**ADDIS ABABA UNIVERSITY**

**ADDIS ABABA INSTITUTEOF TECHNOLOGY**

**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**

# Cost and structural efficiency comparison between rectangular hollow core and solid concrete piers

**By**

**Eyob Tilahun Mengiste**

**May 2016**

# Addis Ababa University
# School of Graduate Studies

## Cost and structural efficiency comparison between rectangular hollow core and solid concrete piers

**Eyob Tilahun**

**Approved By the Board of Examiners**

______________________________ ______________________

**Advisor**

______________________________ ______________________

**Internal Examiner**

______________________________ ______________________

**External Examiner**

______________________________ ______________________

**Chairman**

# Addis Ababa University
# School of Graduate Studies

# Cost and structural efficiency comparison between rectangular hollow core and solid concrete piers

**Eyob Tilahun Mengiste**

**Advisor: Asnake Adamu Dr.**

**A thesis submitted to the school of Graduate studies of Addis Ababa University in practical fulfillment of the requirements for the Degree of Master of Science in Structural Engineering**

**Addis Ababa**
**May 2016**

# Cost and structural efficiency comparison between Rectangular Hollow core and solid concrete piers

**by**

**Eyob Tilahun Mengiste**

**(Abstract)**

Cement is the second most used construction material on earth. Beside the declining abundance of the raw materials to be used for the production of cement, the power and energy saving problems has become the main questions of the world. The search for new smart construction materials and the recycling procedures are currently under research and practices. The efficient use of the materials could also be a significant factor that could give out a valuable contribution in this regard.

Economy, sustainability and environmental changes become the main challenges of engineers in the 21st century. In this thesis, analytic comparison is made by minimizing the volume of concrete used for long and slender concrete piers in order to get buckling and axial compression load resistance as well as economically efficient sections. Bridges of several height are designed and examined under similar Structural loads.

During casting, Concrete experiences different stages of hydrations. These chemical reactions can be categorized as exothermic chemical reactions in which, heat will be released. The amount of high heat released in the core center of the member will not be migrated to the environment so easily and it will result in thermal stress and early stage cracks at the outer most surfaces. The crack formation can be kept minimum by reducing the Concrete volume, which actually results in less thermal stress.

The ductility property of various bridge piers are evaluated and hollow sectioned piers are found to be more ductile. For bridges built in the seismic areas, plastic moment capacity is calculated for piers of hollow and solid core sections. This sample piers are designed with various height and the result indicate that sections of hollow rectangular piers are more ductile and economical.

# Acknowledgement

First and Foremost, I thank God for giving me strength and ability to work and write this thesis along with everything.

I would also like to express my deepest gratitude to my Master thesis advisor Dr. Asnake Adamu. He spends very much time on guiding me to write this thesis. He helped me as best as he can even in my very distant situation. Special thanks are given to AAiT Department of Civil Engineering, Most of my foundations are built in the class lectures and laboratory experiments I had as a student.

I owe the deepest gratitude for NYUAD Engineering faculty who were giving me suggestions and ideas on how to carry on my writing while I was loaded with very tight schedule.

My foremost gratitude is directed towards my wife, for all her enthusiastic support, love and all round help. I feel grateful for my brother, Babi, who were supporting me endlessly through all the time. He was available on every occasion to fulfil my numerous help requests. I do appreciate his heart felt kindness along with all what he did. Also my respected family, this all is due to your strong parenting and bright directions. I am lucky to be your son.

# Contents

# Figures

# Tables

# 1. Introduction

## 1.1 Background

Development of modern infrastructure needs the application and utilization of modern technologies for the efficient and more economical outputs. Fast ground transportation means like railway and motor way demands less gradient on the vertical profiles disregarding the terrain conditions. For the versatile terrain category like Ethiopia, tunnels and long span bridges play an important role to attain the criteria of the designers for a specific case.

The advantage of a tubular section over a solid concrete sections is more flexible. The weight per length ratio of a tubular section is less than the equivalent solid sections. This scenario decreases the earthquake load intensity to act on the structural member. In addition to the material cost reduction, its less weight eliminates high transportation costs for precast members. Considering the economical perspective of constructing long and slender bridge piers, the volume of concrete to be utilized will affect the weight of the entire structure and will escalate the cost of the foundation to be designed. The magnitude of a bending effect is not only governed by the height of the Pier itself, but is also governed by the weight and axial stress emitted from the superstructure. Hence, the span of the superstructure also plays a major role on magnifying the flexural and axial action on the pier system to be utilized.

To make the study more feasible and appropriate for the current Ethiopian Bridge design and construction practices, this thesis will have boundaries on RC concrete bridges with simple span orientation. Economic efficiency starts with minimizing the total quantity of material to be used and completing the design and construction project within a very limited frame of time, which of course will save the man hour payments. This thesis will cover the cost benefit analysis of Hollow Core concrete Pier design and Utilization over the conventional in practice solid core concrete pier.

Utilization of Hollow Concrete piers could be an efficient way of minimizing the cost of construction or even can increase the ductility property and seismic resistance. Hollow bridge piers accommodate the high moment and shear demands by reducing the self-weight and the high bearing demand on pile foundation maximizing the structural efficiency of the strength–mass ratios and reducing the mass contribution of the column to seismic response.

Compared to solid piers, Hollow core piers have the advantage of having significant reduction in the volume of the material, large reduction of dead load, high bending and torsional stiffness. When a hollow cross section exceeds a width to depth ratio of 3, the cost of extra formwork probably exceeds the material savings. But the reduction in dead load can still produce appreciable saving in formwork cost.

In this paper, high emphasis will be given to the seismic capacity of the piers. In both the hollow core and the solid core cross sections, the designed member will be examined for its inelastic redistribution and the properties of plastic hinge regions.

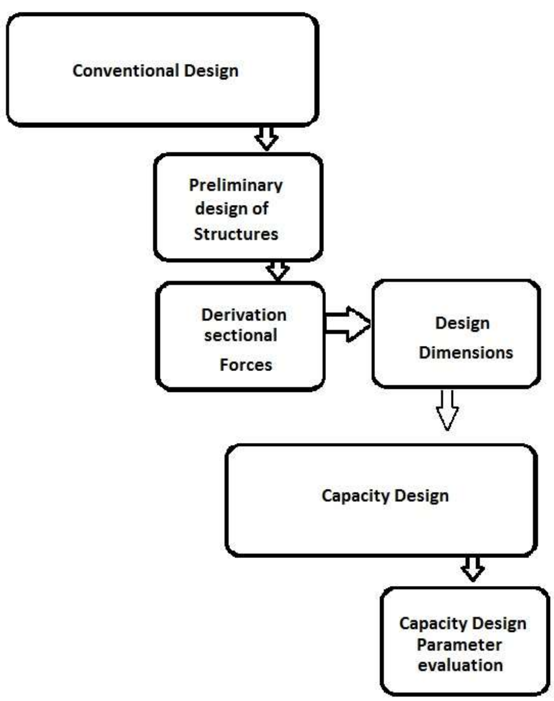



*Figure 1-1: Proposed analysis and design procedure*

The result out of Hollow and Solid models, will be compared in terms of material quantity and structural advantages.

## 1.2 Objectives and scope of study

### 1.2.1 Objectives

As the development hikes up, the need for infrastructures will rapidly grow. This is because of the need to reach inaccessible recourses, to provide facilities for the community at the far end and to provide an infrastructural network. Extensive and massive Bridges with long piers will be necessary to be constructed. As the height of the piers goes up, the monetary and safety questions will force a designer to look for alternative pier sections.

This thesis will address the question by providing a Hollow concrete pier as a best alternative.

The objective of this thesis is to:

Evaluate the economic benefits of Hollow core Concrete Piers over the widely practiced Solid Concrete piers. Using performance based analysis and design of seismic loads.

### 1.2.2 Scope of the study

Structures are assumed to be located in the seismic locations of Ethiopia.

- Usual load cases are used during the conventional structural design scheme
- Orthogonal orientation of column is discussed
- Prismatic (constant) cross section is considered
- Torsion is not considered

## 1.3 Structure of the thesis

This thesis is organized into chapters. Chapters are also subdivided into sub chapters.

Chapter 01 deals with introduction in which the background information is provided. Additionally, objectives, scope and thesis structure are briefed.

Chapter 02 is devoted about the literature review where all the topics covered in this thesis are observed from the perspectives of various notable writers, codes and standards; and are summarized in this chapter. Consensus obtained out of this portion is used as the basic roadmap for the work done on chapter 3 and 4.

Chapter 04 is about the path followed during the modeling and design of the cases under consideration. In this chapter, cases to be considered are set. Geometric and load modeling are carried out and briefed. Brief description including results of conventional and capacity design carried out on the models are also included. To show the effect of size enlargement, heat evolution analysis is also included.

Chapter 05 summarizes all the work. Comparison is made based on economy and structural efficiency. This chapter deals with comparison results. These results are discussed and shown using numbers, charts and brief discussions.

Chapter 06 concludes the outcome of the study. This chapter is concentrated on conclusions and recommendations. In this portion of the study, the final findings are briefly discussed. Recommendations for further study is also provided.

# 2. Literature review

## 2.1 General

Hollow concrete section is often used for column design particularly for very tall bridge columns in seismic areas for reducing the mass and therefore minimizing the self-weight contribution to the inertial mode of vibration during the earthquake. The hollow columns also enables foundation dimensions and thus save the construction cost substantially. Therefore, these advantages have promoted the use of hollow pier columns instead of similar solid members. On the other hand, the seismic behavior of the hollow columns has been controversial due to lack of understanding [31]

Bridges often rely solely on the capacity of the piers to sustain large displacement without collapsing. Failure of bridge piers often cause collapse or failure of bridge span, as it is evident from several major earthquakes. Hence, bridge piers are usually designated as the first structural element to dissipate seismic energy well beyond their elastic limit [13]. Hollow core piers are often used in the construction of long- span structures like in balanced cantilever bridges, cable stayed bridges and in bridges crossing deep valleys. Comparing to solid piers, hollow piers have the advantage of having significant reduction in the volume of material, large reduction of dead load and high bending and torsional stiffness. Despite its wide use, research on the seismic behavior of such piers is limited. Even the most modern codes of practice do not recognize specific problems associated with hollow piers, probably as the consequence of lack of knowledge [3]. However, these types of piers are commonly considered to be vulnerable to seismic action [13].

## 2.2 Conventional Pier Column Design

Design of a pier includes the geometric Size determination, reinforcement Area calculation, and determination of height, cross sectional dimensions, stress and cost. While optimizing Cost, Geometric Size and efficiency towards stress resistance, different Design approaches and theories have been practiced through the history.

From among the major design Methods, working stress design method and limit state design approach were the most influential.
In this thesis, emphasis is placed to LRFD (Load Resistance Factor Method) for designs of structures under consideration.

Reinforced Concrete pier design includes the derivation of sectional properties, loading and configuration. As the first step of design, the pier column shall be modeled to handle shear and moment transferred from the superstructure efficiently. Factored load combinations as per AASHTO bridge design manual shall be transferred without causing instability. Suspected instability cases shall be balanced using the geometry configuration. During this preliminary design stage the material non linearity is not taken into account and the seismic capacity of the structure will not be investigated in detailed manner.
During the utilization of this method, it is expected to obtain significant variation of axial load resisting capacity between hollow and solid core pier columns. Slenderness of the concrete section which depends of the length of the column and the cross sectional parameters in addition to the magnitude of Euler's critical load will determine the advantage of one model over the other.

### 2.2.1 Code Recommendations

For General Applicability, in this thesis AASHTO LRFD Bridge Design Manual and ACI code requirements are widely applied. AASHTO Bridge design manual gives a huge emphasis for the LRFD design technique especially for reinforced concrete Deck slab bridges.

The Load Resistance Factor Design method incorporates various sorts of load combinations. From among this: Strength limit state, Extreme Events, Service and Fatigue are the major divisions that AASHTO describes extensive events.

### 2.2.2 Gravity Load Modeling

**Permanent loads**: in case of pier design and analysis, the most governing load effect is from self-weight of the superstructure and pier concrete weight. In addition to the structural volume of construction, material that the pier will carry, significant magnitude of weight from the aesthetic purpose claddings, utility fixtures and earth pressure could also be taken as the source of the permanent load on the pier.

**Live loads**: live loads are taken as the load kind, which have a variable behavior along the service time of the structure. These loads could be categorized as gravity loads and lateral loads. Vehicular loads are the major forms of gravity live loads on bridges. AASHTO manual regulates the magnitude and intensity of the vehicular loads using the factors from the multiple presence factors and the amount of the Design vehicle configuration. The design load configurations vary to be HS- 93, HS 20 or HS 2. Based on the support conditions, the lane configuration condition, the assigned task, the volume of the traffic forecast and the span length. In addition to the direct loads from the vehicles, there are also indirect considerations of loads. The impact of the wheel to the Bridge element will emit some energy to cause a Dynamic Impact load, which is to be designated by IM load within the AASHTO bridge design manual. This load is expressed as 15% to 75% based on the design technique to be used and also based on the location of the structural element under consideration

In addition to the Vehicular gravity loads, there are also some considerations of the centrifugal forces, Braking forces and collision forces, which are under the live load category.

**Water Load**s: AASHTO Bridge design manual considers the uplift effects of the water on the structural element. In addition, the longitudinal and lateral pressures emitted by the water wave are also modeled through experimental formulas. According to AASHTO, wave loads shall be determined to adjust the scour depth requirements for the foundation design and also to fix the height of the part of the pier to be buried under the water surface and the ground level.

### 2.2.3 Lateral Load Modeling

**Wind Loads**: Wind pressure effects on the Structural element and on the Vehicle element is expressed through empirical formulas and modeled accordingly.

**Earthquake effect:** the earthquake effects on the buried structures are not to be considered on the AASHTO specifications. But for structures with significant height, direct seismic effects and additional seismic effects from the soil liquefaction and slope movements need to be considered.

**Breaking Force:** This is a force is emitted from the action of a design vehicle on superstructure. It can be modeled using formulas given on design codes and standards.

**Centrifugal Force:** Is the effect of normal acceleration of a vehicle when it is traveling along a curved path. Varies with the design speed of the roadway. In this thesis, all of bridges under consideration have straight horizontal alignment and centrifugal forces are assumed to be zero.

**Water Pressure:** Stream pressure on a substructure can be a major load depending on the alignment of the river and the orientation of the bridge. This thesis assumes the purpose of the bridges under consideration is to cross uneven geographic terrain (rolling or mountainous). Accordingly stream pressure is excluded from the analysis.

## 2.3 Bi-axial Bending on Rectangular Solid and Hollow Sections

During the determination of the capacity of the column it is necessary to consider the resistance of the section against the interacted loads. The input of influences from moments and the axial compression load has to be considered. For a compression members of a structure it is necessary to draw the failure interaction diagrams and examine the intensity of actions. Interaction diagram is a graph illustrating the capacity of a structural concrete member to resist a range of combinations of moment and axial force. By changing the location of the neutral axis, giving different size of compression and tension zones, each case will lead to a different capacity calculated from the strain distribution. First the section is in pure compressions, then it will be over reinforced until it reaches the point where it is balanced resigned.

### 2.3.1 Bresler's reciprocal Load Equation

In 1960 [10], Bresler used the failure surface concept to propose two equations which would represent the approximate failure surface. When the inverse of the failure load, $1/P_u$ is approximated as:

$$\frac{1}{P_{ni}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o} \text{ ------------------------------------------ } 2.1$$

Where $P_{ni}$ – the nominal axial load capacity of the section when the load is placed at a given eccentricity along both axis

$P_{nx}$ is the normal axial load capacity of the section when the load is placed at an eccentricity towards X axis

$P_o$ is the nominal axial load capacity of the section when the load is placed with zero eccentricity.

### 2.3.2 Load Contour Equation

Bresler described another failure thrust is plotted against the associated failure moments, $M_{xo}$ and $M_{yo}$, about two major axes. At a level of axial load $P_u$, the failure moments corresponding to the load can be related as:

$$[\frac{M_{xu}}{M_{xo}}]^{a_1} + [\frac{M_{yu}}{M_{yo}}]^{b_1} = 1.0 \text{ ----------------------- } 2.2$$

Where $M_{xo}$ and $M_{yo}$ moments at failure load, $P_u$ about x axis and y- axis, respectively.

$M_{xo}$ = failure moment about x- axis when the axial load Pu acts with uniaxial eccentricity producing moment about the x-axis only. ($M_y$ =0)

$M_{yo}$ = failure of moment about y-axis when axial load $p_u$ acts with uniaxial eccentricity producing moment about the y-axis only ($M_x$ = 0)

$a_1$ and $b_1$ are exponents depending on the shape and column dimensions, the amount and distribution of steel and properties of concrete. And this exponents are assumed to be equal for a rectangle.

### 2.3.3 Capacity Reduction Factors

Depending on the shape and the shear reinforcement selection ϕ values vary. Lower ϕ values are applicable for compression controlled sections because of smaller ductility.

ACI code section 9.3.2.2 states that ϕ for a particular column may be increased linearly from 0.65 or 0.70 to 0.90 as the net tensile strain $\epsilon_t$ increases from 0.002 to 0.005, where the condition undergoes transition from compression controlled to the tensile controlled as the strain increases.

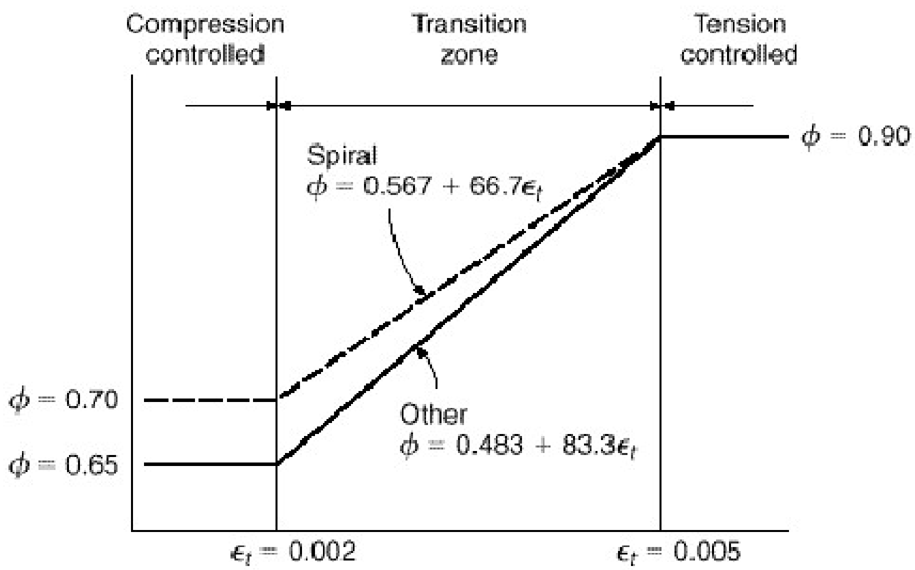



*Figure 2-1: Strain - reduction factor ACI 318-11*

### 2.3.4 Interaction Diagram Modeling

For a rectangular section, the value of $a_1$ and $b_1$ in equation 2.2 are considered to be equal for a rectangular section where,

$$a_1 \text{ or } b_1 = \log 0.5/\log \beta \quad \text{-------------------------------------} \ 2.3$$

Where, β depends primarily on the ratio of Pn/Po and to a lesser degree on the reinforcement ratio, arrangement and strength. Figure 3 gives values the ultimate moment capacity of a column section for different values of β. For lightly loaded columns variation of β is in order of 0.5 to 0.7. As the ratio $P_n/P_o$ increases, the value of β also increases. A lower value of β can be used for a purpose of conservative design.

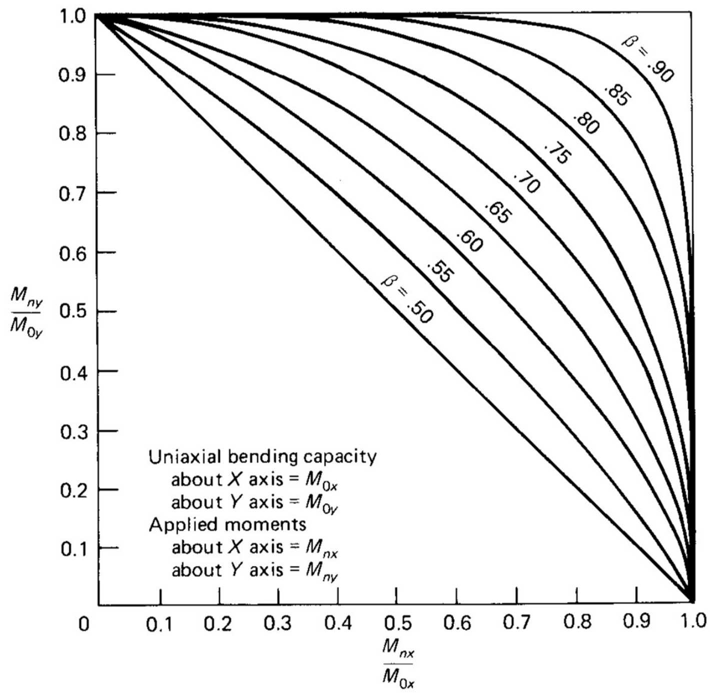



*Figure 2-2: Biaxial moment relationship*

### 2.3.5 Confining Reinforcement Design

Lateral reinforcement for columns normally consists of individual ties or closely spaced continuous spirals. These reinforcements are essential components of the reinforcement cage. Lateral ties and spirals are primarily used to hold longitudinal bars in place. They provide lateral bracing to longitudinal bars to prevent local bar buckling under heavy compression loads. The structural performance of bars can be significantly be affected by the amount and detailing of lateral reinforcement. Columns that are subjected to load reversals has significant implications on their strength and ductility.

Axially loaded columns reinforced with ties generally fail in a sudden and brittle manner when the column capacity is reached. At the failure load, concrete crushes and bulges outward between the ties, buckling of longitudinal bars takes place, and the column cannot withstand the applied load. If the column is reinforced with spiral lateral reinforcement, concrete within the core area is confined. When the same failure load is applied, the core concrete, because of its confinement within the spirals, cannot shear outwards, and the longitudinal bars are not allowed to buckle. At this load level, the unconfined shell concrete spells off, but the column core continues carrying the applied load depending on the amount of spiral reinforcement, the column carrying loads at higher inelastic deformation levels.

### 2.3.6 Long Column effects

When a column bends or deflects laterally, its axial load will cause an increased column moment. This moment will be superimposed onto any moment already in the column. This magnitude will reduce the axial load capacity of the column significantly. ACI code section 10.10.2 states that the design of a compression member should desirably be based on a theoretical analysis of the structure that takes into account the effects of axial load, moments, deflections, duration of loads, varying member sizes and end conditions.

The ACI code (10.10.5) provides an approximate method for determining slenderness effects. This method is based on a moment magnifier which is to be multiplied by the larger moment at the end of the column denoted as $M_2$ and this value will be used for the design.

## 2.4 Capacity Design Method

This method of design addresses the special nature of inelastic structural responses of seismic excitations [1]. In structures designed for ductile seismic response, the locations of potential plastic hinge regions are deliberately chosen to enable the development of a sustainable plastic mechanism. Plastic hinge regions are designed and detailed for adequate ductility. All other regions are provided with additional capacity so that they remain elastic when the chosen plastic hinges develop their over strength [15]

According to AASHTO guide specification for LRFD Seismic Bridge design, depending on the Seismic Deign category (SDC) a structure's design requirements shall be compared to the capacity of the section. Displacement capacity Verification is required for individual piers and bents. Although it is recognized that force redistribution may occur as the displacement increases (AASHTO 2011).

### 2.4.1 Demand and Capacity

The global displacement demand estimate $\Delta_D$ for ordinary bridges can be determined by linear elastic analysis utilizing effective section properties. The global structural displacement $\Delta_D$ is the total displacement at a particular location within the structure or subsystem. The global displacement will include components attributed to foundation flexibility, $\Delta_f$ , flexibility of capacity protected components such as bent caps $\Delta_B$ and the flexibility attributed to elastic and inelastic response of ductile members $\Delta_y$ and $\Delta_p$ respectively. (figure 2-3)

The Displacement ductility demand is a measure of the imposed post-elastic deformation on a member displacement ductility,

$$\mu_D = \Delta_D / \Delta_{Y\,i} \quad \text{------------------------------------------------} \quad 2.4$$

Where,

$\Delta_D$ is the estimated global frame displacement demand

$\Delta_{Y\,i}$ is the yield displacement of the subsystem from its initial position to the formation of plastic hinge

For a single column bent, supported on a fixed foundation, the target ductility demand should be less than or equal to 5.

The structure shall be designed to resist the internal forces generated when the structure reaches its collapse limit state. The collapse limit state is defined as the condition when a sufficient number of plastic hinges have formed within the structure to create a local collapse mechanism.

The column design moments shall be determined by the idealized capacity of the column's cross section, $M_p^{col}$ . The over strength moment $M_o^{col}$ the associated shear $V_o^{col}$ and the moment distribution characteristics of the structure system shall determine the design moments for the capacity protected components adjacent to the column.

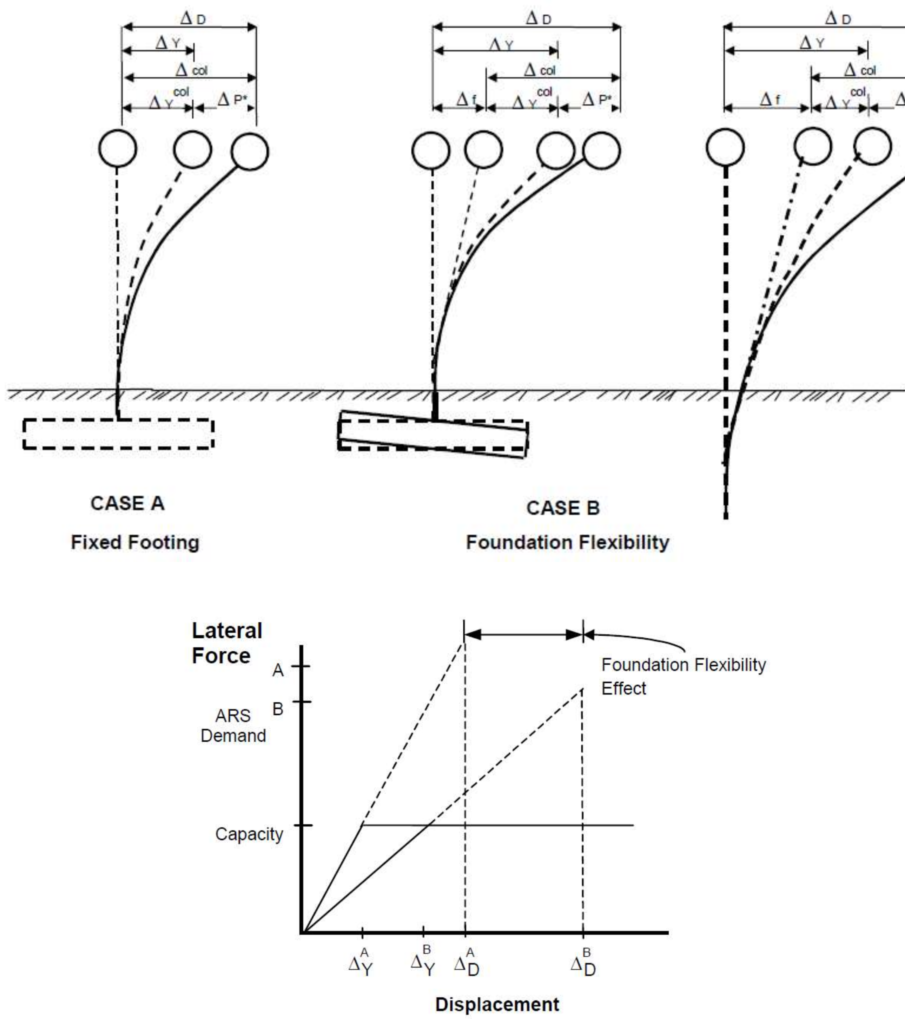



*Figure 2-3: Force deflection curve of a single Pier - caltrans*

The column shear demand and the shear demand transferred to adjacent components shall be the shear force $V_o^{col}$ associated with the overstrength moment $M_o^{col}$ .The designer shall consider all potential plastic hinge locations to insure the maximum possible shear demand has been determined. [21]

The shear demand for pier walls in the weak direction shall be calculated. The shear demand for pier walls in the strong direction is dependent upon the boundary conditions of the pier walls.

### 2.4.2 Displacement Capacity of Ductile concrete members

All columns designed and detailed to behave in a ductile manner are designated as a seismic –critical members. A ductile member is defined as any member that is intentionally designed to deform inelastically for several cycles without significant degradation of strength or stiffness under the demands generated by the design seismic hazards. Seismic-critical members may sustain damage during seismic event without leading to structural collapse or loss of structural integrity.

Local displacement capacity $\Delta_c$ is defined as a member's displacement capacity attributed to its elastic and plastic flexibility. The structural system's displacement capacity $\Delta_c$ is the reliable lateral capacity of the bridge or subsystem as it approaches its collapse limit state. Ductile members must meet the local displacement capacity requirements and the global displacement criteria [21]

The local displacement capacity of a member is based on its rotation capacity, which is based on the curvature capacity. Curvature capacity shall be determined from the moment curvature analysis.

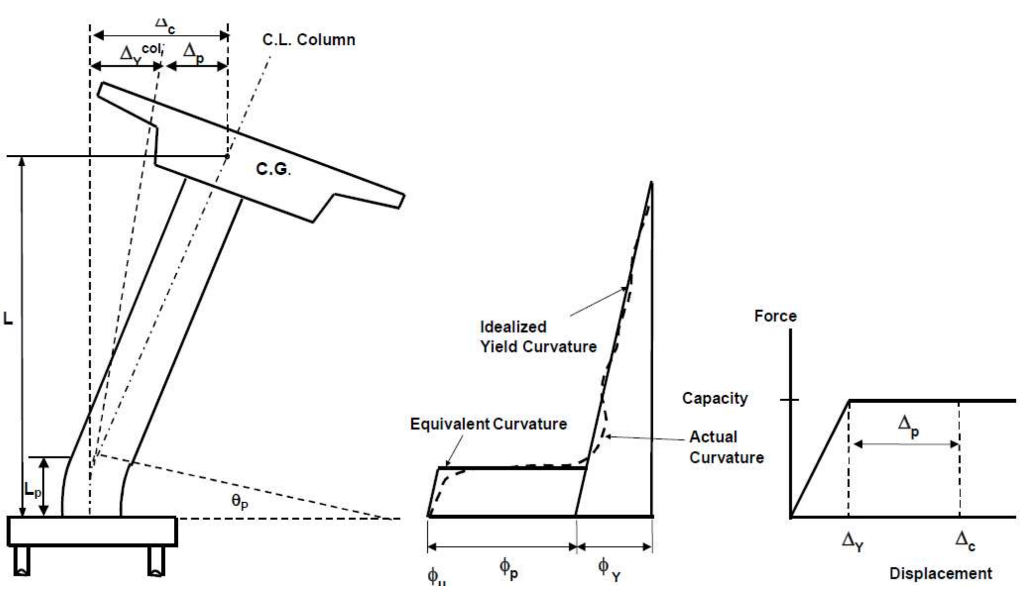



*Figure 2-4: Local displacement capacity cantilever column with fixed base*

Each ductile member shall have a minimum local displacement ductility capacity of 3 to insure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imported to that member.

### 2.4.3 Mander's Confined and Unconfined concrete model

In a seismic design of reinforced concrete columns of buildings and bridge structures, the potential plastic hinge regions need to be carefully detailed for ductility in order to insure that the shaking from large earthquakes will not cause collapse. Adequate ductility of members of reinforced concrete frames is also necessary to ensure that moment redistribution can occur. The most important design consideration for ductility in plastic hinge regions of reinforced concrete columns is the provision of sufficient transverse reinforcement in the form of spirals or circular hoops or of rectangular arrangements of steel, in order to confine the compressed concrete, to prevent buckling of the longitudinal bars and to prevent shear failure.

Mander [29] have proposed a unified stress strain approach for confined concrete applicable to both circular and rectangular shaped transverse reinforcement.

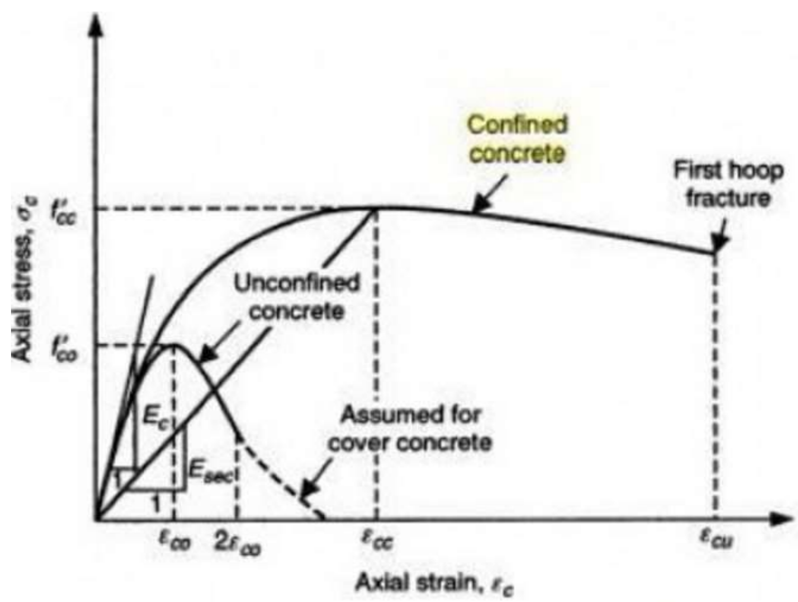



*Figure 2-5: Mander et al stress strain model*

A stress strain model for confined and unconfined concrete shall be used in the analysis to determine the local capacity of ductile concrete members. The initial ascending curve may be represented by the same equation for both the confined and unconfined model since the confining steel has no effect in this range of strains. As the curve approaches the compressive strength of the unconfined concrete, the unconfined stress begins to fall to an unconfined strain level before rapidly degrading to zero at the spalling strain $\epsilon_{sp}$ typically equals to 0.005. The confined concrete model should continue to ascend until the confined compressive strength $f_{cc}'$ is reached. This segment should be followed by the descending curve dependent on the parameters of the confining steel. The ultimate strain $\epsilon_{cu}$ should be the point where strain energy equilibrium is reached between the concrete and the confinement steel.

### 2.4.4 Moment Curvature analysis

The plastic moment capacity of all ductile members shall be calculated by the moment curvature analysis based on expected material properties. Moment curvature analysis derives the curvature associated with the range of moments for a cross sectional based on the principles of strain compatibility and equilibrium of forces. The M-ϕ curves can be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section. The elastic portion of the idealized curve should pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity is obtained by balancing the areas between the actual and the idealized M-ϕ curves beyond the reinforcing bar yield point [21]

The Transverse steel placed near the outside face of the plastic hinge region activates in tension when the section performs in a ductile manner in the inelastic range, and the resulting steel stress then applies confinement pressure to the confinement core.

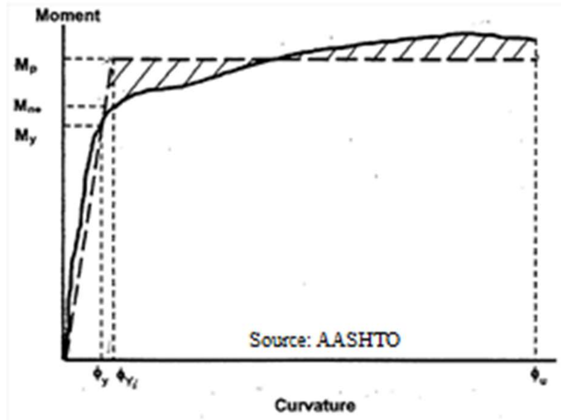



*Figure 2-6: Moment Curvature Diagram*

### 2.4.5 Seismic Performance Assessment- Pushover analysis

Design will always be a compromise between simplicity and reality. With the recognition that reality is a very complex and uncertain in imposed demands and available capacities. And simplicity is a necessity driven by limited economy and limited ability to implement complexity with commonly available knowledge and tools. Static Pushover analysis is the system of evaluating the capacity of a structure when exposed to a savior seismic load. This method of analysis is a part of evaluation process and provides estimates of demands imposed on structures and elements. Evaluation implies that imposed demands have to be compared to available capacities in order to assess the capacity of the design.

POA is based on the assumption that the response of the structure can be related to the response of an equivalent single degree of freedom system. This implies that the response is controlled by a single mode and that the shape of this mode remains constant throughout the time history response. Various studies [33], [18],[17] have indicated that these assumptions lead to a good prediction of the maximum seismic response of multi degree of freedom structure provided their response is dominated by a single mode.

Pushover analysis is an Inelastic Static Analysis generally is used to determine the reliable displacement capabilities of a structure as it reaches its limit of structural stability. This method of analysis shall be performed using expected material properties or modeled members. It is an incremental linear analysis which captures the overall nonlinear behavior of the elements, including soil effects, by pushing them laterally to initiate plastic action. Each increment pushes the frame laterally, through all possible stages, until the potential collapse mechanism is achieved. Because the analytical model accounts for the redistribution of internal actions as components respond inelastically. Inelastic Static Analysis (Pushover analysis) is expected to provide a more realistic measure of behavior.

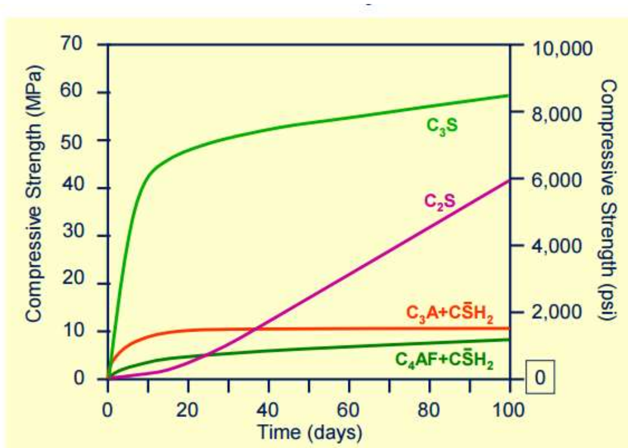
In the pushover analysis it Is assumed that the target displacement for the MDOF structure can be estimated as the displacement demand for the corresponding equivalent SDOF system transformed to the MDOF domain through the use of a shape vector.

## 2.5 Early stage Crack

### 2.5.1 General

Early-age cracking can be a significant problem in concrete. Volume changes in concrete will drive tensile stress development when they are restrained. Cracks can develop when the tensile stress exceeds the tensile strength, which is generally only 10 percent of the compressive strength. At early ages, this strength is still developing while stresses are generated by volume changes. Controlling the variables that affect volume change can minimize high stresses and cracking [35], [36], [37]

According to ACI 207 code, any volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat from hydration of the cement and attendant volume change to minimize cracking.




Compressive Strength development in pastes of pure cement compounds (*Mindess et al, 2003*)

*Figure 2-7: Strength development*

### 2.5.2 Heat of Hydration

Cement hydration is an exothermic reaction in which heat will be released out to the environment. The amount of heat release during the first 3 days is relatively high. From among many factors which affect the amount of heat release are weather (humidity and temperature), additives, cement type, cross sectional area, aggregate type can be stated. Studies has recommend to consider early age cracking and heat of hydration during the design on concrete members which are 1m or 3ft thick [36]

The hydration of cement can be thought of as a two-step process. In the first step, called dissolution, the cement dissolves, releasing ions into the mix water. The concentrations of ionic species in the pore solution increase rapidly as soon as the cement and water are combined. Eventually the concentrations increase to the point that the pore solution is supersaturated, meaning that it is energetically favorable for some of the ions to combine into new solid phases rather than remain dissolved. This second step of the hydration process is called precipitation. A key point, of course, is that these new precipitated solid phases, called hydration products, are different from the starting cement minerals. Precipitation relieves the supersaturation of the pore solution and allows dissolution of the cement minerals to continue. Thus cement hydration is a continuous process by which the cement minerals are replaced by new hydration products. Cement minerals are formed at temperatures exceeding 1400 $^{o}$C, At the much lower temperatures present during cement hydration, the cement minerals are actually quite unstable, meaning that there are many other solid phases that will form preferentially in their place once they dissolve. In fact, the whole point behind the high-temperature cement manufacturing process is to create solid phases that will readily dissolve in water, allowing new phases to form. When one phase is converted into another phase with a lower free energy, there is usually a release of excess energy in the form of heat. Such a reaction is termed exothermic, and the exothermic heat associated with cement hydration has already been defined as the Heat of hydration.

Generated heat shall be conducted into the surrounding environment. As the thickness of the section increases, the heat convection between the environment and the core center will be mild and heat difference between will be higher enough to create thermal cracking.  Heat of hydration evolution is presented below in the graph [36]

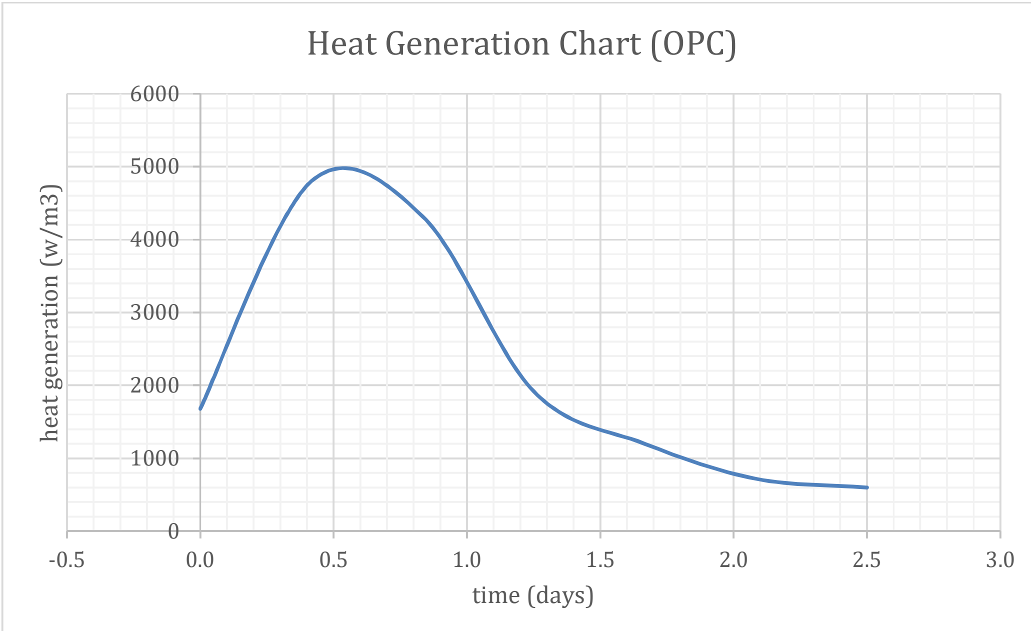



*Figure 2-8: Heat generation vs time*

### 2.5.3 How to reduce heat of hydration

The heat accumulation in the core center of the concrete can be reduced by forming more surface area which is in contact with the surrounding air. Less thick sections has less viability to early age cracks. But also the following methods can be used:

- Using larger size aggregates (as in Roller Compacted Concrete – RRC) for extremely thick concrete laying. Eg. Dams
- Mixing and casting concrete mixed with ice ( construction in high temperature areas)
- Utilization of chemical additives.

In this thesis ANSYS heat conduction simulator is used to elaborate the hydration heat generation and conduction property of designed sections.

# 3. Modeling and Analysis

## 3.1 Introduction

A single rectangular column pier with variable geometric configurations is designed against common design loads factored according to LRFD method.  The comparison will be made according to the ductility capacity and the minimum quantity of materials needed. Based on the proposed work flow, conventionally designed section is examined for its seismic performance capacity.

**LRFD Loading Criteria**

**Strength I** : 1.25DC + 1.5DW + 1.75LL + 1.50*(TU + CR + SH)

**Strength II** : Consideration of the owner specified special design

**Strength III** : When exposed to wind velocity exceeding 55Km/hr

**Strength IV** : When very high dead load to live load force effect ratio is exhibited

**Strength V** : Normal vehicular loading plus wind loading of 55Km/hr velocity is encountered

**Extreme events**: load combination including earth quake

**Extreme events II**: Load combination related to Ice load collisions and certain hydraulic loads

**Service I:** Load consideration when normal operational use is under consideration.

**Service II:** load combination intended to control yielding of the steel structure and slip of slip critical connections due to vehicular loads.

**Service III:** Longitudinal analysis related to the tension in the prestressed concrete superstructure with Crack control objectives.

**Service IV:** Load combination related to tension in prestressed concrete columns with crack control objective

**Fatigue I:** fatigue and fracture load combinations related to infinite load induced fatigue life.

**Permanent Loads**

**Dead load (DC, DW and EV):** from the superstructure will be used nominally as given on a tabular form from table 3.1

Dead load from the self-weight of the pear, from the geometrical considerations

**Earth Loads (EH, ES and DD):** can be calculated based on the height of the buried section of the pier

Stress transfer from the Bearings of the pear has also significant eccentric loading effect on the compression member of the structure. Bearing used may be plate or reinforced rubber but the necessity of using the bearing is to transfer the structural deformation from the structure directly to the pier. Deformations can be rotational, axial or shearing.

For this specific paper, Shearing deformations on the Bearing connections are modeled to be a lateral load effects in both of the sides. This lateral loads may be modeled from the lateral load effects on the super structure and the vehicle. The operation/service deformations along the longitudinal direction on the camber do have a potential of pushing the bearing in the other direction to cause a lateral loading conditions.

**Live loads, Gravity (LL and PL):** For live loading consideration on the Pier design, AASHTO specified vehicular loads shall be used for superstructure. But for this specific paper, the reactions of the superstructure on Bearing and Bearing stresses are modeled in different levels of actions.

In addition to the direct load transfer from the super structure, deformation stress from the bearings will also be considered.

**Dynamic load allowance**: for this specific load effect AASHTO standards table 3.6.2.1-1 shall be used. As per the specifications, 75% load magnification has to be utilized for the deck joints where the piers are mostly located.

**Curved members: Centrifugal forces (CE**) shall be considered. Modeling this load shall be according to AASHTOs standards 3.6.2.3 the amount of load is directly applied on the Pier Cap.

**Braking Force (BR):** shall be modeled and applied on the design pier as per AASHTO Section 3, C3.6.4

**Water load (WL)**: for the Bridge pier design, water load shall be considered. Piers may be exposed to the running water pressure. Where the pressure could be significant depending on the slop of the river bed, shape and the drag coefficient of the pier

**Wind Load conditions (WL and WS):** this loading condition has a direct influence the design of both the super structure and the sub structure parts of the pier. The cumulative effect obtained from the superstructure will be modeled according to the connection type and the resultant directions. But the direct effect at the surface of the pier shall be modeled according to the general guidelines given on the AASHTO specifications and on the Ethiopian codes and standards EBCS part 1, Appendix part.

**Vehicular Collision Loads (CT):** For the worst case, all the Overpass bridge piers shall be designed considering the vehicular collision loads. But for the river bridges, CT shall not be considered.

Earthquake Loads (EQ): shall be modeled according to the EBCS 8 1995, Guidelines given for the specific Geographic conditions of Ethiopia.

## 3.2 Geometry Consideration

Optimizing geometry regarding the material components and the load expectations, due consideration to single column pier with a firm deep foundation with a variable height above the ground level is given. In this study only the rectangular shaped column types will be covered. The column will have a pedestal head at the top with both side-flanged shelves. The shelve will directly support the Bearing system, so that no moment will be transferred from the superstructure, except the unbalanced load condition of to be subjected on the load bearings. The sizes of each element will be governed by the Span and height of the pier. For a fair consideration, the following pairing of span to height ratios are taken. (table 3-1)

Table 3-1: Span to height Pairings

| Case number | Span( both left and right (m) | Hollow Cross section Column Height (m) | Solid Cross section Column Height (m) |
|---|---|---|---|
| Case 04 | 40 | 25 | 25 |
| Case 03 | 40 | 20 | 20 |
| Case 02 | 40 | 15 | 15 |
| Case 01 | 40 | 10 | 10 |

Using the applicable loading conditions and conventional design approach, optimization of the section concrete and reinforcement is done. During this analysis stage, SAP2000 V 18 Eval is utilized along with various excel templates.

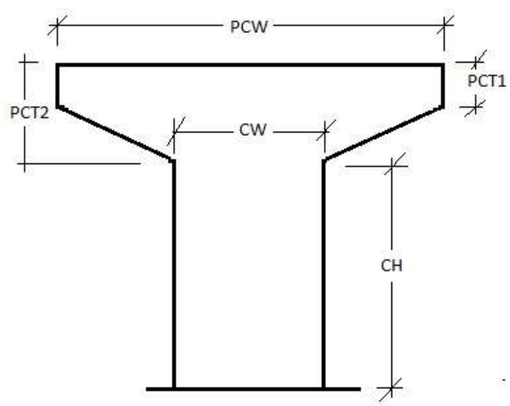



Figure 3-1: General Pier Geometry XZ axis

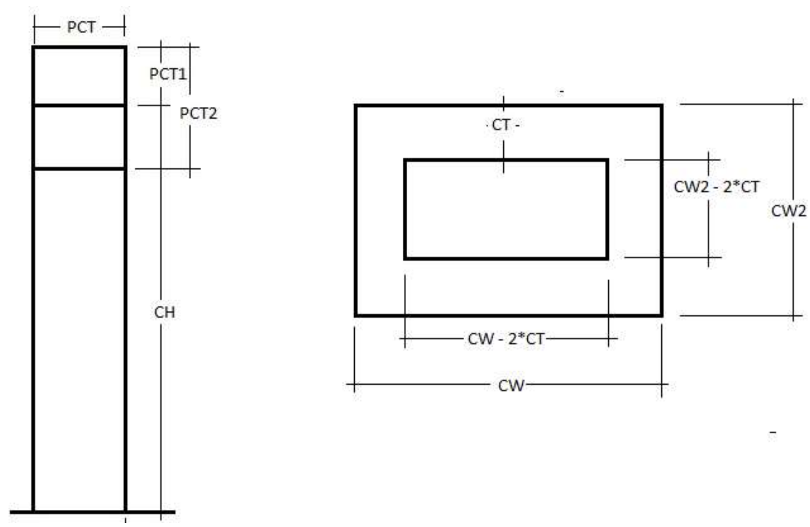



Figure 3-2: Cross section dimensioning - hollow pier

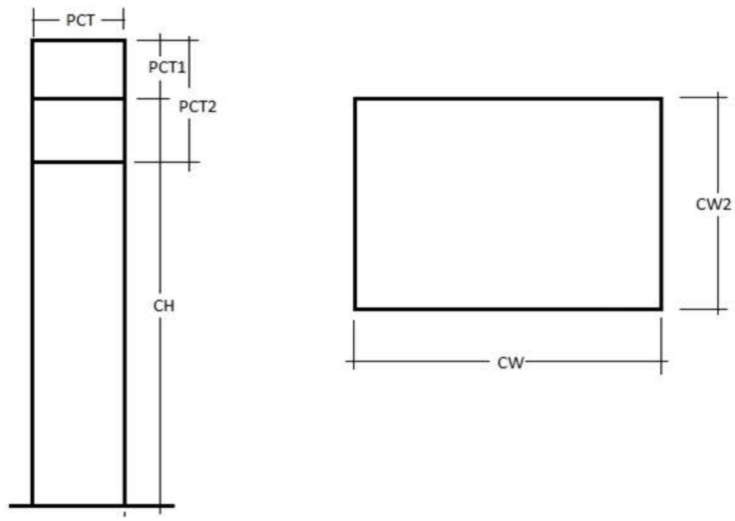



*Figure 3-3: Cross Section Dimensioning - Solid Core*

## 3.3 Material Consideration

From practical point of view, C30 Concrete grade, class I is considered. To prevent segregation and uneven distribution of ingredients within the concrete composition, Super plasticizers may be applied. From the composition, the hydration heat evolution and nominal compressive stress calculations will be carried out based on the ACI and EBCS standards and specifications.

**Concrete:**

Class C30, where $f_{ctk}$ = 1.7MPa, $f_{ctm}$= 2.5MPa, $f_{ck}$ = 24MPa, $E_{cm}$ =32GPa

Where:

$f_{ctm}$ is the mean value of axial tensile strength

$f_{ctk}$ Characteristic tensile strength,

$f_{ck}$ Characteristic compressive strength of concrete.

$E_{cm}$ Secant modulus of Elasticity

Poisson's ratio = 0.2

Shrinkage Strain Given on EBCS 2, 1995 is based on the Relative Humidity, age of loading and size ratios ($2A_c/n$) where $A_c$ is the Area of the section and n is the perimeter

Coefficient of thermal expansion = 10 x $10^{-6}$ per degree centigrade

**Reinforced Steel:**

Characteristic yield strength of Reinforcement steel can be obtained from the manufacturers' manual which could be given as $f_y$ or $f_{0.2}$

For this specific study, Class A high ductile, ribbed steel with 420MPa yield strength Coefficient of thermal expansion = 10 x $10^{-6}$ $^0$C and wieldable reinforcement bar is utilized.

## 3.4 Modeling Procedure

### 3.4.1 Load Calculations

Pier carries an axial load which is subjected on the super structure. Considering an efficient bearing separator between the superstructure and the pier, unbalanced moment from the superstructure is not taken in to consideration in this specific study. But all the gravity loads and the environment emitted loads are considered to their maximum magnitude.

For all of the design cases, the magnitude of un-factored dead load from the superstructure girders is given in the Table 3.7. In addition to the superstructure dead load, the weights of pier cap, the pier column itself is quantified and applied in the analysis.

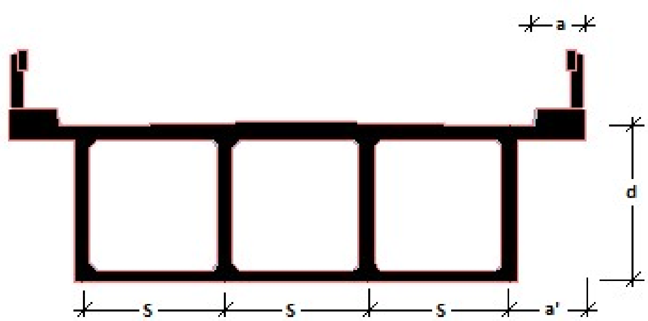



*Figure 3-4: Typical superstructure cross section*

Dead load is calculated considering the volume of all permanent fixtures of the bridge.

**Live Load Calculation**

To calculate the live load, two trucks which are separated in 15 meters are considered. According to ERA 2002. Having one of the trucks stepping at the support, influence lines are developed and maximum shear force (for the superstructure) is obtained. Having a typical superstructure dimensions as provided on table 3.5 below, distribution coefficients for interior and exterior graders are calculated according to AASHTO bridge design manual. According to AASHTO LRFD bridge design manual section 4.6, live load distribution on a cast in - place concrete multi-cell concrete girder can be carried out using the following formulas.

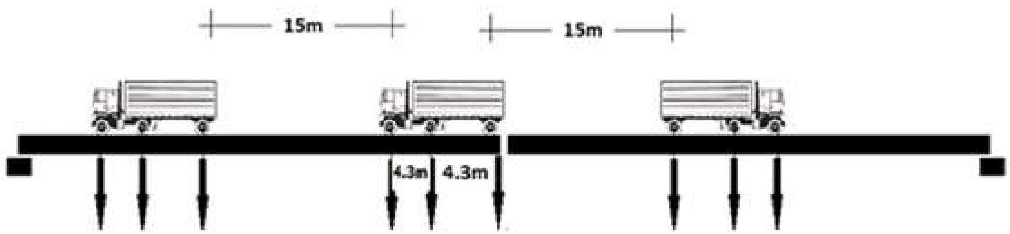



*Figure 3-5: Live load configuration*

Depending on the vehicle configuration shown on Figure 3-5, the total reaction on the central pier is calculated to be 1171.6KN on the interior and 1415.9KN on the exterior girder.

*Table 3-2: AASHTO LRFD Bridge Design manual, live load distribution*

| Type of Superstructure | Applicable Cross-Section from Table 4.6.2.2.1-1 | One Design Lane Loaded | Two or More Design Lanes Loaded | Range of Applicability |
|---|---|---|---|---|
| **Table 4.6.2.2.3b-1—Distribution of Live Load for Shear in Exterior Beams** | | | | |
| Cast-in-Place Concrete Multicell Box | d | Lever Rule | $g = e\, g_{interior}$ <br> $e = 0.64 + \frac{d_e}{12.5}$ | $-2.0 \le d_e \le 5.0$ |
| | | or the provisions for a whole-width design specified in Article 4.6.2.2.1 | | |
| **Table 4.6.2.2.3a-1—Distribution of Live Load for Shear in Interior Beams** | | | | |
| Cast-in-Place Concrete Multicell Box | d | $\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{d}{12.0L}\right)^{0.1}$ | $\left(\frac{S}{7.3}\right)^{0.9}\left(\frac{d}{12.0L}\right)^{0.1}$ | $6.0 \le S \le 13.0$ <br> $20 \le L \le 240$ <br> $35 \le d \le 110$ <br> $N_c \ge 3$ |

Live load distribution coefficient calculation for interior beams

*Table 3-3: Live load distribution coefficient interior beams*

| | Cast in place multi-cell box girder deck: | | | | Single lane loaded | Multiple members loaded |
|---|---|---|---|---|---|---|
| S | 2.25 | m = | 7.401316 | ft | 0.637873 | 0.750188 |
| L | 40 | m = | 131.5789 | ft | | |
| d | 2 | m = | 78.74016 | in | | |
| Nc | 3 | Nc | 3 | | | |
| a | 0.8 | m = | 2.631579 | ft | | |
| a' | 1.15 | m = | 3.782895 | ft | | |
| ccl | 1.8 | m = | 5.921053 | ft | | |

**Lever rule (shear distribution coefficient for Exterior beams)**

Assuming the truck can close the curbstone at the age of assumed pedestrian lane, which is 0.8m from the age of the bridge, and calculating moment about the interior girder considering a multiple presence factor m = 1.2 for a single lane loaded.

$$R_E = \frac{1}{a} mP * (a + d_1 + d_2) = 1.81P$$

In correlation with the diagram below, the above equation gives a shear force distribution coefficient for single lane loaded condition.

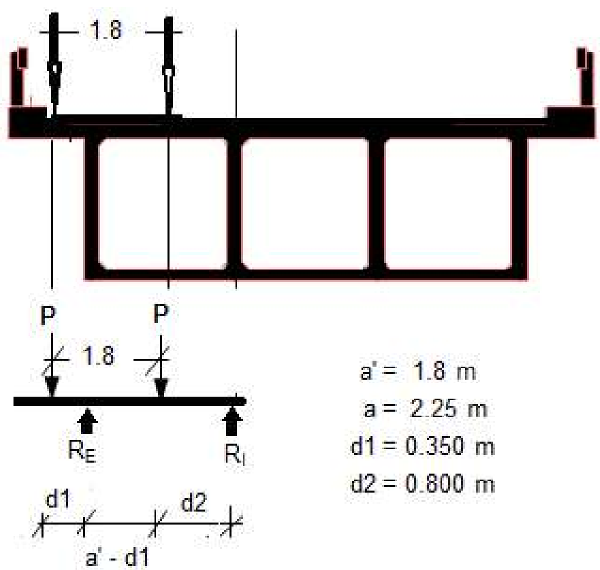



*Figure 3-6: Lever rule*

For an exterior girder (multiple lane loaded) condition the provisional equation given on AASHTO standard is directly used. Summarized result is tabulated

*Table 3-4: Exterior girder shear force distribution coefficient*

| | Cast in place munti cell box beams | | | | | Single lane loaded | Multiple members loaded |
|---|---|---|---|---|---|---|---|
| S | 2.25 | m | = | 7.401316 | ft | 1.813 | 1.365 |
| L | 40 | m | = | 131.5789 | ft | | |
| d | 2 | m | = | 78.74016 | in | | |
| Nc | 3 | web width = 0.25 m | | | | | |

*Table 3-5: Superstructure Loads*

| Superstructure load | Internal Girder | External Girder |
|---|---|---|
| Dead Load | 3258KN | 3732KN |
| Live Load | 1358KN | 1602KN |

According to the specifications, the breaking force shall be taken as:

$$Breaking\ Force \geq \begin{cases} 25\%\ of\ axle\ weight\ of\ the\ design\ truck \\ 25\%\ of\ axle\ weight\ of\ the\ design\ tandum \\ 5\%\ of\ the\ design\ truck\ +lane\ load \\ 5\%\ of\ the\ design\ tandum\ +Lane\ load \end{cases}$$ ---------------------3.1

The breaking force is placed in all design lanes which are considered to be loaded In accordance with AASHTO section 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces are assumed to act horizontally at a distance of 6ft above the road way surface in either longitudinal direction to cause extreme force effects.

Accordingly, for an orthogonally oriented bridge and using 25% of axle weight of the design truck:

Br = 162.50KN, towards the longitudinal direction

**Wind load on superstructure and substructure:**

According to AASHTO Section 3.8.1.1 Wind pressure $P_D$ can be expressed as:

$$P_D = P_B * \left(\frac{V_{DZ}}{V_B}\right)^2$$ --------------------------------------------3.2

Where, $$V_{DZ} = \begin{cases} 2.5 * V_o * \left(\frac{V_{10}}{V_B}\right) \ln\left(\frac{Z}{Z_o}\right) ------ for\ Z > 10m \\ V_B -------------- for\ Z \leq 10m \end{cases}$$ -------------------------3.3

Z is the height of the superstructure above the water level

$Z_o$ depends on the city or non-city characteristics, we will take 1000m assuming

$V_{10}$ = 105km/h, According to ERA 2002, considering the max. Wind speed that could occur in Ethiopia.

$V_B$ = 160km/h

$P_B$ is a base wind pressure, a table below (ERA 2002 table 2-13) gives various wind pressure for various attack angle

**$V_{DZ}$ Calculation**: (for max calculation Urban and sub-urban condition is assumed in this study: $V_o$ = 17.6km/h and $Z_o$ = 1000mm)

*Table 3-6 : Vdz calculation for each design case*

| Pier Study Case | Z (m) | Vdz (km/h) |
|---|---|---|
| Case 01 | 10 | 160 |
| Case 02 | 15 | 79.44 |
| Case 03 | 20 | 87.87 |
| Case 04 | 25 | 94.42 |

**Wind pressure (WL) calculations**

**Wind pressure calculations on Structural Members**

*Table 3-7: ERA Bridge design manual 2002 Base wind pressure recommendation*

| (kPa) | Columns and Arches | | Girders | |
|---|---|---|---|---|
| Skew Angle of Wind, Degrees | Lateral Load | Longitudinal Load | Lateral Load | Longitudinal Load |
| 0 | 3.6 | 0 | 2.4 | 0 |
| 15 | 3.4 | 0.6 | 2.1 | 0.3 |
| 30 | 3.1 | 1.3 | 2.0 | 0.6 |
| 45 | 2.3 | 2.0 | 1.6 | 0.8 |
| 60 | 1.1 | 2.4 | 0.8 | 0.9 |

Using the base wind pressure given above, design wind pressure ($P_D$) is calculated for corresponding skew angle.

*Table 3-8:PD calculation for Girders (PD in kPa and angles in degrees)*

| | Design wind pressure at the superstructure axis 10m | | Design wind pressure at the superstructure axis 15m | | Design wind pressure at the superstructure axis 20m | | Design wind pressure at the superstructure axis 25m | |
|---|---|---|---|---|---|---|---|---|
| Skew Angle | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal |
| 0 | 2.4 | 0 | 0.592 | 0.000 | 0.724 | 0.000 | 0.836 | 0.000 |
| 15 | 2.1 | 0.3 | 0.518 | 0.074 | 0.633 | 0.090 | 0.731 | 0.104 |
| 30 | 2 | 0.6 | 0.493 | 0.148 | 0.603 | 0.181 | 0.697 | 0.209 |
| 45 | 1.6 | 0.8 | 0.394 | 0.197 | 0.483 | 0.241 | 0.557 | 0.279 |
| 60 | 0.8 | 0.9 | 0.197 | 0.222 | 0.241 | 0.271 | 0.279 | 0.313 |

*Table 3-9: PD calculation for Column members (PD in kPa and angles in degrees)*

| | Design wind pressure at the at the center of 10m pier | | Design wind pressure at the at the center of 15m pier | | Design wind pressure at the at the center of 20m pier | | Design wind pressure at the at the center of 25m pier | |
|---|---|---|---|---|---|---|---|---|
| Skew Angle | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal |
| 0 | 3.600 | 0.000 | 0.887 | 0.000 | 1.086 | 0.000 | 1.254 | 0.000 |
| 15 | 3.400 | 0.600 | 0.838 | 0.148 | 1.026 | 0.181 | 1.184 | 0.209 |
| 30 | 3.100 | 1.300 | 0.764 | 0.320 | 0.935 | 0.392 | 1.080 | 0.453 |
| 45 | 2.300 | 2.000 | 0.567 | 0.493 | 0.694 | 0.603 | 0.801 | 0.697 |
| 60 | 1.100 | 2.400 | 0.271 | 0.592 | 0.332 | 0.724 | 0.383 | 0.836 |

**Wind Pressure calculations on Vehicles**

*Table 3-10: Basic wind pressure on vehicle*

| Skew Angle (Degrees) | Normal Component (kN/m) | Parallel Component (kN/m) |
|---|---|---|
| 0 | 1.46 | 0 |
| 15 | 1.28 | 0.18 |
| 30 | 1.20 | 0.35 |
| 45 | 0.96 | 0.47 |
| 60 | 0.50 | 0.55 |

Wind pressure on vehicles is transferred to the top of all piers, Design pressure calculated is presented below

*Table 3-11: PD calculation for vehicles (PD in kN/m and angles in degrees)*

| | Design wind pressure at the superstructure axis 10m | | Design wind pressure at the superstructure axis 15m | | Design wind pressure at the superstructure axis 20m | | Design wind pressure at the superstructure axis 25m | |
|---|---|---|---|---|---|---|---|---|
| Skew Angle | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal |
| 0.000 | 1.460 | 0.000 | 0.360 | 0.000 | 0.440 | 0.000 | 0.508 | 0.000 |
| 15.000 | 1.280 | 0.180 | 0.316 | 0.044 | 0.386 | 0.054 | 0.446 | 0.063 |
| 30.000 | 1.200 | 0.350 | 0.296 | 0.086 | 0.362 | 0.106 | 0.418 | 0.122 |
| 45.000 | 0.960 | 0.470 | 0.237 | 0.116 | 0.290 | 0.142 | 0.334 | 0.164 |
| 60.000 | 0.500 | 0.550 | 0.123 | 0.136 | 0.151 | 0.166 | 0.174 | 0.192 |

**Wind Load calculations**

Wind load acting on superstructure is calculated as follows:

$$Wind\ load_{\ transverse} = P_D b \frac{L_{back} + L_{\ ahe}}{2} \quad \text{--------------------3.4}$$

$$Wind\ Load_{\ longtudinal} = P_D b (L_{back} + L_{\ ahead}) \quad \text{-------------3.5}$$

Wind load on acting on substructure (piers):

$$Wind\ load_{\ transverse} = P_D H C_{w2} \quad \text{----------------------------3.6}$$

$$Wind\ Load_{\ longtudinal} = P_D H C_w \quad \text{-----------------------------3.7}$$

Wind load on acting on Vehicles:

$$Wind\ load_{\ transverse} = P_D \quad (L_{back} + L_{\ ahea} \ ) \quad \text{---------------3.8}$$

$$Wind\ load_{\ longtudinal} = P_D \quad (L_{back} + L_{\ ahead}) \quad \text{------------- --3.9}$$

Couple moment at the top of the pier (using 1.8m + girder depth offset)

$$Moment\ at\ \ the\ tip\ pier = \ Wind\ load_{\ transverse}(1.8 + b) \text{--------3.10}$$

$$Moment\ at\ \ the\ tip\ pier = \ Wind\ load_{\ longtudinal}(1.8 + b) \text{ ---------3.11}$$

Where:

H=the exposed superstructure height, b is the depth of superstructure

$L_{back}$=left span length (40m) and $L_{ahe}$ =right span length (40m)

$C_{w2}$ = week axis size (transverse size of a pie

$C_w$ = Longitudinal (Strong axis size of a column)

Using the above expressions, wind load at the head of a pier (using equations 3.4 and 3.5) at the center of the pier ( using equations 3.6 and 3.7) at the top pf the pier (using equations 3.8, 3.9. 3.10 and 3.11).

*Table 3-12: Resultant Wind Force when pressure acts on girders*

| | Resultand wind Force(KN) at the superstructure axis 10m | | Resultand wind Force(KN) at the superstructure axis 15m | | Resultand wind Force(KN) at the superstructure axis 20m | | Resultand wind Force(KN) at the superstructure axis 25m | |
|---|---|---|---|---|---|---|---|---|
| Skew Angle | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal |
| 0 | 192 | 0 | 47.326 | 0.000 | 57.915 | 0.000 | 66.864 | 0.000 |
| 15 | 168 | 48 | 41.410 | 11.831 | 50.676 | 14.479 | 58.506 | 16.716 |
| 30 | 160 | 96 | 39.438 | 23.663 | 48.262 | 28.957 | 55.720 | 33.432 |
| 45 | 128 | 128 | 31.551 | 31.551 | 38.610 | 38.610 | 44.576 | 44.576 |
| 60 | 64 | 144 | 15.775 | 35.494 | 19.305 | 43.436 | 22.288 | 50.148 |

*Table 3-13: Resultant Wind Force when pressure acts on Piers*

| | Resultand wind Force(KN) at center of of pier surface 10m | | Resultand wind Force(KN) at center of of pier surface 15m | | Resultand wind Force(KN) at center of of pier surface 20m | | Resultand wind Force(KN) at center of of pier surface 25m | |
|---|---|---|---|---|---|---|---|---|
| Skew Angle | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal |
| 0 | 90.000 | 0.000 | 33.276 | 0.000 | 54.295 | 0.000 | 78.356 | 0.000 |
| 15 | 85.000 | 21.000 | 31.427 | 7.764 | 51.279 | 12.669 | 74.003 | 18.283 |
| 30 | 77.500 | 45.500 | 28.654 | 16.823 | 46.754 | 27.449 | 67.473 | 39.613 |
| 45 | 57.500 | 70.000 | 21.260 | 25.881 | 34.689 | 42.230 | 50.061 | 60.944 |
| 60 | 27.500 | 84.000 | 10.168 | 31.058 | 16.590 | 50.676 | 23.942 | 73.133 |

*Table 3-14: Resultant Wind Force when pressure acts on Vehicles*

| | Resultand wind Force(KN) at the superstructure axis 10m | | Resultand wind Force(KN) at the superstructure axis 15m | | Resultand wind Force(KN) at the superstructure axis 20m | | Resultand wind Force(KN) at the superstructure axis 25m | |
|---|---|---|---|---|---|---|---|---|
| Skew Angle | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal |
| 0.000 | 58.400 | 0.000 | 14.395 | 0.000 | 17.616 | 0.000 | 20.338 | 0.000 |
| 15.000 | 51.200 | 7.200 | 12.620 | 1.775 | 15.444 | 2.172 | 17.830 | 2.507 |
| 30.000 | 48.000 | 14.000 | 11.831 | 3.451 | 14.479 | 4.223 | 16.716 | 4.876 |
| 45.000 | 38.400 | 18.800 | 9.465 | 4.634 | 11.583 | 5.671 | 13.373 | 6.547 |
| 60.000 | 20.000 | 22.000 | 4.930 | 5.423 | 6.033 | 6.636 | 6.965 | 7.662 |

*Table 3-15: Resultant Moment when Wind pressure acts on Vehicles 1.8m above surface of superstructure*

| | Resultand wind Moment (KN-m) at the superstructure axis 10m | | Resultand wind Moment (KN-m) at the superstructure axis 15m | | Resultand wind Moment (KN-m) at the superstructure axis 20m | | Resultand wind Moment (KN-m) at the superstructure axis 25m | |
|---|---|---|---|---|---|---|---|---|
| Skew Angle | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal |
| 0.000 | 221.920 | 0.000 | 54.701 | 0.000 | 66.940 | 0.000 | 77.284 | 0.000 |
| 15.000 | 194.560 | 27.360 | 47.957 | 6.744 | 58.687 | 8.253 | 67.756 | 9.528 |
| 30.000 | 182.400 | 53.200 | 44.959 | 13.113 | 55.019 | 16.047 | 63.521 | 18.527 |
| 45.000 | 145.920 | 71.440 | 35.968 | 17.609 | 44.015 | 21.549 | 50.817 | 24.879 |
| 60.000 | 76.000 | 83.600 | 18.733 | 20.606 | 22.925 | 25.217 | 26.467 | 29.114 |

*Table 3-16: Summery of Maximum unfactored wind loads*

| | 10m tall pier | | 15 m tall pier | | 20m tall pier | | 25m tall pier | |
|---|---|---|---|---|---|---|---|---|
| | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal | transverse | Longtudinal |
| Load from pressure acting on girders (KN) | 192.000 | 144.000 | 47.326 | 35.494 | 57.915 | 43.436 | 66.864 | 50.148 |
| Load from Pressure acting on Piers (KN) | 90.000 | 84.000 | 33.276 | 31.058 | 54.295 | 50.676 | 78.356 | 73.133 |
| Load From Pressure acting on Vehicles (KN) | 58.400 | 22.000 | 14.395 | 5.423 | 17.616 | 6.636 | 20.338 | 7.662 |
| Moment from pressure acting on Vehicle (KN - m) | 221.920 | 83.600 | 54.701 | 20.606 | 66.940 | 25.217 | 77.284 | 29.114 |

**Vertical wind load**

The vertical wind load is calculated by multiplying a 0.00096Mpa vertical wind pressure by out-to-out bridge deck width. It is applied to the windward quarter-point of the deck only limit states that do not include wind on live load. Also the wind attack angle must be zero degrees for the vertical wind load to apply

$$W_{vert} = 0.00096\, W_{deck}$$

12.48KN/m

**Earthquake Load**

All of the structure are assumed to be constructed in seismic zone 4. Load analysis and design is carried out as per the detailed procedures explained on ERA 2002 Bridge design manual.

**Acceleration Coefficient, Site effects and Importance Categories**

For safe consideration, this study adopts soil and seismic characteristics of severe conditions. Accordingly, depending on EBCS Zone 4, an acceleration coefficient of 0.1 is taken for further calculation.

Based on ERA Bridge design manual 2002 recommendations for locations with unknown soil conditions, Soil profile II (S = 1.2) is used in this study. This study assumes bridges under evaluation are of Essential bridge category.

ERA Bridge design manual recommends that seismic analysis is not required for single – span bridges. But following AASHTO LRFD bridge design manual section 3, piers evaluated in this study will be designed for EQ load along their weak axis.

**EQ requirement of Superstructure to substructure connection**

Minimum displacement width is obtained from a percentage of the empirical seat width. Minimum support length required:

$$N_1 = (200 + 0.0067H)(1 + 0.000125S^2)$$

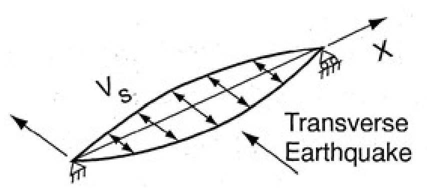
$$N_1 = 336.87\text{mm}$$

**EQ load determination**

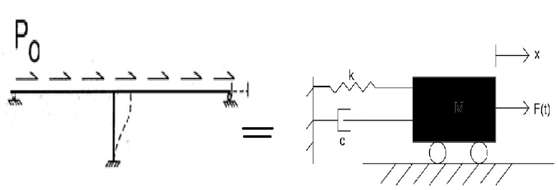
According to ERA Bridge design manual:

- Response modification factor (R) for Essential bridge's single column = 2.0
- Seismic analysis method for Multispan regular bridges are Single – mode elastic method and Uniform load elastic method.

Single –mode elastic method (SM): This method can be used either in longitudinal or transverse direction, and the natural period is calculated by equating maximum potential and kinetic energies associated with the fundamental mode shape.




Uniform load elastic method (UL): can be used on either longitudinal or transverse directions and its assumed mode of vibration is as explained on the figure below




In this study, UL analysis method is used to determine the share of earthquake load absorbed by the pier in its weak axis.

Steps followed:

- Lateral stiffness of the bridge, K, is determined using $K = \frac{P_o L}{V_{s,max}}$

  Where, $P_o$ = 1N/m a uniform arbitrary load

  $V_{s,max}$ is the maximum deformation corresponding to $P_o$

  L is total length of the bridge (80 m) considering 40 meters span both sides of the pier

- Nominal unfactored weight of the bridge including sub structural weights and extreme case live loads (W)
- Period of the bridge $T_m = 2\pi \sqrt{\frac{W}{gK}}$ where g is a gravitational acceleration
- $C_{sm}$ is calculated from $C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A$
- Equivalent static EQ load $P_e = \frac{C_{sm} W}{gK}$

Calculated $P_e$ will be used in the static analysis for conventional design.

*Table 3-17: Summery of EQ calculation*

| H, height of pier | height (m) | E (GPA) | I (m4) | Max diflection (Vs, max) - mm | K (KN/m) | W(KN) | Tm (Period of the bridge ) | Csm | Pe* 2L (kN) to be applied at the top of a pier |
|---|---|---|---|---|---|---|---|---|---|
| 10 Hollow | 10 | 32 | 3.4519 | 0.0002 | 41422.4000 | 9959.7677 | 0.9837 | 0.1456 | 1450.03 |
| 10 Solid | 10 | 32 | 4.5573 | 0.0002 | 54687.5000 | 9971.0246 | 0.8566 | 0.1597 | 1591.92 |
| 15 Hollow | 15 | 32 | 3.4519 | 0.0008 | 18409.9556 | 9964.8689 | 1.4759 | 0.1111 | 1106.96 |
| 15 Solid | 15 | 32 | 4.5573 | 0.0006 | 24305.5556 | 9981.7543 | 1.2856 | 0.1218 | 1215.73 |
| 20 Hollow | 20 | 32 | 3.4519 | 0.0019 | 10355.6000 | 9969.9701 | 1.9684 | 0.0917 | 914.08 |
| 20 Solid | 20 | 32 | 4.5573 | 0.0015 | 13671.8750 | 9992.4840 | 1.7150 | 0.1005 | 1004.29 |
| 25 Hollow | 25 | 32 | 3.4519 | 0.0038 | 6627.5840 | 9975.0713 | 2.4611 | 0.0790 | 788.00 |
| 25 Solid | 25 | 32 | 4.5573 | 0.0029 | 8750.0000 | 10003.2137 | 2.1449 | 0.0866 | 866.09 |

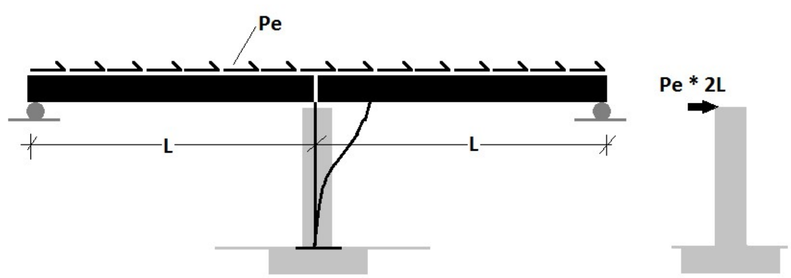



*Figure 3-7: EQ load model*

In order to ultimately load the pier with maximum possible action towards the local weak axis of the cross-section, Pier columns in this thesis are modeled as free at the top and fixed at the bottom along the longitudinal direction. Also, the stiffness of the superstructure and the resistance of abutments at the far ends are neglected. But in practical bridge design, the stiffness of the superstructure and abutment resistance is significant for actions along the longitudinal axis of the bridge system therefore the transverse axis of the bridge is a likely governing axis.

*Table 3-18: Load Factors*

| LOAD | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
|---|---|---|---|---|---|
| DC | **1.25** | **0.9** | **1.25** | **0.9** | **1.00** |
| EV | **1.35** | **1** | **1.35** | **1** | **1.00** |
| EH | **1.5** | **1.5** | **1.5** | **1.5** | **1.00** |
| LL | **1.75** | **1.75** | **0** | **0** | **1.00** |
| BR | **1.75** | **1.75** | **0** | **0** | **1.00** |
| LS | **1.75** | **1.75** | **0** | **0** | **1.00** |
| WS | **0** | **0** | **1.4** | **1.4** | **0.30** |
| WL | **0** | **0** | **0** | **0** | **1.00** |
| CR+SH+TU | **0.5** | **0.5** | **0.5** | **0.5** | **1.00** |

Factored Summery of loads for each case

**Case 01 -1: Hollow Core Pier (10 meters)**

Cross sectional dimensions

| | | |
|---|---|---|
| PCW = | 10 | m |
| PCT1= | 1500 | mm |
| PCT2= | 2500 | mm |
| Thickness (PCT) = | 1500 | mm |
| **Pier Column** | | |
| CH = | 10000 | mm |
| CW= | 2200 | mm |
| CW2 (Thickness) = | 1850 | mm |
| Wall Thickness (CT)= | 400 | mm |

*Table 3-19: Factored Vertical Loads (KN)*

| Vertical Loads Items | Factored Vertical Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $DL_{ET}$ | 4665.00 | 3358.80 | 4665.00 | 3358.80 | 3732.00 |
| $DL_{IT}$ | 4072.50 | 2932.20 | 4072.50 | 2932.20 | 3258.00 |
| $LL_{ET}$ | 2418.50 | 2418.50 | 0.00 | 0.00 | 1382.00 |
| $LL_{IT}$ | 2056.25 | 2056.25 | 0.00 | 0.00 | 1175.00 |
| $W_1$ | 850.50 | 612.36 | 850.50 | 612.36 | 680.40 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{column}$ | 476.52 | 343.09 | 476.52 | 343.09 | 381.22 |
| $W_{column\ bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| ΣVu | 14539.27 | 11721.20 | 10064.52 | 7246.45 | 10608.62 |

*Table 3-20: Factored Horizontal Loads Transverse direction (KN)*

| Horizontal Loads Items | Factored Horizontal Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 268.80 | 268.80 | 57.60 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 126.00 | 126.00 | 27.00 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 58.40 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣHuT** | **0.00** | **0.00** | **394.80** | **394.80** | **143.00** |

*Table 3-21: Factored Horizontal loads- Longitudinal Direction (KN)*

| Horizontal Loads Items | Factored Horizontal Loads | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 201.60 | 201.60 | 43.20 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 117.60 | 117.60 | 25.20 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1450.03 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 22.00 | 0.00 |
| $Br_{Lpier}$ | 284.38 | 284.38 | 0.00 | 0.00 | 162.50 | 0.00 |
| **ΣHuL** | **284.38** | **284.38** | **319.20** | **319.20** | **252.90** | **1450.03** |

*Table 3-22: Factored Horizontal Moments – Transverse (KN-m)*

| Moment due to | Factored Moment (My) | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 4066.68 | 4066.68 | 871.43 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 711.90 | 711.90 | 152.55 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 355.98 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 221.91 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣMy** | **0.00** | **0.00** | **4778.58** | **4778.58** | **1601.87** |

*Table 3-23: Factored Horizontal Moments – Longitudinal (KN-m)*

| Moment due to | Factored Moment (Mx) | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 3050.01 | 3050.01 | 653.57 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 664.44 | 664.44 | 142.38 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 355.98 | 0.00 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 83.60 | 0.00 |
| $Br_{Lpier}$ | 4601.47 | 4601.47 | 0.00 | 0.00 | 2629.41 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 14500.30 |
| **ΣMx** | **4601.47** | **4601.47** | **3714.45** | **3714.45** | **3864.95** | **14500.30** |

**Case 01 - 2: Solid Core Pier (10 meters)**
Cross sectional dimensions

| | | |
|---|---|---|
| PCW = | 10 | m |
| PCT1= | 1500 | mm |
| PCT2= | 2500 | mm |
| Thickness (PCT) = | 1500 | mm |

**Pier Column**

| | | |
|---|---|---|
| CH = | 10000 | mm |
| CW= | 1800 | mm |
| CW2 (Thickness) = | 1600 | mm |

*Figure 3-8: Factored Vertical Loads in (kN)*

| Vertical Loads Item | Factored Vertical Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $DL_{ET}$ | 4665.00 | 3358.80 | 4665.00 | 3358.80 | 3732.00 |
| $DL_{IT}$ | 4072.50 | 2932.20 | 4072.50 | 2932.20 | 3258.00 |
| $LL_{ET}$ | 2803.50 | 2803.50 | 0.00 | 0.00 | 1602.00 |
| $LL_{IT}$ | 2376.50 | 2376.50 | 0.00 | 0.00 | 1358.00 |
| EQ | | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{pier\ cap}$ | 859.50 | 618.84 | 859.50 | 618.84 | 687.60 |
| $W_{column}$ | 716.49 | 515.87 | 716.49 | 515.87 | 573.19 |
| $W_{column\ bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| ΣVu | 15493.49 | 12605.71 | 10313.49 | 7425.71 | 11210.79 |

*Figure 3-9: Factored Horizontal Loads (kN) - Transverse (Strong) axis*

| Horizontal Loads Items | Factored Horizontal Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 268.80 | 268.80 | 57.60 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 126.00 | 126.00 | 27.00 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 58.40 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣHuT** | **0.00** | **0.00** | **394.80** | **394.80** | **143.00** |

*Table 3-24: Factored Horizontal Loads- longitudinal (weak axis)- kN*

| Horizontal Loads Items | Factored Horizontal Loads | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | XTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 201.60 | 201.60 | 43.20 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 117.60 | 117.60 | 25.20 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 22.00 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1591.92 |
| $Br_{Lpier}$ | 284.38 | 284.38 | 0.00 | 0.00 | 162.50 | 0.00 |
| **ΣHuL** | **284.38** | **284.38** | **319.20** | **319.20** | **252.90** | **1591.92** |

*Table 3-25: Factored Moment – Transverse axis (KN-m)*

| Moment due to | Factored Moment (My) | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 4066.68 | 4066.68 | 871.43 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 711.90 | 711.90 | 152.55 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 355.98 |
| $Br_{\ Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣMy** | **0.00** | **0.00** | **4778.58** | **4778.58** | **1379.96** |

*Table 3-26: Factored Moment – Longitudinal axis (KN-m)*

| Moment due to | Factored Moment (Mx) | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 3050.01 | 3050.01 | 653.57 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 664.44 | 664.44 | 142.38 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 355.98 | 0.00 |
| $Br_{Lpier}$ | 4601.47 | 4601.47 | 0.00 | 0.00 | 2629.41 | 0.00 |
| $VW_{Lsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 221.92 | 0.00 |
| un balanced Live load | 228.06 | 228.06 | 0.00 | 0.00 | 130.32 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 15919.20 |
| **ΣMx** | **4829.53** | **4829.53** | **3714.45** | **3714.45** | **4133.59** | **15919.20** |

**Case 02 - 1: Hollow Core Pier (15 meters)**

Cross sectional dimensions

| | | |
|---|---|---|
| PCW = | 10 | m |
| PCT1= | 1500 | mm |
| PCT2= | 2500 | mm |
| Thickness (PCT) = | 1500 | mm |

**Pier Column**

| | | |
|---|---|---|
| CH = | 15000 | mm |
| CW= | 2400 | mm |
| CW2 (Thickness) = | 2000 | mm |
| Wall Thickness (CT)= | 400 | mm |

*Table 3-27: Factored Vertical Loads (kN)*

| Vertical Loads | Factored Vertical Loads | | | | |
|---|---|---|---|---|---|
| Items | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $DL_{ET}$ | 4665.00 | 3358.80 | 4665.00 | 3358.80 | 3732.00 |
| $DL_{IT}$ | 4072.50 | 2932.20 | 4072.50 | 2932.20 | 3258.00 |
| $LL_{ET}$ | 2803.50 | 2803.50 | 0.00 | 0.00 | 1602.00 |
| $LL_{IT}$ | 2376.50 | 2376.50 | 0.00 | 0.00 | 1358.00 |
| $W_1$ | 839.25 | 604.26 | 839.25 | 604.26 | 671.40 |
| $W_{column}$ | 950.13 | 684.09 | 950.13 | 684.09 | 760.10 |
| ΣVu | 15706.88 | 12759.35 | 10526.88 | 7579.35 | 11381.50 |

*Table 3-28: : Factored Horizontal Loads (kN) – Transverse direction*

| Horizontal | Factored Horizontal Loads | | | | |
|---|---|---|---|---|---|
| Loads Items | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 66.26 | 66.26 | 14.20 |
| $W_{Tcap}$ | 0.00 | 0.00 | 7.33 | 7.33 | 1.57 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 51.30 | 51.30 | 10.99 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 14.00 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| ΣHuT | 0.00 | 0.00 | 124.89 | 124.89 | 40.76 |

*Table 3-29: Factored Horizontal Loads (kN) – Longitudinal direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 49.69 | 49.69 | 10.65 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 42.31 | 42.31 | 9.07 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 68.55 | 68.55 | 14.69 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 5.42 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1106.95 |
| $Br_{Lpier}$ | 284.38 | 284.38 | 0.00 | 0.00 | 162.50 | 0.00 |
| **ΣHuL** | **284.38** | **284.38** | **160.54** | **160.54** | **202.32** | **1106.95** |

*Table 3-30: Factored Moment (KN- m)- Transverse Direction*

| Moment due to | Factored Moment (My) | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 1333.68 | 1333.68 | 285.79 |
| $W_{Tcap}$ | 0.00 | 0.00 | 123.85 | 123.85 | 26.54 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 418.10 | 418.10 | 89.59 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 114.80 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 14.39 |
| $Br_{\ Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣMy** | **0.00** | **0.00** | **1875.63** | **1875.63** | **531.11** |

*Table 3-31: Factored Moment (KN-m) – Longitudinal Direction*

| Moment due to | Factored Moment (Mx) | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT |
| $W_{Lpier}$ | 0.00 | 0.00 | 1000.13 | 1000.13 | 214.31 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 715.07 | 715.07 | 153.23 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 558.64 | 558.64 | 119.71 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 114.80 | 0.00 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 5.41 | 0.00 |
| $Br_{Lpier}$ | 6023.35 | 6023.35 | 0.00 | 0.00 | 3441.91 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 16604.25 |
| **ΣMx** | **6023.35** | **6023.35** | **2273.84** | **2273.84** | **4049.38** | **16604.25** |

**Case 02 - 2: Solid Core Pier (15 meters)**

Cross sectional dimensions

| | | |
|---|---|---|
| PCW = | 10 | m |
| PCT1= | 1500 | mm |
| PCT2= | 2500 | mm |
| Thickness (PCT) = | 1500 | mm |

**Pier Column**

| | | |
|---|---|---|
| CH = | 15000 | mm |
| CW= | 1850 | mm |
| CW2 (Thickness) = | 1600 | mm |

*Table 3-32: Factored Vertical Loads (KN)*

| Vertical Loads Items | Factored Vertical Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $DL_{ET}$ | 4665.00 | 3358.80 | 4665.00 | 3358.80 | 3732.00 |
| $DL_{IT}$ | 4072.50 | 2932.20 | 4072.50 | 2932.20 | 3258.00 |
| $LL_{ET}$ | 2803.50 | 2803.50 | 0.00 | 0.00 | 1602.00 |
| $LL_{IT}$ | 2376.50 | 2376.50 | 0.00 | 0.00 | 1358.00 |
| $W_1$ | 858.38 | 618.03 | 858.38 | 618.03 | 686.70 |
| $W_{column}$ | 1105.16 | 795.71 | 1105.16 | 795.71 | 884.13 |
| ΣVu | 15881.03 | 12884.74 | 10701.03 | 7704.74 | 11520.83 |

*Table 3-33: Factored Horizontal Loads (KN) – Transverse Direction*

| orizontal Loads Iter | Factored Horizontal Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 66.26 | 66.26 | 14.20 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 46.58 | 46.58 | 9.98 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 14.00 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| ΣHuT | 0.00 | 0.00 | 112.83 | 112.83 | 38.18 |

*Table 3-34: Factored Horizontal Loads (KN) - Longitudinal direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 49.69 | 49.69 | 10.65 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 43.47 | 43.47 | 9.32 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 5.42 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1215.73 |
| $Br_{Lpier}$ | 284.38 | 284.38 | 0.00 | 0.00 | 162.50 | 0.00 |
| **ΣHuL** | **284.38** | **284.38** | **93.16** | **93.16** | **187.88** | **1215.73** |

*Table 3-35: Factored Moment (KN-m) – Transverse Direction*

| Moment due to | Factored Moment (My) | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 1333.68 | 1333.68 | 285.79 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 379.61 | 379.61 | 81.35 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 114.80 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 54.70 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣMy** | **0.00** | **0.00** | **1713.29** | **1713.29** | **536.63** |

*Table 3-36: Factored Moment (KN-m) – Longitudinal Direction*

| Moment due to | Factored Moment (Mx) | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 1000.13 | 1000.13 | 214.31 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 354.28 | 354.28 | 75.92 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 114.80 | 0.00 |
| $VW_{Lsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 20.66 | 0.00 |
| $Br_{Lpier}$ | 6023.35 | 6023.35 | 0.00 | 0.00 | 3441.91 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 18235.95 |
| **ΣMx** | **6023.35** | **6023.35** | **1354.41** | **1354.41** | **3867.60** | **18235.95** |

**Case 03 - 1: Hollow Core Pier (20 meters)**

Cross sectional dimensioning

| | | |
|---|---|---|
| PCW = | 10 | m |
| PCT1= | 1500 | mm |
| PCT2= | 2500 | mm |
| Thickness (PCT) = | 1500 | mm |

**Pier Column**

| | | |
|---|---|---|
| CH = | 20000 | mm |
| CW= | 2450 | mm |
| CW2 (Thickness) = | 2050 | mm |
| Wall Thickness (CT)= | 400 | mm |

*Table 3-37: Factored Vertical Loads (KN)*

| Vertical Loads Items | Factored Vertical Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $DL_{ET}$ | 4665.00 | 3358.80 | 4665.00 | 3358.80 | 3732.00 |
| $DL_{IT}$ | 4072.50 | 2932.20 | 4072.50 | 2932.20 | 3258.00 |
| $LL_{ET}$ | 2803.50 | 2803.50 | 0.00 | 0.00 | 1602.00 |
| $LL_{IT}$ | 2376.50 | 2376.50 | 0.00 | 0.00 | 1358.00 |
| $W_1$ | 846.00 | 609.12 | 846.00 | 609.12 | 676.80 |
| $W_{column}$ | 1128.96 | 812.85 | 1128.96 | 812.85 | 903.17 |
| ΣVu | 15892.46 | 12892.97 | 10712.46 | 7712.97 | 11529.97 |

*Table 3-38: Factored Horizontal Loads (KN) – Transverse Direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 81.07 | 81.07 | 17.37 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 76.01 | 76.01 | 16.29 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 17.62 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣHuT** | **0.00** | **0.00** | **157.08** | **157.08** | **51.28** |

*Table 3-39: Factored Horizontal Loads (KN) - Longitudinal Direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 60.80 | 60.80 | 13.03 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 70.94 | 70.94 | 15.20 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 6.63 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 914.08 |
| $Br_{Lpier}$ | 284.38 | 284.38 | 0.00 | 0.00 | 162.50 | 0.00 |
| **ΣHuL** | **284.38** | **284.38** | **131.74** | **131.74** | **197.36** | **914.08** |

*Table 3-40: Factored Moment (kN –m ) – Transverse Direction*

| Moment due to | Factored Moment (My) | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 2037.31 | 2037.31 | 436.57 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 809.46 | 809.46 | 173.46 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 173.58 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 66.94 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣMy** | **0.00** | **0.00** | **2846.77** | **2846.77** | **850.54** |

*Table 3-41: Factored Moment (KN- m) – Longitudinal Direction*

| Moment due to | Factored Moment (Mx) | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 1527.89 | 1527.89 | 327.41 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 755.49 | 755.49 | 161.89 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 173.58 | 0.00 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 25.28 | 0.00 |
| $Br_{Lpier}$ | 7445.22 | 7445.22 | 0.00 | 0.00 | 4254.41 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 18281.60 |
| **ΣMx** | **7445.22** | **7445.22** | **2283.38** | **2283.38** | **4942.57** | **18281.60** |

**Case 03 - 2: Solid Core Pier (20 meters)**

Cross sectional dimensioning

| | | |
|---|---|---|
| PCW = | 10 | m |
| PCT1= | 1500 | mm |
| PCT2= | 2500 | mm |
| Thickness (PCT) = | 1500 | mm |

**Pier Column**

| | | |
|---|---|---|
| CH = | 15000 | mm |
| CW= | 2250 | mm |
| CW2 (Thickness) = | 1900 | mm |

*Table 3-42 : Factored Vertical Loads (KN)*

| Vertical Loads Items | Factored Vertical Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $DL_{ET}$ | 4665.00 | 3358.80 | 4665.00 | 3358.80 | 3732.00 |
| $DL_{IT}$ | 4072.50 | 2932.20 | 4072.50 | 2932.20 | 3258.00 |
| $LL_{ET}$ | 2803.50 | 2803.50 | 0.00 | 0.00 | 1602.00 |
| $LL_{IT}$ | 2376.50 | 2376.50 | 0.00 | 0.00 | 1358.00 |
| $W_1$ | 856.13 | 616.41 | 856.13 | 616.41 | 684.90 |
| $W_{column}$ | 1697.65 | 1222.31 | 1697.65 | 1222.31 | 1358.12 |
| ΣVu | 16471.28 | 13309.72 | 11291.28 | 8129.72 | 11993.02 |

*Table 3-43: Factored Horizontal Loads (KN) – Transverse Direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 81.07 | 81.07 | 17.37 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 76.01 | 76.01 | 16.29 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 17.62 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣHuT** | **0.00** | **0.00** | **157.08** | **157.08** | **51.28** |

*Table 3-44: Factored Horizontal Loads (KN) – Longitudinal Direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 60.80 | 60.80 | 13.03 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 70.94 | 70.94 | 15.20 | 0.00 |
| $W_{Lcol.brecing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 6.63 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1004.29 |
| $Br_{Lpier}$ | 284.38 | 284.38 | 0.00 | 0.00 | 162.50 | 0.00 |
| **ΣHuL** | **284.38** | **284.38** | **131.74** | **131.74** | **197.36** | **1004.29** |

*Table 3-45: Factored Moment (KN-m) – Transverse Direction*

| Moment due to | Factored Moment (My) | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 2037.31 | 2037.31 | 436.57 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 809.46 | 809.46 | 173.46 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 173.58 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 66.94 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣMy** | **0.00** | **0.00** | **2846.77** | **2846.77** | **850.54** |

*Table 3-46: Factored Moment (KN -m) – longitudinal Direction*

| Moment due to | Factored Moment (Mx) | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 1527.89 | 1527.89 | 327.41 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 755.49 | 755.49 | 161.89 | 0.00 |
| $W_{Lcol.brecing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 173.58 | 0.00 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 25.28 | 0.00 |
| $Br_{Lpier}$ | 7445.22 | 7445.22 | 0.00 | 0.00 | 4254.41 | 0.00 |
| EQ | -455.25 | -327.78 | -455.25 | -327.78 | -364.20 | 20085.80 |
| **ΣMx** | **6989.97** | **7117.44** | **1828.13** | **1955.60** | **4578.37** | **20085.80** |

**Case 04 - 1: Hollow Core Pier (25 meters)**

Cross sectional dimensioning

| | | |
|---|---|---|
| PCW = | 10 | m |
| PCT1= | 1500 | mm |
| PCT2= | 2500 | mm |
| Thickness (PCT) = | 1500 | mm |

**Pier Column**

| | | |
|---|---|---|
| CH = | 25000 | mm |
| CW= | 2550 | mm |
| CW2 (Thickness) = | 2050 | mm |
| Wall Thickness (CT)= | 400 | mm |

*Table 3-47: Factored Vertical Loads (KN)*

| Vertical Loads Items | Factored Vertical Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $DL_{ET}$ | 4665.00 | 3358.80 | 4665.00 | 3358.80 | 3732.00 |
| $DL_{IT}$ | 4072.50 | 2932.20 | 4072.50 | 2932.20 | 3258.00 |
| $LL_{ET}$ | 2803.50 | 2803.50 | 0.00 | 0.00 | 1602.00 |
| $LL_{IT}$ | 2376.50 | 2376.50 | 0.00 | 0.00 | 1358.00 |
| $W_1$ | 843.75 | 607.50 | 843.75 | 607.50 | 675.00 |
| $W_{column}$ | 1500.00 | 1080.00 | 1500.00 | 1080.00 | 1200.00 |
| **ΣVu** | **16261.25** | **13158.50** | **11081.25** | **7978.50** | **11825.00** |

*Table 3-48: Factored Horizontal loads (KN) – Transverse Direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 93.60 | 93.60 | 20.06 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 109.69 | 109.69 | 23.51 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 20.34 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣHuT** | **0.00** | **0.00** | **203.29** | **203.29** | **63.90** |

*Table 3-49: Factored Horizontal loads (KN) – longitudinal Direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 70.21 | 70.21 | 15.04 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 102.39 | 102.39 | 21.94 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 7.66 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 788.00 |
| $Br_{Lpier}$ | 284.38 | 284.38 | 0.00 | 0.00 | 162.50 | 0.00 |
| **ΣHuL** | **284.38** | **284.38** | **172.59** | **172.59** | **207.15** | **788.00** |

*Table 3-50: Factored Moment (KN-m) – Transverse Direction*

| Moment due to | Factored Moment (My) | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 2820.19 | 2820.19 | 604.33 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 1442.42 | 1442.42 | 309.09 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 238.91 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 20.34 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣMy** | **0.00** | **0.00** | **4262.62** | **4262.62** | **1172.67** |

*Table 3-51: Factored Moment (longitudinal Direction)*

| Moment due to | Factored Moment (Mx) | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 2115.27 | 2115.27 | 453.27 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 1346.38 | 1346.38 | 288.51 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 238.91 | 0.00 |
| $Br_{Lpier}$ | 8867.10 | 8867.10 | 0.00 | 0.00 | 5066.91 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 19700.00 |
| **ΣMx** | **8867.10** | **8867.10** | **3461.65** | **3461.65** | **6047.60** | **19700.00** |

### Case 04 - 2: Solid Core Pier (25 meters)

Cross sectional dimensioning

| | | |
|---|---|---|
| PCW = | 10 | m |
| PCT1= | 1500 | mm |
| PCT2= | 2500 | mm |
| Thickness (PCT) = | 1500 | mm |

**Pier Column**

| | | |
|---|---|---|
| CH = | 25000 | mm |
| CW= | 2500 | mm |
| CW2 (Thickness) = | 2000 | mm |

*Table 3-52: Factored Vertical Loads (KN)*

| Vertical Loads | Factored Vertical Loads | | | | |
|---|---|---|---|---|---|
| Items | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $DL_{ET}$ | 4665.00 | 3358.80 | 4665.00 | 3358.80 | 3732.00 |
| $DL_{IT}$ | 4072.50 | 2932.20 | 4072.50 | 2932.20 | 3258.00 |
| $LL_{ET}$ | 2803.50 | 2803.50 | 0.00 | 0.00 | 1602.00 |
| $LL_{IT}$ | 2376.50 | 2376.50 | 0.00 | 0.00 | 1358.00 |
| $W_1$ | 843.75 | 607.50 | 843.75 | 607.50 | 675.00 |
| $W_{column}$ | 3117.77 | 2244.80 | 3117.77 | 2244.80 | 2494.22 |
| $\Sigma Vu$ | 17879.02 | 14323.30 | 12699.02 | 9143.30 | 13119.22 |

*Table 3-53: Factored Horizontal loads (KN) – Transverse Direction*

| Horizontal | Factored Horizontal Loads | | | | |
|---|---|---|---|---|---|
| Loads Items | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 93.60 | 93.60 | 20.06 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 109.69 | 109.69 | 23.51 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 20.34 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $\Sigma HuT$ | 0.00 | 0.00 | 203.29 | 203.29 | 63.90 |

*Table 3-54: Factored Horizontal loads (KN) – longitudinal Direction*

| Horizontal Loads Items | Factored Horizontal Loads | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 70.21 | 70.21 | 15.04 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 102.39 | 102.39 | 21.94 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 7.66 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 866.09 |
| $Br_{Lpier}$ | 284.38 | 284.38 | 0.00 | 0.00 | 162.50 | 0.00 |
| **ΣHuL** | **284.38** | **284.38** | **172.59** | **172.59** | **207.15** | **866.09** |

*Table 3-55: Factored Moment (KN-m) – Transverse Direction*

| Moment due to | Factored Moment (My) | | | | |
|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I |
| $W_{Tpier}$ | 0.00 | 0.00 | 2820.19 | 2820.19 | 604.33 |
| $W_{Tcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $W_{Tcolumn}$ | 0.00 | 0.00 | 1442.42 | 1442.42 | 309.09 |
| $VW_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 238.91 |
| $VW_{Tsuper}$ | 0.00 | 0.00 | 0.00 | 0.00 | 20.34 |
| $Br_{Tpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Pf | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| **ΣMy** | **0.00** | **0.00** | **4262.62** | **4262.62** | **1172.67** |

*Table 3-56: Factored Moment (KN-m) – Longitudinal Direction*

| Moment due to | Factored Moment (Mx) | | | | | |
|---|---|---|---|---|---|---|
| | STRENGTH I | STRENGTH Ia | STRENGTH III | STRENGTH IIIa | SERVICE I | EXTREME EVENT I |
| $W_{Lpier}$ | 0.00 | 0.00 | 2115.27 | 2115.27 | 453.27 | 0.00 |
| $W_{Lcap}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $WL_{column}$ | 0.00 | 0.00 | 1346.38 | 1346.38 | 288.51 | 0.00 |
| $W_{Lcol.bracing}$ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| $VW_{Lpier}$ | 0.00 | 0.00 | 0.00 | 0.00 | 238.91 | 0.00 |
| $VWL_{super}$ | 0.00 | 0.00 | 0.00 | 0.00 | 7.66 | 0.00 |
| $Br_{Lpier}$ | 8867.10 | 8867.10 | 0.00 | 0.00 | 5066.91 | 0.00 |
| EQ | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 21652.25 |
| **ΣMx** | **8867.10** | **8867.10** | **3461.65** | **3461.65** | **6055.26** | **21652.25** |

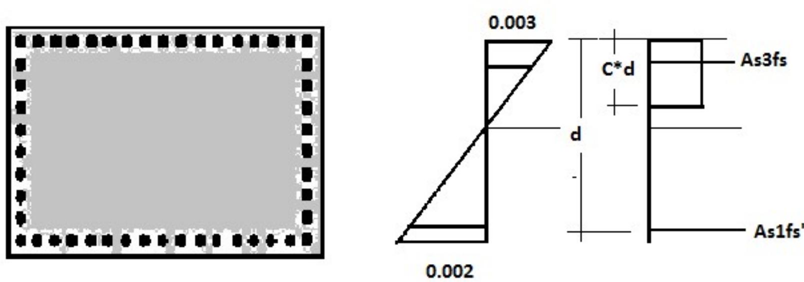
**Analysis**

First order structural analysis is done using SAP2000 after modeling each case structural member geometry and loading condition. The analysis results for along with the design member cross sections will be presented after the design.

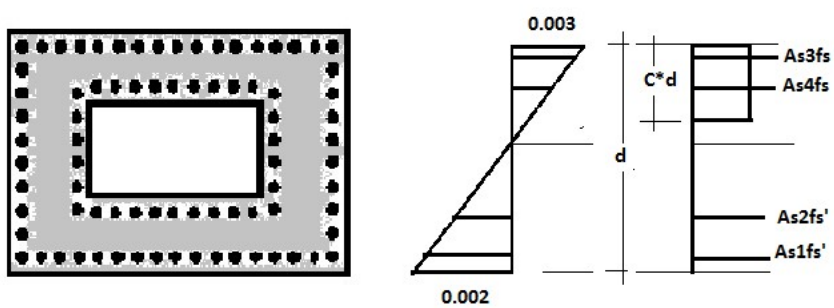
### 3.4.2 Interaction Development

Interaction curves are developed for each case considering the 0.002 and 0.003 to be the ultimate strain of steel and concrete respectively. Assuming the columns are compression controlled, the capacity reduction factor used is 0.65. To develop an interaction diagram the plastic centroid is assumed to be shifting. The coefficient C of the location of the plastic centroid is assumed from zero to 1, when it is assumed to be zero, the section will be entirely under tension and when it is 1, the section will be under compression. The same analysis procedure is carried out in both of the major directions. The principal axis is determined using the tangent value of the ratio of Maximum moment capacities on x and y.

The tabulated results obtained by changing the value of C and contour maps are plotted.




Solid Core section

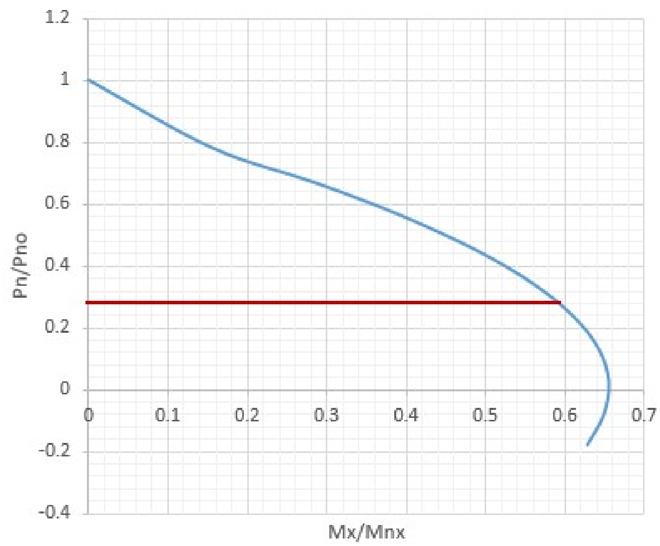



Stress Hollow core section

*Figure 3-10: Stress and Strain block diagram*

Cross sectional axial and flexural strengths of each section is calculated using the formulas stated on figure 3-10. Detailed numbers obtained through this analysis is tabulated on Appendix A. Moment resistance in both of the directions is calculated varying the value of C from 0 to 1. Interaction diagram is plotted using Bresler's reciprocal load method.

**Case 01 – 1: 10m long Hollow Pier Section**



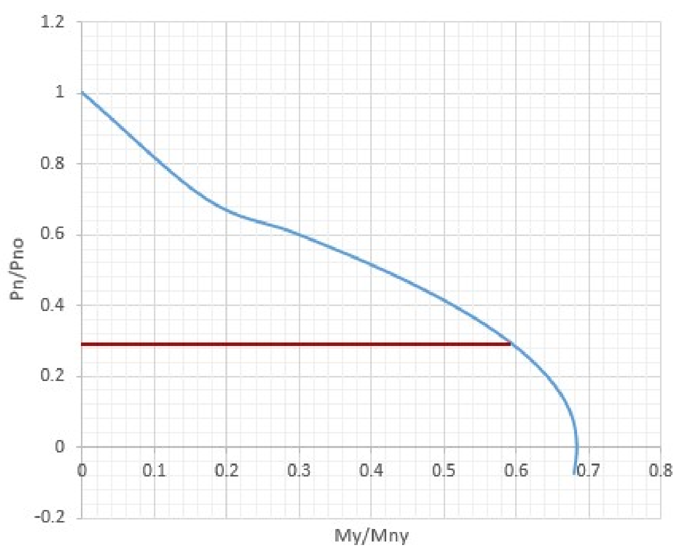




*Figure 3-11: Resistance moment contour Hollow 10m*

Using Bresler's equation, the nominal axial load capacity of the section ($P_{ni}$) = **22.47MN**

$P_{ni}/P_{no}$ = 0.2728

$P_{no}$ (maximum axial load carrying capacity) = 82.3MN

$M_{ny}$ (Maximum moment carrying capacity of the section along Y – axis) = 57MN-m

$M_{nx}$ (Maximum moment carrying capacity of the section along X-axis) = 51.8MN-m

Design factored resistance moment along x = 0.58*51.8 = **30.0MN- m**

Design factored resistance moment along y = 0.59* 57.0 = **33.69MN- m**

### Case 01 – 2:  10m long Solid Pier Section

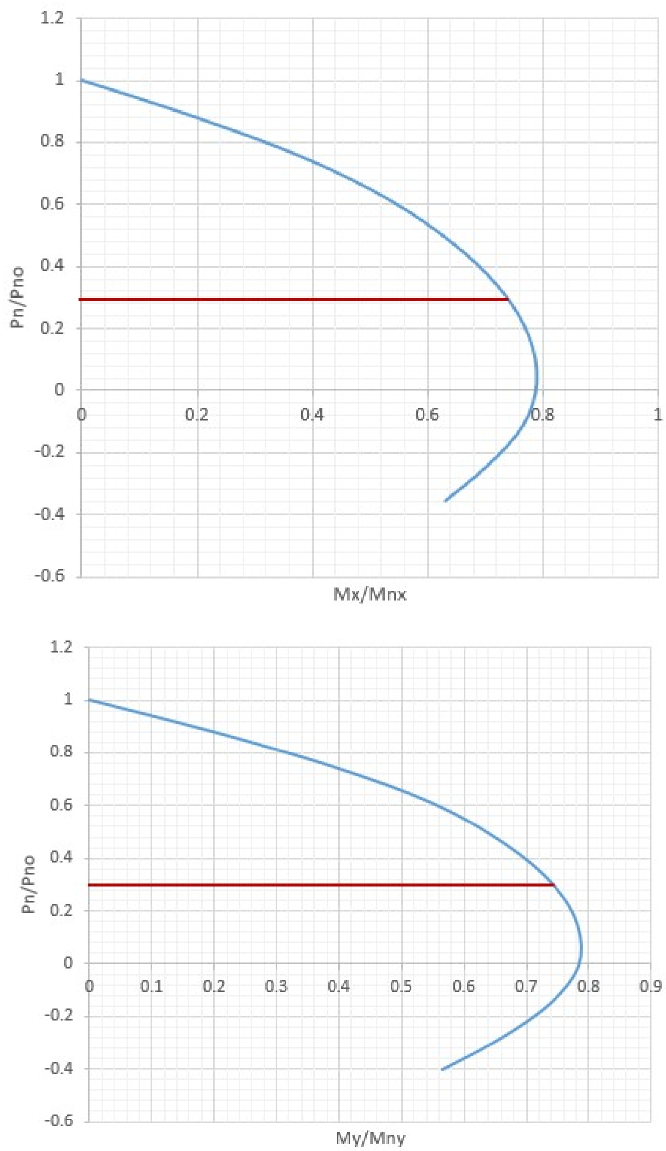



*Figure 3-12: Resistance moment contour Solid 10m*

Using Bresler's equation, the nominal axial load capacity of the section ($P_{ni}$) = **23.9MN**

$P_{ni}/P_{no}$ = 0.29

$P_{no}$ (maximum axial load carrying capacity) = 81.9MN

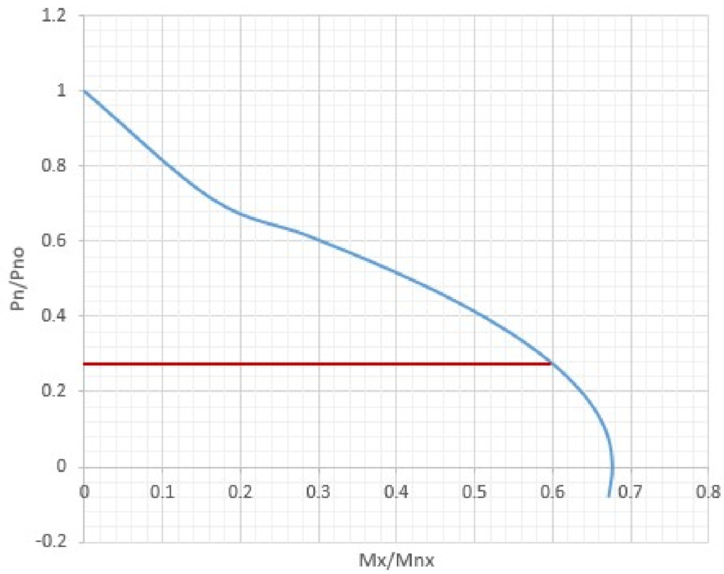
$M_{ny}$ (Maximum moment carrying capacity of the section along Y – axis) = 42.2MN-m

$M_{nx}$ (Maximum moment carrying capacity of the section along X-axis) = 39.8MN-m

Design factored resistance moment along x = 0.67*39.8 =   **34.7MN- m**

Design factored resistance moment along y = 0.72* 42.2 = **30.38MN- m**

**Case 02 – 1: 15m long Hollow Pier Section**



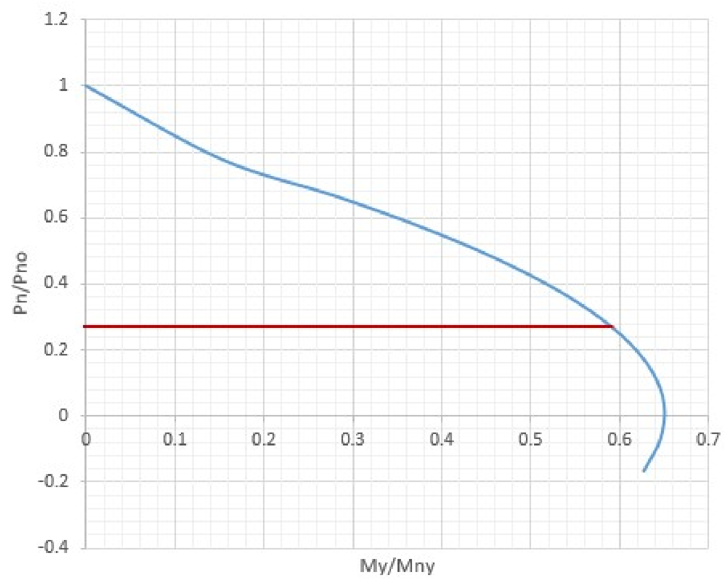




*Figure 3-13: Resistance moment contour Hollow 15m*

Using Bresler's equation, the nominal axial load capacity of the section ($P_{ni}$) = **24.01MN**

$P_{ni}/P_{no}$ = 0.276

$P_{no}$ (maximum axial load carrying capacity) = 87.01MN

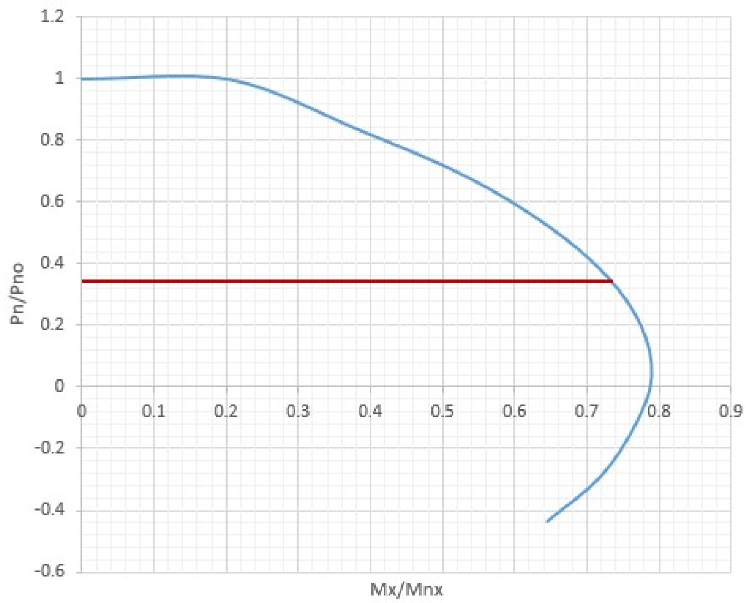
$M_{ny}$ (Maximum moment carrying capacity of the section along Y – axis) 64.18MN-m

$M_{nx}$ (Maximum moment carrying capacity of the section along X-axis) = 60.13MN-m

Design factored resistance moment along x = 0.60*60.13 = **36.08MN- m**

Design factored resistance moment along y = 0.58* 64.19 = **37.23MN- m**

**Case 02 – 2: 15m long Solid Pier Section**



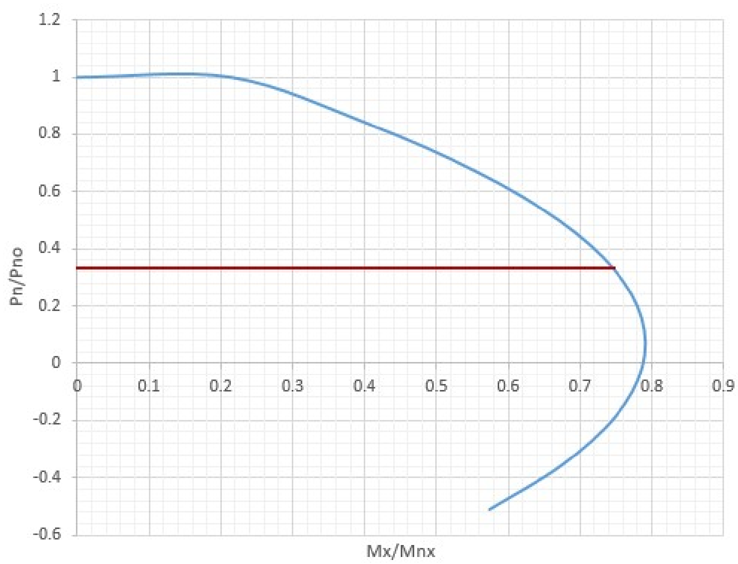




*Figure 3-14: Resistance moment contour Solid 15m*

Using Bresler's equation, the nominal axial load capacity of the section ($P_{ni}$) = **26.6MN**

$P_{ni}/P_{no}$ = 0.346

$P_{no}$ (maximum axial load carrying capacity) = 77MN

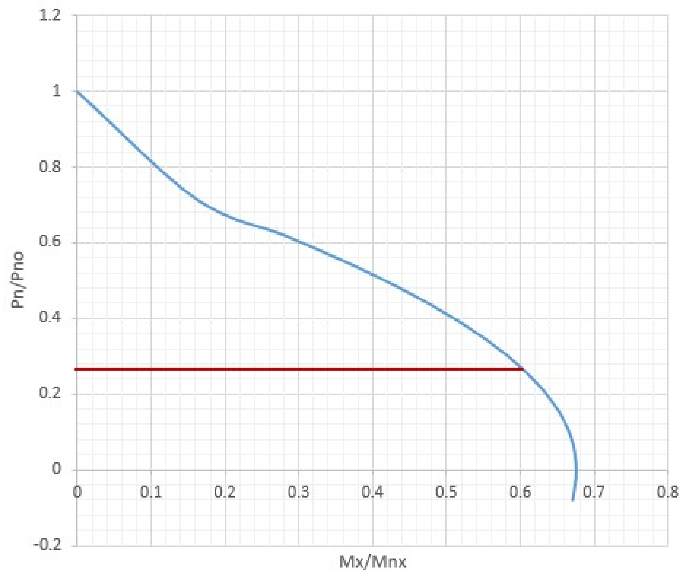
$M_{ny}$ (Maximum moment carrying capacity of the section along Y – axis) 45.12MN-m

$M_{nx}$ (Maximum moment carrying capacity of the section along X-axis) = 41.15MN-m

Design factored resistance moment along x = 0.74*60.13 = **30.45MN- m**

Design factored resistance moment along y = 0.75* 64.19 = **33.83MN- m**

**Case 03 – 1:  20m long Hollow Pier Section**



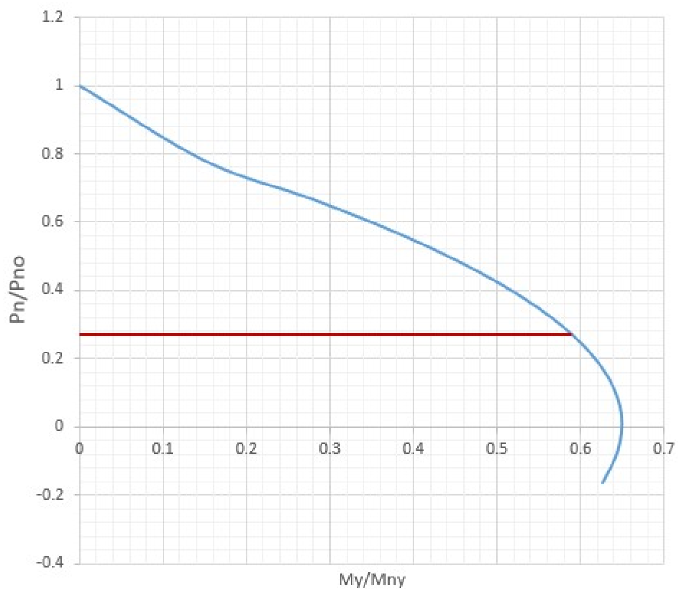




*Figure 3-15: Resistance moment contour Hollow 20m*

Using Bresler's equation, the nominal axial load capacity of the section ($P_{ni}$) = **24.6MN**

$P_{ni}/P_{no}$ = 0.0.277

$P_{no}$ (maximum axial load carrying capacity) = 88.9MN

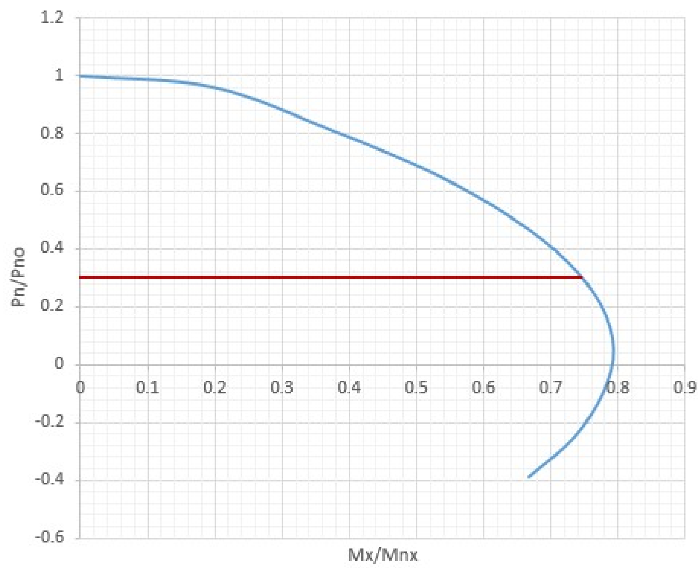
$M_{ny}$ (Maximum moment carrying capacity of the section along Y – axis) = 67.34MN-m

$M_{nx}$ (Maximum moment carrying capacity of the section along X-axis) = 63.32MN-m

Design factored resistance moment along x = 0.62*63.32=   **39.2MN- m**

Design factored resistance moment along y = 0.58* 67.36 = **40.42MN- m**

**Case 03 – 2: 20m long Solid Pier Section**



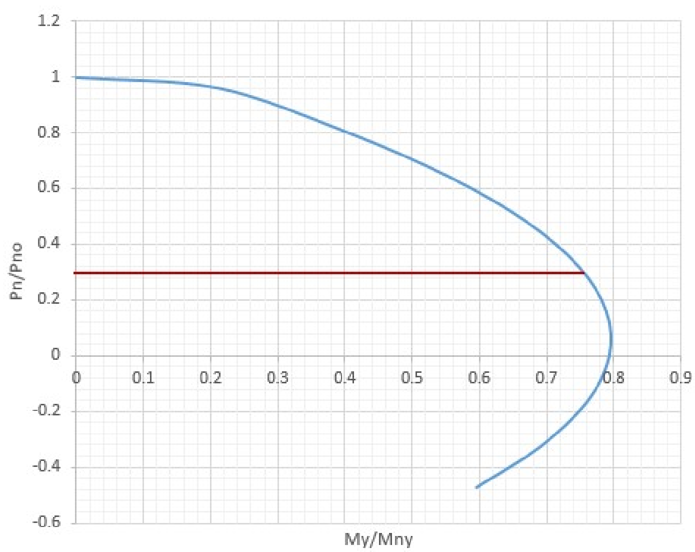




*Figure 3-16: Resistance moment contour Solid 20m*

Using Bresler's equation, the nominal axial load capacity of the section ($P_{ni}$) = **35.46MN**

$P_{ni}/P_{no}$ = 0.314

$P_{no}$ (maximum axial load carrying capacity) = 112.9MN

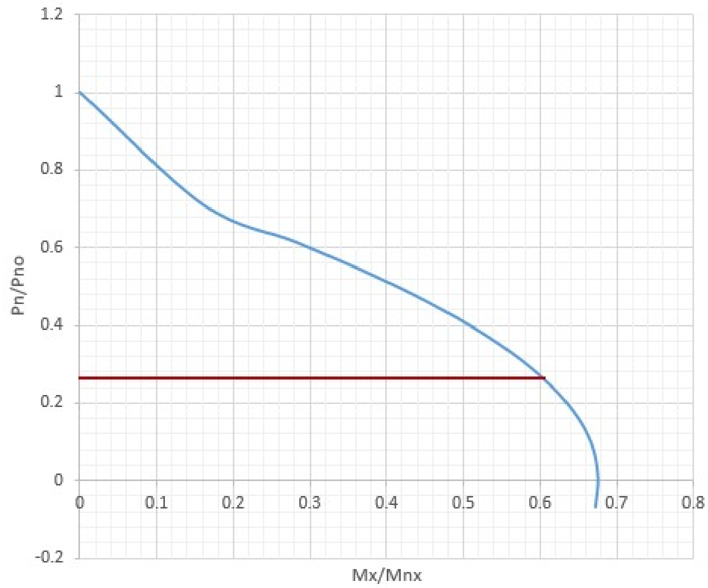
$M_{ny}$ (Maximum moment carrying capacity of the section along Y – axis) = 76.91MN-m

$M_{nx}$ (Maximum moment carrying capacity of the section along X-axis) = 67.40MN-m

Design factored resistance moment along x = 0.75*67.40= **50.56MN- m**

Design factored resistance moment along y = 0.76* 76.91 = **58.47MN- m**

### Case 04 – 1: 25m long Hollow Pier Section



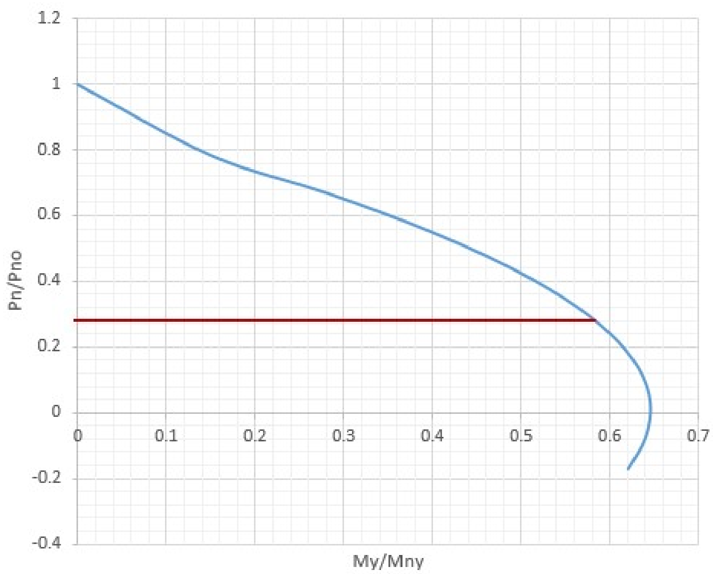




*Figure 3-17: Resistance moment contour Hollow 25m*

Using Bresler's equation, the nominal axial load capacity of the section ($P_{ni}$) = **25.25MN**

$P_{ni}/P_{no}$ = 0.277

$P_{no}$ (maximum axial load carrying capacity) = 91.03MN

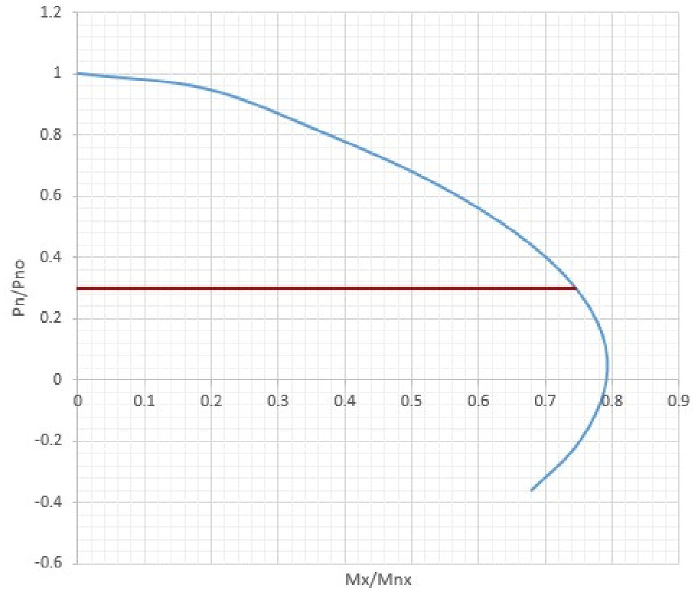
$M_{ny}$ (Maximum moment carrying capacity of the section along Y – axis) = 71.85MN-m

$M_{nx}$ (Maximum moment carrying capacity of the section along X-axis) = 65.03MN-m

Design factored resistance moment along x = 0.61*65.02= **39.66MN- m**

Design factored resistance moment along y = 0.58* 71.85 = **41.67MN- m**

**Case 04 – 2: 25m long Solid Pier Section**



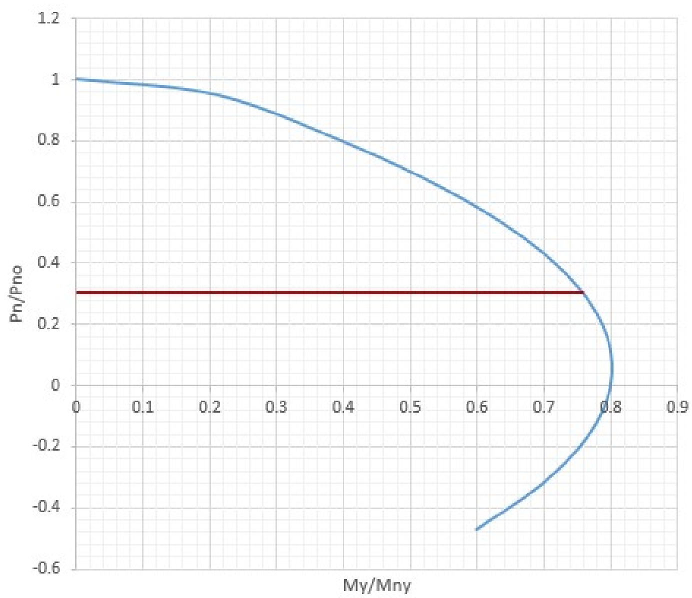




*Figure 3-18: Resistance moment contour Solid 25m*

Using Bresler's equation, the nominal axial load capacity of the section ($P_{ni}$) = **25.25MN**

$P_{ni}/P_{no}$ = 0.277

$P_{no}$ (maximum axial load carrying capacity) = 91.03MN

$M_{ny}$ (Maximum moment carrying capacity of the section along Y – axis) = 71.85MN-m

$M_{nx}$ (Maximum moment carrying capacity of the section along X-axis) = 65.03MN-m

Design factored resistance moment along x = 0.61*65.02= **39.66MN- m**

Design factored resistance moment along y = 0.58* 71.85 = **41.67MN- m**

## 3.5 Conventional Design

Shear design: tie design for solid column and "shear wall" model transverse reinforcement design comparison

A column with high slenderness ratio will have a considerable reduction in strength, whereas a low slenderness ratio means that the column is relatively short and the reduction in strength may not be significant. The slenderness ratio is the ratio of the column effective height to the radius of gyration.

Effective length factor along the longitudinal axis of all of the column cases can be estimated to be 0.8 (assuming fixed foundation and pinned support at the top) and 2.1 for transverse direction (fixed foundation and free top edge). The code (ACI 318 11) allows a column to be considered as non-sway if Stability index (Q) is not greater than 0.05. In this thesis, the foundation is assumed to be fixed and effective length factor used is 0.8 for design along the transverse direction and 1.2 for design along the longitudinal direction.

Where:

$$Q = \frac{\sum P_{u*\Delta_o}}{V_{us}*l_s} \text{------------------------------------} 3.12$$

$P_u$ and $V_{us}$ are the total factored vertical load and story shear respectively. $\Delta_o$ is the first order deflection between the top and the bottom of the story due to the story shear. $L_c$ is the length of the column measured from joint to joint.

According to ACI code 10.10.4, section properties shall be represented using the modulus of elasticity Ec and the moment of inertia for a column as only 70% (ACI 10.10.4.1) of the gross moment of inertia of the cross section. And accordingly, depending on the sway and non-sway property of the pier column, appropriate moment magnification factor is obtained. Separate results for each of the cases are presented below.

The effect of slenderness may be neglected when

$$\frac{Kl_u}{r} \leq \begin{cases} 22 & \text{for unbraced (non} - \text{sway frame} \\ 34 - \dfrac{12M_1}{M_2} & \text{for braced frame} \end{cases}$$ --------------------3.13

Moment magnifier for non- sway frame is calculated using:

$$\delta_{ns} = \frac{1}{\left(1 - \frac{P_u}{0.75P_c}\right)} \geq 1$$ -------------------------------- 3.14

$$M_c = \delta_{ns} M_2$$

The Euler buckling load $P_c$ is computed = $\frac{\pi^2 EI}{(kl_u)^2}$

For sway frames,

$$\delta_s = \frac{1}{(1-Q)} \geq 1$$ -----------------------------------------3.15

If $\delta_s$ Is greater than 1.5, then use

$$\delta_s = \frac{1}{\left(1 - \frac{\sum P_u}{0.75 \sum P_c}\right)} \geq 1$$ ---------------------------------3.16

Design moments are determined using the expression below:

$$M_1 = M_{1ns} + \delta_s M_{1s}$$ ---------------------------- 3.17(a)

$$M_2 = M_{2ns} + \delta_s M_{2s}$$ ------------------------------ 3.17(b)

Where:

- $M_{1s}$ and $M_{2s}$ are moments that cause side sway. Considered load combinations for this scenario are:
    - Strength III
    - Strength IIIa
    - Extreme events I
- $M_{1ns}$ and $M_{2ns}$ are moments that are not cause for side sway. Considered load combinations for this scenario are:
    - Strength I
    - Strength 1a

The design output is presented, discussed and the output comparison is made at the results section.

Cm is taken as 1, considering the probability of existence of loads between the supports. (All of the moment quantities are expressed in Mega Newton per unit meter)

**Case 01-1 – 10m Hollow section**

*Table 3-57: Slenderness effect Hollow 10m*

| | Trans. Direction | Longt. Direction |
|---|---|---|
| | | |
| **Check for Stability Index, Q =** | 0.018345049 | 0.012406887 |
| **recommendations =** | NON SWAY | NON SWAY |
| | | |
| | | |
| **Klu/r =** | 19.1568222 | 6.073122806 |
| **Boundary values =** | 22 | 22 |
| **Recommendations=** | Slenderness Ignored! | Slenderness Ignored! |

**Case 01-2 – 10m Solid Section**

*Table 3-58: Slenderness effect Solid 10m*

| | Trans. Direction | Longt. Direction |
|---|---|---|
| | | |
| **Check for Stability Index, Q =** | 0.035484219 | 0.027348479 |
| **recommendations =** | NON SWAY | NON SWAY |
| | | |
| | | |
| **Klu/r =** | 22.4030836 | 7.450822286 |
| **Boundary values =** | 22 | 22 |
| **Recommendations=** | Design for Slenderness! | Slenderness Ignored! |

Moment Magnification factor calculation (moment in MN-m)

| | Trans. Direction | Longt. Direction |
|---|---|---|
| Cm = | **1.000** | |
| dns = | **1.102** | |
| Mc = | **17.540** | |

**Case 02 -1 – 15m Hollow Section**

*Table 3-59: Slenderness effect Hollow 15m*

| | Trans. Direction | Longt. Direction |
|---|---|---|
| | | |
| **Check for Stability Index, Q =** | 0.03451323 | 0.024952132 |
| **recommendations =** | NON SWAY | NON SWAY |
| | | |
| | | |
| **Klu/r =** | 26.12695984 | 8.558310631 |
| **Boundary values =** | 22 | 22 |
| **Recommendations=** | Design for Slenderness! | Slenderness Ignored! |

Moment Magnification factor calculation (moment in MN-m)

| | Trans. Direction | Longt. Direction |
|---|---|---|
| Cm = | **1.000** | |
| dns = | **1.105** | |
| Mc = | **18.347** | |

**Case 02-2 - 15 Solid Section**

*Table 3-60: Slenderness effect Solid 15m*

| | Trans. Direction | Longt. Direction |
|---|---|---|
| | | |
| **Check for Stability Index, Q** | 0.079961428 | 0.057987967 |
| **recommendations =** | SWAY | SWAY |
| | | |
| | | |
| **Klu/r =** | 33.70817635 | 10.85958198 |
| **Boundary values =** | 34 | 34 |
| **Recommendations=** | Slenderness Ignored! | Slenderness Ignored! |

According to equation 3.2, slenderness effect can be neglected for sway frames with slenderness effect $Kl_u/r > 34$.

**Case 03-1- 20m Hollow Section**

*Table 3-61: Slenderness effect Hollow 20m*

| | Trans. Direction | Longt. Direction |
|---|---|---|
| | | |
| **Check for Stability Index, Q =** | 0.057580948 | 0.041977169 |
| **recommendations =** | SWAY | NON SWAY |
| | | |
| | | |
| **Klu/r =** | 33.83445098 | 11.13176941 |
| **Boundary values =** | 34 | 22 |
| **Recommendations=** | Slenderness Ignored! | Slenderness Ignored! |

**Case 03 – 2 -20m Solid Section**

*Table 3-62: Slenderness effect Solid 20m*

| | Trans. Direction | Longt. Direction |
|---|---|---|
| | | |
| **Check for Stability Index, Q =** | 0.074555338 | 0.051263826 |
| **recommendations =** | SWAY | SWAY |
| | | |
| | | |
| **Klu/r =** | 34.83225883 | 10.91333347 |
| **Boundary values =** | 34 | 34 |
| **Recommendations=** | Design for Slenderness! | Slenderness Ignored! |

Moment Magnification factor calculation (moment in MN-m)

| | Trans. Direction | Longt. Direction |
|---|---|---|
| Cm = | **1.000** | |
| dns = | **1.081** | |
| Mc = | **28.821** | |

**Case 04 -1: 25m Hollow Section**

*Table 3-63: Slenderness effect Hollow 25m*

| | Trans. Direction | Longt. Direction |
|---|---|---|
| | | |
| **Check for Stability Index, Q =** | 0.111030511 | 0.075466705 |
| **recommendations =** | SWAY | SWAY |
| | | |
| | | |
| **Klu/r =** | 42.40567531 | 13.51218417 |
| **Boundary values =** | 34 | 34 |
| **Recommendations=** | Design for Slenderness! | Slenderness Ignored! |

Moment Magnification factor calculation (moment in MN-m)

| | Trans. Direction | Longt. Direction |
|---|---|---|
| Cm = | **1.000** | |
| dns = | **1.125** | |
| Mc = | **31.028** | |

**Case 04- 2: 25m Solid Section**

*Table 3-64: Slenderness effect Solid 25m*

| | Trans. Direction | Longt. Direction |
|---|---|---|
| | | |
| **Check for Stability Index, Q =** | 0.120098839 | 0.073157272 |
| **recommendations =** | SWAY | SWAY |
| | | |
| | | |
| **Klu/r =** | 40.61386344 | 11.94111426 |
| **Boundary values =** | 34 | 34 |
| **Recommendations=** | Design for Slenderness! | Slenderness Ignored! |

Moment Magnification factor calculation (moment in MN-m)

| | Trans. Direction | Longt. Direction |
|---|---|---|
| Cm = | **1.000** | |
| dns = | **1.136** | |
| Mc = | **33.475** | |

Summery of conventional design is presented on table 3.67

*Table 3-65: Conventional design summery*

| | Hollow | | | | Solid | | | |
|---|---|---|---|---|---|---|---|---|
| Case | Case 01-1 | Case 02-1 | Case 03-1 | Case 04 -1 | Case 01-2 | Case 02-2 | Case 03-2 | Case 04-2 |
| **Proposed width - mm** | 2200 | 2400 | 2450 | 2550 | 1600 | 1850 | 2250 | 2500 |
| **Proposed length - mm** | 1850 | 2000 | 2050 | 2050 | 10000 | 1600 | 1950 | 2000 |
| **Proposed height – mm** | 10000 | 15000 | 20000 | 25000 | 0 | 15000 | 20000 | 25000 |
| **Proposed wall thickness – mm** | 400 | 400 | 400 | 400 | 69.12 | 0 | 0 | 0 |
| **Concrete weight - kg/m$^3$** | 62.4 | 69.12 | 71.04 | 72.96 | 39676.22 | 71.04 | 105.3 | 120 |
| **Reinforcement area longitudinal – mm$^2$** | 87126.84 | 93828.9 | 96509.73 | 99190.55 | 311.4583 | 40346.43 | 49729.32 | 54420.76 |
| **Reinforcement - kg/m** | 683.9457 | 736.5569 | 757.6014 | 778.6458 | 0.001411 | 316.7195 | 390.3752 | 427.203 |
| **lateral reinforcement/meter – mm$^2$/m** | 0.00096 | 0.000642 | 0.000512 | 0.000429 | 11.07792 | 0.001124 | 0.000649 | 0.000507 |
| **lateral reinforcement kg/ m** | 7.534352 | 5.03656 | 4.018572 | 3.363725 | 15919.2 | 8.825755 | 5.09465 | 3.979165 |
| **Action factored moment longitudinal- KNm** | 14500.3 | 16604.25 | 18281.6 | 19700 | 4778.575 | 18235.95 | 20085.8 | 21652.25 |
| **Action factored moment transverse – KNm** | 4778.575 | 1713.286 | 2846.772 | 4262.618 | 1591.92 | 1813.286 | 2846.772 | 4262.618 |
| **Action factored shear longitudinal - KN** | 1450.03 | 1106.95 | 914.08 | 788 | 394.8 | 1215.73 | 1004.29 | 866.09 |
| **Action factored shear transverse - KN** | 394.8 | 112.8344 | 157.08 | 203.294 | 15493.49 | 112.8344 | 157.08 | 203.294 |
| **Factored action axial load - KN** | 14569.72 | 15610.22 | 15941.32 | 16324.55 | 34700 | 15881.03 | 16950.96 | 17879.02 |
| **Action magnified longitudinal moment-slenderness effect KN-m** | 14500.3 | 16604.25 | 18281.6 | 19700 | 17540 | 18235.95 | 20085.8 | 21652.25 |
| **Action magnified transverse moment-slenderness effect KN-m** | 4778.575 | 18347 | 2846.772 | 31028 | 23686.97 | 1813.286 | 28821 | 33475 |
| **Factored Resistance longitudinal moment-KN-m** | 30000 | 36080 | 39200 | 39660 | 30380 | 30450 | 50560 | 39660 |
| **Factored Resistance transverse moment-KN-m** | 33690 | 37230 | 40420 | 41670 | 23900 | 33830 | 58470 | 41670 |
| **Factored axial load resistance** | 22470 | 24010 | 24600 | 25250 | 46119.78 | 26600 | 35460 | 25250 |

## 3.6 Capacity Design

A structure designed conventionally is also checked for its seismic performance using capacity design mechanism. To satisfy the global performance criteria of a bridge, the displacement generated by the global analysis or the stand alone analysis or the larger of the two shall be less than the frame displacement when any plastic hinge reaches its ultimate capacity.

**Structural displacement demand:**

$$\Delta_D^L \leq \Delta_C^L$$

Where:

$\Delta_D^L$ is the displacement demand taken along the local principal axis of the ductile member (is calculated as the displacement of the pier, assuming fixed foundation.

$\Delta_C^L$ is the displacement capacity taken along the local principal axis corresponding to $\Delta_D^L$ of the ductile member as determined using pushover analysis.

**Member ductility requirements**

AASHTO Seismic design category SDC D is taken under consideration for all of bridge elements in this thesis.

For a single pier column along the wall direction $\mu_D \leq 5$, and for a pier along strong direction $\mu_D \leq 1$.

Where, $\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_{yi}}$

$\Delta_{pd}$ is the plastic displacement demand and

$\Delta_{yi}$ is the idealized yield displacement corresponding to the idealized yield curvature.

**Moment curvature (M-C) analysis**

**Material Properties**

The main input for M-C analysis is the material property. The mechanical property of materials to be used for design is specified. Stress strain characteristics at points of yield, ultimate and crack should be known to determine Moment and curvature parameters.

**Concrete:** For all moment curvature analysis cases, Mandar's Confined concrete model is used. Following the shear reinforcement design, sample design is attached at the appendix B. In average, a shear reinforcement amount of 14mm diameter bar with 150mm c-c spacing. This tie reinforcement causes confinement in the concrete and it is necessary to assume Mander's confined concrete model. In this thesis. For each section, corresponding confined stress-strain graph is developed and used in for moment V curvature analysis. The development of stress- strain graph of is made using SAP 2000 V18 Section developer.

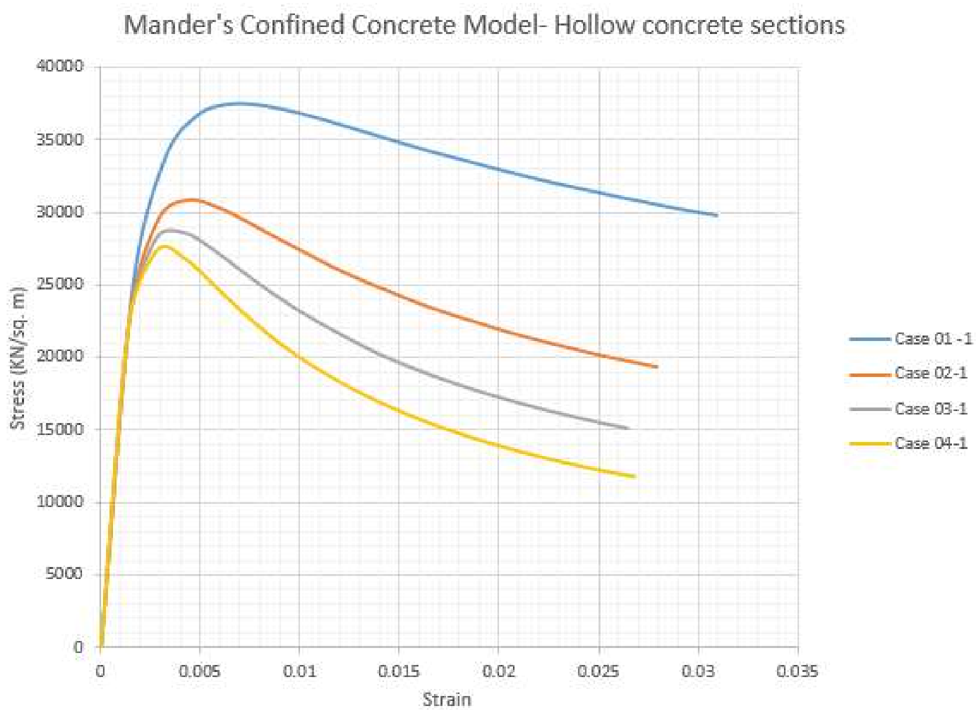



*Figure 3-19: Mander's confined concrete - Hollow sections (Stress in kPa)*

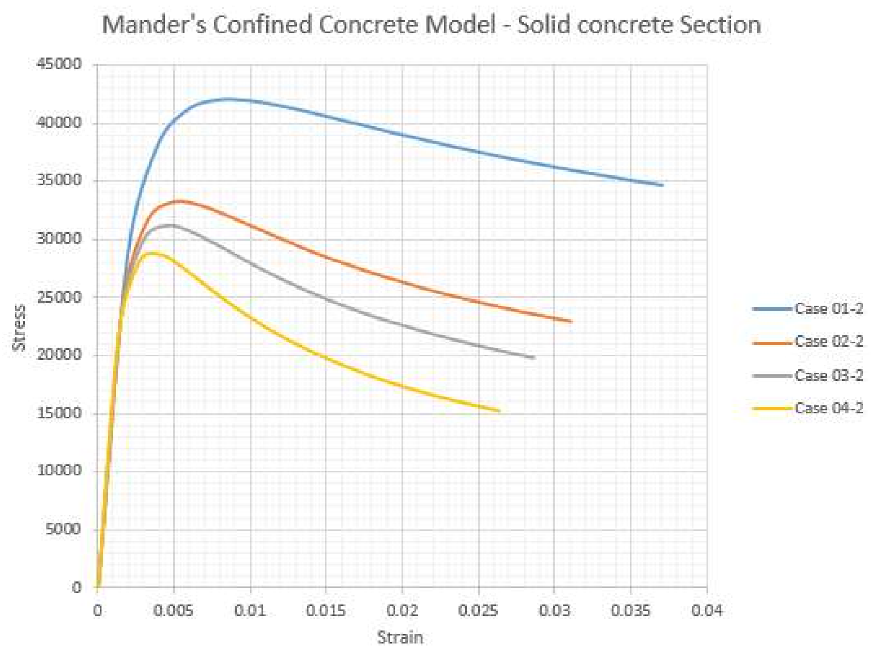



*Figure 3-20: Mander's Confined concrete model- Solid Sections (Stress in KPa)*

In addition to the concrete model, the reinforced steel used in the concrete model has the following physical characteristics.

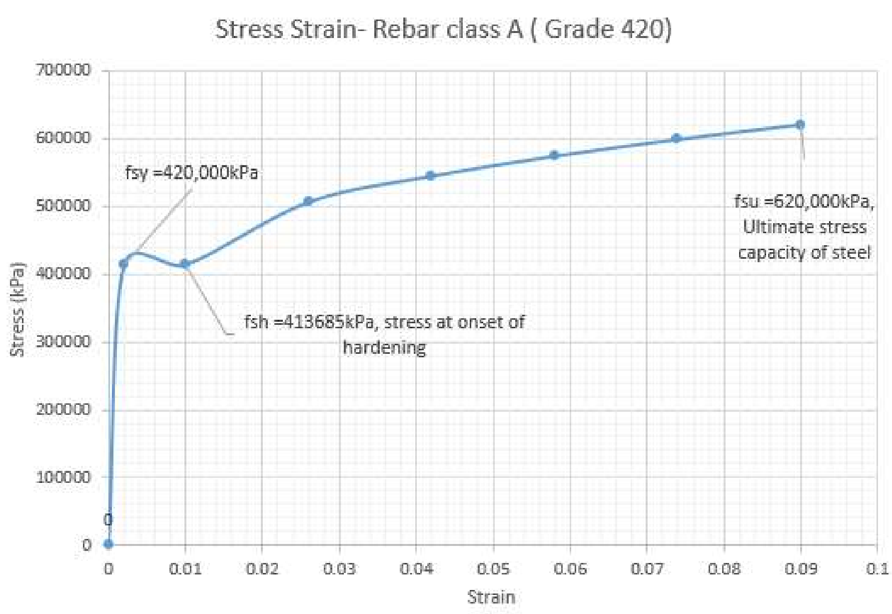



*Figure 3-21: Steel grade 420, stress strain curve*

The formulas used are:

For $\varepsilon_s \leq \varepsilon_{sy}, f_s = E_s\varepsilon_s$

For $\varepsilon_{sy} < \varepsilon_s \leq \varepsilon_{sh}, f_s = f_y$

For $\varepsilon_{sh} < \varepsilon_s \leq \varepsilon_{su}, f_s = f_{sy} + \frac{f_{su} - f_{sy}}{(\varepsilon_{su} - \varepsilon_{sh})^{\frac{1}{2}}}$

Moment – curvature analysis in this thesis is entirely been carried out using SAP 2000 V 18Eval version. The software outputs are verified with the results of an approximate Matlab program, the output is confirmed to be acceptable.(Appendix C)

Moment Curvature Analysis formulas:

- **Onset of Cracking:**

$$M_{cr} = \frac{f_{rc} I_e}{Y};\ I_e \ used\ is\ 70\%\ of\ I_g \quad \text{--------------------------------------3.19}$$

$f_{rc}$ – is strength of the material just before yield ( can be taken as the area under confined Stress – strain curve of concrete up to the yield point)

$$\emptyset_{cr} = \frac{M_{cr}}{E_c I_e} \quad \text{----------------------------------------------3.20}$$

- **Onset of Yield**

$\rho = A_s/A_{ec}$ , where $A_{ec}$ is effective confined concrete area,

$\rho' = A'_s/A_{ec}$

$$k = \sqrt{(\rho + \rho')^2 n^2 + 2\left(\rho + \frac{\rho' d'}{d}\right) n} - (\rho + \rho')n \quad \text{-----------3.21}$$

$$f'_s = \frac{d-d'}{d-kd} f_y \quad \text{--------------------------------------3.22}$$

$$M_y = A_s f_y \left(d - \frac{kd}{3}\right) + A'_s f'_y \left(\frac{kd}{3} - d'\right) \quad \text{-------------------3.23}$$

$$\emptyset_y = \frac{\varepsilon_y}{d-kd} \quad \text{-----------------------------------------3.24}$$

- **Ultimate**

  Iteration needed, assume still in compression will yield first, $\varepsilon'_s > \varepsilon_y$

$$c = \frac{A_y f_y - A'_s f'_s}{0.85 f'_{cu} b \beta_1} \quad \text{----------------------------------------3.25}$$

$$M_u = 0.85 f'_{cu} \beta_1 A_{ec} \left(d - \frac{\beta_1 c}{2}\right) + A'_s f'_s (d - d')$$

$$\emptyset_u = \frac{\varepsilon_u}{c}$$

Idealized M-C curve is derived from the estimated actual moment curvature diagram, by balancing the areas between the actual and the idealized curves beyond the first reinforcing bar yield point. The data used for this plots is attached on Appendix D

**Moment curvature analysis results**
**Case 01-1- 10m Hollow**

Data input:

Axial Compression = 14,570KN,

Reinforcement bar diameter: 32mm

Confining reinforcement bar diameter = 14mm, c-c 140mm

Moment (transverse direction- weak axis)

Cross section dimension: 2300 mm x 1850mm and wall thickness = 400 mm

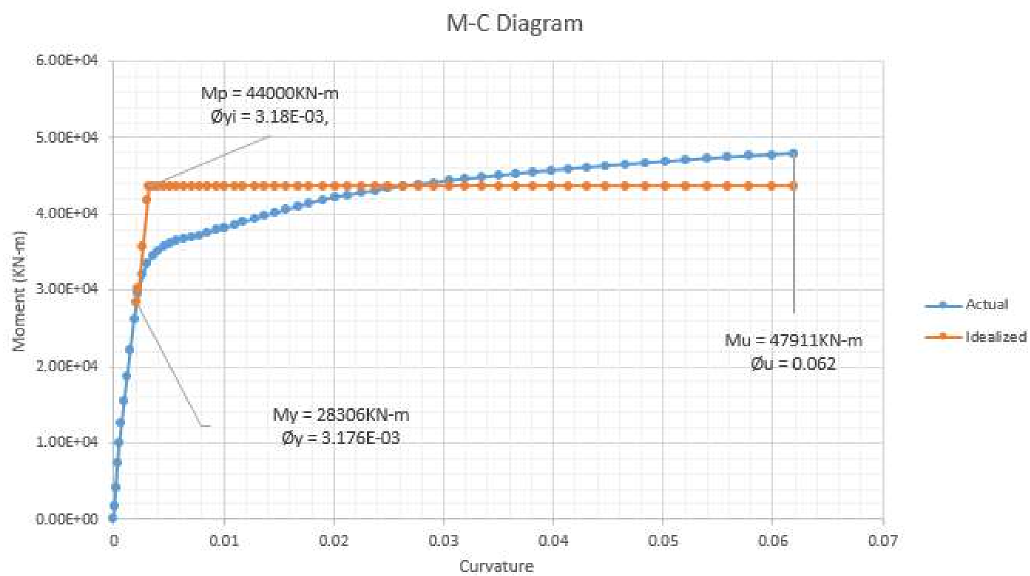



*Figure 3-22: Moment curvature diagram - c1-1*

**Case 01-2- 10m Solid**

Data input:

Axial Compression = 15,493KN,

Reinforcement bar diameter: 32mm

Confining reinforcement bar diameter = 14mm, c-c 150mm

Moment (transverse direction- weak axis)

Cross section dimension: 1800 mm x 1600mm

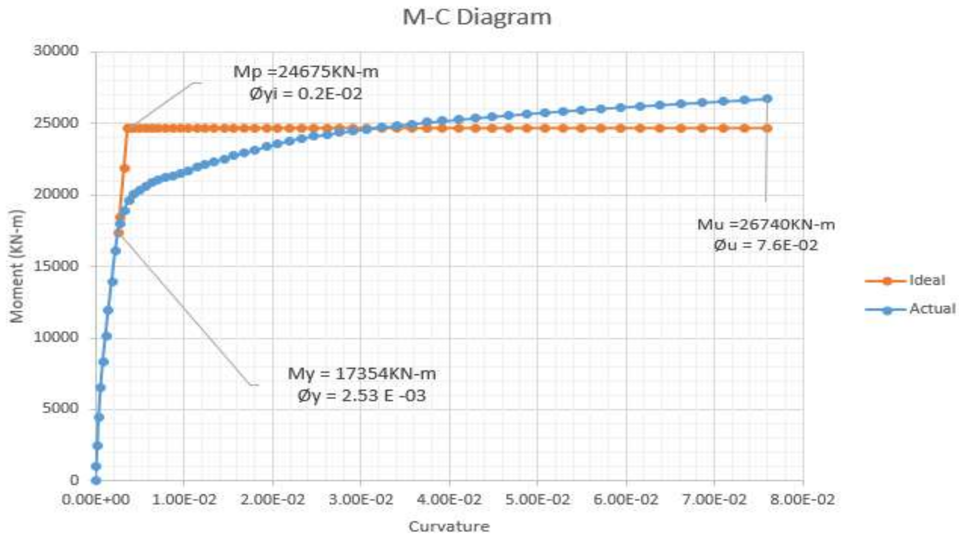



*Figure 3-23: Moment curvature diagram - c1-2*

**Case 02-1- 15m Hollow**

Data input:

Axial Compression = 15,610KN,

Reinforcement bar diameter: 32mm

Confining reinforcement bar diameter = 14mm, c-c 223mm

Moment (transverse direction- weak axis)

Cross section dimension: 2400 mm x 2000mm and wall thickness = 400 mm

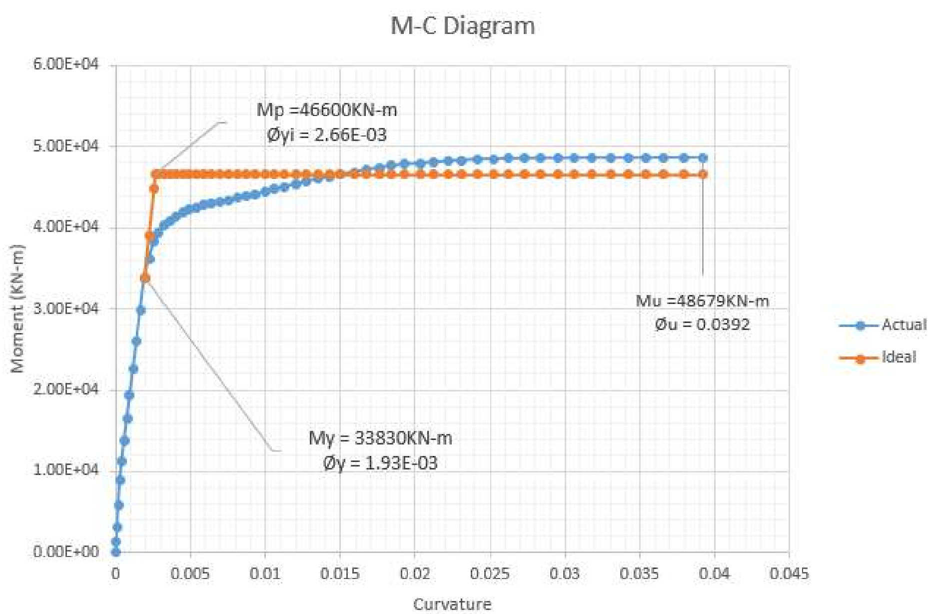



*Figure 3-24: Moment curvature diagram - c2-1*

**Case 02-2- 15m Solid**

Data input:

Axial Compression = 15,881KN

Reinforcement bar diameter: 32mm

Confining reinforcement bar diameter = 14mm, c-c 125mm

Moment (transverse direction- weak axis)

Cross section dimension: 1850 mm x 1600mm

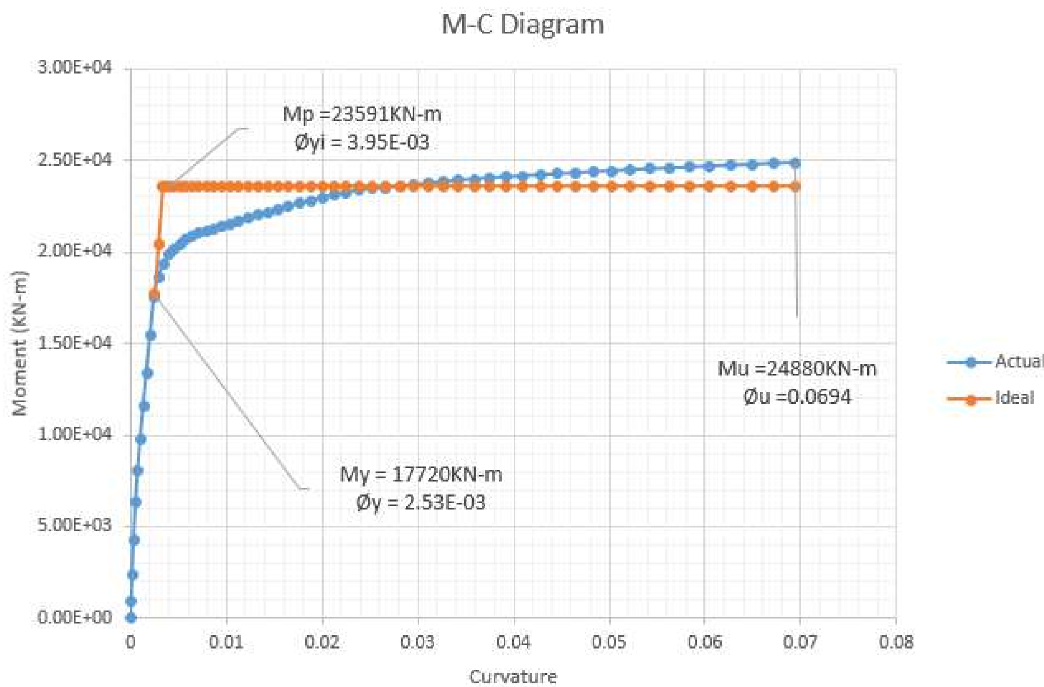



*Figure 3-25: Moment curvature diagram - c2-2*

**Case 03-1- 20m Hollow**

Data input:

Axial Compression = 15,941KN,

Reinforcement bar diameter: 32mm

Confining reinforcement bar diameter = 14mm, c-c 280mm

Moment (transverse direction- weak axis)

Cross section dimension: 2450 mm x 2050mm and wall thickness = 400 mm

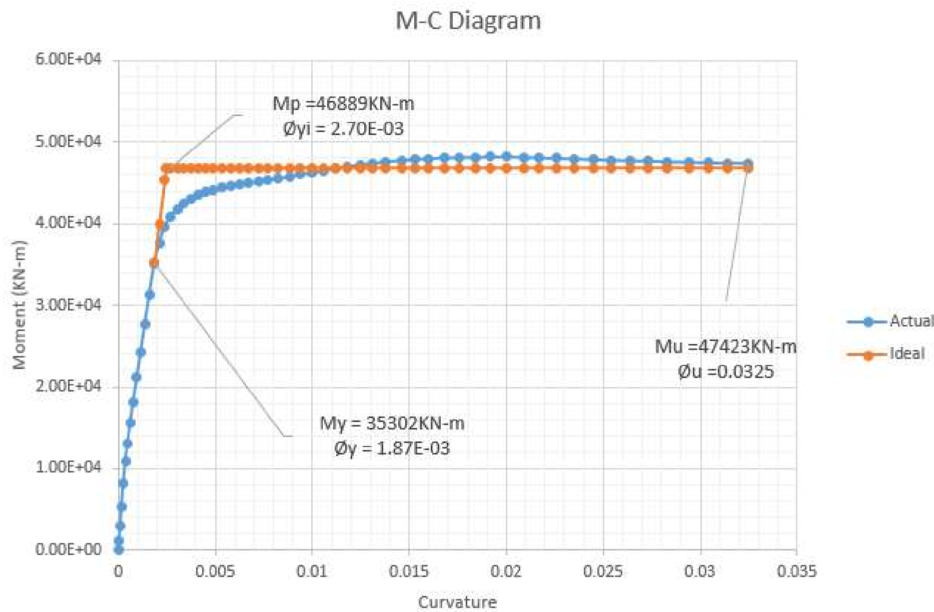



*Figure 3-26: Moment curvature diagram - c3-1*

**Case 03-2- 20m Solid**

Data input:

Axial Compression = 16,896KN

Reinforcement bar diameter: 32mm

Confining reinforcement bar diameter = 14mm, c-c 210mm

Moment (transverse direction- weak axis)

Cross section dimension: 2250 mm x 1900mm

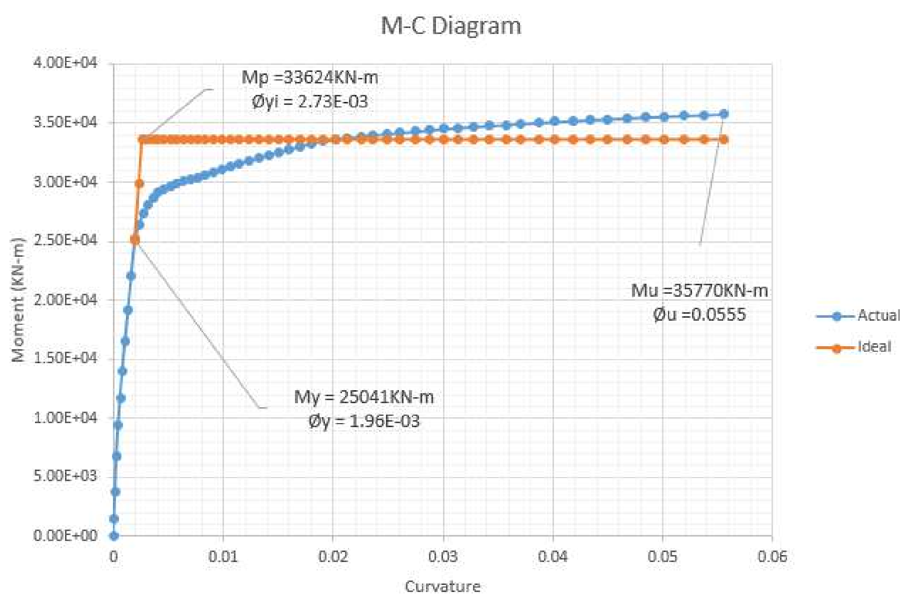



*Figure 3-27: Moment curvature diagram - c3-2*

**Case 04-1- 25m Hollow**

Data input:

Axial Compression = 16,325KN,

Reinforcement bar diameter: 32mm

Confining reinforcement bar diameter = 14mm, c-c 330mm

Moment (transverse direction- weak axis)

Cross section dimension: 2550 mm x 2050mm and wall thickness = 400 mm

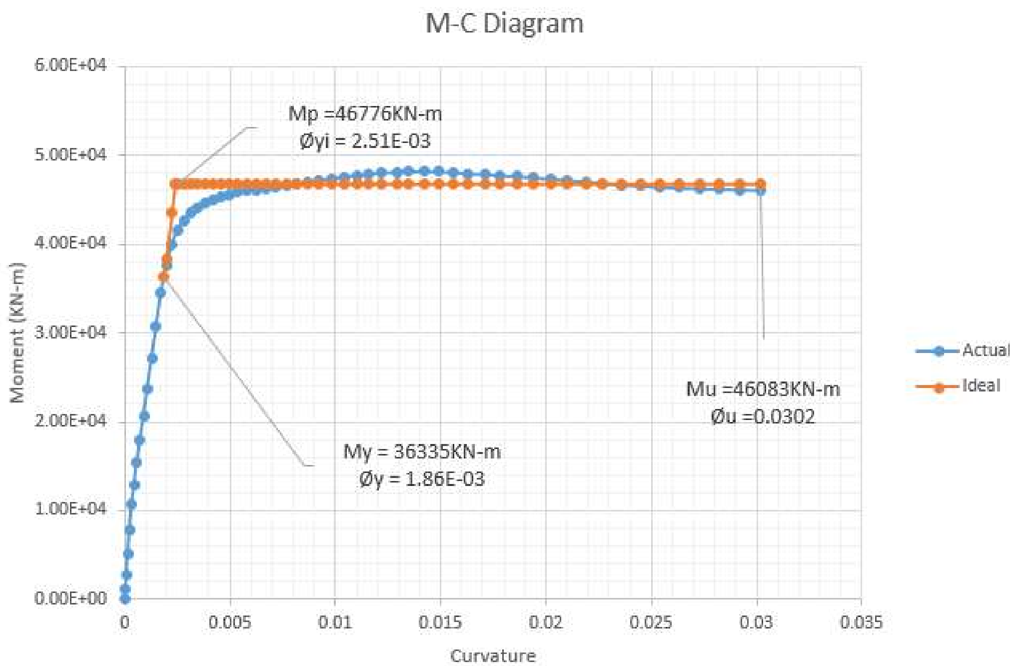



*Figure 3-28: Moment curvature diagram - c4-1*

**Case 04-2- 25m Solid**

Data input:

Axial Compression = 17,879KN

Reinforcement bar diameter: 32mm

Confining reinforcement bar diameter = 20mm, c-c 280mm

Moment (transverse direction- weak axis)

Cross section dimension: 2500 mm x 2000mm

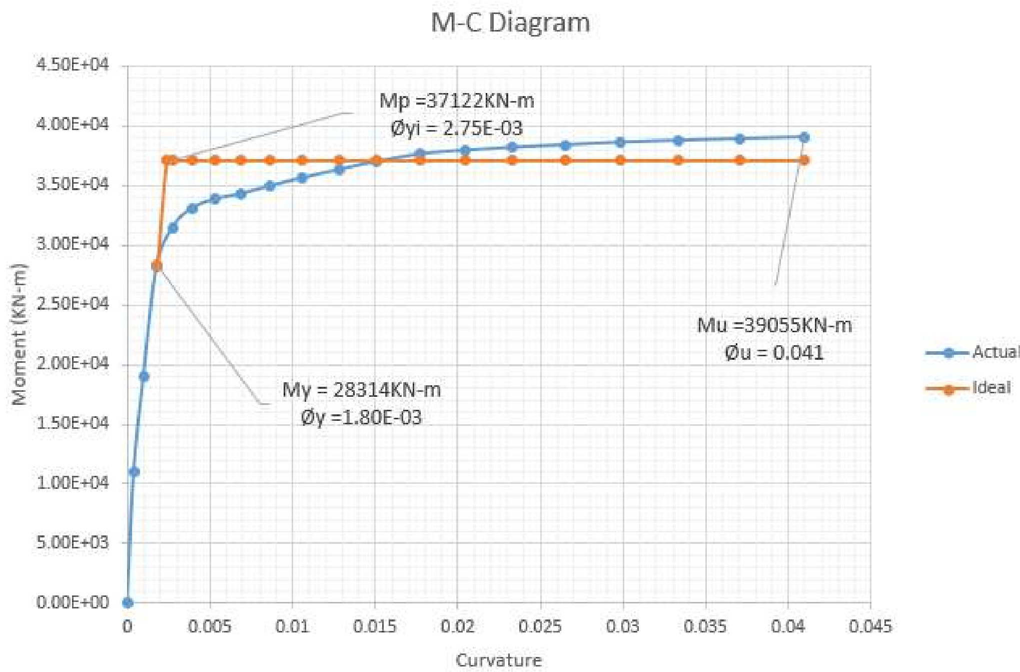



*Figure 3-29: Moment curvature diagram - c4-2*

**Ductility Calculation:**

According to Caltrans 1.7 Seismic Design Criteria,

$\Delta_{pd} = \theta_p \times \left(L - \frac{L_p}{2}\right)$, where - -----3.26

$\Delta_{pd}$ is idealized plastic displacement capacity due to rotation of the plastic hinge

$\theta_p$ is Plastic rotation capacity in radians = $L_p \times \emptyset_p$

$L$ is the length between max. moment and the point of contra-flexure (total height for this thesis, assuming cantilever column)

$L_p$ is an equivalent analytical plastic hinge length defined with:

$L_p = 0.08L + 0.022 f_{ye} \geq 0.044 f_{ye} d_{bl}$ (Considering all of the columns in this thesis has their plastic hinge at the end and are all supported by footings.

$f_{ye}$ is expected yield strength of longitudinal reinforcement steel, according to the graph (figure 3-21) (strength at the onset of hardening ) = 414MPa

$d_{bl}$ is nominal longitudinal reinforcement bar diameter ( 32mm)

$\emptyset_p$ is idealized plastic curvature capacity, is calculated as = $\emptyset_u - \emptyset_y$

$\Delta_{yi} = \frac{L^2}{3} \times \emptyset_y$ , ------ ---------3.27

$\Delta_{yi}$ Is idealized yield displacement of the column at the formation of the plastic hinge.

Following the calculation of components, ductility is calculated using equation 3.28

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_{yi}} \text{ ---------------------------} \qquad 3.28$$

Final results for each case is tabulated in table 3.67

*Table 3-66: : Ductility Calculation Chart*

| | Hollow | | | | Solid | | | |
|---|---|---|---|---|---|---|---|---|
| **Case** | **Case 01-1** | **Case 02-1** | **Case 03-1** | **Case 04 -1** | **Case 01-2** | **Case 02-2** | **Case 03-2** | **Case 04-2** |
| **Height (mm)** | 10000 | 15000 | 20000 | 25000 | 10000 | 15000 | 20000 | 25000 |
| **Idealized yield curvature $\phi_{yi}$** | 0.00318 | 0.00266 | 0.0027 | 0.00251 | 0.002 | 0.00395 | 0.00273 | 0.00275 |
| **Ultimate curvature $\phi_u$** | 0.062 | 0.0392 | 0.0325 | 0.0302 | 0.00253 | 0.0694 | 0.0555 | 0.041 |
| **Yield curvature $\phi_y$** | 0.003176 | 0.00193 | 0.00187 | 0.00186 | 0.00253 | 0.00253 | 0.00196 | 0.0018 |
| **Idealized plastic curvature capacity $\phi_p$** | 0.058824 | 0.03727 | 0.03063 | 0.02834 | 0.07347 | 0.06687 | 0.05354 | 0.0392 |
| **Nominal reinforcement diameter. (mm)** | 32 | 32 | 32 | 32 | 32 | 32 | 32 | 32 |
| **expected yield strength $f_{ye}$ (MPa)** | 420 | 420 | 420 | 420 | 420 | 420 | 420 | 420 |
| **Equivalent plastic hinge length $L_p$ (mm)** | 1095.68 | 1495.68 | 1895.68 | 2295.68 | 1095.68 | 1495.68 | 1895.68 | 2295.68 |
| **Plastic rotation Capacity $\theta_p$ (radians)** | 64.45228 | 55.74399 | 58.06468 | 65.05957 | 80.49961 | 100.0161 | 101.4947 | 89.99066 |
| **Idealized Plastic displacement capacity ($\Delta_p$) mm** | 609213.3 | 794472.3 | 1106258 | 1551811 | 760895.2 | 1425446 | 1933693 | 2146472 |
| **Idealized Yield displacement ($\Delta_y$) mm** | 105866.7 | 144750 | 249333.3 | 387500 | 84333.33 | 189750 | 261333.3 | 375000 |
| **Ductility ratio $\mu_D$** | **6.75** | **6.49** | **5.43** | **5.00** | **10.00** | **8.51** | **8.40** | **6.72** |
| **Target maximum ductility on pier (Weak direction)** | **5** | **5** | **5** | **5** | **5** | **5** | **5** | **5** |

## 3.7 Heat evolution and early stage crack

To check if the extent of remedial measure to be taken in order to reduce early age cracking generated from the temperature variation between the core center and the construction environment, the FEA is conducted using ANSYS Steady state thermal software. For the analysis concrete itself is considered as a heat generating material with the following rate of temperature evolution with time

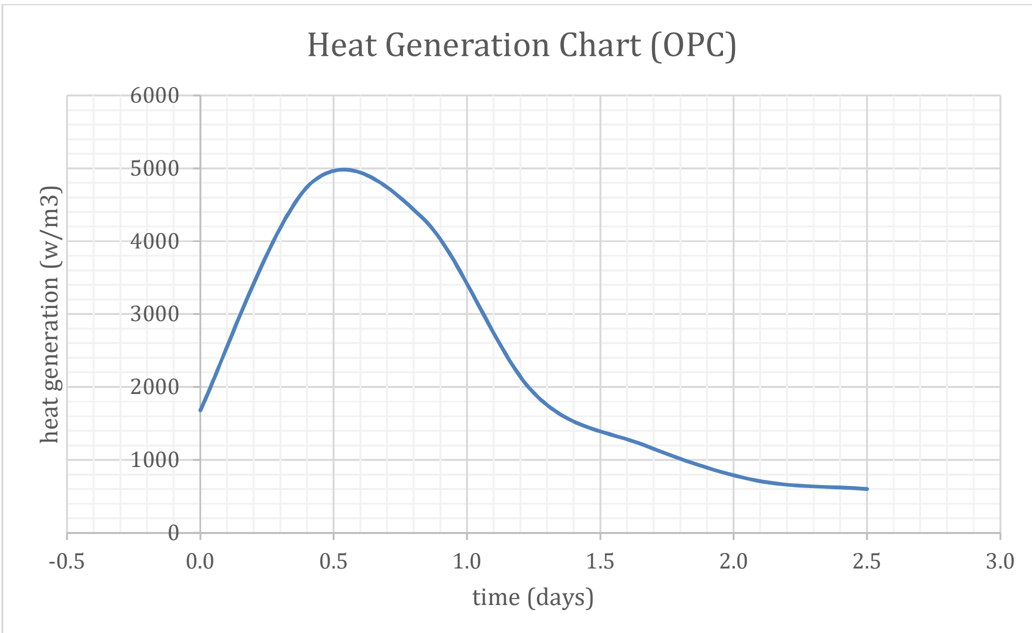



*Figure 3-30: Heat evolution Chart*

The Analysis considered the heat variation between the environment (22 °C) and the core section. For hollow section, the internal surface temperature is also considered to be a room temperature.

The geometry modeled for all of column sections are subjected to this temperature variation and the maximum difference obtained through time is noted for further analysis. The modeled concrete has the following physical properties.

Coefficient of thermal expansion = 1.0 x $10^{-5}$/° C

Poisson's ratio = 0.2

Isotropic thermal conductivity = 0.72W/m° C

Specific heat= 780J/Kg ° C

Obtained results of sample sections (10 and 25 meters hollow and solid) are presented below. Sample finite element design software output is attached on appendix E

**Case 01-1: 10m Hollow (2300mm x 1850mm wall thickness =400mm)**

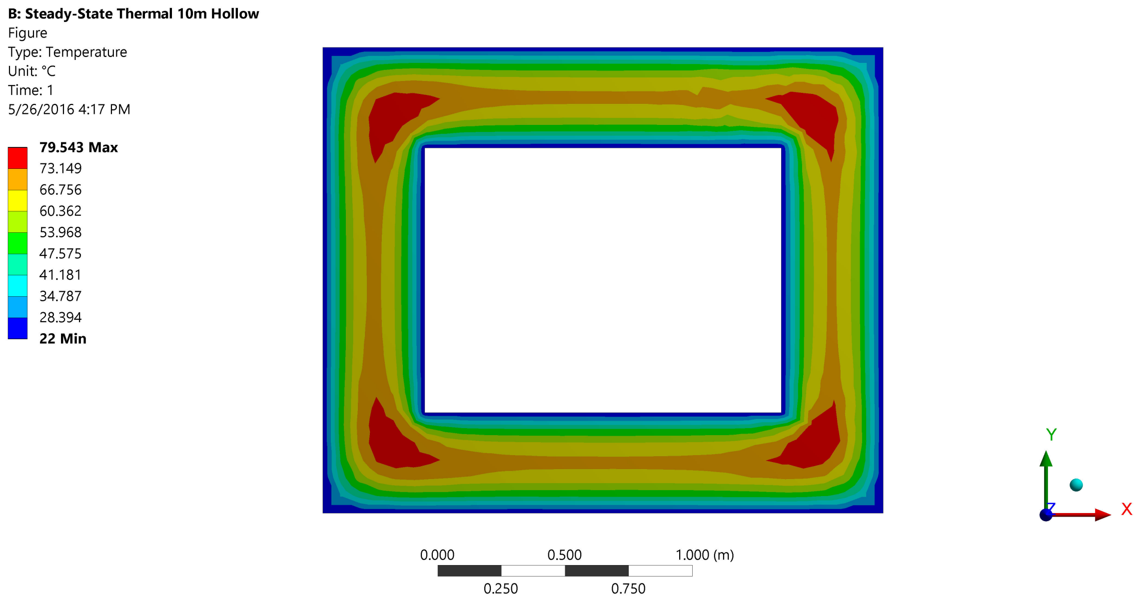



*Figure 3-31: Temperature gradient 10m solid*

Max = 79.15°C, Min = 22 °C

**Case 01-2: 10m Solid (1800mm x 1600mm)**

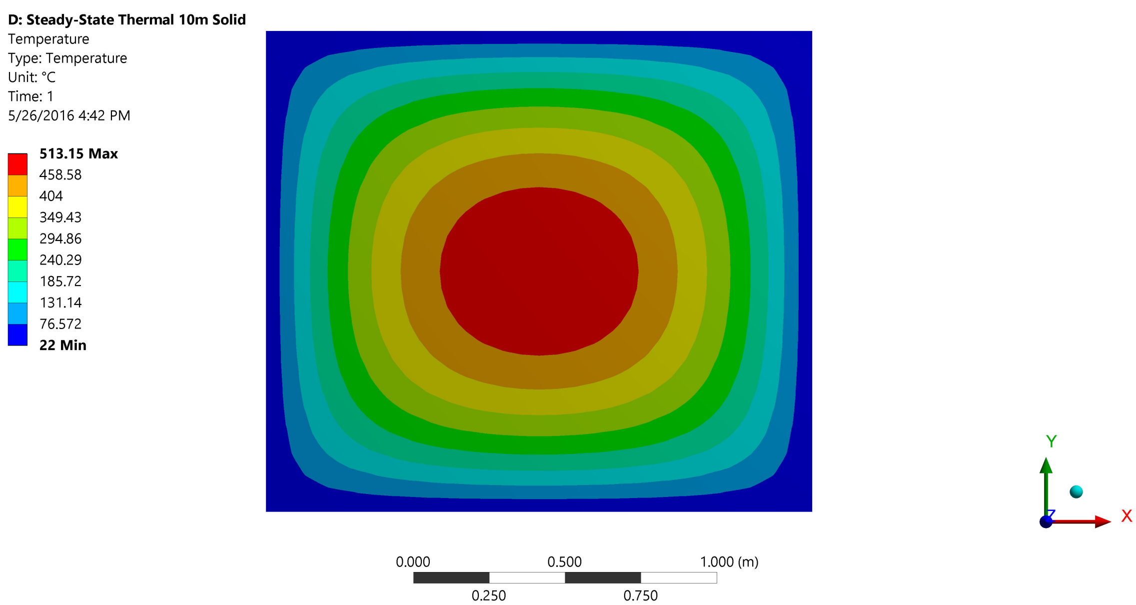



*Figure 3-32: Temperature gradient 10m hollow*

Max = 513.58 °C, Min = 22 °C

**Case 02-1: 15m Hollow (2400mm x 2000mm wall thickness =400mm)**

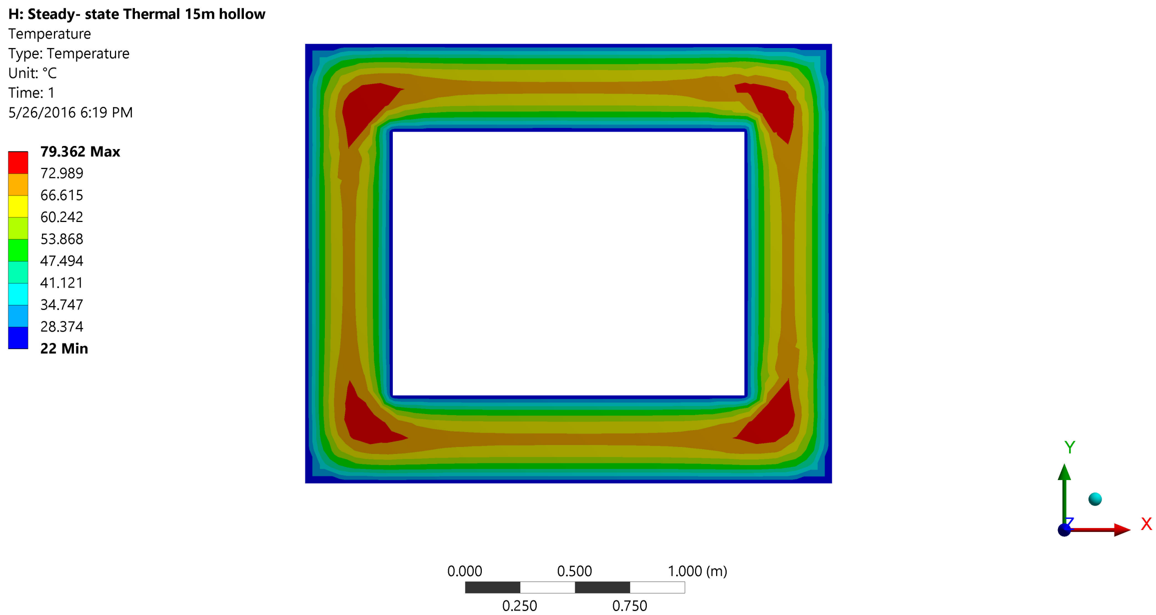



*Figure 3-33: Temperature gradient 25m hollow*

Max = 79.362 °C
Min = 22 °C

### Case 02-2: 15m Solid (1850mm x 1600mm)

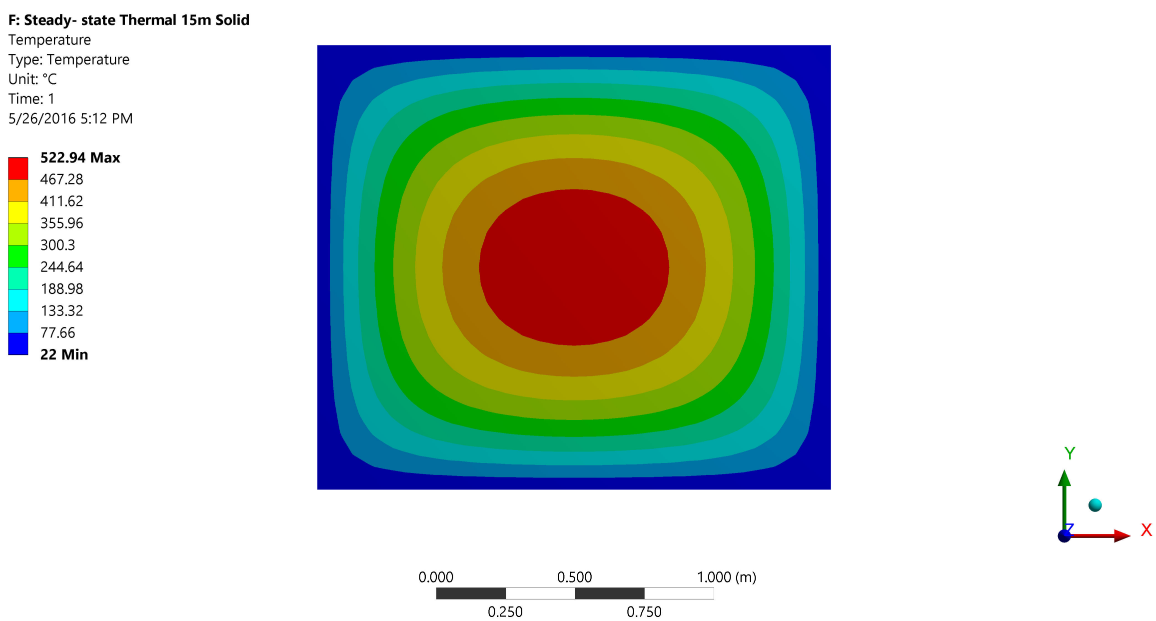



*Figure 3-34: Temperature gradient 25m Solid*

Max = 522.28 °C
Min = 22 °C

### Case 03-1: 20m Hollow (2450mm x 2050mm wall thickness =400mm)

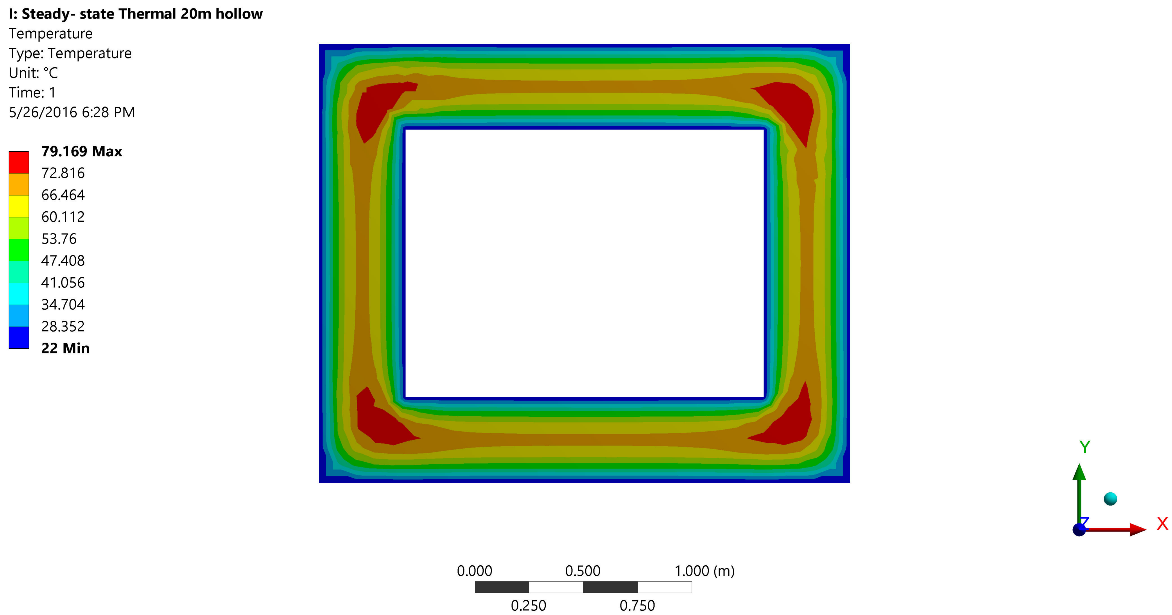



*Figure 3-35: Temperature gradient 10m solid*

Max = 79.169°C
Min = 22 °C

**Case 03-2: 20m Solid (2250mm x 1900mm)**

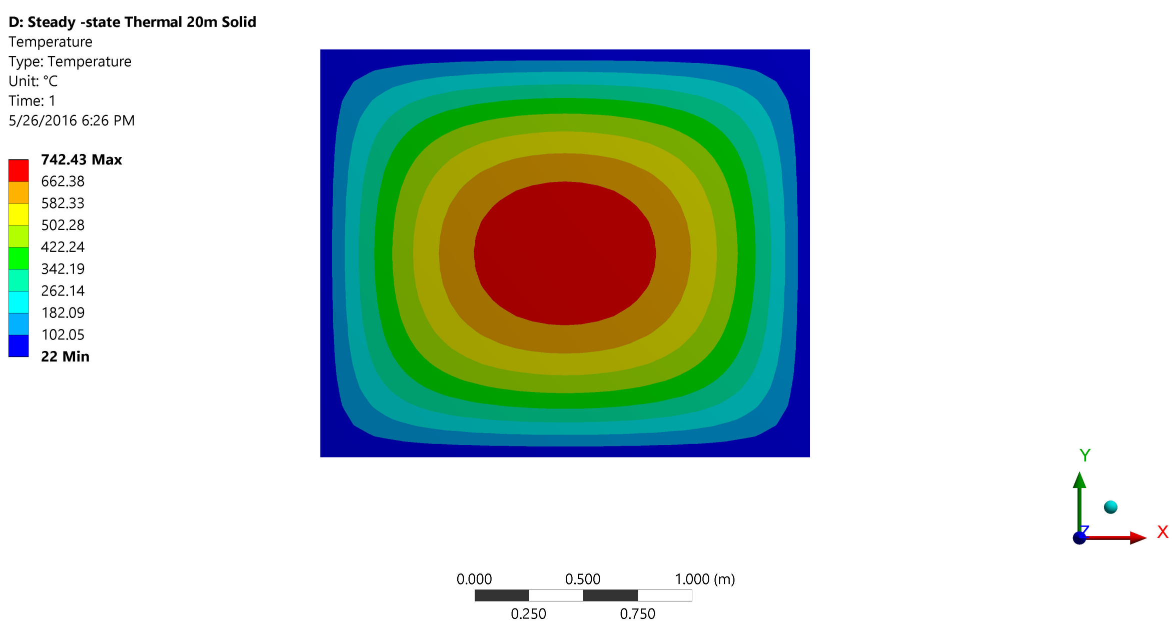



*Figure 3-36: Temperature gradient 10m hollow*

Max = 742.43 °C
Min = 22 °C

**Case 04-1: 25m Hollow (2550mm x 2050mm wall thickness =400mm)**

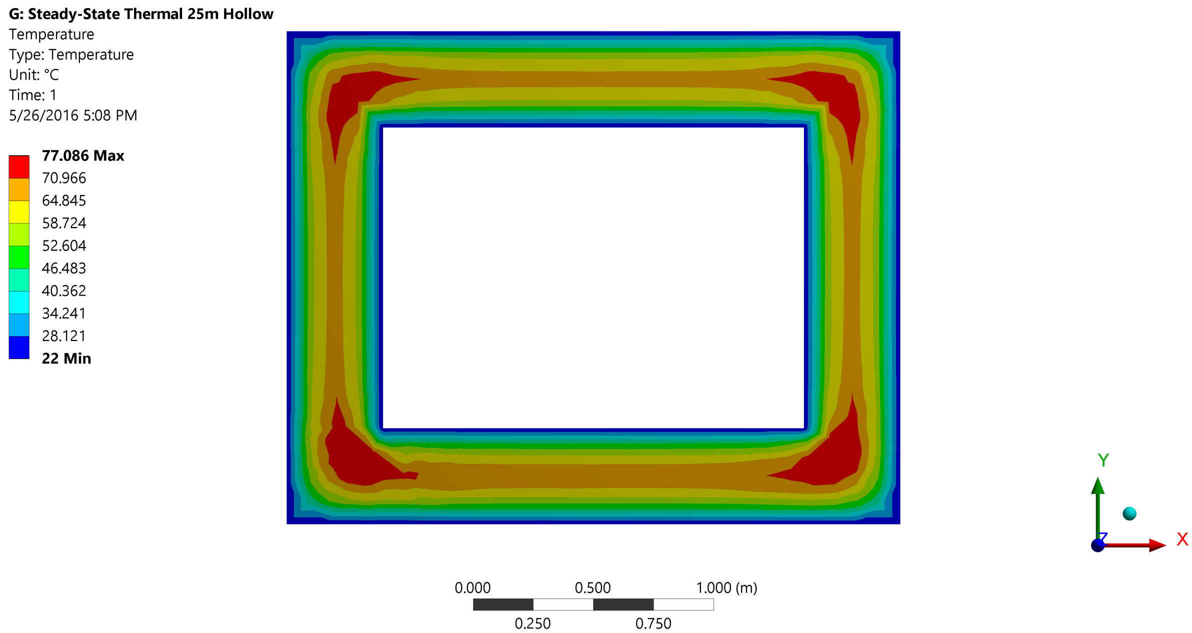



*Figure 3-37: Temperature gradient 25m hollow*

Max = 77.086 °C
Min = 22 °C

## Case 04-2: 25m Solid (2500mm x 2000mm)

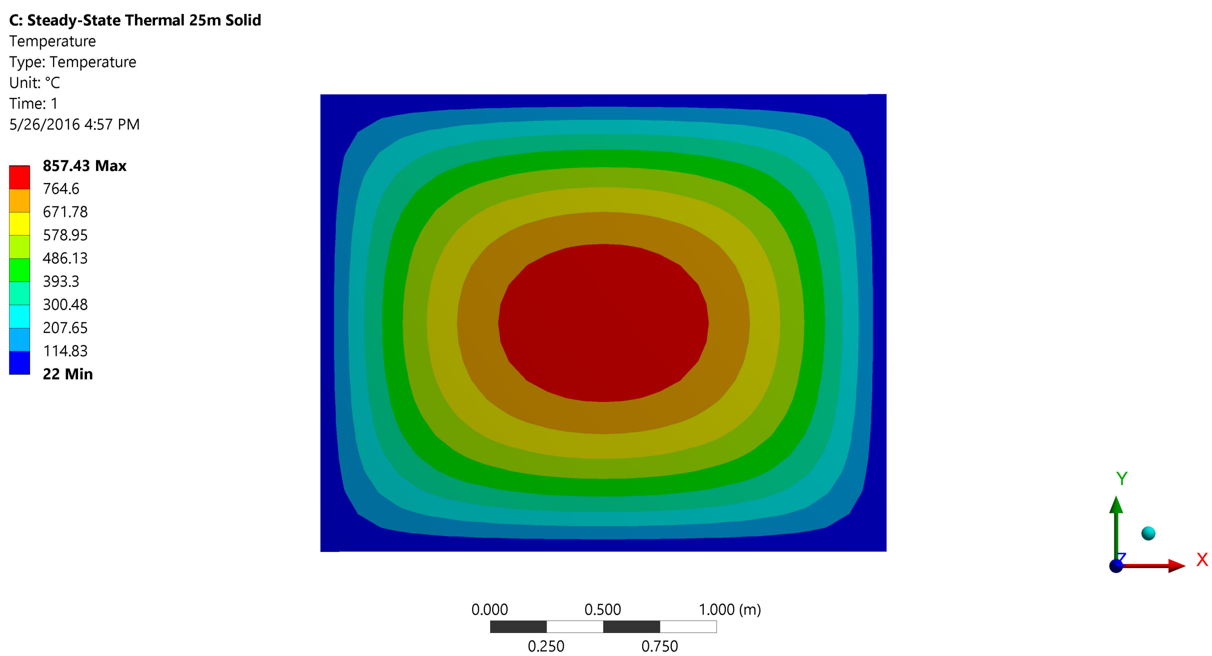



*Figure 3-38: Temperature gradient 25m Solid*

Max = 857.43 °C
Min = 22 °C

The analysis result shows that as the height of the column increases the thermal stress will increase for solid sections, early crack prevention measures has to be made to secure structural safety. On the other hand, hollow section has an advantage because the probability of early stage crack occurrence is lower since the temperature variation is constant with increasing height.

# 4. Structural Efficiency and Economy comparison

## 4.1 Introduction

Depending on the analysis and design of 8 separate pier columns, it has become evident that Hollow concrete column has advantages both structurally and economically. For long columns, slenderness effect plays an important role of demanding more relaxed cross sectional dimensions, which resulted additional self-weight and concrete consumption.

Ductility property of the section is also examined using the moment curvature diagrams of each section. Moment curvature diagrams are developed towards the weak axis of the section where the seismic effect can be severe. Plastic hinge moments are compared to examine the ultimate capacity of the sections. Idealized [21] moment curvature diagrams are developed to make the analysis available for further study and nonlinear analysis implementation.

Mandar's confined concrete section is assumed for all of the cases analyzed. The mechanical property of reinforcement steel consumed is also discussed. In this portion of this thesis, I will discuss the results and conclude the outcomes in the last chapter. Future study recommendations will also be made.

## 4.2 Comparison of Mechanical Property

### 4.2.1 Ductility

Using AASHTO guide and Caltrans Seismic design criteria specifications for LRFD bridge design, the ductility requirement is analyzed and presented in table 3-69. As it is observed, the first two cases both hollow and solid sections do not qualify AASHTO LRFD seismic criteria maximum ductility demand. But hollow sections satisfied the ductility criteria for piers of 20m and above tall.

Solid sections designed in this thesis has evidently proved that they are brittle for all of the heights and an over strength factor shall be applied on the calculated plastic moment capacity to enhance the section's moment distributions capacity at the plastic hinge. The ductility comparison is graphically presented on table 5-1

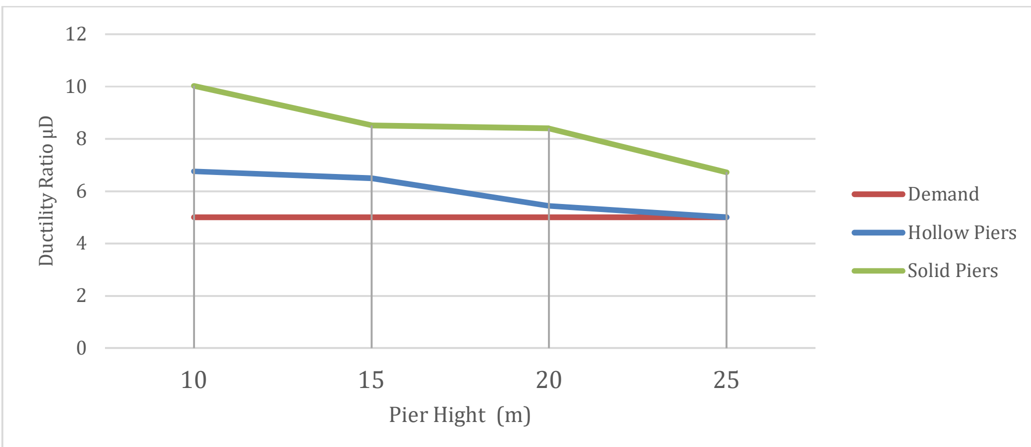



*Figure 4-1: Ductility comparision chart*

From the calculated result, it is evident that height of the pier is inversely proportional to the ductility ratio, to below figure 5-2, shows the probable height that solid core pier can satisfy ductility demand for piers with greater of equal to 35meters height.

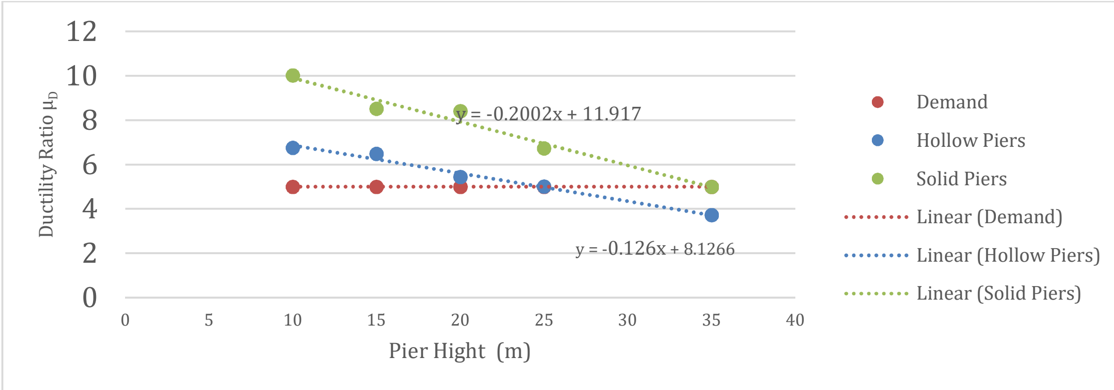



*Figure 4-2: Ductility ratio forecast*

Linear forecast given on the figure above is only to show the tendency of decrement as the pier height increase. This extrapolation does not assure the linearity property and might change due to the expected slenderness and ductility variation. Designers shall consider changing pier geometry or utilization of multi column piers.

### 4.2.2 Moment carrying capacity

From the analysis results it is observed that the hollow concrete section is more ductile and carries more moment before the plastic hinge is formed. Plastic moment capacity of each section is obtained from moment – curvature calculation. Even though over strength factor should be applied on members which are not satisfying the ductility demand, it is helpful to see the comparison between the weak axis plastic moment characteristics of conventionally designed sections. The summarized data is presented below on table 5.1 and figure 5-3.

*Table 4-1: Capacity Moment Comparison Chart*

| Case | 01-1 | 01-2 | 02-1 | 02-2 | 03-1 | 03-2 | 04-1 | 04-2 |
|---|---|---|---|---|---|---|---|---|
| Plastic moment capacity – KN-m | 44000 | 24675 | 46600 | 23591 | 46776 | 33624 | 46889 | 37122 |
| yield moment capacity - KN-m | 28306 | 17354 | 33830 | 17720 | 35302 | 25041 | 36335 | 28314 |
| Ultimate Moment capacity KN-m | 47911 | 26740 | 48679 | 24880 | 47423 | 35770 | 46083 | 39055 |
| % Increase of Plastic moment carrying capacity of Hollow Pier over solid pier | 43.9 | | 49.4 | | 28.1 | | 20.8 | |
| % Increase of Yield moment carrying capacity of Hollow Pier over solid pier | 38.7 | | 47.6 | | 29.1 | | 22.1 | |
| % Increase of Ultimate moment carrying capacity of Hollow Pier over solid pier | 44.2 | | 48.9 | | 24.6 | | 15.3 | |

Capacity analysis has verified the plastic moment capacity of the hollow section is in average 35.5% greater than the equivalent height solid section designed against similar load conditions. Concluding from table 5-1, the percentage increase in moment carrying capacity decreases with height, it can be generalized that for taller piers, Solid sections have higher moment carrying capacity. Depending on graphic linear forecasts shown on diagrams 5-4, 5-5 and 5-6, solid sections taller than 56meter, designed for similar load intensity has greater moment capacity than hollow equivalent piers.

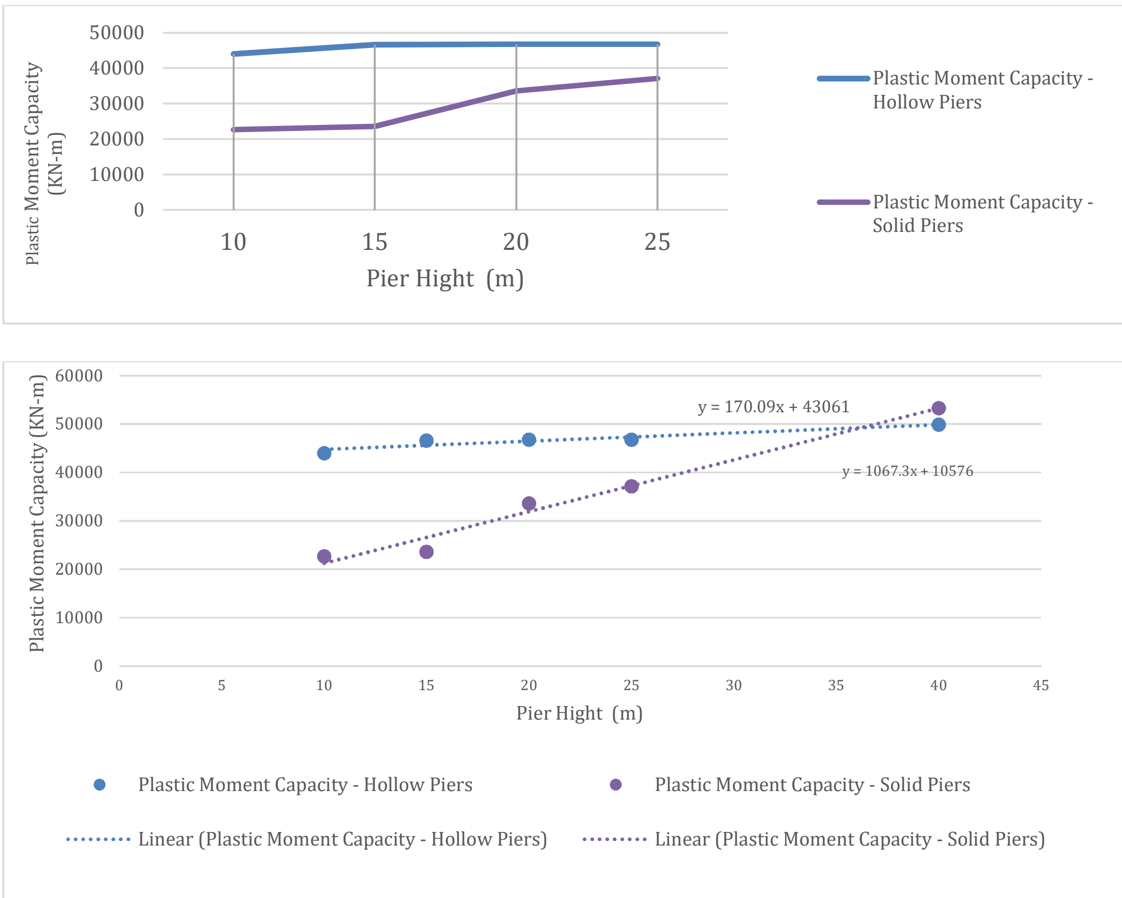
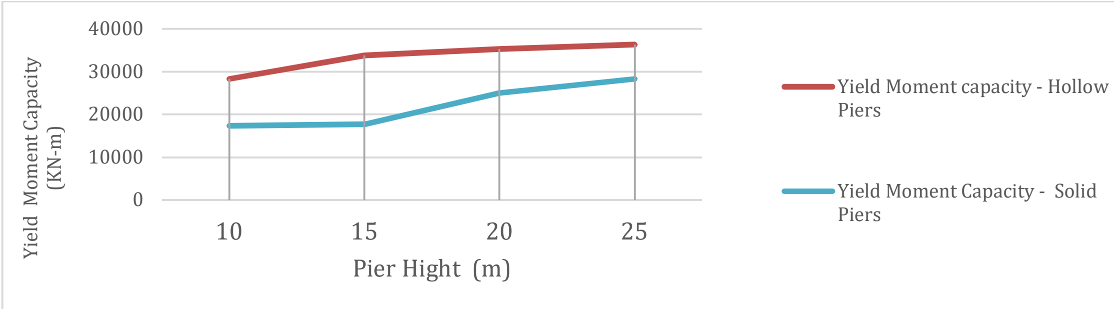



*Figure 4-4: Plastic Moment Capacity*



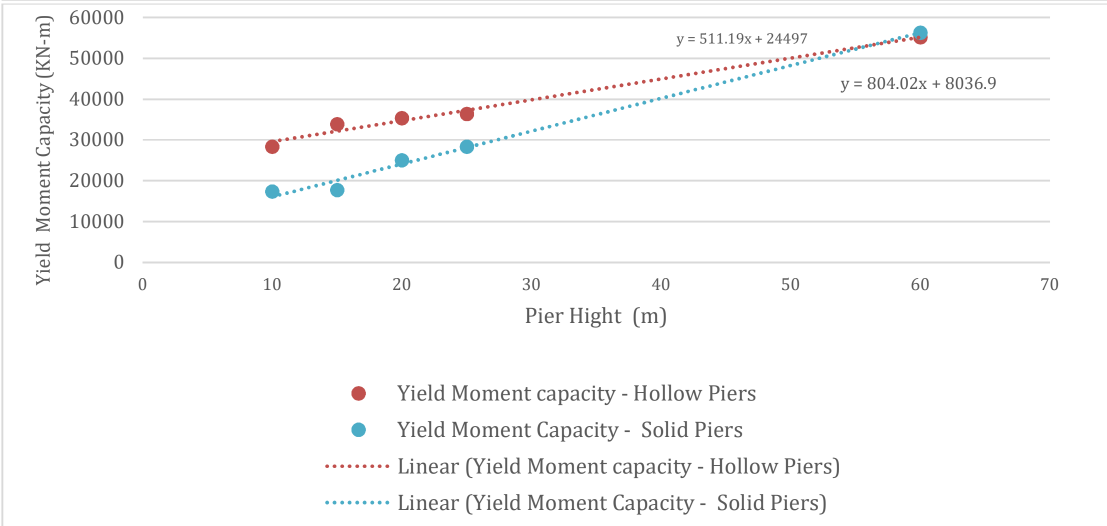
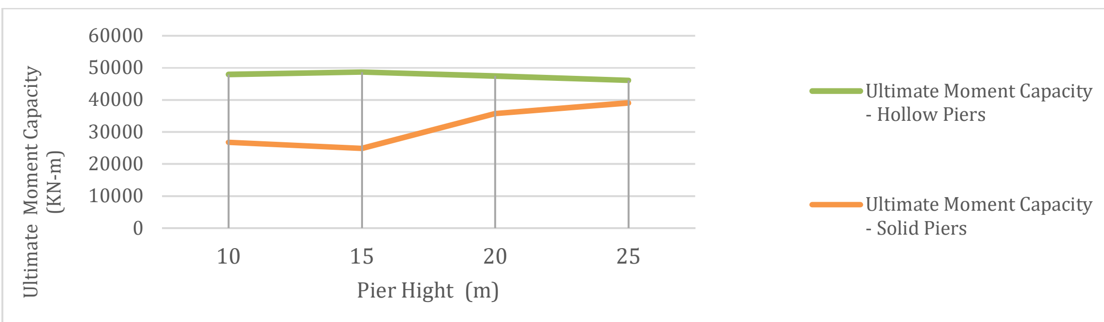




*Table 4-2: Yield Moment Capacity*



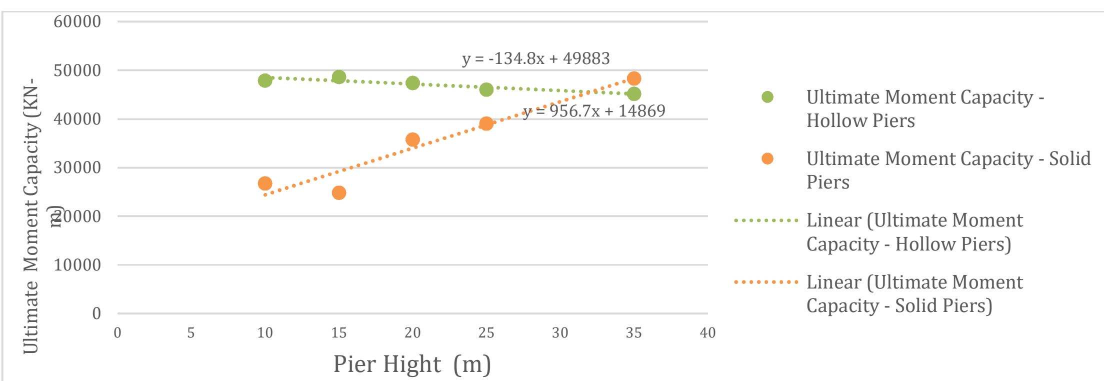




*Table 4-3: Ultimate Moment Capacity*

## 4.3 Economic Comparison

Economically, it is obvious that hollow concrete compressive members have less concrete consumption. But this thesis has also verified that the reinforcement bar quantity proposed for hollow piers is almost as double as the amount of reinforcement quantity required in solid members to support the same amount of design actions.

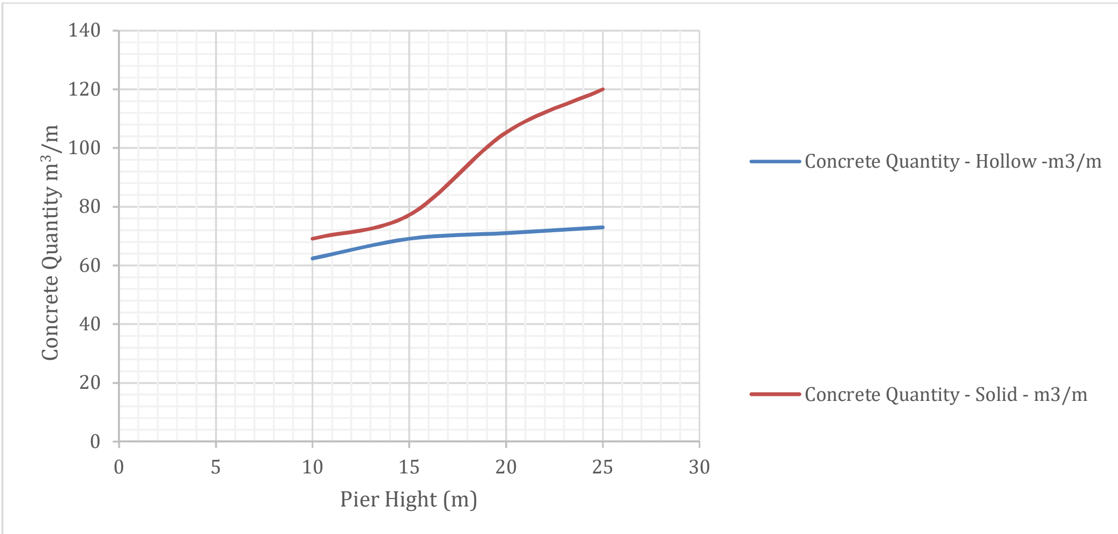



*Figure 4-5: Material comparison chart – Concrete quantity*

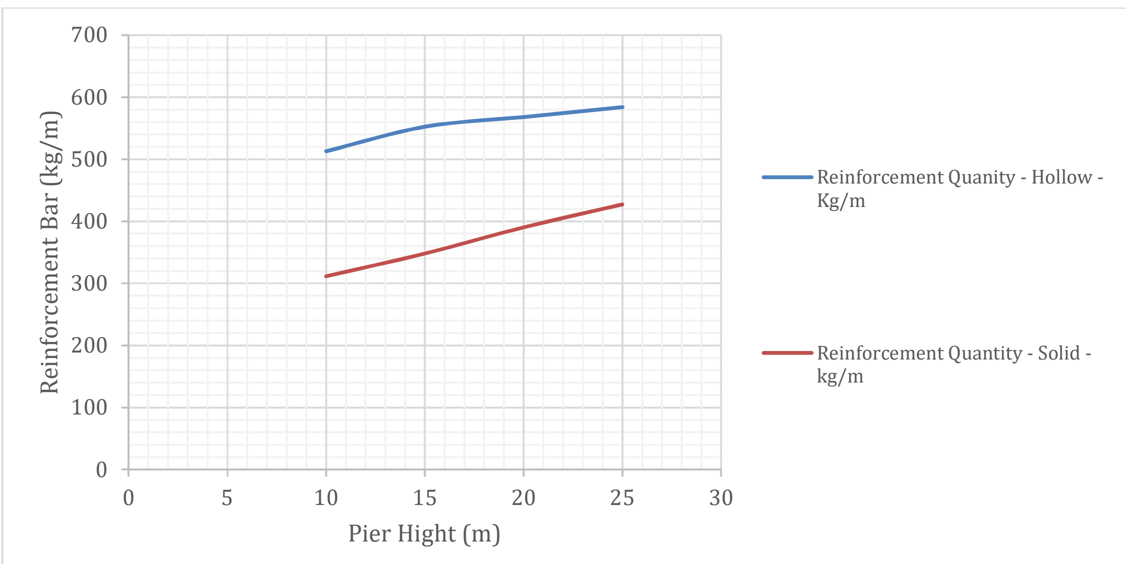



*Figure 4-6: Material comparison chart – Reinforcement quantity*

For ease comparison, costs of materials are adopted from ***http://con.2merkato.com/material_prices*** website, the prices tagged are updated on May 25th 2016.

*Table 4-4: Price Chart*

| | Diameter | Price (birr) | Unit | Remark |
|---|---|---|---|---|
| **Re Bar** | 32 mm | 25.82 | kg | Grade 60 |
| **Re Bar** | 14 mm | 19.28 | kg | Grade 40 |
| **Cement** | | 280 | 1qt | 100kgs, or 2 bags |
| **Sand** | | 540 | $m^3$ | In Addis Ababa, Hauling distance 5kms |
| **Gravel** | | 450 | $m^3$ | In Addis Ababa, Hauling distance 5kms |

*Table 4-5: Concrete price per cubic meter*

| Ratio by volume | Cement (birr) | Sand (birr) | Gravel (birr) | Total (birr) |
|---|---|---|---|---|
| 1:02:03 | 938 | 270 | 333 | 1541 |
| 1:1:5:3 | 1022 | 221.4 | 369 | 1612.4 |
| 1:02:02 | 1134 | 324 | 270 | 1728 |
| 1:1:5:2 | 1260 | 270 | 301.5 | 1831.5 |
| 1:01:02 | 1414 | 199.8 | 337.5 | 1951.3 |

Excluding manpower and machinery costs (assuming the both hollow and solid pier have approximately the same manpower and machinery costs), material consumption comparison cart is developed.

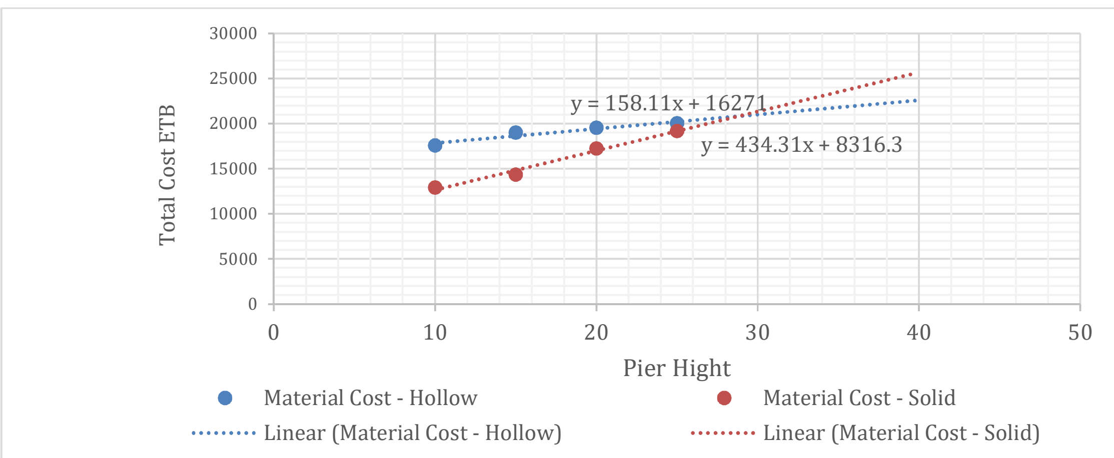



*Figure 4-7: Total Material Cost per meter length*

*Table 4-6: Material Cost Chart*

| Case | 01-1 | 01-2 | 02-1 | 02-2 | 03-1 | 03-2 | 04-1 | 04-2 |
|---|---|---|---|---|---|---|---|---|
| Concrete cost (ETB)/m | 4192.24 | 4643.712 | 4643.712 | 5187.897 | 4772.704 | 7074.405 | 4901.696 | 8062 |
| Re bar cost (ETB)/m | 13244.61 | 8041.854 | 14263.42 | 8988.4449 | 14670.95 | 10079.49 | 15078.48 | 11030.38 |
| Shear Reinforcement (ETB/m) | 145.2623 | 213.5823 | 97.10488 | 170.16056 | 77.47807 | 98.22485 | 64.85262 | 76.7183 |
| Total (birr) | 17582.11 | 12899.15 | 19004.24 | 14346.502 | 19521.13 | 17252.12 | 20045.02 | 19169.1 |

The amount of longitudinal reinforcement in hollow section has played the main role in escalating total material cost of the section. This is due to relatively low compressive strength of hollow piers. Satisfaction of axial compression demand in hollow sections required provisions of higher reinforcement steel. This study has verified that all of elements considered in this thesis has high compression resistance demand. According to figure5 -15 and 5-13, capacity vs demand ratio of axial compression is lower in most of the cases than flexural requirements along both of the axis.

Forecasting cost vs height data, regardless of additional factors (slenderness and foundation requirement), hollow pier application has a probability of having chipper cost for piers taller than 25 meters.

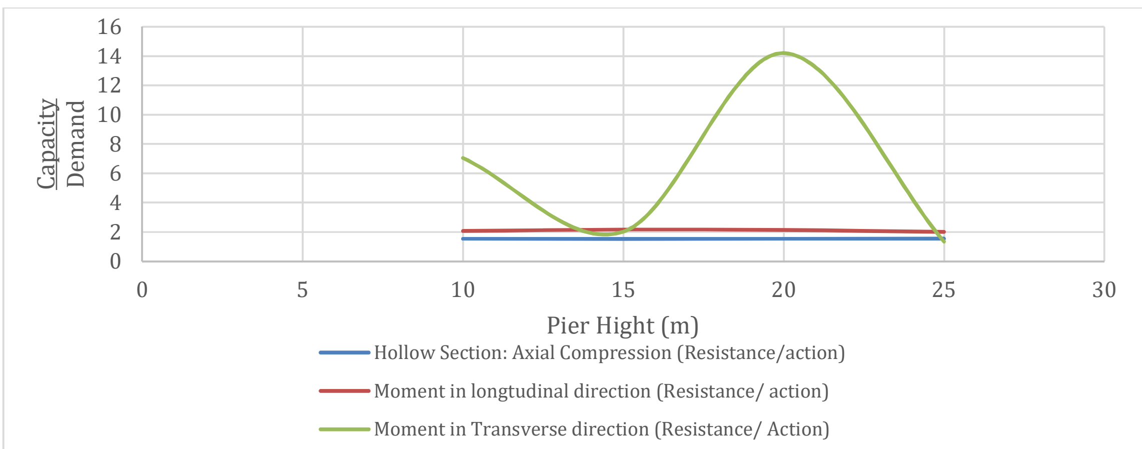



*Figure 4-8: Hollow Section, Capacity demand ratio*

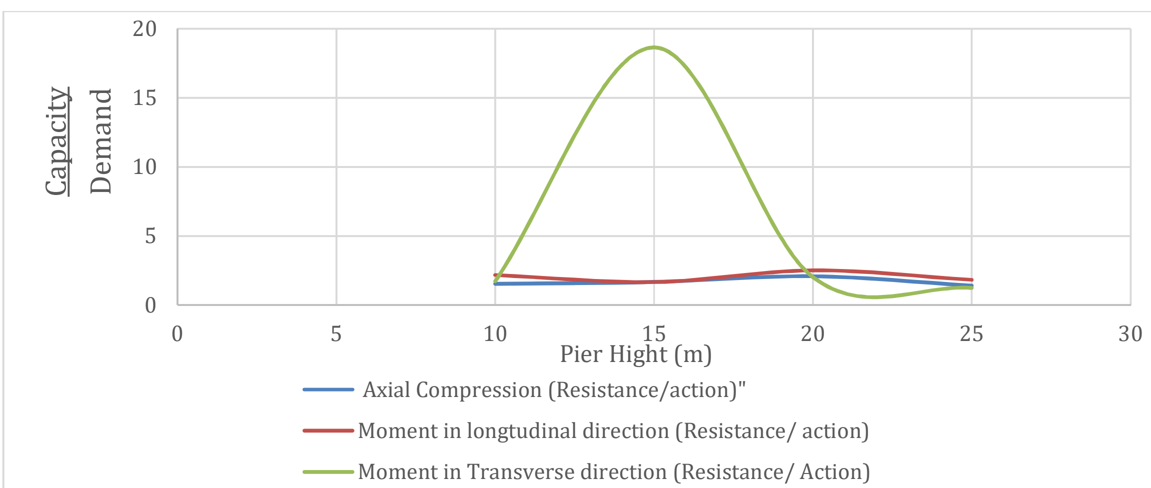



*Figure 4-9: Solid Section, Capacity demand ratio*

The cross sectional dimensions of solid sections are relatively smaller than hollow piers. This is the result of slenderness property and the necessity of magnifying taller solid piers which are under this study. Accordingly, action moments in both x and y directions are magnified. This effect is visible in figure 5- 13, in which the resistance vs ratio of transverse direction moment at 20m and 25m pier heights is smaller.

## 4.4 Total Weight on Foundation

One of the necessary factors while designing the foundation is weight of the structure other permanent and temporary loads subjected to it. The total weight of pier columns over the top of a foundation are graphed on figure 5-14

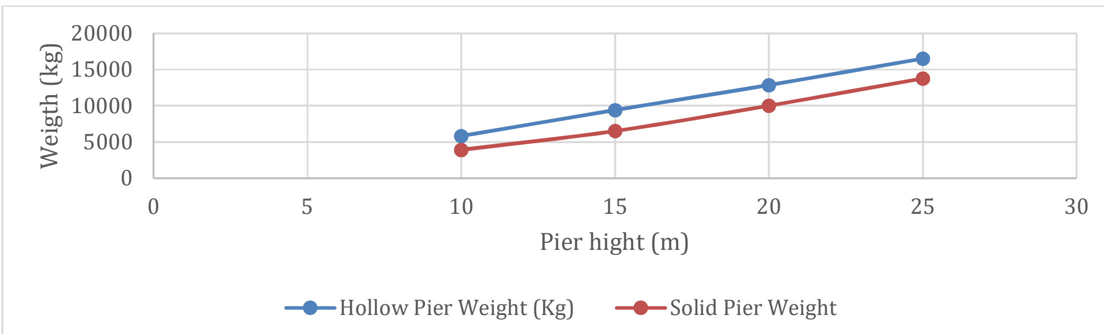



*Figure 4-10: Pier weight Comparison*

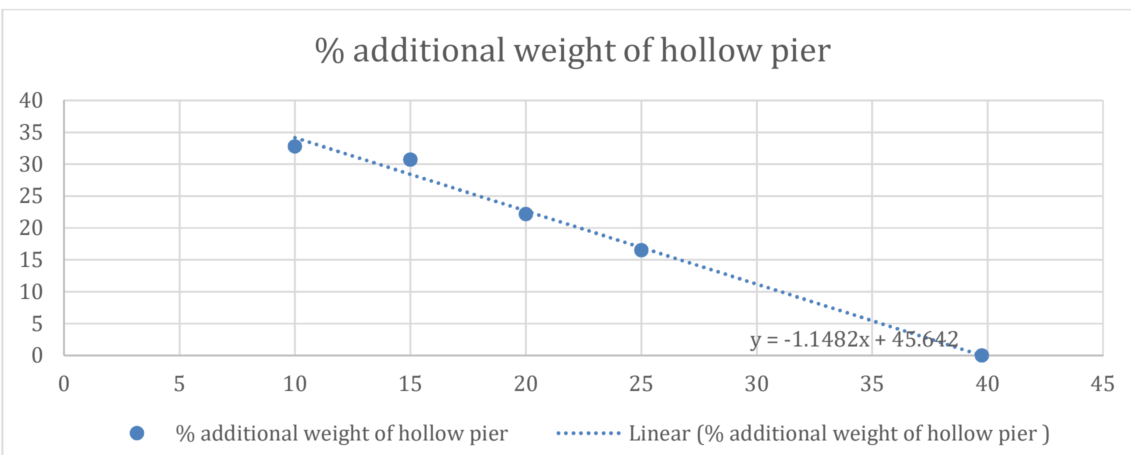



*Figure 4-11: additional Percentage weight increment over solid pier*

Hollow pier is existed to be heavy for piers approximately less than 40m tall.

## 4.5 Heat of hydration and exposure to early stage crack

Depending on the steady state heat analysis done on ANSYS academic software, thermal stress on each cross section is calculated and shown on graph. Thermal expansion coefficient of concrete varies from 1.3 x$10^{-5}$/°C to 0.6 x$10^{-5}$/°C depending of aggregate. Average value of thermal expansion coefficient is used in this thesis.

*Table 4-7: Coefficient of thermal expansion chart*

| **Concrete and aggregate type** | **Coefficient of thermal expansion** |
|---|---|
| Cement Concrete - Quartzite | 1.2 to 1.3 x $10^{-5}$/°C |
| Cement Concrete - Sand Stone | 0.9 to 1.2 x $10^{-5}$/°C |
| Cement Concrete - Granite | 0.7 to 0.95 x $10^{-5}$/°C |
| Cement Concrete - Basalt | 0.8 to 0.95 x $10^{-5}$/°C |
| Cement Concrete - Lime stone | 0.6 to 0.90 x $10^{-5}$/°C |

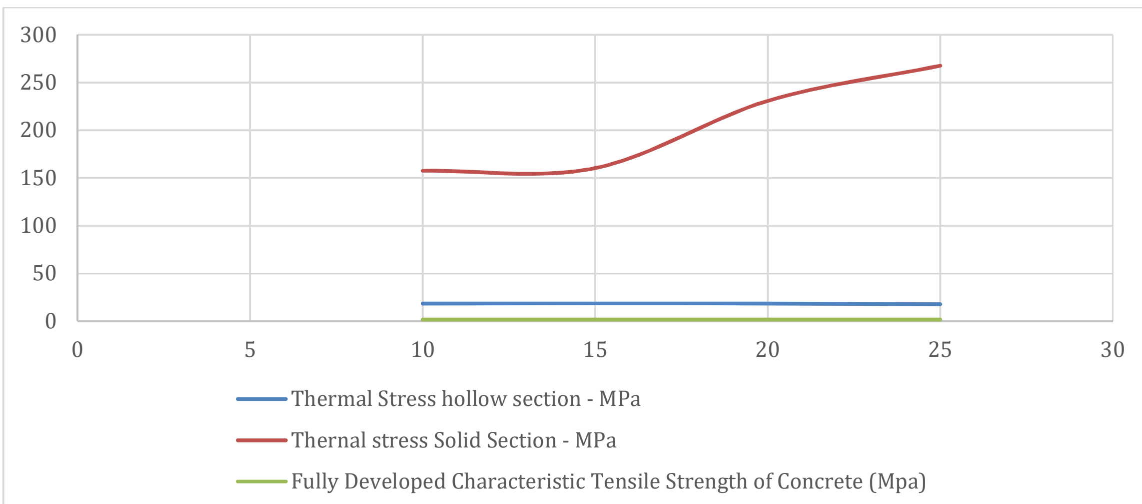



Figure 4-12 Thermal stress (MPa)

In comparison with the fully developed tensile strength of concrete, thermal stress subjected in all cases of piers is much higher. Shrinkage reinforcement required should further be analyzed and proposed. But for this thesis, only comparison in demand for shrinkage reinforcement is conducted. As the height of a pier increases, hollow sections do not show noticeable change on thermal stress requirement, rather the size of solid section increases and this has implication on the development of increasing stress. In general, it is necessary to consider the effect of temperature during construction and design of tall solid core bridge piers and remedial measures shall be taken.

# 5. Conclusion and Recommendations

## 5.1 Conclusion

The main intention of this study was to evaluate all-round applicability of hollow rectangular concrete piers in comparison with conventional solid piers. In order to attain the main goal, example pier height were selected and designed under appropriate permanent, temporary and environmental actions. The outputs of these process are evaluated back again for their performance under seismic loading and ductility property of each pier is calculated.

Analysis and design results are interpreted and brought up for comparison. As the main factor for infrastructural development is economy, economical advantages of one pier over the other is determined. Hollow core piers are less costly in terms of concrete cost. In terms of total cost, It is evident that hollow concrete pier can be economical for piers taller than 25 meters. Hollow pier impacts the cost of 40 meter high or taller bridges by reducing the amount of weight that a foundation has to carry.

Short piers of 20meters high or less, both types of cross- sections do not satisfy the ductility requirements set for SDC D by AASHTO seismic design specification. This will imply that it is necessary to consider applying moment over strength factors and revise the section. Even though the ductility requirement is not satisfied, hollow is observed to have a tendency of satisfying the ductility demand for piers taller than 20 meters. Capacity analysis has verified the plastic moment capacity of the hollow section is in average 35.5% greater than the equivalent height solid section designed against similar load conditions. Solid Sections are confirmed to be brittle for short piers of height less than 35 meters.

Early stage cracking is more probable to be observed on solid sections than in hollow sections. Heat generation increases in solid piers as the height increases. This can result in exceeding the tensile resistance of shrinkage reinforcements. Attention should be given during design and construction as there are remedial measures that could be proposed for such situations. Hollow core sections are advantageous in this regard, since thermal stress is approximately constant as the height increases.

## 5.2 Recommendations

Hollow column axial compression capacity can be improved by using less costly, volumetric stable and light weight material filler. This method can also result in saving reinforcement resources and cost can be kept lower than expected. It is also necessary to keep reinforcement ratio of entire cross section within the limits. Because Hollow sections overcome strength demand with the application of reinforcement area. This could result in giving out over reinforced sections. Filling in the hollow space in the column with less costly material can provide additional compressive strength.

During interaction development, unsymmetrical bending is not considered. Further investigation is recommended since these actions could bring significant change in values and results.

The reduction in the amount of reinforcement in hollow sections could result in better outcomes and economic improvements. In this regard, there are various approaches proposed by researchers recently. Triangular reinforcement method is one of the methods proposed in recent years. [37]. This thesis can be extended by involving the effect of triangular reinforcement in the hollow section design.

Extra skin reinforcement, needed to prevent shrinkage and expansion crack on solid column can be eliminated for hollow section. Shrinkage and expansion can be significant in solid section depending on the cross-sectional dimensions and heat conductivity of a concrete. Hollow core column has significantly small thickness when compared with solid section. This results in the amount of heat variation between inside and outside of the column to be very small and can be resisted using the existing shear and confining reinforcement.

# References


1. Bachmann, Hugo, Peter Linde, and Thomas Wenk. “Capacity Design and Nonlinear Dynamic Analysis of Earthquake-Resistant Structures.” The Tenth European Conference on Earthquake Engineering 1994.

2. Berry, Michael P, and Marc O Eberhard. “Performance Modeling Strategies for Modern Reinforced Concrete Bridge Columns.” April (2008).

3. Calvi, Gian Michele et al. “Experimental and Numerical Studies on the Seismic Response of R.C. Hollow Bridge Piers.” Bulletin of Earthquake Engineering 3.3 (2005): 267–297

4. Denpongpan, Thammanoon, and Hiroshi Shima. “Effect of Axial Load on Ductility of Reinforced.” (2005)

5. Dhakal, Rajesh P. “Structural Design for Earthquake Resistance: Past, Present and Future.” August (2011): 1–34

6. Eberhard, Marc O, and Michael P Berry. “Modeling of Reinforced Concrete Bridge Columns.” (2006):

7. Filippou, Filip C, Angelo D Ambrisi, and Ahmad Issa. “NONLINEAR STATIC AND DYNAMIC ANALYSIS OF REINFORCED CONCRETE SUBASSEMBLAGES. Report No. UCB/EERC–92/08.” August (1992)

8. George, Sujin S, and Valsson Varghese. “General Concepts of Capacity Based Design.” 2 (2012): 211–215

9. Krawinkler.Helmut. “Pushover Analysis Why When and Not to Use It.” Proceedings 1996 Convention. Structural Engineers Association of California 1996: 17–36

10. Mavichak V, Furlong RW. Strength and Stiffness of Reinforced Concrete Columns Under Biaxial Bending. 1976;(2).

11. M, Ravi Kumar C, Vimal Choudhary, and K S Babu. “Moment Curvature Characteristics for Structural Elements of RC Building.” 2.1 (2014): 1–17

12. Matthews, Robert. “□ 2001 Robert Matthews.” (2001)

13. Ranasinghe, Arjuna. “The Design of Precast Concrete Segmental Bridge Piers in New Jersey, USA.”

14. Sheikh, Neaz, A. Vivier, and F. Legeron. “Seismic Vulnerability of Hollow Core Concrete Bridge Piers.” (2007): 1445–1454

15. . Paulay and M.J.N. Priestley, ‘Seismic Design of Reinforced Concrete and Masonry Buildings’, by T Wiley, New York, 1992

16. Themellis, Spyridon. “Pushover Analysis for Seismic Assessment and Design of Structures.” October (2008): 261

17. Ward, Ariel et al. “Investigating the Behavior of Reinforced Concrete Columns through Static and Dynamic Analysis.” c (2014)

18. Wardenier, J. “HOLLOW SECTIONS Hollow Sections in Structural Applications.” Technology (2001): 199

19. Wickline, Joseph E. “A Study of Effective Moment of Inertia Models for Full-Scale Reinforced Concrete T-Beams Subjected to A Tandem-Axle Load Configuration.” September (2002): 117

20. Caltrans. “Seismic Design Criteria Version 1.7.” California Department of Transportation: Sacramento, CA, U.S. April (2013)

21. Cedolin, L et al. “Biaxial Bending of Concrete Columns: An Analytical Solution.” Studies and Researches 26 (2006): 1–30

22. Charney, Finley, Adrian Tola Tola, and Ozgur Atlayan. “FEMA P-751: Chapter 4: Structural Analysis.”

23. Columns, Reinforced Concrete. “Determination of Moment Capacity M.” 20.1 (2009): 1293–1312

24. Helgason, Valdimar Örn. “Development of a Computer Program to Design Concrete Columns for Biaxial Moments and Normal Force.” (2010)

25. Kumar, Manish. “CIE525 : Assignment 3 Moment-Curvature Relationships.” 1–13

26. Kwak, Hyo-gyoung, and Sun-pil Kim. “Nonlinear Analysis of RC Beams Based on Moment – Curvature Relation.” 80 (2002): 615–628

27. Lejano, Bernardo A. “Investigation of Biaxial Bending of Reinforced Concrete Columns Through Fiber Method Modeling.” Journal of Research in Science, Computing and Engineering 4 (2008): 61–73

28. Mander, J. B., M.J.N. Priestley, and R.Park. “Theoretical Stress Strain Model for Confined Concrete.” Journal of Structural Engineering 1988: 1804 – 1825.

29. Mavichak, V, and R W Furlong. “Strength and Stiffness of Reinforced Concrete Columns Under Biaxial Bending.” 2 (1976)

30. Suarez, Vinicio a, and Jose C Hurtado. “RC Virtual Laboratory for Earthquake Engineering.” (2008)

31. Taucer, Fabio F., Enrico Spacone, and Filip C. Filippou. “A Fiber Beam-Column Element for Seismic Response Analysis of Reinforced Concrete Structures.” Ucb/Eerc-91/17 December 1991 (1991)

32. Priestley, MJN , Hoshikuma, Jun-Ichi, “Flexural Behavior Of Circular Hollow Columns With A Single Layer Of Reinforcement Under Seismic Loading” 2000;(59):iii - 79.

33. Gaspersic P, Fajfar P, Fischinger M. An approximate method for seismic damage analysis of buildings. *10Th World Conf Earthq Eng*. 1992:3921-3926.

34. National Ready Mixed Concrete Association. CIP 42- Thermal Cracking of Concrete. 2009.

35. Mukhopadhyay A, Ye D, Zollinger D. Moisture-related cracking effects on hydrating concrete pavement. 2006;7(2):172.

36. Kurtis K. Portland Cement Hydration. 2007:1-35. Papers 2

37. Kim, T. H., et al. "Performance assessment of advanced hollow RC bridge column sections." *Computers and Concrete* 16.5 (2015): 703-722.

38. Ethiopian Road Authority, ERA Bridge design manual, 2002

## Appendix A- 1: Interaction Calculation 10m Hollow
(Moment in MN-m , Forces in MN)

**Longtudinal Direction**

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.019 | 0.0000 | 0.00195 | 0.00 | 24 | 84 | 12.5 | 7.70 | 26.4 | -6.17 | 0.218 | 0.195 | 1.128 | 35.2 |
| 0.185 | 0.0022 | 0.00194 | 0.16 | 30 | 79 | 15.3 | 9.48 | 24.6 | 0.14 | 0.170 | 0.179 | 1.201 | 35.4 |
| 0.370 | 0.0026 | 0.00193 | 0.31 | 36 | 73 | 18.3 | 11.46 | 22.6 | 7.15 | 0.193 | 0.195 | 1.280 | 35.0 |
| 0.555 | 0.0027 | 0.00192 | 0.47 | 42 | 67 | 21.3 | 13.44 | 20.6 | 14.17 | 0.236 | 0.233 | 1.357 | 33.8 |
| 0.740 | 0.0028 | 0.00191 | 0.63 | 48 | 60 | 24.3 | 15.43 | 18.5 | 21.21 | 0.292 | 0.284 | 1.431 | 31.8 |
| 0.925 | 0.0028 | 0.00189 | 0.79 | 54 | 54 | 27.3 | 17.41 | 16.5 | 28.26 | 0.357 | 0.345 | 1.501 | 28.9 |
| 1.110 | 0.0029 | 0.00186 | 0.94 | 60 | 48 | 30.4 | 19.39 | 14.4 | 35.35 | 0.427 | 0.412 | 1.564 | 25.2 |
| 1.295 | 0.0029 | 0.00182 | 1.10 | 67 | 42 | 33.4 | 21.37 | 12.2 | 42.50 | 0.501 | 0.483 | 1.620 | 20.6 |
| 1.480 | 0.0029 | 0.00165 | 1.26 | 73 | 36 | 36.4 | 23.35 | 9.9 | 49.82 | 0.578 | 0.558 | 1.662 | 15.1 |
| 1.665 | 0.0029 | 0.00146 | 1.42 | 79 | 30 | 39.4 | 25.34 | 6.9 | 57.82 | 0.658 | 0.636 | 1.683 | 8.8 |
| 1.832 | 0.0029 | 0.00123 | 1.56 | 84 | 24 | 42.1 | 27.12 | -13.1 | 82.37 | 0.731 | 0.708 | 1.637 | 0.0 |

**Transverse Direction**

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.023 | -0.0035 | 0.00196 | 0.02 | 17 | 92 | 8.9 | 5.33 | 28.9 | -14.58 | 0.224 | 0.192 | 1.242 | 35.9 |
| 0.230 | 0.0023 | 0.00195 | 0.20 | 24 | 85 | 12.3 | 7.55 | 26.6 | -6.76 | 0.164 | 0.174 | 1.342 | 37.1 |
| 0.460 | 0.0027 | 0.00195 | 0.39 | 31 | 77 | 16.1 | 10.02 | 24.1 | 1.94 | 0.231 | 0.214 | 1.452 | 37.5 |
| 0.690 | 0.0028 | 0.00194 | 0.59 | 39 | 70 | 19.8 | 12.48 | 21.7 | 10.65 | 0.339 | 0.282 | 1.561 | 36.7 |
| 0.920 | 0.0028 | 0.00193 | 0.78 | 47 | 62 | 23.6 | 14.94 | 19.2 | 19.36 | 0.468 | 0.366 | 1.669 | 34.9 |
| 1.150 | 0.0029 | 0.00191 | 0.98 | 54 | 54 | 27.3 | 17.41 | 16.7 | 28.08 | 0.610 | 0.457 | 1.775 | 31.9 |
| 1.380 | 0.0029 | 0.00189 | 1.17 | 62 | 47 | 31.1 | 19.87 | 14.1 | 36.82 | 0.759 | 0.555 | 1.877 | 27.8 |
| 1.610 | 0.0029 | 0.00186 | 1.37 | 70 | 39 | 34.8 | 22.34 | 11.6 | 45.59 | 0.914 | 0.656 | 1.975 | 22.6 |
| 1.840 | 0.0029 | 0.00178 | 1.56 | 77 | 31 | 38.6 | 24.80 | 8.9 | 54.46 | 1.072 | 0.760 | 2.064 | 16.3 |
| 2.070 | 0.0029 | 0.00157 | 1.76 | 85 | 24 | 42.4 | 27.26 | 5.9 | 63.70 | 1.232 | 0.866 | 2.136 | 8.9 |
| 2.277 | 0.0029 | -0.00235 | 1.94 | 92 | 17 | 45.7 | 29.48 | -6.3 | 81.48 | 1.379 | 0.963 | 2.076 | 0.0 |

**Legend**

C value = location of nutral axis
Strain Cc = Concrete compressive strain (max = 0.0035
Strain Cs = Reinforcement bar strain (max =0.002)
a= depth of section under compression
Cc= Resistance axial force of concrete under compression
Ts = Resistance axial force of Steel under tension
Cs= Resistance axial force of steel under compression

## Appendix A-2 : Interaction Calculation 10m Solid

(Moment in MN-m , Forces in MN)

### Longtudinal Direction

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.016 | -0.0064 | 0.00194 | 0.01 | | 41 | 0.0 | 0.00 | 12.8 | -26.93 | | 0.635 | 1.108 | 35.8 |
| 0.160 | 0.0021 | 0.00193 | 0.14 | | 36 | 0.0 | 0.00 | 11.2 | -18.32 | | 1.182 | 1.138 | 33.8 |
| 0.320 | 0.0025 | 0.00192 | 0.27 | 1 | 31 | 0.8 | 0.32 | 9.5 | -8.44 | 1.624 | -8.329 | 1.171 | 30.4 |
| 0.480 | 0.0027 | 0.00191 | 0.41 | 4 | 26 | 8.6 | 1.35 | 7.8 | 2.14 | 0.603 | -0.386 | 1.201 | 31.4 |
| 0.640 | 0.0028 | 0.00190 | 0.54 | 10 | 20 | 16.5 | 3.06 | 6.2 | 13.40 | 0.406 | 0.065 | 1.224 | 30.8 |
| 0.800 | 0.0028 | 0.00188 | 0.68 | 15 | 15 | 24.4 | 4.78 | 4.5 | 24.65 | 0.378 | 0.276 | 1.237 | 28.9 |
| 0.960 | 0.0028 | 0.00184 | 0.82 | 20 | 10 | 32.2 | 6.49 | 2.8 | 35.89 | 0.387 | 0.423 | 1.218 | 25.7 |
| 1.120 | 0.0029 | 0.00179 | 0.95 | 26 | 4 | 40.1 | 8.20 | 1.2 | 47.08 | 0.408 | 0.544 | 1.051 | 21.1 |
| 1.280 | 0.0029 | 0.00169 | 1.09 | 31 | 1 | 47.9 | 9.92 | 0.3 | 57.59 | 0.436 | 0.652 | 0.203 | 15.0 |
| 1.440 | 0.0029 | 0.00138 | 1.22 | 36 | | 55.8 | 11.63 | 0.0 | 67.44 | 0.468 | 0.752 | | 7.5 |
| 1.584 | 0.0029 | 0.00000 | 1.35 | 41 | | 62.9 | 13.17 | 0.0 | 76.05 | 0.498 | 0.837 | | 0.0 |

### Transverse Direction

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.018 | -0.0053 | 0.00194 | 0.02 | | 44 | 0.0 | 0.00 | 13.8 | -32.81 | | 0.630 | 1.221 | 40.2 |
| 0.180 | 0.0022 | 0.00194 | 0.15 | | 39 | 0.0 | 0.00 | 12.1 | -23.13 | | 1.013 | 1.254 | 66.2 |
| 0.360 | 0.0026 | 0.00193 | 0.31 | 1 | 33 | 1.0 | 0.32 | 10.2 | -12.05 | 1.905 | 4.043 | 1.289 | 36.2 |
| 0.540 | 0.0027 | 0.00192 | 0.46 | 3 | 27 | 6.7 | 0.92 | 8.3 | -0.70 | 1.345 | -0.718 | 1.319 | 35.2 |
| 0.720 | 0.0028 | 0.00191 | 0.61 | 9 | 21 | 15.5 | 2.85 | 6.4 | 11.96 | 0.776 | 0.050 | 1.340 | 33.0 |
| 0.900 | 0.0028 | 0.00189 | 0.77 | 15 | 15 | 24.4 | 4.78 | 4.5 | 24.62 | 0.739 | 0.326 | 1.342 | 31.3 |
| 1.080 | 0.0029 | 0.00186 | 0.92 | 21 | 9 | 33.2 | 6.70 | 2.7 | 37.26 | 0.775 | 0.503 | 1.286 | 28.1 |
| 1.260 | 0.0029 | 0.00181 | 1.07 | 27 | 3 | 42.0 | 8.63 | 0.8 | 49.84 | 0.835 | 0.643 | 0.806 | 23.2 |
| 1.440 | 0.0029 | 0.00172 | 1.22 | 33 | 1 | 50.9 | 10.56 | 0.3 | 61.17 | 0.906 | 0.766 | 0.126 | 16.5 |
| 1.620 | 0.0029 | 0.00144 | 1.38 | 39 | | 59.7 | 12.49 | 0.0 | 72.22 | 0.983 | 0.879 | | 8.3 |
| 1.782 | 0.0029 | 0.00000 | 1.51 | 44 | | 67.7 | 14.22 | 0.0 | 81.92 | 1.055 | 0.976 | | 0.0 |

**Legend**

C value = location of nutral axis

Strain Cc = Concrete compressive strain (max = 0.0035

Strain Cs = Reinforcement bar strain (max =0.002)

a= depth of section under compression

Cc= Resistance axial force of concrete under compression

Ts = Resistance axial force of Steel under tension

Cs= Resistance axial force of steel under compression

## Appendix A-3 : Interaction Calculation 15m Hollow

(Moment in MN-m , Forces in MN)

**Longtudinal Direction**

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.020 | -0.0045 | 0.00195 | 0.02 | 26 | 91 | 13.4 | 8.25 | 28.5 | -6.90 | 0.216 | 0.195 | 1.222 | 40.3 |
| 0.200 | 0.0023 | 0.00194 | 0.17 | 32 | 85 | 16.3 | 10.18 | 26.6 | -0.09 | 0.172 | 0.180 | 1.302 | 40.6 |
| 0.400 | 0.0026 | 0.00194 | 0.34 | 38 | 78 | 19.6 | 12.32 | 24.4 | 7.49 | 0.199 | 0.200 | 1.388 | 40.2 |
| 0.600 | 0.0028 | 0.00193 | 0.51 | 45 | 72 | 22.8 | 14.46 | 22.2 | 15.08 | 0.247 | 0.243 | 1.472 | 38.8 |
| 0.800 | 0.0028 | 0.00192 | 0.68 | 52 | 65 | 26.1 | 16.60 | 20.0 | 22.68 | 0.309 | 0.300 | 1.552 | 36.4 |
| 1.000 | 0.0029 | 0.00190 | 0.85 | 58 | 58 | 29.4 | 18.75 | 17.8 | 30.30 | 0.379 | 0.367 | 1.628 | 33.1 |
| 1.200 | 0.0029 | 0.00188 | 1.02 | 65 | 52 | 32.6 | 20.89 | 15.6 | 37.95 | 0.456 | 0.440 | 1.698 | 28.7 |
| 1.400 | 0.0029 | 0.00183 | 1.19 | 72 | 45 | 35.9 | 23.03 | 13.3 | 45.67 | 0.536 | 0.518 | 1.759 | 23.4 |
| 1.600 | 0.0029 | 0.00175 | 1.36 | 78 | 38 | 39.2 | 25.17 | 10.8 | 53.55 | 0.621 | 0.600 | 1.806 | 17.1 |
| 1.800 | 0.0029 | 0.00150 | 1.53 | 85 | 32 | 42.4 | 27.32 | 7.6 | 62.11 | 0.707 | 0.685 | 1.831 | 9.7 |
| 1.980 | 0.0029 | -0.00300 | 1.68 | 91 | 26 | 45.4 | 29.25 | -12.4 | 87.00 | 0.787 | 0.763 | 1.788 | -0.1 |

**Tangential Direction**

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.024 | -0.0033 | 0.00196 | 0.02 | 19 | 98 | 10.2 | 6.15 | 30.7 | -14.38 | 0.220 | 0.193 | 1.317 | 40.1 |
| 0.240 | 0.0024 | 0.00195 | 0.20 | 26 | 90 | 13.7 | 8.46 | 28.4 | -6.22 | 0.169 | 0.177 | 1.421 | 41.3 |
| 0.480 | 0.0027 | 0.00195 | 0.41 | 34 | 82 | 17.6 | 11.03 | 25.8 | 2.86 | 0.236 | 0.218 | 1.535 | 41.6 |
| 0.720 | 0.0028 | 0.00194 | 0.61 | 42 | 74 | 21.5 | 13.60 | 23.2 | 11.95 | 0.346 | 0.287 | 1.647 | 40.6 |
| 0.960 | 0.0028 | 0.00193 | 0.82 | 50 | 66 | 25.5 | 16.18 | 20.6 | 21.04 | 0.478 | 0.372 | 1.758 | 38.5 |
| 1.200 | 0.0029 | 0.00192 | 1.02 | 58 | 58 | 29.4 | 18.75 | 18.0 | 30.14 | 0.623 | 0.467 | 1.867 | 35.2 |
| 1.440 | 0.0029 | 0.00190 | 1.22 | 66 | 50 | 33.3 | 21.32 | 15.3 | 39.26 | 0.777 | 0.567 | 1.971 | 30.6 |
| 1.680 | 0.0029 | 0.00186 | 1.43 | 74 | 42 | 37.2 | 23.89 | 12.7 | 48.43 | 0.936 | 0.672 | 2.070 | 24.8 |
| 1.920 | 0.0029 | 0.00179 | 1.63 | 82 | 34 | 41.1 | 26.46 | 9.9 | 57.69 | 1.099 | 0.779 | 2.160 | 17.8 |
| 2.160 | 0.0029 | 0.00158 | 1.84 | 90 | 26 | 45.0 | 29.03 | 6.7 | 67.37 | 1.265 | 0.889 | 2.230 | 9.6 |
| 2.376 | 0.0029 | -0.00217 | 2.02 | 98 | 19 | 48.6 | 31.35 | -6.7 | 86.58 | 1.417 | 0.989 | 2.180 | -0.2 |

**Legend**

C value = location of nutral axis

Strain Cc = Concrete compressive strain (max = 0.0035

Strain Cs = Reinforcement bar strain (max =0.002)

a= depth of section under compression

Cc= Resistance axial force of concrete under compression

Ts = Resistance axial force of Steel under tension

Cs= Resistance axial force of steel under compression

## Appendix A-4 : Interaction Calculation 15m Solid

### Longtudinal Direction

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.016 | -0.0064 | 0.00194 | 0.01 | | 41 | -14.1 | -3.56 | 12.8 | -30.46 | -0.108 | 0.646 | 1.109 | 41.2 |
| 0.160 | 0.0021 | 0.00193 | 0.14 | | 36 | -6.9 | -2.01 | 11.3 | -20.20 | -0.188 | 1.228 | 1.140 | 37.2 |
| 0.320 | 0.0025 | 0.00192 | 0.27 | 1 | 31 | 1.1 | 0.32 | 9.6 | -8.18 | 1.658 | -5.943 | 1.173 | 35.2 |
| 0.480 | 0.0027 | 0.00191 | 0.41 | 4 | 26 | 9.1 | 1.41 | 7.9 | 2.59 | 0.583 | -0.368 | 1.203 | 32.6 |
| 0.640 | 0.0028 | 0.00190 | 0.54 | 10 | 20 | 17.1 | 3.13 | 6.2 | 13.97 | 0.400 | 0.066 | 1.227 | 32.0 |
| 0.800 | 0.0028 | 0.00188 | 0.68 | 15 | 15 | 25.1 | 4.84 | 4.5 | 25.35 | 0.376 | 0.274 | 1.240 | 30.0 |
| 0.960 | 0.0028 | 0.00184 | 0.82 | 20 | 10 | 33.0 | 6.56 | 2.9 | 36.70 | 0.385 | 0.421 | 1.224 | 26.7 |
| 1.120 | 0.0029 | 0.00179 | 0.95 | 26 | 4 | 41.0 | 8.27 | 1.3 | 48.02 | 0.406 | 0.541 | 1.070 | 22.0 |
| 1.280 | 0.0029 | 0.00169 | 1.09 | 31 | 1 | 49.0 | 9.98 | 0.3 | 58.71 | 0.434 | 0.649 | 0.175 | 15.8 |
| 1.440 | 0.0029 | 0.00138 | 1.22 | 36 | | 57.0 | 11.70 | -1.4 | 70.07 | 0.466 | 0.748 | 1.751 | 8.5 |
| 1.584 | 0.0029 | -0.00425 | 1.35 | 41 | | 64.2 | 13.24 | 7.6 | 69.85 | 0.496 | 0.834 | 1.687 | 0.0 |

### Tangential Direction

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.019 | -0.0051 | 0.00195 | 0.02 | | 45 | -20.2 | -4.87 | 14.2 | -39.22 | -0.068 | 0.639 | 1.252 | 45.1 |
| 0.185 | 0.0022 | 0.00194 | 0.16 | | 40 | -11.9 | -3.08 | 12.4 | -27.36 | -0.109 | 1.015 | 1.286 | 41.1 |
| 0.370 | 0.0026 | 0.00193 | 0.31 | 1 | 34 | -2.6 | -1.10 | 10.4 | -14.18 | -0.578 | 3.591 | 1.322 | 39.1 |
| 0.555 | 0.0027 | 0.00192 | 0.47 | 3 | 27 | 6.6 | 0.88 | 8.5 | -1.01 | 1.485 | -0.792 | 1.352 | 36.5 |
| 0.740 | 0.0028 | 0.00191 | 0.63 | 9 | 21 | 15.8 | 2.86 | 6.5 | 12.16 | 0.817 | 0.048 | 1.374 | 35.4 |
| 0.925 | 0.0028 | 0.00189 | 0.79 | 15 | 15 | 25.1 | 4.84 | 4.6 | 25.31 | 0.772 | 0.337 | 1.375 | 33.7 |
| 1.110 | 0.0029 | 0.00186 | 0.94 | 21 | 9 | 34.3 | 6.82 | 2.7 | 38.44 | 0.807 | 0.520 | 1.313 | 30.3 |
| 1.295 | 0.0029 | 0.00182 | 1.10 | 27 | 3 | 43.5 | 8.81 | 0.8 | 51.52 | 0.867 | 0.665 | 0.765 | 25.1 |
| 1.480 | 0.0029 | 0.00173 | 1.26 | 34 | 1 | 52.7 | 10.79 | -1.0 | 64.49 | 0.940 | 0.791 | 2.353 | 18.2 |
| 1.665 | 0.0029 | 0.00146 | 1.42 | 40 | | 62.0 | 12.77 | -2.3 | 77.00 | 1.018 | 0.908 | 1.960 | 9.7 |
| 1.832 | 0.0029 | -0.00341 | 1.56 | 45 | | 70.3 | 14.55 | 8.3 | 76.54 | 1.092 | 1.007 | 1.918 | 0.0 |

**Legend**

C value = location of nutral axis

Strain Cc = Concrete compressive strain (max = 0.0035

Strain Cs = Reinforcement bar strain (max =0.002)

a= depth of section under compression

Cc= Resistance axial force of concrete under compression

Ts = Resistance axial force of Steel under tension

Cs= Resistance axial force of steel under compression

## Appendix A-5: Interaction Calculation 20m Hollow

(Moment in MN-m , Forces in MN)

**Longtudinal Direction**

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.021 | -0.0043 | 0.00195 | 0.02 | 27 | 93 | 13.8 | 8.52 | 29.3 | -7.01 | 0.215 | 0.195 | 1.256 | 42.4 |
| 0.205 | 0.0023 | 0.00195 | 0.17 | 33 | 87 | 16.8 | 10.50 | 27.3 | -0.03 | 0.173 | 0.181 | 1.337 | 42.7 |
| 0.410 | 0.0026 | 0.00194 | 0.35 | 40 | 81 | 20.2 | 12.69 | 25.1 | 7.74 | 0.201 | 0.202 | 1.425 | 42.3 |
| 0.615 | 0.0028 | 0.00193 | 0.52 | 46 | 74 | 23.5 | 14.89 | 22.9 | 15.52 | 0.250 | 0.246 | 1.511 | 40.8 |
| 0.820 | 0.0028 | 0.00192 | 0.70 | 53 | 67 | 26.8 | 17.09 | 20.6 | 23.31 | 0.314 | 0.305 | 1.594 | 38.3 |
| 1.025 | 0.0029 | 0.00190 | 0.87 | 60 | 60 | 30.2 | 19.28 | 18.4 | 31.11 | 0.386 | 0.373 | 1.672 | 34.8 |
| 1.230 | 0.0029 | 0.00188 | 1.05 | 67 | 53 | 33.5 | 21.48 | 16.1 | 38.96 | 0.464 | 0.448 | 1.743 | 30.2 |
| 1.435 | 0.0029 | 0.00184 | 1.22 | 74 | 46 | 36.9 | 23.67 | 13.7 | 46.86 | 0.547 | 0.529 | 1.805 | 24.6 |
| 1.640 | 0.0029 | 0.00176 | 1.39 | 81 | 40 | 40.2 | 25.87 | 11.2 | 54.94 | 0.633 | 0.612 | 1.854 | 17.9 |
| 1.845 | 0.0029 | 0.00151 | 1.57 | 87 | 33 | 43.6 | 28.07 | 7.9 | 63.70 | 0.722 | 0.699 | 1.880 | 10.2 |
| 2.030 | 0.0029 | -0.00288 | 1.73 | 93 | 27 | 46.6 | 30.04 | -12.3 | 88.90 | 0.804 | 0.779 | 1.839 | 0.0 |

**Transverse Direction**

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.025 | -0.0031 | 0.00196 | 0.02 | 20 | 100 | 10.6 | 6.42 | 31.5 | -14.49 | 0.219 | 0.193 | 1.351 | 42.2 |
| 0.245 | 0.0024 | 0.00195 | 0.21 | 27 | 93 | 14.2 | 8.78 | 29.1 | -6.15 | 0.171 | 0.178 | 1.456 | 43.4 |
| 0.490 | 0.0027 | 0.00195 | 0.42 | 36 | 85 | 18.2 | 11.41 | 26.5 | 3.12 | 0.239 | 0.220 | 1.572 | 43.7 |
| 0.735 | 0.0028 | 0.00194 | 0.62 | 44 | 76 | 22.2 | 14.03 | 23.8 | 12.39 | 0.351 | 0.291 | 1.687 | 42.7 |
| 0.980 | 0.0028 | 0.00193 | 0.83 | 52 | 68 | 26.2 | 16.66 | 21.2 | 21.67 | 0.485 | 0.377 | 1.800 | 40.5 |
| 1.225 | 0.0029 | 0.00192 | 1.04 | 60 | 60 | 30.2 | 19.28 | 18.5 | 30.96 | 0.633 | 0.473 | 1.910 | 37.0 |
| 1.470 | 0.0029 | 0.00190 | 1.25 | 68 | 52 | 34.2 | 21.91 | 15.8 | 40.27 | 0.789 | 0.575 | 2.017 | 32.2 |
| 1.715 | 0.0029 | 0.00186 | 1.46 | 76 | 44 | 38.2 | 24.53 | 13.1 | 49.63 | 0.951 | 0.682 | 2.117 | 26.2 |
| 1.960 | 0.0029 | 0.00180 | 1.67 | 85 | 36 | 42.2 | 27.16 | 10.3 | 59.09 | 1.118 | 0.791 | 2.207 | 18.8 |
| 2.205 | 0.0029 | 0.00159 | 1.87 | 93 | 27 | 46.2 | 29.78 | 7.0 | 68.97 | 1.287 | 0.903 | 2.278 | 10.2 |
| 2.426 | 0.0029 | -0.00208 | 2.06 | 100 | 20 | 49.8 | 32.14 | -6.7 | 88.62 | 1.441 | 1.005 | 2.231 | 0.0 |

**Legend**

C value = location of nutral axis
Strain Cc = Concrete compressive strain (max = 0.0035
Strain Cs = Reinforcement bar strain (max =0.002)

a= depth of section under compression
Cc= Resistance axial force of concrete under compression
Ts = Resistance axial force of Steel under tension
Cs= Resistance axial force of steel under compression

## Appendix A-6: Interaction Calculation 20m Solid

(Moment in MN-m , Forces in MN)

### Longtudinal Direction

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.020 | -0.0047 | 0.00195 | 0.02 | | 51 | -20.9 | -4.22 | 15.8 | -40.97 | -0.112 | 0.786 | 1.352 | 47.2 |
| 0.195 | 0.0022 | 0.00194 | 0.17 | | 45 | -10.2 | -2.34 | 14.0 | -26.56 | -0.209 | 1.492 | 1.390 | 52.1 |
| 0.390 | 0.0026 | 0.00194 | 0.33 | 1 | 38 | 1.6 | 0.32 | 11.9 | -9.98 | 2.276 | -7.404 | 1.431 | 55.3 |
| 0.585 | 0.0027 | 0.00193 | 0.50 | 6 | 32 | 13.4 | 1.83 | 9.8 | 5.45 | 0.642 | -0.450 | 1.468 | 56.4 |
| 0.780 | 0.0028 | 0.00191 | 0.66 | 12 | 25 | 25.3 | 3.92 | 7.8 | 21.45 | 0.466 | 0.080 | 1.500 | 55.1 |
| 0.975 | 0.0028 | 0.00190 | 0.83 | 19 | 19 | 37.1 | 6.01 | 5.7 | 37.43 | 0.446 | 0.335 | 1.519 | 51.5 |
| 1.170 | 0.0029 | 0.00187 | 0.99 | 25 | 12 | 49.0 | 8.10 | 3.7 | 53.40 | 0.462 | 0.513 | 1.508 | 45.6 |
| 1.365 | 0.0029 | 0.00183 | 1.16 | 32 | 6 | 60.8 | 10.19 | 1.7 | 69.33 | 0.491 | 0.660 | 1.358 | 37.3 |
| 1.560 | 0.0029 | 0.00174 | 1.33 | 38 | 1 | 72.7 | 12.28 | 0.3 | 84.66 | 0.527 | 0.791 | -0.039 | 26.7 |
| 1.755 | 0.0029 | 0.00149 | 1.49 | 45 | | 84.5 | 14.37 | -1.7 | 100.62 | 0.566 | 0.912 | 2.119 | 14.0 |
| 1.931 | 0.0029 | -0.00313 | 1.64 | 51 | | 95.2 | 16.25 | 6.6 | 104.80 | 0.604 | 1.017 | 2.040 | 0.0 |

### Transverse Direction

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.023 | -0.0037 | 0.00196 | 0.02 | | 55 | -29.8 | -5.80 | 17.4 | -53.07 | -0.070 | 0.778 | 1.523 | 47.2 |
| 0.225 | 0.0023 | 0.00195 | 0.19 | | 49 | -17.5 | -3.63 | 15.3 | -36.45 | -0.124 | 1.237 | 1.566 | 54.5 |
| 0.450 | 0.0027 | 0.00194 | 0.38 | 1 | 41 | -3.9 | -1.22 | 12.9 | -17.98 | -0.787 | 4.413 | 1.610 | 59.8 |
| 0.675 | 0.0028 | 0.00194 | 0.57 | 4 | 34 | 9.8 | 1.19 | 10.5 | 0.49 | 1.686 | -0.958 | 1.648 | 62.2 |
| 0.900 | 0.0028 | 0.00193 | 0.77 | 11 | 26 | 23.5 | 3.60 | 8.1 | 18.95 | 1.001 | 0.059 | 1.677 | 61.8 |
| 1.125 | 0.0029 | 0.00191 | 0.96 | 19 | 19 | 37.1 | 6.01 | 5.8 | 37.39 | 0.955 | 0.410 | 1.683 | 58.5 |
| 1.350 | 0.0029 | 0.00189 | 1.15 | 26 | 11 | 50.8 | 8.42 | 3.4 | 55.81 | 1.000 | 0.632 | 1.622 | 52.3 |
| 1.575 | 0.0029 | 0.00185 | 1.34 | 34 | 4 | 64.5 | 10.83 | 1.1 | 74.19 | 1.075 | 0.808 | 1.087 | 43.2 |
| 1.800 | 0.0029 | 0.00178 | 1.53 | 41 | 1 | 78.1 | 13.24 | -1.1 | 92.46 | 1.163 | 0.962 | 2.891 | 31.2 |
| 2.025 | 0.0029 | 0.00156 | 1.72 | 49 | | 91.8 | 15.65 | -2.8 | 110.27 | 1.259 | 1.104 | 2.373 | 16.4 |
| 2.228 | 0.0029 | -0.00244 | 1.89 | 55 | | 104.1 | 17.82 | 7.1 | 114.82 | 1.349 | 1.224 | 2.320 | 0.0 |

**Legend**

C value = location of nutral axis
Strain Cc = Concrete compressive strain (max = 0.0035
Strain Cs = Reinforcement bar strain (max =0.002)

a= depth of section under compression
Cc= Resistance axial force of concrete under compression
Ts = Resistance axial force of Steel under tension
Cs= Resistance axial force of steel under compression

## Appendix A-7: Interaction Calculation 25m Hollow

(Moment in MN-m , Forces in MN)

### Longtudinal Direction

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.021 | -0.0043 | 0.00195 | 0.02 | 28 | 95 | 14.6 | 9.06 | 29.9 | -6.18 | 0.215 | 0.196 | 1.266 | 43.6 |
| 0.205 | 0.0023 | 0.00195 | 0.17 | 34 | 89 | 17.6 | 11.03 | 27.9 | 0.80 | 0.174 | 0.181 | 1.347 | 43.8 |
| 0.410 | 0.0026 | 0.00194 | 0.35 | 41 | 82 | 21.0 | 13.23 | 25.6 | 8.57 | 0.201 | 0.202 | 1.434 | 43.3 |
| 0.615 | 0.0028 | 0.00193 | 0.52 | 48 | 75 | 24.3 | 15.43 | 23.4 | 16.35 | 0.249 | 0.244 | 1.519 | 41.7 |
| 0.820 | 0.0028 | 0.00192 | 0.70 | 55 | 69 | 27.7 | 17.62 | 21.1 | 24.14 | 0.310 | 0.302 | 1.600 | 39.1 |
| 1.025 | 0.0029 | 0.00190 | 0.87 | 62 | 62 | 31.0 | 19.82 | 18.9 | 31.96 | 0.381 | 0.369 | 1.676 | 35.5 |
| 1.230 | 0.0029 | 0.00188 | 1.05 | 69 | 55 | 34.4 | 22.01 | 16.6 | 39.80 | 0.458 | 0.442 | 1.746 | 30.8 |
| 1.435 | 0.0029 | 0.00184 | 1.22 | 75 | 48 | 37.7 | 24.21 | 14.2 | 47.72 | 0.539 | 0.521 | 1.807 | 25.1 |
| 1.640 | 0.0029 | 0.00176 | 1.39 | 82 | 41 | 41.0 | 26.41 | 11.6 | 55.82 | 0.624 | 0.604 | 1.854 | 18.3 |
| 1.845 | 0.0029 | 0.00151 | 1.57 | 89 | 34 | 44.4 | 28.60 | 8.4 | 64.64 | 0.712 | 0.690 | 1.878 | 10.4 |
| 2.030 | 0.0029 | -0.00288 | 1.73 | 95 | 28 | 47.4 | 30.58 | -13.0 | 91.03 | 0.793 | 0.769 | 1.839 | 0.0 |

### Transverse Direction

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.026 | -0.0029 | 0.00196 | 0.02 | 20 | 103 | 10.6 | 6.43 | 32.6 | -15.53 | 0.219 | 0.193 | 1.403 | 44.7 |
| 0.255 | 0.0024 | 0.00196 | 0.22 | 28 | 96 | 14.4 | 8.89 | 30.1 | -6.85 | 0.172 | 0.179 | 1.513 | 46.1 |
| 0.510 | 0.0027 | 0.00195 | 0.43 | 36 | 87 | 18.5 | 11.62 | 27.4 | 2.79 | 0.247 | 0.225 | 1.635 | 46.5 |
| 0.765 | 0.0028 | 0.00194 | 0.65 | 45 | 79 | 22.7 | 14.35 | 24.6 | 12.44 | 0.366 | 0.300 | 1.755 | 45.5 |
| 1.020 | 0.0029 | 0.00193 | 0.87 | 53 | 70 | 26.8 | 17.09 | 21.8 | 22.10 | 0.508 | 0.392 | 1.873 | 43.2 |
| 1.275 | 0.0029 | 0.00192 | 1.08 | 62 | 62 | 31.0 | 19.82 | 19.1 | 31.77 | 0.664 | 0.494 | 1.988 | 39.4 |
| 1.530 | 0.0029 | 0.00190 | 1.30 | 70 | 53 | 35.2 | 22.55 | 16.3 | 41.45 | 0.828 | 0.601 | 2.100 | 34.4 |
| 1.785 | 0.0029 | 0.00187 | 1.52 | 79 | 45 | 39.3 | 25.28 | 13.4 | 51.18 | 0.998 | 0.713 | 2.205 | 27.9 |
| 2.040 | 0.0029 | 0.00180 | 1.73 | 87 | 36 | 43.5 | 28.01 | 10.5 | 61.01 | 1.173 | 0.828 | 2.301 | 20.1 |
| 2.295 | 0.0029 | 0.00161 | 1.95 | 96 | 28 | 47.7 | 30.75 | 7.2 | 71.24 | 1.350 | 0.945 | 2.376 | 10.8 |
| 2.525 | 0.0029 | -0.00192 | 2.15 | 103 | 20 | 51.4 | 33.20 | -6.2 | 90.79 | 1.511 | 1.051 | 2.331 | 0.0 |

**Legend**

C value = location of nutral axis
Strain Cc = Concrete compressive strain (max = 0.0035
Strain Cs = Reinforcement bar strain (max =0.002)

a= depth of section under compression
Cc= Resistance axial force of concrete under compression
Ts = Resistance axial force of Steel under tension
Cs= Resistance axial force of steel under compression

## Appendix A-8: Interaction Calculation 25m Solid

(Moment in MN-m , Forces in MN)

**Longtudinal Direction**

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.020 | -0.0045 | 0.00195 | 0.02 | -13 | 53 | -21.6 | -4.08 | 16.5 | -42.19 | -0.129 | 0.848 | 1.393 | 55.2 |
| 0.200 | 0.0023 | 0.00194 | 0.17 | -7 | 47 | -9.8 | -2.15 | 14.6 | -26.59 | -0.254 | 1.729 | 1.433 | 60.4 |
| 0.400 | 0.0026 | 0.00194 | 0.34 | 1 | 40 | 3.2 | 0.32 | 12.5 | -8.92 | 2.503 | -4.079 | 1.475 | 63.6 |
| 0.600 | 0.0028 | 0.00193 | 0.51 | 7 | 33 | 16.3 | 2.13 | 10.3 | 8.08 | 0.598 | -0.404 | 1.514 | 64.4 |
| 0.800 | 0.0028 | 0.00192 | 0.68 | 13 | 27 | 29.3 | 4.28 | 8.2 | 25.40 | 0.459 | 0.087 | 1.548 | 62.7 |
| 1.000 | 0.0029 | 0.00190 | 0.85 | 20 | 20 | 42.4 | 6.42 | 6.1 | 42.72 | 0.446 | 0.338 | 1.570 | 58.4 |
| 1.200 | 0.0029 | 0.00188 | 1.02 | 27 | 13 | 55.5 | 8.56 | 4.0 | 60.01 | 0.464 | 0.517 | 1.565 | 51.5 |
| 1.400 | 0.0029 | 0.00183 | 1.19 | 33 | 7 | 68.5 | 10.70 | 2.0 | 77.27 | 0.495 | 0.666 | 1.450 | 42.2 |
| 1.600 | 0.0029 | 0.00175 | 1.36 | 40 | 1 | 81.6 | 12.85 | 0.3 | 94.15 | 0.533 | 0.800 | -0.183 | 30.2 |
| 1.800 | 0.0029 | 0.00150 | 1.53 | 47 | -7 | 94.7 | 14.99 | -1.6 | 111.26 | 0.574 | 0.924 | 2.207 | 15.8 |
| 1.980 | 0.0029 | -0.00300 | 1.68 | 53 | -13 | 106.4 | 16.92 | 6.1 | 117.20 | 0.613 | 1.031 | 2.104 | 0.0 |

**Transverse Direction**

| C value | Strain Cc | Strain Cs | a | # of bars in Comp. | # of bars in T | Cc | Cs | Ts | Pn | Rebar Center C | Concrete center C | Rebar Center T | Moment |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.025 | -0.0030 | 0.00196 | 0.02 | -21 | 61 | -37.6 | -6.70 | 19.2 | -63.48 | -0.062 | 0.835 | 1.681 | 59.7 |
| 0.250 | 0.0024 | 0.00196 | 0.21 | -13 | 53 | -22.9 | -4.29 | 16.8 | -43.97 | -0.114 | 1.282 | 1.728 | 69.2 |
| 0.500 | 0.0027 | 0.00195 | 0.43 | -5 | 45 | -6.6 | -1.62 | 14.1 | -22.30 | -0.716 | 3.531 | 1.776 | 76.3 |
| 0.750 | 0.0028 | 0.00194 | 0.64 | 3 | 37 | 9.7 | 1.06 | 11.5 | -0.64 | 2.345 | -1.338 | 1.818 | 79.6 |
| 1.000 | 0.0029 | 0.00193 | 0.85 | 12 | 28 | 26.1 | 3.74 | 8.8 | 21.01 | 1.202 | 0.048 | 1.847 | 79.2 |
| 1.250 | 0.0029 | 0.00192 | 1.06 | 20 | 20 | 42.4 | 6.42 | 6.2 | 42.65 | 1.118 | 0.463 | 1.849 | 75.0 |
| 1.500 | 0.0029 | 0.00190 | 1.28 | 28 | 12 | 58.7 | 9.10 | 3.6 | 64.27 | 1.156 | 0.716 | 1.765 | 67.1 |
| 1.750 | 0.0029 | 0.00187 | 1.49 | 37 | 3 | 75.1 | 11.78 | 1.0 | 85.84 | 1.234 | 0.914 | 0.938 | 55.5 |
| 2.000 | 0.0029 | 0.00180 | 1.70 | 45 | -5 | 91.4 | 14.45 | -1.5 | 107.29 | 1.330 | 1.086 | 3.059 | 40.1 |
| 2.250 | 0.0029 | 0.00160 | 1.91 | 53 | -13 | 107.7 | 17.13 | -3.4 | 128.28 | 1.434 | 1.243 | 2.612 | 20.9 |
| 2.475 | 0.0029 | -0.00200 | 2.10 | 61 | -21 | 122.4 | 19.54 | 6.7 | 135.24 | 1.533 | 1.378 | 2.562 | 0.0 |

**Legend**

C value = location of nutral axis

Strain Cc = Concrete compressive strain (max = 0.0035

Strain Cs = Reinforcement bar strain (max =0.002)

a= depth of section under compression

Cc= Resistance axial force of concrete under compression

Ts = Resistance axial force of Steel under tension

Cs= Resistance axial force of steel under compression

# Appendix B: Sample Shear Reinforcement Design (sample 10meters)

**Shear reinforcement design: 10 meters hollow (or both hollow and Solid sections)**

When the longtudinal reinforcment is distributed along the circumfrance (ACI 318-11)
When a member is subjected to axial, shear and flexure,

| | | | | | | |
|---|---|---|---|---|---|---|
| f'c = | 24 | Mpa | fy = | 420 | | |
| bw = | | 2300 | mm | | | |
| d= | | 1850 | mm | | | |
| Vu = | | 1450.03 | KN | | | |
| Mu = | | 14500.3 | KN-m | | | |
| Nu = | | 14569.72 | KN | | | |
| Av for dia | 14 | 153.86 | sq. mm | | 4 | ties |
| h = | | 10 | m | | | |
| lamda = | | 1 | | | | |
| Ag = | | 3335069 | sq. mm | | | |

| | | | | | |
|---|---|---|---|---|---|
| Vc = | 704.2722 | KN | | | |
| or | | | | | |
| Vc = | 6289.449 | KN | use Vc = | 704.2722 | KN |
| Vs = | 745.7578 | KN | | | |
| S = | 160.3057 | mm | 0.959791232 | mm | |

Weight of rebar /meter hight : 63.21759403 Kg/m

Designing section as a shear wall: (for hollow sections only)

**Design Shear Strength Calculation**

Vu <= factored Vc + factored Vs, Vu = reduction factor *0.83*sqrt (fc') * h*d = (trans. And longt **1.46 MN, and** **1.02 MN**

Nominal Shear Strength Vn at any horizontal section in the plane of the wall may not be taken greater than 10sqert(fc') times the gross area of the cross section

| | | | | | | |
|---|---|---|---|---|---|---|
| **Vc = 2*lamda *sqrt(fc')*h*d =** | 5878.775383 | N | for factored axial load compression ( ACI 11 10 5) | | | |
| **Vc = 2*lamda *sqrt(fc')*h*d =** | 4115.142768 | N | for factored axial load compression ( ACI 11 10 5) | | | |
| or | | | | | | |
| | **Trans. Direction** | | **Longt. Direction** | | | |
| **VC = 0.25* lamda *sqrt(fc')*h*d + Nud/4lw =** | 0.58788046 | MN | 0.411517198 | MN | ACI 11-29 | |
| or Vc = | 3.009855691 | MN | 1.767213158 | MN | ACI 11-30 | |
| **Vc recommended =** | **0.58788046** | **MN** | **0.411517198** | **MN** | | |
| Factored Vc = | 0.440910345 | **MN** | 0.308637899 | **MN** | | |
| Compare ( factored Vc with Vu) = | **Reinf. Needed** | | **Reinf. Needed** | | ACI 11.9.9 | |
| Area of Shear Reinf./ Spacing = | 0.003788536 | m | 0.003788533 | m | | |
| area of reinforcement per 200mm = | 607.3238976 | mm2 | 607.3233401 | mm2 | | |
| | 27.80771121 | mm | 27.80769845 | mm | | |

## Appendix C: Moment Curvature analysis approximate analysis Matlab function

```
function [ phi,M ] = MCurve( b,D,nbar,dia,d)
% written by Eyob, for Masters thesis
% b is the width of a section
% phi is curvature and M is moment
% D is gross depth of a section
% nbar is the number of bars
% dia is the diameter of nominal reinforcement bar
% d is cover concrete thickness
% this program will give out approximate moment curvature diagram along
% with rough moment curvature data
fc=24; e_cu=0.0035;
fy=420; Ey=200000; ey=fy/Ey;
Ec=4700*sqrt(fc);
n=Ey/Ec;
dc=D-d;
As1b = dia^2*pi*0.25;
As=nbar*As1b; r=As/(2*b*d);
Asc=As/2; rc=Asc/(b*d);
% approximating the nurtal axis will devide total bar area into two
M=[]; phi=[];
%onset of cracking
Ig=(b*(D^3))/12; yt=D/2; fr=0.7*sqrt(fc);
Mcr=fr*0.7*Ig/yt; phi_cr=Mcr/(Ec*Ig);
M=[M;Mcr]; phi=[phi; phi_cr];
%onset of yielding
k=sqrt(((r+rc)*n)^2+2*(r+rc*dc/d)*n)-(r+rc)*n;
fsc=((k*d-dc)/(d-k*d))*fy;
My=As*fy*d*(1-k/3)+Asc*fsc*(k*d/3-dc); phi_y=ey/(d-k*d);
M=[M;My]; phi=[phi;phi_y];
%Ultimate
ct=0.5*d; c=0;
while abs(c/ct-1)>0.0002;
e_sc=((ct-dc)/ct)*e_cu;
fsc=Ey*e_sc;
Cs=Asc*fsc; Cc=0.85*0.85*fc*b*ct; T=As*fy;
c=(T-Cs)/(0.85*0.85*fc*b);
ct=(c+ct)/2;
end

Mu=0.85*0.85*fc*c*b*(d-0.85*c/2)+Asc*fsc*(d-dc); phi_u=e_cu/c;
M=[M;Mu]; phi=[phi;phi_u];
%graph
plot(phi,0.001*M);

end
```

## Appendix D-1: Plotted Moment Curvature graph data (case 01-1- hollow)

(Moment KN-m and Force KN)

| Concrete Strain | Neutral Axis | Steel Strain | Concrete Compression | Steel Compression | Steel Tension | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|
| -2.38E-04 | 0 | -2.38E-04 | -10359 | -4210 | 0 | -14569 | 0 | 7.84E-14 |
| -3.01E-04 | -3.5154 | -1.80E-04 | -10360 | -4210 | 0 | -14570 | 6.77E-05 | 1657.807 |
| -3.95E-04 | -1.4062 | -9.26E-05 | -10360 | -4210 | 0 | -14570 | 0.0001692 | 4144.517 |
| -5.18E-04 | -0.7745 | 2.57E-05 | -10397 | -4254 | 80.0346 | -14570 | 0.0003046 | 7349.235 |
| -6.42E-04 | -0.4291 | 2.04E-04 | -10975 | -4459 | 861.6669 | -14572 | 0.0004738 | 9978.671 |
| -7.74E-04 | -0.2177 | 4.34E-04 | -11963 | -4859 | 2252.0921 | -14570 | 0.0006769 | 12549 |
| -9.21E-04 | -0.0831 | 7.09E-04 | -13231 | -5373 | 4030.4369 | -14574 | 0.0009138 | 15407 |
| -1.09E-03 | 8.22E-03 | 1.03E-03 | -14745 | -5986 | 6158.5046 | -14573 | 0.001185 | 18609 |
| -1.27E-03 | 0.0729 | 1.39E-03 | -16492 | -6695 | 8614.8798 | -14572 | 0.001489 | 22179 |
| -1.47E-03 | 0.1196 | 1.79E-03 | -18437 | -7503 | 11370 | -14570 | 0.001828 | 26100 |
| -1.69E-03 | 0.1569 | 2.23E-03 | -20162 | -8371 | 13961 | -14572 | 0.0022 | 29606 |
| -1.90E-03 | 0.1959 | 2.75E-03 | -21357 | -9107 | 15894 | -14570 | 0.002606 | 31962 |
| -2.09E-03 | 0.2374 | 3.34E-03 | -22081 | -9672 | 17182 | -14572 | 0.003046 | 33442 |
| -2.28E-03 | 0.2771 | 4.00E-03 | -22556 | -10138 | 18128 | -14566 | 0.00352 | 34442 |
| -2.49E-03 | 0.3072 | 4.70E-03 | -23193 | -10230 | 18850 | -14574 | 0.004027 | 35113 |
| -2.70E-03 | 0.3336 | 5.45E-03 | -23798 | -10260 | 19488 | -14570 | 0.004569 | 35651 |
| -2.92E-03 | 0.3575 | 6.26E-03 | -24330 | -10258 | 20018 | -14570 | 0.005144 | 36088 |
| -3.15E-03 | 0.3784 | 7.12E-03 | -24795 | -10257 | 20482 | -14570 | 0.005753 | 36443 |
| -3.38E-03 | 0.3966 | 8.03E-03 | -25232 | -10230 | 20892 | -14570 | 0.006396 | 36735 |
| -3.62E-03 | 0.4127 | 8.99E-03 | -25633 | -10186 | 21246 | -14573 | 0.007073 | 36984 |
| -3.87E-03 | 0.4275 | 1.00E-02 | -25980 | -10118 | 21529 | -14570 | 0.007784 | 37186 |
| -4.14E-03 | 0.4396 | 1.11E-02 | -26380 | -10080 | 21890 | -14570 | 0.008529 | 37513 |
| -4.42E-03 | 0.4504 | 1.22E-02 | -26751 | -10054 | 22234 | -14571 | 0.009307 | 37829 |
| -4.71E-03 | 0.4596 | 1.33E-02 | -27118 | -10035 | 22584 | -14570 | 0.0101 | 38163 |
| -5.02E-03 | 0.4675 | 1.45E-02 | -27486 | -10023 | 22939 | -14570 | 0.011 | 38538 |
| -5.34E-03 | 0.4743 | 1.58E-02 | -27861 | -10021 | 23315 | -14567 | 0.0118 | 38927 |
| -5.68E-03 | 0.4802 | 1.71E-02 | -28227 | -10024 | 23681 | -14570 | 0.0128 | 39327 |
| -6.03E-03 | 0.4854 | 1.84E-02 | -28567 | -10027 | 24025 | -14569 | 0.0137 | 39723 |
| -6.39E-03 | 0.4898 | 1.98E-02 | -28907 | -10046 | 24383 | -14570 | 0.0147 | 40130 |
| -6.78E-03 | 0.4934 | 2.12E-02 | -29248 | -10092 | 24770 | -14570 | 0.0157 | 40547 |
| -7.18E-03 | 0.4966 | 2.27E-02 | -29573 | -10151 | 25149 | -14575 | 0.0168 | 40970 |
| -7.59E-03 | 0.4995 | 2.42E-02 | -29868 | -10211 | 25510 | -14570 | 0.0178 | 41388 |
| -8.02E-03 | 0.5017 | 2.58E-02 | -30175 | -10279 | 25880 | -14575 | 0.019 | 41818 |
| -8.46E-03 | 0.504 | 0.0274 | -30436 | -10325 | 26194 | -14568 | 0.0201 | 42146 |
| -8.92E-03 | 0.5059 | 0.0291 | -30689 | -10382 | 26504 | -14567 | 0.0213 | 42462 |
| -9.40E-03 | 0.5075 | 0.0308 | -30930 | -10450 | 26814 | -14567 | 0.0225 | 42773 |
| -9.89E-03 | 0.5087 | 0.0325 | -31165 | -10531 | 27129 | -14567 | 0.0238 | 43085 |
| -1.04E-02 | 0.5098 | 0.0343 | -31375 | -10613 | 27420 | -14568 | 0.025 | 43362 |
| -1.09E-02 | 0.5109 | 0.0361 | -31564 | -10697 | 27693 | -14567 | 0.0264 | 43612 |
| -1.15E-02 | 0.5119 | 0.038 | -31732 | -10784 | 27946 | -14570 | 0.0277 | 43857 |
| -1.20E-02 | 0.5129 | 0.0399 | -31876 | -10874 | 28180 | -14571 | 0.0291 | 44097 |
| -1.25E-02 | 0.5139 | 0.0419 | -31993 | -10993 | 28416 | -14570 | 0.0305 | 44345 |
| -1.31E-02 | 0.5149 | 0.0439 | -32097 | -11115 | 28642 | -14570 | 0.032 | 44571 |
| -1.37E-02 | 0.5157 | 0.046 | -32193 | -11245 | 28867 | -14572 | 0.0335 | 44795 |
| -1.43E-02 | 0.5164 | 0.0481 | -32280 | -11385 | 29090 | -14575 | 0.035 | 45017 |
| -1.49E-02 | 0.517 | 0.0503 | -32355 | -11529 | 29314 | -14569 | 0.0366 | 45234 |
| -1.55E-02 | 0.5175 | 0.0525 | -32424 | -11685 | 29540 | -14569 | 0.0381 | 45453 |
| -1.62E-02 | 0.518 | 0.0548 | -32484 | -11847 | 29763 | -14568 | 0.0398 | 45667 |
| -1.68E-02 | 0.5184 | 0.0571 | -32533 | -12014 | 29981 | -14566 | 0.0414 | 45874 |
| -1.75E-02 | 0.5187 | 0.0594 | -32573 | -12189 | 30192 | -14570 | 0.0431 | 46076 |
| -1.82E-02 | 0.519 | 0.0618 | -32604 | -12368 | 30401 | -14570 | 0.0448 | 46270 |
| -1.89E-02 | 0.5193 | 0.0642 | -32626 | -12553 | 30608 | -14570 | 0.0466 | 46463 |
| -1.96E-02 | 0.5195 | 0.0667 | -32640 | -12747 | 30818 | -14568 | 0.0484 | 46657 |
| -2.04E-02 | 0.5197 | 0.0692 | -32650 | -12952 | 31032 | -14570 | 0.0502 | 46855 |
| -2.11E-02 | 0.5198 | 0.0718 | -32651 | -13160 | 31243 | -14567 | 0.0521 | 47049 |
| -2.19E-02 | 0.5192 | 0.0744 | -32702 | -13308 | 31436 | -14574 | 0.054 | 47242 |
| -2.28E-02 | 0.5178 | 0.077 | -32793 | -13378 | 31600 | -14571 | 0.0559 | 47410 |
| -2.36E-02 | 0.5164 | 0.0796 | -32875 | -13451 | 31761 | -14565 | 0.0579 | 47576 |
| -2.46E-02 | 0.5148 | 0.0822 | -32953 | -13526 | 31914 | -14565 | 0.0599 | 47743 |
| -2.55E-02 | 0.513 | 0.0849 | -33027 | -13604 | 32063 | -14568 | 0.0619 | 47911 |

## Appendix D-2: Plotted Moment Curvature graph data (case 01-2- Solid)

(Moment KN-m and Force KN)

| Concrete Strain | Neutral Axis | Steel Strain | Concrete Compression | Steel Compression | Steel Tension | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|
| -3.08E-04 | 0 | -3.08E-04 | -12869 | -2623 | 0 | -15493 | 0 | 4.62E-14 |
| -3.74E-04 | -3.7104 | -2.47E-04 | -12870 | -2623 | 0 | -15493 | 8.30E-05 | 996.056 |
| -4.74E-04 | -1.4841 | -1.56E-04 | -12870 | -2623 | 0 | -15493 | 0.0002075 | 2490.14 |
| -6.07E-04 | -0.8245 | -3.39E-05 | -12870 | -2623 | 0 | -15493 | 0.0003735 | 4482.2521 |
| -7.60E-04 | -0.5085 | 1.31E-04 | -12977 | -2858 | 341.3054 | -15494 | 0.000581 | 6530.8469 |
| -9.21E-04 | -0.3094 | 3.52E-04 | -13306 | -3200 | 1012.3723 | -15494 | 0.00083 | 8327.9728 |
| -1.09E-03 | -0.1741 | 6.27E-04 | -13832 | -3576 | 1914.0568 | -15494 | 0.00112 | 10103 |
| -1.28E-03 | -0.0776 | 9.53E-04 | -14534 | -3991 | 3031.0622 | -15494 | 0.001452 | 11962 |
| -1.47E-03 | -6.05E-03 | 1.33E-03 | -15400 | -4439 | 4345.2935 | -15494 | 0.001826 | 13957 |
| -1.68E-03 | 0.0483 | 1.75E-03 | -16421 | -4945 | 5867.2415 | -15499 | 0.002241 | 16119 |
| -1.90E-03 | 0.0966 | 2.24E-03 | -17294 | -5419 | 7217.9045 | -15496 | 0.002697 | 18022 |
| -2.09E-03 | 0.1475 | 2.82E-03 | -17557 | -5779 | 7841.5846 | -15495 | 0.003195 | 18935 |
| -2.28E-03 | 0.1905 | 3.45E-03 | -17694 | -6149 | 8350.1079 | -15494 | 0.003735 | 19618 |
| -2.49E-03 | 0.2232 | 4.13E-03 | -17983 | -6243 | 8726.3203 | -15500 | 0.004316 | 20063 |
| -2.72E-03 | 0.2502 | 4.86E-03 | -18269 | -6245 | 9019.2022 | -15495 | 0.004938 | 20376 |
| -2.95E-03 | 0.2733 | 5.64E-03 | -18526 | -6273 | 9305.4201 | -15494 | 0.005602 | 20644 |
| -3.19E-03 | 0.2939 | 6.48E-03 | -18737 | -6295 | 9538.0402 | -15494 | 0.006308 | 20862 |
| -3.44E-03 | 0.312 | 7.38E-03 | -18942 | -6299 | 9746.079 | -15495 | 0.007055 | 21056 |
| -3.70E-03 | 0.3285 | 8.33E-03 | -19117 | -6274 | 9896.9875 | -15494 | 0.007843 | 21210 |
| -3.96E-03 | 0.3429 | 9.34E-03 | -19306 | -6246 | 10058 | -15494 | 0.008673 | 21363 |
| -4.24E-03 | 0.3557 | 0.0104 | -19484 | -6215 | 10205 | -15494 | 0.009545 | 21523 |
| -4.53E-03 | 0.3666 | 0.0115 | -19668 | -6183 | 10357 | -15494 | 0.0105 | 21717 |
| -4.84E-03 | 0.376 | 0.0127 | -19865 | -6186 | 10557 | -15494 | 0.0114 | 21917 |
| -5.16E-03 | 0.3841 | 0.0139 | -20076 | -6190 | 10772 | -15494 | 0.0124 | 22125 |
| -5.50E-03 | 0.391 | 0.0151 | -20302 | -6195 | 10997 | -15500 | 0.0134 | 22339 |
| -5.84E-03 | 0.3977 | 0.0164 | -20489 | -6198 | 11193 | -15494 | 0.0145 | 22544 |
| -6.20E-03 | 0.4037 | 0.0178 | -20658 | -6201 | 11365 | -15494 | 0.0156 | 22746 |
| -6.57E-03 | 0.4089 | 0.0192 | -20836 | -6205 | 11547 | -15494 | 0.0168 | 22952 |
| -6.97E-03 | 0.4133 | 0.0207 | -21022 | -6211 | 11734 | -15499 | 0.018 | 23161 |
| -7.37E-03 | 0.417 | 0.0222 | -21211 | -6218 | 11929 | -15499 | 0.0193 | 23372 |
| -7.80E-03 | 0.4203 | 0.0237 | -21405 | -6227 | 12133 | -15499 | 0.0205 | 23587 |
| -8.25E-03 | 0.4229 | 0.0253 | -21603 | -6237 | 12341 | -15499 | 0.0219 | 23804 |
| -8.70E-03 | 0.4256 | 0.0269 | -21765 | -6247 | 12518 | -15494 | 0.0232 | 23973 |
| -9.17E-03 | 0.428 | 0.0286 | -21916 | -6257 | 12676 | -15497 | 0.0246 | 24112 |
| -9.65E-03 | 0.4302 | 0.0304 | -22061 | -6268 | 12831 | -15497 | 0.0261 | 24244 |
| -0.0102 | 0.432 | 0.0322 | -22204 | -6280 | 12988 | -15496 | 0.0276 | 24374 |
| -0.0107 | 0.4335 | 0.034 | -22343 | -6294 | 13142 | -15495 | 0.0291 | 24498 |
| -0.0112 | 0.4348 | 0.0359 | -22478 | -6310 | 13290 | -15498 | 0.0307 | 24621 |
| -0.0118 | 0.4363 | 0.0378 | -22573 | -6323 | 13402 | -15494 | 0.0323 | 24737 |
| -0.0123 | 0.4376 | 0.0398 | -22666 | -6342 | 13512 | -15495 | 0.034 | 24853 |
| -0.0129 | 0.439 | 0.0419 | -22734 | -6386 | 13625 | -15495 | 0.0357 | 24976 |
| -0.0135 | 0.4404 | 0.044 | -22788 | -6431 | 13728 | -15490 | 0.0374 | 25081 |
| -0.0141 | 0.4416 | 0.0461 | -22842 | -6479 | 13827 | -15494 | 0.0392 | 25187 |
| -0.0147 | 0.4426 | 0.0483 | -22891 | -6528 | 13925 | -15494 | 0.041 | 25289 |
| -0.0153 | 0.4435 | 0.0505 | -22939 | -6580 | 14025 | -15494 | 0.0429 | 25391 |
| -0.0159 | 0.4443 | 0.0528 | -22984 | -6634 | 14126 | -15493 | 0.0448 | 25493 |
| -0.0166 | 0.4449 | 0.0551 | -23029 | -6690 | 14225 | -15494 | 0.0468 | 25595 |
| -0.0173 | 0.4451 | 0.0575 | -23088 | -6730 | 14321 | -15497 | 0.0488 | 25694 |
| -0.018 | 0.4454 | 0.0599 | -23139 | -6771 | 14415 | -15496 | 0.0508 | 25786 |
| -0.0187 | 0.4454 | 0.0623 | -23191 | -6813 | 14509 | -15495 | 0.0529 | 25876 |
| -0.0195 | 0.4454 | 0.0648 | -23243 | -6857 | 14604 | -15495 | 0.055 | 25967 |
| -0.0203 | 0.4452 | 0.0674 | -23293 | -6901 | 14700 | -15495 | 0.0571 | 26057 |
| -0.0211 | 0.445 | 0.07 | -23342 | -6948 | 14794 | -15495 | 0.0593 | 26147 |
| -0.0219 | 0.4448 | 0.0726 | -23385 | -6997 | 14887 | -15495 | 0.0616 | 26236 |
| -0.0227 | 0.4445 | 0.0753 | -23423 | -7049 | 14977 | -15495 | 0.0639 | 26322 |
| -0.0236 | 0.4442 | 0.078 | -23457 | -7102 | 15065 | -15495 | 0.0662 | 26404 |
| -0.0244 | 0.4437 | 0.0807 | -23491 | -7157 | 15154 | -15495 | 0.0686 | 26488 |
| -0.0253 | 0.4433 | 0.0835 | -23525 | -7213 | 15243 | -15495 | 0.071 | 26572 |
| -0.0262 | 0.4427 | 0.0864 | -23557 | -7271 | 15333 | -15495 | 0.0734 | 26656 |
| -0.0272 | 0.4421 | 0.0893 | -23588 | -7330 | 15423 | -15495 | 0.0759 | 26740 |

## Appendix D-3: Plotted Moment Curvature graph data (case 02-1- Hollow)

(Moment KN-m and Force KN)

| Concrete Strain | Neutral Axis | Steel Strain | Concrete Compression | Steel Compression | Steel Tension | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|
| -2.48E-04 | 0 | -2.48E-04 | -10817 | -4791 | 0 | -15609 | 0 | 3.04E-14 |
| -2.91E-04 | -5.7824 | -2.08E-04 | -10819 | -4792 | 0 | -15610 | 4.29E-05 | 1284.2495 |
| -3.55E-04 | -2.313 | -1.48E-04 | -10819 | -4792 | 0 | -15610 | 0.0001072 | 3210.6238 |
| -4.41E-04 | -1.285 | -6.77E-05 | -10819 | -4792 | 0 | -15610 | 0.0001929 | 5779.1228 |
| -5.45E-04 | -0.8166 | 3.52E-05 | -10875 | -4852 | 115.3276 | -15612 | 0.0003001 | 8819.8195 |
| -6.48E-04 | -0.5107 | 1.81E-04 | -11380 | -5030 | 798.6965 | -15612 | 0.0004286 | 11305 |
| -7.56E-04 | -0.306 | 3.63E-04 | -12193 | -5380 | 1956.503 | -15617 | 0.0005787 | 13747 |
| -8.75E-04 | -0.1658 | 5.76E-04 | -13210 | -5822 | 3417.455 | -15614 | 0.0007501 | 16402 |
| -1.01E-03 | -0.0657 | 8.19E-04 | -14415 | -6345 | 5146.659 | -15613 | 0.000943 | 19323 |
| -1.15E-03 | 8.15E-03 | 1.09E-03 | -15794 | -6956 | 7138.434 | -15612 | 0.001157 | 22534 |
| -1.30E-03 | 0.0642 | 1.39E-03 | -17341 | -7622 | 9351.801 | -15611 | 0.001393 | 26044 |
| -1.47E-03 | 0.1078 | 1.72E-03 | -19049 | -8382 | 11820 | -15611 | 0.00165 | 29862 |
| -1.66E-03 | 0.139 | 2.07E-03 | -20792 | -9254 | 14435 | -15611 | 0.001929 | 33835 |
| -1.84E-03 | 0.177 | 2.48E-03 | -21860 | -9934 | 16183 | -15611 | 0.002229 | 36214 |
| -2.01E-03 | 0.2102 | 2.92E-03 | -22744 | -10651 | 17782 | -15613 | 0.00255 | 38274 |
| -2.18E-03 | 0.2479 | 3.42E-03 | -23201 | -11171 | 18759 | -15614 | 0.002893 | 39432 |
| -2.35E-03 | 0.2795 | 3.95E-03 | -23658 | -11511 | 19554 | -15615 | 0.003258 | 40307 |
| -2.53E-03 | 0.3044 | 4.51E-03 | -24194 | -11630 | 20214 | -15611 | 0.003644 | 40928 |
| -2.73E-03 | 0.327 | 5.11E-03 | -24680 | -11758 | 20828 | -15610 | 0.004051 | 41466 |
| -2.92E-03 | 0.3488 | 5.75E-03 | -25087 | -11854 | 21330 | -15610 | 0.004479 | 41891 |
| -3.12E-03 | 0.3673 | 6.41E-03 | -25501 | -11925 | 21815 | -15611 | 0.004929 | 42282 |
| -3.32E-03 | 0.3846 | 7.12E-03 | -25820 | -11969 | 22173 | -15616 | 0.005401 | 42560 |
| -3.54E-03 | 0.3994 | 7.86E-03 | -26134 | -12023 | 22543 | -15614 | 0.005894 | 42827 |
| -3.76E-03 | 0.4136 | 8.64E-03 | -26374 | -12071 | 22835 | -15611 | 0.006408 | 43022 |
| -3.98E-03 | 0.4263 | 9.45E-03 | -26592 | -12125 | 23104 | -15613 | 0.006944 | 43192 |
| -4.22E-03 | 0.4371 | 0.0103 | -26816 | -12197 | 23407 | -15606 | 0.007501 | 43394 |
| -4.47E-03 | 0.4464 | 0.0112 | -27046 | -12304 | 23739 | -15610 | 0.00808 | 43663 |
| -4.73E-03 | 0.4554 | 0.0121 | -27219 | -12393 | 24004 | -15608 | 0.00868 | 43898 |
| -5.00E-03 | 0.4629 | 0.013 | -27395 | -12507 | 24291 | -15612 | 0.009302 | 44157 |
| -5.28E-03 | 0.4686 | 0.0139 | -27591 | -12634 | 24615 | -15611 | 0.009945 | 44462 |
| -5.59E-03 | 0.4733 | 0.0149 | -27780 | -12782 | 24951 | -15611 | 0.0106 | 44776 |
| -5.90E-03 | 0.4773 | 0.0159 | -27950 | -12942 | 25276 | -15616 | 0.0113 | 45094 |
| -6.23E-03 | 0.4809 | 0.017 | -28090 | -13108 | 25586 | -15611 | 0.012 | 45406 |
| -6.57E-03 | 0.4842 | 0.0181 | -28199 | -13279 | 25863 | -15614 | 0.0127 | 45714 |
| -6.92E-03 | 0.487 | 0.0192 | -28290 | -13463 | 26144 | -15609 | 0.0135 | 46025 |
| -7.28E-03 | 0.4892 | 0.0203 | -28371 | -13670 | 26430 | -15611 | 0.0143 | 46343 |
| -7.71E-03 | 0.4873 | 0.0214 | -28548 | -13734 | 26671 | -15611 | 0.015 | 46625 |
| -8.18E-03 | 0.484 | 0.0225 | -28723 | -13795 | 26905 | -15614 | 0.0159 | 46900 |
| -8.68E-03 | 0.4801 | 0.0236 | -28875 | -13869 | 27131 | -15613 | 0.0167 | 47166 |
| -9.21E-03 | 0.4754 | 0.0247 | -29003 | -13957 | 27346 | -15613 | 0.0176 | 47420 |
| -9.77E-03 | 0.47 | 0.0259 | -29105 | -14058 | 27551 | -15612 | 0.0184 | 47664 |
| -0.0103 | 0.4652 | 0.027 | -29158 | -14161 | 27707 | -15612 | 0.0193 | 47825 |
| -0.0109 | 0.4595 | 0.0282 | -29181 | -14280 | 27845 | -15616 | 0.0203 | 47969 |
| -0.0116 | 0.4524 | 0.0294 | -29184 | -14376 | 27947 | -15613 | 0.0212 | 48087 |
| -0.0123 | 0.4456 | 0.0306 | -29153 | -14505 | 28047 | -15611 | 0.0222 | 48216 |
| -0.013 | 0.4385 | 0.0318 | -29100 | -14644 | 28134 | -15610 | 0.0231 | 48338 |
| -0.0137 | 0.4307 | 0.033 | -29027 | -14819 | 28235 | -15611 | 0.0242 | 48444 |
| -0.0146 | 0.421 | 0.0341 | -28940 | -14973 | 28302 | -15612 | 0.0252 | 48495 |
| -0.0154 | 0.4116 | 0.0353 | -28826 | -15138 | 28354 | -15611 | 0.0262 | 48529 |
| -0.0163 | 0.4021 | 0.0365 | -28694 | -15326 | 28409 | -15611 | 0.0273 | 48559 |
| -0.0172 | 0.3926 | 0.0377 | -28539 | -15603 | 28532 | -15610 | 0.0284 | 48581 |
| -0.0182 | 0.3834 | 0.0389 | -28368 | -15891 | 28652 | -15608 | 0.0295 | 48598 |
| -0.0191 | 0.3753 | 0.0401 | -28186 | -16171 | 28741 | -15616 | 0.0306 | 48625 |
| -0.0201 | 0.3673 | 0.0414 | -27995 | -16428 | 28808 | -15615 | 0.0318 | 48646 |
| -0.0212 | 0.3581 | 0.0426 | -27805 | -16648 | 28842 | -15611 | 0.033 | 48642 |
| -0.0222 | 0.3495 | 0.0439 | -27611 | -16869 | 28873 | -15607 | 0.0342 | 48636 |
| -0.0233 | 0.3424 | 0.0452 | -27409 | -17072 | 28870 | -15610 | 0.0354 | 48647 |
| -0.0244 | 0.3347 | 0.0465 | -27208 | -17230 | 28826 | -15611 | 0.0366 | 48652 |
| -0.0255 | 0.3273 | 0.0478 | -27003 | -17390 | 28782 | -15611 | 0.0379 | 48662 |
| -0.0266 | 0.3204 | 0.0492 | -26798 | -17552 | 28739 | -15610 | 0.0392 | 48679 |

## Appendix D-4: Plotted Moment Curvature graph data (case 02-2- Solid)

(Moment KN-m and Force KN)

| Concrete Strain | Neutral Axis | Steel Strain | Concrete Compression | Steel Compression | Steel Tension | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|
| -3.03E-04 | 0 | -3.03E-04 | -13300 | -2580 | 0 | -15880 | 0 | 3.24E-14 |
| -3.64E-04 | -3.9915 | -2.47E-04 | -13300 | -2581 | 0 | -15881 | 7.59E-05 | 944.8403 |
| -4.55E-04 | -1.5966 | -1.64E-04 | -13301 | -2581 | 0 | -15881 | 0.0001897 | 2362.1008 |
| -5.76E-04 | -0.887 | -5.23E-05 | -13301 | -2581 | 0 | -15881 | 0.0003415 | 4251.7814 |
| -7.20E-04 | -0.5551 | 9.50E-05 | -13373 | -2755 | 243.1902 | -15885 | 0.0005312 | 6308.6041 |
| -8.70E-04 | -0.346 | 2.94E-04 | -13645 | -3067 | 830.1473 | -15882 | 0.0007589 | 8080.3912 |
| -1.03E-03 | -0.2035 | 5.43E-04 | -14106 | -3413 | 1636.8897 | -15882 | 0.001024 | 9789.7114 |
| -1.20E-03 | -0.1013 | 8.40E-04 | -14735 | -3789 | 2642.2536 | -15881 | 0.001328 | 11551 |
| -1.38E-03 | -0.0254 | 1.18E-03 | -15520 | -4202 | 3840.7331 | -15881 | 0.00167 | 13423 |
| -1.57E-03 | 0.0325 | 1.57E-03 | -16454 | -4654 | 5221.2943 | -15887 | 0.002049 | 15439 |
| -1.78E-03 | 0.0767 | 2.00E-03 | -17492 | -5156 | 6766.8219 | -15882 | 0.002466 | 17571 |
| -1.98E-03 | 0.1229 | 2.50E-03 | -17854 | -5555 | 7522.2308 | -15886 | 0.002922 | 18648 |
| -2.17E-03 | 0.1645 | 3.07E-03 | -17969 | -5958 | 8044.7259 | -15882 | 0.003415 | 19335 |
| -2.37E-03 | 0.1989 | 3.68E-03 | -18075 | -6270 | 8463.1517 | -15881 | 0.003946 | 19855 |
| -2.60E-03 | 0.2241 | 4.33E-03 | -18381 | -6291 | 8790.7362 | -15882 | 0.004515 | 20191 |
| -2.83E-03 | 0.2467 | 5.02E-03 | -18618 | -6306 | 9041.7486 | -15881 | 0.005122 | 20451 |
| -3.08E-03 | 0.2667 | 5.77E-03 | -18828 | -6346 | 9292.2002 | -15882 | 0.005768 | 20682 |
| -3.33E-03 | 0.2845 | 6.57E-03 | -19015 | -6369 | 9502.3796 | -15882 | 0.006451 | 20877 |
| -3.59E-03 | 0.2996 | 7.41E-03 | -19213 | -6366 | 9697.3673 | -15882 | 0.007171 | 21042 |
| -3.86E-03 | 0.3133 | 8.30E-03 | -19349 | -6359 | 9826.4466 | -15881 | 0.00793 | 21158 |
| -4.15E-03 | 0.3251 | 9.24E-03 | -19491 | -6351 | 9961.5391 | -15881 | 0.008727 | 21266 |
| -4.45E-03 | 0.3348 | 0.0102 | -19657 | -6347 | 10116 | -15888 | 0.009562 | 21387 |
| -4.76E-03 | 0.3437 | 0.0112 | -19799 | -6339 | 10257 | -15882 | 0.0104 | 21539 |
| -5.09E-03 | 0.3513 | 0.0123 | -19947 | -6333 | 10399 | -15881 | 0.0113 | 21698 |
| -5.44E-03 | 0.3576 | 0.0134 | -20101 | -6330 | 10550 | -15881 | 0.0123 | 21856 |
| -5.81E-03 | 0.3628 | 0.0146 | -20264 | -6331 | 10708 | -15887 | 0.0133 | 22017 |
| -6.19E-03 | 0.3671 | 0.0157 | -20429 | -6335 | 10877 | -15887 | 0.0143 | 22179 |
| -6.60E-03 | 0.3706 | 0.017 | -20594 | -6360 | 11069 | -15885 | 0.0154 | 22341 |
| -7.02E-03 | 0.3736 | 0.0182 | -20745 | -6386 | 11250 | -15882 | 0.0165 | 22500 |
| -7.46E-03 | 0.3763 | 0.0195 | -20870 | -6414 | 11398 | -15886 | 0.0176 | 22655 |
| -7.92E-03 | 0.3786 | 0.0209 | -20990 | -6440 | 11549 | -15882 | 0.0188 | 22807 |
| -8.40E-03 | 0.3797 | 0.0223 | -21143 | -6440 | 11697 | -15886 | 0.02 | 22959 |
| -8.92E-03 | 0.3803 | 0.0237 | -21296 | -6440 | 11851 | -15886 | 0.0212 | 23110 |
| -9.46E-03 | 0.3804 | 0.0251 | -21451 | -6440 | 12006 | -15885 | 0.0225 | 23261 |
| -0.01 | 0.3804 | 0.0266 | -21585 | -6440 | 12139 | -15886 | 0.0239 | 23384 |
| -0.0106 | 0.3806 | 0.0281 | -21685 | -6440 | 12240 | -15885 | 0.0252 | 23462 |
| -0.0112 | 0.3804 | 0.0297 | -21783 | -6440 | 12339 | -15884 | 0.0266 | 23537 |
| -0.0118 | 0.3799 | 0.0313 | -21879 | -6440 | 12435 | -15884 | 0.0281 | 23606 |
| -0.0124 | 0.3796 | 0.0329 | -21945 | -6474 | 12536 | -15882 | 0.0296 | 23686 |
| -0.0131 | 0.3792 | 0.0346 | -22002 | -6513 | 12632 | -15883 | 0.0311 | 23763 |
| -0.0138 | 0.3786 | 0.0363 | -22055 | -6553 | 12725 | -15883 | 0.0326 | 23838 |
| -0.0145 | 0.3778 | 0.038 | -22105 | -6596 | 12818 | -15883 | 0.0342 | 23913 |
| -0.0152 | 0.3768 | 0.0398 | -22149 | -6639 | 12906 | -15883 | 0.0359 | 23985 |
| -0.0159 | 0.3757 | 0.0416 | -22188 | -6685 | 12990 | -15883 | 0.0375 | 24054 |
| -0.0167 | 0.3745 | 0.0435 | -22216 | -6731 | 13064 | -15883 | 0.0392 | 24113 |
| -0.0175 | 0.3733 | 0.0454 | -22240 | -6780 | 13138 | -15882 | 0.041 | 24169 |
| -0.0183 | 0.372 | 0.0473 | -22259 | -6832 | 13208 | -15882 | 0.0428 | 24226 |
| -0.0191 | 0.3707 | 0.0492 | -22269 | -6887 | 13274 | -15882 | 0.0446 | 24279 |
| -0.02 | 0.3692 | 0.0512 | -22275 | -6945 | 13338 | -15882 | 0.0464 | 24333 |
| -0.0209 | 0.3677 | 0.0533 | -22278 | -7004 | 13401 | -15882 | 0.0483 | 24388 |
| -0.0218 | 0.3662 | 0.0553 | -22277 | -7078 | 13473 | -15883 | 0.0503 | 24443 |
| -0.0228 | 0.3646 | 0.0574 | -22271 | -7169 | 13558 | -15882 | 0.0522 | 24498 |
| -0.0237 | 0.363 | 0.0595 | -22257 | -7263 | 13637 | -15882 | 0.0543 | 24548 |
| -0.0247 | 0.3614 | 0.0617 | -22235 | -7360 | 13713 | -15882 | 0.0563 | 24595 |
| -0.0257 | 0.3599 | 0.0639 | -22210 | -7461 | 13789 | -15882 | 0.0584 | 24642 |
| -0.0267 | 0.3583 | 0.0661 | -22181 | -7567 | 13865 | -15882 | 0.0605 | 24690 |
| -0.0278 | 0.3567 | 0.0684 | -22147 | -7676 | 13942 | -15882 | 0.0627 | 24739 |
| -0.0289 | 0.3551 | 0.0707 | -22111 | -7790 | 14019 | -15882 | 0.0649 | 24789 |
| -0.03 | 0.3535 | 0.073 | -22071 | -7908 | 14098 | -15881 | 0.0671 | 24841 |
| -0.0311 | 0.3517 | 0.0753 | -22036 | -8015 | 14174 | -15878 | 0.0694 | 24880 |

# Appendix D-5: Plotted Moment Curvature graph data (case 03-1- Hollow)

(Moment KN-m and Force KN)

| Concrete Strain | Neutral Axis | Steel Strain | Concrete Compression | Steel Compression | Steel Tension | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|
| -2.44E-04 | 0 | -2.44E-04 | -11151 | -4789 | 0 | -15940 | 0 | 6.84E-14 |
| -2.80E-04 | -6.8553 | -2.10E-04 | -11152 | -4790 | 0 | -15941 | 3.55E-05 | 1169.737 |
| -3.35E-04 | -2.7421 | -1.58E-04 | -11152 | -4790 | 0 | -15941 | 8.89E-05 | 2924.342 |
| -4.08E-04 | -1.5234 | -9.03E-05 | -11152 | -4790 | 0 | -15941 | 0.0001599 | 5263.816 |
| -4.99E-04 | -0.9787 | -4.94E-06 | -11155 | -4787 | 0 | -15942 | 0.0002488 | 8177.619 |
| -5.93E-04 | -0.6434 | 1.12E-04 | -11448 | -4908 | 412.4441 | -15943 | 0.0003554 | 10818 |
| -6.87E-04 | -0.4062 | 2.65E-04 | -12110 | -5180 | 1348.5668 | -15942 | 0.0004798 | 13105 |
| -7.89E-04 | -0.2432 | 4.45E-04 | -12971 | -5538 | 2563.9771 | -15945 | 0.0006219 | 15542 |
| -9.00E-04 | -0.1262 | 6.51E-04 | -14004 | -5971 | 4030.3095 | -15944 | 0.0007818 | 18193 |
| -1.02E-03 | -0.0395 | 8.82E-04 | -15197 | -6473 | 5727.4659 | -15943 | 0.0009595 | 21090 |
| -1.15E-03 | 0.0264 | 1.14E-03 | -16543 | -7041 | 7640.7255 | -15943 | 0.001155 | 24250 |
| -1.30E-03 | 0.0778 | 1.42E-03 | -18034 | -7669 | 9761.3978 | -15942 | 0.001368 | 27679 |
| -1.45E-03 | 0.118 | 1.72E-03 | -19656 | -8363 | 12074 | -15945 | 0.001599 | 31367 |
| -1.62E-03 | 0.1461 | 2.04E-03 | -21251 | -9195 | 14504 | -15941 | 0.001848 | 35125 |
| -1.79E-03 | 0.1797 | 2.41E-03 | -22278 | -9881 | 16215 | -15943 | 0.002115 | 37530 |
| -1.96E-03 | 0.2087 | 2.80E-03 | -23161 | -10606 | 17823 | -15944 | 0.002399 | 39679 |
| -2.11E-03 | 0.2443 | 3.25E-03 | -23593 | -11112 | 18764 | -15942 | 0.002701 | 40845 |
| -2.26E-03 | 0.277 | 3.73E-03 | -23932 | -11602 | 19599 | -15936 | 0.003021 | 41816 |
| -2.43E-03 | 0.3009 | 4.23E-03 | -24426 | -11768 | 20251 | -15943 | 0.003358 | 42471 |
| -2.61E-03 | 0.3227 | 4.76E-03 | -24874 | -11911 | 20843 | -15942 | 0.003714 | 43016 |
| -2.79E-03 | 0.3431 | 5.32E-03 | -25274 | -12029 | 21356 | -15948 | 0.004087 | 43483 |
| -2.97E-03 | 0.3616 | 5.91E-03 | -25630 | -12119 | 21807 | -15942 | 0.004478 | 43865 |
| -3.16E-03 | 0.3778 | 6.53E-03 | -25939 | -12220 | 22218 | -15942 | 0.004887 | 44178 |
| -3.36E-03 | 0.3924 | 7.18E-03 | -26207 | -12319 | 22585 | -15941 | 0.005313 | 44440 |
| -3.57E-03 | 0.4058 | 7.86E-03 | -26431 | -12410 | 22896 | -15945 | 0.005757 | 44654 |
| -3.78E-03 | 0.418 | 8.56E-03 | -26618 | -12495 | 23172 | -15941 | 0.006219 | 44831 |
| -3.99E-03 | 0.4289 | 9.30E-03 | -26787 | -12607 | 23452 | -15942 | 0.006699 | 44994 |
| -4.22E-03 | 0.4393 | 0.0101 | -26920 | -12708 | 23685 | -15943 | 0.007197 | 45128 |
| -4.46E-03 | 0.4472 | 0.0108 | -27075 | -12844 | 23977 | -15941 | 0.007712 | 45358 |
| -4.71E-03 | 0.4544 | 0.0117 | -27197 | -12967 | 24222 | -15942 | 0.008245 | 45563 |
| -4.97E-03 | 0.4604 | 0.0125 | -27308 | -13107 | 24471 | -15944 | 0.008796 | 45771 |
| -5.24E-03 | 0.4651 | 0.0133 | -27415 | -13277 | 24745 | -15947 | 0.009364 | 46020 |
| -5.53E-03 | 0.4694 | 0.0142 | -27494 | -13451 | 25002 | -15942 | 0.009951 | 46272 |
| -5.83E-03 | 0.4729 | 0.0151 | -27561 | -13648 | 25268 | -15941 | 0.0106 | 46532 |
| -6.14E-03 | 0.4755 | 0.016 | -27613 | -13873 | 25539 | -15948 | 0.0112 | 46799 |
| -6.51E-03 | 0.474 | 0.0169 | -27733 | -13977 | 25769 | -15942 | 0.0118 | 47022 |
| -6.91E-03 | 0.471 | 0.0178 | -27846 | -14056 | 25960 | -15942 | 0.0125 | 47224 |
| -7.34E-03 | 0.467 | 0.0187 | -27933 | -14154 | 26142 | -15946 | 0.0131 | 47416 |
| -7.79E-03 | 0.4619 | 0.0197 | -27990 | -14272 | 26316 | -15946 | 0.0138 | 47593 |
| -8.28E-03 | 0.456 | 0.0206 | -28015 | -14405 | 26479 | -15942 | 0.0146 | 47754 |
| -8.83E-03 | 0.4471 | 0.0215 | -28027 | -14520 | 26602 | -15944 | 0.0153 | 47878 |
| -9.42E-03 | 0.4372 | 0.0224 | -27994 | -14654 | 26705 | -15943 | 0.016 | 47977 |
| -0.01 | 0.4266 | 0.0233 | -27925 | -14807 | 26790 | -15942 | 0.0168 | 48055 |
| -0.0107 | 0.4146 | 0.0241 | -27827 | -14975 | 26858 | -15944 | 0.0176 | 48105 |
| -0.0115 | 0.4016 | 0.025 | -27692 | -15172 | 26916 | -15948 | 0.0184 | 48131 |
| -0.0122 | 0.39 | 0.0259 | -27517 | -15409 | 26984 | -15942 | 0.0192 | 48177 |
| -0.0129 | 0.3795 | 0.0268 | -27315 | -15644 | 27017 | -15941 | 0.02 | 48170 |
| -0.0137 | 0.3669 | 0.0277 | -27105 | -15835 | 26997 | -15943 | 0.0209 | 48123 |
| -0.0146 | 0.3539 | 0.0286 | -26875 | -16064 | 26994 | -15945 | 0.0217 | 48064 |
| -0.0155 | 0.3417 | 0.0294 | -26634 | -16349 | 27039 | -15944 | 0.0226 | 48005 |
| -0.0164 | 0.3297 | 0.0303 | -26384 | -16636 | 27078 | -15942 | 0.0235 | 47944 |
| -0.0173 | 0.3165 | 0.0312 | -26131 | -16845 | 27030 | -15946 | 0.0245 | 47856 |
| -0.0183 | 0.3049 | 0.0321 | -25870 | -17052 | 26984 | -15937 | 0.0254 | 47781 |
| -0.0193 | 0.2939 | 0.033 | -25609 | -17314 | 26982 | -15941 | 0.0264 | 47716 |
| -0.0203 | 0.2839 | 0.034 | -25341 | -17635 | 27031 | -15945 | 0.0273 | 47670 |
| -0.0213 | 0.2728 | 0.0349 | -25076 | -17900 | 27040 | -15936 | 0.0283 | 47599 |
| -0.0224 | 0.2624 | 0.0359 | -24811 | -18169 | 27041 | -15939 | 0.0294 | 47532 |
| -0.0235 | 0.2529 | 0.0368 | -24548 | -18441 | 27049 | -15941 | 0.0304 | 47481 |
| -0.0245 | 0.2447 | 0.0378 | -24290 | -18706 | 27057 | -15939 | 0.0314 | 47443 |
| -0.0256 | 0.2382 | 0.0389 | -24028 | -18970 | 27063 | -15935 | 0.0325 | 47423 |

## Appendix D-6: Plotted Moment Curvature graph data (case 03-2- Solid)

(Moment KN-m and Force KN)

| Concrete Strain | Neutral Axis | Steel Strain | Concrete Compression | Steel Compression | Steel Tension | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|
| -2.22E-04 | 0 | -2.22E-04 | -14565 | -2331 | 0 | -16896 | 0 | 1.47E-14 |
| -2.80E-04 | -3.6629 | -1.69E-04 | -14565 | -2331 | 0 | -16896 | 6.07E-05 | 1527.0047 |
| -3.66E-04 | -1.4651 | -8.82E-05 | -14565 | -2331 | 0 | -16896 | 0.0001517 | 3817.5116 |
| -4.81E-04 | -0.81 | 2.02E-05 | -14584 | -2379 | 59.4692 | -16903 | 0.0002731 | 6791.2226 |
| -6.04E-04 | -0.4723 | 1.75E-04 | -14794 | -2693 | 588.9904 | -16898 | 0.0004248 | 9390.2975 |
| -7.34E-04 | -0.259 | 3.79E-04 | -15250 | -3036 | 1388.122 | -16898 | 0.0006069 | 11719 |
| -8.72E-04 | -0.1138 | 6.31E-04 | -15920 | -3414 | 2436.637 | -16897 | 0.0008193 | 14055 |
| -1.02E-03 | -9.91E-03 | 9.28E-04 | -16786 | -3828 | 3717.785 | -16897 | 0.001062 | 16521 |
| -1.18E-03 | 0.0671 | 1.27E-03 | -17836 | -4277 | 5216.516 | -16897 | 0.001335 | 19179 |
| -1.35E-03 | 0.1257 | 1.65E-03 | -19064 | -4777 | 6937.123 | -16904 | 0.001639 | 22067 |
| -1.53E-03 | 0.1721 | 2.08E-03 | -20417 | -5300 | 8818.966 | -16899 | 0.001972 | 25140 |
| -1.69E-03 | 0.2253 | 2.59E-03 | -20795 | -5706 | 9602.33 | -16899 | 0.002337 | 26398 |
| -1.86E-03 | 0.2703 | 3.15E-03 | -21013 | -6105 | 10219 | -16898 | 0.002731 | 27331 |
| -2.02E-03 | 0.309 | 3.76E-03 | -21118 | -6509 | 10730 | -16898 | 0.003156 | 28060 |
| -2.19E-03 | 0.3428 | 4.43E-03 | -21139 | -6927 | 11170 | -16896 | 0.003611 | 28651 |
| -2.37E-03 | 0.3717 | 5.14E-03 | -21168 | -7269 | 11532 | -16906 | 0.004097 | 29127 |
| -2.57E-03 | 0.3929 | 5.89E-03 | -21437 | -7288 | 11827 | -16898 | 0.004612 | 29427 |
| -2.77E-03 | 0.4124 | 6.69E-03 | -21641 | -7297 | 12041 | -16897 | 0.005159 | 29665 |
| -2.99E-03 | 0.4296 | 7.53E-03 | -21848 | -7334 | 12285 | -16897 | 0.005735 | 29895 |
| -3.20E-03 | 0.4454 | 8.43E-03 | -21996 | -7365 | 12464 | -16897 | 0.006342 | 30071 |
| -3.43E-03 | 0.4585 | 9.37E-03 | -22169 | -7377 | 12649 | -16897 | 0.006979 | 30226 |
| -3.67E-03 | 0.47 | 0.0104 | -22336 | -7377 | 12816 | -16897 | 0.007647 | 30390 |
| -3.93E-03 | 0.4795 | 0.0114 | -22530 | -7379 | 13012 | -16897 | 0.008345 | 30619 |
| -4.20E-03 | 0.4876 | 0.0124 | -22736 | -7384 | 13223 | -16896 | 0.009073 | 30858 |
| -4.48E-03 | 0.4943 | 0.0135 | -22960 | -7391 | 13447 | -16905 | 0.009832 | 31110 |
| -4.77E-03 | 0.5005 | 0.0147 | -23150 | -7397 | 13650 | -16897 | 0.0106 | 31349 |
| -5.08E-03 | 0.5058 | 0.0159 | -23321 | -7404 | 13829 | -16896 | 0.0114 | 31584 |
| -5.41E-03 | 0.5102 | 0.0171 | -23504 | -7416 | 14015 | -16904 | 0.0123 | 31826 |
| -5.74E-03 | 0.5138 | 0.0184 | -23686 | -7430 | 14212 | -16904 | 0.0132 | 32069 |
| -6.10E-03 | 0.5167 | 0.0197 | -23868 | -7448 | 14415 | -16902 | 0.0141 | 32313 |
| -6.48E-03 | 0.5189 | 0.0211 | -24052 | -7471 | 14625 | -16899 | 0.015 | 32559 |
| -6.87E-03 | 0.5205 | 0.0225 | -24238 | -7500 | 14841 | -16896 | 0.016 | 32806 |
| -7.29E-03 | 0.5211 | 0.0239 | -24446 | -7509 | 15058 | -16897 | 0.017 | 33052 |
| -7.73E-03 | 0.5209 | 0.0253 | -24669 | -7513 | 15280 | -16902 | 0.018 | 33301 |
| -8.19E-03 | 0.5208 | 0.0268 | -24856 | -7517 | 15469 | -16904 | 0.0191 | 33492 |
| -8.65E-03 | 0.5213 | 0.0284 | -24973 | -7517 | 15592 | -16897 | 0.0202 | 33623 |
| -9.13E-03 | 0.5213 | 0.0299 | -25091 | -7520 | 15709 | -16902 | 0.0213 | 33748 |
| -9.64E-03 | 0.5209 | 0.0315 | -25208 | -7526 | 15831 | -16903 | 0.0225 | 33870 |
| -0.0102 | 0.5202 | 0.0332 | -25318 | -7535 | 15951 | -16903 | 0.0236 | 33983 |
| -0.0107 | 0.5193 | 0.0349 | -25427 | -7547 | 16072 | -16902 | 0.0249 | 34093 |
| -0.0113 | 0.5181 | 0.0366 | -25527 | -7562 | 16189 | -16901 | 0.0261 | 34195 |
| -0.0119 | 0.5167 | 0.0383 | -25621 | -7584 | 16307 | -16897 | 0.0274 | 34296 |
| -0.0125 | 0.5155 | 0.0401 | -25681 | -7644 | 16426 | -16899 | 0.0287 | 34411 |
| -0.0131 | 0.5142 | 0.042 | -25735 | -7709 | 16545 | -16899 | 0.03 | 34523 |
| -0.0137 | 0.5128 | 0.0438 | -25776 | -7777 | 16654 | -16899 | 0.0314 | 34615 |
| -0.0144 | 0.5113 | 0.0457 | -25811 | -7850 | 16762 | -16899 | 0.0328 | 34705 |
| -0.0151 | 0.5097 | 0.0477 | -25838 | -7927 | 16867 | -16899 | 0.0342 | 34790 |
| -0.0158 | 0.5079 | 0.0496 | -25862 | -8009 | 16972 | -16898 | 0.0357 | 34875 |
| -0.0165 | 0.506 | 0.0516 | -25881 | -8097 | 17079 | -16898 | 0.0371 | 34960 |
| -0.0172 | 0.5041 | 0.0537 | -25893 | -8190 | 17185 | -16898 | 0.0387 | 35044 |
| -0.018 | 0.5021 | 0.0557 | -25896 | -8291 | 17289 | -16898 | 0.0402 | 35127 |
| -0.0188 | 0.5 | 0.0578 | -25894 | -8397 | 17393 | -16898 | 0.0418 | 35210 |
| -0.0196 | 0.498 | 0.06 | -25882 | -8506 | 17490 | -16898 | 0.0434 | 35283 |
| -0.0204 | 0.496 | 0.0621 | -25863 | -8620 | 17588 | -16895 | 0.045 | 35355 |
| -0.0213 | 0.494 | 0.0643 | -25838 | -8738 | 17687 | -16889 | 0.0467 | 35426 |
| -0.0222 | 0.4919 | 0.0666 | -25815 | -8863 | 17781 | -16897 | 0.0484 | 35502 |
| -0.0231 | 0.4898 | 0.0689 | -25779 | -8990 | 17876 | -16893 | 0.0501 | 35573 |
| -0.024 | 0.4869 | 0.0711 | -25785 | -9081 | 17969 | -16897 | 0.0519 | 35643 |
| -0.025 | 0.4836 | 0.0734 | -25797 | -9163 | 18063 | -16898 | 0.0537 | 35709 |
| -0.0261 | 0.4803 | 0.0757 | -25802 | -9247 | 18152 | -16897 | 0.0555 | 35770 |

## Appendix D-7: Plotted Moment Curvature graph data (case 04-1- Hollow)

(Moment KN-m and Force KN)

| Concrete Strain | Neutral Axis | Steel Strain | Concrete Compression | Steel Compression | Steel Tension | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|
| -2.43E-04 | 0 | -2.43E-04 | -11385 | -4938 | 0 | -16323 | 0 | 1.67E-13 |
| -2.77E-04 | -7.3608 | -2.11E-04 | -11386 | -4938 | 0 | -16325 | 3.30E-05 | 1127.0916 |
| -3.28E-04 | -2.9443 | -1.64E-04 | -11386 | -4938 | 0 | -16325 | 8.26E-05 | 2817.7289 |
| -3.95E-04 | -1.6357 | -1.01E-04 | -11386 | -4938 | 0 | -16325 | 0.0001486 | 5071.912 |
| -4.80E-04 | -1.0515 | -2.14E-05 | -11386 | -4938 | 0 | -16325 | 0.0002312 | 7889.6409 |
| -5.72E-04 | -0.7065 | 8.33E-05 | -11586 | -5042 | 302.426 | -16325 | 0.0003302 | 10668 |
| -6.60E-04 | -0.4563 | 2.24E-04 | -12192 | -5271 | 1138.131 | -16325 | 0.0004458 | 12947 |
| -7.56E-04 | -0.2831 | 3.91E-04 | -13004 | -5613 | 2289.505 | -16328 | 0.0005779 | 15330 |
| -8.60E-04 | -0.1587 | 5.81E-04 | -13985 | -6029 | 3687.408 | -16327 | 0.0007265 | 17910 |
| -9.73E-04 | -0.0664 | 7.96E-04 | -15123 | -6513 | 5309.202 | -16327 | 0.0008916 | 20722 |
| -1.10E-03 | 3.81E-03 | 1.03E-03 | -16409 | -7060 | 7143.192 | -16326 | 0.001073 | 23784 |
| -1.23E-03 | 0.0585 | 1.29E-03 | -17836 | -7669 | 9179.597 | -16325 | 0.001271 | 27106 |
| -1.37E-03 | 0.1019 | 1.58E-03 | -19401 | -8339 | 11415 | -16325 | 0.001486 | 30694 |
| -1.53E-03 | 0.135 | 1.88E-03 | -21039 | -9096 | 13805 | -16331 | 0.001717 | 34467 |
| -1.69E-03 | 0.1637 | 2.21E-03 | -22389 | -9881 | 15944 | -16327 | 0.001965 | 37674 |
| -1.85E-03 | 0.1943 | 2.57E-03 | -23319 | -10584 | 17578 | -16325 | 0.002229 | 39904 |
| -2.01E-03 | 0.2245 | 2.97E-03 | -23991 | -11232 | 18897 | -16326 | 0.00251 | 41629 |
| -2.15E-03 | 0.2577 | 3.42E-03 | -24356 | -11740 | 19767 | -16330 | 0.002807 | 42680 |
| -2.30E-03 | 0.2871 | 3.89E-03 | -24679 | -12162 | 20511 | -16329 | 0.003121 | 43518 |
| -2.47E-03 | 0.3088 | 4.38E-03 | -25129 | -12313 | 21117 | -16325 | 0.003451 | 44087 |
| -2.64E-03 | 0.329 | 4.89E-03 | -25523 | -12457 | 21656 | -16325 | 0.003798 | 44568 |
| -2.82E-03 | 0.3478 | 5.44E-03 | -25879 | -12584 | 22134 | -16329 | 0.004161 | 44990 |
| -3.00E-03 | 0.3646 | 6.01E-03 | -26194 | -12686 | 22555 | -16325 | 0.004541 | 45332 |
| -3.19E-03 | 0.3794 | 6.61E-03 | -26459 | -12803 | 22937 | -16325 | 0.004937 | 45609 |
| -3.38E-03 | 0.3926 | 7.23E-03 | -26683 | -12920 | 23278 | -16325 | 0.00535 | 45840 |
| -3.59E-03 | 0.4047 | 7.88E-03 | -26863 | -13034 | 23569 | -16328 | 0.005779 | 46026 |
| -3.79E-03 | 0.4157 | 8.56E-03 | -27005 | -13147 | 23827 | -16325 | 0.006225 | 46176 |
| -4.01E-03 | 0.4254 | 9.26E-03 | -27125 | -13281 | 24080 | -16325 | 0.006687 | 46310 |
| -4.23E-03 | 0.4345 | 9.99E-03 | -27209 | -13416 | 24295 | -16330 | 0.007166 | 46415 |
| -4.47E-03 | 0.4413 | 0.0107 | -27306 | -13580 | 24562 | -16325 | 0.007661 | 46609 |
| -4.72E-03 | 0.4471 | 0.0115 | -27376 | -13750 | 24801 | -16325 | 0.008173 | 46791 |
| -4.98E-03 | 0.4522 | 0.0123 | -27421 | -13932 | 25027 | -16326 | 0.008702 | 46966 |
| -5.26E-03 | 0.456 | 0.0131 | -27456 | -14147 | 25278 | -16325 | 0.009247 | 47168 |
| -5.55E-03 | 0.459 | 0.0139 | -27472 | -14388 | 25531 | -16328 | 0.009808 | 47396 |
| -5.85E-03 | 0.4616 | 0.0148 | -27458 | -14639 | 25773 | -16324 | 0.0104 | 47617 |
| -6.22E-03 | 0.4587 | 0.0156 | -27517 | -14767 | 25958 | -16325 | 0.011 | 47788 |
| -6.63E-03 | 0.4534 | 0.0164 | -27568 | -14875 | 26114 | -16329 | 0.0116 | 47929 |
| -7.06E-03 | 0.4469 | 0.0172 | -27577 | -15006 | 26255 | -16328 | 0.0122 | 48048 |
| -7.54E-03 | 0.4391 | 0.018 | -27540 | -15164 | 26373 | -16331 | 0.0129 | 48146 |
| -8.07E-03 | 0.4284 | 0.0188 | -27476 | -15291 | 26439 | -16329 | 0.0135 | 48193 |
| -8.65E-03 | 0.4159 | 0.0195 | -27345 | -15459 | 26478 | -16327 | 0.0142 | 48199 |
| -9.27E-03 | 0.4026 | 0.0203 | -27164 | -15698 | 26538 | -16324 | 0.0149 | 48181 |
| -9.98E-03 | 0.3853 | 0.021 | -26953 | -15908 | 26531 | -16331 | 0.0156 | 48092 |
| -0.0107 | 0.3678 | 0.0217 | -26702 | -16125 | 26505 | -16322 | 0.0163 | 47974 |
| -0.0115 | 0.3518 | 0.0224 | -26406 | -16389 | 26474 | -16320 | 0.0171 | 47866 |
| -0.0123 | 0.3348 | 0.0231 | -26090 | -16682 | 26451 | -16321 | 0.0178 | 47750 |
| -0.0131 | 0.3187 | 0.0238 | -25760 | -16984 | 26423 | -16322 | 0.0186 | 47635 |
| -0.014 | 0.3015 | 0.0245 | -25424 | -17256 | 26358 | -16322 | 0.0194 | 47499 |
| -0.015 | 0.2828 | 0.0251 | -25076 | -17603 | 26355 | -16324 | 0.0202 | 47335 |
| -0.0159 | 0.2681 | 0.0258 | -24730 | -17925 | 26333 | -16322 | 0.021 | 47211 |
| -0.0169 | 0.2531 | 0.0265 | -24385 | -18189 | 26251 | -16322 | 0.0219 | 47042 |
| -0.0179 | 0.2381 | 0.0272 | -24041 | -18435 | 26152 | -16324 | 0.0227 | 46861 |
| -0.0189 | 0.2249 | 0.028 | -23701 | -18784 | 26161 | -16324 | 0.0236 | 46712 |
| -0.0199 | 0.2134 | 0.0287 | -23366 | -19140 | 26183 | -16323 | 0.0245 | 46602 |
| -0.021 | 0.1998 | 0.0294 | -23036 | -19438 | 26155 | -16318 | 0.0254 | 46464 |
| -0.022 | 0.1892 | 0.0302 | -22711 | -19714 | 26102 | -16322 | 0.0263 | 46364 |
| -0.0231 | 0.1795 | 0.0311 | -22396 | -19991 | 26057 | -16331 | 0.0273 | 46282 |
| -0.0241 | 0.1709 | 0.0319 | -22082 | -20348 | 26103 | -16326 | 0.0282 | 46219 |
| -0.0252 | 0.1612 | 0.0327 | -21774 | -20675 | 26128 | -16321 | 0.0292 | 46151 |
| -0.0264 | 0.1509 | 0.0335 | -21473 | -20985 | 26134 | -16324 | 0.0302 | 46083 |

## Appendix D-7: Plotted Moment Curvature graph data (case 04-2- Solid)

(Moment KN-m and Force KN)

| Concrete Strain | Neutral Axis | Steel Strain | Concrete Compression | Steel Compression | Steel Tension | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|
| -1.98E-04 | 0 | -1.98E-04 | -15675 | -2203 | 0 | -17878 | 0 | 6.53E-14 |
| -5.63E-04 | -0.4352 | 1.96E-04 | -15984 | -2639 | 737.5012 | -17885 | 0.0003923 | 10981 |
| -9.52E-04 | 0.0296 | 9.45E-04 | -18206 | -3820 | 4143.241 | -17883 | 0.0009809 | 19003 |
| -1.40E-03 | 0.2054 | 2.01E-03 | -21919 | -5241 | 9280.602 | -17879 | 0.001766 | 28184 |
| -1.82E-03 | 0.3361 | 3.49E-03 | -22783 | -6411 | 11305 | -17888 | 0.002746 | 31517 |
| -2.26E-03 | 0.4248 | 5.33E-03 | -22710 | -7562 | 12390 | -17881 | 0.003923 | 33088 |
| -2.76E-03 | 0.4788 | 7.48E-03 | -23107 | -7852 | 13074 | -17885 | 0.005297 | 33856 |
| -3.32E-03 | 0.516 | 9.96E-03 | -23482 | -7919 | 13521 | -17880 | 0.006866 | 34279 |
| -3.99E-03 | 0.5382 | 0.0127 | -24034 | -7934 | 14089 | -17879 | 0.008632 | 34955 |
| -4.74E-03 | 0.5526 | 0.0157 | -24538 | -7943 | 14601 | -17879 | 0.0106 | 35630 |
| -5.61E-03 | 0.5602 | 0.0191 | -25044 | -7978 | 15139 | -17884 | 0.0128 | 36311 |
| -6.61E-03 | 0.5624 | 0.0226 | -25548 | -8063 | 15731 | -17881 | 0.0151 | 36992 |
| -7.76E-03 | 0.5604 | 0.0264 | -26017 | -8135 | 16271 | -17881 | 0.0177 | 37629 |
| -9.04E-03 | 0.5571 | 0.0304 | -26324 | -8157 | 16596 | -17885 | 0.0204 | 37932 |
| -0.0105 | 0.5509 | 0.0347 | -26598 | -8207 | 16922 | -17884 | 0.0233 | 38178 |
| -0.0121 | 0.543 | 0.0391 | -26802 | -8319 | 17240 | -17881 | 0.0265 | 38391 |
| -0.0139 | 0.5351 | 0.0438 | -26865 | -8561 | 17544 | -17881 | 0.0298 | 38602 |
| -0.0158 | 0.5264 | 0.0487 | -26859 | -8845 | 17824 | -17880 | 0.0333 | 38760 |
| -0.0179 | 0.5171 | 0.0538 | -26800 | -9184 | 18105 | -17879 | 0.0371 | 38914 |
| -0.0202 | 0.5071 | 0.0591 | -26706 | -9546 | 18372 | -17880 | 0.041 | 39055 |

# Appendix E: Steady state temperature finite element analysis sample report

ANSYS Academic Teaching Introductory

```
     **** CENTER OF MASS, MASS, AND MASS MOMENTS OF INERTIA ****

 CALCULATIONS ASSUME ELEMENT MASS AT ELEMENT CENTROID

 TOTAL MASS =   6808.0

                    MOM. OF INERTIA        MOM. OF INERTIA
 CENTER OF MASS          ABOUT ORIGIN       ABOUT CENTER OF MASS

 XC =   1.1986        IXX =   0.1245E+05     IXX =    3966.
 YC =  0.99781        IYY =   0.1672E+05     IYY =    5237.
 ZC =  0.50000        IZZ =   0.2466E+05     IZZ =    8101.
                      IXY =   -8142.         IXY =   0.6033
                      IYZ =   -3397.         IYZ =   0.1364E-10
                      IZX =   -4080.         IZX =   0.1000E-10

 TYPE     MASS
   1   6808.00

Range of element maximum matrix coefficients in global coordinates
Maximum = 0.146884458 at element 32.
Minimum = 0.116147113 at element 173.

  *** ELEMENT MATRIX FORMULATION TIMES
 TYPE    NUMBER   ENAME      TOTAL CP  AVE CP

   1       798  SOLID90      0.125   0.000157
Time at end of element matrix formulation CP = 1.296875.

SPARSE MATRIX DIRECT SOLVER.
 Number of equations =       2671,    Maximum wavefront =     66
 Memory allocated for solver =         15.259 MB
 Memory required for in-core =          3.408 MB
 Memory required for out-of-core =       1.017 MB


*** NOTE ***                           CP =       1.391   TIME= 18:28:09
The Sparse Matrix solver is currently running in the in-core memory
mode.  This memory mode uses the most amount of memory in order to
avoid using the hard drive as much as possible, which most often
results in the fastest solution time.  This mode is recommended if
enough physical memory is present to accommodate all of the solver
data.
Sparse solver maximum pivot= 0.590563914 at node 2709 TEMP.
Sparse solver minimum pivot= 0.108883389 at node 602 TEMP.
Sparse solver minimum pivot in absolute value= 0.108883389 at node 602
TEMP.
```

```
 *** ELEMENT RESULT CALCULATION TIMES
 TYPE    NUMBER   ENAME      TOTAL CP  AVE CP

   1      798  SOLID90      0.125   0.000157

 *** NODAL LOAD CALCULATION TIMES
 TYPE    NUMBER   ENAME      TOTAL CP  AVE CP

   1      798  SOLID90      0.000   0.000000
 *** LOAD STEP     1   SUBSTEP     1  COMPLETED.    CUM ITER =      1
 *** TIME =   1.00000        TIME INC =   1.00000      NEW TRIANG MATRIX


 *** ANSYS BINARY FILE STATISTICS
  BUFFER SIZE USED= 16384
      1.250 MB WRITTEN ON ASSEMBLED MATRIX FILE: file.full
      1.250 MB WRITTEN ON RESULTS FILE: file.rth
 *************** Write FE CONNECTORS *********

 WRITE OUT CONSTRAINT EQUATIONS TO FILE=
file.ce

 ****************************************************
 *************** FINISHED SOLVE FOR LS 1 *************

 PARAMETER _DS_PROGRESS  DELETED.

 *GET  _WALLASOL  FROM  ACTI  ITEM=TIME WALL  VALUE=  18.4694444

 *** NOTE ***                            CP =       1.656   TIME= 18:28:10

Compiler: Intel(R) FORTRAN Compiler Version 14.0.0  (Build: 20140422)
          Intel(R) C/C++ Compiler Version 14.0.0  (Build: 20140422)
          Intel(R) Math Kernel Library Version 11.1.3 Product Build 20140917

Total number of cores available         :    4
Number of physical cores available      :    2
Number of processes requested           :    1
Number of threads per process requested :    2
Total number of cores requested         :    2 (Shared Memory Parallel)

GPU Acceleration: Not Requested

Job Name: file
Working Directory: C:\Users\Eyob\Desktop\Desktop April 09 2016\Thesis
End\ANSYS\Thermal\_ProjectScratch\Scr4F7F

Total CPU time for main thread                    :        1.4 seconds
Total CPU time summed for all threads             :        1.7 seconds

Elapsed time spent pre-processing model (/PREP7)  :        0.0 seconds
Elapsed time spent solution - preprocessing       :        0.0 seconds
```

```
Elapsed time spent computing solution           :       0.2 seconds
Elapsed time spent solution - postprocessing    :       0.0 seconds
Elapsed time spent post-processing model (/POST1) :     0.0 seconds

Equation solver computational rate              :     838.9 Mflops
Equation solver effective I/O rate              :    2795.1 MB/sec

Maximum total memory used                       :      32.0 MB
Maximum total memory allocated                  :    2112.0 MB
Maximum total memory available                  :         8 GB

+------------------ E N D   A N S Y S   S T A T I S T I C S -------------------+
```