Final Design of Two Coastal Erosion Options for Eastern Tongatapu, Tonga

Prepared for:

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Cover page: A small ad hoc groyne at the northern end of Talafo'ou Village.

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1 Background

This report provides the Final output for SPC contract CC/13/95 Final Engineering Design and Cost for Coastal Protection Works in Eastern Tongatapu, Tonga; the Draft Design Document describing the two coastal protection options, justification of their selection, prioritisation of the options, engineering drawings and specifications, detailed costs and schedule for completion of works for each option. SPC, specifically the Global Climate Change Alliance: Pacific Small Island States (GCCA:PSIS) project in the Strategic Engagement Policy and Planning Facility, has commissioned eCoast Marine Consulting and Research to provide final engineering design and costing for coastal protection works in eastern Tongatapu (Figure 1.1).

Following review of the historic and recent information (Mead *et al*., 2013a), a Draft Design Document was prepared (Mead *et al*., 2013b) that included a description of the two pilot construction options (one hard, one soft engineering solution), justification of their selection, prioritisation of the options, engineering drawings and specifications, detailed costs and schedule for completion of works for each option. A monitoring and evaluation plan has also been prepared in order to determine the efficacy of the trails and learn how the beaches are responding to the 2 trial interventions (Mead *et al*., 2013c). The monitoring will be conducted by the Government of Tonga, over at least a 2 year post-construction period for each option and the borrow site used for sand recharge at the trial sites. The project has a €0.5M cap.

This Final Design document has been produced from the Draft Design document following consultation with key stakeholders, including the GCCA:PSIS project team; SPC-AGTD, Ministry of Lands, Environment, Climate Change and Natural Resources, Ministry of Infrastructure; and others through one-on-one meetings and a national planning workshop in Nuku'alofa involving the aforementioned stakeholders and the affected communities and obtain input. Consultation and the workshop was undertaken 21/22 June 2013, and included a site visit for all delegates.

An important factor that is being considered during this pilot study is the ADB project for the same area of Tonga, which is also considering climate change resilience and the trialling and monitoring of coastal protection options in this area of northeastern Tongatapu. While the ADB project is likely to follow this project by approximately 12 months, the potential benefits include extending monitoring to 4 years, trialling more options and developing a better understanding of the existing coastal processes than currently available for the site. A better and quantified understanding of the existing coastal processes will lead to the development of sustainable and effective methods for these types of environments in Tonga and in the Pacific.

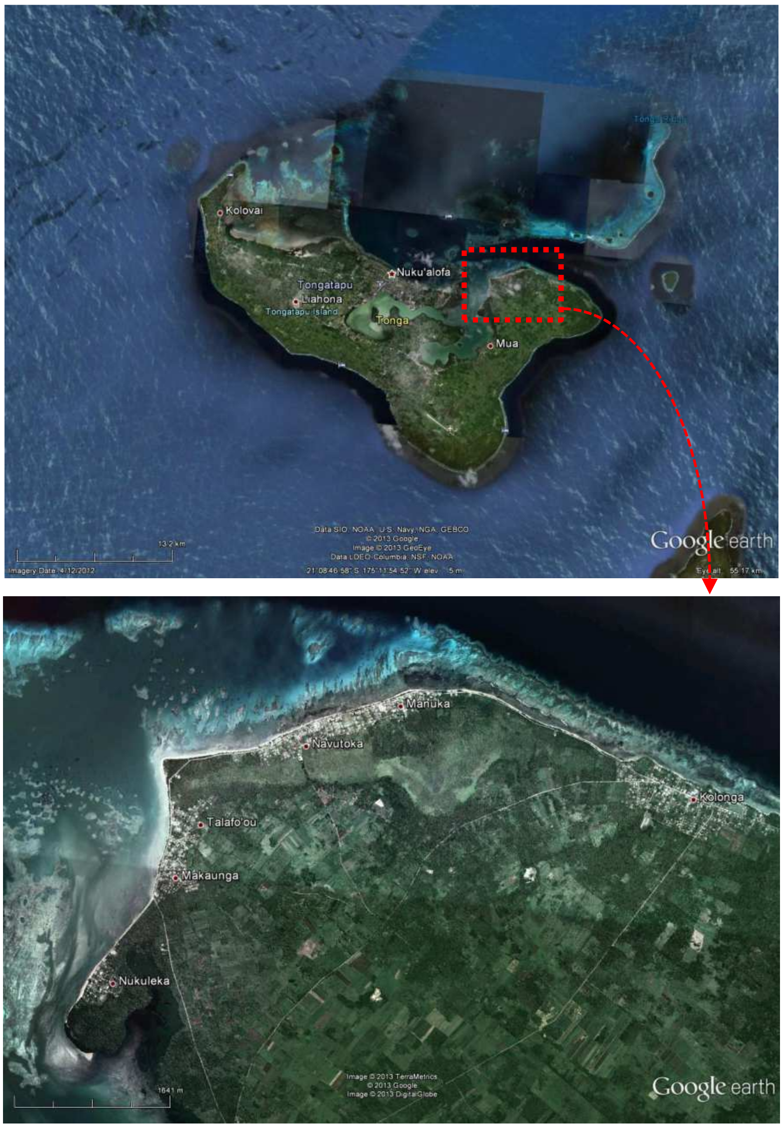

Figure 1.1. Location map of the 5 villages in eastern Tongatapu where the coastal protection pilot studies are planned. The study site incorporates the villages of Nukuleka, Makaunga/Talafo'ou, Navutoka, Manuka and Kolonga (Source Google Earth 2013)

2 Selection of Two Options for Trials

The review of the historical and recent investigations of the area of interest (Figure 1.1) is presented in the first output of this consultancy (Mead *et al*., 2013a). This review presents not only the historical erosion since 1968 based on geo-referenced aerial photographs, but also discusses the 4 options put forward by CTL (2012a,b). The brief calls for the selection of one hard and one soft option for coastal protection/climate change resilience. However, the options presented in the CTL (2012a,b) reports do not include a soft solution, but rather hybrid solutions that incorporate a combination of hard and soft engineering, which are more appropriate for the sites. Keeping both the budget-cap and ADB's involvement in mind, the two pilot studies that we have selected are within A3 and C2 (as per CTL, 2013a,b), that is the 350 m stretch to the east of Manuka Village and the beach front along Talafo'ou and Makaunga Villages (Figure 2.1).

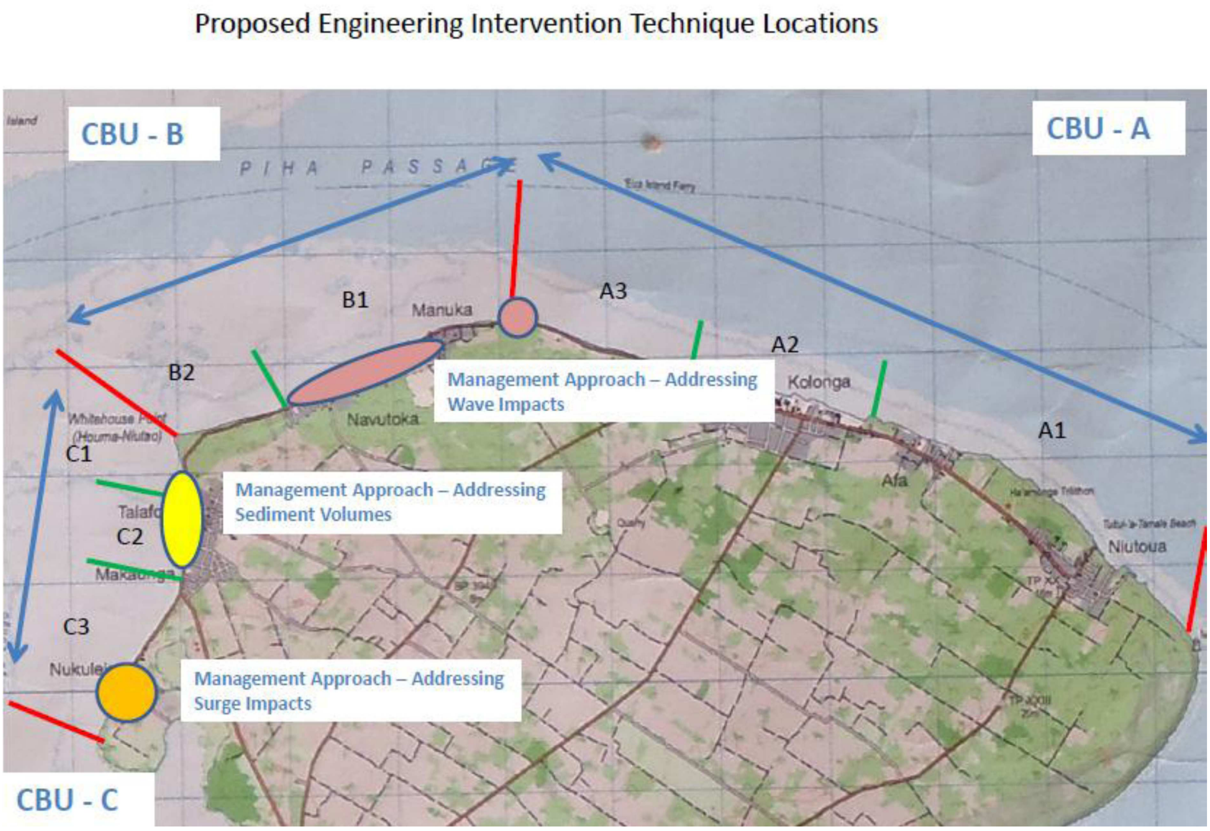

Figure 2.1. The geomorphological units determined by CTL (2013a,b), including the proposed engineering intervention technique locations – orange = hard defence, yellow = soft defence, and pink = hard and soft defence.

2.1 “Soft Option” Area C2 – Talafo’ou and Makaunga

This area has been selected and prioritised due to the following factors:

1. This site is in front of 2 of the villages that require increased resilience for climate change/sealevel rise.
2. It combines a soft option (renourishment) with a hard option (permeable groynes).
3. The hard component to the intervention is novel and incorporates the potential for flexibility with respect to sediment transmission by orientation of the construction units (sedi-tunnels). Indeed, permeable groynes alleviate one of the potential issues with groynes by allowing sediment to move in both directions along the shore.
4. The sedi-tunnel units are already being built in Tonga, and so new processes do not have to be developed.
5. Observations indicate that groyne structures will retain sand in this area, which is conducive to groyne structures being efficient where it is alongshore currents rather than waves that are transporting sand.
6. Monitoring of the sediment extraction zone (C1 – Figure 2.1) will provide data to determine whether or not this area is a sediment ‘sink’, and along with monitoring of the trail area, provide an indication of the sustainability of the project.
7. The beach replenishment of Makaunga and Talafo’ou villages is also considered from the tourism aspects by MEC (2012) to have local economic potential.

The trial will consist of varying spaced 10 m long groynes with varying permeability, sand recharge and planting of salt-tolerant shrubs.

As described in the initial report (Mead *et al*., 2013a), CTL (2012b) suggested 4 x 10 m long groynes along some 900 m of beach. However, given that groynes are normally 2-3^{rds} the length of the spacing between them (Basco and Pope, 2004), and the proposed groynes are 10 m long, tighter spacing and more groynes are recommended for effective widening of the full length of beach. Longer groynes could be built to reduce the number required, however, longer groynes have greater potential to cause downcoast erosion due to sand trapping and ‘starvation’ of the downcoast beach.

It is noted that the 1:3 design ratio for groynes was developed for temperate sandy coasts, not for coral sand/reef/lagoon situations. In addition, given the relative shelter from wave activity (this area is current-dominated) and the comparably larger grain size of coral sand (which leads to accretion more often than erosion (e.g. Dean, 1988), larger spacings are likely to be applicable. Indeed, observations indicated that along this stretch of the study area, either small protrusions of rocks or substantial trees just off the beach result in an obvious fillet of sand of greater length than 3x the obstruction (Figure 2.2 and cover picture). Therefore, a range of separation distances will be trialled to determine the most appropriate for the site through the monitoring and evaluation procedure.

Figure 2.2. A fillet of sand on the northern side of a small groyne in Makaunga Village.

In addition to the varied spacing of groynes, the sedi-tunnels provide the opportunity to present varying levels of permeability by rotating the units through 90° (Figure 2.3 and Figure 2.4). Groynes are often associated with downcoast erosion since they are basically sand-traps and one-way valves – once the sand moves past the groyne it cannot move back around the groyne in a beach system dominated by uni-directional alongshore currents. The permeable potential means that the groynes can allow some sand to move through the structure while also not acting as a one-way valve and allowing sand to move in the opposite direction as conditions dictate. Permeable groynes have been successful in other parts of the world (e.g. Dette *et al*., 2004). The sedi-tunnel permeable groyne is a novel methodology, and when set on a base allows for modification to the structures permeability following construction. The 1 m square unit sizes also allows for easy lengthening of groynes.

This project has been set-up to test potential erosion protection/climate change resilience. Area C2 is considered conducive to the application of groynes, while the structural units (sedi-tunnels) and design specifications (i.e. length, spacing) allow a trial design to be set up that will lead to an understanding of the most effective configuration for the site. Therefore, different spaced groynes of 10 m in length will be trialled with different permeability along Area C2. The details of the groyne layouts are detailed in Section 3 below.

A further feature of the Area C2 trial is the renourishment of 2,800 m^3 of sand extracted from Area C1 (Figure 2.1). It is best practise to fill beaches to the expected response volume based on the structural intervention being applied to ensure that downcoast impacts are minimised. In the present case, 2,800 m^3 will be sufficient to fill the groyne-field along Area C2, with ~900 m length, and a filled beach gradient of 1:12 (H:V) which is close to natural for a coral sand beach, i.e. (900*10*0.375)*10/12 = 2,812). Environmental impacts of sand removal at the western Whitehouse Point site have been found to be minimal (MEC, 2012). Monitoring will consider the efficacy of the groynes, downcoast impacts, and impacts at the sand extraction site.

Finally, salt-tolerant coastal species will be planted along on the upper beach to provide further coastal resilience.

Figure 2.3. Sedi-tunnel units manufactured in Tonga to protect under-ground cables.

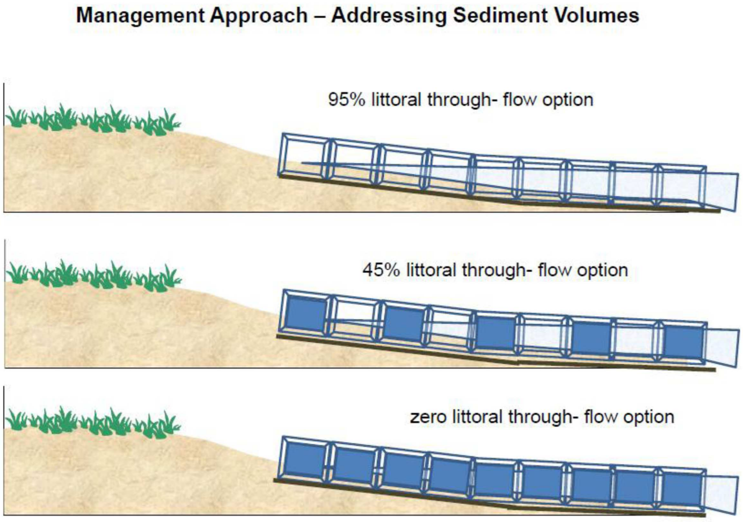

Figure 2.4. Rotation of the sedi-tunnel units allows for varying the permeability of the structures (from CTL, 2012b).

2.2 “Hard Option” A3 – East of Manuka Village

This area has been selected and prioritised due to the following factors:

1. The failed seawall at this site has resulted in the formation of tombolos and habitat suitable for mangrove to establish, i.e. evidence of a method.
2. The failed seawall provides material for foundations on which to build limestone rock detached breakwaters, reducing costs in this funding capped project.
3. The site combines both hard and soft solutions to achieve managed advance of the shoreline, which is critical to climate change resilience.
4. This demonstration site is the most exposed of all the areas, and successful results could be applied along the slightly less exposed beach fronts of Navutoka and Manuka Villages to the west.
5. The establishment of mangrove habitat provides some niche habitat in an area of degraded marine ecology (CTL, 2012a).
6. The protruding seawall and holes in the nearshore zone to the west of Area A3 effectively separate this trail site from Area B1 (Manuka Village) and so downcoast impacts are not expected (Figure 2.5).
7. Over-topping of the road occurs at the eastern half of this 390 m length during even relatively mild wind/wave events (Figure 2.8) and needs protection.

Figure 2.5. The Area A3 trial site is effectively separated from Area B1 to the west by the protruding seawall and nearshore holes.

The failure of the previous seawall at the eastern end of Manuka has provided good evidence that tombolos can be formed in the lee of semi-emergent breakwaters (which are proving successful in similar locations in Fiji) and can also capture fines that lead to improved soil conditions for mangroves (Figure 2.6). It is unlikely that mangroves would survive on this part of the coast without protection from waves and modification of soils, which has been a positive result of the failed seawall (Figure 2.6). Some sections have remained to attenuate wave energy, while sand/sediment has been transported shoreward through the gaps and created a buffer zone along some areas of this 350 m long site.

Figure 2.6. Tombolos and mangroves growing in the lee of failed sections of the seawall.

Therefore, this area (Area A3) will be used to trail low-crested detached breakwaters on this wave-exposed section of the study site. This will be accompanied by beach recharge with the aim to create the kind of beach response shown in the stylised Figure 2.7. It is recommended that mangroves are not planted in this area for at least 1 year following construction to allow for the beach to adjust to the structures and sand recharge. It may be that different salt-tolerant species will be more applicable to the site following construction.

The details of the detached layouts are detailed in Section 3 below.

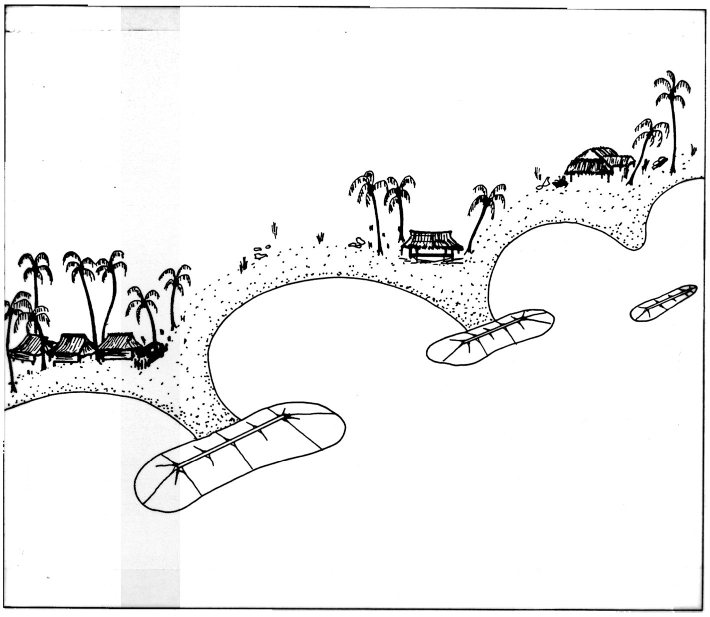

Figure 2.7. Stylised detached breakwaters leading to managed advance (SOPAC 1994).

Figure 2.8. Over-topping along the eastern part of the 390 m long trial site.

3 Option Specifications

3.1 Area C2 – Sedi-Tunnel Groynes and Sand Re-Charge

3.1.1 Locations and Configurations

The locations of the fifteen 10 m long sedi-tunnel groynes are shown in Figure A1-1 (Appendix 1). The first groyne is located some 30 m south of the most northern dwelling in Talafo'ou (21°08'01.80"S 175°07'17.68"W), and then the groynes are staggered at intervals (30 m, 60 m and 120 m), which are repeated down the beach southwards.

Each sedi-tunnel unit is 0.75 m high by 1 x 1 m square, with one side open to allow sediment to pass through as shown in Figure 2.3 – these units are manufactured on Tongatapu and will be sourced straight from the contractor. Bases are 5.2 m long by 1.4 m wide (i.e. 2 bases per groyne) and 100 mm thick. Both components of the sedi-tunnel groyne are reinforced, with reinforcing optimally central to the concrete (i.e. 50 mm deep). Sedi-tunnel units can be placed as to either block sediment or allow it to pass alongshore by rotating the units 90°, while the total permeability of the groynes can be adjusted by the configuration of each of the units.

Groynes 1-3 will be built with 45% transmission, i.e. every second unit will be rotated to allow sediment to pass through the opening in it (as shown in Figure 2.4). Groynes 4-6 will have zero permeability, with all 10 units being aligned to prevent any sediment transmission (Figure 2.4). Groynes 7-9 will be built to allow 95% transmission of sediment, with all units positioned to allow for sediment to pass through (Figure 2.4). Groynes 10-12 will built with a variety of transmission types, with groyne 10 having the shoreward 5 non-transmissive and the seaward 5 units all transmissive; groyne 11 having the opposite (shoreward transmissive and seaward non-transmissive); groyne 12 will have 3 shoreward non-transmissive, the next three transmissive, the next 2 non-transmissive and the final 2 transmissive. Groynes 13-15 will be constructed to all have zero permeability, and will be adjusted during the 3-4 years of the evaluation period based on the results of the monitoring to replicate the performance of other groynes in the groyne-field. The configuration of each sedi-tunnel groyne is provided in Figure A1-2 to A1-6.

3.1.2 Construction Procedure

While the fine details of construction should be carried out by the contractor and ensured by the Project Manager, the following provides some particulars that should be followed during construction:

1. The most shoreward sedi-tunnel of a groyne should be positioned 1 m shoreward of the existing high tide mark, with the beach being levelled to properly support the placement of the 5.2 m base units.
2. Once the position of the groyne has been located and confirmed, the beach should be prepared from 1 m shoreward of high tide along a ~1.5 m wide by ~11 m strip to ensure both base plates are level and supported along their lengths. Figure A1-7 provides some examples of sedi-tunnel groynes placed on base units on varying beach slopes.
3. Sedi-tunnel units are then placed on the base units in the specified configuration starting from the most shoreward unit (Figures A1-2 to A1-6).
4. Sand is placed in the between the groynes to come level half the height of the most shoreward unit (~0.375 m high), and graded down to 2 m beyond the sedi-tunnel groyne at a gradient of 1:10 to 1:12 (Figure A1-7). A total of 2,800 m^3 of sand will be required to nourish the whole length of beach.
5. Planting of coastal vegetation should occur in the margin from the existing high tide mark (i.e. pre-construction high tide) and shoreward.

3.2 Area A3 – Detached Breakwaters and Sand Re-Charge

3.2.1 Locations and Configurations

The locations and configuration of the 9 low-crested detached breakwaters for Area A3 east of Manuka Village are presented in Figure A2-1 (Appendix 2). All breakwaters will be located a nominal distance of 20 m from the edge of the existing coast to the inside top of the breakwater crest – over-topping of the eastern end is in part due to the remnant seawall being within 3-5 m of the road.

Limestone rock of 0.8 m diameter has been specified based on the application of the Hudson formula. The US Army Corps of Engineers' method for determining proper rock sizing relies on the Hudson formula which is expressed as:

$$W = \gamma_r H^3 / K_D (\gamma_r/\gamma_w - 1)^3 \cot\theta$$

Where,

W = weight of the outer layer armour unit

γ_r = unit weight of armour unit

H = design wave height

K_D = stability coefficient from armour size, shape and material

γ_w = unit weight of water

θ = angle of structure slope

Where K_D is a stability coefficient taking onto account all the other variables. K_D values in the literature are for "no damage" conditions defined so that up to 5% of the armour rock may be displaced. H in the equation being taken as $H_{1/10}$ i.e., the highest 1/10th of all waves.

Without local wave climate data, Gourlay's (1994) empirical formula for the estimation of maximum wave height, which is based on the maximum water depth over the lagoon/fringing reef, was applied to develop the maximum wave height at the shore (i.e. H_{10} of ~1.8 m during extreme events). The results of applying this maximum wave height to the Hudson formula led to a Dn_{50} (median rock diameter) of 0.74 m, with ~10% safety factor resulting in the specified 0.8 m limestone rocks at a gradient of 2:1 (H:V).

Some variation has been incorporated into the design of the breakwater's length, which varies between 15 and 25 m (crest length). The aim is to form tombolos (i.e. create a beach response where the beach welds onto the shoreward side of the breakwater (Figure 2.7)), which according to Gourlay (1981) will occur when the ratio of the length of the breakwater (B) to the distance of the breakwater off of the existing coast (S) is >0.67. The Shore Protection Manual (1984) states that a tombolos form when the B/S ratio >1. Thus, with a nominal distance offshore of 20 m (S), breakwaters of 15, 20 and 25 m result in B/S ratios of 0.75, 1 and 1.25, respectively. In addition, the spacings between are varied between 20 and 25 m. The breakwater length and spacing configurations for breakwaters 1-4 are shown in Figure A2-2, and for breakwaters 5-9 in Figure A2-3.

Similar to design specifications for groynes, these design specifications have been developed for temperate sandy coasts, which are not comparable to coral sand beaches on reef flats. Monitoring of the beach response along the trial site will provide valuable insight into the efficacy of detached breakwaters for managed advance and their application at similar semi-exposed sites in Tonga and around the Pacific.

As each breakwater is constructed the beach will be charged with sand from the borrow site (C1) and placed in a tombolo-like formation with the sand welded to the shoreward side of the detached breakwater (20 m wide) and curving to result in an approximately 10 m wide beach at the narrowest point (Figure A2-5). Approximately 2,200 m^3 of sand will be required to nourish the 390 m trial site. It is recommended that planting does not occur in the trial area until a year after construction to allow for the beach to adjust and respond to the detached breakwaters.

3.2.2 Construction Procedure

A detailed construction plan should be carried out by the contractor and endorsed by the Project Manager; the following provides some particulars that should be followed during construction:

1. A detailed survey of the site will be the first requirement, to provide both existing beach levels with respect to a known datum (and hence highest astronomical tide (HAT)) out to 30 m from the shore along the full length of the trial site (and so, the required rock and sand volumes for breakwaters and recharge, respectively, can be estimated), as well as an indication of the

extent and volume of remnant seawall that will be used for foundation material for the proposed breakwaters.

2. The results of the survey will then be used to develop a basic design and build programme that will incorporate foundation preparation and breakwater volume calculations.
3. Construction will be undertaken with earthmoving equipment (a large digger), which will progressively prepare breakwater construction areas by removal and stockpiling of the failed seawall and excavation/preparation of foundations.
4. Construction will begin from the western end of the trial site with a 10 m extension from the corner of the existing seawall (Figure A2-2). All structures will have foundation preparation that will include excavation to 1.5 m below the existing beach level, unless hard material is encountered at a shallower depth, which is likely to be the case along parts of this site where nearshore rock outcrops are visible. Approximately 0.5 of rumble will be positioned in the footprint of the detached breakwater and extent approximately 1.0 m beyond the base of the structure (Figure A2-4). If hard substrate is present, rubble will be used to develop an even platform for detached breakwater construction if necessary.
5. Limestone rock (0.8 m diameter) will then be placed by the digger to form the breakwaters with sides at gradients of 2:1 (H:V) and crest height of +0.5 m highest astronomical tide (HAT).

4 Costing and Construction Schedule

Prices have been provided from several sources and contractors in Tonga to provide the best estimate of the total cost for each option. Contingency funds have also been added, while the total cost is well within the €500K budget, allowing for support of monitoring and other activities throughout the 3-4 year trial period (e.g. maintenance, monitoring, the installation of further sedi-tunnel groynes within the 120 m compartments, additional nourishment, etc.).

Charging/nourishing will be undertaken at both trial sites. Sand is to be excavated from the nearshore lagoon (i.e. between 25 and 50 m off the beach), with the most likely cost-effective methods being a digger or front-end loader at spring low tide and trucks to move the sand to the site and dump at intervals along the beach. The digger or a bulldozer can then be used to 'groom' the sand into place between structures (sedi-tunnel groynes and detached breakwaters).

4.1 Area C2 Construction Cost – Sedi-Tunnel Groynes and Sand Re-Charge

The these costs include the construction of 15 x 10 m long groynes from sedi-tunnel units with different littoral transport flows and different spacings along the full length of the villages and nourishment with 2,800 m^3 of sand from the C1 borrow site (Table 4.1).

Sedi-tunnel units are already manufactured on Tongatapu, being used to house and protect underground cables (Figure 2.3). The units and bases are to be reinforced with steel re-bar to ensure structural integrity and prolong the unit's design-life.

Table 4.1. Cost for Area C2 construction:

Item	Units required	Unit Cost (TP$)	Total Cost (TP$)
Manufacture and Supply of "Sedi-Tunnel" units (1000x1000x750 by 100 thick) also refer to attached drawing	165 (10% contingency for breakage and replacement)	$813.78	$134,274.32
Construction of "Sedi-Tunnel" concrete base (5.2 m unit lengths by 1.4 m wide by 100 mm thick – each groyne requiring 2 base units) also refer to attached drawing	34 (>10% contingency for breakage and replacement)	$1,309.78	$44,532.49
Construction of 15 groynes	150 Sedi-tunnel units and		$134,700.00

	30 base units		
Sand recovery, transport and placement	2,800 m^3	$36.00	$100,800.00
Supply and plant of local robust salt tolerant trees/shrubs	100		$3,000.00
Total Cost			**$417,306.81**

4.2 Area A3 – Construction Cost – Detached Breakwaters and Sand Re-Charge

The these costs include the construction of 9 x 15-25 m long detached breakwaters from 0.8 m diameter limestone blocks with varied spacings along the full length of the 390 m trial site and nourishment with 2,200 m^3 of sand from the C1 borrow site (Table 4.2).

Table 4.2. Cost for Area A3 construction.

Item	Units required	Unit Cost (TP$)	Total Cost (TP$)
Unit cost of 0.8 m diameter limestone rocks (which can be per 1 m3). In addition, it will include the transport to Manuka and the preparation and placement (per 1 m^3) of the limestone with a digger.	5,000 m^3 (includes 10% contingency for estimated volumes)	$84	$420,000.00
Sand recovery, transport and placement	2,200 m^3	$36.00	$79,200.00
Supply and plant of local robust salt tolerant trees/shrubs	100		$3,000.00
Total Cost			**$502,200.00**

4.3 Total Cost for Construction of Both Options – Reduced Option C2

The total cost for both options is **TP$919,506.81**, which equates to **US$496,533.70.** Applying a contingency of 10%, which may be required for project management and cost over-runs (noting 10% contingency has already been applied to sedi-tunnel units, bases and 0.8 m diameter rocks in the costings above), the total expected cost is **TP$1,0011,457.49**, which equates to **US$546,235.67.** However, from the project budget, a total of **US$463,100** is available for both construction trials. Therefore, the funding will be distributed to <u>fully fund the trial of detached breakwaters</u> at Area A3, and the remaining funds will be used to fund the construction of <u>9 of the 15 sedi-tunnel groynes</u> at Area C2.

The funds allocated to Area A3 breakwater construction and renourishment, including 10% contingency, will be **US$304,549** (approximate **TP$564,000**), i.e. full funding of all breakwaters and renourishment. The funds allocated to Area C2 sedi-tunnel groyne construction and renourishment, including 10% contingency, will be **US$158,551** (approximate **TP$293,600**). Table 4.4 presents the construction cost breakdown for the reduced construction of 9 sedi-tunnel groynes and 2,000 m^3 of renourishment. The drawings for the 9 sedi-tunnels are A1-A4 and A7 (cross-section) in Appendix 1, that is the first 9 groynes starting from the north. There is potential for the remaining 7 groynes to be constructed during the following UNDP project for this same area of northeastern Tongatapu.

Table 4.3. Cost for Reduced Area C2 construction:

Item	Units required	Unit Cost (TP$)	Total Cost (TP$)
Manufacture and Supply of "Sedi-Tunnel" units (1000x1000x750 by 100 thick) also refer to attached drawing	99 (10% contingency for breakage and replacement)	$813.78	$80,564.59
Construction of "Sedi-Tunnel" concrete base (5.2 m unit lengths by 1.4 m wide by 100 mm thick – each groyne requiring 2 base units) also refer to attached drawing	20(>10% contingency for breakage and replacement)	$1,309.78	$26,195.58
Construction of 15 groynes	90 Sedi-tunnel units and 18 base units		$80,820.00
Sand recovery, transport and placement	2,000 m^3	$36.00	$72,000.00
Supply and plant of local robust salt tolerant trees/shrubs	65		$2,000.00
Total Cost			**$261,580,17**
Total Cost with ~10% Contingency			**$293,600.00**

4.4 Work Schedule

Work schedules for both options are provided in Table 4.4 and Table 4.5 below. It is expected that these will be modified by the successful contractors following tendering. Prioritising of construction of Options could be based on the season that construction is to occur to reduce down time due to winds, with Option C2 occurring within the period May-Oct (SE winds dominate) and Option A3 in Nov-April (lighter wind period). However, contractors advise that they would undertake construction of both trials simultaneously, with an expected total duration of 4-5 months.

Table 4.4 Cost for Option C2 construction.

Task	Wk 1	Wk 2	Wk 3	Wk 4	Wk 5	Wk 6	Wk 7	Wk 8	Wk 9	Wk 10
Site Inspection and preparation										
Transport and storage of units/bases										
Construction										
Renourishment										
Planting of coastal shrubs										

Note: The time to fabricate sedi-tunnel units and bases is not included.

Table 4.5. Construction schedule for Option A3.

	Wk 1	Wk 2	Wk 3	Wk 4	Wk 5	Wk 6	Wk 7	Wk 8	Wk 9	Wk 10	Wk 11	Wk 12	Wk 13	Wk 14	Wk 15	Wk 16	Wk 17
Site Inspection, survey, planning																	
Transport and Delivery of Rock																	
Construction																	
Renourishment																	

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Appendix 1 – Drawings for Option C2

Appendix 2 – Drawings for Option A3