PICK EVERARD

CROUCH WATERFALL

WATCHET HARBOUR

GEOTECHNICAL ANALYSIS REPORT

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Contents

Executive Summary

The report summarises a modelling exercise carried out on the existing harbour wall at Watchet Harbour; this exercise has been undertaken to gain an understanding of the effect of loading on the existing structure as a result of various scenarios.

The east quay harbour wall at Watchet, West Somerset comprises three distinct sections of wall:

- Northern section – masonry/concrete wall;
- Central section – sheet pile wall supported by deadman anchors;
- Southern section – masonry/concrete wall;

Each section of the harbour wall has been modelled using various tidal loading scenarios, detailed below:

- High tide with harbour silt;
- Low tide with harbour silt;
- Low tide with harbour silt removed (dredged);

Due to the proximity of Hinkley Point (located 14km to the east) to the site, the tide levels for Hinkley have been adopted for Watchet Harbour (from the UK National Tidal & Sea Level Facility).

Surcharge loading scenarios have been detailed by the Client, as per the following:

- A **10kPa** load is placed over a 10-wide strip immediately behind the wall. This simulates a generic load for day-to-day use of the quay/harbour wall – it represents the 'current' situation;
- A **20kPa** load replaces the 10kPa load mentioned above. This 20kPa load is applied over a 10m-wide strip immediately behind the wall and simulates the operation of a mobile crane used to lift boats in to/out of the marina;
- Static (dead) load of **50kPa** imposed by the proposed new development, located 14.5m (minimum) away from the harbour wall;

Load combinations have been analysed for all three sections of the wall and all tidal situations as follows:

Load Scenario 1: 10kPa loading – 'current' situation;
Load Scenario 2: 20kPa loading – potential crane loading;
Load Scenario 3: 10kPa + 50kPa loading – 'current' load + development load;
Load Scenario 4: 20kPa + 50kPa loading – crane load + development load

<u>Northern Section – Masonry Wall</u>
Analysis of the northern section of wall has concluded that, under 'current' marina conditions (ie: silt present), using Load Scenarios 1 and 2, the wall has a minimum Factor of Safety of 1.06. This figure, albeit greater than 1.0, already represents a reduced Factor of Safety, as the minimum acceptable FoS was set at 1.25. Analysis was carried out to SLS conditions of the Eurocode for gauging of the current condition of the wall. This being said, should the silt be dredged from the base of the marina then the Factor of Safety drops below 1.0.

<u>Central Section – Sheet Piled Wall</u>

Detailed sensitivity analysis has been completed on Larssen 22 and Larssen 25 sheets with varying thicknesses (as requested by the Client) and steel grades. This has given some insight into how much degradation/loss of section is required, at differing steel grades, before the Factor of Safety falls below 1.25. Determination of the steel grade (through chemical testing) would give valuable insight into predicting the performance of the sheet piles.

Load Scenarios 1 and 2 have been determined as having significant impacts on the harbour wall, enough to reduce the FoS to <1.0. Based on the Limit Stage analysis, the addition of the development surcharge (Load Scenarios 3 & 4) is not deemed significant enough to affect the harbour wall.

The fact that the model predicts failure but the wall remains standing is believed to be (partly) due to the cyclical nature of the tides and the limited length of time that the wall is left exposed to excessive bending moments. Once the tide starts rising again, so returns the stabilising force of the high tide, and thus the Factor of Safety rises in turn.

Southern Section – Masonry Wall
Analysis of the southern section of wall has concluded that, under 'current' marina conditions (ie: silt present), using Load Scenarios 1 and 2, the wall has a minimum Factor of Safety of 1.4. This Factor of Safety remains unchanged should the silt be dredged from the marina.

When the development load is applied to the wall, the Factor of Safety does not change, suggesting that the development has little to no effect on this section of the harbour wall.

Summary of East Quay Harbour Wall Scenarios, Watchet Harbour

<table>
<tr><th colspan="3"></th><th colspan="2">North Section</th><th colspan="4">Central Section</th><th colspan="2">South Section</th></tr>
<tr><th rowspan="3">Load Scenario</th><th rowspan="3">Surcharge</th><th rowspan="3">Loading (kPa)</th><th rowspan="3">Low Tide + Silt</th><th rowspan="3">High Tide + Silt</th><th colspan="2">Low Tide + Silt</th><th colspan="2">High Tide</th><th rowspan="3">Low Tide + Silt</th><th rowspan="3">High Tide + Silt</th></tr>
<tr><th colspan="4">Larssen Sheet Pile* Equivalent – Bending Moment FOS - in mms</th></tr>
<tr><th>22mm</th><th>25mm</th><th>22mm</th><th>25mm</th></tr>
<tr><td>1</td><td>Current Loading</td><td>10kPa</td><td>1.3 - 7</td><td>4 - 10+</td><td>0.7</td><td>1.1</td><td>2.7</td><td>4.1</td><td>1.8 - 10+</td><td>10+</td></tr>
<tr><td>2</td><td>Crane Loading</td><td>20kPa</td><td>1.06 - 1.3</td><td>2.5 - 10+</td><td>0.7</td><td>1.1</td><td>2.7</td><td>4.1</td><td>1.4 - 6</td><td>4 - 10+</td></tr>
<tr><td>3</td><td>Current + OC Building</td><td>10kPa + 50kPa</td><td colspan="2" rowspan="2">No impact on Northern wall from development.</td><td colspan="4" rowspan="2">As there were no changes from Load Scenario 1 to Load Scenario 2 it is concluded that there will be no further changes as a result of Load Scenarios 3 and 4.</td><td>1.8 - 10+</td><td>10+</td></tr>
<tr><td>4</td><td>Crane + OC Building</td><td>20kPa + 50kpa</td><td>1.4 - 6</td><td>4 - 10+</td></tr>
</table>

Assumed Onion Collective development to generate 50kPa sited 14.5m+ from the edge of the Central and Southern Harbour Walls. No impact on Northern wall section hence discounted from calculations.

All numbers are Factor of Safety (FoS) numbers. FoS = 1.25 is the minimum required by British Standards. As this is an existing structure it has not been analysed against Eurocode 7 partial factors.

Central Section assumed 240 MPa Yield Strength Steel – weakest

Megapascal (MPa) is the mega-unit used to measure the intensity of pressure. MPa in these works can be summarised as the capacity of a material, such as a structure or ground, to 'resist pressure' – the higher the number, the more resistance.

The overall Factor of Safety for each scenario should be taken as the lowest figure for the pile and tidal situation. Numbers in red fail the Factor of Safety assessment or fall outside the margin of safety required.

**Larssen 22/25 are the types of sheet piles at Watchet Harbour – modelling has been done on equivalent sheets to estimate likely current performance depending on the grade of steel (this is unknown at this stage).*

1. Introduction

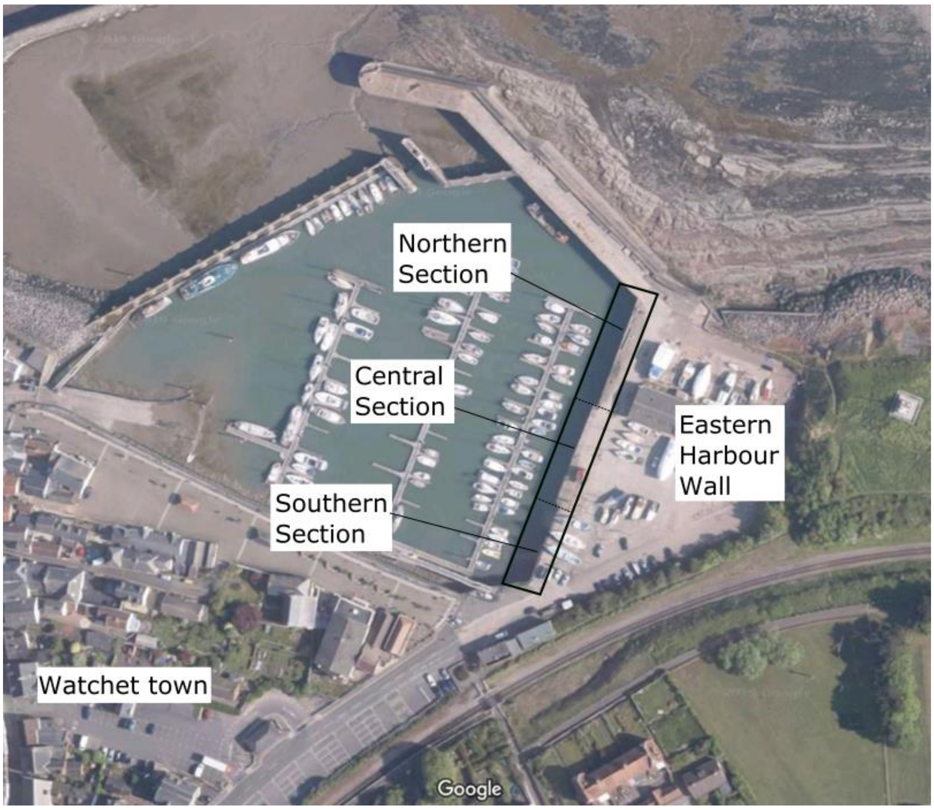
Crouch Waterfall have been commissioned by Pick Everard ('the Client') to undertake detailed geotechnical analysis of the existing east quay wall ('the asset') at Watchet Harbour, West Somerset, TA23 0AQ. The ultimate client and owner of the harbour and quay is Somerset West & Taunton Council.

The town of Watchet is located in south-west England, on the northern Somerset coast, 15 miles to the west of Bridgwater. The town is home to a marina which is contained by a combination of concrete/masonry and sheet piled harbour walls to the north and east, with the town of Watchet lying to the south (see Figure 1 of Watchet Harbour, below).

The eastern harbour wall backs onto the east quay which is currently used as a boat park and storage area. The east quay fulfils a number of functions including flood defence for the town and a working quay for the marina (including boat parking and storage area). It is proposed to redevelop this area with a community arts centre that will lie 14.5m from the harbour wall at its closest point. The proposed redevelopment will be serviced by an access road from the Esplanade, that runs immediately behind the edge of the harbour wall. The proposed development is to be designed and constructed by others.

The focus of this report is to look in detail at the make-up of the eastern harbour wall along its length and attempt to determine the state of this asset and how it might react to proposed future loading scenarios.

Figure 1: Overview of Eastern Harbour Wall, Watchet



2. Existing Information

2.1. Report References

The following reports were supplied by the Client and used during the analysis:

- Scope Document, issued by Pick Everard, June 2019, JRBB/MGA/190315/17-3/R102 Issue 1;
- Assessment of Potential Lateral Loads on the Quay Wall due to Raft Loads, issued by Red Rock Geo, June 2019, RP7090/C001;
- Watchet Harbour – Sea Wall Investigation, issued by Henderson Thomas Associates, December 2018, L/1748/18/WDT Rev 2;
- Quay Wall Survey – Watchet Marina, issued by Marine & Civil Solutions, November 2018;
- Geotechnical and Geoenvironmental Assessment of Watchet Harbour, issued by South West Geotechnical, January 2019, Ref 10501 Issue 2;

2.2. Eastern Harbour Wall

The eastern harbour wall comprises two distinct forms of construction, as per Figures 2-4, and summarised in Table 1 (levels/thicknesses taken from dive survey report provided by the Client) below:

- Stone masonry / in-situ concrete (believed to be unreinforced);
- Sheet piles supported by deadman anchors;

The stone masonry / in-situ concrete make up the northern and southern sections of the harbour wall, with the sheet piles located in the central section.

A thickness of soft silt has built up over the base of the marina. The thickness of this silt was found to fluctuate along the line of the harbour wall, varying from 1.55m to 3.35m at the time of the survey. The variation in thickness of silt deposits within the marina is predominantly believed to come from the proximity to the marina entrance: i.e.: thickest in the north which is closest to the marina entrance. Other factors might include tidal scour from an outgoing tide. In addition to these, Watchet Harbour Marina have proposed dredging the silt from the marina, in order to increase the draft under boats that use the marina.

TABLE 1: EASTERN HARBOUR WALL DETAILS

Section ID & Make-up	Full wall height (excl. embedment)	Thickness of silt
Northern – masonry/concrete	10.05m	3.35m
Central - sheet pile circa 1970's High level ties/northern half	9.70m	3.1m
Central - sheet pile circa 1950's Low level ties/southern half	9.70m	3.1m
Southern – masonry/concrete	8.45m	1.55m

Figure 2: Indicative Cross-Section through Northern Section of Harbour Wall

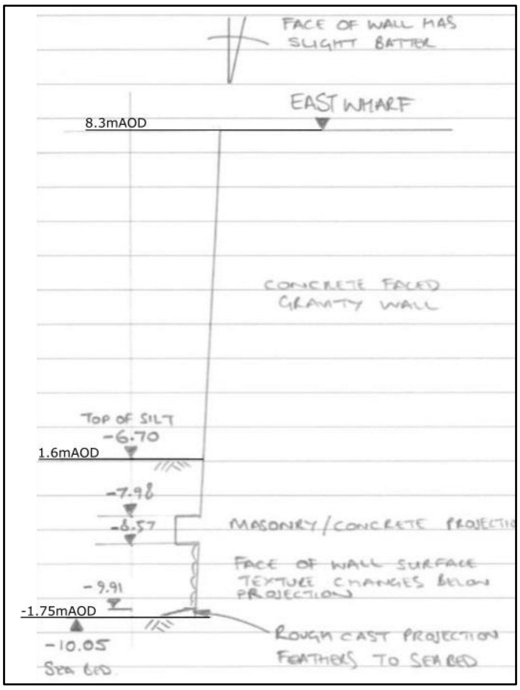


Figure 3: Indicative Cross-Section through Central Section of Harbour Wall

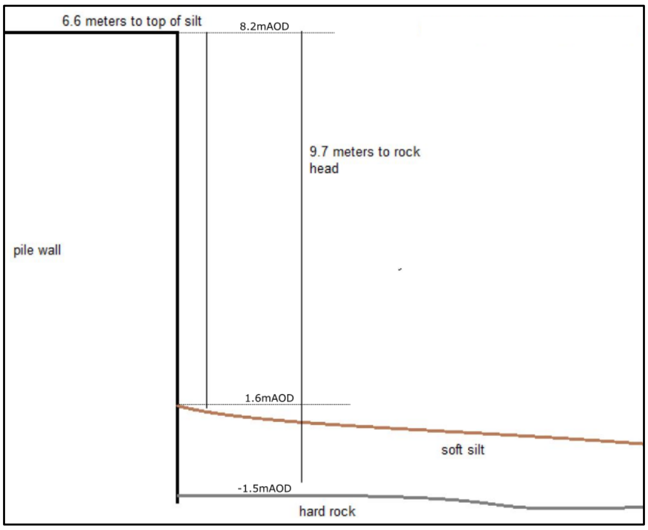
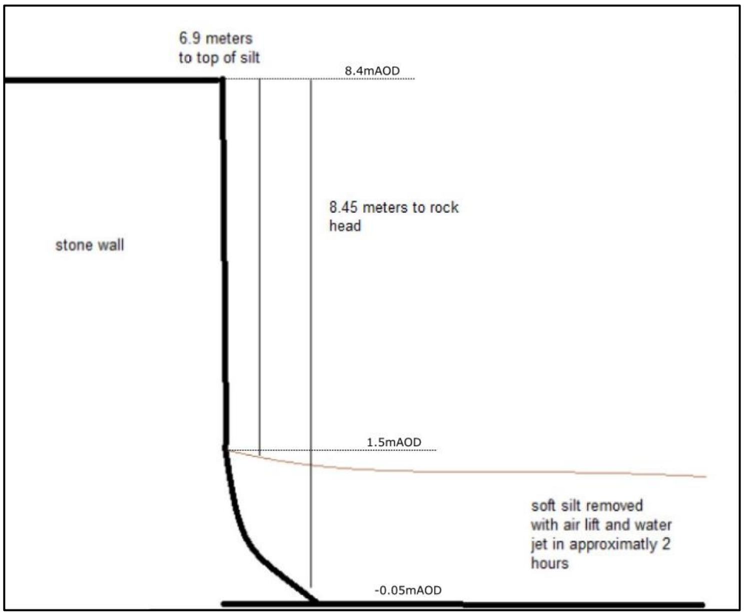


Figure 4: Indicative Cross-Section through the Southern Section of Harbour Wall



2.2.1. Central Section – Sheet Piled Wall

It is understood that the sheet piles were installed on two separate occasions: the first in the 1950s and the second in the 1970s. Installation of sheet piles on both occasions is believed to have been as a result of collapse or failure of the masonry wall, but no as-built information or details of the construction methodology has been made available. These two separate installations can be identified by the anchor heads located at two distinct levels: the 1970s installation having high-level anchors (located in the northern half of the central section), and the 1950s installation using low-level anchors (located in the southern half of the central section).

The selection of the sheet-pile sections is based on measured/estimated dimensions. Larssen 22 sheets have been identified in the northern half of the Central Section (installed in 1970s), utilising high level anchor ties. Larssen 25 sheets are believed to be present in the southern half of the Central Section (installed in 1950s), utilising low level anchor ties.

Based on dive survey findings, a 3.1m-thick layer of silt was encountered in front of the sheet pile wall. The diver was not able to tell how far the piles penetrated into the underlying bedrock. The length of the sheet piles, to the point where they enter the bedrock, have been measured at 9.7m.

The thickness of the existing sheet piles has been estimated based on the published parameters of the Larssen 22/Larssen 25 sheets. However, from dive surveys commissioned by the Client, it is understood that the sheets have developed large areas of rust, and therefore the loss of thickness due to corrosion is uncertain and could be significant.

Following a ground investigation, the two different sets of anchors were discovered lying at 2.0m/6.2mAOD (northern half, installed in 1970s) and 3.5m/4.7mAOD (southern half, installed in 1950s) below the top of the sheet pile wall. The horizontal spacing of the anchors has been estimated at 0.77m-1.0m based on photographs provided by the Client. The anchor bars have been measured at 14.3m to 14.4m long and 63.5mm diameter. The anchor ends are set into concrete blocks of varying sizes.

2.2.2. Northern & Southern Sections – Concrete/Masonry Wall

The masonry/concrete sections of the harbour wall were surveyed using ground penetrating radar (GPR) as well as cored sections taken from multiple points on the face of the walls. Based on the GPR results the wall thicknesses have been estimated at 1.0m to 1.3m for both sections. However, the cored sections for each of the walls give varying thicknesses of intact concrete, ranging from 170mm to 1260mm. For the purposes of simplicity in the modelling exercise, the masonry walls have been modelled as 1.0m thick, as per the GPR survey.

The full wall height in the northern section has been measured as 10.05m, with 3.35m of silt at its base. The full wall height in the southern section has been measured at 8.45m, with 1.55m of silt at its base. The foundations for each of the masonry wall sections are unknown. The dive survey did not find any evidence of a shear key binding the wall to the underlying bedrock.

2.3. Ground Model

A ground model has been produced based on the information within the Geotechnical Investigation Report (GIR) produced by South West Geotechnical (Ref. 10501). This investigation comprised 5 No. boreholes, 2 No. plate load tests and assorted lab testing. Individual ground profiles were produced for each of the Northern, Central and Southern sections of the harbour wall, based on the closest boreholes. These are summarised in the tables below.

Table 2: Northern Section

Level top (mAOD)	Level base (mAOD)	Soil Description	Comments
8.3	2.3	MADE GROUND: clayey GRAVEL	Based on BH101
2.3	1.3	Clayey GRAVEL	
1.3	-4.2	Weak-medium strong Mercia MUDSTONE	

Table 3: Central Section

Level top (mAOD)	Level base (mAOD)	Soil Description	Comments
8.2	1	MADE GROUND: clayey GRAVEL	Based on BH103
1	-4.3	Extremely weak Mercia MUDSTONE	

Table 4: Southern Section

Level top (mAOD)	Level base (mAOD)	Soil Description	Comments
8.4	3.3	MADE GROUND: clayey GRAVEL	Based on BH105. Limestone bands encountered in BH105 have been ignored in the design
3.3	-2	Very weak Blue Lias MUDSTONE	

In the above tables, the Blue Lias Mudstone and Mercia Mudstone will be treated as one and the same.

A table summarising the ground parameters assigned to these soil types is presented below.

Table 5: Soil Parameters

Soil Type	Unit Weight γ (kN/m³)	Young's Modulus E (MPa)	Poisson's Ratio ν	Angle of Shearing Resistance ϕ (° deg)	Cohesion c' (kPa)
MADE GROUND: clayey GRAVEL	18	16 *	0.4	34 ᐞ	16 ᐞ
Clayey GRAVEL	19	16	0.35	33 ^	1
MUDSTONE	22	30	0.45	0	400 '
Harbour SILT	18	5	0.3	20	1
Masonry wall FILL	22	100	0.1	-	-

Notes:

** This figure is based upon the results from the 2 No. plate load tests completed on site. Two stiffness values were calculated for the Made Ground material, and the more conservative value has been used in this analysis.*

ᐞ This figure is based on shear-box testing results (Taken from South West Geotechnical GIR, Ref 10501)

^ This value is based upon in-situ SPT testing.

ᵗ *This value is based upon the unconfined compressive strength (UCS) laboratory testing. The most conservative test result produced a UCS of 0.8MPa (UCS/2 ≈ c').*

Based on the borehole information and particle size laboratory testing, Crouch Waterfall have concluded that the risk of the soils/foundations degrading due to freeze/thaw action is negligible.

2.4. Tide Levels

Historic tide level data was not available for the site at the time of writing; however, long term monitoring has been taking place at Hinkley Point since 1990. Due to the proximity of Hinkley Point (located 14km to the east) to the site, the tide levels for Hinkley have been adopted for Watchet Harbour. The highest and lowest tide levels for the period 2008 – 2026 are listed in the table below (taken from the UK National Tidal & Sea Level Facility https://www.ntslf.org).

It is understood that the marina is partially impounded and therefore never completely empties at low tide (thus ensuring that the boats always remain afloat). This impounded level has been estimated at +1.5mAOD. For the purposes of this modelling exercise, the impounded level (+1.5mAOD) has been used instead of the *actual* low tide level (-6.09mAOD).

TABLE 6: TIDE LEVELS

Scenario	Level (Tidal Datum)	Level (Ordnance Datum mAOD)
High Tide	13.02m	+7.12mAOD
Low Tide	-0.19m	-6.09mAOD
Low Tide - Impounded Level	-	+1.5mAOD

2.5 Assumptions, Exclusions and Caveats

Modelling has been undertaken with due regard to the available information. However, there are significant areas in which information is not available and has had to be assumed for the purposes of modelling, and are as follows:

- Sheet-pile embedment is assumed to be 0.5m;
- Based on investigative surveys completed by the Client, the deadman anchors, supporting the sheet pile wall, are determined to be free from corrosion and are not detrimentally affecting the structural integrity of the harbour wall;
- The presence of a shear key has been discounted;
- The thickness of the masonry wall has been assumed as being 1.0m;
- The masonry and concrete wall is assumed to be unreinforced;
- The steel grade (yield strength) of the sheet pile wall has been assumed as 240MPa;
- Ground strength information is based on available information and published data;
- Accurate limits for the high tide and low tide levels were not available for Watchet Harbour, and so the tide levels have been taken from the nearby tidal measuring station at Hinkley Point power station;

3. Modelling Results and Interpretation

3.1 Introduction

Both finite element modelling (FEM) and limit-state modelling has been carried out on all three structural sections. Initially, FEM modelling has been carried out to gain an understanding of the forces acting on the existing structures. These forces have then been incorporated into limit-state models in order to provide Factor of Safety (FoS) values for the structures.

3.2. Finite Element Modelling

Finite element models for the northern, central and southern sections were produced using the profiles and parameters mentioned above. In addition, three tidal situations were chosen by the Client, namely:

- High Tide with harbour silt present in front of the existing structures;
- Low Tide (impounded) with harbour silt present in front of the existing structures;
- Low Tide (impounded) with the harbour silt dredged/removed to harbour floor level;

Various surcharges to accurately model the possible future development of the quay were chosen by the Client, namely:

- A **10kPa** load is placed over a 10m-wide strip immediately behind the wall. This simulates a generic load for day-to-day use of the quay/harbour wall – it represents the 'current' situation;
- A **20kPa** load replaces the 10kPa load mentioned above. This 20kPa load is applied over a 10m-wide strip immediately behind the wall and simulates the operation of a mobile crane used to lift boats in to/out of the marina.
- Static (dead) load of **50kPa** imposed by the development, located 14.5m (minimum) away from the harbour wall;

Load combinations (as specified by the Client) have been analysed for all three sections of the wall and all tidal situations as follows:

Load Scenario 1: 10kPa loading – 'current' situation;
Load Scenario 2: 20kPa loading – proposed crane loading;
Load Scenario 3: 10kPa + 50kPa loading – 'current' load + proposed development load;
Load Scenario 4: 20kPa + 50kPa loading – crane load + proposed development load

GEO5 Finite Element Modelling (FEM) software was used to perform the analysis for all three sections of the harbour wall.

A detailed drawing showing the layout of the harbour and locations of the crane operating area and proposed development is appended to this report.

3.2.1. Central Section – Reduced Thickness Sheet Pile Wall

Crouch Waterfall were advised by the Client on the types of sheet piles used in the Central section of the wall. However, following the findings of the dive survey, it was necessary to take into account corrosion and subsequent loss of section of the sheet piles. This was completed by following guidance in BS EN 1993-5:2007 Eurocode 3 – Design of Steel Structures – Piling. The following values have been generated following a reduction in the sheet-pile section based on this guidance:

TABLE 7: SHEET PILE - REDUCED THICKNESSES

Sheet Pile ID	Original thickness (mm)	Loss of thickness on Soil Side (mm)		Loss of thickness on Seawater Side (mm)		Reduced Sheet Pile Thickness (mm)	
		After 45yrs	After 65yrs	After 45yrs	After 65yrs	After 45yrs	After 65yrs
Larssen 22	10	0.55	0.75	3.5	5.0	5.95	4.25
Larssen 25	25	0.55	0.75	3.5	5.0	20.95	19.25

Following the guidance in BS 1993-5, the corrosion values for the Low Water/Splash Zone have been used as these are most onerous case. It should be noted that if it is assumed that Larssen 22 sheets were installed in the 1970s, then these sheets will be approaching the end of their design life, with potentially only ~6mm of thickness remaining in the most corroded parts of the piles.

In order to model the performance of a 45yr/65yr old Larssen 22 or 25 sheet pile, a sheet pile with the appropriate thickness must be modelled in its place (i.e.: 5.95mm or 4.25mm for a Larssen 22 and 20.95mm or 19.25mm for a Larssen 25). Larssen 25 sheets were replaced with Larssen 605 sheets, with a thickness of 12.5mm. However, a comparable sheet for the Larssen 22 could not be found; therefore, a GU6N pile was used in its place.

Following the application of reduced section values, modelling was undertaken to evaluate the performance of the harbour wall after 45yrs/65yrs of corrosion has reduced the thickness of the piles. In the following sections, only the reduced thickness sheet piles will be analysed (GU6N and Larssen 605) and the original, full thickness sheets (Larssen 22 and 25) have been ignored in the analysis.

3.2.2. Central Section – Sheet Pile Wall

Computations covering the various tidal scenarios and surcharges (Loading Scenarios 1-4) were completed for the central section of the harbour wall. A pile embedment depth of 0.5m into the underlying bedrock was assumed. This figure was chosen following analysis into the minimum embedment depth required to ensure stability of the wall, under the low tide without silt condition. This resulted in a total length of sheet pile of 10.2m. Given the nature of the underlying Mercia/Blue Lias mudstone a maximum embedment value is expected to be in the order of 2.0m. This depth is based on engineering judgement and working knowledge of the Mercia/Blue Lias mudstone.

The following results were achieved:

TABLE 8: SHEET PILED WALL (10.2M LONG SHEETS) – HORIZONTAL DISPLACEMENTS & BENDING MOMENTS

Model	Sheet Pile ID & Section	Max Bending Moment (Capacity) of Sheet (kNm)	Load Scenario 1		Load Scenario 2		Load Scenario 3		Load Scenario 4	
			Disp (mm)	Bending Moment (kNm)	Disp (mm)	Bending Moment (kNm)	Disp (mm)	Bending Moment (kNm)	Disp (mm)	Bending Moment (kNm)
High Tide + Silt	GU6N (6mm)	150	39	87	41	89	51	88	53	90
Low Tide + Silt			42	45	44	47	55	47	56	49
High Tide + Silt	Larssen 605 (12.5mm)	484	31	172	33	178	43	178	45	183
Low Tide + Silt			40	136	41	142	52	142	53	148

Load Scenario 1: This gives some indication as to how the wall is reacting to the 'current' applied load.

Load Scenario 2: With the addition of the crane load, the sheet pile wall is noted to displace an extra 1-2mm, with minor increases in bending moment.

Load Scenario 3 & 4: Once the development load is added to the 'current' situation, the displacements were noted to increase by ~12mm. The same can be said when the development load is applied in addition to the crane loading.

The significant increase in bending moment for the High Tide with Silt scenario is believed to be due to the increased water pressure acting on the back of the sheet pile wall.

In general terms, Table 8 shows that the sheet piles do not exceed their bending moment capacity in any of the Load Scenarios, despite experiencing some significant horizontal displacements.

3.2.3. Northern & Southern Sections – Masonry/Concrete Wall

Computations covering the various tidal scenarios and surcharges were completed for the northern and southern sections of the harbour wall. Due to the difficulties in accurately modelling a masonry wall in finite element software, only the horizontal displacements have been computed.

The results of the analysis are tabulated below.

TABLE 9: NORTHERN MASONRY WALL - HORIZONTAL DISPLACEMENTS

Scenario	Load Scenario 1	Load Scenario 2
	Horizontal Displacement (mm)	
Low Tide + Silt	41	43
High Tide + Silt	62	66

For the Northern masonry wall analysis, Load Scenarios 3 & 4 were ignored under direction from the Client. The northern masonry wall is located far enough from the proposed development for it to lie outside the zone of influence.

A small increase of 2-4mm is noted in the transition from the ‘current’ situation (Load Scenario 1) to the addition of the crane load (Load Scenario 2).

As with the Central section, the increased displacements during the High Tide with Silt scenario are believed to be caused by the increased water pressure acting on the back of the masonry wall. It is not clear why this is only evident in the analysis of the Northern section.

TABLE 10: SOUTHERN MASONRY WALL - HORIZONTAL DISPLACEMENTS

Scenario	Load Scenario 1	Load Scenario 2	Load Scenario 3	Load Scenario 4
	Horizontal Displacement (mm)			
Low Tide + Silt	38	39	50	51
High Tide + Silt	32	36	43	47

As with the central sheet pile section above, only minor increases in displacements are noted when moving from Load Scenario 1 to 2. More substantial displacements are noted when the development load is included in Scenarios 3 and 4.

3.3. Limit State Modelling

Limit State models were employed in an to attempt to assess the predicted performance of the harbour wall in terms of a Factor of Safety (FoS) value.

In the following analyses, Eurocode 7 partial factors were ignored, in order to have greater parity with the original British Standard design methods that would have been used at the time.

3.3.1. Central Section – Sheet Pile Wall

The Central section of the harbour wall was modelled using Larssen 22 and Larssen 25 sheet piles with progressively reduced thicknesses (a product of corrosion). In addition to this, a sensitivity analysis was carried out on the effect of steel grade on the performance of the sheet piles. The piles were assigned an embedment depth of 0.5m (total pile length of 10.2m)

In order to complete this analysis, typical sections of Larssen 22 and Larssen 25 sheets were modified to reduce the thickness of the sheet (as though being corroded), and in doing so compute the reduced structural parameters that would accompany the loss of section. A steel grade of 240MPa was chosen for the yield strength. It should be noted that changing the thickness and grade of steel only affects the Factor of Safety on the bending moment of the sheet piles. The following tabulated results were calculated for Load Scenario 1 (10kPa surcharge behind the wall). The addition of the development load (Load Scenarios 3 & 4) has been determined to have no effect on the Central Section of the harbour wall, due to the location of the development.

The same analysis was completed for Load Scenario 2 (20kPa crane load), and separately, for the Low Tide with silt dredged scenario, but neither of these resulted in a significant change in the bending moment factors of safety shown below, and therefore the results have not been replicated here.

TABLE 11: LIMIT STATE ANALYSIS - 240MPA YIELD STRENGTH STEEL – LOAD SCENARIO 1

Scenario	Sheet Pile ID & Thickness (mm)	Bending Moment FoS		Scenario	Sheet Pile ID & Thickness (mm)	Bending Moment FoS
Low Tide + Silt	Larssen 22 – 6mm	0.7		High Tide + Silt	Larssen 22 – 6mm	2.7
	Larssen 25 – 15mm	1.1			Larssen 25 – 15mm	4.1

In Table 11 above, the bending moment factors of safety have been calculated for both Larssen 22 and Larssen 25 sheet piles, with varying levels of reduced thickness. The calculated factors of safety which fall below 1.25 have been highlighted in red. For the basis of this investigation, a Factor of Safety greater than 1.25 is deemed 'acceptable' (the minimum Factor of Safety required by British Standards is 1.25). The stated thicknesses (6mm for Larssen 22 and 15mm for Larssen 25) were advised by the Client.

3.3.2. Northern & Southern Sections – Masonry/Concrete Wall

The same Limit State analysis was performed for the northern and southern sections of masonry wall. During the analysis, there was a degree of uncertainty around the dimensions of the foundations for the walls, as well as the presence of any kind of shear key. To maintain simplicity of analysis, the foundation dimensions (on the seawater side) that were recorded during the dive survey have been mirrored on the soil side of the wall.

The following Overturning (OVT) and Sliding (SLI) Factor of Safety values were recorded:

TABLE 12: NORTHERN SECTION - LIMIT STATE ANALYSIS – FACTOR OF SAFETY

Scenario	Load Scenario 1		Load Scenario 2	
	SLI	OVT	SLI	OVT
Low Tide + Silt	7	1.3	1.3	1.06
High Tide + Silt	10+	4	10+	2.5

For the Northern masonry wall analysis, Load Scenarios 3 & 4 were ignored under direction from the Client. The northern masonry wall is located far enough from the proposed development for the increased loading to be insignificant.

From the table above it is apparent that under 'current' conditions (Load Scenario 1) the lowest FoS the northern masonry wall might experience is FoS=1.3. Should Somerset West & Taunton council decide to dredge the silt from the marina then this would drop to FoS=1.1. Factors of Safety that are <1.25 have been highlighted in red.

Should the proposed crane be used within the 10m-wide strip immediately behind the northern harbour wall, the FoS drops to FoS=1.06 (under current marina conditions). Again, should Somerset West & Taunton council decide to dredge the marina silt, the FoS drops to less than unity (FoS=<1).

TABLE 13: SOUTHERN SECTION - LIMIT STATE ANALYSIS – FACTOR OF SAFETY

Scenario	Load Scenario 1		Load Scenario 2		Load Scenario 3		Load Scenario 4	
	SLI	OVT	SLI	OVT	SLI	OVT	SLI	OVT
Low Tide + Silt	10+	1.8	6	1.4	10+	1.8	6	1.4
High Tide + Silt	10+	10+	10+	4	10+	10+	10+	4

The southern section of masonry wall is not as tall as the northern section, and this is reflected in the higher Factors of Safety.

Load Scenario 1: The lowest safety factor values produced by the analysis occurred during the low tide scenario (FoS=1.8).

Load Scenario 2: When the crane load was applied to the model, the FoS dropped to FoS=1.4, for the low tide scenario.

Load Scenario 3 & 4: When the development load was applied in Load Scenarios 3 & 4, the FoS did not change, suggesting that the construction of the development will not have an effect on the southern section of masonry wall.

4. Conclusions & Recommendations for Additional Works

4.1. Conclusions

Following detailed geotechnical analysis into the different sections of the harbour wall, and the various loading scenarios / tidal scenarios that are being applied to the wall, it is possible to comment on the 'robustness' of the harbour wall.

Northern (Masonry) Section

Only a minimal increase (1-2mm) in horizontal displacement was recorded during FEM analysis when moving from the current situation (Load Scenario 1) to operating with the crane immediately behind the wall (Load Scenario 2). However, when this analysis was conducted using Limit State methods, this transition from Scenario 1 to 2 resulted in the Factor of Safety falling to FoS=1.06.

Central (Sheet Pile) Section

As with the northern section, only small increases in horizontal displacement were recorded during FEM analysis when moving from Scenario 1 to 2. Larger increases of 10-13mm were recorded when the development load was applied to the model.

In terms of Limit State analysis: calculations were completed on varying thicknesses of both Larssen 22 and 25 sheet piles, for both the high tide- and low tide- with silt scenarios. The factor of safety remained >1.25 for all of the high tide with silt scenarios (ie: both Larssen 22 and 25 sheets). The low tide with silt scenario produced some factors of safety <1.25 for Larssen 22 and 25 sheets, as shown in Table 11. Determining the steel grade and sheet pile thickness would give considerable confidence towards predicting the sheet pile capacity.

It is often difficult to reconcile hypothetical results from analysis of an existing situation, where the analysis predicts failure (FoS<1.0), and yet the structure remains standing. The reasoning behind this is believed to be (partly) due to the continuous, cyclical action of the tides. It is likely that the unstable low tide condition does not last long enough to bring about failure of the sheet piles, before the tide, and the Factor of Safety, start rising again. The excessive bending, and ultimate failure, of the sheet piles, would be a progressive process rather than a singular catastrophic event. It is believed that if the harbour were left dry for any significant period of time then there is a distinct possibility of bending failure of the wall.

Southern Section

Limit State modelling of the southern section did not produce any situations where the Factor of Safety dropped below 1.0. This is believed to be partly due to the fact that the southern wall has the smallest retained height, and therefore the lateral forces are less.

4.2. Recommendations for Additional Works

Whilst every attempt has been made to use realistic assumptions and not impose undue conservatism into the models, there are still some key areas of uncertainty.

4.2.1. Sheet Pile Walls

Several significant uncertainties still surround the central sheet piled section of the harbour wall. These include:

- The sheet piles were modelled as either Larssen 22 or 25 sheets, with reduced (assumed) thicknesses/strength parameters as appropriate due to corrosion.
- Detailed sensitivity analysis has been completed on the effect of reducing the thickness of the sheet and also reducing the grade of steel. If either/both of these parameters could be established, then it would give greater confidence in predicting the behaviour of the sheet pile wall. Determination of steel grade is possible through chemical testing of samples of the steel.
- Depth of embedment into bedrock: this could potentially be achieved through the use of geophysical surveys;

4.2.2. Masonry Walls

- Significant uncertainties surround the base of walls and their foundations: are the foundations embedded to any extent? The thickness/dimensions of the walls is also key to ensuring the existing situation is modelled accurately;
- Based on the investigations completed to date there appears to be some variation over the thickness of the masonry/concrete;

However, notwithstanding the above recommendations there is a strong possibility that further investigation work would **not** necessarily result in significantly better/improved model outputs that reduce the perceived risk to the harbour walls. Therefore, it may be prudent to consider other options that could reduce the impact on the harbour walls, such as: limiting the extent of the crane operating area, effecting repairs, or strengthening the harbour wall.

Consideration must also be given to the fact that theoretical reduction of section as a result of corrosion, coupled with the results of the dive survey, suggests that the sheet-piled section of the development is nearing the end of its working life. While improvements such as propping could be considered to increase the capacity of these structures, this is not seen as a workable solution in the medium – long term.

Drawings