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# Analysis and Design of Amphibious House

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*Abstract*— **Flooding is one of the most natural common disaster in India and this phenomenon has been worsening due to current global warming conditions. The disaster cost the government a huge amount of money and effort to handle it in terms of its catastrophe and mitigation strategy. It also has an adverse effect on land value in the floodplain area. Therefore, it is crucial to resort for more effective flood mitigation approach in India. This need has motivated this research to develop a new sustainable, environment friendly and flexible flood mitigation measure known as amphibious house within an amphibious urbanisation environment. This concept enabled the dwellers to live with flood rather than to confront it. The amphibious house system consists of the pit system concept with horizontal support, a specially formulated light weight concrete material, special designed precast pontoon system and precast glass fibre reinforced gypsum panels, which provide floatation to the single storey house during flooding. This research is conducted in phases using qualitative and quantitative data collection approach to develop reliable and stable amphibious house. The initial approach consists of interview with expert panels and using questionnaire survey to establish potential of using the amphibious house as a sustainable flood mitigation strategy. This followed with fuzzy materials selection for developing sustainable lightweight concrete ground floor slab composition and design with ferrocement pontoons composition and design. The amphibious house system has been vigorously analysed to meet the standard floatation and stability requirements in the hydraulic laboratory which includes hydraulic testing for drag forces, tilting angle and point load test on a floodplain model. Finally, this research produces program to assist the designer to analyse the amphibious nature and stability of the amphibious house system. The findings from this research have successfully established a novel concept of amphibious house system. This concept provides potential development of flood proof urbanisation especially in low lying area with very minimum risk to flood disaster.**

## I. Introduction

I.1 Flood vulnerability and flood mitigation strategies:

Fighting and protection against flood vulnerability is one of the most important afford of human species. The destructive impact of flood as a natural disaster is jumped to an alarming rate. The term of the natural disaster is defined regarding to the human impact. Flood vulnerability triggers economic, environmental, and social effect in floodplain area. This means that, people, property, society, and the environment are suffering more and more from flood danger. Climate changes and global warming escalate flood risk and rising of the level of the sea in South India. Increasing the risk of flash-flood generates different approaches and strategies to mitigate vulnerability of this natural disaster. Climate change, land subsidence, and need of space for water are the main reasons resolute the need of smart and sustainable water management policy in India.

Floating houses considered as an approach with the advantage of flexibility in both vertical direction (moving with a fluctuating water level) and horizontal direction to float the buildings whereas, amphibious house limited for horizontal movement. However, both of the approaches have been used in water side locations previously. On the other hand, the first and novel type of amphibious houses might be happened in Malay Peninsula in Malaysia. The wooden houses are built on stilts which rest on, but without being fixed to the ground underneath the house are stacked horizontally, hundreds of bamboos. In addition, each house has four or more wooden poles and ropes are latched onto these poles. During flash flood, the entire community, with its houses, shops, a public pavilion and dog kennels, is automatically afloat. Thus, the integrated and new design of amphibious house which used concrete pontoons and pit system should be developed and applied as a flood mitigation strategy in India.

The benefits of amphibious urbanization are similar to floating urbanization and could be stated as:

- Cost efficiency
- Environmentally friendly
- Easy to construct
- Durability
- Suitable mooring and movement system.

The amphibious house concept developed by this research adapt to the nature of flooding environment. However, there is a need to address the issue of stability of the house during flooding. Pontoon system provides good solution to ensure not only for floatation mechanism but also to provide stability of the house. This floating system has to be designed by using a suitable lightweight material which is strong enough, durable and sustainable. The house floating system need to be tested for its stability with acceptable tilting design tolerance during flood time.

I.2 Aim and Objective of the Research:

The aim of this research is to develop amphibious house system, which can float during flooding by using the special lightweight concrete pontoon within pit system technology in floodplain areas in India. The objectives of this research are:

i. To evaluate alternative sustainable flood mitigation strategies in India.
ii. To evaluate the suitability of using amphibious house system as a flood mitigation strategy in India
iii. To evaluate suitable lightweight concrete material for pontoon design
iv. To develop the conceptual design of amphibious house system
v. To evaluate and analyse the buoyancy and hydraulic reaction on amphibious house system.
vi. To develop a computer assisted system to analyse and design amphibious house.

I.3 Scope and Limitation of the Research

Floods can be categorized in two main types: riverine and coastal. In the case of riverine floods, they are mainly caused by the over flow of channels. Riverine or fluvial floods are common issues in India. Thus, in this research, the amphibious house is developed and evaluated for riverine (fluvial) floods. In general, this research is related to following main area of focus. The first category is floating platform with buoyant compartments. More emphasize is given on the pontoon material selection. The shape of concrete pontoon considered as simple hollow boxes for economic purposes. The fabrication of pontoon component is made using IBS system. This research proposed the especial sustainable lightweight concrete as pontoon. The investigated cementing pozzolans are limited to: fly ash, silica fume, rice husk ash, palm oil fuel ash. Lightweight aggregates are limited to more common and available aggregate in India. The design of the floating platform is suitable for single unit house in the open area.

The research is applied for flat floating platform to mount the house. The seismic consideration is neglected in this research. The internal design of houses is not considered. The next area of focus is the hydro-force on the lateral support system. This research contains a physical model to establish, and analyse required loads for vertical support system. The models validate the lateral stability and anchoring of the house. Hydrodynamic reaction forces in the structure boundary estimate for river (fluvial) floods only in laboratory scale. The other area is the pit area. The research does not cover the detail design for pit system. The conceptual and geometrical designs of the pit with implementation steps are developed based on hydraulic results. Finally, the computer assisted program is developed. This program is able to check the tilting due to point load. It determines the number and pattern of pontoons for different types of house based on their areas and weights.

I.4 Significance of the Research:

The significant of this research is to establish the importance of using amphibious house as a novel and sustainable flood mitigation strategy in India as:

i. This new strategy increases the usability of the low laying land in flood prone area.
ii. It helps to boost the land value in the floodplain area.
iii. This system provides economic and flexible solution for diverse urban societies in flood plain areas.
iv. This system is completely aligned with IBS technology.
v. The research provides novel strategy to mitigate riverine flood in India.
vi. It can reduce catastrophic vulnerability due to flooding and reduce the hassle to evacuate people and valuable things if the situation is merit.
vii. The research provides buoyancy for floating platform by a special designed pontoon system.
viii. The research determines sustainable and the most suitable lightweight concrete material to be used for reinforced concrete pontoons.
ix. The research provides maximum stability for the house and its occupants during the flood.
x. The research establishes the relationship between forces acting and suitable lateral restraining structure to support platform and the house system.
xi. The research provides non-turbulent area for flood water and initiates smooth uplift and settlement of the house by the pit system.
xii. The research develops the computer-assisted software for analysing the loads and design of the platform area and position of the buoyant components.

## II. LITTÉRATURE SURVEY

Permanently elevating houses, in some areas by as much as 12-15 feet, may be solution to the problem of flooding but it creates new problems, such as difficult access to living areas, loss of neighborhood character and increased vulnerability of the structure to wind damage. With permanent static elevation, even if a house is raised to the HFL or higher, it can still flood in an extreme event. In the meantime, residents must live with daily inconvenience and a reduced quality of life in the hope of avoiding flooding in a future event that is statistically very rare indeed. A look at floating docks and houseboats suggests that there may be an alternative approach, one that would allow a house to remain close to the ground under normal conditions but rise as much as necessary, even if far above the HFL, when flooding occurs.

Figure 1 Permanent static elevation of homes

II.1Amphibious Housing

There is a growing number of cost-effective amphibious houses around the world. Best known are the amphibious houses designed and built earlier this decade by Factor Architected and DuraVermeer at

Maasbommel in the Netherlands. Unhampered by building codes in These areas, local residents and vacationing fishermen devised an amphibious foundation system that has been keeping their homes and fishing camps dry for over three decades. Large blocks of EPS (expanded polystyrene, or Styrofoam) are secured underneath the home which has been raised to an elevation 3 - 4 ft above the ground. Long poles or pipes are sunk into the ground near the corners of the house. When flooding occurs, the EPS blocks raise the house. Sleeves that have been placed around the poles and attached to the structural frame of the home are able to slide up and down, allowing the home to rise and fall with the level of flooding.

Figure 2 Amphibious house in Dry Summer, Floating during flood.

Figure 3 Amphibious house in rural Louisiana, the same house in September and February.

II.2FLOAT House in New Orleans

Make It Right (MIR) Foundation in 2007, promising to give to former residents of the Lower Ninth Ward in New Orleans 150 affordable, sustainable and storm-resistant new homes. Morphosis Architects designed the amphibious FLOAT House for MIR. It was completed in October 2009. The base of the house is a "chassis" formed of EPS encased in fiberglass reinforced concrete. It acts as a raft, allowing the house to rise vertically by sliding on two guide posts that pass-through sleeves in the chassis, one at each end, inside the house. The house can float up to twelve feet as water levels rise.

Figure 4 The FLOAT House, New Orleans. Elevation, interior guide post and sleeve details.

II.3 Lakeview Amphibious House in New Orleans

This house in the Lakeview neighborhood of New Orleans was built by a contractor as a commercial spec house. It was completed more than a year ago, but to date it has not been occupied. Due to difficulties with the permitting process, the contractor has been unable to obtain a Certificate of Occupancy. The house appears to be supported on a hollow steel box that provides its buoyancy. The box rests on a concrete slab-on grade. Four wood vertical guidance posts are set near the corners of the house. Each post is attached to the house by two steel sleeves welded to the steel box. The sleeves are capable of sliding up and down on the posts.

Figure 5 Lakeview House, New Orleans. Elevation and detail of connection to post.

II.4 LIFT House in Dhaka, Bangladesh

This prototype of a low-cost, sustainable amphibious house for urban slum-dwellers broke ground in November of this year in Dhaka, Bangladesh. Initiated and designed by Prithula Prosun, currently a Master of Architecture student at the University of Waterloo School of Architecture, each independent structure consists of two to eight floatable bamboo dwellings clustered around a shared courtyard. A stationary brick base supporting the dwellings contains plumbing, utilities and rainwater storage cisterns. Each two-room amphibious bamboo dwelling unit provides living and sleeping quarters for a single family.

Figure 6 The LIFT House in Dhaka, Bangladesh. Eight-unit and two-unit clusters.

Figure 7 The LIFT House in Dhaka, Bangladesh. Concrete and brick base structure
under construction; buoyancy block made of recapped, recycled plastic bottles.

II.5 The Buoyant Foundation Project

The Buoyant Foundation Project (BFP) is a non-profit research initiative founded in 2006 at the LSU Hurricane Centre with the goal of designing and implementing retrofittable buoyant foundations for New Orleans "shotgun" houses.

II.5.1. What is a Buoyant Foundation?

A buoyant foundation is a type of amphibious foundation that is specially designed to be retrofitted to an existing south Louisiana shotgun house. It allows the house to sit just above the ground like a normal elevated house under normal conditions, but to rise up and float safely on the water when there is a flood. It has a structural subframe that attaches to the underside of the house and supports the flotation elements, or buoyancy blocks. Extensions of the structural subframe attach to the tops of vertical guidance poles near the corners of the house that telescope out of the ground to provide resistance to lateral forces from wind and flowing water.
When flooding occurs, the flotation blocks lift the house, with the structural subframe transferring the forces between the house, blocks and poles. The vertical guidance poles keep the house from going anywhere except straight up and down on top of the water. The elements of the structural subframe are inserted underneath the house in pieces.
Most of the pieces are small and light enough to be installed by two persons without machinery. After the buoyant foundation is in place, the house remains supported on its original piers except when flooding occurs. Utility lines have either long, coiled "umbilical" lines or self-sealing "breakaway" connections that disconnect gas and sewer lines when the house begins to rise.

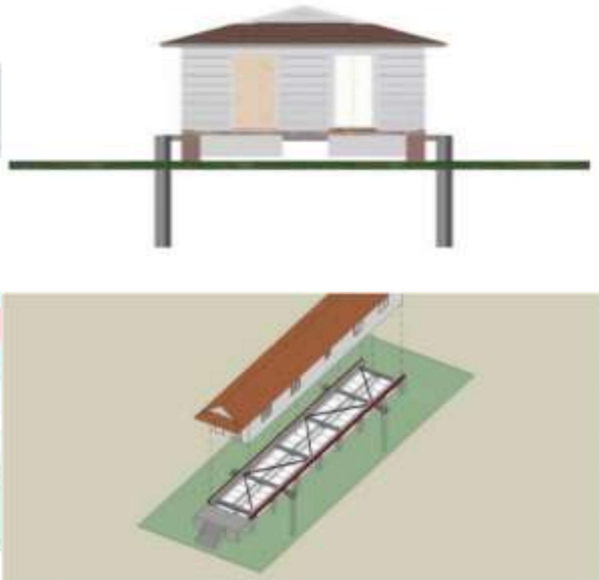


Figure 8 Schematic diagrams of the buoyant foundation system.

## III. FERROCEMENT

III.1 Introduction to ferrocement

Ferrocement techniques though of recent origin, have been extensively used in many of countries, notably U.K., New Zealand and China. There is a growing awareness of the advantages of this technique of construction all over the world. It is well known that the conventional reinforced concrete members are too heavy, brittle cannot be satisfactorily repaired if damaged, develop cracks and reinforcements are liable to be corroded. The above disadvantages of normal concrete make it inefficient for certain types of work.

Ferrocement is a relatively new material consisting of wire meshes and cement mortar. This material was developed by P.L. Nervi, an Italian architect and engineer, in 1940. It consists of closely spaced wire meshes which are impregnated with rich cement mortar mix. The wire mesh is usually of 0.5 to 1.0 mm diameter at 5 mm to 10 mm spacing and cement mortar is of cement sand ratio of 1: 2 or 1: 3 with water/ cement ratio of 0.4 to 0.45. The ferrocement elements are usually of order 2 to 3 cm. in thickness with 2 to 3 mm external cover to the reinforcement. The steel content varies between 300 kg to 500 kg per cubic meter of the mortar. The basic idea behind this material is that concrete can undergo large strains in the neighbourhood of the reinforcement and the magnitude of the strains depends on the distribution and subdivision of reinforcement throughout of the mass of concrete.

Ferrocement is widely accepted in U.K., New Zealand and United States as boat building material. It has also found various other civil engineering applications. The main advantages are:

1. Simplicity of its construction
2. Lesser dead weight of the elements due to less thickness
3. High tensile strength
4. Less crack widths compared to conventional concrete
5. Easy repairability
6. Non corrosive nature
7. Easy mouldability to any required shape.

There is also saving in basic materials namely, cement and steel. This material is more suitable to special structures like shells which have high strength through forms and structures like roofs, silos, water tanks and pipelines.

The material is under active research in various countries and attempts are being made to give a sound theoretical backing to establish the material behaviour. This is highly suitable material for precast products, because of its easy adaptability to prefabrication and lesser dead weight of the unit cast. The development of ferrocement depends on suitable casting techniques for the required shape. Development of proper prefabrication techniques for ferrocement is still not a widely explored area and gap needs to be filled.

3.2 Casting techniques

There are four methods of casting:

i. Hand plastering
ii. Semi-mechanised process (using hand plastering)
iii. Centrifuging and
iv. Guniting.

III.2.1 Hand plastering

It is the most critical operation in ferro cement casting. If the mortar impregnation is not proper, the quality of the structure will not be good and it will not give desired performance.

A sufficient quantity of mortar is dashed from outside through the layers against a G-1 sheet held on the other side. The flexible G-1 sheet is moved around and the mortar is dashed from the outside. The process is continued till the whole structure is built up. During process of putting the mortar, it should be ensured that no voids are left in the body of the structure.

It can be ensured by using a wooden hammer of about 100 mm diameter with 150 mm long wooden handle. The mild hammer blows are given over the temporarily held form to remove the voids. This will give sufficient vibrations for compacting the mortar. The whole thickness is built up gradually in two or three consecutive dashing of mortar and then both internal and external surfaces are made smooth.

Shells and boat hulls like structures are built by the technique known as two operation mortar impregnations. In this system, first the outside mesh is plastered and the inner layer is left exposed. The excess mortar is scrapped by trowel and wire brushes. The mortar is left for setting till it attains sufficient strength for carrying the load from the inside during the application of a second layer of mortar. Before applying the second layer, fine cement slurry is sprayed over the entire inner surface.

In structures where many layers are used as reinforcement and thickness is more than 20 mm, in such cases it is desirable to do the casting in three layers. The middle or core layer is applied first covering the skeleton steel and one layer of wire mesh. This core provides a firm surface for mortar application on its top and bottom. The core is cured at least for 3 days before the other two layers of mortar are applied. For getting the good bond, between old and new mortar cement slurry should be sprayed over the middle layer.

For thin cylindrical units of about 1 m diameter, 6 mm diameter steel rods at a spacing of 15 cm be used for making a cage of cylindrical shape and then woven or chicken mesh can be tied to the mesh and impregnated or plastered. In such type of construction, the use of chicken mesh is not advisable as it is very flexible and plastering over it may not be satisfactory.

In this method the control of thickness is difficult and the minimum thickness of the section works out to be more than 20 mm. The greater thickness not only makes it uneconomical, but also some technical advantages are lost. The strength obtained by hand plastering or impregnation is lower compared to other methods due to poor compaction of mortar by this method.

The units cast by this method may be used for pipes, storage structures and gas holder units etc. This method of casting is suitable of units of shapes for which making of mould is difficult. This method can also be applied for making cylindrical shaped units of size approximately 60 cms in diameter or above.

III.2.2 Semi-mechanised process

A semi mechanised process for making ferro cement cylindrical units has been developed by Structural Engineering Research Centre (SERC) Roorkee. In this process a central cylindrical mould is used. Over this central mould one layer of wire mesh is wound. Over this layer a 4 mm diameter wire is tied at a spacing of 150 mm in both directions. One layer of chicken mesh is wound over this wire layer. This forms the complete wire mesh system of reinforcement.

Now the cement sand mix prepared is impregnated or plastered layer by layer. Due to the tightly wound mesh around the form work, the thickness of the unit is reduced. With this system, units up to one cm thickness can be cast containing two layers of wire mesh in that thickness i.e. within 1 cm thickness. This system is called semi mechanised as the mould can be rotated to facilitate dashing of mortar.

Advantage of Semi Mechanised Method:
Following advantages have been observed of this method:

i. In this method better compaction can be obtained by means of a straight edge pressed against the inner mould.

ii. The uniformity of thickness is better in this method than hand plastering.

iii. The wire mesh can be wound tightly over the mould and also can be tightened during the casting operation. This helps in avoiding unevenness of thickness and looseness in the mesh.

iv. This system does not need any sophisticated equipment and electricity.

v. Local, un-skilled people can handle this process.

vi. This process can be adopted easily in rural areas.

vii. The cylindrical units of size up to 1.0 m or above can be cast by this process.

III.2.3 Centrifuging

For the fabrication of concrete cylindrical units, generally centrifuging process is adopted. The first crack strength of ferro cement has been observed higher in comparison of normal reinforced concrete. Thus, the pipe thickness can be reduced, resulting in lesser dead weight. In the existing centrifuging process, the mild steel reinforcement cage has been replaced by wire mesh layers cage. The trial casting at SERC Roorkee has shown that this method can be adopted for casting ferro cement units. Due to good compaction, ferro cement pipes cast by centrifuging process can be used as high-pressure pipes.

III.2.4. Guniting

The process of guniting can be adopted for applying the mortar to the wire mesh system. This process seems to be suitable for mass production of ferro cement prefabricated units. A continuous process of layer guniting with an interval of about, an hour will yield good results. If the process is applied properly by an experienced gun man can produce good compacted and uniform surface.

III.3 Properties of Ferro Cement:

i. Compressive strength. The behaviour of thin ferro cement element under compression primarily is controlled by the properties of cement mortar matrix. Its compressive strength varies from 27.5 to 60 MPa.
ii. Tensile strength. The tensile strength of ferro cement depends mainly on the volume of reinforcement in the direction of force and the tensile strength of the mesh. The ultimate tensile strength is 34.5 MPa and allowable tensile stress is taken as 10.0 MPa.

The tensile behaviour may be divided into three groups:

i. Pre cracking stage.
ii. Post cracking stage.
iii. Post yielding stage.

A ferro cement member subjected to increasing tensile stresses behaves like a liner elastic material till the development of first crack in the matrix. Once the cracks are developed, the material enters the stage, of multiple cracking and this stage continues up to the point where wire mesh starts to yield.

In this stage numbers of cracks go on increasing with the increase in tensile stress without any significant increase in the width of the crack. With the yield of reinforcement, the mortar enters the stage of crack widening. At this stage the number of cracks remains constant, but the width of crack goes on increasing. The behaviour mainly is controlled by the reinforcement bars.

III.3.1 Fatigue Strength:

The fatigue behaviour of ferro cement flexural elements is governed by the tensile fatigue properties of mesh like reinforced and pre stressed concrete beams. The fatigue strength of ferro cement is poor under cyclic loading.

III.3.2 Impact Strength:

The impact strength of ferro cement has been found to increase linearly with the increase of specific surface (volume fraction) and ultimate strength of mesh reinforcement. Further for the same reinforcement fraction, the element having welded wire mesh reinforcement showed highest impact strength while chicken mesh reinforced section showed lowest impact strength. The impact strength of woven mesh reinforced ferro cement is found higher than chicken wire mesh and lower than welded wire mesh reinforced elements.

III.4 Applications of Ferro Cement:

Due to the very high percentage of well distributed and continuously running steel reinforcement, the ferro cement behaves like steel plates. Its cracking resistance, ductility, impact and fatigue resistances are higher than that of normal concrete. The impermeability of ferro cement product is far superior than ordinary R.C.C. products.

Due to these properties the ferro cement can be used for the following purposes:

i. Ferro cement can be used successfully for casting domestic overhead tanks. These tanks being light and flexible can be transported and hoisted without difficulty. The inlet and out let connections also can be done easily with the help of modem adhesives like “m seal”. These tanks will be cheaper than any other type of material tanks.
ii. These tank units can also be modified into silos for storing grains in villages. These tanks will help in preserving grains from moisture effect and rodents.
iii. Due to the favourable properties of ferro cement, this material has been widely used for boat building in U.K., U.S. and New Zealand. It has been reported that 14 m long ferro cement boat weighs only 10% more than the wooden boats. Ferro-cement boats are found 300% cheaper than fibre reinforced concrete boats, 200% cheaper than steel boats and 35% cheaper than timber boats.
iv. The cost of ferro cement is only about 10% of the cost of cast iron. Thus, the use of ferro cement manhole covers is becoming very popular, where these manhole covers are not subjected to heavy vehicular traffic.
v. Ferro cement is becoming more popular material for pre-fabricated roof units. The folded plates of ferro cement being light can be advantageously used as prefabricated roof units. A 3 cm thick ferro cement folded plate with two layers of chicken wire mesh can be used safely over a span of 3.5 m. It can also be used for prefabricated channel units for roof construction.
vi. Ferro cement being a light material, considerable reduction in self-weight of structure and foundation cost can be reduced to a great extent. A 30% reduction in dead weight on supporting structure, 15% saving in steel consumption and 10% saving in roof cost has been observed in USSR by the use of ferro cement.
vii. Ferro cement is found most suitable material for the production of pressure pipes. It is much lighter than normal RCC pipes.
viii. Ferro cement also is found suitable for casting curved benches for parks, gardens, and open cinema theatre. It can also be used to cast tree guards. They can be cast in two parts to facilitate their removal at a later data.

III.5 Advantages of Ferro Cement:

Following are the advantages of the ferro cement:

1. The construction technique of ferro cement is simple. It does not require skilled labour.

2. Complete or partial elimination of form work is possible.

3. Ferro cement construction is easily amenable to repairs in case of local damage due to abnormal loads as impact load.

Table 23.4. Normal range of composition and properties of ferro cement

| S. No. | Parameters | Range |
|---|---|---|
| 1. | Wire mesh | |
| | (a) Type of mesh | Welded wire or square woven or chicken wire mesh or galvanized wire mesh or expanded metal |
| | (b) Diameter of wire | Diameter of wire may be between 0.5 to 1.5 mm |
| | (c) Size of mesh opening | It may be between 5 to 25 mm |
| | (d) Distance between mesh layers | The distance between two layers should be greater than 2 mm |
| | (e) Volume fraction of reinforcement | Upto 8% in both directions corresponding to 650 kg/m$^3$ of concrete |
| | (f) Specific surface of rainforcement | Upto 4 cm$^2$/cm$^3$ in both directions |
| 2. | Skeleton reinforcement | |
| | (a) Type | Rods, strands, wires or wire fabrics |
| | (b) Diameter | 3 to 10 mm |
| | (c) Grid size | 50 to 100 mm |
| 3. | Mortar composition | |
| | (a) Portland cement | Any type depending on application |
| | (b) Cement sand ratio | 1:2 to 1:3 or 1:1.75 to 1:2.5 |
| | (c) Water/cement ratio | It may range from 0.35 to 0.6, but normally a w/c ratio of 0.35 to 0.4 is used |
| | (d) Fine aggregate | Fine sand all passing IS 4.75 mm sieve and 5% by mass passing through IS 1.18 mm sieve. Thus the sand of continuous grading curve between 1.18 mm and 2.36 mm size should be used. |
| 4. | Matrix Properties | |
| | (a) Thickness | The mortar thickness may be between 10 to 60 mm but normally it should be between 20 to 30 mm |
| | (b) Cover to reinforcement | 1.5 to 5 mm |
| | (c) Compressive strength | 27.5 to 60 MPa |
| | (d) Ultimate tensile strength | 34.5 MPa |
| | (e) Allowable tensile stress | 10.0 MPa |
| | (f) Modulus of rupture | 55.0 MPa |

Table I: showing Summary of composition and properties of commonly used ferro cement.

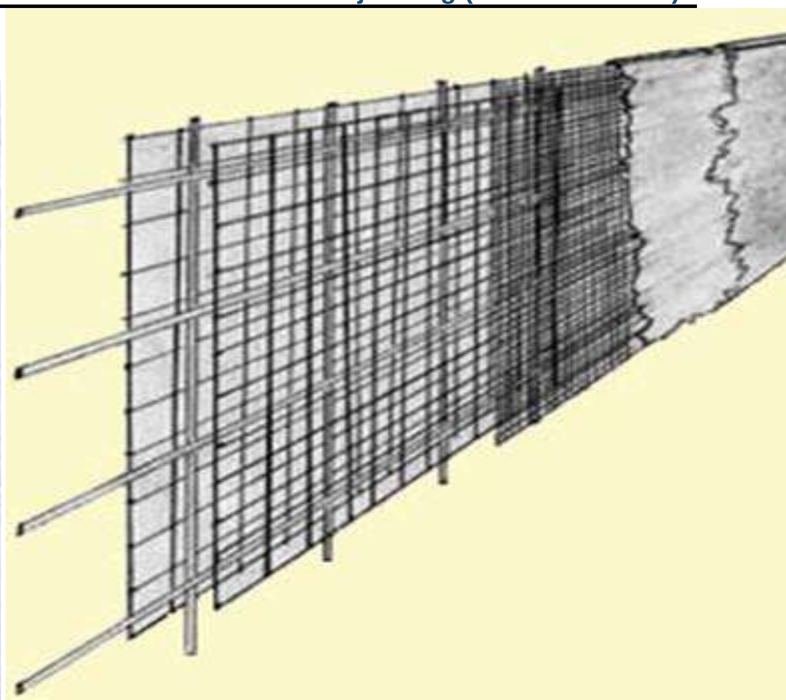


Figure 9 Casting of ferrocement concrete on site

## IV. GLASS FIBRE REINFORCED GYPSUM (GFRG)

IV.1 Introduction

GFRG is the abbreviation for glass fibre reinforced gypsum. It is the name of a new building panel product, made essentially of gypsum plaster, reinforced with glass fibres, and is also known in the industry as Rapid wall. This product, suitable for rapid mass-scale building construction, was originally developed and used since 1990 in Australia. GFRG is of particular relevance to India, where there is a tremendous need for cost-effective mass-scale affordable housing, and where gypsum is abundantly available as an industrial by-product waste. The product is not only eco-friendly or green, but also resistant to water and fire.

GFRG panels are presently manufactured to a thickness of 124 mm, a length of 12m and a height of 3m, under carefully controlled conditions. The panel can be cut to required size. Although its main application is in the construction of walls, it can also be used in floor and roof slabs in combination with reinforced concrete. The panel contains cavities that may be filled with concrete and reinforced with steel bars to impart additional strength and provide ductility. The panels may be unfilled, partially filled or fully filled with reinforced concrete as per the structural requirement. Experimental studies and research have shown that GFRG panels, suitably filled with reinforced concrete, possess substantial strength to act not only as load-bearing elements, but also as shear walls, capable of resisting lateral loads due to earthquake and wind. It is possible to design such buildings up to ten storeys in low seismic zones (and to lesser height in high seismic zones). However, such construction needs to be properly designed by a qualified structural engineer. Manufacture of GFRG panels with increased thickness (150 mm, 200 mm) with suitable flange thickness can facilitate design and construction of taller buildings.

GFRG panels can also be used advantageously as infills (non-load bearing) in combination with reinforced concrete (RC) framed columns and beams (conventional framed construction of multi-storey buildings) without any restriction on the number of storeys. Also, GFRG panels with Embedded micro-beams and RC screed (acting as T-beams) Can be used as floor/roof slabs.

4.1.1. Some of the advantages of construction using GFRG panels are:

i. Substantial reduction in the structural weight of the building
ii. No plastering requirement for walls and ceiling
iii. Increased speed of construction with less manpower
iv. Saving of cement, steel, river sand, burnt clay bricks / concrete blocks and hence saving of energy and reduced $CO_2$ emissions, contributing to environmental protection and mitigating climate change.
v. Use of reprocessed / recycled industrial by product, viz., waste gypsum, to manufacture.

GFRG panel, helping to abate pollution and protect the environment. GFRG building systems can be constructed only with technical support or supervision by qualified engineers and constructors, based on structural designs carried out in detail complying to prevailing standards; this is applicable even for low-rise and affordable mass housing, to provide for desirable safety margins against natural disasters (such as earthquakes and cyclones).

GFRG panels can be unfilled when used as partition walls, but when used as external walls, need to be suitably designed (with reinforced concrete filling) in order to resist the design wind pressures. For single-storey construction (suitable for affordable mass housing), unfilled GFRG panels can be used for walls as well as roof (which may be pitched suitably), with local reinforced concrete filling at the joints between walls and between the roof and walls. It is mandatory to provide embedded RC horizontal tie beams over all the walls below the floor slab / roof slab.

Government of India, has accorded approval of GFRG panels for construction in India. However, to facilitate such construction, it was felt necessary to bring out appropriate design and construction manuals that meet the statutory requirements of relevant Indian Standards.

There is a need for continued structural testing of various aspects of GFRG panels, manufactured in India. It is strongly recommended that structural engineers and building designers associated with GFRG panel construction should be thoroughly familiar with the various structural design aspects. It is also recommended that architects and construction engineers who undertake GFRG / Rapid wall building design and construction gain familiarity with the properties and material characteristics of Rapid wall and its applications and construction systems.

IV.2. Dimensions of GFRG Panels

These design guidelines are applicable to GFRG panels, presently manufactured as rapid walls, for the typical dimension and material properties describes in this project. Typical dimensions of a GFRG building panel are 12.0m × 3.0m × 0.124 m, as shown in Fig. 4.2.1. Each 1.0 m segment of the panel contains four ‘cells. Each cell is 250 mm wide and 124 mm thick, containing a cavity 230 mm × 94 mm, as shown in Fig. 11.

The various cells are interconnected by solid ribs (20 mm thick) and flanges (15mm thick), comprising gypsum plaster reinforced with 300 – 350 mm glass fibre roving, randomly but centrally located. The skin thickness is 15mm and the rib thickness is 20 mm.

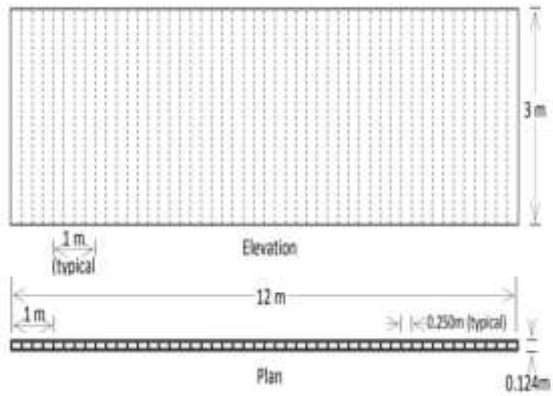



Fig. 10 Typical cross section of GRFG panel

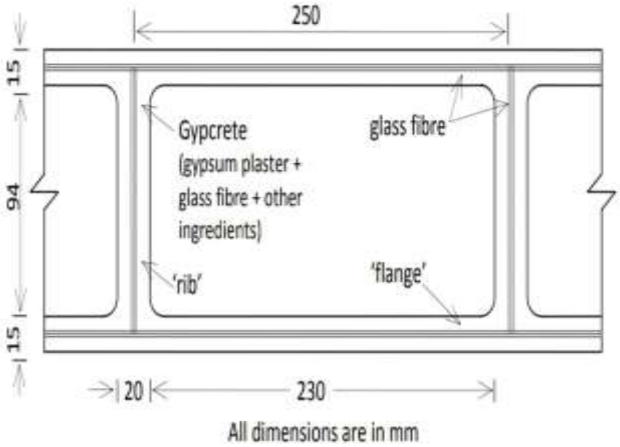



Fig. 11 Enlarged view of typical cell

IV.3. Applications

GFRG building panels are generally used structurally in the following seven ways:

i. As load bearing walling in buildings. When the cavities are filled with reinforced concrete, the strength of the panel to resist vertical and lateral loads gets enhanced considerably, rendering such load bearing constructions suitable for multi-storeyed housing. In single or two storeyed constructions, the cavities can remain unfilled or suitably filled with non-structural core-filling such as insulation, sand, quarry dust, polyurethane or lightweight concrete.
ii. As partition infill walls in multi-storey framed buildings. Panels can also be filled suitably. Such walls can also be used as cladding for industrial buildings or sport facilities, etc. As compound walls / security walls.
iii. As horizontal floor slabs / roof slabs: with reinforced concrete micro beams and screed (T- beam action). This system can also be used in inclined configurations, such as staircase waist slabs and pitched (sloped) roofing.

IV.3.1. Use as Load Bearing Structural Walling

In typical multi-storeyed constructions involving the use of GFRG as load bearing structural walling, the connections between cross walls and with the foundations and floor/roof are achieved through reinforced concrete filling or R.C beams. All GFRG wall panels at the ground floor are to be erected over a network of RC plinth beams supported on suitable foundation (refer Fig 17). 'Starter bars' shall be embedded in the RC plinth beams, at the precious locations where the cavities are to be filled with reinforced concrete, with appropriate lap length. In this manner, the connection at the ground storey between super structure and foundation, spread over the entire wall length over the network of RC plinth beams is ensured.

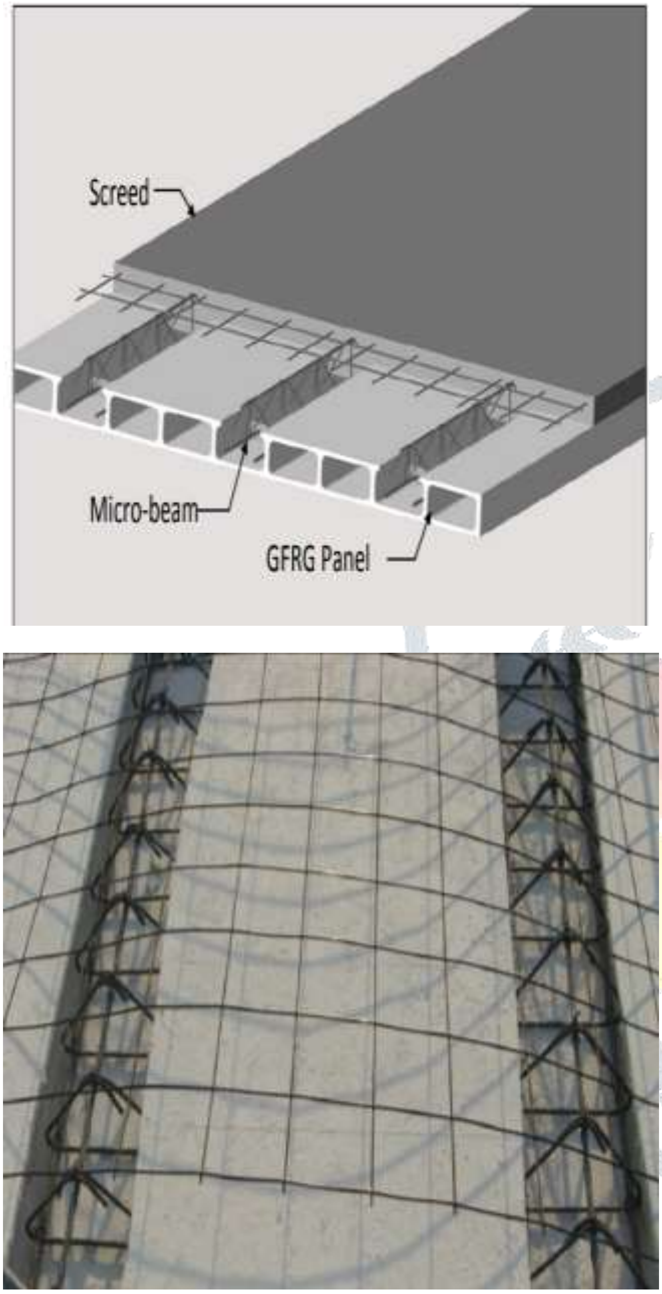



Fig 12 GFRG floor slab with micro beam and screed.

Fig. 13. Erection of GFRG panels over plinth beam at site.

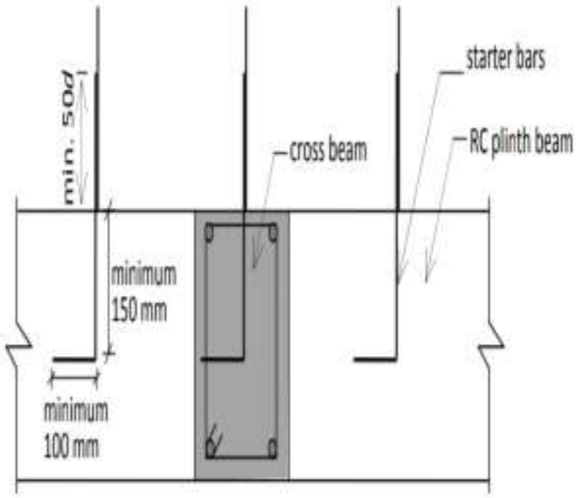



Fig 14 Provision for starter bars in RC plinth beam for erection of GFRG panels.

For constructing an additional GFRG floor above an existing RC building, connectivity between GFRG wall and the existing floor can be achieved by proper detailing (insertion of starter bars with proper anchorage) as shown in Fig. 18. If the existing floor slab does not have sufficient depth for anchorage, an additional RC beam may be constructed above the roof before erecting the GFRG walls.

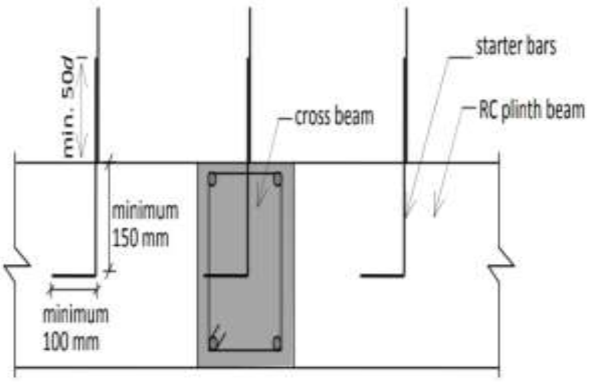



Fig 15 Starter bars in case of existing RC floor / roof slab.

When GFRG panel is used as structural walling, an embedded horizontal RC tie beam has to be provided on top of all the walls. Tie beam size of 200 mm depth and 94 mm width is suggested by cutting and removing the top portion of the web of GFRG panels as shown in fig. 4.3.2.1.

IV.3.2. Use as Floor / Roof slabs

GFRG panel can also be used for intermediate floor slab/roof slab in combination with RC. The strength of GFRG slabs can be significantly enhanced by embedding reinforced concrete micro beams. For providing embedded micro beams, top flange of the respective cavity is cut and removed in such a way that minimum 25 mm flange on both ends is protruded as shown in Fig. 4.4. RC concrete screed of minimum 50 mm thickness is provided above the GFRG floor panel, which is reinforced with weld mesh of minimum size of 10-gauge 100 mm × 100 mm. This RC screed and micro beam act together as series of embedded T-beams. The thickness of the RC screed, reinforcement and interval of embedded RC micro beams depends on the span and intensity of imposed load. The connectivity between the horizontal tie beam, embedded RC micro beams, concrete screed and vertical rods in GFRG wall, ensures perfect connection between floor/roof slab and walling system.

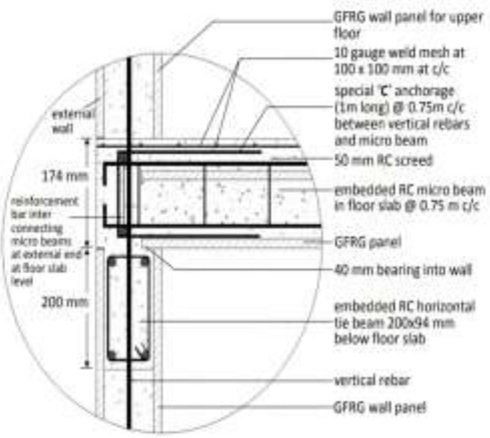



Fig. 16 Connectivity between floor slab and wall.

IV.4. Mechanical Properties of GFRG panels

Table II provides a summary of typical mechanical properties of the GFRG building panel. These properties have been determined from tests on GFRG building panel. The compressive strength to be considered when the panel is filled with concrete is given in Table 4.4.2.

| Mechanical Property | Nominal Value | Remarks |
|---|---|---|
| Unit weight | 0.433 kN/m$^2$ | |
| Modulus of elasticity | 7500 N/mm$^2$ | |
| Uni-axial compressive strength, $P_{uc}$ | 160 kN/m | Strength obtained from longitudinal compression / tension tests with ribs extending in the longitudinal direction |
| Uni-axial tensile strength, $T_{uc}$ | 34 - 37 kN/m | |
| Ultimate shear strength, $V_{uc}$ | 21.6 kN/m | |
| Out-of-plane moment capacity, Rib parallel to span, $M_{uc}$ | 2.1 kNm /m | |
| Out-of-plane moment capacity, Rib perpendicular to span, $M_{uc\text{-}perp}$ | 0.88 kNm /m | |
| Mohr hardness | 1.6 | |
| Out-of-plane flexural rigidity, *EI*, Rib parallel to span | 3.5×10$^{11}$ Nmm$^2$/m | |
| Out-of-plane flexural rigidity, *EI*, Rib perpendicular to span | 1.7×10$^{11}$ Nmm$^2$/m | |
| Coefficient of thermal expansion | 12×10$^{-6}$ mm/mm /°C | |
| Water absorption | 1.0 % : 1hr<br>3.85 % : 24hrs | Average water absorption by weight % after certain hours of immersion |
| Fire resistance: Structural adequacy/Integrity/Insulation | 140/140/140 minutes | CSIRO, Australia |
| Sound transmission class (STC) | 40 dB | ISO 140-3-1996[7] |

Table II: Mechanical properties of GFRG building panel (unfilled)

| Property | Nominal Value | Remarks |
|---|---|---|
| Uni-axial compressive strength, $P_{uc}$ (Both ends hinged) | 1310 kN/m* | Obtained from longitudinal compression tests with ribs in the longitudinal direction |
| Uni-axial compressive strength, $P_{uc}$ (one end fixed and other end hinged) | 1360 kN/m* | – as above – |
| Ultimate shear strength, $V_{uc}$ | 61 kN/m† | Longitudinal cracks (parallel to the ribs) |
| Fire resistance: Structural adequacy/Integrity/Insulation | 241/241/241 minutes | CSIRO, Australia |

Table III: Properties of compressive strength of GFRG building panels (filled with minimum M20 grade of concrete in all the cores).

IV.5 Design Philosophy

The design capacities given in these guidelines are based on limit states design procedures, considering the ultimate limit state for strength design, treating the 3.0 m high GFRG building panel as the unit material, and considering the strength capacity as obtained from test results. The design should be such that the structure should withstand safely all loads (as per relevant Indian Standards) likely to act on the structure during its lifetime. It shall also satisfy serviceability requirements, such as limitations of deflection and cracking. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

IV.5.1. Limit State Design

For ensuring the design objectives, the design should be based on the characteristic values of material strengths and applied loads (actions), which take into account the probability of variations in material strength and load. The design values are derived from the characteristic values through the use of partial safety factors, both for material strengths and for loads, for limit states of collapse and serviceability.

IV.5.1.1. Partial Safety factors for loads $\gamma_f$

The design must account for various combinations of loads acting on the structure simultaneously. The various load combinations and corresponding partial safety factor for loads shall be used as given in IS 456: 2000 as summarized in Table IV

| Load Combination | Limit State of collapse | | | Limit State of Serviceability | | |
|---|---|---|---|---|---|---|
| | DL | LL | WL/EL | DL | LL | WL/EL |
| DL+LL | 1.5 | 1.5 | - | 1.0 | 1.0 | - |
| DL+WL/EL | 1.5 or 0.9(*) | - | 1.5 | 1.0 | - | 1.0 |
| DL+LL+WL/EL | 1.2 | 1.2 | 1.2 | 1.0 | 0.8 | 0.8 |

Table IV: Values for partial safety factor for loads.

Note: - For the limit state of serviceability, the values of $\gamma_f$ given in this table are applicable for short-term effects. While assessing long term effects due to creep, the dead load and that part of live load likely to be permanent should be considered.

IV.5.1.2. Partial safety factor for materials $\gamma_m$

The magnitude of partial safety factor for the material must take into account the uncertainty related to the material strength. Although GFRG building panels are manufactured under carefully controlled conditions, it is considered prudent to treat the material like concrete, for which the partial safety factor specified in IS 456: 2000 is 1.50. The partial safety factor for the GFRG building panel (with and without concrete infill) shall be taken as $\gamma_m$ = 1.50 in general. The above partial safety factor $\gamma_m$ = 1.50 is applicable to situations involving out of plane bending where the observed mode of failure is brittle as well as in plane bending of RC filled GFRG panels where the mode of failure is expected to be ductile. In the case of reinforcing steel, the partial safety factor shall be taken as $\gamma_s$ = 1.15 in all cases, as recommended in IS 456: 2000. While investigating serviceability limit states, the partial safety factor for all materials should be taken as unity.

IV.5.1.3. Response reduction factor for Earthquake Resistant Design

Earthquake resistant design shall be carried out in compliance with the requirements of IS 1893 (Part 1). In such design, an important and difficult task is the determination of the response reduction factor (R). This is traditionally arrived at, based on the general observed performance of similar buildings during past earthquakes, estimates of general system toughness and the amount of damping present during inelastic response. As GFRG buildings constitute a new type of structure, a reasonable choice of R factor can only be made by comparing the GFRG building system with traditional structures, such as reinforced concrete wall building systems for which the response modification factors are already available.

GFRG walls are composite members with partial interaction, and the ductility of a partially interactive member is generally greater than that of a fully interactive reinforced concrete member. In terms of strength reserve, it is recommended that the safety margin adopted for the design of GFRG walls be at least as large as that adopted for concrete structures. Therefore, it is not unreasonable to treat buildings constructed with GFRG walls as reinforced concrete shear wall structures and to adopt the R values from the respective code of practice (Wu 2009) (10). Hence, the response reduction factor (R) is taken as 3.0 (IS 1893-2002) for seismic load calculations.

IV.6. Axial Load Capacity

While assessing the axial loads capacity GFRG panels (under compression), it is important to consider possible eccentricities in loading. A minimum eccentricity causing (out of plane bending) must always be accounted for in the design.

IV.6.1. Minimum eccentricity

According to IS 456: 2000 (cl.32.2.2), the design of a reinforced concrete wall shall take into account the actual eccentricity of the vertical force subjected to a minimum value of 0.05t (6.2 mm for panel thickness t = 124 mm). As per IS 1905: 1987, the design of a masonry wall shall consider appropriate eccentricity, which in no case shall be taken to be less than t/24 (5.2 mm for t = 124 mm).

In the case of wall panels supporting floor slabs from one side only, the eccentricity to be considered should be more than the minimum values indicated above. It is recommended that a value of minimum eccentricity equal to t/6 (i.e., 20.7 mm) shall be considered conservatively. Additional value of eccentricity may be considered when out-of-plane bending is explicitly involved (for example, action of local wind effects on an exposed wall panel).

IV.6.2. Axial Compressive Strength

The characteristic values of axial compressive strength of the GFRG building panel, expressed in kN/m, are obtained from compression test results on GFRG building panel for full height panel, subject to various eccentricities of loading (20 mm, 30 mm and 45 mm) and different boundary conditions (1,8,11). In general, it is conservative to assume pinned – pinned condition, as shown in fig. 4.6.2.1.

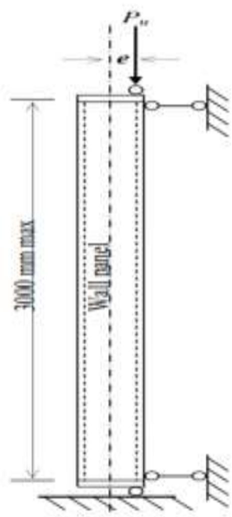



Fig. 17 Experimental Setup for pinned – pinned panels

It may be noted that, for design purposes, the reported nominal values should be divided by γm = 1.5. The design values (including partial safety factor) are depicted in Figs 7.2 and 7.3, corresponding to unfilled and filled cases respectively, assuming a linear variation of axial load with eccentricity.

IV.7. Compressive Strength

The unit shear strength capacity of the 124 mm thick, 3.0 m high GFRG panel is given in Table 9.1. The ultimate design shear strength of a GFRG panel is given by the unit shear capacity in Table V. multiplied by the length of the panel.

| Application | Design Shear Capacity, $V_{ud}$ (kN/m) |
|---|---|
| Unfilled GFRG panel | 14.4 |
| GFRG panel filled with 20 MPa concrete | 40.0 |
| GFRG panel partially filled with 20 MPa concrete | $14.4 + 25.6\eta$ (where, $\eta$ is defined as in Eq. 5.3) |

Table V. Shear strength of GFRG panels as Vertical walls.

In a multi-storeyed construction, using GFRG wall panels as load bearing construction, different walls will be subjected to different shear forces, at any storey level under consideration. Larger walls, which are stiffer, will attract more lateral shear. The maximum length of an individual shear wall segment may be limited to 3.5 m in the finite element model used for analysis under factored loads. The average value of factored shear force calculated for all walls in any one direction at any storey level shall not exceed the value indicated in Table 9.1. In few walls, some local increase (up to 20 percent) in shear capacity may be permitted, provided the average value for all walls (combined) is within the prescribed limit. Double walls may be provided, if there is a higher demand for shear strength.

In wall construction for multi-storeyed buildings, all cavities should be filled with concrete (of grade not less than M20) and reinforced appropriately. The design of such reinforcement is discussed in the Chapter 10. The rebars shall be provided for the full height in filled GFRG panels. In any case, for both filled and unfilled GFRG wall panels at the interface with foundation plinth beam, starter bars should be provided in each cell embedded in concrete (of strength not less than 20 MPa) for a depth of 450 mm for adequate shear transfer.

IV.8. Design of Lintel

GFRG panels above windows and door cuts can be used as lintels to support superimposed loads. These design guidelines are based on simply supported conditions. For lintels, that are actually framed above an opening, to limit creep deflection, they may be considered as simply supported.

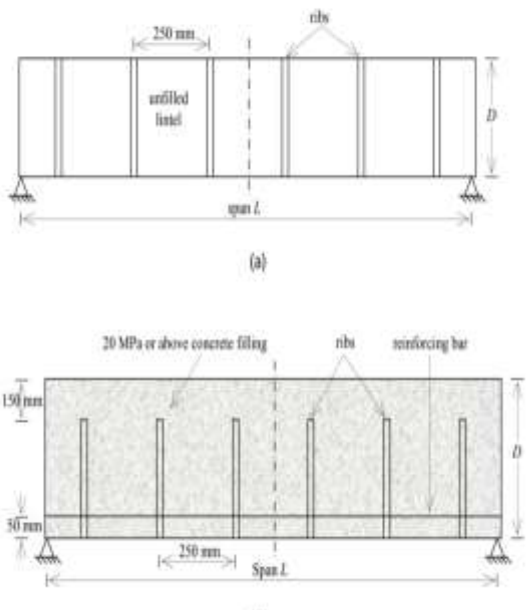



Fig. 18 Details of Lintel

IV.9. Design of Floor / Roof Slab

As GFRG panels with ribs aligned in direction of bending possess flexural strength (refer Table 8.1), such panels can be used as flexural slab, whose strength can be significantly enhanced by embedding 'micro beams', filled with reinforced concrete. Unfilled GFRG panels can be used as pitched roofs for single storeyed small span buildings (refer Fig. 12.1). Some nominal filling with reinforcement may be done at eaves and ridge locations as shown in Fig. 19.

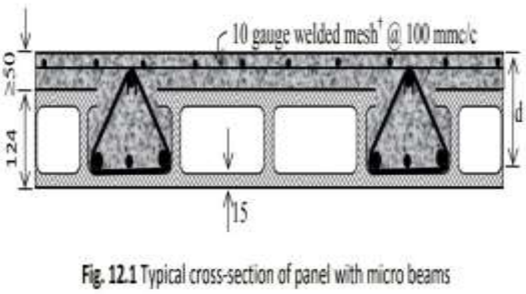



Fig. 19. Typical cross section of panel with micro beams

GFRG-RC composite slab systems can be used efficiently in floor slabs and roof slabs. The ribs are to be oriented along the shorter span, supported on GFRG wall panels. For convenience in design, the contribution of GFRG towards the flexural strength can be ignored and the GFRG treated as lost formwork. Reinforced concrete micro beams are to be provided by filling cavities at regular intervals (typically every third cavity) and provided with reinforcement suitably designed, with a screed concrete of thickness not less than 50 mm as shown in Fig.19. One-way slab action may be assumed for strength and deflection check, considering T beam action of the embedded micro beams. In the screed concrete, suitable welded wire fabric (of required gauge and spacing) shall be provided. The design of reinforcement in the micro beams shall conform to the requirements of IS 456: 2000. Such slabs can be conveniently designed up to spans of 5m.

IV.10. GFRG building panel as pitched (slope) roofing element

GFRG building panels can be used as roofing elements for buildings with pitched (sloped) roof, which is commonly adopted for low income group housing as shown in Fig. 4.10.1. Design of such roofing for typical low-income group housing with a plinth area of 21.06 m2 is illustrated here.
Live load on the roof is taken as per IS: 875 (Part-2) 1987 as 0.65 kN/$m^2$ (on projected plan area). It is assumed that water-repellent paint is applied on the top of the roof as in the case of external surface of the building wall panels. The roof panels are assumed to be integrally connected with the wall panel (through provision of reinforced concrete grouts as shown in Fig. 12.4). A finite element analysis has been carried out for a dead load (self-weight) of 0.43 kN/m2 in the unfilled portion and an additional 1.98 kN/m2 in the filled regions, and for a live load of 0.65 kN/m2 on the projected plan area of the building. The variation of bending moments in the roof

i. Along the slope (ribs parallel to slope).
ii. Perpendicular to the rib and in the wall (both vertical and horizontal bending moment) are shown in Fig. 20 From these figures, it can be seen that the maximum design moment parallel to the rib is less than the design moment capacity of 1.4 kNm/m and the maximum design moment perpendicular to the rib is less than the corresponding design moment capacity of 0.59 kNm/m. This means that panels can be used as sloping roof without any reinforced concrete infilling.

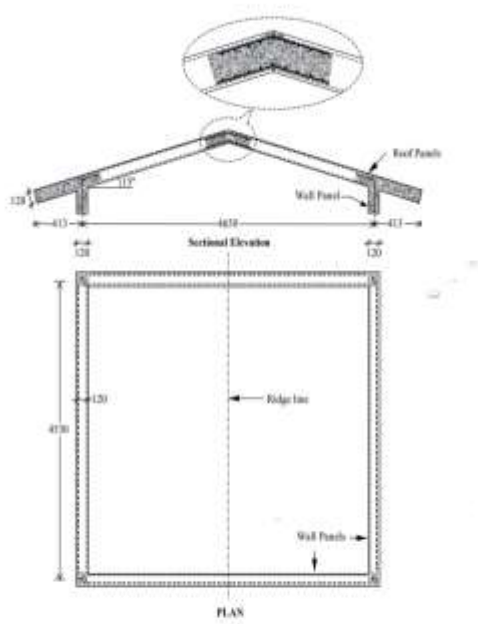



Fig. 20. Typical Unfilled GFRG pitched roof

## V. Design of an Amphibious House

### V.1. Notations

| | | |
|---|---|---|
| Width of Cavity | = | bc |
| Depth of cavity | = | dc |
| Height of panel | = | hp |
| Number of panels | = | np |
| Unit weight of concrete | = | $\gamma_{conc.}$ |
| Depth of slab | = | ds |
| Width of slab | = | bs |
| Length of slab | = | ls |
| Shear Force | = | Vu |
| Effective Length | = | Leff |
| Factored Load | = | Wu |
| Shear Stress | = | Tv |
| Shear Strength | = | Tu |

Note: All the other Notations have their usual meanings.

### V.2. Weight of the structure

Unit weight of light weight conc. used with reinf = 15 KN/$m^3$
Unit weight of GFRG panel =0.433 KN/$m^2$
Unit weight of concrete used for base slab = 25 KN/$m^3$
Unit weight of ferrocrete used for pontoons = 24 KN/$m^3$
Unit weight of conc. in bottom surface of floor slab = 16 KN/$m^3$
Unit weight of steel = 78 KN/$m^3$

W (light weight conc. in GFRG) = $b_c$ X $d_c$ X hp X np X $\gamma_{conc.}$
Weight of light weight concrete in GFRG panels = 7000 kg. _(i)

Weight of GFRG panels = unit weight panels X A(panels)X np
Weight of GFRG panels =9352kg. ____________________(ii)

Weight of bottom slab = ds X ls X $\gamma_{conc.}$ X bs
Weight of bottom slab =30375kg. ____(iii)

Weight of Floor slab = depth of screed layer X surface area X unit weight + cavity width X depth X length X no. of cavities filled X unit weight + unit weight of panel X surface area
=15186kg_______________(iv)

**(Thickness of pontoon = 0.1 m)**
Weight of pontoon = conc. Used in one pontoon X no. of block X unit wt.
= 53568 kg. ________________(v)

W (steel beam in base slab) = sectional area X length X no. of steel beams X unit Weight steel

=886.9kg. ______________(vi)

Total weight of structure = (i) + (ii) + (iii) + (iv) + (v) + (vi)
= 116367.9 kg

Weight pontoon can lift= {(width/4–thicknessof pontoon– width of flange of Flange of steel beam) X height of block X length X No. of blocks) X (unit weight of water kg/$m^3$)}

Weight pontoon can lift = 136800 kg.

**Hence, the pontoon can lift the house.**

Now, check the freeboard height**:**
Weight of house = A(pontoon) X height X density of water
116367.9 = (1.9 X 9 X 4) X h X 1000
Height, h = 1.7 m

Freeboard = 2 – 1.7
Freeboard = 0.3m

### V.3. Design of depth of base slab

Now, assuming the weight of pontoon to be as an U.D.L. on the slab 14.88 KN/m.
Now, taking depth of slab as 110mm and checking for all the conditions.

| | | |
|---|---|---|
| L.L. | = 14.88 + 2 | = 16.88 KN/m |
| FLOOR FINISH | | = 0.6 KN/m |
| SELF WEIGHT | | = 2.75 KN/m |
| TOTAL U.D.L | | = 20.23 KN/m |
| FACTORED U.D.L. | | = 30.3 KN/m |
| $L_{EFF}$ | | = length + d |
| $L_{EFF}$ | | = 2.36 m |

Max. Bending Moment = $W_u L_{EFF}^2 / 8$
Max. Bending Moment = 21 KN.m

Check for flexure:
$B.M._{max.}$ = $0.138 f_{ck} b d^2$
Depth, d = 66 m. <110mm ∴ safe.

Shear force, Vu = $W_u L_{EFF}^2 / 2$
Vu = 35.75 KN
Area of Steel, Ast = $0.5 f_{ck} bd/fy$ X {1 – (1- 4.6 X Mu/ fck $bd^2$)}
= 563.21 $mm^2$
Provide 6 bars of 12 mm diameter.

Spacing Sv,

a. **Sv** $= \dfrac{\frac{\pi}{4} X\ sq.dia\ X\ 1000}{\frac{\pi}{4} X\ sq.dia\ X\ no.of\ bars}$
= 167 mm
b. Sv = 3d or 300mm

Provide, 12 mm dia. Bars at 167mm c/c.

Distribution Steel = 0.12% of bD
= 156 mm.
Use 8mm dia bars, provide 4no. of bars.
Sv = 250 mm.
Sv = 450mm or 5d

Provide, 8mm dia. Bars at 250 mm c/c.

**Check for Shear**

A. Shear stress, Tv = Vu/ bd

= 0.32 N/mm$^2$

% Pt = Ast/bd X 100

= 0.6 %

Shear strength, Tc = $k_s$X0.536(from table 19 I.S. 456: 2000)

= 0.69 N/mm$^2$

Tv < Tc

Hence Safe.

Check for Deflection:

$F_s$ = 0.58 X fy X $Ast_{req.}$ / $Ast_{pro.}$

= 200 N/mm$^2$

For % Pt = 0.6% and Fs = 200 N/mm$^2$, M.F.T. = 1.45

Depth, d = Leff / 20 MFT

= 81.38 mm < 110 mm Hence safe.

V.4. Design for First floor slab

Effective Span:

Effective depth assuming 12mm diameter bars, d = 145 mm.

Effective span as per clause 22.2 of IS:4562000: Effective span (l) is the minimum of:

Clear span + effective depth, l = 4.5 + 0.145 = 4.645m

Weight of infilled conc. ( every 3rd cavity filled) =

$$= 0.05 \times 15 + \frac{0.094 \times 0.23 \times 16}{0.75}$$

= 1.21 kN/m$^2$

Floor finish = 0.75 kN/m$^2$

Live load as per IS: 875 (Part-2)1987 = 2.00 kN/m$^2$

Total service load, *w*

Mud = 1.5 X 4.39 X 4.645$^2$

= 17.75 KNm/m

Design BM rib, Mud = 0.75 X 17.75

= 13.3 KNm

Mud/bd$^2$ = 13.36 X 10$^6$ / 230 X (145)$^2$

= 2.75 N/mm$^2$ From table 3 of SP-16,

% Pt = 0.89%

Ast = 296.815 mm$^2$

Provide 2 bars of 12 mm dia. And 1 bar of 10 mm dia. giving an area of 305 mm$^2$.

Shear Force , Vu = 1.5 X 4.39 X 2.75 X (2.25 – 0.5)

Vu = 10.39 KN

Tu = 0.311 N/mm$^2$

From Table 19 I.S. 456-2000,

From,

% Pt = 0.94 %

Tc = 0.62 N/mm$^2$ > 0.32 N/mm$^2$

Hence, only nominal stirrup steel is required.

Minimum stirrup steel,

Asv = (0.4bsv) (0.87 fy)

Maximum spacing, SV max = 0.75 × 145 = 108 mm ≅ 100 mm

$Asv_{min}$ = 42.3 mm$^2$

Provide 6 mm dia. two-legged mild steel stirrups @ 100c/c.

Nominal steel for screed concrete = (0.12 / 100) × 50 × 103 = 61.2 mm$^2$ / m

Provide 10-gauge welded mesh @ 100 mm c/c on top.

Note :

There will be nominal bending moments in the long span (perpendicular) direction, resisted by the composite action of GFRG (without ribs) and screed concrete.

The limiting tensile strain should not be exceeded at the soffit of the GFRG, to avoid cracking of GFRG in this direction.

V.5 Check for Panel

Weight of GFRG panel = unit weight × length × breadth

= 11.691 kN

Weight of concrete in hollow space of GFRG panel = length of hollow space × depth of hollow No. of spaces filled × height of panel= 8.75 kN

U.D.L. of GFRG panel and light weight concrete = $\frac{11.691 + 8.75}{9}$

= 2.27 kN/m..(i)

Total udl on the panels of slab = depth of screed × density + udl for panel + live load + floor finish + weight of empty slab panel

$$= 0.05 \times 15 + \frac{0.094 \times 0.23 \times 16}{0.75} + 2 + 0.75 + 0.433$$

= 4.39 kN/m$^2$

UDL on panels of 1 floor due to slab

$$= \frac{(u.d.l \times area\ of\ slab)}{length\ of\ panel \times no.\ of\ panel}$$

= 9.87kN/m (ii)

Total u.d.l on panels of ground floor = (i) + (ii)

= 14.26 kN/m

= 1.21 kN/m$^2$

= 0.75 kN/m$^2$

= 2.00 kN/m$^2$

= 4.39 kN/m$^2$

The factored axial load on the wall panel is given by 1.5 U.D. L as:

Pu = 1.5 × (14.26)

= 21.39 kN/M

Minimum eccentricity = t/6 = 20.7 mm

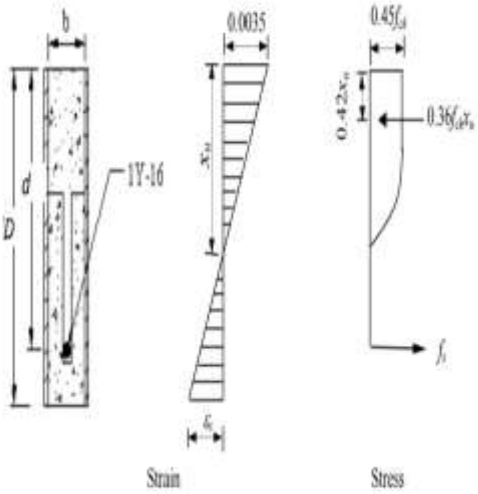
Design capacity in axial compression,

$P_{ud}$ = (68 – 0.9 × 20.7)

= 49.37 kN/m > $P_u$

Hence, ok.

V.7. Design of Lintel




Effective Depth = 300 – 30 – 8
= 262 mm

Design of Shear Force = (1.5/2 – 0.262) X 14.26
= 7.12 KN < 25 KN Hence Safe.

Depth of neutral axis assuming section as under reinforced,

Xu =0.87fyAst/0.36fckb
= 262 mm

$Xu_{max.}$ = 0.48Xu
= 126 mm

Hence, the section is under reinforced.

$M_R$= 0.87fy Ast (d- 0.42 Xu)
= 9.21 KN.m

Design Bending Moment,

Mu = $Wl^2/8$
= 4 KN.m < $M_R$

Hence, the section is safe.

We have provided 12 mm dia. Bar reinforcement.

VI. PLAN OF THE AMPHIBIOUS HOUSE

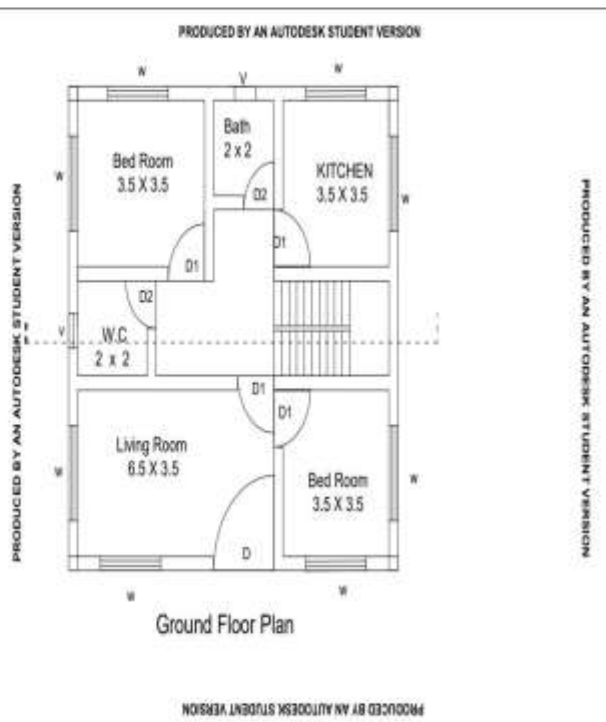



Fig. 21 Ground Floor plan

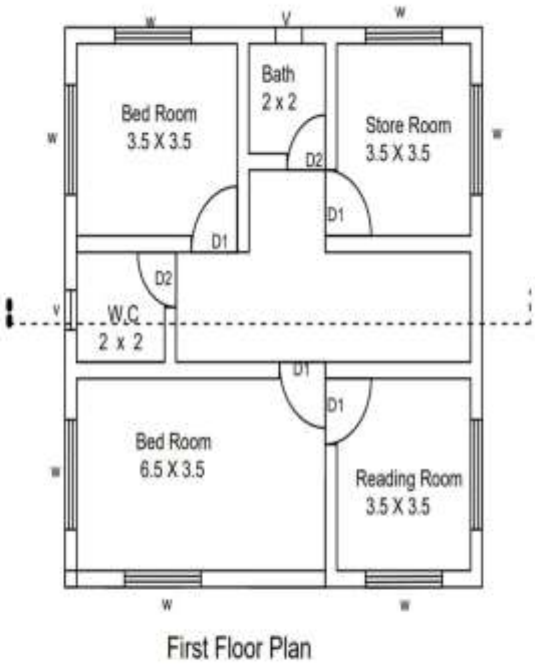



Fig. 22 First floor plan

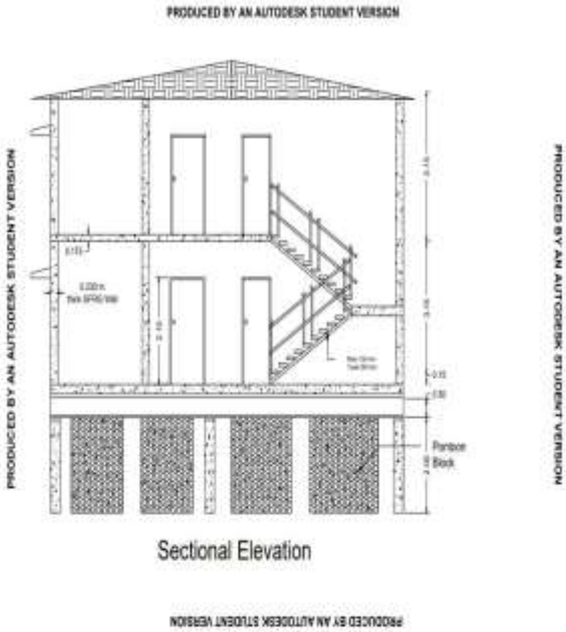



Fig. 23 Sectional Elevation

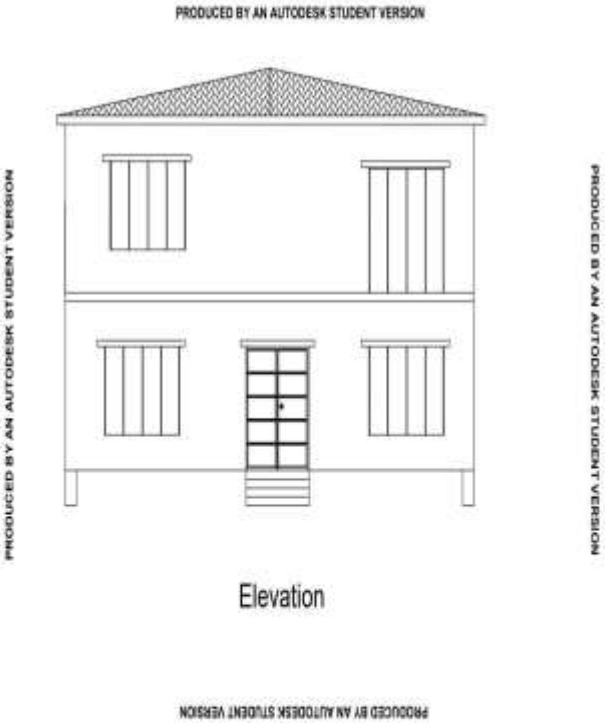



Fig. 24 Elevation

## VII. CONCLUSION

Floating urbanization is a high-potential strategy for flood vulnerability mitigation. There were different types of mooring and floating system, which could be applied for floating urbanization. By concluding from all, amphibious house with concrete pontoon is the most appropriate and applicable choice in India. Lateral forces during flood rush are transferred by roller fenders and absorbed by lateral columns. The mounting system designed for normal house loading, and point load analysis have been conducted for it. Pre-cast concrete pontoons, which are filled by expanded polystyrene blocks (EPS) are one of the approaches towards economic and time saving strategy. These pontoons provide buoyancy for the whole system. The number of pontoons, and pattern of positioning calculated. Tilting of the system is considered for less than 5 degrees and the free board of floating house was considered 20 cm. Based on the questionnaire survey the acceptance and suitability perception of lowland settlers is in a moderate level, and it triggers to be high while their perception of floating house was increased. On the other hand, this method helps to boost the land value and give buffer time for any evacuation, if it is necessary. The use of GFRG panels also helps to reduce the construction cost of super structure up to 30% and it is eco-friendly. Thus, by more attempt on promoting amphibious system and making the show rooms the acceptance level would be increased. Based on the results following strategies should be applied to implement amphibious house in South-East Asia.

i. Promoting R&D and training Centre;
ii. Subsidizing from government for low population and rural area;
iii. Promoting the exhibitions and showrooms for different approaches.

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## REFERENCES

1. STABILITY AND RELIABILITY OF AMPHIBIOUS HOUSE FOR FLOODPLAIN AREAS by Mohammed Ali Nikoi – Universiti Malaysia
2. AMPHIBIOUS HOUSE, a novel practice as flood mitigation strategy in flood plain area by Department of Structures and Material, Faculty of Civil Engineering.
3. AMPHIBIOUS FOUNDATIONS AND BOUYANT FOUNDATION PROJECT: Innovating strategies for flood resilient housing by Elizabeth C English, University of Waterloo.
4. Better living conditions for floodplain areas by Naing N, Indonesia.
5. SP-16 design aids for reinforced concrete.
6. Rajput. A text book of fluid mechanics. 4th ed. GK publishers.
7. http://www.iiste.org/Journals/
8. IS 875 (Part 1 and 2) – Dead load and Imposed load – code of practice for design load for building and structure. Bureau of Indian Standards. 1987.
9. Sundarraj M. Study of compact ventilator. Middle - East Journal of Scientific Research. 2013; 16(12):1741–3. ISSN: 1990-9233.
10. GFRG design manual. Structural Engineering Division Department of Civil Engineering, IIT Madras.