The Study of Beam - Column Joint of Typical School Building Structure in Malaysia

by

Rashidi bin Saemar

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

JUNE 2007

Universiti Teknologi PETRONAS
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CERTIFICATION OF APPROVAL

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A project dissertation submitted to the
Civil Engineering Programme
Universiti Teknologi PETRONAS
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BACHELOR OF ENGINEERING (Hons)
(CIVIL ENGINEERING)

Approved by,

____
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UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

June 2007

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.

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(RASHIDI BIN SAEMAR)

ABSTRACT

This report described the study and further analysis of the critical portion of frame structure – beam column joint of a school building in Malaysia, where it could be said 'neglected' in the design. The objective of seismic analysis is to assess the force and deformation demands and capacities on the structural system and its individual components. The projects is focused on identifying the possible outcome that seismic load could give onto the structure such as slippage of reinforcement bars in the joint. Preliminary studies and plan interpretation was done in the early stage. Several properties on structure members were identified and calculated such as the ultimate moment and shear capacity. The structure is analyzed using Equivalent Linear Static analysis. Equivalent static analysis is best suited for structures or individual frames with well balanced spans and uniformly distributed stiffness. The seismic load shall be assumed as an equivalent static horizontal force applied to individual frames. The horizontal force was applied at the vertical center of mass of the superstructure and distributed horizontally in proportion to the mass distribution. The analysis was done using the STAAD.Pro software to demonstrate the UBC Zone 2 loading. Variation of drifts, deformations, displacements and stresses were identified. However, none of the joint failed. By using more sophisticated software, evaluation of the bond stress between the reinforcement bars and the concrete in the joint region was done by using the pseudo – static forces. The results showed that both bars for beam and column for the joint had only experienced a low range of stresses, far below from the recommended values. Thus, a conclusion was made that the structure could actually withstand the loading.

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LIST OF ABBREVIATIONS

RC : Reinforced concrete

PBEE : Performance Based Earthquake Engineering

CAE : Computer – Aided Engineering

CAD : Computer – Aided Design

CHAPTER 1

INTRODUCTION

1.1 Background of Study

In the past, when a large scale earthquake hit Acheh, consequently produced a tsunami that washed away a major part of area, we here in Malaysia could also felt the impact of the vibration of the seismic wave from there. The city of Banda Aceh, Indonesia suffered catastrophic damage as a result of the impact of tsunami that struck on 26 December 2004, causes the extent of flooding, and widespread devastation to the city and surrounding areas. The great earthquake occurred at 00:58:50 (UTC), at 6:58 a.m. local time, on Sunday produced magnitude 9.0 events was located off the West coast of Northern Sumatra. This is the fourth largest earthquake in the world since 1900 and is the largest since the 1964 Prince William Sound, Alaska earthquake. The earthquake had a depth of 10 km.

The tsunami crossed into the Pacific Ocean and was recorded along the west coast of South and North America. Tsunamis also occurred on the coasts of Cocos Island, Kenya, Mauritius, Reunion and Seychelles. State liked Kuala Lumpur, Selangor and Pulau Pinang has experienced some of the effect of the seismic from our neighbour. Even though the distance was far, unfortunately we were also effected. In many years from now, we cannot predict what will happen thus an action should be consider – the building structure in Malaysia, especially beam-column joints where can be said as the critical part of a building. A typical school building structure is taken as the research subject in this course where in Malaysia the design approach doesn't take seismic load into consideration.

1.2 Problem Statement

In RC buildings, portions of columns that are common to beams at their intersections are called *beam-column* joints. Since their constituent materials have limited strengths, the joints have limited force carrying capacity. When forces larger than these are applied during earthquakes, joints are severely damaged. Repairing damaged joints is difficult, and so damaged must be avoided. Thus, beam-column joints must be designed to resist earthquakes. Here in Malaysia, we neglected all those critical considerations when designing a structure – a RC structure, where it cannot bare to resist the load transferred through the seismic wave.

1.3 Objective of Project

The objective of this project is to measure the behaviour of the beam column joint by using methods like, equivalent linear static, frame analysis and also using ANSYS simulation. Before that, partly initial calculations were done in order to obtain the loading of the school structure. After that, a more detail study was done regarding the bond stress of the concrete and the rebar inside the joint.

1.4 Scope of Work

The scope of work for this project is:

- Doing a preliminary research on s school structure regarding the structural frame; flexural capacity, shear capacity, torsion, development length, axial length and etc.
- To get familiar with the softwares to analyze the structural behaviour of the beam-column joints etc. frame analysis, ANSYS simulation, CATIA.

CHAPTER 2

LITERATURE REVIEW

Some studies are made to gain more understanding about the project. Generally the project involves three main elements which are the beam-column joints and the seismic studies/earthquake and also the analyze system that will be used. Journals/references are studied and which literally related to the project:

2.1 Behaviour of Joints

Under earthquake shaking, the beams adjoining a joint are subjected to moments in the same direction. Under these moments, the top bars in the beam-column joint are pulled in one direction and the bottom ones in the opposite direction. These forces are balanced by bond stress developed between concrete and steel in the joint region. If the column is not wide enough or if the strength of concrete is low, there is insufficient grip of concrete on the steel bars. In such circumstances, the bar slips inside the joint region, and beams loose their capacity to carry load (*C.V.R Murty*). However, among the various famous tests in beam - column joint components, they are not sufficient to describe the complete behaviour of the beam – column joint. For example, the measurement of the lateral expansion of joint is always been neglected whereas it is related to the axial forces arising in the beam, and also suggested that, experimentally, inelastic action of joints is sometimes significant for response of frame under earthquake loading.(*Fumio Kusuhara and Hitoshi Shiohara, 2006*).

2.2 Cross – sectional size and Bond Stress

The concrete in the joint will develops diagonal cracks if the cross – sectional size is insufficient. The action of the pull – push forces at top and bottom ends of the joint, undergoes some geometric distortion where the diagonal lengths of the joint elongate and compress.

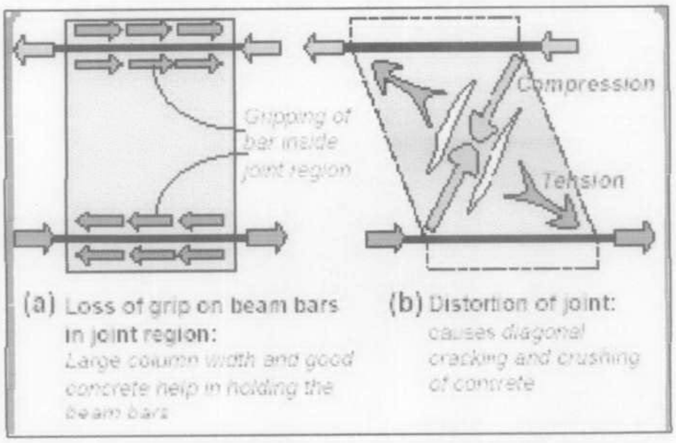

Figure 2.1: Pull – push forces in joints

Under the action of the moment in the joint, the bars in the joint will be pushed in one direction and the other one in the other direction (compression and tension). This is where bond stresses that are developed balanced the above forces. The bond stresses are developed between concrete and steel in the joint region. In this part, the size of the column plays an important role followed by the concrete strength (*C.V.R Murty*) – the grip between the bars and concrete will be lesser/smaller if the size and strength of the concrete is small thus causing the joint to loose its ability to handle/carry load.

2.3 Exterior Joints

One of the special attentions must be given to the external joints where development of nodes and bond transfer is needed. In contrast to the interior joints, no compression force acts to develop strut mechanism, and node is essential. By developing a 90^0 hook on the beams bars or an anchor plate-the hook must be turned down to create the desired node and the tail must be within the joint region. Beam bars on reverse loading are subjected to compressive loads, can be pushed through the back face of the joint if not resisted by bond stresses in the joint (*Robert E. Englekirk*). The $10d_b$ adjacent to the beam – column interface not be relied upon to develop the strength of the bars and that ties to be located so as to restrain the hooks when the bars are subjected to compression (*Paulay and Priestley*). There are varieties of measurements in tests of the beam – column joint, and as far as the performance based earthquake engineering (PBEE) is concerned, the prediction of the damage distribution in the structural system is very critical beside of stiffness and strength criteria. Hence the test data is very important to validate models used for the PBEE. Flexural deformation and shear deformation, the most popular joint components, likely to be broken into subcomponents. Some of the deformation subcomponents proposed by *Fumio Kusuhara and Hitoshi Shiohara* are the chord rotation of beam and column, rigid body rotations of beam, displacements of faces of beam – column joint panel, relative displacements between adjoined member ends, displacement of member ends and rotation of faces of the beam- column joint.

2.4 Expansion of Joint

According to *Fumio Kusuhara and Hitoshi Shiohara,* five half-scale reinforced concrete beam –column subassemblages with no traverse and slab were constructed. The specimens then were loaded with cyclic loading with increasing amplitudes and a constant axial load of 216kN was applied throughout the test. The amount of bars chosen was based on the required joint shear stress – 0.9 times as specified in the AIJ Guidelines and the longitudinal bars are passing through the joint. The type of expansions observed was horizontal and vertical directions. The both expansion increased as the story drift increased and the horizontal direction is expected to lead the axial force in beams and constrain force in the joint in actual frames.

2.5 Relative rotations between the joint faces

The relative rotation is defined by taking the differences between rotations of two adjoined faces and they are increment or decrement of angles between adjoined column and beam. When test conducted by *Fumio Kusuhara and Hitoshi Shiohara,* a pair of upper column and south beam is opening in positive loading while the other pair is closing, indicates that the stiffness of opening is smaller than closing. By that, the difference in stiffness causes asymmetric deformation of the joint panel.

A preliminary study was performed to investigate the structural behaviour of the storage rack due to ground motion by *Danny H. Chan* and *Raymond Yee.* By using finite element simulation – ABAQUS 2002, the dynamic analysis is implemented and broken down to two steps (a) natural frequency extraction (b) acceleration response spectrum analysis.

In the simulation, a proper specification of damping is included for obtaining accurate results since structural members have some levels o inherent capability to minimize vibration by damping.

Results obtained as follows:

- When applying the longitudinal direction (2 direction), a maximum displacement was recorded for about 163mm.
- it is seemed that localized stresses are occurred at the beam column connection due to geometric effect.
- The top level of the rack experienced the most displacement.
- When acceleration response spectrum applied in traverse direction, the displacement was much lower.
- The stress at the connection was reduced to one third from previous value.

2.6 Shear in RC Beam – Column Joint

A research has done and according to *J.F. Stanton, D.E. Lehman, S.G.Walker and C.M. Yeargin,* almost all old building didn't have shear reinforcement in the joint region. The primary variables were the joint stress demand and the load history. Two different shear stress demand and four different imposed displacement histories were used. In the experiment, all the beams and columns were the same size and were concentric. The reinforcement used was with nominal strength of 60 ksi and the concrete strength of 5000 psi. The axial load assigned was 0.1fc'Ag onto the column.

The all joints were suffered from serious joint shear damage and had loss great quantities of concrete and column bars are easily seen. With drift about 1.5%, the result indicates that low damage and repairable. Rectification can be made by installing a stiff shear wall to restrict the 1.5% drift to occur. The shear strength envelope of the joint peak at a drift of approximately 1.5% in all cases. When reaching the peak of the shear stress, it suddenly dropped gradually to an approximate value.

- <u>Analytical studies</u>

  According to *John W. Wallace* and *Thomas H. Kang*, punching failure is predicted to occur at a displacement of 0.01m or an inter story drift ratio of 0.35% when tested with static analysis without shear reinforcement. However when the joint is equipped with shear reinforcement, the punching failure was delayed and occurred at a displacement of 0.06m or an inter story drift ratio of 1.5%.

Figure 2.2: Punching Failure and flexural yielding.

2.7 Bond Strength / Stress

The bond action between the concrete and the rebars is composed mainly of three components, namely the chemical adhesion between the concrete and the rebars, the friction due to the surface roughness of the rebars, and the mechanical interlock of the rebars due to the ribbed surface profile. When smooth rebars are used in RC beams, the first two components will contribute to the bond strength of the bars. In contrast, the bond strength of deformed bars in beams would involve all of the aforementioned components and, more importantly, the last factor will form the major portion. In order to ensure that the rebars can be loaded to their yield points, a minimum length of embedment is usually required and this is commonly named as the development length.

The calculated bond stresses that been stated by Sezen.H and Moehle.J are normalized by fc′. For the twelve columns that has been tested and considered, the average bond stress is 0.95 $\sqrt{fc'}$ MPa (11.4 fc′ psi), and the standard deviation is 0.2 $\sqrt{fc'}$ MPa (2.5 fc′ psi). In this study, a uniform bond stress of 1.0 $\sqrt{fc'}$ MPa is assumed in the elastic range. In the portion of the reinforcing bar over which the yield strain is exceeded, a uniform bond stress of 0.5 $\sqrt{fc'}$ MPa is used as suggested. Furthermore, a study by Jingyao Cao and D.D.L. Chung also proved that the bond failure did not occur up to 400 cycles of shear loading, at which testing was stopped. The bond strength before any cyclic shear was 6.68 ± 0.24 MPa; that after the abrupt increase was 5.54 ± 0.43 MPa. Thus, even though the abrupt increase did not cause visually observable damage, bond degradation occurred.

CHAPTER 3

METHODOLOGY

This research will be based on the following project design. This methodology was design to have a basic view of the project and a preparation for this research. Figure 3.1 below shows the design that will be followed in carrying out this project.

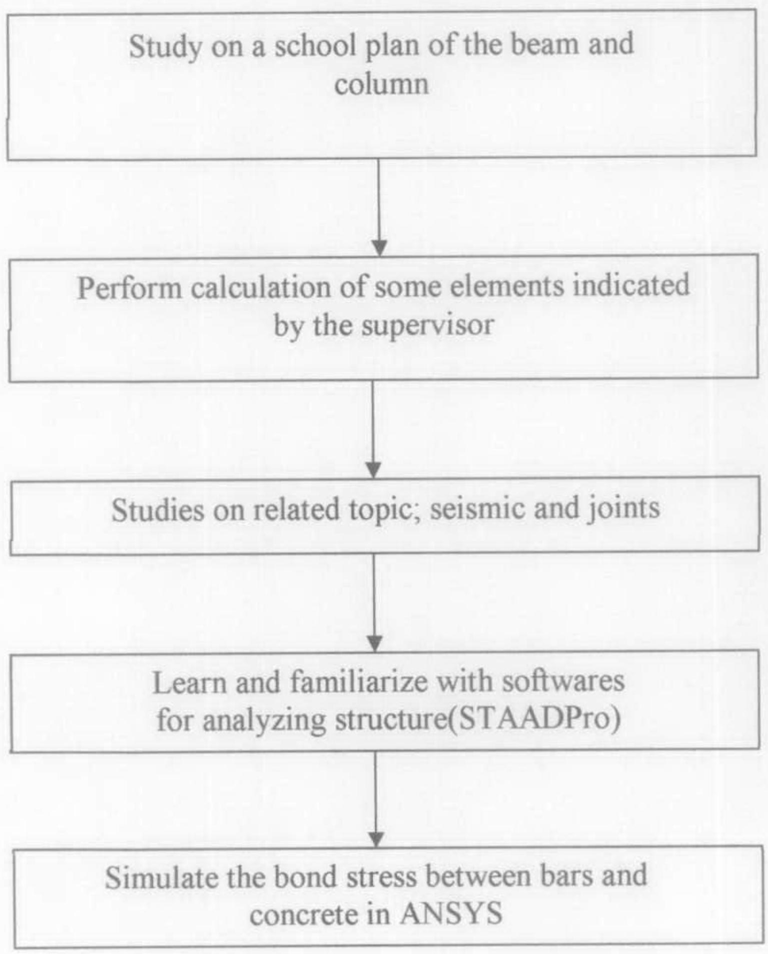

Figure 3.1: Project Design Flow Chart

The details methods for the project will be identified after this stage. Researches will be done from time to time so that better understanding can be reached. High-end engineering software such as STAAD PRO, CATIA and ANSYS WORKBENCH are needed in performing the analysis.

3.1 Plan Interpretation and Calculations

In order to understand the behaviour of a structure, especially when analyzing it, some of important elements must be familiarized. For the early stage of the project, some of the elements are considered (British Standard code):

Flexure/Bending Capacity: $M_u = 0.156 f_{cu} b d^2$

$M_f = 0.45 f_{cu}\, b h_f (d - h_f/2)$

Shear Capacity : $Vu = \text{Ø}Vc + \text{Ø}Vs$

$Vc = (\sqrt{f'c}/6)\, b_w d$

$Vs = \tfrac{1}{3}\sqrt{f'c}\, b_w\, d$

Torsional Capacity : $T_u \leq \text{Ø}\, T_n$ *where* $T_n = T_c + T_s$

A standard drawing/plan of a typical school structure in Malaysia is been studied and used as a research material in this course. A building of three stories building of school is been analyzed its structural frame, especially the beam – column joint part. The drawing is designed based on the BS: 8110, done by the JKR and the building consisting of three type of frame; K1, K2 & K3 and lastly K4, from the side angle. Overall, the frames consist of approximately 30 types/sizes of beams and column.

Formulas involved in calculation of beam and column capacities using ACI Code due to its reliability and accuracy rather than the BS. From the plan, it shows that the concrete covers used are 25mm for beams, 40mm for ground beams and 40mm for columns. The steel yield strength fy is 460N/mm2 and the concrete strength, fc' is 30N/mm2. For the beams, the important perimeters that are calculated and taken into consideration are the Ultimate Moment Strength, Mn, torsion, Tu, shear capacity, Vu and the development length, l_d / d_b . As for the column, the axial load capacity is studied.

The following perimeters that are calculated as below:

3.1.1 Beams

a. Ultimate Moment Strength, Mn

- $M_n = T\,(d\text{-}a/2)$ (unit: kN.m)
    - Tension, $T=A_s.f_y$
    - Compression, $C=0.85f'_c.a.b$
    - a is obtained from equation when T=C (equilibrium)

- T- Beams

  $\text{Ø}\,M_n = \text{Ø}T\,z$ (unit: kN.m)
    - Tension, $T=A_s f_y$
    - Area of concrete in compression, $A_c=T/0.85f'_c$
    - $a=Ac/b_w$; b_w is the effective width
    - $z = d - a/2$
    - Reduction factor, Ø= 0.90

b. Shear, V_u

- $Vu= \text{Ø}V_c + \text{Ø}V_s$ (unit: kN)
    - $Vc= (\sqrt{f'c}/6)\, b_w d$ (In SI)
    - $Vs= \frac{1}{3} \sqrt{f'c}\, b_w\, d$ (In SI)
    - $Av = \dfrac{\text{Ø}\sqrt{f'c}}{12} \quad \dfrac{A^2 cp}{p_{cp}}$
    - Ø = 0.75

c. Torsion, T_u

- $T_u = \dfrac{\text{Ø}\sqrt{f'c}}{12}\,(A^2{}_{cp}/\,p_{cp})$ (unit : kN.m)
    - A_{cp} = area enclosed by outside perimeter concrete cross-section
    - p_{cp} = perimeter (outside) of beam cross-section

d. Development Length

It can be defined as the minimum length of embedment of bars that is necessary to permit them to be stressed to their yield point plus some extra distance to ensure member toughness. The bar stresses must be transferred to the concrete by bond between the steel and the concrete before the bars can be cut off.

- $\dfrac{l_d}{d_b} = \dfrac{9}{10} \dfrac{f_y \acute{\alpha}\beta\gamma\lambda}{\sqrt{f'c}\,\dfrac{(c+K_{tr})}{d_b}}$ (unit: diameters)

  - $\acute{\alpha}$ = reinforcement location factor (table 7.1; ACI 12.2.4)
  - β = coating factor (table 7.1; ACI 12.2.4)
  - γ = reinforcement size factor (table 7.1; ACO 12.2.4)
  - C = lightweight aggregate concrete factor (table 7.1; ACI 12.2.4)

  - K_{tr} = transverse reinforcement index;
  - K_{tr} = 0, permitted by ACI Code in Section 12.24
  - Computed $K_{tr} = \dfrac{A_{tr}\, f_{yt}}{10sn}$ (In SI units)
    - A_{tr} = total cross-section area of all transverse reinforcement having the center-to-center spacing s and a yield strength f_{yt}
    - n =number of bars being developed along the lane of splitting

3.1.2 Columns

1. Axial Load Capacity, P_u
   - $P_u = ØP_n = 0.80Ø\ [\ 0.85f'_c (A_g - A_{st}) + f_y A_{st}]$ (unit: kN)

2. Shear
   - $Vc = 1 + \dfrac{N_u}{14A_g} * \dfrac{\sqrt{f'c}}{6} * b_w * d$ (unit: kN)

ά (alpha) = reinforcement location factor	
Horizontal reinforcement so placed that more than 12 in. of fresh concrete is cast in the member below the development length or slice	1.3
Other reinforcement	1.0
β (beta) = coating factor	
Epoxy-coated bars or wires with cover less than $3d_b$ or clear spacing less than 6 d_b	1.5
All other epoxy-coated bars or wires	1.2
Uncoated reinforcement	1.0
However, the product of ά β need not be taken greater than 1.7	
γ (gamma) = reinforcement size factor	
No. 6 and smaller bars and deformed wires	0.8
No. 7 and larger bars	1.0
In SI units,	
No.19 and smaller bars and deformed wires	0.8
No. 22 and larger bars	1.0
γ (lambda)= lightweight aggregate concrete factor	
When lightweight aggregate concrete is used	1.3
However, when fct is specified, γ shall be permitted to be taken as 6.7 $\sqrt{f'c}$ / fct Its $\sqrt{f'c}$/ 1.8fct in SI, but not less than	1.0
When normal weight concrete is used	1.0
c = spacing or cover dimension	
Use the smaller of either the distance from center of the bar or wire to the nearest concrete surface, or one-half the center-to-center spacing of the bars or wires being developed	

Table 3.1: Factors for Use in the Expressions for Determining Required Development Lengths for Deformed Bars and Deformed Wires in Tension (ACI 12.2.4)

3.2 STAAD.Pro Analysis

A portal frame of the school building structure, figure 3.7, has been modeled by using the STAAD.Pro 2004, to further investigate the behaviour of the beam-column joint analytically. The software is used to generate model which can then be analyzed using the STAAD engine. After the analysis and design is completed, the Graphical User Interface (figure 3.1) is used to view the results graphically. The type of structure that is used to analyze the building is the *Space* structure which is three dimensional frame structure with loads applied in any plane. The model is generated using the beam element by connecting lines to nodes constructed.

Then the building was analyzed using Second Order Static analysis- P-Delta Analysis. Structures subjected to lateral loads often experience secondary forces/moments due to the movement of the point of application of vertical loads. This effect is called P-Δ effect- calculated for frames members and plate element only, not included solid element. The analysis is restricted to structures where members and plate elements carry the vertical load from one structure level to the next.

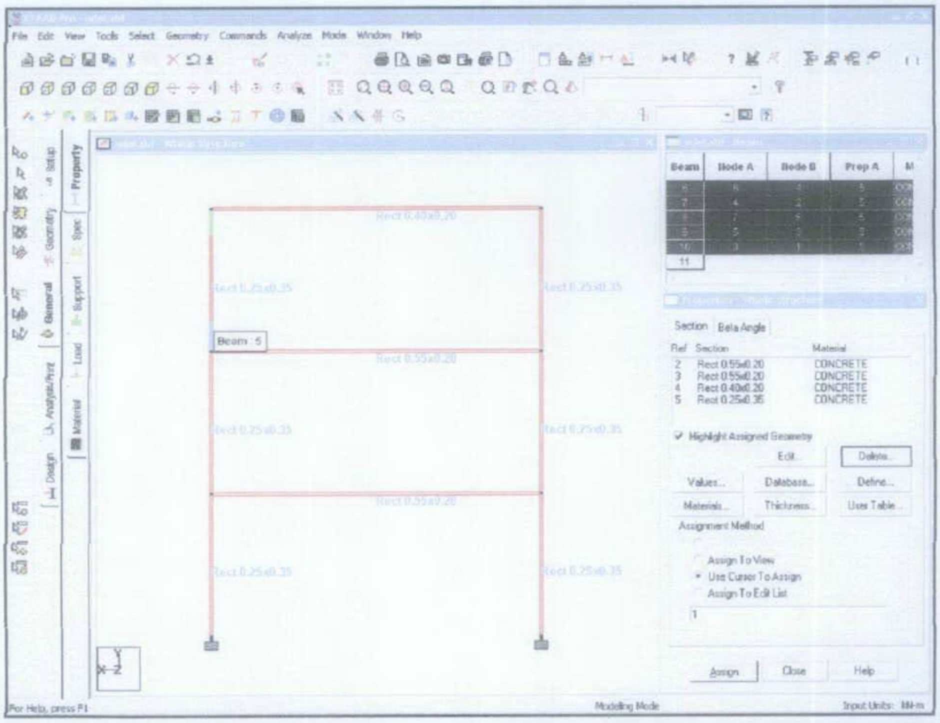

Figure 3.2: STAAD.Pro Interface

3.3 CATIA Modeling

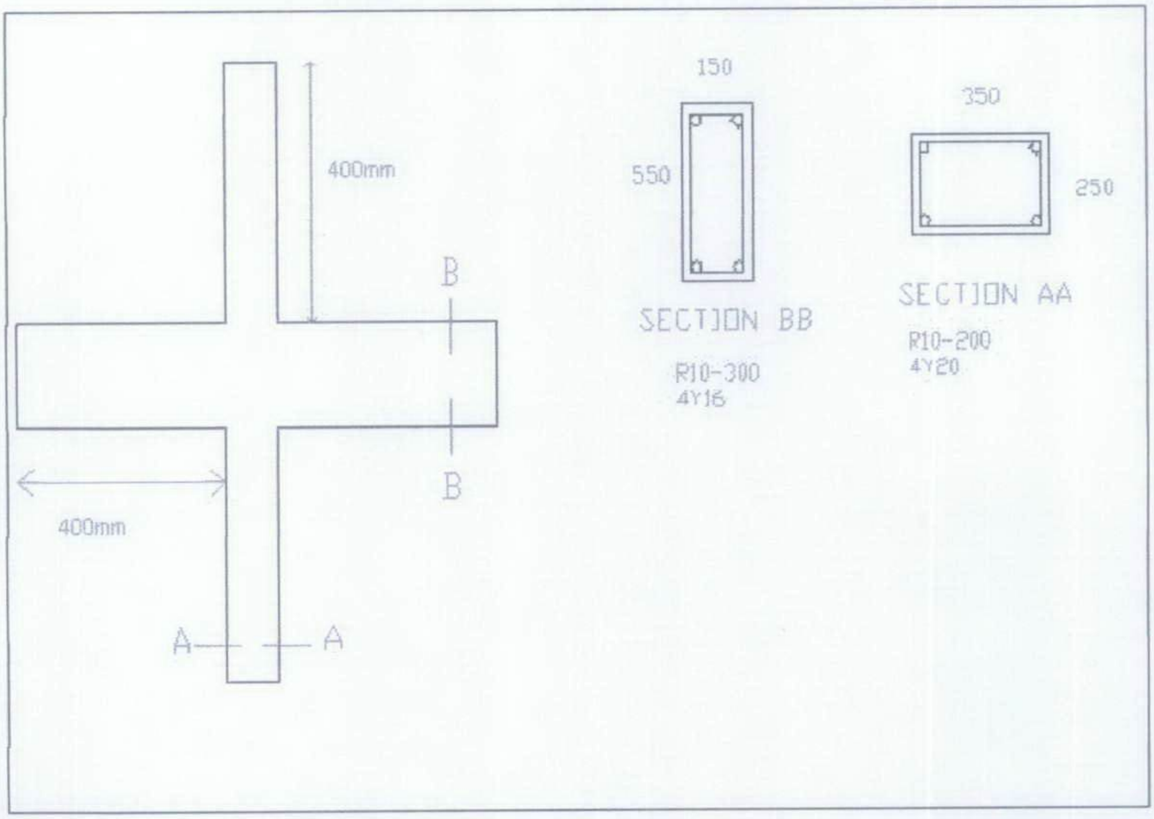

Figure 3.3: Beam-column joint dimension.

The figure above is the dimensions and cross-sections one of the joint in the school building frame. Likely the joint will be taken and analyzed to study the behaviour of joint under seismic load/condition. The joint was originally taken from the STAAD simulation and be considered as the most joint in the building

The beam is sized 550mm x 150mm and located at the second floor of the building. The traverse bars are using the R10 – 300 and 4Y16 as the longitudinal bar. The column is 350mm x 250mm and using R10 – 200 as the traverse while 4Y20 as the main bar.

In order to continue the study of behaviour of the joint, a model representing the interior joint is made using the CATIA. The model was on 3D interface so that the finite element could be used. The model was 525mm and 675mm in lengths from the center joint and no traverse bars were included.

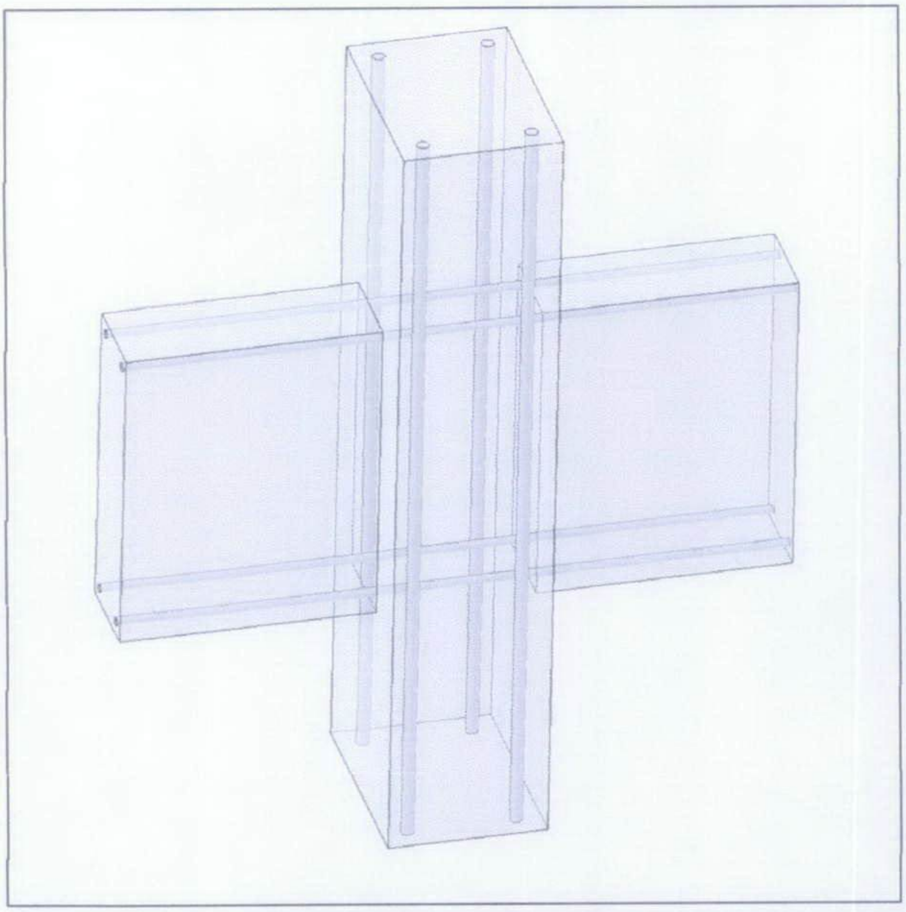

Figure 3.4: CATIA modeling 1

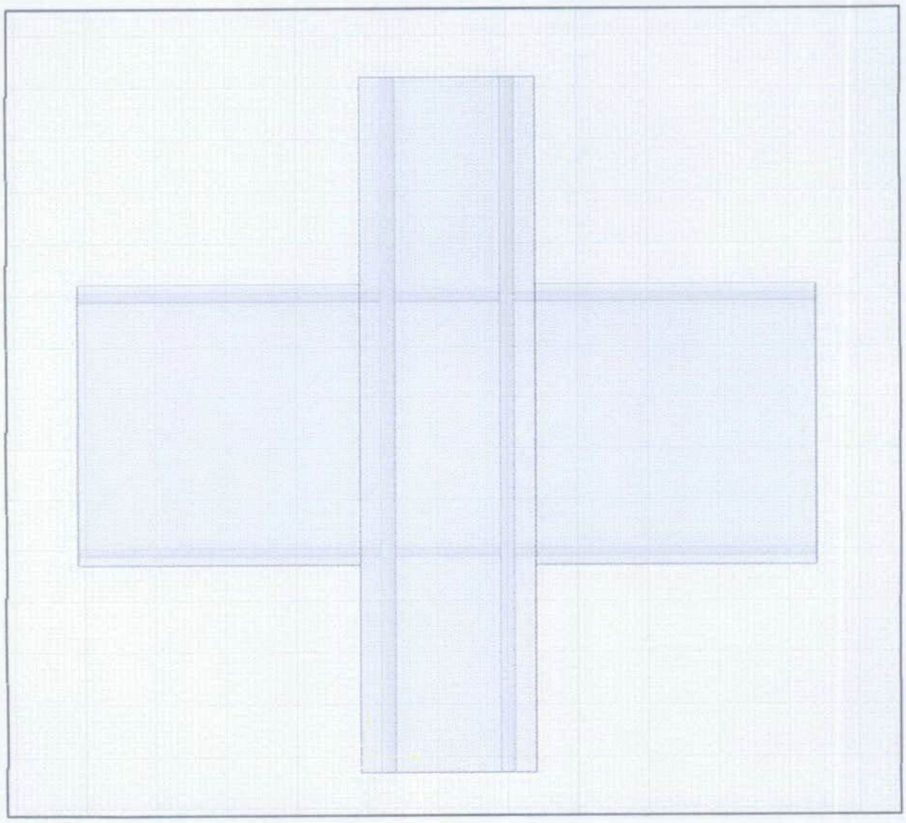

Figure 3.5: CATIA modeling 2

3.4 ANSYS Simulation

The last step for the project was to simulate the behaviour of the beam – column joint in the ANSYS Workbench software programme. The ANSYS CAE software programme was used in conjunction with 3D CAD solid geometry to simulate the behaviour of beam – column joint under the structural loading conditions. The results of a simulation provide insight into how the joint may perform and how the design might be improved.

The model was obtained from the design modeler assembly which was imported from CATIA. With the body of 525mm and 675mm in both lengths, the joint weighing 443.43 kg and has a total volume of $0.19m^3$. The joint was formed by two elements which were concrete and structural steel (rebar). The simulation was done at a uniform temperature of 28^0C according to Malaysia's temperature range.

3.4.1 Structural loading

Name	Type	Magnitude
upper column	axial	-1.04 KN
	shear	-30.5KN
	moment	-0.814KNm
lower column	axial	+1.9KN
	shear	+98.3KN
	moment	+1.8KNm
right beam	shear	+19.9KN
	moment	+0.69KNm
left column	shear	-23KN
	moment	+3.73KNm

Table 3.2: Structural loadings

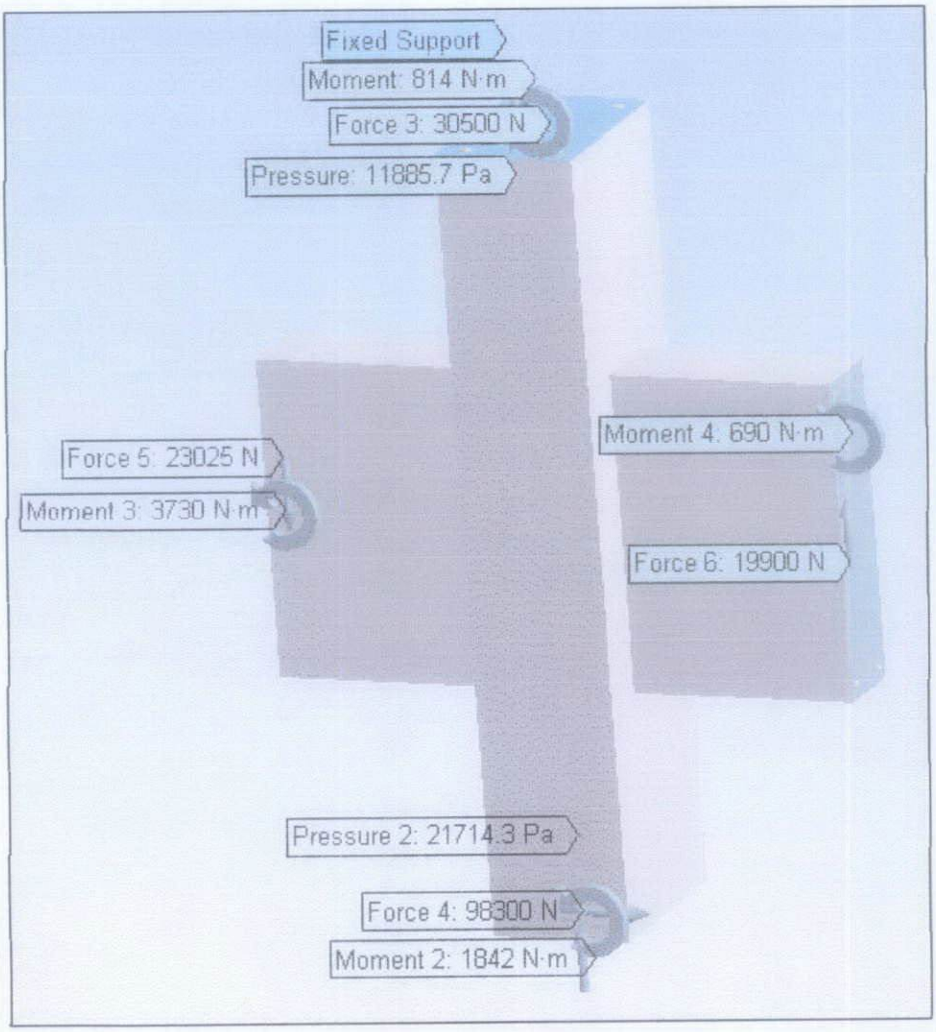

Figure 3.6: Loadings

The figure above shows the placement of the loadings in the ANSYS. As pictured, all the forces and moments were taken from the STAAD results. The joint was analyzed using the pseudo-static forces in order to get the nearest actual results of joint behaviour due to seismic loading. The loadings taken from the STAAD were under seismic loading.

There were 10 loadings (pseudo-static) putted on the model. All the loadings were putted on the 4 surface planes of the model, as shown from figure above. All the forces and moments were defined on the planes as component; x,y,z direction, except for the axial forces in column, defined as vector and putted as pressure.

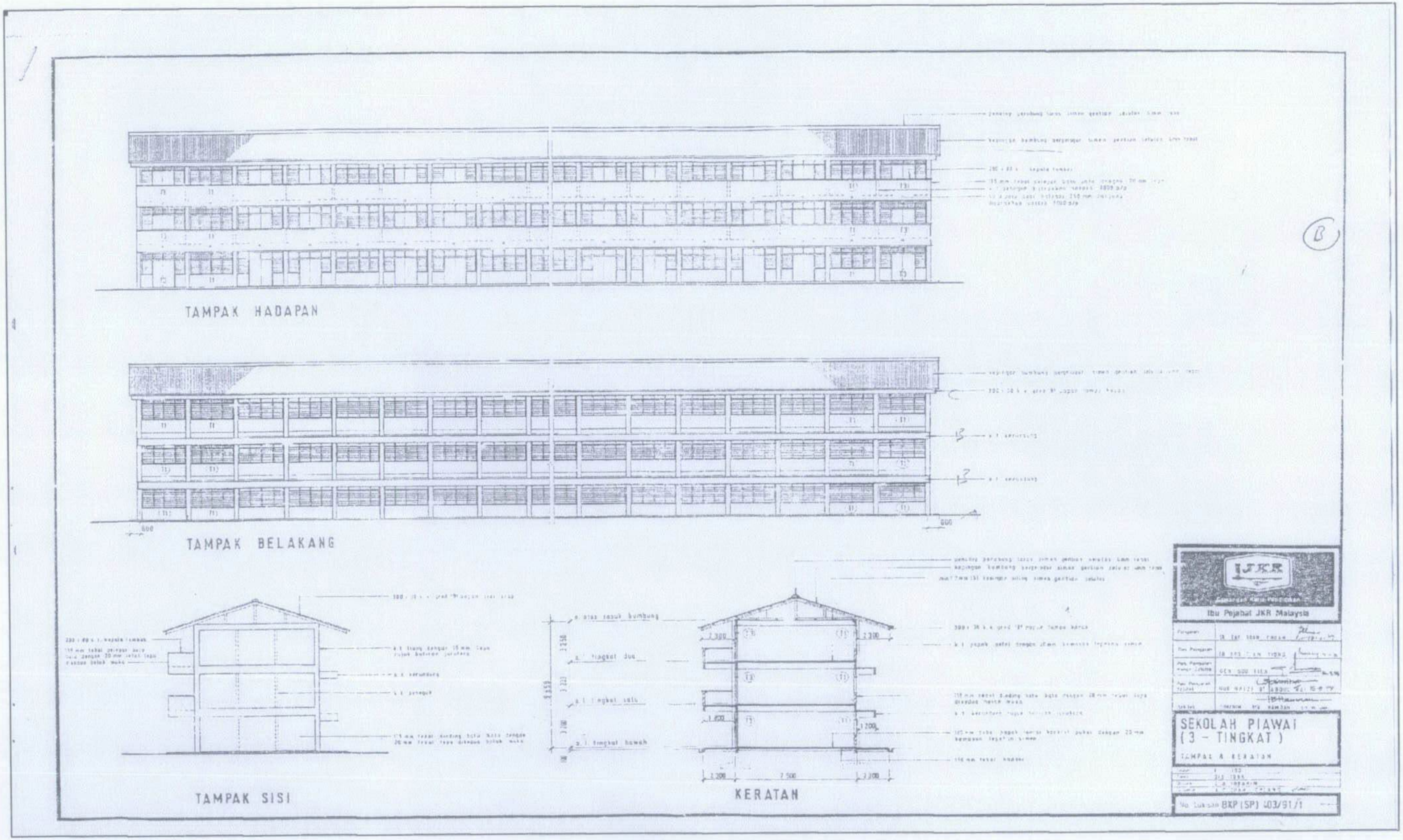

Figure 3.7: School Building Sideview / Frontview

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Elements Calculation

Some of the beams that have been calculated its capacity selected from the entire school building frame type, two beams from each frame. The value as shown in tables below:

Ultimate Moment Strength

Beams	b(mm)	d(mm)	a	fy(N/mm2)	fc'(N/mm2)	As(mm)	Mn(N.mm)
3-3(k1)	200	365	128.08	460	30	1420	196587584
10-10(k1)	200	400	153.69	460	30	1704	253300201
5-5(k3&k2)	200	365	128.08	460	30	1420	196587584
12-12(k3&k2)	200	400	153.69	460	30	1704	253300201
5-5(k4)	200	365	128.08	460	30	1420	196587584
12-12(k4)	200	400	153.69	460	30	1704	253300201

Shear Capacity

Beams	fc'(N/mm2)	bw(mm)	d(mm)	Vc(N)	Vs(N)	Φ	Vu(N)
3-3(k1)	30	200	365	66639.58	133279.16	0.75	149939.05
10-10(k1)	30	200	400	73029.67	146059.35	0.75	164316.77
5-5(k3&k2)	30	200	365	66639.58	133279.16	0.75	149939.05
12-12(k3&k2)	30	200	400	73029.67	146059.35	0.75	164316.77
5-5(k4)	30	200	365	66639.58	133279.16	0.75	149939.05
12-12(k4)	30	200	400	73029.67	146059.35	0.75	164316.77

Torsion Capacity

Beams	fc'(N/mm2)	Pcp(mm)	Φ	Acp(mm2)	Tu(N.mm)
3-3(k1)	30	1200	0.75	80000	1825742
10-10(k1)	30	1300	0.75	90000	2132958
5-5(k3&k2)	30	1200	0.75	80000	1825742
12-12(k3&k2)	30	1300	0.75	90000	2132958
5-5(k4)	30	1200	0.75	80000	1825742
12-12(k4)	30	1300	0.75	90000	2132958

Development length

Beams	fy(N/mm2)	fc'(N/mm2)	Ktr	c(mm)	db	area of bar(mm2)	ld/db(diameters)
3-3(k1)	460	30	0	25	42.52	1420	129
10-10(k1)	460	30	0	40	46.58	1704	88
5-5(k3&k2)	460	30	0	25	42.52	1420	129
12-12(k3&k2)	460	30	0	40	46.58	1704	88
5-5(k4)	460	30	0	25	42.52	1420	129
12-12(k4)	460	30	0	40	46.58	1704	88

Tables 4.1: Beam and column properties (theoretically)

4.2 Portal Frame Analysis

A simple 3 – story frame with the height of 9.45m and width of 3.0m for each bay is constructed. The frame is supported by fixed support as its foundation and a total length of 54.0m. As seen from Figure 4.1a, the green line indicating the beams and columns connected from node to node in the STAAD. The frame is constructed using the beam element in the software and all the actual properties such as material used and dimensions are based from the drawing/plan. By using the beam element, the reinforcements inside the beam/column were able to be generated, thus giving a much better analysis and results.

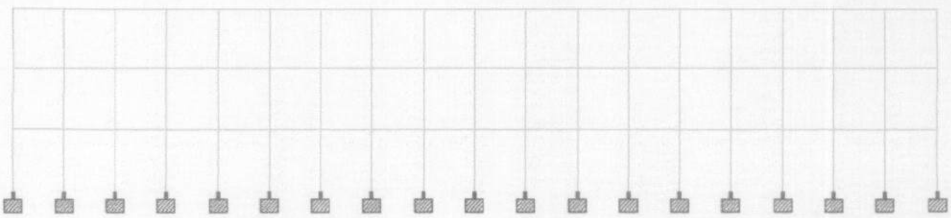

Figure 4.1a: Portal Frame Analysis

Loads calculated for portal frame analysis (refer figure 4.17):

- Ordinary slab thickness 120 mm
- One way slab: ly/lx > 2 → (0.5* n*lx)
- Two way slab: ly/lx < 2 → (n*lx)/3 or [n*lx*[3-(lx/ly)^2)]]/6

1st and 2nd floor

Beam A: (0.5 x 6.6 x 3) Kn/m → from slab

= (9.9 + selfweight + brickwall)

= 20.284 KN/m

Beam B (secondary beam):

= load from slab + selfweight

= 7.9 KN/m

Beam C (secondary beam):

= load from slab1 + selfweight + load from slab2

= 7.418 KN/m

Beam D:

= load from slab + selfweight + brickwall

=11.596 KN/m

Roof level:

Beam A = beam selfweight = 1.92 KN/m

Beam B = beam selfweight = 2.16 KN/m

Dead load
1-uniform member load in global Y direction of -11.596 KN/m (1st and 2nd floor) 2-uniform member load in global Y direction of -2.22 KN/m (roof floor) 3-selfweight in Y direction of factor -1 4-point load in Y direction of -2.5KN 5-point load in Y direction of -1.25KN 6-UBC load in X direction with factor of 0.75
Live load
1-uniform member load in global Y direction of -1.912KN/m (only at 1st and 2nd floor)

Table 4.2: Types of loading analyzed with STAAD

Two loads are applied onto the frame structure analysis; dead and live load (Table 4.2). For the analysis, the seismic load is defined according UBC 1994 and using the Zone 2 properties. The values that have been used were loading with factor of 0.75 in X direction, importance factor of 1, soil factor of 1.5, CT of 0.032, Rw in X direction of 9 and Rw in Z direction o 9. The seismic load is put under the dead load and analyzed together with the live load as a combination load using the 1.4 (dead load) + 1.6 (live load) factors.

The STAAD analyzed the structure by using 3 stages of load; (1) dead load (2) live load and (dead + live load). Referred to the Table 4.3, the node 40 has experienced a drift/displacement in X direction for 1.395mm and Y direction for -0.476mm. The resultant drift is around 1.48mm and experienced no rotation. From figure 4.1b, the red line shows the displacement of the frame which has been scaled due to the small drift value. The red dotted joint in the figure is the node 40, which also been taken to do further analysis which will be explain later. The reason node 40 is chose because it is one of the critical joints after the analysis.

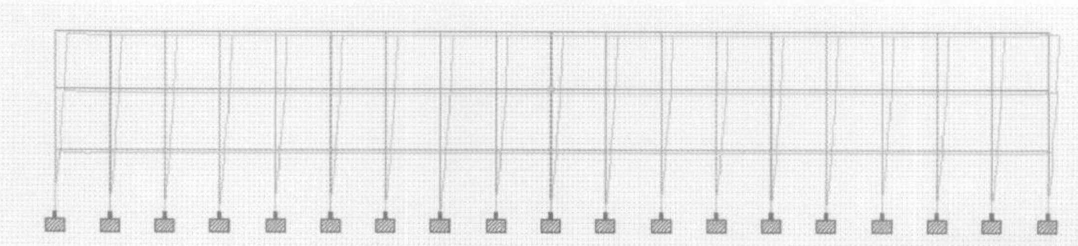

Figure 4.1b: Displacement/drift after seismic loading

Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
40	1 DEAD LOA	0.997	-0.307	0.000	1.043	0.000	0.000	-0.000
	2 LIVE LOAD	-0.000	-0.029	0.000	0.029	0.000	0.000	0.000
	3 DL+LL	1.395	-0.476	0.000	1.475	0.000	0.000	-0.000
41	1 DEAD LOA	0.527	-0.194	0.000	0.562	0.000	0.000	-0.000
	2 LIVE LOAD	-0.000	-0.019	0.000	0.019	0.000	0.000	0.000
	3 DL+LL	0.738	-0.303	0.000	0.798	0.000	0.000	-0.000

Table 4.3: Node/Joint drifts (displacement)

Beam	L/C	Node	Fx N	Fy N	Fz N	Mx kNm	My kNm	Mz kNm
53	1 DEAD LOA	38	198.680	19195.400	0.000	0.000	0.000	8.482
		40	-198.680	21424.118	0.000	0.000	0.000	-11.825
	2 LIVE LOAD	38	49.875	2867.718	0.000	0.000	0.000	1.433
		40	-49.875	2868.285	0.000	0.000	0.000	-1.434
	3 DL+LL	38	357.952	31461.906	0.000	0.000	0.000	14.168
		40	-357.952	34583.019	0.000	0.000	0.000	-18.850
54	1 DEAD LOA	40	207.101	19200.275	0.000	0.000	0.000	8.489
		42	-207.101	21419.220	0.000	0.000	0.000	-11.818
	2 LIVE LOAD	40	49.875	2868.284	0.000	0.000	0.000	1.434
		42	-49.875	2867.717	0.000	0.000	0.000	-1.433
	3 DL+LL	40	369.742	31469.641	0.000	0.000	0.000	14.160

Table 4.4a: Beam forces (No.53&54).

Beams	fc'(N/mm2)	bw(mm)	d(mm)	Vc(N)	Vs(N)	Φ	Vu(N)
54&53	30	150	500	68465.32	136930.64	0.75	154046.97

Table 4.4b: Beam forces (No.53&54) by theoretical calculation.

As referred to the values obtain from STAAD, the shear loadings for the frame analyzed, using the portal frame method, are far below the theoretical value calculated manually using the ACI standard earlier. The values from the beams 53 and 54 (highlighted above from Table 4.4a), are 34.6 KN and 31.5 KN. However, the value of ultimate shear loading for respective beams is around 154 KN (Table 4.4b). The difference is too big and thus indicating that all the loadings onto the frame are held sufficiently by the beams.

The shear values obtain from the table are taken for beam 53 and 54 and axial load values for beam 108 and 27 (Table 4.5), and used to analyzed the joint element (node 40). Not to forget the bending moment.

Beam	L/C	Node	Fx N	Fy N	Fz N	Mx kNm	My kNm	Mz kNm
108	1 DEAD LOA	79	15524.647	740.007	0.000	0.000	0.000	1.176
		40	-21812.651	-740.007	0.000	0.000	0.000	1.081
	2 LIVE LOAD	79	0.254	0.000	0.000	0.000	0.000	0.000
		40	-0.254	-0.000	0.000	0.000	0.000	0.000
	3 DL+LL	79	21734.911	1036.010	0.000	0.000	0.000	1.646
		40	-30538.115	-1036.010	0.000	0.000	0.000	1.514
109	1 DEAD LOA	80	15524.663	734.172	0.000	0.000	0.000	1.167

Table 4.5a: Column axial force (No.108)

Beam	L/C	Node	Fx N	Fy N	Fz N	Mx kNm	My kNm	Mz kNm
27	1 DEAD LOA	40	63687.046	1391.475	0.000	0.000	0.000	2.255
		41	-70284.294	-1391.475	0.000	0.000	0.000	2.198
	2 LIVE LOAD	40	5736.822	0.000	0.000	0.000	0.000	0.000
		41	-5736.822	-0.000	0.000	0.000	0.000	0.000
	3 DL+LL	40	98340.775	1948.065	0.000	0.000	0.000	3.156
		41	-107 57692E	-1948.065	0.000	0.000	0.000	3.077

Table 4.5b: Column axial force (No.27)

4.3 Joint Analysis Using STAAD

In Malaysia, RC structure is more preferable than the steel structure due to many reasons, like cost. Additionally, the mind set of local authorities is focused more on the RC, but in this recent years the steel industries are began to develop. The RC joints in structure often build without shear reinforcement/ traverse reinforcement. The analysis describes a physical test that was conducted on graphical beam – column that represents the condition typical of school building using the STAAD taken from node 40.

The figure 4.2 below shows the beam - column assemblage, generated by the STAAD in 3D model. Each beam has the length of 1.5m from the center joint and the column is 1.6m each, which is half of the original length in the portal frame earlier. This is considering portal frame analysis is based on the assumption of zero moment at mid span of beam and mid height of column, thus these members can be used with hinges at zero moment locations. The cross section size of beam 53 and 54 is 150mm x 500mm and the column 108 and 27 is 250mm x 350mm.

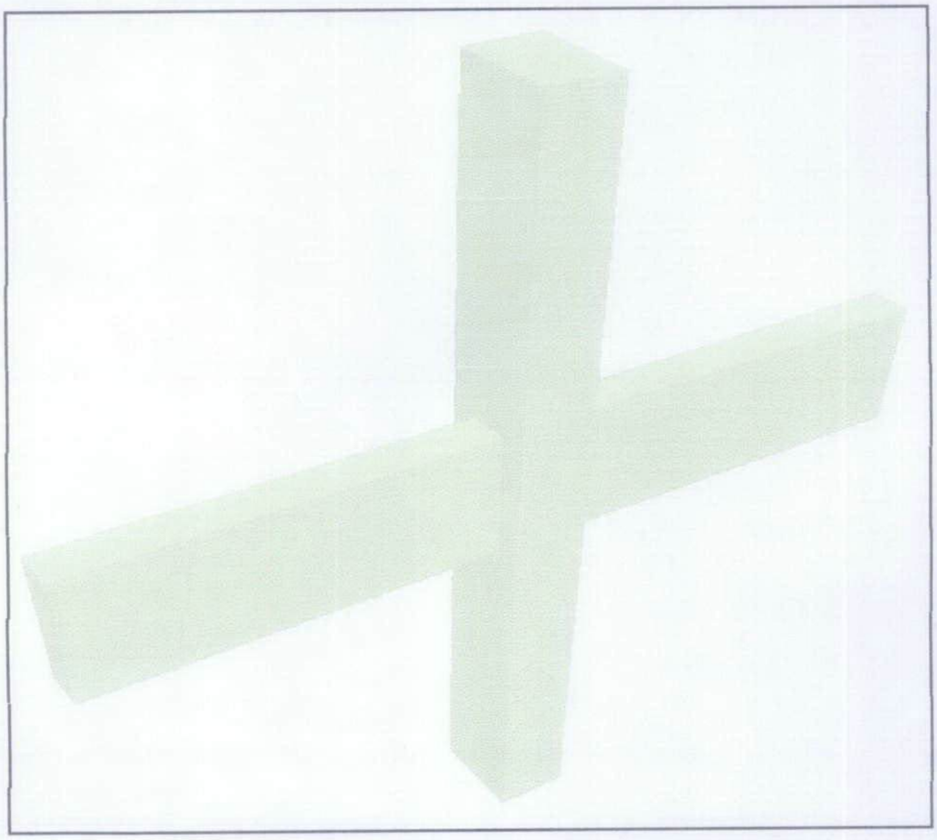

Figure 4.2: 3D generated model of joint/node 40

Values of shear, bending moment and axial load (Table 4.4 and 4.5) obtain from the frame analysis earlier is converted and manipulated to be distribute onto the joint assemblage. At every each end of the members, a pin support is located thus carries no moment loading. For beams, point loads are assigned as shear load at four different distances – 0m, 0.375m, 0.75m and 1.5m (all based from center joint). Point loads act as axial loads are assigned directly onto column 108 and 27, at the support. The bending moment loads are assigned at the center joint only since pin support doesn't have moment. The distribution of loads is pictured in figure 4.3 below.

***Attention: Beam 53=Beam 10, Beam 54=Beam 11, Column 108=Beam/Column 8, Column 27=Beam/Column 9**.

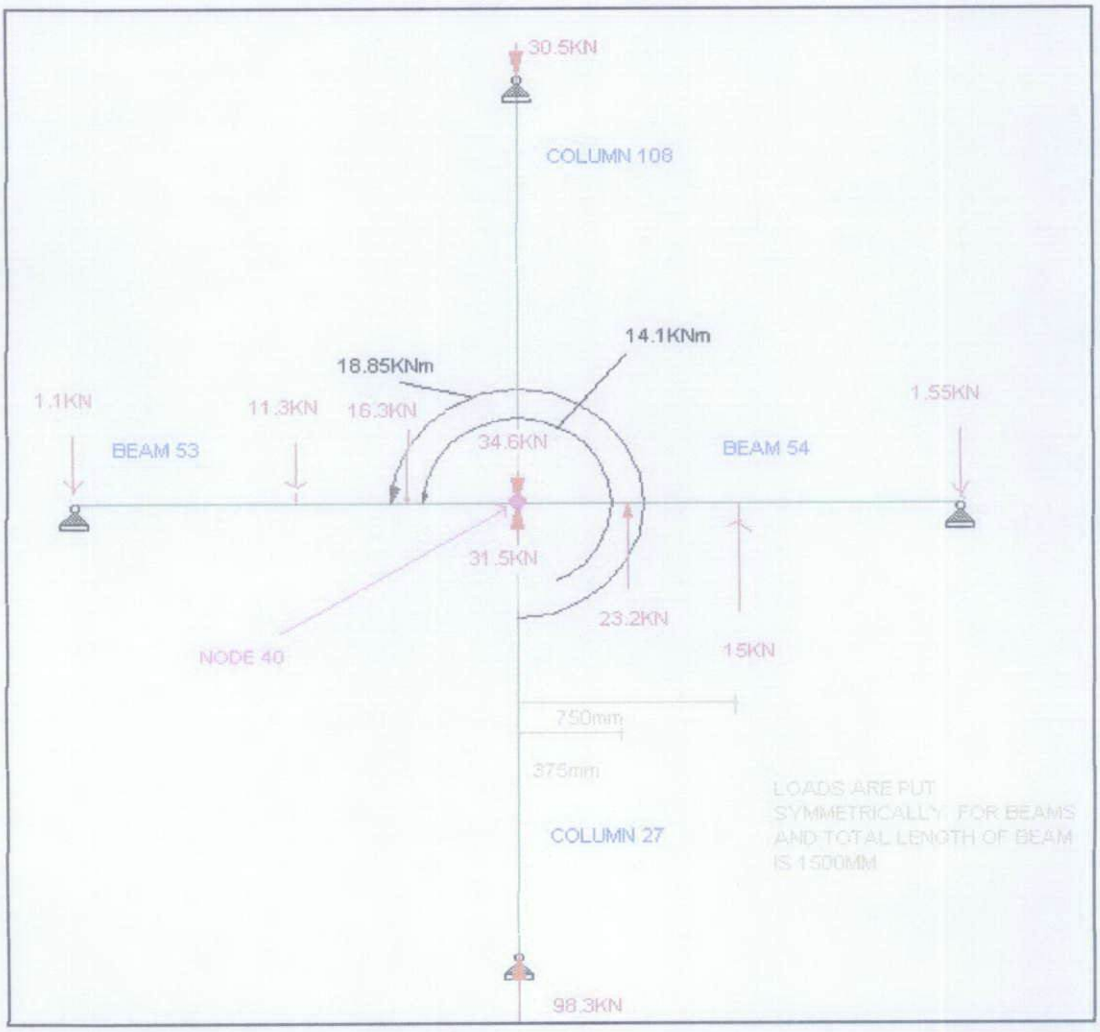

Figure 4.3: Loads distribution on the joint/node 40

After the analysis has finished, the results obtained and viewed from the post – processing mode. In that mode, all the results such as bending moment, shear, axial and stress can be shown in the most presentable way. The STAAD is equipped with animation engine processing tool which a property like deflection can be animated (figure 4.4). It is seemed that the deflection had taken the most at mid span of members and had causing a small displacement of the center joint for about 0.002mm in Y direction and 0.001mm in Z direction causing the resultant of 0.003mm. The bending moment and shear diagram is shown in figure 4.5 and 4.6. The high bending moment that resulted is mostly at the mid span till the center joint at all members and the negative value of shear occurred at the center joint, slightly bigger value at beam 54.

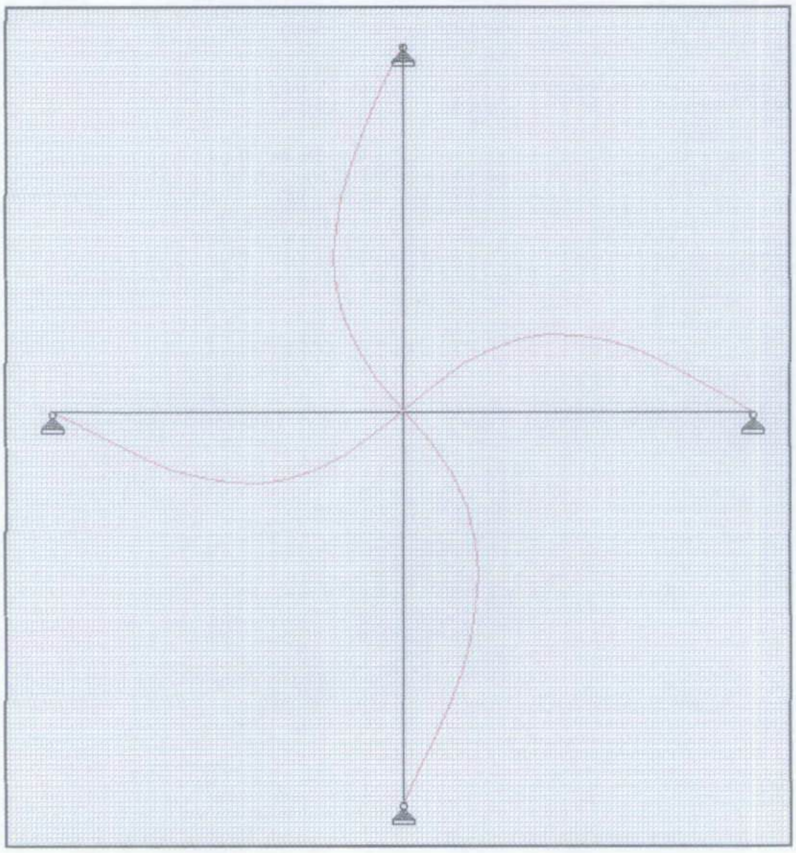

Figure 4.4: Deflection of node 40

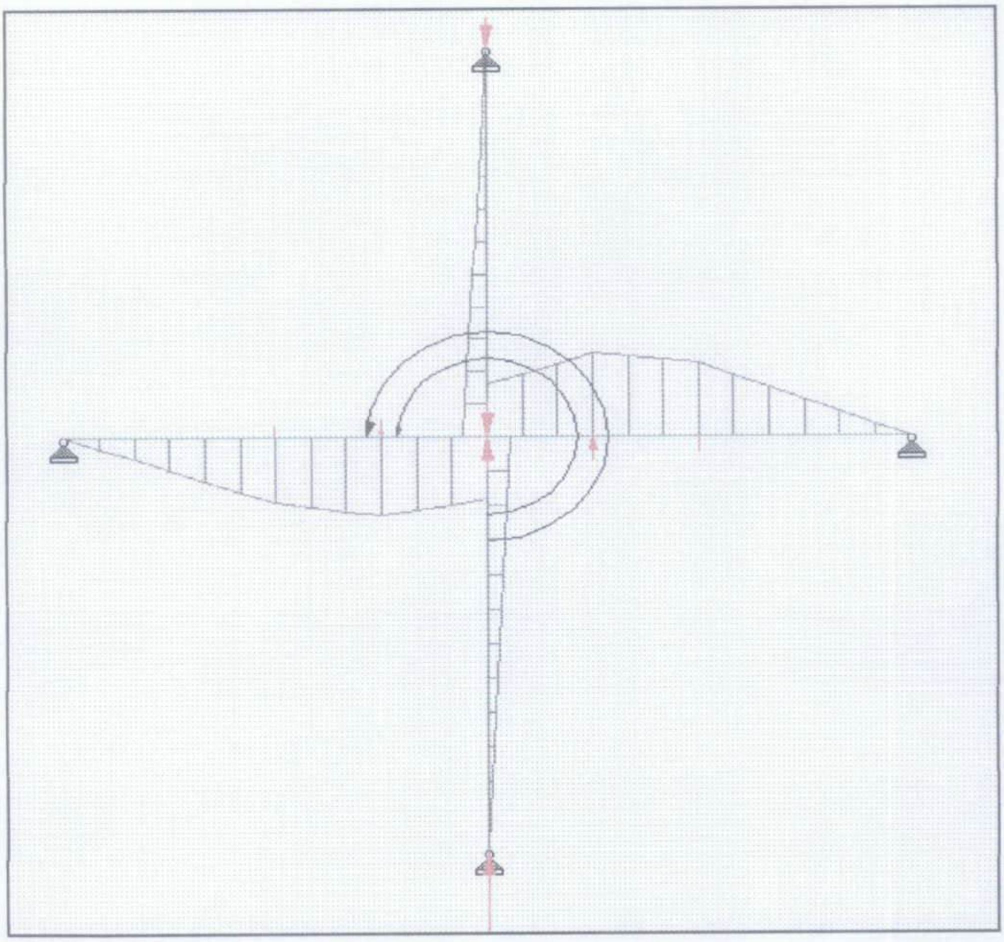

Figure 4.5: Bending diagram.

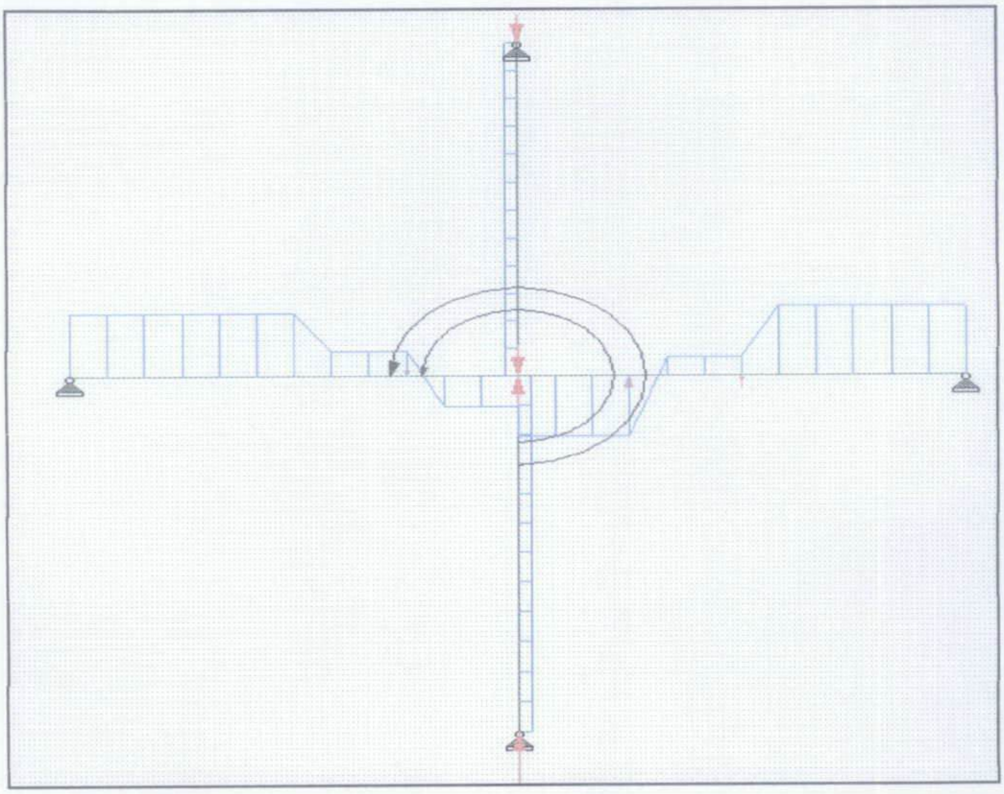

Figure 4.6: Shear diagram

Concerning the beam element, two types of stresses are considered – compression and tension. Stresses are been calculated by dividing the force over area, giving the unit of N/mm^2. Table 4.6 shows the result of stresses occurs along the members at the corner and the maximum stresses available. For these stresses, the STAAD analyzed the joint using the load combination earlier and displays them as the max stresses occurred according to the certain distance. For example, for beam 8@column 108, at distance 0m (center joint), the max tensile stress is 1.267 N/mm^2 and compression of 1.332 N/mm^2 .

			Corner Stress				Max Stress	
Beam	L/C	Dist m	Corner 1 N/mm2	Corner 2 N/mm2	Corner 3 N/mm2	Corner 4 N/mm2	Max Comp N/mm2	Max Tens N/mm2
8	1 LOAD 1	0.000	-1.267	-1.115	1.332	1.179	1.332	-1.267
		0.400	-0.942	-0.828	1.007	0.892	1.007	-0.942
		0.800	-0.618	-0.541	0.682	0.606	0.682	-0.618
		1.200	-0.293	-0.254	0.357	0.319	0.357	-0.293
		1.600	0.032	0.032	0.032	0.032	0.032	0.000
9	1 LOAD 1	0.000	-1.105	-1.264	1.044	1.203	1.203	-1.264
		0.425	-0.836	-0.956	0.776	0.895	0.895	-0.956
		0.850	-0.568	-0.647	0.507	0.586	0.586	-0.647
		1.275	-0.299	-0.339	0.238	0.278	0.278	-0.339
		1.700	-0.030	-0.030	-0.030	-0.030	0.000	-0.030
10	1 LOAD 1	0.000	-0.002	-0.002	-0.002	-0.002	0.000	-0.002
		0.375	0.921	0.920	-0.924	-0.923	0.921	-0.924
		0.750	1.843	1.841	-1.847	-1.845	1.843	-1.847
		1.125	2.205	2.202	-2.209	-2.206	2.205	-2.209
		1.500	1.759	1.755	-1.763	-1.759	1.759	-1.763
11	1 LOAD 1	0.000	-1.448	-1.452	1.452	1.455	1.455	-1.452
		0.375	-2.320	-2.323	2.324	2.327	2.327	-2.323
		0.750	-2.042	-2.044	2.046	2.048	2.048	-2.044
		1.125	-1.020	-1.021	1.024	1.025	1.025	-1.021
		1.500	0.002	0.002	0.002	0.002	0.002	0.000

Table 4.6: Beam and Column Stresses.

Graphical data has been generated using color contour to describe and differentiate the compression and tensile stresses. The tensile stress is shown by the blue color and compression by the red color. The figures below from 4.7 till 4.10 show the distribution of the stresses along the members. The light yellow plane at the left hand side of the figure indicates the joint region. At the right hand side of the figure, the corner stresses is shown together with the value legend.

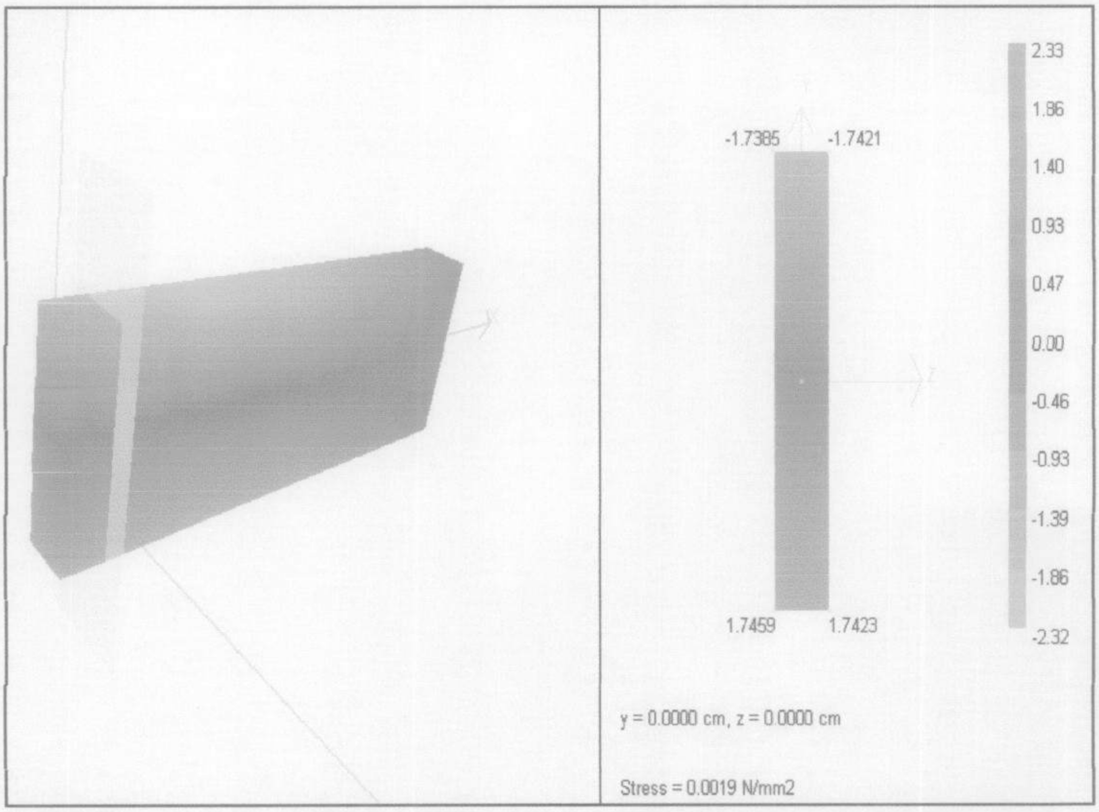

Figure 4.7: Stresses on Beam 11-Compression (Red) Tension (Blue)

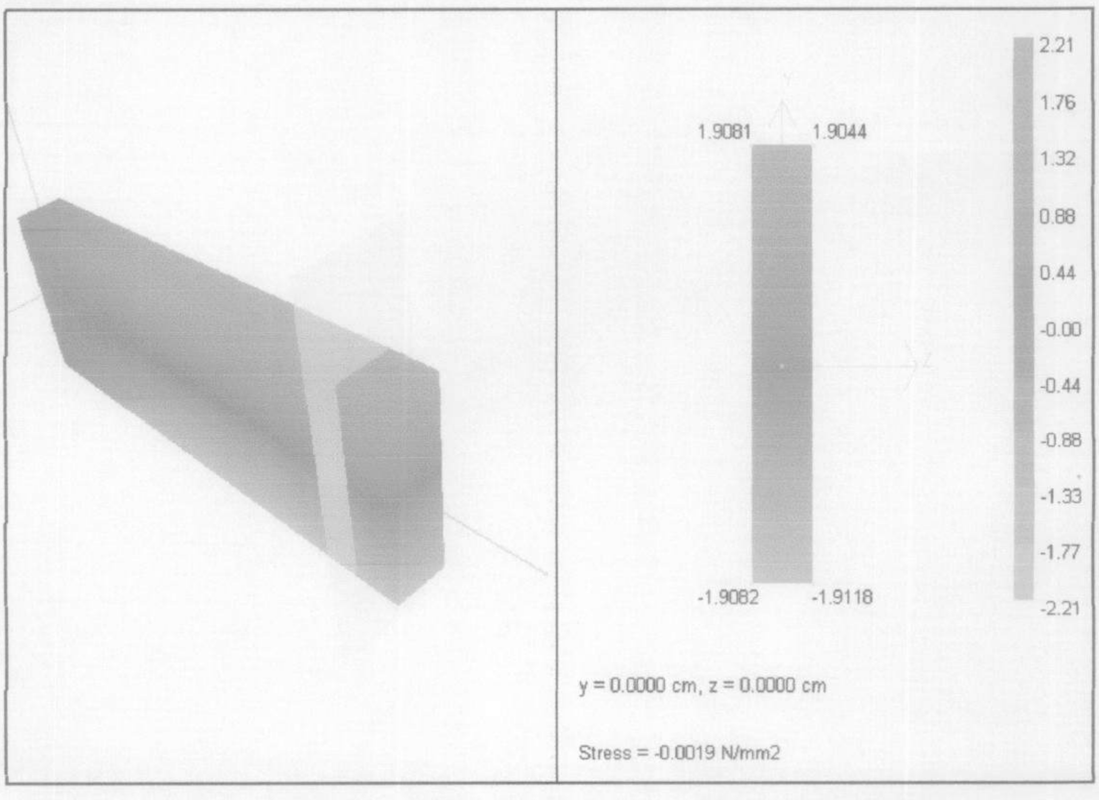

Figure 4.8: Stresses on Beam 10-Compression (Red) Tension (Blue)

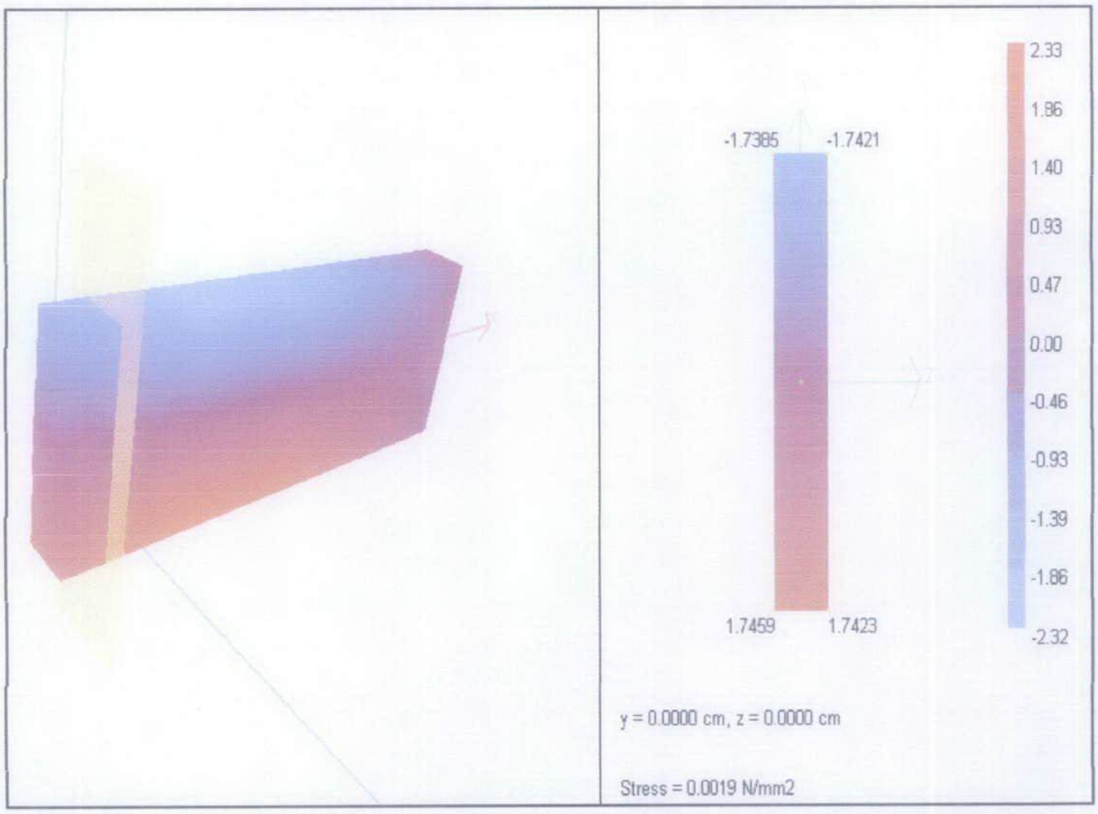

Figure 4.7: Stresses on Beam 11-Compression (Red) Tension (Blue)

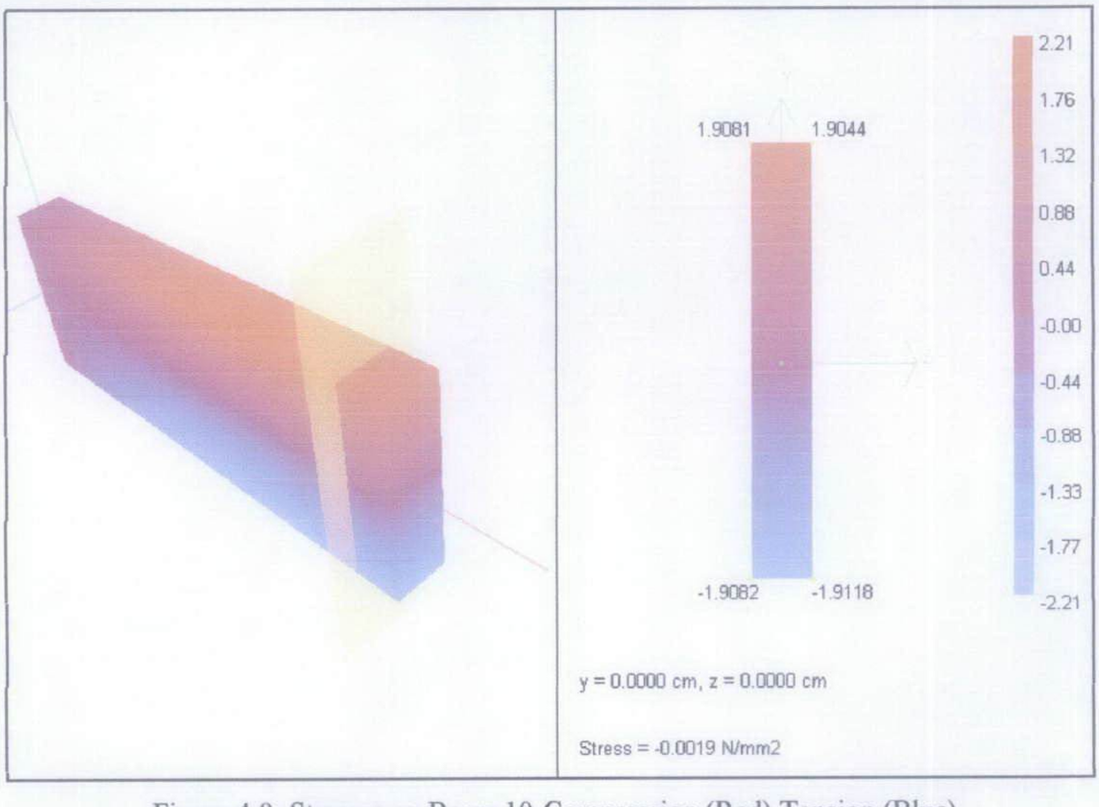

Figure 4.8: Stresses on Beam 10-Compression (Red) Tension (Blue)

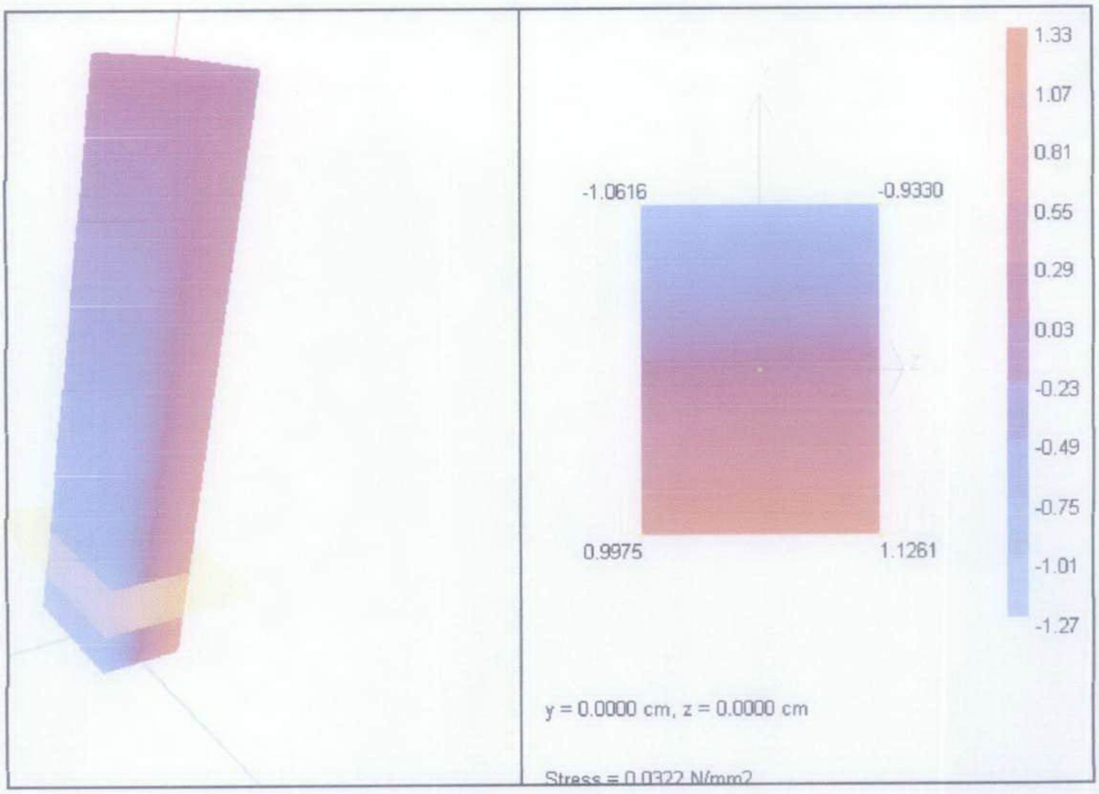

Figure 4.9: Stresses on Beam (Column) 8-Compression (Red) Tension (Blue)

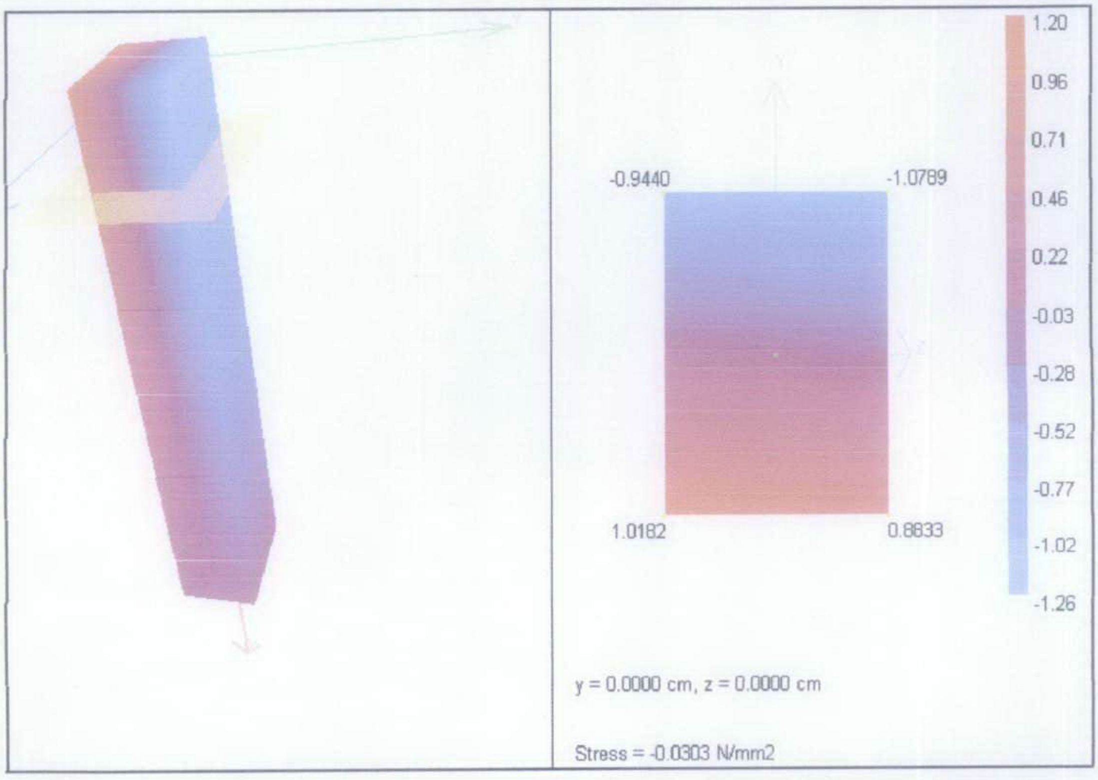

Figure 4.10: Stresses on Beam (Column) 9-Compression (Red) Tension (Blue)

4.4 ANSYS Simulation

From this simulation, we could know and see the effect of loadings onto the joint; cracking due to tension and compression, the stress distribution in the joint and also the bonding stress between concrete and the reinforcement bars, especially longitudinal bars.

4.4.1 Tension and Compression

Normal stress is stress that acts perpendicular to the face of the material. It can act in *compression* or in *tension*. Compressive stress is the stress applied to materials resulting in their compaction; decrease of volume. When a material is subjected to compressive stress then this material is under compression. Loading a structural element or a specimen will increase the compressive stress until the reach of compressive strength. According to the properties of the material, failure will occur as yield for materials with ductile behaviour or as rupture for brittle behaviour. Compressive stress has stress units (force per unit area), usually with *negative values* to indicate the compaction. For this specimen, failure would only occur when the stress exceeding the ultimate yield strength of 15.46N/mm2.

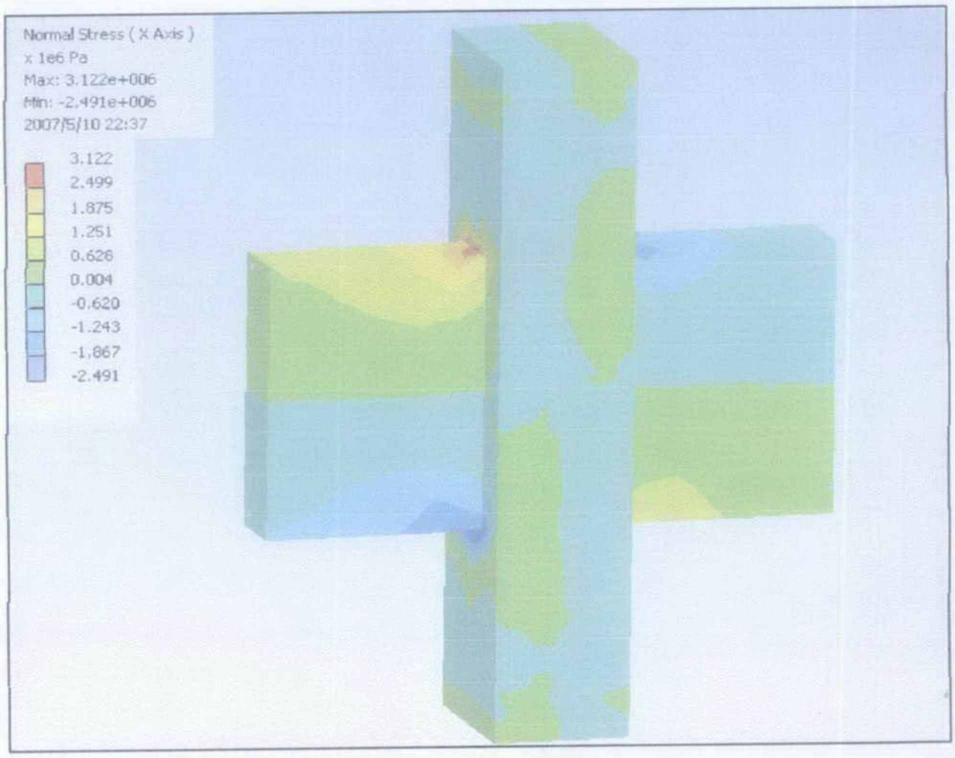

Figure 4.11: Normal stress distribution

Meanwhile, the tensile stress or tension is the stress state leading to expansion; that is, the length of a material or compression member tends to increase in the tensile direction. The volume of the material stays constant. Tension has the *positive values.* The figure 4.11 above showed the distribution of the normal stress on the beam – column joint after the simulation. From the figure, it could be tell that most of the joint's parts were in tension indicated by the greenish contour. The maximum value was 3.12 N/mm2 (tension) and - 2.49 N/mm2 (compression). The joint tended to rotate counter-clockwise as shown in figure 4.12, resulting the upper left and lower right experienced tension (figure 4.13).

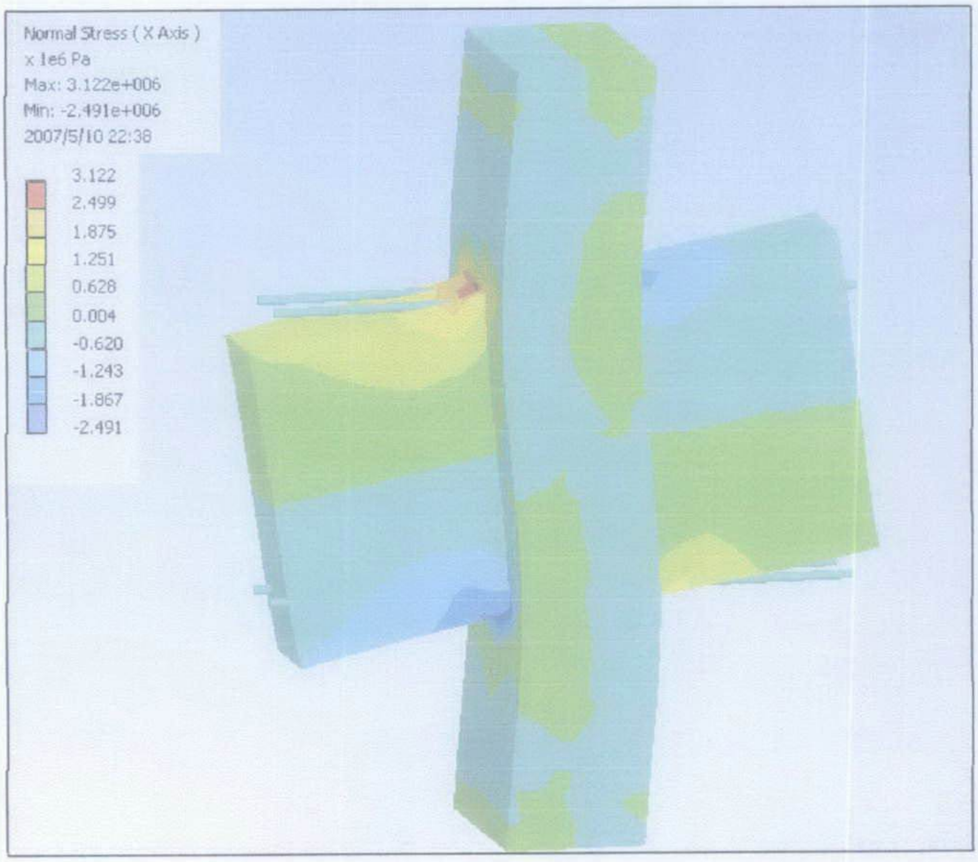

Figure 4.12: Normal stress distribution with 1300 scale enlargement

From the picture below (figure 4.13), the circle areas were identified as the critical edges between beam and column. In those parts, diagonal tensile would develop and it did happen after the simulation. The maximum value was spotted at those edges. According to fact, cracks would happen perpendicular to the tension diagonal area; between the two circles. However, with the resulted maximum value of 3.12N/mm2, the crack was far from existed, compared to 15.4N/mm2 – cracks began to occur.

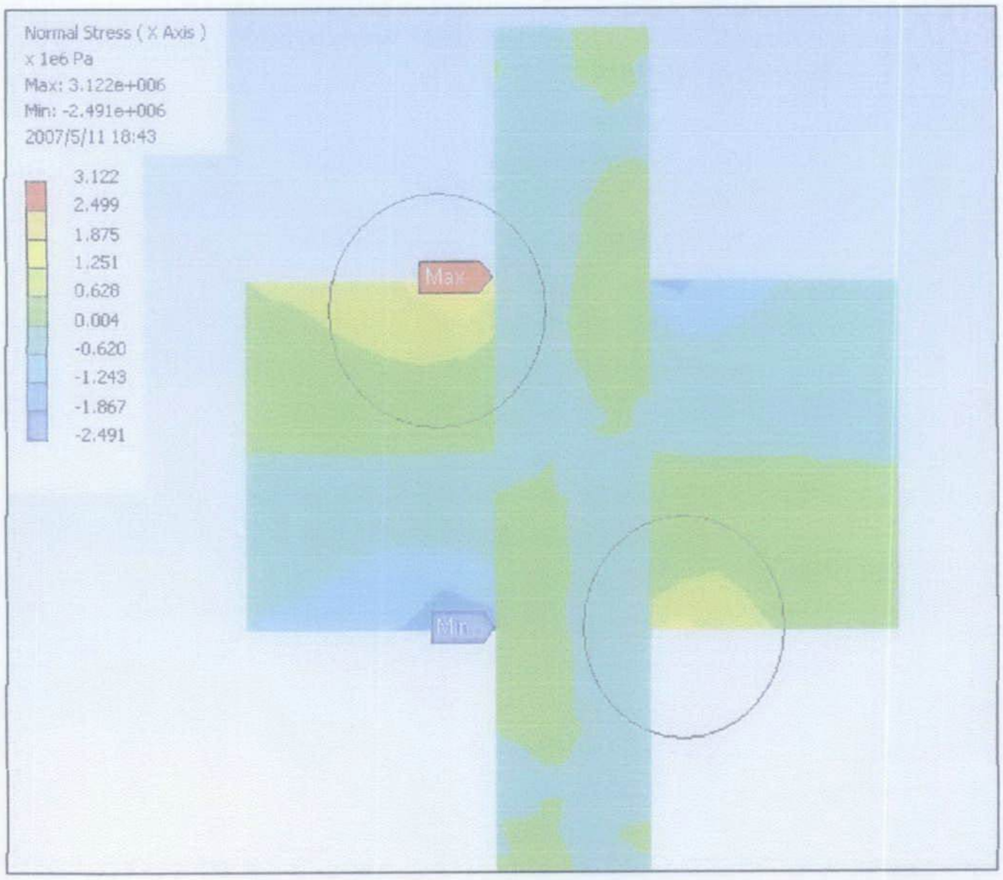

Figure 4.13: Maximum value (tension)

4.4.2 Bond Stress / Bond Strength

Steel-reinforced concrete is a widely used structural material. The effectiveness of the steel reinforcement depends on the bond between the steel reinforcing bar and the concrete. The effects of admixtures, water/cement ratio, curing age, rebar surface treatment and corrosion on the steel ± concrete bond were very critical onto the bond strength. It is universally accepted that a good bond which allows effective stress transfer from the concrete matrix to the embedded steel reinforcing bars is essential in reinforced concrete members.

Ideally, a perfect bond involves no rebar slippage. However, if the bond between the concrete and the rebars is imperfect, internal rebars may slip and the composite action may not be fully developed. In such a case, development of cracks and sudden collapse of the RC section may be predicted.

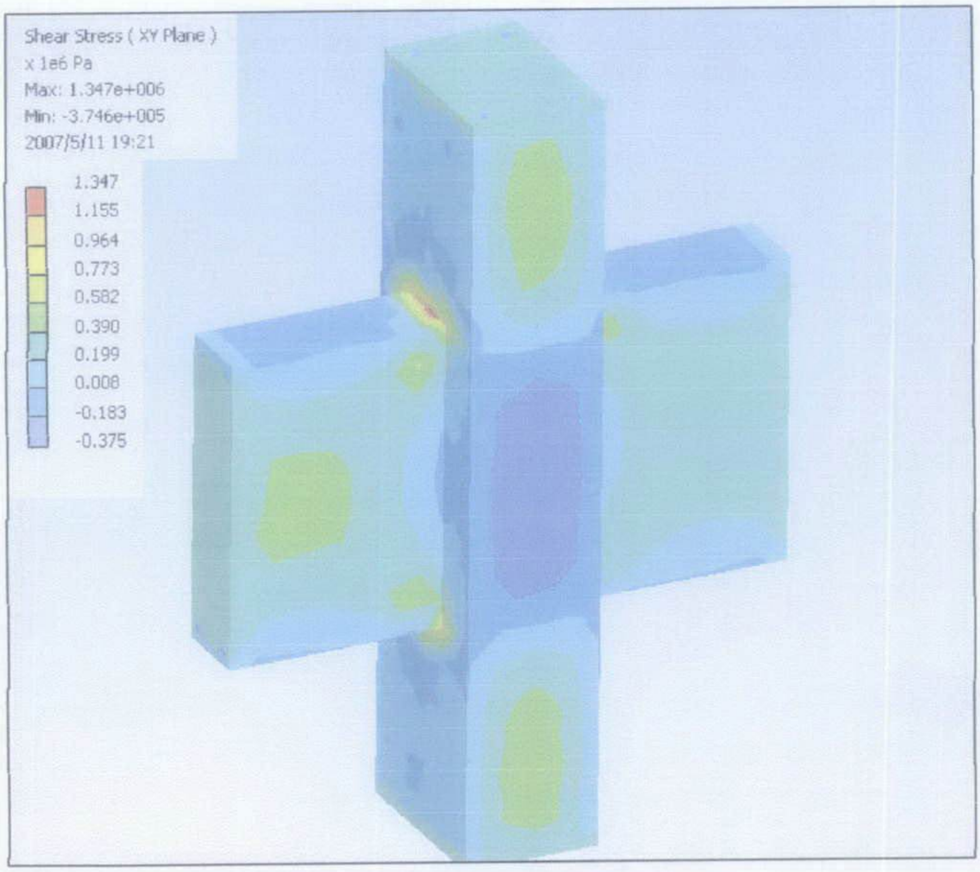

Figure 4.14: Shear stress distribution.

The high internal forces (pseudo-static) developed at plastic hinges cause critical bond conditions in the longitudinal reinforcing bars passing through the joint and also inflict high shear demand in the joint core. The figure 4.14 above showed the distribution of the shear stress on the joint, most of the area was in tension. As it showed, the maximum value of shear stress was at the edges between beam and column where diagonal crack would occur.

Figure 4.15a showed the contact between beam reinforcement bars and concrete and the stresses in the joint, while figure 4.15b showed the closer view of the circled area to see the contact and bond stresses that occurred along the reinforcement bars. The bond stress value that obtained along the rebars was in range of 0.008 N/mm2 to 1.2 N/mm2. This value represented all the beam rebars inside the joint and indicated that the value was far below from the 0.95 $\sqrt{fc'}$ MPa @ N/mm2 value stated by Sezen.H and Moehle.J in their research of *Bond-slip Behavior of Reinforced Concrete Members.* The 0.95 $\sqrt{fc'}$ MPa or 5.2 N/mm2 bond stress was the average bond stress for concrete fc′ = 30 N/mm2 which was used for the school building.

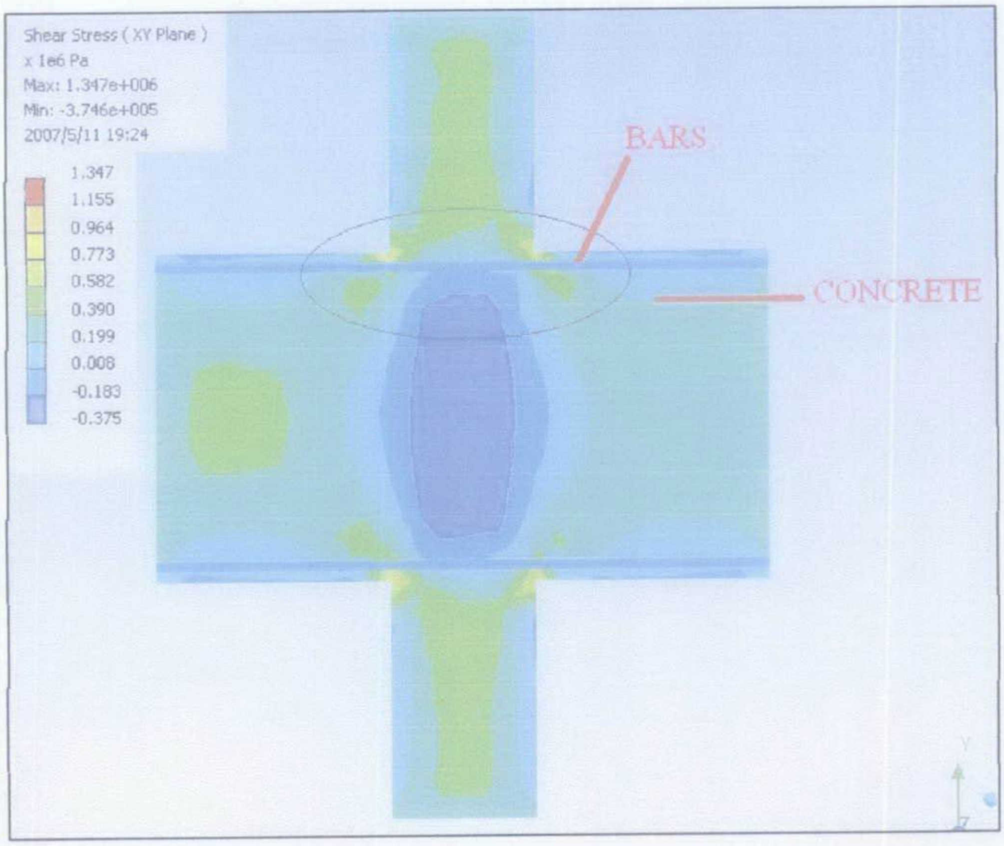

Figure 4.15a: Sliced-plane view of joint (beam)

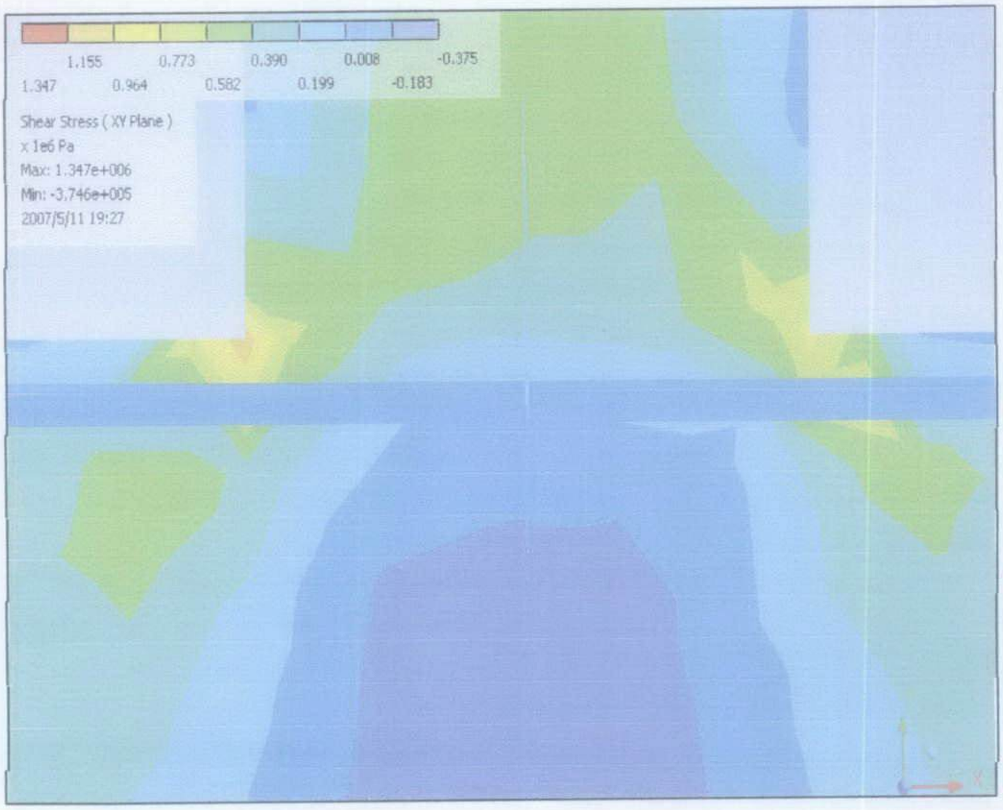

Figure 4.15b: Bond Stress (beam)

The average bond stress that from Sezen.H and Moehle.J, were actually quite close to the value obtained by Jingyao Cao and D.D.L. Chung from Composite Materials Research Laboratory, State University of New York, Buffalo, NY which was 5.54 ± 0.43 MPa @ N/mm2. This proved that the value was reliable. The resulted value that below the suggested value, has verified that the bond stress generated onto the building did not exceeded the bar – slip stress limit. In order for a bar to slip from the concrete, the bond stress needs to go beyond 5.2 N/mm2, at least.

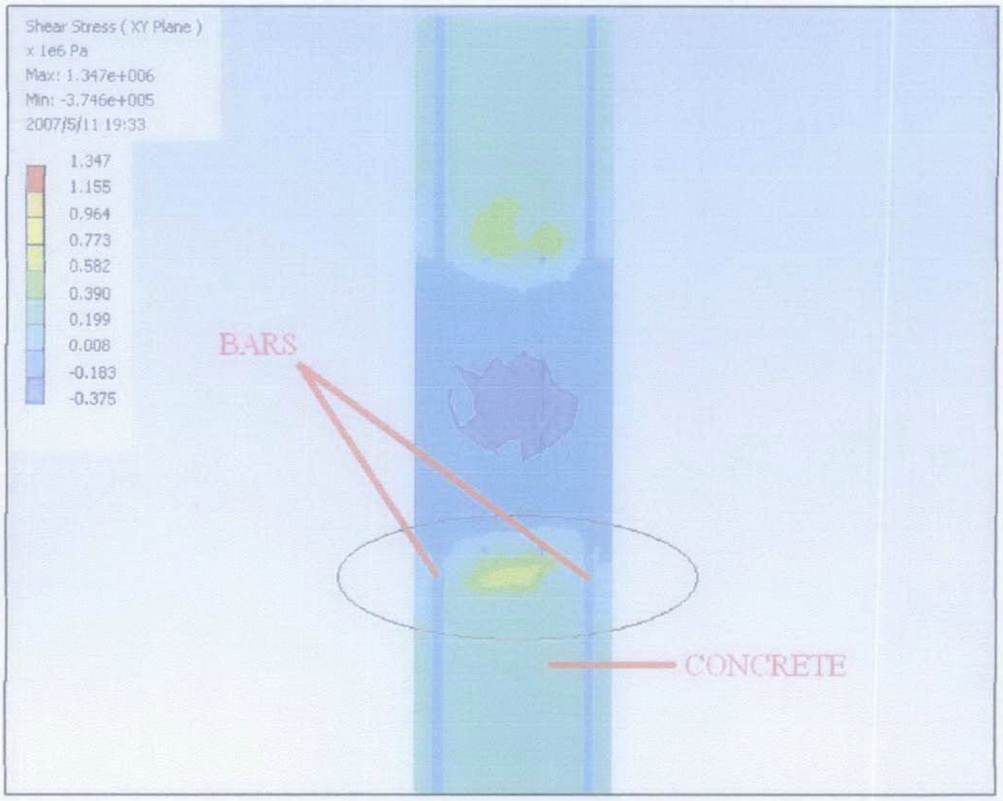

Figure 4.16a: Sliced-plane view of joint (column)

The value range of 0.008 N/mm2 to 1.2 N/mm2 earlier was for the beam bond stress. For the column, a range of 0.05 N/mm2 to 0.26 N/mm2 was collected. From this we could tell that they were more stresses on beam rebars than the columns.

The value obtained from the column indicated that no slip would occur between the rebar and the concrete. As from Dr. S. R. Uma and Prof. A. Meher Prasad, in order for the column to experienced bond deterioration, the column depth should be at least 28 bars diameter. For example, the school column used the 20 mm bar size, so the depth of 560 mm should be provided, which it was provided – 550 mm.

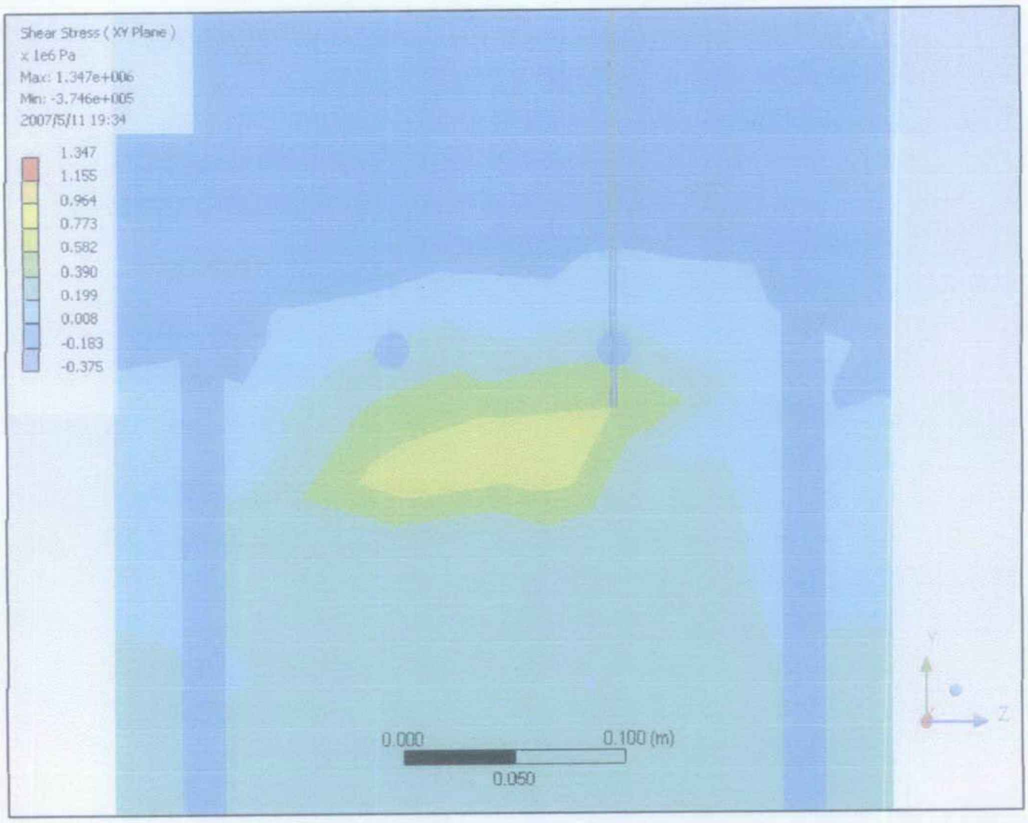

Figure 4.16b: Bond Stress (beam)

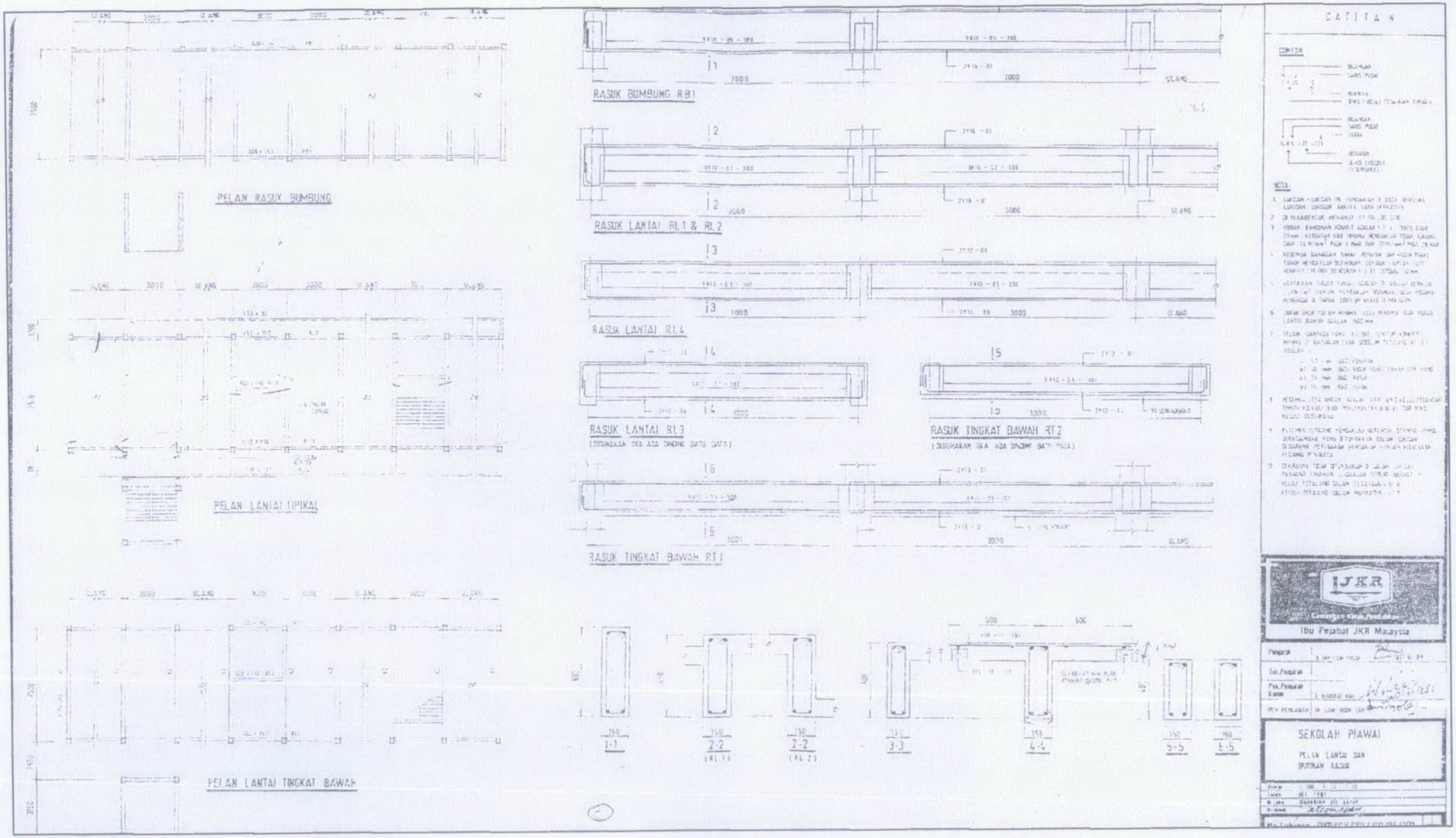

Figure 4.17: Floor plan

CHAPTER 5

CONCLUSION

This research is base on analyzing the behaviour of the critical part of a RC structure – beam column joints of a typical school building in Malaysia in order to cope with the impact of the earthquake seismic loading. Several tests can be modeled or performed to analyze the behaviour such as frame analysis and ANSYS simulation. From the test conducted using the equivalent linear static analysis, it is determined that the Zone 2 UBC loading doesn't give a major impact onto the structure. The existing beam and column can handled the load sufficiently. However, better software may give a better and more accurate result than STAAD. Later on, the joint is tested using the ANSYS Workbench simulation. Before that, a model of the joint is done using the CATIA. By using the pseudo-static forces obtained from the STAAD, the ANSYS simulation has given quite fascinating result. The result from the ANSYS also shows that the conclusion earlier with STAAD is right. With a low value of bond stress occurs between the concrete and the rebar, the school structure is definitely safe from the Zone 2 UBC seismic loading. With that, the overall objective of this study for these two semesters has achieved.

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