NUMERICAL STUDY ON STRENGTH OF REINFORCED BEAM-COLUMN JOINTS

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CIVIL ENGINEERING
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Numerical Study on Strength of Reinforced Beam-Column Joints

by

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

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CERTIFICATION OF APPROVAL

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Approved by, ON BEHALF OF

NABILAH ABU BAKAR
FYP COORDINATOR

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UNIVERSITI TEKNOLOGI PETRONAS
TRONOH, PERAK
January 2009

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.

LINZIANA BINTI MUSTAFFA

ABSTRACT

The location of Malaysia is not within the "Ring of Fire" zone of frequent earthquake. Thus, in Malaysia, the seismic hazard is low with high consequences. The possibilities of the building to experience the vibration when subjected to a lateral or horizontal force ground motion due to an earthquake is low. After the incident of undersea megathrust earthquake of 2004 Indian Ocean Earthquake, called Tsunami swept across Malaysia, this disaster has made the engineers, architects and local authorities pay more attention on the effective seismic design of concrete structures. This project is about the numerical study on strength of reinforced beam-column joints due to severe earthquake, which leads to building collapse. Research regarding the shear strength and ductility of the joints has been found to be the important design factor to achieve satisfactory structures. The simulation is conducted by using STAAD.Pro to observe the shear, bending moments and torsion of the frame structure of 4-storey school building and make sure that the value doesn't exceed the static capacity which the beam can sustain. From the results obtain, finite element analysis is conducted for the most critical section of beam-column joint and determine the stress, cracking pattern and crushing pattern on the beam-column joint. Based on the simulation that has been done, this research concludes that the school building is still able to withstand seismic loading and safe to be used.

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CHAPTER 1
INTRODUCTION

1.1 Background

On 26 December 2004, an earthquake disaster had swept across Malaysia. Pulau Langkawi and Pulau Pinang were said to be the most affected area. The killer wave, called 2004 Indian Ocean Earthquake, known as Tsunami is indicate to be the largest earthquake on earth since the 9.20-magnitude Good Friday Earthquake which struck Alaska, USA, on March 27, 1964 and the fourth largest since 1900. The 9.0 of moment magnitude earthquake struck the Indian Ocean off the western coast of northern Sumatra, Indonesia on December 26, 2004 causing thousands of deaths. The catastrophe hits coastal regions all over the Indian Ocean including Aceh, Sri Lanka, Indian state of Tamil Nadu, Phuket Island, Thailand, Somalia, Africa and even in Malaysia.

The impact of this sudden earthquake has made the building designers in Malaysia more cautious about the building safety. Most of the building structures in Malaysia are designed with less consideration of vibration due to force ground motion since Malaysia is claimed to be out of frequent earthquake zone. The design load for most of the building in Malaysia is only the lateral load due to wind and neglect the earthquake load. Although Malaysia fortunately escaped from the damages that struck beaches thousands of miles further away, the amount of deaths was still the tragic incident for Malaysians and the future tremor hits which are greater from this might be possible.

Many researches and studies from the past have been conducted in relation to the seismic response of reinforced beam-column joints. Experiments and modelling were done in upgrading the ductility and shear strength of beam-column joints when subjected to seismic loading. The study on seismic performance and structural behaviour of beam-column joints is also one of the researches in terms of cyclic loadings. As a result, the researchers come out with different ways of improving the durability of reinforced beam-column joints subjected to cyclic loading.

In this project, the numerical study on the strength of reinforce beam-column joints under seismic loading is conducted by taking the elasticity, ductility and shear strength of beam-column joints into consideration. The simulation by STAAD.Pro software has been conducted to analyse the behaviour and properties of a frame structure subjected to earthquake loading using spectrum analysis. STAAD.Pro is the most popular structural engineering software product for 3-dimensional model generation, analysis and multi-material design. The types of concrete being used are also the most important thing to study in order to make the structure more effective and efficient under the seismic response.

1.2 Problem Statement

In Malaysia, the reinforced concrete building designs are based on British Standard Institution, BS 8110. The structural use of concrete in buildings and structures are recommended in the BS 8110. The existing reinforced concrete structures in Malaysia are mostly designed with consideration of wind and gravity load. However, there are little or no buildings structures that have been design with the provision of seismic load. They are not designed on the basis of earthquake design code and make no direct use of ground motion.

The tremor felt by Malaysians on 2004 has become the important consideration on designing the buildings structures for the safety of people. This is due to rapid construction of high rise structures in Peninsular Malaysia which may create high seismic risk in terms of structural damages and deaths due to high population and commercial activities taking place in the structure. Thus, the building must be able to withstand the vibration due to earthquake when subjected to a lateral or horizontal force ground motion. Thus, structural failures and deaths can be reduced or prevented.

1.3 Objectives and Scope of Study

1.3.1 The purpose of the project

The objectives of the project are:

1. To analyse behaviour and properties of a frame structure of a 4-storey school building subjected to earthquake loading using spectrum analysis with STAAD.Pro software.
2. To conduct the finite element analysis of the most critical section of beam-column joint when subjected to seismic loading and determine the stress, cracking pattern and crushing pattern on the beam-column joint.

1.3.2 The feasibility of the project

The scope of this study would be on dynamic loading by analysis and modeling. For this project, it would focus only on the analysis. The initial works are to calculate the loading subjected to the beam. Furthermore, the capacity of the beam and column under shear, moment and tension will be calculated for the checking purpose. From the structural drawing, a model can be simulated under seismic loading and analysis on the shear, moment, tension and deflection of the whole structure can be done. After checking the resistance of the building under seismic loading, one specific connection of the beam-column joint which has the maximum values of axial force and bending moment will be analysed. The analysis will be done by using finite element analysis. This project is only limited to 2 dimensional (2D) view of beam-column joint for stress, cracking and crushing analysis.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Seismic loading is the application of the earthquake-generated effects on the structures. In ductile frame, the seismic load path flows through the beam-column joint. Thus, ductility is an essential attribute to a structure in order to withstand the strong vibration of the ground motion under seismic loading. Instead of ductility, the shear strength with effectively shear transfer mechanism is the most important thing in designing the reinforced concrete structures in order to prevent the building from collapse when subjected to seismic loading. The basic understanding on the limit state is essential in designing the load paths in the concrete systems. The approach for designing the structure is to design it on the most critical limit state without exceeds the remaining limit states. Two lateral loads such as wind and vibration loads are the major design factors to be considered in designing the building structures.

2.2 Ductility

Most structural design procedures serve the elastic behaviour as the basis of development. The induced level of deformation is significantly exceeding the idealized elastic limit of the system. Therefore, the elastic model must be used in the understanding of structures that will be subjected to earthquake-induced ground motions. Ductility is the relationship between the anticipated level of displacement and the displacement at idealized. According to the force-displacement relationship of ductile structures, at some point, the displacement increases with little or no increase in applied force. Energy, which is dissipated by the ductile structures, is taken into design methodologies consideration. To reduce the level of system response, the dissipated energy is converted into equivalent structural damping.

2.3 Shear Strength

Concrete component which is subjected to cyclic loads and postyield deformations need an effective shear transfer. Once the concrete cracked, the shear is transferred by two mechanisms which are by the cracked concrete and by a truss mechanism. Generally, the shear transfer in concrete is by interlocking in aggregate along the cracked surface. The load path of reinforced shear follows an internally developed truss or ties and strut model. The assumptions that the truss panel points are square and uniform in compression field is used to develop the codified mechanism strength for this load path.

2.4 Ground Behavior

Earthquake is due to violent shaking on the ground. The effects are temporarily to increase lateral and vertical forces and also to disturb intergranular stability of non-cohesive soils. The violent shaking also imposes the strains directly on surface material where the fault plane reaches the surface. This means that any soil structures that are capable of movement are at risk of this transient increase in lateral and vertical forces. Earthquake in Peru on 1970 and in Anchorage, Alaska on 1964 are the examples of resulting types of damages which is landslips. One village, Yungay, in Peru was destroyed almost entirely. 18 000 lives were lost by a debris flow involving tens of millions of tons of rock and ice.

The consolidation of both dry and saturated material are caused by the disturbance of the granular structure of soils by shaking which is due to the closer of packing grains. Temporary liquefaction which is caused by the increase of pore pressure of saturated sands by shaking can lead to massive foundation failure. Shear movement results from the soil displacement such as landslips and consolidation. Furthermore, inelastic displacement also occurs and it is critical in the piles' design.

2.5 Reinforced Concrete

Reinforced concrete is one of the building materials used in construction. It is a strong durable material that can be formed into many varied sizes and shapes ranging. Concrete is strong in compression and protects the steel to give durability and fire resistance.

There are many types of typical damage to elements subjected to bending, with or without direct force. The typical damage are diagonal cracking in the core, cracking in the tension zone, loss of concrete cover, the concrete core breaking into lumps by reversing diagonal cracking, stirrups bursting outwards and buckling of the main reinforcement. These typical damage leads to bond failure, which is particularly in zones where there are high cyclic stresses in the concrete and also direct shear failure of short elements. The failure also occurs at the beam-column intersection zone which is called shear cracking.

From the review of Bonacci and Pantazopoulou (1993) about the 86 building joint subassemblages tested in the laboratory, they had found that joint failure for 19 specimens was contributed by the failure of anchorage. Furthermore, joint failure by shear failure of the joint core was determined at 51 of 86 laboratory test specimens.

Meinheit and Jirsa (1977) had done a laboratory testing of the building assemblages with design details typical of pre-1970's construction. From the experiment, it shows that joints with little to no transverse reinforcement and relatively high shear and bond-stress demand exhibit severe stiffness and strength loss. Furthermore, Durrani and Wight (1985) had also observed the strength and stiffness for elements with moderate volumes transverse reinforcement and moderate shear and bond-stress demand.

2.6 Seismic Loadings

Corinaldesi and Moriconi (2006) conducted a study about the beam-column joints behavior made of sustainable concrete under cyclic loading. It is being done by preparing the substituting natural aggregates with recycled aggregates from building demolition. This experiment is based on submitting some real-scale beam-column joints to cyclic loading either natural or recycled-aggregate concrete to compare their behavior. Their aims are to promote the structure safety regarding the environmental issues. For the experiment, a commercial Portland-limestone cement type was used, which is according to the European Standards. Two different kind of aggregate, either recycle or natural of the same diameter was prepared for the concrete specimens. Three test was being conducted which were compression test and modulus of elasticity, splitting tension test and pull-out test. Furthermore, in order to compare the concretes by means of monotonic and low-cycle loading, the bond behavior of cyclic loading was being studied. Two types of concrete for beam-column joint were being made by natural-aggregates concrete (REF) and REC with another made of REC+FA. For the first concrete, the damage was observed in the beam portion close to the joint as predictable. While for the second concrete, the crisis occurred just in the joint. They conclude that there is different rupture mechanism which can characterize the beam-joint column due to its very low elastic modulus value for the recycled-aggregate concrete. The column and the joint should be more stiff than usual to obtain all the same ductile failure. In case of seismic design, to get better performance when the structure is shaken by the earthquake, it is noted that when fly ash is added to recycled-aggregate concrete, the higher deformability can be achieved.

Lowes and Altoontash (2003) have developed two constitutive models which are constitutive model for the shear panel and bar-slip component of the beam-column joint element. For the 1st model, the earthquake loading of joint results in substantial shear loading of the joint core. The inelastic response of the joint core is simulated by the shear-panel component. The response of joint subassemblages had been used. The MCFT is developed to characterize the global response of RC panels subjected to uniform shear and uniform shear plus axial load and to define the response of the shear panel component for several reasons. Stevens et al. (1991) has done a study for the MCFT extended simulation of response under cyclic loading. The response envelope is defined on the basis of the MCFT and experimental data provided by Stevens et al. Concrete compressive strength is reduced using the factor proposed by Stevens and a concrete tensile stress-strain model is derived from the Stevens data and used in the current implementation of the MCFT. The behavior is attributed to the opening and closing of cracks in the concrete-steel composite. For the 2nd model, it is developed to define the load-deformation history of the bond-slip springs that simulate inelastic anchorage-zone response. The experimental data of joint subassemblies testing is used to define the bar stress-slip relationship. The bar-stress versus slip relationship is developed on the basis of several simplifying assumptions about joint anchorage-zone response. As a conclusion, they indicate that the proposed model is appropriate for use in simulating response under earthquake loading.

Solberg et al. (2008) has conducted an experiment and computational on the seismic performance of damage-protected beam-column joints. It is about the 80% scale precast concrete three dimensional beam-column joint subassembly designed with damage-protected rocking connections. Rigid body kinematics has been identified as the theoretical basis of rocking system where the precast members are tied together using unbounded prestressed tendons. The hybrid systems were introduced and the investigation has been done about the behavior of these systems through a testing of a 5-storey 3D frame and wall system. As a result, less damage has been observed than would be expected with monolithic frames and negligible residual displacements observed in both frame and walls.

2.7 Seismic Analysis

Structural response due to earthquake is referring to stress, acceleration, displacement, shear, velocity or any other parameter affected by the ground motion. The dynamic analysis of a structure responding to dynamic forces is used to establish the strength and ductility requirements of the structure.

Pantelidas et al. (2008) has done an experimental research program about the seismic rehabilitation of reinforced concrete (RC) frame interior beam-column joints with FRP composites. The RC frame has been designed for gravity loads. By using carbon FRP (CFRP) and deficient under seismic loads, strengthening of RC beam-column interior joints in building frames was being addressed to improve the story shear capacity, displacement ductility, energy dissipation and inelastic rotation capacity of joints under simulated seismic loads. The experiment was done by measuring the load applied at the beam ends by using loads cells which attached in series with two actuators that applied the quasistatic cyclic loads. The column is subjected to constant axial load which is equivalent to 0.1 Agf_c' through an actuator at the column bottom. The assumption for beam-column joint design is the points of contraflexure occur at mid height of the columns and midspan of the beams. There are two types of beam-column joints were tested in this research with specific criteria and been divided into as-built condition and rehabilitated with CFRP composites. As a result for as-built specimen, concrete shear crack has developed. While for the rehabilitated with CFRP composites, CFRP delimitation has been observed. As a conclusion, ductile behavior has been successfully observed as the brittle joint shear failure and pullout of the beam bottom steel bars at the joint can be delayed and postponed the loss of stiffness and strength.

Al-Salloum and Almusallam (2007) have conducted an experimental study on the efficiency and effectiveness of carbon fiber-reinforced polymers (CFRP) in upgrading the shear strength and ductility of seismically deficient beam-column joints. The objective of this experiment is to evaluate seismic performance of as-built reinforced concrete (RC) interior connection. The comparison between connection performance with that of CFRP-repaired and CFRP-strengthened specimens has done. With non-optimum design parameters, four as-built RC interior beam-column subassemblages were constructed. The specimens has been divided into 2 parts; 2 specimens used as baseline specimens and 2 specimens were strengthened with CFRP sheets under two different schemes. Then, these specimens were being subjected to cyclic lateral load histories. The purpose is to provide equivalent of severe earthquake damage. After that, the damaged control specimens were being repaired using CFRP sheets. For the test of control specimens, shear cracks were observed in diagonal directions and propagated toward the ends of joints and also in the beams and columns which the cracks in the beams were higher than those in column. Then, the damage specimens were repaired through injecting epoxy into the cracks and bonding the specimens with CFRP sheets externally under either Scheme 1 or 2. These two schemes show the significant delay of shear failure of the joint which is due to either debonding or crushing/cracking of concrete. It also shows that the joint gains strength to such an extent and cause shifting of mode of failure from the joint to the beam. As a conclusion, both the shear strength and ductility of beam-column joints can be effectively improved by bonded the CFRP sheets externally. However, it may also shift the failure mode from the joint to the adjacent member.

Megawati et al. (2004) had developed a new set of attenuation relationship on rock site due to distant Sumatran-subduction earthquakes. The relationship is for shallow crustal earthquake in stable continent and active tectonic region for Singapore and the Malays Peninsula since the number of recorded ground motions in the region is very limited. This research has come out with the facts that the Sumatran Fault Segments have the potential to generate a specified level of response spectral acceleration in Singapore and Kuala Lumpur. It is based on the newly derived ground motion models.

2.8 Modelling of Concrete Behavior

Feenstra and Rots (2001) had made a comparison of four popular constitutive models for reinforced concrete on their merits for monotonic and cyclic loading. The four constitutive are multiple-fixed crack model with von Mises to model the crushing, Rankine-von Mises plasticity model, total strain-based fixed model and total strain-based rotating model. The monotonic analysis is performed by applying the vertical loading and monotonic increase at the center of the top slab. Inertia effect is negligible and the loading is considered being applied within the time domain. From this research, two aspects had been observed as the cause of monolithic and cyclic loading behavior. The aspects are the allowable stress and the unloading and reloading behavior. The failure surface and the evolution of the failure surface dominates the behavior for monotonic loading while for cyclic loading, the unloading and reloading of the models dominates the behavior. The results are influenced by the type of structure used, the reinforcement ratio and the material parameters.

Rose et al (2001) had developed a reinforced concrete model to analyse the inelastic behavior of reinforced concrete beam column members. In this research, the author uses a composite steel-concrete constitutive law to analyse the shear and flexural behavior of reinforced concrete beam columns with the model which based on the Modified Compression Field Theory. The model has been successfully implemented for cyclic loads. As a result, they observed that the panel can develop two types of failure modes when it is under pure shear which depends on the quantity of the reinforcing steel present. The failure modes are compressive crushing of the concrete struts and crack sliding. Throughout the experiments, it shows an excellent correlation for panels subjected to cyclic and monotonically increasing load.

In order to develop an excellent performance structures, the design needs to be safe, durable and serviceability. Maekawa et al. (2001) had developed in-plane spatially averaged constitutive models of RC elements with up to 4-way cracking. The structure is developed to predict the dynamic behavior. Using an active crack coordinate concept, compression, tension and shear stress-strain relationships had been applied on it. From this research, it indicates that the FEM tool is the best way for seismic performance evaluation of RC structures. Furthermore, in order to predict the deformation capacity of RC structures, modeling the buckling of main reinforcing bars and spalling of cover concrete is the most important thing for the prediction.

CHAPTER 3
METHODOLOGY

3.1 Introduction

This project is to study on strength of the reinforced beam-column joints when subjected to seismic loading. Before starting the modeling, some literature review through journals and readings material has been done regarding the seismic loading which affects the reinforced beam-column joints. Research regarding the shear strength and ductility of the joints has been found to be the important design factor to achieve satisfactory structures.

After some research has been done, the structural drawing of four storey school building has been chosen to be analysed. The building model is made of reinforced concrete and owned by Jabatan Kerja Raya (JKR) Malaysia. The structural design of the building is analysed by using STAAD.Pro software. Figure 3.1 shows the flow chart of the project. Detail for the project schedule can be referred at Appendix A.

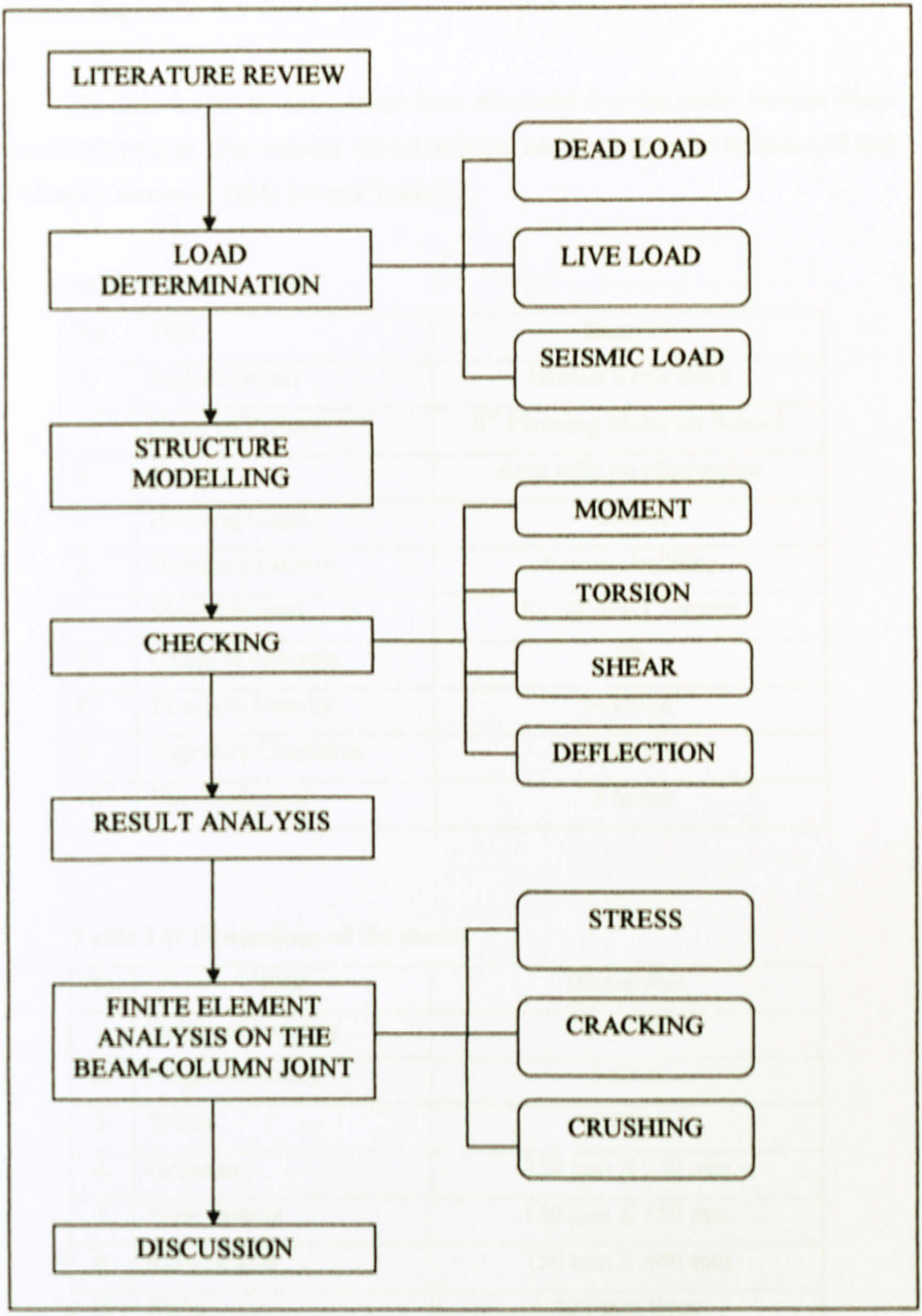

Figure 3.1: Flow Chart of the Project

3.2 Modeling

The data for the model is taken from structural drawing under Jabatan Kerja Raya (JKR) project. The 4-storey school building had been chose to be analysed and the detail is shown in Table 3.1 and Table 3.2.

Table 3.1: Model Data

No	Title	Remark
1.	Project Owner	Jabatan Kerja Raya
2.	Name of Project	8^{th} Planning Malaysia School
3.	Terrain	Area with no obstruction
4.	Building usage	School
5.	Number of storey	4 storey building
6.	Materials used	Reinforced Concrete
7.	Grade of concrete	30
8.	Concrete Density	24 kN/m^3
9.	Exposure Condition	Moderate
10.	Fire Resistance	2 hours

Table 3.2: Dimensions of the model

No.	Title	Dimension
1.	Height of building	14.40 m
2.	Height of Storey	3.60 m
3.	Width	7.80 m
4.	Columns	350 mm X 250 mm
5.	Floor Beams	150 mm X 550 mm
6.	Roof Beams	150 mm X 600 mm
7.	Slabs	125 mm thick

Figure 3.2, 3.3 and 3.4 shows the 3-D view, side view and the front view of the school building dimensions.

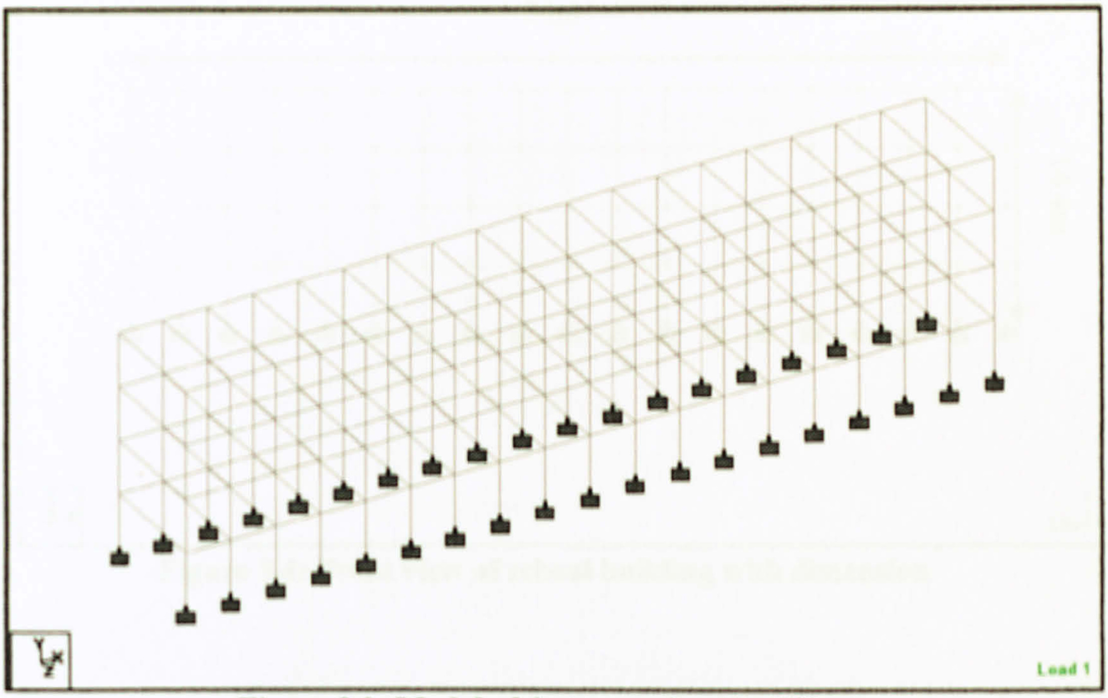

Figure 3.2: Model of 4-storey school building

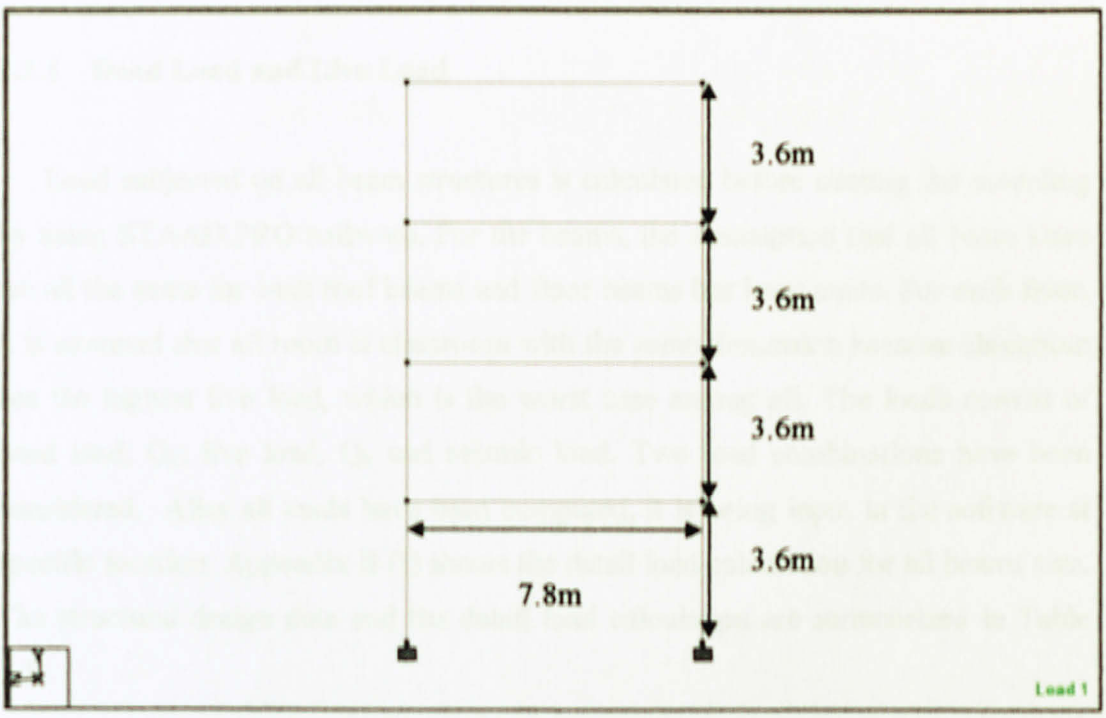

Figure 3.3: Side view of school building with dimension

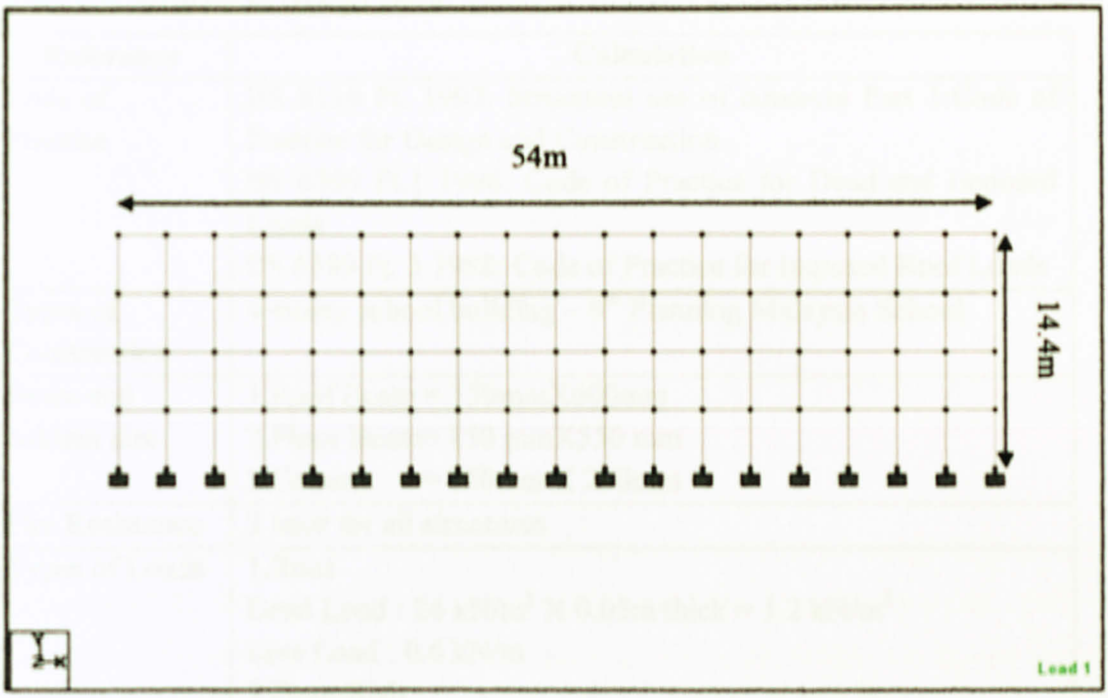

Figure 3.4: Front view of school building with dimension

3.3 Load Determination

3.3.1 Dead Load and Live Load

Load subjected on all beam structures is calculated before starting the modeling by using STAAD.PRO software. For the beams, the assumption that all beam sizes are all the same for each roof beams and floor beams has been made. For each floor, it is assumed that all room is classroom with the same dimension because classroom has the highest live load, which is the worst case among all. The loads consist of dead load, G_k; live load, Q_k and seismic load. Two load combinations have been considered. After all loads have been computed, it is being input in the software at specific location. Appendix B (I) shows the detail load calculation for all beams size. The structural design data and the detail load calculation are summarized in Table 3.3.

Table 3.3: Structural Design Data

Reference	Calculation
Code of Practice	BS 8110 Pt. 1997: Structural use of concrete Part 1:Code of Practice for Design and Construction BS 6399 Pt.1 1996: Code of Practice for Dead and Imposed Loads BS 6399 Pt. 3 1988: Code of Practice for Imposed Roof Loads
Types of Construction	4-storey school building – 8^{th} Planning Malaysia School
Beam and column size	1.Roof Beam = 150mmX600mm 2.Floor Beam= 150 mmX550 mm 3.Column = 350mm X 250mm
Fire Resistance	2 hour for all structures
Types of Loads	1.Roof Dead Load : 24 kN/m^3 X 0.05m thick = 1.2 kN/m^2 Live Load : 0.6 kN/m 2.Floor/Slab Dead Load : (0.125m x 24 kN/m^3) + 0.75 kN/m^2 = 3.75 kN/m^2 Live Load : 3.0 kN/m^2 3.Wall Dead Load: 1.0 kN/m^2 Live Load : 3.1 kN/m^2 (115 mm thick brickwall with 3.6m height) 4. Ceiling Dead Load : 24 kN/m^3 X 0.0032m = 0.1 kN/m^2
Materials	Grade of Concrete: f_{cu} = 30 N/mm2
Reinforcement strength	Characteristic strength of reinforcement : f_y = 410 N/mm2 Characteristic strength of link reinforcement: f_{yv} = 250 N/mm2
Concrete Cover	Beam cover = 25 mm
Concrete Density	Dead Load of Concrete = 24 kN/m^3

3.3.2 Seismic Load

During the earthquake, the ground surface moves in X, Y and Z direction. The movements parallel to the ground surface, which is at X and Y direction, generally cause the largest part of damaging effects on the stationary structures because structures are normally designed to support vertical gravity loads (Ambrose and Vergun, 1999). In this study, seismic load in the form of spectrum analysis is applied on the structure for the analysis. The response spectrum is taken from Arshad et.al (2007).

Response spectra are the plots of maximum response of single degree of freedom (SDOF) systems subjected to a specific excitation. It is simply a plot of the peak of a series of oscillators of varying natural frequency, which are forced into motion by the same base vibration. For this study, each plot is for SDOF systems having a fixed damping ratio of 0.05. The maximum modal responses are combined using Complete Quadratic Combination method (CQC). It is noted that once the combination method of CQC are applied, the sign of the results is lost. Consequently, results of a spectrum analysis such as displacement, reactions and forces do not have any sign.

In this study, the building is considered constructed on a very dense soil and soft rock (soil class C). The design spectra acceleration and the time period are shown in Table 3.4.

Table 3.4: Time Period for Soil Class C (Arshad et. al, 2007)

Period	Acceleration(m/sec^{2})
0.09	0.247
0.47	0.247
0.80	0.1463
1.00	0.1170
1.50	0.0780
2.00	0.0585
2.50	0.0468
3.00	0.0390
3.50	0.0334
4.00	0.0293
4.50	0.0260
5.00	0.0234
5.50	0.0213
6.00	0.0195
6.50	0.0180
7.00	0.0167
7.50	0.0156
8.00	0.0146
8.50	0.0138
9.00	0.0130
9.50	0.0123
10.00	0.0117

3.3.3 Load Combination

Load combinations for concrete structure are base on the British Standard BS8110. The loads are as follows:

a) Load Combination 1 : $U = 0.75G_k + 0.75Q_k + 0.75EQ$

b) Load Combination 2 : $U = 0.75G_k + 0.75Q_k - 0.75EQ$

Where,

U = Ultimate Load resulting from load combination

G_k = Dead Load

Q_k = Live Load

EQ= Seismic Load

After applying the loads on the beams, the software will run the analysis. The results of axial force, deflection, bending moment, torsion and shear are analyzed and compared with the value of static capacity which the beams can sustain.

3.4 Finite Element Method

After the analysis of the building structures has been done, finite element analysis of beam-column joint is conducted. The beam-column joint is designed by using nodes and plate elements. The end of the column is assumed as fixed support while the forces are applied on the beams through nodes.

By using the maximum values of axial force and bending moment taken from the result analysis from previous model, the forces and bending moments are distributed through each node. For axial forces, it is uniformly distributed on each node while for the moments; it is converted to resultant force and distributed evenly according to the stress diagram. Refer Appendix B (II) for the detail calculation.

The analysis of this beam-column joint is to determine the stress, cracking and crushing pattern and also to determine the location of cracking and crushing development. Crushing will develop if the compression of the joint is higher than the compressive strength of concrete; F_C, while cracking will develop if the tension of the joint is higher than tensile strength of concrete; F_T. Figure 3.5 shows the 2D view of beam-column joint. The vertical figure is the column with fixed end support while the horizontal figure is the beam.

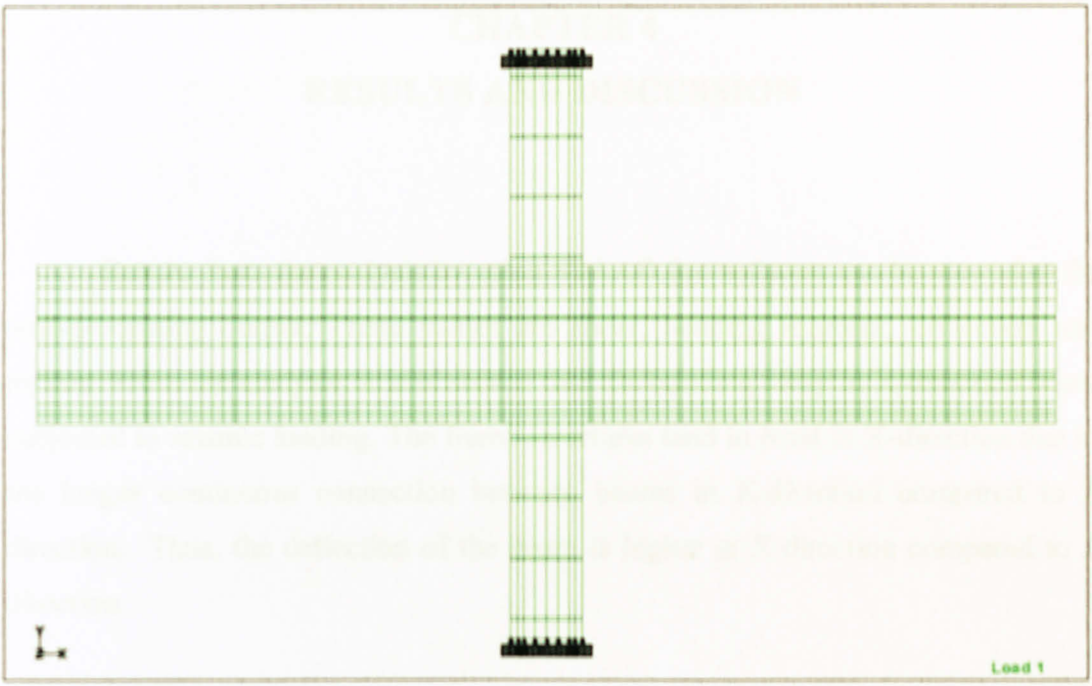

Figure 3.5: 2D view of beam-column joint

3.5 Ergonomics on Computer Workstation

3.5.1 Monitors

Monitor can affects both eyes and the musculoskeletal system of human beings. Thus, users must pay attention on the placement and maintenance of the monitor which can brings bad effects to them. While using the computer workstation, the development of eye strain, shoulder fatigue and neck pain can be prevented by make sure the surface of the viewing screen is clean and adjust the brightness and contrast to optimum comfort.

3.5.2 Chair Adjustment

Sitting on a chair for a very long time can brings to increasing pressure on the intervertebral discs. To avoid this, the height of backrest must be adjusted to support the natural inward curvature of the lower back and adjust the height of chair so feet rest flat on the floor.

CHAPTER 4

RESULTS AND DISCUSSION

Results from the analysis by using STAAD.Pro software can be viewed at the post-processing output. These values are shear, bending moment, deflection and torsion. From the analysis, it is observed that the building sway in X-direction when subjected to seismic loading. The frame structures tend to bend in X-direction due to the longer continuous connection between beams in X-direction compared to Z direction. Thus, the deflection of the beam is higher at X direction compared to Z direction.

4.1 Axial Force, Shear, Bending Moment and Torsion Due to Seismic Loading

The school building's shear, bending moment and torsion due to seismic loading are compared with the manual calculation of static capacity. The values are moment capacity, shear capacity and total torsional resistance which are the maximum limit of moment, shear and torsion that the structure can sustain. Figure 4.1 shows the specific location for the selected beam and column while Table 4.1 shows the result of maximum forces by section properties for the whole structure of school building due to seismic load.

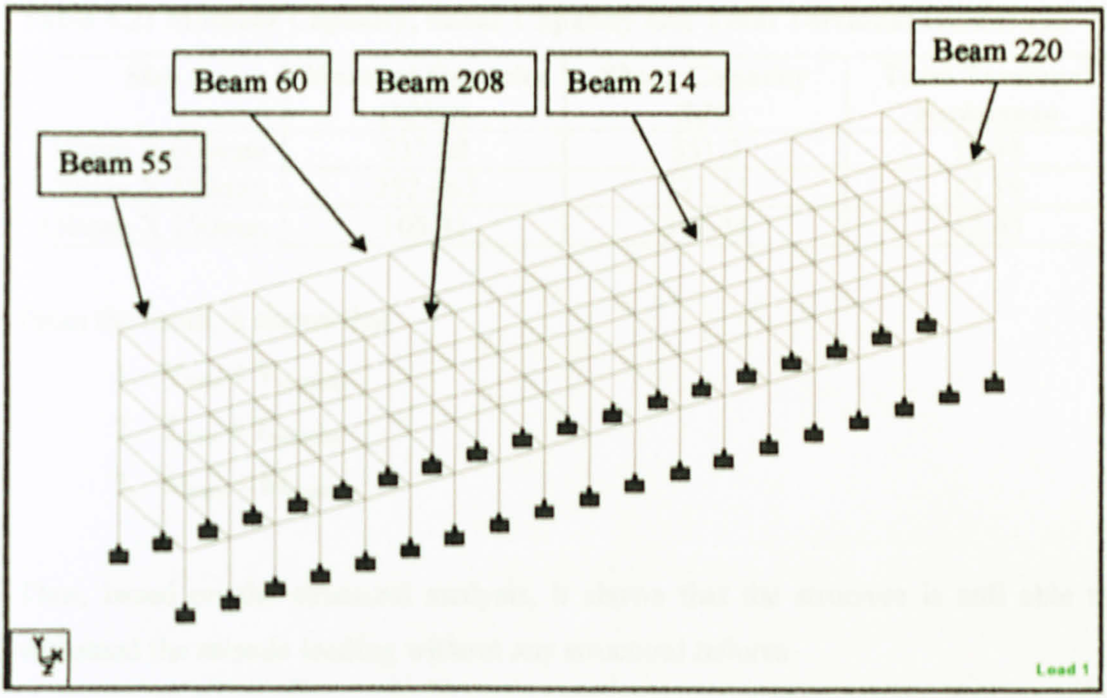

Figure 4.1: Location of the selected beam and column

Table 4.1: Forces by Section Properties: Whole Structure for [I] Maximum +ve and [II] Maximum -ve

		Axial	Shear		Torsion	Bending Moment	
Section		Max Fx kN	Max Fy kN	Max Fz kN	Max Mx kNm	Max My kNm	Max Mz kNm
Beam 150x600	[I]	24.023	25.269	0.589	0.595	1.161	37.364
	[II]	-1.380	-25.269	-0.589	-0.595	-1.161	-28.163
Beam 150x550	[I]	16.906	131.485	0.466	1.264	0.912	168.562
	[II]	-17.606	-131.485	-0.466	-1.264	-0.912	-143.576
Column 350x250	[I]	674.307	16.094	49.93	0.356	92.657	29.703
	[II]	-84.467	-16.094	-49.93	-0.356	-92.657	-28.235

Moment capacity, shear capacity and total torsional resistance are calculated manually based on the structural drawing given by the JKR. Table 4.2 shows the value of moment capacity, shear capacity and total torsional resistance. Refer Appendix B (III) for further detail calculation.

Table 4.2: Moment Capacity, Shear Capacity and Total Torsional Resistance

Size	Moment Capacity (kNm)	Shear Capacity (kN)	Total Torsional Resistance
150mm X 600mm	237.60	351.2	18.98
150mm X 550mm	252.465	321.31	17.19
350mm X 250mm	108.41	311.34	52.65

From the result, it shows that

1. $V_{max} < V_{capacity}$
2. $T_{max} < T_{capacity}$
3. $M_{max} < M_{capacity}$

Thus, based on the structural analysis, it shows that the structure is still able to withstand the seismic loading without any structural failures.

4.2 Deflection Due to Seismic Load

Table 4.3 below shows the result of maximum and minimum deflection for 4 storey school building model. The results show the deflection in X, Y and Z direction.

Table 4.3: Deflection of beam and column due to seismic loading

	Beam/ Column	Load/ Combination	Horizontal X (mm)	Vertical Y (mm)	Horizontal Z (mm)	Resultant Resultant (mm)
Max X	220	SL	20.929	0.31	17.997	27.604
Min X	220	LC 2	-15.739	-1.966	-13.531	20.848
Max Y	55	SL	20.926	0.31	17.997	27.602
Min Y	60	LC 2	-15.672	-2.693	-16.753	23.098
Max Z	208	SL	20.909	0.129	22.399	30.642
Min Z	208	LC 2	-15.666	-2.693	-16.845	23.161
Max Resultant	214	SL	20.917	0.129	22.399	30.647

*SL = Seismic Load

*LC2 = Load Combination 2

The maximum allowable deflection for the structural members due to Uniform Building Code (UBC) 1997 is:

$$\text{Maximum deflection} = \frac{\text{length of structure}}{240}$$

$$= \frac{5400 \text{ mm}}{240}$$

$$= 225 \text{ mm}$$

From the result, it shows that the maximum deflection occurs when subjected to seismic loading alone. Furthermore, the maximum deflection doesn't exceed the maximum allowable deflection of the structure. Thus, the school building is still able to withstand the dynamic load during the earthquake.

4.3 Finite Element Analysis of Beam-Column Joint

Results from the finite element analysis of beam-column joint gives the intensity of the forces distributed over the plates which is shown from the plate stress contour. For this project, only two types of stresses is considered for observation which are stress in Y direction; SY and stress in X direction; SX. The positive value indicates a tensile stress while negative value indicates a compressive stress. Figure 4.2 and 4.3 shows the plate stress contour for stress in X direction, SX and in Y direction,SY.

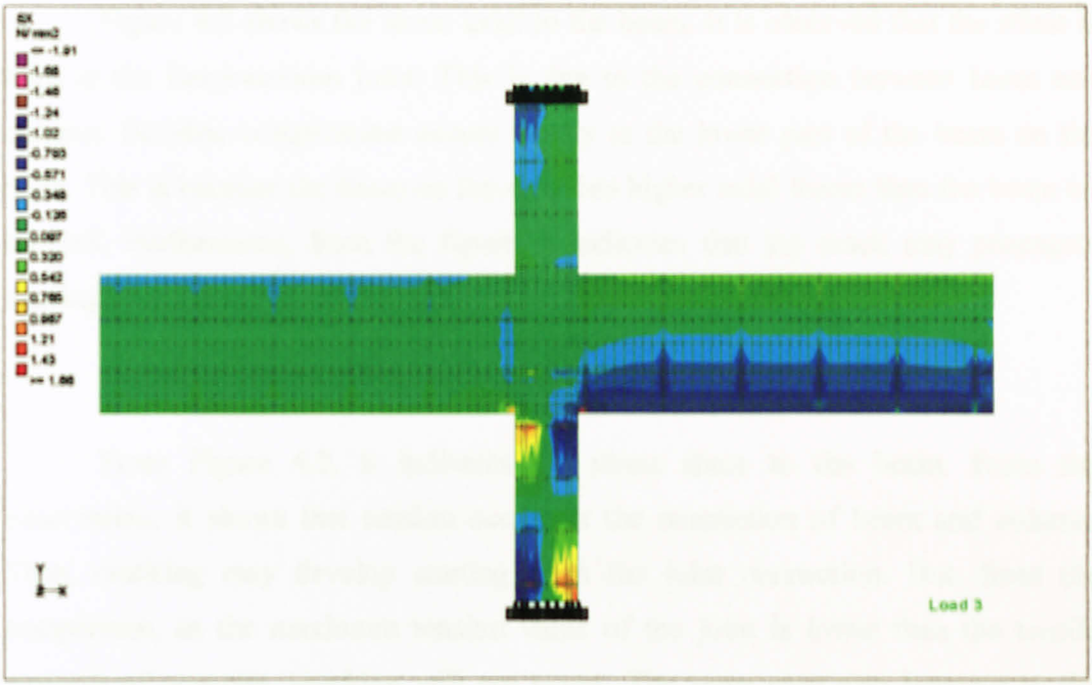

Figure 4.2: Plate Stress Contour of Stress in X direction, SX.

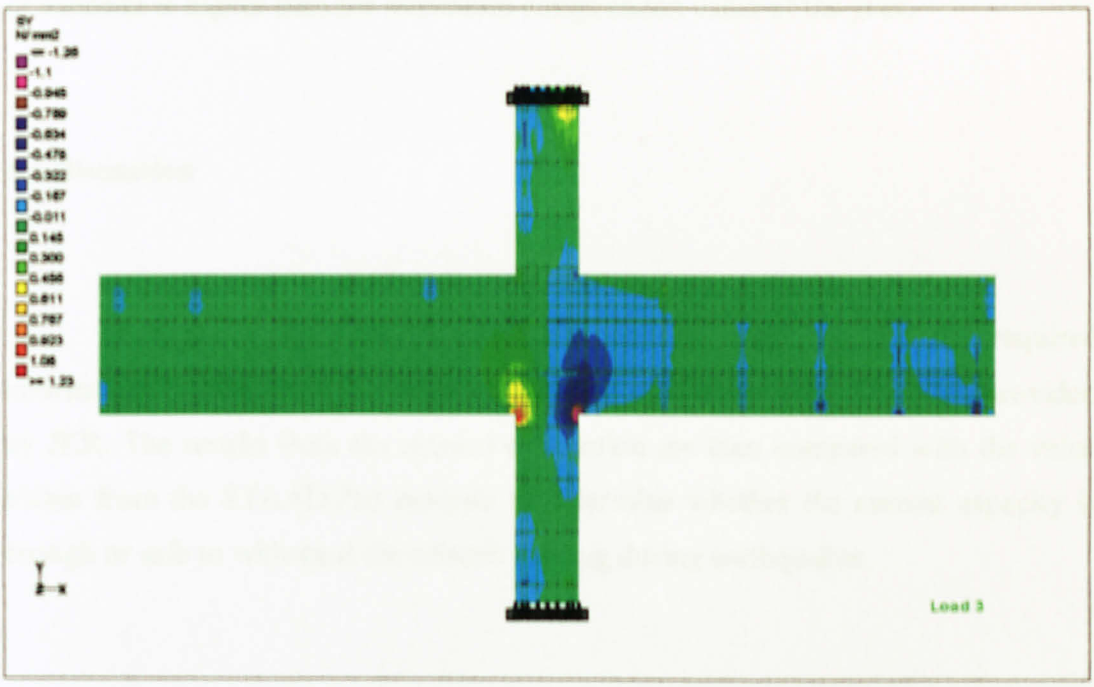

Figure 4.3: Plate Stress Contour of Stress in Y direction, SY.

Figure 4.2 shows the stress axial to the beam. It is observed that the stress is high at the beam-column joint. This is due to the connection between beam and column. Besides, compression occurs mostly at the lower part of the beam on the right. This is because the beam on the right has higher axial forces than the beam on the left. Furthermore, from the figure, it indicates that the crack may propagate starting from the corner of the joint.

From Figure 4.3, it indicates the stress shear to the beam. From the observation, it shows that tension occurs at the connection of beam and column. Thus, cracking may develop starting from the joint connection. But, from the comparison, as the maximum tension value of the joint is lower than the tensile strength of concrete, cracking will not occur. The same case also happens to the compression of the joint where crushing will not occur as the compression strength of concrete is higher than the maximum compression value of the joint.

4.4 Discussion

From the calculation of beam and column capacities, all the required information is taken from the structural drawing of 4-storey school building provided by JKR. The results from the manual calculation are then compared with the value obtain from the STAAD.Pro analysis to determine whether the current capacity is enough or safe to withstand the seismic loading during earthquakes.

Before starting the modeling process, dead load and live load are calculated manually according to the BS 8110 and BS 6399 standard. These values are then being input to STAAD.Pro for analysis of the frame. Spectrum analysis is used for the definition of seismic loading.

In Post-Processing results, the shear force, torsion, bending moment and axial forces enveloped on beams and columns are obtained. After comparing the values with the maximum capacity of shear, bending moment and torsional resistance, it is found that the beams and columns are still able to withstand the seismic load.

CHAPTER 5

CONCLUSIONS & RECOMMENDATION

A four-storey school building model has been developed for the analysis and design of reinforced concrete building using STAAD.Pro 2005 Software. The model has been generated according to the structural drawing and the calculated design load according to the British Standard.

From the analysis of the four-storey school building in Malaysia which is never been designed for the earthquake resistance, the columns and beams are slightly affected by the seismic loading. The deflection, bending moment, torsion and shear force of the whole structures is not exceeding the capacity that has been designed. The school structure is still able to withstand the seismic loading and safe to be used.

The results from the finite element analysis show that the tension and compression value doesn't exceed the concrete's tensile strength and compressive strength. Thus, no cracking or crushing occurs. However, the accuracy of the result can be improved by upgrading the finite element analysis from 2D view to 3D view. More accurate result of plate stress contour can be obtained for the beam-column joint when depth of the beam and column is taking into consideration.

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APPENDICES

APPENDIX A

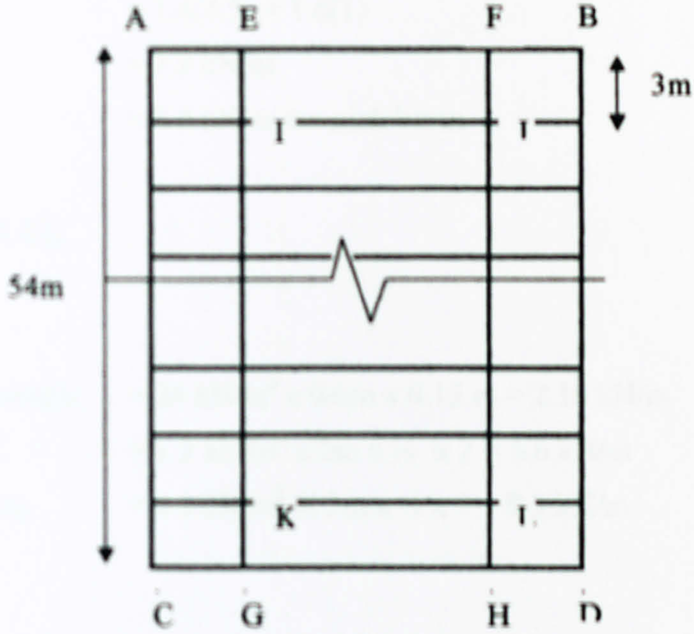
Gantt Chart

No.	Detail/ Week	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	Design Spectrum Analysis	■	■	■	■						Mid-semester break					
2	Finite Element Analysis					■	■	■	■	■		■	■	■		
3	Submission of Preliminary/Progress Report						●									
4	Poster Presentation												●			
5.	Dissertation Report Submission													●		
6.	Oral Presentation															●

Manual Calculation

(I) Load Calculation

Load Calculation of Roof Beam Structure



7.8 / 3 = 2.6 > 2 (1 way slab)

All beam sizes = 150 mm X 600 mm

<u>Beam EF and GH</u>

Dead Load;

Self weight $= 24\ kN/m^3 \times 0.6m \times 0.15\ m = 2.16\ kN/m$

Roof $= 1.2\ kN/m^2 \times 3m \times ½ = 1.8\ kN/m$

Ceiling $= 0.1\ kN/m^2\ X\ 3m \times ½ = 0.15\ kN/m$

Live Load;

Roof $= 0.6\ kN/m^2 \times 3m \times ½ = 0.9\ kN/m$

Design Load;

G_k $= 2.16 + 1.8 + 0.15 = 4.11\ kN/m$

$\approx 4.5\ kN/m$

Q_k = 0.9 kN/m

≈ 1 kN/m

Design Load, $F = 1.4\ G_k + 1.6Q_k$

= 1.4(4.5) + 1.6(1)

= 7.9 kN/m

≈ 8.0 kN/m for each beam

Beam IJ until KL

Dead Load;

Self weight = 24 kN/m^3 x 0.6m x 0.15 m = 2.16 kN/m

Roof = 1.2 kN/m^2 x 3m x ½ x 2 = 3.6 kN/m

Ceiling = 0.1 kN/m^2 X 3m x ½ x 2 = 0.3 kN/m

Live Load;

Roof = 0.6 kN/m^2 x 3m x ½ x 2 = 1.8 kN/m

Design Load;

G_k = 2.16 + 3.6 + 0.3 = 6.06 kN/m

≈ 6.1 kN/m

Q_k = 1.8 kN/m

Design Load, $F = 1.4\ G_k + 1.6Q_k$

= 1.4(6.1) + 1.6(1.8)

= 11.42 kN/m for each beam

Beam EI & FJ until KG & LH

Dead Load;

Self weight = 24 kN/m^3 x 0.6m x 0.15 m = 2.16 kN/m

Roof = 1.2 kN/m^2 x (2.3 m x 1/3 + 7.8m X 1.3) = 4.04 kN/m

Ceiling = 0.1 kN/m^2 X (2.3 m x 1/3 + 7.8m X 1.3) = 0.34 kN/m

Live Load;

Roof $= 0.6\ kN/m^2 \times (2.3\ m \times 1/3 + 7.8m \times 1.3) = 2.02\ kN/m$

Design Load;

G_k $= 2.16 + 4.04 + 0.34 = 6.54\ kN/m$

$\approx 7.0\ kN/m$

Q_k $= 2.02\ kN/m$

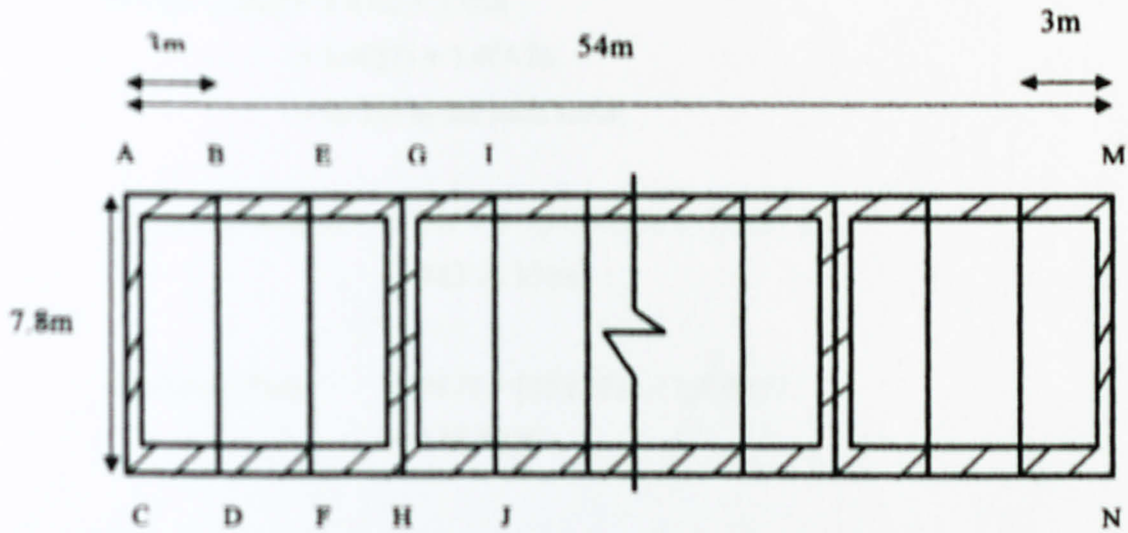
$\approx 2.1\ kN/m$

Design Load,F $= 1.4\ G_k + 1.6Q_k$

$= 1.4(7.0) + 1.6(2.1)$

$= 13.16\ kN/m$ for each beam

b) Load Calculation of Floor Beam Structure



7.8/3 = 2.6 > 2 one way slab

All beam sizes = 150 mm X 550 mm

<u>Beam AC & MN</u>

Dead Load;

Finishes $= 1.2\ kN/m^2 \times 7.8\ m = 9.36\ kN/m$

Self weight $= 24\ kN/m^3 \times 0.55m \times 0.15\ m = 1.98\ kN/m$

Slab $= 3.75\ kN/m^2 \times 3m \times ½ = 5.625\ kN/m$

Wall(115mm) $= 3.1\ kN/m^2\ X\ (3.6 - 0.55)m = 9.46\ kN/m$

Live Load;

Slab $= 3.0\ kN/m^2 \times 3m \times ½ = 4.5\ kN/m$

Design Load;

G_k $= 9.36 + 1.98 + 5.625 + 9.46 = 26.43\ kN/m$

$\approx 27\ kN/m$

Q_k $= 4.5\ kN/m$

Design Load,F $= 1.4\ G_k + 1.6Q_k$

$= 1.4(27) + 1.6(4.5)$

$= 45$ kN/m for each beam

Maximum moment $= PL^2 / 8 = [45\ kN/m \times (7.8m)^2]/8$

$= 342.23\ kNm$

Maximum Shear $= PL/2 = [45\ kN/m \times 7.8\ m]\ /2$

$= 175.5\ kN$

<u>Beam GH and similar beams to it</u>

Dead Load;

Finishes $= 1.2\ kN/m^2 \times 7.8\ m = 9.36\ kN/m$

Self weight $= 24\ kN/m^3 \times 0.55m \times 0.15\ m = 1.98\ kN/m$

Slab $= 3.75\ kN/m^2 \times 3m = 11.25\ kN/m$

Wall(115mm) $= 3.1\ kN/m^2\ X\ (3.6 - 0.55)m = 9.46\ kN/m$

Live Load;

Slab $= 3.0\ kN/m^2 \times 3m = 9.0\ kN/m$

Design Load;

G_k $= 1.98 + 9.36 + 11.25 + 9.46 = 32.05\ kN/m$

$\approx 32.1\ kN/m$

Q_k $= 9.0\ kN/m$

Design Load, $F = 1.4\ G_k + 1.6Q_k$

$= 1.4(32.1) + 1.6(9.0)$

$= 60$ kN/m for each beam

Maximum moment $= PL^2 / 8 = [60\ kN/m \times (7.8m)^2]/8$

$= 456.3\ kNm$

Maximum Shear $= PL/2 = [60\ kN/m \times 7.8\ m]/2$

$= 234\ kN$

<u>Beam BD and similar beams to it</u>

Dead Load;

Finishes $= 1.2\ kN/m^2 \times 7.8\ m = 9.36\ kN/m$

Self weight $= 24\ kN/m^3 \times 0.55m \times 0.15\ m = 1.98\ kN/m$

Slab $= 3.75\ kN/m^2 \times 3m = 11.25\ kN/m$

No brickwall on the beam

Live Load;

Slab $= 3.0\ kN/m^2 \times 3m = 9.0\ kN/m$

Design Load;

G_k $= 1.98 + 9.36 + 11.25 = 22.59\ kN/m$

$\approx 23\ kN/m$

Q_k $= 9.0\ kN/m$

Design Load,F = 1.4 G_k + 1.6Q_k

= 1.4(23.0) + 1.6(9.0)

= 47 kN/m for each beam

Maximum moment = $PL^2 / 8$ = [47 kN/m x $(7.8m)^2$]/8

= 357.435 kNm

Maximum Shear = PL/2 = [47 kN/m x 7.8 m] /2

= 184 kN

<u>Beam AB and similar beams to it</u>

Dead Load;

Finishes = 1.2 kN/m^2 x 3 m = 3.6 kN/m

Self weight = 24 kN/m^3 x 0.55m x 0.15 m = 1.98 kN/m

Slab = 3.75 kN/m^2 x 7.8m x 1/3 = 9.75 kN/m

Wall(115mm) = 3.1 kN/m^2 X (3.6 – 0.55)m = 9.46 kN/m

Live Load;

Slab = 3.0 kN/m^2 x 7.8m x 1/3 = 7.8 kN/m

Design Load;

G_k = 1.98 + 3.6 + 9.75 + 9.46 = 24.79 kN/m

≈ 25 kN/m

Q_k = 7.8kN/m

Design Load,F = 1.4 G_k + 1.6Q_k

= 1.4(25) + 1.6(7.8)

= 48 kN/m for each beam

Maximum moment $= PL^2 / 8 = [48 \text{ kN/m} \times (7.8\text{m})^2]/8$

$= 54 \text{ kNm}$

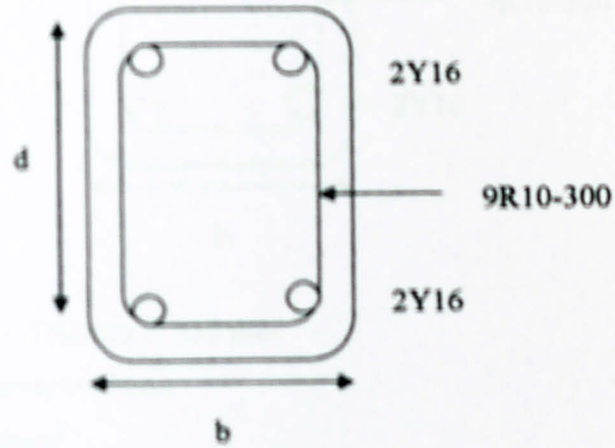
Maximum Shear $= PL/2 = [48 \text{ kN/m} \times 7.8 \text{ m}] /2$

$= 72 \text{ kN}$

(II)Design Calculation

1) Calculation of Moment Capacity

a) Roof Beam



Beam size = 150mm X 600 mm

Concrete cover = 25mm

$f_y = 410\ N/mm^2$

$f_{cu} = 30\ N/mm^2$

2h fire resistance = 20mm

$d = 600mm - 10mm - 25mm - 20/2\ mm = 555\ mm$

$d' = 25\ mm + 10mm + 16/2\ mm = 43\ mm$

$As = As' = 402.12\ mm^2$

$F_{st} = F_{cc} + F_{sc}$

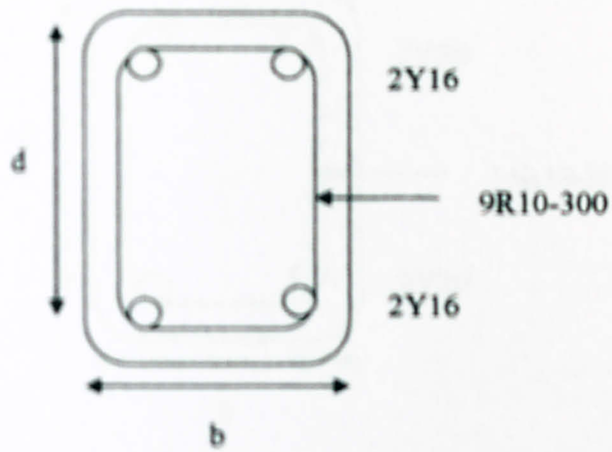
$M = F_{cc}\ (d - s/2) + F_{sc}\ (d\text{-}d')$

$s = 0.9\ X\ (d/2) = 0.9\ X\ (600/2) = 270mm$

Thus,

$$
\begin{aligned}
M &= F_{cc}\ (d - s/2)\ \ F_{sc}\ (d\text{-}d') \\
&= 0.45 f_{cu} bs\ (d\text{-}s/2) + 0.95 f_y As\ (d\text{-}d') \\
&= 0.45(30)(150)(270)(505\text{-}235) + 0.95(410)(402.12)(555\text{-}43) \\
&= 237.60\ kNm
\end{aligned}
$$

b) Floor Beam



Beam size = 150 mm X 550 mm

Concrete cover = 25mm

f_y = 410 N/mm^2

f_{cu}= 30 N/mm^2

2h fire resistance = 20mm

d = 550 mm – 10mm – 25mm – 20/2 mm =505 mm

d'= 25 mm + 10mm + 16/2 mm = 43 mm

As = As' = 402.12 mm^2

$F_{st} = F_{cc} + F_{sc}$

$M = F_{cc}\,(d - s/2) + F_{sc}\,(d\text{-}d')$

s = 0.9 X (d/2) = 0.9 X (505/2) = 227.25mm

Thus,

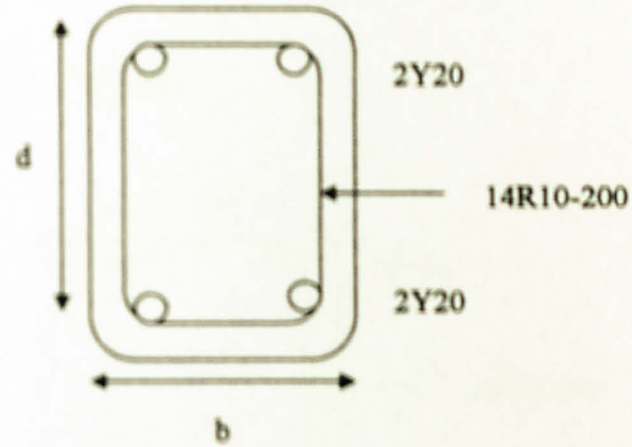
$M = F_{cc}\,(d - s/2) + F_{sc}\,(d\text{-}d')$

$= 0.45 f_{cu} bs\,(d\text{-}s/2) + 0.95 fyAs\,(d\text{-}d')$

$= 0.45(30)(150)(227.25)(505\text{-}113.625) + 0.95(410)(402.12)(505\text{-}43)$

= 252.465 kNm

c) Column



Column size = 350 mm X 250 mm

Concrete cover = 25mm

f_y = 410 N/mm^2

f_{cu}= 30 N/mm^2

2h fire resistance = 20mm

d = 250 mm – 10mm – 25mm – 20/2 mm =205 mm

d'= 25 mm + 10mm + 20/2 mm = 45 mm

As = As' = 628.32 mm^2

$F_{st} = F_{cc} + F_{sc}$

$M = F_{cc} (d - s/2) + F_{sc} (d\text{-}d')$

s = 0.9 X (d/2) = 0.9 X (205/2) = 92.25mm

Thus,

$M = F_{cc} (d - s/2) + F_{sc} (d\text{-}d')$

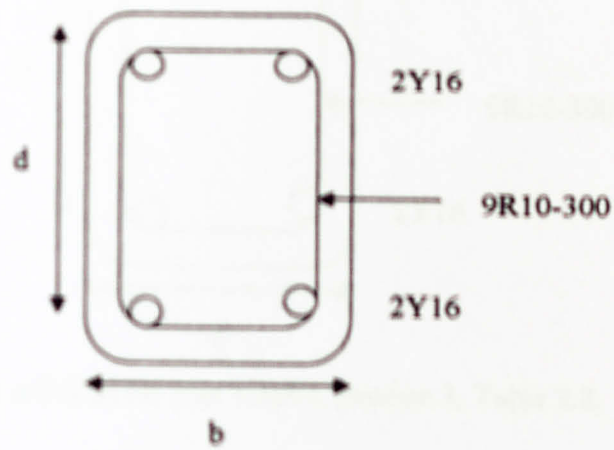
$= 0.45 f_{cu} bs (d\text{-}s/2) + 0.95 fyAs (d\text{-}d')$

$= 0.45(30)(350)(92.25)(205\text{-}46.13) + 0.95(410)(628.32)(205\text{-}45)$

= 108.41 kNm

2) Calculation of Shear Capacity

a)Beam size = 150mm X 600 mm



According to BS 8110: Part 1:1997, Section 3, Table 3.8,

Shear resistance, $v_c = \dfrac{0.79(100A_s/bd)^{1/3}(400/d)^{1/4}}{1.25} \times (f_{cu}/25)^{1/3}$

$= \dfrac{0.79\,(100 \times 402.12/150 \times 555)^{1/3}(400/555)^{1/4}}{1.25} \times (30/25)^{1/3}$

$= 0.49\ N/mm^2$

$= 490\ kN/m^2$

For the stirrups,

$A_{sv} / s_v = 9 \times (78.5 / 300) = 2.355$

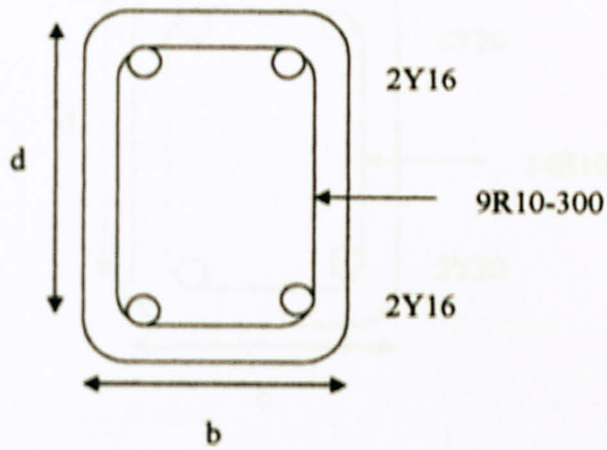
Thus, for the shear resistance of the stirrups plus the concrete,

$V_s = (A_{sv} / s_v) \times 0.95 f_{yv} d + b v_c d$

$= 2.355 \times 0.95(250)(555) + (150)(0.49)(555)$

$= 351.2\ kN$

b) Beam size = 150mm X 550 mm



According to BS 8110: Part 1:1997, Section 3, Table 3.8,

$$\text{Shear resistance}, v_c = \frac{0.79(100As/bd)^{1/3}(400/d)^{1/4}}{1.25} \times (f_{cu} / 25)^{1/3}$$

$$= \frac{0.79\,(100x402.12/150x505)^{1/3}(400/505)^{1/4}}{1.25} x(30/25)^{1/3}$$

$$= 0.513\ N/mm^2$$

$$= 513\ kN/m^2$$

For the stirrups,

$$A_{sv} / s_v = 9\ X\ (78.5 / 300) = 2.355$$

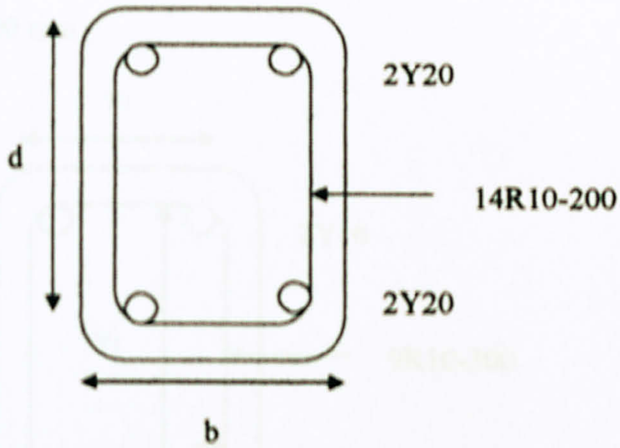
Thus, for the shear resistance of the stirrups plus the concrete,

$$V_s = (A_{sv} / s_v)\ X\ 0.95 f_{yv} d + bv_c d$$

$$= 2.355\ X\ 0.95(250)(505) + (150)(0.513)(505)$$

$$= 321.31\ kN$$

c) Column size = 350 mm X 250 mm



According to BS 8110: Part 1:1997, Section 3, Table 3.8,

$$\text{Shear resistance}, v_c = \frac{0.79(100As/bd)^{1/3}(400/d)^{1/4}}{1.25} \times (f_{cu} / 25)^{1/3}$$

$$= \frac{0.79\ (100x628.32\ /250x205)^{1/3}(400/205)^{1/4}}{1.25} x(30/25)^{1/3}$$

$$= 0.85 N/mm^2$$

$$= 850\ kN/m^2$$

For the stirrups,

$$A_{sv} / s_v = 14 \text{ X } (78.5 / 200) = 5.5$$

Thus, for the shear resistance of the stirrups plus the concrete,

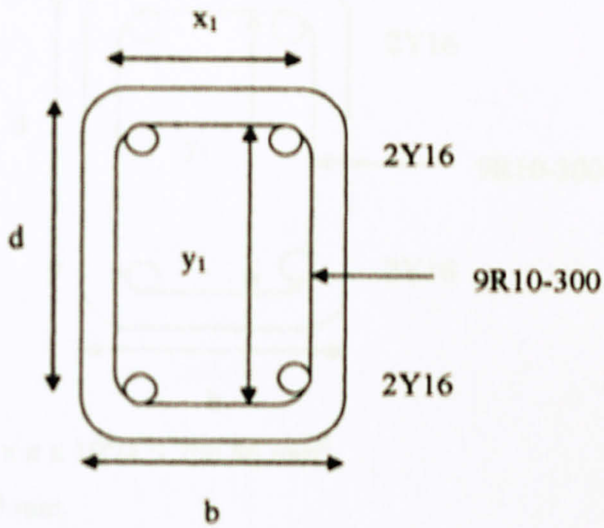
$$V_s = (A_{sv} / s_v) \text{ X } 0.95 f_{yv} d + b v_c d$$

$$= 5.5 \text{ X } 0.95(250)(205) + (250)(0.85)(205)$$

$$= 311.34\ kN$$

3) Total torsional resistance

a)Beam 150 mm X 600 mm



$A_{sv} = 9 \times \pi \times 10^2/4 = 706.86\ mm^2$

$s_v = 300$ mm

$f_{yv} = 250\ N/mm^2$

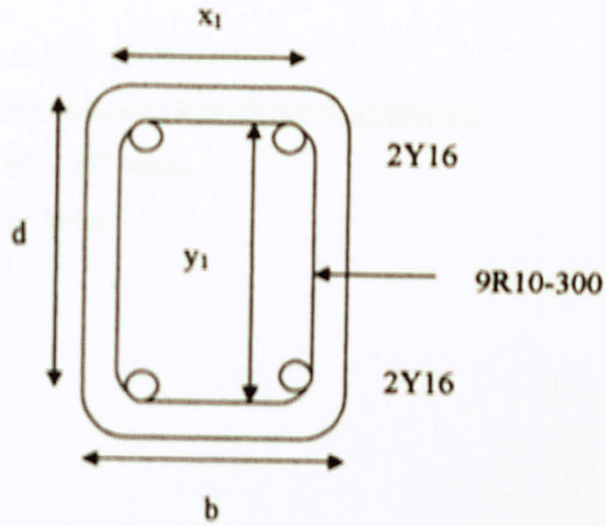
$T = (A_{sv}/s_v)x_1y_1(0.95f_{yv})x0.8$

$= (706.86/300)(530)(80)(0.95x250)x0.8$

$= 18.98 \times 10^6$ Nmm

$= 18.98$ kNm

b) Beam 150 mm X 550 mm



$A_{sv} = 9 \times \pi \times 10^2/4 = 706.86\ mm^2$

$s_v = 300\ mm$

$f_{yv} = 250\ N/mm^2$

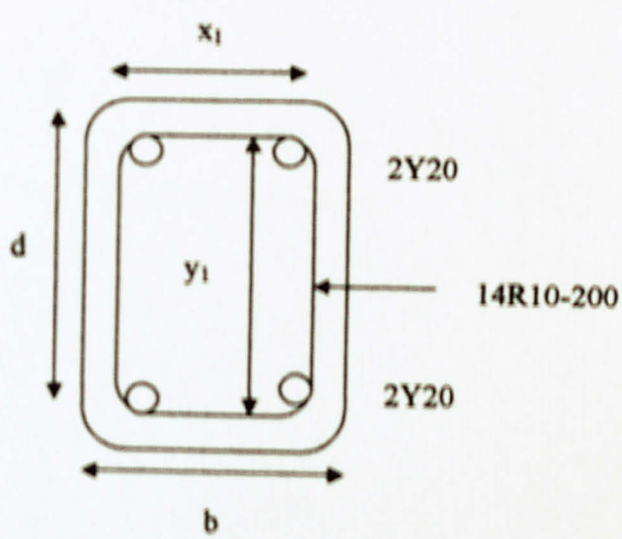
$T = (A_{sv}/s_v)x_1y_1(0.95f_{yv})x0.8$

$= (706.86/300)(480)(80)(0.95x250)x0.8$

$= 17.19 \times 10^6\ Nmm$

$= 17.19\ kNm$

c) Column 350 mm X 250 mm



$A_{sv} = 14 \times \pi \times 10^2/4 = 1099.56\ mm^2$

$s_v = 200\ mm$

$f_{yv} = 250\ N/mm^2$

$$
\begin{aligned}
T &= (A_{sv}/s_v)x_1y_1(0.95f_{yv})x0.8 \\
&= (1099.56/200)(180)(280)(0.95x250)x0.8 \\
&= 52.65 \text{ x } 10^6\ Nmm \\
&= 52.65\ kNm
\end{aligned}
$$

(III) Finite Element Analysis: Load Calculation

a) Beam 1

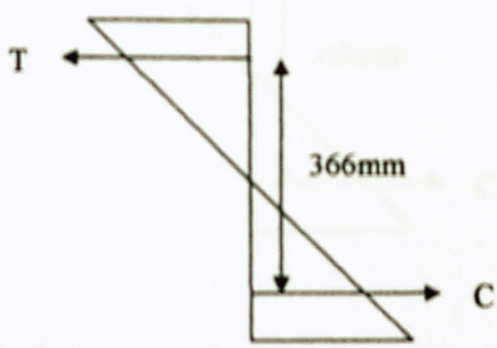
Node of nodes = 13

Axial Force = 0.777 kN

Axial force at each node = $\frac{0.777}{13}$

= 0.06 kN

Bending Moment = 26.676 kNm



To find resultant force,

M = T x d

T = M / d

T = 26.676 x 1000 kNmm / 366mm

= 72.885 kN

b) Beam 2

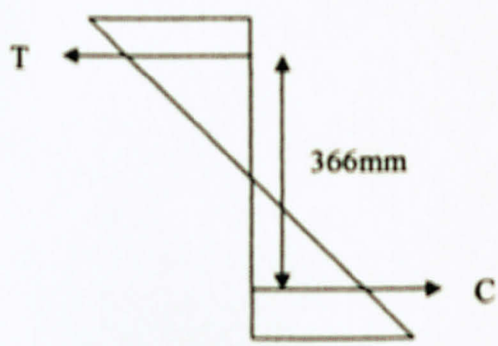
Node of nodes = 13

Axial Force = 12.646 kN

Axial force at each node = $\dfrac{12.646}{13}$

= 0.97 kN

Bending Moment = 112.221 kNm



To find resultant force,

M = T x d

T = M / d

T = 112.221 x 1000 kNmm / 366mm

= 306.61 kN