

EXPERIMENTAL INVESTIGATION ON BEHAVIOR OF JOINTS IN REINFORCED EPS CORE PANEL CONSTRUCTION SYSTEM

A thesis submitted in the partial fulfillment of the requirements for the award of the degree of

**MASTER OF ENGINEERING
IN
CIVIL ENGINEERING (STRUCTURES)**

**BY
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UNDER THE SUPERVISION OF

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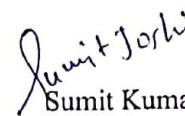
**DEPARTMENT OF CIVIL ENGINEERING
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JUNE 2019

DECLARATION

I, Sumit Kumar Joshi, hereby declare that the work which is presented in this thesis entitled **“EXPERIMENTAL INVESTIGATION ON BEHAVIOR OF JOINTS IN REINFORCED EPS CORE PANEL CONSTRUCTION SYSTEM”** in partial fulfilment of requirements for the award of the **Master of Engineering in Structural Engineering**, submitted in the Civil Engineering Department, Thapar Institute of Engineering and Technology, Patiala, is an authentic record of the work carried out by me under the guidance of **Dr. Naveen Kwatra, Professor**, Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala. The matter presented in this has not been submitted either in part or full to any other university or institute for the award of any other degree.

Dated: 15/7/2019


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CERTIFICATE

This is to certify that the above declaration made by the student concerned is correct to the best of my knowledge and belief.


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ACKNOWLEDGMENT

In the accomplishment of this dissertation successfully, many people have bestowed upon me their blessings and the heart pledged support, this time I am utilizing to thank all the people who have been concerned with this dissertation. It would not be fair on my part if I don't say a word of thanks to all those whose sincere advice made this period an educative, enlightening, pleasurable and memorable one.

Primarily, I would like to express my special thanks of gratitude to **DR. NAVEEN KWATRA, Professor**, Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala, for providing his valuable guidance and support that helped me patch this project report and make it a full proof success. His suggestions and instructions have served as the major contributor to the completion of the dissertation.

I would like to extend my deep gratitude to **Dr. A. B. Danie Roy, Assistant Professor**, Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala, for his gracious efforts and keen pursuits which have remained as a valuable asset for the successful completion of the dissertation. His timely advice, meticulous scrutiny and scholarly advice have helped me to a great extent to accomplish this task.

I would like to extend my sincere gratitude to **Dr. Prem Pal Bansal, Associate Professor** and Head of Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala for his kind cooperation and encouragement which has always been a boost.

Last but not least, I would like to thank my family and friends for their constant encouragement during the entire period of work.

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ABSTRACT

Reinforced EPS panel construction system has become a very efficient, safe and economic construction system in the modern times. These panels are being used both as load bearing and non-load bearing elements. With its increasing popularity it has caught various researcher's attention across the world in many areas. The research on joints or connections in the EPS panel construction is one of the areas which is needed to be studied. A very minor amount of work has been done in the area of joints in EPS panel construction system. This study is completely based on determining the strength, stiffness and ductility of the joints in EPS construction against a loading which develops a pure moment in the joint and with different configurations for different types of connections. A comparison between the connections tested in lab and wall-floor panel connection in buildings comprising EPS core panel system is drawn in the end, to check the efficacy of the different joints tested. The comparison yielded good results as the moment capacity values of all connection specimens with additional configurations are found to be near about the moment capacity of a connection considering a full scale EPS slab panel, which justify the significance of this study.

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CHAPTER I

INTRODUCTION

1.1 GENERAL

In the present study, a thorough investigation of the behavior of the wall-slab panel joints or connections is carried out to simulate the behavior of joints between the wall and slab panels present in the expanded polystyrene (EPS) core panel building system.

EPS core panel building system is a part of concrete sandwich panel (CSP) construction system in which expanded polystyrene is used as the core material which provides the insulation to the system and also increases the stiffness factor. Collins (1954) describes a concrete sandwich panel as the panel consisting of two layers of comparatively thin, high-density components bonded to a comparatively dense, low-density material core. The core's role is to separate the two layers of materials of high density, thus providing a greater stiffness factor. However, owing to the core's reduced heat conductivity material, the CSP possesses greater heat insulation. Figure 1.1 shows a typical cross-section of a sandwich panel of reinforced concrete.

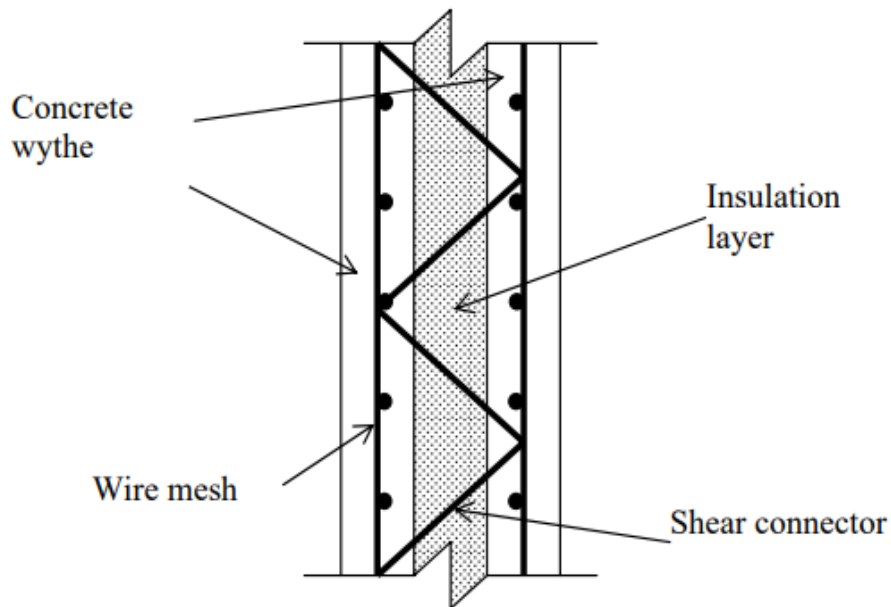


Figure 1.1 Typical cross-section of a reinforced sandwich panel

Seeber et al. (1997) in PCI committee report has presented various type of concrete sandwich panels. Expanded polystyrene and polyurethane, with elevated thermal insulation, low strength and low density, are frequently used as layers of insulation. It is possible to divide the concrete wythe into structural or non-structural wythe. However, shear connectors are mainly categorized as concentrated shear connectors, continuous shear connectors and non-shear connectors.

Einea et al. (1991) give the classification of CSPs based on the structural behavior of the two concrete wythes separated by the inner foam core. Generally, CSPs are classified into 3 categories as

Fully Composite Panels - Sandwich boards whose wythes are so connected that both wythes withstand applying bending loads as an united section. For such case, to obtain the bending stress distribution on the cross-section of the panel as shown in Figure 1.2 (a), the connector must transfer the required longitudinal shear.

Partially Composite Panels - Sandwich panels in which connectors can pass from zero to 100% of the longitudinal shear needed for a completely composite panel. Figure 1.2(b) shows the distribution of bending pressure in this form of a panel.

Non-Composite Panels - Sandwich panels that connect the two concrete wyths to connectors that do not have the ability to transfer longitudinal shear. If the two concrete wyths are of equal rigidity and strength, each wyth resists 50 percent of the load and the distribution of bending stress is as shown in Figure 1.2(c). However, in most non-composite panel systems, the applied loads are resisted by only one of the two wythes, in which cases the bending stress distribution is as shown in Figure 1.2 (d).

Out of all the insulation materials used in concrete sandwich panel construction, EPS has been found to be the most effective insulation material as the thermal resistance of the EPS has been found to remain constant with time compared to other insulation materials used in the sandwich panels (Strzepek, 1990).

Expanded Polystyrene (EPS) core panel construction system is up-to-date, effective, safe and cost-effective construction system for the building construction. These panels can be used as both load-bearing and non-load-bearing bearing components. The EPS key panel is a 3-dimensional panel made of a 3D welded wire space frame with the polystyrene core insulation. After positioning it

in place, the panel shotcrete on both sides. As shown in Figure 1.3, the EPS boards consist of a 3D welded wire space frame using a truss idea for stress transfer and stiffness.

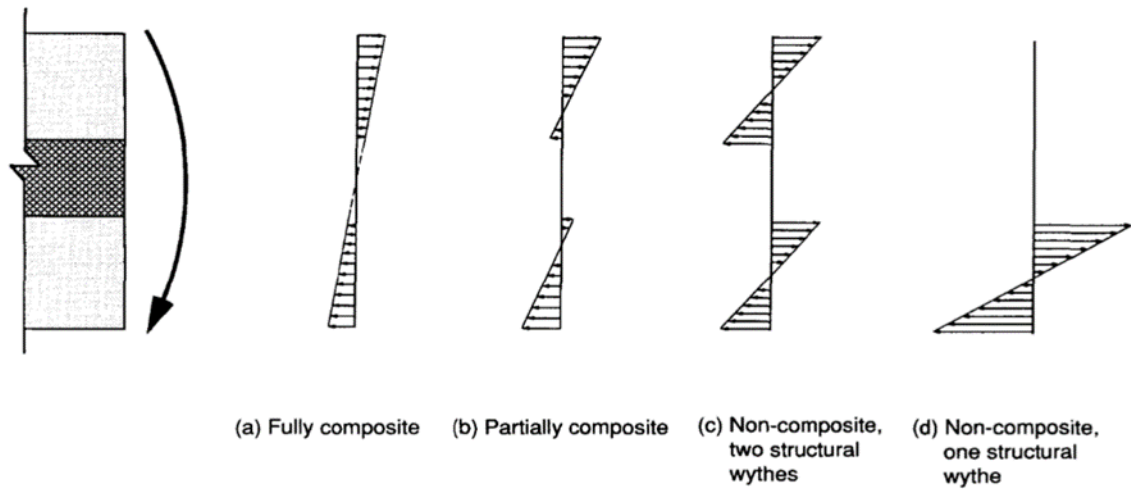


Figure 1.2: Stress distribution diagrams in CSP systems due to pure bending (**Einea et al.1991**)

The welded reinforcing mesh of high strength wire, diagonal wire and self-finishing off extended polystyrene unvarnished concrete are included in an EPS panel, manufactured in the factory and shotcrete is applied to the panel finished on the construction site, providing the bearing capacity of the structure. The following elements are present in the EPS panel after shotcreting:-

- i. The inner and outer layer of shotcrete.
- ii. The welded high resistance wire mesh reinforcement.
- iii. The core of extended sheet of polystyrene.
- iv. Stainless or galvanized wire (Diagonal wire).

The inner strut cables stretch separately between each layer through the core of the panel. In the necessary spacing, these galvanized strut cables are continually welded so that they shape into a triangulated truss system with the welded wire fabric, which significantly improves the panel strength.

EPS panel is a diverse building component designed for floors, walls, partitions, roof and stairs. Figure 1.3 shows the reinforcement mesh welded by the EPS panel. In general, the standard EPS panel is created with a width of 1200 m, a length of 3000 mm and an general thickness of 80-230 mm. The panels are mounted at the site using a minimum cement & sand shotcreting thickness of

30 mm applied under pressure at 1:4. The shotcreting coat comprises the EPS Core with a welded wire mesh of core steel.

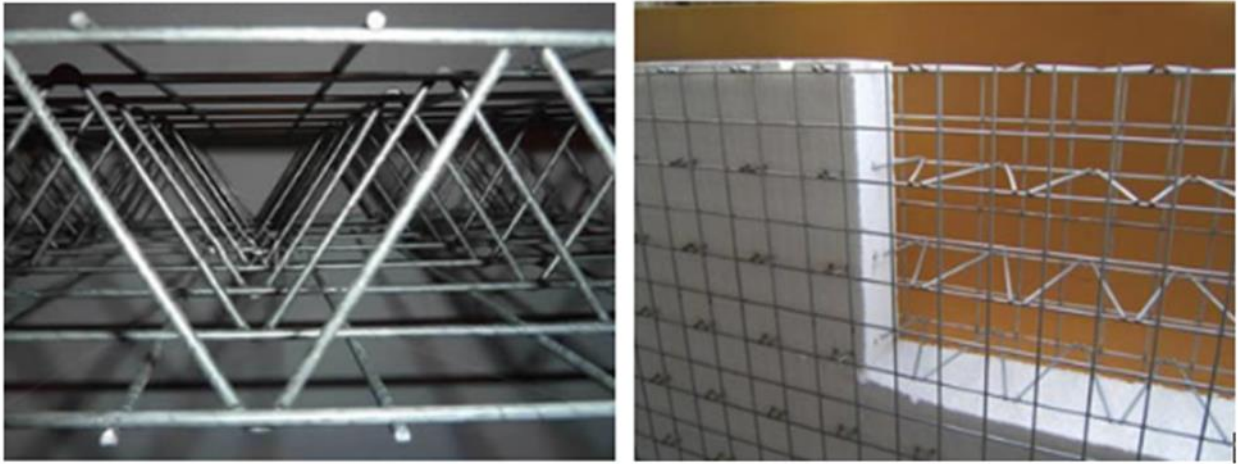


Figure 1.3 Welded reinforcing mesh of EPS panels

Some of the advantages of the EPS Core panel systems are as follows:

- i. Lower building costs, building periods, cost of transportation etc.
- ii. The use of this scheme ensures high concentrations of thermal insulation, sound insulation, sanitary and fire safety.
- iii. EPS 3-D panels do not permit additional costs for buildings to be erected in areas with moving soil, in particular heaving, subsidence, frozen soil and remote areas.
- iv. Provides high strength and resilience. The polystyrene used is practically inactive and does not absorb moisture, is resilient and resistant to decline.

Some of the Restrictions of the Reinforced EPS Panel System:

- i. Unless built by a professional engineer, EPS panel building system may only be used in the building of foundation walls assisting 4 stories or less.
- ii. Concrete must be implemented by either the American Concrete Institute's "shotcrete dry" or "shotcrete wet" method as per ACI 506 R-85, "Guide to Shotcrete."

Eco-friendly and aesthetically attractive is the EPS Core panel system. It can be built rapidly leading in time and money reserving in the building. The technology has been used effectively with the participation of various organizations in many African as well as European nations.

There are now two kinds of building techniques, the conservative cast-in-situ method, and the precast method. The structural elements are cast at the final position in the structure in the

conventional cast-in-situ method; whereas the structural elements are cast separately at other locations in the prefabricated method before being manufactured and installed at the desired position.

In this study, the cast-in-situ method of construction of connections has been considered to compare with the wall-slab connections fabricated in the laboratory.

1.2 Type of Connections in EPS core panel construction system

Only a panel-panel joint, such as wall-floor, wall-roof and wall-wall, etc., is available in the EPS key panel scheme. Horizontal joints and vertical connections can be classified as panel-panel joints. Horizontal connections are the wall-floor and wall-roof connections. However, the vertical connections are the connections between the wall panels in the same floor. As shown in Figure 1.4, the wall-floor joints can further be split into L- joints, T- joints and X- joints.

1.3 Connection Methods for Structural Elements

Basically, structural elements can be connected by two methods:

- i. Cast-in-situ methods
- ii. Mechanical means (bolting, welding, mechanical interlocking, etc.).

Cast-in-situ techniques give an easy connection with quick speed but complex connections are offered by mechanical processes. The cast-in-situ technique of links is recommended by most structure developers as the connections are more effective and likely more cost effective than the mechanical connections.

A cast-in-situ joint is built using normal strengthened concrete principles. However, prior study findings show that the connections do not act as anticipated. Structural distress may lead from a lack of comprehension of the behavior of the joints under the in-service design loads.

Engstrom (2008) explained the force transfer mechanisms in structural connections for the precast concrete buildings. Although, the study is about the precast concrete buildings but the connections in the EPS core panel systems behave almost similar to the precast concrete building connections in terms of force transfer. As per this study, the connection features can be classified by the sort of action to be resisted i.e. shear, tension, compression, flexure, torsion, etc. For-

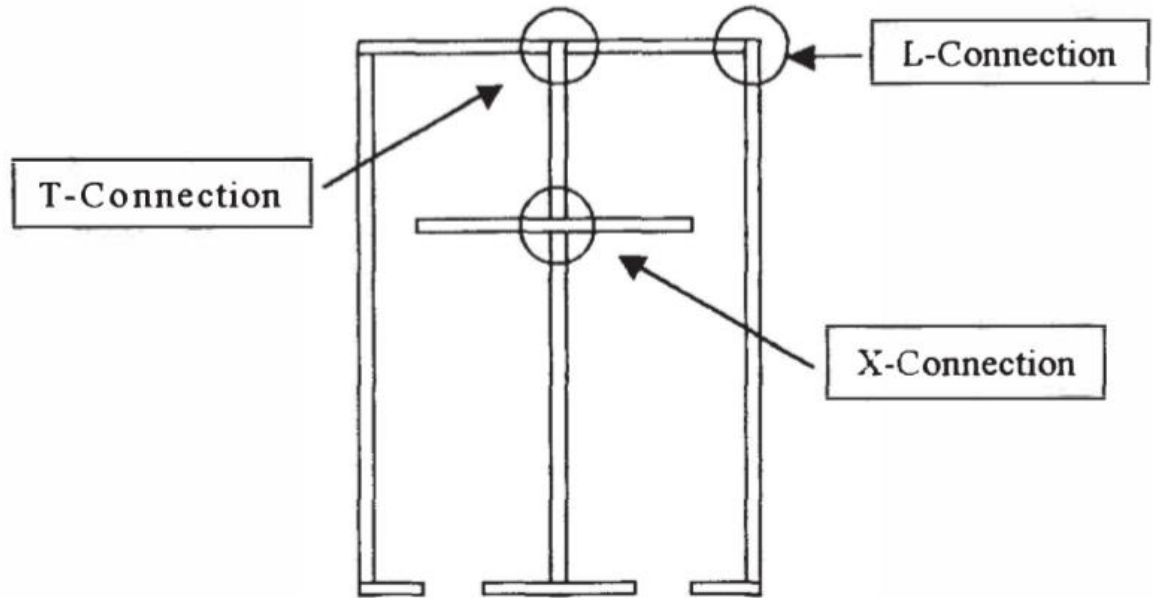


Figure 1.4: Plan of Typical Load Bearing Wall Structure (Chin et al. 2012)

-many structural connections, one of these actions dominates behavior. This dominant action sometimes classifies connections as 'shear connections', 'compression connections' etc. Very often, however, the structural joints should be able to transfer a mixture of these fundamental activities. Connections design about the capacity to transfer forces must be based on the knowledge and understanding of fundamental processes for transferring force.

In a report on reinforced EPS core panel system by building materials & technology promotion council (BMTPC) in 2015 sponsored by government of India, a complete detail regarding this type of system is explained e.g. description of the system, types of panels, production process, design considerations, installation procedure, etc. the design of wall panels and floor panels is explained with the help of some examples. The information that is missing in this report is the design of wall-floor panel joints in the EPS core panel buildings. Although a little information about the tests carried on joints and some joint or connection details has been given but the important details like the procedure of the tests, the dimensions, and properties of connections are not explained. This is a case like most of the other literature where very little guidance can be found regarding the wall-floor panel connections in buildings comprising of EPS core panel systems. This deficiency of information makes the basis of this research work i.e. to check the efficacy of the joints produced in a laboratory against the joints present in the prototype buildings.

1.4 NEED AND OBJECTIVE

1.4.1 Need

A joint or connection between two structural elements in concrete construction is a plane of weakness that does not affect the compressive strength although shear and tensile strength across the joint are markedly reduced. Since the tensile strength of concrete is zero for design purposes, the only worry is the shear strength. Joints should, therefore, be located at sections of minimum shear. In the context of EPS core panel systems, there is a number of studies available in the area of flexural behavior of EPS panels. For instance, Carbonari et al. (2012) studied the flexural behavior of the lightweight sandwich panels composed of concrete and EPS by featuring four-point bending test and a uniformly distributed load test for different experimental programs, Joseph et al. (2018) studied the flexural behavior of concrete sandwich panels under two distinct loading conditions i.e. four-point bending and punching. But when it comes to connections or joints in the EPS core panel system, almost negligible amount of research can be found out e.g. one of rare research in this area is done by Gara et al. (2012) who studied the two different wall-floor nodes with separate connection details to investigate the degree of rotational restraint of different realistic nodes.

In the 1970s, ACI Committee 408 indicated the requirement for research on connections in traditional concrete construction, as it was found that the reinforcing steel bar did not develop expected tensile stress even when adequate anchorage length was delivered for the reinforcing steel bar. Furthermore, it was not possible to prevent the tension cracks at the corner, although a complex reinforcement detail was introduced. As a consequence, a comprehensive inquiry was carried out by Mayfield et al (1971) to study the capabilities of the joint opening and closing moment capacities. Jackson (1995) and Abdul Wahab (1998) examined the strength features of joints acquired through the intersection of U-bars. However, very little literature is accessible on the moment or shear capacity of cast-in-situ connections in concrete sandwich panel scheme between wall and slab panels or, as in this case, EPS panel building system. Construction using reinforced EPS panel system leads to its own complexities as considered in the design of joints. Therefore, a study focusing on the behavior of joints is always welcomed.

1.4.2 Objective

In the EPS core panel construction system, joints are cast-in-situ between wall-floors, wall-roofs, wall-wall panels, etc. This study explores some of the issues in joints construction as discussed above in the EPS core panel construction system by considering the wall-slab panel joints. The main objective of the present investigation is given below:

- To evaluate the performance of joints in terms of their strength, stiffness and ductility.
- To study the failure pattern of L and T joints for different configurations.
- To draw a comparison of moment capacity of the wall-floor joints or connections.

1.5 ORGANISATION SCHEME

The rest of the research is divided into 6 sections. **Chapter II** describes about the research works carried out on the concrete sandwich panels, their insulation properties including thermal and sound insulation, their flexural behavior, joints in the sandwich panel construction system etc. . **Chapter III** deals with the experimental study carried during the entire work, the practical parameters of each material and machinery that have been used in the casting and testing of the connections. The various tests that were conducted on the materials used in this research are also given. **Chapter IV** describes the results and discussion of the various tests performed on the different type of joints to check its strength, stiffness, ductility, etc. Also an analysis involving a comparison between the moment capacities of the connections tested in the lab and wall-floor connections present in EPS core panel building to justify the significance of the research. **Chapter V** describes the conclusion of the work.

CHAPTER II

LITERATURE REVIEW

A literature review helps to recognize the concepts relating to the research topic and the potential relationship between them. It also helps in the proper analysis of data. Moreover, it gives broader vision and even the secondary data to the research.

2.1 GENERAL

Expanded polystyrene (EPS) is an innovative building material that imparts to the design and structural superiority of many building projects. EPS has been renowned as a major insulation material for many past years in the concrete sandwich panel construction, although over the past few years, new applications have swiftly developed. Now, EPS is an influential design element and a perfect choice for green building design, offering real environmental advantages that can get the best out of energy efficiency, providing better indoor environmental quality and increasing durability. This type of construction involves different constructional joints like wall-slab panel joints, wall-wall panel joints, slab-slab panel joints, etc. these joints or connections may be cast-in-situ or are provided by mechanical means. This study is about cast-in-situ wall-slab panel connections. No sufficient experimental results were found in the literature regarding the behavior of joints in this type of construction. The lack of studies on the subject has also motivated this study to evaluate the structural efficiency of joints in this type of construction which involve slab and wall panels.

2.1.1 Concrete sandwich panels (CSP)

Henin et al. (2014) in CSPs for thermally effective floor/roof applications developed a concrete sandwich floor panel that contains truss-shaped glass fiber-reinforced polymer ties, identified as NU ties used in the form of shear connectors because of their structural and thermal efficacy. This research presented the design, detailing, fabrication and testing of three specimens which represented two unlike panel designs. One specimen had solid concrete ends (A1) and other two specimens were fully insulated (B1 & B2) and both types were designed for different values of total superimposed loads depending upon their application in storage rooms, stages, etc. for panel A and corridors, computer rooms for panel B.

Different cracks were identified at both ends on the top surface of the panel in specimen A1, which is discovered because of the restraint induced by the panel's compact end blocks. In B1, the failure happened to owe to horizontal shear, which led some ties to be withdrawn from the concrete

bottom with. To make all the ties participate in the horizontal shear resistance, B2 was loaded with one concentrated load at the mid-span. The results showed that the ultimate strength of A1 and B2(tested under three-point loading) surpassed the theoretical capacity of specimen B1(tested under four-point loading) was lower than predicted because the shear connectors placed between two applied loads did not make a significant contribution to horizontal shear resistance, which resulted in early horizontal shear failure.

Einea et al. (1991) studied the precast concrete sandwich panels to describe the basic constituents of sandwich panels and discussed some of the existing research in this area. This study provided all the information about the concrete sandwich panels e.g. classification of these panels on the basis of their degree of composite behavior, type of insulation material used as core, the need of using insulation material, the connector used across the insulation material, type of connectors, etc. From some of the available research results, it was found that the thermal resistance might be significantly reduced due to the concrete and steel penetrations through the insulation. It was also reported that there might be a production of significant stresses and deformations because of temperature and volume change effects.

Morcous et al. (2011) designed and carried out tests on tornado resistant precast concrete sandwich panels with GFRP ties. Wall panels of CSPs for tornado-resistant buildings is an emerging application. An introduction of the use of large diameter GFRP ties in 24 inch thick CSPs to resist the considerably high wind pressure of tornado zones is made by this research. It experimentally investigated the flexural capacity, shear capacity and deflection of tornado-resistant CSPs by using large diameter GFRP ties. This paper concluded that the flexural design of tested CSPs is satisfactory for withstanding the tornado wind pressure calculated based on FEMA361. The GFRP ties used in the tested tornado-resistant sandwich panels are found to be adequate in number, size, distribution for creating resistance to horizontal shear and attaining the flexural capability of a fully composite panel.

Tuwair et al. (2015) studied the assessment of sandwich panels with multiple foam core and ribs of polyurethane. This research evaluated three potential core alternatives for glass fiber reinforced polymer (GFRP) foam core sandwich panels. Three distinct polyurethane foam configurations were taken for the inner core. Type 1 was produced of polyurethane foam of high density with a

weight density of 96 kg / m³. The Type 2 core consists of thin, interconnecting, glass fiber/resin webs creating an FRP grid consisting of a 32 kg / m³ low-density polyurethane foam. The core of Type 3 consisted of trapezoidal, low-density, polyurethane foam and three-ply web layers. Several kinds of experimental tiny scale research have been performed.

In the restricted area under a centered load, Type 3 was discovered to be less susceptible to stress impacts compared to the other two kinds. There were also big deflections of type 1&2 connected with important core shear deformation. Type 3 decreased the core facing the de-bonding trend found in the standard building of sandwich panels.

2.1.2 Insulation properties of sandwich panels

Zhai et al. (2018) studied the thermal performance of concrete sandwich walls with a new hybrid connector. A study with a novel hybrid connector based on experimental and numerical outcomes of precast concrete sandwich (PCS) wall heat responses. To enhance the thermal performance of the PCS wall, the hybrid connector was covered with nylon. Four PCS walls were manufactured to experimentally assess the impacts on the heat performance of precast concrete sandwich walls of the hybrid connector, insulation layer thickness, and insulation material. Two different core materials i.e. polyurethane (PU) and expanded polystyrene (EPS) were used in different specimens. The findings showed that the thermal transmission of the PCS wall with the hybrid connector increased by about 32.4 percent compared to the PCS wall without the hybrid connector. In addition, by increasing the polyurethane insulation layer thickness from 80mm to 100mm, the same was reduced by 33.9%. PU was found to be the better insulation material compared to EPS as it showed better thermal performance. It was found that the thermal transmission of the precast concrete wall was reduced most effectively by reducing the conductivities of the materials, especially for polyurethane, concrete and rebar located at the center.

Kawasaki and Kawai (2006) studied the wood-based sandwich panel for thermal insulation properties to be used as structural walls and floors with enhanced insulation properties. Thermal insulation and heat possessing properties of dense plywood faced sandwich panels with low-density fiber board (plywood faced sandwich, PSW) produced as wood-based structural insulation material for walls and floors were studied. The properties investigated in the research were thermal conductivity (λ), thermal diffusivity (D) and thermal resistance (R). The findings for PSW panels have been contrasted with those for commercial wood boards, strong wood, and business

insulators. It was discovered that the findings of this study were quite satisfactory. It summarizes the overall trend as follows.

Insulators > PSW > wood > boards are the order for better thermal insulation, and PSW = boards > wood > insulators are the order for characteristics to maintain warmth and for better heat resistance PSW > insulators > wood > boards.

Yossef et al. (2017) developed an embedded insulated concrete sandwich (PVICS) multipurpose photovoltaic (PV) panel that could operate as an inactive energy arrangement through insulation layer and a mobile energy arrangement by collecting solar energy using thin-film solar cells.

In a creative co-curing scheme featured in the panel, where to provide the formwork for the panel, solar cells, FRP shell and polymer concrete were manufactured together. The performance of solar cells was studied through the current density vs voltage curves at different strains. The solar cells were found to work properly under the service load. The performance of the full-scale PVICS panel was evaluated under a three-point bending test. The panel achieved around 82% DCA before the loss of bond between FRP and concrete. After this cracking point, the panel was acting as non-composite or low composite action panel.

Wen-chao and Chung-fai (1998) studied the enhancement of sound insulation using concrete sandwich panels for honeycomb. A forecast model was developed to explain the transmission loss of the honeycomb panels based on the understanding of their structural modal parameters. A number of sample samples with aluminum sheet and reinforced concrete fiber sheets as used on panels were used to investigate the effect of stiffness and damping on the sandwich panel noise transmission loss.

From the outcomes, it was shown that the treatment of stiffness and damping can be designed to decrease the noise transferred through a light weight panel, such as a partition on and below the resonance. Results from the experiment showed that a high-resistance epoxy attaching the sandwich panels directly to the inner side of the skin subpanels was very adequate to improve the loss of low-frequency noise transmission. The partition panels were produced of face sheets of glass fiber strengthened concrete (GRC) and paper impregnated with honeycomb core resin. The relatively excellent settlement was achieved between theoretical projections and experimental noise transmission loss measurements through low-frequency area partition panels.

2.1.3 Flexural behavior of precast concrete sandwich panels

Carbonari et al. (2012) examined the flexural behavior of concrete and EPS lightweight sandwich panels. To assess the different supporting and load conditions, three experimental programs were performed. Dissimilar designs of panels in terms of dimensions of the panel, the thickness of EPS used and inclination angle of shear connectors were used for each experimental program. The first program featured the four-point bending test with a load application 1m apart from the centerline between supports. An evenly distributed load is applied in five consecutive steps by using premixed mortar bags located over the upper surface of the slab for the program second and third program. The configuration of the third experimental program was similar to that of second with the only difference that it evaluated the flexural performance of the slabs with more efficient link between the PSP slabs and walls by means of providing additional U hooked steel bars in order to enhance the transfer of forces and moments between the slab and the wall.

Results showed that under service loads high deformability and a high degree of cracking was observed in precast sandwich panel (PSP) slabs with 90° steel connectors. For all the slabs tested their behavior was found to be little away from the ideal behavior and approaching slightly towards that of independent layers. The overall contribution of the connectors was found to be small.

Hopkins et al. (2014) studied the impact of DCA on the flexural conduct of prefabricated concrete sandwich panels. In this research, the effect of DCA on the flexure behavior of the sandwich panels was studied on the basis of bending test and F.E analysis. At the end of the test, the cracks spread through the concrete bottom wythe and move toward the compression zone of the panel in the top wythe. An unanticipated crack developed on the top concrete wythe which was most probably considered to occur due to radial tensile forces from the prestressing strand and at the ends of the panel.

Between the test results and F.E results, good correlations were observed. The F.E model was applied to study the panel under 0% and 100% DCA. It was found to be a variable of the magnitude of the load acting on the structure and had a noteworthy impact on the stiffness and flexural capacity of the panel.

Qing and Zhengxing (2017) experimentally evaluated the flexural behavior of precast concrete sandwich wall panels with polymer shear connectors reinforced with steel-glass fiber. The shear connector made of reinforced polymer material from steel–glass fiber was used. Under flexure,

twelve full size insulated concrete sandwich panels were screened to examine their pressurized flexural conduct. It was found from the outcomes that lower insulation thickness and lag angle lowered shear deformation amid the two concrete wyths and then improved composite conduct further. The reduced amount of connectors, simple GFRP W-shaped connector, and smaller diameter steel truss connector will result in low original stiffness for the sandwich panels when transforming the flexure shear. Most big diameter connectors were harmed by withdrawing of the concrete wyths devoid of achieving the ultimate strength. Therefore, to completely utilize its power, a decent diameter in the connectors was needed. The sandwich panel's flexural capacity was governed by the shear connectors ' rigidity and strength, and the panel thickness, particularly the thickness of the insulation layer, and the appropriate longitudinal reinforcement ratio.

2.1.4 Stone dust used in fine aggregates as a partial replacement of river sand

Stone dust is a waste material from crusher plants. It had the ability to be used in concrete as a partial substitution of natural river sand. Stone dust is an excellent solution to preserving the natural river sand for future generations, it also increases the quality of concrete.

Singh et al. (2015) studied the effect of stone dust on the compressive strength of concrete. An experimental study was carried out to research the workability and compressive strength of concrete created through stone dust as a partial substitute of fine aggregate in the range of (10 percent -100 percent). It was found that up to 8%-27% increased compressive strength can be achieved by replacing river sand (fine aggregates) with stone dust (10%-100%). Optimum replacement level of the fine aggregates with the stone dust was found to be 60%. However, at each replacement level, the compressive strength of concrete developed using stone dust was found higher than the referral concrete.

Kujur et al. (2014) evaluated the effect of partial replacement of natural river sand with stone dust in concrete. For concrete of M25 grade and 0.46 w/c ratio, a 30%, 40%, 50%, 60% and 70% partial replacement of river sand with stone dust had been taken in the experimental program. In the research, set of cubes and beams were cast to calculate the compressive and split tensile strength respectively.

From the results it was found that the compressive strength of concrete was more than referral concrete at a replacement of 40%. At a replacement of 50% of natural river sand with stone dust,

concrete had more split tensile strength than referral concrete, after this, it started decreasing however it was still more than referral concrete.

2.1.5 Joints in reinforced concrete sandwich panel (CSP) construction

Gara et al. (2012) studied the wall-floor nodes with different connection details to evaluate the effective degree of rotational restraint of different realistic nodes. Evaluated the impacts of the floor's wall-floor joint and bi-dimensional behavior. Two typical connections (node 1&2) were regarded for the experiments on the wall-floor node, the upper panel slab passes the vertical panel in node 1 and is disrupted in node 2. Consequently, node 1 interrupts the wall panel at ground level, whereas node 2 interrupts it continuously.

Results showed that in both instances (node 1&2) since failure included local phenomena happening on the concrete layers of wall panels, the connections resulted in the ultimate moments of bending lower than the final time of the ground panel. Before the ground top slab cracking, the stiffness of the links was discovered to be important, particularly for node 1.

Chin et al. (2002) reported the results of an experimental study to develop an appropriate cast-in-situ resistant connection between two intersecting concrete sandwich panels. Four types of cast-in-situ L-connections with varying anchorage length of extended reinforcing steel bars in each connection had been studied. The appropriateness of each joint was inspected as considered to be the cracking moment, the ultimate moment capability and ductility calculated under gradually growing time in terms of change in included angle between the intersecting panels.

The study specified the superiority of a loop type connection over other connection. The loop type connection exhibited a ductile behavior under increasing moment till failure. The looped extended bars into the joint or connection were found to enhance the ultimate moment by about 120% over the open-ended extended bars.

CHAPTER III

EXPERIMENTAL WORK

3.1 GENERAL

The experiment work of this research does not involve only testing of joint or connection specimens but also other tests of cement and fine aggregates. The main reason for performing tests on cement and aggregates is just to check whether their tests values lies within the limits prescribed in codes.

Experimental work was carried out to simulate the behavior of wall-slab panel joint specimens prepared in the lab to the same type of joints in prototype construction against the total load coming on the wall panel which would be resisted by the joint between wall-slab panels. The tests carried out check their strength, its failure pattern and the rotation of joint against the load applied.

Two different type of joints are studied in this work to produce the possible connections or joints between the wall-slab panels.

3.2 Configuration of connection or joint specimens

Three specimens of each shape i.e. L & T, with different configurations have been cast in this study. The configuration of different specimens is shown below

Table 3.1 Configuration used for different connections

S. No.	Connection type	Reinforcement	Mix
1.	L1 & T1	N	N
2.	L2 & T2	N+R	N
3.	L3 & T3	N+R	SN

The abbreviations used above are explained below

N – Normal reinforcement i.e. the welded wire mesh embedded around the insulation.

N+R – Dowel bars of diameter 8mm in addition to the normal reinforcement

SN – Special mortar mix prepared using stone dust as 30% replacement of the natural river sand.

The different properties of above materials are discussed later in this chapter.

3.3 Constituents of a wall-slab panel connection

Basic constituents used in the production of the wall-slab connection are

1. Reinforced 3D panel
2. Cement mortar mix
3. Reinforcement

3.3.1 Reinforced 3D panel

It is a factory produced panel in which a core of undulated polystyrene is enclosed with interconnected zinc coated welded wire mesh on double sided reinforcement. Two types of sandwich panels are connected with each other in a connection specimen i.e. a wall panel and a slab panel. The dimensions of the wall and slab panel are shown in the figure 3.7 & 3.8

- i. Wall Panel
- ii. Slab Panel

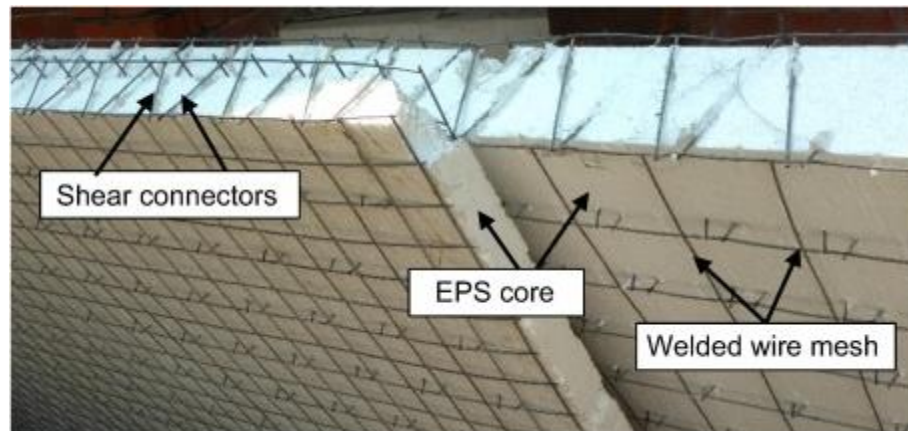


Figure 3.1 Reinforced 3D panel

The components of a concrete sandwich panel are described below:

3.3.1.1 Insulation material

Some of the mostly used types of insulations in concrete sandwich panel systems are expanded polystyrene (EPS), polyurethane, polyisocyanurate, etc. The thermal resistance of EPS is found to be almost constant with time, which makes it the most favorable insulation material to be used in PCSP construction. In this study also EPS is used as the insulation material. This material is either

created through a method of molding or extrusion. Generally a thickness of at least 1 in. (25 mm) will be used. EPS characteristics have low thermal conductivity, high strength of compression, low weight, inert. Properties of the EPS used in this study are given in table 3.2.

Table 3.2 Properties of EPS

S.No.	Property	Value
1.	Density (kg/m ³)	20
2.	Water absorption	0.1
3.	Initial elastic modulus	6
4.	Thermal conductivity (mW/cm °C)	0.33

3.3.1.2 Welded wire reinforcing mesh

Welded wire reinforcing mesh is embedded around the insulation material i.e. EPS with the help of shear connectors. The mesh used in the 3D panels is of standard size which consists of 2mm diameter bars placed at a spacing of 50mm.

3.3.1.3 Connectors

Connectors are used to bind together the welded wire metallic mesh and to maintain insulation. They can be classified into two major categories: shear connectors and non-shear connectors. Shear connectors are those that can transfer longitudinal shear from one wythe to the other as a result of flexure in the panel. Non-shear connectors can transfer only a negligible extent of longitudinal shear from one wythe to the other.

Shear connectors are preferred for most of construction applications and hence are used in this study. In this study the 2mm diameter wires have been used for shear connectors similar to that of the welded wire mesh.

3.3.2 Cement Mortar Mix

The mortar mix is used as concrete wythes in the sandwich panel. The constituents of the cement mortar mix are explained below:

3.3.2.1 Cement

Cement of grade 43 (OPC 43) was used in the mix for the specimen. A number of tests were done as per IS 8112:1989 to obtain its physical properties as listed in Table 3.3.

Table 3.3 Physical properties of the cement

Serial no.	Property	Value obtained	Standard value as per code BIS 8112:1989
1	Standard consistency	28%	-
2	Fineness as retained on 90-micron sieve	8%	Not more than 10%
3	Initial setting time	100 mins	30 mins (minimum)
4	Final setting time	280 mins	600 mins (maximum)
5	Specific gravity	3.12	3.15

3.3.2.2 Fine aggregates

Two types of fine aggregates have been used in the fabrication of wall-slab panel connections. In two connection specimens of each shape, natural river sand (zone II) has been used in the mortar mix as fine aggregates. It is denoted by aggregate I in this study. To enhance the compressive strength of the mix for the third connection specimen of both shapes, stone dust is used as a replacement of sand from fine aggregates and denoted by aggregate II in this study.

Observations and Calculations:-

a) Aggregate I (natural river sand)

Table 3.4 Gradation of aggregate I

Sieve size	Weight retained (g)	Percentage retained (%)	Percentage passing (%)	Cumulative weight retained (g)	Cumulative percentage retained (%)
4.75 mm	30	3.06	96.94	30	3.06
2.36 mm	60	6.12	90.82	90	9.18
1.18 mm	100	10.2	80.62	190	19.4
600 μ	210	21.43	59.19	400	40.82
300 μ	240	24.5	34.69	640	65.31
150 μ	300	30.6	4.08	940	95.92
Pan	40	4.08	0	980	100
	$\Sigma W=980$				$\Sigma F=333.69$

Fineness modulus = $\Sigma F/100 = 3.33$

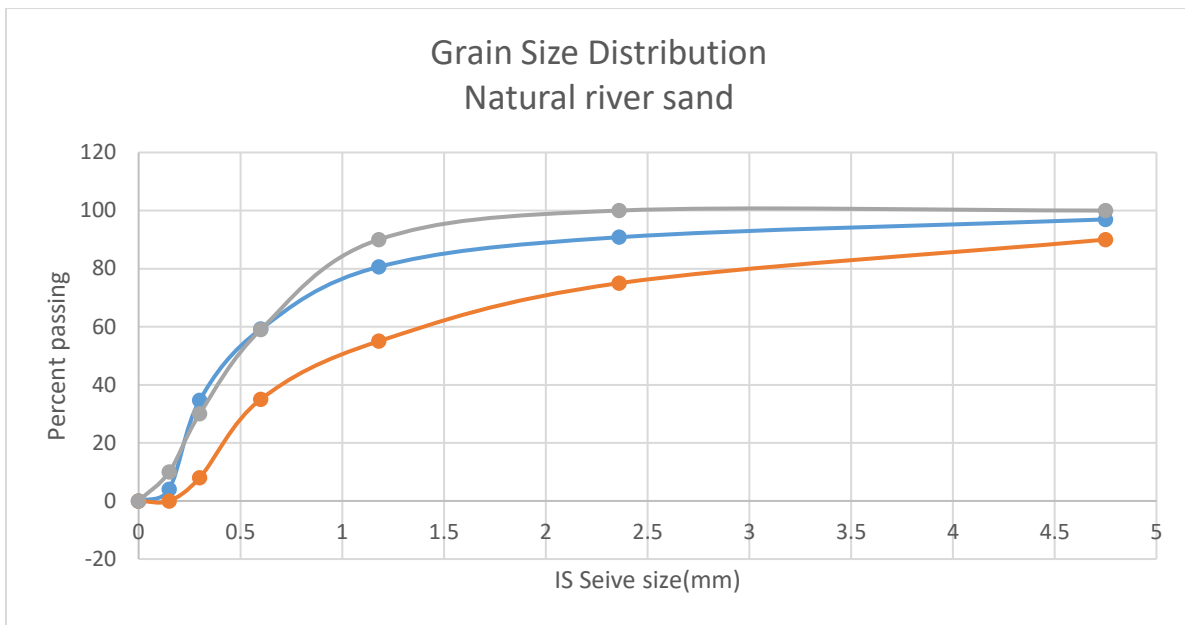


Figure 3.2 Gradation curve for the aggregate I

b) Aggregate II (sand + stone dust)

To find the optimum content of the sand and stone dust, which would comply with the requirements of grading zone 2 as given in IS: 383-1970, three types of mixes had been prepared for sieve analysis as follows

i. Sand (60%) + Stone dust (40%)

Table 3.5 Gradation of aggregate II (40% replacement)

Sieve size	Weight retained (g)	Percentage retained (%)	Percentage passing (%)	Cumulative percentage retained (%)
4.75 mm	25	2.54	97.46	2.54
2.36 mm	130	13.2	84.26	15.74
1.18 mm	140	14.2	70.06	29.94
600 μ	120	12.18	57.88	42.12
300 μ	320	32.5	25.38	74.62
150 μ	230	23.35	2.03	97.97
Pan	20	2.03	0	100
	ΣW=985			ΣF=362.93

Fineness modulus = $\Sigma F/100 = 3.63$

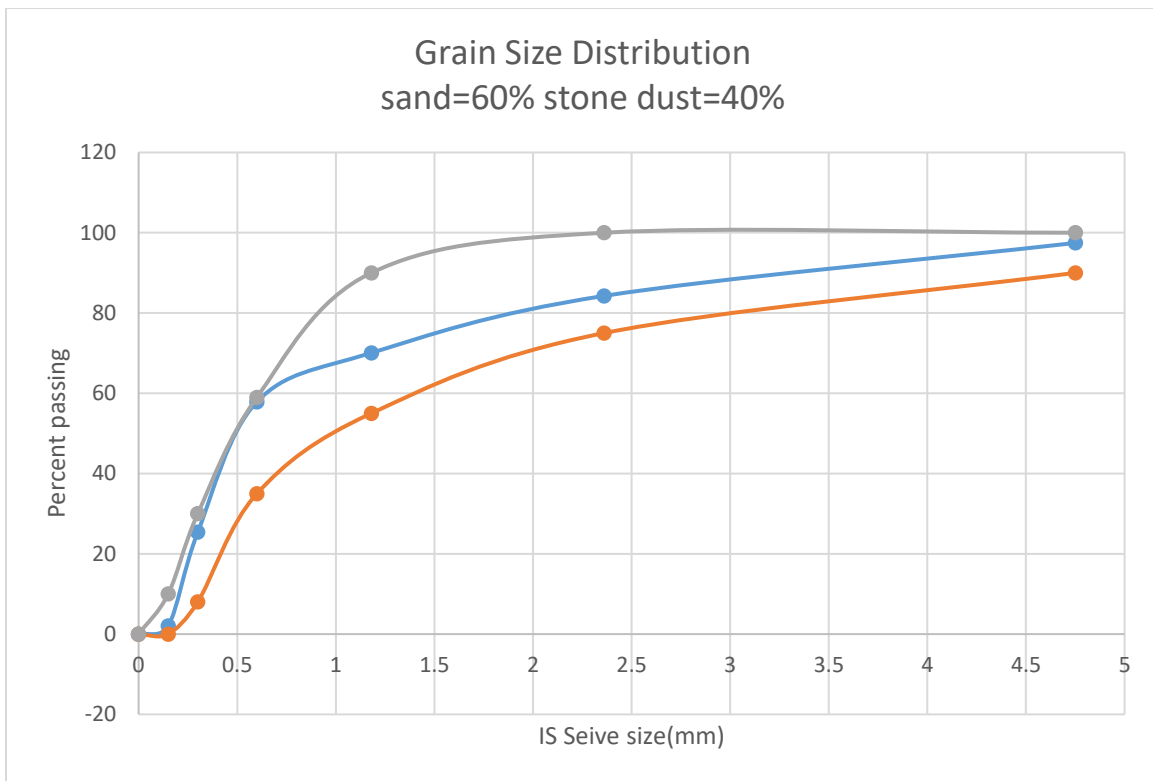


Figure 3.3 Gradation curve for aggregate 2 (sand=60% stone dust=40%)

ii. Sand (65%) + Stone dust (35%)

Table 3.6 Gradation of aggregate II (35% replacement)

Sieve size	Weight retained (g)	Percentage retained (%)	Percentage passing (%)	Cumulative percentage retained (%)
4.75 mm	25	2.55	97.45	
2.36 mm	115	11.73	85.72	2.55
1.18 mm	160	16.33	69.39	14.28
600 μ	125	12.76	56.63	43.3730.61
300 μ	280	28.57	28.06	71.94
150 μ	250	25.5	2.56	97.44
Pan	25	2.55	0	100
	ΣW=980			ΣF=360.18

$$\text{Fineness modulus} = \Sigma F / 100 = 3.6$$

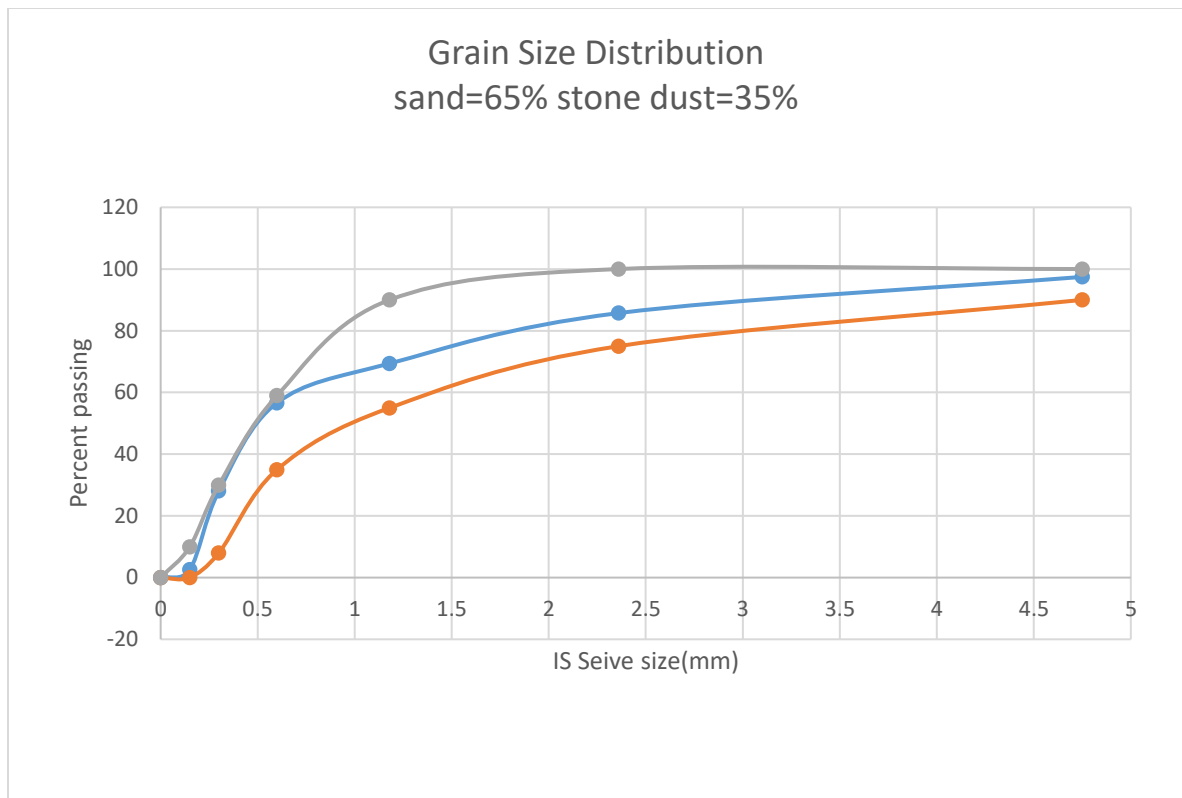


Figure 3.4 Gradation curve for aggregate II (sand=65% stone dust=35%)

iii. Sand (70%) + Stone dust (30%)

Table 3.7 Gradation of aggregate 2 (30% replacement)

Sieve size	Weight retained (g)	Percentage retained (%)	Percentage passing (%)	Cumulative percentage retained (%)
4.75 mm	23	2.34	97.66	2.34
2.36 mm	120	12.21	85.45	14.55
1.18 mm	140	14.24	71.21	28.79
600 μ	150	15.26	55.95	44.05
300 μ	390	39.67	16.28	83.72
150 μ	150	15.26	1.05	98.98
Pan	10	1.1	0	100
	ΣW=983			ΣF=372.51

$$\text{Fineness modulus} = \frac{\Sigma F}{100} = 3.7$$

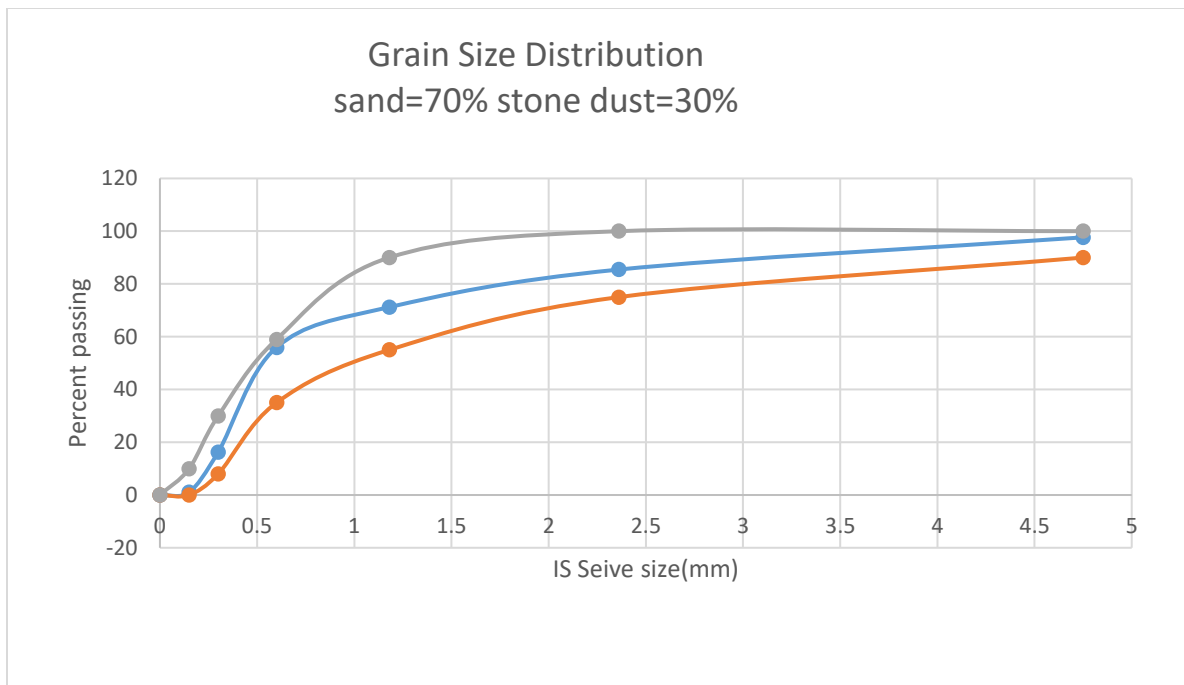


Figure 3.5 Gradation curve for aggregate 2 (sand=70% stone dust=30%)

As we can see, all three types of fine aggregates comply with the requirements of grading zone II as per IS: 383-197, but the third type seems to be most accurate. So the third type of combination i.e. sand (70%) + stone dust (30%) has been used in this study.

Table 3.8 shows the important physical properties of fine aggregates used in this study.

Table 3.8 Physical properties of fine aggregates

Serial no.	Property	Aggregate I	Aggregate II
1	Fineness modulus	3.3	3.7
2	Specific gravity	2.5	2.8
3	Water absorption	0.95%	1
4	Grading zone	Zone II	Zone II

3.3.2.3 Selection of cement mortar mix for casting of specimens

The compressive strength of cement mortar for different configurations is calculated to determine the best ratio of materials for the preparation of the mix.

Two types of mortar mixes have been used in the fabrication of specimens as

- a) Cement + aggregate I + water
- b) Cement + aggregate II + water

Table 3.9 shows the test results for compressive strength of the cement mortar cubes for different configurations.

Table 3.9 Compressive strength of cement mortar for different cement: sand ratios

S. No.	Test observations	Cement + aggregate I + water			Cement + aggregate II + water
		1:3	1:4	1:5	1:3
1.	Breaking load (kN)	111.5	99	90.5	127.75
2.	Compressive Strength(MPa)	22.3	19.8	18.1	25.55

It can be seen from the above observations, the strength of mix with aggregate II has increased by about 15% compared to the mix with aggregate I.



Figure 3.6 Compression testing of a cement mortar cube

3.3.3 Reinforcement

Generally, an angular mesh L-shaped is used as a normal reinforcement. The structural properties of this mesh are similar to the welded wire mesh used in the reinforced 3D panels i.e. 2mm diameter reinforcing wires spaced at 50mm. It is tied at the junction of wall and slab panels to

provide certain rigidity to the connection. The angular mesh is cut in such dimensions so that its length in the direction of one panel is one-third of the length of the panel. Figure 3.7 depicts the use of a typical angular L mesh used in the connection.

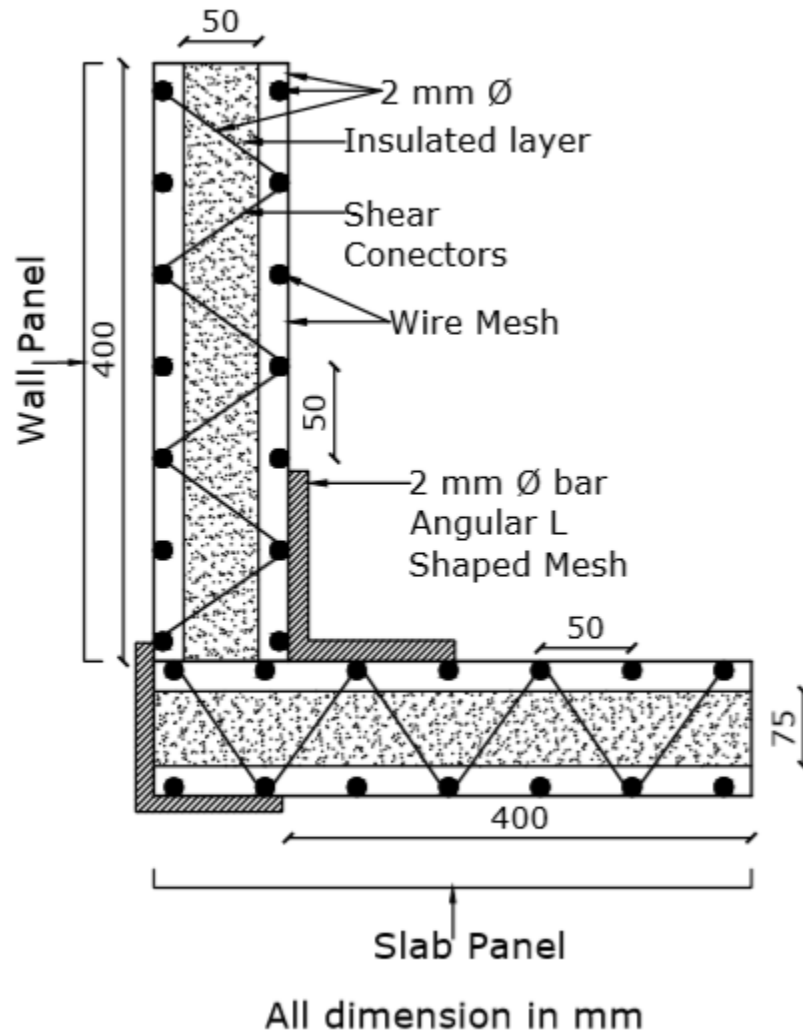


Figure 3.7 Angular L mesh in L-shaped wall-slab panel connection specimen

In addition to the angular mesh in second and third configuration of both L and T shaped joints as described in figure 3.7 and figure 3.8, steel reinforcement bars of 8mm diameter are used to provide the connection more strength and ductility. The length of these bars is kept the same as that of the angular mesh. A total no. of 4 bars are provided in one connection. In the case of L connection, 2 bars in the front and 2 in the back along with the angular mesh at equal spacing are provided. In case of T connection, 2 bars at each side of the web of the specimen at equal spacing.

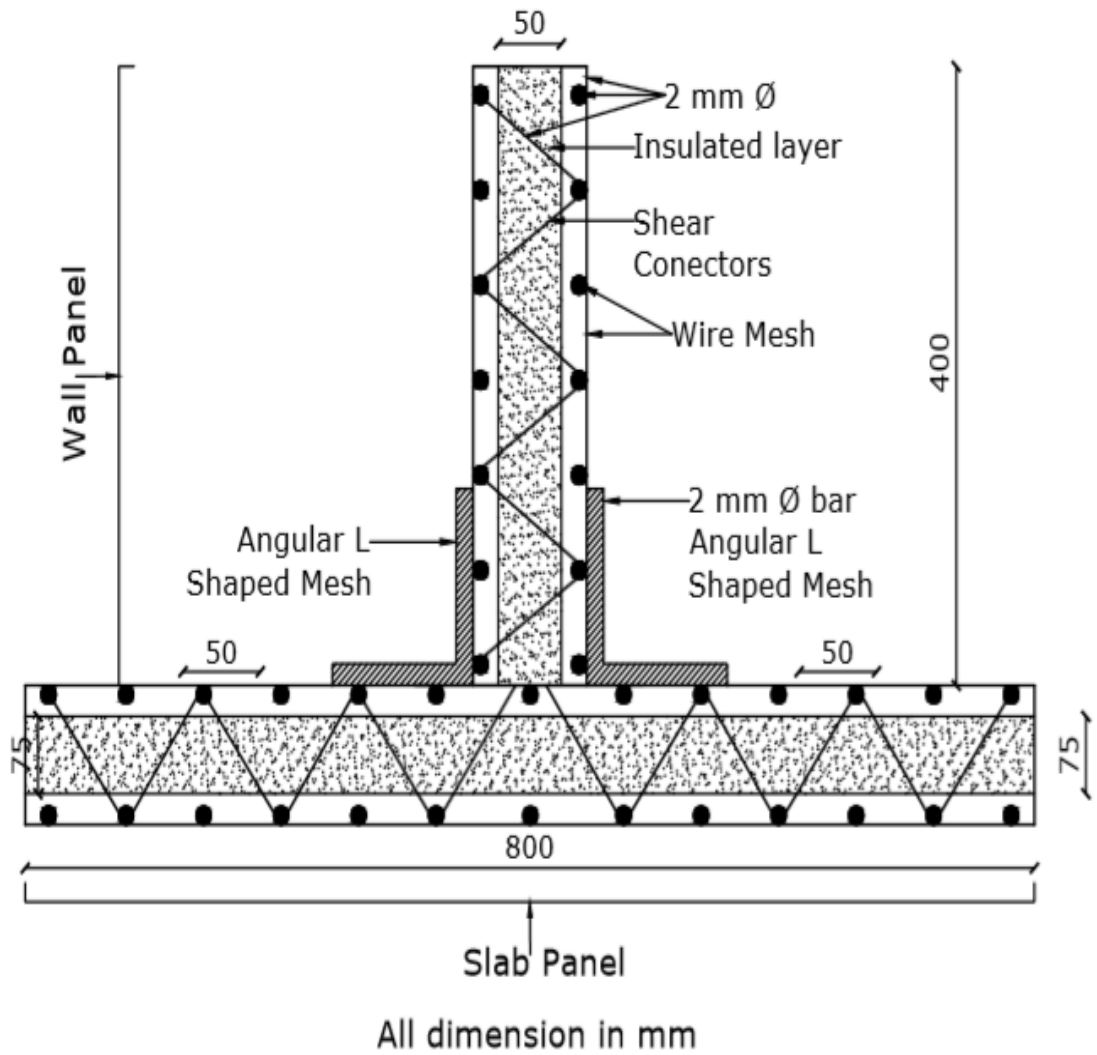


Figure 3.8 Angular L mesh in T-shaped wall-slab panel connection specimen

The flowchart of casting process of different specimens is shown in figure 3.9.

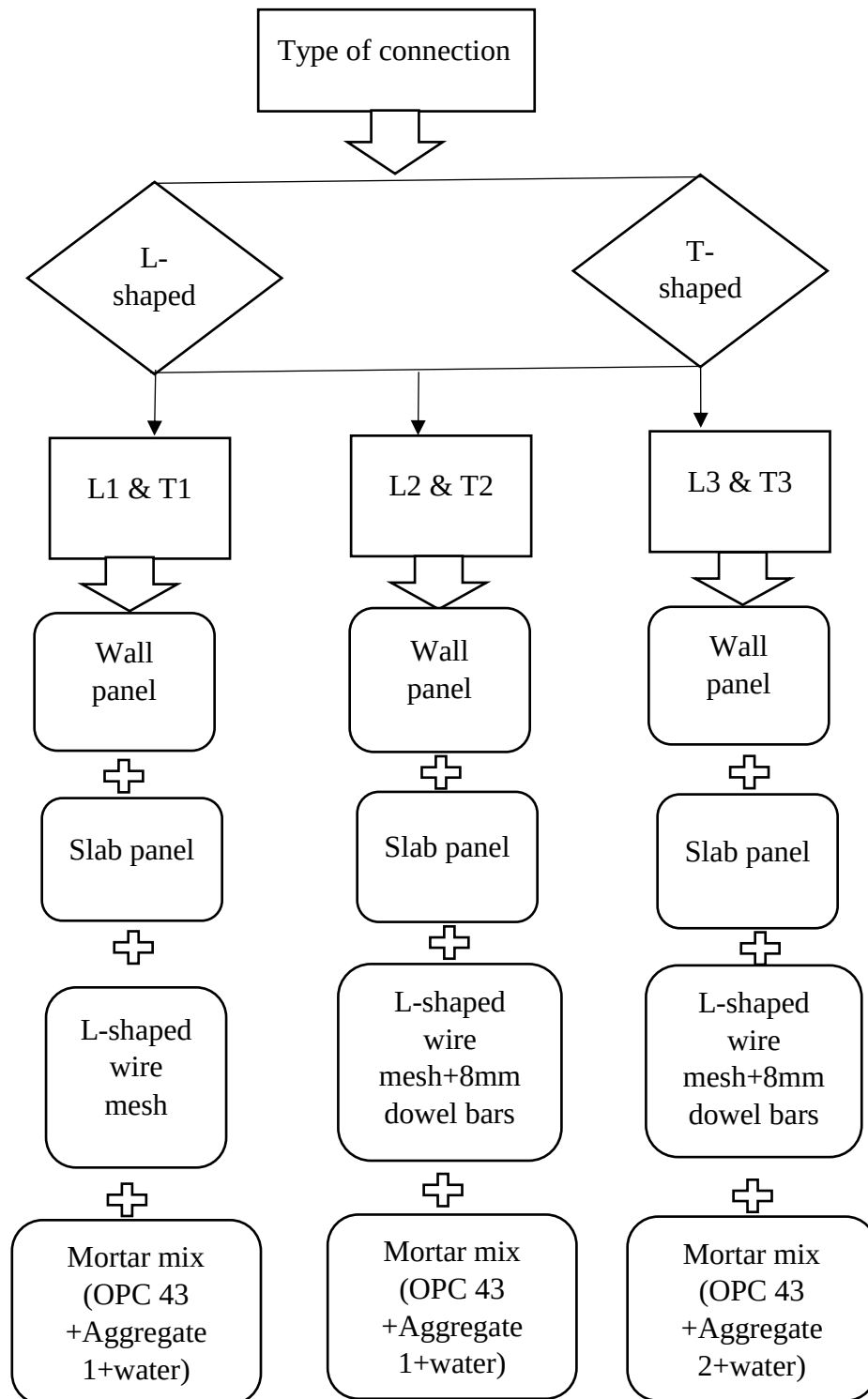


Figure 3.9 Flowchart of casting process

3.4 Testing of wall-slab panel connections

This part can be divided into three categories i.e. firstly selection of dimensions, then casting of connections and finally testing is carried out.

3.4.1 Selection of dimensions

Wall and slab panels required to cast the connections or joints were cut from the factory produced big reinforced EPS panels which are being produced to be used directly in the prototype i.e. buildings. The dimensions of the factory produced wall and slab panels are shown in figure 3.6.

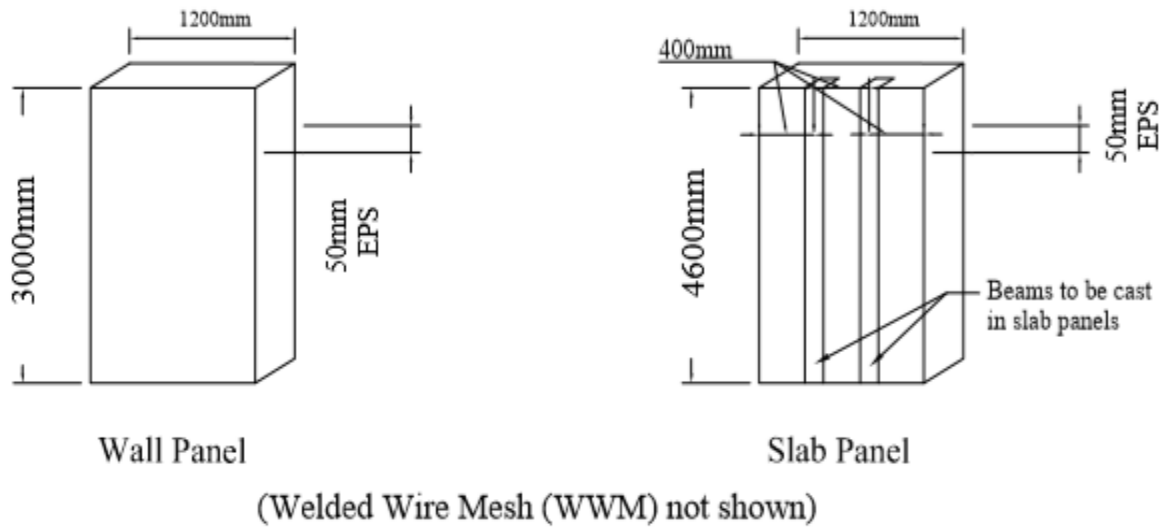


Figure 3.10 Dimensions of factory produced EPS panels

By using these factory produced EPS panels a general wall-slab panel joint in a typical building can be shown as in figure 3.10.

As we can see that the joint between the wall and slab panel is most effective up to a length of 400mm in the slab panel because of the presence of the reinforced concrete beam cast in the slab, which will further transfer the forces into the slab from the joint.

Hence a length of 400mm has been chosen for the slab as well as the wall panel in L-shaped connection and a length of 800mm for slab panel, 400mm for wall panel in T-shaped connection. A common width of 250mm has been chosen for both of the panels to make a connection.

Figure 3.11 depicts a typical L-shaped connection between a wall and a slab panel at the corner of a building. Similarly, a T-shaped connection can be depicted if the slab would be continuous over the wall.

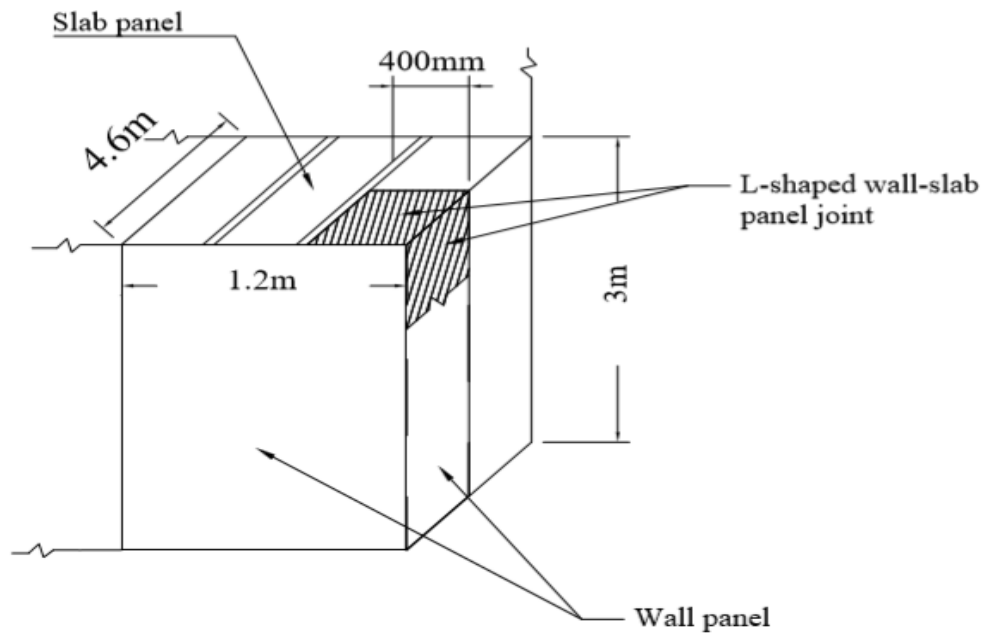


Figure 3.11 L-shaped connection in a building

3.4.2 Casting of connections

After selecting the dimensions, the reinforced EPS panels have been cut from the available factory-



Figure 3.12 Cutting of EPS panels

-produced panels in the required dimensions.

The required wall and floor/slab panel cut, are embedded with each other with the help of the L-mesh. Figure 3.13 shows a typical L shaped connection after being tied up.

The required cement mortar mix is prepared using different configurations for different connections. Concreting is done at the wall-floor panel connections with the required mortar mix as shown in figure 3.14.



Figure 3.13 Wall and Slab 3D panels tied up using L-shaped mesh



Figure 3.14 L-shaped connection after applying the first layer of mortar

The concreting is done in two layers over the panels in approximately 24 hours interval. A layer of 20mm mortar is applied firstly at the surface of the EPS. Then the second layer of 15mm mortar is applied after that, resulting in a total thickness of about 35mm on one side of panels.

These panels were then cured for 28 days. A layer of wall putty was applied to these connections for the visibility of cracks, which would be developed after testing. A finished L-shaped connection is shown in figure 3.15.



Figure 3.15 Fully finished L-shaped connection after 28 days of casting

3.4.3 Testing of connections

In the testing, loading on the wall-floor panel connections is applied by using a hydraulic jack or force transducer having a capacity of 100 kN. The experimental setup used for the testing of these wall-floor connections can be depicted as shown in figure 3.16.

The loading is applied to simulate the loading conditions which are present normally in a building and include only dead and live loads. As we know that in a prototype building, loading occurs in a vertically downwards direction on the slab or floor at a particular floor. It ultimately develops a moment in the wall-floor panel joint by causing a rotation in the joint.

To simulate these conditions, an experimental setup was prepared accordingly as shown in figure 3.16. Two LVDTs were placed on the specimen one at the top of the panel (LVDT top) and one in the middle of the panel (LVDT middle).

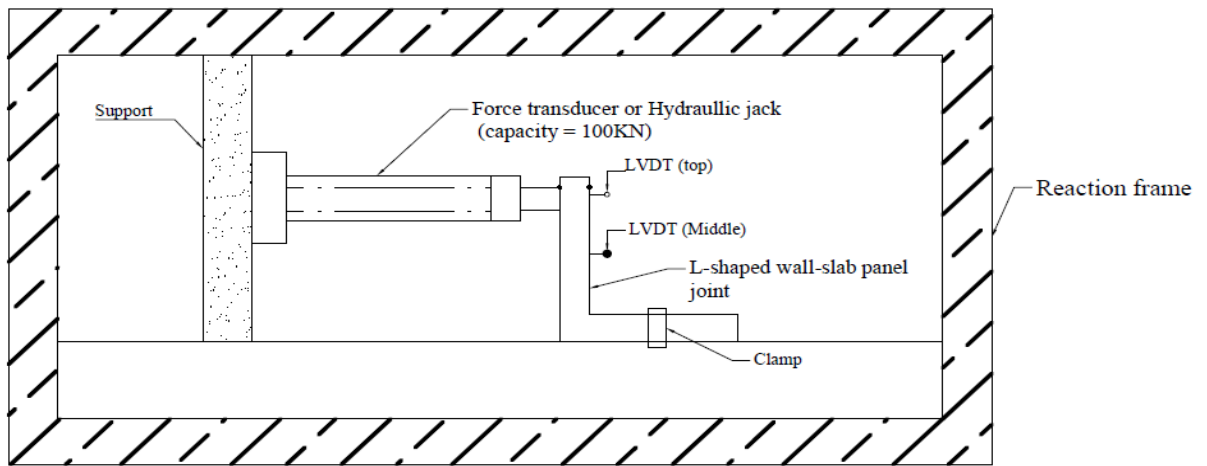


Figure 3.16 Experimental setup for testing of joints

The hydraulic jack is connected to a computer system with the help of a data logger to get the reading of the load acting on the specimen. The LVDTs are also connected to the system to get the values of deflections.



Figure 3.17 L-shaped connection before testing

CHAPTER IV

RESULTS & DISCUSSIONS

4.1 GENERAL

This chapter deals with the test results and the analysis of results that were carried out in the previous chapter. It consists of results of testing done on the connection specimens in the form of their load-deflection behavior.

It can be divided into two parts. The first and second part briefly shows the results of experimental testing performed on the six different types of wall-slab panel connection specimens with different configurations of L and T shape respectively, in the form of their load-deflection behavior and a brief discussion is presented about the cracking pattern, comparison of the load carrying capacities etc.

4.1.1 L-shaped specimens

The load-deflection behavior of L-shaped different wall-slab panel connections is shown below:

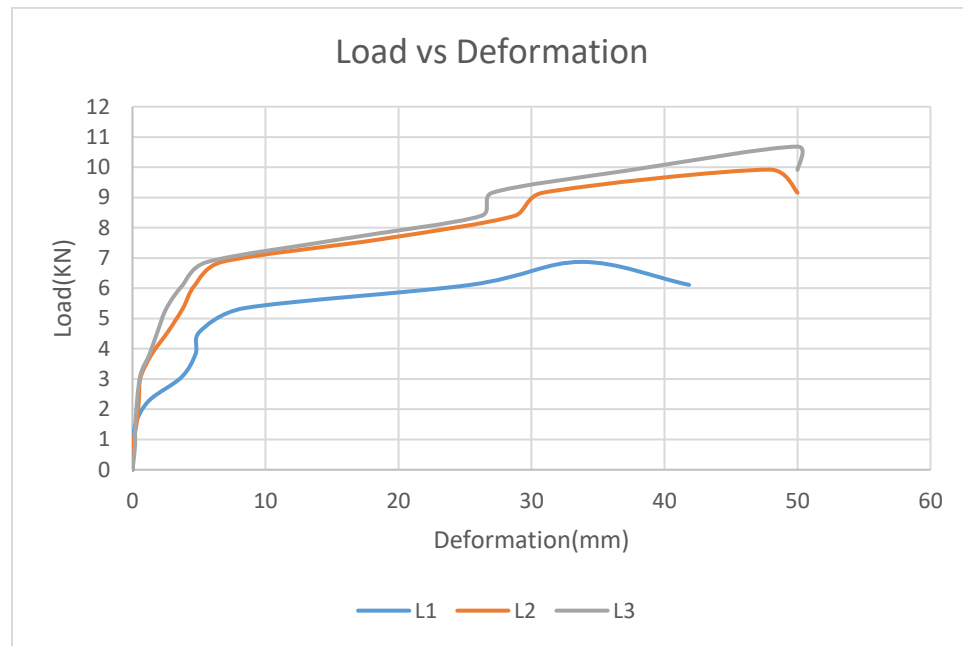


Figure 4.1 Load vs Deformation curve for the L-shaped connections

It can be seen from the above figure that for the connection type L1, it behaved elastically up to a load of 5.34kN and the first crack occurs at this load which was observed from the testing. After the point yielding of the specimen occurs and ultimate failure occurs at 6.87 kN. The cracks produced during testing in L1 type is shown in the figure 4.2, the first visible crack developed

diagonally at the joint between the wall-slab panels. Other small cracks are observed around this area which ultimately lead its failure.



Figure 4.2 Cracks developed in L1 during testing

For L2 type of connection, dowel bars of diameter 8mm are provided in this specimen in addition to the configuration of L1. It is observed that the first crack occurs at a load of 6.87kN up to which the specimen behaved elastically. It failed ultimately at a load of 9.92 kN. Figure 4.3 depicts the cracks developed in the connection type L2. Initial cracks developed near the joint at the panel and the major cracks at ultimate load are observed in the middle of the panel in the figure.



Figure 4.3 Cracks developed in L2 during testing

For connection L3 in which high strength mix has been used in addition to the configuration of the L2. As can be seen from the figure, the first crack occurs at 6.86 kN. At a load of 10.68 kN it shows an ultimate failure. In figure 4.4 the crack pattern is shown for connection type L3. The cracks observed are almost same as that of L2, most of these at the middle of the panel and between the concrete wythe and EPS layer.



Figure 4.4 Cracks developed in L3 during testing

Among the L-shaped specimens the behavior of L2 and L3 is found to be approximately similar as compared to L1. In case of L1, the major crack that caused the ultimate failure in L1 is developed at the joint diagonally. A partially composite behavior has been observed as failure of bond occurs between the EPS and concrete wythe to some extent.

In case of L2 and L3, the major cracks were observed in the middle of the panels between the outer concrete wythe and the EPS in both slab and wall panels. The connection types L2 and L3 maintained a certain strength and ductility for higher values of moment and deflection. The larger ductility and strength values of L2 and L3 can be due to presence of additional dowel bars provided in these specimens, which provide certain extra yielding to the specimens. The load carrying capacity of the connections L2 and L3 is found to be increased by 44.40% and 55.46% respectively as compared to connection type L1.

The moment vs rotation behavior of the L connections is shown below in figure 4.5.

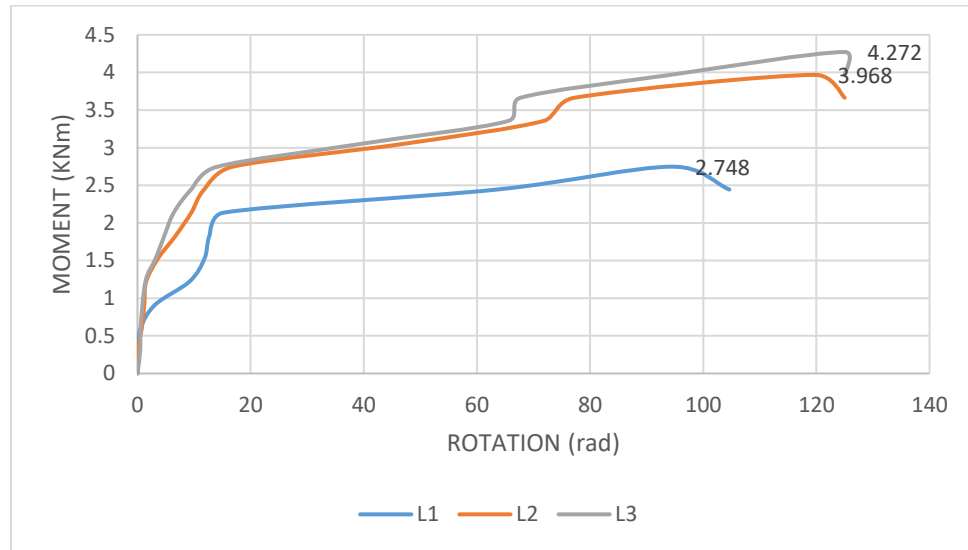


Figure 4.5 Moment vs Rotation for L-shaped connections

As can be seen from figure 4.5, the ultimate moment capacity of the connection type L2 increases by 44.39% as compared to type L1. This value further increases by about 55.4% in case of connection type L3 compared to L1.

4.1.2 T- shaped specimens

The load-deflection behavior for T-shaped type of connections is shown below. It can be seen-

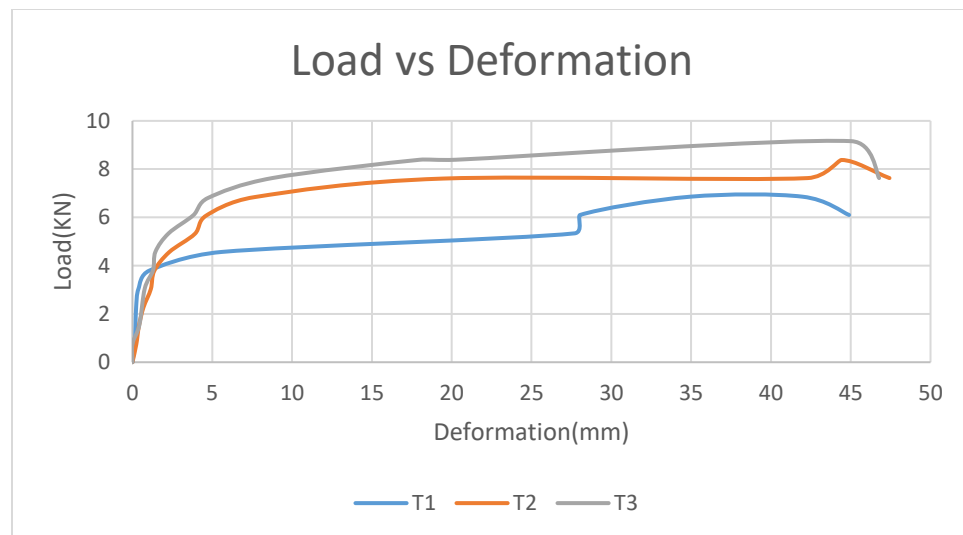


Figure 4.6 Load vs Deformation curve for the T-shaped connections

-from the figure 4.6 that connection type T1 shows an elastic behavior up to a load of 4.57kN and the first crack occurs at this load which was observed from the testing. After this point yielding of the material occurs and ultimate failure occurs at 6.86 kN. Figure 4.7 shows the cracks developed in joint T1. Failure of joint occurs prior to the panel as a crack at the interface of the wall-slab joint can be seen from the figure.



Figure 4.7 Crack pattern in T1 during testing

In connection type T2 which is provided with additional dowel bars of 8mm diameter, the first crack occurs at a load of 6.84 kN up to which the specimen behaved elastically as shown in the figure 4.8. It failed ultimately at a load of 8.38 kN.



Figure 4.8 Crack pattern in T2 during testing

Figure 4.8 shows the cracking pattern of the joint T2 in which initial cracks are observed to be developed in the panel prior to at the joint interface.

For connection T3 in which high strength mix has been used in addition to the configuration of the T2, the first crack is observed to be occurred at 7.63 kN up to which it shows an elastic behavior. After a load of 7.63 kN yielding of the material occurs up to a load of 8.39 kN at which it failed ultimately. The cracking pattern observed in T3 is found to be almost similar as that of T2. As can be seen from figure 4.9, major cracks appear on the panels rather on the on the joint.



Figure 4.9 Cracks developed in T3 during testing

As can be seen from the above results, in case of connection type T1, the failure of joint occurs prior to a wall or slab panel. Whereas in case of T2 and T3, the initial major cracks occur in the panels rather than at the joints and propagated towards the joint from there. In case of T1, the major crack that caused the ultimate failure in L1 is developed at the joint diagonally. The load carrying capacity of the connections T2 and T3 is found to be increased by 22.16% and 33.5% respectively as compared to connection type T1.

The moment vs rotation behavior of the T connections is shown below in figure 4.10.

In the case of T shaped joint specimens, almost similar behavior has been found as that of L shaped connections as depicted in figure 4.10. The ultimately moment capacity of the connection type T2 increases by 22.33% as compared to type T1. From T1 to T3 it is found to increase by around 33.5%.

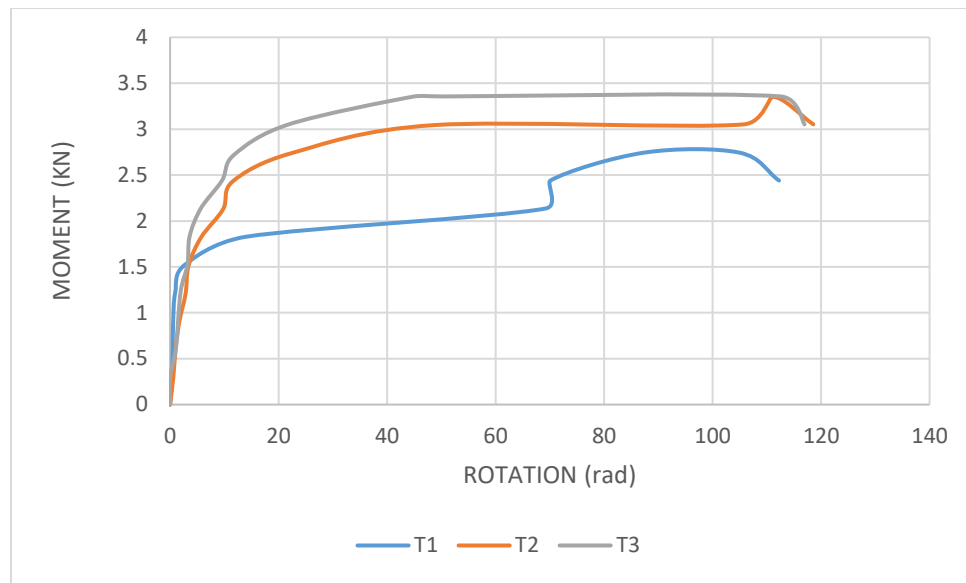


Figure 4.10 Moment vs Rotation for T-shaped connections

Table 4.1 reports the values of ultimate load and cracking load (F_u and F_{cr}) with corresponding values of ultimate bending moment and cracking bending moment (M_u and M_{cr}).

Table 4.1 Load and bending moment at cracking and failure

S. No.	Specimen	F_{cr} (kN)	M_{cr} (kNm)	F_u (kN)	M_u (kNm)
1.	L1	5.34	2.136	6.87	2.748
2.	L2	6.87	2.748	9.92	3.968
3.	L3	6.86	2.744	10.68	4.27
4.	T1	4.57	1.83	6.86	2.744
5.	T2	6.84	2.736	8.38	3.352
6.	T3	7.63	3.052	9.16	3.664

4.2 DISCUSSION

4.2.1 Stiffness and Ductility of connections

The stiffness and ductility values of the connections are calculated using the data given in table 4.1. As we know the stiffness is given as the load per unit deflection, therefore by using the ultimate load values and corresponding deformations for different connections the stiffness is calculated for corresponding specimens as given in table 4.2. Also the ductility of a material is defined as a measure of a material's ability to undergo considerable plastic deformation before cracking, which

may be expressed as % elongation or % area reduction. In this case ductility is calculated by the following method.

$$\text{Ductility} = (\text{Yielding area under load-deflection curve} / \text{total area under same curve}) * 100$$

As shown in the figure 5.11

A_f – total area under load-deformation curve

A_y – area under curve up-to yield point

Therefore, yielding area = $A_f - A_y$

Therefore, ductility = $(A_f - A_y) / A_f * 100$

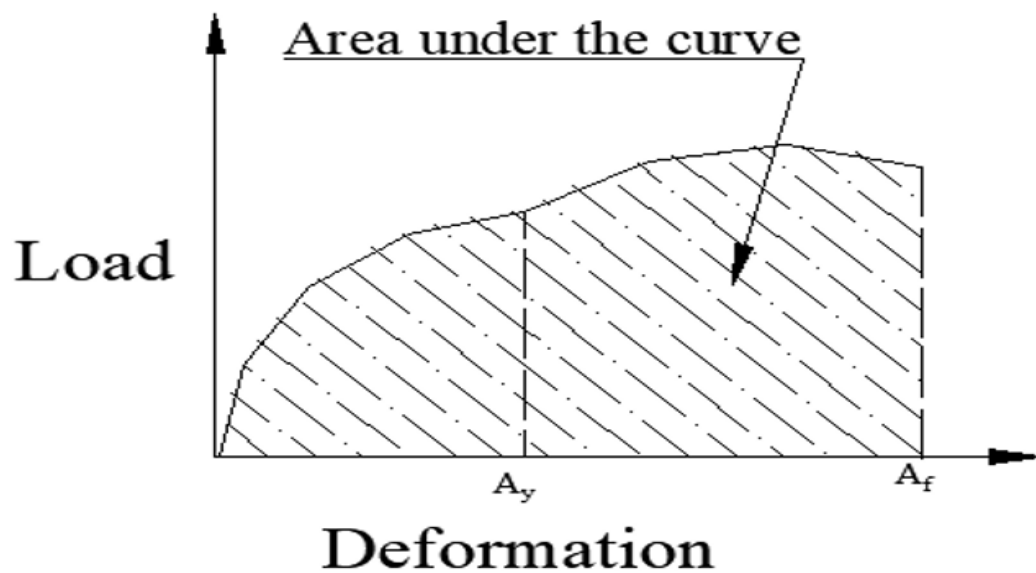


Figure 4.11 Depiction of area under a load-deformation curve

By using this equation ductility calculated for different connections is given in table 4.2.

From the table 4.2 it can be seen that the stiffness in both connection types i.e. L and T shaped is increasing with the modification applied to the connection. For instance from the above table, for connection type L2 stiffness value increase by 13.74% compared to L1, for L3 it increases by 17% compared to L1. In case of T-shaped connections for connection type T2 stiffness increases by 15.95% compared to T1, for type T3 stiffness increases by 24.5% compared to T1.

Table 4.2 Stiffness and Ductility of specimens

S. No.	Specimen	F_u (kN)	Deformation(Δ) in mm	Stiffness ($=F_u/\Delta$ in kNm)	Ductility
1	L1	6.87	37.83	182	87.13
2.	L2	9.92	47.91	207	93.69
3.	L3	10.68	50	213	93.07
4.	T1	6.86	41.98	163	87.84
5.	T2	8.38	44.43	189	89.07
6.	T3	9.16	45.02	203	88.96

Also from table 4.2, the ductility values for the type of connections provided with additional dowel bars and high strength mix are found to be more in comparison to the ductility values of the connections with normal configuration for both L and T shaped connections. In case of L-shaped connections the ductility increases by 7.53% and 6.82% for type L2 and L3 respectively compared to the joint with normal configuration. In case of T-shaped joints the ductility increases by 1.4% and 1.28% respectively for type T2 and T3 compared to the joint with normal configuration.

4.2.2 Comparison of moment capacities of wall-floor connections tested and similar connections in building construction

1. Moment capacity of connections tested in the laboratory

Moment capacity of the connection (M_u) is calculated by multiplying the applied force (F_u) and the distance of the point load from the connection ($a= 0.4\text{m}$).

Maximum of ultimate load of any type of connection = 10.68 kNm (from table 5.1)

Moment produced at the connection = $10.68 \times 0.4 = 4.27 \text{ kNm}$

Moment produced at the connection per meter width of the panel = $4.27/0.25 = 17 \text{ kNm/m}$

2. Moment capacity of connection considering 4.6m x 1.2m panel

As per IS 875 Part-1 & Part-2 1987, the load carrying capacity for the floor in the dwelling units considering only dead load and live loads given as

Dead load = $2 \text{ kN/m} - 3 \text{ kN/m}$

Live load = 2 kN/m

Therefore, total load coming on the of a floor panel = 5 kN/m²

Factored load (W_u) = $1.5 \times 5 = 7.5$ kN/m²

Moment produced at the wall-floor connection due to total load on the floor = $(W \cdot L^2)/10$

.....(Approximate method)

$$= 7.5 \times 4.62/10 = 15.87 \text{ kNm/m}$$

Table 4.3 Comparison of moments produced at wall-floor connection in a building and connection tested in laboratory

S. No.	Connection type	M_1 (kNm/m)	M_u (kNm)	M_2 (kNm/m)	Percentage change from M_1 to M_2
1.	L1	15.87	2.748	10.99	44.38 (decrease)
2.	L2	15.87	3.968	15.872	0.013(increase)
3.	L3	15.87	4.27	17.08	7.62(increase)
4.	T1	15.87	2.744	10.98	30.84(decrease)
5.	T2	15.87	3.352	13.41	15.51(decrease)
6.	T3	15.87	3.664	14.66	7.65(decrease)

From the above calculations it can be observed that the moment capacity of the wall-floor connection tested in the laboratory is about 7% higher compared to the similar connection in the prototype buildings.

In table 4.3, M_1 = Moment capacity of wall-floor connection in a building.

M_2 = Moment capacity of the wall-floor connection tested in lab

It can be seen from the table 4.3, the moment capacity of the connection type L2 & L3 is more than the moment capacity of the wall-floor panel connection in a building for 4.6m long and 1.2m wide floor panel. In all other type of connections i.e. L1, T1, T2 and T3, moment capacity is observed to be lesser.

CHAPTER V

CONCLUSIONS

5.1 GENERAL

From the study it was found that the connections provided with additional dowel bars and high strength mix showed superior structural behavior with high strength mix showed superior structural behavior with high strength, stiffness and ductility compared to the specimen with normal mix and normal reinforcement. Around 50% increase in the ultimate moment capacity has been recorded in the connection types provided with additional configuration which included dowel bars and high strength mix. The following points were concluded from this:

1. The connection specimens of both L and T shape provided with normal configuration of the reinforcement and mortar mix provides less strength, stiffness and ductility to the joint.
2. Use of dowel bars increased the ductility of the material. Also it prevented developing initial cracks at the joint. The ultimate bending moment capacity increased by 44% and 22% in L and T shaped connections respectively for connections with dowel bars compared to the connections without dowel bars.
3. Stone dust proved to be a good replacement of sand to increase the strength of material. The specimens cast using stone dust as replacement of sand, in addition to other configurations used for other specimens, have around 55% and 33% more ultimately bending moment capacity compared to connections with normal configurations for L and T shaped specimens respectively.
4. The specimens with provision of additional configuration have higher stiffness and ductility values compared to specimens with normal configuration. There is at least around 15% increase is recorded in stiffness of the connection specimens with additional configurations. The ductility of the connection specimens provided with additional dowel bars to the normal configuration, is found to be highest, around 7% and 2% higher in L and T type of connections respectively. These properties make these specimens provided with additional configuration in terms of reinforcement and mortar mix, superior with regards to structural behavior.
5. For all connection specimens with additional configurations moment capacities are found to be near about the moment capacity of a connection considering a full scale reinforced EPS slab panel. In connection types L2 & L3 only, it is found to be approximately .013%

and 7.62% higher, making these connection types most suitable. This comparison justifies the efficacy of the joints tested and significance of this research.

5.2 FUTURE SCOPE OF THE WORK

EPS core panel system is a latest technology in the field of construction. The use of this system in a building imparts less weight, economic, faster construction, provides insulation etc. Joints between wall and slab panels make an important part of the system in terms of structural behavior. A little literature can be found on the behavior of joints in EPS core panel system, so a study in this area is always welcomed. Extensive studies need to be carried out on the moment capacities of joints for monotonic loading in one direction in such buildings. In this study only a limited type of connections on the same floor have been studied, so different types of connections present at other locations in a building e.g. a wall-slab joint at the junction of two floors can be studied. Further studies can be carried out on the behavior of joints by the application of different types of loading e.g. earthquake loading, wind loading etc.

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APPENDICES

APPENDIX A

The connection details of the wall-slab connections at a single floor and at different floors is depicted in figures A1 and A2 as reported by the report on reinforced EPS core panel system by Building Materials & Technology Promotion Council (BMTPC), Government of India 2017.

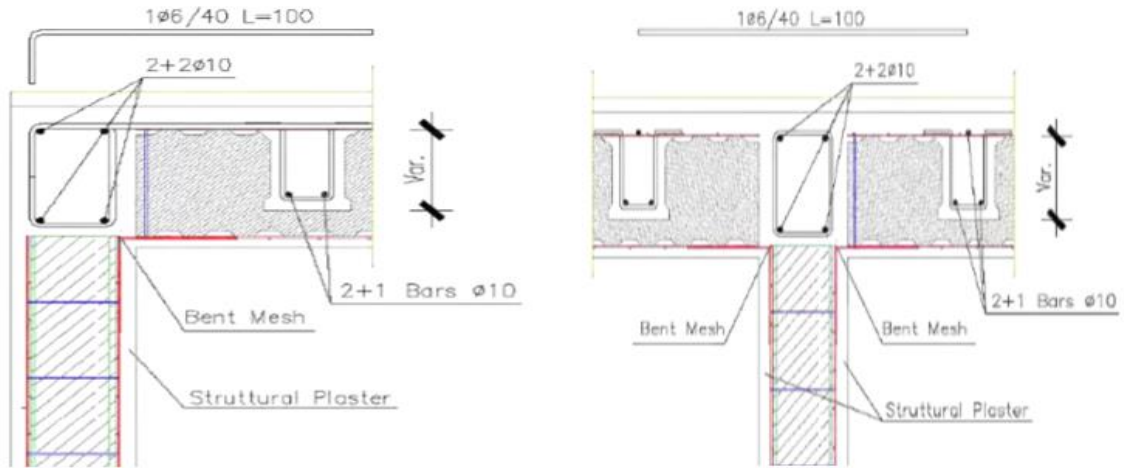


Figure A1 Wall-floor connection in single floor

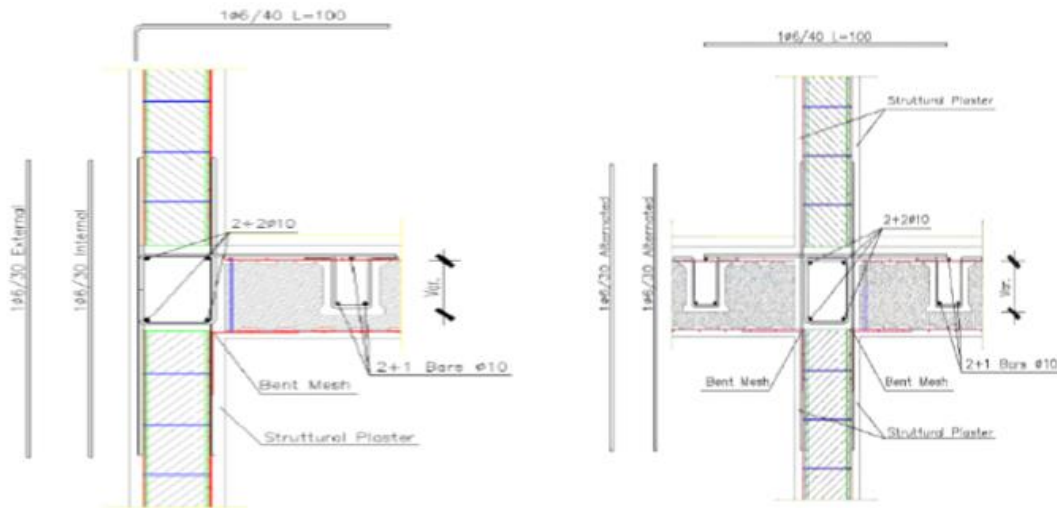


Figure A2 Wall-floor panel connection in different floor

APPENDIX B

The chipping of connections is done after testing with the help of a hammer along the cracks propagated in specimens, to more clearly observe the failure pattern of the different specimens.



Figure B1 Crack pattern in different specimens after testing

