



Mr Tim Cronin
CEO
Cook Shire Council
PO Box 3
Cooktown Qld 4895

Dear Tim,

Re: Cooktown Foreshore Development – Technical Review – Final Report

Following on from my initial technical review and the further wave studies undertaken by Royal Haskoning (RHDHV) I am pleased to provide my final report on the Webber Esplanade Cooktown Foreshore Development. Those sections of the initial review which have not changed following the additional studies are included in this report for the sake of completeness. The review has included :

- Review of GHD technical reports;
- Meeting with GHD geotechnical and risk staff, 9th June;
- Site Visit and meetings with Cook Shire staff, 15th June
- Meeting with EHD staff in Cairns 16th June
- Meeting with GHD sheetpile design staff Cairns 17th June
- Review of the RHDHV wave modelling study Sept 2016

There have been a number of concerns raised with regard to the development and I intend to discuss each of those in turn.

Overall Wall Stability

The issue here is that the causeway was built over the existing soft sediment. There is little known about the properties of this material and concern has been expressed about the potential for sliding or slip circle failure.

Given that we do not have information on the material properties it is not possible to undertake analyses with any certainty.

We are able however to review loads that have been applied during construction and draw some conclusions from that. We do know that the subsurface material will be at its weakest immediately after construction when pore pressures within the material are high. As time goes on those pore pressures dissipate and the strength of the material improves.



We also know that one of the most severe load conditions is when the back of the wall is filled, the water level behind the wall is high and the water level in front of the wall is low.

As evidenced by the attached photographs, water levels were very high during dredging and reclamation filling operations and given that work was undertaken over a period of weeks we can assume that at some stage the water level in front of the wall would have been low. The seawall has therefore experienced one of the most severe load conditions and survived without any evidence of distress. Since that time the material underneath the wall has gained strength.



It would seem that at least some of the wall was preloaded with fill well above the finished level applying significant loads.

It is not possible to estimate the Factors of Safety of the wall at that time or now. We can however be confident that it has likely experienced the worst conditions and survived and therefore the likelihood of future failure is very low.

Piping Failure

The core of the seawall causeway was constructed with a mixture of material including a proportion of quite fine sand size. It is possible that under severe wave conditions or where there is a head of water resulting in a flow of water through the wall some of that fine material could be sucked from the wall resulting in piping failure.

While no grading analysis of the core material has been undertaken GHD have estimated that up to 80% of the material is less than 40mm.

If such piping were to occur sink holes would appear either on the seawall causeway or in the fill behind. For the majority of the wall, which has been in place for over two years, there is no evidence of any sink holes. If they were to occur in the future then they would be evidenced by cracking of the concrete path on top of the wall. They would then need to be repaired by excavating and backfilling with geotextile and graded rock.

There is one exception to the above comments and that is in the area at the south east corner of the of the pool. Here there is evidence of strong flows between the pool and the sea and this has resulted in scour and some sink holes have begun to occur in the surface of the seawall. It is understood that the pool area will be backfilled and this will improve the situation. Notwithstanding there is now a flow path for water and further piping may occur. It will be necessary to excavate where the flow path is and backfill with graded material from sand fines up to 100 mm rock to within 700mm of the surface. At that level a thick geotextile should be installed (Texcel R1200 or



similar) and then backfilled with small armour reclaimed from the seawall. This can then be covered in soil and the area grassed.

Armour Failure

An inspection of the seawall armour has indicated a number of issues:

- The armour is well undersize compared to the original design
- The armour stones are rounded and not well interlocked with adjoining armour stones
- There are gaps in the armour covering
- The toe armour rocks tended to be large and probably will not require modification

A systematic measurement programme was undertaken at nine locations equally spaced along the seawall. Each zone covered about 4 linear meters of wall. In each area the armour stones on the surface were measured with a tape along three dimensions to obtain a measure of the volume of the rock which when multiplied by the specific gravity gives a measure of the weight of each stone. The results of that measurement programme are contained in Appendix A.

Although not extremely accurate the measurement does give a reasonable indication of the average size of the armour and the variation along the seawall.

The results may be summarised as follows :

Existing Seawall

Zone	1	2	3	4	5	6	7	8	9
Average Armour Weight (Tonne)	0.7	0.5	0.5	0.4	0.4	0.5	0.3	0.4	0.4
Median Armour Weight (Tonne)	0.5	0.4	0.4	0.3	0.3	0.2	0.3	0.3	0.3

The original design called for 2 Tonne armour but this was based on a relatively simple wave model which produced a 50 year return period design wave of $H_s=2.75m$.

The Cook Shire Council commissioned RHDHV to undertake a comprehensive wave modelling exercise to establish appropriate 50 year return period design wave and water level conditions. That report is attached at Appendix B.

The following extract from that report provides combined 50 year wave with 20 year water levels and 50 year water levels with 20 year wave conditions :

The seawall points in the tables above approximate those at which rock sampling was undertaken. The design waves are significantly lower than previously assumed varying from approximately 1.75m to just over 2m. The amount of damage that might be expected to the existing seawall will be less but probably still unacceptable.

There are a variety of formulae for determining the size of armour required for given wave conditions. These are normally used for preliminary design and then the final design is based on physical model studies.

A physical model study for this project would be possible but probably not warranted. Given the seawall has already been constructed the money would be better spent on upgrading the wall or some set aside for future maintenance.

Table 9 Wave conditions at the seawall for 20 year ARI water level + 50 year ARI wind speed (from NNE)

	Seawall Point								
	1	2	3	4	5	6	7	8	9
Hs(m)	1.96	2.03	2.08	2.00	1.90	1.88	1.79	1.75	1.74
Tp (Sec)	3.7	3.8	3.8	3.8	3.9	4.0	4.1	4.3	4.6
Dir(degN)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	6(N)	23(NNE)

Table 10 Wave conditions at the seawall for 50 year ARI water level + 20 year ARI wind speed (from NNE)

	Seawall Point								
	1	2	3	4	5	6	7	8	9
Hs(m)	1.85	1.95	2.00	1.94	1.89	1.88	1.81	1.77	1.75
Tp (Sec)	3.7	3.8	3.8	3.8	4.0	4.2	8.5	9.2	9.2
Dir(degN)	23(NNE)	24(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	45(NE)

The two most commonly used formulae are those that were developed by Hudson and Van der Meer. For the Hudson formula you can use the significant wave height, Hs (average of the 1/3rd highest waves in the spectrum) or H_{2%}. If I was using the Hudson formula for a final design without physical modelling I would use H_{2%} if I was using Hudson as a prelude to physical modelling I would quite often use Hs as typically it gives answers closer to the modelled result especially where the seabed slope in front of the wall is gentle.

The following table summarises the design median rock armour size (Tonne) required for each section of the seawall using both wave options for Hudson and Hs for Van der Meer for zero maintenance for the 50 year return period wave conditions.

Zero Maintenance

Design Method	Zone								
	1	2	3	4	5	6	7	8	9
Hs (m)	2.0	2.0	2.1	2.0	1.9	1.9	1.8	1.75	1.75
Hudson (Hs)	0.7	0.7	0.8	0.7	0.6	0.6	0.5	0.5	0.5
Hudson (H _{2%})	1.5	1.5	1.7	1.5	1.3	1.3	1.1	1.0	1.0
Van der Meer	1.5	1.5	1.7	1.3	1.3	1.3	1.1	1.0	1.0

Van der Meer and Hudson H_{2%} are in close agreement requiring between 1 and 1.5 Tonne median armour weight for a zero maintenance design.

Clearly the seawall as it exists is inadequate and would likely suffer very serious damage if not total collapse following a 50 year return period event.

It is possible to upgrade the seawall without a total rebuild. The following table illustrates what could be achieved in terms of average and median armour weights if 8 to 10 small armour stones were to be removed and replaced with 5 to 6 stones in the 1 to 3 Tonne range for each 4 meter length of seawall.

Possible Upgrade

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Zone	1	2	3	4	5	6	7	8	9
Average Armour Weight (Tonne)	1.6	1.1	0.9	1	1	1.4	1	1.2	1.2
Median Armour Weight (Tonne)	1.4	0.8	0.6	0.7	0.7	1.2	0.8	1.2	1.0

While it is not possible to achieve the preferred design level of 1.5 Tonne a significant improvement can be made.

Hudson and Van der Meer can be used to estimate the rock armour sizes required for different levels of damage and maintenance.

The following Table illustrates the median rock armour size required for each zone for between 5% and 10% damage.

5% to 10% Damage

Design Method	Zone								
	1	2	3	4	5	6	7	8	9
Hs (m)	2.0	2.0	2.1	2.0	1.9	1.9	1.8	1.75	1.75
Hudson (Hs)	0.6	0.6	0.7	0.6	0.5	0.5	0.4	0.4	0.4
Hudson (H _{2%})	1.1	1.1	1.3	1.1	1	1	0.8	0.8	0.8
Van der Meer	0.75	0.75	0.9	0.75	0.7	0.7	0.6	0.5	0.5

It can be seen that if the seawall was upgraded as described above it should be possible to achieve a seawall which would perform well under normal circumstances and require maintenance after more severe storms. Note that zone 3 has the largest design wave and currently the smallest median size rock. This area may require greater work than others.

It is important to be aware that the formulae used are empirical and not exact. Whilst it is possible that Hudson (Hs) is applicable and the extent of damage following storms may be relatively low, if Van der Meer is more applicable then maintenance will be required and will be probably be required after storms with a lower return period than 50 years.

The following is recommended :

- Using a skilled operator, in each 4 to 5 meter section of the wall remove 8 to 10 of the smaller rocks and replace them with 5 to 6 large armour stones in the 1 to 3 tonne range. Make sure that all the armour stones are tightly placed such that each armour rock has contact with at least three adjoining stones.
- The small armour rock removed could be placed behind the concrete path on a heavy geotextile such that all rock is below the surface of the path. This could then be filled with soils and the area grassed. This would protect the rear of the wall from erosion in the event that a storm results in heavy wave overtopping.
- A stockpile of armour stones should be kept at the local quarry and the Shire should have a mechanism in place to facilitate urgent response should the seawall be damaged following a storm.
- A formal inspection of the seawall should be undertaken on an annual basis and a short report prepared describing its condition and the types of storms that it has withstood in the previous year and documenting the extent of any maintenance that may have been required..
- The seawall should be informally inspected following all significant storms and any areas repaired that may have suffered damage.

Armour Sources

While in Cooktown I visited the local quarry (Mt Amos). There were some stockpiles of armour on the quarry floor with sizes varying from 1 tonne upwards of 3 tonne. The quarry only does a blast every two or so years. Another blast was undertaken following our visit. The Director of Infrastructure Services at the Shire has visited the quarry following the blast and estimates that there is about 500 Tonne of suitable armour available from this source. There is another quarry at Mt Carbine approximately 200 kilometres from Cooktown and that has an abundant supply of large rock. The cost from this source is of course significantly higher due to the cost of haulage.

It would be worthwhile the shire organising for oversize material from existing stockpiles and from the latest blast at Mt Amos to be separated into say three stockpiles of 1 to 2 tonne and 2 to 3 tonne and larger. In this way the seawall upgrade can be planned so that the larger armour is placed in the most vulnerable areas and in zones which are protecting valuable infrastructure.



Overtopping

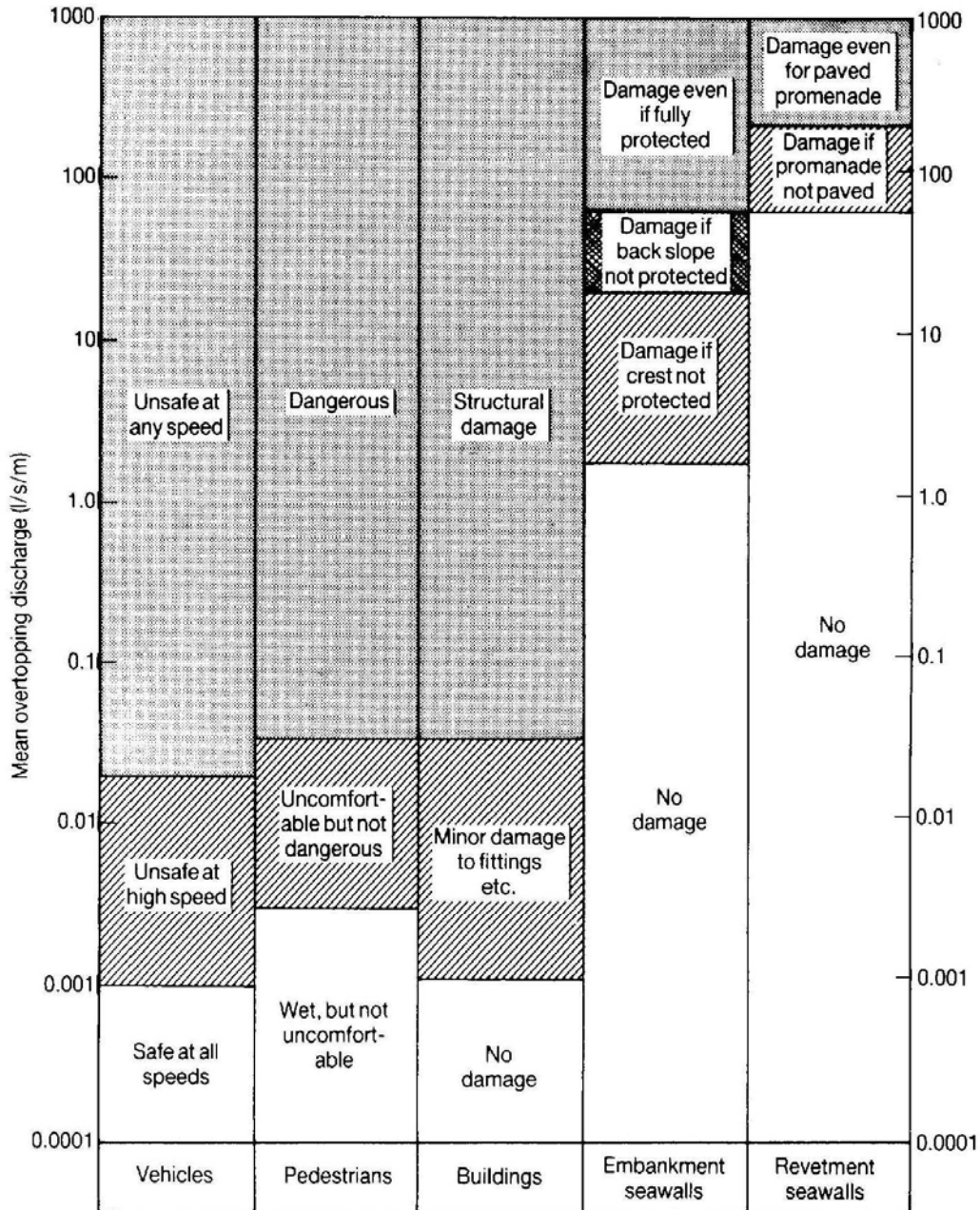
Overtopping calculations have been undertaken based on the methodologies recommended in *Eurotop, 2007, Wave Overtopping of Sea Defences and Related Structures : Assessment Manual*.

Depending on the wave and water level conditions and the locations along the seawall the overtopping during a 50 year return period event would average between 70 and 160 l/m/sec.

The following table from the CIRIA Rock Manual 1991 suggests that at overtopping rates between 70 and 300 l/m/sec crest erosion would occur unless the crest is paved.

Box 46 Critical overtopping discharges

Limiting values of \bar{Q} for different design cases have been suggested, and are summarised in Figure 152. This incorporates recommended limiting values of the mean discharge for the stability of crest and rear armour to types of seawalls and or the safety of vehicles and people.



There is a concrete path at the crest and if the area behind that is further protected with small armour rock recovered from the seawall then structural damage should be minimal.

While the structural integrity of the structure should be able to be maintained it would be dangerous for people to be walking along the path during such an event. Public access during such time should be discouraged or prevented if possible.

Sheetpile Walls

GHD have correctly advised that the integrity of the sheetpile walls at the marina and the tidal pool is not able to be determined at this stage because of the lack of information on the nature of the material at the toe of the walls and how far the sheetpiles penetrated into competent material.

It is understood that the Shire has decided to fill the swimming pool area and place armour rock in front of the marina sheetpile wall.

Based on the armour stability calculations undertaken it is recommended that armour stones well graded between 0.5 tonne and 3 tonne with a median weight of 1.5 Tonne be used. The crest level should be just above HAT so that the rocks are visible at all tides. The crest width should be a minimum of three rocks wide and should be made up of the larger rocks in the range defined above. The slope of the wall should be 2:1.



Summary


The following summarises the finding of my technical review of the Webber Esplanade Foreshore Development :

- The armour rock forming the wave protection to the seawall is undersize and not well interlocked. The stability of the armour can be significantly improved by removal of some of the smaller armour and replacement with larger armour stones placed in such a way that all armour is tightly interlocked with adjacent rocks.
- A study has been undertaken to establish the return period for various deepwater wave heights which were then transformed to locations along the seawall.
- Analysis of those waves has established that it should be possible with an upgraded seawall to withstand a 50 year return period event with 5 % to 10% damage. If appropriate stockpiles of armour are kept at the local quarry repairs should be able to be facilitated promptly following the storm.
- The seawall should be inspected following each significant storm and formally reviewed and reported upon annually.
- The risks of a major slip circle or sliding failure of the seawall are very low given that it has survived conditions worse than should be expected in the future.
- Piping failure is unlikely to be a major problem except in an area in the south east corner of the tidal pool which will require remedial action and ongoing monitoring.
- The sheetpile wall at the swimming pool and the marina will be filled in to ensure ongoing stability.

- Overtopping will occur during storm events. Placement of small armour rocks on the land side of the concrete footpath and then backfill with soil and regrassing should provide adequate protection against such overtopping. Public access during storm events should be actively discouraged.

Should you have any queries or require additional information please contact me.

Yours Sincerely

A handwritten signature in black ink, appearing to read 'Anthony P Byrne', with a long horizontal flourish extending to the right.

Tony Byrne

19th September 2016

Appendix A

Seawall Armour Measurements



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 1

	mm	mm	mm	Tonne
1	900	1300	800	2.4
2	900	1100	800	2.0
3	1000	900	800	1.8
4	800	1100	800	1.8
5	1000	800	700	1.4
6	900	1300	400	1.2
7	800	800	700	1.1
8	600	1200	500	0.9
9	800	900	500	1.0
10	1200	500	600	0.9
11	900	800	500	0.9
12	1000	500	500	0.6
13	800	400	600	0.5
14	900	450	450	0.5
15	600	700	400	0.4
16	600	500	500	0.4
17	500	600	450	0.3
18	500	500	500	0.3
19	400	500	500	0.3
20	400	800	300	0.2
21	500	400	400	0.2
22	500	800	200	0.2
23	400	500	400	0.2
24	400	800	250	0.2
25	500	500	300	0.2
26	300	400	500	0.2
27	300	400	400	0.1
28	400	500	200	0.1
Ave	0.7	Tonne		



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 1

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3				2
4				1.8
5				1.5
6				1.2
7	900	1300	800	2.4
8	900	1100	800	2.0
9	1000	900	800	1.8
10	800	1100	800	1.8
11	1000	800	700	1.4
12	900	1300	400	1.2
13	800	800	700	1.1
14	600	1200	500	0.9
15	800	900	500	0.9
16	1200	500	600	0.9
17	900	800	500	0.9
18	1000	500	500	0.6

Armour Rocks Removed

Ave	1.6	Tonne
Median	1.4	Tonne

Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 2

	mm	mm	mm	Tonne
1	1100	1000	700	2.0
2	1100	1100	600	1.9
3	600	1100	600	1.0
4	600	1000	600	0.9
5	900	600	600	0.8
6	800	800	450	0.8
7	600	800	600	0.7
8	700	800	500	0.7
9	700	1300	300	0.7
10	500	1200	400	0.6
11	500	1100	400	0.6
12	800	600	400	0.5
13	500	600	600	0.5
14	500	500	700	0.4
15	500	800	400	0.4
16	500	500	600	0.4
17	600	600	400	0.4
18	500	600	400	0.3
19	500	600	300	0.2
20	700	600	200	0.2
21	700	300	400	0.2
22	500	400	400	0.2
23	500	400	400	0.2
24	400	400	500	0.2
25	500	500	300	0.2
26	300	600	400	0.2
27	400	600	300	0.2
28	400	500	350	0.2
29	400	400	300	0.1
30	300	400	300	0.1
31	400	400	200	0.1
Ave		0.5	Tonne	



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 2

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3	Armour Rocks Added			2
4				1.8
5				1.5
6				1.2
7	1100	1000	700	2.0
8	1100	1100	600	1.9
9	600	1100	600	1.0
10	600	1000	600	0.9
11	900	600	600	0.8
12	800	800	450	0.7
13	600	800	600	0.7
14	700	800	500	0.7
15	700	1300	300	0.7
16	500	1200	400	0.6
17	500	1100	400	0.6
18	800	600	400	0.5
19	500	600	600	0.5
20	500	500	700	0.4
21	500	800	400	0.4
22	500	500	600	0.4

Armour Rocks Removed

Ave	1.1	Tonne
Median	0.8	Tonne

Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 3

	mm	mm	mm	Tonne
1	800	1000	600	1.3
2	700	900	700	1.1
3	700	1000	600	1.1
4	700	1000	500	0.9
5	600	700	650	0.7
6	700	700	500	0.6
7	500	800	600	0.6
8	500	600	800	0.6
9	400	700	800	0.6
10	600	600	600	0.6
11	800	600	400	0.5
12	400	900	500	0.5
13	500	600	600	0.5
14	500	800	400	0.4
15	500	500	600	0.4
16	400	600	600	0.4
17	300	800	600	0.4
18	500	800	300	0.3
19	400	700	400	0.3
20	400	600	450	0.3
21	400	600	400	0.2
22	600	500	300	0.2
23	600	500	300	0.2
24	400	500	400	0.2
25	400	600	300	0.2
26	600	400	300	0.2
27	300	600	400	0.2
28	300	700	300	0.2
29	400	700	200	0.1
Ave		0.5	Tonne	



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 3

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3	Armour Rocks Added			2
4				1.8
5				1.5
6				1.2
7	800	1000	600	1.2
8	700	900	700	1.1
9	700	1000	600	1.1
10	700	1000	500	0.9
11	600	700	650	0.7
12	700	700	500	0.6
13	500	800	600	0.6
14	500	600	800	0.6
15	400	700	800	0.6
16	600	600	600	0.6
17	800	600	400	0.5
18	400	900	500	0.5
19	500	600	600	0.5
20	500	800	400	0.4
21	500	500	600	0.4
22	400	600	600	0.4
23	300	800	600	0.4
24	500	800	300	0.3
25	400	700	400	0.3

Armour Rocks Removed

Ave	0.9	Tonne
Median	0.6	Tonne

Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 4

	mm	mm	mm	Tonne
1	700	1000	800	1.4
2	600	900	600	0.8
3	800	1000	400	0.8
4	700	800	500	0.7
5	650	800	500	0.7
6	500	700	600	0.5
7	600	700	500	0.5
8	700	500	500	0.4
9	600	800	350	0.4
10	500	600	550	0.4
11	500	600	500	0.4
12	600	600	400	0.4
13	500	700	400	0.4
14	500	400	600	0.3
15	400	800	350	0.3
16	400	700	400	0.3
17	300	700	500	0.3
18	400	500	500	0.3
19	800	400	300	0.2
20	800	400	300	0.2
21	500	600	300	0.2
22	600	500	300	0.2
23	500	400	400	0.2
24	400	400	500	0.2
25	500	400	350	0.2
26	500	300	400	0.2
27	300	500	400	0.2
28	500	350	300	0.1
29	600	300	250	0.1
30	300	500	200	0.1
Ave	0.4		Tonne	



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 4

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3	Armour Rocks Added			2
4				1.8
5				1.5
6				1.2
7	700	1000	800	1.4
8	600	900	600	0.8
9	800	1000	400	0.8
10	700	800	500	0.7
11	650	800	500	0.7
12	500	700	600	0.5
13	600	700	500	0.5
14	700	500	500	0.4
15	600	800	350	0.4
16	500	600	550	0.4
17	500	600	500	0.4
18	600	600	400	0.4
19	500	700	400	0.4
20	500	400	600	0.3
21	400	800	350	0.3

Armour Rocks Removed

Ave	1 Tonne
Median	0.7 Tonne

Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 5

	mm	mm	mm	Tonne
1	700	800	600	0.9
2	600	900	600	0.8
3	700	600	700	0.7
4	600	800	600	0.7
5	600	600	800	0.7
6	800	700	500	0.7
7	500	900	500	0.6
8	400	1000	500	0.5
9	700	400	700	0.5
10	500	600	500	0.4
11	800	800	200	0.3
12	500	500	500	0.3
13	500	600	400	0.3
14	400	600	500	0.3
15	400	800	350	0.3
16	400	700	400	0.3
17	300	700	500	0.3
18	500	600	350	0.3
19	400	500	500	0.3
20	400	600	400	0.2
21	400	600	400	0.2
22	600	500	300	0.2
23	500	600	300	0.2
24	400	500	400	0.2
25	400	500	400	0.2
26	500	450	300	0.2
27	400	500	300	0.2
28	500	400	300	0.2
29	500	400	300	0.2
30	400	500	300	0.2
31	400	500	300	0.2
Ave		0.4	Tonne	



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 5

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3	Armour Rocks Added			2
4				1.8
5				1.5
6				1.2
7	700	800	600	0.9
8	600	900	600	0.8
9	700	600	700	0.7
10	600	800	600	0.7
11	600	600	800	0.7
12	800	700	500	0.7
13	500	900	500	0.6
14	400	1000	500	0.5
15	700	400	700	0.5
16	500	600	500	0.4
17	800	800	200	0.3
18	500	500	500	0.3
19	500	600	400	0.3
20	400	600	500	0.3

Armour Rocks Removed

Ave	1 Tonne
Median	0.7 Tonne

Webber Espanade Rock Armour Measurements

Identified	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 6

	mm	mm	mm	Tonne
1	900	1400	900	2.9
2	800	1200	800	2.0
3	500	900	600	0.7
4	600	700	600	0.6
5	600	1000	400	0.6
6	500	600	600	0.5
7	500	700	500	0.4
8	700	500	500	0.4
9	500	600	500	0.4
10	600	600	300	0.3
11	500	700	300	0.3
12	400	700	350	0.2
13	400	700	300	0.2
14	400	600	350	0.2
15	400	400	500	0.2
16	300	600	400	0.2
17	400	500	350	0.2
18	400	400	400	0.2
19	400	500	300	0.2
20	500	500	200	0.1
21	400	400	300	0.1
22	400	400	300	0.1
23	300	500	300	0.1
24	200	400	400	0.1
25	200	400	250	0.1

Ave 0.5 Tonne



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 6

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3	Armour Rocks Added			2
4				1.8
5				1.5
6				1.2
7	900	1400	900	2.9
8	800	1200	800	2.0
9	500	900	600	0.7
10	600	700	600	0.6
11	600	1000	400	0.6
12	500	600	600	0.5
13	500	700	500	0.4
14	700	500	500	0.4
15	500	600	500	0.4

Armour Rocks Removed

Ave 1.4 Tonne
 Median 1.2 Tonne

Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 7

	mm	mm	mm	Tonne
1	800	800	700	1.1
2	500	1000	800	1.0
3	600	1000	500	0.8
4	600	700	500	0.5
5	800	600	400	0.5
6	400	600	700	0.4
7	800	500	400	0.4
8	500	700	450	0.4
9	500	700	400	0.4
10	500	700	400	0.4
11	800	400	400	0.3
12	400	700	400	0.3
13	600	600	300	0.3
14	500	500	400	0.3
15	500	500	400	0.3
16	400	500	500	0.3
17	400	700	350	0.2
18	300	600	400	0.2
19	300	600	400	0.2
20	400	600	300	0.2
21	400	500	300	0.2
22	400	500	300	0.2
23	300	500	400	0.2
24	300	300	600	0.1
25	200	600	400	0.1
26	500	300	300	0.1
27	400	250	200	0.1
Ave		0.3	Tonne	



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 7

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3	Armour Rocks Added			2
4				1.8
5				1.5
6				1.2
7	800	800	700	1.1
8	500	1000	800	1.0
9	600	1000	500	0.8
10	600	700	500	0.5
11	800	600	400	0.5
12	400	600	700	0.4
13	800	500	400	0.4
14	500	700	450	0.4
15	500	700	400	0.4
16	500	700	400	0.4
17	800	400	400	0.3
18	400	700	400	0.3

Armour Rocks Removed

Ave	1	Tonne
Median	0.8	Tonne

Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 8

	mm	mm	mm	Tonne
1	1300	800	500	1.3
2	600	900	500	0.7
3	500	1000	500	0.6
4	600	500	700	0.5
5	600	700	500	0.5
6	400	700	700	0.5
7	800	600	400	0.5
8	600	900	350	0.5
9	800	400	500	0.4
10	600	600	400	0.4
11	300	800	600	0.4
12	400	800	400	0.3
13	500	500	500	0.3
14	600	500	400	0.3
15	400	600	400	0.2
16	400	600	400	0.2
17	300	500	500	0.2
18	400	500	300	0.2
19	300	500	400	0.2
20	400	500	300	0.2
21	250	400	400	0.1
22	300	400	300	0.1
23	400	400	200	0.1

Ave 0.4 Tonne



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 8

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3	Armour Rocks Added			2
4				1.8
5				1.5
6				1.2
7	1300	800	500	1.3
8	600	900	500	0.7
9	500	1000	500	0.6
10	600	500	700	0.5
11	600	700	500	0.5
12	400	700	700	0.5
13	800	600	400	0.5
14	600	900	350	0.5

Armour Rocks Removed

Ave 1.2 Tonne
 Median 1.2 Tonne

Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 9

	mm	mm	mm	Tonne
1	800	900	600	1.1
2	600	800	800	1.0
3	800	600	800	1.0
4	900	800	500	0.9
5	700	1000	500	0.9
6	700	1100	400	0.8
7	600	700	450	0.5
8	600	500	500	0.4
9	400	900	400	0.4
10	500	700	400	0.4
11	500	500	500	0.3
12	500	500	500	0.3
13	500	800	300	0.3
14	400	600	500	0.3
15	600	500	400	0.3
16	500	600	400	0.3
17	500	700	300	0.3
18	400	500	500	0.3
19	400	500	450	0.2
20	500	400	400	0.2
21	600	500	250	0.2
22	500	500	300	0.2
23	300	600	400	0.2
24	400	700	250	0.2
25	400	500	300	0.2
26	600	200	150	0.0
Ave	0.4	Tonne		



Webber Espanade Rock Armour Measurements

Rounded	Assumed Specific Gravity	2.55
Angular	Assumed Specific Gravity	2.65

Zone 9

Possible Upgrade

	mm	mm	mm	Tonne
1				3
2				2.5
3	Armour Rocks Added			2
4				1.8
5				1.5
6				1.2
7	800	900	600	1.1
8	600	800	800	1.0
9	800	600	800	1.0
10	900	800	500	0.9
11	700	1000	500	0.9
12	700	1100	400	0.8
13	600	700	450	0.5
14	600	500	500	0.4
15	400	900	400	0.4
16	500	700	400	0.4
17	500	500	500	0.3

Armour Rocks Removed

Ave	1.2	Tonne
Median	1	Tonne

Appendix B

Cooktown Wave Modelling Report - RHDHV



REPORT

Cooktown Waterfront Wave Modelling

Technical Note

Client: Cook Shire Council

Reference: M&WPA1438R001D07

Revision: 00/Final

Date: 19 September 2016

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Date: 19 September 2016
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Project number: PA1438
Author(s): James Lewis

Drafted by: James Lewis

Checked by: Evan Watterson

Date / initials: 19-09 / EKW

Approved by: Evan Watterson

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Classification

Open



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Appendices

Appendix A Combined ARI Wave Parameters

1 Background

Royal HaskoningDHV (RHDHV) was engaged by Cook Shire Council (CSC) to undertake a wave modelling study focussing on the seawall built as part of the Webber Esplanade Cooktown Foreshore Development. Following damage to the seawall a Technical Review (TR) was undertaken by specialist consultant Tony Byrne (Byrne, 2016). The TR investigated the following issues regarding the seawall:

- Overall wall stability
- Piping failure
- Armour failure
- Armour sources
- Design waves
- Design water levels
- Combined wave and water levels
- Overtopping

This study follows from the recommendations within the TR document which, in regards to wave modelling investigations, were:

- A study to establish the return period for various deep water wave heights is warranted as is the transformation of those waves from deep water to locations along the seawall.
- Once that information is available it should be possible to combine the wave heights with appropriate return period water levels to establish the return period condition that the upgraded seawall can withstand with minimal damage and the amount of maintenance that might be required following a 50 year return period event.

2 Scope of Work

The CSC are asset custodians of the seawall along the Webber Esplanade foreshore and wish to determine return period wave and water level conditions of the site in order to inform design of upgrade works to the structure.

The CSC has defined the scope of work covered by this consultancy to include;

- assess return period wind conditions for various directions;
- determine deep water waves that may be generated by such winds;
- transform waves from deep water through the navigation channel to various locations along the foreshore; and
- Establish wave parameters along the wall for the 1, 10, 20, 30, 50 and 100 year average recurrence intervals (ARIs) conditions.

The ensuing inclusions/exclusions were noted following communication with CSC:

- wave direction and peak periods at the seawall locations should be reported to inform wave overtopping;
- a curve of return time period vs wave height will need to be produced for the study site
- design water level values should be obtained from:
 - *Ocean Hazard Assessment – Stage 3 July 2004 – Surge Plus Tide Statistics for Selected Open Coast Locations along the Queensland East Coast* (JCU, 2014); and

- Design water levels excluding sea level rise should be used.
- for the purpose of combined wave and water levels, a reduced return period water level with a given return period wave should be used e.g.; a 50 year ARI wave may be taken with a 20 year ARI water level and vice versa; and
- Analysis should take into account the potential for wave breaking at or before the seawall.

Wave conditions determined through the modelling will be required to be assessed at nine (9) locations along the 400m-500m revetment wall. The locations are in close proximity to the sites sampled for rock size in June 2016.



3 Methodology

The following methodology was undertaken to ascertain design wave heights at the Webber Esplanade foreshore seawall following consultation with CSC and following recommendations made within Byrne, 2016.

3.1 Review of Previous Studies and Available Data

This section outlines information sources reviewed in order to inform the numerical modelling stage of the study.

3.1.1 Wind and Tropical Cyclones

Wind speed and direction measurements have been recorded at Bureau of Meteorology (BoM) weather stations in close proximity to the study site. Table 1 shows data sourced from nearby BoM sites and used to inform this study. Figure 1 gives location to each recording station.

Table 1 Cooktown BoM wind data sources

Site	Parameters	Data Start	Data End	Total Record Length
Cooktown Mission Strip	Wind Speed and Direction	03/07/1987	18/09/2002	15.2 years
Cooktown Post Office	Wind Speed and Direction	01/01/1957	03/07/1987	30.5 years
Cooktown Airport	Wind Speed and Direction	24/07/2000	03/08/2016	16.1 years

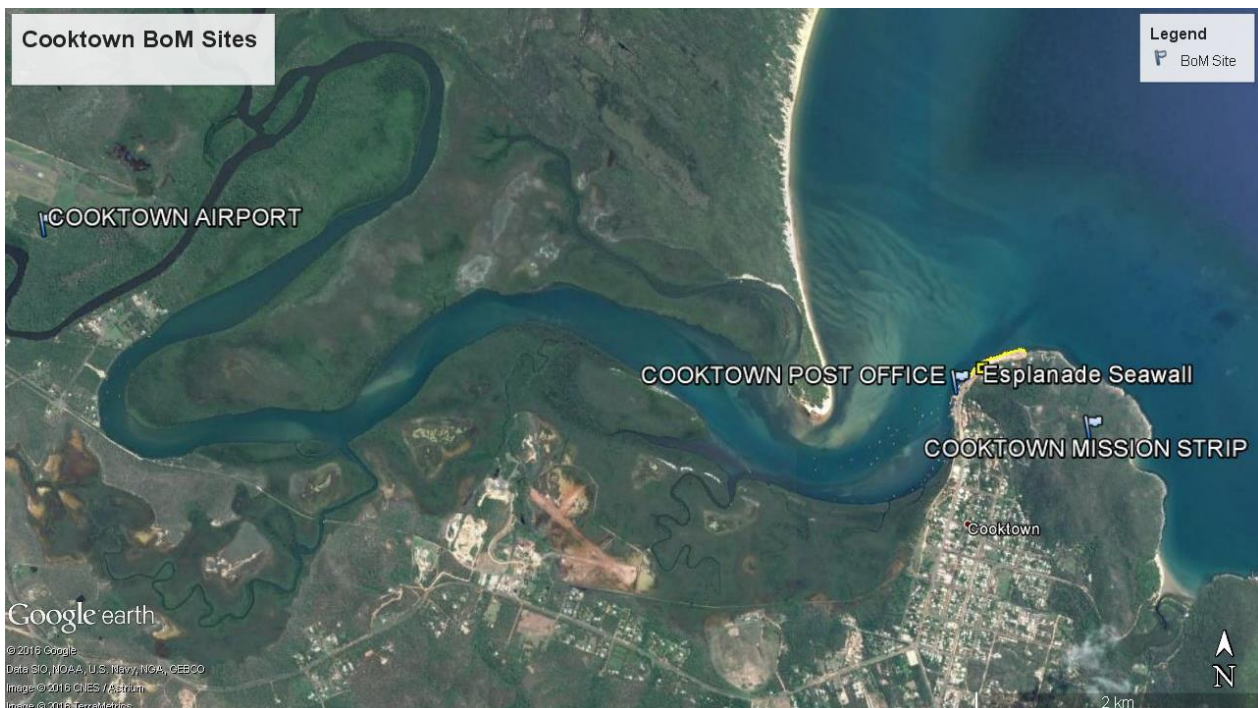


Figure 1 Cooktown BoM wind stations (Google Earth, 2016).

As the Cooktown Post Office had the longest continuous record (30.5 years), RHDVH used this data to undertake an Extreme Value Analysis (EVA) to determine Average Return Intervals (ARI) for wind speeds in the Cooktown area. It should be noted that while this site is located near the coast, it is not considered representative of the overwater wind regime offshore of Cooktown and wind measurement here may be influenced by coastal topographic features (e.g. Grassy Hill).

Due to its location at 15.5°S latitude, Cooktown is vulnerable to the passage of tropical cyclones on a seasonal basis. Cyclones in the Australian region have their maximum occurrence between 15°S and 20°S latitude, commencing in November/December and continuing to March/April. The greatest incidence is in January to March, transferring from east to west as the season advances (Lourensz, 1981). In terms of both cyclone intensity and the likelihood of crossing the coast, the most cyclone-prone area along the Queensland coast is around Mackay (Harper, 1999) 450 km south. A historical representation of cyclone tracks for this region can be seen in Figure 2.

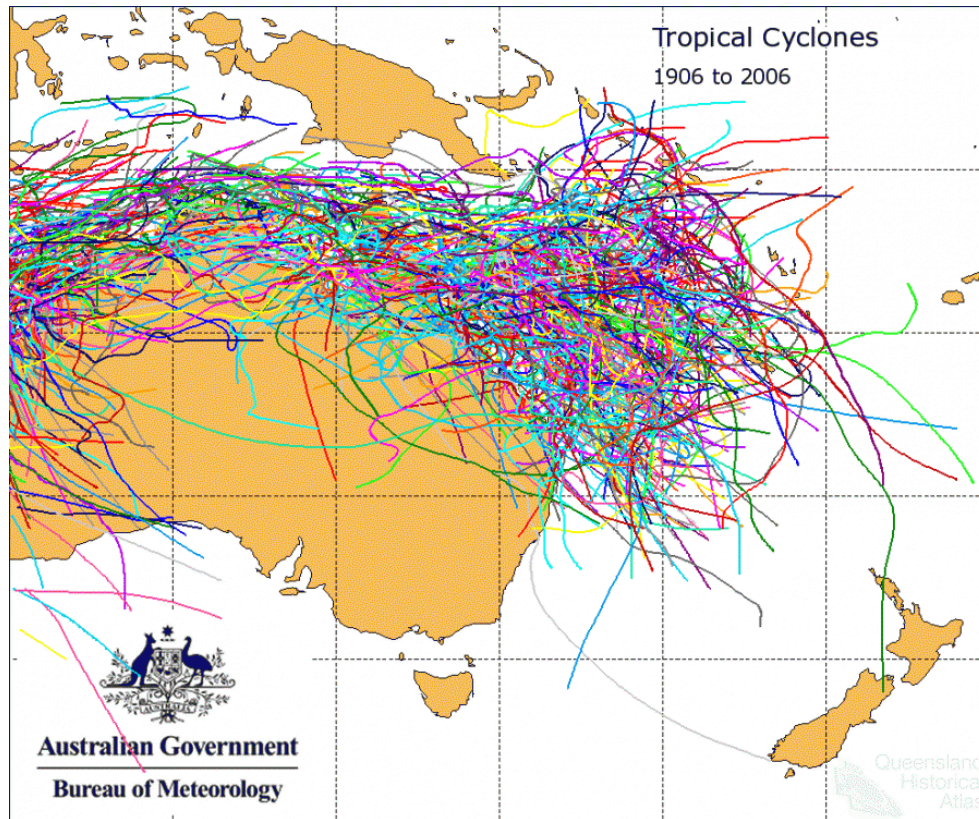


Figure 2 Historical cyclone tracks along the North Eastern coastline of Australia form 1906 to 2013 (BoM, 2016).

Due to the tendency for cyclones to move in such close proximity to the study site, it is sufficient to say that design wave conditions at the site will occur during cyclonic conditions. The results of the ARI analysis from the BoM Cooktown Post Office Site can be seen in Figure 3. The analysis determined the 100 year ARI wind speed (3s gust) to be approximately 30m/s. Harper et al, 1999 undertook a study of extreme tropical cyclonic winds focussed on the nearby coastal location of Townsville. The highest recorded gust at Townsville was due to Cyclone Althea, being approximately 55m/s as seen in Figure 4. The current regional recommendation of the Australian Design Standards (AS1170.2) wind loading code (Region C) is also shown seen in Figure 4. The AS1170.2 recommendation shown here is noted by Harper (1999) to be potentially conservative.

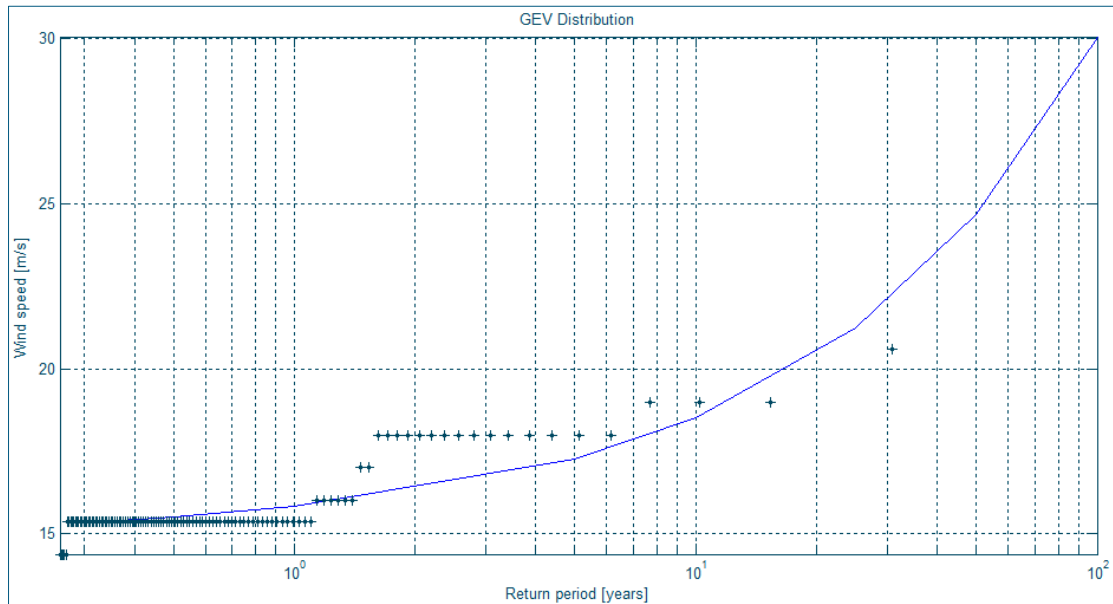


Figure 3 ARI analysis of 9am and 3pm wind speed averages at Cooktown Post Office BoM Weather station Site.

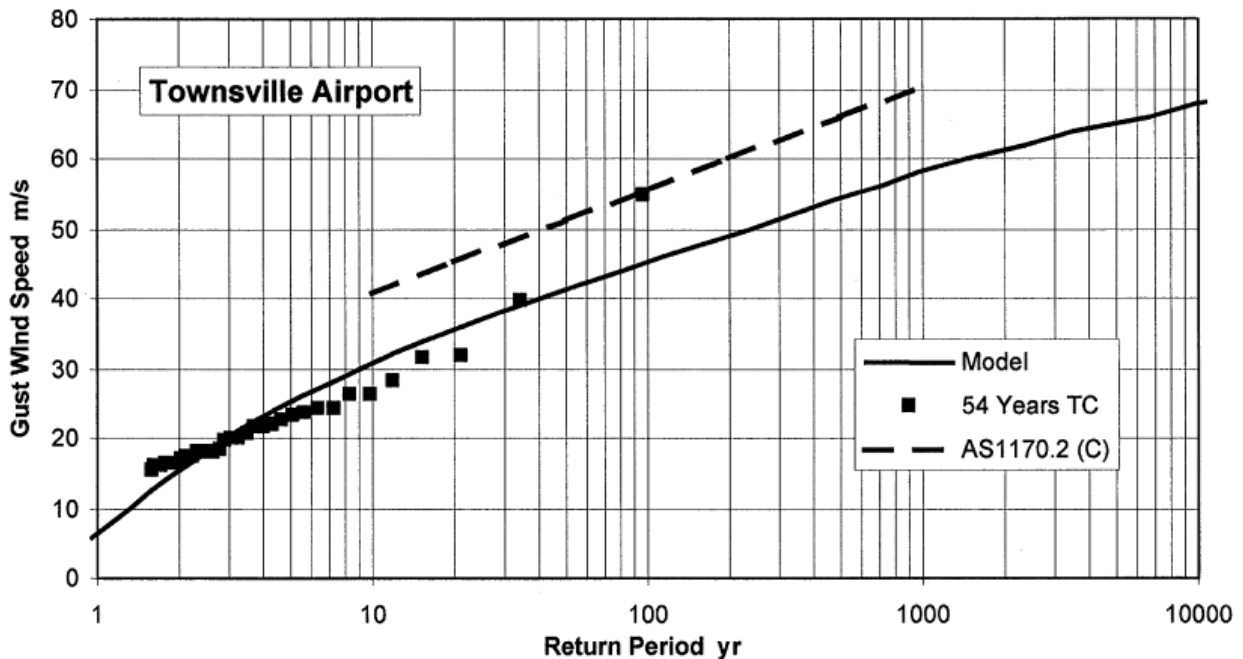


Figure 4 Modelled, measured and Australian Design Standard (AS) extreme winds (3s gusts) comparison at Townsville (Harper, 1999).

A comparison of ARI analyses from Harper's study with ARI values from the Cooktown Post Office site as well as the AS1170.2 values was undertaken to determine worst case scenario cyclonic winds that may develop in the region. The Post Office wind speeds (1 hour averages) have been converted to 3 second gusts using the method outlined in WMO (2008). The results of this comparison can be seen in Table 2. As a conservative measure, the highest of each ARI value (AS1170.2 in most instances) for the region have been adopted to take forward into the wave modelling phase.

Table 2 Average Return Interval (ARI, years) of wind speed (gust) at Central-North QLD locations (Region C)

ARI (years)	Gust Speed (m/s)						
	CAIRNS (3 sec gusts)	MACKAY (3 sec gusts)	TOWNSVILLE (3 sec gusts)	COOKTOWN (3 sec gust, converted)	AS1170.2 Region C (3 sec gusts)	MAX	ADOPTED (MAX x 1.20)
1	10	10	10	26	22	26	31
10	30	27	28	29	41	41	49
20	34	31	33	33	44	45	54
30	37	36	38	37	48	48	58
50	40	38	42	40	52	52	62
100	43	43	45	48	55	55	66

Table 2 represents the highest calculated ARI wind speed (gusts) along Region C. In order to use these values for wave modelling (wind over water) purposes, the over-land gust speeds need to be converted to over-sea gust speeds. Australian Standard AS/NZS1170.2:2011 uses a conversion factor of 1.20 for Region C at a Terrain type of T2. As discussed further below, the over-water gust speeds were then converted to 10-minute average (or sustained) wind speeds which are appropriately conservative inputs to the wind-wave model.

3.1.2 Water Levels

As per the CSC brief, water levels used for this study will be taken from *Ocean Hazard Assessment – Stage 3 July 2004 – Surge plus Tide Statistics for Selected Open Coast Locations along the Queensland East Coast*; (JCU, 2004). The report defines the ARI water level (storm surge + tide) values for Cooktown, detailed in Figure 5 and Table 3.

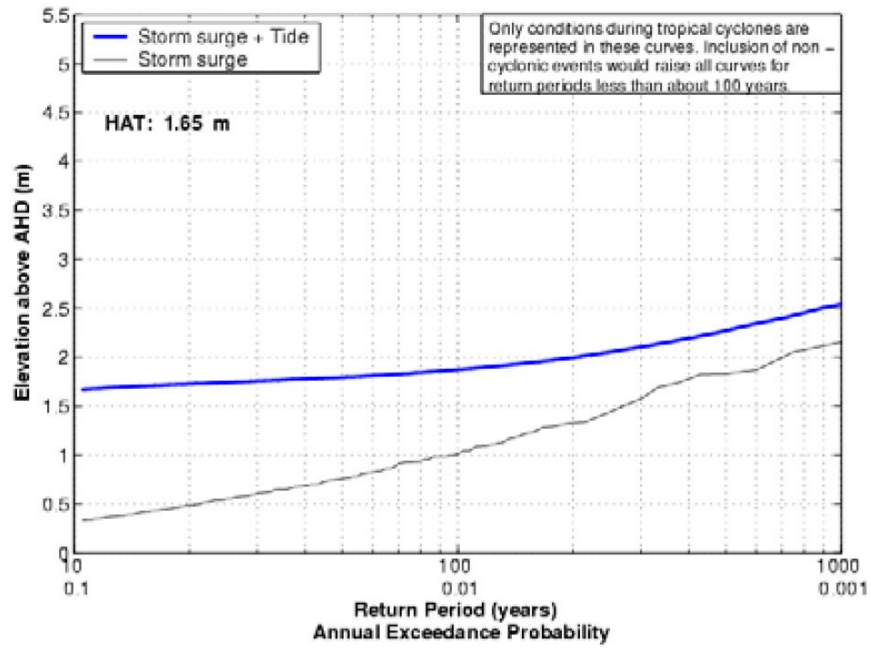


Figure 5 2003 Cooktown Storm Surge plus tide return period curves (JCU, 2004).

Table 3 2003 Cooktown Storm Surge plus tide return period curves (JCU, 2004).

ARI (yrs.)	Water Level+ Storm Surge (mMSL)
1	1.55
10	1.65
20	1.7
30	1.75
50	1.8
100	1.9

3.2 Numerical Model

A MIKE21 Spectral Wave (SW) Flexible Mesh (FM) model was setup for the study site. The MIKE21FM SW model is a third generation spectral wind-wave model based on unstructured meshes, giving the opportunity to vary resolution around complex bathymetric and topographic features. The FM module was selected due to the complexity of the offshore bathymetry of the study site. The model simulates the growth, decay and transformation of wind-generated waves and swells in offshore and coastal areas.

MIKE 21 SW includes the following physical phenomena:

- wave growth by action of wind;
- non-linear wave-wave interaction;
- dissipation due to white capping;
- dissipation due to bottom friction;
- dissipation due to depth-induced wave breaking; and
- Refraction and shoaling due to depth variations.

3.2.1 Domain

The nature of cyclonic winds as they approach the coast means that the direction of the maximum winds as the cyclone crosses the coast may actually have very little impact on the onshore-directed wave field. In the example seen in Figure 6, areas south of the cyclone eye (Townsville, etc.) experience larger waves due to the onshore fetch set up by the clockwise wind pattern. Whereas towns further north will be experiencing strong offshore winds resulting in smaller waves along the shoreline. That is the seawall at Cooktown would be most exposed to a cyclone that tracks to the north of Cooktown.

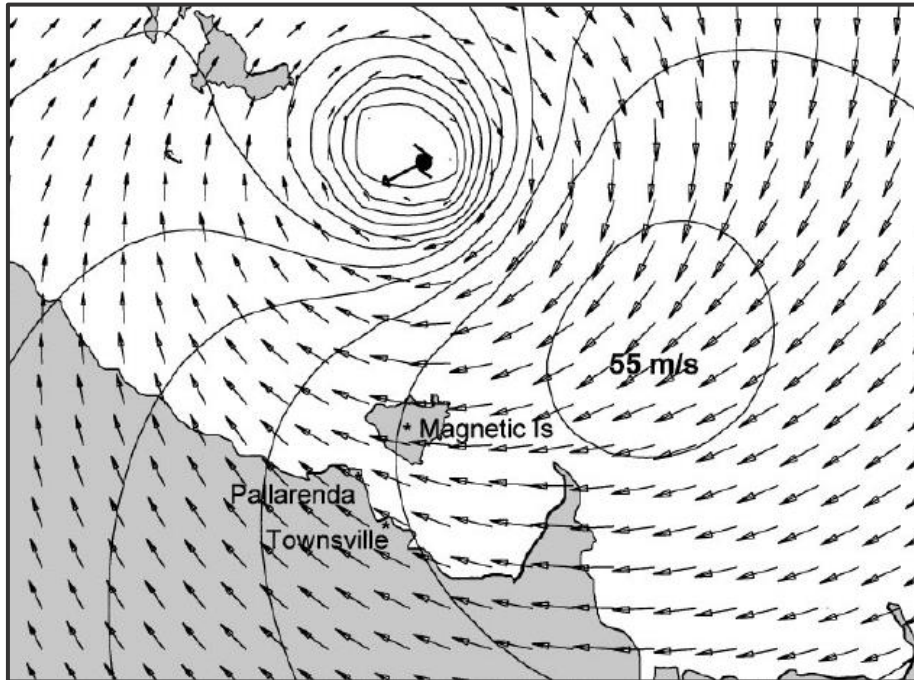


Figure 6 Numerically modelled winds of Cyclone Althea (Harper, 1999).

As the transit path of cyclones along the north eastern Queensland coast is to a certain extent unpredictable, it is sufficient to assume that maximum wind speeds that may affect a coastline due to tropical cyclones are omni-directional. Hence the maximum wind speeds defined in Table 2 can be assumed to occur from any direction. As such, the biggest impact on the maximum wave heights due to these events will be determined by maximum effective fetch distance. Approximate maximum fetch distances (to the outer GBR) for different directions offshore of the study site can be seen Figure 7.

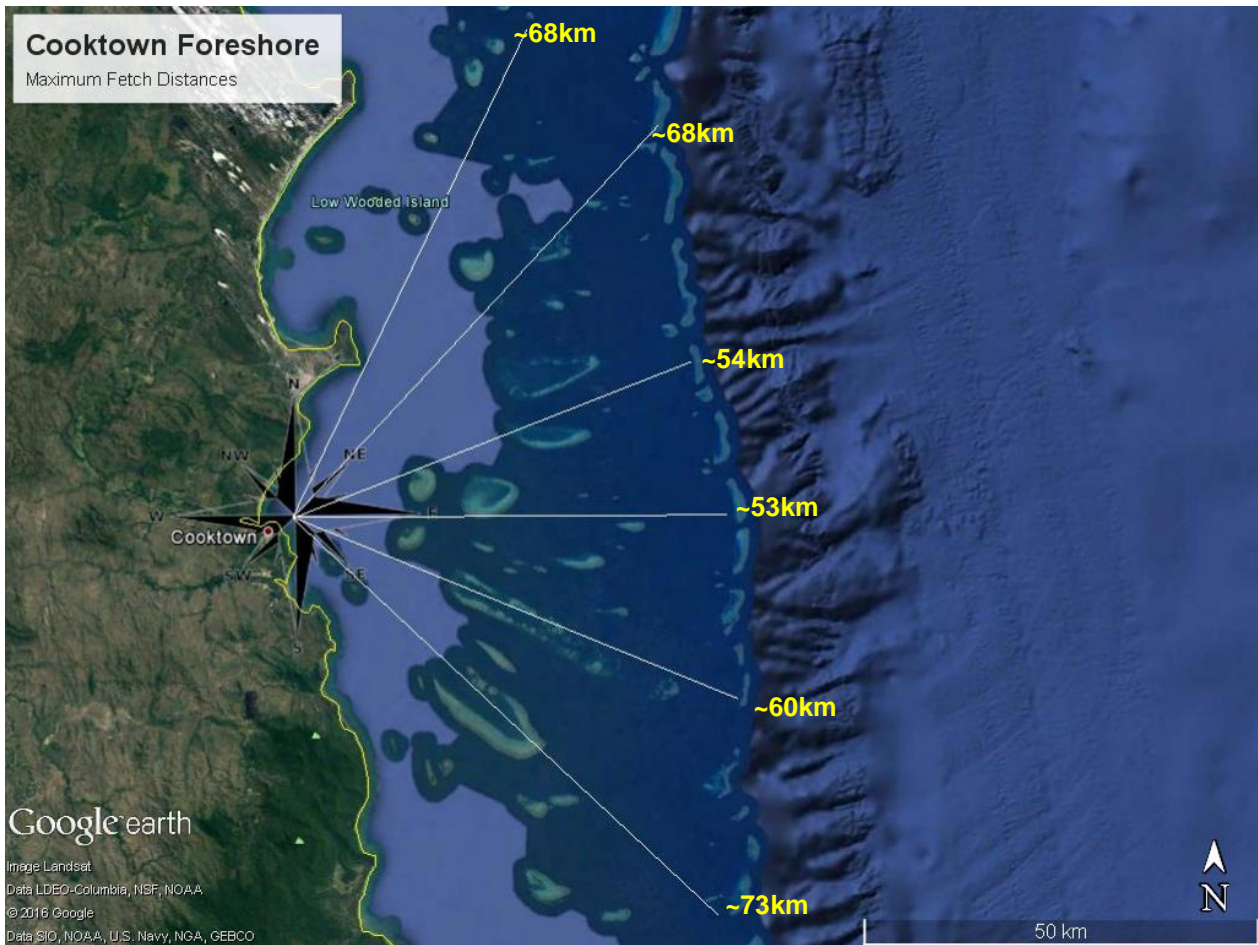


Figure 7 Representation of approximate maximum fetch distances from the Cooktown foreshore (Google Earth, 2016).

The seawall site is most exposed to the fetches located from the NNE to ENE. However, due to the complex nature of the bathymetry, islands and coral cays of the inner GBR and the sheltered nature of the study site, simple nomograms cannot be used to calculate wave heights produced along these fetch distances. This means that the longer fetch distances (SE) will not always produce the largest waves at the study site and instead a numerical spectral wave model should be used to correctly quantify wave growth and transformation within the GBR lagoon.

The domain for the SW model can be seen in Figure 8. Unstructured flexible mesh resolution varies from approximately 1km offshore to 20m in the nearshore areas around the Cooktown channel entrance, with varying resolution in areas of numerical interest; i.e. abrupt changes in bathymetry around islands and coral cays.

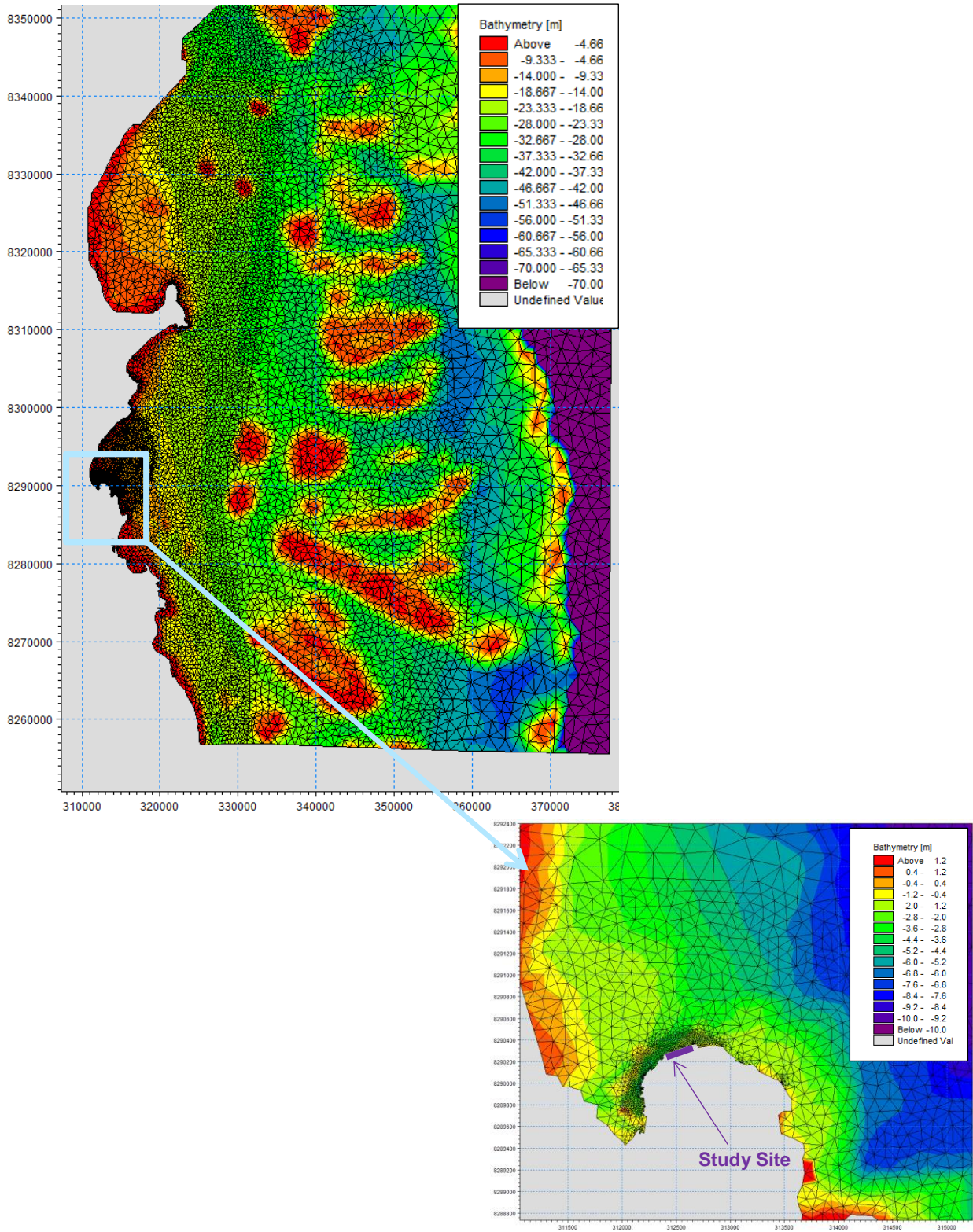


Figure 8 MIKE21FM SW domain used for the study covers both the wave generation and wave transformation areas.

3.2.2 Model Forcing and Parameterisation

Six (6) different simulations of the model domain specified in Figure 8 were run based upon the six water levels determined in the ARI analysis (Figure 5 and Table 3). The wave model was simulated using the following model forcing:

- Spatially uniform wind field using the ARI wind speeds and directions provided in Table 4
- Steady state solution, providing a fully developed sea state for each condition.

This matrix approach was used to simulate all six (6) ARI water levels for each of the six (6) ARI wind speeds. These 36 model simulations were completed for each wind direction (on a 16-point compass) between 22.5°N to 135°N (i.e. six (6) wind directions). A total of 216 simulations were completed.

The SW model was run using a quasi-stationary time formulation, meaning the time is removed as an independent variable and a steady state solution is calculated at each time step. The model used a fully spectral formulation, based on the wave action conservation equation, where the directional-frequency wave action spectrum is the dependant variable. These selections were made to ensure efficiency of model run times and to correctly resolve wind-wave growth through the selected domain.

The time series of wind speed and direction applied across the domain (for each water level iteration) can be seen in Table 4. The wind speeds have been converted to 10 minute averages (as per the WMO (2008)).

Table 4 Wind forcing for Cooktown MIKE21 SW model

ARI	Wind Dir	Angle (°)	Wind Speed (m/s) (ocean)
1	NNE - SE	22.5 - 135	25
10	NNE - SE	22.5 - 135	40
20	NNE - SE	22.5 - 135	44
30	NNE - SE	22.5 - 135	47
50	NNE - SE	22.5 - 135	51
100	NNE - SE	22.5 - 135	54

4 Results

Wave parameters were extracted at nine (9) pre-determined locations in front of the Webber Esplanade seawall, as defined in (Byrne, 2016). These locations can be seen in Figure 9.

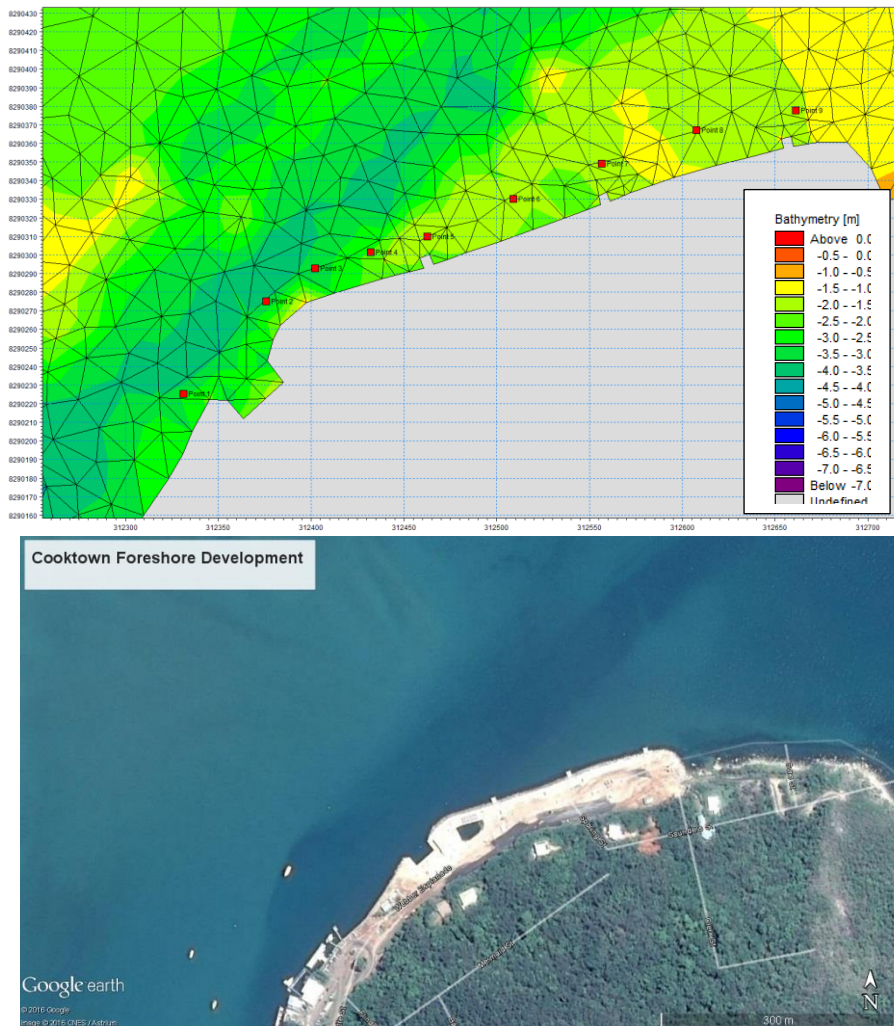


Figure 9 MIKE21FM SW wave data extraction points.

Plots of combined 1, 10, 20, 30, 50 and 100 year ARI water level (+ storm surge) and wind speed (and direction) wave heights, peak wave period (T_p) and peak wave direction (Dir) at each of the nine extraction points are provided in Appendix A.

From the results it can be seen that waves generated along the NE and NNE fetches have the largest Significant Wave Heights (H_s). It can also be seen that there is very little variation in H_s between each of the extraction points for each model scenario (water level and wind speed combinations). A contour map showing the dependence of wave height generation to water level and wind speed along the NNE and NE fetches can be seen in Figure 10 and Figure 11.

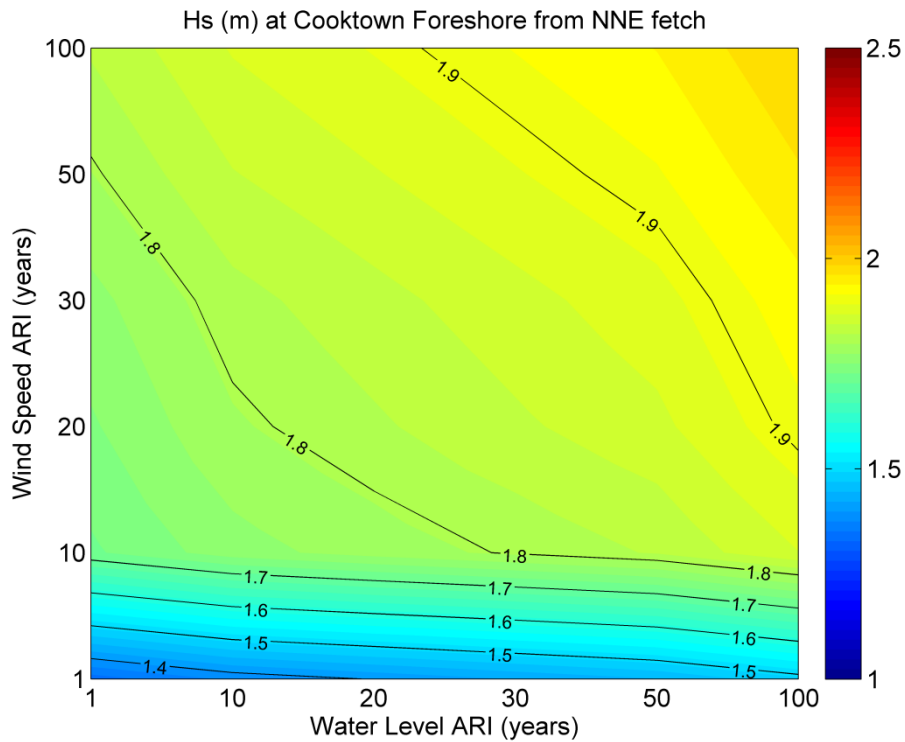


Figure 10 Water level and wind speed influence on Hs along a NNE fetch offshore from the Cooktown Foreshore Seawall.

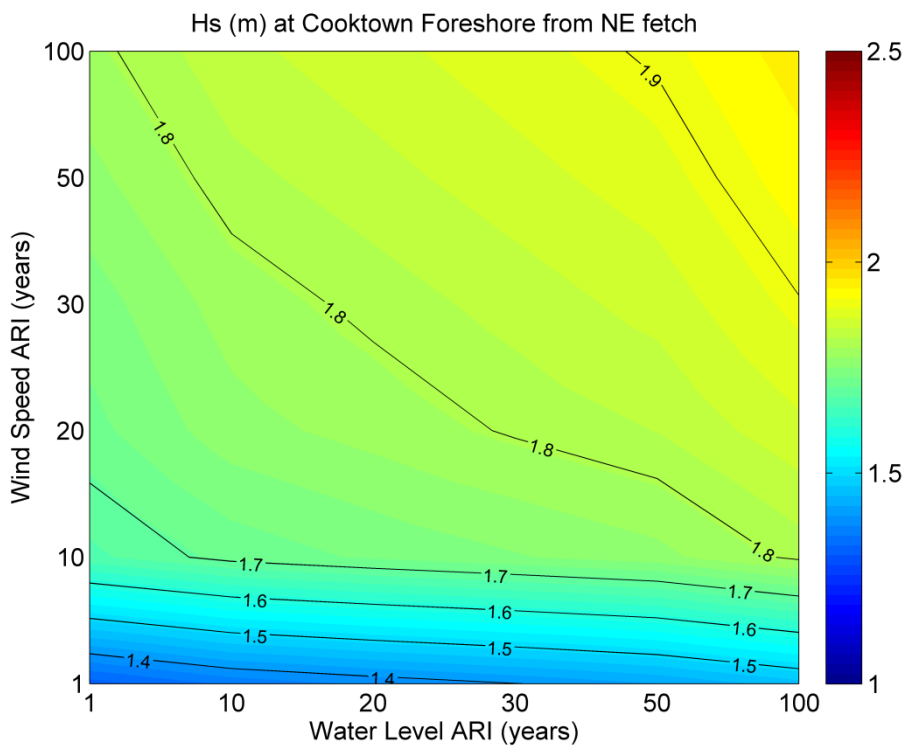


Figure 11 Water level and wind speed influence on Hs along a NE fetch offshore from the Cooktown Foreshore Seawall.

Figure 12 and Figure 13 show the transformation of waves across the model domain. Filtering of wave heights on the outer GBR can be seen as well as the growth of wind-waves from the NE in the inner

portion of the GBR. Focussing of wave heights in the Cooktown entrance channel can be seen due to the deeper bathymetry allowing the waves to travel further in shore compared to adjacent shallow areas dissipating wave energy due to depth-induced wave breaking.

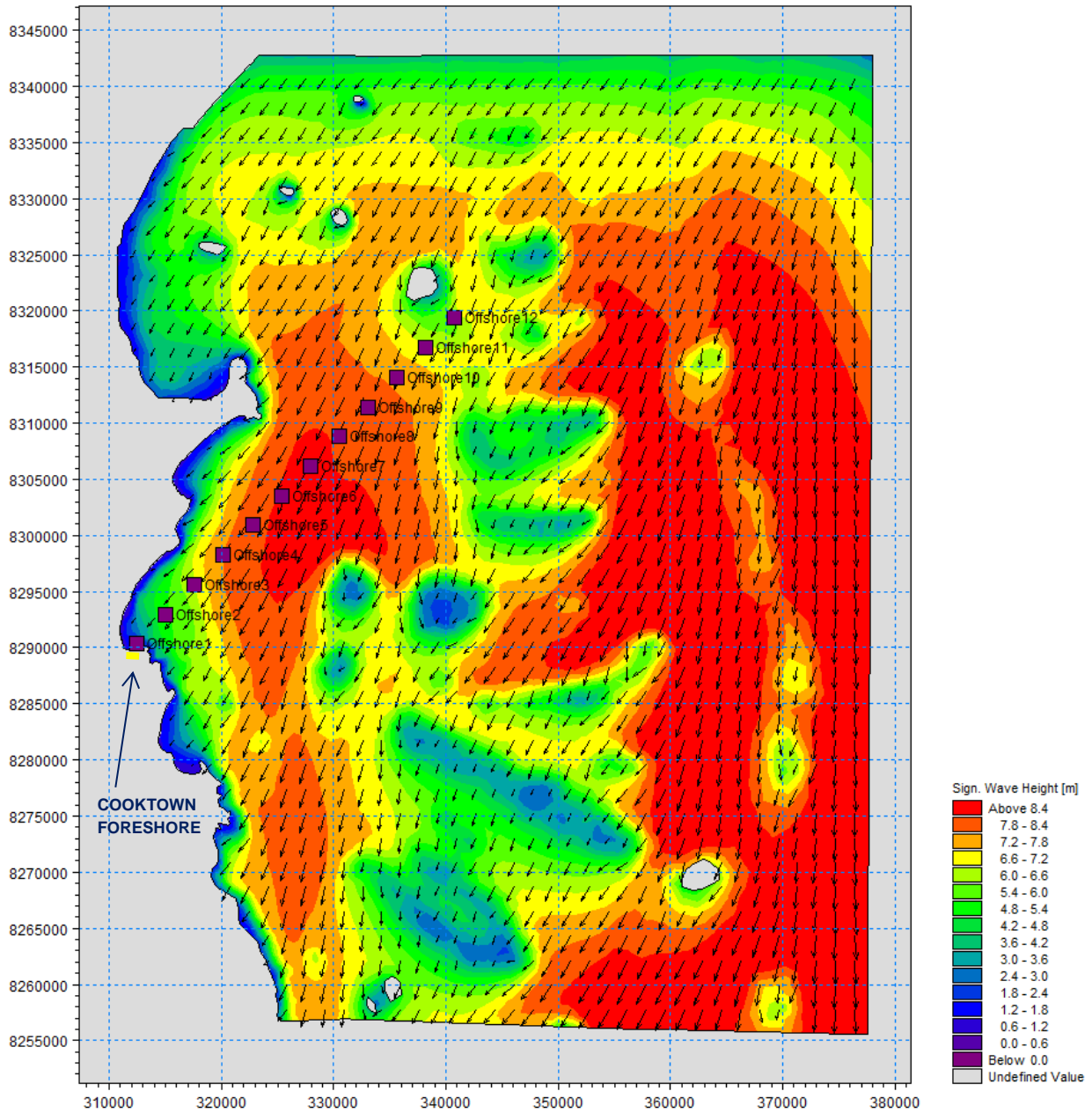


Figure 12 Significant wave height, H_s (m) from 100 year ARI Wind Speed across a NNE fetch at 100 year ARI water level, regional view.

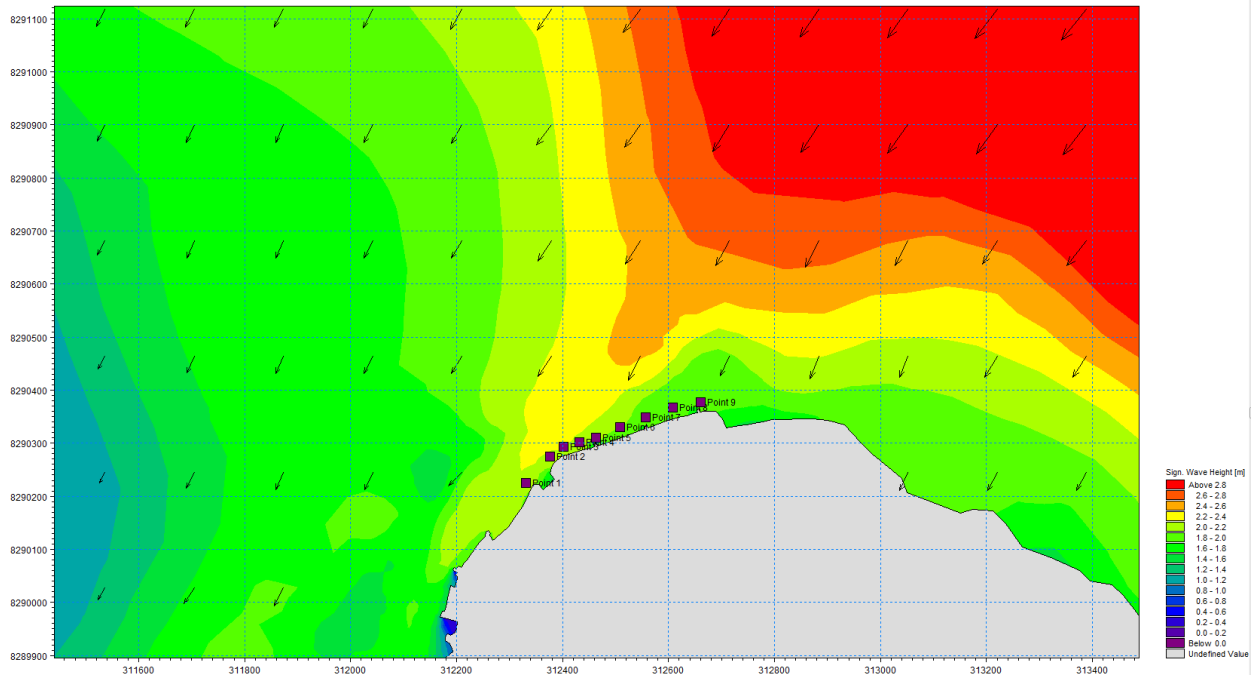


Figure 13 Significant wave height, H_s (m) from 100 year ARI Wind Speed across a NNE fetch at 100 year ARI water level, local view.

Extraction points (OP1 - OP12) across the model domain shown in Figure 12 have been plotted for each model iteration along the NNE fetch to gain an understanding of wave transformation through the domain, Figure 14. This plot also provides the offshore significant wave heights for the various ARI wind speed and water level conditions modelled.

Hs (m) transformation to Cooktown foreshore from NNE fetch

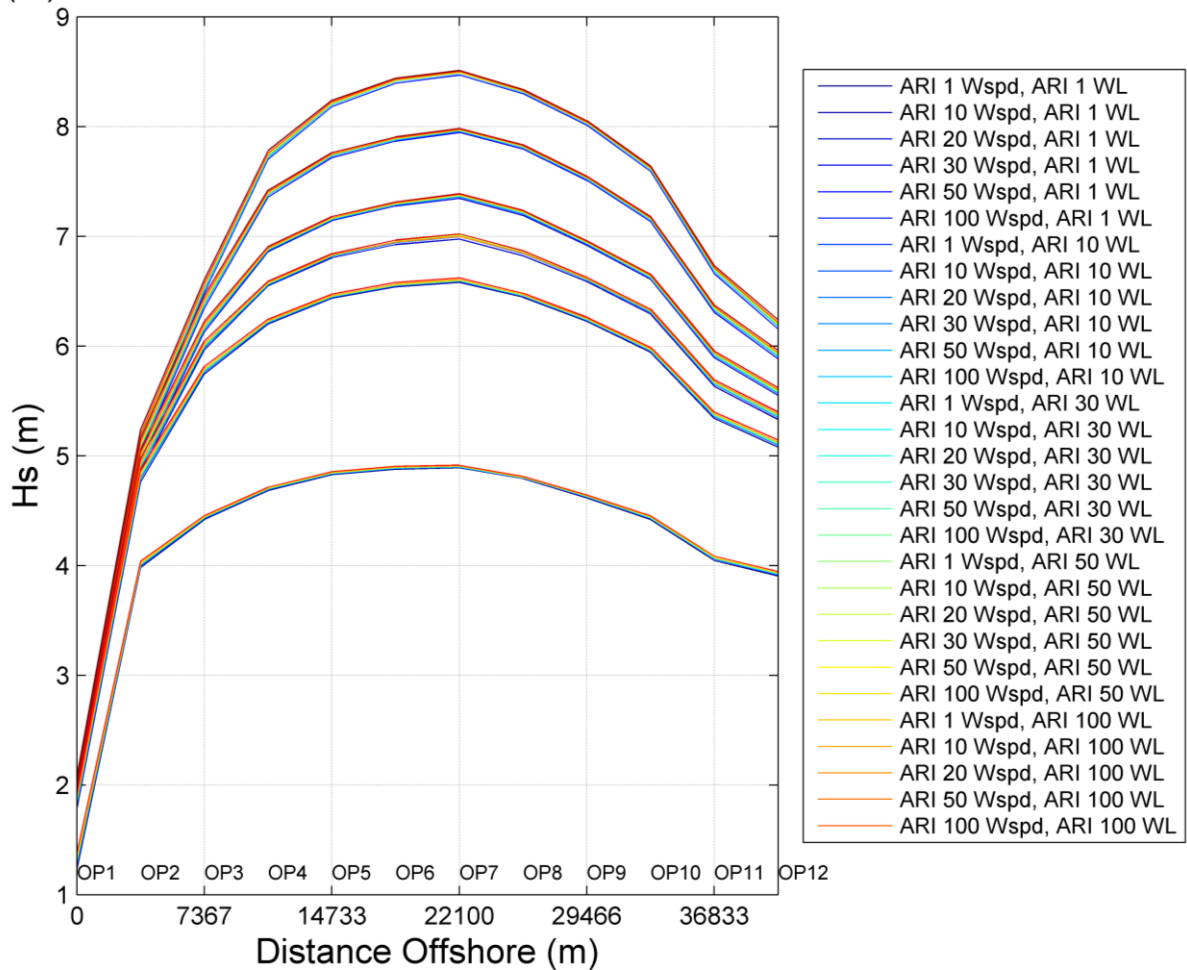


Figure 14 wave transformation (H_s) through the model domain to the Cooktown foreshore for different combinations of wind speed and water level.

5 Discussion

It can be seen from Figure 17 that H_s values at each of the locations appear to be reaching a maximum (~2.25m) this is most likely due to depth-limited wave breaking adjacent to and offshore of the entrance channel. Each of the extraction points in the vicinity of the seawall are located in between approximately 1.5 to 3m (at MSL) water depth (Figure 9) which means waves will become depth-limited due to breaking at $H_s = 1.2$ to 2.3m. This illustrates the dependence of wave height to water level at the study site as increase in water depth at the site results in increased wave heights.

In order to adhere to a conservative design approach, ARI wave parameters were taken from the worst-case scenario fetch orientation or wind direction i.e. the NNE. As wave parameters are seen to vary little between each extraction point, a representative point, Point 5 (approximate centre location) will be taken for analysis of design ARI events. Wave parameters recorded at this location have been summarised in Table 5 to Table 7.

Table 5 Significant wave heights, H_s (m) taken from NNE fetch model simulations at Extraction Point 5

		ARI WATER LEVEL					
		1	10	20	30	50	100
ARI WIND SPEED	1	1.25	1.29	1.31	1.33	1.35	1.39
	10	1.75	1.79	1.81	1.83	1.85	1.89
	20	1.78	1.82	1.84	1.86	1.89	1.93
	30	1.80	1.84	1.86	1.89	1.91	1.95
	50	1.83	1.87	1.90	1.92	1.94	1.99
	100	1.86	1.90	1.93	1.95	1.97	2.02

Table 6 Peak Wave Periods T_p (sec) taken from NNE fetch model simulations at Extraction Point 5

		ARI WATER LEVEL					
		1	10	20	30	50	100
ARI WIND SPEED	1	8.2	8.2	8.2	8.2	8.2	8.2
	10	3.9	3.9	4.0	4.0	6.0	8.1
	20	3.9	3.9	3.9	3.9	4.0	4.0
	30	3.8	3.9	3.9	3.9	3.9	4.0
	50	3.9	3.9	3.9	3.9	3.9	4.0
	100	3.9	3.9	3.9	3.9	4.0	4.0

Table 7 Peak wave direction, Dir (deg) taken from NNE fetch model simulations at Extraction Point 5

		ARI WATER LEVEL					
		1	10	20	30	50	100
ARI WIND SPEED	1	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)
	10	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)
	20	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)
	30	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)
	50	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)
	100	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)	23 (NNE)

Due to the orientation of the seawall and the offshore channel, nearly all of the waves offshore of the seawall come from the NNE (Figure 18, Table 7) with peak wave periods associated with the larger wave heights seen to be below 4 seconds (Figure 19, Table 5 and Table 6). Longer period events (>8 sec) are seen to occur across nearly all fetch orientations under smaller wave heights, Figure 19. The separation in wave periods can be attributed to complex wave processes in and adjacent to the entrance channel

causing a shift in the wave energy distribution, i.e. a shift towards higher frequencies; these tend to be more prevalent under larger events. This frequency shift can be seen in Figure 15 and occur 4 to 7km offshore from the study site in approximately 10m to 15m water depth (at MSL).

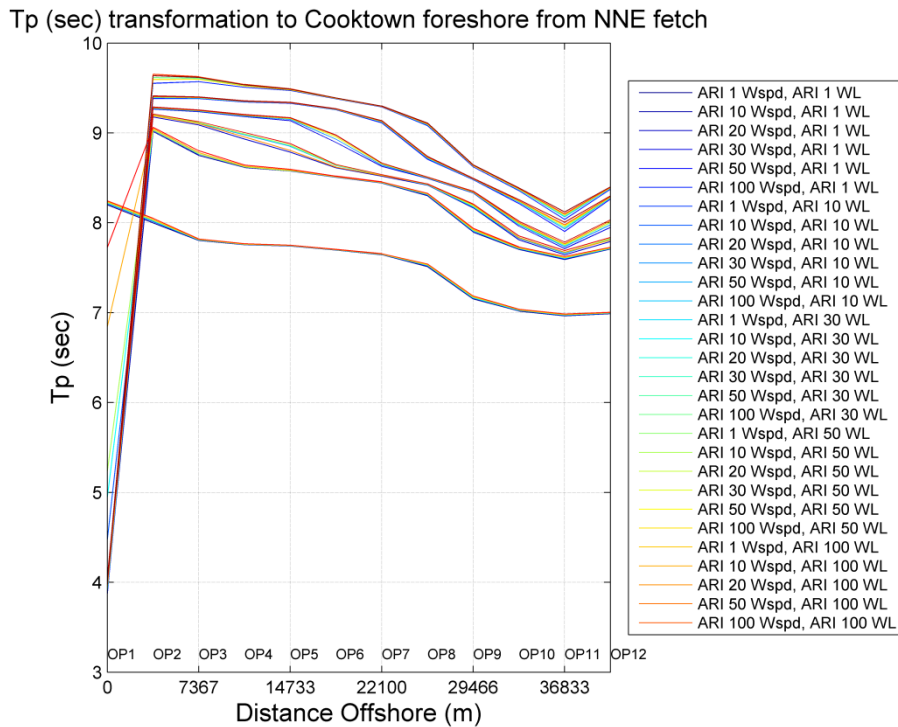


Figure 15 Transformation of peak wave period (T_p) through the model domain to the Cooktown foreshore for different combinations of wind speed and water level.

Wave heights at this location can be seen to be independent of water level variation for each wind scenario. Table 8 shows that there is very little variation compared to the water level fluctuation at Offshore Point 2 (OP2), which is in approximately 6m water depth.

Table 8 Significant wave heights, H_s (m) taken from NNE fetch model simulations at Offshore Point 2 (OP2)

		ARI WATER LEVEL					
		1	10	20	30	50	100
ARI WIND SPEED	1	3.98	4.00	4.01	4.02	4.02	4.04
	10	4.76	4.80	4.81	4.83	4.85	4.88
	20	4.85	4.89	4.91	4.93	4.94	4.98
	30	4.93	4.96	4.98	5.00	5.02	5.05
	50	5.03	5.07	5.09	5.11	5.13	5.16
	100	5.10	5.14	5.16	5.18	5.20	5.24

In order to sufficiently determine ARI wave events from a combination of ARI wind speed and water levels, it is not sufficient to simply assume a 100 year ARI water level combined with a 100 year ARI wind speed will result in a 100 year ARI wave event. Although there will be an expected correlation between increased water level and wind speed at this location due to cyclone-induced storm surge, there is no clear indication

as to when a cyclone will cross (or impact) a coastline in relation to tidal phase. As such, large storm surges may be impacting the site on the low water phase of the tide and the resultant net water level may still be below mean sea level (MSL).

Due to the selection of the ARI wind event based on Australian Design Standards as well as the ARI water levels having been taken from recommended model values (JCU, 2004), a dependency relationship of water level on wind speed cannot be made. In this case (and at the request of the client) a reduced return period for each of the water level and wind speeds has been taken to determine design (100 year ARI wave levels) wave events at the seawall. Combinations of the 20 year ARI and 50 year ARI water levels and wind speeds and their impacts on the wave parameters are shown in Table 9 and Table 10.

Table 9 Wave conditions at the seawall for 20 year ARI water level + 50 year ARI wind speed (from NNE)

	Seawall Point								
	1	2	3	4	5	6	7	8	9
Hs(m)	1.96	2.03	2.08	2.00	1.90	1.88	1.79	1.75	1.74
Tp (Sec)	3.7	3.8	3.8	3.8	3.9	4.0	4.1	4.3	4.6
Dir(degN)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	6(N)	23(NNE)

Table 10 Wave conditions at the seawall for 50 year ARI water level + 20 year ARI wind speed (from NNE)

	Seawall Point								
	1	2	3	4	5	6	7	8	9
Hs(m)	1.85	1.95	2.00	1.94	1.89	1.88	1.81	1.77	1.75
Tp (Sec)	3.7	3.8	3.8	3.8	4.0	4.2	8.5	9.2	9.2
Dir(degN)	23(NNE)	24(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	23(NNE)	45(NE)

It can be seen that there is a gradient in wave period from the western seawall points (1-6) in comparison to the eastern points (7-9) in Table 10. The separation in wave period over this bathymetry under these design conditions can be seen in Figure 16. This can be attributed to complex wave processes occurring due to the difference in offshore bathymetry at the outer seawall points (7-9) from the abrupt change due to the entrance channel offshore of seawall points 1-6, as seen in Figure 8 and Figure 9.

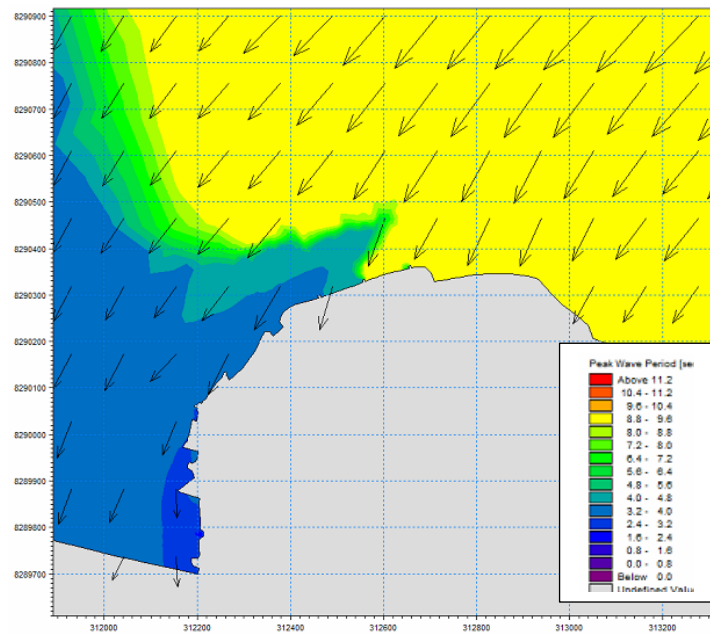


Figure 16 Peak wave period, T_p (sec) for a 50 year ARI water level + 20 year ARI wind speed from a NNE fetch at Cooktown

Although under larger wave heights the wave modelling results have shown that T_p along the seawall reduces, it is advised that the higher T_p values seen for the lower ARI events (8.5 to 9.2 sec) should be used in the design calculations.

5.1 Limitations

In the absence of locally available measured wave data, the wave model used in this study has not been calibrated. Calibration of the wave model would provide a greater level of confidence in the adopted modelling approach, particularly considering (i) the potential sensitivity to the model boundary location, (ii) lack of measured wind speed at Cooktown which can be considered representative of offshore conditions, (iii) the sheltering effect of Cape Bedford observed in model results, and (iv) the limited extent of measured bathymetric data in the area of the seawall and further offshore.

The model results may be sensitivity to the selection of the northern and eastern model boundaries. The location of the boundaries was selected based on the experience of the modeller and the available aerial imagery and bathymetry. A sensitivity analysis to these model boundary locations has not been undertaken as it was beyond the scope of this study.

Complex wave interactions due to reflections from the seawall, wave-current interactions from discharges from the Endeavour River have also not been included in the scope of works of this study.

Due to the nature of the study, the values given above will be conservative and as such to ascertain a true ARI wave event value it is recommended to re-analyse both the wind and water level data following a longer recording period to determine the dependency between the two.

6 References

Byrne, T. 2016. *Cooktown Foreshore Development – Technical Review*. Cook Shire Council Communication.

Harper, B.A, 1999. *Numerical Modelling Of Extreme Tropical Cyclone Winds*. Journal of Wind Engineering and Industrial Aerodynamics 83 (1999) pp35-47

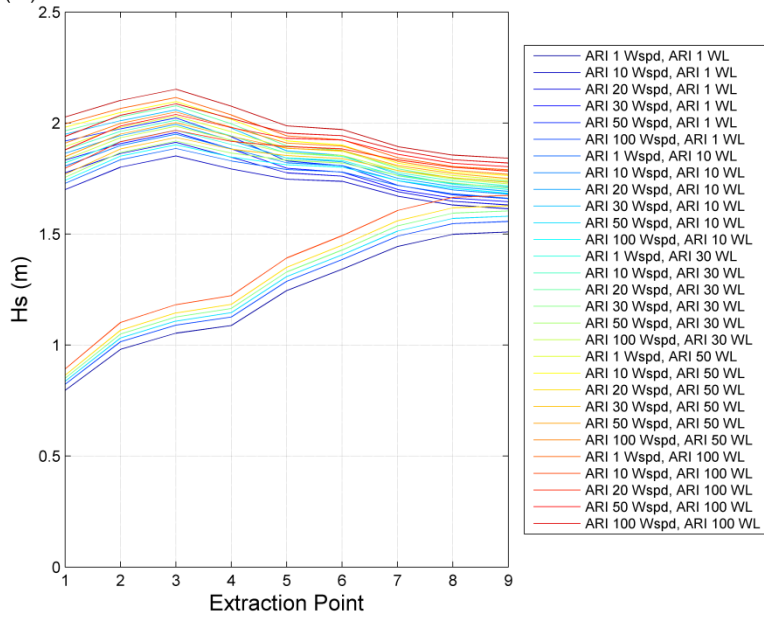
James Cook University (JCU), 2004. *Queensland Climate Change and Community Vulnerability to Tropical Cyclones: Ocean Hazards Assessment – Stage 3*, Report July 2004.

Lourensz, R.S. 1981. *Tropical Cyclones in the Australian Region July 1909 to June 1980*. Australian Bureau of Meteorology, October 1981.

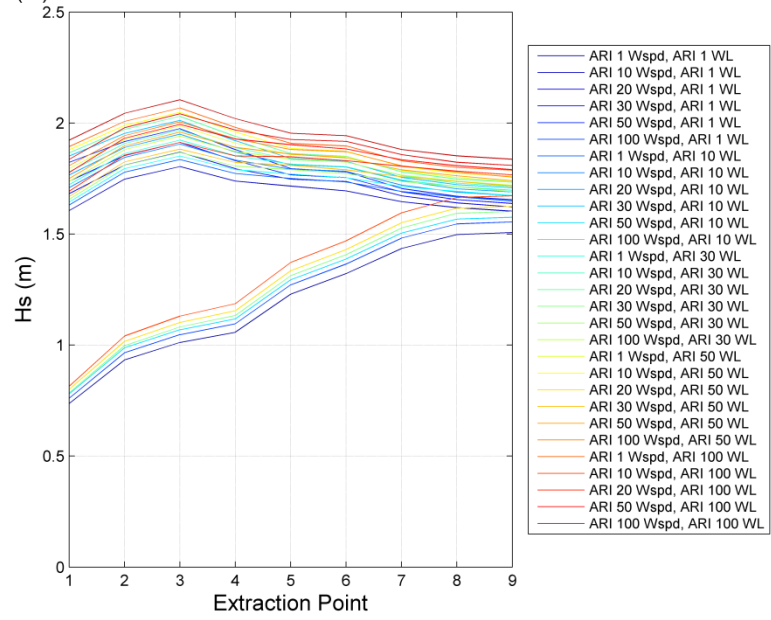
World Meteorological Organization (WMO), 2008. *Guidelines For Converting Between Various Wind Averaging Periods In Tropical Cyclone Conditions*, Appendix II, October 2008

Appendix A Combined ARI Wave Parameters

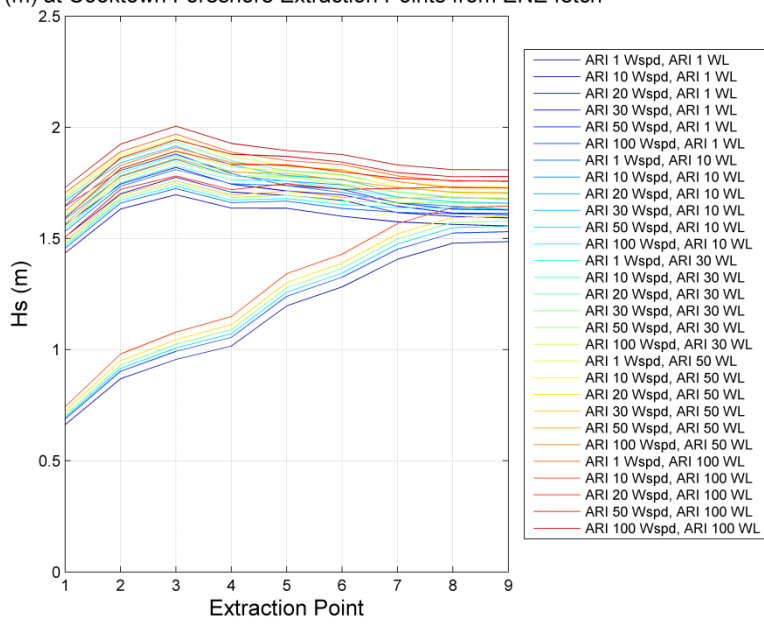
Hs (m) at Cooktown Foreshore Extraction Points from NNE fetch



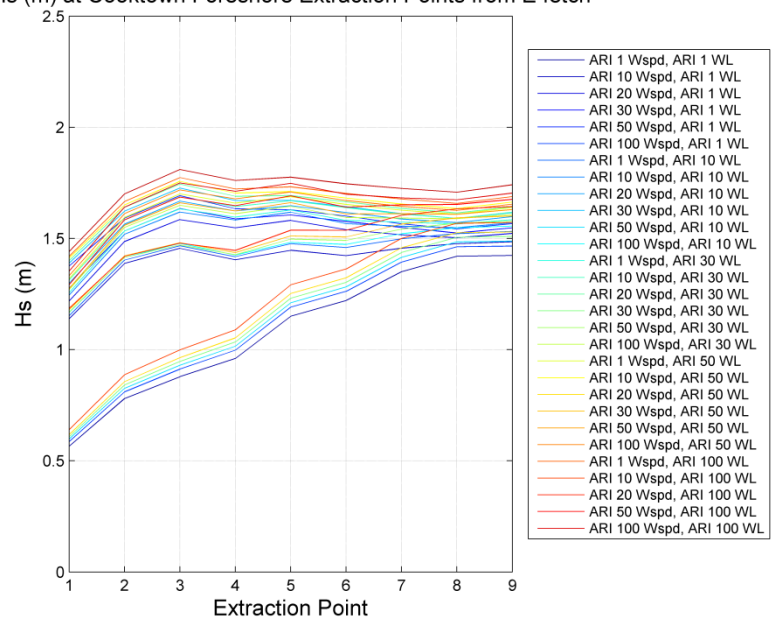
Hs (m) at Cooktown Foreshore Extraction Points from NE fetch



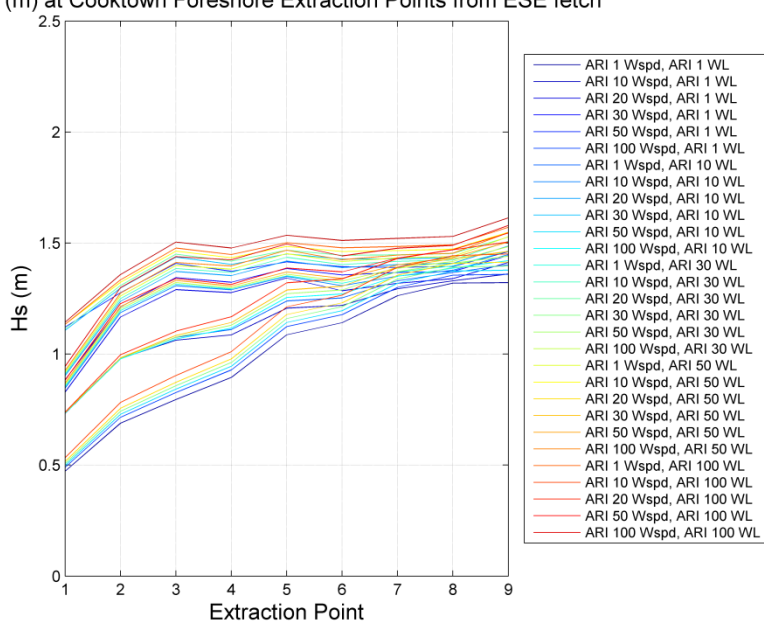
Hs (m) at Cooktown Foreshore Extraction Points from ENE fetch



Hs (m) at Cooktown Foreshore Extraction Points from E fetch



Hs (m) at Cooktown Foreshore Extraction Points from ESE fetch



Hs (m) at Cooktown Foreshore Extraction Points from SE fetch

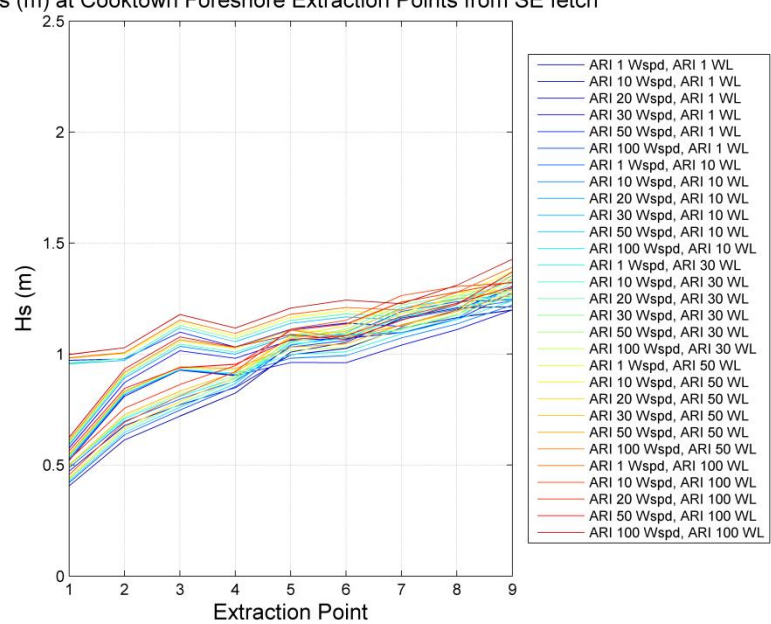
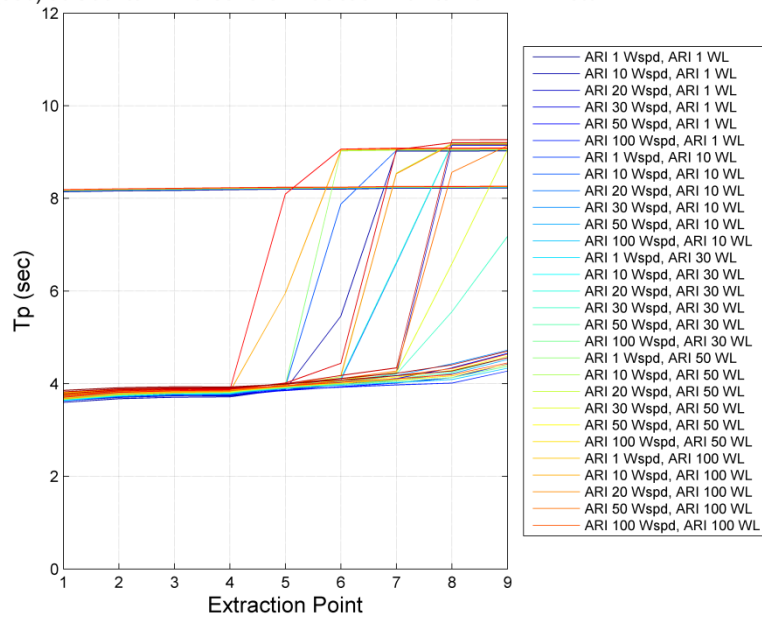
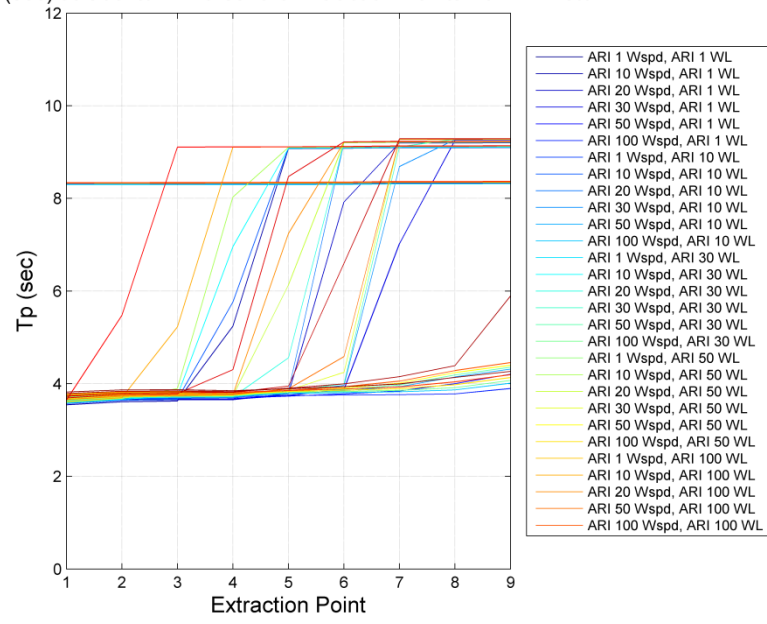


Figure 17 Combined 1, 10, 20, 30, 50 and 100 year ARI water level (+ storm surge) and wind speed (and direction) wave heights at each of the nine extraction points.

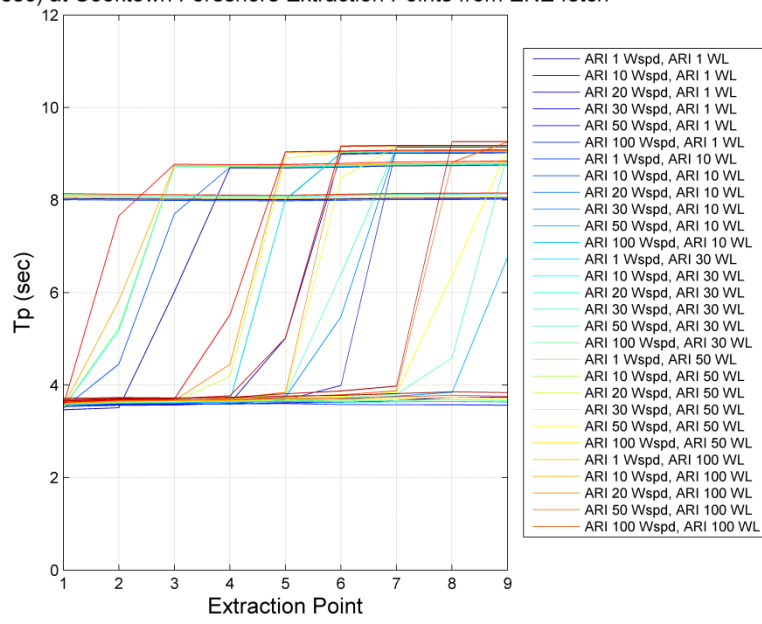
Tp (sec) at Cooktown Foreshore Extraction Points from NNE fetch



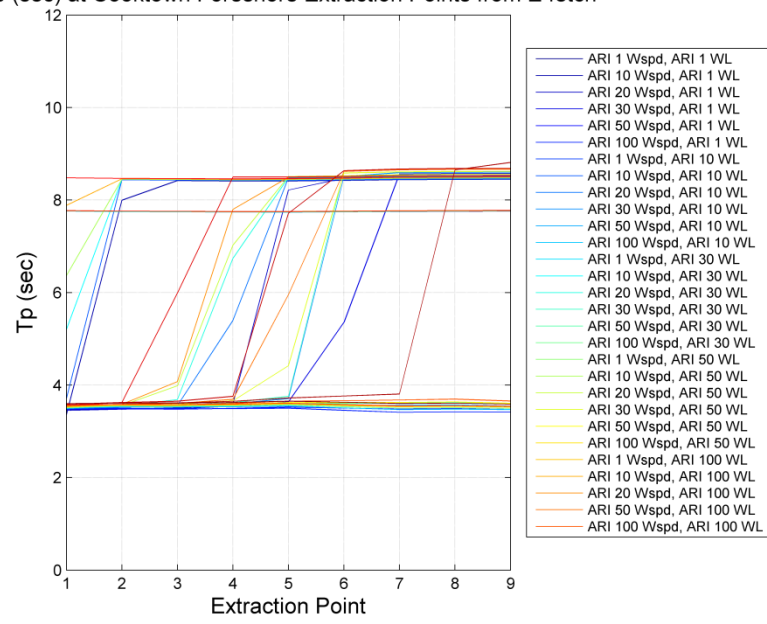
Tp (sec) at Cooktown Foreshore Extraction Points from NE fetch



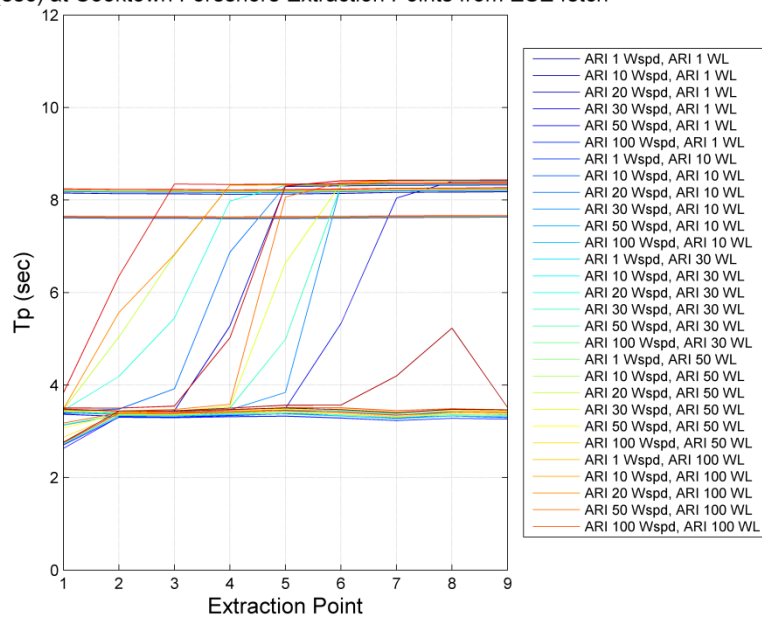
Tp (sec) at Cooktown Foreshore Extraction Points from ENE fetch



Tp (sec) at Cooktown Foreshore Extraction Points from E fetch



Tp (sec) at Cooktown Foreshore Extraction Points from ESE fetch



Tp (sec) at Cooktown Foreshore Extraction Points from SE fetch

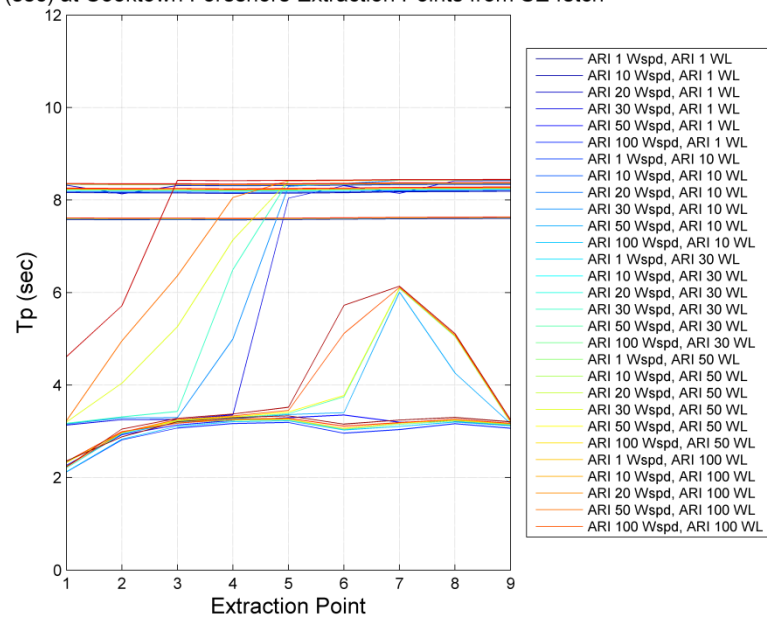
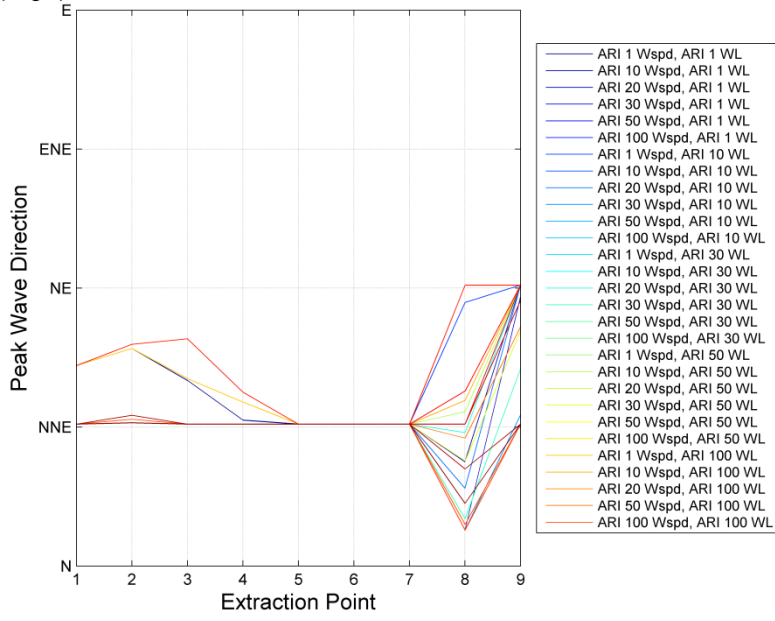
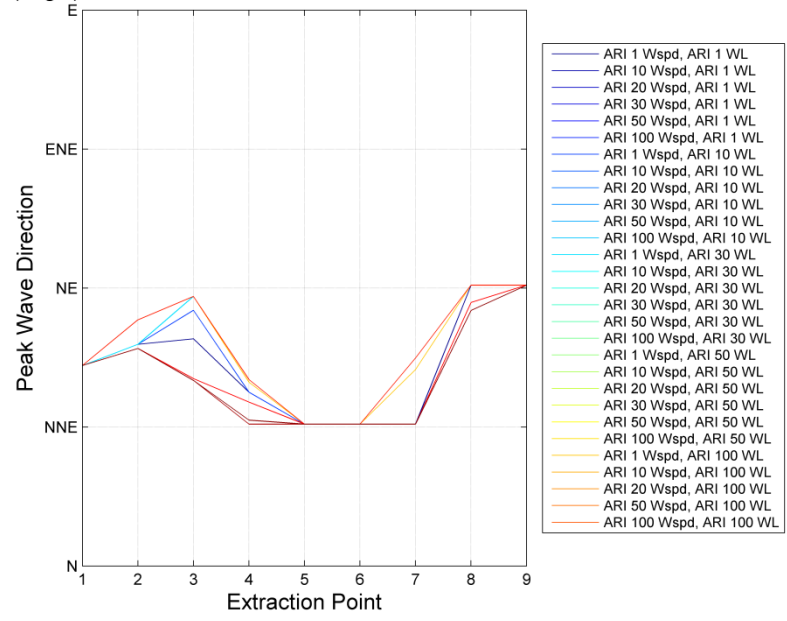


Figure 18 Combined 1, 10, 20, 30, 50 and 100 year ARI water level (+ storm surge) and wind speed (and direction) wave periods at each of the nine extraction points.

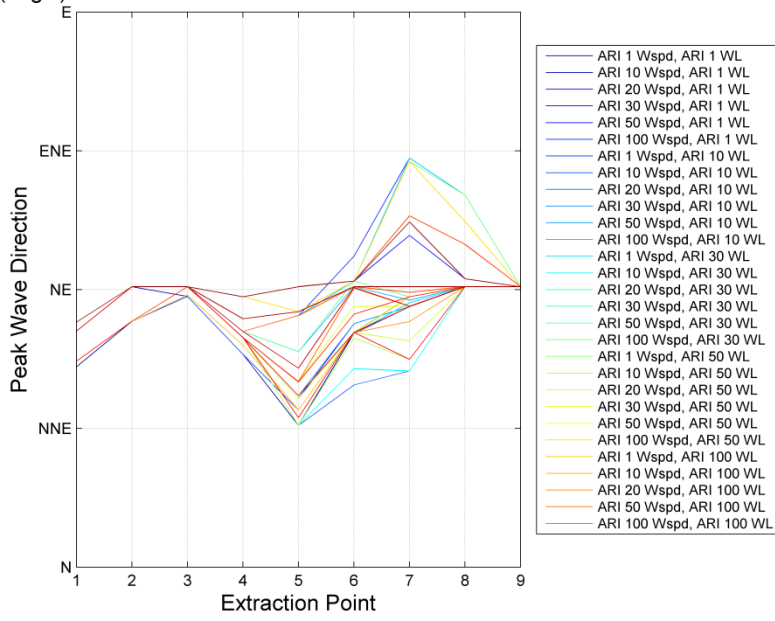
Dir (degN) at Cooktown Foreshore Extraction Points from NNE fetch



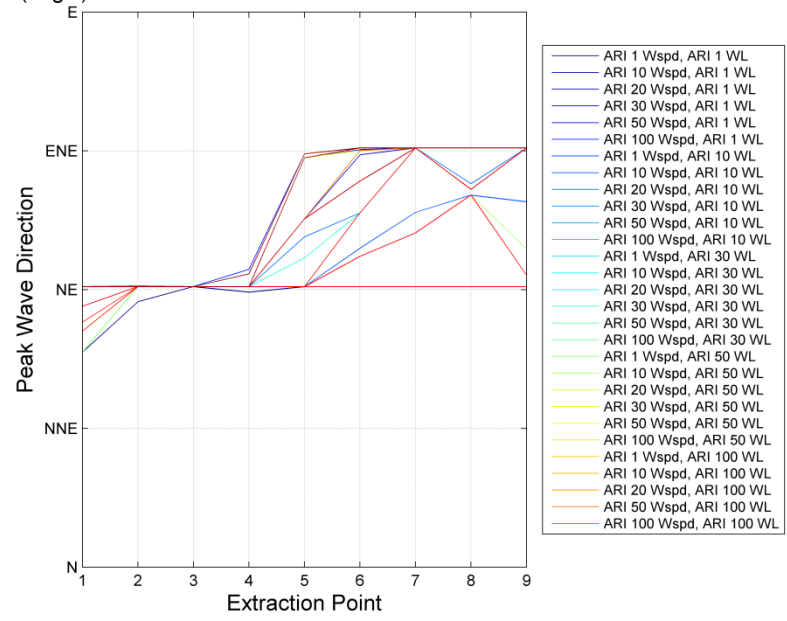
Dir (degN) at Cooktown Foreshore Extraction Points from NE fetch



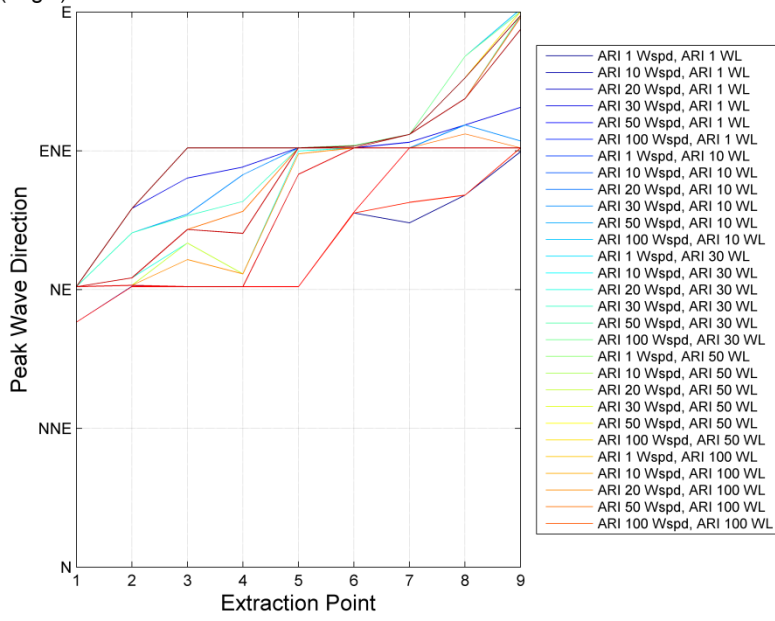
Dir (degN) at Cooktown Foreshore Extraction Points from ENE fetch



Dir (degN) at Cooktown Foreshore Extraction Points from E fetch



Dir (degN) at Cooktown Foreshore Extraction Points from ESE fetch



Dir (degN) at Cooktown Foreshore Extraction Points from SE fetch

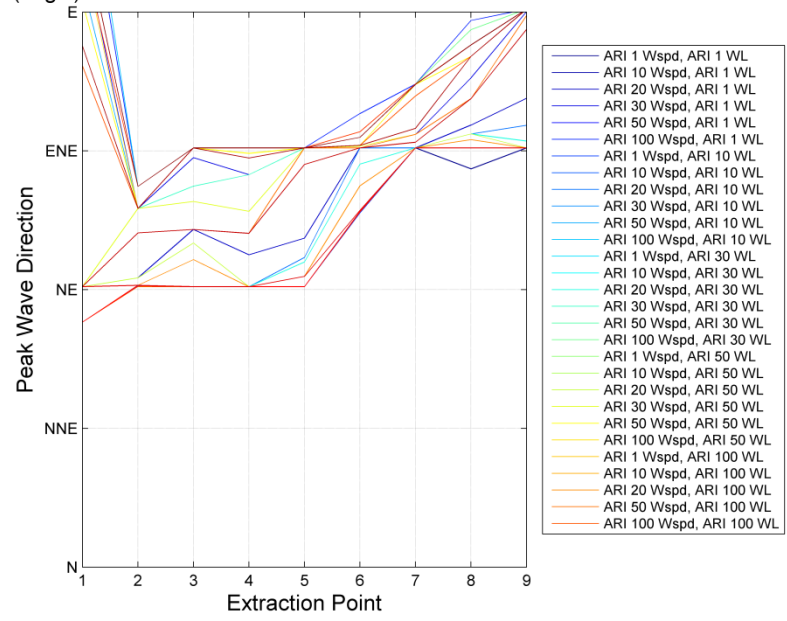


Figure 19 Combined 1, 10, 20, 30, 50 and 100 year ARI water level (+ storm surge) and wind speed (and direction) wave directions at each of the nine extraction points.