BEACH NOURISHMENT AND COASTAL PROTECTION ALONG THE GOLD COAST, AUSTRALIA: A CASE STUDY AT PALM BEACH

DARRELL STRAUSS[1], KRISTEN D. SPLINTER[1*], RODGER TOMLINSON[1#]

1. *Griffith Centre for Coastal Management, Gold Coast Campus, Griffith University, QLD, Australia 4222. [1]d.strauss@griffith.edu.au, [1*]k.splinter@griffith.edu.au, [1#]r.tomlinson@griffith.edu.au.*

Abstract: Effectively managing coastal beaches in areas exposed to high wave climate variability requires an appraisal of all the stakeholder interests and consideration of the above into a decision making framework. The popularity of the Gold Coast beaches as a national and international tourist destination as well as the increasing residential population ensures that the care of beaches maintains a high priority for the local government. Lessons learned from past management strategies are reviewed and a new methodology for assessment of beach condition is presented here which integrates the primary descriptors of beach health for the Gold Coast region of Australia's East Coast.

Introduction

The Gold Coast region, situated along the south-east coast of Queensland, Australia, is an iconic tourist destination known for its white sandy beaches and world class surfing. The gross economic value of the Gold Coast beaches has been estimated between \$106 million and \$319 million per annum (Lazarow et al. 2010) with more than 4.9 million day and 4.4 million overnight tourists visiting the region each year. With such a high economic and social impact, maintaining safe, sufficient beach access for current and future users is a priority coastal management issue at both local and state levels. The Gold Coast City Council currently undertakes a suite of engineering works to maintain approximately 35 km of sandy coastline. These include dredging and beach nourishment operations, sand by-passing schemes, several existing groynes and an artificial reef. Palm Beach is one of the most vulnerable sections along this coast and is frequently eroded back to the seawall. Maintenance dredging of the adjacent Currumbin Creek for flood mitigation purposes delivers 50,000 to 60,000 m^3 of sand to the beach annually. This minor nourishment is considered to be enough for short-term recovery but not enough to withstand a severe storm event. Training walls at creek entrances to the immediate north and south of the 4.2 km stretch of coast, as well as two short groynes along the beach, offer minimal reduction to the net northward longshore transport of sand. The last major nourishment exercise conducted at Palm Beach resulted in 385,000 m^3 of sand being placed on the beach between 2004 and 2005. Recent analysis of beach profiles demonstrates that in 2007 the beach was currently no wider than it was prior to the 2004 nourishment (Tomlinson et al., 2007). Since then a severe storm in May 2009 eroded sections of the beach, exposing the boulder wall. More than 400 m^3/m can be eroded from the upper profile of the beach during episodes of erosion such as this. Roughly half of what eroded in 2009 was deposited in an offshore storm bar, while the rest was lost to adjacent sections of the beach.

Current Design Profiles

After a series of devastating storms of 1954, 1967 and 1974, Delft Hydraulics was contracted to perform an engineering site analysis of the area and provide possible solutions to protect the coastline (Delft Hydraulics Laboratory 1970). A boulder wall (the 'A-line') has now been constructed along most of the coast and serves as a final line of defense against direct wave attack on adjacent infrastructure (Figure 1).

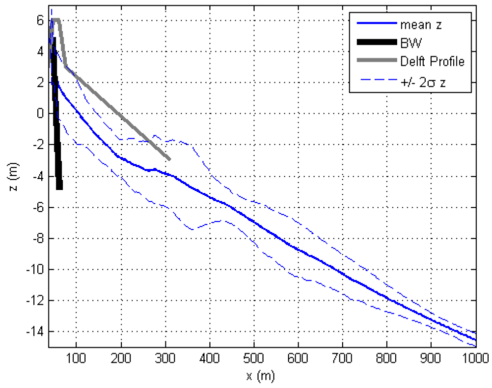

Figure 1. Delft design profile for Palm Beach survey line, ETA32, with boulder wall. Also included are the mean profile based on survey data and +/- 2σ of the mean to show temporal variability.

The wall is buried under the active beach profile and in some locations may be exposed following large storm events. Design profiles call for a primary dune system to cover the boulder wall, and to provide a gently sloping foreshore (1:30). In practice, however, this is rarely obtained, and in most cases, the beach is severely eroded back from the design specifications. The design profile volume adopted by the Beach Protection Authority in 1968 was chosen to be able to withstand the erosion of 414 m^3/m of beach above RL -3.05 m AHD (Australian Height Datum). This volume was based on an analysis of the volume lost from the most severely affected beach as a result of the cumulative effects of a sequence of several storms and tropical cyclones which affected the coast in 1967. With the exception of the southernmost beaches, which receive sufficient nourishment through artificially bypassed and dredged material, the design profile volume of 759 m^3/m above RL -3.05 m AHD is not realized in practice. A comparison of the mean profile and the design profile for Palm Beach indicates that the upper beach volume is substantially lower than the design profile (Figure 1).

Assessing Current Beach Vulnerability

Current practice utilizes a Beach Volume Index, *BVI*, to assess beach health at various locations along the Gold Coast. The BVI is determined by:

$$BVI = \frac{V_n}{V_D} + \frac{V_n - V_{n-1}}{V_D}, \tag{1}$$

where V_n (m^3/m) is the profile volume of sand, at time *n,* in the beach profile above a prescribed datum (currently, RL -3.05 m AHD), and V_D (m^3/m) is the design profile volume. The BVI can be used in two ways:

i) The present health of the beach with respect to the previous survey can be evaluated to prioritize future protection strategies. A value of $BVI = 1$ indicates that the current profile matches the design profile and has not changed since the last survey. This would suggest a degree of stability of both the management strategy and the ability to cope with typical conditions. For a $BVI > 1$ there is adequate sand in the profile and for $BVI < 1$ the beach is vulnerable to storm erosion with lower values indicating a greater risk of exposing the boulder wall.

ii) The trend in the change of the BVI over time for a given location can be used to provide a prediction of change to a more vulnerable state or used to assess the performance of protective measures.

Here we can compare the BVI for the profile of Palm Beach, a narrow beach backed by a low dune, with a profile of Narrowneck beach, 15 km to the north with a substantially taller dune (Figure 2). Direct comparison shows that Narrowneck is far healthier in terms of the BVI. Prior to the construction of the multi-purpose artificial reef offshore of Narrowneck in the early 2000s, the BVI averaged 0.6, while post construction BVI values are around 1 (Figure 2, right panel). The major storm in 2009 removed some of the profile, however the BVI remains around 0.8 and the beach is still considered to be fairly resilient to storm wave attack.

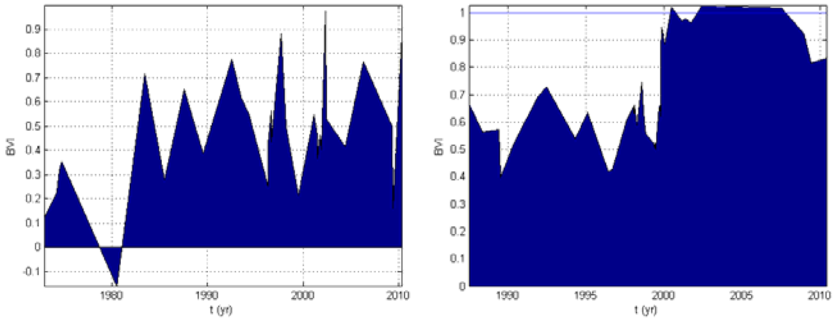

Figure 2. BVI for Palm Beach, ETA32: 1972-2010 (left panel) and Narrowneck, ETA 67: 1987-2010 (right panel).

The same cannot be said for Palm Beach. The boulder wall is covered by a minimal amount of sand above its crest and is much closer to the active beach and shoreline. Nourishment efforts are short lived as observed by the spikes in the BVI (Figure 2, left panel) and natural recovery is limited. Comparison of pre- and post-storm surveys from May 2009 indicates that longshore gradients in sediment transport can affect the profile evolution at this site.

Recent work by Froehlich (2010) suggests that under projected climate change scenarios, the capacity to maintain an active dry beach following a storm event will be greatly reduced at Palm Beach. Here we use a calibrated process-based sediment transport model, XBeach (Roelvink et al. 2009), to estimate the potential resiliency of the design profile to protect the boulder wall from exposure for specific design storms. A synthesized design profile was extended by merging the April 2009 survey with the design profile. The initial dry volume of sand in front of the profile was 345 m^3/m and the initial shoreline position relative to the top of the boulder wall was x = 147 m.

Design Storm

The worst recorded impacts of erosion on Gold Coast beaches were associated with a series of eight severe storms and tropical cyclones that occurred within a six month period in 1967. Parametric data from wave buoys and tide gauges from 1967 was unavailable so a design storm has been synthesized as a sequence of two recent storms where data was available. The two selected storms, an intense coastal depression (known as an East Coast Low) from 1996 and tropical cyclone Roger from 1993 were typical of the scale of individual events encountered during the sequence of 1967 storms (Figure 3). The 1996 East Coast Low (ECL) produced significant wave heights (H_s) commensurate with an annual return interval of 1:50 (Allen and Callaghan, 2000). Anecdotal evidence indicated that a wave setup equal to 10% of H_s could be added to the predicted tide throughout with a 1:50 year storm surge applied during the peak of the first storm as a proxy for observed water elevation (Figure 3). Significant wave height and peak period at 30 minute intervals were derived from wave buoy recordings during the period (01/05/1996 00:00 - 07/05/1996 23:30). Wave angle was set to 90^0N (directly from the East).

Tropical cyclone Roger occurred in March 1993 and came within 250 km of the east coast of Australia. The cyclone had a very large circulation and an extensive area of gale to storm force winds developed. A storm surge of 0.74 m was recorded and the peak water level reached 0.16 m above highest astronomical tide (HAT). The Brisbane wave rider buoy recorded significant and maximum wave heights of 7.5 m and 13.2 m, respectively. Tropical cyclone Roger produced significant wave heights commensurate with an annual return interval of 1:20 (Allen and Callaghan, 2000). Wave data from 11/3/1993 00:00 - 21/3/1993 23:00 was used to create the wave time series. Again, a wave angle of 90^0N was applied. Water elevations were based on predicted tides and included a storm surge component of 10% of H_s during the storm (defined as $H_s > 3$ m).

Figure 3. Design storm conditions used for the 1:100 Hs annual return interval.

Modeling

XBeach Model Description

XBeach (Roelvink et al. 2009), a model designed to estimate eXtreme Beach erosion during storm events was used to model dune erosion and test the resiliency of the design profile. The model solves the depth-averaged nonlinear shallow water equations using a wave action balance formulation:

$$\frac{\partial A}{\partial t} + \frac{\partial c_x A}{\partial x} + \frac{\partial c_y A}{\partial y} + \frac{\partial c_\theta A}{\partial \theta} = -\frac{D}{\sigma}, \tag{2}$$

where t,x,y,θ represent the temporal and spatial dependencies, c (m/s) is the wave action propagation speed and the wave action, A (kg/s),

$$A(x,y,\theta) = \frac{S_w(x,y,\theta)}{\sigma(x,y)}, \tag{3}$$

is represented by the wave energy in each directional bin, S_w (kg/s^2), and σ (Hz) is the intrinsic wave frequency. For instationary waves, the wave dissipation, D (kg/s^2/s), is modeled using the formulation of Roelvink (1993) (model parameter *break = 1*):

$$D = 2\alpha f_{rep} E_w Q_b \,. \tag{4}$$

In equation 4 α is a free parameter of $O(1)$, f_{rep} (Hz) is the representative intrinsic frequency, E_w (kg/s^2), is the wave energy, and Q_b is the fraction of breaking:

$$Q_b = \min\left[1 - e^{-\left(H_{rms}/\gamma h\right)^n}, 1\right]. \tag{5}$$

H_{rms} (m) is the local rms wave height, h (m) is the local water depth and n, γ are free parameters in the model. For the model runs presented here, n = 10 and γ = 0.55. The transfer of momemtum from the waves to the water column via the wave roller is included through a similar roller energy balance formulation.

The model uses the Generalized Lagrangian Mean (GLM) formulation to represent the depth-averaged undertow. The Eulerian depth-averaged velocities, u_E (m/s), are replaced by their Lagrangian equivalent, u_L (m/s):

$$u_L = u_E + u_S \,, \tag{6}$$

where u_S (m/s) represents the Stokes drift:

$$u_s = \frac{E_W \cos\theta}{\rho h c} \,. \tag{7}$$

Wave energy, E_w (kg/s^2), and angle, θ (degrees), are determined from the wave action balance equation and ρ (kg/m^3) is the density of water. This results in the GLM momentum equations:

$$\begin{aligned} \frac{\partial u_L}{\partial t} + u_L \frac{\partial u_L}{\partial x} + v_L \frac{\partial u_L}{\partial y} &= -\frac{\tau_{E,bx}}{\rho h} - g\frac{\partial n}{\partial x} + \frac{F_x}{\rho h} \\ \frac{\partial v_L}{\partial t} + u_L \frac{\partial v_L}{\partial x} + v_L \frac{\partial v_L}{\partial y} &= -\frac{\tau_{E,by}}{\rho h} - g\frac{\partial n}{\partial y} + \frac{F_y}{\rho h} \end{aligned} \,, \tag{8}$$

where u_L,v_L are the cross-shore and alongshore components of velocity, $\tau_{E,bx(y)}$ (kg/ms^2) are the cross-shore (alongshore) Eulerian bed shear stresses, g (m/s^2) is the gravitational acceleration, η (m) is the free surface elevation, and $F_{x(y)}$ (kg/ms^2) are the cross-shore (alongshore) wave-induced stresses.

Sediment transport is modeled using the formulation of Galapatti (1983) with a depth-averaged advection diffusion equation:

$$\frac{\partial hC}{\partial t} + \frac{\partial hCu_{av}}{\partial x} + \frac{\partial hCv_{av}}{\partial y} + \frac{\partial}{\partial x}\left[D_h h \frac{\partial C}{\partial x}\right] + \frac{\partial}{\partial y}\left[D_h h \frac{\partial C}{\partial y}\right] = \frac{hC_{eq} - hC}{T_s} \,, \tag{9}$$

where C is the depth averaged sediment concentration varying on the infragravity time scale, u_{av} (m/s) and v_{av} (m/s) are the cross-shore and alongshore velocity including the effects of wave skewness and asymmetry, D_h is the horizontal diffusion factor, C_{eq} is the equilibrium suspended sediment transport concentration, and T_s (s) is the adaptation time-scale for the entrainment of sediment. The inclusion of wave asymmetry has recently been implemented into XBeach (version 18) and is defined as:

$$u_{av} = V_W \cos\theta_m + u_E$$
$$v_{av} = V_W \sin\theta_m + v_E \qquad (10)$$

where θ_m (degrees) is the mean wave angle and V_W (m/s) is the velocity amplitude:

$$V_W = \gamma_{ua} u_{rms} (S_k - A_s)\ . \qquad (11)$$

γ_{ua} (model parameter *facua*) is a free parameter in the model and determines to what effect short wave properties impact sediment transport. u_{rms} (m/s) is the near-bed rms velocity, while S_k and A_s are the parameterized wave skewness and asymmetry, respectively, and are a function of the Ursell number.

Finally, the adaptation time scale for the entrainment of sediment is:

$$T_s = \max\left(f_{Ts} \frac{h}{w_s}, 0.2 \right) s\ , \qquad (12)$$

and a function of the local depth, h (m), sediment fall velocity, w_s (m/s), and the sediment transport depth factor, f_{Ts}. Bed updating at each time step is based on the continuity equation:

$$\frac{\partial z_b}{\partial t} + \frac{f_{morph}}{1-p}\left(\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} \right) = 0\ , \qquad (13)$$

where p is the sediment porosity (set to 0.4), z_b (m) is the bed elevation, f_{morph} is the morphological acceleration factor and q_x, q_y (m^2/s) are the cross-shore and alongshore sediment transport rate, respectively. An avalanching criterion to account for dune slumping is also imposed on the bed-updating. When the critical slope, m_{cr}, as defined by the user (*wetslp, dryslp*) is exceeded, the following bed updating occurs:

$$\begin{aligned} \Delta z_b &= \min\left(\left(\left| \frac{\partial z_b}{\partial t} \right| - m_{cr} \right) \Delta x, 0.05\Delta t \right), \quad \frac{\partial z_b}{\partial x} > 0 \\ z_b &= \max\left(-\left(\left| \frac{\partial z_b}{\partial t} \right| - m_{cr} \right) \Delta x, -0.05\Delta t \right), \quad \frac{\partial z_b}{\partial x} < 0 \end{aligned}\ , \qquad (14)$$

where Δt is the time step in the model and Δx is the cross-shore grid spacing. A complete description of the model can be found in Roelvink et al. (2009) and the XBeach users manual (www.xbeach.org).

Model Setup

The offshore boundary condition was forced using a Jonswap spectrum (*instat* = 4, γ_J = 3.3). We included Active Reflective Compensation (*ARC* = 1) and bound-long waves (*order* = 2). The offshore boundary was set to a 2D adsorbing-weakly reflective condition (*front* = 1), while the lateral boundaries were set to Neumann conditions (*left=right=0*). Cross-shore grid spacing, Δx (m) was variable and Δy = 5 m in the alongshore direction. The model was run in alongshore uniform profile mode, where the number of alongshore grid points, *ny,* was set to 3. Additional model parameters are summarized in Table 1. Unless otherwise noted, default parameter values were used in the simulations.

Table 1. Summary of parameters used in XBeach runs.

		Value	
Parameter	**Description**	**Default**	**Used**
hmin (m)	Threshold depth for concentration and return flow	0.01	0.2
eps (m)	Threshold depth for drying and flooding	0.1	0.01
CFL	Maximum Courant number	0.2	0.9
facua	Determines relative importance of short-wave forcing on sediment transport	0	0.15
gamma	Breaker parameter in Roelvink dissipation term	0.6	0.55
Morfac	Morphological speed-up factor	0	10

Model Calibration

XBeach has been calibrated to the Gold Coast beaches using the May 2009 ECL. For Palm Beach, two surveys spaced one month apart made this event an ideal calibration exercise. Surveys are oriented with a bearing of 44°35'28" that is roughly perpendicular to the A-line and the bathymetry contours (Figure 4).

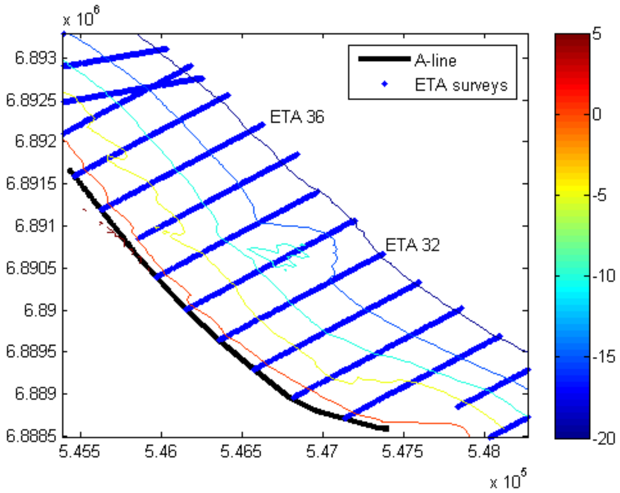

Figure 4. Palm Beach ETA survey lines (blue) with ETA 32 identified, with A-line (black) location. June 2010 bathymetry contours (m) are included in colour.

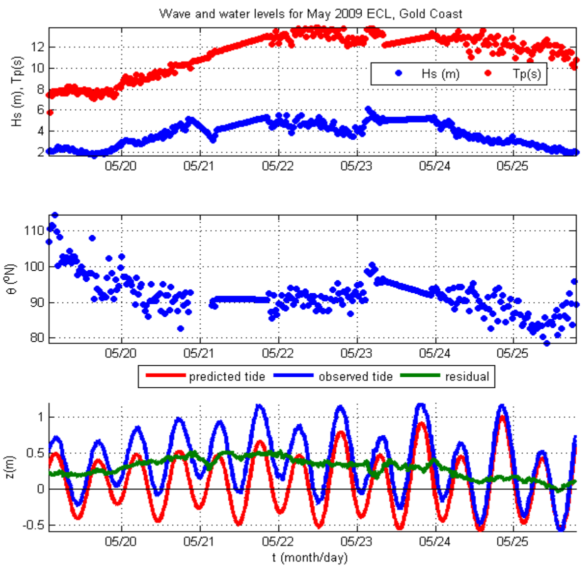

Figure 5. Measured wave statistics and water levels for the May 2009 ECL. (Top) wave height, Hs (m) and peak wave period, Tp (s). (Middle) wave direction, θ (°N). (Bottom) predicted and observed water level.

It should be noted that the model assumes no longshore gradients in transport when run in profile mode, so total volume must be conserved. If considerable longshore transport gradients occurred during this storm, these will not be accounted for in the current set-up. As the focus is on estimating dune erosion, rather than offshore bathymetry during the storm, this is of secondary importance during these runs as cross-shore processes are assumed to dominate dune erosion.

The storm duration was defined by the first and last exceedance of $H_s > 2$ m. ('19-May-2009 01:30:00': '25-May-2009 19:00:00'). Wave conditions in the model were updated every 30 min based on buoy data. Water levels were updated every 10 minutes using water levels from a tide gauge station located nearby at the Southport Marine Operations Base (Figure 5).

Figure 6 shows the calibration result for ETA 32 using the May 2009 ECL data set. The 04/28/2009 survey was extended shoreward from ~ z = 3 m to the boulder wall (based on the 06/02/2009 survey) assuming a linearly increasing dune profile. Observed shoreline retreat, $\Delta x_{s,obs}$, was 22 m, while the model predicted $\Delta x_{s,model}$ = 12 m. Observed dry beach erosion, $\Delta V_{dry,obs}$, (for z > 0) was -77 m^3/m, while the model predicted $\Delta V_{dry,model}$= -63 m^3/m. Total (entire profile) observed change in volume, ΔV_{total}, was -249 m^3/m suggesting that significant gradients in longshore transport were present during this storm. The model qualitatively matched the offshore extent of bar migration (x~ 500 m) however a realistic bar was not formed by the model. This is an acknowledged limitation of the current model formulation as it models very little of the bar generating processes, such as wave skewness. The calibration results suggest the model can provide qualitative estimates of erosion of the upper beach and dune for tests of storm scenarios with reasonable confidence.

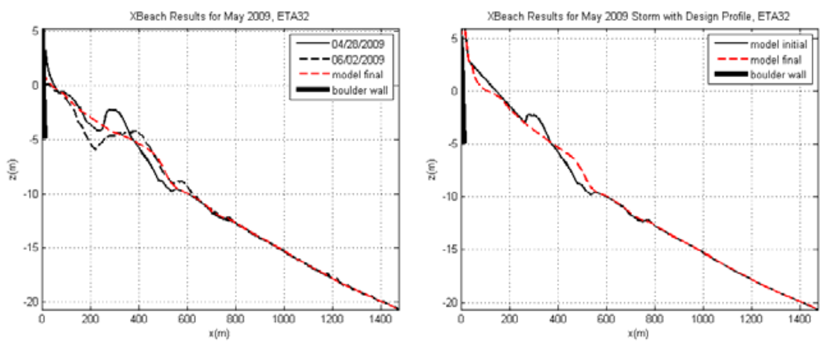

Figure 6. May 2009 ECL storm on profile ETA32. (Left) Calibration results for ETA 32 using pre-storm surveyed profile. (Right) Modeled erosion using design profile.

Design Profile Results

Predicted erosion for the storms varied according to storm characteristics (Table 2). Total eroded volume and shoreline retreat for the May 2009 storm was greater with the design profile (Figure 6, right panel) than with the initial measured profile (Figure 6, left panel). This is to be expected as the pre-storm profile from April 2009 had considerably less sand in front of the boulder wall and the wall was exposed during this event. The results suggest that the erosion potential of the 2009 storm was between a 1:20 and 1:50 wave height annual return interval (ARI) design storm.

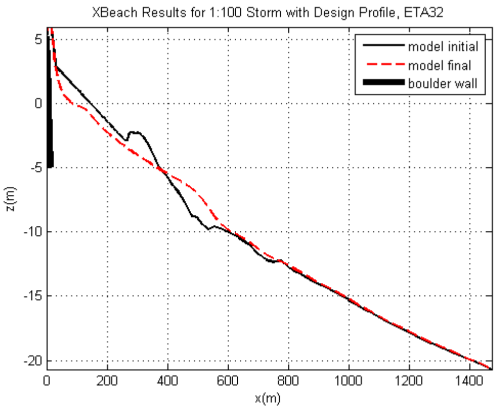

Figure 7 Modeled erosion for 1:100 design storm for ETA 32 using design profile data.

Table 2. Model results for design storms using the design profiles.

Storm Event	**ΔV_{dry} (m^3/m)**	**ΔX_s (m)**
May 2009 ECL	-83.4	-30.4
1:20	-39.6	-8.5
1:50	-104.7	-28.3
1:100	-125.8	-57.5

The shoreline retreat for the May 2009 event was comparable to the 1:50 event while the total erosion was significantly higher. This may be attributed to differences in wave angle, total water levels as well as storm duration. The design profile was capable of withstanding the erosion of the 1:100 ARI sequential storm events (Figure 7). This suggests that if the design profile can be maintained via aggressive management practices, the boulder wall is not likely to be exposed under extreme wave attack.

Conclusions

Under current beach conditions Palm Beach has a high potential for severe erosion and the exposure of the boulder wall during storm events with relatively modest wave height return intervals as observed during the May 2009 event. Similarly, the Beach Volume Index indicates that despite ongoing nourishment and control structures to trap longshore transport, Palm Beach is far more vulnerable than a nearby site that is supported by the implementation of a submerged control structure and nourishment a decade ago. The BVI demonstrates that the reduced dry beach volume at Palm Beach has failed to maintain a profile capable of withstanding the erosion volumes that can be accommodated by a storm with a 1:100 year wave height return interval.

Modeling of the upper beach with XBeach was used to determine the resiliency of the current design profile to withstand a series of storm events. Model results indicate the current design profile is capable of providing adequate protection for the 1:100 wave height return interval design storm under current sea level conditions. However, historical surveys indicate that Palm Beach cannot maintain an increased profile for extended periods of time. This suggests that solely nourishing to the design profile is not sufficient to address the complex sediment transport along Palm Beach. Without further strategies to provide and maintain an increased profile it is expected that the boulder wall will become exposed more frequently and a usable beach will be absent in the most erosion prone sections. Anticipated rises in sea level and changes to wave climate will further increase the dependence on the seawall for the protection of coastal infrastructure.

Acknowledgements

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