DESIGN OF MANGROVE REHABILITATION PROJECTS ON TROPICAL COASTS

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CoMEM

DESIGN OF MANGROVE REHABILITATION PROJECTS ON TROPICAL COASTS

A Study of Possible Options and Recommendations

DELFT UNIVERSITY OF TECHNOLOGY

24 June 2015

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CoMEM Thesis

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Abstract

Mangroves have been proven to be an asset to coastlines because of their myriad advantages relating to coastal protection and stability. Unfortunately, in the past, due to economic necessities or other reasons, many of these mangrove swamps in tropical areas came under threat and were subsequently destroyed. As the awareness about the positive effects of mangroves' is becoming widespread, many places would now want to re-plant mangroves along the coast. However, such mangrove rehabilitation projects face major challenges, in terms of design, costs and implementation. The goal of such a project would be to create a conducive environment for highest mangrove growth with the lowest possible costs. But, though a case study along the eastern coast of the Indian city of Mumbai, it has been found that creation of a 'conducive' environment will involve various factors, addressing the growth requirement of the mangroves as well as protecting the planted area from perturbation due to the sea. These challenges have been tackled through a multi-alternative approach, with various possible solutions for the layout, structural design and drainage of the environment. The optimal design consisting of bamboo-piled walls and drains for such a project has then be worked out based on the overall benefit against the cost incurred. It is to be borne in mind that there is, at present, little guidance available for the design and planning of such mangrove rehabilitation projects. Therefore, a study encapsulating various aspects of the project design, including engineering analysis and cost computation has been presented as a guideline for any such projects in the future.

Acknowledgements

I cannot thank Prof. Henk Jan Verhagen enough for his immense guidance during the course of my work. His incredible patience in dealing with my questions and his availability for discussions at all times is deeply appreciated, and I am aware that his invaluable input has been the most important factor in shaping this thesis to its present form. It is he who encouraged me to pursue this topic and he ensured that I get the right help and direction each time. I started off last year with doing my multidisciplinary project under his supervision and in the many interactions we have had since, he has always inspired me to think like a true engineer.

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The two-year journey of the CoMEM programme has been an extremely enriching experience for me, in more ways than one. Not only did the programme introduce me to different academic environments but also allowed me to experience a variety of cultures which I am sure would not have been possible otherwise. For this, I thank the Erasmus Mundus programme for granting this once-in-a-lifetime opportunity.

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1.

Introduction

The widespread distribution of mangroves along wave-sheltered tropical coasts indicates that such areas across the world are favourable for mangrove rehabilitation, given the availability of suitable substratum in the form of dredged mud. This chapter has been put forward as an introduction of this proposed concept. It goes on to detail the motivations behind the concept and the objectives of this study.

The richest mangrove communities of the world are found in tropical and sub-tropical areas, i.e., between the 30°N and 30°S latitudes where the water temperature is greater than 24°C in the warmest month, where the annual rainfall exceeds 1250 mm and mountain ranges greater than 700 m high are found close to the coast. Mangroves are found practically in almost all the continents, excepting Europe and the Antarctic with the best ones found in Asia, especially in India and Bangladesh - the Sunderbans are the largest mangrove forest in the world both in size as well as biodiversity. The total area of mangroves in India is about 6,740 sq. km, which is about 7% of the world's total area of mangroves (Soonabai Pirojsha Godrej Marine Ecology Centre, n.d.).

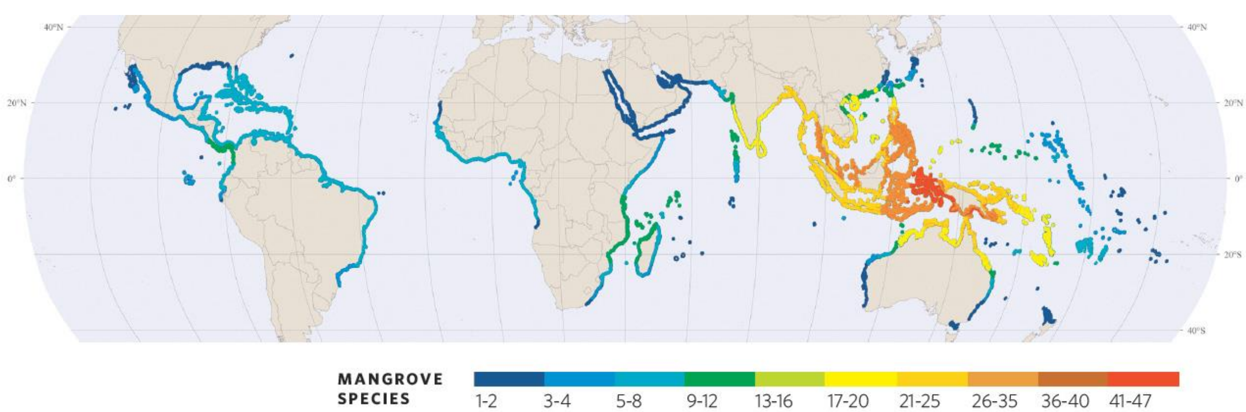

Figure 1-1 World map of the mangrove distribution zones and the number of mangrove species along each region. (US National Ocean Service, 2014)

Dredging is essential for maintenance and development of waterways and harbours. It is also necessary for navigation, remediation and flood protection. Dredging of waterways and harbours creates large volumes of dredged material. This material can be a valuable resource. Coarse material is already widely used as a resource for construction, but also soft material can be re-used in a cost-effective manner (M Ketelaars, 2013). There are several places along the tropics and sub-tropics where disposal or reuse of dredged mud is a major issue. Given the extensive coastline of India, the great number of maritime infrastructure projects, both capital and maintenance, should be no surprise. Mumbai on the north-western coast of India is a prime example. On the eastern side of the Mumbai Estuary a green field container terminal, the first terminal to be privatised, was built in the late 1980s at Nhava Sheva to relieve the congested docks on the western side and some 6.5 million-m^3 soft material were dredged (IADC, 2005).

A lot of such dredged mud goes unused. One way of making use of this mud could be as intertidal building material to plant coastal vegetation such as mangroves. This forms the basis of the intended study which is defined in the section below.

1.1. Problem Definition

The idea of the reuse of dredged fines for plantation and growth of mangroves along the coast is at the core of the study. There are several problems, however, that remain unsolved about such utility of unused dredged mud as building material for mangrove flats. Firstly, mud that is available post-dredging usually has high water content and therefore may need to be drained. The kind of mechanism that is used for such drainage will be crucial in determining the stability of such mud. It is important to mention here that stabilization of mud-flats is a preliminary process in the establishment of mangroves (Soonabai Pirojsha Godrej Marine Ecology Centre, n.d.). Secondly, such mud, when laid along the coast, may tend to recirculate offshore and hence there will be a need of a protective structure at the offshore end of the mud-laid area in order to prevent this from happening. Moreover, this structure will need to be such that it will allow (drained) water to permeate through but not allow mud to pass. This therefore presents a challenge in terms of designing the right kind of structure for such purposes as well as finding an appropriate building material for such a structure.

1.2. Motivations for the study

One of the major premises of this study is the fact that reuse of dredged mud as a building material for natural mangrove plantation in a nearby coastal area will mean local recycling of the mud. Local re-use also implies significant cost-saving on factors such as processing, transportation or discarding of unused dredged mud.

Importantly, such local re-use would open an opportunity for coastal firms to come up with a feasible project plan, which can be paid for by the relatively richer harbour companies which want to get rid of their dredged muds. It will therefore not only help solve a major disposal issue but also make the project financially possible.

Furthermore, presence of mangrove ecosystems have a positive impact on the stability of the coast, the water quality, flora and fauna and on the coastline (AdeKUS, KU Leuven, 2005). It is also important to stress here that such reuse of dredged mud would be a soft approach, a plan which is better integrated with nature, unlike hard approaches which are sometimes implemented for coastal stabilization.

1.3. Aims of the study

a. To develop an appropriate design guideline for usage of dredged fines as building material, which can further be divided into two parts:

    i. To propose an appropriate confining structure that prevents the mud from recirculating offshore

    ii. To look at a suitable way of laying the mud with the right drainage mechanism in place.

b. To evaluate the financial viability of such a project

2.

Background and Site Identification

Before proceeding with a design for mangrove restoration plans, it is important to study relevant literature pertaining to growth of mangroves and suitable host environments. The identification of an actual site, in this study the city of Mumbai along the Indian west-coast is vital so as to obtain realistic perspectives about such projects. This chapter focuses on the salient facts about mangroves essential to the study, as also on the identification and description of the study site.

2.1. Characteristics of Mud Required for Mangroves

In order to determine drainage and layout methodologies for mud, it is first essential to broadly investigate the properties of mangrove soils. When tidal inundation is superimposed on the drainage and aeration properties of mangrove soils, they are characterized by high water content, low oxygen content, high salinity and high levels of soluble sulfides. In addition, these soils are often semi-fluid and poorly consolidated (Saenger, 2002). Some major facts and challenges related to some of these characteristics have been detailed below.

2.1.1. Water content

Mangrove soils are characterized by high water content (Saenger, 2002). However, mangroves encounter problems in areas of poor drainage, infrequent tidal flooding and permanent waterlogging. Mangroves are sensitive to disruptions in freshwater run-off and ground water flow, the magnitude of the currents and the physical composition of the soil. The water circulation in the mangrove swamp controls the chemistry and biology of the swamp and the estuary and hence the growth of mangroves (Field, 2007).

2.1.2. Salinity

High salinity makes it difficult for mangrove plants to take up water for osmotic reasons and it may have a direct effect on the metabolism of plants. High salinity can cause an acute effect on damaged seedlings. Chronic effects will be seen where high soil salinity and infrequently flood soils lead to small stunted trees with extensive root systems (Field, 2007).

It is important to note here that when thinking of dredged muds, such high salinity could be attributed to insufficient drainage of seawater.

2.1.3. Composition of sand and clay (based on available examples)

Studies reveal that mangroves generally occur on fine sandy or muddy substratum. Mangrove soils are made up of sand, silt and clay in different combinations, and 'mud' actually refers to a mixture of silt and clay, both of which are rich in organic matter (detritus). What we see on the surface is topsoil, and are loosely recognised as sandy or clayey types (Peter K L Ng, 2001).

It is also observed that mangroves themselves may influence sediments by promoting siltation. Soils studied at Bhatye and Kalabadevi region (near Mumbai, India) show swampy-muddy nature at many locations of different estuaries (D D Kurlapkar, 2014). In Kalimantan (Indonesia), it is observed that the top layer of the soil is composed (0-20 cm) is composed of less than 35% sand particles, indicating that the surface is composed of mainly small but newly-sedimented particles (could be mud?) (Sukardjo, 1994).

2.1.4. Density variation with depth

Bulk density of a soil is a dynamic property that varies with the soil structural conditions. In general, it increases with profile depth, due to changes in organic matter content, porosity and compaction. The "ideal" soil would hold sufficient air and water to meet the needs of plants with enough pore space for easy root penetration, while the mineral soil particles would provide physical support and plant essential nutrients. Soil bulk density is a basic soil property influenced by some soil physical and chemical properties. When the mud layer grows in thickness, the overburden will cause a stress on the structure by which it will slowly collapse (Pravin R Chaudhari, 2013).

During this process, known as self-weight consolidation, pore water is expelled to the surface and the pressure of the pore water increases relative to the hydrostatic pressure, generating excess pore pressures. The process continues until the strength of the soil skeleton (the effective stress) is in equilibrium with the submerged weight it has to carry and the excess pore pressures are dissipated (Coastal Wiki, 2012).

In case of this study, it is more desirable to know the general variation and stages of consolidation rather than the precise depths and densities. It is also important to note that these densities will also vary depending on drainage.

At some places near Mumbai, dry as well as deep muddy swamps are recorded (D D Kurlapkar, 2014). These are observations recorded for the Bhatye and Kalabadevi studies and it must be noted that mangrove swamps present in other countries such as Indonesia and Australia may have different depths of soft soil.

2.1.5. Aeration

Mangrove soil is mildly aerobic to highly anaerobic due to waterlogging of the soil. To combat this, mangroves have developed special aerial roots in order to keep the underground roots aerated. However, these do not develop until 4-8 months after the seedlings become established so anaerobic conditions can be fatal for seedlings. This lack of oxygen is often caused by poor drainage, bringing the problem back to proper drainage of the soil (Field, 2007).

It may be noted that most concerns related to important characteristics of mangrove growth can be attributed to proper drainage of these soils in order to maintain optimal salt, water and air content.

Characteristic	Attributes that can regulate the level or magnitude of the component
Water content	Depends on drainage
Salinity	In case of dredged muds, may depend on drainage
Composition of sand and clay	Depends on local conditions
Density variation with depth	Depends on local conditions but in case of dredged mud, may depend on drainage
Aeration	Depends on roots, but also linked to drainage

Table 2-1 Soil characteristic and the attributes that need to be controlled to manage these characteristics

2.2. Mangroves growth and plantation

2.2.1. Mangrove growth trajectories

Information about the process and timeline of mangrove growth is crucial to any project that involves re-development of mangrove swamps. Unlike many other plants, the plantation process for mangroves starts not with a seed but what is called a propagule. A germinated mangrove seed is called a propagule. Such germination happens when the seed it attached to the tree. Therefore, for rehabilitation of mangroves, one needs to use these propagules and therefore it also becomes important to analyse the life of this propagule after it falls into water. In any further explanations, the terms propagule and seedling will be used interchangeably since, with respect to plantation of mangroves, the propagules indeed function as a seedling.

Once in water, these propagules travel in an unusual way. In salt water they lie horizontally and move quickly. On reaching fresher (brackish) water they turn vertically, making it easier for them to lodge in the mud. These floating seedlings can survive, in a state of suspended animation, for up to a year in the water. Once lodged in mud they produce roots and begin to sprout.

After sprouting, the first years of growth involve additions to the foliage without any major change in height. The first roots appear within weeks. However, the stronger prop roots appear by the third or fourth year of growth, serving as anchoring and breathing organs. These seedlings and saplings have a very low survival rate thanks to the stresses of salinity, flooding, insufficient light and pollution (Beachbeans, 2004).

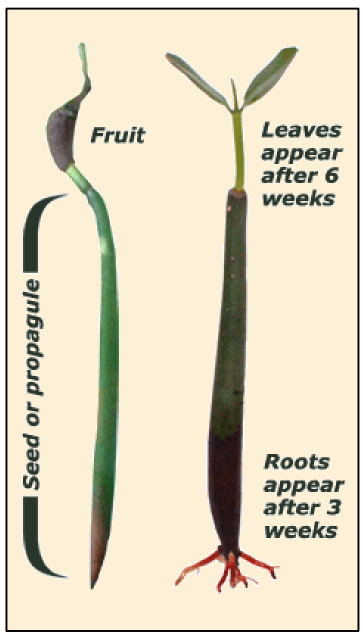

Figure 2-1 A mangrove propagule before and after it catches root, shown here with the corresponding time period required for roots and leaves to appear respectively (Beachbeans, 2004)

A number of investigators have experimentally studied dispersibility and the ability for establishment of mangrove propagules, chiefly by recording the length of time they float in salt and fresh water. Some 'mark and capture' experiments have been carried out. Rabinowitz (1978), for example, established dispersal parameters for the common mangroves of Central America. These include longevity (how long the propagule floats while retaining viability), period of floating, period for establishment (permanent rooting) and period of 'obligate dispersal' (how long a seedling floats before it can initiate the root system that will allow it to anchor). Clearly, the last is an absolute minimum value for time available for dispersal. She gives values from 35 days to more than a year for longevity, from 8 to 40 days for obligate dispersal, and a flotation time from as little as 1 day for *Pelliciera* in fresh water (but 6 days in salt water) to an 'unlimited period' for *Avicennia* (this one is amply found in the Mumbai mangroves at present). It takes most mangroves 5 to 15 days to develop an anchoring root system. Differences between values for experiments carried out in sea water and fresh water are small and do not really affect performance (Tomlinson, 1994).

2.2.2. Mangrove plantation strategies

A brief explanation about the plantation strategies adopted for mangrove restoration has been outlined in this paragraph. Usually most of these directions are meant for mangrove plantation in nurseries and do not apply identically to coastal plantation projects. But some of these guidelines may be useful to know before proceeding with the design of a restoration project. Also, since this study is not meant to focus on the site plantation guidelines but the technical planning of the project, such guidelines should be referred to separately from mangrove plantation manuals such as the 'Mangrove Management Handbook' published by the US Department for Environment and Natural Resources.

Some of the salient points regarding mangrove plantation mentioned in this handbook are:

- The mangrove seedlings or propagules need to be collected periodically at critical times of the year and stored and transported in conditions appropriate for that mangrove species. Sorting of healthy propagules must be done before transportation wherever possible.
- The soil (potting soil for nurseries or in this case, naturally laid soil for large-scale projects) must preferably be prepared in advance before planting the propagules. In the context of this study, such advice would mean that partition of soil and potting, if possible on landward exposed areas, must be carried out before mangrove plantation.
- The mud substrate containing organic matter should be sufficiently thick, taken in this study as at least half a meter.
- Spacing for the seedlings could be variable depending on expected density. Closely spaced propagules move lesser and are better in withstanding wave impacts. However, at least one seedling per square meter needs to be planted to be able to achieve a density of 1000 trees per hectare (even if 10 % propagules hold roots).

2.3. Site Identification

2.3.1. Why a site?

In order to study the possibility of an actual mangrove restoration project, it will be essential to consider an actual study site so as to be able to analyze and solve any challenges that may be posed in a real environment. These challenges may be technical, logistical or economic in nature and only after studying the case of a real site will one be able to say with some sureness whether such a mangrove rejuvenation project will be possible in a realistic scenario.

2.3.2. Prerequisites for the site

The identification of the site for the study is based on the availability of conditions that are suitable for the growth of mangroves using dredged mud from nearby. Consideration has been given to factors such as:

i. Location in a tropical or sub-tropical climatic regime (preferably within the Indian sub-continent)
ii. Situation along the shoreline of the sea/ocean, particularly intertidal environment
iii. Presence of a harbour/dock/navigational channel in close proximity to the site
iv. Evidence of pre-existing mangrove forests in the area to prove the conduciveness of the conditions
v. Commercial landscape of adjoining region and potential environmental benefits

2.3.3. Proposed region

A site that seems to satisfy all of these conditions has been identified in the Indian city of Mumbai, lying within its metropolitan borders. Mumbai is the commercial capital and the richest city, in India (MMRDA, 2008). The total area of Mumbai is 603.4 square kilometer, with 370 square kilometer coming under the Brihanmumbai Municipal Corporation (BMC) and the rest belonging to the Mumbai Port Trust, Defense, Atomic Energy Commission, and Borivali National Park. Due to the deep natural harbour, Mumbai contributes up to 70% of maritime trade in India (Kaujalgi, 2010).

Geographical attribute	Value
Location	18.96° N 72.82° E
Altitude	10 metres

Area	468 sq. km.
Population (2001 census)	12,691,836
Population Density	27,120/sq. km.
State	Maharashtra
District	Mumbai City

Table 2-2 List of important geographical facts related to Mumbai (Anon., n.d.)

Figure 2-2 Location of Mumbai city within India. It is the commercial capital of the country and a major trading port on the western coast of India.

2.3.4. Geography of Mumbai

Mumbai is located on Salsette Island which lies at the mouth of Ulhas River off the western coast of India in the coastal region known as the Konkan. Most of Mumbai is at sea level and the average elevation ranges from 10 to 15 metres. Mumbai is classified as a metropolis of India, under the jurisdiction of the Brihan Mumbai Municipal Corporation. It consists of two discrete regions: the City and the Suburbs, which also form two districts of Maharashtra. Three lakes are located within the metropolitan limits: the Tulsi Lake, Vihar Lake and the Powai Lake. The first two are located within the Borivali National Park. Mumbai also has three small rivers within the city limits originating in the National Park (Anon., n.d.).

The coastline of the city is indented with numerous creeks and bays. Bombay has numerous creeks with close to 71 km² of creeks and mangroves along its coastline. The Vasai Creek to the north and Thane Creek to the east separates Salsette Island from the mainland. Within the city the Malad (or Marve) Creek and the Gorai (or Manori) Creek inundate the suburban region. The Mahim Creek forms the border between the two districts. There are also the Mahul Creek and the Mahim Creek. On the eastern seaboard, large mangrove swamps rich in biodiversity occupy most of the region. Although the islands were merged in the 18th century, islands still dot Thane Creek. Elephanta Island, Butcher Island, Oyster Rock, Cross Island and Middle Ground are scattered across the Creek. The latter three are uninhabited islets owned by the Indian Navy (Wikipedia, 2009). Soil cover in the city region is predominantly sandy due to its proximity to the sea. In the suburbs the soil cover is largely alluvial and loamy (Wikipedia, 2009).

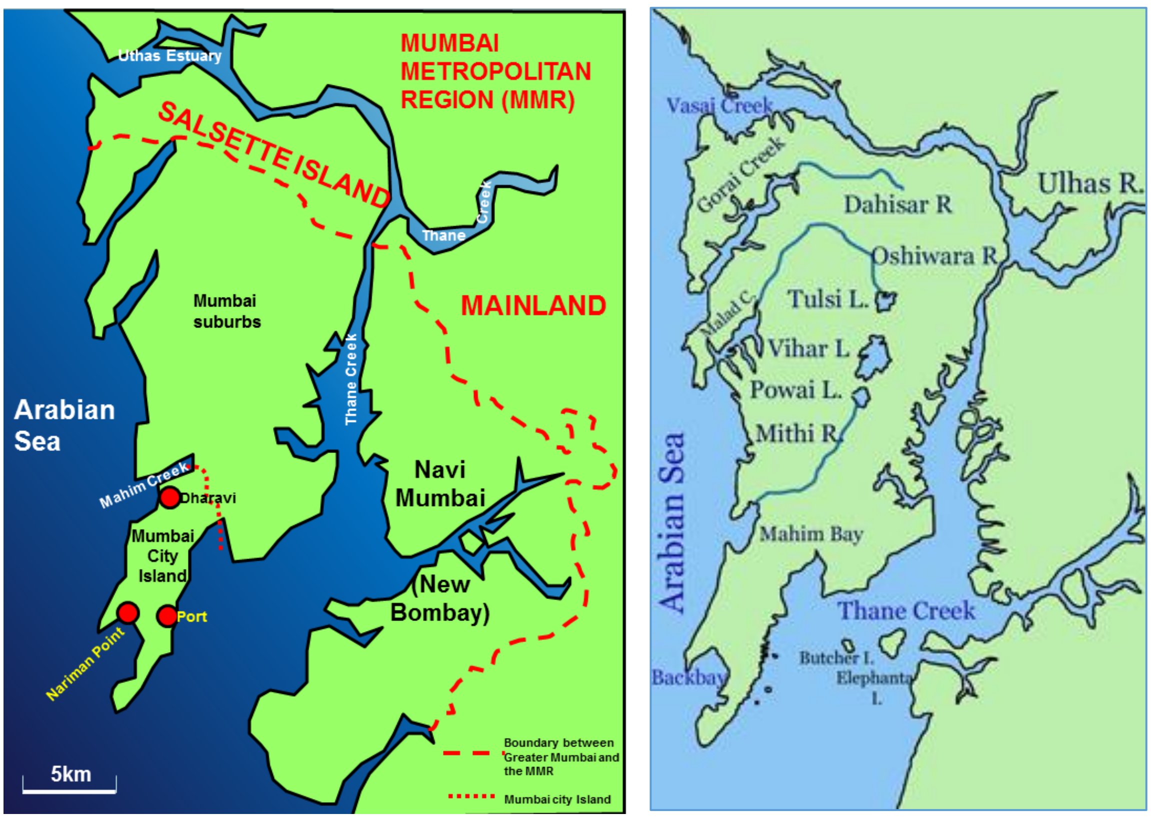

Figure 2-3 (Left) Map presenting city and suburban areas of Mumbai, flanked on the east by the upcoming developing zone of Navi Mumbai (New Bombay) separated from Mumbai city by the Thane creek. (Right) Water bodies within and around Mumbai.

2.3.5. Present situation of mangroves in Mumbai

Mangrove along the coast of Mumbai always faced the challenge of various anthropogenic activities over the decade. In early nineties around 37 sq.km. Of mangrove existed in Mumbai, mostly in Versova, Gorai, Mahim creek, Thane and Ghodbunder. Some sparsely covered patches of mangrove are also found in Bandra, Colaba, Mahul and Malabar Hill.

The most commonly occurring species of mangrove in Mumbai is *Avicennia marina*, this covers the almost 60 per cent of species diversity. The characteristic feature of *Avicennia marina* makes it tolerable for high salinity area. This species also tolerates pollution including heavy metals such as lead, mercury and chromium (Kaujalgi, 2010).

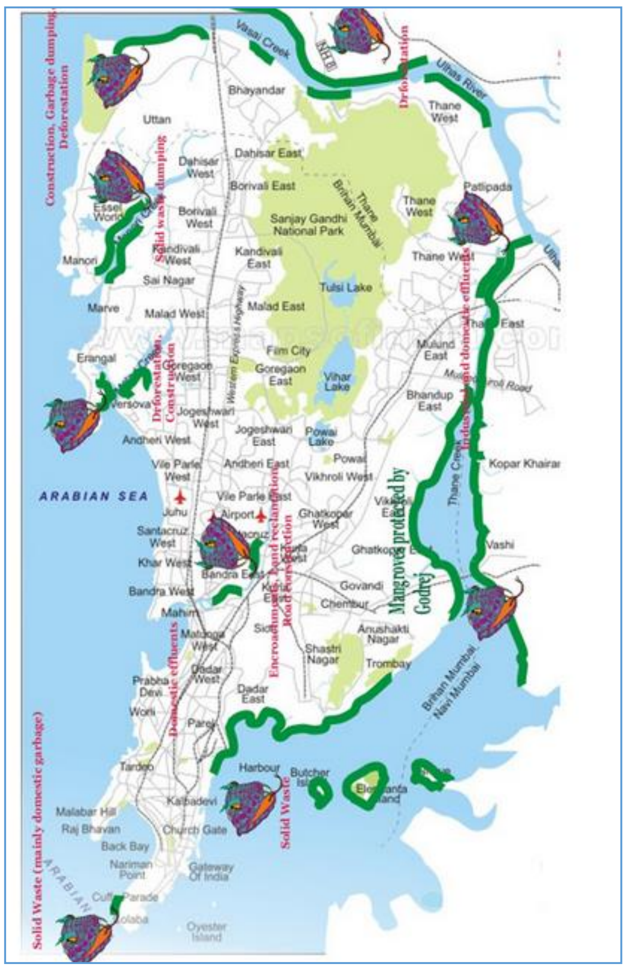

Figure 2-4 Shores (marked in green) along which mangrove swamps are present (Soonabai Pirojsha Godrej Marine Ecology Centre, n.d.)

2.3.6. Dredging as a Side Effect of Development in Mumbai

The rise of New Mumbai along with tis residential and industrial areas along the Trans-Thane creek and large scale reclamations along Mulund, Bhandup, Vikhroli and Ghotkper has caused destruction of the Thane creek; Caught in this double-sided onslaught the creek is dying rapidly. Reclamations for making of Sion causeway and later along Sewri, Wadala stretch, and then recently the Eastern Express way led to destruction of extensive mangrove forests along the creek further reducing the natural silt traps. In addition to this the railway and road bridges in existence have results in construction of water way. The net result is rapid siltation of the whole Sewri mud flats. This sort of siltation must be expected to continue as the sides of the mud flats are reclaimed and the tidal flow over the remaining areas is reduced. In these circumstances gradual deterioration of the depth throughout the harbour is inevitable. The major impact of this has been that the natural flushing of the silt in the harbor is getting reduced thereby, making the Harbour Bay shallower though the port authorities would not readily accept this unpleasant status. The reclamations have altered the flows and currents resulting in increased siltation drastically. With the flow velocities decreasing in the Thane Creek the net result is seen in the increase in amount of silt. Now the natural harbour needs to be continuously dredged. The creek decay has literally chocked the north end of the Harbour Bay. This means that the harbour has to be dredged constantly to maintain a navigable channel with the entries to the docks at desired depths.

While it was being realized that dredging is inevitable its intensity has however not been fully gauged and constantly monitored. The depth of Harbour Bay even in middle stretches did not exceed 7 to 8 m in earlier stages. 20th century navigation demanded approach channels that are at least 11 m plus deep to reach individual docks. Dredging requirement in Mumbai harbour is steeply increasing. The silt mainly accumulates around the corners of the docks and barriers. The studies undertaken by the CWPRS in 2003 indicate that the flood current carries highly silt charged water in tidal cycle as compared to ebb currents. Spring tides are found to be carrying higher silt content as compared to neap tides. The silt is brought from offshore by tidal currents and the weak strength of this current is responsible for deposition of silt in the Harbour Bay, and therefore the high requirement for dredging (Pendse, n.d.).

The reasons summarised above indicate in the near future, the Thane creek where the Port of Mumbai is located, will require frequent maintenance dredging, generating considerable amounts of dredged silt. This means that his location fulfils the premise of the proposed project, the availability of dredged mud. However, it is now important to look at whether there is a possibility of using this dredged silt along the Mumbai shoreline to plant and restore mangroves.

2.3.7. The Need for mangroves in Mumbai

The Maharashtra coastal zone extends between the latitude 15 52'N and 20 10'N and longitude 72 10'E and 73 10'E and falls under five districts from South to North

- Thane District
- Sindhudurg District
- Ratnagiri District
- Raigad District
- Mumbai District

In Mumbai and Thane, mangrove zones are much degraded. During the last 25 years about 40% mangrove area in the Maharashtra coast reduced for constant anthropogenic pressure. Satellite Imagery data shows that in Maharashtra coast the mangrove area is only 148.4 km^2 on the mouth of rivers like the Vashishti, the Thane and the Vaitarana. The minor areas are Ratnagiri, Raigad, Thane, Bombay, Sindhudurg, Mahim, Elephanta Island, Waghotan, Rajapur, Dharamtar, Vasai (Ulhas), Shastri, Vikroli etc (Mangrove Society of India, 2011).

A decade ago, mangrove forests in Mumbai were cleared for living spaces (Vijayaraghavan, 2011). Moreover, accidents such as oil leaks have further destroyed mangrove cover. Recently in 2013, mangroves stretching over several kilometres along the coast in the eastern suburb of Mahul had been destroyed due to an oil leak from a pipeline carrying furnace oil from the sea to a refinery in the area (Deshpande, 2013).

However, it is important to realize that the mangrove forests dissipate wave force and act as barriers. The high tides and rise in water level does not affect the city due to the protective functions of mangroves. The Mumbai Deluge was a flood event on 26 July 2005, which affected the city badly. Loss of lives and property prompted the city's governing officials to carry out an investigation to explain the reasons of the extensive damage. The fact-finding report states that one of the reasons for flooding was the loss of mangroves. Additionally, the fishing industry in Mumbai is largely dependent on mangrove forests. The mangrove stands act as breeding grounds for fishes during the monsoon season. The Koli community in Mumbai worships mangroves because they know that these are breeding and nursery grounds for the marine organisms on which their sustenance depends (Kaujalgi, 2010). The loss of nursery grounds for young fish will lead to decline in the fish population, thus affecting the fishing industry. The ecosystem services that mangroves provide is tremendous. Healthy mangrove forests can be valued anywhere between 0.1 and 0.5 million Indian rupees approximately per hectare. These forests act as natural sand berms and dykes against the tide and high waves apart from acting as carbon sinks. By preserving mangrove forests, the city of Mumbai will have saved itself about 3 billion Indian rupees) every year. Not only will protecting mangroves ensure better air quality and coastal protection for Mumbai, it will also serve to act as a big boost for the fishing community in the area for years to come (Vijayaraghavan, 2011).

2.3.8. Sites in focus

The infrastructural development of the Mumbai Metropolitan Region is planned and managed by the Mumbai Metropolitan Region Development Authority, a body of the government of Maharashtra (MMRDA). Mumbai is divided into six Administrative Zones. Out of the six administrative zones, only three hold large mangrove stands and the potential for additional mangrove restoration (Kaujalgi, 2010).

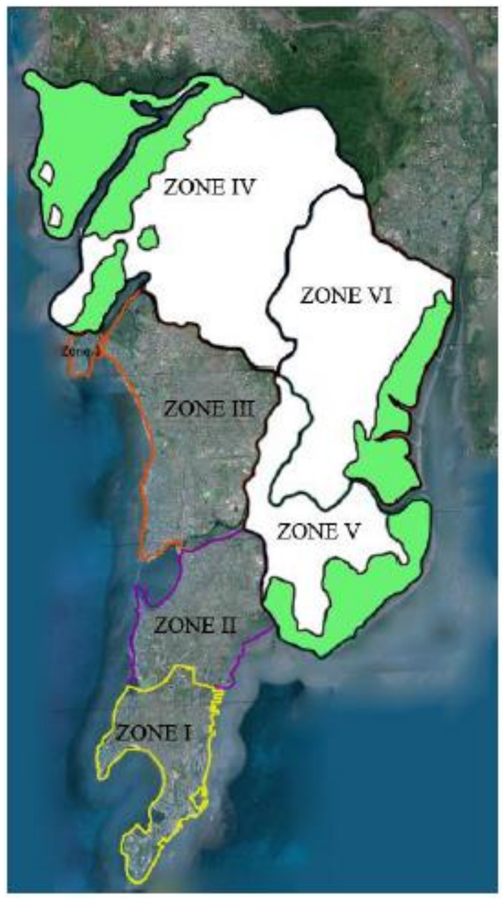

Figure 2-5 The administrative zones (as designated by MMRDA) within which there is scope of mangrove restoration (Kaujalgi, 2010)

The estuaries lining the coast of Mumbai provide ideal settings for mangroves to grow. Mumbai, being an island, is subject to heavy tidal changes on all sides. Mumbai experiences four months of monsoon rains, which are in fact heavy thunderstorms. The water level rises too high for the city to tolerate, this is where the estuaries step in and provide a buffer zone between the sea and land (Kaujalgi, 2010).

An additional important criterion in this case for deciding the ideal location is proximity to the harbour so as to facilitate the local reuse of dredged mud. Since the Mumbai port is located in close proximity to zones V and VI, it is justifiable to consider these two zones for the restoration of mangroves.

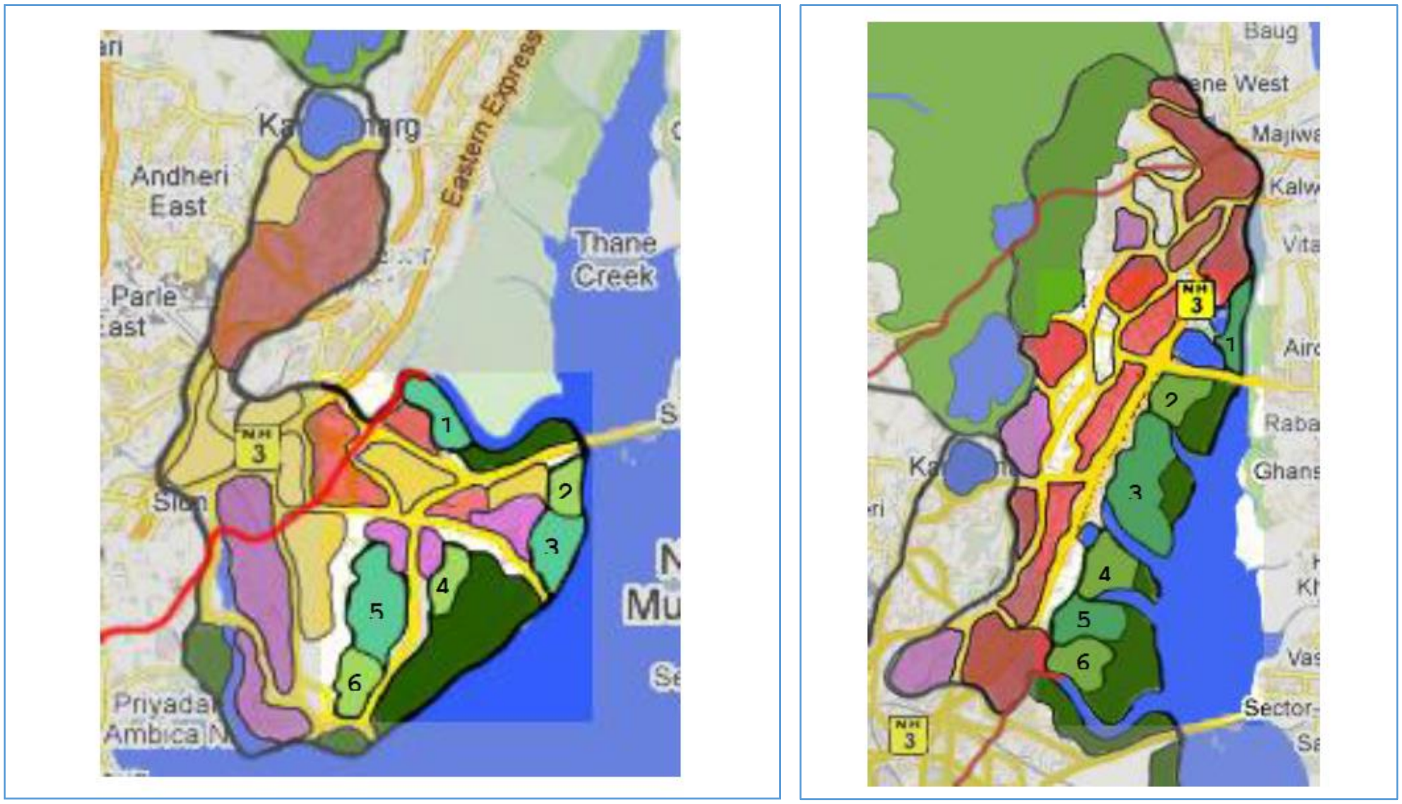

Figure 2-6 (Left) Areas (denoted 1 to 6) within Zone V (refer figure 4) wherein mangrove restoration can be possible, 1 and 2 being high-potential areas. (Right) Areas (denoted 1 to 6) within Zone VI wherein mangrove restoration is possible, areas1, 4 and 5 being high potential (Kaujalgi, 2010)

Also within these zones, studies have been carried out to decide which areas out of 1, 2, 3, 4, 5 and 6 are more suitable for mangrove restoration in each zone. However, in general, it could be concluded that the coast covering zones V and VI along the eastern coast of Mumbai adjoining the Thane Creek and closely accessible from the Mumbai port is the ideal site of focus (figure) for this study which aims to analyze the possibility of mangrove rejuvenation using dredged fines.

Also, within these zones, an offshore reach of about 500 m will be a safe assumption, owing to two reasons. Firstly, as observed from bathymetry, the depth of water does not change drastically up to this distance and this is essential since mangrove plantations require a reasonable low-gradient surface. Second bottleneck is the availability of dredged fines. Maintenance dredging in Mumbai port consists of 7 million m^3 in the main channel, once in for years and 2 million m^3 in the approach and other cross channels every year. A mangrove project of dimensions 27 km X 500 m, with a minimum layer height of 50 cm for the first roots (which grow up to 30-35 cm) puts the requirement for dredged sand up to 7 million m^3, which is also the amount that is roughly available in one cycle of maintenance dredging. Also, it is important to note that not all the dredged material is silt and clay, it also contains a small proportion of sand.

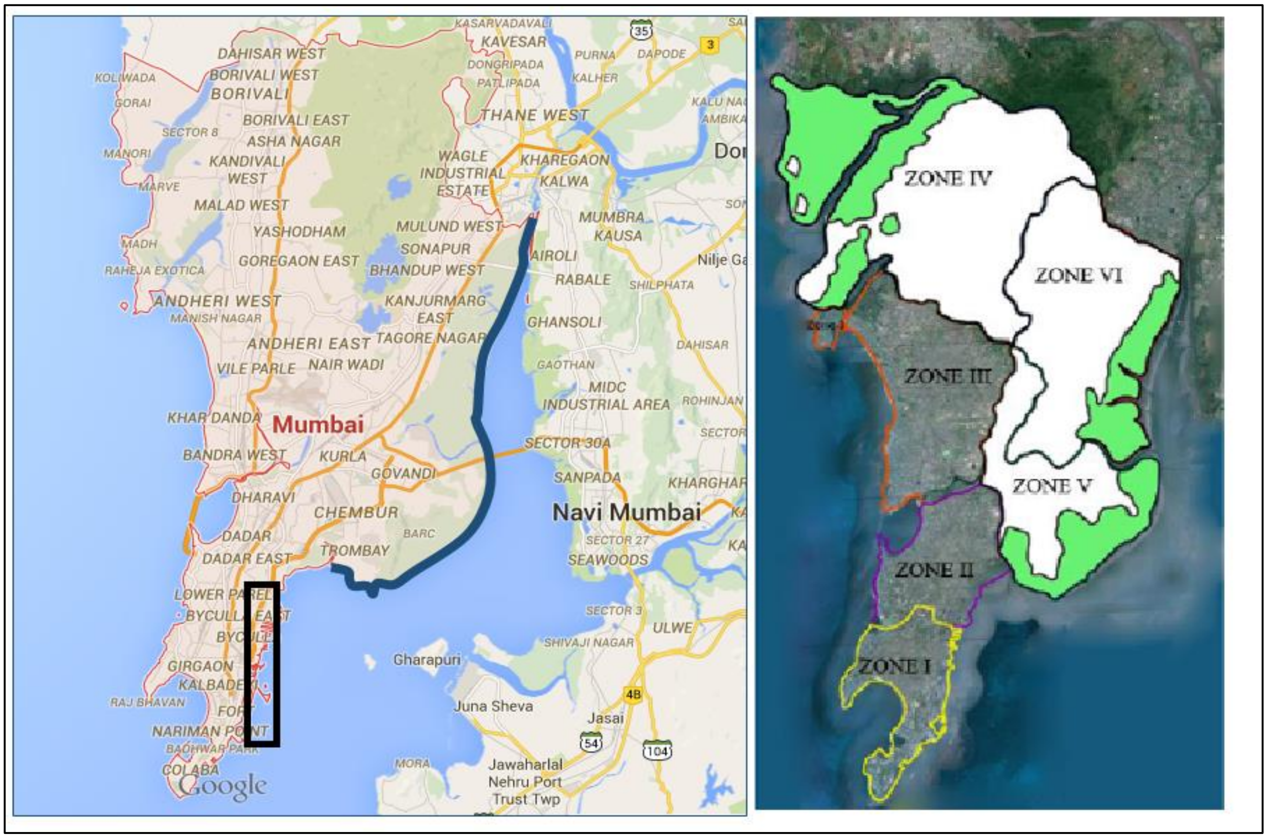

Figure 2-7 (Left) Broad outline of potential coastline selected for study (highlighted in blue), near from the rectangular box covering major docks and harbours under the Mumbai Port Trust and (Right) the zonal map for reference

Details about the selected zone:

Approximate length of the coastline planned for project	27.5 km
Seaward extent of the planned project	500 m
Number of MMRDA Zones covered	2
Area of the basin (measured from Apollo Bunder - Uran mouth)	234300 m^2
Minimum volume of sand required for first layer	7 million m^3

Table 2-3 Facts and geographical dimensions of the region proposed for mangrove restoration

3.

Boundary Conditions

The nature of the surrounding tidal and wave environments is extremely crucial in determining the design criteria for mangrove restoration. This chapter details the environmental boundary conditions, in terms of tidal range, currents and most importantly, the locally generated wave climate due to wind.

3.1. Tide

3.1.1. The Thane Creek as a basin

The shallow funnel-shaped Thane Creek is a semi-enclosed water body open to the Arabian Sea at its southwest approach. Its northern extremity is connected to the Ulhas River through a narrow channel (V S Naidu, 2001).

It is an inlet in the shoreline of the Arabian Sea that isolates the city of Mumbai from the Indian mainland. The creek is divided into two parts. The first part lies between Ghodbunder and Thane, a section from where the river Ulhas flows from the north of Mumbai Island to meet the Arabian Sea on the west. The second part of the waterway lies between the city of Thane and the Arabian Sea at Trombay or Uran, before the Gharapuri islands.

3.1.2. Short basin theory

This is a theory intended for general estimation of flow characteristics in short and rectangular basins. It assumes an average bottom elevation (which is constant) and that water level will be nearly horizontal at all tidal heights. Then, based on conservation of mass, a calculation is done to find the flow velocities (Henk Jan Verhagen, 2001).

After obtaining the flow velocities, it is important to calculate the corresponding length of tide and verify if the short basin assumption is indeed correct or not.

3.1.3. Is Thane Creek a Short Basin?

The major basin Thane Creek extends from the Mulund-Airoli sea-link all the way the mouth of the tidal inlet which roughly extends to the Apollo Bunder in the west to Uran in mainland Maharashtra in the east. Considering this as the tidal basin for study, certain details about the dimensions can be summarised:

Distance from the mouth of the basin inward (westward boundary Measured from Apollo Bunder)	45 km
Width of the inlet to Thane creek	8 km
Area of the basin (measured from Apollo Bunder - Uran mouth)	234300 m^2

Figure 3-1 The basin of the Thane creek with its scale and corresponding are demarcated

The deepest point in the Thane Creek is found at its mouth near the Appollo Bunder at about 12 m. A simple analysis based on the depth yields the celerity of the tide to be 10.8 m/s. Mumbai falls in a predominantly semi-diurnal tidal environment, wherein the influence of semidiurnal tidal components is found to be large (M P Subeesh, 2013). Therefore, the tidal length comes out to be approximately as 550 km **(See APPENDIX A)**. The length of the basin as tabulated in the previous section suggests that it is ten times lesser than the tidal wavelength, implying that although the basin may be small in size, it may not be perfectly approximated as a short basin. One needs to note that there may still be some lag introduced due to this, as has been observed that the tide at the inward end near Airoli lags behind the one at mouth by about half an hour (V S Naidu, 2001).

However, at an early stage in a project, it can be very useful to get a rough idea of the situation by making a simple manual calculation using the theory of Chezy (Henk Jan Verhagen, 2001). Therefore, though the short basin analysis could not be very accurate in this case, it nevertheless can be a good first approximation.

3.1.4. Cross-sectional velocities

The maximum cross-sectional velocity is the maximum velocity during a tidal cycle. To understand its behaviour as a function of the cross-sectional area, it can be approximated as the amplitude of a sinusoidal tidal motion (Judith Bosboom, 2012).

Assuming the approximation of a sinusoidal tidal motion, the equilibrium (cross-sectionally averaged) tidal velocity is determined to be 2 m/s **(See APPENDIX B)**. It is important to note however that this is the velocity at the entrance (mouth) of the basin and the magnitudes of tidal currents inside the basin may vary from this. Nevertheless, one can safely say that the order of these current velocities will be same as that determined above. In fact, the magnitudes currents further inside the basin are found to be relatively lesser and this fact has indeed been verified through studies detailed below.

3.1.5. Verification from Previous Studies

In general, maximum speeds are found to occur between Elephanta Island and the Jawaharlal Nehru Port (JNP). The flow from the mouth is modified by the presence of Elephanta Island and converges between the Island and the JNP area. Under this influence, a strong current is found in this region. On the other hand, during ebb tide, flow from Panvel Creek and a part of the main channel flow from Vashi leads to high values in the same region. During neap tides, maximum current speeds range from 0.3 to 0.4 m/s in both ebb and flood conditions, while during spring tide it varies from 1.0 to 1.4 m/s. The circulation pattern delineates that the water mass entering into the creek from the eastern side of the creek is transported toward Panvel Creek after passing through the

JNP navigational channel, while the current orientation in the western side is mainly toward the northern upstream of the creek.

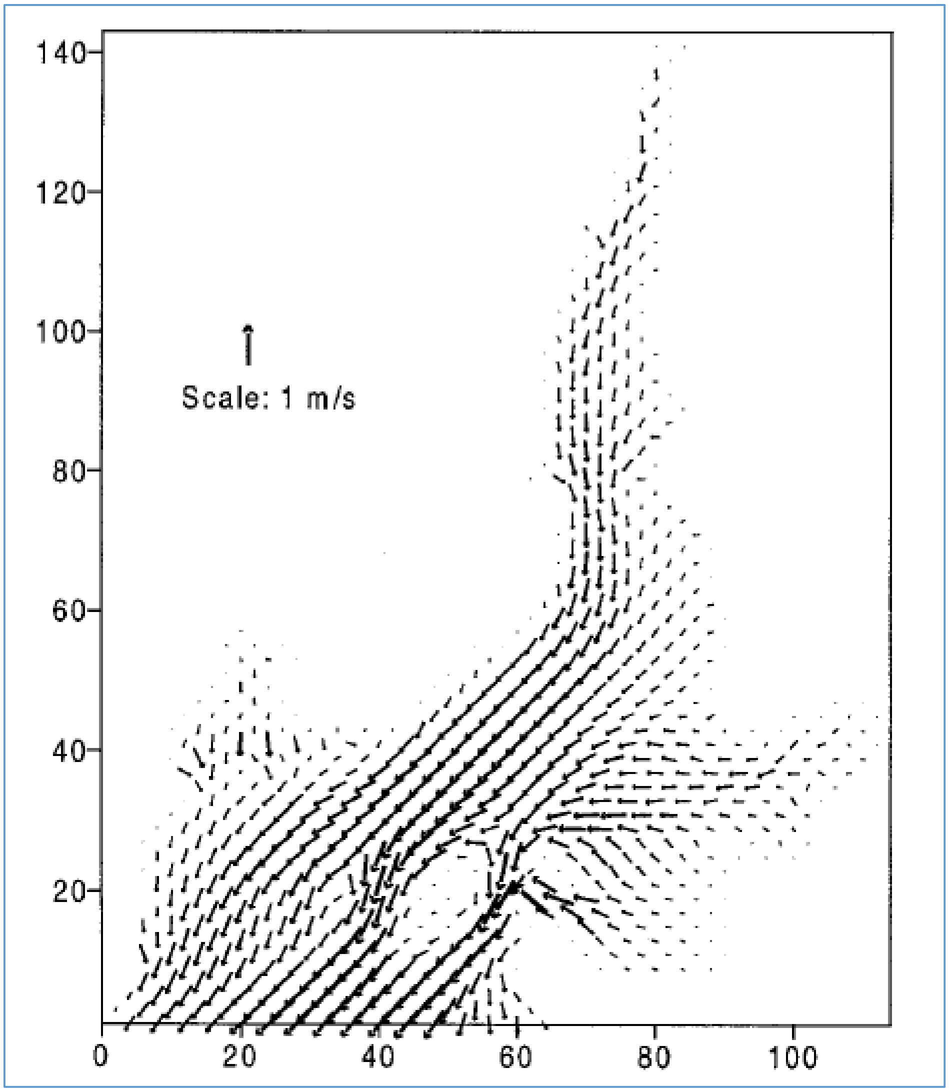

Figure 3-2 Maximum spring tidal current vectors during ebb. Note that the current magnitudes are higher at the mouth of the creek and around the Elephanta islands (V S Naidu, 2001). Closer to the western shoreline, along which the mangrove rejuvenation case is being studied, the magnitudes of the currents are comparatively low. Also, the magnitudes of flood currents are very comparable to the ones during flood, except that they have their directions reversed.

Generally, the western side of the creek has ebb dominated flow, whereas the eastern side experiences slightly flood-dominated currents. The northern portion extending from Pir Pau experiences low residual currents (<0.05 m/s), whereas values of 0.25 m/s are found in the southern part (V S Naidu, 2001).

3.2. Wave

In additional to tidal conditions, the wave conditions in the location of the project have also been looked into. Although the global wave forcing may be a first starting point **(See Appendix C)**, in sheltered short basins such as the Thane Creek, it is the locally generated waves that are most important in determining the design criterion. The driver for these local waves is the wind at the location and therefore local wave estimation has to begin with analyzing the wind data.

3.2.1. Wind data

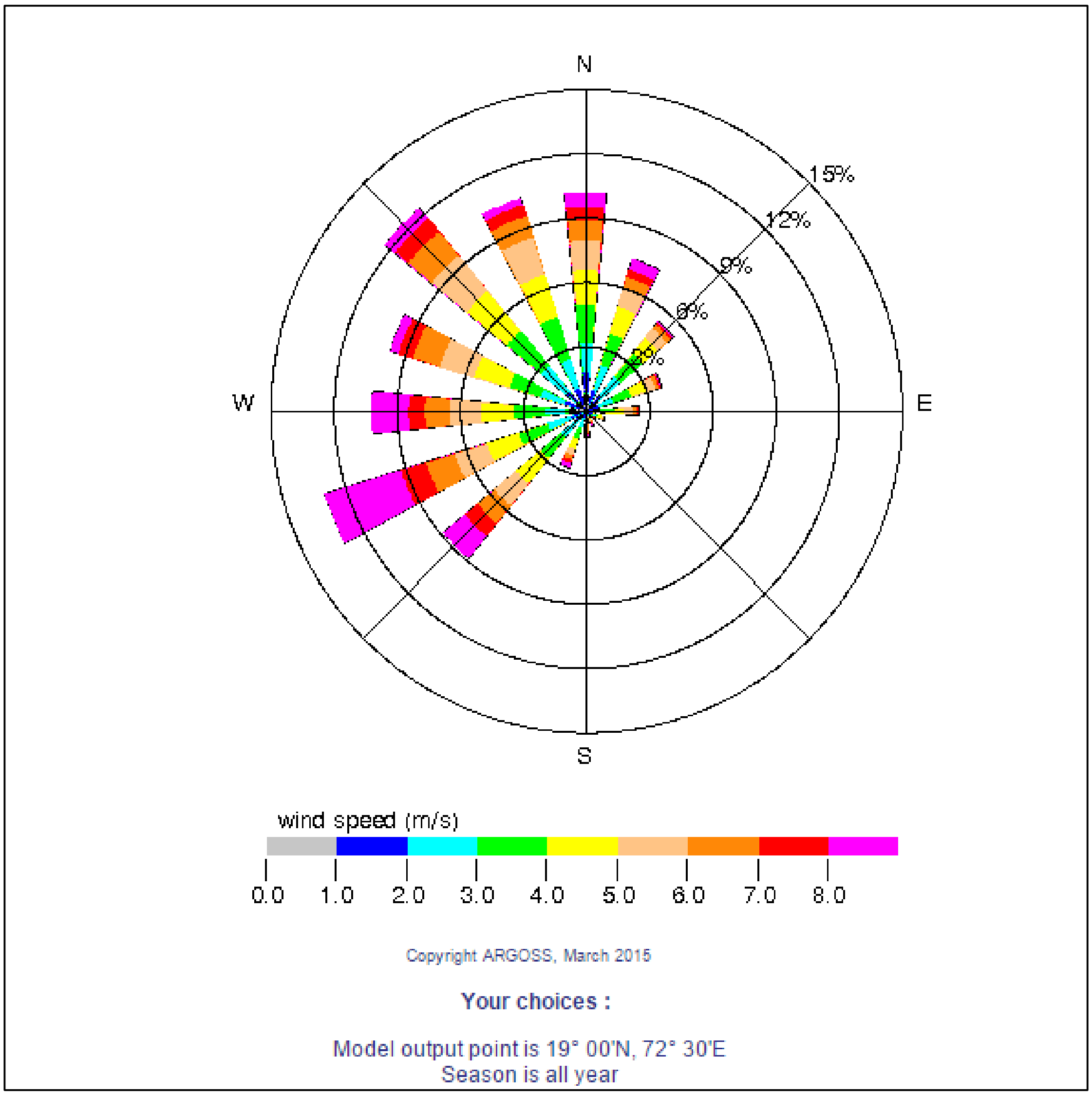

Figure 3-3 Wind scatter diagram for the Mumbai coastal zone for the entire year

3.2.2. Sorting of wind data

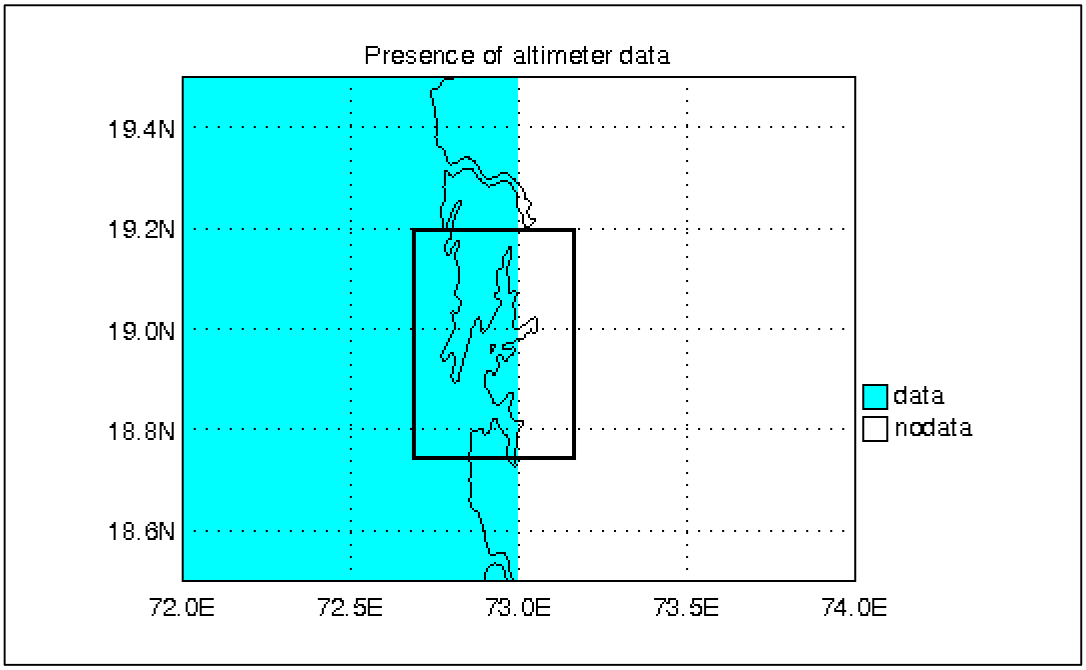

Figure 3-4 The zone for which the aforementioned wind data has been extracted

Wind data for the basin was gathered from the wave-climate database of BMT Argoss. Although direct wave estimates have been detailed on the website, these are mostly annual and monthly averages and additionally are most confined to the swell wave observations made from satellites. In the particular case of this project where the protected basin of the Mumbai harbour-Thane Creek is under consideration, locally generated wind waves within the zone of interest may be more important in deciding the response pf confining structures such as dikes or walls.

After having obtained the data, a decision had to be taken as to which sections of this data would be relevant to the design. Two important factors to be considered here are the direction and seasonality of the wind. First and foremost, those directions of wind which are unlikely to produce any waves towards the region of interest must be discarded. Therefore, the wind data that falls within this zone, which is roughly directed from the North to South-West had to eliminated. Furthermore, since the purpose of temporary structures which need to be protected against such wind conditions is for a short time, it can be ensured that such structures be built in such a way that they remain exposed in the season when the wind is least strong. Out of the relevant wind directions being considered for such exposure, one can note that the period outside of the November-February bracket is relatively safer in terms of exposure to locally generated wind-waves. However, given the peculiar location of Mumbai within the South-West monsoon geographical climatic zone, one also needs to bear in mind that the monsoon season extending from July-September is a season of frequent and torrential rains accompanied with strong winds and swell-wave action from the south-west direction, which could potentially affect the wave climate within the Thane creek. This leaves the window period between March and June as the calmest period compared to the rest of the year. Wind data corresponding to this particular period has been displayed in figure 3-5.

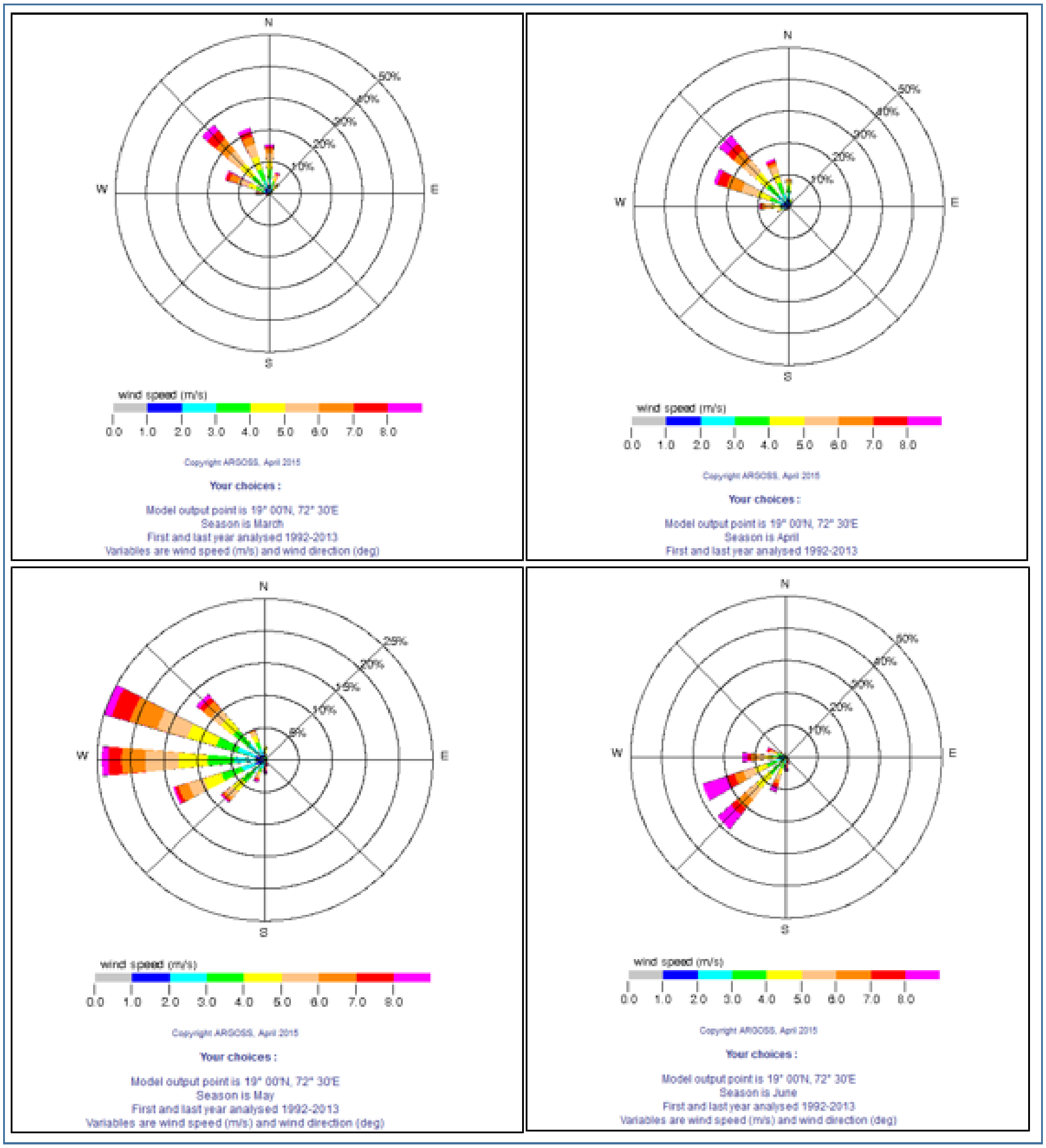

Figure 3-5 Wind scatter (rose) diagrams for the months of March, April, June and May (clockwise from top). These are the calmest months for wind facing the zone of interest.

3.2.3. Wave height estimations

For estimating the locally generated wave heights, it is important to take into account the fetch distances along with the wind speeds and directions. The longer the fetch, the higher the wave generated. Considering the range of wind directions demarcated above, the longest critical fetch distances within that window have been show in the figure 4.

As had been explained earlier, the waves to be generated in the zone of interest are going to be coming from a certain set of directions, which are only a part of the whole directional spectrum for the four months from March-June. At the outset, it needs to be mentioned that of all the possible wind speeds measured at these locations, only 13 % of the winds are expected to be blowing towards the zone of interest. This also helps explain the reason why these four months were chosen as the safest, since there is little wind in these periods towards the eastern coast of Mumbai and therefore there is a correspondingly less probability of waves being generated. However, out of these 13 %, it is important to single out the most risky zones, especially the ones corresponding to longer fetch distances. It is important to note here that a cut-off value of 5 m/s is chosen as the wind speeds for which a wave of considerable height (a minimum of 0.5 m) is generated **(See APPENDIX D)**.

Figure 3-6 The maximum local fetch distances that could be possible for winds blowing from the north or north-east

The maximum wave heights possible have been found to be near 2 m. However, the probability of these wave heights occurring in reality is very miniscule (less than 1 percent), which begs the question of whether a structure compatible to these wave heights is really necessary, especially when it has been established that the mangrove seedlings (which are being protected by the structure) can actually survive in the overtopped waters. On the other hand, it has been found that there is a roughly one in ten year chance of a wave as high as 1.9 m may be caused by the wind in critical zones **(See APPENDIX D)**. Although this also seems a relatively low probability, it can be used as a limiting height for the design of the containing structure. Also, given that there could be errors introduced in the dimensions of the structure due to issues relating to subsoil settlement, a conservative limiting wave height makes added sense. Also to be mentioned is the fact that the probabilities of exceedance are for a 50-year duration and not for longer periods as is the case with many designs. This is because the purpose of service of the structure is about a year, or a couple of years and therefore the exceedance probability for these periods should be sufficient.

4. Global Alternatives for Design

Since the germination of mangrove propagules is linked with the extent of the layout and the level of confinement, various alternative design layouts can be proposed as the outline for the project. These plans will vary in terms of compartmentalization and placement of drains. This chapter explains, in a general sense, what these alternative layouts could be and the reasons behind their conception.

The design methodology for the planned reuse of dredged mud in the basin for mangrove plantation first of all has to deal with the placement and layout of mud. This has to do with two important aspects, the spatial placement and the temporal procedure of that placement.

Mangroves are confined to the upper intertidal zone of sheltered sedimentary coasts and estuaries (Balke, 2013). The mangroves' niche between land and sea has led to unique methods of reproduction. Seed pods germinate while on the tree, so they are ready to take root when they drop. The 'niche' zone to which mangroves are confined puts limitations on the spatial extent of the envisaged project. The placement of the mud should be restricted mainly in the intertidal zones, which helps establish the onshore and offshore extents of the placement. The alongshore extent has previously been delimited during the identification of the study site, based on the conduciveness of the certain part of the shoreline for the mangrove plantation.

Additionally, if a seed falls in the water during high tide, it can float and take root once it finds solid ground. If a sprout falls during low tide, it can quickly establish itself in the soft soil of tidal mudflats before the next tide comes in. A vigorous seed may grow up to two feet (about 0.6 m) in its first year. Some take root nearby while others fall into the water and are carried away to distant shores (American Museum of Natural History, n.d.). These aspects about mangrove germination are crucial when determining the timing of mud placement, especially unfruitful when seeds are planted in a high tide environment accompanied by an impending ebb flow. The seeds will not only fail to sprout but will also be lost off to the sea.

As discussed in the aims of the project, the design will comprise of the placement of mud accompanied by a suitable filtering (permeable) structure offshore. Permeable structures made of local materials such as bamboo, twigs or other brushwood can be placed in front of the coastline. These structures let the sea water pass through, breaking the waves rather than reflecting them. As a result, waves lose height and of sediment trapped at or near the coast. These devices imitate nature - mimicking the structure of a natural mangrove root system (Femke Tonneijck, n.d.). This could be a good starting point to consider for the functionality of the filter. Also, until the mangrove seeds germinate and the mangrove plants actually take root, it will be such permeable structures that will keep the placed dredged material from flowing away offshore.

4.1. Compartmentalized placement through containment paddocks

One of the procedural designs for laying the mud is the consideration of compartmentalized placement. The method basically would firstly mean dividing up the intended coverage area into suitable sections. This is to be followed by set-up of mud-filters or permeable boundaries along the borders of these sections. Having done that, sequential placement of the dredged mud can be carried out in each compartment depending on accessibility and output capacity of that particular section. After this, one has to wait for the mud to undergo self-weight consolidation while monitoring the efficacy of the filters in draining out water.

At the outset, one advantage of such a method could be that a section-by-section placement may help to obtain proper feedback on whether the placement method is indeed working in one compartment. Equipped with some feedback and requisite improvements, the next compartment can be laid more effectively. One such project has been carried out previously (not for purposes of mangrove rejuvenation) in Australia wherein dredged mud is placed into the containment paddocks in a slurry form, dewatered and allowed to undergo self-weight consolidation (Dhanya Ganesalingam, 2011). Another benefit of a compartment-based placement, especially in a busy city like Mumbai, is that such placement may allow some time for alternative projects or utilities in other charted compartments to relocate.

However, some criticism of this method could be in terms of time-delays due to compartmentalized placement. Further deficiencies related to efficiency may be discovered as and when an elaborate design is developed.

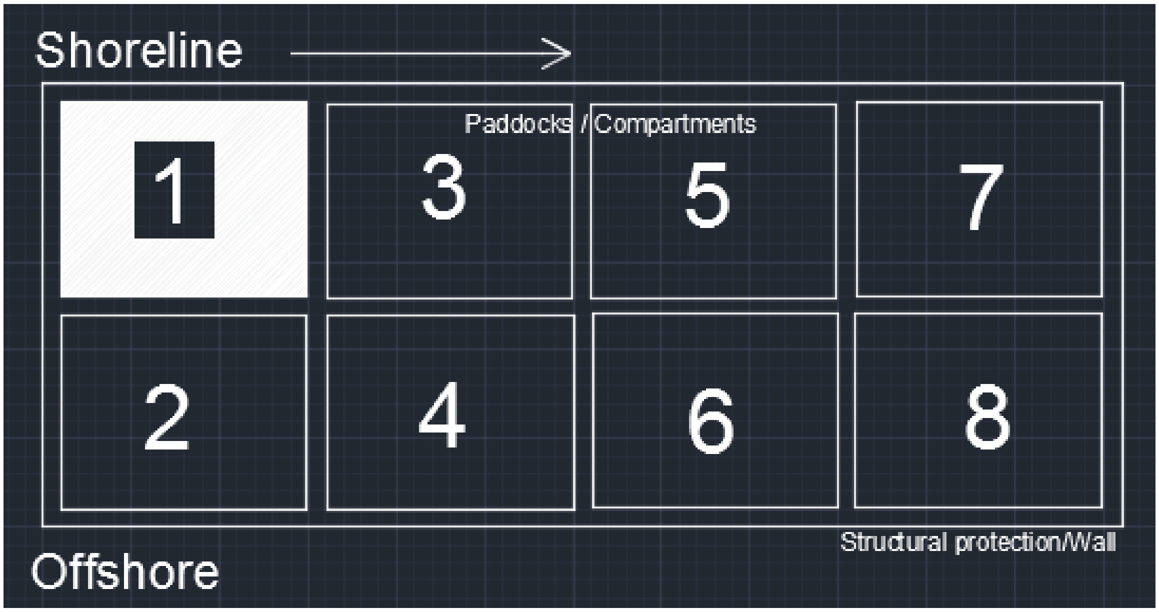

Figure 4-1 Placement of mud into compartments or 'paddocks' leading to section-by-section placement in compartments that would be relatively small compared to the dimensions required for simultaneous vertical or alongshore placement

4.2. Alongshore stage-wise horizontal placement with offshore structural protection

Another alternative worth considering could be step-wise placement of mud in alongshore strips gradually moving offshore to cover the entire intertidal zone. An overview of the directions and magnitudes of currents along the shore may also be significant in such placement.

The need of a confining structure may persist. It is, nevertheless, important to note that the confining structure in this case will have to be lengthier than the one required in the compartmentalized layout. However, as the mangrove swamp gradually takes root and progresses, it will provide reasonable stability in the landward zones. Additionally, the placement could be commenced at favourable low-tide intervals wherein the alongshore water-levels will be significantly low.

Such a method will involve planning of the stages as well as building of a permeable structure that could be effectively moved or a filter that could be relocated further offshore over time. This may pose an important logistic difficulty. Additionally, some delay may be introduced at every stage based on the wait induced by the mangrove germination period.

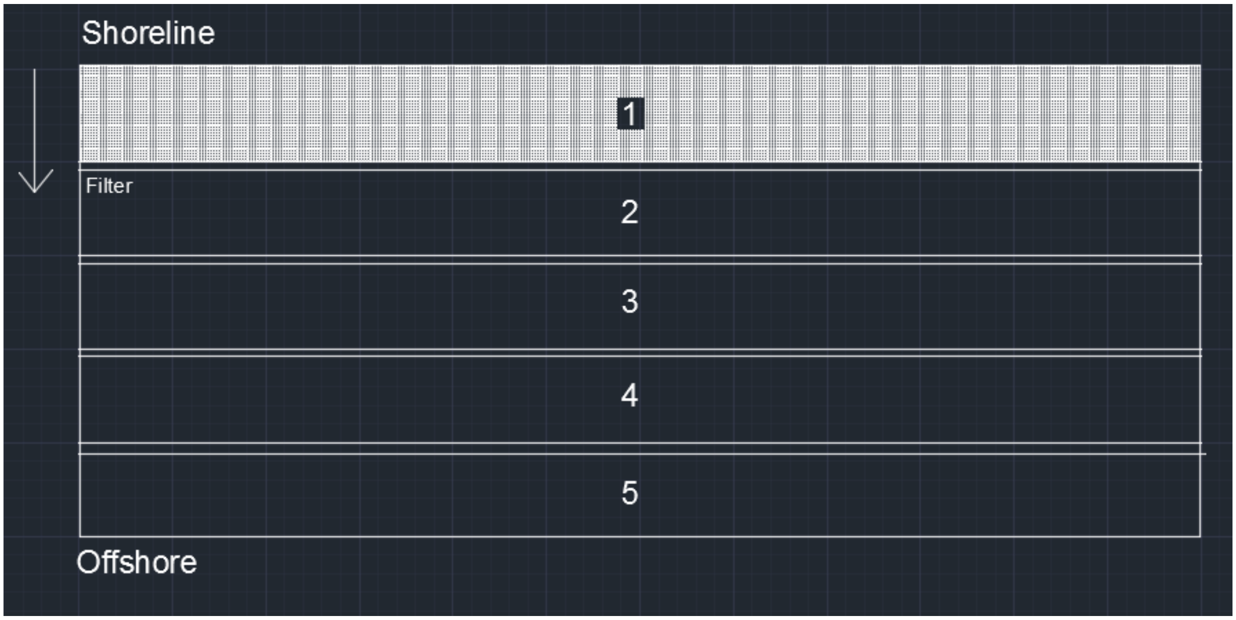

Figure 4-2 Simultaneous placement of mud on an alongshore stretch with gradual movement offshore. The offshore end is to be contained by a permeable structure. Also to be noted is that 'alongshore stretch' here means a significant alongshore distance (at least 3 times the alongshore length of compartment described in the first alternative. However, it is not supposed to imply the entire alongshore coastal stretch of the project (that would be mean a protective structure of the order of 25 km which is not only an engineering challenge but also a heavy investment)

4.3. Simultaneous placement with vertical drains

The third alternative is a layer-by-layer vertical placement of the mud accompanied with installation of vertical drains made of suitable material. Such a placement would lead to gradual increase of the mud-levels akin to the gradual evolution of a natural mud-flat. There are studies that suggest that to accelerate the consolidation process, prefabricated vertical drains and surcharge have often been used (Dhanya Ganasalingam, 2012).

The positives of this method may be simultaneous placement along the entire area without major time-delays. It also may lead to relatively quicker drainage due to presence of vertical drains, though the efficiency will greatly depend on the number, construction material and arrangement of these drains. Additionally, it is extremely important for these drains to be integral with nature since they will stay within the built mud even after the mangroves take root.

However, finding suitable material and arrangement for vertical drainage may be a challenge. It will be important to look at availability of local material such as bamboo or wick. Furthermore, the stability of mud may have to be continuously monitored. The role of geotextiles or rock mattresses along the bed in combination with the vertical drains may be significant for the overall stability of the mud. Also, a simultaneous placement procedure would imply that all existing actives in and around the zone may have to suspended during the period of building the mudflat.

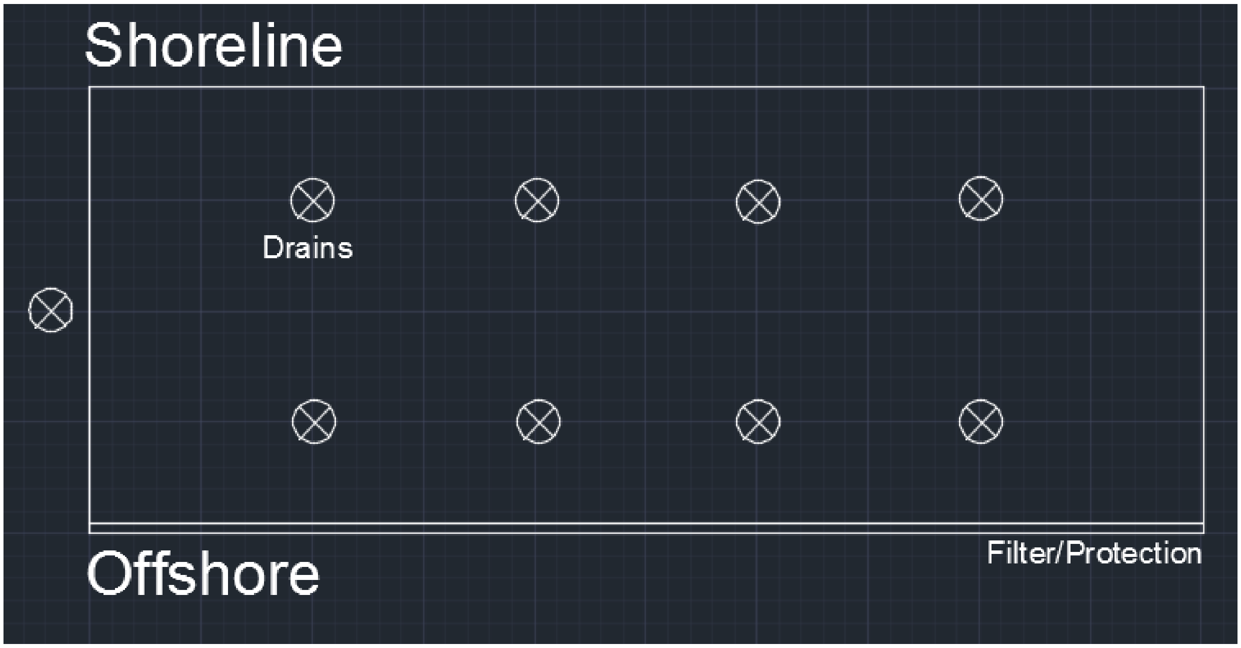

Figure 4-3 Vertical placement of mud with the presence of vertical drains (going into the page in this diagram)

4.4. A Combination?

Consideration should also be given to a suitable combination of these methods, in case they are found to be optimal together. For example, vertical drains in an alongshore placement procedure may hasten the mangrove germination. Similarly, compartmentalization with permeable filters for the vertical placement of drains may aid the consolidation and hence the plantation of mangroves. However, the effectiveness of each of these procedures taken individually will be sufficient to study since the individual performance of each method is believed to superimpose in a combined structure to give added efficiency.

4.5. Dimensions

As has been clarified from the previous study, the length of the stretch selected for the project comes to about 27 km. If one assumes a complete wave height reduction (up to 100 %) requirement along with an average height of the mangrove trees as 5 metre, the approximate width of the mangrove stretch required would be approximately 250 m (refer chart below).

One has to importantly note here that this is only an approximation. Quyhn's charts are based on data collected at 5 marine stations along the Vietnamese coast and covering about 24 species of mangroves (Quynh, 2010). Although some of these species such as the

Mangrove Apple and the common mangrove are also widely found in Mumbai, the direct applicability of this chart to such a situation can only give a good approximation at the best. Additionally, it needs to be clearly observed that this chart mentions an 80% coverage which, if followed, implies that the mangrove restoration project in Mumbai must also adhere to achieving this 80% coverage in the designated zone.

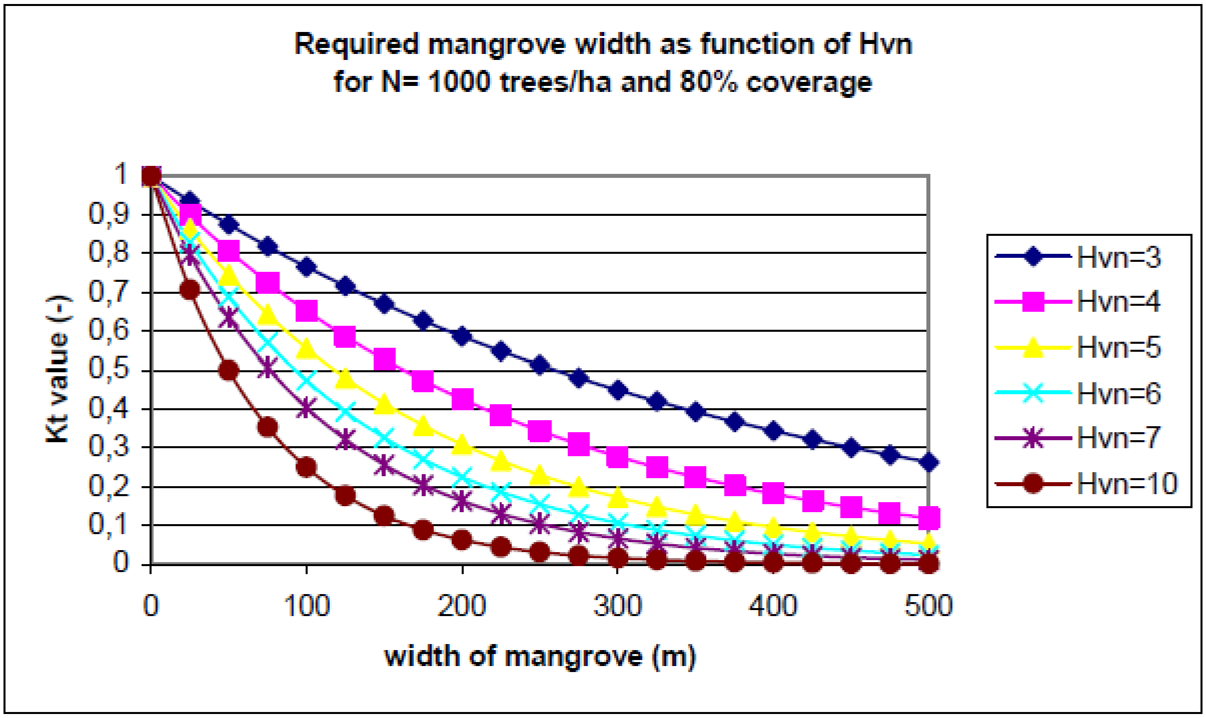

Figure 4-4 Design graphs for the width of mangroves using formulas of Quyhn (Verhagen and Loi, 2012)

4.6. Broad schedule for implementation

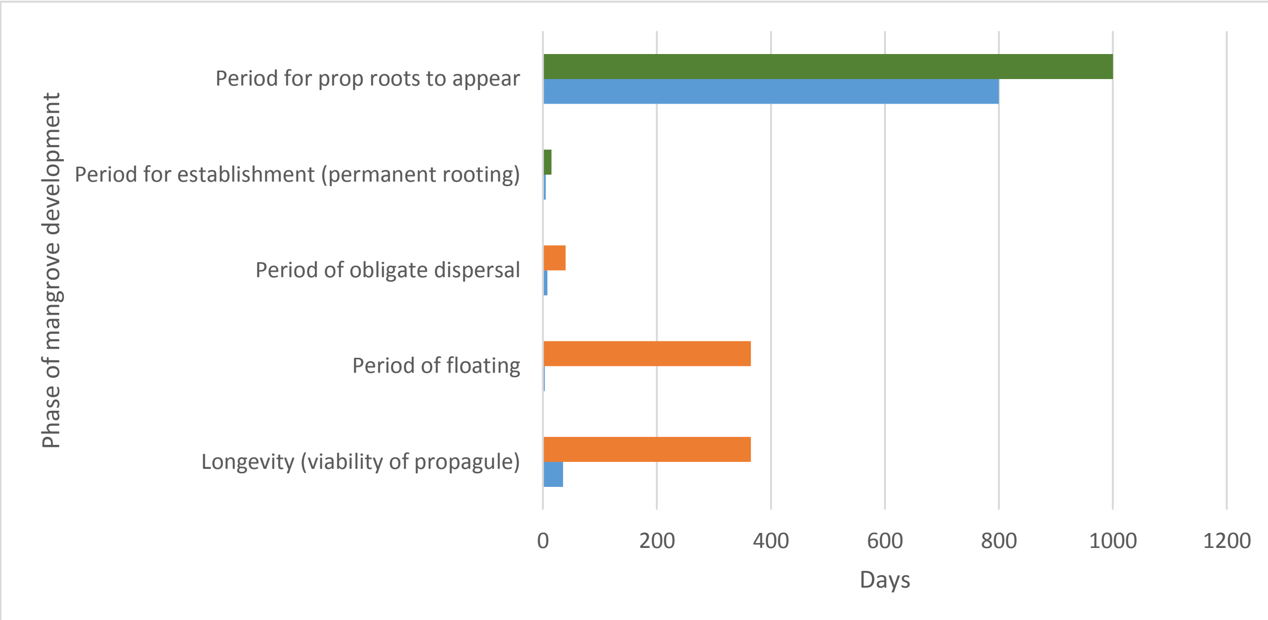

Figure 4-5 Stages of mangrove development shown along with the minimum and maximum time in days required for each stage of development

The time frame from the appearance of the first roots to the final anchoring of the roots in the mud ranges from a few days to 3 weeks after sprouting. However, it is also important to know that the seedlings (propagules) can survive in the water for up to a year, before being lodged in the mud to germinate. This would therefore imply an incubation period of **about a year** until the appearance of prop roots which could take **up to 3 years**. So the period of confinement is expected to last for at least this much amount of time.

Construction of temporary structure (and subsequent drainage mechanism)	0.5 - 1 year
Minimum life of confining structure	3 - 4 years
Dismantling of the structure	Few months

Table 4-1 Rough schedule for the construction, serving duration and dismantling of the confining structure, depending intrinsically with mangrove growth. Some alterations may take place owing to seasonality.

4.7. Link with Boundary conditions

The environmental factors that will be relevant for the determination of the height of the confining structure in this case are the tidal parameters, the wave climate and the settlement properties of the subsoil. It is important to understand that in this case the purpose of the confining structure is not so much to conventionally protect the coastline but it is importantly to provide a confined environment of mud for the mangrove plantation. Such a confinement is intended for a relatively shorter time period and therefore consideration such as storms or sea-level change could be ignored in such a case since these become more important for construction of structures meant for long-term protection.

The tidal conditions in the Thane Creek region have been summarised below. The high water spring value is what is the most relevant to the design. The highest high water recorded may also be taken into consideration but only if the probability of occurrence of such a scenario is available as part of data.

Tide	**Above(+) or Below(-) Chart Datum**
Highest High Water recorded	+ 5.39 m
Mean High Water Spring Tides.	+ 4.42 m
Mean High Water Neap Tides.	+ 3.30 m
Mean Sea Level.	+ 2.51 m

Table 4-2 Some tidal values that may be relevant in determining the height of the retaining structure

About the waves, as has been discussed, the predominant south-west swell does not produce significant heights, generally lesser than 1 m. But, wave height data displayed by ports may not be so relevant because the interest of such data lies mainly in long period swell waves. However, in the case of this study, what will be important are locally generated (short period) waves within the basin due to winds blowing from the eastern end. In this context, it will be important to get some information about the wind conditions in the area and the resulting possibility of wave generation. These waves will depend predominantly on the direction and the magnitude of the wind as well as the fetch within the area of the basin. These studies have been carried out based on the wind data for the specific geography and the design wave height has then been determined for the proposed protective structure to be built. These studies have been summarized in the preceding chapter on boundary conditions.

Also a question about the settlement of coastal mud arises. How much is this value? There are no accurate values for these but through some studies on levees, estimates of settlement for 1-3 years usually do not exceed a few feet (Nguyen, 2006). An approximated analysis of the settlement of soil has been carried out **(See Appendix F)** in order to determine its effect on dike designs being considered in the following chapter.

5. Design of compartments

The division of the project area, as explained earlier, will have to be done either as small rectangular compartments or longitudinal sections. In addition, in every design, an outermost confining structure will be required. This chapter details the design of these confining structures, with dikes and bamboo pile walls as two major design alternatives.

One of the proposed design for the placement of mud for the mangrove project is to divide the project area into several compartments and then place the mud with an appropriate drainage mechanism within these compartments. What is the need have such retaining compartments? Once the water line reaches areas of consolidated mud in combination with significant wave action (either wind waves or boat traffic induced waves), large clumps may break off, resulting in erosion. Pulled up and down by the tidal currents, they roll over the bottom, they may break up into smaller clumps, erode more or less, and eventually become rounded mud pebbles which can roll even better and be transported over longer distances or accumulate in deeper areas. The purpose of the retaining structure is to avoid any such erosion of mud and prevent it from flowing into deeper areas.

Firstly, in order to get an idea about the dimension and scale of these compartments, some indicative figures have been provided in the previous section. Since the stretch of land will extend to about 27 km along the coast and about 500 m offshore, these compartments could ideally be proposed as rectangular sections of 500 m X 250 m. The alongshore distance of 500 m is basically taking into consideration the longshore length along which such a compartment can be structured without making it too large in size, while also maintaining a very gentle gradient. Also, another advantage of this limited confinement will be that the mangrove propagules when laid will have lesser distances to travel (due to obstruction from the bordering dikes) and therefore there will be higher probability of them getting lodged in the mud sooner and germination process will be speedier.

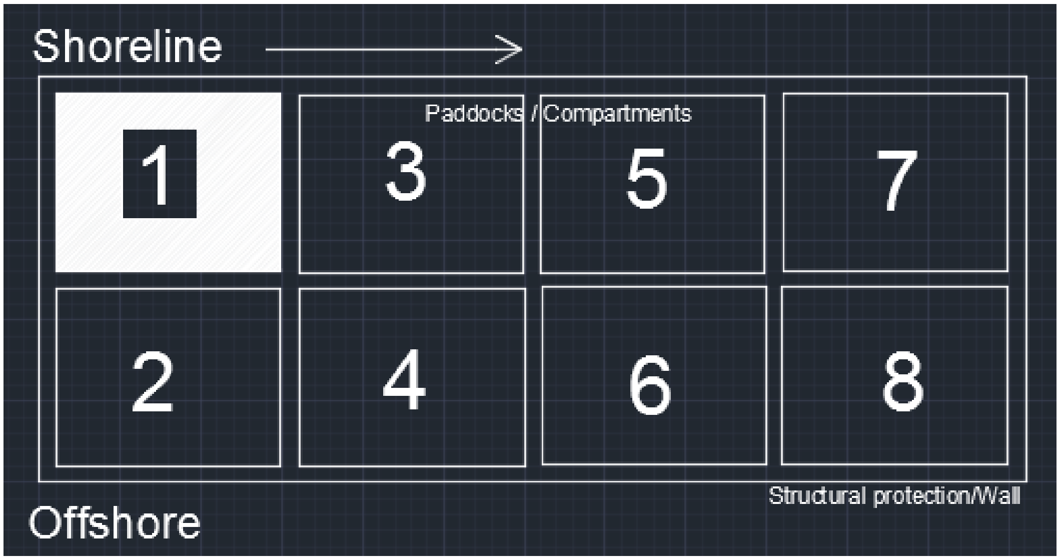

Figure 5-1 Placement of mud into compartments or 'paddocks' leading to section-by-section placement in compartments that would be relatively small compared to the dimensions required for simultaneous vertical or alongshore placement

5.1. Dikes

Since the retaining or confining structure in this case has a purpose that is temporary and is not intended for long term protection of the coast, it will not make much sense to have a hard structure built for confinement. Therefore, options such as building a seawall have to be ruled out at the outset. There have been suggestions about using concrete kerbs but this has been deemed expensive.

One of the foremost options that may be considered for a confining structure is the building of a conventional sand dike. It is again important to understand that in this case the purpose of the dike is not so much to conventionally protect the coastline but it is to provide a closed environment of mud for the mangrove plantation. For the design of the dike, one of the first conditions one needs to assess is whether material for construction of such a dike is available. It has been observed that sand mining activities are being carried in various locations along the western coast of the state of Maharashtra. In fact, there have been records of sand mining taking place along the western shoreline of Mumbai. This could well be diverted for construction of such a dike.

Next, the depth in the region where the dike is to be constructed as well as the wave and tidal conditions are significant. Herein, the depth has been found to be 2 to 4 metre and the wave and tide parameters have been listed in the section on boundary conditions.

The environmental boundary conditions relevant for the calculation of the height of the dike have been mentioned in the previous section.

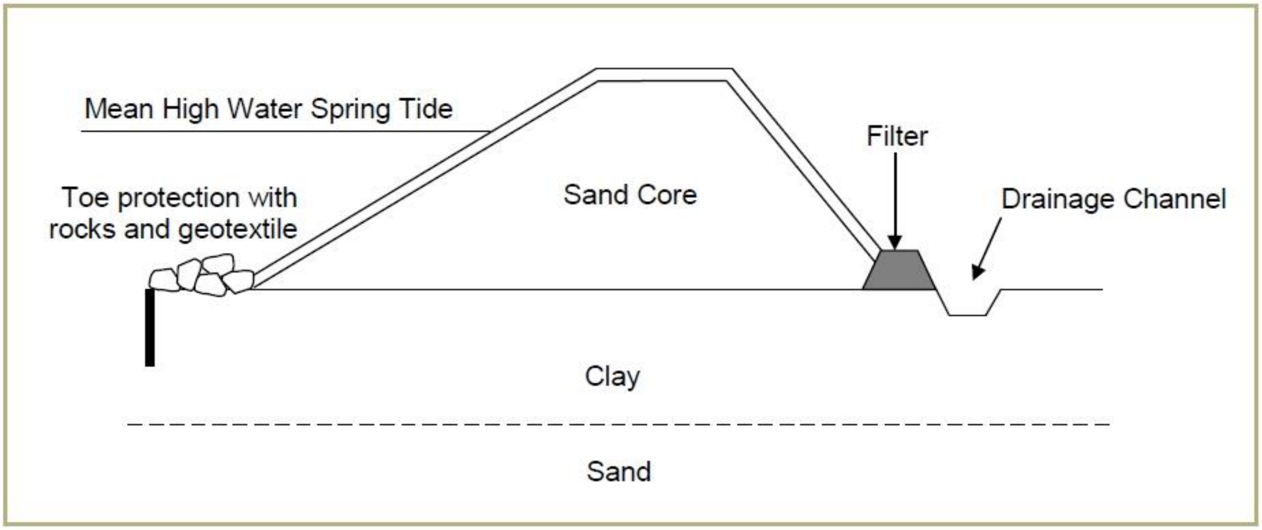

Figure 5-2 Diagram showing the design of a sand dike (ClimateTechWiki, n.d.). This is solely a representative diagram and the dimensions are not indicative of the actual design dimensions.

Considerations for the freeboard are also questionable in this case. Is the run-up or the overtopped water capable of re-suspending and carrying the mud back into the sea? Although this seems less plausible, one may still have to look at the behaviour of mud momentarily saturated by sea-water from run-up or overtopping. As a practice, it is safe to assume a small freeboard in order to make up for any corrections in the design water level.

What could be some issues associated with dikes?

One of the problems associated with a sand dike is the mixing of sand that may take place with the underlying mud. The mangroves may not thrive in a substrate that consists of too much coarse sand. Also, in this context, one major area of concern is the erosion of the rear-slope of the dike due to overtopped water. Such erosion may be detrimental to the underlying mud as it may lead to an excessive concentration of sand within the mud, thus changing the composition of the soil and impacting the possibility of mangrove growth. As will be discussed in section 5.4, having a carpet of grass to prevent run-off over dikes in one of the possible means of overcoming such problems.

5.2. Bamboo piles

As a second option, a fence consisting of bamboo piles can be considered for a retaining structure. Bamboo poles have been used for protection from waves in various cases, such as walls for aquaculture farms or as groynes etc. One such picture of a T-groyne with bamboo wave screens for a project in Vietnam has been shown below. The protection was made as a double bamboo row with reed in between. The bamboo piled wall being proposed for mangrove rehabilitation could have a similar look, with bamboo culms and reed-panels in the gaps. However, when the design of such a structure is being considered, the main points that become necessary to look at are the height of the piles and the bottom support of such piles.

Figure 5-3 Bamboos with reed filling in the gaps being used as wave screens along a T-groyne in Vietnam (picture by H.J. Verhagen)

The height of the pile could have similar deterministic conditions as those mentioned for the height of the dike. However, as for the spacing of the piles, it will be practical to not permit any in this case as the purpose is to prevent mud from escaping. The piles will have to be structured as aligned together, resembling a wall. An arrangement consisting of vertical halved culms, for example, could be considered.

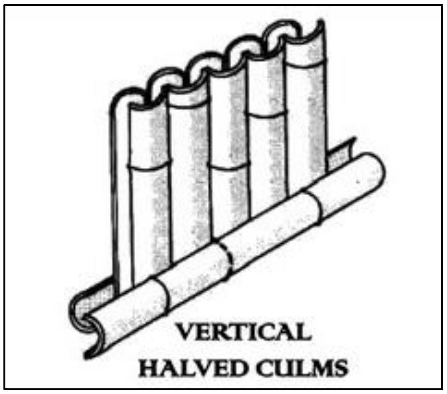

Figure 5-4 Depiction of the bamboo piled wall made up of the vertical halved culms. The joining of the piles and each half needs to be done in a manner that does not allow mud to escape (Chaudhari and Kolamkar, 2012).

For the bottom support, there are some hints available from the design of fish farms that use such bamboo piles as retaining structures. Some designs suggest that boulders are laid about 20 cm thick between the piling to form a floor. Gravel layer of 5 cm thick is spread on top of the boulders, and then compacted. The exposed ends of the piles should be level with the surface of the gravel layer (Cruz, 1983). A similar support consisting of boulders or other large supporting rocks which are available from the Western Ghats could be made use of for supporting such a structure along the eastern coast of Mumbai.

What could be some issues associated with bamboo piles?

One of the major concerns with regard to bamboo piles is about how much of wave action it can withstand. Some reservations have been voiced in such a design for the Lunwei sub-district in China. Bamboo fences are often found to be weak against stronger waves such as typhoon waves (Liu and Chang, 1986). However, waves of such high intensity are not expected to arise at all in this scenario. Secondly, another concern regarding bamboo walls is their mud-tightness. While these walls will favourably allow water to pass through them, they will also lead to passage of mud particles. It must nevertheless be mentioned that absolute mud-tightness is an ideal that is not realistically possible while keeping project costs low. Given that the service periods of these structures are not long, some mud permeability will have to be accepted as a trade-off for lower expenses.

5.3. Designs of Dike as Retaining Structure

While considering various weather conditions and soil parameters for determining the height of a dike as a confining structure, it needs to be reiterated that this is not meant to be a dike giving a totally water-proof compartment. In fact, some amount of water needs to be allowed in. This will not only enable the mangrove seeds to get adapted to the environment in which they would grow but will also save considerable costs by allowing a comparatively lower confining structure.

The concern, however, with allowing overtopped water in is about the rear slope of the dike structure. In reality, the concern could be two-fold because given the history of torrential rains in Mumbai, one may not be entirely sure of the strength of both the slopes, rear and front, while they are built. Therefore, these slopes need appropriate protection. One of the cheapest methods available to protect such slopes from overtopping and run-off is the provision of a grass-top. This grass top will act as a protective layer above the top layer of the dike in protecting its slopes and other surfaces.

For the dikes, four alternative design cases have been considered. Steeper slopes have been given higher priority because gentler slopes will in turn leave very little land for the mangroves to be planted within the compartment. The slopes of 1:3 and 1:4 have been considered. From the results regarding the volume requirements for the different dike sections, a dike with a slope of 1:3 with the provision of a berm is found to be the most optimal **(See APPENDIX E)**. Also, from the point of view of space requirement for the mangroves, a dike with the 1:3 slope but with a berm would be ideal, without the sacrifice of some additional building material for the crest in the dike without the berm.

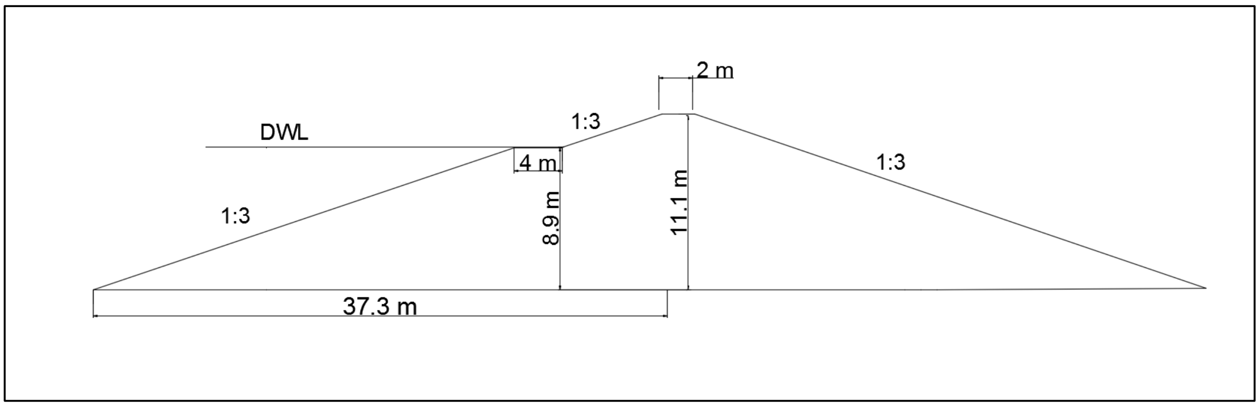

Figure 5-5 The dike design that was found to most optimal in terms of space and volume requirements, of the 4 cases considered

Also, going back to the proposed design plans which included 1) Splitting plantation zones into compartments 2) Splitting zones into alongshore longitudinal strips and 3) No compartmentalization with vertical drainage, all the three plans will result in a different volume requirement for the material of the dike. For example, making compartments will require several confining dikes, resulting in highest demand of material as opposed to the vertical-drain plan where only an outermost confining dike will be necessary. Also, different design plans will lead to a different space availability for the planation of mangroves. Surrounding dike slopes in compartments will occupy a sizeable proportion of the total compartment area whereas in other design, these ratios would be lesser **(See APPENDIX G)**.

Given below is the summary of the two conflicting deciding factors namely volume and space requirements, for each of the design methodologies:

Case	Slope	Berm	Volume requirements in longitudinal (m³)	Proportion of total area taken up by the dike in case of longitudinal design	Volume requirements in compartmental design (m³)	Proportion of total area per compartment for plantation of mangroves	Volume requirements in the vertical drain design (m³)	Proportion of total area per compartment for plantation of mangroves
Case 1	1:3	-	28.5 million	0.25	35 million	0.33	15 million	0.06
Case 2	1:4	-	29 million	0.33	35.5 million	0.47	15.5 million	0.08
Case 3	1:3	4 m	27 million	0.35	33.5 million	0.50	13.5 million	0.08
Case 4	1:4	5 m	28 million	0.45	34.5 million	0.55	14.5 million	0.11

Table 5-1 Different dike design cases with time and space requirements against the different mangrove placement plans

However, a comparison between these designs must not be viewed completely in the light of volume and space requirements. Since the purpose of the design is to create a conducive environment for the mangroves to thrive, the more confined the areas, the better the possibility of the seeds holding root. In that sense, some would argue that the compartmental design, though sacrificial in terms of space and volume, may be ideal for the final cause. However, as has been discussed earlier, there may be a possibility of combining these designs to obtain the advantages of each individual plan. This will be discussed in later chapters.

5.4. Grass revetment

Grass, as a cover layer for dikes and revetments is able to withstand considerable wave loads. Waves, as may occur along rivers are no problem for a good erosion-resistant grass-cover. On sea and lake dikes waves with a height up to 0.72 m (and maybe higher) will not cause any damage on a uniform, closed grass-cover with a high density of roots.

A grass cover with a high erosion resistance consists of a closed vegetation with a high root density in the layer of 0 - 0.15 m. The realisation of a closed grass-cover and dense root system depends on the way of execution of the management, and to a lesser extend of the soil properties. In case of a correct management the erosion resistance of the sod-layer becomes better than the erosion resistance of high quality clay (Schiereck and Verhagen, 2012).

A deep-rooted grass cover can be extremely advantageous from the point of view of protection of the dike. However, not much detailed information is available on the quality of grass, precisely to determine what is good and bad grass. What one would need to ensure, however, is that for waves of 1.4 m on an average, a good quality grass would be able to tolerate such conditions for about 10 hours. Since this a climate dependent phenomenon, it is hard to predict the longevity of such a weather condition. However, given that the specified wave heights do not occur with a probability of more than 1 in 10, one could say that the grass could be more-or-less safe.

Some specification is being proposed for the type of grass that would be suitable for such purposes. The two most important things to check here have been the ready availability (or possibility for growth) of the type of grass in the said environment and the protective properties of the grass against erosion.

It can be seen that Bermuda, Carpet and Ray grasses grow in a common pattern that evenly covers the slope surface. With good development, they may create a continuous mat protecting the dike slopes. Vetiver grass, on the other hand, tends to spread across the ground in separate clumps (Trung, 2014).

Bermuda grass, the selected option:

Bermuda grass is one of the options for the kind of grass that can be used for the protection of dike slopes. It is advantageous as it grows in such a way that it will cover the entire dike slope like a grass-mat with no gaps. Bermuda grass, unlike what its name suggests, is a native grass to the African Savannah region and India and therefore is found aplenty in these regions.

The grass is known as *Arugampul* or *Indian doab* within India (specifically as 'durva' in Maharashtra) and has been used traditionally as cattle fodder as also for erosion control from excess rains or over-watering. In fact, this is one of the purposes in this case of having a grass cover over the dike – protecting the dike slopes from the rainwater run-off during the monsoon season that is accompanied by heavy rainfall.

Bermuda grass can be easily grown from Bermuda seed resulting in "Medium plus" turf grass lawn. The grass type depends on the Bermuda seed used - common or improved varieties. Ideal time for planting or seeding Bermuda grass is when the weather is warm and temperature in the range of 32-40 degrees (C). Bermuda seeds germinate in seven to ten days and fully grows in grass within 60 to 90 days. This makes the grass ideal for growth in the pre-monsoon season because this is ideally the summer season in South Asia, which is marked by warm day temperatures, almost always lying within the temperature range mentioned here.

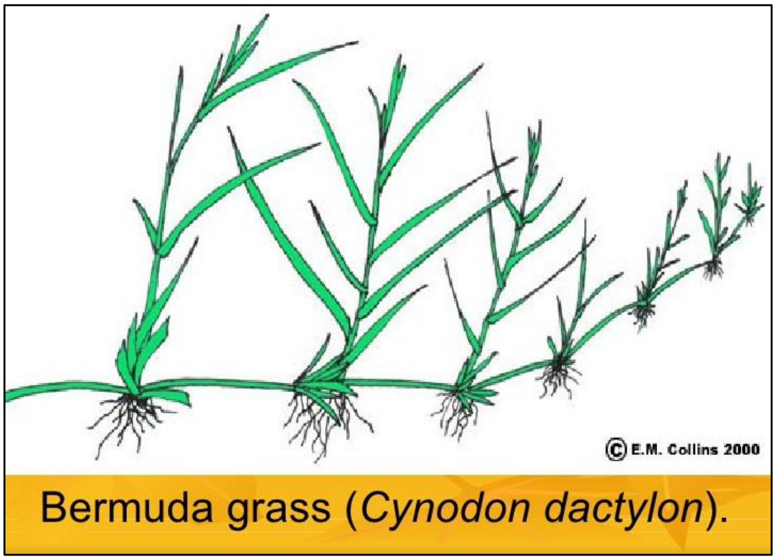

Figure 5-6 Depiction of Bermuda grass

One of the important issues with Bermuda grass is that it will only grow above normal high water in case of a salt water environment. This means in the case of the project along Mumbai coast, a different kind of protection will be needed for the area between the normal HW line (which is about 1.5 m above MSL) and the toe of the dike. For these, either a slat tolerant grass cover or a hard revetment could be considered.

- **Rhodes grass:**

This is one of the turf grass types found widely in India that is believed to bear moderately saline environments. There are particularly two advantages of this grass from the point of view of the mangrove rehabilitation project in Mumbai. One is that it grows in spring and summer (March to June in India), which is mainly the target period for the laying out of the project. Secondly, this grass is known to grow very quickly after seeding i.e. in about 1 to 7 days. However, the slopes need to be protected until the seeds are grown, as will be discussed in later sections.

The downside of Rhodes grass is that though its utility for erosion protection in farm soils is practised, its utility as protection for dike slopes in not well known. There are a couple of reasons why it could still be used. Firstly, its growth in the form of a uniform layer (not clumps) indicates that it is similar to Bermuda grass to some extent and also given the short service duration of the protection (about 4 months during monsoon), this grass will be able to survive rain (it can take about 650-700 mm rainfall per year). Secondly, there are possibilities of mixing the grass with other mildly salt-tolerant species such as *alfalfa* grass which have proven utility as grasses for erosion protection.

- **Hard revetment:**

Hard revetments could be in the form of stones, rip-raps or placed blocks. These would prove to be a robust protection but the question arises whether such strong protection is really needed. Also, the discussion here is about regions below the mean high-water. This means that during higher tides, there will be little wave action in these regions and even lesser rainwater run-off. Also, the purpose of the revetment is mainly for the duration of one Indian season, which is very less compared to the design periods for which such hard structures are considered.

Finally, considering the scale of the project and the dimensions of the dike, the cost and labour required for laying of such a revetment would be very high relative to the grass-revetment options. Although the utility and serviceability of the salt-tolerant grasses is not perfectly known, it still seems as a wiser trade-off for investing in an expensive hard revetment for a short duration.

There is still one additional concern with Bermuda grass. A well-developed Bermuda grass cover can withstand heavy rainfall. However, there might be a problem during the first year after seeding the grass. The extra protection is needed. Reed mats or jute are to be preferred above geo-synthetics, because they are only needed for short periods, and are environmentally friendly. Heavy torrential rains during the monsoon season can lead to considerable run-off over the dikes. The grass revetment on the top of the dike may not be able to withstand these high run-offs on its own. In this case, additional protection for the dike may be required. These can be in the form of reed mats, jute nettings or geo-synthetics.

- **Reed mats:**

Blankets made out of reed are used widely for protection against rainwater run-off. These are especially effective to protect the slope until the grass has not grown. Seeding with grass and overlaying with mulch or mats is done to stabilize cleared or freshly seeded areas (Environmental Preotection Agency (US), 2010). The locally collected reed rhizomes could be planted through gaps in mats or mulches. Types of mulches include reed-like organic materials, straw, wood chips, bark or other wood fibers. In some cases decomposed gravel may also be used for making such mats. However, if such reed mats are made with woven choirs, they should have sufficient tensile strength to sustain on slopes.

- **Jute netting:**

Erosion control blankets made from jute netting can also be an option for protecting the grass slopes from run-off. They are easily accessible in the tropics as also relatively easy to install. The challenge usually is to ensure that water flows over these blankets. To do this one needs to 'key' it into the slope by digging a small trench on the top of the slope.

In this trench, the top end of the material is to be laid so that the trench is lined. To line it the edge is folded underneath itself and then it is secured using staples (University of Washington , 2006). The trench is then filled in to the previous soil level. Such jute netting is easily nature-integrated as well as protective against flows due to rainwater run-off.

- **Geo-synthetics:**

This is another option that is often considered. However, for most of the above protections, and especially for geo-synthetics it is important that they are made flexible in the vertical directions as well. The pressures from the soil and a growing grass-root system can generate tensional stresses in the geo-synthetic, which can fail if it is not made flexible enough in the plane perpendicular to the surface. Especially on inner slopes with grass, these geo-grids will have additional tension (in addition to the tension while installed) due to water stresses (caused by run-off and saturated soils) (Garcia, 2007). Woven geo-grids are known to have better ultimate tensile strengths (up to 370 kN/m width of grass turf) in comparison to no-woven unitized stiff geo-grids (maximum up to 117 kN/m).

5.5. Design of the Bamboo Piled Wall

The designs of the split-bamboo pile walls have not been frequently tried and tested in coastal environments. But bamboo as a construction material is used extensively in tropical areas.

The design of the split bamboo pile walls in this case has been carried out based on the overtopping found for small vertical walls over dikes, and was found to be 11.2 m. This does not indicate a precise value but this is an approximation that has been obtained through design. On the other hand, tolerance of water by mangroves as well as accommodation for subsoil settlement may make up for any deviations in the design height. Apart from the height, other dimensions of the pile have also been estimated from previous examples. Another factor that becomes important in these dimensions are the spacings between piles. These relatively large spacings need to have some kind of panelling that will complete the 'piled wall'. The dimensions for the pile diameters, heights and spacings have been approximated for this design **(See APPENDIX H)**.

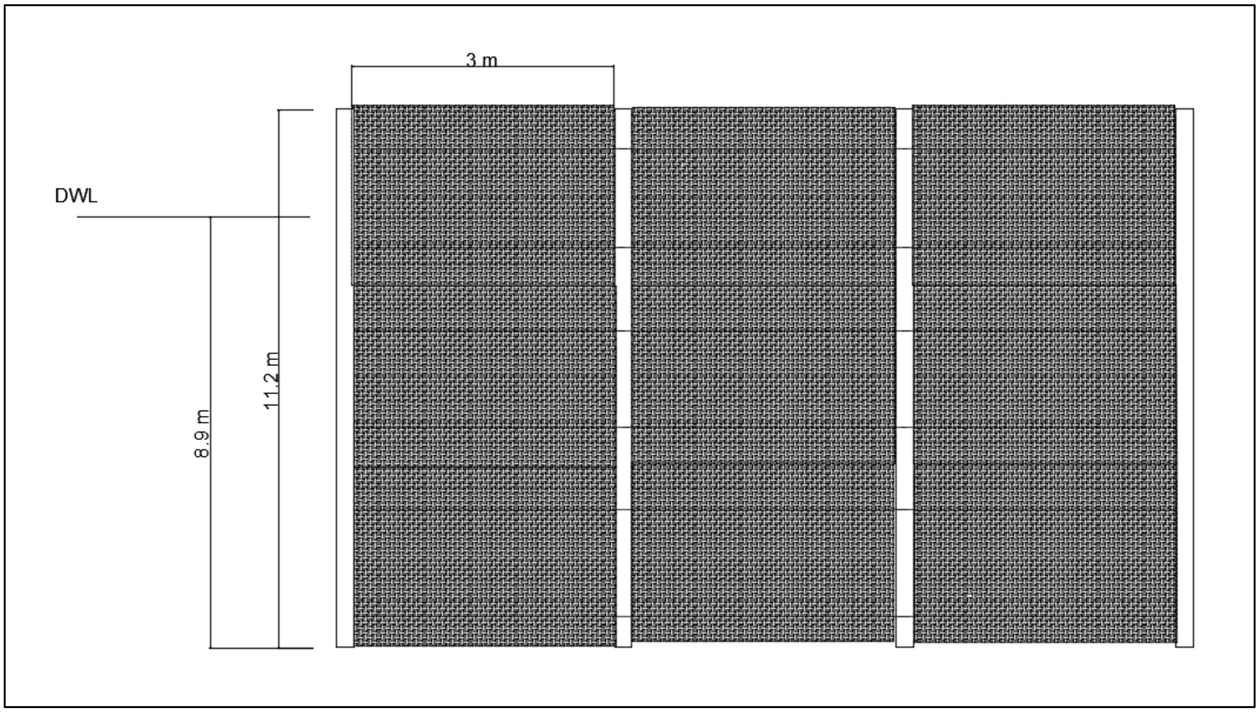

Figure 5-7 General design of the bamboo piles with panels in between

The bamboo piled walls are supposed to consist of the framing members which could be half culms. An infill material is required to fill the gaps between these culms. The infill material is extremely crucial in this case because it is not supposed to allow the clay to pass through, which will be required within the confinement as a substratum for the mangroves to grow. Most information for such infill material has been obtained from the Report by the International Network of bamboo and Rattan (INBAR). While some specific observation with regards to mangroves have been made, the description of the infill members has been reproduced as given in the INBAR report. The infill material could be any of the following:

- **Whole or halved vertical or horizontal bamboo culms, with or without bamboo mats**

  These could be driven directly into the ground. They can also be anchored between horizontal bamboo culms, acting also as bottom support. Else, mats of halved bamboo culms can also be tied using bamboo battens. A vertical arrangement of such half-culms is good for shear resistance. However, the spacing between these bamboo culms should be negligible and they should be mad completely mud-tight.

- **Split or flattened bamboo, with mats and/or plaster**

  These kind of panels may be fixed or tied horizontally to piles (posts) that are surrounding the flattened bamboo. They can be tied or stretched by a wire-mesh in order to carry out plastering. This plastering is supposed to be beneficial especially in this case because it will help in making the spacing elements mud-tight.

- **Wattle (wattle and daub, lath and plaster, quincha)**

  This is a very commonly used material in parts of India and therefore is all the more relevant in this case. It consists of coarsely woven panels of bamboo strips (vertical weft and horizontal warps). Most importantly, these are plastered on both sides, making the structure more integrated and also resilient to mud passing through it. However, such plastering is cost and labour intensive and is not required in a coastal environment, since wet clay can act as natural plaster.

  In the case of bamboo framework, substantial horizontal members made of larger split bamboo sections or two sections tied together should be used at 3 feet intervals to reduce the size of panels to be filled in with smaller split bamboo sections and earth. This bears the weight of the earth plaster adequately, which can otherwise cause frame distortion and make the plaster fall off. Instead of jute or coconut coir rope, good quality galvanized wire or nylon rope should be used for tying together the elements of the framework. However, one of the problems with using synthetic fibre is that these will have to be eliminated after mangrove growth. Even if they aren't, they will certainly not be integrated easily with the mangroves.

- **Woven bamboo, with or without plaster**

  These are very similar to the wattle mats described earlier. However, these are a more flexible design that do not have a fixed methodology at present. The plaster can be chosen as per the requirements. The plaster is to be made as a combination of mud and clay and not with the inclusion of sand as pores within sand may allow clay to pass. Additionally, making the plaster of mud and clay means that after the mangroves begin to grow, these will easily integrate with the surrounding woven bamboo mats that consist of clay-based plaster.

- **Bamboo panels**

  Bamboo panels have a greater importance for consideration here because they will not merely act as infill for the spacings but will also impart structural rigidity to the piled wall. The bamboo panels can be made as a large mesh of vertical and horizontal bamboo rods. The downside of using panels is that it may not be as impermeable as the other options.

  In conclusion, the choice of using a vertical half culms of bamboo without plastering seems as an optimal choice. It may not be as structurally sound as the one using bamboo panels. However, considering that the target period for such a structure is quite less (lesser than 2 years), this seems to be less of a worry. More important is the reasonable mud-tightness that is provided by the bamboo culms and also lower costs because there will be no weaving or plastering operations involved.

6.

Study about drainage options

As has been discussed earlier, drainage of excess water from the muddy environment is quite important in order to provide a conducive condition for growth of mangroves. In this chapter, few drainage options have been discussed, including bamboo drains, geotextile filters and gravel drains. The bamboo piles filled with suitable choir from coconut or jute seems to be the most viable option.

6.1. Split bamboo piles

Split-bamboo piles have been developed to improve the bearing capacity of soft compressible soils and to reduce settlements for various types of construction works, such as buildings, roads, etc. The hollow bamboo culms are filled up with loosely wound coconut coir and jute thread wrapped in jute fabric; holes in the culm permit the water in the soil to trickle in, thus drying out the soil and improving its load-bearing capacity (Stulz and Mukerji, 1988). An advantage of using this methodology for drainage of water from the mud is that it can be well integrated with the bamboo-piled wall design that has been considered for the confining structure.

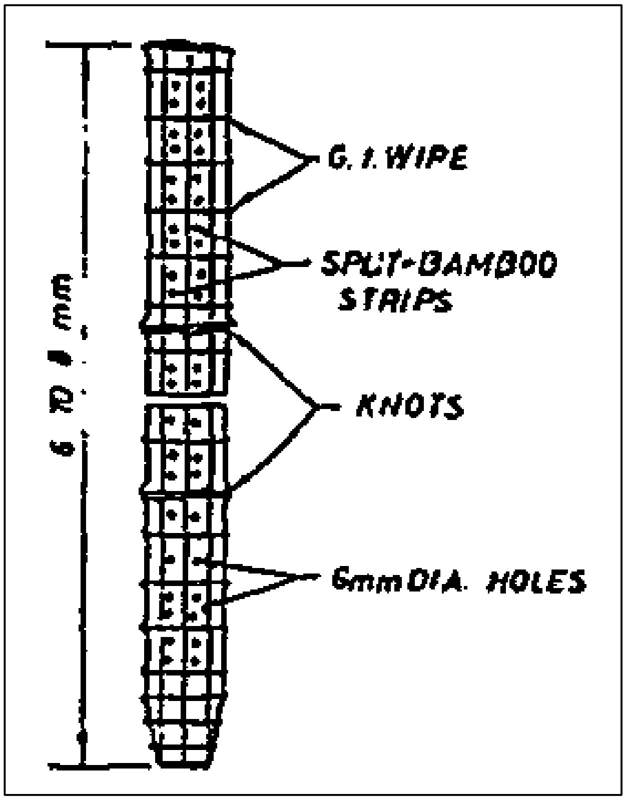

Figure 6-1 Representative design of a bamboo pile being used as a drain for the mud (Stulz and Mukerji, 1988).

In fact, the bamboo piles which have been discussed in the section for confining structures could be replicated also for the purpose of drainage. However, the draining properties of such piles as well as the infill material needs to be given special consideration in this case. In this section, infill material is not referring to the panels or members between piles but the nature of material (drainage medium) that will be filled inside the bamboo culms.

Firstly, the 'drain pipe' in this case will be the bamboo pile itself. It will function as the surrounding support for the drainage media. The bamboo culms can additionally have extremely minute holes pinned into it to allow the water to seep through.

Next consideration should be the type of drainage media that are to be used. Two alternative materials could be proposed for such purposes- coconut fibre or jute threading, both quite easily available in the geography of the proposed project. . However, if materials such as coconut choir and jute are being considered, the diameter of such choir strands needs to be ascertained. For soft soils with a top layer of about 2 m thickness, 6 mm diameter choirs have been used in the split bamboo piles. One of the reason for this would be their ability to trap particles of even larger size (up to 2 mm) while allowing water to pass, thus functioning as a filter. Apart from the size of the choir, other factors such as properties, applicability and availability have been looked at. Some salient points have been mentioned below:

6.1.1. Drainage medium: Coconut fibre

Coconut fibre is extracted from the outer shell of a coconut. The common name, scientific name and plant family of coconut fibre is Coir, Cocos nucifera and Arecaceae (Palm), respectively. There are two types of coconut fibres, brown fibre extracted from matured coconuts and white fibres extracted from immature coconuts. Brown fibres are thick, strong and have high abrasion resistance. White fibres are smoother and finer, but also weaker. Coconut fibres are commercially available in three forms, namely bristle (long fibres), mattress (relatively short) and decorticated (mixed fibres). These different types of fibres have different uses depending upon the requirement. In engineering, brown fibres are mostly used. According to official website of International Year for Natural Fibres 2009, approximately, 500 000 tonnes of coconut fibres are produced annually worldwide, mainly in India and Sri Lanka. Its total value is estimated at $100 million. India and Sri Lanka are also the main exporters, followed by Thailand, Vietnam, the Philippines and Indonesia. Around half of the coconut fibres produced is exported in the form of raw fibre (Ali, 2010).

It has been observed form previous usages of coconut fibre for purposes such as walls and roofs that there is no significant change of moisture content with coconut fibres. There is also no significant effect to water absorption on increasing coconut fibre content. These facts do imply that such fibres are possibly quite water resistant, will not absorb any water and let it drain through.

Coir erosion fabrics provide firm support on slopes and unlike other natural fibre alternatives like cotton or jute, do not degrade until 5 years. They have the necessary strength and come in a number of forms such as matting, rolls and logs and are used for soil stabilization. This indicates that such fibres will be strong enough to sustain the small serving period for the intended drains.

Such choirs are available in white, golden brown and brown varieties with most major manufacturers in India. A table of properties has been displayed below:

Parameter	White	Brown Grade I	Brown Grade II
Fibre Length	10-25 cm	10-25 cm	10-25 cm
Colour	White	Golden Brown	Brown
Moisture	15-18%	15-18%	15-18%

Table 6-1 Details of types of coconut fibre obtained from the manuals of Vaighai Agro Products Limited, India (2015)

6.1.2. Drainage medium: Jute thread

Jute fibre is 100% bio-degradable and recyclable and thus environmentally friendly. Jute is a natural fibre with golden and silky shine and hence called The Golden Fibre. Jute is the cheapest vegetable fibre procured from the bast or skin of the plant's stem. It is the second most important vegetable fibre after cotton, in terms of usage, global consumption, production, and availability. The best source of Jute in the world is the Bengal Delta Plain, which is occupied by Bangladesh and India (Textile Learner, 2012).

The fibres are extracted from the ribbon of the stem. When harvested the plants are cut near the ground with a sickle shaped knife. The small fibres, 5 mm, are obtained by successively retting in water, beating, stripping the fibre from the core and drying.

Advantages of using jute are:

a) It can withstand rotting very easily.
b) *Lignocellulosic*[1] fibres are favourably bonded with phenolic resin to have better water resistance. This will involve treatment of jute which will have to be considered prior to usage.
c) The fibres can easily withstand heat (Tara Sen, 2011).

One concern that is often cited with regard to jute is that it is easily biodegradable. However, such degradation takes a few years and therefore it is actually good in the case of this project, as it will be a good drainage material for the period purported and later will degrade and get naturally assimilated in the environment. Also, since the jute is going to get filled inside the bamboo piles, the chemical treatment is not a major problem as these chemicals, being within the bamboo, will not interfere much with the surrounding clay environment meant for the mangroves.

The kind of fibres available in India range from off-white to brown in colour and are 1-4 m long. These can also be custom-made as per requirements.

Another question that needs to be addressed in this section is about the integration of the containing and draining system. This is quite possible in the above scenario because the kind of pile being used for the wall is similar to the one that is being used for the drainage. However, the heights of the pile could be an issue.

6.2. Geotextile filters

[1] Cellulose fibres are fibres made with ethers of ester or cellulose. Lignocellulose refers to cellulose intimately associated with lignin, both of which are major constituents of jute.

Another alternative for drainage of water through the mud being used for mangrove rehabilitation is by making ditches with rock or gravel fill that will allow the water to drain but not the mud. Such drains, often known as French' drains have been used in wet areas especially to drain rainwater. However, a similar methodology might as well be adopted in this case to allow the sea water to drain.

The important parameter that needs to be ascertained is the size and density of the gravel fill that will be optimum, such that it allows little mud to pass through while draining out major volumes of water. Also, the wrapping material used for such a drain will also be crucial. As depicted in the figure below, it could be an appropriate geotextile material that can hold the rock and gravel in place. However, one concern could be that if one wraps the gravel with geotextile the geotextile actually may collect all the fine particles on its surface and get clogged up stopping the water getting to the gravel and through the drain.

The drainage mechanism being proposed here is consisting of two parts. The outer part is supposed to be a cover material that will contain the drainage medium. The cover material can consist of a geotextile or natural fibre. In some way, the bamboo pile discussed above could be considered as a special case of these cover materials. However, it must be noted that that mechanism differs in terms of the fill material it uses. In this case, the fill materials could be sand, gravel or rock.

The cover material (the wrap) will be crucial for two reasons:

1) The pore volume or voids of the drain cover must not be too high, so as to permit significant amounts of clay to pass through.
2) The strength of the material must be enough to sustain the irregularities in the infill, specially pointed ends of rocks or angular gravel.

In the Mumbai harbour, sediment particles can be classified as sand (>.063 mm), silt (0.002 to .063 mm) and clay (< 0.002 mm). The overall mean content of sand, silt and clay in cores of all sites ranged from 5.27, 72.66 and 22.10 percent respectively. Generally, the depth core fractions were dominated by silt and clay (Ajay Kumar, n.d.). While the percentages are important, in this case the particle sizes are more crucial in terms of determining the cover material. It means that the purpose is to prevent material as small as 63 micrometer from passing through the filter. At the same time, water needs to be allowed to pass through. Often, the conflict here is about which must be preferred, permeability of water or retention of clay? Ideally, both are important. However, in the specific case of mangrove growth, the retention of clay seems to be more important. Mangroves, seedling included, are widely considered to be tolerant to water. Although one of the purposes is to ensure that not too much water enters the clay in the first place (by the confining structure) but if some water does enter, it is not as major an issue as the loss of fines is.

On the other hand, maintenance dredging in Mumbai port consists of 7 million m^3 in the main channel, once in four years and 2 million m^3 in the approach and other cross channels every year (Pendse, n.d.). All of this is anyway disposed, so some may argue that such clay could be diverted to the mangrove rehabilitation compartments in case of loss of clay. However, this process consumes reasonable time, at least several days, given distances of few kilometers. This means that a delay will be introduced in the root-gripping process of the mangroves, in turn affecting the project calendar.

In brief, it could be said that the purpose of the filtering here is to possibly satisfy both clay retention and water-percolation, with more preference for the retentive property.

Are there going to be any effects due to the confining structure itself? The confining structure such as a dike is a relatively soft structure (consisting of sandy material) compared to a stiffer bamboo pile. In both cases, some confining pressures may be exerted on the soil in the vicinity of the retaining structures. High confining pressures decrease the hydraulic conductivity of fine-grained soils, increasing the potential for soil to intrude into, or through, the geotextile filter. Is this to imply that one must select a stronger geotextile filter? Maybe not. Simply placing the drainage filters sufficiently away from the confining walls and more in the central compartmental areas may be an easier way to resolve this problem. This could also be an important point for the rationale of why such drains cannot be integrated with the retaining structure.

Studies regarding the geotextile have yielded that a geotextile preferably with an O_{95} (maximum allowable opening size that will provide adequate retention) lesser than 0.21 mm and a k_g value greater than about 30 cm a day **(See APPENDIX I)**. These helps narrow down to the choice of geotextiles under consideration. These two geotextile, which are also significantly used for filtering fines, fall under the specific requirements:

6.2.1. Woven jute geotextile

Eco-friendly woven jute geotextile can be used as filter sheath in prefabricated vertical drains
(PVDs) which are installed in saturated clayey soils such as marine clay to accelerate consolidation. The PVDs made from natural materials can be used as an alternate system to polymer based PVDs. One of the main concerns in using woven jute geotextile as filter in marine clay is the fine fraction retention ability of jute sheaths. As the woven jute geotextile has wider pore sizes compared to polymer filter sheaths, there is an apprehension of clogging of core of the PVDS.

However, the woven jute geotextiles have multifilament structure with different weights (mass per unit area) and pore sizes. They are labeled as WJT775, WJT700 and WJT550 representing woven jute type having weight of 775 g/m2, 700 g/m2 and 550 g/m2 respectively. The two types of woven jute geotextiles, WJT775and WJT700 can be used as filter sheaths in making of natural PVDs made from core of coir wrapped by woven jute sheaths called as natural PVDs (NPVDs) (B S Asha, 2012).

6.2.2. Non-woven polymer geotextile

The permeability of non-woven polymer geotextile filters is ten times more than the permeability of the marine clay and thus satisfies the generally recommended criteria used for filters used in PVDs installed in clay. The O_{95} values are all lesser than 0.075 mm for these filters (Asha & Mandal, 2012). Several varieties of this type of geotextile are available at the GEOMAT portal. The following varieites are all non-woven and purported to offer good filter properties with high water permeability: Geomatex NTB10, Geomatex NTB20, Geomatex RPES, Geomatex NTI, Bontec NW, Bontec SNW and Bontec VNW. All of them use polypropylene as the polymer.

After having obtained options for the cover material, the options for the infill material need to be discussed. The permeability of sand is an order or 2 higher and gravel and small rocks is almost 3 orders higher than that of sand. The other options could be either the fills discussed previously for the bamboo pile, such as coconut choir and jute threads. Else, gravel or stones could be wrapped in the geotextile with sandfill and used as filter. It should be noted that only clean or washed gravel is being talked about here because for example, limestone gravel would end up acting almost like cement.

6.3. Ditches with gravel fill

In this case, a vertical drain is being considered therefore the head difference for the flow is high, at times it could be higher than design water levels. Here, a distinction needs to be made between geometrically closed and geometrically open filters. The geometrically closed filters have gap-sizes smaller than the particles being filtered whereas the geometrically open ones have larger gap sizes but the filtering in their case is carried out by controlling hydraulic forces on the material being filtered. For high gradients, a geometrically open filter may not be efficient at all because it would easily permit the finer material to pass through with the water (It is a case of perpendicular flow, in a sense similar to the cases considered in bed protections, except in the opposite direction). However, a geometrically closed filter could be quite useful in such a wave since it is not dictated by the hydraulic gradient. For geometrically closed filter, fine or medium sand needs to be used. Where does gravel come in? The drain could be a set of layers with gradually increasing sizes of grains. What this could mean that as long as there is fine to coarse sand in the upper layers, the bottom of the drain could be conveniently filled with gravel. This may also have advantages in terms of ease of installation. As any heavier gravel would anyway sink down due to gravity in a 'vertical' drain. The gravel also need not be too big in size and therefore could be easily obtainable. Fine washed gravel of 2-5 mm size could be one of the options. However, obtaining such fine gravel could be potentially more expensive and more specialized than using simple small stones. High-quality gravel is a product that needs to be bought especially in areas near Mumbai and may carry a lot of pre-planning and expenses. This is indeed being considered as the next option for infill material.

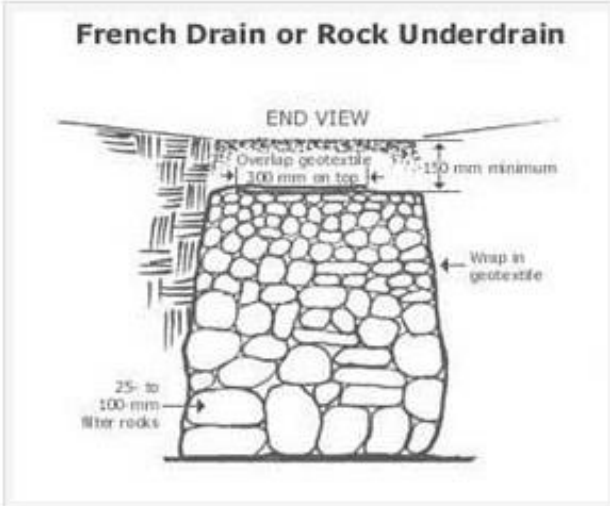

Figure 6-2 The concept of the French drain. Representative figure showing a drain using gravel or rock fill, typically referred to as a French drain (Permaculture design, n.d.).

Stones of small sizes have the same functioning that gravel would be performing in the filter described above. However, discarded stone is far easier to obtain and therefore may be a more prudent option, especially when large quantities are required. The requirements of the material, considering the maximum possible total of about 100 drains could be around 0.1-0.2 million kg, sand and rock together. This may not be a considerable amount for sand, because almost 10 times as much amount is dredged every 2 years in a nearby zone. However, for obtaining gravel, this is a considerable amount and therefore a substitution by stone may be helpful. However, the only concern should be that the geotextiles used for wrapping are strong enough to sustain the stresses due to pointed rock, if there be any.

As will be elaborated in chapter 9, out of the options considered for drainage, the one using bamboo piles is found to be optimal from the point of view of service and cost. The drains that use stone or gravel fill will require layering and therefore extensive effort and investment. Even after that, the benefit of these drains cannot be ascertained and therefore it is more reliable to use the bamboo-pile option, wherein the properties of infill material such as jute or coconut fibre have been known.

7. Cost computations

After having conducted studies regarding various design alternatives, cost computations need to be performed in order to determine the cost of each design, in this case in terms of INR[2]. As illustrated in this chapter, the cost calculations are crucial in deciding which alternative is most feasible. After all, the desire is to obtain the best designed environment for mangrove restoration with least costs.

7.1. Design components

7.1.1. Dikes

What are the contributions to the cost of the dike? One of the components is the cost of the material required for the building of the dike. In addition, there will be costs owing to transportation of such material as also to the construction costs that these structures are expected to incur.

One of the crucial developments in relation to transportation of sand to the region of the project has been that the ban on and mining within the estate of Maharashtra has been lifted in 2014, which will result in relatively cheaper availability of sand in comparison to previous case wherein the sand would have to be imported from neighbouring coastal states. In the Konkan area, one of the closest destinations where sand dredging is carried out on a large scale is the Dapoli area, which is about 200 km from the city of Mumbai.

Additionally, what is important in determining the labour costs is the schedule of the construction, which will give an hourly or daily amount of work that is required for such purposes. One of the factors to consider here is the tidal environment in the area of construction. It is usually recommended to only carry out sand placement operations during low tide, which is also relevant in this case given the considerable depth of water even during low tide scenarios. This also restricts the working period per day to about 12 hours maximum, with sometimes windows with a smaller duration if the work hours are extremely odd. Also, in this context, the machinery being used for the sand placement is quite important. For offshore placement purposes i.e. for the dike facing offshore, the placement could be done using sand-pumping from hopper dredgers. However, for areas

[2] INR refers to Indian Rupee, the local currency in Mumbai which is the location of this project. By market exchange rates, as on 5 June 2015, 1 INR = 0.014 EUR.

closer to land, it is much better to place the sand using land-based machinery such as trucks and cranes.

A rough estimate of the per meter costs for the larger wave-facing dike and the inner protective dikes has been summarized below:

Component	Per unit cost (estimates)	Per metre dike cost (offshore) (INR)	Per metre dike cost (inland) (INR)
Material Cost: Sand (Includes Transportation Cost for about 200 km distance)	INR 500 per tonne	425000	250000
Construction Cost (marine, minimum assuming small hopper dredger)	50 people with INR 2000 a day	17000	10000
Construction Cost (land-based)	500 people with INR 200 a day	17000	10000
Land Cost	INR 20000 per square meter	1400000	1100000

Table 7-1 Per meter cost break-up of dikes based on material, construction and transportation. These costs are applicable for an offshore dike of volume 420 m³ and area 70 m² and a landward dike of volume 250 m³ and area 55 m²

The values for these costs have been calculated for a dike with a slope of 1:3 and with a berm of 4 m width. This dike from section 5.3 has been found to be the most efficient in terms of material requirement, while not adding very significantly to the area footprint under the dike.

These are values for the dike costs per unit length of the dike and therefore the entire project costs will only be calculated when the alternative plan for the dike is decided upon. Among these unit costs, however, it is noteworthy that the costs for construction are quite small compared to the cost of acquisition of land and material for the dike. Given the location of the dike in a place such as Mumbai, these seem to be reasonable results for the estimated prices. In comparison, however, had these been in an extremely remote or rural coast where the land prices are miniscule and sand is available in abundance, the prices for the material can drop by the order of 2 and land prices could fall by an order of 3, making the net prices for material an land far lower than the price for construction, which would not reduce very highly.

However, in addition to these, the availability of mangrove seedlings in the near vicinity and facilities for storage and transportation of these seedlings, which could be made possible in urban areas, are not so feasible in rural lands, especially if the mangrove zones are not nearby. This may add to a considerable cost on top of the existing material costs.

7.1.2. Bamboo-piled walls

It is difficult to precisely ascertain the price of bamboos in any location, because there are several varieties of bamboo available with varying strengths. But the solid Guada bamboos have been used for reference in this case wherein the price could be considered to be about 10 INR per piece and therefore this price has to be taken to be a slightly higher estimate of the actual price.

Also, bamboo mat prices could only best be estimated by bamboo fences used in construction presently. These may not be a very good estimate but are nevertheless the closest possible approximation to the cost of a contiguous bamboo mattress.

Component	Unit Price	Price per m² for seaward wall (INR)	Price per m² for landward wall (INR)
Bamboo piles	INR 600 per piece (for 6-7 m)	1200	700
Coconut choir	INR 400 per kg	50000	25000
Jute threads	INR 300 per kg	50000	25000
Mats	INR 100 per sq.m.	2400	1200

Table 7-2 Per m² cost break-up for bamboo piled walls

The per square meter prices here can be observed to be quite less in comparison to the sand dike in part because of the widespread natural availability of bamboo in many parts of the tropical world. In fact, these materials are used in many cases for construction for the very purpose of bringing down the price of the construction.

Since bamboo availability is found to be near to the zone of the project as well as the fact that there is a huge presence of bamboo dealers in Mumbai, the transportation costs are not expected to be high. In fact, not too many strong bamboo poles will be required given the scale of the project and will be roundabout 200000 for the entire project which could be phased out. Considering a daily installation of 10 piles with the assistance of trained labour of about at least 500, the plan for the sea-facing wall can be expected to be completed in a span of 40 days, Also, considering the labour prices of about INR 500 per worker a day, these process would not be more than INR 25000 in total per day.

7.1.3. Drains

The pricing for the bamboo drains is to be carried out in a similar fashion as that mentioned above. However, the price per unit will not be exactly similar to the ones mentioned above because the length of the drains is not expected to be identical to that of the sea-facing piles. However, this length is not supposed to be very small. In comparison, it would still be similar to those walls which are at the landward side.

Component	Unit Price	Price per m² for drain
Bamboo piles	INR 600 per piece (for 6-7 m)	700
Coconut choir	INR 400 per kg	25000
Jute threads	INR 300 per kg	25000
Mats	INR 100 per sq.m.	1200

Table 7-3 Per m² cost break-up for the bamboo pile drains

Based on the drain spacings for different alternatives and the presence of compartments, the number of drains used will vary and therefore the net price of the drains will be different for different alternatives. Also, it is important here to note that the same drain prices would be much higher if a geotextile material is considered as a covering material instead of the bamboo. Although the non-wovens are comparably lesser in price as compared to the woven ones, they are still higher than the price of naturally available materials such as bamboo. These are available at prices of about INR 100 per square meter, and since quite a few layers of these may be required, such costs could be manifold. At least a cost of INR 6000 per pile could be incurred which is almost 10 times the price of the bamboo required for the same.

7.2. Layouts

7.2.1. Compartmental design

For this design, it needs to be mentioned at the outset that the costs are going to be the highest because the design is meant to have the maximum compartmentalization and therefore more building material will be required for these confining or compartmentalizing structures.

Also notable for this plan is the fact that the planning schedule needs to be more labour-intensive and works need to be kicked off at various zones simultaneously if the project is intended for completion within a 4-month window. Fortunately, this four month window prior to March is considerably conducive for construction with little rain and relatively lower temperatures.

Schedule for construction	Seaward facing dike	Landward dike

Small hopper dredger at least 2 rounds net material (2X2500)	5000 tonne per compartment (500 m)	5000 tonne per compartment (500 m)
500 trucks with a 10 tonne capacity (500X10)	5000 tonne per compartment (500 m)	5000 tonne per compartment (500 m)
Compartmental design	80 days/ compartment (100 zones)	80 days/ compartment (200 zones)

Table 7-4 Daily workload assuming a minimum 6 hour work interval, with resultant total period for construction

For the dikes as confining structures, this design plan will have the following cost estimate:

Contributions to cost	Dike Cost Sea-facing (INR)	Dike Cost Landside Alternative 1 (INR)
Material Cost Sand (Includes Transportation Cost for about 200 km distance)	500 billion	500 billion
Construction Cost (marine, minimum assuming small hopper dredger)	36 million	
Construction Cost (land-based)		72 million
NET	500 billion	500 billion
Land Cost	275 billion	

Table 7-5 Net costs of the sea-facing and landward dikes for compartmental design based on contributions from each cost component

The cost for the construction that uses bamboo piles has been summarized in the table below:

Component	Price Per bamboo column (INR)	Price per bamboo column (INR)	Seaward facing wall (INR)	Landward walls net price (INR)
Bamboo piles	1200	700	108 million	190 million
Coconut choir	50000	25000	4.5 billion	2.5 billion
Jute threads	50000	25000	4.5 billion	2.5 billion
Mats	2400	1200	800 million	400 million
NET			5.5 billion	3 billion

Table 7-6 Net costs of the bamboo piled walls for compartmental design based on contributions from each cost component

It is quite evident from the values in the tables that the bamboo pile walls are far efficient in terms of net cost in comparison to the dikes which are heavy on sand requirement. In fact, the net price for the bamboo piled walls is almost two orders lesser than the net price for the sand dike, owing to two reasons: the higher footprint of the dike and its expansion in the lateral direction while the bamboo pile is almost a 2D structure and also because the bamboo is spaced over distances.

The pricing of the drains, however, in the case of this design is expected to be much lesser than that of the drains in the other designs. In fact, this design due to its heavy compartmentalization will allow little water to seep into the paddocks and therefore may require little drainage. Even if the drains are put in place, they will not number higher than 900 in all (in the case of the bamboo pile design). If the confining structure is a dike, these drains will be even lesser, and could be numbering as low as 300 as the dikes would be covering a considerable part of the compartment. Therefore, the total cost of these will be as shown in the table 7-7:

Component	Unit price	Price per drain (INR)	Price for 900 drains (INR)
Bamboo piles	INR 600 per piece (for 6-7 m)	900	810000
Coconut choir	INR 400 per kg	25000	22.5 million
NET			23.5 million

Table 7-7 Net cost of drains for the compartmentalized design

Here, it can be observed that the prices of the drains are insignificant in comparison to those of the confining structures and therefore the determining factors for the net price will be the cost of the retaining structure.

7.2.2. Longitudinal design

For this design, the costs are going to be the somewhere in between those for the other to design plans.

Also for this plan is the fact that the planning schedule needs to be more labour-intensive and works need to be kicked off at various zones simultaneously if the project is intended for completion within a 4-month window. However, the zonation in this case could be eased out a bit in comparison to that in the case of the compartmentalized design plan.

Schedule for construction	Seaward facing dike	Landward dike
Small hopper dredger at least 2 rounds net material (2X2500)	5000 tonne per compartment (500 m)	5000 tonne per compartment (500 m)

500 trucks with a 10 tonne capacity (500X10)	5000 tonne per compartment (500 m)	5000 tonne per compartment (500 m)
Compartmental design	80 days/ compartment (100 zones)	80 days/ compartment (100 zones)

Table 7-8 Daily workload assuming a minimum 6 hour work interval, with resultant total period for construction

For the dikes as confining structures, this design plan will have the following cost estimate:

	Sea-facing Dike Cost (INR)	Dike Cost Landside Alternative 2 (INR)
Material Cost Sand (Includes Transportation Cost for about 200 km distance)	500 billion	350 billion
Construction Cost (marine, minimum assuming small hopper dredger)	36 million	
Construction Cost (land-based)		36 million
NET	500 billion	350 billion
Land Cost	275 billion	

Table 7-9 Net costs of the sea-facing and landward dikes for longitudinal design based on contributions from each cost component

The only difference that is clearly observable in these costs from that of the compartmental design is the price of the landward dikes. Since this design does not have any lateral compartments, only longitudinal ones, the costs incurred on the landward dikes is lesser in comparison to the previous design plan.

The cost for the construction that uses bamboo piles has been summarized in the table 7-10:

Component	Price Per bamboo column (INR)	Price per bamboo column (INR)	Seaward facing wall (INR)	Landward walls net price (INR)
Bamboo piles	1200	700	108 million	100 million
Coconut choir	50000	25000	4.5 billion	1.2 billion
Jute threads	50000	25000	4.5 billion	1.2 billion
Mats	2400	1200	800 million	200 million
NET			5.5 billion	1.2 billion

Table 7-10 Net costs of the bamboo piled walls for longitudinal design based on contributions from each cost component

Again, it is evident that due to lesser compartmentalization in this design plan, the cost of the inner walls has been lesser. How much lesser is the real question here. The cost is only 10 percent lesser in that it saves 1 billion from a net cost of about 10 billion.

The pricing of the drains in the case of this design plan will be higher. Since there are no lateral compartments, drains will have to be placed along these boundaries as well, taking the net number of drains to about 1200. However, since these values are not comparable to the costs incurred on the confining structures, such tables may only be taken as a necessary calculation but not affecting the final decision on the selection of the design plan.

Component	Unit price	Price per drain (INR)	Price for 1200 drains (INR)
Bamboo piles	INR 600 per piece (for 6-7 m)	900	1 million
Coconut choir	INR 400 per kg	25000	30 million
NET			31 million

Table 7-11 Net cost of drains for the longitudinal design

7.2.3. Single-confining structure (vertical drain) design

For this design, the costs are going to be the lowest also because the design is meant to have the minimum compartmentalization and therefore least building material required for these confining or compartmentalizing structures.

There will be no construction of landward dikes in the case of this design and therefore all costs related to these landward dikes will be null and void. This is also one of the major reasons why this design plan would be the cheapest from the point of view of price of the confining structures.

Schedule for construction		Seaward facing dike	Landward dike
Daily Work Interval (minimum)	6 hours		
Small hopper dredger at least 2 rounds net material (2X2500)	5000 tonne	5000 tonne per compartment (500 m)	0
500 trucks with a 10 tonne capacity (500X10)	5000 tonne	5000 tonne per compartment (500 m)	0
Compartmental design		80 days/ compartment (100 zones)	0

Table 7-12 Daily workload assuming a minimum 6 hour work interval, with resultant total period for construction

For the dikes as confining structures, this design plan will have the following cost estimate:

	Seaward-facing Dike Cost (INR)	Dike Cost Landside Alternative 3 (INR)
Material Cost Sand (Includes Transportation Cost for about 200 km distance)	500 billion	0
Construction Cost (marine, minimum assuming small hopper dredger)	36 million	
Construction Cost (land-based)		0
NET	500 billion	0
Land Cost	275 billion	

Table 7-13 Net costs of the sea-facing and landward dikes for non-compartmental design based on contributions from each cost component

The cost for the construction that uses bamboo piles has been summarized in the table below:

Component	Price Per bamboo column (INR)	Price per bamboo column (INR)	Seaward facing wall net price(INR)	Landward walls net price (INR)
Bamboo piles	1200	700	108 million	0
Coconut choir	50000	25000	4.5 billion	0
Jute threads	50000	25000	4.5 billion	0
Mats	2400	1200	800 million	0
NET			5.5 billion	0

Table 7-14 Net costs of the bamboo piled walls for non-compartmental design based on contributions from each cost component

The bamboo piled walls, as has been previously discussed, are much cheaper in comparison to the dikes.

This design, however will be the one that uses the maximum number of drains. The number of drains in this design could be estimated to be as high as 2500. This design can be considered to be drainage-intensive and the maximum costs are expected to be expended on the drains. However, given the order of these costs, however high, these are not comparable to the costs of the retaining structures.

Component	Unit price	Price per drain (INR)	Price for 2500 drains (INR)
Bamboo piles	INR 600 per piece (for 6-7 m)	900	2.2 million
Coconut choir	INR 400 per kg	25000	62 million
NET			64 million

Table 7-15 Net cost of drains for the non-compartmentalized design

8. Design Guideline

One of the prime goals of the research was to come up with certain points of guidance about the design and decision-making for mangrove rehabilitation projects along tropical coasts. This chapter lays down a guideline for the design of such mangrove rehabilitation projects, mentioning the essential queries and recommendations for each step of the design.

Prior to the concluding section of the report, an attempt has been made to outline a design guideline for projects intended for mangrove rehabilitation using dredged mud in coastal areas. The cases undertaken as a part of this study has been considered as an indicative case for such a development, but must not be taken as a strictly representative cases for the reasons enlisted below:

- The case in Mumbai is one based in a predominantly urban setting. The prices, choices and design decisions may be affected by a rural or an urban setting. In fact if the same project were being considered in a small village 200 km south of Mumbai, the land and sand prices would be vastly different and therefore may affect certain design decisions.

- Also, the case considered is that in the commercial capital of a developing country and therefore the investment choices may depend on the economy and capacity of the country in question (One could consider Australia or the US in contrast). Having said that, this project is also being considered for Mumbai where the corporations and local government bodies (MMRDA) have considerable financial inflows coming to them and therefore local government capacities are also crucial.

- Another important distinction is that of space requirement. The 25 km coastal stretch along the Thane creek is unique for an overburdened and populous city like Mumbai. Such space availabilities are rare in other urban settings, a case in point being Singapore.

- The climatic situation in the region is also one that distinguishes Mumbai. Not every tropical region has a monsoon-type of climate with predominant rains only in a 4-month window. Therefore, the project plan and scheduling will differ for areas which survive under different climatic conditions.

- Finally, the cheap availability of certain construction materials such as bamboo is quite special in this context and may be relatively costlier in places where such bamboo crop is not in abundance. This will, in turn, dictate the structural choice as well as the material cost.

Having summarised the distinctions, the general design guideline can now be laid out based on the design analysis above:

Step 1: Study of case in question

- An assessment of the specific region under consideration must be the foremost step in the design of such a project. The geographic and climatic conditions must be the first ones to be assessed as they will decide whether mangrove growth indeed is possible in the area.
- Next, the space availability must be estimated. A rough idea about the area and scale of the project needs to be built. This information will be crucial when deciding on the design options, especially the format of compartments that such a project would require.
- The following step must be to lookout for the availability of considerable quantities of mud within the area or in close vicinity as it has been shown that mangroves, especially for germination, require a predominantly muddy (or silty) substratum.

Relevant questions to ask
What geographical latitude is the region under consideration?
What kind of climate characterises the region?
How much area/space is available for the project?
Is the project area predominantly along the coastline, or slightly offshore?
Is there mud available in the near vicinity?

Recommendations
Identify set of mangrove species based on geographical area
Analyse the climatic calendar to decide periods conducive to mangrove germination

Estimate scale of project in terms of coastal length and offshore width (this will help envisage the kind of compartmental structure in further design)

Assess any regular dredging practices in the region to estimate mud availability

Step 2: Getting an idea about the boundary (environmental) conditions

- The tidal situation of the area is extremely important in deciding the nature of the project. As demonstrated through the case of Mumbai, the tide in fact dictates several factors, starting from the mangrove species, the structural design, the schedule as well as construction methods. Therefore, this is one data that needs to be made available and must be taken as an important indicator for further design.
- The wave climate must also be estimated. Here, it is important to stress that wave climate includes (sometimes more importantly) locally generated waves. For such a scenario, not only the wave data but the wind data will also be required for estimation of the local wave conditions. As shown in the case of the basin of the Thane creek, the local waves are in fact the waves that decide the design criterion.

Relevant questions to ask
What is the tidal range in the region?
What are the tidal current velocities in the area?
What are the wind conditions like, and therefore what is the local wave climate?
What is the pattern and seasonality of rainfall?

Recommendations
Obtain tidal values for measured data and note differences between high water and low water (this will be crucial for mangrove project planning)
Estimate tidal current velocities within the basin to get an idea of any major disturbance to mud due to currents
Access wind data and conduct analysis to estimate locally generated significant wave (do not directly consider wave data especially when there is a protected basin). The significant wave should further be converted to maximum wave height for design purposes.

Step 3: Design plan for mangrove rehabilitation

- The design plan in most cases consists of alternatives on two fronts. One set of alternatives is based on the nature of the structure and the second set of alternatives is based on the layout of these structures.

- The nature of the structure could depend on what kind of material is used and what kind of structure is built. The two most probable alternatives for such structures- dikes and bamboo-piled walls have been considered in the case studied in this project. It has also been shown that the bamboo-piled wall proves to be far more economically effective when compared to the sand dike. This indicates that, only when sand availability is extremely cheap and bamboo is almost impossible to obtain and given that space requirements are not too constrained to allow wider dikes, the sand dike will become the preferred alternative.

- The second set of alternatives is based on what layout will be chosen for such a design, whether it must be one full stretch or whether it must be a plan broken into compartments or whether it could be a combination. In this particular case of Mumbai, a more efficient alternative combining both the plans has been proposed. It seems to be the most feasible choice in most cases where the project will be in areas adjoining the coastline. However, a set of alternative options must always be developed in order to conclusively state which alternative is the best.

Relevant questions to ask
What are the alternative structural designs possible?
What materials are to be used for such designs?
What alternative patterns of arrangement are possible for these confining structures?
Could any arrangements be used in combination?

Recommendations
In many cases, bamboo-piled walls would be the preferred design although an alternative sand dike design may be considered in parts where sand is cheaply and abundantly available and there are no space constraints
When considering the arrangement, making compartments yields an increased cost

However, for mangrove seed germination, compartments are considered as an aiding factor

A combination of a compartmental and non-compartmentalized design based on drainage and submergence criteria will usually be an optimal solution

Step 4: Design of drainage

- The next step must be the conceptualization and design of the drainage mechanism that needs to be put in place so as to drain water from the area designated for mangrove growth. From the case studies, it has been quite clear that a drainage using bamboo piles seems to be the most effective option. Although drains using geotextile filters and gravel drains could be considered, these may only be chosen in cases where there is no bamboo availability. In such cases, however, a retraction plan for the drains must also be proposed because these drains cannot stay in the natural environment once the mangroves have grown.

Relevant questions to ask

What are the alternative drainage mechanisms possible?

What materials are to be used for such mechanisms?

How efficient are these mechanisms, cost-wise and drainage-wise?

Are these mechanisms such that they can be integrated with the natural mangrove environment?

Recommendations

Conclusively, bamboo piles/rods are found to be the most efficient mechanism for such purposes, considering their cost, efficiency and ability to integrate with natural surroundings

In cases where bamboo is very hard to access and expensive, a geotextile filter cover can be used for the drains with inside filling of jute or coconut choir

Gravel filters seem to be the last and least efficient option. While special layered designs may work better but that is unknown at this point, and therefore such filters may not be worth considering for an immediate project.

Step 5: Cost planning and scheduling

- Once the design for the confining structure and the drainage mechanism have been decided, the costs of the project must be finalised. It must be noted that an approximated cost calculation must also be done in the design alternatives stage because it is one of the major deciding factors in the choice of the alternative.

- Also, a timeline or a rough schedule needs to be proposed based not only on the project's scale and requirements but also taking into consideration the specific climatic calendar of the region.

- Finally, some indication must be given about the possible construction practices for the project. For example, depending on whether it will be a landward or a sea-based construction, appropriate land or marine equipment must be proposed. These may also be crucial for the cost because the labour (skilled or unskilled) required for these equipment will also add to the cost.

Relevant questions to ask
What would be the estimated costs of the confining structures? How much would the drainage practice cost? Which of costs would be a determining factor for the ultimate design? Are the cost differences between alternatives only marginal or are they significant enough to discard a particular alternative? Could the costs be optimized by combining alternatives?

Recommendations
It has been found that for most projects of a medium to large scale (tens of kilometres of coastline), the retaining structures will be the determinants in terms of costs. The drainage mechanism costs are insignificant in comparison. Cost calculations also indicate that in places where natural construction materials such as bamboo and fill materials such as coconut or jute are available, these will yield the design with the least net costs, considerably lower in comparison to sand-based alternatives. When combined in an effective manner, certain compartmental patterns can lead to acceptable costs, with only 10-15 % higher investments in comparison to non-compartmentalized designs.

As for the scheduling, a calm period of at least 4 months needs to be allowed post plantation for the first roots to grow. This should be followed with at least 2-3 years of protection by confining structures. Also, construction intervals must be included in the time-plan.

The steps detailed in the above sections are meant to be merely a guidance based on the study of a particular case. While most of these recommendations are believed to hold good in many similar circumstances, there may sometimes be variations. While it is always preferred to carry out an entire design process from scratch, these guidelines and recommendations are meant to narrow down the design process to certain limited alternatives that have been found to be better when compared to others, in terms of cost or service. For example, the study has clearly showed the differences between sand-based structures and bamboo-piled structures in terms of cost. So when confronted with a dilemma, the designer could take reference from these studies, which will help to make a choice in an informed and efficient manner. Similarly, the studies show, for example, that drainage costs are marginal compared to retaining structure costs, which will also help the designer realize that the structure is finally going to be the decider when it comes to price comparisons. Such indications could be vital not only in making the design process more robust but also in making it faster, thus impacting overall timelines of projects. Also such a study and ensuing recommendations are important because there is currently not much guidance material available for mangrove rehabilitation projects. By addressing at least a few issues related to such projects, this study may be helpful in aiding such projects in many tropical countries, some of which do not mostly have the resources and capabilities to conduct such analyses. However, having concluded this part of work, it is also important to note that such a guideline is by no means meant to be comprehensive or fool-proof. In fact, additions and improvisations may be expected, based on further research as well as in the form of feedback obtained from real projects.

9. Conclusions, Assumptions & Risks

In this concluding chapter, inferences have been drawn about best options from each set of alternatives. Furthermore, a summary of important assumptions in the study has also been presented. This chapter also explains the risks associated with mangrove rehabilitation projects, owing to geographical, financial or planning considerations. Finally, this chapter offers a brief summary of the study.

9.1. Section Specific Conclusions

9.1.1. Confining structure

Among the studies carried out about confining structures for mangrove rehabilitation, the costs clearly indicate that the retaining structure must be one that uses split-bamboo piled walls. The two-fold advantage of using these is the fact that they will occupy a smaller space on the ground and therefore allow a larger area for the plantation of mangroves and secondly they are far cheaper than the option of constructing dikes. Also, the construction schedule of the bamboo piled walls can be a much more eased out process in comparison to that in the case of dikes where operation will have to be carries out simultaneously for different compartments if the project were to be completed within one season.

The only downside of using split bamboo piles is that it leaves higher room for the area to be drained and hence a higher number of drains will have to be installed. However, given the incremental cost of these drains and their small number in comparison to the total number of bamboo piles to be installed for the wall, the increased drainage requirement is not a major concern in the selection.

Factor	Dike Design	Split Bamboo piled walls
Area footprint	High	Very low
Material costs	High	Low (2 orders lesser)
Construction schedule	Busy and long	Eased out and short

Number of drains	Small	Large

Table 9-1 Qualitative comparison of dike design and split-bamboo pile wall design for compartments and confining structures

9.1.2. Drains

In terms of the selection of drainage procedure, the option pertaining to the use of sand-and-gravel mixture as drainage material can be ruled out on the basis of the complex procedure involved in layering these mixtures appropriately within the drain. Even then, there will be no surety of their serviceability. Also, considering the previously outlined costs for sand (in the case of dikes), the use of sand within drains could prove to be an extremely costly affair.

The use of a bamboo drain seems to be the most efficient and cost-effective option available. As has been observed, the cost of the bamboo as a cover material is 10 times lesser than the cost of any other non-woven geotextile and hence would be a huge saving to the project. Also, the use of natural materials such a bamboo and jute means that these will be easily integrated with the mangrove environment and will require no removal after the mangroves are grown. The option of using geotextiles is problematic in this sense too, apart from the factor of being pricy.

Factor	Bamboo drain with natural jute/coconut choir infill	Geotextile-covered drain with natural infill	Geotextile-covered drain with sand-and-gravel infill
Effectiveness	High	High	Low
Integration with nature	Good	Not good	Not good
Cost	Relatively cheap	Relatively expensive	Highly expensive
Integration with confining structure	Good	Not good	Not good

Table 9-2 Qualitative comparison of various drainage alternatives considered for study

9.1.3. Alternative design plans

Among the three alternative design plans, a comparison is made harder by the fact that the degree of aid for the mangrove seedlings due to the presence pf compartments is not known. However, what is known is that the lesser these seedlings or propagules are allowed to move, the higher the chance that they will hold roots within the underlying soil. In addition to this, the compartmental system is also known to aid drainage through the extensive walls that will be surrounding the compartments, which will also in turn act as drains.

The comparison on the factor of costs is however quite simple. It is evident that lesser division in compartments will lead to lesser net costs, which indicates that a design with just an outermost confining wall and inner drains is the cheapest from the point of view of project costs. The costs for the various design vary as shown below (the costs of the drainage have been ignored as these are not significant compared to the cost of the retaining structures):

Design	Net Cost (dike)	Net Cost (bamboo wall)
Alternative 1 Compartments	1000 billion	8.5 billion
Alternative 2 Only longitudinal walls	850 billion	6.7 billion
Alternative 3 Only outermost confining wall	500 billion	5.5 billion

Table 9-3 Cost comparison of various alternative layouts with different design types

All these process are in addition to a net land piece to the tune of 275 billion. It is clearly noticeable as mentioned above that the choice of the confining structure has to be the bamboo wall. However, when one looks at the costs for the bamboo-piled walls, the incremental differences in various design do not seem very high although they are considerable. For example, the compartmentalized plan is about 50 percent costlier than the one with no divisions at all. However, the plan with longitudinal walls is not only about 20 percent higher or lower in cost than the other designs. The trade-off for giving up compartments will have to be the induction of a higher number of drains. Another factor will be that the mangrove seed germination may not be as efficient as that in the compartmentalized design. It has been noted earlier that highly inter-tidal areas are not very suitable for mangrove seed germination. However, the outermost wall, in the absence of extreme wave conditions, is expected to provide such an environment. And, if a high number of seeds are planted, there is a good chance that in the right season, a considerable number of mangroves may germinate. However, post-germination, these mangroves will also need protection from a high-wave environment. So, a plan with the partial implementation of compartments and drains could be an appropriate one.

Factor	Alternative 1	Alternative 2	Alternative 3
Space availability	Low	Medium	High
Cost	Expensive	Medium	Least expensive

Environment for mangrove growth	Most conducive	Partly conducive	Least conducive
Number of drains	Low	Medium	Significantly high

Table 9-4 Qualitative comparison of alternative layouts based on various factors

9.2. Combined Design

As mentioned earlier the combined design could be one that takes advantage of both the design practices that are supposed to aid each other, compartmentalization and individual drains. One of the concerns which supports compartmentalization is too much of water intrusion, especially in the inter-tidal zone which will be further seaward. In order to tackle this issue, the compartments on the seaward end could be retained. Also, having compartments on the seaward end means that the walls could be positioned there and not many intermediate drain installations will be required. These walls could be made in their entirety and then installed whereas individual drains may require personnel going offshore and installing these drains. On the landward side, however, the presence of compartments is not so much necessary as here will be lesser presence pf water and hence just the individual drains may suffice. These drains will also be easier installed since the land is expected to be completely exposed during the low waters.

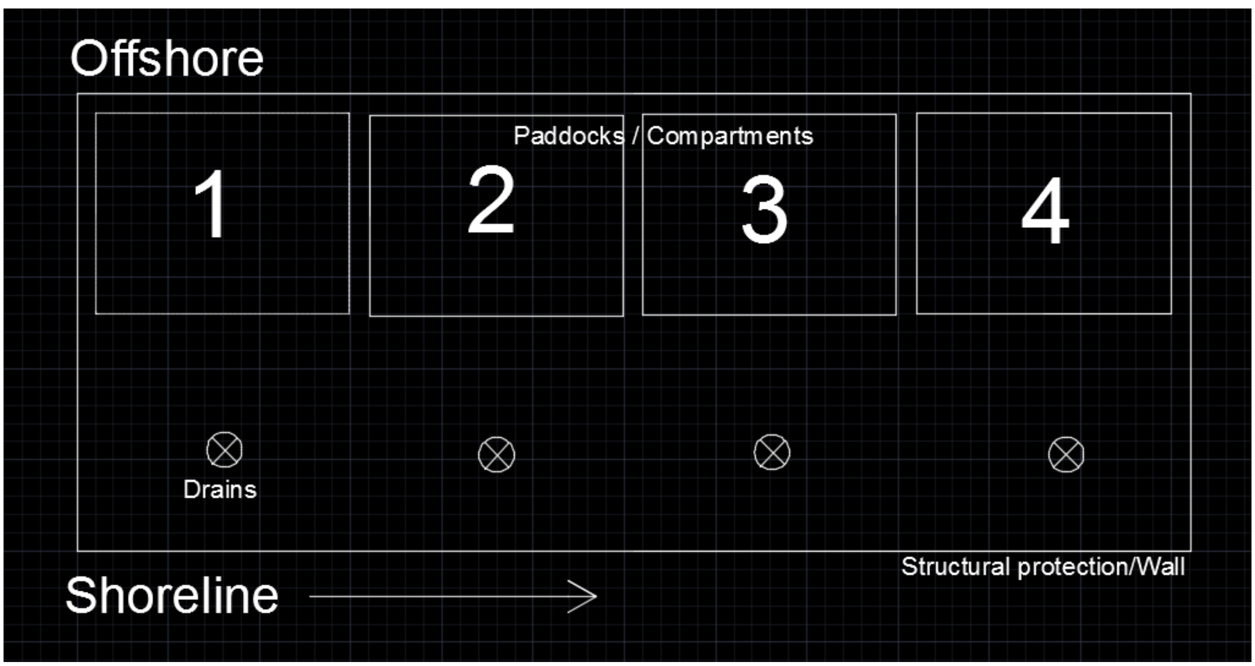

Figure 9-1 Combined design consisting of a partly compartmentalized design along the offshore end with a predominant vertical-drain layout at the landward end. This is one of the optimal plans that could be developed though depending on specific situations, other combinations may also prove to be beneficial.

The combined cost of these could be estimated to be the sum of approximately half of the previously designed plans and hence would lead to, in the case of bamboo piled walls, a net cost of about 6.5 billion, which when compared to the cost of 5.5 billion to be incurred on the no-compartment plan, does not seem much higher. This extra investment of about 15 percent higher cost may give better returns in terms of mangrove seed germination, which is the ultimate purpose of the project. Also, the net requirements for the drains will be reduced considerably to about 1050 drains (450+600) which is less than half of what would be required if there would be no compartments at all.

Therefore, this is a plan that could be found to be effective in terms of design and cost. Though such a combined design could be used for most projects, combinations of suitable layouts may vary on a case to case basis.

9.3. Summary of assumptions

9.3.1. About mangroves

- The characteristics of mangroves vary for different species of mangroves. Also, the parts of the inter-tidal zone that these species inhabit varies depending on their ability to tolerate moisture and salinity. In the case of this study, one species of mangroves has been assumed, as is the case in many projects where purposeful mangrove plantation is carried out.

- Salinity considerations have not been taken into account. In other words, a uniformly saline intertidal area has been assumed. Saline intertidal areas with no freshwater input are considered counterproductive for seed germination. However, the species that has to be selected for a project like this, which has to be similar to the existing most common mangrove species in the region, is *Avicennia Marina* and this species is believed to have high salt-tolerance.
- Also, growth trajectories for mangroves, according to different studies, point to varying periods of germination as well as growth of the root system. In this study, it is safe to state that for the species under consideration, 4-5 days are taken for the germination and about 4 weeks for the roots to grow. The root system, however, may take longer than this duration to strengthen and really catch hold. This is also why a large period of about 4 months has been allocated as the plantation window and also the reason why the service periods of the confining structures have been extended to at least a year’s duration.

- It is important to note at this juncture that the retaining structures play a vital role because for the germination of the mangrove seedling, a highly tidal environment is not conducive and the retaining structure by virtue of its presence avoids excessive tidal activity within the protected plantation zone.

9.3.2. The design boundary conditions

- The estimates for the wind and resultant wave conditions are based on readings of a particular year and have been assumed to be representative of the annual wind and wave climate in the region.

- Also, the selection of four months extending from March to June as the calmest period is based on the wind readings. It needs to be clarified that these windows may depend largely on the construction schedule and could be subject to changes. In these cases, thought the predominant wave heights are unlikely to be higher than the selected period (as indicated by the monthly charts), it is expected that the probability of occurrence of these higher waves will be more and therefore the confining structures may be subject to the design conditions for a larger duration than in these 4 months. Does this constitute a problem for the project? Not per se, because the structure has been designed keeping in mind the possible wave height for a 1 in 50 year situation. However, there is always a possibility, however tiny, of this extremity occurring in any of the selected windows, calm or not calm.

- The extents of the project area have been estimated based on the intertidal extremities of the Thane Creek as also on the feasibility of construction or plantation in these depths. For example, the offshore extent of about 100 m could be reasoned on the basis that the depths for these zones generally lie around 2 m where plantation or even mud-laying operations could be carried out during low-water periods with relative ease. Beyond these regions, as the water gets deeper, the conditions present a challenge both from a construction point of view as well as from the point of view of growth of the mangroves.

9.3.3. The alternatives and choices

- The alternatives have been selected based on a broader idea of division of plantation land. Cases relating to compartmentalization as well as those without any sections have been considered separately. For each of the alternatives, a retaining structure as well as a drainage plan has been worked out.

- The retaining structure may not be a contiguous long structure, as in the case of the longitudinal design alternative or the vertical-drain alternative. What these designs intend to propose is a phase-wise extension of the structure in the longitudinal direction, in the same manner as that in which a compartmental design would gradually progress from compartment to compartment when under construction.

- The final idea has been to work pout each alternative individually and then look at the best alternative form the perspectives of drainage, confinement as also the possibility of mangrove germination, which is again to a great extent linked with confinement.

- The combined design is not intended to be a design as worked from the very basics of the plan, but as the name suggests, it is meant to be a combination of the best picks from each of the confinement, drainage and compartmentalization alternatives.

9.3.4. Cost estimations

- The cost estimations are also meant to be rough calculations that are not a precise assertion of the cost of the project, in part or in full. However, although the absolute values of the cost estimates may be arguable, the values are intended to give a relative comparison between the different alternatives at hand. In this context, it is important to emphasize that, if for example, the uncertainty in cost were high, to about say 50 %, it would still give a 1.5 time inflated value for all the cost estimates in each alternative, thus making the resultant comparison still justifiable since all values will be inflated by a common multiplier.

- The estimates for land prices are based on current prices available for land areas adjoining the coast. The cost of the specific coastal zone may differ from this and may also depend on factors such as whether the available land comes under the purview of the government, and if at all, under the central government of India or the state government of the Maharashtra state. Other factors such as planned tourist zones are also expected to impact the land prices.

- The material cost of the dike is largely assumed to be contributed by the sand requirement and not the clay-silt component. One reason for this is that the clay and silt is readily available in the surrounding region and secondly also because these from a small component of the construction material for the dike. Prices for the grass seeds or grass plantation have been neglected due to the reason that Bermuda grass is widely available in India.

- Also, costs related to mangrove seeds have been neglected as the mangrove seeds are supposed to be available within the vicinity of the site where considerable mangrove swamps already exist. Also, storage and transportation of these seeds (expected to be about 4-6 days) is expected to be over extremely short distances. And since seeds aren't a particularly heavy material to carry and are not to be carried in large amounts over long distances, the increment in costs due to these is expected to be only marginal. Additionally, these seeds will be used in all cases and in all alternatives and therefore the inclusion of these costs will be similar in all cases and therefore is not expected to affect the comparison between alternatives, which is one of the major purposes of the cost estimation exercise.

9.4. Potential risks

When considering a large project of mangrove restoration that requires huge investment and planning, it is important to mention certain major risks associated with the project and if there are any possible ways to reduce them.

9.4.1. Storms and cyclones

Mumbai and its surrounding region does have a history of cyclones, the latest one being cyclone Phyan that hit the city in 2009, damaging also parts of Thane, which is near to the studied project site. Although fully grown mangroves are known to be resilient to cyclones and tropical storms, if such an event occurs before the trees are fully grown, it could possibly lead to considerable damage.

With cyclones such as Phyan, there could be a dual effect, with a rainfall of about 25 cm in a day and winds of about 60 km/hr in the local basin. Therefore, the dike slopes will inevitably fail, as also would the bamboo-piled walls which are weak against storm and typhoons. It is important to know that the confining structures are designed for only a 1 in 50 year storm, unlike others which are designed for longer storm durations. An event that is rarer than that could effect in failure of the structure. Also, a 10 % probability of failure means that there is a chance that such an extreme event could occur within 5 years after the structures are built. However, it must also well be noted that the structure could have been built with just a 10 year storm and a 1 year lifetime and this could serve the basic purpose. But since the design criteria and resultant structural dimensions for a 1 in 50 year storm were found to be only marginally higher, a greater-than-required lifespan of the structure has already been chosen **(See Appendix D)**.

9.4.2. Hike in land prices

Another major risk associated especially with projects in highly commercial cities such as Mumbai, the case of this project, is the constant rise in land prices. Especially if a tourist zone were to come up in the areas surrounding the project, the land prices will shoot up manifold. At current estimates, the net land price is about half that of the dike cost (275 billion against 500). But, if the land price becomes significantly higher, say 5 times of the current prices, it could affect the entire decision-making process. If the dike costs end up being a small proportion of the total land price, then designers could argue for the dike alternative against the bamboo-pile alternative. The deciding factor between the two choices will then no longer be the cost but the utility. Dikes being more impermeable than bamboo walls (which will still have some gaps), they may serve as a better confinement and therefore could then be chosen as a better alternative.

9.4.3. Changes in schedule

Another reason that would increase the risk of damage is the change in the schedule of the project. Though the project should ideally be planned for the calmest duration of the year i.e. March to June, there could be delays induced which would require moving the project.

The season that immediately follows the 'calm' period is the monsoon. Since the monsoon is predominantly a south-west phenomenon in terms of direction, it is not characterised by strong winds in the direction of the project site. However, the real concern in these cases would be the rainwater run-off. This has been addressed in Chapter 5 but in short, this problem needs to be tackled in two ways. Firstly there should be a grass mat developed on the dike slopes and if this is not the case, then the slopes which have been seeded with grass should be protected using reed mats or jute nettings until the grass grows. Bamboo piles, on the other hand, are resistant to rain and in fact help in draining water and therefore the monsoons will not particularly be as much a concern as in the case of dikes.

If further delays are introduced and the project has to shift to the Indian winter season i.e. November to February, it could create more serious risks. This is the period which has maximum wind activity form the north-eastern direction and will affect the wave climate at the project site directly. During this period, almost all of the wind-speeds (of 7.5 m/s and above) are expected to blow from the north-east to south-west, with an average probability of 5 % (BMT Argoss), in contrast to the very small (less than 1 %) probability during the calmer season. This in turn means that the design condition could occur with a much higher chance during this season, and also could be exceeded multiple times. In order to avoid this risk, it is wiser that if the project gets delayed by more than 8 months one may rather wait for the entire year until the next year's calmer months begin. For example, if a project starting March 2016 were to be pushed to November that year, it is better to begin in March 2017 instead of November 2016.

9.5. Brief Summary

In the end, some important junctures of the study need to be stated briefly. The intended goal has been to develop a design guideline for projects in tropical areas that deal with restoration of mangroves. Foremost, the cheap availability of mud in the nearby areas needed to be assessed. The design procedure included two major components, the confining structures for the project area and the drainage mechanism. For retaining structures, options such as dikes and bamboo piled walls have been considered and the bamboo walls have been found to be more cost-effective for the scale and purpose of such projects. Similarly, from multiple alternatives for drainage, the ones that use bamboo poles with natural infill are the most feasible, owing to low cost and easy integration with surroundings. However, despite having inferred these conclusions, several assumptions specific to the case study in Mumbai have also been outlined. Also, risks relating to extreme events, prices and schedule have been addressed and their influence on design decisions has been explained. Finally, it is important to emphasize that in projects of such scale and investment, different design and risk factors carry different weightage in the decision-making, low price and easy availability of materials such as mud and bamboo being crucial. In conclusion, it one must reiterate that it is through a multi-faceted approach involving technical knowledge, commercial feasibility and environmental viability that a final design for such projects has to be developed.

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Appendix A Thane Creek as a Short Basin?

Length of basin (east-west average): 40 km

Tide celerity = √(g*h)

Where, g: acceleration due to gravity = 9.8 m/s

h: water depth = 12 m

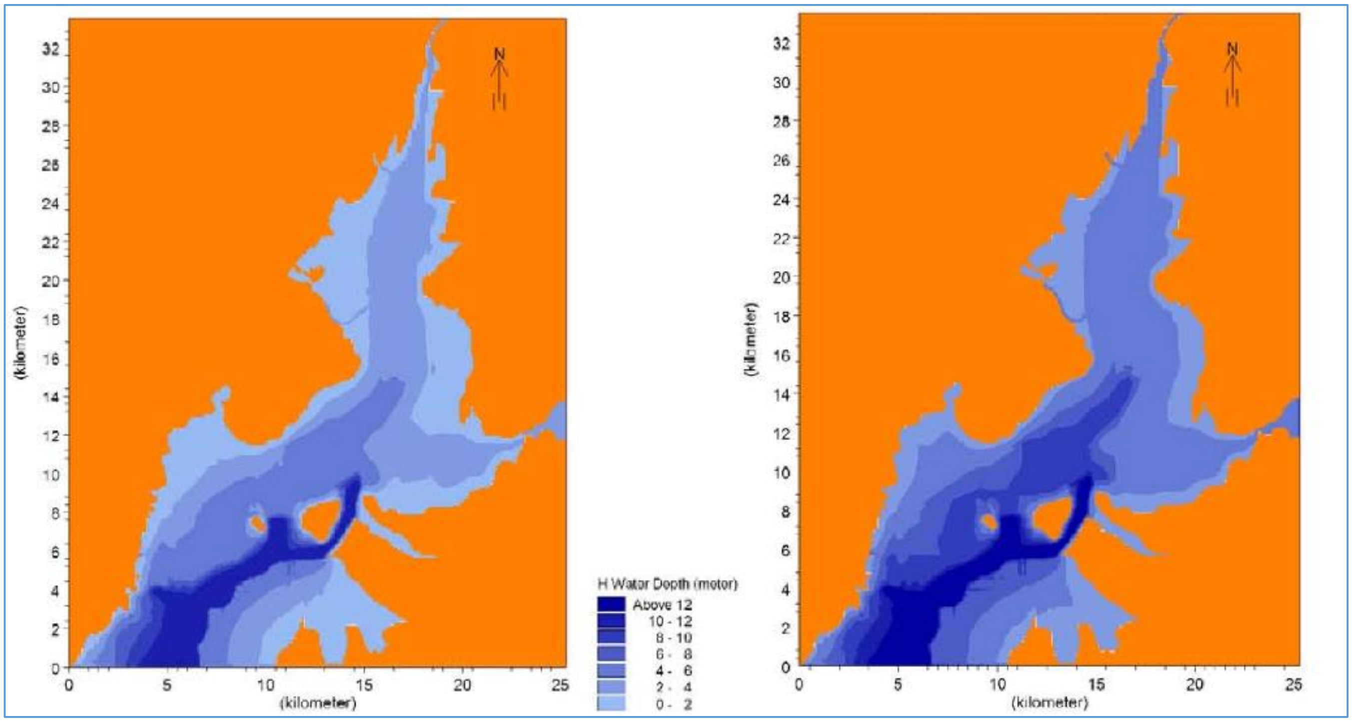


Figure A-1 Bathymetry of Thane Creek (represented as water depths) during low (left) and high (right) tides respectively (Ritesh Vijay, 2014)

Therefore, tide celerity = √ (9.8*12) = 10.8 m/s

Semi-diurnal tide implies 12 hr 40 min period, which implies a period of:

12*3600 + 2400

= 45600 seconds.

Length of tide = Celerity*Period

= 10.8*45600

= 550 km

The length of the shoreline along the western coast of the creek is found to be approximately 45 km. However, taking this as the length of the basin would be arguable since this is the length of a curved coastline and not the length a tidal wave has to traverse inside the creek. The width of the basin at its mouth is about 8 km and the area narrows down towards the creek's landward end, resulting in a relatively small net surface area of about 234 km^2, further reducing the zone within which the tide operates.

In fact, the actual distance travelled by the tide will be significantly smaller, and expected to be about 30-35 km depending on the magnitude and period of the tide. With these values the ratio of dimension of the basin to the tidal length would be about 1/20 to 1/15, making this basin a relatively short basin but not rigidly falling into the 'short basin' category (since that requires this ratio to be lesser than 1/20 always).

Appendix B Calculation of cross-sectionally averaged tidal velocity at the entrance channel

The relevant environmental factors necessary to ascertain the height of the dike required in this case would be the tidal parameters, the wave climate as well as the settlement of the subsoil. It is important to understand that in this case, the purpose of the dike is not necessarily so much to protect the coast than to confine the mud for mangrove plantation. Also, since the duration of such a confinement is relatively lesser than the durations for which dikes are generally implemented for, one can ignore emphasis on storm events which occur rarely in a short time frame as well as effects such as sea-level rise which will be more important for longer durations. The tidal parameters for the basin under consideration near the Mumbai harbour have been summarised below:

Tide	Above(+) or Below(-) Chart Datum
Highest High Water recorded	+ 5.39 m
Mean High Water Spring Tides.	+ 4.42 m
Mean High Water Neap Tides.	+ 3.30 m
Mean Sea Level.	+ 2.51 m
Mean Low Water Neap Tides.	+ 1.86 m
Mean Low Water Spring Tides.	+ 0.76 m
Lowest Low Water recorded.	- 0.46 m
Highest Low Water	+ 2.74 m

Table B-1 Mean tidal values recorded at the Mumbai Harbour, adjoining the basin under consideration (Mumbai Port Trust, n.d.)

The mean height of the tide could therefore be inferred as 3.86 m and mean low tide as 1.31 m, giving an average tidal range of 2.55 m.

In the approximation of sinusoidal tidal motion, the equation for equilibrium velocity at the entrance channel can be given as:

U = 3.14*P/(A*T) (Judith Bosboom, 2012) where U = cross-sectionally averaged equilibrium velocity

P = Tidal prism = Mean tidal range* Area of basin = 2.55*234*10^6 = 597*10^6 cu.m.

A = Cross-sectional area at the entrance of the basin = Mean sea level* Width of entrance

= 2.51*8*10^3

= 20*10^3

T = Period of semi-diurnal tide = 45600 s

Therefore, U = 2 m/s.

Appendix C What about the waves?

At the location, if one looked at globally generated waves, then the predominant waves are the swell waves generated by deep sea storms. These mainly arise just before and during the South West monsoon. The statistical analysis indicates that most wave periods fall between 6 seconds and 10 seconds.

During the continuance of the North-East monsoon, North-Easterly winds known as "Elephantas" blow for short durations during the months of October-November. As the fetch and duration of these winds are limited, the significant height of the resulting waves is not likely to exceed 1 metre with period ranging from 3 to 5 seconds (Mumbai Port Trust, n.d.).

Although the wave heights are not very high compared to open-sea situations, nevertheless one needs to take cognizance of the fact that these waves may exist. Mangroves grow best in a depositional environment with low wave energy according to Tomlinson, 1995 (Ilka C Feller, n.d.). High waves prevent propagule establishment, expose the shallow root systems, and prevent accumulation of fine sediment.

This coast under study, however could be safely classified as low wave energy coast (with wave heights rarely exceeding 1 m (Mumbai Port Trust, n.d.)) while the mean spring tidal range will put the coast in the meso-tidal category. This implies that although the basin will have a mixed energy environment, it will nevertheless be mostly tide-dominated (Judith Bosboom, 2012).

But, this does not mean one can ignore the waves. It only goes to state that the swell may not be very important. However, there will be locally generated waves within the basin caused by wind forcing. These wind waves, as will be proved in the following, will be of considerable height and will therefore constitute a major design criterion.

Appendix D Wave height calculation

For the calculation of wave heights in critical zones, with longer fetch distances and high wind speeds, Young and Verhagen's approximation has been used.

The wave heights are conversely obtained from the dimensionless energy, ϵ.

$$\epsilon = \mathbf{3,64 \cdot 10^{-3}} \cdot \left\{ \tanh[\mathbf{0,292}^{1/n} \cdot \delta^{1,3/n}] \cdot \tanh\left[\frac{(\mathbf{4,396 \cdot 10^{-5}})^{1/n} \cdot X^{1/n}}{\tanh(\mathbf{0,292}^{1/n} \cdot \delta^{1,3/n})}\right]\right\}^{n}$$

$$A1 = 0{,}292^{1/n} \cdot \delta^{1,3/n} = 0{,}493 \cdot \delta^{0,75}$$

$$B1 = (4{,}396 \cdot 10^{-5})^{1/n} \cdot X^{1/n} = 3{,}13 \cdot 10^{-3} \cdot X^{0,57}$$

$$n = 1{,}74$$

In the above box, ϵ = dimensionless energy = gE/u^4

And $E = 0.125gH^2$, wherein H = significant wave height (in m)

$X = gx/u^2$, wherein x = fetch distance (in m) and u = wind speed (in m/s)

$\delta = gd/u^2$, wherein d = water depth (in m).

Using the above formulation, wave heights have been determined for critical cases as shown in the table below:

Angle (from)	Angle (to)	Wind speed	Re-din-gs	Probability Of Occurrence	Fetch (x)	De-pth (d)	δ	X	B1	A1	ϵ	Hs (m)
352.5	22.5	7.5	353	12.77597	8000	6	1.046	1395.2	0.19	0.51	0.001909	0.71
352.5	22.5	10	12	0.434311	8000	6	0.589	784.8	0.14	0.33	0.001067	0.94
22.5	52.5	7.5	69	2.497286	9000	6	1.046	1569.6	0.21	0.51	0.002118	0.75
157.5	187.5	7.5	206	7.455664	9000	6	1.046	1569.6	0.21	0.51	0.002118	0.75
157.5	187.5	10	19	0.687658	9000	6	0.589	882.9	0.15	0.33	0.001182	0.99

Table D-1 Wave angles in predominant fetch directions and corresponding wave heights with occurrence probabilities

From these calculations, a chart depicting the exceedance probabilities was plotted against the wave heights. A smooth curve-fitting equation has been plotted through the depicted points. It is important to note that the curve is an approximation and therefore may induce uncertainties in the wave height estimation. However, given the nature of the curve, even major increments in the probability of exceedance are not supposed to give significant errors. For example, considerations have been given to different structural lifetimes. If one assumes a lifetime of the structure of, say 5 years, and a 10 % probability of failure for the structure, the corresponding extreme event will be about 1 in 50 years. For such a structure, the significant wave height according to the plot turns out to be 1.04 m. Similarly, one could see that for a structure of just a year's lifetime, this storm would be a 1 in 10 year storm for which the wave height (noticeable in the table too) is about 0.99 m. Given the methodology used and the estimation, only the first significant digit of these wave heights actually matters. Therefore, it can be clearly observed that a structure with a year's lifetime or a 5-year lifetime will not lead to significant changes in design, with significant heights for both being around 1 m. However for a structure with a life of 50-100 years, these significant heights change to about 1.1-1.2 m, leading to even higher differences in the maximum wave height (Maximum wave height is about 2 times the significant height). Therefore, in this case, a structure with a lifetime of 5 years and a significant wave height of about 1 m should be sufficient, given its short-term serviceability requirement.

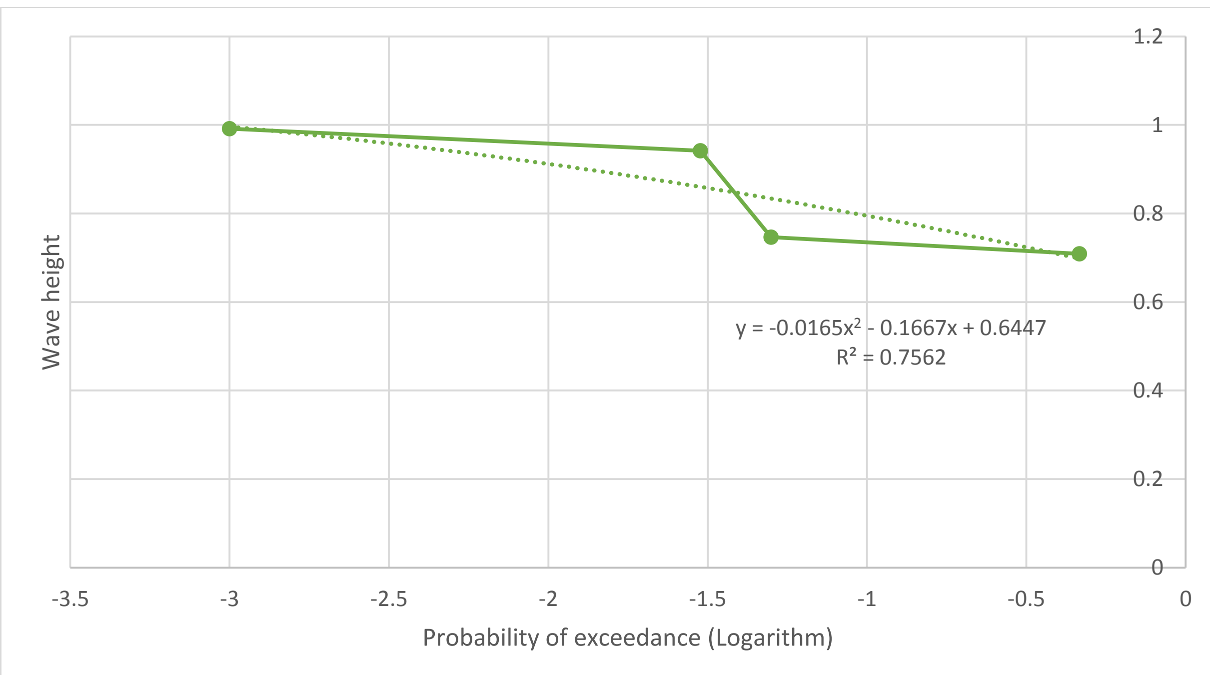


Figure D-1 Wave height plotted against the probability of exceedance of that height (for 1 in 50 year period)

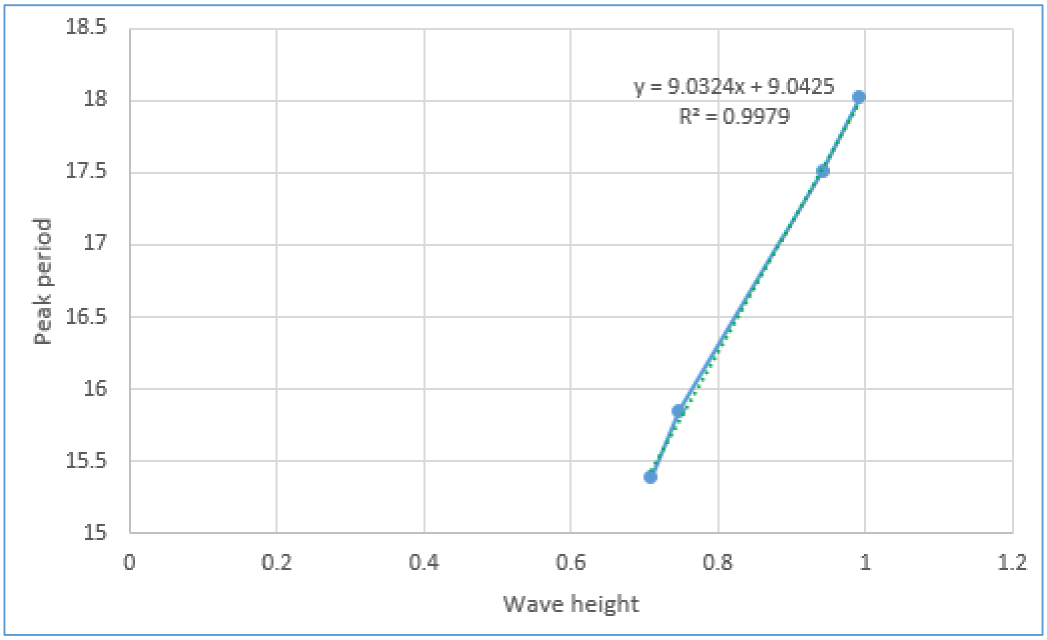


Figure D-2 Peak period plotted against wave height (linear best-fit has been shown)

It is crucial to note that the wave heights obtained the previous table are significant wave heights and not the maximum wave heights. The maximum wave heights are usually (1.8-1.9) times the significant values and therefore, in this case, the critical wave heights could go as far as 2 m. According to the World Meteorological Organization (1998), the maximum wave height can be statistically estimated to be 1.9 times the significant wave and therefore in this case the design wave height will be taken as 1.9 m.

Next important factor under consideration should be that as these waves approach the shoreline, where the intended mangrove swamps are to be planted, these waves may shoal and therefore, increase in height. Another possibility is that of breaking. However, considering a previously assumed depth of 2-4 m in the zone of interest, waves which are less than 1.9 m in height will give the height to depth ratio of

$H/d = 0.7 < 0.78$ (In case of minimum depth higher than 2.4 m)

For depths greater than 2.4 m, this ratio is ideally lesser than the breaking limit for the waves and therefore not too much of wave breaking is to be expected. However, it has been noted that these wave depths may not be uniform throughout the western shoreline of the Thane creek and therefore, breaking in some places will still be expected to happen (in shallow areas of about 2 m depth).

Also, important to note is that for wind speed readings between 5 m/s and 10 m/s, an average speed of 7.5 m/s is considered as a conservative estimate, although most speeds within this bracket lie in the lower half of the range. Speeds lesser than 5 m/s have not been taken into account since these speeds, even in the most critical cases, do not yield wave heights higher than 5 m/s. This can be verified from the table below:

Angle (from)	Angle (to)	Wind speed	Fetch (x)	Depth (d)	δ	X	B1	A1	ϵ	Hs
352.5	22.5	5	8000	6	2.35	3139.2	0.31	0.94	0.004254	0.47
157.5	187.5	5	9000	6	2.35	3531.6	0.31	0.94	0.004717	0.49

Table D-2 The waves generated by wind speeds less than 5 m/s

Appendix E Dike height calculation

The calculation for the height of the dike usually includes parameters related to water level, tide, waves and subsoil settlement. In addition to these, values related to storm surges and sea level rise (SLR) are incorporated in conventional dike designs. However, since the length of service for the intended dike in this project is relatively small, special events such as storms and long-term effects such as SLR have been ignored.

The values for the parameters to be considered are:

MSL = 3-4 m

Some discussion about the MSL values is essential at this point. The bathymetry charts for the region along the western shoreline of the Thane creek show a range of depth between 2-4 m along the area where the mangrove plantation is to be carried out. In an ideal dike design meant for coastal protection, a conservative value of 4 m may be taken because this will give the maximum protection in the worst case scenario.

MHWS = 4.4 m

Tidal readings in the Mumbai harbour area show a highest astronomical tide of above 5 m. However, these levels are the highest which can be predicted to occur under average meteorological conditions, and under any combination of astronomical conditions; these levels will not be reached every year (Anon., 2015). It is therefore not required to consider these extreme values and therefore the MHWS seems as the most practical consideration in this case.

An argument against the consideration of the tidal parameters in the dike height calculation could be that mangroves indeed require tidal circulation in order to survive. However, it is important to know that this is the case for fully grown mangroves. If such tides are allowed in for the mangrove seedlings, most of them would end up getting dispersed or worse still, washed away offshore.

Wave height = 1.9 m

An explanation of why this has been chosen as the design wave height has been provided earlier. This is not the highest wave height but the highest height with a considerable probability of occurrence (at least 10 percent). Also, important to specify is that this is the value calculated for the relatively calmer period extending from March to June.

This wave height will not be immediately relevant in the calculation of the design water level but will be important in the calculation of the freeboard which will be dependent on the wave run-up.

Settlement of subsoil = 0.5 m

There are no accurate values for the settlement of subsoil but from measurements for levees etc., these values are not expected to exceed a few feet in 1-3 years' time. This is very much within the time duration of service of the intended structure. Hence, this approximation may hold true. Yet, how many feet is few feet?

Although this settlement is not much for sea dikes, it can be considerable in areas such as estuaries (the case under consideration) or tidal rivers. In these case, there may be a lot of peat material below the dike and hence the settlement may be enhanced.

An initial value of 0.5 m is assumed for the settlement. After having calculated the height of the dike, a settlement value for the dike will be re-calculated and a corrective settlement value will be iteratively added to the height of the dike. As is evident, this corrective procedure can continue iteratively for multiple times. However, values need to be calculated only as long as they will be practically influential in terms of material and coast of the design. In short, settlement needs to be considered as an important factor only to the extent that the structure is sensitive to the settlement.

The height of the possible dike designs can now be estimated:

Design: With highest MWL (4 m)

Height = MWL + MHWS + S

= 4 + 4.4 + 0.5

= 8.9 m

The design crest level of the dike is usually supposed to include the run-up as well as the freeboard values. Comments regarding the slope of the dike is necessary at this juncture because the value of run-up may vary depending on the slopes of the dikes. The constraints for deciding the slopes in this case are 1) the availability of space; a larger slope would imply a larger space occupied by the dike and therefore very less space left for the mangroves to catch root 2) the stability of the structure; an extremely steep slope may affect the stability of the confining structure adversely.

There are various factors intertwined in the concept of apace availability. The available space inside a paddock (compartment) or a longitudinal section for that matter will depend importantly on the rear slope of the dike. The rear slope of the dike in turn depends on the amount of overtopping that is allowed. And the amount of overtopping that should be allowed is linked with the steepness or gentleness of the outer slope of the dike. Steep outer slopes lead to greater amount of overtopping, which would mean that the inner slope could be kept steep but needs to be made strong and protective against overtopping. First, a consideration is being given to the outermost dike which will be directly facing the sea. Since the limiting dimension for the 'paddock' design is 250 m, assuming different inner slopes the area available for mangrove growth has been tabulated below:

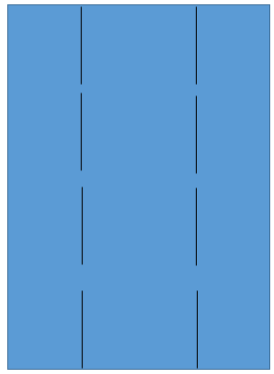


Figure E-1 A rectangle representing a compartment and the dotted line the area occupied by the dike slope

Slope	Approximate area available in sq.m. (length dimension 500 m)
1:3	100000
1:4	85000
1:6	75000
1:8	50000

Table E-1 Slope of dike and corresponding area availability for mangrove plantation as a ratio of parts of the compartmental rectangle covered

From the trend, it can be observed that increasing the slope considerably compromises the area available for mangrove growth. Since this is the very purpose of the project, it must be preferred to have a relatively small slope for the dike.

For the outer slope, one of the criterion important for consideration is that such a project is being carried out in a highly urban area surrounding the metropolis of Mumbai. Even coastal land, however environmentally important it may be, carries a high cost value. Higher dike slopes imply that the dikes will be wider and will therefore occupy larger areas along the coast. For a project of such a scale, wider dikes will lead to occupation of sizeable land, which will be extremely expensive. Various slopes with corresponding approximate areas have been considered. The area occupies by such dikes (only assuming the area lying under the outer slope, assuming that the areas under the inner slope will be corresponding to the seaward size of the dike). A liberal pricing of at least 5000 INR per square meter is assumed as the price of the land (reference collectively from Indian property websites Nestora.in, 99Acre.com).

Slope	Approximate area of entire stretch along coast (approximate length dimension 25 km)	Approximate price prediction (minimum estimate in INR)
1:3	750000	3.75 million
1:4	1000000	5 million
1:6	1500000	7.5 million
1:8	2000000	10 million

Table E-2 Land prices corresponding to dike building exercise. Gentle dikes could be very expensive.

There is now an additional problem in this mix. Mumbai's inaccessible and deserted eastern waterfront could finally be opened to the city. With an ambitious makeover plan, Mumbai Port Trust hopes to convert the 28 kilometre coastline into a tourist hotspot and save its fortune amid declining business. The plan envisages turning the land near the coast, owned largely by the Mumbai Port Trust and currently closed to the public, into revenue earning assets by developing water-sports, opening heritage hotels, floating restaurants, bird watching galleries, among other things (Mandavia, 2013). This means that the price of this land may go up further increasing two-fold per square meter, which means that having wider dikes becomes an even bigger problem.

On the top of these, the requirements for building material for the dike will also shoot up if the slope is gentler. The volume requirements will be 8 times if the width, which is one of the dimensions of the dike, is doubled.

Given that difference between the slopes of 1:3 and 1:4 is still quite but not too much, these two slopes could be considered as design slopes for the dikes. The preference, however, would be to the steeper slope as long as it remains stable and does not entail too much of additional maintenance cost.

Why is the choice of slope important? Although the overtopping of the waves may not be a significant worry from the point of view of mangroves, it could be a problem from the point of view of structural stability of the dike. Two wave overtopping related failure mechanisms at dikes or levees are generally distinguished: erosion of the grass and clay cover layer and sliding of the cover layer as a whole.

Next comes the question about having a berm. Dike cost comparisons have shown that for steeper dikes, in fact having berms do result in considerable differences in cost. Since the slopes being considered are relatively steep ones, it will be worth considering designs including berms as well. This will impact the dike design in terms of reducing its crest elevation dependent on run-up and overtopping, if either of them proves to be significant at all. How wide could the berm be made? Considering the slope of the dike and the initial design height that was calculated. The front-end width of the dike is not supposed to exceed 30-40 m. Therefore, a berm of 11 m to 14 m corresponding to each case would be the most practical.

What about the rear slope? It should be noted that direct erosion of the slope is only one possible failure mechanism. A major failure mechanism on steep inner faces (typically 1:1.5 and 1:2) in the past was slip failure of the (rear) slope. Such slip failures may lead directly to a breach. For this reason most dike designs in the Netherlands in the past fifty years have used 1:3 inner slope, where it is unlikely that slip failures will occur due to overtopping. Failures of this kind need to be avoided specifically in this case not so much for safety but for the soil-composition of the mangrove substratum. Slip failures of the rear slope could be troublesome because they may lead to severe sand infiltration into the clay, which is to be strictly avoided. Therefore, it is prudent that the least slope of 1:3 be adopted for the rear slopes of dikes under consideration.

In order to calculate the run-up, information regarding the wave periods has to be obtained. Returning to the Young and Verhagen's formulation, this time the same can be used to calculate the wave period.

$$\nu = 0.133 \left\{ \tanh\left(0.331\delta^{1.01}\right) \tanh\left[\frac{5.215 \times 10^{-4}\chi^{0.73}}{\tanh\left(0.331\delta^{1.01}\right)}\right] \right\}^{-0.37}$$

In this case, υ is a parameter that is expressed as $\upsilon = 6.28u/T_p.g$ where u is the wind speed (in m/s) and T_p is the peak period of the wave.

Angle (from)	Angle (to)	Wind speed	Re-din-gs	Probability Of Occurrence	Fetch (x)	Depth (d)	δ	X	V	Tp (s)
352.5	22.5	7.5	353	12.77597	8000	6	1.046	1395.2	0.311965	15.4
157.5	187.5	7.5	206	7.455664	9000	6	1.046	1569.6	0.302834	15.8

Table E-3 Peak periods corresponding to the two wave angles considered for the design waves

Once the peak periods are obtained the run-up can be estimated using the run-up formula for the TAW specifications (1989), as referenced from the Technical Report Wave Run-up and Wave Overtopping at Dikes can be applied:

$$z_{2\%}/H_{m0} = 1.77 \cdot \xi_0$$

$$\xi_0 = \frac{\tan\alpha}{\sqrt{s_0}}$$

Where the tangent is the angle of slope and so is the wave steepness. Using this formula, the the run-up for the dike can is calculated, for different slopes as:

Slope	Irribarren number
1:3	4.9
1:4	3.7

Table E-4 Irribarren numbers corresponding to the two dike slopes under consideration

Further, the calculations for the wave run-up and overtopping on the dike will be dependent on the type of revetment that is used. The type that is strongly recommendable in a temporary situation such as this is that of grass.

Using the value for roughness corresponding to the grass and applying it to the formula for run-up and overtopping:

$$z_{2\%}/H_{m0} = 1.75 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_0$$

The berm-related correction factor has been set to the optimal value of 0.6. In fact the berm widths have been so assumed that this factor reaches its lowest possible value.

The roughness factor for a grass revetment is to be taken as 1, because it is considered to be a close to smooth surface.

The maximum angle of attack in the zones with critical fetch distances, as has been observed in the fetch-diagram is 45 degree and therefore this correction factor is to be set to 0.7

Therefore, the run-up in the all the cases can be calculated as:

Case	Slope	DWL (m)	Berm width (m)	Run-up (m)	Overtopping (l/s/m)	Crest level (m)	Volume of sand (per unit length)
Case 1	1:3	8.9	0	6.0	0.44	14.9	333
Case 2	1:4	8.9	0	4.5	0.40	13.4	360

Case 3	1:3	8.9	4	3.6	0.26	12.5	270
Case 4	1:4	8.9	5	2.7	0.24	11.6	314

Table E-5 Dike designs assuming the minimal overtopping

One of the clear indications of the above table is that the overtopping values are far lesser than the ones permitted. As has been predicted, the overtopping and run-up values may in fact not be too significant in this case. It has been repeated than the mangroves can indeed survive even if a considerable amount of overtopping discharge is allowed.

What happens if no accommodation for run-up and overtopping is not accounted for in the crest level calculation?

Case	Slope	DWL (m)	Berm width (m)	Run-up (m)	Overtopping (l/s/m)	Crest level (m)
Case 1	1:3	8.9	0	6.0	1000	8.9
Case 2	1:4	8.9	0	4.5	1550	8.9
Case 3	1:3	8.9	4	3.6	1200	8.9
Case 4	1:4	8.9	5	2.7	900	8.9

Table E-6 Dike designs assuming no crest levels at all. One can see extremely high overtopping values.

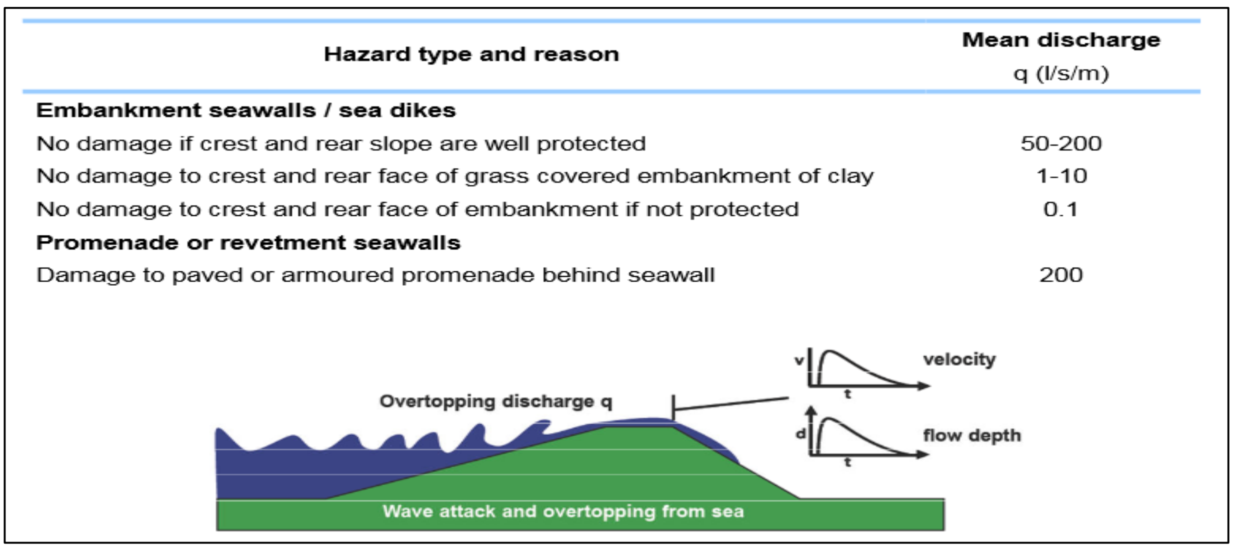
Even now, the overtopping values are well within the tolerable limit. What if instead of providing no freeboard at all, the freeboard is further reduced? As discussed earlier, there is a considerable uncertainty about water depth since it is ranging between 2 and 4. What if the MWL is assumed to be at 3 m and as a result a negative freeboard of 1 m is provided? Also since there is a grass revetment for the dike, limits corresponding to grass-covered slopes must be considered.

Case	Slope	DWL (m)	Berm width (m)	Run-up (m)	Overtopping (l/s/m)	Crest level (m)	Volume of sand

							(per unit length)
Case 1	1:3	8.9	0	6.0	3.1	13.5	273
Case 2	1:4	8.9	0	4.5	4.7	12.0	288
Case 3	1:3	8.9	4	3.6	6.7	11.1	215
Case 4	1:4	8.9	5	2.7	17.5	10.2	246
Case 5	1:4	8.9	5	2.7	3.8	10.7	269

Table E-7 Dike designs assuming a slightly lower crest level than previously assumed (1 m lower)

The overtopping requirements as entailed in the overtopping manuals have been summarised in the figure below. These give significantly high overtopping allowances for dikes which are protected by grass slopes.



Hazard type and reason	Mean discharge q (l/s/m)
Embankment seawalls / sea dikes	
No damage if crest and rear slope are well protected	50-200
No damage to crest and rear face of grass covered embankment of clay	1-10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200

Figure E-2 Limits of overtopping for damage of the crest and rear slope

Figures for the four dike designs being considered for the options have been drawn below. The figures have been drawn on a proportional scale and the relative values of the dimensions are identical to those being proposed for the project.

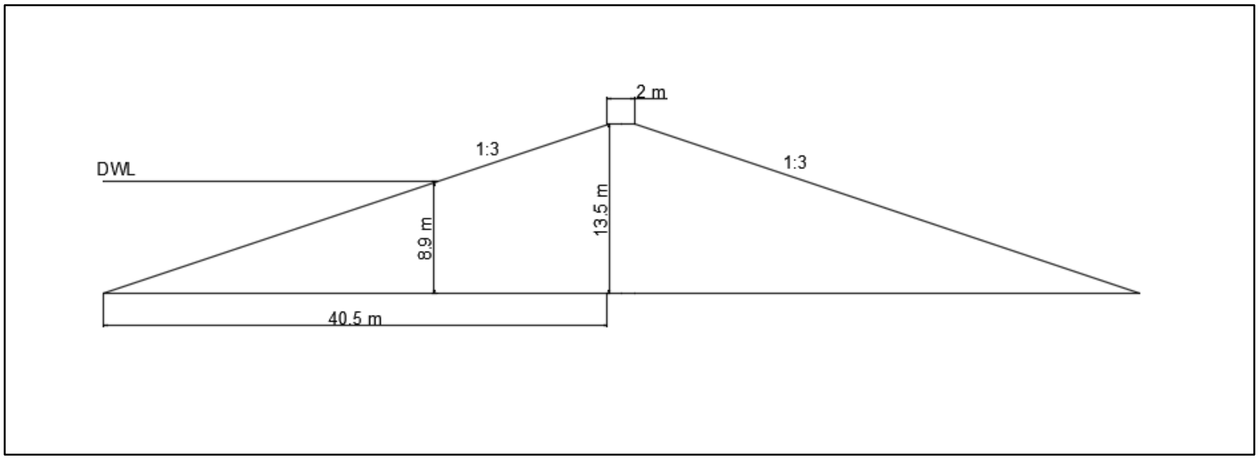


Figure E-3 Dike 1

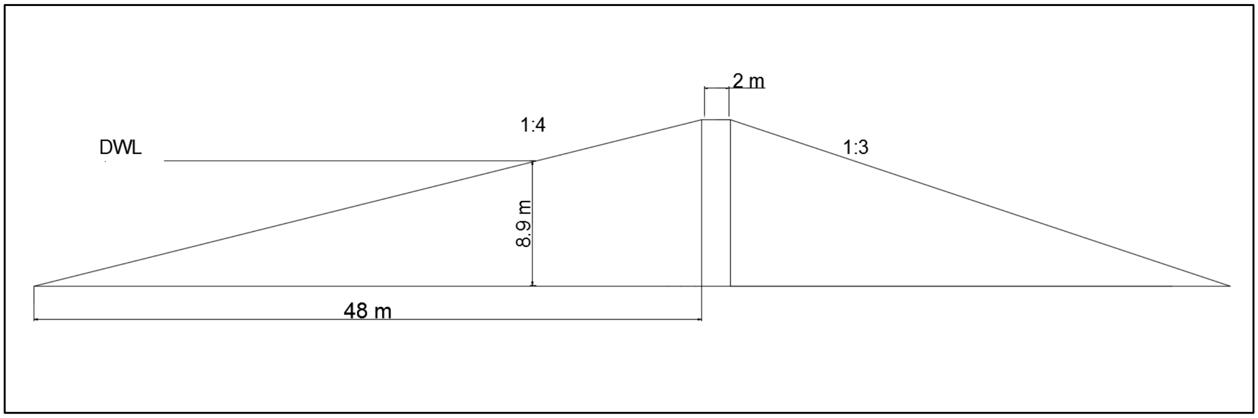


Figure E-4 Dike 2

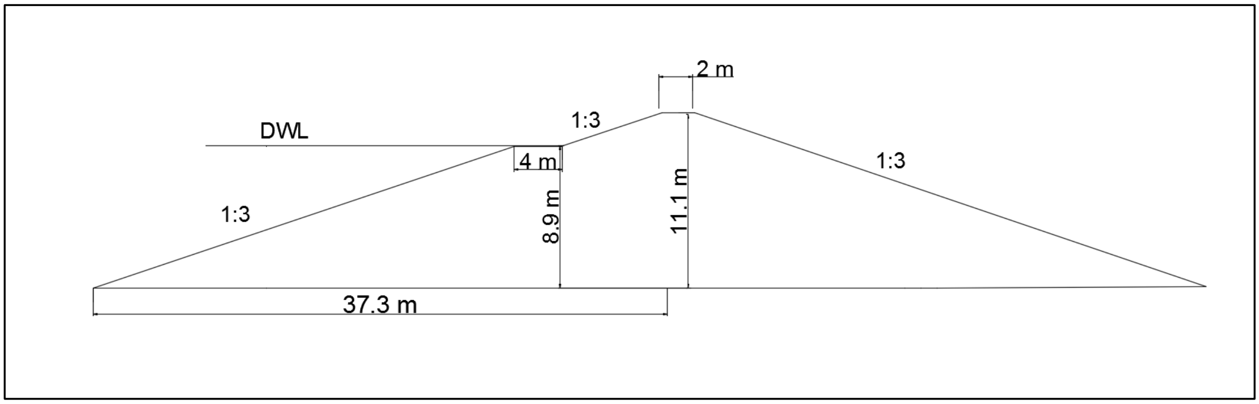


Figure E-5 Dike 3

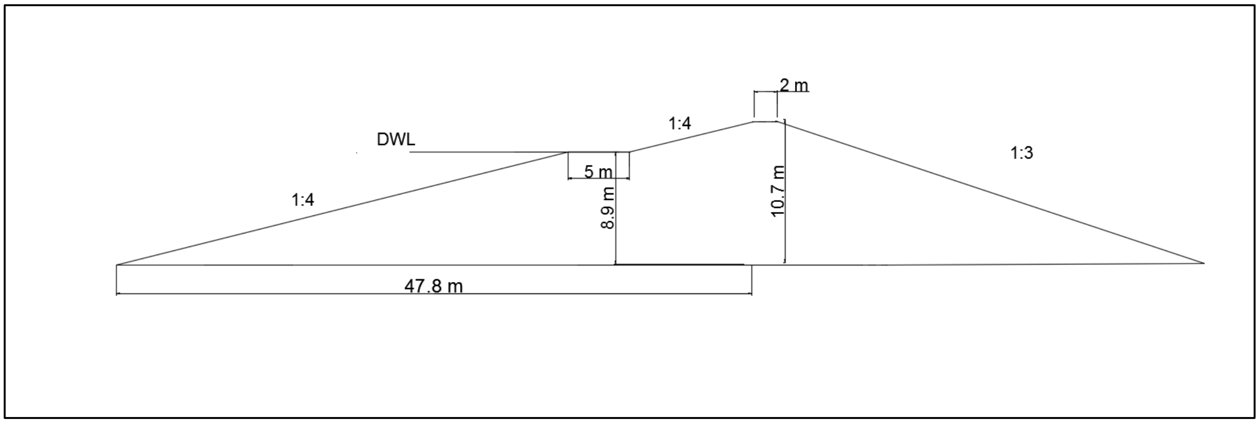


Figure E-6 Dike 4

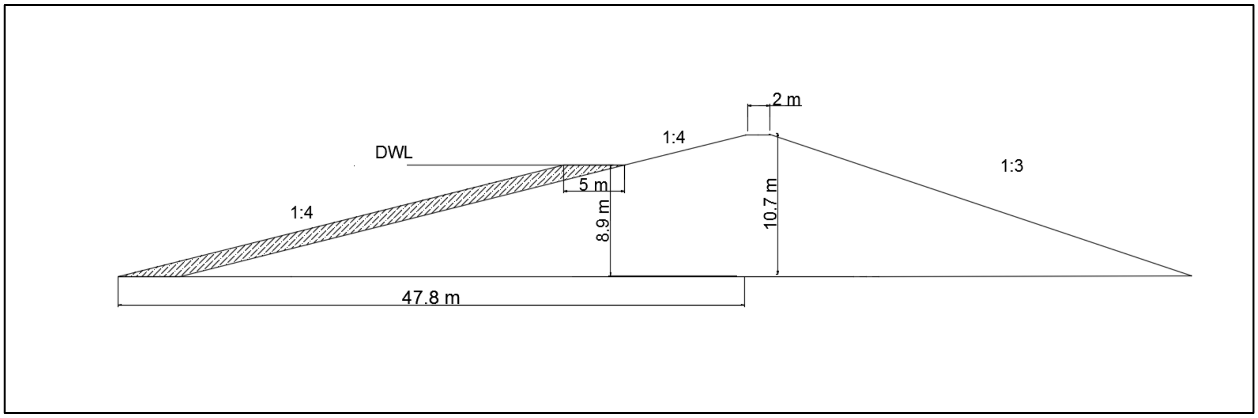
It can be seen that for these cases the values for overtopping are not exactly below the 1 l/s/m specified by the overtopping manual. However, in this case, some flexibility could be permitted and minor erosion of grass layers or clay on the rear slope may not heavily impact the mangroves. In fact, grass being a natural revetment, it can be easily integrated with the mangroves once they are grown. It is also observed that there would be a considerable saving (about 10 to 20 percent) in terms of the volume of material required for building the dike.

At first look, it seems like the dike with a slope of 1:3 (which is good since a higher slope is preferable) with or without a berm would be the optimal from the point of view of volume requirement, as would be the dike with slope 1:4 without a berm. The further decision on the selection of the ideal dike design should depend on space availability. From this perspective, a dike with a slope of 1:3 without a berm will be the best design, as this will be the dike that will occupy the least surface area and in turn allow more space for the mangrove plantations.

Case	Slope	Berm width (m)	Approximate land area (per square m)
Case 1	1:3	0	40.5
Case 2	1:4	0	48
Case 3	1:3	4	37.3
Case 4	1:4	5	45.8

Table E-8 The per meter area footprint for each dike

The amount of areas that may have to be sacrificed if berms are provided to the dike have been represented in the figures below.



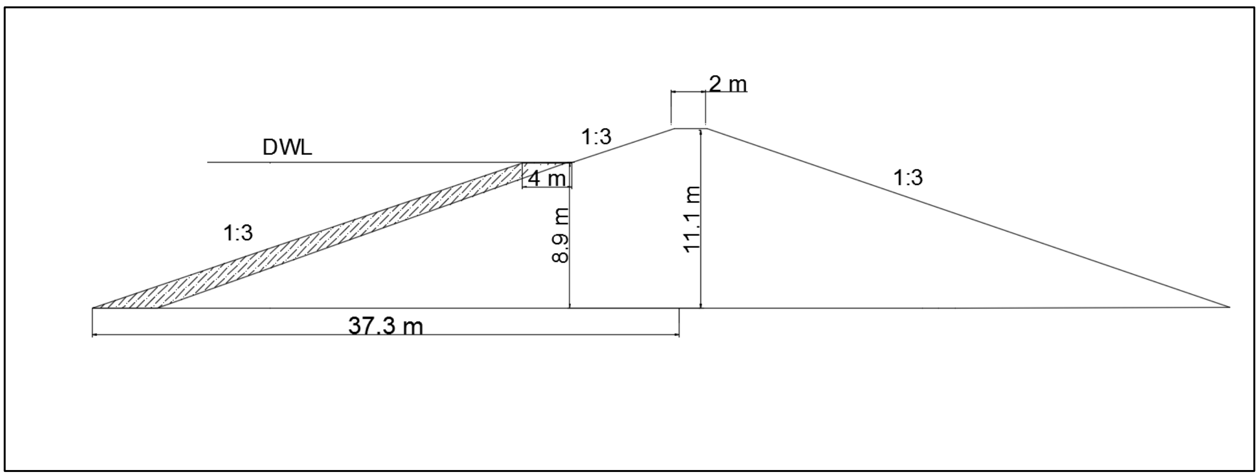


Figure E-7 Figures showing incremental areas (and resultant volumes) due to berms in the dikes

Appendix F Sensitivity analysis of the subsoil

The calculation for the dike heights has been carried out in the previous section. Applying the designed dike as a load, the next step is to calculate the resultant settlement of the subsoil. Would this be significant? If yes, how would it affect the existing overtopping values? Some corrections in the existing crest level may also be required owing to such settlement of clay.

Topography around the harbour:

In the southern harbour between Colaba and Karanja the depth varies from less than a meter to about 11 meters. The background is covered with 6 m thick layer of clay. Onshore rock continues as submarine outcrops near Colaba and extends over 2 km eastward and 4 km southward forming Prongs reef. The bedrock surface has a 1-2 m relief. From Karanja offshore and onshore rock continues 12 km either as submarine outcrops or as subsurface below a thin layer of clay. This rea is covered with a 16 m thick layer of clay. The bedrock here is characterized by considerable relief. Surveys also bring out the submarine meandering channel of the Karanja Island into Dharmatar Creek the bedrock is flat and covered with 2-3 m clay.

Large bedrock outcrops known as Thal shoal stand out as major bathymetric features near the harbour entrance. The uneven topography of the shoal is of a range of 2 to 3 m and exhibits steep seafloor gradient. To conclude we can say that:

- The entire area is covered with an average 3m of clay
- Stratified layers of stiff clay underlie the soft clay layer.
- Several buried channels are present in this area
- The bedrock is comprised of Deccan Trap or its weathered derivatives
- Northern inner harbour has more relief as compared to southern or outer harbour
- Surface sediments show evidences of scouring and mobility.

Consolidation periods:

The time for consolidation to occur can be predicted. Sometimes consolidation can take years. This is especially true in saturated clays because their hydraulic conductivity is extremely low, and this causes the water to take an exceptionally long time to drain out of the soil. While drainage is occurring, the pore water pressure is greater than normal because it is carrying part of the applied stress (as opposed to the soil particles).

Settlement calculation:

The Skempton Bjerrum method (craig, 2004) has been used in order to calculate one-dimensions=al settlement. There may be instances of lateral strain in such a case but these effects have been ignored. The purpose of calculating the settlement is not to get a fixed value for the same but to get a good approximation of the order of settlement. This will be helpful in determining whether the settlement I indeed significant in comparison to the dimensions of the dike.

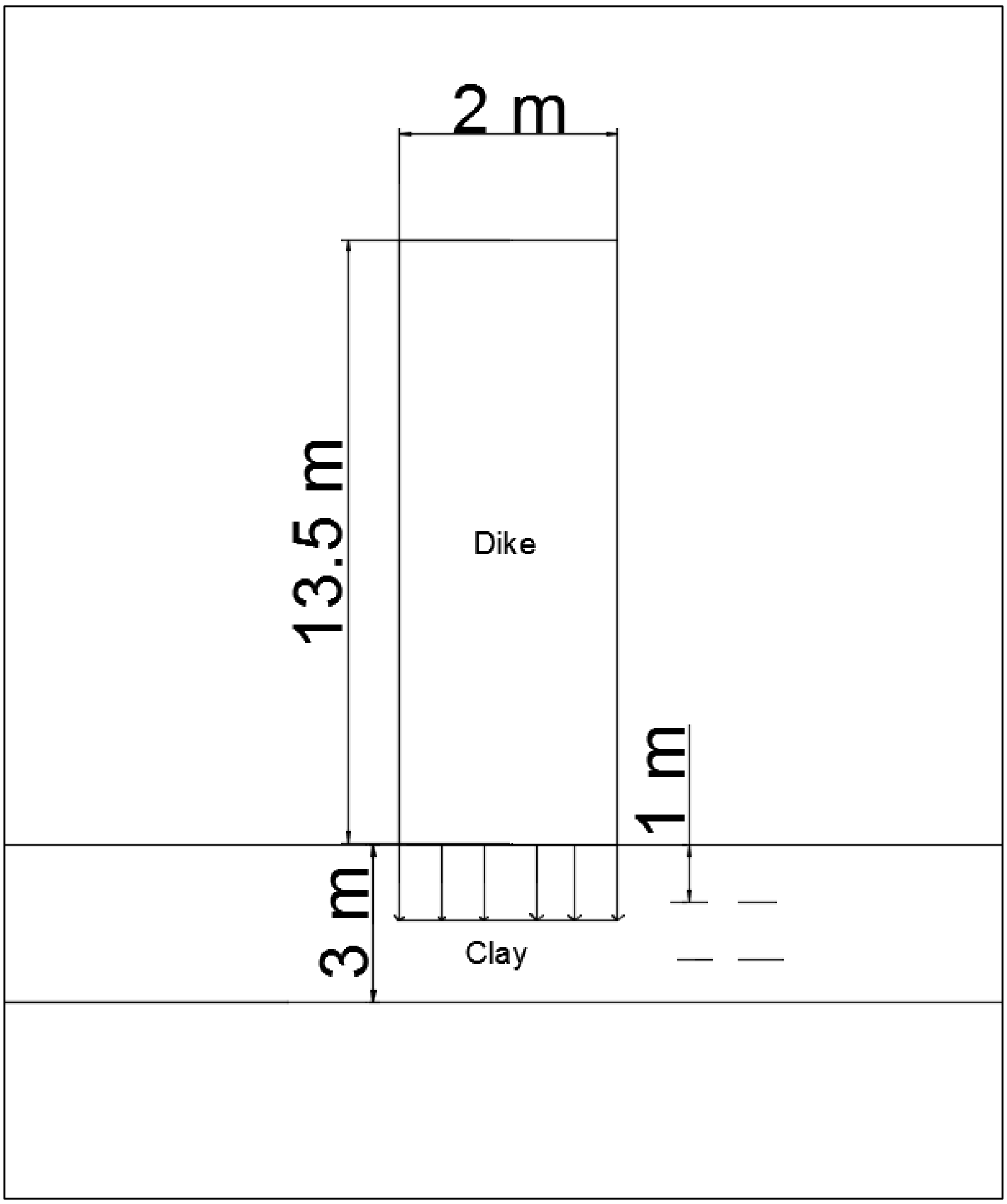


Figure F-1 Soil layer below the heaviest part of the dike.

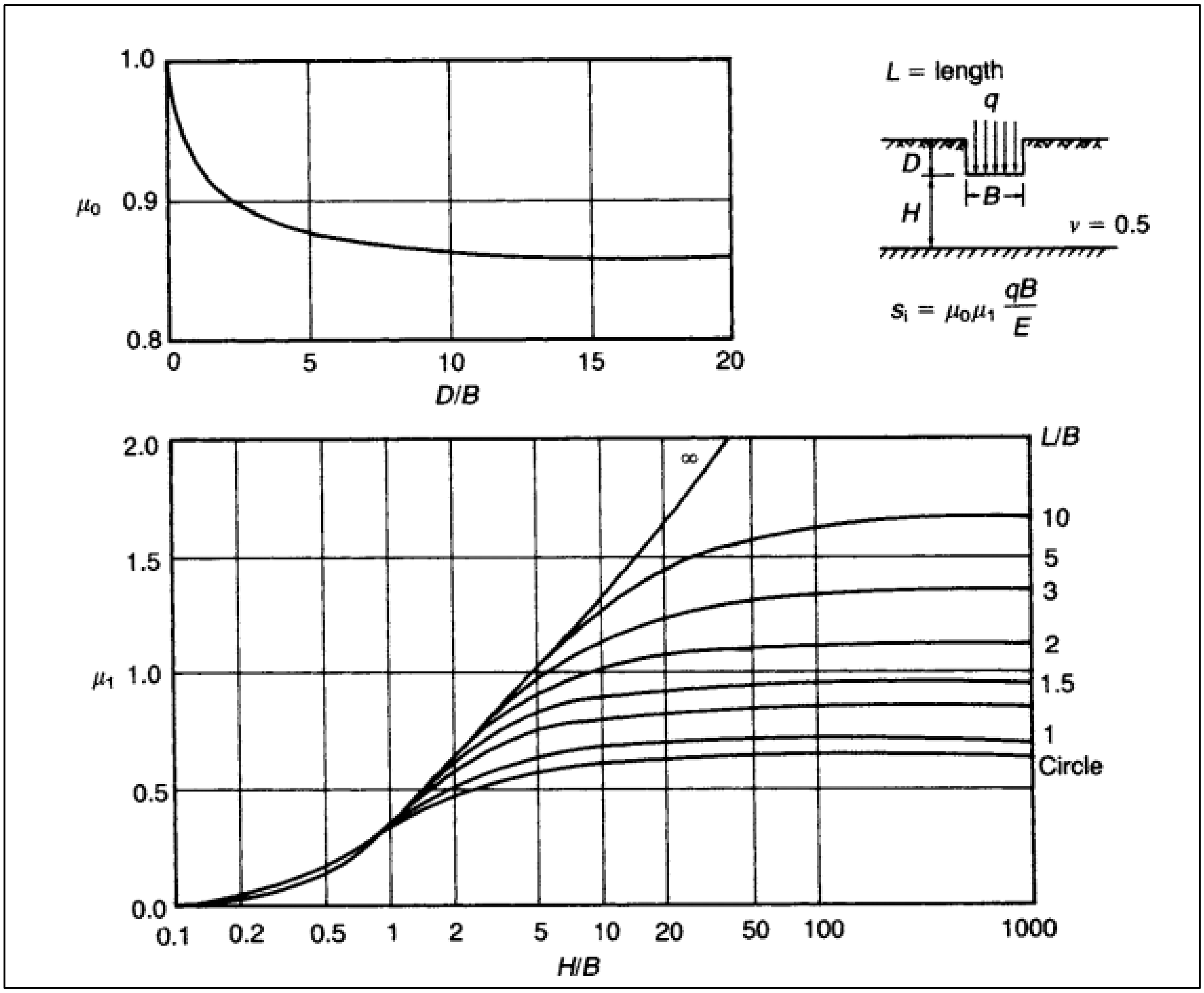


Figure F-2 Reference figure for calculating constants that will need to be used for the initial settlement calculation. These constants depend on the dimensions of the layers being considered.

Initial settlement:

Thickness of the clay layer = 3 m

Dike weight assumed at the highest load i.e. along the crest of the dike, for 1:3 slope without berm is 300 kN/m² (The density of saturated coastal sand is assumed as 2150 kg/m³)

Using this as the load and the values for the coffecients μ as 1 and 0.5 respectively, and assuming the width of the dike crest to be 2 m, the initial settlement is found to be of the order of 12 mm i.e. about 0.012 m which is actually negligible compared to the settlement that was initially assumed while designing the dike (i.e.0.5 m). It is important to note that the settlements due to all other dike designs being considered will be lower than this because this is supposed to be the dike with the highest crest.

Consolidation settlement:

In addition to the initial settlement, a consolidation settlement needs to be calculated. This settlement may not be too important in the case of this project since consolidation processes may take several years in time whereas the applicability of this project is for a shorter duration.

Layer	Z in m (Depth)	m,n (assumed 2m X 2m block)	I	Difference in effective stress	Settlement in mm (assumed m_v = 0.13 m^2/MN)
1	0.5	2	0.233	280	36
2	1.5	0.67	0.121	145	19
3	2.5	0.4	0.060	72	9
Total					64

Table F-1 Settlement calculation based on layers with incremental depth of 1 m

Assuming a value of μ for the clay as 0.5, the total consolidation settlement would be around 30 mm.

Therefore, even the post-consolidation settlement is found to be in the range of 0.03 m (which is less than 10 percent of the initially assumed subsoil settlement of 0.5 m while designing the dike). So, even if the dike sinks due to such settlement, it will still be well in the safe zone, in turn implying that the dike design is not too sensitive to subsoil settlement in such a scenario.

It will also therefore be better to assume a smaller settlement margin for the dike design itself. Instead of the already assumed 50 cm, a total of these two settlements which rounds off to about 10 cm can be considered. This will allow to reduce the initially calculated dike heights by about 40 cm, thus saving considerable material costs. However, this allowance has already been accommodated through a 1 m reduction in the dike height proposed in Appendix E. This 1 m reduction is assumed to be partially due to lesser subsoil settlement and owing partially to the reduction in the average MSL value, which has a considerable level of uncertainty. The assessment of the appropriateness of the dike after such alteration has been based on the overtopping values, which, although significantly higher than the first design, yielded values within the allowable limits for grass-top dikes.

Appendix G The dikes as part of each of the proposed design structures

Compartmental design

For the compartment (paddock) design, the maximum volume of dike material will be required since each of the compartment boundaries will have to be a dike. However, what is important to note here is that not all the compartment boundaries need to be as sturdy as the outermost dike which is exposed to sea. The dike which is facing the sea could be designed based on the principles mentioned above. However, for dikes which would be present on the lee side of this dike, the crest level can be considerably lesser because of two reasons: 1) The wave height will be considerably attenuated, almost reduced to zero, after passing the one facing the sea and 2) Since wave attack will be negligible, not much run-up is expected to be caused. However, the mean water level and the tidal parameters will nevertheless be present.

Therefore, while for the exposed structure the design from the last table needs to be considered, for the inner dikes a design similar to a no-freeboard dike as mentioned in an earlier table can be followed.

The downside of using dikes for the compartmental design plan is that there will be far lesser space available for the mangrove seedlings to catch hold since a considerable expanse of the available areas could be taken up by the dikes.

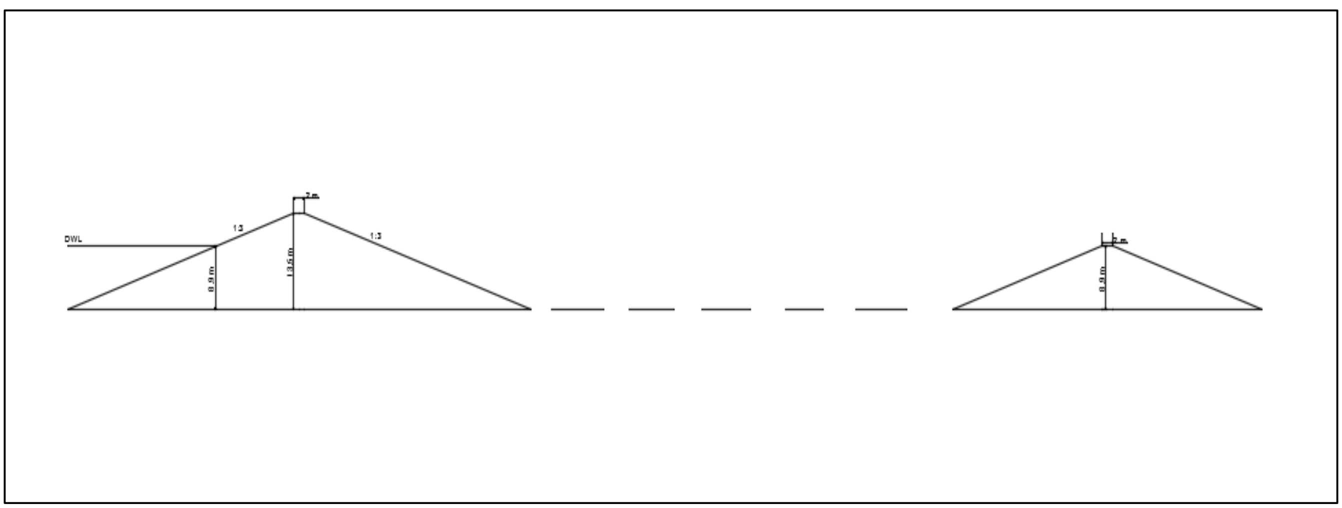


Figure G-1 View of offshore and landward dike separated by area meant for mangrove plantation

For the four cases considered, the minimum occupied areas could be summarised as follows:

Case	Slope	DWL	Berm width	Surface area taken up by the dike per compartment (in sq. m)	Proportion of total area per compartment for plantation of mangroves
Case 1	1:3	8.9	0	18000	0.33
Case 2	1:4	8.9	0	24000	0.47
Case 3	1:3	8.9	4	25200	0.50
Case 4	1:4	8.9	5	27600	0.55

Table G-1 Area taken up by the dikes and corresponding area left for mangrove restoration as a proportion of the total area of compartment

Considering that the total area of the compartment (500*250) will be 125000, the area occupied by the dikes takes up a considerable proportion (at least 1/3rd but also half in most cases).

Additionally, in this design, the total volume requirement of the sand will also be the maximum.

Assuming a 27.5 km alongshore stretch of dikes (with 500 m offshore) with 500*250 size compartments each. This would mean a total of 110 such compartments. Each of the compartments will be consuming a volume equivalent to the half-dike volume along its entire perimeter.

Cases	Volume required by inner dikes	Volume required by sea-facing dike	Total volume requirement (in m^3)
Case 1	20 million	15 million	35 million
Case 2	20 million	15.5 million	35.5 million
Case 3	20 million	13.5 million	33.5 million
Case 4	20 million	14.5 million	34.5 million

Table G-2 Volume of sand required for each of the dikes, both landward and sea-facing

Longitudinal design

For the design with dikes along the direction of the shoreline, without any in the direction perpendicular to it, the application will be similar as discussed in the earlier case. The outermost dike which will be facing the sea would have to be the strong dike which has been proposed with appropriate dimensions in the previous section. However, the subsequent dikes behind it can be dummy dikes with no crest levels corresponding to wave heights and run-ups as these would be negligible.

In these case, the areas taken up by the dike will be relatively lesser as the dikes will be present only in one direction.

Case	Slope	DWL	Berm width	Surface area taken up by the dike per as a proportion of total area
Case 1	1:3	8.9	0	0.25
Case 2	1:4	8.9	0	0.33
Case 3	1:3	8.9	4	0.35
Case 4	1:4	8.9	5	0.45

Table G-3 Area taken up by the dikes and corresponding area left for mangrove restoration as a proportion of the total area of compartment in the longitudinal design plan

Estimates need to be made for the volume requirement for these project as well:

Cases	Volume required by inner dikes	Volume required by sea-facing dike	Total volume requirement (in cu.m.)
Case 1	13.5 million	15 million	28.5 million
Case 2	13.5 million	15.5 million	29 million
Case 3	13.5 million	13.5 million	27 million
Case 4	13.5 million	14.5 million	28 million

Table G-4 Volume of sand required for each of the dikes, both landward and sea-facing in the longitudinal design plan

Vertical drains design

For this design, only the outermost offshore confinement is necessary. The other components are not part of this design plan and therefore the volume requirements for this design will be restricted to those for the outer dike.

Cases	Volume required by sea-facing dike	Total volume requirement (in cu.m.)
Case 1	15 million	15 million
Case 2	15.5 million	15.5 million
Case 3	13.5 million	13.5 million
Case 4	14.5 million	14.5 million

Table G-5 Volume of sand required for each of the dikes, both landward and sea-facing in the vertical drains design plan (non-compartmental

Appendix H Design of split bamboo piled wall

First and foremost, the height (or length) of such a pile needs to be determined. Similar criteria as those used in the dike design could be considered: waves, tides and subsoil settlement. The height of the piles should therefore be kept at least to the crest levels that have been defined for the dikes in the previous sections.

The spacing between the bamboo piles is another design determinant. The design practices suggest that the spacings between these piles must be about 20 times the diameter of the pile. *Bambusa balcooa,* which is one of the most commonly found bamboo species from India. It has tangled clumps which make it difficult to harvest. They are strong, and are often used in construction. The diameter of the pile is of course dependent on the type of bamboo available. The most common type (*Bambusa balcooa*) found in the areas close to the project is about 130-150 mm in diameter. This is also the type that is used in other constructions using bamboo piles and has been recommended as a suitable construction material for bamboo walls. An approximately 10 cm diameter implies that the spacing between the piles can be about 2 m. This spacing of 2 m needs to be filled with infill. A discussion on the infill has been constructed in the report. The most suitable infill is found to be one that uses a wattle or bamboo mat with plastering. The 2 m dimension is small enough and is therefore advantageous from the point of view of making these mats. This is also smaller than the length of tie-bamboo rods available, meaning that multiple mats can be ties together using bamboos itself and a strong enough wire-mesh. These will come under the category of woven mats and any spaces between the weavings must be appropriately plastered using a combination of mud and clay.

A typical piled wall in a wave environment will experience several loading conditions as it is being build and during the various construction stages. The significant forces are hydrostatic pressure, forces due to soil loads, water current forces, wave forces and accidental loads. Accidental forces or storm forces will not be considered for this design considering the period and purpose of the intended structure.

One of the criteria for the design of the freeboard of such a wall would be the possibility of run-up. The Eurotop manual states the following rule for a vertical wall on a dike:

In some situations it may occur that a vertical wall or a very steep slope at the top of a slope has been designed to reduce wave overtopping. These walls are relatively small and not comparable with vertical structures like caissons or quay walls. Due to limited research the application of a reduction factor
Γ is restricted by the following area:

- slopes from 1:2.5 to 1:3.5, possibly with a berm with dimensions B/H= 2-3 or B/L= 0.05-0.08

- the toe of the wall should lie between 1.2 H above and below the still water level
- the minimum height of the wall (with a high toe level) is about 0.5 H and the maximum height (with a low toe level) is about 3 H

With a vertical wall this procedure will very soon lead to a large value of the breaker parameter ξ (Refer appendix A) which means that waves do not break in such situations. In fact the wall is situated at the top of the slope and waves will possibly break on the slope before they reach the wall. In order to keep the relationship between the type of breaking and the breaker parameter, the vertical wall has to be schematised as a 1:1 slope, starting at the toe of the wall. The reduction factor for a vertical wall on top of a slope is $\gamma = 0.65$.

Using formulae for overtopping specified in the Eurotop manual, the values for run-up come to be 2.3 m assuming a 1:3 slope for a supporting dike.

It is extremely important to note at the end of this calculation that this is only an approximation. The piled bamboo wall which is expected to stretch through the entire water depth cannot actually be considered as merely a wall on the top of a dike. Additionally, it does not precisely satisfy all criteria for the application of the formula but it does not deviate too much from the restrictions. Given the lack of formulations available for construction of bamboo piled walls (also because there are extremely few of these in coastal areas), the results of this formula are considered to be the best possible approximation.

The height of the pile can now be determined as addition of design water level, freeboard for run-up.

Height of pile = 8.9 + 2.3 = 11.2 m

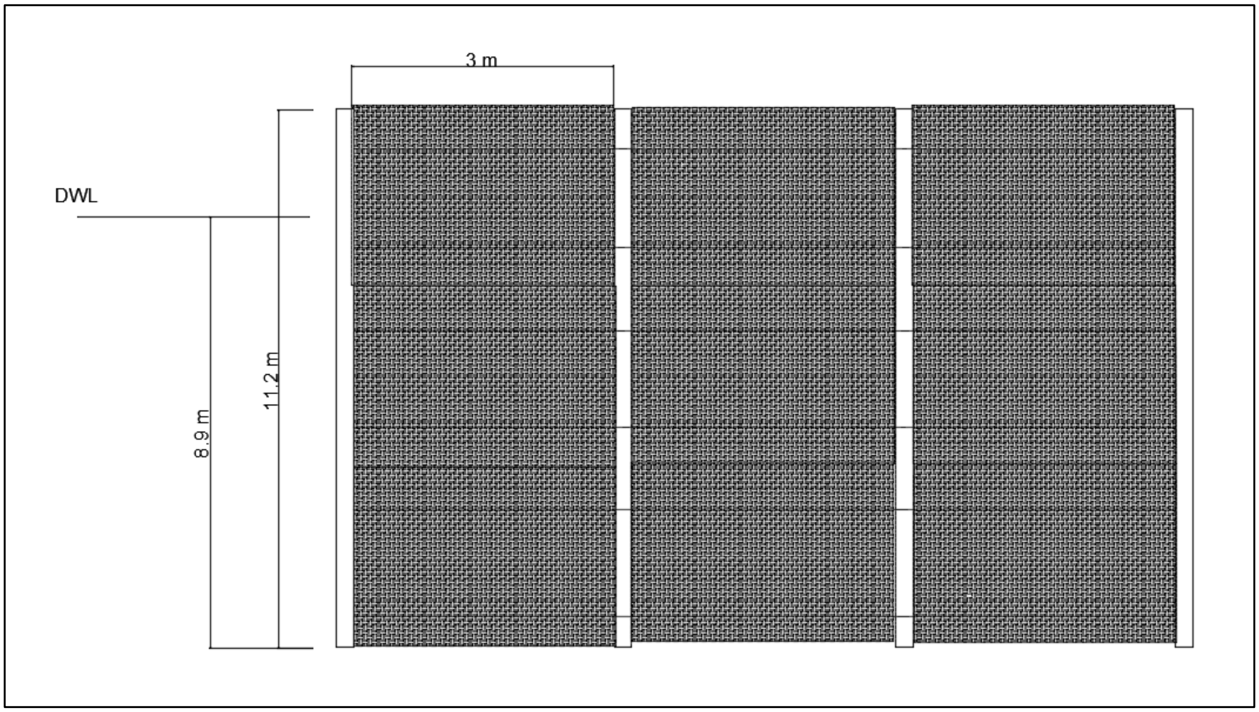


Figure H-1 Depicting the bamboo piled wall with panels

It is arguable if some accommodation in this height should be provided to make up for some sinking due to scour at the bottom. However, considering that such an occurrence would rather be und=expected and if occurred, would lead to a failure at the toe, such adjustment are not being considered here. However, calculations relating to the expected scour depths under wave and current conditions have been made in the following sections.

Some indication also needs to be given about the amount of bamboo that be required for the entire length of construction in the three design procedures proposed as alternatives for the project. The previously found spacing of 3 m is to be assumed and bamboo piles of lengths of about 12 m are to be used.

Design procedure	Number of bamboo piles required (assuming 27.5 km stretch with 500 m offshore expanse)
Compartmental design	36700
Longitudinal design	27500
Design with vertical drains	9200

Table H-1 Number of bamboo piles required for the construction of the wall based on various alternatives for layout

Also, it must be noted that the requirement of infill material will be proportional to the number of piles and therefore those materials will lead to a further increase in case of a compartmental design. One of the advantages of using the bamboo piles for a compartmental design as gainst the dikes will be the amount of space available for mangrove growth. Since this is a vertical wall design, it will take up meagre land area in comparison to dikes with 1:3 slopes.

Additionally, one of the major considerations for the design of such walls is its foundation. The kind of support these bamboo piles will have at the bottom will to a large extent determine their stability and survivability under coastal conditions.

Scour at the bottom of the pile:

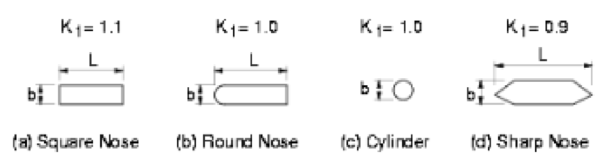
$$\frac{S_m}{(H_{mo})_o} = \sqrt{22.72\,\frac{h}{(L_p)_o} + 0.25}$$

Where
- S_m – Maximum scour depth from bed level
- $(H_{mo})_o$ – Deepwater significant wave height
- h – Pre-scour water depth at wall
- $(L_p)_o$ – Deepwater wavelength associated with T_p

This formula predicts an even lesser scour depth of about 1 m.

$$\frac{S_m}{h} = 2.0\,K_1\,K_2\left(\frac{b}{h}\right)^{0.65} F_r^{\,0.43}$$

Where
- S_m – Maximum scour depth below average bed level
- h – Water depth upstream of pile
- b – Pile width
- F_r – Flow Froude number $[F_r = U/\sqrt{(gh)}]$
- U – Mean current velocity magnitude
- K_1 – Pile shape factor
- K_2 – Pile orientation factor
- θ – Angle of pile orientation
- L – Pile length



$$K_2 = \left(\cos\theta + \frac{L}{b}\sin\theta\right)^{0.62}$$

The scour depth under the current conditions that are supposed to occur due to the tidal environment in Mumbai comes about to be 2.22 m. This may be a point of concern and therefore the pile height must be corrected or this possibility. Proper protection also needs to be provided, whatever support may be used at the bottom of the pile. As rule of thumb, this scour hole is not expected to be higher than the wave height, which means that it won't be more than 1.4 m in the extreme. One of the solutions to this is, given that bamboo is available in higher lengths, such scour prediction could be accounted for in the height of the pile.

For supporting material at the bottom, various options have been considered:

A small sand mound:

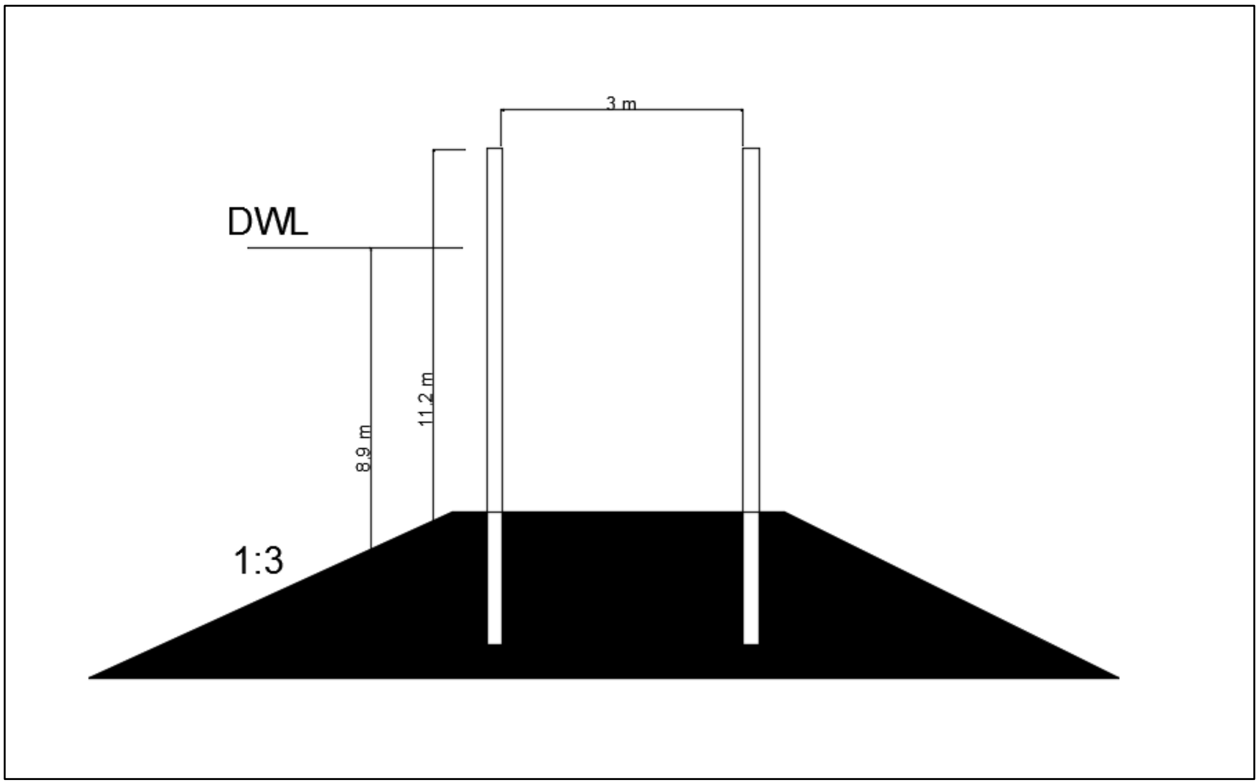


Figure H-2 Piled bamboo wall supported by sand mound

The design of the sand mound would have to be somewhat similar to the dike design discussed earlier, only with a far smaller dimension. However, these structures will be below the design water level and will not be subject to any wave loads. However, considering the conditions in Mumbai, there is still expected to be strong tidal currents to and from the shore. The toe of such a sand mound will also require to be protected against scour. But such an issue may not be a very major one as the scouring process will be accelerated under high wave conditions which are mostly expected to occur during monsoons and not during the season under consideration.

Boulders:

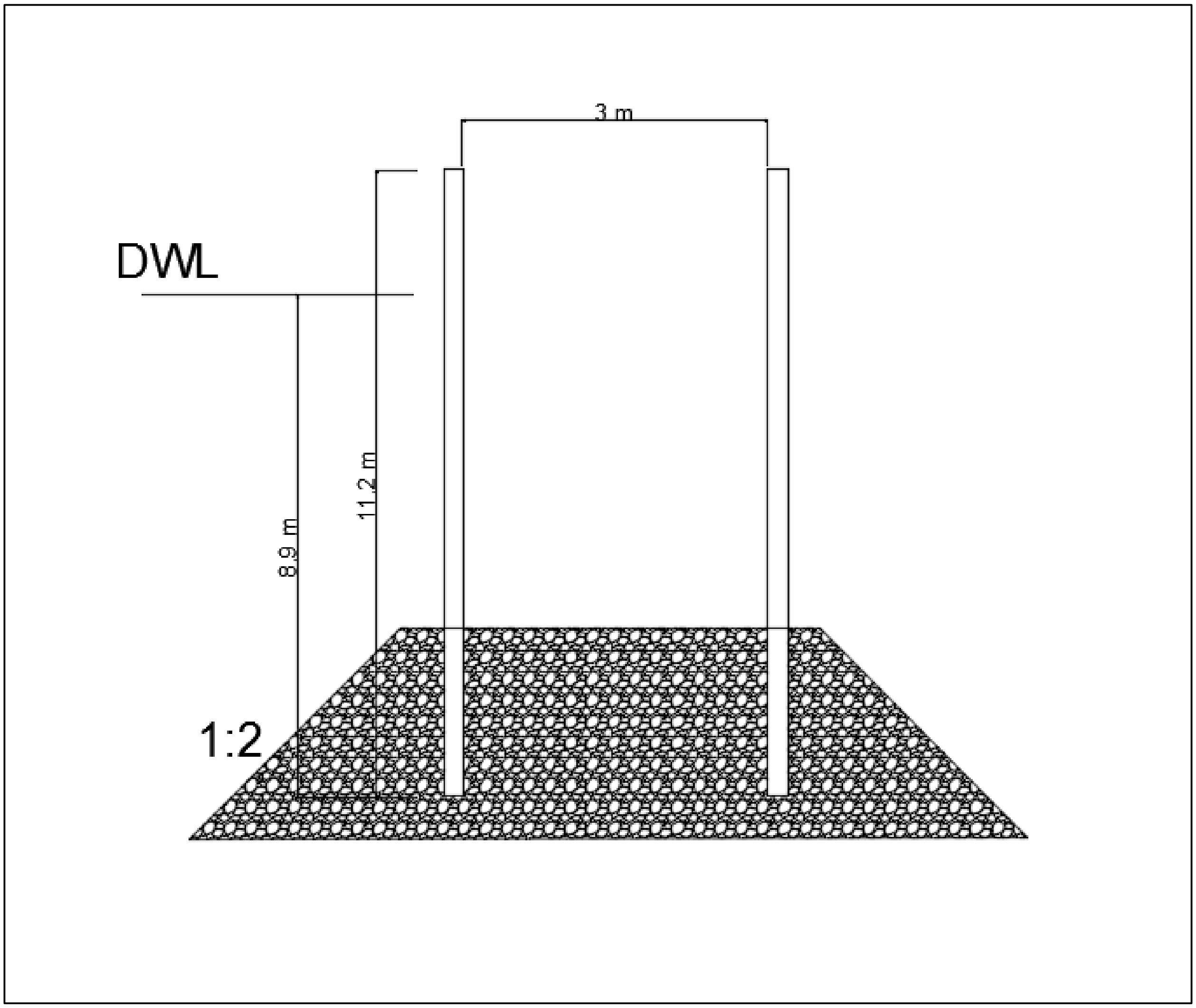


Figure H-3 Piled bamboo wall supported by boulders

Boulders could also be possibly used in order to support the bottom of the piles. An appropriate size of boulders can provide adequate defence against currents for a short period of time. However, one of the problems with using boulders is that clay could escape from under the bamboo piles through the gaps in these boulders. That makes this choice a less preferred one for bottom support. Also, rocky environments are not best suited for mangrove growth. Mangroves can also grow on sandy or rocky coast or on carbonate or reef sediments but tend not to reach the height, density or extent of mangroves on marine clay.

Wooden bars:

Wooden bars could also be an option for bottom support. One of the problems with wooden bars is that they must not be exposed even during low tide. The major problem with wood piles occurs when groundwater levels drop and expose the wood at the top of the pile to air, which will trigger spores of fungi that are naturally in the wood to produce the wood rotting organisms. It will mostly be the case that they are submerged except at the close to shore sites of the project. These bars must be placed laterally along the pile toes in order to provide the best protection.

Bamboo mats:

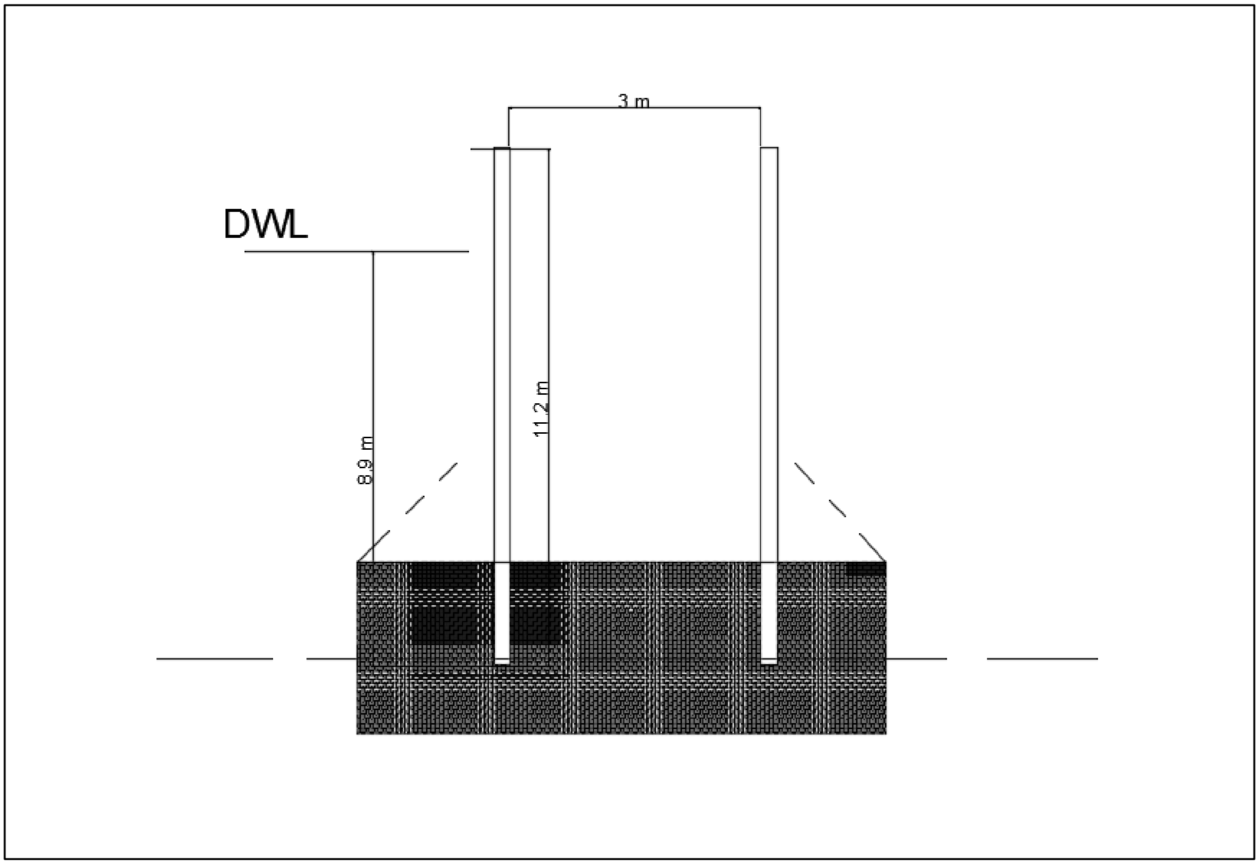


Figure H-4 Piled bamboo wall supported by wooden or bamboo mats

Bamboo mats are made up by weaving together thin strips of bamboo. These can vary in size from 20 mmX2mm to 2mmX1mm. It is evident that for the expected protection against percolation of mud (with grains having even smaller diameters than 1 mm), thin dimensioned strips must not be used as the gaps between these strips could be of the order of magnitude of the dimension, thus allowing too many gaps for the clay to pass through. Relatively thicker mattresses could be held in place by weighty bamboo strips or timber battens making this protection naturally integrated with the mangroves.

At the end of the design procedure for bamboo piled- walls, it is also important to mention some of the properties of the bamboo that are critical and must be clarified for such a situation. Untreated bamboo constructions are usually supposed to survive a lifespan of 1-3 years when in contact with soil and atmosphere even in humid conditions. This is advantageous since the purpose of the intended structure is within this proposed lifespan. However, repeated spells of wetting and drying tend to weaken the structure. Considering a predominantly tidal environment in Mumbai, this could be a critical problem and may hasten the process of failure. Another area of concern is infestation of these bamboos by insects that can slowly destroy the strength of the structure. There are treatment methods available for the bamboos before they are used in construction. Chemical treatment methods using CCA (Copper Chrome Arsenic composition), CCB (Copper Chrome Boron composition) or other oils. However, the effect of such chemicals on vegetation such as mangroves is not certain and therefore may be a concern. Since this is a project oriented not solely for construction but for providing a conducive environment to the vegetation, such chemical treatment methods must be ruled out. Instead, physical methods such as soaking in water (pynot), clump curing and seasoning are available which could help in making the bamboos slightly more resilient to infestation by organisms.

Appendix I Geotextile filter design

The soil particle sizes in the area of interest have already been outlined in the report. The predominant composition is that of fine-grained soils, mostly clay less than 0.063 mm in diameter. This puts the soil in the category of having high clay content, definitely greater than 20 %. Also, since the clay here is not specially being used for any construction purposes, the dispersive behaviour is not so important.

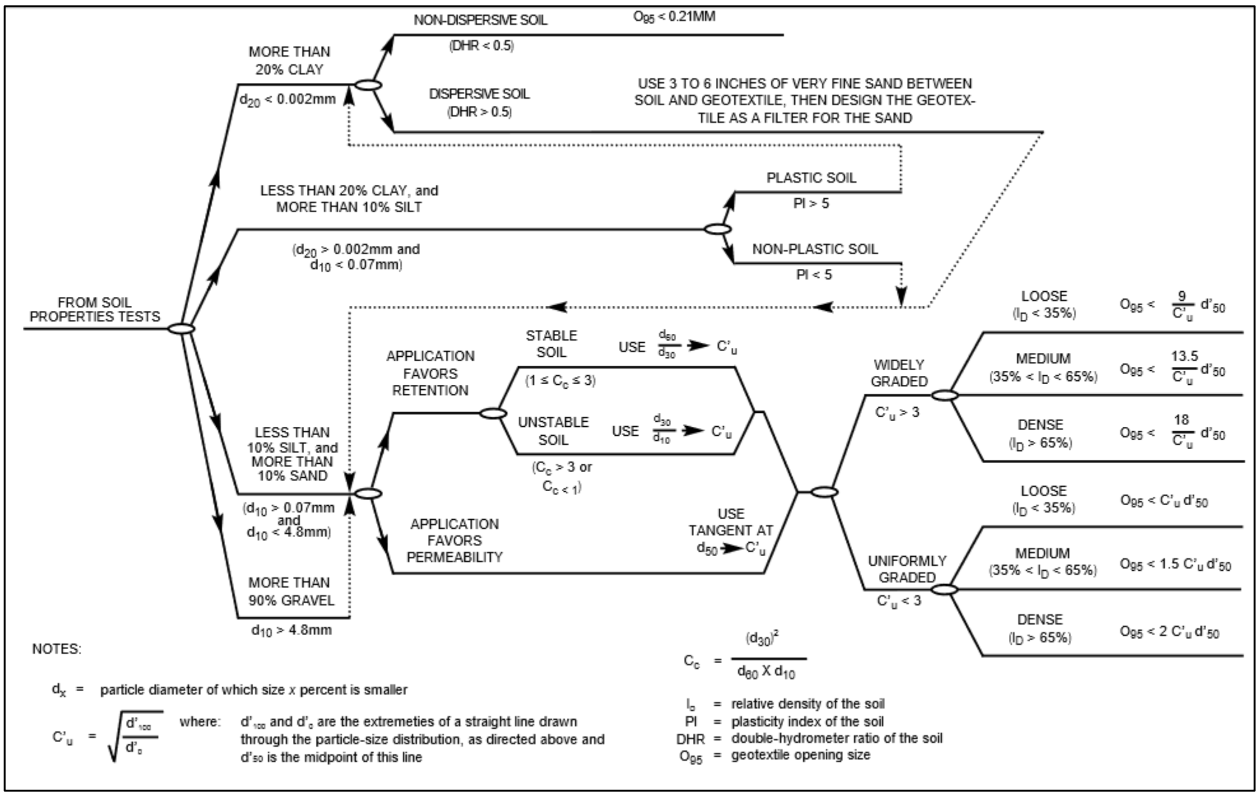


Figure I-1 Representative table for choice of filter for different kinds of soil (Nicolon, n.d.)

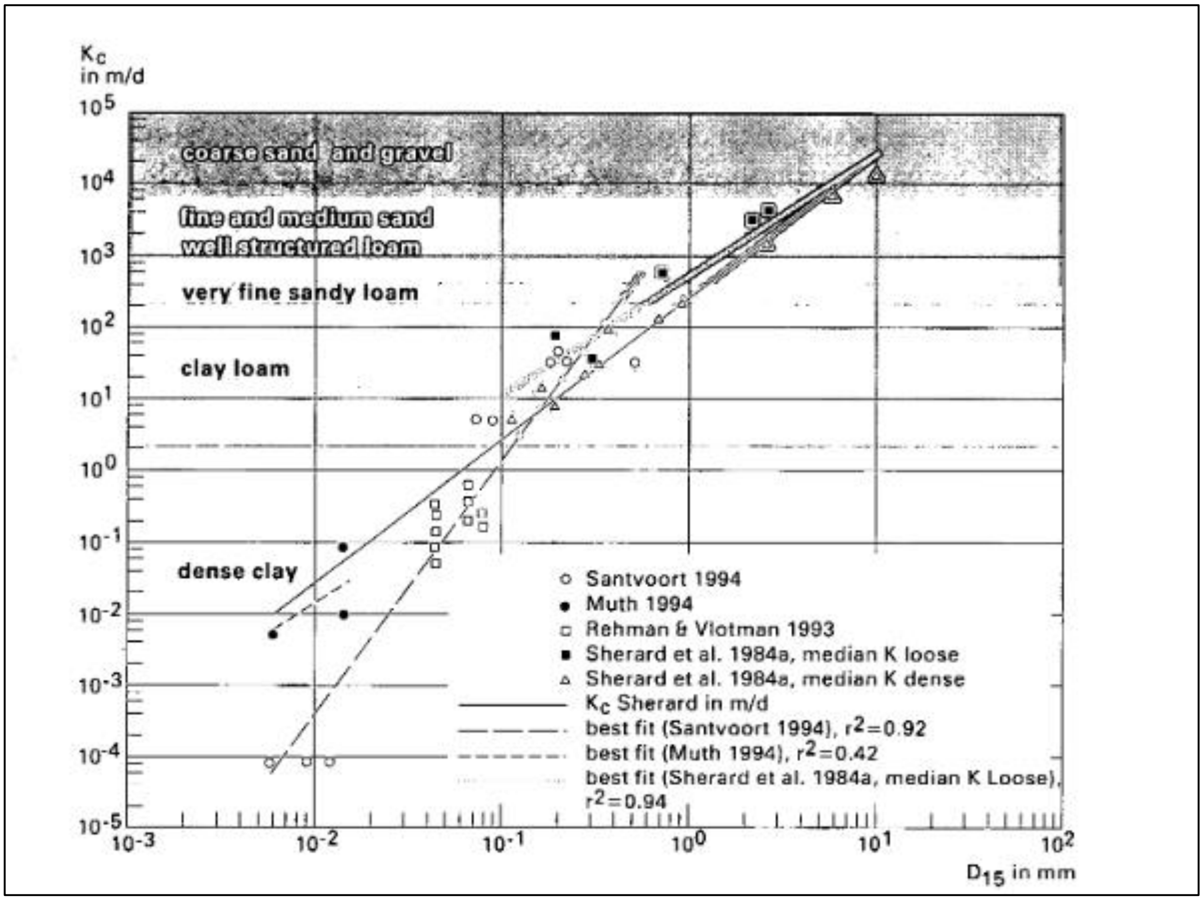


Figure I-2 Diagram showing hydraulic conductivities of different soil types against particle size

The hydraulic conductivity of clay is with the sizes specified in the area of interest should be of the order of 0.1 metre per day. From certain sources in agriculture, this value centres around 0.2 m/day. The gradient value for most vertical drains is taken to be 1.5. Therefore the k_g value for the geotextile should be greater than 0.00035 cm/s or about 0.3 m/day. This value is supposed to be calculated as a product of the permeability and the thickness of the geotextile filter, and is available with most major manufacturers.

Taking, for example the NBT20/200 Geomatex geotextile, permeability is given as 100 l/sq.m./s. This provides a k_g value of about 0.00003 cm/s drainage with a minimalistic thickness of few millimetres. Also, opening size for this geotextile is about 0.075 mm which is lesser than the limit of 0.21 mm. Even the geotextiles with lowest permeabilities in these series satisfy the NBT20/800 or the NBT10/800, satisfy these requirements and therefore all geotextiles in these series can be acceptable for the said purposes.

About jute geotextile filters:

Lee et al. (1989, 1994, 2003) had designed and studied the laboratory and field performance of PVDs made from jute and coir to improve soft clay deposits.

They had used two layers of jute burlap as filter sheath. The AOS of the filter sheath was in the range of 200-600 microns. Their study shows that clay of near liquid limit (70%) did not enter the drain core during installation as well as consolidation process, but were retained by the two burlap layers. From the series of permeability tests, they conclude that the AOS of the jute filter layers need not be too small to prevent clay intrusion into the core, but a larger AOS of the order of 200-600 microns is beneficial in tapping pervious seams and lenses in clay deposits.

They also conclude that the jute filter sheath quickly reached quasi-stable flow condition, but the loss of soil particles continued which could eventually clog the PVD core.

There exists good filtration compatibility between marine clay – polymer / WJT775/WJT700 filter sheaths for the given hydraulic and soil conditions. Whereas, marine clay - WJT550 filter system is not a better filtration system. Hence, while selecting the woven jute geotextiles as better filters in marine clay, it is essential to prefer jute geotextiles having good fibre network structure with appropriate pore sizes as well.

The apparent opening size (AOS) tests revealed that polymer and WJT775 filters satisfied some of the suggested soil retention criteria adopted by other researchers. Although, WJT700 filter did not satisfy some of the referred criteria, it was able to retain clay particles in marine clay - WJT700 filtration tests similar to polymer and WJT775 filters within the test period. This illustrates that filtration compatibility between marine clay – woven jute geotextile filter system can be achieved with the ratio of $O_{95}/ D_{85} \leq 4.5$ and $O_{50}/ D_{50} \leq 100$ (B S Asha, 2012).

About non-woven polymer geotextiles:

Comparison of dirty and cleaned non-woven geotextiles in a test constructed in Pantai Murni, Malaysia showed a difference in pore size of about 8% (The Free Library, 1995). In terms of consistency in the pore size opening measurements of nonwoven geotextiles, these differences are considered insignificant and it can be concluded that the pore size of the geotextile was not significantly influenced during the period the geotextiles were in service.

The results indicated that both the tensile Strength and the puncture resistance of geotextiles were reduced. Significant reduction in puncture resistance of 40% (reduction factor about 1.66) was recorded in the geotextile.

Geotextile material requirement:

Assuming cylindrical drains of 10 cm diameter and a height of about 9 m, the surface area of one drain wrap would be at least 6 m^2. Considering the dimensions of the region to be drained (about 12 km^2), of such drains may have to be built, a minimum of about 1000 such drains may have to be put in place, for the design that relies solely on vertical drains. If such drains are combined with a compartmental design the requirement will decrease to about 300 drains, because compartment borders will already be occupied by dikes and hence drains will not need to be put in there. Also, for the compartmental and longitudinal design, the target area for drainage will be lesser than that for the vertical-drain-only design. This may be taken as a hint towards the fact the final design could be a combination of compartmentalization and vertical drainage.

Gravel/stone filter design:

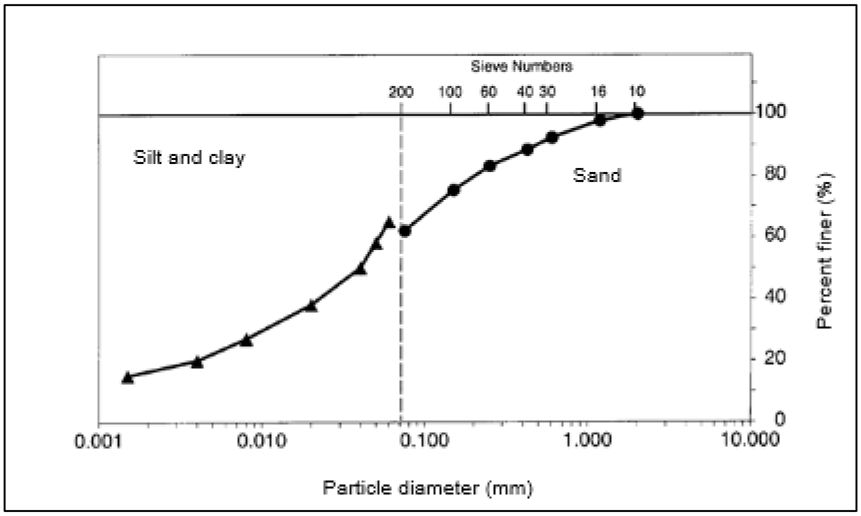


Figure I-3 Sieve curves for silt and clay and on right, the sieve curve for fine sand

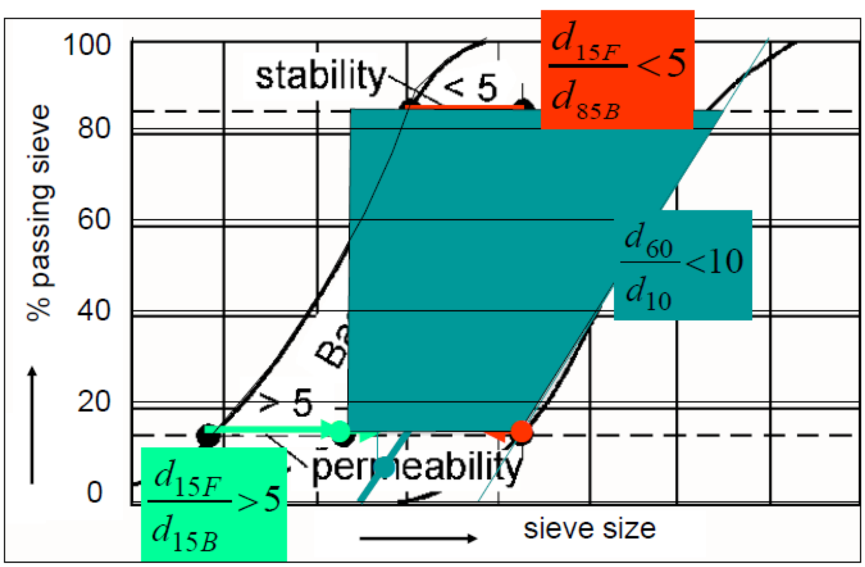


Figure I-4 Diagram encapsulating the design critera for a geometrically closed filter, red (stability), green (permeability) and blue (internal stability of filter)

The design for the geometrically closed filter is being approached by the filter rules. The sieve curve of the base material i.e. clay can be observed on the left hand part of the above diagram. The d_{85} of clay, although not explicit on the figure, but could be extrapolated to a size of about 0.1 mm. In reality, this only means that for the clay particles, a larger variation in diameter on the higher side does not occur. In fact, for higher dimensions, the material would most usually be sand. It also means that since the largest particles in the clay are not that large, the smallest holes in the filter need not be too small. The d_{15} of the filter material should be lesser than at least 0.5 mm. The permeability value however stipulates that this d_{15} should be greater than 5 times the d_{15} of the clay, which would imply it should be greater than 0.01 mm. This implies that the ideal material for such filtering would be fine or medium sand. Such a sand drain could be used for large draining heights, with lower layers consisting of particles with increasingly higher sizes (gravel or stones at the bottom).

Appendix J Diameter of drains and spacing between drains

For diameter of drains, a cue is again to be taken from the concept of French drains and other pipe drains used for clayey soil. Most of them range between 4 to 12 inches in diameter, mostly found as 4-inch, 6-inch or 8-inch diameter drain tiles. We can choose similar diameters which could be 10 cm, 15 cm or 20 cm respectively. It will be seen in the following explanation that drain spacing will not depend so much on drain diameter as much on the conductivity and other parameters. An idea about this can be obtained from the spacings.

There is very little information for drain spacing along shores for the purposes such as in this project. However, there are some solutions available in agricultural practice where they have to drain the clayey soil, in their case not due to overtopping but due to over-irrigation. The required drain spacing between parallel drains necessary to promote trafficability and prevent crop damage from excess water in the root zone by using the Hooghoudt equation.

The parameters required for such calculation, in equivalent terms, are the drainage co-efficient (how much water is to be drained, typically 1 cm/day, the drains diameter and depth (9 m), depth of the underlying restrictive layer (here taken as slightly larger than the length of the drain), minimum water table depth and saturated hydraulic conductivity (0.2 m/day for clay). The minimum water table depth is the depth of water table from surface of soil that the plant can tolerate. Usually, this values are about a feet but since mangroves can tolerate water right up to the surface, a minimal value of 0.1 ft is chosen. The drain spacings for slightly varying diameters between 10-30 cm are all found to be in the range of about 100 m, based on the drain spacing calculator (South Dakota State Climate Department). Note that this is a calculation predominantly used for agricultural purposes and its adaptation, here, however similar in terms of applicability, should only be considered as a rough estimate.

Now, should the drains be large (30 cm) or small (10 cm) in diameter? Having larger drains in this case would mean a minor advantage in drainage property but high material requirement as well as greater area of the rehabilitated land to be occupied by drains. Smaller drains have the dual advantage of being both cost and space efficient. It will be a wiser idea therefore to go with smaller drains, spaced abut hundred meters apart and located reasonably away from confining walls.