ALMA MATER STUDIORUM - UNIVERSITÀ DI BOLOGNA

FACOLTA' DI INGEGNERIA

CORSO DI LAUREA SPECIALISTICA IN INGEGNERIA CIVILE

DICAM

Dipartimento di Ingegneria Civile, Ambientale e dei Materiali

TESI DI LAUREA

in

Tecnica delle Costruzioni LS

DESIGN AND CONSTRUCTION OF CONCRETE SEAWALLS REINFORCED WITH GFRP I-BARS USING AND STAY-IN-PLACE FORMWORKS

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Anno Accademico 2009/10

Sessione III

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ABSTRACT:

Traditional building materials, such as timber and concrete, and non-traditional building materials, such as composite and aluminum, are currently used for waterfront construction. As in the past, the engineering challenge remains durability and low maintenance. A combination of concrete and internal glass fiber reinforced polymers (GRFP) reinforcement represents a solution with high potential, to reduce steel reinforcement corrosion, particularly when construction costs and completion time could be simultaneously reduced by introducing integrated GRFP structural reinforcement and stay-in-place (SIP) formwork.

The research intends to investigate the long-term performance of concrete seawalls using structural SIP panels. The project tasks include design of a prototype, construction, accelerated ageing and deployment at waterfront site, and laboratory testing.

GFRP RC seawalls represent an economically competitive alternative to conventional solutions, such as steel RC or timber seawalls, because of its resistance to corrosion, high strength to weight ratio and excellent fatigue performance.

This study is emphasis is the research about the art of the seawall construction in South Florida.

ABSTRACT

Through field studies, it has been possible to categorize three principal systems of seawall construction:

- Vinyl/FRP or Steel Sheet pile (anchored and not);
- Anchored Panel wall and Vertical pile;
- Panel wall, King pile and Battering pile.

GFRP is sensitive to aggressive environment, such as humidity, high alkalinity, and high temperature. Therefore, the application of GFRP in concrete structures requires the development of design criteria that must take into account both mechanical and durability properties. A review of the research literature reveals that, while the GFRP mechanical properties have been widely investigated and short-term design concepts are well established, experimental data on the long-term physical and mechanical behavior of GFRP is scarce.

The objectives of the activity are to:

- Investigate the longevity of the concrete seawalls reinforced with GFRP SIP panels;
- Provide the scientific community and practitioners with experimental data on the long-term performance of GFRP SIP panels when exposed to harsh environment;
- Provide the engineering community with new design tools to build long-lasting RC seawalls.

Four prototype seawall panels have been constructed. From each panel will be obtained 24 beams specimens, which will be then divided in nineteen triplets and subjected to different ageing treatments over a period of 12 months.

Five types of ageing treatments will be considered: Preserved Ageing; Natural Ageing; Accelerated Ageing at Room Temperature; Accelerated Ageing at 104° F ans Accelerated Ageing at 140° F.

At the end of 12 months, each beam specimen will be subjected to a four-point bending test to evaluate its bending moment at failure

The extrapolation of the bending moment at failure for beam specimens subjected to different ageing treatments will lead to the definition of new design concepts for the design of long-lasting RC seawalls (C_E, Environmental Factor. ACI 440.1R-06).

Also, will a guide to assist the design and the construction of concrete seawalls reinforced with GFRP grating with SIP forms is provided to Strongwell Company. The content of this study is fallowing reassumed:

- In the Chapter 1 and 2 there is a literature review about the seawall structure and design, the overall concept and the different types and components, the hydraulic performance and the modes of failure;

- In the Chapter 3 is reported the state of the art of the seawall construction in South Florida, an work which born from field studies on West Palm Beach, Miami (FL), Manthesan Hammark Park, Miami (FL) and William Island, Aventura (FL);

- In the Chapter 4 is described the software realize to design the Panel Wall adopting the characteristics (dimensions and strength) provided from the Company that finance the research program;

- In the Chapter 5 is possible to find information about the durability of seawall construction and in particular information about the durability of Concrete and FRP, which represent the principal constituents of the RC seawall;

- In the Chapter 6 is reported the Experimental Program that will provide the scientific community and practitioners experimental data on the long-term performance of GFRP SIP panels when exposed to harsh environment.

ABSTRACT

The content of this thesis is an assembly of text and figures that were taken from other sources. These items are not identified through the text. This work cannot be considered original. The original sources are listed as follows:

- Thomas R.B., Seawall Design, Oxford (1992) – Chapter 1 and 2
- Das Braja M., Principles of Foundation Engineering, PWS Publishing Company (1995) – Chapter 1 and 2
- Colombo Pietro, Elementi di Geotecnica, Zanichelli (2004) – Chapter 1 and 2
- ASTM C134/C143M-10 – Section 6.2.2.2
- ASTM C231/C231M-10 – Section 6.2.2.3
- ASTM C138/C138M-10a – Section 6.2.2.4
- ASTM C39/C39M-09a – Section 6.2.2.5

1 CHAPTER:

SEAWALL STRUCTURE – OVERALL CONCEPT AND TYPES

The Chapter 1 is a literature review of three important books about the marine construction and about the geotechnical, that are the “Seawall Design” of Thomas R.S.; the “Principles of foundation Engineering” of Braja Das and the “Elementi di Geotecnica” of Pietro Colombo.

This chapter provides information only background.

To properly asses the requirements for a bulkhead or seawall, the design professional should fully understand and differentiate the purpose of these two structures. Both structures, seawall and bulkheads, serve the purpose of a vertical shoreline stabilization. Both structures utilize similar constructions techniques and similar construction materials. However, the structures are not the same

- A bulkhead is a vertical shoreline stabilization structure that primarily retains soil and surcharge loads behind the wall;

- A seawall is a structure that has two primarily functions:
    - retaining soil and surcharge loads behind the wall, and
    - protection of shoreline from wave loads.

In addition, seawalls typically protect frontline beaches from storm surges, shoreline erosion and wave overtopping. Some waterfront properties are subjected to significant wave activity during the storm surge events, even though they are not exposed to wave actions for the most part of the year (1).

The following design considerations are normally addressed by the designer of a seawall as compared to the designer of a simple bulkhead:

- Direct wave force action
- Uplift force imposed by wave action
- Wave overtopping
- Storm surge
- Toe scour.

Many existing waterfront properties around the country, including both East and West Coast shoreline as well as shoreline of the Great Lake, were designed using a simple bulkhead approach that neglected wave forces. As result, many waterfront properties suffered substantial structural damage and had costly maintenance problems.

The shoreline is an environment hostile to structures placed there. It is also a valuable and sensitive part of the environmental (2).

1.1 Design philosophy

A prerequisite for determining design criteria is the selection of the overall design standards, the required life of the seawall and the acceptable risk of being overwhelmed by exceptional waves or tides.

1.1.1 Life of the seawall

In different aspects of the design process there are a number of various 'lives'.

- Structure life (serviceable, or working). The length of time that the structure actually lasts and is capable of functioning.
- Component life. The life of a particular component of the structure, i.e. the length of time which it lasts before requiring replacement.
- Design life. The minimum length of time the component structure or scheme is designed to last.

Cleary, when the component life is substantially less than the design life, maintenance plays an important role in the eventual structure life.

The choice of design life requires careful consideration and regard for the marine environment in which it will function. The use of a long design life may result in a very costly scheme. A shorter design life may be predetermined by expected changes in the use of that land protected or may show financial advantage even after provision is made for future replacement. It is usual for the design life chosen to be the economic optimum solution by which the functional requirements can be met. The choice is often strongly influenced by the availability of inexpensive local material of limited durability.

The common construction materials are:

- *Concrete* provides services life of 30+ years if correct mix design and proper marine structural design implemented. This material is often used in South Florida due to the locally available aggregate;
- *Steel* provides service life of 25+ years;
- *Aluminum* do not use in waters with low Ph or backfill with clay-mucky soils. Difficult to install in hard substrates, so it is an obsolete material for this kind of construction;
- *Timber* Not often used in South Florida, generally economical material, but limited strength characteristics for high wall heights. Difficult to install in hard substrates;

- *Vinyl/plastic* relative new economical product with services life of 50+ years.

The Code of Practice for Maritime Structures (UK) gives recommended minimum design life for shore-protection works and breakwaters as 60 years and for sea-defense as 100 years.

Seawalls on the coast of Florida come under the jurisdiction of the Florida Department of Environmental Protection (DEP). In addition to evaluating the structural condition of a seawall, the DEP has special requirements for seawalls at or near the Erosion Control Line (ECL). A coastal engineering analysis is required to determine if an existing seawall will be affected by a 30-year coastal storm event. If the existing wall is within the 30-year Erosion Projection, then the property owner must “provide scientific and engineering evidence that the armoring structure (seawall) has been designed, constructed, and maintained to survive the effects of a 30-year storm and provide protection to existing and proposed structures from the erosion associated with that event.” The DEP requires certification by a professional engineer that the seawall was designed, constructed, and is in adequate condition to meet the following criteria:

- The top of the seawall must be at or above the predicted maximum wave crest elevation, considering the eroded beach profile, of the 30-year design storm.
- The seawall must be stable under the 30-year design storm including localized scour, with adequate penetration and toe protection to avoid settlement, toe failure, or loss of material from beneath or behind the armoring.
- The seawall must have sufficient continuity or return walls to prevent flanking under the design storm from impacting the proposed construction.
- The seawall must withstand the static and hydrodynamic forces of the 30-year design storm(2).

1.1.2 Risk of design criteria exceedance

Phenomena such as winds, waves, tides and surges are fundamental design factors.
Extreme values of these are, therefore, essential design criteria. However, there are frequently no absolute maximum values of these. In circumstance when there is no realistic maximum, the solution is to choose extreme values which are very rare. It follows, therefore, that there is a finite chance that the design condition will be exceeded during the life of the wall.

For convenience, this probability of exceedence is normally characterized by "Return Period" (T_R), an event with a return period of N years is likely to be exceeded, on average, once in N years. The most appropriate return period should be chosen in consideration of the consequences of exceedance. The return period for the various design criteria should not be confused with design life. For example, if the return period of any extreme event is set the same as the design life, then is a 63% chance that the extreme event will be exceeded before the end of the design life:

$$R = 1 - \left(1 - \frac{1}{nT_R}\right)^{Ln} \tag{1.1}$$

$$T_R = \frac{1}{n} \cdot \frac{1}{1 - (1 - R)^{1/Ln}} \tag{1.2}$$

From eq. (1.1) is possible find the R = risk, given L = design life and n = number of events.
From eq. (1.2) is possible find T_R = return period.

1.1.3 Probabilistic design

The design of the seawalls is probabilistic to the degree that wave, currents and water levels are evaluated probabilistically, with a statistical risk of exceedance. These factors are then applied deterministically, i.e. by determining the strengths, size of members, etc. that are require to resist or accommodate these factors. Full probabilistic design entails applying probability distributions to all the properties of the wall as well as to the wall. It is costly and requires a very thorough understanding of the mechanics of the wall, although it leads to a more rigorous, reliable and efficient design. Advances in research into the working of seawalls and in computational techniques are making probabilistic design less costly. However, at present it is only for the major schemes that full probabilistic design is feasible(3).

1.1.4 Design criteria

The definition of the problem to be solved, the consideration of environmental aspects, and a preliminary cost/economic assessment should determine the key functional requirements of the wall. Different defense options can then be evaluated, although considerable additional data and analysis may be required before selection of options and subsequent design can be finalized.

1.1.5 Analysis of failure

The central philosophy of approach to seawall design is to consider possible modes of failure and design against them. When such a failure occurs it is a result of the imposed loads exceeding the capacity of the wall to withstand them (including consideration of the coastal slope and foreshore). This may be because of:

- Inadequate assessment of design loads;
- Inadequate assessment of design capacity;

- Construction problems giving actual capacity less than design capacity;
- Construction problem giving actual capacity less than design capacity;
- Maintenance problem giving actual capacity less than design capacity;
- Deterioration of wall capacity with time;
- Increase in loads with time; and
- The capacity of the wall being exceeded as a result of a particularly severe event.

In the case of sea defenses, the following main groups of failure which might lead to flooding can be distinguished as(4):

- Flow under or through the wall;
- Flow over the seawall;
- Damage to the front face leading to breaching of the seawall
- Geotechnical instability;
- Slope instability.

Plan layout

The hydraulic performance of a wall can often be made much more efficient by altering its plan shape.

1.1.6 Location of the structure with respect to shoreline

In general terms it is desirable that the wall be located as far landward as possible to minimize both interference with the coastal regime and the hydraulic loads on the structure. Also are noted the difficulties in assessing the detailed impact of a new seawall on the coastal regime. One very important in trying to assess the impact of the seawall the engineer should be aware of the complexities of coastal processes and facts that are almost always in a state of change. In most cases, even with the most sophisticated investigations, it is not possible to accurately to predict future changes. In siting new walls, two consequences should be recognized:

- By virtue of its existence, the new wall may cut off the natural supply of material to littoral regime, the effects of which may be felt at considerable distance from the site in question;
- If the seawall is well landward of normal high water mark, acting only as a “fallback” in times of severe erosion or extreme surges, the effect on the coastal region is likely to be minimized. If, however, the wall is built further seaward, it will be in deeper water and the consequences of wave reflection and other effects on the coast are likely to be greater. Depending on its location or distance out from the shoreline, local changes or adjustment of the seabed must be allowed for as a result changed wave conditions in front of the wall.

1.1.7 Plan shape and alignment

In general, the wall alignment should follow the average alignment of the beach contours. Where the new structure is a replacement for a previously collapsed or failed structure the engineer should consider the influence, if any, of the previous wall alignment as a contributory factor in the collapse.

Plan layouts of seawalls which embody concave curvatures with respect to incoming wave crests can result in a focusing of reflected wave energy in a limited area offshore with potentially detrimental effects on beach process. A more extreme case arises with sudden changes of direction of walls which involve re-entrant angles, and these are to be avoided if at all possible. They can cause serious concentrations of wave impact on the wall where waves are focused onto a small area.

Re-entrant angles can also result from the introduction of access steps to the beach, concrete buttresses, bastions, outfalls and other discontinuities. Consequently they should, where possible, be avoided.

In both cases the concentration of incoming wave energy will result in local wave set-up and increased overtopping. If the walls in question are vertical or near-vertical, large wave forces will be generated, in particularly in the case of re-entrant angles, and excessive overtopping will occur with wave and spray being projected upwards to a great height.

1.1.8 Junctions and terminations

Some of the most difficult problems in seawalls occur in relation to an existing structure such as a breakwater or the termination of a seawall on an unprotected coast.

The introduction of curves and re-entrant angles in the plan shape of seawalls will automatically call for an assessment of the effect of oblique waves and resulting increases in the beach movement(5).

1.2 Cross section

Conditions vary greatly from site to site, and there is therefore no such thing as a standard cross section for a seawall. Each design should be considered in its own right, taking into account all the wide variety of functional requirements. This is self-evident in establishing the plan layout but is no less pertinent with respect to the cross section.

The success of the wall, whether its aim is to alleviate flooding or prevent erosion, will be fundamentally dependent on:

- Preventing wave action from reaching landward of the wall;
- Maintenance of its integrity under extreme conditions of waves, beach level changes, abrasion, etc.;

- Its intersection with coastal processes.

1.2.1 Seawall classification

To allow general discussion of the various types of seawall it is convenient to classify them under four heading:

- Sloping;
- Vertical
- Porous;
- Non-porous.

These are defined as follows:

Sloping: walls having a slope shallower than 1 in 1 (45°);

Vertical: walls having a slope of 90°;

Porous: walls whose face is permeable to wave action (e.g. a rubble slope, perforated blocks);

Non-Porous: walls whose face is not permeable to wave action (e.g. in-situ concrete slope, stell sheet-piled wall).

The porous/non-porous classification refers only to wave action

1.2.2 Structural concept

A seawall can be regarded as having elements which need to be combined to produce a coherent structure. The elements then comprise components appropriate to the function required of them. In order to examine the options for each element and the choice of suitable components, the following have been adopted for discussion (Figure 1.1):

- Body (including the front face and core);
- Toe;
- Crest (including the back face).

Each element performs specific functions which interact with the other (6).

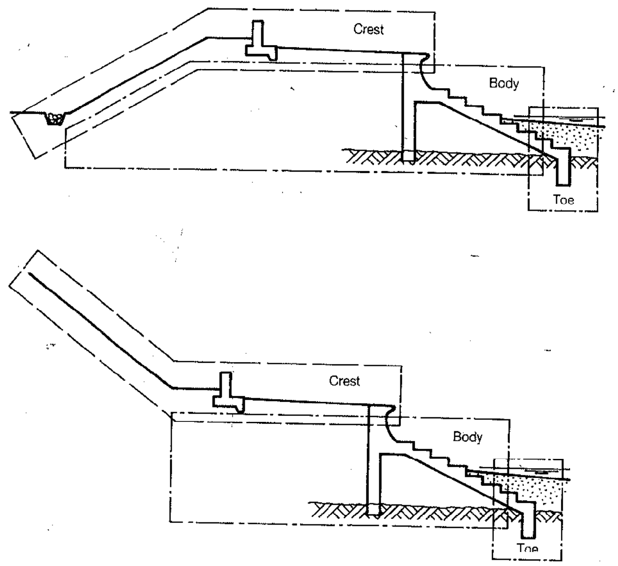

Figure 1.1: the elements of a seawall (6)

1.3 The body of the wall

The body is the major part of the seawall and has a great influence on its performance. In order to select the most appropriate type of construction the suitability of each type of wall structure should be assessed against the functional requirement, including:

- Stability against wave attack;
- Wave reflection;
- Run-up and overtopping;
- Spray;
- Aesthetics;
- Durability and likely life;

- Ease of construction and requirements for construction (access, working area, etc.);
- Availability of materials;
- Require level and ease of maintenance;
- Flexibility (ability to accommodate scour or settlement);
- Strength (ability to resist imposed loads);
- Ease of access along and cross the wall;
- Safety;
- Cost.

1.3.1 The core

The core of a seawall is often a significant proportion of the total cost. In general, when fill is required as backing to a vertical seawall or retaining wall it should be good quality, inert, free-draining granular fill such as would be used in general works of civil engineering.
The choice of fill material for a sloping seawall will be strongly influenced by both local availability and nature of the subsoil at the site. It is inherent in the nature of many sea defenses that locally available fill and foundation soils consist of marsh clays, often in association with silt or peat.

1.3.2 Vertical walls

The type of the seawall that characterized our research is the vertical one, so now we focus about this kind of construction:

- Vertical non-porous wall;
- Vertical porous wall.

1.3.2.1 *Vertical non-porous wall*

Vertical non-porous walls are very reflective. They have advantage of occupying little space compared with that required by a sloping structure but require steps or ramps to effect access to the foreshore. Vertical non-porous walls often prove cost-effective when they also act as retaining walls, for example when at the seaward edge of a promenade or coastal road. In these cases the walls should be designed primarily as retaining walls, taking into account the other functional requirements such as stability, ease of construction, acceptable overtopping, durability, etc. Vertical non-porous walls can therefore be conveniently considered under four headings corresponding to five main types of retaining wall, i.e. gravity, cantilever, sheet-pile, anchored wall and reinforced soil(7).

- *Gravity walls* (Figure 1.2)

Gravity walls, by their nature, are required to be massive and the most suitable materials are mass concrete, masonry, concrete blockwork and brickwork. They depend on their weight and any soil resting on the masonry for stability. Their use was widespread until the last 40-50 years, when increasing labor costs and changing construction techniques often made them more expensive than other types of wall. They require little maintenance subject to sound foundations. In locations such as rocky areas and seaside towns they can blend in well with the local surroundings if faced with local stone.

Reinforced concrete called semi-gravity walls come under the broad heading of gravity walls in that, overall, they function as gravity walls, thereby minimizing the size of the wall sections (8).

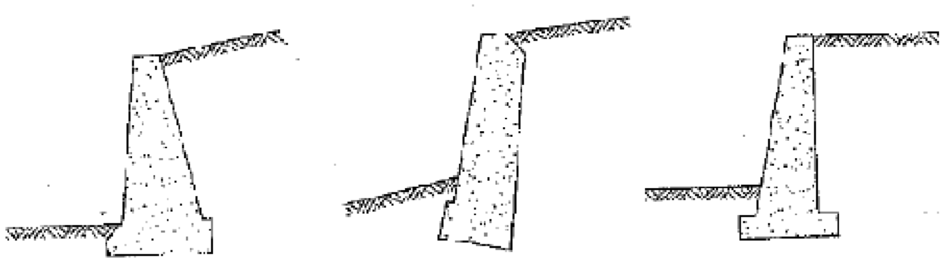

Figure 1.2: gravity and semi-gravity walls (9)

- *Cantilever reinforced concrete walls* (Figure 1.3)

Cantilever walls are made of a reinforced concrete (L-shaped) and consist of a thin stem and base slab. This type of wall is economical to a height of about 25 ft (8 m). The 'cantilever' element refers to the front face, which is a vertical cantilever from the base. They can provide considerable potential for precasting, given the requisite access. Precasting in turn offers improved durability (conditional on adequate reinforcement cover and quality in the finished wall) and alleviates some of the problems associated with tidal working. In this category fall also the type of wall called counterfort retaining walls, that are very similar to cantilever wall, but at regular intervals they have thin vertical concrete slabs known as counterforts that tie the wall and the base slab together. The purpose of the counterforts is to reduce the shear and the bending moments (8).

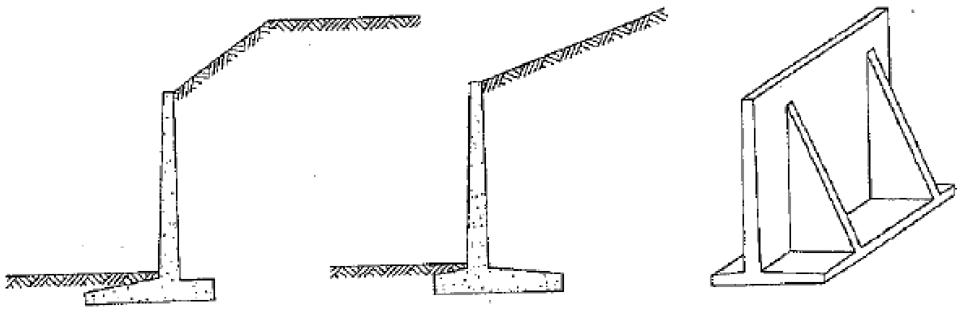

Figure 1.3: cantilever reinforced concrete wall and counterfort wall (9)

- *Cantilever Sheet – pile* (Figure 1.4)

Connected or semi-connected sheet-piles are often used to build continuous walls for waterfront structures that may range from small waterfront pleasure boat launching facilities to large dock facilities. In contrast with the other types of retaining wall, the building of sheet pile walls does not usually require dewatering of the site. Sheet piles are also used for some temporary structures, such as braced cuts. They are normally tied to produce a more effective solution which is more tolerant to variations in soil conditions and ground level. Also they are characterized by relative ease of construction, because there is no need to excavate for foundations or shutting. There are not restrictions due to tidal working.

Several types of sheet pile are commonly used in construction. Wooden sheet piles are used only for temporary light structures and are above the water table. Precast concrete sheet piles (Figure 1.5) are heavy and are designed with steel reinforcements to withstand the permanent stresses to which the structures are subjected during construction. Steel sheet piles (Figure 1.6) in United States are about 0.4-0.5 in (10-13 mm) thick and the shape of the section may be Z, deep arch, low arch or straight web, instead the interlocks are shaped like a thumb-and-finger or ball-and socket for watertight connections. The steel sheet piles are convenient to use because their resistance to high driving stress developed when being driven into hard soil and for their flexibility in being able to accommodate variation in ground levels and conditions along its length. They are also lightweight and reusable. But exposed steel sheet piling lacks durability when used in a coastal environment. Its disadvantages include: corrosion, particularly between high and low-water level; abrasion from beach material moved by waves; an unsightly appearance; difficulties of repair. A new technology is proposed like a good solution, the vinyl or FRP sheet pile(9).

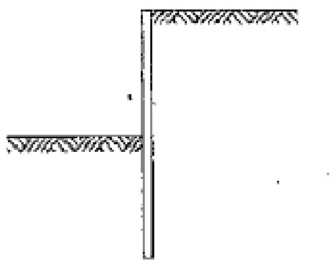

Figure 1.4: cantilever sheet pile (9)

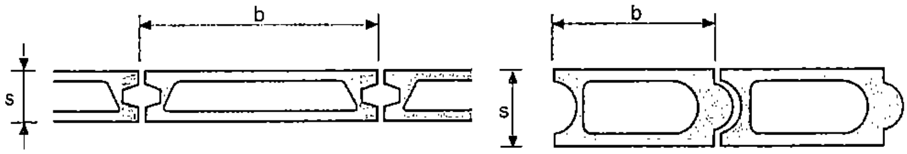

Figure 1.5: example of the shape of the precast-concrete sheet pile (9)

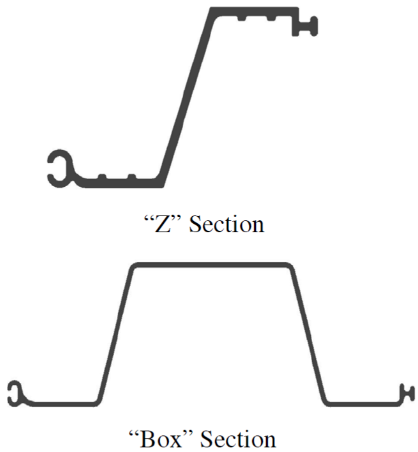

Figure 1.6: two examples of the shape of steel sheet pile (10)

- *Anchored sheet piles* (Figure 1.7)

When the height of the backfill material behind a cantilever sheet pile wall exceeds about 20 ft (6m), tying the sheet pile wall near the top to anchor plates, anchor walls, or anchor piles become more economical. This type of construction is referred to as anchored sheet pile wall or an anchored bulkhead. Anchors minimize the depth of the required penetration by sheet piles and also reduce the cross-sectional area and the weight of the sheet piles needed for construction. However, the tie rods and anchors must be carefully designed.

The general types of anchor used in sheet pile walls are: anchor plates and beam; tie backs, vertical anchor piles, anchor beams supported by batter piles

Anchor plates and beams are generally made of cast concrete blocks. The anchor are attached to the sheet pile by tie rods. A wale is placed at the front or back face of a sheet pile for the purpose of conveniently attaching the tie rod to the wall. To protect the tie rod from corrosion, it is generally coated with paint or asphalt material. In the construction of tie backs, bars or cables are placed in predrilled holes with concrete grout (cable are commonly high-strength, prestressed stell tendons).

The resistance offered by anchor plates and beams is derived primarily from the passive force of the soil in front of them. If the anchor is placed inside the wedge of active pressure, it would not provide any resistance to failure(9).

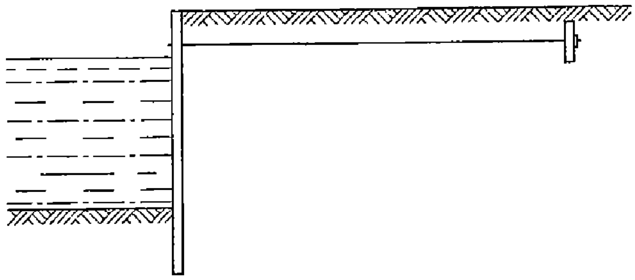

Figure 1.7: anchored sheet pile (9)

- *Reinforced soil* (Figure 1.8)

Reinforced soil is a relatively new system of construction, and has only had very limited use on seawalls. At present, as design techniques are advancing, the engineer should seek assistance from manufactures of the reinforcing system and/or specialist consultants. The system, in situations which can be kept dry during construction, offers potential for substantial cost saving. The fill must, however, be placed carefully and be well controlled, which mitigates against tidal forces. The construction does not require heavy plants. Providing measures are taken to ensure durability of the reinforcing strips and to avoid any loss of fill, i.e. by installing geotextiles down the back face of the facing blocks and by constructing an effective toe, the wall should be effective (7).

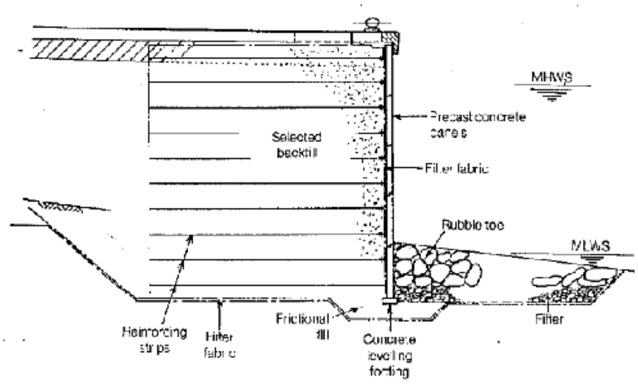

Figure 1.8: vertical seawall using reinforced soil (11)

1.3.2.2 *Vertical porous walls*

Vertical porous walls offer the potential for reducing somewhat the high reflections that occur with non-porous walls. Unlike non-porous walls, they also have the advantage of being permeable, so the potential problems of blockage of groundwater flow are minimized.

Two types of walls are very common, i.e. gabions and cribwork. These basically comprise stone or rock held at vertical or near-vertical slope by a framework or mesh. Other types of vertical porous wall have been used overseas, notably in japan, and consist of concrete block with built-in voids. They were developed, and are more widely used, for breakwater construction.

- *Stone-filled gabions* (Figure 1.9)

Gabions, conditional on suitable stone fill being available offer a cheap solution. However, they are not durable in areas of limited public access and where they are only occasionally reached by the sea they have performed successfully. In such cases they offer a flexible solution and can be aesthetically pleasing, particularly in rural areas. When acting as a retaining wall, care should be taken in the design to include suitable filters down the back face to prevent leaching out of fill and to allow provision for toe scour (7).

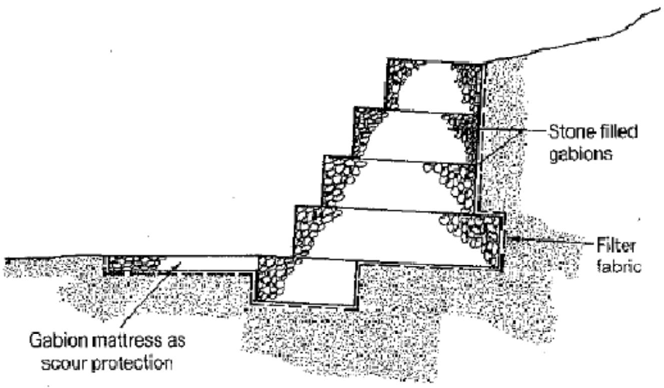

Figure 1.9: vertical porous seawall using gabions (11)

- *Cribwork and breastwork* (Figure 1.11)

Cribwork and breastwork have been used a number of time to protect the base of cliffs from erosion. A cribwork is normally constructed of vertical or sloping timber or steel with cross members. This retains rubble infill.

For the most part, cribwork systems are used as conventional types of seawall, i.e. as abutments to land which might otherwise erode. On occasions they have been used seawards of the toe of a cliff. In this way they act to reduce, rather than eliminate, the wave energy reaching the cliff (7).

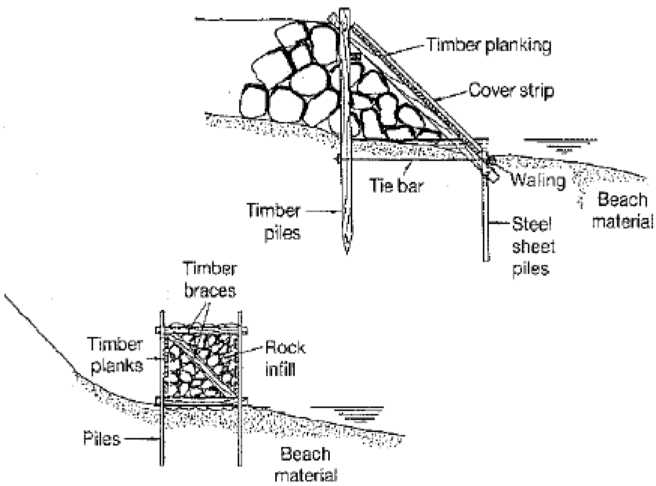

Figure 1.10: porous seawalls using stone-filled cribwork (11)

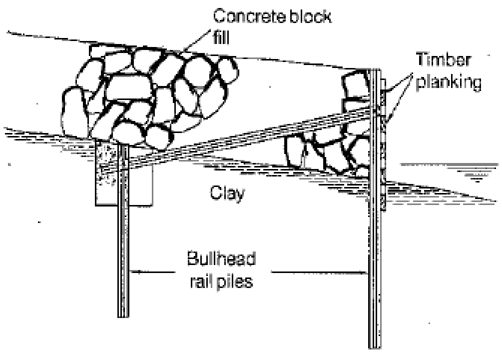

Figure 1.11: porous seawalls using stone-filled cribwork (11)

1.4 The toe of the wall

The toe of the seawall terminates the base of the wall on its seaward face. It is located in that critical zone where the rigid or semi-rigid seawall structure meets the mobile material of the foreshore. CIRIA Report TN 125 indicated that toe erosion is the most common cause of seawall failure, which emphasizes the importance of the toe design.

The term 'apron' is often used to describe a sloping or horizontal construction whose primary purpose is to provide protection against scour and which is located either seaward of the toe or between the body of the wall and the toe.
The primary purpose of the toe is to prevent undermining of the wall. In addiction it may:

- Protect the beach or sub-strata in front of the wall against scour;
- Improve the hydraulic performance of the wall;
- Provide structural support to the wall against sliding or toppling forward;
- Provide a part of the structural foundation of the wall;
- Prevent or restrict seepage.

While these apply to consideration of a new seawall, they apply equally well to the case when works are being carried out to the toe of an existing wall. In satisfying these purpose the toe is likely to be of major significance in the overall stability of the wall. The most likely cause of toe failure is a fall in the beach levels which, with a new seawall in place, cannot be accurately predicted. Even, with a model, the sensitivity of the beach to variations in wave conditions, tides and other changes along the foreshore means that no absolute minimum beach level can be reliably predicted unless the fall is restricted by a hard stratum. Having regard to the potential for overall failure if the toe is undermined, the engineer should be conservative in his estimate of minimum beach levels, and should also carefully consider the consequences of a fall in the beach level below that predicted.

1.4.1 Nature of the toe

In order for it to be successful the toe needs to be able to extend down to a level sufficiently below the lowest likely beach level to ensure that it can be perform all its functions listed above. Alternatively, it can be designed in the knowledge that the beach may well fall below the toe, but that it is sufficiently flexible to drop down to the new lower level without damage. Introducing an apron into the toe construction has the advantage of taking the scour problems further away from the foundation of the wall.

The method of founding the toe may be selected from the following:

- Founded in resistant strata

Where strata, which are relatively resistant to erosion, are exposed or occur within a reasonable depth of the beach level, the toe is founded on the stratum with little risk of undermining.

- Founded in formation with limited resistance

In many instances, clay or other strata with limited resistance to erosion occurs at a shallow depth below the beach and erodes only slowly when it is uncovered. Founding within such strata usually, reduces the risk of undermining arising from unexpected beach changes.

- Founded in beach or another erodible material

Where there is not resistant foundation at reasonable depth in which to found the toe it is necessary to predict the lowest beach level during the design life and found below this level. The risk depends largely on the reliability of prediction and the design of conservation adopted.

- Falling apron in erodible material

As an alternative, prevision can be made to accommodate a fall in beach level below the underside of the toe by means of an apron of rock or other construction which is sufficiently flexible to adjust to the lower beach without suffering damage.

- Prevision for toe extension

As another alternative, prevision can be made for the construction of a seaward extension to the toe at some future date when the beach fallow below a certain level. Such a policy carries a high degree of risk, requires careful monitoring and calls for the ability to mobilize resources at short notice for emergency works. It may nevertheless be the most acceptable solution in economic terms where future changes are uncertain.

1.4.2 Design consideration

The hydraulic performance of the toe should be considered in general terms in conjunction with the whole seawall. Particular consideration should be given to the changes of the beach level on the toe design and the possible progressive exposure of the surfaces which were originally buried and which may give rise to wave reflection. Where an apron is being added to an existing then a specific investigation into the combined hydraulic performance may be necessary. Environmental aspects arising from possible future changes must also be considered in relation to access, amenity and appearance. This can be overriding consideration on an amenity beach.

The design of the toe should also be considered in relation to overall structural design of the wall. In many cases the toe can be designed to assist in resisting sliding, rotation or toppling movements or as a foundation. Structural analysis is normally carried out on the bases of the worst design foreshore conditions that can be expected.

Consideration of construction problems, always essential in seawall design, becomes particularly important at the toe because of tidal working. Account must be taken of the nature of the subsoil and the overlying beach both in terms of:

- The ease of construction of a particular solution;
- The effect of the construction on the subsoil.

Both pile driving and excavation for the toe can affect the structure of the subsoil, leading to additional local scour if it becomes exposed. Design considerations relating to water pressure are described elsewhere (Chapter 2). If the toe design is such that it will affect the flow of tidal and groundwater under the seawall, the impact of such changes will need to be analyzed.

On shingle and cobble beaches, and under exposed conditions, abrasion is likely to be the most severe at the beach level, i.e. at the toe. The effects of temporary and progressive exposure of the toe to abrasion due to changes in beach level should be taken into account. Steel sheet pile toes in particular are vulnerable to perforation for abrasion. If this occurs below the level of any backing concrete, the fill or underlying beach may be washed out by wave, tidal and groundwater flows. This has in several cases led to seawall collapse.

1.4.3 Toe construction – non porous

- *Steel sheet piling*

Given suitable ground conditions for pile driving, steel piling can provide a deep cut-off without the problems of foreshore excavation or the damage to the subsoil. In consequence it is used extensively in situation where cyclical or progressive beach changes are expected. It is particularly vulnerable to combined abrasion and corrosion. The vertical face of the piling can became unsightly and cause wave reflection should it became exposed above beach level. It is therefore usually designed so that it is not exposed above beach level under normal conditions.
Steel sheet piling can be used at the toe of large variety of forms of construction of the body of the wall. These uses may be divided into two categories:

- Where the piling is structurally separate and usually of cantilever construction, as in the case of a rock armored slope or pattern placed armour units;
- Where the piling are structurally connected to form an integral part of the body of the wall, such as in reinforced concrete wall.

In the former case steel sheet piling is usually provided with a wailing or other stiffening members but is self-supporting. In the latter is most commonly used in conjunction with some form of concrete construction which provides the necessity stiffness and support. Structural considerations usually require adequate attachment to transfer loading from the pile head to the seawall. Where steel sheet piling is used with a concrete cap and backing, to form a composite toe, the concrete backing can provide a safeguard against loss of fill due to perforation of the piling and the concrete should extend below the lowest design beach level.

Steel sheet pile also offers an effective means of constructing a new toe to underpin an existing seawall where excavation would cause undermining and consequent risk to the wall.

- *Concrete*

Concrete is particularly suitable for seawall toe construction where the toe is to be founded in a relatively resistant material or on a reasonably stable beach. The use of reinforcement in the toe and the extent of precasting is largely a matter for structural considerations and design for construction. The profile of concrete as used in toe construction can vary from a narrow vertical cut-off to a horizontal slab. The profile is determinate both by structural requirements and the practicalities of construction. The latter is often related to the slopes at which the excavated subsoil will stand. In the cases of a toe in resistant strata it is usually important to completely fill the excavated trench with concrete.

- *Asphalt and bitumen*

Asphalt and composite bitumen and stone materials are used as flexible scour aprons to protect structures or aprons in front of existing seawalls in order to improve hydraulic performance. Care should be taken in using such a toe on a coastal beach. The rapid fall in beach levels that can occur within a few hours of a storm can be too rapid for the asphalt to accommodate without rupture.

- *Timber*

A low timber breastwork can contain and protect the seaward edge of other materials forming a sloping seawall or apron. This is normally either a continuous line of timber sheet piles with a top wailing or capping or timber king piles with horizontal planking.

1.4.4 Toe construction – porous

- *Armour*

Armour comprising either rock or pre-cast concrete units can provide a flexible toe in the form of a 'falling apron'. It can adjoin all forms of wall construction and has the advantage that it can be durable and flexible. However, it can make access to the beach difficult.

- *Rip-rap*

Graded rock rip-rap can also provide a flexible toe either in the form of a berm or flattened slope particularly when there is a possibility of extensive redistribution of the materials under extreme conditions. It has similar advantages to armuor, although the smaller pieces of stone inherent in the winder grading of rip-rap may well cause abrasion problem, when they are moved in storms if the main seawall consist of, for example, steel sheet piling.

- *Gabions*

Gabions provide a porous semi-rigid form of toe construction, although they lack durability.

- *Cribwork*

Design consideration to the use of either sloping or vertical cribwork in a toe are usually similar to their use in the body of the wall(10).

1.5 The crest of the wall

The crest provides the interface between the seawall and the land behind. As such, on flood defenses the crest has been taken to include the back face of flood embankment or dyke. In addition to forming an element in new walls, crests can be added to existing walls to improve their hydraulic performance. An analysis of failure modes by CIRIA, Report TN 125, has indicated that partial crest failure is the second most common type of seawall damage. The most common cause of breaching of flood bank is erosion of the back face as a result of overtopping.
The primary function of the crest is to prevent overtopping, that is to satisfy one or more of the following:

- Prevent the flooding of the land behind;
- Prevent scour of the back face or crest itself;
- Contain the beach and prevent it from being washed onto the land behind; and
- Prevent the build-up of water on the crest which may result in excessive water loads on the land immediately behind the crest.

Additionally the crest may well be required to:

- Resist scour if the wall does overtop;
- Provide a collecting area for spray;
- Provide access;
- Retain or collect fallen rock or soil movement behind the wall;
- Provide a termination to the body of the wall.

In most circumstances the choice of the crest should follow that of the body of the wall. Exceptions arise where the main purpose of the wall is in the crest, e.g. prevision of a road along a causeway or the base of a cliff. With its primary function likely to be the prevention of overtopping, we discuss first the hydraulic performance in Section 1.6.1 and consequently the other design considerations in Section 1.6.2. Various types of crest are described in Section 1.6.3.

1.5.1 Hydraulic performance

The toe and the body will govern the nature of wave run-up that reaches the crest, and so an evaluation of hydraulic performance should be based on considerations of the overall wall. Nonetheless, the crest itself does offer considerable potential for the improving the hydraulic performance of the wall with respect to overtopping. It is unlikely to make much difference to wave reflections.

The crest can prevent or alleviate overtopping in three ways:

- By virtue of height alone;
- By increasing the amount of dissipation;
- By deflecting the upsrushing wave back to the sea.

1.5.2 Design consideration

Crest level

The cost of reducing overtopping should be weighed against the damage that is caused by it. A more pragmatic approach is to adopt a solution resistant to overtopping erosion which is less sensitive to errors in the estimate of the extreme event. Allowable overtopping discharge should also be considered in relation to the penetration of seawater under non-porous slabs where there is a risk of uplift. Sloping wall facing on granular material are particularly vulnerable.

Structural, construction and maintenance considerations

While the design of the crest should be considered in relation to the structural design of the wall, this is not normally a major consideration. The structural design of the crest itself, however, may involve the fallowing:

- Wave loading on parapets, floodwall and other cantivelered structures;
- Bearing loads and settlement problems over backfilled areas;
- Stability of back slopes;
- Support to the coastal slopes or cliffs;
- Access and traffic loads.

1.5.3 Environmental considerations

Because the crest is at the interface between the seawall and the land, the design may involve planning and/or environmental considerations which either override or modify purely technical considerations of hydraulic performance and structural design. These may include provisions for:

- Safety of those on or near the wall;
- Pedestrian or vehicular access along the wall or across to the beach;
- Boat launching or other special facilities;
- Floodgates or scuppers in connection with the above;
- Special finishes for appearance;
- Limitation of crest level to avoid interference with views from properties behind.

Consideration should be given to the appearance, particularly the detailing of joints in areas where the public may be expected to walk. Areas with poor drainage falls or discharges from weepholes should be avoided, as slippery surfaces may develop and become a risk to the public.

1.5.4 Drainage

Surface water drainage will need to be considered both in terms of rainwater and the potentially larger flows involved in spray and overtopping.

1.5.5 Types of crest

Taking account of the general discussion of the functions of the crest, the crest of seawall should consist of one or more of fallowing:

- Walls (to increase the effectiveness of the seawall in preventing overtopping);
- Slope protection;
- Decking and surfacing.

- *Wave return walls*

The purpose of these is to divert the uprushing wave seawards and so reduce overtopping. They are most effective when used at the top of a smooth slope, so that run-up of water is guided smoothly around the face of the wall and thrown back. Wave return walls atop a rubble slope, which produce more turbulent run-up, are less likely to be so successful, although it must be remembered that rubble walls in themselves will produce much less run-up in the first place.

Wave return wall need to be carefully dimensioned, smooth and structurally strong, so are normally built of concrete. A crest wall is a strong visual feature and care is needed to achieve a shape and surface finish that is both initially acceptable and will weather in pleasing way. Concrete is very adaptable in that a wide variety of special surface textures or colors can be provided where required to enhance the visual impact. Access to the shore past large return walls is difficult if the hydraulic performance is to be preserved. Sometimes there is a little choice to live gaps and install floodgates at point of access. The concrete of the wall will require to be strong and durable to resist reinforcement corrosion, and adequate joints will be required if settlement is likely.

- *Parapet walls*

Where there is limited space, the crest of the sloping seawall can be raised with a vertical wall, although this is unlikely to be as effective in resisting run-up as would be an extension of the slope up to the same level. On more exposed walls, they are used to retain the top of armoured slope. In urban areas they are normally constructed in concrete which, with careful detailing, can produce an aesthetically suitable result. As for wave return walls, the concrete should be adequately designed in respect of durability and joints. In rural areas, cantilever steel sheet pile have been used to provide a modest raising in the wall which also overcomes some of the problems due to fissuring. This piling can, however, lead to problems because of:

- Pressure build-up at the pile toe during driving, which locally reduces the strength of the soil;
- Concentration of seepage around the pile toe which leads to washing out to fill.

- *Splash walls, flood walls and flood banks*

Wave return and parapet walls are intended to resist waves at the top of the slope. Alternatively, or in addition a wall can be placed further landwards. This has advantages that:

- The wall is removed from the full force of the wave attack so it need not be so strong;
- Waves are allowed to spend their energy before reaching the wall, thus making it more effective than if it were further seawards;
- It allows easy access and uninterrupted sight from the decking to the beach and sea.

However, it does mean that the seawall overall occupies more space than with a parapet at the top of the main body of the wall. Another consequence is that the decking between the splash wall and the seaward slope is not fully protected against waves and flooding.

- *Slope protection and deckings*

The degree and the nature of slope protection at the crest varies a great deal with the severity of the wave climate, the nature of the body of the wall and the additional uses to which the crest is put (e.g. a roadway). A vertical wall in a sever climate will generate large amounts of spray which will be carried onto the land in onshore winds. The land immediately adjacent will be subject to quite heavy dynamic loads from water falling. On the other hand, a flood bank should be designed so it will not breach if design wave/water level conditions are exceeded, when it will be subject to water flowing down the back face. Types of slope protection and deckings are broadly similar to those that might be used on front face, but of a lighter weight because they are less exposed. The following systems are particularly appropriate to the crest and back face of a seawall (11).

- Concrete slabs;
- Concrete desk (these are less flexible then the slab system and can crack seriously if the supporting soil settles);
- Asphaltic concrete (respect the simple asphaltic, the concrete asphaltic can resist severe avertopping);
- Hogging (when fissuring is a problem in a clay flood banks).

2 CHAPTER:

SEAWALL DESIGN

The Chapter 3 is a literature review of three important books about the marine construction and about the geotechnical, that are the “Seawall Design” of Thomas R.S.; the “Principles of foundation Engineering” of Braja Das and the “Elementi di Geotecnica” of Pietro Colombo.
This chapter provides information only background.

The essence of the design process is to consider possible modes of failure. Where the structural performance of individual elements is being considered, limit state design should be used. In this respect the designer should take into account:

- The likelihood of failure, bearing in mind the frequency of occurrence of the load case;
- The consequences of failure;
- The reliability of the assumed load case,
- The reliability of the assumed structure strength.

2.1 Hydraulic performance

When waves meet a seawall their energy is spent by two main processes: dissipation and reflection. Dissipation is the conversion of the wave energy into turbulence by the surface roughness of the seawall, by flow in and out of its pores and by wave breaking. The energy which is neither dissipated nor transmitted past the structure by overtopping must return to the sea by way of the reflected wave.
Hydraulic performance is concerned with the way in which a seawall accommodates the process mentioned above. The effects which are of greatest engineering importance are:

- Wave run-up;
- Wave overtopping;
- Wave reflection.

2.1.1 Run-up

The run-up and the run-down are defined as the maximum and the minimum levels reached by waves at a seawall relative to still the water. If the run-up level exceed the crest freeboard R_c, then overtopping will occur (Figure 2.1).

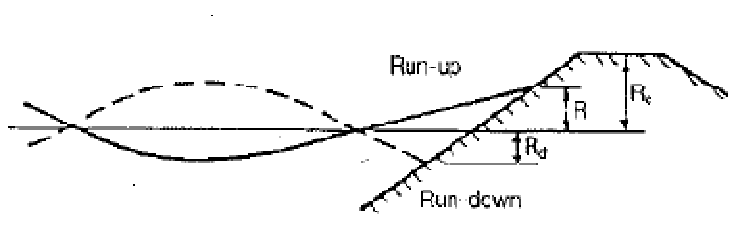

Figure 2.1: seawall hydraulics-definition sketch (12)

Under the action of random waves, run-up level will vary from wave to wave. It is therefore more meaningful to determinate a characteristic measure of run-up which is typical of the sea state R.

The prediction formulae for run-up vertical walls are based on regular waves, and generally assume deep water at the seawall toe and are based on the work of Saintflou.

The Shore Protection Manual[3] gives prediction graphs on model test for relative run-up R/H_0, where the wave height H_0 is the deep water height.

2.1.2 Wave overtopping

Most likely, the engineer will wish to design the seawall, so it limits overtopping to a specific quantity or rate, given wave and water level. Guidance on acceptable amounts of overtopping is given by Owen[4], which takes account of:

- The stability of the crown and back face of the seawall;
- The discharge capacity of drainage channels behind seawall;
- The total volume available for storage of flood waters behind the seawall until the tidal level falls sufficiently for tidal outfalls to come into operation;
- The possibility of damage or injury to buildings, vehicles or members of the public located behind the seawall.

Prediction methods for overtopping of different seawalls vary considerably. Overtopping of non-porous vertical walls has been considered by Goda[5] and in the Shore Protection Manual (SPM method). Both sources are based on regular wave tests, but postulated methods by which these results might be extended to random wave. Douglass[6] has compared the use of these two methods for various cases. The engineer should use these methods for preliminary design but will need model tests to gain more reliable values for detailed work.

No prediction method is available to describe overtopping of porous vertical walls such as cribwork or gabions.

Almost all data on wave overtopping have been obtained in laboratory studies without taking wind effects into account. However, in nature, larger waves will frequently be associated with onshore wind. These winds may influence the overtopping discharge in several ways, including:

- Raising the still water level (wind set-up);
- Increasing wave run-up on the seawall;
- Blowing spray over the seawall.
- The relative importance of these factors depend largely on the type of seawall being considered.

The Shore Protection Manual quotes a formula for wind effect, based on regular waves can be applied for seawalls with slope up to vertical. Calculating overtopping rates are multiplied by a wind correction factor.

2.1.3 Wave reflection

The interaction of incident reflected waves often leads to a confused sea in front of the structure, with occasional steep and unstable waves of considerable hazard to small boat. Reflected waves can also propagate into areas of harbor previously sheltered from wave action. These will lead to increased peak orbital velocities, increasing the likelihood of movement of beach material. Under oblique waves, reflection will increase littoral currents and hence local sediment transport. All coastal structures reflect some proportion of incident wave energy. This is often described by a reflection coefficient C_r, define in terms of the incident and reflected wave heights, or the total incident and reflect energies.

In particular, vertical non-porous walls will reflect almost 100% of the incident wave, so the C_r=1 (12).

2.2 Design of vertical walls

Vertical seawall can be designed using the standard procedures for conventional retaining walls.

2.2.1 Modes of failure

Gravity walls and semi-gravity walls can be considered together in stability terms as they share the common geotechnical modes of failure. These walls derive their stability largely from self-weight, although in the case of reinforced concrete cantilever walls the majority of the weight is actually provide by the soil which rest on the wall base. Instead, tied walls are considered separately because of their rather different modes of failure.

Figure 2.2 shows the four major types of failure which can affect gravity and RC cantilever walls. Failure of overturning occurs when overturning moment due to applied loads exceeds the restoring moment due to the weight of the wall. Sliding takes place when the frictional resistance over the base of the wall, together with the passive resistance at the toe, are insufficient to withstand the applied loads. Bearing capacity failure occurs when the contact pressure beneath the toe of the wall exceed the bearing capacity of the foundation soil. Deep-seated rotational slips can happen in poor ground and represent a slope type failure. In addition cantilever walls can suffer structural failure from overstressing of the stem.
Figure 2.3 shows the four principal ways in which a tied wall can fail. Rotation of the wall about the tie takes place when there isn’t an inadequate passive resistance at the toe. Rupture of the tie rods or overstressing of the wall material are structural modes of failure and are often precipitated by corrosion. Deep-seated rotational slips can occur as described above for gravity walls.

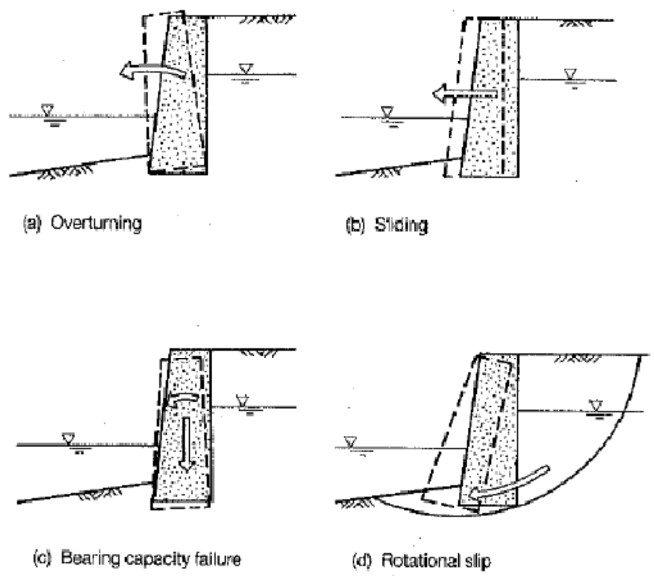

Figure 2.2: modes of failure of gravity walls (4)

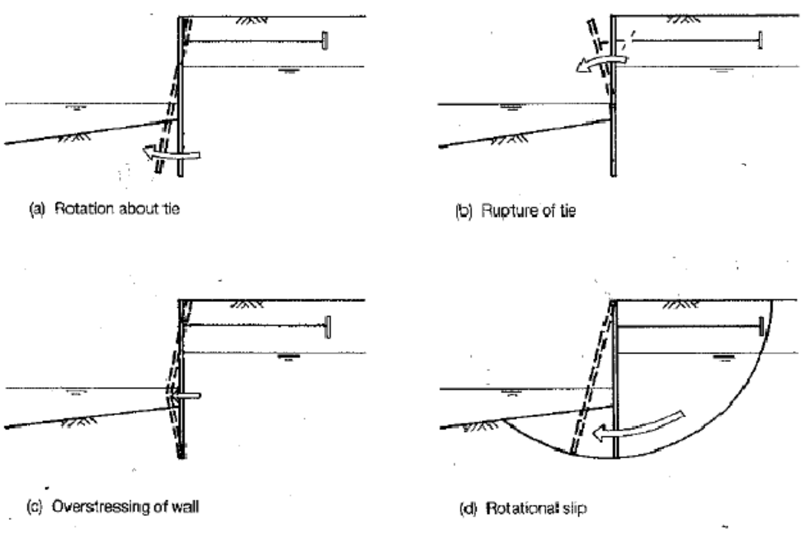

Figure 2.3: modes of failure of tied walls (4)

The soil on the seaward side of the wall in the passive region plays very important role in the stability of the wall. Loss of material from this area for whatever reason can lead in turn to one of above modes of failure.

2.2.2 Analysis

The starting point in analyzing the stability of vertical walls is the prediction of the soil and water pressures which will act upon them. When considering overall stability, active conditions can usually be assumed on the retained side of the wall.
To design retaining walls properly, an engineer must know the basic soil parameters - that is, the unit weight, angle of friction, and cohesion – for the soil retained behind the wall and the soil below the base slab. Knowing the properties of the soil behind the wall enables the engineer to determine the lateral pressure distribution that has to be designed for.
There are two phases in the design of a retaining wall. First, with the lateral earth pressure known, the structure as a whole is checked for stability, that includes checking for possible overturning, sliding and bearing capacity failure. Second, each component of the structure is checked for adequate strength, and the steel reinforcement of each component is determinated.
To calculate the active pressure (minimum value of pressure that the soil can bear before collapse) and passive pressure (maximum value of pressure that the soil can bear before collapse) which act on the gravity or semi-gravity wall, two theories exist:

- Rankine's theory; and
- Coulomb's theory.

Each theory has some basic assumptions:
For the Rankine theory, we consider a semi definite mass limited of a horizontal surface and a wall frictionless. Instead Coulomb proposed a theory to calculate the lateral pressure with granular soil backfill, so this theory takes wall friction into consideration, but in the other hand, with Coulomb' theory, is imposed a plane failure surface in the soil mass, that means assumed a possible soil failure wedge.

Although the actual failure surface in soil for the case of active pressure is somewhat different from that assumed in the calculation of the coulumbian pressure, the results are not greatly different. However, in the case of passive pressure, as the value of the angle of friction between the soil and the wall increases, Coulomb’s method of calculation gives increasingly erroneous values of passive force. This factor of error could lead to an unsafe condition because the values of passive force would became higher than the soil resistance. To remedy this situation, several investigations have been done to calculate the value of passive force using the type of failure surface shown in Figure 2.4.

Note that the failure surface BC is curved and that the failure surface CD is a plane. The investigators conduced their researches changing the BC profile, for example Packshaw (1969) used a circle assumption for the curved part of the failure surface, Caquot and Kerisel (1948) used a ellipse one, Schelds and Tolunay (1972,1973), Jams and Bransby (1971), Terzaghi end Peck (1967) used a long spiral one (13).

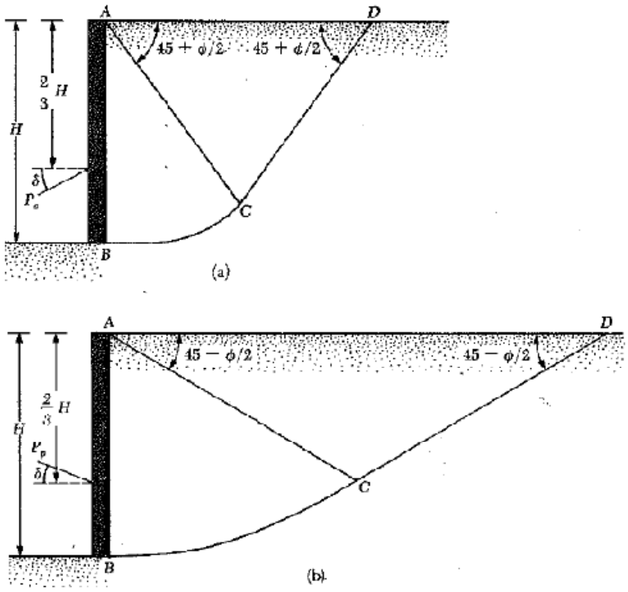

Figure 2.4: nature of failure surface in soil with wall friction for (a) active pressure and (b) passive pressure case (13)

Instead for the sheet pile walls we have three calculation methods:

- Method of ultimate equilibrium;
- Method derived from Winkler's model;
- Method of calculation through FEM.

These methods have increasing difficulties, because of the quality of the geotechnical parameters required.

The first method is used frequently to determine the depth of penetration in the ground, and it is available for the structures which have a kinematics easily identifiable such as the cantilever sheet pile or anchored walls with only one tie at the top.

For the calculation we consider the sheet pile subjected on active pressure, generated from the retained soil. The pile tends to turn on the side of excavation and comprises the soil in the opposite side. In this way, in the area on the top of rotation centre we have an passive pressure, instead at bottom of rotation centre there are passive pressure on the soil side, and active pressure on the sea side (14) (Figure 2.5).

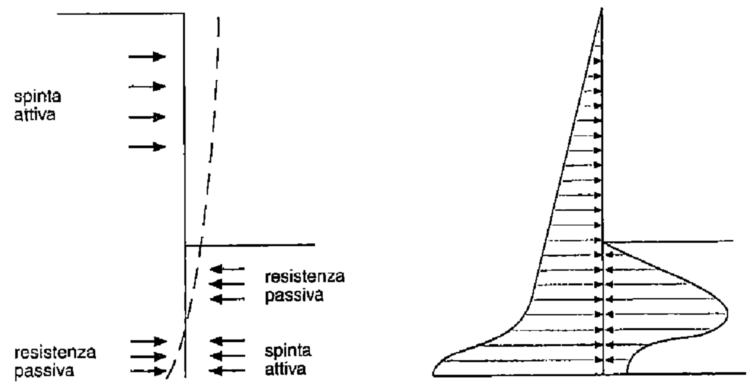

Figure 2.5: distribution of passive and active pressure for the Method of Ultimate Equilibrium (9)

When the height of the backfill material behind a cantilever sheet pile wall exceeds about 20 ft ($\approx 6m$), tying the sheet pile wall near the top to anchor plates, anchor walls, or anchor pile becomes more economical. Anchors minimize the depth of required penetration by sheet piles and also reduce the cross section area and the weight of the sheet piles needed for construction. However, the tie rods and anchors must be carefully designed.

The two basic method of constructing anchored sheet pile walls are:

- The free earth support method (Figure 2.6);
- The fixed earth support method (Figure 2.7).

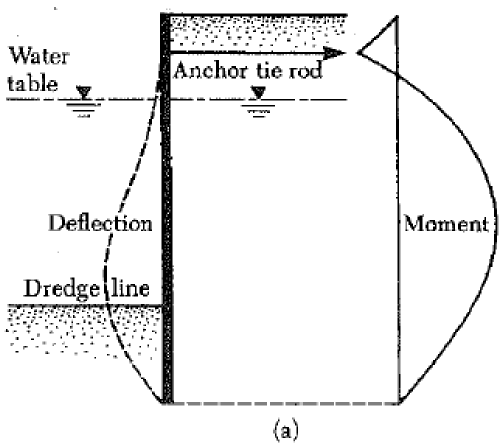

Figure 2.6: nature of variation of deflection and moment for anchored sheet piles: (a) free earth support method: (15)

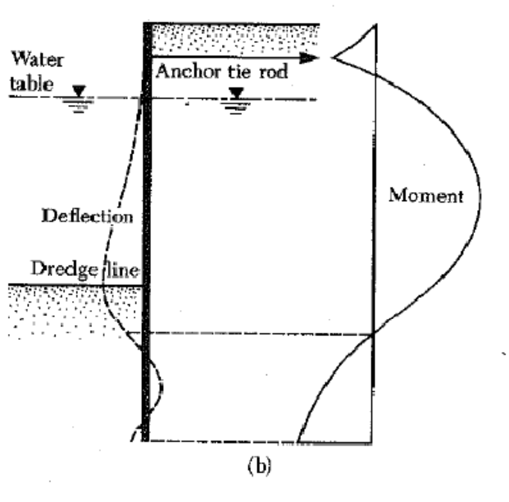

Figure 2.7: (b) fixed earth support method (15)

The water pressures acting on the wall are often higher than the soil pressures, and care should be taken to choose the appropriate values. Groundwater levels will vary seasonally and fluctuate with the tide. On a falling tide there will be a lag between the groundwater level behind the wall and the sea level in front of the wall. The height of the tidal lag will depend on the nature of the retained soil, the porosity of the wall, and the effectiveness of the drainage system behind the wall.

Overtopping can result in a build-up of water pressure behind the wall and this can be a major consideration in design. The extent to which it affects wall stability depends on the amount of overtopping, the permeability of the decking, and the rate at which the excess water can be discharged.

The stability of gravity walls should be checked under the most adverse combinations of tidal lag, groundwater seepage and uplift. These conditions can result in a reduced resistance of sliding and overturning, or a reduction in passive resistance. Stability against overturning is assessed by checking that the resultant force acting on the base of the wall lies within the middle third. This requirement theoretically prevents any tension occurring beneath the base. If the wall is founded on permeable material, the middle-third rule should be satisfied by the resultant force in effective stress terms, using the worst assumptions regarding water pressure distribution.

The stability of tied walls should be checked under the most adverse groundwater and seepage conditions. Where the wall penetrates cohesive soil, short-term stability should be assessed in total stress terms using undrained shear strength parameters. Long-term stability should be assessed in effective stress terms, particularly when lower beach levels are being allowed for in the future (15).

2.2.3 Foundation

Gravity walls should generally be founded on a suitable stratum of soil or rock. Where granular strata occur beneath the wall, an adequate groundwater cut-off should be provided to prevent seepage leading to piping or bearing capacity failure. Where piles are required to facilitate construction or limit differential settlements, they should be taken down into a component bearing stratum. The penetration of sheet pile is normally determinate by stability requirements and need to provide adequate passive resistance. Where they need to carry significant vertical loads, they should be treated as bearing piles and taken down into a competent bearing stratum. In all cases a knowledge of the soil profile is required to ensure that the bearing stratum is of adequate thickness and is not underlain by compressible material. The construction of a vertical walls results in a considerable net loading of the ground. Estimates of consolidation settlement should be made so that a settlement allowance can be built into the design crest level of the wall.

2.3 Design of crest

In terms of design, the crest can be considered as one of three broad types. The first, wave walls and wave return walls, placed at the immediate crest of the wall will be required to resist wave loads. The second, splash walls, are set back and are required to withstand a short-term ‘hydrostatic’ head to prevent water flowing onto the land behind and other live loads such as vehicle impact. The third type, deckings and slabs, are basically facings to the fill material beneath. They should be designed to resist the erosive forces imparted by wave run-up and overtopping, as well as other live loads due to vehicles and people. The resistance to overtopping is very important in consideration of the back face.

2.4 Design of the toe

2.4.1 Modes of failure

In structural terms the main purpose of the toe is to prevent undermining of the body of the wall. Toe stability is essential since failure of the toe will generally lead to overall failure of the wall. Under the most adverse conditions it should prevent soil from being removed from the base of the body of the wall. The two most significant modes of failure are:

- Geotechnical instability when the beach is drawn down;
- Lack of resistance to wave action in severe weather when the beach is drawn down.

We already spoken (Section 1.5.3 and Section 1.5.4) about the different types of toe, but for our kind of project, of vertical seawall, the toe is a component included in the body of the wall.

2.5 Design for the construction and maintenance

The engineer should ensure that the construction will be stable during the proposed construction sequence, taken account of stresses likely to be induced in the structure during the construction, and consider the effect that construction methods may have on a long-term durability and hence on future maintenance requirements. It is important to recognize ease and economy of construction and maintenance as major functional requirements. The design should be based on concepts which are realistically achievable in the particular location and a particular time. The level and the type of maintenance should be thoroughly assessed during design so that they can be taken into account in selecting the most appropriate design (16).

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3 CHAPTER:

STATE OF THE ART OF THE SEAWALL CONSTRUCTRION IN SOUTH FLORIDA

In this Section is shown the common types of seawall used in South Florida.the information came from field studies on West Palm Beach, Miami (FL), Manthesan Hammark Park, Miami (FL) and William Island, Aventura (FL).

3.1 Seawall systems used in South-Florida

Through field studies, it has been possible to categorize three principle systems of seawall construction:

- vinyl/FRP or steel sheet pile (anchored and not);
- anchored panel wall and vertical pile;
- panel wall, king pile and battering pile.

The steel sheet pile is the strongest system, with its possibility to reach greater depths, but the cost of this material is very high. Also, this system offers a service life of 25 years at most due to corrosion. This adds additional costs for maintenance.

The system formed by reinforced concrete panels and piles is more common in South Florida because concrete is the locally available aggregate and it also it offers a service life of 50 years. Thus it represents the most economical choice. On the other hand, the disadvantage of the RC system is the difficulty to penetrate hard layers, and sometimes, when the soil is not sand, an excavation is required to place the pile at the desired depth.

3.1.1 Vinyl/FRP or steel sheet pile

Sheet pile walls are divided into two basic categories: cantilever and anchored. In the construction of sheet pile walls, sheet pile may be driven into the ground and then the backfill placed on the land side, or the sheet pile may be driven into the ground after the soil around of the sheet pile is dredged. In any case, the soil used for the backfill behind the sheet pile wall is usually granular. The soil below the dredge line may be sandy or clayey soil. The surface of soil on the water side is referred to as the mud line or dredged line.

Thus construction methods generally can be divided into two categories (Tsinker, 1983):

- Backfilled structure
- Dredged structure

The sequence of construction for a backfilled structure is as follows (Figure 3.1):

Step1. Dredge the in situ soil in front and in back of the proposed structure.

Step2. Drive the sheet piles.

Step3. Backfill up to the level of the anchor and place the anchor system.

Step4. Backfill up to the top of the wall.

For a cantilever type of wall, only steps 1,2 and 4 apply.

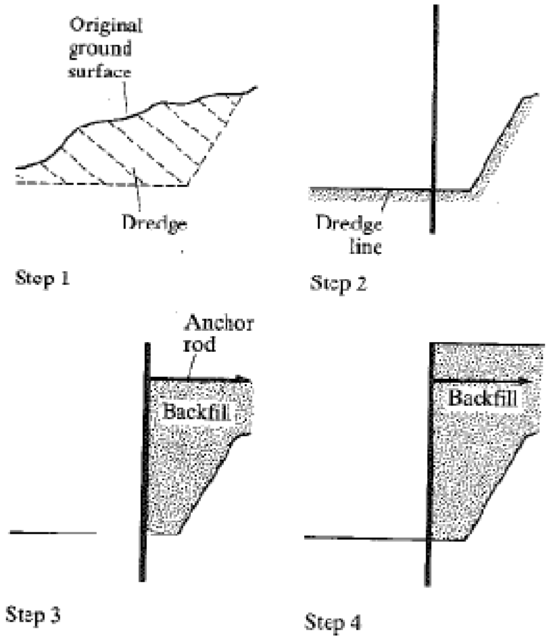


Figure 3.1: sequence of construction for Backfilled structure (17)

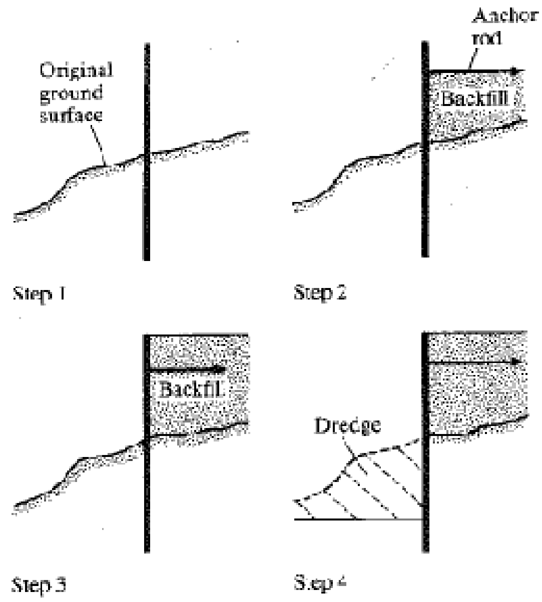


Figure 3.2: sequence of construction for dredged structure (17)

The sequence of the construction for a dredged structure is as follows (
Figure 3.2):

Step1. Drive the sheet piles.

Step2. Backfill up to the anchor level and plane the anchor system.

Step3. Backfill up to the top of the wall.

Step4. Dredge the front side of the wall.

For a cantilever sheet pile walls, step 2 is not required (17).

Here are pictures from a construction site in West Palm Beach showing some practical aspects of the construction of the seawall through the method of sheet piles.

In the Figure 3.3, it is possible to see the ease of the installation of the sheet piles. In this construction site, the soil is sand, so the placement of the sheet pile is made by hand. The excavation equipment simply holds the pile in the right position.

Instead the Figure 3.4 shows a different case where it is necessary to use a vibration sheet pile driver that can be vibratory hammer or vibratory plate compactors.

It possible, also, to note in the Figure 3.3 and Figure 3.4 the presence of a temporary driving guide that is recommended for building a straight wall. By assembling a driving guide before installation, accurate wall position and a solid driving surface is established keeping the sheet piling plumb.

Figure 3.3: installation of sheet piles by hand in sand soil

Figure 3.4: installation of sheet pile by vibration machine driver

When all the sheet piles are positioned, the next step consists of the installation of the anchor.

The most popular anchoring method involves driving or burying large bodies in the ground with which to secure the wall. This method involves excavation which may be not possible or feasible in every situation.

In general, the anchor consists in two parties: the tie rods and the ‘dead man’, that is the large body in the ground.

The rod selection is an important decision. A large portion of the load acting on a tie-back wall is carried by the tie rods to the anchors. Rods specs as well as spacing need to be given careful design consideration. Many walls fail because the tie rods or anchor fail. The tied rods are made in steel or aluminum and need protection against corrosive environments, for this reason, generally, the first part is covered by polymer coating.

The range of the dimension varies for the diameter from 1.5” to 2” and for the length from 15 ft to 32 ft.

For a tie rod of 1.5” dimension of diameter the strength loaded is of 12.000 lb (54 N), while for a tie rod of 2”dimension of diameter the strength loaded is of 48.700 lb (216.7 N).

The tie rod can be oblique or vertical, and can be placed on the sheet pile or on the cap. These choices depend from the necessities of the design.

In the Figure 3.5 is showed the common type of sheet pile system with anchor.

Figure 3.5: anchored sheet pile system

The Figure 3.6 represents a particular about the connection between sheet pile and tie back.

Figure 3.6: particular about the connection between sheet pile and tie back

After the installation of the anchor there is the coverage with soil of the tied rods and the ‘dead men’ to protect the anchor system against corrosive environment (Figure 3.7).

Figure 3.7: anchored sheet pile system

The type of backfill to use is an important decision that is often overlooked. The choice in backfill influences the wall structure more than any other single factor. Hydrostatic pressure behind a wall is the primary cause of bulkhead failure. It is suggested to use granular, free draining backfill, and if the site condition permits, the use of a drainage or weep holes is also recommended.

It is a good idea to backfill in lifts or layers of 1-2 ft, compacting the soil at each layer. It is very important to ensure that there are no voids in the backfill and that it has a good contact with the entire surface of the wall. It is particularly important in the corners, where it may take some extra effort to ensure good backfilling and compaction.

In the Figure 3.7 is also showed the construction of the cap beam.

In the case represented in the below picture the cap beam is made by concrete and it requires the construction of formwork as showed in the Figure 3.8.

The cap beam can be realized also in aluminum with a generic C shape (Figure 3.9).

Figure 3.8: formwork for concrete cap beam

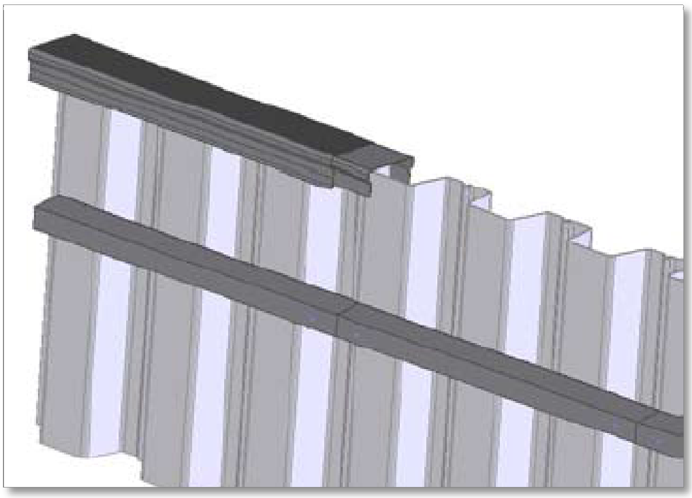


Figure 3.9: cap beam in aluminum

Finally, after the retaining structure is complete, it is a good practice to periodically check back to make sure that everything is aging as expected and no maintenance is required. Though the metals used are designed for marine environments, all materials corrode eventually. It should make sure that no premature corrosion is taking place, and if it is found, the corroded material should be removed and sealed to prevent damage to the integrity of the structure.

The Figure 3.10 shows the work at the construction site completed (18).

Figure 3.10: the completed work

3.1.2 Anchored Panel wall and Vertical pile

The anchored panel wall + vertical pile system is a reinforced concrete system. The stages of the construction are:

1. Driving the vertical pile (Figure 3.12)
2. Set the panel
3. Placing the anchor
4. Casting of the cap beam.

In general the concrete piles may be divided into two basic categories: precast piles and cast in situ piles. For the seawall construction the concrete piles used are precast and can be prepared by using ordinary reinforcement.

Reinforcement is provided to enable the pile to resist the bending moment developed during pickup and transportation, the vertical load, and the bending moment caused by lateral load.

Precast piles can also be prestressed by the use of high-stress steel prestressing cables. The ultimate strength of these steel cables is about 260 ksi (≈1800 MN/m^2). During casting of the piles, the cables are pretensioned to about 130-190 ksi (≈900-1300 MN/m^2), and concrete is poured around them. After curing, the cables are cut, thus producing a compressive force on the pile section.

The length of the concrete pile depends on previous geotechnical study because they have to reach the bed-rock layer. Consequently, the mechanism of load transfer to the soil adopted by piles used in the seawall construction is the point bearing piles.

The typical range of depth varies from 30-50 ft (10-15m) for the precast concrete piles, and from 30-150 ft (10-35m) for the precast prestressed concrete piles.

The usual shape of the cross section the pile is square and its dimension varies typically from 12” to 14” (Figure 3.11).

The following pictures are taken from a construction site in West-Palm Beach.

Figure 3.11: precast concrete pile used in the West-Palm Beach’ site construction

If the soil is sand, the pile is driven through jetting into sand or using a hammer, but if the site is characterized by the presence of hard layers is necessary to practice a previous excavation.

Figure 3.12: the first step for the construction of a seawall trough the Vertical Pile System

The second step of the construction is the placing of the panel behind the vertical pile.

The panel has like typical dimensions:

1. Wide: 6-10 ft
2. Deep: 6-12 ft
3. Thickness: 8 in

The panel are cast horizontally (typically on the job site) with conventional reinforcement (Figure 3.13).

Figure 3.13: casting of the panel

The Figure 3.14 shows the type of formwork used to create panels with exposed rebar on the side where will be cast the cap beam.

Figure 3.14: formworks used for the casting of the Panel

After as little as one day, a polyvinyl sheet and side forms are placed on the prior-cast panel surface and a new panel is cast (up to 4 panels per stack).
Panels sit for curing awaiting their place in the seawall (Figure 3.15).

Figure 3.15: completed panels

As is shown in the Figure 3.15, the panels are cast with a C profile. In this way it is possible to create a good connection between the panels, behind the vertical pile.

This connection consists of a hole that afterwards will be filled by concrete. Before coverage with backfill, a geotextile sheet is placed on the connection, (Figure 3.16).

Figure 3.16: particular about the connection between the panels

After placing the panel behind the vertical pile, the next step is the collocation of the anchors. This process is already showed in the Section 3.1.1 for the sheet pile system.

The last step of the construction is the casting of the cap beam (Section 3.1.1).

The completed panel wall and vertical pile system is shown in the Figure 3.17.

Figure 3.17: completed seawall through the method of panel wall and vertical pile.

3.1.3 Panel wall, King pile and Battering pile system

The Panel wall, king pile and battering pile system is a reinforced concrete system.

The stages of the construction are:

1. Driving the king pile in place using an alignment template. Use jetting, hammering, or some combination to fix the king pile in the soil.

2. Panels are placed behind the king piles.

3. Batter piles are placed in the manner similar to that used for the king piles.

4. Casting of the cap beam.

The ability to install driven piles on an angle, or batter, gives them a distinct advantage with respect to their ability to carry lateral loads. Batter piles carry lateral loads primarily in axial compression and/or tension while vertical deep foundations carry lateral loads in shear and bending. When subjected to lateral loading, batter piles will therefore generally have a greater capacity and be subject to smaller deformations than vertical piles of the same dimensions and material. Through the installation of the batter piles, the anchors become useless.

In the Figure 3.18 it is possible to see the phase of jetting the batter pile into the sand.

Figure 3.18: jetting of the batter pile in the sand

In the Figure 3.19 is shown the entire system complete of king piles, batter piles and panel wall. The only missing piece is the beam cap.

Figure 3.19: King piles, Batter piles and Panel wall

3.1.4 Modes of failure

There are six principle modes of failure, reported to the typical system of seawall used in Florida:

1. Failure due to toe scour
2. Rotational slip surface failure
3. Failure of wall construction material
4. Failure due to the anchor pullout
5. Failure due to the absence of anchor's contribution, caused by cutting during following construction works
6. Failure due to degradation by corrosion of parts of the seawall

All of these failure modes occur through a rotation of the seawall structure that generates important cracks (and consequently missing pieces) on the head of the pile and on the panel at the bottom of the cup beam (the vertical cracks on the panel are in the most of the cases the normal shrinkage cracks) or misalignment of the seawall. The presence of the cracks is the first advice of a structural problem that requires maintenance.

1- Failure due to toe scour (Figure 3.20)

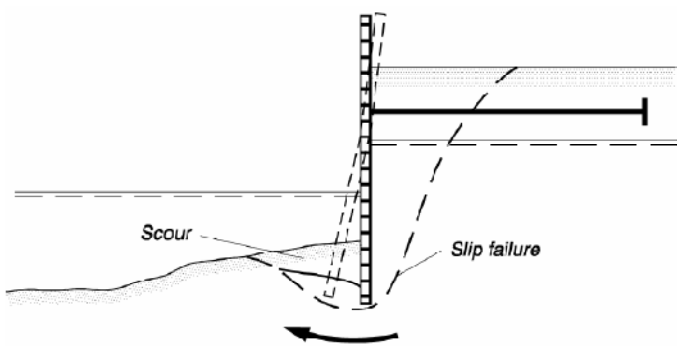


Figure 3.20: failure due to the scour

Toe scour reduces or eliminates the passive pressure from the soil.

Subsequent rotation of the wall happened when the loads from the active soil pressure and pressure from the groundwater exceed the passive pressure.

2- Rotation slip surface failure (Figure 3.21)

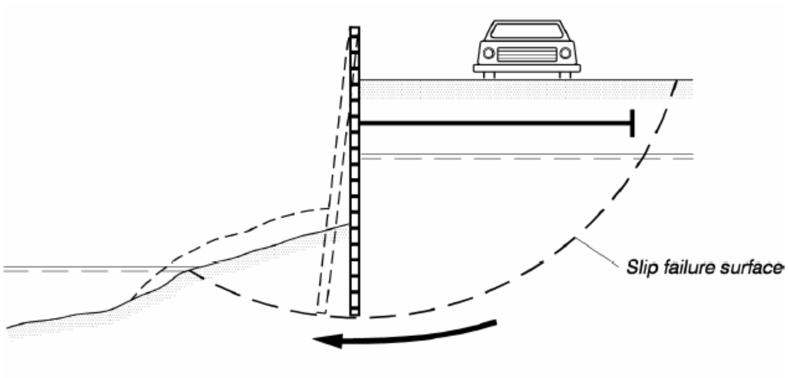


Figure 3.21: Rotation slip surface failure

Rotational slip failure occurs when the driving moments from the weight of the soil and the surface loads exceed the restoring moment given by the soil strength.

3- Failure of wall construction material (Figure 3.22)

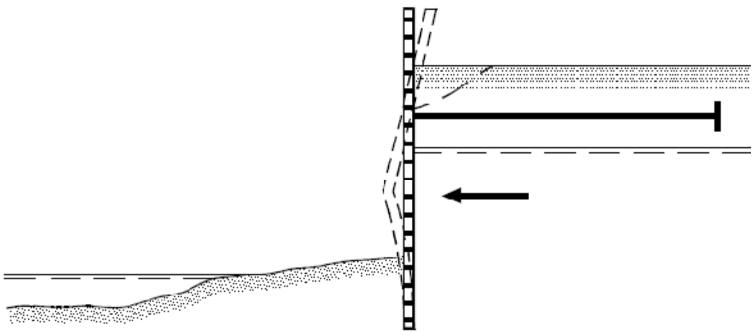


Figure 3.22: Failure of wall construction material

Yielding in the sheet wall due to stress exceeding the strength.

4- *Failure due to the anchor pullout (Figure 3.23)*

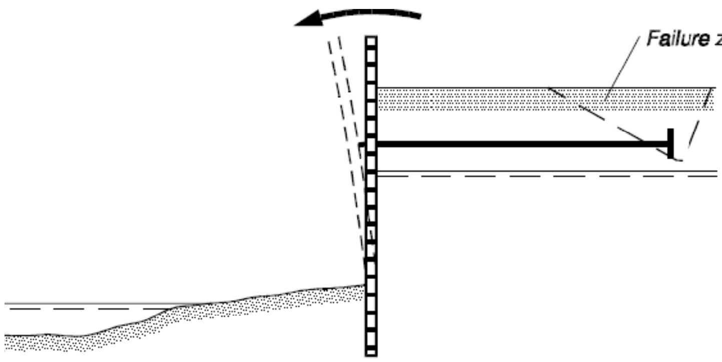


Figure 3.23: Failure due to the anchor pullout

Excess loads from active soil pressure and high groundwater table or too small wall anchor slabs might lead to anchor pullout and subsequent collapse of the sheet pile.

The same mode of failure happens for the failure due to the absence of anchor's contribution caused by cutting during following works.

6- *Failure due to degradation by corrosion of parts of the seawall*

There were just described the principal mechanisms of failure, but another important cause of collapse is the corrosion of the steel reinforcement or of the steel sheet pile or the tie-back.

Displacement cracking and spalling can occur through local overstressing due to poor detailing, but the real problem is that after the cracking it triggers a corrosion process that is deleterious for the life of construction (20).

So, the types of failure described above require restoration as introduction of a new toe deeper (*for failure 1 and 2*) or the introduction of batter pile or sheet pile behind the deck (*for failure 3,4 and 5*) as shown in the Figure 3.24and in the Figure 3.25, but if the collapse is caused by corrosion (*failure 6*) the solution to have a durable construction is to find a way to prevent the corrosion, for this reason Strongwell corporation together the team of research of Dr. Nanni are trying to create a new system of RC FRP (Section 3.2).

Figure 3.24: restoration trough batter piles (Old Cutler by Mathesan Hammark Park, Miami (FL))

Figure 3.25: restoration trough steel sheet pile (Williams Island, Aventura (FL))

3.2 New Seawall system

The idea to realize a new seawall system reinforced with GFRP grid-form, born like review of a project about an innovation system of FRP internal reinforcement for the concrete bridge deck that was built in Greene Country, Missouri, completed in November 2005 (Figure 3.27, Figure 3.28 and Figure 3.28: FRP RC Bridge in Greene County completed) (21).

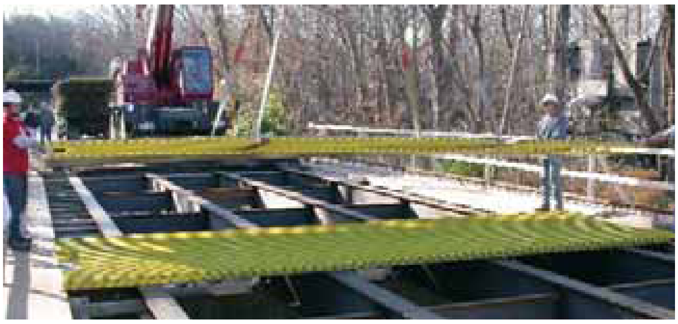


Figure 3.26: positioning of the Strongwell Gridform Slab

Figure 3.27: deck casting of the new Bridge in Greene County, Missuri

Figure 3.28: FRP RC Bridge in Greene County completed

The design of the prototype of the Panel Wall is shown in the Section 3.2.1.
It consists in two-layer reinforcement, prefabricated by assembling off-the-shelf pultruded glass/vinyl ester profiles, which are normally used in grating application for corrosive environmental, and the GFRP internal reinforcement is represented by the $\mathrm{I}-bars$, which are the main load-carrying elements and of the cross-rods, which provide shrinkage and temperature reinforcement and constrain the core concrete to ensure load transfer into the $\mathrm{I}-bar$.

The idea for the new system of seawall consists in the realization of a structure completely in reinforced concrete with GFRP, in this way it will be possible understand if the RC Seawall with GFRP grid-form and stay in place gridform is more resistant against the external environment than the traditional RC Seawall with steel internal reinforcement.
As largely stated in Chapter 1 and in the Sections 3.1.2 and 3.1.3, the concrete seawall structure is made by three principal parts: the Panel wall (anchored or not), the pile (vertical end/or batter) and the cap beam. The project involves the construction of all these parts in reinforced concrete with GFRP.
The research has started with the realization of the panel wall prototype (Section 3.2.1) and following modified by practices that occurred at the time of casting and information obtained from contractors (Rafael Marin and Kevin McCabe for the job side in Williams Island, Aventura (FL)) and site visits (West-Palm Beach's job site).

The next steps in the evolution of the project will regard the construction of the tie-rods in FRP and its anchor in reinforced concrete with FRP, and the construction of the pile, working with Raymond's technology but with a tube shaped in FRP.

3.2.1 Prototype of Seawall Panel Wall

At the beginning of the research the design of the seawall provided the use of two prefabricated panels that integrate a pultruded grating with stay-in-place (SIP) forms, as shown in the Figure 3.29.

The two-layer reinforcement is prefabricated by assembling off-the-shelf pultruded glass/vinyl ester profiles, which are normally used in grating application for corrosive environmental.

The components are:

- $\mathrm{I}-bar$ running continuously in the direction perpendicular to ground;
- Cross rods running through pre-drilled holes spaced at 100 mm on-center in the $\mathrm{I}-bar$ web in the direction perpendicular to the principal reinforcement;
- Vertical connectors that space the layers 100 mm apart;
- Channel as outer edge of the seawall.

$\mathrm{I}-bars$ are the main load-carrying elements and their shape and spacing are design to realize the economic solution that satisfies, at the same time, the external conditions of load (Section 4.1).

The cross-rods provide shrinkage and temperature reinforcement and constrain the core concrete to ensure load transfer into the $\mathrm{I}-bar$ (Figure 3.32, Figure 3.32 and Figure 3.32).

Figure 3.29: the first model of Strongwell’s panel

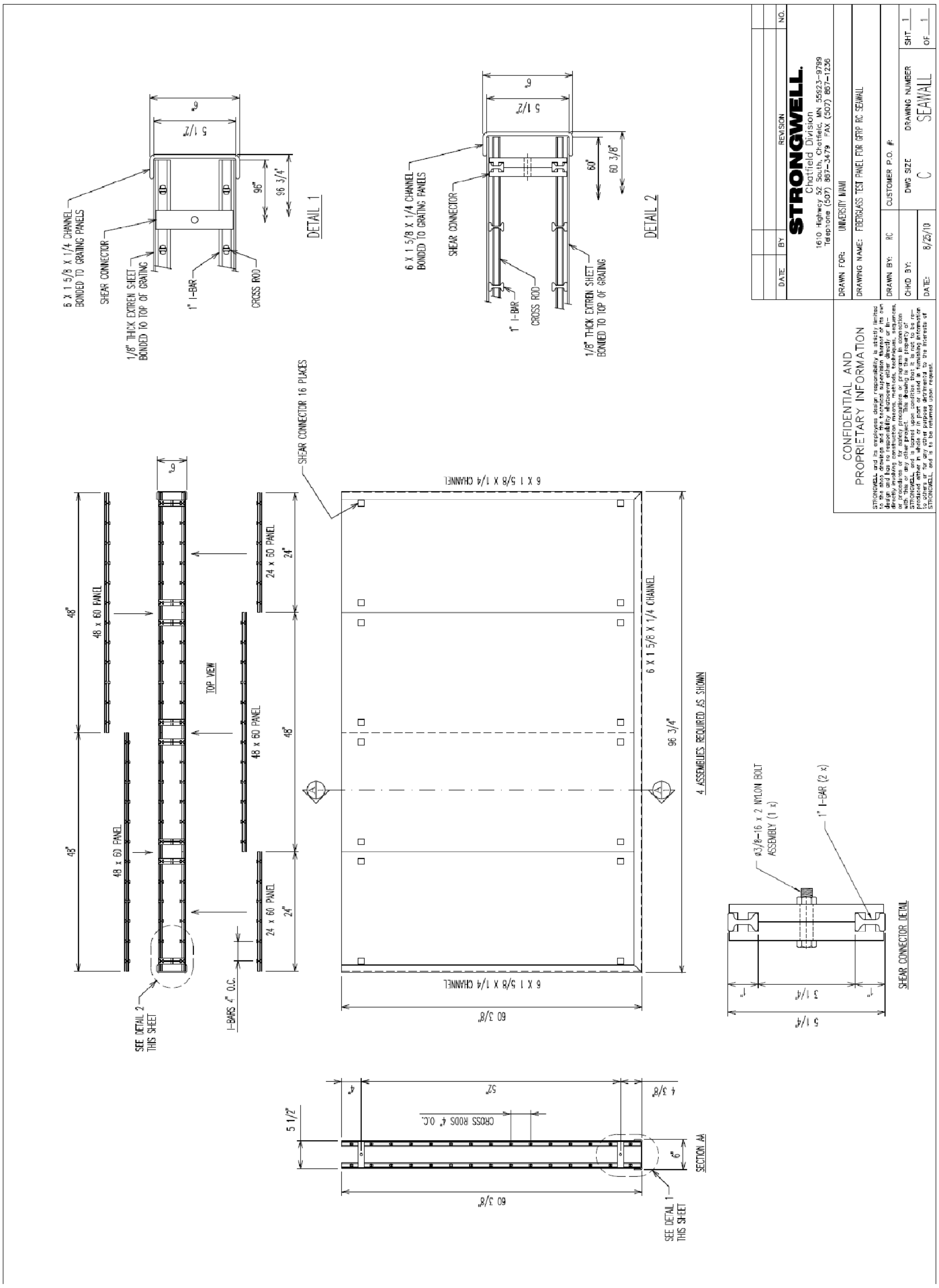


Figure 3.30: autocad drawing of the Panel realized

Figure 3.31: details about the C-channel and the shear connectors

Figure 3.32: detail about internal FRP reinforcement of the panel

After the casting of this first system of seawall, issues emerged (Section 6.2) which led the team of research to change the design of wall.

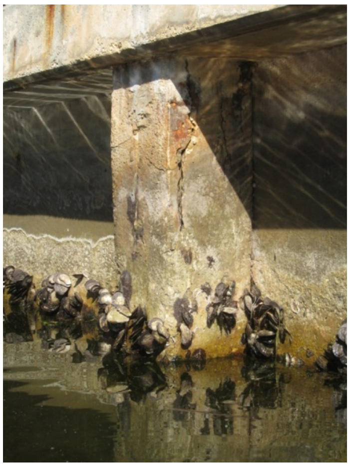
To create the new design of the panel wall have been done field investigations and these brought two radical modifications:

- Horizontal casting;
- The FRP Panel Wall can be constructed without SIP formwork
- Introduction of FRP eye bolts that will be embedded in the concrete, these will be used to hoist the panel walls with a crane.

3.2.2 Future Steps

The future steps, for the project to realize a seawall structure completely in GFRP, consist in the possibility to create an anchor system and reinforced concrete pile plus cap beam with GFRP.

From the feasibility and marketability studies that we conducted, the main problem for existing structures emerged that require rehabilitation is the damage to the head of foundation piles (Figure 3.34, Figure 3.34 and Figure 3.35)



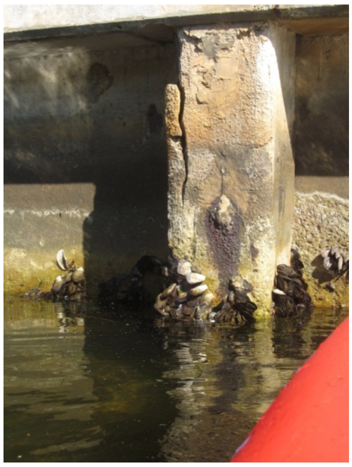


Figure 3.33: example of damage Old Cutler by Mathesan Hammark Park, Miami (FL)

Figure 3.34: example of damage Williams Island, Aventura (FL)

Figure 3.35: example of damage Williams Island, Aventura (FL)

Hairline cracks have been induced during their installation with a pile driver. Saltwater seeped into the cracks and corroded the steel reinforcing bars within the piles. To make the process of degradation faster, there is the fact that the top of the pile is a joint where stresses on the seawall are focused.

For these reasons the type of pile that will be used in the project is still concrete and cast-in-situ pile, but will be adopted as construction method the Raymond’s technology, in which, instead the use of steel casing will be used GFRP one.

The Raymond’s technology consists to driving a casing into the ground with the help of a mandrel placed inside the casing. In this way we can avoid creating cracks on the top of the pile and at the same time with the FRP casing left in the soil we create an external reinforcement that is resistant for the corrosive environmental.

Another advantage, that comes from the use of FRP casing, is the untying from the decision in advance of the start of the work about the length of the pile, because the cutting of FRP is very easy and without complications.

The FRP casing has a square cross section varies from 9 inches to 12 inches and thickness of ½ inch.

In addition, to give grater rigidity to the pile, the casing has also an internal longitudinal member.

When the pile reaches the proper depth, the mandrel is withdrawn and the casing is filled with concrete. On the top of the pile is inserted a armor of reinforcement that acts, also, like a rebar system for the cup beam (crest).

Has already been found the contractor to install test pile on the job side.

4 CHAPTER:

SEAWALL DESIGN PROGRAM

In this section a manual for the design of reinforced concrete seawalls with double layer 3D pultruded gridforms is presented.
The manual comes with a software.
The software consist of the following work sheets:

- Input;
- Design parameters;
- Calculation; and
- Result.

4.1 Input sheet

The input sheet is the user's interface for the design program. Options, input parameters and reduction factor that are required to be placed in the program, for the design analysis, are located on this sheet.

4.1.1 Properties

In the Section 4.1.1.1 the properties of the glass-fiber reinforced composite used as internal reinforcement of the Strongwell's panel wall are listed; while in Section 4.1.1.2 the characteristics of the concrete used for the casting of the prototype are discussed.

4.1.1.1 *FRP material properties*

Table 4.1 presents the suggested guaranteed design values (SGDV) from the samples tested by manufacturer (Conachen, 2005).

		Tensile strength(ksi)	Tensile modulus(ksi)	Ultimate tensile strain(ksi)
Main bars	Average	89.1	5290	0.0017
	St. deviation	9.5	238	0.002
	SGDV	60.6	5290	0.012
Cross rods	Average	160	6928	n/a
	St. deviation	2.5	88	n/a
	SGDV	153	6930	n/a

Table 4.1: FRP material properties

$f_{fu\ bar}$ = $main\ reinforcement\ ultimate\ tensile\ strength$

$f_{fu\ cr}$ = $cross\ rod\ ultimate\ tensile\ strength$

$\varepsilon_{fu\ bar}$ = $main\ reinforcement\ ultimate\ tensile\ strain$

$E_{f\ bar}$ = $modulus\ of\ elasticity\ for\ main\ reinforcement$

$E_{f\ cr}$ = $modulus\ of\ elasticity\ for\ cross\ rod$

Design Example: FRP Material Properties Selection Experimentally determined by Strongwell values are used.

$f_{fu\ bar}$ = 60.6 ksi $f_{fu\ cr}$ = 153 ksi $\varepsilon_{fu\ bar}$ = 0.01 $E_{f\ bar}$ = 5290 ksi $E_{f\ cr}$ = 6930 ksi

4.1.1.2 *Concrete material properties*

Fallowing the ACI 318 (23),

f_c' = $concrete\ compressive\ strength$

E_c = $modulus\ of\ elasticity\ for\ concrete$ = $57000\sqrt{f_c'}$

ε_{cu} = $ultimate\ concrete\ strain$

Design Example: Concrete Material Properties Selection f_c' = 5000 psi E_c = 4030 ksi ε_{cu} = 0.003

4.1.2 Environmental reduction factor

ACI 440 Section 7 as per,

“The material properties provided by the manufacturer, such as the guaranteed tensile strength, should be considered as initial properties that do not include the effects of long-term exposure to the environment. Because long-term exposure to various types of environments can reduce the tensile strength hand creep rupture and fatigue endurance of FRP bars, the material properties used in design equations should be reduced based on the type and level of environmental exposure.”

From Table 7.1 (ACI 440) for concrete exposed to earth and weather and reinforced with glass fiber the Environmental Reduction Factor $C_E = 0.7$

4.1.3 Geometry

The geometry parameters are divided into ‘ fixed’ and ‘variable’.

- The fixed parameters are:

    1. $D = panel\ thickness$
    2. $B = panel\ base$
    3. $T_W = tickness\ of\ main\ reinforcement = 0.16\ in.$
    4. $D_{cr} = cross\ rod\ diameter = 0.50 in.$

- The variable parameters:

    1. $A_{f\,bar} = cross\ area\ I - bar\ reinforcement$

    The options to choice for the $A_{f\,bar}$ are three: 0.3221 in^2, 0.3560 in^2 and 0.5536 in^2.

    2. $s = spacing\ between\ I - bar\ reinforcement$

    The options to choice for the s are three: 3 in, 3.5in and 4 in.

3. *d = distance from extreme compression fiber to the centroid of tension reinforcement (Eq.*4.1*)*

$$d = D - \frac{h_I}{2} \tag{4.1}$$

4. $h_I = I - bar\ height$

5. $A_n = net\ area\ of\ main\ reinforcement\ (Eq. 4.2)$

$$A_n = A_{f\,bar} - A_p \tag{4.2}$$

6. $A_p = projected\ area\ of\ cross\ rod\ in\ web\ (Eq. 4.3)$

$$A_p = T_W \cdot D_{cr} \tag{4.3}$$

Design Example: Geometry

For the example is chosen the cheaper solution: the minimum area and the maximum spacing.

$D = 5.5\ in$

$B = 12\ in$

$A_f = minimum\ area = 0.3221\ in^2$

$A_n = net\ area = 0.3221 - (0.16 \cdot 0.5) = 0.3221 - 0.8 = 0.2421\ in^2$

$T_W = 0.16\ in$

$D_{cr} = 0.50\ in$

$s \;=\; maximum\,spacing \;=\; 4\,in$
$h_l \;=\; 1\,in$
$d \;=\; 5\,in$

4.1.4 Loads

Loads have to be given as input data.

This software does not include the structuralanalysis of the seawall.

The software requires for the design analysis:

$Mu \;=\; the\,factored\,moment\,at\,section\,(k-ft)$

$Vu \;=\; factored\,shear\,force\,at\,section\,(kip)$

Design Example: Loads
$Mu \;=\; 3\,k-ft$
$Vu \;=\; 0.702\,kips$

4.1.5 Result

Provided to the software the external stresses, the seawall's design is defined by the following checks:

Check 1: Flexure – ACI 440.1R 6 Section 8

Check 2: Shear – ACI 440.1R 6 Section 9

Check 3: Temperature and shrinkage – ACI 440.1R 6 Section 10

4.2 Design parameters sheet

The calculation sheet contains preliminary calculations: Design material properties (24).

$f_{fd\ bar}$ = $main\ reforcement\ design\ tensile\ strenght$ ($Eq.$ 4.4)

$$f_{fd\ bar} = C_E \cdot f_{fu\ bar} \tag{4.4}$$

$f_{fd\ cr}$ = $cross\ rod\ design\ tensile\ strenght$ ($Eq.$ 4.5)

$$f_{fd\ cr} = C_E \cdot f_{fu\ cr} \tag{4.5}$$

$\varepsilon_{fd\ bar}$ = $main\ reforcement\ design\ tensile\ strain$ ($Eq.$ 4.6)

$$\varepsilon_{fd\ bar} = C_E \cdot \varepsilon_{fu\ bar} \tag{4.6}$$

Design Example: Design Material Properties

$C_E = 0.7$

$f_{fd\ bar} = 0.7 \cdot 60.6 = 42.42\ ksi$

$f_{fd\ cr} = 0.7 \cdot 153 = 107.1\ ksi$

$\varepsilon_{fd\ bar} = 0.7 \cdot 0.01 = 0.007$

4.3 Calculation sheet

4.3.1.1 *Check 1: Flexure (ACI 440.1R 6 Section 8)*

"Assumptions:

- Strain in the concrete and the FRP reinforcement is proportional to the distance from the neutral axis;
- The maximum usable compressive strain in the concrete is assumed to be 0.003;
- The tensile behavior of the FRP reinforcement is linearly elastic until failure;
- Perfect bond exists between concrete and FRP reinforcement."

The strength design philosophy states that the design flexural strength at a section of a member must exceed the factored moment (Eq.4.7).

Design flexural strength refers to the nominal flexural strength of the member multiplied by a strength reduction factor ϕ.

$$\phi M_{n\,tot} \geq M_u \tag{4.7}$$

The nominal flexural strength of FRP-reinforced concrete member can be determined on strain compatibility, internal force equilibrium, and the controlling mode of failure.

To define the reduction factor ϕ (Eq. 4.10) is necessary to determine the failure mode by comparing the FRP reinforcement ratio (ρ_f), obtained through the Eq. 4.8, to the balanced reinforcement ratio (ρ_{fb} concrete crushing and FRP rupture occur simultaneously - Eq. 4.9).

$$\rho_f = \frac{A_n}{bd} \tag{4.8}$$

$$\rho_{fb} = 0.85\beta_1 \frac{f_c'}{f_{fd\ bar}} \frac{E_f \varepsilon_{cu}}{\left(E_{f\ bar}\varepsilon_{cu} + f_{fd\ bar}\right)} \tag{4.9}$$

$$\phi = \begin{cases} 0.55 & for \quad \rho_f \leq \rho_{fb} \;\; failure\ by\ FRP\ rupture \\ 0.3 + 0.25\dfrac{\rho_f}{\rho_{fb}} & for \;\; \rho_{fb} < \rho_f < 1.4\rho_{fb} \\ 0.65 & for \quad \rho_f \geq 1.4\rho_{fb} \;\; failure\ by\ concrete\ crushing \end{cases} \tag{4.10}$$

If the reinforcement ratio is less than the balanced ratio ($\rho_f < \rho_{fb}$), FRP rupture failure mode governs. Otherwise, ($\rho_f > \rho_{fb}$) concrete crushing governs.

FRP fails is a brittle manner. Concrete crushing is a slightly more favorable failure mode than FRP rupture, which is reflected in the strength reduction factor.

Because FRP members do not exhibit ductile behavior, a conservative strength reduction factor should be adopted to provide a higher reserve of strength in the member. The Japanese recommendations for design of flexural members using FRP suggest a strength reduction factor equal to 0.77 (JSCE 1997b). Other researchers (Benmkrane et.al. 1996a) suggest a value of 0.75 determined based on probabilistic concepts.

Based on ACI 318-05, the ϕ factor for compression controlled failure is 0.65, with a target reliability index between 3.5 to 4. A reliability analysis on FRP-reinforced beams in flexure using Load Combination 2 from ACI 318-05 for live to dead load ratios between 1 and 3 indicated reliability index between 3.5 and 4 when the ϕ factor was set to 0.65 for concrete crushing failure, and 0.55 for reinforcing bar rupture failure .

A non-linear sectional analysis of curvatures at failure showed that the curvatures of typical FRP reinforced beams at failure varied between 0.0138/d and 0.0176/d for tension-controlled failures, and between 0.0089/d and 0.012/d for compression-controlled failures (Gulbandsen 2005).

ACI 318-05 considers a failure tension-controlled whenever the curvature is greater than 0.008/d. this indicates that due to the low modulus of elasticity of reinforcement, FRP-reinforced beams will have large deflections at ultimate, and that FRP-reinforced beams that fail by FRP reinforcing bar rupture will have larger deflections at ultimate than those that fail by concrete crushing. Even though the curvature values of FRP-reinforced beams are larger than those of equivalent steel-reinforced beams, the committee recommends a ϕ factor of 0.55 for tension-controlled failure to maintain a minimum reliability index of 3.5.

While a concrete crushing failure mode can be predicted based on calculations, the member as constructed may not fail accordingly. For example, if the concrete strength is higher than specified, the member can fail due to FRP rupture. For this reason and to establish a transition between the two values of a section controlled by concrete crushing is defined as a section in which $\rho_f \geq 1.4\rho_{fb}$, and a section controlled by FRP rupture is defined as one in which $\rho_f < \rho_{fb}$.

Design Example: Failure mode

$\beta_1 = 0.8\ (stress\ block)$

$\rho_{fb} = 0.0218$

$1.4\rho_{fb} = 0.030$

$\rho_f(A_n = 0.2421\ in^2) = 0.0121$

$\therefore \rho_f < \rho_{fb} \rightarrow FRP\ rupture$

$\phi = 0.55$

Only tensile reinforcement is included in the flexural reinforcement and tensile strength of concrete is ignored.

FRP is not permitted as compression reinforcement.

The nominal resistance is dependent on the failure mode ($M_{n\,tot}$).

$M_{n\,tot}$ is computed from Eq.4.11

$$M_{n\,tot} = M_n \cdot p \tag{4.11}$$

Where:

M_n = *nominal flexural strength that is calculated considering the wall system like a beam;*

p = *factor that bring back to wall system (Eq.4.12)*

$$p = \frac{B}{s} \tag{4.12}$$

$B = panel\ base$

$s = spacing\ between\ I - bar\ reinforcement$

When $\rho_f > \rho_{fb}$ the failure of the member is initiated by crushing of the concrete, and the stress distribution in the concrete can be approximated with the ACI rectangular stress block.

The nominal strength is given in the fallowing Eq. 4.13, based on the equilibrium of forces and strain compatibility:

$$M_n = A_n \cdot f_f \left(d - \frac{a}{2}\right) \tag{4.13}$$

Where

$a = depth\ of\ equivalent\ rectangular\ stress\ block, in\ (mm)\ (Eq. 4.14)$

4.14

$$a = \frac{A_n \cdot f_f}{0.85 \cdot f_c' \cdot b}$$

f_f = $stress\ in\ FRP\ reinforcement\ in\ tension, psi\ (MPa)$(Eq. 4.15) $\leq f_{fd\ bar}$

4.15

$$f_f = \left(\sqrt{\frac{(E_f \varepsilon_{cu})^2}{4} + \frac{0.85\beta_1 f_c'}{\rho_f} E_f \varepsilon_{cu}} - 0.5 E_f \varepsilon_{cu} \right)$$

$b = s = spacing\ between\ the\ main\ reinforcement$

$A_n = net\ area\ of\ main\ reinforcement$

$E_{f\ bar} = modulus\ of\ elasticity\ for\ main\ reinforcement$

$\varepsilon_{cu} = ultimate\ concrete\ strain$

$\beta_1 = 0.8\ (stress\ block)$

If failure of member is not controlled by FRP rupture, the minimum amount of reinforcement to prevent failure upon cracking is automatically achieved.

When $\rho_f < \rho_{fb}$, the failure of the member is initiated by rupture of the FRP bar, and the ACI stress block is not applicable because the maximum concrete strain (0.003) may not be attained.

The analysis incorporates two unknowns: the concrete compressive strain at failure ε_{cu} and the depth of the neutral axis c. in addition, the rectangular stress block factor, β_1 is unknown.

The analysis involving all these unknowns becomes complex.

Nominal flexure strength at a section can be computed as shown in Eq.4.16.

4.16

$$M_n = A_n f_{fd\ bar} \left(d - \frac{\beta_1 c}{2} \right)$$

For a given section, the product of $\beta_1 c$, varies depending on the material properties and FRP reinforcement ratio. The maximum value for this product is equal to $\beta_1 c_b$ and is achieved when the maximum concrete strain (0.003) is attained. A simplified and conservative calculation of the nominal flexural strength of the member can be based on the Eq.4.17 and Eq.4.18 as follows:

$$M_n = A_n f_{fd\ bar}\left(d - \frac{\beta_1 c_b}{2}\right) \tag{4.17}$$

$$c_b = \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fd\ bar}}\right) d \tag{4.18}$$

If a member is designed to fail by FRP rupture ($\rho_f < \rho_{fb}$), a minimum amount of reinforcement should be provided to prevent failure upon concrete cracking.

The minimum reinforced area for FRP-reinforced members is obtained by the fallow Eq.4.19 (25).

$$A_{f,min} = max\left(4.9\sqrt{f_c'}; 330\right)\frac{bd}{f_{fd\ bar}} \tag{4.19}$$

Design Example: Check 1 – Flexure (for the economic choice)
$$B = 12\ in$$
$b = s = 4\ in$
$A_{fbar} = 0.3221\ in^2$
$A_n = 0.3221 - (0.16 \cdot 0.50) = 0.2421\ in^2$

$T_w = 0.16\ in$

$D_{cr} = 0.50\ in$

$\rho_f = 0.0121$

$\rho_{fb} = 0.0218$

$\therefore \rho_f < \rho_{fb} \rightarrow FRP\ rupture$

$\phi = 0.55$

$M_n = 0.2421 \cdot 42420 \left(5 - \frac{0.8 \cdot 1.5}{2}\right) = 45187.48\ psi - in = 3.76 kip - ft$

$c_b = \left(\frac{0.003}{0.003 + 0.007}\right) \cdot 5 = 1.5\ in.$

$M_{n\ tot} = 3.76 \cdot 3 = 11.29\ kip - ft$

$p = \frac{12}{4} = 3$

$\phi \cdot M_n = 6.2\ kip - ft > M_u$

$A_{f,min} = \max(0.163; 0.155) = 0.163\ in^2$

$\therefore A_n > A_{f,min}$

Design check 1 OK

4.3.1.2 *Check 2: Shear (ACI 440.1R 6 Section 9)*

The shear design philosophy is the same as the flexure one.

The strength reduction factor of 0.75 given by ACI 318-05 for reducing nominal shear of steel-reinforced concrete members should also be used for FRP reinforcement.

The factored shear strength ϕV_n must be larger than the factored maximum shear force V_u (Eq.4.20)

$$\phi V_n \geq V_u \tag{4.20}$$

According to ACI 318-05 the nominal shear strength of a reinforced concrete cross section V_n is the sum of the shear resistance provided by concrete V_c and the FRP shear reinforcement V_f .

Compared with a steel-reinforced section with equal areas of longitudinal reinforcement, a cross section using FRP flexural reinforcement after cracking has smaller depth to the neutral axis because of the lower axial stiffness.

The compression region of the cross section is reduced, and the crack widths are wider.

The contribution of longitudinal FRP reinforcement in terms of dowel action has not been determined. Because the lower strength and stiffness of FRP bars in the transverse direction, however, it is assumed that their dowel action contribution is less than of an equivalent steel area.

In the Strongwell’s system of seawall the shear reinforcement is represented by the connectors and for safety their contribution to the shear resistance is neglected. Consequently, the nominal shear strength is computed from the Eq. 4.21 (26).

$$V_n = V_c = 5\sqrt{f_c'}bc \tag{4.21}$$

Where

c =*distance from extreme compression fiber to the neutral axis, in. (mm) (Eq.*4.22*)*

$$c = kd \quad (4.22)$$

$k = ratio\ of\ depth\ of\ neutral\ axis\ to\ reinf.depth$ *(Eq.*4.23*)*

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \quad (4.23)$$

$n_f = ratio\ between\ modulus\ of\ elasticity\ of\ FRP\ and\ concrete$ $(Eq.$ 4.24)

$$n_f = \frac{E_f}{E_c} \quad (4.24)$$

$b = s = spacing\ between\ the\ main\ reinforcement$

Design Example: Check 2 – Shear (for the economic choice)
n_f = 1.31
ρ_f = 0.0121
$b = s =$ 4 in
d = 5 in
k = 0.16
c = 0.82 in
$V_n = V_c$ = 1153 psi

$\phi \cdot Vn = 0.75 \cdot 1153 = 864.84\, psi > V_u = 701\, psi$

Design check 2 OK

4.3.1.3 *Check 3: Temperature and Shrinkage (ACI 440.1R 6 Section 10)*

Shrinkage and temperature reinforcement are intended to limit crack width.
The stiffness and strength of reinforcing bars control this behavior. Shrinkage cracks perpendicular to the member span are restricted by flexural reinforcement. Thus, shrinkage and temperature reinforcement are only required in the direction perpendicular to the span.
The lower and the upper bound limits are 0.0014 and 0.0036.
Spacing of shrinkage and temperature of FRP reinforcement should not exceed three times the slab thickness or 12 in (300mm).
The amount of reinforcement should be determined by using of following Eq.4.25:

$$\rho_{f,ts(bar,;\, cross)} = 0.0018 \cdot \frac{60000}{(f_{fd\ bar}; f_{fd\ cross})} \frac{E_s}{(E_{f\ bar}; E_{f\ cross})} \qquad 4.25$$

Both layers of the reinforcement are used to calculate the temperature and shrinkage reinforcement ratios (Eq.4.26 and Eq.4.27)

$\rho_{bar,ts} = main\ bar\ temp\ and\ shink\ reinf.\ ratio$

$$\rho_{bar,ts} = \frac{2 \cdot A_{fbar}}{D \cdot b} \qquad 4.26$$

$\rho_{cr,ts} = cross\ rod\ bar\ temp\ and\ shink\ reinf.\ ratio$

$$\rho_{cr,ts} = \frac{2 \cdot A_{cr}}{D \cdot b_{cr}} \tag{4.27}$$

Where

A_{cr} = $cross\ rod\ area\ (Eq. 4.28)$

$$A_{cr} = \frac{\pi}{4} D_{cr}^2 \tag{4.28}$$

b_{cr} = $cross\ rod\ spacing$

The check is satisfied if are verify the following inequities (Eq.4.29 and Eq.4.30) (27).

$$\rho_{bar,ts} \geq \rho_{f,ts\ bar} \tag{4.29}$$

$$\rho_{cr,ts} \geq \rho_{f,ts\ cross} \tag{4.30}$$

Design Example: Check 3 – Temperature and Shrinkage (for the economic choice)
E_s = $modulus\ of\ elasticity\ \ for\ steel$ = 29000 ksi
$\rho_{f,ts\ bar\ main}$ = 0.0098 max $controls$ = 0.014
A_{fbar} = 0.3221 in^2
b = s = 4 in
D = $thickness\ panel\ (fixed\ parameter)$ = 5.5 in
$\rho_{bar,ts}$ = 0.0293

$\rho_{bar,ts} > \rho_{f,ts\ bar\ main}$

Design check 3 for main reinforcement OK

$A_{cr} = \frac{\pi}{4}(0.5)^2 = 0.196\ in^2$

$b_{cr} = 4\ in$

$\rho_{f,ts\ cross} = 0.0055$

$\rho_{cr,ts} = 0.0178$

$\rho_{cr,ts} > \rho_{f,ts\ cr}$

Design check 3 for cross rod OK

4.4 Result sheet

In this sheet are summarized the results of the checks.

Design Example:

Check 1 – Flexure

$M_n = 11.3\ kip - ft$

$\phi \cdot M_n = 6.21\ kip - ft > M_u$

Design check 1 **OK**

Check 2 – Shear

$V_n = V_c = 1153\ psi$

$\phi \cdot Vn = 0.75 \cdot 1153 = 864.84\ psi > V_u = 701\ psi$

Design check 2 **OK**

Check 3 – Temperature and Shrinkage

$\rho_{f,ts\ bar\ main} = 0.014$

$\rho_{bar,ts} = 0.029$

$\rho_{bar,ts} > \rho_{f,ts\ bar\ main}$

$\rho_{f,ts\ cross} = 0.0055$

$\rho_{cr,ts} = 0.0178$

$\rho_{cr,ts} > \rho_{f,ts\ bar\ main}$

Design check 3 **OK**

5 CHAPTER:

DURABILITY OF GFRP RC SEAWALLS

Durability is the ability of a generic material to last a long time without significant deterioration.

From a sustainability perspective, a material, component or system may be considered durable when its useful service life (performance) is fairly comparable to the time required for related impacts on the environment to be absorbed by the ecosystem.

A durable material helps the environment by conserving resources and reducing wastes and the environmental impacts of repair and replacement. Construction and demolition waste contribute to solid waste going to landfills. The production of new building materials depletes natural resources and can produce air and water pollution.

Nowadays, for the majority of the types of construction, sustainability is a research topic for some time and is beginning to bear fruit in the modern construction design. Instead, for the seawall constructions, as for the past, the durability and low maintenance are still challenges to be won.

A combination of concrete and internal GFRP reinforcement represent a solution with high potential, particularly when construction costs and completion time could be simultaneously reduced by introduction integrated GFRP structural reinforcement.

The research proposed intends investigate the long-term performance of concrete seawall using structural GFRP reinforcement.

In this section is showed the casting of the prototype, the construction of the specimens, the beginning of the experimentation for the accelerated ageing and deployment at waterfront site, and the laboratory test at zero time.

5.1 Durability of seawall construction

Deterioration of reinforced concrete elements and structures is a natural consequence of the ageing process. Clearly, the relative importance of the various mechanisms will vary from country to country and even region to region, and no generally applicable ordering of the mechanisms can be made, but a comparison among the different definitions of durability could be interesting in terms of general meanings:

"Durability is the capability of maintaining the serviceability of a product, component, assembly, or construction over a specified time...in particular, concrete durability is the resistance to weathering action, chemical attack, abrasion and other degradation processes" (28).

"Durability, defined as conservation of the physical and mechanical proprieties of materials and structures, is the essential propriety to allow the maintenance of security levels during the service life" (29).

"A durable structure shall meet the requirements of serviceability, strength and stability throughout its intended working life, without significant loss of utility or excessive unforeseen maintenance" (30),

The establishment of functional requirements for the seawall and the choice of suitable options for design have been outline in Chapter 2. The choice of the materials will be a relevant consideration in the identification of options at each stage and subsequently in making a choice as to the preferred option. The engineer should choose materials that are:

- Suitably durable;
- Environmental acceptable;
- Economical in use.

Seawalls, and the materials that compromise them, must survive in a hostile environment. They must resist abrasion, chemical attack, corrosion, marine borers and vandalism. Durability of materials, and especially loss by abrasion, is very site-specific.

The general description, durability, is taken to include resistance to the climate, abrasion by foreshore material, chemical attack/corrosion and biodegradation by bacteria, worms or molluscs.

The Figure 5.1 is a diagrammatic representation of a seawall and foreshore which shows the limits of waves and tides, a representative time distribution for water level and four zones used to describe the differing environments encountered around the crest, body and toe of a seawall.

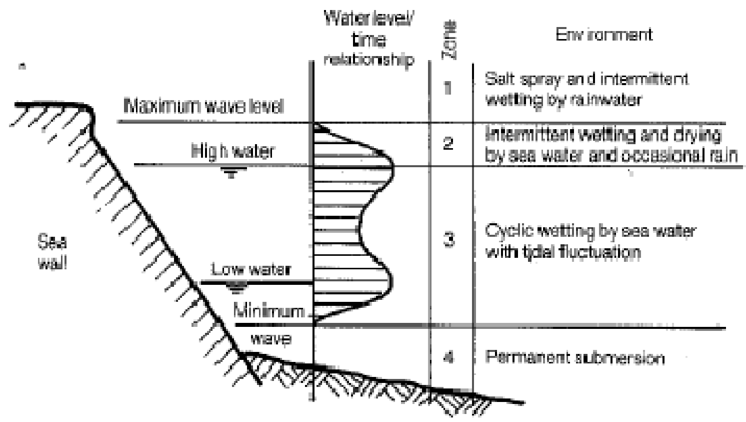


Figure 5.1: the weathering zones of a seawall (30)

The seawall above highest wave level is in zone 1 and receives wind-blown salt spray plus rain and exposure to the sun and drying winds. It is an environment conductive to corrosion and chemical attack.

The face of the seawall between highest wave level and high water, zone 2, is intermittently wetted by both seawater and rainwater, which is conducive to both chloride penetration and corrosion.

The cyclic wetting and drying of the seawall between high water and lowest wave level in zone 3 provides conditions for chloride penetration, corrosion and abrasion by foreshore materials moved by wave or tidal currents.

Permanent submersion of the seawall in zone 4 inhibits corrosion and normally produces only limited abrasion.

The manner in which abrasion, chemical attack/corrosion and biodegradation reduces durability varies both with the location in the seawall and with the type of material considered.

5.1.1 Resistance to abrasion

A seawall on a sand or shingle foreshore will be subject to abrasion by particles of the foreshore carried by tide and wave currents and drive into contact with the wall. If the material size is large enough to resist motion no abrasion will occur, but the larger the particles carried, the greater will be the rate of abrasion.

High rates of abrasion resulting from waves over 3 m high and shingle foreshore can be expected to scour away the face of concrete to expose the large aggregate in few months. The surface removal beyond this stage will be much slower provided (31):

- The coarse aggregate used is as hard as the beach shingle;
- The fine aggregate used is the minimum needed to ensure a dense concrete;
- The cement/aggregate ratio is adequate;
- The water/cement ratio is the minimum to facilitate placing and compaction of the concrete;
- The concrete has been fully compacted.

5.1.2 Resistance to chemical attack and corrosion

The chemistry of the seawater and its temperature range are relevant to the chemistry of the degradation.

Salinity of sea water along the coastal areas of the conterminous United States varies with the month of the year as well as with geographic location. For example, the salinity of the ocean water off Miami Beach, Fl., varies from about 34.8 o/oo (34.8 pounds of salt per 1000pounds of seawater) in October to 36.4 o/oo in May and June, while diagonally across the country, off the coast of Astoria, Oregon, the salinity of sea water varies from 0.3 o/oo in April and May to 2.6 o/oo in October. The water off the coast of Miami Beach has a high salt content because it is undiluted sea water. Off the coast of Astoria, however, the sea water is less saline because it is mixed with the fresh water of the mighty Columbia river.

The range of temperature that characterized Miami (Fl) varies from the average of maximum temperatures that is 84° Fahrenheit to the average of the minimum temperatures that is 70° Fahrenheit.

The attention is focus, now, on the case of reinforced concrete.

Seawater salts will react with constituents of concrete at rates limited by the temperate climate. In the presence of oxygen and water, steel will corrode. Where steel embedded in the concrete corrodes, the corrosion products will cause expansive forces which will burst the concrete.

The process of chemical attack and corrosion are briefly described below and rates of degradation are assessed. Factors which increase the rate of degradation and actions which might be used to slow it are reviewed.

Seawater attack on reinforced concrete in seawalls occurs in three stages:

- The reduction of the chemical protection given to reinforcement by concrete surrounding it (loss of passivity);
- Oxidation of the reinforcement and generation of expansive forces within the concrete (rusting);

- Spalling away of the concrete cover to the reinforcement and accelerated corrosion.

Visible spalling, the third stage, is an indication of a substantial problem which is likely to lad ultimately to failure. The stages describe above are discussed in more detail below.

Initially, the reinforcement is protected by a combination of chemical reactions on the surface of the steel and the barrier provided by the concrete cover. Fresh concrete is very alkaline with pH values ranging from 12.6 to 13.5 and at these pH levels the electrochemical reactions of the corrosion processes are inhibited. Chlorides from seawater penetrate the concrete during the life of the structure and reduce its alkalinity. The rate ingress of the chloride is controlled by the thickness of the concrete cover and the permeability of the concrete. These factors also determine the rate of diffusion of oxygen and moisture which are also necessary for corrosion to occur.

Corrosion of the reinforcement in a concrete seawall is usually as a result of chloride attack. However, an alternative process, known as carbonation, should be mentioned. Carbonation is the effect of atmospheric carbon dioxide reacting with the alkaline materials in concrete to reduce the alkalinity and allow corrosion to commence. The onset of corrosion does not take place until the carbon dioxide diffuse through the concrete cover. Corrosion will then only occur when sufficient moisture is present. Carbonation appears to be largely inhibited in a seawall environment by the presence of the chlorides.

Once the alkaline environment is destroyed by the ingress of chlorides the metal ions react in the presence of oxygen and water to form ferrous of ferric oxides. The oxides occupy considerably more volume than that of the steel destroyed and exert internal expansive forces large enough to spall off the concrete cover.

By the time the concrete cover is spalled away, the bond between concrete and steel is much reduced and the reinforced concrete is no longer an effective composite material. The reinforcement is even more vulnerable to corrosion, as it exposed to a ready supply of oxygen and water with the result that the process is accelerated.

The penetration of chlorides into the concrete will reduce alkalinity and eventually make the maintenance of passivity impossible. Recent results from the Concrete in the Oceans programme reported by Leeming[1] indicate that in the splash zone:

- Chloride will penetrate cover of 25 to 75 mm to reach the reinforcement in ten years;
- Chloride reaches the reinforcing steel down cracks as a small as 0.1 mm at the surface early in the design life of structure, and this is forerunner of corrosion to the steel;
- Moist curing of green concrete in the absence of chlorides reduces chloride penetration relative to that for air-cured concrete;
- Curing with seawater increases the rate of penetration;
- The addition of some 20% pulverized fuel ash reduces penetration of chloride.

This work suggests that mass concrete should be used in preference to reinforced concrete within zones 2 and 3 wherever this is a practical proposition. It further indicates that where reinforcement is provided all possible steps should be taken to minimize surface cracking, and that workmanship in placing concrete should be such as will ensure no defects in the cover to the reinforcement. Curing should be moist and free from chlorides.

While the penetration of chlorides will lead to loss of chemical protection of reinforcement from corrosion this stage is by no means the end of structural integrity of a seawall. If the reinforcement is in close contact with the cement paste, then the diffusion of oxygen proceeds very slowly and rust formation is inhibit. Given concrete with complete absence of avoidable defects, then the life expectancy is very long. The use of material and the construction process should be such as will come as close as possible to the creation of defect-free concrete (31).

Below is showed the typical process of corrosion for structure in reinforced concrete, in the Section 5.2 will be illustrated the materials, in particular the potentiality of the application of GFRP in this area.

5.2 Characteristics of materials used in in the prototype of seawall

For the prototype of seawall's project, the materials used are:

- Concrete
- GFRP

In this section will be illustrated their characteristics.

5.2.1 Concrete

Concrete is the most commonly used material in seawall construction. The opportunity to form suitable shapes of a relativity high density having potentially good durability makes it an attractive all-purpose material. Concrete is reasonably economic in most situation, and not too visually intrusive when used with care. It is important to recognize that the environment requiring good-quality concrete for durability is also a difficult one in which to create a good product.

5.2.1.1 *Constituents*

The cement likely used for seawalls are:

- Ordinary Portland cement
- Rapid-hardering Portland cement;
- Portland blast furnace cement
- Sulphate-resisting Portland cement

The bulk of all works should use ordinary Portland cement with a rapid-hardering variety being an option where specific site conditions warrant the additional cost and handling problems. Sulphate-resisting Portland cement5 may be justified where waters are warmed or polluted by local discharge, but the basic requirement for a dense product remains unchanged.

For the aggregates, if the abrasion is of particular concern, then the coarse aggregate should be at least as hard as the material causing abrasion and the fine aggregate reduced to a minimum, consistent with density requirements and workability.

The water used in the concrete should be free from harmful substance: as a generality, water fit for consumption can be used.

Admixtures for purposes of entraining air, improving flow characteristics for pumping, allowing the use of lower water/cement ratio, delaying settling, etc. should only be used where it can be shown by site tests that required effect can be achieved without deterioration of concrete or steel reinforcement.

Self-consolidating concrete (SCC), also known as self-compacting concrete and SCC, is a highly flow able, non-segregating concrete that spreads into place, fills formwork, and encapsulates even the most congested reinforcement, all without any mechanical vibration. It is defined as a concrete mix that can be placed purely by means of its own weight, with little or no vibration. As a high-performance concrete, SCC delivers these attractive benefits while maintaining all of concrete's customary mechanical and durability characteristics.

5.2.1.2 *Durability of concrete*

The ingress of various ions, liquids and gases from the environment is responsible for the deterioration of concrete directly or indirectly. For instance, the ingress of chlorides or carbon dioxide would depassivate the steel in concrete, and in the presence of oxygen and water, steel may start corroding. Similarly, the ingress of chemicals, such as acids, alkalis and sulfates are responsible for the chemical deterioration of concrete. Moisture movement during freezing and thawing action also causes deterioration of concrete [36]. Important degradation mechanisms in concrete structures include the following:

5.2.1.2.1 Chemical attack

- *Leaching and efflorescence*

Efflorescence occurs quite frequently on surface of concrete when water can percolate through the material, either continuously or intermittently, or when an exposed face is alternately wetted and dried. Efflorescence consists of deposited salts that are leached out of the concrete and are crystallized on subsequent evaporation of the water or interaction with carbon dioxide in the atmosphere. Typical salts are sulfates and carbonates of sodium, potassium, or calcium, the major constituent being calcium carbonate.

Efflorescence, in itself, is an aesthetic rather than a durability problem, but it does indicate that substantial leaching is occurring within the concrete. Extensive leaching causes an increase of porosity, thereby lowering the strength of the concrete and increasing its vulnerability to aggressive chemicals.

Thus, pastes that have a high content of calcium hydroxide are likely to be more prone to leaching and efflorescence and to have a greater potential for deterioration in unfavourable condition. The rate of leaching is dependent on the amount of dissolved salts contained in the percolating water. Soft waters, such as rainwater, are the most aggressive, while hard waters containing large amounts of calcium ions are the less dangerous. The temperature is also a consideration, since calcium hydroxide is more soluble in cold water than it is in warm water. Leaching is, indeed, most prevalent when the water can seep though the concrete, particularly under pressure (32).

- *Sulfate attack*

Sulfate attack is most simply defined as a series of chemical reactions between components of hardened concrete and sulfate ions. Usually it is the cement paste that is attacked. Sulfate ions can originate from within the concrete, such as from portland cement or admixtures, or from outside the concrete from sources including soil and industrial pollution in water and the atmosphere.

Sulfate attack can manifest in the form of expansion of concrete. When concrete cracks, its permeability increases and the aggressive water penetrates more easily into the interior, thus accelerating the process of deterioration.
Sulfate attack can also take the form of a progressive loss of strength and loss of mass due to deterioration in the cohesiveness of cement hydration products.
In hardened concrete, the formation of ettringite by sulfate attack can, but does not always, result in expansion and lead to cracking of the concrete. The conditions under which ettringite formation produces damage in the concrete are uncertain (32).

- *Alkali-aggregate reactions*

Expansion and cracking, leading to loss of strength, elasticity, and durability of concrete can also result from chemical reaction involving alkali ions from portland cement (or from other sources), hydroxyl ions, and certain siliceous constituents that may be present in the aggregate. The phenomenon is referred to as alkali-silica reaction. This alkali-aggregate reaction has two forms: alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR).
Alkali-silica reaction (ASR) is of more concern because aggregates containing reactive silica materials are more common. In ASR, aggregates containing certain forms of silica will react with alkali hydroxide in concrete to form a gel that swells as it adsorbs water from the surrounding cement paste or the environment. These gels can swell and induce enough expansive pressure to damage concrete. Typical indicators of ASR are random map cracking and, in advanced cases, closed joints and attendant spalled concrete. Cracking due to ASR usually appears in areas with a frequent supply of moisture, such as close to the waterline in piers, near the ground behind retaining walls, near joints and free edges in pavements, or in piers or columns subject to wicking action.
Alkali-carbonate reactions (ACR) are observed with certain dolomitic rocks.
The deterioration caused by ACR is similar to that caused by ASR; however, ACR is relatively rare because aggregates susceptible to this phenomenon are less

common and are usually unsuitable for use in concrete for other reasons. Aggregates susceptible to ACR tend to have a characteristic texture that can be identified by petrographers.

- *Acid and alkalis*

Generally, naturally occurring acidic groundwaters are not common, being confined to marshy or peaty regions, where mining operations and stockpiling of mine tailings have occurred. Highly acidic conditions may exist in agricultural and industrial wastes, particularly from the food and animal processing industries.
Acids that can attack concrete are: sulfuric acid, hydrochloric acid, nitric acid, organic acids such ad acetic and solution of CO_2. The rate of attack on the cement matrix depends on the solubility of the salts that are formed and thus on the nature of the anions involved.
Corrosion of reinforcing steel embedded in concrete (chemical effects, explained in the 5.1 Section):

- carbonation of concrete
- chloride induced

5.2.1.2.2 Physical attack

- *Freeze-thaw damage*

When the temperature reach values below 0°C, water contained in the pores of concrete can freeze, causing an increase in volume. Then generated tensile stresses may result in scaling, cracking or spalling of the concrete and, eventually, in its complete disintegration.
Three mechanisms of freeze-thaw deterioration can occur within concrete. The first mechanism is hydraulic pressure.
The second and third mechanisms of freeze-thaw deterioration occur within the microstructure of the concrete, or within the cement paste, as a result of osmotic and vapor pressures (32).

- *Wetting and drying*

Cyclic wetting and drying causes continuous moisture movement through concrete pores.

This cyclic effect accelerates durability problems because it subjects the concrete to the motion and accumulation of harmful materials, such as sulphates, alkalies, acids, and chlorides.

Cyclic wetting and drying is a problem for RC structures exposed to chlorides and its effects are most severe in mainly marine structures.

When concrete is dry or partially dry, and then exposed to salt water, it will imbibe the salt water by capillary suction. The concrete will continue to suck in the salt water until saturation or until there is no more reservoir of salt water. A concentration gradient of chlorides will develop in the concrete, stopping at some point in the interior of the concrete. If the external environment becomes dry, then pure water will evaporate from the pores, and salts that were originally in solution may precipitate out in the pores close to the surface. The point of highest chloride concentration may now exist within the concrete. On subsequent wetting, more salt solution will enter the pores, while re-dissolving and carrying existing chlorides deeper into the concrete. Cyclic wetting and drying increases the concentrations of ions such as chlorides, by evaporation of water. The drying of the concrete also helps to increase the availability of the oxygen required for steel corrosion, as oxygen has a substantially lower diffusion coefficient in saturated concrete (32).

- *Temperature changes*

A main problem can be created by the presence of large amount of evaporable water. If the rate of heating is high and the permeability of cement paste is low, damage to concrete may take place in the form of surface spalling. Spalling occurs when the vapour pressure of steam inside the material increases at a faster rate than the pressure relief by the release of steam into the atmosphere (32).

- *Wear and abrasion*

In certain application, severe wearing of concrete surface may lead to service problems. Three distinct types of wear have been distinguished:

Abrasion. Wearing by repeated rubbing of frictional processes (attrition). This term is used in connection with the traffic wear on pavement and industrial floors.

Erosion. Wearing by abrasive action of fluids and suspended solids. Erosion is a special case of abrasion and occurs in water-supply installations: canals, conduits, pipes, and spillways.

Cavitation. Impact damage caused when a high-velocity liquid flow is disturbed. It will occur at spillways and sluiceways in dams and irrigation installations (32).

- *Mechanical loads*

Concrete strength is defined as the maximum stress recorded during the load testing of specimens carried out to failure. The type of loading excites different types of strength: compressive-tensile-shear strengths are of main importance, both for static and dynamic (earthquake) loading. Typically dams have strength requirements at high maturity and have a rather high scatter of strength values as compared to common civil structures.

When a reinforced concrete member is overloaded or the design load is applied with an inadequate amount of curing time, damage to the concrete may occur due to these loads. The result of excessive or early loading is often represented by cracks developing in the tensile zone of the member or with shear cracking developing where the shear capacity of the beam was exceeded.

5.2.1.3 *Quality control*

The engineer should use through quality control of concrete to ensure durability of the finished product.

Slump tests (ASTM C143/C143M - 10) are unlikely to provide a suitable measure with the low water/cement ratios in use and compacting factor. At the seawall and immediately before the concrete is placed, samples should be taken and test cylindrical made to prove that density (ASTM C138/C138M – 10a) and 28-day strength (ASTM C39/C39M – 09a) complies with that required. These cubes should be made, cured and tested to the standardized procedure (32).

5.2.2 FRP – Fiber Reinforced Polymer

Recently, composite materials made of fibers embedded in a polymeric resin, also known as FRP's, have become an alternative to steel reinforcement for concrete structures. Because FRP materials are nonmagnetic and noncorrosive, the problems of electromagnetic interference and steel corrosion can be avoided with FRP reinforcement. Additionally, FRP materials exhibit several properties, such as high tensile strength, that take them suitable for use as structural reinforcement.
The use of FRP is predicated on performance attributes linked to their light weight, high stiffness-to-weight and strength-to-weight ratio, ease of installation in the field, potential lower system cost, and potentially high overall durability.
The mechanical behavior of FRP reinforcement differs from the behavior of conventional steel reinforcement, so it needed a change in the traditional design philosophy of concrete structures. FRP materials are anisotropic and are characterized by high tensile strength only in the direction of the reinforcing fibers. This anisotropic behavior affects the shear strength and dowel action of FRP bars as well as the bond performance. Furthermore, FRP materials do not yield; rather, they are elastic until failure. Design procedures must account for a lack of ductility in structural concrete members reinforced with FRP bars.
The analytical and experimental phases for FRP construction are sufficiently complete; therefore, has been made the ACI 440.R-06, a document that establishes recommendations for the design of structural concrete reinforced with FRP bars (it has been used for the realization of the software illustrated in the 4 Chapter)

5.2.2.1 *Constituents*

FRP composites materials are formed trough the physical combination of two or more phases, at least one of which is a reinforcing phase, and one is a matrix phase.

The concept is analogous of reinforced concrete wherein steel reinforcement is the reinforcing phase and concrete is the matrix phase. In a FRP composite, the reinforcing elements are fibers, most commonly glass, carbon or high tensile steel, and occasionally aramid. These fiber provide strength and stiffness to the composite and act as the main load carrying elements. The matrix (typically a thermoset resin such as polyester, vinyl ester, or epoxy) binds the fiber together allowing shear transfer between fibers; provides durability to the slender; and serves as a protective barrier.

E-glass fiber offer high tensile strength and economical cost; without the protection of an appropriate resin system, however, they are susceptible to degradation due to moisture and alkalinity (GFRP). Similarly, aramidic fibers (AFRP) are resistant to abrasion and impact, but show a propensity to creep, absorb moisture, and degrade under ultraviolet exposure. Carbon fiber (CFRP) are relativity inert to the environment.

5.2.2.2 *Application and use*

The material characteristics of FRP reinforcement need to be considered when determining whether FRP reinforcement is suitable or necessary in a particular structure.

The Figure 5.2 lists some of the advantages and disadvantages of FRP reinforcement for concrete structures when compared with conventional, steel reinforcement.

Advantages of FRP reinforcement	Disadvantages of FRP reinforcement
High longitudinal tensile strength (varies with sign and direction of loading relative to fibers)	No yielding before brittle rupture
Corrosion resistance (not dependent on a coating)	Low transverse strength (varies with sign and direction of loading relative to fibers)
Nonmagnetic	Low modulus of elasticity (varies with type of reinforcing fiber)
High fatigue endurance (varies with type of reinforcing fiber)	Susceptibility of damage to polymeric resins and fibers under ultraviolet radiation exposure
Lightweight (about 1/5 to 1/4 the density of steel)	Low durability of glass fibers in a moist environment
Low thermal and electric conductivity (for glass and aramid fibers)	Low durability of some glass and aramid fibers in an alkaline environment
	High coefficient of thermal expansion perpendicular to the fibers, relative to concrete
	May be susceptible to fire depending on matrix type and concrete cover thickness

Figure 5.2 lists of the advantages and disadvantages of FRP reinforcement

The corrosion resistance of FRP reinforcement is a significant benefit for structure in highly corrosive environments such as seawalls and other marine structures; or the use of FRP materials is a smart solution for structures that support magnetic resonance imaging. But, due to lack of experience in its use, FRP reinforcement is not recommended for moment frames or zones where moment redistribution is required. FRP should not be relied on to resist compression (33).

5.2.2.3 *Material characteristics*

To characterize the FRP materials are reported below three tables about the densities (Table 5.1), the typical coefficients of thermal expansion (Table 5.2) and tensile properties (Table 5.3).

The material characteristics are, also, subdivided in three different categories, in function of the types of fiber used (34).

Steel	GFRP	CFRP	AFRP
493.00 (7.90)	77.8 to 131.00 (1.25 to 2.10)	93.3 to 100.00 (1.50 to 1.60)	77.80 to 88.10 (1.25 to 1.40)

Table 5.1: typical densities of reinforcing bars, lb/ft^3 (g/cm^3)

Direction	CTE, $\times 10^{-6}$/°F ($\times 10^{-6}$/°C)			
	Steel	GFRP	CFRP	AFRP
Longitudinal, α_L	6.5 (11.7)	3.3 to 5.6 (6.0 to 10.0)	–4.0 to 0.0 (–9.0 to 0.0)	–3.3 to –1.1 (–6 to –2)
Transverse, α_T	6.5 (11.7)	11.7 to 12.8 (21.0 to 23.0)	41 to 58 (74.0 to 104.0)	33.3 to 44.4 (60.0 to 80.0)

Table 5.2: typical coefficients of thermal expiation for reinforcing bers

	Steel	GFRP	CFRP	AFRP
Nominal yield stress, ksi (MPa)	40 to 75 (276 to 517)	N/A	N/A	N/A
Tensile strength, ksi (MPa)	70 to 100 (483 to 690)	70 to 230 (483 to 1600)	87 to 535 (600 to 3690)	250 to 368 (1720 to 2540)
Elastic modulus, $\times 10^3$ ksi (GPa)	29.0 (200.0)	5.1 to 7.4 (35.0 to 51.0)	15.9 to 84.0 (120.0 to 580.0)	6.0 to 18.2 (41.0 to 125.0)
Yield strain, %	0.14 to 0.25	N/A	N/A	N/A
Rupture strain, %	6.0 to 12.0	1.2 to 3.1	0.5 to 1.7	1.9 to 4.4

Table 5.3: usual tensile properties of reinforcing bars

5.2.2.4 *Durability of FRP composites*

Anecdotal evidence provides substantial reason to believe that, if appropriately designed and fabricated, FRP systems can provide long service life and lower maintenance than equivalent structures fabricated from conventional materials. Actual data on durability, however, are sparse and not well documented, and in case where available, not easy accessible to the practicing engineer.

The durability of the resin system depends on several factors including the resin components and proportions as well as curing time and conditions. Resin system

available commercially are typically designed for specific end-use applications and their formulations are usually proprietary. Resin formulations are typically designed by manufacturer for specific applications based on mechanical, physical, chemical, electrical, or other considerations associated with the intended operating environment. Mechanical properties of the resin are published by manufacturers and are based on standard test method. Typically properties of FRP composite are reported above and can be found in ACI 440.1R-06 and ACI 440.2R-08.
Below the service conditions that FRP composite used with concrete are likely to encounter are summerize. Each section gives a brief summary of the effect that these environments have on the constituents usually found in FRP composite used with concrete.

- *Moisture (water and salt solution)*

All resins absorb moisture, with the percentage of moisture absorption depending on the resin structure, degree of cure, and temperature. The two primary effects of moisture uptake are plasticization (through hydrolysis) and a reduction in glass transition temperature, T_g. in general, moisture effects over the short-term cause more pronounced degradations in strength as opposed to stiffness of the concrete. In some application, performance is improved with the use of a protective barrier. Long-term exposure under specific conditions, especially in under-cured systems can result in irreversible degradation. Salt solution can cause blistering due to osmotic effects. In some cases, moisture has been observed to wick along the fiber-matrix interface, resulting in a loss of structural integrity. Moisture may also affect the fracture toughness of FRP composite with reported results being somewhat contradictory. In some cases, increase in fracture energy and fracture toughness are reported due to an increase in compliance resulting from the plasticization process. In other cases, reductions in fracture energy and toughness are reported due to degradation in the fiber-matrix interphase region.
In the case of glass fibers, degradation is initiated by moisture extracting ions from the fiber. This results in etching and pitting of the fiber (35).

- *Chemical solutions*

In most cases, chemical solutions affect the resin system rather than the fibers. The presence of specific salts/chemicals in the solution can accelerate deterioration of the resin system (35).

- *Alkaline environment*

Alkaline solutions, such as the pore water of concrete, have a high pH and a high concentration of alkali ions. Concrete pore water has a pH level as 13 for newly placed concrete, but will decrease with time. The combination of alkali ions and high pH has minimal effect on carbon fibers but may lead to degradation of the resin and interphase regions. E-glass fiber system must be properly designed and fabricated with the appropriate resin system (vinyl ester, epoxy) to protect the reinforcement from alkali attack.

It is critical that the resin be well cured before exposure to alkaline environments, which generally only occurs in a moist environment. The lack of moisture to aid in the migration of alkali ions results in a dramatic decrease in effect.

Alkali resistant glass fibers are available and can decrease the rate of deterioration. Corrosion resistant E-glass fibers have also been shown to be less susceptible to alkaline solutions than simple E-glass fibers (Wen et al.,2003).

Aqueous solutions with high pH are known to degrade the tensile strength and stiffness of GFRP bars (Porter and Barnes 1998), although individual results vary widely according to differences in tests methods (Robert et al.,2009). Furthermore, Rostasy (1997) documented that durability tests un highly concentrated alkaline solutions do not realistically depict the true damage phenomenon. Higher temperature and long exposure time tend to accelerate degradation. (Rostasy 1997; Sen et al. 1998; Takewaka and Khin 1996; Sheard et al. 1997; GangaRoa and Vijay 1997) (35).

- *Extreme temperature and thermal cycling*

Within the normal range of operating temperatures seen in the civil infrastructure environment, no major changes in performance are anticipated as long as the system is properly selected and processed such that the resulting glass transition temperature T_g is always higher than the maximum operational limit of the component/structure. The primary effects of temperature are on the visco-elastic response of the resin, and hence of the composite. As temperature increases, the elastic modulus of the resin will decrease. A slight elevation in temperature, however, can result in a beneficial post-cure of the composite. If the temperature exceeds the glass transition temperature (T_g), however, the FRP composite performance will decrease substantially.

Thermal cycling below T_g generally does not cause deleterious effects although extended thermal cycling of brittle resin systems can result in microcrack formation (35).

Temperature raging from -40° to 50° C doesn't influence the tensile strength and flexural modulus of elasticity of GRFP composites, which appear stable (Robert et. al. 2010).

The coefficient of thermal expansion (CTE) for FRP composite can be quite different from those of steel and concrete and vary considerably with fiber and resin type as well as fiber orientation and constituent volume fractions. For the case of unidirectional reinforced glass fiber reinforced composite, the CTE in the fiber direction is similar to that of steel and concrete. For unidirectional carbon and aramid fiber reinforced composites, the CTE in the fiber direction is generally lower than that the steel and concrete.

- *Freeze and freeze-thaw*

In general, freeze and freeze-thaw conditions do not affect fibers; although they can affect the resin and the fiber-resin interface. Polymeric resin system are known to embrittle under cold conditions, resulting in increased strength and stiffness. Freeze-thaw effects can be more severe due to moisture-initiated effects causing microcrack growth and coalescence because of cycling. The presence of road salts in wet conditions combined with freeze thaw cycling can cause microcrack formation and gradual composite degradation due to crystal formation and increased salt concentration.

If the concrete, after FRP external strengthening, becames critically satured and subjected to freezing and thawing, this can demage the concrete interface and cause bond failure even with air-entrained concrete (35).

- *Creep and relaxation*

Polymer resin generally exhibit creep and relaxation behavior. Since glass and carbon fibers are linear elastic to failure, the addition of these fibers increases the creep resistance of the resins. Consequently, creep and relaxation are more pronounced when load is applied transverse to the fibers or when the composite has low fiber volume fraction. Creep can also be an issue in shear for unidirectional reinforcement. Creep can not be a significant factor if the stress levels are kept sufficiently low. Typically. Thermosetting resins are more resistant to creep than are thermoplastic. Creep-stress relaxation properties are dominated by the resin dependent properties, rather than the fiber or interfacial properties. Under-cured resins are susceptible to creep during the early stages of service. This susceptibility diminishes with the time. Carbon fiber are the least susceptible to creep rupture; aramid fibers are moderately susceptible; and glass fibers are the most susceptible to creep rupture.

E-glass fibers are susceptible to environmental stress corrosion cracking. This is a delayed brittle fracture effect that is caused by synergism between stress and the environment (Jones 1999). Nkurunziza et al. (2005) conducted two series of stress rupture tests on GFRP bars under two different stress corrosion environmental conditions: de-ionized water and alkaline solution. After 417 days of exposure, the average residual tensile strength was found to be 139 and 144% of the design tensile strength for bars conditioned in de-ionized water at 25 and 38% stress level respectively. In alkaline solution, this range was 126 to 97%. These results showed that the testes GFRP bar performed very well under these extreme loading and environmental conditions. More important, no significant change in the elastic modulus was observed under the stress levels and environmental conditions used (Nkurunziza et al., 2005) (35).

- *Fatigue*

It has been demonstrated (Curtis, 1989) that the matrix composition has greater effect on the fatigue performance of FRP than the type of fiber used. No significant difference in fatigue performance of FRP materials was observed when four different carbon fiber types were placed in the same matrix, however, using the same fiber in different epoxy resin matrices had an effect on the slopes of resulting experimental curves. (Curtis, 1989) The individual fiber within unidirectional composites have few defects and are consequently resistant to crack initiation. Additionally, any crack that does form travels through the matrix and is not transmitted adjacent fibers. It is these thoughness and crack arresting properties that contribute to the good fatigue performance of FRP materials (35). Fatigue loading conditions have been clearly shown to affect the bond performance of externally bonded FRP systems (Harries and Aidoo 2006)

- *Ultraviolet radiation (UV)*

In general, effects are rarely severe in terms of mechanical performance, although some resins can show significant embrittlement and surface erosion. The most deleterious effect of UV exposure is generally the UV-related damage, which is limited to the FRP surface, but the potential for increased penetration of moisture and other agents via the damaged region. FRP composite can be protected from UV-related degradation with appropriate additives in the resin (35).

6 CHAPTER:

EXPERIMENTAL PROGRAM

In this section the experimental program about the durability of GFRP RC seawalls will be explain.

This study focuses on the longevity of a concrete seawall when pultruded GFRP grating with SIP forms are used as reinforcement.

6.1 Program of the experimental on the long term performance of GFRP materials exposed to harsh environment

The objective of the proposed research is to investigate the longevity of concrete seawalls reinforced with GFRP SIP panels and provide the scientific community and practioners with experimental data on the long term performance of GFRP SIP panels when exposed to harsh environment.

6.1.1 Specimen design

The cross-section of the GFRP-RC prototype seawall is shown in Figure 6.1. The GFRP internal reinforcement consists of two layers of prefabricated panels that integrate a pultruded grating with stay-in-place (SIP) forms. The two layers are held together by horizontal connectors.

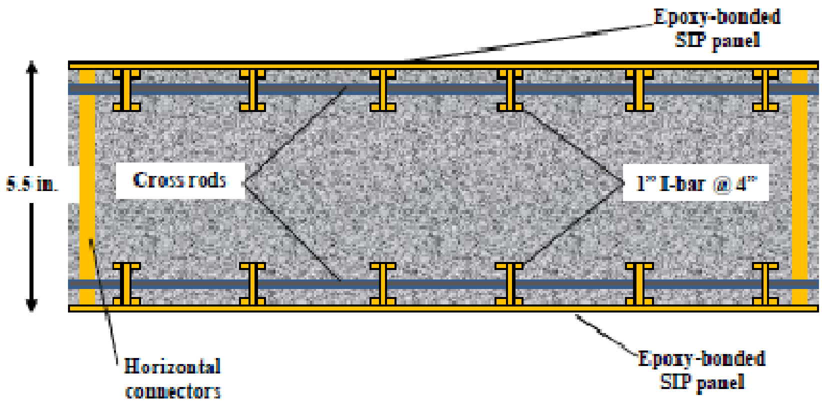


Figure 6.1: seawall cross-section

The design of the GFRP-RC prototype is based on an existing design of a steel RC seawall. A typical steel RC seawall is made of RC panels 10 ft tall, 6 ft wide, and 0.5 ft thick.

The internal reinforcement is a 1 ft by 1 ft grid of #4 MMFX steel rebars. The reinforcing steel grid is place at the center of the seawall panel. Minimum cover over the rebar is 2.5”.

The thickness of the prototype is 5.5 in., while the height is reduced to 5 ft because the area interested to the tidal action and subjected the worst environmental conditions is estimated to be 4÷6 ft .

6.1.2 Specimen preparation

Four prototypes of seawall panels were realized.

The casting took pace at Supermix precast plan in Miami, Fl.

A 5000 psi concrete mix design was used.

The seawall prototypes were casted vertically at the same time and from the same batch of concrete.

The prototype seawall panels were cut in horizontal position, to obtain fifty-seven 4 in × 5.50 in × 60 in beam specimens (Figure 6.2).

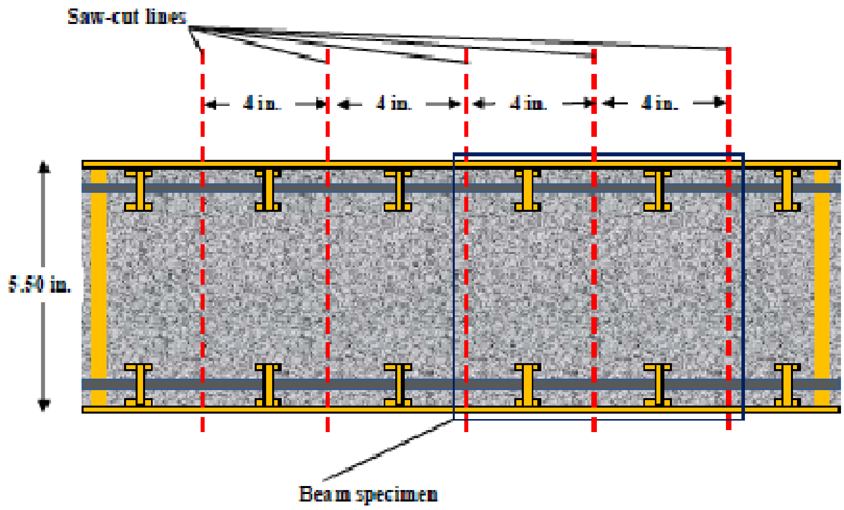


Figure 6.2: detail of the saw-cutting of the seawall panel

6.1.3 Experimental program

The beam specimens are divided in nineteen triplets and subjected to different ageing treatments over a period of time of 12 months as shown in Table 6.1.
Five types of ageing treatments are considered.

1. *Preserved ageing (LAB)*: the specimen is aged in an indoor environment at room temperature. This condition is referred to as LAB.

2. *Natural ageing (NA)*: the specimen is aged at a waterfront site subjected to tidal action.

3. *Accelerated ageing at room temperature (AA-RT)*: the specimen is aged in a highly alkaline wet environment at room temperature.

4. *Accelerated ageing at 104° F (AA-104F)*: the specimen is aged in a highly alkaline wet environment at a temperature of 104° F.

5. *Accelerated ageing at 140° F (AA-140F)*: the specimen is aged in a highly alkaline wet environment at a temperature of 140° F.

6.

Specimens belonging to triplets AA-RT, AA-104F, and AA-140F. The beam specimen is half immersed in a water (H_20) solution containing NaCl. The presence of NaCl is meant to reproduce the seawater environment. An alkaline solution would have been the worst situation for the GFRP but would have not been representative of a real field application. A water-heater will be used for specimens belonging to triplets AA-104F, and AA-140F

Specimen triplets	**Ageing duration and type**
12mLAB	12 months in LAB
6mNA	6 months under natural ageing and 6 months in LAB
12mNA	12 months under natural ageing
3mAA-RT	3 months under accelerated ageing at room temperature and 9 months in LAB
3mAA-104F	3 months under accelerated ageing at 104° F and 9 months in LAB
3mAA-160F	3 months under accelerated ageing at 140° F and 9 months in LAB
6mAA-RT	6 months under accelerated ageing at room temperature and 6 months in LAB
6mAA-104F	6 months under accelerated ageing at 104° F and 6 months in LAB
6mAA-160F	6 months under accelerated ageing at 140° F and 6 months in LAB
9mAA-RT	9 months under accelerated ageing at room temperature and 3 months in LAB
9mAA-104F	9 months under accelerated ageing at 104° F and 3 months in LAB

9mAA-160F	9 months under accelerated ageing at 160° F and 3 months in LAB
12mAA-RT	12 months under accelerated ageing at room temperature
12mAA-104F	12 months under accelerated ageing at 104° F
12mAA-160F	12 months under accelerated ageing at 140° F
24mNA	12 months under natural ageing
36mNA	36 months under natural ageing
48mNA	48 months under natural ageing
60mNA	60 months under natural ageing

Table 6.1: specimen ageing treatments

At the end of the 12 month ageing treatment, each beam specimen will be subjected to a four-point bending test to evaluate its bending moment at failure.

6.1.4 Deliverables

The most important deliverable of the research is the definition of the long-term properties of the material (GFRP) and of the structural element (GFRP-RC seawall).

The ratio between the average bending moment at failure, M_{aged}, for each specimen triplet will be compared with the average bending moment at failure of specimens belonging to triplet LAB, M_{bench}, and plotted versus time (expressed in hours) on a log-log plot.

Figure 6.3 explains how the data will be processed, using as an example the case of the specimens belonging to triplets 3m AA-140F, 6m AA-140F, 9m AA-140F, and 12m AA-140F. Each point shown on the plot is the ratio M_{aged} / M_{bench} at 3, 6, 9, and 12 months, respectively. The dashed line is the trend line identified by the four points. The trend line allows to extrapolate the value of the ratio M_{aged} / M_{bench} at 100 years.

The extrapolation of the bending moment at failure at 100 years for beam specimens subjected to different ageing treatments will lead to the definition of new design concepts for the design of long-lasting RC seawalls.
Data from beam specimens belonging to triplets 24m NA, 36m NA, 48m NA, and 60mNA will be ultimately used to validate the new proposed design concepts (36).

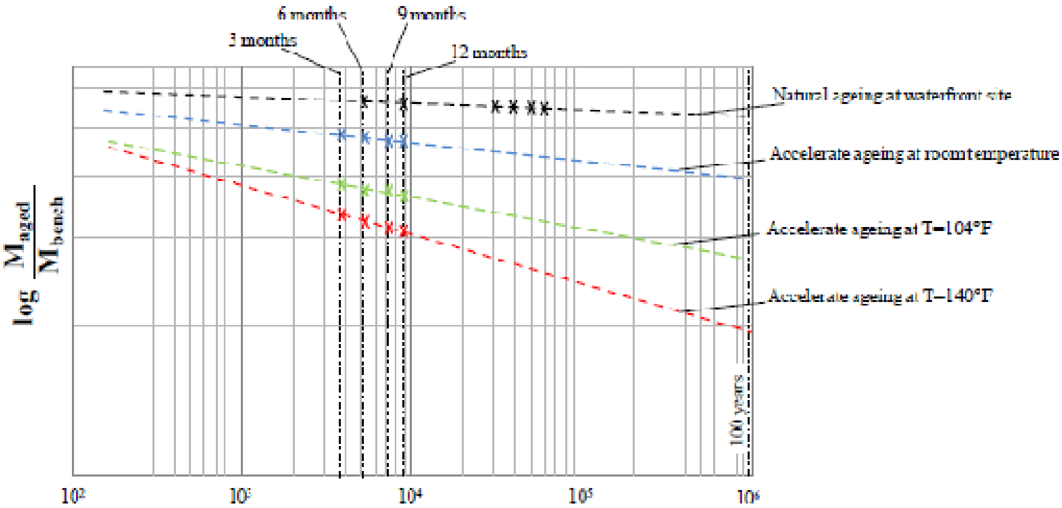


Figure 6.3: expected outcome

6.2 Casting of prototype concrete seawalls

6.2.1 GFRP SIP panels

Four prototype concrete seawalls were cast GFRP SIP panels (Figure 6.4 and Figure 6.5).
The design of the GFRP-RC prototype is based on an existing design of a steel RC seawall. A typical steel RC seawall is made of RC panels 10 ft tall, 6 ft wide, and 0.5 ft thick. The thickness of the prototype is 5.5 in., while the height is reduced to 5 ft because the area interested to the tidal action and subjected the worst environmental conditions is estimated to be 4÷6 ft.

In the Figure 6.6, Figure 6.8 and Figure 6.8 is possible to see the internal reinforcement system, whose components are:

- $\mathrm{I}-bar$ running continuously in the direction perpendicular to ground;
- Cross rods running through pre-drilled holes spaced at 100 mm on-center in the $\mathrm{I}-bar$ web in the direction perpendicular to the principal reinforcement;
- Vertical connectors that space the layers 100 mm apart;
- Channel as outer edge of the seawall.

$\mathrm{I}-bars$ are the main load-carrying, and the cross-rods provide shrinkage and temperature reinforcement and constrain the core concrete to ensure load transfer into the$\mathrm{I}-bar$

In Figure 6.9 is showed the drawing the machine shop generated by the manifacturer.

Figure 6.4: GFRP SIP panels

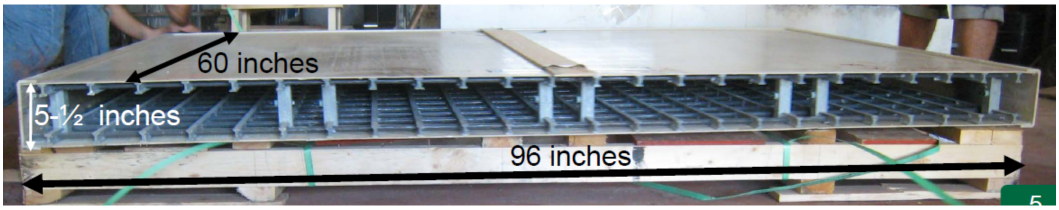


Figure 6.5: dimensions of prototype

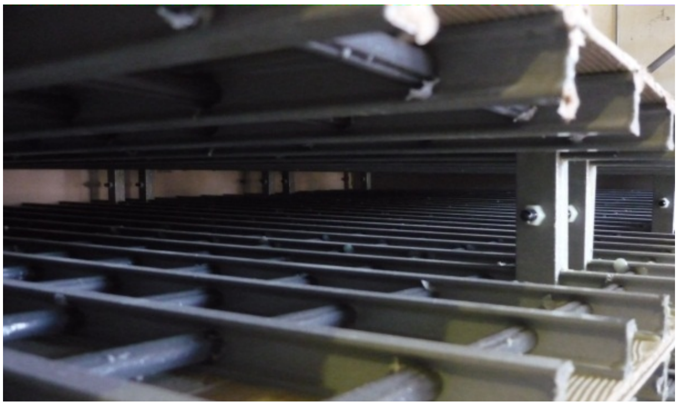


Figure 6.6: internal reinforcement of the seawall system

Figure 6.7: comparison between the dimension of the panel’s thickness and the maximum size of the aggregate

Figure 6.8: detail about C-channel and shear connectors

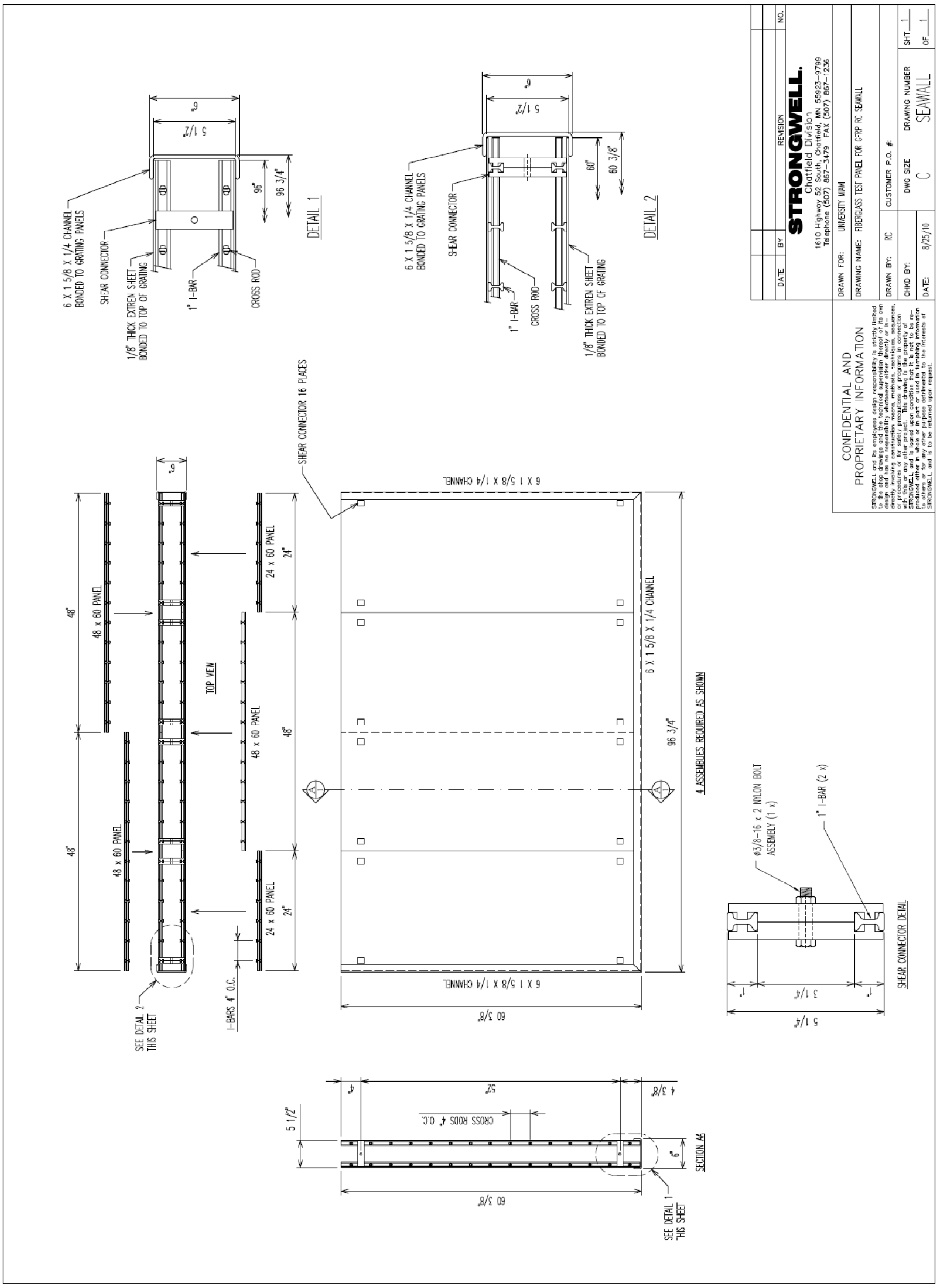


Figure 6.9: drawing of the GFRP sip panels gave from Strongwell company

6.2.2 Steps of the casting

- Bracing of the SIP panels (Figure 6.10 and Figure 6.11)

Figure 6.10: lateral view of the bracing

Figure 6.11: the top view of the bracing

- Casting of the seawalls.

The SIP panels are filled in thirds of their heights.

Concrete was vibrated during each phase (Figure 6.12)

The finishing of the panel has been done by hand (Figure 6.13).

Figure 6.12: casting of the seawalls

Figure 6.13: finishing of the casting

6.2.2.1 *Characteristics of concrete used*

A 5000 psi mix design was used.

The expected slump was 5 in ± 1 in.

Mix features are shown below:

- Cement 755 lbs/c.y.

- Rock #89 (course) 850 lbs/c.y
- Sand (fine) 1,777 lbs/c.y.
- Water 41 gls/c.y.
- Water reducer (WRDA 64) 30 oz/c.y.
- Air entrained (DAREX AEA) 2 oz/c.y.
- Superplasticizer 2 oz/c.y.

WRDA 64 is a polymer based aqueous solution of complex organic compounds. It is a ready-to-use low viscosity liquid which is factory pre-mixed in exact proportions to minimize handling, eliminate mistakes and guesswork. WRDA 64 produces a concrete with lower water content (typically 8 to 10% reduction), greater plasticity and higher strength.

Darex AEA admixture is an aqueous solution of complex mixture of organic acid salts. It is specially formulated for the use as an air-entraining admixture for concrete and is manufactured under rigid conrol provides uniform, predictable performance. Darex AEA is supplied ready-to-use and does not require pre-mixing with water. Darex AEA imparts workability to the mix, it is particularly effective with slag, lightweight, or manufactured aggregates which tend to produce harsh concrete. It also makes possible the use of natural sand deficient in fines. Air is entrained by the development of a semi-microscopic bubble system, introduced into the mix by agitation

For this kind of design mix concrete the performances are:

- W/C ratio: 0.45
- Unit weight 137.9 lb/cu.ft.

The concrete is designed to work in any structural member that requires the above mentioned strength at 28 days or by spec’s. the concrete can be placed by bucket, chute or specified pump hose diameter.

The following quality control tests were performed:

1- Standard Test Method for slump of Hydraulic-Cement Concrete (ASTM C134/C143M-10) – Section 6.2.2.2;
2- Standard Test Method for Density (Unit Weight) and Air Content (Gravimetric) of Concrete (ASTM C138/C138M-10a) – Section 6.2.2.
3- Standard Test Method for Density (AASTM C138/C138M-10a) – Section 6.2.2.
4- Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens (ASTM C39/C39M-09a) – Section 6.2.2.;

6.2.2.2 *Standard Test Method for slump of Hydraulic-Cement Concrete (ASTM C134/C143M-10)*

The following text is copied viability from the ASTM Standards.
This test method covers determination of slump of hydraulic-cement concrete.
A sample of freshly mixed concrete is placed and compacted by rodding in a mold shaped as the frustum of cone. The mold is raised, and the concrete allowed to subside. The vertical distance between the original and displaced position of the center of the top surface of concrete is measured and reported as the slump of the concrete.

- Significance and use

This method is intended to provide the user with a procedure to determine slump of plastic hydraulic-cement concrete.
This test method was originally developed to provide a technique to monitor the consistency of unhardened concrete. Under laboratory conditions, with strict control of all concrete materials, the slump is originally found to increase proportionally with the water content of a given concrete mixture. Under field conditions, however, such a strength relationship is not clearly and consistently

shown. Care should be taken in relating slump results obtained under field conditions to strength.
We used in the concrete mix design a course of 3/8 in, so this type of test is applicable.

- Apparatus

Mold – the test specimen shall be formed in a mold made of metal not readily attacked by the cement paste. The metal shall not be thinner than 0.060 in. (1.5 mm) and if formed by spinning process, there shall be no point on the mold at which the thickness is less than 0.045 in. (1.15 mm).
The mold shall be in the form of the lateral surface of the frustum of a cone with the base 8 in. (200mm) in diameter, the top of 4 in. (100mm) in diameter, and the height of 12 in. (300mm). individual diameters and heights shall be within 1/8 in. (3 mm) of the prescribed dimensions. The base and the top shall be open and parallel to cach other and at right angles to the axis of the cone. The mold shall be provided with foot pieces and handles.
The mold shall be constructed without a seam. The interior of the mold shall be relatively smooth and free from projections.
Tamping rod – a round, straight steel rod, with 5/8 in. (16 mm) ± 1/16 in. (2 mm) diameter. The length of the tamping rod shall be at least 4 in. (100 mm) greater than the depth of the mold in which rodding is being performed, but not greater than 24 in. (600 mm) in overall length. The length tolerance is for the tamping rod shall be ± 1/8 in. (4 mm).
Measuring device – a ruler, metal roll-up measuring tape, or similar rigid or semi-rigid length measuring instrument marked in increments of ¼ 1n. (5 mm) or smaller.
Scoop – of a size large enough so each amount of concrete obtained from the sampling receptacle is representative and small enough so it is not spilled during placement in the mold.

- Procedure

Dampen the mold and place it on a rigid, flat, level moist, nonabsorbent surface, free of vibration, and that is large enough to contain all of the slumped concrete. It shall be held firmly in place during filling and perimeter cleaning by the operator standing on the two foot pieces. From the sample of concrete immediately fill the mold in three layers, each approximately one third the volume of the mold. Place the concrete in the mold using the scoop (Figure 6.14).

Move the scoop around the perimeter of the mold opening to ensure an even distribution on the concrete with minimal segregation.

Rod each layers 25 times uniformly over the cross section with the rounded end of the rod (Figure 6.15). For the bottom layer, this will necessitate inclining the rod slightly and marking approximately half of the strokes near the perimeter, and then progressive vertical strokes spirally toward the center. Rod the bottom layer throughout its depth. For each upper layer, allow the rod to penetrate through the layer being rodded and into the layer below approximately 1 in. (25 mm).

In filling and rodding the top, heap the concrete above the mold before rodding is started. If the rodding operation results in subsidence of the concrete below the top edge of the mold, add additional concrete to keep an excess of concrete above the top of the mold at all times. After the top layer has been rodded, strike off the surface of the concrete by means of a screeding and rolling motion of the tamping rod (Figure 6.16).

Continue to hold the mold down firmly and remove concrete from the area surrounding the base of the mold to preclude interference with the movement of slumping concrete.

Remove the mold immediately from the concrete by raising it carefully in a vertical direction.

Immediately measure the slump by determining the vertical difference between the top of the mold and the displacement original center of the top surface of the specimen (Figure 6.17).

If a decided falling away or shearing off of concrete from one side or portion of the mass occurs, disregard the test and make a new test on another portion of the sample (37)

Figure 6.14: the scoop and the sample of concrete

Figure 6.15: the rodding of the first layer

Figure 6.16: the mold is full

Figure 6.17: measuring the slump

6.2.2.3 *Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method (ASTM C231/C231M-10)*

The following text is copied viability from the ASTM Standards.
This test method covers determination of the air content of freshly mixed concrete from observation of the change in volume of concrete with a change in pressure.
This method is intended for the use with concretes and mortars made with relatively dense aggregates for which the aggregate correction factor (G) was be determined by the Supermix Company, and equal to 0.5. It is not applicable to concretes made with lightweight aggregates, air-cooled blast-furnace slag, or aggregates of high porosity. This method is also not applicable to non-plastic concrete such as in commonly used in the manufacture of pipe and concrete masonry units.

- Significance and Use

This method covers the determination of the air content of freshly mixed concrete. The test determines the air content of freshly mixed concrete exclusive of any air that may exist inside voids within aggregate particles.

- Apparatus

Air Meters - there are available satisfactory apparatus of two basic optional designs employing the principle of Boyle’s law. For purposes of reference herein these are designed Meter Type A and Meter Type B.
It was used the Meter Type B (Figure 6.19), an air meter consisting of a measuring bowl and cover assembly conforming to the requirements, which will be listed at following.
The optional principle of this meter consists of equalizing a known volume of air at known pressure in a sealed air chamber with the unknown volume of air in the

concrete sample, the dial on the pressure gauge being calibrated in terms of percent air for observed pressure at which equalization takes place. Working pressures of 50 to 205 kPa [7.5 to 30 psi] have been used satisfactorily.

The measuring bowl shall be essentially cylindrical in shape, made of hard material not readily attacked by the cement paste, having a minimum diameter equal to 0.75 to 1.25 times the height, and a capacity of at least 6.0 L [0.20 ft^3]. It shall be flanged or otherwise constructed to provide for a pressure tight fit between measuring bowl cover assembly. The interior surface of the measuring bowl and surfaces of rims, flanges, and other component fitted parts shall be machined smooth. The measuring bowl and cover assembly shall be sufficiently rigid to limit the expansion factor, D, of the apparatus assembly to not more than 0.1% of air content on the indicator scale under normal operating pressure.

The cover assembly shall be made of hard material not readily attacked by cement paste. It shall be flanged or otherwise constructed to provide for a pressure –tight fit between measuring bowl and cover assembly and shall have machined smooth interior surfaces countered to provide an air space above the level of the top of the measuring bowl. The cover shall be sufficiently rigid to limit the expansion factor of the apparatus assembly.

The cover shall be fitted with a means of direct reading of the air content.

The cover assembly shall be fitted with air valves, air bleeder valves, and petcocks for bleeding off or trough which water may be introduced as necessary for particular meter design. Suitable means for clamping the cover to the measuring bowl shall be provided to make a pressure-tight seal without entrapping air at the joint between the flanges of the cover and measuring bowl. A suitable hand pomp shall be provided with the cover either as an attachment or as an accessory (Figure 6.18).

Calibration Vessel- a measure having an internal volume equal to a percent of the volume of the measuring bowl corresponding to the approximate percent of air in the concrete to be tested.

Spray Tube– a brass tube of appropriate diameter, which may be an integral part of the cover assembly, or which may be provided separately. It shall be so constructed that when water is added to the container, it is sprayed to the walls of the cover in such a manner as to down the sides causing a minimum of disturbance to the concrete.

Trowel– a standard brick mason's trowel.

Tamping Rod– to see Section 6.2.2.2.

Scoop– to see Section 6.2.2.2.

Mallet

Strike-Off bar– a flat straight bar of steel at least 3 mm thick.

Figure 6.18: components of Cover Assembly

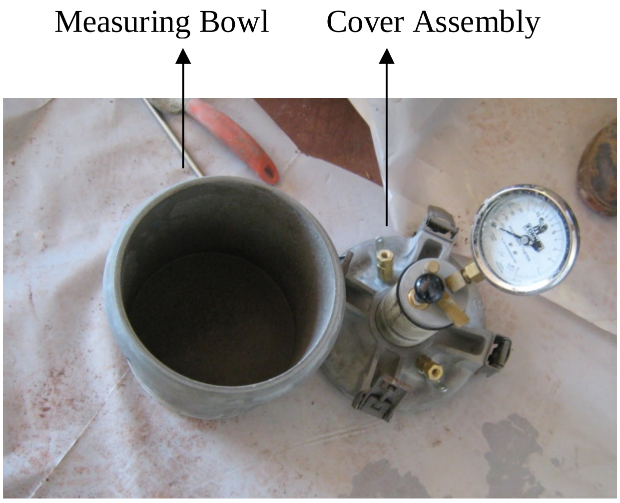


Figure 6.19: Air Meter Type B

- Procedure

Dampen the interior of the measuring bowl and place it on flat, level firm surface. Using the scoop place the concrete in the measuring bowl in three layers of approximately equal volume. Rod each layer 25 times uniformly over the cross section with the rounded end of the rod. Rod the bottom layer throughout its depth. In rodding this layer, used care not to damage the bottom of the measuring bowl. For each layer, allow the rod to penetrate through the layer being rodded and into the layer below approximately 25 mm.[1 in]. after each layer is rodded, tap the sides of the measuring bowl smartly 10 to 15 times with the mallet to close any voids left by the tamping rod and release any large bubbles of air that may have been trapped. Add the final layer of concrete in a manner to avoid axcessive overfilling.

After consolidation of concrete, strike-off the top surface by sliding the strike-off bar across the top flange

Thoroughly clean the flanges or rims the measuring bowl and the cover assembly so that when the cover is clamped in place a pressure-tight seal will be obtained. Assemble the apparatus. Close the main air valve between the air chamber and measuring bowl and open both petcocks on the holes through the cover. Using a rubber syringe, inject water through one petcock until water emerges from the opposite petcock. Jar the meter gently until all air is expelled from this same petcock.

Close the air bleeder valve on the air chamber and pump air into the air chamber until the gauge hand is on the initial pressure line. Allow a few seconds for the compressed air to cool to normal temperature. Stabilize the gauge hand at the initial pressure line by pumping or bleeding-off air as necessary, tapping the gauge lightly by hand.

Close both petcocks on the holes through the cover. Open the main air valve between the air chamber and the measuring bowl. Tap the sides of the measuring bowl smartly with the mallet relive local restraints. Lightly tap the pressure gauge by hand to stabilize the gauge hand. Read the percentage of air on the dial of the pressure gauge (Figure 6.20). Release the main air valve. Failure to close the main air valve before releasing the pressure from either the container or the air chamber will result in water being drawn into the air chamber, thus introducing error in subsequent measurements (39).

Figure 6.20: measuring of Air Content refers to the sample of concrete used for the casting of the panel

- Calculation

Calculate the air content of the concrete in the measuring bowl as shown in the Eq.

$$A_s = A_l - G \tag{6.1}$$

Where:

$A_s = air\ content\ of\ the\ sample\ tested, \%$

$A_l = apparent\ air\ content\ of\ the\ sample\ tested, \%$

$G = aggregate\ correction\ factor, \%$

As shown in the Figure 6.20, A_l, refers to the concrete used for the casting of the Panel Wall, is 3.8%.

$$\therefore\ A_s = 3.8 - 0.5 = 3.3\%$$

6.2.2.4 *Standard Test Method for Density (ASTM C138/C138M-10a)*

The following text is copied viability from the ASTM Standards.
This test method covers determination of the density of freshly mixed concrete

- Apparatus

Balance- a balance or scale accurate to 0.1 lb [45 g] or to within 0.3% of the test load, whichever is greater, at any point within the range of use. The range of use shall be considered to extend from the mass of the measure empty to the measure of the mass plus its contents at 160 lb/ft^3 [2600 kg/m^3].
Tamping rod- see Section 6.2.2.2
Measure- a cylindrical container made of a suitable material, not attacked by the cement paste.

Strake-Off Plate- a flat rectangular metal plate least ¼ in. [6 mm]. thick and width at least 2 in. [50 mm].

Mallet

Scoop- to see Section 6.2.2.2.

- Procedure

Place the concrete in the measure using the scoop. Move the scoop around the perimeter of the measure opening to ensure an even distribution of the concrete with minimal segregation.

Place the concrete in the measure in three layers of approximately equal volume. Rod each layer with 25 strokes of tamping.rod each layer uniformly over the cross section with the rounded end of the rod using the required number of strokes. Rod the bottom layer throughout its depth. In roddding this layer, use care not to damage the bottom of the measure. For each layer, allow the rod to penetrate through the layer being rodded and into the layer below approximately 25 mm.[1 in]. after each layer is rodded, tap the sides of the measuring bowl smartly 10 to 15 times with the mallet to close any voids left by the tamping rod and release any large bubbles of air that may have been trapped. Add the final layer of concrete in a manner to avoid excess overfilling.

After strike-off, clean all excess concrete from the exterior of the measure and determine the mass of the concrete (40).

- Calculation

Calculate the net mass of the concrete in pounds or in kilograms by subtracting the mass of the measure, M_m, from the mass of the measure, filled with concrete, M_c (Figure 6.21). Calculate the density, D, ft^3 or yd^3, by dividing the net mass of concrete by the volume of the measure, V_m, as shown in the Equation 6.2:

$$D = (M_c - M_m)/V_m \tag{6.2}$$

Figure 6.21: measuring of M_m and M_c

$M_m = 7.52\ lb$

$M_c = 42.44\ lb$

$V_m = 0.25\ ft^3$

$$\therefore D = \frac{42.44 - 7.52}{0.25} = 140\ pcf\ [2240\ kg/m^3]$$

6.2.2.5 *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens (ASTM C39/C39M-09a)*

The following text is copied viability from the ASTM Standards.

This test method covers determination of compressive strength of cylindrical concrete specimens such as molded cylinders and drilled cores. It is limited to concrete having a density in excess of 800 Kg/m^3.

The concrete used for the casting of the seawall has a density of 140 lb/ft^3 that means 2,240 Kg/m^3 (Section 6.2.2.4).
This test method consists of applying a compressive axial load to molded cylinders at a rate which is within a prescribed range until failure occurs. The compressive strength of the specimen is calculated by diving the maximum load attained during the test by the cross-sectional area of the specimen.

- Significance and use

Care must be exercised in the interpretation of the significance of compressive strength determination by this test method since strength is not a fundamental or intrinsic property of concrete made from given materials. Values obtained will depend on the size and shape of the specimen, batching, mixing, procedures, the method of sampling, molding, and fabrication and the age, temperature, and moisture conditions during curing.
The results of this test method are used as a basis for quality control of concrete proportioning, mixing, and placing operations; determination of compliance with specifications; control for evaluating effectiveness of admixture; and similar uses.

- Apparatus

Testing Machine – (Figure 6.22) the testing machine shall be of a type having sufficient capacity and capable of providing the rates of load prescribed.
The Design, Accuracy and Load Indication required for the testing machine are listed on the 5.1.2, 5.1.3 and 5.3 sections on the C39/C39M-09a.

Figure 6.22: Satec System machine used for the compressive test

Figure 6.23: a detail about the Control Screen

Figure 6.24: detail about Load Frame

- Specimen

During the casting 75 cylindrical specimens were poured (Figure 6.25 and Figure 6.26) which will be subjected to the same type of ageing of the beams and, also, they will be tested at same time.

The specimen is filled in third of the heights and each layer is compacted through 25 shots uniformly over the cross section.

Figure 6.25: casting of the specimens

Figure 6.26: finish of the specimens

Specimens shall not be tested if any individual diameter of cylinder differs from any other diameter of the sane cylinder by more than 2%.prior to testing, neither end of test specimens shall depart from perpendicularity to the axis by more than 0.5° (approximately equivalent to 1 mm in 100 mm [0.12 in. in 12 in.]).

The ends of compression test specimens that are not plane within 0.050 mm [0.002 in.] shall be sawed or ground to meet tolerance.

The diameter used for calculating the cross-sectional area of the test specimen shall be determined to the nearest 0.25 mm [0.01 in.] by averaging two diameters measured at right angles to each other at about mid height of the specimen.
The number of individual cylinders measured for determination of average diameter is not prohibited from being reduced to one for each ten specimens or three specimens per day, if all cylinders are known to have been made from a single lot of reusable or single-use molds which constantly produce specimens with average diameters within a range of 0.5 mm [0.02 in.].
The dimensions of the molds used for the casting of the 75 specimens are: 8 inches of height and 4 inches of diameter.
The average diameter determined measuring the diameter of one to each ten specimens is 4 in.

- Procedure

Place the plain (lower) bearing block, with its hardened face up, on the table or platen of the testing machine directly under the spherically seated (upper) bearing block. Wipe clean the bearing faces of the upper and lower bearing blocks and test specimen and place the test specimen on the lower bearing block. Carefully align the axis of the specimen with the center of thrust of spherically seated block.
Prior to testing the specimen, verify that the load indicator is set to zero.
Apply the load continuously and without shock.
The load shall be applied at a rate of movement corresponding to a stress rate on the specimen of 0.25 ± 0.05 MPa/s [35 ± 7 psi/s].
Apply the compressive load until the load indicator shows that the load is decreasing steadily and the specimen display a well-defined fracture pattern (Types 1 to 4 in Figure 6.27). When tasting with unbounded caps, a corner fracture similar to a Type 5 or 6 patterns shown in Figure 6.27 may occur before the ultimate capacity of the specimen has been attained. Continue compressing the specimen until the user is certain that the ultimate capacity has been attained (38).

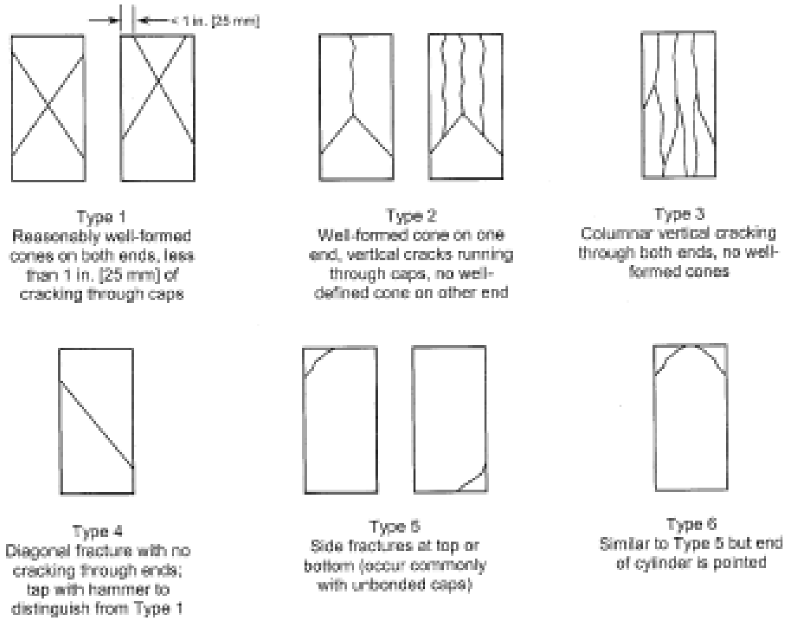


Figure 6.27: schematic of Typical Patterns (37)

For the compressive test at 28 days after the casting of the wall panels, has been used a silicon cap during the compression, and the specimens showed the 3 Type of Rupture (Figure 6.27).

In the following figures are showed the ruptures of the four samples tested on 7 December 2010 (Figure 6.28, Figure 6.29, Figure 6.30 and Figure 6.31)

Figure 6.28: specimen number 1

Figure 6.29: specimen number 2

Figure 6.30: specimen number 3

Figure 6.31: specimen number 4

- Calculation

Calculate the compressive strength of the specimen by dividing the maximum load carried by the specimen during the test by the average cross-sectional area. Have been tested four specimens at 28 days after the casting.

The average cross section area for the four specimens used in the test is 12.56 in^2. In the Table 6.2 there are reassumed the results obtained from the test at 28 days.

TEST AT 28 DAYS (7 DEC. 2010): TYPE 3 FAILURE				
Specimens	Weight (g)	Load (lb)	Time(min)	σ (psi)
1	3,569	64,200	2:40	5,111
2	3,541	65,160	2:43	5,180
3	3,546	65,430	2:35	5,209
4	3,581	67,600	2:20	5,382
Average		…………………………………………		**5223**…..
Standard Deviation				**198.1**

Table 6.2: results of the test at 28 days from the casting

6.2.3 Cutting of the panels into beams specimen

The four wall panels have been cut into 96 beams specimen (Figure 6.32 and Figure 6.33). From these specimens are selected 57 beams to put under environment ageing.

Figure 6.32: cutting of the Wall Panels

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Figure 6.33: cutting of the Wall Panel

6.2.4 Preliminary results

6.2.4.1 Durability tests

The 57 beam specimens are divided in nineteen triplets and subjected to different ageing treatments over a period of time of 12 months as shown in Table 6.1 (Section 6.1.3).

Five types of ageing treatments are considered.

1. *Preserved ageing (LAB)*: the specimen is aged in an indoor environment at room temperature. This condition is referred to as LAB.

2. *Natural ageing (NA)*: the specimen is aged at a waterfront site subjected to tidal action.

3. *Accelerated ageing at room temperature (AA-RT)*: the specimen is aged in a highly alkaline wet environment at room temperature.

4. *Accelerated ageing at 104º F (AA-104F)*: the specimen is aged in a highly alkaline wet environment at a temperature of 104º F.

5. *Accelerated ageing at 140º F (AA-140F)*: the specimen is aged in a highly alkaline wet environment at a temperature of 140º F.

Specimens belonging to triplets AA-RT, AA-104F, and AA-140F. The beam specimens are immersed in a three-foot-deep seawater (H_20) solution containing NaCl. An alkaline solution would have been the worst situation for the GFRP but would have not been representative of a real field application. A water-heater, collocated in the containers shown in Figure 6.34, will be used for specimens belonging to triplets AA-104F, and AA-140F (Figure 6.35).

Figure 6.34: containers used for the environment ageing.

Figure 6.35: heaters used for the environment ageins

6.2.4.2 *Flexural tests*

The ageing treatments start after the four-point bending test at ‘Time 0’.

The four-point bending test consists into the application of monotonic load as shown in the Figure 6.36.

Figure 6.36: the load frame for the bending test.

During the test, the displacement in the middle of the beam and the load is recorded through the LVDT instrumentation and load cell (Figure 6.37).

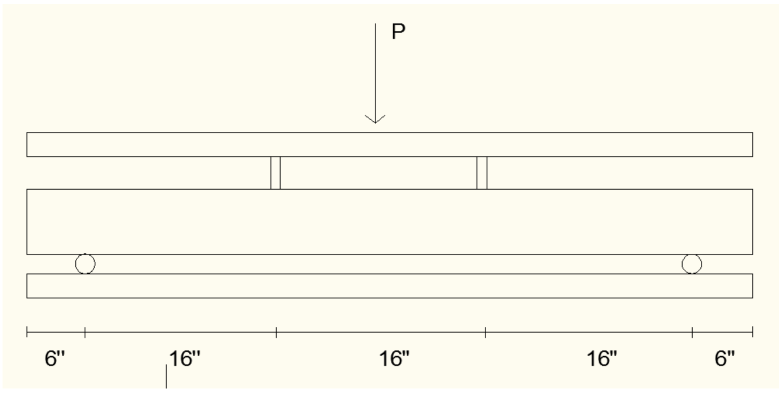

Figure 6.37: schematization of the bending test

Three specimens were tested, and they show shear failure (Figure 6.38 and Figure 6.39).

Figure 6.38: shear collapse

Figure 6.39: shear collapse

The experimental results attest the strength capacity trusted from the characteristic qualities.

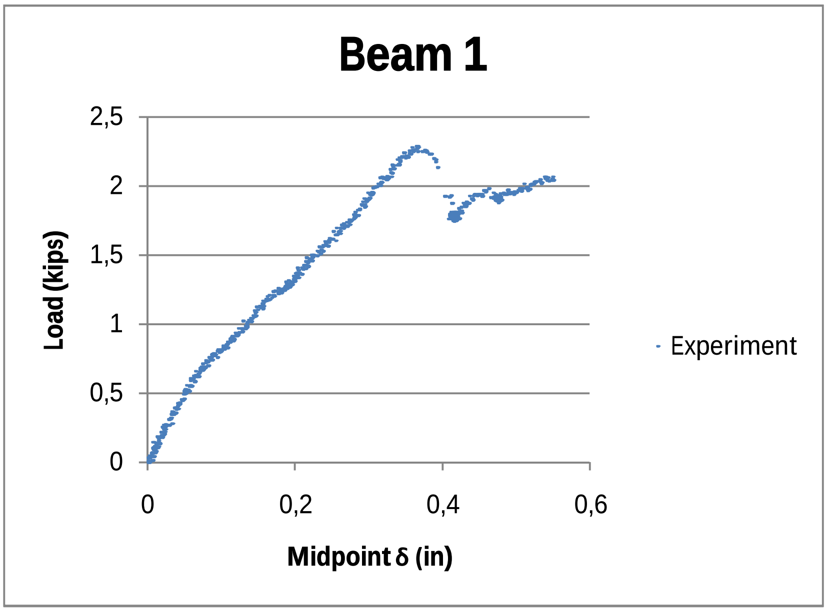

Figure 6.40: result of bending test referred to the Specimen 1

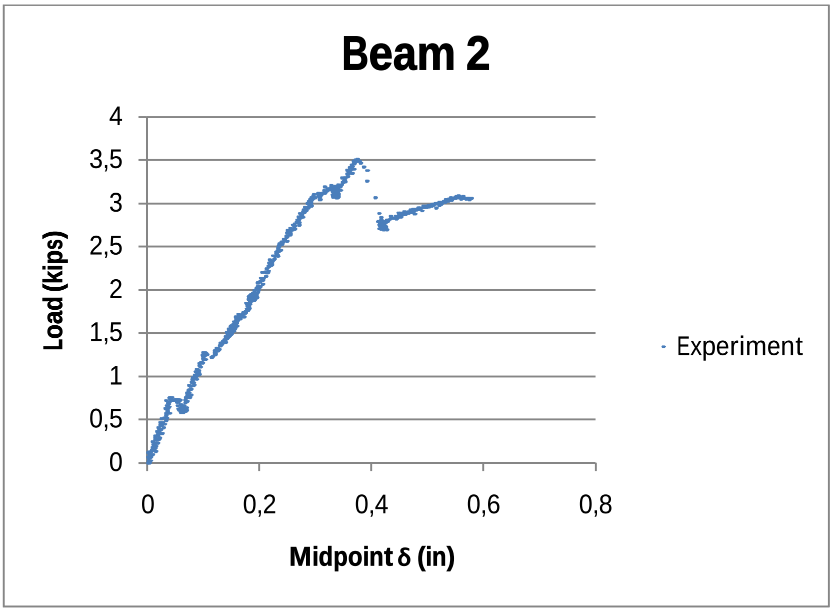

Figure 6.41: result of bending test referred to the Specimen 2

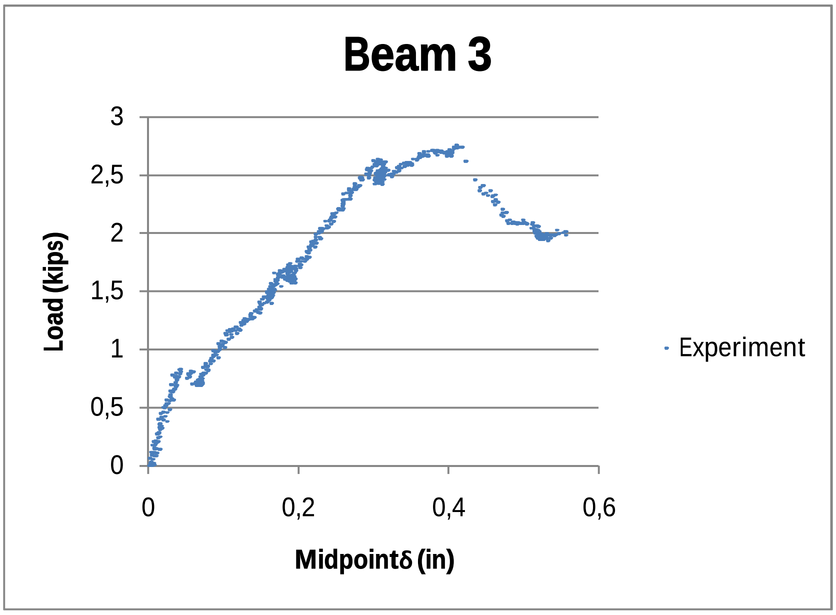

Figure 6.42: result of bending test referred to the Specimen 3

CONCLUSION:

The study, reported in this thesis, represents the beginning of a long experimental program about the durability of RC Seawall with GFRP bars, and also the beginning of a structural project that has the purpose to realize an Seawall System completely in concrete reinforced by FRP reinforcement.

For the first purpose, that is the part of experimental program about the durability of seawall structure reinforced by GFRP bars, the objectives reached are:

- the creation of the experimental project, with timing and instrumentation necessary (Section 6.1.1; Section 6.1.2; Section 6.1.3; Section 6.1.4);
- the realization of beams specimens from the panel wall prototype (Section 6.2.4);
- the four-point bending tests at ‘Time 0’ and consequently the starting of ageing treatments (Section 6.2.5).

Four prototype seawall panels were constructed. From each panel were product 14 beams specimens, which will be divided in nineteen triplets and subjected to different ageing treatments over a period of 12 months.

Five types of ageing treatments are considered: Preserved Ageing; Natural Ageing; Accelerated Ageing at Room Temperature; Accelerated Ageing at 104° F and Accelerated Ageing at 140° F.

At the end of 12 months, each beam specimen will be subjected to a four-point bending test to evaluate its bending moment at failure

The extrapolation of the bending moment at failure for beam specimens subjected to different ageing treatments will lead to the definition of new design concepts for the design of long-lasting RC seawalls (C_E, Environmental Factor. ACI 440.1R-06).

For the second purpose, that is the creation of a Seawall System completely in concrete reinforced by GFRP reinforcement, the objectives reached are:

- the knowledge about the types of RC Seawall used in Florida, particularly in Miami (Section3.1);
- the design and creation of the prototype of Panel Wall, that represents a component of the RC seawall system (Section 6.2);
- the realization of the Software to design the Panel Wall in base to the performance of the materials that the Strongwell company provides (Section 4);

GFRP RC seawalls represent an economically competitive alternative to conventional solutions, such as steel RC or timber seawalls, because of its resistance to corrosion, high strength to weight ratio and excellent fatigue performance.

Through field studies is been possible individuate three principal systems of seawall construction:

- Vinyl/FRP or Steel Sheet pile (anchored and not);
- Anchored Panel wall and Vertical pile;
- Panel wall, King pile and Battering pile.

The steel sheet pile is the stronger system, with it is possible reach greater depths, but the cost of this material is very high. Also this system offers a service life of 25 years at most due to corrosion, that represents also an additional costs for the maintenance.

The system formed by reinforced concrete panels and piles is more common because this material is the locally available aggregate and it also it offers an service life of 50 years so it represents the most economic choice. On the other hand the disadvantage of the RC system is the difficulty to penetrate hard layers, and sometimes, when the soil is not sand, is required an excavation to place the pile at the desired depth.

The realization of Seawall System completely in RC wit GFRP reinforcing represents a new challenge that research team together the Strongwell Company want convert in reality. Right now, the first part of the Seawall System, that is the Wall Panel, it was realized, the next steps will regard the creation of the pile in RC with GFRP.

At the end, will be provided a guide to assist the design and the construction of concrete seawalls reinforced with GFRP grating with SIP forms (Strongwell Company).

RINGRAZIAMENTI

Eccomi a 25 anni, con il mio bagaglio di esperienze, piacevoli e non, ma che mi hanno resa la persona che sono oggi. Sicuramente cambiata, diversa, con le prime rughette, cresciuta e spaventata, come il primo giorno delle scuole elementari, solo che questa volta non ci sarà la mamma ad aiutarti a salire le scale portando il tuo zaino tra le mani e che ti lascia scomparire dietro il grande portone, la cui maniglia sembrava cosi alta ed irraggiungibile.
Ringrazio tutti per essere qui con me, oggi, perché avete capito l'importanza di questo traguardo.

Ringrazio il Professore Mazzotti, che con il suo "Cara Ivana", anche se dall'altra parte del mondo, non mi ha mai fatto sentire abbandonata a me stessa.

Ringrazio il Professore Nanni perché mi ha dato l'opportunità di aprire i miei orizzonti e di vivere un'esperienza indimenticabile.

Ringrazio i miei genitori.
Cosa dire, mi hanno messa al mondo e mi hanno sostenuto ed accompagnato per tutta la vita.

RINGRAZIAMENTI

Li ho odiati, amati, ammirati, criticati, visti con gli occhi da bambina e con quelli da donna e posso sicuramente affermare che loro sono la mia più grande fortuna.
La Mamma, lei mi ha reso cosi speciale o “pazza” come dicono in molti!
Non so se questa caratteristica sia propriamente un bene, ma sono certa che è una parte importante della mia personalità, che mi contraddistingue dagli altri e di sicuro per questo la ringrazio.
Mio Padre, c’è chi lo teme, ma solo perché non lo conosce come me.
Io, il suo “maschietto mancato”, la figliola a cui proprio non sa dire no, per la quale soffre in silenzio (la mamma non direbbe poi così tanto in silenzio!) ed è costantemente preoccupato.
Grazie per avermi sempre spronato a fare meglio, destato nei momenti difficili e dato la pace quando tutto sembrava andar per il verso sbagliato.

La mia Sorellona, cosa dire, l’ esempio di vita da imitare, unico e vero legame inscindibile della mia vita, sola certezza del mio futuro.
Cosi piccola e forte, che mi ha regalato (grazie al fondamentale contributo di Emanuele) una splendida nipotina, Patrizia, tutto pepe e riccioli d’oro.

Annamaria…la mia “migliore” amica, ci sei sempre stata e per questo ti ringrazio dal più profondo del mio cuore.
Ti sosterrò e ti farò ridere nei momenti di sconforto, e se anche io sarò giù…beh allora ci deprimeremo insieme, ma la certezza è che saremo insieme!

Beh come scordare “grazia e graziella”, e si dico proprio a voi…Lisa ed Elena, mie compagnie di avventura, di dolore e di ubriacatura!!
Credo che la vera forza per andare avanti in questi anni, a volte veramente pesanti, sia derivata dalla nostra unione.
Questa amicizia è una delle cose più importanti che l’esperienza universitaria mi ha dato…quindi, occhio, non perdiamoci!!

RINGRAZIAMENTI

Ilaria, Dario, Antonio…i miei amici storici, oggi siete qui con me e questo gesto parla da sé. Sono davvero fortunata ad avere persone come voi al mio fianco, su cui sono sicura di poter contare sempre perché “cumpà amu cresciutu assieme!” e anche se la vita ci porterà a vivere distanti, non conta, saremo sempre uniti. Grazie!

Grazie a Giovanni e Matteo, due inaspettati amici trovati per caso dall’altra parte del modo, che hanno reso la permanenza all’estero meno difficile perché semplicemente erano la mia famiglia lì! Grazie, Grazie!

Ringrazio Antonio e Steven, che mi hanno aiutato nel mio lavoro di tesi, sempre molto gentili e disponibili.

Grazie alle mie coinquiline…la mia “famiglia allargata”…

Grazie ai miei Nonni!! Che mi rendono così fiera dei miei traguardi.
Grazie al Nonno Dino, l’uomo della famiglia con la lacrima facile, che al telefono lascia le frasi a metà dalla commozione, questo gesto, nonno, mi ha fatto sempre sentire molto amata, grazie!
Grazie alla Nonna Teresa, e alle sue storie che mi hanno fatto crescere credendo che oltre le nuvole ci sia qualcosa di più grande di noi!
Grazie al Nonno Cocò, il “nonno che gioca” e che costantemente prega per me…grazie!

Grazie agli zii, oggi tutti qui, e che dopo tanti anni mi rendono orgogliosa di far parte di una “vera, grande e numerosa famiglia del sud”!!
Grazie a tutte le persone che oggi sono qui per condividere con me questo momento indimenticabile.

Grazie!

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