APPLICATION FORM
NOTIFICATION OF INTENT TO DEVELOP (NID)
SECTION 38 (1) AND SECTION 38 (8)

Heritage Western Cape Reference No:
To be completed by the applicant

22020203

Completion of this form is required by Heritage Western Cape for the initiation of all impact assessment processes under Section 38 (1) & (8) of the National Heritage Resources Act (NHRA)

As per Section 38 (1) (e) of the NHRA, submission of the NID must be initiated at the earliest stage of development. Should the development trigger any other legislation, practitioners may submit the NID without formal submission to other statutory bodies in order to comply with the NHRA.

This form is to be read in conjunction with the HWC Notification of Intent to Develop, Heritage Impact Assessment, (Pre-Application) Basic Assessment Reports, Scoping Reports and Environmental Impact Assessments, Guidelines for Submission to HWC

Whilst it is not a requirement, it may expedite processes and in particular avoid calls for additional information if certain of the information required in this form is provided by a heritage specialist/s with the necessary qualifications, skills and experience. All sections of the form must be completed in order to deem the application to be complete.

Making an incorrect statement or providing incorrect information may result in all or part of the application having to be reconsidered by HWC in the future, or submission of a new application.

The following information is to be included upon submission to HWC:

1. Proof of payment with correct reference number
2. Completed and signed application form – the application form must be completed in full in order to be considered
3. Power of Attorney
4. Locality Map
5. Images of the site and its context
6. Additional information pertaining to the heritage of the site

Application and associated documentation to be emailed to ceoheritage@westerncape.gov.za

A. APPLICABILITY OF THE NATIONAL ENVIRONMENTAL MANAGEMENT ACT (NEMA)

Department of Environmental Affairs Development Planning (Western Cape); Department of Mineral Resources (National); Department of Environmental Affairs (National);
Reference Number (if applicable): Ad-hoc setback line application not yet submitted to DEADP

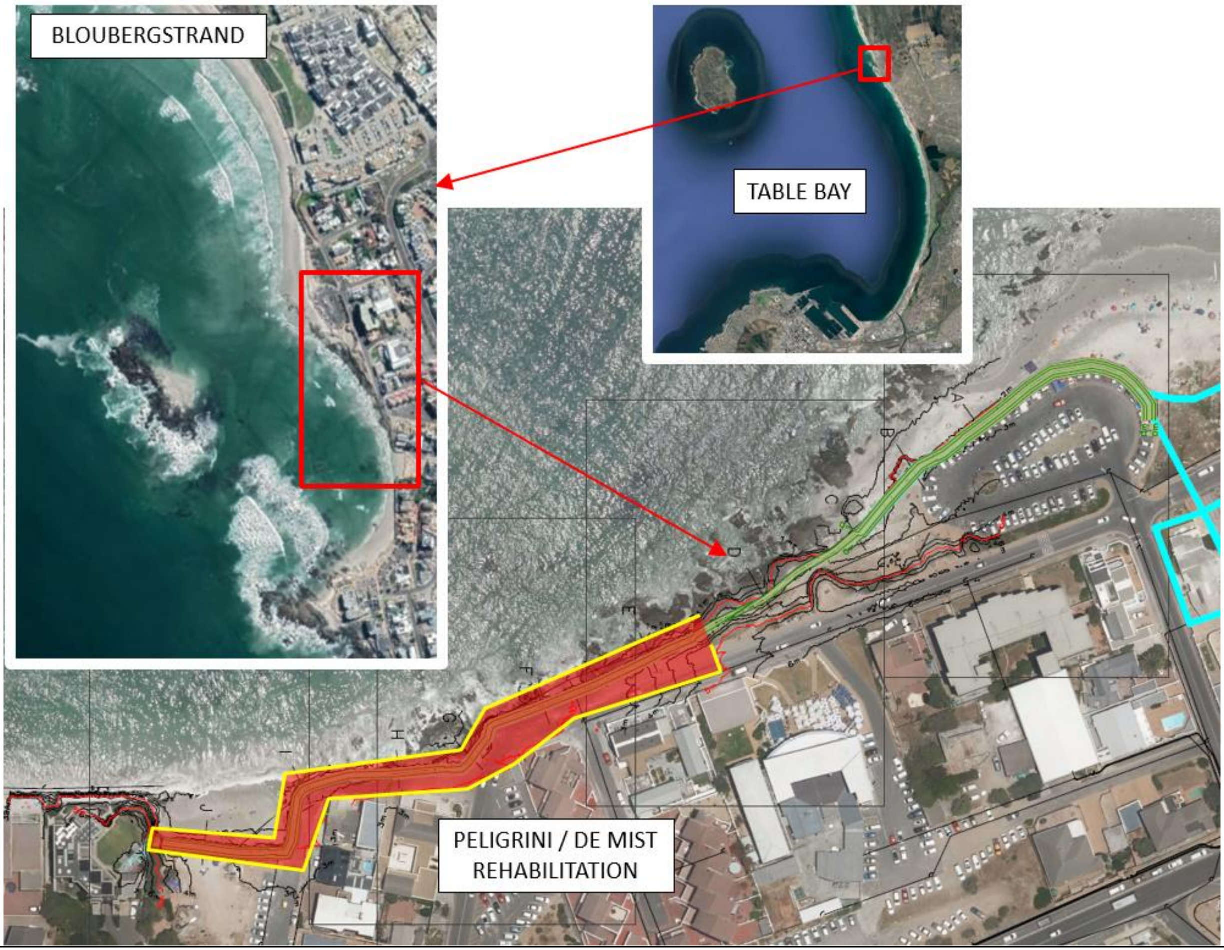
Please tick the applicable section:

☒	This application is made in terms of Section 38(8) of the NHRA and an application under NEMA has been made to the following authority: DEADP
☐	This development will not require a NEMA application.

B. BASIC DETAILS

PROPERTY DETAILS: Small Bay Seawall - Pelegrini / De Mist Rehabilitation

Name of property: Small Bay Sea Wall – area between De Mist road Parking in the North and Small Bay Park in the South. The site location of the Pelegrini De Mist Rehabilitation Project is shown in the Figures below.



Street address or location: Pelegrini and Popham Road, Blaauwberg - Small Bay

Erf or farm number/s: Portion of erf 241 Blaauwbergstrand Portion of Erf 253 Blaauwbergstrand unallocated land that is road and parking (See Appendix A showing land use and erf diagram	Coordinates: Start of site: S 18° 27' 29.91" E -33 47' 47.92" End of Site: S 18° 27' 28.06" E -33 47' 43.29" (A logical centre point. Format based on WGS84.)
Town or District: Blaauwberg – Small Bay	Municipality: City of Cape Town
Extent of property: Erf 241/ zoned vacant business/1959m2 Erf 253/ Zoned residential/ 744m2	Current use: seawall, car park, road, park.

Predominant land use/s of surrounding properties: residential and coast

REGISTERED OWNER OF PROPERTY:

Name and Surname: City of Cape Town

Address: Private Bag x9181, Cape Town. 8000

Telephone 021 487 2532	Cell 0839408143	E-mail gregg.oelofse@capetown.gov.za

APPLICANT/ AUTHORISED AGENT:

Name and Surname: Natalie Newman

Address: 44 Wale Street, Cape Town

Telephone 021 487 2123	Cell 072 495 9715	E-mail Natalie.newman@capetown.gov.za

By the submission of this form and all material submitted in support of this notification (ie: 'the material'), all applicant parties acknowledge that they are aware that the material and/or parts thereof will be put to the following uses and consent to such use being made: filing as a public record; presentations to committees, etc; inclusion in databases; inclusion on and downloading from websites; distribution to committee members and other stakeholders and any other use required in terms of powers, functions, duties and responsibilities allocated to Heritage Western Cape under the terms of the National Heritage Resources Act. Should restrictions on such use apply or if it is not possible to copy or lift information from any part of the digital version of the material, the material will be returned unprocessed. All sections of the form have been completed.

Signature of Owner: Gregg Oelofse Digitally signed by Gregg Oelofse Date: 2022.02.08 08:49:37 +02'00' Date:

Should the owner not be able to sign, the applicants/ agents must attach copy of power of attorney to this form.

Signature of Applicant/ Authorised Agent: Date:

Applicants/ agents must attach copy of power of attorney to this form.

C. DEVELOPMENT DETAILS:

Please indicate below which of the following Sections of the National Heritage Resources Act, or other legislation has triggered the need for notification of intent to develop.

☒	S38(1)(a) Construction of a road, wall, powerline, pipeline, canal or other similar form of linear development or barrier over 300m in length.		S38(1)(c) Any development or activity that will change the character of a site -
☐	S38(1)(b) Construction of a bridge or similar structure exceeding 50m in length.	☐	(i) exceeding 5 000m² in extent;
☐	S38(1)(d) Rezoning of a site exceeding 10 000m² in extent.	☒	(ii) involving three or more existing erven or subdivisions thereof;
☐	Other triggers, eg: in terms of other legislation, (ie: National Environment	☐	(iii) involving three or more erven or divisions thereof which have been

Management Act, etc.) Please set out details:

consolidated within the past five years.

If you have checked any of the three boxes above, describe how the proposed development will change the character of the site:

There are 3 key components of the project; (1) seawall, (2) sewer and (3) roadway. All of the 3 components are existing infrastructure, which has deteriorated to such an extent that it has to be re-constructed. Although the existing footprint and configuration of the re-constructed infrastructure/components are similar to the existing (like-for-like), there are minor changes to the re-constructed infrastructure such as follows:

- Seawall: like for like in terms of location, orientation, footprint, structure type (gravity block wall), and structure height. Minor changes comprise structural design (e.g. anchoring of block wall to bedrock) as well as a handrail for public safety which was designed as a concrete stanchions/barrier instead of typical steel handrails. Reasons for concrete barrier instead of steel handrail is due to maintenance and mitigation of wave overtopping to some extent.

- Sewer: Shall change in terms of existing sewer to be re-routed slightly landwards (instead of located inside existing seawall at present, which result in risk of failure, re-routed more landwards to be located inside roadway). Sewer length, flows and configuration remain the same, therefore like-for-like.

- Roadway: Damaged portion of road (Popham and Pelegrini) to be refurbished like for like. Road alignment, elevations and configuration (width) to remain the same, however type of surface to change from asphalt to concrete at certain locations to make allowance for wave overtopping unto road.

See appendix B

If an impact assessment process has also been / will be initiated in terms of other legislation please provide the following information: As the project is the refurbishment /rehabilitation of what was there previously there was no impact assessment, however an Environmental Management Programme (EMPR) will be included with the construction tender to mitigate any environmental or heritage impacts.

Authority / government department (ie: consenting authority) to which information has been /will be submitted for final decision: Department of Environmental Affairs and Development Planning

Present phase at which the process with that authority stands: The City is in the process of submitting the setback line application to the Department.

Estimated value cost of the project in South African Rands: R 45 million

D. ANTICIPATED IMPACTS ON HERITAGE RESOURCES

Section 3 of the National Heritage Resources Act sets out the following categories of heritage resource as forming part of the national estate. Please indicate the known presence of any of these by checking the box alongside and then providing a description of each occurrence, including nature, location, size, type

Failure to provide sufficient detail or to anticipate the likely presence of heritage resources on the site may lead to a request for more detailed specialist information.

Provide a short history of the site and its environs (Include sources where available):
The affected properties are situated on the coast and has historically mainly housed public facilities.

Please indicate which heritage resources exist on the site and in its environs, describe them and indicate the nature of any impact upon them:

☐	**Places, buildings, structures and equipment of cultural significance** N/A Description of resource: Description of impact on heritage resource:
☐	**Places to which oral traditions are attached or which are associated with living heritage** N/A Description of resource:

	Description of impact on heritage resource:
☐	**Historical settlements and townscapes** N/A Description of resource: Description of impact on heritage resource:
☐	**Landscapes and natural features of cultural significance** N/A Description of resource: Description of impact on heritage resource:
☐	**Geological resources of scientific or cultural importance** N/A Description of resource: Description of impact on heritage resource:
☒	**Archaeological resources** (Including archaeological sites and material, rock art, battlefields & wrecks)**:** Description of resource: Coastal archaeology including shell middens (incl burials) and shipwreck material have been recorded along this stretch of coast. Description of impact on heritage resource: Although the affected area is situated in an area of known archaeological sensitivity, the archaeological potential risk is low considering the project area has already been transformed.
☒	**Palaeontological resources** (ie: fossils)**:** Description of resource: The proposed work is located in an area identified on SAHRIS as an area of moderate palaeosensitivity. Description of impact on heritage resource: There is little reason to believe that palaeontological heritage resources will be impacted as the work proposed in an area that is already fairly disturbed and the work proposed not of any significant depth.

☐	**Graves and burial grounds** (eg: ancestral graves, graves of victims of conflict, historical graves & cemeteries): N/A Description of Resource: Description of Impact on Heritage Resource:
☒	**Other human remains:** Description of resource: Human remains associated with shell middens along the coast. Description of impact on heritage resource: Although the affected area is situated in an area of known archaeological sensitivity, the archaeological potential risk is low considering the project area has already been transformed.
☐	**Sites of significance relating to the history of slavery in South Africa:** N/A Description of resource: Description of impact on heritage resource:
☐	**Other heritage resources:** N/A Description of resource: Description of impact on heritage resource:

Describe elements in the environs of the site that could be deemed to be heritage resources:
Description of impacts on heritage resources in the environs of the site:

There is a **low likelihood** that archaeological heritage resources may be impacted during the course of the work proposed in this area.

Summary of anticipated impacts on heritage resources:

Accidental disturbance of archaeological heritage resources (shell middens and associated burials) are a possibility, but are **not likely** given that the project area has been extensively altered and transformed.

E. ILLUSTRATIVE MATERIAL:

Attach to this form a minimum A4 sized locality plan showing the boundaries of the area affected by the proposed development, its environs, property boundaries and a scale. The plan must be of a scale and size that is appropriate to creating a clear understanding of the development.

Attach also other relevant graphic material such as maps, site plans, satellite photographs and photographs of the site and the heritage resources on it and in its environs. These are essential to the processing of this notification.

Please provide all graphic material on paper of appropriate size and on CD/ USB in JPEG format. It is essential that graphic material be annotated via titles on the photographs, map names and numbers, names of files and/or provision of a numbered list describing what is visible in each image.

F. RECOMMENDATION

In your opinion do you believe that a heritage impact assessment is required? ☐ Yes ☒ No

Recommendation made by:

Name Natalie Newman

Capacity Project Management (Head: Coastal Management)

With the assistance from Harriet Clift and Sonja Warnich-Stemmet of the Environmental Management Branch, City of Cape Town

PLEASE NOTE: No Heritage Impact Assessment should be submitted with this form or conducted until Heritage Western Cape has expressed its opinion on the need for such and the nature thereof.

G. INFORMATION TO BE PROVIDED AND STUDIES TO BE CONDUCTED AS PART OF THE HERITAGE IMPACT ASSESSMENT (HIA)

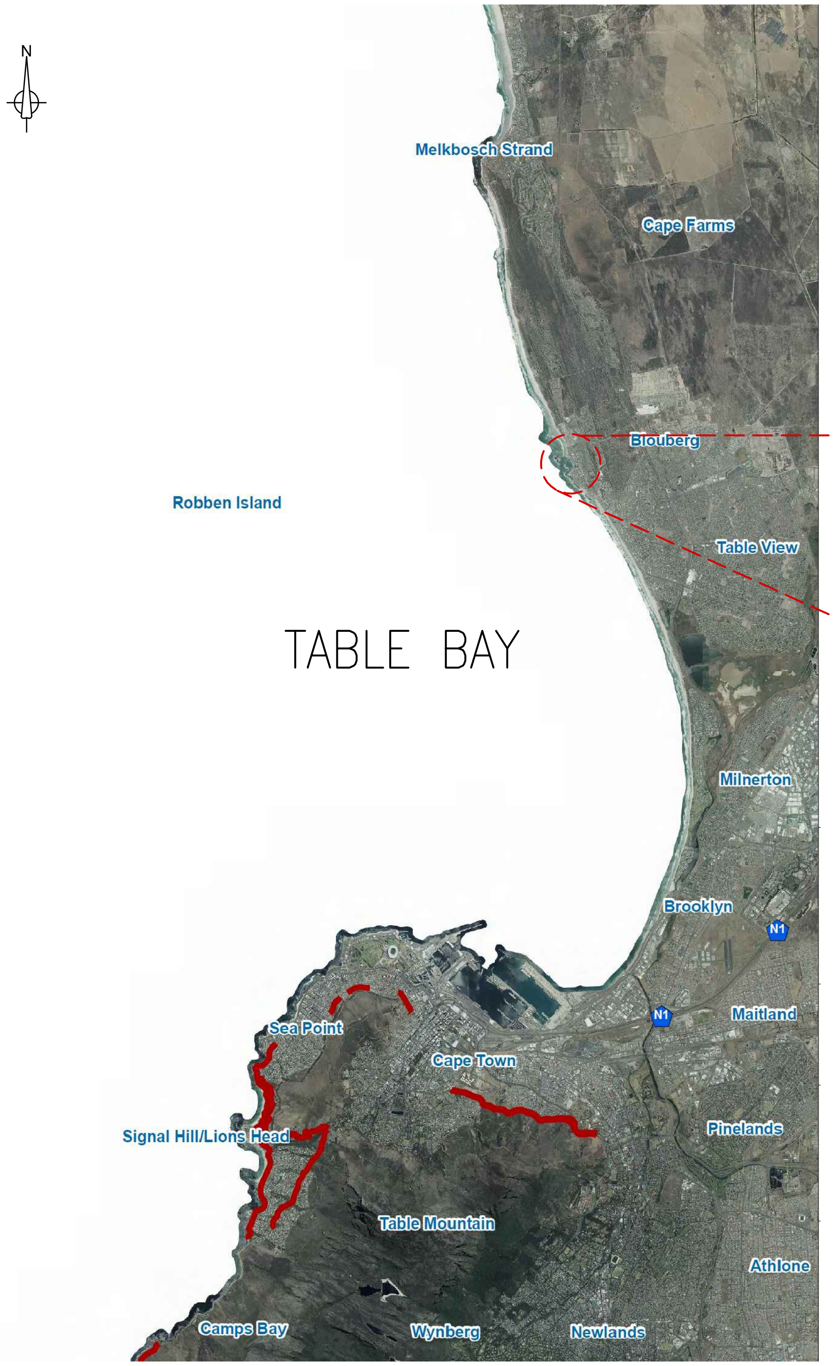
If it is recommended that an HIA is required, please complete this section of the form. N/A

DETAILS OF STUDIES TO BE CONDUCTED IN THE INTENDED HIA

In addition to the requirements set out in Section 38(3) of the NHRA, indicate envisaged studies:

☐	Heritage resource-related guidelines and policies.
☐	Local authority planning and other laws and policies.
☐	Details of parties, communities, etc. to be consulted.
☐	Specialist studies, eg: archaeology, palaeontology, architecture, townscape, visual impact, etc. Provide details:
☒	Other. Provide details: Recommended that archaeological monitoring be implemented in the EMPr with specific instructions to the EP in order to facilitate the mitigation of heritage resources accidentally impacted, should it be necessary to mitigate. • Should concentrations of shell be observed that work in that area cease until it can be inspected (Metro office archaeologists can assist to identify whether occurrence is natural or cultural), and HWC must advise as to the mitigation process. • Should any human remains be disturbed, work in the area must cease immediately and the site secured. SAPS and HWC must be notified immediately in order to confirm whether the remains are recent or archaeological. If the remains are recent, SAPS processes will be followed. Should the remains be archaeological, HWC must advise on the mitigation process.

PLEASE NOTE: Any further studies which Heritage Western Cape requires should be submitted must be in the form of a single, consolidated report with a single set of recommendations. Specialist studies must be incorporated in full, either as chapters of the report, or as annexures thereto.
Please refer to the *Guidelines for Heritage Impact Assessments required in terms of Section 38 of the National Heritage Resources Act (Act 25 of 1999)*



GENERAL LOCATION

PROJECT LOCATION

SCALE 1:2000

REVISIONS		
NO	DATE	DESCRIPTION
A	2022-01-06	INITIAL ISSUE FOR APPROVAL

SURVEYED	N/A
DESIGNED	M REBEL
CAD	M REBEL
EXAMINED	M LE ROUX
APPROVED	M LE ROUX

HEAD:
COASTAL ENGINEERING
AND DESIGN

CITY OF CAPE TOWN

SPATIAL PLANNING AND ENVIRONMENT

SMALL BAY SEAWALL REFURBISHMENT

SETBACK LINE APPLICATION
LOCALITY PLAN

DRAWING STATUS: APPROVAL

CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD
Making progress possible. Together.

SCALE: AS SHOWN

19379-CMB-GA-1000

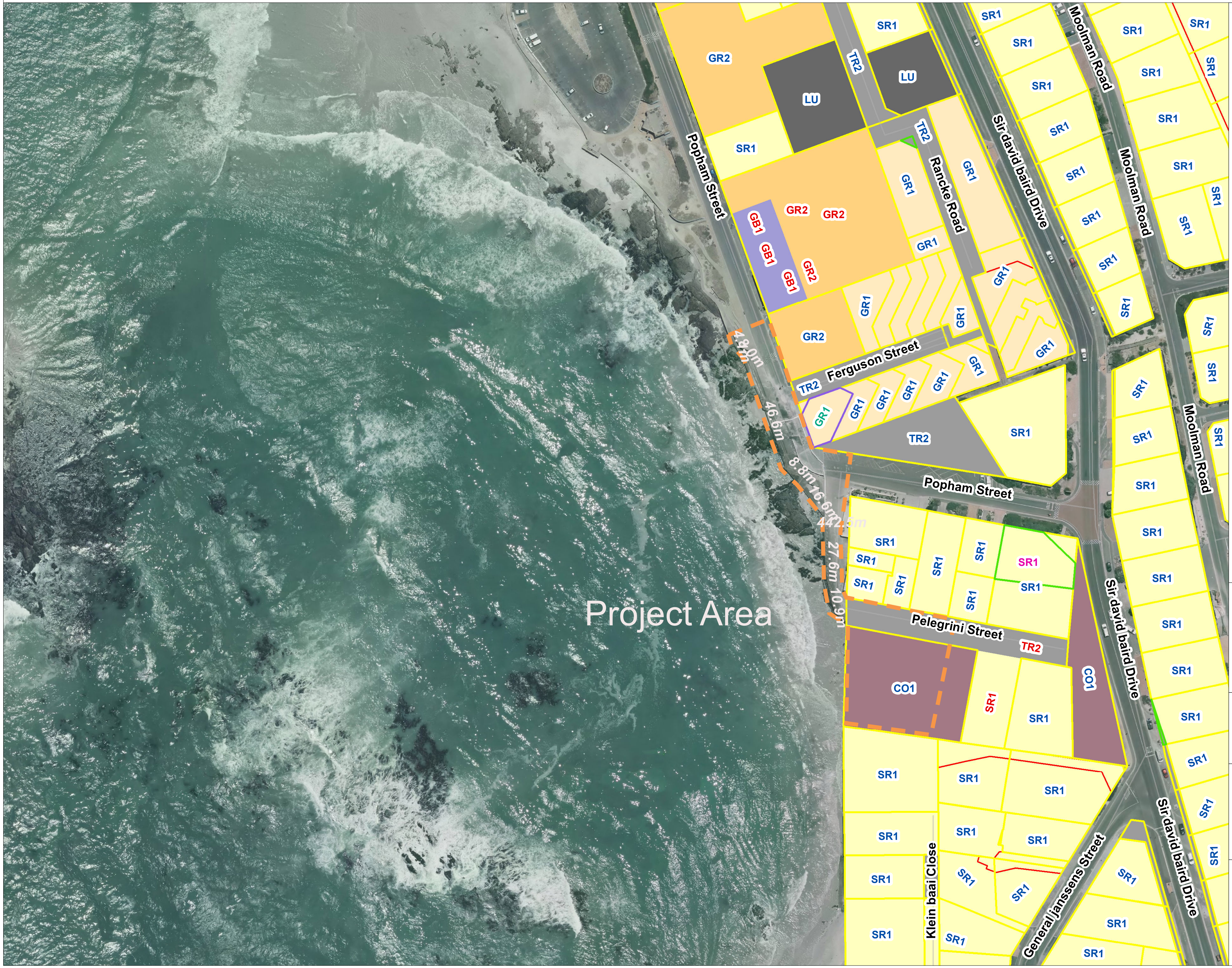
A3

REVISIONS A

A:\19379_AC_Small Bay seawall\05 Drawings\02 Current\03 Drawings\CCT-19379-CMB-GA-Setback-1000_Rev A.dwg

PLOT DATE: 2022/01/06 14:36:17

Zoning Plan



Legend

— Street Name
▭ SG Approved
▭ Confirmed
▭ Registered
▭ Servitude

Zoning Split Zoning

INT_ZONE_DESC

▬ General Business 1
▬ General Residential 2
▬ Single Residential 1 : Conventional Housing
▬ Transport 2 : Public Road and Public Parking

Zoning SG Approved Properties

INT_ZONE_DESC

▬ Single Residential 1 : Conventional Housing
▬ Transport 2 : Public Road and Public Parking

Zoning Confirmed Properties

INT_ZONE_DESC

▬ General Residential 1 : Group Housing

Zoning Registered Properties

▭ <all other values>

INT_ZONE_DESC

▬ Community 1 : Local
▬ General Residential 1 : Group Housing
▬ General Residential 2
▬ Limited Use Zone
▬ Single Residential 1 : Conventional Housing
▬ Transport 2 : Public Road and Public Parking

N

1:1 500

Transverse Mercator Projection, Central Meridian 19º East,
WGS84 Ellipsoid using the Hartebeesthoek94 Datum

THIS MAP WAS GENERATED BY THE

CityMap Viewer

Information & Knowledge Management Department

Author: M Rebel
Date: 06 January 2022

CITY OF CAPE TOWN
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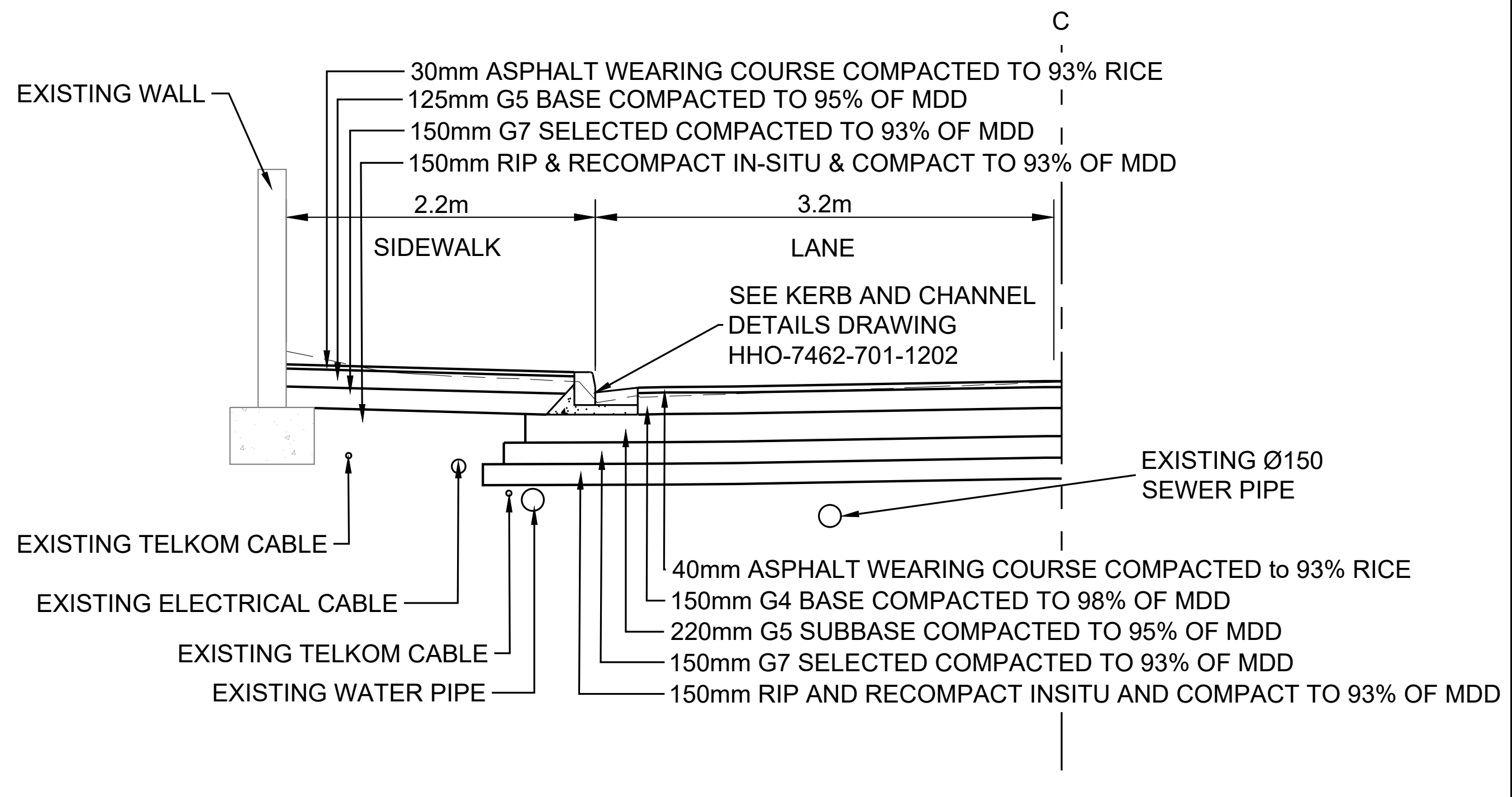
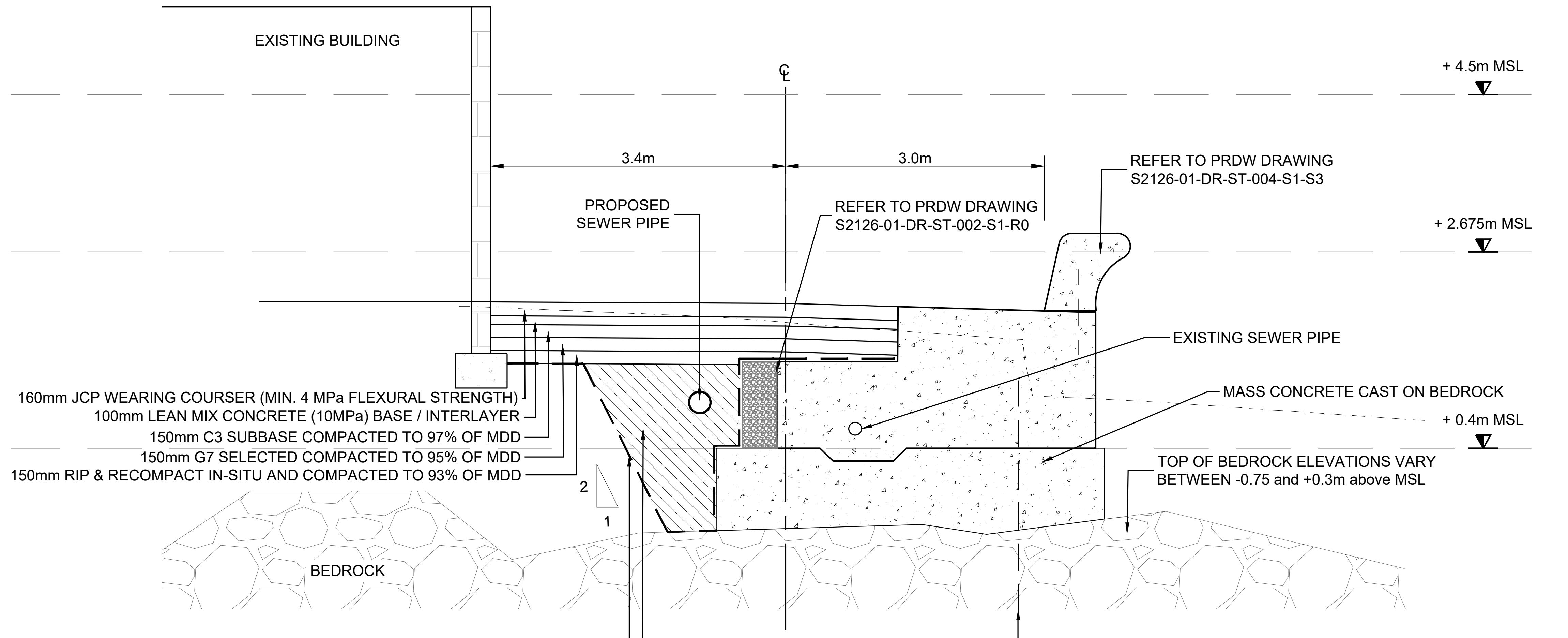
Making progress possible. Together.



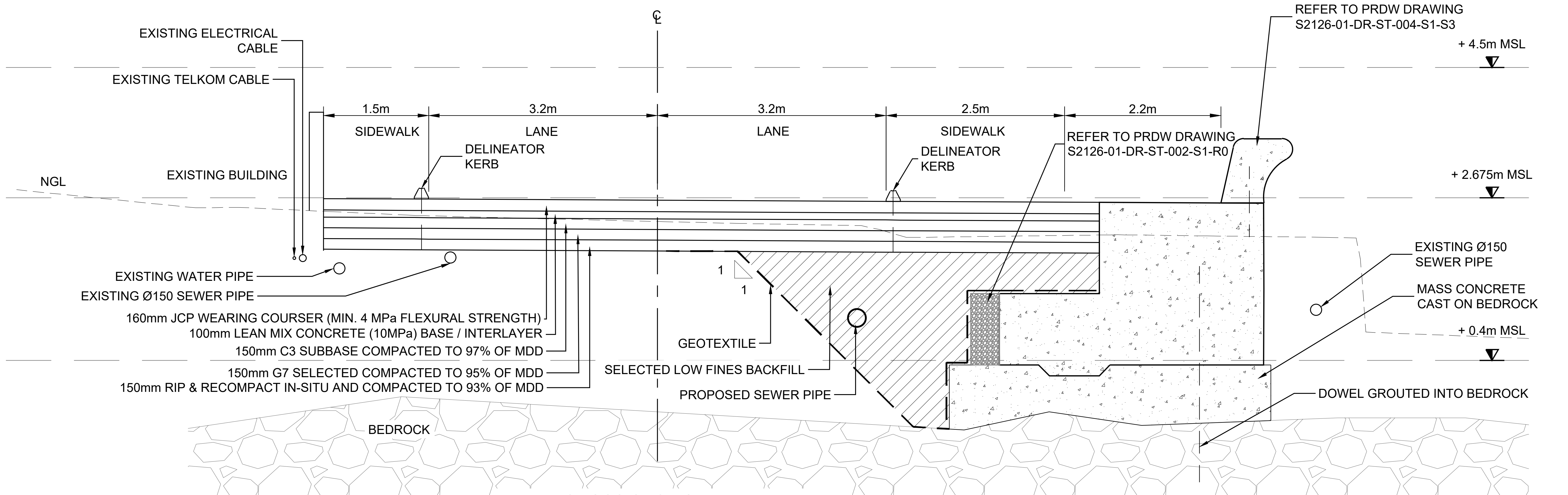
CROSS SECTION D - D
POPHAM STR SV 75



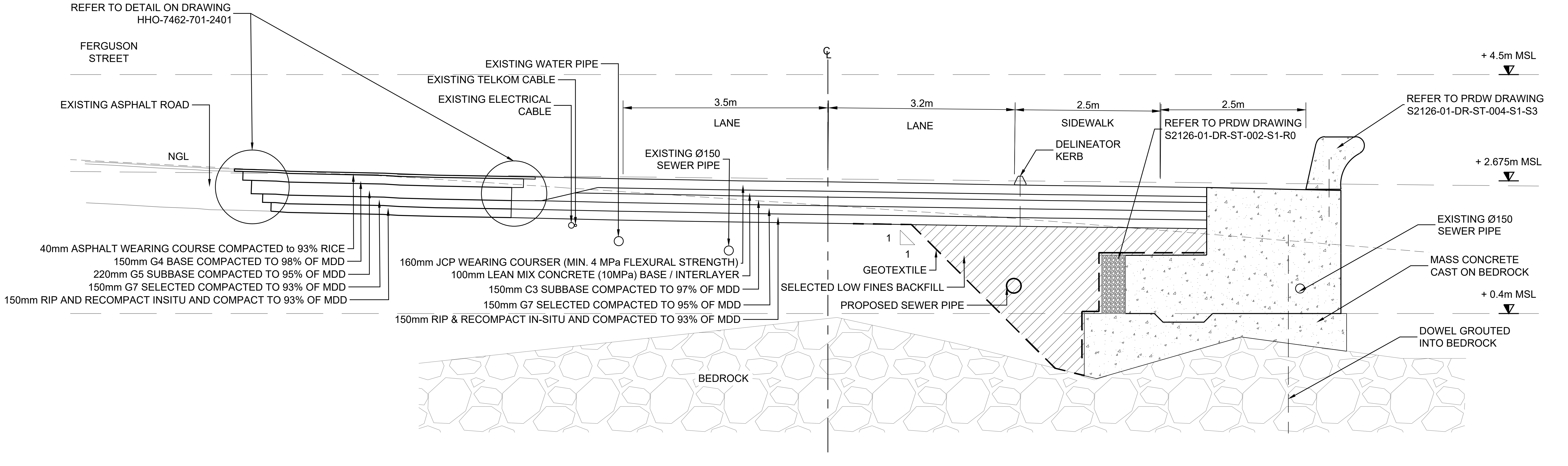
CROSS SECTION A - A
PELEGRINI STR SV 46

NOTE:

1. TYPICAL CROSS SECTIONS TO BE READ IN CONJUCTION WITH GENERAL LAYOUT DRAWING HHO-7462-701-1001
2. FOR EXISTING AND PROPOSED FOUL SEWER LAYOUT SEE DRAWINGS HHO-7462-701-1601 TO 1602
3. FOR EXISTING AND PROPOSED STORMWATER LAYOUT SEE DRAWINGS HHO-7462-701-1621
4. FOR EXISTING ELECTRICAL & TELECOMMUNICATIONS LAYOUT SEE DRAWING HHO-7462-701-1641
5. FOR EXISTING WATER LAYOUT SEE DRAWING HHO-7462-701-1661
6. FOR SEWER DETAILS SEE DRAWING HHO-7462-701-2501
7. FOR SEA WALL LAYOUT REFER TO PRDW DRAWING S2126-01-DR-ST-003-S1 TO S8



CROSS SECTION B - B
POPHAM STR SV 30



CROSS SECTION C - C
POPHAM STR SV 55

NEW CONCRETE CARRIAGEWAY PAVEMENT SPECIFICATION	
LAYER / MATERIAL	SPECIFICATION
160mm JOINTED CONCRETE PAVEMENT (JCP) WEARING COURSE	UNREINFORCED JOINTED CONCRETE SLABS WITH A MINIMUM FLEXURAL STRENGTH OF 4 Mpa AT 28-DAYS
100mm BASE / INTERLAYER	MINIMUM 10 MPa LEAN MIX CONCRETE (28-DAY STRENGTH)
150mm CEMENT STABILISED MATERIAL (C3) SUBBASE	MINIMUM G5 QUALITY MATERIAL CONFORMING TO COTO SPECIFICATION STABILISED WITH CEMENT TO ACHIEVE A MINIMUM UNCONFINED COMPRESSIVE STRENGTH (UCS) OF 1,5 Mpa AND MAXIMUM OF 3,0 Mpa WHEN COMPACTED TO 100% OF MAXIMUM DRY DENSITY (MDD). THE INDIRECT TENSILE STRENGTH (ITS) SHALL BE MINIMUM 250kPa AND MAXIMUM 400kPA AT 100% OF MDD. THE 150mm THICK LAYER WILL BE COMPACTED TO A MINIMUM OF 97% OF MDD.
150mm G7 SELECTED LAYER	G7 GRAVEL CRUSHED STONE OR NATURAL GRAVEL MATERIAL COMPACTED TO 95% OF MDD. THE MATERIAL SHALL HAVE A MINIMUM 7 DAY SOAKED CBR OF 15% WHEN COMPACTED TO 93% MAXIMUM DRY DENSITY (MDD) AND A MAXIMUM PLASTICITY INDEX (PI) OF 3 TIMES THE GRADING MODULUS ON THE MATERIAL PASSING THE 0,425mm SIEVE. THE GM SHALL BE WITHIN THE BOUNDARY LIMITS OF 0,75 AND 2,7.
IN-SITU SUBGRADE	WHERE THE IN-SITU MATERIAL CONFORM TO A MINIMUM G8 QUALITY MATERIAL IT SHALL BE RIPPED AT A DEPTH OF 150mm AND COMPACTED TO A MINIMUM OF 93% MAXIMUM DRY DENSITY (MDD) (100% FOR SAND). WHERE THE IN-SITU MATERIAL DOES NOT CONFORM TO A MINIMUM G8 QUALITY MATERIAL, IT SHALL BE REMOVED AT A DEPTH OF 150mm AND REPLACED WITH A G8 GRAVEL CRUSHED STONE / NATURAL GRAVEL / SAND MATERIAL AND COMPACTED TO 93% OF MDD FOR A GRAVEL OR 100% FOR SAND. THE MATERIAL SHALL HAVE A MINIMUM 7 DAY SOAKED CBR OF 25% WHEN COMPACTED TO 95% MDD AND A MAXIMUM PLASTICITY INDEX (PI) OF 3 TIMES THE GRADING MODULUS ON THE MATERIAL PASSING THE 0,425mm SIEVE. THE GM SHALL BE WITHIN THE BOUNDARY LIMITS OF 0,75 AND 2,7. THE IN-SITU MATERIAL SHALL BE RIPPED AND RECOMPACTED TO 93% MDD BEFORE CONSTRUCTING THE G8 LAYER.
NEW FLEXIBLE PAVEMENT SPECIFICATION	
LAYER / MATERIAL	SPECIFICATION
40mm ASPHALT WEARING COURSE	10mm MAXIMUM AGGREGATE SIZE USING 50/70 PENETRATION GRADE BITUMEN AND COMPACTED TO A MINIMUM DENSITY OF 93% OF RICE.
150mm G4 BASE LAYER	G4A GRAVEL CRUSHED STONE MATERIAL COMPACTED TO 98% MAXIMUM DRY DENSITY (MDD). THE MATERIAL SHALL HAVE A MINIMUM 7 DAY SOAKED CBR OF 80% WHEN COMPACTED TO 100% MDD AND A MAXIMUM PLASTICITY INDEX (PI) OF 6 ON THE MATERIAL PASSING THE 0,425mm SIEVE. THE SWELL AT 100% OF MDD SHALL BE LESS THAN 0,2%.
220mm G5 SUBBASE LAYER	G5B GRAVEL CRUSHED STONE OR NATURAL GRAVEL MATERIAL COMPACTED TO 95% OF MDD. THE MATERIAL SHALL HAVE A MINIMUM 7 DAY SOAKED CBR OF 45% WHEN COMPACTED TO 95% MAXIMUM DRY DENSITY (MDD) AND A MAXIMUM PLASTICITY INDEX (PI) OF 15 ON THE MATERIAL PASSING THE 0,425mm SIEVE. THE GM SHALL BE WITHIN THE BOUNDARY LIMITS OF 1,5 AND 2,4.
150mm G7 SELECTED LAYER	G7 GRAVEL CRUSHED STONE OR NATURAL GRAVEL MATERIAL COMPACTED TO 93% OF MDD. THE MATERIAL SHALL HAVE A MINIMUM 7 DAY SOAKED CBR OF 15% WHEN COMPACTED TO 93% MAXIMUM DRY DENSITY (MDD) AND A MAXIMUM PLASTICITY INDEX (PI) OF 3 TIMES THE GRADING MODULUS ON THE MATERIAL PASSING THE 0,425mm SIEVE. THE GM SHALL BE WITHIN THE BOUNDARY LIMITS OF 0,75 AND 2,7.
IN-SITU SUBGRADE	WHERE THE IN-SITU MATERIAL CONFORM TO A MINIMUM G8 QUALITY MATERIAL IT SHALL BE RIPPED AT A DEPTH OF 150mm AND COMPACTED TO A MINIMUM OF 93% MAXIMUM DRY DENSITY (MDD) (100% FOR SAND). WHERE THE IN-SITU MATERIAL DOES NOT CONFORM TO A MINIMUM G8 QUALITY MATERIAL, IT SHALL BE REMOVED AT A DEPTH OF 150mm AND REPLACED WITH A G8 GRAVEL CRUSHED STONE / NATURAL GRAVEL / SAND MATERIAL AND COMPACTED TO 93% OF MDD FOR A GRAVEL OR 100% FOR SAND. THE MATERIAL SHALL HAVE A MINIMUM 7 DAY SOAKED CBR OF 25% WHEN COMPACTED TO 95% MDD AND A MAXIMUM PLASTICITY INDEX (PI) OF 3 TIMES THE GRADING MODULUS ON THE MATERIAL PASSING THE 0,425mm SIEVE. THE GM SHALL BE WITHIN THE BOUNDARY LIMITS OF 0,75 AND 2,7. THE IN-SITU MATERIAL SHALL BE RIPPED AND RECOMPACTED TO 93% MDD BEFORE CONSTRUCTING THE G8 LAYER.
SIDEWALK PAVEMENT SPECIFICATION	
LAYER / MATERIAL	SPECIFICATION
30mm ASPHALT WEARING COURSE	10mm MAXIMUM AGGREGATE SIZE USING 50/70 PENETRATION GRADE BITUMEN AND COMPACTED TO A MINIMUM DENSITY OF 93% OF RICE.
125mm G5 BASE LAYER	G5B GRAVEL CRUSHED STONE OR NATURAL GRAVEL MATERIAL COMPACTED TO 95% OF MDD. THE MATERIAL SHALL HAVE A MINIMUM 7 DAY SOAKED CBR OF 45% WHEN COMPACTED TO 95% MAXIMUM DRY DENSITY (MDD) AND A MAXIMUM PLASTICITY INDEX (PI) OF 15 ON THE MATERIAL PASSING THE 0,425mm SIEVE. THE GM SHALL BE WITHIN THE BOUNDARY LIMITS OF 1,5 AND 2,4.
150mm G7 SELECTED LAYER	G7 GRAVEL CRUSHED STONE OR NATURAL GRAVEL MATERIAL COMPACTED TO 93% OF MDD. THE MATERIAL SHALL HAVE A MINIMUM 7 DAY SOAKED CBR OF 15% WHEN COMPACTED TO 93% MAXIMUM DRY DENSITY (MDD) AND A MAXIMUM PLASTICITY INDEX (PI) OF 3 TIMES THE GRADING MODULUS ON THE MATERIAL PASSING THE 0,425mm SIEVE. THE GM SHALL BE WITHIN THE BOUNDARY LIMITS OF 0,75 AND 2,7.
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ISSUED FOR
APPROVAL

ON ORIGINAL
0 5 10 15 20 25 30 40 50 60mm

Rev	Date	Prepared by	Checked by	Approved by	Consultant Revision Details	Status
D	08/12/2021	SA	MW	FdV	ISSUED FOR APPROVAL	IFA

Client
CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD
Making progress possible. Together.

Client Approval .... Date ....
Consultant Signature .... Date ....

Project Title
SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

Drawing Title
TYPICAL CROSS SECTIONS

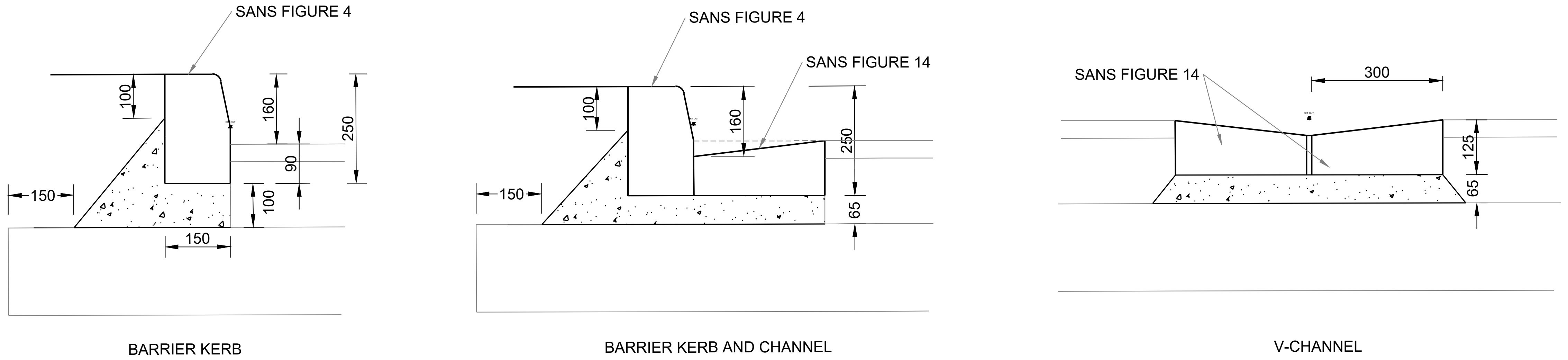
hho
14th FLOOR, THE TOWERS SOUTH,
2 HERTZOG BOULEVARD,
CAPE TOWN,
SOUTH AFRICA,
8001
TEL: +27 21 425 2870
FAX: +27 21 419 4689
EMAIL: enquiries@hho.co.za

Client Number
SB_19379_C_201_D

Scale
1:100 @ A1

Drawing Number
HHO-7457-701-1101

Rev
D



BARRIER KERB

BARRIER KERB AND CHANNEL

V-CHANNEL

KERB & CHANNEL DETAILS

SCALE 1:10

NOTE:
ALL UNITS TO BE LAID ON 25MPa CONCRETE HAUNCHING/BEDDING.

PROPOSED ASPHALT SETTING OUT COORDINATES SYSTEM WGS 84 Lo 19

POINT	Y	X	Z
P1	50 141.657	3 741 249.854	3.350m
P2	50 147.880	3 741 252.077	3.325m
P3	50 138.480	3 741 258.535	3.026m
P4	50 144.719	3 741 260.768	2.882m
P5	50 128.557	3 741 266.011	2.900m
P6	50 131.917	3 741 267.198	2.850m
P7	50 132.352	3 741 267.651	2.800m
P8	50 132.543	3 741 268.293	2.792m
P9	50 132.980	3 741 268.620	2.782m
P10	50 126.155	3 741 272.838	3.000m
P11	50 128.505	3 741 274.428	2.817m
P12	50 130.061	3 741 277.167	2.786m
P13	50 108.483	3 741 300.731	3.042m
P14	50 109.685	3 741 306.950	2.915m
P15	50 100.172	3 741 302.140	3.600m
P16	50 101.025	3 741 308.909	3.462m
P17	50 144.682	3 741 250.897	3.448m
P18	50 140.261	3 741 253.669	3.163m
P19	50 143.229	3 741 254.764	3.200m
P20	50 146.468	3 741 255.959	3.068m
P21	50 127.464	3 741 266.946	2.900m
P22	50 130.020	3 741 267.869	2.833m
P23	50 089.957	3 741 357.890	2.684m
P24	50 085.041	3 741 358.802	2.900m
P25	50 085.473	3 741 361.268	2.950m
P26	50 090.565	3 741 361.167	2.652m
P27	50 086.541	3 741 367.356	2.850m
P28	50 091.487	3 741 366.554	2.678m

PROPOSED DELINEATOR KERB SETTING OUT COORDINATES WGS84

POINT	Y	X
DK1	50138.480	3741258.535
DK2	50125.008	3741268.202
DK3	50130.383	3741280.412
DK4	50125.898	3741292.819
DK5	50125.036	3741294.584
DK6	50144.623	3741260.725
DK7	50132.230	3741294.936
DK8	50124.625	3741303.327
DK9	50129.014	3741299.885

LEGEND

▨	PROPOSED UPGRADED ASPHALT PAVEMENT
░	PROPOSED UPGRADED CONCRETE PAVEMENT
▓	PROPOSED SEA WALL
— — —	PROPOSED DELINEATOR KERB SEE DETAIL DRAWING HHO-7462-701-2401
—	EXISTING DEVELOPMENT BOUNDARY

NOTE:

1. FOR GENERAL LAYOUT SEE DRAWING HHO-7462-701-1001
2. FOR TYPICAL CROSS SECTIONS REFER TO DRAWING HHO-7462-701-1101
3. FOR CONCRETE JOINT DETAILS SEE DRAWING HHO-7462-701-2401

ISSUED FOR APPROVAL

ON ORIGINAL

0 5 10 15 20 25 30 40 50 60mm

Rev	Date	Prepared by	Checked by	Approved by	Consultant Revision Details	Status
A	30/11/2021	RA	FdV	FdV	ISSUED FOR APPROVAL	IFA

Client

CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD
Making progress possible. Together.

Client Approval .... Date ....

Consultant Signature .... Date ....

Project Title
SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

Drawing Title
PROPOSED ASPHALT ROAD AND
DELINEATOR KERB SETTING OUT

hho
14th FLOOR, THE TOWERS SOUTH,
2 HERTZOG BOULEVARD,
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8001
TEL: +27 21 425 2870
FAX: +27 21 419 4689
EMAIL: enquiries@hho.co.za

Client Number
SB_19379_C_302_A

Scale
1:250 @ A1

Drawing Number
HHO-7462-701-1202

Rev
A

Saleem J:\data\7462 375C Small Bay (FdV)\70 Urban\03 Drawings\01 Layout Dwgs\HHO-7462-701-1202 Rev A.dwg 09/12/2021

FOUL SEWER SETTING OUT
COORDINATE SYSTEM WGS84

NAME	Y	X	CL	IL1	IL2	NOTES
FSMH01	50194.764	3741168.121	3.557	2.60		TYPE A MANHOLE
FSMH02	50163.986	3741202.383	5.829	2.29		TYPE B MANHOLE
FSMH03	50146.853	3741249.441	3.603	1.95		TYPE B MANHOLE
EX.MH01	50191.698	3741160.185	3.88	3.06		
EX.MH02	50185.779	3741134.767	6.15	5.20		
EX.MH03	50163.284	3741195.915	5.90	3.99		
EX.MH04	50150.932	3741229.256	4.95	2.99		
EX.MH05	50160.115	3741241.104	3.19	1.33		MANHOLE TO BE DEMOLISHED

NOTES
1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE PROJECT SPECIFICATIONS AND THE FOLLOWING DRAWINGS:
    - HHO-7462-701-1001 GENERAL LAYOUT
    - HHO-7462-701-1011 PROPOSED PHASING OF CONSTRUCTION
    - HHO-7462-701-2501 SEWER DETAILS
    - HHO-7462-701-2511 SEWER PIPE LONGSECTION
2. ALL uPVC PIPES TO BE IN ACCORDANCE WITH SANS 4435.
3. ALL HDPE PIPES TO BE IN ACCORDANCE WITH SANS 4427.
4. CONSTRUCTION OF SEWER TO BE IN ACCORDANCE WITH SANS1200DB, SANS1200LD AND SANS 1200LB.

ISSUED FOR APPROVAL

LEGEND

—▶— S —	EXISTING SEWER PIPE
⩘▶⩘ S ⩘	EXISTING SEWER PIPE TO BE REMOVED
—▶— S —	PROPOSED SEWER PIPE PIPE
—— ——	EXISTING DEVELOPMENT BOUNDARY
▨	PROPOSED SEA WALL

ON ORIGINAL
0 5 10 15 20 25 30 40 50 60mm

Rev	Date	Prepared by	Checked by	Approved by	Consultant Revision Details	Status
D	30/11/2021	MW	FdV	FdV	ISSUED FOR APPROVAL	IFA

Client
CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD
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Client Approval .... Date ....

Consultant Signature .... Date ....

Project Title
SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

Drawing Title
EXISTING AND PROPOSED FOUL SEWER
LAYOUT: SHEET 1 OF 2

hho
14th FLOOR, THE TOWERS SOUTH,
2 HERTZOG BOULEVARD,
CAPE TOWN,
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8001
TEL: +27 21 425 2870
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EMAIL: enquiries@hho.co.za

Client Number
SB_19379_C_401 D

Scale
1:250 @ A1

Drawing Number
HHO-7462-701-1601

Rev
D

PROPOSED REALIGNED SEWER PIPE

EXISTING SEWER PIPE TO BE REMOVED

PROPOSED PIPE TO BE CORE DRILLED INTO EXISTING CHAMBER - FLOAT LEVEL TO BE CONFIRMED PRIOR TO DRILLING

EXISTING PUMPSTATION CONTROL PANEL AND POWER CABLES TO BE LOCATED BY HAND PRIOR TO ANY EXCAVATION AND MUST BE PROTECTED

FOUL SEWER SETTING OUT
COORDINATE SYSTEM WGS84

NAME	Y	X	CL	IL1	IL2	NOTES
FSMH04	50137.436	3741275.307	2.636	1.43		TYPE C MANHOLE
FSMH05	50129.560	3741296.942	2.643	1.27		TYPE C MANHOLE
FSMH06	50123.135	3741302.092	2.555	1.21		TYPE C MANHOLE
FSMH07	50116.324	3741313.032	2.562	1.12		TYPE C MANHOLE
FSMH08	50115.461	3741342.372	2.198	0.92		TYPE C MANHOLE
FSMH09	50112.452	3741357.147	1.998	0.81		TYPE C MANHOLE
FSMH10	50087.976	3741362.099	2.779	0.64		TYPE B MANHOLE
FSMH11	50088.478	3741366.413	2.803	1.00	0.61	TYPE B MANHOLE
FSMH12	50089.979	3741378.010	2.812	TBC		TYPE A MANHOLE
FSMH13	50086.435	3741384.921	3.08	1.80	0.58	TYPE A MANHOLE
FSMH14	50089.680	3741397.688	3.51	0.72		TYPE B MANHOLE
FSMH15	50096.891	3741403.986	2.75	0.82		TYPE A MANHOLE

FOUL SEWER SETTING OUT
COORDINATE SYSTEM WGS84

NAME	Y	X	CL	IL1	IL2	NOTES
EX.MH06	50143.813	3741277.606	2.23	1.13		MANHOLE TO BE DEMOLISHED
EX.MH07	50134.515	3741274.256	2.60	1.51		
EX.MH08	50127.462	3741292.611	2.29	1.67		MANHOLE TO BE DEMOLISHED
EX.MH09	50134.923	3741301.343	2.13	0.94		MANHOLE TO BE DEMOLISHED
EX.MH10	50122.495	3741297.364	2.50	1.77		MANHOLE TO BE DEMOLISHED
EX.MH11	50118.898	3741320.926	2.05	0.84		MANHOLE TO BE DEMOLISHED
EX.MH12	50115.955	3741353.739	1.80	0.63		MANHOLE TO BE DEMOLISHED
EX.MH13	50092.084	3741377.593	2.77	0.83	-0.17	MANHOLE TO BE DEMOLISHED
EX.MH14	50089.459	3741365.036	2.76	1.12		MANHOLE TO BE DEMOLISHED
EX.MH15	50091.776	3741379.840	2.83	1.41		MANHOLE TO BE DEMOLISHED
EX.MH16	50067.084	3741403.333	3.97	3.22		
EX.MH17	50109.490	3741331.766	2.64	TBC		
EX.MH18	50109.775	3741334.141	2.65	1.79		
EX.RE	50096.891	3741403.986	2.75	0.82		RODDING EYE TO BE DEMOLISHED

NOTES

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE PROJECT SPECIFICATIONS AND THE FOLLOWING DRAWINGS:

HHO-7462-701-1001	GENERAL LAYOUT
HHO-7462-701-1011	PROPOSED PHASING OF CONSTRUCTION
HHO-7462-701-2501	SEWER DETAILS
HHO-7462-701-2511	SEWER PIPE LONGSECTION

2. ALL uPVC PIPES TO BE IN ACCORDANCE WITH SANS 4435.
3. ALL HDPE PIPES TO BE IN ACCORDANCE WITH SANS 4427.
4. CONSTRUCTION OF SEWER TO BE IN ACCORDANCE WITH SANS1200DB, SANS1200LD AND SANS 1200LB.

ISSUED FOR APPROVAL

LEGEND

—▶— S —	EXISTING SEWER PIPE
⌇▶⌇ S ⌇	EXISTING SEWER PIPE TO BE REMOVED
—▶— S —	PROPOSED SEWER PIPE PIPE
— —	EXISTING DEVELOPMENT BOUNDARY
▨	PROPOSED SEA WALL

DO NOT SCALE FROM THIS DRAWING. ONLY WRITTEN DIMENSIONS SHALL BE USED. DIMENSIONS AND LEVELS ARE TO BE VERIFIED ON SITE BY THE CONTRACTOR BEFORE COMMENCING ANY WORK. ANY DISCREPANCIES ARE TO BE REPORTED TO THE ENGINEER FOR VERIFICATION.

ON ORIGINAL

0 5 10 15 20 25 30 40 50 60mm

Rev	Date	Prepared by	Checked by	Approved by	Consultant Revision Details	Status
C	30/11/2021	MW	FdV	FdV	ISSUED FOR APPROVAL	IFA

Client

CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD
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Client Approval .... Date ....

Consultant Signature .... Date ....

Project Title
SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

Drawing Title
EXISTING AND PROPOSED FOUL SEWER
LAYOUT: SHEET 2 OF 2

hho
14th FLOOR, THE TOWERS SOUTH,
2 HERTZOG BOULEVARD,
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Client Number
SB_19379_C_402_C

Scale
1:250 @ A1

Drawing Number
HHO-7462-701-1602

Rev
C

ISSUED FOR APPROVAL

DO NOT SCALE FROM THIS DRAWING. ONLY WRITTEN DIMENSIONS SHALL BE USED. DIMENSIONS AND LEVELS ARE TO BE VERIFIED ON SITE BY THE CONTRACTOR BEFORE COMMENCING ANY WORK. ANY DISCREPANCIES ARE TO BE REPORTED TO THE ENGINEER FOR VERIFICATION.

ON ORIGINAL

0 5 10 15 20 25 30 40 50 60mm

Rev	Date	Prepared by	Checked by	Approved by	Consultant Revision Details	Status
B	9/12/2021	RE	MW	FdV	ISSUED FOR APPROVAL	IFA

Client

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STAD KAAPSTAD
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Client Approval .... Date ....

Consultant Signature .... Date ....

Project Title

SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

Drawing Title

EXISTING ELECTRICAL AND
TELECOMMUNICATIONS

hho
14th FLOOR, THE TOWERS SOUTH,
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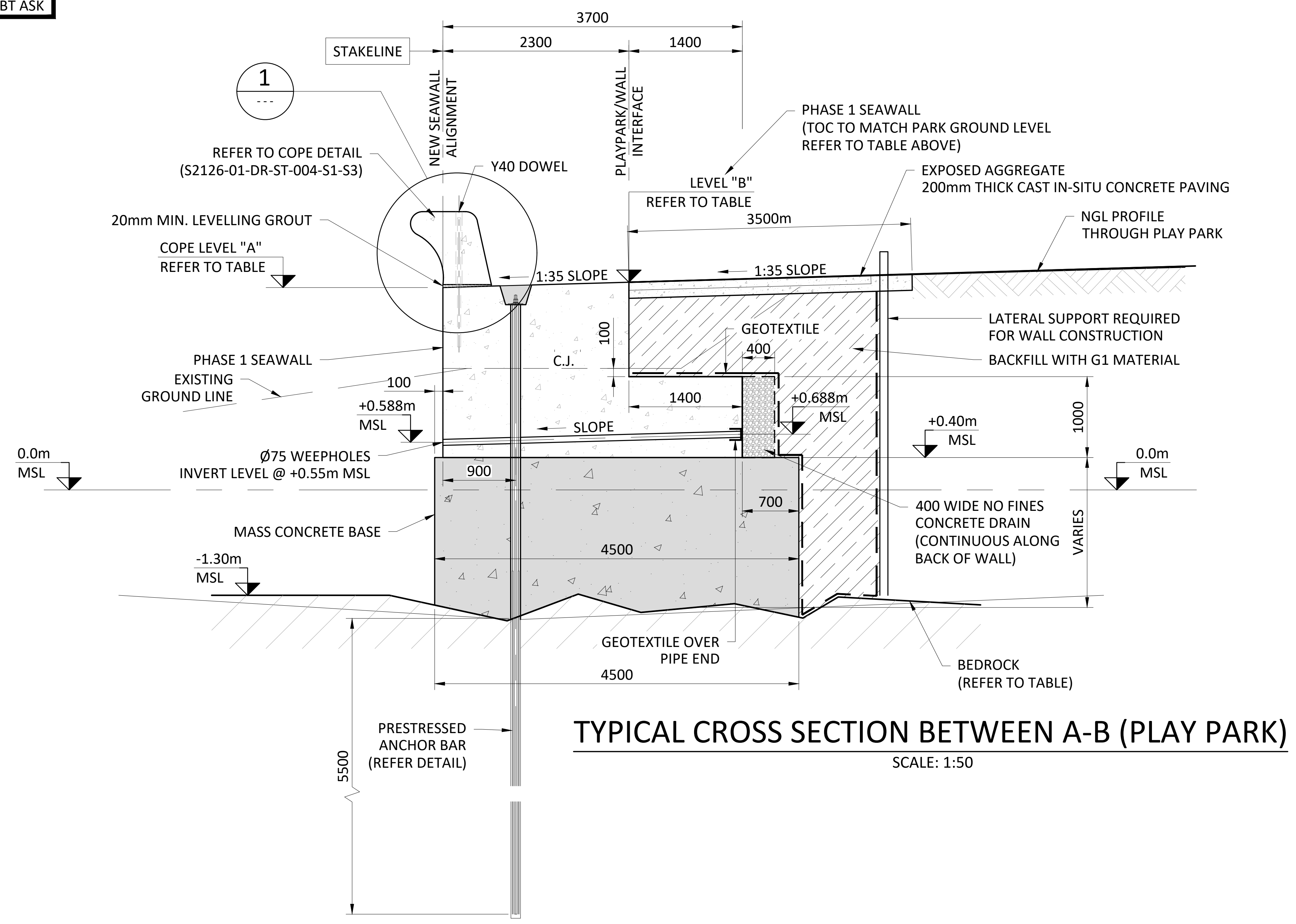
Client Number: SB_19379_C_421_B

Scale: 1:250 @ A1

Drawing Number: HHO-7457-701-1641

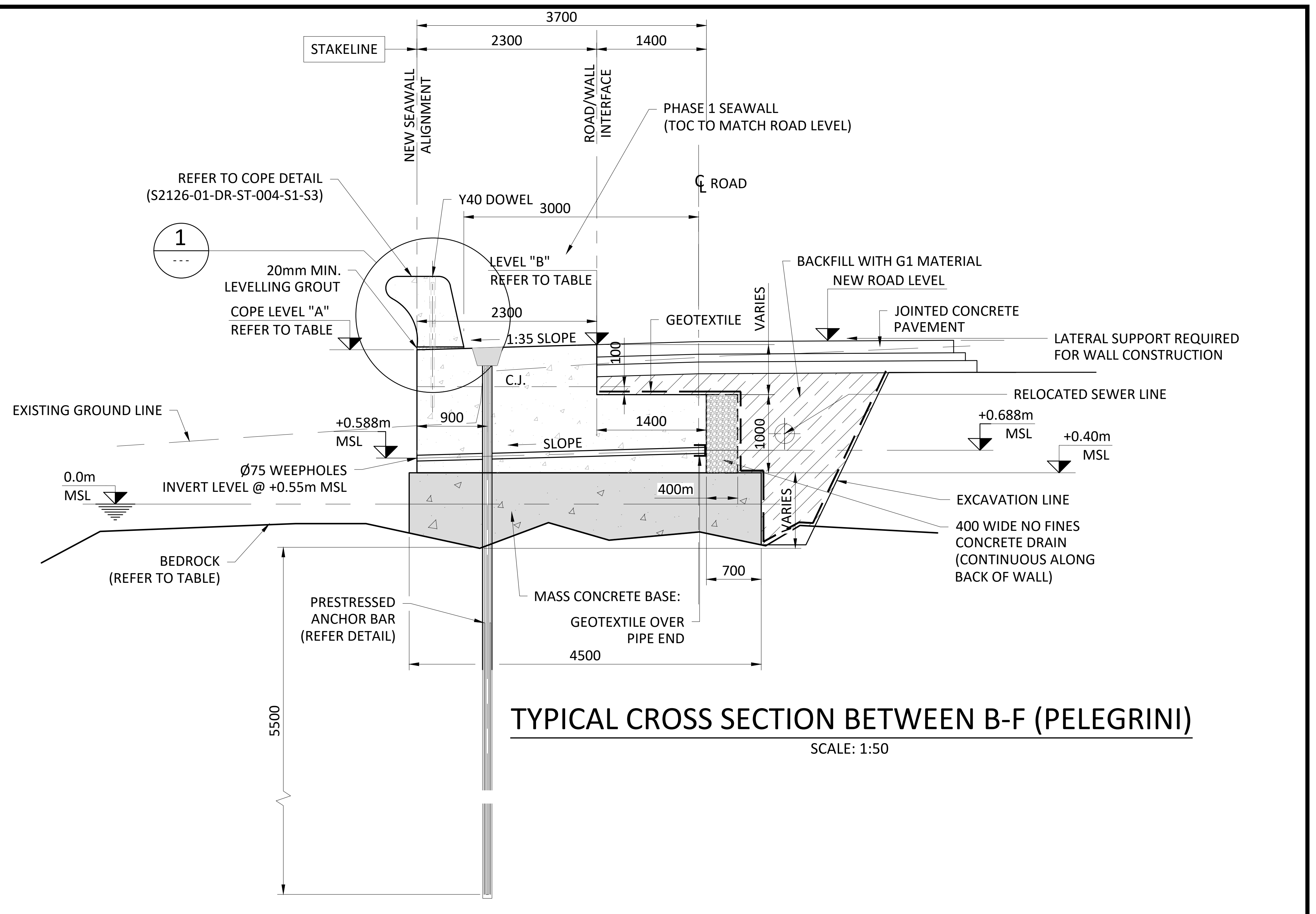
Rev: B

A1 DO NOT SCALE - IF IN DOUBT ASK



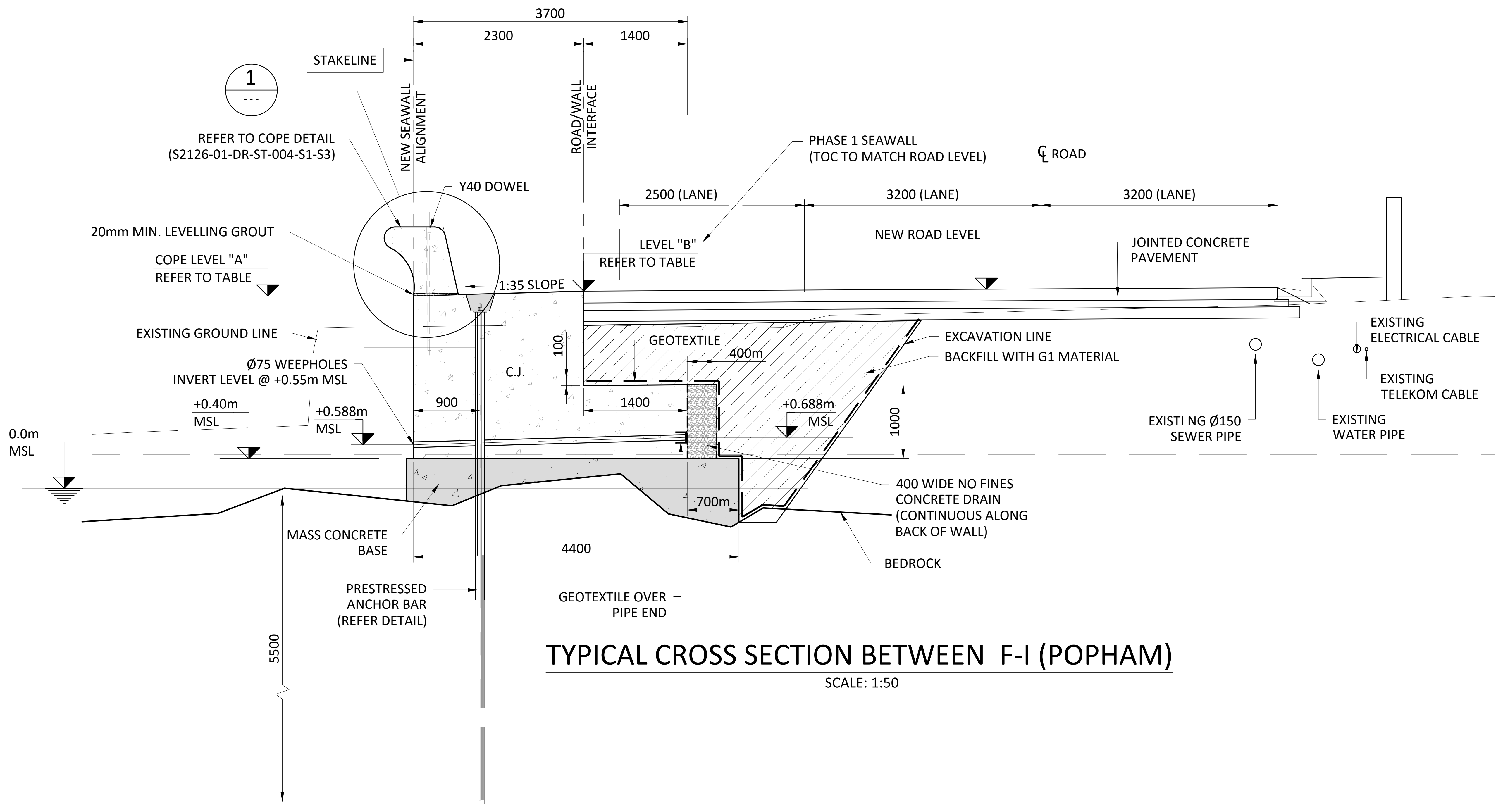
TYPICAL CROSS SECTION BETWEEN A-B (PLAY PARK)

SCALE: 1:50



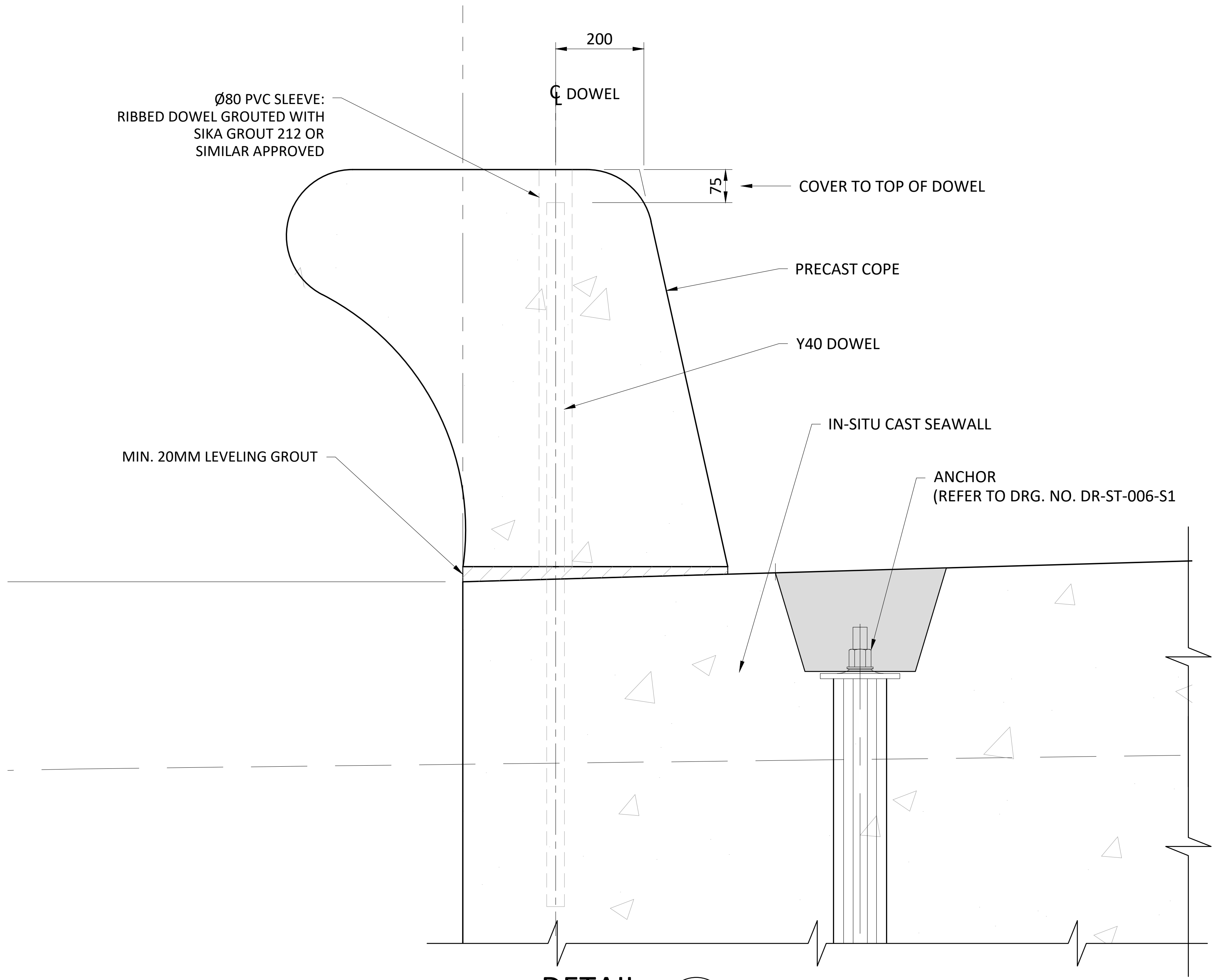
TYPICAL CROSS SECTION BETWEEN B-F (PELEGRINI)

SCALE: 1:50



TYPICAL CROSS SECTION BETWEEN F-I (POPHAM)

SCALE: 1:50



DETAIL 1

SCALE: 1:10

REFERENCE DRAWINGS

DRAWING NO.	REFERENCE
HHO-7462-701-1101	TYPICAL CROSS SECTIONS
S2126-01-DR-ST-006-S1	PRESTRESS ANCHOR DETAILS
S2126-01-DR-GA-000-S1	SEAWALL SPECIFICATION SHEET

NOTES:

NO.	DESCRIPTION	STATUS	BY	CHK'D	DATE
0	FOR APPROVAL	C	GDT	PES	01/12/2021

STATUS CODE
(A) PRELIMINARY
(B) PROPOSAL OR QUOTE
(C) FOR APPROVAL
(D) APPROVED FOR CONSTRUCTION
(E) SUPERSEDED
(F) AS BUILT

REVISIONS

	NAME	SIGNED	DATE
DRAWN BY	GdT		01/12/2021
DRAWING CHECKED	SMT		01/12/2021
DESIGN CHECKED	PES		01/12/2021
APPROVED	GPH		01/12/2021

FILE NAME: S2126-01-DR-ST-002-S1-R0.dwg

TIME STAMP: Tuesday, 07 December 2021 16:02:15

CLIENT

CITY OF CAPE TOWN
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PROJECT

SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

DRAWING TITLE

TYPICAL CROSS SECTIONS

SCALE: 1:50

CODE Nº CATEGORY DISCIPLINE CONSECUTIVE SHEET REVISION DRAFT

S2126-01 - DR - ST - 002 - S1 - R0

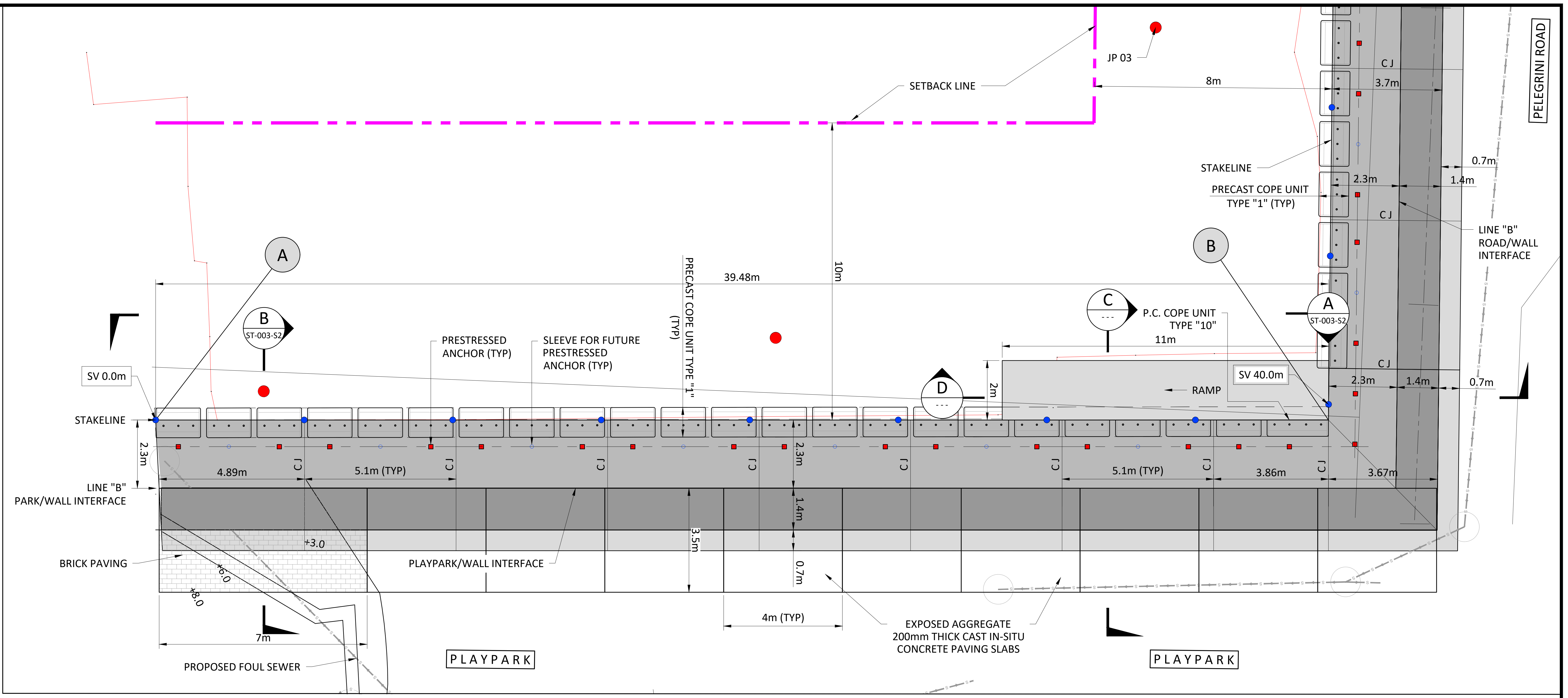
CLIENT NUMBER: SB_19379_M_102_B

X:\1. Projects\1. Current\Bloubergstrand (S2126) Small Bay Seawall\Drawings\2. Internal Working\2. Drawings in Progress\S2126-01-DR-ST-002-S1-R0.dwg

A1 | DO NOT SCALE - IF IN DOUBT ASK

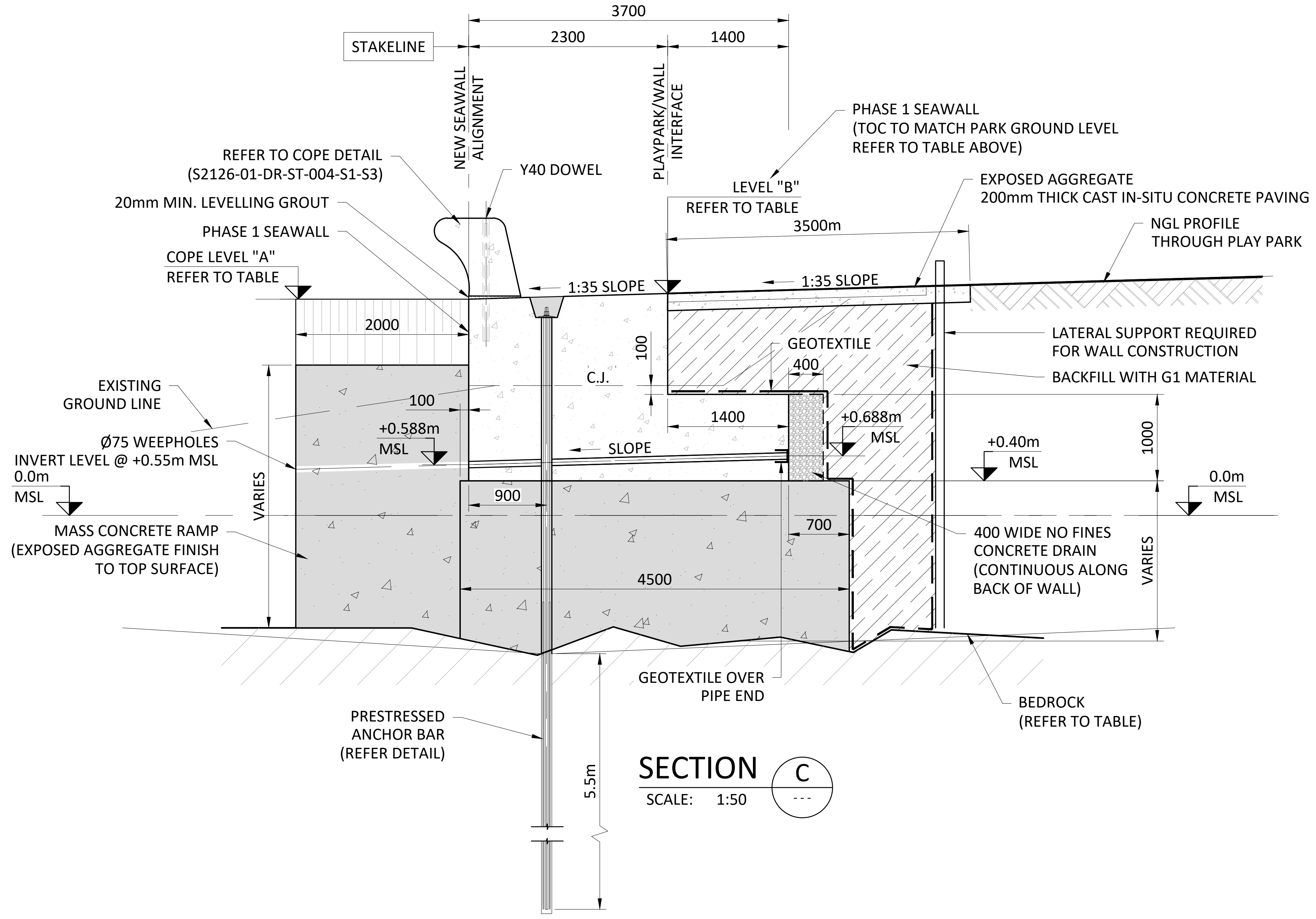
KEY PLAN

JET PROBE CO-ORDINATES AND LEVELS				
JET PROBE	X	Y	GROUND LEVEL (MSL)	ROCK LEVELS (MSL)
JP 1	+3 741 401.4600	+50 100.2700	+0.81	-1.30
JP2	+3 741 384.1500	+50 099.4700	+1.06	-1.12



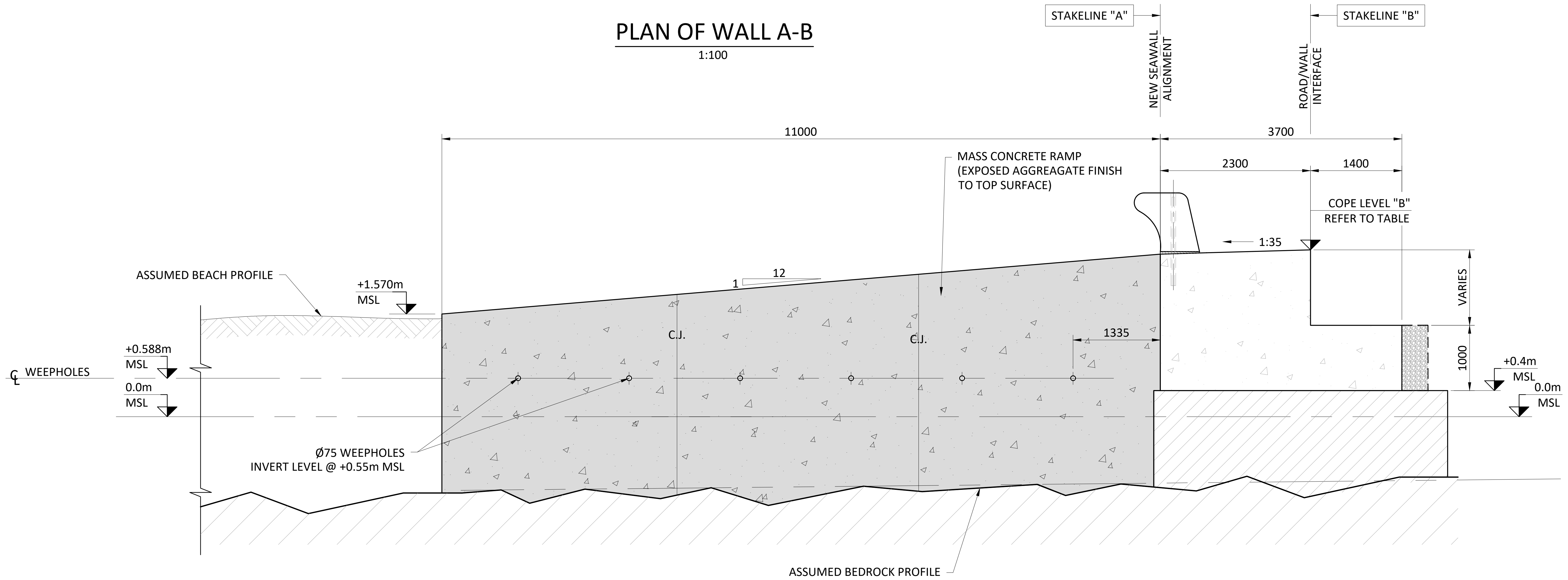
PLAN OF WALL A-B

1:100



SECTION C

SCALE: 1:50



SECTION D

SCALE: 1:50

(S.V. 45 STAKELINE "B")

REFERENCE DRAWINGS	
DRAWING NO.	REFERENCE
HHO-7462-701-1101	TYPICAL CROSS SECTIONS
S2126-01-DR-ST-006-S1	PRESTRESS ANCHOR DETAILS
S2126-01-DR-ST-002-S1	TYPICAL CROSS SECTIONS
S2126-01-DR-GA-000-S1	SEAWALL SPECIFICATION SHEET

NOTES:

1) REFER TO DRAWING No. GA-000-S1 FOR SEAWALL DRAWING NOTES
2) SEE DRAWING HHO-7462-701-1101 FOR ROAD LEVELS AND PAVEMENT LAYERS.

Note:
Coordinate System: Gauss Conform WGS84 LO 19
Heights relative to MSL

NO.	DESCRIPTION	STATUS	BY	CHK'D	DATE
0	FOR APPROVAL	C	GDT	PES	03/12/2021

STATUS CODE: (A) PRELIMINARY (B) PROPOSAL OR QUOTE (C) FOR APPROVAL (D) APPROVED FOR CONSTRUCTION (E) SUPERSEDED (F) AS BUILT

REVISIONS

	NAME	SIGNED	DATE
DRAWN BY	GdT		03/12/2021
DRAWING CHECKED	SMT		03/12/2021
DESIGN CHECKED	PES		03/12/2021
APPROVED	GPH		03/12/2021

FILE NAME: S2126-01-DR-ST-003-S1-S8-R0.dwg

TIME STAMP: Thursday, 09 December 2021 10:01:34

CLIENT

CITY OF CAPE TOWN
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PROJECT

SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

DRAWING TITLE

SEAWALL LAYOUT
PLAY PARK

SCALE: AS SHOWN

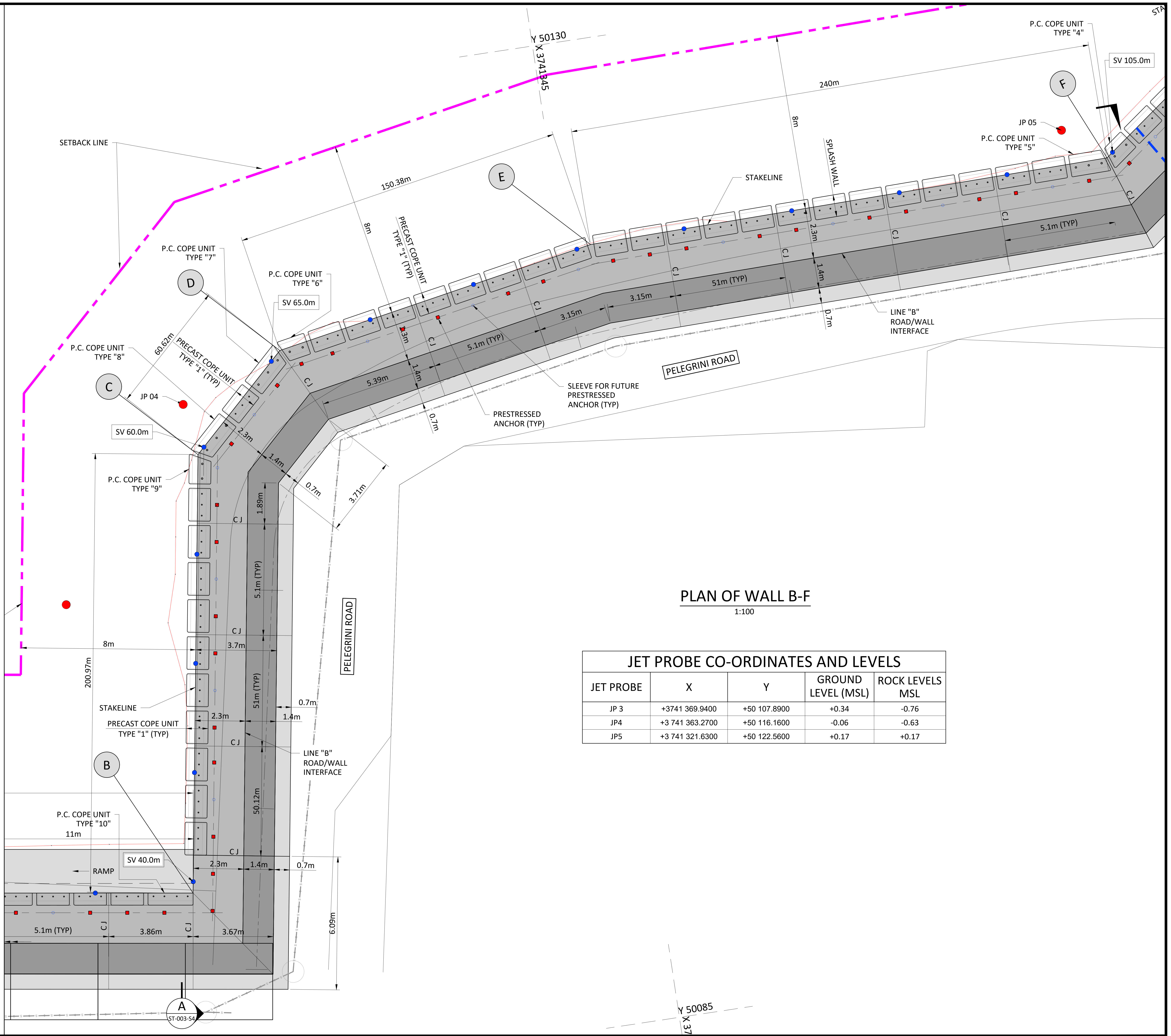
CODE	Nº	CATEGORY	DISCIPLINE	CONSECUTIVE	SHEET	REVISION	DRAFT
S2126	01	DR	ST	003	S1	R0	

CLIENT NUMBER: SB_19379_M_101_B

X:\1. Projects\1. Current\Bloubergstrand (S2126) Small Bay Seawall\Drawings\2. Internal Working\2. Drawings in Progress\S2126-01-DR-ST-003-S1-S8-R0.dwg

A1 | DO NOT SCALE - IF IN DOUBT ASK

KEY PLAN



PLAN OF WALL B-F

1:100

JET PROBE CO-ORDINATES AND LEVELS

JET PROBE	X	Y	GROUND LEVEL (MSL)	ROCK LEVELS MSL
JP 3	+3741 369.9400	+50 107.8900	+0.34	-0.76
JP4	+3 741 363.2700	+50 116.1600	-0.06	-0.63
JP5	+3 741 321.6300	+50 122.5600	+0.17	+0.17

X:\1. Projects\1. Current\Bloubergstrand (S2126) Small Bay Seawall\Drawings\2. Internal Working\2. Drawings In Progress\S2126-01-DR-ST-003-S1-S8-R0.dwg

REFERENCE DRAWINGS

DRAWING NO.	REFERENCE
HHO-7462-701-1101	TYPICAL CROSS SECTIONS
S2126-01-DR-ST-006-S1	PRESTRESS ANCHOR DETAILS
S2126-01-DR-ST-002-S1	TYPICAL CROSS SECTIONS
S2126-01-DR-GA-000-S1	SEAWALL SPECIFICATION SHEET

NOTES:

1) REFER TO DRAWING No. GA-000-S1 FOR THE SEAWALL DRAWING NOTES.
2) SEE DRAWING HHO-7462-701-1101 FOR ROAD LEVELS AND PAVEMENT LAYERS.

Note:
Coordinate System: Gauss Conform WGS84 LO 19
Heights relative to MSL

REVISIONS

NO.	DESCRIPTION	STATUS	BY	CHK'D	DATE
0	FOR APPROVAL	C	GDT	PES	03/12/2021

STATUS CODE
(A) PRELIMINARY
(B) PROPOSAL OR QUOTE
(C) FOR APPROVAL
(D) APPROVED FOR CONSTRUCTION
(E) SUPERSEDED
(F) AS BUILT

	NAME	SIGNED	DATE
DRAWN BY	GdT		03/12/2021
DRAWING CHECKED	SMT		03/12/2021
DESIGN CHECKED	PES		03/12/2021
APPROVED	GPH		03/12/2021

FILE NAME: S2126-01-DR-ST-003-S1-S8-R0.dwg

TIME STAMP: Thursday, 09 December 2021 10:01:34

CLIENT

CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD

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PROJECT

SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

DRAWING TITLE

SEAWALL LAYOUT
PELEGRINI ROAD

SCALE	CODE	N°	CATEGORY	DISCIPLINE	CONSECUTIVE	SHEET	REVISION	DRAFT
AS SHOWN	S2126-01		DR	ST	003	S3	R0	

CLIENT NUMBER: SB_19379_M_101_B

A1 | DO NOT SCALE - IF IN DOUBT ASK

PLAN OF WALL F-I

1:100

KEY PLAN

JET PROBE CO-ORDINATES AND LEVELS				
JET PROBE	X	Y	GROUND LEVEL (MSL)	ROCK LEVELS MSL
JP 5	+3 741 321.6300	+50 122.5600	+0.17	+0.17
JP 6	+3 741 280.3300	-50 145.4200	+0.52	-0.11
JP 7	+3 741 262.9600	+50 151.2400	+0.80	+0.80

REFERENCE DRAWINGS

DRAWING NO.	REFERENCE
HHO-7462-701-1101	TYPICAL CROSS SECTIONS
S2126-01-DR-ST-006-S1	PRESTRESS ANCHOR DETAILS
S2126-01-DR-ST-002-S1	TYPICAL CROSS SECTIONS
S2126-01-DR-GA-000-S1	SEAWALL SPECIFICATION SHEET

NOTES:

1) REFER TO DRAWING No. GA-000-S1 FOR THE SEAWALL DRAWING NOTES.
2) SEE DRAWING HHO-7462-701-1101 FOR ROAD LEVELS AND PAVEMENT LAYERS.

Note:
Coordinate System: Gauss Conform WGS84 LO 19
Heights relative to MSL

NO.	DESCRIPTION	STATUS	BY	CHK'D	DATE
0	FOR APPROVAL	C	GDT	PES	03/12/2021

STATUS CODE
(A) PRELIMINARY
(B) PROPOSAL OR QUOTE
(C) FOR APPROVAL
(D) APPROVED FOR CONSTRUCTION
(E) SUPERSEDED
(F) AS BUILT

REVISIONS

	NAME	SIGNED	DATE
DRAWN BY	GdT		03/12/2021
DRAWING CHECKED	SMT		03/12/2021
DESIGN CHECKED	PES		03/12/2021
APPROVED	GPH		03/12/2021
FILE NAME	S2126-01-DR-ST-003-S1-S8-R0.dwg		
TIME STAMP	Thursday, 09 December 2021 10:01:34		

CLIENT

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PROJECT

SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

DRAWING TITLE

SEAWALL LAYOUT
POPHAM ROAD

SCALE: AS SHOWN

CODE	Nº	CATEGORY	DISCIPLINE	CONSECUTIVE	SHEET	REVISION	DRAFT
S2126-01		DR	ST	003	S6	R0	

CLIENT NUMBER
SB_19379_M_101_B

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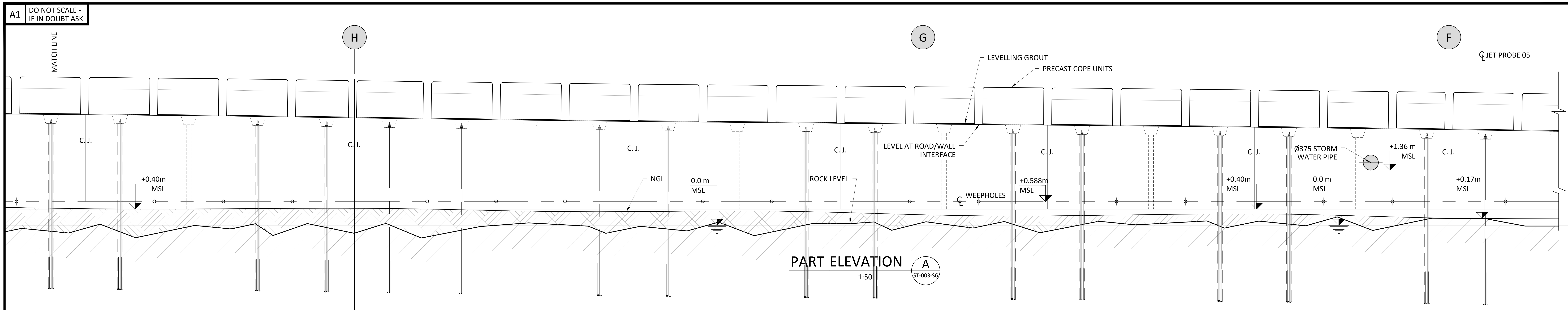

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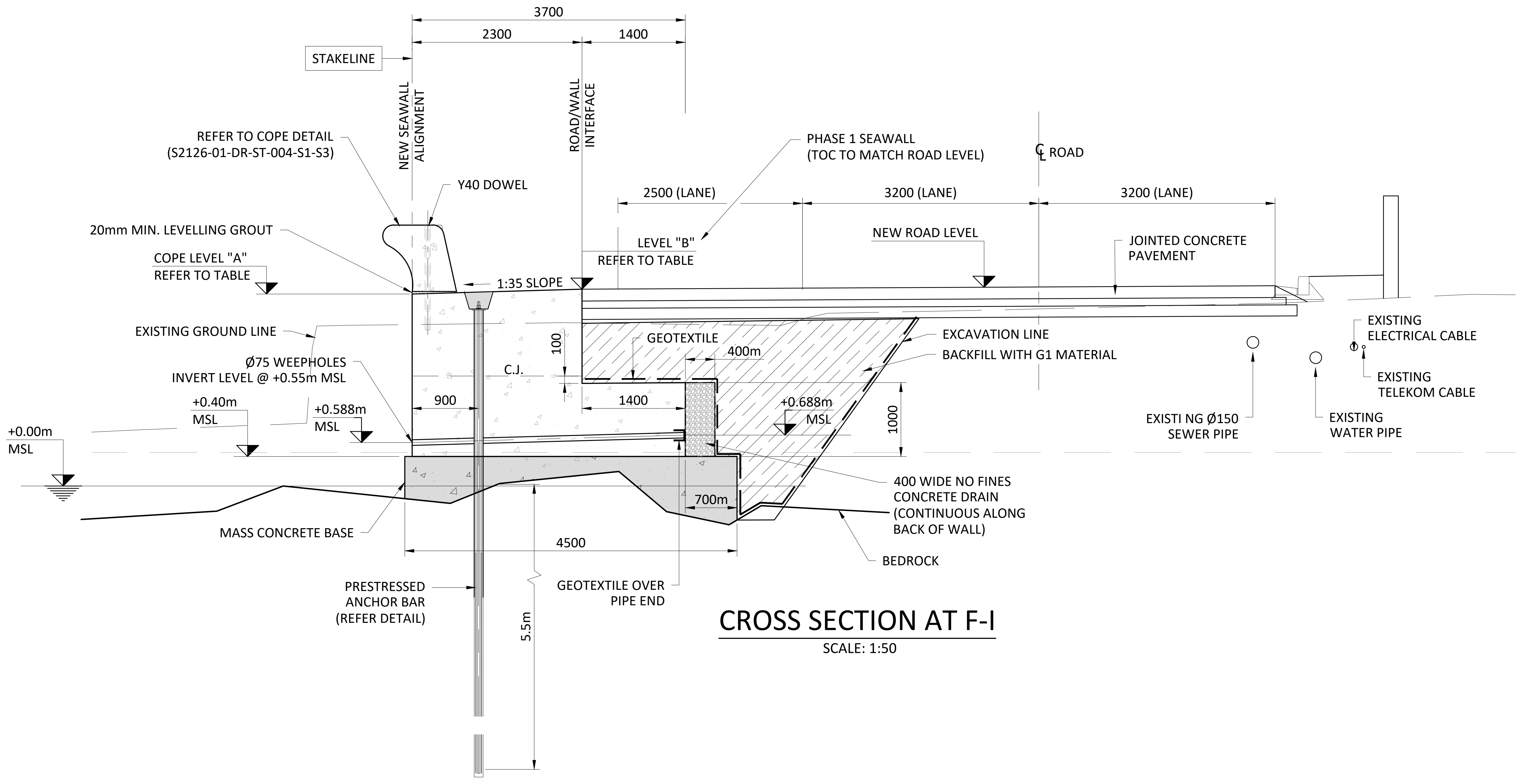
JET PROBE		
	NGL	+0.17
	ROCK	+0.17



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SCALE: 1:50

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REFERENCE DRAWINGS

DRAWING NO.	REFERENCE
HHO-7462-701-1101	TYPICAL CROSS SECTIONS
S2126-01-DR-ST-006-S1	PRESTRESS ANCHOR DETAILS
S2126-01-DR-ST-002-S1	TYPICAL CROSS SECTIONS
S2126-01-DR-GA-000-S1	SEAWALL SPECIFICATION SHEET

NOTES:

1) REFER TO DRAWING No. GA-000-S1 FOR THE SEAWALL DRAWING NOTES.
2) SEE DRAWING HHO-7462-701-1101 FOR ROAD LEVELS AND PAVEMENT LAYERS.

Note:
Coordinate System: Gauss Conform WGS84 LO 19
Heights relative to MSL

REVISIONS

NO.	DESCRIPTION	STATUS	BY	CHK'D	DATE
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STATUS CODE: (A) PRELIMINARY (B) PROPOSAL OR QUOTE (C) FOR APPROVAL (D) APPROVED FOR CONSTRUCTION (E) SUPERSEDED (F) AS BUILT

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DRAWING CHECKED	SMT		03/12/2021
DESIGN CHECKED	PES		03/12/2021
APPROVED	GPH		03/12/2021
FILE NAME	S2126-01-DR-ST-003-S1-S8-R0.dwg		
TIME STAMP	Tuesday, 07 December 2021 16:10:40		

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SMALL BAY SEAWALL UPGRADE
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AS SHOWN	S2126-01		DR	ST	003	S7	R0	

CLIENT NUMBER: SB_19379_M_101_B

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PART ELEVATION

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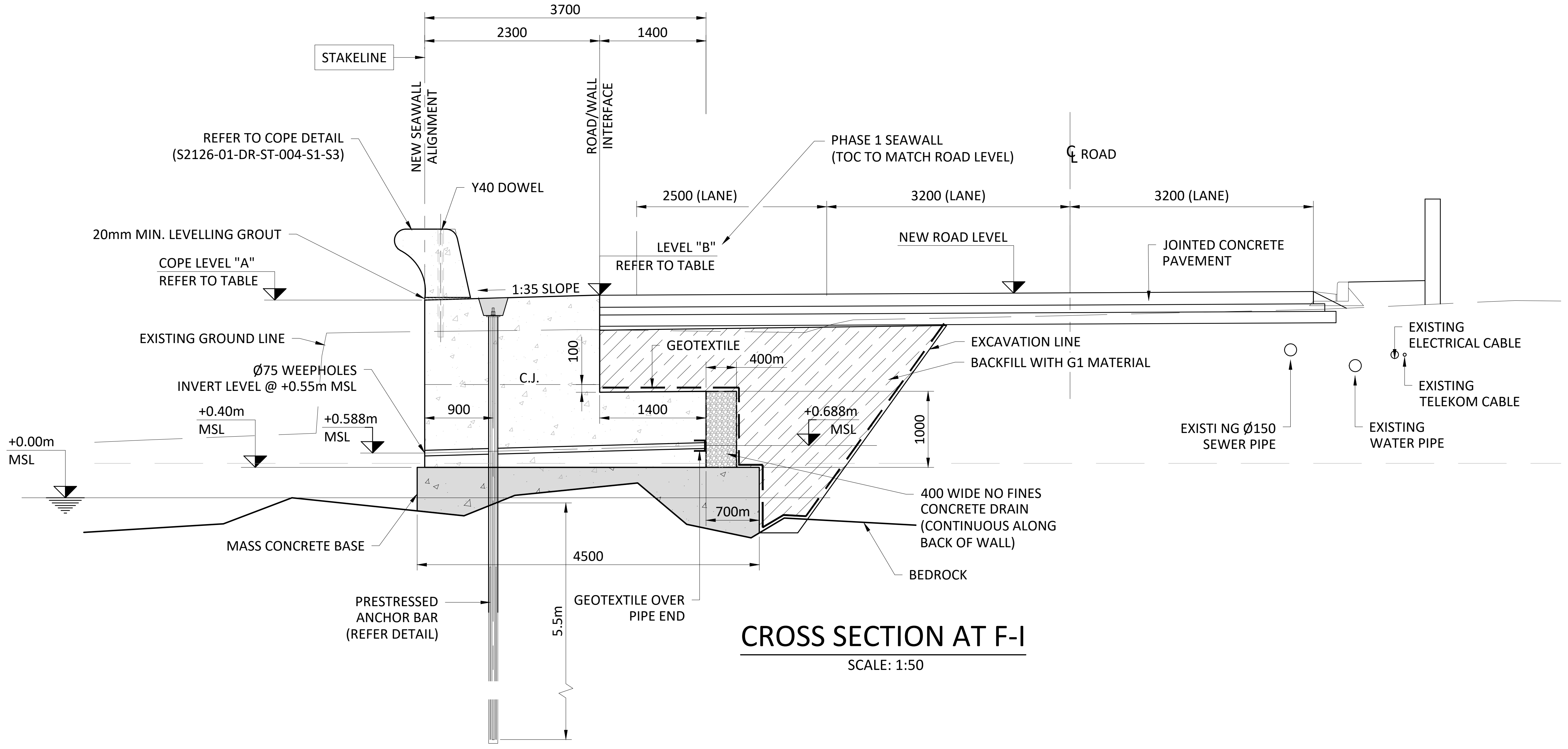
JET PROBE ROCK					+0.80							-0.11			



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Note:
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Heights relative to MSL

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S2126-01		DR	ST	003	S8	R0	

CLIENT NUMBER SB_19379_M_101_B

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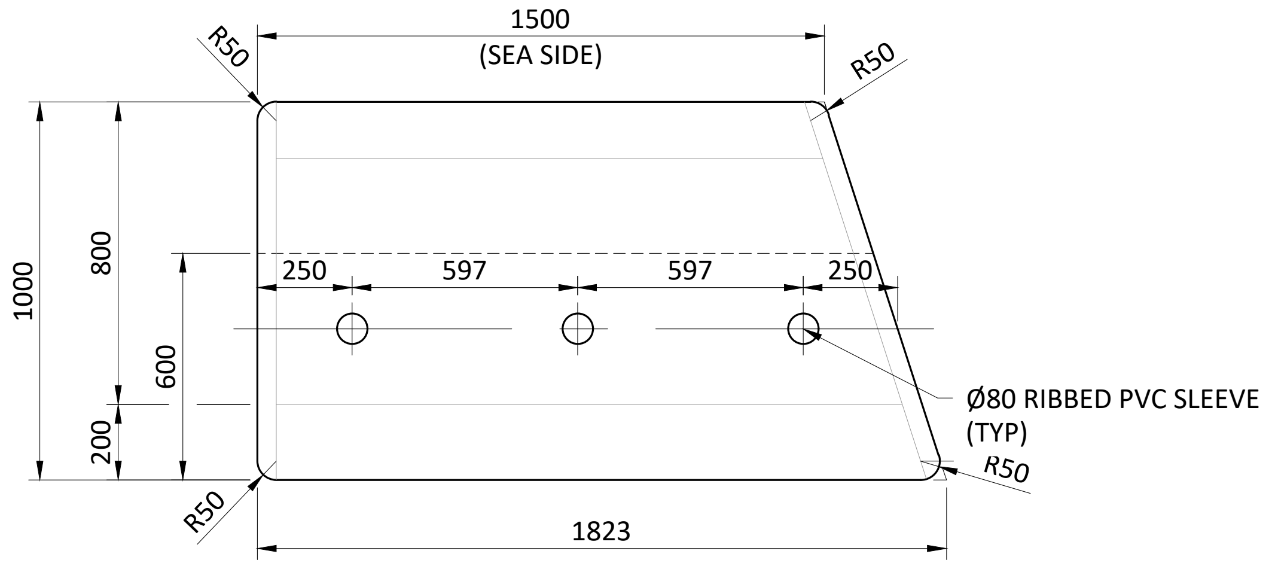

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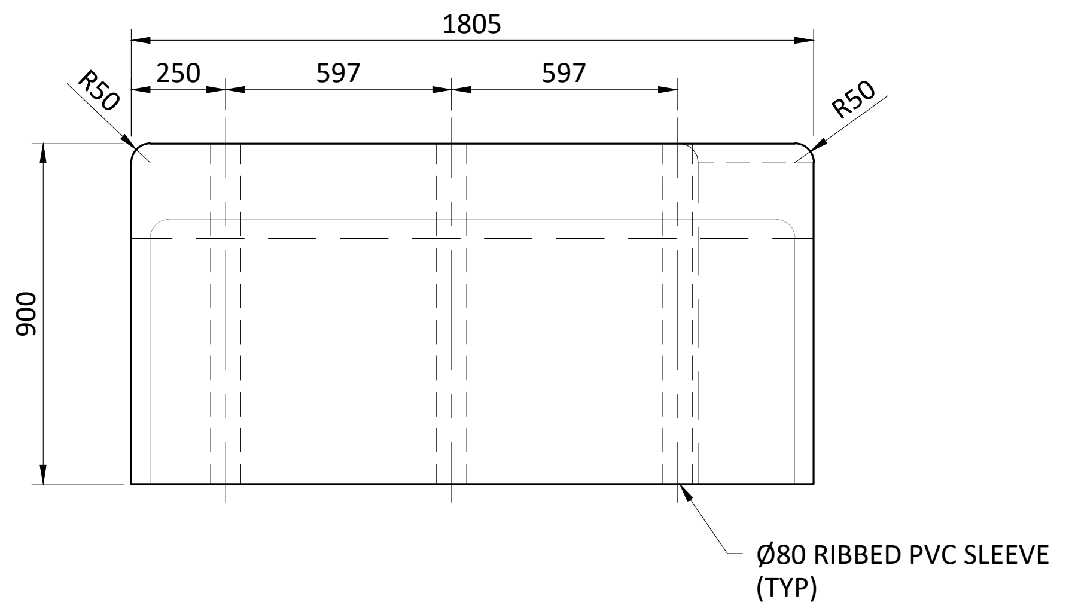

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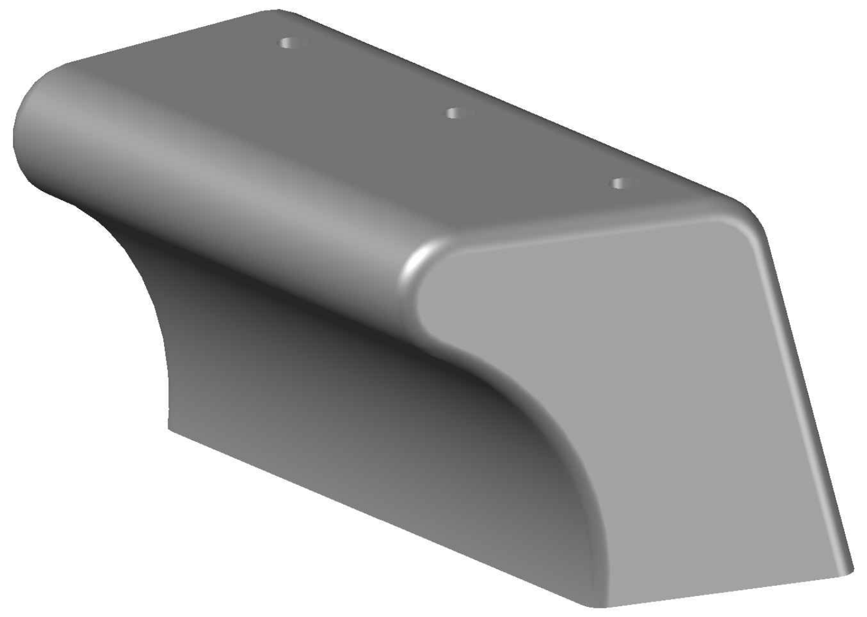

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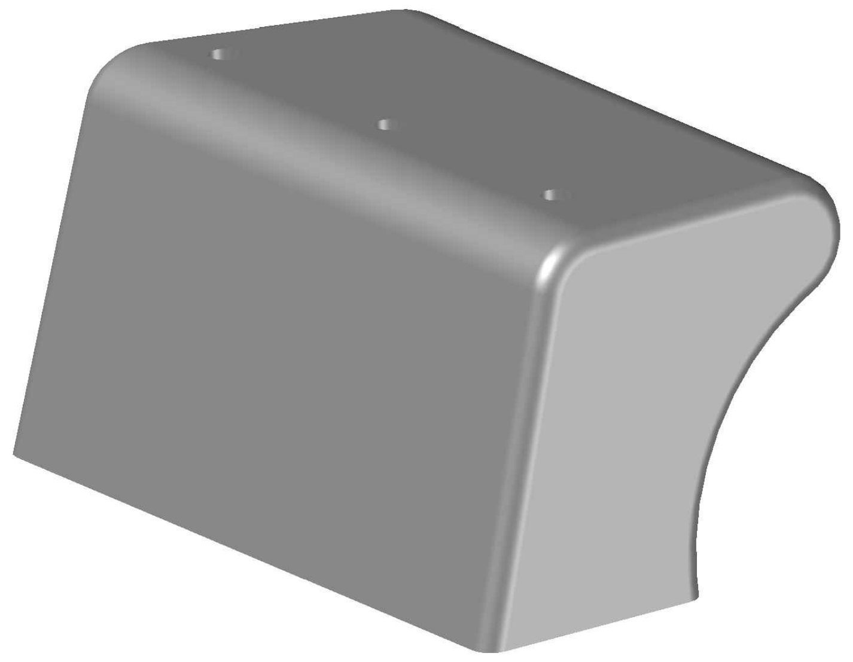

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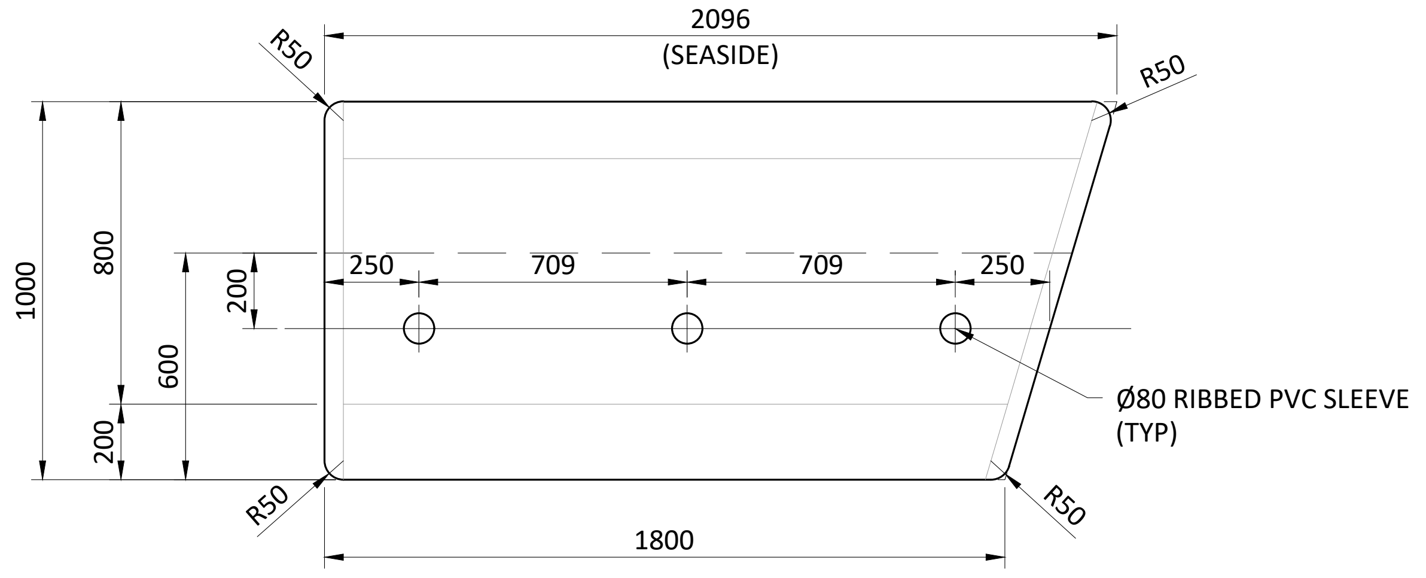

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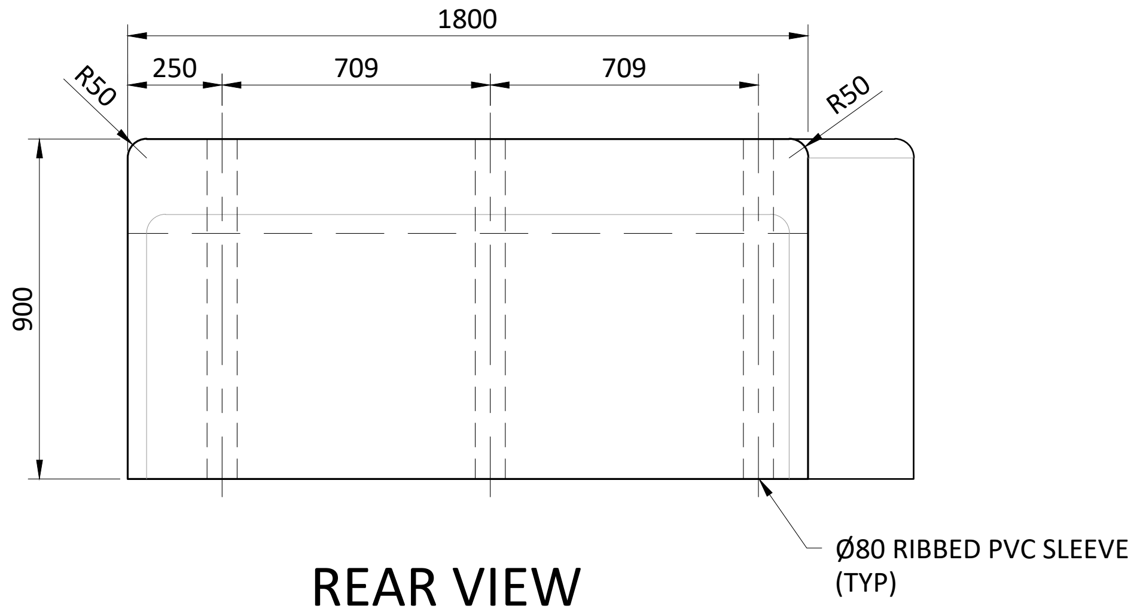

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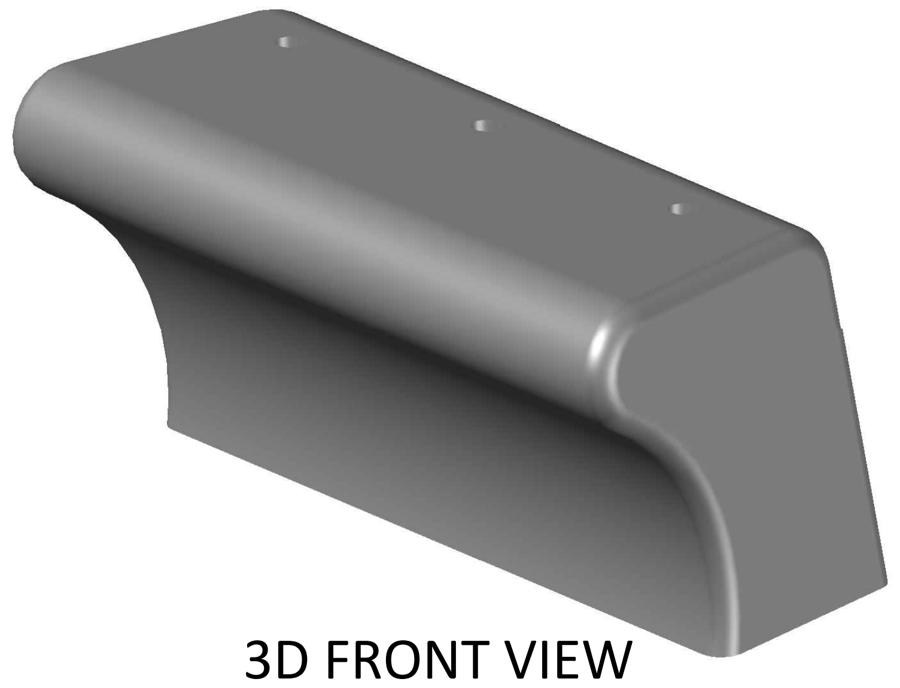

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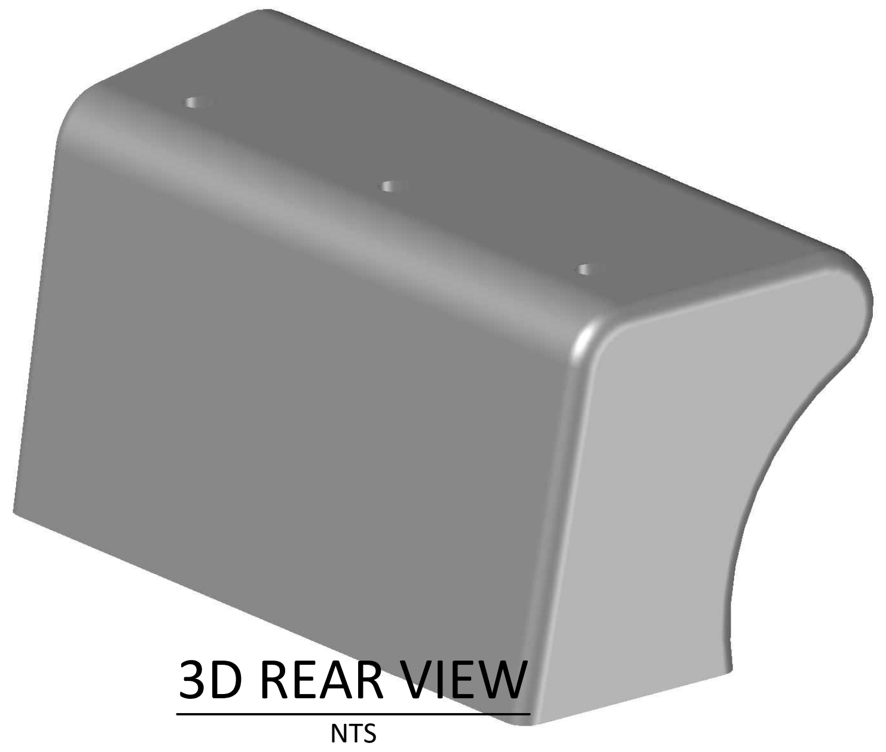

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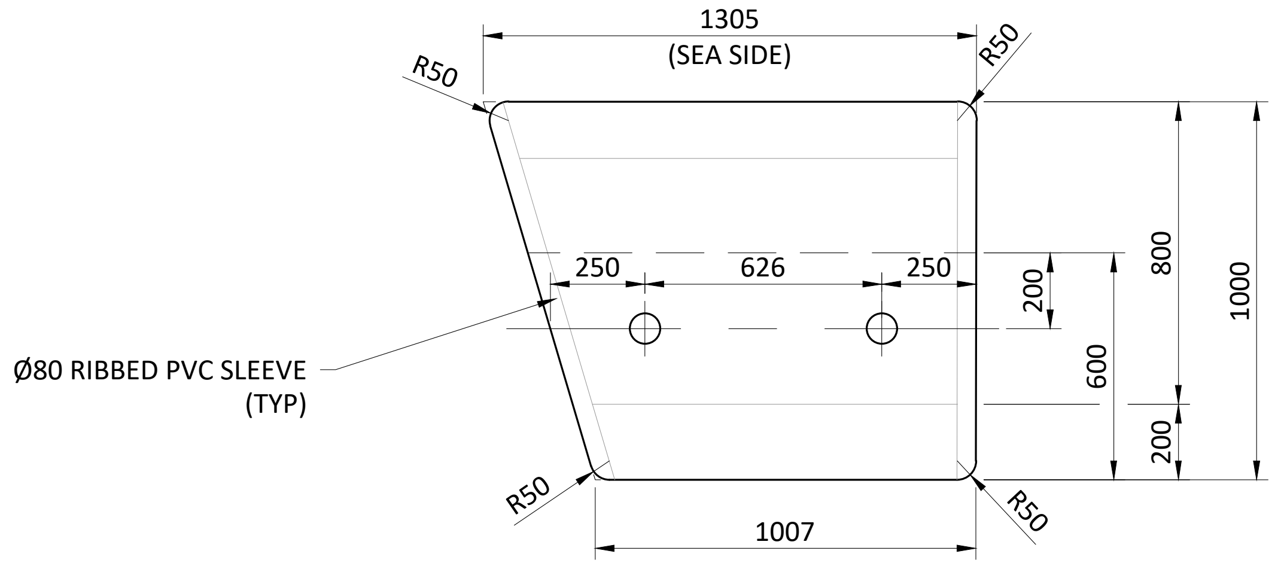

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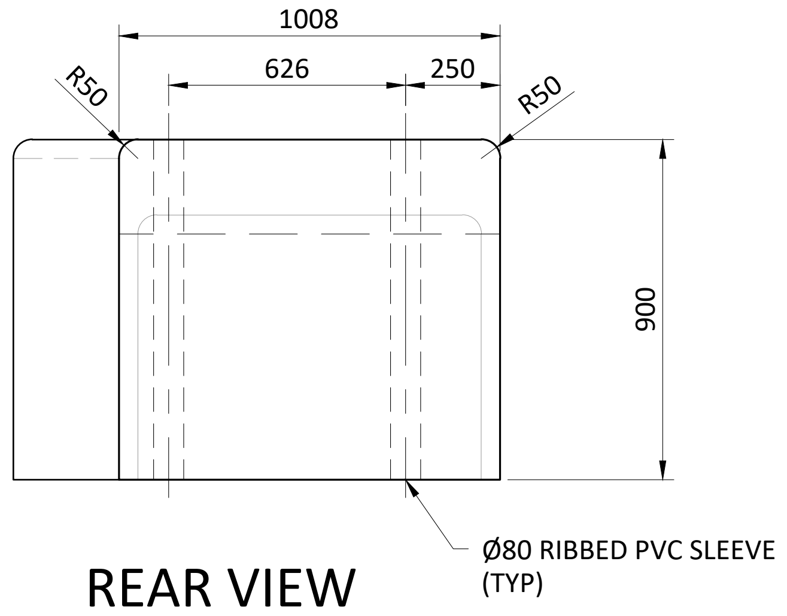

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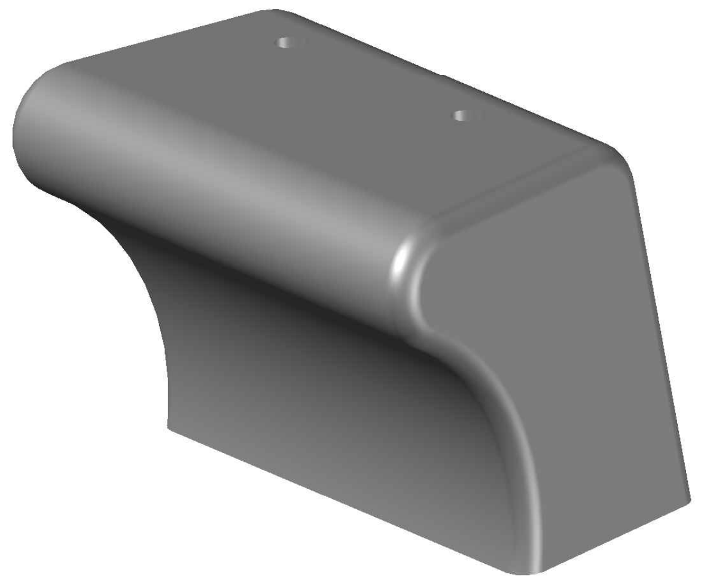

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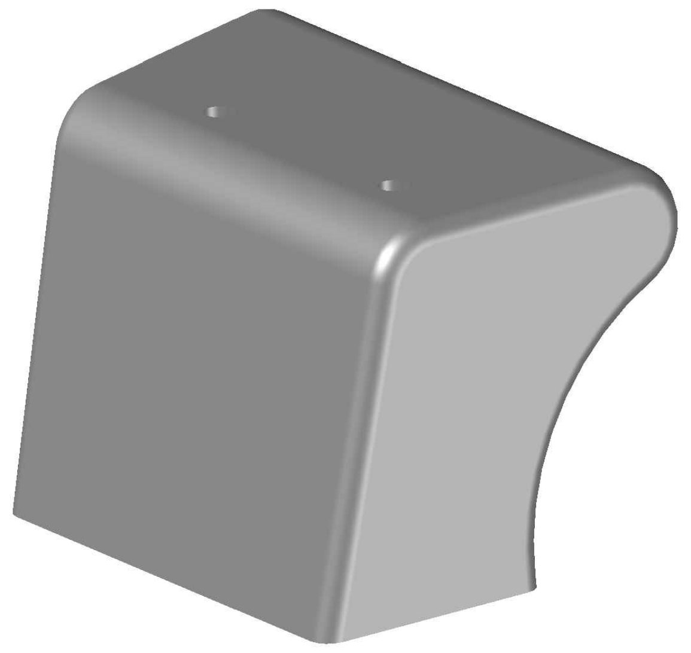

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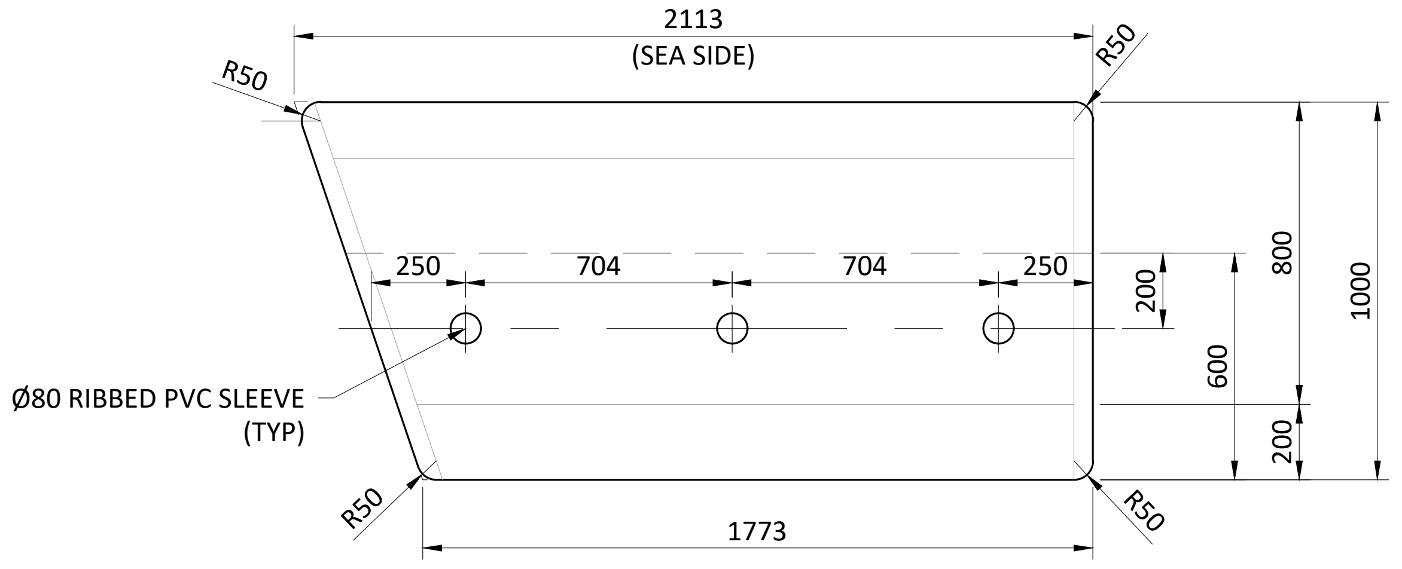

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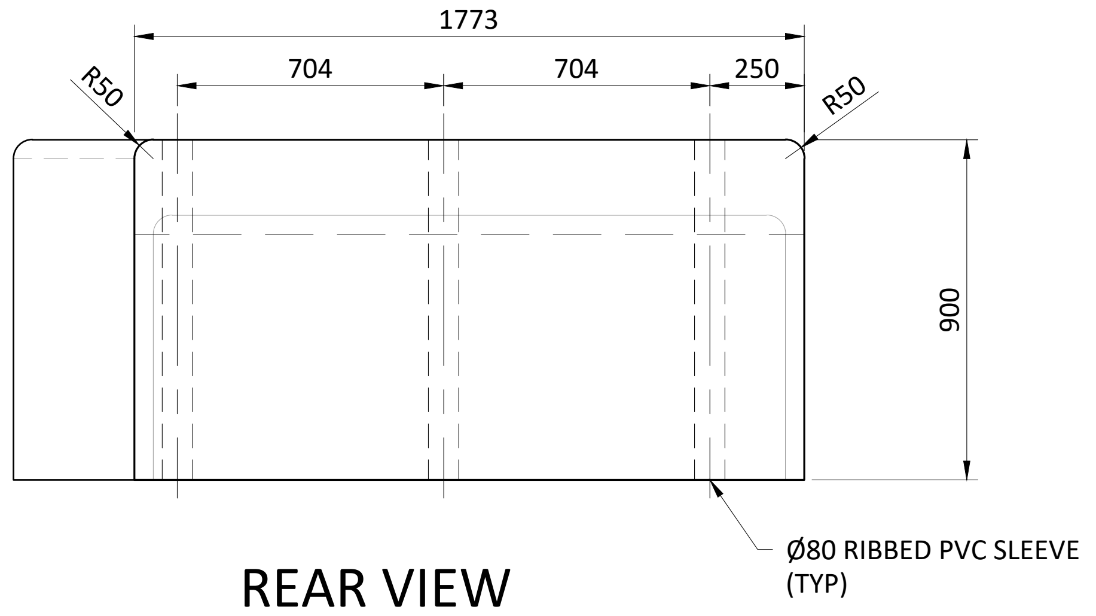

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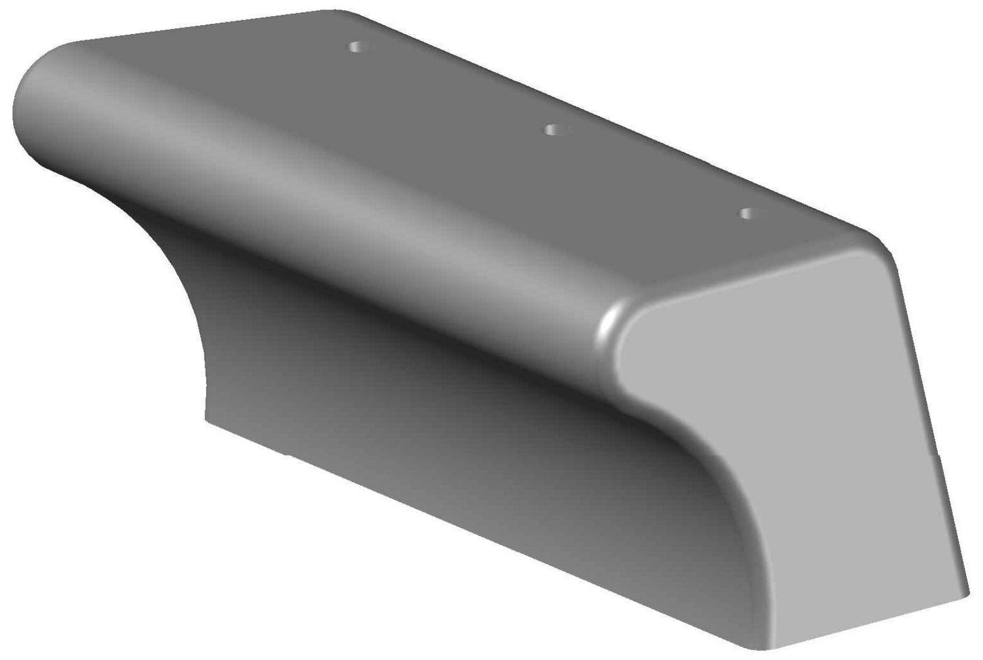

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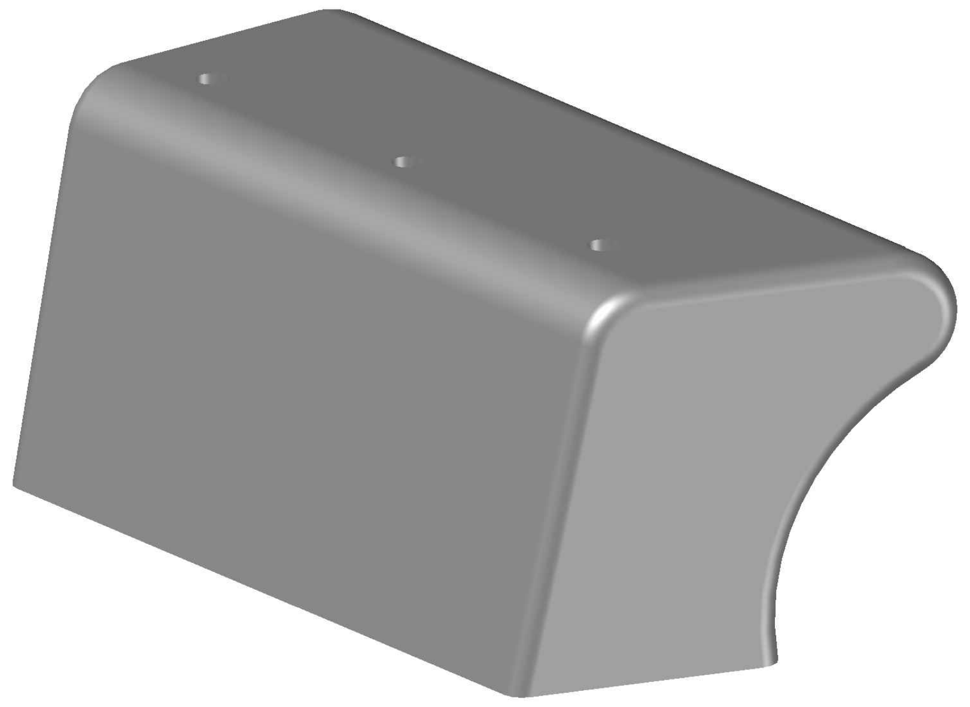

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REFERENCE DRAWINGS

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S2126-01-DR-ST-003-S7-R0	SEAWALL LAYOUT - PLAY POPHAM ROAD
S2126-01-DR-ST-003-S6-R0	SEAWALL LAYOUT - PLAY POPHAM ROAD
S2126-01-DR-ST-003-S5-R0	SEAWALL LAYOUT - PLAY PELEGRINI ROAD
S2126-01-DR-ST-003-S4-R0	SEAWALL LAYOUT - PLAY PELEGRINI ROAD
S2126-01-DR-ST-003-S3-R0	SEAWALL LAYOUT - PLAY PELEGRINI ROAD
S2126-01-DR-ST-003-S2-R0	SEAWALL LAYOUT - PLAY PARK
S2126-01-DR-ST-003-S1-R0	SEAWALL LAYOUT - PLAY PARK
S2126-01-DR-ST-002-S1	TYPICAL CROSS SECTIONS
S2126-01-DR-GA-000-S1	SEAWALL SPECIFICATION SHEET

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SMALL BAY SEAWALL UPGRADE
375C-2018-19: WP-013

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CLIENT NUMBER
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City of Cape Town

Small Bay Seawall Upgrade: Final Preliminary Design Report

SB_19379_TR_005_01
REP-7462-700-8005 Rev 01

ISO number: 9001: 2015

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DOCUMENT VERIFICATION

Title:
SMALL BAY SEAWALL UPGRADE: Preliminary Design Report

Client Project No: 19379_SB
Client Report No: SB_19379_TR_005_01
HHO Project No: 7462
HHO Report No: REP-7462-700-8005 Rev 01

Date: 30 August 2021
Report Status: Final

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Client:
City of Cape Town
44 Wale Street
Cape Town
8000

Author: Megan Woodward
Client Contact: Maria le Roux

Synopsis: Preliminary design report for the Small Bay seawall upgrade project

Quality Verification:

Rev	Date	Prepared by	Checked by	Approved by	Description	Status
A	19/07/2021	Megan Woodward	Fred de Villiers	Fred de Villiers		Draft
00	21/07/2021	Megan Woodward	Fred de Villiers	Fred de Villiers		Final
01	30/08/2021	Megan Woodward	Fred de Villiers	Fred de Villiers		Final
		Signature	Signature	Signature		

Design Specialist Contact Details

Role	Contact Person	E-mail	Telephone No.
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Project Manager	Megan Woodward	megan@hho.co.za	021 425 2870

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CLIENT APPROVAL

The undersigned agrees that the preliminary design report is acceptable and approved.

Maria le Roux	Project Manager	2 Sep 2021	MlRoux
Name	Designation	Date	Signature
Name	Designation	Date	Signature

1 EXECUTIVE SUMMARY

Introduction

The Bloubergstrand Beach node is an important recreational and tourism destination situated on the west coast of Cape Town. The existing concrete seawall and walkways, which extend from the play park in Small Bay to the De Mist parking area, have been undermined for a number of years. This has resulted in damage to the adjacent road infrastructure. Of specific concern is the exposure of the existing sewer line which is located within the existing sea wall along Popham and Pelegrini Streets. There is an elevated risk that the sewer line could fail, which could create an environmental emergency.

The study area comprises of the shoreline between the play park in Small Bay and the De Mist parking area which includes the seawall, roadway and associated infrastructure.

Site Conditions

Site investigations were undertaken during this phase of the project. These investigations included aerial, hydrographic and topographic surveys with geotechnical and materials investigations behind and in front of the existing seawall. The foreshore geotechnical investigations comprised of jet probe investigations to determine sand thickness and bedrock levels in front of the seawall. The investigations showed that the site is characterised by a shallow bedrock which deepens towards its Northern and Southern extents of the site to a maximum depth of approximately -1.30 m MSL. The results of the jet probe compared well with the trial pit investigations undertaken in previous investigations. An underground services detection survey was also completed which confirmed and detected the location and depth of all existing services at the site. These investigations were undertaken to characterise the project and inform the various design processes documented in this report.

A coastal modelling study was undertaken to define the metocean conditions along the site. This information was used as input into the project decision making and seawall design. Water levels, waves, overtopping rates and resulting flooding were investigated for various scenarios. The results from the coastal modelling show that future still water levels and wave climate will still result in significant flooding of landside infrastructure even after a new seawall has been constructed.

Coastal Engineering Design

Development of the new seawall has been split into two phases. The requirement of the first phase is to provide reinstatement of and protection to the City’s landside infrastructure, in particular the edge wall, sewer line, roadway, and pedestrian access. The project brief, however, requires that the seawall foundation and base be designed so that the seawall can be raised to a final crest elevation of +4.5 m MSL in the future. This is required to provide further protection from wave action and overtopping. The seawall will not initially be built to the +4.5 m MSL level due to: budgetary constraints; the boundary that a raised crest level will create between the land and shoreline access; the obstruction of sea views; and the potential challenges and delays in getting the necessary authorisations.

The structural solution of the seawall comprises a cast in-situ, reinforced concrete wall which is anchored to the bedrock using galvanised rock anchors. A cast in-situ construction method has been adopted given the various site constraints such as site access, site storage, space availability and traffic accommodation. The seawall structure, therefore, differs from the mass gravity structure with precast concrete blocks that was previously proposed. The seawall has a design working life of 50 years and is designed for a 100-year storm event with an encounter probability of 39.60%. The stability analysis shows that landward wave forces govern the stability of the seawall.

Civil Engineering Design

In addition to the seawall, the design of Popham and Pelegrini streets was undertaken. Popham Street has suffered complete structural failure and has been closed to traffic, while Pelegrini Street has experienced some localised failures. A jointed concrete pavement road, with a minimum design life of 30 years, has been proposed for both roads. This pavement is best suited for this location due to its durability and low maintenance requirement. The pavement will be designed during the detailed design phase of the project. The vertical alignment of both roads has made allowance for overland drainage and where possible, the roads have been raised to help reduce wave overtopping volumes.

The existing foul sewer pipe along Popham and Pelegrini Streets requires realignment, as well as an upgrade from DN150 to DN200. The proposed pipe is a SDR34 uPVC pipe which runs in the north bound lane of Popham Street, and along Pelegrini Street behind the seawall before tying into the pump station at the play park. A second existing foul sewer pipe has been identified for a realignment to allow for the construction of the Small Bay play park seawall. The DN200 HDPE pipe will be offset from the wall. The construction coordination of the foul sewer will need to be carefully controlled, making use of over pumping to prevent any disruptions to service or a sewage spill.

Construction Details

The constructability of the works has been considered in the preliminary design. Various site constraints such as site access, space, residential infrastructure and constructing within exposed tidal zone conditions have been considered. A cast in-situ methodology has been adopted for the seawall construction which is different to that previously proposed. Additionally, an indicative construction sequence has been formulated along with an associated construction program. It is estimated that the construction of the works will have a duration of at least 16 months.

Cost Estimate

A preliminary design cost estimate was completed, which split the works into two sections, as well as into 2 phases. The associated capital costs for all works, including preliminary and general, design development, contingencies and escalation is as follows:

- Phase 1 from the play park to Ferguson Road – R 45 493 791
- Phase 1 De Mist parking area – R 29 374 896
- Phase 2 future works – R 19 476 257 (excluding escalation)

Based on the available budget, it is unlikely that the De Mist parking work will be feasible unless additional budget is secured.

2 INTRODUCTION

2.1 Background

The Bloubergstrand Beach node is an important recreational and tourism destination situated on the west coast of Cape Town. The existing concrete seawall and walkways, which extend from the play park in Small Bay to the De Mist parking area, have been undermined for a number of years. This has resulted in damage to the adjacent road infrastructure. Of specific concern is the exposure of the existing sewer line which is located within the existing sea wall along Popham and Pelegrini Streets. There is an elevated risk that the sewer line could fail, which could create an environmental emergency. The project is therefore most urgent.

Contract 375C/2018/19 was approved by the City of Cape Town (CoCT) as a transversal panel appointment contract over a period of 7 years. The Environmental Management Branch has obtained due approval to appoint the required professional team members available through this contract for the Small Bay rehabilitation project. This includes the detailed design, documentation, construction and close-out phases of the project.

2.2 Study Area

The study area comprises of the coastline between the play park in Small Bay and the De Mist parking area. This includes the seawall, roadway and associated infrastructure. The extent of the Small Bay study area is shown in the **Figure 2.1** below.

Figure 2.1: Extent of Small Bay Seawall project

2.3 Project Description

The existing concrete seawall and walkways, which extend from the play park in Small Bay to the De Mist parking area, have been undermined for a number of years. This has resulted in damage to the adjacent road and underground services infrastructure. The following specific problems have been identified (and are indicated in the figures which follow).

2.3.1 Seawall

The study area has several sections of seawall that require reconstruction:

- De Mist public parking lot – this section has been undermined causing severe settlement of the sidewalk and seawall structure. The seawall has been undermined and rotated seaward, away from the parking lot roadway as can be seen in **Figure 2.3**. The portion of the de Mist seawall alignment is illustrated in **Figure 2.2** as the red line.

- Popham Street seawall – significant undermining of the seawall has occurred which has caused the material beneath, and behind the seawall to be lost as can be seen in **Figure 2.4**. The structure itself shows minimal signs of distress or settlement, however visible cracking of the foul sewer manholes is evident. Additionally, the undermining of the seawall has damaged, and collapsed Popham Street's pavement layers and has left these open to continual water ingress and wave attack. The portion of the Popham seawall alignment is illustrated in **Figure 2.2** as the blue line.

- Pelegrini Street seawall - portions of the seawall show separation of the L-shaped coping from the roadway, eventually separating from the roadway completely (shown in **Figure 2.8**). This has left the underlying concrete wall and the concrete pavement slab exposed to the water ingress and wave action. The portion of Pelegrini seawall alignment is illustrated in **Figure 2.2** as the green line.

- Small Bay play park seawall – this section of seawall which forms the boundary between the beach and the play park has been severely undermined. This has recently resulted in a sink hole and damage to an adjacent foul sewer pipe, as shown in **Figure 2.9**. The portion of the play park seawall alignment is illustrated in **Figure 2.2** as the yellow line.

All seawall reconstruction must be founded on the shallow bedrock present on site. This is to ensure a robust foundation solution and prevent future undermining and retention of material behind the seawall.

In addition to the reconstruction of the seawall, the Client has requested the Consultant Team to investigate various seawall crest levels to reduce overtopping volumes over the seawall during storm conditions.

Figure 2.2: Sections along the existing seawall.

2.3.2 Roadworks

A portion of Popham Street, from its intersection with Pelegrini Street up until the speed bump located north of the Ferguson Street intersection has experienced structural failure and is in a very poor condition, as shown in **Figure 2.5**. This portion of the street has been closed to traffic and the adjacent sidewalk has also collapsed, shown in **Figure 2.6**. This road requires a full reconstruction of the pavement layerworks, and the proposed road levels also need to tie in with the raised seawall. Drainage of overtopping sea water needs to be considered, as well as any existing underground drainage networks.

Pelegrini Street has experienced localised failures which are visible in its concrete surface. However, the underlying layerworks are thought to be undermined or missing in much larger areas than what is visible on the surface. This will lead to further damage to the concrete road surface which will eventually collapse over time, if not attended to. Therefore, a full reconstruction of this road is also required for the entire concrete surface portion. The levels of the road are constrained by existing properties. However, the road will be raised wherever possible. This was a request from the Client.

2.3.3 Underground Services

An existing DN150 foul sewer pipe is located along the Popham seawall edge and below Pelegrini Street. This pipe is at a high risk of failure at several locations as follows:

- The Popham Street seawall has been undermined and the pipe has become exposed. It is therefore not protected from storm conditions or high tide events.
- The sewer manhole which is built in the Popham seawall is progressively breaking away from the wall which could lead to a sewer spill, as shown in **Figure 2.7**.
- The foul sewer which crosses the beach near Small Bay play park was exposed during a large storm event. There is an exposed pipe coupling which could fail should the pipe be further undermined.

A recently installed sewer which was installed behind the play park seawall is at risk of damage should the seawall be further undermined. A sink hole has already developed in the southern corner where a rodding eye is located. In addition, the construction of the new seawall in this location may put this pipe at risk of collapse.

Figure 2.3: Settlement of seawall and sidewalk along De Mist public parking

Figure 2.4: Undermined seawall along Popham Street

Figure 2.5: Popham Street structural failure

Figure 2.6: Popham Street sidewalk collapse

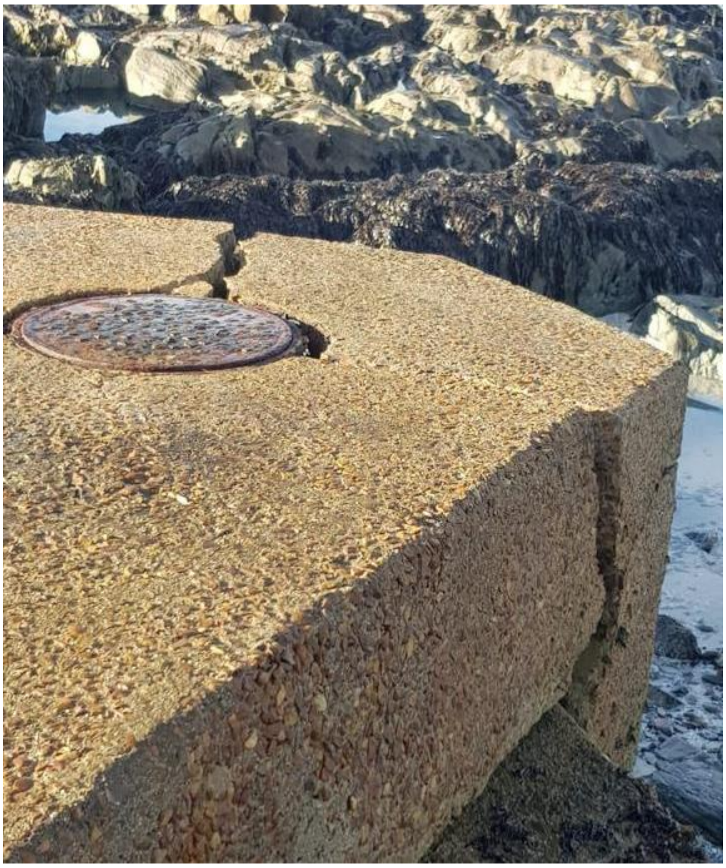


Figure 2.7: Sewer manhole breaking away from seawall.

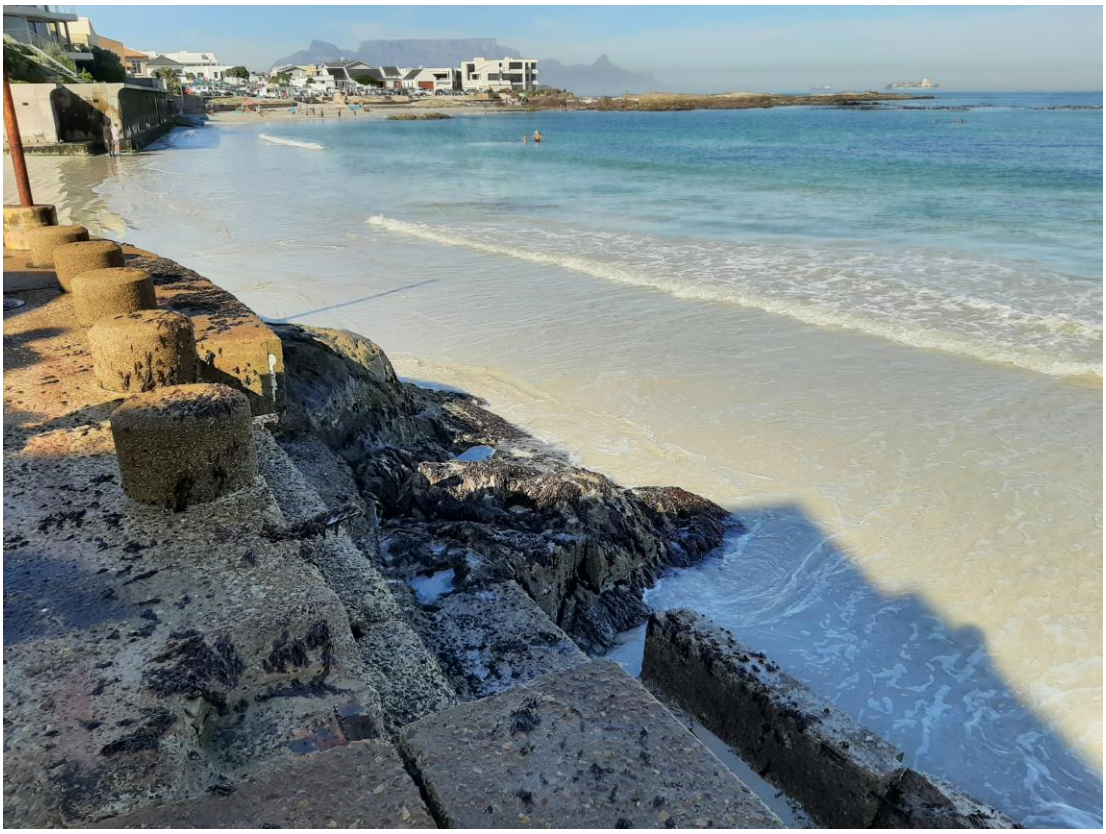


Figure 2.8: Localised failure of seawall along Pelegrini Street

Figure 2.9: Undermining at Play Park showing cavity beneath the seawall and damage behind it.

2.4 Project Objectives

The main goal of the project is to design and construct a new robust sea wall that would last at least 50 years to protect the adjacent City of Cape Town infrastructure and services, whilst preventing any long-term negative impact on the coastline (typically more than 100 years).

The rehabilitation of the Small Bay coastline shall comprise the following:

- Replace the existing seawall with a new seawall along the Small Bay coastline. The seawall design should allow for the seawall to eventually be raised with a final crest height of +4.5 m MSL.
- Re-route the existing foul sewer line landwards (±160 m long). It is currently encased in the existing sea wall.
- Reconstruct ±60 m of Popham Street between the Ferguson Street speed hump and the Pelegrini Street intersection.
- Reconstruct ±70 m of Pelegrini Street.
- Upgrade the De Mist Street parking area. A new parking layout and surface is required to align with the new seawall.

It is understood that the estimated construction cost for all these elements may not exceed the available budget. It may therefore be necessary to prioritise the various elements of the project, which would be implemented in order of importance. This will take place once a preliminary cost estimate is completed.

3 SITE CONDITIONS

3.1 Coastal Site Investigation

3.1.1 General

Site investigations were undertaken by Tritan Survey (Pty) Ltd and included:

- Aerial photogrammetry survey (conducted 28th – 30th April 2021).
- Hydrographic survey of the adjacent coastline and small bay area (conducted 1st – 3rd May 2021).
- Jet probe survey on the beach in front of the seawall (conducted 13th – 14th May 2021).

3.1.2 Aerial Survey

An aerial photogrammetry survey was carried out of the shoreline, beach and rock reefs/outcrops, the extent of which is shown in **Figure 3.1** (actual orthophoto produce by the survey). The survey was undertaken using a manned fixed wing aircraft. To maximise the area covered, the survey was carried out during calm sea and weather conditions, within 2 hours of low tide and within 2 days of a spring tide.

Figure 3.1 Aerial survey area

The result of the photogrammetric survey is further illustrated in **Annexure A**.

3.1.3 Hydrographic Surveys

A multibeam survey was carried out from a survey vessel, in the deeper water, offshore of the site and of the Big Bay area with track lines at 80 m c/c perpendicular to shoreline and with three perpendicular check lines. The survey along the coastline and in the shallower areas was carried out during high tide, to maximise the area covered and to overlap with the other surveys of the area. The area of the multibeam survey is shown in **Figure 3.2**.

Figure 3.2: Extent of multibeam survey area

A single beam hydrographic survey was carried out in the Small Bay area. Survey track lines were at a spacing of 40 m c/c perpendicular to the shoreline and included at least two perpendicular check lines. This equated to approximately 7 line km's. The survey was carried out in calm sea conditions, close to a spring high tide.

The results of the hydrographic surveys are further presented in **Annexure A**.

3.1.4 Combined models

The three datasets were combined into a single 3D TIN model. The data agreed well in areas of overlap and triangles were manually edited across large spans to best represent the expected topography. A one-meter grid of points was interpolated on the full tin surface, for ease of data imports into CAD packages.

The points and triangle model can be found in the associated CAD drawings. The combined data set showing the extent of the data overlap is illustrated in **Figure 3.3**. Additionally, **Figure 3.4.** illustrates the digital terrain model (DTM) developed using the survey techniques described above.

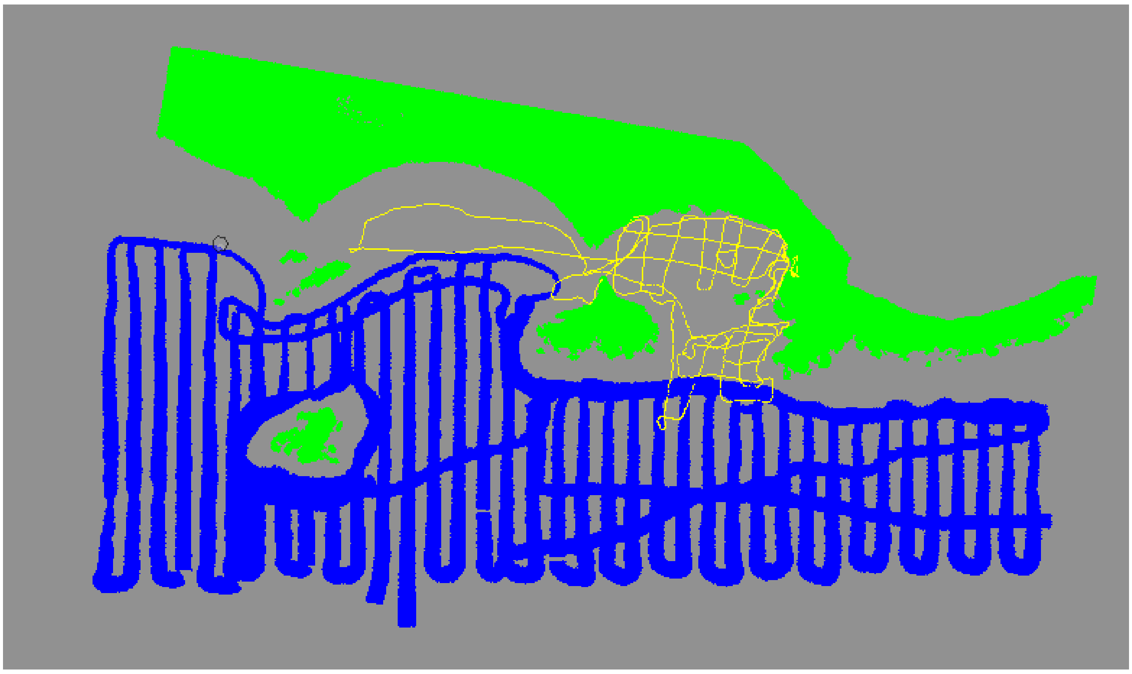


Figure 3.3: Combined data model with Aerial survey (green), Single beam (yellow) & Multibeam (blue).

Figure 3.4: Bloubergstrand Small Bay DTM.

3.1.5 Jet Probe Investigation

Jet probes (JP) were carried out to determine bedrock levels along the existing seawall and on the beach. JP's were conducted at the locations shown in **Figure 3.5** below.

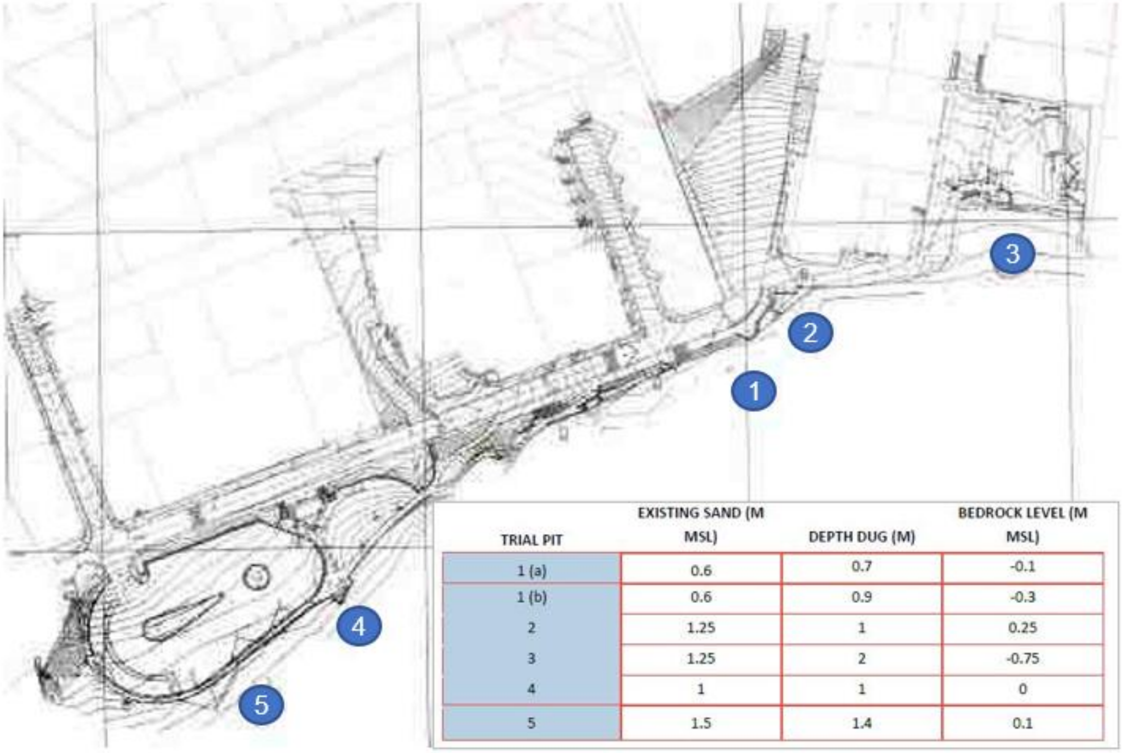
The depth to refusal and the nature of the material that each probe refuses on (e.g. hard rock, clay, shells, etc.) was recorded along with any hard layers or objects encountered during the probe. Material coming out of the probe wash was visually assessed and recorded.

Figure 3.5: Jet probe investigations

In general, the site was washed open at the time of the JP investigation with large areas of exposed bedrock clearly visible. Refusal was reached shortly after commencing each JP with minimal effort required to reach refusal due to the lack of overlying material.

Moderate to stiff resistance was encountered in the majority of the JP's due to the presence of dispersed pebbles however these were easily shifted by forcing and manoeuvring the probe until distinct, solid refusal was felt.

The results from the JP investigation showed good correlation with trial pit investigations conducted during previous project studies which indicated bedrock levels within the same order of magnitude. The position and results of these trial pits are illustrated in **Figure 3.6.**



TRIAL PIT	EXISTING SAND (M MSL)	DEPTH DUG (M)	BEDROCK LEVEL (M MSL)
1 (a)	0.6	0.7	-0.1
1 (b)	0.6	0.9	-0.3
2	1.25	1	0.25
3	1.25	2	-0.75
4	1	1	0
5	1.5	1.4	0.1

Figure 3.6: Previous trial pit investigations by KZR.

Appreciable probe depths were reached in the areas of the de Mist parking lot and the Small Bay play park. A summary of the JP results is presented in **Table 3.1**. The JP results typically indicate that the site is characterised by shallow bedrock, with these levels becoming increasingly deeper towards the northern and southern ends of the site boundary. Additionally, the bedrock level decreases in the seaward direction away from the seawall alignment. The lowest refusal level, along the seawall alignment, occurred at JP01 (highlighted in **Figure 3.5**) which indicated potential bedrock at -1.30 m MSL.

Table 3.1: JP investigation summary.

Name	Location	Comparative trial pit	Ground Level	Rock/Refusal Level	Sediment Thickness
JP01	PP	-	0.81	-1.30	2.11
JP02	PP	TP01	1.06	-1.12	2.18
JP03	Pele & PP		0.34	-0.76	1.11
JP04	Pele		-0.06	-0.63	0.58
JP05	Pele	TP02	0.17	0.17	0.00
JP06	Pop	TP01	0.52	-0.11	0.63
JP07	Pop	-	0.80	0.80	0.00
JP08	De Mist	TP04	1.11	0.00	1.11
JP09	De Mist	-	1.91	0.76	1.16
JP10	De Mist	TP05	1.19	0.14	1.05
JP11	De Mist	-	1.66	0.07	1.58
JP12	De Mist	-	2.17	0.28	1.89
JP13	De Mist	-	1.53	-0.30	1.83
JP14	De Mist*	-	-0.43	-0.79	0.36
JP15	De Mist*	-	-0.86	-1.24	0.38
JP16	De Mist*/Pop*	-	-1.22	-1.82	0.60
JP17	Pop*	-	-0.94	-1.45	0.51
JP18	Pop*	-	-1.31	-1.73	0.42
JP19	Pele*/PP*	-	-1.06	-1.39	0.33
JP20	PP*	-	-0.66	-1.60	0.94
JP21	PP*	-	-0.62	-0.62	0.00

Notes:
PP = Play Park
Pele = Pelegrini Street
Pop = Popham Street
De Mist = de Mist Parking Lot
* = JP taken within the bay away from seawall alignment

The positions and results from the JP investigation are further presented in **Annexure A**.

3.2 Coastal Modelling

3.2.1 General

PRDW was appointed by CoCT to undertake a coastal modelling study to define the metocean conditions at the site. This information was used as input into the project decision making and seawall design. The following inputs were developed:

- Design water levels

- Design waves, and
- Overtopping rates and resulting flooding

The coastal modelling considered the following:

- Three climate change scenarios were considered: 2024 (present day), 2049 (mid-way through 50-year design life) and 2074 (end of design life).
- Measured water levels at Cape Town were analysed to extract the extreme storm surge residuals, e.g. 1:1, 1:5, 1:100 and 1:500 year return periods. These were added to the predicted tides and sea level rise to obtain the design still water level.
- The deep water spectral wave climate offshore of Cape Town were extracted from the NCEP global wave hindcast model.
- The storm events were transformed to offshore of Small Bay using the MIKE Spectral Waves model and the results analysed to obtain the extreme nearshore wave climate.
- The sediment transport and associated variability in seabed level in front of the seawall was assessed based on jet probe measurements, along with previous sediment studies undertaken by PRDW in this area.
- The MIKE 3 Wave Model was used to model the wave transformation within Small Bay to determine the design waves and water levels at the seawall, as well as the overtopping of the seawall and the resulting flooding.
- A large number of cases comprising combinations of sea level rise, storm return period and wall crest level are possible. The cases to be modelled were confirmed with the CoCT to ensure optimum use of the available time and budget.
- The cases which have been considered are presented in **Table 3.2**. The cases which are highlighted in green have been considered for the preliminary design. It is recommended the cases which are not highlighted be considered during the detailed design phase.

The detailed results of the coastal modelling are further presented in **Annexure B.**

Table 3.2: Combination of cases.

Case no.	Output	Crest level [m MSL]	SLR [year]	Return period [years]	Notes
1	Overtopping	Existing	2024	1	Wall and road level as per HHO design.
2	Overtopping	Existing	2024	5	
3	Overtopping	Existing	2024	100	
4	Overtopping	Existing + 1 m	2024	1	

5	Overtopping	Existing + 1 m	2024	5	1.0 m high x 0.5 m wide x 2.0 m long cope units with 0.2 m gap, modelled as 1.0 m high x 1 m wide x 20 m long cope units with 2 m gap.
6	Overtopping	Existing + 1 m	2024	100	
7	Overtopping	Existing + 1 m	2049	1	Halfway through design life (25 years).
8	Overtopping	Existing + 1 m	2049	5	
9	Overtopping	Existing + 1 m	2049	100	
10	Overtopping	Existing + 1 m	2074	1	End of design life (50 years).
11	Overtopping	Existing + 1 m	2074	5	
12	Overtopping	Existing + 1 m	2074	100	
13	Overtopping	4.5	2074	1	Raised crest level at the end of design life (50 years).
14	Overtopping	4.5	2074	5	
15	Overtopping	4.5	2074	100	
16	Wave forces	n/a	2074	100	Seawall modelled as a wave absorbing structure.
17	Wave forces	n/a	2074	500	Accidental case (TBC in detailed design).

3.2.2 Extreme water levels

The extreme high-water levels for 2024, 2049 and 2074 for the upper 95th percentile storm surge residual are provided below.

Table 3.3: Extreme water levels.

Return Period [years]	Tide Level (90th percentile high tide) [m MSL]	Sea Level Rise [m]			Storm Surge Residual [m]	Water Level [m MSL]		
		2024	2049	2074		2024	2049	2074
0.1	0.93	0.09	0.26	0.52	0.39	1.41	1.58	1.84
0.5	0.93	0.09	0.26	0.52	0.42	1.44	1.61	1.87
1	0.93	0.09	0.26	0.52	0.43	1.45	1.62	1.88
5	0.93	0.09	0.26	0.52	0.58	1.60	1.77	2.03
10	0.93	0.09	0.26	0.52	0.68	1.70	1.87	2.13
50	0.93	0.09	0.26	0.52	1.02	2.04	2.21	2.47
100	0.93	0.09	0.26	0.52	1.22	2.24	2.41	2.67
500	0.93	0.09	0.26	0.52	1.83	2.85	3.02	3.28

3.2.3 Waves

The modelled upper 95% confidence extreme significant wave height, period, direction and directional spreading at a location offshore of Small Bay in -13 m MSL depth are provided below.

Table 3.4: Extreme nearshore waves.

Return Period [years]	H_{m0} [m]	T_p [s]	MWD [deg]	DSD [deg]
0.1	3.20	13.06	250	30
0.5	3.45	13.56	250	30
1	3.57	13.79	250	30
5	4.16	14.89	250	30
10	4.48	15.45	250	30
50	5.47	17.07	250	30
100	6.01	17.90	250	30
500	7.54	20.05	250	30

3.2.4 Sediment processes

The sediment transport in Small Bay is wave-driven and the following conclusions can be made from previous studies and data:

- Small Bay will typically lose sand to the north during summer which returns during winter.
- The sediment transport in Small Bay occurs closer to the shoreline and the sediment transport rates are significantly lower than in Big Bay. This is due to the offshore reef reducing the wave heights in Small Bay and the rocky seabed in Small Bay.
- Cross-shore erosion will move sand offshore during storm events thus lowering the inshore sand levels.
- Although the sand levels in Small Bay vary, no significant long-term trend can be discerned.
- Based on available beach profiles, the vertical variability in the sand levels is less than 1 m at depths deeper than +2 m MSL.

3.2.5 Overtopping and flooding

The modelled mean overtopping rates over the seawall for the proposed road level are provided below. **Figure 3.7** illustrates the sections considered.

SRW\Models\M3W\02\Overtopping_Sections.png

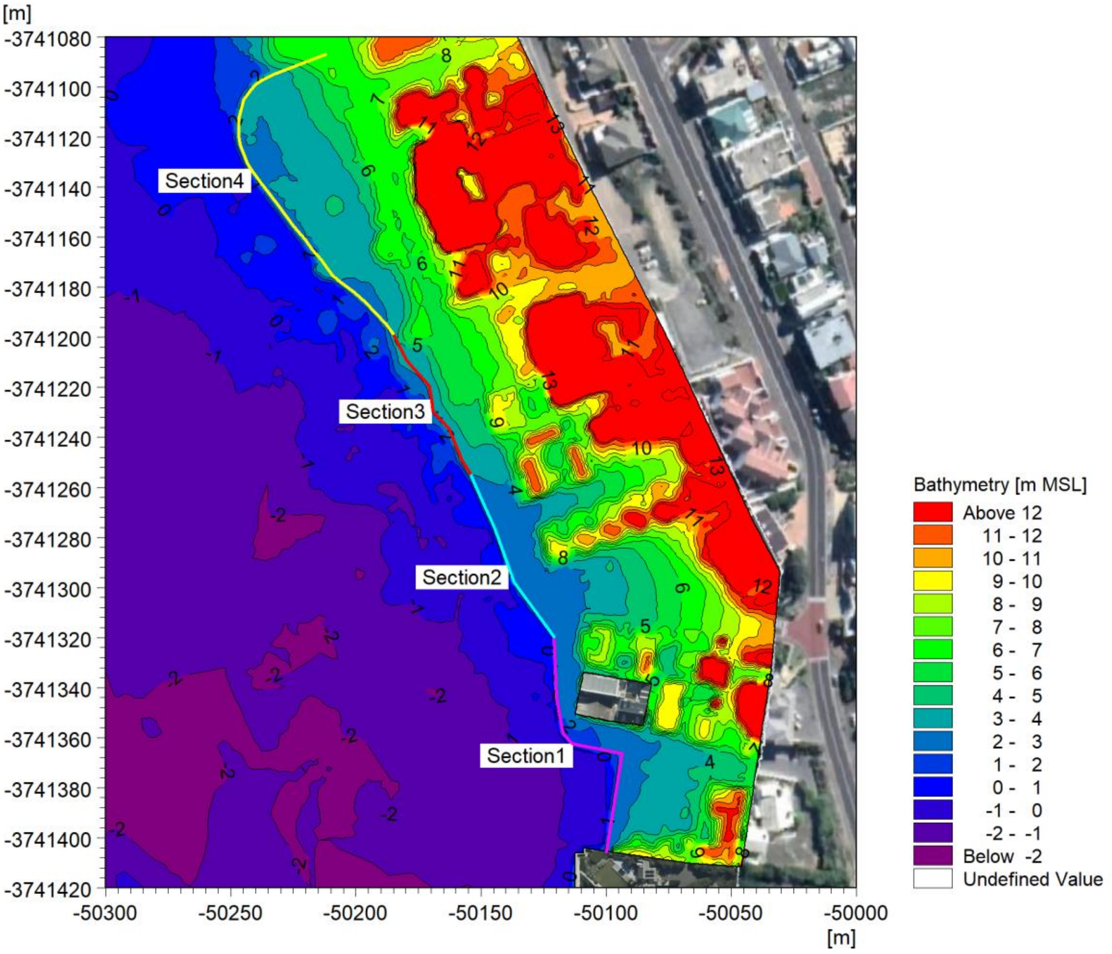


Figure 3.7: Overtopping sections with bathymetry at low sand levels.

Table 3.5: Modelled mean overtopping rate for the proposed road level.

Combined Return Period [years]	Wave Return Period [years]	Residual Return Period [years]	Mean Overtopping Rate [l/s/m]				
			Total	Section 1	Section 2	Section 3	Section 4
1	0.1	1	24.2	62.7	30.4	3.9	3.8
1	1	0.1	24.3	62.8	29.2	3.4	4.7
5	0.5	5	44.1	95.0	65.3	12.4	12.8
5	5	0.5	35.8	80.6	50.2	9.2	9.9
100	10	100	222.1	376.0	257.1	127.6	141.3
100	100	10	124.1	225.6	159.9	63.5	63.8

The modelled mean overtopping rates over the seawall for the 1 m raised seawall (with gaps) are provided below.

Table 3.6: Modelled mean overtopping rate for the 1 m raised seawall (with gaps).

Combined Return Period [years]	Wave Return Period [years]	Residual Return Period [years]	Mean Overtopping Rate [l/s/m]				
			Total	Section 1	Section 2	Section 3	Section 4
1	0.1	1	6.7	18.8	7.3	0.8	0.7
1	1	0.1	6.9	19.5	7.0	0.5	0.9
5	0.5	5	12.6	29.6	19.0	2.2	2.4
5	5	0.5	11.4	28.3	14.3	2.8	2.2
100	10	100	Model results to be added as soon as available				
100	100	10	Model results to be added as soon as available				

The results above show that raising the wall by 1 m significantly reduces the mean overtopping rate by a factor 4 on average.

Comparing the modelled overtopping rates to the limits from EurOtop, shows that for the proposed road level, the 1-year return period is hazardous to vehicles for Sections 1 and 2 and is hazardous to pedestrians for all sections. The 5-year return period is hazardous to vehicles and pedestrians for all sections. For the 100-year return period, Sections 1 and 2 are expected to see damage to paved promenade surfaces.

For the raised wall level, the reduction in the overtopping results in the 1-year return period presenting no hazard for Sections 3 and 4, while Section 2 becomes safe for vehicles. For the 5-year return period, the raised wall reduces the hazard for vehicles for Sections 3 and 4, while Sections 1 and 2 remain hazardous for vehicles and pedestrians.

The modelled maximum flood depths and current speeds behind the wall for the proposed road level are provided below.

Table 3.7: modelled maximum flood depths and current speeds behind the wall for the proposed road level.

Combined Return Period [years]	Wave Return Period [years]	Residual Return Period [years]	Maximum Flood Depth [m]					Maximum Current Speed [m/s]				
			Total	Section 1	Section 2	Section 3	Section 4	Total	Section 1	Section 2	Section 3	Section 4
1	0.1	1	1.51	1.51	1.15	0.73	0.62	6.00	6.00	5.04	4.17	4.06
1	1	0.1	1.25	1.25	1.13	0.70	0.79	5.34	5.34	5.21	2.84	4.29
5	0.5	5	1.44	1.44	1.35	0.87	0.90	6.46	6.46	5.82	4.45	4.45
5	5	0.5	1.53	1.53	1.47	0.94	1.05	5.69	5.69	5.28	4.37	5.21
100	10	100	3.20	3.20	2.53	2.13	2.68	8.77	8.77	7.68	7.36	8.09
100	100	10	2.76	2.76	2.41	1.75	1.98	7.06	7.06	6.73	6.14	6.58

The modelled maximum flood depths and current speeds behind the wall for the 1 m raised level (with gaps) are provided below:

Table 3.8: modelled maximum flood depths and current speeds behind the wall for the 1 m raised seawall (with gaps).

Combined Return Period [years]	Wave Return Period [years]	Residual Return Period [years]	Maximum Flood Depth [m]					Maximum Current Speed [m/s]				
			Total	Section 1	Section 2	Section 3	Section 4	Total	Section 1	Section 2	Section 3	Section 4
1	0.1	1	1.46	1.46	1.16	0.78	0.63	5.82	5.82	5.31	4.33	3.85
1	1	0.1	1.48	1.48	0.98	0.53	0.72	5.73	5.73	5.26	2.83	4.97
5	0.5	5	1.54	1.54	1.50	0.84	0.90	5.16	5.16	4.63	4.52	3.80
5	5	0.5	1.85	1.85	1.30	0.92	1.09	7.58	6.08	5.28	4.40	7.58
100	10	100	Model results to be added as soon as available									
100	100	10	Model results to be added as soon as available									

The results above show that raising the wall by 1 m results in similar maximum water depths behind the wall, despite the significant reduction in overtopping rates. This is due to the overtopped water taking longer to drain back through the gaps in the wall and ponding against the inside of the wall. The raised wall does however reduce the flood depths and extent of flooding further inland away from the wall.

The results above show that raising the wall by 1 m results in similar maximum current speeds behind the wall, despite the significant reduction in overtopping rates. The raised wall results in high current speeds at the gaps in the wall where the water flows back to the sea, which is partly a model artefact due to simulating 2 m gaps at 20 m spacings instead of 0.2 m gaps at 2 m spacings. The raised wall does however reduce the current speeds and extent of flooding further inland away from the wall.

Comparing the maximum depths to vulnerability curves for general flood hazards, shows that the maximum flood depths for the 1-year return period are unsafe for people and vehicles in Section 1, while Sections 2 to 4 are unsafe for vehicles and the elderly and children. The maximum flood depths for the 5-year return period, Section 2 becomes unsafe for all people as well as vehicles, while Sections 3 and 4 remain unsafe for vehicles, children and the elderly. The maximum flood depths for 100-year return period are unsafe for vehicles and pedestrians and could cause some structural damage landwards of the seawall. However, the seawall, sewer and new road infrastructure are designed for these events however residential infrastructure may be a risk to damage.

Comparing the maximum currents to vulnerability curves for general flood hazards shows that the maximum current speeds for all returns periods are unsafe for pedestrians and vehicles and could cause damage to structures. It should be noted that these current speeds are the maximum at any point and the spatial maximums presented in **Annexure B** can be used to assess the risks at specific sites of concern.

Overtopping and drainage are discussed further in Section **4.13**.

3.2.6 Design Waves and Water Levels

The design waves and water levels were modelled to calculate the wave forces as part of the structural design. The results show that for the 100-year return period and SLR to 2074, the maximum wave height (Hmax) anywhere along the wall was 3.86 m.

3.2.7 Recommendations

The overtopping model should be used to test additional cases to optimise the solution. These can include:

- Testing SLR for 2049 and 2074.
- Testing different drainage options.
- Testing different wall heights.

3.3 Master Topographical Survey

Parker Surveys were responsible for compiling a master topographical survey for the study area based on previous surveys which were made available by the Client, as well as the surveys conducted by Tritan Surveyors for the coastal site investigation. In order to compile the master survey, a ground truthing investigation was required. This was done in order to determine the accuracy of the existing topographical surveys and any discrepancies. A list of missing features was prepared, along with a spot check on various levels previously surveyed. This information is included in **Annexure C**.

While there were some omissions (such as dustbins, bollards and steps) or very slight level discrepancies, Parker Surveys confirmed that the existing surveys were acceptable to be used. Following this, they compiled a master topographical survey which stitched the original topographical surveys with the additional coastal surveys that were completed. This is included in **Annexure C**.

3.4 Geotechnical and Materials Investigation

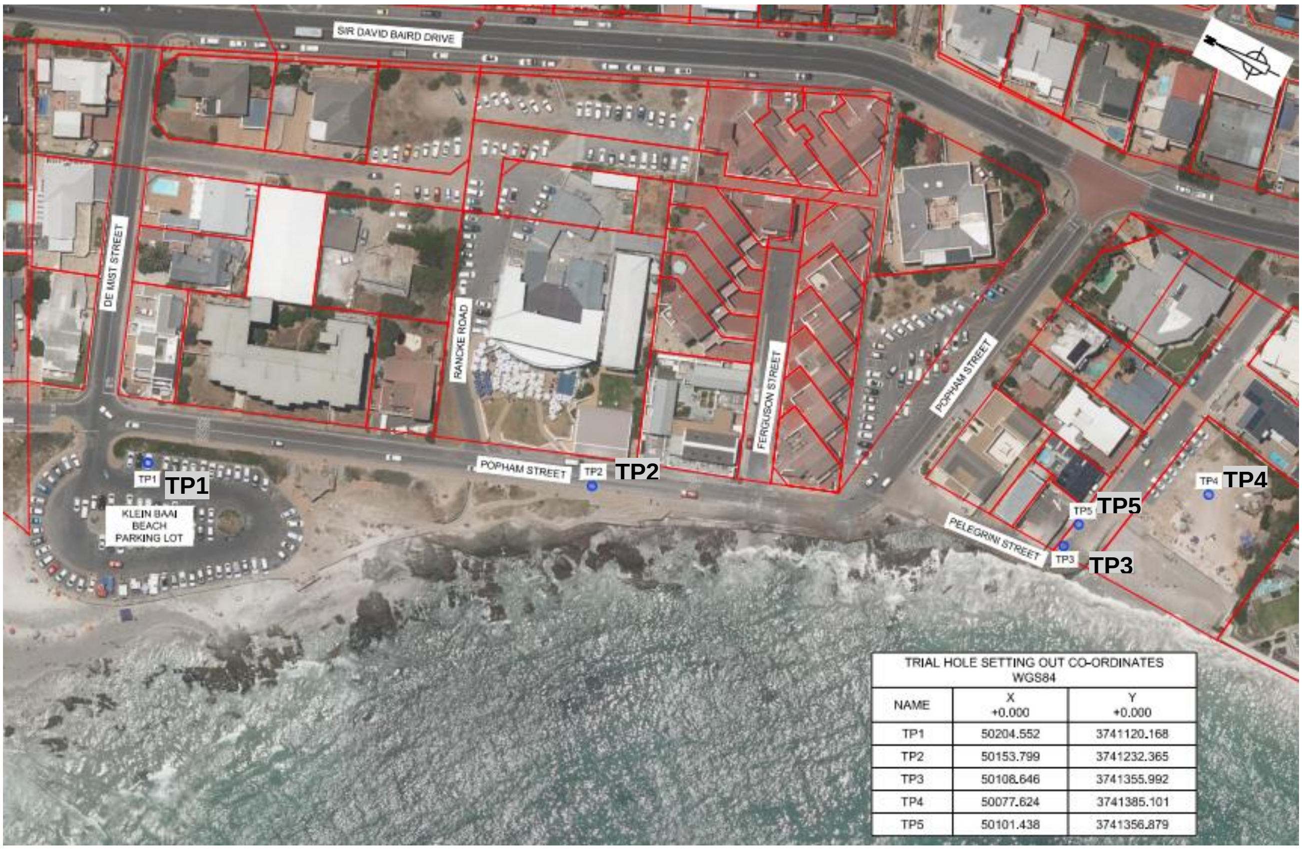
A geotechnical and pavement materials investigation was conducted in order to provide sufficient information for the design of the proposed foul sewer and road reconstruction. This investigation included a desk study of the site and surrounds using Google Earth aerial photo imagery, as well as an examination of the following published geological maps:

- 1:250 000 scale Geological Series 3318 Cape Town (Geological Survey, 1990)
- 1:50 000 scale Geological Series 3318 CD Cape Town (Geological Survey, 1984)
- 1:50 000 scale Geotechnical Series 3318 CD Cape Town (Council for Geoscience, 2006)

The geology of the site, according to the 1:250 000 scale geological map, is Quaternary age (recent) transported deposits of the Witzand Formation, comprising of “unconsolidated white sand with comminuted shell, pebbles and shells locally along the beach”. These are deflation products of modern beaches and occur sporadically along the coast in the greater Cape Town area.

Rock units of the Tygerberg Formation of the Malmesbury Group are shown to form the Bloubergstrand peninsula and series of small islands that characterise the Small Bay coastline. The Tygerberg Formation comprises of greywacke (poorly sorted, fine-grained sandstone), phyllite (foliated metamorphic rock) and quarzitic sandstone. These rock units underlie the transported sands and were observed along the coast (within and adjacent to the intertidal area) and were encountered with depth in some test pits. The Tygerberg Formation comprises of interlayered siltstone/phyllite which is relatively easily weathering and hard-weathering greywacke/sandstone. This leads to sharply different weathering depths. The variable rock hardness coupled with steep bedding orientation results in the jagged rock profile observed along the coastline.

Following the desk study, a field investigation was conducted by SGS Matrocast between 24 and 26 May 2021 which comprised of hand excavation, profiling and backfilling of five test pits; sampling of soils for laboratory testing and dynamic cone penetrometer testing. The test pits were excavated to depths of between 0.60 and 2.20 m at selected locations to assess the pavement structure and subgrade conditions beneath existing roads and parking areas and to determine the deep soil profile at the proposed sewer replacement location. The location of the trial pits is shown in **Figure 3.8** and a summary of the findings is provided in **Table 3.9.**



TRIAL HOLE SETTING OUT CO-ORDINATES
WGS84

NAME	X +0.000	Y +0.000
TP1	50204.552	3741120.168
TP2	50153.799	3741232.365
TP3	50108.646	3741355.992
TP4	50077.624	3741385.101
TP5	50101.438	3741356.879

Figure 3.8: Trial pit locations

Table 3.9: Summary of trial pits

Trial Pit	Methodology	Final Depth Test Pit (m BGL)	Depth DCP Tests (mm BGL)
TP1	Test pit through parking area pavement DCP test x 2	1.35 m (R)	40-964 1000-2617
TP2	Test pit through road pavement DCP tests x 2	1.60 m	0-929 1450-3384
TP3	Test pit through concrete road DCP test	0.60 m (R)	400-1016
TP4	Test pit in grassed park DCP tests x 2	2.20 m	0-936 150-1959
TP5	Test pit through concrete road	1.30 m (R)	None
R- Refusal			

The test pits indicate that the natural soil profile comprises of predominantly sandy or gravelly transported deposits with shell layers (interpreted as aeolian or littoral in origin) of the Springfontyn Formation. An extremely variable bedrock profile was observed in the test pits or inferred from the DCP tests. TP1 and TP2 were both conducted within existing asphalt roads. The layerworks were classified as shown in **Table 3.10**.

Table 3.10: Classification of TP1 and TP2 layerworks

Layer Thickness	Description	Classification (COLTO)
TP1	Asphalt	
40mm	Crushed stone base	G5
100mm	Natural gravel subbase	G5
100mm	Selected subgrade	G8
±150mm	In situ	G8
TP2		
30mm	Asphalt	
150mm	Calcrete base	G8
200mm	Shell/sand subbase	G7
	Bulk fill	G8

Test pits TP3 and TP5 were excavated to assess the subgrade conditions beneath the concrete roadway and to determine the thickness of the concrete. TP3 indicates that the concrete slab is 300 mm thick in this area and is underlain by an approximately 150 mm thick layer of no-fines concrete. A gabion basket was encountered beneath the concrete and a void was observed on the seaward side of the test pit, beneath the slab. Strong airflow through this void was felt when waves washed against the sea wall, indicating that the void may be continuous with the openings beneath the sea wall to the South. A large void was also observed to the north-west of TP3, where a broken section of slab adjacent to the sea wall exposed a void extending away from the wall beneath the slab. This void was 650 mm deep and extended to >2 m away from the opening

The concrete at TP5 was 200 mm thick and was underlain by gravelly fill and sandy alluvium without any voids below the concrete.

TP4 was excavated within the grassed park and encountered sandy fill to a depth of 0.35 m, underlain by alternating sands and rounded cobbles with shell layers to a depth of 2.20 m. No sidewall collapse was observed at TP4. However, it is recommended that allowance is made for shoring or battering back deep excavations.

Overall, the bedrock profile was noted to be extremely variable, and bedrock maybe encountered at shallow depths during excavation and trenching on site. The sandy soils are deemed to be highly erodible to both wind and water. The control of stormwater drainage is critical to preventing erosion and undermining of infrastructure from water running off of structures and hardened surfaces.

More detailed analysis, as well as test pit logs, DCP test results and laboratory test results can be found within **Annexure D**.

3.5 Additional Geotechnical Investigations

3.5.1 Rock levels at De Mist ablutions

During the detailed design phase of the project, further geotechnical investigations will be undertaken in the vicinity of the De Mist ablution block. DCPs and trial pits will be conducted to determine the depth of rock along the proposed pipe alignment. This will confirm whether any hard excavation will be required during construction.

3.5.2 Borehole investigations

The proposed seawall consists of a gravity concrete wall founded directly on bedrock. Due to site constraints the width of the foundation block is limited, and rock anchors are required to achieve the overall stability requirements. Typically, one would carry out borehole investigations along the seawall alignment to get a better understanding of the bedrock quality, hardness and fracturing. This information would then be used to help inform the anchor design depth requirements. However, even if exploratory boreholes are drilled, there will always be uncertainty in the condition and quality of the bedrock between the borehole positions. Due to this uncertainty the design Engineer will still apply a level of conservatism in the rock anchor design.

The rock anchor length has been calculated by applying the weight of a rock cone appropriate to a highly fractured rock at a reasonable lower bound density (a conservative approach). All highly weathered rock will be removed from the wall base footprint, therefore bearing pressures will be well within the capacity of the founding rock and no further information is required for the bearing pressures.

Rock anchor tests will be performed during construction to confirm that rock anchor pull out resistance is sufficient and that the anchors will perform as required. The estimated cost of installing the rock anchors between the Play Park and Ferguson Street is in the order of R1 million. The cost of installing slightly longer rock anchors to satisfy a conservative design is considered significantly more cost effective than carrying out borehole investigations, which will still not give absolute rock properties or accurate design parameters.

It is always good practise to carry out borehole investigations for a project like this, however considering the above explanation, with the relatively high cost of carrying out borehole investigations relative to the cost of the rock anchors, with the limited value add from borehole information, and together with the limited project budget, it is recommended that the detail design continue without carrying out borehole investigations. The capacity of the anchors will be confirmed by in-situ testing of selected anchors.

3.6 Underground Services Investigation

An underground services detection survey was completed by Hydrometrix Technologies on 20 May 2021. This survey confirmed and detected the location and depth of all existing services which included cables, fibre optics, water, foul sewer and stormwater pipes within the study area. While existing services information was requested from the relevant service authorities (as part of the wayleaves), it is well known that these plans can be out of date or incorrect.

The survey was conducted using GPR and ELM locating equipment. Intermediate grid surveys were conducted every 20m to ensure any unknown services were detected. All utilities were mapped using GPS survey equipment and finally coordinated AutoCAD drawings were produced. This information was very important for the design of the proposed sewer which is required to tie into the existing pipes at several points, as well as avoid collisions with other existing services. The detailed survey plans can be found in **Annexure E**.

4 COASTAL ENGINEERING DESIGN

4.1 General

A preliminary design has been undertaken for the proposed seawall reconstruction. The purpose of the preliminary design is to develop the seawall concept to a suitable level of detail to inform the project costing and to carry through into the detail design phase.

The new seawall provides a robust marine infrastructure solution that will remain serviceable (with minimal maintenance required) for the specified design life. The seawall will protect the adjacent CoCT infrastructure and services.

This section of the report documents the concept development and design of the seawall accordingly.

4.2 Seawall Constraints

There are a variety of site constraints which govern the seawall alignment including the selection of appropriate structural cross sections. These constraints are as follows:

- 50 m^2 encroachment limit seawards of the existing surveyed disturbed footprint.
- Existing residential and civil infrastructure especially along Pelegrini Street.
- Existing road width and associated functional requirements.
- Existing road levels relative to design still water levels and wave crest heights.
- Existing foul sewer lines and pump station at the play park.
- Space, site access and storage constraints during construction.

4.3 Seawall Alignment

The proposed 306 m of seawall reconstruction has been divided into 4 sub-sections as illustrated in **Figure 4.1**. These are:

- Popham Street (61 m) – blue
- Pelegrini Street (76 m) – green
- Play park (41 m) – yellow
- De Mist parking lot (128 m) – red

Figure 4.1: Seawall sections under consideration

4.4 Seawall Phasing

Development of the new seawall has been split into two phases. The requirement of the first phase is to provide protection to the Cities landside infrastructure, in particular the sewer line, roadway, and pedestrian access. The project brief however requires that the seawall foundation and base be designed so that the seawall can be raised to a final crest elevation of +4.5 m MSL in the future. This is required to provide further protection from wave action and overtopping.

The seawall will not initially be built to the +4.5 m MSL level due to the following:

- Budgetary constraints,
- The boundary it will create between the land and shoreline access,
- The obstruction of sea views and,
- The potential challenges and delays in getting the necessary authorisations.

These two project phases are further summarised as follows:

4.4.1 Phase 1

The primary purpose of phase 1 is to provide a robust seawall that protects the new sewer pipeline and roadway. The seawall crest level will align with the adjacent road level and allow for all overtopping seawater and storm water to drain directly over the top of the seawall and back into the sea. Therefore, there will be no need for catchpits or drainage pipes through the seawall. The foundation and base of the seawall will be designed so that the seawall can be raised in the future to a level of +4.5 m MSL, without having to reconstruct the seawall foundation.

For safety requirements a barrier is required along the seaward edge of the seawall to prevent vehicles or pedestrians from falling off the wall. According to legislation this barrier needs to be at least 0.9 m high. The most cost-effective barrier is to provide open railings. This is however not a robust solution and may be damaged after significant storm events. An alternative and more robust barrier would be to construct a concrete cope wall along the edge. It is proposed that 1 m high and 2 m long precast reinforced concrete recurved cope units be placed along the seawall edge. These units will have a 200 mm gap between them to allow for drainage of storm water and overtopping seawater. This cope wall will be low enough to provide sea views over the wall while providing the added benefit of providing some protection against wave overtopping and giving some protection to pedestrians and vehicles using the road behind the seawall.

4.4.2 Phase 2

The level of the seawall can be raised in phase 2 to provide added protection again overtopping which will increase with the predicted future sea level rise.

Phase 2 construction will require the following:

- Remove the handrails or cope wall units along the top of the phase 1 seawall.
- Drill and install additional vertical rock anchors through the foundation blocks to help improve the overall wall stability.
- Drilling and grouting reinforcing starter bars in the top of the phase 1 wall.
- Construct a new in-situ reinforced concrete seawall along the top of the phase 1 wall.
- Provide large gaps in the phase 2 wall extension, at strategic low points, in order to drain overtopping and storm water.

The functionality and effectiveness of the phase 2 seawall extension is questioned for the following reasons:

- It will provide added protection against overtopping however the coastal modelling results show that extensive flooding will still occur behind the seawall during storm events.
- The road levels along Pelegrini Street range from +1.9 m to +2.5 m MSL. As shown in the coastal modelling section some of the design still water levels are higher than these levels and with waves on top of this, the area behind the wall will experience extensive flooding for prolonged periods during the more extreme storm events. Flood water will not just be from the overtopping volumes but also from water coming through at drainage points. This issue will be most prevalent along Pelegrini street.
- A significant barrier of between 2 m to 2.5 m high will be created between the road and the sea and this will obstruct sea view from the road, de Mist parking and the play park.

4.5 Seawall Structural Solution

The seawall structure differs from the mass gravity structure with precast concrete blocks that was previously proposed. With the spatial constraints on site and especially in the Pelegrini Street and Play Park areas, it is difficult to provide sufficient wall mass to resist the design wave loadings with only the weight of the wall.

The site constraints, as described in Section **4.2**, require a narrower wall foundation block to help improve constructability and accommodate the relocation of the foul sewer line. This is compounded by significant design storm conditions creating large destabilising wave forces and buoyancy influences due to the high-water levels. The preliminary design has therefore identified the requirement of rock anchors to ensure the overall stability of the wall.

4.5.1 Popham Street

Currently, this portion of the existing seawall shows signs of significant undermining which is made evident by:

- Visible cavities under the seawall in various locations.

- Collapsed and eroded road paving and layer works immediately behind the seawall.
- Cracked foul sewer manholes.

This damage is illustrated above in Figure 2.4 through to Figure 2.7**.**

The foul sewer line runs within this portion of the existing seawall and is to be relocated as part of the main project scope. The new seawall will therefore protect the new sewer line which will be installed in compacted fill behind the wall base,

No pedestrian access to the beach will be provided from this point to the Small Bay Play Park.

The structural cross section along Popham Street makes use of a partial gravity wall solution whereby the mass of the wall is maximised, but rock anchors are required to achieve the full factored stability. This type of solution is feasible here given the increased available space compared to Pelegrini Street and the Play Park. The seawall cross section is illustrated in Figure 4.2. The following notable characteristics are:

- Cast in-situ, reinforced concrete construction.
- A founding layer of mass concrete used to level off the exposed bedrock. This is combined with the demolition of high spots in the bedrock (by land-based equipment/plant) to ensure a constant foundation level, with a minimum overlying concrete thickness of at least 300 mm.
- 4.80 m wide base with:
    - Passive rock anchors to help resist overturning,
    - Shear key to prevent sliding and over utilisation of the rock anchors.
- Vertical face to the seaward side of the wall to maximise space, road width and mass influence during overturing.
- Inclusion of a 1m high precast, recurved cope unit.
- Phase 1 construction to be level to the new road levels along Popham street.

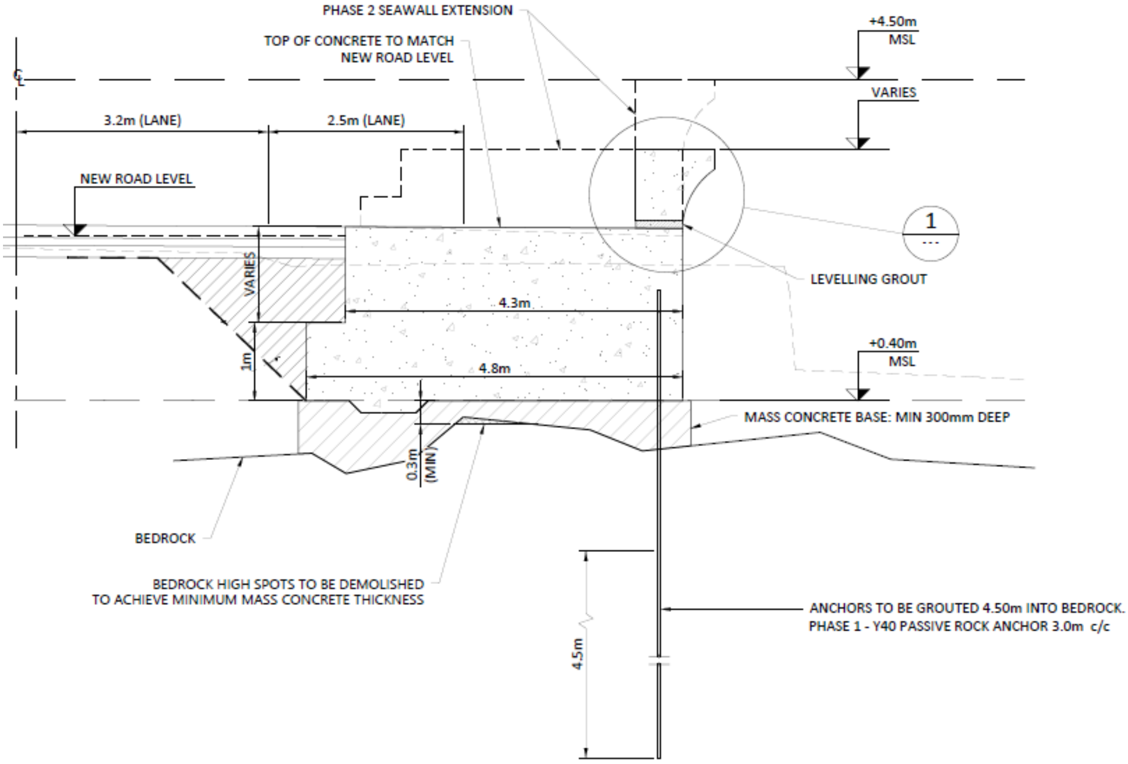


Figure 4.2: Popham Street cross section.

4.5.2 Pelegrini Street

Currently, this portion of the existing seawall shows signs of undermining which is made evident by:

- Visible cavities under the seawall in various locations
- Deterioration of the wall and its coping units
- Loss of material from under the road surface resulting in sink holes
- Exposure of the foul sewer line visibly daylighting through the Pelegrini Street seawall and crossing the play park beach.

This damage is illustrated in **Figure 2.8**.

The foul sewer line also runs within this portion of the existing seawall and is to be relocated as part of the main project scope. The new seawall will therefore protect the new sewer line which will be installed in compacted fill behind the wall base,

The structural cross section along Pelegrini Street makes use of an anchored, cantilever wall arrangement. A full gravity type solution is not feasible here given the constrained space along Pelegrini Street mainly due to the proximity of the residential infrastructure. The relocation of the foul sewer line, and the requirement for maintaining the current usable road width, further constrain the cross-section.

It therefore not possible to provide the required mass and foundation width to achieve stability and rock anchors are required. Additionally, the road levels behind the seawall do not have the required height to provide any meaningful passive resistance to wave attack, The seawall cross section is illustrated in Figure 4.3 and has the following notable characteristics:

- Cast in-situ, reinforced concrete construction.
- A founding layer of mass concrete used to level off the exposed bedrock. This is combined with the demolition of high spots in the bedrock to ensure a constant foundation level, with a minimum overlying concrete thickness of at least 300 mm.
- 3.70 m wide base with:
    - Passive rock anchors to resist overturning,
    - Shear key to prevent sliding and over utilisation of the rock anchors.
- Vertical face to the seaward side of the wall to maximise space, road width and mass influence during overturing.
- Inclusion of a 1m high precast, recurved cope unit.
- Phase 1 construction to be level with the new road levels along Pelegrini street.

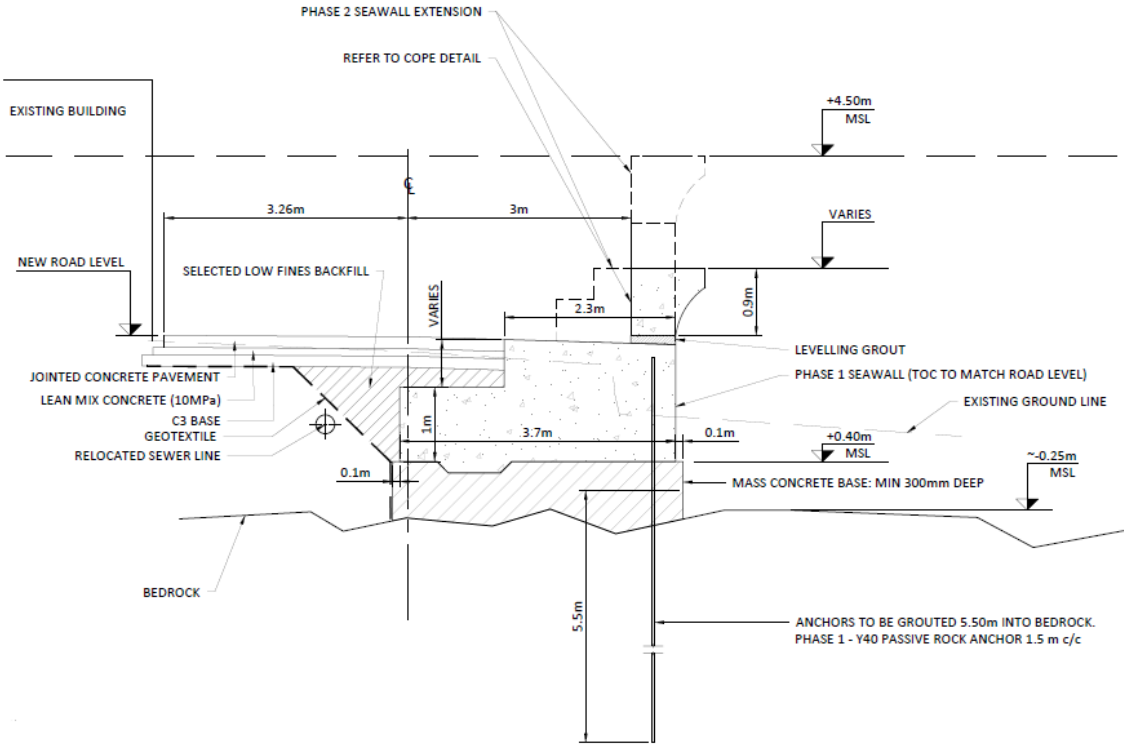


Figure 4.3: Pelegrini Street cross section.

4.5.3 Play Park

Currently, this portion of the existing seawall shows signs of significant undermining which is made evident by:

- Visible cavities and spanning of the seawall.

- Significant loss of back of wall material which has damaged the paving and exposed the foul sewer line immediately behind the wall.

This damage is illustrated in **Figure 2.9**.

The existing foul sewer line runs immediately behind this portion of the existing seawall. The proximity of the sewer line makes the construction of the seawall difficult given the deep excavation which will be required to reach the anticipated bedrock levels. Relocation of this sewer line has been considered which will help to reduce the sensitivity of construction along this section of seawall.

The typical structural cross section along the Play Park makes use of a partial gravity wall arrangement anchored to the bedrock with passive rock anchors. A full gravity type solution is not feasible here given the constrained space and depth of bedrock.

The seawall cross section is illustrated in **Figure 4.4** and has the following notable characteristics:

- Cast in-situ, reinforced concrete construction.

- A founding layer of mass concrete is used to level off the exposed bedrock. This is combined with the demolition of high spots in the bedrock to ensure a constant foundation level, with a minimum overlying concrete thickness of at least 300 mm.

- 3.00 m wide base with:

    - Passive rock anchors to resist overturning,

    - Shear key to prevent sliding and over utilisation of rock anchors.

- Vertical face to the seaward side of the wall to maximise space and mass influence during overturing.

- Inclusion of a 1m high precast, recurved cope unit (this unit may be removed to decrease the project cost estimate).

- Phase 1 construction to be level to the existing play park levels (~ +2.50 m MSL).

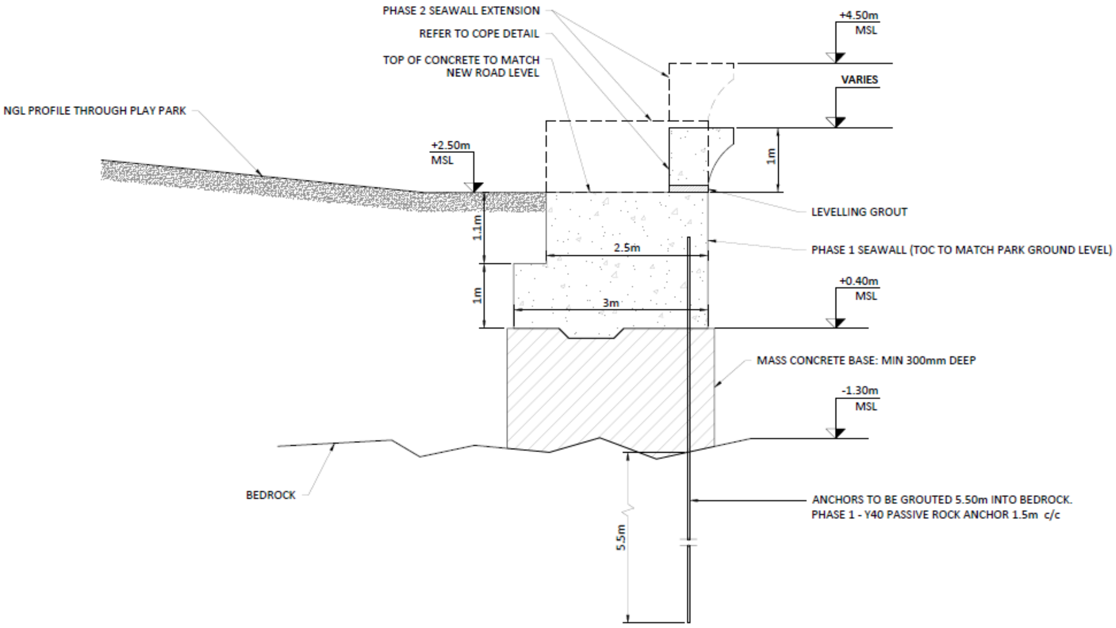


Figure 4.4: Play park cross section.

4.5.4 De Mist parking lot

Currently, this portion of the existing seawall shows signs of significant undermining which is made evident by:

- Rotation, and subsidence of the seawall seawards.

This damage is illustrated in **Figure 2.3**.

The preliminary design intends on interfacing this portion of the alignment with the existing concrete pedestrian walkway which has been cast directly onto the exposed bedrock at the southern end of the viewing platform.

The typical structural cross section along De Mist makes use of the same partial gravity wall solution utilised along Popham Street.

4.6 Codes and Specifications

The following codes, standards and references have been used for the design of the marine works:

BS 6349-1-1	British Standard, Maritime Structures: Code of practice for planning and design for operations
BS 6349-1-2	British Standard, Maritime Structures: Code of practice for assessment of actions
BS 6349-1-3	British Standard, Maritime Structures: Code of practice for geotechnical design
BS 6349-1-4	British Standard, Maritime Structures: Code of practice for materials
BS 6349-2	British Standard, Maritime Structures: Code of practice for the design of quay walls, jetties and dolphins
BS EN 1990	Eurocode: Basis of structural design
BS EN 1991	Eurocode 1: Actions on structures

BS EN 1992	Eurocode 2: Design of concrete structures
BS EN 1993	Eurocode 3: Design of steel structures
BS EN 1997	Eurocode 7: Geotechnical design
BS EN 1998	Eurocode 8: Design of structures for earthquake resistance

4.7 Design Criteria

The following section lists the various design criteria given by the Client:

- The seawall is to be founded on bedrock.
- Two project phases are to be considered.
    - Phase 1: The top of the seawall is to be constructed to the level of the road so that stormwater drains over the seawall. A handrail or concrete recurved cope unit wall will be placed along the edge of the seawall. (seawall crest level varies).
    - Phase 2: The seawall can be raised in the future to a final crest level of +4.50 m MSL.
- The seawall foundation is to be designed to allow for the wall to be raised to crest level of +4.50 m MSL in the future.
- The seawall footprint shall not encroach more than 50 square meters seawards of the existing footprint.
- The seawall is to have a design life of 50 years.

4.8 Design Philosophy

The analysis and design of the seawall is based on the limit states design approach set out in the Eurocode suite of documents (EN 1990 – 1998). The design philosophy is supplemented by the specific requirements and partial factors obtained from BS 6349 specific to maritime structures.

The preliminary design considers a selected range of varied load situations which are considered to be the most critical for the engineering stage considered.

The detailed design will consider a range of severe and varied load situations which can reasonably occur during the design life of the seawall. The assessment, combination of actions and design conditions shall be in accordance with EN 1990 and BS 6349.

All limit states (i.e. overturning, sliding and bearing) will be identified, analysed and incorporated into the design.

Although not considered in the preliminary design, due consideration must be given to credible accidental design situations to ensure that an adequate level of structural robustness and reliability is achieved. Accidental design situations will be defined as recommended in EN 1990, and BS 6349 and will be included in the detailed design.

4.9 Design Life and Conditions

A minimum design working life of 50 years has been specified for the permanent marine infrastructure.

The seawall will be designed for the following storm events:

- 100-year return period (design): 39.60% encounter probability

- 500-year return period (accidental): 9.52% encounter probability (TBC in detailed design)

4.10 Design Actions

4.10.1 Permanent actions

Dead loads (self-weight) is based on the following material densities:

- Unreinforced concrete: 23.5 kN/m^3
- Reinforced concrete: 25 kN/m^3
- Structural steel: 77 kN/m^3

All superimposed dead loads will be added on an ad hoc basis as required.

4.10.2 Geotechnical actions

Geotechnical actions are based on the following material parameters for compacted backfill material:

- Saturated unit weight: 19 kN/m^3
- Effective friction angle: 35°
- Effective cohesion: 0 kPa

Geotechnical actions are based on the following lateral earth pressure limits:

- Stabilising and destabilising pressures are both taken at rest.

The use of at rest pressures are considered appropriate as negligible translation of the wall is anticipated given the rigidity of the structure. This prevents full mobilisation of active and passive pressures.

4.10.3 Variable actions

4.10.3.1 Surcharge

The following surcharges are applied to the sea wall, where appropriate:

- Traffic: 20 kPa

4.10.3.2 Design waves and water levels

Specific wave and water level parameters are extracted from the wave model at various points along the seawall alignment. These parameters are used in the respective wave load estimation methods. The positions of the points considered along the seawall are shown in Figure 4.5.

Figure 4.5: Points considered for wave load determination.

Wave loads are dependent on the sensitivity of the site to sediment processes as well as the joint probability between waves and water levels. Wave parameters extracted from the wave model are based the effective 100-year return period storm event which considers two combinations of wave and water levels:

- 10- and 100-year return period for wave height and water levels, respectively.
- 100- and 10-year return period for wave height and water levels, respectively.

Landward wave pressures on the seawall are calculated using Goda's method appropriate for vertical faced seawalls. The wave pressure distribution (P1, P2, P3 and Pu) is illustrated in **Figure 4.6**.

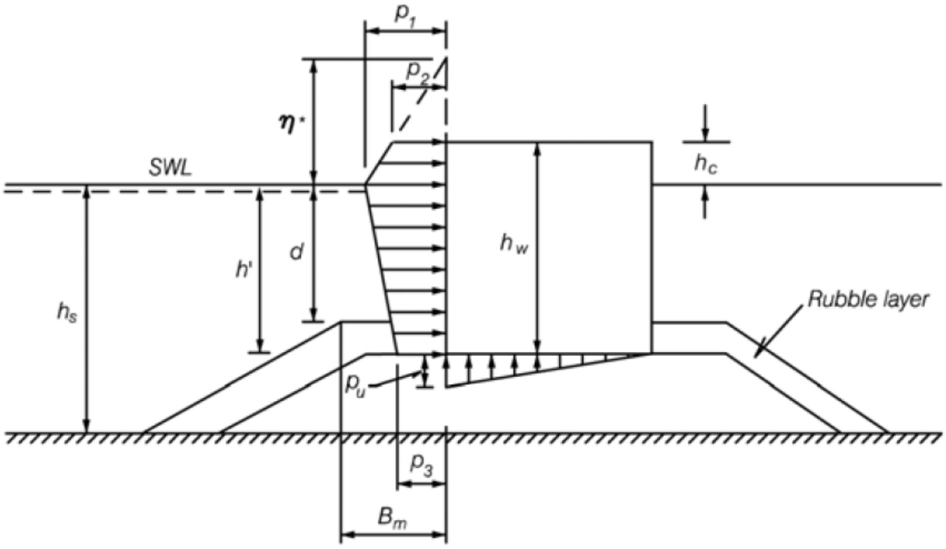


Figure 4.6: Goda method for vertical walls.

It should be noted that Goda's method was primarily developed, and calibrated, to estimate horizontal forces on concrete caisson breakwaters founded on rubble mounds. The use of this method, although outside its validation range, is generally considered best practice for the estimation of wave loads on vertical seawalls especially if physical model testing is not available.

The seawall is supported on an impermeable solid bedrock which is extended by a mass concrete foundation cast directly onto the bedrock. Uplift pressures could be generated if pressure infiltration is possible in between the bottom of the seawall and its foundation. This infiltration may be possible due to various factors such as seawall construction method, seawall rocking during extreme events, crack opening along the foundation interface and construction tolerances. All of these increase the probability of allowing pressure to penetrate the interface which will lead to some magnitude of uplift.

It may be justifiable to assume a reduced uplift pressure (Pu) given the plausible variations in opening sizes, their extent, spatial orientation, and location. A conservative approach is therefore taken by applying full uplift pressures as calculated by Goda's method where there is limited sediment build up in front of the wall.

The landward pressures applied to the seawall are presented in **Table 4.1**.

Table 4.1: Landward wave pressures calculated using Goda's method.

Description	**Position**	**Play Park**	**Popham**	**Pelegrini**	**De Mist**
	Point	**Point 1b**	**Point 4**	**Point 3**	**Point 7**
	Model (1)	**Low**	**Low**	**Low**	**Current (3)**
	Combination (2)	**100/10**	**100/10**	**100/10**	**10/100**
SWL	**m MSL**	2.61	2.54	2.55	2.92
Hmax	**m**	3.70	3.20	3.80	3.10
P1	**kPa**	46.40	38.70	46.20	54.00
P2	**kPa**	25.80	20.00	26.30	13.37
P3	**kPa**	45.80	38.10	45.60	53.70
Pu	**kPa**	40.10	34.60	40.70	33.90

Notes:

(1) Two bathymetry cases were investigated i.e. current and low bathymetry.

(2) Two combinations of wave and water level return periods were investigated.

(3) This case is characterised by a large sediment level in front of the seawall which requires modifications of the typical Goda pressure distribution.

The following methodology was used where modification of the Goda pressure distribution was required due to high sediment levels in front of the wall (i.e. at de Mist):

- P3 is applied at the depth contour in front of the wall.
- P3 is reduced to 0.50P3 at the base of the seawall.
- 0.50Pu is used as the uplift pressure.

It must be noted that the assumption of pressure reduction through sediment layers is not based on any specific physical model testing or published literature. However, pressure reduction is anticipated. Since the reduction is uncertain, a conservative approach would be to assume no reduction. This is likely to result in increased cost which needs to be considered by the CoCT. Without a solid basis for estimating the reduction the CoCT may choose a conservative design in which no reduction is applied.

Wave and water pressures, pushing the wall seawards, are considered in the preliminary design. The highest destabilising action in this direction coincides with the wave trough occurring at the wall accompanied by a flooded back of wall area. The seaward pressure is calculated by considering a conservative assumption on the influence of wave clapotis effects which lower the wave trough level at the face of the seawall. The wave model predicts highly asymmetrical waves close to the seawall. As such, Sainflou's method is therefore not applicable. Clapotis effects are allowed for by assuming the wave trough manifests at a depth of 2 times the wave trough depth identified in the wave model. These calculations show that there is potential drying which occurs in front of the seawall where elevated sediment levels are present (i.e. at the Play Park, and De Mist). A trough level of +0.60 m MSL is considered at all locations along the seawall assuming no sediment in front of the seawall.

The seaward pressures are therefore based on the difference in hydrostatic head between the seaward and landward water levels.

4.11 Partial Factors and Load Combinations

All load combinations are developed in accordance with EN 1990 and BS 6349 and consider a variety of actions such that worst case load envelopes are produced.

The developed load combinations account for the principles of leading and accompanying variable actions and implement the respective partial factors and combination factors as recommended by the design codes of practice.

Specific consideration is given to the limit states (overturning, sliding and bearing) being investigated such that appropriate partial factors are applied to favourable and unfavourable actions for each.

In general, the following partial factors apply:

- Unfavourable:
  - Permanent: 1.35
  - Variable: 1.50
- Favourable:
  - Permanent: 0.95 (for EQU) and 1.00 (for others)
  - Variable: 0.00

4.12 Stability Verification

4.12.1 General

The preliminary design is based primarily on the verifications of overturning (EQU) and sliding. The bearing limit state is not considered critical at this stage given that the seawall will be founded on bedrock, filled in and raised with overlying mass concrete.

4.12.2 Analysis

The stability of the seawall has been analysed using GEO5 Prefab Wall software.

The analysis models considered the following:

- Three seawall cross sections, namely Popham, Pelegrini and the Play Park.
- Each of which considered the respective phase 1 and 2 crest levels:
    - Phase 1 crest levels follow the reinstated road levels. Therefore, crest levels change along the alignment. The lowest crest level in the alignment was considered for combinations where landward actions dominated, whilst the highest crest level was considered where seaward pressures dominated.
    - Phase 2 crest level taken as +4.50 m MSL for all cross sections where the height of the retained material behind the wall was taken the same as the phase 1 crest levels.

Analysis models where sub-divided according to the direction of the wave load being considered (i.e. landward or seaward). In general, the following methodology was adopted:

- Landward pressures (i.e.Goda forces):
    - Phase 1 and phase 2 crest levels considered.
    - Lowest road level behind seawall considered for resistance.
    - At rest pressure for backfill used for resistance.
    - Water levels behind seawall are increased in intervals up to the SWL.
- Seaward pressures (i.e. wave trough):
    - Phase 1 and phase 2 crest levels considered.
    - Highest road level behind seawall considered for destabilising action.
    - At rest pressure for backfill used.
    - Flooding up to +3.50 m MSL considered in all cases.
    - Water level behind seawall reduced in intervals until backfill is dry.
    - Traffic surcharge included when flood water behind the wall has drained to the level of the road, or less.
    - Wave trough taken at +0.60 m MSL for all cases.

A typical analysis model is presented in **Figure 4.7.** The figure illustrates the wave pressure distribution for a landward wave attack and shows the analysis case which produces the highest anchor force.

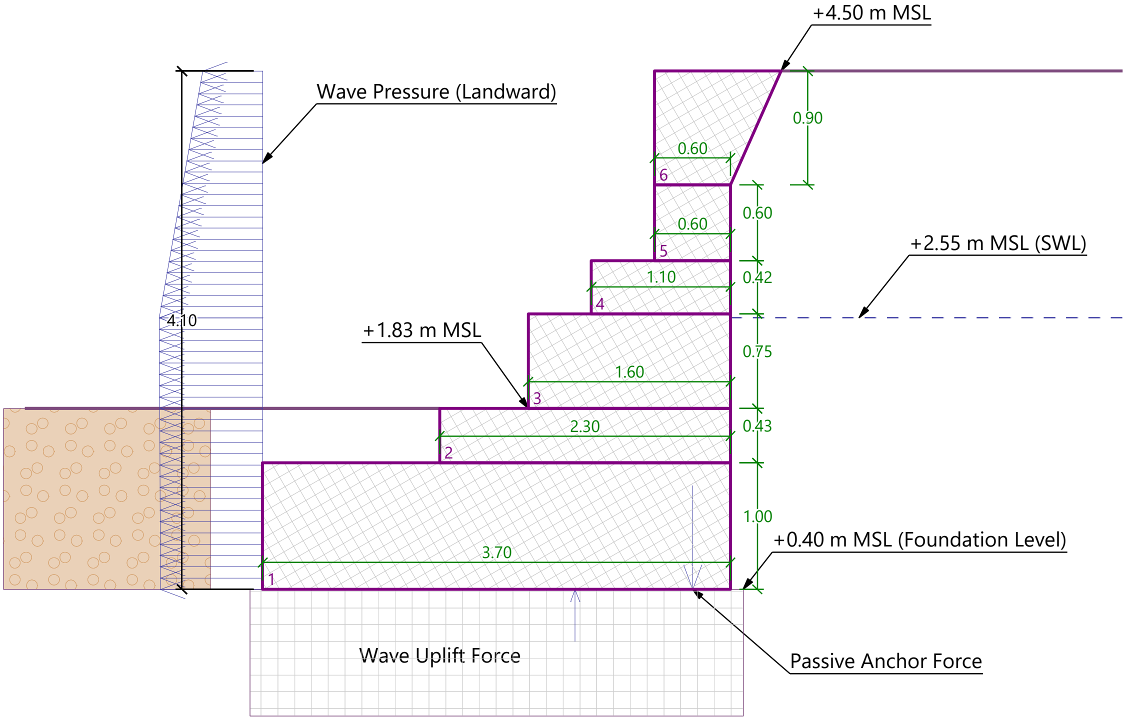
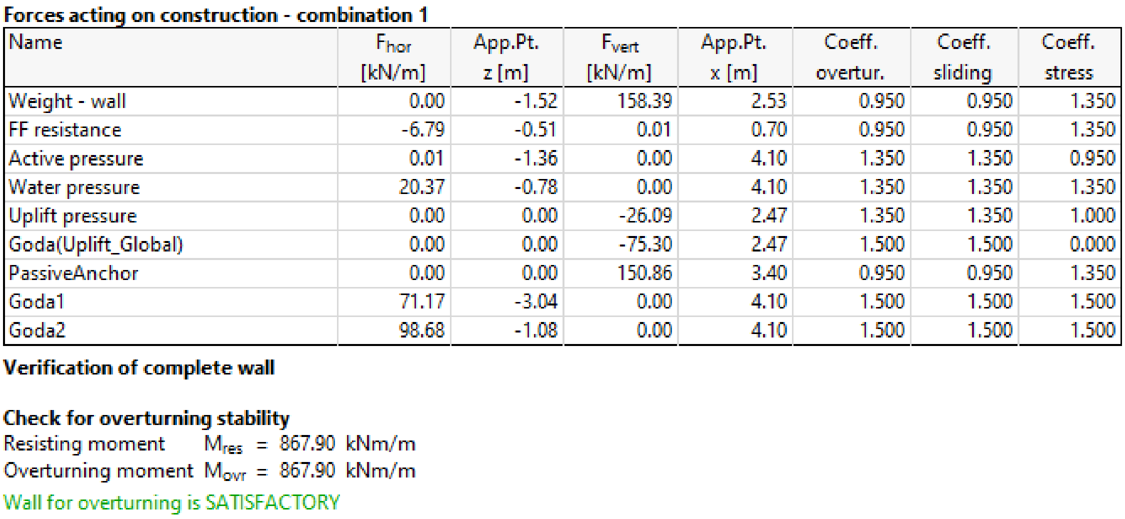


Figure 4.7: Typical GEO5 Prefab Wall model (Phase 2: Pelegrini, Landward pressures).

4.12.3 Results

The overturning (EQU) verification for the Phase 2 Pelegrini cross section illustrated above is presented in **Table 4.2**.

Table 4.2: EQU verification – Phase 2, Pelegrini Street + Landward pressure.



Forces acting on construction - combination 1

Name	F_{hor} [kN/m]	App.Pt. z [m]	F_{vert} [kN/m]	App.Pt. x [m]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - wall	0.00	-1.52	158.39	2.53	0.950	0.950	1.350
FF resistance	-6.79	-0.51	0.01	0.70	0.950	0.950	1.350
Active pressure	0.01	-1.36	0.00	4.10	1.350	1.350	0.950
Water pressure	20.37	-0.78	0.00	4.10	1.350	1.350	1.350
Uplift pressure	0.00	0.00	-26.09	2.47	1.350	1.350	1.000
Goda(Uplift_Global)	0.00	0.00	-75.30	2.47	1.500	1.500	0.000
PassiveAnchor	0.00	0.00	150.86	3.40	0.950	0.950	1.350
Goda1	71.17	-3.04	0.00	4.10	1.500	1.500	1.500
Goda2	98.68	-1.08	0.00	4.10	1.500	1.500	1.500

Verification of complete wall

Check for overturning stability
Resisting moment M_{res} = 867.90 kNm/m
Overturning moment M_{ovr} = 867.90 kNm/m
Wall for overturning is SATISFACTORY

The results from the stability analysis are summarised in **Table 4.3** which has been developed using a total of 21 different analysis models considering the various sections, and their loadings, along the seawall.

Table 4.3: Stability analysis results.

Section	Required Anchor Force (kN/m)		Anchor Depth (1) (m)	Anchor Spacing (m)	EQU (%) (2)		Sliding (kN/m) (3)	
	Phase 1	Phase 2			Phase 1	Phase 2	Phase 1	Phase 2
Popham (4)	84.38	59.53	4.50	3.00	100	100	199.8	263.8
Pelegrini	103.41	150.86	5.25	1.50	100	100	164.6	275.3
Play Park	125.96	150.77	5.25	1.50	100	100	217.6	265.5

(1) Required anchor diameter = 40 mm including corrosion allowances (fy = 450 MPa).

(2) All EQU utilisation ratios are 100% since rock anchors are required for stability.

(3) Sliding forces are shown here instead of utilisation since all analysis models show sliding instability. Shear keys are therefore required to resist the full sliding force.

(4) It is envisaged that the Popham Street and the de Mist parking lot will use the same structural cross-sections.

In summary, the seawall overturning (EQU) and sliding stability is governed by the landward wave pressures. This is due to the low retained material levels behind the seawall which are critical in developing resisting earth pressures. This is further compounded by the available space requirements which force reductions in seawall base width. Additionally, the elevated crest levels attract considerable wave forces whilst high SWL's which create buoyancy influences. Uplift forces also contributed greatly to the instability issues.

As indicated in **Table 4.3**, passive rock anchors are required to prevent overturning. The capacity verification of these anchors considered the following:

- Overall factor of safety (FOS) = 1.50 used with rock cone approach.
- Submerged rock cone weight used assuming a half angle of 30°
- Overlapping influences of adjacent anchors
- Corrosion allowance over 50 years at an average rate of 0.055 mm/year.
- Stress limitation of 60% of the anchor yield strength at the end of the design life.

Preliminary calculations show that an unreinforced shear key can be provided which has sufficient structural capacity to resist the anticipated sliding forces. Alternatively, shear dowels can be installed into the bedrock. The provision of a shear key prevents potential over utilisation of the rock anchors should these tend to resist shear force. Shear keys shall be constructed near the back face of the seawall to ensure that the shear key in engaged during the design storm events.

4.13 Overtopping and Drainage

Large sections of the existing seawall regularly experience wave overtopping, in particular along Pelegrini Street. Currently all the overtopping water drains back over the existing seawall.

One of the main challenges in this area is the low road and adjacent ground levels (+2 m to +2.5 m MSL) compared to the design still water levels, sea level rise and wave conditions means that the area behind the seawall will be flooded in extreme events, unless a suitably high and watertight seawall is constructed which is not practical for this area.

Phase 1 of the seawall construction has the top of the seawall at the same level as the new road and any overtopping water or storm water will drain directly back over the wall into the sea. If a 1 m high cope wall is constructed along the edge of the new seawall, then it will have 200 mm gaps between each cope wall unit to allow for drainage.

When the wall is raised in phase 2, this will prevent free drainage over the wall. In this event it is proposed that the overtopping and storm water be drained through large gaps in the raised portion of the seawall. These gaps would be at low areas of the road, at the following locations:

- At the bottom of Ferguson Road
- At the intersection of Popham and Pelegrini Street
- At the bend at the bottom of Pelegrini Street

Seawater will flood through these gaps and add to the flooded water behind the wall during extreme events.

The option of putting pipes through the wall with one-way valves to keep the water out, has been considered, however this is not recommended as they require a suitable water head to function, they are expensive, have a high risk of becoming blocked or damaged and are a high maintenance item.

The overtopping and flooding results presented in Section **3.2.5** confirm and validate these concerns. These results have illustrated that even low return period storm events pose a hazard to pedestrians and vehicles.

5 CIVIL ENGINEERING DESIGN

A preliminary design has been undertaken for the reconstruction of Popham and Pelegrini Streets, and the realignment of the sewer. This section of the report will explain in more detail how these designs were carried out.

5.1 Roadworks

The scope of roadworks which forms part of the civil engineering design includes the reconstruction of the following:

- Approximately 78m of Popham Street from its intersection with Pelegrini Street until the speed bump located north of the Ferguson Street intersection. The extent is shown in **Figure 5.1** below.
- Approximately 77m of Pelegrini Street from its intersection with Popham Street until the end of the existing concrete pavement. The extent is shown in **Figure 5.2**.

Figure 5.1: Extent of Popham Street reconstruction

Figure 5.2: Extent of Pelegrini Street reconstruction

5.1.1 Existing road information

Popham and Pelegrini Streets are both classified as Class U5 Urban Local Streets according to the Technical Recommendations for Highways 26 (TRH26) guidelines.

Popham Street is asphalt surfaced with layerworks as described in **Section 3.4** (represented by trial pit 2). The existing road has completely failed and consequently been closed to traffic.

Pelegrini Street is a concrete road which has some localised failures visible on the surface. In addition, considerable sections appear to be undermined – as detailed in the geotechnical investigations (**Annexure D**). This road is used by local residents and some tourists visiting the play park and beach. Pelegrini Street is the only road access for these residents.

5.1.2 Client requirements

The Client requirements for the roadworks were as follows:

- Both roads are to have a full depth reconstruction,
- Both roads must be concrete roads,
- Drainage of stormwater to flow towards the sea, overland and not in underground pipes
- Raise Pelegrini Street by approximately 200mm (the maximum amount, while taking into account existing properties and maintaining sufficient fall for drainage),
- Raise Popham Street to a minimum of +2.5m MSL,

- Maintain existing road width along Pelegrini Street – no need for a separate pedestrian facility.

5.1.3 Road design constraints

The following constraints were identified during the preliminary design stage:

- Popham Street is required to tie in with both Pelegrini Street and Ferguson Street – the road levels at these two points are therefore, fixed.

- Popham Street's levels are further limited by the existing adjacent property's drainage holes which need to remain above road level.

- The existing Pelegrini Street pinch point (at the corner) is very narrow and allows service vehicles (refuse trucks and fire trucks) one way access. Therefore, this width cannot be reduced.

- The house on the Pelegrini Street corner limits the amount to which the road can be raised.

5.1.4 Design codes

The following codes, standards and references will be applied for the design of the civil engineering design:

- Urban Transport Guidelines (UTG) 7: Geometric Design of Urban Local Residential Streets (1989)
- COTO: Standard Specifications for Road and Bridge Works for South African Road Authorities (2020)
- South African Road Traffic Signs Manual (SARTSM, May 2012)
- City of Cape Town: Standards and Guidelines for Roads and Stormwater (2020)
- SANS 1200 DM: Earthworks (Roads, Subgrade)
- COLTO – Standard Specifications for Road and Bridge Works for State Authorities
- South African Pavement Engineering Manual (SANRAL, 2013)
- Department of Transport Pedestrian and Bicycle Facility Guidelines (2003)

5.1.5 Pavement evaluation and design

Popham and Pelegrini Streets are low trafficked roads which primarily provide access to local residents. The roads are located where the sea can play a major role in "design fit for purpose" and as such, the design cannot be based solely on trafficking but must include this proximity to the sea. The coastal modelling has indicated that in large storm events, waves could be expected to crash onto the road surfaces. This loading is similar to what is experienced on pavements adjacent to quay walls. In addition to crashing waves, still water is expected to pond on the road surface for several hours during the peak of a storm.

Based on the roads' locations, a concrete road is preferred to a bitumen or gravel base road. Concrete is more durable, resistant to wind and water and would result in lower maintenance. The design life of the pavement, taking into account the additional life-time protection of a sea wall, will be a minimum of 30 years. The pavement layerworks proposed for the two roads are as follows:

- 160mm jointed concrete pavement (JCP) wearing course/base

- 100mm lean mix 10MPa concrete upper subbase interlayer

- 150mm C3 lower subbase

- Selected and subgrade materials

The City of Cape Town's previous service provider suggested a jointed concrete pavement on a bitumen stabilised gravel supporting upper subbase interlayer. However, a bitumen stabilised material is not considered an optimal solution in a water intense environment, and therefore a lean mix concrete has been proposed instead. An alternative to the JCP is a continuously reinforced concrete (CRC) pavement. CRC is generally more expensive than JCP, as there a large steel component, as well as additional labour. This also means that a CRC pavement takes longer to construct. In the case of Popham and Pelegrini Streets, both roads have horizontal bends which the CRC is not suited to. CRC pavements are used for straight sections of roads and piers. For these reasons, a CRC pavement was not considered a viable option for these roads.

The JCP will be designed to prevent longitudinal or transverse movement. This will be done using steel ties at joints, including into the seawall. This design will be undertaken during the detailed design stage of the project. Ensuring the concrete panels are correctly tied into each other and the seawall will provide optimal stability during storm events and wave loading.

In addition to the proposed layerworks, a portion of the backfill material which is closest to the seawall needs to be a low fines material, such as crushed rock. This will prevent the loss of fines through or underneath the wall which would lead to significant cavitation and possible road collapse. This rock fill will be separated from the in-situ fill material using a geofabric filter/reinforcement layer. This design can be seen in drawing HHO-7462-701-1101 in **Annexure H**.

5.1.6 Popham Street geometric design

Popham Street runs from its intersection with Sir David Baird Drive in the south-east to De Mist Street in the north. The portion of road which is being reconstructed is shown in **Figure 5.1**. The existing road is approximately 6.5m wide, with a 3m sidewalk on the seaside and a 1.3m sidewalk against the houses. The road has an existing low point on the corner with Pelegrini Street (at SV35), towards which all stormwater from south of Ferguson Street and from the public parking lot to the east, flows. The road is in cross fall with a slope ranging from 1 to 3%. The longitudinal grade is approximately 1.8% from the low point travelling east; and 2.4% from the low point towards Ferguson Street.

The proposed horizontal road alignment was required to match the existing road, in terms of lane widths. The north bound lane is 3.2m wide and the south bound lane is 3.5m wide. The proposed sidewalk on the seaside extends from the road edge until the proposed seawall coping. This is approximately 5m wide and widens further to the north of Ferguson Street, as the seawall alignment changes. The future phase of the seawall (when it is raised to 4.5m), will reduce the sidewalk width to approximately 1.5m at its narrowest. Details of this future phase were considered in the traffic study, which is described in more detail in **section 5.1.7** below.

The proposed vertical road alignment made allowance for the client design requirement to raise the road to a minimum height of +2.5m MSL. In order to maintain a minimum longitudinal grade of 2%, an intermediate high point was created along the road at SV40. This can be seen on drawing HHO-7462-701-2001 in **Annexure H**. This will result in two low points, from where stormwater and overtopping sea water will flow towards the seawall. The minimum crossfall along the road is 0.5%, which has been deemed sufficient for overland drainage. The proposed road contours can be seen on drawing HHO-7462-701-1622 in **Annexure H**. The proposed alignment ties in with the existing Ferguson Street intersection. Drainage from Ferguson Street will no longer be via the catchpit on the Popham corner (which will now be removed), but rather overland towards the seawall where it will exit through one of the gaps in the cope unit.

5.1.7 Phase 2 seawall – Popham Street

As discussed in **section 4.5.1**, the seawall has been designed for a future height of 4.5m, with the walkway at 3.5m. This seawall raise impacts the cross section of the road, specifically the sidewalk width, from which access onto the seawall walkway needs to be accommodated. A traffic study was conducted to consider the impact of this future phase on the existing road. Two options were considered to address accessibility:

i. A ramp access to the sidewalk be provided at the start and endpoints of the elevated section

ii. Conversion of the existing two-way road to a one-way road as to accommodate a continuous stairway parallel to the sidewalk, i.e. available road space is reconfigured.

The first option would not allow for continuous access to the seawall walkway along the road. However, it would prevent the need for converting the road into a one-way. Ramps would be accommodated where sufficient sidewalk width is available. At narrower sections, the sidewalk would be a minimum of 1.5m wide, with a side wall of approximately 1.1m running parallel to the back of the sidewalk.

The second option was a client proposal which allows for the elevated seawall walkway to be continuously accessible via a parallel stairway. Accommodating the continuous stairway would require reconfiguring the available road space with a reduction of the vehicle lanes from two to one. An assessment was undertaken to evaluate the impact of the conversion on property access and egress. An ‘area of influence’ was determined by identifying the extent of properties that are within the immediate vicinity of the converted link, taking into account the degree of impact on ‘direct’ access or egress, where ‘direct’ refers to the shortest path between the property and the nearest higher-order route. A property impact legend as follows.

Table 5.1: Property Impact Legend (One-way conversion)

Low Impact	**Medium Impact**	**High Impact**
No direct property access/ egress route was impacted by the conversion.	One direct property access/ egress route impacted by the conversion.	Two direct property access/ egress routes were impacted by the conversion.

Sir David Baird Drive was identified as the nearest higher-order route. The one-way conversion is classified as a high impact, as several houses within Ferguson Street would no longer have access to Sir David Baird Drive via Popham Street. Instead, residents would be required to travel via De Mist Street.

Following TMH 16 Volume 2 (COTO, 2014), one-way conversions are a means of managing external traffic flow through a residential area. The motivation to convert this section of Popham Street to a one-way is not related to traffic management and thus TMH 16 Volume 2 (COTO, 2014) does not apply in this context. The benefits of the conversion will need to be scrutinised by the needs of the community through an appropriate public participation process.

Further information can be found on the traffic study in **Annexure F**.

5.1.8 Pelegrini Street geometric design

Pelegrini Street is an existing concrete road which runs from its intersection with Popham Street to the corner of the play park seawall. The continuation of Pelegrini Street past the park is an asphalt surfaced road. The portion of road which is being constructed is shown in **Figure 5.2**. The existing road varies in width from approximately 6 to 10m.

There is no dedicated sidewalk as the road is a shared facility for vehicles and pedestrians. The road currently has a crossfall of 3.5%, which results in a significant height difference from the left edge to the right edge. The existing longitudinal grade of the road varies from 0% to 3.7% with a low point at SV45 and SV55.

The proposed horizontal road alignment maintains the status quo width. This was a specific requirement to allow for service vehicles such refuse trucks and firefighting vehicles to traverse the roadway. **Figure 5.3** below represents a simulation check that was done using AutoTURN, to confirm the minimum allowable width at the Pelegrini corner for such a vehicle. The horizontal alignment of the seawall and road were carefully coordinated to maintain the existing width, while not encroaching past the existing coastal development line.

Figure 5.3: AutoTURN simulation of Pelegrini corner

The proposed vertical alignment aimed to raise the level of the roadway approximately 200mm, while still tying into existing boundary conditions along the residential property walls. In order to achieve this, the crossfall of the road was reduced – ranging between 0.5% and 1.5%. In addition, a single low point was developed at SV60. This can be seen on drawing HHO-7462-701-2001 in **Annexure H**. While the low point could have remained at the existing position (SV45-SV55), the future phase of the seawall will not accommodate overland drainage. Creating drainage through the seawall will then be necessary. The corner of the road receives the highest wave impact during a storm and therefore, is not an optimal position for a stormwater outlet. Thus, the future outlet will be positioned around the corner at SV60, in line with the proposed Phase 1 low point. The proposed road contours can be seen on drawing HHO-7462-701-1622 in **Annexure H**.

5.1.9 Ancillary road design details

Both Popham and Pelegrini streets currently make use of 'half penny' bollards. In Popham Street, these separate moving vehicles from pedestrians on the sidewalks. In Pelegrini Street, the bollards provide a barrier along the seawall edge to assist in preventing vehicles from driving over the edge. The proposed seawall makes use of a cope unit which in the case of Pelegrini Street will function in the same way, and therefore no additional barrier is required. Along Popham Street, the roadway will be at the same level as the sidewalks – using one continuous concrete surface. Bollards are not the preferred method for designating zones, as they can be dangerous projectiles when impacted by a moving vehicle, as well as often become detached from their footing creating additional road maintenance.

It is, therefore, recommended that a red line is painted onto the road edge and that delineator kerbs are placed along the road edge next to the line. These kerbs will have gaps in between to allow the flow of the overland drainage. In addition, several benches can be placed along the sidewalk parallel to the roadway in order to increase the visual barrier for road users.

5.2 Foul Sewer Realignment

The existing DN150 foul sewer pipe gravity feeds from the ablution block at De Mist parking to the foul sewer pump station in the Small Bay play park. The alignment follows the existing seawall along Popham Street, then travels along Pelegrini Street and finally crosses the Small Bay beach, where it has recently been undermined. Several foul sewer manholes are located within the existing seawall and are showing signs of failure, with cracks widening every month. With the reconstruction of the seawall and adjacent roads, the sewer pipe is required to be realigned along Popham and Pelegrini Street.

In addition to the existing DN150 pipe, another foul sewer pipe which is located parallel to the Small Bay play park wall has been earmarked for a possible realignment. The realignment of the existing DN200 HDPE pipe may be required to allow for the construction of the proposed seawall.

5.2.1 Client Requirements

The Client requirements were as follows:

- Upgrade the existing DN150 pipe along the Popham Street seawall to a SDR34 uPVC DN200 pipe, which must also be realigned to run within the north bound lane in Popham Street.
- Tie in to the existing DN150 pipe at the De Mist parking ablution block.
- Avoid other service relocations if possible.
- Avoid any pump station configuration changes.
- Determine the need for and feasibility of realigning the existing DN200 HDPE pipe along the play park seawall.

5.2.2 Design constraints

The following constraints were identified during the preliminary design stage:

- The existing sewer pipe is contained for a large section within the existing seawall which will be demolished before the proposed pipe is constructed.
- The existing pump station has several incoming pipes which combine at one inlet manhole – making this a particularly difficult area to work in.
- Three residential sewer connections enter the existing sewer pipe along Pelegrini Street.
- The proposed Small Bay play park seawall requires significant excavation to reach the bedrock level, resulting in a large cut back slope which impacts the existing DN200 sewer line.

- Five residential sewer connections enter the existing DN200 sewer pipe between the play park and Ons Huisie (upstream point).

5.2.3 Design codes

The following codes, standards and references will be applied for the design of the civil engineering design:

- SANS1200LD: Sewers
- SANS 1200DB: Earthworks (Pipe trenches)
- City of Cape Town: Standards and Guidelines for Roads and Stormwater (2020)
- Guidelines for Human Settlement Planning and Design Vol 2 (2005)

5.2.4 Proposed uPVC DN200 alignment

The position of the proposed pipe's tie in at its most upstream point considered two options:

- Adjacent to the De Mist ablution block – this option requires shoring along Popham Street and the reinstatement of a retaining wall after pipe trenching is completed. The invert levels associated with this option allow for gradients of 1:150 and 1:100 along the entire pipe route. This option is shown in **Figure 5.4** below.

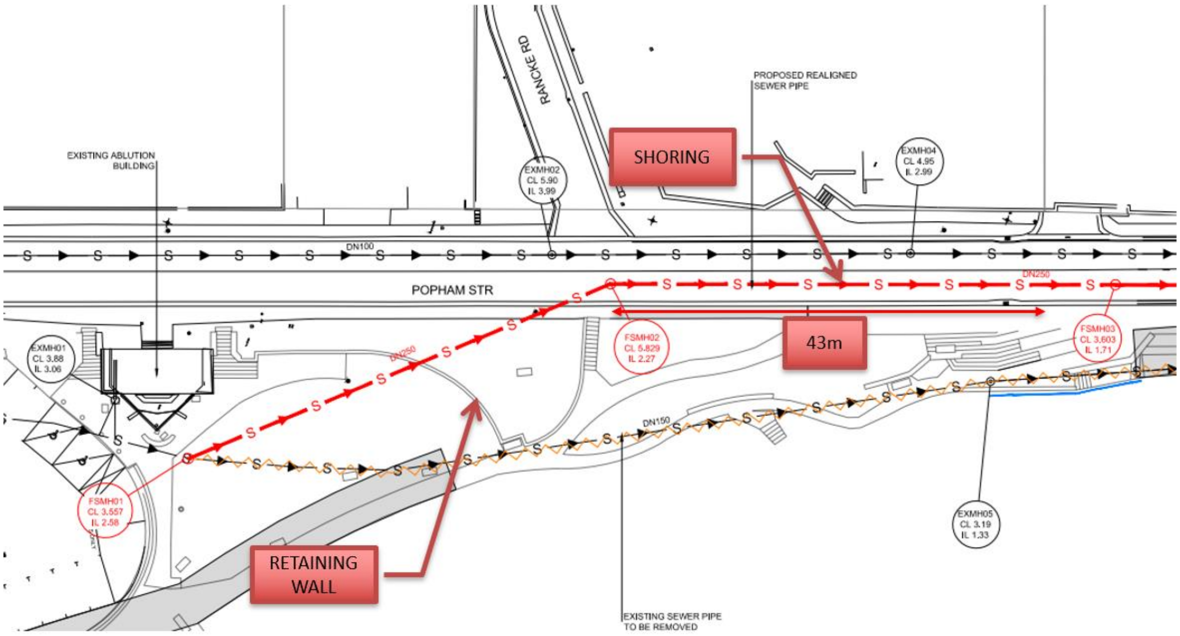


Figure 5.4: Option 1 – DN200 uPVC tie in at De Mist ablution block

- At EXMH05 – this option was considered as a cost saving alternative. There would be a reduction in shoring along Popham Street. However, in order to connect EXMH05 to the proposed manhole in Popham Street, the demolition of an existing concrete staircase and associated walkway would be required. Following this, the seawall would need to be extended up to EXMH05. This results in significant extra cost. The invert levels associated with this option will result in a gradient of 1:200, which is not ideal. During low flows, blockages could take place. The deeper pipe trenches will also have an associated construction cost. This option is shown in **Figure 5.5**.

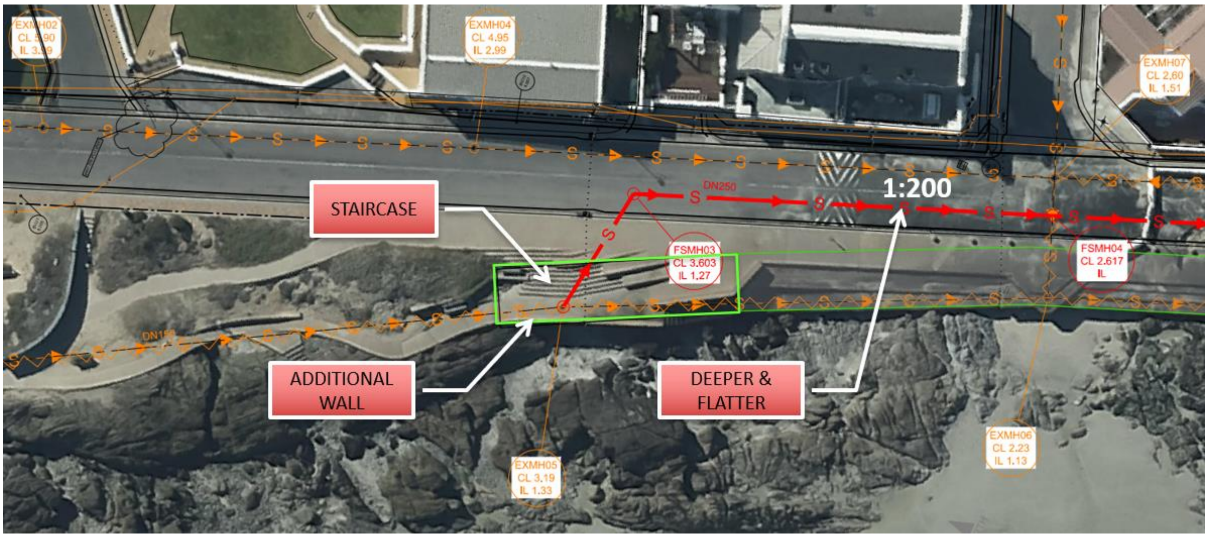


Figure 5.5: Option 2 – DN200 uPVC tie in at existing manhole EXMH05

Following a foul sewer design workshop, **it was decided by CoCT Water and Sanitation that the De Mist ablution block would be the preferred location based on the advantages and benefits presented by HHO.**

The proposed alignment is shown on drawing HHO-7462-701-1602 (**Annexure H**) and should be a visual reference for the works. The proposed SDR34 uPVC DN200 pipe starts at FSMH01 alongside the De Mist parking ablution block. This new manhole will be built using precast rings which are sunk over the existing DN150 sewer pipe. This allows the existing pipe to continue operating. From this manhole the pipe travels south-east towards Popham Street at a grade of 1:150. Once the pipe reaches Popham Street, it travels south in the north bound lane of the road. FSMH04 ties into the existing DN150. Following this, the sewer pipe continues along Popham Street until FSMH06, at which point the pipe changes direction into Pelegrini Street.

The foul sewer along Pelegrini Street has been specifically offset from the seawall by 1m. This will allow enough space for manholes FSMH08 and FSMH09 to be built, while not resulting in deep excavations which are too close to existing building foundations. The sewer continues around Pelegrini corner before crossing the corner of the park and terminates at FSMH12. This manhole will be built in the same manner as FSMH01, to prevent unnecessary disruption.

The coordination of the proposed sewer construction needs to be carefully planned to prevent any service disruption to local residents. Significant amounts of over pumping will be required to transfer sewage directly to the pump station. The construction of the sewer is discussed further in **chapter 7**.

5.2.5 Proposed HDPE DN200 realignment

The existing DN200 HDPE sewer pipe is located parallel to the play park seawall until the southernmost corner, at which point the pipe changes direction at 90-degree bend and passes through the seawall towards the sea. A full-bore rodding eye is also located at this corner. The pipe is class PN100 and butt-welded, making it fairly flexible. The pipe on the seaward side of the wall is encased in mass concrete which, according to previous design drawings from 2019, is founded on bedrock. The proposed play park seawall (described in **section 4.5.2**) is founded on bedrock which is approximately 4m deep. This will require a large cut back slope during construction which will lead to the existing sewer pipe being undermined. A foul sewer design workshop was held with City officials to discuss the feasibility of realigning the foul sewer pipe.

Two alignment options were considered:

- Option 1 proposed a new manhole FSMH15 (shown below in **Figure 5.6**) would be built on the existing HDPE pipe, from which point the pipe would be diverted away from the seawall to FSMH14. From this manhole, it would run parallel to the seawall, connect to an existing DN100 pipe at FSMH13 and then enter directly into the pump station wet well via a new core drilled hole. This option had the following advantages and disadvantages:
    - The proposed manhole FSMH15 will be susceptible to future wave damage and is difficult to build,
    - The proposed realignment will protect the pipe from damage during construction
    - The existing manholes, EXMH13 and EXMH15 are due for an upgrade if they were to continue being used – in this option they will be demolished as they are no longer required

Figure 5.6: Option 1 – DN200 HDPE diversion at FSMH15

- Option 2 proposed leaving the existing PN100 HDPE pipe in its current location up until a proposed manhole FSMH16 (shown in **Figure 5.7**). From this manhole the pipe would be diverted away from the seawall to a new manhole FSMH14 built on the existing DN100 foul sewer pipe, from which point it would enter directly into the pump station wet well via a new core drilled hole. The advantages and disadvantage of this option are as follows:
    - The existing pipe can continue to be used
    - The length of pipe proposed is shorter than Option 1

- The existing pipe will need protection during construction and will be at risk of damage from heavy machinery
- The pipe location prevents a future concrete slab being installed for protection against wave overtopping
- The existing manholes, EXMH13 and EXMH15 are due for an upgrade if they were to continue being used – in this option they will be demolished as they are no longer required

Figure 5.7: Option 2 – DN200 HDPE diversion at FSMH16

The two options do not have a large cost difference, and therefore the construction risk, constructability and long-term risk of damage were the key factors. Taking into consideration the long-term operational benefits by re-routing the existing sewer pipe more landwards, the final preferred design selected by Water & Sanitation and supported by the Coastal Management was a revised Option 1. Instead of using a manhole in the southern corner, the existing rodding eye will be replaced with a butt welded 45-degree bend. The pipe will then be rerouted away from the seawall, and pass through another butt welded 45-degree bend before reaching a proposed manhole on the existing DN100 foul sewer pipe. Both bends will have a full-bore rodding eye installed that will ensure blockages can be cleared. These rodding eyes will be installed inside DN1000 precast manhole rings that are fitted with a cover. This will protect the rodding eye from wave overtopping damage and allow for a concrete walkway slab to be installed along the back of the seawall. The proposed design can be seen in drawing HHO-7462-701-1602 (**Annexure H**).

The construction of this realignment will take place before the existing seawall is demolished. The installation of the first butt welded bend will need to take place at a low flow period, i.e. the middle of the night. In addition, sewage upstream will be overpumped at Ons Huisie using a truck. In addition, over pumping will likely be required from EXMH16 directly to the pump station. The construction of the sewer is discussed further in **chapter 7**.

5.2.6 Ancillary sewer design details

A custom manhole detail is intended to be used at several locations in the project where a proposed pipe ties in with an existing pipe at the same invert level. The manhole will be constructed using precast rings which are sunk into the ground without any large excavation. Once the rings reach the top of pipe level, the internal portion of the manhole will be excavated to below the pipe, then filled with in situ concrete with allowance made for benching and the proposed future pipes. Once the proposed pipe is in place, the existing pipe can be cut open, with the outgoing pipe blocked up. A detail of this manhole can be seen on drawing HHO-7462-701-2501.

6 UPGRADE TO DE MIST PARKING AREA

6.1 Seawall Retreat – Parking Optimisation

The initial Client requirement for the De Mist parking seawall was to retreat approximately 5m landwards. A traffic study (included in **Annexure F**) was undertaken to consider the existing parking availability, parking demand and propose a parking lot concept design.

A parking survey was undertaken by HHO Consulting Engineers on Saturday 17 April 2021 as part of the traffic data collection exercise. The total survey duration was 3 hours (11h00 – 14h00) and observations were carried out in 30min intervals. The objective of this survey was to capture the parking demand at the De Mist parking area. The existing parking lot has the following bays:

- 71 normal bays
- 2 disabled persons bays
- 5 trading bays (3 small, 1 medium and 1 large)

The demand survey had the following findings:

- Parking demand across the full count duration (3 hours) on a weekend midday peak period exceeded the provided capacity. The overspill parking demand was observed to initially circulate the parking area before navigating toward nearby parking lots located along Popham Street and behind the main entrance of The Blue Peter Hotel. The existing parking capacity is thus not sufficient for the demand and any proposed reduction of parking will have a negative impact on the users.
- No on-street parking is allowed on streets in the vicinity of the parking area, as indicated by red (no stopping) lines. From the observations, this is adhered to by motorists. It is assumed that overspill demand is accommodated on Sir David Baird Drive, which has extensive embayed parking in the vicinity.
- It can be reasonably inferred that during peak recreational periods, the parking area would be continually fully utilised.

Based on the findings of the study, the concept design's main objective was to mitigate the impact of the realigned seawall on the De Mist parking by ensuring that any lost parking could be reinstated within a reasonable vicinity of the impacted area. The proposed design (which can be seen in **Figure 6.1**) resulted in a loss of 20 bays, of which 4 were trading bays. The traffic study considered alternative locations for the lost parking as follows:

i. A total of 9 bays could be reinstated along De Mist Street; however, this would require the conversion of a section of Popham Street into a one-way road.

ii. A total of 21 perpendicular bays could be reinstated along Verkouteren Street; however, this would likely impact the character of the cul-de-sac and require widening of the road infrastructure seaward.

Following the traffic study and its findings, the Client decided that parking could not be accommodated elsewhere and therefore, the seawall should not retreat. The design of the seawall (as described in **section 4.5.4**) was based on this change.

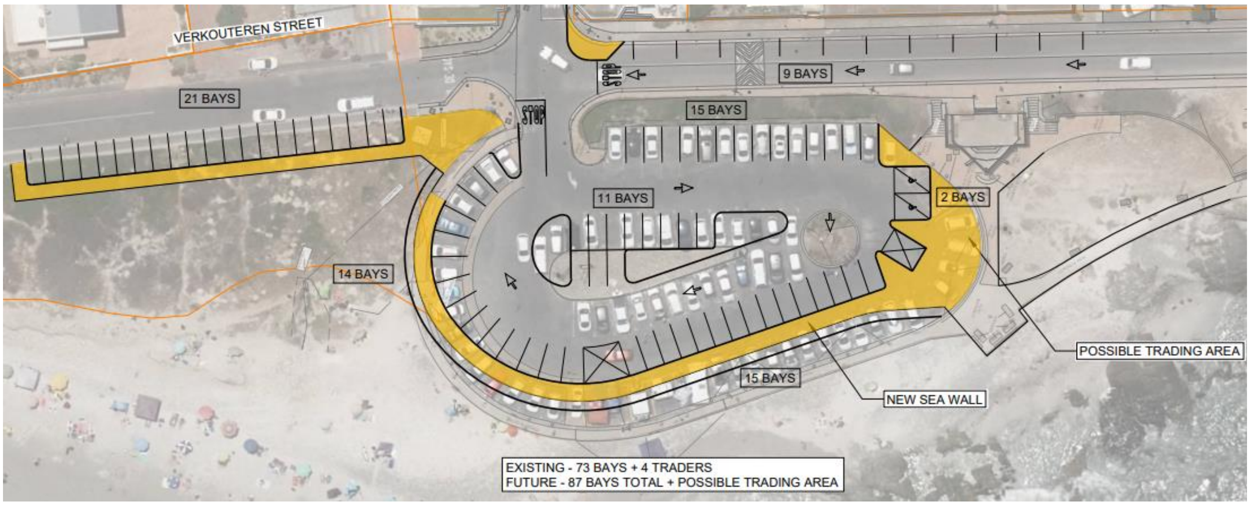


Figure 6.1: Proposed parking layout – seawall retreat

6.2 Revised Design – Parking Optimisation

The proposed De Mist seawall, despite not retreating, impedes into the existing parking lot. A preliminary design was completed in order to optimize the parking lot layout. This proposed layout has increased the parking lot capacity. The number of bays available would be as follows:

- 77 normal bays
- 2 disabled persons bays
- 4 trading areas (2 large, 2 medium)

It would be possible to convert some normal bays into additional trading areas, if there is a demand. The proposed layout can be seen in **Figure 6.2**.

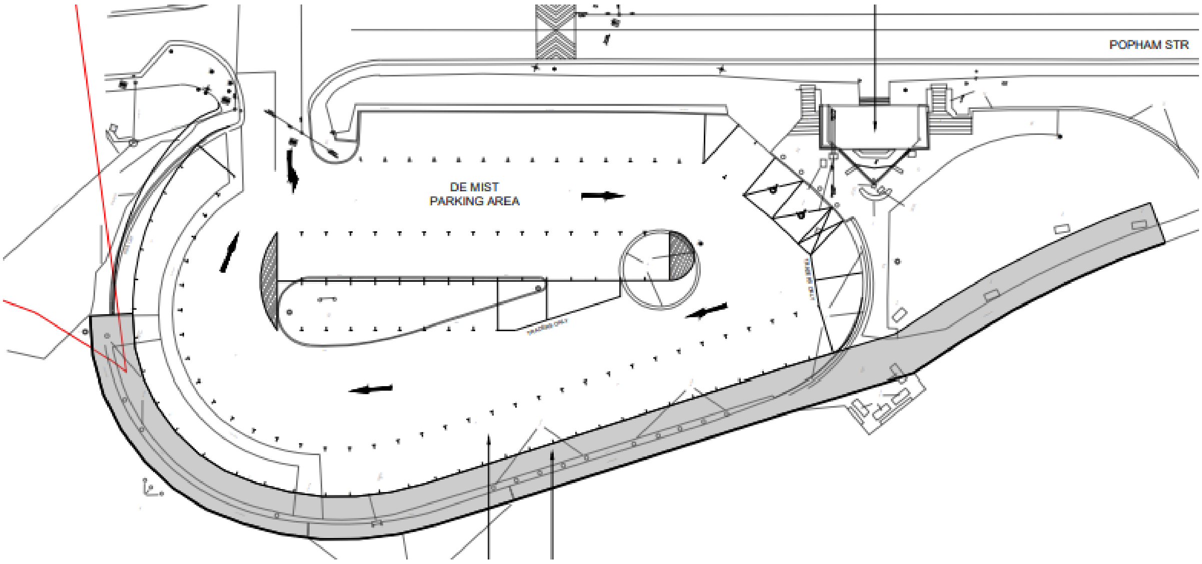


Figure 6.2: Revised parking layout – no seawall retreat

6.3 Project Scope Change

Following the completion of the preliminary design and the detailed costing, as presented in **Chapter 9**, the Client decided to omit the De Mist parking upgrade from the project scope due to budget constraints.

7 CONSTRUCTION DETAILS

7.1 General

The preliminary design of the seawall considered the following with respect to the construction of the works:

- Location and proximity to existing infrastructure:
    - Residential infrastructure
    - Existing services
    - Traffic accommodation
    - Limited space and storage areas
- Temporary works required during construction:
    - *Protection of seawall construction against wave action:* One of the challenges of constructing the new seawall is the exposure of the site to wave action, especially during high tide conditions. The seawall will be constructed in sections of between 10 m and 20 m long. During construction, these sections will need to be protected against wave action. It is recommended that the temporary works include the construction of a Geobag bund wall along the seaward face of the cofferdam. The Geobags will need to be large enough to withstand the wave attack and these bags should be at least 2.2m x 1.4m x 0.4m high and when filled will have a weight of approximately 1.8 tons. These Geobags will then be reused for the next section of coffer dam.
    - *Water levels and dewatering:* The seawall needs to be founded on competent bedrock cleaned of all sand, marine growth and loose material. The bedrock level varies along the seawall alignment from approximately -1.30 m MSL to +1.40 m MSL with the average level being approximately +0.30 m MSL. This means that a large portion of the seawall foundation will be underwater especially during the high tide conditions. The Geobag bund wall will not create a dry coffer dam and the contractor will need to deal with water during the seawall foundation construction. In certain areas the contractor will be restricted to working during low tide conditions and they will also most likely need to pump water out of the cofferdam before they can carry out their work. When casting the concrete blinding layer in the lower bedrock levels, the contractor may need to tremie the concrete blinding layer. The top of the contrate blinding layer is at + 0.4 m MSL which is above most of the tidal range, except for the high tide conditions. The top of the lower seawall block is at +1.4 m MSL which is above the full tidal range. The rock anchors will be installed from the top of the lower block so that this process is not disrupted by tides.
    - *Stabilisation of excavations landwards*: Slopes will be cut back at 1:1 and stabilised using sand bags. The proposed sewer pipe along Popham and Pelegrini Street has been specifically positioned to prevent any damage to or undermining of existing infrastructure and residential houses.
    - *Shoring of pipe trenches:* The construction of the proposed DN200 sewer beneath the existing asphalt surfaced Popham Street will require shoring, in order to minimize the width of excavation and damage to the parallel DN150 sewer line in the south bound carriageway.
- Construction method:

- *Precast or insitu concrete:* In previous studies it was suggested that the seawall be constructed using precast concrete blocks. It is, however, recommended that the seawall be constructed with insitu concrete for the following reasons:
    - Disadvantages of insitu Concrete:
        - Longer time working on site in a coffer dam
        - Potential construction disruptions during storm events
        - Surface finish may not be as good/uniform as precast units
    - Advantages of insitu concrete:
        - No need for large cranes to place units (up to 15t) especially with the very restricted access at the site.
        - No need to grout uneven surface between base slab and precast unit. Alleviate spanning and over stressing of precast units
        - Easier to manage joints between blocks (no need to grout or seal joints)
        - Better interlocking between blocks.
        - No need for special units at bends in the wall.
        - Relatively easy to pump and place concrete.
        - Easier to vary the thickness of the top block in order to align with the varying road levels
        - Easier to accommodate the existing stormwater pipe outlets (i.e. cast into the wall)
        - No need to store precast blocks on site before installation
- *Custom sewer manhole:* In order to prevent disruption to existing sewer lines, as well as minimise the excavation required in confined areas, a custom sewer manhole detail has been proposed (as described in **section 5.2.6**). This will reduce the amount of over pumping required and prevent damage to nearby infrastructure and properties.

7.2 Construction Sequence

The general construction sequence of the seawall and adjacent infrastructure can be described in the following steps (see **Figure 7.1** for reference to steps listed below):

A. Site clearance and demolition of existing seawall.

B. Construct temporary bund wall with Geobags to protect the works from wave action. Between 10m and 20m length wall sections to be constructed at a time.

C. Excavate down to bedrock and clean all sand, marine growth and debris for foundation preparation. It may also be required to break back any weathered or highly jointed loose rock that is not suitable for founding on.

D. Cast insitu concrete blinding layer up to + 0.4 m MSL. The contractor may need to tremie concrete areas where the bedrock is deeper and that cannot be dewatered.

E. Cast the lower concrete block insitu, up to a level of +1.4 m MSL.

F. Drill from the top of the lower block for the rock anchors and install rock anchors.

G. Place bedding layer and install sewer pipeline.

H. Backfill behind the seawall.

I. Cast the upper seawall block insitu, up to the final road level.

J. Construct the road and road concrete surface.

K. Install 1m high precast concrete cope wall along the edge of the seawall.

Figure 7.1 shows a typical section through the proposed seawall along Pelegrini Street and is used to illustrate the construction sequence.

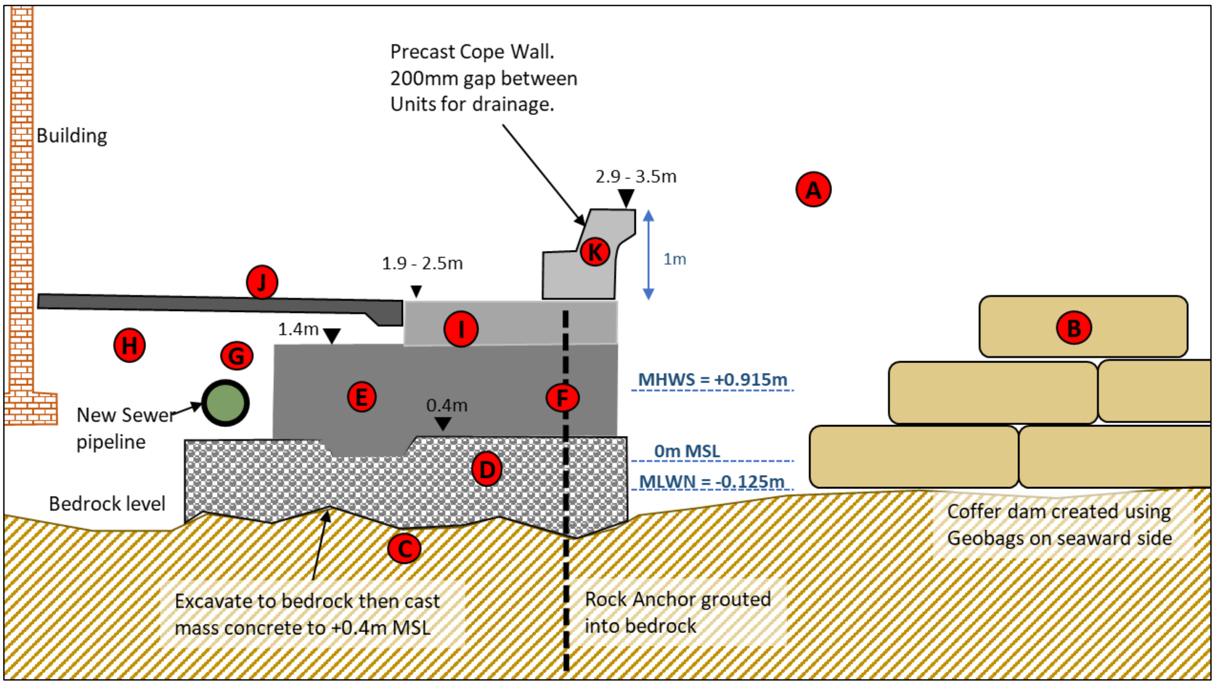


Figure 7.1: Typical section through seawall at Pelegrini Street.

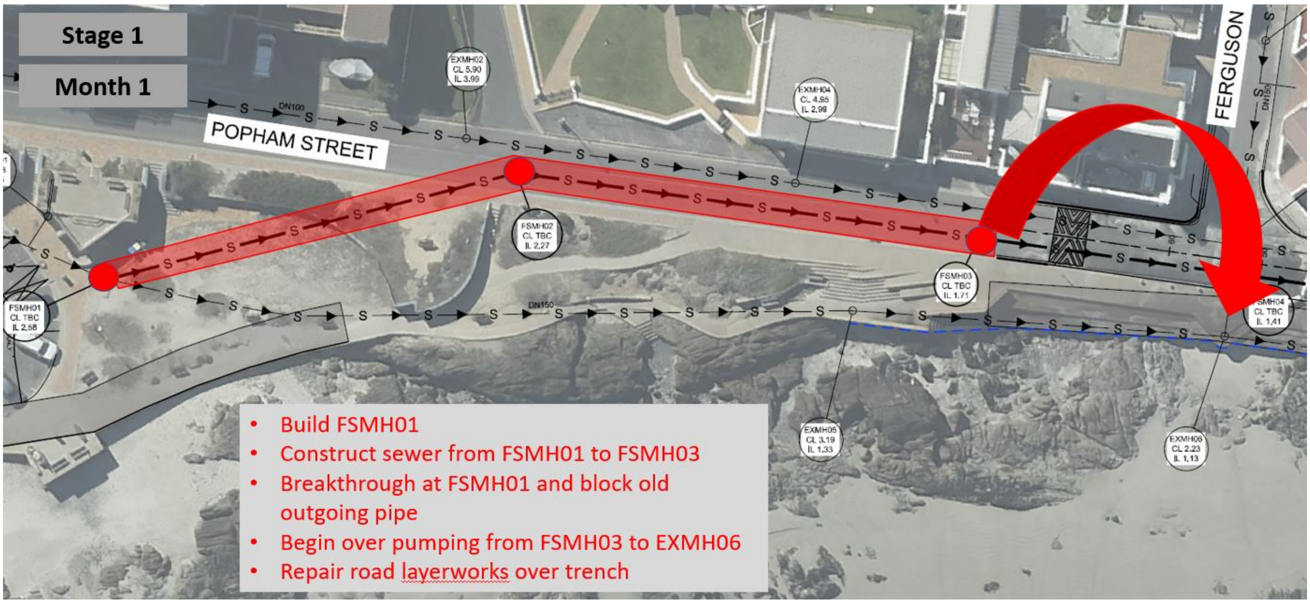
7.3 Construction Programme

A high-level program has been formulated to determine the stages of the construction and the amount of time per stage. The proposed construction period is estimated at 16 months, based on the challenging working environment. The three main components – seawall, roadworks and sewer are interdependent during construction. In order to reduce the construction period, additional teams would be required. However, this may be limited due to NEMA's temporary works requirements.

Figures 7.2 to **7.7** illustrate the five main stages of the construction process, which align with the traffic accommodation discussed in **7.4.1**. The De Mist parking lot seawall and associated works have been assumed to take place in parallel with the rest of the works. Therefore, this work is not allocated to a specific stage, and is shown to take place at the beginning from months 1 to 4. **Table 7.1** provides a summary of the construction programme.

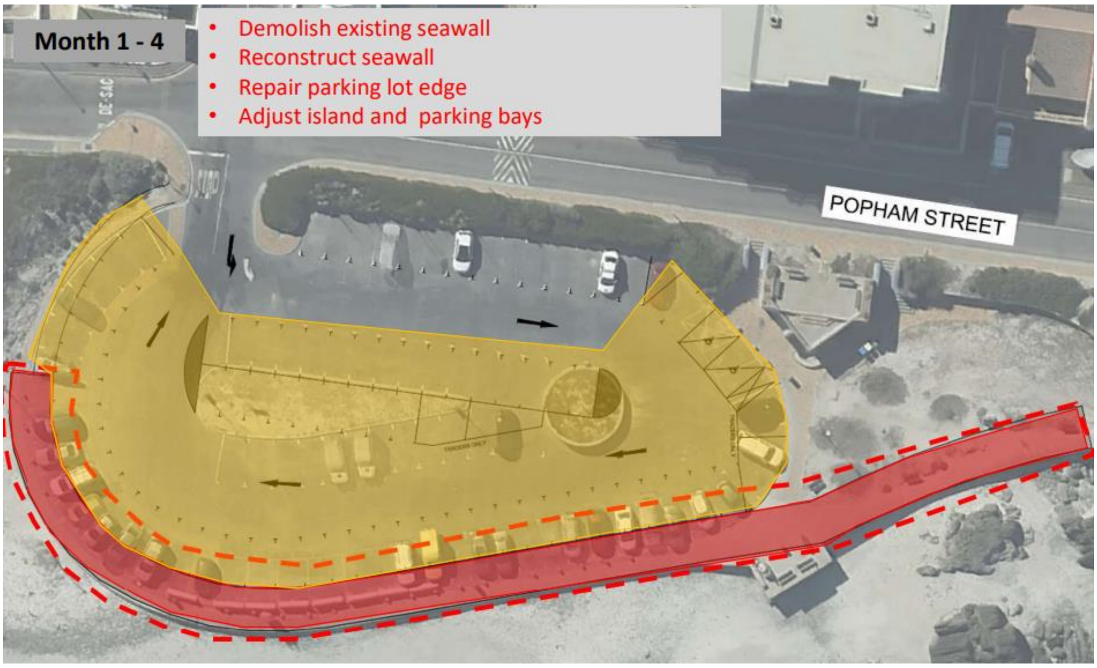
Table 7.1: Construction program summary

Stage	Description of construction activities	Months
De Mist parking lot	• Seawall construction • Parking lot construction	4
Stage 1	• Foul sewer (FS) relocation • FS over pumping • New FS construction • Restore road layer works and surface	1
Stage 2	• Demolitions • FS over pumping • Road, new FS and seawall construction	2
Stage 3	• Demolitions • FS over pumping • New FS construction • Road, new FS and seawall construction	3
Stage 4	• Demolitions • FS over pumping • New FS construction • Road, new FS and seawall construction	7
Stage 5	• Demolitions • FS over pumping • New FS construction • Road, new FS and seawall construction	3
Total Duration		**16**



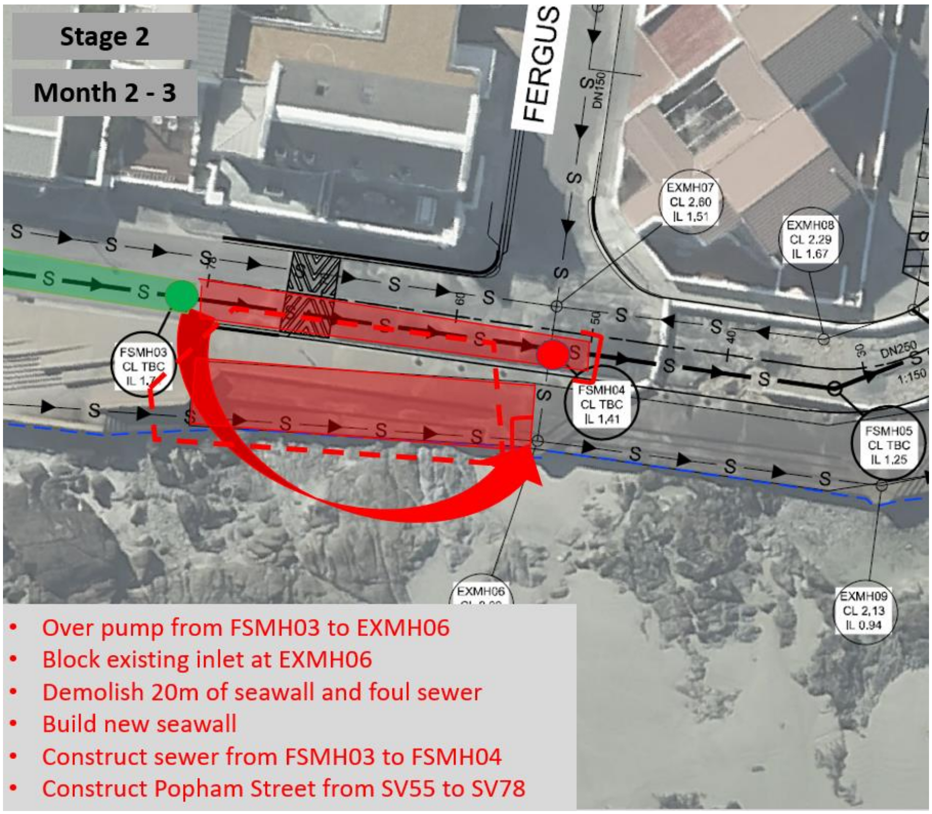
- Build FSMH01
- Construct sewer from FSMH01 to FSMH03
- Breakthrough at FSMH01 and block old outgoing pipe
- Begin over pumping from FSMH03 to EXMH06
- Repair road layerworks over trench

Figure 7.2: Stage 1 of construction (1 month)



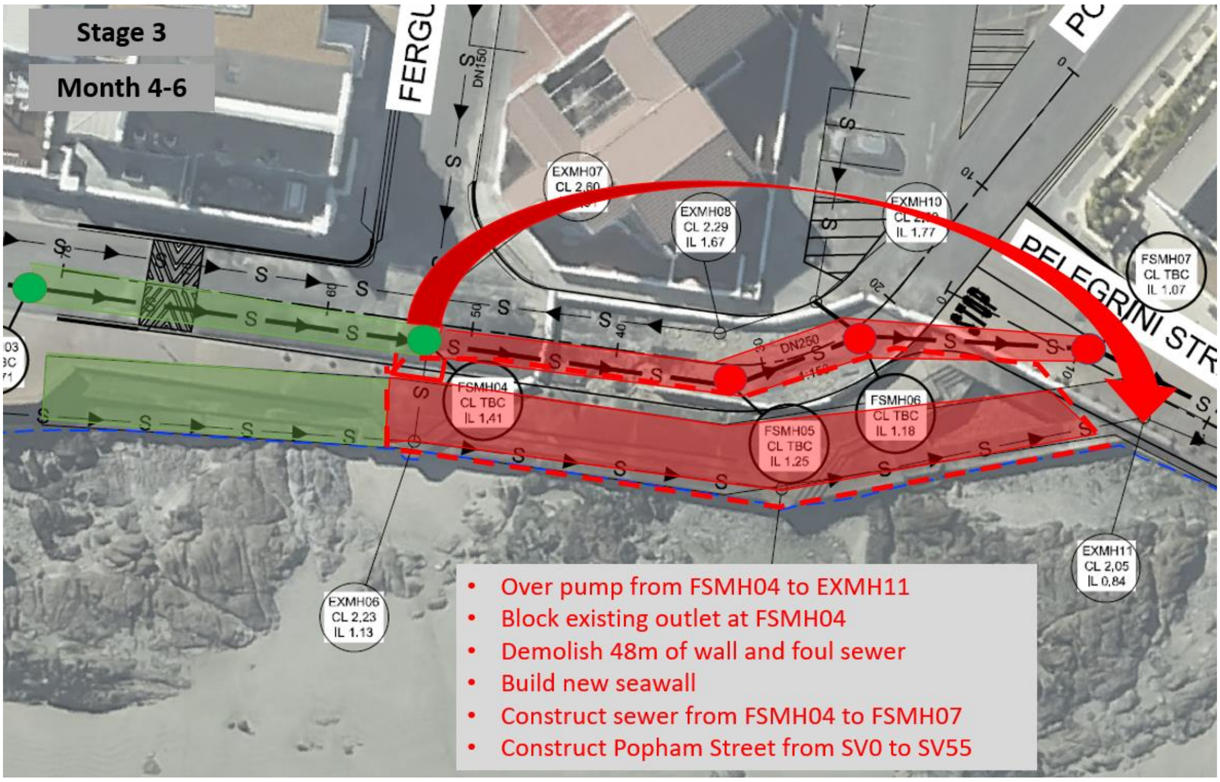
- Demolish existing seawall
- Reconstruct seawall
- Repair parking lot edge
- Adjust island and parking bays

Figure 7.3: Construction of De Mist parking lot (4 months)



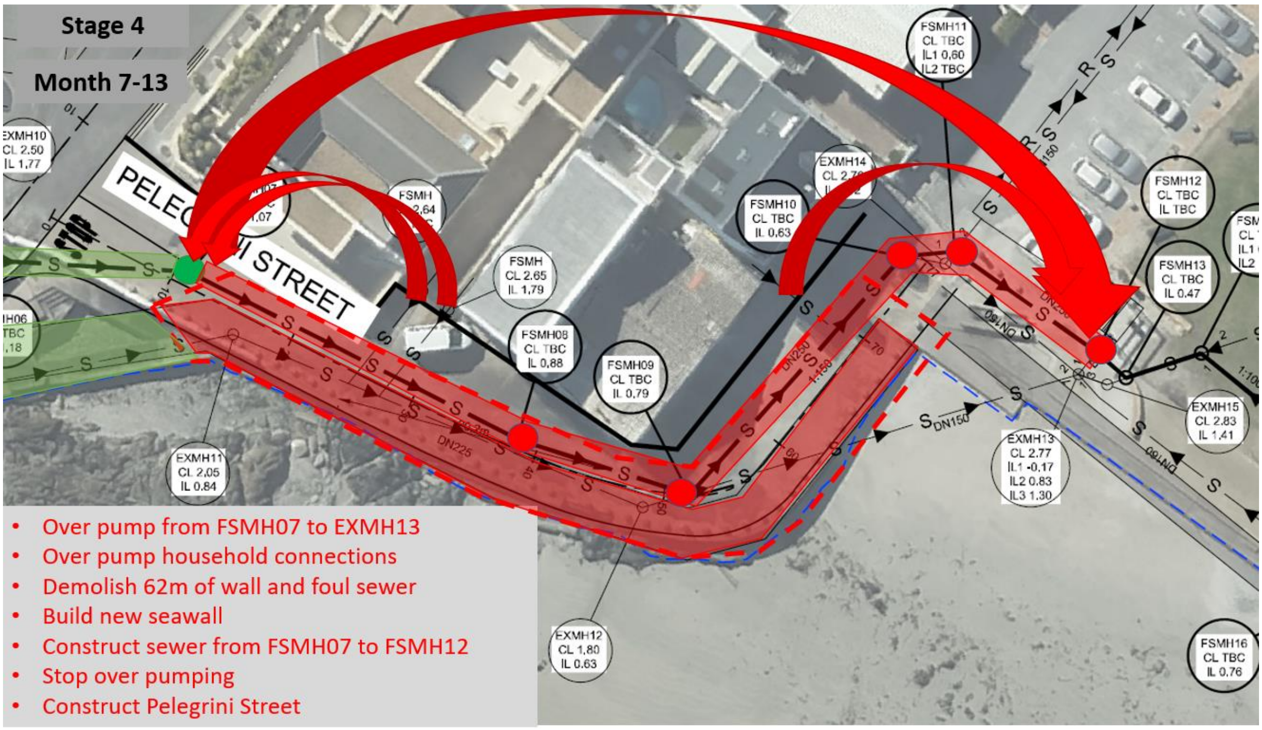
- Over pump from FSMH03 to EXMH06
- Block existing inlet at EXMH06
- Demolish 20m of seawall and foul sewer
- Build new seawall
- Construct sewer from FSMH03 to FSMH04
- Construct Popham Street from SV55 to SV78

Figure 7.4: Stage 2 of construction (2 months)



- Over pump from FSMH04 to EXMH11
- Block existing outlet at FSMH04
- Demolish 48m of wall and foul sewer
- Build new seawall
- Construct sewer from FSMH04 to FSMH07
- Construct Popham Street from SV0 to SV55

Figure 7.5: Stage 3 of construction (3 months)



- Over pump from FSMH07 to EXMH13
- Over pump household connections
- Demolish 62m of wall and foul sewer
- Build new seawall
- Construct sewer from FSMH07 to FSMH12
- Stop over pumping
- Construct Pelegrini Street

Figure 7.6: Stage 4 of construction (7 months)

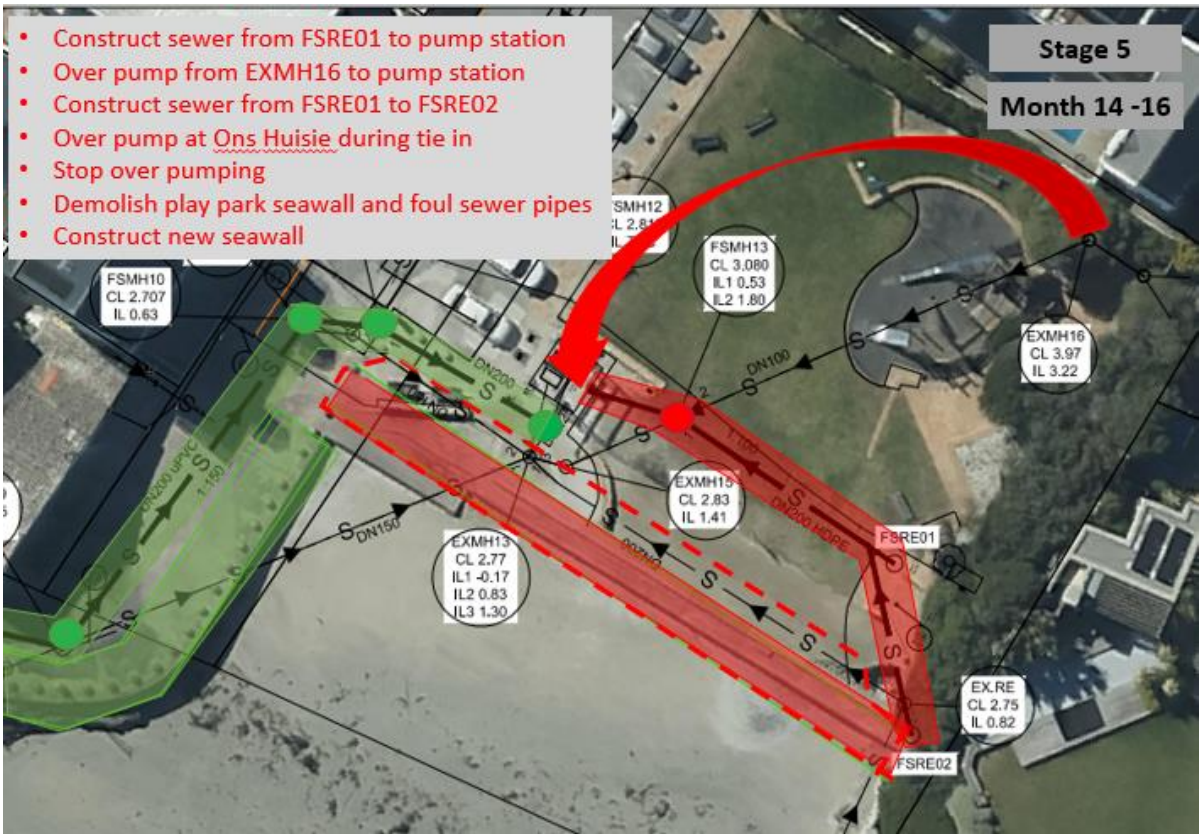


Figure 7.7: Stage 5 of construction (3 months)

7.4 Social and Environmental Considerations

In addition to the construction methodology, the work to be carried out must also consider all social and environmental requirements. The following aspects must, therefore, be considered:

- Traffic accommodation during construction
- Compliance with health and safety requirements
- Adherence to environmental regulations, specifically NEMA

7.4.1 Traffic accommodation

In order to formulate a traffic accommodation plan for the local residents during construction, it was first necessary to identify the vehicular access points associated with each property that may be affected by the construction. **Figure 7.8** below indicates these access points with a red arrow.

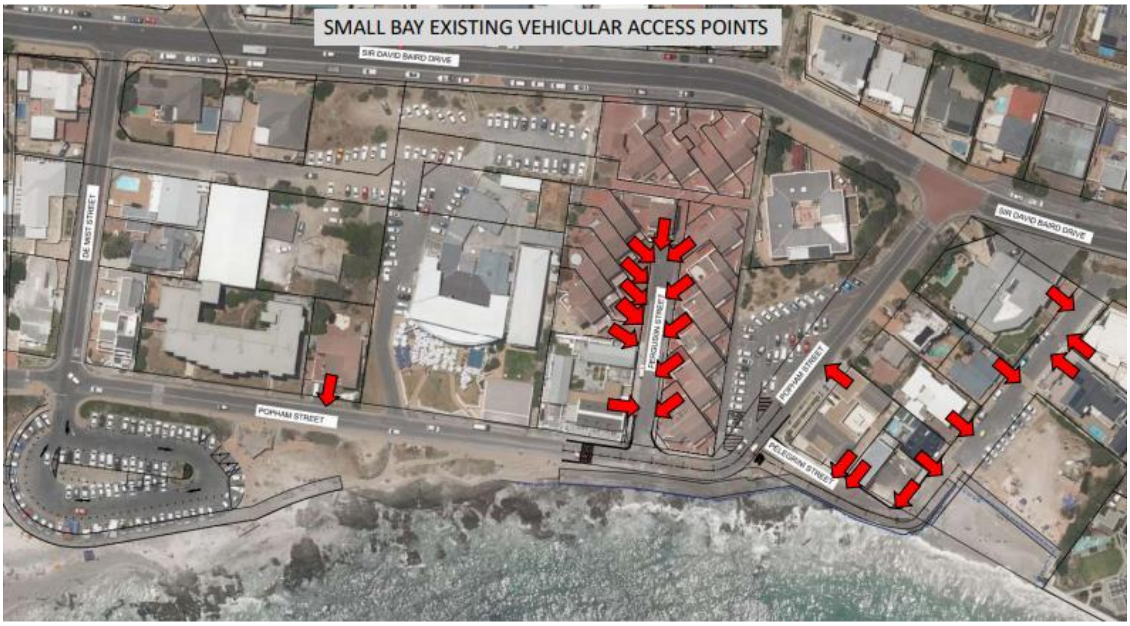


Figure 7.8: Existing vehicular access points

Once the access points were identified, the stages of construction (as detailed in **chapter 7.3**) were considered, and temporary access routes highlighted. These are detailed in **Figures 7.9** to **7.13** which follow. Stage 1 requires a temporary access along Popham Street, over the currently collapsed portion of roadway. It is proposed to fill the existing cavities with fill material and provide a gravel residents-only access. Popham Street will be closed between Ferguson Street and Rancke Road. De Mist parking will also be closed.

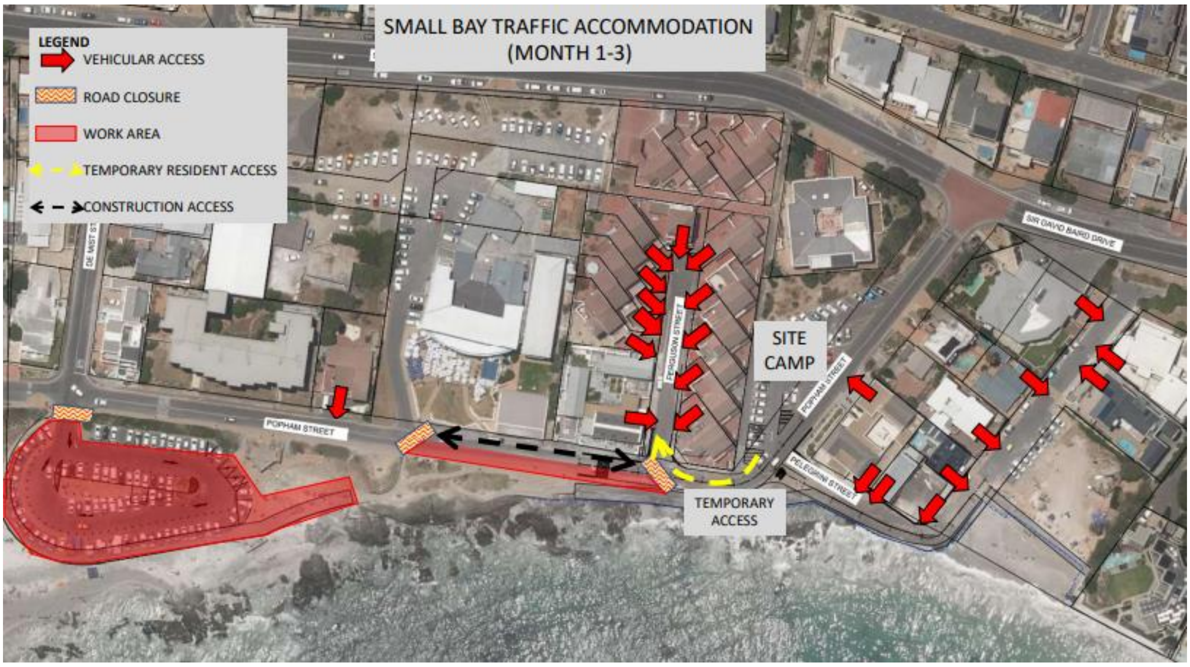


Figure 7.9: Stage 1 traffic accommodation

Stage 2 still requires De Mist parking to be closed. However, access to Ferguson Street will be via the northern portion of Popham Street.

Figure 7.10: Stage 2 traffic accommodation

Stage 3 of construction focuses on the portion of Popham Street between Pelegrini Street and Ferguson Road. Therefore, access to Ferguson Road will be from the north. De Mist parking will be opened to the public. Residents will not be severely impacted.

Figure 7.11: Stage 3 traffic accommodation

Stage 4 of construction has the worst impact on local residents, specifically those along Pelegrini Street. Nine residents were identified that have vehicular access that may be impossible to reach for extended periods of time. A temporary access off Sir David Baird Drive has been identified as a possible solution. The height difference between Pelegrini

Street and Sir David Baird Drive is approximately 2.8m. However, an exact measurement and available space would need to be confirmed prior to this access being built.

A reserved residents only parking area will be made available within the parking lot on Popham Road. If necessary, security will need to be provided overnight. However, it is expected that the contractor's site camp will already have security overnight.

Figure 7.12: Stage 4 traffic accommodation

The final stage of construction will not impact local residents' vehicular access, apart from making allowance for construction vehicles to access the play park. The park itself will most likely be closed to visitors.

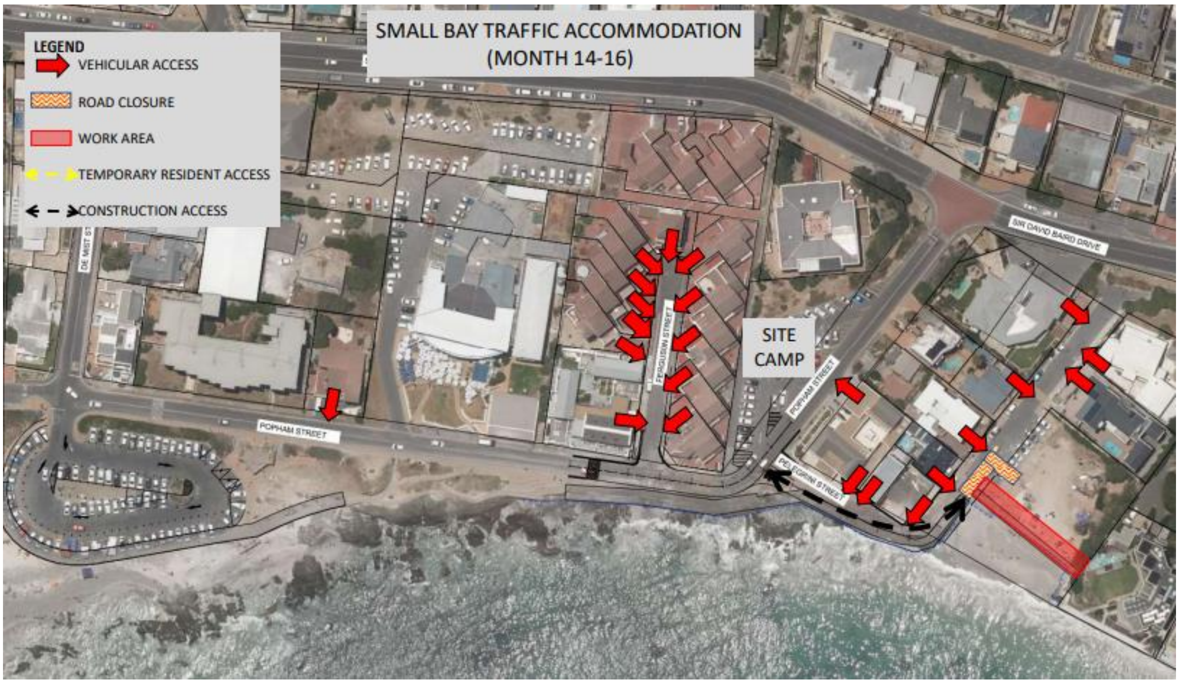


Figure 7.13: Stage 5 traffic accommodation

7.4.2 Occupational health and safety

During the preparation of the tender document, an OHS specialist will be appointed to prepare the relevant specifications. This same specialist will also be appointed during construction to ensure compliance with all requirements.

7.4.3 Environmental regulations

The construction of the seawall, temporary works and sewer realignment are incredibly sensitive works and require careful monitoring in order to comply with the National Environmental Management Act (1998). An environmental officer shall be appointed to perform all activities required, and to ensure the project complies with the Act.

8 MAINTENANCE SCHEDULE

Period (after construction)	Asset		
	Seawall	Road	Sewer line
1 – 5 years	Storm events < 1:100 year Annual replacement of steel stanchions (if any) between recurve cope units: Estimate R200 000/year (excluding escalation)	No maintenance required	Annual jet flushing of pipes: Estimate R280 000/year (excluding escalation)
5 – 10 years	Storm events < 1:100 year Annual replacement of steel stanchions (if any) between recurve cope units: Estimate R200 000/year (excluding escalation)	No maintenance required	Every 5 years, camera inspection: Estimate R420 000 (excluding escalation)
10 – 50 years	Storm events < 1:100 year Annual replacement of steel stanchions (if any) between recurve cope unit: Estimated R300 000/year (excluding escalation). Minor repair to joints and/or minor cracks: Estimated R 100 000/year (excluding escalation) Replacing of all recurve cope units during a 30-year period and repair of cavities along foundation of wall: Estimate R 3 000 000 over 30-year period (excluding escalation). Extreme storm events ≈ 1:100 year Major structural repair may be required but unlikely. Replacement of steel stanchions (if any) between recurve cope units: Estimate R100 000 (excluding escalation). Replace damaged recurved cope units: Estimate R 500 000 (excluding escalation). Accidental storm events > 1:100 Major structural repair may be required due to significant damage to seawall, road and sewer line. Not possible to estimate.	15 years after construction, joint maintenance will be required (estimate R10 000 excluding escalation) 30-40 years after construction, the road pavement will need to be reconstructed.	Every 10 years, detailed inspection: Estimate R420 000 (excluding escalation) Major rebuild of sections, if required: Estimate R28 000/m (excluding escalation)

9 COST ESTIMATE

9.1 Structure of the Estimate

The estimate is structured to provide indicative CAPEX pricing for the complete project scope of works.

The estimate is subdivided into the following elements:

- Direct capital costs associated with the various marine and civil works, including:
    - Demolition of the existing seawall, sewer pipeline and roadway
    - Temporary works for the protection of permanent works including but not limited to dewatering, shoring and geobags
    - Temporary works associated with the realignment of the sewer pipes
    - Concrete seawall
    - Mass concrete foundational fill
    - Precast cope
    - Rock anchors
    - Realigned uPVC DN200 and HDPE DN200 foul sewers
    - Popham Street roadworks
    - Pelegrini Street roadworks
    - Reinstatement of De Mist parking lot layerworks
    - Parking layout changes at De Mist parking lot
- Design development allowance
- Pre-tender and post Contract escalation
- Project contingencies

9.2 Basis of Estimate

The capital cost estimate is prepared considering the layouts and basic engineering information presented from previous studies and supporting drawings. Additional considerations include:

- The Estimate Class is set at an AACE Class 4 / FEL 2 level with an agreed level of accuracy of -20% to +30%.
- The estimate is derived using a combination of measured preliminary quantities and corresponding current or escalated unit rates, largely based on PRDW and HHO's internal rates databases. Built-up rates and prices are used where no relevant rates or prices are available.

9.3 Scope Exclusions and Assumptions

The estimate does not consider the following:

- Phase 1: Particular Exclusions
    - Landscaping and irrigation requirements
    - Streetlighting
    - Street furniture such as benches
- Phase 2: Particular Exclusions
    - Pretender award and post-contract award escalation
- Phase 1 and Phase 2: General Exclusions
    - Professional Fees including additional study work (shown separately below)
    - Construction Supervision costs (shown separately below)
    - Project-wide contingency allowance and management reserves (included in HHO's overall estimate)
    - Owners team costs
    - Purchase/lease of land and/or relocation and restitution costs
    - Local or other authority permits and approvals, rights and licenses.
    - Allowance for compensation to third parties
    - Allowance for market adjustment due to local and international demand, availability of skills, resources and materials
    - Environmental fees for assessments and management plans
    - Rate of exchange adjustment
    - Force Majeure
    - Cost of finance and working capital.
    - Value Added Tax or other foreign or South African taxes, royalties and duties.

The estimate is based upon the following assumptions:

- Precast elements to be manufactured and supplied by a commercial precast concrete manufacturer – no additional on-site storage area has been allowed for.
- Seawall is constructed in 3m sections to limit exposure to the sea and potential damage due to storm event. Excavations are done between 10 – 20 m, with the seawall cast in 3 m lengths.
- No delays due to unforeseen delays
- Phase 1 will comprise Play Park to Ferguson and De Mist Parking for escalation purposes.

- Project tender period to start March 2023.
- Two-year construction period

9.4 Estimate Base Date and Foreign Currency

The scope of the estimate is subject to the following assumptions:

- Cost base as of July 2021
- All elements will be locally sourced.

No allowance has been included for fluctuations in the rate of exchange used for imported content or changing market conditions and escalation beyond the base date of the estimate.

9.5 Estimating Method

The marine works component of the capital cost estimate for the Bloubergstrand Small Bay Seawall was prepared in terms of PRDW's Cost Estimation Guideline S5005-DG-CS-002-R0 taking into consideration the layouts and basic engineering information presented in the previous study reports together with other preliminary information following from this.

9.6 Base Capital Cost

The base capital costs are the expenses incurred by a contractor for labour, material, equipment, plant and any other items required to construct the works, plus overheads and the contractor's profit for each of the elements.

9.7 Preliminary and General Allowances

An allowance for Preliminary and General (P&G) costs is included in the capital cost estimate of each cost element. Each P&G allowance has been presented as a percentage of the total value of base capital cost for that particular cost element. The P&G allowance is set at 25%, due to the nature of the works.

9.8 Design Development Allowance

A design development allowance of 15%, is included to cover design and pricing uncertainties due to the level of design information available at this FEL 2 stage of the project. The design development allowance is included in the base capital cost estimate as a percentage of the total value of base capital cost, including P&G allowances.

9.9 Escalation

The estimate includes an allowance to cover the potential increase in cost from the base date of the estimate to the completion of construction. The pre-tender escalation is based on the indices published by the Medium-Term Forecasting Associates Bureau for Economic Research (BER) and amounts to approximately 7% of the direct capital cost (base cost including P&G, design development allowances). A post-tender escalation allowance of approximately 3.5% per year for 2 years has also been included to account for increases in cost during the construction period. This allowance is based on the Derived Haylett CPAP indices, published by the STATS SA.

9.10 Capital Cost Estimate Summary

The estimated capital costs for the Seawall development subject to the assumptions and exclusions as listed above are summarised in **Tables 9.1** to **9.3**. The detailed capital cost estimate is included in **Annexure I** of this report.

Table 9.1: Capital cost estimate for Phase 1 – Play Park to Ferguson Road

Description	Seawall (1)	Roadworks	Foul Sewer	Total
Demolition, excavation and temporary works	R 3 332 000		R 2 827 000	R6 159 000
Capital cost	R 12 470 000	R 3 844 000	R 2 774 400	R 19 088 400
Preliminary & General (25%)	R 3 950 500	R 961 000	R 1 400 350	R 6 311 850
Design Development (15%)	R 2 962 875	R 720 750	R 1 050 263	R 4 733 888
Subtotal Direct Capital Cost	**R 22 715 375**	**R 5 525 750**	**R 8 052 013**	**R 36 293 138**
Pre-Tender Escalation	R 1 567 361	R 381 277	R 555 589	R 2 504 227
Post Tender Escalation	R 1 602 661	R 389 864	R 568 102	R 2 560 627
Contingency (10%)	R 2 588 540	R 629 689	R 917 570	R 4 135 799
Total	**R 28 473 937**	**R 6 926 580**	**R 10 093 273**	**R 45 493 791**

(1) Recurved cope units could be removed from the phase 1 play park alignment, at an estimated saving of R 235 000.00 (excluding escalation)

Table 9.2: Capital cost estimate for Phase 1 – De Mist parking lot

Description	Seawall	Roadworks	Total
Demolition, excavation and temporary works	R 2 384 000		R 2 384 000
Seawall (1)	R 11 820 000	R2 098 000	R 13 918 000
Preliminary & General (25%)	R 3 551 000	R 524 500	R 4 075 500
Design Development (15%)	R 2 663 250	R 393 375	R 3 056 625
Subtotal Direct Capital Cost	**R 20 418 250**	**R 3 015 875**	**R 23 434 125**
Pre-Tender Escalation	R 1 408 859	R 208 095	R 1 616 955
Post Tender Escalation	R 1 440 589	R 212 782	R 1 653 371
Contingency (10%)	R 2 326 770	R 343 675	R 2 670 445
Total	**R 25 594 469**	**R 3 780 428**	**R 29 374 896**

(1) Recurved cope units could be removed from the phase 1 De Mist alignment, at an estimated saving of R 740 000.00 (excluding escalation).

Table 9.3: Capital cost estimate for Phase 2

Description	Play Park - De Mist Parking
Demolition, excavation and temporary works	R 217 000
Seawall	R 11 730 000
Services Infrastructure	R 370 000
Sub-total	**R 12 317 000**
Preliminary & General (25%)	R 3 079 250
Design Development (15%)	R2 309 438
Subtotal Direct Capital Cost	**R 17 705 688**
Contingency (10%)	R 1 770 569
Total	**R 19 476 257**

9.11 Additional Phase 1 Project Costs

The following phase 1 project costs are to be considered, which do not form part of the direct capital costs listed above:

- The professional fees associated with Phase 1's scope of works (Play Park to De Mist parking lot) are calculated using a percentage of 4.7%. This amounts to R 3 518 828.
- The construction monitoring fees make allowance for the following:
  - Civil engineer: Level 3, Category C (16 months) – R 2 039 684
  - Coastal engineer: Level 3, Category D (13 months) – R 911 091
- Employer's health & safety agent – R 172 198
- Environmental officer – R59 252
- Additional services (project lead fee, specialist studies, surveys etc.) excluding printing and travelling disbursements –R 2 510 493

The total additional project costs amount to R 9 211 546 (excl VAT).

10 CONCLUSIONS

The Bloubergstrand Beach node is an important recreational and tourism destination situated on the west coast of Cape Town. The existing concrete seawall and walkways, which extend from the play park in Small Bay to the De Mist parking area, have been undermined for a number of years. This has resulted in damage to the adjacent road infrastructure. Of specific concern is the exposure of the existing sewer line which is located within the existing sea wall along Popham and Pelegrini Streets. There is an elevated risk that the sewer line could fail, which could create an environmental emergency.

Site investigations were undertaken by Tritan Survey and SGS Matrocast in May 2021. These investigations included aerail and hydrographic survey with geotechnical investigation in front and behind of the existing seawall. These investigations characterised the current insitu condition of the site. The foreshore geotechnical investigations by Tritan used a jet probe survey to investigate the bedrock levels at the site which illustrated that the site is characterised by a shallow bedrock level which deepens towards its Northern and Southern extents. A bedrock level as deep as -1.30 m MSL is anticipated. Additionally, an underground services detection survey was completed by Hydrometrix Technologies which confirmed and detected the location and depth of all existing services at the site.

Development of the new seawall has been split into two phases. The requirement of the first phase is to provide reinstatement of and protection to the Cities landside infrastructure, in particular the edge wall, sewer line, roadway, and pedestrian access. The project brief however requires that the seawall foundation and base be designed so that the seawall can be raised to a final crest elevation of +4.5 m MSL in the future. This is required to provide further protection from wave action and overtopping. The seawall will not initially be built to the +4.5 m MSL level due to budgetary constraints, the boundary that a raised crest level will create between the land and shoreline access, the obstruction of sea views and, the potential challenges and delays in getting the necessary authorisations.

The coastal modelling study undertaken by PRDW has defined the metocean conditions at the site. Water levels, waves (offshore and at the wall), overtopping rates and resulting flooding were investigated for various scenarios. In general, the coastal modelling has shown that the design water levels are high compared to the raised road levels and that large waves are anticipated in front of the seawall. A design water level of +2.67 m MSL and an offshore wave height of Hm0 = 6.01 m have been calculated for the design storm event (1:100 year). Additionally, the maximum wave height (Hmax) calculated along the length of the seawall was 3.86 m. The results from the overtopping and flooding analysis for the phase 1 seawall levels (including cases with a +1 m recurved cope extension) have shown that there is a hazard to pedestrians and vehicles during storm events with low return periods (1:1 and 1:5 year) particularly at the Pelegrini Street corner. This has been based on calculated flood water depths and current velocities behind the seawall. The results show that the extent to the flooding behind the wall is appreciable, and that a large portion of the landside infrastructure is impacted. The depth of flood water determined in the analysis verifies the design assumptions used in the preliminary design. Additionally, the analysis further shows that the use of a 1m high cope unit significantly decreases overtopping by a factor of 4 for the model cases considered which illustrates the benefit of installing these units.

The preliminary design has illustrated that a feasible structural solution for the seawall is possible. The chosen structural solution of the seawall comprises a cast in-situ, reinforced concrete wall which is anchored to the bedrock using galvanised rock anchors. A cast in-situ construction method has been adopted given the various site constraints such as site access, site storage, space availability and traffic accommodation. The seawall has a design working life of 50 years and is designed for a 100-year storm event with an encounter probability of 39.60%. The stability analysis shows that landward wave forces govern the stability of the seawall.

In addition to the seawall, the design of Popham and Pelegrini streets was undertaken. Popham Street has suffered complete structural failure and has been closed to traffic, while Pelegrini Street has experienced some localised failures. A jointed concrete pavement road, with a minimum design life of 30 years, has been proposed for both roads. This pavement is best suited for this location due to its durability and low maintenance requirement. The pavement will be designed during the detailed design phase of the project. The vertical alignment of both roads has made allowance for overland drainage and where possible, the roads have been raised to help reduce wave overtopping volumes.

The realignment and upgrade of the existing DN150 foul sewer was completed. The proposed DN200 pipe is a SDR34 uPVC pipe runs in the north bound lane of Popham Street, and along Pelegrini Street behind the seawall before tying into the pump station at the play park. A second existing foul sewer pipe has been identified for a realignment to allow for the construction of the Small Bay play park seawall. The DN200 HDPE pipe will be offset from the wall. The construction coordination of the foul sewer will need to be carefully controlled, making use of over pumping to prevent any disruptions to service or a sewage spill.

A cast insitu methodology has been adopted for the seawall construction which is different to that previously proposed. Additionally, a feasible construction sequence has been formulated along with an associated construction program. The construction sequence is based on the various site constraints and takes into consideration the proximity of the neighbouring residential infrastructure and construction in the tidal zone. A high-level construction program has been developed and it is estimated that the construction of the works will have a duration of at least 16 months.

A preliminary design cost estimate was completed, which split the works into two sections, as well as into 2 phases. The associated capital costs for all works, including preliminary and general, design development, contingencies and escalation is as follows:

- Phase 1 from the play park to Ferguson Road – R 45 493 791
- Phase 1 De Mist parking area – R 29 374 896
- Phase 2 future works – R 19 476 257 (excluding escalation)

Based on the available budget, it is unlikely that the De Mist parking work will be feasible unless additional budget is secured.

ANNEXURE A: COASTAL SITE INVESTIGATION

ANNEXURE B: COASTAL MODELLING

ANNEXURE C: GROUND TRUTHING INVESTIGATION

ANNEXURE D: GEOTECHNICAL AND MATERIALS INVESTIGATION REPORT

ANNEXURE E: UNDERGROUND SERVICES SURVEY

ANNEXURE F: TRAFFIC STUDY

ANNEXURE G: COASTAL ENGINEERING PRELIMINARY DESIGN DRAWINGS

ANNEXURE H: CIVIL ENGINEERING PRELIMINARY DESIGN DRAWINGS

ANNEXURE I: CAPITAL COST ESTIMATE

CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD

Appendix G: Photos

SMALL BAY SEAWALL PROJECT

Environmental Management Department: Coastal Management Branch

Making progress possible. Together.

Existing sewer line indicated in red


Popham
Play Park
Pelegrini


Popham
Play Park
Pelegrini

FOUL SEWER


1


2


3


4


5


6


7


1


2


3


1.5 m

CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD

1:1000 GUIDE PLAN AREA G.N. 270/1982

BLAAUWBERG MUNICIPALITY PROC. 147 / 1995

BHSN-3474

COMPILED IN THE OFFICE OF THE SURVEYOR-GENERAL, CAPE TOWN NOV. 1984

TRIG SHEET No. 3318 (60)

J. SWINERD
G. MOSTERT

SYMBOLS SHOWN THUS:-
● LO CONNECTED BEACONS
◉ LO CONNECTED BEACONS ESTABLISHED
○ BEACONS CONNECTED TO BOSMAN SYSTEM
◎ BEACONS NOT CONNECTED ESTABLISHED
+ + + + ALLOTMENT, MUNICIPAL, GROUP, BLACK AREA BOUNDARIES
. . . . ESTABLISHED BOUNDARIES

BHSN-3474

M 3184

ERF 241, BLAAUWBERGSTRAND, P:0 (CAPE TOWN)

Deeds Office Property Erf

Suite G01, Waterview 2, Waterview Close, Century City
Tel: +27 860 340 000
Website: https://www.searchworks.co.za

SEARCH INFORMATION

Summary	
Search Type	DEEDS OFFICE PROPERTY ERF
Search Description	ERF 241, BLAAUWBERGSTRAND, P:0 (CAPE TOWN)
Reference	ENVIRONMENTAL MANAGEMENT
Date	27/01/2022

ERF INFORMATION

Summary	
Deeds Office	CAPE TOWN
Property Type	ERF
Township	BLAAUWBERGSTRAND
Erf Number	241
Portion Number	0
Previous Description	-
Registration Division	CAPE RD
Municipality	CITY OF CAPE TOWN
Province	WESTERN CAPE
Diagram Deed	T2994/920
Size	1959.0000 SQM
LPI Code	C01600050000024100000
Street Address	-

OWNER SUMMARY

Owner Name	ID / Reg. Number	Purchase Price	Purchase Date
REGIONAL SERVICES COUNCIL-CAPE METROPOLE	-	-	-

OWNER INFORMATION

Owner 1 of 1	
Owner Name	REGIONAL SERVICES COUNCIL-CAPE METROPOLE
ID / Reg. Number	-
Owner Type	COMPANY
Title Deed	T2719/1967
Purchase Date	-
Registration Date	15/02/1967
Purchase Price	-
Multiple Owners	NO
Multiple Properties	-
Share	-
Microfilm Reference No.	-

ENDORSEMENT(S)

Document Number	Microfilm Reference Number	Institution	Value
No information available.			

HISTORY INFORMATION

Document Number	Microfilm Reference Number	Owner	Value
No information available.			

REPORT INFORMATION

Date of Information	27/01/2022 14:28
Print Date	27-01-2022 14:44
Generated By	GARTH MANASSE
Reference	ENVIRONMENTAL MANAGEMENT
Report Type	DEEDS OFFICE PROPERTY ERF

ERF 253, BLAAUWBERGSTRAND, P:0 (CAPE TOWN)

Deeds Office Property Erf

Suite G01, Waterview 2, Waterview Close, Century City
Tel: +27 860 340 000
Website: https://www.searchworks.co.za

SEARCH INFORMATION

Summary	
Search Type	DEEDS OFFICE PROPERTY ERF
Search Description	ERF 253, BLAAUWBERGSTRAND, P:0 (CAPE TOWN)
Reference	COASTAL MANAGEMENT - ENVIRONMENTAL MANAGEMENT
Date	27/01/2022

ERF INFORMATION

Summary	
Deeds Office	CAPE TOWN
Property Type	ERF
Township	BLAAUWBERGSTRAND
Erf Number	253
Portion Number	0
Previous Description	-
Registration Division	CAPE RD
Municipality	CITY OF CAPE TOWN
Province	WESTERN CAPE
Diagram Deed	CPF8-13/1844
Size	1389.0000 SQM
LPI Code	C01600050000025300000
Street Address	-

OWNER SUMMARY

Owner Name	ID / Reg. Number	Purchase Price	Purchase Date
K2011139723 SOUTH AFRICA PTY LTD	201113972307	ESTATE	-

OWNER INFORMATION

Owner 1 of 1	
Owner Name	K2011139723 SOUTH AFRICA PTY LTD
ID / Reg. Number	201113972307
Owner Type	COMPANY
Title Deed	T5764/2021
Purchase Date	-
Registration Date	08/02/2021
Purchase Price	ESTATE
Multiple Owners	NO
Multiple Properties	-
Share	-
Microfilm Reference No.	20210219 08:19:51

ENDORSEMENT(S)

Document Number	Microfilm Reference Number	Institution	Value
No information available.			

HISTORY INFORMATION

Document Number	Microfilm Reference Number	Owner	Value
B60051/1995	20210219 08:17:46	ABSA BANK	R 550 000,00
B60051/1995	20210219 08:17:46	-	R 550 000,00
B60133/2006	20210219 08:20:31	-	-
B18587/2009	20210219 08:23:24	-	-
T29761/1976	2006 155 8 :03:20	STARKE BASIL EDMUND	UNKNOWN

REPORT INFORMATION

Date of Information	27/01/2022 14:49
Print Date	27-01-2022 14:50
Generated By	GARTH MANASSE
Reference	COASTAL MANAGEMENT - ENVIRONMENTAL MANAGEMENT
Report Type	DEEDS OFFICE PROPERTY ERF

A. VIR AKTEBESORGER SE GEBRUIK / FOR CONVEYANCER'S USE:

(a) Gelyktydiges met ander registrasiekantore / deeltitels: Simuls with other registries / sectional titles:

	Kode/Code	Firma / Firm	Eiendom / Property	Kantoor / Office
1.				
2.				
3.				
4.				

(b) Kliënt afskrifte van aktes permanent in Aktekantoor geliasseer:
Client copies of deeds filed permanently in Deeds Office:

Aard en nommer van akte / Nature and number of deed	Cover No. / Omslag Nr.	Parawe van ondersoekers Initials of Examiners

(c) Notas / Notes:

B. VIR AKTEKANTOOR GEBRUIK / FOR DEEDS OFFICE USE:

Interdikte nagesien Interdicts checked			Opmerkings / Remarks	Paraaf Initials
Datum / Date	Paraaf Initials	(1) Dorp goedgekeur (geproklameer) Township approved (proclaimed)		
		(2) Begiftigingserwe Endowment erven		
		(3) Begiftiging Endowment		
		(4) Voorwaardes Conditions		
		(5) Mikro Micro		
		(6) Algemene Plan General Plan		
		(7) Titelakte Title Deed		
		(8) Verbande teen dorpstitel Bonds against township title		
		(9) Datum nagesien Date checked		

BLACKBOOK
08-02-2021
DIPONTSHENG LEEUW

Kantoor instruksies/Office instructions:

Seksie/Section

Deeds Reg A/C No.
CTN
056
Akte Kant Rek Nr.

WALKERS

TELEPHONE: 464-1400

EXECUTION/ UITVOERING

Ref No / Verw. Nr W43081

A. VIR AKTEKANTOOR GEBRUIK / FOR DEEDS OFFICE USE:

Datum van indiening / Date of lodgement

04 FEB 2021

LODGED
28-01-2021
INGEDIEN

RCC 09/02/2021

	Ondersoekers / Examiners	Kamers Rooms	Skakeling / Linking		Reject Verwerp	Passeer Pass
1.	N. MENZIWA 1230		4	1		
2.	R.Harris 1352					
3.						

B. VIR AKTEBESORGER SE GEBRUIK / FOR CONVEYANCER'S USE

Aard van Akte / Nature of Deed: Transfer
EIL B.E. Starke
t.g.v. / i.f.o. K2011139723 SA Pty Ltd

T 000005764 / 2021

Skakeling / Linking		Titelaktes / Title deeds within
4	1	T 29761/1976 in no. 2 of batch

GELYKTYDIGES / SIMULS

No. in stel / batch	Kode / Code	Name van Partye / Names of Parties	Naam van Firma / Name of Firm	Firma Firm No.
1	T	EIL Starke/K2011139723	Walkers	56
2	BL	" / ABSA	Bissets	52
3	BL	" / "	Bissets	52
4	BL	" / "	Bissets	52
5				
6				
7				
8				
9				
10				
11				
12				

HANDED IN FOR EXECUTION
05 FEB 2021
CAPE TOWN
HANDED IN FOR EXECUTION

EXAMINER'S NOTES INSIDE

080007493510

Registrasie versoek deur / Registration requested by: KARA FISCHER LPCM No. 96488

Datum / Date: 08 FEB 2021

(Kort beskrywing van eiendom (slegs para 1 in Akte) / Brief description of property (merely para 1 in Deed)

ERF 253 Blaauwbergstrand

WALKERS

① The extending clause is incomplete - dgm ref is omitted. Retype pg 2.

done.

56 WALKERS INC.
Telephone (021) 464 1400

Prepared by me

CONVEYANCER
JOHANNA PHILNA PHILLIPS
(85836)

Deeds Office Registration fees as per Act 47 of 1937		
	Amount	Office Fee
Purchase Price	R 11 000 000.00	R 3397.00
Reason for Exemption 0	Category Exemption....................	Exemption i t o. Sec/Reg....................... Act/Proc.......................

DATA / VERIFY
10 -02- 2021
VUYELWA LAMANI

T 000005764 / 2021

DEED OF TRANSFER

BE IT HEREBY MADE KNOWN THAT

KARA FISCHER
LPCM No. 96488

appeared before me, the Registrar of Deeds at Cape Town, the said appearer, being duly authorised thereto by a power of attorney granted to him by

The Executor in the Estate of the late BASIL EDMUND STARKE
Estate number 2965/2018

signed at Pinelands on 1 December 2020

DATA / CAPTURE
10 -02- 2021
JENNY VAN WYK

And the appearer declared that:

Whereas the Transferor had, with the consent of the Master of the High Court as more fully provided for in terms of Section 42(2) of Act, No. 66 of 1965, truly and legally sold the undermentioned property on 13 February 2020 by Private Treaty,

Now therefore the Appearer on behalf of the Transferor, did by these presents, cede and transfer to and on behalf of

K2011139723 (SOUTH AFRICA) PROPRIETARY LIMITED
Registration Number 2011/139723/07

its successors in title or assigns, in full and free property

REMAINDER ERF 253 BLAAUWBERGSTRAND, IN THE CITY OF CAPE TOWN, CAPE DIVISION, WESTERN CAPE PROVINCE

IN EXTENT 1389 (ONE THOUSAND THREE HUNDRED AND EIGHTY NINE) SQUARE METRES

FIRST TRANSFERRED by Deed of Grant dated 12 August 1844 (Cape Freeholds Volume 8 Number 13) with a diagram relating thereto and held by Deed of Transfer No. T29761/1976

SUBJECT to the conditions referred to in the Deed of Transfer No. T302/1920.

WHEREFORE the appearer, renouncing all the right and title the said

The Estate of the Late BASIL EDMUND STARKE

heretofore had to the premises, did, in consequence also acknowledge him to be entirely dispossessed of, and disentitled to, the same; and that, by virtue of these presents, the said

K2011139723 (SOUTH AFRICA) PROPRIETARY LIMITED

its successors in title or assigns, now is and henceforth shall be entitled thereto, conformably to local customs; the State, however, reserving its rights, and finally acknowledging that the purchase price is the amount of R11 000 000,00 (Eleven Million Rand).

IN WITNESS WHEREOF I, the said Registrar, together with the appearer, have subscribed to these presents, and have caused the seal of office to be affixed thereto.

THUS DONE AND EXECUTED at the Office of the Registrar of Deeds at Cape Town on

0 8 FEB 2021

Signature of appearer *q.q.*

In my presence

Registrar of Deeds

① 1-8-9

56 WALKERS INC.
Telephone (021) 464 1400

Prepared by me

CONVEYANCER
JOHANNA PHILNA PHILLIPS
(85836)

POWER OF ATTORNEY TO PASS TRANSFER

I, the undersigned

RICHARD ERNEST BROOME in my capacity as the Execut~~rix~~or in the Estate of the late
BASIL EDMUND STARKE
Estate number 2965/2018
Acting under Letters of Executorship issued by the Master of the High Court of South Africa, Western Cape Division, Cape Town on 11 April 2018 in terms of Section 13 and 14 of the Administration of Estates Act No 66 of 1965

Do hereby nominate, constitute and appoint

ARNO WATSON (80234) or HANS WERNER MENNEN (80824) or JOHANNA PHILNA PHILLIPS (85836) or JOHN ANTHONY LEE (82295) or JOLINE NORMAN (90999) or KARA FISCHER (96488) or LYNNE BOTHA (82076)

with the power of substitution to be my true and lawful attorney and agent to appear before the Registrar of Deeds at Cape Town, or any other competent official in the Republic of South Africa

And then and there to declare that the Transferor did, with the consent of the Master of the High Court as more fully provided for in terms of Section 42(2) of Act, No. 66 of 1965, on 13 February 2020 sell by Private Treaty to

K2011139723 (SOUTH AFRICA) PROPRIETARY LIMITED
Registration Number 2011/139723/07

for the sum of R11 000 000,00 (Eleven Million Rand)

the following property:

REMAINDER ERF 253 BLAAUWBERGSTRAND, IN THE CITY OF CAPE TOWN, CAPE DIVISION, WESTERN CAPE PROVINCE

IN EXTENT 1389 (ONE THOUSAND THREE HUNDRED AND EIGHTY NINE) SQUARE METRES

HELD BY DEED OF TRANSFER T29761/1976

② ①

And further cede and transfer the said property to the said transferee; to renounce all right, title and interest which the Transferor heretofore had in and to the said property, and generally, for effecting the purposes aforesaid, to do or cause to be done whatsoever shall be requisite, as fully and effectually, to all intents and purposes, as Transferor might or could do if personally present and acting therein, hereby ratifying, allowing and confirming all and whatsoever the said agent shall lawfully do or cause to be done in the premises by virtue of these presents.

Signed at Pinelands on 01/12/2020 in the presence of the undersigned witnesses.

WITNESSES:

1. ______________________

2. ______________________

Broome
RICHARD ERNEST BROOME

SERTIFIKAAT

Ek bevestig hiermee, in terme van Artikel 42 (2), Wet No. 66 van 1965, dat daar geen besware is teen transport soos hierin vermeld.

CERTIFICATE

I hereby certify, in terms of Section 42 (2), Act No. 66 of 1965, that there is no objection to transfer as stated herein.

MEESTER VAN DIE HOË HOF
MASTER OF THE HIGH COURT

KAAPSTAD/CAPE TOWN 18/1/2021

⑥

3

Q

SARS

Transfer Duty Declaration

TDREP

Reference Details

Transfer Duty Reference Number: TDE03D6955

Details

Details of Seller / Transferor / Time Share Company

Surname / Registered Name	STARKE (Estate Number 2965/2018)	Full Name	Executor in Estate Late BASIL EDMUND
ID Number	3803235029083	Date of Birth (CCYYMMDD)	1938-03-23
Company / CC / Trust Reg No.		Marital Status	NOT MARRIED

Details of Purchaser / Transferee

Full Name	LIMITED	Surname / Registered Name	K2011139723 (SOUTH AFRICA) PROPRIETARY
Company / CC / Trust Reg No.	201113972307	Marital Notes if applicable	

X

Details of the Property

Date of Transaction/Acquisition (CCYYMMDD)	2020-02-13		
Total Fair Value	R 11000000.00	Total Consideration	R 11000000.00

Calculation of Duty and Penalty / Interest

Transfer Duty Payable on Natural Person	R 11000000.00

Property Description

1 REMAINDER ERF 253 BLAAUWBERGSTRAND, IN THE CITY OF CAPE TOWN, CAPE DIVISION, WESTERN CAPE PROVINCE IN EXTENT 1389 (ONE THOUSAND THREE HUNDRED AND EIGHTY NINE) SQUARE METRES

Receipt

Receipt Details

Transfer Duty Reference Number	TDE03D6955	Receipt No.	1200757317
Receipt Amount	R 1063000.00		

Declaration by Conveyancer / Attorney

I certify that this is a true copy of the transfer duty declaration / receipt / exemption certificate drawn from the SARS eFiling site, which will be retained by me for 5 years from the date of registration of transfer.

xxxxxxxxxxxxxxxxxxxxxxxxx
xxxxxxxxxxxxxxxxxxxxxxxxx

Please ensure you sign over the 2 lines of "X"s above

ff9f4b66a2c8828558a569
dc6d772559b3069f71

Date (CCYYMMDD) 2 0 2 1 0 1 2 7

For enquiries go to www.sars.gov.za or call 0800 00 SARS (7277)

JOHANNA-PHILNA PHILLIPS

4

ISIXEKO SASEKAPA
CITY OF CAPE-TOWN
STAD KAAPSTAD

CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD
Making progress possible. Together.

CERTIFICATE IN TERMS OF SECTION 118 OF THE LOCAL GOVERNMENT: MUNICIPAL SYSTEMS ACT, 2000 (ACT No. 32 OF 2000)

(AS PRESCRIBED IN TERMS OF SECTION 120 OF ACT No. 32 OF 2000)

ISSUED BY CITY OF CAPE TOWN

In terms of section 118 of the Local Government: Municipal Systems Act, 2000 (Act No. 32 of 2000), it is hereby certified that all amounts that became due to City of Cape Town in connection with the undermentioned property situated within that municipality for municipal service fees, surcharges on fees, property rates and other municipal taxes, levies and duties during the two years preceding the date of application for this certificate, have been fully paid.

DESCRIPTION OF PROPERTY (see definition of property in section 1 of Act 32 of 2000)

21 Digit Code (or Municipal Reference Number): 60440567/2020
Erven/Property Description: 253
Portion:
Extension: BLAAUWBERGSTRAND
Zoning: SINGLE RESIDENTIAL ZONE 1 : CONVENTIONAL HOUSING
Registration division / Administrative District: CAPE
Suburb/Allotment: BLAAUWBERGSTRAND
Town: CITY OF CAPE TOWN
Sectional Title unit number:
Exclusive use area and number as referred to on the registered plan:
Real right:
Sectional registration number:
Sectional Title Scheme Name:
Registered Owner: BASIL EDMUND STARKE (3803235029083)
Name and Identity/Registration number of all purchaser/s: K2011139723 (SOUTH AFRICA) PROPRIETARY LIMITED (2011/139723/07)

This Certificate is valid until: 09/02/2021
Given under my hand at: CAPE TOWN on: 11/12/2020
Date issued: 11/12/2020

City of Cape Town
Digitally signed by City of Cape Town
DN: cn=City of Cape Town, c=ZA, l=Cape Town, st=Western Cape, o=City of Cape Town, ou=Signed for Municipal Manager, email=RatesClearance.ElectronicApplication@capetown.gov.za
Reason: I attest to the accuracy and integrity of this document
Date: 2020.12.11 08:12:21 +02'00'

Authorised Official: Annabelle Minnette Cooper

MUNICIPAL MANAGER
CAPE TOWN

Certificate By Conveyancer:

I JOHANNA PHILNA PHILLIPS (full name and surname) hereby certify that this is a print-out of a data message in respect of the original clearance certificate electronically issued by the City of Cape Town.

Conveyancer Signature:
Date: 27/01/2021

⑤

CONVEYANCER'S CERTIFICATE

I, JOHANNA PHILNA PHILLIPS, the undersigned Conveyancer, attending to the estate transfer of the following property:

REMAINDER ERF 253 BLAAUWBERGSTRAND, IN THE CITY OF CAPE TOWN, CAPE DIVISION, WESTERN CAPE PROVINCE

IN EXTENT 1389 (ONE THOUSAND THREE HUNDRED AND EIGHTY NINE) SQUARE METRES

HELD BY DEED OF TRANSFER T29761/1976

By:

The Executor in the Estate of the late BASIL EDMUND STARKE
Estate number 2965/2018

To:

K2011139723 (SOUTH AFRICA) PROPRIETARY LIMITED
Registration Number 2011/139723/07

hereby certify that the property is not an asset in a joint estate.

SIGNED at Cape Town on 27 January 2021.

CONVEYANCER
JOHANNA PHILNA PHILLIPS

e ⑥

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PROD         DEEDS REGISTRATION SYSTEM -  CAPE TOWN                 DATE :   20210128 TIME :  11:50:10.2 PAGE :       1
PREPARED BY : DRS08231 - MADAMA LITHA

                                                  TRACK NUMBER :     80007493510


          BLACK-BOOKING ENQUIRY ON NAME -  STARKE BASIL EDMUND
                              ID NUMBER -  3803235029083
                             BIRTH DATE -   19380323
                         MARITAL STATUS -  MARRIED OUT
                            MAIDEN NAME -
                         TYPE OF PERSON -  PRIVATE PERSON


 PERSON HAS NO CONTRACTS/INTERDICTS


** PLEASE NOTE : THE INFORMATION APPEARING ON THIS PRINTOUT IS FURNISHED FOR PURPOSES OF INFORMATION ONLY.
           FOR MORE DETAILED INFORMATION, PLEASE REFER TO THE REGISTERED SOURCE DOCUMENTS.

                                          * * * END OF REPORT * * *
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7

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PROD         DEEDS REGISTRATION SYSTEM -  CAPE TOWN                DATE :   20210128 TIME :  11:50:16.3 PAGE :       1
PREPARED BY : DRS08231 - MADAMA LITHA

                                                    TRACK NUMBER :    80007493510


        BLACK-BOOKING ENQUIRY ON NAME -  STARKE BASIL EDMUND B-E
                            ID NUMBER -
                           BIRTH DATE -         0
                       MARITAL STATUS -
                          MAIDEN NAME -
                       TYPE OF PERSON -


  PERSON DOES NOT EXIST


** PLEASE NOTE : THE INFORMATION APPEARING ON THIS PRINTOUT IS FURNISHED FOR PURPOSES OF INFORMATION ONLY.
                 FOR MORE DETAILED INFORMATION, PLEASE REFER TO THE REGISTERED SOURCE DOCUMENTS.

                                              * * * END OF REPORT * * *
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PROD         DEEDS REGISTRATION SYSTEM -  CAPE TOWN                DATE :   20210128 TIME : 11:50:25.5 PAGE :       1
PREPARED BY :  DRS08231 - MADAMA LITHA

                                              TRACK NUMBER :    80007493510

   PROPERTY DETAILS PRINT FOR PORTION   0     (R/E)
                                ERF NO     253
                                TOWNSHIP   BLAAUWBERGSTRAND
                                REG DIV    CAPE RD

PROVINCE          WESTERN CAPE
PREV DESCRIPTION
DIAGRAM DEED NO   CPF8-13/1844
EXTENT            1389 SQM
CLEARANCE         CITY OF CAPE TOWN

NO INTERDICTS

DOCUMENTS                   HOLDER & SHARE                       AMOUNT            O/P/A   SCAN/MICRO REF     MMDD
B18587/2009                 A B S A BANK LTD                     R2190000.00               20181218111308     0710
B60051/1995                 A B S A BANK LTD                     R550000.00                20181218111140     0906
B60133/2006                 A B S A BANK LTD                     R460000.00                20181218111224     0621


OWNER DETAILS

FULL NAME & SHARE                    PURCH DATE  AMOUNT/REASON  O/P/A IDENTITY       TITLE DEED      MMDD  MICROFILM REF
STARKE BASIL EDMUND                                                   3803235029083  T29761/1976     0923  20091111121937

 *   O/P/A -   O - MULTIPLE OWNER   P - MULTIPLE PROPERTY   A - MULTIPLE OWNER AND PROPERTY

** PLEASE NOTE : THE INFORMATION APPEARING ON THIS PRINTOUT IS FURNISHED FOR PURPOSES OF INFORMATION ONLY.
                 FOR MORE DETAILED INFORMATION, PLEASE REFER TO THE REGISTERED SOURCE DOCUMENTS.

                                        * * * END OF REPORT * * *
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BC