USE OF STRUCTURAL CONCRETE INSULATED PANELS (SCIP) SYSTEM IN PAKISTAN AND STUDY OF ITS DIFFERENT ASPECTS

by

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This is to certify that the

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DEDICATED

TO

MY PARENTS, FAMILY AND FRIENDS

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ABSTRACT

After the earthquake of October 2005, almost 70% RCC buildings were either destroyed or declared unfit for living. The government of Pakistan (GoP) as well as some local and international donor agencies including United Nations (UN) and Asian Development Bank (ADB) undertook the task of reconstruction of infrastructure in the region. Some new technologies were adopted for the reconstruction of residential, office, school and hospital buildings including Light Gauge Steel (LGS), Structural Concrete Insulated Panels (SCIP) and Straw Bale Construction. In this research work, SCIP technology is studied comprehensively to evaluate its advantages and disadvantages over conventional buildings in all respects.

Structural Concrete Insulated Panel is a special type of Sandwich Panel or Structural Insulated Panel (SIP) which is composed of thick polystyrene or polyurethane layer and welded GI Wire mesh crossed between the foam sandwiched between two layers of shotcreting (1.5 inch thick on each side). Wall panels and Roof Panels are manufactured in 4 ft. in width and 8 ft. height, available in thicknesses range of 2” to 6".

The conventional building system in Pakistan take much longer time for construction and also it is very poor in terms of energy conservation. According to a conservative estimate, buildings in Pakistan consume more than 40% of the total electricity produced. The demand of this sector is growing at the rate of almost 14% per annum, the highest among all other sectors. To cope up with the challenges of the construction industry, energy efficient fast construction buildings solution is indispensable.

In this research work, a typical school building was modelled and analysed using SAP2000 software and its results were evaluated thoroughly. A comparison between SCIP walls of different thickness is also carried out. Financial comparison is done to evaluate the cost efficiency of the SCIP building in Pakistan.

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Chapter 01

INTRODUCTION

1.1 BACKGROUND

Pathetic energy preservation and extended construction period of conventional building system in Pakistan is responsible for the conflicts between architects and engineers. Approximately 40% of the total power supply produced is consumed by the building sector and is increasing by 14% per annum according to ministry of water and power. Energy efficient and quick construction of building is the only way to meet the challenges of construction industry.

The earthquake of October 2005 in Pakistan had caused huge loss of life and property in AJ&K and KPK. Thousands of Schools, Hospitals, Roads and Bridges were completely destroyed besides human life loss. After that destruction, rebuilding phase started and different types of construction techniques were adopted. Most significant techniques used for the construction works are as under:

a) Structural Concrete Insulated Panels (SCIP)
b) Light Gauge Cold Formed Steel Structure, and
c) Straw Bale Construction etc.

It became evident at the start of the reconstruction process that conventional means of construction cannot fetch the desired level of quality. The problems associated with the construction of conventional concrete structures were accessibility to the remote locations, severe weather conditions, unavailability of water and logistic arrangements at project sites. Keeping in view the light weight nature of the SCIP, excellent structural compatibility to seismic prone areas, efficient thermal and sound insulation and much lesser time of construction compared to conventional RCC construction, SCIP System is a much better option for reconstruction in earthquake affected areas of AJK and KPK.

A brief introduction of the above mentioned technologies used after earthquake for construction is as under:

1.2 STRUCTURAL CONCRETE INSULATED PANELS (SCIP) SYSTEM

SCIP wall system is becoming more and more popular among the users as an alternative construction material for almost every type of buildings worldwide. Since there are many types of sandwich panel building systems are available in the market, but SCIP is the panels made from a thick layer of foam (polystyrene or polyurethane) and welded Galvanized Iron (GI) Wire mesh crossed between the foam sandwiched between two layers of shotcreting (1.5 inch thick on each side). If SCIP panels of particular sizes are transported to the jobsite as per structural drawings, the panels can be easily and rapidly erected by the labor within one month without extensive training. Structural Concrete Insulated Panels are generally manufactured in standard size of 4ft. by 8ft, with thickness varies from 2 to 6 inches.

1.2.1 Attributes

- **Strength:** If we investigate the load carrying capacity of SCI wall panels, their strength varies from 40,000 lbs to 120,000 lbs of load per linear foot of wall and these panels also have the capacity to withstand 200 mph hurricane winds and 7.5 magnitude earthquakes.
- **Energy Efficient:** Polystyrene foam cores have their R values as high as R4.2 per inch of foam as compared to the concrete whose R value is as low as R0.08 and that of brick is R0.2.
- **Speed of Construction:** A normal SCIP building can be erected in less time than wood framing and brick structures and much less time than concrete block.
- **Sound Insulation:** As sound insulation is very much important in urban areas and in multi user applications. Therefore, super insulated PS walls are superior to almost every other wall system available in case of sound insulation.

1.3 LIGHT GAUGE COLD FORMED STEEL STRUCTURE

For the manufacturing of cold formed steel structure sections, flat sheets of steel are bent at ambient temperature into shapes which would support more load than the flat sheets themselves. These steel sheets are being produced for more than a century. Now in recent years, cold formed steel structural members are manufactured in such a way that they possess a much higher strength and a wider range of sections as compared to the traditional heavier hot rolled steel structural members [1].

1.3.1 Materials

Materials used for the construction of Light Gauge Cold Formed Steel Structures are as under:

- Foundation: Reinforced Cement Concrete dry foundation.
- Structural Frame: Cold formed Galvanized Steel channels of different shapes.
- Insulation: Glass wool/Mineral wool.
- Wall Cladding: Fiber Cement Board/Glass Reinforced Cement Board.
- Roofing: Corrugated GI sheet.
- False Ceiling: MDF Board/Fiber Cement Board

1.3.2 Advantages

Light Gauge Cold Formed Steel Structure Buildings have the following advantages:

- Fast Track Construction.
- Thermally efficient.
- Sound Proof.
- Ideal for seismic zones.
- High Strength to weight ratio.
- Reduced Labour Cost.
- Can be easily shifted from one place to another.
- Ideal for mountainous and difficult approaches.

1.4 STRAW BALE CONSTRUCTION SYSTEM

Just like all other natural fibers materials e.g. wood, paper, cotton fabric, etc., straw bale also degrade slowly in typical conditions. But by maintaining certain conditions for the preservation of straw bale, it can last many thousands of years. The storage conditions of straw bale highly affect its degradation process. Most effective factors which affect the degradation process of stored straw bale are moisture content and temperature. If we pay thorough attention the moisture control, a straw bale structure should be able to last as long as any conventional wood framed home. We can say that the useful life of a straw bale structure mostly depend upon its environmental conditions [8].

1.4.1 Advantages

In straw bale building, walls can be raised easily which suit both the stakeholders i.e owner and the builder, and labour cost can be cut short to a large extent. Also the material cost is much less than the other common wall systems adopted. Apart from these advantages of straw bale building, it has been observed that such buildings are fire and earthquake resistant, have extremely high heat and sound insulation values, almost ten times as much as wood and bricks, energy efficient, and require minimum maintenance. With such attributes, it is clearly evident that a straw bale building possesses significant cost advantage as compared to other types of conventional building systems [8].

1.5 NEED OF ALTERNATE CONSTRUCTION TECHNOLOGIES

If we take a look at the seismic zoning map of Pakistan (Appendix C), it can be seen that considerable part of the country is lying under severe seismic zones. Under such circumstances where the probability of catastrophic disaster is on higher side, the necessity of alternate construction techniques rises considerably. SCIP is one of those technologies which can be constructed in a speedy manner and at the same time it can resist the lateral forces effectively. Due to the light weight nature of SCIP technology, the SCI wall panels can be transported to the remote areas quite easily and comfortably. A SCIP building can be constructed in much lesser time which saves the money to a large extent. Due to the energy efficient nature of SCI panels, these type of materials can considerably reduce the consumption of energy which ultimately help the energy sector to deal with the energy crisis tenure. These types of construction technologies can be used for every kind of building such as recreational, residential, official, hospitals etc. As these construction techniques are speedy, so it results in a lot of time and money saving.

1.6 AIMS AND OBJECTIVES OF RESEARCH

The main aims and objectives of this research are:

- To evaluate the **flexural capacity** of walls constructed with SCIP technology in accordance with ACI 318-02.
- To evaluate the **shear capacity** of walls constructed with SCIP technology in accordance with ACI 318-02.

- To evaluate the **axial load capacity** of walls constructed with SCIP technology in accordance with ACI 318-02.
- To make a **comparison of SCIP wall panels of different thicknesses** ranging from 4 to 7 inches and to evaluate their flexural capacity, shear capacity and axial load capacity. and
- To make a **financial comparison** between a SCIP structure and a conventional frame structure building.

Chapter 02

LITERATURE REVIEW

2.1 STRUCTURAL CONCRETE INSULATED PANEL OR SANDWICH PANEL

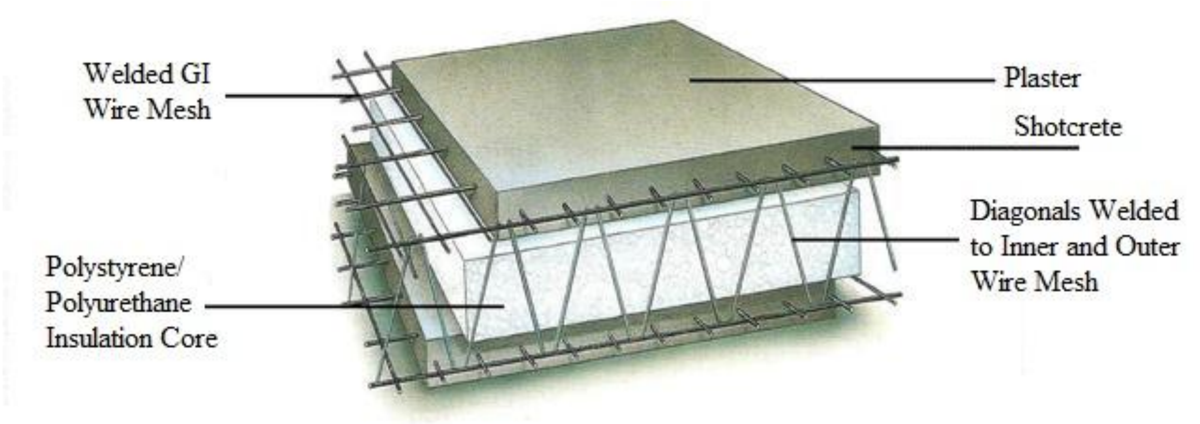


Figure 2.1: Standard Wall Section of SCIP Panel

Structural insulated panels (SIPs) are a type of composite building material. A SIP panel consists of two structural board layer of any material and space between these two layers is filled with a layer of foam which acts as an insulation. Due to their structural assembly, they are call as sandwich panels. Normally plywood or oriented strand boards are used in making SIP panels. Expanded polystyrene foam (EPS), extruded polystyrene foam (XPS) or polyurethane foam is used as an insulation layer between the structural board layers. Both skins of structural insulated panel are structure-less initially, but once the pressure laminates them together under controlled conditions, a structural material that has structural properties is created. These panels are used in roofs, exterior and interior walls and floor system of residential and light commercial building. They serve as a replacement to various conventional components of building. Once the SCIP panels of appropriate sizes as per structural drawings arrive to the jobsite, the panels can be rapidly assembled by workers without extensive training. Overall, the SIP construction method allows for rapid erection of an exterior building envelope that is strong, airtight, and energy efficient [4].

Another variation of the SIP is a structural concrete insulated panel (SCIP), which can be used for wall, floor, and roof applications. This SCIP system is a steel trussed sandwich concrete panel made of expanded polystyrene foam, flanked by galvanized wire mesh on both sides, and connected with galvanized vertical steel wire trusses spaced at 6 in. (150 mm). The assembly is then coated on site with 1 in. (25 mm) of Portland cement plaster by low-velocity shotcreting on both sides to form a composite panel. The advantages of the SCIP system over other types of sandwich panels that are currently in the market include the ease of manufacturing; the reduction of construction time and cost; the production of structural elements with greater structural integrity; and lower risk in fire and termite attack since the skins are not a wood product. A unique advantage of the SCIP system is its lower cost because it does not require any complicated machinery for construction [4].

2.2 BACKGROUND AND INTRODUCTION

It became evident at the start of the reconstruction process in earthquake affected areas of AJK and KPK that conventional means of construction cannot fetch the desired level of quality. The problems associated with the construction of conventional concrete structures were accessibility to the remote locations, severe weather conditions, unavailability of water and logistic arrangements at project sites. Keeping in view the light weight nature of the SCIPs, excellent structural compatibility to seismic prone areas, efficient thermal and sound insulation and much lesser time of construction compared to conventional RCC construction, it was decided to deploy the use of SCIPs in earthquake affected areas.

2.3 DESIGN

Once the shotcrete layers make bond with insulation layer, it creates a web-and-flange structural strength (along the same principal as an I-beam) across the length and breadth of the panel. Having capacity to resist all the demand forces like axial loading, flexural loading, cracking and shear loading, properly designed and erected SCIP buildings can easily and effectively replace the conventional framing. At the same time such type of building can withstand high wind and seismic forces. For low rise construction, SCIP panels are designed using standard charts for wind, snow and seismic load resistance capacity which have been developed by different manufacturers. SCIP panels are modeled as mesh elements in finite element based structural design softwares (SAP2000, ETABS, STAAD, STAAD Pro etc) and each element is checked for the permissible value of stress against the thermal, gravity, seismic and wind loads applied in accordance with the ACI, UBC and other relevant codes [5].

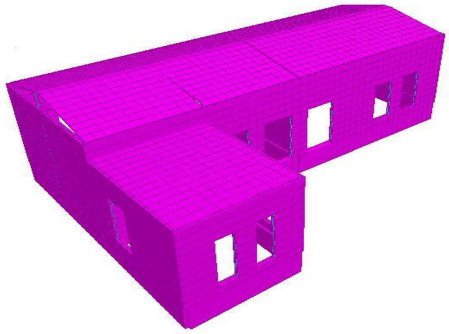


Figure 2.2: 3-D Model Rendered View

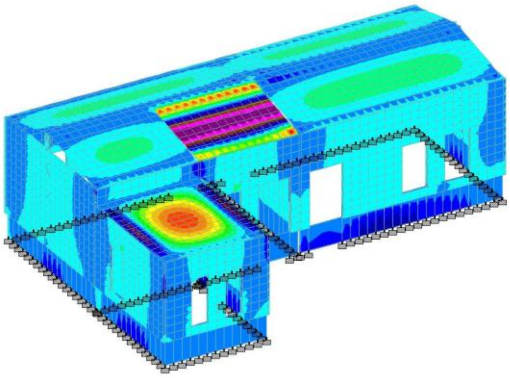


Figure 2.3: Stress Analysis on 3-D Model

2.4 SPECIFICATIONS AND CODE REQUIREMENTS

2.4.1 Wire Mesh: (ASTM A82 and ASTM A185)

- ***Diameters:***

  Longitudinal wires: 2.6 mm diameter

  Transverse wires: 2.6 mm diameter

  Joint wires: 2.6 mm diameter

- ***Strength:***

  Yield Strength: > 6,500 psi

  Ultimate Strength: > 7,500 psi

2.4.2 Reinforcing Steel: (ASTM A615)

- ***Diameters:***

  It depends on design, may vary from #3 bars to #8 bars.

- ***Strength:***

  Yield Strength: = 40,000 psi

  Ultimate Strength= 58,000 psi

2.4.3 Plain and Reinforced Concrete: (ACI-318-05)

- ***Compressive Strength:***

  Reinforced Concrete = 3,000 psi

  Blinding Concrete = 1,000 psi

Shotcrete = 3,000 psi (Polypropylene PP fibers must be added)

2.4.4 Polystyrene: (ASTM C578)

- ***Specifications:***

Minimum Density = 15 Kg/cu.m

Thickness = 2 inch (Internal Walls) and 4 inch (External Walls)

Thermal Conductivity < 0.040 W/m. K (Watts per meter Kelvin)

2.5 AVAILABILITY OF MATERIAL

Structural Concrete Insulated Panels are manufactured from a starting width of 4' x 8' length. The panels can be assembled up to 40' in length in 8" increments. Truss wire gauges available are 11, 12.5 and 14.

2.6 LOAD CARRYING CAPACITY

Fouad et al. (2004) calculated the maximum service uniform distributed live load for the tested panels from the ultimate failure load, and by using load factors of 1.6 and 1.2 for live and dead loads respectively according to the ACI 318-05. A flooring load of about 15 psf (0.72 kPa) was also assumed. The service uniform live loads for the panels for residential buildings vary from 10 to 40 psf (0.50 to 2.00 kPa) except stairs and balconies. The panels may have service live load capacity in excess of 40 psf (2.00 kPa) if span-to-depth ratio is more than 32 for typical reinforced concrete structures but the lower service live load capacity should be expected because originally the span-to-depth ratio doesn't go much higher. Therefore, it is recommended to use a panel span-to-depth ratio of about 20 or less to ensure a reasonable amount of service live load capacity.

2.7 ADVANTAGES OF SCIP OVER SIP SYSTEM

The SCIP system offers numerous advantages over other types of SIPs. The advantages include the ease of manufacturing; the reduction of construction time and cost; and the production of structural elements with greater structural integrity. A unique advantage of the SCIP system is its lower cost because it does not require any complicated machinery for construction. The SCIP system exhibits a semi-composite behavior under ultimate loads. A strength calibration factor (SCF) was derived by comparing the test data to theoretical calculations based on a fully composite behavior according to ACI 318 Code. This factor, which accounts for the partial composite behavior, may be used to estimate the ultimate flexural strength of the SCIP system [4].

2.8 DESIGN LOADS

Structural Concrete Insulated Panels used for the construction in earthquake affected areas are designed for all anticipated loads during the life of the structure and should at least cater for the following:

- ***Dead Loads***

  It comprises the self-weight of structure, false ceiling, insulation and other supporting members.

- ***Live Loads***

  The live load on the roof should not be less than 0.72 kN/m^2 (15 psf)

- ***Wind Loads***

  Wind load calculated according to UBC-97 with following parameters:

Basic Wind Speed (V)	= 70 Miles/hr
Exposure Category	= C
Importance Factor (I)	= 1.0

- ***Earthquake Loads***

  Earthquake Loads calculated according to the UBC-97 Provisions with following parameters

Seismic Zone Factor (Z)	= 0.4
Soil Profile Type	= SD
Importance Factor (I)	= 1.0

  Parameters R & T shall be set according to the basic structural system adopted.

- ***Temperature Loads***

  Yearly maximum and minimum temperature variations are as follows.

Minimum Ambient Temperature	=	-5^{o}C
Maximum Ambient Temperature	=	+45^{o}C

- ***Load Combinations***

  All load combinations conforming to ACI 318-05.

2.9 CODES AND STANDARDS

The building along with foundations and compound wall etc. are designed in accordance with the following codes and standards:

a)	ASTM A82	Standard specifications for steel wire, plain, for concrete reinforcement
b)	ASTM A185	Standard specifications for steel welded wire fabric, plain, for concrete reinforcement.
c)	ACI 506R-90	Guide to shotcrete
d)	ACI 506R.4R-94	Guide for evaluation of shotcrete
e)	ACI 506.2-95	Specifications for shotcrete
f)	ACI 318	Building Code Requirements for Structural Concrete.
g)	ASTM C578	Standard specifications for rigid cellular polystyrene thermal insulation.

2.10 SERVICEABILITY LIMITS

Allowable deflections in the buildings constructed with SCIP technology in earthquake affected areas are restricted to the following limits:

- Vertical deflections due to Live Loads should not exceed L/360, where L is the effective span of the member under consideration. While members are loaded with live load as well as dead load, the deflection should not exceed the maximum allowable deflection of L/240.
- Horizontal deflections/sway should not exceed H/240 where H is the height of the building.

2.11 FLEXURAL BEHAVIOUR OF SCI PANEL

Fouad et al. (2004) conducted an experimental program to study the flexural behaviour of SCIP system. The results of the test program were used to refine the analytical procedures that were developed to predict the panel strength.

For the structural concrete insulated panel (SCIP) to be widely used in the construction industry and accepted by the design engineer, a reliable design procedure for predicting the panel strength must be developed. A test program was planned and conducted to provide a basic understanding of the flexural behavior of the panel and to refine the analytical procedures that were developed to predict the panel's flexural strength.

2.11.1 Experiment Conducted to Check Flexural Strength

Ten full-scale specimens were tested to study the flexural behavior of SCIP system. The ten specimens were divided into four groups; each group had a different span-to-depth ratio. All the specimens tested were 24 in. (600 mm) wide. The first two groups consisted of three specimens each with a span-to-depth ratio of 8.73 and 16, respectively. The other two groups consisted of two specimens each with a span-to-depth ratio of 20 and 32, respectively. The test specimen dimensions are given in Table 2.1 [4].

The shotcrete on each side of the panel was 1 in. (25 mm) thick and was reinforced with 14 gauge wire mesh of 1 x 1 in. (25 x 25 mm), centred in the cementitous wall, with ½ in. (13 mm) concrete cover. The steel trusses were made of 3/16 in. (5 mm) diameter wire spaced at 6 in. (150 mm) centre to centre longitudinally. The top and bottom chords of the steel trusses were embedded for about ½ in. (13 mm) into the shotcrete thickness. Steel truss wire used in the panels was tested in tension. Table 2.2 gives the results of the steel wire tensile tests. The shotcrete mixture proportions used are given in Table 2.3 [4].

Table 2.1: Test specimen dimensions

Group No.	Span-to-depth ratio	Specimen ID	Width (ft.) [mm]	Span (ft.) [mm]	Thickness (in.) [mm]
Group 1	8.73	3R	2 [600]	8 [2400]	11 [280]
		4R	2 [600]	8 [2400]	11 [280]
		11R	2 [600]	8 [2400]	11 [280]
Group 2	16	1R	2 [600]	8 [2400]	6 [150]
		2R	2 [600]	8 [2400]	6 [150]
		13R	2 [600]	8 [2400]	6 [150]
Group 3	20	5R	2 [600]	10 [3000]	6 [150]
		6R	2 [600]	10 [3000]	6 [150]
Group 4	32	8R	2 [600]	16 [4800]	6 [150]
		9R	2 [600]	16 [4800]	6 [150]

Table 2.2: Steel Wire Tensile Tests

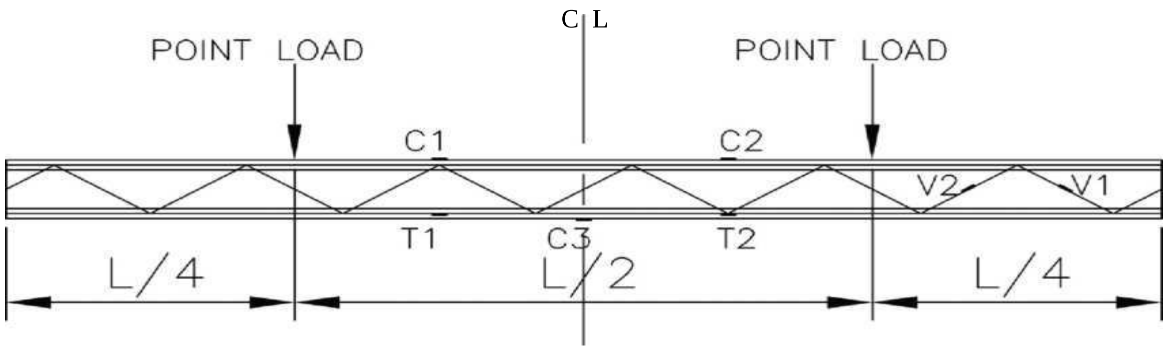
Steel type	Sample Number	Average dia. (in.) [mm]	Tensile strength (psi) [MPa]	Average tensile strength (psi) [MPa]
Steel wire mesh	Sample # 1	0.1000 [2.5]	93,500 [645]	92,333 [635]
	Sample # 2	0.1000 [2.5]	91,000 [625]	
	Sample # 3	0.1000 [2.5]	92,500 [640]	
Steel truss wire	Sample # 1	0.1820 [4.55]	105,937 [730]	112,965 [780]
	Sample # 2	0.1700 [4.25]	122,478 [845]	
	Sample # 3	0.1735 [4.34]	110,480 [760]	

Table 2.3: Wet Shotcrete Mixture Proportions

Water cement ratio w/cm ratio	Portland cement Type I/II (weight %)	Concrete sand (weight %)
0.45	0.23	0.77

The test specimens were tested under 2-point loading according to ASTM E 72, with load points located at quarter points of the panel span. The panels were oriented horizontally and seated on roller supports. The marked supports lines were aligned with the rollers. Bearing plates with rubber pads were used to prevent crushing of the material at the loading points and supports. A spreader beam was used to distribute the load into two point loads on the panel. A 10 kip (44 kN) load cell was used for measuring the applied load and a linear displacement potentiometer (LVDT) was used to measure the deflection at the centre of the panel. The weight of the spreader beam, roller supports and loading hardware acting on the panel was measured and accounted for in the calculations. The self-weight of the panels was also measured and considered in the calculations. The span length of the panels, measured between the roller supports, was 4 in. (100 mm) shorter than the overall panel length. The test setup is shown in Figure 2.4. For some selected specimens, the concrete and the steel wire were strain gauged to obtain data that would further verify the load transfer mechanism. Figure 2.5 shows a schematic diagram of the strain gauge locations [4].

Figure 2.4: Flexural test setup [Fouad et al. (2004)]



LEGEND:
C1&C2: STRAIN GAUGE OF CONCRETE IN COMPRESSION
C3: STRAIN GAUGE OF CONCRETE IN TENSION
T1&T2: STRAIN GAUGE OF WIRE MESH IN TENSION
V1: STRAIN GAUGE OF TRUSS WIRE IN COMPRESSION
V2: STRAIN GAUGE OF TRSUSS WIRE IN TENSION

Figure 2.5: Schematic diagram of the strain gauge locations [Fouad et al. (2004)]

2.11.2 Findings of the experiment

The load was applied to the panels in increments of about 1000 lbs (4.5 kN). There was a pause after each load increment application to allow time to check for the development of any cracks in concrete, measure crack widths, and inspect for any structural distress that might have occurred. The load deflection curve of panel 3R with a span-to-depth ratio of 8.73 is shown in Figure 2.6. Figure 2.7 shows the load deflection curve of panel 1R with a span-to-depth ratio of 16. It could be seen from the load deflection curves that the panels are ductile with a fairly large range of non-linear behavior. It could also be seen that the deflection is significantly affected by the span-to-depth ratio of the panels. The deflection at failure for panel 3R with span-to-depth ratio of 8.73 was 3.02 in. (77 mm), while for panel 1R with a span-to-depth ratio of 16, the deflection at failure was 1.38 in. (35 mm) [4].

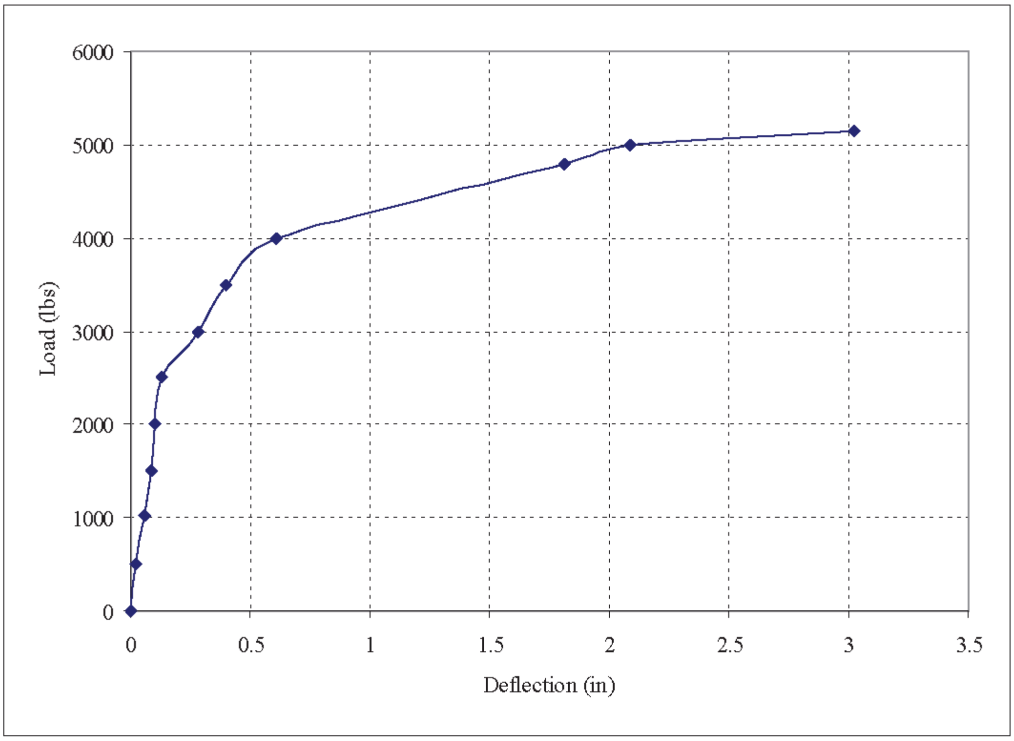


Figure 2.6: Load deflection curve of panel number 3R (1 in = 25.4 mm, 1000 lbs = 4.5 kN) [Fouad et al. (2004)]

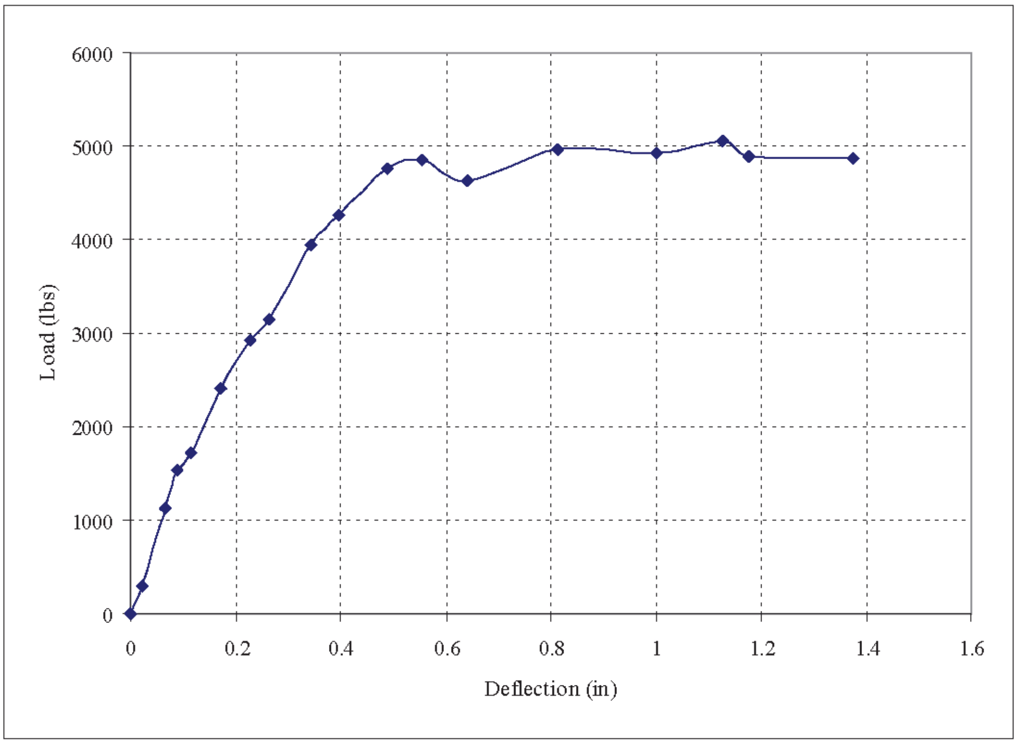


Figure 2.7: Load deflection curve of panel number 1R (1 in = 25.4 mm, 1000 lbs = 4.5 kN) [Fouad et al. (2004)]

2.12 LIGHT GAUGE COLD FORMED STEEL

Light Gauge Cold-formed steel members are being used in construction industry, automobile industry and various other equipments. Such type of cold formed steel sections are made from sheets of steel, strips, plates or flat bars either in roll forming machines or by bending operations or by using press brakes. The thickness of these steel members usually ranges from 0.0147in to about 0.25in. Even steel plates and bars of 1 inch thickness can also be successfully transformed into cold formed structural shapes. Till now, no standard series of cold formed steel sections have been developed. Only a few dimensional requirements are specified in AISI standard for cold formed steel framing. These sections are more expensive than hot rolled sections. Their thickness varies from 0.020in to 0.125in when manufactured from sheet or strip steel [1].

2.12.1 How Cold-Formed Shapes are Made

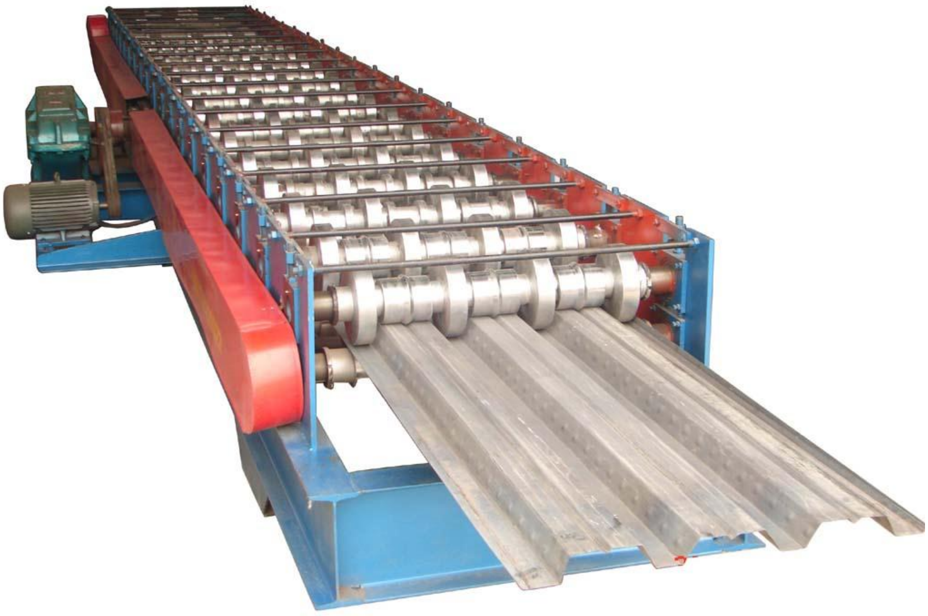


Figure 2.8: Cold Roll Forming Machine

The cold formed shapes are available in two forms i.e. galvanized or uncoated (black). Galvanized col formed shapes are expensive but more preferred at conditions prone to increased corrosion. The ASTM standards for structural quality galvanized sheets come under

ASTM 653. These cold formed sheets are ordered and manufactured in millimetre or decimal thickness [7].

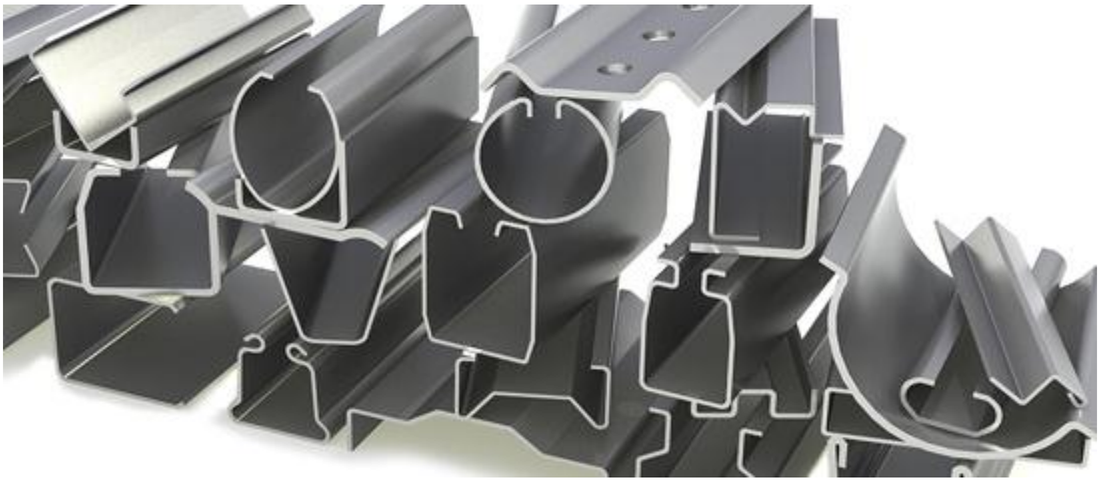


Figure 2.9: Various Shapes of Cold Formed Steel Sections

2.12.2 Properties of Light Gauge Cold Formed Steel Sections

Some of the main properties of cold formed steel are as follows:

- Light in weight
- Larger strength and stiffness
- Prefabrication and mass production can be done easily
- Erection and installation can be done easily and in a speedy manner
- Considerable elimination of delays due to weather
- More accurate detailing
- Non shrinking and non-creeping at ambient temperatures
- No formwork required
- Termite-proof and rot proof
- Uniform quality
- Economy in transportation and handling
- Non combustibility
- Recyclable material

Chapter 03

METHODOLOGY

Figure 3.1: Typical SCIP System

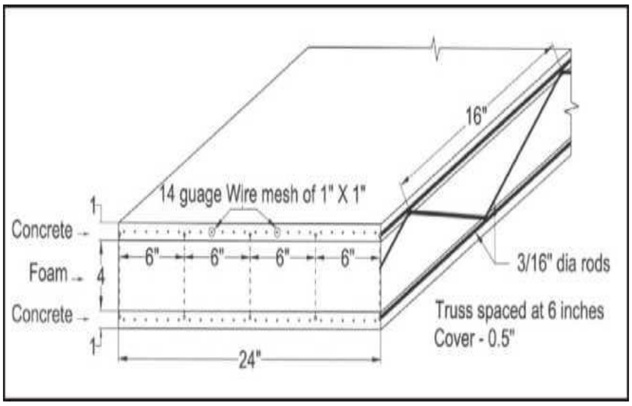


Figure 3.2: Dimensions and Reinforcement Details of Typical SCIP System

3.1 SANDWICH PANEL SYSTEM

Since there had been severe earthquakes in Pakistan, there is a public concern to improve the seismic performance of the buildings. Sandwich panel system is an advanced technique to improve the seismic performance of the building. Sandwich panel system comprise of three panels, two concrete layers with a layer of wire mesh each and one insulation layer between the two concrete layers. This sandwich panel is used for walls while ordinary reinforced concrete structure and cold-formed steel structure are used in some parts of the building as required. All panels are connected by using dowel bars. Concrete or mortar spraying method is used with temporary support to cast this panel system.

3.2 FINITE ELEMENT MODEL AND ANALYSIS

The entire building was modeled as three-dimensional finite element model for analysis. Linear static and response spectrum analysis were performed to investigate its response under applied loading. Finite element model was created based on the drawings generally used in most of school buildings used in Pakistan. The building is 85 ft long and 27 ft wide and there is no floor slab in the building. The walls are provided as load bearing wall system and built of sandwich panel. The steel sheet roof is directly supported by the walls and columns. The spread footing is used for foundation system. Ordinary RC structure is used for the foundation and cold-formed steel is used for roof truss and columns. Details of architectural drawing for this building are shown in the following figures.

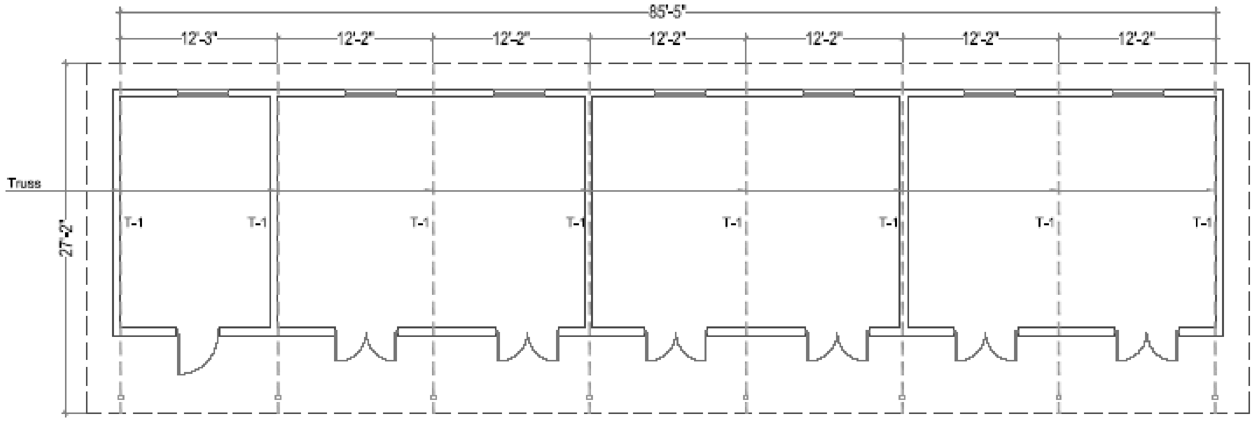


Figure 3.3: Floor Plan

3.3 ANALYSIS METHODOLOGY

A comprehensive modeling and analysis of the structure and its components under study has been carried out with an objective to understand the behavior of the structure, to predict its response to various possible loads including earthquake, and to evaluate the safety and serviceability states. A full three dimensional, finite element model of the building and associated components has been constructed by using appropriate finite element software. Series of linear static and response spectrum analyses were carried out for various objectives.

3.4 ANALYSIS TOOLS

SAP2000 software was selected as the main tools for the modeling, analysis and design checks. SAP2000 is a special program for analysis and design of RC, Steel and Composite buildings. This is a well-known and sophisticated program for the analysis and design of structural systems and components, developed by Computers and Structures Inc. (CSI), Berkeley California, USA. It provides powerful capabilities for modeling a wide range of structures, including bridges, dams, tanks and buildings. The Windows-based graphical interface allows for quick model creation using templates. The creation and modification of models, execution of the analysis, viewing of the results, and design optimization are all performed interactively within the same interface.

3.5 FINITE ELEMENT MODEL

Three-dimensional finite element model of the structural system have been created in SAP2000 based on the architectural drawings. The finite element model is comprised of shell and frame element to represent structural components. Appropriate finite element modeling techniques were used to model the structural components as shown in following table.

Table 3.1: Structural Component and Finite Element Model

Structural component	Finite element model
Wall spread footing	Shell element
Column	Frame element
Truss	Frame element
Sandwich panel wall	Shell element

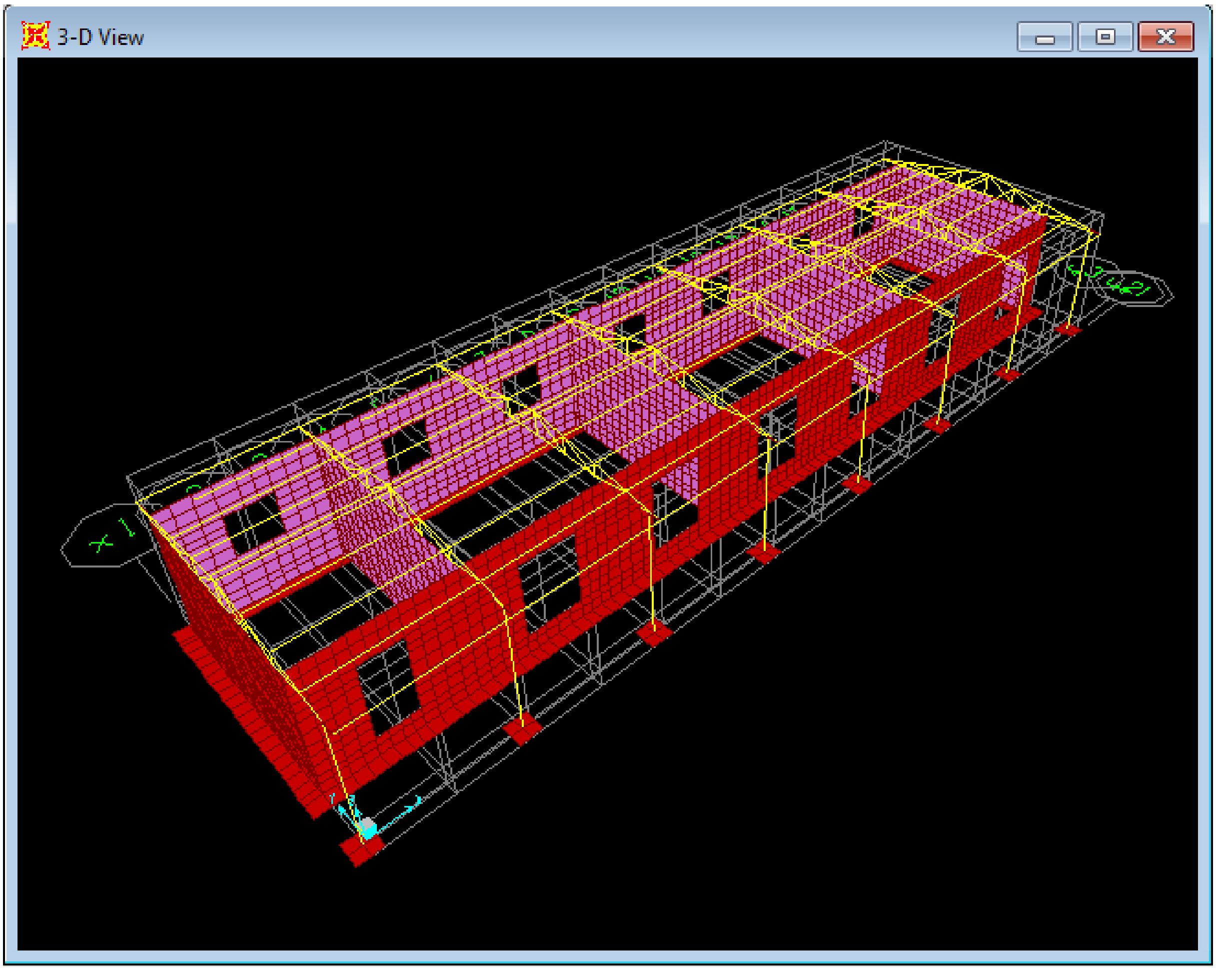


Figure 3.4: Finite Element Model

3.6 MATERIAL PROPERTIES

The same material properties are used in the model for the ordinary concrete for RC component, the sandwich panel for wall. For cold-formed steel structures, ASTM A 572-50 steel is used.

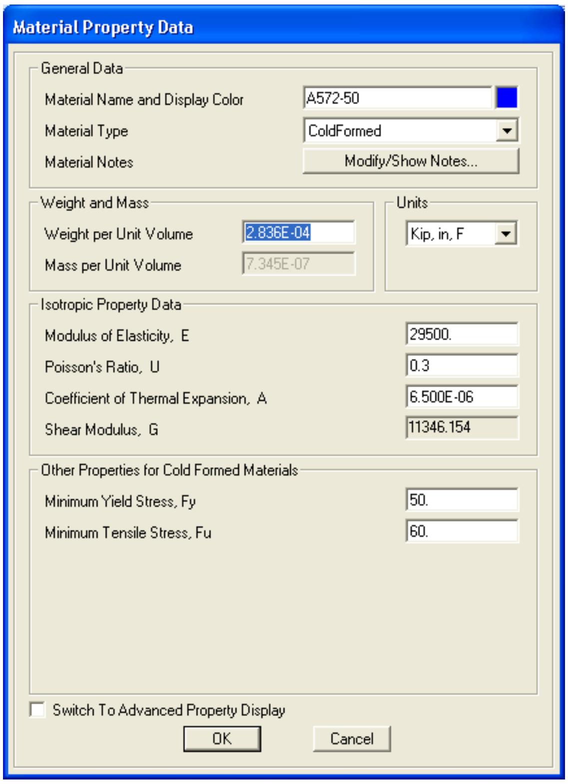


Figure 3.5: Material Property for cold-formed steel

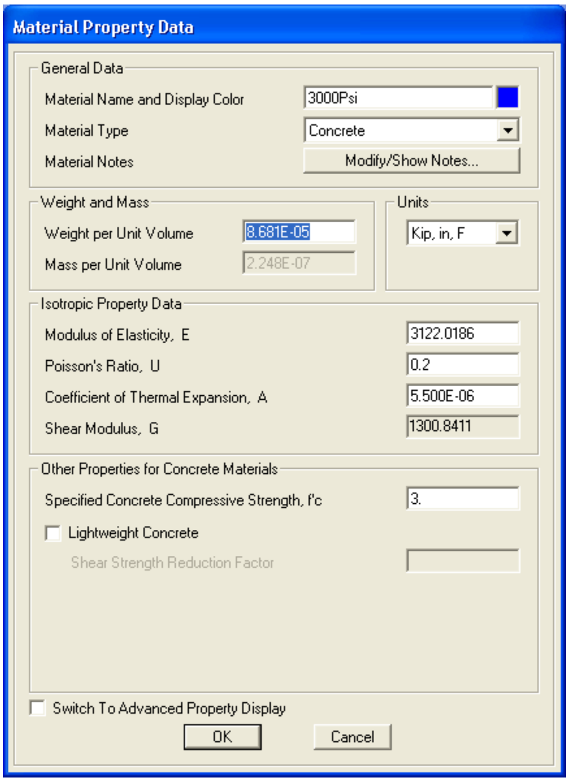


Figure 3.6: Material Property for concrete

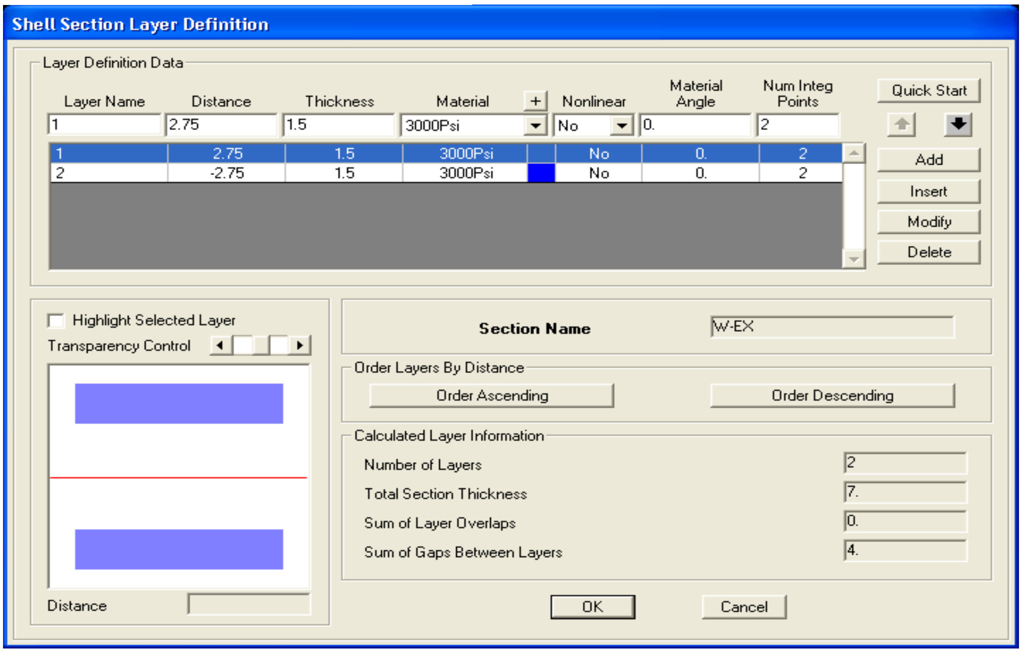


Figure 3.7: Material Property for Shell Section for Wall

3.7 MODELING OF STRUCTURAL COMPONENTS

All columns and trusses have been modeled as frame element. The sandwich panel for walls is modeled as multi-layer shell elements. All the walls are modeled with total thickness of 7 in. The spread footing is modeled as the single layer thin shell elements. To represent the soil interaction between the soil and the spread footing, the horizontal and vertical springs are applied on the footing.

3.8 EARTHQUAKE LOAD

Response spectrum analysis using UBC 97 spectrum is performed to evaluate the response of the building under earthquake load. Seismic zone 4 parameters are used in the seismic analysis.

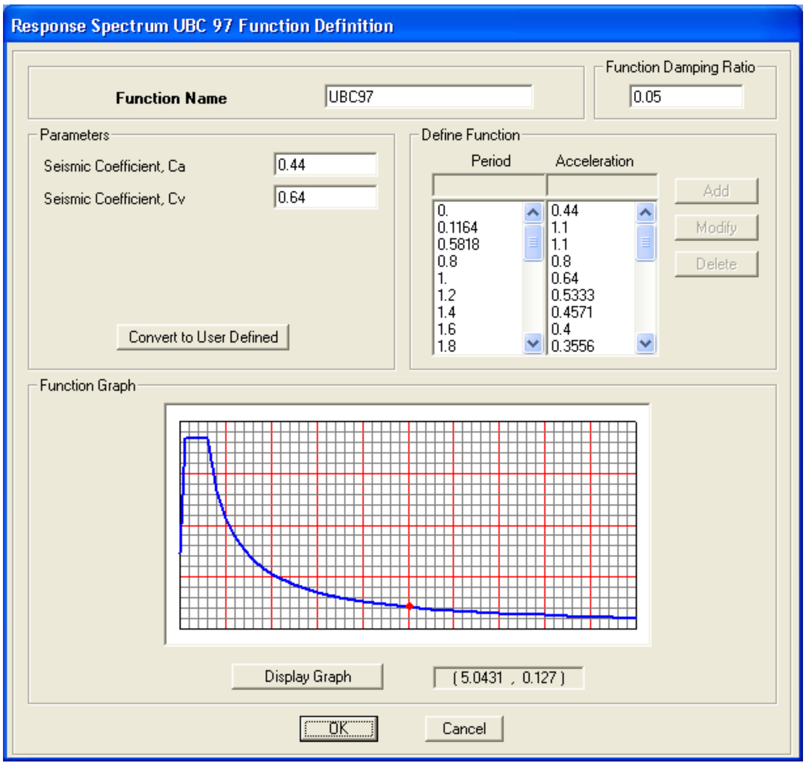


Figure 3.8: UBC 97 Response Spectrum

3.9 WIND LOAD

Wind load is applied in accordance with ASCE 7-05. Wind speed of 70 mph is applied and the exposure type "C" is used.

3.10 LOAD CASES AND COMBINATIONS

Complete linear static and response spectrum analysis have been carried out for different load cases. This section describes the external loads applied to the structure and used in the analysis. All load cases are summarized in following the table.

Table 3.2: Load Case Details

<table>
<tr><th>LOAD CASE NAME</th><th>LOAD CASE TYPE</th><th>DETAILS</th></tr>
<tr><td>DL</td><td>Dead</td><td>Self-weight of structural component
Using self-weight multiplier in SAP2000
10 kg/m^2 of ceiling load
10 kg/m^2 of roof material</td></tr>
<tr><td>LL</td><td>Live</td><td>30 kg/m^2 on the roof</td></tr>
<tr><td>TEMP</td><td>Temp</td><td>45^{o}C temperature changes</td></tr>
<tr><td>WX</td><td>Wind
in x direction</td><td rowspan="2">Static loading in accordance with ASCE 7-05</td></tr>
<tr><td>WY</td><td>Wind
in y direction</td></tr>
<tr><td>ESX</td><td>Earth quake
in x direction</td><td rowspan="2">Equivalent static load</td></tr>
<tr><td>ESY</td><td>Earth quake
in y direction</td></tr>
<tr><td>ERX</td><td>Earth quake
in x direction</td><td rowspan="2">Response spectrum analysis using UBC97 response spectrum</td></tr>
<tr><td>ERY</td><td>Earth quake
in y direction</td></tr>
</table>

Total of 24 design load combinations have been considered for structural design of RC components based on ACI 318-02 and 23 service load combinations have been used for serviceability based on UBC97. 31 design load combinations based on AISC LRFD-93 have been considered for design of cold-formed steel components. All load combination details are shown in the following table.

Table 3.3: Load Combinations

	RC Design Load Combinations	Service Load Combinations	Steel Design Load Combinations
1	1.4 DL	DL	1.4 DL
2	1.2 DL + 1.6 LL	DL + LL	1.2 DL + 1.6 LL
3	1.2 DL + 1.0 LL	DL + WX	0.9 DL + 1.3 WX
4	1.2 DL + 0.8 WX	DL - WX	0.9 DL + 1.3 WY
5	1.2 DL + 0.8 WX	DL + WY	0.9 DL – 1.3 WX
6	1.2 DL + 0.8 WY	DL - WY	0.9 DL – 1.3 WY
7	1.2 DL - 0.8 WY	DL + ERX/1.4	1.2 DL + 1.3 WX + 0.5 LL
8	1.2 DL + 1.6 WX + 1.0 LL	DL– ERX/1.4	1.2 DL + 1.3 WY + 0.5 LL
9	1.2 DL - 1.6 WX + 1.0 LL	DL ERY/1.4	1.2 DL - 1.3 WX + 0.5 LL
10	1.2 DL + 1.6 WY + 1.0 LL	DL– ERY/1.4	1.2 DL - 1.3 WY + 0.5 LL
11	1.2 DL - 1.6 WY + 1.0 LL	0.9 DL + ERX/1.4	0.9 DL + 1.0 ERX

	RC Design Load Combinations	Service Load Combinations	Steel Design Load Combinations
12	1.2 DL + 1.0 ERX + 1.0 LL	0.9 DL - ERX/1.4	0.9 DL + 1.0 ERY
13	1.2 DL - 1.0 ERX + 1.0 LL	0.9 DL + ERY/1.4	0.9 DL - 1.0 ERX
14	1.2 DL + 1.0 ERY + 1.0 LL	0.9 DL– ERY/1.4	0.9 DL - 1.0 ERY
15	1.2 DL - 1.0 ERY + 1.0 LL	DL + LL + WX	1.2 DL + 1.0 ERX
16	0.9 DL + 1.6 WX	DL + LL - WX	1.2 DL + 1.0 ERY
17	0.9 DL - 1.6 WX	DL + LL + WY	1.2 DL – 1.0 ERX
18	0.9 DL + 1.6 WY	DL + LL - WY	1.2 DL - 1.0 ERY
19	0.9 DL - 1.6 WY	DL + LL + ERX/1.4	1.2 DL + 0.5 LL + 1.0 ERX
20	0.9 DL + 1.0 ERX	DL + LL – ERX/1.4	1.2 DL + 0.5 LL + 1.0 ERY
21	0.9 DL - 1.0 ERX	DL + LL + ERY/1.4	1.2 DL + 0.5 LL - 1.0 ERX
22	0.9 DL + 1.0 ERY	DL + LL – ERY/1.4	1.2 DL + 0.5 LL - 1.0 ERY
23	0.9 DL - 1.0 ERY	DL + TEMP + LL	1.2 DL + 1.6 (LL + TEMP)

	RC Design Load Combinations	Service Load Combinations	Steel Design Load Combinations
24	1.2 (DL + TEMP) + 1.6 LL		1.2 DL + 0.5 (LL + TEMP) + 1.3 WX
25			1.2 DL + 0.5 (LL + TEMP) + 1.3 WY
26			1.2 DL + 0.5 (LL + TEMP) - 1.3 WX
27			1.2 DL + 0.5 (LL + TEMP) - 1.3 WY
28			1.2 DL + 0.5 (LL + TEMP) + ERX
29			1.2 DL + 0.5 (LL + TEMP) + ERY
30			1.2 DL + 0.5 (LL + TEMP) – ERX
31			1.2 DL + 0.5 (LL + TEMP) - ERY

Chapter 04

ANALYSIS AND RESULTS

4.1 INTRODUCTION

This chapter discusses the analysis of structural components of the building based on the results given by the software. Design load combinations are used in component capacity check for ultimate limit and the service load combinations are used for serviceability limit state check.

4.2 FUNDAMENTAL MODES AND BASE SHEAR

The fundamental modes and time periods are presented in the following table.

Table 4.1: Fundamental Periods

Mode	Time Period (s)
1	0.47526
2	0.21576
3	0.20014

The seismic base shear from equivalent static analysis and response spectrum analysis is presented in the following table. The appropriate scale factor is calculated in order to scale the parameters for response spectrum analysis. The detail of scale factor calculation is described in appendix A.

Table 4.2: Base Shear

	Base Shear (Ton)	% Dead Load
ESX	25	24
ERX (unfactored)	88	85
ERX (factored)	26	25
ESY	25	24
ERY (unfactored)	79	76
ERY (factored)	23	22
Dead Load (Ton)	103	
Scale factor of g (ERX)	0.2966	
Scale factor of g (ERY)	0.2966	

4.3 STRUCTURAL COMPONENT CAPACITY

4.3.1 Ultimate Limit

The flexural capacity, shear capacity and axial load capacity of structural components are evaluated in accordance with ACI 318-02. The details of manual calculations are shown in appendix B. The structural component capacity of each component is summarized in the following table.

Table 4.3: Structural Component Capacity

	Bending (Ton-m/m)		Shear (Ton/m)	Compression (Ton/m)	Tension (Ton/m)
	Positive	Negative			
Wall	0.8	0.8	13	81	10
Wall joint	1.6	1.6	21.8	88	21
Footing	1.2		8.8		
Footing wall	1.2	1.2	22	237	15

4.3.2 Serviceability Limit

The cracking stress in concrete, the allowable compressive stress in concrete and the allowable tensile stress in steel are checked for serviceability limit state.

Cracking stress in concrete	= 7.5 x sqrt (f_c') (f_c' is in psi)
	= 7.5 x sqrt (3000)
	= 410 psi (28 ksc)
Allowable compressive stress in concrete	= 0.45 x f_c'
	= 0.45 x 210 ksc
	= 94 ksc

Allowable tensile stress in steel for Grade 60 = 1120 ksc

Allowable tensile stress in steel for Grade 80 = 2200 ksc

4.4 CAPACITY CHECK

The capacity check is done by comparing the analysis results and the capacity of structural components for ultimate limit and serviceability limit. To determine the overstressed members, the limit of contour range of member forces is set up at the capacity of members. The flexural and axial load capacities are checked for the walls. The shear capacity of walls is checked against the in-plane shear force

Since the cracking stress in concrete (28 ksc) is minimum, the serviceability limit is checked against the cracking stress in concrete. If the stress in the analysis results is within this limit, the allowable tensile stresses in steel will be satisfactory. The allowable compressive stress in concrete is also checked.

4.5 ANALYSIS RESULTS

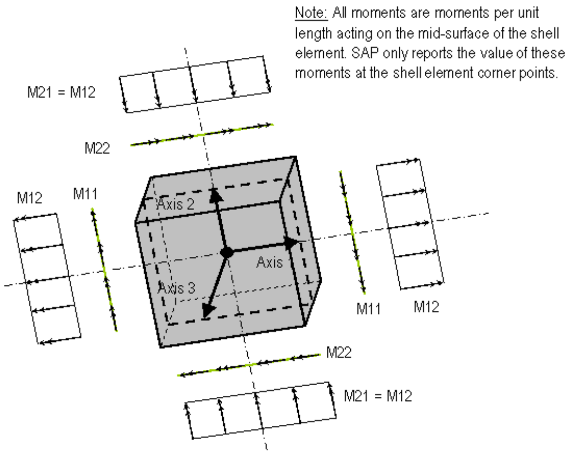


Figure 4.1: Moments M11, M22 and M12

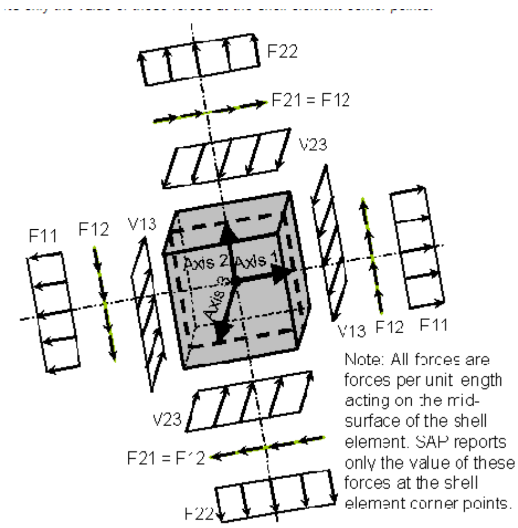


Figure 4.2: Axial Forces F11, F22 and Shear Forces F12, V13, V23

The analysis results of maximum moment, shear force and axial force contours in the walls and footing are shown in this section. The contour limits are set up based on the capacity of the members.

4.5.1 Analysis Results of Walls

4.5.1.1. Ultimate Membrane Forces

The ultimate membrane forces in walls are shown in the following figures. The M11, M22, F11, F22 and F12 are checked against the flexural capacity, axial load capacity and shear capacity. The results from envelope of design load combinations are used.

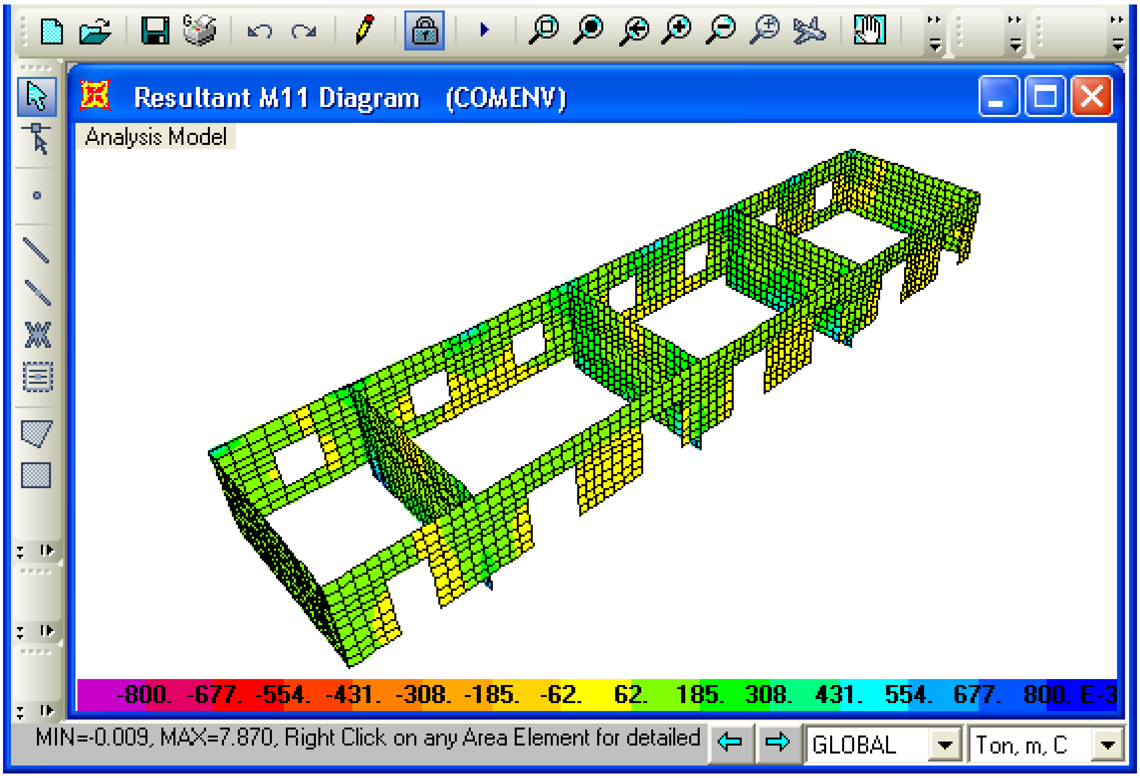


Figure 4.3: M11 Diagram of Wall (ENVMAX)

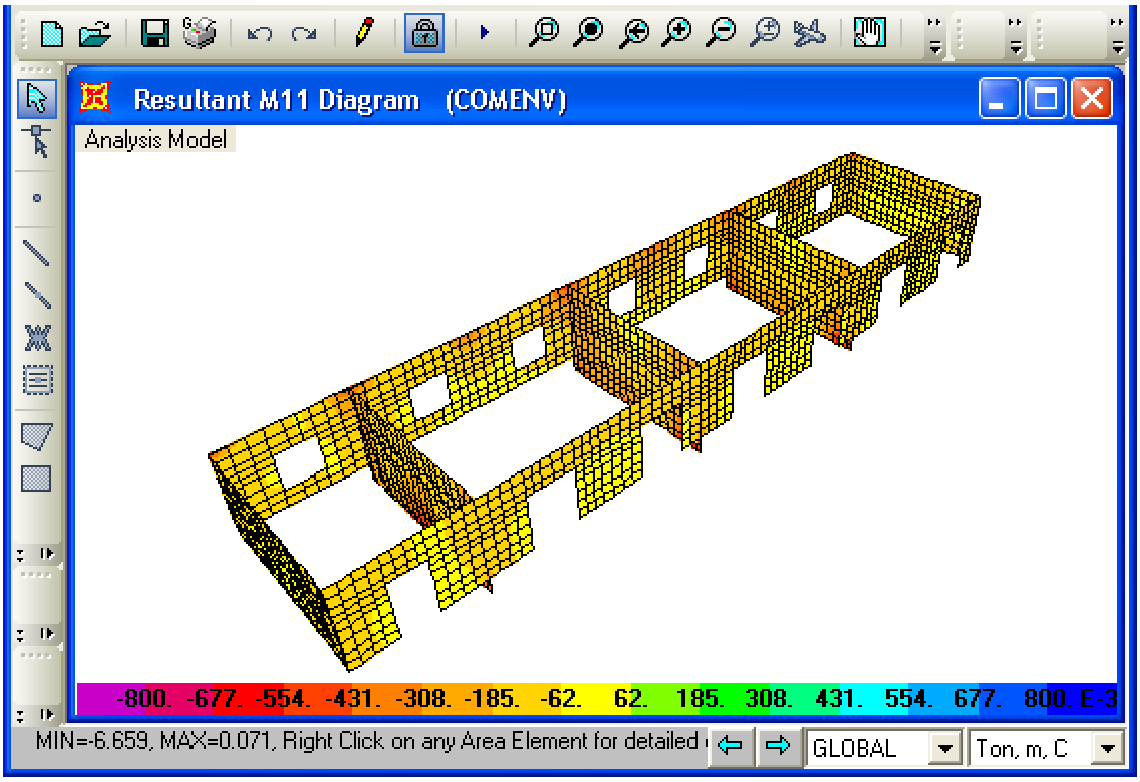


Figure 4.4: M11 Diagram of Wall (ENVMIN)

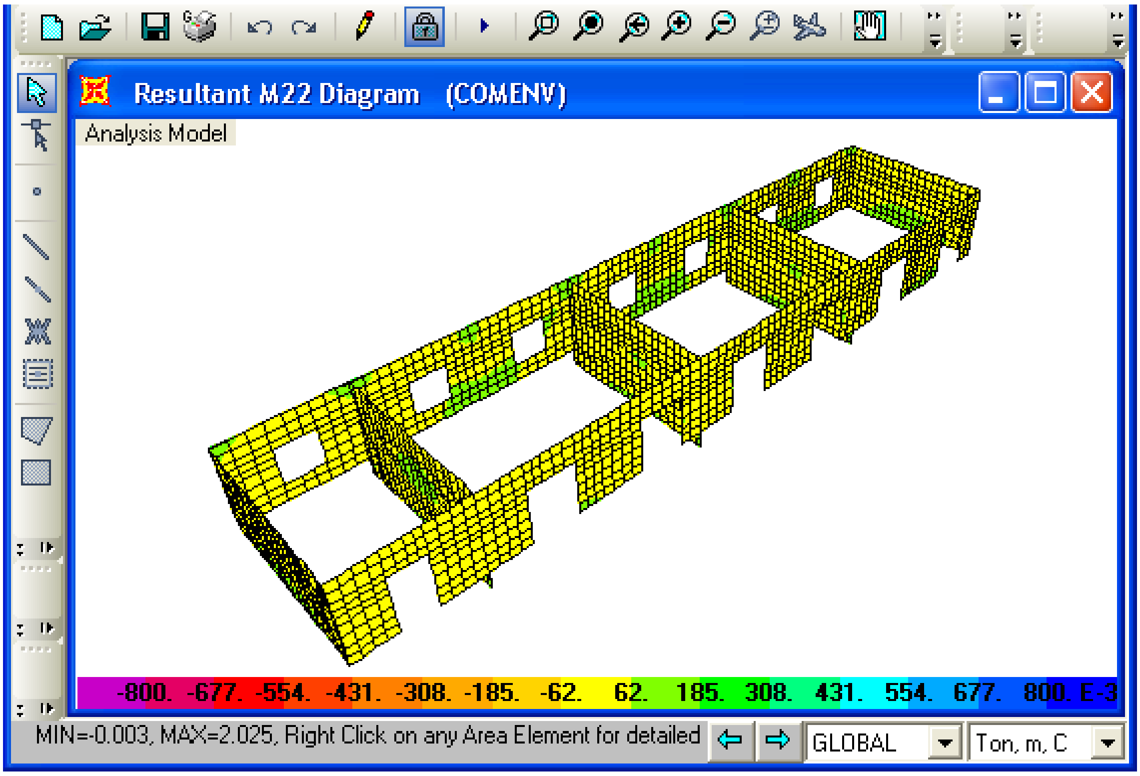


Figure 4.5: M22 Diagram of Wall (ENVMAX)

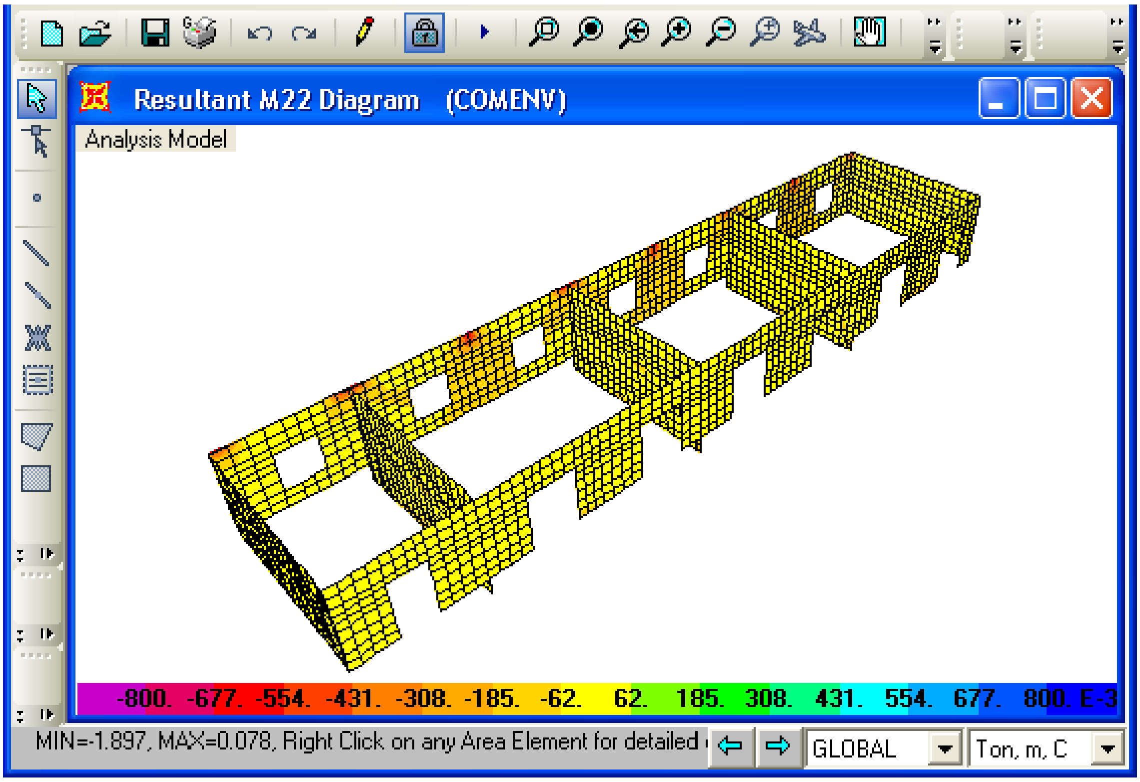


Figure 4.6: M22 Diagram of Wall (ENVMIN)

4.5.1.2. Discussion about Moment Results

It can be seen from the table 4.3, the flexural capacity of 7 inch thick SCI Wall Panel is calculated as 0.8 Ton-m/m in accordance with ACI 318-02. The contour limits are defined in SAP2000 based on this flexural capacity of the member. The contour range is setup between +0.8 and -0.8 and then it was checked in the software. If we look at the figures 4.3, 4.4, 4.5 and 4.6, it is clearly evident that all the bending stresses are well within the limits of the capacity of the building. So it can be concluded that the building is safe against flexural stresses.

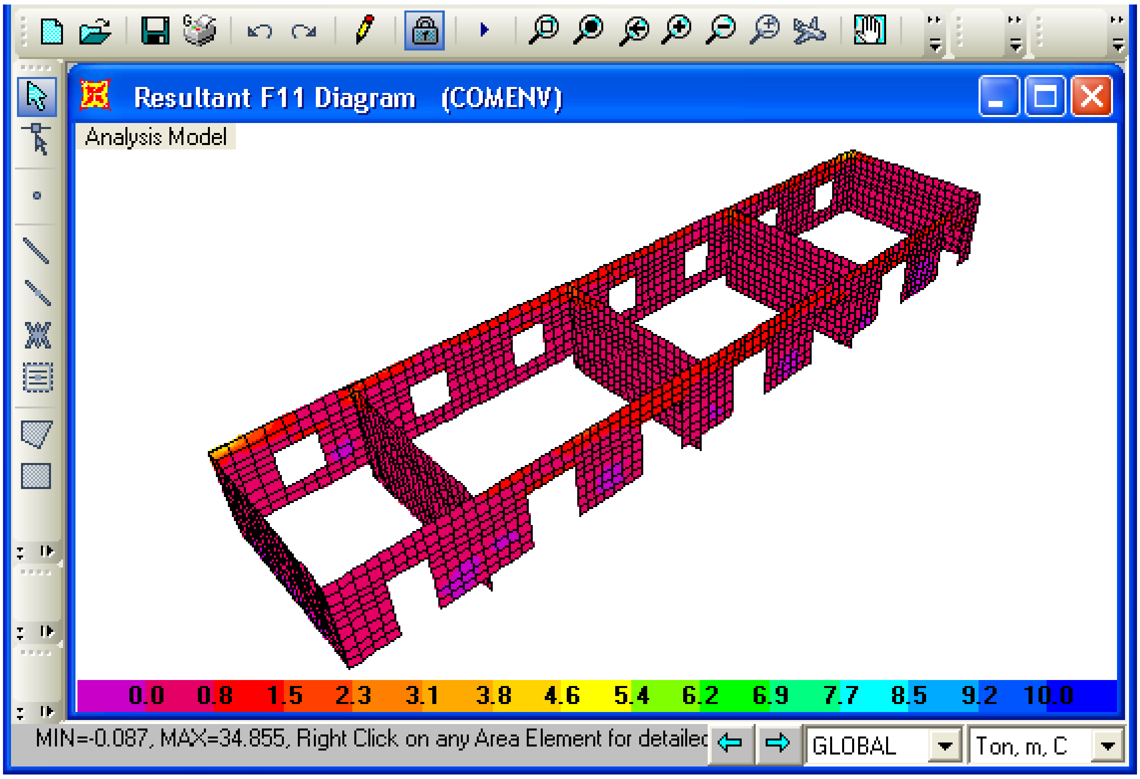


Figure 4.7: F11 Diagram of Wall (ENVMAX)

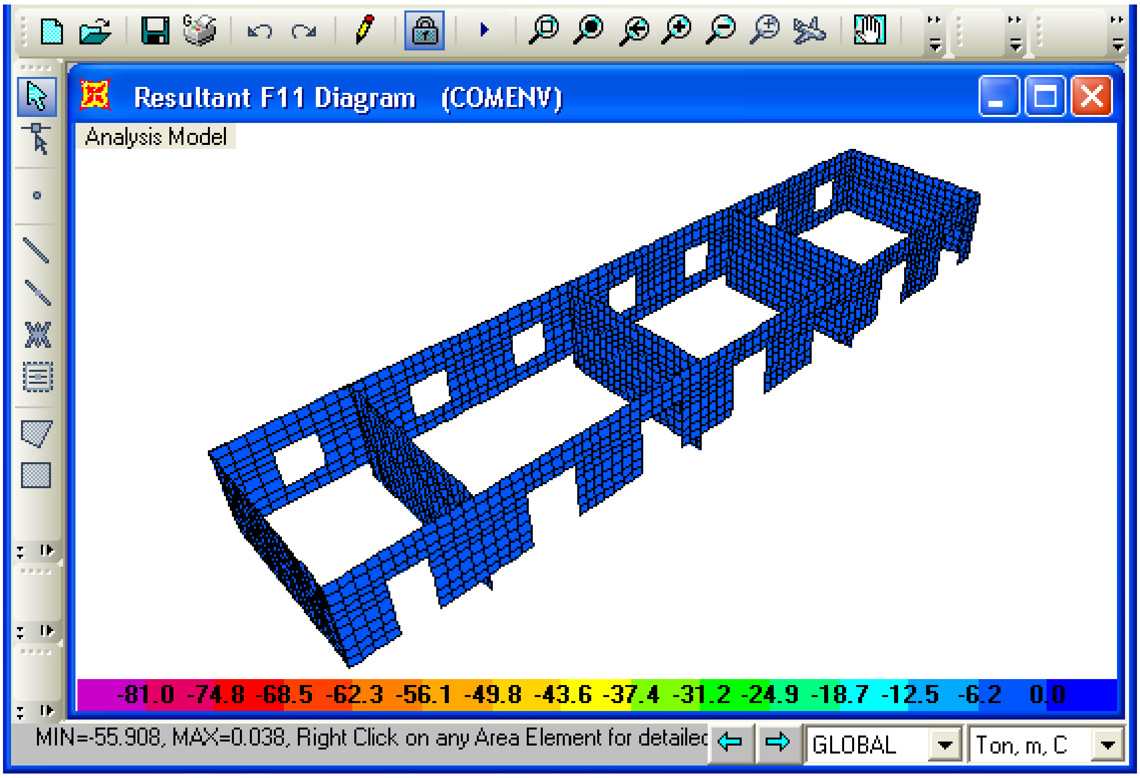


Figure 4.8: F11 Diagram of Wall (ENVMIN)

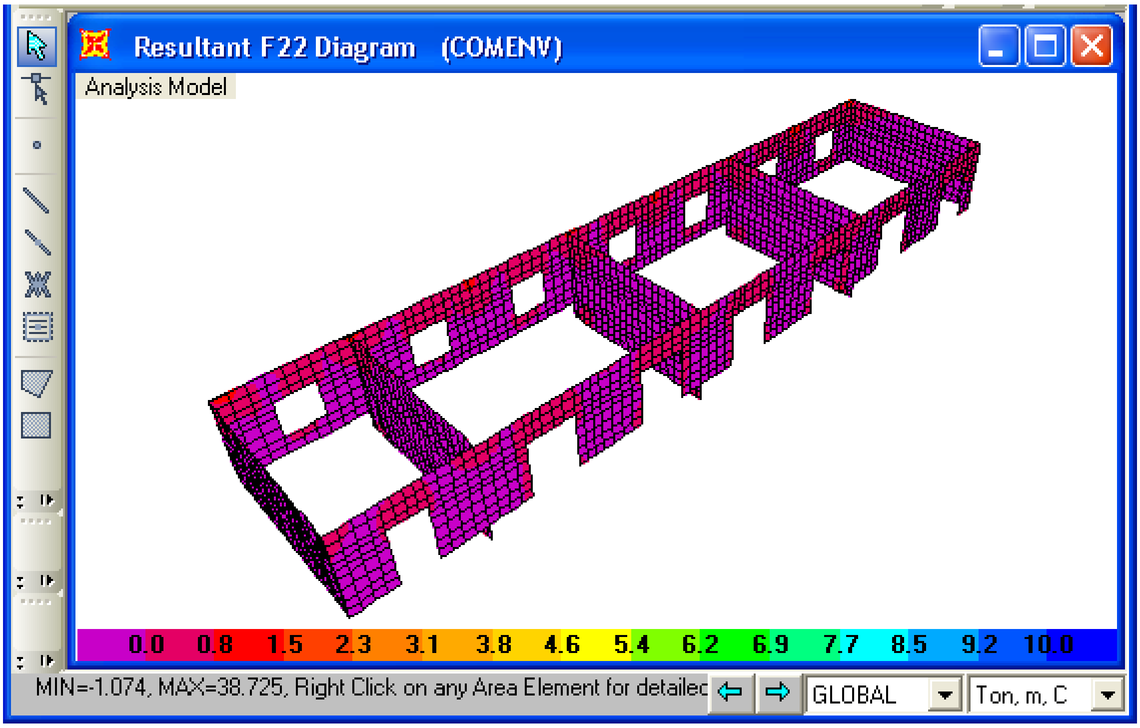


Figure 4.9: F22 Diagram of Wall (ENVMAX)

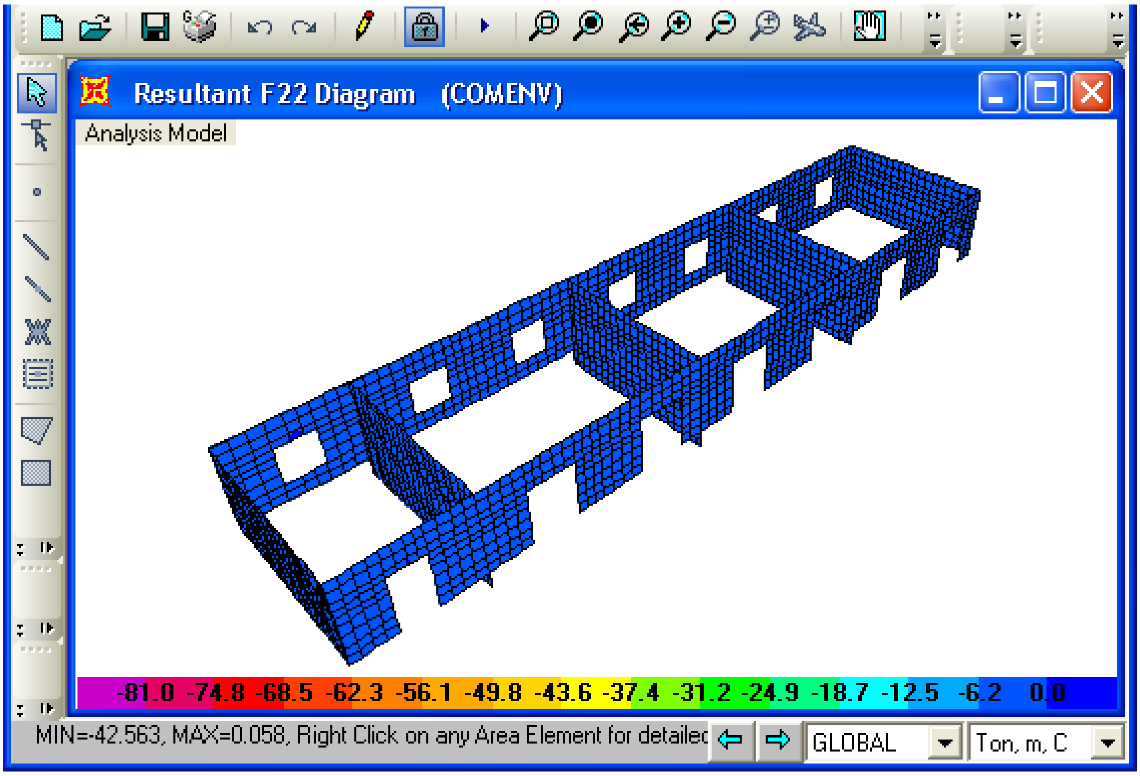


Figure 4.10: F22 Diagram of Wall (ENVMIN)

4.5.1.3. Discussion about Axial Force Results

It can be seen from the table 4.3, the compressive load capacity of 7 inch thick SCI Wall Panel is calculated as 81 Ton/m and tensile capacity as 10 Ton/m in accordance with ACI 318-02. The contour limits are defined in SAP2000 based on these compressive and tensile capacities of the member. The contour range is setup between 0 and 10 for tension stresses and then 0 to -81 for compression stresses and then it was checked in the software. If you look at the figures 4.7, 4.8, 4.9 and 4.10, it is clearly evident that all the compressive and tensile stresses are well within the limits of the capacity of the building. So it can be concluded that the building is safe against axial stresses.

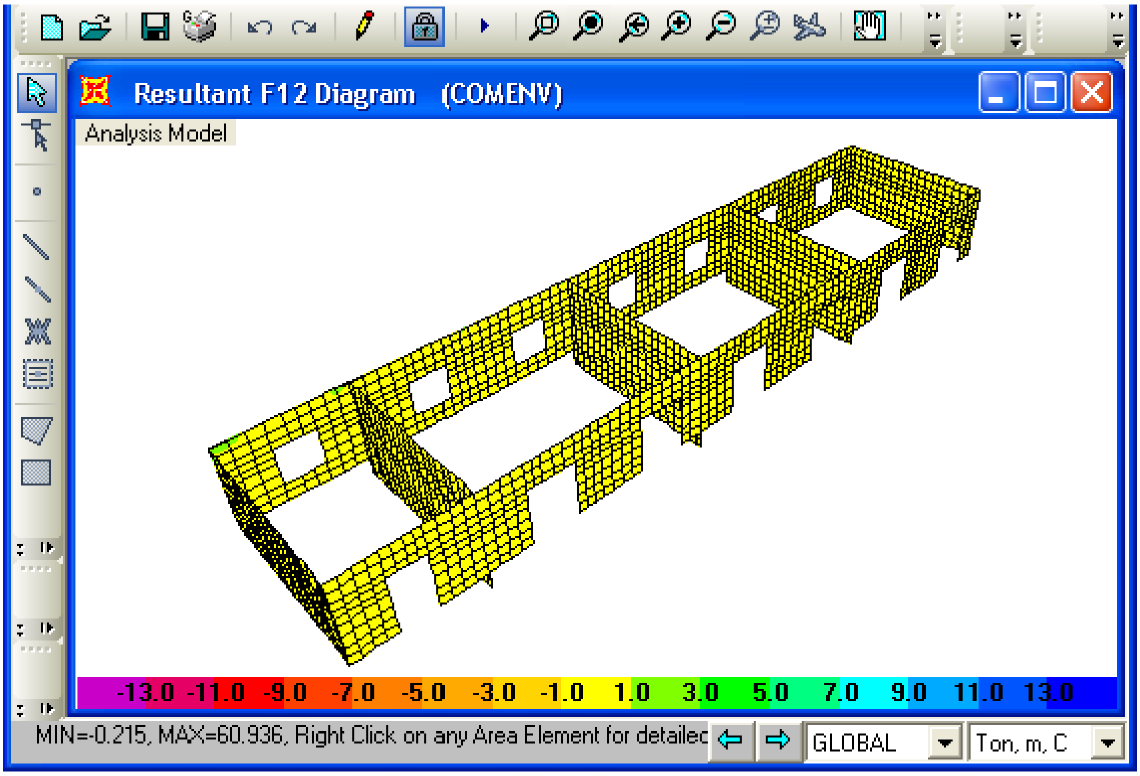


Figure 4.11: F12 Diagram of Wall (ENVMAX)

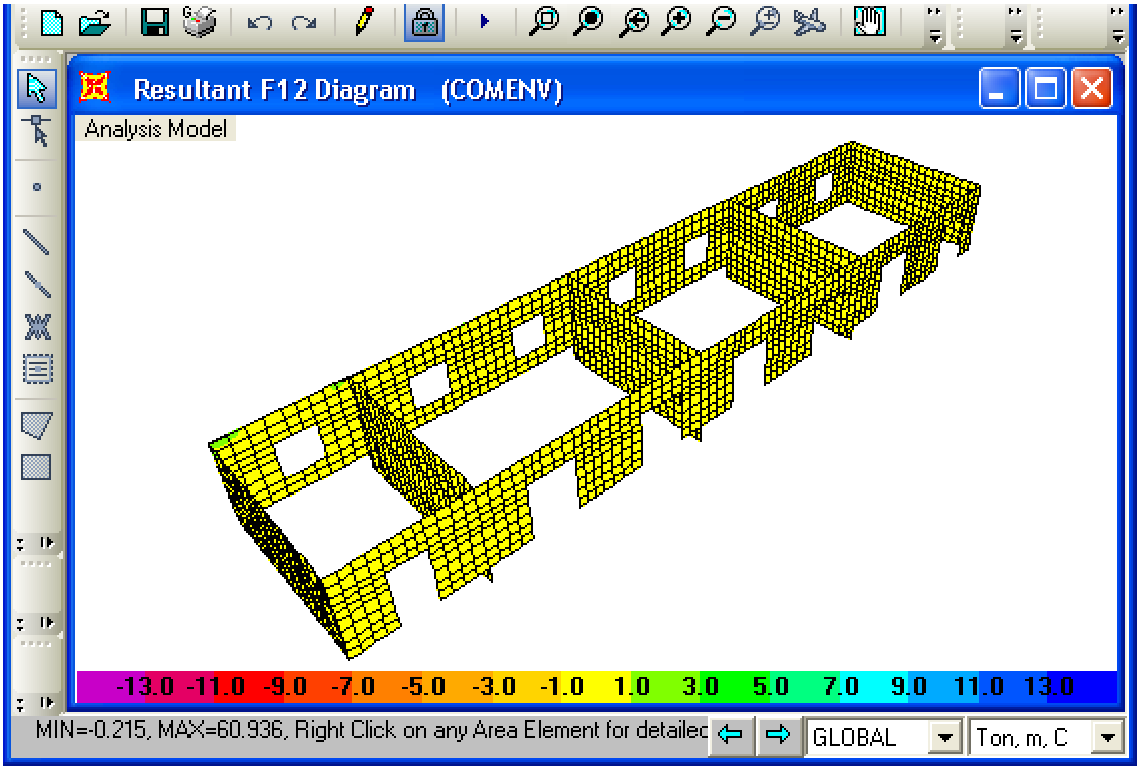


Figure 4.12: F12 Diagram of Wall (ENVMIN)

4.5.1.4. Discussion about Shear Force Results

It can be seen from the table 4.3, the shear capacity of 7 inch thick SCI Wall Panel is calculated as 13 Ton/m in accordance with ACI 318-02. The contour limits are defined in SAP2000 based on this shear capacity of the member. The contour range is setup between +13 and -13 for shear stresses and then it was checked in the software. If you look at the figures 4.11 and 4.12, it is clearly evident that all the shear stresses are well within the limits of the capacity of the building. So it can be concluded that the building is safe against shear stresses.

4.5.1.5. Cracking Stress

The cracking stress in walls is shown in the following figures. The S11 and S22 stresses are checked against the cracking stress capacity and allowable compressive stress. The results from envelope of service load combinations are used.

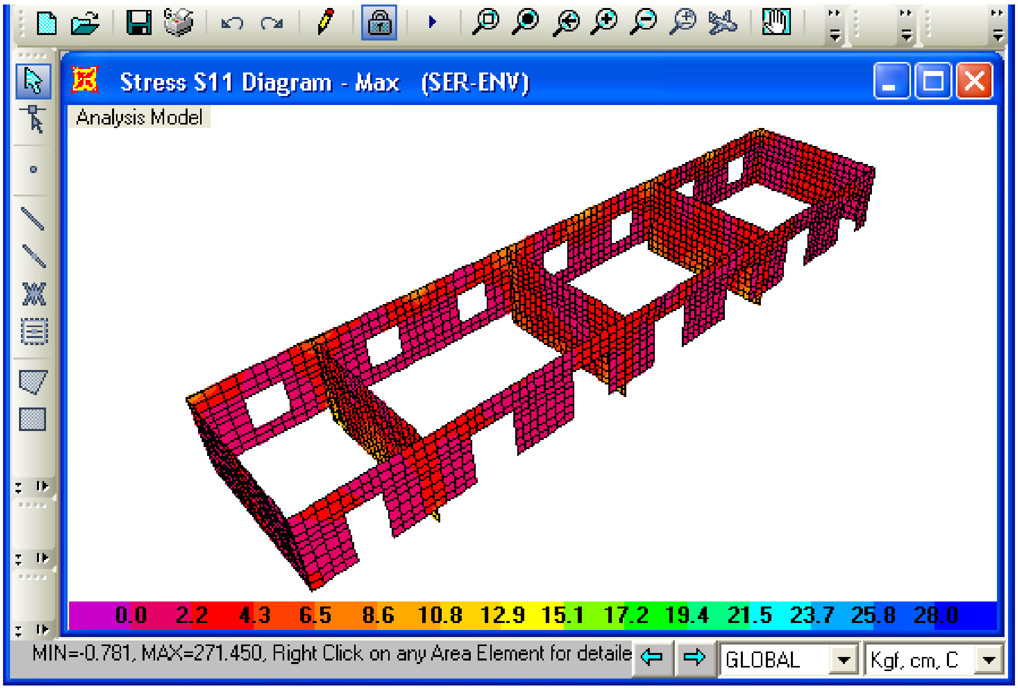


Figure 4.13: Maximum S11 Diagram of Wall (ENVMAX)

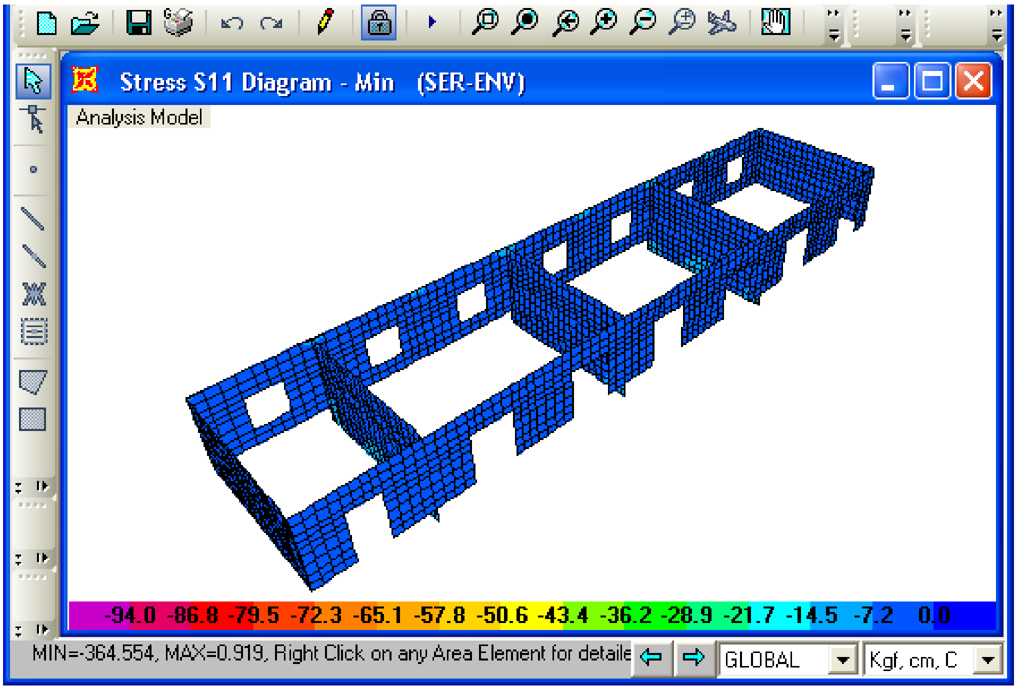


Figure 4.14: Minimum S11 Diagram of Wall (ENVMIN)

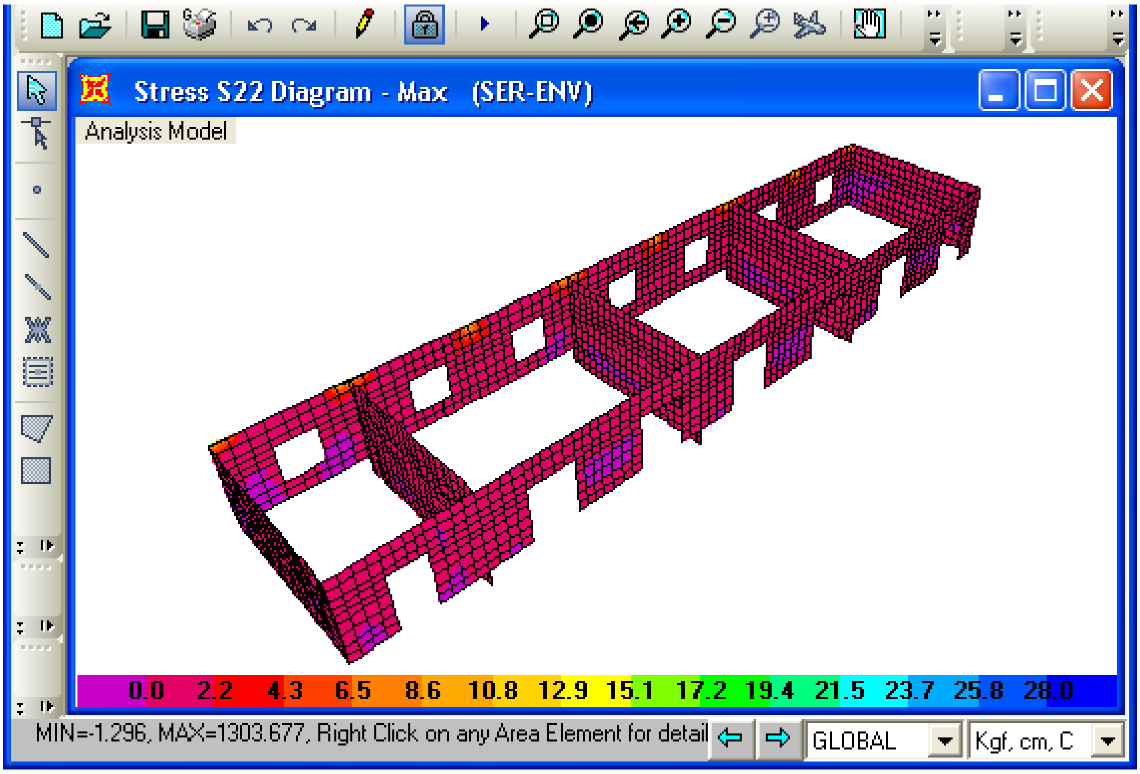


Figure 4.15: Maximum S22 Diagram of Wall (ENVMAX)

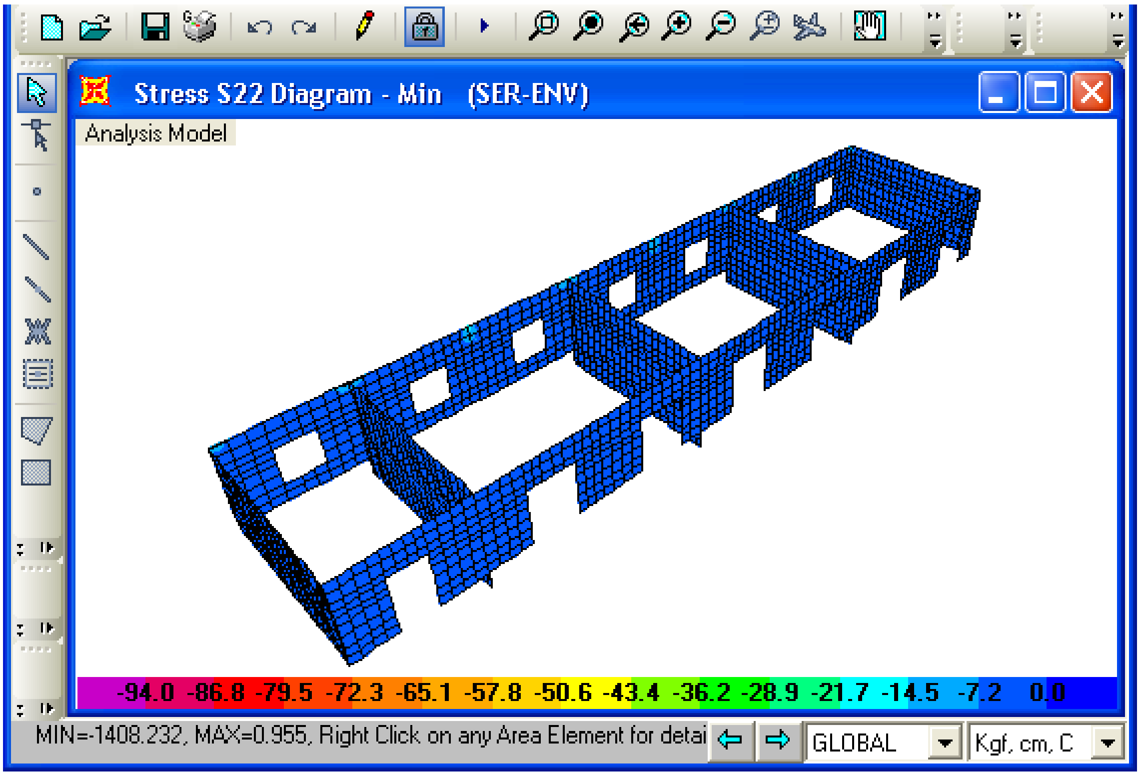


Figure 4.16: Minimum S22 Diagram of Wall (ENVMIN)

4.5.1.6. Discussion about Cracking Stress Results

The cracking stress and allowable compressive stress in concrete of 7 inch thick SCI Wall Panel are calculated as 28 ksc and 94 ksc respectively in accordance with ACI 318-02. The contour limits are defined in SAP2000 based on these capacities of the member. The contour range is setup between 0 and 28 for cracking stresses and then 0 to -94 for compressive stresses and then it was checked in the software. If you look at the figures 4.13, 4.14, 4.15 and 4.16, it is clearly evident that all the cracking and compressive stresses are well within the limits of the capacity of the building. So it can be concluded that the building is safe against cracking and compressive stresses in concrete.

4.6 COMPARISON OF DIFFERENT THICKNESSES

To make a comparison between the SCIP wall panels of different thicknesses, four (04) SCIP sections of different thicknesses were modeled in SAP2000 software. The detail of these sections is given hereunder;

4 inch thick SCIP wall panel with 1 inch thick insulation layer

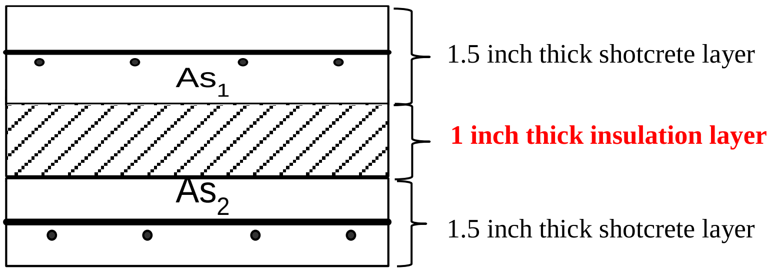


Figure 4.17: 4" Thick SCIP Wall Panel

5 inch thick SCIP wall panel with 2 inch thick insulation layer

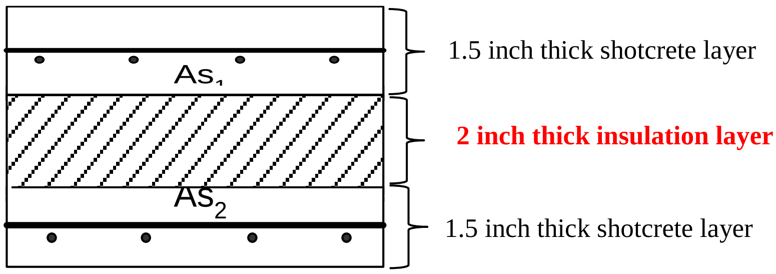


Figure 4.18: 5" Thick SCIP Wall Panel

6 inch thick SCIP wall panel with 3 inch thick insulation layer

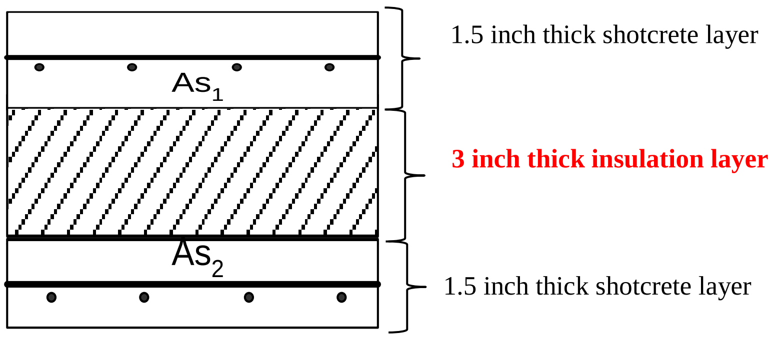


Figure 4.19: 6" Thick SCIP Wall Panel

7 inch thick SCIP wall panel with 4 inch thick insulation layer

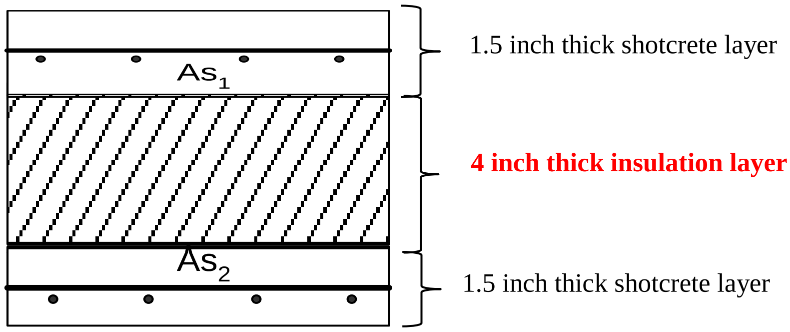


Figure 4.20: 7" Thick SCIP Wall Panel

All these panels having insulation thickness variation from 1 inch to 4 inch and shotcrete layer of 1.5 inch (on both sides) were analyzed in SAP2000 for moments M11, M22 and M12, axial forces F11 and F22 and shear forces F12, V13 and V23. Then these SCIP Wall Panels were checked for translations U1, U2 and U3 and rotations R1, R2 and R3. The analysis results obtained from the SAP2000 software were then plotted on graphs to understand the pattern of variation of above mentioned forces (flexural, axial and shear) with variation of insulation thickness by keeping the concrete cover same.

As the negative sign indicate the direction only, so during the plotting of graphs, these negative forces are multiplied with -1 to transform them towards positive side so that the pattern of curve can properly be identified.

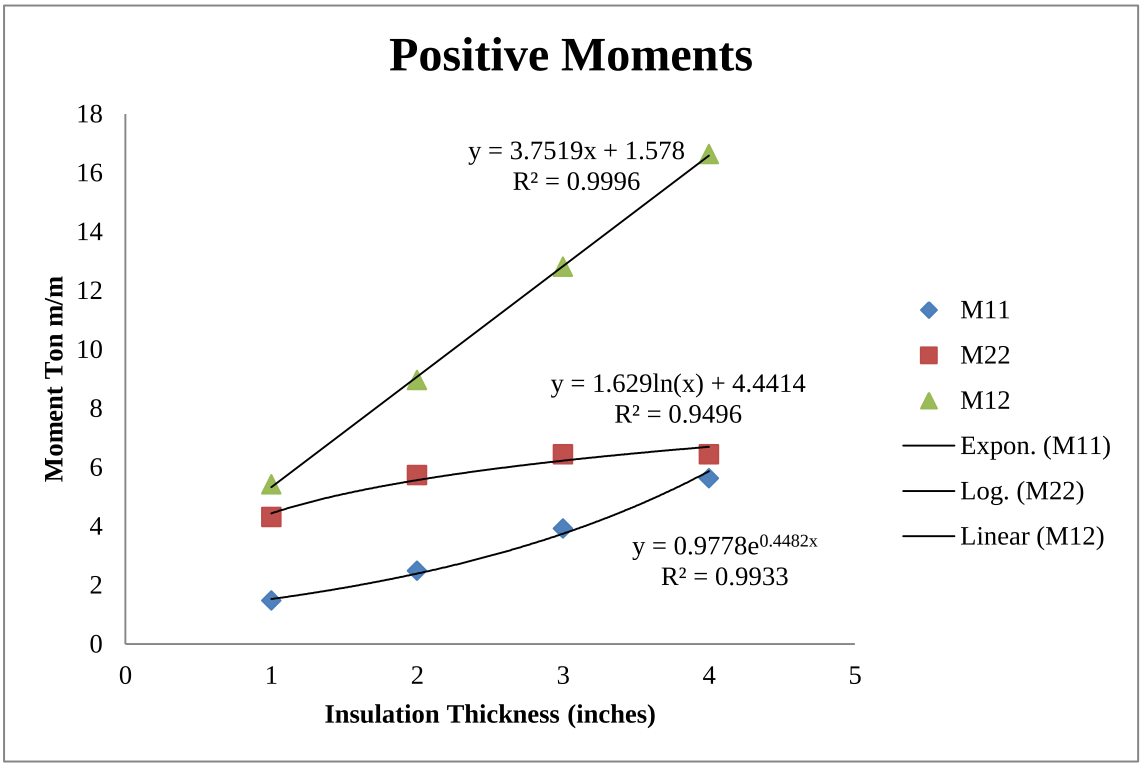


Figure 4.21: Positive Moments

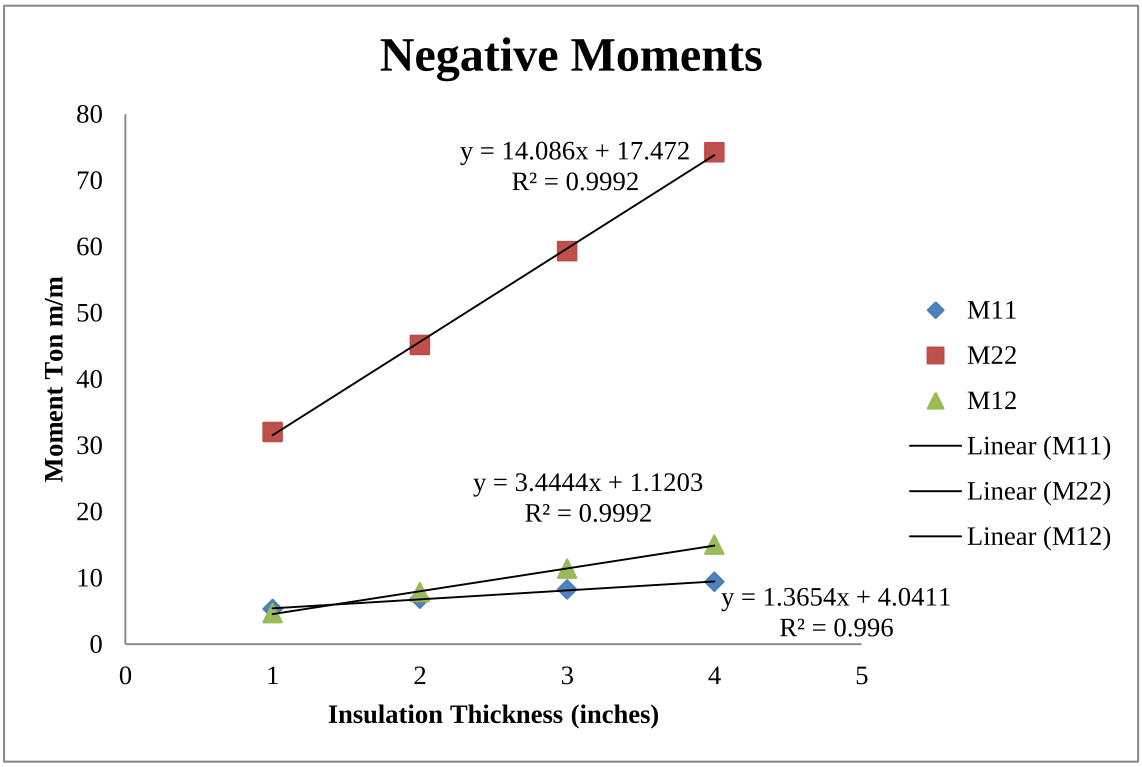


Figure 4.22: Negative Moments

4.6.1. Discussion about Moment Charts

In figure 4.21, it is observed that the positive moment M11 increase exponentially as a function of insulation thickness, the positive moment M22 increases logarithmically as a function of insulation thickness and the positive moment M12 increase linearly with increase in insulation thickness. The relations obtained from analysis results are:

$M11 = 0.9778e^{0.4482x}$

$M22 = 1.629\ln(x) + 4.4414$

$M12 = 3.7519x + 1.578$

Where "x" is the thickness of insulation

In figure 4.22, it is observed that all the negative moments M11, M22 and M12 increase linearly as a function of insulation thickness. The relations obtained from the analysis results are:

$M11 = 1.3654x + 4.0411$

$M22 = 14.086x + 17.472$

$M12 = 3.4444x + 1.1203$

Where "x" is the thickness of insulation

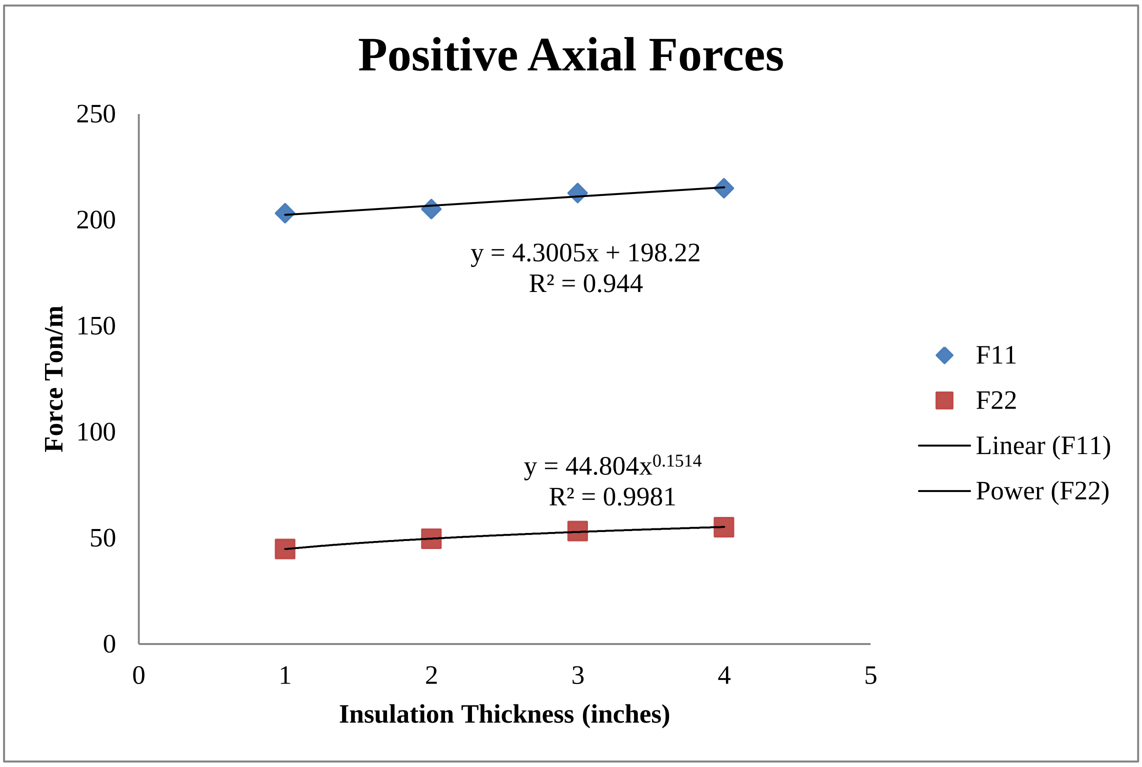


Figure 4.23: Positive Axial Forces

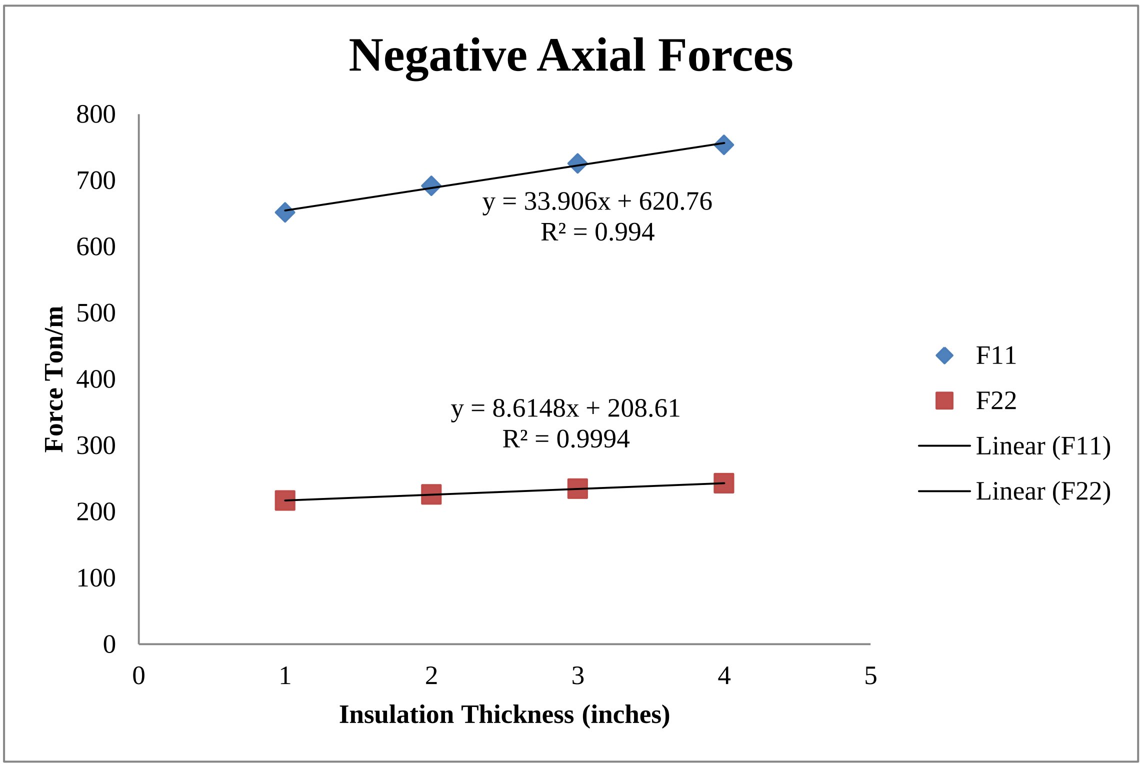


Figure 4.24: Negative Axial Forces

4.6.2. Discussion about Axial Forces Charts

In figure 4.23, it is evident that positive axial force F11 increases linearly while F22 increases as a power function of thickness yielding following relations:

$F11 = 4.3005x + 198.22$

$F22 = 44.804x^{0.1514}$

Where "x" is the thickness of insulation

In figure 4.24, it can be seen that the negative axial forces F11 and F22 increase linearly as a function of insulation thickness. The relations obtained from analysis results are:

$F11 = 33.906x + 620.76$

$F22 = 8.6148x + 208.61$

Where "x" is the thickness of insulation

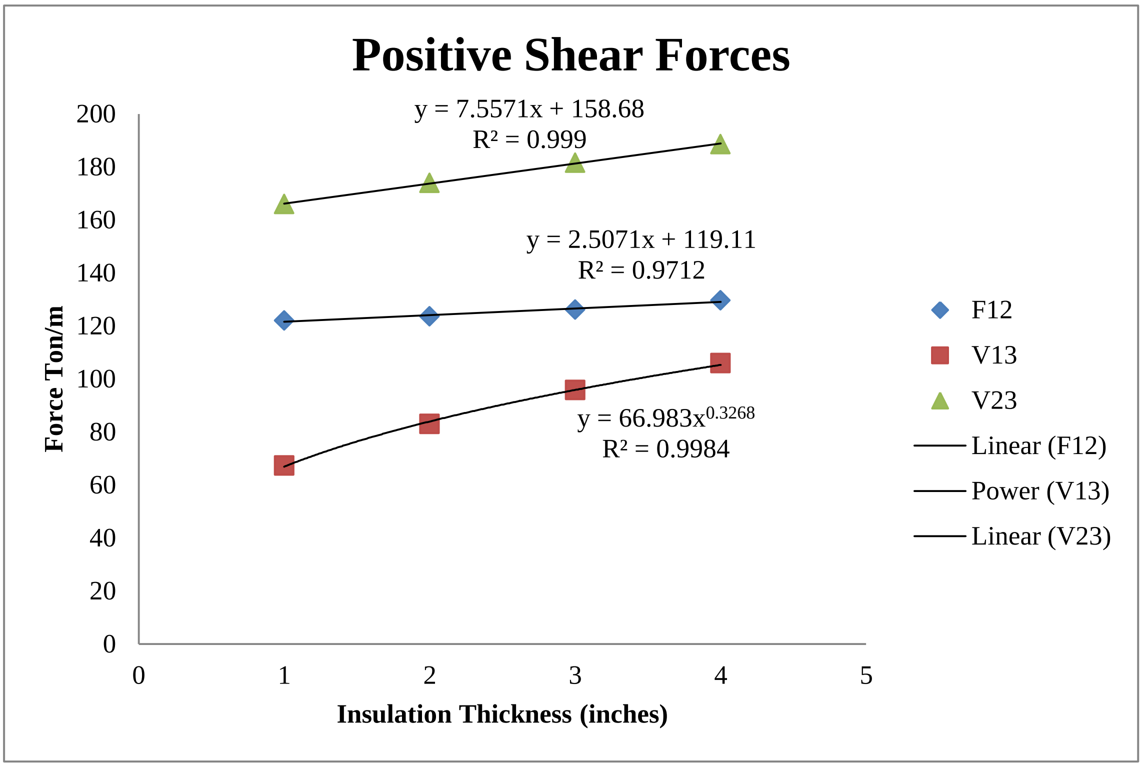


Figure 4.25: Positive Shear Forces

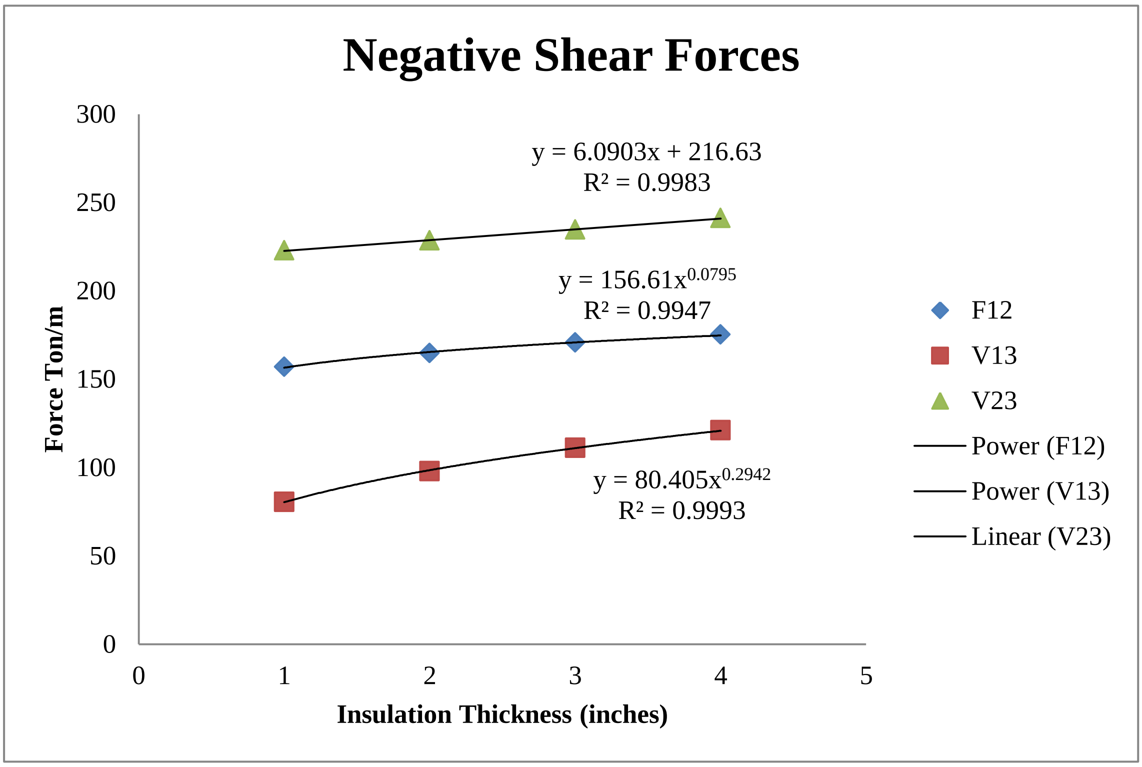


Figure 4. 26: Negative Shear Forces

4.6.3. Discussion about Shear Forces Charts

From figure 4.25, it is observed that the positive shear forces F12 and V23 increase linearly as a function of insulation thickness, while V13 increase as a power function of insulation thickness. The relations obtained from analysis results are:

$$F12 = 2.5071x + 119.11$$

$$V13 = 66.983x^{0.3268}$$

$$V23 = 7.5571x + 158.68$$

Where "x" is the thickness of insulation

From figure 4.26, it can be seen that the negative shear forces F12 and V13 increase as a power function of insulation thickness whereas V23 increase linearly as a function of insulation thickness. The relations obtained from analysis results are:

$$F12 = 156.61x^{0.0795}$$

$$V13 = 80.405x^{0.2942}$$

$$V23 = 6.0903x + 216.63$$

Where "x" is the thickness of insulation

Table 4.4: Positive Translations (in mm)

Insulation Thickness (inch)	U1	U2	U3
1	34.401	33.123	3.936
2	34.373	32.241	3.741
3	34.340	31.495	3.547
4	34.144	30.942	2.700

Table 4.5: Negative Translations (in mm)

Insulation Thickness (inch)	U1	U2	U3
1	34.255	27.288	117.995
2	34.232	26.946	117.044
3	34.206	26.678	115.953
4	33.984	26.122	114.287

Table 4.6: Positive Rotations (in radians)

Insulation Thickness (inch)	R1	R2	R3
1	0.013306	0.010526	0.004791
2	0.013384	0.010519	0.004675
3	0.013403	0.010508	0.004566
4	0.013211	0.010452	0.004332

Table 4.7: Negative Rotations (in radians)

Insulation Thickness (inch)	R1	R2	R3
1	0.011801	0.010498	0.00489
2	0.010216	0.010493	0.004785
3	0.008804	0.010486	0.004667
4	0.007363	0.010423	0.004411

4.6.4. Discussion about Deflections

If we look at the tables 4.4 and 4.5, the displacements U1, U2 and U3 are in millimeters and it is evident from the tables that the difference between the linear displacements is not significant. So we can say that by keeping the concrete cover same, there is no significant difference on deflections is observed by increasing the insulation thickness.

It is observed from tables 4.6 and 4.7 that the angular displacements R1, R2 and R3 also show the same behavior as we have observed in case of translations. The rotations also remain almost same if we keep the concrete cover same. There is no significant difference in rotations is observed by increasing the insulation thickness only.

4.7 FINANCIAL COMPARISON

A financial comparison is also made between the SCIP building and a conventional frame structure. The detail is given hereafter;

Table 4.8: Financial Comparison

Structural Concrete Insulated Panel (SCIP) (7 inch total thickness)			
Item Description	**Rate Per Sq. Ft. (PKR)**	**Area of Building (Sq.Ft.)**	**Cost (PKR)**
Design of complete architectural, structural, civil, electrical, plumbing & public health works, analysis and design for the whole building, on covered area basis	52	**2295**	119,340/-
Supply of pre-fabricated structural and non-structural parts of the building including roofing, insulation, false ceiling with their connections and accessories, on covered area basis	470		1,078,650/-
Excavation for foundations, construction and installation/erection of all architectural, structural, civil, electrical works, on covered area basis	1093		2,508,435/-
TOTAL			**3,706,425/-**
Cost of Conventional Building with same covered area			**4,704,750/-**
Difference			**- 998,325/-** **27% Cheaper**

The financial comparison clearly indicates that a SCIP building is 27% cheaper from the conventional frame structure building with the same covered area.

4.8 ERECTION SEQUENCE

4.8.1 Foundation

The first step for the construction of a SCIP building is the preparation of foundation. A foundation is built up with conventional reinforced concrete of 3000psi. The site is properly compacted before the laying of foundation. Dowels are also provided at the locations where the walls of SCI Panels to be erected.

Figure 4.27: Foundation

4.8.2 Super Structure Installation

After the preparation of RCC foundation, the second step is the installation of super structure. In this phase, Structural Concrete Insulated Wall Panels are installed according to the drawings and design of the building. These wall panels are properly joined where two panels meet with each other and provided with extra layer of GI wire mesh. This provision of extra layer of GI Wire mesh is effective to protect the wall from cracking at the joints.

Figure 4.28: Super Structure

4.8.3 Shotcreting and Plaster

After the completion of erection work, third phase starts which include the shotcreting and plastering of the building. Shotcreting is done by spraying concrete containing pan size aggregate followed by plastering. Shotcreting and plastering is done on both faces of the SCI Panels which result in a highly finished surface.

Figure 4.29: Shotcreting and Plaster

4.8.4 Finishing Works

At the end, finishing works are done which include fixing of doors and windows, installation of false ceiling and electrical fixtures, flooring and painting.

Figure 4.30: Finishing Works

Chapter 05

CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL INFORMATIONS

The building designed and considered under this research work is a typical single storey primary school building. The buildings with same dimensions and covered area are constructed in earthquake affected areas of AJK and KPK with several other materials like block masonry, brick masonry, frame structure and light gauge cold formed steel structure.

5.2 CONCLUSIONS

i. Capacity of Structural Members

Structural Concrete Insulated Wall Panels of **seven (07) inch thickness** are checked for flexural capacity, axial load capacity and shear capacity. If we look at the results yielded by the software, it is evident from **figures 4.3 to 4.16** that the SCIP walls have sufficient capacity to resist the moments, axial stresses and shear stresses. None of the wall portion exceeds the allowable limits of moments, axial forces and shear forces.

ii. Comparison of SCIP Walls of Different Thicknesses

From the **figures 4.21 to 4.26** shown above, as the thickness of insulation increases the moments, axial forces and shear stresses also increase with different patterns. Some forces show linear behaviour, some increase exponentially, some increase in the form of power and some increase logarithmically. In case of deflections, if we look at the **tables 4.4 to 4.7**, as the thickness of insulation increases the displacements reduce accordingly. But it is evident from the tables shown above that the difference in deflections not significant. So we can say that by increasing insulation thickness with same concrete cover depth (i.e 1.5 inch) there will be no significant change in deflections.

iii. Financial Comparison

A SCIP building having covered area of **2295 $ft.^2$** and seven (07) inch wall thickness can be constructed (complete in all respects) in **PKR 3,706,425/-** while a conventional frame structure building with same covered area can be constructed (complete in all respects) in

PKR 4,704,750/-. It can be concluded that a SCIP building is almost **27% cheaper** than the conventional frame structure building.

5.3 RECOMMENDATIONS

i. Considerable part of the country is earthquake prone as indicated in the map provided by Pakistan Building Code. Hence it is recommended that SCIP technology must be promoted as a construction material in all earthquake prone areas to avoid any severe consequences in case of any disaster.

ii. In the scenario of post-earthquake circumstances, it is of supreme importance to introduce such kind of construction technologies which can withstand against the earthquake forces more effectively and at the same time they can be constructed in speedy manner.

iii. SCIP system is the most advanced technology which should be introduced in Pakistan for reconstruction of office and school buildings in earthquake affected areas of AJK and KPK.

iv. If we look at the seismic zoning map of Pakistan (Appendix C), significant area of the country lies under zone 3 and 4. In such circumstances, where the probability of earthquakes is extremely high, alternative construction technologies should be adopted so that the damage should be minimized.

v. For future research work, it is recommended that experimental work can be done to study the actual behaviour of SCIP wall panels. Then the comparison between analytical work and experimental work can also be done to check the difference between analytical work and experimental work.

REFERENCES

1. Gregory J. Hancock, Thomas M. Murray and Duane S. Ellifritt, "*Cold-Formed Steel Structures to the AISI Specification*"

2. American Society of Civil Engineers (ASCE), "*Minimum Design loads for Buildings and Other Structures*", ASCE 7-05

3. Anil K. Chopra, "*Dynamics of Structures, Theory and Applications to Earthquake Engineering*", Third Edition, Prentice-Hall India

4. Fouad FH, Jarrell F, Heath M, Shalaby A and Vichare A,.Behavior of the MR Sandwich Panel in Flexure, ACI Journal 260 (2004), 73-88.

5. Steven B. Taylor, Harvey B. Manbeck, John J. Janowiak and Dennis R. Hiltunen, Modelling Structural Insulated Panel (SIP) Flexural Creep Deflection, Journal of Structural Engineering 123(12), (1997), 1658-1665

6. Salmon DC, and Einca A, Partially Composite Sandwich Panel Deflection, ASCE Journal of Structural Engineering, 121(4) (1995), 778-783

7. ASTM A653 – "*Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy Coated (Galvanized)*" by Hot-Dip Process.

8. Garas G., Allam M. and El Dessuky R. "straw bale construction as an economic environmental building alternative- a case study"

9. ACI 506R-90 (Reapproved 1995) Guide to Shotcrete, American Concrete Institute, Farmington Hills, MI.

10. ASTM C42 "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of concrete,"

11. Annual Book of ASTM Standards, American Society for Testing and Materials, West Conshohocken, PA

12. ASTM E 72 (2002) "Standard Test Methods of Conducting Strength Tests of Panels for Building Construction," Annual Book of ASTM Standards, American Society for testing and Materials, West Conshohocken, PA

13. PCI Sandwich Wall Panel Committee, "PCI Report – Precast Sandwich Wall Panels," PCI Journal, Chicago, IL (May-June 1997).

14. ACI 318 (2005) Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, MI.

15. ASCE 7 -02 (2002) "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, Reston, VA.

16. Bruce King, [2006] Design of Straw Bale Buildings. Green Building Press, San Rafael, CA

17. Pakistan Straw Bale and Appropriate Building (PAKSBAB), http://www.paksbab.org/index.ph

18. ACI 318-08 – "Building Code Requirements for Structural Concrete"

19. Awad, Z.K., Aravinthan, T. and Zhuge, Y., (2010), "Cost optimum design of structural fibre composite sandwich panel for flooring applications." CICE 2010, Beijing, China.

20. SAP2000, Ver.14, User Manual.

21. AQ Bhatti, SZ Ul Hassan, Z Rafi, Z Khatoon and Q Ali, (2011) "Probabilistic seismic hazard analysis of Islamabad", Pakistan. J Asian Earth Sci 42 (3), 468-478.

22. AQ Bhatti, N Kishi, T Ohno and H Konno, (2006) "Dynamic response analysis for a large-scale RC girder under a falling-weight impact loading" Advances in Engineering Structures, Mechanics & Construction, 99-109.

23. AQ Bhatti, H Varum and Z Alam, (2012) “Seismic vulnerability assessment and evaluation of high rise buildings in Islamabad”, 15th WCEE, World Conference on Earthquake Engineering.

24. BCP. (2007). Seismic Provision for Building Code of Pakistan. Ministry of Housing and Works Government of Pakistan Islamabad.

25. Edward L Wilson. (2000). “Three Dimensional Static and Dynamic analysis of Structures” 3rd Edition. Computers and Structures, Inc. Berkeley, California, USA

APPENDIX A

RESPONSE SPECTRUM SCALE FACTOR CALCULATION

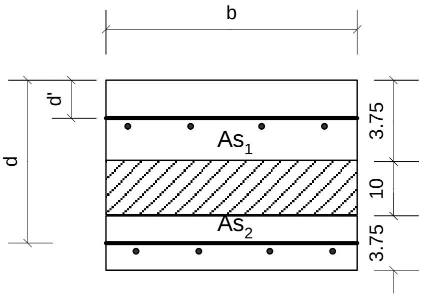
ESX =	25 Ton	ERX (unfactored) =	88 Ton
ESY =	25 Ton	ERY (unfactored) =	79 Ton
Base shear (Static) =	sqrt($25^2 + 25^2$)		
=	35 Ton	ERX (factored) =	26 Ton
Base shear (RS) =	sqrt($88^2 + 79^2$)	ERY (factored) =	23 Ton
=	118 Ton		
Scale factor of g =	35 / 118		
=	0.2966		
Scale factor of g in other direction =	0.2966 x 30%		
=	0.088		

APPENDIX B

REINFORCED CONCRETE DESIGN (ACI 318-02)

a) Structural Component Capacity (SCIP Wall)

i. Moment Capacity



f_y = 5,500 Kg/cm^2

f_c' = 210 Kg/cm^2

A_{s1} = 1.06 cm^2

A_{s2} = 1.06 cm^2

b = 100 cm

d = 15.625 cm

d' = 1.875 cm

a = A_{s2} x f_y / [0,85 x f_c' x b]

= 0.32 cm

ϕM_n = ϕ A_{s2} f_y [d – a/2]

= 81144 kg-cm

= 0.8 Ton-m

ii. Shear Capacity

f_y = 5,500 kg/cm^2

A_v = 0.053 x 2

= 0.106 cm^2

f_c' = 210 kg/cm^2

Width (b) = 7.5 cm

depth (d) = 100 cm

Φ V_c = 0.75 x 0.53 x $(f_c')^{1/2}$ x b x d

	=	$0.75 \times 0.53 \times (210)^{1/2} \times 7.5 \times 100$	kg
	=	4,320	kg
	=	4.3	Ton
ΦV_s	=	$0.75 \times A_v \times f_y \times d / s$	
	=	0.75 x 0.106 x 5500 x 100 / 5	
	=	8745	kg
	=	8.7	Ton
$\Phi V_c + \Phi V_s$	=	4.3 + 8.7	
	=	13	Ton

iii. Compressive Load Capacity

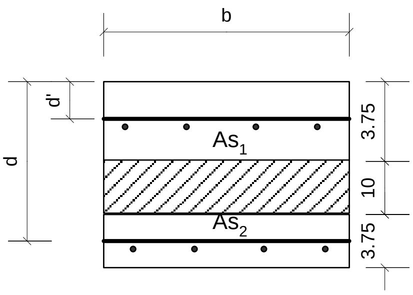
f_y	=	5,500	kg/cm^2
f_c'	=	210	kg/cm^2
A_s	=	2.12	cm^2
Width (b)	=	100	cm
Thickness (th)	=	7.5	cm
A_c	=	750	cm^2
P	=	$0.8 \times 0.7 (A_c \times 0.85 \times fc' + As\ fy)$	
	=	81500	kg
	=	81	Ton

iv. Tensile Load Capacity

f_y	=	5,500	kg/cm^2
A_s	=	2.12	cm^2/m
T	=	$0.9 \times A_s \times f_y$	kg
	=	10494	kg
	=	10	Ton

b) Structural Component Capacity (Wall Joint)

i. Moment Capacity



f_y	=	5,500	Kg/cm^2
f_c’	=	210	Kg/cm^2
A_{s1}	=	2.12	cm^2
A_{s2}	=	2.12	cm^2
b	=	100	cm
d	=	15.625	cm
d’	=	1.875	cm
a	=	A_{s2} x f_y / [0,85 x f_c’ x b]	
	=	0.65	cm
ϕM_n	=	ϕ A_{s2} f_y [d – a/2]	
	=	160558	kg-cm
	=	1.6	Ton-m

ii. Shear Capacity

f_y	=	5,500	kg/cm^2
A_v	=	0.053 x 4	
	=	0.212	cm^2
f_c’	=	210	kg/cm^2
Width (b)	=	7.5	cm
depth (d)	=	100	cm
Φ V_c	=	0.75 x 0.53 x $(f_c')^{1/2}$ x b x d	
	=	0.75 x 0.53 x $(210)^{1/2}$ x 7.5 x 100	kg
	=	4,320	kg
	=	4.3	Ton

ΦV_s	=	$0.75 \times A_v \times f_y \times d / s$	
	=	0.75 x 0.212 x 5500 x 100 / 5	
	=	17490	kg
	=	17.5	Ton
$\Phi V_c + \Phi V_s$	=	4.3 + 17.5	
	=	21.8	Ton

iii. Compressive Load Capacity

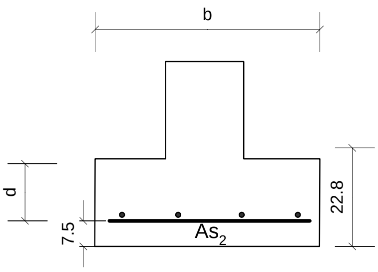
f_y	=	5,500	kg/cm^2
f_c'	=	210	kg/cm^2
A_s	=	4.24	cm^2
Width (b)	=	100	cm
Thickness (th)	=	7.5	cm
A_c	=	750	cm^2
P	=	$0.8 \times 0.7 (A_c \times 0.85 \times fc' + As\ fy)$	
	=	88029	kg
	=	88	Ton

iv. Tensile Load Capacity

f_y	=	5,500	kg/cm^2
A_s	=	4.24	cm^2/m
T	=	$0.9 \times A_s \times f_y$	kg
	=	20988	kg
	=	21	Ton

c) Structural Component Capacity (Footing)

i. Positive Moment Capacity



f_y	=	2800	Kg/cm^2
f_c'	=	210	Kg/cm^2
A_{s1}	=	0	cm^2
A_{s2}	=	3.1	cm^2
b	=	100	cm
d	=	15.3	cm

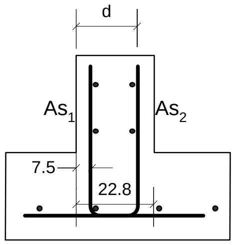
a	=	A_{s2} x f_y / [0,85 x f_c' x b]	
	=	0.48	cm
ϕM_n	=	ϕ A_{s2} f_y [d – a/2]	
	=	117648	kg-cm
	=	1.2	Ton-m

ii. Shear Capacity

f_y	=	2800	kg/cm^2
f_c'	=	210	kg/cm^2
Width (b)	=	100	cm
depth (d)	=	15.3	cm
$\Phi\ V_c$	=	0.75 x 0.53 x $(f_c')^{1/2}$ x b x d	
	=	0.75 x 0.53 x $(210)^{1/2}$ x 100 x 15.3	kg
	=	8800	kg
	=	8.8	Ton

d) Structural Component Capacity (Footing Wall)

i. Moment Capacity



f_y = 2800 Kg/cm^2

f_c' = 210 Kg/cm^2

A_{s1} = 3.1 cm^2

A_{s2} = 3.1 cm^2

b = 100 cm

d = 15.3 cm

cm

a = A_{s2} x f_y / [0,85 x f_c' x b]

= 0.48 cm

ϕM_n = ϕ A_{s2} f_y [d – a/2]

= 117648 kg-cm

= 1.2 Ton-m

ii. Shear Capacity

f_y = 2800 kg/cm^2

A_v = 0.7 x 2

= 1.4 cm^2

f_c' = 210 kg/cm^2

Width (b) = 22.8 cm

depth (d) = 100 cm

Φ V_c = 0.75 x 0.53 x $(f_c')^{1/2}$ x b x d

= 0.75 x 0.53 x $(210)^{1/2}$ x 22.8 x 100 kg

= 13133 kg

= 13 Ton

ΦV_s	=	$0.75 \times A_v \times f_y \times d / s$	
	=	0.75 x 1.4 x 2800 x 100 / 30.5	
	=	9639	kg
	=	9.6	Ton
$\Phi V_c + \Phi V_s$	=	13 + 9.6	
	=	22	Ton

iii. Compressive Load Capacity

f_y	=	2800	kg/cm^2
f_c’	=	210	kg/cm^2
A_s	=	6.2	cm^2
Width (b)	=	100	cm
Thickness (th)	=	22.8	cm
A_c	=	2280	cm^2
P	=	$0.8 \times 0.7 (A_c \times 0.85 \times fc' + As\ fy)$	
	=	237630	kg
	=	237	Ton

iv. Tensile Load Capacity

f_y	=	2800	kg/cm^2
A_s	=	6.2	cm^2/m
T	=	$0.9 \times A_s \times f_y$	kg
	=	15624	kg
	=	15.6	Ton

APPENDIX C

SEISMIC ZONING OF PAKISTAN