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Designing a sustainable beach replenishment scheme for a site in Malta

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DESIGNING A SUSTAINABLE BEACH REPLENISHMENT SCHEME FOR A SITE IN MALTA

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Abstract
The Malta Tourism Authority is planning to undertake environmentally sound beach replenishment along a rocky stretch of coastline immediately south of Qawra Point, Salina Bay. Refinement and confirmation of an initial beach layout through numerical modelling was undertaken to support Environmental Impact Studies required for the development permit for the creation of a recreational beach. There is no offshore sand available for replenishment in Malta. Instead, sand must be crushed from rock originating from an overseas quarry; local limestone is too friable. This reduces the risk of biological contamination and enables the granulometric characteristics of the sediment to be designed to suit the wave conditions and meet the design specifications. The proposed design life for the artificial replenishment, before major replenishment becomes necessary (i.e. when the cumulative dry beach area loss reaches 30%), was set at 10 years. It was recognised that hard modifications to the existing coastline, i.e. control structures, may be required to stabilise the replenishment and prevent migration of the sand however, such modifications were to be kept to a minimum and not compromise the marine environment. Important features of the site included a tidal rock pool (overtopped under certain conditions) along the spit linking Qawra Point to the mainland and the location of nearshore *Posidonia oceanica* meadows (an Annex I priority habitat under the European Union Habitats Directive) restricting the extent of the beach. Wave modelling was undertaken to establish extreme and morphologically representative wave conditions. The results were used to assess the typical beach plan shape which was found to be quite stable and not subject to large variations. The stable beach profile was assessed to provide an indication of the overall footprint of the beach within the bay and cross-shore sediment transport modelling was used to determine beach draw-down during storm events. Modelling studies showed that it may be possible to create a beach at the site. However, there are several factors for consideration such as the offshore extent of the beach toe, thought to place some risk on the nearshore seagrass, and overtopping from the tidal pool. Mitigation methods suggested included the adoption of coarser, narrowly graded material as well as the use of beach retaining structures. A 3D mobile bed physical model was recommended to further refine the scheme.

1 Introduction

The Malta Tourism Authority (MTA) is planning to undertake environmentally sound beach replenishment along a rocky stretch of coastline immediately south of Qawra Point, Salina Bay, Malta (Figure 1). The proposed beach initially developed by MTA was envisaged to be 15 metres wide at the centre of the bay, tapering away at the edges. The proposed design life for the beach replenishment, before major replenishment becomes necessary (i.e. when the cumulative dry beach area loss reaches 30%), was set at 10 years. In Malta, there are no significant offshore sources of sand available. Instead, sand must be crushed from rock originating from an overseas quarry; local limestone is too friable. The upper bound (d_{max}) is limited to 4mm for amenity reasons and the lower bound (d_{min}) is limited to around 50-60 μm to prevent the shipping of dust at great expense.

The refinement and confirmation of the concept beach design was undertaken using a suite of numerical models combined with the examination of other beach replenishments in the area and the outputs of a site investigation. Special consideration was taken of any potential offshore losses of the imported beach material due to the presence of environmentally important seagrass offshore.

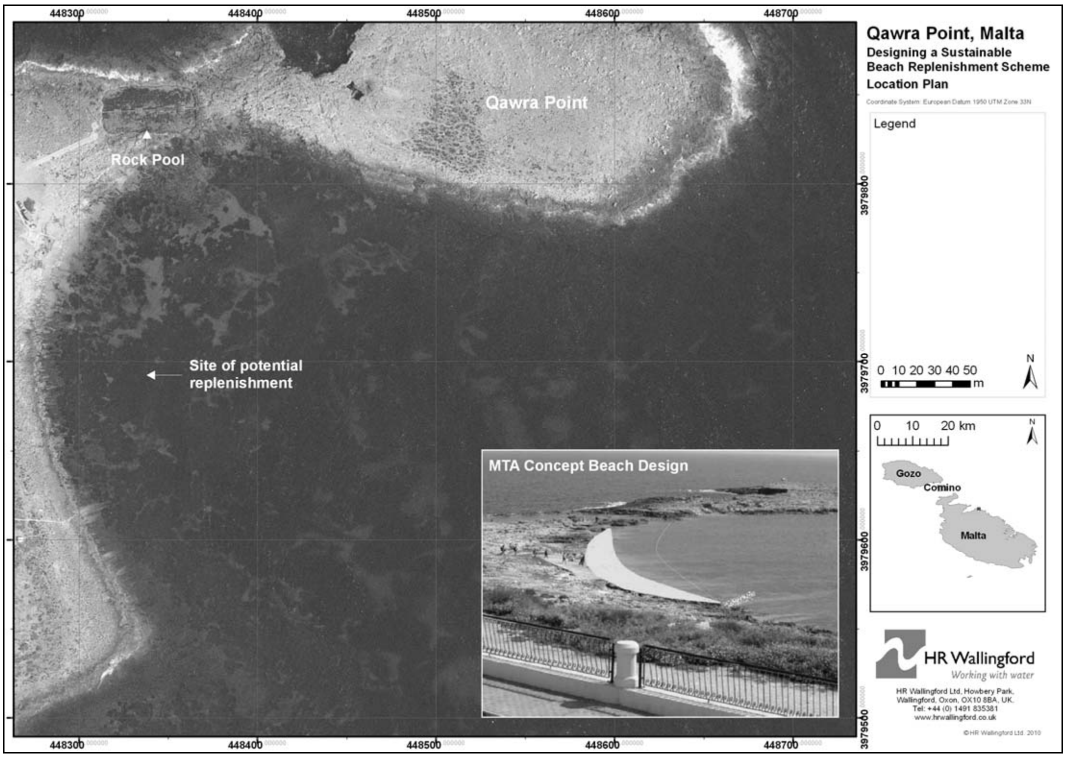

Figure 1 Location of the site

1.1 Site description

The proposed location for the beach is a rocky shoreline to the immediate south of Qawra Point (Figure 1). The site is composed of limestone and old coraline crags; there is very little beach but some coarse sand, shingle and cobbles washed up on the rocks. Typically, the bed is rocky with very sparse sand pockets. Offshore, benthic assemblages include a fringe of photophilic algae growing on the rocks below mean sea level, which quickly give way to a biocoenosis of infralittoral algae intermixed with patches of stones and pebbles, sands and gravel, and stands of the Lesser Neptune Grass. This seagrass gradually grades into an association with *Posidonia oceanica*, which eventually forms extensive meadows interspersed with patches of sands and gravel, some 50m offshore. *Posidonia oceanica* is a flowering plant that has evolved to grow in a completely marine environment. It is endemic to the Mediterranean, where it forms extensive meadows over large expanses of the basin. *Posidonia oceanica* is a very important marine ecosystem, functioning as a nursery, fishing, and recruitment ground for several species of invertebrates (molluscs, echinoderms, crustaceans, etc) as well as fish. The plant, which grows on a dense matrix of rhizomes called a "matte", is also an important part of the beach dynamics on the island as it helps buffer incoming waves while also trapping sand moving offshore. *Posidonia oceanica* is an Annex I priority habitat under the European Union Habitats Directive.

The present coastline is particularly exposed to storms from the north to east-southeast sectors. The largest wave conditions occur from the northeast (*gregale*) due to wave diffraction around Qawra Point.

A further important feature of the site is the tidal rock pool along the spit, linking Qawra Point to the mainland. Under certain conditions water is overtopped into the rock pool, generating strong flows over the reef and towards the southeast into the proposed beach location. This could, potentially, cause some additional erosion of any placed beach material. In addition it will be important to ensure that sand from the proposed beach is not washed into the tidal pool, thus compromising the marine environment.

2 Review of St George's Bay Beach Replenishment Project

A brief review of the St George's Bay beach replenishment project was undertaken to determine how the nourished beach has responded, having been *in-situ* for several years. An initial replenishment using medium to coarse sand occurred in May-June 2004 and further beach replenishment works to enlarge the beach were carried out shortly after June 2007.

Whilst the site at St George's Bay is considerably different to the proposed replenishment site at Qawra Point, similar crushed material is proposed for the nourishment of Qawra beach. Particular attention was paid to the slope and stability of the nourished beach and in particular the extent of the 'toe' (i.e. where the beach intersects the existing seabed).

A review of survey data indicated that the steeper more active part of the beach, between +0.5m and -1m, has steepened to a slope of around 1:7, compared to an initial slope of 1:10. This is of consequence to Qawra Point, as a steep slope will be required to help prevent the toe of the beach extending into the seagrass meadows.

3 Wave Modelling

Mathematical models (TELURAY and ARTEMIS) were run to assess the wave climate at the entrance of the bay, within the bay with particular reference to diffraction of waves around Qawra Point, and to the north of Qawra Point where wave overtopping effects cause flow through the tidal pool. The largest waves reaching the site under the 1 in 1 year return period conditions were from the 20ºN sector, which refract and diffract around the headland and are incident at the site from the east. Within the 40ºN and 60ºN range there is significant focussing of wave energy by the shoal to the Northwest of Qawra Point into the bay, and an interference pattern associated with it to the west.

A number of extreme and morphologically representative conditions were also run in the model to determine how the wave crests enter the site under a range of wave conditions (Figure 2). The morphological beach alignment (annual averaged) is governed by morphologically averaged wave conditions impacting on the coast. Hence, whilst extreme waves will cause occasional adjustments, it is necessary to understand how the beach responds to more frequent but less energetic events. Furthermore, the beach alignment is expected to be more sensitive to the incident wave angles than to changes in wave height.

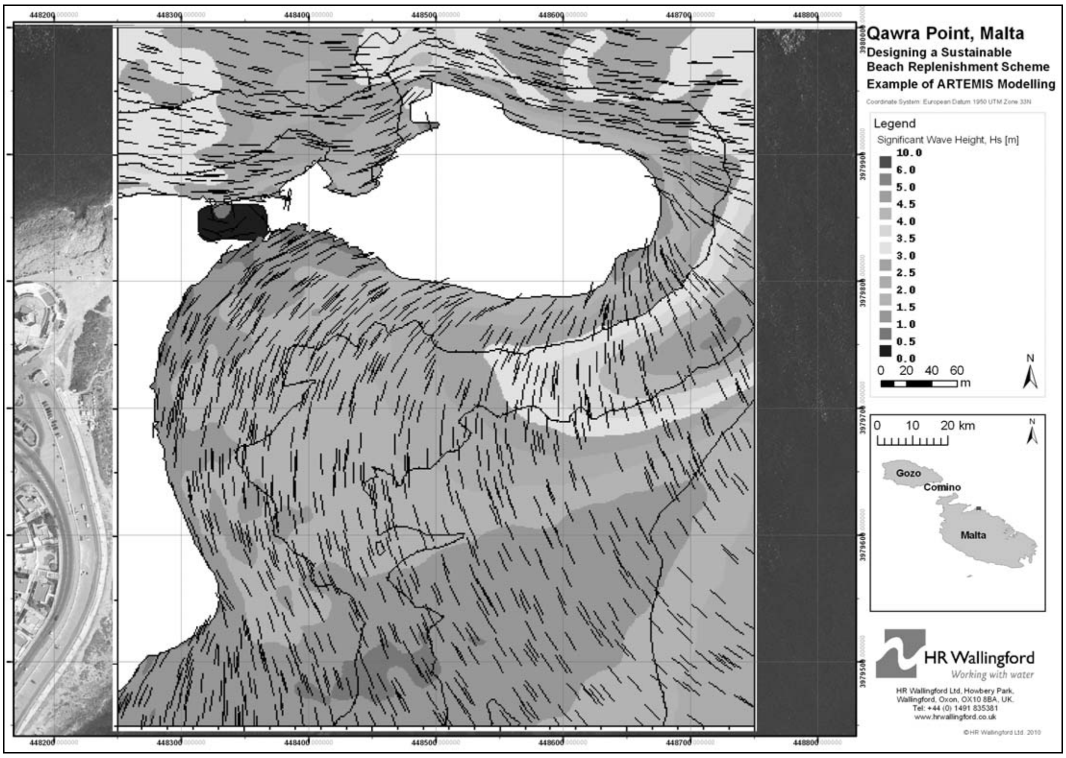

Figure 2 Example of ARTEMIS wave modelling from 20°N shaded contours of Hs, contours of bathymetry contour lines, and wave direction represented as mean orientation of wave crests

3.1 Concept beach design

Having completed the wave modelling, attention was turned to the concept design of Qawra beach. Bathymetric and topographic datasets, provided by the Client, were reviewed and used to produce a DEM (digital elevation model) which is shown in Figure 3.

The following boundary constraints on the extent and excursion of the beach were identified:

- The northern limit is the tidal rock pool.
- The southern limit has been taken to be the southern promontory.
- The landward limit is not as constrained, if sand spills over onto the rocks this is not considered to be of great concern. Consideration could be given to forming a perched beach/reclamation on the rocks above the dynamic beach level. This part of any proposed works was, however, beyond the scope of the study.
- The seaward limit was identified following the completion of a Marine Benthic Survey, undertaken by Ecoserv Ltd. (Borg, 2006). The survey identified different habitat types; the beach should not extend beyond the zone of mixed and sparse grasses into the seagrass (*Posidonia oceanica)* meadows; this limit of beach excursion is marked by a dashed line in Figure 3.

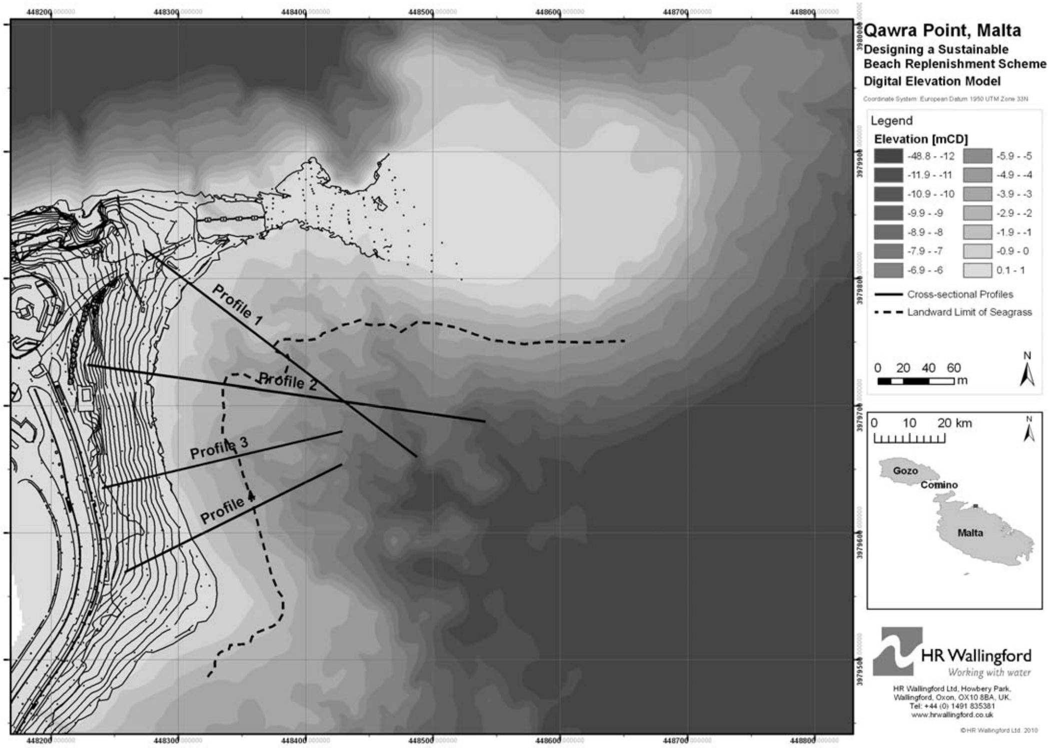

Figure 3 Digital Elevation Model

4 Evaluation of Beach Plan Shape

The alignment of the wave crests from the wave modelling results (figure 2), were used to establish the initial indication of the natural beach alignment that the beach will tend to adopt. This alignment will be different for different incident wave conditions, thus yielding a range of possible beach alignments. As the wave alignment is largely dictated by the incident wave condition and diffraction effects, the wave results were only applied to the pre-developed case (i.e. existing layout) to understand the alignment the beach would tend to adopt.

The movement of beach sediment along the coastline, i.e. parallel to the beach contours, is termed “longshore drift”, and this is predominantly caused by waves breaking at an angle to the beach contours. Longshore drift causes change in the plan-shape of beaches, i.e. increasing the beach width at some points (beach accretion) and reducing it elsewhere (beach erosion).

Longshore drift takes place in shallow water, i.e. where the water depths are less than twice the height of the breaking waves. Where barriers such as rocky headlands jut out into deeper water than this, they effectively halt the longshore drift, retaining sediment in the embayments between them, forming so-called pocket bay beaches. In a pocket bay, the beaches tend to align themselves parallel to the crests of the incoming waves, i.e. becoming “swash aligned” and minimising the alongshore transport of sand as the waves are breaking normal to the beach slope. However, as the direction of the incoming waves changes, these beaches will also alter their alignment in response. These changes in beach “plan shape” are achieved by the movement of sand along the beaches, i.e. with retreat of the beach contours at one end and advance at the other end of the bays, and do not need to involve any change in the overall volume of sand within a bay. It was important to understand the extent of this variability to accommodate it in the design of the beach and any control structures deemed necessary. The wave modelling indicated that the wave crests are very similar irrespective of the incident wave direction. This suggests that the plan shape would be quite stable and not subject to large variations.

Wave set-up is another important phenomenon in the surf-zone hydrodynamics. This is a local elevation in the mean water level on the foreshore, caused by the reduction in wave height through the surf-zone. The wave set-up is related to the wave height at breaking. Gradients in wave set-up, in partly sheltered areas for example, can generate local circulation in the surf zone towards the sheltered area. This is important as it can cause secondary flows offshore with potential to transport sediment offshore. Examination of the wave results indicated that wave heights are relatively uniform within the bay. This is primarily due to the way the waves are transformed (i.e. diffracted/refracted) around the point. Therefore, whilst secondary currents are still expected, they can be managed with well designed control structures.

In some locations, tidal currents are strong enough to significantly modify the longshore drift caused by obliquely breaking waves. At Qawra Point, however, the tidal currents are expected to be weak, even offshore from the beaches, and will therefore not significantly affect the longshore sediment transport.

5 Beach Profile

Having considered the beach plan shape, the footprint of the beach in terms of the underwater extent, i.e. between MHWS (mean high water springs) and the beach toe where the profile intersects the bedrock was examined. Important aspects of the beach profile considered were the closure depth, natural slope of the existing beach profile, lateral extent of the 'dry profile', wave set-up and run-up and the geometric constraint imposed by the rock upper shore and cliffs.

5.1 Natural Beach Profile at the Site

Four cross-sectional profiles were extracted from the DEM and are plotted in Figure 4 (in the plot the 0m chainage is taken to be location of existing shoreline at level of MHWS). Data between chainages -5m and 10m were excluded due to the sparse data points along the line of the coastline. Each of the four profiles intercepted the seagrass, approximately 50m seaward of the coastline, at depths of -2.2mCD to -3.6mCD. The four profiles exhibited slopes of 1:10 to 1:20 between chainages of +10m and +50m.

On the premise that the proposed beach cannot encroach onto the seagrass, the toe of the beach was designed to be situated landward of this limit. In order to achieve this, it was necessary to compromise on the width of the beach depending upon the natural stable slope at which the sediment can be placed and remain stable. In general, the larger the sediment size the steeper the slope at which the beach can be placed.

5.2 Closure Depth

An initial estimate of the closure depth, defined as the depth beyond which no significant longshore or cross-shore sediment transport takes place due to littoral transport processes, was calculated according to Hallermeier (1981):

$$D_l = 2.28 H_{s,12\,h/g} - 68.5 \frac{H^2_{s,12\,h/g}}{g T_s} \tag{1}$$

where: D_l is the depth of closure; $H_{s,12h/g}$ is the nearshore wave height (approximately 1:1 year return period); and T is the associated wave period.

The closure depth is important as it provides an indication of the depth at which transport of mobile bed material becomes limited. If sufficient mobile bed material were present at a depth where the important habitats are found, this material would be expected to be transported and deposited within the area of the important marine habitats. Using the Hallermeier formula and the incident wave data at the -10m contour (just outside the wave breaking zone) obtained from the wave modelling, the closure depth was estimated to be around -5.2mCD. It should be noted that, the Hallermeier formula does not consider grain size, and a bespoke coarse grain size may be required to achieve a suitable steep slope; coarser sized sediment will require more ‘energy’ before it becomes mobile. Due to the complex bathymetry affecting the way in which the waves diffract and refract inshore, the closure depth calculated is considered to be conservative and a change in wave height by 0.5m would result in a change in closure depth of 1.0m.

5.3 Wave Run-up

The wave run-up level was used to determine the level of the beach crest. A calculation of wave run-up was undertaken using a modified equation for sand beaches developed by Nielson and Hanslow (1991). The run-up height exceeded by 2% of the run-up events is denoted $R_{2\%}$ and can be obtained from:

$$R_{2\%} = 0.36 g^{1/2} \tan \beta H_s^{1/2} T$$

where: g is gravity; $\tan\beta$ is the slope of the beach in degrees; H_s is the wave height; and T is the wave period.

The calculation of wave run-up used the offshore significant wave height and an average bed slope into deeper water. However, from previous experience, using this wave condition can significantly over-estimate the wave run-up. Instead it is more appropriate to use the nearshore wave condition, with a bed slope corresponding more to the nearshore region. A 1:100 year significant wave height (from 20°N) of 2.0m was selected with an associated period of 11.5s (the depth was approximately -4m) and a slope of 1:10, i.e. the likely slope of the nourished beach, was used. The porosity of the beach material can also have an effect on the wave run-up on a beach and the Nielson and Hanslow (1991) equation does not take beach porosity into account. For this study, it was estimated that the beach crest height will typically be around 2.5mCD (2m above MHWS).

5.4 Replenished Profiles

Sand for the replenishment must be crushed from rock originating from an overseas quarry. Hence the granulometric characteristics of the sediment need to be designed to suit the wave conditions and meet the design specifications. Coarse sand with very little fine-grained sediment filling the interstices, i.e. well-sorted sediment, usually results in the beach being very permeable and consequently has a very steep beach profile, with a gradient more typical of a gravel beach than a sand beach. The crest height for the proposed nourished beach was taken to be 2.5mCD.

Beach profiles were estimated using the Dean formula (Dean, 1987), for a range of sediment sizes, where the slope of the beach is a factor of the grain size and reduces in the offshore direction. The MHWS level was taken as the still water level in the calculation of the Dean Profile. To create the required recreational beach width it was assumed that the MHWS contour will be advanced seaward by 15m. Below MHWS the profiles then fit the Dean Profile for the given grain size, which flatten off as they extend offshore and intersect with the various existing bed profiles (Figure 4). The finest sediment (d_{50} = 0.5mm) would not be suitable for the nourishment as the sediment would encroach on the seagrass at all locations within the bay.

Whilst the 4mm material would be more suitable (as it intercepts the existing seabed landward of the seagrass) the 2mm material was preferred, as it provided a better quality amenity beach and it only encroached into the seagrass meadows at some locations within the bay.

An alternative solution was therefore sought, and a recreational beach width of 15m was created without advancing the MHWS seaward (Figure 5). This was achieved through the utilisation of the existing rocky backshore to provide the required beach width. This would allow the 2mm sediment to be used with minimum encroachment onto the seagrass offshore. Furthermore it was noted from the examination of the beach data from St Georges Bay that the granultic crushed material used, stands at a steeper slope than suggested by Deans Profile.

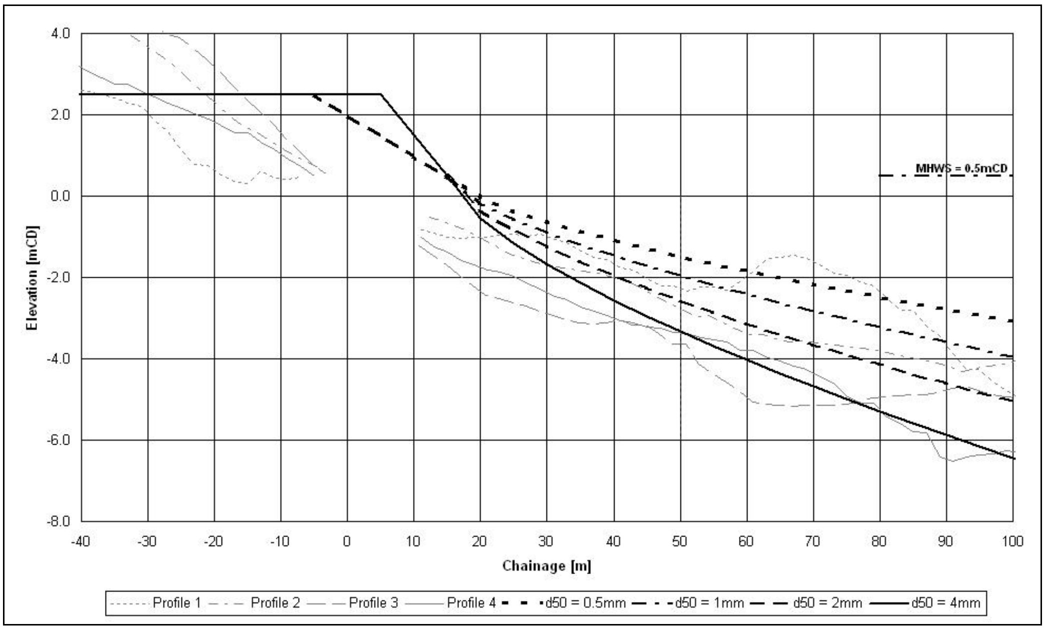

Figure 4 Natural and replenished cross-sectional profiles (MHWS line advanced by 15m)

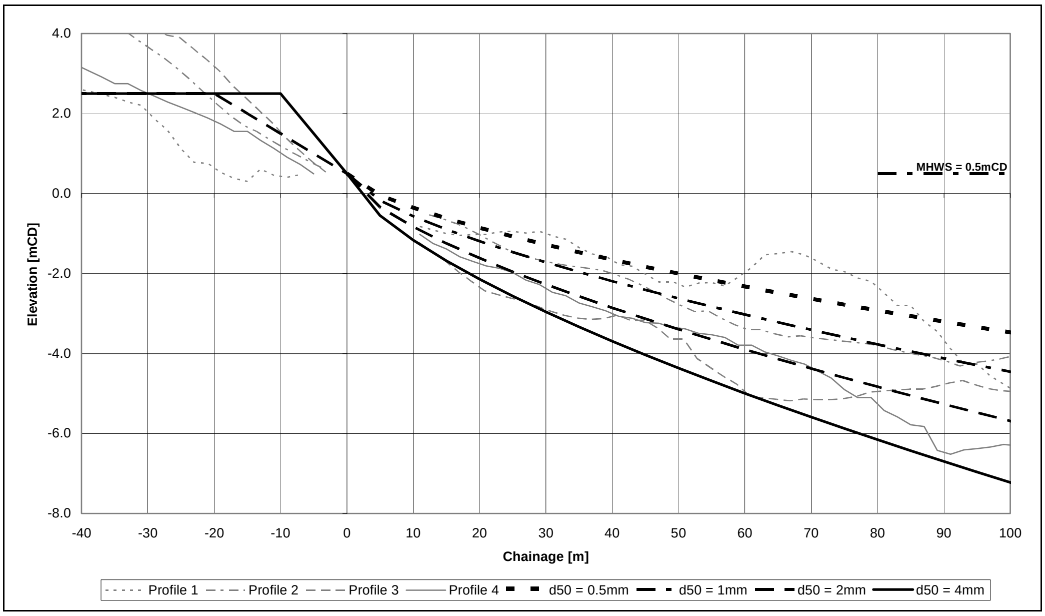

Figure 5 Natural and replenished cross-sectional profiles (MHWS line advanced by 0m)

6 Cross-shore Sediment Transport

Cross-shore sediment transport principally causes changes in the beach profile, i.e. making it steeper or flatter. It is usual for beach sediment to move offshore in stormy weather and then return from the nearshore seabed to the beaches in calmer conditions. These cross-shore interchanges are typically important in the short-term, i.e. over days or months. Such onshore-offshore transport of sediment rarely results in any significant long-term changes in beach volume, i.e. over several years. However, if there is a net long-term offshore transport, the beach will erode, i.e. recede landwards, and sediment is likely to be deposited upon the seagrass.

Beach profile changes, in response to short-lived but severe storm wave action, were predicted using the COSMOS and SHINGLE beach profile models. The results were used to determine whether the imported beach material will be transported offshore into the seagrass area.

The sediment size of 4mm (that could be used for the beach renourishment) is unusual, being intermediate between shingle and sand beaches. The sediment grain size is greater than most sand beaches and little is known about the porosity of the sediment to be used for the nourishment. The permeability of the beach will also play an important part in the slope at which the beach can be placed and its stability. Whilst, it may be possible to engineer the crushed granite sediment so that only the larger sized particles remain i.e. sieve out the fines to ensure that the sample is well sorted there is no guarantee that the existing finer sediment will not become incorporated into the recharge material and thereby flattening the slope of the beach profile in the longer term.

Because of the uncommon grain size required for the beach recharge, two computational modelling methods were used to examine how the beaches may change in severe storm conditions. COSMOS was used to model the short-term cross-shore response of a sand beach during a storm, and SHINGLE was used to predict the potential reaction of shingle beaches to storms.

Due to the lack of relevant validation of both models for the 4mm sized sediment, the model results were compared and the information was used to provide an indication of how the beach may respond to storm events and whether the sediment may be transported offshore into the area of the seagrass.

6.1 Description of COSMOS Modelling Results

The model was run for a range of sediment sizes (1mm, 2mm and 4mm) for storm events with return periods of 1, 5, 10, 20, 50 and 100 years for a duration of 12 hours. Profile 2 (Figure 3) was chosen to be representative of the underlying bedrock primarily because of its proximity to the seagrass (55m between the shoreline and the seagrass), it is also located at one of the widest parts of the beach and has a profile that is intermediate to Profiles 1, 3 and 4 in terms of depth. The current shoreline is located at 0m chainage. Incident wave conditions were obtained from the wave modelling.

For the 2mm results with the advanced 15m beach, sediment is deposited seaward of the beach limit, i.e. within the seagrass meadows under all conditions. For the 1:100 year storm event, a bar type feature is formed between 100m and 140m chainage with a height of up to 1m. This amount of sediment is unlikely to be environmentally acceptable. For the 1:1 year storm event, sediment is deposited no further than 105m at a thickness of no more than 0.5m. Despite the 4mm sediment size being outside the valid range for this model, the results were useful in providing information on how a steeper profile responds; the sediment was still transported offshore onto the seagrass meadows but to not such a great extent.

Sensitivity testing to smaller wave heights was also undertaken, as it was recognised that COSMOS does not account for the continued diffraction that is known to occur within the bay, thus predicting a larger transport of sand offshore. A smaller wave height, extracted from the ARTEMIS wave modelling, resulted in the 2mm sediment only being transported 75m offshore (i.e.20m into the seagrass meadows). Therefore if the beach was not advanced by 15m then there would be minimal encroachment of sediment onto the seagrass meadows, although further testing is recommended to confirm this.

Finally, the model was run with a submerged sill/berm (for a 2mm sediment size with the advanced 15m beach) to determine if such a structure could be used to reduce the amount of offshore transport onto the seagrass. The results indicated that, for a 1:100 year storm, the sill had a considerable effect on reducing offshore transport of sediment onto the seagrass. The berm reduced the depth of material deposited onto the seagrass by 80%.

6.2 Description of SHINGLE Modelling Results

The SHINGLE model was run for a 4mm profile. Coarse sediment is likely to be eroded from the upper part of the profile and ‘pushed’ up onto the beach. It should be noted that the scenario for which this model was run ‘pushes’ the model to the limit of/beyond its capabilities as a result of the physical processes involved. However, it does suggest that sediment is unlikely to be transported offshore and deposited onto the seagrass. It is feasible that relatively long period waves could push the profile above the proposed beach crest of 2.5mCD during a storm.

7 Requirement for Beach Control Structures

The need for a terminal groyne or alternative beach retaining structure was assessed, guided by the modelling results, the determinants being the (virtual) extent of the bay, and the beach profile at the point of intersection with the control structure. Apart from the functional requirements, engineering feasibility and visual impact were also taken into account in the conceptual design and advice regarding any proposed structure was also provided.

The results of the wave agitation modelling indicated that the beach would naturally align itself across the tidal pool. However due to the strong current flowing out of the tidal pool, known to flow towards the south-east, the beach is unlikely to remain in place at this location. Therefore in order keep the beach stable and not compromise the marine environment inside the rock pool through inundation of sediment, or transport of beach sediment offshore onto the areas of seagrass, the use of a groyne/training wall just to the south-west of pool that was recommended (Figure 6).

It is also possible that there will be a requirement for a southern groyne/training wall to prevent any material from being transported around the southern promontory and lost from the system. The actual requirement for this structure is difficult to discern as the wave agitation modelling shows this end of the beach would be relatively stable. The concern however, is that wave setup currents may transport the material east, away from the site where it is then moved out of the beach system. A structure at this end of the beach could help to prevent this, if this is found to be a notable problem. This structure, could be located just to the south of the intersection with the MHWS contour of the proposed replenished beach and the existing shoreline (Figure 6); it could also be used as a recreational platform. It appears that there is also an area of higher ground which should help to naturally curtail the beach as well as prevent the structure from needing to be constructed in deeper water which could lead to cost savings.

Given the results of the cross-shore sediment transport modelling, the proximity of the beach toe to the seagrass (especially for the finer 2mm material) and the complex nature of the site, it was recommended that the potential additional performance offered by a berm/sill should be further investigated at the detailed design stage, preferably through the use of a physical model.

Using a coarse sized sediment would likely reduce the need for/extent of the southern groyne as well as potentially reducing the need for the berm to contain the sand and prevent it from being transported in to the seagrass meadows, as a coarser sediment will sit at a steeper slope and is also less mobile.

However, the northern structure will still be needed for two reasons: (1) to provide a barrier to the strong currents which flow from the rock pool and will most likely wash away the beach compromising its stability; and (2) to prevent beach material from being transported into the tidal pool compromising the marine environment.

The lengths of the structures, indicated in Figure 6, are based on the best estimate of the likely length required using 2mm material with minimal fines and without the beach being advanced by 15m. It was also anticipated that the height of the structure will need to be around 0.5-1m above the height of the nourished beach profile. Whilst a 4mm sized sediment will sit at a steeper slope, it is not recommended to reduce the length or height of the structures without further testing and optimising in a physical model.

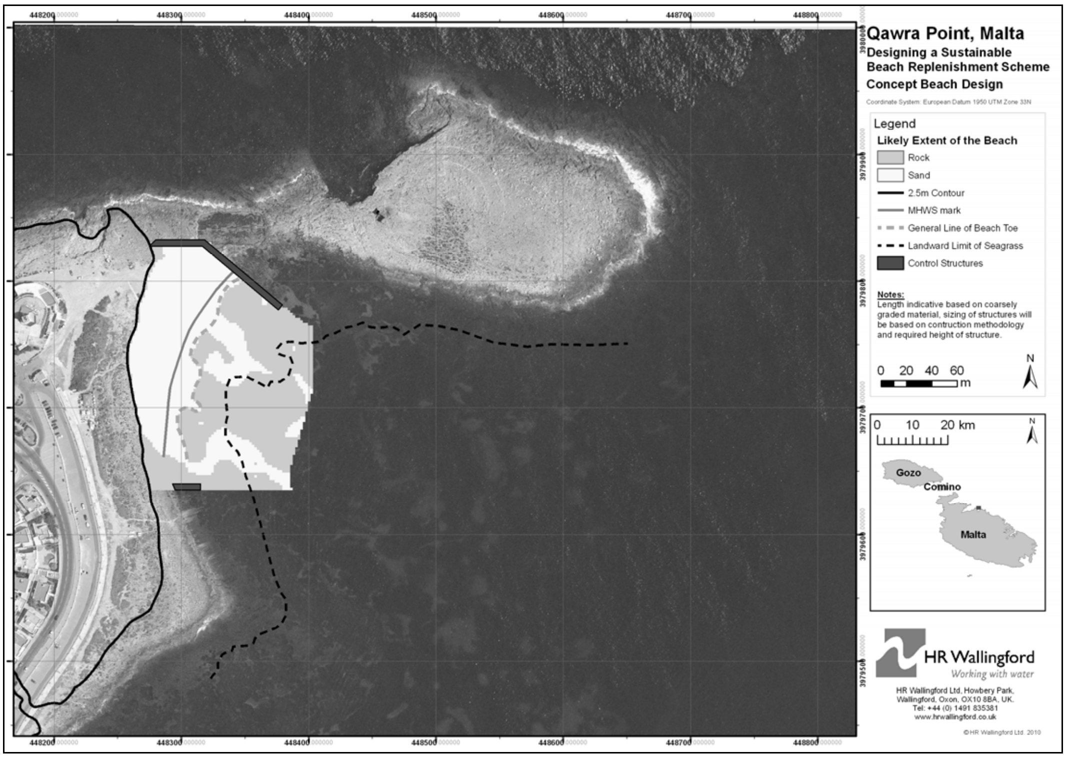

Figure 6 Concept Beach Design (based on a 2mm sediment size and a MHWS advanced by 0m)

8 Conclusions and Recommendations

Wave agitation modelling was undertaken to assess the typical beach plan shape and its variability, working on the premise that the beach will tend to align itself with the incoming wave crests. The results of the modelling indicate that the beach would naturally align itself across the tidal pool. However, due to the strong current flowing out of the tidal pool, known to flow towards the south-east, the beach is unlikely to remain in place at this location. Therefore, in order keep the beach stable and not compromise the marine environment inside the rock pool through inundation of sediment, and to prevent transport of beach sediment offshore into the areas of seagrass, the use of a groyne/training wall just to the south-west of pool was recommended. A structure might also be required at the southern end of the beach to prevent circulation driven by wave set-up from transporting material too far out to the east.

An assessment of the stable beach profile was undertaken to provide an indication of the overall footprint of the beach within the bay and hence it's maximum seaward excursion. Cross-shore sediment transport models were used to determine the draw-down (i.e. the likelihood of beach material being transported offshore) of the beach during severe storm conditions. The results of the modelling indicated that a draw down of sediment onto the seagrass is likely. However the potential for this can be reduced by adopting a berm/sill and by utilising the rocky backshore to create the beach instead of advancing the MHWS contour by 15m.

The development of a beach at this location is considered to place some risk to the nearshore seagrass. Possible methods to mitigate this have been investigated, including the adoption of coarser material, or limiting the proportion of fines in the nourished material and consideration of a berm /sill.

If a beach is to be adopted, it is recommended that a 3D mobile bed physical model be used for the detailed design. This modelling approach would help refine the scheme with a better level of confidence on how the recommended structures and the beach imported beach material will behave.

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References

Adi Associates Environmental Consultants Ltd., 2006, Proposed Upgrading of Coastal Area Development of a Sandy Beach (Ix-Xtajta tal-Qawra, Qawra, St Paul's Bay) – Project Description Statement. Report prepared for Malta Tourism Authority, 20pp. and Appendices, August 2006.

Borg, J.A., 2006. Report on a Marine Benthic Survey of an Area Adjacent to Qawra Point, Identified for Beach Replenishment Works, made in August 2006. Survey commissioned by the Malta Tourism Authority through Adi Associates Environmental Consultants Ltd. Ecoserv Ltd. Report, August 2006.

Dean, R.G., 1987. Coastal Sediment Processes: Toward engineering solutions. Proceedings Coastal Sediments' 1987, AM. So. Civ. Eng., 1-24

Hallermeier, 1981. A profile zonation for seasonal sand beaches from wave climate. Coastal Engineering, Vol 4, 253-277.

Neilson, P., and Hanslow, D. J., 1991. Wave Run-up Distributions on Natural Beaches. Journal of Coastal Research, 7 (4), 1139-1152.

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