ANALYSIS OF REINFORCED CONCRETE DAPPED-END BEAMS USING VECTOR2

by

SARAH JOY BINTI NUR

Dissertation submitted in partial fulfillment of

the requirements for the

Bachelor of Engineering (Hons)

(Civil Engineering)

SEPTEMBER 2013

UniversitiTeknologi PETRONAS
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CERTIFICATION OF APPROVAL

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Approved by,

____

DR TEO WEE

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TRONOH, PERAK

SEPTEMBER 2013

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 sourced or persons.

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ABSTRACT

The concept of dapped-end beams is expansively used in buildings and other structures as well as it provide better lateral stability. The design of dapped-end connections is an important consideration in a precast concrete structure even though its analysis is complex. Moreover, the implication of the Concorde Bridge incident has attracted many researchers and this collapse prompted detailed investigation of structural analysis for Dapped-end Beam whereby the determination of the maximum load capacity of this structure is very crucial for the sake of the next designing purpose. Therefore, this project aims to determine the failure load of the Dapped-end Beam by using this sophisticated two-dimensional non-linear finite element analysis program called VecTor2 and a data from an experiment which has been done by other researchers, will be used for corroboration. In this study, 5 specimens of the Dapped-end Beam were tested to obtain the load capacity and identify the part of this structure that contributes more to failure. As the results, most of these specimens were failed at the diagonal tension at re-entrant corner of this structure. Besides that, the comparison shows that the proposed method able to predict the failure load very close to the existing results.

ACKNOWLEDGEMENT

First and Foremost, I am grateful to Allah S.W.T. for the blessing and strength that has provided me with the inspirations to pursue and finally, completed my final year project successfully.

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CHAPTER 1

INTRODUCTION

1.1 Project Background

Precast structural members provide several advantages to building designers and contractors, especially in situations where the speed of construction is emphasized. Precast manufacturing expedites the construction process by allowing large pieces of a building project, such as beam, slabs, and thin-stemmed ‘Tee’ members, to be cast off-site and then transported to the worksite rather than forming and casting each structural element in place and then allowing for time to cure. Moreover, prestressing these precast structural elements can further optimize efficiency by allowing members to span longer distances and carry higher loads than those reinforced with mild steel alone. The other advantage of precast concrete manufacturing is able to provide better quality control than traditional concrete construction due to the repetitive, controlled, industrial production. Such a setting allows for a reduction in construction error and the creation of favorable casting and curing conditions.

Besides that, one often used in buildings, bridges and parking garages that is unique to precast concrete construction is the dapped end. A dapped end is created when the web or stem of a beam is notched at the bottom corner, moving the bearing location higher in the cross-section. The notch itself is known as the “dap” and the portion of concrete remaining above the dap is referred to as the nib. The dapped end detail enables the overall depth of a precast floor or roof structure to be reduced by recessing the supporting corbel or ledge into the supported beam. By allowing for a reduction in floor height, the dapped end detail can significantly reduce the overall height of a building.

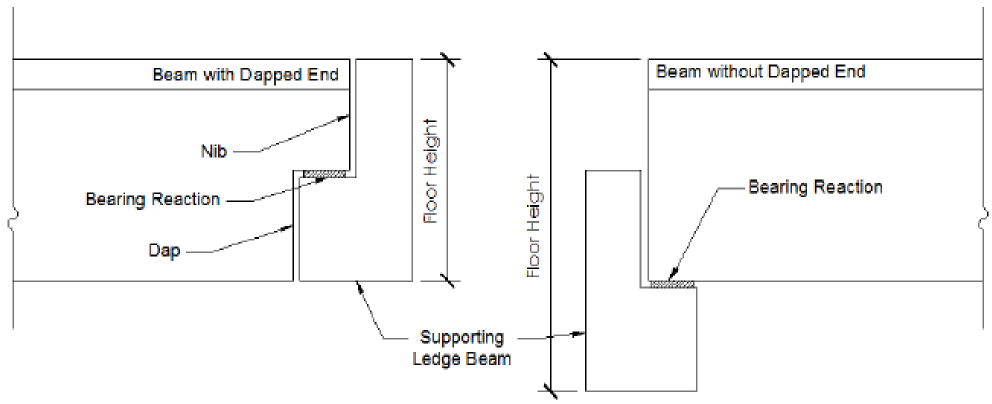

Figure 1.1: Beams with and without Dapped Ends

Figure 1.2: Typical Dapped End in a Precast Concrete Beam

The design and detailing of a dapped end connection must consider the severe stress concentration that develops at the re-entrant corner. Dapped ends are often subjected to high bearing reactions that must be safely resisted by transferring forces into the main cross-section of the beam through the reduced cross-section of the nib. A bearing point that is eccentric to the dap face and the potential for additional axial loads from bearing friction and axial shortening due to creep complicate the design. Furthermore, design of the dapped end can be more complicated by the presence of prestressing strands below the nib or through the nib. Prestressed strands may transfer additional horizontal and splitting forces into the section in the dap region. The magnitude of the forces applied to the section by prestressing can be many times the magnitude of the primary dap reaction. In addition, in prestressed beams with dapped ends, the need may arise to transfer forces between mild steel reinforcement and prestressing strands through lap splices.

In the precast concrete industry, the design of dapped end beams typically follows the provisions outlined in the PCI Design Handbook (Precast Prestressed Concrete Institute, 2010). However, dapped end reinforcing details are not standardized across the industry. Therefore, few research and experimental studies of the dapped-end beams have been conducted for more understanding and to answer any questions that arise regarding this kind of structure. This study aimed to investigate the behavior of various dapped end beam reinforcement details in precast structures. Multiple dapped-end beams details were chosen from the previous research and will be used to analyze in the sophisticated Non-linear Finite Element program called VecTor2 and also PCI design method. Furthermore, this yields from the advances in the combination of both the computer power and mathematical techniques as they have led us to more sophisticated investigations.

1.2 Problem Statement

Issues regarding the dapped-end beam failure have been catching the interest of many researchers since after the incident of the south halves of the Concorde overpass structure collapsed in Laval, Quebec on the 30th of September 2006 and killed five people with six injured. The fact that this bridge failed after nearly forty years in service and essentially under its own weight was concern for the safety of the other bridge of a similar age. For this case, shear capacity prediction is very crucial to determine the failure load of the structure thus able to design the structure under shear capacity. Besides that, PCI design method is often used as the guideline for designing the dapped-end beam. However, the effectiveness of this design method in dapped-end beam analysis was not proven yet.

Figure 1.3: Aerial view of the Concorde Bridge

1.3 Objectives and Scope of Study

The prime objective of this project is to study the design of the Dapped-end Beam for Precast Concrete structures. In order to achieve the general aims, the following specified objectives are proposed to be achieved:

- To determine the failure load of the Dapped-end Beam by using VecTor2.
- To compare the analysis results obtain from VecTor2 and PCI design approach with the experimental result that has been done by the other researchers.

1.4 Relevancy of the Project

The reason behind the idea is to identify the problems that rise up from the beginning of the project until the project completed and also find solution on how to overcome it. The assessment of the behavior of the Precast Dapped-end Beam requires tools that can be used to analyze and design structural element that will improve the state of the art of protective design.

1.5 Feasibility of the Project

Up to this moment, the project has been conducted in accordant with the plan showed in the Gantt chart. In the first progress, collecting and gathering all the data and information about the dapped-end Beam and understand more about the PCI Design method are very crucial since this project is mainly used this method to carry out the analysis. The next step was to perform the analysis for the selected Dapped-end Beams details from the previous research. Whereby for this case, the results obtained compare with the existing result of this analysis. In conclusion, this project was able to achieve its main objectives within the time frame given based on the scope of study.

CHAPTER 2

LITERATURE REVIEW

2.1 Dapped End Connection

Nowadays, Precast Concrete (PC) structures have become more popular in the construction industries. The widely use of PC in particular has been shown to be technically advantageous, economically competitive and esthetically superior because of the reduction of cross-sectional dimension and consequent weight savings and larger shear force resistance. The use of this kind of concrete can improve the quality of the final products, decrease construction time and assist the progress of construction in adverse weather conditions. Unlike a cast-in-place Reinforced Concrete (RC) structure that is by nature massive and continuous, a precast concrete structure is composed of individual prefabricated members that are connected by different types of connections. The type of connections used to determines the behavior of a precast structure when subjected to a certain load. The concept of dapped-end beams is widely used in bridges or buildings due to its feasibility to provide better lateral stability and reduce the floor-to-floor height. Examples of dapped-end application are as a cantilever and suspended span type of structure, drop-in between corbels and also as a hide-away type of beam-to-beam and beam-to-column connection.

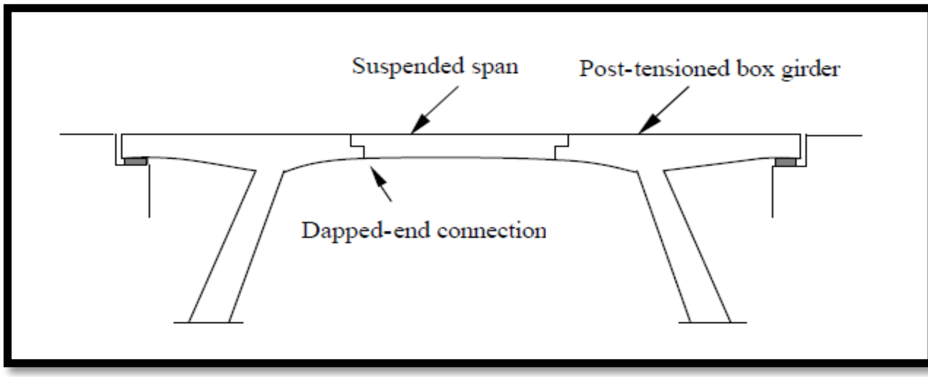

Figure 2.1: As a Cantilever Suspended Span Bridge

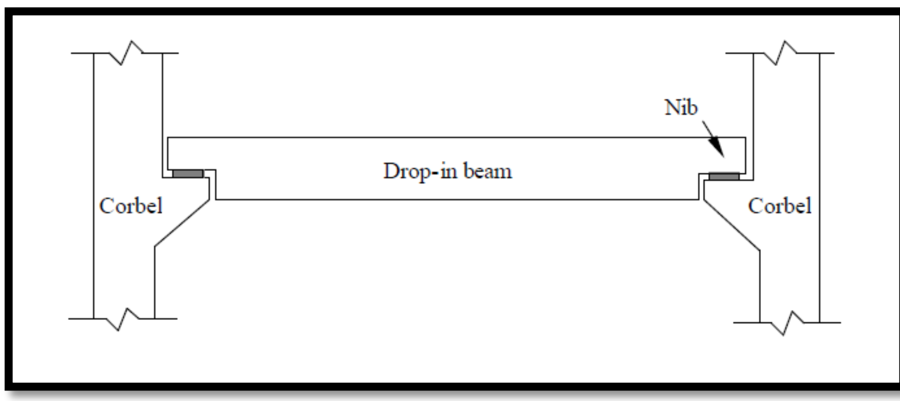

Figure 2.2: As a Drop-in Beam Supported by Corbels

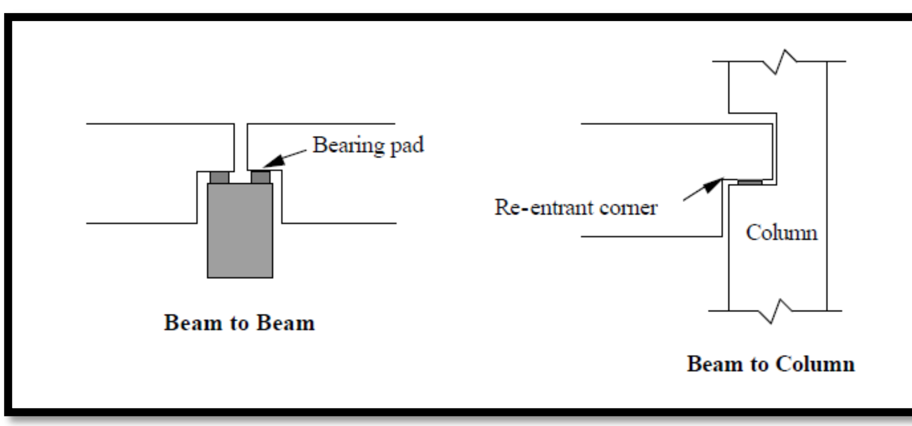

Figure 2.3: As a Hide-Away Type Connection

The design of dapped-end connections is one of the most important considerations in a precast structure. However, the analysis of connections in dapped-end structures is very complex. The unusual shape of the dapped-end beam develops a severe stress concentration at the re-entrant corner. In this case, flexural theory is only partially applicable. Furthermore, in addition to the calculated forces from external loads, dapped-ends are also sensitive to horizontal tension forces arising from restraint of shrinkage or creep shortening of a member. Therefore, if suitable reinforcement is not provided close to the re-entrant corner, the diagonal tension crack may propagate rapidly and failure may occur with little or no warning. Figure 2.4 shows the stress concentration at re-entrant corner of different a/d ratios, where a is the shear span and d is the effective nib depth. As compared to a conventional straight end, the solid contour lines represent tension, while the broken dotted line represented crack direction.

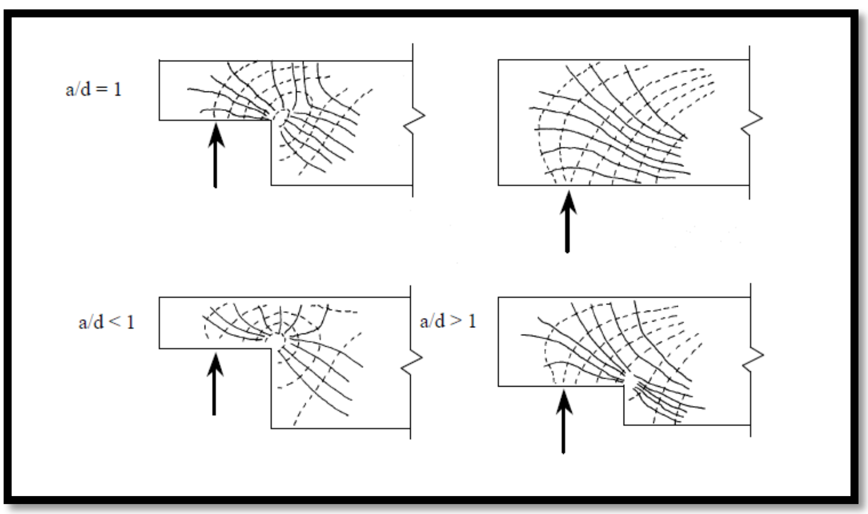

Figure 2.4: Stress Concentration at Re-entrant Corner of Dapped-end Beams

2.2 Previous works on Dapped-end design

Various researches have been performed on dapped-end beams until 1969 when Reynold presented his paper "The Strength of Half Joints in Reinforced Concrete Beam". In 1970, a comprehensive research was carried out by Mattock at the University of Washington in Seattle. Research on dapped-end design then produced practical criteria. While, Reynold in 1969 have carried out the test to developed suitable reinforcement details evolving a design procedure for dapped-end members. However, he then noticed that joints can be designed by a straightforward consideration of equilibrium, horizontal stirrups should be included against the misplacement of diagonal stirrups and axial tension, and tensile reinforcement should be extended to the end of the beam to offer anchorage for stirrups. As the result, diagonal stirrups provide suitable reinforcement.

The other method for the Dapped-end design is to use the Finite Element analysis to determine the behavior and strength of dapped-end beams (Sargious and Tadrus, 1970). Werner and Dilger in 1973 have done the research on determination of first cracking shear at re-entrant corner using Finite Element Method (FEM) and also the concrete contribution to cracking shear. As the result of their research, cracking load can be taken as contribution of concrete, vertical and inclined shear reinforcement seem to be equally efficient in resisting shear. Besides that, they also have conclude that shear strength is the summation of the concrete, shear reinforcement, and prestressing tendons.

Another research is to develop the mechanics of diagonal shear cracks (Hamoud et al., 1975). Based on this research, shear strength of prestressed dapped-ends can be predicted based on elastic analysis. In addition to that, shear cracking load for beams with post-tensioned bars equal to failure loads and beams with low values of reinforcement and high prestress failed in flexure, while low prestressed beam failed by concrete rupture. Hence, ultimate shear strength increased with an increase in prestress and a/d ratio.

Mattock and Chan in 1979 have performed the research about the corbel design application to dapped-end and determination of the concrete capacity and if the shear span “a” should be measured from load center to the re-entrant corner or to be center of stirrups. As the result of this research, the reduced depth of dapped-end may be designed as corbel if “a” is measured to the center gravity of the hanger reinforcement. Closed stirrups should be provided close to the end face of full-depth beam to resist the vertical component of the inclined compression in the nib. The full-depth part of the beam should be designed to satisfy moment and force equilibrium. Besides that, the main nib reinforcement should be provided with a positive anchorage as close to the end and the horizontal stirrups should be positively anchored near the end face of the beam and concrete contribution should be ignored.

However, another research have verified this Mattock and Chan’s design proposals for beams by having a/d ratio less and equal to 1.0, utilizing the horizontal stirrups only in the nib (Khan, 1981). Khan also had verified beams having a/d ration is greater and equal to 1.0, utilizing a horizontal and vertical stirrups in the nib. Results obtained showed the validity of Mattock and Chan recommendation for beams with a/d ratio less than 1.0 and the behavior of dapped-ends was in agreement with the assumption of a “truss-like” behavior. Ultimate strength of a dapped-end with 45 degree inclined reinforcement should have twice the strength of a dapped-end with horizontal or vertical reinforcement (Liem, 1983). Liem have conducted the studied about the maximum shear strength of a dapped-end or corbel with inclined reinforcement and compare to Mattock's study. He also have mentioned that a limit yield of steel to be 40 ksi in order to prevent a secondary collapse.

Chung in 1985 used two a/d ratios, one greater than 1.0 and one less than 1.0 to compare to Chan and Khan's study. Based on this analysis, Chung have noticed that Mattock and Chan's design leads to satisfactory behavior from strength and serviceability viewpoints in the case of h/H=0.5, the hanger reinforcement carries the total shear. Positive anchorage must be provided for both nib and beam flexural reinforcement at the faces of the beam. Horizontal stirrups are only satisfactory in dapped-end beam nibs with a/d less and equal to 1.0.

Ajina in 1986 have investigated the cracking and shear capacity of the connections with different patterns of shear reinforcement. As the result, 1.2% steel fibers can be considered as reinforcement proficient enough to substitute for the vertical stirrups and only h/H greater and equal to 0.5 should be allowed in precast dapped-end beams when steel fibers are not to use. Also in the same year, another research which have been carried out by Theryo to investigate the behavior of a dapped-end at ultimate can be modeled using an analogue truss by providing 45 degree, 60 degree and 90 degree lop anchor hanger reinforcement at their upper end. According to this analysis, the behavior of a dapped-end can be modeled using an analogous truss, whereby a contribution can be included if 50% of the total prestressing strands pass through the nib. Besides that, the vertical and inclined hanger reinforcement seems to be equally efficient in resisting shear. However, the inclined hanger reinforcement is much more effective in controlling cracking at service and it is suggested to provide a minimum 1.0 inch bottom concrete cover to hanger reinforcement instead of 0.75 inch.

The ultimate shear capacity of strut and tie model details exceeded the design ultimate substantially and was in the range as the PCI and Menon/Furlong details (Barton, 1988). He also mentioned that as load increased beyond the design load of 100 kips, the distribution of internal forces changed. This resulted from partly the result of the method of testing and partly present of force transfer mechanisms not considered by the strut and tie model. Anchorage requirements based upon the strut and tie model are found to be conservative. Proper anchorage of the horizontal reinforcement within the dap flexure reinforcement was found to be particularly important.

Another method for dapped-end beam design is the strut and tie models which are also capable of estimating the failure load and the inclined dapped-end which is more efficient comparing to the rectangular dapped-end (So, 1989). Besides that, Mader in 1990 have carried out the analysis and compared the PCI method and the strut and tie model to determine how prestressing forces effect the load path in a beam. According to his research, all design methods resulted in beam ends that carried loads 15~20% higher than predicted except for the PCI method. While this strut and tie model specimens were 11~29% more efficient than the PCI models.

2.3 PCI Design Provisions

The design of a dapped-end termination is based on the shear-friction theory. The PCI Provisions require that several potential failure modes be investigated separately. Design of connections which are recessed or dapped into the end of the member greater than 0.2 times the height of the member (H in Figure 2.5), requires the investigation of several potential failure modes. These are numbered and shown in Figure 2.5 and listed below along with the reinforcement required for each. It should be noted that the design equations given in this section are based primarily on previous works by Mattock, A.H and Chan in 1979.

1. Flexure (cantilever bending) and axial tension in the extended end.
   Provide flexural reinforcement, $f\ A$, plus axial tension reinforcement, $n\ A$, equal to $s\ A$.
2. Direct shear at the junction of the dap and the main body of the member.
   Provide shear-friction reinforcement composed of $vf\ A$ and $h\ A$, plus axial tension reinforcement, $n\ A$.
3. Diagonal tension emanating from the reentrant corner.
   Provide shear reinforcement, $sh\ A$.

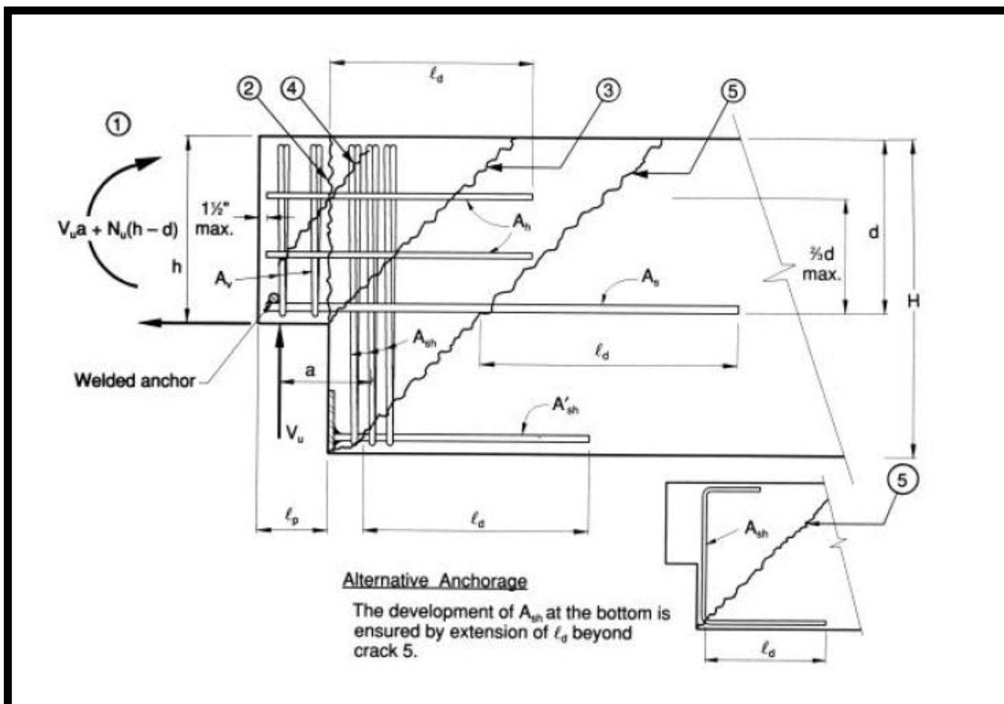

Figure 2.5: Potential Failure Modes and Required Reinforcement in Dapped-end Connection

4. Diagonal tension in the extended end. Provide shear reinforcement composed of $h\ A$ and $v\ A$.
5. Diagonal tension in the undapped portion. This is resisted by providing a full development length for $s\ A$ beyond the potential crack. Each of these potential failure modes should be investigated separately. The reinforcement requirements are not cumulative, that is, $s\ A$ is the greater of that required by 1 or 2. $n\ A$ is the greater of that required by 2 or 4.

CHAPTER 3

METHODOLOGY

3.1 RAPID Methodology

The method used mainly for this project is based on observation, Software skill, self study and discussion with the supervisor. Student have to studies and research more to understand about the project and further improvement will be made inside this project, uses mostly RAPID methodology.

I. **Results**

The result expected by the supervisor, is that this project can be used as the baseline and continued by the next administrator.

II. **Align**

The data required for the project are based on discussion, observation and self study with supervisor. Data collection is a continuous effort.

III. Pilot

This project will mainly refer to the experimental work which has been done by the other researchers. By using the VecTor2 software and PCI design method to conduct the analysis and compare the data obtain with experimental results.

IV. Insure

Supervisor is asked to check the progress of this project and give guidance and advice for further improvement.

V. Deploy

Data collected for this project are deployed after an utter discussion and the draft database are agreed with the supervisor.

3.2 VecTor2 and FormWorks

VecTor2 is a two-dimensional finite element program, used to analyze the concrete structures under various types of loads such as static, cyclic and thermal loads and the program is based on Modified Compression Field Theory formulations (Vecchio and Collins, 1986). While FormWorks is a multiple document interface with its application window encloses one or more child Workspace windows. Each Workspace is a unique document that can be saved and opened as a FormWorks file and contains all the information required to generate the input files for one VecTor2 finite element mode. The application title bar indicates the name of the active workspace in brackets. For example Workspace1, which created by default when the FormWorks application opens. The finite element model appears in the Workspace window as it is created.

Basically for this project, it is very necessary to be familiarizing with the software before starting the project. As for the beginning, a simply supported beam that subjected to static loading was used as an example to run the system and it can be conclude that this software does not have any problem to conduct such analysis. Thus, for the next step, 5 specimens from the experimental works will be taken out from different research papers and will be used to analyze in VecTor2 and PCI design method.

To run the VecTor2, there are several procedures that must be followed:

1. The first step in creating the VecTor2 input is to define the Job Data. Input the job data as described in the subsequent sections and when done select the **Models** page.

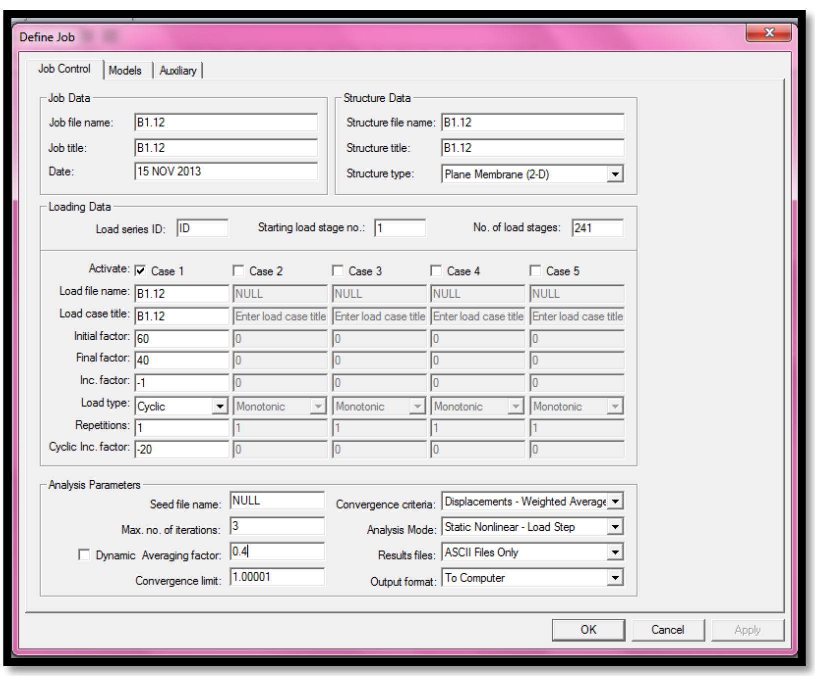

Figure 3.1: Job Control dialog box

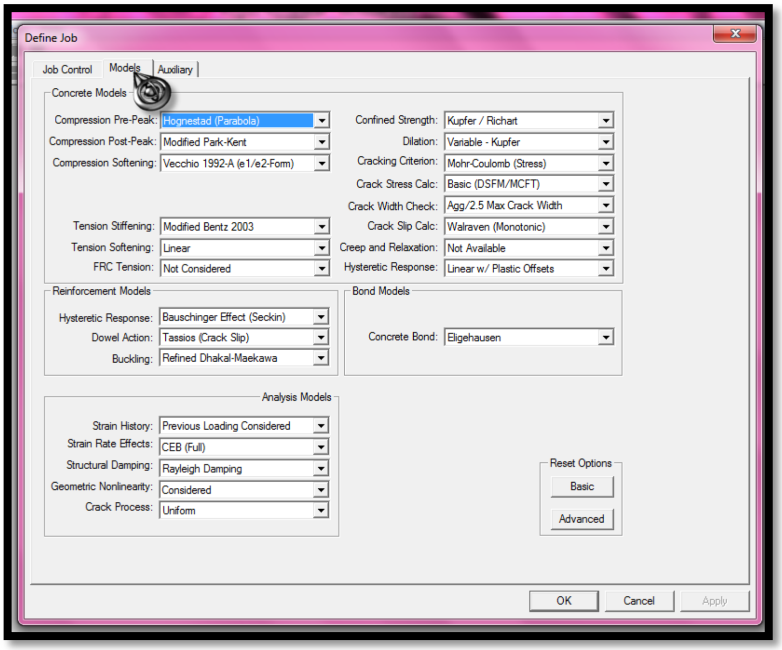

Figure 3.2: Models dialog box

2. Define Reinforced concrete properties
- Reinforced concrete materials types described concrete with or without one or more smeared reinforcement components. These material types are applied to rectangular, quadrilateral, or triangular elements.
- All the data inserted are based on table 2 the specimen properties.

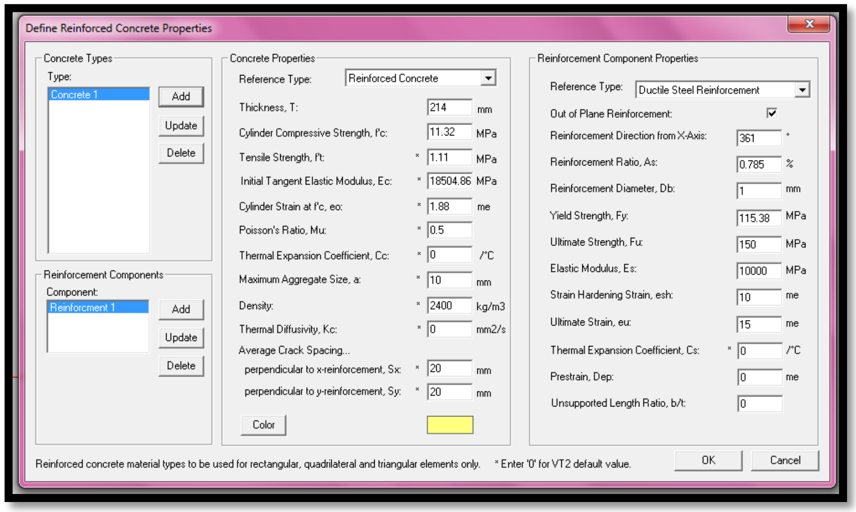

Figure 3.3: Reinforced Concrete Properties dialog box

3. Define Reinforcement properties

- Reinforcement materials types describe steel or FRP reinforcement materials for truss bar elements.
- All the data inserted are based on table 2 the specimen properties.

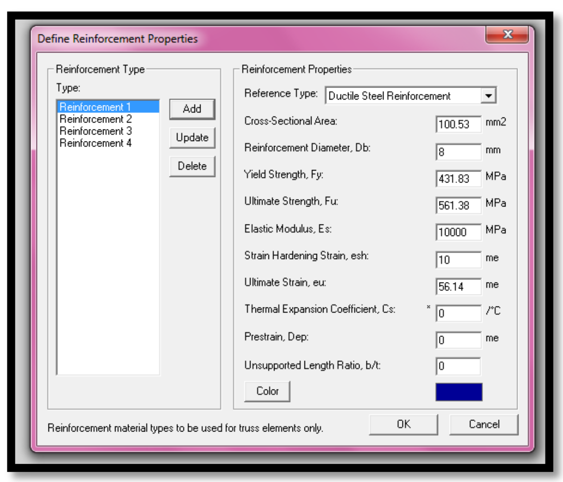

Figure 3.4: Reinforcement Materials Properties dialog box

4. Define Mesh and Structure

- To define the concrete region.

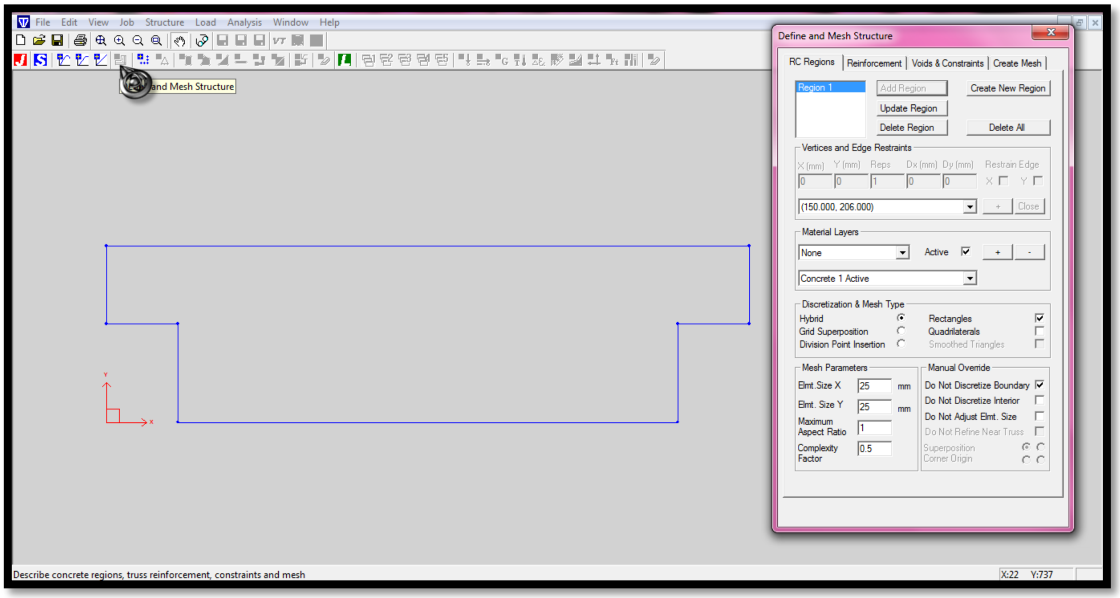

Figure 3.5: RC Region dialog box

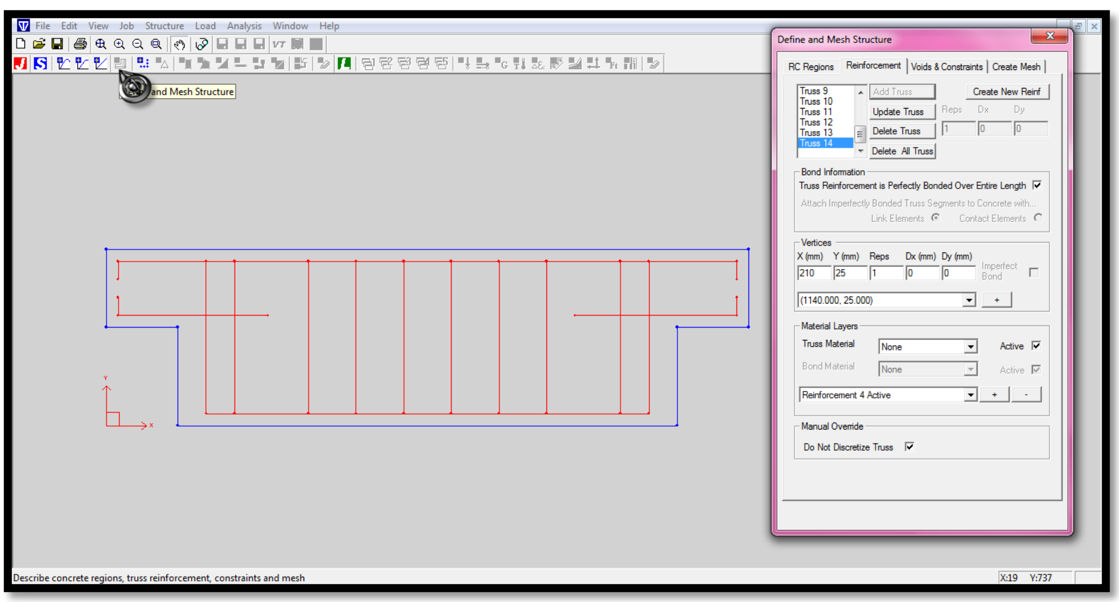

Figure 3.6: The Reinforcement dialog box

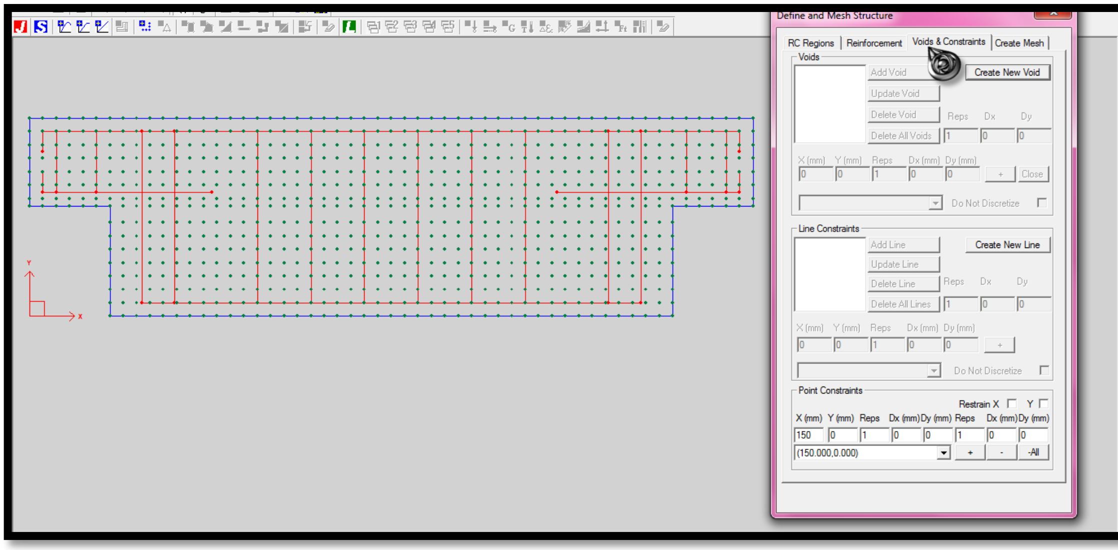

Figure 3.7: Voids and Constraint dialog box

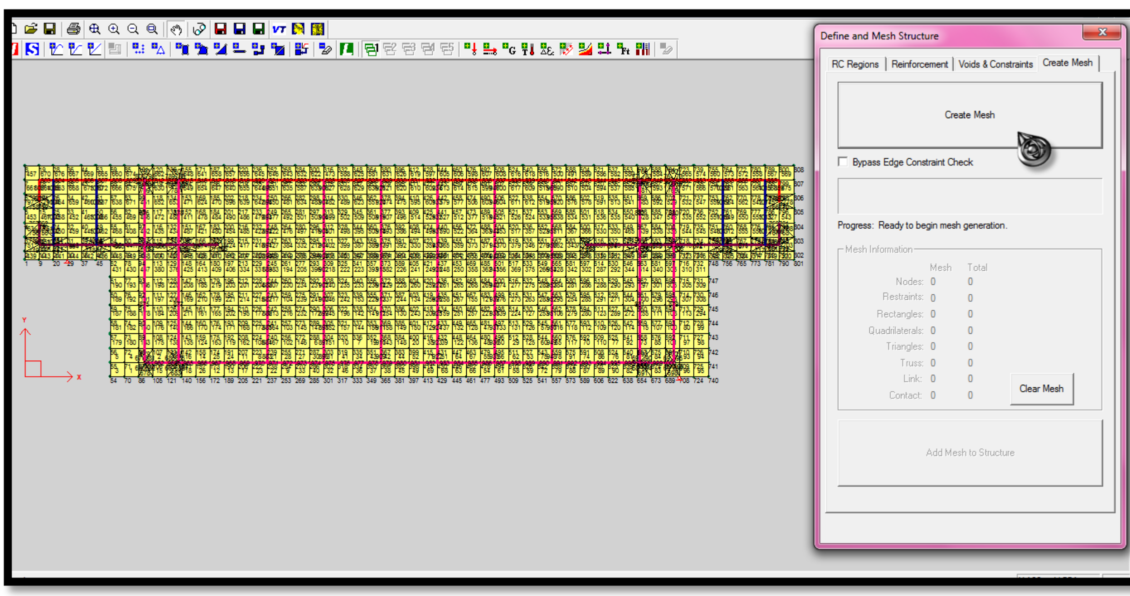

Figure 3.8: Create Mesh dialog box

5. Create support restraints

- Create the support restraint at the selected nodes.

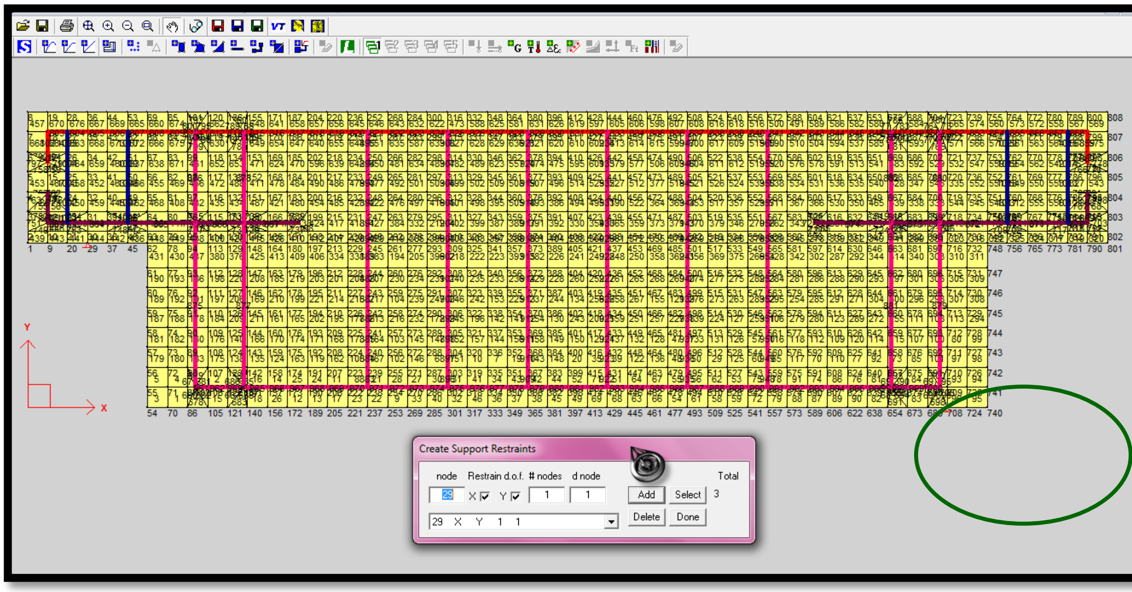

Figure 3.9: Support Restraints dialog box

6. Assign Material Types

- Assign material types for the concrete and steel reinforcement.

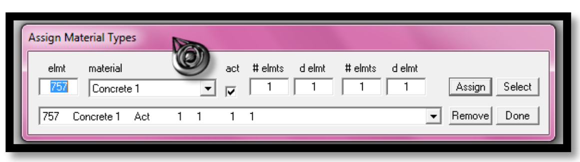

Figure 3.10: Material Types dialog box

7. Apply the load
   - Nodal loads for static loading.
   - Applying the load at the selected nodes.

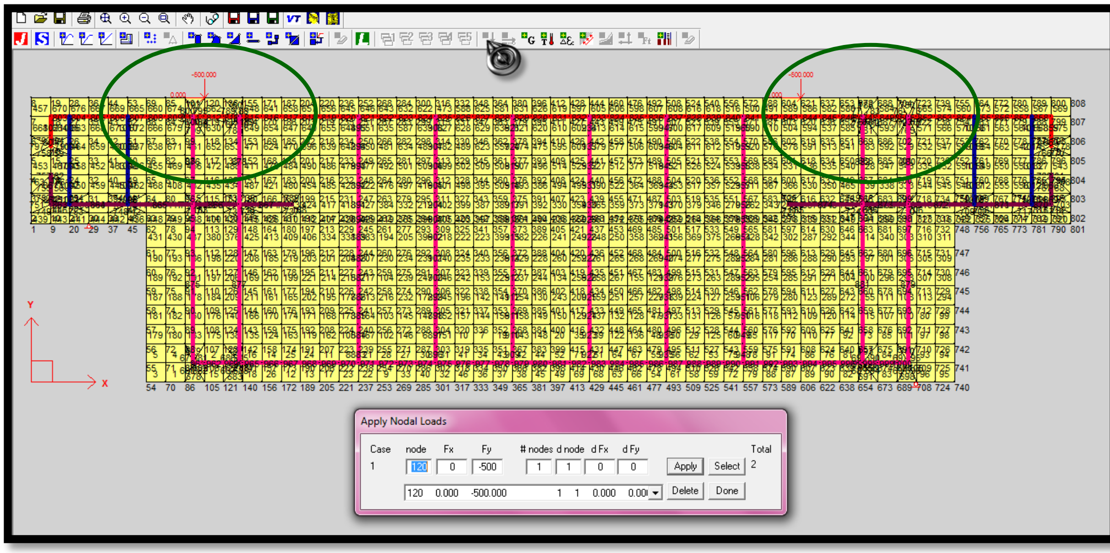

Figure 3.11: Nodal Loads dialog box

8. Saving the file
   - Save job file, save structure file and save load file.

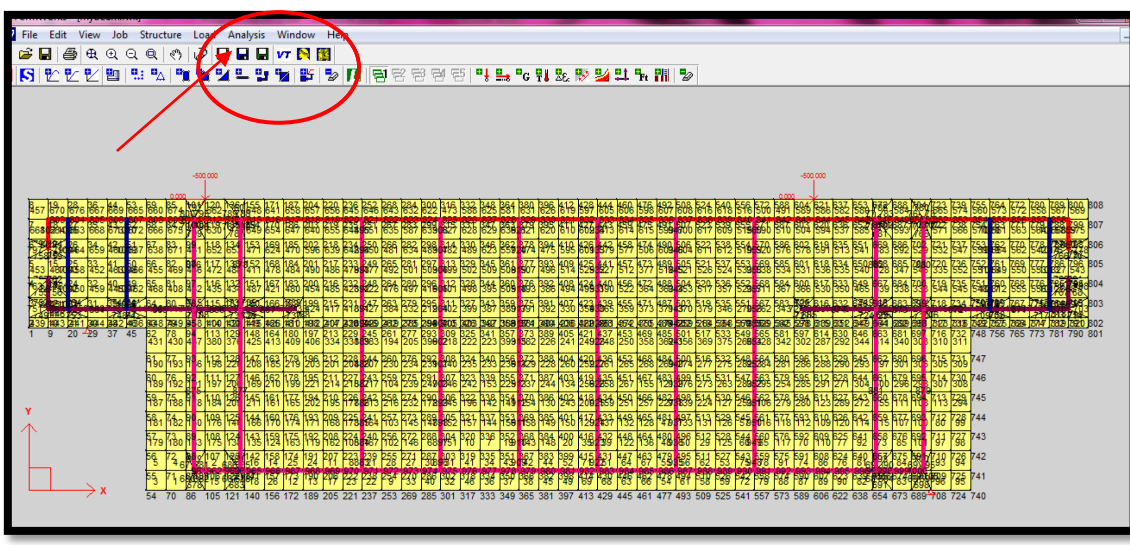

Figure 3.12: Three Save File Icon

9. Run VecTor2 processor

- To allow the system to read, before attempting to open the file in the Augustus Postprocessor.
- Providing there are no errors in the input, the analysis proceeds until all specified load steps are performed, or until the stiffness matrix is no longer invertible.

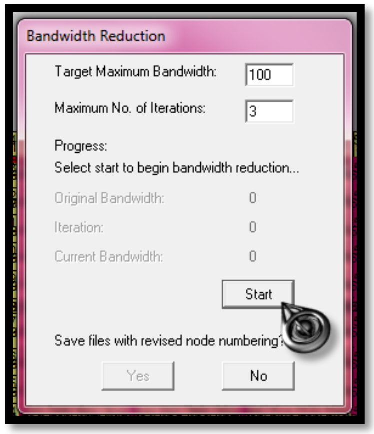

Figure 3.13: Bandwidth Reduction dialog box

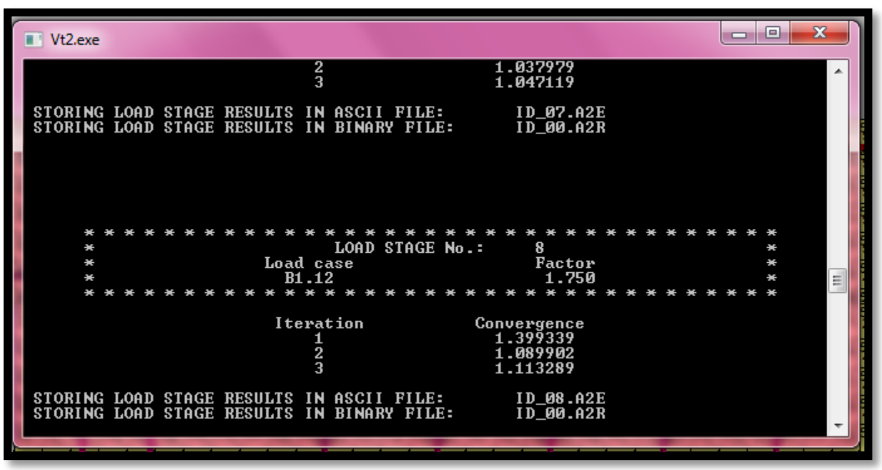

Figure 3.14: VecTor2 Analysis Proceeding

10. Run Augustus postprocessor

- Main function is to allow the input from the FormWorks readable.

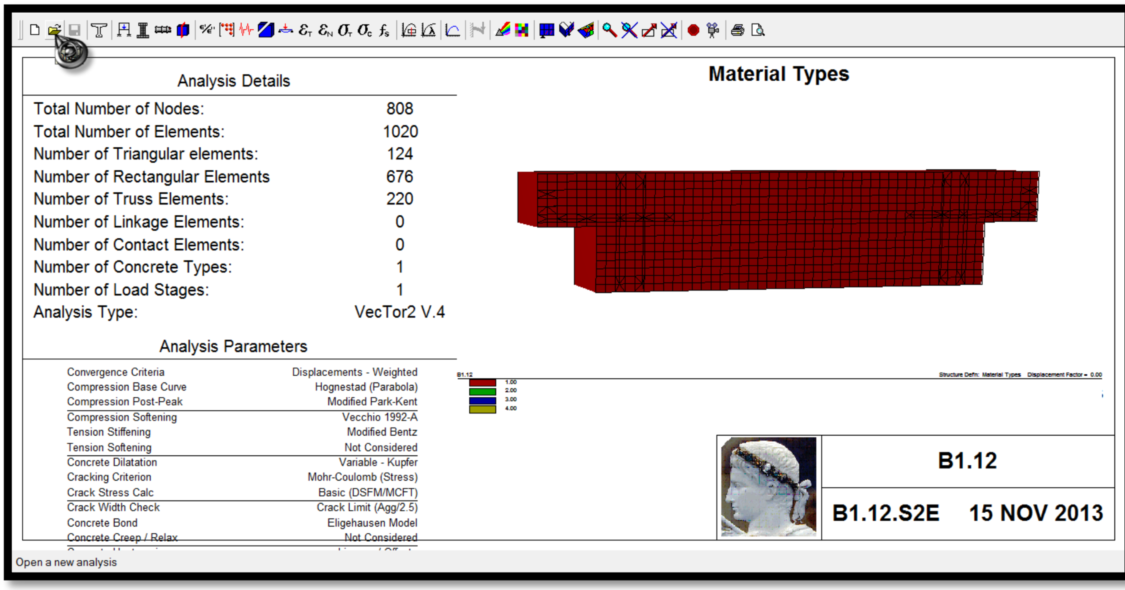

Figure 3.15: Augustus Analysis Details

3.3 PCI Design Procedures

The calculation to predict the failure load of the specimens will mostly refer to the PCI design handbook seventh edition. This is the newly released edition that includes the new and updated information for design guide for Precast and Prestressed concrete structures that provides an easy to follow the design procedures.

The steps of the calculations are as follows:

1. **The Flexure and Axial Tension in Extended End**

$$A_s = A_f + A_n = \frac{1}{\phi f_y}\left[V_u\left(\frac{a}{d}\right) + N_u\left(\frac{h}{d}\right)\right]$$

Where;

ϕ = 0.75
a = shear span, measured from load to center of A_{sh}, in.
h = depth of component above dap, in.
d = distance from top to center of reinforcement A_s, in.
f_y = yield strength of flexural reinforcement, psi
N_u = 0.2 times sustained load portion of V_u unless otherwise calculated (when bearing pads are used),[11] lb

2. **Direct Shear**

- Refers to the potential vertical crack.

$$A_s = \frac{2V_u}{3\phi f_y \mu_e} + A_n$$

Where;

$$A_n = \frac{N_u}{\phi f_y}$$

$$A_h = 0.5(A_s - A_n)$$

ϕ = 0.75
f_y = yield strength of A_s, A_n, A_h, psi

$$\mu_e = \frac{\phi 1000 \lambda b h \mu}{V_u}$$

- The shear strength of the extended end, ***me*** (Equation above) is limited by the maximum values given in the table 1. Whereby, in this case the maximum value used was 3.4.

Case	Crack interface condition	μ^a	Maximum μ_e	Maximum V_u/ϕ
1	Concrete to concrete, cast monolithically	1.4λ	3.4	$0.30\lambda f'_c A_{cr} \leq 1000\lambda A_{cr}$
2	Concrete to hardened concrete, with roughened surface	1.0λ	2.9	$0.25\lambda f'_c A_{cr} \leq 1000\lambda A_{cr}$
3	Concrete placed against hardened concrete not intentionally roughened	0.6λ	Not applicable[b]	$0.20\lambda f'_c A_{cr} \leq 800\lambda A_{cr}$
4	Concrete to steel	0.7λ	Not applicable[b]	$0.30\lambda f'_c A_{cr} \leq 800\lambda A_{cr}$

Table 1: Recommended Shear Friction Coefficients

3. Diagonal Tension at Re-entrant Corner

- Refers to the reinforcement that required resisting the diagonal tension cracking starting from the re-entrant corner.

$$A_{sh} = \frac{V_u}{\phi f_y}$$

Where;

ϕ = 0.75
V_u = applied factored load, lb
A_{sh} = vertical or diagonal bars across potential diagonal tension crack, in.^{2}
f_y = yield strength of A_{sh}, psi

4. Diagonal Tension in the Nib

- Concrete Capacity

$$2bd\lambda\sqrt{f_c'}$$

- Vertical Reinforcement in the nib

$$A_v = \frac{1}{2f_y}\left(\frac{V_u}{\phi} - 2bd\lambda\sqrt{f_c'}\right)$$

3.4 Key Milestone and Gant Chart

Activities	Week
Submission of Progress Report	8
Pre-SEDEX	11
Submission of Draft Report	12
Submission of Dissertation and Technical Paper	13
Oral Presentation	14

Table 2: Key Milestone

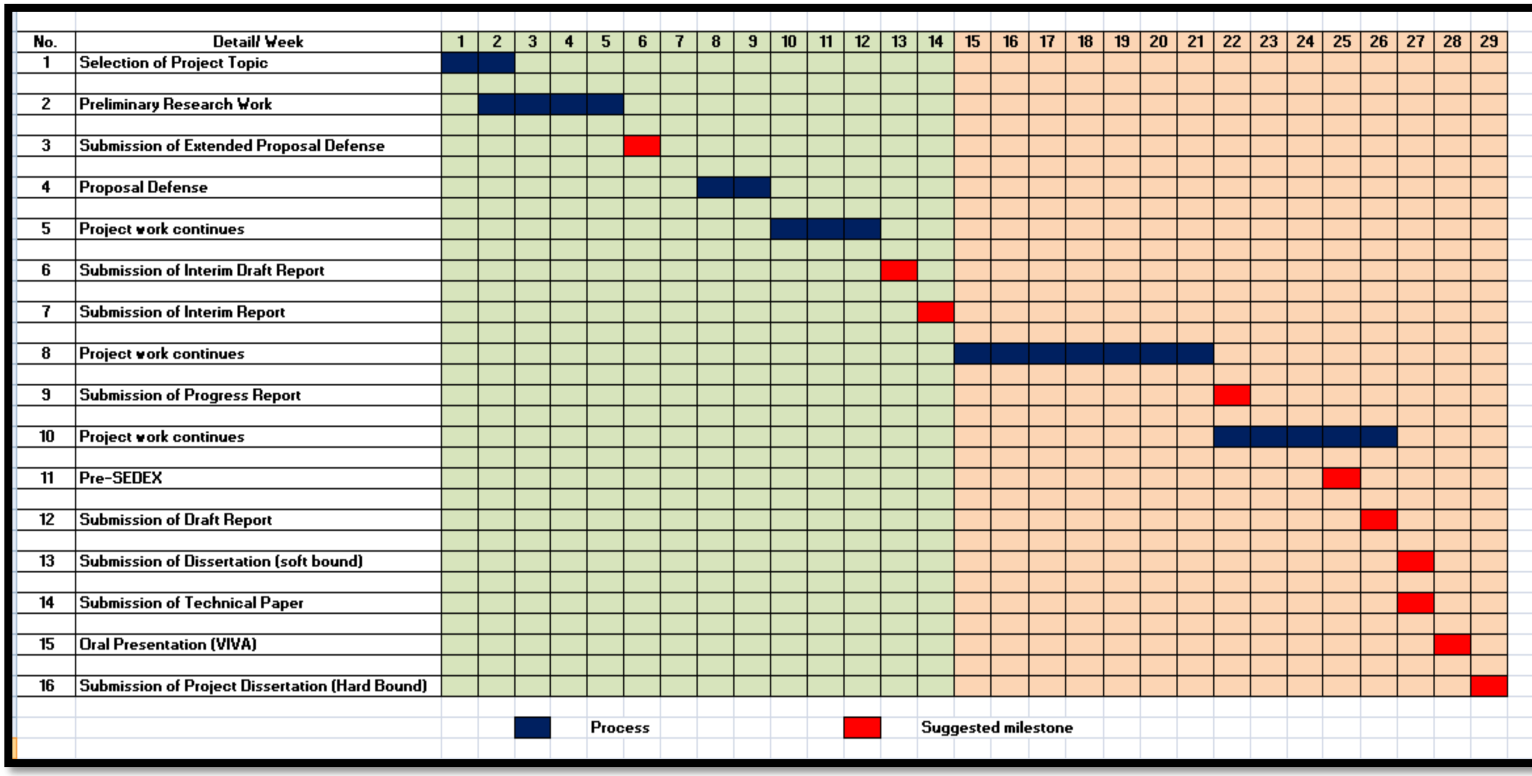

Figure 3.16: Gant Chart

CHAPTER 4

RESULT AND DISCUSSION

4.1 Result

In this study, five dapped-end beams details has been taken out from Lu et al (2003) and Wang et al (2005) research papers. These specimens was used to analyze in VecTor2 and also in the PCI design approach and the results obtain was then compared with data provided from this previous research.

Specimen	f'c (MPa)	b (mm)	h (mm)	hd (mm)	ao (mm)	a/h	a1/hd	ϕSH	ϕS	ϕV	ϕh	ϕN	Exp. Vn (kn)	Source
1	11.32	214	370	164	60	1.2	0.8	0.027	0.118	0.049	0	0	42	Wang et al.(2005), B1.12
2	34	200	600	300	80	0.7	0.5	0.134	0.097	0	0.026	0	561	Lu et al.(2003), Speimen 1
3	33.7	200	600	300	80	0.7	0.5	0.108	0.065	0	0.026	0	458	Lu et al.(2003), Speimen 7
4	62.6	200	600	300	80	0.7	0.5	0.087	0.053	0	0.014	0	705	Lu et al.(2003), Speimen 2
5	62.6	200	600	300	80	0.7	0.5	0.058	0.035	0	0.014	0	599	Lu et al.(2003), Speimen 8

Table 3: List of the specimen details

4.1.1 VecTor2 Analysis

The results of all the specimens based on VecTor2 analysis are shown in the following graphs:

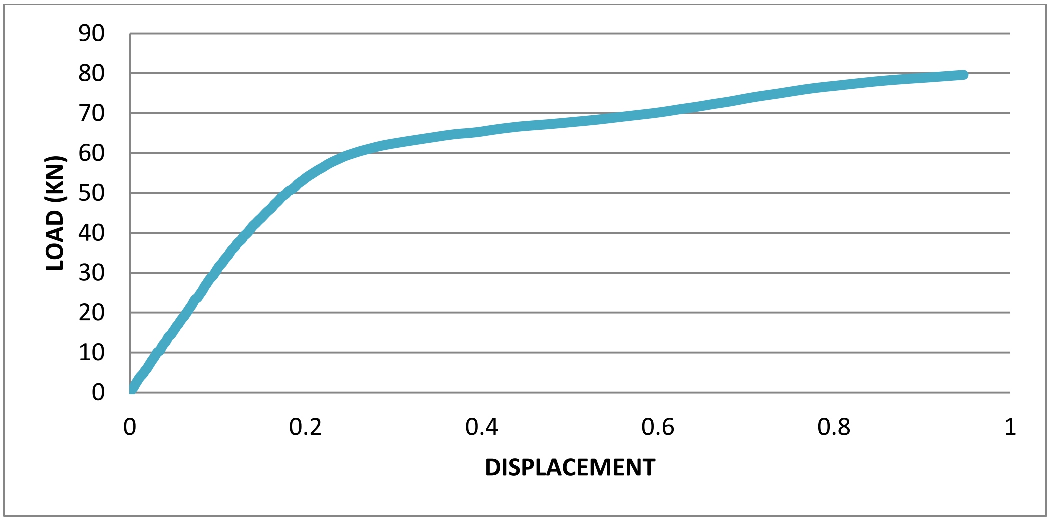

Figure 4.1: Failure Load for **Specimen 1**

Specimen 1 was tested for two failures load on its span, therefore the value of the predicted failure load need to be divided into two in order to get the finalize result for this specimen since from the previous experimental result, its only state for one failure load. Thus the predicted failure load for this specimen is **39.8 KN**.

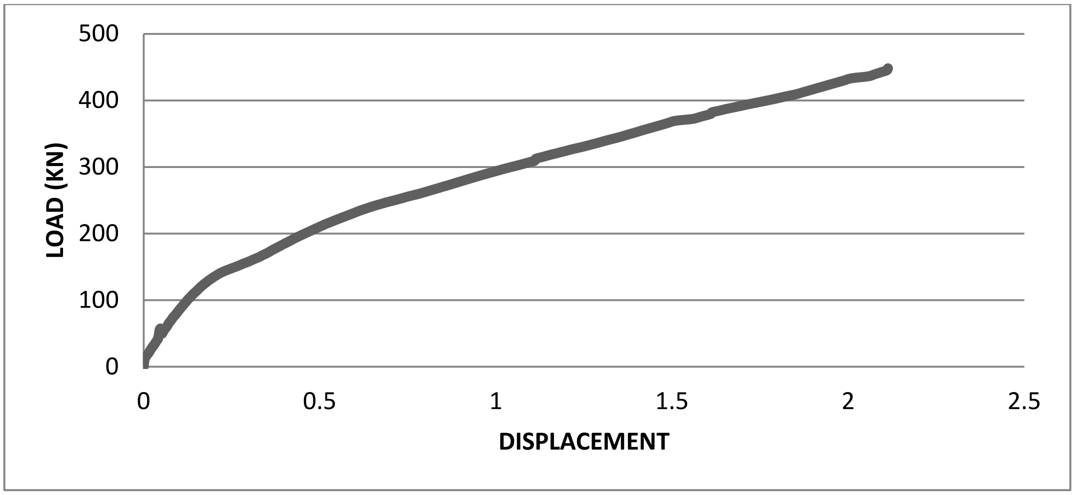

Figure 4.2: Failure Load for **Specimen 2**

Specimen 2 was tested for only one loading which is at 650 mm from the total of 3000 mm long span and the predicted failure load for this specimen is **448.2 KN.**

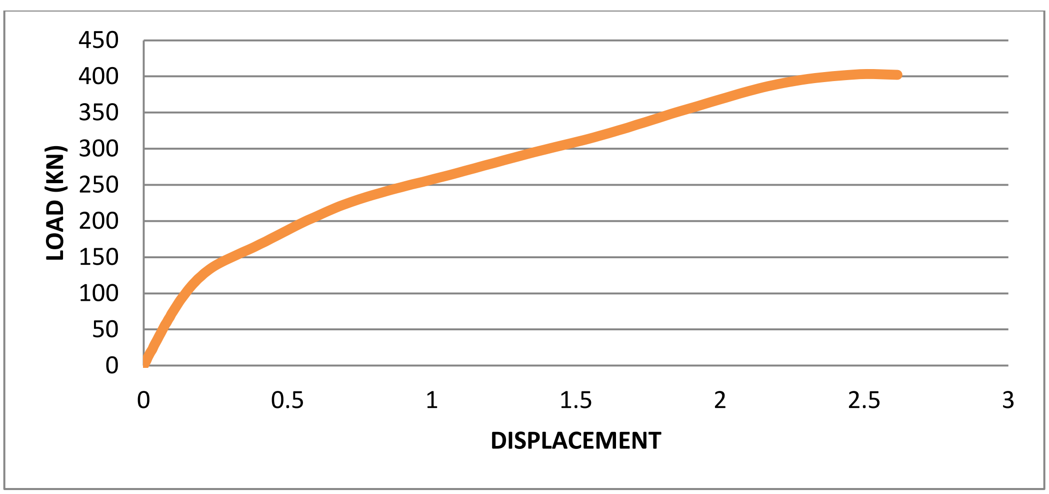

Figure 4.3: Failure Load for **Specimen 3**

Besides that, for Specimen 3, 4 and 5, the procedure was almost the same with Specimen 2 and their failure loads was **402.2 KN, 702.5 KN** and **405 KN**.

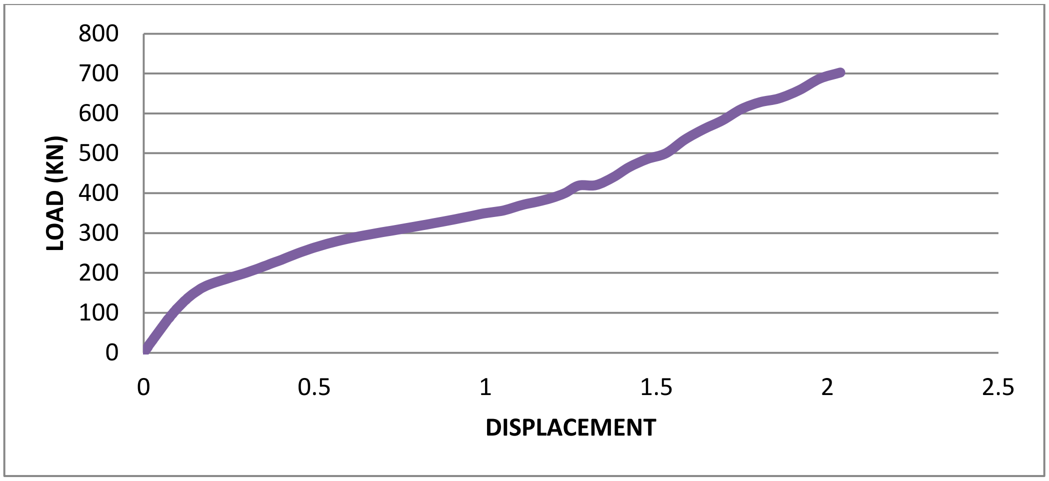

Figure 4.4: Failure Load for **Specimen 4**

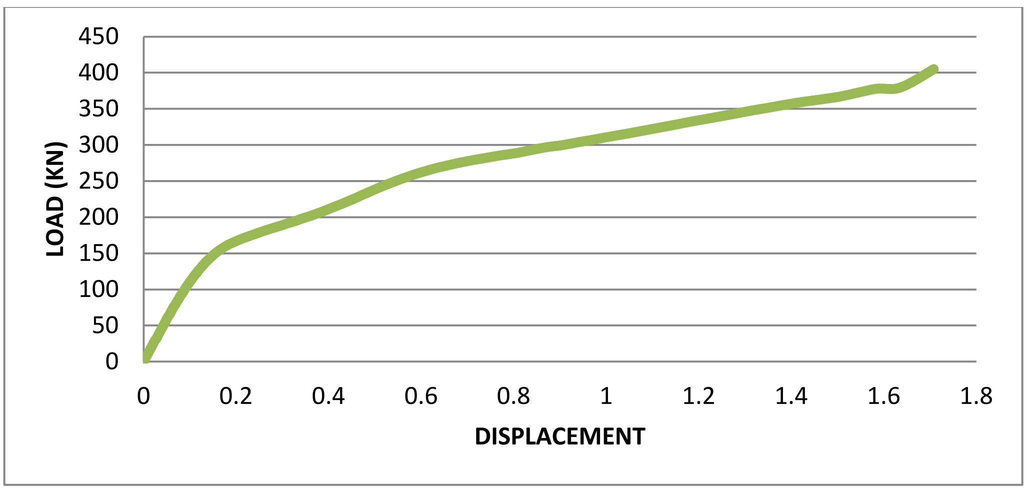

Figure 4.5: Failure Load for **Specimen 5**

4.1.2 PCI 7th Edition Design Provision

Mathematical technique was also needed to support the findings or the results obtain from this computer power method. In this case, the Precast and Prestressed Concrete Institution (PCI) Design Handbook 7th Edition was used as the guideline to perform the analysis (hand calculation) to predict the failure load. Sample of hand calculation for all specimens are attached in Appendices.

4.2 Discussion

The results of all the specimens from the VecTor2 analysis and PCI Design Approach have been gathered in a table as below:

Specimen	Vn : KN			(Vn) Exp./ (Vn) Pre.	
	VecTor2	PCI Design Approach	Experimental	VecTor2	PCI
1	39.8	36.67	42	1.055	1.145
2	448.2	408.38	561	1.252	1.374
3	402.2	327.12	458	1.139	1.4
4	702.5	491.72	705	1.003	1.434
5	405	327.12	599	1.479	1.831

Table 4: Summary of the Results

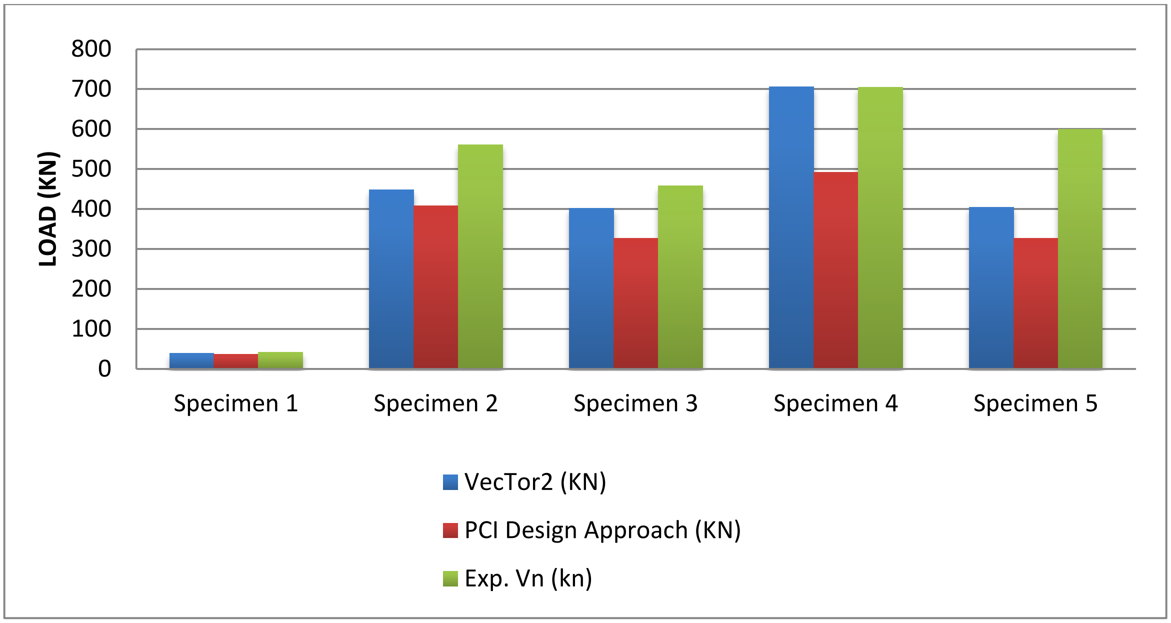

Figure 4.6: Comparison of the Results

Based on this analysis:

- The results obtain from the VecTor2 and PCI Design approach was very close to the provided experimental results.
- Most of the specimens failed at the diagonal tension at re-entrant corner which is at the hanger reinforcement in the nib according to PCI Design Approach.
- However, during conducting the analysis by using the VecTor2, comparatively challenging whereby it's required a lot of effort to master this software especially on part extracting the data or result from the software itself.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion and Recommendation

This research was aim to study the behavior of Reinforced Concrete structures specifically Dapped-end beam. For this purpose, a sophisticated two-dimensional non-linear finite element program, VecTor2 and also the PCI Design handbook was used to conduct the analysis. However, there is uncertainty about the ability of these methods to carry out such analysis. Therefore, main objective was to determine the failure load of the Dapped-end Beam and also to compare the analysis results with the experimental result that has been done by the other researchers. Based on the results, it shows that there is no significant difference between the results obtained from the VecTor2, PCI design provision or from the existing experimental results. Thus, it can be conclude that VecTor2 can be used to perform the analysis of the Reinforced Concrete Dapped-end Beam.

As for recommendations, the author suggested to do further investigate to increase more findings and to make the data more accurate. It is better to have more research in this area of study because it is now increasing practiced in structural engineering as being quicker, more economical, and allow more data to be taken than the other present methods.

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APPENDICES

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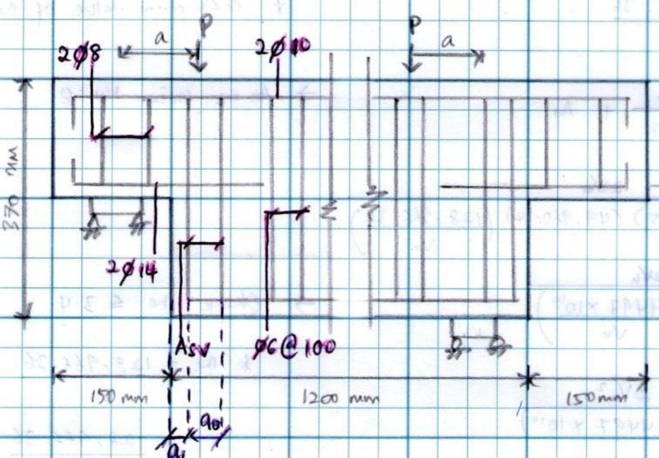
PCI Design Analysis
Subject : FYP II (Specimen 1)

Based on PCI Design Handbook, 7th Edition (2010)

* the predicted failure load for specimen 1 =



Given Data ;
f_{cu} = 16.90 mpa
f_c = 11.32 mpa
b = 214 mm
h = 370 mm
h_1 = 164 mm
h_{10} = 139 mm
a_1 = 60 mm
a_0 = 60 mm

a/h = 1.2
a_1/h_d = 0.8
ϕ_{SH} = 0.027
ϕ_s = 0.118
ϕ_v = 0.049
ϕ_h = 0

$\therefore a/h = 1.2$

$a/164 = 1.2$

$a = 1.2(164)$

$= 196.8$ mm

a) the flexure and axial tension in the rib.

$* A_s = [V_u . (a/d) + N_u . (h/d)] / (\phi . f_y)$

$0.48 = [V_u . (1.42) + 0] / (0.75 \times 49{,}960.41)$

$0.48 = \dfrac{1.42 V_u}{37{,}470.31}$

$V_u = \dfrac{17985.75}{1.42} = 12666.02$ lbs

$\therefore V_u = (12666.02 \times 4.45 \times 10^{-3})$

$= 56.36$ kN #

$\rightarrow A_s = 2(\frac{1}{4} \times \pi \times 14^2)$

$= 307.88$ mm^2

$= [307.88 \times 10^{-2} \times 0.155]$

$= 0.48$ in^2

$\rightarrow f_y = 344.23$ mpa $\times 145.137$

$= 49{,}960.41$ psi

$\rightarrow h/d = 164/139 = 1.18$

$\rightarrow a/d = 196.8/139 = 1.42$

$\rightarrow N_u = 0, \ \phi = 0.75$

b) Direct shear

$\rightarrow$ $\mu e = \dfrac{1000\ \phi\ \lambda\ b h\ \mu}{V_u} \leq 3.4$

$\mu e = \dfrac{1000\ (0.75)(1)(8.43)(14.57)(1.4)}{V_u}$

$\mu e = \dfrac{128{,}966.36}{V_u}$

$\rightarrow$ $b = 214$ mm $= 8.43$ in
$h = 370$ mm $= 14.57$ in
$\mu = 1.4$
$\phi = 0.75$
$\lambda = 1$

* maximum value of $\mu e = 3.4$.

$\rightarrow$ $A_s = \dfrac{2V_u}{3\phi\ f_y\ \mu e} + A_n$

$\rightarrow$ $A_n = 0$ since $N_u = 0$

$0.48 = \dfrac{2V_u}{3(0.75)(49{,}960.14)\left(\dfrac{128{,}966.36}{V_u}\right)}$

$0.48 = \dfrac{2V_u}{\left(\dfrac{1.4497\times10^{10}}{V_u}\right)}$

$0.48 = \dfrac{2V_u^2}{(1.4497\times10^{10})}$

$V_u^2 = 3{,}479{,}334.600$
$V_u = 58985.88$ lbs
$= [58985.88\times(4.45\times10^{-3})]$
$= 262.49$ kN #

$\rightarrow$ Check $\mu e \leq 3.4$

* $\mu e = \dfrac{128{,}966.36}{V_u}$

$= \dfrac{128{,}966.36}{58985.88}$

$= 2.19 \leq 3.4$. (ok).

c) ~~Direct shear~~ Diagonal Tension at Re-entrant Corner:

$\rightarrow$ Hanger Reinforcement. $(1\phi 6)$

$\phi = 6$ mm $= 0.24$ in (2 nos ; 2 legs)

$A_s = 2[2(\frac{1}{4}\times\pi\times6^2)] = 113.097$ mm^2
$= [113.097\times10^{-2}\times0.155]$ in^2
$= 0.175$ in^2.

* $A_s = V_u / (\phi f_y)$
$V_u = A_s(\phi f_y)$
$= 0.175(0.75\times62{,}674.51)$
$= 8240.17$ lbs
$= [8240.17\times(4.45\times10^{-3})]$ kN
$= 36.67$ kN #.

$\therefore$ f_y value, given
$A_{sv} f_{yv} = 24.42$ kN.

$\rightarrow$ $A_s = 1[2\times\frac{1}{4}\times\pi\times6^2]$
$= 56.55$

$\rightarrow$ $A_s f_y = 24.42\times10^3$

$f_y = \dfrac{24.42\times10^3}{56.55}$
$= 431.83$ mpa

$\therefore f_y = 431.83\times145.137$
$= 62{,}674.51$ psi

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Subject : Specimen 1

d) Diagonal tension in the rib.

i) Concrete Capacity $= 2\lambda bd\sqrt{f'c}$

$= 2(1)(8.43)(6.46)(\sqrt{1642.95})$

$= 4414.71$ lbs

→ $h = d = 164$ mm
$= 6.46$ in

→ $f'c = 11.32$ mpa
$= (11.32 \times 145.137)$
$= 1642.95$ psi.

ii) Vertical Reinforcement in the rib.

$$Av = \frac{(Vu/\emptyset) - 2\lambda bd\sqrt{f'c}}{2fy}$$

$$2Avfy = (Vu/\emptyset) - (2\lambda bd\sqrt{f'c})$$

$$\therefore \quad Vu = \emptyset[2Avfy + (2\lambda bd\sqrt{f'c})]$$

$$= 0.75[2(0.312)(62,674.51) + 4414.71]$$

$$= 0.75[39108.89 + 4414.71]$$

$$= 32642.70 \text{ lbs}$$

$$= [32642.70 \times (4.45 \times 10^{-3})] \text{ kN}$$

$$= 145.26 \text{ kN } *$$

$\therefore \emptyset = 8$ mm $= 0.31$ in
[2 nos ; 2 legs]
$Av = 2[2(\frac{1}{4} \times \pi \times 8^2)]$
$= 201.06$ mm^2
$= [201.06 \times 10^{-2} \times 0.155]$
$= 0.312$ in^2

$fy = 431.83 \times 145.137$
$= 62,674.51$ psi.

∴ thus, the predicted failure load for this specimen is 36.67 kN (smallest value). It can be conclude that this specimen was failed at the diagonal tension at re-entrant corner which is at the hanger reinforcement.

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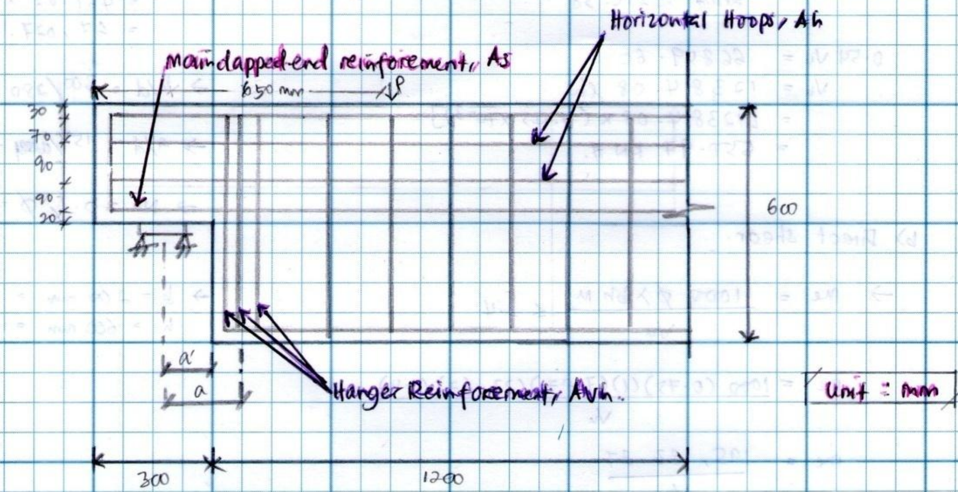
PCI Design Analysis

Subject : FYP II (specimen 2)

Based on PCI Design Handbook, 7th Edition (2010).

* The predicted failure load for specimen 2 :



Given Data;

	Main-Dapped end Reinforcement	Horizontal Hoops	Hanger Reinforcement
Bars	3 - #6	2 - #3	5 - #4
As (mm²)	859.6	285.3	1266.8
fy (MPa)	461.82	368	430.14
P.fy (MPa)	7.39	1.95	-

Additional Data;

$f'c = 34$ Mpa	$b = 200$ mm	$\phi_{SH} = 0.134$
$a/d = 0.56$	$h = 600$ mm	$\phi_s = 0.097$
$a = 150$ mm	$a/h = 0.7$	$\phi_v = 0$
$a' = 80$ mm	$a_1/hd = 0.5$	$\phi_h = 0.026$.

a) the flexure and axial tension in the rib.

* $A_s = [V_u \cdot (a/d) + N_u \cdot (h/d)] / (\phi \cdot f_y)$

$1.33 = [V_u \cdot (0.54) + 0] / (0.75 \times 67{,}027.17)$

$1.33 = \dfrac{0.54\, V_u}{50270.38}$

$0.54\, V_u = 66859.60$

$V_u = 123814.08$ lbs

$= [123814.08 \times (4.45 \times 10^{-3})]$

$= 550.97$ KN #.

→ $A_s = 859.6\ mm^2$

$= (859.6 \times 10^{-2} \times 0.155)$

$= 1.33\ in^2$.

→ $F_y = 461.82$ mpa

$= 461.82 \times 145.137$

$= 67{,}027.17$ psi

→ $h/d = 300/280 = 1.07$

→ $a/d = 150/280 = 0.54$

→ $N_u = 0$, $\phi = 0.75$

b) Direct shear.

→ $m_e = \dfrac{1000\ \phi \times bh\ \mu}{V_u} \leq 3.4$

→ $b = 200$ mm $= 7.87$ in

$h = 600$ mm $= 23.62$ in

$m_e = \dfrac{1000\ (0.75)(1)(7.87)(23.62)(1.4)}{V_u}$

$m_e = \dfrac{195{,}183.87}{V_u}$

→ $A_s = \dfrac{2V_u}{3\phi\, f_y\, m_e} + \cancel{A_n}^{\,0}$

→ $A_n = 0$ since $N_u = 0$.

$1.33 = \dfrac{2V_u}{3(0.75)(67{,}027.17)\left(\dfrac{195{,}183.87}{V_u}\right)}$

$1.33 = \dfrac{2V_u^2}{(2.94 \times 10^{10})}$

$2V_u^2 = 3.915 \times 10^{10}$

$V_u = 139910.23$ lbs

$= 622.60$ KN #.

* check $m_e \leq 3.4$.

→ $m_e = \dfrac{195{,}183.87}{V_u}$

$= \dfrac{195{,}183.87}{139910.23}$

$= 1.4 \leq 3.4$ _(ok).

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dent's No : 15743 Subject : specimen 2 .

c) Diagonal Tension at Re-entrant

* Hanger Reinforcement

→ $A_{vh} = 1266.8 \text{ mm}^2 = [1266.8 \times 10^{-2} \times 0.155] = 1.96 \text{ in}^2$
$f_{yvh} = 430.14 \text{ MPa} = 430.14 \times 145.137 = 62429.23$ psi.

* $A_{vh} = \dfrac{V_u}{(\phi f_{yvh})}$, $\phi = 0.75$

$V_u = A_{vh} (\phi f_{yvh})$

$V_u = 1.96 (0.75 \times 62429.23)$

$= 91770.97$ lbs

$= [91770.97 \times (4.45 \times 10^{-3})]$

$= 408.38$ kN #.

* - thus, the predicted failure load for this specimen is 408.38 kN #.
- Failed at diagonal tension at re-entrant corner.

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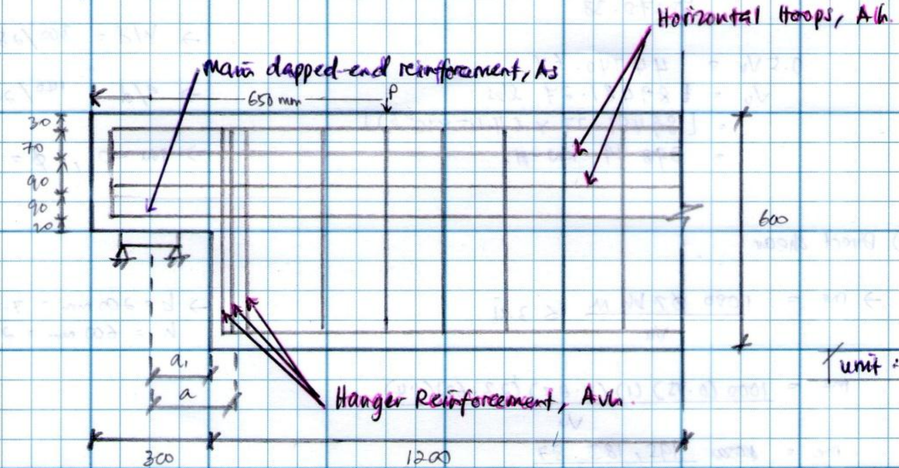
PCI Design Analysis
Subject : FYP II (specimen 3)

Based on PCI Design Handbook, 7th Edition (2010)

* The predicted failure load for specimen 3 :



unit : mm

Given Data ;

	Main dapped-end Reinforcement	Horizontal Hoops	Hanger Reinforcement
Bars	2 - #6	2 - #3	4 - #4
As (mm²)	573.0	285.3	1013.4
fy (mpa)	461.82	368	430.14
Pfy (mpa)	5.08	1.95	-

Additional Data ;

$f'c$ = 33.7 Mpa.
a/d = 0.52
a = 140 mm
a' = 80 mm

b = 200 mm
h = 600 mm
hd = 300 mm
a/h = 0.7
a_1/hd = 0.5

ϕ_{SH} = 0.108
ϕ_S = 0.065
ϕ_V = 0
ϕ_h = 0.026
ϕ_N = 0.

a) the Flexure and axial tension in the rib.

$\rightarrow A_s = 573.0 \text{ mm}^2$
$= [573 \times 10^{-2} \times 0.155]$
$= 0.89 \text{ in}^2$

$\rightarrow F_y = (461.82 \times 145.137)$
$= 67,027.17 \text{ psi}$

$\rightarrow h/d = 300/280 = 1.07$

$\rightarrow a/d = 140/280 = 0.5$

$\rightarrow N_u = 0, \ \phi = 0.75$.

$* \ A_s = [V_u (a/d) + N_u (h/d)] / (\phi f_y)$

$0.89 = [V_u.(0.5) + 0] / (0.75 \times 67,027.17)$

$0.89 = \dfrac{0.5 V_u}{50270.38}$

$0.5 V_u = 44740.64$

$V_u = 89481.27 \text{ lbs}$

$= [89481.27 \times (4.45 \times 10^{-3})]$

$= 398.19 \text{ kN } \#.$

b) Direct shear.

$\rightarrow b = 200 \text{ mm} = 7.87 \text{ in}$
$h = 600 \text{ mm} = 23.62 \text{ in}$

$\rightarrow m_e = \dfrac{1000 \ \phi \lambda b h \mu}{V_u} \leq 3.4$

$m_e = \dfrac{1000 (0.75)(1)(7.87)(23.62)(1.4)}{V_u}$

$m_e = \dfrac{195,183.87}{V_u}$

$\rightarrow A_s = \dfrac{2V_u}{3 \phi f_y m_e} + A_n$

$\rightarrow A_n = 0$ since $N_u = 0$.

$0.89 = \dfrac{2V_u}{3(0.75)(67,027.17)\left(\dfrac{195,183.87}{V_u}\right)}$

$0.89 = \dfrac{2V_u^2}{(2.94 \times 10^{10})}$

$0.89 (2.94 \times 10^{10}) = 2V_u^2$

$2V_u^2 = 2.6166 \times 10^{10}$

$V_u = 114380.94 \text{ lbs}$

$= [114380.94 \times (4.45 \times 10^{-3})]$

$= 508.995 \text{ kN } \#.$

To check $m_e \leq 3.4$:

$\rightarrow m_e = \dfrac{195,183.87}{V_u}$

$= \dfrac{195,183.87}{114380.94}$

$= 1.7 \leq 3.4$, (OK)

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ne : Sarah Joy Bt. Nur

dent's No : 15743 Subject : specimen 3.

c) Diagonal tension of Re-entrant

* Hanger Reinforcement

→ A_{vh} = 1013.4 mm² = [1013.4 x 10⁻² x 0.155] = 1.57 in².

→ f_{yvh} = 430.14 mpa = (430.14 x 145.137) = 62,429.23 psi.

* $A_{vh} = \frac{V_u}{\phi f_{yvh}}$, $\phi = 0.75$

$V_u = A_{vh} (\phi f_{yvh})$

$V_u = 1.57 (0.75 \times 62429.23)$

$= 73510.42$ lbs.

$= [73510.42 \times (4.45 \times 10^{-3})]$

$= 327.12$ kN #.

. Thus, the predicted failure load for this specimen is 327.12 kN
- Failed at the hanger reinforcement in the rib.

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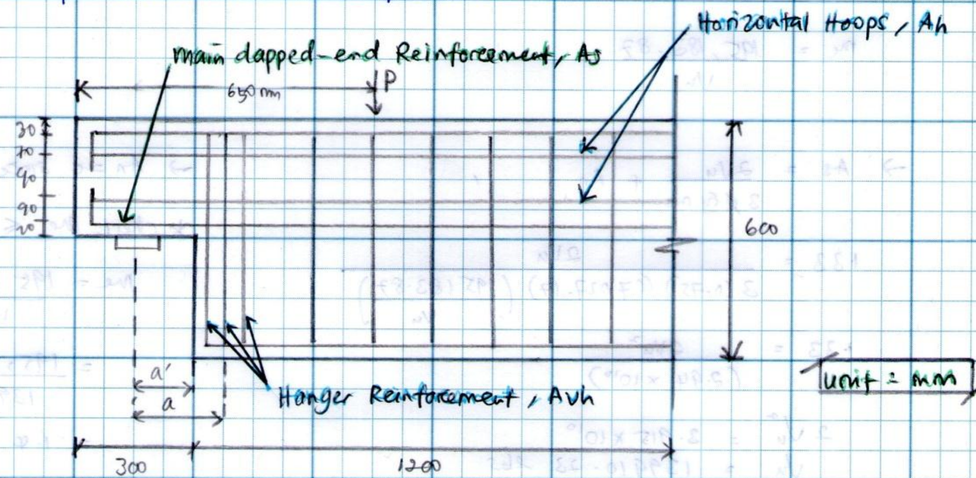
Name : Sarah Joy Binti Nur — PCI Design Analysis
Student's No : 15743 — Subject : FYP II (specimen 4)

Based on PCI Design Handbook, 7th Edition (2010)

* The predicted failure load for specimen 4 :



Given Data,

	Main dapped-end Reinforcement	Horizontal Hoops	Hanger Reinforcement
Bar	3 - #6	2 - #6	6 - #4
As (mm^2)	859.6	285.3	1520.2
Fy (Mpa)	461.82	368.0	430.14
f_y (Mpa)	7.39	1.95	-

Additional Data,

$f'c = 62.6$ Mpa $a/d = 0.56$ $\phi sh = 0.087$

$b = 200$ mm $a/h = 0.7$ $\phi s = 0.053$

$h = 600$ mm $a'/hd = 0.5$ $\phi h = 0.014$

$a' = 80$ mm $\phi v = 0.$

$a = 160$ mm

a) the Flexure and Tension in the rib.

$* \; A_s = [V_u . (a/d) + N_u (h/d)] / (\phi . f_y)$

$1.33 = [V_u . (0.57) + 0] / (0.75 \times 67\,027.14)$

$$1.33 = \frac{0.57 V_u}{50\,270.38}$$

$V_u = 117\,297.54$ lbs

$= 521.97$ kN #

$\rightarrow A_s = 859.6 \; mm^2$

$= 1.33 \; in^2$

$\rightarrow F_y = 461.82$ mpa

$= 67\,027.17$ psi

$\rightarrow h/d = 300/280 = 1.07$

$a/d = 160/280 = 0.57$

b) Direct shear.

$\rightarrow \mu_e = \dfrac{1000\ \phi \lambda b h \mu}{V_u} \leqslant 3.4$

$\rightarrow b = 200\ mm = 7.87\ in$
$h = 600\ mm = 23.62\ in.$

$\mu_e = \dfrac{1000\ (0.75)(1)(7.87)(23.62)(1.4)}{V_u}$

$\mu_e = \dfrac{195{,}183.87}{V_u}$

$\rightarrow A_s = \dfrac{2V_u}{3 \phi f_y \mu_e} + A_n$,

$\rightarrow A_n = 0$ since $N_u = 0$

$1.33 = \dfrac{2V_u}{3(0.75)(67027.17)\left(\dfrac{195183.87}{V_u}\right)}$

$1.33 = \dfrac{2V_u^2}{(2.94 \times 10^{10})}$

$2V_u^2 = 3.915 \times 10^{10}$

$V_u = 139910.23$ lbs

$= 622.60$ kN #.

* Check $\mu_e \leqslant 3.4$:

$\mu_e = \dfrac{195\ 183.87}{V_u}$

$= \dfrac{195{,}183.87}{139910.23}$

$= 1.4 \leq 3.4$, OK.

c) Diagonal Tension at Re-entrant:
* Hanger Reinforcement:

$\rightarrow A_{vh} =$ ~~[illegible]~~ $1520.2\ mm^2 = [1520.2 \times 10^{-2} \times 0.155] = 2.36\ in^2$
$f_{yvh} = 430.4\ mpa = 430.4 \times 145.137 = 62429.23\ psi$

* $A_{vh} = \dfrac{V_u}{\phi f_{yvh}}$

$V_u = A_{vh}(\phi f_{yvh})$

$V_u = 2.36\ (0.75 \times 62429.23)$

$V_u = 110499.74$ lbs

$= 491.72$ kN #.

* This specimen failed at Diagonal Tension at re-entrant corner with failure load of 491.72 kN #.

Civil Engineering Department
Universiti Teknologi PETRONAS

ame : Sarah Joy Binti Nur

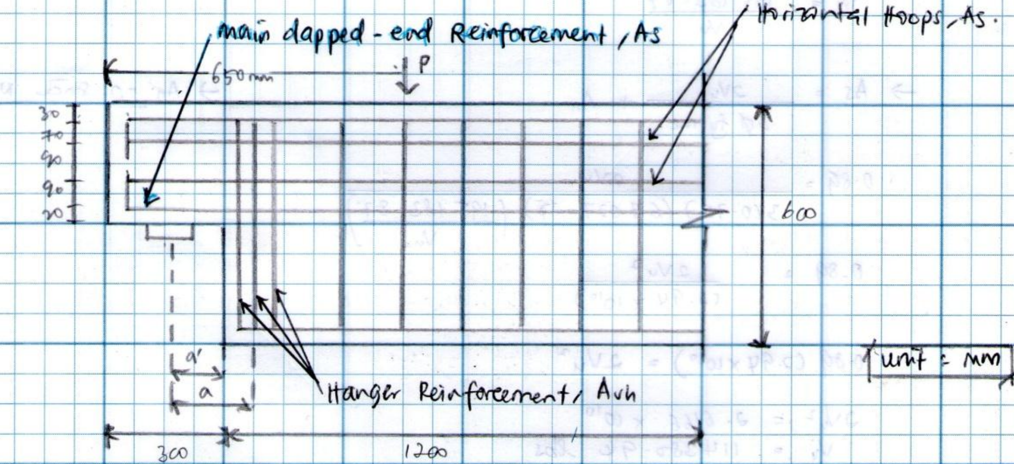
PCI Design Analysis

tudent's No : 15743

Subject : FYP II (specimen 5)

Based on PCI Design Handbook, 7th Edition (2010)

* the predicted failure load for specimen 5 :



Given Data ;

	main Dapped-end Reinforcement	Horizontal Hoops	Hanger Reinforcement
Bars	2 - #6	2 - #3	4 - #4
As (mm²)	573	285.3	1012.4
Fy (MPa)	461.82	368	430.14
Pfy (MPa)	5.08	1.95	-

Additional Data ;

$f'c$ = 62.6 MPa
a = 145 mm
a' = 80 mm
a/d = 0.52

b = 200 mm
h = 600 mm
hd = 300 mm
a/h = 0.7

a_1/hd = 0.5
ϕ_{sH} = 0.058
ϕ_s = 0.035
ϕ_h = 0.014.

a) the Flexure and axial tension in the nib.

$$* \; A_s = [V_u \cdot (a/d) + N_u (h/d)] / (\phi f_y)$$

$$0.89 = [V_u \cdot (0.52) + 0] / (0.75 \times 67,027.17)$$

$$0.89 = \frac{0.52 V_u}{50270.38}$$

$$V_u = 89481.27 \text{ lbs}$$
$$= 398.19 \text{ kN} \#$$

→ A_s = 573.0 mm²
= [573 × 10^{-2} × 0.155]
= 0.89 in².

→ f_y = (461.82 × 145.137)
= 67,027.17 psi.

→ h/d = 300/280 = 1.07
a/d = 140/280 = 0.52
→ N_u = 0, ϕ = 0.75

b) Direct shear:

$\rightarrow \mu_e = \dfrac{1000\,\phi\,\lambda\, bh\,\mu}{V_u} \leq 3.4$

$\rightarrow b = 200\text{ mm} = 7.87\text{ in}$
$h = 600\text{ mm} = 23.62\text{ in}$

$\mu_e = \dfrac{1000\,(0.75)(1)(7.87)(23.62)(1.4)}{V_u}$

$\mu_e = \dfrac{195\,183.87}{V_u}$

$\rightarrow A_s = \dfrac{2V_u}{3\phi\, f_y\, \mu_e} + A_n$

$\rightarrow A_n = 0$ since $N_u = 0$.

$0.89 = \dfrac{2V_u}{3(0.75)(67\,027.17)\left(\dfrac{195\,183.87}{V_u}\right)}$

$0.89 = \dfrac{2V_u^2}{(2.94 \times 10^{10})}$

$0.89\,(2.94 \times 10^{10}) = 2V_u^2$

$2V_u^2 = 2.6166 \times 10^{10}$
$V_u = 114380.94\text{ lbs}$
$= [11438.94 \times (4.45 \times 10^{-3})]$
$= 508.995\text{ kN}$ #.

* check $\mu_e \leq 3.4$:

$\rightarrow \mu_e = \dfrac{195\,183.87}{V_u}$
$= \dfrac{195\,183.87}{114380.94}$
$= 1.7 \leq 3.4$, OK.

c) Diagonal tension at Re-entrant
* Hanger Reinforcement

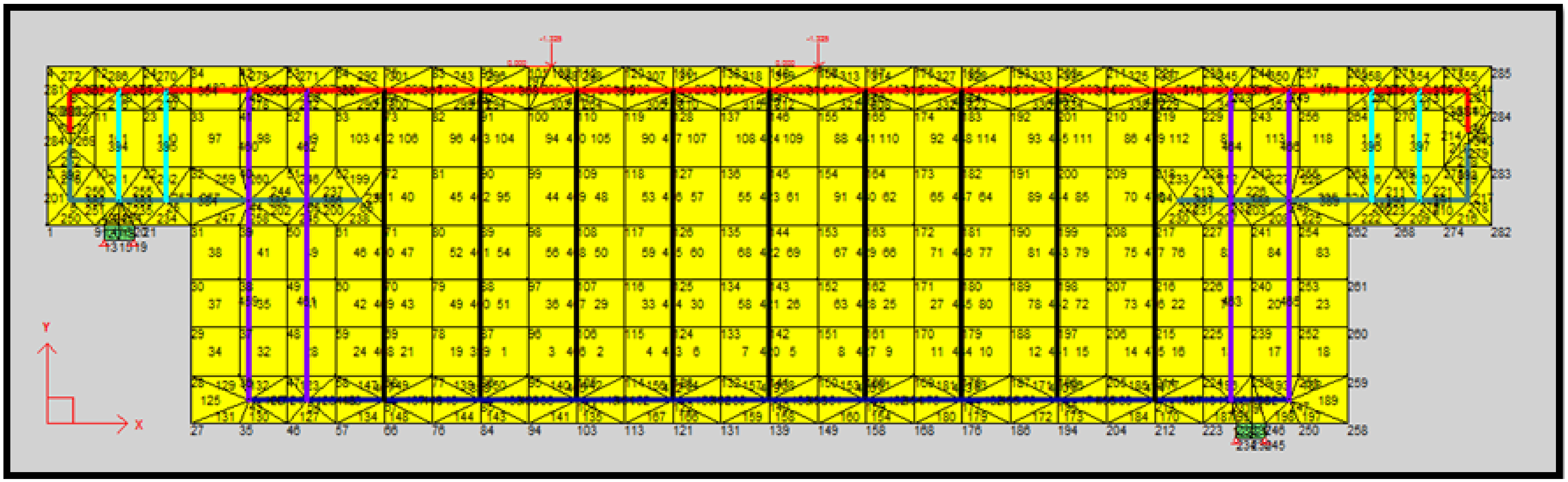
$\rightarrow A_{vh} = 1013.4\text{ mm}^2 = 1.57\text{ in}^2$
$f_{yvh} = 430.14\text{ mpa} = 62\,429.23\text{ psi}$

* $A_{vh} = \dfrac{V_u}{\phi\, f_{yvh}}$, $\phi = 0.75$

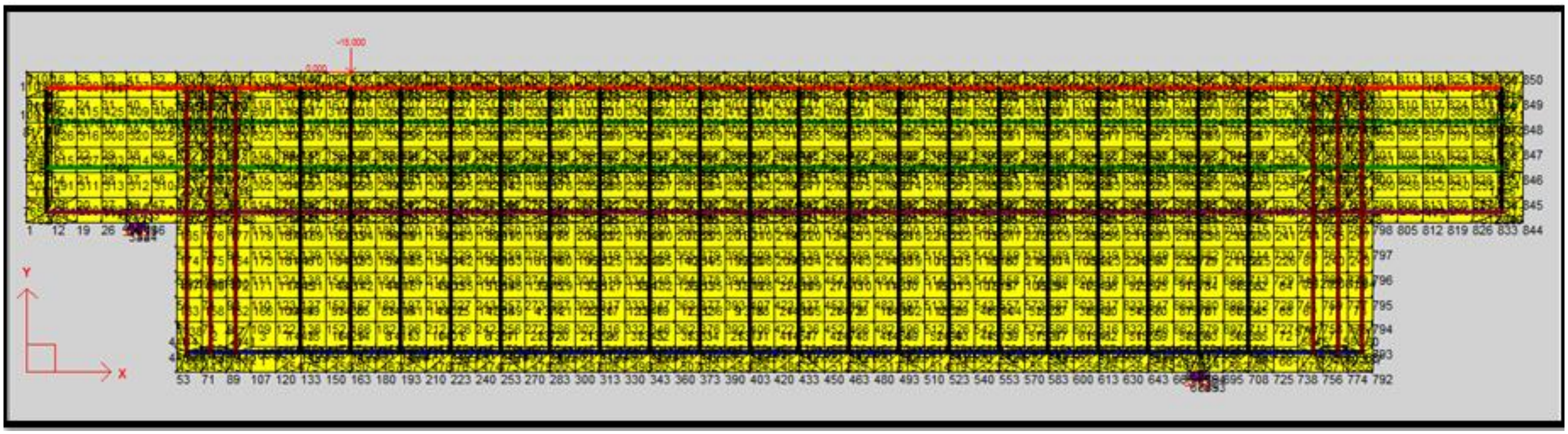
$V_u = A_{vh}\,(\phi\, f_{yvh})$

$V_u = 1.57\,(0.75 \times 62\,429.23)$
$= 73510.42\text{ lbs}$
$= 327.12\text{ kN}$ #.

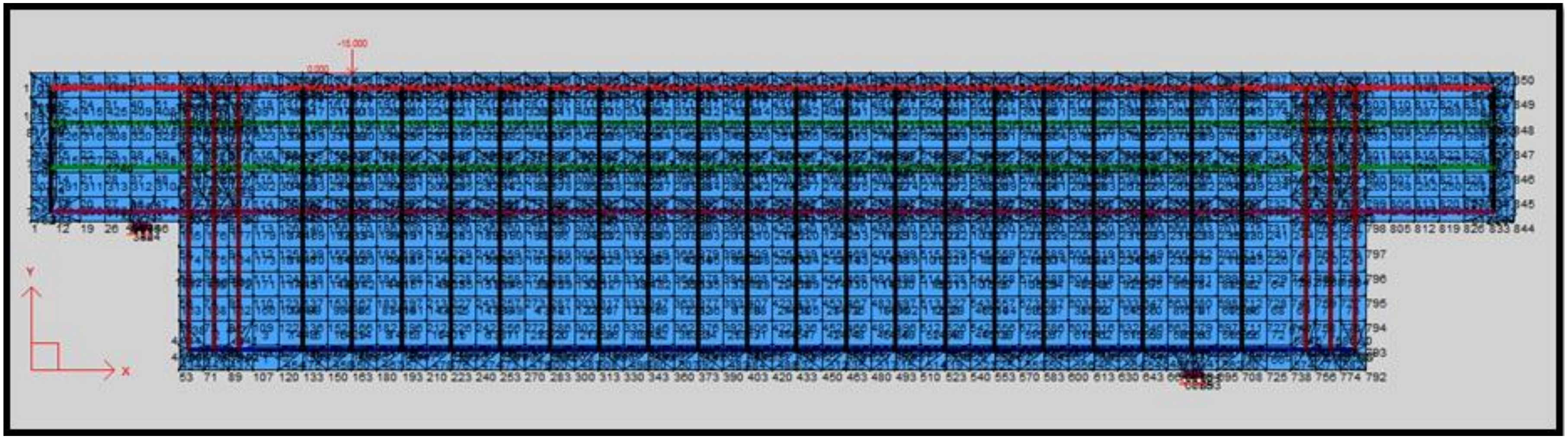
* Thus, this specimen failed ~~in~~ at the Hanger reinforcement with the failure load of 327.12 kN #



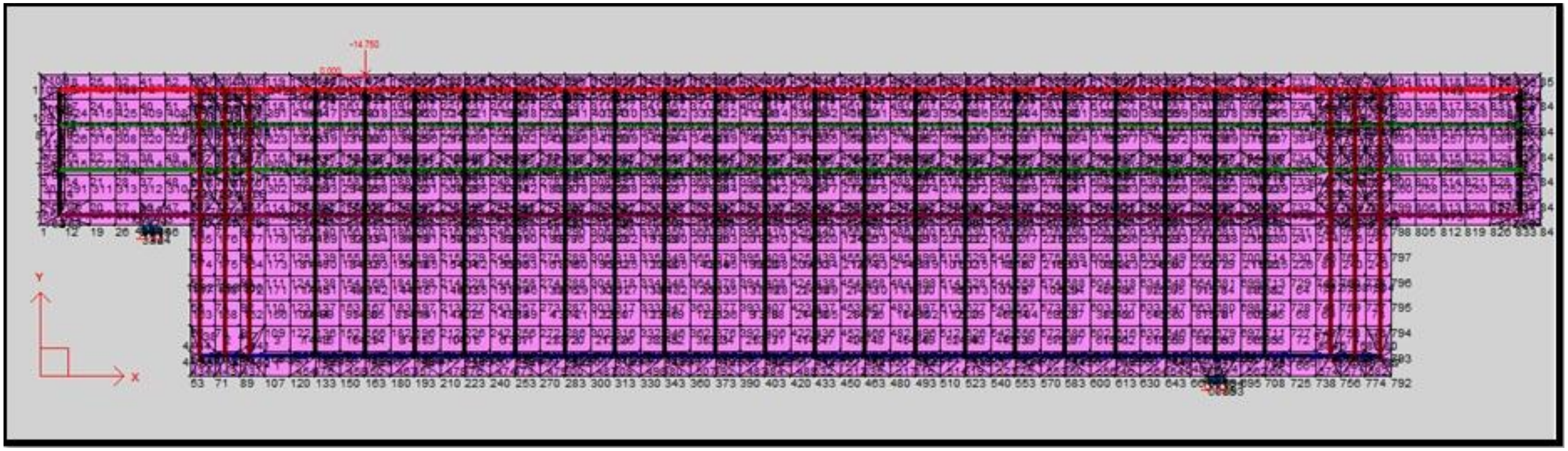
VecTor2: Specimen 1



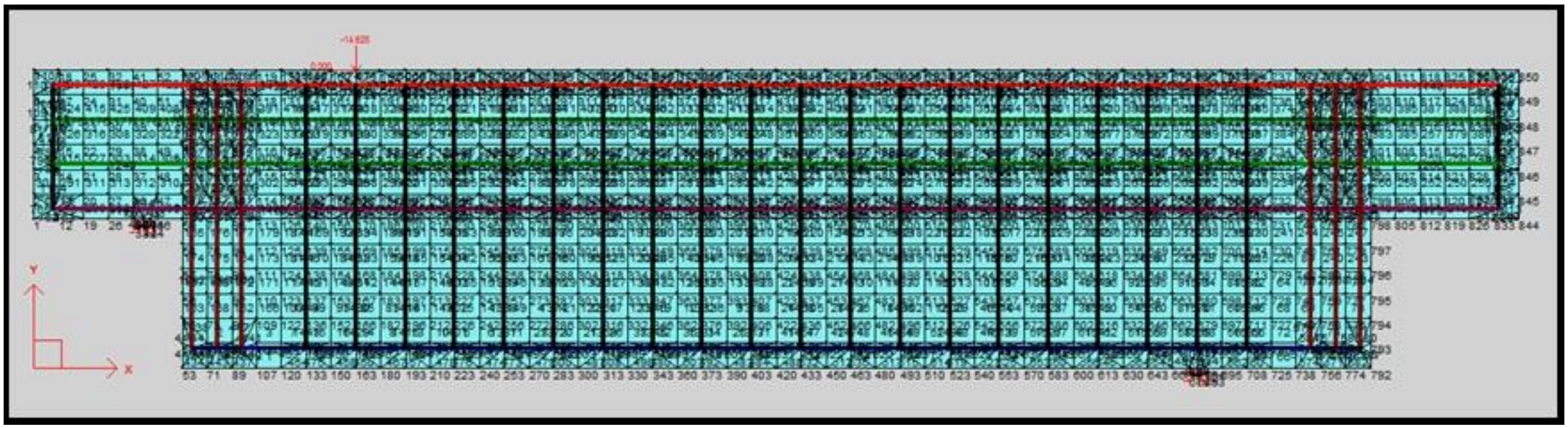
VecTor2: Specimen 2



VecTor2: Specimen 3



VecTor2: Specimen 4



VecTor2: Specimen 5