STRUCTURAL FEASIBILITY OF A PRECAST BUILDING SYSTEM

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Degree of Master of Science

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Thesis submitted in partial fulfilment of the requirements for the degree Master of Science

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DECLARATION

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ABSTRACT

In a global context where the need for quality housing is increasing with improved standard of living and growing population, the over exploitation of natural resources as building materials is becoming a serious problem. In addition, many construction related issues are coming up due to the increased costs, higher construction time and lack of construction labour. In this context, precast building systems with pre-stressed concrete slabs, beams and columns along with Expanded Polystyrene (EPS) based light weight concrete sandwich panels as an infill material has a great potential to become an alternative construction technique.

The research presented in the thesis was aimed at assessing the structural feasibility of the proposed system. To this end, design and constructability aspects related to the pre-stressed concrete slabs, beams and columns are presented along with a comparison of a two-storey house between this system and a conventional reinforced concrete structure. In addition, the design of pre-stressed columns is discussed in detail through the development of interaction diagrams. This system is beneficial due to the use of reduced section sizes, saving on steel and elimination of formwork and falsework.

Also, in this research, the use of mechanically recycled expanded polystyrene as 50 % of the total EPS used in a composite foam concrete panel has been assessed experimentally. The results of the experimental program have been interpreted with respect to various useful structural behaviours. It is shown that the use of this foam concrete along with cement fibre boards produces a light weight wall panel that can be used very effectively for non-loadbearing walls in houses, apartment buildings, hotels and commercial buildings thus reducing the overall weight of the building while allowing rapid construction that results in a durable system.

Key words: interaction diagram, mechanically recycled EPS, pre-stressed concrete

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f_{cu} = Characteristic strength of concrete

f_{pu} = Characteristic strength of a pre-stressing tendon

1 INTRODUCTION

1.1 Background

Sri Lanka is slowly but steadily moving into the status of a developing country (The Annual Report of the Central Bank of Sri Lanka for the Year 2015). This has led to improved standards of living and hence, people are seeking better housing. Therefore, the increase in construction has been inevitable and hence, the overexploitation of natural resources for building materials has become a serious problem. In addition, the labour requirement for conventional construction techniques is rather high and this has also led to increased time and cost. In this context, alternative techniques of construction look attractive due to their optimized use of material and labour, which in turn would reduce the use of natural resources. However, design and construction experiences gained through years of practice in relation to conventional techniques, could be a major obstacle when shifting to alternative approaches.

Additionally, given the growing concerns regarding disaster resisting structures due to the recent history of earthquakes and cyclones in the South East Asia region, these new building systems should be checked for its ability to withstand such disasters. Especially, given the lack of design guidelines for such techniques, satisfactory experimental results are required to convince the general public of its ability to withstand such disasters.

The concept of pre-cast construction is not new to Sri Lanka and hence, it is important to investigate the existing technics in order to add a meaningful contribution to the pre-cast industry in Sri Lanka. Many reputed construction firms in Sri Lanka are already producing pre-cast elements. Pre-stressed concrete elements such as Pre-stressed hollow core slab system, SBS floor slab system, pre cast piles by International Construction Consortium (ICC) and Pre-stressed purlins by State Development and Construction Corporation (SD&CC), as well as pre cast wall panels like ACOTEC Precast Wall Panels by ICC are some common examples. In addition many small scale attempts are carried out to produce such pre-cast elements. One such pre cast element system has been investigated in detail in this thesis. A pre-cast sandwich wall panel system made out of Expanded Polystyrene (EPS) based light weight concrete is presented in detail.

1.2 Objectives

Main objective: Determination of the structural feasibility of the proposed pre-cast building system

Sub objectives:

- Pre-stressed concrete – Assess the structural feasibility of a new system with pre-stressed concrete frame and EPS panels for walls and roofs.

- (EPS) based light weight concrete sandwich wall panels –To investigate the structural and other characteristics of a light weight wall panel which uses mechanically recycled EPS as one of the raw materials.

1.3 Methodology

The following steps were carried out in order to achieve the above objectives.

- A comprehensive literature review was carried out in order to identify the knowledge gap in relation to pre-stressed concrete design and EPS based light weight concrete sandwich wall panels.

- A comparative study was carried out for a two-storey house constructed out of reinforced concrete frame with cement sand blocks and roofing sheets and pre-stressed concrete frame with EPS panels for walls and roof.

- A foam concrete mix that uses EPS was developed experimentally to obtain a mix with a lower density while having sufficient strength.

- Wall panels were constructed and tested to determine the compressive strength and the flexural strength.

- The structural adaptability of the wall panels was checked using the above results.

1.4 Arrangement of Thesis

- Chapter 2 of the thesis presents the findings of the literature review.
- Chapter 3 provides the design procedure for a framed structure with pre-stressed elements.
- Chapter 4 and 5 presents the experimental program and the results related to the EPS based light-weight concrete sandwich wall panels.
- Chapter 6 highlights the constructability and applicability of the proposed wall panel and its construction related aspects are presented using a case study.
- Chapter 7 presents the conclusions.

2 LITERATURE REVIEW

2.1 General

Concrete, which is a widely used construction material, has undergone changes of different forms over the years. There have been many researches that have been carried out to improve its performance. One such development is the usage of concrete for the construction of pre-cast elements.

The proposed pre-cast system consists of pre-stressed concrete beams, columns and slabs and wall panels constructed using Expanded Polystyrene (EPS). The literature review was carried out with the intention of identifying the knowledge gap in relation to the usage of pre-stressed concrete in design. Also, previous research related to concrete mixes with EPS was reviewed in order to determine the possibility of using EPS in a light-weight wall panel with sufficient strength parameters.

2.2 Pre-stressed Concrete

Pre-stressed concrete uses the principle of applying an initial force to overcome concrete's inherent weakness in resisting tensile forces. This pre stress force is an external force which will not be considered as a normal load which the structure is expected to experience during its design life. This pre-stress force is applied via a steel wire, strand or bar (Kaylor, 1961). In modern day construction, pre-stressed concrete is widely used as a substitute for reinforced concrete and its advantages and disadvantages are given in Table 2.1.

Table 2.1: Advantages and disadvantages of pre-stressed concrete

Advantages	Disadvantages
• This enables to have a crack free structure under service loads, which would in turn result in higher stiffness, smaller deflection and greater durability in aggressive environments.	• It would be more expensive due to using of high strength materials. • Larger capital would be required to facilitate the pre-

• Slender sections can be used in structures and hence, there will be a reduction in the self-weight of structures. • Deflections of flexural elements under service loads will be small because pre-stressing would cause an upward camber initially. • The using of higher strength steel and concrete would provide a high quality product, compared to using reinforced concrete. • Facilitates repetitive and rapid construction.	stressing operation. • The design and production of pre-stressed concrete will require highly trained personnel.

Two most common modes of pre-stressing is pre-tensioning and post-tensioning. Pre-tensioning involves applying the pre-stress force, before the element is concreted, where as in post-tensioning, the pre-stress force is applied only after the concreting of the element. Pre-tensioning is often used in situations where straight tendon profiles could be maintained, because providing curved profiles under this method requires extra work in the form of holding down (Nawy, 2006). The main features of these two methods are given in Table 2.2.

Table 2.2: Pre-tensioning verses Post-tensioning

Pre-tensioning	Post-tensioning
• Generally suitable for shorter spans. • Employs straight tendon profiles. • It is comparatively cheaper	• Generally suitable for medium and large spans. • Can facilitate curved tendon profiles. • It is comparatively expensive

Concrete and steel are the two main materials used in pre-stressed concrete. Each of these materials would require certain properties to ensure that the pre-stressed concrete would behave in the desired way. As far as concrete is concerned, concrete with high early strength is preferred, since the transmitting of pre-stress force will happen within the few days of casting the concrete. The steel wires, strands and bars used for pre-stressing have tensile strengths in the range of 1600-1800 N/mm^2, which is far greater than the yield stress of tor steel used for reinforced concrete work. The general range of sizes for wires, strands are bars are 3-7 mm, 8-18 mm and 20-40, mm respectively. In addition, special corrosion prevention mechanisms such as galvanizing, cathodic protection etc. are incorporated in to the steel, to enhance its durability.

Design procedures related to pre-stressed concrete is found in BS 8100: Part 1:1985, BS EN 1992-1-1 2004 and ACI Code. Design of pre-stress concrete elements is examined for two stages, namely; transfer stage and service stage. During the transfer stage, the pre-stress force is transmitted to the concrete elements and the concrete will still be in the process of gaining its full strength. This will not be a major concern because in addition to the pre-stress force, the element would only have to carry its self-weight during this period. However, the concrete is expected to gain its full strength by the service stage and in addition to the loads experienced during the transfer stage, the elements would also have to carry the superimposed dead and imposed loads, which act during the normal course of the design life of a structure. The codes provide maximum permissible tensile and compressive stresses for pre-stressed concrete during each of these stages and the governing inequalities for the design of pre-stressed elements are derived based on these stress limits.

Another critical phenomenon in pre-stressed concrete is the loss of pre-stress due to changes in concrete as well as the pre-stressing steel. Some of these losses may occur within the first few days (short term), while some of it could happen over a longer period of time. Elastic shortening of concrete, anchorage set and friction between ducts and tendon could cause a loss of pre-stress in the short run. Out of the above types of losses, only the first is related to pre-tensioning, while all three may be found in post-tensioning. As far as long term losses are concerned, creep and shrinkage of concrete and relaxation of pre-stressing steel could cause losses. It is important to quantify these losses to ensure sufficient pre-stress force is present within the element, during its design life. There are provisions in the BS code to quantify these losses and there have been extensive research work afterwards to quantify it more accurately.

2.2.1 Pre-stressing of beams and slabs

Pre-stressing of beams and slabs is carried out to counter the tension induced due to the flexural effects due to loading. The compressive pre-stressing force is expected to create a slight initial hog in the beams and slabs, in order to counter the sagging effect due to the loading. Therefore, the deflections, both upward and downward need to be calculated in order to prevent excessive deflections.

2.2.2 Pre-stressing of columns

Columns are primarily designed to carry compressive forces and hence, applying additional pre-stress forces could seem redundant. In fact, this might even look disadvantageous because this could reduce the compressive load carrying capacity of a column. However, columns in a frame are most likely to experience tensile stresses due to bending moments, which are in fact caused due to eccentric application of the axial loads. Therefore, as with the case with elements where flexural effects predominate, the introduction of the pre-stressing force on columns would transform the cracked section into an un-cracked section. This is because the pre-stressing steel could increase the resistance to cracking. Thus, the entire section would be able to resist the bending effects caused due to the eccentricities (Valaparambil et al., 2008 and Itaya, 1964).

Also, in the case of slender columns, pre-stressing steel could be beneficial. This is because, the column will act as a homogenous member due to the fact that the pre-stressing steel will be in contact with the surrounding concrete thorough out the entire length of the column. Therefore, the buckling capacity could be considered to be reliable due to this homogeneous nature of the pre-stressed column (Valaparambil et al., 2008 and Itaya, 1964).

Furthermore, non pre-stressed reinforcements are also added to the columns in order to increase its ductility and to increase compressive and tensile resistance in the ultimate load range.

As far as design of pre-stressed concrete columns is concerned, the eccentricity of the load is of significant interest. For large eccentricities, the primary mode of failure could be due to the yielding of steel due to tension at the column face farthest from the load. But for smaller eccentricities, the primary mode of failure is due to the maximum strain in concrete exceeding the maximum permissible strain of concrete.

Generally, as it is with reinforced concrete columns, interaction diagrams can be used to design a pre-stressed concrete column. Figure 2.1 illustrates an interaction diagram for a pre-stressed column.

When subjected with an eccentric load (P_n) a column may develop an equal and opposite axial force to resist it. Therefore, these forces would create a moment and hence the column will also develop an internal moment (M_n) to resist this. P_n and M_n can be calculated by considering requirements of equilibrium for the column. Therefore, the failure line in the interaction diagram corresponds to combinations of P_n and M_n at the point of failure of a pre-stressed column. Furthermore, the two extreme points on the graph P_o and M_o correspond to concentric compression failure and pure bending failure respectively (Nilson, 1987).

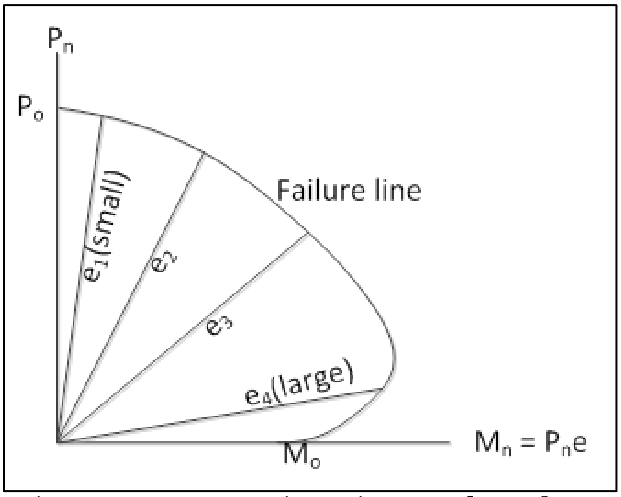


Figure 2.1: Interaction Diagram for column

The forces, stresses and strains are illustrated in Figure 2.2.

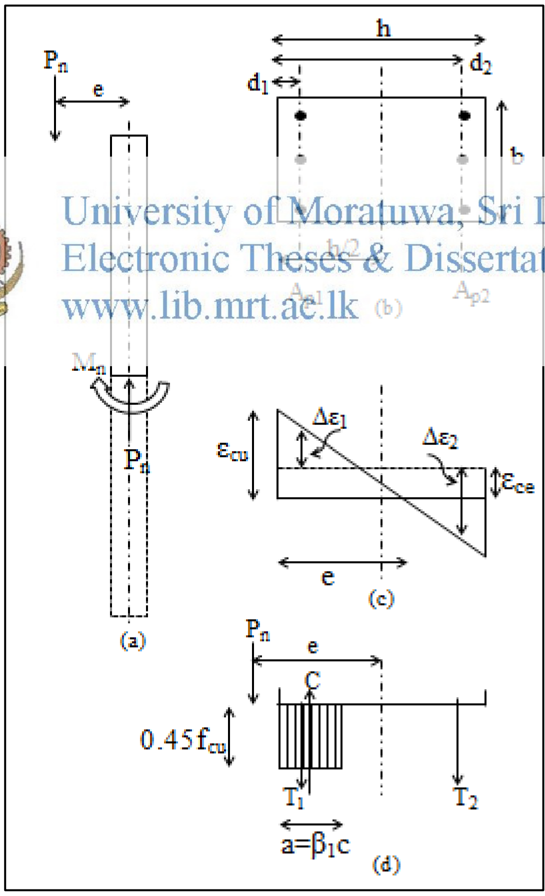


Figure 2.2: Short column with eccentric loading. (a) Free body of half column. (b) Cross section. (c) Strain distribution. (d) Forces and equivalent stresses

The terms illustrated in the above figures could be explained as follows.

C = Compressive stress in the concrete = $0.45f_{cu}$

T_1 = Tensile force due to the steel on the left face

T_2 = Tensile force due to the steel on the right face

d_1 = Depth to the steel reinforcement on the loaded side of the column

d_2 = Depth to the steel reinforcement on the other side of the column

ε_{pe} = Strain in pre-stressing steel

ε_{ce} = Strain in concrete

ε_{cu} = Limiting value of concrete strain at compression

b, h = Plan dimensions of the column

e = Eccentricity of the applied force

c = The distance to the neutral axis at failure from the face of the column

a = The approximated width of the stress block, usually expressed as βc, where β is the reduction factor which is specified as 0.9 in the British Standard codes.

Hence, by considering equilibrium conditions for strain compatibility method, the following equations could be derived for P_n and M_n at failure (Nilson, 1987).

$P_n = C - T_1 - T_2$

$M_n = P_n e = C(h/2 - a/2) - T_1(h/2 - d_1) + T_2(d_2 - h/2)$

The calculations for P_n and M_n can be carried out by using the following equations.

$$\varepsilon_{pe} = \frac{f_{pe}}{E_p} = \frac{P_e}{(A_{p1} + A_{p2})E_p}$$

$$C = 0.45 f_{cu} * a * b$$

$$T_1 = A_{p1}E_p(\varepsilon_{pe} - \varepsilon_{cu}\frac{c - d_1}{c} + \varepsilon_{ce})$$

$$T_2 = A_{p2}E_p(\varepsilon_{pe} + \varepsilon_{cu}\frac{d_2 - c}{c} + \varepsilon_{ce})$$

Therefore, for given column geometry, material strengths and pre-stress force, different failure combinations of P_n and M_n can be derived by selecting arbitrary locations for the ultimate neutral axis (c).

Furthermore, the slenderness effects of columns must also be taken into consideration. This could be a main factor in pre-stressed columns because, generally, smaller section sizes are used. The primary moment (as identified earlier), would cause the column to deflect laterally and this displacement would cause an increase in the bending moment due to the increased eccentricity at which the applied force will be acting. This effect should be taken in to account when producing the interaction diagram and the following equation could be used for the failure moment (M_n) (Nilson, 1987).

$$M_{max} = M_o \frac{1}{1 - {}^{p}/_{p_c}}$$

P_c, which is the critical lateral buckling load, can be calculated using the following equation.

$$P_c = \frac{\pi^2 EI}{(kl_u)^2}$$

The other terms in the above equations can be defined as below.

M_o = Primary moment due to the eccentric axial load

P = The applied axial load

EI = Flexural stiffness of the member

l_u = Unbraced length

k = Effective length factor, which takes into account the different conditions of end restraint

As illustrated in Figure 2.3 the variation between P_n and M_n would be non-linear and the failure would eventually occur at a lower force and a higher moment (Nilson, 1987).

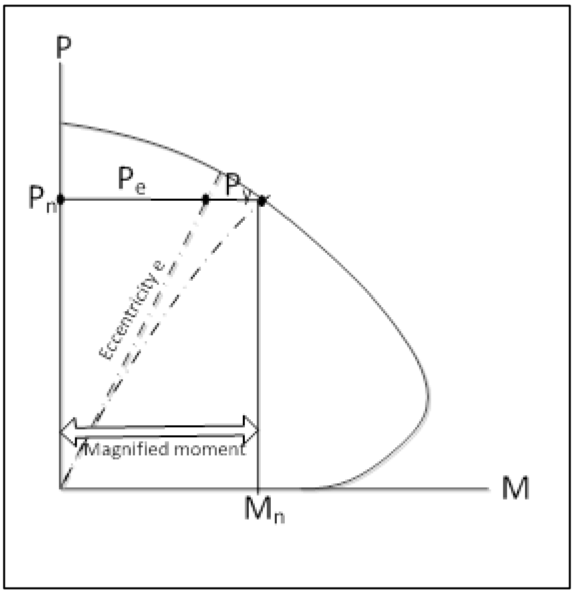


Figure 2.3: Interaction diagram for slender column

2.3 EPS based Light weight concrete

Use of light-weight concrete for construction work has gained wider popularity due to its many benefits, structurally and otherwise. The lower weight of the construction material would mean that the sections of the elements constructed out of it could be reduced and therefore eventually, there would be benefits at the foundation level too (Kan and Demirboga, 2007). Additionally, when compared to ordinary concrete, light weight concrete has displayed other encouraging properties such as higher specific strength, thermal insulation and energy absorption (Xu et al., 2012). Light weight concrete is commonly used in applications such as cladding panels, curtain walls, composite flooring systems and load bearing concrete blocks (Kan and Demirboga, 2007).

Previous researches have revealed that there are different ways of producing light weight concrete. One such way is to introduce foaming agents. Another way is by replacing the aggregate content, fully or partially, with light weight mineral aggregates. Also, light-weight concrete could be produced by replacing the aggregate content with plastic granules such as EPS (Kan and Demirboga, 2009 (a)).

The research presented in this thesis concentrates on an EPS based light weight concrete.

EPS is produced by adding a blowing or expanding agent to polystyrene, which is a vinyl polymer (Kan and Demirboga, 2007). EPS is commonly used as a packing and insulating material. However, due to its artificial nature and the structural composition, its disposal has been problematic. If disposed into the nature, it would not decompose and hence could cause land and water related environmental problems (Kan and Demirboga, 2009). Therefore, using of EPS as an aggregate in light-weight concrete could be beneficial, especially when the environmental impacts are taken into consideration. However, the structural and mechanical properties of EPS need to be studied.

EPS has a density in the range of 10-30 kg/m^3 (Xu et al, 2012 and Kan and Demirboga, 2007). The reason for such low densities is mainly due to its composition. Generally, an EPS bead comprises of 98% of air and only 2% of polystyrene. Therefore, this composition of EPS would mean that its compressive strength could be very low and that could affect the overall strength of the light weight concrete. In fact, the compressive strength of EPS is expected to be between 0.068-0.39 MPa (Kan and Demirboga, 2007). Due to such low values, it is generally assumed that EPS beads have almost zero strength (Kan and Demirboga, 2009 (a)). Therefore, use of EPS based light weight concrete in structural applications could be problematic and hence, it is generally recommended for non-structural applications such as panel walls, concrete briquettes, etc. (Kaya and Kar,2016).

Another important property of EPS is its hydrophobic nature. This property is both advantageous and disadvantageous. Its advantages could be explained in terms of the water requirement of concrete at fresh and hardened states. Since EPS is hydrophobic, it would not absorb the water in the concrete at fresh state. Therefore, this would eliminate the necessity to add extra water in order to maintain sufficient levels of it for workability of concrete and the hydration of cement. As for concrete at hardened state, the hydrophobic and closed cell structure of EPS would mean that effects due to bleeding and shrinkage of concrete can be minimized (Ranjbar et al.,2009). However, the main disadvantage of the hydrophobic property is that, there could be segregation in the concrete due to lack of bonding between EPS and the mortar. Therefore, admixtures need to be added to ensure there will be a homogeneous mix in the concrete (Babu and Babu, 2003).

The size of EPS beads plays a major role in the strength properties of the light weight concrete. According to past research, the size of EPS beads could vary between 1-10 mm. It is reported that for a given density, concretes of higher compressive strength can be obtained by using smaller size beads. However, for EPS concretes with densities less than 1000 kg/m^3, the size of the EPS beads would not have a significant bearing on the compressive strength of the concrete (Miled et al., 2003 and Miled et al.,2006). In addition to the sizes, the surface texture of the EPS beads could also affect the properties of the concrete. It is reported that finer EPS granules allow a higher compressive strength than larger granules (Laukaitis et al., 2005).

It is important to study the effect of the above properties on the EPS based light-weight concrete. EPS beads tend to have a higher surface area compared to natural aggregates (Kan and Demirboga, 2009 (a)). Also, due to the very low density of EPS, there could be segregation while mixing of the materials and casting the concrete (Babu and Babu, 2003). Therefore, both these factors would reduce the workability of the concrete. In order to overcome these shortcomings, a polycarboxylate based super-plasticizer will be added to increase the workability of the concrete. (Kan and Demirboga, 2009 (a)). In addition, in order to achieve a homogeneous concrete, mixing, pouring and compacting of the concrete must be carried out in such a way, so as to ensure, the possibility of segregation is minimized (Kan and Demirboga, 2007). Furthermore, over vibration of the concrete could cause segregation and hence, hand compacting of the concrete is preferred (Sayanthan et al., 2013).

Density of the EPS concrete is a significant factor, because many other physical and mechanical properties depend on it. The dominant factor that controls the density of the concrete is the volume of EPS. Water cement ratio and cement and sand contents play a less significant role (Xu et al., 2012). The densities of EPS based concrete could vary between 400-2000 kg/m^3 (Kan and Demirboga, 2007). However, if it is to be used for structural purposes, the density of the concrete needs to be higher. In fact, it is reported that EPS based light-weight concrete with densities in the range of 1440-1850 kg/m^3, have a compressive strength of about 17 N/mm^2 (Babu and Babu, 2003). So it is clear that the compressive strength of the concrete would increase with the density of the concrete. However, thermal insulation of the concrete will be higher when it has a lower density (Xu et al., 2012). Also, using of higher volumes of EPS to reduce the density of concrete, would lead to greater voids and thereby, low strength of concrete (Tamut et al., 2014). Therefore, the density of the concrete needs to be decided based on the requirements of the purpose it would be used for.

Furthermore, the low strengths of EPS based light-weight concrete could be attributed to the fact that EPS beads have almost zero strength and due to the lack of adhesion between EPS and the mortar paste (Kan and Demirboga, 2009 and Kan and Demirboga, 2009 (a)). In order to overcome the issue related to the lack of adhesion, the EPS beads are pre-wetted with some of the water and the super plasticizer. This would increase the bonding between the cement paste and EPS in the interfacial zone (Kan and Demirboga, 2007). Another way of improving the bond between EPS beads and the cement paste is by adding ultra-fine silica fume (Xu et al., 2012).

Fly ash is another waste product that can be used to partially replace cement due to its pozzolanic action. In foam concrete, the cement content is in the range of 350 kg/m^3 or more. Therefore, it is possible to use fly ash to make a significant gain in strength (Chousidis et al., 2015). The percentage of fly ash could be up to about 25%. To ensure continuous hydration, Type F fly ash has been used. Type F fly ash refers to siliceous fly ash with low amounts of CaO (Chousidis et al., 2015). The presence of fly ash can improve workability (Babu et al., 2005). The partial replacement of cement can assist in reducing the embodied energy of foam concrete.

2.4 Summary

The use of pre stressed concrete can allow the development of pre-cast systems that has the potential to reduce the labour required for construction. The foam concrete panel that uses EPS can be used the infill panels of this system. For wider use of precast panels, its strength and durability properties have to be assessed, while assuming that its weight can be kept as low as possible. The precast system will need the development of a robust system of connection that can allow the transfer of adequate moments for resisting the specified lateral loads. It is also necessary to compare this system with that can be constructed with conventional materials to quantify the likely benefits.

A summary of the literature review is shown below in Figure 2.4

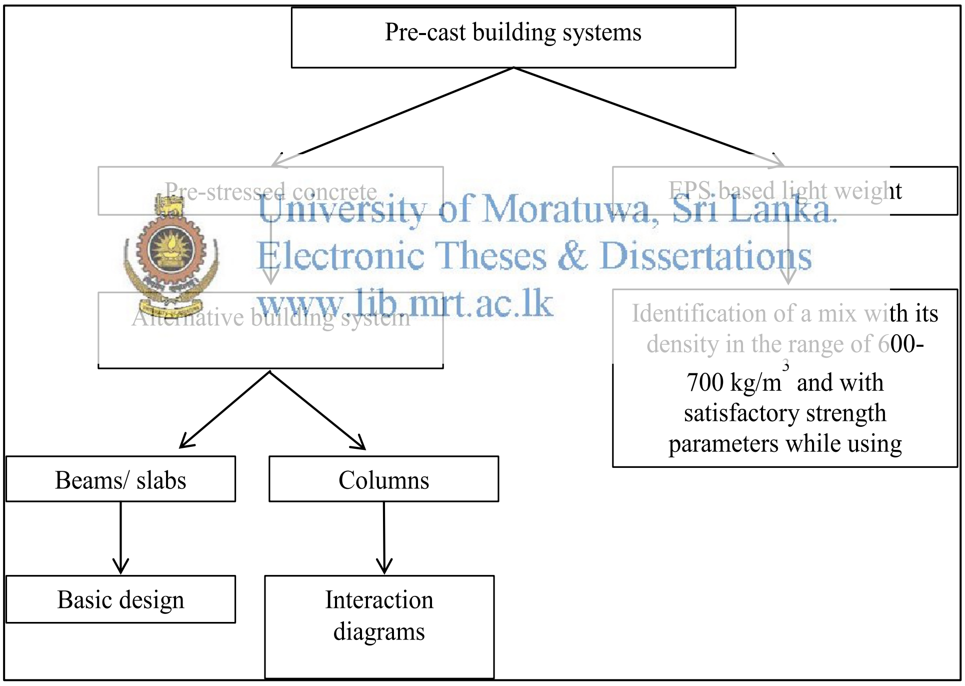


Figure 2.4: Summary of the literature review

3 DESIGN OF PRE-STRESSED CONCRETE ELEMENTS

3.1 General

Pre-stressed concrete elements will be beneficial if the section sizes could be kept as small as possible while assuming that the tendency for buckling failure is eliminated. This condition will be especially applicable for the columns. For this, a system that has already been developed was assessed to check whether it is adequate or would require further refinement.

3.2 Pre-stressed concrete elements

3.2.1 Slabs

It is useful to utilize pre-stressed, precast panels that have the potential to eliminate the need for scaffolding and also make a saving in the steel requirement. The precast slabs can be made effective for any span up to 6 m. For a span of 4 m, it is possible to use a thickness of 65 mm. When it is 6 m, the thickness should be higher and it has to be 75 mm.

The pre-stress was applied with 5mm diameter high tensile steel wires. The characteristic tensile strength is 1770 N/mm^2. The initial stress is maintained at about 60-70% of the characteristic strength. For 75 mm thick panels, 16 wires would be needed. For 65mm panel, it would be sufficient to have about 12-13 wires. After finishing placing the panels, 40-45 thick screed can be laid after placing 6mm wires to form a mesh of 200 mm x 200 mm to control the cracks. This is illustrated in Figure 3.1.

Figure 3.1: Reinforcement net on a pre-stressed slab panel prior to concreting

3.2.2 Beam

With pre-stressed concrete, the beam can be of a smaller cross section. The width of the beam could be maintained at 150 mm and the depth could be 250 mm or 300 mm. In order to assure adequate shear capacity, shear links should be provided. The cover to shear links could be maintained as 20 mm since Grade 40 concrete is used. The beam could be pre-stressed axially or with some eccentricity. A typical cross section of a beam is illustrated in Figure 3.2 and the casting yard of a pre-stressed concrete beam is shown in Figure 3.3.

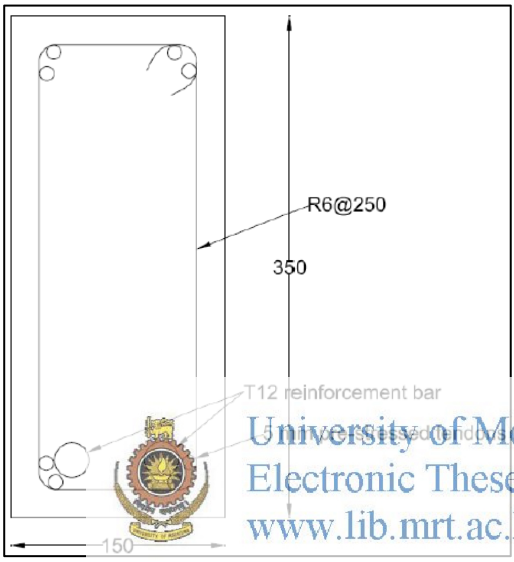


Figure 3.2: Cross section of a typical beam

Figure 3.3: Casting of pre-stressing beams

3.2.3 Columns

The column used would be of 150 mm x 250 mm when the beams are supported only in one direction. For T junctions, it is possible to use 150 mm x 350 mm columns. The use of 150 mm for one side would make it slender and hence, there could be a possibility for buckling. This has to be checked carefully. In addition, it is possible to use 200 mm x 200 mm sections or 200 mm x 350 mm sections depending on the spanning direction of the beams. Charts to determine the axial load and moment capacities have been developed and presented in this thesis. The tendon arrangement of a pre-stressed column is shown on Figure 3.4.

Figure 3.4: Pre-stressed concrete columns

3.2.4 Beam-column junction

Generally, joints in pre-stressed concrete structures are designed as simply supported, as opposed to the usual practice of monolithic joints in reinforced concrete. This would eliminate the hogging moments at the joints and therefore render the top reinforcement of beams to be nominal. However, this would increase the sagging moments at mid span of beams and hence would require additional pre-stressing to ensure the additional sagging moment is countered through the hogging moment caused due to pre-stressing.

However, achieving continuity at joints is critical during construction. Constructing connections that achieve continuity at supports is usually complex. In order to achieve connectivity, two numbers of Dowell bars of 20 mm or 25 mm diameter were anchored 400 mm into the columns using cement grout based anchoring, to ensure adequate strength at connection joints. One such joint type is shown in Figure 3.5.

However, a critical aspect that needs consideration is the moment carrying capacity of a beam-column junction. Since the frame is expected to carry the moments induced due to the lateral loads, the moment carrying capacity of the junction needs to be greater than the moment due to the lateral loads. The reinforcing steel on opposite faces of the column at the junctions could be expected to carry tension and compression due to the bending effect. Since, the stress in steel at tension is limited to $0.87f_y$, the moment carrying capacity of the junction could be reasonably approximated as the allowable tensile force times the lever arm. This calculation is numerically justified in the design example of the house.

Figure 3.5: A typical beam-column joint

3.3 Development of interaction diagrams for pre-stressed columns

Development of interaction diagrams was carried out for column sizes of 400 mm x 400 mm, 350 mm x 350 mm, 300 mm x 300 mm, 250 mm x 250 mm, 200 mm x 200 mm, 250 mm x 150 mm and 350 mm x 150 mm. Additionally, for each column size, different levels of pre-stressing was considered. This was carried out by changing the number of pre-stressing wires. The resulting interaction diagrams for 12, 10, 8 and 6 are presented.

In this, the calculations were performed as for BS 8110: Part 1:1985. Since the ultimate capacity is determined, the axial loads and moments should be the factored values.

The following procedure was followed.

Step 1:

The required data related to the dimensions of the columns, pre-stressing steel and concrete were included. 5mm pre-stressing wire with a Nominal tensile strength of 1770 N/mm2 and a failure load of 34.7 kN was used to apply the pre-stressing force. The other input data are presented in Figure 3.6.

DATA		
b-Column dimension 1	350	mm
h-Column dimension 2	350	mm
d1	25	mm
d2	325	mm
No of wires per layer	3	
Diameter of wire	5	mm
No.of layers	2	
Steel area of each layer	58.9125	mm2
Total steel area	117.825	mm2
Ap1-steel area on left face	58.9125	mm2
Ap2-steel area on right face	58.9125	mm2
Ece	28000	N/mm2
Yield strain	0.006	
Effective prestress	867.3	N/mm2
Ep	205000	N/mm2
εpe	0.004230732	
Concrete strength	[illegible]	N/mm2
εce	2.97929E-05	
εcu	0.0035	

Figure 3.6: Step 1 of column interaction diagram design

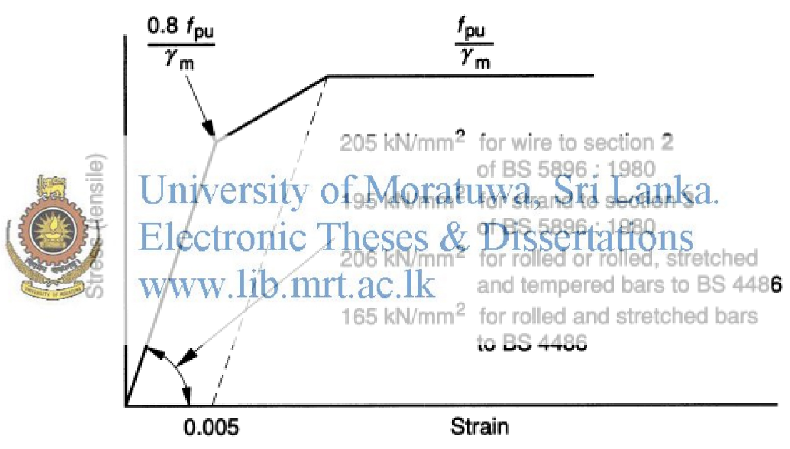
Step 2:

As shown in Figure 3.7, the axial force (P_n), moment due to eccentricity (M_n) and the eccentricity (e) were calculated based on different values for the distance of the neutral axis to the loaded face of the column. (denoted as c). This calculation assumes that the steel has not yielded and hence, the calculation of the stress is based on the strain and elastic modulus of steel.

However, if the steel has yielded the corresponding stress cannot be calculated using the above method. Therefore, the stress corresponding to the strain was calculated based on the stress-strain curve for pre-stressing tendons from BS8110:Part1:1985 (BS 8110, 1985). This graph is illustrated in Figure 3.8. Since a 5 mm wire with a nominal strength of 1770 N/mm^2 was used f_{pu} was taken as 1770 N/mm^2 and the gradient of the stress-strain curve was taken as 205 kN/mm^2. In addition, based on Table 2.2 of BS 8110:Part1:1985, a γ_m value of 1.15 was used.

CALCULATIONS		
Assumed c	20	mm
β	0.9	
a	18	mm
ε left face	0.005135525	
ε right face	0.057635525	
Compression	113400	N
Tension on the left face	62022.05157	N
Tension on the right face	90674.02174	N
Pn	-39.29607331	kN
Mn	23.12219553	kNm
e	-588.4	mm

Figure 3.7: Step 2 of column interaction diagram design



NOTE f_{pu} is in N/mm^2.

Figure 2.3 — Short term design stress-strain curve for prestressing tendons

Figure 3.8: Short term design stress-strain curve for pre-stressing tendons

Step 3:

The calculated values for P_n and M_n were plotted to obtain the interaction diagram for the chosen column size. These diagrams are illustrated in Figures 3.9 to 3.16. The following aspects should be taken in to account when using the above charts for designing of pre-stressed concrete columns.

According to BS 8110: Part 1:1985, a minimum moment of 0.05e times the axial force must be considered, where e refers to the overall dimension of the column in the plane of bending (BS 8110, 1985). However, the maximum permitted eccentricity is 20 mm. Therefore, when using the above interaction diagrams, points corresponding to lower moments than this calculated minimum moment must not be used.

Furthermore, the above charts have been developed without considering the slenderness effects. Therefore, in order to consider the slenderness effects, the expected moment on the column must be modified using the moment modification factor which takes into account the height and the axial load of the column. Hence, the point corresponding to the axial load and the modified moment must be within the accepted zone of the interaction diagram.

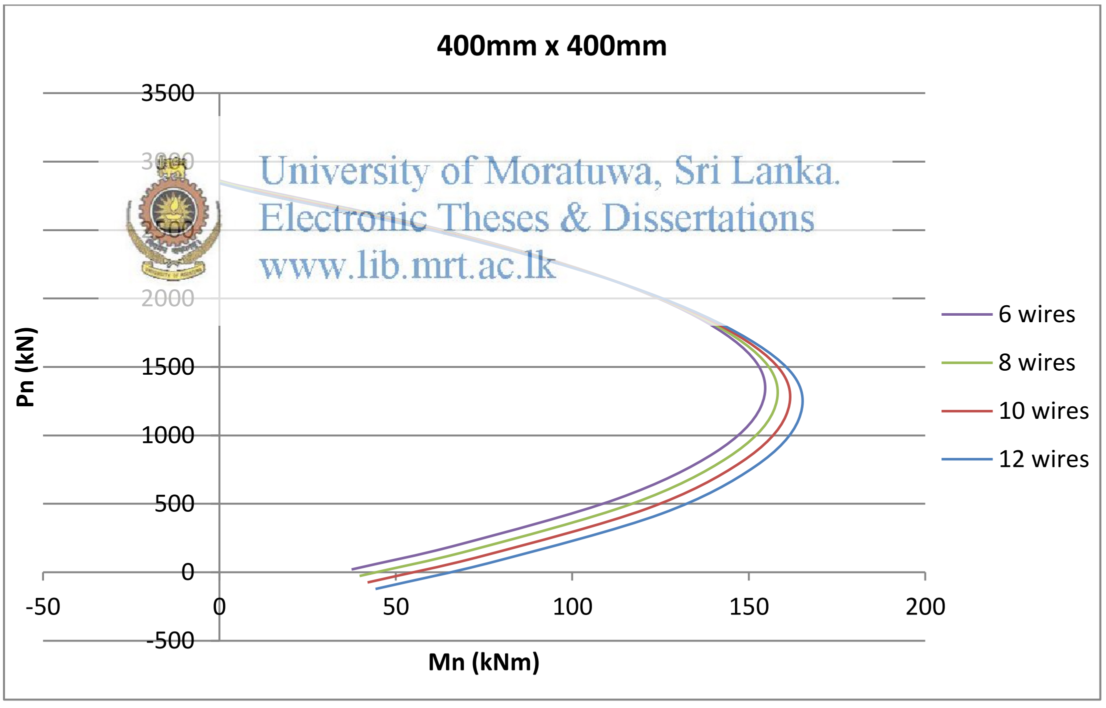


Figure 3.9: Interaction diagrams for 400 mm x 400 mm columns with 12, 10, 8 and 6 pre-stressing wires

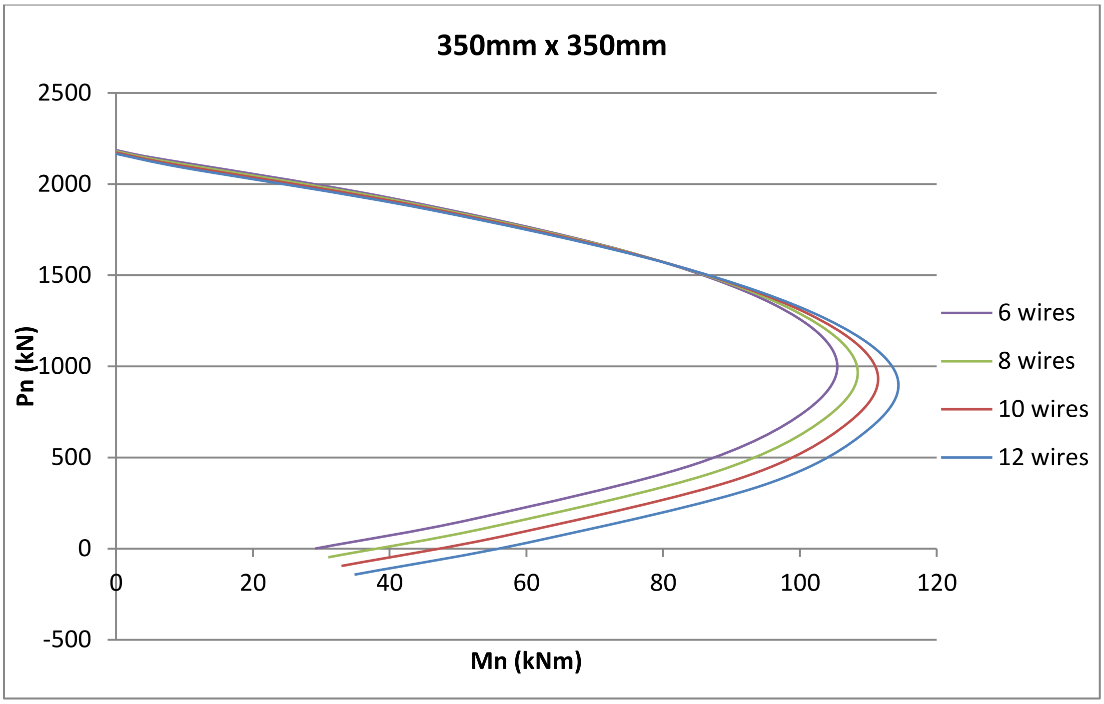


Figure 3.10: Interaction diagrams for 350 mm x 350 mm columns with 12, 10, 8 and 6 pre-stressing wires

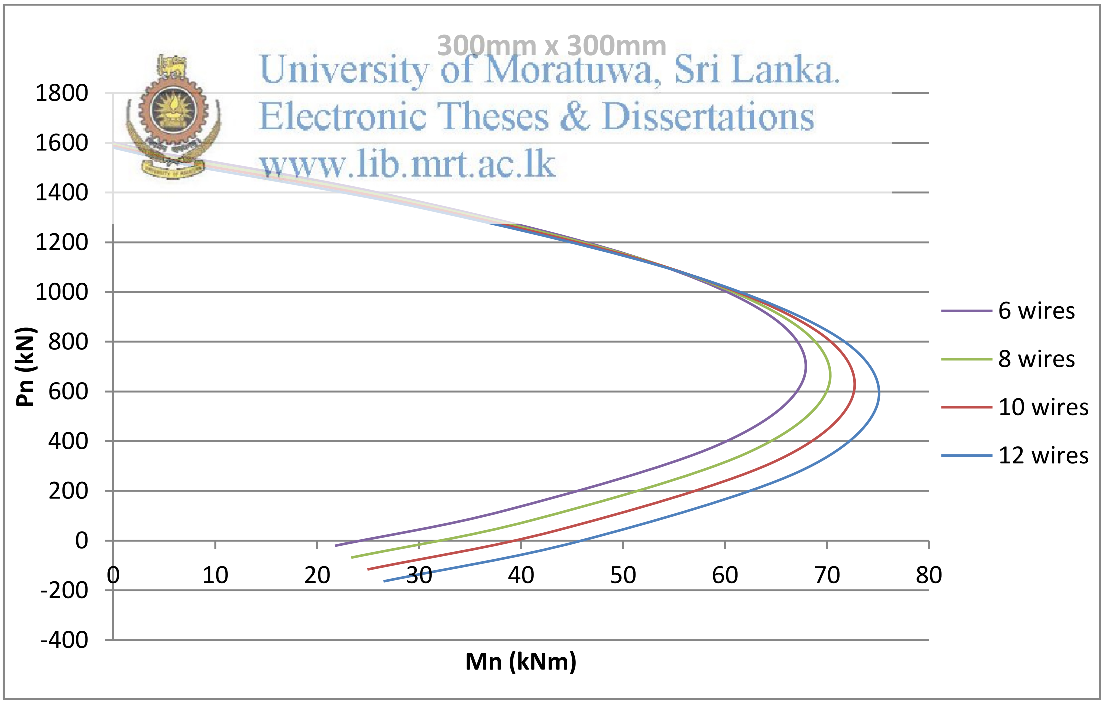


Figure 3.11: Interaction diagrams for 300 mm x 300 mm columns with 12, 10, 8 and 6 pre-stressing wires

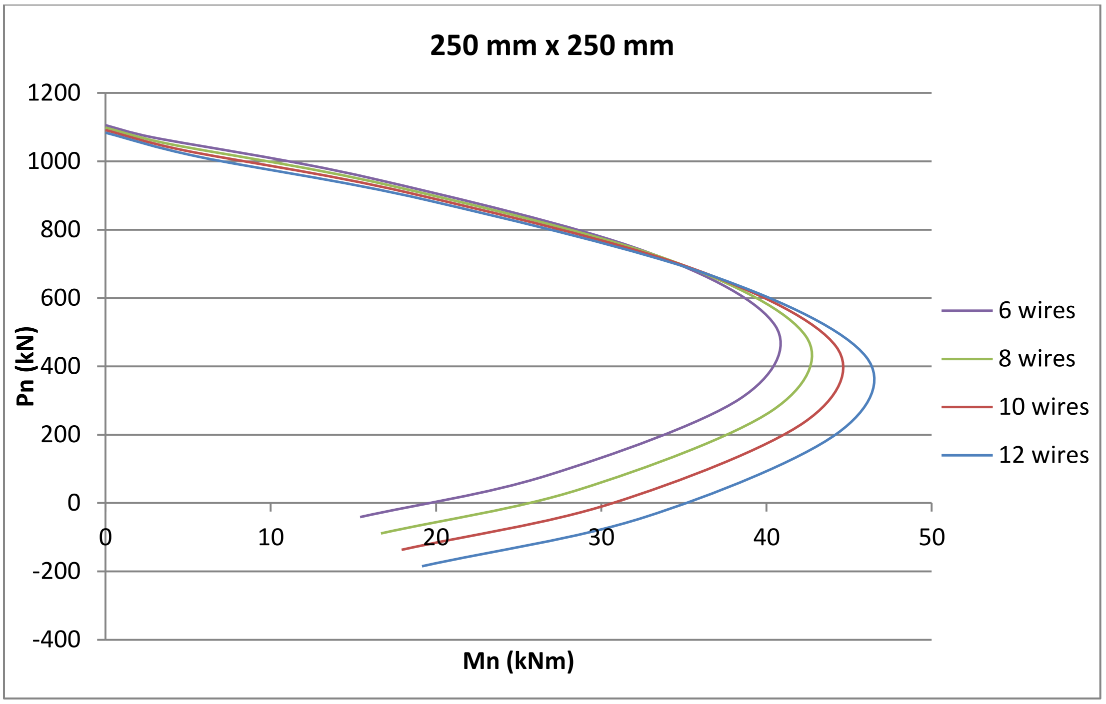


Figure 3.12: Interaction diagrams for 250 mm x 250 mm columns with 12, 10, 8 and 6 pre-stressing wires

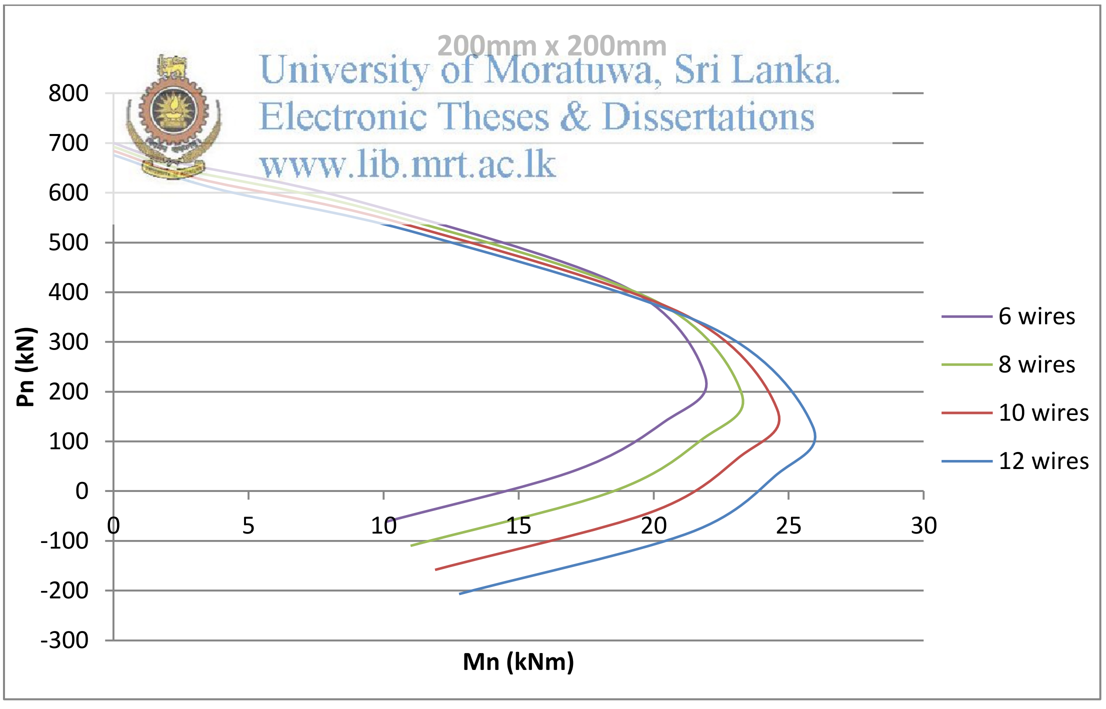


Figure 3.13: Interaction diagrams for 200 mm x 200 mm columns with 12, 10, 8 and 6 pre-stressing wires

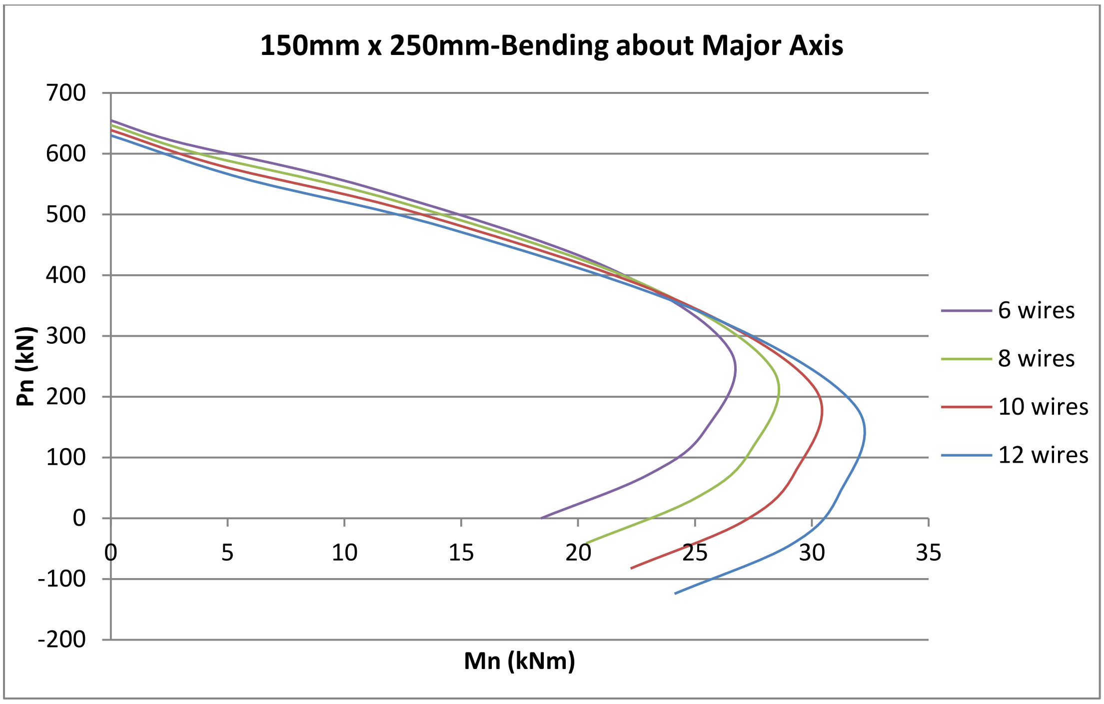


Figure 3.14: Interaction diagrams for 150 mm x 250 mm columns with 12, 10, 8 and 6 pre-stressing wires-bending about major axis

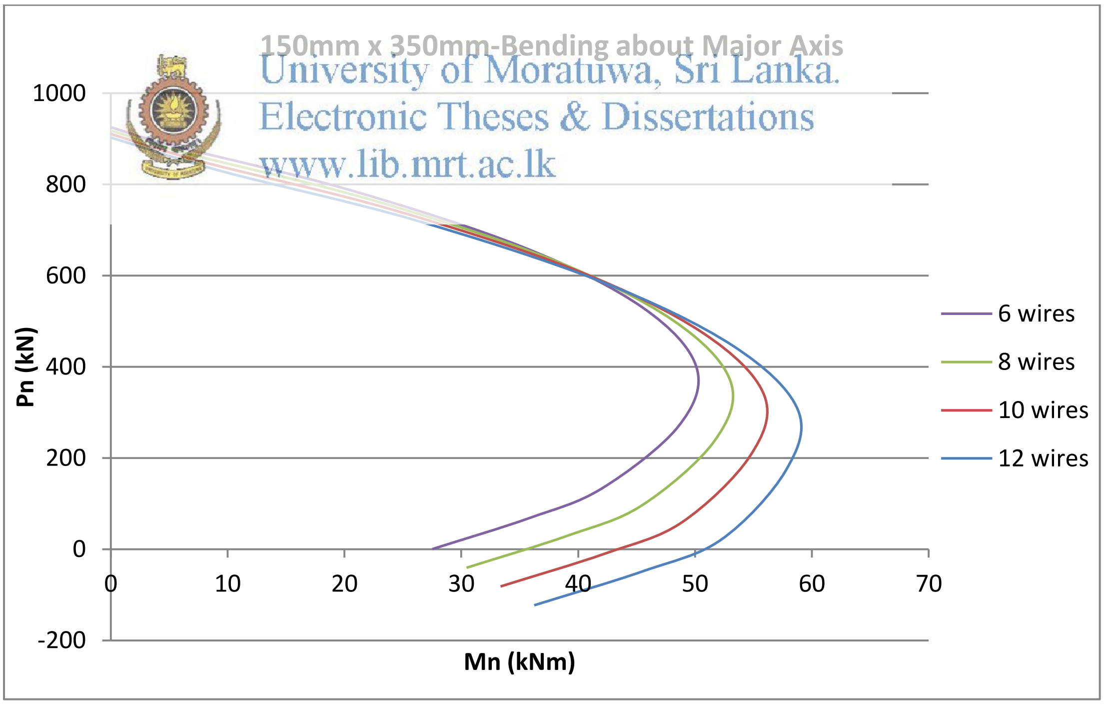


Figure 3.15: Interaction diagrams for 150 mm x 350 mm columns with 12, 10, 8 and 6 pre-stressing wires-bending about major axis

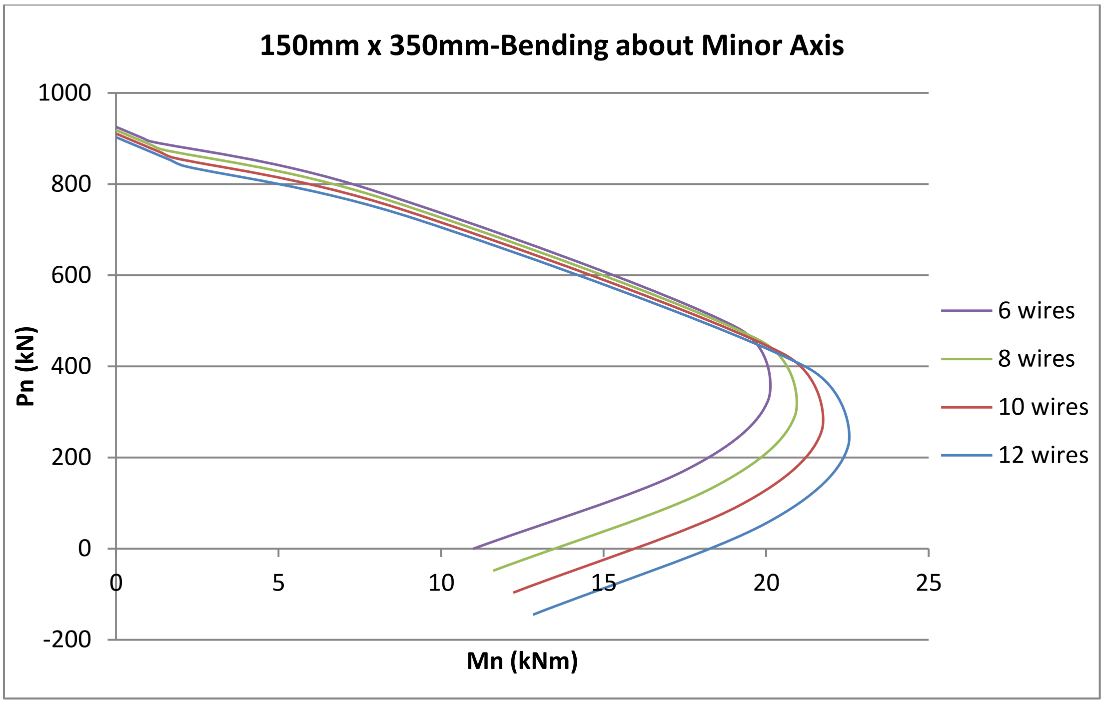


Figure 3.16: Interaction diagrams for 150 mm x 350 mm columns with 12, 10, 8 and 6 pre-stressing wires- bending about minor axis

3.4 Comparison for precast system

In order to determine the application of pre-stressed concrete precast system with wall panels, a typical two storey house has been considered. It is shown in Figure 3.17. This house consists of precast columns, beams and slabs. The walls are out of EPS panels. The arrangement of columns and beams are shown in Figure 3.18. All the columns have been considered as 350 mm x 150 mm in cross section. The spanning direction of slabs has also been marked in Figure 3.19. The slabs are all connected using a reinforced mesh of 6mm diameter at 200 mm center to center.

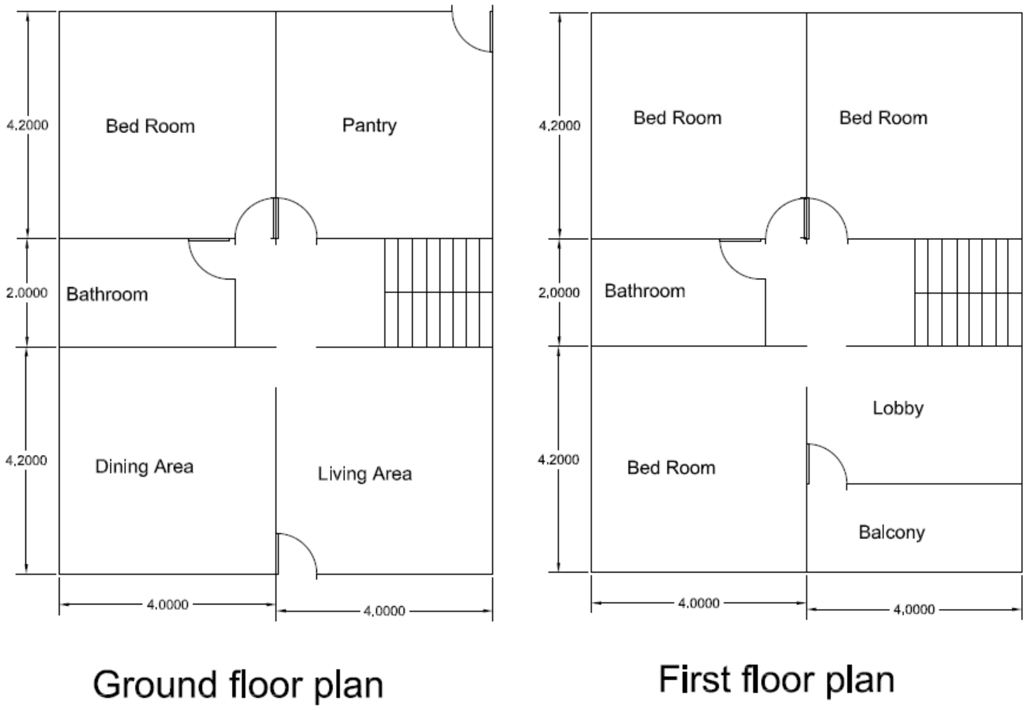


Figure 3.17: House plan

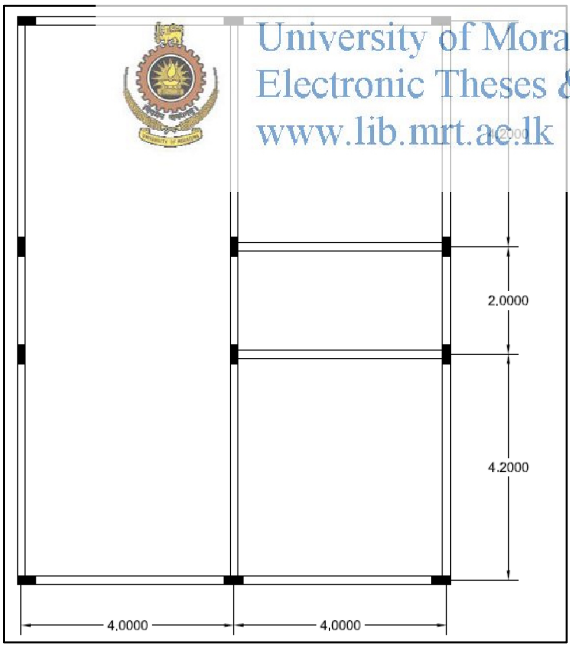


Figure 3.18: Beam column arrangement

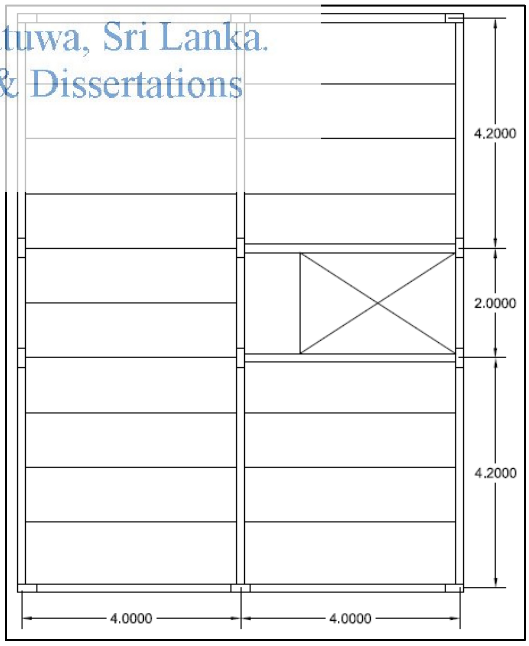


Figure 3.19: Spanning direction of the slab panels

One of the key considerations is the orientation of the columns. The columns have to be arranged with the major axis in a particular way to ensure adequate lateral stability. This is shown in Figure 3.18. The column size is 350 mm x 150 mm. The moment that has to be resisted at each joint is determined by using SAP2000. Based on the orientation of the columns, the lateral loads will be resisted by two types of frames and this is illustrated in Figures 3.20 and 3.21.

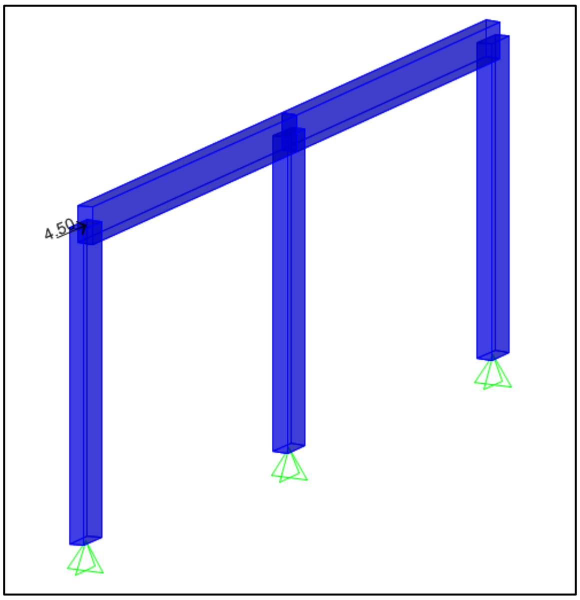


Figure 3.20: Frame resisting a lateral load parallel to the shorter direction of the house

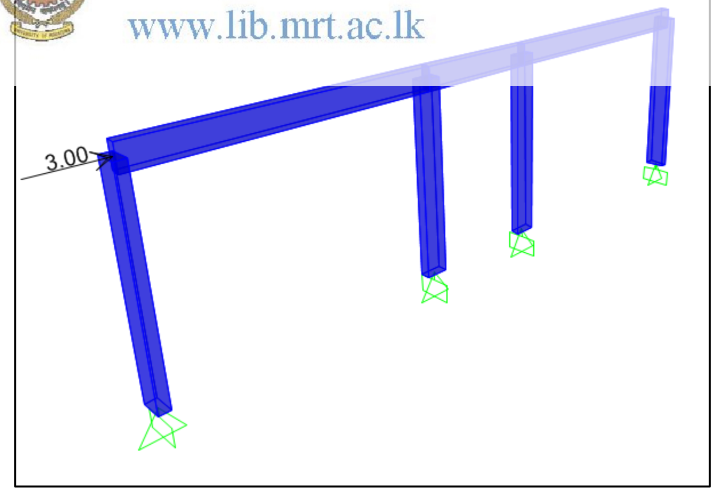


Figure 3.21: Frame resisting a lateral load parallel to the longer direction of the house

In order to compare the material quantities, the same house constructed with reinforced concrete frame and block work infill has been considered. For this columns were considered as 200 mm x 200 mm. The beams have been of the size 225 mm width and 250 mm depth. The reinforced concrete slab has been 125 mm. The reinforcement in the columns has been 4 numbers of 12 mm diameter bars. The slab reinforcement has been 10 mm bars 200 mm centers in both directions. Over the supports, 10mm bars are provided to a distance of 0.3 times the span at 200 mm centers.

3.4.1 Lateral loads on the structure

Since it is a two storey house, the frames consisting of columns and beams have been designed to carry a load of $0.015G_k$. When the weight of the slab and the walls are considered, this could be a representative load instead of the wind loads. The frames considered are given in Figures 3.20 and 3.21.The bending moment at the joints has been less than 10 kNm. Hence, a value of 14 kNm has been used to design the column. Since, the columns are 350 mm x 150 mm, the slenderness ratio is high. Hence, the slenderness effects also have been considered.

From charts, it is possible to find that 350 mm x 150 mm section with 12 wires can be used. For the beams, section of 150 mm width and 250 mm depth can be used. The number of 5 mm diameter wires is 7. The shear links should be provided at 100 mm centers of 6mm diameter. The slabs will be of 65 mm thickness and each 1m wide panel will need 13 numbers of 5 mm diameter pre-stressing wires.

3.4.2 Comparison for column

The columns of insitu connection will be 200 mm x 200 mm provided with 4 numbers of 12 mm diameter bars. The concrete and steel quantities can be compared as given in Table 3.1.

3.4.3 Comparison for beam

The beams of reinforced concrete construction consist of 250 mm deep and 225 mm wide beams. The reinforcement will be 12 mm and 16 mm diameter bars. The material quantities are given in Table 3.1. The beams of precast construction are 250 mm deep and 150 mm wide.

3.4.4 Comparison for slabs

The reinforced concrete slab consists of 125 mm thickness and 10 mm diameter reinforcement. The precast slab will have a thickness of 65 mm and a screed of 45 mm provided with a 6 mm diameter mild steel mesh at 200 mm centers to prevent shrinkage cracks and also to create monolithic behaviour for panels so that the individual panels will now behave as one unit. The comparison for materials is given in Table 3.1.

Table 3.1: Comparison of material quantities

Item	Insitu construction			Precast construction		
	Concrete (m^3)	Steel (kg)	Formwork (m^2)	Concrete (m^3)	Mild steel (kg)	Pre-stressing steel (kg)
Columns	1.44	182	29	1.9	48	70
Beams	3.6	322	32	2.5	43	147
Slabs	9.45	800	70	8.3	185	155

3.5 Summary

In terms of construction, use of precast systems could lead to significant benefits. The main advantage of the precast panels is the rapid construction also no need for formwork and falsework. This indicates that in addition to rapid construction that can be achieved with precast construction, it can bring about a significant saving for the steel used for slabs and some saving for the beams and columns. The use of less labour is also useful in the current context where the labour is expensive and also in short supply. It can also be seen that the system could resist both axial and lateral loads satisfactorily and hence, the precast system is structurally feasible.

4 EXPERIMENTAL PROGRAM

4.1 General

The experimental program was designed to assess the structural feasibility of using EPS based light weight concrete in wall panels. Since it is expected to use these wall panels as non-load bearing ones, the density could be lower than the densities desired for light weight concrete in structural applications. Hence, the research was aimed at identifying a mix that will provide a wall panel with a density in the range of 600-700 kg/m^3. However, since the strength of the panels directly depends on its density, the structural performance has to be assessed.

As far as the selection of a mix is concerned, since the possibility of replacing new EPS with mechanically recycled EPS is also under study, samples were prepared with only new EPS (Sample 1) and 50 % of the EPS requirement being replaced by mechanically recycled EPS (Sample 2).

The fresh EPS was produced by expanding the fresh polystyrene beads in a steamer. The EPS prior to processing and after processing is shown in Figure 4.1 and the steamer is shown in Figure 4.2

Figure 4.1: Polystyrene before and after processing

Figure 4.2: Steamer used to expand the polystyrene

Figure 4.3: Waste polystyrene

The mechanically recycled EPS was produced by sending the waste polystyrene shown in Figure 4.3 through a machine, as illustrated in Figure 4.4. The end product is illustrated in Figure 4.5.

Figure 4.4: Recycling machine

Figure 4.5: Recycled EPS

The size of mechanically recycled EPS bead was around 5 mm, whereas the fresh EPS shown in Figure is slightly smaller at around 3 mm.

In order to determine the strength parameters, two different types of wall panels were cast. The first type (Type A), represents the actual wall types that is to be used for construction work and is illustrated in Figure 4.6. The EPS based light weight concrete is cast between two asbestos free cement fibre boards. Since the boards are asbestos free, it can be used for construction without any health related concerns.

Figure 4.6: Type A panel

Figure 4.7: Type B panel

The second type of panels (Type B) was cast without the cement fiber boards. This was cast for comparison purposes to assess the structural behaviour of the EPS based light weight concrete under unconfined conditions. A Type B panel is illustrated in Figure 4.7.

The panels were cast in thicknesses of 75 mm, 100 mm and 150 mm. However, all the panels had a width of 600 mm and a height of 2400 mm. The dimensions of the panels were limited and governed by the machine and equipment used for casting. The machine set up at the construction plant is shown in Figure 4.8.

Hence, the experimental program could be summarized as below.

- Comparison of density between a mix with new EPS and a mix with 50 % of the EPS as mechanically recycled while having the same mix proportions.
- The testing of wall panels for compressive strength and buckling characteristics.
- The testing of wall panels for flexural strength.

Figure 4.8: Machine setup at production plant

4.2 Selection of a mix

The wall panels are to be cast using cement, sand, fly ash, EPS and water. Also, one main objective of the procedure was to produce a mix with a density in the range of 600-700 kg/m³. Hence, the main consideration in this step was to identify a mix with a density in the above range while exploring the possibility of using mechanically recycled EPS. The identified mix is given below in Table 4.1.

Table 4.1: A mix proportion giving a low density

Materials	Content (kg/m^3)	By weight
Cement	380	41.4%
Sand	136	14.8%
Water	282	30.7%
EPS	22	2.4%
Fly ash	98	10.7%

As evident from the Table 4.1, cement has the highest content, thus making a significant contribution to the strength of the mix. On the other hand, EPS has the lowest content, but also has the highest percentage by volume. Therefore, the light weight EPS would dominate the volume of the mix and thereby provide a mix with a low density. Therefore, a compromise between density and strength could be achieved by the proportions of cement and EPS.

In order to identify the effect of the addition of mechanically recycled EPS on the density of the mix, a comparative study based on density was carried out. Cubes were cast in moulds of 150 mm x 150 mm x 150 mm for this study.

At the yard where the wall panels are actually constructed, the materials are mixed in a machine. Also they are mixed in a certain sequence. Hence, the same procedure was followed at the laboratory to ensure the actual situation during production is represented while preparing the moulds. This is an important consideration since the results of the experimental program need to be compatible with the actual conditions. As far as mixing is concerned, first, a certain amount of sand, cement and fly ash were mixed with water until it formed a consistent mortar mix. To this mix, EPS beads were added along with the rest of the sand, cement, fly ash and water. This entire mix was mixed thoroughly in order to achieve a homogeneous mix. This complete mixing process took between 7 to 10 minutes. The mix was then cast into the moulds. As far as compacting is concerned, the cubes were hand compacted since machine compaction could lead to segregation of the mix, especially due to the EPS. The cubes were removed from the mix after 24 hours, and then kept immersed in water for 7 days. After 7 days, the cubes were taken out of the water and was left to dry for several hours. Then the weight and the dimensions of the cubes were measured in order to calculate the density of the cubes. The above procedure was carried out for Samples 1 and 2. Figure 4.9 shows one such EPS based light weight concrete mix.

Figure 4.9: EPS based lightweight concrete

4.3 Testing of compressive strength

Although the walls are expected to act as non-load bearing walls, it is important to assess its performance when subjected to axial compression. Failure under compression could be of two forms: crushing or buckling. Especially, given the dimensions of a full panel, buckling effects could be anticipated when loaded axially. Therefore, the possibility of studying the structural behaviour of the EPS based light weight concrete could be affected. This is particularly critical since there could be some degree of shortening due to the compressibility of EPS. Also other critical indicators such as stress-strain behaviour and confinement effect due to the presence of the cement fibre boards may also get affected by the buckling effects and may not reflect the actual behaviour of the composite material.

Therefore, in order to overcome this potential shortcoming, the testing of compressive strength was carried out in two stages. Firstly, short panels with a height of 690 mm were tested, in order to study the structural behaviour. Secondly, full panels were tested to assess the buckling effects. In both cases, the width and the thickness of the panels were 600 mm and 100 mm, respectively.

4.3.1 Testing of short panels for compressive strength

Short panels tested this way had a slenderness ratio of 6.9 (height/thickness = 690/100) and therefore the possibility of slenderness effects dominating the behaviour of the panels was low. Short panels were obtained by cutting each full panel in to three parts. The instrument set up is shown in Figure 4.10. The deformations at the top and bottom were recorded by using dial gauges which were fixed at the top and the bottom. The loads were applied at steps of 0.5 tons using a universal compression testing machine. Dial gauge readings at each 0.5 ton intervals were recorded during the experiment, in order to determine the deformations. Additionally, the panels were carefully observed to detect any hint of failure, especially in the form of cracking and crushing. Both Type A and Type B panels were tested in order to compare the behaviour of the short panels with and without the cement fibre boards.

Figure 4.10: Short Wall panels being tested for compressive strength

4.3.2 Testing of full panels for compressive strength

The full panels had a slenderness ratio of 24 (height/thickness = 2400/100) and hence, slenderness effects were anticipated. Since full panels will be used for actual construction, the behaviour of the panels during this test would provide a more realistic indication regarding its performance. The instrument set up is illustrated in Figure 4.11.

The panels were placed between a special steel frame and the loads were applied at a constant rate using a hydraulic jack. Placing of the panel was carried out carefully, to ensure the loads were applied symmetrically, without any eccentricities. The light-weight nature of the panel was helpful during the setting up process. While the panels were loaded, the behaviour of the panels was carefully observed to identify the mode of failure. Both Type A and Type B panels were tested under this procedure.

Figure 4.11: Full wall panel being tested for compressive strength

4.4 Testing of panels for flexural strength

Testing of panels for flexural strength is critical because the panels are expected to resist lateral loads, predominantly in the form of wind loads, when used as external wall panels. The flexural test was carried out according to ASTM C78: Standard Test

Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) (ASTM, 2016). The instrument set up is illustrated in Figure 4.12.

Figure 4.12: A Wall panel being tested for flexural strength

The panels were placed horizontally and loaded with a hydraulic jack. The behaviour of the panels was carefully observed to determine the mode and location of failure. A similar procedure was carried out for both Type A and Type B panels.

4.5 Summary of the experimental program

The experimental programme was designed to identify two key parameters related to EPS based light weight concrete: density and strength. The summary of the experimental program is given Figure 4.13 below.

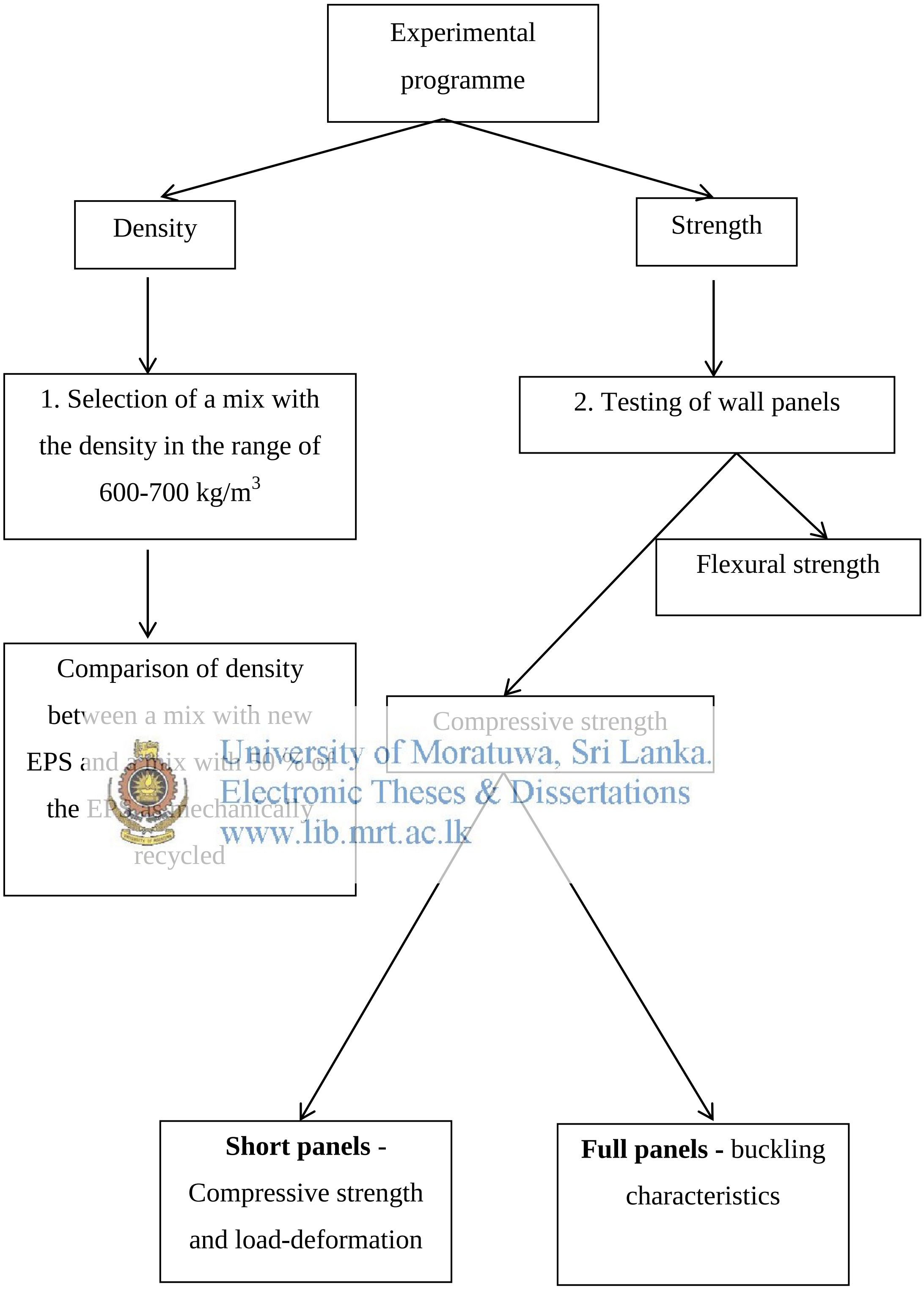


Figure 4.13: Summary of the experimental programme

5 RESULTS AND ANALYSIS

5.1 General

The results of the experimental program were interpreted to assess the structural feasibility of the proposed wall panel. The key parameters that required interpretation were the density, the behaviour under compressive and flexural actions. Furthermore, the results of the compression and flexural strengths were compared with other common wall construction techniques to determine its suitability.

5.2 Selection of a mix

As explained in the experimental program, the EPS based light-weight concrete was produced and cast into cubes in order to determine its densities. The results of the density for Samples 1 and 2 are presented in Table 5.1.

Table 5.1: Density of samples

	Density (kg/m^3)	Average Density (kg/m^3)
Sample 1- 1	711	
Sample 1-2	741	738
Sample 1- 3	761	
Sample 2- 1	628	
Sample 2- 2	653	629
Sample 2- 3	607	

The average density for each sample clearly indicates that the addition of mechanically recycled EPS would provide a mix with a lower density. In this case the reduction in density is about 15%.

Figure 5.1 illustrates the macro nature of the concrete with respect to Samples 1 and 2. As can be seen, both the mixes appeared to be homogeneous with an even spreading of fresh EPS and mechanically recycled EPS in Sample 2. This is an encouraging observation since the EPS was expected to replace coarse aggregates in performing the role of infill material, when compared to normal concrete.

Figure 5.1: Sample 1(LHS) and Sample 2(RHS)

5.3 Testing of compressive strength

The experimental program for the compressive strength was carried out to determine two main features regarding the behaviour of the EPS based light weight concrete and also of the panels. The results of the short panels were used to determine the load deformation characteristics of the panel while the results of the full panels were used to assess the performance of the individual panels under actual conditions. In both cases, the performance of the panels with and without the cement fibre boards was also compared.

5.3.1 Testing of short panels

It was observed that the failure mode of the panels was crushing at the top or at the bottom. As far as the failure of the panels was concerned, both Types of panels failed gradually. This is an encouraging observation because this indicates that the EPS based light weight concrete has the ability to carry and transfer loads. Also the fact that there is no sudden failure is a desirable characteristic of a wall panel. Also, the Type A panels did not disintegrate and hence, indicated that the bond between the cement fibre boards and the EPS based light weight concrete is strong enough to maintain the composite action until failure. The failure patterns are illustrated in Figure 5.2

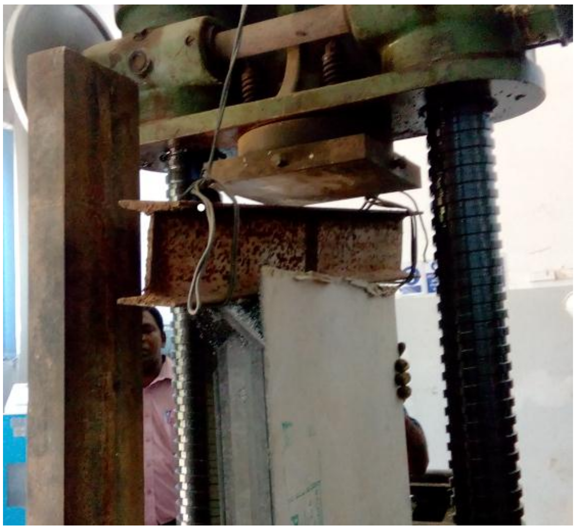


Figure 5.2: Failure of short panels during the compression test

A summary of the results is presented in Table 5.2.

Table 5.2: Test results of the compression test on short panels

Panel	Panel number	Failure load(kN)	Failure stress(N/mm^2)	Average stress(N/mm^2)
Type A	1	237.4	3.89	4.06
	2	247.2	4.05	
	3	258.9	4.25	
Type B	1	142.4	2.33	2.13
	2	112.8	1.84	
	3	135.4	2.22	

One key observation from Table 5.2 is that the Type A panels has a higher failure stress and thereby a higher load carrying capacity, which is almost twice to that of Type B panels. From the structural behaviour point of view, this could be because of the confinement effects caused by the cement fibre boards or the ability of the cement fibre boards to carry some part of the load or a combination of the above reasons. But it is clear that the presence of the cement fibre boards has a beneficial effect on this light-weight wall panel.

Since the compressibility of the EPS based light weight concrete could be a concern, the load deformation characteristics of the short panels was analysed to determine the degree of shortening due to the influence of the axial load. The load deformation curves were generated based on the obtained results. Figure 5.3 illustrates one such curve for Type A panels.

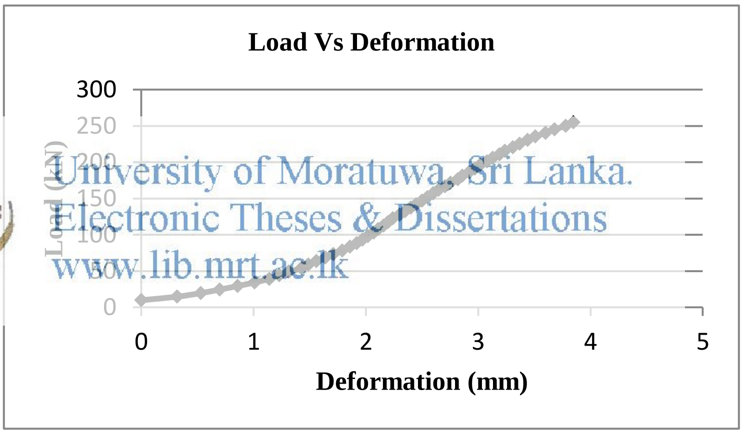


Figure 5.3: Load Deformation curve for short Type A panels

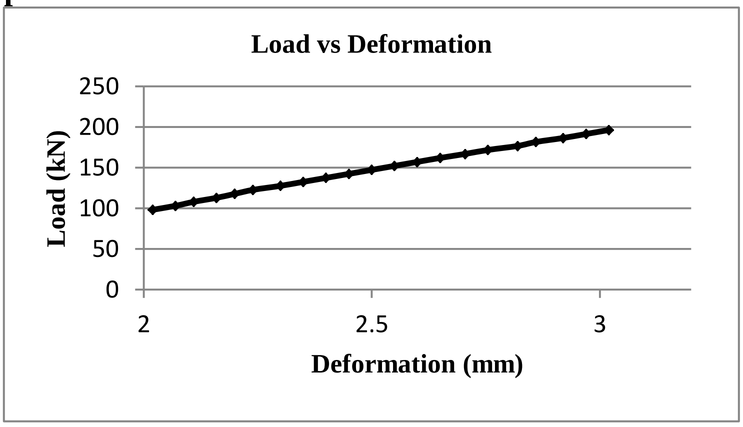


Figure 5.4: Linear portion of the Load Deformation curve for short Type A panels

The initial part of the graph indicates a significant deformation, which is then followed by a relatively linear portion. The significant deformation at the beginning could be attributed to the different types of deformations that may take place in cross heads prior to the loads being applied properly to the panel. Therefore, this part of the graph will not be of significant concern, when assessing the performance of the EPS based light weight concrete. Hence, the linear portion of the graph is illustrated in Figure 5.4 could be used.

During the experiment, it was observed that the cross section of the short panels did not experience significant changes. Hence, it is reasonable to assume that the cross section of the panels remained unchanged throughout the time the panel was deforming. As evident by Figure 5.4, the panel shortened by 1mm, when the applied load increased from 100 kN to 200 kN. For a cross section of 600 mm x 100 mm, this would amount to an elastic modulus of 1.15 kN/mm^2. This is relatively low, when compared to the elastic modulus of bricks and cement blocks.

Figures 5.5 and 5.6 shows the load vs. deformation curve for Type B panels and it can be seen that the variation takes a similar form to that of Type a panels. As far as the elastic modulus of the linear portion is concerned, a shortening of about 1 mm when the load is increased from 50 kN to 115 kN would give an elastic modulus of 0.75 kN/mm^2. Therefore, it is clear that the presence of the cement fiber boards provide an additional stiffness to the panel, which is reflected by the higher elastic modulus of Type A panels.

Therefore, it can be said that the cement fibre board not only adds to the load carrying capacity of the panel, but also provides an additional stiffness to it. However, given the relatively low values for the elastic modulus, it is important to determine the performance of the full panel under working stress conditions.

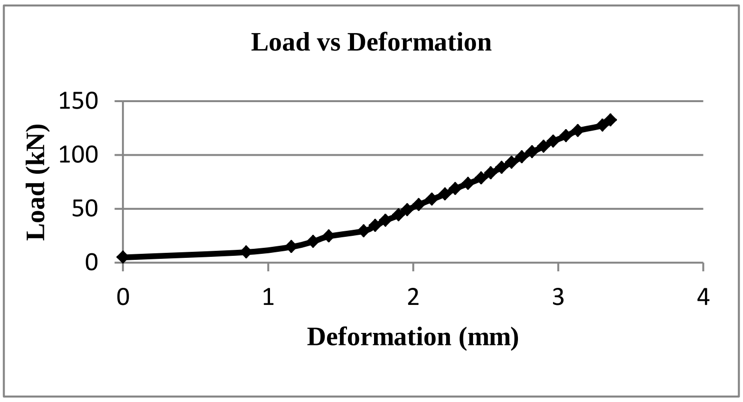


Figure 5.5: Load Deformation curve for short Type B panels

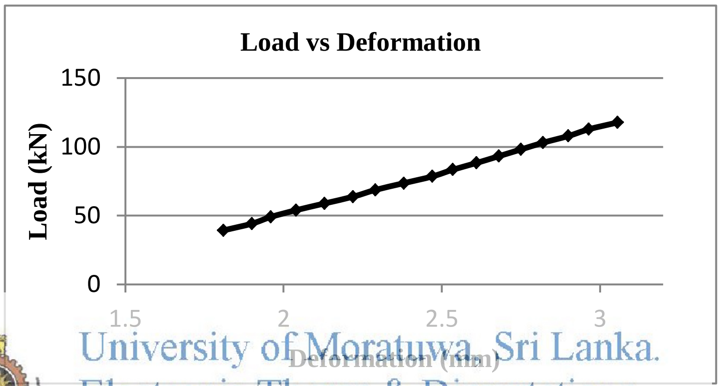


Figure 5.6: Linear portion of the Load Deformation curve for short Type B panels

5.3.2 Testing of full panels

As was the case with the short panels, it was observed that the failure mode of the full panels was by crushing at the bottom as shown in Figure 5.7. This is could mainly be due to the self-weight of the panel. However, despite a high slenderness ratio, the panels did not fail by buckling. The results of the experimental program are summarized in Table 5.3. The results further prove that the cement fibre boards provide additional robustness to the wall panels.

Table 5.3: Test results of the compression test on full height panels

Panel Type	Panel	Failure load(kN)	Failure stress (N/mm^2)	Average stress(N/mm^2)
Type A	1	186.3	3.05	2.89
	2	166.7	2.73	
Type B	1	98.0	1.61	1.69
	2	107.8	1.77	

Figure 5.7: Failure due to crushing at the bottom in the full panels

Since, the wall panels failed by crushing at the bottom, it would be necessary to compare the failure stress with the expected stress at the bottom level due to the self-weight of the panel. In this case, only the self-weight was considered because the panels will be used as non-load bearing walls. The weight of a 100 mm thick full panel was about 90 kg and hence, the stress at the bottom due to the self-weight was about 0.015 N/mm^2. Therefore, both Type A and Type B panels have a significantly high factor of safety, when used for non-load bearing purposes.

Axial shortening of the full panel due to the self-weight is another important parameter. Based on an elastic modulus of 1.15 kN/mm^2 obtained by the testing of short panels and assuming a purely linear variation between stress and strain, a stress of about 0.015 N/mm^2, would result in an axial shortening of only 0.03 mm. Hence, it is unlikely to be significant despite having a relatively low elastic modulus.

5.4 Testing of panels for flexural strength

It was observed that all the panels displayed brittle failure. Under actual conditions, the absence of prior warning could be of significant concern. Failed panels are shown in Figure 5.8.

Figure 5.8: Failure due to flexure

The failure location of all the panels, both Types A and B, were within the middle third of the panel and hence, equation found in ASTM C78- Simple beam with third-point loading was used to determine the stress at failure (ASTM,2016).

$R=Pl/bd^2$

Where

R = MOR in Mpa

P= Maximum applied load indicated by the testing machine in N

l= Span length in mm

b= Average width of the specimen in mm

d= Average thickness of the specimen in mm

The results of the experimental program are provided in Table 5.4.

Table 5.4: Test results of the flexural test on panels

Panel Type	Panel	Failure Location	Failure load (kN)	Failure stress(N/mm^2)	Average stress (N/mm^2)
Type A	1	Within middle third	3.6	1.7	1.8
	2		3.3	1.56	
	3		4.9	2.15	
Type B	1	Within middle third	0.3	0.42	0.5
	2		0.5	0.53	
	3		0.6	0.54	

From the above table, it is evident that the Type A panels with the cement fibre boards has a significantly higher flexural capacity than masonry. The flexural strength values used in BS 5628: Part1:1992 was 0.4 N/mm^2 parallel to bed joints and about 0.9 N/mm^2 perpendicular to bed joints (Jayasinghe et al., 2016).

5.5 Summary

The summary of the results could be presented as follows:

With these results, it can be stated with confidence that the foam concrete panels have adequate compressive strength and flexural strength to be used as a partition panel. In the case of single storey construction, the panel would be strong enough to resist the weight of the roof as well. A summary of the test results are presented in Figure 5.9.

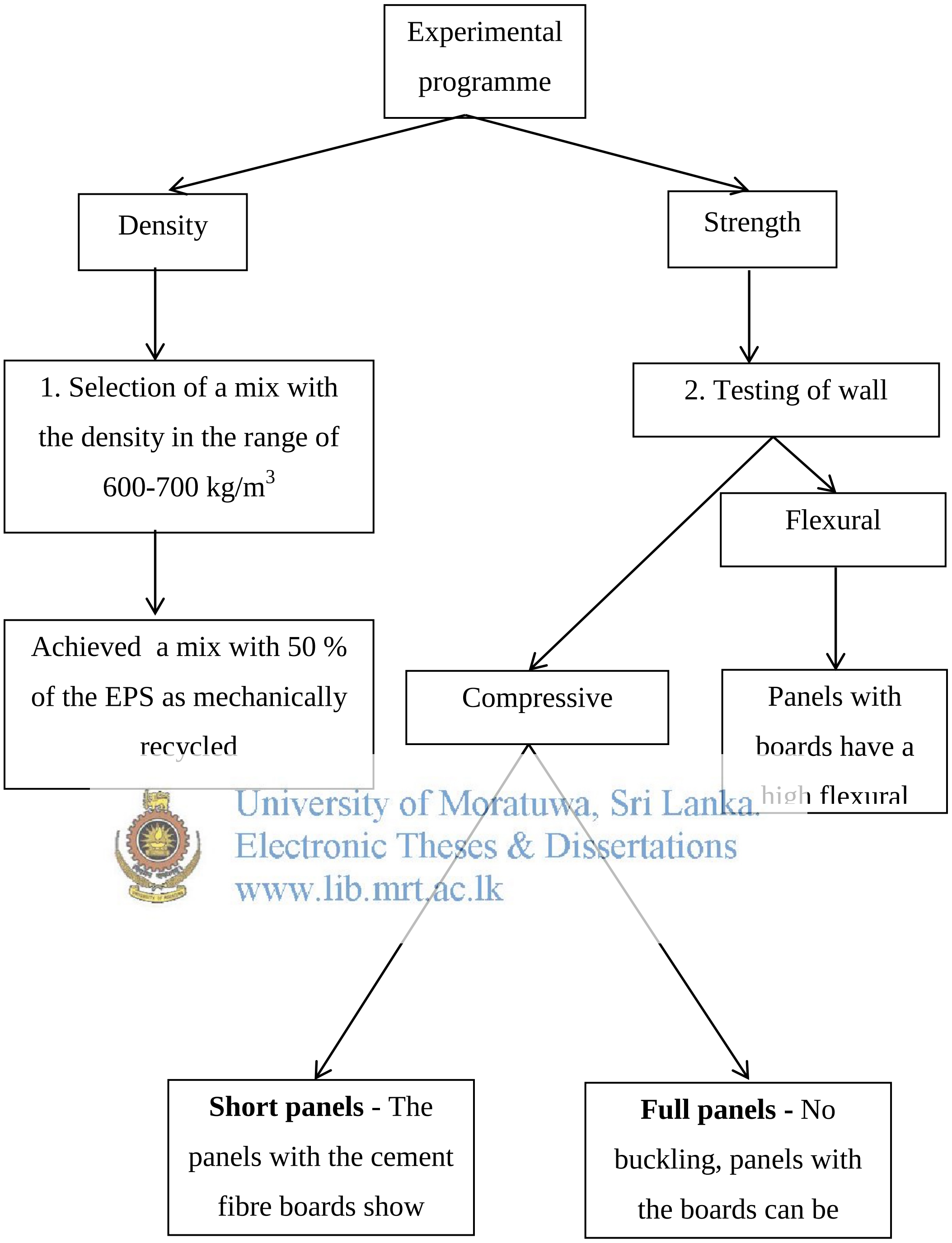


Figure 5.9: Summary of the test results

6 CONSTRUCTABILITY AND APPLICATIONS

6.1 General

As identified by the experimental program, the proposed EPS based light weight concrete sandwich panels can be used as non-load bearing elements. However, the tests of the individual panels would only provide part of the full picture and hence its constructability and applicability needs to be studied in detail.

6.2 Constructability

One main aspect that needs attention is the connectivity of the panels. As shown in Figure 6.1, the tongue and groove arrangement in each of the panels would facilitate the inter-connection between panels. In addition, a cement based grout could be used in these tongue and groove joints. Other important points of connection are at the slab level and at the soffit of the beam. These connections could be provided by using galvanized steel components. The dimensions of these steel components could be customized based on the requirements during construction.

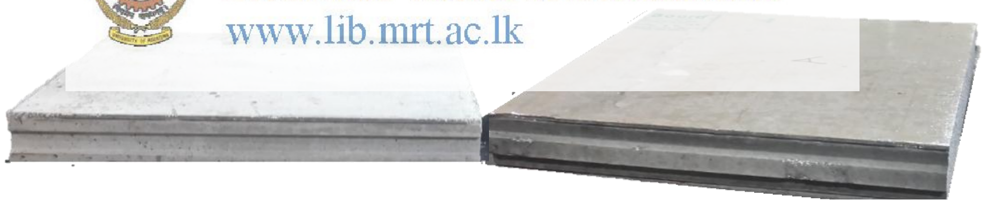


Figure 6.1: Tongue and groove arrangement

Also, given that the weight of a 100 mm thick panel is less than 100 kg, two labourers should be able to carry and manoeuvre it. This will be helpful during casting, loading, unloading of panels and also during construction.

6.3 Applicability

Although the panel is strongly recommended for non-load bearing partition walls in single storey houses, due to its high margin of safety during the compression test, it can be said with confidence that the panels will be able to carry the load of a typical roof with insulation. Furthermore, due to sufficient flexural strength, the panels could be used as ceiling panels along with a Zn-Alum sheet covering. Even in this case, the wall panels will be able to carry the weight of the roofing system due to its relatively low weight.

Another key aspect that requires attention in terms of the panel's applicability is the performance of the proposed wall panel in the event of a fire. The presence of the cement fibre boards which will resist the fire to a greater extent.

However, the effect of EPS on fire resistance will also have to be studied before using such panels for construction applications. The fire resistance of EPS based concrete largely depends on the density of the concrete. In effect, this is an indication that the density would have a significant bearing on the fire resistance because the density of the light weight based concrete is significantly influenced by the volume of EPS. According to Sayadi et al., an EPS based concrete mix with a density of 400 kg/m^3 with an EPS volume of about 45% did not show significant failure even after 3 hours, when subjected to the fire test (Sayadi et al.,2016). Since the density of the mix used for the proposed panel is in the range of 600-700 kg/m^3, the EPS based concrete mix would be expected to perform satisfactorily in the event of a fire.

Also, in case the fire penetrates through the wall and the concrete mix catches fire, its effect on the wall's strength would be minimal. This is because, the EPS only acts as an infill material and the strength of the panel is provided by the mortar skeleton.

Other potential applications of this light weight wall panels are as partitions for apartment buildings, hotels, commercial buildings etc. This would be beneficial in terms of ease of construction due to repetitive work. Also structurally, using of these panels would reduce the self-weight of the building, which would enable using of smaller section sizes and less loads at the foundation level. Also due to the low mass of the buildings, the dynamic effects due to cyclic loads such as earth quakes will be less.

6.4 Constructability of a house with EPS wall panels

As explained in the previous chapter, the structural testing results and the behaviour of the proposed EPS wall panel system reveals that it could be recommended for non-structural applications such as partitions walls in a building. However, due to the encouraging test results of these lightweight wall panels, the possibility of using these panels in structural application, i.e. as load bearing walls in a building with relatively low loads (for example a single storey house) could be explored. In addition to the sufficient margin of safety in terms of the compressive and flexural strengths of individual wall panels, the confinement effect created in an assembly of panels must also be taken into consideration in such a scenario. This chapter covers a case study related to the construction of a single storey house using the proposed EPS wall panel system. The house has a total area of 98 m^2 while the total wall area is 242 m^2.

6.4.1 Initial planning

Houses constructed in this form falls under the broader category of modular houses. Hence, during the planning stage, the designer is expected to incorporate critical aspects related to the wall panel into the proposed building. For example, since, the panel has a width of 600 mm, having wall dimensions in multiples of 0.6m would ensure convenience during construction. However, the tolerance due to the thickness of the panels at the interconnecting joints should also be taken into account. In addition, the shape of the building should also be taken into account, since achieving certain shapes (for example arch shaped openings) may not be feasible with the proposed rectangular shaped panels. All these aspects could be broadly categorized as ‘panel layout’ and hence, it is important to identify a suitable layout during the planning stage.

Also, the sizes of compartments within the house could affect the confinement effect of the panels. Smaller compartments would create a more robust structure. However, a compromise may have to be reached between this structural consideration and other non-structural considerations when deciding on plan dimensions.

The height of the walls is also a critical factor. If improperly planned, there could be wastage of around 10% of wall panels when used during actual construction. However, due to attention to detail, this particular house has only had wastage less than 1%. For example, a concept similar to 'cut and fill method' can be incorporated when constructing Gabel walls, to ensure, reusing of the panel off cuts. Although the height of the walls would be primarily governed by non-structural factors such as free ventilation space, aesthetics etc. the structural implications of this decision should also be considered. The vertical cantilever effect of walls during erection, number of connection joints, and requirements for staggering, requirement of gabel walls to support the roof structure will all depend on the height of the walls. The maximum height of the walls panels in this house is about 4.5m. Additionally, there are three types of gabel walls which vary in height 2.7 m to 3.9 m, 2.7 m to 4.2 m and 3.5 m to 4.5 m. One such gabel wall is illustrated in Figure 6.2.

Figure 6.2: Gabel walls

It is also important to decide on the materials used for each element in the house as this would contribute significantly to both structural (Ex: weight and connections) and non-structural (Ex: aesthetics, thermal comfort) aspects. Table 6.1 summarizes the elements of this house.

Table 6.1: Materials used for each element

Element	Material(s)
Foundation	Random rubble with cement block work
Intermediate beams at foundation level	Reinforced concrete
Screed	Trowel concrete- Grade 25
Wall panels	EPS panels with asbestos free cement fiber boards on either sides
Lintels	Reinforced concrete
Roof	EPS panels with asbestos free cement fiber boards on either sides
Purlins	ICC inverted T purlins
Roof covering	Zinc-Alum sheets

6.4.2 Foundation

As is the case with normal construction, the soil type and its properties, along with the expected loadings would govern the foundation design. Although there were marshy areas adjacent to the site, a hard lateritic soil was found underneath the overlying soil. The level of the ground water table was about 0.6 m. As far as loading is concerned, the expected total weight of the house is less because of the light weight of the panels. Considering all these factors, the settlement effects would be insignificant and a random rubble foundation with cellular hollow cement blocks can be used. The width and the depth of the foundation are about 300 mm and 75 mm respectively. A typical arrangement of the foundation is illustrated in Figure 6.3.

Additionally, intermediate beams of 75 mm x 75 mm were placed along wall corners and locations along which the length of the wall panels exceeded 3 m. The reason behind this was to prevent slipping of the floor concrete due to possible expansion.

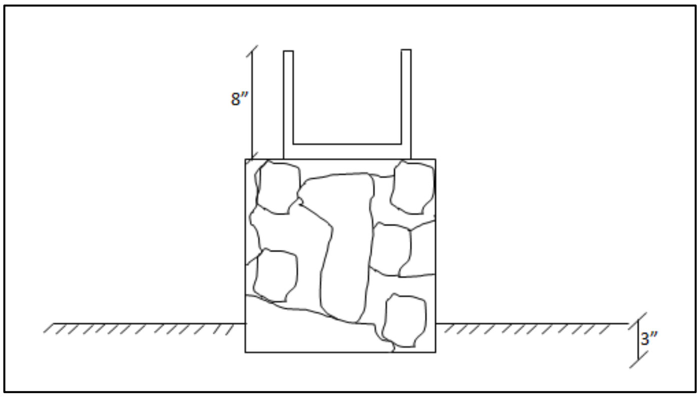


Figure 6.3: Typical arrangement of the foundation

6.4.3 Ground Screed

As shown in Figure 6.4, a ground screed of 65 mm thickness was placed and it was trowel finished. However, this also meant that additional considerations such as deciding and placing of plumbing, electrical and ground supply requirements at the ground level were carried out at this stage. Termite treatment was not used due to the soil conditions as well as the presence of the cement fibre boards in the wall panels.

Figure 6.4: Ground screed

6.4.4 Wall Panels

One main advantage of these kinds of panels is that it reduces the weight of the building, thus less dynamic effects in the event of an earthquake. However, its performance in the event of a sudden impact load could be questionable, although the confinement effect would mitigate it to a greater extent.

The location of the walls was identified and the required wall heights were determined. The panels were cut and placed accordingly. As far as cutting of the wall panels were concerned, a technic called ‘water cutting’ was employed. This was due to the high friction of the cement fibre boards and the dust caused by cementitious materials in the panel. One such instance of cutting is illustrated in Figure 6.5.

Figure 6.5: Cutting of a wall panel

As far as connections of panels are concerned, the connections at the bottom, top and at the sides are of significant importance. At the bottom level, the panels were fixed to the ground using an angled shaped R6 bar. Its dimensions are illustrated in Figure 6.6. However, initially it was planned to use L angled sections, but during construction it was found out that the using of such sections could damage the wall, because the bar did not enter smoothly, when hammered in.

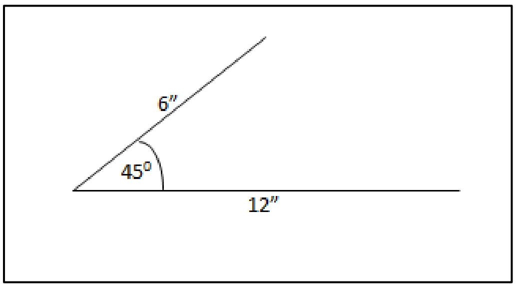


Figure 6.6: Angled connection

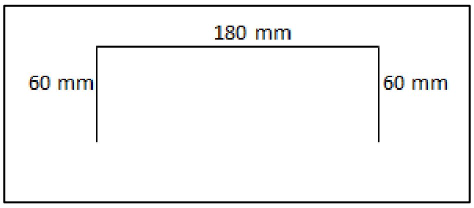


Figure 6.7: U connection

U shaped bars were used to connect the panels at the top. This is illustrated in Figure 6.7. However, it had been noted that some panels, especially the ones at directly under the Gabel walls tend to wobble, even when these types of connectors were used. Therefore, as illustrated in Figure 53, a cross pin arrangement was employed. These bars were expected to serve as temporary fixing agents and hence would allow a certain amount of movement if needed at the bottom. This would be useful for aligning purposes. Also, these steel connectors were exposed to the outside and hence, carry the risk of corrosion. However, this would not be of major concern because these connectors serve a temporary function during the construction period, which is relatively short. But the main form of connection is through the tongue and groove arrangement at the sides of the panels.

In addition, a Tokyo Superbond tile adhesive was poured in between the tongue and the groove to enhance the connection.

An important consideration is the height of the walls. Since all the walls have a height greater than 2.4 m, there will be connection joints at the vertical direction too. To ensure no weak planes along the wall, the wall panels were staggered although literature suggests that staggering is not required if the wall height is less than 3.0 m

As illustrated in Figure 6.8, in addition to placing wall panels vertically, at certain locations, they were also placed horizontally. Generally, placing of horizontal panels is not recommended, because the connections between the panels won't be as strong due to the absence of the tongue and groove arrangement. Therefore, there could be movements and deflections, which could lead to a weak plane in the wall. However, these adverse effects could be minimized if the panels on either side (top and bottom) of the horizontal panel are well connected to its adjoining member.

During construction, it was observed that the walls tend to sway en bloc, if a force was exerted on them. The walls which acted as vertical cantilevers had a sway of about 15 mm at the top. However, once the entire wall system was completed, the robustness of the wall due to the confinement effect reduced this sway to around 7 mm. However, this sway is expected to be 0 mm, once the roof is constructed, due to the support action from the roof at the top of the walls. Therefore, this highlights the importance of studying the properties of the system as opposed to a single panel, when determining the performance of such wall panel systems.

Figure 6.8: Placing of horizontal panels over vertical panels

Additionally, it must also be noted that free standing walls will not be feasible, while using this system, because such walls would eliminate the confinement effects. However, it is possible to have such walls if steel U sections are placed on top of such wall panels. But, the discontinuities due to the openings such as doors and windows will not be critical because the Aluminium frames would transfer the loads.

Walls with an inclined face were a special feature incorporated to the walls. These were convenient to implement due to the panelled wall system. As shown in Figure 6.9 walls with inclined faces are used as a passive technic of providing shading as well as lighting to the house. One of the edges is inclined and hence, construction of such walls with conventional technics like bricks and cement blocks could be very difficult.

Figure 6.9: Wall with an inclined face

Figure 6.10: A lintel for a large opening

Reinforced concrete lintels were placed on top of the wall panels at locations where the width of the openings are greater than 2m. In this house, there was one such location, and this is illustrated in Figure 6.10.

6.4.5 Roof

The 75mm thick EPS panel was used as a ceiling material. This would further reduce the self-weight of the building. However, this could adversely affect the cyclone resistance of the building. Therefore the panels would have to firmly fixed to the purlins.

6.4.6 Further recommendations based on the construction work

Certain practical issues were identified during the construction of this house and these could be used to make recommendations for future applications.

Customization of the tongue and groove arrangement based on the requirement at site is one such modification. This was identified in cases where both the connecting panels had grooves and hence, providing a strong connection required the addition of a 'biscuit'. A biscuit, is in effect is a cut made of a tongue and is inserted between the grooves to fill the voids. However, this requires extra work during construction and hence needs addressing. This issue occurred due to the design of the panel. Certain panels were produced with a flat end on one side and a groove on the other (these were used at the bottom with the flat end resting on the ground screed) while the others were produced by having tongue on one side and groove on the other (these were placed on top of the other type of panel). Due to the nature of the two types of panels, there is the possibility of two grooves meeting at a connection joint. One way of overcoming this shortcoming is by producing panels with tongues at both ends. This is a change that needs to be incorporated from the production stage.

The workmen also faced practical issues while cutting panels. This is because three people were required for the cutting process. Given that the panels were erected by only 3 people, the construction came to a standstill during the cutting procedure. This could have been avoided if a clamping system was used to hold the panels in place. Also, adjusting the orientation of the panels was also carried out manually. When placing a panel, it is first set in place at the bottom and then the alignment at the top is adjusted. However, this is a time consuming task. This process could be simplified if a guide rail can be installed at the bottom.

6.5 Summary

Foam concrete based panels are a new concept that has been introduce to Sri Lanka. It can be said that the workmen are still in a learning cycle and hence, there may be room for many improvements. Theses have to be addressed carefully so that the panel based homes will become a robust structure and also gradually lead to it becoming a mainstream walling material.

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

Pre-stressed concrete precast members consisting of slabs, beams and columns have been presented as an alternative to the insitu cast construction. This is gradually becoming a need due to shortage of construction labour. These precast systems also allow the construction to be carried out at a rapid rate.

The structural design aspects related to pre-stressed concrete columns have been dealt with in detail. Column interaction charts also have been presented.

Another system that can be combined with the precast system is foam concrete based panel system. This also can allow rapid construction. In order to assess the structural performance, an experimental program was carried out to identify an EPS based lightweight concrete mix with a density in the range of 600-700 kg/m^3 with 50% of the EPS content being replaced with mechanically recycled EPS. In addition, testing was carried out to identify the strength properties of a wall panel. It is advisable to have the cement fibre sheets for the partition panels though foam concrete on its own would have sufficient compressive strength. One of the key parameters that are important for a wall panel is its elastic modulus in the working stress region. It is shown that this value could be in the range of 1 kN/mm^2 even when the cement fibre boards provide an additional stiffness. Hence, these panels can be recommended primarily for non-loadbearing partitions. One encouraging observation was the ability of the cement fibre boards to retain the composite action until the ultimate loads where the failure was generally due to crushing at the bottom. Even for partition walls, certain degree of robustness is needed. It is shown that when foam concrete is cast between the cement fibre sheets, the flexural capacity of a panel is reasonably high.

With the cement fibre boards and with tongue and groove joint for connections, these panels could allow a rapid construction rate. It is possible to finish the walls without any plaster. It would need a thin fibre based tape at the joint to provide a neat finish.

With the ability to contain up to 50% recycled EPS, the wider usage of this panel could have many benefits environmentally.

7.2 Future studies

- The performance of the wall panels under dynamic loading needs to be studied and the results of a full scale model testing can be used to model its behaviour for future designs.

- The construction of a two-storey house by incorporating both these systems could be studied. However, this would require testing of a real scale model to identify the load transferring mechanism between the pre-stressed concrete elements and the EPS based lightweight concrete wall panels.

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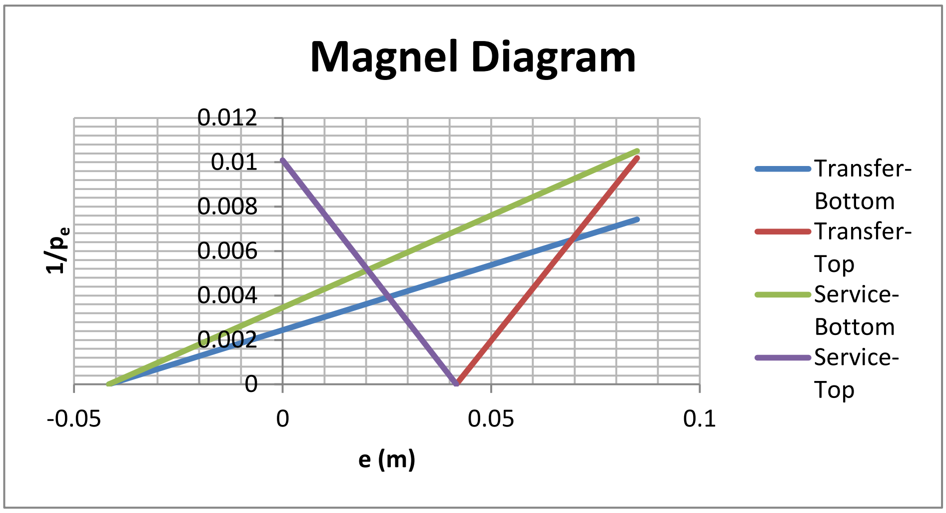
Step 2:

Trial Section sizes were chosen and checked for its adequacy based on the expected loadings.

Section Properties		
Length	4	m
Breadth	0.15	m
Depth	0.25	m
Y bottom(Y1)	0.125	m
Y top(Y2)	0.125	m
Cross sectional area	0.0375	m2
Second moment of area	0.000195313	m4
Section modulus bottom (Z1)	0.0015625	m3
Section modulus top (Z2)	0.0015625	m3
Applied Stresses		
Loads		
Self weight	0.9	kN/m
Super imposed dead weight	5.77	kN/m
Imposed Load	1.58	kN/m
M0	1.8	kNm/m
Mdl	11.544	kNm/m
Mil	3.15	kNm/m
Msmax	16.494	kNm/m
Msmin	13.344	kNm/m
Suitability of Section Sizes		
Achieved section modulus	0.00019631	m3
Result-bottom	HENCE OK	
Result-top	HENCE OK	

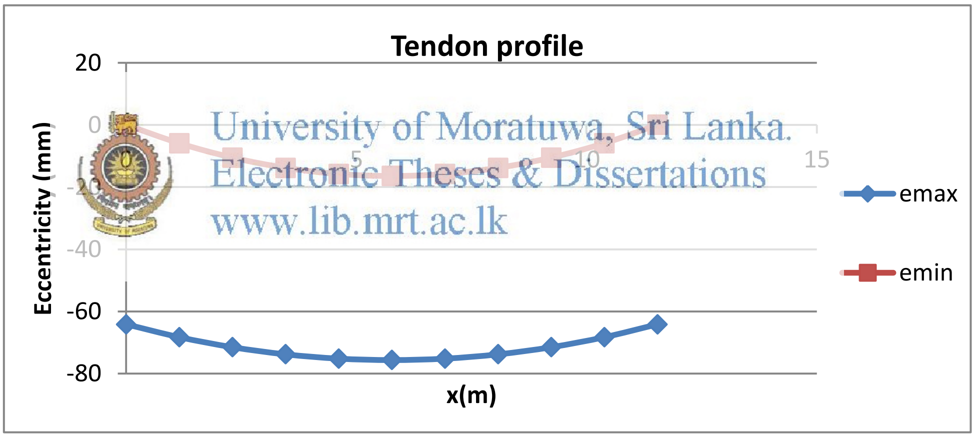
Step 3:

The Magnel Diagram was developed based on the inequalities developed for limiting the tensile and compressive stresses at the top and bottom most fibres at the mid-section. (Where the bending moment and hence the bending stresses are greatest)



Step 4:

A suitable combination for P_e and e were chosen from the Magnel Diagram and the expected tendon profile for this combination was developed in order to determine the possibility of maintaining a linear tendon profile.



Step 5:

The requirement of pre-stressed strands was determined for the chosen P_e and e value.

Number of Tendons		
Selected 1/pe	0.008	1/kN
Selected pe	125	kN
Breaking load of a tendon(pb)	34.7	kN
Number of tendons required	7	
Diameter of pre stress wire/strand/bar	5	mm
Selected e	60	mm

Step 6:

The expected short and long term losses were determined to see if the pre-determined loss ratio is adequate.

Short term losses		
Elastic Shortening of Concrete		
Pj	166.6667	kN
A	0.0375	m2
e	0.06	m
I	0.000195	m4
M	1.8	kNm
fco	6963.484	kN/m2
Esteel	205	kN/mm2
Econc	28	kN/mm2
Δfps	50.98265	N/mm2
Loss due to elastic shortening(Δpse)	6.869575	kN
Long term losses		
Steel Relaxation		
Pj	166.6667	kN
Relaxation factor	1.2	
1000h relaxation test value	2.5	%
Loss due to steel relaxation(Δpsr)	5	kN
Shrinkage of Concrete		
Shrinkage strain	0.0003	
Esteel	205	kN/mm2
Loss due to shrinkage of concrete(Δpss)	8.286718	kN
Creep of Concrete		
Creep coefficient Φ	1.8	
Econc	28	kN/mm2
Esteel	205	kN/mm2
fco	6963.484	kN/m2
Loss due to creep of concrete(Δpsc)	12.36524	kN
Total short term losses	6.869575	kN
Total long term losses	25.65195	kN
Total losses	32.52153	kN
Pi	159.7971	kN
Pe	134.1451	kN
α	83.94717	HENCE OK

A.2 Pre-stressed concrete slab design

Since these panels are one way spanning slabs, the procedure for beams could be followed. However, the effect of the screed concrete must also be taken in to consideration.

Step 1: First the allowable stresses for compression and tension under transfer and service conditions were determined.

Allowable Stresses			
Concrete cube strength(fcu)		40	N/mm2
Strength of Concrete at transfer(fci)		25	N/mm2
Density of concrete		24	kN/m3
Loss ratio		0.8	
Compressive			
At Transfer			
Select Stress distribution type	Triangular stress distribution		
Allowable stress(famaxt)		12.5	N/mm2
Under service loads			
Select loading type	Loaded in bending		
Allowable stress(famax)		13.2	N/mm2
Tensile			
At transfer			
Select class	Class 2		
Select type of pre stressing	Pre tensioning		
Allowable stress(famint)		-2.25	N/mm2
Under service loads			
Select class	Class 2		
Select type of pre stressing	Pre tensioning		
Allowable stress(famin)		-2.85	N/mm2

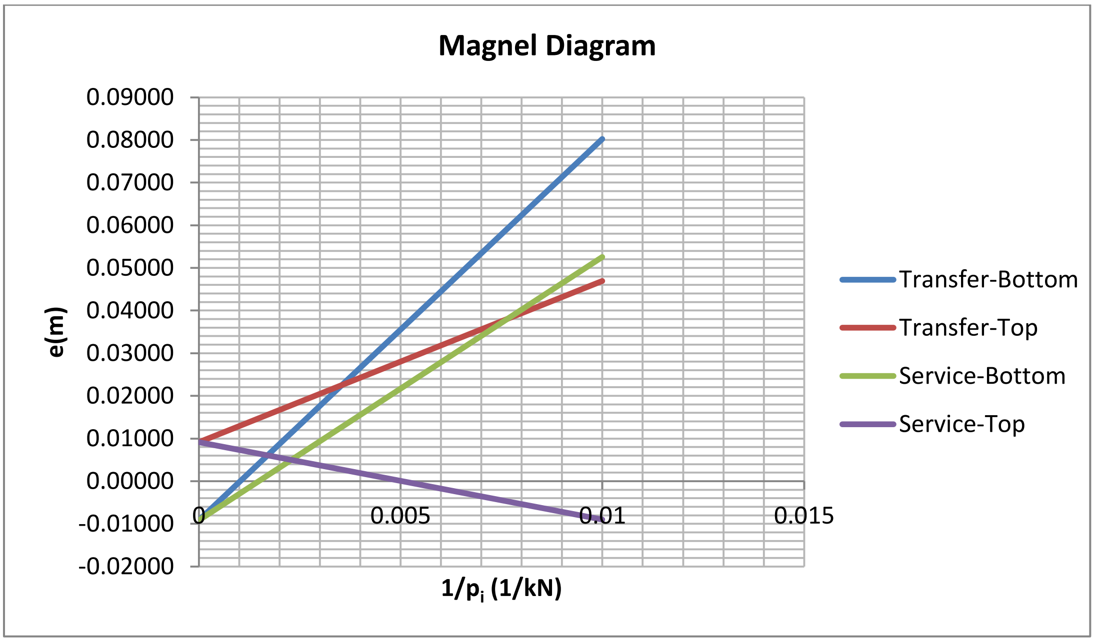
Step 2:

The expected loading was calculated by incorporating the effect of the screed.

Section Properties		
Length	4	m
Breadth	1	m
Depth	0.055	m
Screed thickness	0.05	m
Y top(Y1)	-0.0275	m
Y bottom(Y2)	0.0275	m
Cross sectional area	0.055	m2
Second moment of area	1.38646E-05	m4
Section modulus top (Z1)	-0.000504167	m3
Section modulus bottom (Z2)	0.000504167	m3
Ic	9.64688E-05	m4
Z1c	-0.0385875	m3
Z2c	0.0018375	m3
Applied Stresses		
Loads		
Self weight	1.32	kN/m
Super imposed dead weight_1	1.25	kN/m
Super imposed dead weight_2	0.25	kN/m
Imposed Loads	2.00	kN/m
M0	2.64	kNm/m
Mdl_1	2.5	kNm/m
Mdl_2	0.5	kNm/m
Mil	4	kNm/m
Msmax	9.14	kNm/m
Msmin	5.14	kNm/m

Step 3:

The Magnel Diagram was developed based on the inequalities developed for limiting the tensile and compressive stresses at the top and bottom most fibres at the mid-section. (Where the bending moment and hence the bending stresses are greatest)



Step 5:

The requirement of pre-stressed strands was determined for the chosen P_i and e value.

1/pi	0.0032	
pi	312.5	kN
Breaking load of a tendon	34.7	kN
Number of tendons	13	