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JERSEY FUTURE HOSPITAL

CO004 – SITE OPTION REPORT

APPENDIX 6 Technical Site Appraisal

TN-TI-001 - TECHNICAL NOTE – PRELIMINARY TIDAL IMPACT STUDY

QUALITY ASSURANCE

Sign off: Peter Thomas

Position: Senior Engineer

April 2015

States of Jersey

Jersey Future Hospital

Preliminary Tidal Impact Study

TN-TI-001

P3 | 02 April 2015

Job number 237035

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ARUP

Document Verification

ARUP

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<tr><td></td><td>Prepared by</td><td>Checked by</td><td>Approved by</td></tr>
<tr><td>Name</td><td>Kambiz Ayoubkhani</td><td>Nick Ashby</td><td>Peter Thomas</td></tr>
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<tr><td>Name</td><td>Kambiz Ayoubkhani</td><td>Peter Thomas</td><td>Nick Ashby</td></tr>
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<tr><td>Name</td><td>Kambiz Ayoubhkani</td><td>Kambiz Ayoubhkani/Peter Thomas/Nick Ashby</td><td>Gareth Williams</td></tr>
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Issue Document Verification with Document ✓

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Overtopping Analysis

1 Introduction

This technical note has been prepared to support the preparation of the Site Validation Exercise that forms Change Request Nr. 4 as part of the Jersey Future Hospital Scheme.

The four options being reviewed as below:

Option A - Dual Site Options

Option B - Overdale Hospital Site, 100% New Build Option

Option C - Existing General Hospital, 100% New Build Option

Option D - Waterfront Site, 100% New Build Option

These four options are associated with three sites as detailed in Figure 1 below.

Figure 1 Site Locations

The risk of flooding is primarily from extreme tidal events casing wave overtopping of the sea defences and flood water collecting in the lower areas of St Helier.

The Overdale site is located at high ground and there will be no risk of flooding from wave overtopping. Therefore for the purposes of this report, the Overdale site has not been considered further.

This report provides the findings as the outcome of a review of the available information. It also recommends further work that is recommended to be undertaken as part of the next design stages. This will inform the next stages' design progression including completion of a comprehensive Flood Risk Assessment should one of options A, C or D be considered as the preferred option.

2 Scope of Study

This report is based on previous flooding assessments Arup have undertaken in 2011 for commercial developments in this area of St Helier. In addition, a Digital Terrain Map (DTM) of the area has been supplied by the States which has been used to create a 2D surface and plot contours.

The report does not review the risk of flooding from:

- Sewer Flooding
- Overland pluvial flooding
- Groundwater flooding
- Flood risk to basements
- Fluvial Flooding

It should be noted that the Overdale Hospital site will need to be assessed for the above mentioned flood risks.

3 Wave overtopping

Drawings of the coastal wall structures were obtained from TTS in 2011(Appendix A) which were used to analyse the wave overtopping.

The EurOtop, "Wave Overtopping of Sea Defences and Related Structures: Assessment Manual" was used to derive wave overtopping volumes. This is standard industry guidance for predicting the magnitude of wave overtopping and provides methods for determining the overtopping discharges for given met-ocean conditions on standard wall structure types.

The assessment considers the probability of an extreme tide and wind generated wave occurring simultaneously.

Four structures were considered to be vulnerable to wave overtopping that could reach the sites:

- Vertical sea wall running along Victoria Avenue (Type 1)
- Slipway to the south of Victoria Park (Type 2)
- Concrete terrace sea wall adjacent to Les Jardins de la Mer (Type 3)
- Rock armour revetment to the south of the terraced wall section. (Type 4)

The location of these different wall types is shown in the figure 2 below. This figure also shows the wave direction that was considered within this analysis. A wave direction of approximately 230° allows waves to enter St Aubins Bay and reach Victoria Avenue without obstruction, resulting in the maximum wave overtopping potential.

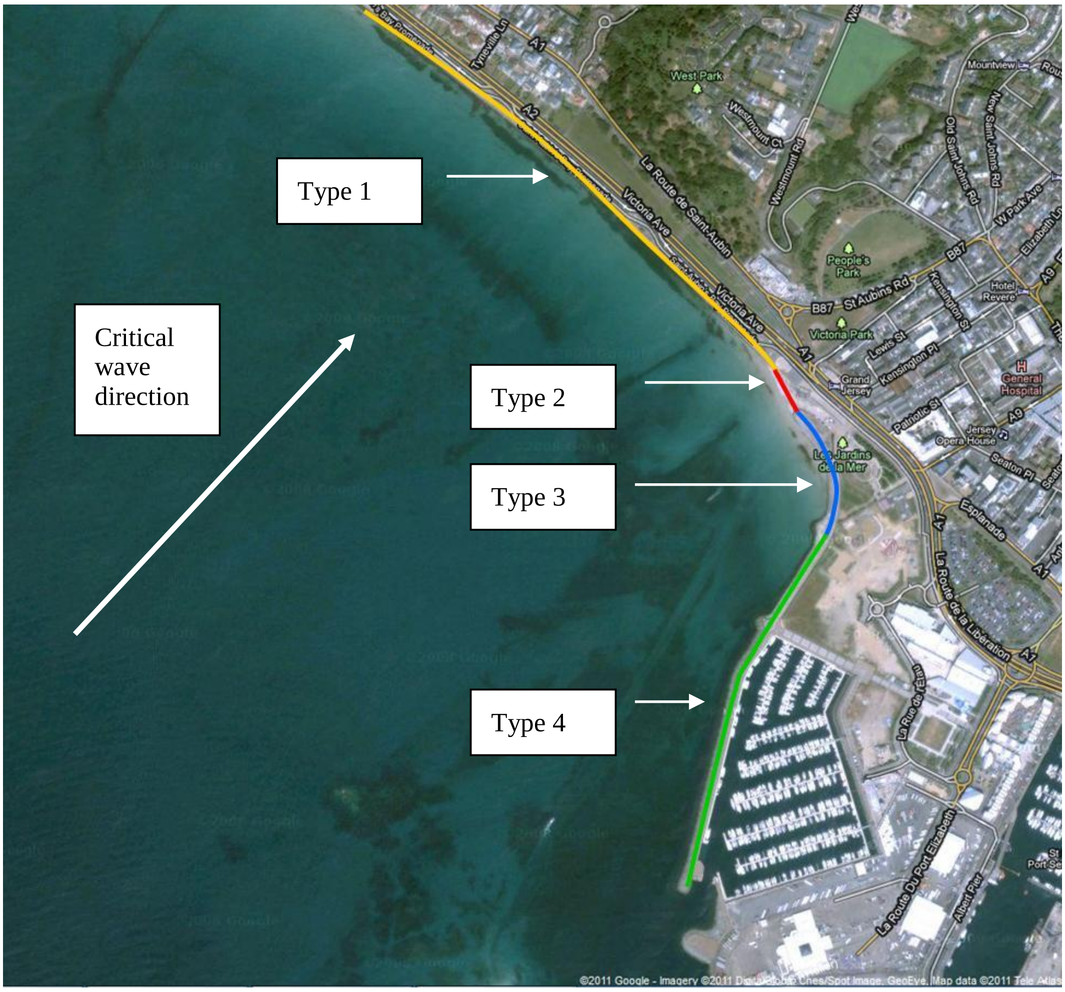


Figure 2 - Wall section locations

3.1 Victoria Avenue (Type 1)

The Victoria Avenue wall, reproduced in Appendix A, is a steeply sloped masonry structure with a curved lower section. Along some sections there is a stepped toe. Figure 3 below is a photograph of this wall type.

Figure 3 - Wall Type 1 photograph

The wall geometry was taken from the drawing provided by TTS with the crest of the wall at +9.1mOD and the toe at +2.0mOD.

Figure 4 below is an image from the BBC Jersey news website taken during the March 2008 floods showing the wave overtopped volume discharging into Victoria Avenue.

Figure 4 - Overtopped discharge down Victoria Avenue (BBC Jersey)

Figure 5 below shows the partial collapse of the flood defences following the 2014 storms.

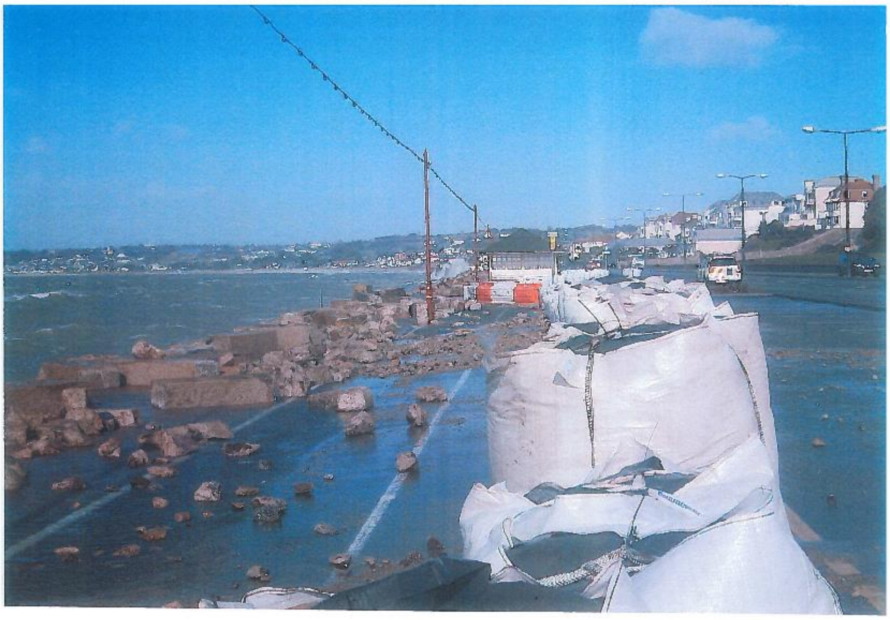


Figure 5 – Partial collapse of defences in 2014 storms

3.2 Slipway (Type 2)

Figure 6 below shows a photograph of the slipway, wall Type 2.

Figure 6 - Wall Type 2 - slipway

The slipway has a width of 10m, a maximum slope angle of approximately 5% and a crest height of +8.0mOD. The surface is made up of blocks, resulting in a reduction factor to account for the increased surface roughness.

3.3 Terrace Revetment (Type 3)

Drawings of this wall section were received from TTS, reproduced within Appendix A. This wall structure is made up of a sloped revetment of terraced blocks placed on a rock filter layer and central rubble core. The toe of the wall is supported on a line of sheet piles driven to rock level. A photograph of this section of wall is shown in the figure 7 below.

Figure 7 - Wall Type 3

From the drawings provided by TTS the crest level and slope angle were determined to be 9.7mOD and 1:2 respectively.

3.4 Rock Armour Revetment (Type 4)

Drawings of this section of wall was received from TTS and are reproduced in Appendix A. This section of wall is a rock revetment type structure with a double layer of 2 to 3 tonne rock armour placed on a stone rubble core. A photograph of the structure is shown in Figure 8 below.

Figure 8 - Wall Type 4

From the drawings provided by TTS the crest level and slope angle were determined to be 9.7mOD and 1:2 respectively.

4 Wave Overtopping Discharges

The overtopping assessment is included in Appendix B. The critical section of wall for wave overtopping is the vertical wall along Victoria Avenue. This vertical wall geometry produces the largest overtopping volumes compared to the more sloped sections of the other wall types. The vertical wall along Victoria Avenue is also the only wall type subject to perpendicular head-on wave impact which significantly increases the overtopping volumes.

4.1 Boundary Assessment

To determine the overtopping volumes across three scenario bounds the following assumptions have been varied. This is to allow a range of values to be presented. It is worth noting that the empirical formulae used to determine wave overtopping volumes can never accurately model the complex wave, structure interactions that take place during an overtopping event for a given set of met-ocean conditions. As such they are intended as a guide to the order of magnitude.

For the lower bound approach the current day water levels have been used. This acts to increase the free-board at the wall structure reducing the overtopping volume. A probabilistic approach has also been adopted which uses a lower confidence factor to assess the resulting discharge volumes.

For the upper bound approach the predicted future water levels and a deterministic approach were adopted. The increased water levels will act to reduce the free-board of the wall structures increasing overtopping. Using a deterministic approach applied a higher confidence factor within the discharge volume determination, increasing the resulting volumes by almost a factor of 2.

The middle bound overtopping volumes were calculated as an average of the upper and lower bounds.

The overtopping volumes from the calculations in Appendix B seem to be high given that reported flooding is less frequent than the figures would suggest. This is especially the case for the middle and upper bound figures. It should be noted that the HR Wallingford Study (Jersey Coastal Management Study 1991) produced even higher figures for a similar section of sea wall further to the West along Victoria Avenue.

This apparent anomaly can possibly be explained by:-

- Localised flooding from wave overtopping at Victoria Avenue is unreported because it is short lived and no sensitive structures are affected.
- Part of the overtopped water usually flows into the local storm drainage network and where possible back into the sea
- Historic overtopping would have been less frequent because sea levels are rising.

5 Flood routes and levels assessment

Figure 9 below illustrates the principal flow routes from the overtopping source. The key points to note are as follows:-

- There is a 'flood hump' in the region of the Gloucester Street and Esplanade junction. This is at a level of +7.4mOD and as such will act to block surface water from travelling down Esplanade and Gloucester Street until it is overtopped. This hump will cause the backing up of water behind it during a flood event.
- Once flood waters overcome the above flood hump, water will pass down Gloucester Street, Seaton Place and the Esplanade. The area is relatively low lying so the flood water will pond in this area and begin to spread over a wider area.
- Flood water will then continue up Castle Street and Commercial Street to the North East and South East respectively.
- At the junction of Esplanade and Castle Street flood flows will not pass this point until overtopping a ridge in the road at approximately +7.25m OD. Beyond this level, water will pass down towards the marina and the southern entrance to the underpass of the La Route de la Liberation.

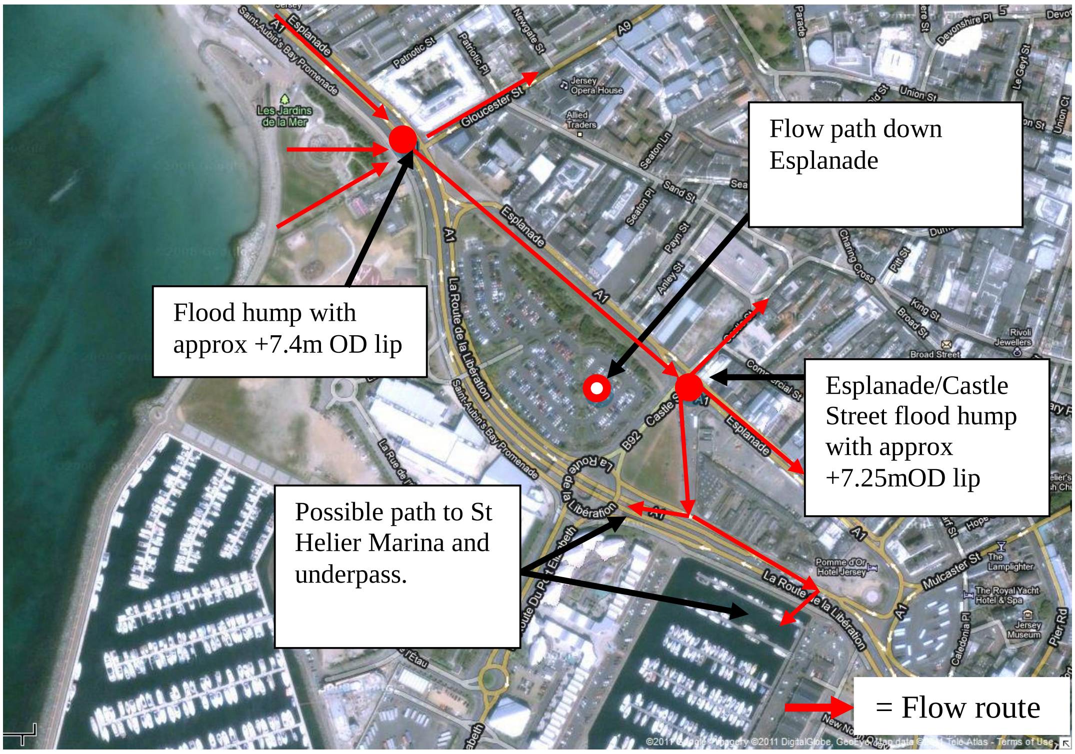


Figure 9 – Principal Flow Routes and Key Levels

Appendix B summarises the conclusions of the wave overtopping assessment.

The following variables were altered to determine lower and upper bound overtopping volume rates across the different wall types. The middle bound was the average of the upper and lower bounds.

Lower bound

- Current day water levels were used (i.e. no allowance for climate change)
- A 10% reduction in wave height was allowed for wave transformation inshore
- A reduced correlation between maximum extreme water levels and wave heights was used (approximately 0.5). Ie. an extreme wave event of return period 50 years was combined with an extreme water level with return period 25 years.

A probabilistic assessment was used. This relates the fit of the empirical wave overtopping model to the recorded data. Using the probabilistic tools 50% of the recorded data points exceed the model prediction and 50% fall below the predicted values.

Upper bound

- Future (50 year) water levels were used.
- No wave height reduction was allowed for – to allow for possible wave concentration.
- A correlation of 1.0 between extreme water levels and wave heights was assumed. Such that the combination of events with the same return period was considered, i.e. a 50 year return period wave event was combined with a 50 year water level event.

Table 1 below reproduced from Appendix B, indicates the volumes of flood water likely to be overtopped for varying return periods and lower to upper bound conditions.

Return Period	**1 year**	**10 year**	**20 year**	**50 year**	**100 year**
	Volume	**Volume**	**Volume**	**Volume**	**Volume**
Bound	**[m³]**	**[m³]**	**[m³]**	**[m³]**	**[m³]**
Lower	3,850	21,500	33,500	56,000	135,000
Middle	11,425	62,250	96,250	155,500	257,000
Upper	19,000	103,000	159,000	255,00	379,000

Table 1 – Wave overtopping volumes

SK-TI-001 shows the depth contours obtained from the Digital Terrain Model at 0.5m intervals ranging from 6m OD to 8.50m OD.

A 3D surface of the contours has been created and draped over the DTM surface to calculate the storage volume contained below each contour interval. Table 2 below indicates the calculated storage volumes available.

Depth Interval m	Storage Volume m3
6.00 – 7.0	3,540
6.00 – 7.50	74,130
6.00 – 8.0	177,740
6.00 – 8.50	392,330
6.00 – 9.00	638,960

Table 2 – Storage volumes available

Comparison of predicted flood volumes in table 1 with the available storage volumes in table 2 indicates that based on the simplified assumption that all flood water will be evenly stored within the low lying areas, the flood levels are unlikely to rise above 8.50m OD.

6 Conclusions

The available information, which is some 4 years old, indicates that in extreme tidal and wave events, the sea defences are likely to be overtopped and flood parts of St Helier. In the absence of comprehensive hydraulic modelling and calculations to latest data sets which will be undertaken at subsequent design stages, the significant difference in volumes, for a 1 in 100 return period in the upper bound, between the depth intervals 6.00-8.50m and 6.00-9.00m provides reassurance that the 8.50m OD flood levels remain appropriate.

Minor flooding is likely to occur in a 1 year event.

If flood levels were to reach 8.50m OD, the impact on the two proposed sites can be assessed as follows:

6.1 Waterfront Site

The Waterfront site would be flood free as it is at or above 9.00m OD. However. The means of access/egress would be flooded to a maximum depth of some 2.5m.

Options have been considered to provide a flood free access to the site. For the purposes of this study a proposal of the following has been allowed for at this early stage:

- a strengthening and raising of the sea wall.
- relocation of the tidal protection hump further to the north west to prevent potential flooding and enable a new at-grade junction arrangement to be provided.

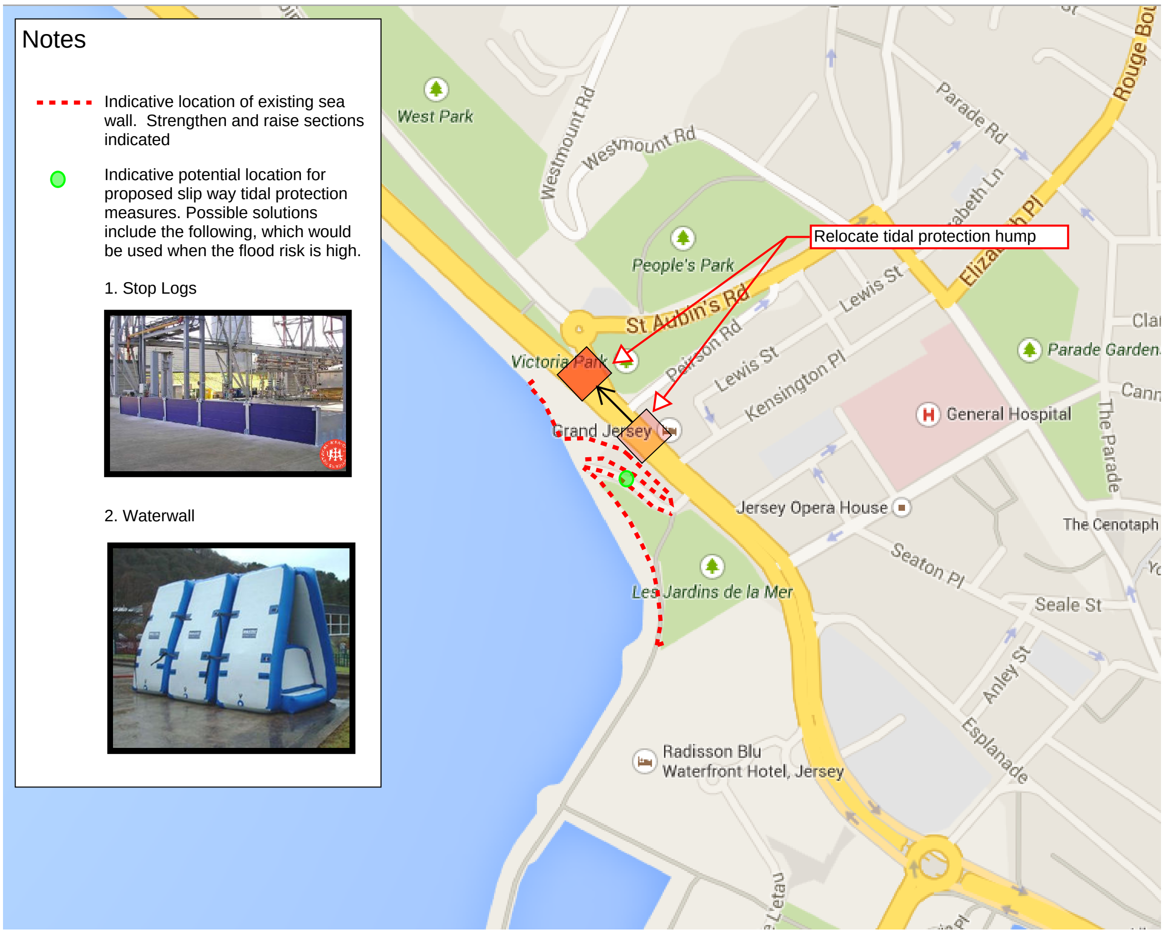


Figure 10 – Waterfront Access Initial Proposals

6.2 Existing General Hospital

The existing hospital site would generally remain flood free, although there is a risk that some basements could be flooded. However, with careful management of the proposed basement thresholds and the provision of flood gates, the hospital can remain flood free and operational.

Access/egress to the site will be available from the higher ground to the north and east but the south western routes onto Gloucester Street and Newgate Street may be inundated.

7 Proposed Further Work

To provide a more robust assessment of flood risk for the proposed hospital sites, it is recommended that the additional information is collated and the study extended as described below:

1. Update the overtopping assessment model to incorporate the latest requirements and recommendations on climate change.
2. Undertake Lidar survey of the low lying areas below 9.50m OD contour with a 2.0m grid and accuracy of +- 150mm.
3. Undertake topographical survey to complement the Lidar survey and accurately determine the level of flood defences, kerb lines and any obstructions to flood routes.
4. Undertake 2D hydraulic modelling using Tuflow or MIKEFLOOD software package to determine flood plain extent and depth and velocity of flood water.
5. Assess impact of sewers, including the flood tunnels on the surface flooding.
6. Assess impact of flooding from other sources such as pluvial, fluvial and groundwater.

Figures

Appendix A

Drawings of Sea Walls

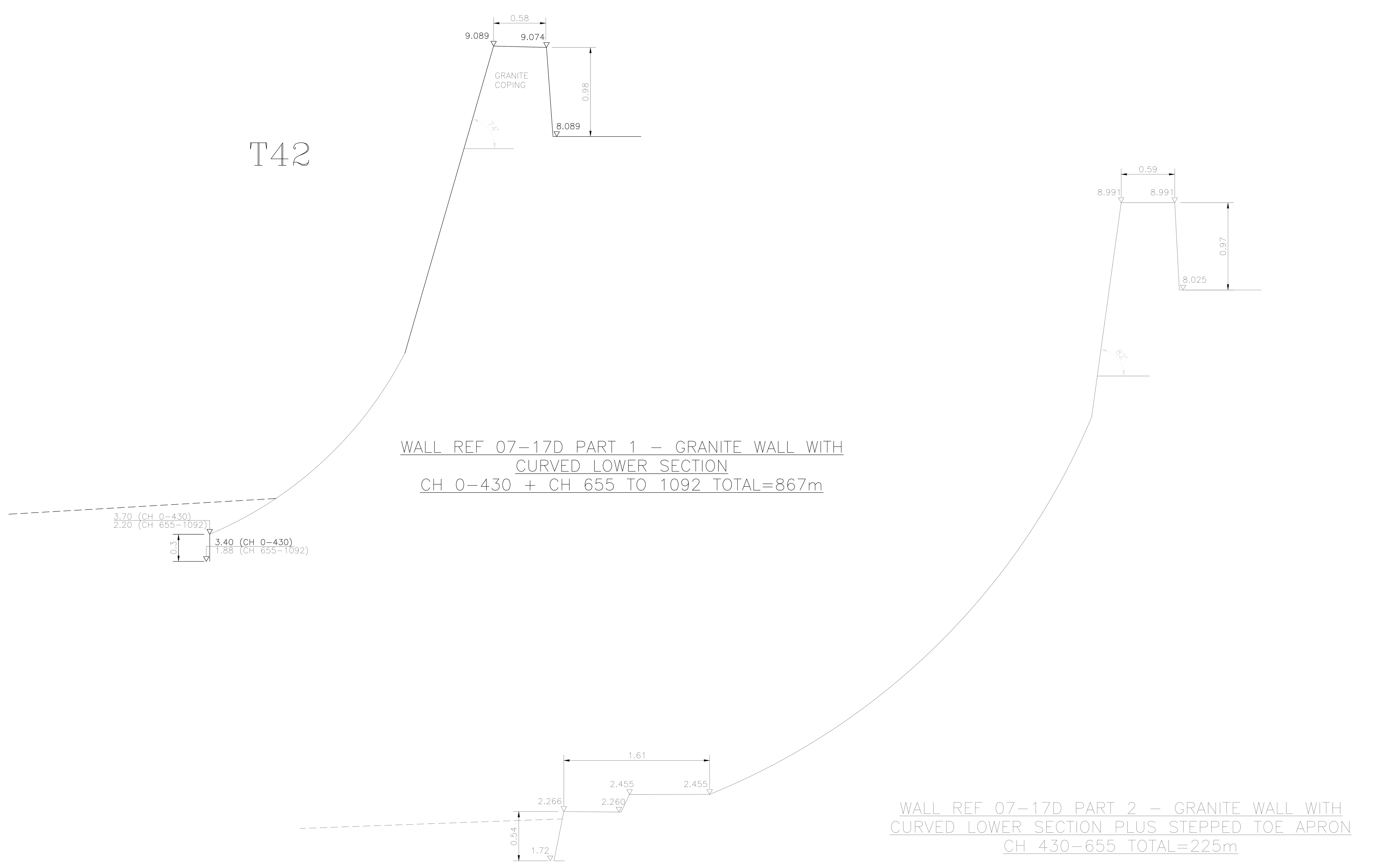
A1 Drawings of Sea Walls

Drg. No. 07-17D-01

NOTES

ALL LEVELS SHOWN ARE m AOD. FOR ADMIRALTY LEVELS ADD 5.892m

NOTE
CH 75-427 AND 470-653 - TOE PROTECTED BY ROCK ARMOUR, PREVENTING EXCAVATION



Rev.	Date	Ints.	Details

STATES OF JERSEY

SOUTH HILL, St.HELIER
JERSEY JE4 8UY
Tel - 01534 601690
Fax - 01534 68950

PUBLIC SERVICES DEPARTMENT

Project
COASTAL DEFENCE STRATEGY

Title
WALL REFERENCE
07-17D
CONSTRUCTION DETAILS

Drawn by : SCS
Designed by :
Checked by :
Project Engineer : CS
Date : JUNE 2002

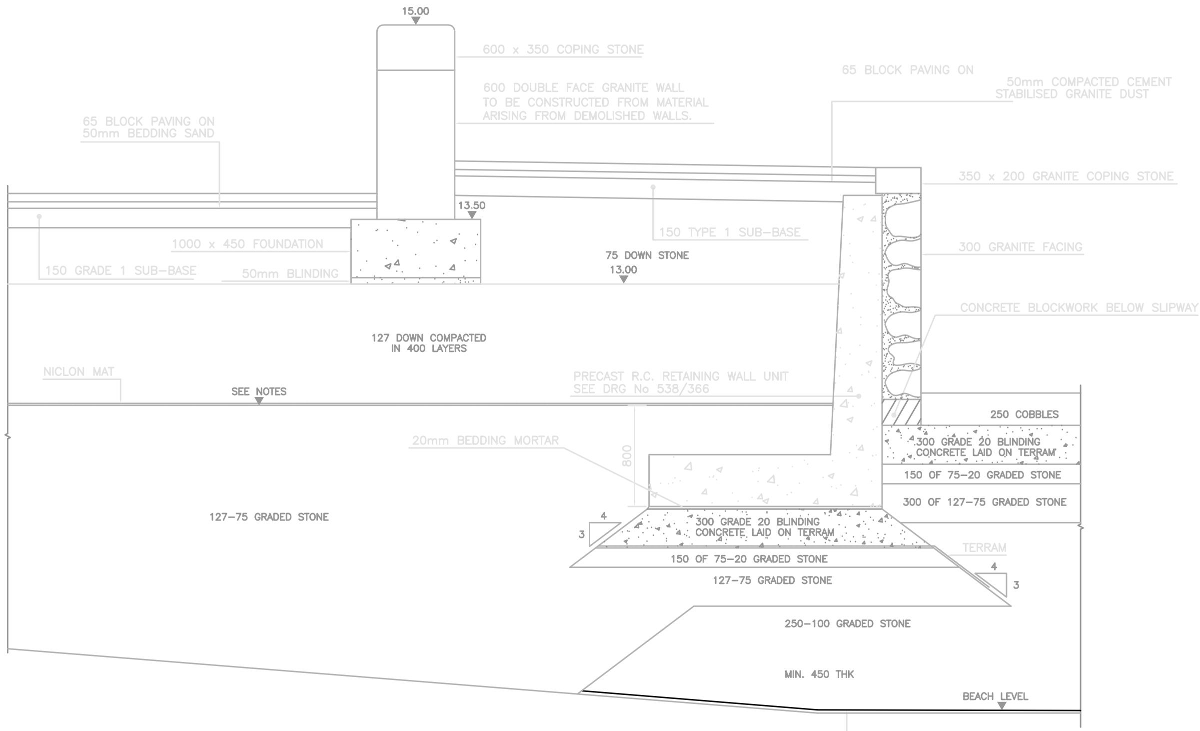
Scales 1:25

Drg. No. 07-17D-01

Drg. No. 538/543 (E)

NOTES

1. FOR SECTIONS D-D & E-E SEE DRG No 538/542
2. FOR SECTION LOCATIONS SEE DRG. No. 538/541
3. FOR G.A. & REINFORCEMENT DETAILS OF PRECAST UNITS SEE DRG. No. 538/366
4. BACKFILLING BEHIND THE PRECAST RETAINING WALL SHALL BE LIMITED TO ONE THIRD OF THEIR HEIGHT UNTIL THE SLIPWAY BLINDING CONCRETE LAYER IS IN PLACE.
5. FOR PAVING DETAILS SEE DRG. No. 538/601



SECTION F-F

AS BUILT

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D	03-01-94	BEL	MOVED FOUNDATION LEVEL AS SHOWN & CHANGED BLOCK PAVING DEPTH
C	16-09-93	BEL	CHANGED FILL MATERIALS AS SHOWN
B	11-06-93	BEL	ISSUED FOR TENDER
A	02-04-93	BEL	ISSUED FOR TENDER

STATES OF JERSEY

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PUBLIC SERVICES DEPARTMENT

Project
WEST OF ALBERT
PHASE II
LAND RECLAMATION

Title
SLIPWAY
DUKW AREA
CROSS-SECTION
SHEET 2

Contract No.	538	Sheet No.	
Designed by :	BEL/CS	Drawn by :	G.M.M
Checked by :	BEL	Date :	MARCH 93

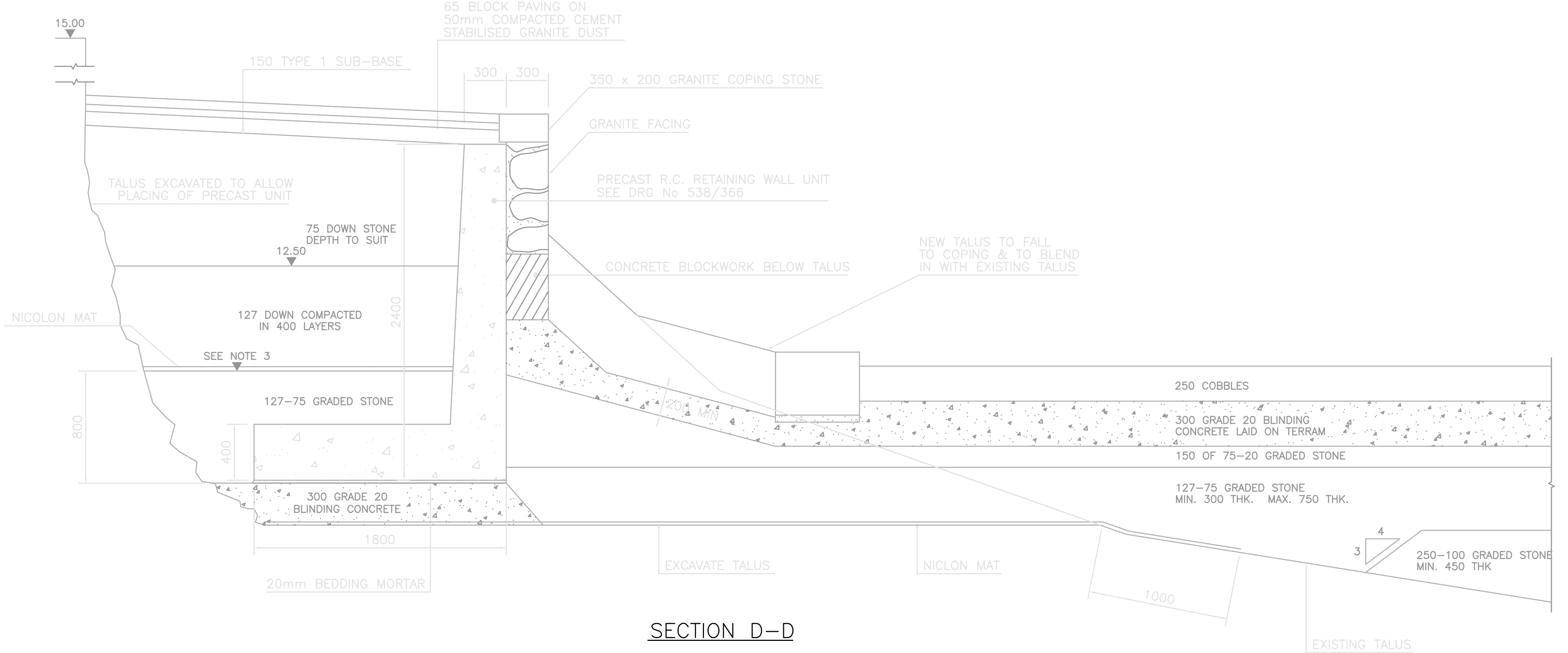
Scales 1:20

Drg. No. 538/543 (E)

Drg. No. 538/542 (E)

NOTES

1. FOR SECTION LOCATIONS SEE DRG. No. 538/541
2. FOR G.A. & REINFORCEMENT DETAILS OF PRECAST UNITS SEE DRG. No. 538/366
3. BACKFILLING BEHIND THE PRECAST RETAINING WALL SHALL BE LIMITED TO ONE THIRD OF THEIR HEIGHT UNTIL THE SLIPWAY BLINDING CONCRETE LAYER IS IN PLACE.
4. FOR PAVING DETAILS SEE DRG. No. 538/601



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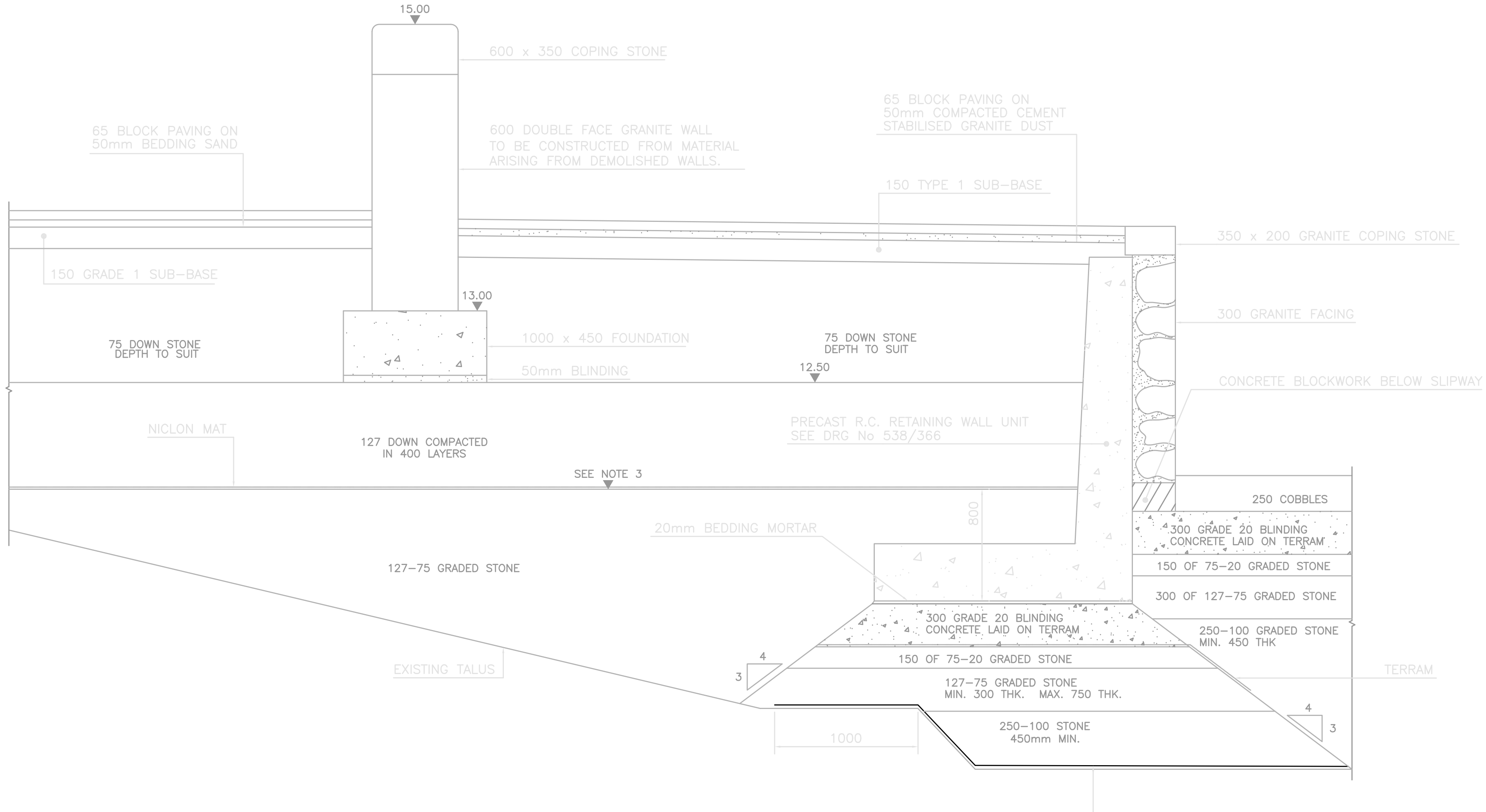

SECTION E–E

AS BUILT

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C	16-09-93	BEL	CHANGED FILL MATERIALS AS SHOWN
B	11-06-93	BEL	ISSUED FOR CONSTRUCTION
A	11-06-93	BEL	ISSUED FOR TENDER

STATES OF JERSEY

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PUBLIC SERVICES DEPARTMENT

Project
WEST OF ALBERT PHASE II LAND RECLAMATION

Title
SLIPWAY DUKW AREA CROSS-SECTION SHEET 1

Contract No. 538	Sheet No.
Designed by : BEL/CS	Drawn by : G.M.M
Checked by : BEL	Date : MARCH 93

Scales 1:20

Drg. No. 538/542 (E)

Drg. No. 538/532 (E)

NOTES
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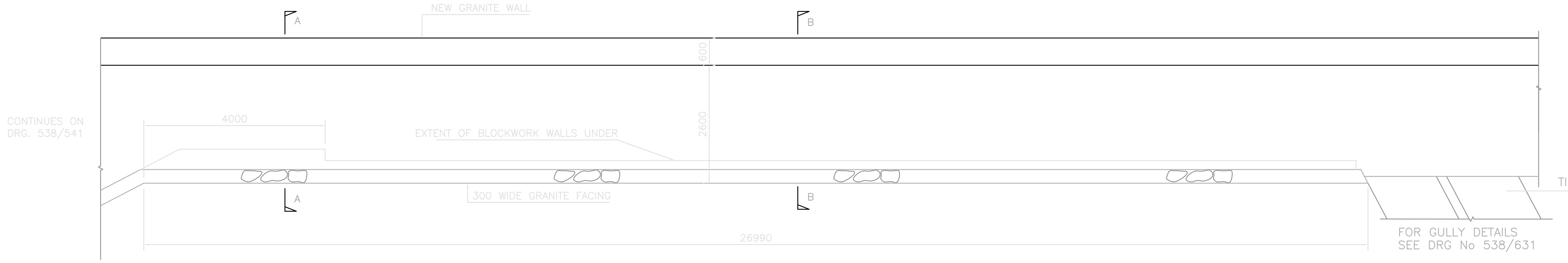
2. FOR HAND RAIL DETAILS SEE DRG. N° 538/621



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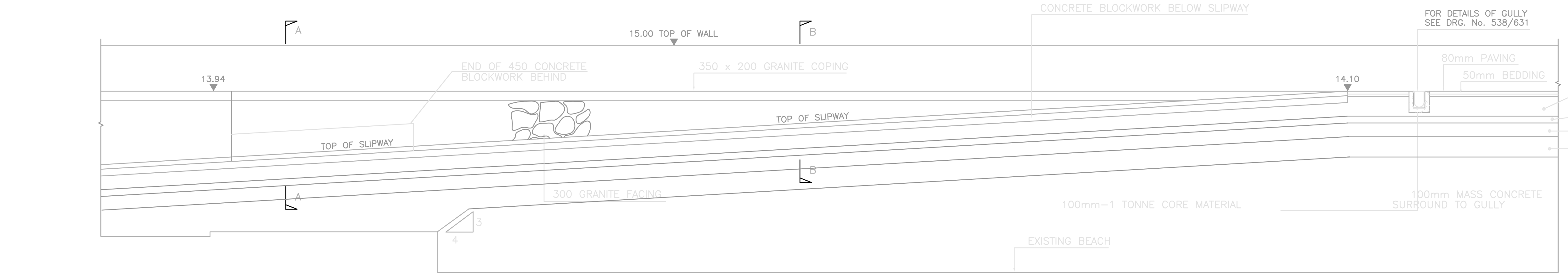

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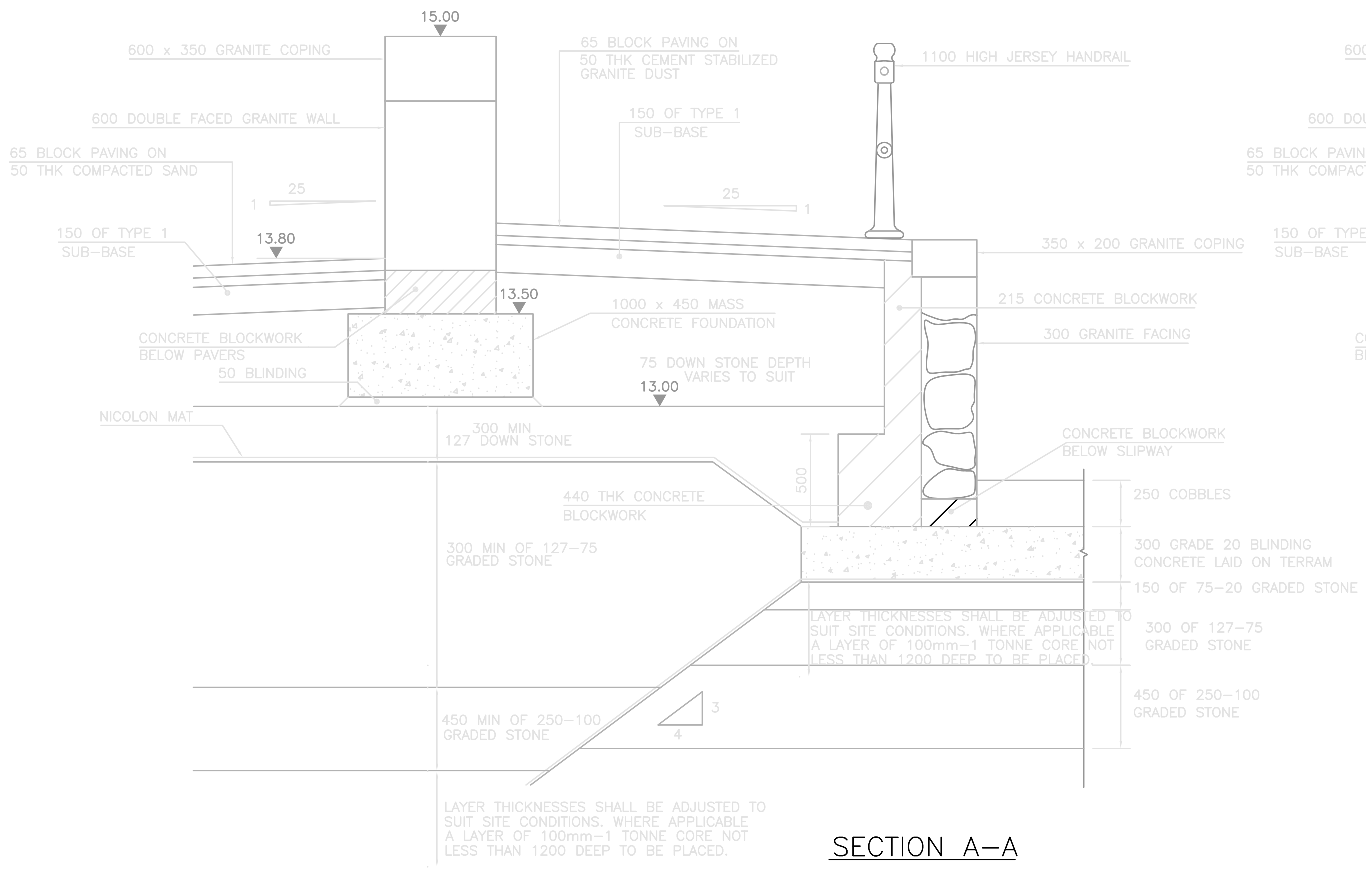

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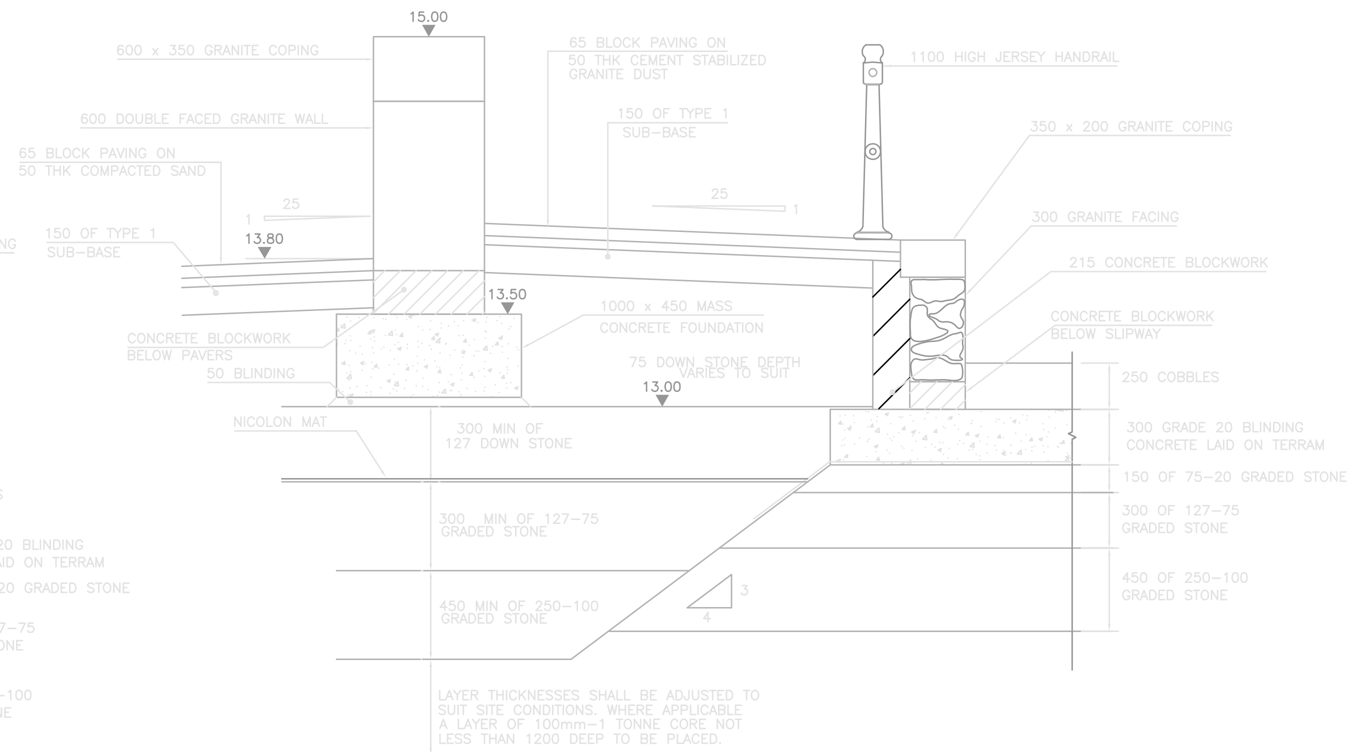

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AS BUILT

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D	03-01-94	BEL	ADDED NOTES TO A-A & B-B
C	16-09-93	BEL	CHANGED UPPER MATERIALS AS SHOWN
B	11-06-93	BEL	ISSUED FOR CONSTRUCTION
A	02-04-93	BEL	ISSUED FOR TENDER

STATES OF JERSEY
SOUTH HILL, St.HELIER JERSEY JE4 8UY
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PUBLIC SERVICES DEPARTMENT

Project
WEST OF ALBERT PHASE II LAND RECLAMATION

Title
SLIPWAY TYPICAL SECTION THROUGH ACCESSS WALKWAY

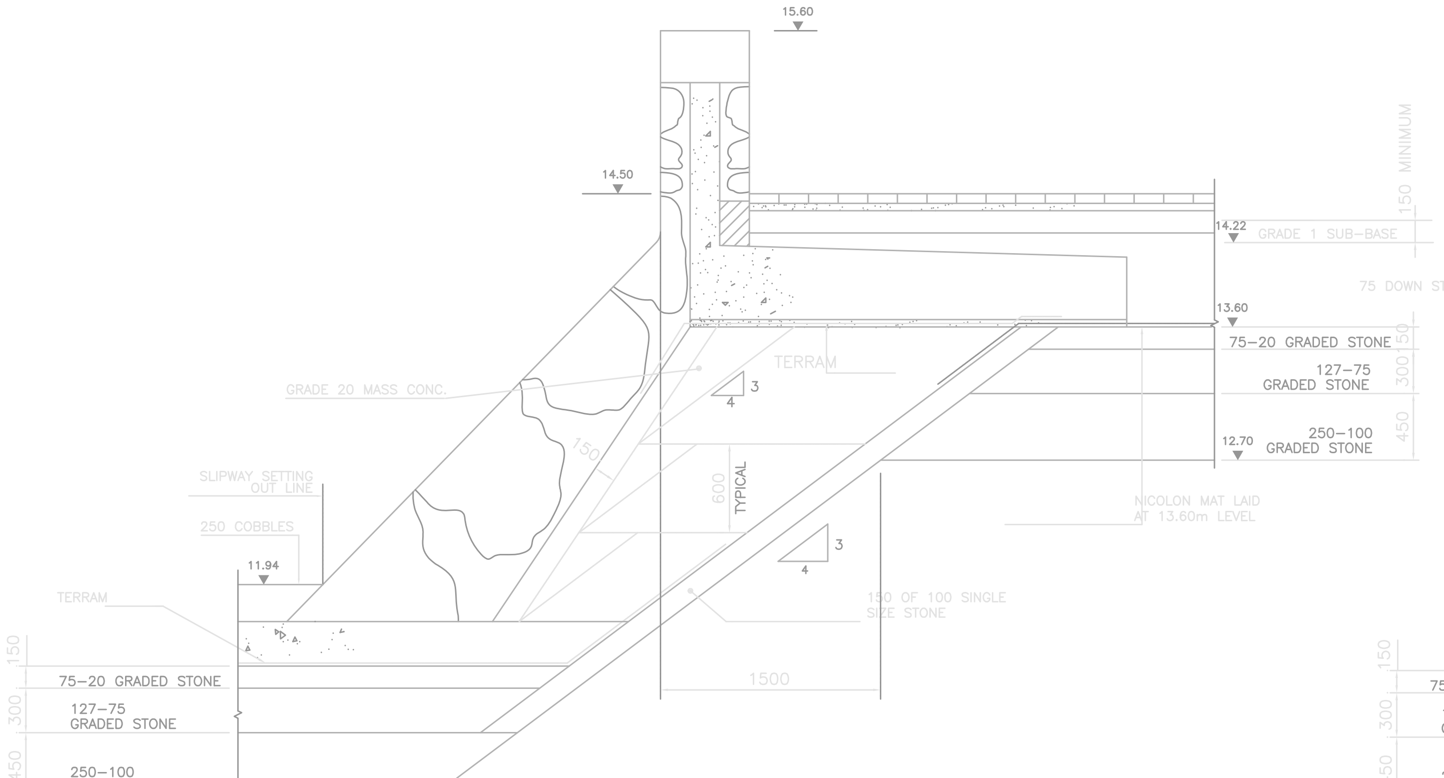
Contract No. 538	Sheet No.
Designed by : BEL/CS	Drawn by : G.M.M
Checked by : BEL	Date : MAR. 1993

Scales 1:50

Drg. No. 538/532 (E)

Drg. No. 538/522 (C)

NOTES



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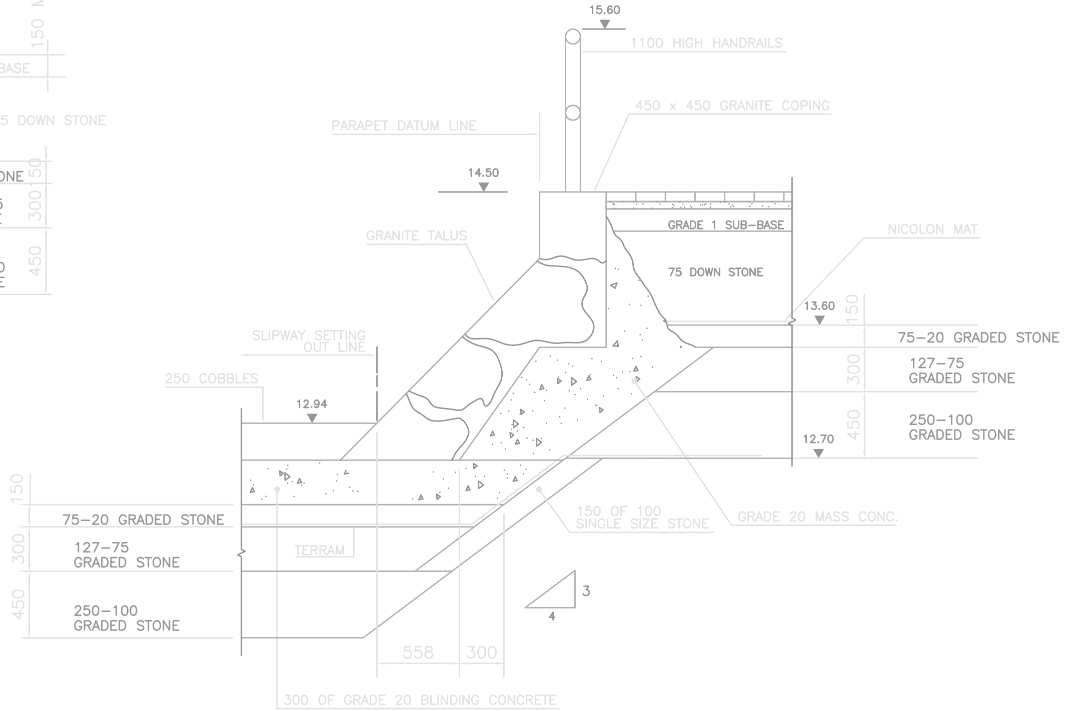

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(SLIPWAY Ch.70)

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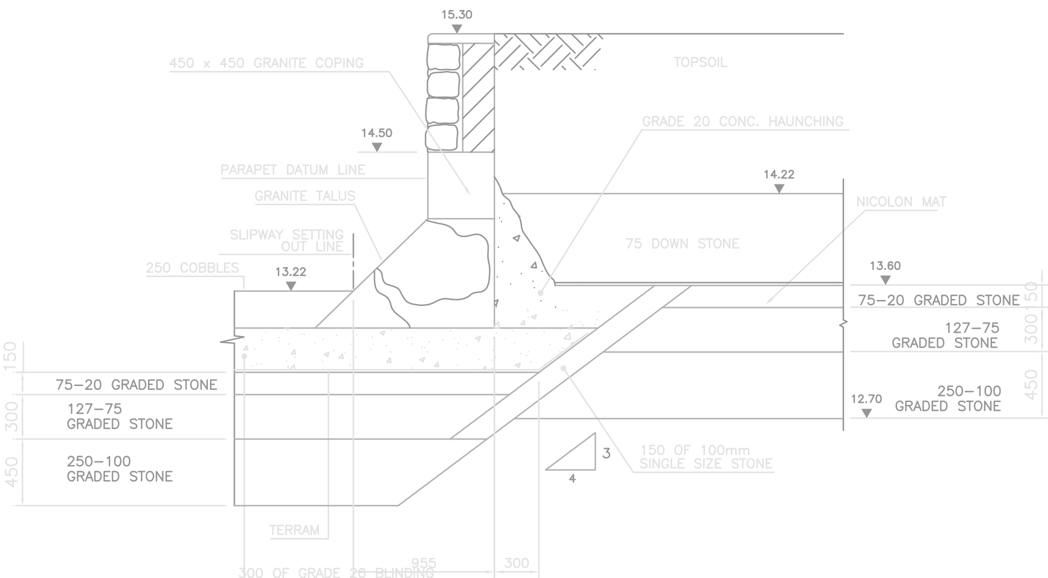

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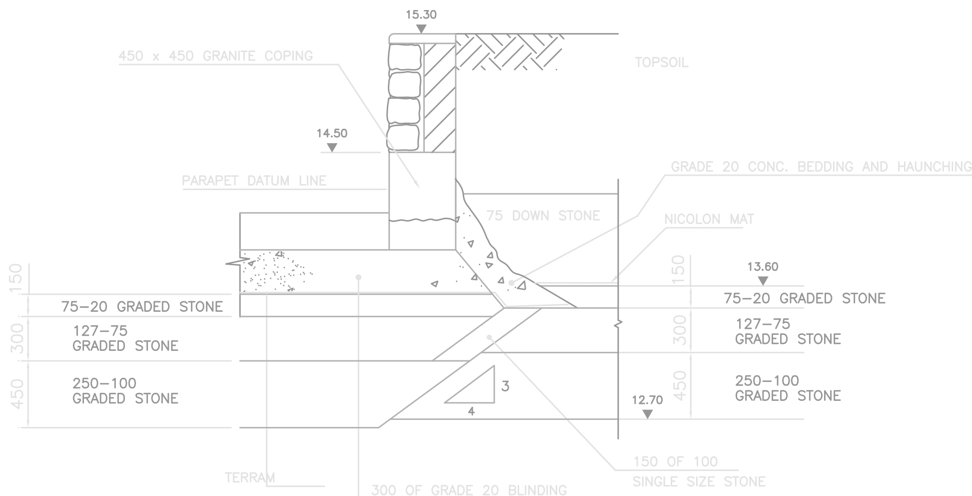

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(SLIPWAY Ch. 87.5)

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B	11–06–93	BEL	ISSUED FOR CONSTRUCTION
A	02–04–93	BEL	ISSUED FOR TENDER

STATES OF JERSEY

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PUBLIC SERVICES DEPARTMENT

Project
WEST OF ALBERT
PHASE II
LAND RECLAMATION

Title
SLIPWAY
LANDWARD TOE DETAILS OF GRANITE TALUS

Contract No. 538	Sheet No.
Designed by : BEL	Drawn by : G.M.M
Checked by : BEL	Date : MAR. 1993

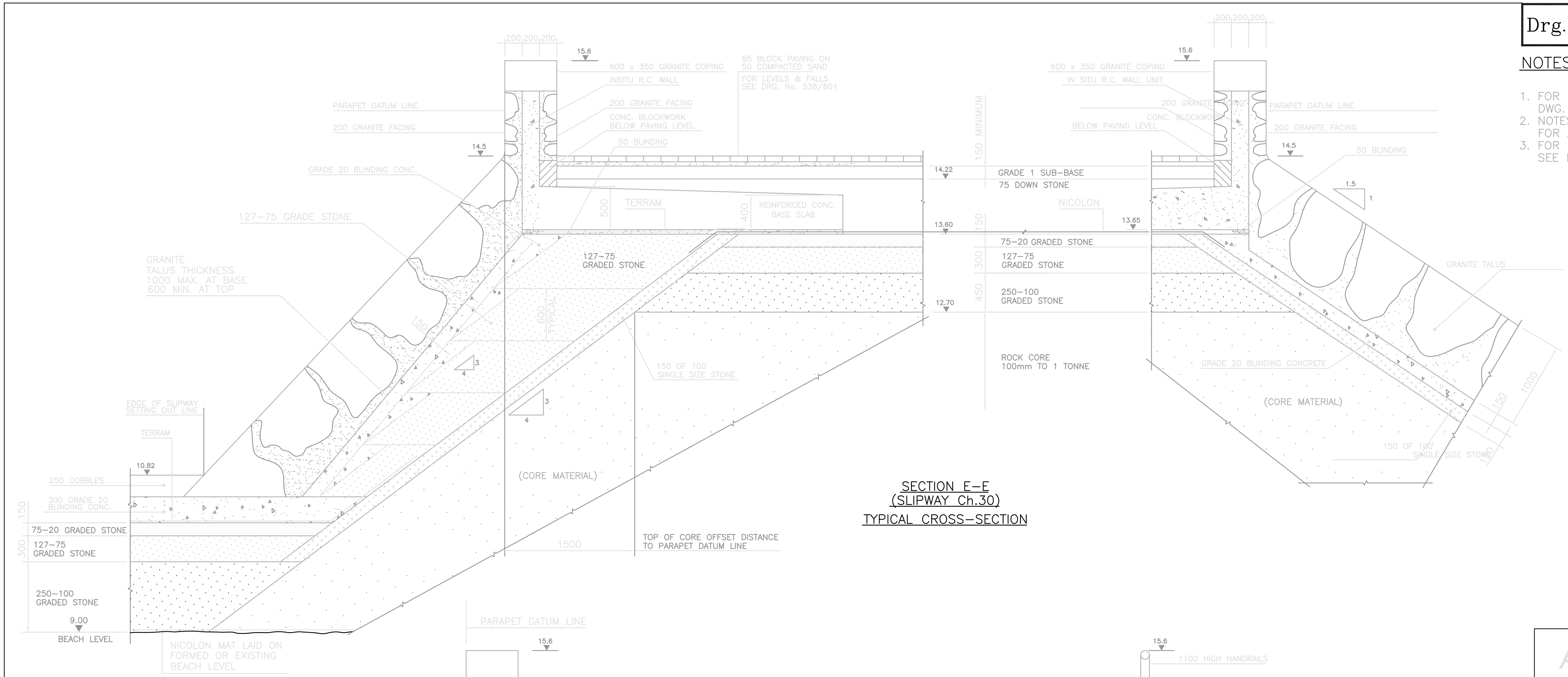
Scales 1:25

Drg. No. 538/522 (C)

Drg. No. 538/511 (C)

NOTES

1. FOR LOCATION OF SECTIONS SEE DWG. No. 538/524
2. NOTES TO SECTION E–E TYPICAL FOR ALL SECTIONS.
3. FOR DETAILS OF R.C. SLAB & WALL SEE DRG. No's 538/551 & 538/552



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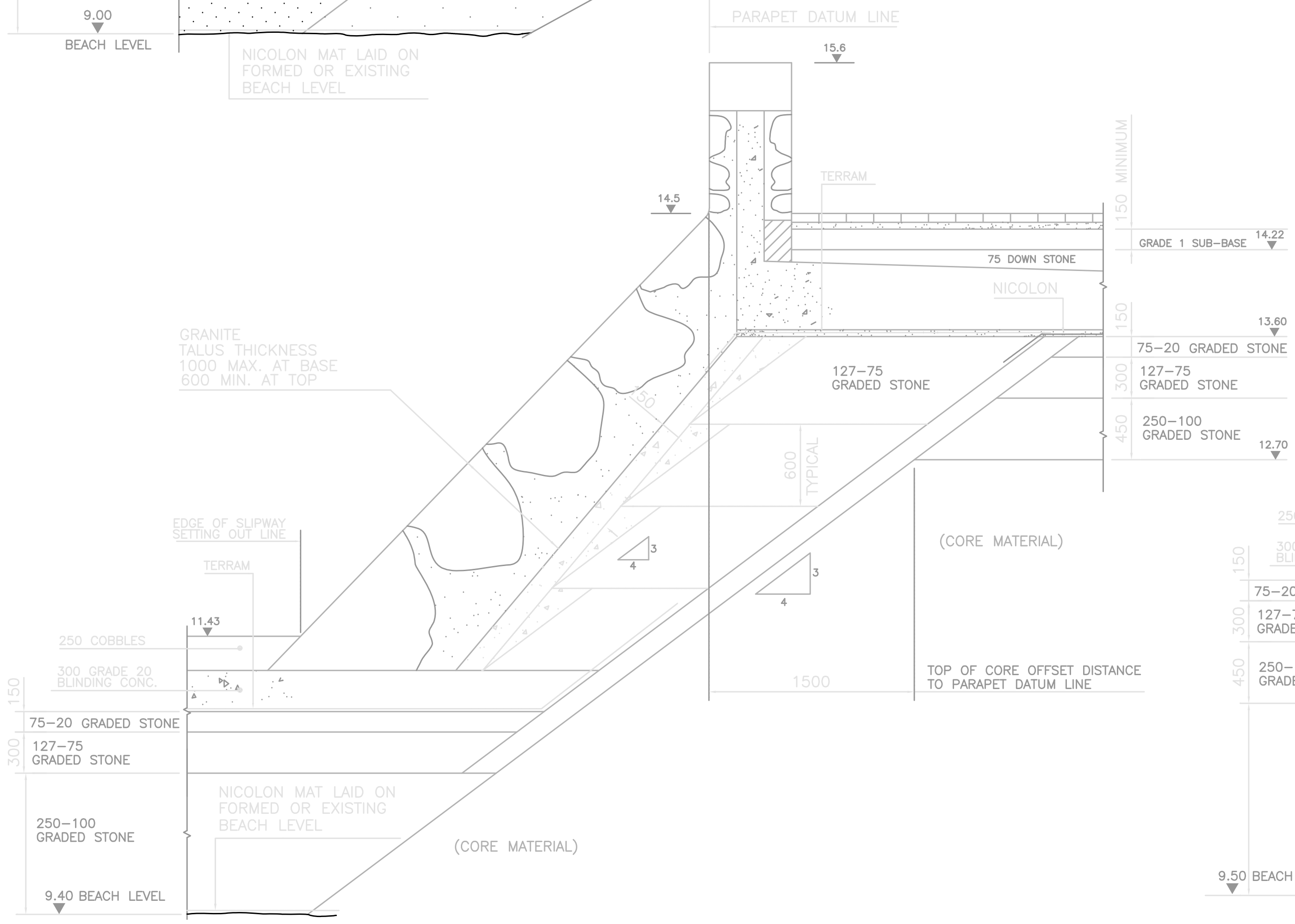
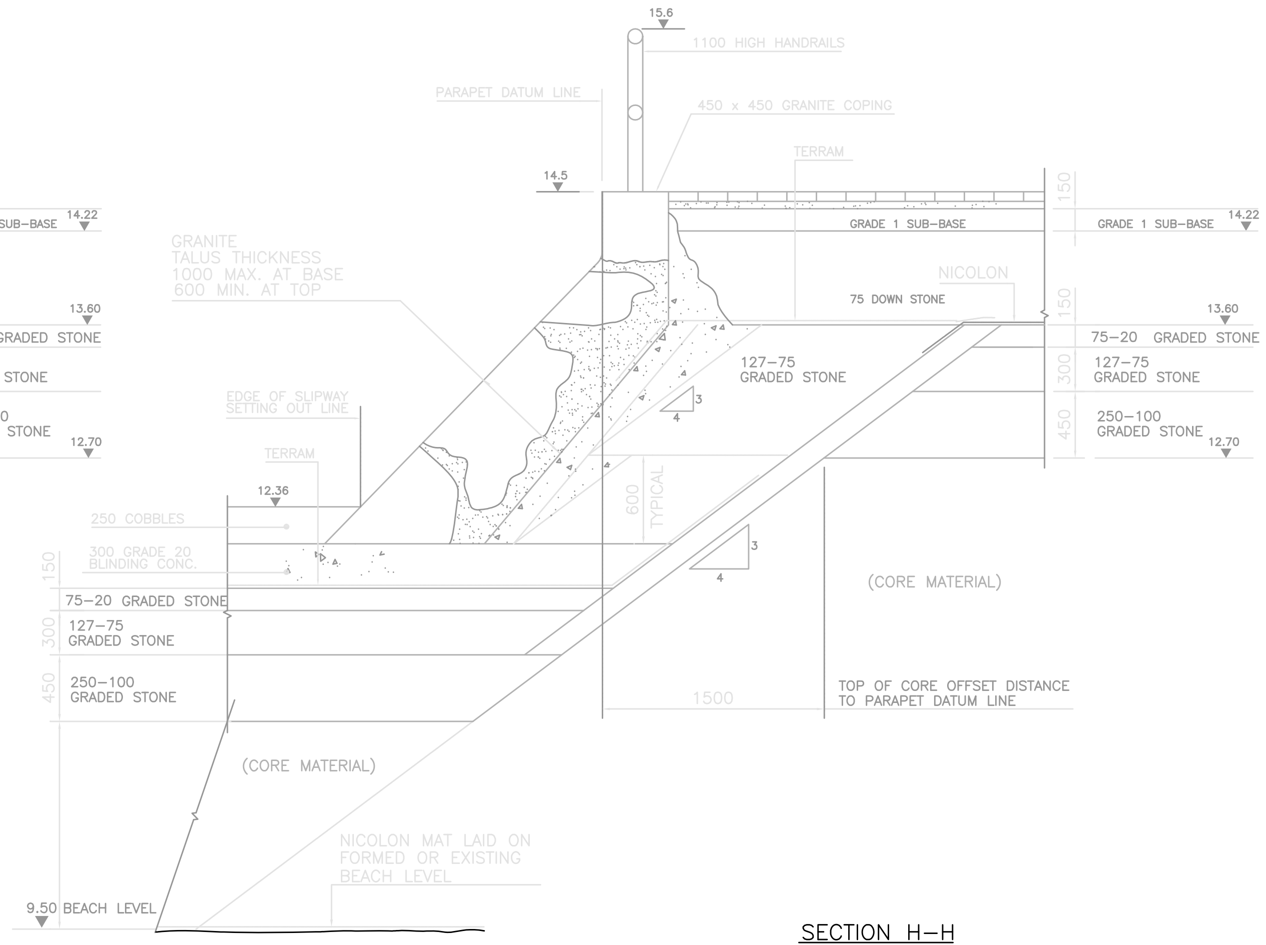
(SLIPWAY Ch.30)
TYPICAL CROSS–SECTION



SECTION F–F
(SLIPWAY Ch.40)



SECTION H–H
(SLIPWAY Ch.60)

AS BUILT

Rev	Date	Ints.	Details
C	02–06–98	SCS	AS BUILT
B	11–06–93	BEL	ISSUED FOR CONSTRUCTION
A	02–04–93	BEL	ISSUED FOR TENDER

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Project
WEST OF ALBERT
PHASE II
LAND RECLAMATION

Title
SLIPWAY
TALUS
TYPICAL CROSS–SECTION & DETAILS

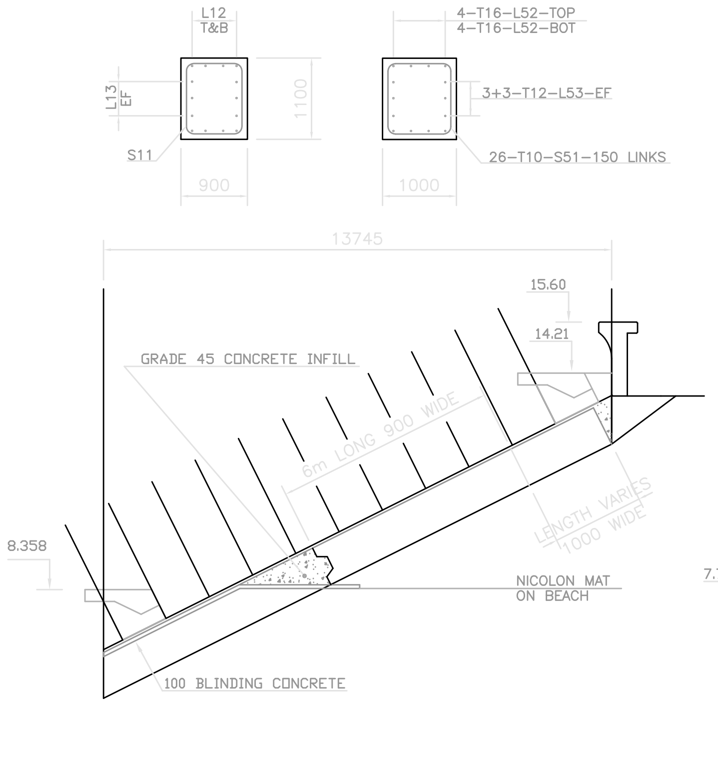
Contract No. 538	Sheet No.
Designed by : BEL/CS	Drawn by : G.M.M
Checked by : BEL	Date : MARCH 93

Scales 1:25

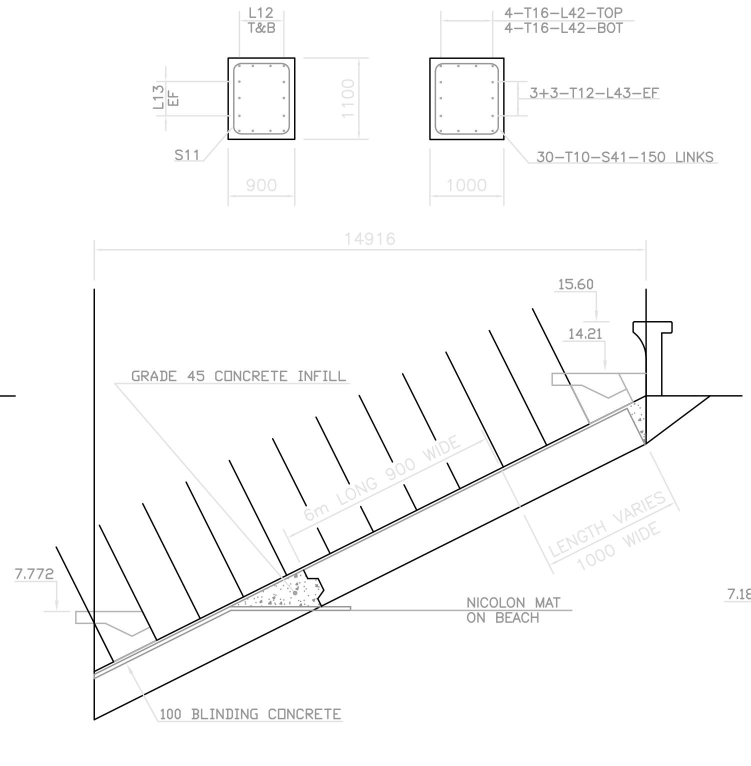
Drg. No. 538/511 (C)

Drg. No. 538/254 (B)

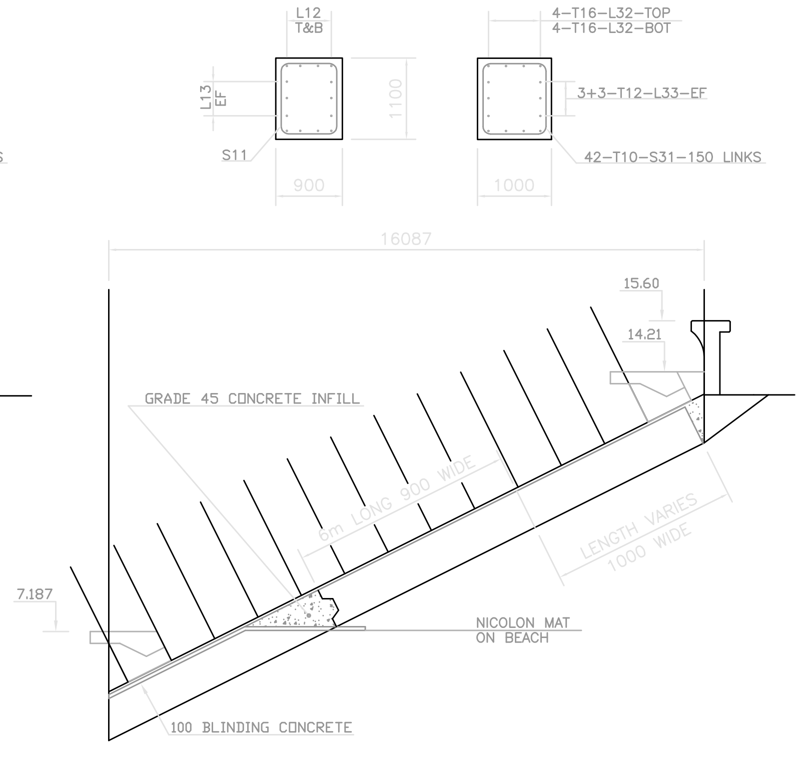
NOTES



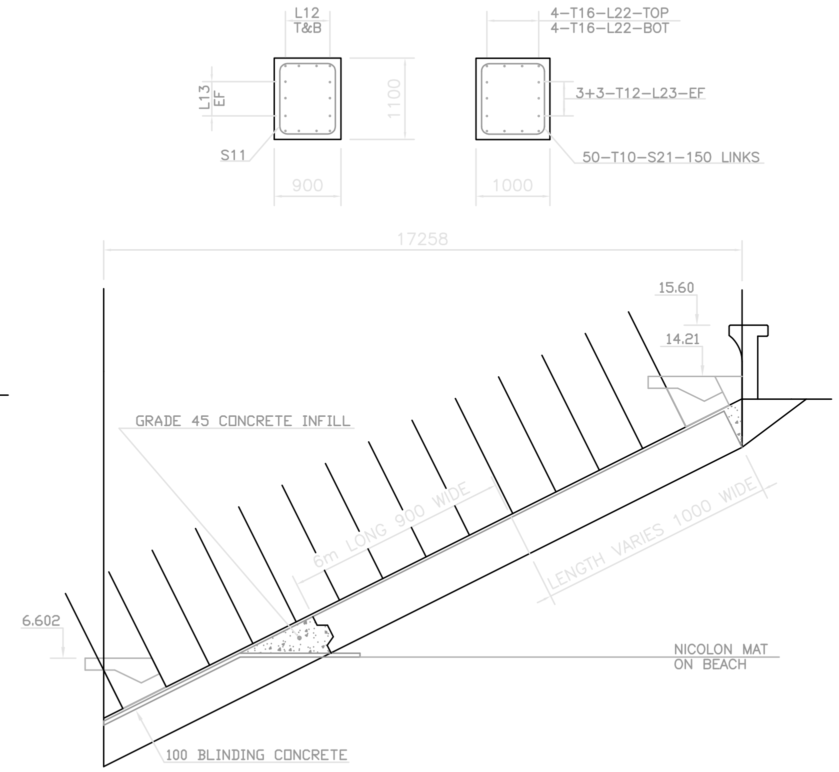
11 BLOCK SECTION
(20 GROUND BEAMS THUS)



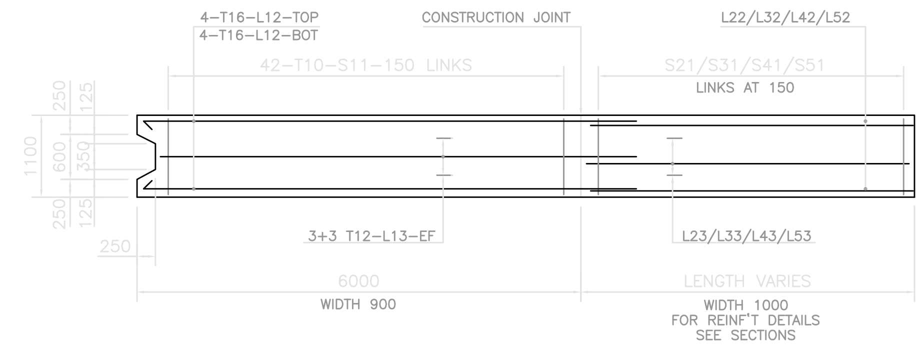
12 BLOCK SECTION
(14 GROUND BEAMS THUS)



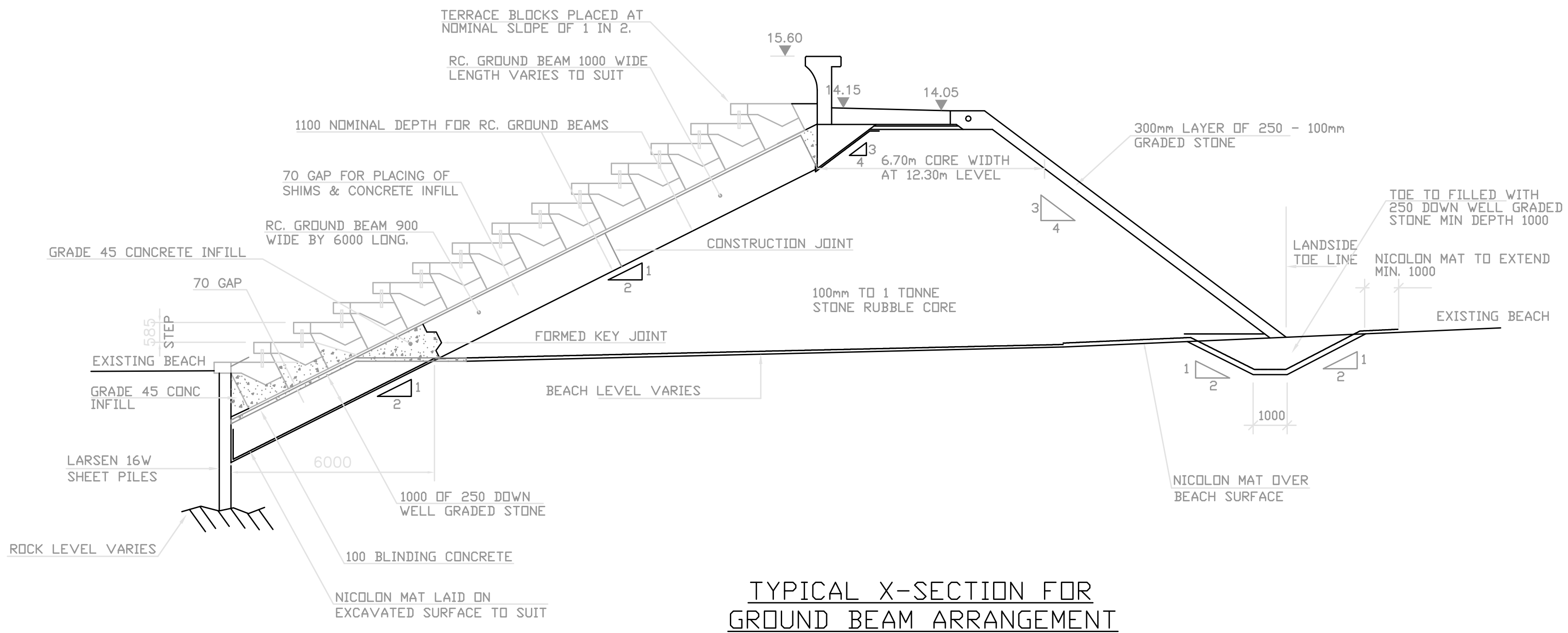
13 BLOCK SECTION
(9 GROUND BEAMS THUS)



14 BLOCK SECTION
(9 GROUND BEAMS THUS)



GROUND BEAM REINFORCEMENT DETAILS
(1:50)



TYPICAL X-SECTION FOR
GROUND BEAM ARRANGEMENT

NOTES :

1. FOR ASSEMBLY TOLERANCES REFER TO THE SPECIFICATION.

AS BUILT

Rev	Date	Ints	Details
B	22-05-98	SCS	AS BUILT
A	14-01-94	BEL	ISSUED FOR CONSTRUCTION

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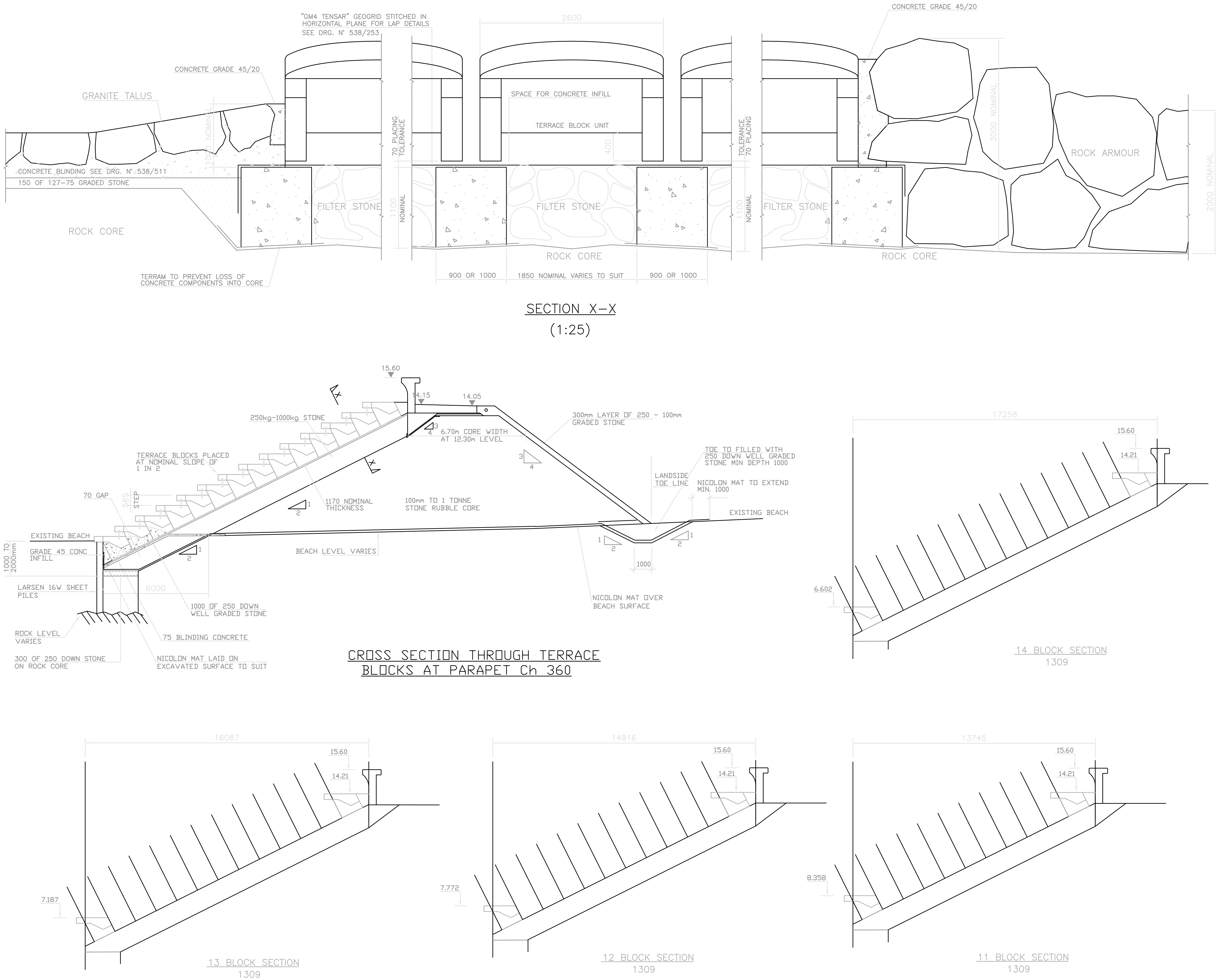
Project
WEST OF ALBERT
PHASE II
LAND RECLAMATION

Title
TERRACE BLOCKS
TYPICAL GROUND BEAMS
G.A. AND REINFORCEMENT

Contract No. 538	Sheet No.
Designed by : BEL/CS	Drawn by : CS/GM
Checked by : BEL	Date : MARCH 93

Scales 1:100

Drg. No. 538/254 (B)



Drg. No. 538/252 (D)

NOTES

1. FOR ASSEMBLY TOLERANCES REFER TO THE SPECIFICATION OR AS DIRECTED BY THE ENGINEER.

AS BUILT

Rev	Date	Ints.	Details
D	22-05-98	SCS	AS BUILT
C	24-01-94	BEL	AMENDED TOE DETAIL AND ADDED SECTION DETAIL
B	11-06-93	BEL	ISSUED FOR CONSTRUCTION
A	02-04-93	BEL	ISSUED FOR TENDER

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Project
WEST OF ALBERT
PHASE II
LAND RECLAMATION

Title
TERRACE BLOCKS
TYPICAL X-SECTION
LAYOUT

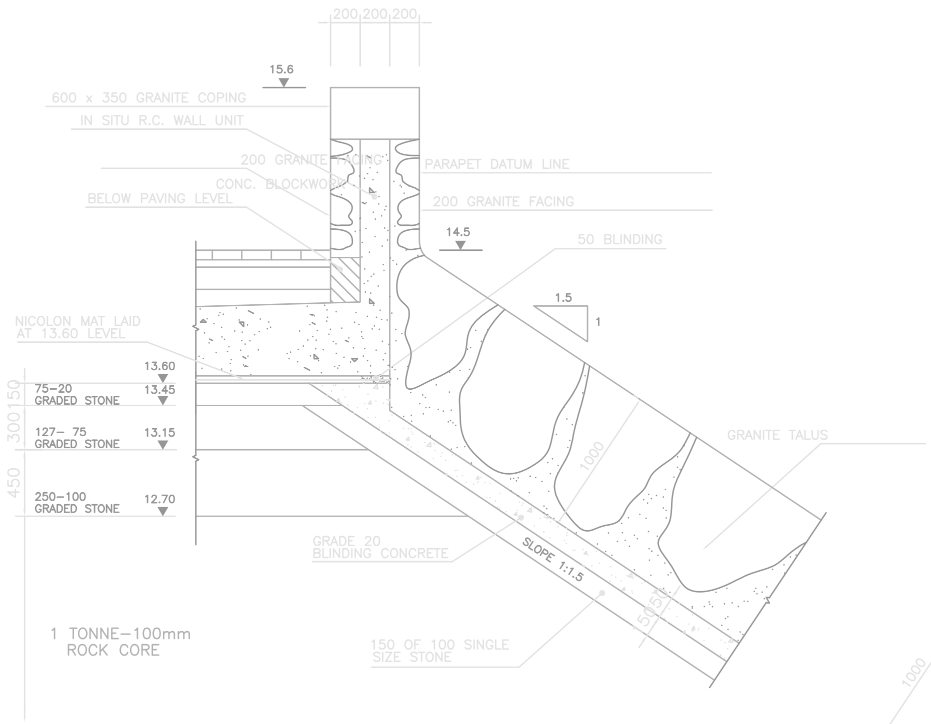
Contract No. 538	Sheet No.
Designed by : BEL/CS	Drawn by : CS/GM
Checked by : BEL	Date : MARCH 93

Scales 1:100

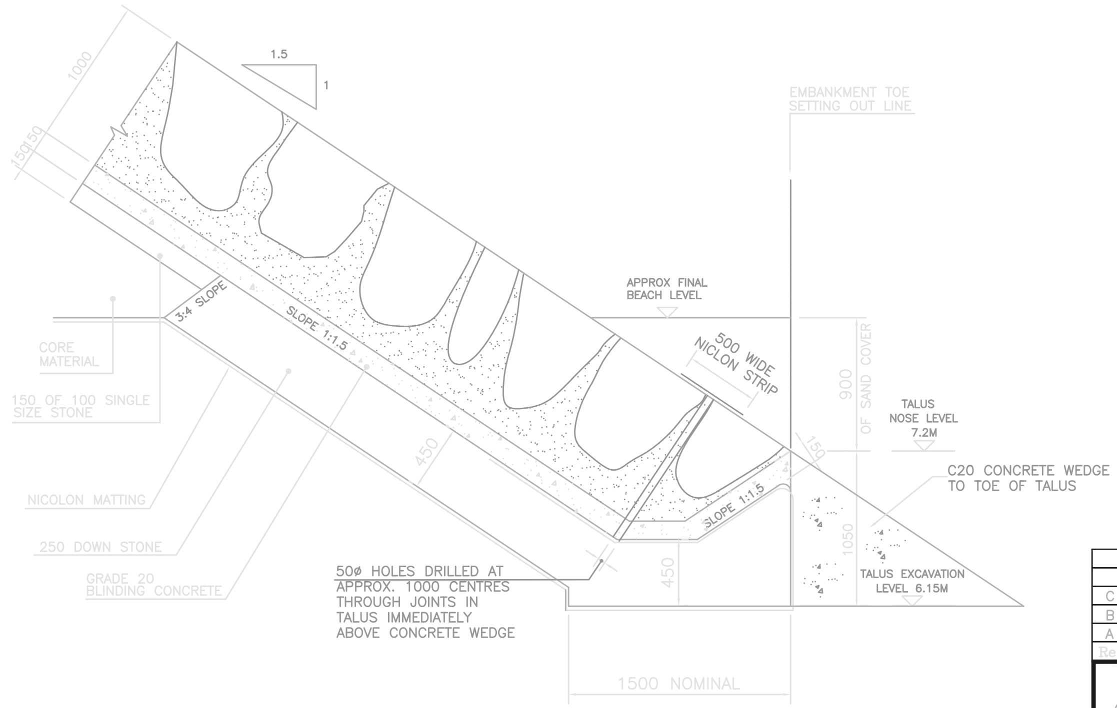
Drg. No. 538/252 (D)

Drg. No. 538/203 (C)

NOTES



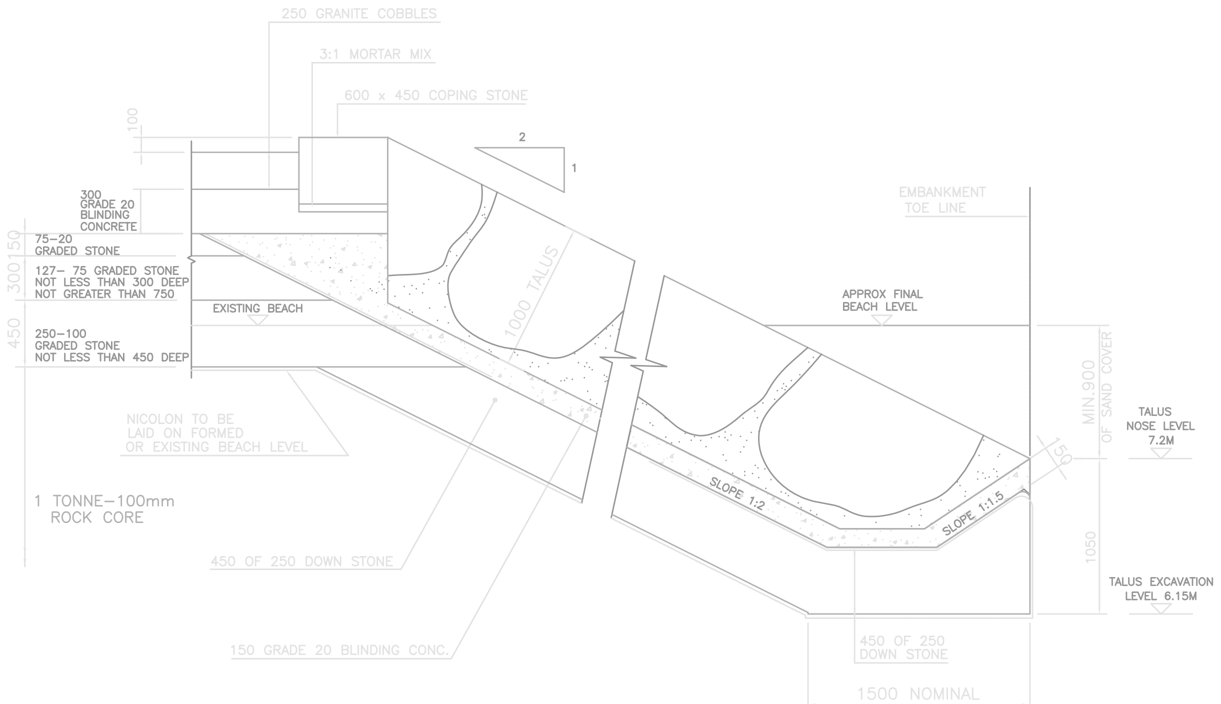
TYPICAL SECTION THROUGH TALUS



AS BUILT

Rev	Date	Ints.	Details
C	22–05–98	SCS	AS BUILT
B	11–06–93	BEL	ISSUED FOR CONSTRUCTION
A	02–04–93	BEL	ISSUED FOR TENDER

NOTE
1. LAYER THICKNESSES SHALL BE ADJUSTED TO SUIT EXISTING BEACH LEVELS. MINIMUM LAYER CONSTRUCTION BENEATH BLINDING CONCRETE SHALL BE 600mm.
2. DEPTH TO EXISTING BEACH VARIES



SLIPWAY SECTION D–D
REFER TO DRG. No. 538/524

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Project
WEST OF ALBERT
PHASE II
LAND RECLAMATION

Title
EMBANKMENT
TYPICAL CROSS–SECTION
OF GRANITE TALUS

Contract No. 538	Sheet No.
Designed by : BEL/CS	Drawn by : G.M.M
Checked by : BEL	Date : MARCH 93

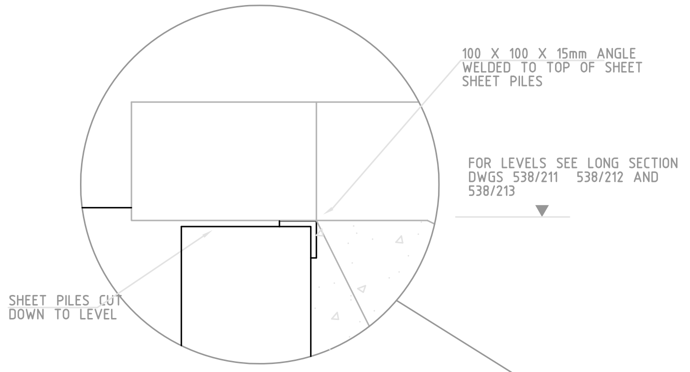
Scales 1:25

Drg. No. 538/203 (C)

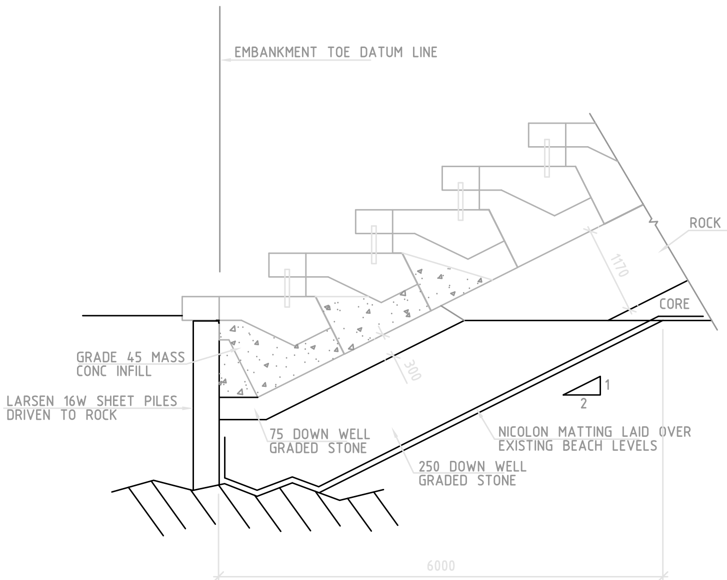
Drg. No. 538/202 (C)

NOTES

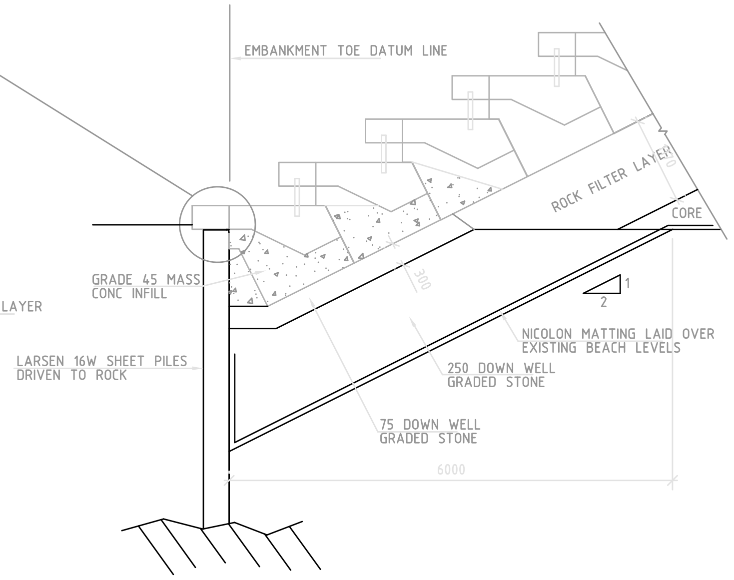
1. FOR DETAILS OF TERRACE BLOCKS SEE DRAWINGS 538/252 AND 538/261
2. FOR DETAILS OF BLOCK TO BLOCK ANCHOR DETAILS SEE DRAWING 538/263
3. FOR DETAILS OF TERRACE BLOCK LEVELS SEE DRWG
4. ROCK FILTER LAYER MATERIAL 250kg TO 1000kg TO BE PLACED WITH CARE TO FORM A SOUND FOUNDATION FOR TERRACE BLOCKS



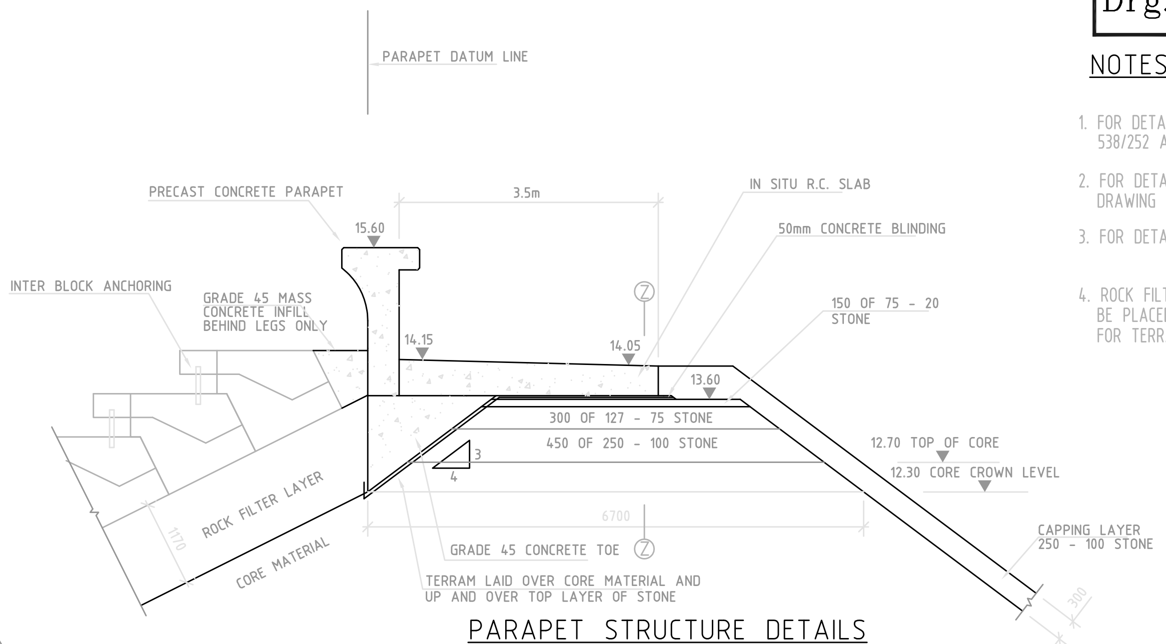
TERRACE BLOCK NOSE SUPPORT DETAIL



TOE DETAIL WHERE ROCK HEAD IS HIGHER THAN TYPICAL TOE DETAIL



TYPICAL TOE DETAIL

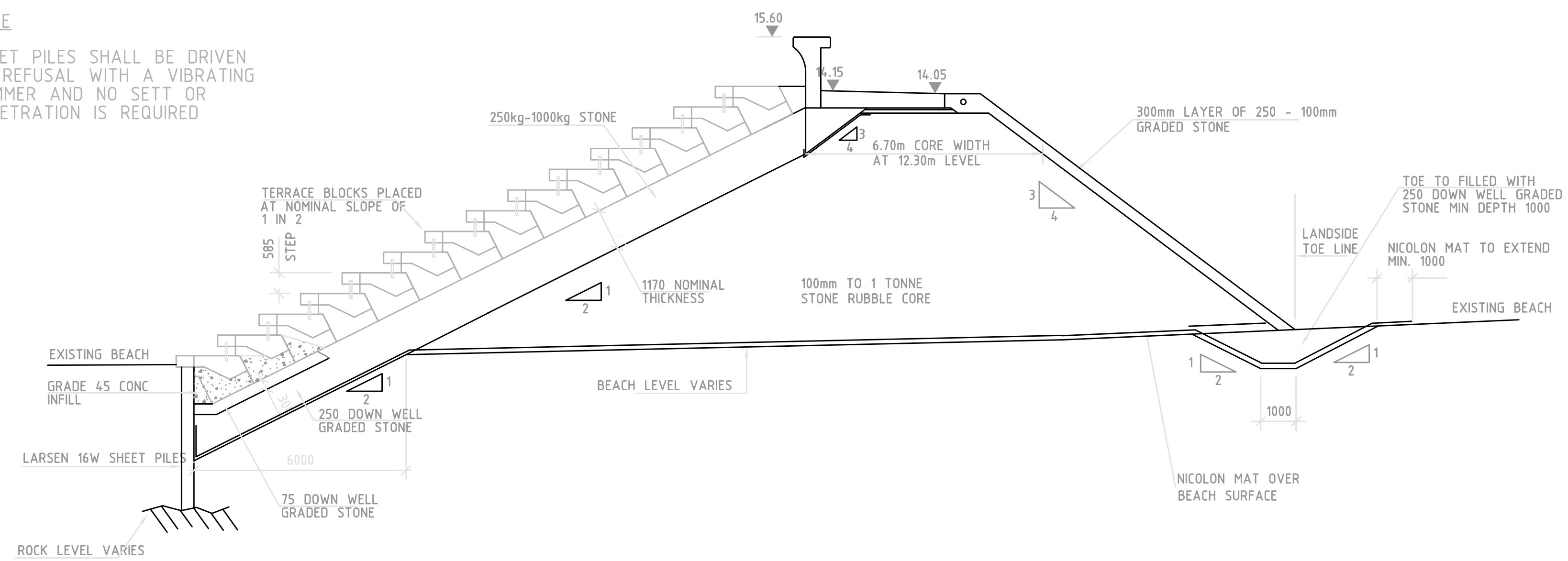


PARAPET STRUCTURE DETAILS ABOVE CROWN AT 12.30m LEVEL

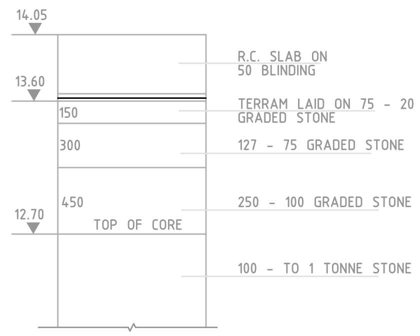
AS BUILT

NOTE

SHEET PILES SHALL BE DRIVEN TO REFUSAL WITH A VIBRATING HAMMER AND NO SETT OR PENETRATION IS REQUIRED



CROSS SECTION THROUGH TERRACE BLOCKS AT PARAPET Ch 360



SECTION Z - Z (1 : 25) GRADED STONE LAYER STRUCTURE

Rev	Date	Ints	Details
C	22-05-98	SCS	AS BUILT
B	11-06-93	BEL	ISSUED FOR CONSTRUCTION
A	02-04-93	BEL	ISSUED FOR TENDER

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PUBLIC SERVICES DEPARTMENT

Project
WEST OF ALBERT PHASE II LAND RECLAMATION

Title
EMBANKMENT TYPICAL CROSS SECTIONS WITH TERRACE BLOCKS

Contract No. 538	Sheet No.
Designed by : BEL	Drawn by : CS
Checked by : BEL	Date : MAR.1993

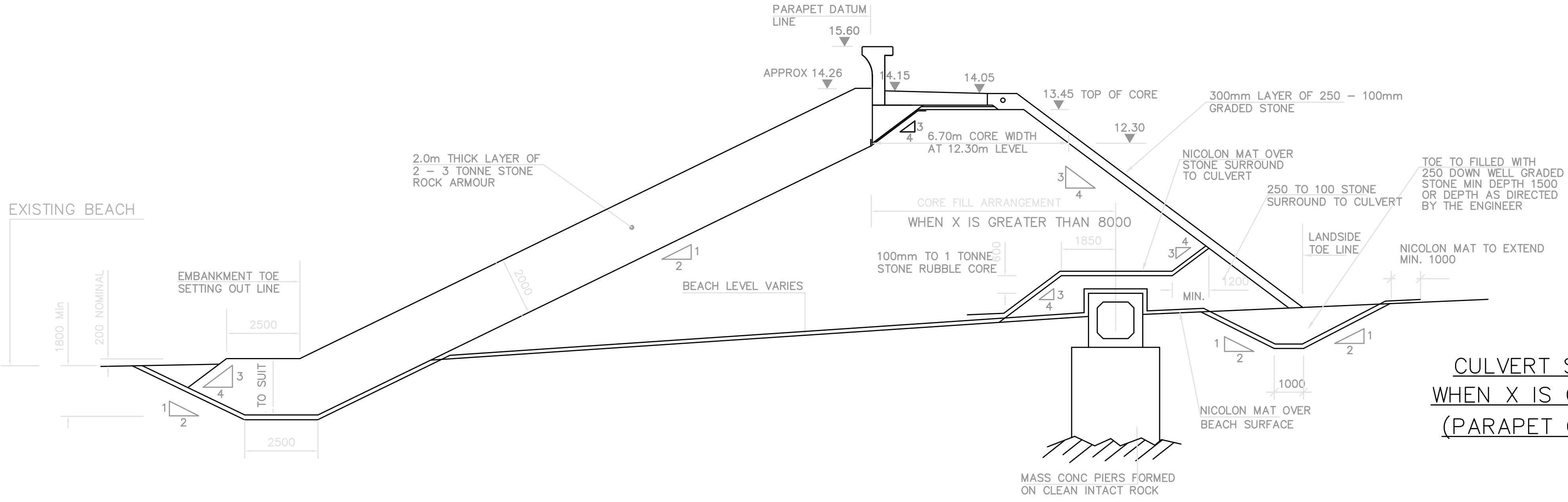
Scales 1:100 1:50

Drg. No. 538/202 (C)

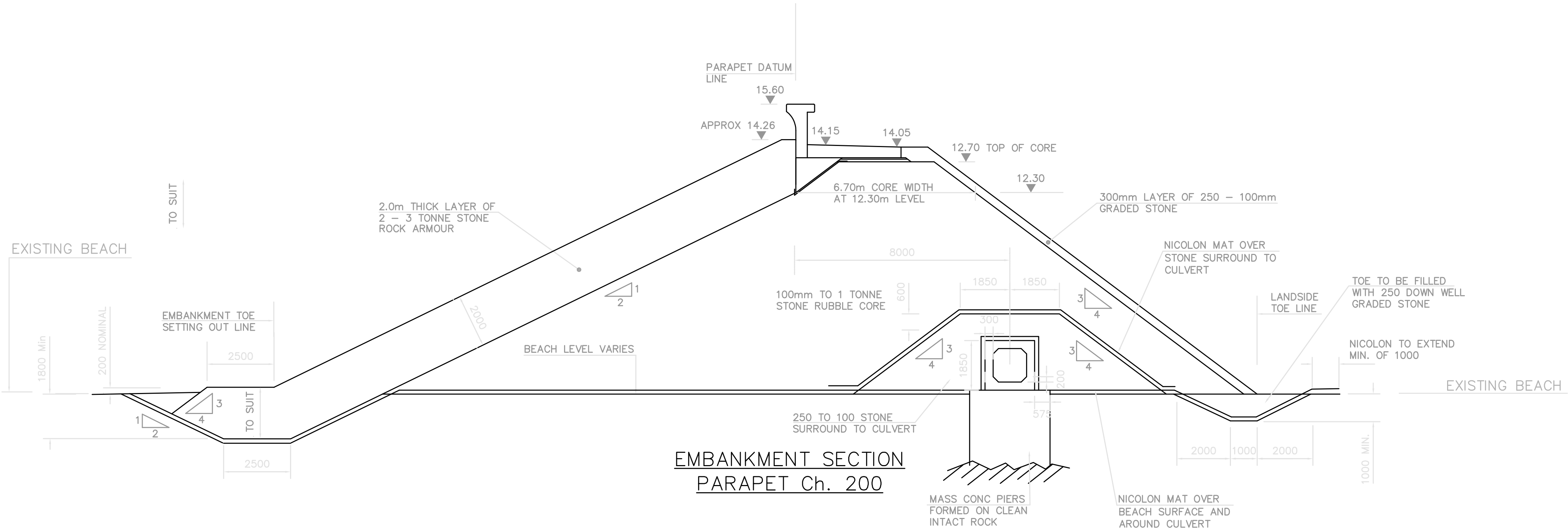
Drg. No. 538/201 (C)

NOTES

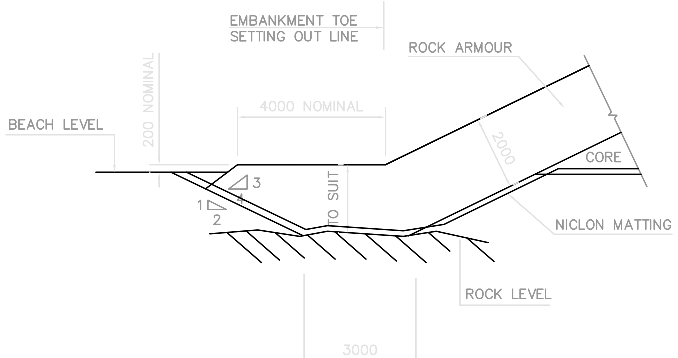
1. STONE SURROUND TO CULVERT STRUCTURE SHALL BE AS SHOWN EXCEPT FOR :–
   (a) SMALL REDUCTIONS IN COVER AT THE OUTFALL SEE DWG 538/421
   (b) CHANGES IN SECTION AS CULVERT EMERGES FROM CORE SURROUND SEE DWG 538/202
2. FOR GRADES OF NICOLON AND TERRAM SEE SPECIFICATION



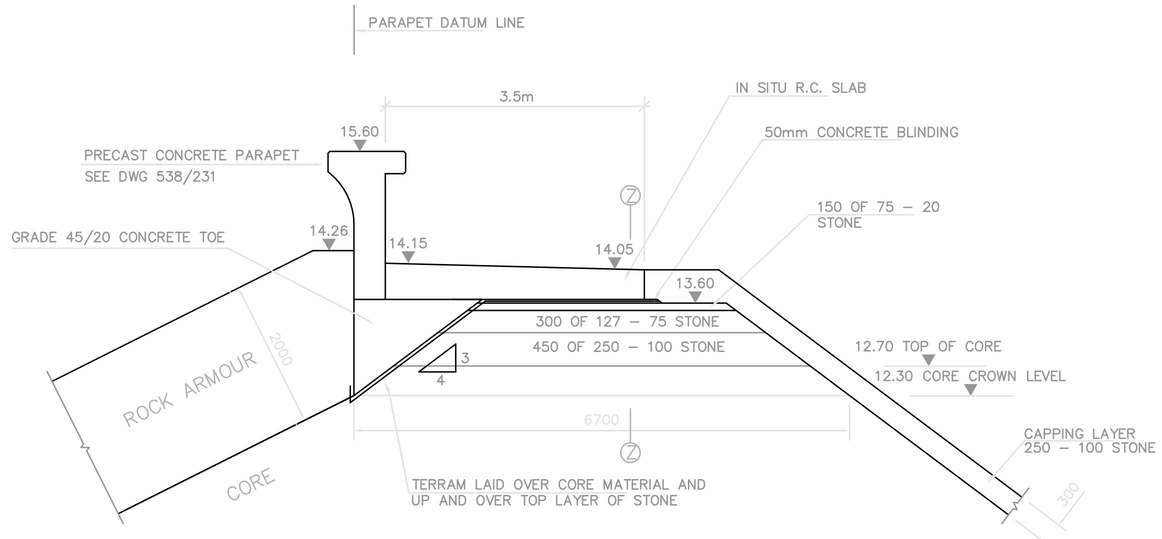
CULVERT SURROUND DETAIL
WHEN X IS GREATER THAN 8000
(PARAPET Ch 315 ONWARDS)



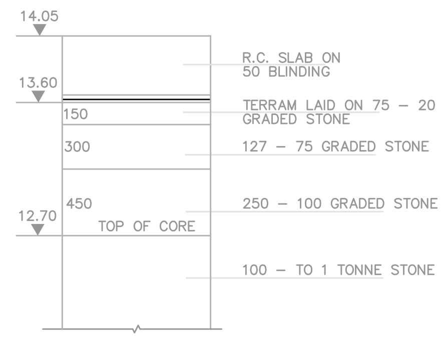
EMBANKMENT SECTION
PARAPET Ch. 200



ALTERNATIVE TOE DETAIL
(WHEN ROCK ENCOUNTERED AT LESS THAN 1.8m BELOW EXTG BEACH LEVEL)



PARAPET STRUCTURE DETAILS
ABOVE CROWN AT 12.30m LEVEL



SECTION Z – Z (1 : 25)
GRADED STONE LAYER STRUCTURE

AS BUILT

Rev	Date	Ints.	Details
C	22–05–98	SCS	AS BUILT
B	11–06–93	BEL	ISSUED FOR CONSTRUCTION
A	02–04–93	BEL	ISSUED FOR TENDER

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Project
WEST OF ALBERT
PHASE II
LAND RECLAMATION

Title
EMBANKMENT
TYPICAL CROSS SECTIONS
WITH ROCK ARMOUR

Contract No. 538	Sheet No.
Designed by : BEL/CS	Drawn by : CS
Checked by : BEL	Date : MAR.1993

Scales 1:100

Drg. No. 538/201 (C)

Appendix B

Overtopping Analysis

B1 Overtopping Analysis

D1 Overtopping Parameters

D1.1 Victoria Avenue

The figure shows the standard vertical wall type from EurOtop has been assumed to be representative of this section of wall.

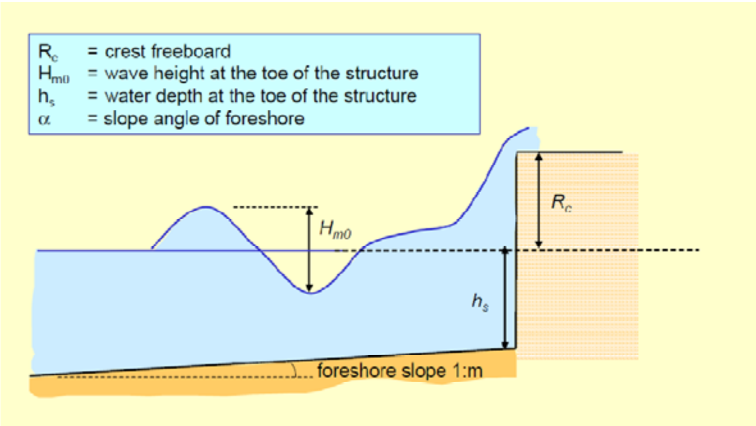

Figure D1 - Representation of Wall Type 1

It has been assumed that the toe, if present, will have a relatively small effect on the wave overtopping regime along this section of wall. We have witnessed physical model testing of a similar shaped wall which demonstrated that this is a robust assumption.

Considering the wave direction shown in Figure 1 in the main report, the waves have been assumed to hit this stretch of wall head on so no obliquity has been considered. Based on an assessment of the levels along Victoria Avenue a wall length of 100m of Wall Type 1 has been considered to drain to the east towards the site.

D1.2 Slipway

The slipway has been modelled as a standard simple slope structure from EurOtop to determine the wave overtopping volume as show in the figure below.

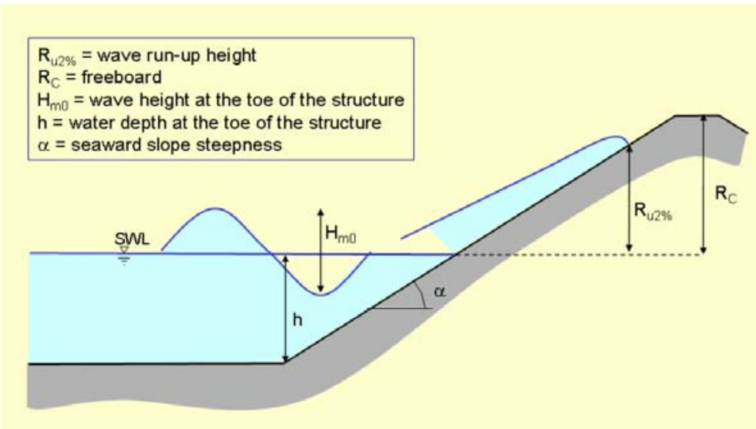

Figure D2 - Representation of Wall Type 2

For this section of wall an incident wave obliquity of 45° has been considered. Given the relatively shallow slope of the slipway and narrow extent, it does not contribute significantly to the total overtopping discharge along the complete length of defences being considered.

D1.3 Terrace Revetment

This is an uncommon form of wall construction and as such is not represented directly within the EurOtop manual. However a reasonable estimation of the expected levels of overtopping discharge can be made by applying the outline geometry of this wall section to a standardised typical wall section from EurOtop.

The standard section show below has been used to represent wall Type 3.

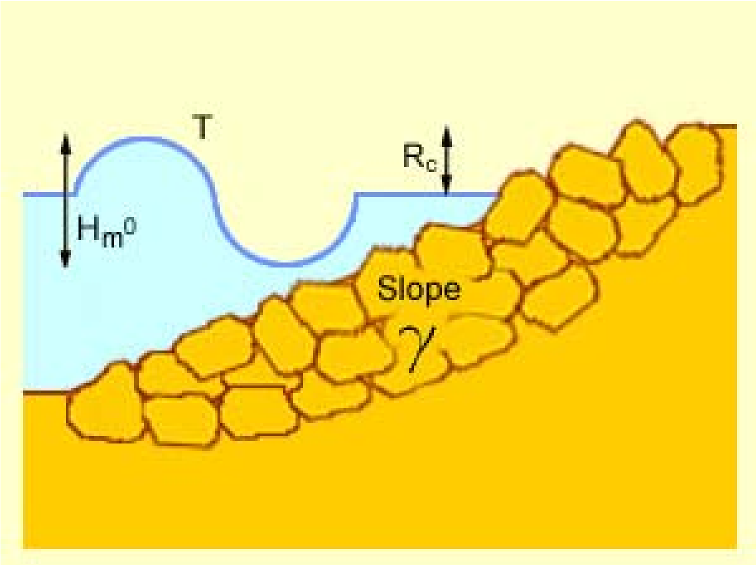

Figure D3 – Representation of Wall Type 3

The use of this standard wall type gave us the flexibility to manipulate the slope material properties to represent the terraced steps that would be expected to cause significant wave breaking, reduce wave run-up and therefore limit wave overtopping.

The slope material reduction factor has been set to a relatively low level of 0.45 which provides a representation of the degree to which the wave energy will be disrupted by the terraced blocks.

This wall section has been determined to be 140m in length and an incident wave obliquity of 45° has been assumed.

D1.4 Rock Armour Revetment

This is a common wall construction type and geometry, and the representative section below has been used to assess its wave overtopping volume potential.

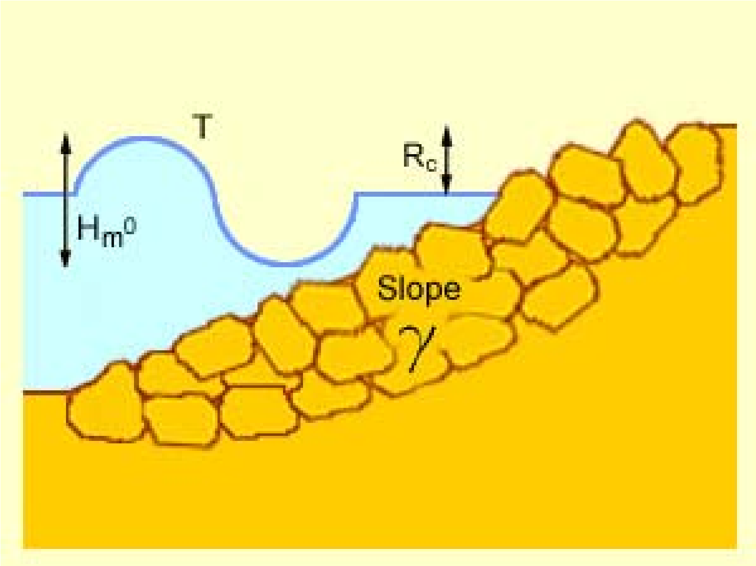

Figure D4 - Representation of Wall Type 4

The slope material reduction factor has been set to a value of 0.55, based on a standard reduction coefficient value for two layer rock armour structures.

Whilst this wall section extends to the tip of Elizabeth Harbour, a length of 50m has been considered where overtopped water is likely to drain towards Esplanade. An incident wave obliquity of 45° has been assumed.

D2 Met-Ocean Conditions

The met-ocean conditions for the site have been derived from existing reports completed by HR Wallingford (HRW).

The wave conditions were extracted from HRW Report EX5964 (2009) for a point offshore of the site. HRW computed the near-shore wave height within St Aubins Bay which showed a reduction in the wave height in the order of 10%. However the nearshore location HRW considered was away from the more complex geometry to the eastern end of the bay around the slipway which could act to concentrate wave activity, increasing the wave heights.

HRW have derived a relationship between the mean wave period, Tm, and offshore wave height, Hs, in their report HRW EX4020 (2001). This relationship was determined for wind generated waves propagating in a similar direction to those that will affect our site and has been used to determine wave periods for this analysis.

It was determined that for spectral waves the effect of wave transformation inshore considering wave shoaling, white capping and bed friction would possibly result in an increase in low frequency wave energy, resulting in an increase in wave period. As such a change in the wave period has not been used for the wave overtopping assessment.

The water levels derived within HRW Report EX5255 (2006) have been updated to give current day (2011) and future (2059) water levels, considering climate change. The combination of extreme water level and wave height event has been considered as discussed in Section D3.

D3 Overtopping Output

Three confidence bounds (upper, middle and lower) for the wave overtopping volume were calculated to take into account the uncertainties involved within wave overtopping discharge determination.

The following variables were altered to determine lower and upper bound overtopping volume rates across the different wall types. The middle bound was the average of the upper and lower bounds.

Lower bound

- Current day water levels were used (i.e. no allowance for climate change)
- A 10% reduction in wave height was allowed for wave transformation inshore
- A reduced correlation between maximum extreme water levels and wave heights was used (approximately 0.5). i.e. an extreme wave event of return period 50 years was combined with an extreme water level with return period 25 years.
- A probabilistic assessment was used. This relates the fit of the empirical wave overtopping model to the recorded data. Using the probabilistic tools 50% of the recorded data points exceed the model prediction and 50% fall below the predicted values.

Upper bound

- Future (50 year) water levels were used.
- No wave height reduction was allowed for – to allow for possible wave concentration.
- A correlation of 1.0 between extreme water levels and wave heights was assumed. Such that the combination of events with the same return period was considered, i.e. a 50 year return period wave event was combined with a 50 year water level event.
- A deterministic assessment was used. Using the deterministic design tools returns values as the mean value plus one standard deviation, allowing for model uncertainty.

The following table has the results of the wave overtopping analysis for return periods up to 100 years at the different bounds.

Return Period	**1 year**	**10 year**	**20 year**	**50 year**	**100 year**
	Volume	**Volume**	**Volume**	**Volume**	**Volume**
Bound	**[m³]**	**[m³]**	**[m³]**	**[m³]**	**[m³]**
Lower	3,850	21,500	33,500	56,000	135,000
Middle	11,425	62,250	96,250	155,500	257,000

Upper	19,000	103,000	159,000	255,00	379,000

Table D1 - Wave overtopping volumes

Wave overtopping volumes for event return periods exceeding 100 years were calculated but they have not been included because of the significant wave overtopping volumes that resulted at lower return periods.

As discussed in the main body of the report the resulting wave overtopping volumes have been assessed with respect to various drainage and water loss mechanisms to determine the final resulting wave overtopping volumes that could be expected at the site.

D4 Robustness

The EurOtop Manual is the most current and commonly used industry standard guidance for the derivation of wave overtopping discharges. The overtopping rates are calculated from empirically derived equations and should be regarded as being within, at best, a factor of 1 – 3 of the actual overtopping rate.