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Structural Behavior of Symmetric & Asymmetric Built-up Composite Columns Subjected to Lateral Loads

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Structural Behavior of Symmetric & Asymmetric Built-up Composite Columns Subjected to  
Lateral Loads

THESIS

submitted in partial satisfaction of the requirements  
for the degree of

MASTER OF SCIENCE

in Civil Engineering

by

Nowell Verghese Philips

Thesis Committee:  
Professor Ayman S. Mosallam, Chair  
Assistant Professor Mo Li  
Assistant Professor Mohammad Javad Abdolhosseini Qomi

2017



## **DEDICATION**

To my thesis advisor and Professor Dr. Ayman S. Mosallam for guiding, supporting and motivating me throughout the way.

## TABLE OF CONTENTS

CHAPTER NO	TITLE	PAGE NO
	LIST OF FIGURES	vi
	LIST OF TABLES	ix
	ACKNOWLEDGEMENTS	x
	ABSTRACT OF THE THESIS	xi
1	<b>INTRODUCTION</b>	1
1.1	Overview	1
1.2	Sandwich Panels	2
1.3	Objectives and Scope of the Study	3
1.4	Research Methodology	4
1.5	Thesis Outline	4
2	<b>LITERATURE REVIEW</b>	6
2.1	Introduction	6
2.2	Background	7
2.3	Characteristics of Sandwich Panels	8
2.4	Expanded Polystyrene Foam (EPS)	13
2.5	Lateral Loading on Composite Columns	14
3	<b>DESCRIPTION OF SPECIMENS &amp; EXPERIMENTAL TEST PROGRAM</b>	16
3.1	Built-up Column Sections	16
3.1.1	Material Properties & Structural Joints	20
3.1.2	Expanded Polystyrene (EPS) Foam Core	21

3.1.3	Welded Wire Reinforcement	22
3.1.4	Structural Mortar	23
3.1.5	RC Column Footing	25
3.1.6	Column-Footing Connection	26
3.2	Construction of Composite Built-up Columns	28
3.2.1	Preparation of Symmetric & Asymmetric Column Specimens	28
3.2.2	Development of Wall Panels	28
3.2.3	Mortar Application	29
3.2.4	Curing of Specimens	32
3.3	Testing Instrumentation	33
3.3.1	Hydraulic Jack	34
3.3.2	String Potentiometers	35
3.3.3	Electronic Strain Gages	37
3.3.4	Data Acquisition System	42
3.4.	Testing Protocol	43
3.5	Experimental Results	49
3.5.1	Symmetric I-section Column	49
3.5.2	Asymmetric C-Section Column	53
3.6	Analytical Design Procedure	54
3.6.1	Flexural Strength	55
3.6.2	Shear Capacity	57
3.6.3	Deflection	58

3.7	Summary & Concluding Remarks	60
4	<b>NUMERICAL ANALYSIS USING ANSYS</b>	62
4.1	Modeling Approach	62
4.2	Materials Definitions	63
4.3	Modal Analysis & Meshing	64
4.4	Loads & Boundary Conditions	68
4.5	Verification of Results & Comparison to Experimental Results	69
5	<b>CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH</b>	74
5.1	Summary	74
5.2	Conclusions	74
5.3	Recommendations for Future Research	76
	<b>References</b>	77
	Appendix (A)– Details of Test Equipment	82
	Appendix (B) – Risk Assessment	84

## LIST OF FIGURES

FIGURE NO	TITLE	PAGE NO
2.1	Strain Distribution (Source: Benayoune et al., 2008)	12
2.2	EPS Sandwich Panel with Diagonal Shear Connectors	13
3.1	Typical Cross Section of Sandwich Panel with Diagonal Shear Connectors	16
3.2	Layout of Composite I-Section Column	18
3.3	Layout of Composite C-Section Column	19
3.4	Three-Dimensional (3D) View of I-Section Column	19
3.5	Three-Dimensional (3D) View of C-Section Column	20
3.6	EPS Sandwich Panels with Welded-wire Reinforcements	22
3.7	28-day Compressive Strength Testing of Mortar Cylinder	24
3.8	#4 Rebars Installed to Foundation Using High-Strength Epoxy	27
3.9	Steel dowels Connecting Footing to Built-up Columns	27
3.10	Hand-applied Mortar on I-Section Column	29
3.11	Mortar Poured from the Top to the Inner Faces of Column-Wall Connection	30
3.12	Hand-applied mortar on symmetric I-Section Column	31
3.13	Hand-applied mortar on Asymmetric C-Section Column	32
3.14	I-Section & C-Section Column Specimens After Curing & Painting	33
3.15	Hydraulic Cylinder	34



3.16	Hydraulic Pump	35
3.17	Location of String Pots on I-section Column	36
3.18	Location of String Pots on C-section Column	37
3.19	Strain Gauge Installed on Mortar Surface of Columns	38
3.20	Strain Gauge Locations on Steel Wire Reinforcements of I-section Column	39
3.21	Strain Gauge Locations on Mortar Layer of I-section Column	40
3.22	Strain Gauge Locations on Steel Wire Reinforcements of C-section	41
3.23	Strain Gauge Locations on Mortar Layer of C-section Column	42
3.24	National Instrument Data Acquisition Chassis	43
3.25	Layout for Experimental Test Setup for Composite I-Section	44
3.26	Layout for Experimental Test Setup for Composite I-Section	45
3.27	Experimental Setup for Composite I-Section	47
3.28	Experimental Setup for Composite C-Section	47
3.29	Crack Pattern for Composite I-section Column	48
3.30	Crack Patterns for Composite C-Section Column	49
3.31	Large Cracks Observed at the Column-Footing Joint of I-section Column	50
3.32	Load-Displacement Curve for Composite I-section Column	51
3.33	Load-Strain Curve for Steel Wires on Tension Side of I-section Column	51
3.34	Load- Strain Curve for Mortar Layer on I-section Column (Compression Side)	52
3.35	Cracks Observed on the Mortar Surface of C-section Column	53

3.36	Load-Displacement Curve for Composite C-section Column	54
4.1	Modelled Geometry of I-section Column	65
4.2	Modelled Geometry of C-section Column	66
4.3	Meshing of I-Section Column Model	67
4.4	Meshing of I-Section Column Model	68
4.5	Von-Misses Distribution in I-Section Column	70
4.6	Von-Misses Distribution in C-Section Column	70
4.7	Deformation Pattern in I-section Column	71
4.8	Deformation Pattern in C-section Column	71
4.9	Load vs Deformation of I-Section Column	72
4.10	Load vs Deformation of C-Section Column	72
4.11	Load Comparison of I-section Column	73
4.12	Load Comparison of C-section Column	73

## LIST OF TABLES

<b>TABLE NO</b>	<b>TITLE</b>	<b>PAGE NO</b>
3.1	Dimensions of Column Specimens (Including Mortar Thickness)	17
3.2	Dimensions of Wall Panel Sections (Including Mortar Thickness)	18
3.3	Specifications of the EPS Panel	21
3.4	Mortar Mis Design Proportion	23
3.5	28-Day Mortar Compressive Strength for Symmetric I-Section Column	24
3.6	28-Day Mortar Compressive Strength for Asymmetric C-Section Column	25
3.7	Maximum and cracking Experimental Load for Built-up Column Sections	61
3.8	Analytical Results of Flexural Strength, Shear Strength, Deflection of Columns	61

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## **ABSTRACT OF THE THESIS**

Structural Behavior of Symmetric & Asymmetric Built-up Composite Columns Subjected to Lateral Loads

By

Nowell Philips

Master of Science in Civil Engineering

University of California, Irvine, 2017

Professor Dr. Ayman S. Mosallam, Chair

The main goal of this research study is to evaluate the structural behavior of symmetric and asymmetric built-up composite columns subjected to lateral loading. The composite columns were made up of two outer layers of high-strength structural mortar with three-dimensional welded-wire space truss incorporating diagonal cross wires welded to welded-wire reinforcement (WWR) on each side of a fire retardant Expanded Polystyrene (EPS) foam inner core. This study deals with the experimental investigation and methodology carried out for the fabrication and testing of these built-up columns with the EPS sandwich panels. The peak load, maximum strains and horizontal displacements of each of the specimens were obtained and the different failure modes were identified and recorded.

This thesis also presents the detailed output of the nonlinear 3-D finite element analysis of the composite columns modeled using ANSYS. The numerical analysis results were comparable with the experimental results and were conservative. The conclusions from the experimental test results were drawn and the scope for future research work is also discussed.

# CHAPTER 1

## INTRODUCTION

### 1.1 Overview

The main aim of this study is to experimentally evaluate the structural behavior of built-up composite column sections made of Expanded Polystyrene (EPS) sandwich panels under monotonic lateral load and to compare the results with the numerical analysis using Finite Element Analysis.

These built-up sections for building construction is made up of sandwich panels which consist of a three-dimensional welded wire space frame provided with a polystyrene insulation core and wythes of mortar applied around it. All the tests involved in the experimental program consist of the panel described. Various specimens were designed and fabricated to study and evaluate numerous structural components. The structural components include walls, columns and joints. The diverse specimens were designed and constructed based on the sandwich panel material and design to maximize the strength of the structural component when subjected to the lateral loadings.

The structural evaluation of the built-up sections made of sandwich panels through numerous strength tests is an important and large part of the research. From the strength test the maximum loads, forces, strains, displacements and modes of failure are acquired to be able assess the behavior of the panels. The results attained from the experimental program are then compared to theoretical and numerical results.

The evaluation of the sandwich panels through experimental test, theoretical analysis and numerical modeling allows for a better understanding of their behavior and limitations of the panel.

## **1.2. Sandwich Panels**

Sandwich panels are an excellent way to obtain extremely lightweight panels with very high bending stiffness, high strength and high buckling resistance. Sandwich wall system is an innovative prefabricated wall panel which consists of an integrated core insulation allowing very low energy design and retrofit of buildings. Sandwich panel is a three-layer element which consists of two thin flat facing plates of relatively higher strength material and in between a thick core of relatively lower strength and density is encased or it could consist of thin skin box of relatively higher strength material in-filled with relatively weaker and lower density material known as core. When the inner core is made of an insulating material like expanded polystyrene, thermal comfort of the building can be drastically improved leading to energy efficiency.

Sandwich construction form has distinct advantages over conventional solid structural sections as it provides high strength-to-weight ratio and high stiffness when compared to a solid panel. Sandwich composite panel possesses excellent flexural and shear properties. Their inherent lightweight characteristics make them ideal structural components where weight reduction is desirable. Thus, structural sandwich panels are becoming important elements in modern lightweight construction. Thus, the sandwich panels have a very good potential to withstand

seismic as well as impulsive loads. In sandwich panels transfer of shear between the wythes is achieved by providing shear connectors. The various types of sandwich panels are (i) Non-composite system (ii) Composite system and (iii) Semi-composite system. The insulation core within sandwich panels varies. The core material used generally falls into one of the following categories are (i) Non-combustible mineral wool or fiberglass (ii) Polyisocyanurate Foam (PIR) (iii) Polyurethane Foam (PUR) and (iv) Expanded or Extruded Polystyrene (EPS and EXPS).

In the present study, Expanded Polystyrene (EPS) was used as the inner insulation core and the outer wythes were made of cement mortar for the development of sandwich panel.

### **1.3. Objectives & Scope of the Study**

The main objectives of this research are:

1. Examine the load carrying capacity of the composite built-up sections under lateral loading.
2. Analyze observations and test results from experimental program.
3. Compare the experimental results to the theoretical and/or numerical analysis.
4. Evaluate the structural performance and behavior of the symmetric and asymmetric sandwich panel built-up specimens.
5. Formulate the load-deformation curve and study the structural behavior of the prepared specimens.



The purpose of this study is to develop various built-up composite column sections and to investigate its static lateral load carrying capacity of the specimens. The experimental evaluation is followed by the numerical analysis of the composite columns by utilizing the finite element software, ANSYS 17.1.

Prototype models were prepared and analyzed. The experimental results were compared with the numerical model analysis results and the conclusions were drawn.

#### **1.4. Research Methodology**

A substantial part of this research study focusses on the laboratory experiments that were carried out in the Structural Engineering Testing Hall (SETH Lab) at the University of California, Irvine (UCI). Lateral loading test was performed on the symmetric (I- Section) specimen and asymmetric (C-section) composite built-up specimens. Experimental results were recorded and compared with theoretical and numerical results.

#### **1.5. Thesis Outline**

This thesis is structured as follows:

Chapter 1 of the thesis introduces the research study carried out on the composite built-up sections made of sandwich panels and describes the main objectives and scope of the study.

Chapter 2 details the various studies done in the past on this area of research and their findings and observations along with the significance of using the composite

sandwich panels.

Chapter 3 explains the methodology followed for the experimentation part of the research and elaborates the material properties and presents the results of the experimental testing.

Chapter 4 shows the finite element analysis procedure using ANSYS software and presents the detailed finite element models and the results obtained from the numerical analysis.

Chapter 5 briefs the concluding remarks and summarizes the study and discusses the scope for future research in this field of study. This chapter is followed by references.

## **CHAPTER 2**

### **LITERATURE REVIEW**

#### **2.1. Introduction**

The presence and use of sandwich panels for building construction has been known for many decades, however, limited applications of this innovate system has been reported for US construction market due to limited comprehensive research work, except for the several studies initiated at University of California, Irvine, in the past few years.

Most of the literature on advanced sandwich construction is associated with the aerospace industry utilizing this system for several applications because of its light-weight properties, however, there is also a considerable level of implementation in the building industry by using the sandwich panels for fabricating structural elements. Although the current knowledge about the relevance of sandwich panels is much less than the conventional building technologies, the improvement of the material properties and design has led to an increase in acceptance and practice of sandwich panel construction.

The changes in material has made the sandwich panels a more direct choice for construction because of the lower impact it has on the environment compared to the conventional methods of construction. The idea of incorporating fibre composite properties into the design of panels for construction building has increased the capacity of the various building components. The background, design and previous

testing of sandwich panels for building construction are discussed more in detail in the following paragraphs.

## **2.2. Background**

Construction methods and materials have historically been determined on available resources within a region. Generally, as population grows, those resources become scarce, for example timber in the United States. The over use of natural resources eventually has a major impact on the natural environment, but as the society has evolved, so has the construction industry. More sustainable alternatives to conventional methods of construction have been developed over the years. The use of sandwich panels as structural components would reduce lumber consumption, energy consumption, carbon production, and building waste (Baginski, 2006).

As stated earlier, prior to the use of sandwich panels in construction, the use of sandwich panels had become popular in the aerospace applications and saw mass production during Second World War (Howard G. Allen, 1969). The Second World War "Mosquito" aircraft is often quoted as the first major structure to incorporate sandwich panels. Sandwich construction has been used in many fields earlier, but in less spectacular circumstances. It is not clear who first developed the idea of sandwich panels for construction purposes, but it has been seen in the United States since the early 20<sup>th</sup> century. In 1935 the Forest Products Laboratory, established by the United States Department of Agriculture, built homes out of sandwich panels constructed of plywood and hardboard sheathing (Structural Insulated Panel, (SIP),

Background and Research, 2013). The first residential buildings in the United States built with sandwich panels with foam as the core insulation were constructed in Midland, Michigan in 1952 (Dufree, 1993). Those panels consisted of plywood faces and a styrofoam foam core. The sandwich panels effectively function by the stressed-skin principle, where the combination of all components greatly improves the overall strength compared to each component individually (Dufree, 1993). This principle is similar to characteristics of an I-beam where the flanges of the I-beam can be compared to the faces of the sandwich panel and the core of the sandwich panel could be considered as the web of the I-beam. Oriented strand board (OSB), plywood, cementitious materials, and metal can all serve as the structural skin of the sandwich panels. Many innovations and improvements have been made ever since the introduction of sandwich panels to the field of civil engineering.

### **2.3 Characteristics of Sandwich Panels**

The sandwich panels being evaluated in this study consist of expanded polystyrene foam core, two wire mesh faces, shear connectors passing through the foam core to connect the two wire mesh faces, and mortar sprayed on both faces of the foam core (refer to Figure (2.2)).

Depending on the structural element, the dimensions of each of those components could vary in size. After the application of mortar to both sides of the foam core the panel achieves the concept of a sandwich panel. According to the sandwich concept, the mortar along with the welded-wire reinforcements takes both the compressive and tensile loads causing the stiffness and strength of the panel to increase and the

wires connecting the two faces of the welded-wire reinforcement sheets serves to transfer the shear loads between faces (Bajracharya, Lokuge, Karunasena, Lau and Mosallam, 2012). The structural behaviour of the sandwich panel is largely dependent on the positioning, strength and stiffness of the shear connectors (Benayoune, Abdul Samad, Trikha, Abang Ali & Ellinna, 2008). The shear connectors also contribute to the degree of composite action achieved by the sandwich panel; fully composite, semi-composite or non-composite (Benayoune, Abdul Samad, Trikha, Abang Ali & Ellinna, 2008). The purpose of the foam core is to reduce the weight of the structure as well as acts as an insulation against thermal, acoustics, and vibration (Bajracharya, Lokuge, Karunasena, & Lau, 2012). The thickness of the foam core depends on the structural component being designed and the desired thermal resistance of the panel (Salmon, Einea, Tadros, & Culp, 1997).

By achieving composite action the sandwich panels have a superior advantage over traditional construction building methods. Two disadvantage of having steel wire diagonals is the possibility of thermal bridging and thermal bowing due to different temperatures at the interior and exterior surfaces of the sandwich panel (Einea, Salmon, Tadros, & Culp, 1994). The foam core's more efficient thermal insulation and ability to be manufactured with recycled material allows the sandwich panel to conserve a larger amount of energy and create a more comfortable living condition compared to tradition insulation material (Canadian Building Digest). The high quality, proven durability, fast erection, attractive architectural appearance and previously mentioned advantages of the sandwich panels has led to various

investigations of the sandwich panels structural behaviour to further understand its capabilities.

Experimental testing on sandwich panels for building construction has been previously done on distinct designs of panels to verify their maximum strength, modes of failures, and full extent of the composite action. Previous experimental test includes both element level, and coupon level tests.

Both full scale and coupon level testing were done at the National University of Engineering in Peru. Axial tension test of the three-dimensional wire truss with no mortar were completed to know the mechanical characteristics of the three-dimensional reinforcement and axial compression test of the sections of panels were completed in order to study the characteristics of the composed material (Zavala, 2008). A full-scale test of a two-story scale model was also tested at the National University of Engineering in Peru with the goal of obtaining information of the behavior of the structural system due to the full scale.

Coupon level test were also completed at the University of California, Irvine to investigate whether the Classical Laminate Theory can be applied to the sandwich panels. The study focused on the development of the engineering constants for the steel reinforced mortar skins of the sandwich panels through uniaxial tension, uniaxial compression and in plane shear tests (Eriksson, 2014). Although the focus of the experimental program of this study is of element level tests, the various coupon level tests allow for initial assumptions of the different components of the sandwich panels.

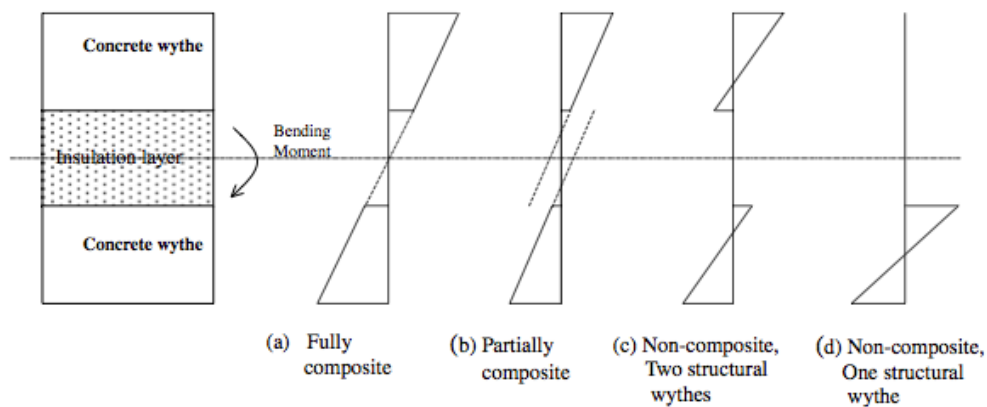
Numerous element level tests have been performed with the goal of obtaining an improved understanding of the structural behavior of sandwich panels.

Full scale seismic tests were performed on walls with and without openings with fixed ends, made of EPS sandwich panels. The sandwich wall panels proved to have coupling between flexure and shear as the panel geometry was basically subjected to axial stresses and in-plane shear. The steel reinforcement provided in the panel helped to limit the brittle failure due to diagonal tension (Pavese et al., 2011). The experimental program in Kabir (2005) included 3 flexural specimens and a Finite Element Model to compare the results. It was concluded that the load deflection behaviour of those panels demonstrated acted as partially composite panels (Kabir & Nasab, 2005). The National University of Engineering in Peru performed experimental testing on multiple flexure specimens to formulate moment curvature curves. Previous experimental cyclic shear wall test was performed at the National University of Engineering in Peru and at the University of California, Irvine.

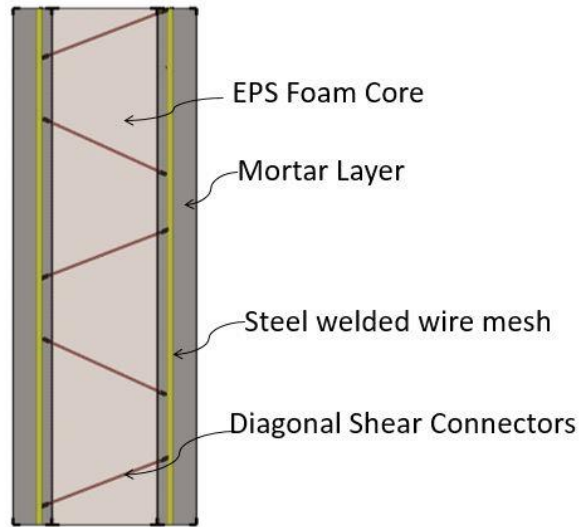
Experimental testing on 3D sandwich wall panels under in-plane quasi-static lateral cyclic loading found out that a sizable structural damage such as wall cracks, concrete spalling and fracture of longitudinal reinforcement bars at the bottom corner was occurred to the wall panel (Hamid, 2008). The conclusions of the test on wall panels subjected to monotonic axial and cyclic lateral loading reveals that the panel stiffness and deflections were affected by the type and configuration of the shear transfer mechanism and also by the type of foam used in the panels (Frankl et al., 2011).



Benayoune (2007) presented experimental results of sandwich panels subjected to axial loading. The results are compared to a Finite Element Models and calculated values from ACI design practices and other available design formulations developed from solid walls to determine if these procedures can be applied to sandwich panels (Benayoune A., Samad, Abang Ali, & Trikha, 2007). Experimental results were found to be similar to the results from the Finite Element Model, while the calculated values were much more conservative. In 2006, Benayoune also presented results from eccentrically loaded sandwich panels. Experimental results were compared to both calculated values using conventional approach based on reinforced concrete principles and to results obtained from a Finite Element Model. The Finite Element Model results were found to be similar to the experimental results, but the calculated values based on reinforced concrete principles underestimates the ultimate strength (Benayoune A. ,Samad, Trikha, Abang Ali, & Ashrabov, 2006).



**Figure (2.1):** Strain Distribution (Source: Benayoune et al., 2008)



**Figure (2.2):** EPS Sandwich Panel with Diagonal Shear Connectors

#### **2.4 Expanded Polystyrene Foam (EPS)**

EPS is a potential material that can be used in the fabrication of light weight panels. The material is molded into large blocks and cut to the proper shapes for use in structural panels. The EPS core also reduces the panel's weight compared to some other prefabricated structural panel systems, making EPS panels easier to construct and better suited for seismic active regions.

Expanded Polystyrene has several advantages which makes it a suitable material in the civil and construction industry. EPS being a very lightweight material consists of 98% air is derived from petroleum and natural gas by-products and it is very compatible with construction materials such as cement, concrete, masonry, brick, mortars, bitumen based membranes, etc. EPS is very versatile material as it can be cut and shaped to any size and pattern.

One of the importance benefits of using EPS panels in the building industry is that it is fire retardant. The fire-retardant properties of EPS have been tested and it was found that it does not support the spread of flames. EPS combined with building materials such as concrete or mortar as outer wythes to a certain thickness can prevent fire to pass through it. The fire resistance depends on the concrete thickness layers and the quality of aggregates used. A fire rating of 90 minutes could be achieved with a 40mm thick concrete applied on both side of the EPS panel. This property of EPS is significant to this research study because the composite built-up columns sections and walls could be used as a compound boundary or fence in places where there are chances of a fire breakout.

When used in combination with harder building materials, EPS increases the sound insulation. It has got very good thermal insulating properties, moisture resistant, greater durability, cost effective, easy to transport, recyclable and environmental friendly. These advantages make it a preferable product in the manufacture of light weight structural sandwich panels.

## **2.5 Lateral Loading on Composite Columns**

The lateral loads acting on structural vertical components are vital to be studied and analyzed especially in seismic prone areas. Several studies have been carried out on the steel and concrete columns, however the experimental analysis of composite columns made of EPS sandwich panels for lateral loading are very limited.

The experimental investigation conducted on Steel-Encased composite columns to evaluate the seismic performance of the columns done in the past have showed that such columns has high cyclic strength and ductility when confinement of the concrete core in the flexural plastic hinge zone is adequately provided (Ricles, 1994). Similar static and cyclic experimental and analytical tests to examine the behavior of steel-concrete composite columns has taken place in the past to assess the applicability of composite columns in seismic zones.

The monotonic and cyclic lateral load testing carried out at the Technical University of Cluj-Napoca, Romania on composite steel-concrete columns with steel encased profile indicates that the lateral force corresponding to the displacement and lateral loading had a significant rise even if the elastic displacement was decreasing. Due to improved fire protection and exceptional seismic performances, the steel encased composite columns are an apt solution for seismic zones (Campian, Nagy & Pop, 2015).

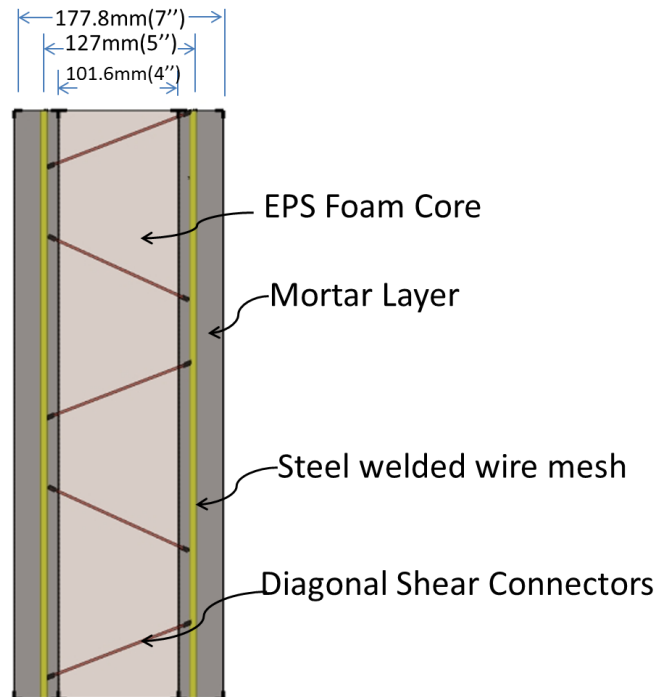
When conventional RC columns are subjected to cyclic lateral load, it undergoes a brittle shear failure along with the flexural yielding of longitudinal steel reinforcement bars. However, the seismic performance and parameters such as ultimate load-carrying capacity, stiffness, ductility of the RC columns could be improved by giving debonding casting for the reinforcement over the potential plastic hinge zone (Mitra & Bindu, 2015).

## CHAPTER 3

### DESCRIPTION OF SPECIMENS & EXPERIMENTAL TEST PROGRAM

#### 3.1 Built-up Column Sections

A detailed description of the symmetric and asymmetric built-up column sections is discussed in this chapter. Three composite column sections were tested and all the specimens were subjected to monotonic lateral loads. The composite column sections were built-up of 3-D sandwich panels composed of Expanded Polystyrene (EPS) foam insulation core that is sandwiched between the two layers of high strength welded-wire mesh reinforcements that are connected to each other via diagonal cross wires that are welded.



**Figure (3.1):** Typical Cross Section of Sandwich Panel with Diagonal Shear Connectors

Two different column specimens were evaluated in this study; (i) a symmetric I-section column, and (ii) an asymmetric C-section column. The EPS core and the welded-wire reinforcements were cut and shaped as required according to the structural design requirement to fabricate the I-shaped symmetric column section and C-shaped asymmetric column section. The sandwich panels cut in the required dimensions were attached together and for all the joints in the specimens, galvanized 9-gauge steel tie wires (conforming to ASTM A-185 and ASTM A-82) was used to create the meshed face sheets and diagonal reinforcement that connected the opposing face sheets into a rigid three-dimensional structure. The dimensions of the built-up composite columns are summarized in Table 3.1.

Wall sections made of the same sandwich panels were inserted to the built-up column sections and the column-to-wall joints were connected using galvanized 9-gauge steel tie wires (conforming to ASTM A-185 and ASTM A-82) and the structure was casted with high-strength mortar. The wall dimensions for both the built-up column sections were the same and are given in Table 3.2.

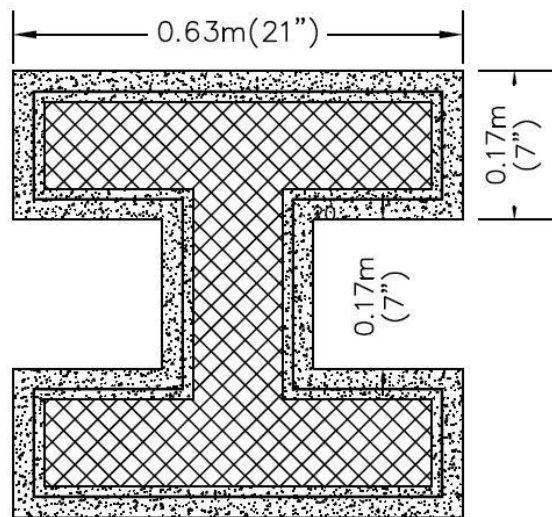
**Table (3.1):** Dimensions of Column Specimens (Including Mortar Thickness)

<b>Test Code</b>	<b>Depth of Section D</b>	<b>Width of Section B</b>	<b>Thickness of Web t</b>	<b>Thickness of Flange T</b>	<b>Height of Column H</b>
I	0.6334m (21in)	0.6334m (21in)	0.1778m (7in)	0.1778m (7in)	1.2446m (49in)
C	0.6334m (21in)	0.3556m (14in)	0.1778m (7in)	0.1778m (7in)	1.2446m (49in)

**Table (3.2):** Dimensions of Wall Panel Sections (Including Mortar Thickness)

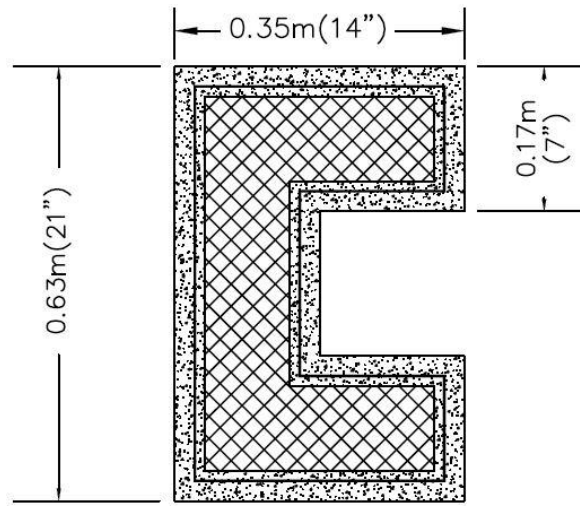
Test Code	Length	Width	Thickness
I	1.0668m (42in)	0.6334m (21in)	0.1778m (7in)
C	1.0668m (42in)	0.6334m (21in)	0.1778m (7in)

The built-up columns were constructed on a reinforced cement concrete (RCC) foundations. The foundations were prepared earlier and the columns were attached to the dowel anchor bars drilled and inserted to the foundation by using high strength epoxy. The details of the column-to-foundation connection is described in section 3.1.5.



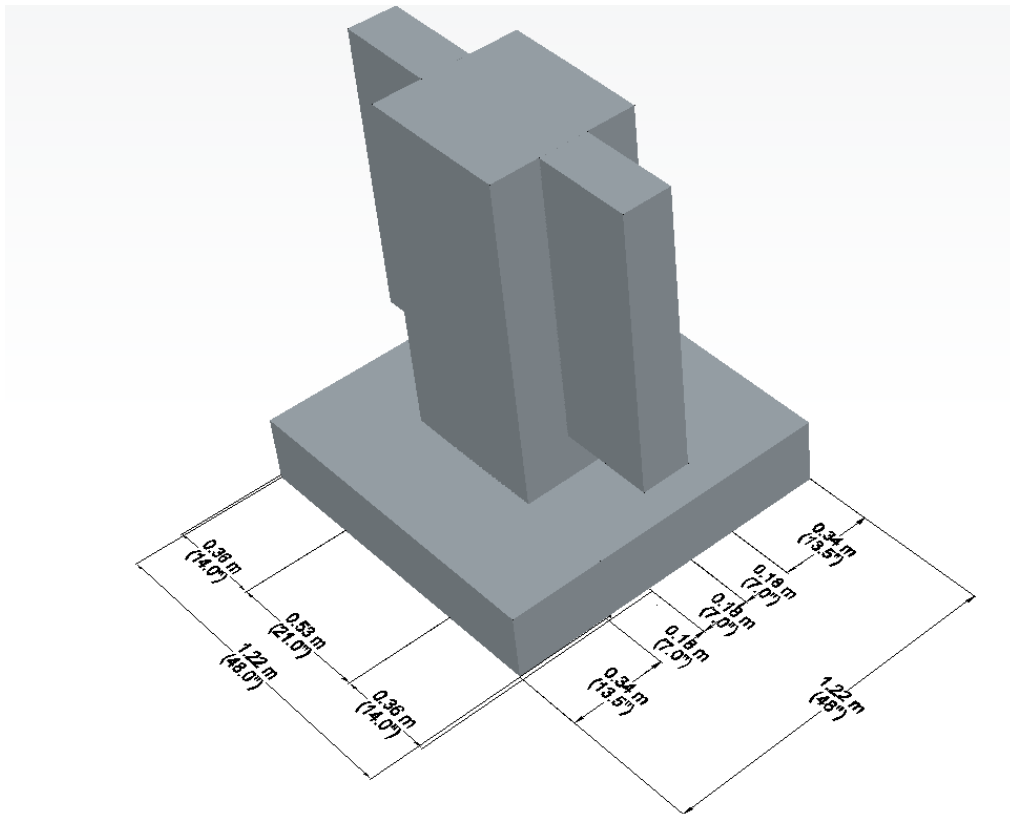
I- Section

**Figure (3.2):** Layout of Composite I-Section Column



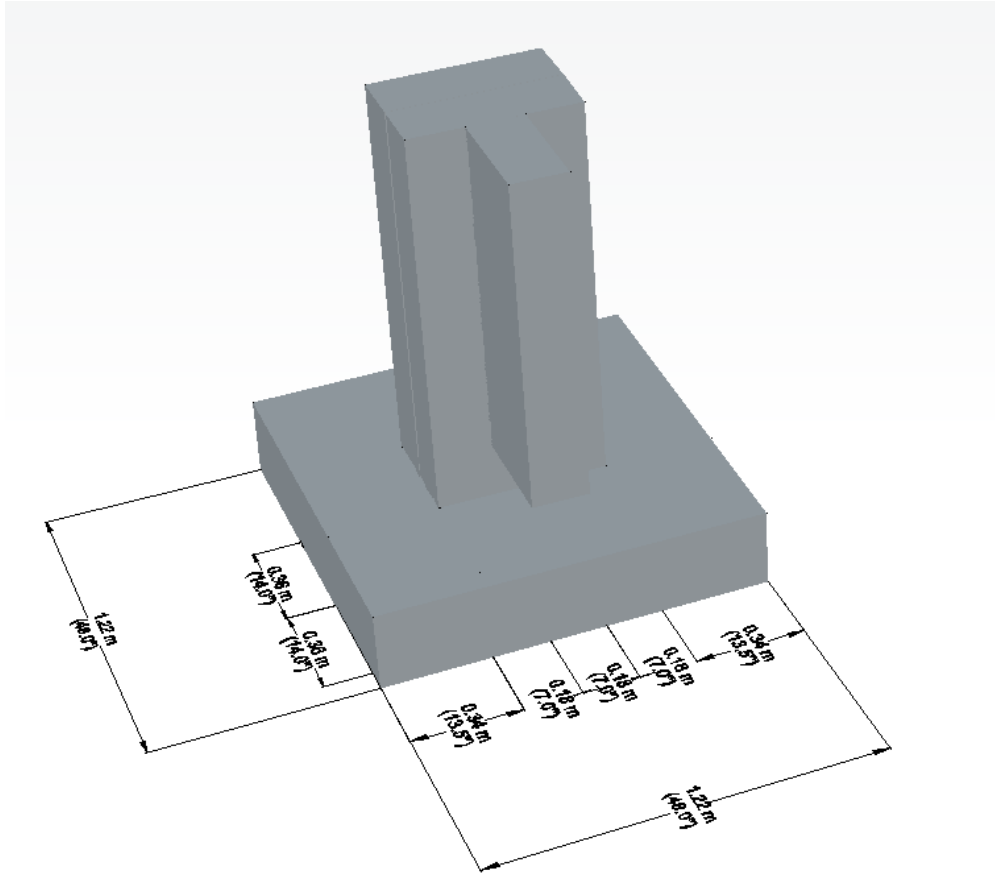
C- Section

**Figure (3.3):** Layout of Composite C-Section Column



**Figure (3.4):** Three-Dimensional (3D) View of the I-Section Column





**Figure (3.5):** Three-Dimensional (3D) View of the C-Section Column

### 3.1.1. Material Properties & Structural Joints

The materials utilized in this research study for the fabrication of built-up composite columns are described in this section. All the materials used were selected as per the structural design requirements.

The built-up composite columns were placed on a reinforced concrete foundation and the elaborate details of the same is presented in the section below. The joint connection between the columns and the wall panels are also presented. The

construction procedure and experimental testing protocol are detailed in the following sections.

### 3.1.2. Expanded Polystyrene (EPS) Foam Core

The core is a Type I modified EPS foam plastic complying with ASTM C578, having a normal density of  $16.0\text{kg/m}^3$  ( $1.0\text{ lb/ft}^3$ ) as manufactured by Poliestireno Alfa Gamma, S.A. de C.V., Insulfoam, LLC and Fanosa S.A. de C.V. The insulating core has a flame-spread index of 25 or less and a smoke-developed index of 450 or less when tested in accordance with ASTM E84 at a 152mm (6") thickness for EPS boards recognized under ICC-ES ESR-1788 and ESR-1006, and at a 102mm (4") thickness for EPS boards recognized under ICC-ES ESR-2744.

In this study, the expanded polystyrene foam core utilized for the evaluation had a thickness of 102mm (4"). The EPS foam was cut and shaped into the desired sizes to build the columns and wall panels.

**Table (3.3):** Specifications of the EPS Panel

Thickness	102mm (4")
Density	16 kg/m <sup>3</sup> (1 lb/ft <sup>3</sup> )
Nature	Fire retardant

### 3.1.3. Welded-Wire Reinforcement

The welded-wire reinforcement (WWR) was made up of galvanized and bright welded wire complying with ASTM A185. The welded-wire reinforcement was spaced 12.7mm (0.5”) from the faces of the EPS insulation core on both sides. The steel wire mesh size was 50 mm x 50 mm. The clear spacing was provided to ensure that the wire mesh was properly embedded in the mortar shell during the construction of the built-up specimens. The diagonal truss wires that pierce through the EPS core, act as shear transfer elements and comply to ASTM A82. The welded-wire reinforcement, including the diagonal wires, have a minimum yield stress of 420MPa (60 ksi). Minimum yield stress of 420 MPa (60 ksi) must comply with Section 3.5.3 of ACI 318 and IBC Section 1903.



**Figure (3.6):** EPS Sandwich Panels with Welded-wire Reinforcements

### 3.1.4. Structural Mortar

The outer wythes of the composite columns and the wall panels were composed of cementitious skins and the mortar was prepared in the Structural Engineering Test Hall (SETH) of the Department of Civil and Environmental Engineering (CEE) at the University of California, Irvine (UCI). Compressive strength properties of the cementitious skin were established in accordance with the protocols of ASTM C109/C109M-11, Standard Test Method for Compressive Strength of Hydraulic Cement Mortars, ASTM C-1140/C-1140M-11 Standards. A minimum compressive strength of 17.2 MPa (2500 psi) is required under ADIBC Appendix L, Section 5.1.1 at 28 days.

The mix proportion used for the preparation of mortar for all the specimens is given in Table 3.4.

**Table (3.4):** Mortar Mix Design Proportion

Cement	Sand	Water	W/C Ratio
22 kg (47 lbs)	35 kg (117 lbs)	10 liters (2.7 Gallons)	0.48

The 28-day compressive strength,  $f_c$ , of the cementitious skin were determined from an average of 102 mm X 203 mm (4" Dia. X 8" Ht.) control cylinders tested in accordance with ASTM C39-71. The average 28-days compressive strength of the

mortar cylinders for composite I-Section column and composite C-Section column are tabulated in table 3.4 and 3.5 respectively.



**Figure (3.7):** 28-day Compressive Strength Testing of Mortar Cylinder

**Table (3.5):** 28-Day Mortar Compressive Strength for Symmetric I-Section Column

Period		Specimens			Average Compressive Strength MPa(psi)
		1	2	3	
28 Days	Load, kN (lb)	206 (46,310)	206.53 (46,430)	204.560 (45,987)	25.384 (3681.71)
	Compressive Strength MPa(psi)	25.421 (3,687.10)	25.487 (3,696.65)	25.244 (3,661.38)	

**Table (3.6):** 28-Day Mortar Compressive Strength for Asymmetric C-Section Column

Period		Specimens			Average Compressive Strength MPa(psi)
		1	2	3	
28 Days	Load, kN (lb)	203.46 (45,740)	205.17 (46,125)	205.53 (46,206)	25.264 (3,664.30)
	Compressive Strength MPa(psi)	25.108 (3,641.72)	25.320 (3,672.37)	25.364 (3,678.82)	

### 3.1.5 RC Column Footing

Individual reinforced concrete (RC) column footings were designed and constructed for placing the composite columns for the lateral loading test evaluation. The RC foundations were 1.2192m (48”) in length by 1.2192m (48”) in width by 0.254m (10”) in thickness. Eight Grade 60 hot rolled #4 (12.7mm (0.5”) Diameter) reinforcement bars were used in the tension and compression faces of the foundation. The footings after construction was cured and the built-up columns were securely attached to the steel dowel bars inserted to the footing. The mortar shells were applied to the columns after they were fixed to their foundations. RC foundations were designed to be able to firmly be fixed to the outside strong floor of the SETH lab at UCI with the assistance of high strength Dywidag steel rods (38.1mm (1.5”) diameter), nuts and washers.

### 3.1.6 Column-Footing Connection

Precision holes were drilled on the reinforced concrete foundations for inserting the dowels, to which the columns were to be attached. Eight Grade 60 hot rolled #4 (12.7mm (0.5") Diameter) reinforcement bars were used to connect and secure the columns to the footing. The reinforcement bars to be inserted were accurately positioned at all the corners and ends of the column section and were embedded to a depth of eight inches into the footing and ten inches within the columns. The minimum anchorage bar length from the surface of footing was determined by multiplying twenty to the diameter of reinforcement dowel bars used (12.7mm X 20= 254mm (10")). These dowels were provided to connect the column to the footing firmly and they also act as shear connectors.

Eight Grade 60 hot rolled #4 ((12.7mm (0.5") Diameter) reinforcement bars were anchored to the foundations in their predetermined positions using high-strength anchoring two-part, room temperature cured epoxy. The high strength anchoring epoxy had a fast setting time of 15 minutes. Each rebar was properly inserted and attached well to the foundation. 16 numbers of #4 hot rolled steel dowels (one at each corner and end of the section and two each on both the outer faces of the width of the column section) were provided to support the I-section symmetric column to the foundation. 10 numbers of #4 hot rolled steel dowels (one at each corner and end of the section and two at the outer face of the width of the column section) were provided for the C-section asymmetric column to be attached to the concrete foundation.



**Figure (3.8):** #4 Rebars Installed to Foundation Using High-Strength Epoxy



**Figure (3.9):** Steel dowels Connecting Footing to Built-up Columns



## **3.2. Construction of Composite Built-up Columns**

### **3.2.1. Preparation of Symmetric & Asymmetric Column Specimens**

Symmetric and asymmetric built-up columns were developed by cutting into the shapes as per the structural engineering design and attaching the panels together with galvanized 9-gauge steel tie wires. Expanded Polystyrene form was cut out from the bottom part of the panels for a length of ten inches to make a 25.4mm (1") cavity surrounding the steel dowel bars connected from the footing to the column. The polystyrene was removed to provide mortar around the steel dowel bars to ensure that the dowels are fully embedded.

I-section column and the C-section column was attached to the dowels on the foundation and fastened by galvanized wires with appropriate means and it was made sure that the column is firmly connected to the reinforced concrete foundation.

### **3.2.2. Development of Wall Panels**

Wall panels were prepared by removing the core polystyrene from the part where it is embedded into the column as specified by the structural design. The empty space in the EPS core was filled with mortar to improve the strength of the column-wall connection joint. A U-shaped welded wire reinforcement was added to the part from which the foam was removed to ensure that no part of EPS core is exposed without welded-wire reinforcements all around it. Wall panels were attached to the column in such a way that the wall was not in contact with the foundation. A 177.8mm (7") gap from the surface of the footing was maintained as per the design.

### 3.2.3. Mortar Application

Hand application was the method followed to apply the mortar to the faces of the built-up column sections. Mortar was applied on all the outer faces of the column and the wall panels. For the inner parts of the specimen, mortar was poured inside from the top and vibrated, in a manner that complies with the IBC.

Mortar thickness was 25.4mm (1") over the wire mesh on all the side of the column and wall panel section. The constant thickness of the mortar shell on the specimen was achieved by placing guiding pieces of form wood at the ends of each joint. Exceptional care was taken to ensure complete filling of all the void spaces between the EPS insulation core and the welded-wire reinforcement.



**Figure (3.10):** Hand-applied Mortar on I-Section Column

Wood formwork was provided around the wall panel section to pour mortar into the column-wall connection joint from where the insulation foam core was removed. High strength structural mortar was placed from the top of the column to fill all the void space between the EPS core and welded-wire reinforcement and between the column-wall connection.



**Figure (3.11):** Mortar Poured from the Top to the Inner Faces of Column-Wall Connection



**Figure (3.12):** Hand-applied mortar on symmetric I-Section Column



**Figure (3.13):** Hand-applied mortar on Asymmetric C-Section Column

#### **3.2.4. Curing of Specimens**

All the specimens were cured for 7 days after the casting with mortar. Twenty-hours after the casting, the wood formwork around the wall panels were removed. Wet rugs were put around the specimens to keep the mortar in a wet condition during the curing period to attain the maximum strength for the mortar. The specimens were cured for seven days and were painted before taken for the testing procedure.



**Figure (3.14):** I-Section & C-Section Column Specimens After Curing & Painting

### **3.3. Testing Instrumentation**

In this section, the equipment and measuring instruments utilized for the lateral load testing of the built-up column specimens are presented elaborately. The equipment used for the test program consisted of hydraulic pumps to generate force, hydraulic jacks to impart loads, steel test frames to transfer the load uniformly throughout the specimens, tie-rods to constrain motion, strain gages to record strains of steel reinforcement and mortar, displacement sensors and transducers to capture the horizontal deflections and a calibrated data acquisition system to observe and record the experimental measurements. As per requirement, additional tools and supplemental equipment were also utilized.

### 3.3.1. Hydraulic Jack

A calibrated hydraulic jack with the hydraulic pump was used to apply the full monotonic lateral load. The hydraulic jack used was the ENERPAC RCH-603 Hollow Plunger Cylinder with a maximum operating pressure of 10,000 psi and a maximum stroke length of 3 inches. The cylinder had an effective area of 12.73 in<sup>2</sup> and a capacity of 60 tons. Weight of the hydraulic cylinder was 62 lbs.



**Figure (3.15):** Hydraulic Cylinder

A WIKA Model S-10 pressure transmitter was utilized to convert the pressure into analog electrical signals for the data and recording. The hydraulic cylinder and the pressure transmitter were calibrated and certified and were connected to a hydraulic pump. The loading was carried out manually and the loading frequency was maintained uniformly with the gage reading noted and recorded throughout.



**Figure (3.16):** Hydraulic Pump

### 3.3.2. String Potentiometers

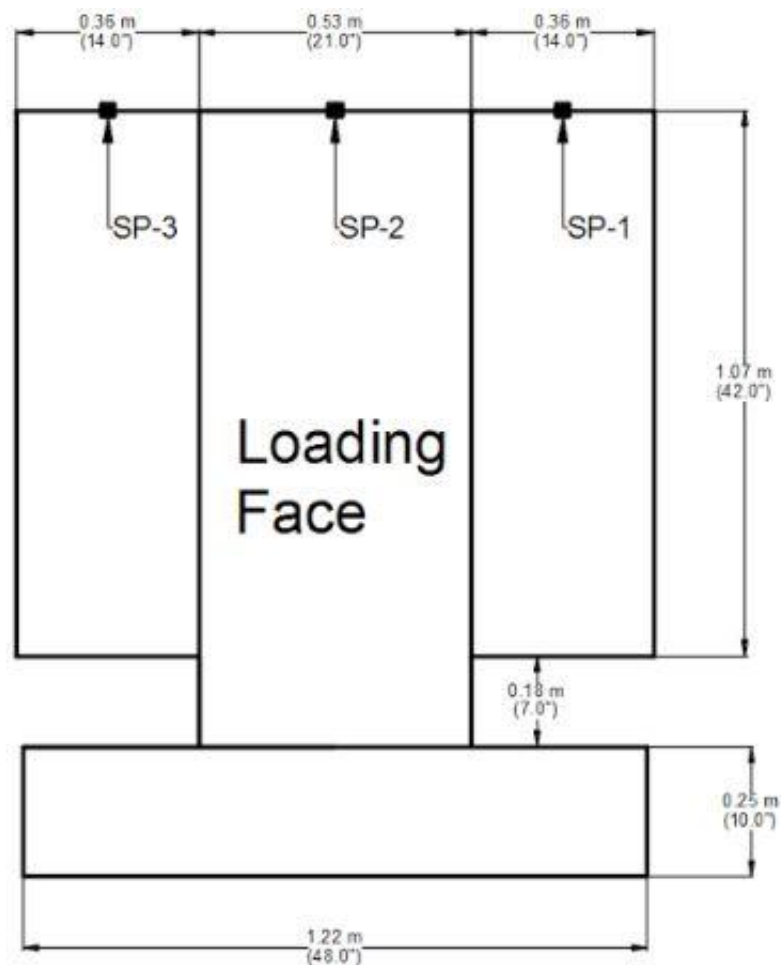
The lateral displacements were measured using String Potentiometers installed on the concrete wall against the built-up column specimens. Three String Potentiometers (String Pots) were used for both the column specimens and were individually calibrated. All deflection data was collected and recorded using a computerized data acquisition system. Details of the potentiometers are provided in Appendix (A).

For the I-section composite column, three string pots (*two on the wall panels and one on the column*) were placed on the laboratory concrete strong wall and was attached

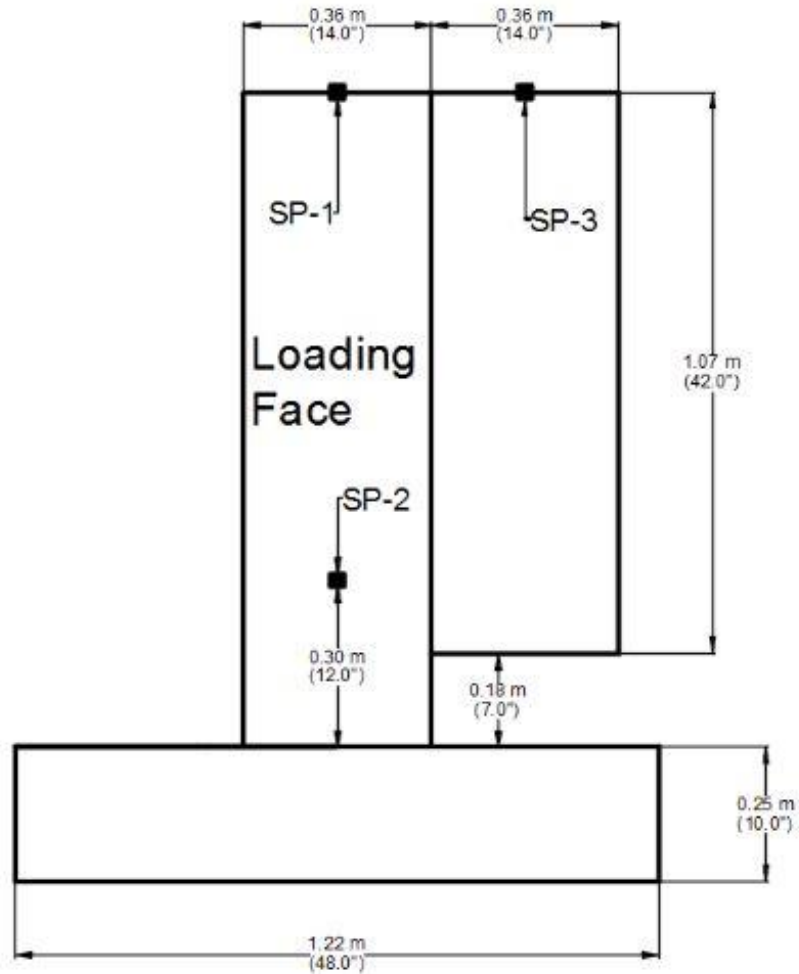


to the top of the vertical portion of the specimen by galvanized string ropes to record the horizontal deflections of the column and walls. For the C-section column, two string pots were installed on the laboratory concrete strong wall and was connected to the top vertical portion of the column and the wall of the specimen by means of string ropes and one string pot was connected at quarter-height of the column to record the horizontal displacement at the bottom of the column.

The locations of the string pot placement are showed in the following figures below.



**Figure (3.17):** Location of String Pots on I-section Column



**Figure (3.18):** Location of String Pots on C-section Column

### 3.3.3. Electronic Strain Gages

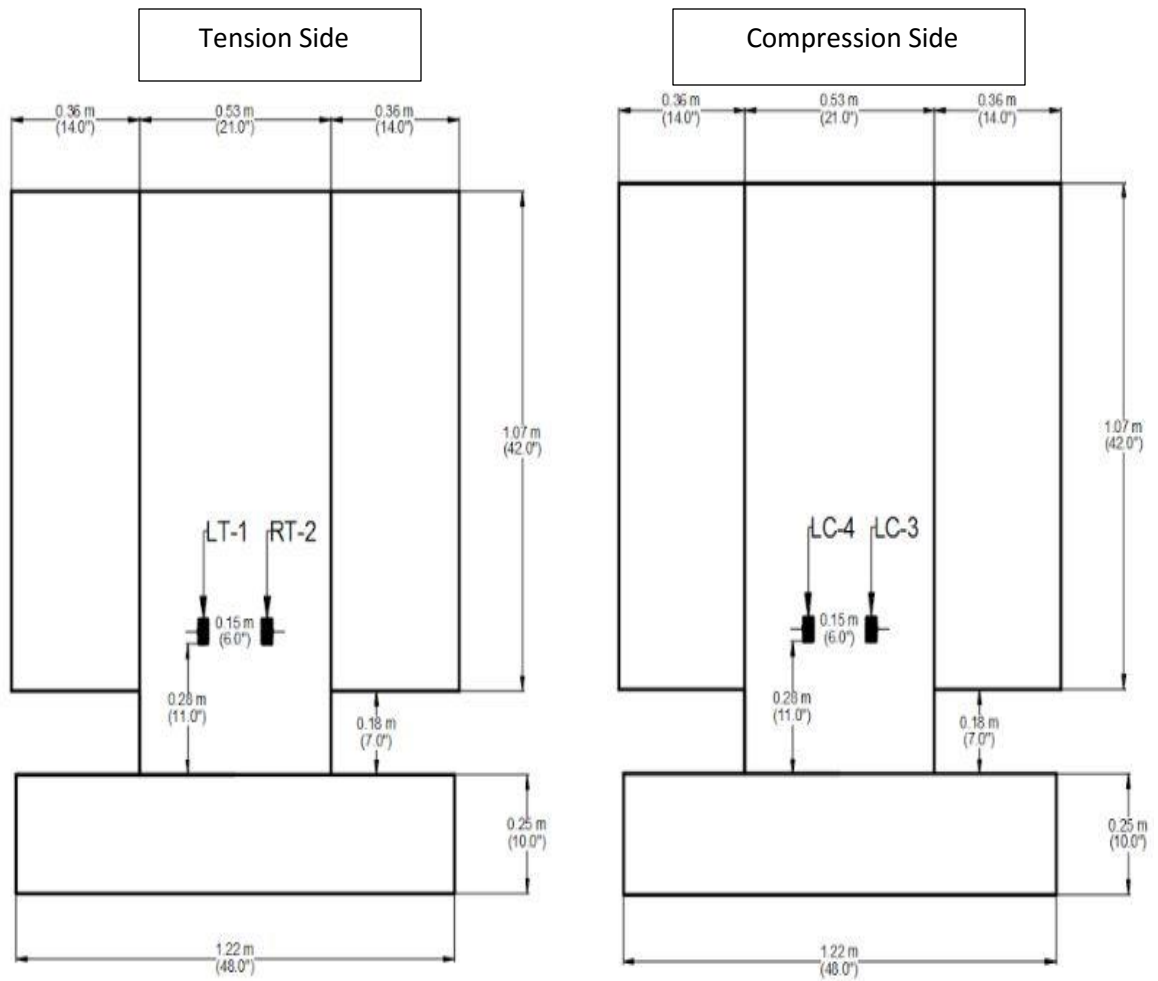
The composite column sections were instrumented with electronic foil resistance strain gages bonded at critical locations on each face of the mortar layer and on the internal welded-wire reinforcements. One strain gage each was installed on the mortar layer of the wall and column section on the compression side to measure the maximum strain values. Details of strain gages used are given in Appendix (A.)



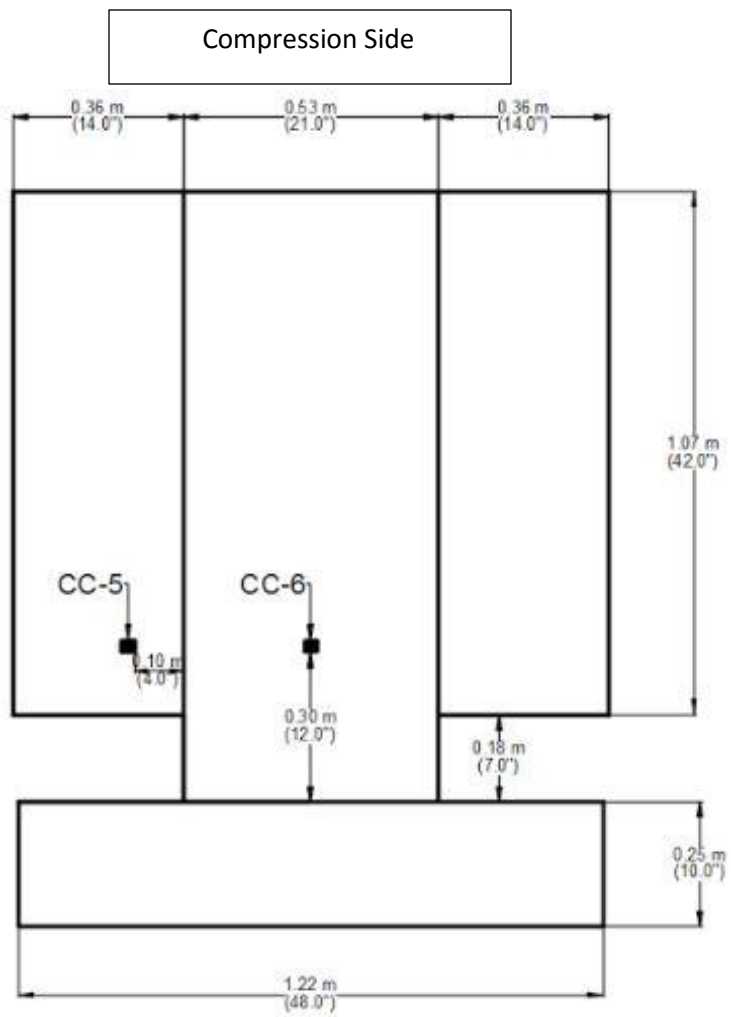
**Figure (3.19):** Strain Gauge Installed on Mortar Surface of Columns

The location of the strain gages on the I-section and C-section columns are illustrated in Figures 3.20 to 3.23. Electronic strain gages were installed on both the steel wire reinforcements and on the mortar layer of the composite columns to determine the change in strain with the corresponding loading. On the steel reinforcements of the I-section column, four strain gages (*two on tension side and two on compression side*) were installed and two strain gages (*one on the wall panel and one on the column*) were attached to the mortar layer of the column.

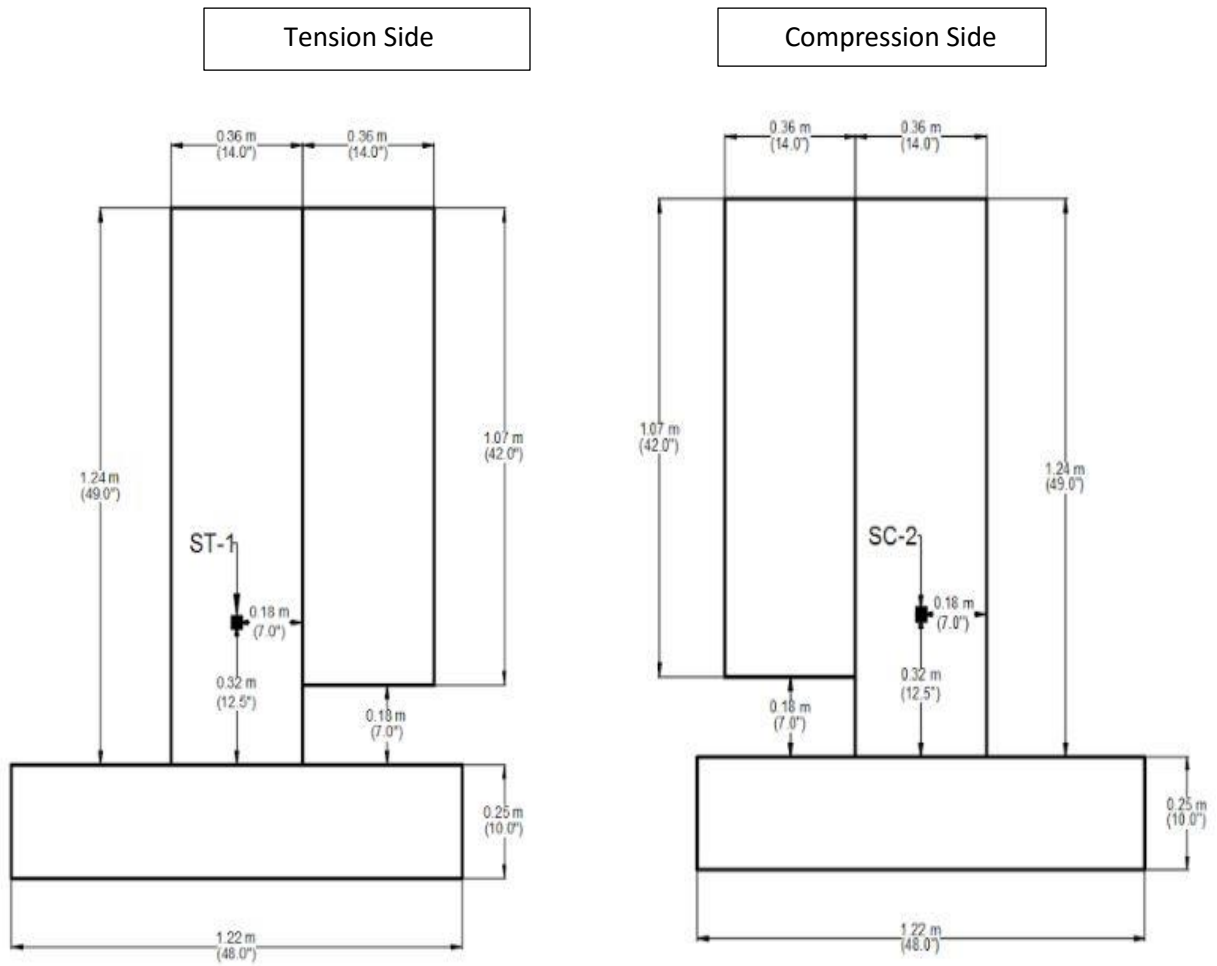
Similarly, for the C-section column, two strain gages (*one on the tension side and one on the compression side*) were installed on the predetermined location on the steel wire reinforcements and two strain gages (*one on the wall panel and one on the column*) were placed on the mortar layer on the compression side of the specimen.



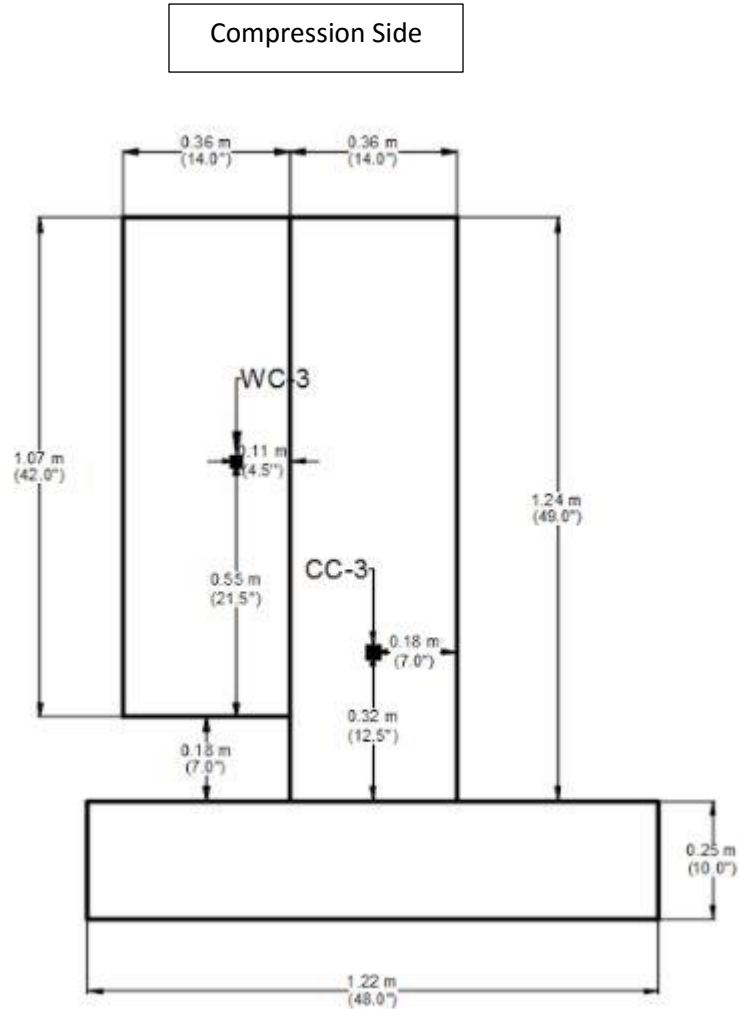
**Figure (3.20):** Strain Gauge Locations on Steel Wire Reinforcements of I-section Column



**Figure (3.21):** Strain Gauge Locations on Mortar Layer of I-section Column



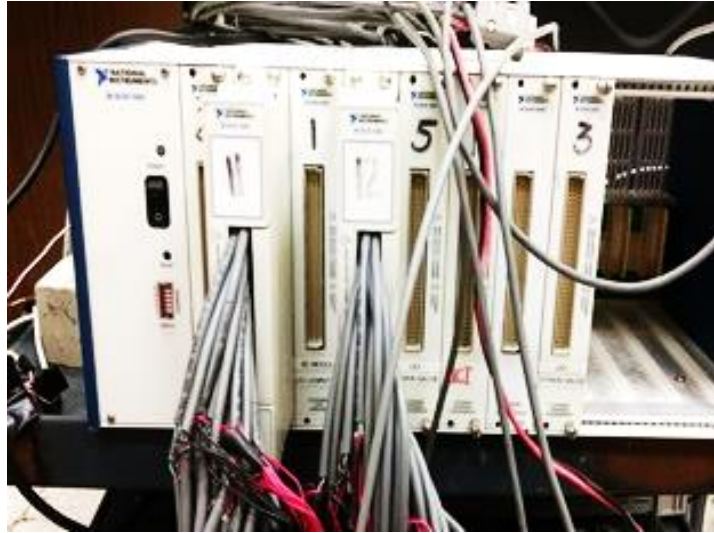
**Figure (3.22):** Strain Gauge Locations on Steel Wire Reinforcements of C-section



**Figure (3.23):** Strain Gauge Locations on Mortar Layer of C-section Column

### 3.3.4. Data Acquisition System

The National Instruments SCXI-1001 data acquisition chassis was used for acquiring the data of the experimental test evaluations. The data acquisition system was calibrated and the NI system documented data from load cells (pressure sensor), displacement transducers (string potentiometers), strain gages to provide the full perspective of the behavior of the test specimens under defined loading protocol.

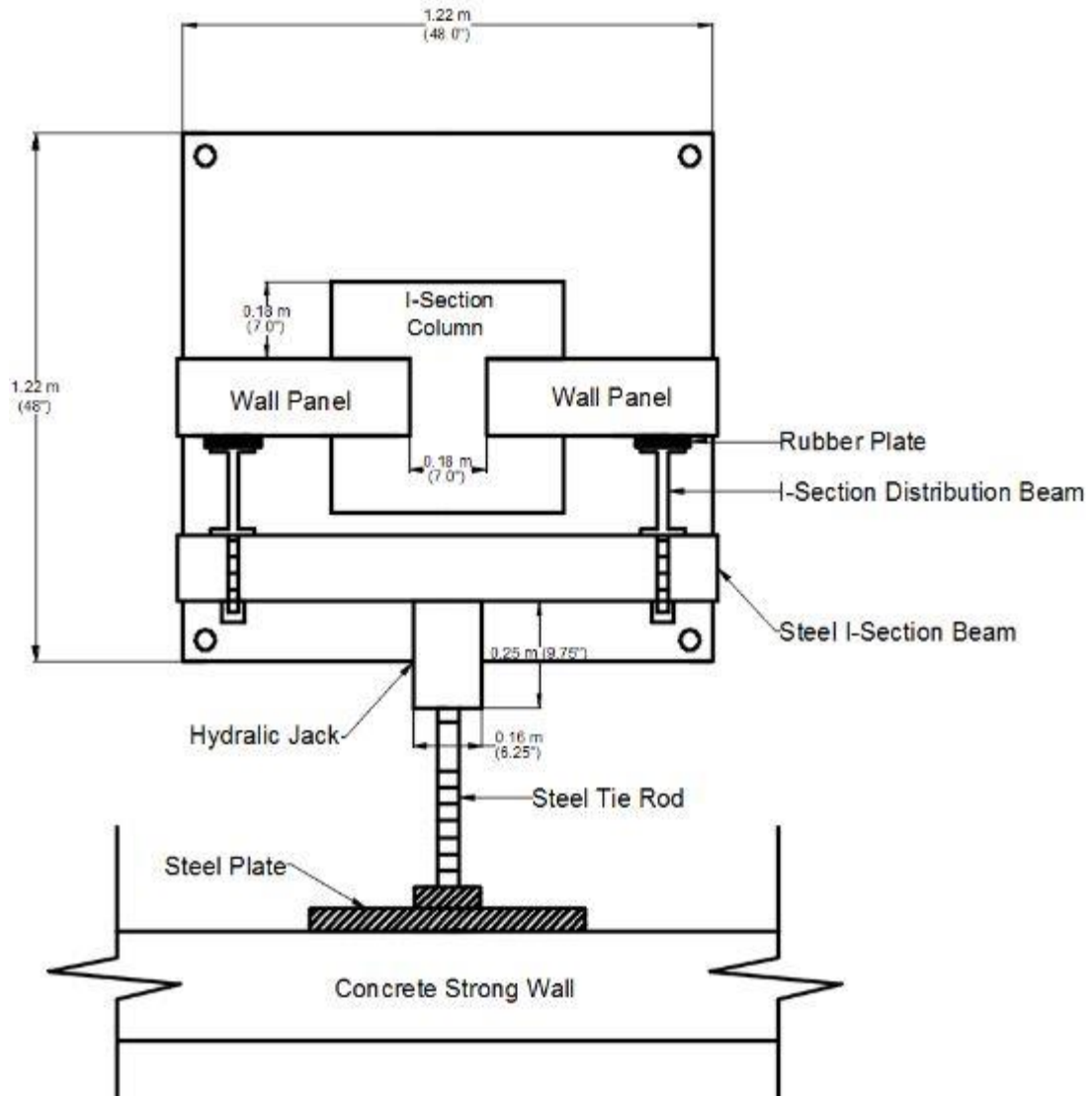


**Figure (3.24):** National Instrument Data Acquisition Chassis

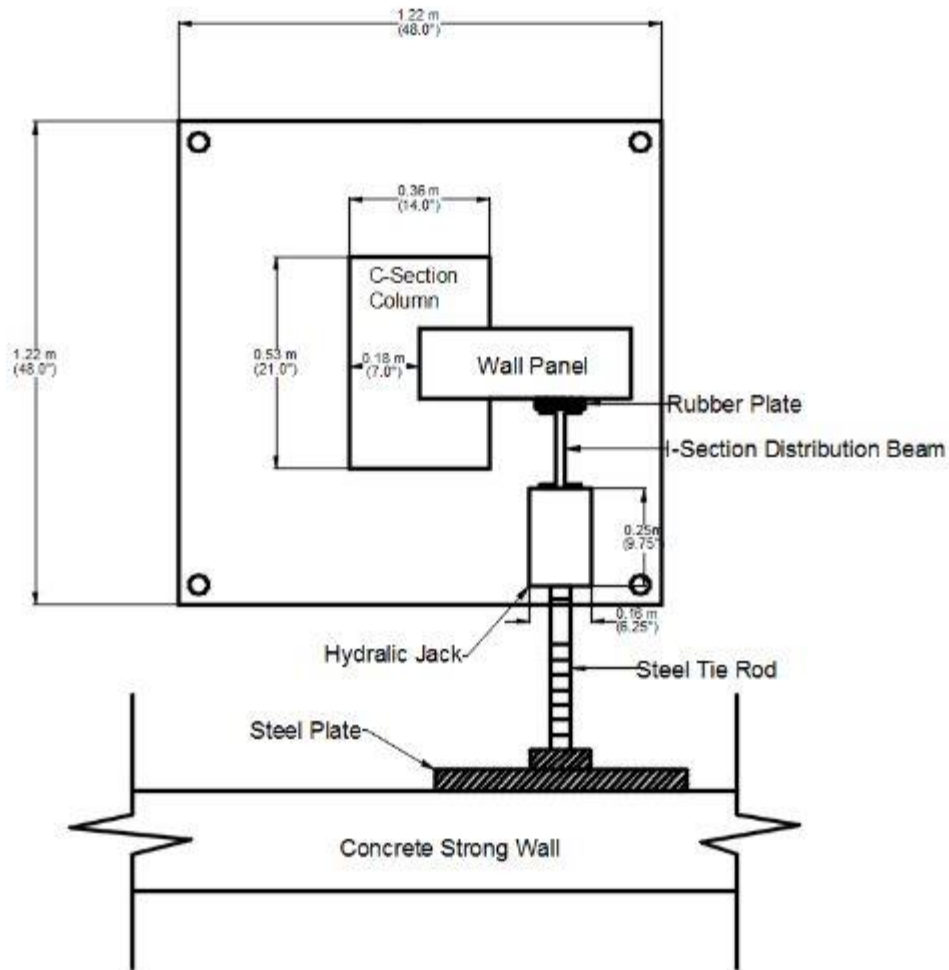
### **3.4. Test Protocol**

The experimental test set-up was prepared at the outside strong floor area of the SETH lab at UCI. The specimens were to be subjected to monotonic lateral loading. The loading points on the I-section were on the wall panels attached to the column. A uniformly distributed load was to be applied at the vertical portion of the specimen as shown in figure 3.35. For the asymmetric C-section column, the lateral load was applied on the wall panel projecting from the column. The steel testing frame was set-up accordingly to comply the testing on both the specimens.





**Figure (3.25):** Layout for Experimental Test Setup for Composite I-Section Column



**Figure (3.26):** Layout for Experimental Test Setup for Composite C-Section Column

The hydraulic jack was placed at the center of the loading frame so that the static lateral loads were uniformly applied over the phase of the wall panels that were attached to the built-up column, thereby inducing the forces to the columns. The loading of the hydraulic jack was done at a uniform frequency and a constant incremental loading rate was maintained and the readings were recorded during both the loading and unloading phase of the testing. The laptop connected to the data acquisition system constantly recorded and stored all the measurements obtained for

all the instruments installed to the columns. The data acquisition system was recording the strain values from the strain gages, the displacement of the columns from the potentiometers and the load applied at each stage from the load cell.

Load was applied until the complete failure on both the column sections. The maximum load was observed when the load reached a point where further loading was not possible and a drop in the load value was observed in the system. The total duration of the loading for the I-section column was 20 minutes and the duration for the C-section column was 15 minutes.

Crack initiation and propagation during the test and the crack width were measured manually and documented throughout the time of the testing. The failure mode and the crack pattern were identified and marked on the specimens during the testing. After the completion of the test all the cracks and failures were diagnosed and marked. The experimental test setup for the I-section column and the C-section column are shown in Figure 3.27 and 3.28 respectively.



**Figure (3.27):** Experimental Setup for Composite I-Section



**Figure (3.28):** Experimental Setup for Composite C-Section



**Figure (3.29):** Crack Pattern for Composite I-section Column



**Figure (3.30):** Crack Patterns for Composite C-Section Column

### **3.5 Experimental Results**

#### **3.5.1 Symmetric I-Section Column**

The lateral load was constantly applied on the column till it reached the ultimate failure and the data was monitored and stored. The maximum load taken by the composite I-section column was **93.506 kN** (21.021 kips).

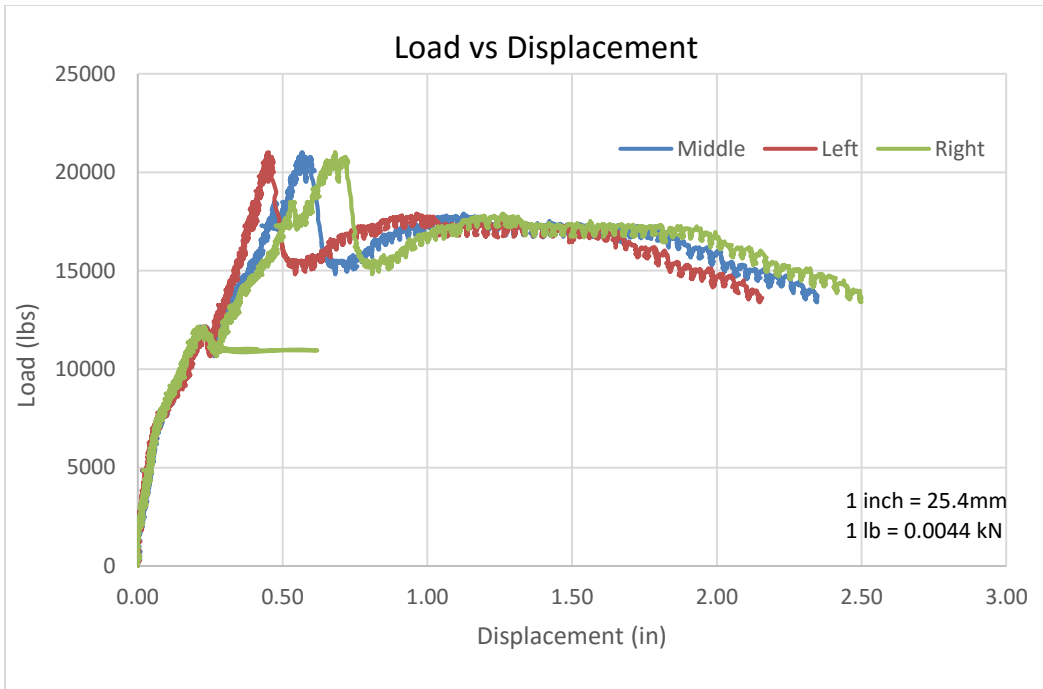
No cracks were seen in the first stage of loading where the force was applied uniformly at a constant pace. As the load increased, diagonal hair cracks on the mortar

layer of the column and the wall panels were observed. The first crack was observed at **42.258 kN** (9.50 kips). Cracks were observed on the tension face of the column and wall panels. Large cracks were seen at the column-footing connection and the few cracks were observed on the foundation as well.

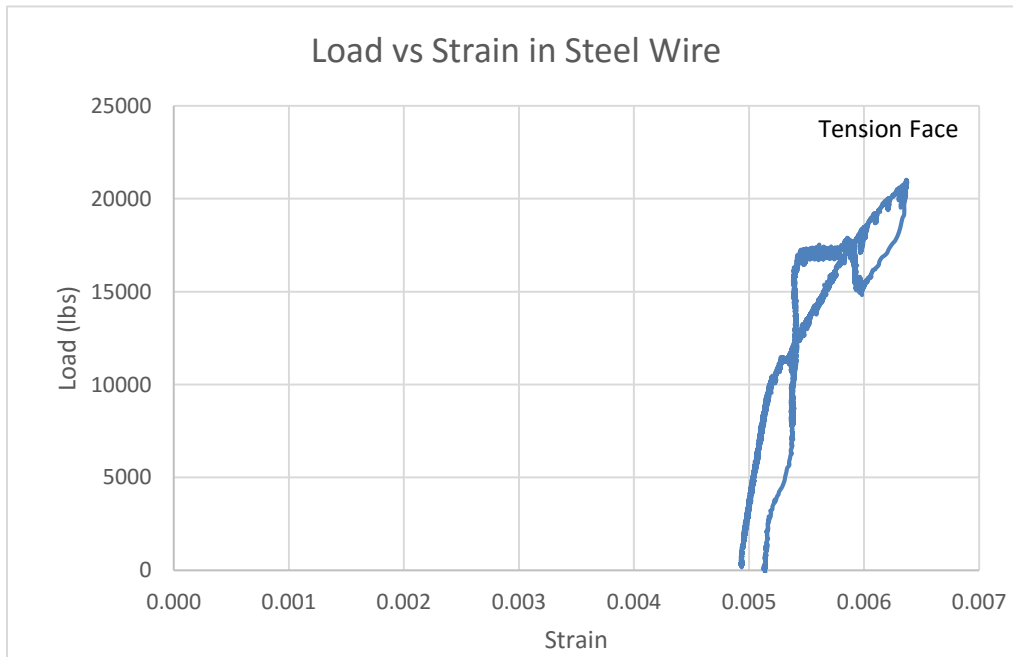
The deflection of the top of the column with respect to the bottom was measured by the potentiometers installed and the maximum displacement occurred to the specimen at peak load was 19.05mm (0.75 inches). The failure mode shows that the ultimate failure occurred at the column-footing connection. The relationships between the load vs displacement is presented in Figure 3.32.



**Figure (3.31): Large Cracks Observed at the Column-Footing Joint of I-section Column**

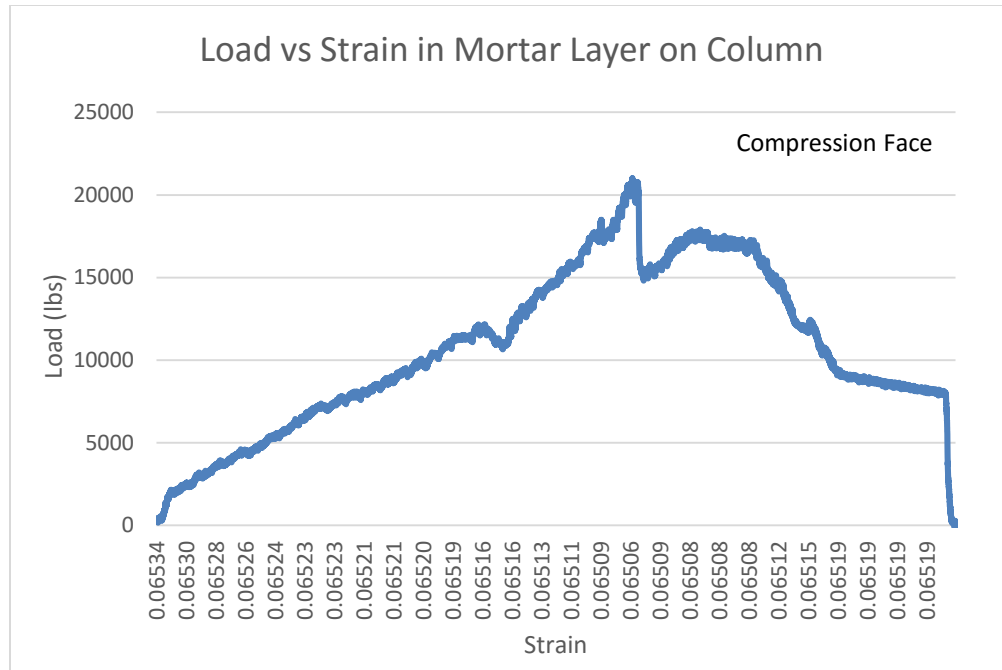


**Figure (3.32):** Load-Displacement Curve for Composite I-section Column



**Figure (3.33):** Load-Strain Curve for Steel Wires on Tension Side of I-section Column





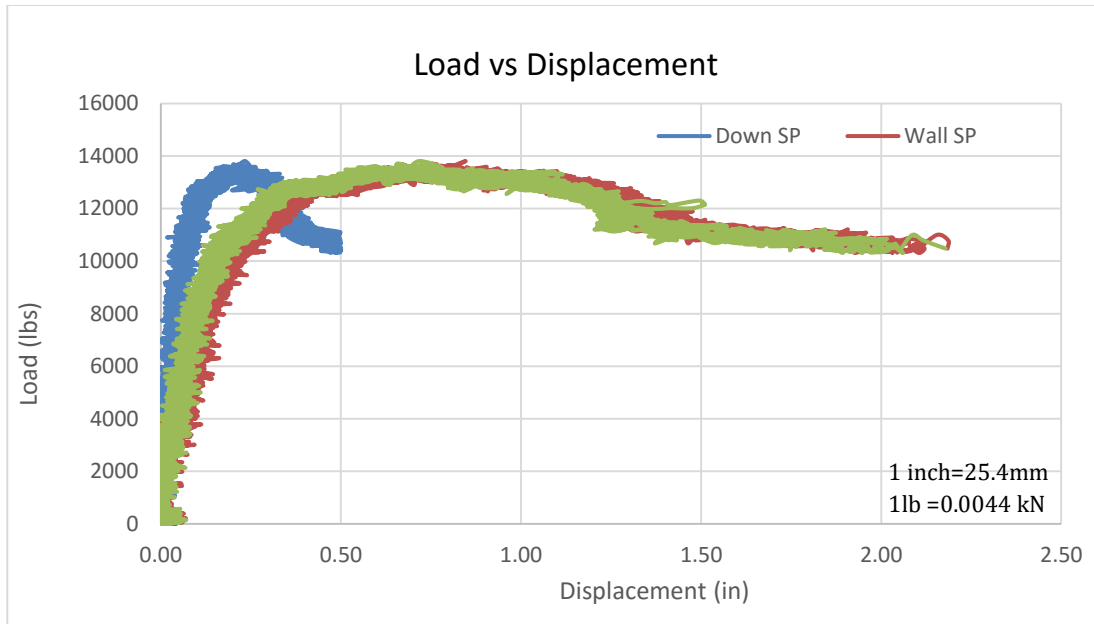
**Figure (3.34):** Load- Strain Curve for Mortar Layer on I-section Column (Compression Side)

### 3.5.2 Asymmetric C-Section Column

The lateral load was constantly applied and the maximum load taken by the composite I-section column was **61.425 kN** (13.809 kips). Diagonal and transverse cracks were observed on the tension face of the column and wall panels. The first crack on the surface of the mortar layer was observed at **32.916 kN** (7.40 kips). Large cracks were seen at the column-footing connection and the few cracks were observed on the foundation as well. The maximum displacement occurred to the specimen at peak load was 22.098mm (0.87 inches) and this is the deflection of the top of the column with respect to the bottom of the column.



**Figure (3.35):** Cracks Observed on the Mortar Surface of C-section Column



**Figure (3.36):** Load-Displacement Curve for Composite C-section Column

### 3.6 Analytical Design Procedure

The theoretical analysis of the built-up columns was performed and all the calculations comply with the provisions of ACI 318-14 and the modifications required for determining the flexural capacity, shear and deflection of the composite columns were executed based on the Structural Engineering Handbook of EVG 3D Construction System.

For the analysis, the strength of the EPS foam was neglected as it does not contribute to the strength of the built-up sections. It was assumed that a full composite action was achieved for the structural element as the inclined shear connectors was provided in the sandwich panel to carry and transfer the shear forces.

### 3.6.1 Flexural Strength

The flexural calculations for the built-up column sections are governed by ACI 318-14. The strength reduction factor for flexure is taken as  $\Phi = 0.75$ . The ultimate failure of the built-up sections was observed from the experimental testing at the tension side of the columns due to the inherent brittle nature of the steel wire mesh reinforcements.

#### I-Section Column

Yield strength of steel (for ST 500),  $f_y = 500 \text{ N/mm}^2$  (72518.86 psi)

Compressive strength of mortar,  $f_c = 25.384 \text{ MPa}$  (3681.71 psi)

No. of steel wire reinforcements = 22

Mortar cover,  $c = 25.4 \text{ mm}$  (1")

We have,  $a = 0.85 \times c = 0.85 \times 25.4 \text{ mm} = 21.59 \text{ mm}$  (0.85")

Diameter of the welded-wire reinforcement = 3 mm (0.118")

Area of steel,  $A_{st} = \text{No. of steel wires} \times \text{Area of each steel wire}$ , or

$$A_{st} = 22 \times \frac{\pi d^2}{4} = 22 \times \frac{\pi \times 3^2}{4} = 155.50 \text{ mm}^2 \text{ (0.240 in}^2\text{)}$$

Maximum Moment from factored loads,

$$M_u = \Phi \times A_{st} \times f_y \left( d - \frac{a}{2} \right)$$

$$= 0.75 \times 155.50 \times 500 \left( 177.8 - \frac{21.59}{2} \right)$$

$$\mathbf{M_u = 9.720 \text{ kN.m (86.035 kip-in)}}$$

Nominal flexural strength with the welded-wire reinforcements and mortar wythes,

$$M_n = \frac{Mu}{\Phi} = \frac{9.720}{0.75} = 12.960 \text{ kN.m (114.714 kip-in)}$$

$$\text{Hence, Load, } P_n = \frac{M_n}{e} = \frac{12.960}{0.1778 \text{ m}} = 72.890 \text{ kN (16.385 kips)}$$

### **C- Section Column**

Yield strength of steel (for ST 500),  $f_y = 500 \text{ N/mm}^2 (72.518 \text{ ksi})$

Compressive strength of mortar,  $f'_c = 25.264 \text{ MPa}(3.6643 \text{ ksi})$

No. of steel wire reinforcements = 14

Mortar cover,  $c = 25.4 \text{ mm (1")}$

We have,  $a = 0.85 \times c = 0.85 \times 25.4 \text{ mm} = 21.59 \text{ mm (0.85")}$

Diameter of the welded-wire reinforcement = 3 mm = 0.118 in.

Required strength  $\leq$  Design Strength

Area of steel,  $A_{st} = \text{No. of steel wire} \times \text{Area of each steel wire}$

$$A_{st} = 14 \times \frac{\pi d^2}{4} = 14 \times \frac{\pi \times 3^2}{4} = 98.96 \text{ mm}^2 (0.153 \text{ in}^2)$$

Maximum Moment from factored loads,

$$M_u = \Phi \times A_{st} \times f_y \left( d - \frac{a}{2} \right)$$

$$= 0.75 \times 98.96 \times 500 \left( 177.8 - \frac{21.59}{2} \right)$$

$$M_u = 6.181 \text{ kN.m (54.713 kip-in)}$$

Nominal flexural strength with the welded-wire reinforcements and mortar wythes,

$$M_n = \frac{Mu}{\Phi} = \frac{6.181}{0.75} = 8.24 \text{ kN.m (72.951 kip-in)}$$

$$\text{Hence, Load, } P_n = \frac{M_n}{e} = \frac{8.240}{0.1778 \text{ m}} = 46.344 \text{ kN (10.421 kips)}.$$

### 3.6.2 Shear Capacity

The total shear capacity is equivalent to the sum of the concrete shear strength and the steel shear strength. The shear force taken by the diagonal shear connectors can be ignored for determining the total shear capacity of the symmetric and asymmetric composite columns. As there were no loads acting on top of the columns, it was assumed that the columns behave as a vertical cantilever. So, one end of the column was considered as a fixed support and the other end free and therefore no displacement or rotation constraints at the free end. Hence, for all the calculations, the gravity load acting on the column sections were neglected.

$$V_c = 2\sqrt{f'_c}hd \quad \text{ACI-318 14 Eq. 22.5.5.1}$$

Area = d x h (where, d = only one layer depth of concrete = 25.4mm (1"); h= height of the built-up column section)

$$V_s = \frac{A_s f_y d}{s} \text{ (where, } s \text{ is the spacing between the wires and } d \text{ is the effective horizontal depth of the column section)}$$

Each wire area = 7.74 mm<sup>2</sup> (0.012 in<sup>2</sup>) spaced at 50.8 mm (2") (at both sides of the panel)

$$A_s = 2 \times 7.74 = 15.48 \text{ mm}^2 \left(0.024 \frac{\text{in}^2}{\text{ft}}\right)$$

$$\text{Spacing, } s = 50.8 \text{ mm (2")}$$

Therefore, Nominal Shear Capacity,  $Q = V_c + V_s$  (kN) or (kips)

### **I-Section Column**

$$V_c = 26.450 \text{ kN (5.9463 kips)}$$

$$V_s = \frac{A_s f_y d}{s} = 15.48 \times 500 \times 533.40 / 50.8 = 81.280 \text{ kN (18.274 kips)}$$

**Hence, total shear capacity,  $Q = V_c + V_s = 107.73 \text{ kN (24.220 kips)}$ .**

### **C-Section Column**

$$V_c = 26.387 \text{ kN (5.9322 kips)}$$

$$V_s = \frac{A_s f_y d}{s} = 15.48 \times 500 \times 355.6 / 50.8 = 54.180 \text{ kN (12.1830 kips)}$$

**Hence, total shear capacity,  $Q = V_c + V_s = 80.579 \text{ kN (18.115 kips)}$**

### **3.6.3 Deflection**

For a column that is fixed at the bottom and free at the top without any supports, the total in-plane lateral deflection at the top of the column with respect to the bottom of the column can be calculated as following:

$$\Delta_c = \Delta_b + \Delta_v = \frac{Vh^3}{3E_m I_{eff}} + \frac{1.2Vh}{E_v A}$$

Where,  $\Delta_b$  = In-plane lateral deflection due to flexure (mm or in)

$\Delta_v$  = In- plane lateral deflection due to shear (mm or in)

V = Lateral load at nominal load (kN or kips)

H = Total height of the column (mm or in)

$E_c$  = Modulus of Elasticity of mortar ( $E_c = 4700\sqrt{f'_c}$  MPa (or)  $57000\sqrt{f'_c}$  lb/in<sup>2</sup>)

$E_v$  = Shear Modulus =  $0.4 E_c$  (Mpa or ksi) (ACI 530-92/ASCE 5-92)

A = Horizontal cross-sectional area of the column faces (mm<sup>2</sup> or in<sup>2</sup>)

$I_{eff}$  = Effective moment of inertia =  $2I_{gross}/5$  (Conservative approach) (mm<sup>4</sup> or in<sup>4</sup>)

### **I-section Column**

V= 72.890 kN (16.385 kips)

h= 1244.6 mm (49")

$E_c = 4700\sqrt{f'_c}$  or  $57000\sqrt{f'_c} = 23679.79$  MPa ( $3.4585 \times 10^6$  psi)

$E_v = 0.4 E_c = 9471.916$  MPa ( $1.3834 \times 10^6$  psi)

A =  $2 \times 25.4 \times 1244.6 = 63225.68$  mm (98")

$I_{eff} = 2I_{gross}/5 = 2.631 \times 10^9$  mm<sup>4</sup> ( $6322.634$  in<sup>4</sup>)

$$\Delta_c = \Delta_b + \Delta_v = \frac{72.890 \times 10^3 \times 1244.6^3}{3 \times 23679.79 \times 2.631 \times 10^9} + \frac{1.2 \times 72.890 \times 10^3 \times 1244.6}{9471.916 \times 63225.68}$$



**Hence,  $\Delta_c = 0.9245 \text{ mm (0.0364 in)}$**

### **C-section Column**

$V = 46.344 \text{ kN (10.421 kips)}$

$h = 1244.6 \text{ mm (49")}$

$E_c = 4700\sqrt{f'_c}$  or  $57000\sqrt{f'_c} = 23623.754 \text{ MPa (3.450 x 10}^6 \text{ psi)}$

$E_v = 0.4 E_c = 9449.501 \text{ MPa (1.380 x 10}^6 \text{ psi)}$

$A = 2 \times 25.4 \times 1244.6 = 63225.68 \text{ mm (98")}$

$I_{\text{eff}} = 2I_{\text{gross}}/5 = 1.765 \times 10^9 \text{ mm}^4 (4241.766 \text{ in}^4)$

$$\Delta_c = \Delta_b + \Delta_v = \frac{46.344 \times 10^3 \times 1244.6^3}{3 \times 23623.754 \times 1.765 \times 10^9} + \frac{1.2 \times 46.344 \times 10^3 \times 1244.6}{9449.501 \times 63225.68}$$

**Hence,  $\Delta_c = 0.8229 \text{ mm (0.0324 in)}$**

### **3.7 Summary & Concluding Remarks**

The experimental testing of the I-section and C-section composite columns provided the maximum lateral load carrying capacity of the specimens and the deflection of the column specimens and were presented. The analytical calculations of the symmetric I-shaped built-up column and asymmetric C-shaped built-up column were also discussed in this chapter.

The comparison between the experimental load capacity and the predicted

theoretical load capacity indicates that the maximum load obtained from the experimental testing of both the columns are higher than the analytical results and hence the design is said to be conservative and are comparable to one another. The summarized results from the experimental evaluation and the analytical calculation are tabulated in Tables 3.6 and 3.7.

**Table (3.6):** Maximum Load, Deflection and Cracking Load from Experimental Results

<b>Column Section</b>	<b>Maximum Load (<math>P_{max}</math>) kN (kips)</b>	<b>Cracking Load (<math>P_{cr}</math>) kN (kips)</b>	<b>Maximum Deflection <math>\Delta_{max}</math>, mm(in)</b>
I	93.506 (21.021)	42.258 (9.50)	19.05 (0.75)
C	61.425 (13.809)	32.916 (7.40)	22.098 (0.87)

**Table (3.7):** Analytical Results of Flexural Strength, Shear Strength, Deflection of Columns

<b>Column Section</b>	<b>Maximum Load, P kN (kips)</b>	<b>Flexural Capacity <math>M_n</math> kN.m(kips-in)</b>	<b>Shear Capacity, <math>V_n</math> kN (kips)</b>	<b>Total Lateral Deflection <math>\Delta_c</math> mm(in)</b>
I	72.887 (16.385)	12.960 (114.71)	107.735 (24.220)	0.9245 (0.0364)
C	46.344 (10.421)	8.24 (72.951)	80.579 (18.115)	0.8229 (0.0324)

## CHAPTER 4

### NUMERICAL ANALYSIS USING ANSYS

#### 4.1 Modeling Approach

This chapter of the thesis explains the steps followed for the modelling and analysis of built-up composite columns in finite element analysis program. It was required to model the complete structure that includes the footings, walls and column in an existing finite element based analysis software and then carry out the analysis. To get the best results, it was necessary to model the system as accurately as possible.

First test case was taken as the symmetric I-section column and was modelled in ANSYS 17.1. After modelling, it was analyzed for static lateral loading as it was done in the experimental evaluation. The results were then compared to the actual results from the experimental analysis. It was made sure that the model which was developed can produce results as the same way it will do in the case of actual experiments. Similarly, for the second test case, the asymmetric C-section column was designed in the ANSYS's Design Modeler and the boundary conditions as well as the loading parameters were given as input. Then the section was run to analyze so as to determine the maximum load carrying capacity and the displacements taken by the composite columns.

ANSYS 17.1 Workbench was utilized to model and analyze the composite column specimens. Depending on the kind of analysis that must be carried out, there were

diverse options in ANSYS program as it was a Multiphysics software. Out of those, ANSYS Mechanical – Static Structural was used, as our model is subjected to static load testing.

The model was subjected to various stages of development before the actual analysis. The first step was designing the geometry and structure of the column specimens. In the Design Modeler of ANSYS, the whole specimen model was developed part by part. Modelling of each structural component was advanced carefully examining the dimensions and designing on its basis. All the components were designed separate to the other components and were assigned material properties correspondingly. The engineering properties of all the materials were defined and were assigned to the components at the start of the modelling. The material properties and definition are described in the following section.

## **4.2 Materials Definitions**

The engineering data for each component was defined for each material type and was later assigned to each component accordingly. For the column, it was assumed to be an elastic material with a bilinear isotropic plasticity. Each part of the column sections was initially drawn as per their geometric dimensions and the material properties were allocated separately.

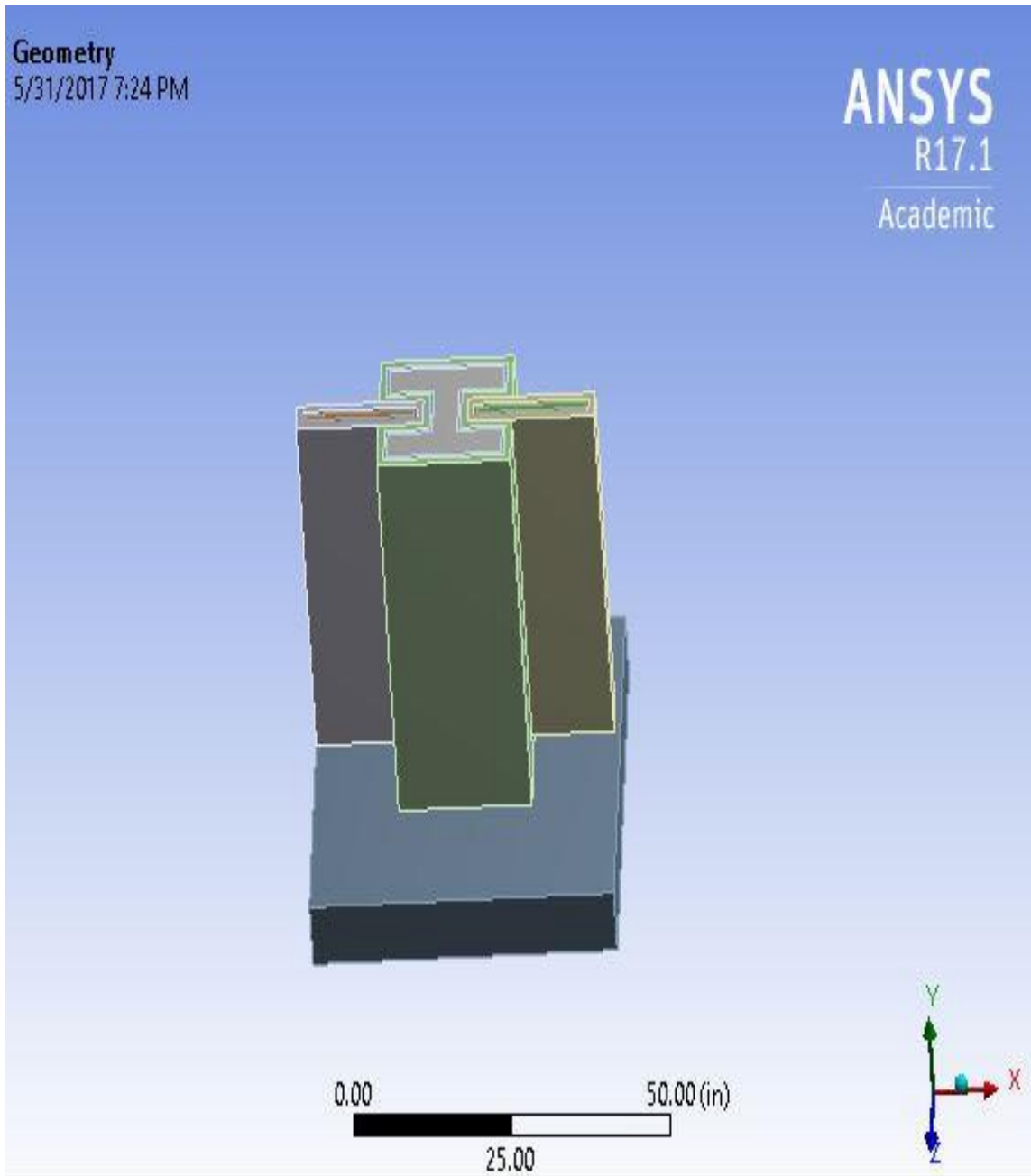
Density, Poisson's ratio, yield strength and the ultimate strength, were also defined for the steel separately. The Poisson's ratio for the steel wire mesh and mortar was given as 0.3 and 0.15 respectively. The tensile yield strength and the compressive

yield strength of steel was designated in the Engineering Data section of the ANSYS software. The compressive yield strength and compressive ultimate strength of the mortar as calculated from the testing results for the two different column sections were given as input definition. The expanded polystyrene core form has very low density and modulus of elasticity and hence and the EPS was assumed in this study to have no structural influence in the structural behavior of the specimens, and hence, it was not considered in the FEM model.

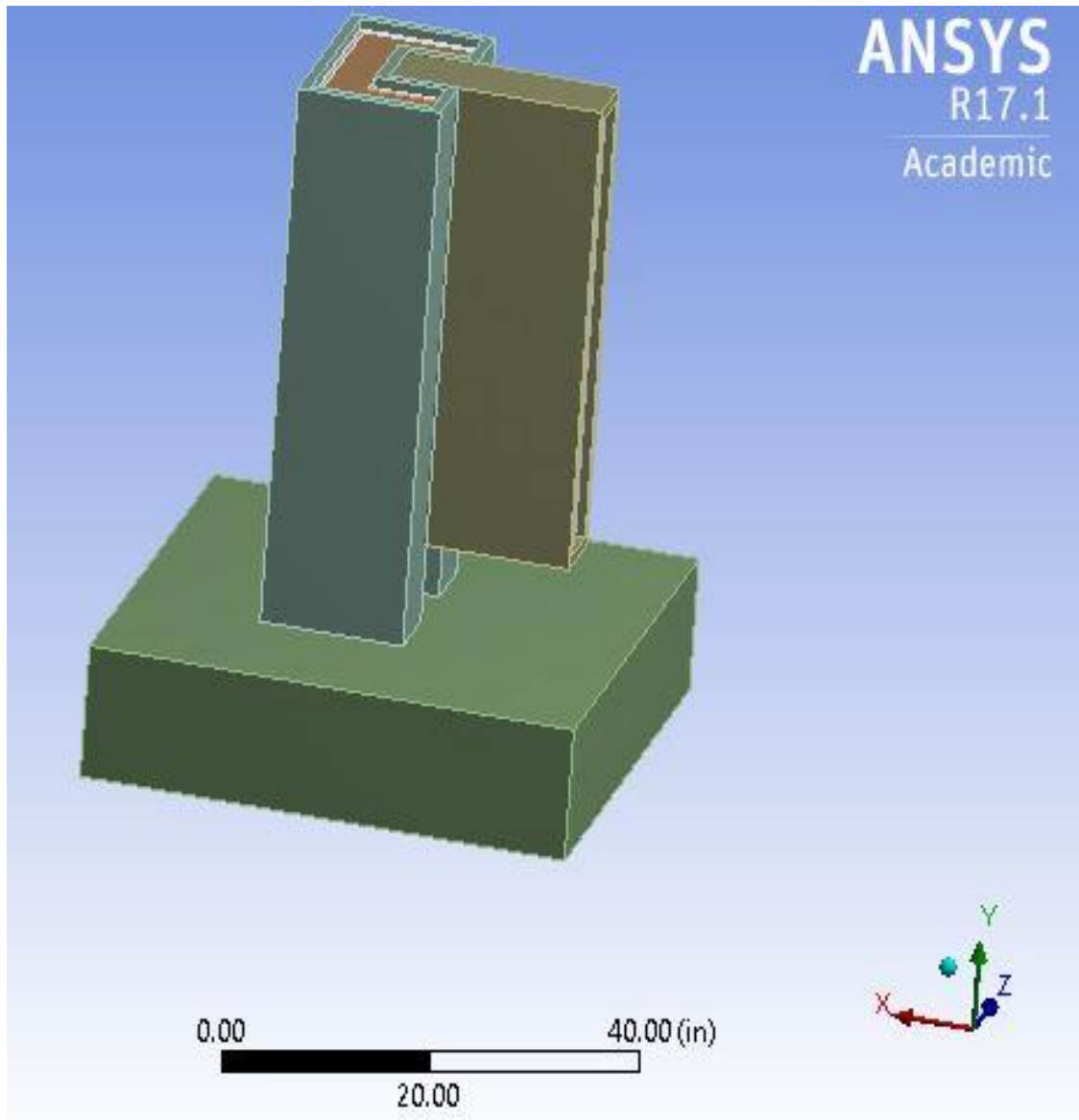
The material properties for each of the structural element were then applied to each of the respective materials comprising the finite element models. Each material was identified by the distinct color codes assigned to them. The EPS core foam material was white, the steel wire mesh that forms the second layer was yellow and the outer mortar wythes were colored green.

### **4.3 Modal Analysis & Meshing**

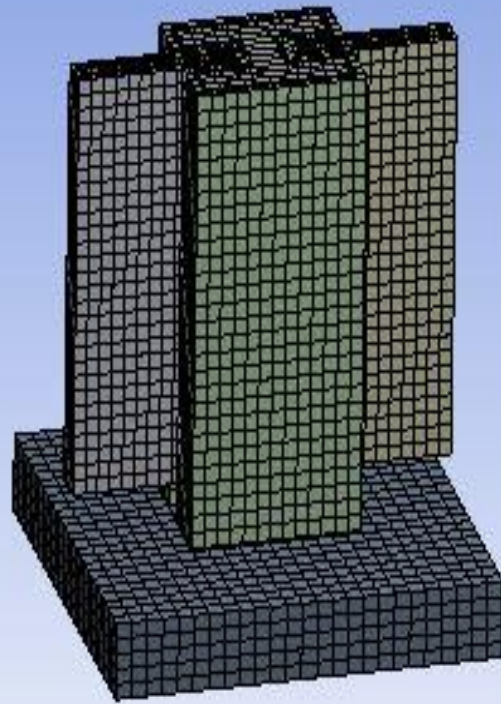
After the designing, the contacts between each set of components were defined separately according to their behavior in the structure. ANSYS has the capability to automatically recognize if two bodies are touching together and then they represent it as a contact. These contacts were then defined separately according to the behavior of each column specimens. Mesh size of 25.4mm x 25.4mm (1"x1") was chosen for the mortar shell elements. Element size of 25.4mm (1") was given for the steel welded-wire reinforcements. All the elements in the model were given the same mesh size so that the nodal location does not change and the boundary conditions are applied.



**Figure (4.1):** Modelled Geometry of I-section Column

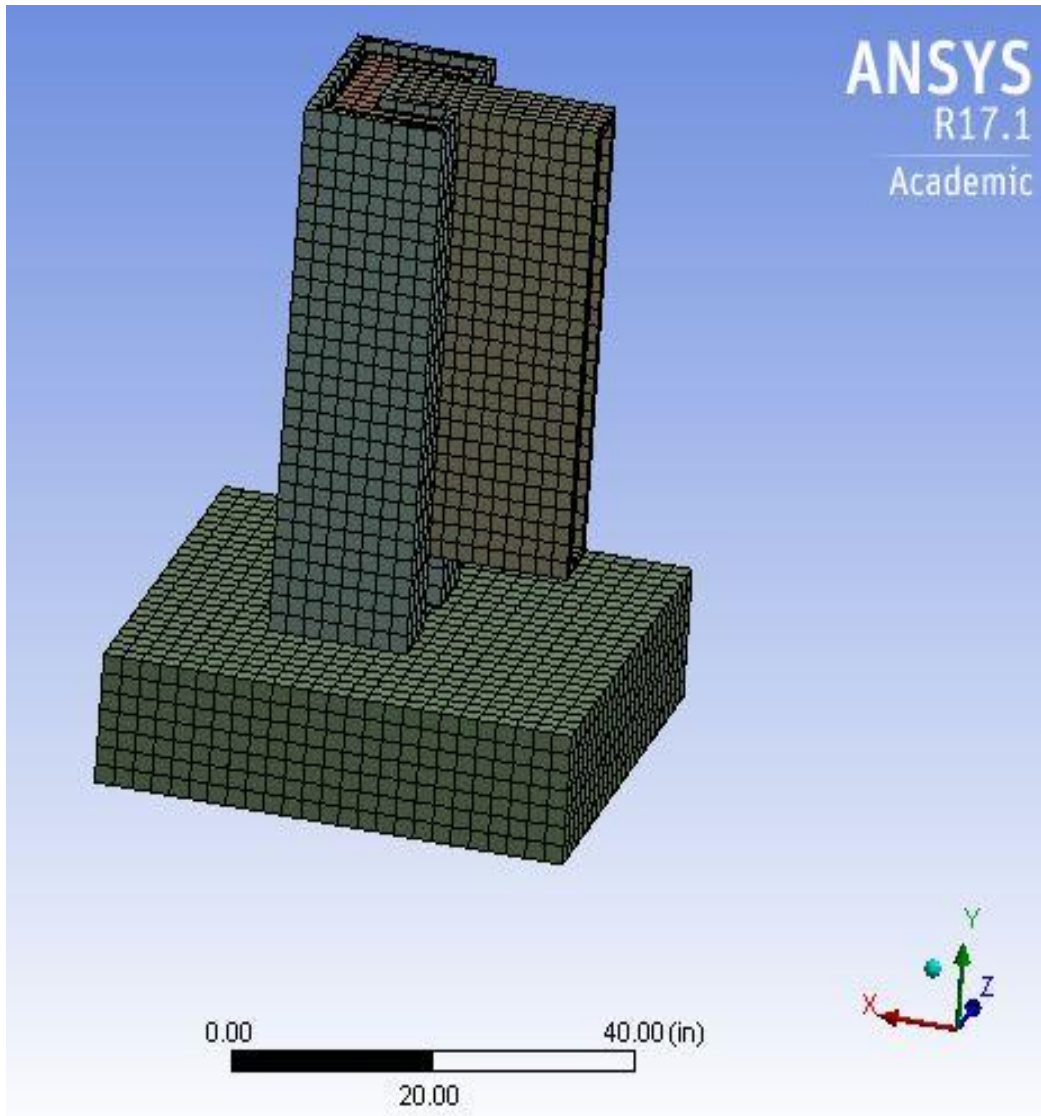


**Figure (4.2):** Modelled Geometry of C-section Column



**Figure (4.3):** Meshing of I- section Column Model





**Figure (4.4):** Meshing of C- section Column Model

#### **4.4 Loads & Boundary Conditions**

For the Finite Element Analysis, the lateral load was applied as a monotonic incremental displacement applied along the vertical surface of the wall panels. The loads were uniformly distributed along the face of the wall panels attached to the column sections. The eccentricity of the loading location was considered and defined

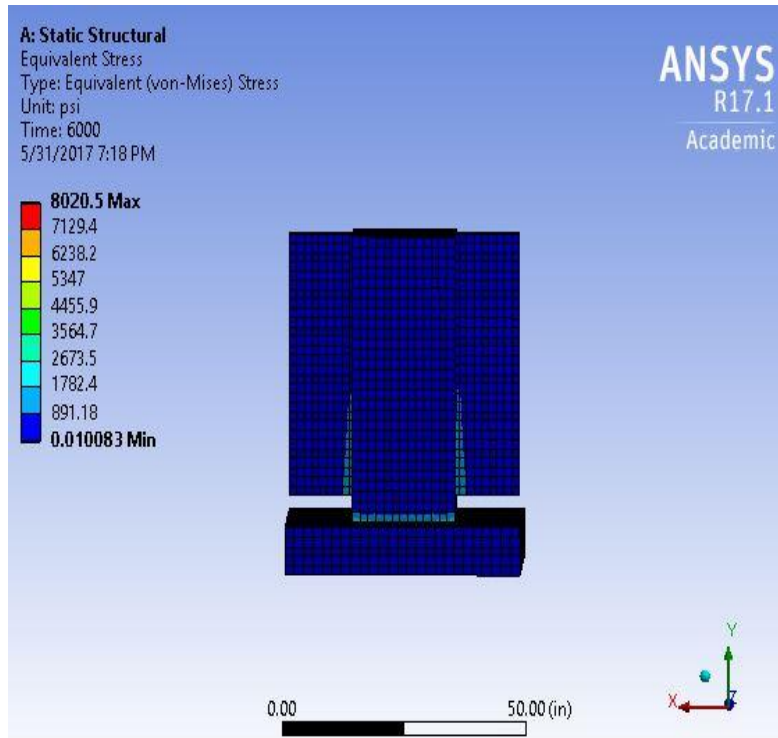
the similar as done in the experimental testing. A maximum load of 133.44 kN (30 kips) was incorporated on the columns, so that the columns fail before reaching the maximum load when the failure occurs. The boundary conditions applied were the fixed foundations to the strong floor of the laboratory.

#### **4.5 Verification of Results & Comparison to Experimental Results**

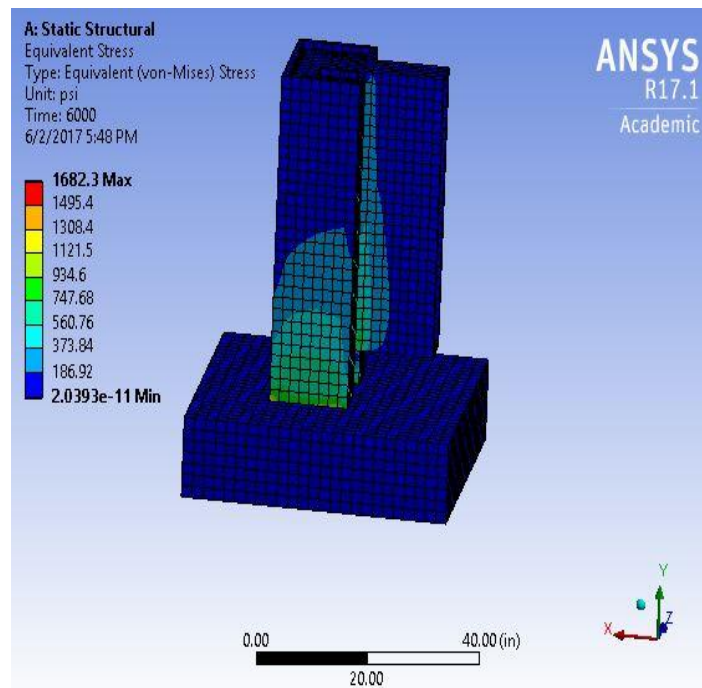
The results from the FEM analysis was compared to the results of the experimental evaluation and theoretical calculation of strength parameters of the I and C shaped built-up column specimens. The numerical load-displacement curve was obtained from the model and is plotted the load-displacement curve from the experimental results. The stresses and strains produced on the column section models were recorded at each predefined increment till the failure of the sections. The results obtained after running the analysis is presented in this section.

The combined stress distribution and the maximum stress concentration at the connections between the column-footing and the column-wall were observed and displayed in the results.

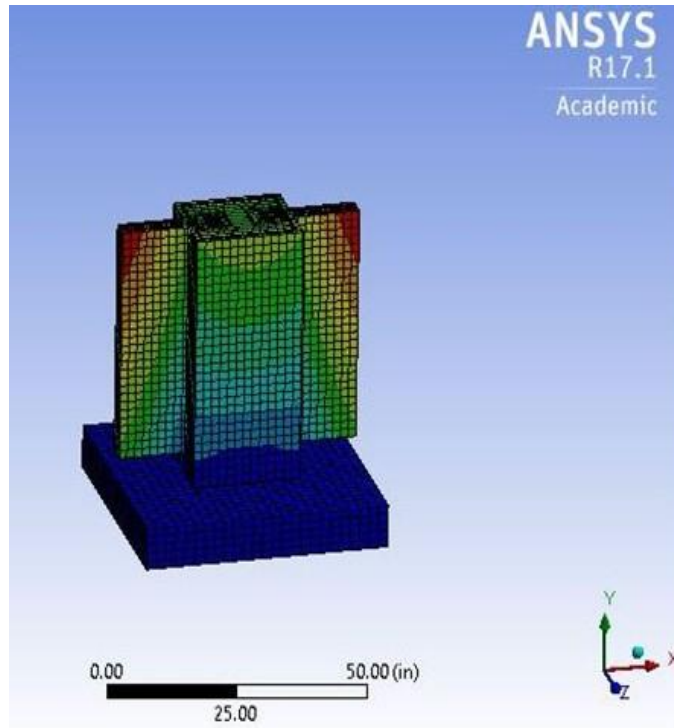
The failure pattern and the crack propagation in the numerical was obtained and was compared with the experimental observations and were found similar. The load carrying capacity of the specimens in the numerical, theoretical and experimental evaluation was comparable and were satisfactory.



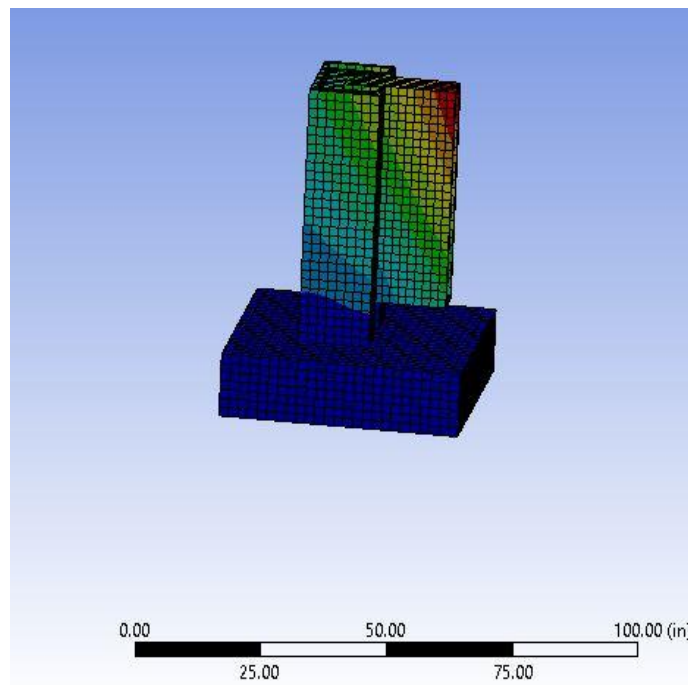
**Figure (4.5):** Von-Misses Stress Distribution in I-section Column



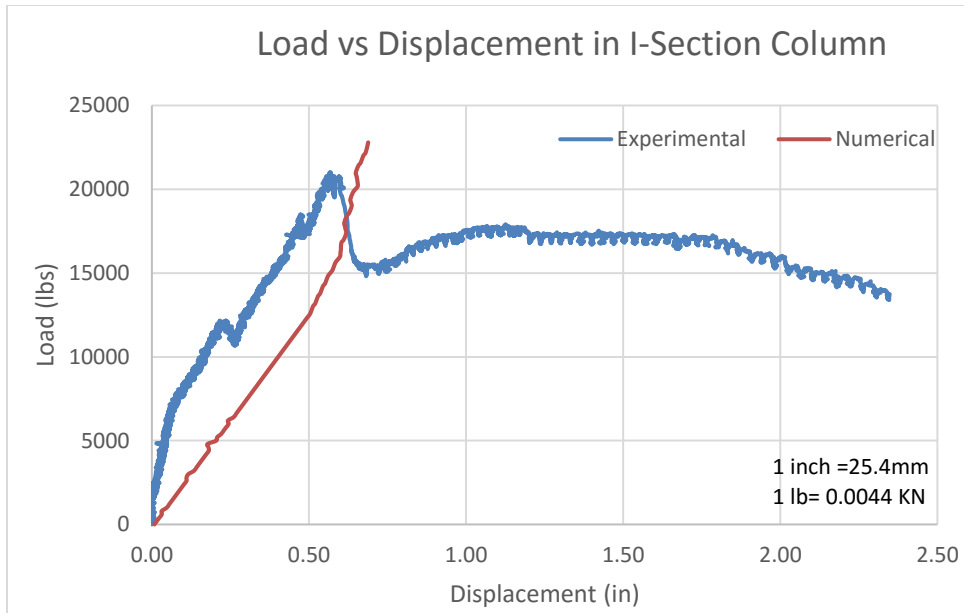
**Figure (4.6):** Von-Misses Stress Distribution in C-section Column



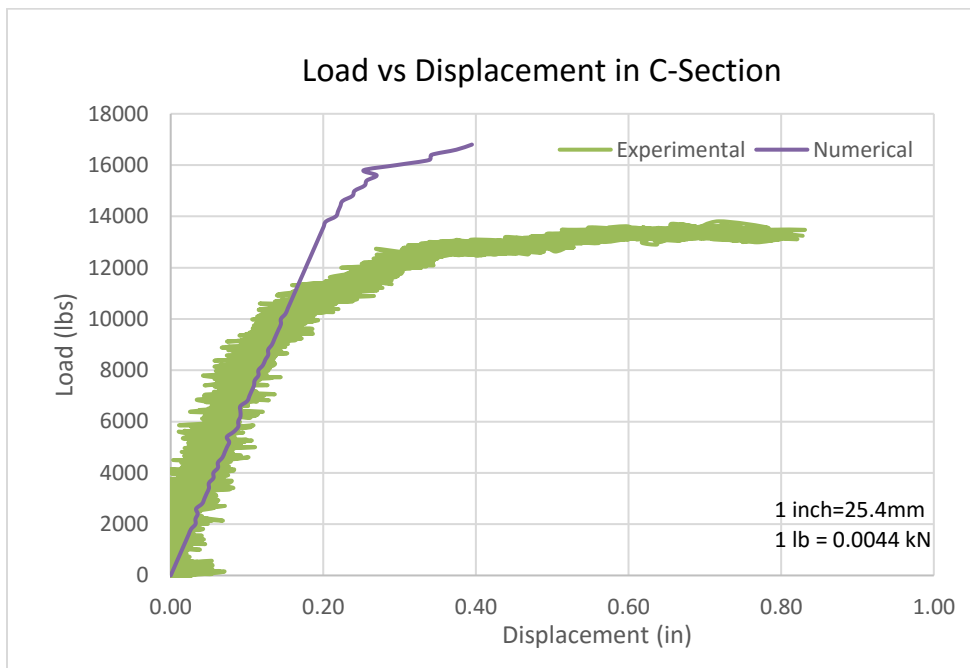
**Figure (4.7):** Deformation Pattern in I-section Column



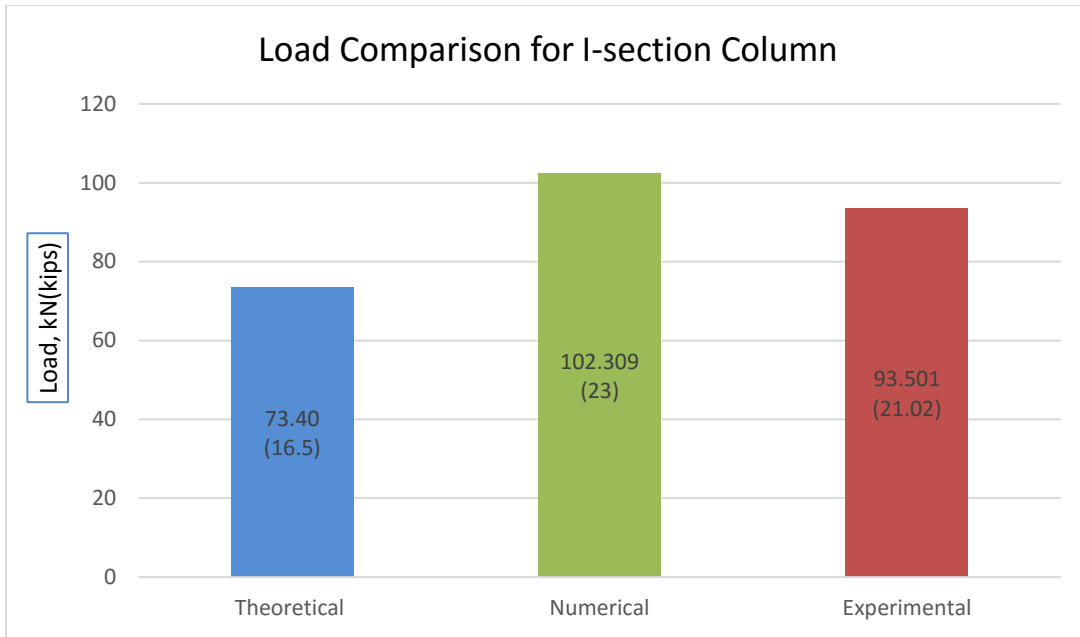
**Figure (4.8):** Deformation Pattern in C-section Column



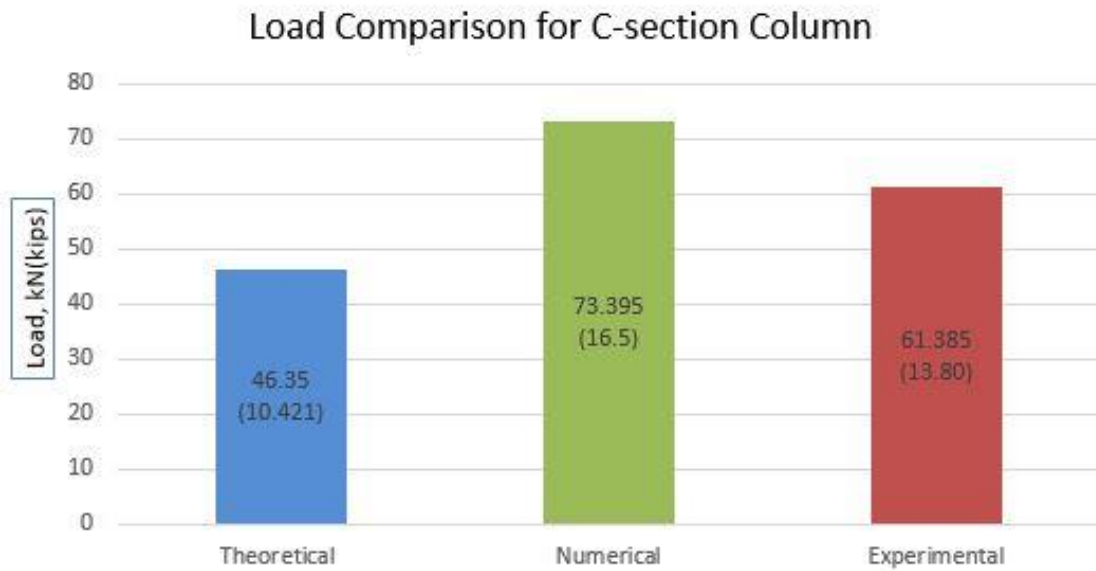
**Figure (4.9):** Load vs Deformation in I-Section Column



**Figure (4.10):** Load vs Deformation in C-Section Column



**Figure (4.11):** Load Comparison for I- section Column



**Figure (4.12):** Load Comparison for C- section Column

## CHAPTER 5

### CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

#### 5.1. Summary

This study aimed at assessing the structural behavior of built-up composite columns subjected to lateral loading. This system is very useful in constructing different types of wall structures including fences, sound and sea walls, shear and bearing walls, as well as fire walls. Both strength and stiffness characteristics of the built-up columns that were fabricated from EVG™ Tridipanel™ sandwich materials with cementitious skins were assessed experimentally and results were validated via ACI design equations as well as finite element numerical simulation. Both symmetric and asymmetric shaped built-up column sections were designed, analyzed and tested and the conclusions are briefed in the section below.

Based on a comprehensive literature review, it was found that this study on sandwich built-up columns is considered to be the first research work in this area. In this chapter, conclusions drawn from experimental, numerical and analytical results are presented. Recommendations for the future research in this area are also discussed.

#### 5.2. Conclusions

Based on the experimental, numerical and theoretical results obtained from this study, the following conclusions are drawn:

1. The symmetrical I-shaped composite built-up column achieved a lateral load carrying capacity of 93.501kN (21 kips) while the unsymmetrical C-shaped

composite built-up column has a lower lateral load carrying capacity of 61.385 kN (13.8 kips) as expected.

2. Numerical results indicated that the experimental results are relatively lower than those obtained from the FE models. Theoretical values that were based on ACI code equations for the peak load were more conservative and were lower than both the experimental and numerical results.
3. The experimental results and the failure pattern indicated that the connection between the column and the foundation is considered to be a controlling factor for achieving the maximum capacity of these built-up columns, hence, attention should be paid when designing proper steel anchoring reinforcement. In addition, the thickness of the mortar at these locations should be thicker than the rest of the columns in order to provide adequate confinement to the steel rebars and steel wire mesh at these regions.
4. These kinds of composite columns with the wall panels could be easily constructed as fences or compound walls and can be modified and made into any required shape or size. The main advantage of using these light-weight structural components for construction is that these could be quickly fabricated and erected in a job site with very less amount of concrete as compared to the conventional concrete columns and walls.
5. Results of this study provides practicing engineers with important preliminary information for designing different types of structures with different geometry including shear walls, bearing walls, fences, sound and sea walls, etc. for utilizing this composite column system for practical field



applications.

### **5.3 Recommendations for Future Research**

Based on the results and observations gained from the study, several research areas have been identified for future work:

1. Due to both time and budget constraints, limited numbers of specimens have been experimentally evaluated. In order to increase the reliability of the conclusions, it is recommended to conduct more comprehensive tests with a large number of specimens. For sandwich structures that are potential will be subjected to wind, and seismic loads, cyclic dynamic experimental and analytical investigations are required.
2. In some cases, openings may be introduced to these types of structures for passing conduits, pipes, etc. In these cases, a more detailed tests program on similar prototypes with different types and locations of openings are recommended.
3. In all the tests performed in this study, gravity loading was not included. However, in cases where such walls and columns are subjected to both gravity and lateral loadings, additional tests with both gravity and lateral loads are needed.
4. Characterization of the joint details between the wall panels and the built-up sandwich columns needs further investigation since it potentially can affect both the deformability, ductility and strength of such structures

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## **APPENDIX (A)**

### **DETAILS OF TEST EQUIPMENT**

#### **HYDRAULIC CYLINDER**

- ENERPAC RCH 603 – Hollow Plunger Cylinder
- Effective Cylinder Area = 12.73 in<sup>2</sup>
- Total Area = 13.49 in<sup>2</sup>
- Cylinder Capacity = 60 tons = 120 kips (Max. = 63.6 tons)
- Stroke = 3 inches
- Oil Capacity = 38.20 in<sup>3</sup>
- Outside Diameter = 6.25 in; Center Hole Diameter = 2.12 in
- Collapsed Height = 9.75 in
- Weight = 62 lbs

#### **PRESSURE TRANSMITTER (LOAD CELL)**

- WIKA Model A-10
- Measuring ranges 0-10,000 psi (0-1000 bar)
- Output Signal: Current (2-wire) – 4-20 mA

Voltage (3-wire) – DC 0-10 V

Power Supply – DC 14-30 V

## STRING POTENTIOMETERS

- Linear Motion Transducers – AC (or) DC

Manufacturer	Model	SL. No	Quantity (No.)	Sensitivity (mV/V/in)	Electrical	
					Excitation	Impedance
1. Unimeasure Inc.	PA-40-HG	SP 3	1	24.20	AC / DC 30 Volts Max.	I: 1000 $\Omega$ O: 0-1000 $\Omega$
2. AMETEK Rayelco	P20- A	SP 1 (NI 6.7254) SP 2 (NI 6.6966)	2	49.77 49.20	AC / DC 25 Volts Max.	I: 500 $\Omega$ O: 0-500 $\Omega$

## STRAIN GAGES

Manufacturer	Model	Resistance	Quantity (No.)		Gage Factor Value
			I- Section Sample	C-Section Sample	
Omega	SGD- 1.5/120- LY11	120 $\Omega$	5	4	2.13



## APPENDIX (B)

### RISK ASSESSMENT

Sl No.	Risk	Impact Scale	Precaution
<b>Risk in Data Acquisition System</b>			
1	Noise or disturbance in recording data from strain gauges and string pots	Moderate	Calibration should be done precisely and properly select the precision for system.
2	Tangling of wires used for connection	Low	<ol style="list-style-type: none"><li>1. Wire should be properly labelled and properly connected with recording device.</li><li>2. Cables should also be fixed with ground with the help of sticking tapes.</li><li>3. Multiple stress release loops should be provided.</li></ol>
<b>Risk in Hydraulic Jack</b>			
1	Leakage	Low	<ol style="list-style-type: none"><li>1. Make sure all the connections are properly fastened.</li><li>2. Strokes should be applied at constant rate.</li><li>3. Number of stokes should be noted.</li></ol>

### Risk in Built-Up Section

1	Yield line recording	High	<ol style="list-style-type: none"><li>1. Crack pattern and timing of occurrence should be recorded precisely.</li><li>2. Position of observer should be safe and visibility should be clear.</li><li>3. Notation for shear and flexure cracks should be properly recorded.</li></ol>
2	Failure point	High	<ol style="list-style-type: none"><li>1. Record type of failure and time of failure.</li><li>2. Failure shape should be precisely recorded.</li><li>3. Observer should record accurately and safely.</li></ol>