CERTIFICATION OF APPROVAL

Analysis And Design Of A Multi-Storey Reinforced Concrete Structure Using Staad Pro And Robot Millennium

by

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

LEE TSE WENG

ABSTRACT

The study on comparative analysis and design of Reinforced Concrete Structures using Application Software available in UTP is presented in the Final Year Project. A Reinforced Concrete structure model, which is created with STAAD PRO and ROBOT MILLENNIUM are analyzed. To verify the effectiveness of these software, the Reinforced Concrete beams, columns and slabs are analyzed according to British Standard (BS) 8110. During the progress stage of the research, a few reinforced concrete structures examples have been analyzed and designed. These examples consist of 2 dimensional and 3 dimensional frame structures. It is observed in the analysis that, the operability and the result output has some slight difference. Geometrical and material modeling plays an important role in determining the accuracy of the results in the reinforced concrete analysis. The analysis result indicates that a study on local behavior and effects must be carried out to ensure better result. Later, the research will focus on the common results between the software, whereby certain degrees of variations will be compared with manual calculations. Finally, discussion will be made on the variations and recommendations will be suggested based on these analysis.

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CHAPTER 1
INTRODUCTION

1.1 BACKGROUND

In Malaysia, concrete plays an important role as building material in construction works. Concrete is a strong durable material, which made up from mixture of cement, sand, aggregates, and water with specific ratio standard can be formed into varied shapes and sizes. Nowadays, there is a high demand in construction development, and there is a need to accelerate the design process. Therefore, design software is used to speed up the analysis, design, detailing of structures in the design office. Precise methods of analysis of such as three-dimensional structures can effectively only are carried out using these design software. Thus, it can eliminate the tedious manual calculation works. However, the flood of analysis and design software in the market has aroused the question of the effectiveness in terms of analysis, design and detailing.

A lot of structural software is being used for design purposes in the market nowadays. In UTP, there are a few structural and design software purchased for the benefit of the structural engineering community in the university. However, up to this moment the software have not been implemented in any structural courses yet. It is important to verify the results obtained before it can be implemented in the course. Many aspects must be considered from analysis to design points of view. For this reason, some structural software in UTP will be analyzed thoroughly. It is then verified with manual calculation.

1.2 RESEARCH OBJECTIVE

The purpose of the research is to perform analysis and design of multiple structures building according to British Standard (BS) 8110 using design software. Throughout the research, the usage for the STAAD PRO and ROBOT MILLENNIUM can be determined. This research will help to improve understanding of the analysis and design of Concrete Building(s) and individual elements, design processes, design philosophy, method and approach. From the obtained results, the detailing ability embedded in these software can be identified and compared.

1.3 SCOPE OF STUDY

This research involved numerical and theoretical analysis. These analysis are based on load-deflection, load- strain and cracking behavior of the reinforced concrete structure.

The scope of the studies can be divided into: the study of Reinforced Concrete related to the research, the study of operation and usability and verification of results from STAAD PRO and ROBOT MILLENNIUM design software. Results obtained were then were analyzed and discussed.

CHAPTER 2
LITERATURE REVIEW

2.1 HISTORY OF REINFORCED CONCRETE

Concrete is a compound material made from sand, gravel and cement. The cement is a mixture of various minerals which when mixed with water, hydrate and rapidly become hard binding the sand and gravel into a solid mass. The oldest known surviving concrete is to be found in the former Yugoslavia and was thought to have been laid in 5,600 BC using red lime as the cement.

The first major concrete users were the Egyptians in around 2,500 BC and the Romans from 300 BC. It is from the Roman words 'caementum' meaning a rough stone or chipping and 'concretus' meaning grown together or compounded, that we have obtained the names for these two now common materials.[1]

In 1830, a publication entitled, "The Encyclopedia of Cottage, Farm and Village Architecture" suggested that a lattice of iron rods could be embedded in concrete to form a roof. Eighteen years later, a French lawyer created a sensation by building a boat from a frame of iron rods covered by a fine concrete which he exhibited at the Paris Exhibition of 1855. Steel reinforced concrete was now born.[2]

It is not only fire resistance that is improved by the inclusion of steel in the concrete matrix. Concrete, although excellent in compression, performs poorly when in tension or flexure. By introducing a network of connected steel bars, the strength under tension is dramatically increased allowing long, unsupported runs of concrete to be produced. Concrete also protects the steel, both physically and chemically.[2]

The Romans made many developments in concrete technology including the use of lime and Pozzolana concretes were used for nearly two millennium before the next major development occurred. In 1824 when Joseph Aspdin of Leeds took out a patent for the manufacture of Portland cement, so named because of its close resemblance to Portland stone. Aspdin's cement, made from a mixture of clay and limestone, which had been crushed and fired in a kiln, was an immediate success. Although many developments have since been made, the basic ingredients and processes of manufacture are the same today.[2]

This history clearly describes the importance of Reinforced Concrete as a building tool in construction material. Therefore, this fundamental process can be identified for future development application.

2.2 DEVELOPMENT OF BRITISH STANDARD CODES

The design procedure done for this research is according to British Standard (BS) Codes. For this reason, it is important to identify the guidelines information of this Code. This part of BS 8110: *Code of Practice for the Structural use of Concrete* has been prepared to replace CP110: Part 1:1972. This code covers the fields of CP110 and encompasses the structural use of reinforced and prestressed concrete both cast in situ and precast.

Although there are no major changes in principle from the previous edition, the text has largely been rewritten with alterations in the order and arrangement of topics.

The redrafting and alterations have been made in the light of experience of the practical convenience in using CP110. They have also been made to meet criticism of engineers preferring the form of CP114. In this respect sections two to five have been rewritten with shorter clauses, avoiding as much as possible lengthy paragraphs dealing with the matters that could be broken down into separate subclauses, to make specific references easier to understand. From this development, consideration had been given to include the load factor method, which had been introduced into CP114 in 1957.[3]

BS 8110 is divided into 3 parts:

Part 1: Code of Practice for Design and Construction. This section covers the design objectives and general recommendations, design and detailing for reinforced concrete and prestressed concrete. This section also provides important information on concrete: materials, specification and construction. Besides that, the specification and workmanship were also explained thoroughly.

Part 2: Code of Practice for Special Circumstances. This Part gives guidance on ultimate limit state calculations and the derivation of partial factors of safety, serviceability calculations with emphasis on deflections under loading and on cracking

Part 3: Design Charts for Singly Reinforced Beams, Doubly Reinforced and Rectangular Columns. The design charts in this section have been prepared in accordance with the assumption laid down in Part 1, with the intention that they may be used as standard charts and avoid duplication of effort by individual design offices.[3]

2.3 DESIGN SOFTWARE DEVELOPMENT

Since the research of this project is done on design software, it is important to identify the background history of these design software. These software were written by programming software called **FOR**MULA **TRAN**SLATOR (FORTRAN) and C++ Programming Language.

Software engineering revolution began since last 30 years ago. It all begins when FORTRAN was invented. This wonderful first FORTRAN compiler was designed and written from scratch in 1954-57 by an International Business Machine (IBM) team lead by John W. Backus and staffed with super-programmers. However, problems aroused because it was difficult to implement: they were more complicated than traditional finite difference methods, and often the data structures involved are not easily represented in the traditional procedural programming environments used in scientific computing.[4]

In order to solve this problem, a collection of libraries written in a mixture of Fortran and C++ Programming Language were used, In this approach, the high-level data abstractions are implemented in C++, while the bulk of the floating point work is performed on rectangular arrays by Fortran routines. The design approach used here is based on two ideas. The first is that the mathematical structure of the algorithm domain specified above maps naturally into a combination of data structures and operations on those data structures, which can be embodied in C++ classes. The second is that the mathematical structure of the algorithms can be factored into a hierarchy of abstractions, leading to an analogous factorization of the framework into reusable components, or *layers.*

Object oriented techniques, and C++ in particular, seem to be taking the software world by storm. Nevertheless, it seems that C++ itself is a major factor in this latest phase of the software revolution. C++ is a programming language suitable for real world projects that is also a more expressive software design language. This results in a more robust design, in essence a better-engineered design.[5]

With the aid from design software, engineers can get through with analysis, design, and detailing in the most convenient method. They are also pretty sure it can be built using accepted construction techniques. Before such a design is actually built the engineers do structural analysis; they build computer models and run simulations; they build scale models and test them.

In short, the software can give designers to make sure the design is a good design before it is built such as:

- Automatic calculation of all building dead loads from structural components.
- Automatic distribution of all uniform and/or concentrated slab loading onto supporting members.
- Automatic creation of necessary analysis models to perform complete building design including automatic pattern loading in accordance with building codes.
- All elements can be designed together in an automated batch design mode. Alternatively, you can interactively control the design of every element or element group
- Layout plans
- All slab reinforcement layouts in plan and/or in section.
- All beam elevation drawings including all reinforcement detailing
- Column Schedules and elevation drawings.
- Complete summary of all analysis output including lateral analysis summaries.
- Complete design calculations for all elements.
- Generation of all material quantities.

2.4 DESCRIPTION ON TALL BUILDINGS STRUCTURE.

2.4.1 INTRODUCTION.

Since the research of this project will analyze the RC structures, it will design the structure for tall building later on. Hence it is necessary to understand some criteria in designing this structure.

For the structural engineer the major difference between low and tall buildings is the influence of the wind forces on the behavior of the structural elements. Generally, a tall building structure is one in which the horizontal loads are an important factor in the structural design. In terms of lateral deflections a tall concrete building, which the structure, sized for gravity loads only, will exceed the allowable sway due to additionally applied lateral loads. This allowable drift is set by the code of practice. If the combined horizontal and vertical loads cause excessive bending moments and shear forces the structural system must be augmented by additional bracing elements.

The analysis of tall structures pertains to the determination of the influence of applied loads on forces and deformations in the individual structural elements such as beams, columns and walls. The design deals with the proportioning of these members. For reinforced concrete structures this includes sizing the concrete as well as the steel in an element. Structural analyses are commonly based on established energy principles assume linear elastic behavior of the structural elements. Non-linear behavior of the structure makes the problem extremely complex. It is very difficult to formulate, with reasonable accuracy, the problems involving inelastic responses of building materials.

At present the forces in structural components and the lateral drift of tall structures can be determined by means of elastic method of analysis regardless of the method of design. Non-linear methods of analysis for high-rise structures are not readily available.[6]

2.4.2 DESIGN AND ANALYSIS CONSIDERATIONS.

As stated in of BS 8110: Part 1, clause 2.1, the aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactory during their intended life. For multi-storey structures the imposed floor loads can be substantially reduced in the design of columns, walls, beams and foundations. Details are given in BS 6399: Part 1, clause 5. BS 8110 contains additional clauses for structures consisting of five storey or more.

a) Ultimate limit state

i. Structural stability

Tall slender frames may buckle laterally due to loads that are much smaller than predicted by buckling equations applied to isolated columns. Instability may occur for a variety of reasons such as slenderness, excessive axis loads and deformations, cracks, creep, shrinkage, temperature changes and rotation of foundations. Most of these are ignored in a first-order analysis of tall structures but may cause lateral deflections that are much larger than initially expected. The increased deformations can induce substantial additional bending moments in axially loaded members. This will increase the probability of buckling failure. In principle the instability of the multi-storey building structure is no different from that of a low structure but because of the great height of such buildings horizontal deflections must be computed with great accuracy. The deflected shapes of individual structural members should be taken into account in the final analysis of tall slender structures.

ii. Robustness

All structures should be capable of safely resisting a notional horizontal load applied at each floor or roof level simultaneously. In the design of tall structures it will also be necessary to identify key elements. These can be defined as important structural members whose failure will result in an extended collapse of a large part of the building.[5]

b) Serviceability limit state

Ideally the limit states of lateral deflection should be concerned with cases where the side sway can

i. limit the use of the structure
ii. influence the behavior of non-load bearing elements
iii. affect the appearance of the structure

c) Assumptions for analysis

The structural form of a building is inherently three-dimensional. The development of efficient methods of analysis for tall structures is possible only if the usual complex combination of many different types of structural members can be reduced or simplified whilst still representing accurately the overall behavior of the structure. A necessary first step is therefore the selection of an idealized structure that includes only the significant structural elements with their dominant modes of behavior. Achieving a simplified analysis of a large structure such as a tall building is based on two major considerations:

i. the relative importance of individual members contributing to the solution
ii. the relative importance of modes of behavior of the entire structure

The user of a computer program is a simple plane frame or a general finite element program, can usually assign any value to the properties of an element even if these are inconsistent with the actual with the actual size of that member. Several simplifying assumptions are necessary for the analysis of tall building structures subject to lateral loading. The following are the most commonly accepted assumptions.

1. All concrete members behave linearly elastically and so loads and displacements are proportional and the principle of superposition applies. Because of its own weight the structure is subjected to a compressive prestress and pure tension in individual members is not likely to occur;
2. Floor slabs are fully rigid in their own plane. Consequently, all vertical members at any level are subject to the same components of translation and rotation in the horizontal plane. This does not hold for very long narrow buildings and for slabs which have their widths drastically reduced at one or more locations;
3. Contribution from the out-of-plane stiffness of floor slabs and structural bents can be neglected;
4. The individual torsional stiffness of beams, columns and planar walls can be neglected;
5. Additional stiffness effects from masonry walls, fireproofing, cladding and other non-structural elements can be neglected;
6. Deformation due to shear in slender structural members can be neglected;
7. Connections between structural elements in cast-*in-situ* buildings can be taken as rigid;
8. Concrete structures are elastically stable.

One additional assumption that deserves special attention concerns the calculation of the structural properties of a concrete member. The cross-sectional area and flexural stiffness can be based on the gross concrete sections. This will give acceptable results at service loads but leads to underestimation of the deflections at yielding. In principle the bending stiffness of a structural member reflects the amount of reinforcing steel and takes account of cracked sections, which cause variations in the flexural stiffness along the length of the member. These complications, however, are usually not taken into account in a first-order analysis.[6]

2.5 CRITERIA IN DESIGNING TALL BUILDINGS STRUCTURE.

2.5.1 INTRODUCTION

A building which height creates different conditions in the design, construction and use than the conditions exist for common buildings of a certain region or period. For the structural engineer; a tall building can be defined as one whose structural system must be modified to make it sufficiently economical to resist lateral forces induce due to wind and earthquakes within the prescribe criteria for:

a.) Strength and stability
b.) Drift
c.) Comfort of occupants

The progression of lateral load resisting schemes from elemental beam and column assemblage towards the notion of an equivalent vertical cantilever is a fundamental to any structural system methodology.

At one end of spectrum there are moment resisting frames, which are efficient for buildings in the range of 20 to 30 stories; at the other end there is the generation of tubular systems were placed with the idea that the application of any particular form is economical only over a limited range of building heights.[6]

2.5.2 TALL BUILDINGS STRUCTURE CLASSIFICATION

The classification of tall buildings could be based on certain engineering and system criteria, which define both the physical as well as the design aspects of the building:

a) ***Materials:*** steel, concrete, and composites
b) ***Gravity load resisting systems***: floor framing (beam, slabs), columns, trusses and foundations
c) ***Lateral load resisting system***: walls, frames, trusses diaphragms
d) ***Type and magnitude of lateral loads***: wind, seismic
e) ***Strength and serviceability requirements***: drift, acceleration, ductility

In 1984, a rigorous methodology for cataloguing of tall buildings with respect to their structure systems has been developed. The classification involves four distinct levels of framing oriented divisions:

a) Primary Framing System
b) Bracing Sub-System
c) Floor Framing
d) Configuration and Load Transfer

2.5.3 FACTORS AFFECTING GROWTH, HEIGHT, AND STRUCTURAL FORM OF TALL BUILDINGS

The feasibility and desirability of high-rise structures have always depended on:

a) the available materials
b) the level of construction technology
c) the state of development of the services necessary for the use of the building

As a result significant advances have occurred from time to time with the advent of a new material, construction facility, or form of service. The main reasons behind the rapid growth of high-rise buildings were:

a) The socio-economic problems that followed industrialization development
b) Increasing demand for space in growing major cities

Development in the high-rise building design and construction is due to:

a) Different structural systems, which have gradually evolved for residential and office buildings, reflecting their differing functional requirements.
b) Advancements in the major construction materials and other services.
c) Advancement in construction machineries, methods and techniques, particularly pre-cast technology.
d) Development of 4^{th} generation structural software and IT technology, etc.

2.6 STRUCTURAL DESIGN CRITERIA AND PHILOSOPHY

The structural design criteria for tall buildings define the following aspects, which control the design:

a) Structural Loading
b) Structural Materials
c) Structural System

2.6.1 STRUCTURAL LOADING

The term load refers to any effect that result in a need for some resistive efforts on the part of the structure. There are many sources of loads and many ways in which they can be classified. The principal kinds and sources of loads on building structures are the following:

i) Gravity
ii) Wind
iii) Earthquake

iv) Hydraulic pressure
v) Soil pressure
vi) Thermal Changes
vii) Shrinkage
viii) Vibration
ix) Internal Actions
x) Handling

2.6.2 STRUCTURAL MATERIALS

In studying or designing a structure, particular properties of materials are concern. These critical properties may split into:

a) Essential structural properties
b) General properties

Essential structural properties include:

i) Strength
ii) Deformation
iii) Hardness
iv) Fatigue resistance
v) Uniformity of physical structure
vi) Creep, shrinkage, and temperature effects

General properties are:

i) Form
ii) Weight
iii) Fire resistance
iv) Coefficient of thermal expansion
v) Durability
vi) Workability
vii) Appearance
viii) Availability and cost

2.6.3 STRUCTURAL SYSTEMS

For selecting a structural systems and optimized design, following are the necessary considerations.

a) Strength and Stability
b) Stiffness and Drift Limitations
c) Human Comfort Criteria

a) Strength and Stability

For the ultimate limit state, 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.

b) Stiffness and Drift Limitations

The provision of adequate stiffness, particular lateral stiffness, is the major consideration in the design of tall building for several important reasons. In terms of serviceability limit state:

i) Deflection must be maintained at a sufficiently low level to allow the proper functioning of non-structural components, such as elevators, doors, etc.

ii) 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, infill, cladding or glazing.

iii) The structure must be sufficiently stiff to prevent dynamic motions to becoming large enough to cause discomfort to occupants, prevent delicate work being undertaken

One parameter that can estimate the lateral stiffness of a building is the drift index, defined as the ratio of maximum deflection at the top of building to the total building height. The control of lateral deflections is particular importance for modern buildings.

c) Human Comfort Criteria

If a tall flexible structure is subjected to lateral or torsion deflections under the action of wind loads, the resulting oscillatory movements can induce a wide range of responses in the building occupants. 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.[6,7]

2.7 RIGID FRAME STRUCTURES

2.7.1 INTRODUCTION

Rigid frame high-rise structure comprises parallel arranged bents consisting of columns and beams with moment resistant joints. Resistance to horizontal loading is provided by the bending resistance of the columns, beams and joints.

2.7.1.1 RIGID FRAME BEHAVIOUR

The horizontal stiffness of a rigid frame is governed mainly by the bending resistance of the beams, the columns, and the connections, and, in a tall frame, by the axial rigidity of the columns.

The accumulated horizontal shear above any storey of a rigid frame is resisted by shear in the columns of that storey as shown in Figure 2.1 below.

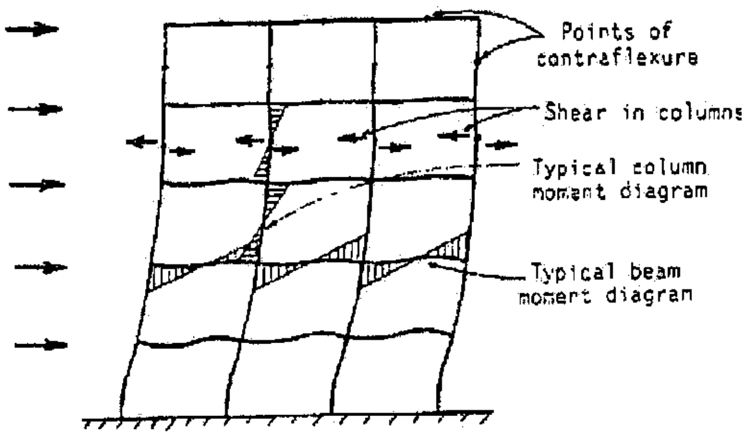


Figure 2.1 Forces and Deformations caused by external shear

The s hear c auses b y s torey-height c olumns t o b end i n d ouble c urvature, w ith point of contraflexure at approximately mid span. These deformations of the columns and beams allow raking of the frame and horizontal deflection in each storey. The overall deflected shape of a rigid frame structure due to raking has a shear configuration with concavity upwind, a maximum inclination near the base, and a minimum inclination at the top. This mode of frame deflection is also called shear mode, and such frames may be framed as shear frames.

The overall moment of the external horizontal shear is resisted in each storey level by the couple resulting from the axial tensile and compressive forces in the columns on opposite sides of the structure as shown in Figure 1.2.

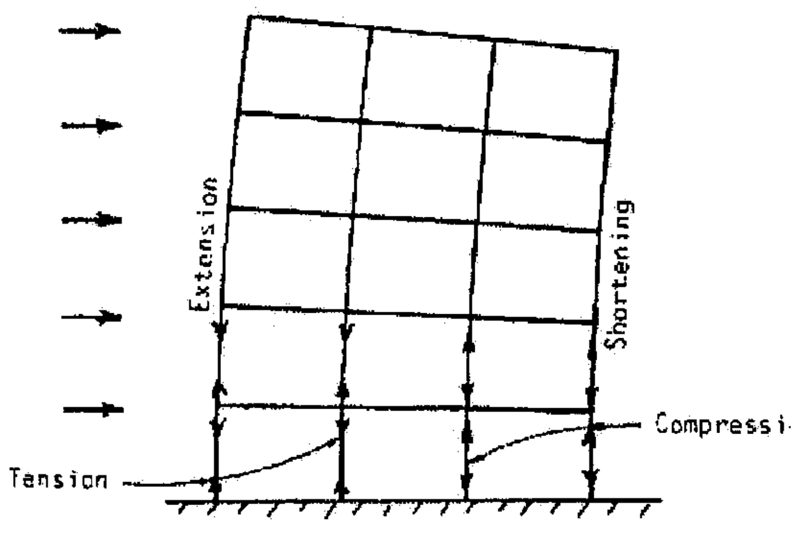


Figure 2.2: Forces and Deformations caused by external moments

The external and shortening of columns cause overall bending and associated displacements of the structure. The contribution of overall bending to the total drift, however, will usually not exceed 10% of that raking, except in very tall, slender rigid frames. Therefore the overall deflected shape of a high-rise rigid frame usually has a shear configuration.

2.7.2 ANALYSIS OF RIGID FRAME STRUCTURE

As highly redundant structures, rigid frames are designed initially on the basis of approximate analysis, after that a detailed analysis and checks are made. The procedure may typically include the following stages:

i. Estimation of gravity load forces in beams and columns by approximate method.
ii. Preliminary estimate of member sizes based on gravity load forces with arbitrary increase in sizes to allow for horizontal loading.
iii. Approximate allocation of horizontal loading to bents and preliminary analysis of member forces in bents.
iv. Check on drift and adjustment of member sizes if necessary.
v. Check on strength of members for worst combination of gravity and horizontal loading, and adjustment of member sizes if necessary.
vi. Computer analysis of total structure for more accurate check on member strengths and drift, with further adjustment of sizes where required. This stage may include the second-order P-Δ effects of gravity loading on the member forces and drift.
vii. Detailed design of members and connections.

2.7.2.1 APPROXIMATE DETERMINATION OF MEMBER FORCES CAUSED BY GRAVITY LOADING

Since a rigid frame is highly redundant; consequently, an accurate analysis can be made only after the member sizes are assigned. Initially therefore member sizes are decided on the basis of approximate forces estimated either by conservative formulas or by simplified method of analysis that are independent of member properties.

a) Determination of Beam Forces Using Code recommended Formulas

Code recommended formulas for determining the beam forces can be used upon the following conditions:

i) These are applicable of two or more spans, when the longest span does not exceed the shortest by more than 20%.
ii) The uniformly distributed design live load does not exceed three times the dead load.

2.7.2.2 APPROXIMATE ANALYSIS OF MEMBER FORCES CAUSED BY HORIZONTAL LOADING

a) Allocation of Loading Between Bents

A first step in approximate analysis of a rigid frame is to estimate the allocation of the external horizontal force to each bent. The loading will come from Wind Analysis.

b) Member Force Analysis by Portal Method

The portal method allows an approximate analysis for rigid frames without having to specify member sizes and therefore, it is very useful for a preliminary analysis.

This method is most appropriate to rigid frames that deflect directly by raking. Therefore, it is suitable for structure of moderate slenderness and height, and is commonly recommended as useful structures up to 25 storeys height, and a height to width ratio not greater than 4:1.

It is analogous between a set of single single-bay portal frames and a single storey or multi-bay rigid frames as shown in Figure 2.3a and b.

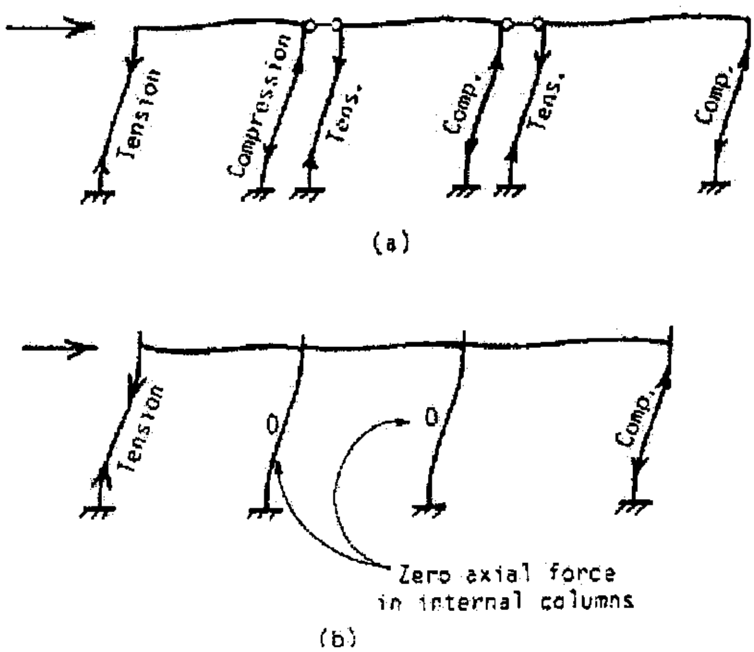


Figure 2.3(a): Separate Portal Analogy (b) Separate Portal Superposed

When each of the separate portals carries a share of the horizontal shear, tension occurs in the windward columns and compression in the leeward columns. If these are superposed to simulate the multi-bay frame, the axial forces of the interior columns are eliminated.

The analysis is based on the following assumptions:

i. Horizontal loading on the frame causes double curvature bending of all the columns and beams, with point of contraflexure at mid height of columns and mid span of the beams.
ii. The horizontal shear at mid storey levels is shared between the columns in proportion to the width of passageway each column support.

The method is used to analyze the whole frame, or just a portion of the frame at a selected level. The analysis of the whole frame considers in turn the equilibrium of separate frame modules, each module consisting of a joint with its column and beam segments extending to the nearest points of contraflexure. The sequence of analyzing the modules is from left to right, starting at the top and working down to the base.

The procedure for a whole frame analysis is as follows:

i. Draw a line diagram of the frame and indicate on it the horizontal shear at each mid-story level.

ii. In each story allocate the shear to the columns in proportion to the aisle widths they support, indicating the values on the diagram.

iii. Starting with the top-left module, compute the maximum moment just below the joint from the product or the column shear and the half-storey height.

iv. Find the girder-end moment just to the right of the joint from the equilibrium of the column and girder moments at the joint. The moment at the other end of the girder is of the same magnitude but corresponds to the opposite curvature.

v. Evaluate the girder shear by dividing the girder end-moment by half the span.

vi. Consider next the equilibrium of the second joint, repeating steps iii to v to find the maximum moment in the second column, and the moment and shear in the second girder from the left.[7]

2.8 PROSPECT OF WIND-DRIVEN NATURAL VENTILATION IN TALL BUILDINGS.

a) Wind Climate of Peninsular Malaysia

The mean surface winds over peninsular Malaysia are generally mild, with the mean speed of about 1.5 m/s, and a maximum speed of less than 8 m/s. The main direction is variable.[8]

2.9 WIND ANALYSIS

Blowing wind tends to exert loads on buildings and other structures exposed to the wind blowing. The amount of loads induced by wind loading depends on:

a. Wind Speed

b. Building Geometry and Configuration

c. Site Location and Topographical Condition

2.9.1 WIND SPEED

a) Basic Wind Speed (V)

According to BS 6399-2, 1997, a basic wind speed is the hourly, mean wind speed at height of 10 m over completely flat terrain at sea level that would occur if the roughness of the terrain was uniform everywhere.

b) Site Wind Speed (V_b)

The basic wind speed modified to account for the altitude of the site and the direction of wind being considered.

c) Effective (Design) Wind Speed (V_s)

The site wind speed modified to gust speed by taking account of the effective height, the size of the building or structural elements.

2.9.2 BACKGROUND

A moving mass of air has kinetic energy; the amount of this energy is directly proportional to the square of the wind velocity:

$$\boxed{KE = \frac{1}{2} mV^2}$$

Where, KE, is the kinetic energy, m, is the wind mass, and V, is the wind velocity. This kinetic energy translates into strain energy when it encounters a stationary object, such as buildings, through deformations induced in that object.

BS Codes presented the following simplified procedure of wind analysis of building structures. The design wind speed, V_b is converted into dynamic pressure, q_i at different levels of a building as shown in figure below using the formula:

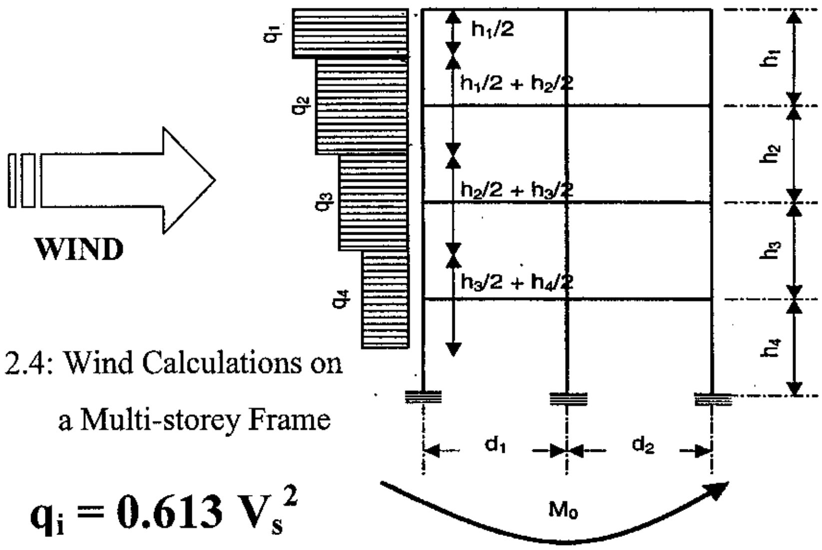


Figure 2.4: Wind Calculations on a Multi-storey Frame

$$q_i = 0.613\ V_s^2$$

Where:

q_i = is dynamic pressure in KN/m^2
V_b = is basic wind speed of a given site in m/s
V_s = is design wind speed in m/s

Where:

$V_s = S_1S_2S_3V_b$

Where, $S_1S_2S_3$ are given in the table

Wind velocities and pressures

Characteristic wind pressure

w_k $0.613V_s^2$ N/m²
where:

V_s design wind speed in m/s $= VS_1S_2S_3$

V basic wind speed in m/s (read from adjoining map)

S_1 multiplying factor relating to topology

S_2 multiplying factor relating to height above ground and wind braking

S_3 multiplying factor related to life of structure

Relation between design wind speed V_s and characteristic wind pressure w_k

V_s (m/s)	w_k (N/m²)
10	61
12	88
14	120
16	157
18	199
20	245
22	297
24	353
26	414
28	481
30	552
32	628
34	709
36	794
38	885
40	981
42	1080
44	1190
46	1300
48	1410
50	1530
52	1660
54	1790
56	1920
58	2060
60	2210
62	2360
64	2510
66	2670
68	2830
70	3000

Values of factor S_1

S_1 may generally always be taken as unity except in the following cases:
On sites adversely affected by very exposed hill slopes and crests where wind acceleration is known to occur: $S_1 = 1.1$
On sites in enclosed steep-sided valleys completely sheltered from winds: $S_1 = 0.9$

Values of factor S_3

S_3 is a probability factor relating the likelihood of the design wind speed being exceeded to the probable life of the structure. A value of unity is recommended for general use and corresponds to an excessive speed occurring once in fifty years.

Values of factors S_2

Structure		Topo-graphical factor	Height of structure h (m): 5	10	15	20	30	40	50	60	80	100	120	140	160	180	200
Cladding etc.		1	0.88	1.00	1.03	1.06	1.09	1.12	1.14	1.15	1.18	1.20	1.22	1.24	1.25	1.26	1.27
		2	0.79	0.93	1.00	1.03	1.07	1.10	1.12	1.14	1.17	1.19	1.21	1.22	1.24	1.25	1.26
		3	0.70	0.78	0.88	0.95	1.01	1.05	1.08	1.10	1.13	1.16	1.18	1.20	1.21	1.23	1.24
		4	0.60	0.67	0.74	0.79	0.90	0.97	1.02	1:05	1.10	1.13	1.15	1.17	1.19	1.20	1.22
Maximum vertical or maximum horizontal dimension	≯ 50 m	1	0.83	0.95	0.99	1.01	1.05	1.08	1.10	1.12	1.15	1.17	1.19	1.20	1.22	1.23	1.24
		2	0.74	0.88	0.95	0.98	1.03	1.06	1.08	1.10	1.13	1.16	1.18	1.19	1.21	1.22	1.24
		3	0.65	0.74	0.83	0.90	0.97	1.01	1.04	1.06	1.10	1.12	1.15	1.17	1.18	1.20	1.21
		4	0.55	0.62	0.69	0.75	0.85	0.93	0.98	1.02	1.07	1.10	1.13	1.15	1.17	1.19	1.21
	> 50 m	1	0.78	0.90	0.94	0.96	1.00	1.03	1.06	1.08	1.11	1.13	1.15	1.17	1.19	1.20	1.21
		2	0.70	0.83	0.91	0.94	0.98	1.01	1.04	1.06	1.09	1.12	1.14	1.16	1.18	1.19	1.21
		3	0.60	0.69	0.78	0.85	0.92	0.96	1.00	1.02	1.06	1.09	1.11	1.13	1.15	1.17	1.18
		4	0.50	0.58	0.64	0.70	0.79	0.89	0.94	0.98	1.03	1.07	1.10	1.12	1.14	1.16	1.18

Notes

h is height (in metres) above general level of terrain to top of structure or part of structure. Increase to be made for structures on edge of cliff or steep hill.

Topographical factors

1. open country with no obstructions
2. open country with scattered wind-breaks
3. country with many wind-breaks; small towns; suburbs of large cities
4. city centres and other environments with large and frequent obstructions.

Figure 2.5: Wind velocities tables

CHAPTER 3
METHODOLOGY

Throughout the project, these steps have be taken to ensure the completion of tasks:

1. Following the examples for each software program to ensure the appropriate ways to operate with the software.

2. Extending the knowledge from **Step 1** to solve simple problem. Each problem will give more understanding on how the analysis and design is achieved.

3. Verifying the results obtained from simple structure.

4. Using the software to solve multiple Reinforced Concrete structure problems.

5. Analyzing the results from multiple structures.

6. Discussion and recommendation will be made according to the results.

3.1 THE DESIGN PROCESS

Design in any field is a logical creative process, which requires a wide variety of skills. As a complete process, structural engineering design can be divided into three main stages:

a. Conceptual design
b. Preliminary analysis and design
c. Detailed analysis and design

The first stage consists of the drawing up the structural schemes, which are safe, buildable, economical and robust. The second stage consists of performing preliminary calculations to determine if the proposed structural schemes are feasible. Rules of thumb are used to determine preliminary sizes for the various members and approximate methods used to check these sizes and to estimate the quantities of reinforcement required. In the third stage, the adequacy of the preliminary member sizes is verified and the quantities of reinforcement calculated accurately.[10]

Following completion of these stages, drawings and specifications are prepared for the construction of the chosen structure.

3.2 ANALYSIS OF FRAMES (MANUAL CALCULATION)

Most concrete buildings contain a structure of beams and columns which, when rigidly connected, make up a continuous frame. The framework of this building concealed behind wall panels which protect the occupants of the building from the external environment.

The analysis of a complete three-dimensional frame can be carried out by hand or by computer using any appropriate method such as the stiffness method. However, the, mathematical complexity of the solution process generally makes it unfeasible to analyze a complete three-dimensional structure by hand. Even when analyzing by computer, the solution may become unduly complex.

One particular aspect of analysis which makes it as yet impractical to design a complete three-dimensional structure is the need to consider all possible arrangements of load. In theory, every possible combination of permanent, variable and wind loading must be considered to determine the critical load effects in each member. The greater the number of members in the frame, the greater the number of possible combinations of applied load. For this reason, certain assumptions and simplifications are commonly made before the structure is analyzed.

In order to overcome the complexity, of considering the full multi-storey skeletal structure and to facilitate frame with smaller, two-dimensional sub-frames. This substantially reduces the total number of load cases which must be considered for each sub-frame and simplifies the process of describing the structural model to the computer. The precise method of simplification depends on whether or not the original frame is braced against horizontal loads. A frame which is braced against horizontal loads using substantial bracing members is termed as non-sway frame.

Owing to the presence of such stiff bracing members, there is little or no lateral deflection in non-sway frame. For this reason, such a frame is designed to resist only the applied vertical loads. A frame that undergoes significant horizontal deflection under applied horizontal loads especially wind load is known as a sway frame. Sway frames must be designed to resist both vertical and horizontal loads.[11]

3.2.1 ANALYSIS OF NON-SWAY FRAMES

The first simplification which can be made is to assume that, in the E-W direction, the frame can be represented by three two-dimensional non-sway frames. Note that the vertical loadings for the two outer plan frames are the same and hence only one need to be analyzed. The central plan frame carries a greater vertical load since it supports a greater floor area.

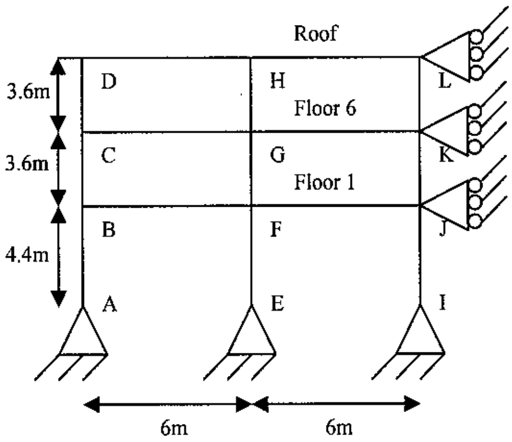


Figure 3.1: Two dimensional Sub-frame

The plane frame can be readily be analyzed by computer for each possible arrangement of load. However, two alternative methods are available for further simplifying the plane frame to facilitate a hand solution.

The first of these methods is to divide the plane frame into a set of sub-frames, each of which is analyzed separately. Each sub-frame is made up of the beams at one level together with the columns connected to these beams. The plane frame can be divided into the three sub-frames below. The columns meeting the beams are assumed to be fixed at their ends.[9]

These sub-frames can readily be analyzed by hand using the moment distribution method to give the moments, shears, etc., in both beams and the columns.

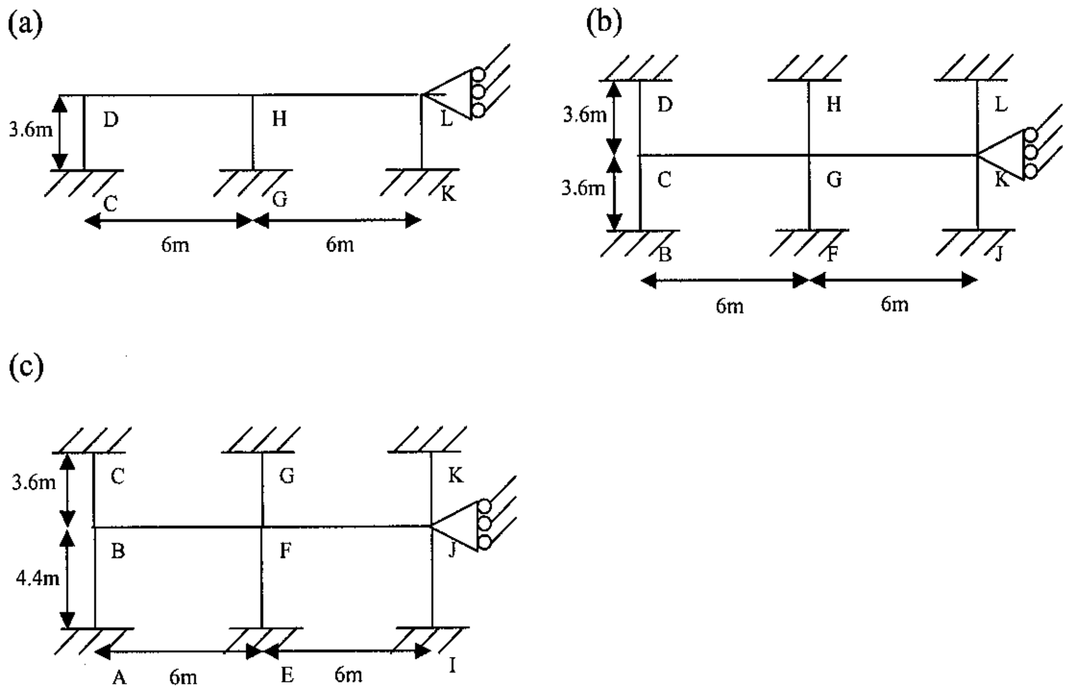


Figure 3.2: Sub-frames for the frame of Figure :(a)top; (b) middle; (c) bottom

DL

Slab finishes = 0.5 x 1.7 KN/m^2 x 3m	= 2.55 KN/m
Wall Load	= 15.12 KN/m
Beam Self-Weight = 0.2m x 0.45m x 24 KN/m^3	= 2.16 KN/m
Total	= 19.83 KN/m

LL

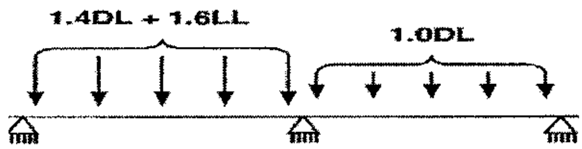
Imposed = 0.5 x 3 KN/m^2 x 3m = 4.5 KN/m

Ultimate Load

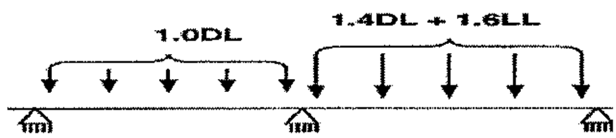
W = 1.4 DL +1.6 LL = 34.96 KN/m

3.2.1.1 CRITICAL LOADING ARRANGEMENT

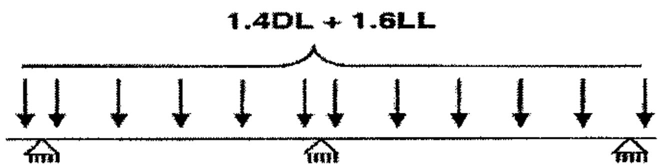
For analysis of continuous beam and/or slabs, load is arranged in different manners of load patterns, in order to get the most unfavorable response of the structure. Typical load patterns are shown as:



LOAD PATTERN-1



LOAD PATTERN-2

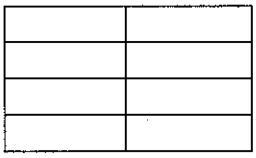


LOAD PATTERN-3

Figure 3.3: Load Pattern Arrangement

3.2.2 ANALYSIS OF SWAY FRAMES

Wind Analysis



12m

12m Figure 3.4

Height = 3.6m each floor

Figure 3.4 : Plan view of single floor

$q = 0.613Vs^2$
Vs = S1S2S3Vb
S1 = S3 = 1 Vb = 8 m/s (maximum wind velocity in Malaysia)
Vs =S2(8m/s)

Table 3.1: Wind Load acting on each floor at different height

hi(m)	S2	Vs(m/s)	q1(N/m²)	q1(KN/m²)	Point Load(KN)
44.00	0.950	7.60	35.41	0.0354	**0.191**
40.40	0.941	7.53	34.74	0.0347	**0.375**
36.80	0.919	7.35	33.13	0.0331	**0.358**
33.20	0.890	7.12	31.08	0.0311	**0.336**
29.60	0.861	6.89	29.08	0.0291	**0.314**
26.00	0.828	6.62	26.90	0.0269	**0.290**
22.40	0.792	6.34	24.61	0.0246	**0.266**
18.80	0.756	6.05	22.42	0.0224	**0.242**
15.20	0.714	5.71	20.00	0.0200	**0.216**
11.60	0.668	5.34	17.51	0.0175	**0.189**
8.00	0.617	4.94	14.94	0.0149	**0.161**
4.40	0.567	4.54	12.61	0.0126	**0.151**

Wind load per floor:
At typical levels q1 x 3.0 x 3.6
At the roof level q1 x 3.0 x 1.8
At the ground level q1 x 3.0 x (1.8+2.2)
Shear in the top story = 0.191KN

3.2.2.1 LATERAL FORCE CALCULATION

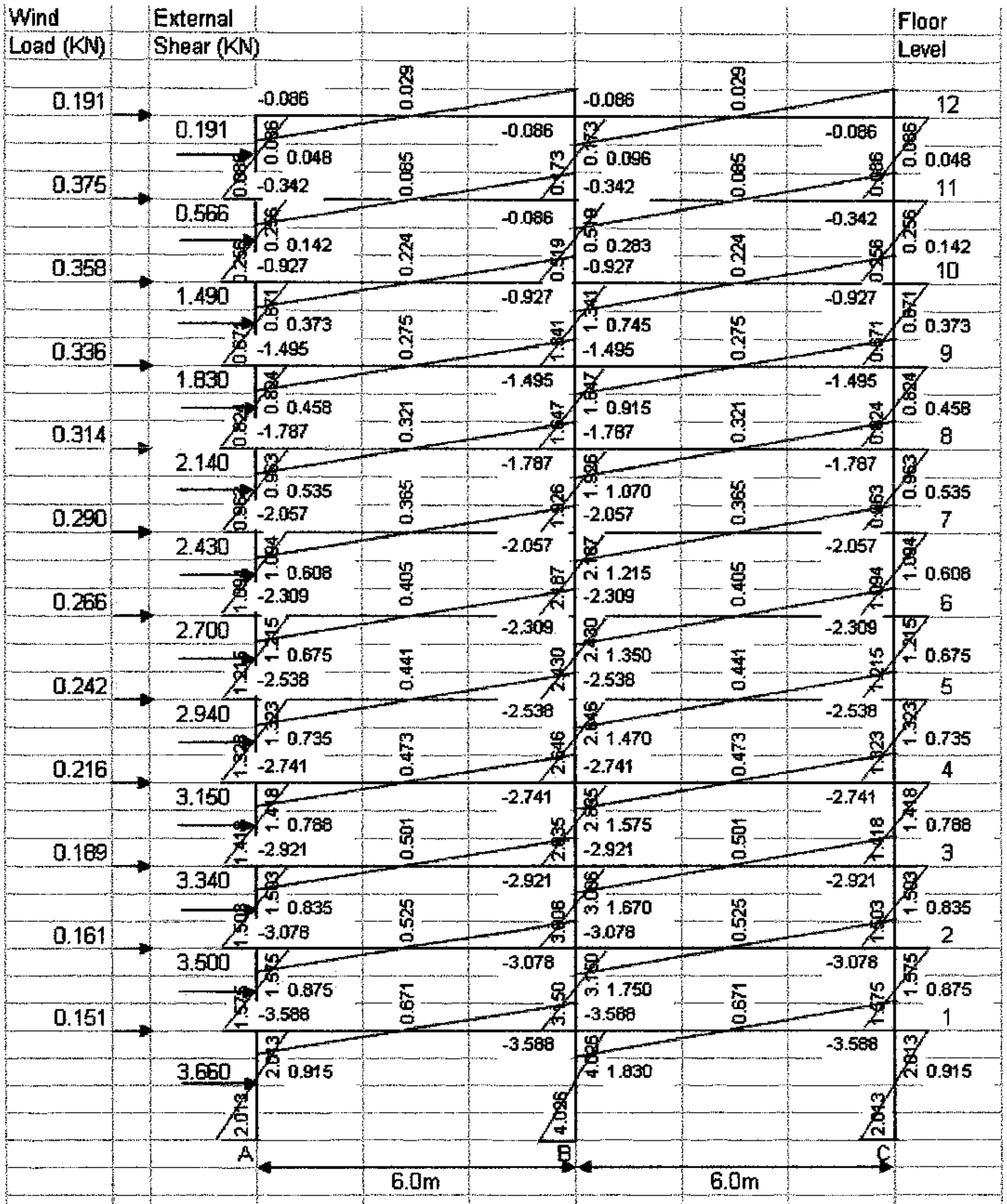


Figure 3.5: Wind analysis using Portal Method

3.2.2.2 METHODS OF CALCULATIONS

Distributing this shear between the top-story columns in proportion to the widths of aisle supported:

For column A: 0.191 x 3/12 = 0.048 KN

For column B: 0.191(3 + 3)/12 = 0.096 KN

For column C: 0.191 x 3/12 = 0.048 KN

The shear in columns of respective stories is allocated.

Moment at top of column = column shear x half-story height

= 0.048 x 1.8 = 0.0864 KNm

From moment equilibrium of the joint, the moment at left end of first girder

= -0.0864 KNm

Shear in girder = girder-end moment/half girder length

= 0.0864/3 = 0.029KN

Because of the mid-length point of contra flexure, the moment at the right end of the girder has the same value as at the left end. Similarly, the column moments at the top and bottom of a story are equal. The sign convention for numerical values of the bending moment is that an anticlockwise moment applied by a joint to the end of a member is taken as positive.

Moment at top of column = column shear x half-story height

= 0.096 x 1.8 = 0.173 KNm

From moment equilibrium of the joint, the moment at left end of second girder

$$= -(0.173-0.0864)= -0.0864 \text{ KNm}$$

Shear in second girder = girder moment/half girder length

$$= 0.0864/3 = 0.029\text{KN}$$

CHAPTER 4
RESULTS AND DISCUSSION

4.1 COMPARISON: SOFTWARE RESULTS TO THEORETICAL RESULTS

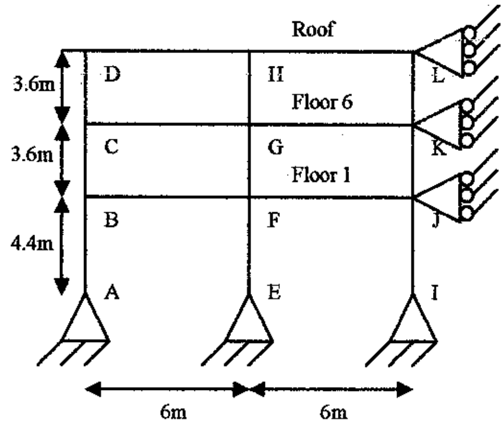


Figure 4.1: Two dimensional Sub-frame

FLOOR 1 BEAMS
MEMBER BF

Table 4.1: Reaction and Moment results for floor 1 beam BF

Item	Position	Theoretical	Staad Pro	(%)	Robot	(%)
Reaction (KN)	At left-hand support	108.34	102.23	5.6	-	-
	At right-hand support	-103.25	-107.54	4.2	-108.48	5.1
Moment (KNm)	At left-hand support	96.47	92.26	4.4	-	-
	At right-hand support	90.58	108.20	19.5	109.03	22.4

MEMBER FJ

Table 4.2: Reaction and Moment results for floor 1 beam FJ

Item	Position	Theoretical	Staad Pro	(%)	Robot	(%)
Reaction (KN)	At left-hand support	106.51	105.92	0.6	-	-
	At right-hand support	-108.34	-103.85	4.1	-105.60	2.5
Moment (KNm)	At left-hand support	83.40	103.55	24.2	104.05	24.8
	At right-hand support	103.65	97.32	6.1	-	-

FLOOR 1 COLUMNS (4.4m)

Table 4.3: Moment results for floor 1 columns

Member	Theoretical	Staad Pro	(%)	Robot	(%)
AB	-45.40	-34.03	25.0	-37.96	16.4
BC	54.66	57.87	5.9	56.86	4.0
IJ	-45.40	-36.38	19.9	-39.69	19.9
JK	54.66	60.94	11.5	59.70	9.2

FLOOR 6 BEAMS
MEMBER CG

Table 4.4: Reaction and Moment results for floor 6 beam CG

Item	Position	Theoretical Typical Floor [1]	Staad Pro	(%)	Robot	(%)
Reaction (KN)	At left-hand support	109.53	99.80	8.9	-	-
	At right-hand support	-104.53	-109.97	5.2	-109.57	4.8
Moment (KNm)	At left-hand support	66.83	70.83	6.0	-	-
	At right-hand support	59.81	101.33	69.4	102.71	71.7

[1] Indicates calculation from subframe with 3.6m height, whereas the software generate the exact floor 6 level.

MEMBER GK

Table 4.5: Reaction and Moment results for floor 6 beam GK

Item	Position	Theoretical Typical Floor	Staad Pro	(%)	Robot	(%)
Reaction (KN)	At left-hand support	105.23	108.78	3.4	105.56	0.3
	At right-hand support	-109.53	-101.00	7.8	-	-
Moment (KNm)	At left-hand support	59.81	98.27	64.3	99.04	65.6
	At right-hand support	66.83	74.93	12.1	-	-

FLOOR 6 COLUMNS

Table 4.6: Moment results for floor 6 columns

Member	Theoretical	Staad Pro	(%)	Robot	(%)
BC	-33.42	-32.44	2.9	-32.10	3.9
CD	33.42	38.38	14.8	38.25	14.5
JK	-33.42	-34.69	3.8	-34.79	4.1
KL	33.42	40.24	20.4	40.58	21.4

ROOF BEAMS
MEMBER DH

Table 4.7: Reaction and Moment results for floor 12 beam DH

Item	Position	Theoretical	Staad Pro	(%)	Robot	(%)
Reaction (KN)	Cantilever support	6.59	6.59	0	6.66	1.1
	At left-hand support	20.34	28.87	41.9	29.00	42.6
	At right-hand support	-19.80	-10.69	46.0	-	-
Moment (KNm)	Cantilever support	0.55	3.30	83.3	3.33	84.2
	At left-hand support	8.43	35.04	75.9	35.15	76.0
	At right-hand support	8.16	-19.50	58.1	-	-

MEMBER HL

Table 4.8: Reaction and Moment results for floor 12 beam HL

Item	Position	Theoretical	Staad Pro	(%)	Robot	(%)
Reaction (KN)	At left-hand support	19.74	10.63	46.1	-	-
	At right-hand support	-20.34	-28.94	42.3	-29.08	43.0
	Cantilever support	6.59	6.59	0	6.66	1.1
Moment (KNm)	At left-hand support	8.16	-19.66	58.5	-	-
	At right-hand support	8.43	35.26	76.1	35.42	78.4
	Cantilever support	0.55	3.30	83.3	3.33	84.2

ROOF COLUMNS

Table 4.9: Moment results for floor 12 columns

Member	Theoretical	Staad Pro	(%)	Robot	(%)
CD	-8.43	-31.74	73.4	-38.55	78.1
KL	-8.43	-31.97	73.4	-38.72	78.2

4.2 COMPARISON: SOFTWARE DETAILING RESULTS

FLOOR 1 BEAMS

Table 4.10: Reinforcement details for floor 1 beams

	Layers Position						
Source	**Top End**	**Bottom End**	**Top Middle**			**Bottom End**	**Top End**
	Left	**Right**	**Left**	**Center**	**Right**	**Right**	**Right**
Staad Pro	2T20	2T16	2T20	3T20	2T20	2T16	2T20
Robot Millennium	2T12+ 2T12	2T16	2T20	2T20+ 2T8	2T20	2T16	2T20

FLOOR 6 BEAMS

Table 4.11: Reinforcement details for floor 6 beams

	Layers Position						
Source	**Top End**	**Bottom End**	**Top Middle**			**Bottom End**	**Top End**
	Left	**Right**	**Left**	**Center**	**Right**	**Right**	**Right**
Staad Pro	2T20	4T12	2T20	3T20	2T20	4T12	2T20
Robot Millennium	3T12+ 1T12	3T12	3T12+ 2T8	3T12+ 2T12	3T12+ 2T8	3T12	3T12+ 1T12

FLOOR 12 BEAMS

Table 4.12: Reinforcement details for floor 12 beams

	Layers Position						
Source	**Top End**	**Bottom End**	**Top Middle**			**Bottom End**	**Top End**
	Left	**Right**	**Left**	**Center**	**Right**	**Right**	**Right**
Staad Pro	2T12						
Robot Millennium	2T16+ 2T16	2T16	2T8	-	2T8	2T16	2T16+ 2T16

COLUMNS

Table 4.13: Reinforcement details for columns with different sizes

	Size		
Source	**250mmx250mm**	**350mmx250mm**	**400mmx400mm**
Staad Pro	8T16		
Robot Millennium	8T12	6T12	6T12

4.3 DETAILING BEHAVIOR: LOAD INCREMENT ANALYSIS ON BEAM

STAAD PRO

Table 4.14: Reinforcement details under different live load imposed using Staad Pro

	Layers Position						
Live Load (KN/m)	**Top End**	**Bottom End**	**Top Middle**			**Bottom End**	**Top End**
	Left	**Left**	**Left**	**Center**	**Right**	**Right**	**Right**
4.50	2T20	2T16	2T20	3T20	2T20	2T16	2T20
6.00	3T20	2T16	2T20	3T20	2T20	2T16	3T20
7.50	3T20	2T16	2T20	3T20	2T20	2T16	3T20
9.00	3T20	2T12+2T12	2T20	3T20	2T20	2T12+2T12	3T20
10.50	3T20	2T12+2T12	2T20	3T20	2T20	2T12+2T12	3T20
12.00	2T32	3T16	2T32	2T32	2T32	3T16	2T32
13.50	2T32	3T16	2T32	2T32	2T32	3T16	2T32

ROBOT MILLENNIUM

Table 4.15: Reinforcement details under different live load imposed using Robot Millennium

	Layers Position						
Live Load (KN/m)	**Top End**	**Bottom End**	**Top Middle**			**Bottom End**	**Top End**
	Left	**Left**	**Left**	**Center**	**Right**	**Right**	**Right**
4.50	2T12+ 2T12	2T16	2T20	2T20+ 2T8	2T20	2T16	2T20
6.00	3T12+ 2T12	3T12	3T12+ 2T8	3T12+ 3T12	3T12+ 2T8	3T12	3T12+ 3T12
7.50	2T20	2T12	2T16+ 2T8	2T16+ 2T16	2T16+ 2T8	2T12	2T16+ 2T16
9.00	2T20	2T12	2T16+ 2T8	2T16+ 2T16	2T16+ 2T8	2T12	2T16+ 2T16
10.50	3T12+ 3T12	3T12	3T16+ 2T8	3T20	3T16+ 2T8	3T12	3T16+ 1T16
12.00	3T12+ 3T12	3T12	3T16+ 2T8	3T20	3T16+ 2T8	3T12	3T16+ 1T16
13.50	3T16+ 1T16	3T12	3T20+ 2T8	3T20	3T20+ 2T8	3T12	3T20

4.4 DISCUSSION

According to the analysis and design done using Staad Pro, Robot Millennium and manual calculations, there are various different values obtained. It is clearly observed that the results from manual calculations gave a higher value in terms of analysis.

These structural software produced slightly different results, because of different assumptions, specifications, different safety factors and analysis. Staad Pro used matrix displacement method, while Robot Millennium used iterative solver application (Gauss elimination) to avoid factorization of a large-scale matrix. Matrix displacement method used few matrix combinations to obtain the values. The Gauss elimination will solve simultaneous linear equations and simplifying the numerical values.

In the analysis results, the software gave smaller values because the results were analyzed in three-dimensional model while manual calculations only can be done in two-dimensional model. This aspect is very important in the sense that the three-dimensional model allow the beam and column connection joints resist more loads due to the many load distribution to other members. However, in the case of two-dimensional model, fewer members only resisting the loads imposed and can be done by sub-frame calculations. Thus, it will lead to a much higher values.

From Table 4.1 and Table 4.2, the reaction results gave a very small difference up to 5.6%. The interior moment results gave up to 25% compared with the exterior with only up to 6.1%. This because the interior beam is stressed by members from many directions: upward, downward, leftward and rightward. These effects also affect the columns moment from Table 4.3, showing that longer span gave higher values.

According to Table 4.4 and Table 4.5, the theoretical results in reaction and moment gave smaller values compare with the software results. These theoretical values is determined using sub-frame calculations and without any influence from the upper and lower floors. Table 4.6 shows that the moments from theoretical results are constant, which is relatively not a precise method to obtain the actual results compared with the software.

Furthermore, manual calculations are done only in two-dimensional model by considering the gravity and lateral load only. This because the three-dimensional model using manual calculations are very complicated. The manual calculation using the most conservative way by always considering the larger values and is not the most accurate way to determine the analysis and design of structure. By considering the higher result values will lead to a larger size of bars in detailing, thus increase the construction cost. Therefore, the design software used to save the time used in designing of structure and also helping in determine the effective construction cost.

The Tables 4.7 to Table 4.9 show that the difference from 40% to 80% in terms of reaction and moment. These values were affected by column sizes and connections between them. The column sizes are more dominant at the upper floor, thus leading to a very huge difference. However, both of these software gave approximately similar values.

According to the detailing results, Staad Pro has provided higher bars for beams used in floor 1 (Table 4.10) and 6 (Table 4.11) compared with Robot Millennium. In floor 1 the top center gave 3 T20 (942mm^2) compared 2 T20 + 2 T8 (729mm^2). In floor 6 the top center gave 3T20 (942mm^2) compared 3T12 + 2T12 (565mm^2). However, in the floor 12 (Table 4.12), Robot Millennium gave higher bars used especially at the cantilever portion.

The detailing figures show that Staad Pro split the bars into few parts and not continuous to the end. It shows only the important bars and symmetrically distributed. As for columns from Table 4.13, Staad Pro provides a continuous bar of 8T16 from larger size towards the smaller sizes. The reason is to reduce less tensioning of bars especially at the connection of bars with different sizes of column. Robot Millennium has given exact values to be used which is not practical for construction purposes and need adjustments.

In terms of load increment analysis done on a single beam, it is clearly showed difference of bars used with a gradually increment in live loads for each 3.0 KN/m increment (Table 4.14 and Table 4.15). From this behavior, is showed that Staad Pro gave larger bars used compared with Robot Millennium. According to Table 4.14, Staad Pro has provided higher bars for beams. In the top center layers gave 3T20 (942mm^2) compared 2T20 + 2T8 (729mm^2) under live load 4.5 KN/m. The live load is increased 1.5 KN/m gradually until 13.5 KN/m.

The difference used in Staad Pro (Table 4.14) is relatively constant increment and symmetrical. However, Robot Millennium gave different approach by selecting the most precise method, by using different bars at different layers (Table 4.15), thus making risky and need further adjustment before issued to site construction.

CHAPTER 5
CONCLUSION AND RECOMMENDATION

As a conclusion, both of the software: STAAD PRO and Robot Millennium have some difference of output in terms of results. Throughout the research, the usage for the STAAD PRO and ROBOT MILLENNIUM has been determined. From the obtained results, the detailing ability and effectiveness of analysis embedded in these software has been identified. These software design can eliminate the tedious manual calculation works and also help the design engineer to appreciate the capability of the software design.

In terms of analysis, the software gave smaller values in terms of shear force and bending moment when compared with theoretical c alculations. Small differences have been found from floor 1 and floor 6. The huge difference at top floor is affected by higher values because column sizes are more dominant. The joint connections between beam and column are supported by lower columns only.

In terms of detailing and load increment analysis effects, Staad Pro gave larger bars used compared with Robot Millennium. Staad Pro detailing gave more adjustment in terms of safety and easy during installation on site. Robot Millennium using the precise method in detailing can gave the designer of the minimum requirement of steel to be used. Both of these software in detailing output need to be adjusted before it is issued to site. T hese results affect the cost reduction; improve safety procedures, and ease of installations on site.

As a recommendation for future work, the research can be continued on improving the calculation to detailing results. This research can determine the procedure of transferring the calculations into detailing AutoCAD software.

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APPENDICES

APPENDIX A
MANUAL CALCULATION

(TWO DIMENSIONAL SUB-FRAME METHOD)

SPAN B-F-J
Stiffnesses, ***k***
Beam

$$I = \frac{bd^3}{12} = \frac{0.2 \times 0.45^3}{12} = 1.52 \times 10^{-3}\ m^4$$

Spans BF and JF

$$k_{BF} = k_{JF} = \frac{1.52 \times 10^{-3}}{6.0} = 0.253 \times 10^{-3}\ m^4$$

$$k_{FB} = k_{FJ} = \frac{1.52 \times 10^{-3}}{6.0} = 0.253 \times 10^{-3}\ m^4$$

Columns

$$I = \frac{bd^3}{12} = \frac{0.4 \times 0.4^3}{12} = 2.13 \times 10^{-3}\ m^4$$

Upper

$$k_U = \frac{2.13 \times 10^{-3}}{3.6} = 0.593 \times 10^{-3}\ m^4$$

Lower

$$k_L = \frac{2.13 \times 10^{-3}}{4.4} = 0.485 \times 10^{-3}\ m^4$$

$$k_U + k_L = (0.593 + 0.485) \times 10^{-3} = 1.08 \times 10^{-3}\ m^4$$

Distribution Factors
Joints B and J

$\sum k = 0.25 + 1.08 = 1.33$

$D.F._{BF} = D.F._{JF}$
$= 0.25/1.33 = 0.19$

$D.F._{COLS} = 1.08/1.33 = 0.81$

Joint F

$\sum k = 0.25 + 0.25 + 1.08 = 1.58$

$D.F._{FB} = D.F._{FJ}$
$= 0.25/1.58 = 0.16$

$D.F._{COLS} = 1.08/1.58 = 0.68$

Fixed End Moment (F.E.M.)
For 1.4DL + 1.6LL

$M_{BF} = M_{JF} = WL^2 / 12 = 34.96(6)^2 / 12 = 104.89$ KNm

$M_{FB} = M_{FJ} = -WL^2 / 12 = -34.96(6)^2 / 12 = -104.89$ KNm

For 1.0DL

$M_{BF} = M_{JF} = WL^2 / 12 = 19.83(6)^2 / 12 = 59.49$ KNm

$M_{FB} = M_{FJ} = -WL^2 / 12 = -19.83(6)^2 / 12 = -59.49$ KNm

First Loading Case

$$\text{Shear } V_{BF} = \frac{\text{Load}}{2} - \frac{(M_{BF} - M_{FB})}{L}$$

$$= \frac{209.76}{2} - \frac{(-96.79 + 86.99)}{6}$$

$$= \underline{106.51 \text{ KN}}$$

$$\text{Shear } V_{FB} = \text{Load} - V_{BF}$$
$$= 209.76 - 106.51$$
$$= \underline{103.25 \text{ KN}}$$

$$\text{Maximum Moment, span BF} = \frac{V_{BF}^2}{2w} + M_{BF}$$
$$= \frac{106.51^2}{2 \times 34.96} - 96.79$$
$$= \underline{\mathbf{65.46\ KNm}}$$

Distance from B, $a_1 = \frac{V_{BF}}{w}$

$= 106.51/34.96$

$= 3.05$ m

Second Loading Case

Shear $V_{BF} = \frac{Load}{2} - \frac{(M_{BF} - M_{FB})}{L}$

$= \frac{118.98}{2} - \frac{(-60.01 + 37.27)}{6}$

$= \underline{63.28 \text{ KN}}$

Shear V_{FB} = Load - V_{BF}

$= 118.98 - 63.28$

$= \underline{55.70 \text{ KN}}$

Maximum Moment, span BF $= \frac{V_{BF}^2}{2w} + M_{BF}$

$= \frac{63.28^2}{2 \times 19.83} - 60.01$

$= \underline{\mathbf{40.96 \text{ KNm}}}$

Distance from B, $a_1 = \frac{V_{BF}}{w}$

$= 63.28/19.83$

$= 3.19$ m

Third Loading Case

Shear $V_{BF} = \frac{Load}{2} - \frac{(M_{BF} - M_{FB})}{L}$

$= \frac{209.76}{2} - \frac{(-100.06 + 79.29)}{6}$

$= \underline{108.34 \text{ KN}}$

Shear V_{FB} = Load - V_{BF}
= 209.76 – 108.34
= <u>101.42 KN</u>

Maximum Moment, span BF $= \frac{V_{BF}^2}{2w} + M_{BF}$

$= \frac{108.34^2}{2 \times 34.96} - 100.06$

= **<u>67.81 KNm</u>**

Distance from B, $a_1 = \frac{V_{BF}}{w}$
= 108.34/34.96
= 3.10 m

<u>First Loading Arrangement</u>
<u>Column Moment</u>

$M_{BC} = 96.79 \times \frac{0.59}{1.08} = 52.88$ KNm

$M_{BA} = 96.79 \times \frac{0.49}{1.08} = 43.91$ KNm

$M_{FG} = 124.26 \times \frac{0.59}{1.08} = 67.88$ KNm

$M_{FE} = 124.26 \times \frac{0.49}{1.08} = 56.38$ KNm

$M_{JK} = 60.01 \times \frac{0.59}{1.08} = 32.78$ KNm

$M_{JI} = 60.01 \times \frac{0.49}{1.08} = 27.23$ KNm

<u>Second Loading Arrangement</u>
<u>Column Moment</u>

$M_{BC} = 60.01 \times \frac{0.59}{1.08} = 32.78$ KNm

$M_{BA} = 60.01 \times \frac{0.49}{1.08} = 27.23$ KNm

$M_{FG} = 124.26 \times \frac{0.59}{1.08} = 67.88$ KNm

$M_{FE} = 124.26 \times \frac{0.49}{1.08} = 56.38$ KNm

$M_{JK} = 96.79 \times \frac{0.59}{1.08} = 52.88$ KNm

$M_{JI} = 96.79 \times \frac{0.49}{1.08} = 43.91$ KNm

Third Loading Arrangement

Column Moment

$M_{BC} = 100.06 \times \frac{0.59}{1.08} = 54.66$ KNm

$M_{BA} = 100.06 \times \frac{0.49}{1.08} = 45.40$ KNm

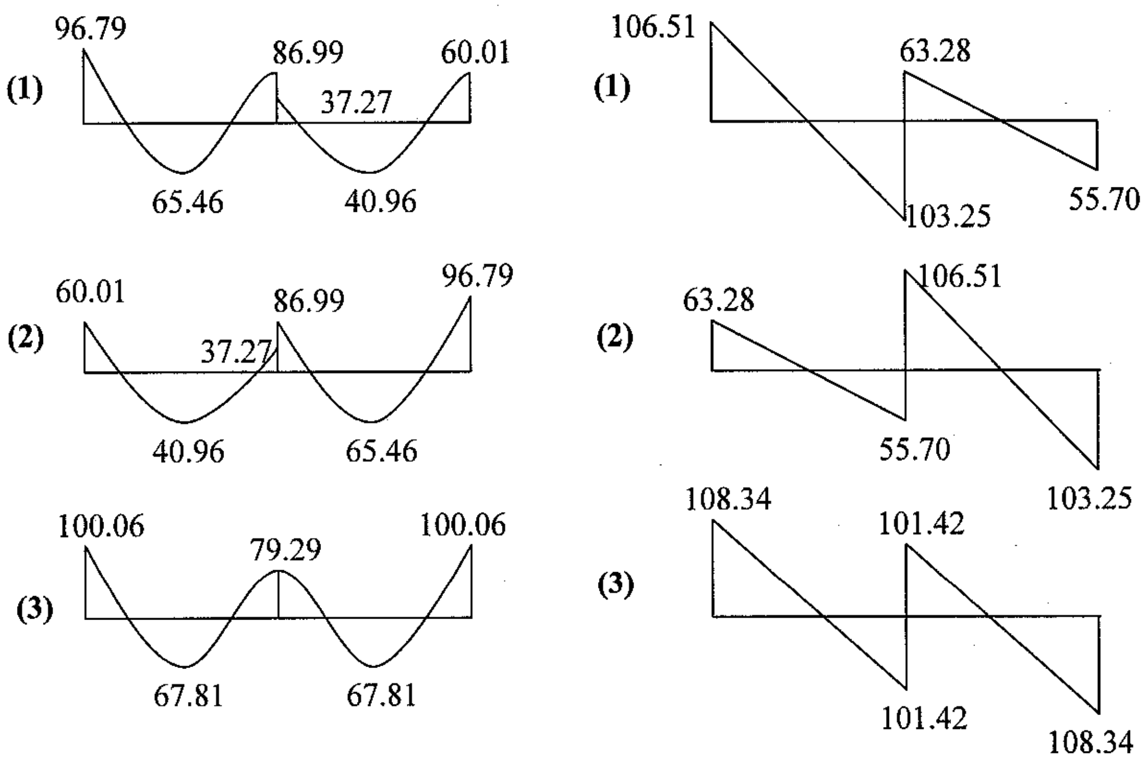
$M_{FG} = 158.58 \times \frac{0.59}{1.08} = 86.63$ KNm

$M_{FE} = 158.58 \times \frac{0.49}{1.08} = 71.95$ KNm

$M_{JK} = 100.06 \times \frac{0.59}{1.08} = 54.66$ KNm

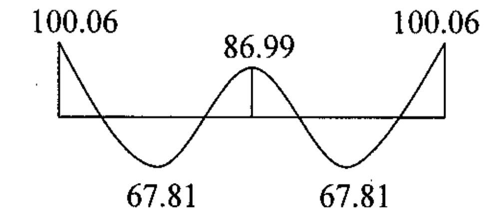
$M_{JI} = 100.06 \times \frac{0.49}{1.08} = 45.40$ KNm

Moment and Shear Force Diagram for All Loads Arrangements

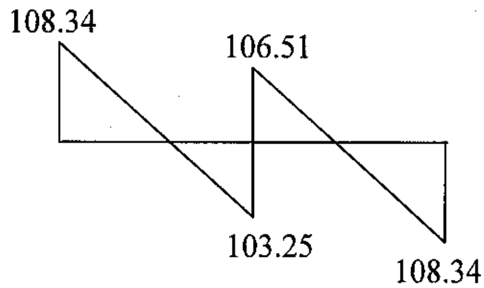


Moment and Shear Force Envelopes

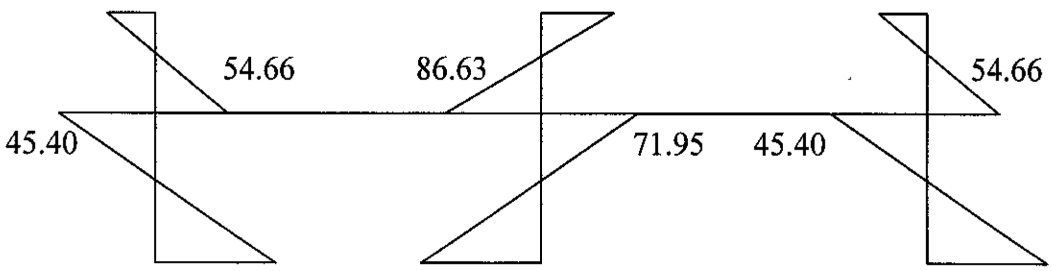
Moment and Shear Force Envelopes are the superimposed moment and shear force diagrams obtained from different possible load arrangements. Critical values from moment's envelopes are selected for design purpose.



Bending Moment Envelope (KNm)



Shear Force Envelope (KN)



Column Bending Envelope (KNm)

SPAN C-G-K

Stiffnesses, ***k***

Beam

$$I = \frac{bd^3}{12} = \frac{0.2 \times 0.45^3}{12} = 1.52 \times 10^{-3}\ m^4$$

Spans CG and KG

$$k_{CG} = k_{KG} = \frac{1.52 \times 10^{-3}}{6.0} = 0.253 \times 10^{-3}\ m^4$$

$$k_{GC} = k_{GK} = \frac{1.52 \times 10^{-3}}{6.0} = 0.253 \times 10^{-3}\ m^4$$

Columns

$$I = \frac{bd^3}{12} = \frac{0.25 \times 0.25^3}{12} = 0.33 \times 10^{-3}\ m^4$$

Upper

$$k_U = \frac{0.33 \times 10^{-3}}{3.6} = 0.09 \times 10^{-3}\ m^4$$

Lower

$$k_L = \frac{0.33 \times 10^{-3}}{3.6} = 0.09 \times 10^{-3}\ m^4$$

$$k_U + k_L = (0.09 + 0.09) \times 10^{-3} = 0.18 \times 10^{-3}\ m^4$$

Distribution Factors

Joints C and K

$$\sum k = 0.25 + 0.18 = 0.43$$

$$D.F._{CG} = D.F._{KG}$$
$$= 0.25/0.43 = 0.58$$

$D.F._{COLS} = 0.18/0.43 = 0.42$

Joint G

$\sum k = 0.25 + 0.25 + 0.18 = 0.68$

$D.F._{GC} = D.F._{GK}$
$= 0.25/0.68 = 0.37$

$D.F._{COLS} = 0.18/0.68 = 0.26$

Fixed End Moment (F.E.M.)

For 1.4DL + 1.6LL

$M_{CG} = M_{KG} = WL^2 / 12 = 34.96(6)^2 / 12 = 104.89$ KNm

$M_{GC} = M_{GK} = -WL^2 / 12 = -34.96(6)^2 / 12 = -104.89$ KNm

For 1.0DL

$M_{CG} = M_{KG} = WL^2 / 12 = 19.83(6)^2 / 12 = 59.49$ KNm

$M_{GC} = M_{GK} = -WL^2 / 12 = -19.83(6)^2 / 12 = -59.49$ KNm

First Loading Case

$$\text{Shear } V_{CG} = \frac{\text{Load}}{2} - \frac{(M_{CG} - M_{GC})}{L}$$

$$= \frac{209.76}{2} - \frac{(-61.90 + 59.81)}{6}$$
$$= \underline{105.23 \text{ KN}}$$

$$\text{Shear } V_{GC} = \text{Load} - V_{CG}$$
$$= 209.76 - 105.23$$
$$= \underline{104.53 \text{ KN}}$$

$$\text{Maximum Moment, span BF} = \frac{V_{CG}^2}{2w} + M_{CG}$$

$$= \frac{105.23^2}{2 \times 34.96} - 61.90$$

$= \mathbf{\underline{96.47\ KNm}}$

Distance from C, $a_1 = \frac{V_{CG}}{w}$

$= 105.23/34.96$

$= 3.01\ m$

<u>Second Loading Case</u>

Shear $V_{CG} = \frac{Load}{2} - \frac{(M_{CG} - M_{GC})}{L}$

$= \frac{118.98}{2} - \frac{(-42.83 + 1.24)}{6}$

$= \underline{66.42\ KN}$

Shear $V_{GC} = Load - V_{CG}$

$= 118.98 - 66.42$

$= \underline{52.56\ KN}$

Maximum Moment, span CG$= \frac{V_{CG}^2}{2w} + M_{CG}$

$= \frac{66.42^2}{2 \times 19.83} - 42.83$

$= \mathbf{\underline{68.41\ KNm}}$

Distance from C, $a_1 = \frac{V_{CG}}{w}$

$= 66.42/19.83$

$= 3.35\ m$

<u>Third Loading Case</u>

Shear $V_{CG} = \frac{Load}{2} - \frac{(M_{CG} - M_{GC})}{L}$

$= \frac{209.76}{2} - \frac{(-66.83 + 38.95)}{6}$

$= \underline{109.53 \text{ KN}}$

Shear $V_{GC} = \text{Load} - V_{CG}$

$= 209.76 - 109.53$

$= \underline{100.23 \text{ KN}}$

Maximum Moment, span CG $= \dfrac{V_{CG}^2}{2w} + M_{CG}$

$$= \frac{109.53^2}{2 \times 34.96} - 66.83$$

$$= \mathbf{\underline{104.75 \text{ KNm}}}$$

Distance from C, $a_1 = \dfrac{V_{CG}}{w}$

$= 109.53/34.96$

$= 3.13 \text{ m}$

<u>First Loading Arrangement</u>
<u>Column Moment</u>

$M_{CD} = M_{CB} = 61.90 \times \dfrac{0.09}{0.18} = 30.95 \text{ KNm}$

$M_{GH} = M_{GF} = 61.05 \times \dfrac{0.09}{1.08} = 30.53 \text{ KNm}$

$M_{KL} = M_{KJ} = 42.83 \times \dfrac{0.09}{1.08} = 21.42 \text{ KNm}$

<u>Second Loading Arrangement</u>
<u>Column Moment</u>

$M_{CD} = M_{CB} = 42.83 \times \dfrac{0.09}{0.18} = 21.42 \text{ KNm}$

$M_{GH} = M_{GF} = 61.05 \times \dfrac{0.09}{1.08} = 30.53 \text{ KNm}$

$M_{KL} = M_{KJ} = 61.90 \times \dfrac{0.09}{1.08} = 30.95 \text{ KNm}$

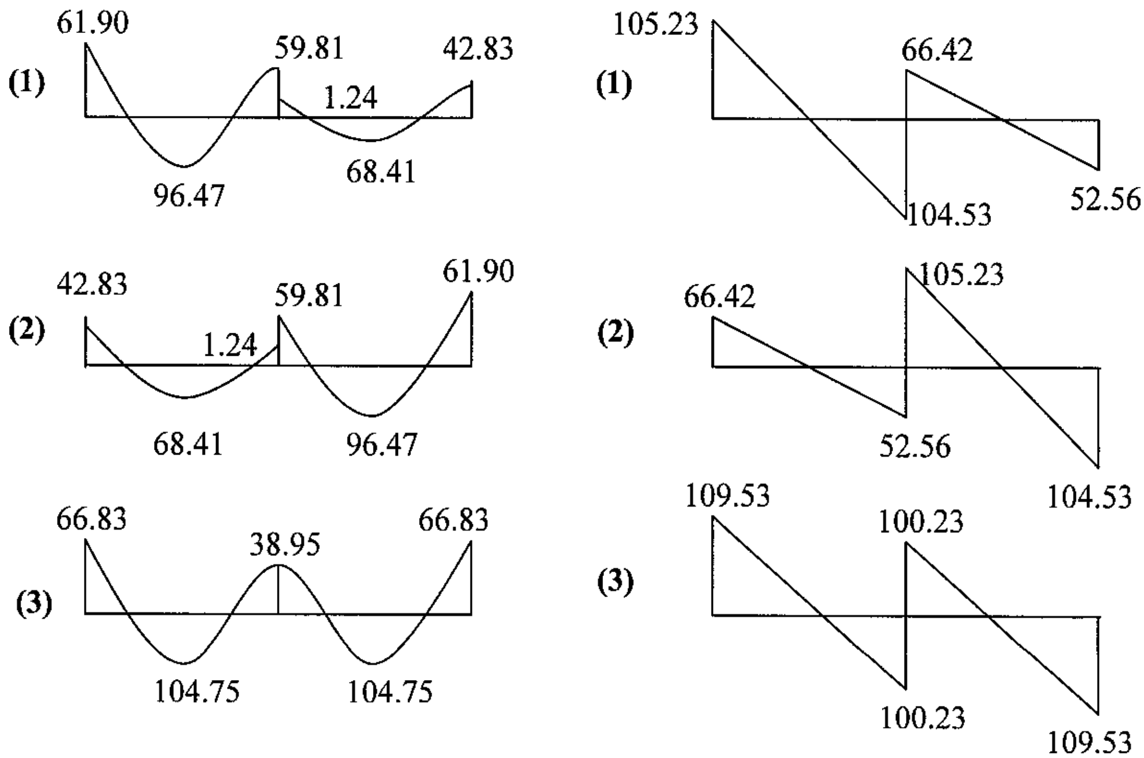
Third Loading Arrangement
Column Moment

$$M_{CD} = M_{CB} = 66.83 \text{ x } \frac{0.09}{0.18} = 33.42 \text{ KNm}$$

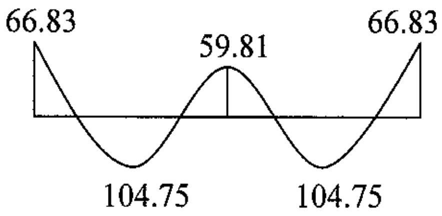
$$M_{GH} = M_{GF} = 77.91 \text{ x } \frac{0.09}{1.08} = 38.96 \text{ KNm}$$

$$M_{KL} = M_{KJ} = 66.83 \text{ x } \frac{0.09}{1.08} = 33.42 \text{ KNm}$$

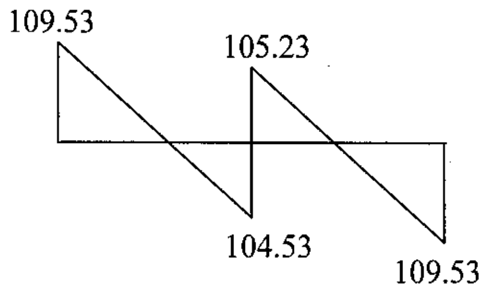
Moment and Shear Force Diagram for All Loads Arrangements



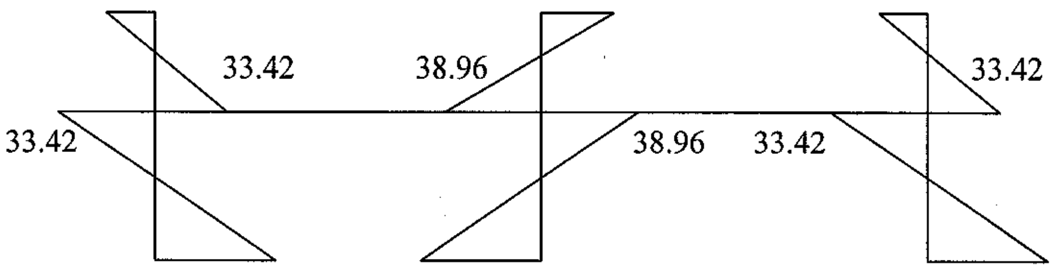
Moment and Shear Force Envelopes



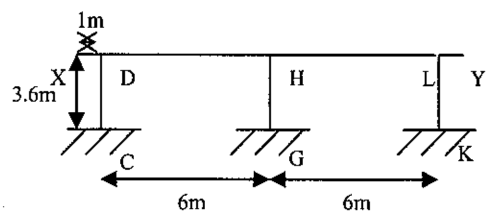
Bending Moment Envelope (KNm)



Shear Force Envelope (KN)



Column Bending Envelope (KNm)



Stiffnesses, *k*

Beam

$$I = \frac{bd^3}{12} = \frac{0.2 \times 0.45^3}{12} = 1.52 \times 10^{-3}\ m^4$$

Spans DH and LH

$$k_{DH} = k_{LH} = \frac{1.52 \times 10^{-3}}{6.0} = 0.253 \times 10^{-3}\ m^4$$

$$k_{HD} = k_{HL} = \frac{1.52 \times 10^{-3}}{6.0} = 0.253 \times 10^{-3}\ m^4$$

Columns

$$I = \frac{bd^3}{12} = \frac{0.25 \times 0.25^3}{12} = 0.33 \times 10^{-3}\ m^4$$

Lower

$$k_L = \frac{0.33 \times 10^{-3}}{3.6} = 0.09 \times 10^{-3}\ m^4$$

Distribution Factors

Joints D and L

$\sum k = 0.25 + 0.09 = 0.34$

$D.F._{DH} = D.F._{LH}$

$= 0.25/0.34 = 0.74$

$D.F._{DX} = D.F._{LY}$
$= 0$

$D.F._{COLS} = 0.09/0.34 = 0.26$

<u>Joint H</u>

$\sum k = 0.25 + 0.25 + 0.09 = 0.59$

$D.F._{HD} = D.F._{HL}$
$= 0.25/0.59 = 0.42$

$D.F._{COLS} = 0.09/0.59 = 0.16$

<u>Fixed End Moment (F.E.M.)</u>
<u>For 1.4DL + 1.6LL</u>

$M_{DH} = M_{LH} = WL^2 / 12 = 6.59(6)^2 / 12 = 19.78$ KNm

$M_{HD} = M_{HL} = -WL^2 / 12 = -6.59(6)^2 / 12 = -19.78$ KNm

$M_{XD} = M_{YL} = 0$ (no support)

$M_{DX} = M_{LY} = -WL^2 / 12 = -6.59(1)^2 / 12 = -0.55$ KNm

<u>For 1.0DL</u>

$M_{DH} = M_{LH} = WL^2 / 12 = 4.71(6)^2 / 12 = 14.13$ KNm

$M_{HD} = M_{HL} = -WL^2 / 12 = -4.71(6)^2 / 12 = -14.13$ KNm

$M_{XD} = M_{YL} = 0$ (no support)

$M_{DX} = M_{LY} = -WL^2 / 12 = 4.71(1)^2 / 12 = -0.39$ KNm

First Loading Case

$$\text{Shear } V_{DH} = \frac{\text{Load}}{2} - \frac{(M_{DH} - M_{HD})}{L}$$

$$= \frac{28.26}{2} - \frac{(-6.50 + 0.43)}{6}$$

$$= \underline{15.14 \text{ KN}}$$

$$\text{Shear } V_{HD} = \text{Load} - V_{DH}$$
$$= 28.26 - 15.14$$
$$= \underline{13.12 \text{ KN}}$$

$$\text{Maximum Moment, span DH} = \frac{V_{DH}^2}{2w} + M_{DH}$$
$$= \frac{15.14^2}{2 \times 4.71} - 6.50$$
$$= \underline{\mathbf{17.83 \text{ KNm}}}$$

$$\text{Distance from D, } a_1 = \frac{V_{DH}}{w}$$
$$= 17.83/4.71$$
$$= 3.79 \text{ m}$$

Second Loading Case

$$\text{Shear } V_{DH} = \frac{\text{Load}}{2} - \frac{(M_{DH} - M_{HD})}{L}$$

$$= \frac{39.54}{2} - \frac{(-7.96 + 8.16)}{6}$$
$$= \underline{19.74 \text{ KN}}$$

$$\text{Shear } V_{HD} = \text{Load} - V_{DH}$$
$$= 39.54 - 19.74$$
$$= \underline{19.80 \text{ KN}}$$

$$\text{Maximum Moment, span DH} = \frac{V_{DH}^2}{2w} + M_{DH}$$

$$= \frac{19.74^2}{2 \times 6.59} - 7.96$$

$$= \underline{\mathbf{21.61\ KNm}}$$

Distance from D, $a_1 = \frac{V_{DH}}{w}$

$= 21.61/6.59$
$= 3.28$ m

Third Loading Case

Shear $V_{DH} = \frac{Load}{2} - \frac{(M_{DH} - M_{HD})}{L}$

$$= \frac{39.54}{2} - \frac{(-8.43 + 5.01)}{6}$$

$= \underline{20.34\ KN}$

Shear $V_{HD} = Load - V_{DH}$
$= 39.54 - 20.34$
$= \underline{19.20\ KN}$

Maximum Moment, span DH $= \frac{V_{DH}^2}{2w} + M_{DH}$

$$= \frac{20.34^2}{2 \times 6.59} - 8.43$$

$$= \underline{\mathbf{22.96\ KNm}}$$

Distance from D, $a_1 = \frac{V_{DH}}{w}$
$= 22.96/6.59$
$= 3.48$ m

First Loading Arrangement
Column Moment

$M_{DC} = 6.50$ KNm

$M_{HG} = 8.59$ KNm

M_{LK} = 7.96 KNm

Second Loading Arrangement
Column Moment

M_{DC} = 7.96 KNm

M_{HG} = 8.59 KNm

M_{LK} = 6.50 KNm

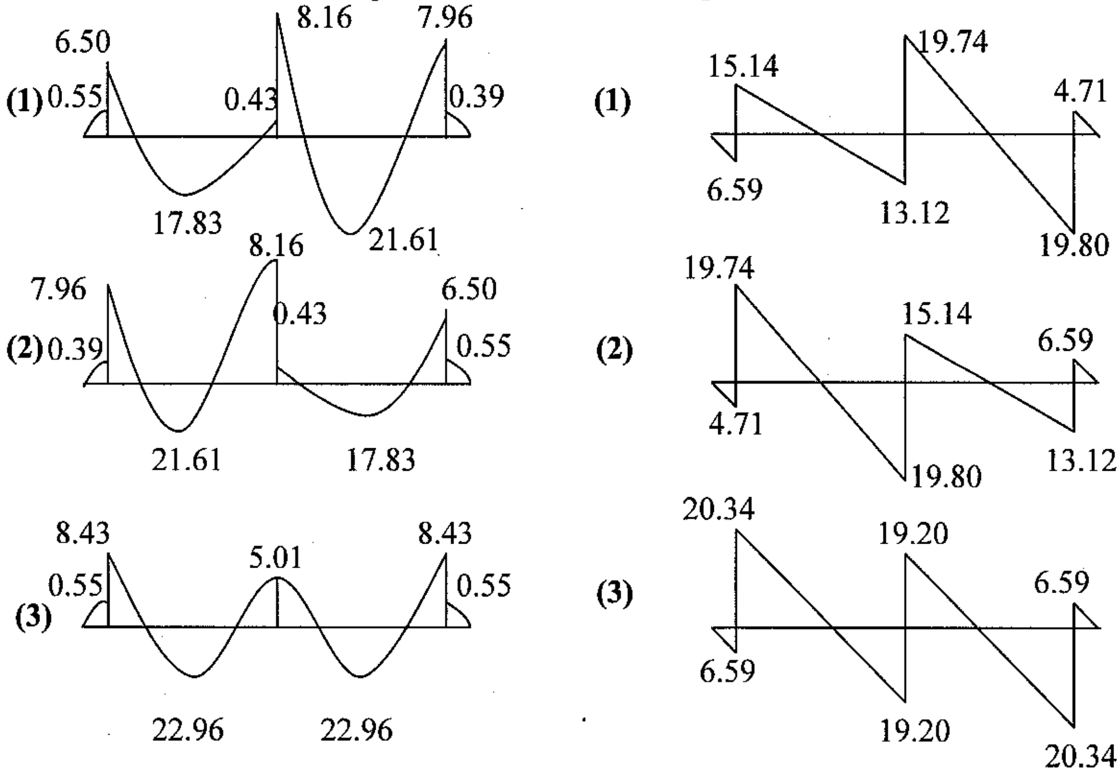
Third Loading Arrangement
Column Moment

M_{DC} = 8.43 KNm

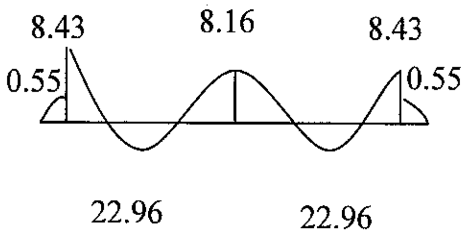
M_{HG} = 10.01 KNm

M_{LK} = 8.43 KNm

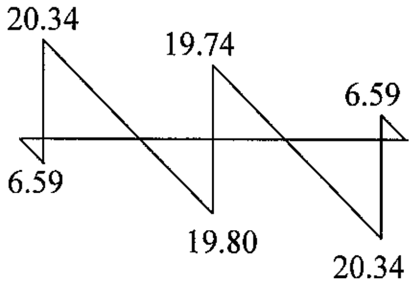
Moment and Shear Force Diagram for All Loads Arrangements



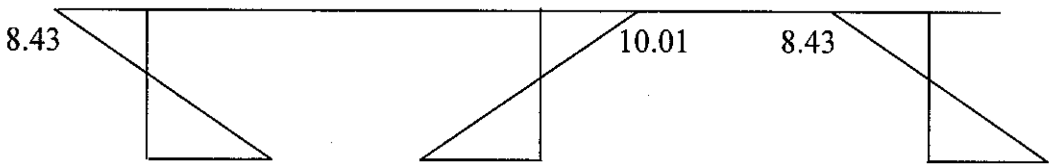
Moment and Shear Force Envelopes



Bending Moment Envelope (KNm)



Shear Force Envelope (KN)



Column Bending Envelope (KNm)

APPENDIX B
MANUAL CALCULATION
(MOMENT DISTRIBUTION METHOD)

SPAN BFJ

Moment distribution for first loading case

	B			F				J	
	Cols. (ΣM)	BF		FB	Cols. (ΣM)	FJ		JF	Cols. (ΣM)
D.F.s	0.81	0.19		0.16	0.68	0.16		0.19	0.81
Load KN			209.76				118.98		
F.E.M.		104.89		-104.89		-59.49		59.49	
Bal.	-84.96	-19.93		26.30	111.78	26.30		-11.30	-48.19
C.O.		13.15		-9.96		-5.65		13.15	
Bal.	-10.65	-2.50		2.50	10.62	2.50		-2.50	-10.65
C.O.		1.25		-1.25		-1.25		1.25	
Bal.	-1.01	-0.24		0.40	1.70	0.40		-0.24	-1.01
C.O.		0.20		-0.12		-0.12		0.20	
Bal.	-0.16	-0.04		0.04	0.16	0.04		-0.04	-0.16
M(KNm)	**-96.79**	**96.79**		**-86.99**	**124.26**	**-37.27**		**60.01**	**-60.01**

Moment distribution for second loading case

	B			F				J	
	Cols. (ΣM)	BF		FB	Cols. (ΣM)	FJ		JF	Cols. (ΣM)
D.F.s	0.81	0.19		0.16	0.68	0.16		0.19	0.81
Load KN			118.98				209.78		
F.E.M.		59.49		-59.49		-104.89		104.89	
Bal.	-48.19	-11.30		26.30	111.78	26.30		-19.93	-84.96
C.O.		13.15		-5.65		-9.96		13.15	
Bal.	-10.65	-2.50		2.50	10.62	2.50		-2.50	-10.65
C.O.		1.25		-1.25		-1.25		1.25	
Bal.	-1.01	-0.24		0.40	1.70	0.40		-0.24	-1.01
C.O.		0.20		-0.12		-0.12		0.20	
Bal.	-0.16	-0.04		0.04	0.16	0.04		-0.04	-0.16
M(KNm)	**-60.01**	**60.01**		**-37.27**	**124.26**	**-86.99**		**96.79**	**-96.79**

Moment distribution for third loading case

	B			F				J	
	Cols. (ΣM)	BF		FB	Cols. (ΣM)	FJ		JF	Cols. (ΣM)
D.F.s	0.81	0.19		0.16	0.68	0.16		0.19	0.81
Load KN			209.76				209.76		
F.E.M.		104.89		-104.89		-104.89		104.89	
Bal.	-84.96	-19.93		33.56	142.65	33.56		-19.93	-84.96
C.O.		16.78		-9.96		-9.96		16.78	
Bal.	-13.59	-3.19		3.19	13.55	3.19		-3.19	-13.59
C.O.		1.60		-1.60		-1.60		1.60	
Bal.	-1.30	-0.30		0.51	2.18	0.51		-0.30	-1.30
C.O.		0.26		-0.15		-0.15		0.26	

Bal.	-0.21	-0.05	0.05	0.20	0.05	-0.05	-0.21
M(KNm)	**100.06**	**100.06**	**-79.29**	**158.58**	**-79.29**	**100.06**	**-100.06**

SPAN CGK

Moment distribution for first loading case

	C			G				K	
	Cols. (∑M)	CG		GC	Cols. (∑M)	GK		KG	Cols. (∑M)
D.F.s	0.42	0.58		0.37	0.26	0.37		0.58	0.42
Load KN			209.76				118.98		
F.E.M.		104.89		-104.89		-59.49		59.49	
Bal.	44.05	-60.84		60.82	42.74	60.82		-34.50	-24.99
C.O.		30.41		-30.42		-17.25		30.41	
Bal.	12.77	-17.64		17.64	12.39	17.64		-17.64	-12.77
C.O.		8.82		-8.82		-8.82		8.82	
Bal.	-3.70	-5.12		6.53	4.59	6.53		-5.12	-3.70
C.O.		3.26		-2.56		-2.56		3.26	
Bal.	-1.37	-1.89		1.89	1.33	1.89		-1.89	-1.37
M(KNm)	**61.90**	**61.90**		**-59.81**	**61.05**	**-1.24**		**42.83**	**-42.83**

Moment distribution for second loading case

	C			G				K	
	Cols. (∑M)	CG		GC	Cols. (∑M)	GK		KG	Cols. (∑M)
D.F.s	0.42	0.58		0.37	0.26	0.37		0.58	0.42
Load KN			118.98				209.76		
F.E.M.		59.49		-59.49		-104.89		104.89	
Bal.	24.99	-34.50		60.82	42.74	60.82		-60.84	-44.05
C.O.		30.41		-17.25		-30.42		30.41	
Bal.	12.77	-17.64		17.64	12.39	17.64		-17.64	-12.77
C.O.		8.82		-8.82		-8.82		8.82	
Bal.	-3.70	-5.12		6.53	4.59	6.53		-5.12	-3.70
C.O.		3.26		-2.56		-2.56		3.26	
Bal.	-1.37	-1.89		1.89	1.33	1.89		-1.89	-1.37
M(KNm)	**42.83**	**42.83**		**-1.24**	**61.05**	**-59.81**		**61.90**	**-61.90**

Moment distribution for third loading case

	C			G				K	
	Cols. (ΣM)	CG		GC	Cols. (ΣM)	GK		KG	Cols. (ΣM)
D.F.s	0.42	0.58		0.37	0.26	0.37		0.58	0.42
Load KN			209.76				209.76		
F.E.M.		104.89		-104.89		-104.89		104.89	
Bal.	-44.05	-60.84		77.62	54.54	77.62		-60.84	-44.05
C.O.		38.81		-30.42		-30.42		38.81	
Bal.	-16.30	-22.51		22.51	15.82	22.51		-22.51	-16.30
C.O.		11.25		-11.25		-11.25		11.25	
Bal.	-4.73	-6.53		8.33	5.85	8.33		-6.53	-4.73
C.O.		4.16		-3.26		-3.26		4.16	
Bal.	-1.75	-2.42		2.42	1.70	2.42		-2.42	-1.75
M(KNm)	**-66.83**	**66.83**		**-38.95**	**77.91**	**-38.95**		**66.83**	**-66.83**

SPAN DHL

Moment distribution for first loading case

		D				H				L	
	DX	Cols. (ΣM)	DH		HD	Cols. (ΣM)	HL		LH	Cols. (ΣM)	LY
D.F.s	0	0.26	0.74		0.42	0.16	0.42		0.74	0.26	0
Load KN				28.26				39.54			
F.E.M.	0.55		14.13		-14.13		-19.77		19.77		0.39
Bal.	0	-3.67	-10.46		14.24	5.42	14.24		-14.63	-5.14	0
C.O.	0		7.12		-5.23		-7.31		7.12		0
Bal.	0	-1.85	-5.27		5.27	2.01	5.27		-5.27	-1.85	0
C.O.	0		2.63		-2.63		-2.63		2.63		0
Bal.	0	-0.68	-1.95		2.21	0.84	2.21		-1.95	-0.68	0
C.O.	0		1.11		-0.97		-0.97		1.11		0
Bal.	0	-0.29	-0.82		0.82	0.31	0.82		-0.82	-0.29	0
M(KNm)	**0.55**	**-6.50**	**6.50**		**-0.43**	**8.59**	**-8.16**		**7.96**	**-7.96**	**0.39**

Moment distribution for second loading case

		D				H				L	
	DX	Cols. (ΣM)	DH		HD	Cols. (ΣM)	HL		LH	Cols. (ΣM)	LY
D.F.s	0	0.26	0.74		0.42	0.16	0.42		0.74	0.26	0
Load KN				39.54				28.26			
F.E.M.	0.39		19.77		-19.77		-14.13		14.13		0.55

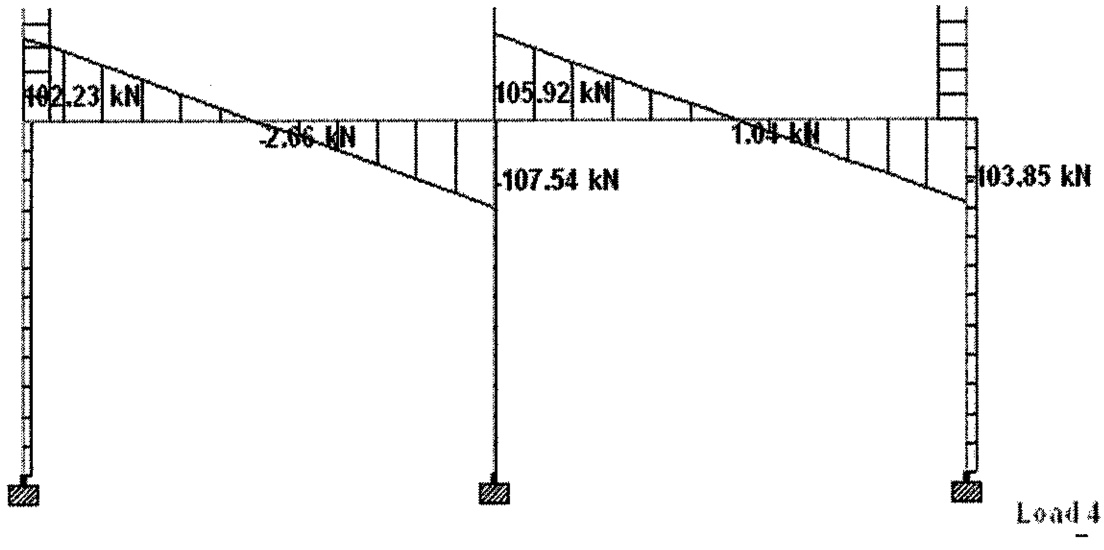
Bal.	0	-5.14	-14.63		14.24	5.42	14.24		-10.46	-3.67	0
C.O.	0		7.12		-7.31		-5.23		7.12		0
Bal.	0	-1.85	-5.27		5.27	2.01	5.27		-5.27	-1.85	0
C.O.	0		2.63		-2.63		-2.63		2.63		0
Bal.	0	-0.68	-1.95		2.21	0.84	2.21		-1.95	-0.68	0
C.O.	0		1.11		-0.97		-0.97		1.11		0
Bal.	0	-0.29	-0.82		0.82	0.31	0.82		-0.82	-0.29	0
M(KNm)	**0.39**	**-7.96**	**7.96**		**-8.16**	**8.59**	**-0.43**		**6.50**	**-6.50**	**0.55**

Moment distribution for third loading case

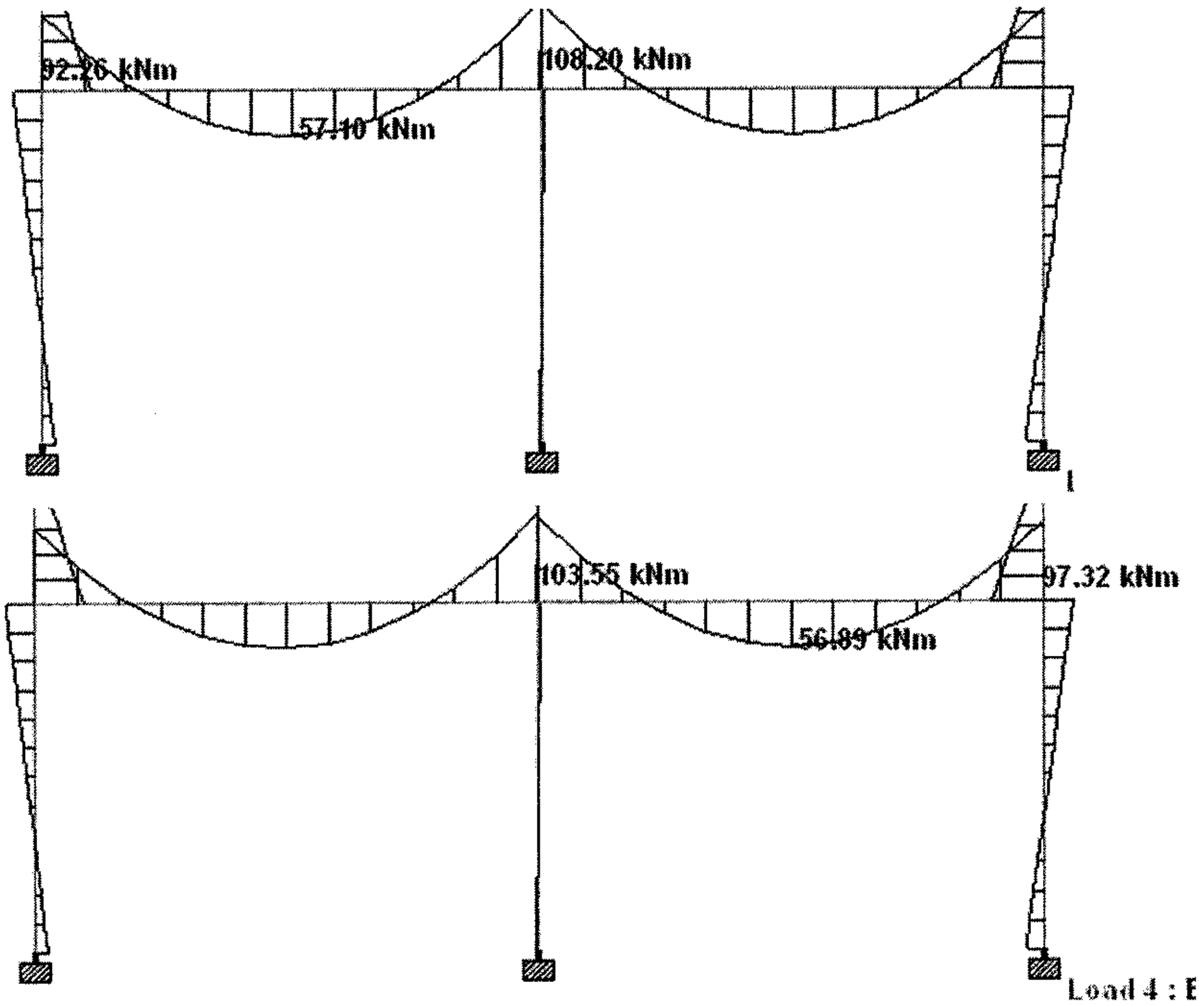
	D				H				L		
	DX	Cols. ($\sum M$)	DH		HD	Cols. ($\sum M$)	HL		LH	Cols. ($\sum M$)	LY
D.F.s	0	0.26	0.74		0.42	0.16	0.42		0.74	0.26	0
Load KN				39.54				39.54			
F.E.M.	-0.55		19.77		-19.77		-19.77		19.77		-0.55
Bal.	0	-5.14	-14.63		16.61	6.33	16.61		-14.63	-5.14	0
C.O.	0		8.30		-7.31		-7.31		8.30		0
Bal.	0	-2.16	-6.14		6.14	2.34	6.14		-6.14	-2.16	0
C.O.	0		3.07		-3.07		-3.07		3.07		0
Bal.	0	-0.80	-2.27		2.58	0.98	2.58		-2.27	-0.80	0
C.O.	0		1.29		-1.14		-1.14		1.29		0
Bal.	0	-0.34	-0.95		0.95	0.36	0.95		-0.95	-0.34	0
M(KNm)	**0.55**	**-8.43**	**8.43**		**-5.01**	**10.01**	**-5.01**		**8.43**	**-8.43**	**0.55**

APPENDIX C
STAAD PRO AND ROBOT MILLENNIUM ANALYSIS RESULTS DIAGRAM
(BEAMS AND COLUMNS)

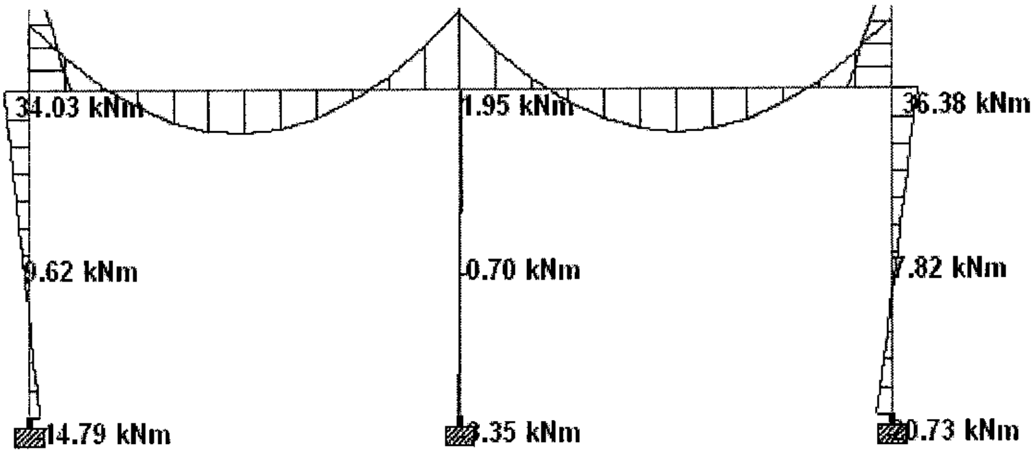
Shear Force Floor 1



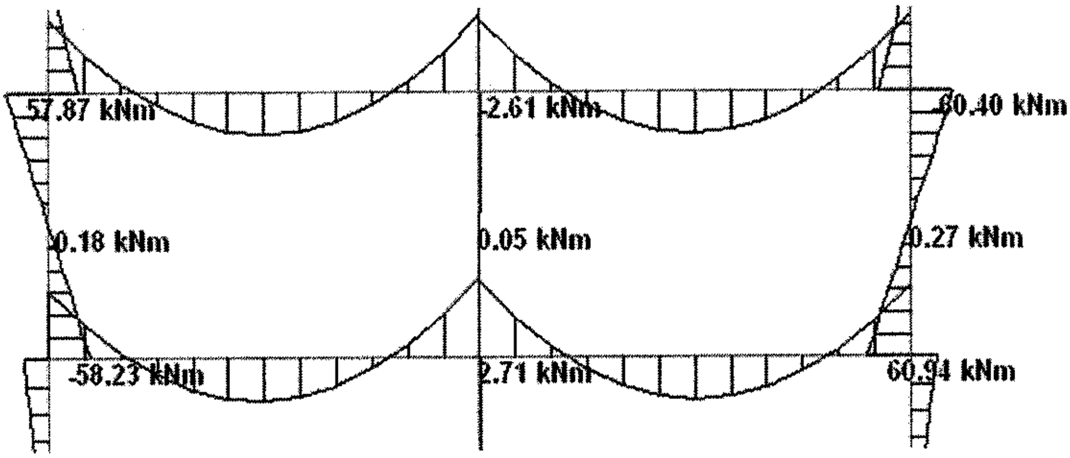
Bending Moment Floor 1



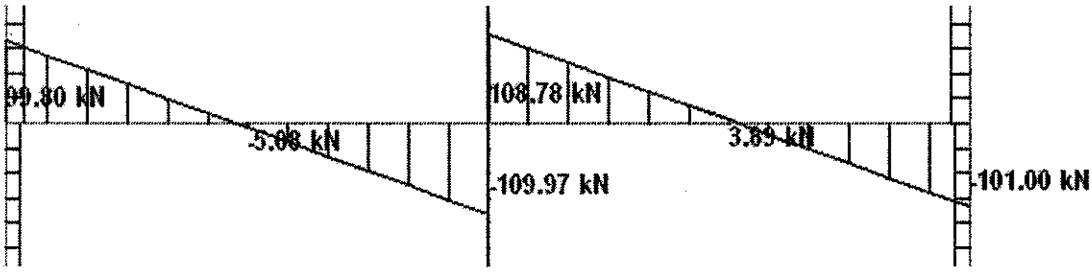
Column Moment Floor 1
Lower



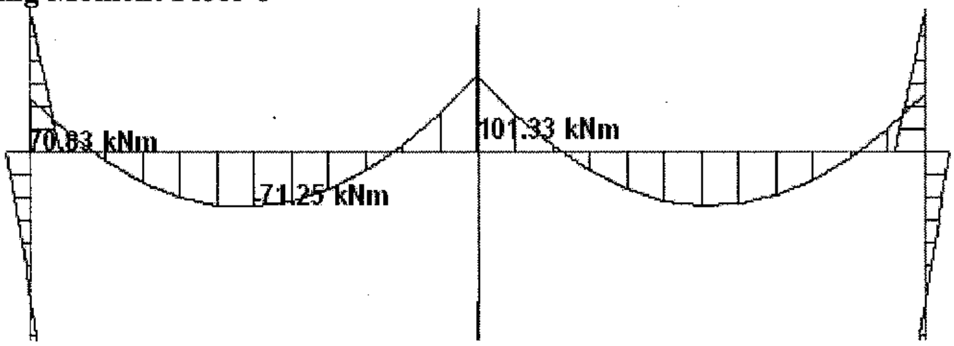
Upper

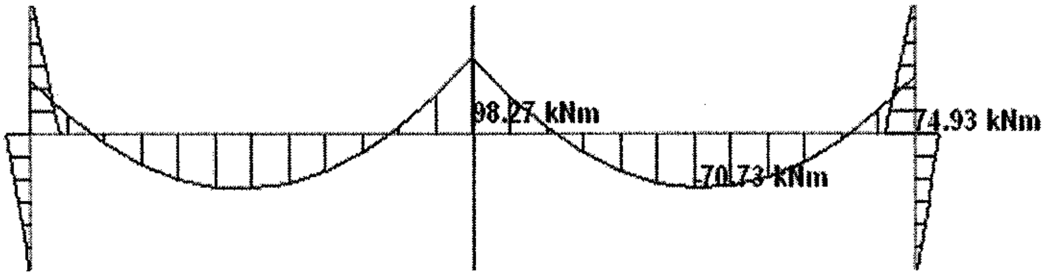


Shear Force Floor 6



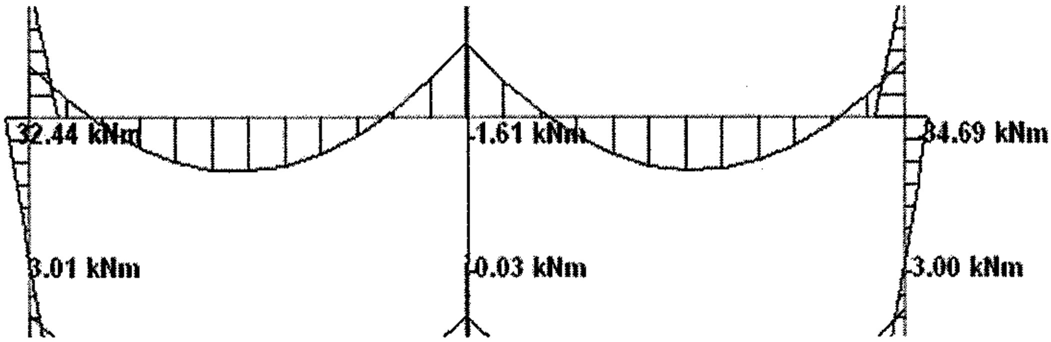
Bending Moment Floor 6



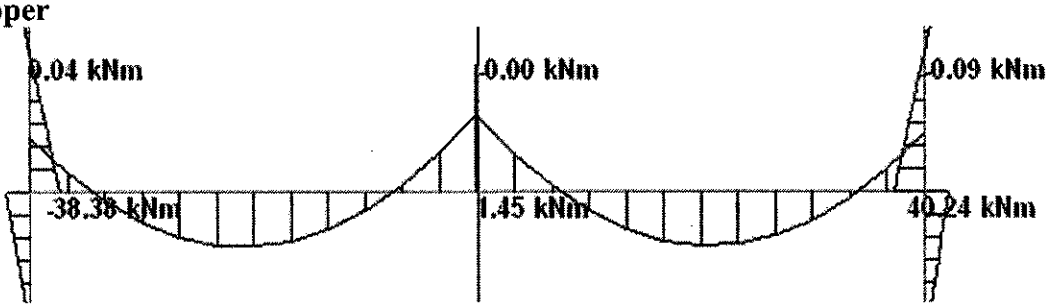


Column Moment Floor 6

Lower

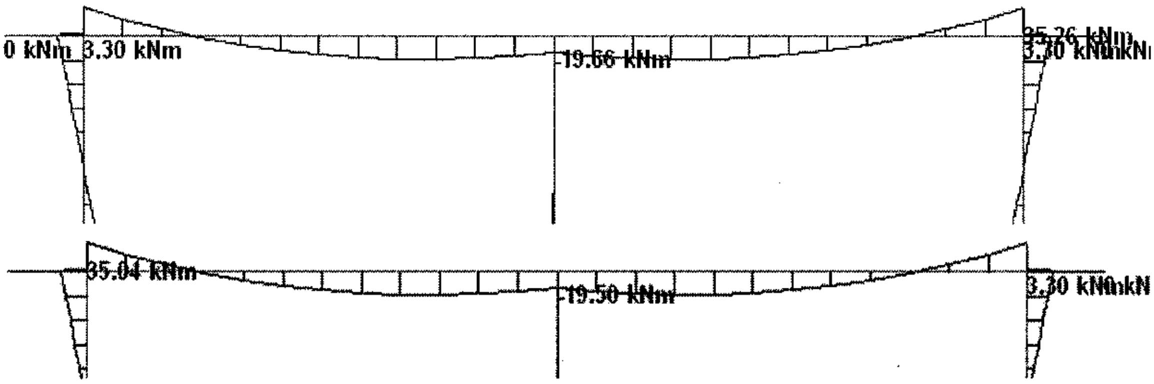


Upper

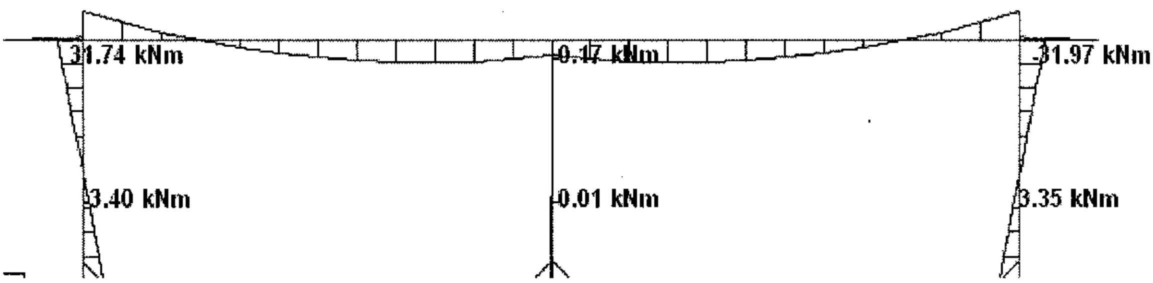


Shear Force Floor 12

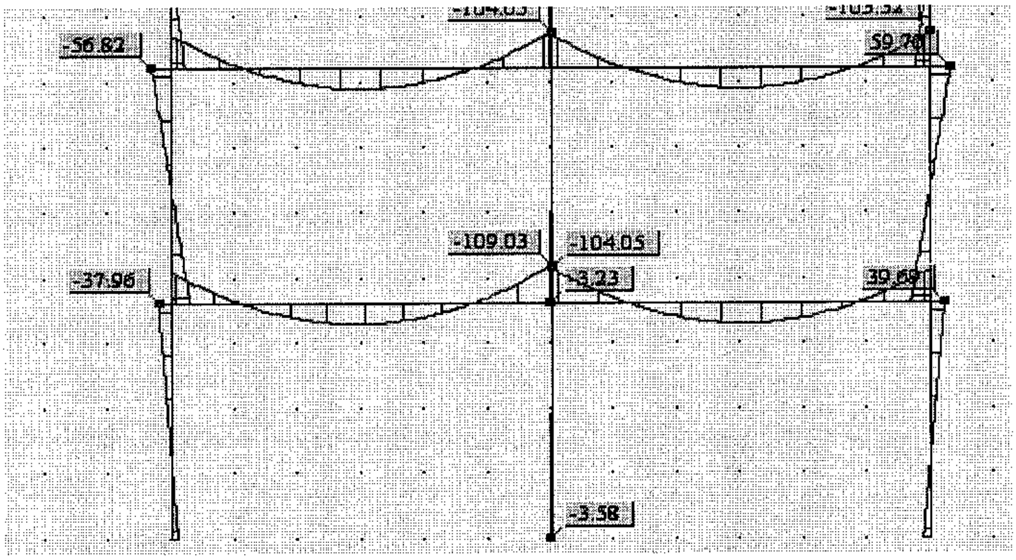
Bending Moment Floor 12



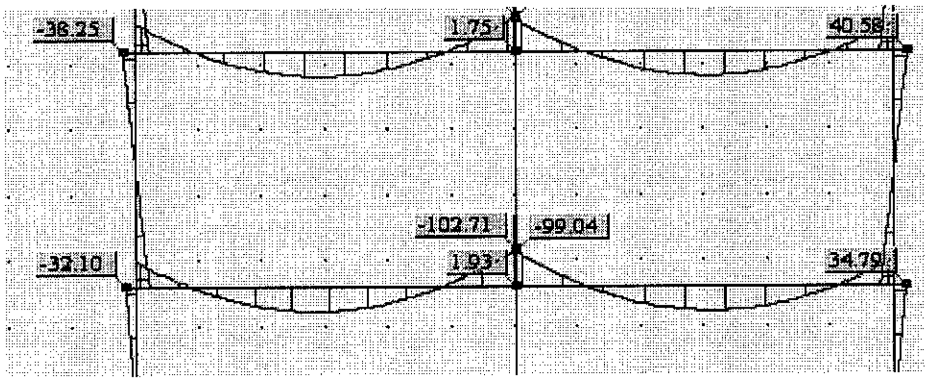
Column Moment Floor 12
Lower



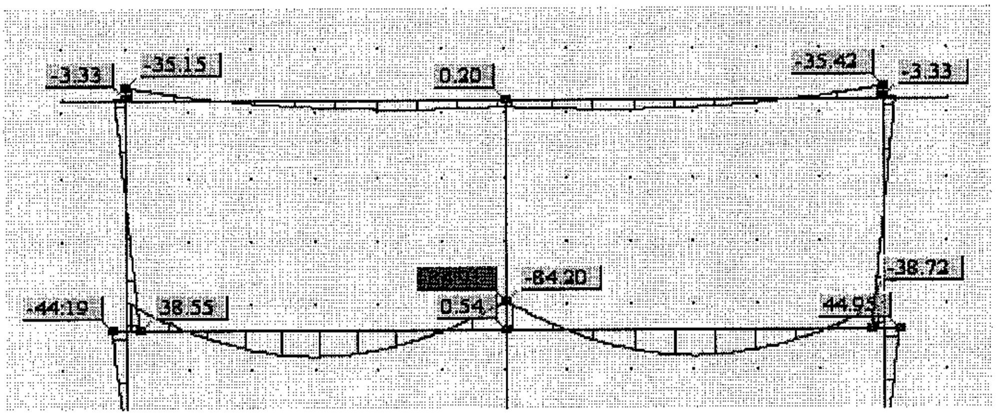
Bending Moment Floor 1



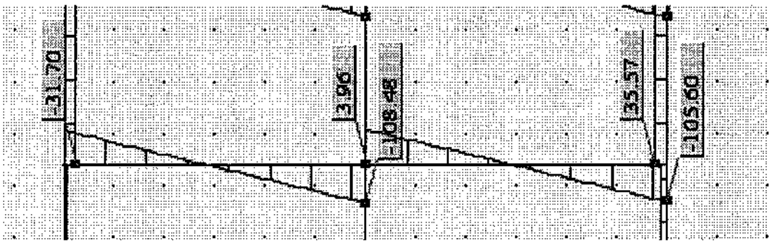
Bending Moment Floor 6



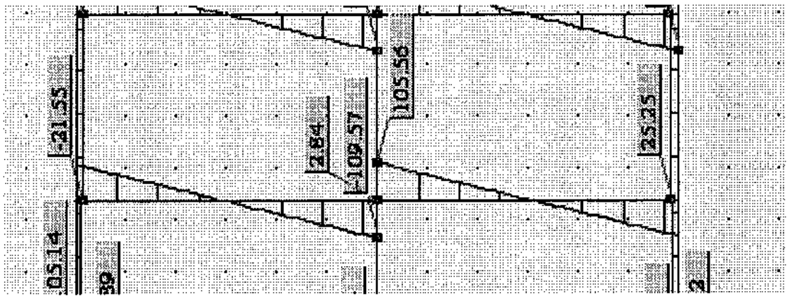
Bending Moment Floor 12



Shear Force Floor 1

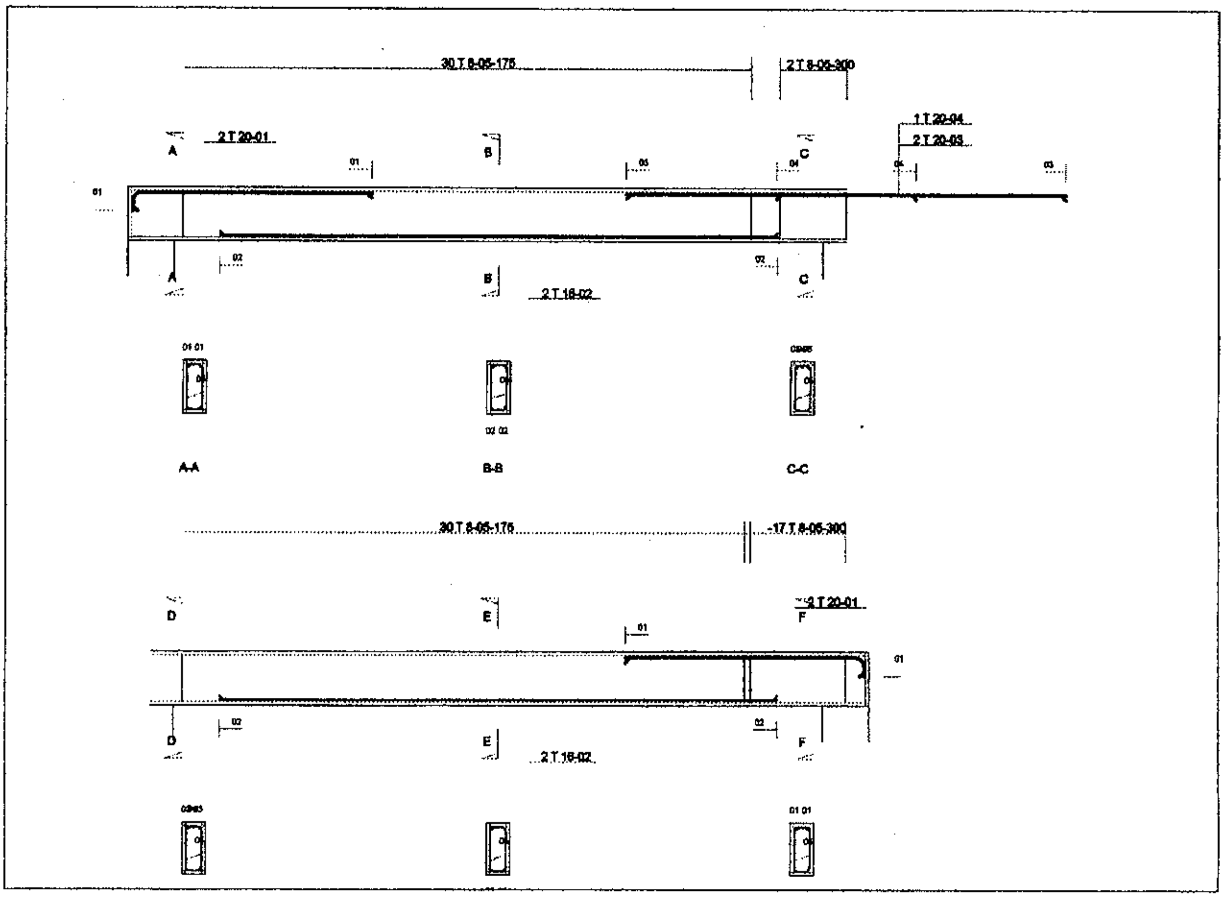


Shear Force Floor 6



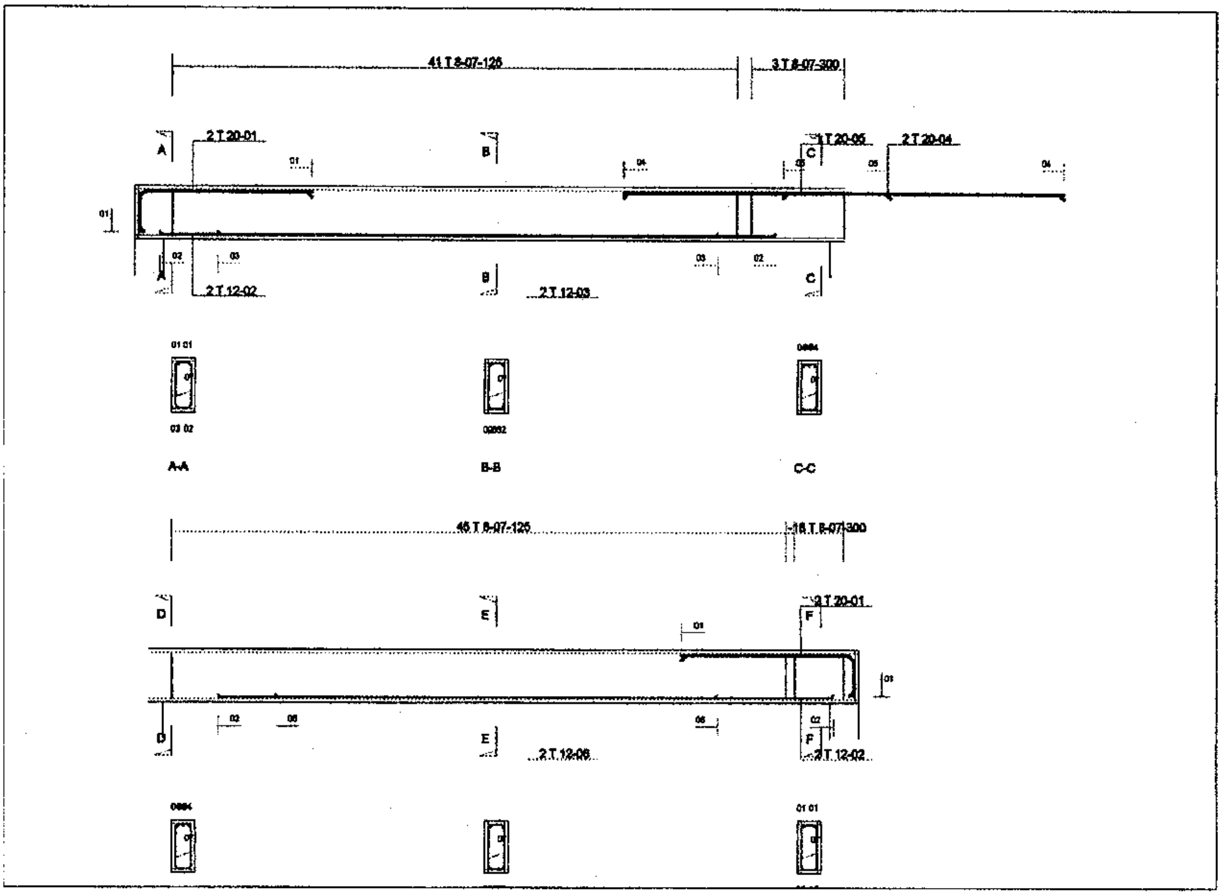
Shear Force Floor 12

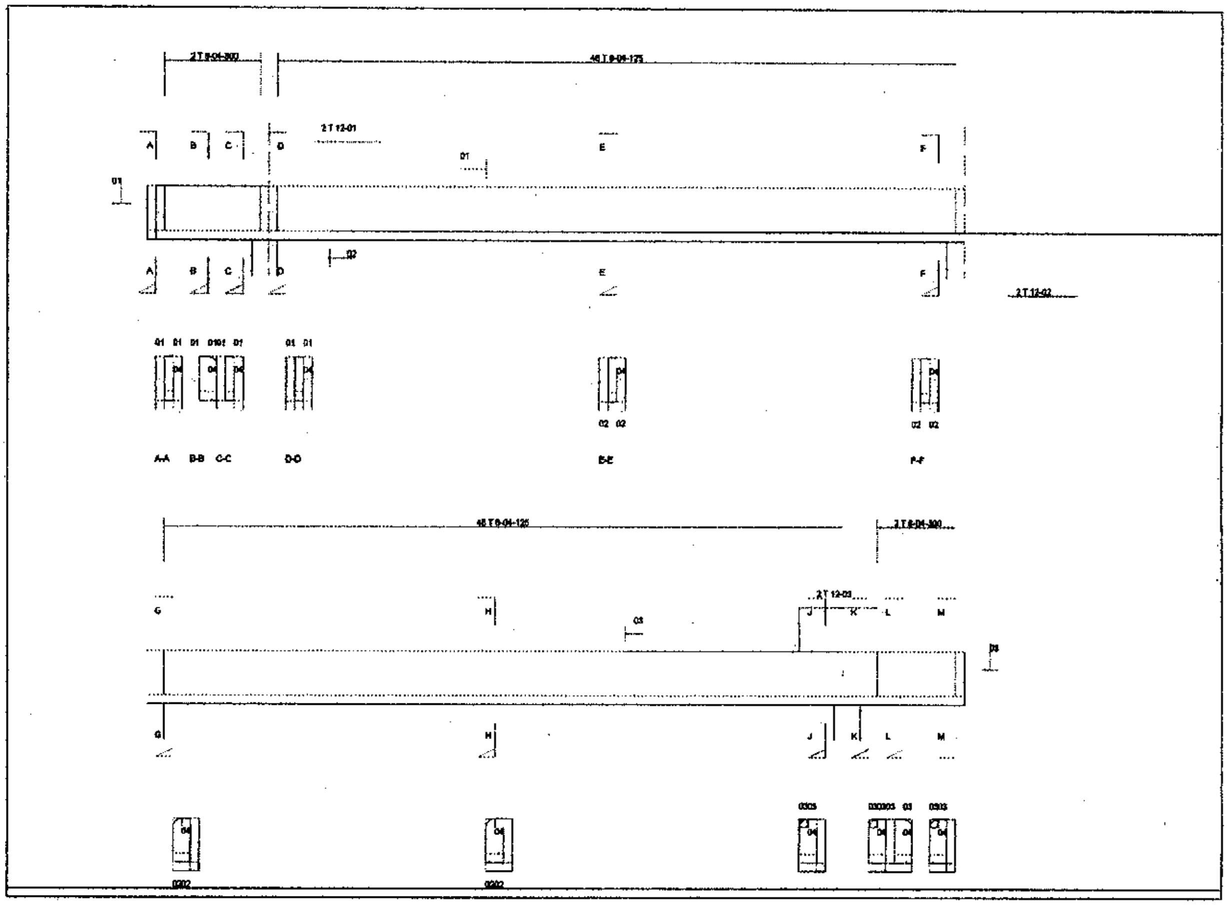
APPENDIX D
STAAD PRO AND ROBOT MILLENNIUM DETAILING RESULTS DIAGRAM
(BEAMS AND COLUMNS)



30 T 8-05-175
2 T 8-05-300
2 T 20-01
1 T 20-04
2 T 20-03
2 T 16-02
A-A
B-B
C-C
30 T 8-05-175
17 T 8-05-300
2 T 20-01
2 T 16-02

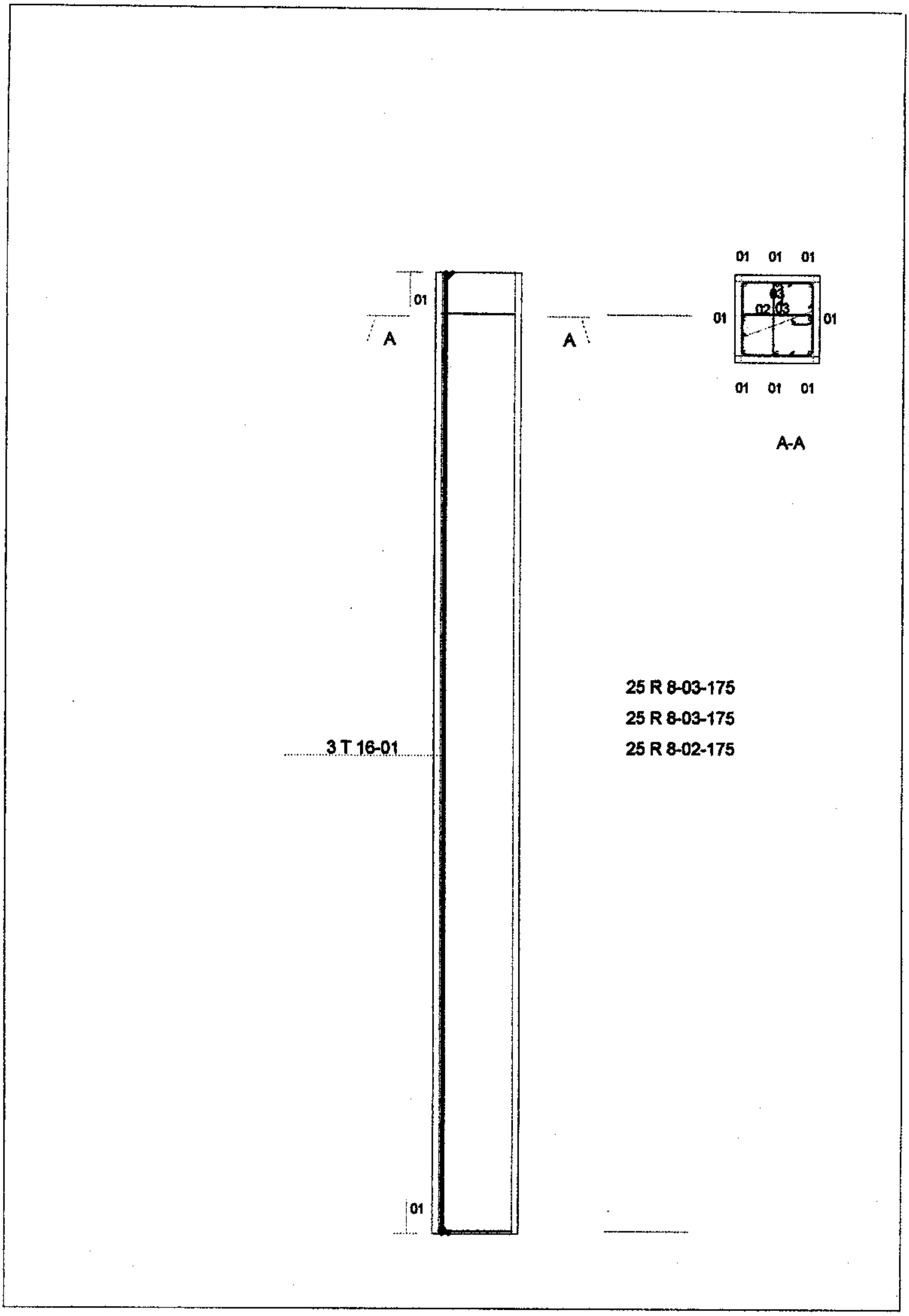
Final Year Project Software licensed toSnow Panther [LZ0]	Job No	Sheet No 2	Rev
	Part		
Title SIX FLOOR	Ref		
	By	Date 17-Sep-04	Chd
	File	Date/Time	

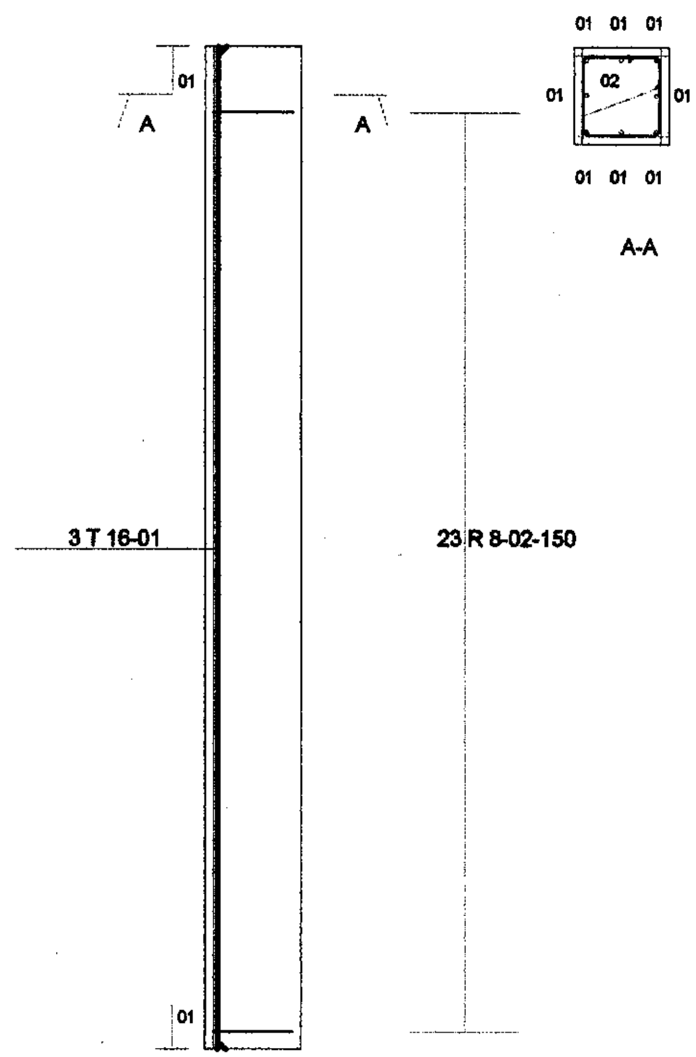




2 T 12-01
2 T 12-02
A-A
B-B
C-C
D-D
E-E
F-F

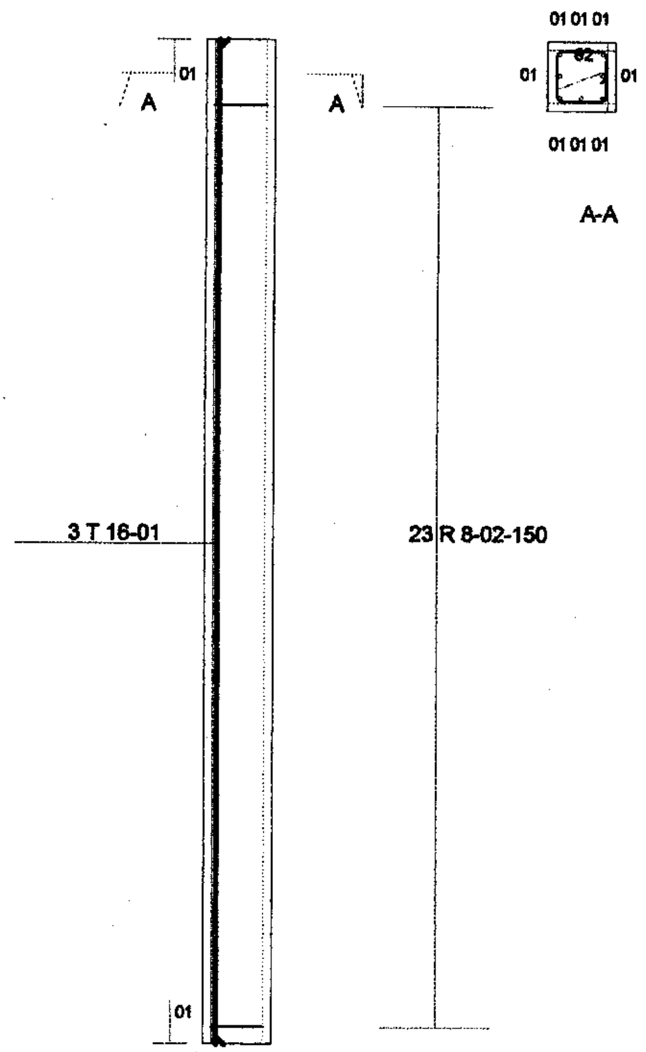
Final Year Project Software licensed toSnow Panther [LZ0]	Job No	Sheet No 1	Rev
	Part		
Title 400mm x 400mm column	Ref		
	By	Date 17-Sep-04	Chd
t	File	Date/Time	



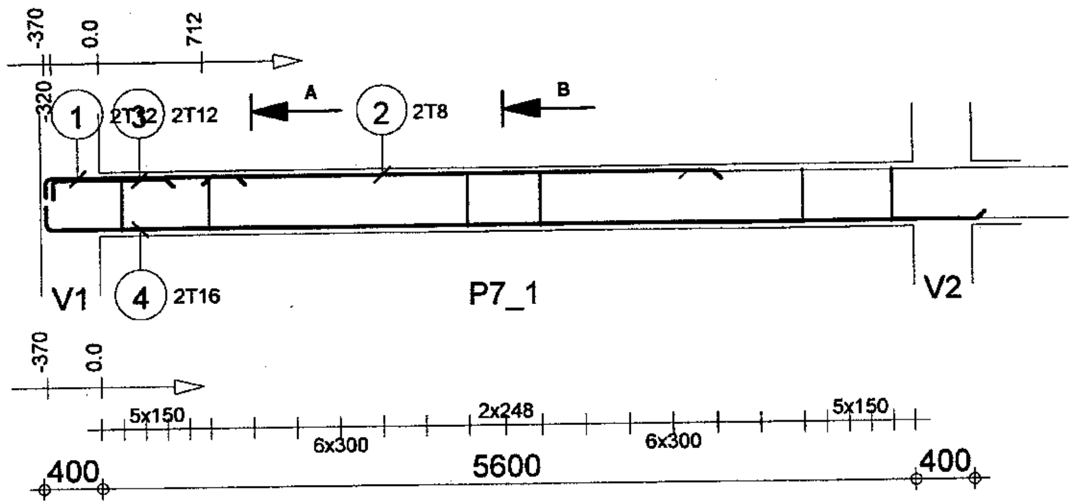


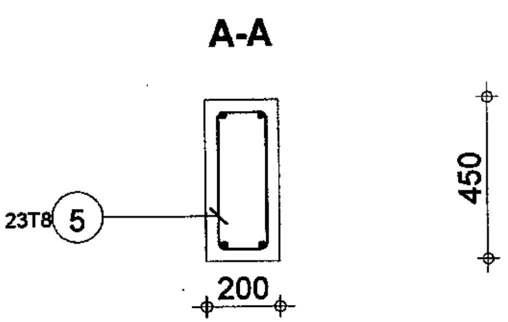
01
A
A
01 01 01
02
01
01
01 01 01
A-A
3 T 16-01
23 R 8-02-150
01

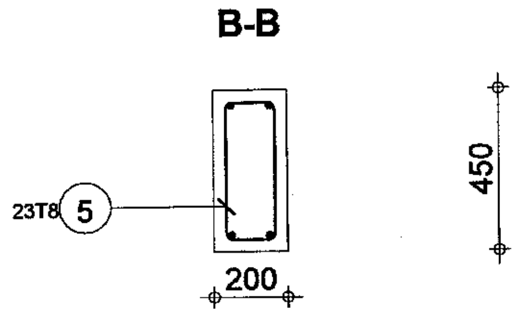
Final Year Project	Job No	Sheet No 3	Rev
Software licensed toSnow Panther [LZ0]	Part		
b Title 250mm x 250mm column	Ref		
	By	Date 17-Sep-04	Chd
ient	File	Date/Time	



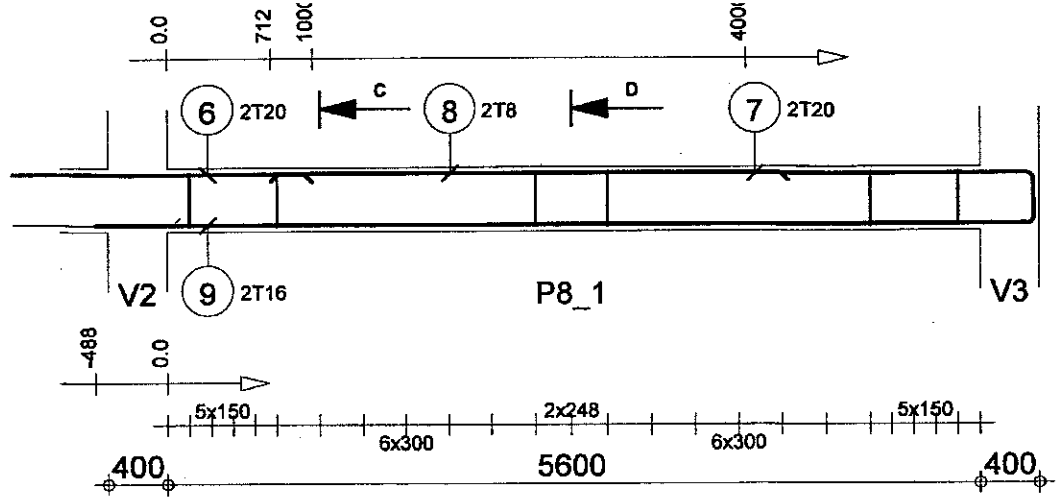
1	2T12	l=1477	37	1370
2	2T8	l=3576	20	3576
3	2T12	l=935	37	132 828
4	2T16	l=6601	37	176 6458
5	23T8	l=1153	60	390 140



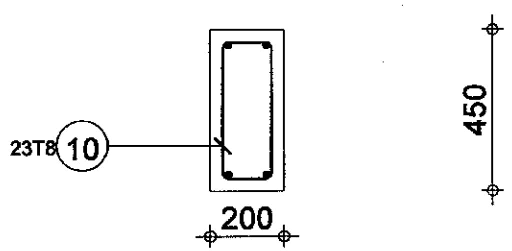


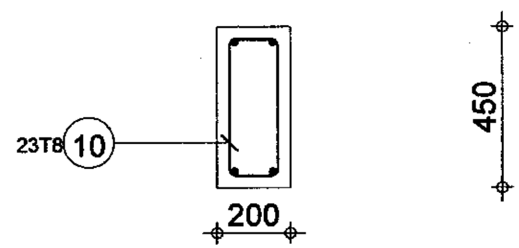


Standard Level **STRUCT 2D**	**Beam7...8** (FLOOR 1) **Section 200x450**	Steel T = 38.4kg — Concrete = 0.558m3 Formwork = 6.79m2 Bottom cover 30cm — Top cover 30cm Side cover 30cm View scale 1:50 Section scale 1:20



7	2T20	l=2149	37	220 1970
8	2T8	l=3576	20	3576
9	2T16	l=6601	37	176 6458
10	23T8	l=1153	60	390 140





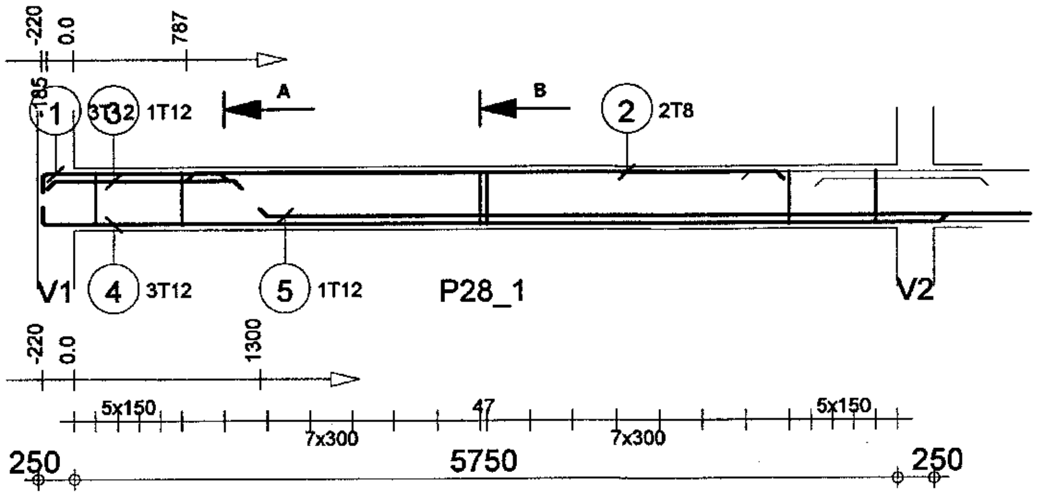
Standard Level

STRUCT 2D

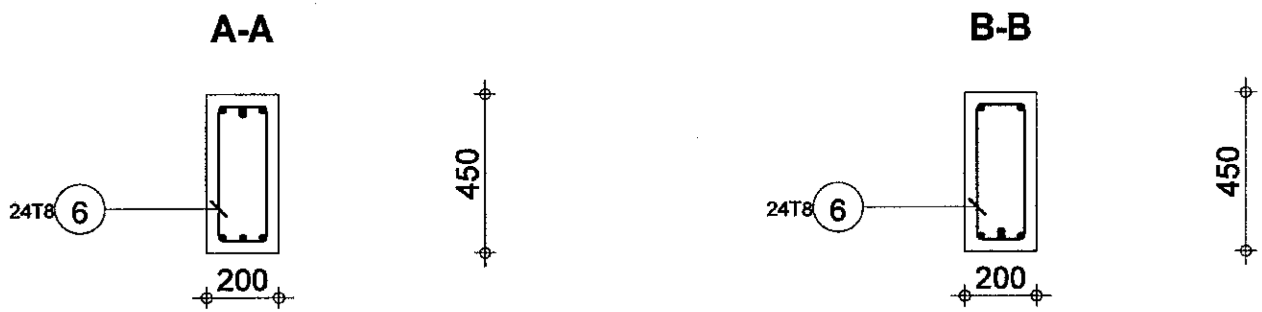
Beam7...8 (FLOOR 1)

Section 200x450

Steel T = 59.5kg	Concrete = 0.558m3
	Formwork = 6.79m2
Bottom cover 30cm	Top cover 30cm
Side cover 30cm	
View scale 1:50	
Section scale 1:20	



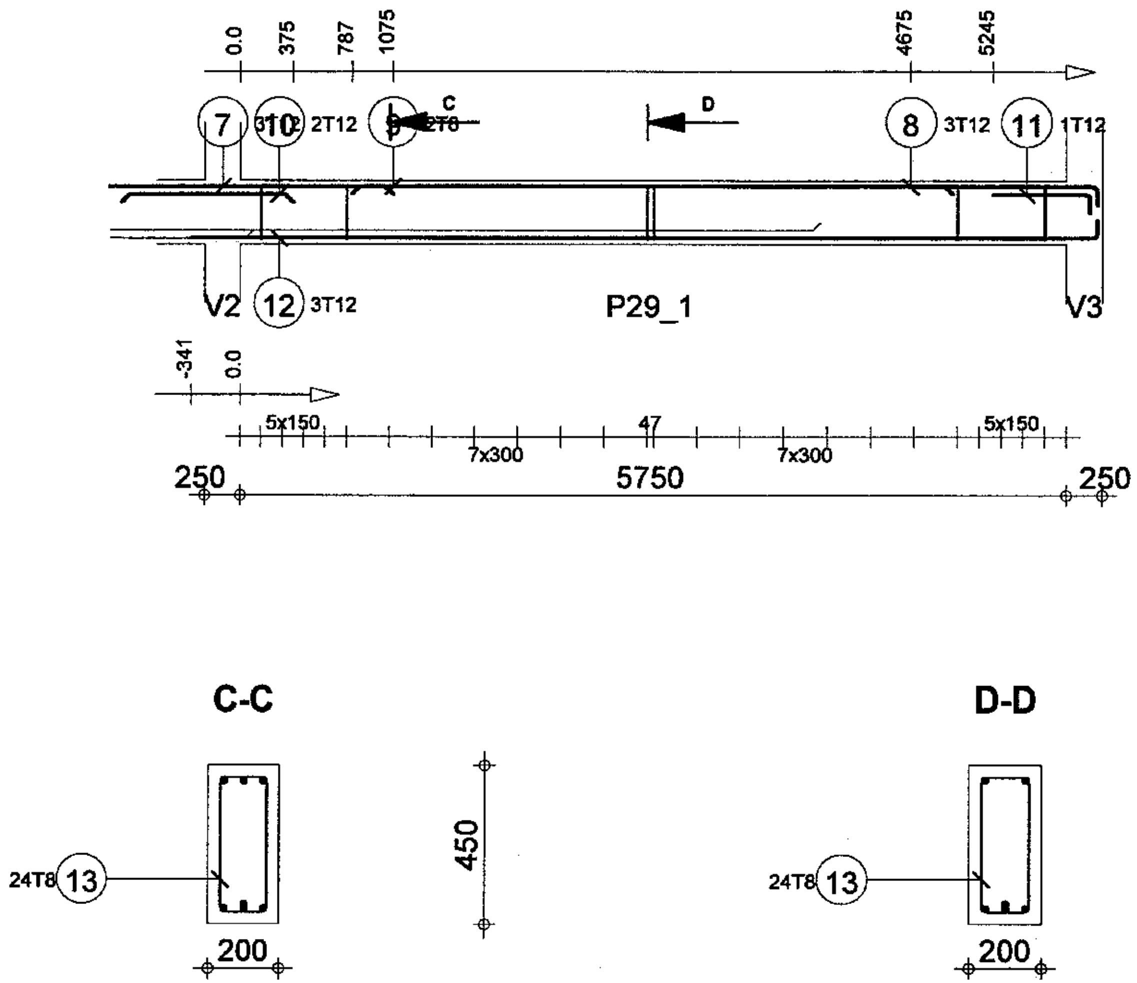
1	3T12	l=1402	37	132 / 1295
2	2T8	l=4176	20	4176
3	1T12	l=1360	20	1360
4	3T12	l=6418	37	132 / 6311
5	1T12	l=8750	20	8750
6	24T8	l=1153	60	390 / 140



Standard Level
STRUCT 2D

Beam28...29 (FLOOR 6)
Section 200x450

Steel T = 44kg	Concrete = 0.551m3 Formwork = 6.75m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	



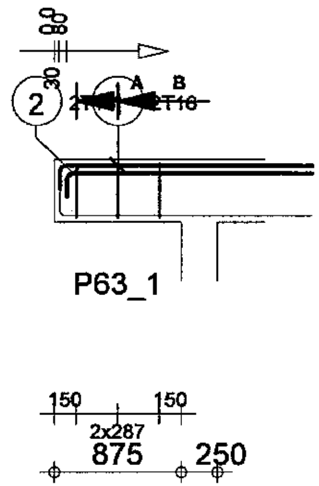
7	3T12	l=2400	20	2400
8	3T12	l=1402	37	132 1295
9	2T8	l=4176	20	4176
10	2T12	l=1200	20	1200
11	1T12	l=782	37	132 675
12	3T12	l=6418	37	132 6311
13	24T8	l=1153	60	390 140

Standard Level
STRUCT 2D

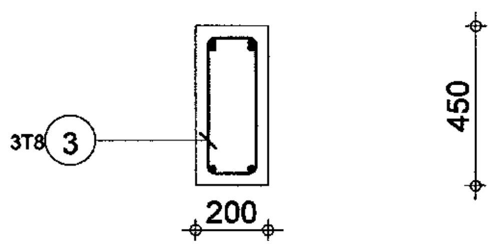
Beam28...29 (FLOOR 6)
Section 200x450

Steel T = 44.3kg	Concrete = 0.551m3 Formwork = 6.75m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	

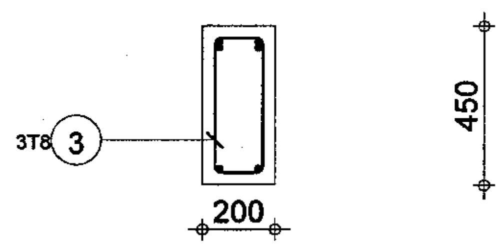
1	2T16	l=4715	37	4570
2	2T16	l=2563	37	176 2420
3	3T8	l=1153	60	390 140



A-A



B-B



Standard Level

STRUCT 2D

Beam40...64 (FLOOR 12)

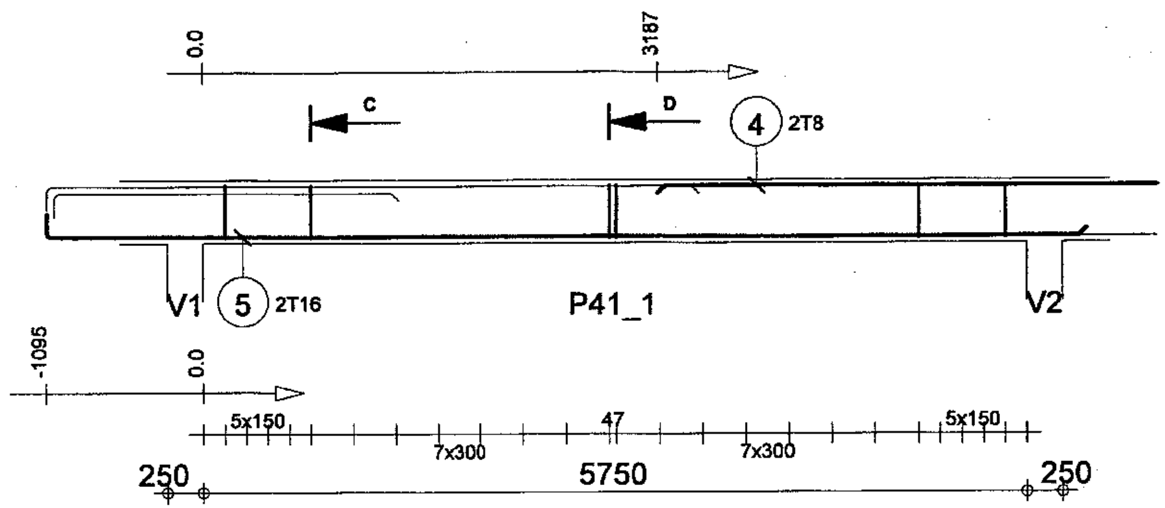
Section 200x450

Steel T = 24.3kg
Concrete = 0.09m3
Formwork = 1.17m2

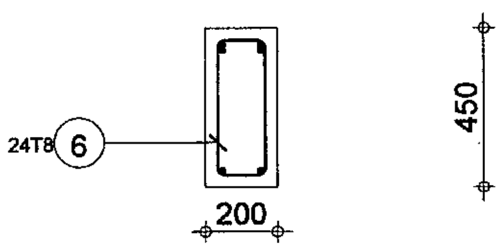
Bottom cover 30cm
Top cover 30cm
Side cover 30cm

View scale 1:50
Section scale 1:20

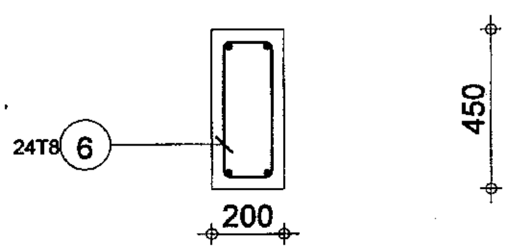
4	2T8	l=7176	20	7176
5	2T16	l=7401	37	176 7258
6	24T8	l=1153	60	390 140



C-C



D-D

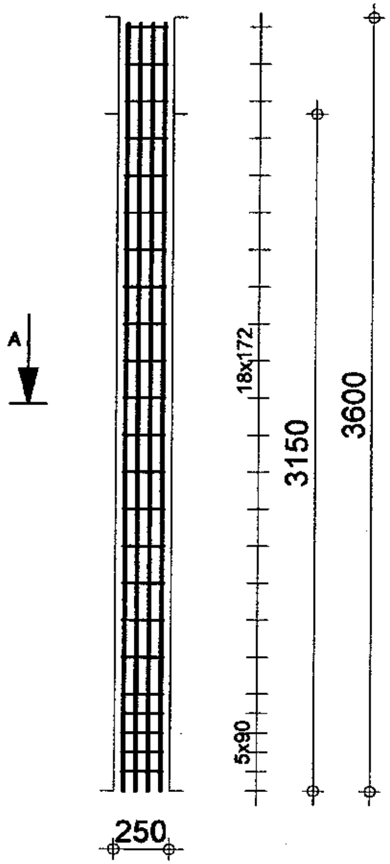


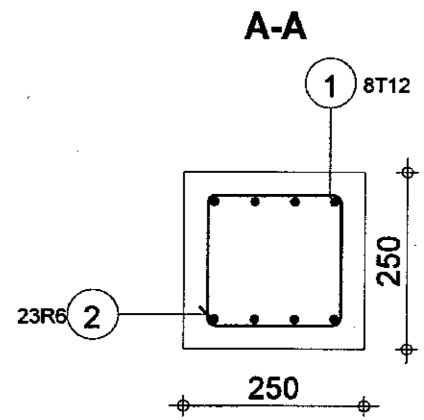
Steel T = 39.9kg	Concrete = 0.54m3 Formwork = 6.55m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	

Standard Level
STRUCT 2D

Beam40...64 (FLOOR 12)
Section 200x450

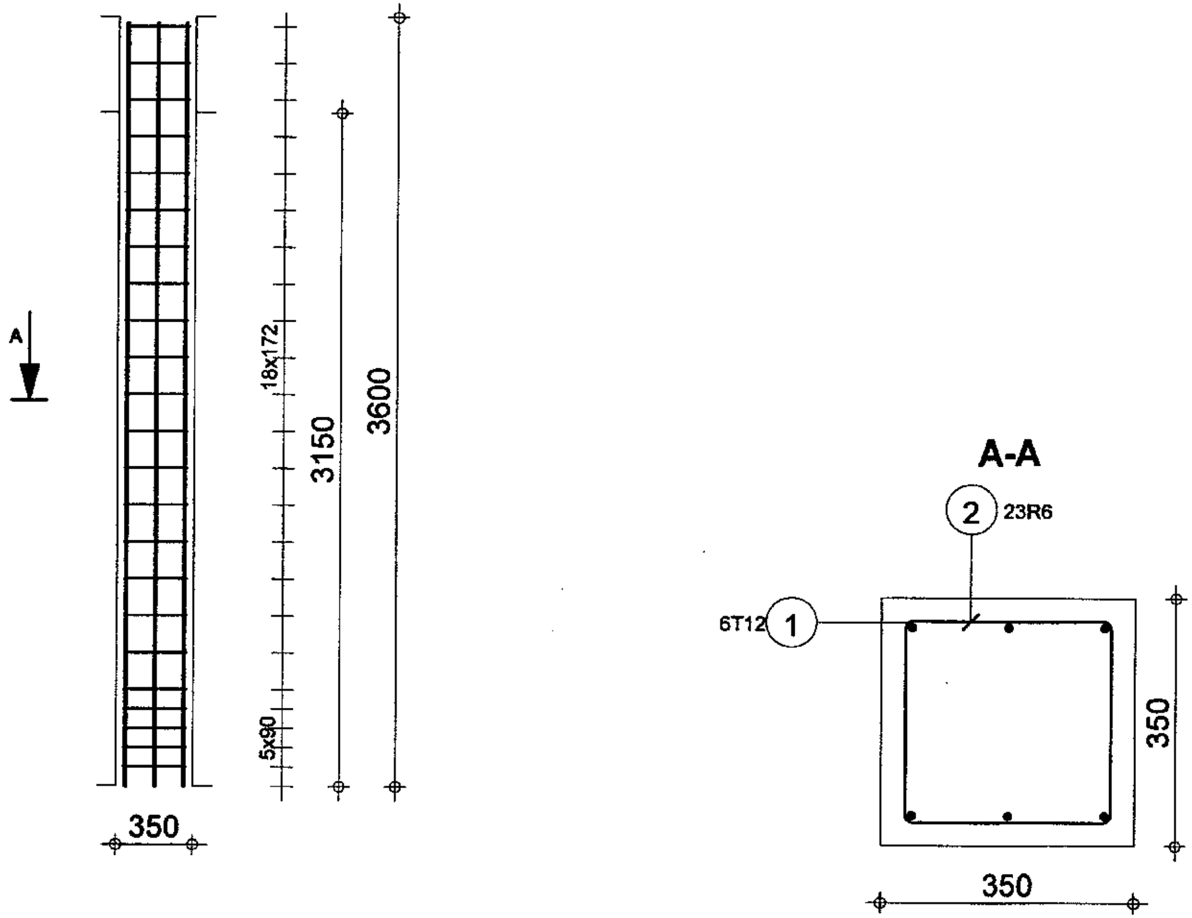
1	8T12	l=3570	20	3570
2	23R6	l=830	60	190 × 190



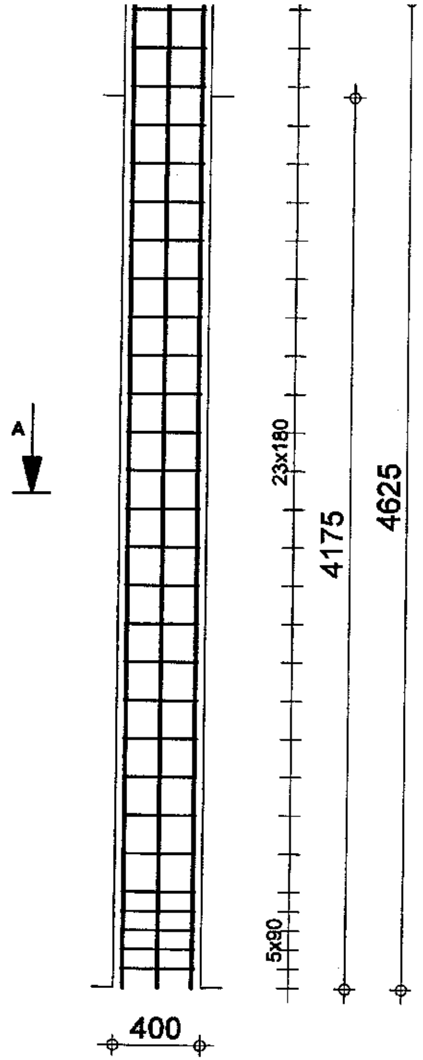


Standard Level STRUCT 2D	Tel. Fax Column60 Section 250x250	Steel T = 25.4kg Concrete = 0.197m3 Steel R = 4.24kg Formwork = 3.15m2 Cover 30cm View scale 1/33 Section scale 1/10

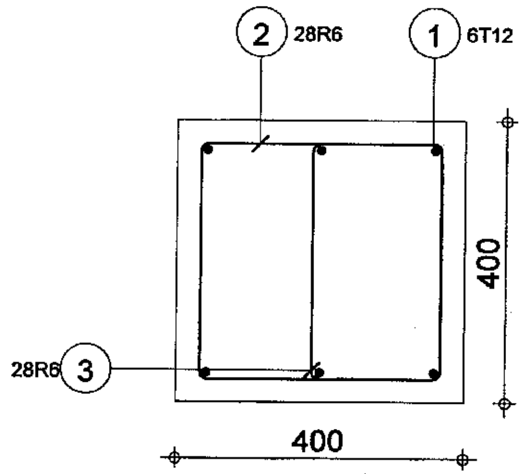
1	6T12	[illegible]	20	[illegible]
2	23R6	l=1230	60	290 / 290



	Tel. Fax	Steel T = 19kg Concrete = 0.386m3 Steel R = 6.28kg Formwork = 4.41m2
Standard Level **STRUCT 2D**	**Column14** **Section 350x350**	Cover 30cm
		View scale 1/33 Section scale 1/10



A-A

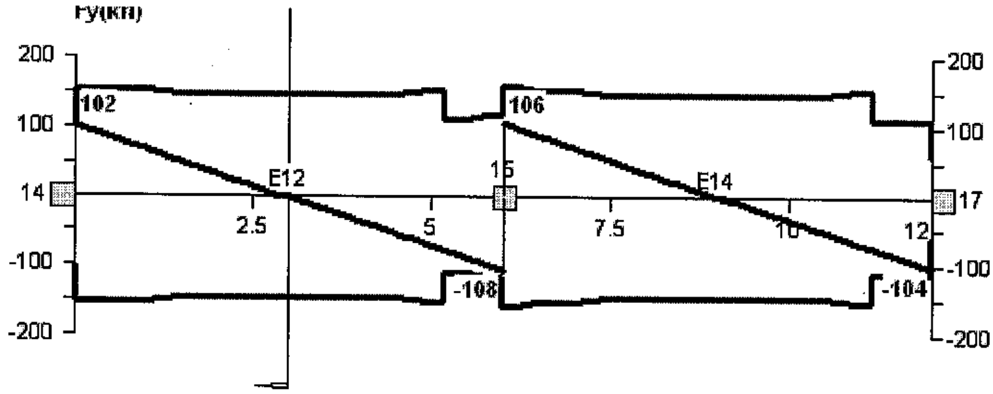


1	6T12	l=4595	20	4595
2	28R6	l=1430	60	340 340
3	28R6	l=442	35	340

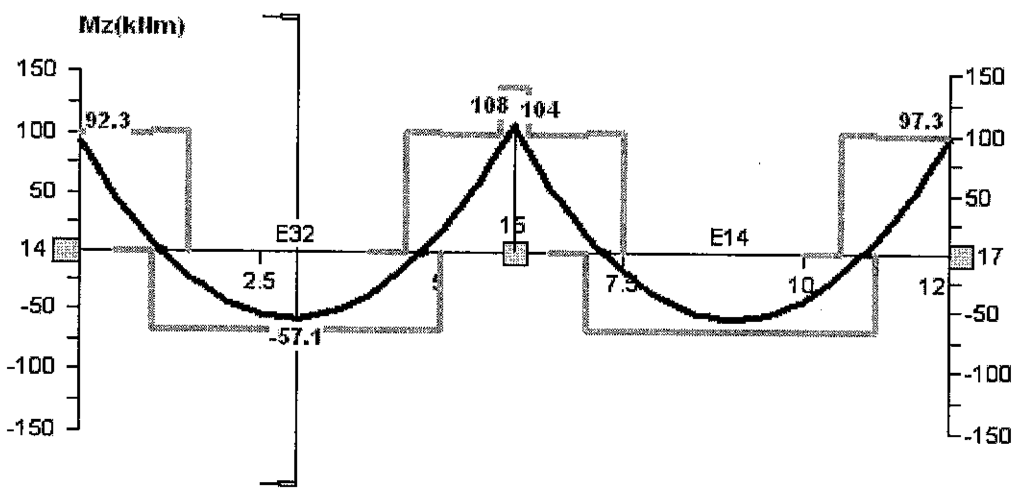
Standard Level STRUCT 2D	Tel. Fax **Column1** Section 400x400	Steel T = 24.5kg Steel R = 11.6kg	Concrete = 0.668m3 Formwork = 6.68m2
		Cover 30cm	
		View scale 1/33 Section scale 1/10	

APPENDIX E
STAAD PRO AND ROBOT MILLENNIUM DETAILING CALCULATIONS
(SELECTED BEAMS AND COLUMNS)

1ST STOREY
BEAM 1(M1)
(200mm x 450mm)



SHEAR FORCE DIAGRAM



BENDING MOMENT DIAGRAM

Member M1 Span 1

Detailed BS8110 Design Requirements

Section Property: 200 x 450

	Span Length	= 6.000 m	Rectangular section	
	Width	= 200 mm	Depth	= 450 mm
Covers:	Top = 30 mm	Bottom = 30 mm	Side = 30 mm	

Member M1 Span 1

Detailed BS8110 Main Reinforcement

Hogging: at 0.000 m from the start of the member

Moment applied to section		= 92.28 kNm	
Effective depth of tension reinforcement	d	= 402 mm	
Depth to compression reinforcement	d'	= 54 mm	
Redistribution < 10%, hence	K'	= 0.156	3.4.4.4
$K = \dfrac{M}{bd^2 f_{cu}}$		= 0.082	
$K \le K'$ hence compression steel not required.			
$z = d\left(0.5 + \left(0.25 - \dfrac{K}{0.9}\right)^{1/2}\right) \le 0.95d$		= 361.49 mm	
$A_{sr} = \dfrac{M}{0.95 f_y z}$		= 582.55 mm²	
Tension Bars provided = 2T20			
Actual area of tension reinforcement		= 628.32 mm²	
Minimum area of tension reinforcement		= 0.13 %	3.12.5.3
Maximum area of tension reinforcement		= 4 %	3.12.6.1
Actual % of tension reinforcement		= 0.70 %	

Member M1 Span 1

Detailed BS8110 Main Reinforcement Cont...

Minimum horizontal distance between bars = h_{agg} + 5mm	= 25 mm	3.12.11.1
Smallest actual horizontal space between bars	= 84 mm	
Maximum spacing of tension bars $= \dfrac{47000}{f_s} \le 300$	= 165 mm	3.12.11.2.4
Largest actual space between tension bars	= 84 mm	
Maximum clear distance between beam face and nearest main bar in tension = max tension bar spacing / 2	= 83 mm	3.12.11.2.5
Actual clear distance between beam face and nearest main bar in tension	= 38 mm	
Actual neutral axis depth of section	= 96.65 mm	
Moment capacity of section	= 99.41 kNm	
	∴ OK	

Sagging: at 3.000 m from the start of the member

Moment applied to section = 57.10 kNm

Effective depth of tension reinforcement d = 404 mm

Depth to compression reinforcement d' = 54 mm

Redistribution < 10%, hence K' = 0.156 — 3.4.4.4

$$K = \frac{M}{bd^2 f_{cu}} = 0.050$$

$K \leq K'$ hence compression steel not required.

$$z = d\left(0.5 + \left(0.25 - \frac{K}{0.9}\right)^{0.5}\right) \leq 0.95d = 380.16 \text{ mm}$$

$$A_s = \frac{M}{0.95 f_y z} = 342.84 \text{ mm}^2$$

Tension Bars provided =2T16

Actual area of tension reinforcement = 402.12 mm²

Minimum area of tension reinforcement = 0.13 % — 3.12.5.3

Maximum area of tension reinforcement = 4 % — 3.12.6.1

Actual % of tension reinforcement = 0.45 %

<u>Member M1 Span 1</u>

<u>Detailed BS8110 Main Reinforcement Cont...</u>

Minimum horizontal distance between bars = h_{agg} + 5mm = 25 mm — 3.12.11.1

Smallest actual horizontal space between bars = 92 mm

Maximum spacing of tension bars $= \frac{47000}{f_s} \leq 300$ = 180 mm — 3.12.11.2.4

Largest actual space between tension bars = 92 mm

Maximum clear distance between beam face and nearest main bar in tension
= max tension bar spacing / 2 = 90 mm — 3.12.11.2.5

Actual clear distance between beam face and nearest main bar in tension
= 38 mm

Actual neutral axis depth of section = 61.99 mm

Moment capacity of section = 66.09 kNm

∴ OK

Hogging: at 6.000 m from the start of the member

Moment applied to section = 108.20 kNm

Effective depth of tension reinforcement d = 402 mm

Depth to compression reinforcement d' = 54 mm

Redistribution < 10%, hence K' = 0.156 — 3.4.4.4

$$K = \frac{M}{bd^2 f_{cu}} = 0.096$$

$K \leq K'$ hence compression steel not required.

$$z = d\left(0.5 + \left(0.25 - \frac{K}{0.9}\right)^{0.5}\right) \leq 0.95d = 353.40 \text{ mm}$$

$$A_s = \frac{M}{0.95 f_y z} = 698.69 \text{ mm}^2$$

Tension Bars provided =3T20

Actual area of tension reinforcement = 942.48 mm²

Minimum area of tension reinforcement = 0.13 % — 3.12.5.3

Maximum area of tension reinforcement = 4 % — 3.12.6.1

Actual % of tension reinforcement = 1.05 %

Member M1 Span 1

Detailed BS8110 Main Reinforcement Cont...

Minimum horizontal distance between bars = h_{agg} + 5mm	= 25 mm	3.12.11.1
Smallest actual horizontal space between bars	= 32 mm	
Maximum spacing of tension bars $= \frac{47000}{f_s} \leq 300$	= 207 mm	3.12.11.2.4
Largest actual space between tension bars	= 32 mm	
Maximum clear distance between beam face and nearest main bar in tension = max tension bar spacing / 2	= 103 mm	3.12.11.2.5
Actual clear distance between beam face and nearest main bar in tension	= 38 mm	
Actual neutral axis depth of section	= 146.28 mm	
Moment capacity of section	= 138.64 kNm	
	∴ OK	

Member M1 Span 1

Detailed BS8110 Span / Effective Depth Check

Basic span / effective depth ratio		= 26.0	3.4.6.3
$f_s = \frac{2 f_y A_{s,req}}{3 A_{s,prov}} \times \frac{1}{\beta_b}$	$= \frac{2 \times 460 \times 342.8}{3 \times 402.1} \times 1$	= 261.45 N/mm²	3.4.6.5
Mod. factor for tension rft.	$= 0.55 + \frac{(477 - f_s)}{120(0.9 + M/bd^2)} \leq 2.0$	= 1.23	3.4.6.5
Mod. factor for compression rft.	$= 1 + \frac{100A'_{s,prov}}{bd} / \left(3 + \frac{100A'_{s,prov}}{bd}\right) \leq 1.5$	= 1.00	3.4.6.6
Hence, modified span / effective depth ratio		= 31.93	
Actual span / effective depth ratio		= 14.86	SAFE

Member M1 Span 1

Detailed BS8110 Shear Reinforcement

High shear zone: 0.001 m to 5.175 m

Maximum shear force within zone, V		= 102.23 kN	
$v = \frac{V}{b_v d}$		= 1.27 N/mm²	3.4.5.2
$\leq 0.8 \sqrt{f_{cu}}$ and 5 N/mm²	hence dimensions adequate		
$v_c = 0.79 \left(\frac{100 A_s}{b_v d}\right)^{1/3} \left(\frac{400}{d}\right)^{1/4} / \gamma_m$			3.4.5.4
	$= 0.79 \times 0.78^{1/3} \times 1.00^{1/4} / 1.25$	= 0.65 N/mm²	

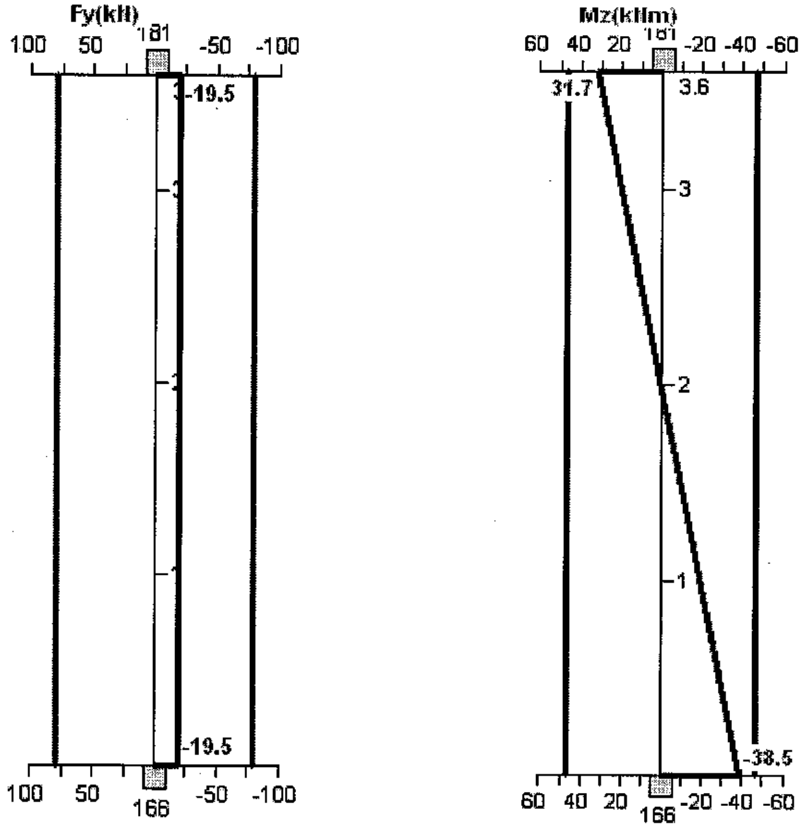
Member M1 Span 1
Detailed BS8110 Shear Reinforcement Cont...

$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ or 5 N/mm²		3.4.5.3
spacing provided, s_v	= 175 mm	
minimum area of links = $b_v s_v (v - v_c) / 0.95 f_{yv}$	= 49.68 mm²	
area of links provided (2T8), A_{sv}	= 100.53 mm²	
distance between main bar in compression zone and a restrained bar ≤150 mm		3.12.7.2
	∴ OK	

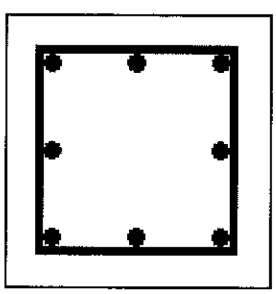
High shear zone: 5.175 m to 5.999 m

Maximum shear force within zone, V	= 107.54 kN	
$v = \frac{V}{b_v d}$	= 1.34 N/mm²	3.4.5.2
$\leq 0.8\sqrt{f_{cu}}$ and 5 N/mm² hence dimensions adequate		
$v_c = 0.79\left(\frac{100\,A_s}{b_v d}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4} / \gamma_m$		3.4.5.4
$= 0.79 \times 1.17^{1/3} \times 1.00^{1/4} / 1.25$	= 0.75 N/mm²	
$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ or 5 N/mm²		3.4.5.3
spacing provided, s_v	= 300 mm	
minimum area of links = $b_v s_v (v - v_c) / 0.95 f_{yv}$	= 47.43 mm²	
area of links provided (2T8), A_{sv}	= 100.53 mm²	
distance between main bar in compression zone and a restrained bar ≤150 mm		3.12.7.2
	∴ OK	

11TH FLOOR
COLUMN 1(M32)
(250MM X 250MM)



Main reinforcement T16



23R8 @ 150mm

250 x 250mm

Member 32 - Detailed BS8110 Design Requirements

Section Property: 250 x 250

Storey height	= 3.600 m			
Rectangular section:	Width	= 250 mm	Depth	= 250 mm
	Cover	= 30 mm		

Member 32 - Detailed BS8110 Main Reinforcement

Shear bars not designed. Depth of main reinforcement taken as cover + brief link size

Axial N		= 35.46 kN	
Major M,	end 1,	= 31.74 kNm	
	end 2,	= -38.53 kNm	
Minor M,	end 1,	= 0.00 kNm	
	end 2,	= 0.00 kNm	
Column Type:	SHORT		3.8.1.3
Design M_x		= 38.53 kNm	
Design M_y		= 0.00 kNm	
Area of steel required		= 914.45 mm²	
Actual neutral axis depth in section, x		= 59.21 mm	
Force produced by concrete stress block		= 205.49 kN	
Force produced by main steel		= -170.03 kN	
Σ internal forces		= 35.46 kN	
Internal forces $\geq$ axial N		$\therefore$ OK for axial resistance	
Moment produced by concrete stress block		= 20.24 kNm	
Moment produced by main steel		= 28.32 kNm	
Σ internal moments		= 48.56 kNm	
Internal moments $\geq$ design moment		$\therefore$ OK for moment resistance	
Min distance between bars	= h_{agg} + 5 mm or bar diameter	= 25 mm	3.12.11.1
Smallest actual space between bars		= 63 mm	
Max allowable spacing of bars in designed faces = $\frac{47000}{f_s} \leq 300$		= 202 mm	3.12.11.2.4
Largest actual clear space between bars in designed faces		= 63 mm	
Max allowable spacing of bars in other faces		= 160 mm	Table 3.28
Largest actual clear space between bars in other faces		= 63 mm	
Actual area of main reinforcement (8T16)		= 1608.50 mm²	

Member 32 - Detailed BS8110 Main Reinforcement Cont...

Min area of compression reinforcement	= 0.4 %	3.12.5.3
Max area of reinforcement	= 6 % (8 % if horizontally cast)	3.12.6.2
Actual area of main reinforcement	= 2.57 %	
Max area of reinforcement at laps	= 10 %	3.12.6.2
∴ area by which steel can increase at laps	= 4641.50 mm^2	
	∴ OK	

Member 32 - Detailed BS8110 Shear Reinforcement

Spacing provided, s_v	= 150 mm	
Major axis:		
Shear force, V	= 19.52 kN	
Moment, M	= 36.53 kNm	
$v = \dfrac{V}{b_v d}$	= 0.38 N/mm^2	3.4.5.2
$<= 0.8 \sqrt{f_{cu}}$ and 5 N/mm^2 ∴ dimensions adequate		
$v_c = 0.79 \left(\dfrac{100 A_s}{b_v d}\right)^{1/3} \left(\dfrac{400}{d}\right)^{1/4} / \gamma_m = 0.79 \times 1.18^{1/3} \times 1.98^{1/4} / 1.25$	= 0.79 N/mm^2	3.4.5.4
$f_{cu} > 25$ ∴ $v_c = v_c \times (f_{cu} / 25)^{1/3}$ (f_{cu} limited to 40)	= 0.88 N/mm^2	Table 3.8
$v_c' = v_c + 0.6 \dfrac{N}{A_c} \dfrac{V h}{M} = 0.88 + 0.6 \times 0.57 \times 0.13$	= 0.93 N/mm^2	3.4.5.12
$v \le v_c'$ but M/N > 0.6 h ∴ provide links as per clause 3.4.5.3 using v_c' instead of v_c		3.8.4.6
Area of links required = 0.4 $b_v s_v$ / 0.95 f_{yv}	= 63.16 mm^2	
Area of links provided (2R8), A_{sv}	= 100.53 mm^2	

Minor axis:		
Shear force, V	= 19.52 kN	
Moment, M	= 0.00 kNm	
$v = \dfrac{V}{b_v d}$	= 0.38 N/mm^2	3.4.5.2
$<= 0.8 \sqrt{f_{cu}}$ and 5 N/mm^2 ∴ dimensions adequate		
$v_c = 0.79 \left(\dfrac{100 A_s}{b_v d}\right)^{1/3} \left(\dfrac{400}{d}\right)^{1/4} / \gamma_m = 0.79 \times 1.18^{1/3} \times 1.98^{1/4} / 1.25$	= 0.79 N/mm^2	3.4.5.4
$f_{cu} > 25$ ∴ $v_c = v_c \times (f_{cu} / 25)^{1/3}$ (f_{cu} limited to 40)	= 0.88 N/mm^2	Table 3.8
$v_c' = v_c + 0.6 \dfrac{N}{A_c} \dfrac{V h}{M} = 0.88 + 0.6 \times 0.57 \times 1.00$	= 0.88 N/mm^2	3.4.5.12
$v \le v_c'$ ∴ only nominal links required		3.4.5.12
Area of links required = 0.4 $b_v s_v$ / 0.95 f_{yv}	= 63.16 mm^2	
Area of links provided (2R8), A_{sv}	= 100.53 mm^2	

Member 32 - Detailed BS8110 Shear Reinforcement Cont...

Minimum size of link bars	= 6 mm	3.12.7.1
Actual size of link bars	= 8 mm	
Maximum spacing of links	= 153 mm	3.4.5.5/3.12.7.1
	∴ OK	

1 Level:

- N ame : Standard Level
- R eference level : ---
- F ire rating : 0 (h)
- M aximum cracking : 0.30 (mm)
- E nvironment class : moderate
- C oncrete creeping coefficient: $\varphi_p = 2.00$

2 Beam: Beam7...8 Number: 1

2.1 Material properties:

- C oncrete : C35 $f_{cu} = 35.00$ (N/mm2)
  Unit weight : 24.00 (kN/m3)
- L ongitudinal reinforcement : T $f_y = 460.00$ (N/mm2)
- T ransversal reinforcement : T $f_y = 460.00$ (N/mm2)

2.2 Geometry:

2.2.1 Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P7_1	**Span**	**0.40**	**5.60**	**0.40**

Span length: $L_0 = 6.00$ (m)
Section from 0.00 to 5.60 (m)
200.00 x 450.00 (mm)
without left slab
without right slab

2.2.2 Span	Position	L.supp. (m)	L (m)	R.supp. (m)
P8_1	**Span**	**0.40**	**5.60**	**0.40**

Span length: $L_0 = 6.00$ (m)
Section from 0.00 to 5.60 (m)
200.00 x 450.00 (mm)
without left slab
without right slab

2.5 Calculation options:

- C alculations according to : BS 8110
- P recast beam : no
- C over : bottom c = 30.00 (mm)
  : side c1 = 30.00 (mm)
  : top c2 = 30.00 (mm)

2.7 Calculation results:

No.	Type	State	Span	x(m)	Value	Capacity	Safety factor
1.	M [kN*m]	ULS	1	0.40	-71.21	-59.37	0.83
2.	M [kN*m]	ULS	2	12.00	-83.75	-53.67	0.64

2.7.1 Internal forces in ULS

Span	Mtmax. (kN*m)	Mtmin. (kN*m)	Ml (kN*m)	Mp (kN*m)	Ql (kN)	Qp (kN)
P7_1	57.19	0.00	-71.21	-91.30	94.31	-101.48
P8_1	56.67	0.00	-79.80	-83.75	97.19	-98.58

2.7.2 Internal forces in SLS

Span	Mtmax. (kN*m)	Mtmin. (kN*m)	Ml (kN*m)	Mp (kN*m)	Ql (kN)	Qp (kN)
P7_1	0.00	0.00	0.00	0.00	0.00	0.00
P8_1	0.00	0.00	0.00	0.00	0.00	0.00

2.7.4 Required reinforcement area

Span	Span (mm2)		Left support (mm2)		Right support (mm2)	
	bottom	top	bottom	top	bottom	top
P7_1	347.53	0.00	0.00	440.32	0.00	579.91
P8_1	344.14	0.00	0.00	499.01	0.00	526.49

2.7.5 Deflection and cracking

$a_{t(s-t)}$ - initial deflection due to total load
$a_{p(s-t)}$ - initial deflection due to long-term load
$a_{p(l-t)}$ - long-term deflection due to long-term load
a - total deflection
a_{all} - allowable deflection

Cw - width of perpendicular cracks

Span	$a_{t(s-t)}$ (mm)	$a_{p(s-t)}$ (mm)	$a_{p(l-t)}$ (mm)	a (mm)	a_{all} (mm)	Cw (mm)
P7_1	0.0000	0.0000	0.0000	0.0000=(Lo/--)	-24.0000	0.00
P8_1	0.0000	0.0000	0.0000	0.0000=(Lo/--)	-24.0000	0.00

2.8 Theoretical results - detailed results:

2.8.1 P7_1 : Span from 0.40 to 6.00 (m)

Abscissa (m)	ULS M max. (kN*m)	M min. (kN*m)	SLS M max. (kN*m)	M min. (kN*m)	A bottom (mm2)	A top (mm2)
0.40	0.00	-71.21	0.00	0.00	0.00	440.32
0.80	0.00	-34.87	0.00	0.00	0.00	206.55
1.40	7.01	0.00	0.00	0.00	117.00	0.00
2.00	36.33	0.00	0.00	0.00	215.43	0.00
2.60	53.05	0.00	0.00	0.00	320.77	0.00
3.20	57.19	0.00	0.00	0.00	347.53	0.00
3.80	48.75	0.00	0.00	0.00	293.24	0.00
4.40	27.72	0.00	0.00	0.00	162.80	0.00
5.00	0.00	-5.90	0.00	0.00	0.00	117.00
5.60	0.00	-52.09	0.00	0.00	0.00	314.70
6.00	0.00	-91.30	0.00	0.00	0.00	579.91

Abscissa (m)	ULS Q max. (kN)	SLS Q max. (kN)	Cw (mm)
0.40	94.31	0.00	0.00
0.80	80.32	0.00	0.00
1.40	59.34	0.00	0.00
2.00	38.37	0.00	0.00
2.60	17.39	0.00	0.00
3.20	-3.57	0.00	0.00
3.80	-24.55	0.00	0.00
4.40	-45.54	0.00	0.00
5.00	-66.50	0.00	0.00
5.60	-87.48	0.00	0.00

6.00	-101.48	0.00	0.00

2.8.2 P8_1 : Span from 6.40 to 12.00 (m)

	ULS		SLS			
Abscissa (m)	M max. (kN*m)	M min. (kN*m)	M max. (kN*m)	M min. (kN*m)	A bottom (mm2)	A top (mm2)
6.40	0.00	-79.80	0.00	0.00	0.00	499.01
6.80	0.00	-42.31	0.00	0.00	0.00	252.72
7.40	1.30	0.00	0.00	0.00	117.00	0.00
8.00	32.35	0.00	0.00	0.00	190.97	0.00
8.60	50.80	0.00	0.00	0.00	306.34	0.00
9.20	56.67	0.00	0.00	0.00	344.14	0.00
9.80	49.96	0.00	0.00	0.00	300.93	0.00
10.40	30.65	0.00	0.00	0.00	180.63	0.00
11.00	0.00	-1.22	0.00	0.00	0.00	117.00
11.60	0.00	-45.71	0.00	0.00	0.00	274.01
12.00	0.00	-83.75	0.00	0.00	0.00	526.49

	ULS	SLS	
Abscissa (m)	Q max. (kN)	Q max. (kN)	Cw (mm)
6.40	97.19	0.00	0.00
6.80	83.20	0.00	0.00
7.40	62.23	0.00	0.00
8.00	41.25	0.00	0.00
8.60	20.27	0.00	0.00
9.20	-0.69	0.00	0.00
9.80	-21.68	0.00	0.00
10.40	-42.66	0.00	0.00
11.00	-63.62	0.00	0.00
11.60	-84.61	0.00	0.00
12.00	-98.58	0.00	0.00

2.9 Reinforcement:

2.9.1 P7_1 : Span from 0.40 to 6.00 (m)

Longitudinal reinforcement:

- bottom (T)
  - 2 ϕ16.0 l = 6.60 from 0.03 to 6.49
- assembling (top) (T)
  - 2 ϕ8.0 l = 3.58 from 1.11 to 4.69
- support (T)
  - 2 ϕ12.0 l = 1.48 from 0.03 to 1.40
  - 2 ϕ12.0 l = 0.94 from 0.08 to 0.91

Transversal reinforcement:

- main (T)
  - stirrups 23 ϕ8.0 l = 1.15
    e = 5*0.15 + 6*0.30 + 1*0.25 + 1*0.25 + 6*0.30 + 4*0.15 (m)

2.9.2 P8_1 : Span from 6.40 to 12.00 (m)

Longitudinal reinforcement:

- bottom (T)
  - 2 ϕ16.0 l = 6.60 from 5.91 to 12.37
- assembling (top) (T)
  - 2 ϕ8.0 l = 3.58 from 7.11 to 10.69
- support (T)
  - 2 ϕ20.0 l = 3.00 from 4.40 to 7.40
  - 2 ϕ20.0 l = 2.15 from 10.40 to 12.37

Transversal reinforcement:

- main (T)
  - stirrups 23 ϕ8.0 l = 1.15
    e = 5*0.15 + 6*0.30 + 1*0.25 + 1*0.25 + 6*0.30 + 4*0.15 (m)

Level:

- Name : Standard Level
- Reference level : ---
- Fire rating : 0 (h)
- Environment class : mild

Column: Column60 Number: 1

2.1 Material properties:

- Concrete : C35 f_{cu} = 35.00 (N/mm2)
  Unit weight : 24.00 (kN/m3)
- Longitudinal reinforcement : T f_y = 460.00 (N/mm2)
- Transversal reinforcement : R f_y = 250.00 (N/mm2)

2.2 Geometry:

2.2.1 Rectangular 250.00 x 250.00 (mm)
2.2.2 Height: L = 3.60 (m)
2.2.3 Slab thickness = 0.00 (m)
2.2.4 Beam height = 0.45 (m)
2.2.5 Cover = 30.00 (mm)

2.3 Calculation options:

- Calculations according to : BS 8110
- Precast column : no
- Pre-design : no
- Slenderness taken into account : yes
- Ties :to slab
- Non-sway structure

2.4 Loads:

Case	Nature	Group	g_f	N (kN)	Myu (kN*m)	Myl (kN*m)	Myi (kN*m)	Mzu (kN*m)	Mzl (kN*m)	Mzi (kN*m)
COMB1	design	60	1.00	40.75	35.15	-39.35	-15.73	0.00	0.00	0.00

g_f - load factor

2.5 Calculation results:

2.5.1 Slenderness analysis

Direction Y: Non-sway structure
Direction Z: Non-sway structure

	l_o (m)	l_e (m)	b
Direction Y:	3.60	2.70	0.75
Direction Z:	3.60	2.70	0.75

ley/h =10.80 Short column (slenderness not taken into account).
lez/b =10.80 Short column (slenderness not taken into account).

2.5.2 ULS Analysis

Design combination: COMB1 (M2)

N = 40.75 (kN)		My = -39.35 (kN*m)		Mz = 0.00 (kN*m)
• E ccentricity:		e_y (mm)	e_z (mm)	
static	e_0:	0.00	-965.97	
total	e_{tot}:	0.00	965.97	
Reinforcement - required area:		A = 896.80 (mm2)		
Ratio:		m = 1.45 %		

2.6 Reinforcement:

Main bars (T):

• 8 f12.0 l = 3.57 (m)

Transversal reinforcement (R):

stirrups: 23 f6.0 l = 0.83 (m)

pins

APPENDIX F
STAAD PRO AND ROBOT MILLENNIUM LIVE LOAD INCREMENT DETAILING DIAGRAM
(SELECTED BEAMS ONLY)

Final Year Project

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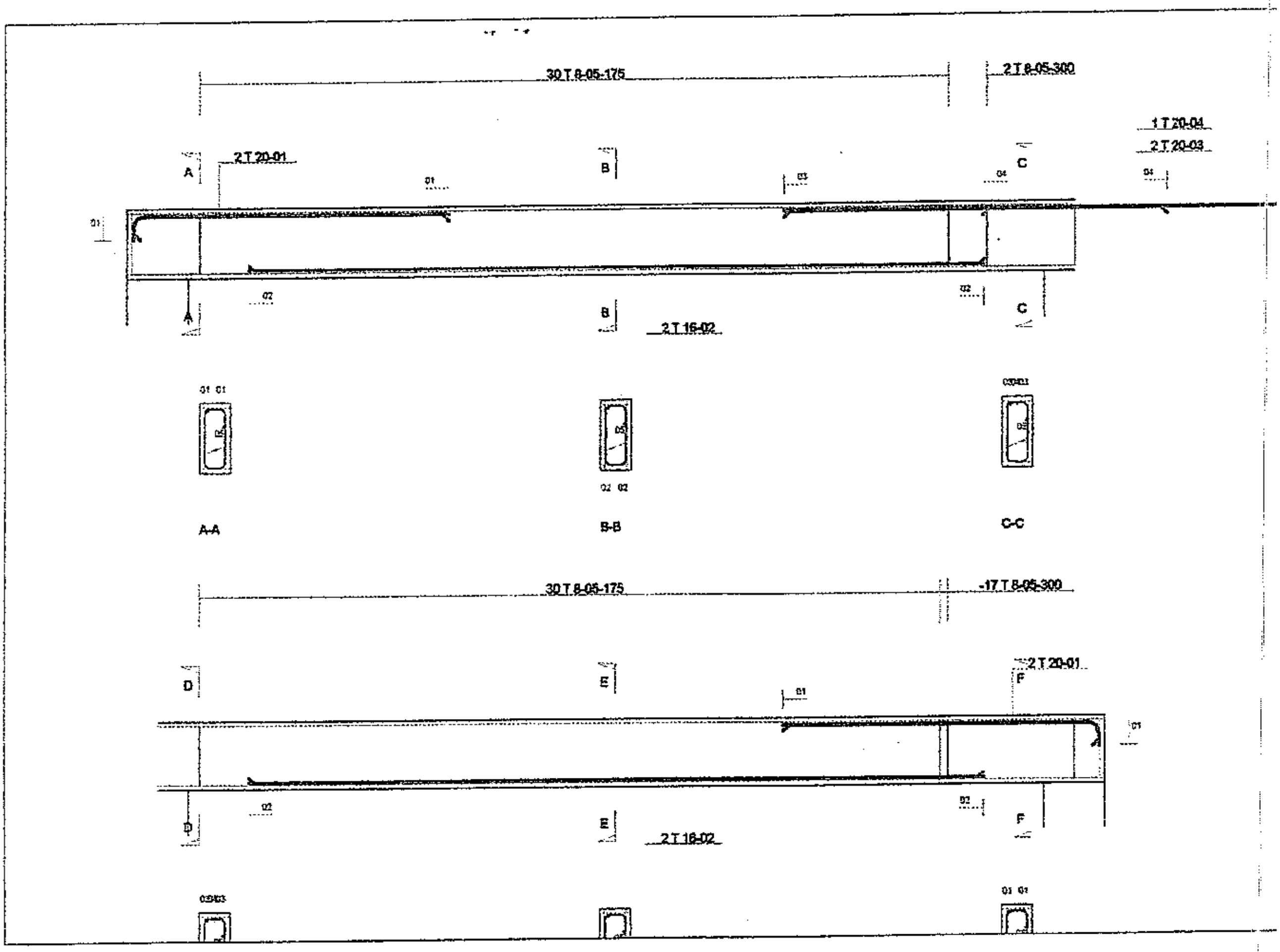
Job No	Sheet No 1
Rev	
Part	
Ref	
By	Date 17-Sep-04
Chd	
File	Date/Time

Title LIVE LOAD INCREMENT ANALYSIS = 4.5 KN/m

nt



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Software licensed to Snow Panther [LZ0]	Part		
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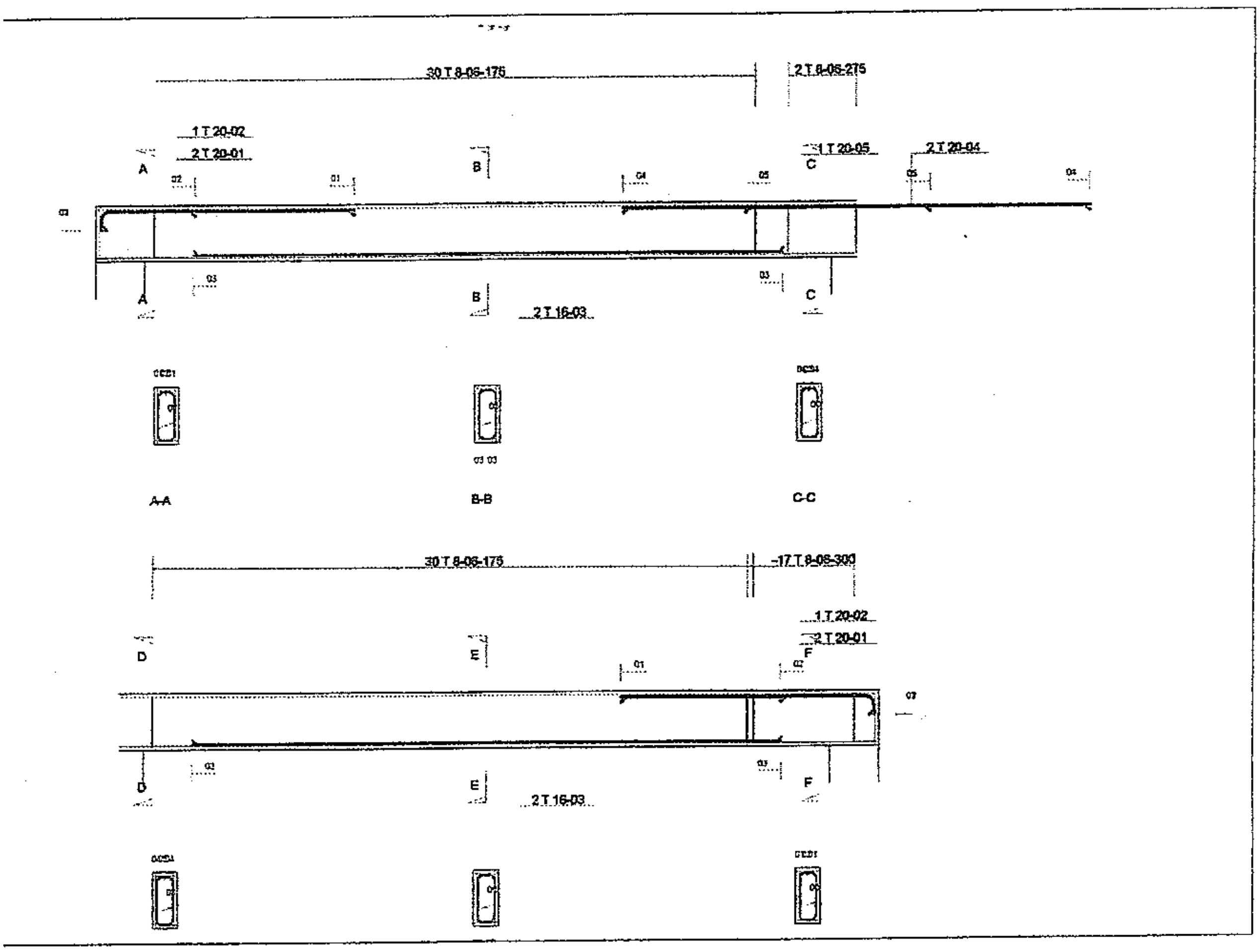

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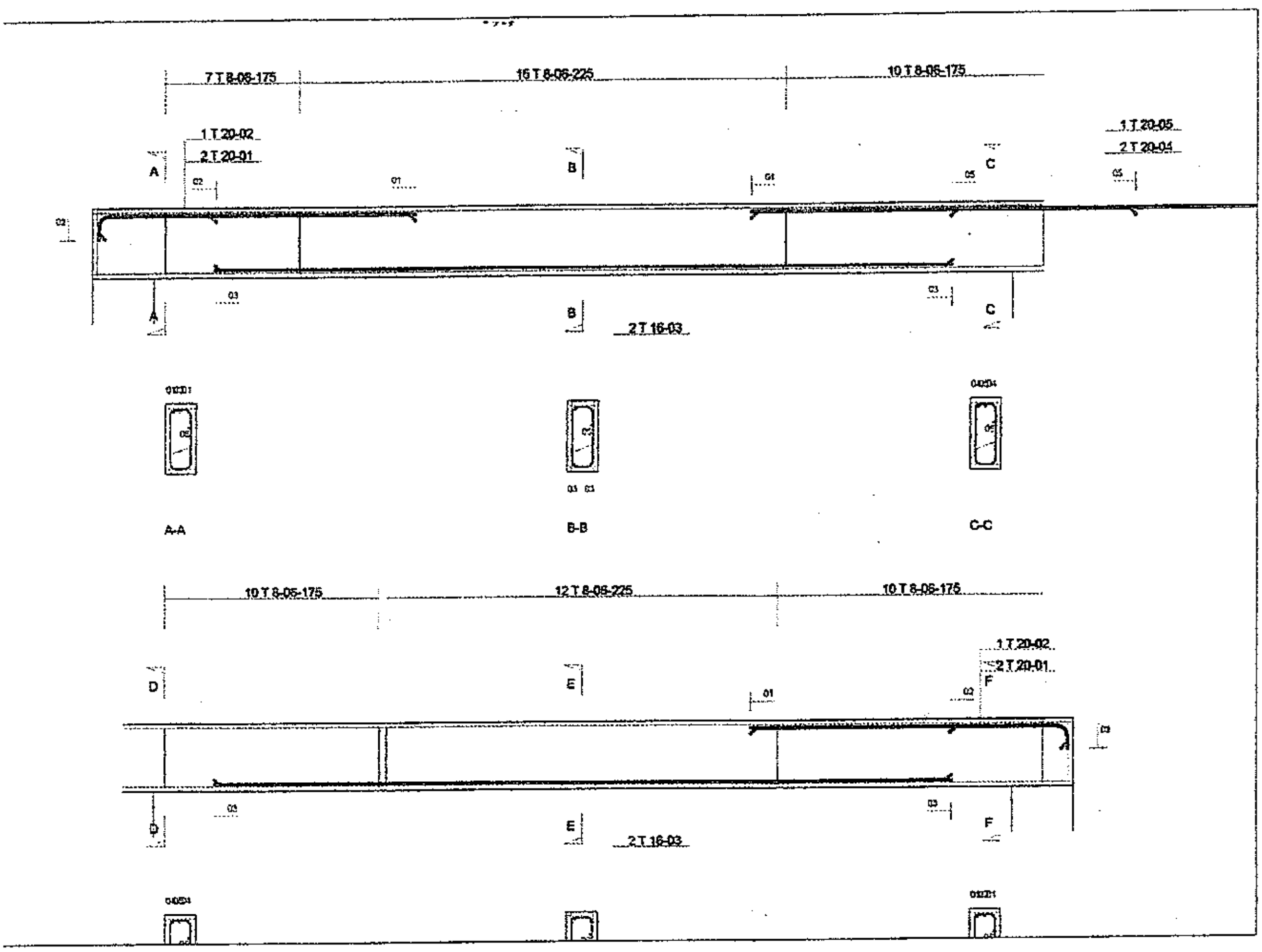

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Software licensed toSnow Panther [LZ0]	Part		
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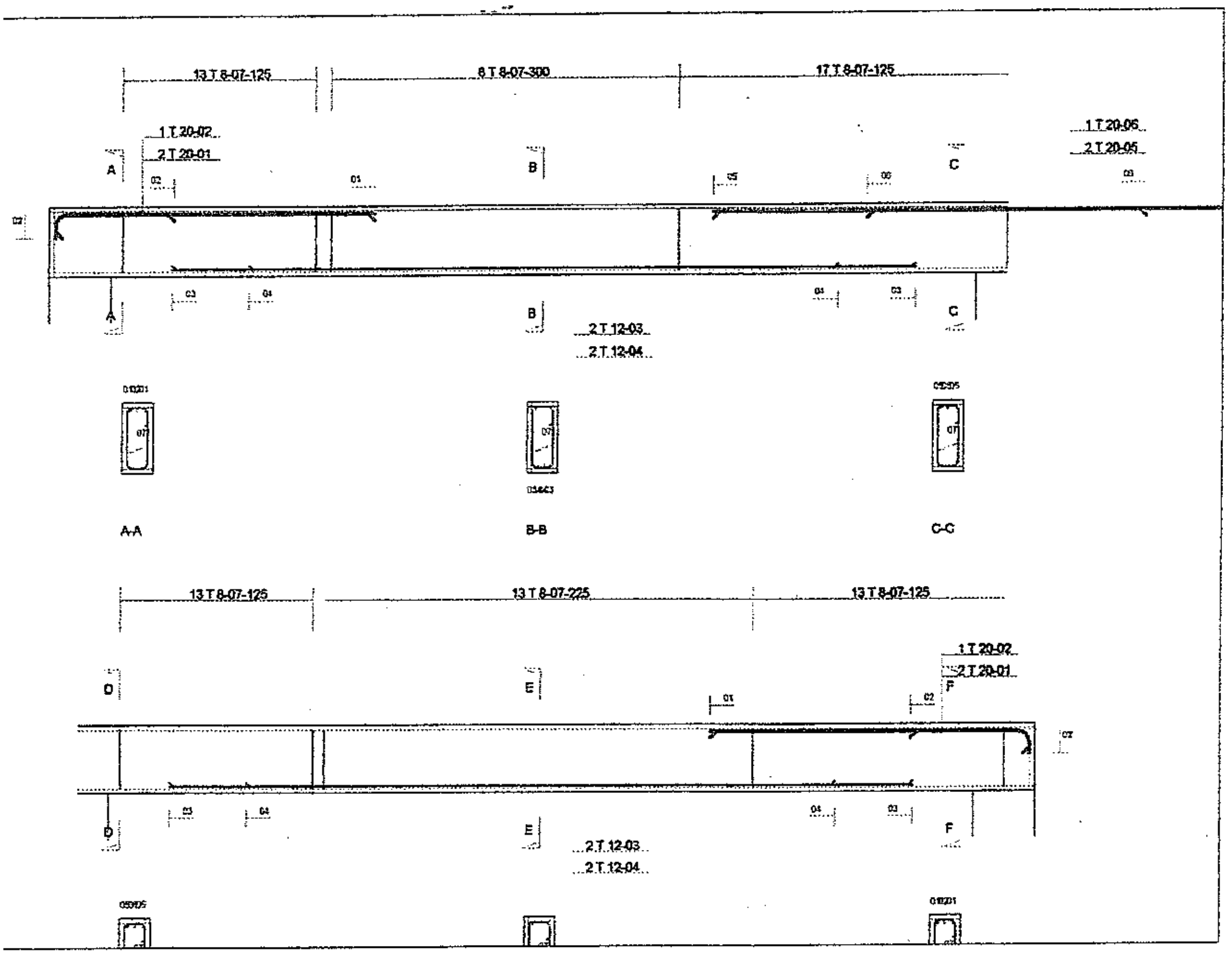

Final Year Project

Software licensed toSnow Panther [LZ0]

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Part		
Ref		
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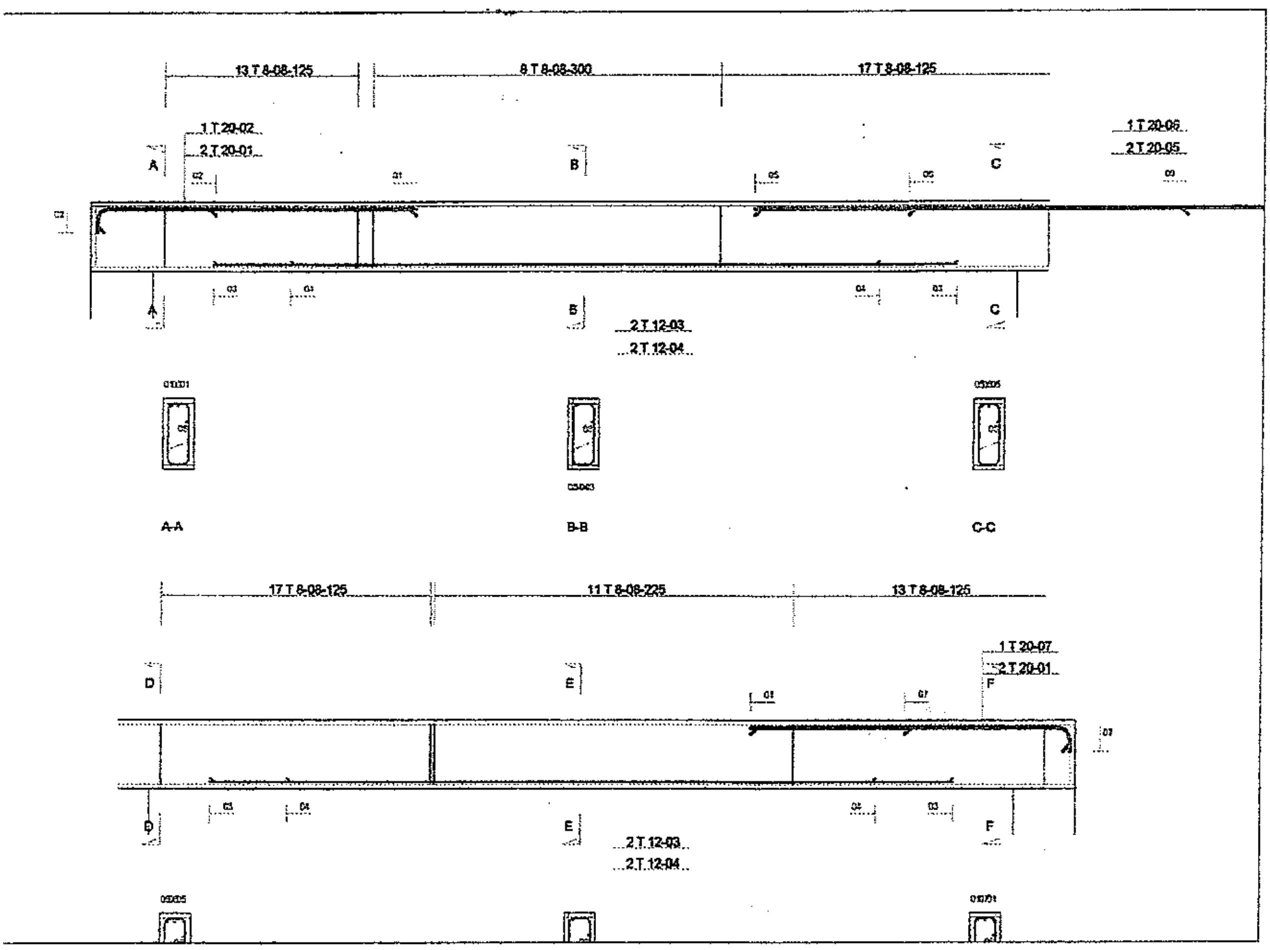

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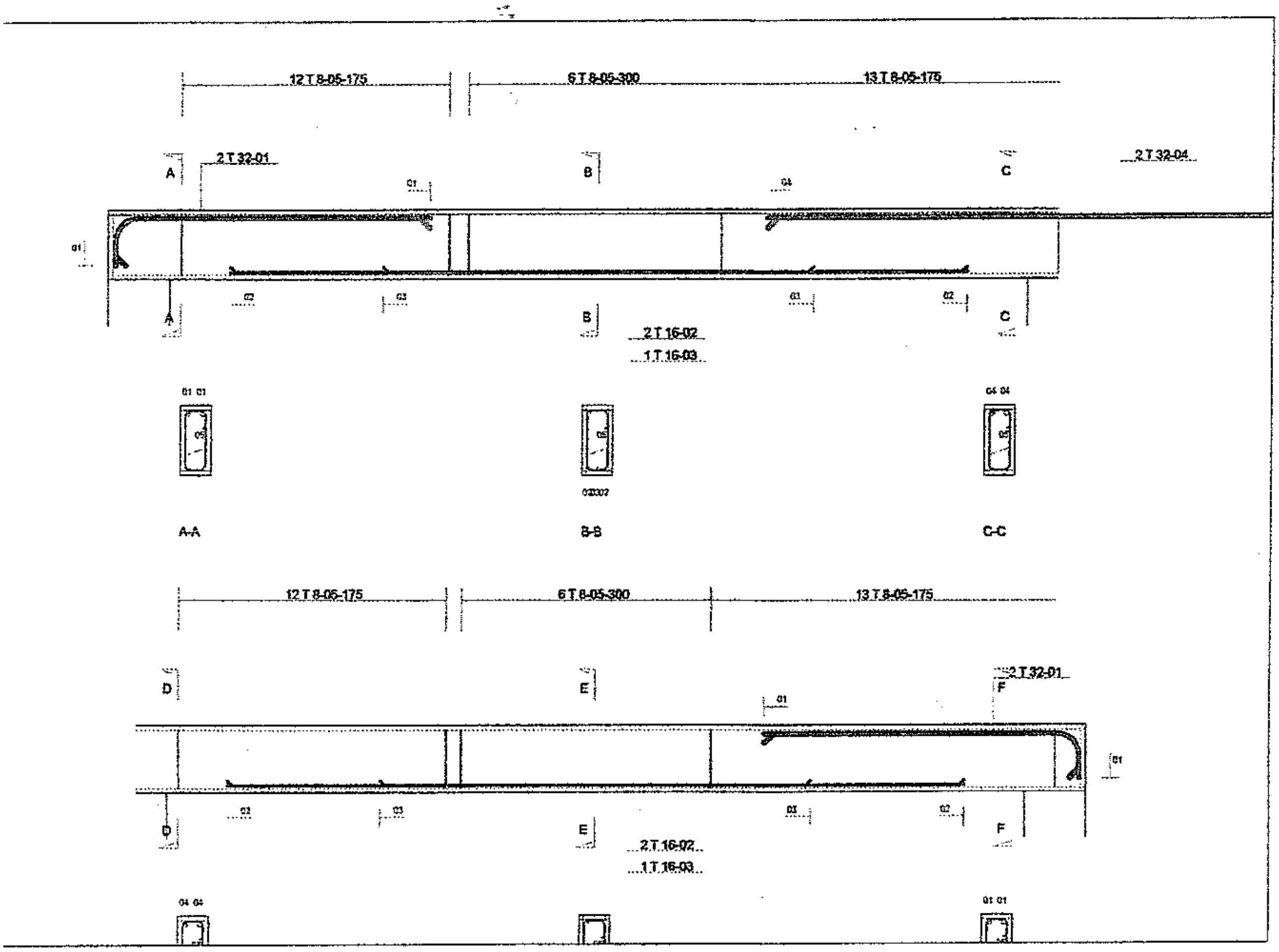

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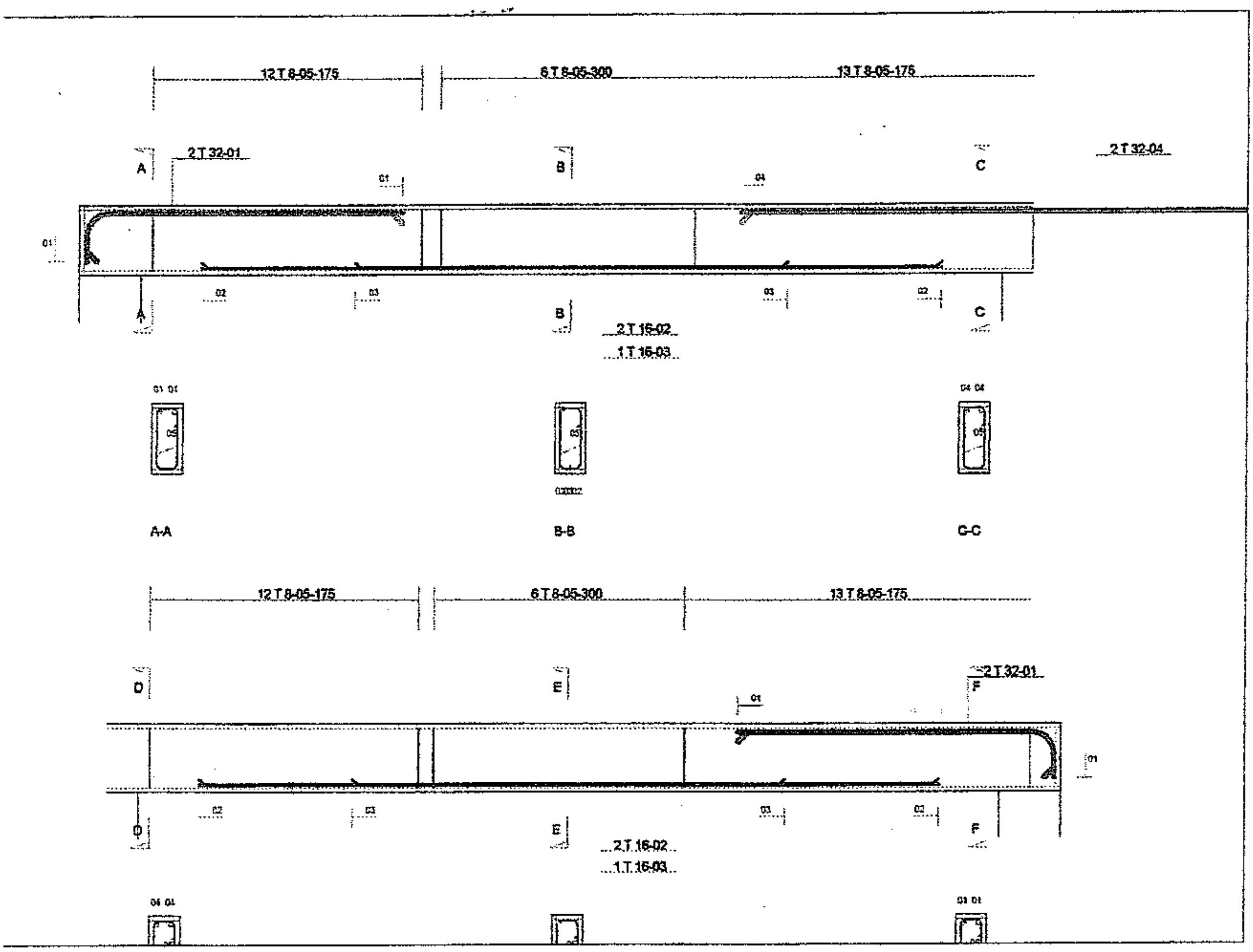
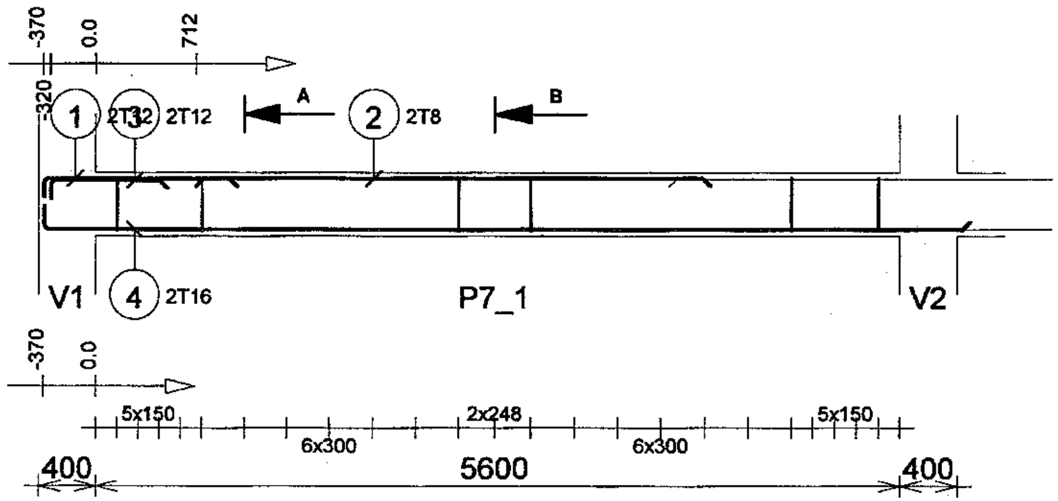
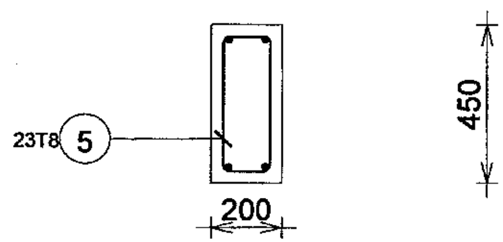




1	2T12	l=1477	37	132 / 1370
2	2T8	l=3576	20	3576
3	2T12	l=935	37	132 / 828
4	2T16	l=6601	37	176 / 6458
5	23T8	l=1153	60	390 / 140

A-A



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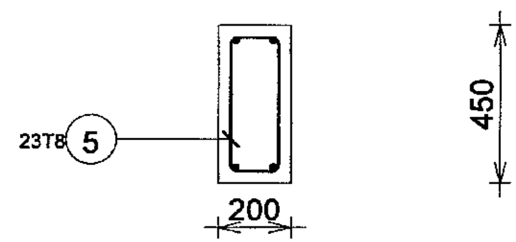
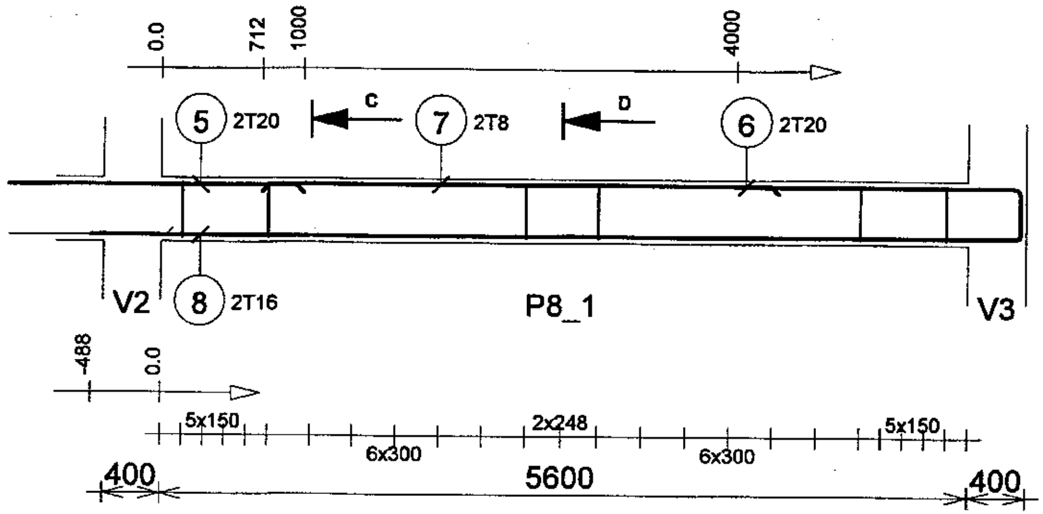


Standard Level **STRUCT 2D**	**Beam7...8** (LIVE LOAD-4-5 KN/M) **Section 200x450**	Steel T = 38.4kg Concrete = 0.558m3 Formwork = 6.79m2 Bottom cover 30cm Top cover 30cm Side cover 30cm View scale 1:50 Section scale 1:20

5	2T20	l=3000	20	3000
6	2T20	l=2149	37	220, 1970
7	2T8	l=3576	20	3576
8	2T16	l=6601	37	176, 6458
9	23T8	l=1153	60	390, 140



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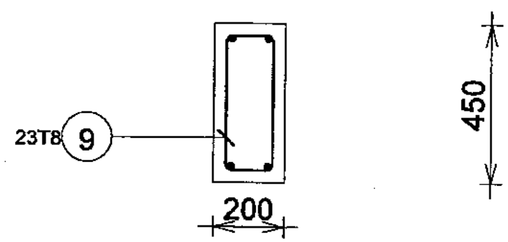

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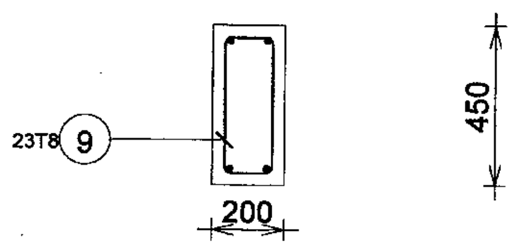


Standard Level

STRUCT 2D

Beam7...8 (LIVE LOAD= 4.5KN/m)

Section 200x450

Steel T = 59.5kg

Concrete = 0.558m3

Formwork = 6.79m2

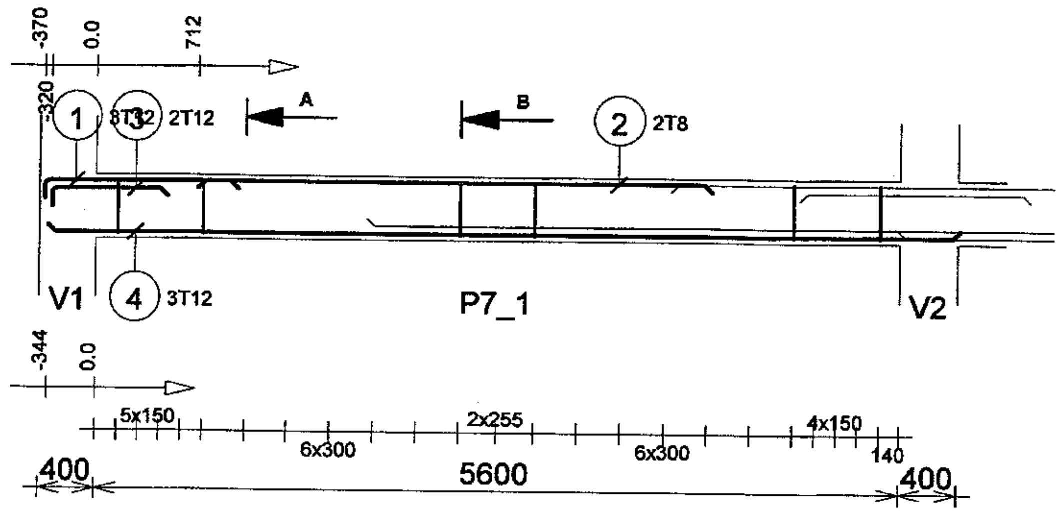
Bottom cover 30cm

Top cover 30cm

Side cover 30cm

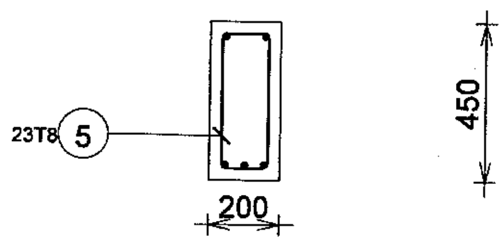
View scale 1:50

Section scale 1:20



1	3T12	l=1477	37	132 1370
2	2T8	l=3576	20	3576
3	2T12	l=927	37	132 820
4	3T12	l=6360	20	6360
5	23T8	l=1153	60	390 140

A-A



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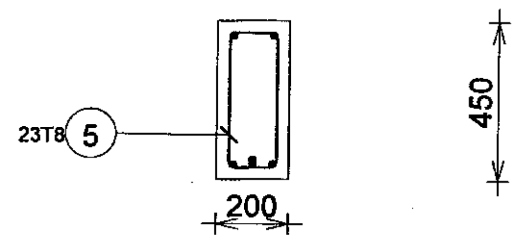
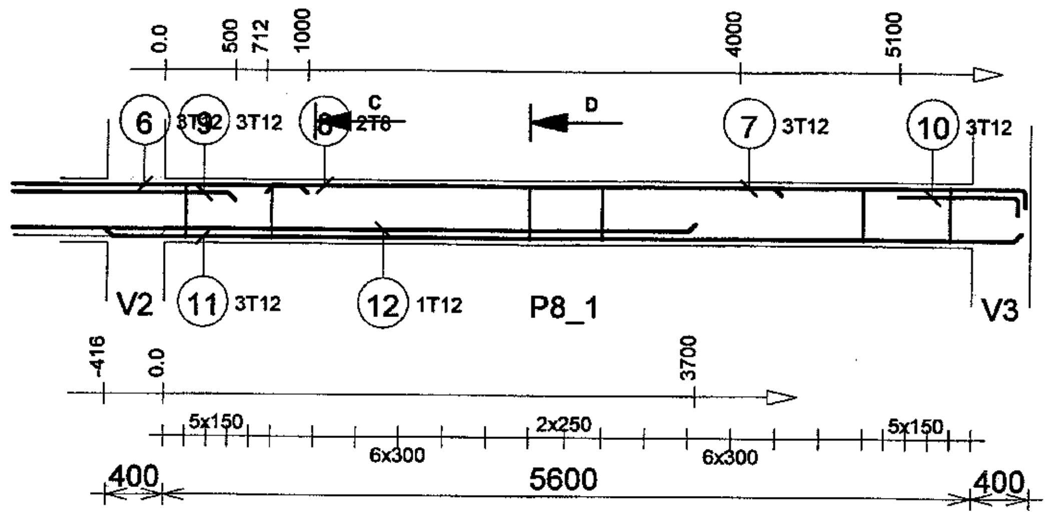


Standard Level
STRUCT 2D

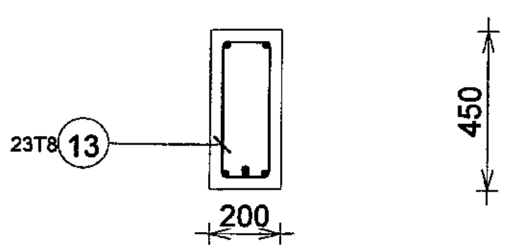
Beam7...8 (LIVE LOAD = 6.0 KN/m)
Section 200x450

Steel T = 35.8kg	Concrete = 0.558m3 Formwork = 6.79m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	



(6)	3T12	l=3000	20	3000
(7)	3T12	l=2077	37	132 1970
(8)	2T8	l=3576	20	3576
(9)	3T12	l=1600	20	1600
(10)	3T12	l=927	37	132 820
(11)	3T12	l=6360	20	6360
(12)	1T12	l=7800	20	7800
(13)	23T8	l=1153	60	390 140

C-C



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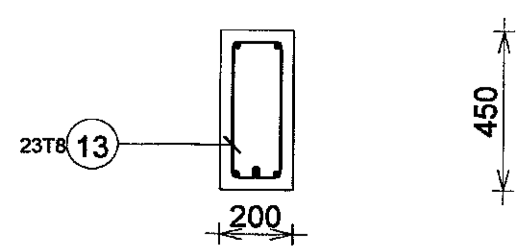
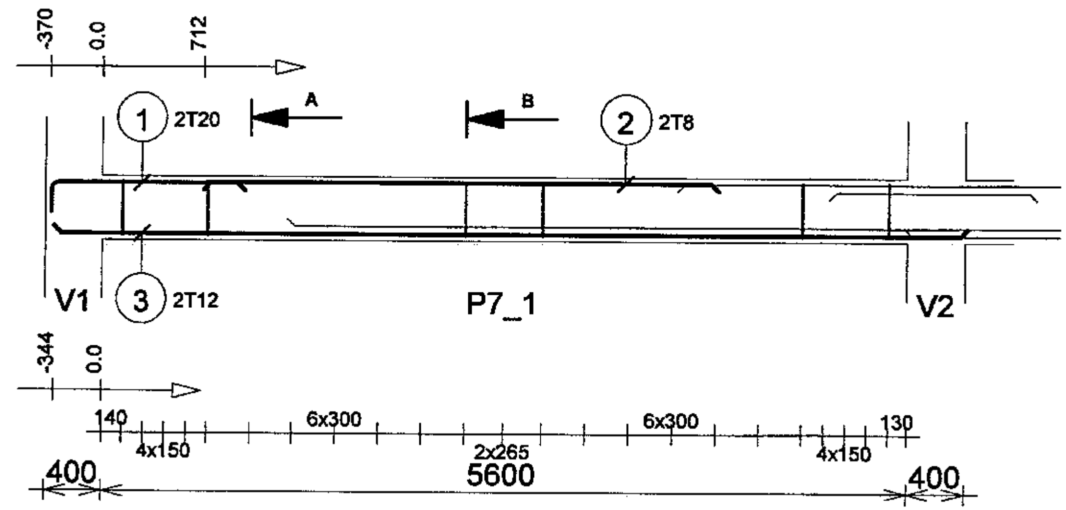


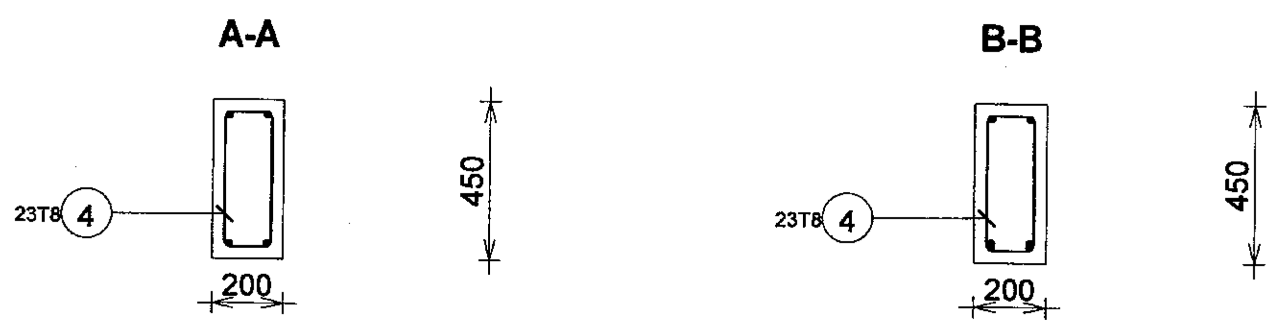
Standard Level
STRUCT 2D

Beam7...8 (LIVE LOAD = 6.0 KN/m)
Section 200x450

Steel T = 57.4kg
Concrete = 0.558m3
Formwork = 6.79m2
Bottom cover 30cm
Top cover 30cm
Side cover 30cm
View scale 1:50
Section scale 1:20

1	2T20	l=1549	37	220 1370
2	2T8	l=3576	20	3576
3	2T12	l=6360	20	6360
4	23T8	l=1153	60	390 140





Standard Level

STRUCT 2D

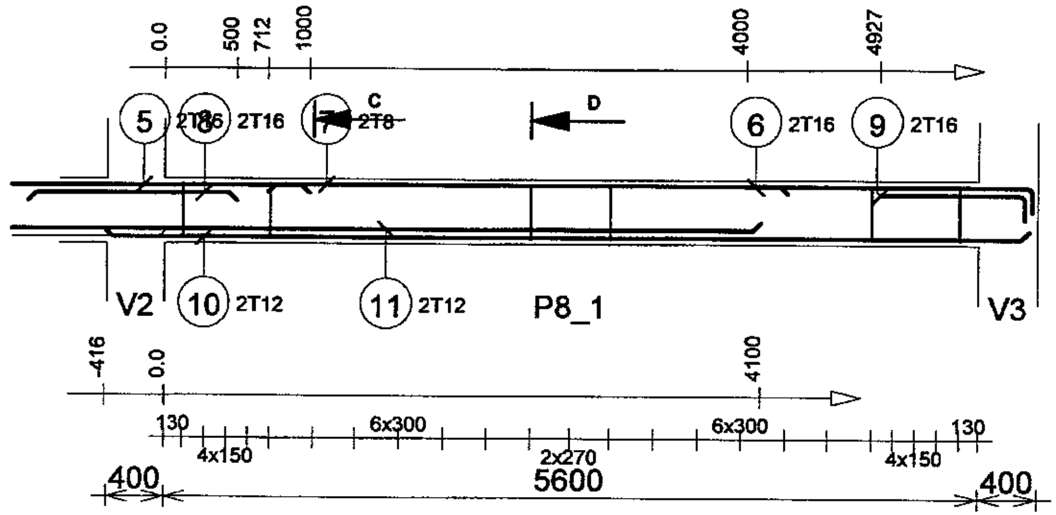
Beam7...8 (LIVE LOAD = 7.5 kN/m)

Section 200x450

Steel T = 32.2kg
Concrete = 0.558m3
Formwork = 6.79m2

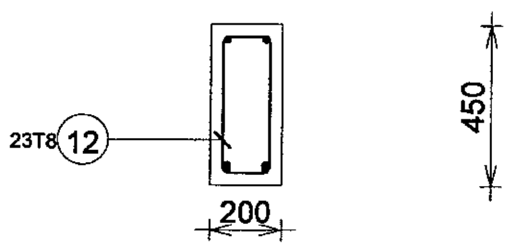
Bottom cover 30cm
Top cover 30cm
Side cover 30cm

View scale 1:50
Section scale 1:20



5	2T16	l=3000	20	3000
6	2T16	l=2113	37	176 1970
7	2T8	l=3576	20	3576
8	2T16	l=1447	20	1447
9	2T16	l=1136	37	176 993
10	2T12	l=6360	20	6360
11	2T12	l=8800	20	8800
12	23T8	l=1153	60	390 140

C-C



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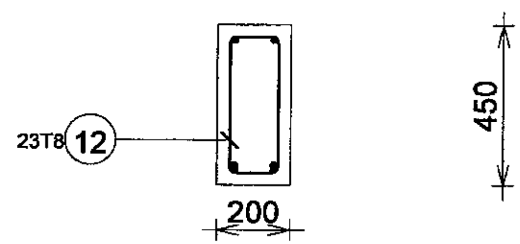


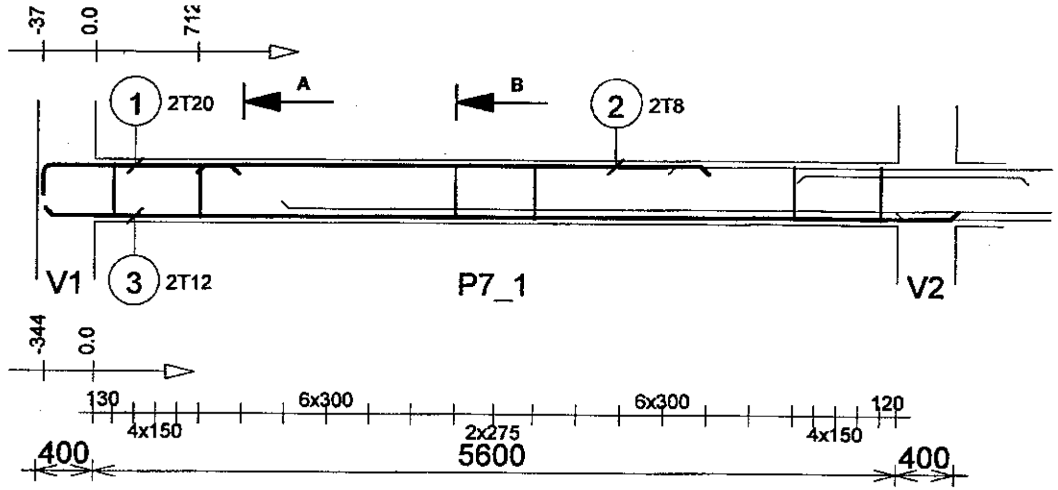
Standard Level

STRUCT 2D

Beam7...8 (LIVE LOAD = 7.5 kN/m)

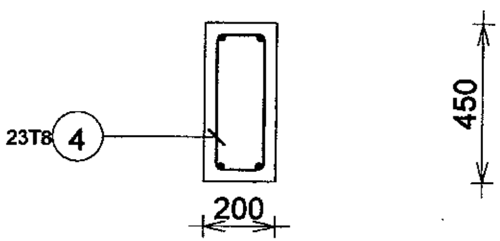
Section 200x450

Steel T = 64.5kg	Concrete = 0.558m3 Formwork = 6.79m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	



2	2T8	l=3576	20	3576
3	2T12	l=6360	20	6360
4	23T8	l=1153	60	390 / 140

A-A



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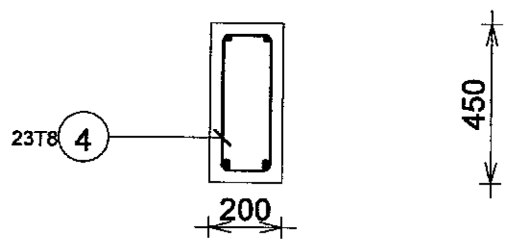
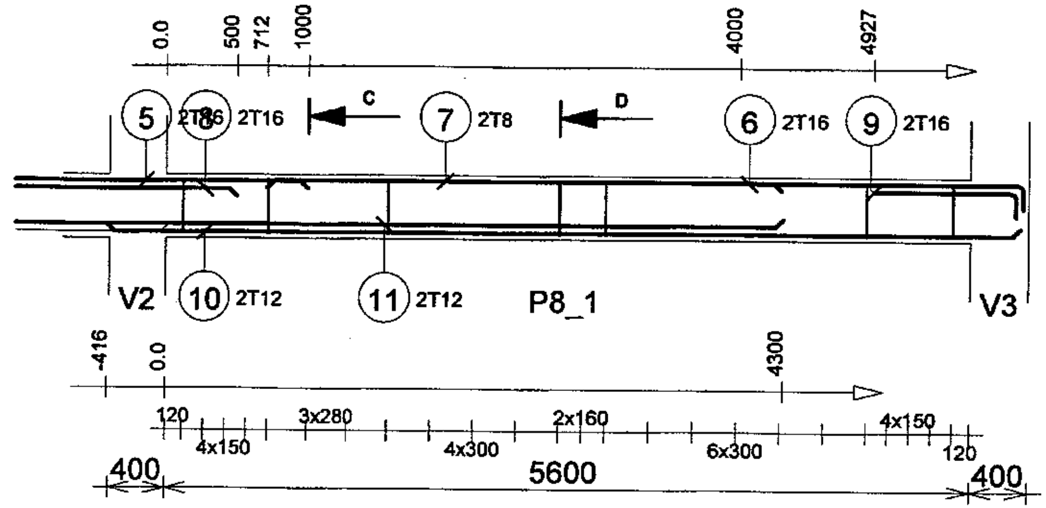


Standard Level
STRUCT 2D

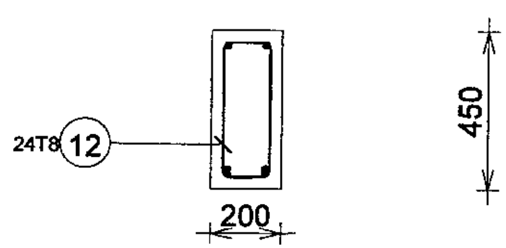
Beam7...8 (LIVE LOAD = 9.0 KN/m)
Section 200x450

Steel T = 32.2kg	Concrete = 0.558m3 Formwork = 6.79m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	

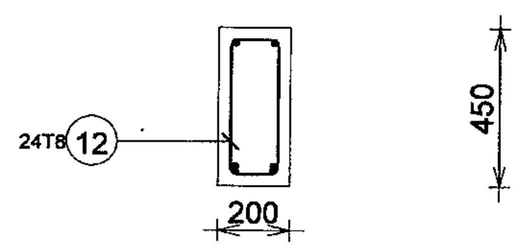


5	2T16	l=3000	20	3000
6	2T16	l=2113	37	176 1970
7	2T8	l=3576	20	3576
8	2T16	l=1600	20	1600
9	2T16	l=1136	37	176 993
10	2T12	l=6360	20	6360
11	2T12	l=9000	20	9000
12	24T8	l=1153	60	390 140

C-C



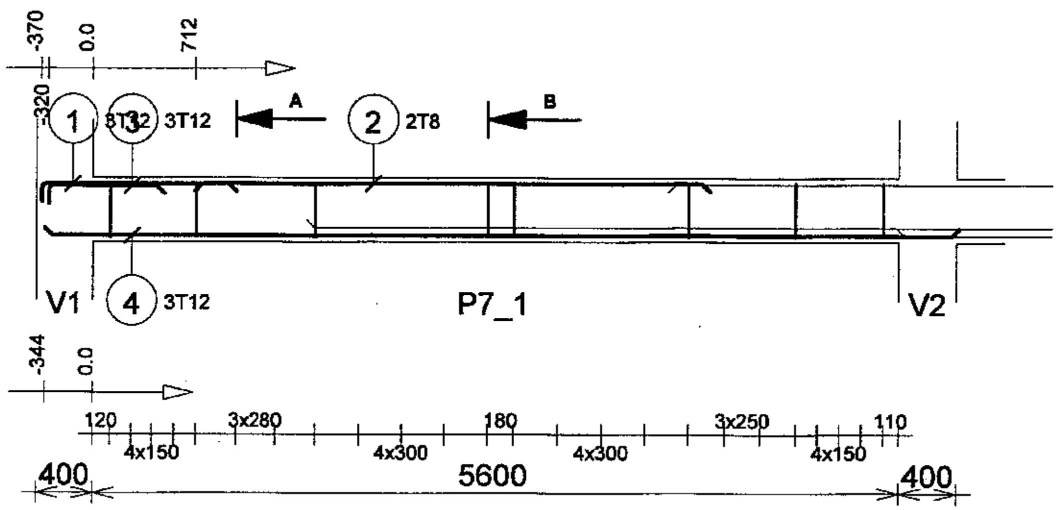
D-D



Standard Level
STRUCT 2D

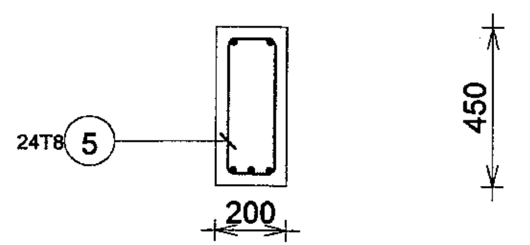
Beam7...8 (LIVE LOAD = 9.0KN/m)
Section 200x450

Steel T = 65.8kg	Concrete = 0.558m3 Formwork = 6.79m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	

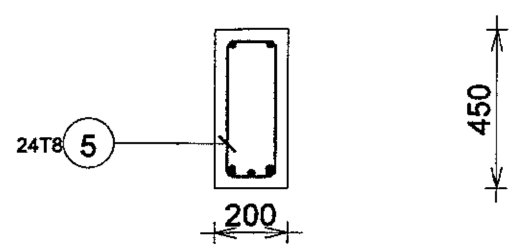


1	3T12	l=1477	37	132 1370
2	2T8	l=3576	20	3576
3	3T12	l=927	37	132 820
4	3T12	l=6360	20	6360
5	24T8	l=1153	60	390 140

A-A



B-B



Standard Level

STRUCT 2D

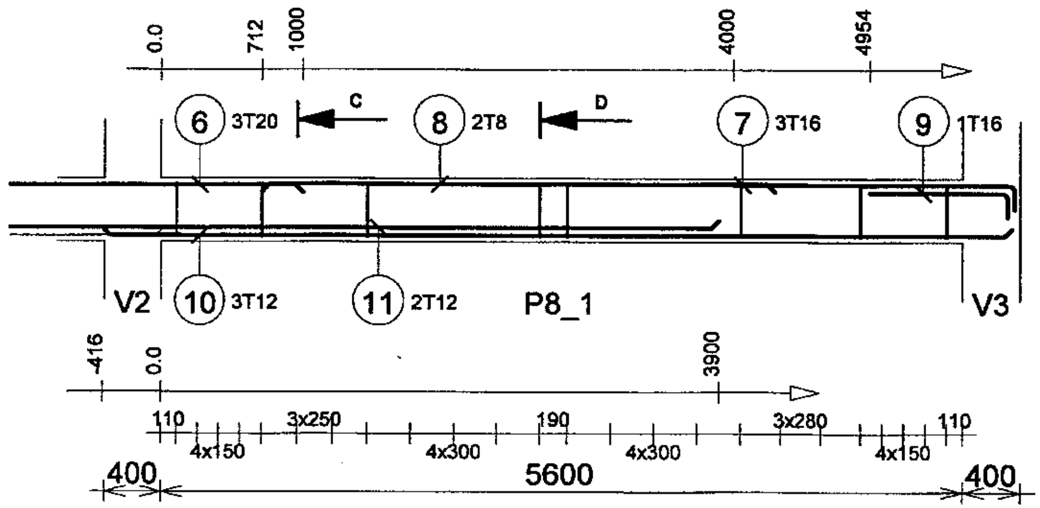
Beam7...8 (LIVE LOAD = 10.5 kN/m)

Section 200x450

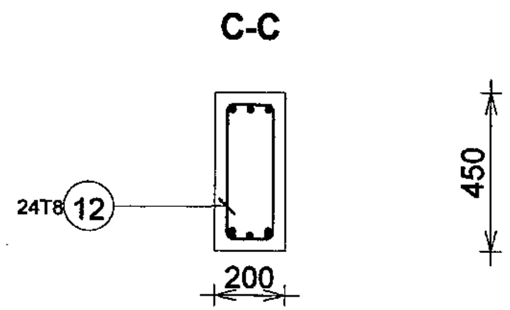
Steel T = 37.1kg
Concrete = 0.558m3
Formwork = 6.79m2

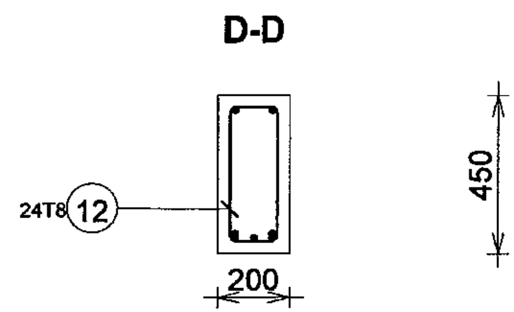
Bottom cover 30cm
Top cover 30cm
Side cover 30cm

View scale 1:50
Section scale 1:20



6	3T20	l=3000	20	3000
7	3T16	l=2113	37	176 1970
8	2T8	l=3576	20	3576
9	1T16	l=1108	37	176 966
10	3T12	l=6360	20	6360
11	2T12	l=8400	20	8400
12	24T8	l=1153	60	390 140

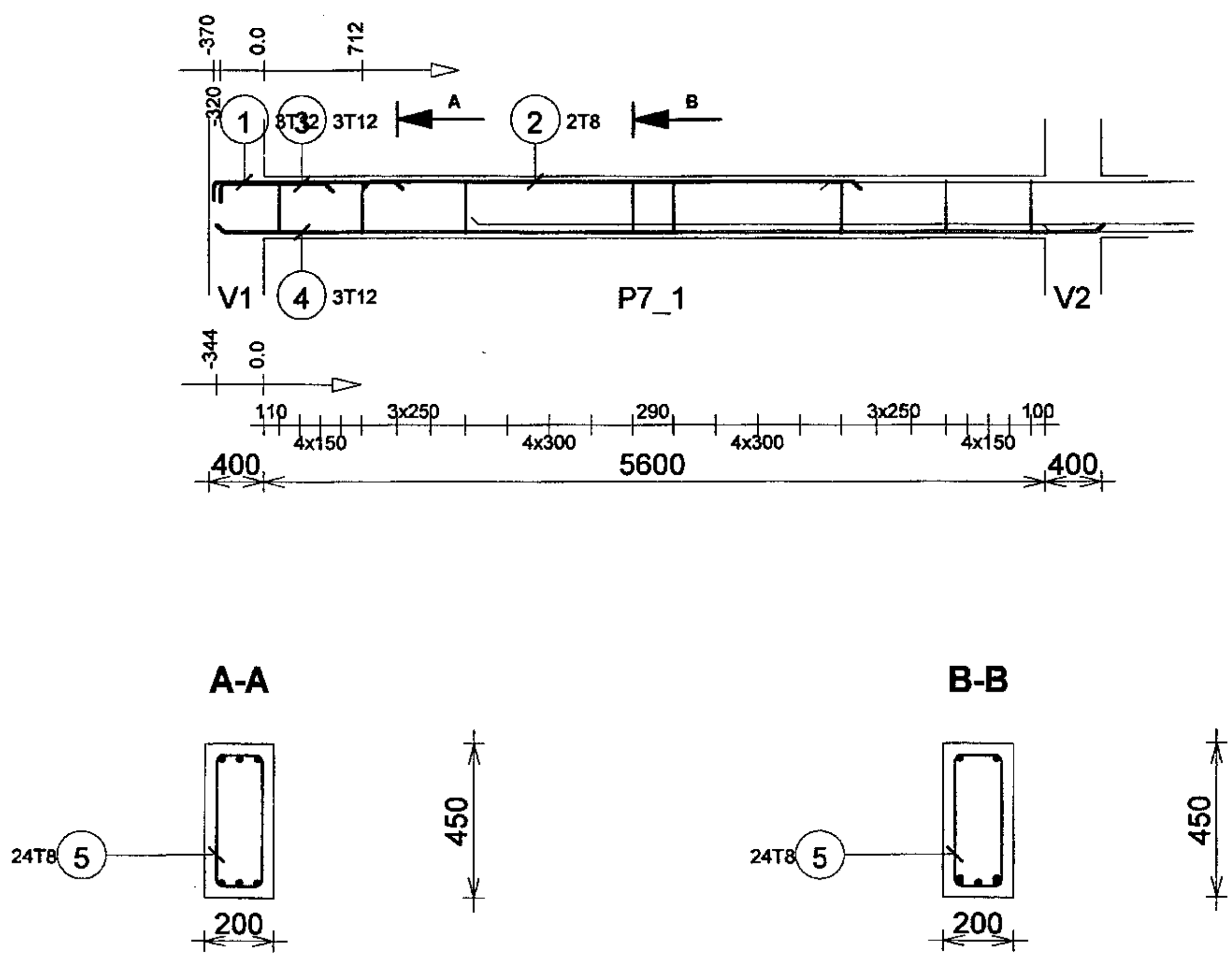




Standard Level
STRUCT 2D

Beam7...8 (LIVE LOAD =10.5 KN/m)
Section 200x450

Steel T = 79.5kg	Concrete = 0.558m3 Formwork = 6.79m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	



1	3T12	l=1477	37	132 1370
2	2T8	l=3576	20	3576
3	3T12	l=927	37	132 820
4	3T12	l=6360	20	6360
5	24T8	l=1153	60	390 140

Standard Level

STRUCT 2D

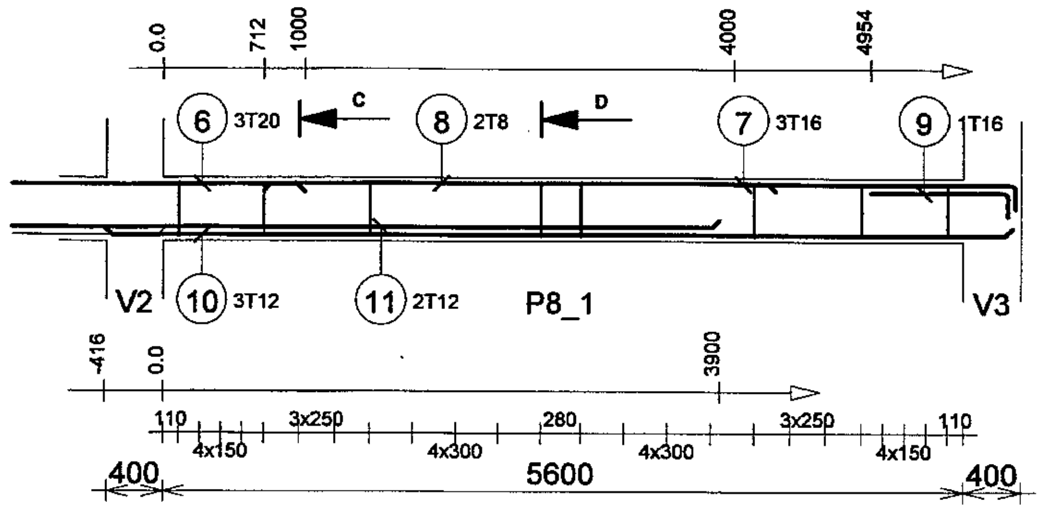
Beam7...8 (LIVE LOAD = 12.0KN/m)

Section 200x450

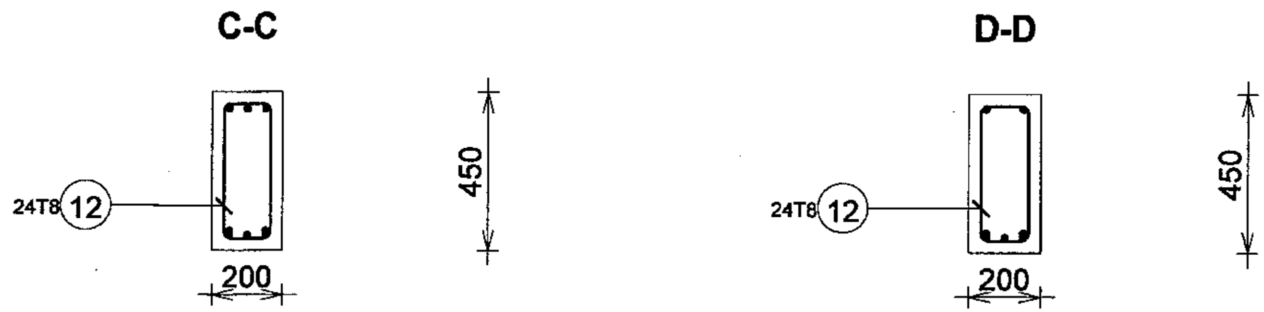
Steel T = 37.1kg
Concrete = 0.558m3
Formwork = 6.79m2

Bottom cover 30cm
Top cover 30cm
Side cover 30cm

View scale 1:50
Section scale 1:20



6	3T20	l=3000	20	3000
7	3T16	l=2113	37	176 1970
8	2T8	l=3576	20	3576
9	1T16	l=1108	37	176 966
10	3T12	l=6360	20	6360
11	2T12	l=8400	20	8400
12	24T8	l=1153	60	390 140



Standard Level

STRUCT 2D

Beam7...8 (LIVE LOAD = (2.0 kN/m)

Section 200x450

Steel T = 79.5kg

Concrete = 0.558m3

Formwork = 6.79m2

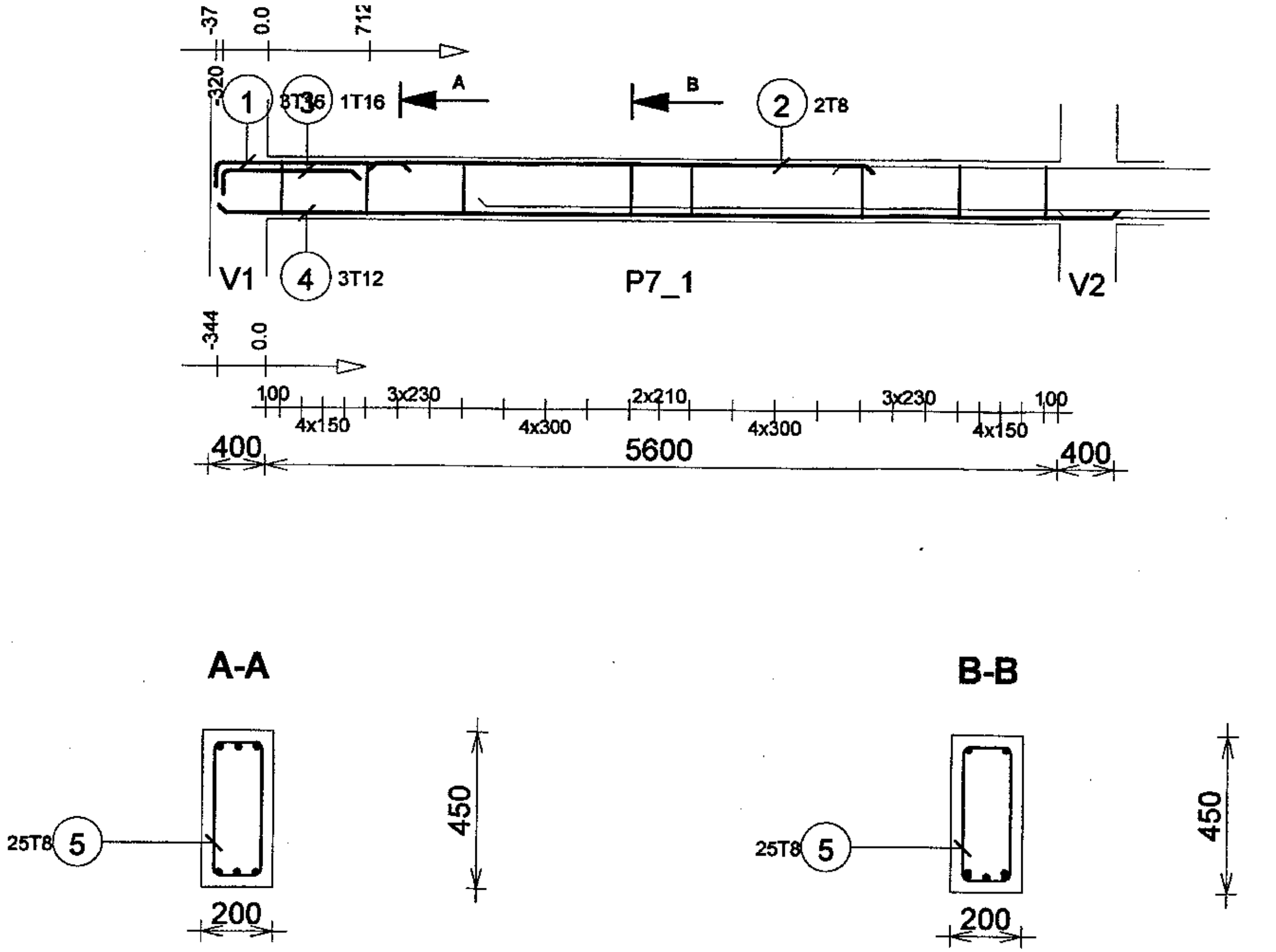
Bottom cover 30cm

Top cover 30cm

Side cover 30cm

View scale 1:50

Section scale 1:20

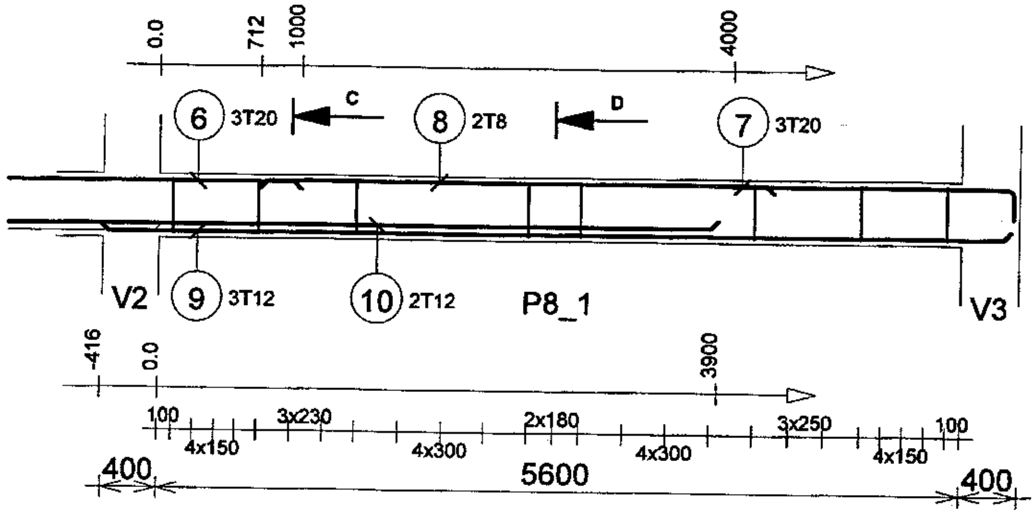


1	[illegible]	[illegible]	[illegible]	[illegible]
2	2T8	l=3576	20	3576
3	1T16	l=1108	37	176 966
4	3T12	l=6360	20	6360
5	25T8	l=1153	60	390 140

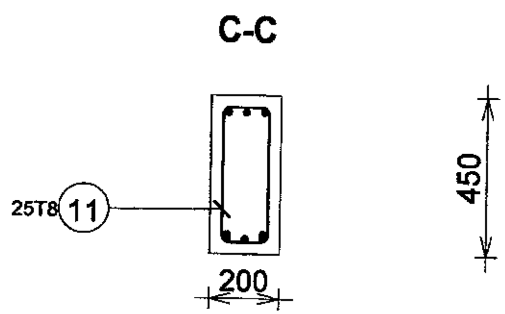
Standard Level
STRUCT 2D

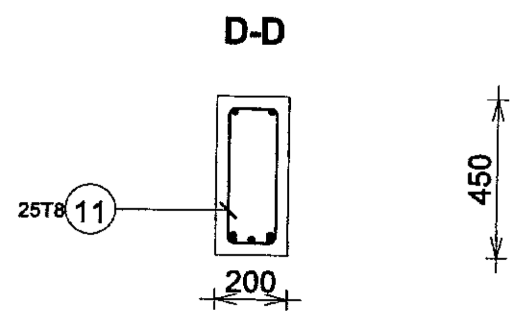
Beam7...8 (LIVE LOAD = 13.5 KN/m)
Section 200x450

Steel T = 40kg	Concrete = 0.558m3 Formwork = 6.79m2
Bottom cover 30cm Side cover 30cm	Top cover 30cm
View scale 1:50 Section scale 1:20	



6	3T20	l=3000	20	3000
7	3T20	l=2149	37	220 1970
8	2T8	l=3576	20	3576
9	3T12	l=6360	20	6360
10	2T12	l=8400	20	8400
11	25T8	l=1153	60	390 140





Standard Level STRUCT 2D	**Beam7...8** (LIVE LOAD = 13.5kN/m) **Section 200x450**	Steel T = 84.1kg	Concrete = 0.558m3 Formwork = 6.79m2
		Bottom cover 30cm Side cover 30cm	Top cover 30cm
		View scale 1:50 Section scale 1:20	

APPENDIX G
GANTT CHART FOR FINAL YEAR PROJECT

Suggested Milestone for the First Semester of 2 Semester Final Year Project

No.	Detail/ Week	1	2	3	4	5	6	7	8	9	10	11	12	13	14	16	18
1	Selection of Project Topic	■	●														
	-Propose Topic																
	-Topic assigned to students																
2	Preliminary Research Work		■	■	●												
	-Introduction																
	-Objective																
	-List of references/literature																
	-Project planning																
3	Submission of Preliminary Report				■	●											
4	Project Work				■	■	■	■									
	-Reference/Literature																
	-Practical/Laboratory Work																
5	Submission of Progress Report								■	●							
6	Project work continue								■	■	■	■	■				
	-Practical/Laboratory Work																
7	Submission of Interim Report Final Draft													■	●		
8	Submission of Interim Report															■	
9	Oral Presentation																■

● Actual milestone
■ Suggested Process

Suggested Milestone for the Second Semester of 2 Semester Final Year Project

No.	Detail/ Week	1	2	3	4	5	6	7	8	9	10	11	12	13	19	20
1	Project Work Continue	▒	▒	▒												
	-Perform Actual Analysis and Design															
2	Submission of Progress Report 1				●											
3	Project Work Continue				▒	▒	▒	▒								
	-Reference/Literature															
	-Practical/Laboratory Work															
	-Comparison of Results															
4	Submission of Progress Report 2								●							
6	Project work continue								▒	▒	▒	▒	▒			
	-Practical/Laboratory Work															
	-Propose or Improved Existing Design															
7	Submission of Dissertation Final Draft													●		
8	Oral Presentation														●	
9	Submission of Project Dissertation															●

● Actual milestone

▒ Suggested Process