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LONGARD TUBE APPLICATIONS MANUAL

PREPARED FOR

ALDEK A-S
LONGARD DIVISION

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PREFACE

In May 1981, Tetra Tech Inc. submitted a report to Aldek A-S entitled "Overview of Longard Tube Application and Design." As a result of recommendations made in that report, Aldek A-S requested Tetra Tech to perform site inspections of various Longard tube installations in Europe, which would broaden Tetra Tech's experience base of projects using the Longard tube system. Site visits to projects in Italy, Belgium, and Germany were made in October 1981.

From information collected previously, and the impression from the site visits in Europe, Tetra Tech prepared this report, "Application Manual for Longard Tube Structures", for Aldek A-S. The intent of this report is to provide possible applications of the Longard tube system in coastal and hydraulic applications. It is not intended to be a substitute for sound engineering design practices which is based on experience and thorough understanding of the hydrodynamic and physical processes, but is suitable for most preliminary planning and design purposes.

Tetra Tech is especially appreciative of the time and information offered by persons who have been involved with the Longard projects. In Italy, our host was Engineer Guiseppe Sarti to whom we are grateful. We would also like to thank Engineers Ferdinando Gambardella and Sergio Montori, the officials responsible for the projects in Italy; Mr. L. Burki of Dragages Decloedt and Fils, and the Belgian officials Mr. Verslype and Mr. Blommer for showing us the projects at Klemskerke and Nieuwpoort; and the German officials Mr. Felden and Mr. Ukena for their valuable input into the project at Norderney. The people from Aldek A-S made this project possible, and we extend our gratitude to Mr. Peter Thomsen, Mr. Willy Konge, and Mr. John Larsen.

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LONGARD TUBE APPLICATIONS

1.1 SEA WALLS, BULKHEADS AND REVETMENTS

Seawalls, bulkheads, and revetments are structures placed parallel to the shoreline, to separate a land area from a water area. The primary purpose of a bulkhead is to retain or prevent sliding of the land, with an additional purpose of providing protection to the upland against damage by wave action. The primary purpose of a seawall or revetment is to protect the land and upland property from damage by waves. There are no precise distinctions between the three types of structures, and often the names are used interchangeably. These structures are generally used where it is necessary to maintain the position of a shoreline, where there is a scant supply of littoral material, and little or no protective beach, or where it is desired to maintain a depth of water along the shoreline.

The planning of seawalls, bulkheads, and revetments constructed using the Longard tube system is an elementary process, since their primary function is simply the maintenance of fixed boundaries. Factors in designing these structures are: use and shape of structure, location with respect to shoreline, length, height, and stability. Earth and water pressures, and the bearing capacity of the foundation determine tube stability.

Of particular consideration in the design of these structures are their placement relative to possible water levels and important land areas, and the specific details of connections or transition sections which must be designed to prevent unwanted erosion, settlement or undermining of the structure. Adequate drainage of the backfill material, especially for seawalls subjected to wave overtopping, must be maintained to prevent piping of fines through the soil matrix, or the liquefaction of backfill material resulting in excess lateral pressures on the retaining tubes. Employing gravel blankets, drains and synthetic filter materials are possible methods for controlling pore pressure and groundwater. End or transition sections of these structures should be tied back into the shoreline to prevent flanking by wave action resulting in the loss of backfill around the ends of the structure.

Typical plans and cross-sections of seawalls, bulkheads, and revetments are shown in Figures 1 through 3.

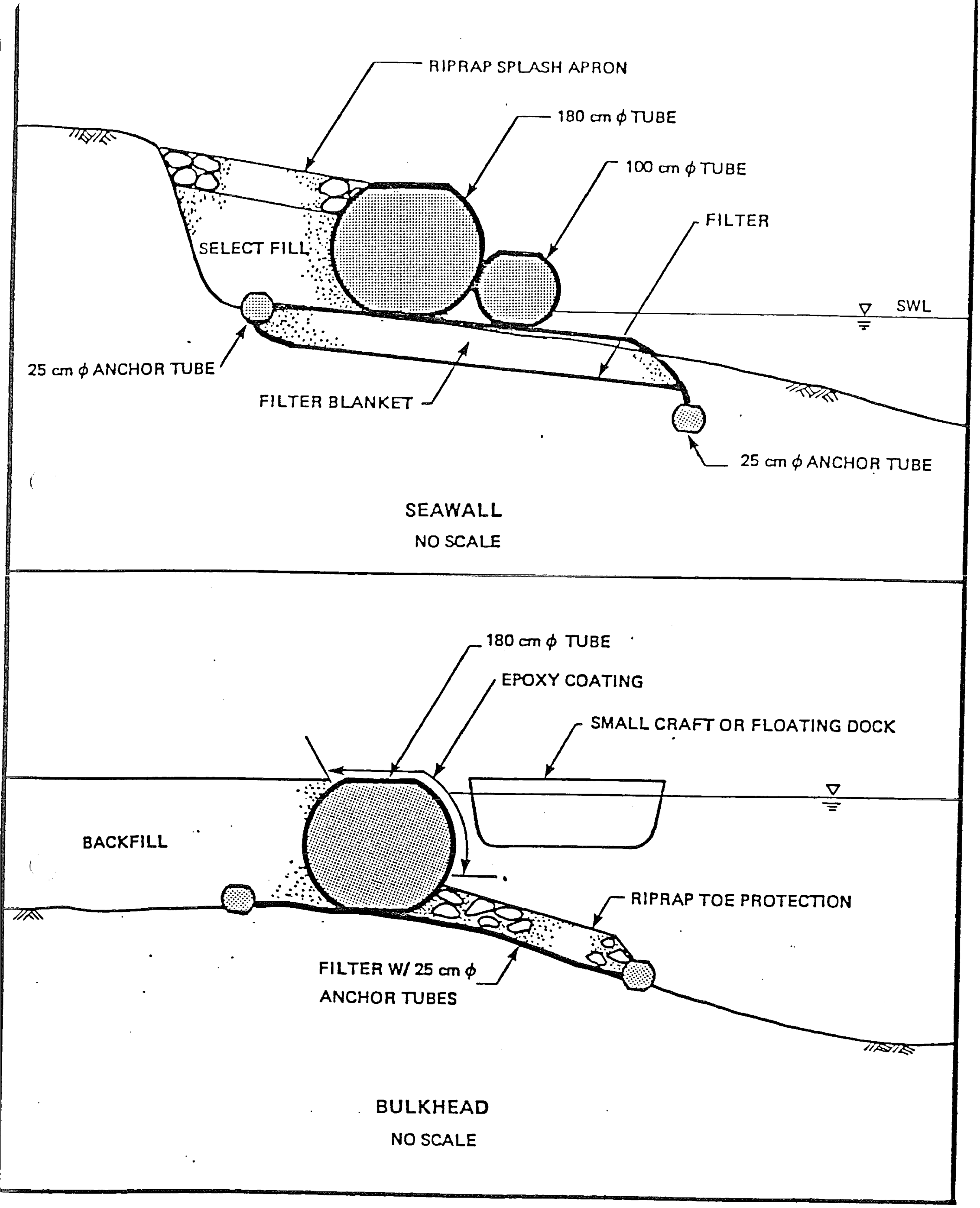


FIGURE 1: SEAWALL AND BULKHEAD SECTION

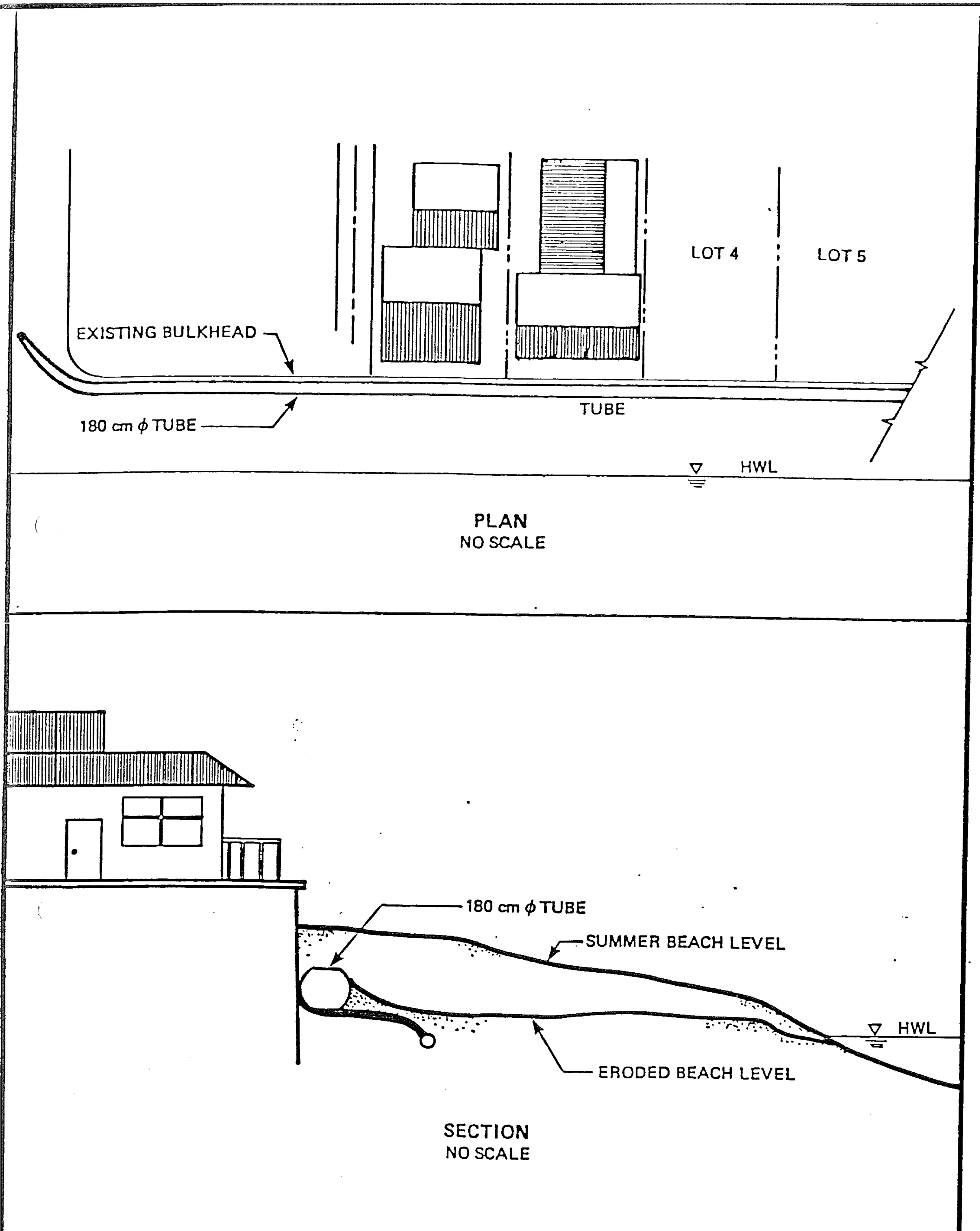
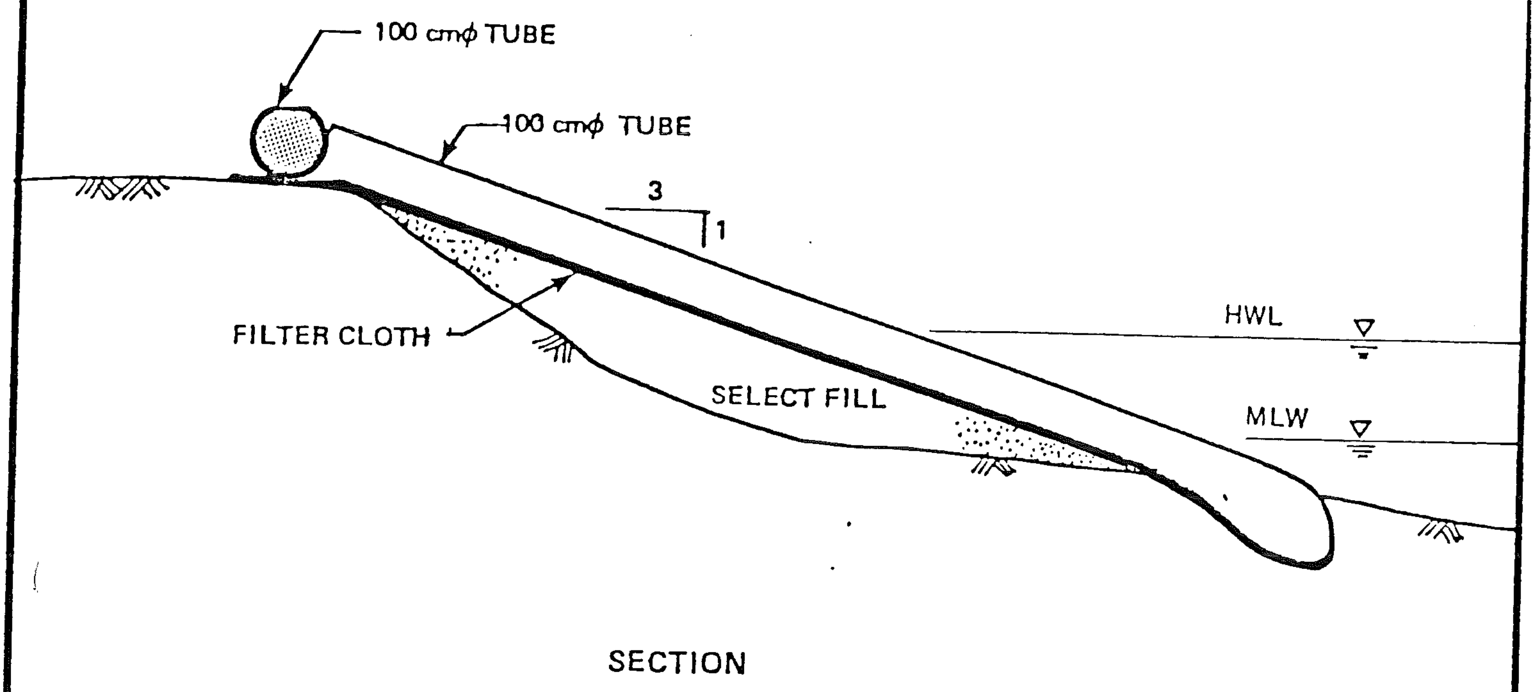


FIGURE 2: SEAWALL FOR EMERGENCY PROTECTION



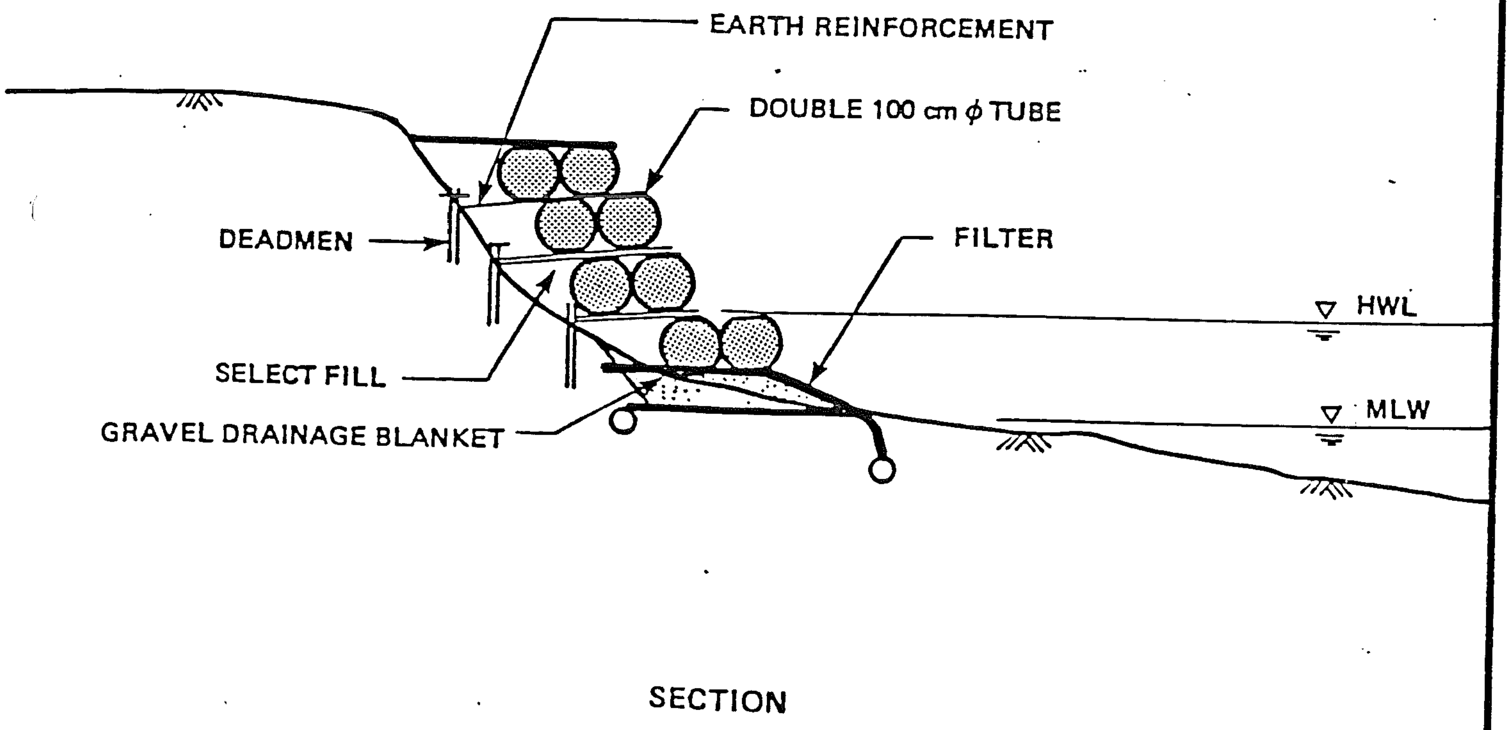


FIGURE 3: REVETMENT SECTION

1.2 ARTIFICIAL CREATION OF PROTECTIVE BEACHES

Structures which cause littoral material to be artificially retained upon the shoreline can effectively reduce erosion problems by producing the protection qualities attributable to natural beaches. Artificial beaches are formed from the retained material, causing wave energy to be dissipated before reaching erodible dunes or bluffs.

Techniques for retaining littoral material generally involve groins, offshore breakwaters, or artificial beach nourishment (periodically placing sand on the shoreline by dredging or other means). Periodic beach nourishment in conjunction with groins or breakwater construction is generally the most effective measure for retaining a protective beach. In any case, the creation of artificial beaches requires an adequate supply of littoral material.

1.2.1 Groins

Groins are among the oldest known methods of shore protection. Where there is an abundant supply of littoral drift, the role of the groin is to build or widen a beach through the trapping of the sediment moving along the shore. Furthermore, properly designed groins will reduce the longshore transport of sediment by reorienting the compartmented shoreline, so that it will be in closer equilibrium to the predominant wave direction. Possible disadvantages of groin structures are increased offshore sediment losses resulting from the formation of concentrated rip currents, the formation of scour holes around the groins which maybe hazardous to bathers on recreational beaches, and the reduction in the supply of littoral drift available to the downdrift coast. The latter disadvantage can be particularly serious where experience has shown that attempts to mitigate an erosion problem with a groin field has often resulted in an aggravated erosion problem downdrift.

The length, spacing, elevation, and number of groins are very important and should be developed through site specific design efforts by a qualified coastal engineer. However, for planning purposes, the general guidelines presented below can be followed. The typical groin extends from the breaker zone to the top of the beach berm, and landward to a point where the groin will not be endangered by flanking during its design life. The minimum crest elevation is set equal to the height of the desired beach berm. The height of the shoreward section of groin above this level is then a careful trade-off between desired levels of beach buildup and sand by-passing quantities necessary to nourish downdrift beaches. Groin spacing is determined by the predicted shape of compartmentalized shorelines, and as a general guideline is usually two to three times the length of the individual groins.

The shape of future shorelines are generally assumed to align its orientation to parallel the incident wave crests under predominant conditions, taking into account wave reflection, refraction, and diffraction. Beaches with wave climates having a distinct seasonal characteristics in wave direction should consider prevailing wave conditions by season in addition to annual predominant wave conditions.

Generally, groins should be planned as a field of multiple structures of uniform length rather than as an individual structure. Groin fields should be constructed in stages, starting at the extreme downdrift end of the area to be protected. In this way, the effect of a single groin can be studied carefully before completing the layout of the groin field.

Example groin plans and cross-sections are shown in Figure 4.

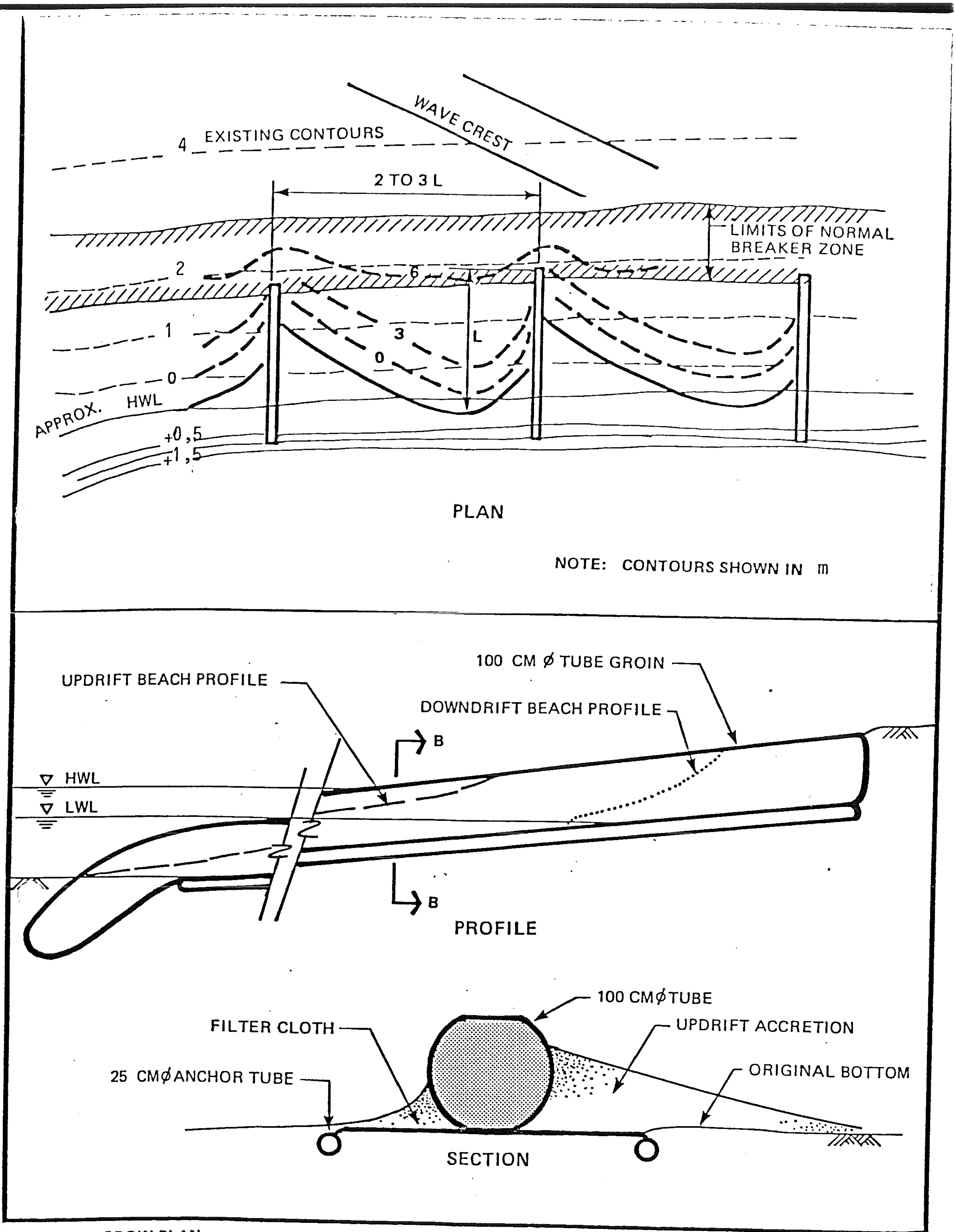


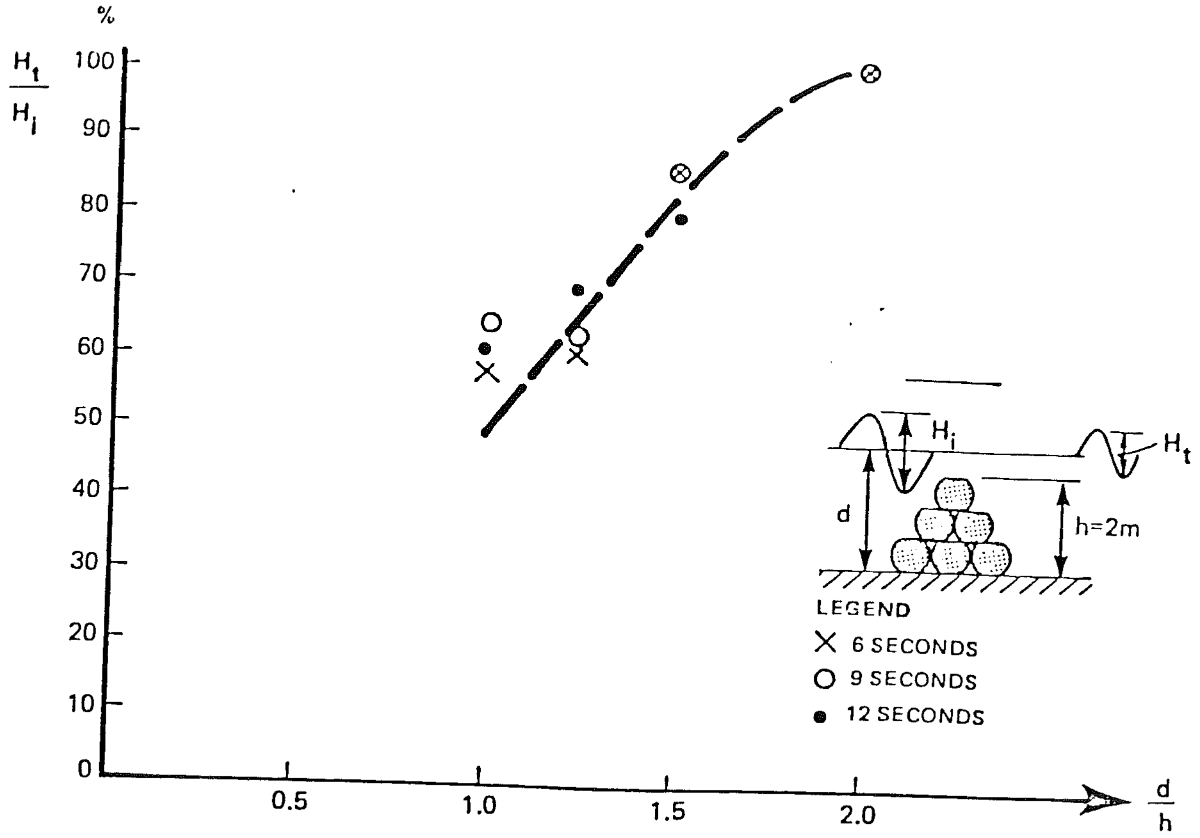
FIGURE 4: GROIN PLAN

1.2.2 Offshore Breakwaters

Offshore breakwaters can be used to mitigate the force and vary the direction of waves striking the shore, and therefore can be used to reduce shore erosion. They are located away from the shore, and are normally partially submerged during all tidal levels. Breakwaters tend to reduce littoral transport in the lee of the structure, and under certain conditions will induce the formation of a tombolo. Offshore breakwater systems provide erosion protection without impairing the usefulness of the beach and can provide sheltered water for bathing. Because sediment transport is directly related to the incoming wave action, the degree of wave attenuation by an offshore breakwater will determine the extent to which littoral drift will be impounded in the lee of a breakwater.

Hydraulic model studies performed at the Danish Hydraulic Institute have examined the effectiveness of Longard tube breakwaters in reducing wave height, and results of these tests may be used in calculating incident wave characteristics. Figure 5 presents wave height transmission coefficients as a function of the relative submergence of a Longard tube breakwater.

Determination of the optimal alignment, height, length, spacing and distance offshore for breakwaters should only be established with full consideration of site specific conditions. If shoreline stabilization is the primary objective, it is usually desirable to employ a series of segmented offshore breakwaters which will regulate the rate of longshore sand movement without completely impounding littoral materials. Figure 6 presents typical plans and cross-sections for a detached breakwater system.



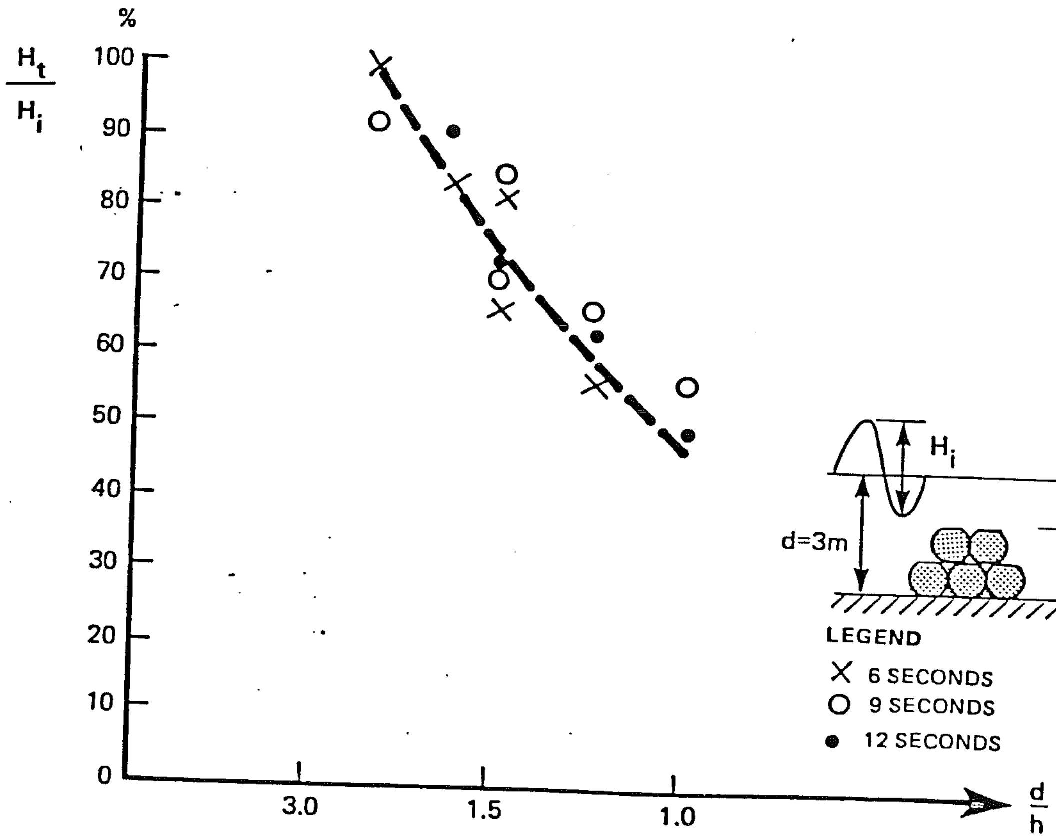


FIGURE 5: WAVE TRANSMISSION OVER SUBMERGED BREAKWATERS FROM SCALE MODEL TESTS (DHI, 1970)

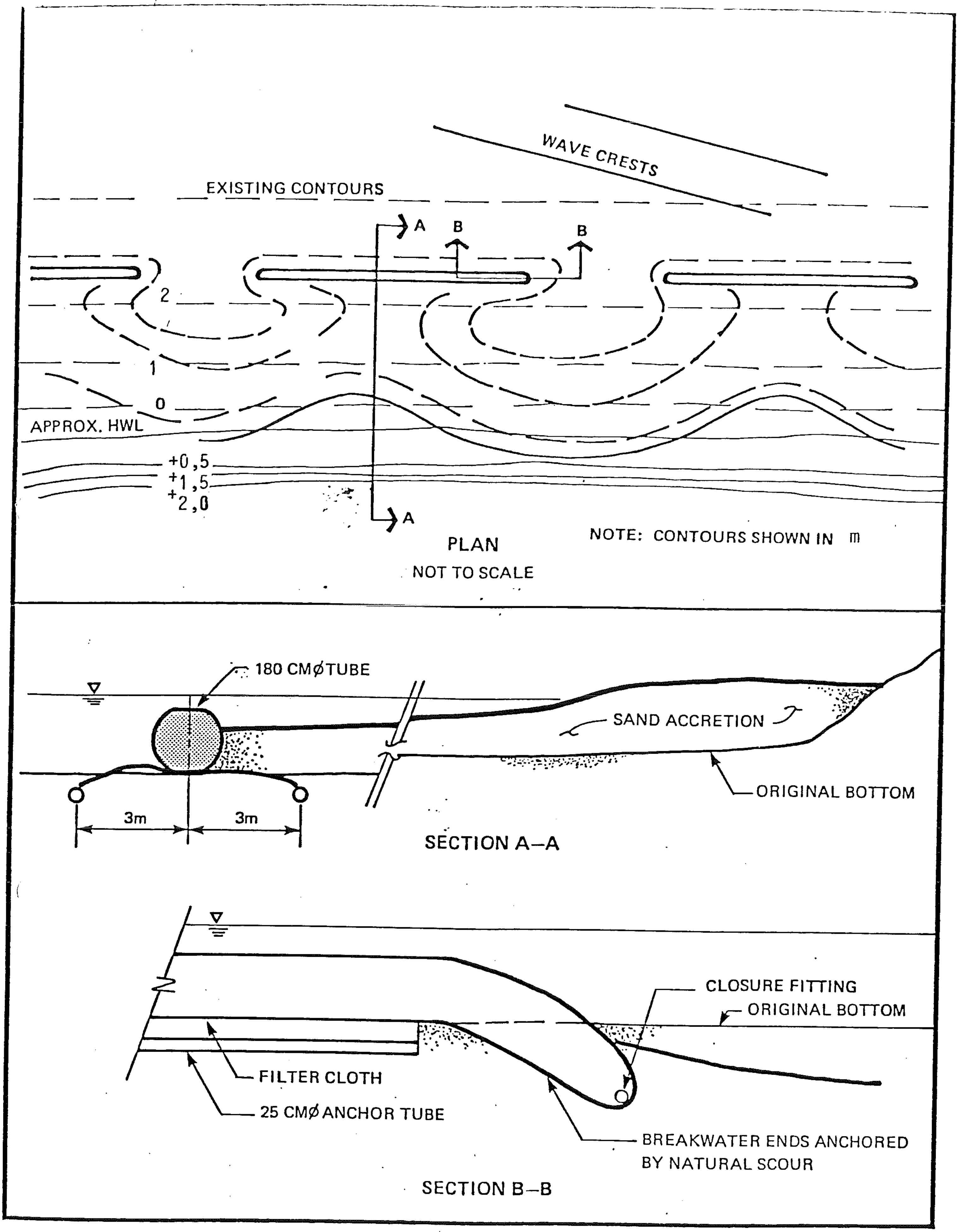


FIGURE 6: OFFSHORE BREAKWATER PLAN

1.2.3 Perched Beaches

The perched beach concept involves retaining a wide protective beach or shallow offshore area by means of terracing with the construction of beach retaining sills. Wave energy is dissipated while propagating over this shallow region by breaking and through bottom friction. Waves hence have a reduced erosive effect upon impinging on the shoreline. In general, perched beach shore protection schemes are most applicable along shorelines with a relatively flat, gently sloping nearshore profile. As with other protective beach concepts, an abundant supply of littoral material in the active wave zone is necessary for beach sills to function properly.

In the planning of a perched beach system, the widest practical beach width should be obtained for maximum effectiveness in wave energy attenuation. Several terraces may be the most effective in order to obtain shallow water for all tidal stages. Figure 7 demonstrates the perched beach concept.

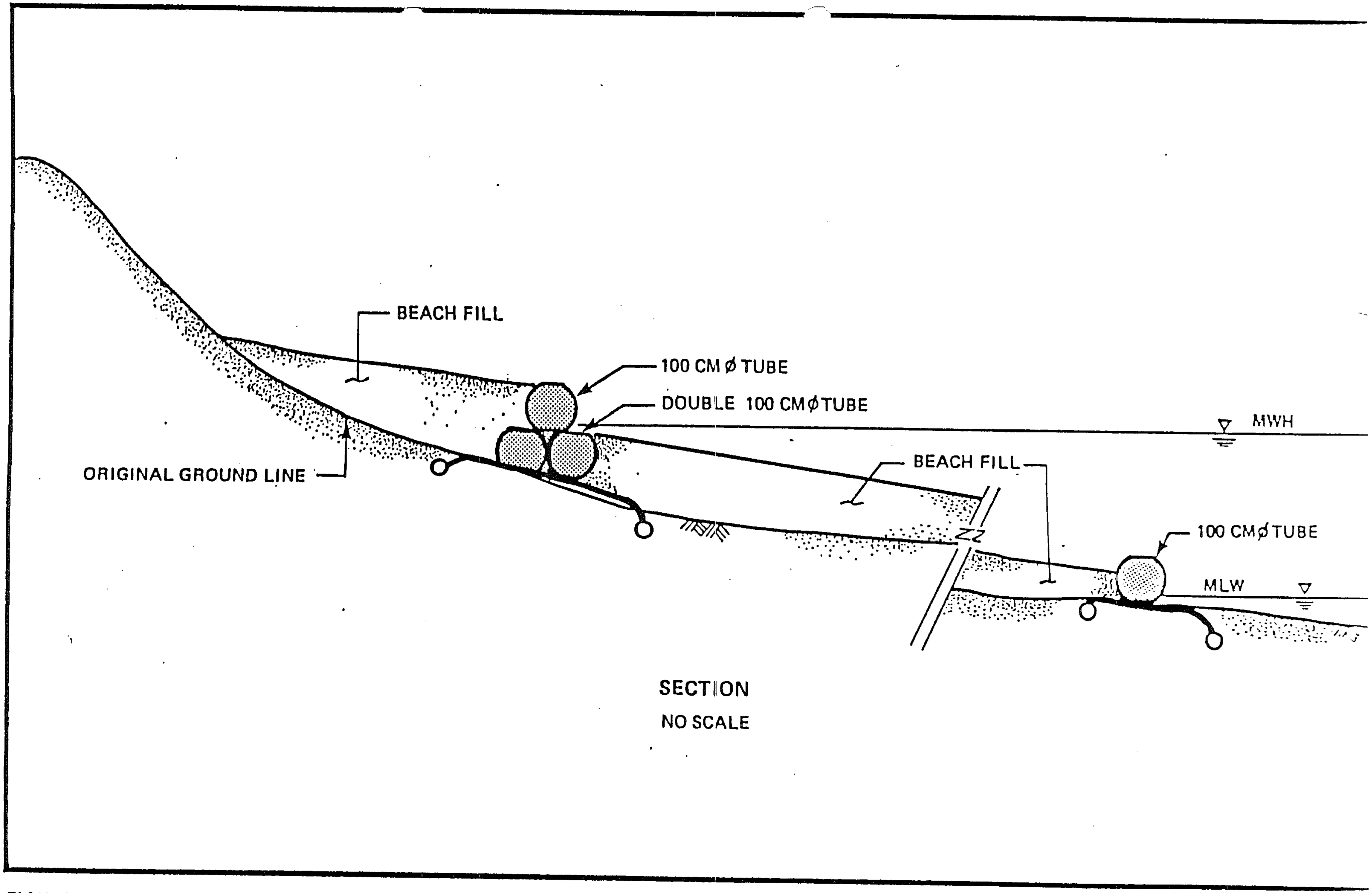


FIGURE 7: PERCHED BEACH CONCEPT

1.2.4 Composite Schemes

A combination of various schemes for retaining a protective beach is often the preferred method for protecting a shoreline while providing the maximum benefits. As an example, groins are useful in stabilizing shorelines but can be detrimental to the recreational value of a beach due to nearshore scour holes hazardous to bathers, the focusing of wave energy and their unsightly appearance. Also, during exceptionally high storm wave conditions, rip currents formed along the groins transport littoral drift farther offshore than what would normally occur without the groins, thus removing this material from the active littoral zone. To lessen these adverse effects, a composite scheme of groins and offshore breakwaters can be used. Safe bathing areas are created in the lee of the breakwaters, while shore realignment and beach buildup can still be obtained with the use of groins. The reduction of incident wave energy by the breakwaters will also decrease the intensity of rip currents and the loss of material offshore.

A composite scheme combining revetments with massive beachfills can also be employed to create artificial headlands and a series of pocket beaches. Figure 8 presents composite schemes with the Longard tube system.

1.3 BLUFF STABILIZATION

Erosion of coastal bluffs is caused by groundwater or soil discontinuities, surface runoff and toe erosion by wave action. In all cases of bluff stabilization, protection of the toe from wave erosion is of primary concern. Upper bluff stabilization cannot be effective until wave erosion is terminated. Methods of mitigating toe erosion are identical to those presented as those for hardening the shoreline or retaining protective beaches. That is, seawalls, bulkheads, revetments, groins, offshore breakwaters, perched

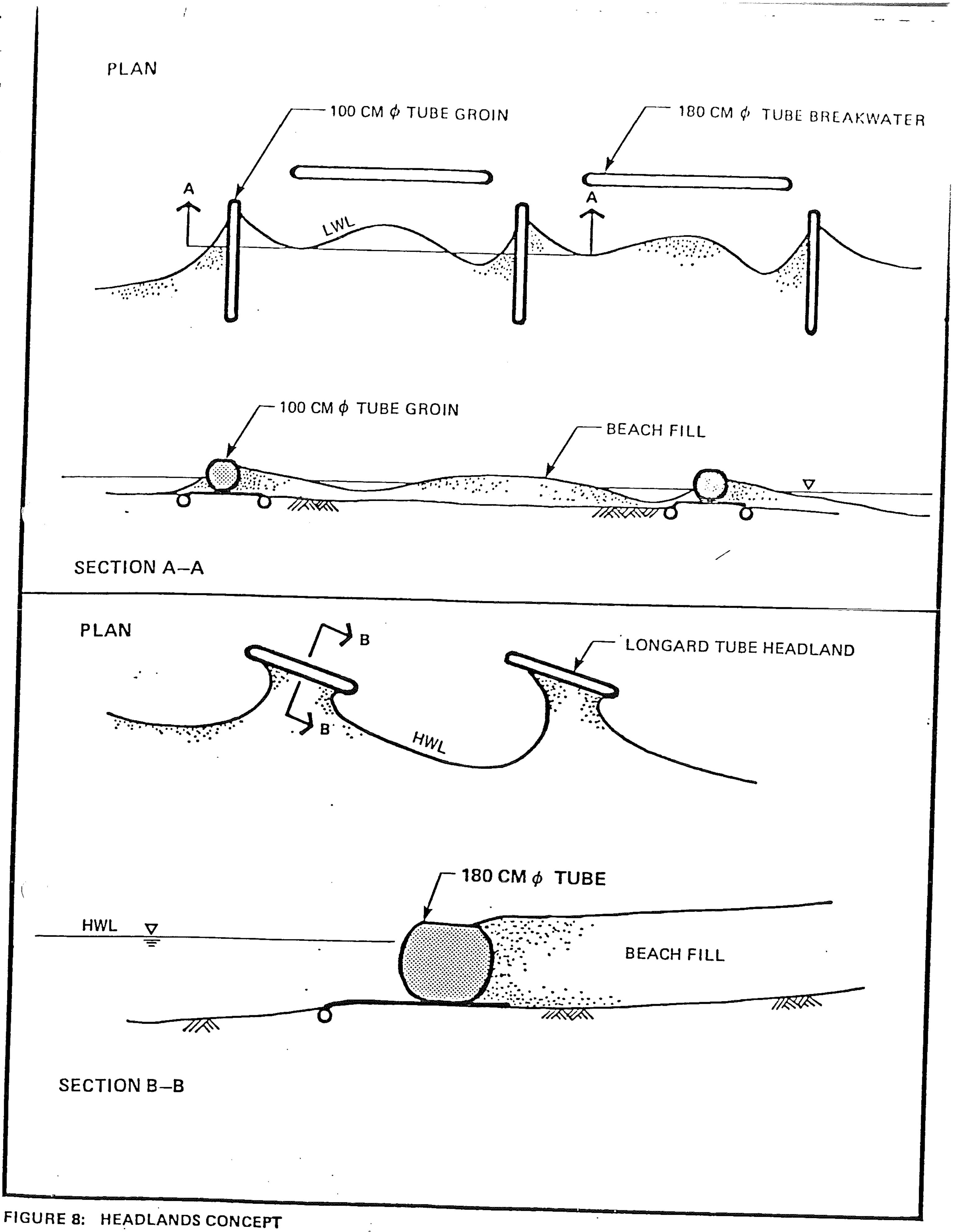


FIGURE 8: HEADLANDS CONCEPT

beaches and combinations of those are all possible methods for providing toe protection to eroding bluffs.

Stabilization of the upper bluff can be accomplished by installing Longard tube retaining walls along with appropriate grading, vegetative plantings, dune fences and drainage provision. Controlling groundwater seeps and gullying effects of surface water runoff is the major objective in stabilizing the upper bluff. Figure 9 presents the design features of a stabilized coastal bluff.

1.4 MISCELLANEOUS USES

The unique characteristics of the Longard system presents various applications which are often not feasible by other construction methods. The flexibility of the system along with the ability to fill a tube in normally inaccessible areas tremendously reduces site preparation and access requirements along with associated mobilization and demobilization costs. Equipment needs are minimal, normally consisting only of the patented filling machine, water pump, and front-end loader. Examples of miscellaneous applications are presented below.

1.4.1 River Training/Flood Control Structures

Bank stabilization and erosion control is somewhat similar in concept to that applicable on the open coast; the major differences being the absence of large waves and normally a uni-directional flow pattern. Groins are typical structures constructed with Longard tubes and used to reduce bank erosion and to stabilize meandering rivers.

Groins, spurs, or spur dikes, function by contracting river flow. These structures are used to establish normal channel width; direct the axis of flow; promote scour and sediment deposition where required; and to build up new banks. In the use of river groins, experience is chiefly

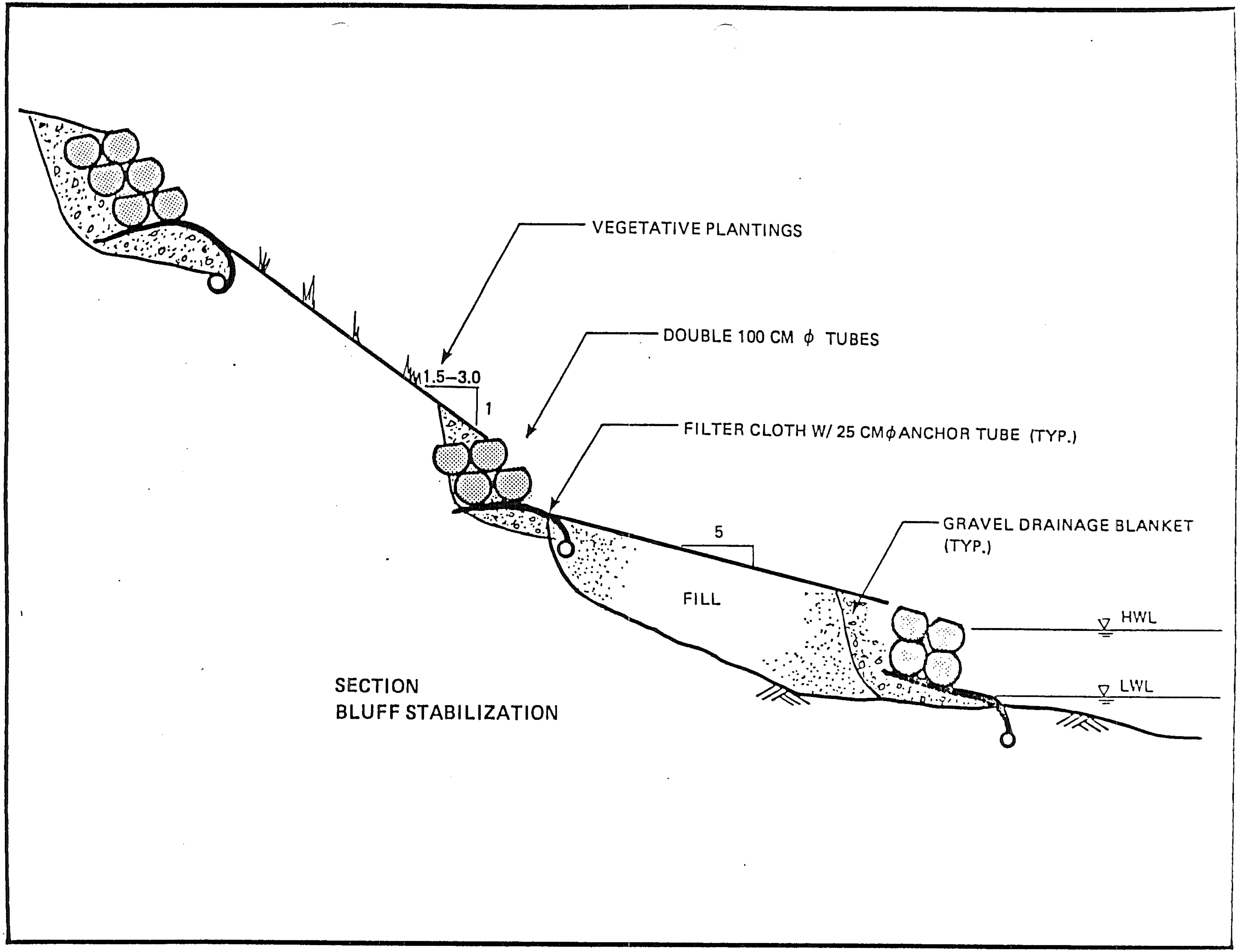


FIGURE 9: BLUFF STABILIZATION

relied upon in determining appropriate sizing and alignment of the groins. In general, large isolated groin structures should be avoided where a group of smaller groins, in a groin field, can be used to obtain the desirable results more gradually. Abrupt modifications to the natural river flow regime may often result in undesirable effects along another reach of the river, especially if a structure is installed causing a sharp change in flow direction. Figure 10 presents an example of Longard tubes used to control erosion along the Po River, Italy.

1.4.2 Jetties

A jetty is a structure extending into the water to confine river or tidal flow in a channel, and to mitigate shoaling of the channel by littoral drift. When located along navigable entrance channels, they also serve to eliminate cross currents and to provide wave protection.

The design of jetties requires a careful understanding of both the littoral regime and hydraulic characteristics of the inlet or channel. The effect of a jetty on longshore sediment transport, tidal prism, salinity intrusion and inlet flow characteristics must be assessed to avoid undesirable impacts, and to achieve desirable results. Because of the complex nature of this design problem, the use of general guidelines for optimum inlet cross-sectional areas and jetty sizing and alignment are not recommended. However, once optimal jetty locations and alignments are determined, the use of Longard tubes in the actual construction of the jetty is a simple matter. For small inlets in areas of limited tidal range, single 100cm or 180cm diameter tubes can serve as jetties for inlet stabilization.

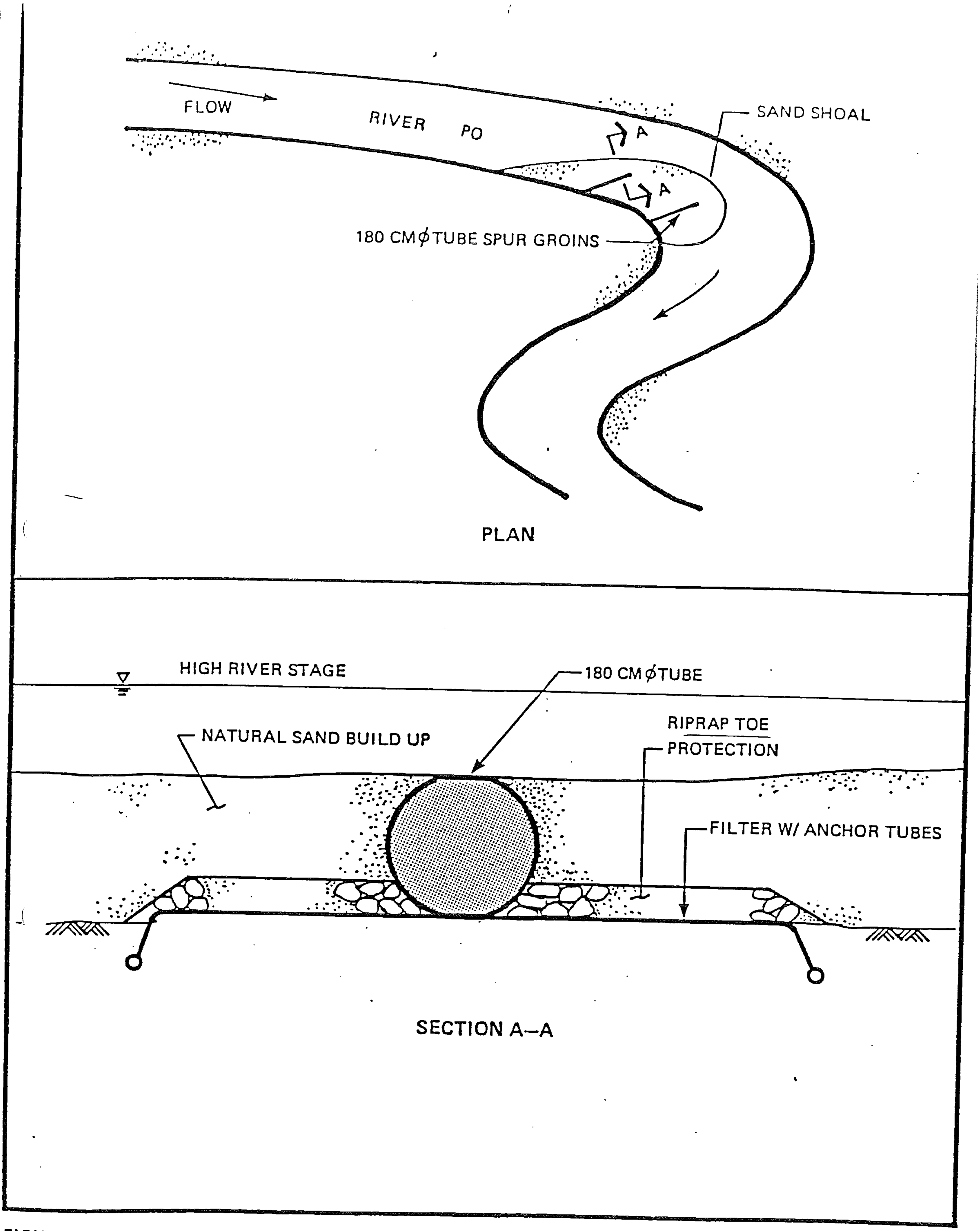


FIGURE 10: RIVER TRAINING GROINS

1.4.3 Dredge Containment Dikes

Dredging of harbors, channels and other navigable waterways often involves spoils with a high percentage of fines. Hydraulic dredging of this material with disposal onshore creates a need for retainment dikes or berms to prevent this slurry from running back to the sea, while providing a wide settling basin. The benefits in using dredge containment dikes are twofold: first, a greater amount of dredge spoil is retained and second, there is reduced turbidity of the returning dredge effluent which is less disturbing to the environment.

The use of Longard tubes for containment dikes is an economic alternative to conventional earth dike construction. A 180cm tube can provide a dike approximately 1.8m high requiring less fill, with select material, and a shorter construction time than the conventional earth dike.

1.4.4 Artificial Islands

Artificial islands constructed with the Longard tube system presents an additional use for both underwater construction techniques and shore protection schemes. Uses for these islands include working platforms for offshore energy exploration and development, offshore terminals and recreational purposes. Advantages for using the Longard tube system for artificial island construction include the reduction of required fill materials, speed of construction, and low relative costs. Island removal is also easily accomplished as is often required for temporary energy exploration structures. Figure 11 presents a conceptual island construction scheme using the Longard tubes.

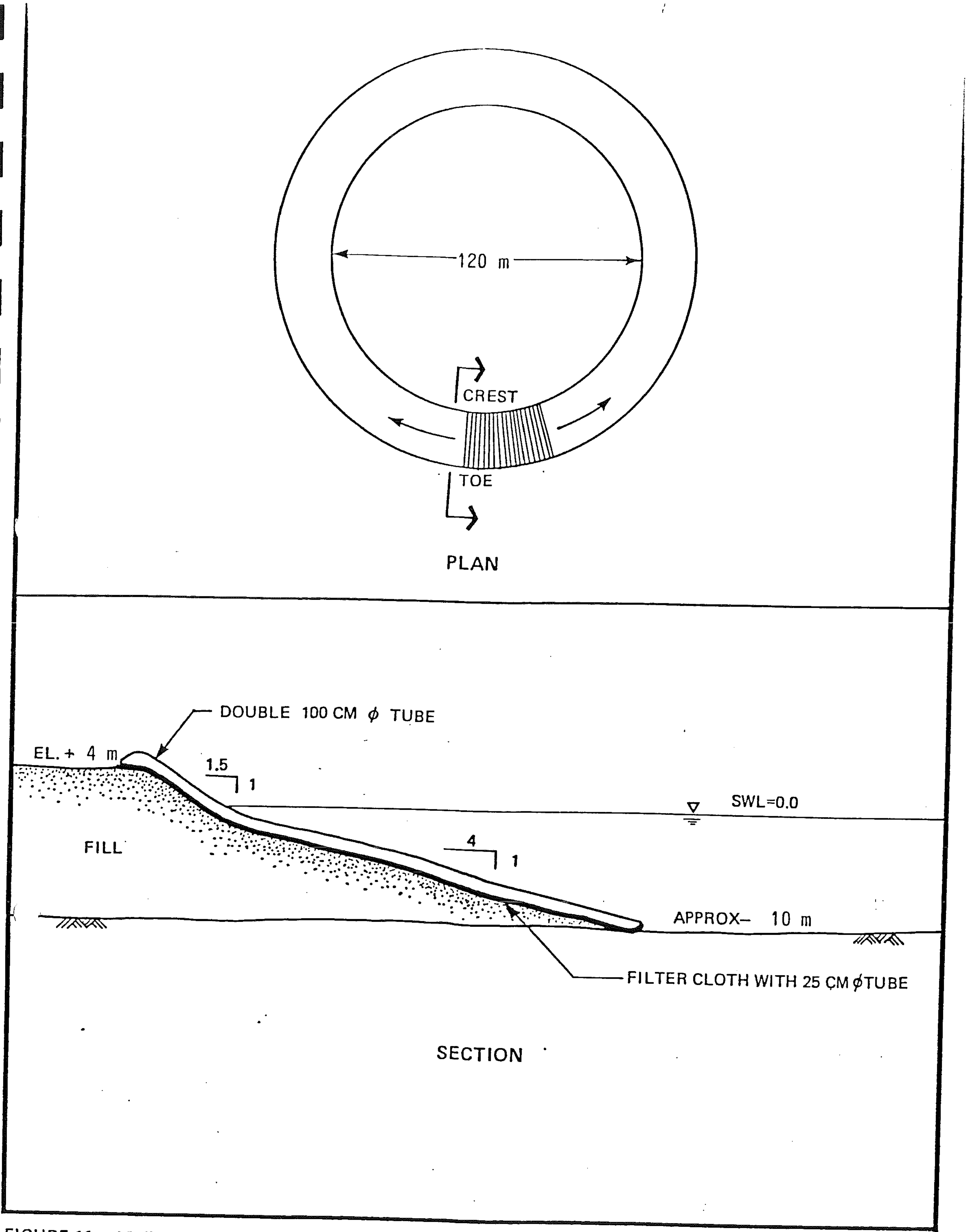


FIGURE 11: ARTIFICIAL ISLANDS

2.0 GENERAL DESIGN GUIDELINES

2.1 PHYSICAL PROPERTIES

2.1.1 Tube Materials and Sizes

The Longard tube actually consists of two tubes: an inner tube of impermeable polyethylene sheet and an outer tube of woven polyethylene fabric. The function of the inner tube is to aid in the filling process. Structural strength is provided by the outer tube, which is woven of a high density polyethylene fabric, is UV-stabilized and has great resistance to rot, oil and chemicals likely to occur in coastal zones. Technical specifications for the tube material are presented in Tables 1 through 4.

Longard tubes are available in standard diameters of 25, 100 and 180cm and in lengths up to 120 m. The 25 cm tube is typically used as an anchor for the filter cloth; where the filter sheet and anchor tube are sewn together with specified filter widths. Single 100 or 180cm tubes comprise the primary structural elements of the Longard tube system. Double 100 cm tubes are also available. These tubes are woven together into one unit and are filled simultaneously. The use of double 100 cm tube is recommended where extra latteral stability is required.

2.1.2 Filling Materials

The Longard tube system can be filled with any cohesionless soil capable of being transported hydraulically and graded to have a maximum grain size diameter of less than approximately 2-5 cm. Naturally occurring beach sand at the project site is the usual choice of fill material, however, select fill material should be considered if a local borrow area is not available, or if the fines or coarse gravel content exceeds approximately 15 percent. Typical fill material consists of beach sand having a grain size ranging from 0.1 to 5.0 milimeters in diameter.

Achieving a relatively high unit weight for a filled tube is essential for stability under severe hydraulic conditions where buoyancy effects can reduce tube stability. Figure 12 can be used to approximate the unit weight of a filled tube with the knowledge of available fill material. For a fully filled tube, the relative density will be firm to dense, with a range in relative density of up to 90 percent. Table 2 presents typical values of weight per unit length of filled Longard tubes.

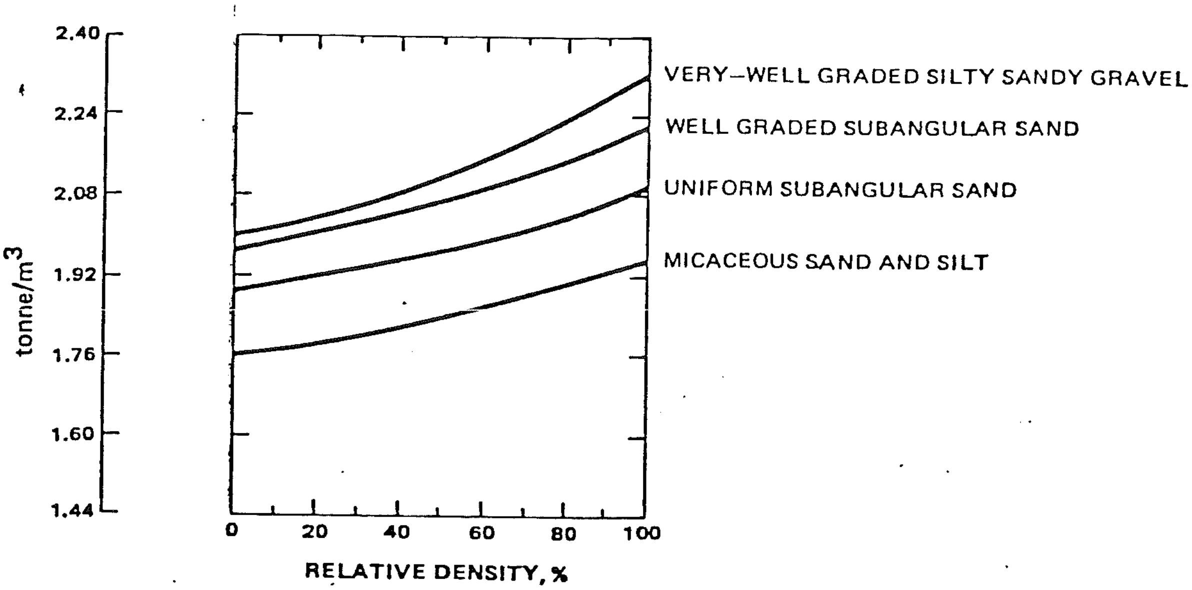


FIGURE 12: SATURATED WEIGHT VS. RELATIVE DENSITY OF FILL MATERIAL

TABLE 2

IN-PLACE WEIGHT OF LONGARD TUBES

TUBE DIAMETER (cm)	SECTIONAL AREA (m^2) with 98% Fullness	WEIGHT OF TUBE (tonne/m)		
		Low Density (1.95 tonne/m^3)	Typical Density (2.08 tonne/m^3)	High Density (2.34 tonne/m^3)
25	0.0491	0.10	0.10	0.11
100	0.770	1.5	1.6	1.8
180	2.49	4.9	5.2	5.8

2.1.3 Geometric Properties

Nominal dimensions for Longard tubes are given as the diameter of an equivalent cylinder having the same diameter as the woven tube. In practice, the cross-sectional shape of a filled Longard tube is approximated by a circle with a flat top, the properties for which are shown in Figure 13. Field experience has demonstrated that it is possible to fill Longard tubes to 98 to 99 percent of their theoretical maximum capacity; while 95 percent fills are easily obtainable with well-graded sand. Figure 13 can be used to obtain dimensions of filled Longard tubes.

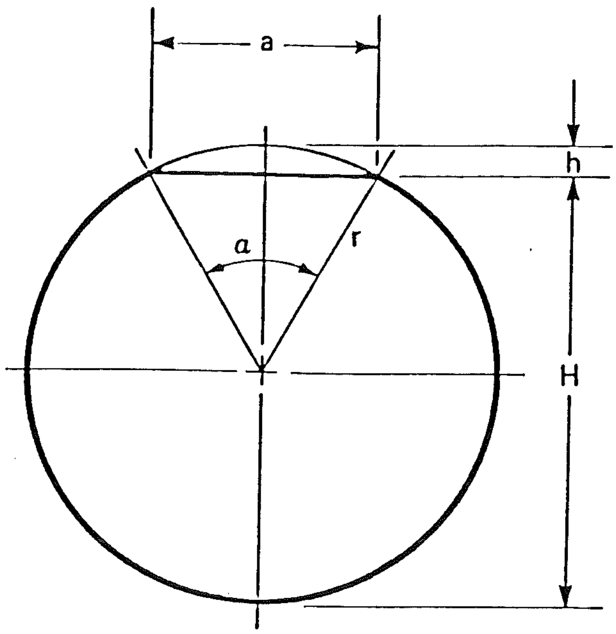
2.1.4 Protective Coating

While under many conditions no further protection is required, in certain situations it may be necessary to consider the likelihood of vandalism, or the possibility of damage in a particularly harsh environment.

For these situations, an epoxy coating is available, which may be spread on after the tube is installed and provides a durable yet flexible protection. This coating may be sprinkled with the surrounding material to visually blend in with the surroundings. It is completely waterproof and may be applied to wet surfaces.

2.2 EARTH PRESSURES

Calculations of earth pressure for the design of Longard tubes as retaining walls or bulkheads can use the approximations for active earth pressure for cohesionless soils. This theory assumes that the tube will displace away from the soil mass sufficiently for active earth pressures to develop, and is thoroughly presented in most soil engineering textbooks (i.e., Sowers and Sowers, 1970).



TUBE FULLNESS	NOMINAL TUBE DIAMETER 100cm	180cm
98% OF THEORETICAL MAXIMUM CAPACITY		
a	44.6cm	80.3cm
H	94.7cm	170.5cm
SECTIONAL AREA	0.770m^2	2.49m^2
95% OF THEORETICAL MAXIMUM CAPACITY		
a	59.1cm	106.4cm
H	90.3cm	162.6cm
SECTIONAL AREA	0.746m^2	2.42m^2

FIGURE 13: GEOMETRIC PROPERTIES OF FILLED LONGARD TUBES

In general, tubes used singly with level backfills are massive enough to resist sliding for all normal conditions, and stability calculations will not be necessary. For conditions involving stacked tube construction, steeply sloping backfills, or poor drainage provisions and the possibility of the backfill becoming fully saturated, earth pressure calculations should be performed and the stability of the tube assessed.

The active pressure, p_A at a distance, h below the ground surface is:

$$p_A = \gamma h \tan^2 \left(45 - \phi/2\right)$$

where,

γ = unit weight of the soil,

ϕ = angle of internal friction of the soil.

For this triangular pressure distribution, shown in Figure 14, the total horizontal force acting on the tube is:

$$P_A = \frac{\gamma H^2}{2} \tan^2 \left(45 - \phi/2\right)$$

where H = height from the bottom of the tube to the top of the backfill.

For high water tables or fully saturated backfills, the effective stress in the soil matrix is used for calculating the active earth pressure and the hydrostatic water pressure is added to obtain the net horizontal force.

A definition sketch for calculating active earth pressures involving sloping backfills-is presented in Figure 15. The approximations are:

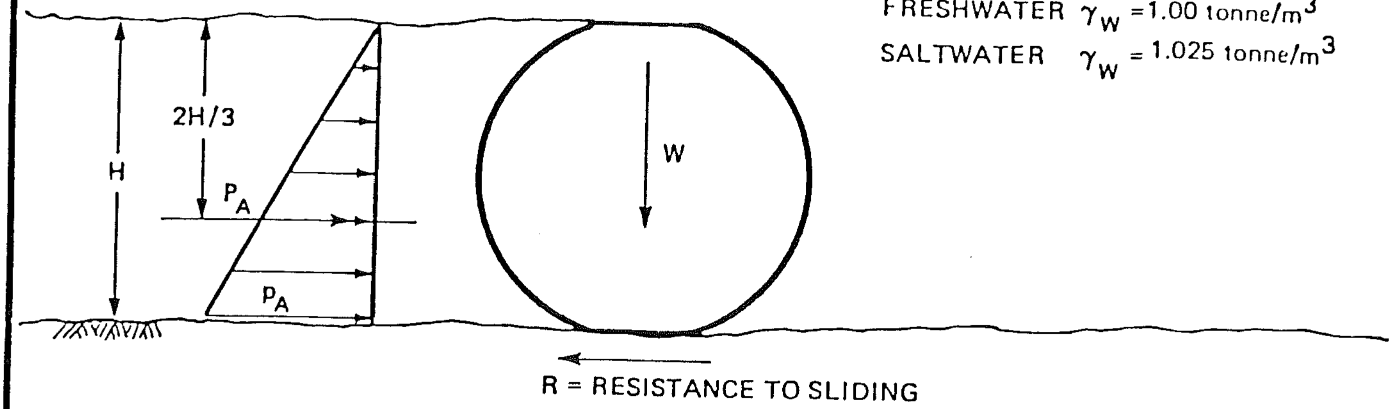
$$p_A = \gamma h \cos\beta \quad \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta \quad \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$P_A = \frac{\gamma H^2}{2} \cos\beta \quad \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

where,

β = the angle between the backfill surface and the horizontal.

The direction of this force is parallel to the backfill slope with a line of action passing through a point 1/3H above the bottom of the tube. Figures 14, 15, and 16 present example problems and a design aid for computing earth pressures.



CASE I. WATER TABLE IS AT THE BOTTOM OF THE TUBE

$$p_A = \gamma_S H \tan^2 (45-\phi/2)$$

$$P_A = \tfrac{1}{2}\gamma_S H^2 \tan^2 (45-\phi/2)$$

CASE II. WATER TABLE LOCATED AT TOP OF BACKFILL

$$'p_A = (\gamma_S-\gamma_W)\, H \tan^2 (45-\phi/2)$$

$$'P_A = \tfrac{1}{2}(\gamma_S-\gamma_W)\, H^2 \tan^2 (45-\phi/2)$$

$$P_W = \tfrac{1}{2}\gamma_W H^2$$

$$P_{TOTAL} = P'_A + P_W$$

RESISTANCE TO SLIDING CAN BE CALCULATED AS:

$$R = \mu W$$

WHERE W = WEIGHT OF THE FILLED TUBE AND
μ = STATIC COEFFICIENT OF FRICTION AT THE SLIP PLANE.

FIGURE 14: ACTIVE EARTH PRESSURE WITH HORIZONTAL BACKFILL

EXAMPLE: DETERMINE THE STABILITY OF A 180cm ϕ TUBE WITH A LEVEL BACKFILL SUBJECTED TO WAVE OVERTOPPING. GIVEN: $\phi = 30^o$, UNIT WEIGHT OF BACKFILL = 1.92 tonne/m^3, AND UNIT WEIGHT OF LONGARD TUBE = 2.08 tonne/m^3.

SOLUTION: FOR A 180cm ϕ TUBE FILLED TO 98% CAPACITY, SECTIONAL AREA = 2.49m^2, H = 170,5cm WEIGHT OF TUBE = (2.49m^2) (2.08 tonne/m^3 = 5.18 tonne/m^3.

SLIDING RESISTANCE

$$R = \mu W$$
$$= \tan 30^o \; (5.18 \text{ tonne/m} \;)$$
$$= 2.99 \text{ tonne/m}$$

ACTIVE EARTH PRESSURE

$$P'_A = ½ (\gamma_S - \gamma_W) H^2 \tan^2 (45 - \phi/2)$$
$$= ½ (1.92 - 1.025) (1.705)^2 \tan^2 (45 - 30/2)$$
$$= 0,434$$

$$P_W = ½ \gamma_W H^2$$
$$= ½ \; 1.025 \; (1.705)^2$$
$$= 1.490 \text{ tonne/m}$$

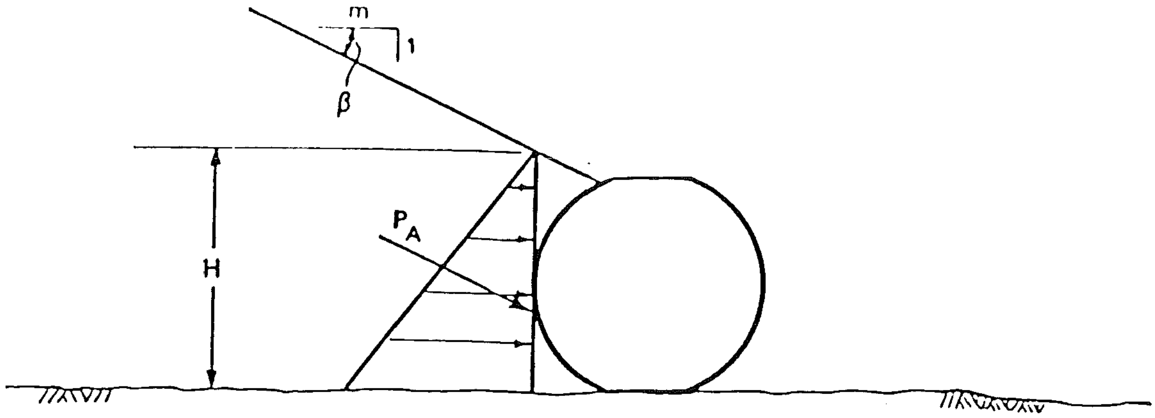
$$P_{A_{TOTAL}} = P'_A + P_W$$
$$= 0.434 + 1.490$$
$$= 1.924 \text{ tonne/m}$$

FACTOR OF SAFETY FOR SLIDING

$$F.S. = R/P_{A_{TOTAL}} = 2.99/\,1.924 = 1.55$$

∴ TUBE IS STABLE

FIGURE 14 (CONTINUED): ACTIVE EARTH PRESSURE WITH HORIZONTAL BACKFILL



TO CALCULATE ACTIVE EARTH PRESSURE:

$$P_A = \tfrac{1}{2}\gamma H^2 \cos\beta\ (\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi})\ /\ (\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi})$$

TOTAL DISPLACEMENT FORCE ON TUBE CONSISTS OF EARTH PRESSURE AND WATER PRESSURE DUE TO DIFFERENTIAL ELEVATIONS OF STATIC HEAD ON THE TWO SIDES OF THE TUBE.

EXAMPLE: DETERMINE THE STABILITY OF A 180cm ϕ TUBE BACKFILLED WITH SAND HAVING A SLOPE OF 3.0 HORIZONTAL ON 1.0 VERTICAL CONSIDER A WELL DRAINED BACKFILLED WITH:

$\phi = 30^o$ AND UNIT WEIGHT = 1.92 tonne/m^3

FROM EXAMPLE 1, THE TUBE'S STATIC RESISTANCE TO SLIDING = 2.99 tonne/m

ACTIVE EARTH PRESSURE:

$$P_A = \tfrac{1}{2}\gamma H^2 \cos\beta\ (\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi})\ /\ (\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi})$$

FROM FIGURE 17 FOR $m = 3.0$ AND $\phi = 30^o$

$$\cos\beta\ (\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi})\ /\ (\cos\beta + \sqrt{\cos\beta - \cos^2\phi}) = 0.40$$

H ≅ TUBE HEIGHT = 1.705 m

$P_A = \tfrac{1}{2}\ (1.92)\ (1.7059)^2(0.40) = 1.12$ tonne/m

HORIZONTAL COMPONENTS OF P_A

$P_{A_H} = P_A \cos\beta$

$= 1.12 \cos 18.4^o$

$= 1.06$ tonne/m

FACTOR OF SAFETY FOR SLIDING

F.S. = $R / P_{A_H} = 2.99 / 1.06 = 2.82$

∴ TUBE IS STABLE

FIGURE 15: ACTIVE EARTH PRESSURE WITH A SLOPING BACKFILL

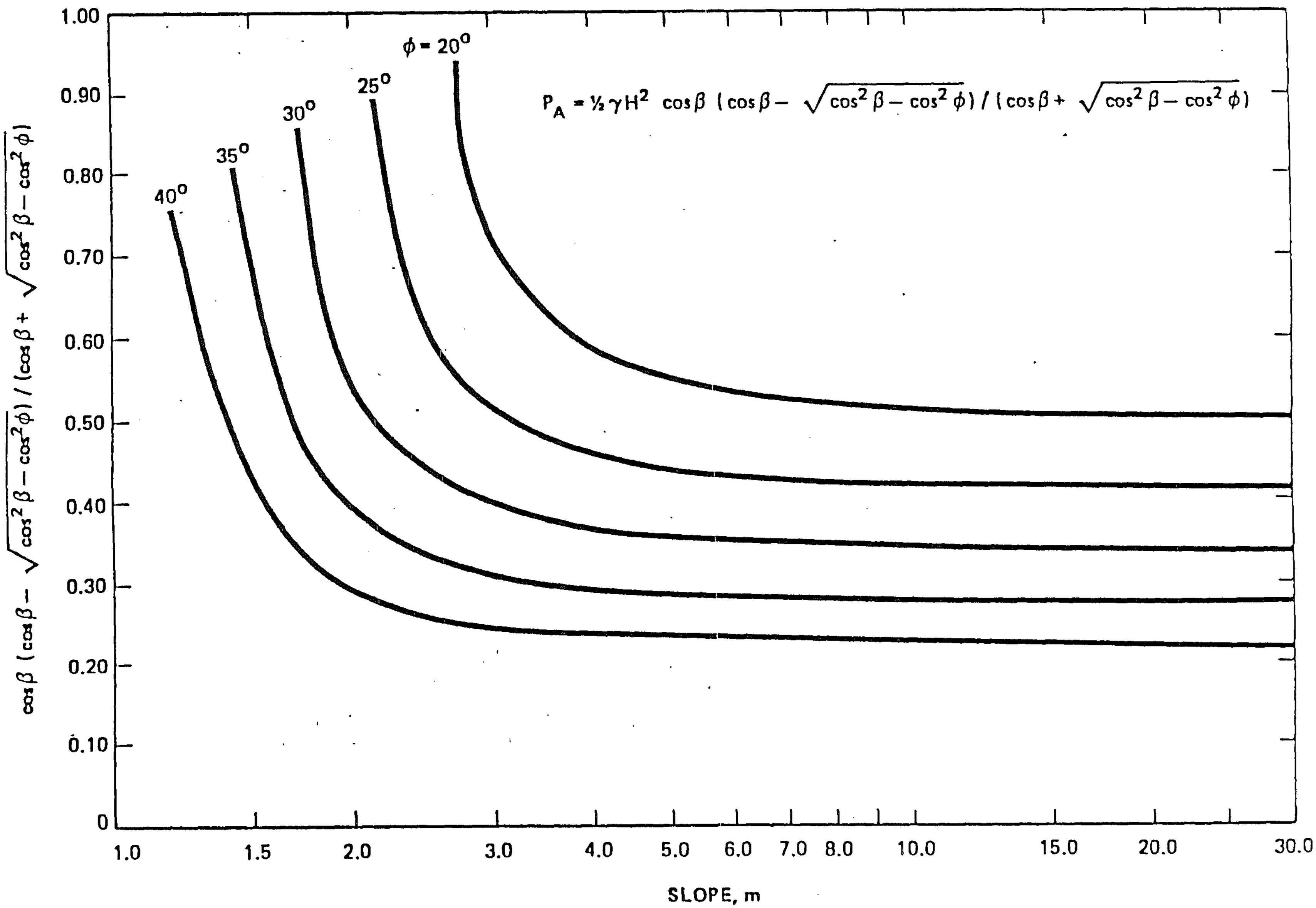


FIGURE 16: APPROXIMATE SOLUTION TO ACTIVE EARTH PRESSURE WITH A SLOPING BACKFILL

2.3 WAVE FORCES

The approximation of wave forces on Longard tube structures is useful to assess the stability of a tube subject to wave attack. Four general wave conditions of wave attack on a shore parallel tube are:

I. Wave forces on a fully submerged tube,

II. Non-breaking wave forces on a non-submerged or marginally submerged tube,

III. Breaking wave forces on a non-submerged or marginally submerged tube, and

IV. Broken wave forces on a non-submerged tube.

Only approximate methods are available to estimate these forces, hence, for structures requiring precise estimates of wave forces, hydraulic model studies will be required.

Conditions I or III will normally constitute the design condition, with the latter generally being capable of producing the greatest force. Methods for approximating wave forces under these conditions, with the wave's direction perpendicular to the tube, are given in the following sections. Selection of a factor of safety to apply to these forces should weigh the seriousness of failure of the tube structure and should be made with a qualified engineer. Non-breaking wave forces on a non-submerged tube and broken waves forces (cases II and IV) can be predicted using methods presented in publications such as the Shore Protection Manual (1977).

2.3.1 Wave Forces on a Fully Submerged Tube

Prediction of wave forces on a fully submerged tube can use models and data developed for wave forces on submarine pipelines. For non-breaking waves and a water depth much greater than the tube diameter, a method proposed by Grace (1978) is suggested. The necessary parameters include water particle kinematics and the physical properties of the water and of the tube. Referring to Figure 17, the following equations can be used to estimate peak forces:

$$F_{MAX} = \begin{cases} C_I^* \ \rho \left(\frac{\pi D^2}{4}\ell\right) \dot{U}_{MAX} & , \ \psi \leq 1 \\ C'_{MAX} \ \frac{\rho}{2} \left(D\ell\right) U^2_{MAX} & , \ \psi > 1 \end{cases}$$

$$P_{MAX} = \begin{cases} \simeq \ F_{MAX} & , \ \psi \leq 1 \\ K'_{MAX} \ \frac{\rho}{2} \ D\ell \ U^2_{MAX} & , \ \psi > 1 \end{cases}$$

where,

F_{MAX} = maximum horizontal force

P_{MAX} = maximum vertical force

ρ = density of water

D = tube diameter

ℓ = tube length

U_{MAX} = maximum water particle velocity

$\dot{U}_{MAX}$ = maximum water particle acceleration

C_I^* = theoretical potential flow inertia coefficient (C_I = 3.3 for a tube resting on the seabed perpendicular to the oncoming flow).

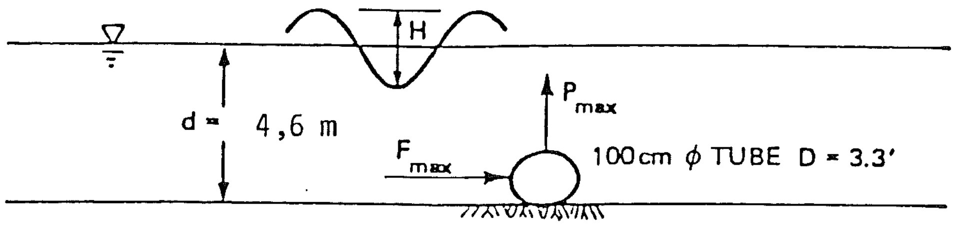
C'_{MAX} = maximum force coefficient, obtained from Figure 18.

Peak horizontal and vertical forces can be assumed to occur simultaneously for design purposes. Estimation of water partical kinematics, i.e., U_{max} and $\dot{U}_{max}$, can be made with the appropriate wave theory. Figure 19 allows an approximation of near bottom kinematics with linear wave theory by:

$$U_{MAX} = \frac{\pi H}{T} \left(\frac{1}{\sin\left(\frac{2\pi d}{L}\right)}\right)$$

$$\dot{U}_{MAX} = \frac{2\pi}{T} \ U_{MAX}$$

EXAMPLE: ESTIMATE THE WAVE FORCE ON A FULLY SUBMERGED 100cm ϕ TUBE WHERE d = 4,6 m H = 1,8 m AND T = 8 sec.



$L_0 = ½(g/\pi)\,T^2 = ½(9{,}82/\pi)\,(8^2) = 100$ m

$d/L_0 = 4.6/100 = 0{,}046$

FROM FIGURE 19 $1/\sinh(2\pi d/L) = 1.69$

$U_{max} = (\pi H/T)\,(1/\sinh(2\pi d/L)) = (\pi\,1.8/8)\,(1.69) = 1{,}19$ m/s

$\dot{U}_{max} = (2\pi/T)\,U_{max} = (2\pi/8)\,(1{,}19) = 0{,}93$ m/s^2

$\psi = U^2_{max}/\dot{U}_{max} D = 1{,}19^2/(0{,}93)\,(1) = 1{,}52$

FROM FIGURE 18 $C'_{max} = 3.78$

$K'_{max} = 4.20$

$F_{max} = C'_{max}\,½P\,(DI)\,U^2_{max}$

$= (3.78)\,½(71\,\text{slugs/m}^3)\,(1 \times I)\,U^2_{max}$

$= 190$ kg f/m

$P_{max} = K'_{max}\,½P\,(DI)\,U^2_{max}$

$= (4.20)\,½\,(71$ slugs/m^3 $(1 \times I)\,U^2$ max

$= 211$ kg f/m

TOTAL RESULTANT MAXIMUM FORCE $= \sqrt{190^2 + 211^2}$

$= 284$ kg f/m

FIGURE 17: WAVE FORCES ON A FULLY SUBMERGED TUBE

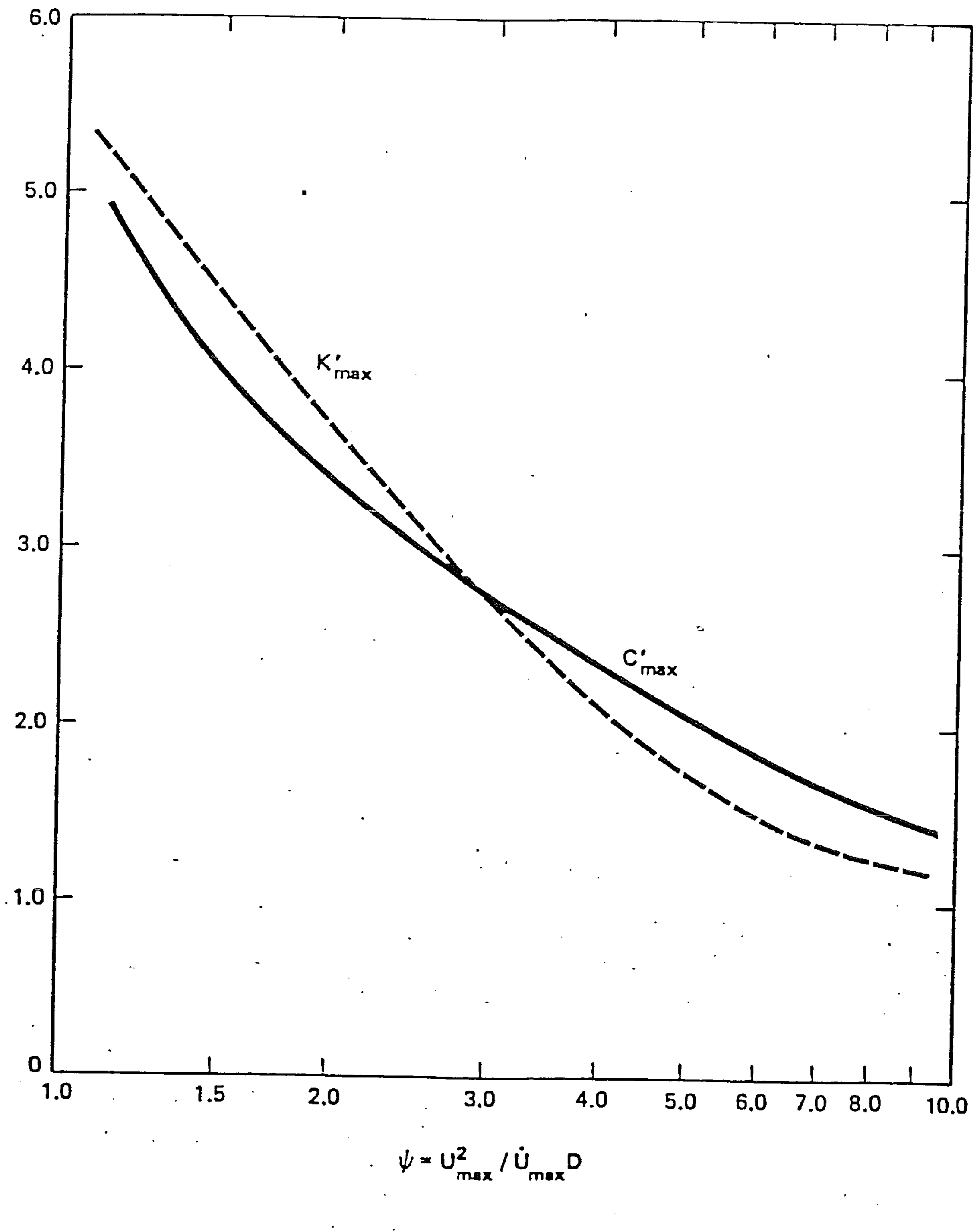


FIGURE 18: MAXIMUM FORCE COEFFICIENTS FOR WAVES ON A FULLY SUBMERGED TUBE

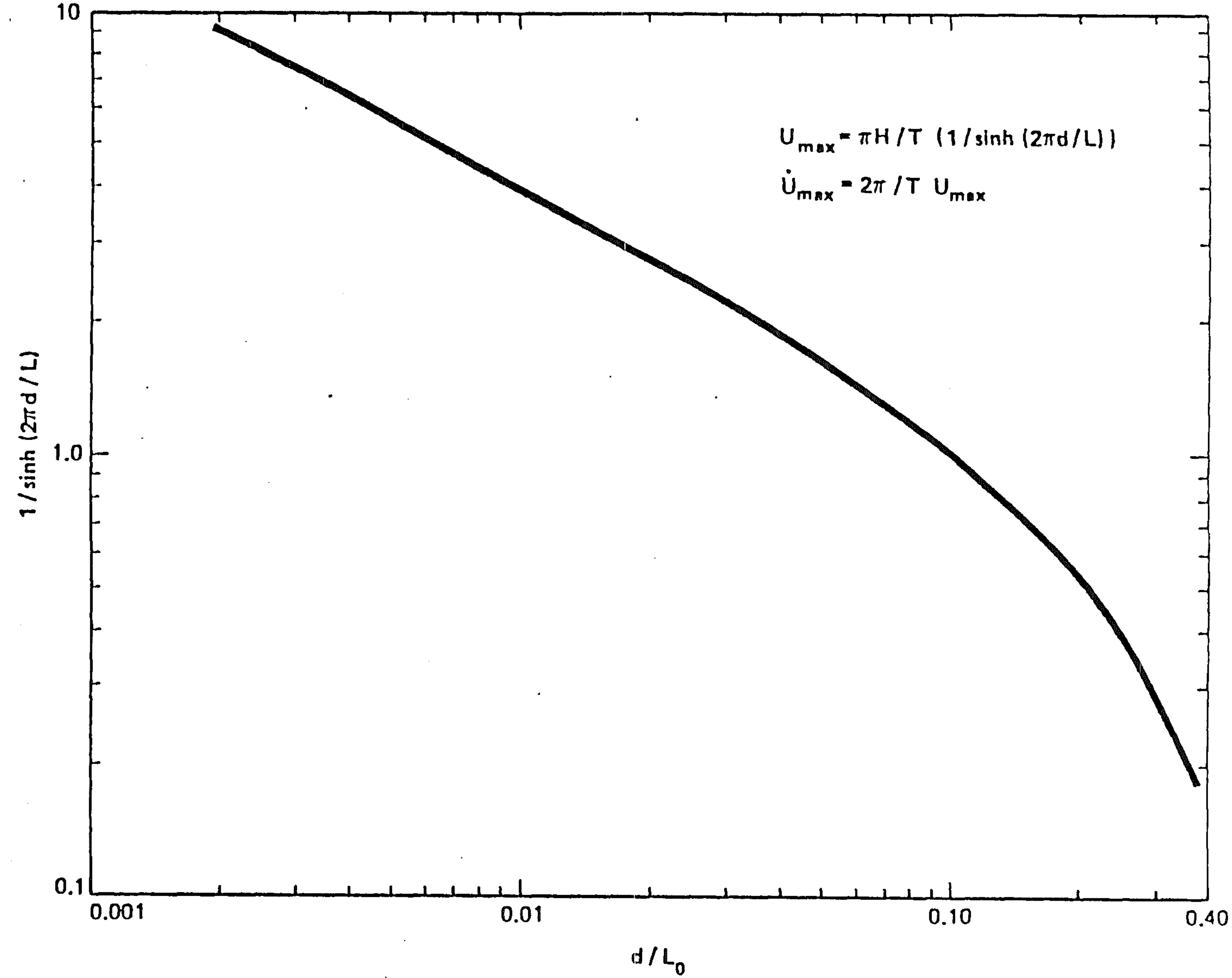


FIGURE 19: LINEAR APPROXIMATIONS FOR NEAR BOTTOM WATER PARTICLE KINEMATICS UNDER NON-BREAKING WAVES

where, H = wave height
T = wave period
d = water depth, and
L = wave length.

An example calculation for using this method is given in Figure 17.

2.3.2 Breaking Wave Forces on a Non-submerged Tube

Longard tubes placed parallel to shore in the active surf zone will normally be subjected to breaking waves during design conditions. These forces are of short duration and are generally not applicable for assessing the static equilibrium of a single tube system. However, its importance for non-typical tube placements, i.e., pyramidal shaped breakwaters, must be recognized and considered in design. The Minikin method for estimating breaking wave forces can be used to approximate the magnitude of the impact loading due to breaking waves.

In this method, the maximum pressure is assumed to act at the still water level and is given by

$$p_m = 101w \frac{H_b}{L_D} \frac{d_s}{D} (D+d_s)$$ where,

w = unit weight of the water
H_b = breaking wave height
D = depth one wave length in front of the tube
d_s = depth at the toe of the tube
L_D = wave length in water of depth D

The pressure distribution is assumed to decrease parabolically from at the still water line to zero at a distance of $H_b/2$ above and below the still water line, as shown in Figure 20. The horizontal force due to this pressure distribution is equal to:

$$F_H = \frac{p_m H_b}{3}$$

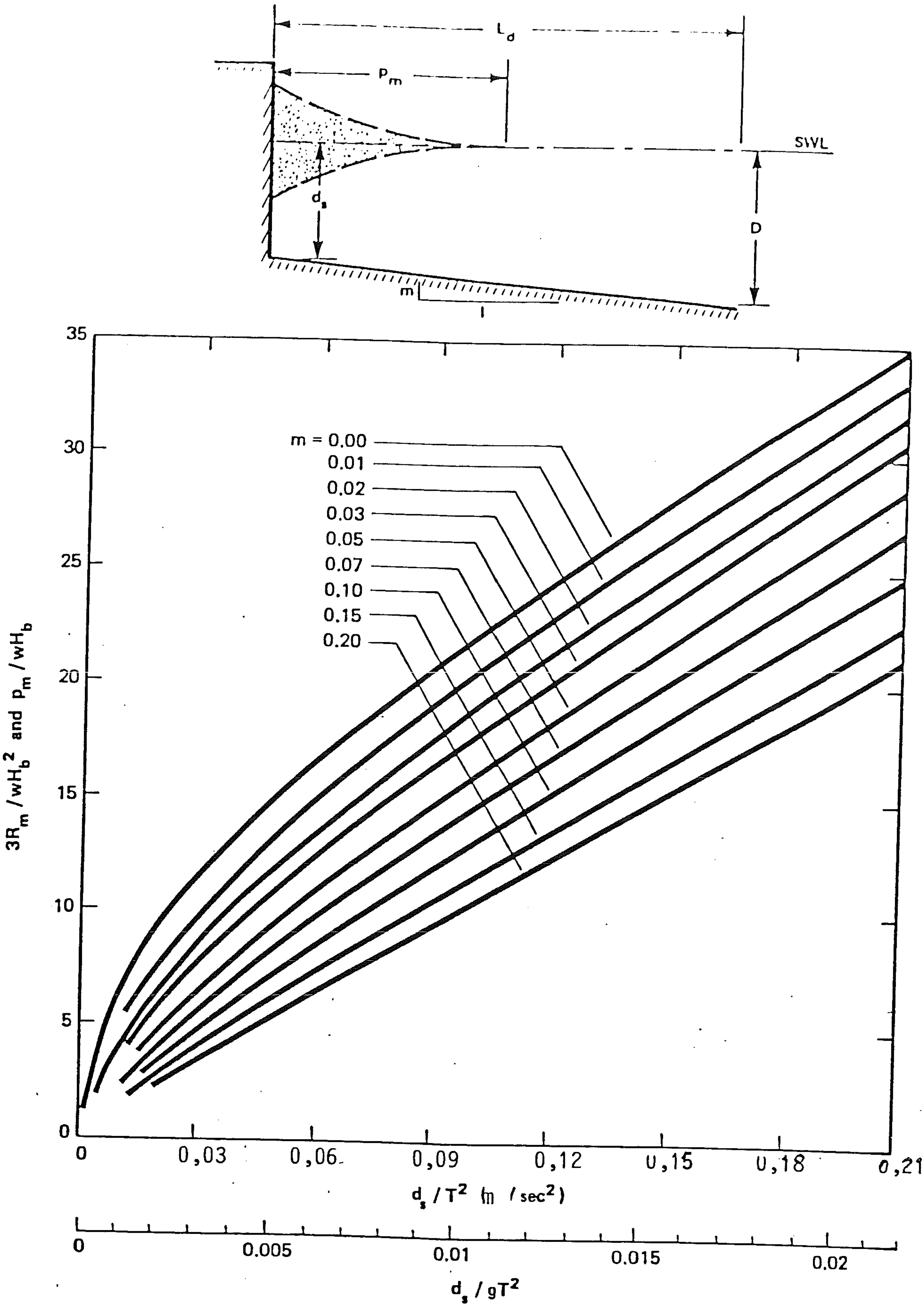
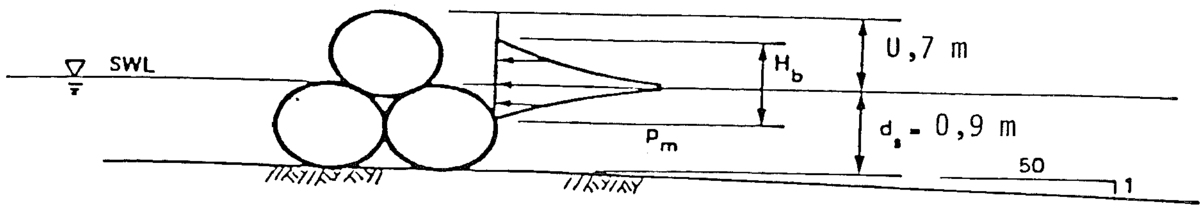


FIGURE 20: DIMENSIONLESS MINIKIN WAVE PRESSURE AND FORCE (AFTER SPM, 1977)

If the top elevation of the tube is located below an elevation of $H_b/2$ above the still water line, i.e., the tube is overtopped by the breaking wave, a pressure distribution truncated at the top elevation of the tube can be assumed. Figure 20 is an aid for calculating the peak dynamic pressure and forces, and Figure 21 presents an example problem.

EXAMPLE: ESTIMATE THE BREAKING WAVE FORCE ON A 100cm ϕ TUBE USED IN A PYRAMIDAL BREAKWATER CONSTRUCTION SHOWN IN THE DIAGRAM BELOW $T = 4sec$, $H_b = 0,9$ m



$d_s/T^2 = 0,9/4^2 = 0,05625\ m/s^2$

FROM FIGURE 20 $p_m/wH_b = 13$

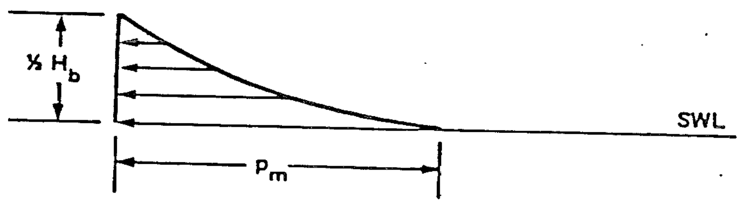
$p_m = 13\ (1,025)(0,9) = 12,0\ t/m^2$

TOTAL BREAKING WAVE FORCE ON BREAKWATER

$F_H = p_m H_b/3$

$F_H = (12)(0,9)/3 = 3,6\ t/m$

FORCE ON TOP TUBE IS EQUAL TO THE AREA UNDER THE PRESSURE DIAGRAM BETWEEN SWL AND ½ H_b



$F_{TOP} = 1/3\ (p_m)\ (H_b/2)$

$= 1/3\ (12)\ (0,9)/2)$

$=\ 1,8\ t/m$

FIGURE 21: EXAMPLE PROBLEM FOR BREAKING WAVE FORCES ON LONGARD TUBES

3.0 REFERENCES

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