Appendix A: Concept Designs
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1. Introduction

Severe erosion at Portsea Front Beach in recent times has resulted from changes to the distribution of wave energy along the Weeroona Bay foreshore. Wave transformation studies showed a clear focussing of wave energy coming off the Quarantine Bank, directed towards Portsea Front Beach (Figure 1).

![Wave transformation model](image)

**Figure 1.** Results from the wave transformation modelling [Boussinessq] showing the distribution of swell wave heights in the vicinity of Portsea Front Beach for an offshore significant wave height of 1.5 m with a peak wave period of 12 seconds on mid-tide level. Colour shading represents increasing wave height and energy from blue (low wave height) to dark red.

The uneven distribution of wave energy along the Weeroona Bay foreshore has the following impacts in respect of littoral drift transport:

1) Increased wave energy impacting on the section of the beach at and to the east of Portsea Pier causing severe dune erosion and cross-shore transport of littoral drift.

2) Increased wave energy impacting on the section of the beach at and to the east of Portsea Pier causing removal of littoral drift from the area through increased rates of alongshore littoral drift transport to the east.

3) A relatively low rate of littoral drift transport eastward along the western part of Weeroona Bay, reducing the supply of sand to Portsea Front Beach relative to the rate at which it is being transported eastward and away from the beach.
Options to ameliorate the erosion process at Portsea Front Beach aim to address each of these processes. The following options have been considered:

- configuration dredging
- breakwaters (detached and attached)
- groynes
- rock revetment
- sand nourishment
- removal of the existing sand bag revetment.

In the following these options are designed in principle for assessment by modelling, costing and environmental impact.
2. Configuration Dredging

2.1 Introduction

The two-dimensional (horizontal) refraction and diffraction of long crested surface gravity waves by pits and trenches is a classic problem in fluid dynamics that has been studied and for which solutions have been provided by researchers over many years (Kirby & Dalrymple, 1983). As waves propagate shoreward into shallow water, wave refraction results in their crests trending towards becoming parallel to the orientation of the local isobaths. Abrupt bathymetric discontinuities that are aligned at acute angles with the direction of swell wave propagation can induce severe wave refraction. Depending on the relative water depths, the level of the surrounding seabed, the wavelength (that is, wave period) and the angle of incidence to the channel, among other factors, wave energy may be reflected off such bathymetric discontinuities (Nielsen et al. 2011a, b). This can result in a significant re-distribution of wave energy along the shore, changing sediment transport regimes and benthic habitats.

At locations where swell waves are somewhat unidirectional, bathymetric discontinuities, such as those instigated by dredging, can have significant impacts on swell wave transformation, wave energy distribution along the shoreline, transport of littoral drift and shoreline alignment. For example, wave reflection off the Platypus Channel in Cleveland Bay, Townsville, North Queensland is well-documented by vertical aerial and ground photography, in-situ wave data collection and detailed numerical modelling (Nielsen et al. 2011a,b). Other examples can be found in Price et al. (1981) and Misra et al. (2008).

The focussing of wave energy onto Portsea Front Beach presents as a near-unidirectional wave train for the range of swell wave periods considered, providing the opportunity for considering configuration dredging as an option to change the inclement wave conditions causing erosion. As indicated schematically in Figure 2, two opportunities that have been identified include an offshore trench on the Quarantine Bank and a trench nearshore to Portsea Front Beach inshore of the Portsea Hole.

![Figure 2. Schematic diagrams of possible wave reflection trenches on the Quarantine Bank (left) and inshore of the Portsea Hole (right).](image-url)
A dredged configuration nearshore to Portsea Front Beach would be preferred because:

- the lower rates of sand transport there would require less maintenance dredging to maintain the configuration; the rates of sand transport on the Quarantine Bank being high (Advisian, 2016)
- it would intercept wave focusing onto Portsea Front Beach that is caused by Portsea Hole
- it would have no impact whatsoever on tidal currents or wave energy over the Portsea Hole, being downstream/down-drift/in the lee of it
- being closer to the beach its effectiveness and impacts would be predicted more accurately.

### 2.2 Design Parameters

The parameters relevant to the design of a wave reflection trench or pit are as follows (after Nielsen et al. 2011a, b):

- $d_1$: surrounding water depth on approach to the trench or pit
- $d_2$: water depth in the trench or pit
- $W$: width of the trench or pit
- $\theta$: direction of wave advance with respect to the trench orthogonal
- $T_p$: spectral peak wave period
- $L_1$: local wavelength in the water depth on approach to the trench or pit

Nielsen et al. (2011a,b) presents nomograms, reproduced in Figure 3, that allow for the conceptual design of a wave reflection trench. The concept design has been based on a peak wave period of 13 s and a direction of wave advance of around 35°.

The offshore trench would be located in a water depth of around 10 m. In this depth the local wavelength would be around 130 m. For the following parameters, Figure 3 gives the following transmitted wave height coefficients:

- For an angle of 35° (i.e., 55° in Figure 3), the transmitted wave height coefficient is 0.3
- For an average trench width $W = 100$ m, $W/L_1 = 0.8$ and the transmitted wave height coefficient is 0.35
- For an average trench depth of 5 m, the average water depth in channel would be $d_2 = 15$ m, $d_2/L_1 = 15/130 = 0.12$, for which the transmitted wave height coefficient is 0.2
- For $d_1/L_1 = 10/130 = 0.08$, the transmitted wave height coefficient is 0.15.

For a transmitted wave height coefficient of 0.3, the reflected wave energy would be 91% of the incident swell wave energy.
### Appendix A: Concept Designs

**Figure 3.** Reflected and transmitted significant wave height coefficients for variations in dependent parameters (Nielsen et al., 2011a, 2011b).

#### 2.3 Concept Design

##### 2.3.1 Offshore Dredged Configuration

Several offshore dredged configurations have been investigated, the most promising concept design for which is shown in Figure 4. The depression has a maximum depth of 10 m (compared with the surrounding seabed level) and has side slopes of 1V:10H.
Figure 4. Concept design footprint for an offshore dredged configuration designed to redistribute the inclement wave energy away from Portsea Front Beach equitably along the Weeroona Bay foreshore. Impacts on wave focussing are shown to decrease incident wave heights to the east of the pier and to increase wave energy along the western shore of Weeroona Bay (compare with Figure 1). Wave height focussing is shown to be increasing from green to yellow to red.

The dredged trench has a footprint area of some 130,000 m² and a dredged volume of 560,000 m³. Of this some 300,000 m³ would be used to prograde the entire beachfront of Weeroona Bay (Figure 5) with the remaining 260,000 m³ being side-casted into adjacent deeper waters. A cross section of the proposed beach nourishment of Weeroona Bay, taken at Portsea Pier, is shown in Figure 5. The beach profile extends from RL 2 m AHD to RL −4 m AHD (Advisian 2016). The entire length of Weeroona Bay is 1,200 m. Therefore, nourishment of some 300,000 m³ would prograde the beach by some 42 m (300,000/(1,200×6)).
Pertinent to the assessment of this option is the distribution of sediment transport over the Quarantine Bank, which is shown in Figure 6.

Figure 6. Distribution of the net rates of sediment transport (m$^3$/m/a) over the Quarantine Bank superimposed onto the bathymetry modified by the offshore dredged configuration (Trench 11)
For an average sediment transport rate of 750 m$^3$/m/a approaching the trench and for the 500 m length of the trench (projected normal to the transport direction), it is estimated that the trench would infill within 1.5 years ($560,000/(500*750)$). In reality, the trench is likely to become wider and shallower over time, which is likely to occur over several years. Nevertheless, the requirement for maintenance dredging is most likely to rule this option unviable.

### 2.3.2 Nearshore Dredged Configuration

Potential nearshore dredged configurations would appear to have limitations on the depth of dredging that may be achieved, which may affect their effectiveness. As indicated in Figure 7, in the nearshore area of Portsea Front Beach, the −10 m isobath presents as a broad shelf, suggesting shallowly underlying hardpan at around that level.

![Nearshore bathymetry at Portsea Front Beach. Isobath intervals are at 2 m increments from 0 m AHD at the shore. Portsea Hole is depicted in white.](image)

The site is located on Quaternary age dunes comprising siliceous and calcareous sands, cemented to varying extents, often forming rock (PB 2003). The less stable areas of cliff at Point Franklin exhibit sandstone and sandy slopes where bedding is dipping towards the coast. The more stable areas have limestone present in the cliff face (PB 2003).
Figure 8. Levels to sound bedrock as gleaned from piling records 2007 (Appendix A)

Point Franklin exhibits features of wave erosion processes with undercutting of the cliffs having occurred in many places forming flat-roofed and deep caves with undercuts up to 12 m deep (PB 2003; see Figure 9). The piling contractor (for the Portsea Pier repairs) described the underlying seabed strata as being layered calcareous sands with the original piling being founded on higher level layers of indurated sand overlying soft sands.

Figure 9. Point Franklin exhibiting wave erosional features of the calcareous sandstone (photos: PB 2003)

It has been assumed that the hardpan underlying the beach and offshore seabed area comprises calcareous sandstone as an extension of the headland of Point Franklin. If the nature of the hardpan is similar to the calcareous sandstone comprising Point Franklin, as described above, then it may be assumed that the hardpan can be dredged with a backhoe, albeit with some effort and cost.

Several concept designs for a nearshore dredged configuration trench have been investigated. Shallow pits and trenches dredged to depths above the assumed level of hardpan were ineffective. The most promising option is presented in Figure 10. The trench aims to reflect the high wave energy train that is focussed onto Portsea Front Beach away to be dissipated on the offshore sand banks north of Point Franklin.
Figure 10. Concept design for a nearshore dredged configuration trench to reflect wave energy away from Portsea Front Beach. The diagram shows results from the wave transformation modelling (Boussinesq) with the high wave energy train directed towards Portsea Front Beach (yellow/red) being reflected north-eastwards to be dissipated on the sand bank areas north of Point Franklin. It is instructive to compare this with Figure 1.

The level to which the base of the trench would be excavated has been assumed to be −16 m AHD, the hardpan being side-casted and swept onto the adjacent seabed area on the offshore side of the trench to raise the seabed level there by around 4 m. This enhances the effectiveness of the trench to divert the wave energy offshore. The viability of dredging to this level would need to be confirmed by borehole investigation. The volume of sand that could be dredged from the trench in Figure 10 would be around 200,000 m$^3$, which could be used to prograde the entire Weeroona Bay foreshore beach by around 30 m (see Figure 11). The volume of rock has been estimated to be around 430,000 m$^3$, which has been distributed over an area 700 m × 170 m on the seaward side of the trench raising the seabed there by around 4 m. These volumes would need to be confirmed by offshore borehole investigation.

Figure 11. Cross-section of beach profile at Portsea Jetty nourished by 30 m width.
3. Detached Breakwaters

3.1 Introduction

Detached breakwaters are shore-parallel structures designed to dissipate wave energy and reduce wave impacts at the shore. They induce sediment deposition in their lee by reducing the rate of alongshore transport of littoral drift and offshore transport during storms. Their crests may be emergent or submerged. The sediment that is deposited between the breakwater and the shoreline may form a tombolo, which joins the shoreline to the breakwater, or a salient (Figure 12).

![Figure 12. Detached Breakwaters at Tel Aviv-Yafo, Israel forming salients to the left of the marina and tombolos to the right (photo courtesy Google earth)](image)

3.2 Design Principles

The parameters that determine whether a salient or a tombolo forms are depicted in Figure 13 and include:

- \( L_s \) Breakwater length
- \( L_g \) Gap width
- \( W \) Design beach width
- \( Y \) Offshore distance to breakwater
- \( Y_o \) Offshore distance to breaker line
- \( Y_t \) Offshore distance to salient tip
- \( Y_g \) Onshore distance to salient trough
- \( Y_{\text{min}} \) Beach width to allow for the design storm
- \( d_s \) Depth at breakwater below mean sea level

To allow littoral drift to continue to be transported through the project area to downdrift beaches, salients are the preferred shoreline response for a detached breakwater system. Salients are likely...
to predominate when the breakwaters are sufficiently far from shore (i.e., beyond the breaker line), are short relative to incident wavelength and relatively transmissible to wave energy (low crested and/or with large gaps). Wave action and longshore currents tend to keep the salient from connecting to the structure.

Three dimensionless ratios, $Y/d_s$, $L_s/L_g$ and $L_s/Y$ determine salient or tombolo response.

Long breakwaters close to shore favour tombolo formation. For a single breakwater close to the breaker line, a tombolo is likely to form when the breakwater length is around 2.5 times its distance offshore (Suh & Dalrymple 1987 Figure 7).

For segmented breakwaters of segment length 1.5 times the distance offshore, tombolos are likely to form where the breakwater segments are longer than the wavelength and the gap between the segments.

Conversely, short breakwaters at greater distances from shore favour salient formation with, generally, breakwater length less than the distance offshore ($L_s/Y < 1$). Dally and Pope (1986) recommend $L_s/Y < 0.67$ for salient formation.

Hawkins et al. (2010) presents wavelength explicitly as a relevant normalising parameter for determining salient/tombolo formation. However, the relationships presented are very nearly linear for conditions where the breakwater length is longer than the wavelength and its distance offshore is farther than one wavelength, indicating that for those conditions the wavelength is not a relevant parameter and that for those conditions the data indicated that tombolos would form when the breakwater length is greater than its distance offshore. However, for structures closer to the shore or of a scale smaller than one wavelength the graphical data presented could not be interpolated clearly. Nevertheless, the graphical data indicated that for structures located around one wavelength offshore, their length would need to be far shorter than one wavelength to prevent tombolo formation; that is, for salient formation the length of the breakwater, $L_s$, would be far shorter than the local wavelength ($L_s << L$).

For a single breakwater, Figure 14 presents the relationship for the growth of a salient, which is based on empirical field data compiled by Sylvester & Hsu (1997).
Figure 13. Definitions of key variables for nearshore breakwaters (CEM 2008 Figure V-3-21)
\[
\frac{(Y-Y_s)}{L_s} = 0.6784 \left(\frac{Y}{L_s}\right)^{1.2148}
\]

Equation (1) can be re-arranged as follows:

\[
L_s = 0.1642 \left(\frac{Y^{5.6555}}{(Y-Y_s)^{6.6555}}\right)
\]

From Equation (2) the requisite breakwater length can be determined from its distance offshore and the scale of the salient required.

However, Figure 14 shows that where there are multiple empirical ordinate data points for a particular abscissa value, the range given for the extent of the salient is large, with values varying by around a factor of 2, indicating that the prediction of the salient extent is not accurate.
3.3 Concept Design

3.3.1 Introduction

A concept design has been based on the methods presented in CEM (2008). A salient solution has been adopted to maintain some littoral drift transport to down-drift beaches. The basic assumptions made included that the structure would be located offshore of the breaker zone to avoid tombolo formation.

Based on Hallermeier (1983), the water depth, $d_{sa}$, offshore of which a detached breakwater should be placed to avoid tombolo formation, is given by:

$$d_{sa} = \frac{2.9H_e}{\sqrt{(S-1)}} - \frac{110H_e^2}{(S-1)gT_e^2}$$

(1)

where:

- $d_{sa}$ = annual seaward limit of the littoral zone
- $H_e$ = deepwater wave height exceeded 12 hours per year
- $S$ = ratio of sediment to fluid density
- $g$ = acceleration due to gravity
- $T_e$ = wave period corresponding to $H_e$

The design wave adopted was that occurring for 12 hours per year. At the point of erosion at Portsea Front Beach, the significant wave height in 5 m water depth occurring for 12 hours per year (i.e., some 0.14% of the time) was determined to be $H_{10,5m} = 0.8$ m (Advisian 2016 Appendix C Table 5-8). This wave has a period around 13.5 s. For that condition, $H_{10,5m} = 1.0$ m and $H_{max,5m} = 1.5$ m (Goda’s method). At breaking, $H_{sb} = 1.6$ m, $H_{10,b} = 1.8$ m and $H_{max,b} = 2.5$ m and the “unrefracted” deepwater wave heights would be, $H_{s,o} = 0.6$ m, $H_{10,o} = 0.8$ m and $H_{max,o} = 1.1$ m (respectively).

As indicated in Figure 15, during severe storm conditions waves are breaking onto Portsea Front Beach close to the shore and well inside of the Portsea Pier head. The breaking wave height in Figure 15 is estimated to be around 2 m and would be breaking in around 2 m water depth. This would be equivalent to the 12 hours per year $H_{10}$ breaking wave height and similar to the design condition. According to Hallermeier’s equation (1), for this condition the breakwater would need to be located in a water depth greater than around 2.5 m to prevent tombolo formation; that is, referring to Figure 16, beyond some 60 m from the shoreline. At Portsea Pier, modelling indicated that cross-shore transport had the potential to extend some 80 m from the shoreline (Figure 16).

If breakwaters are placed too far offshore their impact on the beach will be minimal. The relevant distance as well as the breakwater dimensions should refer to the local wavelength. In around 5 m water depth, for a wave of period 14 s, the wavelength is around 100 m.
Figure 15. Swells rolling in on Portsea Front Beach 25 June 2014 (photo courtesy Dept. Env., Land, Water & Planning, Vic.)

Figure 16. Distribution of littoral drift transport at Portsea Pier (Advisian 2016)
3.3.2 Nearshore Segmented Breakwater

For a nearshore segmented breakwater the breakwaters would be placed some 70 m ($Y$) from the nourished shoreline in a water depth of around 2 m and with breakwater lengths $L_s = 40$m with gaps $L_g = 50$ m (Figure 17). This is considered to be far enough offshore to prevent tombolo formation but not so far as to have minimal effect. In accordance with Figure 18, this combination is likely to produce well developed salients. In accordance with Suh & Dalrymple (1987), $Y_s = 34$m. In accordance with Berenguer & Enriquez (1988), $Y_g = -20$ m. The foreshore would be nourished to prograde the beach some 40 m so that the salient formation would not reduce the beach width to less than 20 m. This would comprise some 120,000 m$^3$ of sand nourishment.

Figure 17. Proposed detached breakwater layout; $L_s = 40$ m, $L_g = 50$ m, $Y = 70$ m from nourished shoreline.

Figure 18. Morphological response as a function of structural parameters (after Pope and Dean 1986) and the prediction of well-developed salients for the proposed concept design (red dot)
The nearshore segmented breakwater solution is not favoured. Portsea Front Beach, inshore of the Pier Head, is a very popular SCUBA diving and swimming location, including the Portsea Swim Classic and nippers training, and dive charter boats work regularly at Portsea Pier. The segmented breakwater could present obstacles for navigation, generate currents and interrupt the visual amenity.

3.3.3 Offshore Single Breakwater

For a single breakwater with a salient length 56 m and breakwater distance offshore of 180 m, well outside the surf zone, from Equation (2) the required breakwater length is 170 m. The relationship of salient length/breakwater length/distance offshore is plotted on the relationship graph of Silvester and Hsu (1997) in Figure 19 and on the graph of Suh and Dalrymple (1987) in Figure 20.

![Figure 19. Plot of single breakwater concept design on graph of Silvester & Hsu (1997)](image)
For the concept design parameters, Silvester and Hsu (1997) predicted a salient amplitude of 56 m. However, it is noted that the data show that where the breakwater length approximates its distance offshore ($L_s/Y=1$), the salient amplitude can vary from $0.2Y$ to $0.5Y$, which, in this case, ranges from 36 m to 90 m.

### 3.4 Structural Design

#### 3.4.1 Sheet Piling Wall

A concept design utilising sheet piling is presented in Figure 21. The beach is prograded some 20 m and the salient is shown with a 56 m maximum extent. The total volume of sediment required would be 84,000 $m^3$. 

![Diagram showing relationship between salient amplitude, distance offshore, and surfzone width/distance offshore.](image-url)
3.4.1.1 **Sheet piling wall in sand**

The following parameter values have been adopted for preliminary design:

- Crest level = 2 m AHD
- Seabed level = -6 m AHD
- Maximum wave height = 1 m
- Water level = 1 m AHD
- Submerged sand mass = 1,200 kg/m³
- Coefficient of passive earth pressure = 1.5

![Diagram showing design forces and soil reactions on a sheet piling detached breakwater](image)
$H_s=0.5\text{m}; \ H_{\text{max}}=1\text{m}$ (non-breaking)

Assume 1 m water pressure from the crest at +2 m AHD with water level at +1 m AHD.

Adopt Factor of Safety = 2; working stress design

Overturning moment around toe:

$\text{Mo}=1000*(8+x)^2/2*10/1000 = 0.5*(8+x)^2$ kN-m/m of wall

Resisting moment:

$K_p = 3$; for FoS of 2, $K_p = 1.5$, $\gamma_{\text{sdb}}=1200 \text{ kg/m}^3$

$\text{Mr} = 0.5*1.5*1200*x^3/(3*1000) = 0.3x^3$ kN-m/m of wall

### Table 1. Global Overturning Stability for Sheet Piling Wall

<table>
<thead>
<tr>
<th>x</th>
<th>Mo</th>
<th>Mr</th>
<th>Mo/Mr</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>40.5</td>
<td>0.3</td>
<td>135.0</td>
</tr>
<tr>
<td>2.0</td>
<td>50.0</td>
<td>2.4</td>
<td>20.8</td>
</tr>
<tr>
<td>3.0</td>
<td>60.5</td>
<td>8.1</td>
<td>7.5</td>
</tr>
<tr>
<td>4.0</td>
<td>72.0</td>
<td>19.2</td>
<td>3.8</td>
</tr>
<tr>
<td>5.0</td>
<td>84.5</td>
<td>37.5</td>
<td>2.3</td>
</tr>
<tr>
<td>6.0</td>
<td>98.0</td>
<td>64.8</td>
<td>1.5</td>
</tr>
<tr>
<td>7.0</td>
<td>112.5</td>
<td>102.9</td>
<td>1.1</td>
</tr>
<tr>
<td>7.3</td>
<td>117.2</td>
<td>117.2</td>
<td>1.0</td>
</tr>
<tr>
<td>9.0</td>
<td>144.5</td>
<td>218.7</td>
<td>0.7</td>
</tr>
<tr>
<td>10.0</td>
<td>162.0</td>
<td>300.0</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Solution $x=7.3\text{m}$

### Table 2. Bending Moment in SPW at Distance $Y$ from Top

<table>
<thead>
<tr>
<th>Y (m)</th>
<th>Mo  (kN-m)</th>
<th>Mr  (kN-m)</th>
<th>Mo-Mr (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.5</td>
<td>0.3</td>
<td>0.2</td>
</tr>
<tr>
<td>2.0</td>
<td>2</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>3.0</td>
<td>4.5</td>
<td>4.5</td>
<td>0</td>
</tr>
<tr>
<td>4.0</td>
<td>8</td>
<td>8</td>
<td>0</td>
</tr>
<tr>
<td>5.0</td>
<td>12.5</td>
<td>12.5</td>
<td>0</td>
</tr>
<tr>
<td>6.0</td>
<td>18</td>
<td>18</td>
<td>0</td>
</tr>
<tr>
<td>7.0</td>
<td>24.5</td>
<td>24.5</td>
<td>0</td>
</tr>
<tr>
<td>8.0</td>
<td>32</td>
<td>32</td>
<td>0</td>
</tr>
<tr>
<td>9.0</td>
<td>40.5</td>
<td>0.3</td>
<td>40.2</td>
</tr>
<tr>
<td>10.0</td>
<td>50</td>
<td>2.4</td>
<td>47.6</td>
</tr>
<tr>
<td>11.0</td>
<td>60.5</td>
<td>8.1</td>
<td>52.4</td>
</tr>
<tr>
<td>12.0</td>
<td>72</td>
<td>19.2</td>
<td>52.8</td>
</tr>
<tr>
<td>13.0</td>
<td>84.5</td>
<td>37.5</td>
<td>47</td>
</tr>
<tr>
<td>14.0</td>
<td>98</td>
<td>64.8</td>
<td>33.2</td>
</tr>
<tr>
<td>15.3</td>
<td>117.2</td>
<td>117.2</td>
<td>0</td>
</tr>
</tbody>
</table>

Max Bending Moment = 53 kN-m/m.
Using these parameters the required depth of penetration is 7.3 m and the maximum bending moment is 53 kN-m/m. The adopted section is a fibre reinforced plastic Box-section sheet pile as shown in Figure 23.

![Box Profile](image)

<table>
<thead>
<tr>
<th>Section</th>
<th>Width</th>
<th>Height</th>
<th>Thickness</th>
<th>Profile</th>
<th>Allowable Moment</th>
<th>Section Modulus</th>
<th>Moment of Inertia</th>
</tr>
</thead>
<tbody>
<tr>
<td>UC-50</td>
<td>914</td>
<td>254</td>
<td>9</td>
<td>8.3</td>
<td>Box</td>
<td>77.10</td>
<td>1,118</td>
</tr>
</tbody>
</table>

Figure 23. Proposed sheet pile wall section for offshore breakwater (JSTEEL or similar)

For the depth of penetration of 7.3 m a cantilever sheet piling solution is not viable as the seabed is underlain shallowly by hardpan at around -9 m CD, which limits piling penetration to less than 4 m. The structure would need to be a braced sheet piling wall or a rubble mound or a mound of interlocking concrete units.

### 3.4.1.2 Braced Sheet Piling Wall Pinned on Rock

Neglect sand resistance (pin in rock prevents sufficient strain to develop passive pressure)
Assume UDL 1m water pressure
SPW pinned at top with 1:10 raker piles (1H:10V) & at bottom in rock
3.4.2 Rubble Mound Breakwater

A rubble mound detached breakwater scheme is presented in Figure 24. The structure is in around 6 m water depth with a crest at RL 2 m AHD. The side slopes are set to 1V:2H.

The breakwater length is 170 m.

The design parameters are as follows:
- Crest level = 2 m AHD; Seabed level = −6 m AHD; Water level = 1 m AHD
- Design significant wave height (non-breaking) $H_s = 1.1$ m; $H_{10} = 1.4$ m
- Rock density = 2,650 kg/m$^3$
- Side slopes for rock fill 1V:2H (factor of safety of 2 for geotechnical stability)

Armour size for preliminary design has been determined using the Hudson formula as documented in Table 3 and Table 4. The total volume of rock fill is 22,000 m$^3$ and the volume of rock armour is 5,300 m$^3$. 

![Figure 24. Rubble mound or concrete unit detached breakwater](image-url)
### Table 3. Rock Armour Design Parameters and Solution for Breakwater Trunk using the Hudson Equation

<table>
<thead>
<tr>
<th>Inputs</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Unit rock</td>
<td></td>
</tr>
<tr>
<td>Unit Wt Sea Water</td>
<td>1,025.00 kg/cu m</td>
</tr>
<tr>
<td>Unit Wt Armour Stone</td>
<td>2,650 kg/cu m</td>
</tr>
<tr>
<td>Design Wave H, (H10)</td>
<td>1.40 m</td>
</tr>
<tr>
<td>Stability Coefficient (KD)</td>
<td>2.0</td>
</tr>
<tr>
<td>Layer Coefficient</td>
<td>1.00</td>
</tr>
<tr>
<td>Porosity</td>
<td>37%</td>
</tr>
<tr>
<td>Cotan Structure Slope</td>
<td>2.00</td>
</tr>
<tr>
<td>No. Units Comprising Layer</td>
<td>2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cover Layer</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{max}$</td>
<td>0.58 m</td>
</tr>
<tr>
<td>$D_{10}$</td>
<td>0.64 m</td>
</tr>
<tr>
<td>$D_{min}$</td>
<td>0.69 m</td>
</tr>
<tr>
<td>Layer Thickness</td>
<td>0.56 m</td>
</tr>
<tr>
<td>Minimum Crest Width</td>
<td>1.11 m</td>
</tr>
<tr>
<td>No. Units per Surface Area</td>
<td>4.07</td>
</tr>
<tr>
<td>$W_{max}$</td>
<td>342 kg</td>
</tr>
<tr>
<td>$W_{10}$</td>
<td>456 kg</td>
</tr>
<tr>
<td>$W_{min}$</td>
<td>570 kg</td>
</tr>
</tbody>
</table>

### Table 4. Rock Armour Design Parameters and Solution for Breakwater Roundhead using the Hudson Equation

<table>
<thead>
<tr>
<th>Inputs</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Unit rock</td>
<td></td>
</tr>
<tr>
<td>Unit Wt Sea Water</td>
<td>1,025.00 kg/cu m</td>
</tr>
<tr>
<td>Unit Wt Armour Stone</td>
<td>2,650 kg/cu m</td>
</tr>
<tr>
<td>Design Wave H, (H10)</td>
<td>1.40 m</td>
</tr>
<tr>
<td>Stability Coefficient (KD)</td>
<td>1.4</td>
</tr>
<tr>
<td>Layer Coefficient</td>
<td>1.00</td>
</tr>
<tr>
<td>Porosity</td>
<td>37%</td>
</tr>
<tr>
<td>Cotan Structure Slope</td>
<td>2.00</td>
</tr>
<tr>
<td>No. Units Comprising Layer</td>
<td>2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cover Layer</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{max}$</td>
<td>0.55 m</td>
</tr>
<tr>
<td>$D_{10}$</td>
<td>0.72 m</td>
</tr>
<tr>
<td>$D_{min}$</td>
<td>0.78 m</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>0.63 m</td>
</tr>
<tr>
<td>Layer Thickness</td>
<td>1.25 m</td>
</tr>
<tr>
<td>Minimum Crest Width</td>
<td>1.88 m</td>
</tr>
<tr>
<td>No. Units per Surface Area</td>
<td>3.21</td>
</tr>
<tr>
<td>$W_{max}$</td>
<td>489 kg</td>
</tr>
<tr>
<td>$W_{10}$</td>
<td>652 kg</td>
</tr>
<tr>
<td>$W_{min}$</td>
<td>815 kg</td>
</tr>
</tbody>
</table>
4. Attached Breakwater

4.1 Introduction

An attached breakwater reduces wave energy at the shoreline in its lee. It is also a complete barrier to littoral drift transport.

4.2 Concept Design

The concept design for an attached breakwater is presented in Figure 25 and comprises a “T-groyne” that could provide for a small boat harbour. The western wing of the “T” provides sheltering to allow build-up of littoral drift for bypassing. Without this wing there would be no build-up of sand because wave incidence, enhanced by wave reflection off the structure, would result in high wave energy at the shore and offshore-running rip that would deposit the sand into deep water. Initial nourishment to prograde the beach 20 m would be 60,000 m$^3$. A sand bypassing system would be required to transport the up-drift fillet to the down-drift beaches. The rate of sand build-up has been estimated at 2,000 m$^3$/a (Appendix B). The “T-type” breakwater would need to be a rock rubble mound, accessed from shore, because the shallowly underlying bedrock prevents it to be constructed from sheet piling.

![Figure 25. Concept design of a “T-type” attached rubble mound breakwater](image)

4.3 Structural Design

The design parameters are as follows:
- Offshore breakwater length = 260 m; Western “T” length 70 m; Distance offshore = 180 m
- Crest width 5 m; Crest level = 2 m AHD; Seabed level = -6 m AHD; Water level = 1 m AHD
- Design significant wave height (non-breaking) $H_s = 1.1$ m; $H_{10} = 1.4$ m
- Rock density = 2,650 kg/m$^3$
- Side slopes for rock fill 1V:2H (factor of safety of 2 for geotechnical stability)

Armour size for preliminary design has been determined using the Hudson formula as documented in Table 3 and Table 4. The total volume of rock armour is 15,000 m$^3$ and rock fill is 45,000 m$^3$. 
5. Headland Groyne

5.1 Introduction

Groynes are linear structures built approximately perpendicular to the shoreline extending across part, or all, of the littoral zone. They are used on coasts with differential rates of alongshore transport of littoral drift that may be causing erosion. They are designed to interfere with littoral drift transport in such a way as to increase beach width and retain sand, thereby stabilising the shoreline. The groynes alter the shoreline orientation locally, thereby reducing locally the rate of alongshore littoral drift transport.

Groynes may be constructed from timber, concrete, steel and plastic sheet piling or rock rubble and they may or may not be linear features (Figure 26).

![Figure 26 Left: Concrete sheet pile groyne; Center: Timber sheet pile groyne; Right: Rock groynes](image)

5.2 Design Principles

The basic concept behind groyne design is to allow for the shoreline to become re-oriented to the local wave direction, thereby reducing the rate of alongshore transport of littoral drift (Figure 27).

The general rule of the thumb is to make the groyne spacing to length ratio about two to four (USACE 2002). The length controls water depth at the end and, hence, the amount of sediment by-passing around the tip. But the cross-sectional elevation in the swash zone controls over-passing, the length and elevation on the beach berm control shore-passing, and the structural materials control through-passing as takes place in rubble-mound and permeable groins. Tidal range, predominant wave characteristics (height, period, direction), net and gross rates of alongshore sediment transport and grain size are key hydrodynamic and sediment parameters.

Unless combined with beach nourishment, the long term effect of groynes is to impound sand on the up-drift side causing erosion of beaches on the down-drift side. For this reason, if groynes are placed in a continuous littoral drift system, they must be prefilled to minimise their impact on adjacent beaches (Dean, 1992). The most appropriate locations for groyne usage are at the ends of littoral drift systems (Dean, 1992).
5.3 Concept Design

Wave energy incidence at Portsea Pier to 200 m eastward is considerably higher than that on the down-drift section of coast nearer to Point Franklin (at L11) and on the up-drift section along the western beach of Weeroona Bay (Figure 28). Further, the beach alignment and, hence, the angle of wave incidence is more obtuse to the incoming wave approach angle in the vicinity of Portsea Pier than on the beach nearer to Point Franklin (Figure 29). As the rates of littoral drift transport vary proportionally to the breaking wave height squared and to the sine of twice the angle of incidence (USACE 2002), both of these factors result in higher rates of littoral drift transport potential at Portsea Pier.

The littoral drift potential at Portsea Pier (L8) is greater than that on the beach to the east nearer to Point Franklin (L11) by a factor of 3.7 (Advisian 2016). This is as a result of wave height difference \((0.15/0.13)^2 = 1.33\); see Figure 28), with the remainder (factor 2.80) due to the difference in wave approach angle/beach alignment. To reduce the littoral drift transport potential it would be necessary to either reduce the incident wave height, change the foreshore alignment or do both.
With a groyne it is possible only to change the foreshore alignment. In this case, the groyne would need to allow bypassing of littoral drift so that down-drift beaches do not erode. However, the groyne compartment may still erode if the supply of littoral drift transport from the western side of Weeroona Bay is insufficient to feed the littoral drift transport demand at Portsea Pier, which appears to be the case at present with erosion being exacerbated by a lack of sand supply from the beach to the west of Portsea Pier. The nourished foreshore east of the Pier would be exposed to the severe wave climate experienced at present because the groyne option does not modify the incident wave climate. The nourishment sand, therefore, would be susceptible to severe cross-shore erosion during storms and may be lost to the foreshore.

Two concept designs for a headland groyne solution are presented in Figure 30 (groyne at Point Franklin) and Figure 31 (groyne on Portsea Front Beach). Both designs aim to alter the foreshore alignment in such a way as to reduce the rate of littoral drift transport away from the eroded area. Given the shallowly underlying bedrock, the groyne structure cannot utilise piling and, therefore, is a rock-fill rubble-mound.
The groyne at Point Franklin would need to be some 110 m long to achieve a 10° rotation in shoreline alignment and a 40 m beach width in front of the eroded area on Portsea Front Beach. The total volume of nourishment sand would be some 100,000 m$^3$.

A groyne on Portsea Front Beach, as shown in Figure 31, would need to be some 110 m long to achieve a 10° rotation in shoreline alignment with a 40 m beach width in front of the eroded area, with some 50,000 m$^3$ of nourishment sand required to reinstate bypassing to obviate down-drift erosion. The foreshore slope at Portsea Front Beach is around 1V:30H, putting the head of a groyne in a depth of around 4 m. The volume of rock would be some 10,000 m$^3$. 
5.4 Structural Design

5.4.1 Design Parameters

The design parameters are as follows:

Crest level = 3 m AHD; Seabed level at head = −4 m AHD; Water level = 1 m AHD
Maximum breaking wave height = 2.5 m
Rock density = 2,650 kg/m$^3$
Side slopes for rock fill 1V:2H (factor of safety of 2 for geotechnical stability)

5.4.2 Rubble Mound Armour

A rubble mound structure could be built from shore comprising rock fill and rock armour. Armour size for preliminary design has been determined using the Hudson formula as documented in Table 5.

Table 5. Rock Armour Design Parameters and Solution for Groyne Head using the Hudson Equation

<table>
<thead>
<tr>
<th>Inputs</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Unit rock</td>
<td>rock</td>
</tr>
<tr>
<td>Unit Wt Sea Water</td>
<td>1,025.00</td>
</tr>
<tr>
<td>Unit Wt Armour Stone</td>
<td>2,650 kg/cu m rough angular</td>
</tr>
<tr>
<td>Design Wave H$<em>t$ (H$</em>{10}$)</td>
<td>2.50 m breaking</td>
</tr>
<tr>
<td>Stability Coefficient (Kd)</td>
<td>0.8</td>
</tr>
<tr>
<td>Factor of safety = 2</td>
<td></td>
</tr>
<tr>
<td>Layer Coefficient</td>
<td>1.00</td>
</tr>
<tr>
<td>Factor of safety = 2</td>
<td></td>
</tr>
<tr>
<td>Porosity</td>
<td>47%</td>
</tr>
<tr>
<td>Factor of safety = 2</td>
<td></td>
</tr>
<tr>
<td>Cotan Structure Slope</td>
<td>2.00</td>
</tr>
<tr>
<td>Factor of safety = 2</td>
<td></td>
</tr>
<tr>
<td>No. Units Comprising Layer</td>
<td>2</td>
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<tr>
<td>Cover Layer</td>
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</tr>
<tr>
<td>$D_{min}$</td>
<td>1.41 m</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>1.55 m</td>
</tr>
<tr>
<td>$D_{max}$</td>
<td>1.67 m</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>1.35</td>
</tr>
<tr>
<td>Layer Thickness</td>
<td>2.70 m</td>
</tr>
<tr>
<td>Minimum Crest Width</td>
<td>4.04 m</td>
</tr>
<tr>
<td>No. Units per Surface Area</td>
<td>0.69</td>
</tr>
<tr>
<td>$W_{min}$</td>
<td>4,871 kg</td>
</tr>
<tr>
<td>$W_{50}$</td>
<td>6,495 kg</td>
</tr>
<tr>
<td>$W_{max}$</td>
<td>8,118 kg</td>
</tr>
</tbody>
</table>

For the head of the rubble mound, the armour would comprise 6,500 kg rock with $D_{50}$ = 1,500 mm. The crest width would need to be 5 m to allow for truck access. Total volume of rock would be 8,000 m$^3$. 
6. Rock Revetment

6.1 Introduction

This option replaces the existing sand bag revetment with a rock revetment (Figure 32). The structure aims to prevent any further recession of the dune face.

![Figure 32. Schematic cross-section of a terminal rock revetment.](image)

6.2 Design Parameters

The design parameters are as follows:
- Crest level = 4.5 m AHD; Toe level = −1.0 m AHD; Storm water level = 1 m AHD
- Seabed level at toe = 0 m AHD
- Seabed slope 1V:16H
- Maximum breaking wave height $H_b = 2.5$ m; Wave period = 15 s
- Water depth at break = 2 m
- Wavelength at break = 67 m
- Rock mass = 2,650 kg/m$^3$
- Side slopes for rock fill 1V:2H (factor of safety of 2 for geotechnical the stability of the sand dune)

6.3 Structural Design

6.3.1 Armour and Underlayer Size

The requisite rock armour size is in Table 6. “Special Placement” has been invoked, meaning that the armour stones will need to be tightly packed with a reduced porosity of 27%. This can be achieved easily at this site because the revetment will be constructed for the most part above water level “in the dry”. The advantages of “Special Placement” include greater stability with smaller rock.
Table 6. Requisite Rock Armour Size for Rock Revetment.

<table>
<thead>
<tr>
<th>Type of Unit rock</th>
<th>Unit Vt (cu m)</th>
<th>Unit Vt Armour Stone</th>
<th>Design Wave H (m)</th>
<th>Stability Coefficient (KD)</th>
<th>Layer Coefficient</th>
<th>Porosity</th>
<th>Factor of safety</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2.6</td>
<td>2.50</td>
<td>2.6</td>
<td>1.00</td>
<td>37%</td>
<td>2</td>
</tr>
</tbody>
</table>

**Inputs**

- **Slope length** = 11.3 m
- **Crest length and toe length** = 6*0.65 m = 3.9 m
- **Armour layer thickness** = 1.8 m
- **Length of revetment** = 150 m
- **Volume of armour** = 150*15.2*1.8 = 4,100 m³
- **Weight of armour rock** = (1 - 0.27)*4,100*2.65 = 8,000 t
- **Volume of under-layer rock** = 150*15.2*0.5 = 1,140 m³
- **Wt underlayer rock** = 1,140*1.8 = 2,000 t
- **Area GTX 1200R** = 15.2*150 = 2,300 m²

### 6.3.2 Quantities

- **Slope length** = 11.3 m
- **Crest length and toe length** = 6*0.65 m = 3.9 m
- **Armour layer thickness** = 1.8 m
- **Length of revetment** = 150 m
- **Volume of armour** = 150*15.2*1.8 = 4,100 m³
- **Weight of armour rock** = (1 - 0.27)*4,100*2.65 = 8,000 t
- **Volume of under-layer rock** = 150*15.2*0.5 = 1,140 m³
- **Wt underlayer rock** = 1,140*1.8 = 2,000 t
- **Area GTX 1200R** = 15.2*150 = 2,300 m²

### 6.3.3 Placement

An example of “Special Placement” is shown in Figure 33.
Figure 33. Cabbage Tree Harbour revetment, Wyong Shire, NSW with “Special Placement” rock armour (photo credit A F Nielsen, 27/07/2011).
7. Beach Nourishment

This option is depicted schematically in Figure 34.

Figure 34. Schematic representation of beach nourishment.

To widen the beach by 20 m, the volume of sand required would be 60,000 m$^3$ (500 × 6 × 20). The amount of sand to cater for the long term erosion rate for, say, 6 years is 180,000 m$^3$. The total volume of sand required would be in the order of 240,000 m$^3$, which would widen the beach by some 80 m. A sand source is indicated in Figure 36, Appendix BB. Nourishment of 180,000 m$^3$ would need to be undertaken periodically every 6 years.

With such volumes of sand nourishment material, some back beach foredune construction would be undertaken with dune planting. The area of foredune planting is estimated at 10,000 m$^2$ (20 m × 500 m)
8. Discussion

In Table 7, each option is assessed for its effectiveness qualitatively in respect of its impacts on the littoral processes causing erosion at Portsea Front Beach.

Table 7. Assessment of the Effectiveness of Each Option in respect of Treating the Coastal Erosion Processes.

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>Configuration Dredging Nearshore Trench</td>
<td>Reduces considerably the cross-shore transport of beach sand and dune erosion but allows still some wave energy onto the beach face</td>
<td>Reduces considerably the alongshore transport rate of beach sand away from the eroded area arresting erosion, but allows some wave energy onto the beach to maintain sand supply to the east</td>
<td>Does not significantly increase wave energy on the western shore of Weeroona Bay but balances the transport rate with the rate of supply of littoral drift to the eroded area</td>
</tr>
<tr>
<td>Configuration Dredging Offshore Trench</td>
<td>Reduces considerably the cross-shore transport of beach sand but allows still some wave energy onto the beach face</td>
<td>Reduces considerably the longshore transport rate of beach sand away from the eroded area, arresting erosion, but allows some wave energy onto the beach to maintain sand supply to the east</td>
<td>Increases wave energy on the western shore of Weeroona Bay, thereby increasing the supply of littoral drift to the eroded area.</td>
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<tr>
<td>Detached Breakwater</td>
<td>Reduces considerably the cross-shore transport of beach sand but allows still some wave energy onto the beach face</td>
<td>Reduces considerably the longshore transport of beach sand away from the eroded area but allows some sand transport to the east</td>
<td>Does not change wave energy levels on the western shore of Weeroona Bay nor increase the supply of littoral drift to the eroded area.</td>
</tr>
<tr>
<td>Attached Breakwater</td>
<td>Reduces to zero the cross-shore transport of sand from the eroded area</td>
<td>Reduces to zero the longshore transport of sand away from the eroded area</td>
<td>Prevents littoral drift transport from the western beach of Weeroona Bay to the beaches to the east, requiring bypassing</td>
</tr>
<tr>
<td>Groyne</td>
<td>Has no impact on the cross-shore transport of beach sand</td>
<td>Reduces but does not eliminate the longshore transport of beach sand away from the eroded area</td>
<td>Has no impact on the rate of littoral drift transport from the western beach of Weeroona Bay to the beaches to the east</td>
</tr>
<tr>
<td>Revetment</td>
<td>Has no impact on the cross-shore transport of beach sand</td>
<td>Has no impact on the longshore transport of beach sand away from the eroded area</td>
<td>Has no impact on the rate of littoral drift transport from the western beach of Weeroona Bay to the beaches to the east</td>
</tr>
<tr>
<td>Sand Nourishment</td>
<td>Has no impact on the cross-shore transport of beach sand</td>
<td>Has no impact on the longshore transport of beach sand away from the eroded area</td>
<td>Has no impact on the rate of littoral drift transport from the western beach of Weeroona Bay to the beaches to the east</td>
</tr>
</tbody>
</table>
9. References


Pope, J. & Dean, J.L. 1986. Development of design criteria for segmented breakwaters, Proceedings, 20th International Conference on Coastal Engineering, November 9-14, Taipei, Taiwan, American Society of Civil Engineers, pp 2144-2158

