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The Owners
29, 31 and 33 Pacific Street and 23a, 23b and 25c Ocean View Drive
Wamberal NSW 2260

8 March 2017

***Horton Coastal Engineering Pty Ltd ats NSW Coastal Panel* (Land and Environment Court No. 324345 of 2016), Response to List of Information Required for Section 34 Conciliation Conference**

1. INTRODUCTION AND BACKGROUND

As an outcome of a Section 34 Conciliation Conference held on 1 February 2017 for *Horton Coastal Engineering Pty Ltd ats NSW Coastal Panel*, the NSW Coastal Panel provided a List of Information Required on 9 February 2017. The 10 items listed by the Coastal Panel are responded to in turn in subsequent sections.

A peer review of the document herein, drawings, and accompanying structural engineering and geotechnical engineering reports, has been completed by Greg Britton from Haskoning Australia, which is provided as **Appendix E**.

2. ITEM 1: DESCRIPTION OF THE PROPOSED DESIGN AND MATERIALS

2.1 General

In response to comments from the Coastal Panel, the original proposed design (original DA) has been modified to:

- be entirely on private land (entirely within the 6 subject properties of 29, 31 and 33 Pacific Street and 23a, 23b and 25c Ocean View Drive Wamberal); and
- include a vertical piled toe (permanently anchored contiguous/secant piles[1] with reinforced concrete capping beam) at the base of the rock revetment.

The purpose of the piled toe, which adds significant cost to the protection works, is to prevent the rock revetment from being undermined in the design event. Even with beach scour down to -3m AHD seaward of the piled toe, the protection works would have a factor of safety against global instability exceeding 1.5 for the design event, and no scour would extend under the revetment, thus preventing any movement of rocks due to undermining.

The rock revetment generally has a similar design and materials to the original DA, namely basalt primary armour rock in 2 layers (individual rock median about 3.8 tonne mass and 1.3m dimension), and basalt secondary armour rock in 2 layers (individual rock median about 380kg mass and 0.6m dimension). However, to improve revetment permeability performance and for

[1] Contiguous piles down to -8m AHD, with a plug pile nestled landward in the gaps between the contiguous piles down to -4m AHD. This thus acts essentially as a secant pile wall (complete barrier) above -4m AHD.

ease of construction in conjunction with use of anchors, geotextile is no longer proposed and has been replaced by a second underlayer (individual rock median about 19kg mass and 0.22m dimension).

The advantages of a hybrid pile and rock structure compared to a full height purely vertical structure include:

- more effective dissipation of wave energy on the rock revetment (less reflected energy);
- less complexity and more effective tying in with the rock revetment at Manyana (25 Pacific Street) located south of the subject properties;
- less visual impact when beach levels lower after a coastal storm;
- less potential end effects (comparing a vertical wall located at the "hybrid pile and rock structure" piled toe position);
- more structural redundancy and lesser depth of piling; and
- less cost.

To prevent the protection works extending on to the Department of Planning and Environment Land at The Ruins (north of the subject properties), a vertical wall would be constructed along a portion of the northern boundary of 25c Ocean View Drive. The crest of this wall would follow the sloped upper profile of the rock revetment and the existing land levels landward of the revetment. To ensure that the protection works at Manyana are not weakened by the proposed works at the southern boundary of 29 Pacific Street, rocks forming the proposed works would be interlocked with the Manyana boulders. The Manyana revetment would be significantly strengthened by the proposed works, as it would no longer be at significant risk of outflanking from the north.

The design has been completed including coastal engineering input from Horton Coastal Engineering Pty Ltd, structural engineering input from James Taylor & Associates (**Appendix B** and **Appendix C**), geotechnical engineering input from JK Geotechnics (**Appendix D**), and overall peer review from Greg Britton of Haskoning Australia Pty Ltd (**Appendix E**).

2.2 Items 1a and 1b: Detailed Survey Plan and Cross Sections

A survey of the subject properties completed in June 2016 is provided as **Appendix A**.

Drawings depicting the proposed footprint of the structure, existing surveyed property boundaries and specifying all relevant dimensions of the proposed structure are provided in **Appendix B**. These drawings include sufficiently accurate cross sections of the structure at each of the 6 properties, and its proposed points of termination at the northern end of 25c Ocean View Drive and southern end of 29 Pacific Street. These cross sections clearly specify all relevant dimensions (including crest, toe and existing and pre-storm beach levels).

2.3 Item 1c: Adopted Engineering Principles, Codes or Standards Applied to the Design of the Structure

For the design of the rock revetment, general documents considered included the *Coastal Engineering Manual* (US Army Corps of Engineers, 2011[2]), the *Rock Manual* (CIRIA et al, 2007) and *Collaroy–Narrabeen Beach Coastal Protection Works Design Specifications* (Northern Beaches Council, 2016). Other documents considered are also listed in subsequent sections.

[2] Specifically, Burcharth and Hughes (2011).

Separate reports from the structural engineer (**Appendix C**) and geotechnical engineer (**Appendix D**) list adopted engineering principles, codes and standards applied to the design of the protection works (including the piling, concrete capping beam, anchors and excavations) from the perspective of these disciplines respectively.

2.4 Item 1d: Proposed Life of Structure

A design life of 60 years has been proposed for the protection works (that is, at the year 2077). As outlined in Horton et al (2014) and Horton and Britton (2015), this design life is considered to be appropriate in relation to beachfront development (that relies on the protection works for protection against erosion/recession over the design life) as:

- it is consistent with Australian Standards applying to the residential development landward of the protection works:
    - in *AS 3600*, a 50 years ± 20% design life[3] (that is, 40 year to 60 years) is used in devising durability requirements for concrete structures;
    - in *AS 2870*, for design purposes the life of a structure is taken to be 50 years for residential slabs and footings construction;
    - in *AS 1170.0*, the design life for normal structures is generally taken as 50 years; and
    - in *AS 4678,* the design life for earth-retaining structures (structures required to retain soil, rock and other materials) is noted as 60 years for river and marine structures and residential dwellings.
- the cost of new residential development is amortised for tax purposes over 40 years based on Subdivision 43-25 of the *Income Tax Assessment Act 1997*; and
- a design life of at least 50 years would be considered to be reasonable for permanent structures used by people (AGS, 2007a, b).

A minimum 60-year design life was adopted in the *Collaroy–Narrabeen Beach Coastal Protection Works Design Specifications* (Northern Beaches Council, 2016). As noted therein, this design life recognises, among other things, that redevelopment of beachfront properties typically occurs within such a period. In practice, rock incorporated within the proposed works would have a considerably greater life than 60 years based on the durability criteria specified on the drawings in **Appendix B**.

2.5 Item 1e: Adopted Design Probability and Risk Used in the Design

A 100-year Average Recurrence Interval (ARI) storm event has been adopted for design. This exceeds the minimum 50-year ARI requirement in Northern Beaches Council (2016) adopted in the particular case of Collaroy-Narrabeen Beach, which has a similar coastal storm exposure to Wamberal. Sensitivity testing was also undertaken with a 50 year ARI storm as discussed in Section 2.10.

It is important to understand that adoption of the 100 year ARI event for design actually leads to a much rarer storm being able to cause "failure" of the rock revetment. This is because:

- the revetment design is to the 0-5% damage level (generally referred to as the "no damage" condition, whereas failure is generally considered to be the 20% damage level;
- the required rock mass is governed by wave height, which is depth limited;

[3] Period for which a structure or a structural member is intended to remain fit for use for its intended purpose with appropriate maintenance.

- the revetment is only subject to the design wave at the end of the design life, after projected sea level rise (the highest possible water level) has been realised;
- for most of the design life the design wave height cannot occur as it is depth limited;
- water levels only increase slightly as ARI's become exponentially rarer; and
- rock structures can accommodate some damage without failure.

Considering a single parameter (such as wave height, or water level), a 100-year ARI event has a 45% probability of occurring over a 60-year life. However, this event for a single parameter only has a 1% probability of occurring in Year 60 of the design life, which is the only year when the design wave height would be physically able to occur due to being depth-limited in earlier years[4].

In Australian Standard *AS 4997-2005*, a design life of 50 years is recommended for normal maritime structures (specifically excluding rock structures), in conjunction with a 500 year ARI design wave height (this event has a 10% probability of occurring over the design life). However, this does not apply to rock structures (both explicitly within the Standard, and as explained by the logic above). The design of the concrete and piling components is governed more by scour level and retained land level than directly by wave height. That stated, due to depth-limited conditions it is reiterated that the design wave height cannot occur until the last year of the design life, such that the selection of a 100 year ARI design event is considered to be conservative for the overall protection works.

2.6 Item 1f: Ocean Water Levels

Based on Department of Environment, Climate Change and Water (2010a), the 100-year ARI ocean water level (in the absence of wave action) as of 2010 is 1.44m AHD. This is also consistent with Manly Hydraulics Laboratory (2016a).

As noted in Table 23 of the *Gosford Beaches Draft Coastal Zone Management Plan* (CZMP), the adopted sea level rise values therein and adopted formally by the then Gosford Council (relative to 2015) are 0.39m at 2070 and 0.74m at 2100. These are considered to be reasonable values based on information that has been peer-reviewed and widely accepted by scientific opinion, as outlined in Coastal Environment and Whitehead & Associates (2015). Linearly interpolating for the 60-year design life at 2077 (which is conservative given the projected non-linear increase in rate of sea level rise towards 2100), this gives a sea level rise (relative to 2015) of 0.47m.

Increasing the 100-year ARI water level above of 1.44m AHD by 4mm/year for actual sea level rise between 2010 and 2015[5], the corresponding value at 2015 is 1.46m AHD.

Therefore, the 100-year ARI still water level at 2077 adopted herein is 1.93m AHD.

Wave setup, caused by breaking waves adjacent to a shoreline, can also increase water levels, as discussed further in Section 2.7.

[4] There is a 10% probability of the design storm occurring in any of the last 10 years of the design life, again only considering a single parameter. However, it should be noted that the sea level rise value adopted (see Section 2.6) is not a certain occurrence, and has an associated probability of exceedance that can be approximately estimated in the order of 50%. Similarly, the scour level adopted (Section 2.7) is not a certain occurrence. That is, use of a 100 year ARI wave height and water level cannot be used to directly imply particular encounter probabilities for structural damage due to the multiple probabilistic factors at play.
[5] As discussed in Coastal Environment and Whitehead & Associates (2015).

2.7 Item 1g: Wave Heights

Extreme value offshore wave conditions have recently been re-evaluated for Sydney (also applicable to Wamberal) by Louis et al (2016), based on offshore Waverider buoy records. They determined 100-year ARI offshore significant wave heights (H_s) of 9.5m and 8.7m for 1 hour and 6 hour durations respectively.

Assuming that wave setup is 10% of the 100 year ARI offshore H_s (consistent with PWD, 1994), which is an appropriately conservative rule of thumb for waves breaking a plunging distance from the proposed protection works, this gives 0.95m and 0.87m setup for 1 hour and 6 hour durations respectively. Conservatively adopting the 1 hour duration value herein[6], the 100 year ARI total water level (including wave setup) at 2077 (at a plunging distance from the proposed works) is 2.9m AHD.

A present-day design back-beach scour level (away from structures) of -1m AHD is considered to be reasonable. Beach scour at a plunging distance from the proposed works is considered unlikely to be below -2m AHD over the design life, taking long term recession over the design life into account (see Section 5 and Section 6.1) as causing a lowering of scour levels by 1m. Therefore, the design water depth in 2077 is estimated to be 4.9m.

The method of Goda (2010) for incipient breaking of significant waves (Equation 8 therein) was employed with the following parameters:

- water depth of 4.9m as defined above;
- L_o of 264m based on a wave period of 13s; and
- beach slope of 1:33 which matches the slope of the upper profile down to about 5m depth presented in Appendix L (Figure L9) of WorleyParsons (2014).

This gave an H_s for incipient breaking of 2.9m, with a breaker index of 0.6. For a beach slope of 1:43 (the slope out to the depth of closure), H_s reduces to 2.8m, while for a beach slope of 1:20 say (about double this slope), H_s increases to 3.2m.

2.8 Item 1h: Toe Scour Levels

Immediately adjacent to the proposed piled wall, a scour level of -3m AHD has been adopted for structural design purposes. This is based on an additional scour of 1m adjacent to the wall, and another 1m of additional scour due to long term recession as discussed above in Section 2.7. This scour level is considered to be conservative and would only be short-term if realised, prior to post-storm beach recovery.

2.9 Item 1i: Crest Levels

A crest level of 6.0m AHD has been adopted for the proposed revetment, generally consistent with or slightly higher than current backyard land levels at the proposed landward edge of the revetment. Crest levels of rock revetments are readily adaptable to raising over time should this be required.

The capping beam has a crest level of 2.5m AHD.

[6] The value is also conservative as it ignores the fact that the subject properties may not be fully exposed to the offshore storm wave climate. Furthermore, note that Manly Hydraulics Laboratory (2016b) considered that the 6-hour duration was more appropriate to use.

2.10 Item 1j: Type of Hydraulic Stability Assessment and Parameters Adopted

2.10.1 Adopted Design

The type of hydraulic stability assessment used to underpin the proposed design and parameters adopted are outlined below. Note that this has been assumed to apply to the rock revetment only (that stated, the piling and concrete capping beam has been designed, among other things, for the hydraulic loading from waves).

As described in CIRIA et al (2007), a modified Hudson equation as proposed by van der Meer (1988)[7] was applied with parameters as follows:

- H_s of 2.9m (from Section 2.7);
- (basalt) rock density of 2,650kg/m^3;
- (sea) water density of 1,025kg/m^3;
- K_D (stability coefficient) of 4.0 (permeable core[8], with a damage level of 0-5%);
- structure slope of 1:1.5 (vertical:horizontal); and
- S_d of 2 as applies for 0-5% damage.

This resulted in a primary armour median rock mass of 3.8 tonnes (typical dimension 1.3m). The range of primary armour, secondary armour and second underlayer rock masses (and typical dimensions) adopted are specified on the drawings in **Appendix B**.

2.10.2 Sensitivity

As the required rock mass is sensitive to the wave height selected, note that using a higher H_s of 3.2m (as per the 1:20 slope in Section 2.7) would lead to a median primary armour rock mass of 4.9 tonnes. However, it is conservative to design a rock revetment using 100 year ARI parameters that can only occur at the end of the design life, as discussed above in Section 2.5. For a more realistic 50 year ARI event, parameters are as follows:

- 2010 ocean water level (in the absence of wave action) of 1.41m AHD;
- as above at 2015 equal to 1.43m AHD;
- as above at 2077 equal to 1.90m AHD;
- offshore H_s for 6-hour duration of 8.2m;
- (10%) wave setup of 0.82m;
- total water level (including wave setup) at 2077 of 2.7m AHD;
- scour level of -1.5m AHD;
- water depth in 2077 of 4.2m; and
- H_s for incipient breaking of 2.5m for a slope of 1:33 (and 2.7m for 1:20), giving primary armour median rock masses of 2.5 tonnes and 3.2 tonnes respectively.

To obtain the adopted median rock mass of 3.8 tonnes for the 50 year ARI parameters, a steeper beach slope of 1:16 would have to be input to the calculations, but this relatively steep slope is unlikely to be sustained under storm conditions when the beach face tends to flatten due to erosion. That is, the adopted 3.8 tonne mass design is considered to be robust for a range of adopted beach slopes, and suitably conservative.

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[7] Equation 5.135 of CIRIA et al (2007).
[8] Applicable when no geotextile is used.

2.11 Item 1k: Estimated Level of Damage (and what Form this will take) Anticipated over the Proposed Life of the Structure

As a 0-5% damage level has been adopted for the rock revetment, if the design event occurs over the design life (particularly towards the end of the design life when projected sea level rise may have been realised), then there may be some movement of up to 5% of the rocks in the primary armour layers of the structure. This does not represent failure of the rock structure, or impact on the structural stability of the protection works, indeed it is often known as the "no damage criterion". Furthermore, the piled toe would reduce the potential movement of rocks.

An inspection of the structure would be undertaken following storm events by a suitably qualified engineer in accordance with an approved revetment maintenance management plan. Any rocks that had moved would be appropriately repositioned.

Other components of the protection works (concrete, piling, anchors) would not be expected to be significantly damaged for the design event over the design life.

2.12 Item 1l: Any Movement (or Spread/Migration) of the Structure that could be Anticipated over the Proposed Life of the Works

No global movement of the protection works would be expected over the design life, if the design event occurred over that life. The geotechnical engineer has determined that the protection works have a factor of safety exceeding 1.5 against global slope stability failure.

As discussed in Section 2.11, there may be some movement of up to 5% of the rocks in the primary armour layers of the structure if the design event occurs towards the end of the design life after projected sea level rise may have been realised. This should not be considered to be "movement, spread or migration" of the structure itself, and can be managed as part of an approved revetment maintenance management plan.

2.13 Item 1m: Tying in at North and South

This was not listed as a specific item by the Coastal Panel, but has been denoted separately for clarity herein. The request was as follows:

> "The engineering report should also detail how the proposed structure will tie in to existing structures to the north and south of the proposed development, and how the expected differences in engineering standards will be managed with respect to the considerations outlined in s55M of the *Coastal Protection Act 1979*".

There are no existing structures to the north of the proposed works. A vertical wall would be constructed along a portion of the northern boundary of 25c Ocean View Drive. The crest of this wall would follow the sloped upper profile of the rock revetment and the existing land levels landward of the revetment. This would maintain the integrity of the protection works for the design event at the northern end, even if severe storms cause erosion into The Ruins.

The proposed revetment would have a similar slope and crest elevation to the adjacent Manyana revetment to the south, enabling it to 'tie-in'. Where possible, the piled toe would return a short distance along the southern boundary of 29 Pacific Street to assist in maintaining the integrity of the proposed works.

It is recognised that the Manyana works would be of a lower standard than the proposed works, although the proposed works would reduce the risk of damage to the Manyana works (from outflanking). Any failure of the Manyana works may cause some damage to the southern portion of the proposed works, which would be made good as part of an approved revetment maintenance management plan.

3. ITEM 2: PROPOSED CONSTRUCTION METHOD, INCLUDING ACCESS ARRANGEMENTS TO WORKS SITE AND CONSENT REQUIREMENTS

In general terms, the construction sequence and method would be as follows (also reproduced in Section 4.1 of **Appendix D**):

1. Preparation of a Construction Environmental Management Plan (CEMP) and Construction Methodology Plan (CMP).
2. Review and approval of the CEMP by the relevant statutory bodies and review and approval of the CMP by the project coastal, structural and geotechnical consultants.
3. Complete dilapidation surveys and detailed condition surveys of the neighbouring properties and, if required, dilapidation surveys of the subject properties.
4. Establishment of appropriate construction zone fencing/traffic control, etc to Council requirements.
5. Geotechnical consultant to complete a piling rig working platform design based on information supplied by the piling contractor.
6. Excavate along seaward property boundary to remove any obstructions (boulders etc).
7. Reinstate sand up to piling working platform level (approximately the underside of the capping beam) in accordance with geotechnical advice.
8. Form a sand bund seaward of works site.
9. Form the batter slope within the seaward portion of the subject properties to enable rock revetment construction. Excavated sand to be placed on the beach seaward of the works.
10. Install contiguous pile wall and landward ‘plug’ piles down to design toe levels.
11. Form and pour concrete capping beam.
12. Install and test permanent ground anchors.
13. Place second underlayer of revetment (0.4m thick) over the batter slope.
14. Place 2 layers of secondary armour rock (1m thick).
15. Place 2 layers of primary armour rock (2.3m thick).
16. Wash sand into the revetment to fill voids, fill interstitial voids at the primary armour crest with rip rap and place geotextile over the revetment crest as a foundation for the reinstated lawn areas
17. Reinstate remaining seaward portion of rear yard areas within the subject properties, including establishing lawn areas landward of the rock revetment
18. Replace sand mounded seaward of the pile wall back over the revetment as required. Where necessary, material would be screened to remove any inclusions.
19. Planting of the sand dune formed over the boulder revetment with suitable dune vegetation.
20. Post construction dilapidation survey.

As noted previously in the DA, access to the site for construction plant and equipment would be via the southern end of Pacific Street then along Wamberal Beach. Trucks, piling rigs and other plant and equipment would access the beach via the existing ramp at this location. Any damage would be made good after completion of the revetment construction works.

A Parks and Reserves Temporary Access Application to Central Coast Council would be made prior to the works being undertaken to obtain consent to access the beach for construction activities.

Prior to works being undertaken to construct the buried vertical wall along the northern boundary of 25c Ocean View Drive, temporary access over land owned by the Department of Planning and Environment will be sought. To the extent that temporary access is required over land along Wamberal Beach owned by Mary Elizabeth Brown, her heirs or assigns, that consent will also be sought prior to construction. If one or either of these consents are unable to be agreed, a neighbouring land access order will be sought from the Court.

4. ITEM 3: IDENTIFICATION OF ANY THIRD PARTIES REQUIRED TO PROVIDE OWNERS' CONSENT FOR WORKS ON, OR ACCESS ACROSS THEIR LAND

As some works may extend slightly on to the Manyana land (to ensure the integrity of the proposed works and adequate tie-in), consent from the Owners Corporation of Manyana has been obtained. It is understood that the form of that consent was not acceptable to the NSW Coastal Panel, and thus a replacement owners consent is being prepared.

Manyana is not one of the parties to the DA, as the works proposed to be undertaken there are for the sole purpose of providing sufficient integrity to the works at the subject properties, and not to upgrade the Manyana works. The proposed works are otherwise entirely within the 6 subject property boundaries.

To enable construction of the works, temporary access and construction activities would need to be undertaken on the Brown land, and the Department of Planning and Environment land at The Ruins. For example, a sand bund would need to be constructed on the Brown land to provide protection to the works area during construction, and a piling rig would partially extend on to The Ruins land to be centred over the northern boundary of 25c Ocean View Drive while installing the piles.

Consent for temporary access will be requested from the Department of Planning and Environment. To the extent that temporary access is required over land along Wamberal Beach owned by Mary Elizabeth Brown, her heirs or assigns, that consent will also be sought prior to construction. If one or either of these consents are unable to be agreed, a neighbouring land access order will be sought from the Court.

Depending on how MHWM is defined, with reference to Thompson (2016), some beach access and construction activities may also be carried out on Crown Land.

5. ITEM 4: DESCRIPTION OF COASTAL PROCESSES AND HAZARDS AFFECTING BEACH

5.1 General

For this Item, the specific request was:

> "A description of coastal processes and hazards (within the meaning of the *Coastal Protection Act 1979*), including sea level rise (being projected sea levels which have been peer-reviewed and widely accepted by scientific opinion) and other associated climate change impacts (as relevant) predicted to affect the beach in the vicinity of the proposed works".

The general coastal processes and hazards applying to Terrigal-Wamberal Beach have been outlined in WorleyParsons (2014). Specific hazards as per the *Coastal Protection Act 1979* are considered in the sections below, where relevant.

5.2 Beach Erosion and Shoreline Recession

As also provided in the original DA, coastline hazard lines at the subject properties defined in 2014 at the landward edge of the Zone of Slope Adjustment (ZSA), and for Immediate, 2050 and 2100 planning periods, are depicted in Figure 1. This shows that beach erosion can extend into private property, well landward of the public beach and Brown land, in severe coastal storms at present. Beach erosion would extend further into private property as any long-term recession is realised, unless protection works (as proposed) are constructed.

With regard to long term recession, WorleyParsons (2014) adopted a long-term recession rate due to net sediment loss of 0.2m/year. They did not specifically identify what may be causing that recession (only listing potential processes identified in previous investigations). Review by the author of the photogrammetric data on which this was based would suggest that anthropogenic influences on historical beach behaviour, and data limitations, were not completely considered in this assessment. That is, it is considered that there is unlikely to be significant long term recession due to net sediment loss occurring at Terrigal-Wamberal Beach (so sand eroded off the subaerial beach in coastal storms thus generally returns to the subaerial beach under calmer conditions). This is further supported by:

- the Office of Environment and Heritage (OEH) review of the original DA, dated 19 December 2016, in which it was stated that "Wamberal Beach is not currently experiencing significant long term recession and so the immediate impact of the structure on beach access and amenity would be expected to be only temporary following storm events[9]"; and
- the position of the 1886 MHWM and 1917 MHWM depicted in Thompson (2016), which is about 50m and 40m landward respectively (on average) of the current MHWM based on an elevation of 0.5m AHD, thus indicating substantial long term progradation of Wamberal Beach over the last 130 or so years.

Sea level rise has been considered in Section 2.6 herein. With regard to long term recession due to sea level rise, WorleyParsons (2014) adopted an inverse slope of the active beach profile of 43. Therefore, with 0.47m of sea level rise to 2077, this would cause 20m of recession based on the Bruun Rule. The effect of long term recession on beach profiles, in relation to the proposed works, is considered further in Section 6.1.

[9] Note that with the proposed works now entirely within the subject property boundaries, the works would not have any effect (even temporary) on public beach access (or more correctly, public access along both the Brown land and Crown Land) seaward of the properties.

Figure 1: Coastline hazard lines and Building Line at subject properties

5.3 Coastal Lake or Watercourse Entrance Instability

The ocean entrances to Terrigal Lagoon and Wamberal Lagoon are located about 300m south and 1,000m north of the subject properties respectively, and any entrance instability would not directly impact on the properties. As noted by WorleyParsons (2014), the Lagoons are unlikely to be significant in terms of the overall sediment budget.

5.4 Coastal Inundation

Extreme ocean water levels were considered above in Section 2.6, and also in Section 2.7 including wave setup. Individual waves also cause temporary water level increases above the still water level due to the process of wave runup or uprush.

WorleyParsons (2014) estimated an approximate present-day wave runup level of 6m AHD in the vicinity of the subject properties. Wave runup is not currently a significant issue at the properties as development is either setback well landward (4 northern properties), and/or ground levels rise moving landward (all properties), and/or ground floor levels are well elevated (all properties).

6. ITEM 5: EFFECT OF COASTAL PROCESSES AND HAZARDS ON STRUCTURE, AND IMPACT OF STRUCTURE ON COASTAL PROCESSES AND HAZARDS

6.1 Item 5a: Potential Effect of Coastal Processes and Hazards on Structure

The potential effect of coastal processes and hazards on the proposed protection works is considered in this Section.

The protection works have been designed to withstand (with 0-5% damage of the rock revetment) a 100 year ARI wave and water level event (leading to corresponding beach erosion) occurring at the end of a 60-year design life (thus incorporating long term recession due to sea level rise), including scour down to -3m AHD. Therefore, beach erosion up to the 100 year ARI event and projected long term recession over a 60-year life have been accommodated in the design, and there would be expected to be no structural damage to the works due to the effect of coastal processes and hazards for this design event and life. Severe beach erosion would cause sand to be eroded from seaward and over the works, which would naturally return over time in post-storm beach recovery (although in the long term, sand volumes seaward of the works would diminish over time caused by recession due to sea level rise, unrelated to the works).

The effects of Terrigal Lagoon and Wamberal Lagoon (and any related entrance instability) on the proposed works would be expected to be insignificant due to their distance from the works.

Coastal inundation, wave runup and any wave overtopping would not be expected to damage the proposed works for the design event and life. Any damage of areas landward of the works due to wave overtopping would be expected to be relatively minor, potentially requiring some restoration of landscaped areas such as lawns, but not an issue in terms of structural stability for dwellings or the protection works themselves. Wave overtopping is considered further in Section 7.

6.2 Item 5b: Likely Impacts of Proposed Structure on Coastal Processes and Hazards (Including Items 5c and 5d)

6.2.1 Preamble

The likely impacts of the proposed protection works on coastal processes and hazards are considered in this Section. To assist in this, cross sections depicting historical beach profiles (and also shifted landward to account for long term recession over the design life) in relation to the proposed position of the works are provided in Section 6.2.2, with general discussion provided in Section 6.2.3.

As per Item 5c, this response is to "include details of the extent to which the proposed structure will be exposed from lowered beach conditions over the course of its proposed life and how this will affect public access and beach usage" (see Section 6.2.4). Furthermore, as per Item 5d, this response is to "provide estimates of the impacts of the proposed structure on the beach's sediment budget, including through storm erosion, underlying recession and projected sea level rise over the design life" (see Section 6.2.5).

6.2.2 Beach Profiles in Relation to Proposed Works

As discussed in the original DA, OEH has collected historical beach profile elevations derived from aerial photography (photogrammetric data) along Wamberal Beach. Dates that have been assessed were in 1941, 1954, 1965, 1969, 1973, 1974, 1977, 1978, 1985, 1986, 1990, 1993, 1996, 1999, 2006, 2008 and 2016[10]. The location of the photogrammetric profiles in the vicinity of the subject properties is provided in Figure 2, with profile numbering from 1 to 5 designated for use herein.

Beach profiles for each photogrammetric date, as well as from a 24 June 2016 survey undertaken after the June 2016 storm (**Appendix A**), are depicted in Figure 3 to Figure 7 for Profile 1 to Profile 5 respectively. These Figures are repeated, but with the beach profiles translated landward by 20m to approximately account for long term recession[11], in Figure 8 to Figure 12[12].

The positon of the proposed works is shown on each Figure. In the legend of each Figure, the pre-storm 5 April 2016 date is designated as "2016a", while the post-storm 24 June 2016 date is designated as "2016b". Note that the 1954 date is certainly in error for Profile 1 and Profile 2, and has not been considered in the analysis.

For the receded profiles, some profiles had to be extrapolated seaward by continuing the profiles at the same slope as the previous 2 data points that were sloping down moving seaward. This was undertaken for all 1974, 1978 and 2016b profiles[13].

[10] The 2016 date is additional to the dates assessed in the original DA. It was pre-storm, ie before June 2016, collected on 5 April 2016.
[11] This ignores any potential for structure-beach interaction. Carley et al (2015) gave an example of using SBEACH numerical modelling for a structure-beach interaction assessment, but as the model cannot be relied upon in this regard, and does properly represent the physics of a reflected wave, this was not considered to be warranted.
[12] Of course, in reality these profiles would not be realised landward of the works as the works themselves would form the future profile within the subject properties.
[13] Except for Profile 3 in 2016b, which was extrapolated at a 1:11 slope (average of all profiles), since using the previous 2 data points was not appropriate due to being overly steep and more representative of the upper dune.

Figure 2: Location of OEH photogrammetric profiles in vicinity of subject properties (zero chainage is at landward end)

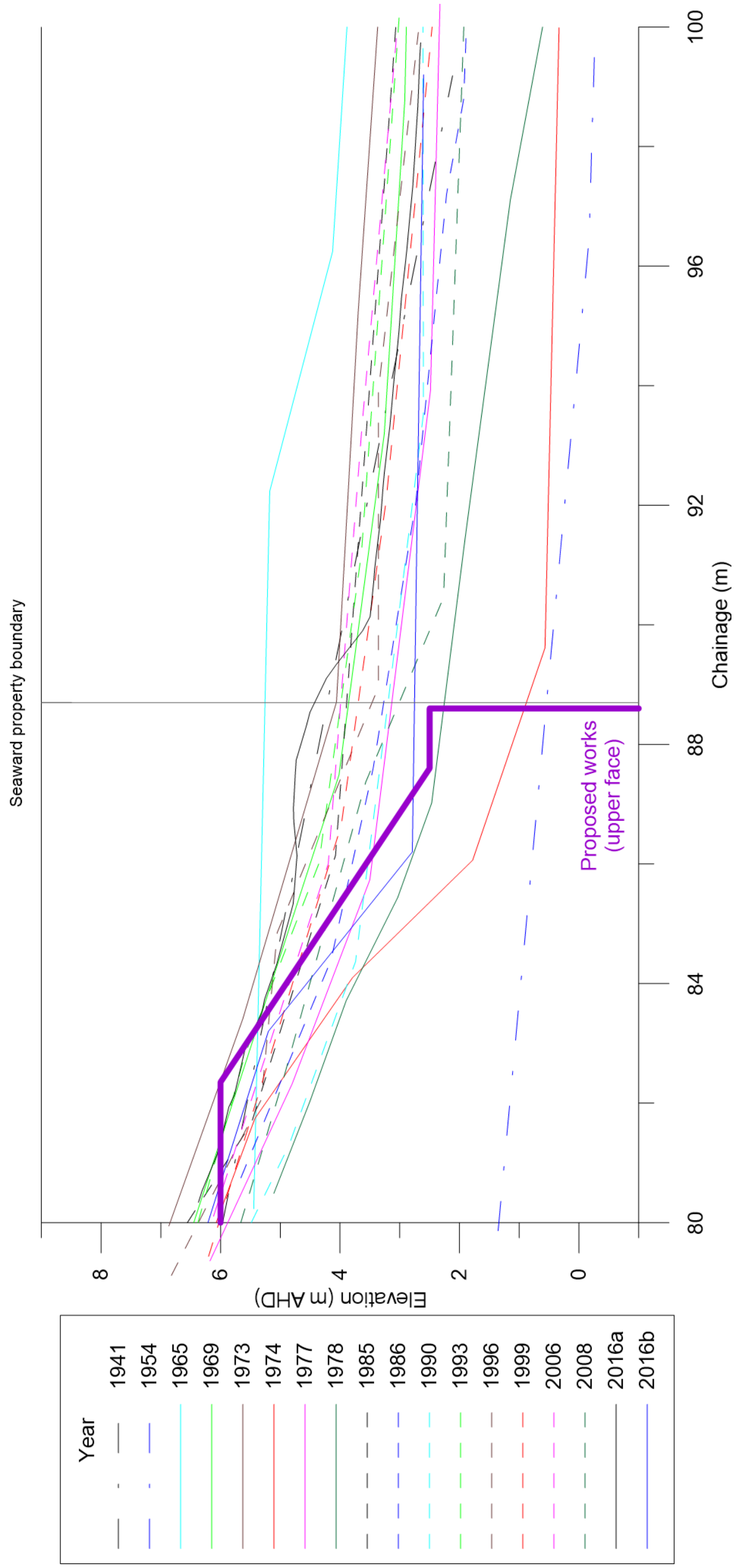

Figure 3: Historical beach profiles at Profile 1, with outline of upper face of proposed works shown

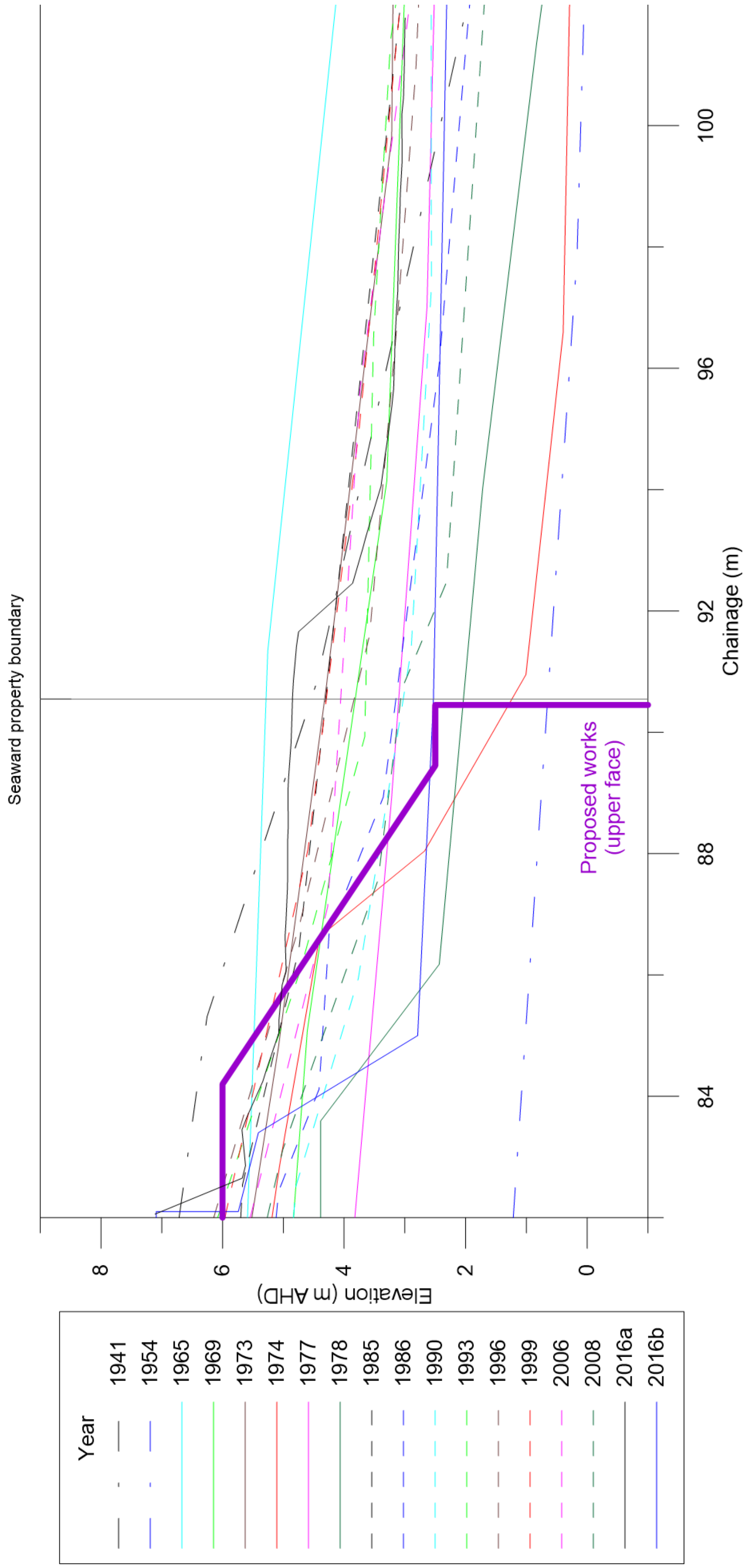

Figure 4: Historical beach profiles at Profile 2, with outline of upper face of proposed works shown

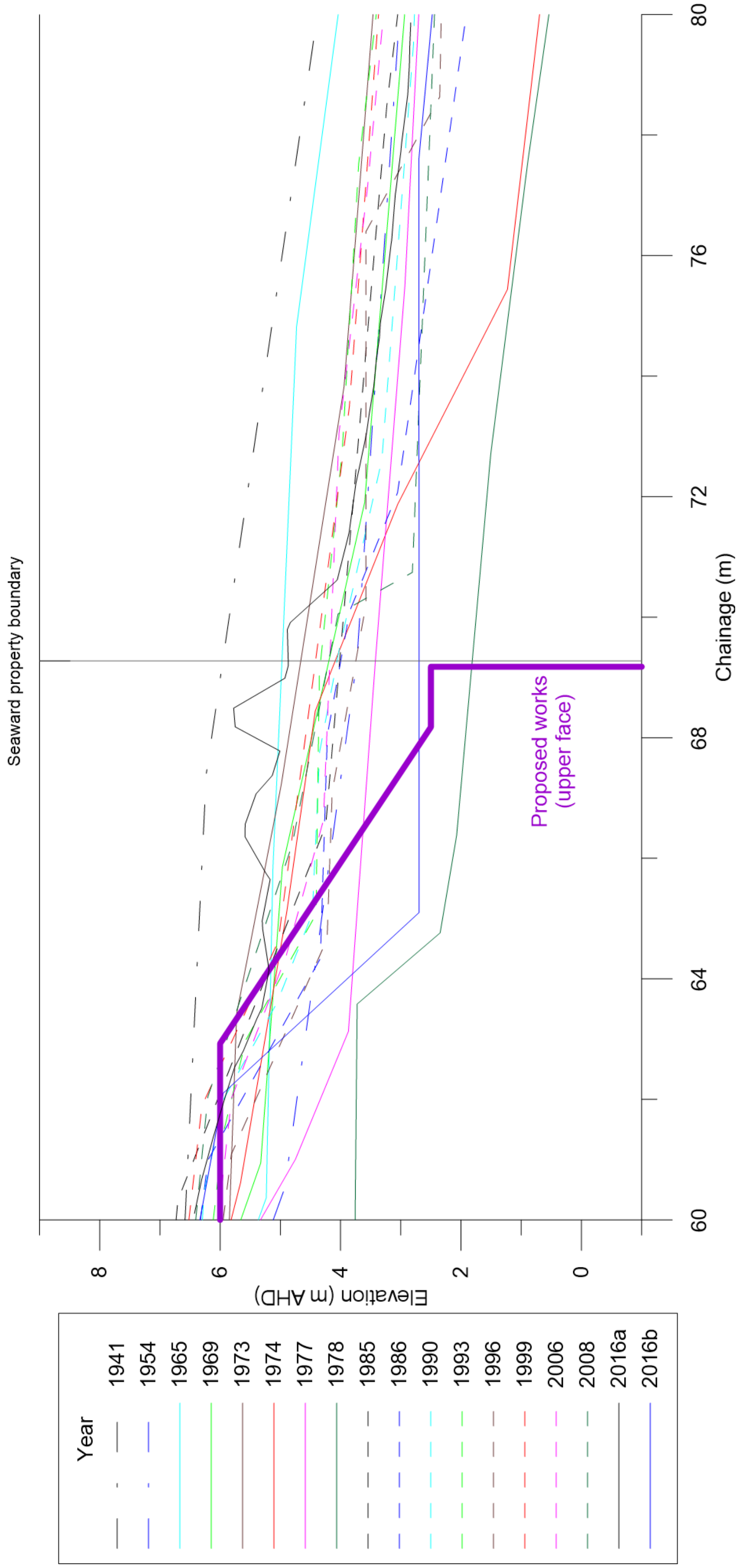

Figure 5: Historical beach profiles at Profile 3, with outline of upper face of proposed works shown

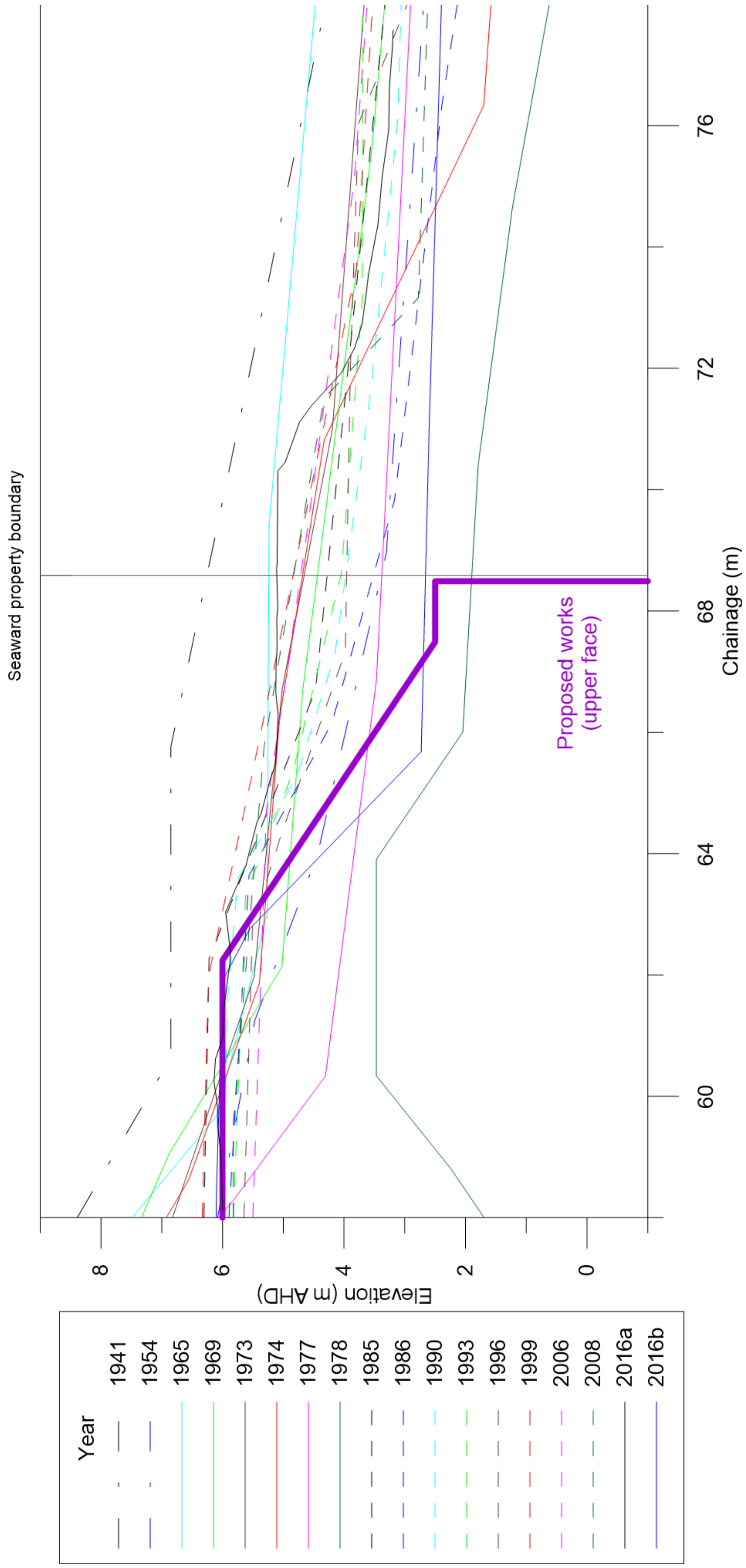

Figure 6: Historical beach profiles at Profile 4, with outline of upper face of proposed works shown

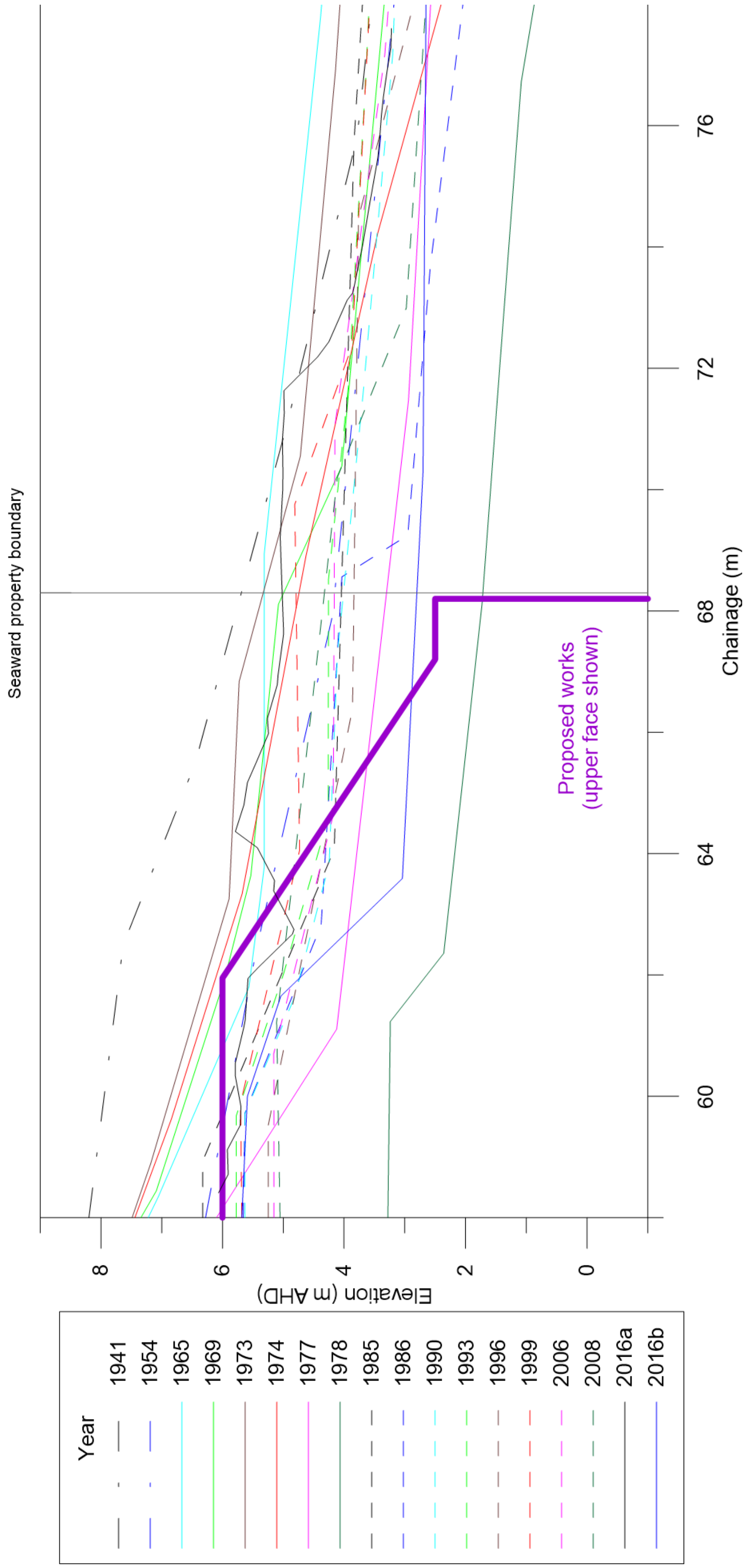

Figure 7: Historical beach profiles at Profile 5, with outline of upper face of proposed works shown

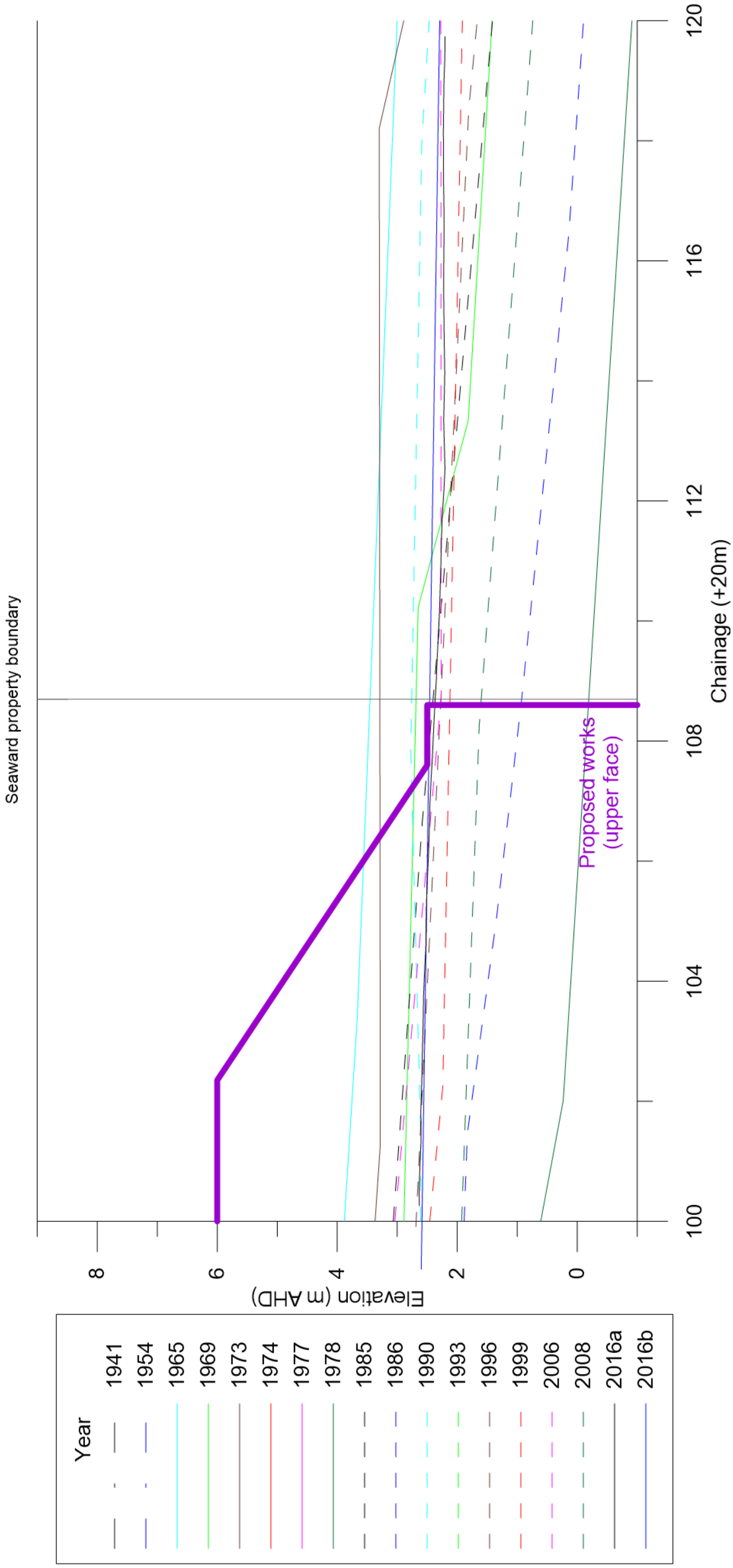

Figure 8: Receded beach profiles at Profile 1, with outline of upper face of proposed works shown

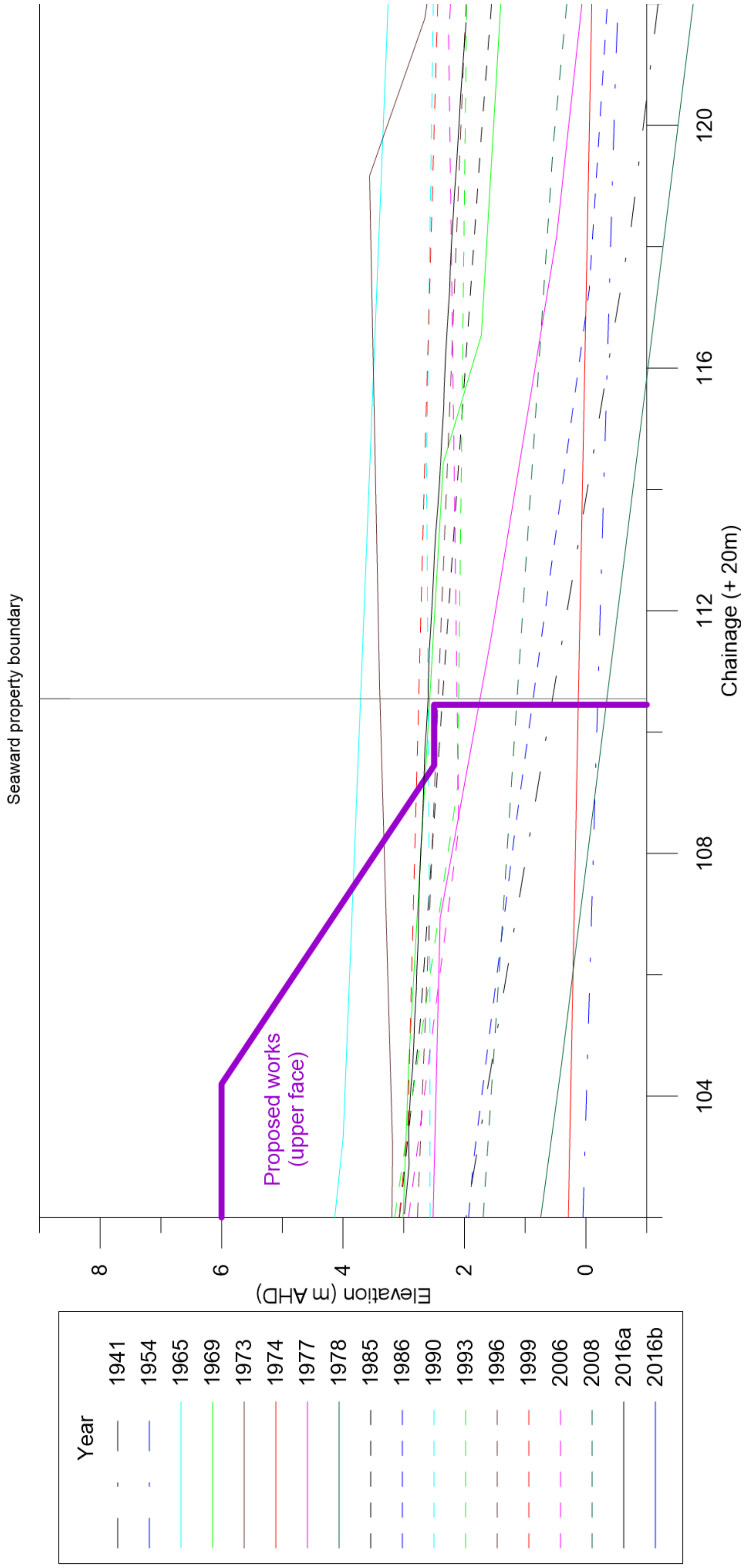

Figure 9: Receded beach profiles at Profile 2, with outline of upper face of proposed works shown

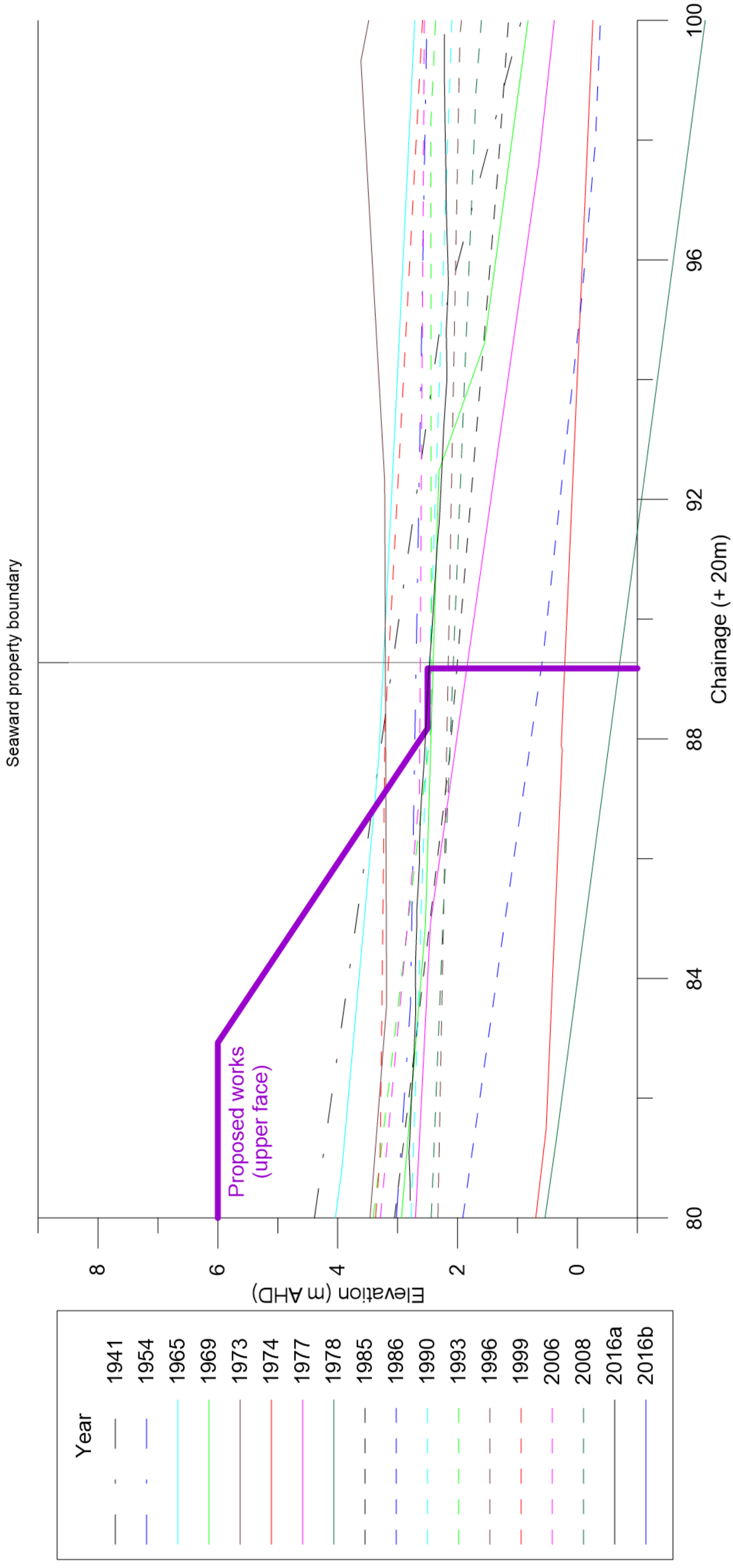

Figure 10: Receded beach profiles at Profile 3, with outline of upper face of proposed works shown

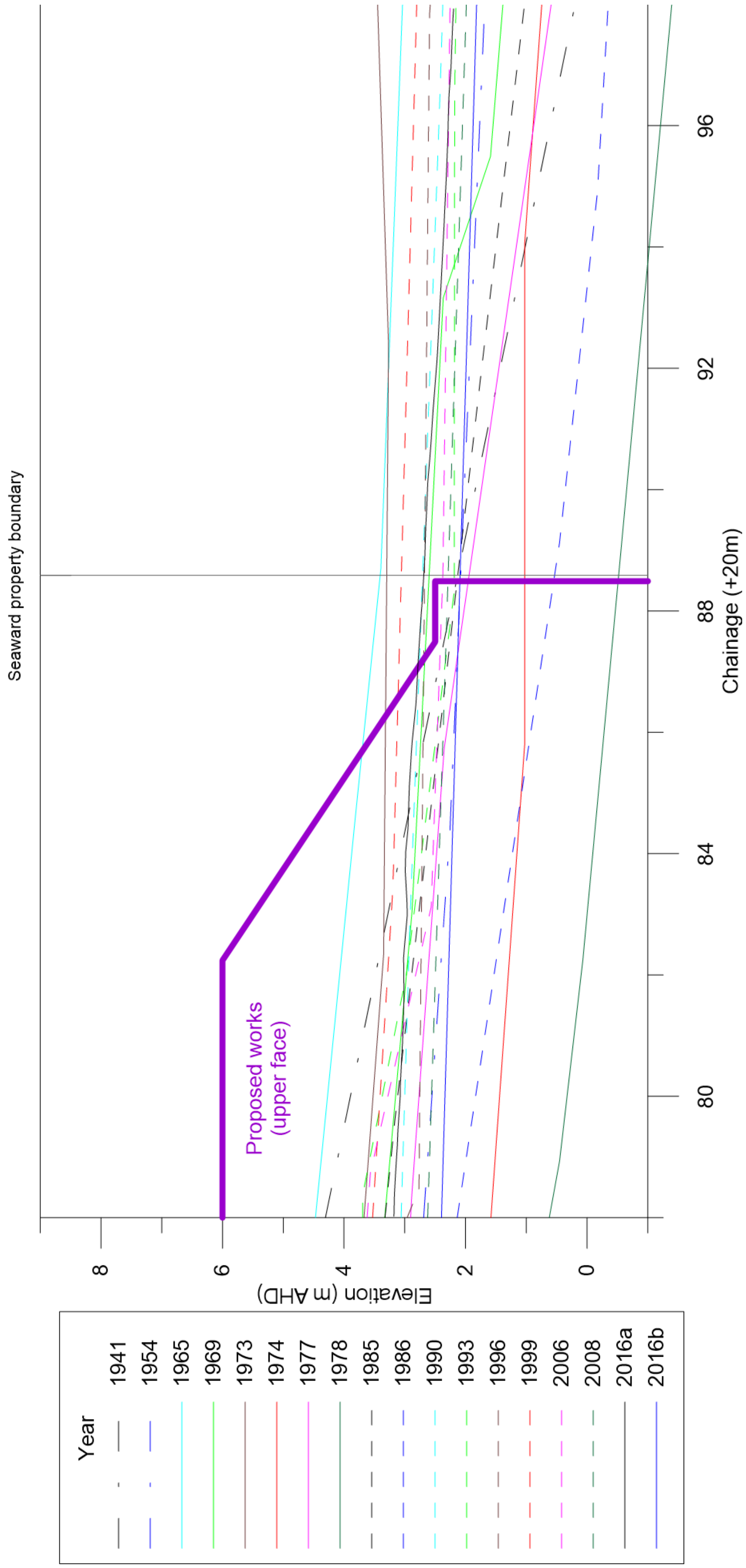

Figure 11: Receded beach profiles at Profile 4, with outline of upper face of proposed works shown

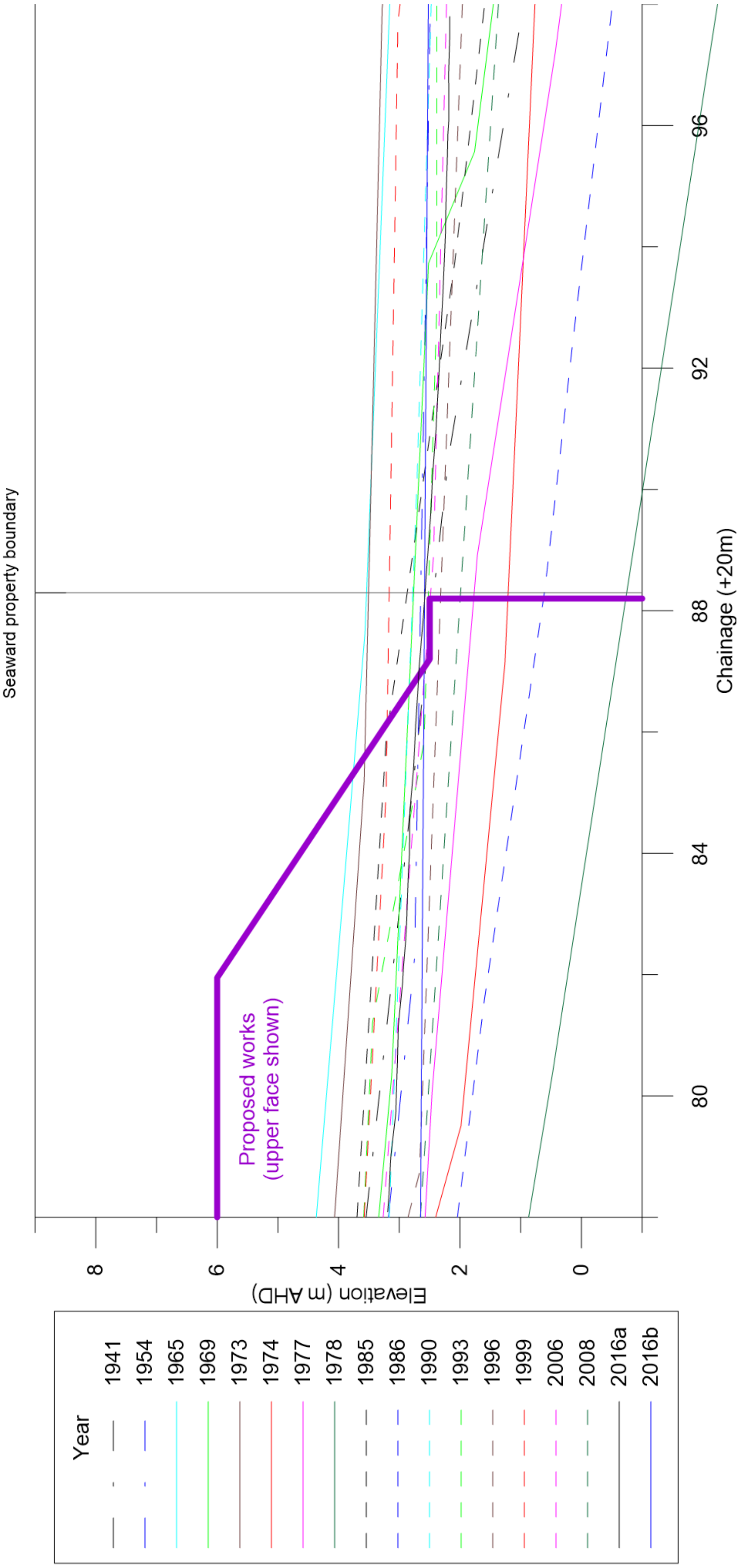

Figure 12: Receded beach profiles at Profile 5, with outline of upper face of proposed works shown

A summary of the number of historical profiles (out of 18) with the piled toe exposed[14], and the typical sand level on the structure (had it been constructed) is provided in Table 1. This is provided for both the historical profiles as is, and the historical profiles with 20m recession (landward translation) applied (that is, potential future receded profiles).

Table 1: Exposure of piled toe and typical sand level on structure for historical and future receded profiles

Profile	Number of profiles (out of 18) with piled toe exposed, had it been constructed		Typical sand level on structure (m AHD), had it been constructed	
	Historical profiles	Receded profiles	Historical profiles	Receded profiles
1	2	12[15]	5	2.5
2	2	11[16]	4.5	2.5
3	1	7[17]	5	2.5
4	1	10[15]	5.5	2.5
5	1	6[15]	5.5	2.5

It is evident that based on historical profiles, the piled toe would be buried under sand for most of the time, with only the most severe storms exposing it[18]. The typical sand level over the revetment, based on historical profiles, would be about 5m AHD, although formation of a vegetated dune over the revetment would assist in encouraging sand levels to be maintained at a higher elevation.

6.2.3 General Discussion

The proposed protection works would limit the landward extent of beach erosion, preventing storm demand from extending into private property as it would without the works being constructed. The works would have no significant long term effect on the amount of sand eroded off the Brown Land or Crown Land in coastal storms seaward of the works. The only land being "locked up" (prevented from eroding) by the proposed works is the land which is within the subject property boundaries, which has been lawfully subdivided and registered in title by the NSW Government, and for which the owners are subject to Council rates and other government levies such as land tax as applicable.

The works may cause end effects (additional erosion) to the north of the works at The Ruins until protection works are constructed there, but would not cause any significant additional erosion to the south of the works due to the existing potential end effects caused by discrete protection works to the south (see Section 8 for further discussion on end effects).

The proposed protection works would also limit the landward extent of shoreline recession (exactly as whole-of-beach protection works would, which have been the recommended and

[14] Defined as including the capping beam, that is any exposure below 2.5m AHD at the seaward property boundary.
[15] However, only 3 profiles exceeded 0.5m of exposure.
[16] However, only 7 profiles exceeded 0.5m of exposure.
[17] However, only 4 profiles exceeded 0.5m of exposure.
[18] Based on the proportion of profiles exposed (1 or 2 out of 18), the frequency of exposure of the piled toe would be about 5-10%. However, in reality, the proportion of time that the piled toe would be exposed would be lower, as a number of photogrammetric dates have been selected on the basis of capturing post-storm profiles (that is, not random selection but skewed towards eroded beaches), and sand volumes recover in the lower part of the beach relatively quickly. Conservatively assuming that the 1974 and 1978 events would have taken 6 months to recover and restore sand levels up to 2.5m AHD, the proportion of time that the piled toe would have been exposed in the last 76 years would be about 1%.

adopted management option for Wamberal Beach for over 21 years[19]). Subaerial sand volumes seaward of the works would be expected to diminish over time under long term recession due to sea level rise, as they would without the works in place. Over the life of the works, this would again have no significant long term effect on the amount of sand eroded off the Brown Land or Crown Land in coastal storms seaward of the works. This is because a sufficient volume of sand would continue to exist (on average) seaward of the works over the design life such that the works would generally remain buried, as illustrated in Section 6.2.2.

The proposed protection works would have no significant effect on the stability or sediment budget of Terrigal Lagoon or Wamberal Lagoon.

The proposed protection works would limit the landward extent of wave overtopping by limiting the landward extent of erosion/recession (thus preventing wave action propagating inland), by increasing the slope permeability (at the rock revetment), and by increasing the vertical momentum of runup while reducing its horizontal momentum (at the piled toe). Further discussion on wave overtopping is provided in Section 7.

6.2.4 Item 5c

With regard to Item 5c, details have been provided of the extent to which the proposed structure will be exposed from lowered beach conditions over the course of its proposed life, both through an allowance of a scour level of -3m AHD in design (Section 2.8 and Section 6.1) and in terms of typical profiles (Section 6.2.2).

As the proposed works are entirely on private land, they would have no significant impact on public access and beach usage (or more correctly, public access along both the Brown land and Crown Land) seaward of the works. Any potential increased scour seaward of the works (piled toe) would only be short term (until relatively rapid post-storm beach recovery), and long term beach levels would not be altered as a result of the proposed works over its design life (rather, they would be altered by long term recession due to sea level rise).

6.2.5 Item 5d

Previous studies have not quantified the sediment budget at Terrigal-Wamberal Beach. Inferring from Public Works Department (1994) and WorleyParsons (2014), the Terrigal-Wamberal embayment would essentially be closed to sediment transport across its boundaries, with no significant alongshore transport in or out of the embayment.

The largest component of the current sediment budget at the beach would be cross-shore sediment transport. In a severe storm, in the order of 500,000m^3 of sand can be eroded off the subaerial beach, moving offshore in a matter of days to be slowly worked back onshore over months to years.

As discussed in Section 6.2.3, the proposed protection works would limit the landward extent of beach erosion, thus preventing the full storm demand from occurring in severe storms that would otherwise extend into private property. This would not alter the sediment budget of the sand on the Brown Land and Crown Land, but for a 100 year ARI storm demand of 250m^3/m, the works would prevent about 105m^3/m of private land being eroded, or about 10,500m^3

[19] Adoption of the protection works management option for Wamberal Beach by Council in 1995 and 2004, and as part of the draft CZMP, was made in the full knowledge of the effects of long term recession. The proposed works are essentially the initial phase of the introduction of these works.

overall over the 6 properties. This is only about 2% of the ±500,000m^3 cross-shore sediment budget for the entire beach.

As long term recession is realised, the proportion of storm demand prevented from eroding by the works (all on private property) in a 100 year ARI storm would increase to approximately 22,000m^3, or about 4% of the total cross-shore sediment budget.

A whole-of-beach protection works option, as adopted in the draft CZMP, would have far more impact on the overall cross-shore sediment budget of Terrigal-Wamberal Beach than the proposed works alone. This whole-of-beach protection works option was found to be acceptable in the "Wamberal Beach and Property Protection Environmental Impact Statement" (Manly Hydraulics Laboratory [MHL], 2003).

If protection works are to be undertaken at Wamberal Beach, as adopted as the preferred management option for over 21 years[20], then (by default) sand volumes will decrease seaward of the works as long term recession due to sea level rise is realised. Beach nourishment was the management option envisaged in MHL (2003) and the draft CZMP to avoid or reduce diminishing beach sand volumes over time, as has been adopted in other similar circumstances such as at Collaroy-Narrabeen Beach. A future loss of sand volume due to sea level rise recession is not considered a reasonable argument to prevent the protection works being completed for existing development, given the present risk to that development. Given that protection works have been adopted as the preferred management option for Wamberal Beach, it would be unreasonable to prevent the adoption of protection works on the basis of an issue that is known to inherently apply to those works and cannot be avoided except through a beach-wide (or even over multiple Council areas or State-wide) beach nourishment approach, which is also known to be impractical for individual landowners to implement.

7. ITEM 6: WAVE OVERTOPPING

For this Item, the specific request was:

> "An assessment of wave overtopping of the proposed structure and how this will be managed to ensure the safety of humans and assets located landward of the structure and the structural integrity of the protection works themselves. This assessment should include all relevant calculations to estimate the wave overtopping rates".

The Neural Network for Wave Overtopping Predictions (van Gent et al ,2007) associated with EurOtop (van der Meer et al, 2016), Version 2.04 (March 2016), was utilised to calculate average wave overtopping rates in a 100 year ARI storm at the proposed works for both present conditions and in 2077 at the end of the design life.

Input parameters are summarised in Table 2. To approximately discretise the proposed works in the model, the piled toe was treated as a berm of 1.0m width, as recommended. In this situation, the water depth in front of the structure is set to be equal to the water depth at the toe (with a present water level of 2.4m AHD and scour level of -1m AHD, and 2077 water level of 2.9m AHD and scour level of -2m AHD). Note that the mean wave period was derived as the peak spectral wave period of 13s divided by 1.1. The roughness coefficient was derived from Table 6.2 of van der Meer et al (2016) for rocks in 2 layers with a permeable core.

[20] As stated in Gosford City Council (1995), "a terminal protection structure in the nature of a buried rock revetment is to be designed and constructed to the satisfaction of Council and NSW Public Works, such construction to occur as soon as practicable and in an orderly, co-ordinated manner".

Table 2: Input parameters for Neural Network for Wave Overtopping Predictions

Parameter	Value	
	Present-day	2077
Angle of wave attack (°)	0	0
Water depth in front of structure (m), see Section 2.7	3.4	4.9
Significant wave height at the toe of structure (m), see Section 2.7	2.1	2.9
Mean wave period (s)	11.8	11.8
Water depth at the toe of structure (m)	3.4	4.9
Width of toe (m)	0	0
Roughness coefficient	0.4	0.4
Angle of down slope (cotangent)	1.5	1.5
Angle of upper slope (cotangent)	1.5	1.5
Crest freeboard in relation to SWL (m)	3.6	3.1
Berm width (m)	1.0	1.0
Water depth at the berm of the structure (m)	-0.1	0.4
Berm slope (tangent)	0	0
Armour freeboard in relation to SWL (m)	3.6	3.1
Armour width (m)	4.5	4.5

The resulting mean overtopping discharges were 0.8L/s/m for the present day, and 14L/s/m at 2077. Historically, based on the previous (2007) version of EurOtop, a 50L/s/m overtopping discharge would have been considered a threshold for damage to a grassed or lightly protected promenade or reclamation cover. That is, based on the 2007 version of EurOtop, the estimated mean overtopping discharges would not have been considered to be damaging even to grass cover over the revetment.

In the latest version of EurOtop (van der Meer et al, 2016), there is more of a focus on linking tolerable overtopping with the peak volume, and hence on the wave height that causes the overtopping, thus changing the limits for tolerable overtopping. For a grass covered crest and landward slope, maintained and closed grass cover and with H_{m0} (spectral significant wave height) of between 1m and 3 m (as applies here), a limit of 5L/s/m was adopted.

On this basis, the landscape cover (eg lawn) over the proposed works would not be expected to incur significant overtopping damage in the 100 year ARI storm at present, but would begin to do so towards the end of the design life. Landscaping damage is not considered to be a significant issue, with the owners able to restore damaged turf areas after any significant storms. The works themselves would not expected to be damaged by overtopping for the design storm over the design life, nor any of the dwellings at the subject properties.

To adapt to increasing overtopping volumes as sea level rise is realised, it would be possible to raise the crest of the revetment. At 2077, raising the crest by 0.5m reduces the mean overtopping discharge to 6L/s/m. Furthermore, in the calculations above it was assumed that only the primary armour contributed to the armour width. If the primary armour was extended to cover the secondary armour and second underlayer at the crest, this would increase the armour layer crest width to 7m, which for the proposed crest level of 6m AHD would reduce the mean overtopping discharge to 5L/s/m (in combination with raising the crest to 6.5m AHD, this would reduce the mean overtopping discharge to 2L/s/m).

Overtopping is not a significant issue at the properties in terms of inundation of dwellings, as natural ground levels rise moving landward and ground floor levels are well elevated.

Furthermore, for the 4 northern properties, development is setback well landward (about 20m) from the revetment crest.

With regard to safety of humans, a tolerable limit of 0.3L/s/m (for H_{m0} of 3m) and 1L/s/m (for H_{m0} of 2m) is noted in van der Meer et al (2016) for people at the revetment crest with a clear view of the sea. A range of 1 to 10L/s/m was adopted in the 2007 version of EurOtop for pedestrians (trained staff, well shod and protected, expecting to get wet).

For their safety, it would be necessary for people to remain several metres landward of the revetment crest in severe storms. However, the subject properties would be far more unsafe in severe storms if the protection works were not constructed.

8. ITEM 7: END EFFECTS

8.1 Preamble

For this Item, the specific request was:

> "An assessment of end effects to both the north and south of the structure –including a diagram illustrating the potential end effects of the structure.
>
> (a) Such diagram to be prepared in accordance with the methodology in McDougal et al 1987 or other reputable methods or modifications and is to include a specification of the (S), (r), (e) and (L_s) values used and how they were determined. If the shape of the end effects calculated does not conform to the shape in McDougal et al 1987 or other reputable methods, an explanation of the variation is to be provided.
> (b) Such assessment to include consideration of the cumulative impact of the structure, having regard to its proposed connection or interaction with end effect impacts of other existing structure (sic) within the active beach margins around the embayment".

Calculation of end effects considering existing protection works and the subject proposed protection works is provided in Section 8.2. Further discussion on the significance of end effects on relation to the proposed works is provided in Section 8.3. Management of end effects is discussed in Section 8.4.

8.2 Calculation of End Effects

As noted in Section 8.1, use of McDougal et al (1987) for calculation of end effects was recommended by default, although "other reputable methods or modifications" may be used. As previously applied in this matter, the methodology of Carley et al (2013) has been used to calculate end effects herein, without endorsing the method as being reasonable. McDougal et al (1987) is not appropriate to use for various reasons as outlined in Carley et al (2013), although it can be noted that for a proposed protection works length (L_S) of 98.7m, the additional erosion (r or AE) from McDougal et al (1987) is 10m and alongshore impact distance (S) is 68m, far less than determined using Carley et al (2013) as outlined below.

In the assessment below, existing protection works causing potential end effects were considered, namely at Manyana (25 Pacific Street), 19 and 21 Pacific Street[21], and 33 and 35 Ocean View Drive. Calculations using Carley et al (2013) were based on the following inputs:

- 2006 photogrammetric profiles, which are appropriate as these were the base profiles used to define the hazard line position in the CZMP, and represent a typical average beach-full position;
- 100 year ARI storm demand of 250m^3/m as defined in the CZMP, applied to 2006 profiles, with only present day conditions considered[22];
- average sand volume seaward of proposed works and above 0m AHD in 2006 of 140m^3/m (this is coincidently the same as for the original DA design, having been recalculated for the new design), averaged over the 6 photogrammetric profiles covering the revetment footprint, giving a non-dimensional volume (NDV) of 0.56 and additional erosion in the impact area (AE) of 110m^3/m or 18m based on a dune height of 6m AHD;
- this volume exceeds a 10 year ARI storm demand volume, so no potential groyne effect recession has to be assessed using a shoreline evolution model as per Carley et al (2013), with it being reiterated that only present day conditions should be assessed;
- average sand volume seaward of existing Manyana revetment and above 0m AHD in 2006 of 130m^3/m, averaged over the 2 photogrammetric profiles covering the revetment footprint, giving an NDV of 0.52 and AE of 120m^3/m or 20m based on a dune height of 6m AHD;
- proposed revetment length of 98.7m, giving an alongshore impact distance (S) of 159m;
- Manyana revetment length of 30m, giving an S value of 120m;
- L_s of 30m and NDV of 0.5 at 33 and 35 Ocean View Drive, giving an AE of 125m^3/m (21m based on a dune height of 6m AHD) and S of 120m;
- L_s of 38m and assumed NDV of 0.5 at 19 and 21 Pacific Street, giving an AE of 125m^3/m (21m based on a dune height of 6m AHD) and S of 123m; and
- excess erosion shapes based on Figure 9.2 of Carley et al (2013), by georeferencing the shape with 3 control points (at protection works, additional erosion extent with an assumption of its alongshore distance, and alongshore extent).

It is considered that end effects would not be cumulative, eg that the 33-35 Ocean View Drive end effect would not add to the end effect of the proposed works. Taking the most landward end effect when combining end effects is considered to have the most likely physical basis, rather than a summation.

The individual calculated end effects, based on Carley et al (2013), are depicted in Figure 13, along with the Immediate Hazard Line from the draft CZMP[23]. End effects calculations and the Immediate Hazard Line are depicted continuing into properties with protection works in Figure 13 to avoid confusion, whereas in reality erosion would not extend into protected properties. In Figure 14, the end effects are combined to depict the existing Immediate Hazard including end effects, and the corresponding Immediate Hazard including end effects after completion of the proposed protection works.

[21] The works at 21 Pacific Street have been extended seaward since the June 2016 storm, that is since the aerial image in Figure 13 was photographed just after the storm.
[22] Potential end effects at the end of the design life have not been considered based on the presumption that The Ruins is likely to be protected, and the fact that if a 100 year ARI coastal storm occurred at the end of the design life then dwellings at unprotected properties along Wamberal Beach would have been completely undermined without any end effects, and hence end effects become inconsequential.
[23] The yellow "combined proposed works and Manyana" end effect is initially identical to the blue Manyana end effect immediately south of Manyana, hence the blue line is not visible in Figure 13.

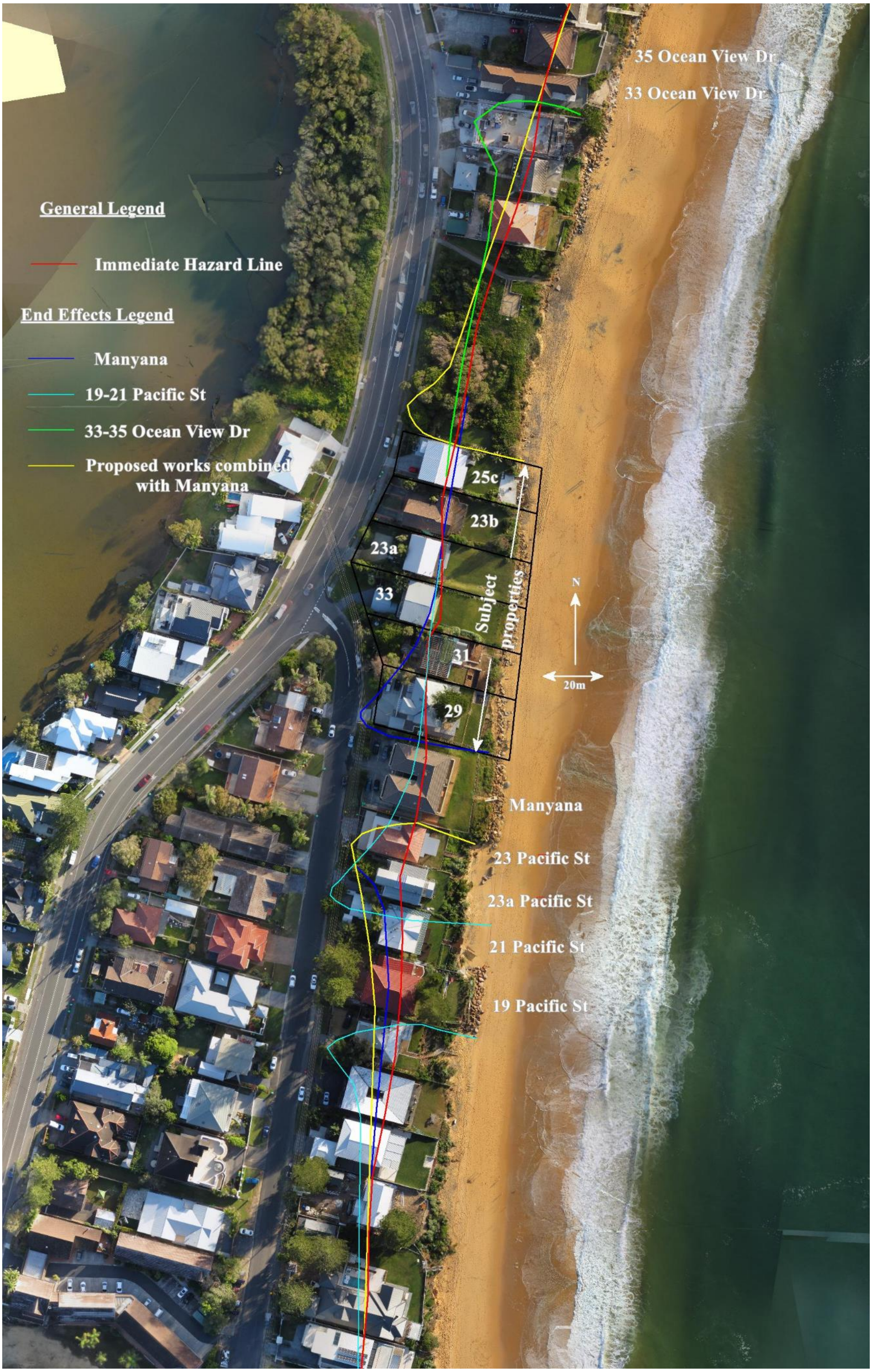

Figure 13: Individual end effects in vicinity of subject properties based on Carley et al (2013)

Figure 14: Comparison of Immediate Hazard including end effects for existing situation and after construction of proposed works, based on Carley et al (2013)

8.3 Discussion

8.3.1 General

It is recommended that broad trends are deduced from Figure 13 and Figure 14, rather than fine measurements, as the accuracy of the Carley et al (2013) is only approximate.

It is evident from Figure 14 that the potential additional end effect from the proposed works (compared to existing), based on the Carley et al (2013) methodology, would be confined to the southern end of The Ruins, the two lots south of Manyana (23 and 23a Pacific Street), and at 19-21 Pacific Street. However, with existing protection works at 19-21 Pacific Street, the end effect would not be realised there in reality. Therefore, only The Ruins and the lots south of Manyana are considered further herein.

8.3.2 The Ruins

The end effect at The Ruins is essentially identical to that presented previously for the original DA design. The existing situation is that the 6 subject properties would have dwellings undermined in the 100 year ARI storm, due to the 100 year ARI storm erosion and Manyana end effect. The proposed situation is that there would be an end effect causing additional erosion at the southern end of The Ruins, of no consequence to development. On balance, the proposed works are considered to be preferable, removing the risk of undermining from 6 dwellings while not increasing the risk of undermining of any other dwellings.

End effects can only potentially occur at The Ruins while protection works are not in place there. Should the Department of Planning and Environment choose to construct protection works along the seaward frontage of The Ruins in the future, the potential end effect of the proposed works would be nullified.

8.3.3 Lots South of Manyana (23 and 23a Pacific Street)

Any additional erosion (end effect) at a dwelling that has already been essentially entirely undermined is not of significant consequence. This is the case at these properties, as the 100 year ARI storm and end effect associated with 19-21 Pacific Street would have already destroyed the dwellings at 23 and 23a Pacific Street. The potential additional erosion at 23 and 23a Pacific Street and associated with the proposed works is inconsequential.

8.3.4 Synthesis

Using the methodology of Carley et al (2013), it has been demonstrated that the only additional erosion associated with the proposed works would be at The Ruins and at 23 and 23a Pacific Street. The proposed works are considered to be preferable, removing the risk of undermining from 6 dwellings while not increasing the risk of undermining of any other dwellings in the 100 year ARI storm.

The potential end effect at The Ruins related to the proposed works is of no consequence to development. The potential additional erosion at 23 and 23a Pacific Street is also inconsequential, as dwellings at these properties would have already been completely undermined without any additional erosion associated with the proposed works.

8.4 Management of End Effects

If the NSW Coastal Panel can devise a reasonable consent condition that the landowners have to import a volume of suitable sand to infill demonstrated additional end effects erosion (that does not infill naturally within say 6 months of a storm), that could be considered. This is only considered to be reasonable to apply if the NSW Government states that it will be constructing protection works at The Ruins, and only over the next 5 years (say), as that should be ample time for works to be designed and constructed. If the NSW Government chooses not to protect The Ruins, it is not considered to be reasonable to apply this condition.

9. ITEM 8: MAINTENANCE

For this Item, the specific request was:

> "Details of the proposed long term inspection, monitoring, management and maintenance regime both for the structure itself, as well as the monitoring and mitigation of impacts of the structure, over the life of the structure, on the adjoining beach and surrounding areas. Include details of who is proposed to be responsible to implement each regime element as well as proposed conditions offered to ensure implementation".

The following activities are considered to be the responsibility of the owners, and are put forward for discussion rather than adoption, being subject to change.

As noted in Section 2.11, an inspection of the structure would be undertaken following storm events by a suitably qualified engineer in accordance with an approved revetment maintenance management plan. Any rocks that had moved would be appropriately repositioned.

As noted in Section 2.13, any failure of the Manyana works in a storm event may cause some damage to the southern portion of the proposed works, which would be made good as part of an approved revetment maintenance management plan.

Over the next 5 years, the owners would be prepared to implement a condition that they were responsible to infill demonstrated additional end effects erosion (that does not infill naturally within 6 months of a storm). This 5-year period is intended to allow the owner of The Ruins sufficient time to consider and construct protection works there. This condition should include an allowance for the owners to undertake beach scraping on the Brown Land and Crown Land as required to achieve this, which introduces some landowner's consent complexities. Central Coast Council should logically be seeking approvals to undertake beach scraping on the Brown Land and Crown Land as required themselves, so any landowner's consent issues could potentially be resolved as part of this.

Any increased erosion on the beach seaward of the works, and related to the works, would be expected to only be of short duration. If there was clear evidence of a lowering of the beach (exceeding say 1m in elevation and averaged over the alongshore length of the properties and over a cross-shore width of say 20m) seaward of and related to the works (and therefore not evident in adjacent areas) after 6 months from a storm event, then the owners could potentially undertake beach scaping to mitigate that lowering. However, logically and logistically beach scraping should be a Council responsibility as it is in other areas, eg Collaroy-Narrabeen Beach, as it produces public benefit and is difficult for many different landowners to implement efficiently.

The owners would be responsible for monitoring sand levels at The Ruins and seaward of the works to assess if conditions had been triggered 6 months after a storm.

The NSW Coastal Panel has referred to other conditions that they considered may be relevant to be imposed, namely:

1. Pursuant to the provisions of Section 55M of the *Coastal Protection Act 1979*, a legally binding arrangement for the life of the works being negotiated and executed with Central Coast Council to ensure (i) the restoration of the beach, or land adjacent to the beach, if any increased erosion of the beach or adjacent land is caused by the presence of the works, and (ii) the maintenance of the works.
2. An easement in favour of Central Coast Council being created over the portion of the property affected by the protection works, and a positive covenant under Section 88BA of the *Conveyancing Act 1919* being established over the easement, burdening the owners of the property and their successors to maintain the protection works to the satisfaction of the Council. Such maintenance is to also include management of future “outflanking”, public safety and upgrading of the works if necessary in the future to meet changed climatic conditions.
3. A Bank Guarantee in the sum of (to be advised), in favour of Central Coast Council, being arranged to cover the completion or removal of the proposed protection works in the event that the wall is commenced but not completed within 5 years from the date at which the Deferred Commencement conditions have been formally satisfied. In such circumstances construction of the wall will be completed or removed by the Council utilising the Bank Guarantee funds. The quantum of the Bank Guarantee can be progressively reduced to three quarters (3/4), half (1/2), and one quarter (1/4) of this amount as Council is satisfied that each of these stages of construction has been satisfactorily achieved.

At this point in time, the owners are seeking legal advice and considering their positon on these conditions.

10. ITEM 9: ABORIGINAL CULTURAL HERITAGE VALUES

It is intended that this Item is addressed in a separate report as soon as practicable.

11. ITEM 10: THREATENED SPECIES

This item was not able to be addressed in the time available.

12. REFERENCES

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13. SALUTATION

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Yours faithfully
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