

DESIGN OF COASTAL STRUCTURES FOR RECREATIONAL PURPOSES
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1. INTRODUCTION

1 1 Although the major aspect of this paper deals with a specific part of the South African coast line it is believed that certain fundamental parameters which emerge are applicable to the greater part of the Southern African coast line of 2954 km.

Typical of the African continent the nature of the Southern African coast line is regular and even with few bays or inlets and has long unprotected beaches subject to long- and off-shore current patterns.

The absence of barrier islands as well as the few in number of protected lagoons and inlets, requires the creation of artificial tidal enclaves or the protection of selected sections of beach to ensure safe bathing conditions where further extensions of existing areas is indicated both by an increase in the population as well as changing socio-economic conditions which place access to the coastal recreation areas, as an increasing facility, to a steadily growing number of people.

This need has been intensified in South Africa by the rapid urbanisation and socio-economic development of the rural population as well as the geographic dispersion of the population of which a high percentage lives at or near the coast.

It is unfortunately so that the need to provide additional access to the beach areas exists in such areas where maximum exploitation of the beach frontage has already taken place and where new development must take place accordingly in less favourable conditions and subject to various constraints imposed by existing development in these areas.

These constraints are inter alia the development of industrial areas right onto the beach front, urban area development on the beach front and sewage outfall works discharging raw sewage into the surf zone.

The absence of an adequate inland area contiguous to the beach, forces planners to absorb an increasing part of the primary dune area to accommodate parking areas and other facilities required for the accommodation of the mass migration to the beaches.

These very constraints imposed upon the planner as well as the capital limits to development, in itself creates a further concentration of people in the most vulnerable part of the beach environment.

- 1.2 The paper now laid before this congress seeks to evaluate various considerations and factors which lead to the adoption of certain design parameters which may satisfy the criteria of safety, user preference, health standards, beach stability and sympathetic environmental treatment.

The creation of a pleasing and sensible development of beach recreation areas may in turn be seen to create a need for a service infrastructure including access under peak conditions not dissimilar to those associated with a new and popular sports stadium.

The submissions made provide certain suggested standards for the service infrastructure for the accommodation of peak conditions. Notwithstanding that the objective of the submissions is to achieve a common philosophy in planning which may be applied generally, the analyses undertaken are relevant to particular sections of the coast line and are user orientated to that section of the population most likely to make use of the facilities thus created.

However, the successes attained as well as the errors incurred may hopefully stimulate sensible and responsible planning for the future.

- 1.3 Whereas the design of extensive coastal structures involves the complete spectrum of environmental, sociological, transport, future urbanisation and economic research, it is patently not within the terms of reference of this paper to deal with these other than in general terms when such have a direct bearing on the structure design and general arrangement being promoted.

The author expresses the opinion however that controlled and planned exploitation of selected portions of the coast to accommodate the needs of a growing and economically developing population is of greater overall environmental benefit than is a lack of planned development and a consequent concentration of human activity in the ecologically sensitive areas as an unlimited dispersion of temporary sojourners without a sense of permanent involvement.

2. USER CONSIDERATIONS AND PREFERENCES

- 2.1 Other than in the case of natural parkland areas where the emphasis is in the maintenance of the environment

and ecological system, the creation of artificial bathing areas is both to accommodate man and also to control his further exploitation of the coastal areas for recreation purposes.

In this regard no sensible planning can be undertaken without an indepth study of the user characteristics and preferences. User preferences vary from the east coast to the west coast over the length of the coast line of 2954 km. where the warm Mozambique-Agulhas current, which skirts the east and south coasts as far as Cape Point, is replaced by the Cold Benguela System flowing north along the west coast, with climatic conditions which vary from dominantly subtropical conditions on the North and East coast to generally cool temperate conditions of the South coast with the uninvitingly cold waters of the West.

These climatic as well as water temperature conditions determine the relative degree of emphasis on bathing or dry beach activity.

- 2.2 User preferences are also related to the heterogeneous character of the South African population and demographic considerations pertinent to the community involved. These in turn are affected by the degree of cultural and social heterogeneity prevalent as well as economic standards of different sections of the community.

Notwithstanding this complexity of influencing factors, man tends to adjust himself in his recreation to the circumstances and environment existing, provided such satisfies certain norms and in this regard the following is quoted from the report titled "Coastal Development Project - False Bay Coast - Cape Town". (Reference No 1).

"The relevant quality of bathing and beach enjoyment conditions, whether natural or man created, is of considerable but not sole importance, and are evaluated by the individual in terms of a number of other facets of his recreational pursuits and his decision to utilize a particular recreation area is the result of his assessment of all relevant factors in their inter-related importance, of which the accessibility of the beach area, provided that all other aspects are reasonably satisfied, may be of highest priority to the individual.

Under accessibility we consider that the actual point at which the motorcar is parked or where the bus delivers passengers, relative to the point where the individual may start 'doing his thing' is, in relative importance of equal value to the route and quality of access road from home to recreation destination.

It has been established that particularly in the case of the more affluent section of the total society, people will travel considerably further to reach a beach where they may step out of the vehicle onto the sand or where they may park their vehicle in the immediate vicinity of the beach and bathing area with good sight of beach activity and a continuous sense of involvement in the beach environment.

Throughout South Africa the significance of this aspect is revealed by the degree of preference given to those areas where parking is immediately adjacent to the beach, lagoon or bathing pool."

This inter-relationship between accessibility and quality of recreation, places an additional constraint on the selection of sites for development and requires the coastal engineer to be pleasingly creative under the most adverse physical conditions in order to create safe bathing conditions as near as practical to immediate access from motor vehicle parking areas which are served by arterial road systems.

The capital cost of the transport infrastructure synonymous with beach development schemes encourages the urban planner to create out of the area a grand terminus for all conceivable recreation as representing the optimum utilisation of capital resources.

The presence however in the vicinity of the beach area of other forms of entertainment such as park and playground development does not of necessity increase the popularity of the beach area but attracts generally a differently motivated recreation seeker whose presence in the area places a fresh complexity on the service infrastructure required and due to various factors adversely affects the comfort, safety and security of parents with young children.

The population age structure in respect of communities in the vicinity of which new beach recreation areas are being planned and which communities accordingly will be the dominant users of the facilities provided is particularly significant as will appear from the following figures applicable to the False Bay area.

Population Age Structure	
0 - 4 years	16,3 percent
0 - 9 years	32,1 percent
0 - 14 years	45,0 percent
0 - 19 years	55,5 percent
0 - 24 years	65,2 percent

For planning purposes, while giving the family factor due weight, the visitor population age structure is

likely to be :

3 - 9 years	45 percent
10 - 15 years	16 percent
16 - 24 years	14 percent
25 years and over	25 percent
Total :	100 percent

This reveals an important emphasis on the lower age group.

3. DEVELOPMENT OF DESIGN CRITERIA

3.1 All design, irrespective as to its nature, requires an evaluation of system loading and in the case of a tidal pool or bathing enclave with its related supporting infrastructure this is equally true, in as much as it is necessary to make a prediction of the visitor normal peak and the manner in which this may be accommodated in its occurrence within the bathing area, on the beach, within the change rooms and ablution blocks and in terms of the transportation system.

There is no universal formula which can be applied and each particular set of circumstances requires its own evaluation based on, among others, the following factors and considerations :

- 1) The overall completeness and attractiveness of the beach recreation scheme and its immediate environs.
- 2) The relationship of parking areas and other transportation termini to the point where the individual may commence 'doing his thing'.
- 3) The capacity and quality of arterial road systems, their congestion factors and the distance to be travelled.
- 4) The socio-economic standards of the respective population.
- 5) The demographic structure of the relevant population.
- 6) The spatial relationship of urban residential areas and population densities.

3.2 In the studies and planning undertaken in the False Bay area of Cape Town the careful weighting and evaluation of the factors referred to, led to the preparation of a visitor projection graph for normal peak days of the form

$$p = k_1 - k_2 S^{\frac{1}{2}}$$

where p - resultant percentage of resident population who would visit the beach on a warm, sunny, low wind, vacation day.

k_1 - maximum effective percentage for people resident within walking distance of beach.

k_2 - spatial relationship - population dispersion factor.

S - best route travelling distance in kilometres.

The importance of the formula lies not solely in an assessment of visitor peak influx but also in the rational assignment of traffic flows to various routes and transportation modes.

The values adopted for the False Bay report are :

$$K_1 = 40 ; \quad K_2 = 5$$

- 3.3 Notwithstanding the peak tendency, the facilities provided will have their own optimum capacity beyond which the user congestion factor acts to limit the abnormal peak and deflect the balance of visitors to other or adjoining areas.

For the planner however it is important that capacities throughout the system be consistent and compatible one with the other. In other words the overloading of the bathing enclave would be accompanied by an overload on the dry beach area, parking areas, toilet and ablution blocks, etc.

This requires the determination of a comfort level optimum capacity bearing in mind that man is gregarious in nature and beach crowding is part of the scene he loves.

Up to a water depth of 800 mm a loading of 20 people per square metres is acceptable; from 800 mm to 1500 mm the loading should not exceed 12 people per 100 square metres and over 1500 mm in depth the loading should be less than 8 persons per 100 square metres.

This would place the optimum capacity of the Strandfontein tidal pool at 5000 people whereas a peak of over 15000 has been experienced.

The probable ratio at peak hour of people in the water to people on the dry beach is 1 to 2.

- 3.4 A structural and geometric design philosophy for the attainment of favourable bathing conditions is functionally related to the measure of assurance of the

ordinary bather, when committing himself to the relatively strange environment, in his ability to predict changes in underwater topography as well as the dynamic forces to which he may be subjected.

In practical terms this design philosophy can be stated as ensuring that :

- 1) No localised changes of any serious consequence occur in the beach or tidal enclave topography in the design area.
- 2) The occurrence of irregular and/or strong currents be eliminated.
- 3) That a beach gradient of good regularity and not exceeding 1 in 50 in the surf bathing zone between High Water Mark and Mean Sea Level - 3,0 metres should be aimed at.
- 4) That in the case of a tidal pool the floor gradient should not exceed 1 in 30.
- 5) That a sand area is to be preferred to a rock and sand area in the bathing zone.

3.5 Service To be Provided

The minimum number of plumbing fittings required for male and female changerooms should be in accordance with the applicable Building Regulations for places of public entertainment based on sixty percent of the assessed recreation area optimum capacity divided evenly between both sexes.

Changerooms should provide accommodation for peak usage at three percent of the optimum capacity at 2,5 square metres per person.

External showers should be provided on the ratio of ten percent of changeroom capita accommodation. An equal number of faucets is required within each changeroom block.

4. THE FALSE BAY RESEARCH AND CONSTRUCTION PROGRAMME

- 4.1 A report evaluating the need for development in the False Bay area and recommending further research was presented in 1979 and resulted in the construction of a tidal pool at Strandfontein Point as well as an extended research programme in respect of protective works at Middlebank and Kapteinsklip.
- 4.2 The Strandfontein Tidal Pool Model Tests were carried

out at an early stage of the False Bay Model Studies and were directed at :

- 1) an examination of wave action under various wave conditions with a view to predicting the inflow of freshwater to the pool area for the geometric configuration and full supply levels recommended by the Consultants
- 2) the determination of the stability of the toe armouring proposed
- 3) the determination of the overturning forces on the seawall due to wave action
- 4) a correlation between inflow and sediment transport into the pool.

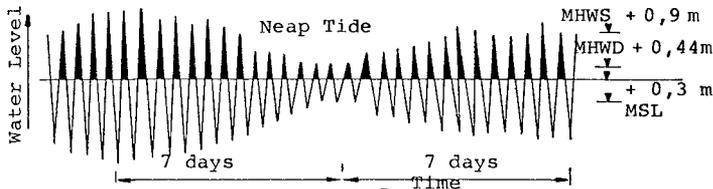
Various tests for different wave conditions and tidal levels were undertaken by the C.S.I.R. the results of which in practice after completion of construction were found to be of a high order of accuracy of prediction.

The following limiting conditions of inflow from the model studies were ascertained for proto type crest level of seawall of R.L. 1,50 m M.S.L.

- 1) That inflow took place only during high tide conditions and when sea levels rose above R.L. 0,30 m M.S.L.
- 2) That no inflow under any conditions took place for wave heights (H_g) 0,40 m and less - which condition pertains for 16 percent of the time.
- 3) That during 50 percent of the high tides an inflow of more than 40 000 cubic metres could be expected.

Spring Tide

Spring Tide



These results together with the structural stability results for armouring size and overturning moments were employed in the final design.

A good indication of sediment inflow probability was available from the correlation calculations and in all the use of the model was found to be completely

successful and fully justified and served to confirm the consultants recommendations. It is not possible in this paper to deal with the C.S.I.R. report in depth but to provide the following salient features.

Scales applicable to the model testing in accordance with the Froude laws were :

Geometrical	1 : 15
Velocity	1 : 15 ^{1/2} (1 in 3,87)
Time	1 : 15 ^{3/2} (1 in 3,87)
Force	1 : 15 ³ (1 in 3375)

In view at that time of the early stage of the False Bay project research as well as the urgency of the tidal pool aspect of the research being undertaken, model calibration was based on wave directions determined from aerial photographs then available and field observations from survey rods located in accordance with the prototype geometrical configuration at 20 metre intervals.

- 4.3 The objective of the experimental measurements of inflow due to wave action was to optimize the height of the seawall so as to achieve a satisfactory fresh water replenishment rate without the occurrence of unduly rough conditions within the pool.

A full supply level of M.S.L. 1,50m based upon previous experience was adopted for experimental purposes and found to be the optimum level for geometric design.

The lowering of the crest level by 200 mm revealed in the experimental analysis virtually no change to the conditions of no-inflow but did affect quite appreciably the total inflow and bathing conditions.

A final crest level for construction purposes of 1,40 m M.S.L. was adopted.

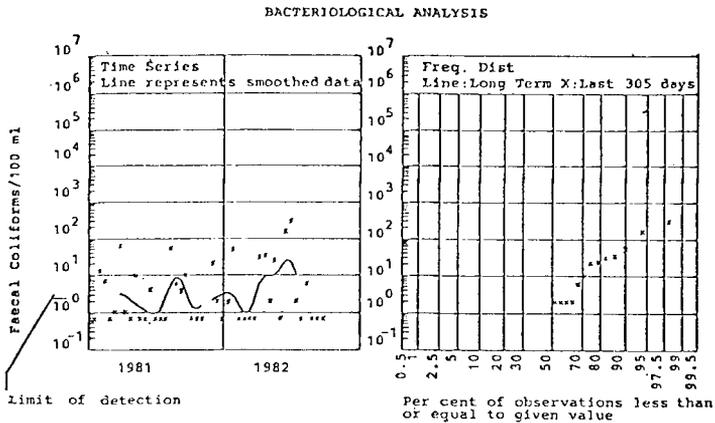
The experimental mean inflow per tide was found to be 40 000 cubic metres with a maximum at sea level 0,90 m (M.S.L.) at M.H.W.S.T. of 160 000 cubic metres.

In general it was established that the best sustained inflow conditions occur during the summer months and in particular December and January when incidentally the most severe visitor loading of the tidal pool takes place.

During December 1978 the sustained period of S.W.L. below M.H.W. was two days; for the same month in 1979 on no occasion; one day in January 1978 and two days in January 1979 as ascertained from the S.A. Tide Tables for 1978 and 1979.

The occurrence during the remainder of the summer season of suitable water levels for wave actuated inflow needs however to be equated within the limiting parameter of minimum significant wave height of 0,40 metres in respect of which value waves lower in height occur for 16 percent of the time. The median inflow per tide for the adopted crest level of 1,40 m M.S.L. has been assessed as being 50 000 cubic metres.

Monitoring of water quality by examination of the faecal coli count under peak conditions has been undertaken by the City Engineer and found under all conditions to comply with acceptable standards for public swimming baths thus justifying the crest level adopted and the model analyses of wave inflow.



4.4 Siting of the pool was subject to geographic, environmental and foundation constraints. The extension of the residential suburbs in an easterly direction placed emphasis on coastal recreation development within easy reach of these areas while at the same time creating a new problem of environmental control consequent to the selective exploitation for recreation purposes for several hundreds of thousand of people of a part of the coast line as yet untouched in the False Bay area.

Foundation problems as well as the type of structure limited the choice of site in as much as the least exposure of the structure to the open sea was sought in view of the economic inaccessibility of bedrock which lies upto ten metres beneath the unconsolidated sediments.

From a regional planning point of view the City Engineer of Cape Town favoured the development of three nodal points in the study area and accordingly studies were directed at the establishment of a tidal pool complex at Strandfontein Point and groyne systems at Middle Bank and Kapteinsklip to cater for larger beach crowds. This planning concept was found to be compatible with the hinterland development, forming a sensible whole.

The location as primary objective of a large tidal pool at Strandfontein Point involving the daily movement in season of scores of thousands of visitors into the area made the removal of primary dune and fynbos environment and the provision of landscaped grassed areas and parking allotments an inescapable feature of the project development.

- 4.5 Pursuant to the preliminary report more extensive investigations of foundation conditions were undertaken. Although general uniformity of geological conditions pertain in the study area the occurrence of an extensive, albeit soft, calcarenite shelf at Strandfontein Point offered the most protected section of the coast for a minor structure.

In this respect the author suggests that the geometric limitations of the structure and the need to create a light and pleasing arrangement required a greater need for design sensitivity than might otherwise be the case.

Geological investigations of offshore conditions both in the field as well as from aerial photographs revealed discontinuous platforms of calcarenite in the surf zone. The calcarenite rock took the form of a surface layer overlying an extensive depth of unconsolidated sands.

Sites were selected for exploratory drilling and boreholes were advanced through unconsolidated sands by washboring methods using bentonite to support the sides of the holes.

Representative samples of the unconsolidated materials being penetrated were taken at intervals of 1,5 metres.

The calcarenite beach rock was penetrated using a diamond bit and NX size cores were recovered in this consolidated material.

Standard Penetration Tests were carried out in the unconsolidated materials at intervals of 500 mm i.e. with gaps of 200 mm followed by 300 mm of penetration over which the Terzaghy N-value was recorded.

The following is an abstract from the report prepared by Dr T Partridge on the geological investigation :

"The properties of the various materials, deduced from the four exploratory boreholes, augmented by Standard Penetration Tests in one case, are as follows :

a) Surface littoral sand : ranges in thickness from 1,3 to 6,0 metres. Usually a coarse, medium and fine shelly sand of loose consistency (N-value 4-8), which would correspond with an in situ dry density of 1450-1600 kg/m³. Maximum allowable foundation pressure 50-100 kPa (Reference 1).

b) Calcarenite : ranges in thickness from 0,35 m to about 1,0 m. Usually a moderately weathered, medium textured soft rock material which becomes slightly softer towards the base. Joints are generally infrequent. This material is lenticular in its occurrence and may thicken and lens out sporadically under the surface.

c) Clayey sand : ranges in thickness from about 0,9 m to 3,0 m. Usually a clayey medium and fine sand of firm to stiff consistency (N-value 11-27). Maximum allowable foundation pressure 100 - 200 kPa. (Reference 1) May contain channels, as reflected by water losses during drilling of up to 35 litres per minute in this material and in underlying littoral sands. The material becomes progressively less clayey with depth and grades into underlying littoral sands; precise position of lower contact therefore uncertain.

d) Littoral sand : ranges in thickness from about 2,15 m to 4,55 m, but may be thicker in places. In the uppermost two metres it is usually a coarse, medium and fine shelly sand of very loose to loose consistency (N-value 1-70, which would correspond with an in situ dry density of about 1300-1500 kg/m³. Maximum allowable foundation pressure about 50 kPa (Reference 1). The lower levels of this material are of similar texture, but have a medium dense to dense consistency (N-value 21-48), which would correspond with an in situ dry density of about 1600-1900 kg/m³. Maximum allowable foundation pressure 200-400 kPa (Reference 1).

Horizons e, f and g are, in general, repetitions of horizons b, c and d respectively.

It should be noted that, in the tidal channels, horizon

(c) and the upper part of horizon (d) may be absent beneath the collapsed calcarenite slabs, depending on the depth to which scour has occurred locally."

Four principal factors emerge :

- 1) The relatively extensive depths of the unconsolidated littoral deposits. (In certain boreholes over ten metres)
 - 2) The occurrence of two horizons of low N-values and related high porewater pressures.
 - 3) The identification of a zone of firm to dense and stiff consistency of clayey medium and fine sand with N-values of 11 to 27. (Generally referred to in the report as the marl layer)
 - 4) Marked lack of uniformity of foundation conditions over the length of the proposed wall.
- 4.6 From a structural point of view it was patent at the outset that in view of the previously referred to localised variations in foundation conditions a monolithic structure was desirable.

A semi-circular structure presented the following advantages :

- 1) A good natural geometric relationship between shallow and deep water relative to bather preference.
- 2) The most economical wall length per unit of surface area.
- 3) Certain dynamic load advantages in as much as that although a particular wave train could theoretically impose a simultaneous force system over the length of the wall, the maximum force acts over a limited length of the wall. In the structure under consideration the maximum force may be considered to act over a length of 60 metres i.e. over a 14 percent section of the wall.

It is a corollary that most of the wave inflow takes place over this section.

Beyond this section of wave frontal attack there is an appreciable progressive reduction in dynamic forces due to the effect of a changing angle of wave approach which decreases from a theoretical 90° for that portion of the structure at right angles to the wave direction to 0° at the diametral point.

From the model study photographs it may be observed that progressively away from the frontal attack section

the angle of wave approach tends to increase due to wave refraction over the shallow calcarenite reef. This is however accompanied by an appreciable refractive energy loss.

Disregarding wave refraction along the length of the wall the incident angle varies linearly to length of wall and by applying the equation $R^1 = R \sin^2 \alpha$ the reduction factor for dynamic force calculation along the length of the wall becomes :

1) At frontal attack section	1,00
2) At three-quarter point	0,85
3) At half point	0,50
4) At quarter point	0,15
5) At diametral point	0,00

To satisfy the condition of simultaneous dynamic forces, the forces in their adjusted value for incident angle act radially resulting in an axial ring stress with secondary moments and shear forces due to load change along the length.

From the structural analysis it became apparent that the critical design condition is represented by an internal loading system comprising a pool filled with sand and water at L.W.S.T. tide state.

From the geological investigation it became apparent that in order to secure reasonable waterholding properties it was essential that the sheetpiling as curtain membrane be taken well into the dense clay sand layer. This depth as well as the minimum section required for driving the sheetpiling resulted in a fairly rigid simple cantilever and the effect of a ring beam apportionment of load was disregarded in the factor of safety analysis. This is illustrated by the low stress analysed in the ring beam when maximum stress is developed in the sheetpile in bending.

4.7 In order to determine the axial load resistance of the proposed sheetpiling a series of sheetpile driving and load tests were carried out employing six metre piles driven 4,40 metres into the substrata. Loading using 1,08 KN units took place five days after completion of driving and the following worst deflections recorded :

- 1) At working load 28,5 KN per metre, settlement 0,3 mm.
- 2) At test load 57,1 KN per metre, total settlement from unloaded level 0,6 mm.
- 3) At time zero plus 24h00 pile rose 1,0 mm.
- 4) At time zero plus 40h00 pile settled 1,0 mm.

5) At time zero plus 66h00 pile was unloaded.

6) At time zero plus 92h00 pile level was 0,4 mm above original unloaded position.

Spring tide conditions prevailed during the test period.

The geological section on the line of the sheet pile curtain membrane dictated the use of five metre and six metre piles driven to form an interlocking diaphragm to predetermined depths in accordance with the geological findings.

The pile section adopted was a BZ12 Arbed imported from Luxemborg.

Overturning moments transferred to the sheetpile are absorbed in passive resistance by the substrata in accordance with the deformation condition imposed by the sheetpile in bending deflection.

In the analyses the pile section adopted allows for loss of 50 % of steel section due to corrosion over a 40 year period. At the point where maximum bending moment occurs it is likely that anaerobic conditions prevail and that the assumption for reduced section modulus applicable at this point are conservative.

4.8 The following design considerations apply :

1) Worst possible internal load condition is pool full of sand submerged, at tide state L.W.S.T.

2) Worst possible external load condition occurs at maximum wave forces with pool empty.

3) The submerged sand and substrata mean angle of shearing resistance ϕ , employed in the calculations, is 20°.

4) The solid weight of sand comprising quartzitic and shell particles is 20 KN/m³.

5) The well compacted sand percentage voids is 30 %.

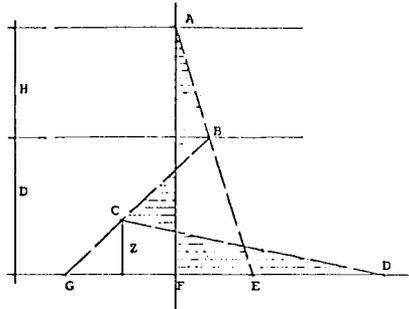
6) From the foregoing the active pressure development factor K_p is 16,2 KN/m² per metre of depth.

7) The passive pressure development factor is 36,6 KN/m² per metre of depth.

8) Horizontal deflection of the superstructure results in a peripheral strain in the concrete section of where δ is the deflection and r the radius of the

pool.

4.9 Calculations are based on Terzaghi Theory for a Cantilever Sheetpile :



$$Z = \frac{D^3 K_p - (H + D)^2 K_a}{(H + 2D)(K_p - K_a)} \dots\dots\dots (1)$$

and

$$\left(\frac{D}{H}\right)^3 + \frac{(K_p - 3K_a)}{K_p - K_a} \left(\frac{D}{H}\right)^2 - K_a \frac{(7 K_p - 3 K_a)}{(K_p - K_a)^2} \left(\frac{D}{H}\right) - K_a \frac{(5 K_p - K_a)}{(K_p - K_a)^2} \left(\frac{D}{H}\right) - \frac{K_a K_p}{(K_p - K_a)^2} = 0 \dots\dots\dots (2)$$

The following values of 'D' have been calculated compatible with the probable values of K_p and K_a .

Sand Level in Pool	Required 'D'
RL 01,40	4,86 m
RL 01,05	4,41 m
RL 00,70	3,92 m
RL 00,40	3,48 m
RL 00,00	3,13 m

'Z' for the extreme condition of 'D' = 4,86 m has been calculated to be 1,01 m, while developing a passive resistance of 55,9 kPa at a depth of 3,85 m equal to a linear rate of development of passive pressure = 14,5 KN/m² per metre of depth.

The 'D', value for the sheet piles installed is 5,40 m.

4.10 Seepage Losses

The substrata formation is as may be seen from the geological section complex and this together with a bedrock horizon of between four metres and eight metres below the lower limit of the sheet pile diaphragm places a low confidence level on the projection of seepage losses. Nevertheless actual observations of water loss conform well to the calculated values.

For the pool full at 01,40 m M.S.L. and tide state - 0,90 the maximum seepage hydraulic gradient is :

$$\frac{2,3}{(4,5 + 5,4)} = 0,23$$

and the critical gradient at which boiling displacement of the sand at L.W.S.T. on seaside can take place :

$$i_{cr} = \frac{\gamma - \gamma_w}{\gamma_w}$$

$$= \frac{20 - 9,8}{9,8} = 1,04$$

$$\text{Factor of Safety} : \frac{1,04}{0,23} : 4,52$$

Assuming the maintenance of a high pore pressure condition on the landward side of the pool which tends to stabilise water movement inland and secondly that the marl layer acts as a semi-pervious homogeneous blanket of effective thickness 1,80 metres as deduced from the geological reports, with permeability $K = 1 \times 10^{-4}$ cm/sec then the mean seepage gradient through the marl layer would be $\frac{1,4}{1,8} = 0,78$ and seepage rate 67 mm/day.

For a period of three days of no inflow the waterlevel would drop by a total of 200 mm.

This has been largely confirmed in the field.

4.11 Concrete Design

The reinforced concrete pile cap was designed to be sufficiently rigid for the structure to act monolithically so as to transfer localised foundation failure conditions to the remainder of the structure while at the same time providing a minimum cover on all steel of 100 mm.

A 30 MPa concrete was employed with 0,30 percent steel reinforcement.

A 0,25 to 1 seaface batter was adopted for wave induced inflow, reduction in dynamic forces and for aesthetic considerations.

4.12 Armouring Design

It is essential to provide adequate toe armouring to prevent progressive scour in the seafloor foundation area, while at the same time limiting the top reduced level of the armouring to the extent necessary to ensure retention of sufficient wave energy for wave induced inflow.

Reasonable assessments of the controlling levels based on experience were made but it is indubitably the case that the model experiments were essential to establish the adequacy of inflow.

Armouring size was determined by the Hudson Formula :

$$W = \frac{W_r H^3}{K_D (S_r - 1)^3 \cot \theta}$$

W_r = unit weight in lbs/ft³ (25 KN/m³)
 = 156 lbs/ft³

H = Design wave height in feet
 = 6,5 ft (2,0 m)

S_r = Specific gravity of armour unit
 = 2,5

W_w = Unit weight seawater (10 KN/m³)
 = 64,0 lbs/ft³

θ = Structure slope angle in degrees
 = 14°

K_D = Stability co-efficient
 = 2,5

W = 1270 lbs (577 kg)
 Say 600 kg.

4.13 Construction

The construction method adopted by the contractor involved the provision of a rubble mound as a sea cofferdam with a sand access road on the inside of the berm. The rubble mound was constructed to a height of 4,00 m M.S.L.

Sheet piles five to six metres in length were driven using a compressed air pile driver suspended from a 30 ton crawler crane off the access road.

The contractor successfully designed a system of driving to geometric configuration by use of steel

channel walers anchored to the previous piles and forward anchored to a transportable concrete block of some two tons mass.

A moveable shutter arrangement capable of being trimmed to the radius of curvature and hung from a gantry system on rails was a further successful innovation by the contractor.

The concrete was cast in alternate section of 7,5 metre length with continuity reinforcement but without waterstops.

On completion of construction the major portion of the rubble mound was employed to form the toe armouring.

The unavoidable inclusion in the armouring of under-size material resulted in an inflow of sharp rock fragments into the pool and future designs should provide particular attention to the temporary cofferdam in order to employ material which later may be utilized in the armouring zone.

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2) Research by the S.A. Institute of Oceanology of the S.A. Council of Scientific and Industrial Research has been fundamental to the submissions made and particular reference is made to the assistance of Dr D H Swart.

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4. Shore Protection Manual of U.S. Army Coastal Engineers Research Centre.