Progressive collapse analysis of reinforced concrete structure

by

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CERTIFICATION OF APPROVAL

PROGRESSIVE COLLAPSE ANALYSIS OF REINFORCED CONCRETE STRUCTURE

By

MUHAMMAD IMRAN B. ROSLAN

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Approved by,

(Dr. Teo Wee)

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TRONOH, PERAK

December 2010

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.

MUHAMMAD IMRAN BIN ROSLAN

ABSTRACT

The event of local failure that occurred in structural system could lead to a disastrous incident. What actually happen during the occurrence of collapse and what triggers the process could not be interpreted accurately because the occurrence is unintended and unwanted. This research is on the subject of investigating the event of progressive collapse using the method of strut and tie model. This analysis will help give a better perspective of what actually happened during the local failure of a structure in term of stress distribution. The result will improve the understanding in term of discontinuity, disturbance and detail of the strain distribution during the event of progressive collapse and describe how the load is transferred during the local failure. This is the first attempt in analyzing the progressive collapse using strut and tie method and can be very helpful information if the analysis achieve a successful result.

This approach will require the design of a structural system that can represent the structure that is modeled to simulate the situation during the process of a collapsing building or a structure. This simulation is done to create the load and stress that exist at some stage in the event that lead to the maximum loading and eventually initiate the collapse or failure to the structural system. The approach that is used to create this situation is by removing one of the critical support structures that hold the system as a whole. The model is first designed to work as a normal structure and carry all the imposed load and dead load for the intended use. By removing one of the critical supporting member such as beam that connect the sections of the structure and the column that hold a section whether in the middle or at the structures end, it will a produce a condition that replicate the state of local failure due the loss of the supporting member.

By means of analyzing the behavior of the structure after the local failure using strut and tie analysis, this "missing column" and "missing beam" scenarios can be computer-generated and the load path of the structure can be observed and analyzed.

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INTRODUCTION

1.1 PROBLEM STATEMENT

Progressive collapse denotes an extensive structural failure initiated by local structural damage, or a chain reaction of failures following damage to a relatively small portion of a structure. This can be also characterized by the loss of load-carrying capacity of a relatively small portion of a structure due to an abnormal load which, in turn, triggers a cascade of failures affecting a major portion of the structure.

The whole world was shocked by the event on 11th September that changed the view of Lower Manhattan, New York City forever. The disaster saw the two 110 storey skyscrapers collapsed into ashes after two planes crashed on to the buildings triggering somewhat peculiar type of structural damage that witness both of the towers fall down vertically towards the ground and another building, WTC7 that also collapsed insignificantly affecting the buildings surrounding it. Before this, there were two distinctions in the modern human society era, B.C and A.D., but the collapse of World Trade Center marks a new phase of modern world, before and after 9/11.

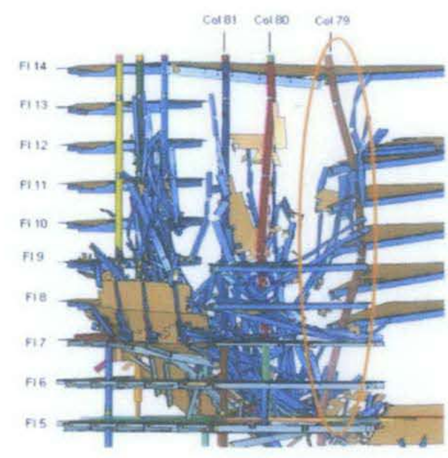


Image from http:// www.nist.gov/bfrl/ disaster_resilience/wtc /wtc7_collapse.cfm

Figure 1.1: Graphic showing the buckling of WTC 7 Column 79 (circled area), the local failure identified as the initiating event in the building's progressive collapse.

If there is an event that displays a significantly minor damage that imbalance the structure and the leads to the collapse of the major part or even the whole structure, then it is a disproportionate collapse. When the collapse starts with the failure of one or a few structural components and then the progresses over successive other components, a fitting label would be progressive collapse. Although the two terms are often used interchangeably, a differentiation can be made to the both of the terms where, 'disproportionate collapse' is more appropriate in the context of design and performance because a precise definition of 'disproportionate' requires the choice of design objectives but whereas progressive collapse is more towards the physical and mechanism of the collapse.

The term 'event' includes all types of potential triggering circumstances. Since these occur with a low probability or even a wholly unexpectedly, there are called accidental circumstances. Before, this referred more to events like accidents and natural disasters or design and construction flaws. In the light of emerging new threats, however, it nowadays also includes deliberate damage inflicted by explosion and other kinds of malicious action.

Different structures are susceptible to progressive collapse to different degrees. Such differences remain unrecognized, though, even in modern verification procedures (Baldridge, 2003)using partial safety factors. This follows, in particular, from not factoring in the structural response to an initial local failure. Additional considerations are therefore necessary concerning both the initial local failure and the ensuing response of the structure. Such considerations have in the past been made only in isolated cases such as for embassy buildings or very long bridges, that is, obviously exposed or susceptible structures and for the most part at the consideration of the design engineer. (Starossek, Progressive collapse of structures, 2009)

The prediction of possible progressive collapse under specific conditions may provide very important information that could be used to control or prevent progressive collapse. It is now clear that abnormal loadings must be taken into account when designing structures. Abnormal load events could arise from a number of sources: gas explosion; confined dust or vapor conflagration; machine malfunction; high explosive effects; missile impact etc. However, to date, no adequate tools exist that can perform a progressive collapse analysis with acceptable reliability. Therefore, two main analysis techniques should be developed to serve as the fundamental bases for fundamental and practical progressive collapse assessment. In the design state, it is very important to predict the behavior of possible progressive collapse, as accurately as possible, for the various abnormal loads that should be considered.

1.2 OBJECTIVES

The objective of this project is to analyse the outcome of progressive collapse to reinforced concrete structures and damage assessment methodology of partially collapsed structures by analyzing the load distribution using strut-and-tie model.

This analysis are intended to determine the capacity of a structure either to resist an abnormal loading, thereby preserving the load carrying capacity of the critical elements, or to redistribute gravity loads of a critical load-bearing when one of the elements is removed.

At the end of the study, we should be able to summarize the effect of loss of elements in beams to the structure itself and provide relevant data and calculations. At the moment, there is very limited information on large deformation and stress distribution using strut-and-tie model. The data gathered from this study will contribute to the database on collapse behavior of RC structures and it is also hoped that the result of the analysis in this paper will contribute to the future development of collapse resistant design method.

1.3 SCOPE OF WORKS

This project will be done mainly using software analysis programs such as StaadPro, AutoCAD, and CAST. Most of the data for the analysis will be based on previous experiments and journals that have been done from various sources. The study is a task to perform progressive collapse analysis that considers the loss of portions of the structure in numerous "missing column" and "missing beam" scenario. The design measures resulting from such considerations can either at increasing the level of safety against local failure or at limiting the total damage following the local failure. The design of the building will be fluctuating between the variables such as the level of continuity, redundancy, and ductility so that alternative load paths can develop following the loss of an individual member. The analysis will be done using the strut-and-tie model to explain the internal force flow (stress fields) due to the loss of element in the structure.

This approach will require the design of a structural system that can represent the structure that is modeled to simulate the situation during the process of a collapsing building or a structure. This simulation is done to create the load and stress that exist at some stage in the event that lead to the maximum loading and eventually initiate the collapse or failure to the structural system. The approach that is used to create this situation is by removing one of the critical support structures that hold the system as a whole. The model is first designed to work as a normal structure and carry all the imposed load and dead load for the intended use. By removing one of the critical supporting member such as beam that connect the sections of the structure and the column that hold a section whether in the middle or at the structures end, it will a produce a condition that replicate the state of local failure due the loss of the supporting member.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Progressive collapse of structures is typically characterized by a disproportion between a small triggering event and the resulting collapse of a major part or even the whole of the structure. Although the disproportion between cause and effect is a defining and common feature, there are various differing mechanisms of collapse that produce such an outcome. The amenability to conceptual, theoretical, and computational treatment can vary accordingly. Collapse-promoting features, possible or preferable countermeasures, and the suitability of indices for quantifying robustness and collapse resistance will likewise depend on the mechanism of collapse. Finally, different kinds of structures are susceptible to different mechanism of collapse.

It is thus useful to distinguish and describe the different types of progressive collapse and gives classification to the types of collapse. These types of progressive collapse are not a standard yet and the types used in this review is based on the types that is suggested by Uwe Starrosek, a lecturer at the Hamburg University of Technology, a former Structural Engineer and author to the book Progressive Collapse of Structures where he included the type of progressive collapse.

The typology and classification of progressive collapse of structures is developed that is founded on a study of the various underlying mechanisms of collapse. Six different types and four classes are discerned, the characteristic features of each category are described and compared, and a terminology is suggested. On this basis, the theoretical treatment of progressive collapse and the development of countermeasures are facilitated because they differ for different types of collapse. Some conclusions drew here concern analogies that should be pursued further, collapse-promoting features, and possible countermeasures.

2.2 PREVIOUS AND ONGOING STUDIES

There are a lot of studies that has been done regarding to the analysis progressive collapse both experimental and software based simulation and analysis. All of these studies have contributed to increase the level of understanding in the behavior of structural elements during the failure of the structure.

2.2.1 Seismic design in preventing progressive collapse

(Baldridge, 2003), clarified that the inherent ability of seismically designed RC beam-column frames to resist progressive collapse. The study has shown that both new and existing structures designed and detailed with such a system already have an inherent ability to better resist progressive collapse and they also concluded that seismically designed RC moment-resisting frames provide a structure with continuity, redundancy and ductility.

The study that was conducted using a building that was taken from the design manual entitled *Design of Concrete Buildings for Earthquake and Wind Forces*. The structure consists of five 24 ft (7.3 m) bays in the longitudinal direction and three 24 ft (7.3 m) bays in the transverse direction. Typical floor-to-floor height is 12 ft (3.7 m) except for the first story, which is 15 ft (4.6 m). Service loads of 50 lb/ft2 (2.4 kPa) live load and 42.5 lb/ft2 (2.0 kPa) superimposed dead load were assumed in the analysis. Analysts evaluated each progressive collapse scenario using a 3-D linear elastic model of the structure created in ETABS Plus, Version 7.18 (Extended 3D Analysis of Building Systems), developed by Computers and Structures, Inc., Berkeley, CA.

The study proved that the seismically designed RC beam-column frames based on the (UBC, 1991) are able to withstand the loss of an element in the structure and the system already have the ability to better resist progressive collapse without substantial increase in project cost.

2.2.2 Finite element analysis on beam

There is also work done to analyse progressive collapse RC structure using finite element by Sasani and Kropelnicki, 2007. A seven-storey was designed and the exterior first floor column was removed to simulate the damage to a support in structure. The model of beam in the second floor was analyzed using ANSYS to develop a detailed finite element model. The model utilizes eight-node solid elements for the concrete and two-node element for the reinforcing bars.

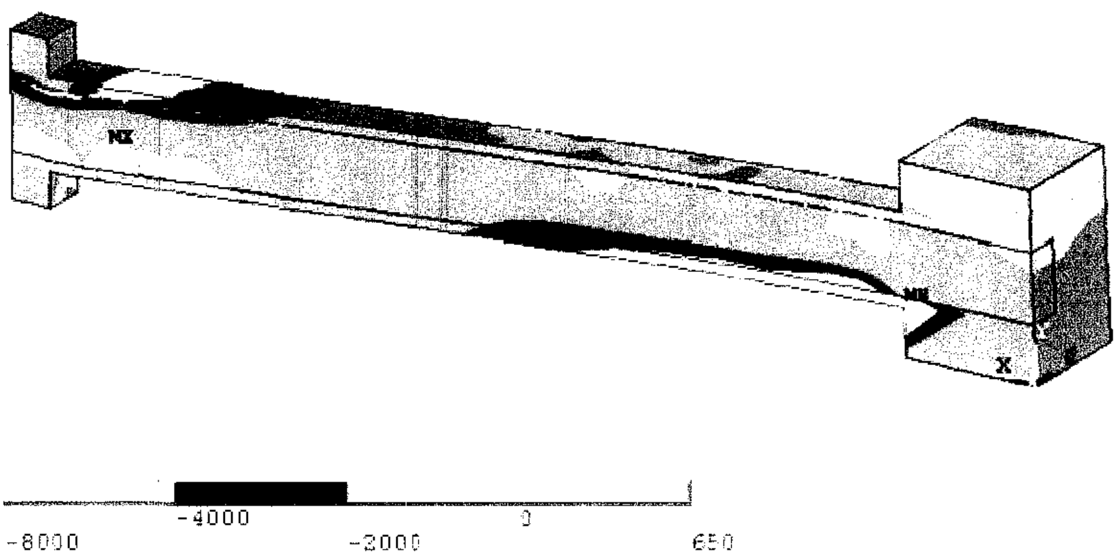


Figure 2.1: Longitudinal concrete stress in tested beam

The finite element mesh is refined in areas with high stress gradients and verified through sensitivity analysis of multiple mesh schemes. The figure shows that the axial compressive stress in the cover concrete at the top and close to the centre stub is reduced to zero following concrete crushing. Furthermore, the concrete stress at the top of the core close to the centre stub remains high. Due to three dimensional confining effects, the stress is increased above the uniaxial strength of concrete.

They concluded that, a detailed finite element model of the beam is developed that closely predicts the behavior of the beam up to the fracture of the bottom reinforcing bars. Such a calibrated and verified analytical modeling along with hybrid analyses can be utilized to evaluate potential progressive collapse of RC structures.

2.2.3 Catenary action during progressive collapse using experimental study

Yi and et al, 2008 conducted an experimental study to observe the behavior of RC frame structures in resisting progressive collapse. This static experimental study was done to investigate progressive failure of a reinforced concrete frame due to the loss of a lower storey column. A four-bay and three-storey one third scale model representing a segment of a larger planar frame structure was tested. A constant vertical load was applied to the top of the middle column by a servo-hydraulic actuator to simulate the gravity load of the upper floors and the failure of the middle column of the first storey was simulated by unloading a mechanical jacking system. The frame collapse, defined in this study as the rupture of tension steel bars in the floor beams, occurred at a vertical unloading displacement of 456mm (18in.) that corresponds to a beam drift angle(rotation with respect to the horizontal) of 10.3 degrees. Based on the experimental observations, the mechanical behavior of the model of the model frame is analyzed and the redistribution and transition of the load resisting mechanism is discussed.

The experimental result was divided into four stages which was very helpful in describing the state of the structure. The study also analyzed approximately the internal force for each of the three stages, elastic, inelastic (plastic) and catenary stage with the help of the vertical displacement curve and the structural behavior obtained from the experiment.

Based on the experimental and analytical result, they acknowledged that failure resulting from progressive collapse of the RC concrete frame structure was ultimately controlled by the rupture of the reinforcing steel bars in the floor beams. While this is different from the normal limit state for beam bending, which is controlled either by crushing of concrete in compression or shear failure. They suggested that if the strain in the tensile bars can be distributed more uniformly along the length the load-carrying capacity can be enhanced through the observed catenary mechanism.

The study also mentioned the experiment was conducted under static condition. Where, in reality, structural collapse resulting from accidental loads is a dynamic phenomenon that requires consideration of numerous additional factors.

2.2.4 Comparison between methods of analysis

An example of study that compares the results using different approach is the comparison between finite element and macromodel-based simulation of progressive collapse by Bao and et al, 2008. The potential for progressive collapse of a typical reinforced concrete RC moment frame structure initiated through the loss of one or more first-story columns is numerically simulated using a macromodel-based approach. The development of the simulation model is guided by the realization that the characterization of nonlinear behavior associated with the transfer of forces through the joint is critical to predict the large deformation response associated with progressive collapse. A simplified simulation model of a beam-column joint is used to represent essential and critical actions in the floor beams and the transfer of these forces through the joint region to the vertical elements. The validity of the macromodel developed is evaluated through comparison of both overall response and element actions with those obtained from high-fidelity finite-element analyses.

Two prototype buildings designed for lateral load requirements in a non seismic and seismic region are considered in this study. Two-dimensional models of the frames are subjected to gravity loads and then one or more first-story columns are removed, and the resulting large displacement inelastic dynamic response of each frame is investigated. It is demonstrated that the proposed approach using a validated macromodel is a viable methodology for progressive collapse analysis. The study also finds that special RC moment frames detailed and designed in zones of high seismicity perform better and are less vulnerable to progressive collapse than RC frame structures designed for low to moderate seismic risk.

The open-source platform OpenSees (2007) is used in this study to develop and demonstrate the utility of the proposed macromodel approach as shown in Figure 2.2. The nonlinear finite-element analysis is carried out using the commercial software DIANA (2006) which employs a total Lagrangian formulation to consider geometric effects.

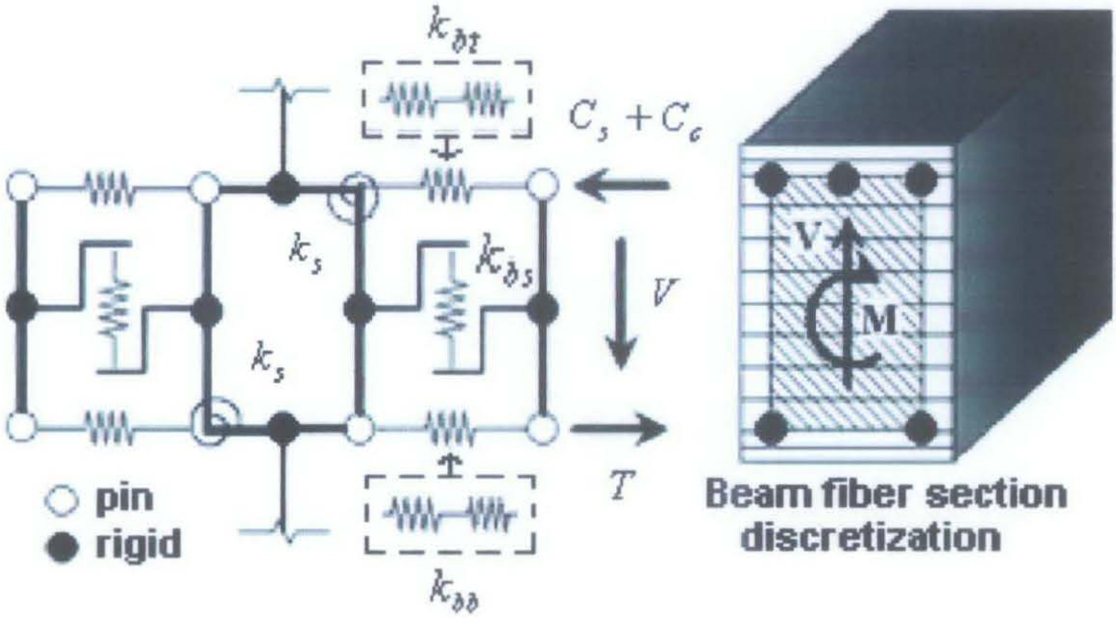


Figure 2.2: Spring-assembly representing typical interior joint and beam-joint interface and the properties of the spring elements that constitute the macromodel

Based on this previous studies that were done to provide relevant objectives, this study to analyze using strut and tie model can be done to create a different perspective on the field of progressive collapse analysis. This analysis can provide a significance comparison to the previous studies because there is still no research regarding progressive collapse analysis using strut and tie model and this research can become a first attempt to provide the data and analysis.

2.3 STRUT-AND-TIE MODEL

The idea of the strut-and-tie method came from the truss analogy method introduced independently by Ritter and Mörch in the early 1900s for shear design of B-Regions. This method employs the so-called truss model as its design basis. The model was used to idealize the flow of force in a cracked concrete beam. In parallel with the increasing availability of experimental results and the development of limit analysis in plasticity theory, the truss analogy method has been validated and improved considerably in the form of full member or sectional design procedures. The truss model has also been used as the design basis for torsion. The strut-and-tie method came after Marti, 1985, and Schlaich et al., 1991, promoted the use of truss model in D-Regions.

The STM is based on the lower-bound theory of limit analysis. In the STM, the complex flow of internal forces in the D-Region under consideration is idealized as a truss carrying the imposed loading through the region to its supports. This truss is called *strut-and-tie model* and is a statically admissible stress field in lower-bound (static) solutions. Like a real truss, a strut-and-tie model consists of *struts* and *ties* interconnected at *nodes* (also referred to as *nodal zones* or *nodal regions*). A selection of strut-and-tie models for a few typical 2-D D-Regions is illustrated in Figure 2.3. As shown in the figure, struts are usually symbolized using broken lines, and ties are usually denoted using solid lines.

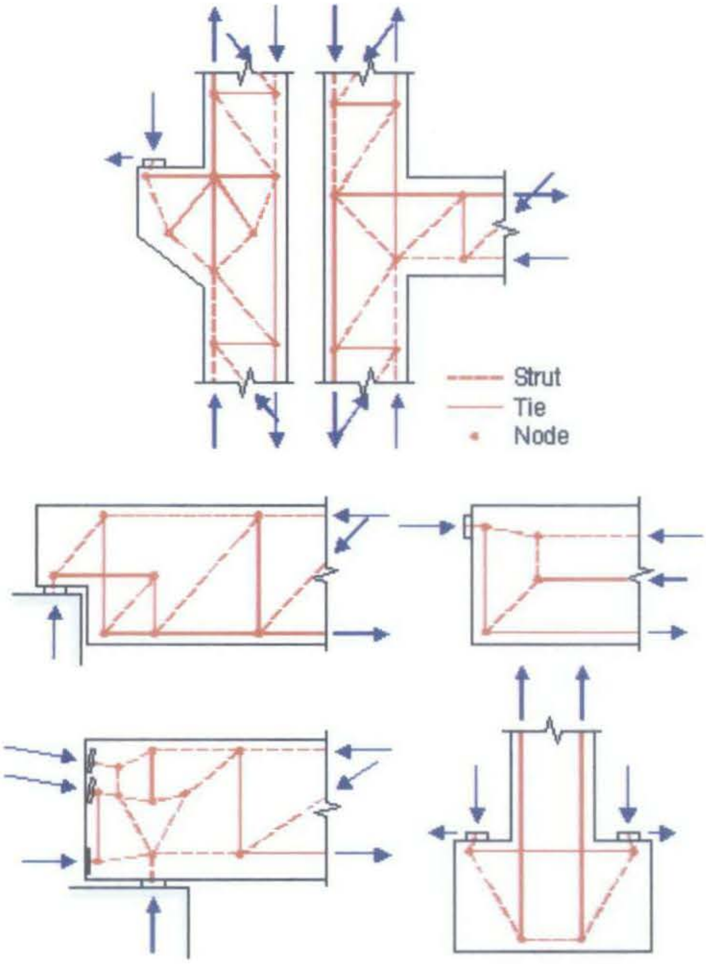


Figure 2.3: Examples of strut-and-tie models for common structural concrete members

Strut-and-Tie Model Components

Struts are the compression members of a strut-and-tie model and represent concrete stress fields whose principal compressive stresses are predominantly along the centerline of the strut. The idealized shape of concrete stress field surrounding a strut in a plane (2-D) member, however, can be prismatic (Figure 2.4(a)), bottle-shaped (Figure 2.4(b)), or fan-shaped (Figure 2.4(c)). Struts can be strengthened by steel reinforcement, and if so, they are termed reinforced struts.

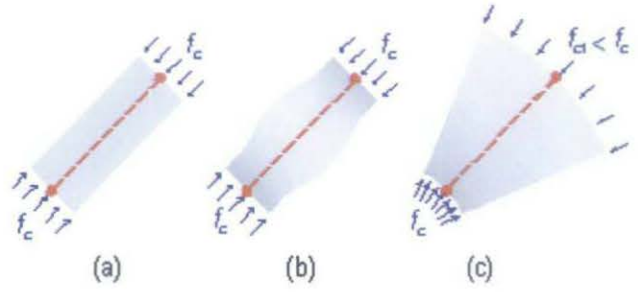


Figure 2.4: Basic Type of Struts in a 2-D Member: (a) Prismatic (b) Bottle-Shaped (c) Fan-Shaped

Ties are the tension members of a strut-and-tie model. Ties mostly represent reinforcing steel, but they can occasionally represent prestressing steel or concrete stress fields with principal tension predominant in the tie direction.

Nodes are analogous to joints in a truss and are where forces are transferred between struts and ties. As a result, these regions are subject to a multidirectional state of stress. Nodes are classified by the types of forces being connected. Figure 2.5 shows basic types of nodes in a 2-D member; in the figure, *C* is used to denote compression and *T* is used to denote tension.

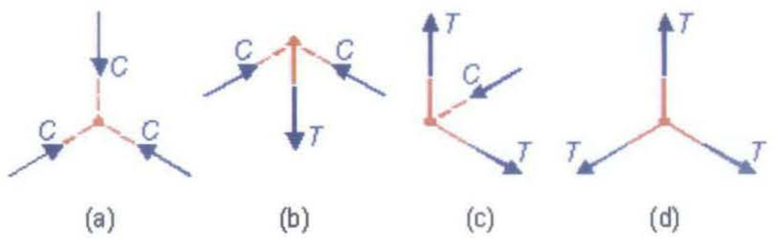


Figure 2.5: Basic Type of Nodes: (a) CCC (b) CCT (c) CTT (d) TTT

Uniqueness of Strut-and-Tie Models

As a statically admissible stress field, a strut-and-tie model has to be in equilibrium externally with the applied loading and reactions (the boundary forces) and internally at each Node. In addition, reinforcing or prestressing steel is selected to serve as the ties, the effective width of each strut is selected, and the shape of each nodal zone is constructed such that the strength is sufficient. Therefore, only equilibrium and yield criterion need to be fulfilled for an admissible strut-and-tie model. The third requirement in solid mechanics framework, namely the strain compatibility, is not considered.

As a result of these relaxed requirements, there is no unique strut-and-tie model for a given problem. In other words, more than one admissible strut-and-tie model may be developed for each load case as long as the selected truss is in equilibrium with the boundary forces and the stresses in the struts, ties, and nodes are within the acceptable limits. The lower-bound theorem guarantees that the capacity obtained from all statically admissible stress fields is lower than or equal to the actual collapse load. However, as a result of limited ductility in the structural concrete, there are only a small number of viable solutions for each design region. Figure 7 illustrates an example in which one solution is preferable to another. Due to the point load at the tip of the cantilever portion, the upper part of the beam is likely to develop horizontal tensile stresses along the beam. Therefore, the model with the upper horizontal tie (Figure 2.6(a)) is preferable to that shown in Figure 2.6(b). The latter only effectively resists the tension in the upper region near the middle support.

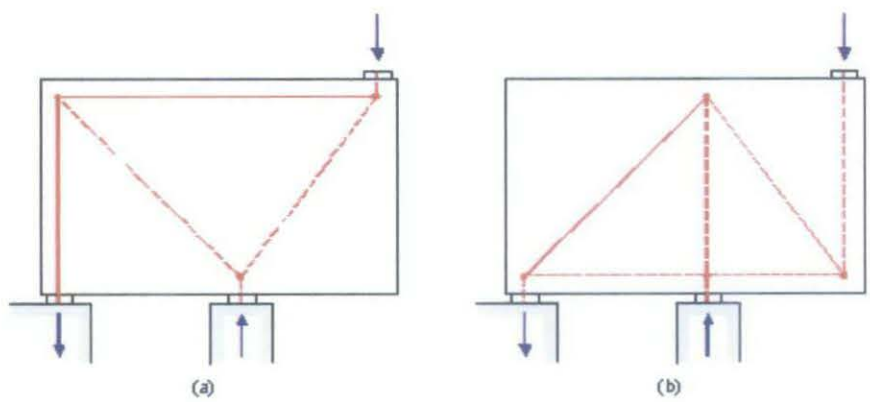


Figure 2.6: Two statically admissible strut-and-tie models for a cantilevered deep beam under vertical loading: (a) Workable truss (b) Less favorable truss due to excessive ductility demands.

2.4 GSA PROGRESSIVE COLLAPSE GUIDELINES

The U.S. General Services Administration (GSA) developed the "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" to ensure that the potential for progressive collapse is addressed in the design, planning and construction of new buildings and major renovation projects. Mr. Bruce Hall, P.E., of the Office of the Chief Architect, initiated this work in 1999 and served as the GSA Project Manager. The Guidelines, initially released in November 2000, focused primarily on reinforced concrete structures. GSA subsequently identified the need to update the November 2000 Guidelines to address the progressive collapse potential of steel frame structures. Preparation of the updated Guidelines was performed by Applied Research Associates, Inc. with assistance provided by Myers, Houghton & Partners, Inc., Simpson, Gumpertz and Heger, Inc., the U.S. Army Corps of Engineers, and the U.S. Department of State.

The purpose of these Guidelines is to:

- Assist in the reduction of the potential for progressive collapse in new Federal Office Buildings
- Assist in the assessment of the potential for progressive collapse in existing Federal Office Buildings
- Assist in the development of potential upgrades to facilities if required

To meet this purpose, these Guidelines provide a *threat independent* methodology for minimizing the potential for progressive collapse in the design of new and upgraded buildings, and for assessing the potential for progressive collapse in existing buildings. It should be noted that these Guidelines are not an explicit part of a blast design or blast analysis, and the resulting design or analysis findings cannot be substituted for addressing blast design or blast analysis requirements. The requirements contained herein are an independent set of requirements for meeting the provisions of Interagency Security Committee (ISC) Security Criteria regarding progressive collapse. The procedures presented herein are required for the treatment of progressive collapse for U.S. General Services Administration (GSA) facilities.

The previous guidelines, "Progressive Collapse Analysis and Design for New Federal Office Buildings and Major Modernization Projects", November 2000 focused primarily on analysis and design for progressive collapse of reinforced concrete structures. This update includes lessons learned and adds a separate section pertaining to structural steel buildings

2.5 TERM DEFINITIONS

Continuity, Redundancy, Ductility

Redundancy: Initially, the structural redundancy is described as indeterminate degree of a system. Although many definitions are presented for redundancy in structures, recently the definition of structural redundancy has been related to the configuration of structural system and the number of lateral load transferring directions in the structure.

Ductility: Capacity of a material to deform permanently (e.g., stretch, bend, or spread) in response to stress. Most common steels, for example, are quite ductile and hence can accommodate local stress concentrations. Brittle materials, such as glass, cannot accommodate concentrations of stress because they lack ductility, and therefore fracture easily. When a material specimen is stressed, it deforms elastically (*see* elasticity) at first; above a certain deformation, called the elastic limit, deformation becomes permanent.

Robustness and Collapse resistance

Robustness: Robustness is the insensibility of a structure to local failure. Robustness is purely structural property in the sense that the cause and probability of local failure and thus also the nature, extent, and probability of the triggering accidental circumstances are immaterial.

Collapse resistance: Collapse resistance is defined as insensitivity to accidental circumstances, that is, to unforeseeable and low-probability events.

Elastic stage, Elasto-plastic stage, Plastic hinge mechanism state, Catenary action state and Collapse limit state

Elastic stage: This type of deformation is reversible. Once the forces are no longer applied, the object returns to its original shape.

Elasto-plastic stage: This type of deformation is irreversible. However, an object in the plastic deformation range will first have undergone elastic deformation, which is reversible, so the object will return part way to its original shape.

Plastic hinge mechanism state: In structural engineering beam theory the term, plastic hinge, is used to describe the deformation of a section of a beam where plastic bending occurs.

Catenary action state: After the plastic stage, the concrete strain in the compression zone at the beam ends reaches its ultimate compressive strain, and the frame begin to enter into catenary action stage. During this phase, the original compressive steel bars are gradually subject to tension with increasing displacement; whereas the original tensile steel bars continue to remain in tension.

Collapse limit state: Phase when the structure has collapsed.

CHAPTER 3

METHODOLOGY

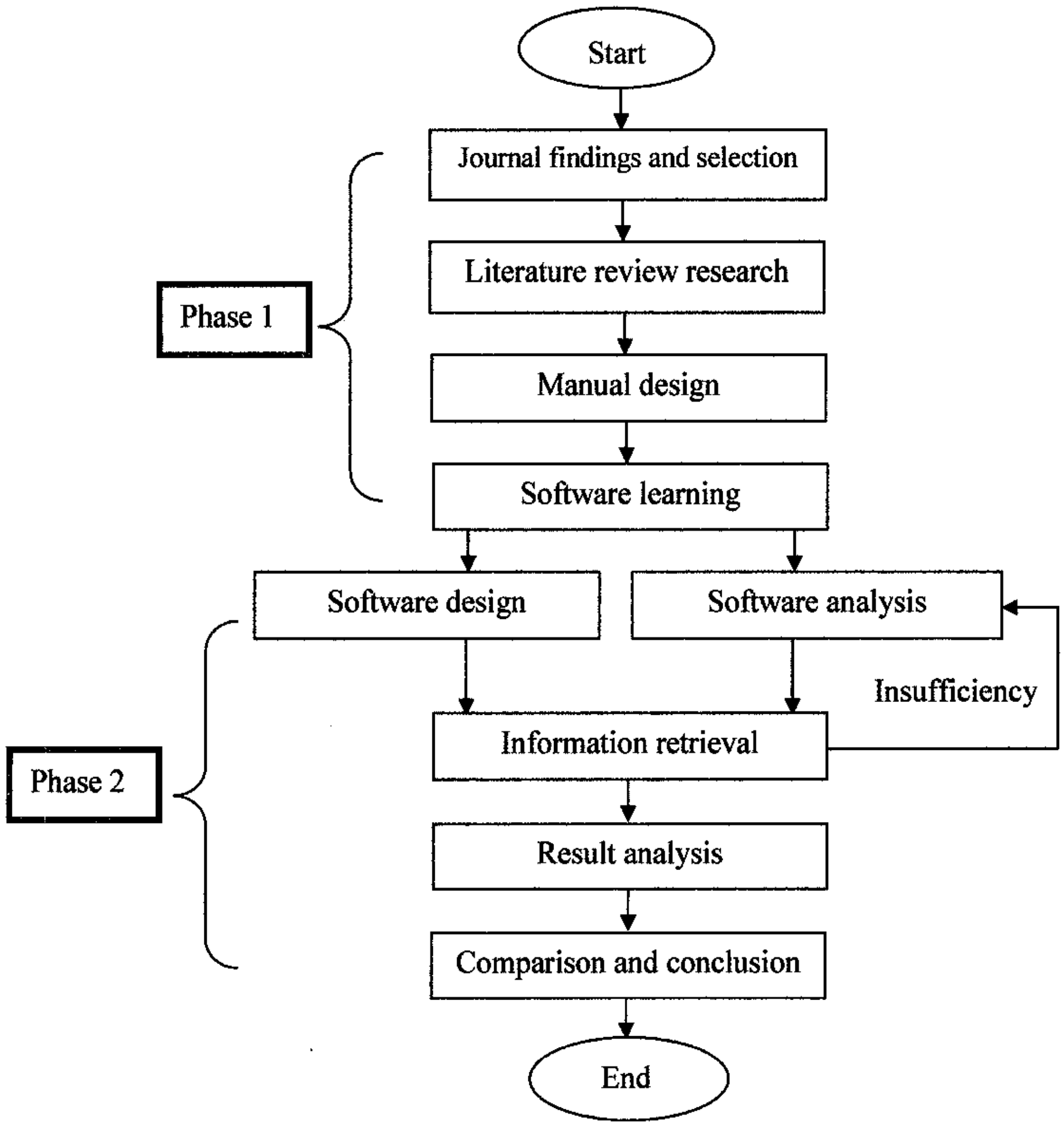


Figure 3.1: Methodology diagram

3.1 Phase I – Progressive Collapse Analysis Methodology

The objective of this phase is to develop progressive collapse theories and to establish the corresponding analysis procedures. This phase is a critical part of the study, and it could be accomplished, as shown below:

1. **Identify Problems**

   Find the characteristics of progressive collapse to be studied and determine the approach to analyze progressive collapse. The necessary theoretical basis will be established by a comprehensive literature review.

2. **Theory Review and Manual Design**

   The identified theories to analyze progressive collapse are modified or clarified, including; geometrical and material nonlinearities, stress stiffening, and stiffness variation theories. This procedure includes the selection of criteria (e.g. stress, stain, buckling, etc.) that might be applied to progressive collapse analysis. Also, address the expected uncertainties in progressive collapse (e.g. debris load, transient loads, damage, etc.) that might be modeled in closed form. The possible mathematical and numerical design should be proposed in this part of the study.

3. **Numerical Approaches and Computer Software Review**

   Review and assess the analytical capabilities of commercial computer software for their suitability to apply reviewed and selected theories for simple and advanced structural models. This assessment will enable one to define the items that should be selected and/or modified in such analyses, and how to apply them, and, also, for progressive collapse analysis. For this case, the CAST software is selected to analyse the model

3.2 Phase II – Software and Numerical Analysis

This phase of the study is aimed at establishing relationships between the strut-and-tie model analysis and the previous studies. Comparisons are done to highlight the different features and advantages mostly on the internal force flow.

The following steps are proposed;

1. **Apply strut-and-tie model software**

   The development of models to enable structural collapse simulation should take into consideration all actions and interactions within the frame elements beam columns and connection regions joints that occur during the large deformation response. In particular the model should incorporate features to adequately account for the transfer of forces from beams and columns through the joint. Bao et al, 2008.

   This is the critical part where the model is designed and inserted with respective value and criteria are analyzed. The model here is based on the previous works that have been mentioned in the literature review. The process is recurring if the analysis failed or lack in information and done until it produce relevant result.

2. **Result harvesting and comparison**

   The data that was obtained through the software analysis using CAST and manual calculations are collected together and compared with the previous studies.

3. **Analysis and Conclusion**

   Comparison with previous work will provide concise and clear findings that will increase the understanding in progressive collapse behavior and how the catenary effect of each model is portrayed in the internal force flow.

3.3 Design Methodology

Design Process Using CAST Software

1. Setup Model Basic Properties
   a. Define the General Properties
   b. Define the D-Region thickness(width)
   c. Define the Concrete Compressive Strength(f'_c)
   d. Define the Non-Prestressed Reinforcement Yield Strength(f_y)
2. Construct Model
   a. Construct the Guidelines
   b. Construct the Outer D-Region based on the model dimensions
   c. Establish the Strut and Tie Elements inside the D-Region
   d. Assign the Boundary Condition where the support and force point are located
   e. Input the loading and the Load Point.
3. Obtain Strut and Tie Model Forces
   a. Run Design Calculation to determine whether the model is stable
   b. Identify compression and tension member
4. Defining and Assigning Properties
   a. Define Strut, Tie and Node property type
   b. Strut : Bottled Shape or Prismatic
   c. Tie : Reinforcement Properties
   d. Node : CCC,CCT,CTT
5. Check Stress
   a. Run Truss Analysis
   b. Check stresses in strut, ties and nodes

3.4 Economic Benefits

The cost for research

As for all the analysis and test the author have done, no cost was involved as all the software and material required is easily obtained available on the internet and library. All the journals are either extracted from the internet or attained from the Information Resource Center. The main software that is CAST is downloaded legally from the source website.

Cost of the product

Because of the study focuses on the analysis only on the software and numerical approach, the product of this study is purely theoretical and could only benefit the Civil and Structural Engineering field as valuable information for the design considerations and precautions.

Business elements

On the business perspective, this study could benefit the construction technology because it can reduce the cost of maintenance if the design procedure is adequate to withstand progressive collapse if it happens. This study also may help the field of engineering to reduce cost of reconstructing damage buildings due to progressive collapse.

CHAPTER 4

RESULTS AND DISCUSSION

1. DESIGN LAYOUT

The design layouts of the Strut-and-Tie Model are based on (Su, Tian, & and Song, 2009) that have done the experimental works. Their works are interpreted into the design for the Strut-and-Ties Model for the CAST software. Mostly the designs are quite similar to each other but the different lies towards the reinforcement details of each individual beams. The dimensions of the samples are also different according to the past journal's experimental specimens. The design layouts for all the beams can be referred to the Appendix A.

1.1.1 DATA COLLECTION FOR SAMPLES

Table 4.1: Sample Properties

Test	*bxh* mm(in.)	l_n mm (in.)	l_n/h	F_{cu} Mpa (psi)	Longitudinal reinforcement and ratio		Ties
					Top	Bottom	
A1	150 x 300 (5.9 x 11.8)	1225 (48)	4.08	32.3 (4680)	2ϕ12, P = 0.55%	2ϕ12, P = 0.55%	ϕ 8 at 100
A2	150 x 300 (5.9 x 11.8)	1225 (48)	4.08	35.3 (5120)	3ϕ12, P = 0.83%	3ϕ12, P = 0.83%	ϕ 8 at 80
A3	150 x 300 (5.9 x 11.8)	1225 (48)	4.08	39.0 (5660)	3ϕ14, P = 1.13%	3ϕ14, P = 1.13%	ϕ 8 at 80
A4	150 x 300 (5.9 x 11.8)	1225 (48)	4.08	28.8 (4180)	2ϕ12, P = 0.55%	1ϕ14, P = 0.38%	ϕ 8 at 100
A5	150 x 300 (5.9 x 11.8)	1225 (48)	4.08	33.1 (4800)	3ϕ12, P = 0.55%	2ϕ12, P = 0.55%	ϕ 8 at 80
A6	150 x 300 (5.9 x 11.8)	1225 (48)	4.08	35.8 (5190)	3ϕ14, P = 1.13%	2ϕ14, P = 0.75%	ϕ 8 at 80
B1	150 x 300 (5.9 x 11.8)	1975 (78)	6.58	23.2 (3360)	3ϕ14, P = 1.13%	3ϕ14, P = 1.13%	ϕ 8 at 100
B2	150 x 300 (5.9 x 11.8)	2725 (107)	9.08	24.1 (3500)	3ϕ14, P = 1.13%	3ϕ14, P = 1.13%	ϕ 8 at 120
B3	150 x 300 (5.9 x 11.8)	2725 (107)	9.08	26.4 (3830)	3ϕ14, P = 1.13%	2ϕ14, P = 0.75%	ϕ 8 at 120
C1	100 x 200 (3.9 x 7.9)	1225 (48)	6.12	19.9 (2890)	2ϕ12, P = 1.30%	3ϕ14, P = 1.13%	ϕ 8 at 80

For the A series test, the dimension in term of length and width are constant, but the value are varied in the longitudinal reinforcement. The top and bottom reinforcement are differed with regard to number of bars and the diameter size of the bar. The distances between the stirrups are also varied as well as the concrete compressive strength.

In the B series tests, the beams has the same cross section as the A series but with different span length. The beams for C series initially made to be tested with different loading case to the samples but the loading cases cannot be done on the software so there is only one sample that were used in the software.

Table 4.2: Reinforcement Properties

Steel type	Diameter, mm (in.)	Yield strength, MPa (ksi)	Ultimate strength, MPa (ksi)	Elongation, %
ϕ 8	8 (0.31)	290 (42)	455 (66)	33
ϕ 12	12 (0.47)	350 (51)	540 (78)	26
ϕ 14	14 (0.55)	340 (49)	535 (78)	27

All the samples were tested with 20mm (0.79in.) clean cover for the longitudinal reinforcement bars.

2. DESIGN CALCULATIONS

The dimensions of the struts and nodes are evaluated in order to ensure that the capacity of the struts and nodes are enough to carry the truss member forces. The verification of the capacity should be done by comparing the area required with the available area. So, the effective width, $W_{req,}$ for the struts are calculated using the following equation.

Because there are three conditions for each sample, the calculation are done for each of the load amount.

For Sample A1,

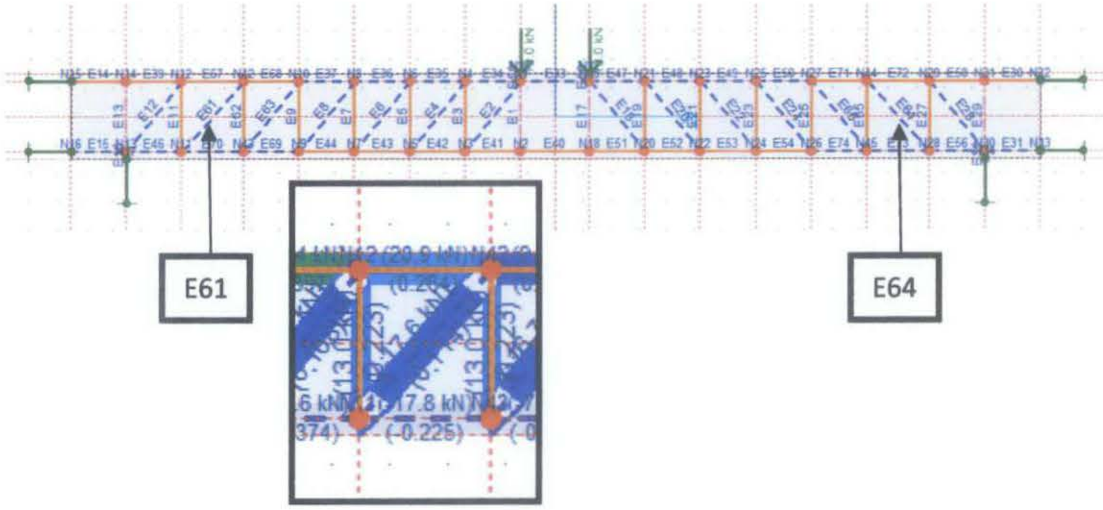
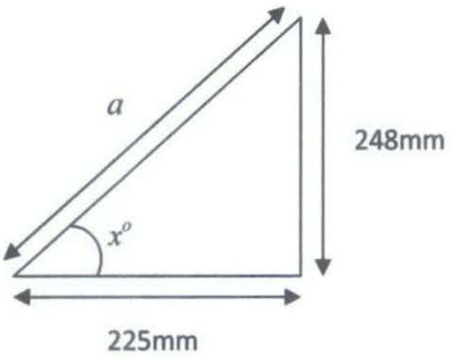


Figure 4.1: The position of Element 61 from sample A1



$\tan x^o = 248\text{mm}/225\text{mm}$

$x^o = 47.78^o$

$a = 225\text{mm} \cos 47.78^o + 248\text{mm} \sin 47.78^o$

$a = 334.86\text{mm}$

Using;

$\emptyset$ = resistance factor = 1.0

$\beta 2$ = factor for account for the effect of cracking and confining reinforcement on the effective compressive strength of a strut.

fc = specified compressive strength of concrete

b = beam width

Effective width,

a. Cracking load

$$W_{req} = \frac{Fu}{\emptyset \times 0.85 \times \beta 2 \times fc \times b}$$

$$= \frac{17.6 \times 10^3}{1 \times 0.85 \times 0.75 \times 32.3 \times 150}$$

$W_{req} = 5.70$mm

b. Support yield

$$W_{req} = \frac{79.0 \times 10^3}{1 \times 0.85 \times 0.75 \times 32.3 \times 150}$$

$W_{req} = 25.6$mm

c. Peak load

$$W_{req} = \frac{113.4 \times 10^3}{1 \times 0.85 \times 0.75 \times 32.3 \times 150}$$

$W_{req} = 36.7$mm

Where;

$\beta 2$ = 0.75; it is an inclined strut (bottle-shaped) - from Appendix B.

b = 150mm beam width

Fu = Internal force of the strut

According to the geometry, maximum width of 348.5mm can be applied, width of 50mm is proposed for Element 10 and it is between the acceptable ranges.

Element 12 and 28

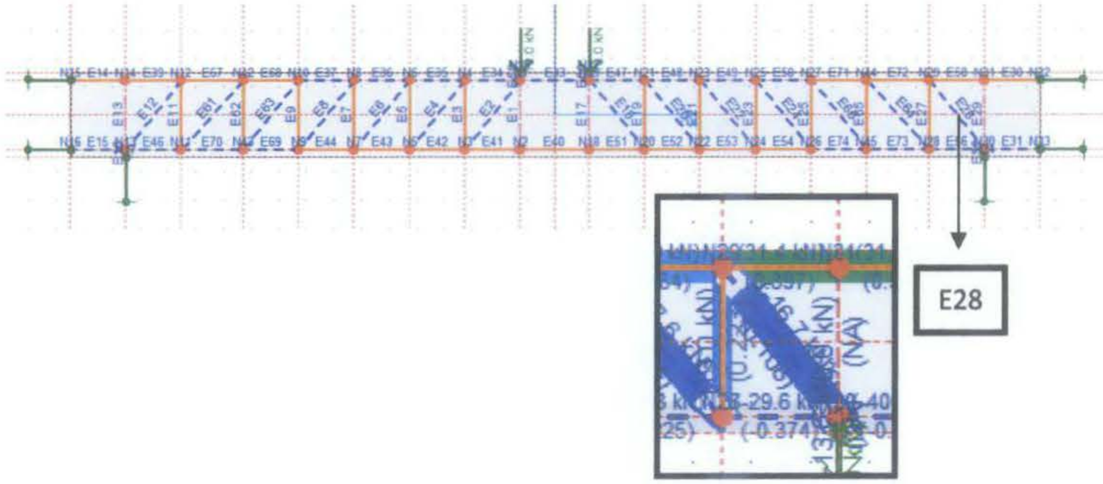
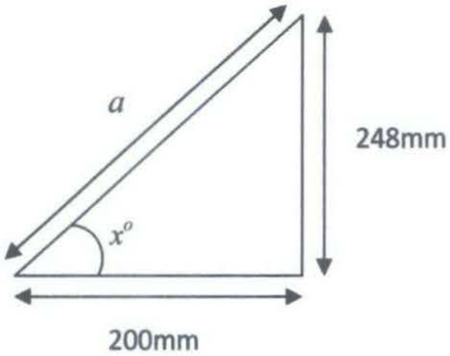


Figure 4.2: The position of Element 28 from sample A1

The strut for the element E28 and other elements with similar dimension struts are calculated,



$$\tan x^o = 248\text{mm}/200\text{mm}$$

$$x^o = 51.12^o$$

$$a = 200\text{mm} \cos 51.12^o + 248\text{mm} \sin 51.12^o$$

$$a = 318.58\text{mm}$$

Effective width,

a. Cracking load

$$W_{req} = \frac{Fu}{\emptyset \times 0.85 \times \beta 2 \times fc \times b}$$

$$= \frac{16.7 \times 10^3}{1 \times 0.85 \times 0.75 \times 32.3 \times 150}$$

$W_{req} = 5.41mm$

b. Support yield

$$W_{req} = \frac{75.2 \times 10^3}{1 \times 0.85 \times 0.75 \times 32.3 \times 150}$$

$W_{req} = 24.35mm$

c. Peak load

$$W_{req} = \frac{107.9 \times 10^3}{1 \times 0.85 \times 0.75 \times 32.3 \times 150}$$

$W_{req} = 34.93mm$

According to the geometry, maximum width of 318.58mm can be applied, width of 50mm is proposed for Element 12 and Element 28 and it is between the acceptable ranges.

Strength check for other struts can be observed from the Table 4.3 and the strength for other struts can be observed in the Appendix B.

Table 4.3: Verification of Strength of Strut of Model A1

A1												
Element	Property Type	β_2	θ	Strut Force (kN)			W_{req} (mm)			W_{prov} (mm)		
				Crack Load	Yield Load	Peak Load	Crack Load	Yield Load	Peak Load	Crack Load	Yield Load	Peak Load
12	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
28	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
2	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
4	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
6	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
8	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
10	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
18	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
20	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
22	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
24	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
26	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
63	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
66	Bottle Shaped	0.75	51.12	16.7	75.2	107.9	5.41	24.3	34.9	50	50	50
61	Bottle Shaped	0.75	47.78	17.6	79	113.4	5.7	25.6	36.7	50	50	50
64	Bottle Shaped	0.75	47.78	17.6	79	113.4	5.7	25.6	36.7	50	50	50

3. REINFORCEMENT LAYOUT

The reinforcement detailing of the beams are design based on the experimental work that have been done. For the A series beam the length is 1225mm for the beam span and the horizontal reinforcement are the 12 and 14 diameter deformed steel bar. The stirrups used in the reinforcement are 8 diameter smooth bars. The beams were measured 150mm in width and 300mm in height.

The B series consists of two 2725 and one 1975 long beam with bigger stirrups spacing with the same width and height as the A series.

The C series beams are measured smaller than the previous beam series, with 100mm width and 200mm width. The beam use 12mm diameter deformed steel bar as reinforcement.

All the specimen use 20mm clean cover for the longitudinal reinforcement and sample of the reinforcement drawing can be observed in the figures.

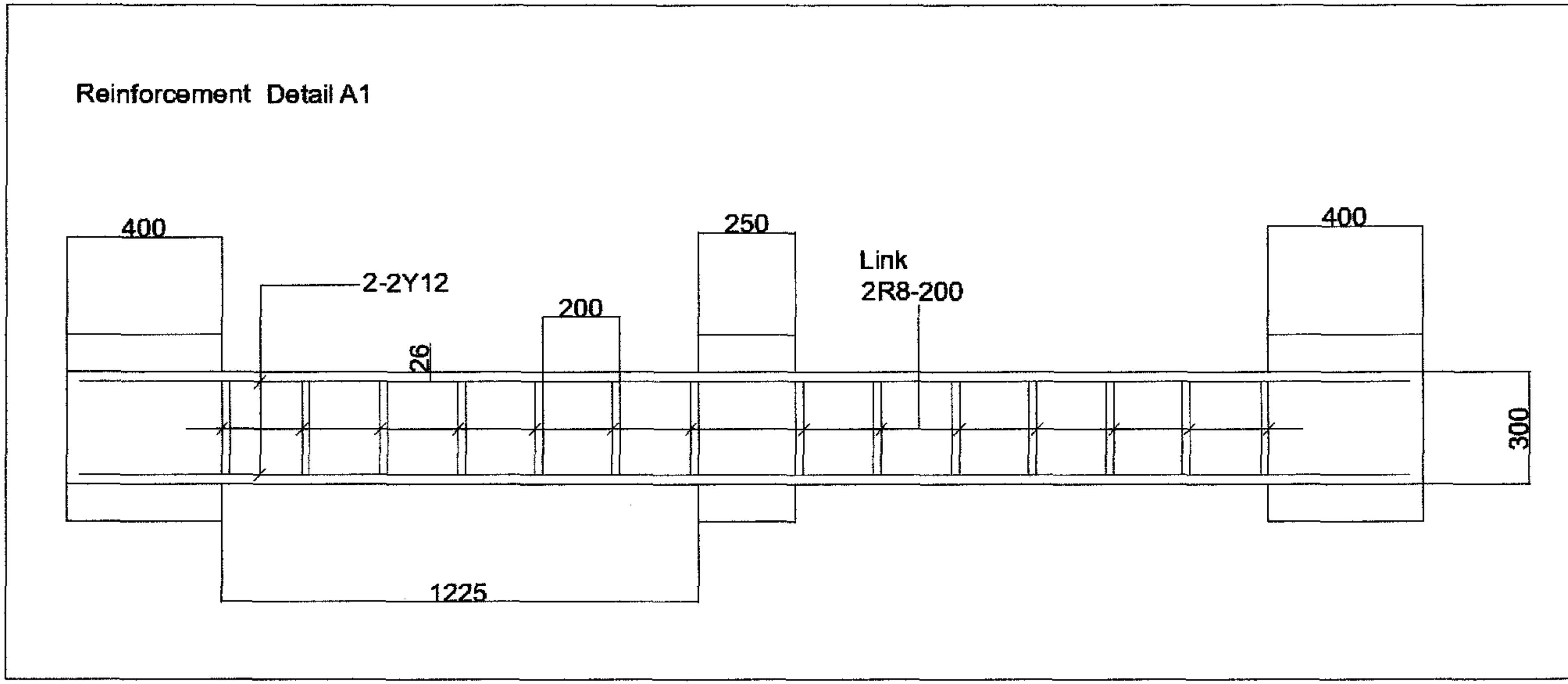


Figure 4.3: Reinforcement Detail for Beam A1

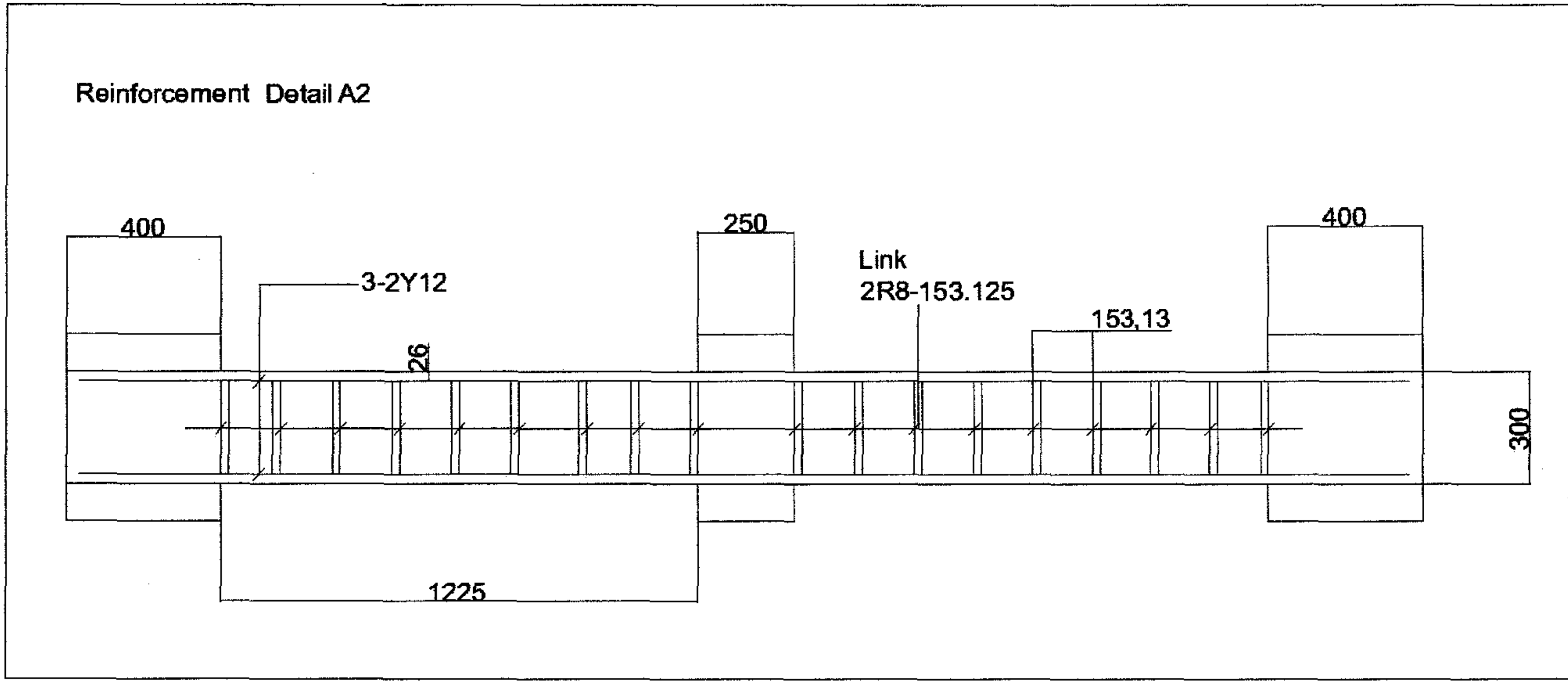


Figure 4.4: Reinforcement Detail for Beam A2

4. LOADING RESULT

Samples that are used in the project are based on the experimental result from the journal. There are three stages of the load that was taken into consideration. They are cracking load, load at yielding at supports and peak load. These loads are used in order to compare the result that was obtained by the experimental work in the journal. The experimental result also produced the horizontal reaction which can be compared with the software analysis. The Table 4.4 shows the result from the experiment.

Table 4.4: Loading and horizontal reaction list

Test	Cracking load, kN (kip)	Load at yielding at supports, kN (kip)	Peak load, kN (kip)	Horizontal reaction, kN (kip) at peak load	
				Experiment Result	CAST Result
A1	26	117	168	281	259.1
A2	30	148	221	318	332.8
A3	29	152	246	296	373.6
A4	24	82.3	147	309	227.6
A5	29	129	198	340	298.2
A6	27	153	226	177	343.1
B1	13	105	125	211	289.9
B2	10	73.2	82.9	190	232.2
B3	9.9	65	74.7	172	257.8
C1	8.0	48.2	60.9	91.6	153.7

5. CAST RESULT AND ANALYSIS

The result that was generated from the CAST software has provided with much information of the internal forces that was acting during the three stages of loading. The forces that were acting internally can be view and compared from the load at cracking until the peak load that can be obtained by the beams.

As an example, Figure 4.5 shows sample A3 at cracking load. The ratios are all below 1 and the struts are still intact. When the load is increased in Figure 4.6, the beam have already failing at the area highlighted with red where the Stress Ratio is more than 1 and we can see clearly in Figure 4.7 that the structure is failing because of the overstressed area at the beam.

Similar result are also obtained from other beams and it is can manipulated in order to obtain a safer design in reinforcing concrete structures such as beams which is more susceptible to progressive collapse.

The result for the CAST analysis and detailed information can be observed in the Appendix C

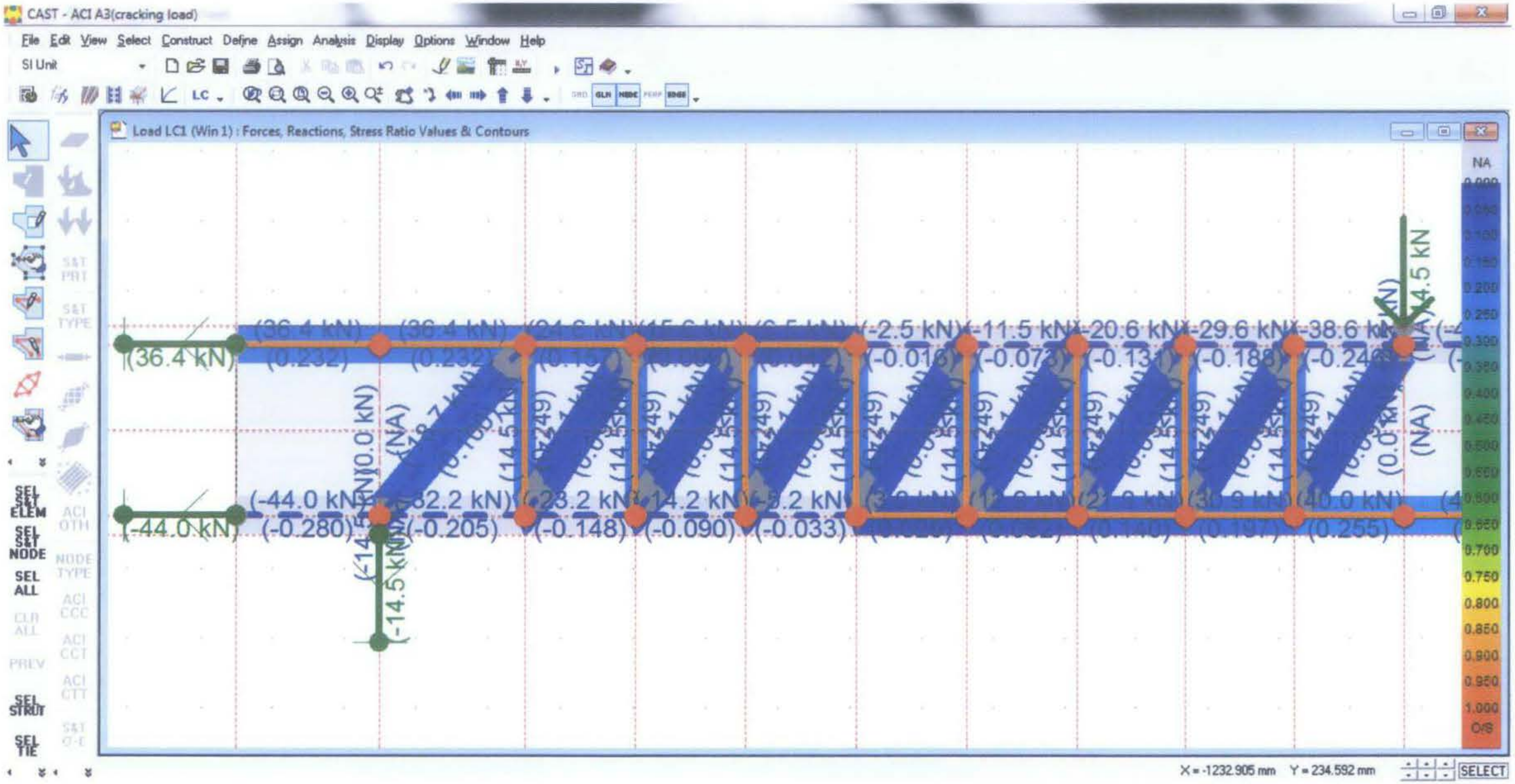


Figure 4.5: CAST loading result for sample A3 at cracking load

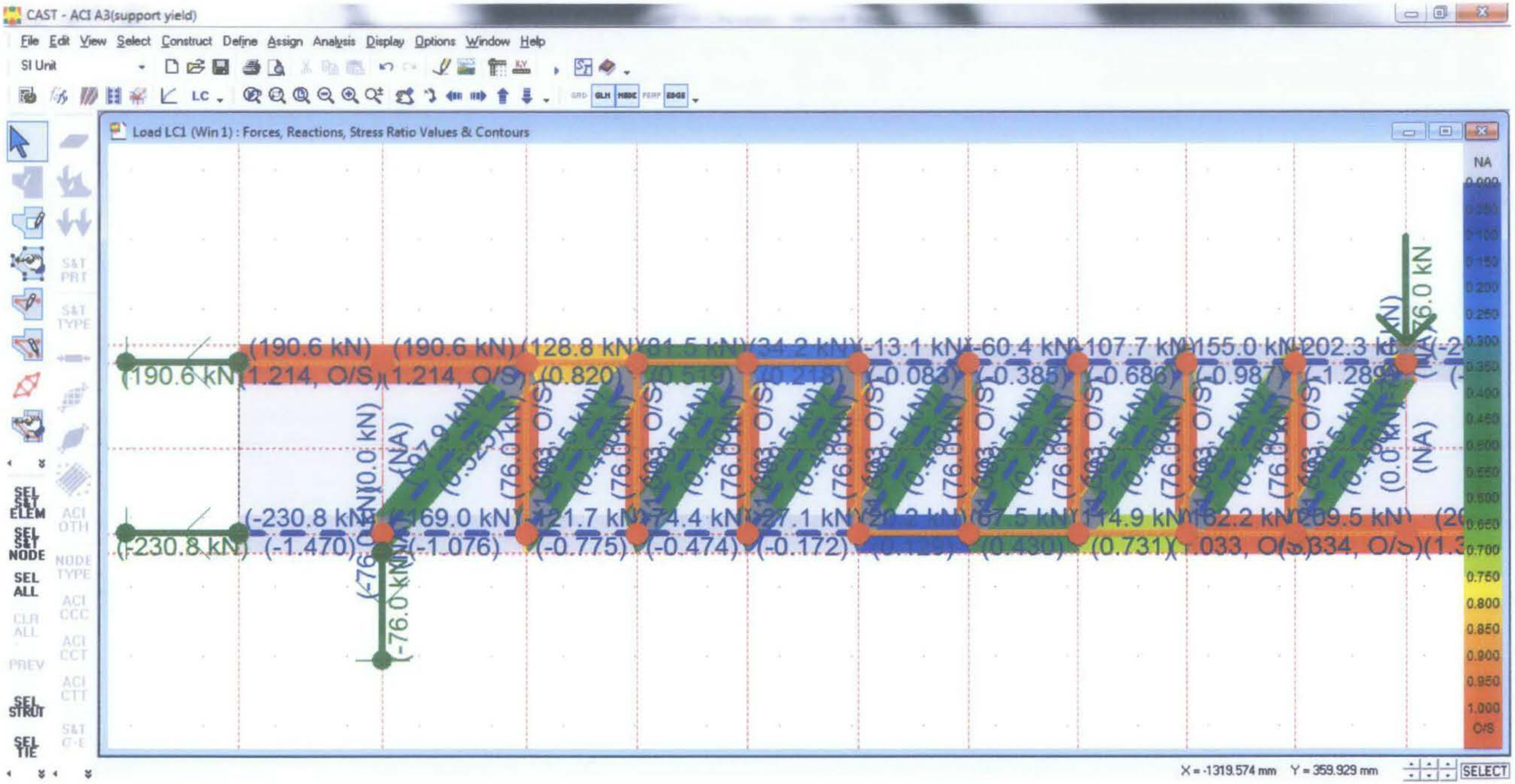


Figure 4.6: CAST loading result for sample A3 at support yield load

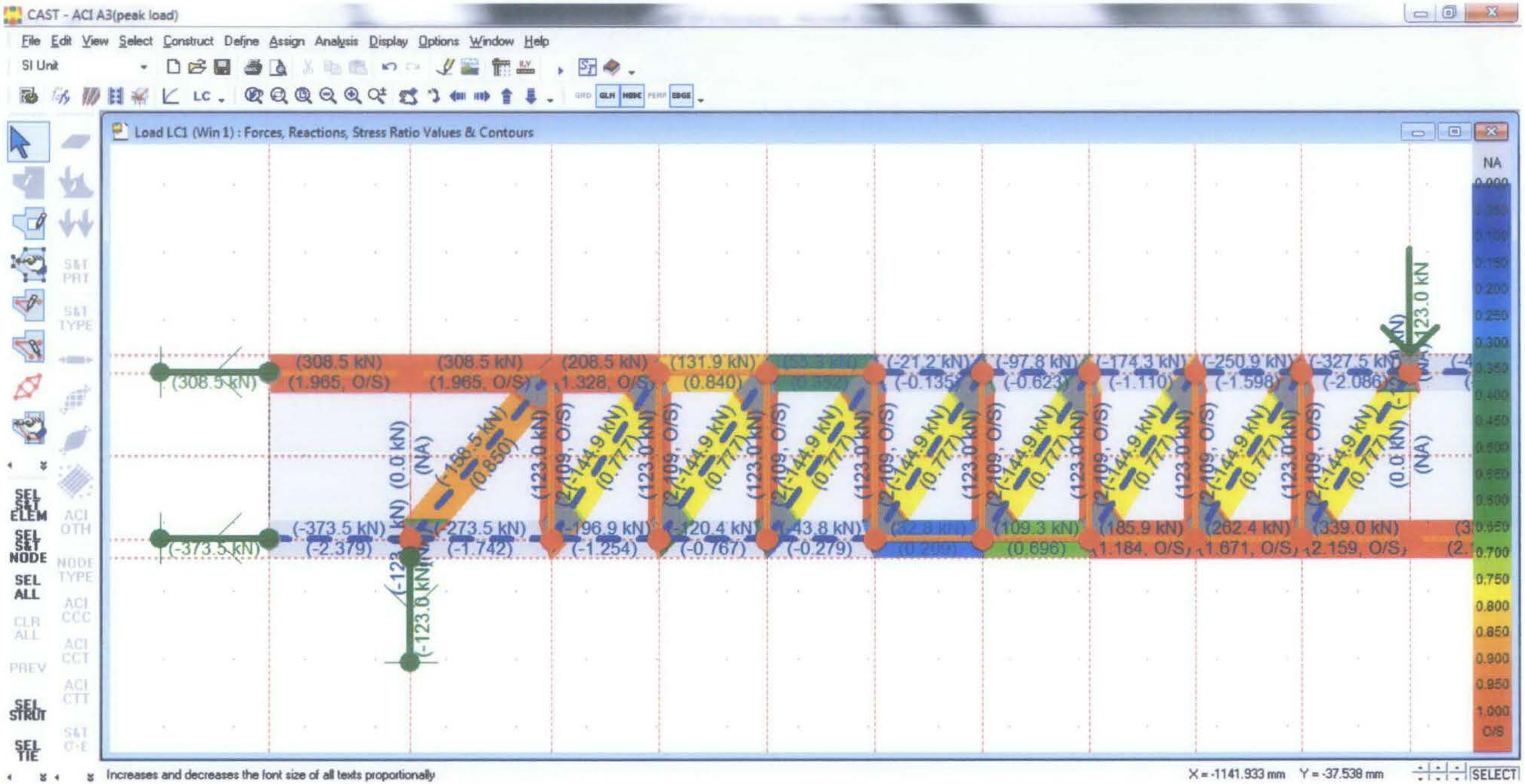


Figure 4.7: CAST loading result for sample A3 at peak load

Yield at Support Load

From the CAST analysis that we have done, we can compare the loading to achieve the support yield to the loading that was obtained from the experimental result. The loading is adjusted in order to obtain the loading that will initiate the yielding of the reinforcement at the support location. This result displays the actual loading that the beam can undergo by showing the ratio of the beam overstress factor.

Table 4.5: Load at yielding at supports for experiment and CAST analysis

Test	Load at yielding at supports, kN (kip)	
	Experimental Result	CAST Analysis
A1	117	65.56
A2	148	95.5
A3	152	125.2
A4	82.3	65.3
A5	129	95.5
A6	153	125.2
B1	105	79.7
B2	73.2	58.2
B3	65	58.2
C1	48.2	38

As an example we can see the differences between the experimental result and by using the CAST analysis in the Figure 4.8 and Figure 4.9. In the Figure 4.8 the loading for the sample B3 is 65kN and the overstress ratio at the support is already 1.117. By using CAST analysis, we can predict the yielding at the support more accurately as we can see in the Figure 4.9 and Figure 4.11

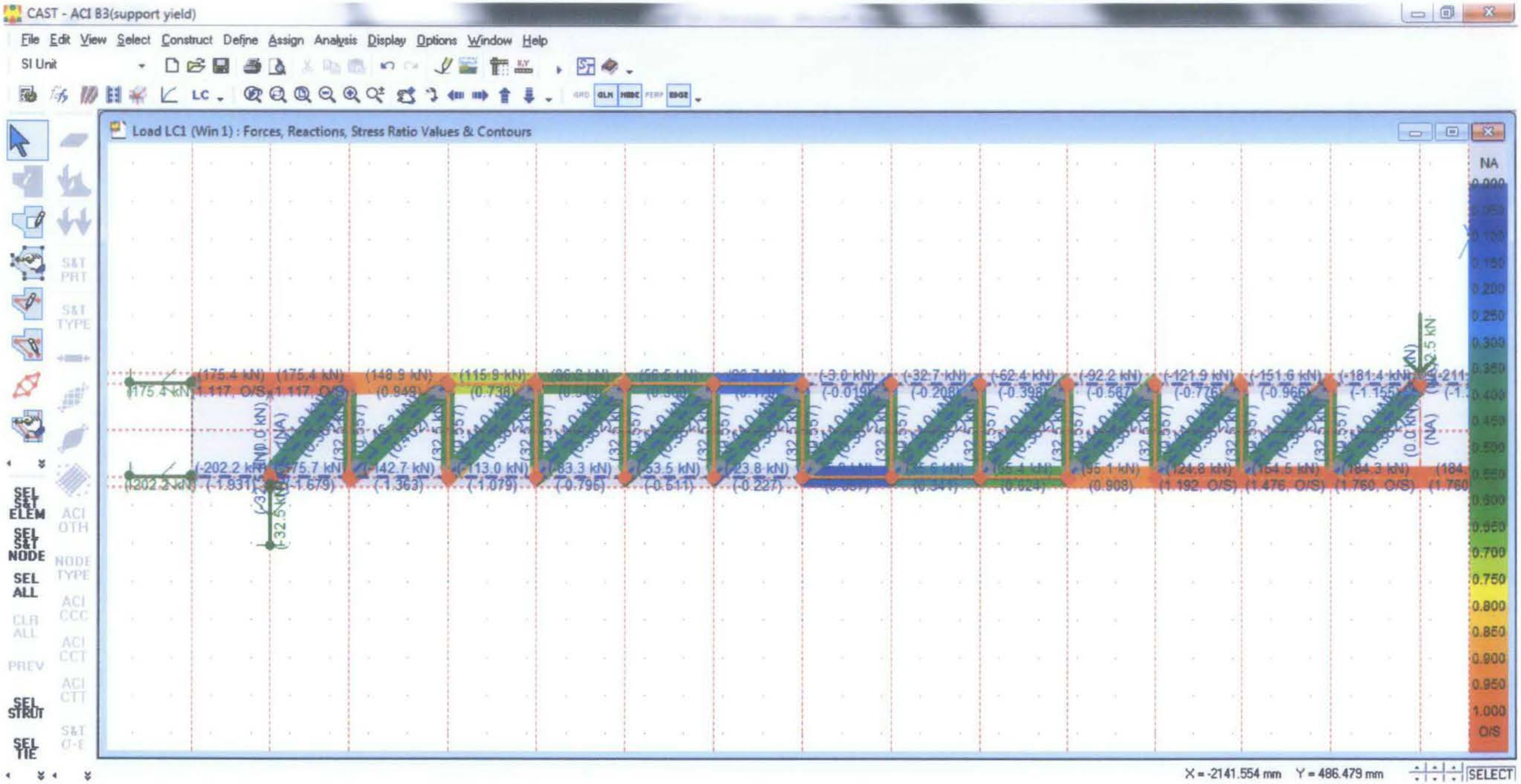


Figure 4.8: Support yield result for sample B3

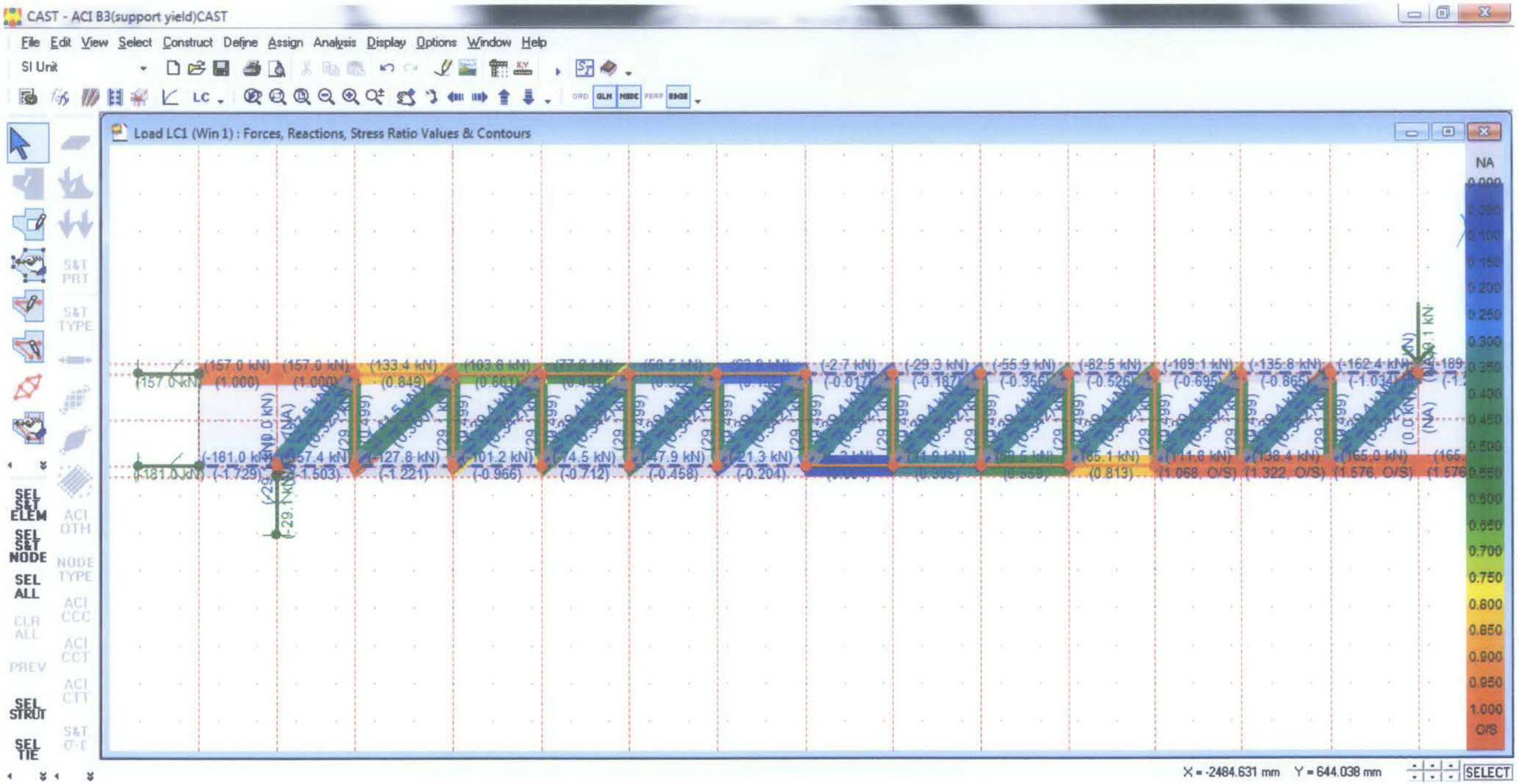


Figure 4.9: CAST loading result for sample B3 for support yield

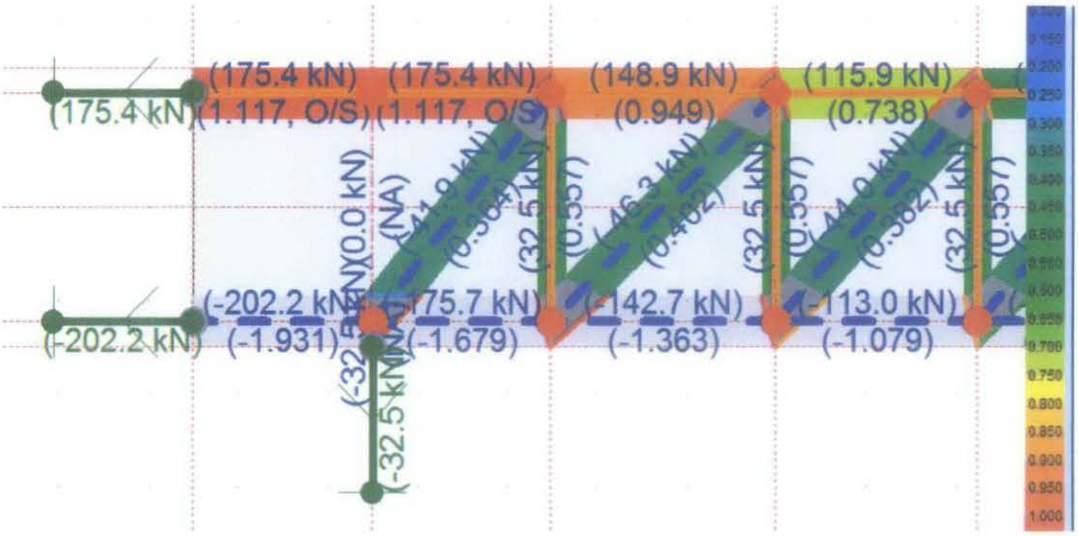


Figure 4.10: CAST loading result for sample B3 at peak load

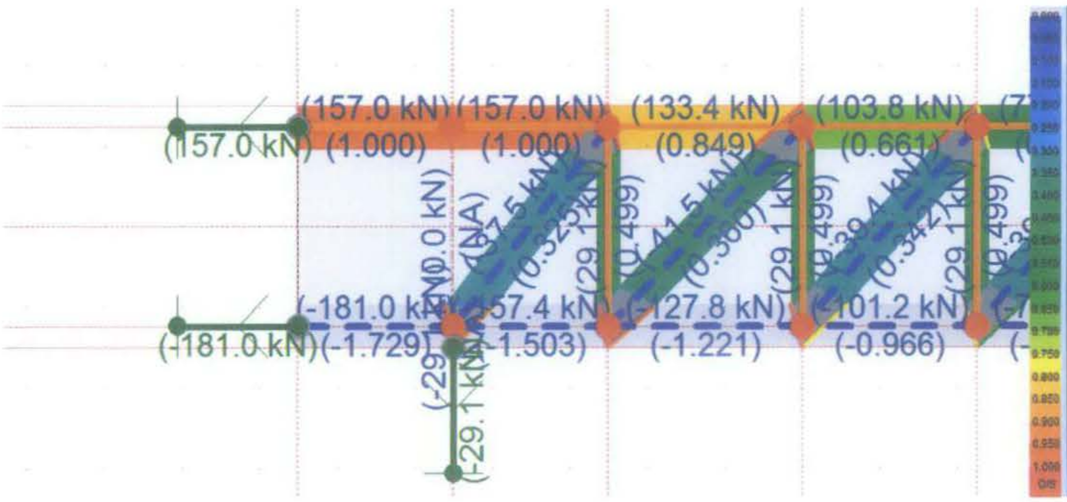


Figure 4.11: CAST loading result for sample B3 at peak load

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

Strut and Tie Model analysis using CAST software has been able to analyze the load distribution of a partially collapsed structure by displaying the individual stress of the element in the structure. By applying the dimension and forces based on the journal, we are able to observe the effect and reaction of individual samples.

The analyses demonstrate that the load is distributed mostly on the middle bottom reinforcement area and at the support area of the top reinforcement area. This information has given us an insight of where we could anticipate the force will be concentrated. The analysis also provide where are the compression and tension elements are located that is useful for considering the reinforcing material to be used in the designing.

All the data gained are valuable in order to help in designing better structure and resisting progressive collapse in reinforced concrete structure. The result has provided us with better understanding in how the load is distributed when the force is applied towards the structure.

There are also some recommendation for the analysis where we can analyze the data first using Strut-and-Tie Model CAST software and then do the experimental work to provide some justification on the software reliability.

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