 2797; 4776 3398 3399 2799 2798; 4777 3399 3400 2800 2799;
4778 3400 2795 2796 2800; 4779 2787 2804 3397 2788; 4780 2804 2812 3398 3397;
4781 2812 2811 3399 3398; 4782 2811 2810 3400 3399; 4783 2810 2794 2795 3400;
4784 2786 2803 2804 2787; 4785 2803 2813 2812 2804; 4786 2813 2816 2811 2812;
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4805 2818 2819 3402 3401; 4806 2819 2820 3403 3402; 4807 2820 2821 3404 3403;
4808 2821 2822 2830 3404; 4809 2828 3405 2837 2829; 4810 3405 3406 2838 2837;
4811 3406 3407 2839 2838; 4812 3407 3408 2840 2839; 4813 3408 2835 2836 2840;
4814 2827 2844 3405 2828; 4815 2844 2852 3406 3405; 4816 2852 2851 3407 3406;
4817 2851 2850 3408 3407; 4818 2850 2834 2835 3408; 4819 2826 2843 2844 2827;

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4829 2824 2841 2842 2825; 4830 2841 2845 2854 2842; 4831 2845 2846 2855 2854;
4832 2846 2847 2848 2855; 4833 2847 2831 2832 2848; 4834 2823 3401 2841 2824;
4835 3401 3402 2845 2841; 4836 3402 3403 2846 2845; 4837 3403 3404 2847 2846;
4838 3404 2830 2831 2847;
MEMBER PROPERTY AMERICAN
*External Frame
4839 TO 5078 PRIS YD 1 ZD 0.5
5079 TO 5318 PRIS YD 0.9 ZD 0.45
5319 TO 5558 PRIS YD 0.8 ZD 0.4
5559 TO 5798 PRIS YD 0.7 ZD 0.35
5799 TO 6038 PRIS YD 0.6 ZD 0.3
6039 TO 6278 PRIS YD 0.5 ZD 0.25
6279 TO 6518 PRIS YD 0.4 ZD 0.2
*Internal Frame
6519 TO 6758 PRIS YD 1.1 ZD 1.1
6759 TO 6998 PRIS YD 1 ZD 1
6999 TO 7238 PRIS YD 0.9 ZD 0.9
7239 TO 7478 PRIS YD 0.8 ZD 0.8
7479 TO 7718 PRIS YD 0.7 ZD 0.7
7719 TO 7958 PRIS YD 0.6 ZD 0.6
7959 TO 8198 PRIS YD 0.5 ZD 0.5
ELEMENT PROPERTY
2354 TO 4838 THICKNESS 0.25
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 3.6e+007
POISSON 0.17
DENSITY 24
ISOTROPIC MATERIAL2
E 2.95e+007
POISSON 0.17
DENSITY 24
END DEFINE MATERIAL
CONSTANTS
MATERIAL MATERIAL1 MEMB 4839 TO 8198
MATERIAL MATERIAL2 MEMB 2354 TO 4838
SUPPORTS
1 TO 48 FIXED
LOAD 1 DEAD LOAD
ELEMENT LOAD
2354 TO 4838 PR 1.7
LOAD 2 LIVE LOAD
ELEMENT LOAD
2354 TO 4838 PR 3
LOAD 3 WIND LOAD
JOINT LOAD
2817 2829 FX 5.67
2777 2789 FX 11.32
2737 2749 FX 11.26
2697 2709 FX 11.2
2657 2669 FX 11.15
2617 2629 FX 11.09
2577 2589 FX 11.03
2537 2549 FX 10.98
2497 2509 FX 10.92
2457 2469 FX 10.87
2417 2429 FX 10.81
2377 2389 FX 10.75
2337 2349 FX 10.7
2297 2309 FX 10.64
2257 2269 FX 10.58
2217 2229 FX 10.53
2177 2189 FX 10.48
2137 2149 FX 10.42
2097 2109 FX 10.37
2057 2069 FX 10.31
2017 2029 FX 10.26
1977 1989 FX 10.2
1937 1949 FX 10.15
1897 1909 FX 10.1
1857 1869 FX 10.04
1817 1829 FX 9.99
1777 1789 FX 9.93
1737 1749 FX 9.88
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1657 1669 FX 9.77
1617 1629 FX 9.72
1577 1589 FX 9.66
1537 1549 FX 9.58
1497 1509 FX 9.5
1457 1469 FX 9.42
1417 1429 FX 9.34
1377 1389 FX 9.26
1337 1349 FX 9.2
1297 1309 FX 9.09
1257 1269 FX 8.99
1217 1229 FX 8.88
1177 1189 FX 8.78
1137 1149 FX 8.68
1097 1109 FX 8.58
1057 1069 FX 8.48
1017 1029 FX 8.36
977 989 FX 8.23
937 949 FX 8.11
897 909 FX 7.99
857 869 FX 7.87
817 829 FX 7.75
777 789 FX 7.6
737 749 FX 7.42
697 709 FX 7.23
657 669 FX 7.04
617 629 FX 6.82
577 589 FX 6.6
537 549 FX 6.38
497 509 FX 6
457 469 FX 5.59
417 429 FX 5.19
377 389 FX 4.83
337 349 FX 4.5
297 309 FX 4.18
257 269 FX 3.86
217 229 FX 3.47
177 189 FX 3.1
137 149 FX 2.75
97 109 FX 2.34
49 92 FX 1.94
2823 TO 2828 FX 11.34
2783 TO 2788 FX 22.63
2743 TO 2748 FX 22.52
2703 TO 2708 FX 22.4
2663 TO 2668 FX 22.29
2623 TO 2628 FX 22.18
2583 TO 2588 FX 22.06
2543 TO 2548 FX 21.95
2503 TO 2508 FX 21.84
2463 TO 2468 FX 21.73
2423 TO 2428 FX 21.61
2383 TO 2388 FX 21.5
2343 TO 2348 FX 21.39
2303 TO 2308 FX 21.28
2263 TO 2268 FX 21.16
2223 TO 2228 FX 21.06
2183 TO 2188 FX 20.95
2143 TO 2148 FX 20.84
2103 TO 2108 FX 20.73
2063 TO 2068 FX 20.62
2023 TO 2028 FX 20.51
1983 TO 1988 FX 20.4
1943 TO 1948 FX 20.3
1903 TO 1908 FX 20.19
1863 TO 1868 FX 20.08
1823 TO 1828 FX 19.97
1783 TO 1788 FX 19.86
1743 TO 1748 FX 19.76
1703 TO 1708 FX 19.65
1663 TO 1668 FX 19.54
1623 TO 1628 FX 19.44
1583 TO 1588 FX 19.31
1543 TO 1548 FX 19.15
1503 TO 1508 FX 18.99
1463 TO 1468 FX 18.83

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1383 TO 1388 FX 18.52
1343 TO 1348 FX 18.37
1303 TO 1308 FX 18.17
1263 TO 1268 FX 17.97
1223 TO 1228 FX 17.76
1183 TO 1188 FX 17.56
1143 TO 1148 FX 17.36
1103 TO 1108 FX 17.16
1063 TO 1068 FX 16.96
1023 TO 1028 FX 16.71
983 TO 988 FX 16.46
943 TO 948 FX 16.22
903 TO 908 FX 15.98
863 TO 868 FX 15.74
823 TO 828 FX 15.5
783 TO 788 FX 15.2
743 TO 748 FX 14.83
703 TO 708 FX 14.46
663 TO 668 FX 14.08
623 TO 628 FX 13.63
583 TO 588 FX 13.19
543 TO 548 FX 12.76
503 TO 508 FX 11.99
463 TO 468 FX 11.17
423 TO 428 FX 10.38
383 TO 388 FX 9.66
343 TO 348 FX 9
303 TO 308 FX 8.36
263 TO 268 FX 7.72
223 TO 228 FX 6.94
183 TO 188 FX 6.2
143 TO 148 FX 5.5
103 TO 108 FX 4.67
52 62 68 74 80 86 FX 3.88
LOAD COMB 4 1.4DL + 1.6LL
1 1.4 2 1.6
LOAD COMB 5 1.2DL + 1.2LL + 1.2WL
1 1.2 2 1.2 3 1.2
LOAD COMB 6 1.0DL + 1.4WL
1 1.0 3 1.4
PERFORM ANALYSIS
LOAD LIST 4 TO 6
START CONCRETE DESIGN
CODE BS8110
UNIT MMS NEWTON
FC 60 MEMB 4839 TO 8198
FC 35 MEMB 2354 TO 4838
FYMAIN 460 MEMB 4839 TO 8198
FYSEC 410 MEMB 4839 TO 8198
MINMAIN 12 MEMB 4839 TO 8198
MAXMAIN 32 MEMB 4839 TO 8198
MINSEC 10 MEMB 4839 TO 8198
DESIGN COLUMN 4839 TO 8198
CONCRETE TAKE
END CONCRETE DESIGN
FINISH
```

APPENDIX II

(Example of Staad.Pro 2002 Software Interface)

Figure I: The skeleton of the structure is built by assigning the geometry information

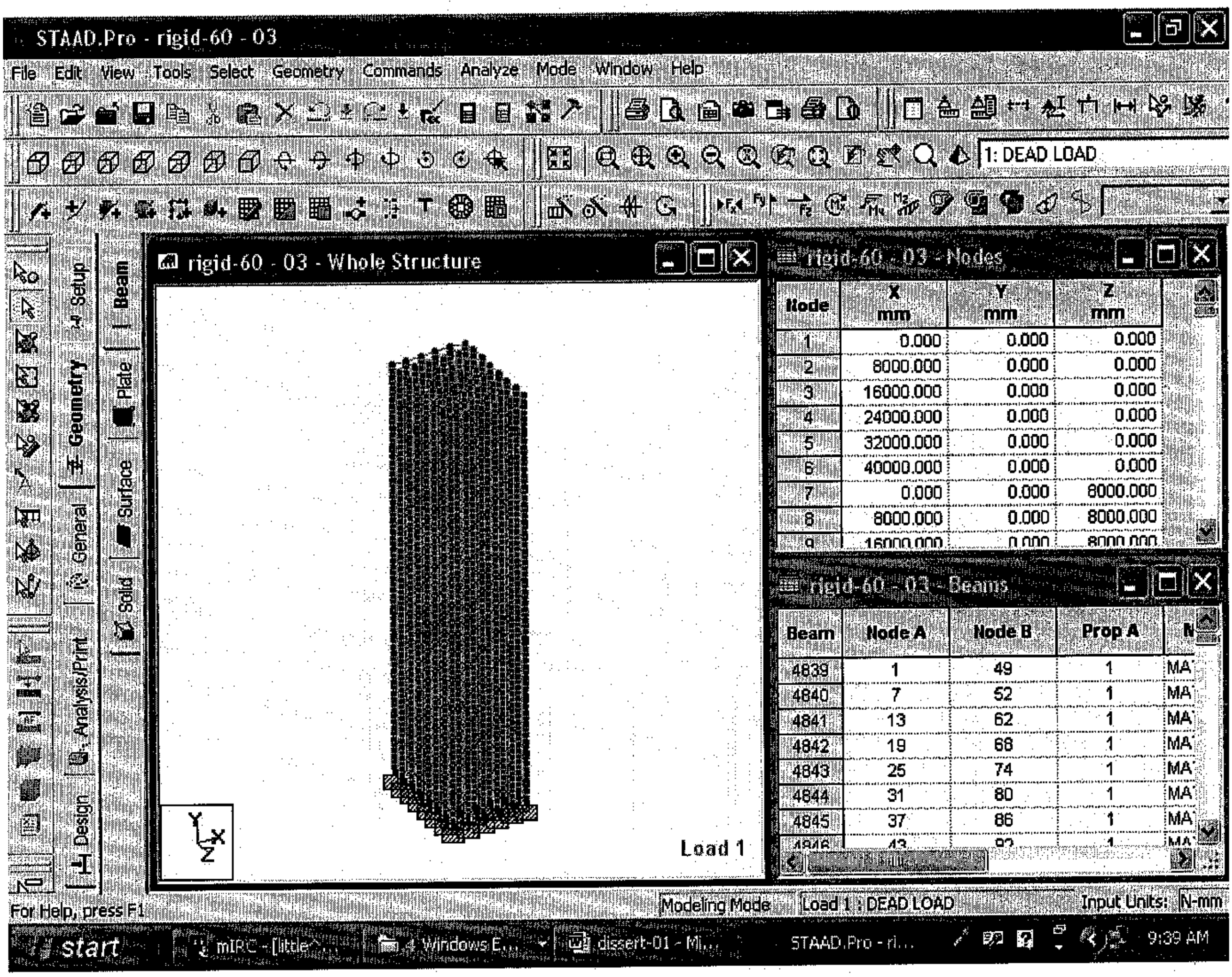

Figure II: Structural properties assigned to the structure

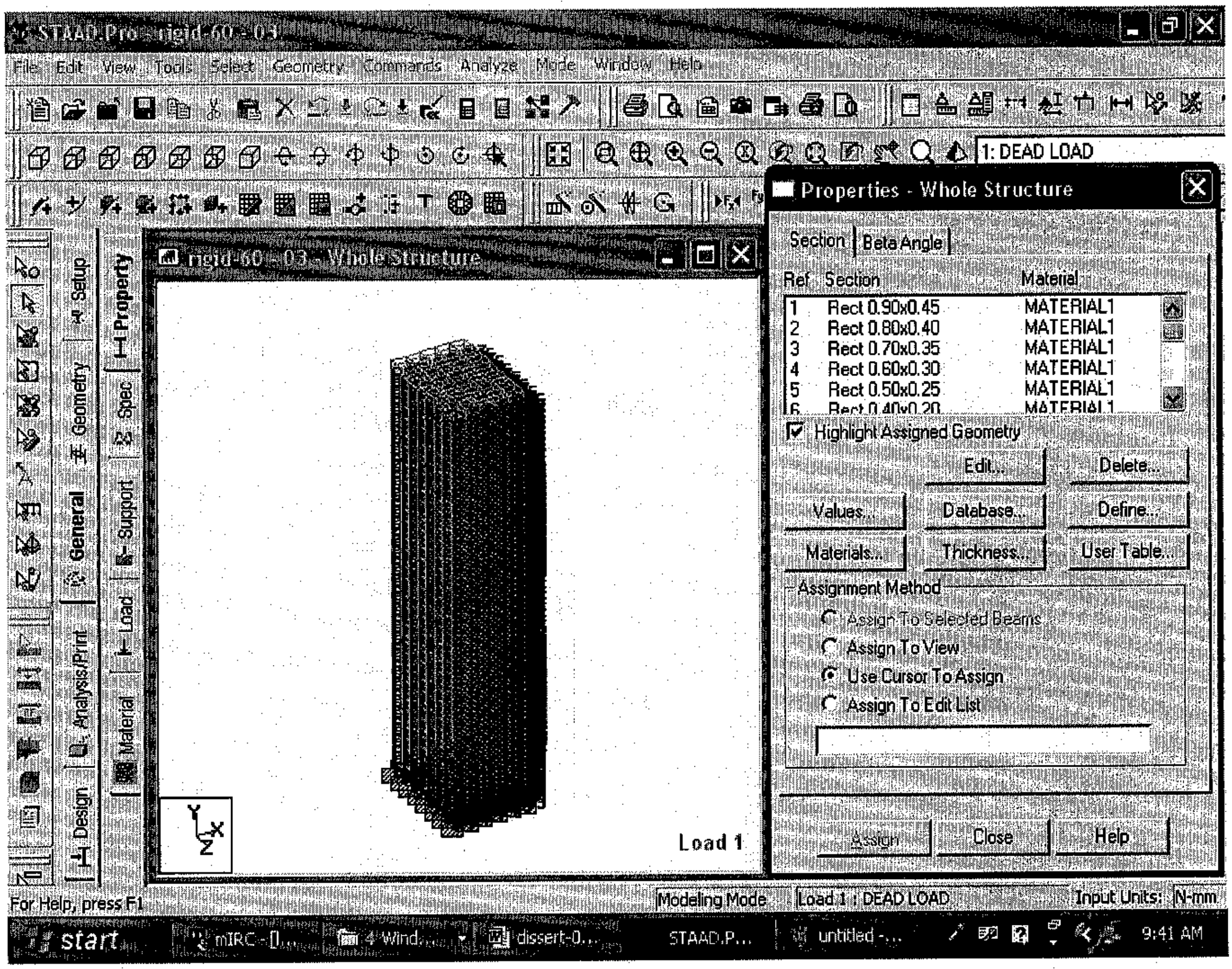

Figure III: Wind load are assigned to the structure

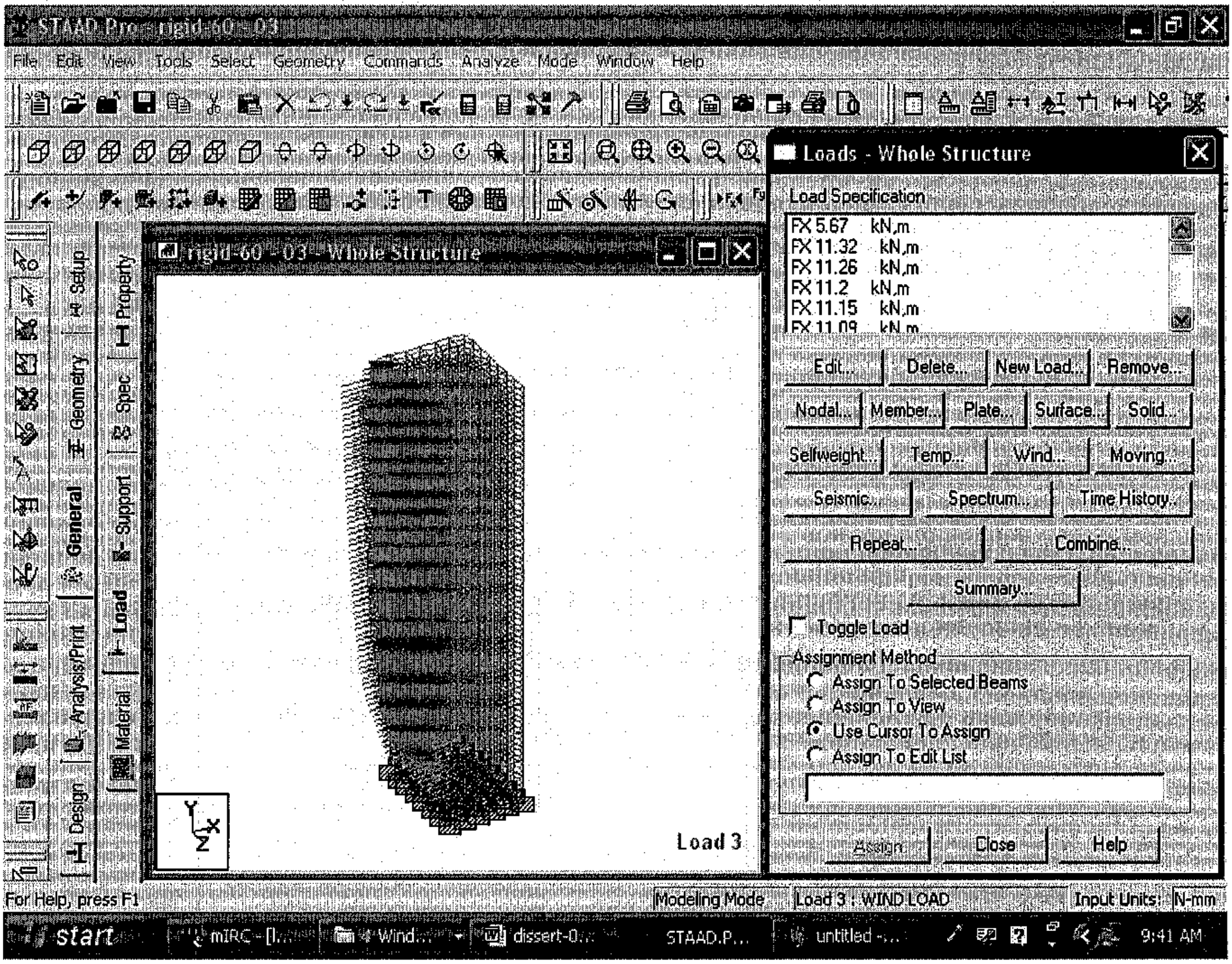

Figure IV: Displacement of the structure was shown in the post processing mode