Main Pier Sheet Pile Condition Survey at Daingean FHC, Co. Kerry

Condition Survey Report

DOCUMENT CONTROL SHEET

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(Provided on CD for reference)

1.0 Executive Summary

Survey of the sheet pile wall (approx 195m length on plan) and exposed sections of the tubular steel bearing piles (35Nr) of the Main Pier at Dingle Harbour, Co. Kerry (13/14 Aug 14) has identified varying condition of these steel elements with some notable deterioration.

Sheet Pile Wall:

- Deterioration and significant loss of wall thickness was noted on the Southern 40m of the pier wall with the most severe corrosion from chainage 0m to 10m. The majority of thickness readings obtained in this location represented greater than 50% loss of nominal wall thickness. This zone spans northwards from the final sheet pile near the southern tip of the structure to and encloses the tubular piles of the structure which finishes in line with the revetment/bund structure to the east of the main pier. This zone was characterised by widespread corrosion, blistering, pitting and significant loss of wall thickness and includes some evidence of possible accelerated low water corrosion. It is suggested to investigate this further.
- Some loss of wall thickness was noted across the remaining length of the wall; however this is significantly lower than the Southern 40m zone noted above.

Steel Tubular Bearing piles:

- Some loss of pile wall thickness was encountered. However readings obtained were largely consistent and no evidence of significant corrosion beyond normal rates was noted.
- While beyond the scope of this appointment deterioration of the substructure concrete elements beneath the deck of the pier was noted at multiple locations. This should be investigated further at earliest opportunity.

2.0 Introduction

RPS were appointed by the Department of Agriculture, Food and the Marine to carry out a condition inspection of the sheet pile wall and bearing piles of the Main Pier at Dingle Harbour, Co. Kerry. The appointment included inspection of the piles, reporting on condition and recommendation of any repairs or improvements required.

Inspections were carried out on the 13th and 14th August 2014.

Above water inspections of the sheet piles were carried out by an Engineer from RPS assisted by a representative from ABCO Divers from a small locally piloted vessel.

Below water inspections of the sheet piles were carried out by ABCO Divers under the direction of an Engineer from RPS via hard-wired audio line.

Inspection of the tubular piles was undertaken at low tide from above water level within the structure by an Engineer from RPS assisted by 2 representatives from ABCO Divers.

Non-destructive thickness measurements were taken using an ultrasonic thickness gauge. Measurements were supported by notes taken by the engineer and record photography taken by ABCO and RPS.

<u>Sheet Pile Wall</u>

The 195m length of sheet pile wall on the Eastern side of the structure (including piles on the Eastern side of the Punt Slip) was inspected. Thickness measurements were taken at 5m intervals below water and at accessible intervals between berthed vessels above water.

Below water the diver was able to swim between measurement locations and perform a general inspection of the piles. The full surface area of all piles was not inspected in this sweep; however a general condition inspection was undertaken across the diver's field of view and it was noted by the inspecting diver that the water visibility was very good (with the exception of a brief period following movement of a berthed vessel) and that the majority of the full surface area of the wall could be seen below water level.

Inspection locations were setup at regular intervals along the structure with inspection and thickness measurements taken at multiple heights within the water column. At each location measurements were taken on the web of the Outpan (OP), the web of the Inpan (IP) and the Flange (or side face: SF) of the pile with a schedule of results recorded. These results are discussed and analysed within section 4 of this report with a full schedule of all recorded measurements presented under Appendix 1 to this document.

Tubular Bearing Piles

The tubular bearing piles at the Southern tip of the pier are enclosed. These are behind the final 50m (approx) of sheet pile wall on the Eastern face of the structure and the concrete wall panels on the Southern and Western faces of the structure.

Inspection was undertaken within the structure above water level at low tide with measurements taken at regular heights between bed and maximum accessible height (arms length above head height - approx 2.30m).

This report presents the results of site inspections, discusses the general condition of the piles and makes recommendations for remedial, improvement or protective works where appropriate.

Survey results and record photographs are presented in Appendices (1 to 3) of this report.

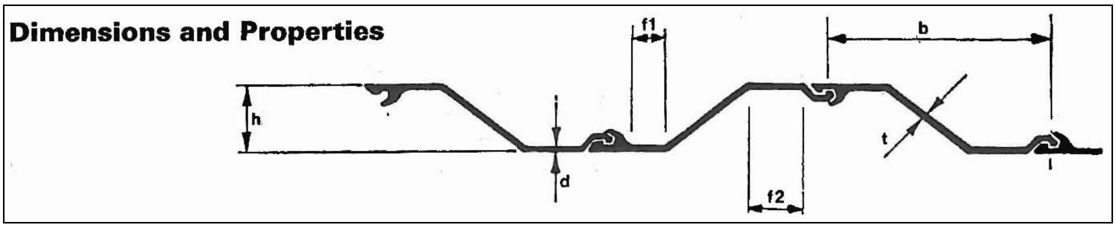
3.0 Form of Construction

3.1 Structure History

The main pier at Dingle Fishery Harbour Centre was has been upgraded and extended on a number of occasions following its initial construction.

Obtained record drawings and report indicate that the original solid masonry pier structure was extended in 1927 and widened in 1975. The works in 1975 included the installation of Frodingham 1BXN steel sheet piles installed on the Eastern side of the pier.

A typical view of the installed cross section of the sheet piles is shown in Figure 1 below with a typical photo elevation in Plate 1.



b mm (nominal):	476mm
h mm (nominal):	143mm
d mm	12.7mm
t mm (nominal)	12.7mm
Mass	62.1kg/m

Figure 1: Main Pier Sheet Pile Wall – Pile size diagram

Plate 1 – Typical view of Eastern face sheet pile wall

More recently a Southerly projecting structural extension of almost 60m was constructed in 2000 to 'square off' the structure just beyond the end of existing sheet pile section on the eastern face of the structure.

This extension features dual concrete wall panels (with concrete filled void between) to the Southern (end) face and western face of the structure.

A new suspended reinforced concrete deck spans between the wall panels and 35Nr new 660mm x 25m CHS steel tubular piles. (Plate 2).

Plate 2 – Tubular steel bearing pile with collar prepared for testing

The open pile structure is also braced below deck level with an arrangement of reinforced concrete sub structure elements which it is understood were installed in 1975.

Given the above it is understood that the steel structural members inspected are of the currently of the following age:

Element	Constructed	Current Age
Sheet Pile Wall	1975	Approx 40 years
Tubular Bearing Piles	2000	Approx 15 years

Further details on the form of construction are shown on the structure record drawings and historical report under Appendix 4 to this report.

3.2 Structure Configuration

As noted in previous sections of this report; this inspection is focused on the condition of the steel sheet piles and bearing piles of the structure, their remaining lifespan and the possible options for their maintenance/remediation as appropriate.

However with the multiple extension and upgrades to the structure to date; it is also relevant to note the current configuration and the structural contribution provided by these elements. This will provide additional clarity and assistance when prioritising any remedial action to elements of major structural or operational significance.

A sketch showing structural developments to the structure is shown in Figure 2 below:

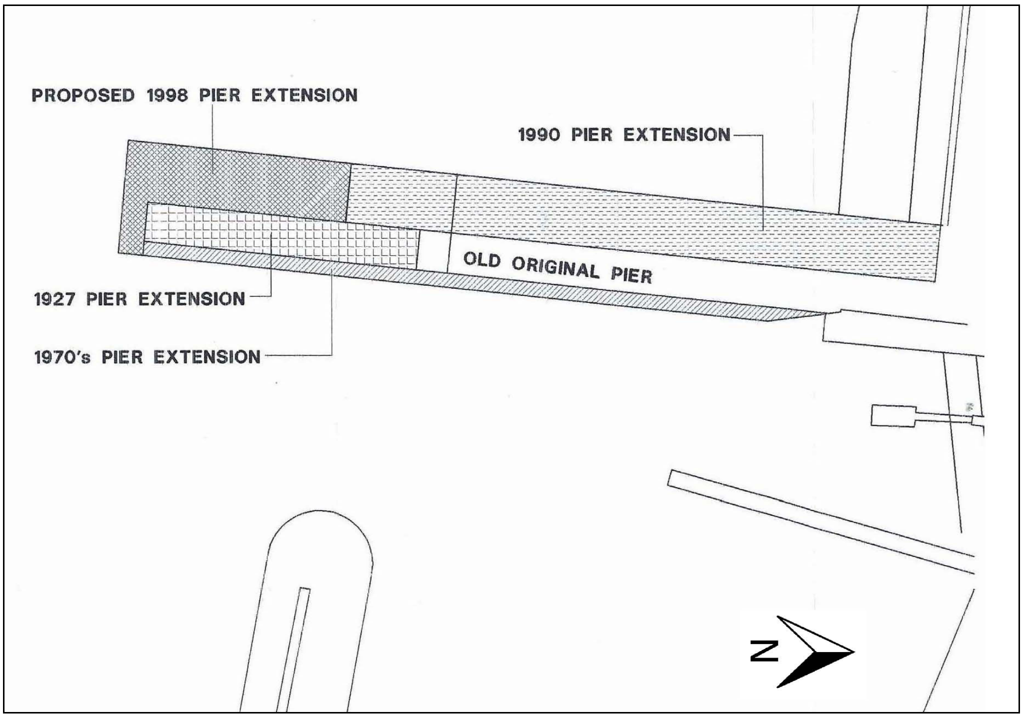


Figure 2: Structural Development works

Steel Tubular Bearing Piles

The extension//refurbishment works of 2000 was conducted to widen and strengthen the Southern end of the structure to allow the installation of the Ice Plant. The steel tubular bearing piles provide vertical support for the deck slab.

(Additional vertical support and lateral stability is provided by an arrangement of vertical and horizontal (at multiple heights) concrete members below deck level between the steel tubular piles installed previous to the works of 2000. (Plate 3).

Plate 3 – Support arrangement below deck including central tubular steel pile

Steel Sheet Piles

The steel sheet piles on the Eastern face of the structure span between the root of the pier and approx 6m from the Southern end of the structure.

“Old Original Pier” portion of the structure:

Record documentation provides the section size, date and indicative depth of installation but further details on the exact toe depth is not available.

It is understood that the sheet piles are tied back to reinforced concrete piles (installed approx 1973) with the surrounding void backfilled.

No further detail is available to calculate the exact structural contribution of these elements to the stability of the structure. Furthermore; confirmation of the exact arrangement of the structural connections and current condition will not be possible without destructive investigation.

The piles remain integral to the structure which has not been modified in this location following their installation in 1975.

Southern end of the Structure (Open Pile):

In its current structural form; the final 50m (approx) length of sheet piles span (curtain wall) across the open pile section of the eastern face of the structure.

As per the landward section - historical record drawings (under Appendix 4) indicate that these sheet piles were installed in front of a number of RC piles (15” x 15” at this location).

It is unknown if the 1927 extension to the structure was solid or open pile and if the sheet piles were installed to provide a structural contribution to the stability of the pier or as a wave screen to improve berthing conditions and/or provide a vertical berthing face for fender installation.

During the inspection it was identified that the upper sections of the sheet piles do have some fixings to the longitudinal edge beam of the deck slab, however these are in poor condition and many appear to have been released/loosened previously.

Given the above and the recent upgrade works of 2000 to extend and strengthen the deck and install piles to accommodate the increased deck loading of the ice plant; it is considered that these sheet piles are not currently providing any significant structural contribution to the support or stability of the structural members of this area of the structure.

While these piles may not be providing significant structural contribution they are not redundant members and their condition and requirement for retention should be considered carefully:

The sheet piles have timber fenders and rubbing strips installed on outpans at regular intervals which are used for berthing vessels.

The overall structure including the piles at this location, contribute to the current harbour conditions to the East of the Main Pier. Removal of piles may cause some impact on harbour disturbance levels. However it is considered that this impact will be minor as the southern end of the structure now features a solid wall on the western and southern faces which provide protection to waves from the predominant directions. This comment follows review of April 2008 Wave Study by RPS.

Given the current form of construction of the main pier and the understood contribution of both the steel tubular piles and the steel sheet piles the following is considered for each element:

Steel Tubular Piles

- It is important that the condition and integrity of the tubular bearing piles is maintained to ensure that the structure remains fit for purpose to support the ice plant and accommodate operational loads.

Steel Sheet Piles

- The requirement for retention of the steel sheet piles at the Southern end of the structure which span the length of the open pile section of the structure is to be considered. Deterioration of their condition may compromise the integrity of the fenders. These may be removed if required pending modification works to reinstate the fenders on the piles (if confirmed that berthing loads may be taken by the piles) and removal works create an acceptable impact on harbour conditions.
- The steel sheet piles of the solid portion of the structure are to be retained for structural purposes. These remain part of the structure as installed in 1975.

4.0 Condition of Piles

A full schedule of survey thickness results is given in Appendix 1 to this report with a plan of the survey area with reference chainages given in Appendix 2.

The results of the survey are discussed in two categories.

1. Condition of Sheet Piles
2. Condition of Bearing Piles.

Descriptions of the condition(s) encountered and results obtained are given below.

4.1 Condition of Main Pier Sheet Piles

No severe defects were encountered during the inspection of the sheet piles. However there are a number of isolated notable elements of deterioration:

Significant Loss of Sectional Thickness

Significant loss of sectional thickness was noted within the first 40m of the wall with the greatest loss of thickness within the first 10m where the majority of readings indicated loss of sectional thickness at greater than 50% as summarised below:

CH0m to CH10m (61 readings):

≥50% loss: 40 readings

25 to 49% loss: 7 readings

10 to 24% loss: 9 readings

5 readings indicate less than 10% loss of sectional thickness.

In order to compare/contrast this zone with the remainder of the structure a brief description summary is provided below

CH0m to 10m	Severe loss of pile sectional thickness
CH10 to 35m	Significant loss of pile sectional thickness
CH35 to 170m	Limited loss of pile sectional thickness

An analysis of the average readings obtained on the main pier sheet pile wall (excl the Punt slip) is provided within Table 1 overleaf. Categories have been coloured for visual comparison.

A full schedule of values obtained is provided under Appendix 1 to this report.

Nominal Pile Dims	d 12.7	d 12.7	t 12.7
	IP	OP	SF
Chainage	Average	Average	Average
0m	4.4	4.0	6.2
5m	6.3	4.7	6.5
10m	7.3	3.9	6.2
15m	7.8	7.9	7.6
20m	7.3	6.7	7.4
25m	9.2	12.4	9.4
30m	10.8	11.3	9.3
35m	10.8	9.6	9.5
40m	10.4	10.5	10.1
45m	11.7	11.1	11.0
50m	11.1	11.0	11.4
55m	11.0	11.0	11.1
60m	11.2	10.7	10.3
65m	11.3	10.4	9.9
70m	10.8	10.6	10.5
75m	10.7	11.3	11.0
80m	10.7	11.1	10.7
85m	10.3	10.3	10.7
90m	11.0	10.8	10.6
95m	10.6	10.4	10.8
100m	11.3	10.4	10.3
105m	11.0	11.3	11.1
110m	11.3	11.4	11.2
118m	11.1	11.6	11.5
130m	11.3	10.7	11.0
140m	11.8	11.5	11.3
150m	11.6	11.0	10.8
160m	12.2	11.8	11.9
170m	11.4	11.3	11.5

Overall Values

Max	**13.3**	**20.7**	**13.2**
Min	**1.4**	**1.5**	**1.4**
Average	**10.4**	**10.2**	**10.2**

0 to 9% loss	*<10% loss*
10 to 24% loss	*10% loss = 11.43m reading*
25 to 49% loss	*25% loss = 9.53mm reading*
50% + loss	*50% loss = 6.35mm reading*

Table 1: Analysis of Average Sheet Pile Readings obtained on Main Pier Wall

(all thickness values are quoted in mm)

Evidence of Corrosion

Evidence of ongoing corrosion was noted on the sheet piles of the main pier and the punt slip.

This was most severe between CH0m and CH40m (See appendix 2 for survey area plan with chainages). In this location widespread orange patches and pitting on the webs of the piles were noted across the full width of the pile face typically 400mm in height. Isolated locations were noted extending to 2m in height concentrated in the low water zone. Black sludge deposits behind orange patches were noted in many instances within this area. This indicates the possible presence of Accelerated Low Water Corrosion (ALWC).

The inspecting diver noted that oranges patches were also incident at upper tidal levels however patches were generally smaller than those at low water level and with limited/no encountered black sludge deposits.

Sporadic oranges patches of corrosion (including location at Ch150m) were identified along the remaining length of the structure beyond CH40m however the level (size and frequency) of corrosion noted by the inspecting diver was visibly reduced beyond CH60m.

The sheet pile wall exhibits widespread corrosion in the splash zone extending downwards from the crown of the piles over an approximate height of 1.8m (Plate4).

Plate 4 – Corrosion of sheet piles in Splash zone

Inspection identified typical degradation with ‘blistered’ and corroded steel and some loss of steel thickness when compared to areas where the pile was intact. It should be noted however that the deviation of thicknesses encountered (between the visually corroded and intact areas) was minor.

Holes

- Single hole identified in the permanently immersed zone at CH15m (150mm wide x 450mm ht). (Plate 5)
- Small holes in intertidal zone (4Nr) believed to be formed at CH0m, CH160m and CH170m.
- 2 Larger holes at approx CH40m in the splash zone (Plate 6)

Plate 5 – Hole in sheet pile wall at CH15m

Plate 6 – Holes in sheet piles

While holes are typically significant: the encountered holes are small and are suitable for localised repairs are not of major concern.

This is valid further for the holes within the first 57m of the wall (open pile section of the structure) given the current consideration that the sheet piles in this location are not essential structural elements.

Punt Slipway Wall

The above water survey conducted on the punt slipway wall at low water indicated a similar condition to that encountered on the main pier wall above water level.

(Piles were inspected at low tide level and as a result a below water inspection was not required and the full height of piles above ground/revetment level was accessible.)

An analysis of the average readings obtained on Punt slipway wall is provided within Table 2 overleaf.

A full schedule of values obtained is included within Appendix 1 to this report.

	d	**d**	**t**
Nominal Pile Dims	12.7	12.7	12.7
	IP (mm)	OP (mm)	SF (mm)
Chainage	Average	Average	Average
0m (Top of Slope)			
5m	10.3	10.1	10.3
	9.6	11.3	11.8
	12.4	9.8	10.6
	12.1	9.9	7.9
10m	9.8	11.0	8.7
	12.3	11.4	8.6
	11.6	11.4	8.8
15m	11.6	11.4	11.4
	10.7	11.6	12.4
	11.1	11.4	11.0
20m	11.1	11.6	11.0
	8.6	10.7	
25m	12.0	12.1	12.0
30m			

Overall Values

Max	12.4	12.1	12.4
Min	8.6	9.8	7.9
Average	11.0	11.1	10.4

0 to 9% loss		*<10% loss*
10 to 24% loss		*10% loss = 11.43m reading*
25 to 49% loss		*25% loss = 9.53mm reading*
50% + loss		*50% loss = 6.35mm reading*

Table 2: Analysis of Average Sheet Pile Readings obtained on Punt Slipway Wall

(all thickness values are quoted in mm)

Although the visual condition is similar to that noted on the sheet piles of the Eastern wall of the main pier structure, pile thicknesses recorded are generally higher than that of the main pier wall (thus less loss of sectional thickness versus the main sheet pile wall).

Also, like the main sheet pile wall surface corrosion is also firmly adhered to the surface of the pile and is providing a level of protection to the intact steel behind. Indeed it should be noted that the piles were extremely difficult to prepare for thickness testing (removing surface corrosion to bare steel behind by hand methods). A typical view of current condition is shown in Plate 7.

Plate 7 – Typical view on Punt slipway sheet pile wall

4.2 Condition of Main Pier Tubular Bearing Piles

No significant defects of concern were encountered during the inspection of the 35Nr tubular bearing piles of the structure however some degree of loss of sectional thickness was noted.

It was intended to inspect the pile within the structure from below water level at a high tide. However it was noted on initial inspection and survey planning that the complexity of the pile and bracing element arrangement would limit navigation by inspecting diver by surface supply restricting speed and level of access to the piles. It was also noted that due to the strength of the incident tides – the internal area of the open pile section of the structure almost completely dried out at low water. Thus it was decided to conduct the inspection from above water level either side of low tide.

It is relevant to note:-

1. With the exception of personal torches; the environment was almost completely dark. However intensive inspection using torches was conducted to all piles from existing bed level to the soffit of the deck slab.
2. Extensive marine growth was also incident on the piles and limited the full extent of the survey; however a large amount of this was cleaned at multiple heights on every pile to allow inspection at each measurement location.

Of the 171nr readings taken during the survey 43 readings obtained indicated a loss of sections thickness exceeding 10%. This relates to reading values between 22.5mm and 18.8mm = a loss of 2.5mm to 6.2mm thickness.

This equates to an upper rate of 0.18mm to 0.44mm loss of sectional thickness per year.

Table 25 from BS6349-1:2000 provides typical rates of corrosion for structural steels in temperate climates. Mean values are noted between 0.04 and 0.08mm/side/year and Upper Limits are noted between 0.1 and 0.17mm/side/year. If water enters the pile and there is adequate oxygen for corrosion to occur these rates equate to 0.16mm (mean) to 0.36mm (upper) per year for the measured pile wall thickness (corrosion on 2 sides).

These values equate roughly to those calculated from survey results and the quoted pile wall thickness. However, corrosion of the internal face of tubular piles is only likely to occur if there are holes in the piles or transfer of air up though the piles from the toe.

It should also be noted that the presence of steel collars/sleeves of various lengths between existing bed and the soffit of the deck slab was identified on the majority of piles. While further details are not currently available these would appear to be of similar wall thickness to the tubular piles (typical view of this arrangement in plate 2). It is not clear if these are sleeves from installation or collars from post installation repairs/upgrade and thus their function and additional structural contribution cannot be confirmed. Furthermore the extent and installation height does not appear to be consistent across the 35Nr tubular piles.

(The location of each survey measurement (pile or collar) is noted in the full schedule of results under Appendix 1).

Although readings obtained indicate (for the record nominal wall thickness) ongoing corrosion equal to the upper level of that expected for temporal climates it should be noted that no visual evidence of significant corrosion was encountered during the survey. Pending confirmation of the installed nominal wall thickness and thus the rate of any corrosion, it is considered that the piles are in reasonable condition for their age. (Plate 8)

Plate 8 – Typical view on tubular steel bearing pile

Further analysis of the values obtained is provided under Table 3.

A full schedule of readings obtained is given within Appendix 1 to this report.

Nominal wall thickness (mm)	25.0

Pile Thickness Reading Analysis		Reading (mm)	Loss of Thickness (Residual - Nominal (mm)): -ve = loss of thk +ve = no loss of thk	Percentage thk remaining
All Piles	Max	26.6	1.6	106.4%
	Min	20.1	**-4.9**	80.4%
	Average	24.4	**-0.6**	97.4%
Row Bb	Max	20.4	-4.6	81.6%
	Min	20.1	**-4.9**	80.4%
	Average	20.2	**-4.8**	80.8%
Row B	Max	26.2	1.2	104.8%
	Min	24.8	-0.2	99.2%
	Average	25.5	**0.5**	101.8%
Row C	Max	25.8	0.8	103.2%
	Min	22.0	**-3.0**	88.0%
	Average	24.7	**-0.3**	98.8%
Row D	Max	26.6	1.6	106.4%
	Min	24.6	-0.4	98.4%
	Average	25.5	**0.5**	102.1%
Row E	Max	25.7	0.7	102.8%
	Min	21.9	-3.1	87.6%
	Average	24.6	**-0.4**	98.5%
Row F	Max	25.7	0.7	102.8%
	Min	21.8	-3.2	87.2%
	Average	24.7	**-0.3**	98.6%
Row G	Max	25.6	0.6	102.4%
	Min	21.0	-4.0	84.0%
	Average	24.8	**-0.2**	99.3%
Row H	Max	25.5	0.5	102.0%
	Min	21.2	-3.8	84.8%
	Average	23.4	**-1.7**	93.4%
Row J	Max	25.5	0.5	102.0%
	Min	21.2	-3.8	84.8%
	Average	23.5	**-1.5**	94.2%
Row K	Max	25.5	0.5	102.0%
	Min	25.4	0.4	101.6%
	Average	25.5	**0.5**	101.9%

0 to 9% loss	(white)	*<10% loss = >22.5mm reading*
10 to 24% loss	(orange)	*10% loss = 22.5mm reading*

Table 3: Analysis of Average Tubular Pile thickness Readings

(all thickness values are quoted in mm)

Additional Note: Concrete Sub structure elements

While beyond the scope of this appointment, deterioration of the concrete sub structure elements was noted during the survey. Cracking, boast concrete and spalling was noted on the soffits of primary and secondary beams. The severity of this varied over the length of the structure however the secondary beams were typically subject to more severe deterioration.

While this condition was identified during the survey it should be noted that a greater provision of lighting and a further detailed investigation would be required to inspect this condition fully.

From the experience of conducting the current survey: this condition should be investigated at low water during a spring tide to allow maximum time to inspect during in the dry during each tidal window.

It is recommended that this condition is inspected within the next 12 to 18 months.

5.0 Discussion

Main Pier Sheet Piles

Southern end of the Structure (Open Pile):

It is noted that the condition of the sheet pile wall varies significantly along the length of the structure. Furthermore the structural purpose of these piles also varies along its length.

CH0 to 50m they form a facing panel/wave screen to the open pile section of the structure. It is understood that these were installed within the works conducted in 1975.

As these piles are subject to exposure on both sides the rate of corrosion is doubled as corrosion can attack both faces (sides) of the pile.

As noted previously; Table 25 from BS6349-1:2000 provides typical rates of corrosion for structural steels in temperate climates. Mean values are noted between 0.04 and 0.08mm/side/year and Upper Limits are noted between 0.1 and 0.17mm/side/year.

Hence if the piles were from 1975 levels of typical corrosion expected could be in the order of 6.4mm. In relation to the nominal thickness of 12.7mm this would leave 6.3mm residual thickness. When compared to the results encountered this is not atypical, however some results have indicated thicknesses less than this in particular at locations where there is evidence of possible Accelerated Low Water Corrosion (ALWC).

ALWC will often result in significantly increased rates of corrosion which can be up to or in excess of 1mm per side per year. Thus, if ALWC is present it may be assumed that this may have only occurred at a relatively recent time as the majority of results are not significantly beyond normal rates.

It was also noted by the Harbour Master and DAFM following the survey that fresh / treated water is discharged from the Mal river culvert at the South side of the marina revetment structure to the East of the Main Pier and water is also released from the Dingle WWTW approx 230m to the South of the harbour entrance. It was noted by the Harbour Master that a regular sediment plume was visible. These could create conditions which could accelerate corrosion of the steel piles.

Both the structural arrangement and exposure factors above may be contributing to the increased loss of sectional thickness encountered at this location.

As discussed previously, it is not currently considered that this portion of the sheet pile wall is providing significant structural contribution to the integrity of this section of the pier. However as noted, these piles (along with those towards the root of the structure) are providing a mounting surface for the timber fenders and rubbing strips. As noted previously they may also be acting as a wave screen – however with the Southern and Western walls of the structure being solid (post 2000 works) the level of benefit is likely to be small.

The level of residual sectional thickness indicates that the structure may thus be approaching the end of its usable/functional life. However as this is not a structural member the piles will continue to function until they no longer facilitate competent fixture of the fender units assuming that this is their current sole function.

At this point it should be considered if measures should be taken to maintain the current condition of the piles, monitor them and the fender connections, modify and mount the fenders onto the bearing piles, or remove them entirely and reinstate the fenders on the pies behind.

“Old Original Pier” portion of the structure:

The remaining sheet piles have been installed against the existing structure (installed 1975).

In contrast to those of the open pile section of the pier, these piles are only exposed on a single side. They also remain within the marina revetment structure (no sediment plume has been noted beyond the revetment).

The piles in this area have higher residual thicknesses than those of the first 50m and have a typical loss of sectional thickness at approx 2mm over the length of the structure. These levels of corrosion are lower than that which would typically be expected for piles of this age.

In this respect and given the age of the structure an approach centred on monitoring and proactive structural management (with maintenance/repairs conducted as identified) is likely to be the most prudent and cost effective approach. These could include actions such as:

- Localised cleaning and plate repairs.
- Localised/global cleaning/grit blasting to remove corroded material and application of suitable paint protection system.
- Installation of anodes to reduce the onset of further corrosion.

However action is not currently an urgent requirement as it is felt that the piles are currently in good condition for elements of this age and the current condition is unlikely to change significantly within the next 3 to 5 years given the current structural and exposure condition.

It should be noted that comments are based on condition only and not capacity.

Punt Slipway Sheet Piles

As noted the condition of these piles is largely typical of the condition encountered on the splash zone of the main pier sheet piles however the readings obtained indicate a lower loss of wall thickness.

The surface corrosion remains firmly adhered to the piles providing some ongoing protection. Furthermore it should be noted that the exposed length of pile is limited as the structure is abutted by the adjacent rock armour revetment. For this reason significant deflection or structural failure is unlikely.

Given the above it is suggested to monitor these piles to determine any ongoing deterioration prior to conducting any repairs. The current condition does not present any current integrity concerns and this element of the structure is unlikely to require maintenance/remedial works for 3 to 5years.

Main Pier Tubular Bearing Piles

It is understood that these piles have been installed for approx 15 years.

As noted within section 4.2 the thickness readings obtained are generally higher than that expected for exposure on a single side, however if the pile is not plugged and water and oxygen are allowing corrosion from the inside, then the majority of readings encountered are inline with that expected for steel elements in temporal climates however;

- Multiple readings of almost twice this rate were noted in row Bb and row H. It may be possible that these piles have a thinner wall thickness however this remains to be confirmed.
- Readings of up to 4mm loss were also noted on further individual piles.

In this respect an approach centred on monitoring and proactive management (with maintenance/repairs conducted as identified) is likely to be the most prudent and cost effective approach to ensure the piles remain in good condition and fit for purpose.

In particular, the condition of piles: Bb1, Bb3, C4, E4, F1, G4, H2, H3, H4, J1 and J4 should be monitored and checked for residual thickness against the results baseline of this survey to determine the level of deterioration over time. This could be considered at a 3 to 5 year interval.

If/when required maintenance may be in the form of cleaning and application of a proprietary paint coating system or surface coating. It should be noted that as the piles dry out at low water sacrificial anodes fixed to the piles would not be appropriate and installation on the seabed with connections to the piles may not be possible given the varying bed levels and conditions and the limited space between piles and structural bracing elements.

However the above comments relate to comparison of the inspection results with the quoted wall thickness on construction record drawings. Comments relating to a typical level of corrosion would for structural elements of this age present classic signs of surface deterioration and corrosion. However no evidence of significant corrosion was noted during the inspection.

There were no notable signs of this level of corrosion incident on the piles during the inspection. For this reason it is suggested that the nominal installed wall thickness is investigated for completeness prior to confirming the above comments and taking further action.

6.0 Load Assessment and Life Span

To analyse a structure and complete a load assessment the following is required;

- A detailed knowledge of the structural format and composition.
- Details of the current structural condition of all members.

For this structure there is currently inadequate information in both of the above categories to allow a load assessment to be completed at this time. Further detail is noted below for each area of the structure.

6.1 Solid Structure (Northern end)

Structural Format and Composition

Although some structural record drawings are available for the steel sheet piles which indicate the year of installation and approximate profile; further information is required to enable a load assessment for the overall structure to be carried out.

These include:

- Installed toe depths of the sheet piles.
- The nature and form of fill behind the sheet pile wall and within the original structure.
- The configuration and design details of any tie rods
- Construction details of the 1998 pier extension on the Western side of the pier.
- Details relating to the form of the original structure.
- Structural details of the concrete deck to allow assessment of its current capacity.
- Structural details relating to the integration and connections between the structural elements and different phases of structure development over the years.

Structural Condition

This inspection was a visual non destructive survey confined to the steel piles.

As discussed previously, without further investigations including intrusive testing on the remaining structural elements, it is not possible to determine their current condition or capacity. This is required to make judgements as to the reduced capacity of the idealised structure once a model has been created.

The same is true for the estimation of remaining lifespan of the steel piles. As the true structural utilisation of the sheet piles cannot be determined and the condition and residual capacity of the structure cannot be quantified at this stage an estimation of the remaining life of the sheet piles cannot be made.

6.2 Open Pile Structure (Southern end)

In similar regard; no design / installation details (depths) are available for the steel tubular bearing piles at the Southern end of the structure (It is unlikely that these will be available as Irishenco Construction who designed and installed these piles are no longer in operation.)

These details are required to assess the load capacity of the piles and their contribution to the structural capacity of this portion of the structure. Thus a structural assessment of the load capacity of the structure is not possible at this stage.

7.0 Recommendations

The inspection of the steel tubular and sheet piles has revealed variance in their current condition.

While there are no defects which currently pose significant structural concern it should be noted that corrosion and loss of sectional thickness of steel does represent deterioration of the steel elements. As a result it is suggested to take proactive action to maximise the lifespan of the structural elements and the structure as a whole.

The level and urgency of action required is dependant on the member's current condition and its contribution to the integrity of the structure.

<u>Main Pier Tubular Bearing Piles</u>

- Initiate monitoring campaign with inspections initially on 1 to 2 year frequency.

The schedule of results from this survey represents a base line to which future surveys may be compared. Only at this point will the rate of any corrosion be fully understood.

It should also be noted that while values obtained during this survey do indicate some loss of sectional thickness, no evidence of significant corrosion was noted. While it may be possible for corrosion to be progressing on the inner face of the tubular pile it is unlikely to be significantly greater than that on the outer face.

It may be assumed that the piles have been designed with an allowance for corrosion over their lifespan as there is no evidence of an installed Cathodic Protection system.

As noted previously, if possible it should be checked if there are any department records that note lower sectional wall thickness on any of the collars or piles of the structure.

For this reason and given that some residual thickness has been lost to date it is recommended to monitor the piles on an ongoing basis (every 1 to 2 years to determine the rate of any incident corrosion) and at the appropriate point consider steps to prevent its progression.

Punt Slipway Sheet Piles

- Initiate monitoring campaign with inspections on 3 to 5 year frequency.

These piles are exhibiting corrosion however as noted the exposure and structural condition of these does not make them an element of prime concern. This inspection for significant deterioration could be conducted at a longer interval than that of larger more significant structural members.

Main Pier Sheet Piles (Solid structure)

- Initiate monitoring campaign with inspections initially on 3 to 5 year frequency.

The survey results from this area of the sheet pile wall indicate that the piles are in good condition for their age. However it will be prudent to commence a monitoring campaign to monitor and document any ongoing corrosion. As they have lost limited sectional thickness since their installation the duration between inspections may be extended as required following initial repeat inspections.

Main Pier Sheet Piles (Open Pile structure)

The survey results from this area of the structure have indicated significant loss of pile thickness and note characteristic evidence of possible ALWC in some locations. Further investigation and testing to confirm the presence of ALWC may be possible to reduce long term liability and future issues if required.

It is noted that these piles are not providing significant contribution to the integrity of the structure. However they are acting as a mounting surface for the timber fenders which it is understood are to be retained.

For this reason there are a number of options in relation to managing these piles and retaining the fenders:

Option1

Initiate a frequent inspection schedule of the fender piles and connections and locally replace fittings or remove fenders at locations where pile deterioration as resulted in fender instability.

Option 2

If Option 1 is not considered acceptable; action could be taken to locally disconnect the fenders from the sheet piles, burn/remove sections of the sheet piles at each location and reinstall the fenders on the tubular piles behind.

It should be noted that it will only be ordinarily possible to install the fenders where a tubular pile is incident behind the current fender location (tubular piles at 5.2m and 6.0m centres).

Some further structural assessments will be required to ensure that the tubular piles can accommodate the fender loads; however this is not anticipated to be a major issue.

A budget estimate of cost for this approach may be in the order of €110,000 - €115,000

(It may be possible to install horizontal members to mount additional fenders on however this will significantly increase the cost of the works.)

A previously considered option was to consider complete removal of the piles; however this will also significantly increase the cost of the works and is not an essential requirement.

Option 3

A further and possible long term option could be to prefabricate and install steel plates on the piles on which the fenders could be reinstalled. This is quite a complex operation and will involve extended use of a commercial dive team.

A budget estimate of cost for this approach may be in the order of €150,000 - €170,000

Option 4

A final option could be the installation of a concrete panel on which the fenders may be mounted. While material costs of this approach may be lower, the result of integrating this element with the structure will increase the final value of the works which is likely to be higher than other options. Furthermore there are additional implications of this approach on the operation of the pier.

- Significant breaking out of the deck slab will be required in order to install the new concrete panel. Further consideration will also be required to confirm if this approach is feasible.
- Installation of a concrete panel will likely step out the berthing line of the structure over its length.

A budget estimate of cost for this approach for materials and installation only may be in the order of €100,000 – 110,000.

However further additional costs will be required for preparative and enabling works to allow installation. Further investigation on estimate of these costs can be provided on request.

Closing Comments:

It is not considered that immediate remedial action on this portion of the sheet pile wall is essential and noted that monitoring the piles for integrity of the fender units may be the most cost effective action until the point where it is visually evident that action is required.

However if this is not considered an option and possible instability of any fender unit (timber + rubbing strip) cannot be allowed; then it is suggested that option 2 may be the most cost effective and logical solution to the current condition.

APPENDICES

UNDER SEPARATE COVER (CD)

APPENDIX 1	-	PILE THICKNESS READINGS
APPENDIX 2	-	SURVEY PLAN
APPENDIX 3	-	RECORD PHOTOGRAPHY
APPENDIX 4	-	STRUCTURE RECORD DRAWINGS & REPORT

APPENDIX 1

PILE THICKNESS READINGS

IBM0558 - Dingle Main Pier Survey Readings - Sheet Piles, Main Pier

Time	Chainage	Dips		Measurements				Notes
		Deck to Bed	Deck to WL	Ht above Bed	IP (mm)	OP (mm)	SF (mm)	
17:34:00	0m (Southern end pier)	8.30m	1.80m	0.5m	4.2	4.1	10.6	Some pitted steel. Hole (ABCO photo 1) at 1.5m above bed 2"w x 12" ht - looks like cut hole.
				1m	3.4	3.1	4.4	
				1.5m	9.6	3.2	3.4	
				2m	3.6	4.6	2.9	
				2.5m	3.2	4.6	12.6	
				3m	2.2	4.3	3.1	
18:00:00	5m	8.00m	1.20m	0.5m	3.8	3.6	9.3	marine growth with clean steel beneath no corrosion good condition patches of corrosion at aprox 3m ht above bed.
				1m	8.5	3.6	5.3	
				1.5m	4.7	2.6	10.1	
				2m	10.4	2.7	8.3	
				2.5m	11.9	11.9	6.7	
				3m	1.8	1.7	1.4	
				3.5m	2.7	1.7	1.6	
	5m	8.00m	2.25m	5.9m	NR	10.1	8.9	
18:31:00	10m	8.00m	1.00m	0.5m	10.8	1.6	10.3	corrosion noted on pile. Thick friable corrosion. Diver noted pile felt frgile and felt could almost "clean through" pile when testing
				1m	10.4	2.7	4.7	
				1.5m	9.3	4.1	7.4	
				2m	1.4	3.7	2.7	
				2.5m	1.5	1.5	1.6	
				3m	10.2	12.1	11.9	
				3.5m	NR	1.6	4.7	
19:00:00	15m	7.90m	1.00m	0.5m	10.5	10.7	10.3	Hole 2m above bed. 450mm ht and 150mm wide
				1m	9.6	6.4	8.4	
				1.5m	11.7	12.5	12.0	
				2m	12.0	12.2	12.1	
				2.5m	1.6	3.8	1.4	
				3m	1.5	1.5	1.6	
19:20:00	20m	7.80m	1.00m	0.5m	10.1	6.7	11.7	
				1m	9.6	8.1	7.8	
				1.5m	4.0	4.2	4.0	
				2m	2.9	4.5	3.7	
				3m	12.5	12.4	12.6	
				3.5m	4.5	4.4	4.7	

Time	Chainage	Dips		Measurements				Notes
		Deck to Bed	Deck to WL	Ht above Bed	IP (mm)	OP (mm)	SF (mm)	
09:45:00	25m	7.80m	2.80m	0.5m	11.7	11.3	11.3	Orange patched of varied size. Typ 400h and 300w on side faces of piles. Black sludge and corroded steel behind. Upper readings also taken on orange patches but steel behind more pitted than lower areas. At upper levels towards HW thiner corroded 'crust' with smaller orange patches.
				1m	10.8	11.9	11.2	
				1.5m	11.5	12.0	11.3	
				2m	5.6	11.8	6.0	
				2.5m	5.8	11.3	6.7	
				3m	11.7	10.9	10.4	
				3.5m	6.7	9.4	4.9	
				4m	9.4	20.7	6.7	
					NR	11.8	NR	
				4.5m	12.4	12.6	13.0	
				5m	6.7	12.4	12.9	
10:10:00	30m	7.60m	2.80m	0.5m	11.6	11.4	9.7	Good visibility (almost full water column) corrosion on side face patch approx 2m ht (0.5m to 2.5m above bed) approx 300w. Orange patch with pitted steel behind (no black sludge). Corrosion but with reduced pitting at upper levels.
				1m	10.4	11.6	6.7	
				1.5m	11.5	10.2	10.5	
				2m	10.8	12.2	4.6	
				2.5m	8.6	10.9	5.6	
				3m	9.0	9.8	5.6	
				3.5m	10.5	9.7	12.0	
				4m	11.7	12.3	13.2	
				4.5m	12.3	12.4	12.4	
				5m	11.3	12.4	12.4	
10:21:00	35m	7.80m	2.75m	0.5m	11.3	10.3	8.6	Corrosion mainly on side faces (seaward facing). Level of corrosion varies with ht (300 to 1.5m ht): approx 2.5m ht 1m ht x 300w patch (full width). 2nd orange patch 2.5m ht. Square hole cut on inpan: 100x100mm.
				1m	10.4	10.5	9.2	
				1.5m	12.3	7.1	12.4	
				2m	10.3	9.4	6.7	
				2.5m	6.7	8.1	6.4	
				3.5m	12.5	9.4	8.4	
				4m	11.2	10.7	11.9	
				4.8m	11.9	11.4	12.0	
	40m	7.75m	2.70m	0.5m	10.0	10.7	10.1	
				1m	9.9	10.7	10.1	
				1.5m	10.1	9.1	9.2	
				2m	9.9	10.6	10.2	
				2.5m	9.0	9.7	10.4	
				3.5m	9.7	10.1	8.3	
				3m	10.2	9.6	10.4	
				4m	10.4	11.1	9.8	
				4.5m	12.2	11.8	12.1	
	40m	7.80m	2.40m	5.6m	12.5	11.3	NR	

Time	Chainage	Dips		Measurements				Notes
		Deck to Bed	Deck to WL	Ht above Bed	IP (mm)	OP (mm)	SF (mm)	
11:10:00	45m	7.50m	3.40m	0.5m	11.6	10.7	10.4	
				1m	11.7	10.5	10.6	
				1.5m	10.8	11.4	11.8	
				2m	11.3	10.7	11.6	
				2.5m	11.1	10.4	10.0	
				3m	11.3	9.7	10.4	
				3.5m	13.3	12.9	11.9	
				4m	12.4	12.4	11.6	
	50m	7.70m	3.60m	0.5m	11.2	10.9	11.0	heavy marine growth at depth. Thins as travel upwards. Orange patch 2.5 to 3m ht on seaward facing side face. Some orange patcheson outpans approx 300mm wide. No holes
				1m	11.1	11.0	10.7	
				1.5m	10.9	11.4	10.8	
				2m	10.7	11.3	10.1	
				2.5m	11.0	11.6	12.2	
				3m	11.2	10.2	10.6	
				3.5m	10.9	10.9	12.2	
				4m	11.8	10.7	13.2	
	55m	7.80m	3.90m	0.5m	10.4	11.2	10.8	Large orange stain 300mm w and 1m ht at chainage 51m. No holes. Multiple orange patches at varying levels.
				1m	10.9	13.3	9.9	
				1.5m	11.0	12.0	10.5	
				2m	11.7	9.6	10.8	
				2.5m	10.4	9.7	10.5	
				3m	10.9	10.9	12.4	
				3.5m	10.5	11.2	12.1	
				4m	12.0	10.4	11.6	
11:44:00	60m	7.50m	3.90m	0.5m	11.1	11.5	8.8	
				1m	11.2	9.8	9.5	
				1.5m	10.9	9.6	10.0	
				2m	11.2	10.8	10.8	
				2.5m	10.7	10.9	11.1	
				3m	10.6	10.6	10.2	
				3.5m	11.2	11.1	10.6	
	60m	7.50m	2.50m	5.3m	12.3	11.4	11.2	
11:55:00	65m	7.65m	3.95m	0.5m	11.2	8.8	11.0	fewer orange patches - mainly marine growth.
				1m	11.3	11.1	10.8	
				1.5m	11.6	10.9	11.4	
				2m	11.4	10.2	10.9	
				2.5m	11.5	10.6	3.5	
				3m	11.2	11.6	11.1	
				3.5m	11.0	9.7	10.4	

Time	Chainage	Dips		Measurements				Notes
		Deck to Bed	Deck to WL	Ht above Bed	IP (mm)	OP (mm)	SF (mm)	
12:05:00	70m	7.70m	4.00m	0.5m	11.2	10.2	11.0	1 orange spot 1m above seabed on side face: approx 750mm ht x 300mm w.
				1m	10.7	10.5	11.2	
				1.5m	10.8	10.7	11.1	
				2m	10.9	10.5	10.9	
				2.5m	10.5	11.0	8.0	
				3m	11.2	11.1	11.5	
				3.5m	10.6	10.1	9.7	
12:14:00	75m	7.20m	4.00m	0.5m	10.7	11.1	11.2	no signs of corrosion fairly uniform condition minor pitting dull steel surface
				1m	10.4	11.1	11.0	
				1.5m	9.6	11.4	11.2	
				2m	10.8	11.9	11.0	
				2.5m	11.2	11.1	11.2	
				3m	11.3	10.9	10.1	
12:23:00	80m	7.10m	4.10m	0.5m	10.9	11.1	10.6	1 orange patch approx 2m above bed 0.5m ht x 200mm w on side face.
				1m	11.0	11.3	11.2	
				1.5m	10.7	10.5	11.3	
				2m	10.2	10.9	10.5	
				2.5m	10.7	11.2	10.4	
				3m	10.9	10.9	11.2	
	80m	7.20m	2.40m	5m	NR	11.1	11.4	
				6.3m	NR	11.9	8.9	
12:28:00	85m	7.10m	4.40m	0.5m	10.5	10.5	11.0	no orange patches.
				1m	11.2	10.1	10.9	
				1.5m	10.3	10.7	10.6	
				2m	10.7	10.5	10.5	
				2.5m	10.2	9.9	10.5	
				3m	9.0	10.0	10.9	
12:37:00	90m	7.20m	4.40m	0.5m	10.9	11.0	11.2	single bright orange patch 1.5m ht x 300mm w.
				1m	12.0	10.7	11.3	
				1.5m	10.4	10.7	10.1	
				2m	11.1	10.6	10.8	
				2.5m	10.8	11.2	9.8	
12:48:00	95m	7.20m	4.40m	0.5m	10.5	10.6	11.2	no orange patches.
				1m	10.6	10.4	10.9	
				1.5m	10.8	10.2	10.9	
				2m	10.8	10.6	10.7	
				2.5m	10.1	10.4	10.4	

Time	Chainage	Dips		Measurements				Notes
		Deck to Bed	Deck to WL	Ht above Bed	IP (mm)	OP (mm)	SF (mm)	
12:54:00	100m	7.30m	4.35m	0.5m	11.3	10.7	10.4	
				1m	11.4	10.9	10.2	
				1.5m	11.3	10.7	10.5	
				2m	11.2	9.8	10.4	
				2.5m	11.1	9.8	10.2	
12:59:00	105m	7.35m	4.35m	0.5m	10.8	11.6	12.0	
				1m	11.8	12.5	12.0	
				1.5m	11.2	11.2	11.4	
				2m	10.3	10.0	10.6	
				2.5m	10.9	11.2	9.6	
18:30:00	110m	6.90m	1.60m	0.5m	11.4	11.6	11.4	
				1m	11.4	11.5	11.6	
				1.5m	11.0	10.9	10.8	
				2m	10.1	11.0	10.5	
				2.5m	8.5	10.8	11.2	
				3m	9.5	10.9	11.4	
				3.5m	12.6	12.4	11.8	
				4m	12.6	12.3	12.0	
				4.7m	12.5	11.7	12.1	
	110m	7.10m	2.50m	4.75m	12.5	12.0	12.2	
				5.6m	12.1	NR	8.3	
				6.35m	11.8	10.3	NR	
18:44:00	118m	7.20m	1.40m	0.5m	11.2	11.3	11.4	
				1m	11.3	11.4	11.1	
				1.5m	10.9	10.9	11.2	
				2m	11.2	10.9	11.3	
				2.5m	10.1	11.3	11.1	Orange patch on outpan from 1.5m above bed to 2.60m above bed. Pile looks abraded on outpan.
				3m	8.7	10.8	11.2	
				3.5m	11.3	11.7	12.0	
				4m	11.1	11.9	12.1	
				4.5m	12.2	12.3	12.0	
				5m	11.9	12.4	12.3	
				5.5m	12.6	12.6	10.3	

Time	Chainage	Dips		Measurements				Notes
		Deck to Bed	Deck to WL	Ht above Bed	IP (mm)	OP (mm)	SF (mm)	
19:07:00	130m	6.80m	1.40m	0.5m	10.4	11.2	10.7	
				1m	10.2	10.9	10.2	
				1.5m	10.0	9.9	9.5	
				2m	10.9	10.3	10.2	
				2.5m	11.2	10.8	10.1	
				3m	11.8	11.4	12.1	
				3.5m	12.2	11.4	11.9	
				4m	12.1	11.8	12.1	
				4.5m	12.1	12.0	12.1	
				5m	8.9	8.4	11.1	
	130m	6.80m	2.70m	4.2m	12.2	11.9	12.1	
				5.3m	12.2	8.6	9.6	
				6m	12.1	NR	NR	
19:30:00	140m	6.50m	1.10m	0.5m	11.7	10.8	11.2	
				1m	10.8	11.4	11.2	
				1.5m	11.4	10.7	10.9	
				2m	11.2	11.4	10.2	
				2.5m	11.9	11.9	12.1	
				3m	11.4	11.2	10.9	
				3.5m	12.1	12.1	12.0	
				4m	12.2	12.1	12.0	
				4.5m	12.1	12.2	11.8	
				5m	12.7	11.4	10.9	
19:45:00	150m	6.20m	1.10m	0.5m	10.5	10.8	9.8	Steel crust, rust and black residue below orange patches. At 0.5m above bed.
				1m	10.7	10.9	11.1	
				1.5m	11.5	11.1	10.4	
				2m	11.5	10.8	10.5	
				2.5m	12.0	12.0	11.9	
				3m	12.1	11.9	12.1	
				3.5m	12.4	11.4	12.4	
				4m	12.2	11.2	11.2	
				4.5m	11.1	9.0	7.0	
16:39:00	150	6.20m	2.90m	5.3m	NR	NR	11.3	

Time	Chainage	Dips		Measurements				Notes
		Deck to Bed	Deck to WL	Ht above Bed	IP (mm)	OP (mm)	SF (mm)	
20:00:00	160m	6.50m	1.10m	0.5m	11.1	9.8	10.4	small hole on outpan 100 ht x 300w hole (triangle shape). Minor abrasion on outpan
				1m	11.2	10.7	10.6	
				1.5m	12.3	12.2	12.2	
				2m	12.5	11.9	11.4	
				2.5m	12.4	12.4	12.4	
				3m	12.6	12.4	12.5	
				3.5m	12.3	11.9	12.3	
				4m	12.0	11.9	12.1	
16:39:00	160	5.60m	3.10m	2.5m	12.5	12.3	12.5	
				3.8m	12.3	12.5	12.5	
				4.6m	12.6	11.4	12.3	
20:14:00	170m	5.10m	1.20m	0.5m	9.4	10.2	11.1	2 small formed holes.
				1m	10.2	9.9	11.2	
				1.5m	11.3	11.8	11.6	
				2m	12.3	11.9	11.8	
				2.5m	12.0	11.7	11.8	
				3m	11.2	11.2	11.2	
				3.5m	11.8	11.4	11.5	
16:39:00	170m	4.90m	3.30m	1.75m	12.3	11.6	11.9	widespread blistering - further readings not possible
				2.4m	12.1	11.7	11.8	

10 to 24% loss		*10% loss = 11.43m reading*
25% to 49% loss		*25% loss = 9.53mm reading*
50% + loss		*50% loss = 6.35mm reading*

IBM0558 - Dingle Main Pier Survey Readings - Sheet Piles, Main Pier

Time	Chainage	Dips		Measurements				Notes
		Deck to Bed	Deck to WL	Ht above Bed	IP (mm)	OP (mm)	SF (mm)	
	0m (top of slipway slope)							blistering at tops of piles.
	5m	3.5m	3.4m	0.7m	10.3	10.1	10.3	
				1.4m	9.6	11.3	11.8	
				2.2m	12.4	9.8	10.6	
				3.1m	12.1	9.9	7.9	
	10m	2.9m	2.55m	1m	9.8	11.0	8.7	
				1.7m	12.3	11.4	8.6	
				2.4m	11.6	11.4	8.8	
	15m	2.7m	2.15m	0.7m	11.6	11.4	11.4	
				1.4m	10.7	11.6	12.4	
				2m	11.1	11.4	11.0	
	20m	2m	1.5m	0.8m	11.1	11.6	11.0	some pitting and orange patches
				1.6m	8.6	10.7		
	25m	1.4m	0.8m	0.5m	12.0	12.1	12.0	approx 1m ht of pile deck slab soffit to bed.
	30m							
				Max	12.4	12.1	12.4	
				Min	8.6	9.8	7.9	
				Average	11.0	11.1	10.4	
					d	d	t	
				Nominal	12.7	12.7	12.7	

	d	**d**	**t**
Nominal Pile Dims	12.7	12.7	12.7
	IP (mm)	OP (mm)	SF (mm)
Chainage	Average	Average	Average
0m (Top of Slope)			
5m	10.3	10.1	10.3
	9.6	11.3	11.8
	12.4	9.8	10.6
	12.1	9.9	7.9
10m	9.8	11.0	8.7
	12.3	11.4	8.6
	11.6	11.4	8.8
15m	11.6	11.4	11.4
	10.7	11.6	12.4
	11.1	11.4	11.0
20m	11.1	11.6	11.0
	8.6	10.7	
25m	12.0	12.1	12.0
30m			

Overall Values

Max	12.4	12.1	12.4
Min	8.6	9.8	7.9
Average	11.0	11.1	10.4

0 to 9% loss		*<10% loss*
10 to 24% loss		*10% loss = 11.43m reading*
25 to 49% loss		*25% loss = 9.53mm reading*
50% + loss		*50% loss = 6.35mm reading*

Pile Ref	Approx Ht Above Bed (m)	Approx Level (mOD Malin)	Measurement (mm)	Location (Pile or Collar?)	Total length(ht) of exposed pile	Steel Collar Length (ht)	Steel Collar Location: Top or Bottom ?	Tubular Pile Length (ht)	Tubular Pile Location: Top or Bottom ?	Notes - Tubular Pile	Notes Seabed	Notes - Concrete ***(brief visual notes only)***
Bb1	0.50		20.3	Collar								
	0.70		20.1	Collar								
	1.00		20.2	Collar	3.80	3.70	Top	0.10	Bottom	-	-	large element between row B and row Bb
	1.80		20.4	Collar								
	2.50		20.4	Collar								
Bb3	0.00		20.1	Pile								
	0.50		20.2	Collar								U/s deck may be corrugated steel?
	1.00		20.1	Collar	3.80	3.70	Top	0.10	Bottom	Appears good condition - no defects	-	Range of conc forms of construction. Some evidence of
	1.80		20.1	Collar								spalling
	2.50		20.2	Collar								
B1	0.00		25.3	Collar								
	0.50		25.4	Collar	1.90	1.90	All	-	-	-	-	horizontal bracing good condition
	1.00		25.5	Collar								
B2	0.00		25.2	-								
	0.50		25.3	-								Secondary beam: poor localised repairs (photo)
	1.00		25.3	-	2.35	-	-	-	-	Appears good condition - no defects	-	primary beam: large crack
	1.80		25.3	-								bracing elements: appear good condition (larger csa)
	2.50		25.3	-								
B3	0.10		25.3	Collar								
	0.50		25.4	Collar								
	1.00		25.2	Collar	3.20	2.50	Bottom	0.70	Top	Appears good condition - no defects	-	Some spalling to underside of concrete beams (photo)
	1.80		25.4	Collar								
	2.50		24.8	Pile								
B4	0.00		26.1	Pile								
	0.50		25.9	Collar								
	1.00		25.5	Collar	3.20	2.90	Top	0.30	Bottom	Appears good condition - no defects	-	-
	1.80		26.2	Collar								
	2.50		25.7	Collar								
C1	0.00		25.6	Collar								concrete elements in reasonable condition
	0.50		25.6	Collar						Appears good condition - no defects		tie rods noted through sheet pile wall into outer concrete
	1.00		25.8	Collar	2.70	1.60	Bottom	1.10	Top	(top of pile like 'new')	-	primary beam
	1.80		24.9	Pile								horizontal bracing good condition (photo)
	2.50		24.8	Pile								
C2	0.00		25.4	Collar								
	0.50		25.4	Collar								
	1.00		25.5	Collar	2.15	2.00	Bottom	0.15	Top	-	-	-
	1.80		25.7	Collar								
	2.10		25.3	Pile								
C3	0.00		25.4	Collar								
	0.50		25.5	Collar								
	1.00		25.5	Collar	3.00	2.30	Bottom	0.70	Top	Appears good condition - no defects	-	-
	1.80		25.3	Collar								
	2.50		25.1	Pile								
C4	0.00		22.2	Collar								
	0.50		22.4	Collar								
	1.00		22.0	Collar	2.70	2.60	Bottom	0.10	Top	Appears good condition - no defects	-	-
	1.80		22.1	Collar								
	-		*Not accessible*	Pile								
D1	0.00		25.7	Collar								
	1.00		25.6	Collar								Horizontal bracing at bed level + 2nd set at 2m ht.
	1.70		25.6	Collar	3.40	2.90	Bottom	0.50	Top	-	-	Some minor spalling of u/s of conc primary beams
	2.10		25.7	Collar								Pilecaps appear in good condition.
	2.50		26.6	Collar								
	-		*Not accessible*	Pile								

Pile Ref	Approx Ht Above Bed (m)	Approx Level (mOD Malin)	Measurement (mm)	Location (Pile or Collar?)	Total length(ht) of exposed pile	Steel Collar Length (ht)	Steel Collar Location: Top or Bottom ?	Tubular Pile Length (ht)	Tubular Pile Location: Top or Bottom ?	Notes - Tubular Pile	Notes Seabed	Notes - Concrete ***(brief visual notes only)***
D2	0.00		25.6	Collar	2.90	2.30	Bottom	0.60	Top	-	-	-
	0.50		25.7	Collar								
	1.00		25.7	Collar								
	1.80		25.6	Collar								
	2.50		25.2	Collar								
	-		*Not accessible*	Pile								
D3	0.00		25.4	Collar	2.60	1.90	Bottom	0.70	Top	Appears good condition - no defects	-	Underside of conc beams slightly better than adjacent areas. However some staining to u/s of beams and cracks on u/s of secondary beams.
	0.50		25.3	Collar								
	1.00		25.4	Collar								
	1.80		25.4	Collar								
	2.50		24.6	Pile								
D4	0.20		25.5	Collar	3.10	2.60	Bottom	0.50	Top	Appears good condition - no defects (photo taken)	Cobble and boulder composition sloped over 1m up on 1 side of pile.	
	0.50		25.7	Collar								
	1.00		25.6	Collar								
	1.80		25.4	Collar								
	2.50		25.0	Pile								
E1	0.00		24.5	Pile	3.30	2.40	Top	0.90	Bottom	Appears good condition - no defects	-	No evident spalling (photo)
	0.50		24.5	Pile								
	1.00		25.4	Collar								
	1.80		25.4	Collar								
	2.50		25.7	Collar								
E2	0.00		25.4	Collar	2.65	2.30	Bottom	0.35	Top	-	-	-
	0.50		25.4	Collar								
	1.00		25.4	Collar								
	1.80		25.4	Collar								
	2.10		25.4	Collar								
	2.35m (0.3 below u/s deck)		24.7	Pile								
E3	0.00		25.5	-	2.90	-	-	-	-	-	-	Some severe cracking of secondary beams. Primary beams appear in better condition.
	0.50		24.4	-								
	1.00		25.4	-								
	1.80		25.5	-								
	2.50		25.2	-								
E4	0.00		25.7	-	3.10	-	-	-	-	Appears good condition - no defects	Cobble and boulder composition sloped over 1m up on 1 side of pile.	-
	0.50		22.0	-								
	1.00		22.0	-								
	1.80		22.1	-								
	2.50		21.9	-								
F1	0.00		21.9	Collar	3.00	2.10	Bottom	0.90	Top	Appears good condition - no defects	-	Primary Beam - some spalling to u/s Secondary beam: poor towards pile rows 2,3,4 but this bay fairly good. Tie rods running into outer primary beam.
	0.50		21.8	Collar								
	1.00		21.9	Collar								
	1.80		21.9	Collar								
	2.50		24.7	Pile								
F2	0.05		24.9	Pile	2.70	2.60	Top	0.10	Bottom	-	-	-
	0.50		25.5	Collar								
	1.00		25.3	Collar								
	1.80		25.4	Collar								
	2.20		25.3	Collar								
	2.50		25.7	Collar								

Pile Ref	Approx Ht Above Bed (m)	Approx Level (mOD Malin)	Measurement (mm)	Location (Pile or Collar?)	Total length(ht) of exposed pile	Steel Collar Length (ht)	Steel Collar Location: Top or Bottom ?	Tubular Pile Length (ht)	Tubular Pile Location: Top or Bottom ?	Notes - Tubular Pile	Notes Seabed	Notes - Concrete (***brief visual notes only***)
F3	0.20		25.3	Collar	3.10	2.00	Bottom	1.10	Top	-	-	-
	0.20		25.4	Collar								
	0.50		25.3	Collar								
	1.00		25.3	Collar								
	1.80		24.6	Pile								
	2.50		24.9	Pile								
F4	0.00		25.5	Collar	4.10	3.20	Bottom	0.80	Top	-	-	mixed forms of construction
	0.50		25.5	Collar								
	1.00		25.5	Collar								
	1.80		25.5	Collar								
	2.50		25.4	Collar								
G1	0.00		25.4	Collar	3.00	1.60	Bottom	1.40	Top	Appears good condition - no defects	-	Primary Beam: some spalling and cracking to u/s between rows 1 and 2. Minor deterioration of secondary beam.
	1.00		25.4	Collar								
	1.80		24.9	Pile								
	2.50		24.9	Pile								
G2	0.00		25.5	Collar	2.80	2.30	Bottom	0.50	Top	-	-	Primary beam: minor staining Secondary beam: some deterioration
	0.30		25.4	Collar								
	0.50		25.3	Collar								
	1.00		25.4	Collar								
	1.80		25.4	Collar								
	2.50		24.7	Pile								
G3	0.00		25.6	Collar	3.10	2.00	Bottom	1.10	Top	-	-	-
	0.50		25.5	Collar								
	1.00		25.4	Collar								
	1.80		25.3	Collar								
	2.50		24.6	Pile								
G4	0.50		24.7	Pile	4.30	3.30	Top	1.00	Bottom	Some marine growth at low level	-	-
	1.30		21.0	Collar								
	1.80		21.1	Collar								
	2.20		25.5	Collar								
	2.50		25.3	Collar								
H1	0.00		24.4	Pile	3.20	3.00	Top	0.20	Bottom	Appears good condition - no defects		Primary Beam: some staining on surface. Secondary beam: Some deterioration between rows 1 + 2. Pilecaps and horizontal bracing appear in good condition.
	0.50		25.3	Collar								
	1.20		25.3	Collar								
	1.80		25.3	Collar								
	2.50		25.3	Collar								
H2	0.00		22.1	Pile	2.80	2.15	Bottom	0.65	Top	-	-	Secondary Beams: some minor cracking to u/s Primary Beams: small areas of staining + minor deterioration.
	0.50		22.1	Pile								
	1.00		22.2	Pile								
	1.80		24.8	Pile								
H3	0.20		21.9	Collar	3.05	1.85	Bottom	1.20	Top	-	-	Primary Beams - staining and possibly 1 area boast. Secondary Beams - concrete staining Minor damage around weep hole on u/s of deck.
	0.50		21.9	Collar								
	1.00		22.0	Collar								
	1.80		25.0	Pile								
H4	0.00		21.2	Pile	3.90	None	None	3.90	All	Appear good condition	Crusher run material	-
	0.50		22.0	Pile								
	1.00		21.9	Pile								
	1.80		22.1	Pile								
	2.50		25.5	Pile								
J1	0.30		22.0	Collar	3.10	2.90	Top	0.20	Bottom	No defects (low light levels) - easy to clean, minimal marine growth.	-	-
	0.80		22.1	Collar								
	1.70		22.2	Collar								
	2.10		22.1	Collar								
J2	0.10		25.5	-	2.75	-	-	-	-	No defects (low light levels) - easy to clean, minimal marine growth.	-	-
	0.50		25.2	-								
	1.60		25.3	-								
	2.00		24.8	-								
	2.60		24.7	-								
J3	0.30		25.5	Collar	3.70	3.10	Bottom	0.60	Top	No defects (low light levels) - easy to clean, minimal marine growth.	Cobbly silty silty material sloped at base of exposed pile	-
	0.70		25.4	Collar								
	1.20		25.3	Collar								
	1.80		25.3	Collar								
	2.50		25.5	Collar								
J4	0.00		21.3	-	3.60	-	-	-	-	No defects (low light levels) - easy to clean, minimal marine growth.	Piles in 4th row have crusher run stone at bed level. Inner rows have larger masonry and waste material between	-
	0.50		21.2	-								
	1.00		21.3	-								
	1.80		21.4	-								
	2.50 (0.5 down from top)		21.3	-								
K4	0.00		25.5	Collar	2.50	2.10	Bottom	0.40	Top	-	-	-
	0.50		25.5	Collar								
	1.00		25.5	Collar								
	1.80		25.4	Collar								

APPENDIX 2

SURVEY PLAN


The Wood
Strand Street
R559
N
CH 0m
CH 5m
CH 10m
CH 15m
CH 20m
CH 25m
CH 30m
SLIPWAY SHEET PILES
CH 170m
CH 160m
CH 150m
CH 140m
CH 130m
CH 120m
CH 110m
CH 100m
CH 90m
CH 80m
CH 70m
CH 60m
CH 50m
CH 40m
CH 30m
CH 20m
CH 10m
CH 0m
MAIN PIER SHEET PILES
203m (CURRENT LENGTH)
166m (SHEET PILES INSTALLED 1975)
122m (LENGTH PRE 1927)
44m
1927 EXTENSION
57m
2000 EXTENSION
MAIN PIER TUBULAR PILES
K
J
H
G
F
E
D
C
B
Bb
4
3
2
1
2
1

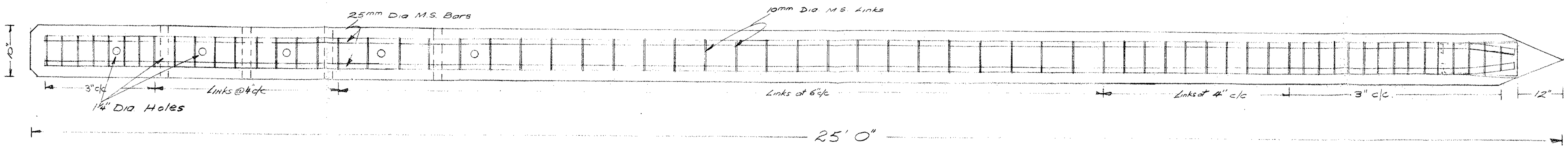
APPENDIX 3

RECORD PHOTOGRAPHY

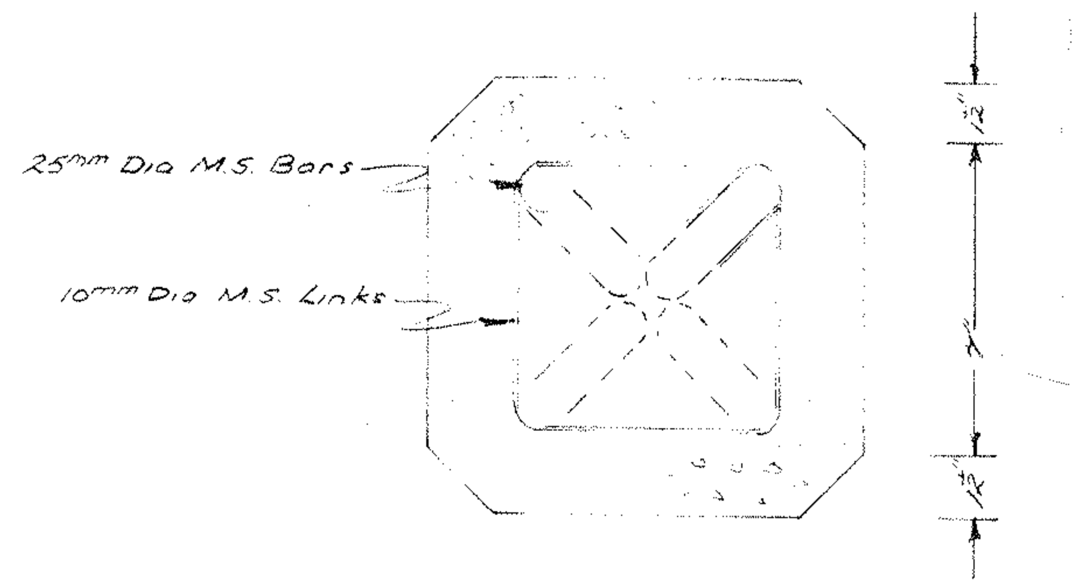
(Supplied digitally)

APPENDIX 4

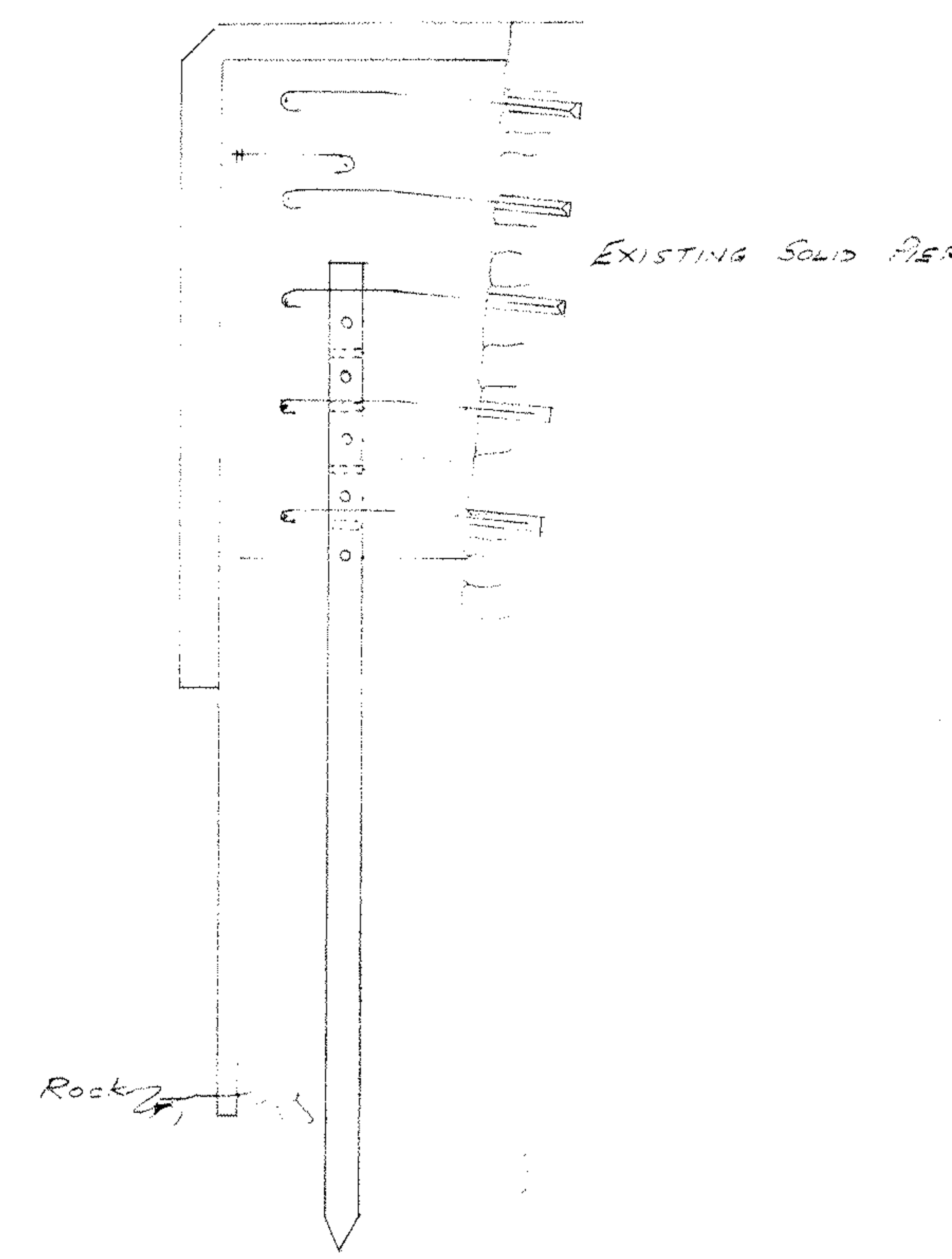
STRUCTURE RECORD DRAWINGS & REPORT



ELEVATION OF R.C. PILE   SCALE 1" TO 1'0"



PLAN THRO' PILE   SCALE: 3" TO 1'0"



SECTION THRO EXTENSION
SCALE: ¼"-TO-1'0"

STEEL SCHEDULE

	BARS PER UNIT	TOTAL No BARS	DIA	OUT OF STRAIGHT BAR	BENDING
25' LONG PILE	4	144	25mm	24' 0"	10"
" "	65	2340	10mm	2' 9"	6"

OFFICE OF PUBLIC WORKS DUBLIN

DINGLE HARBOUR Co KERRY

DETAIL OF R.C. PILE

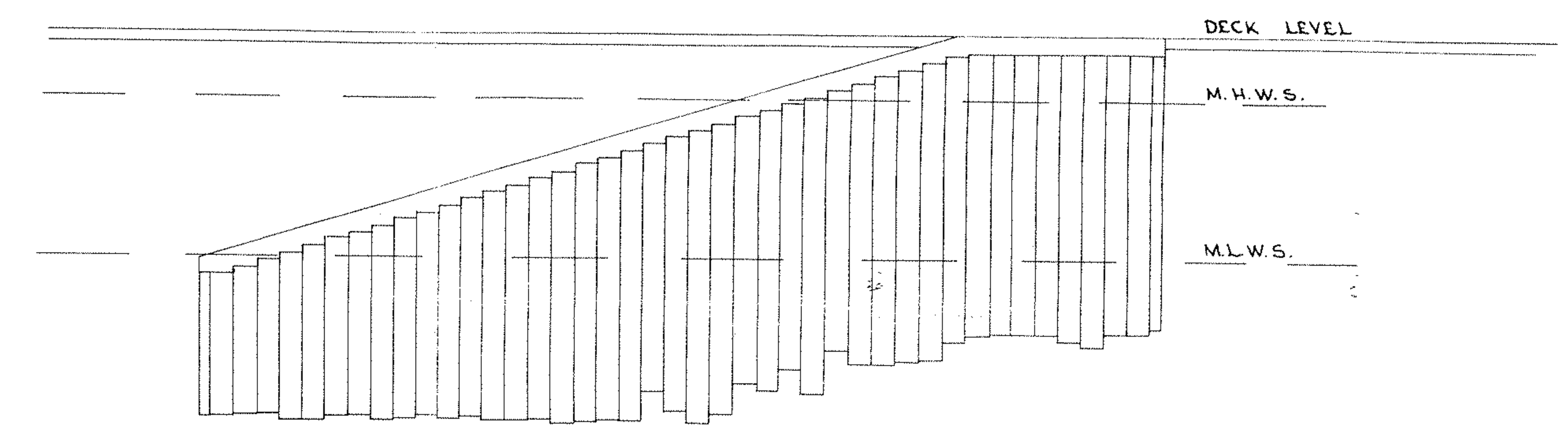
SCALES: 3", 1" & ¼" TO 1'0"

DATE: JAN '73   DRG No. 762/17

DRAWN: L Walsh
CHECKED: S. O Súilleabháin

ELEVATION

SCALES HORIZONTAL 16'-0" TO 1"
VERTICAL 8'-0" TO 1"

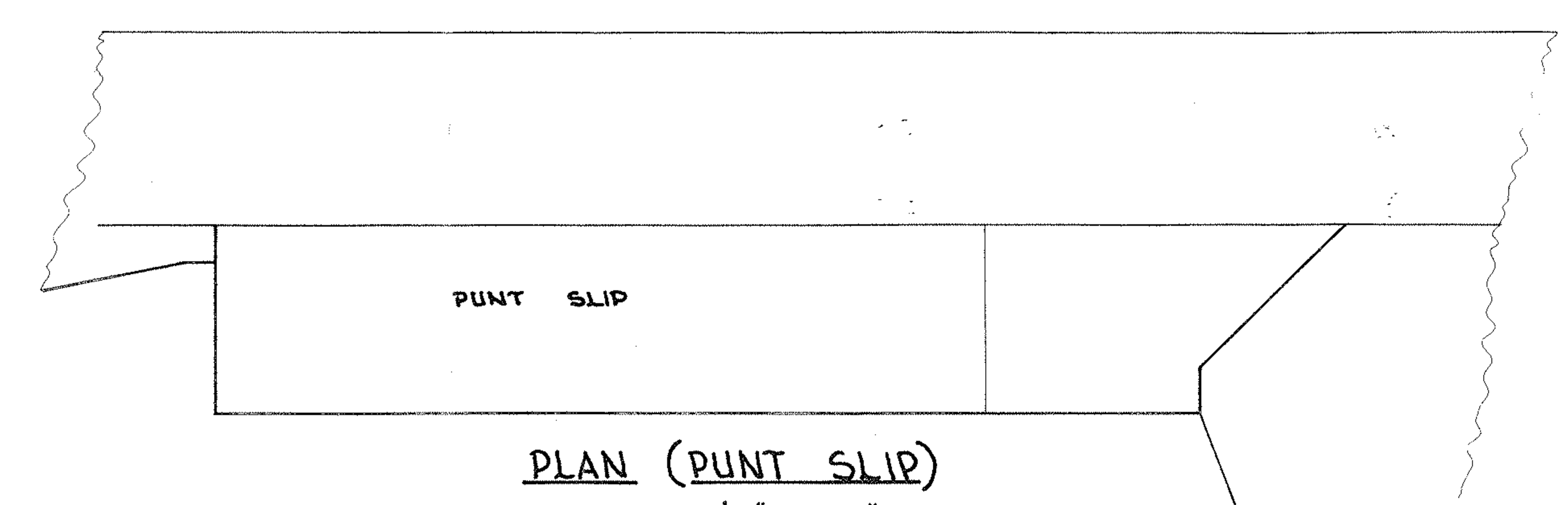


ELEVATION (PUNT SLIP)

SCALES HORIZONTAL 16'-0" TO 1"
VERTICAL 8'-0" TO 1"

PLAN

SCALE — 16'-0" TO 1"



PLAN (PUNT SLIP)

SCALE — 16'-0" TO 1"

OFFICE OF PUBLIC WORKS — DUBLIN

DINGLE HARBOUR, Co. KERRY
PIER IMPROVEMENTS

PENETRATION OF STEEL SHEET PILES
& R.C. PILES

SCALES HOR. 16' TO 1" VERT. 8' TO 1" | DATE – DEC. 1975 | DRG. No. 762/21

DRAWN BY McB.
TRACED BY McB.
CHECKED BY ......

762

PLAN NO. Dingle Harbour PLANFILE NO. 762/21
PLAN TITLE Penetration of Piles

METROPOLITAN EQUIPMENT CO. LTD.
24 MERCHANTS QUAY
DUBLIN 1

DINGLE PIER DEVELOPMENT

INTRODUCTION

The Department of Marine and Natural Resources appointed Malachy Walsh & Partners in January, 1998 to act as Consulting Engineers on proposed developments in Dingle Harbour. The brief evolved into preparation of details for an extension to the existing pier, primarily to accommodate a proposed new ice plant, assessment of the existing suspended structures and dredging in rock material to provide access to the ice plant.

This Report assesses the technical and cost implication of these elements and recommends an optimum solution for this phase of the harbour development.

HISTORICAL BACKGROUND

The existing facility at Dingle is an amalgam of various extensions and additions to the original masonry pier. It is understood that this original masonry pier was constructed in the 1900's There was an extension in 1927 comprising a braced suspended reinforced concrete construction with a further side extension to same in the 1970's. In the late 80's and early 90's, a major development took place in Dingle Harbour. This development was phased over a number of years and included the construction of the new eastern breakwater, the new western breakwater and a major reinforced concrete extension to the pier.

The relationship between the various pier extensions and additions is illustrated on Drawing 3165/P1 in Appendix no. 1. The current development now envisages squaring off the head of the pier and providing for a new ice plant at the terminus. Dredging is required in the environs of the extension and for a substantial turning circle around same to provide for access to the new ice plant.

EVOLUTION OF BRIEF.

The current proposed developments in Dingle Harbour were initiated by the need to provide a new ice plant facility for the fishing fleet. This facility is being provided by B.I.M. and is essentially independent of work to be undertaken by the Department of the Marine. Technical details of the proposed new ice plant are being provided by the Tralee Firm of Architects, O' Sullivan Campbell. Details of their proposals are illustrated on drawing 3165/P2, in Appendix No. 2.

The new ice plant is a substantial building in length and in height. It is also has a substantial loading requirement, i.e. $20Kn/m^2$. This has obvious implications in terms of capacity of the existing suspended structures. There are specific constraints relating to the orientation of the ice plant vis a vis the main axis of the pier to facilitate unloading of ice into the receiving boats. Also, logistical matters such as circulation of pier traffic around the plant and operation of credit facilities at the plant must be taken into account. There are similar constraints relating to marine access. Dredging to -5.0 M.O.D. is a definite requirement if 24 hour use of the ice plant is envisaged for the larger boats.

Given these constraints, the brief to Malachy Walsh & Partners evolved into the following defining parameters;

- There is an absolute logic in providing a pier extension to square off the head of the pier. This extended pier in tandem with the existing provides adequate space for the installation and operation of the ice plant.

- It is inevitable that, given operational requirements at the pier, the ice plant will have to straddle both the existing and new structures. The new structures can obviously be designed to take the load from the ice plant. This, as will be recalled, is substantial and concentrated.

  The partial siting of the ice plant on the old pier has substantial load implications for the old pier. Structures such as this would not have been designed for loadings of the order of $20Kn/m^2$. Also, even a cursory examination of the underside of this suspended structure will highlight very significant reinforcement deterioration problems. All of these difficulties would suggest that a new method of vertical support should be identified for the old pier. This new support must be capable of transmitting the loads from the new ice plant.

- The brief recognises that dredging at the head of the pier will be necessary in order to facilitate 24 hour use of the ice plant. This dredging, which should be carried out to -5.0 M.O.D., will be used to link the berthing areas to the access channels and provide, at the same time, adequate turning circle space for boats using the facility.

BREAKDOWN OF REPORT

This Report assesses a number of different but inter related elements in arriving at an optimum solution to meet the proposed brief. These elements are examined separately in this report under the following heading;

PIER EXTENSION

- The Report examines a variety of possible solutions to achieve the requisite pier extension. These include suspended and piled structures of varying construction types. Also assessed are gravity and cofferdam type structures. The final choice of structure for the pier extension is made on cost grounds. However cognisance has also to be taken of its inter relationship with the existing pier and with the achievement of a cost effective way of dealing with the fact that a rock berm exists at the head of the pier and limits, to some extent, the possibility of dredging close to the head of the pier.

- ### *CONDITION SURVEY OF EXISTING PIER.*

It was recognised, from the onset, that the existing pier, given its age and construction type, might be suffering from chloride induced reinforcement corrosion. This would obviously reduce its ability to sustain new loading and would limit its prognosis in terms of design life. This Report sets out the site and laboratory tests that were carried out in order to define the durability prognosis on this structure and reports on its ability to sustain new loadings, especially loadings from the proposed ice plant. The Report ultimately defines how the extended pier structure and the upgrading work required on the old pier are intermeshed to provide an efficient and cost effective solution for the overall structure. ||

5

- ***DREDGING***

This Report examines the site investigation data, both new and existing, and defines an optimum dredge area. It highlights the problems associated with dredging in the rock that underlies much of the Dingle Bay and sets out the advantages and disadvantages of either disposing of dredge material at sea or on land. These constraints are primarily governed by environmental requirements. The report also examines possible ways of disposing of the dredge material in a useful fashion on land to initiate construction of a new length of breakwater on the eastern shores of Dingle Bay.

SITE SURVEYS AND LABORATORY TESTING

A variety of site and laboratory tests were necessary in order to provide the requisite information to address the issues defined in the brief. The site and laboratory tests included;

(a) Dimensional Surveys

A dimensional survey of both above deck and below deck elements was carried out in the area of the 1927 extension. These surveys are illustrated on drawings 3165/P3 and 3165/P4 in Appendix no. 3. The dimensional surveys are essentially self explanatory but the following important dimensional parameters should be noted;

- The 1927 extension is some 300mm lower than the main body of the pier. Given spring tide levels in Dingle Harbour, the existing 4.6 M.O.D. level would be considered to be low. However, it is suited to overslabbing either by way of structural upgrading and/or allowing for a raised floor level for the new ice plant.

- The below deck survey indicates a variety of suspended beam and slab elements. The location of all of these beams, cross braces, vertical braces, etc, are somewhat restrictive to locating any new structure that would penetrate the deck slab and would be founded on the underlying rock.

(b) Site Investigation

There was considerable site investigation data available from previous developments in Dingle Harbour. This information was very useful in defining preliminary solutions for the new pier structure and for assessing the problems associated with the dredging work. However, given the extent of the development and the difficulties associated with providing foundations for the new pier structure and with rock removal over the dredge area, it was considered prudent to initiate further and more concentrated site investigations. Following a tendering process, Site Investigation Ltd. carried out this new work. This included shell and augur boreholes to determine the nature of the overburden over the rock and rock coring to determine the strength and rippability of the rock. A copy of the site investigation report is included in Appendix No. 4. Also, enclosed are copies of drawings 3165/P5, P6, P7, P8, P9, P10, & P11. These drawings take sections through the new pier extension and the dredge area and thereby define visually the extent of the work. It will be obvious that much of the dredging is in difficult ground conditions, necessitating either removal of weathered rock or solid rock. Provision of adequate foundations, for whatever structure is ultimately decided upon, is not a problem.

C) Photographic Record.

A photographic record of the 1927 and 1970's extension is included in Appendix No. 5. This photographic record defines the visual condition of the suspended elements to the underside of the deck. It will be noted that widespread spalling and cracking is occurring on all beams. From visual inspection alone, it is obvious that very substantial reinforcement deterioration is occurring in the structure. This is not surprising, given that similar deterioration has been observed on many coastal structures of similar design and lesser vintage. The implications of these durability problems is discussed in detail later in this Report.

(d) Laboratory Tests.

The durability prognosis on the suspended elements of the 1927 and 1970's extension is also defined by the extent of chloride ingress into the concrete. Chlorides are naturally occurring in the seawater and, when they penetrate into reinforced concrete, they will initiate very substantial corrosion problems with consequent breakdown and spalling of the concrete. A detailed background assessment of the problems associated with deterioration of reinforced concrete is given in Appendix No. 6. X

In order to define the durability prognosis of this structure, a range of core and spall samples was taken from the structure and tested by Southern Scientific Services. A copy of their report and the test locations on same (Ref. 3165/P12) is included in Appendix No. 7. The results of the Southern Scientific laboratory tests arediscussed in more detail later in this report. However, even a cursory inspection of the report will indicate very serious and substantial levels of chloride present at depth in the concrete. The extent of this chloride penetration ultimately defines the durability prognosis on the structure. This prognosis is not good.

PIER EXTENSION

DESIGN PARAMETERS AND CONSTRAINTS

A variety of structural options were developed in order to arrive at an optimum solution for the pier extension. The design requirements for all of the options was essentially similar i.e.

- The extended structure must be capable of supporting significant dead loads i.e. of the order of 20Kn/m².

- The extended structure must be self supporting in terms of lateral strength.

- A clean vertical face, suitable for berthing without use of fendering, is required. This concrete face should extend at least 1m below Mean Low Water Spring levels.

The constraints relating to the pier extension are also similar for each of the considered solutions;

- The extension is dimensionally constrained by the boundaries of the existing structures.

- Rock occurs at shallow depth below bed level. The inevitable finding of any site investigation report is that the structural loads should be transferred directly to rock.

- A rock berm was provided at the head of the pier to prevent undermining of the old structure. Ultimately, dredging of rock would have to be considered right up to the face of the existing structure if access to the new ice plant is to be provided. This has obvious implications for the stability of the existing suspended deck/caisson support structure.

OPTIONS:

The pier extension options examined in this report are included in Appendix no. 8. The structure type include;

1. ***OPTION NO. 1*** - ***TIED SHEET PILE WALL***

This option is indicated on drawing 3165/P13. It envisages steel sheet piling, tied across the width of the extension and capped at deck level with a new ground bearing insitu reinforced concrete deck slab. This solution envisages substantial dredging under the footprint of the pier extension to remove any soft and unsuitable material.

In any critical examination of this solution, cognisance must be taken of the fact that deterioration of sheet pile structures is a well recognised phenonomen and, unless substantial pile sections are used, maintenance within 20 years is inevitable. Also, an extent of hardwood fendering to the berthing face is also desirable if the the sheet pile structure is used.

2. ***OPTION NO. 2*** - ***SUSPENDED DECK.***

Option no. 2 is indicated on drawing 3165/P14. This option envisages the load transferral to foundations using driven steel piles. These piles will be designed with a sacrificial thickness to cater for corrosion of the steel. The suspended deck elements would be provided by a series of either insitu or precast beams supporting a precast deck, topped with an insitu screed.

A vertical reinforced concrete face is provided on all of the berthing faces using a suspended precast concrete panel system tied to the new structure. This solution does not require dredging under the footprint of the proposed pier.

3. ***OPTION NO. 3*** - ***SUSPENDED PRECAST BEAM DECK.***

Option no. 3 is indicated on drawing 3165/P15. This option is somewhat similar to Option no. 2 but only two rows of piles with rakers are envisaged. The longer span at deck level is provided by precast pretensionded beams spanning across the deck and topped with an insitu concrete slab. The berthing face, as with Option 2, is provided by a precast concrete panel system hung off the new structure.

4. ***OPTION NO. 4*** - ***GRAVITY STRUCTURE.***

Option no. 4 is illustrated on drawing 3165/P16 in Appendix no. 7. This option envisages excavation of all of the material under the footprint of the pier extension down to good rock and the blinding of the rock with leanmix concrete. Cellular reinforced concrete cofferdams would then be used to make up the proposed structure. These cofferdams would be self supporting both laterally and vertically and would provide the vertical berthing face required in the Brief.

5. ***OPTION NO. 5*** - ***ALTERNATIVE GRAVITY STRUCTURE.***

Option no. 5 which is illustrated on drawing 3165/P17, is a variation on Option No. 4 with horizontal precast block rings being used to form the gravity structure. This structure would again be founded on the underlying bedrock and would have no problem in providing the vertical berthing face required in the Brief.

6. ***<u>OPTION NO. 6.</u>*** - ***<u>FILLED GRAVITY STRUCTURE</u>***

Option no. 6 is a further variation of the gravity structure and is formed using interlocking reinforced concrete units with infill precast wall units. As with the previous gravity structures, this structure is founded on the bedrock. (Ref. P18)

7. ***<u>OPTION NO. 7.</u>*** - ***<u>CIRCULAR STEEL COFFERDAM</u>***

Option no. 7, which is illustrated on drawing 3165/P19, is formed using interlocking circular steel pile cofferdams filled with hardcore and capped with an insitu groundbearing reinforced concrete slab. This structure relies on hoop tension to retain its stability. It does not require the same extent dredging as the other gravity structure, but nonetheless, would require that the hardcore material be placed on ground of a reasonable bearing capacity.

Given that the interlocking cofferdams would present an irregular and unsuitable face for berthing, a fender system would have to be provided.

Previous experience with circular cofferdam construction would suggest possibilities of differential settlement within each cofferdam. This would compound the problem of providing a regular rectangular deck without compromising different and differing settlement rates in the cofferdams. Notwithstanding this, the steel cofferdams, once erected properly in an agreed sequence are stable and will provide an acceptable gravity type construction.

COMPARISON

A comparative assessment of the various options has been prepared. This comparative assessment is based on the following;

- **COST**
- **BUILDABILITY**
- **DURABILITY**
- **TIME FOR ERECTION**
- **INTERGRATION WITH OLD PIER**
- **POTENTIAL PROBLEMS AT HEAD OF THE OLD PIER**

The results of this comparative review is set out in figure . The comparative cost plans for the various options is included in Appendix No. 9. It will be noted that Option 3, i.e. the precast suspended deck with steel piles is the most favoured on cost and buildability grounds. Equally, this solution can be easily intermeshed with the old pier especially if a new slab overlay with piles is required to upgrade this pier. This interaction is discussed in more detail later in this report.

HEAD OF PIER

The intent of the brief was that 24 hour marine access should be provided to the ice plant. Implicit in this is the requirement that dredging to -5m O.D. will be necessary. This would obviously create problems at the head of the old pier. Here, it will be recalled, that a rock berm was provided to prevent undermining of the old pier. Excavation in this rock berm and in underlying rock would be necessary to depths varying from 3 to 5m if the Brief is to be satisfied.

This report has examined the engineering constraints relating to dredging at the head of the old pier. Ultimately, it was clear that two options are feasible;

- Dredge to -5 M.O.D., while at the same time, providing some means of vertical and lateral support to the old pier.

- Extend end of pier by some 6m using a suspended and piled structure, thereby superceding the need to dredge adjacent to the old pier.

UNDERPINNING OLD PIER.

If dredging up to the face of the old pier was envisaged, then lateral support would have to be provided by some form of contiguous bored piles. These piles would be used to retain the excavated face of the berm and the rock under. Since the piles would be in the form of a contiguous wall, they would be continuous over the full width of the pier head.

PIER HEAD EXTENSION.

The alternative option to the underpinning solution is to extend the pier head by some 6m using a suspended structure as per Option 3. Details of this suspended structure are indicated on drawing 3165/P20, in Appendix no. 10. This solution, as will be seen from the drawing, has superceded the necessity to remove the rock berm especially close to the old pier head. This has obvious attraction in terms of buildability and prevention of engineering claims during construction.

A comparative assessment has been made between the cost of the contiguous bored pile wall and the pier extension. These costs are;

Contiguous Wall	-	£
Pier Head Extension	-	£

An argument could be made for using the bored pile solution on cost ground,. However, this report strongly recommends the pier extension on the following grounds;

- The pier extension obviously gives 6m additional length on the pier. This would be welcome in any scenario.

- Notwithstanding the cost differential of £ , excavation close to the old pier structure and provision of the bored pile supports is very vulnerable to any problems that might be encountered during construction stage. In such a scenario, the favourable cost differential would be wiped out with an engineering claim by the contractor. A situation could arise where the underpinning work would have to be abandoned or modified significantly in order to preserve the stability of the old pier.

CONDITION SURVEY

The ice plant, as located on drawing 3165/P2, in Appendix No. 2, indicates that the plant will straddle the new pier extension and the old 1927 pier. As has already been pointed out, the increased loadings on the old pier would be significant i.e 20Kn/m^2 live load. The old pier, or some improved version of same, must obviously be able to sustain this increased loading and must have a good durability prognosis in terms of design life.

In the absence of detailed reinforcement drawings of the 1927 extension, it would be difficult to assess its capacity to sustain increased loading. However, the need to do this is superceded by the results of the durability studies on the pier extension. It will be recalled that the photographic survey has highlighted very substantial reinforcement deterioration problems on the suspended elements. The laboratory test results in Appendix No. 7 has confirmed this view.

These tests list chloride concentrations by weight of concrete. Using a factor 5 to estimate the cement content, surface salt levels vary from 0.54% by weight of cement to 5.35%, by weight of cement. By any standards, these are very substantial chloride concentrations, especially when one considers that corrosion activity is initiated at around 0.35% chlorides by weight of cement. The results from sample 7 is some 15 times that activation level.

The comparison given above is a surface comparison and would generally not correspond with chlorides levels at the depth of the reinforcement, i.e. 50 to 75mm from the surface. If these latter levels are examined, then chloride concentrations vary from 0.25% by weight of cement to 3.1% by weight of cement. This latter figure is some 10 times activation level. Given that it is measured at the depth of the reinforcement, it signifies a very poor outlook on the durability prognosis of the structure.

ASSESSMENT OF OLD PIER - CONCLUSION

The inevitable conclusion of the assessment of the old pier is that its future design life is very limited. Also, previous experience with marine concrete repairs would suggest limited benefits only in the initiation of a repair programme. A repair programme would probably involve substantial concrete breaking out, recasting of beams using patent replacement concrete, protective coatings to beams and general upgrading. The cost implication of this work is substantial and, given that significant chloride contaminated concrete would inevitably remain in the structure, the time to future repair would certainly not be in excess of 12 years.

Given the poor durability outlook of this structure and since the loads from the new ice plant are very substantial anyway, Malachy Walsh & Pts. have developed an alternative support scenario for the existing pier. This new scenario assumes that all vertical loads will be taken by a new piled structure, driven down through the existing structure on to the bedrock. A new heavy reinforced concrete overlay will be used to support the ice plant and associated pier loadings. Details of this new option are indicated on drawing 3165/P21 in Appendix no. 11.

An examination of this proposal indicates the advantages of proceeding on this basis:

1. The increased load from the ice plant and the adjacent pier is now supported directly off the existing bedrock.

2. The new pile structure with overlay slab is entirely compatible with the piled pier extension already proposed. Equally, these structures will intermesh successfully with the new piled pier head extension.

3. While the overlay structure with supporting piles has a vertical load bearing capacity it is not strong in terms of sway and lateral support. However, given that it can be linked to the pier extension and, since this pier extension has raking steel piles, then adequate lateral support is provided for the total structure.

The conclusion and recommendation of this report is that an overlay slab with supporting piles should be provided to the 1927 pier extension. This will supercede the need to carry out any repairs to the existing structure under. The old structure can continue to deteriorate given that it only will be a sacrificial shuttering to the new slab.

EXISTING SHEET PILE WALL

It is instructive, at this stage, to comment on the condition and future prognosis on the existing sheet pile wall that runs for the full extent of the pier at Dingle. This sheet pile extension was provided in the 1970's and was used to support a timber fendering system providing a berthing phase for the fishing fleet. The sheet piling does appear to have operated in a satisfactory fashion and, notwithstanding the fact that deterioration has occurred, it can give, for the medium future, a useful service to the pier.

However, as already pointed out, some deterioration is occurring, especially at connections. Also, if the present pier extension occurs, then the eastern face will be the only face where timber fendering is used. Experience on pier structures would suggest that a reinforced concrete berthing face is more favourable, both in terms of strength and longterm durability. It is the considered opinion of this Report that, in the future, a new reinforced concrete skin overhang will be provided for the full extent of the eastern face to the pier. The proposed overlay slab can be designed to support this new skin.. However, provision of this new skin does not form part of this report.

DREDGING

Dredging at the head of the pier and for an appropriate turning circle around same is required in order to provide 24 hour access to the ice plant. The extent of the dredging and sections to same are indicated on drawings 3165/P5 to P11 in Appendix no. 4. It will be noted that significant volumes of dredging are in rock. These volumes are;

Silt Dredging -

Rock Dredging -

In terms of a significant dredging contract that would attract international contractors, the dredge volume at m^3 is obviously very small. This low volume has substantial implications for the cost of mobilising any specialised dredging plant to site. It highlights the importance of deciding whether the disposal of the dredge material at sea or on land will be specified.

DREDGE MATERIAL - DISPOSAL

The implication of specifying disposal at sea are significant. In the first instance, as already pointed out, disposal at sea will attract significant costs in bringing a specialised dredger and appropriate barge to site. Also, a disposal licence will be required. While this almost certainly will be forthcoming, acquisition of such licenses is getting more difficult. If the material is disposed of at sea, then the excavated rock will not be put to any further use. Obviously, in both practical and environmental terms, this is not desirable.

Land disposal has many attractions. In the first instance, the rock material is useful and has an added on value, however small. Also, no disposal licenses are required.
If, however, land filling of some type was specified then planning permission might be required. However, this should not be a problem provided that the disposal location is not contentious.

Removal of the material for land disposal is a significantly difficult operation. In the first instance, under water rock breaking equipment will have to be used. Also, a long reach machine, on a floating barge, will be necessary in order to access material. In that regard, it might be noted that significant volumes of the material could be accessed from the existing pier.

In the Cost Plan, included in Appendix No. of the report, disposal on land is assumed. Malachy Walsh & Pts. did carry out an exercise assuming disposal at sea. If this is assessed on cost grounds only, then it would be more expensive by a factor of over the land disposal option. Disposal at sea is therefore not to be recommended in any scenario.

NEW BREAKWATER

Dingle Harbour Commisssoners have a view that provision of a new length of breakwater on the eastern shore of the bay, adjacent to the sewerage treatment plant and running west from same, would provide significant additional protection in the inner harbour from south easterly gales. This view has already been confirmed by the fact that the temporary access road provided by Kerry County Council to the outfall pipe did provide added protection in the inner harbour. A very tentative location for this breakwater is included on drawing 3165/P22 in Appendix No. 12. While no site surveys, site investigations or any wave or current analysis have been carried out for this breakwater, it is assumed that the cost per meter run for same would be of the order of £3,000 excluding VAT. Using a breakwater length of then the overall cost of this new breakwater would be of the order of £ ?.

The material being excavated as part of the pier development would be suitable for core material only. If one assumes that the cross section at the area of the core if the breakwater would be of the order of m^3, then adequate core material would exist for approximately m. length of new breakwater. However, while the Harbour Commissioners might save £7 per cubic meter in provision of the material, it would cost the same again to place it. Also, since the material would be placed as core material, then rock armouring would have to be carried out in order to protect it, even if only a short length of same was completed.

There is obvious merit in investigating the provision of the new breakwater. There is also an obvious cost benefit if the material dredged from the pier development can be used. However, additional expenditure will be required. The costs of this, other than the broad figures given above, have not been highlighted in this Report.

COST PLAN

A detailed cost plan for the preferred and recommended development solution for the pier extension and associated dredging is included in Appendix no. 13. The master plan for same is indicated on 3165/P23 in Appendix 14. This preferred solution entails;

- The pier extension will be formed using steel piles and a prestressed concrete beam deck.

- A 6m extension will be provided at the head of the old pier in order to prevent undermining of same.

- The dredge area will be as specified in Appendix 12.

- Disposal of the dredge material will be carried out on land.

- The support for the new ice plant and the associated overlay to the old pier will be formed using a reinforced concrete slab supported on steel piles penetrating through the old deck.

The Cost Plan, in Appendix 13, has identified the overall expenditure required for this project to be of the order of excluding VAT. The cost breakdown is as follows;

- Pier Extension excl. VAT

- Pier Head Extension,

- Overlay to old deck with pile support.

- Dredging.

This recommended solution is considered to be the most cost effective and the least likely to be susceptible to claims at construction stage. If problems do arise and if variations occurs at contract stage, then they are likely to be caused by different interpretations of the difficulty and costs associated with rock removal. It is very difficult to control this in the Contract Documents other than to provide very clear and precise documentation and minimise any contradictions in the site investigation data. The intent is that this should be done.

STATUTORY APPROVALS

Statutory approval will be required for this development as follows;

- Planning permission will be required for the ice plant. This is obviously a B.I.M. project. Nonetheless, it does interact with the pier. The intent is that a combined planning permission should be provided.

- A foreshore licence will be required for the pier extension and the associated dredge area. If work is anticipated to be carried out at the possible new breakwater, then this will also have to be covered by the foreshore licence.

- Fire Certification will obviously be required for the B.I.M. building. Again, this is a matter that will be processed by B.I.M.

In recent months, Malachy Walsh & Pts. in association with B.I.M and in association with O' Sullivan Campbell, Architects, have carried out a consultative process with local interests in Dingle, with regard to this development. Generally, this has been very favourable. Consultation has also been carried out by O' Sullivan Campbell with Kerry County Council with regard to any difficulties associated with planning of the ice plant. Obviously , the ice plant is a significant structure, especially in height. Nonetheless, Kerry County Council do recognise that an operating fishing harbour need an ice plant. The present location is considered to be by far the most practical and satisfactory.

CONCLUSION.

This Preliminary Report assesses the site and laboratory tests carried out on site by Malachy Walsh & Pts. It identifies that the optimum pier extension will be a suspended concrete deck supported on steel piles. The Report recommends a 6m extension at the head of the old pier to prevent undermining of same. The extent of the dredge area is identified and disposal of the dredge material on land is recommended. The overall cost for the project is assessed to be of the order of £ excluding VAT. This does not include professional fees.

APPENDIX NO. 1

HISTORICAL BACKGROUND

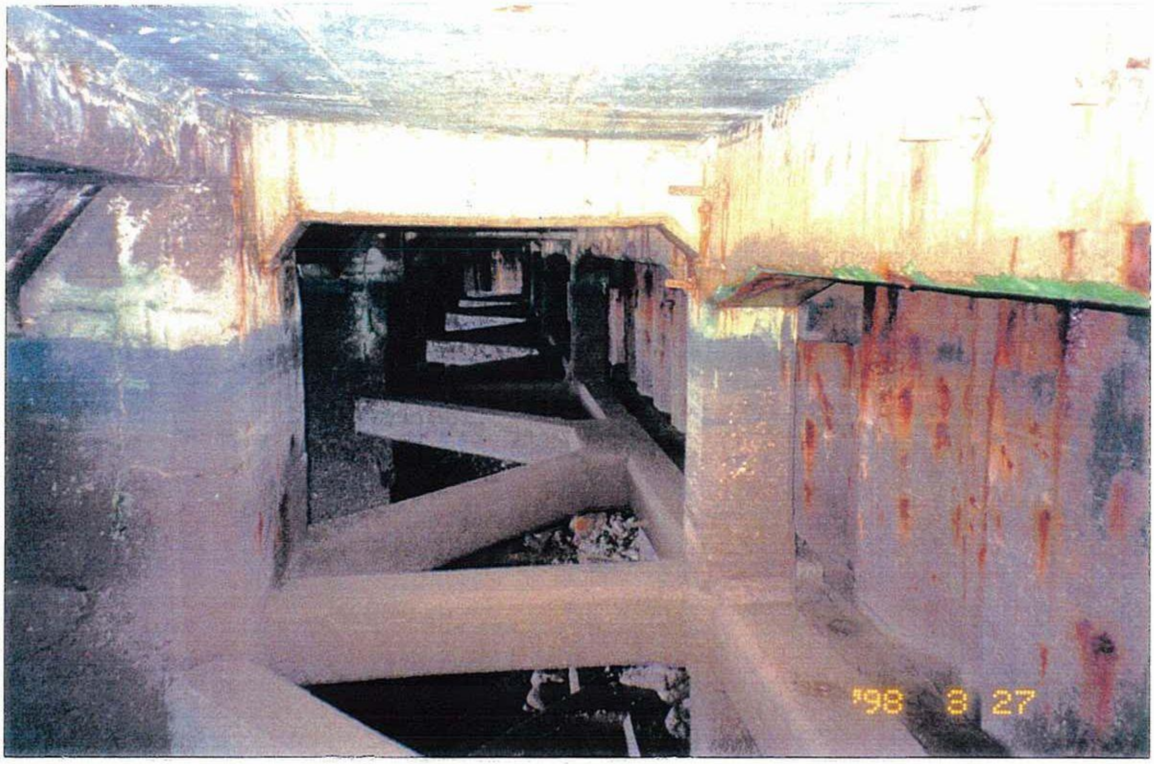
EXTENSIONS & ADDITIONS

APPENDIX 5

PHOTOGRAPHIC REGISTER

P.1. - WEST ELEVATION

P.2. - EAST ELEVATION



P.3. - 1970's EXTENSION

P.4. - GENERAL ARRANGEMENT / 1927 EXTENSION

P.5. - TYPICAL DETERIORATION / SECONDARY BEAMS

P.6. - SLAB DETERIORATION

P.7. - DETERIORATION OF BEAM ELEMENTS

P.8. - DITTO AS P7

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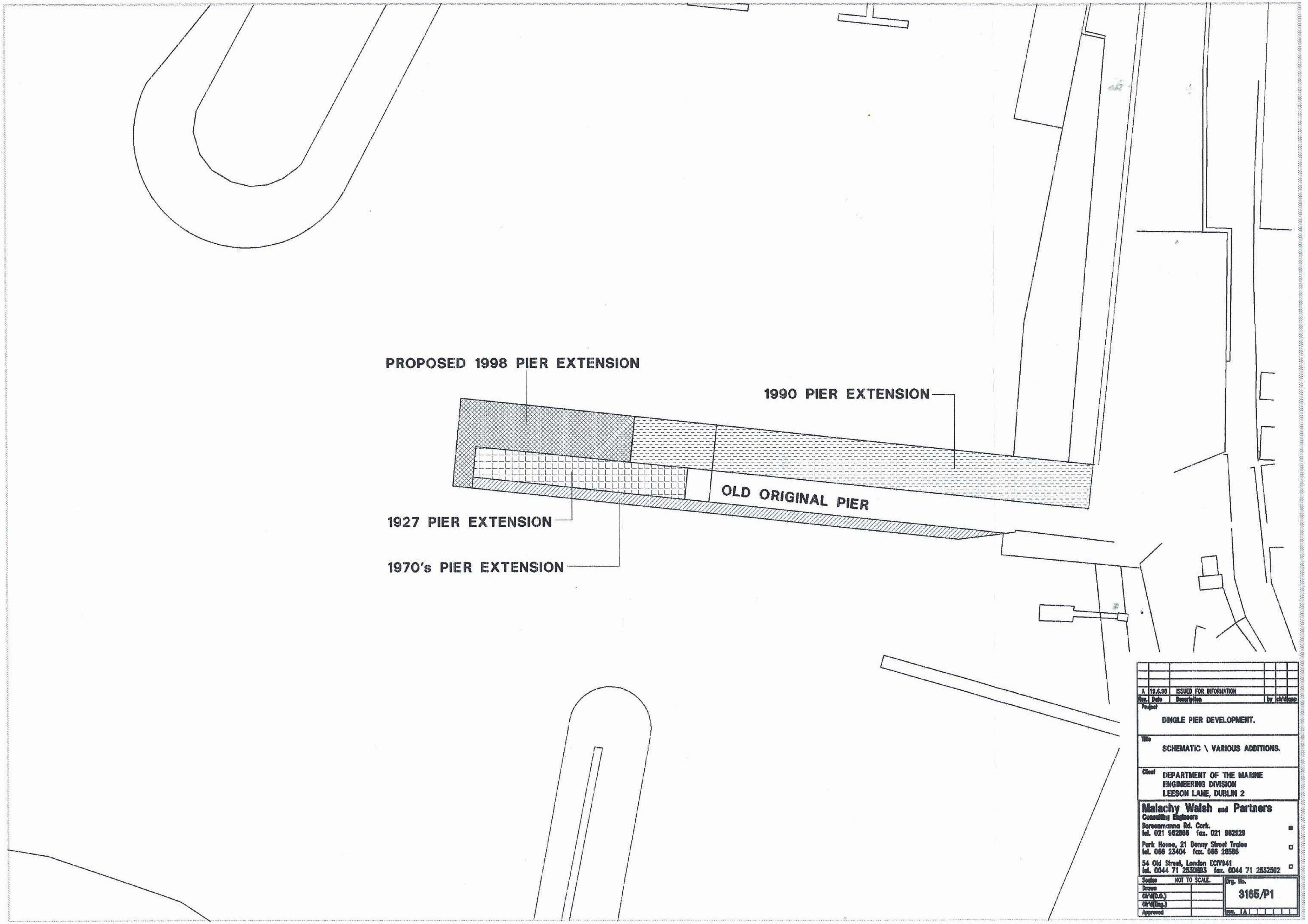
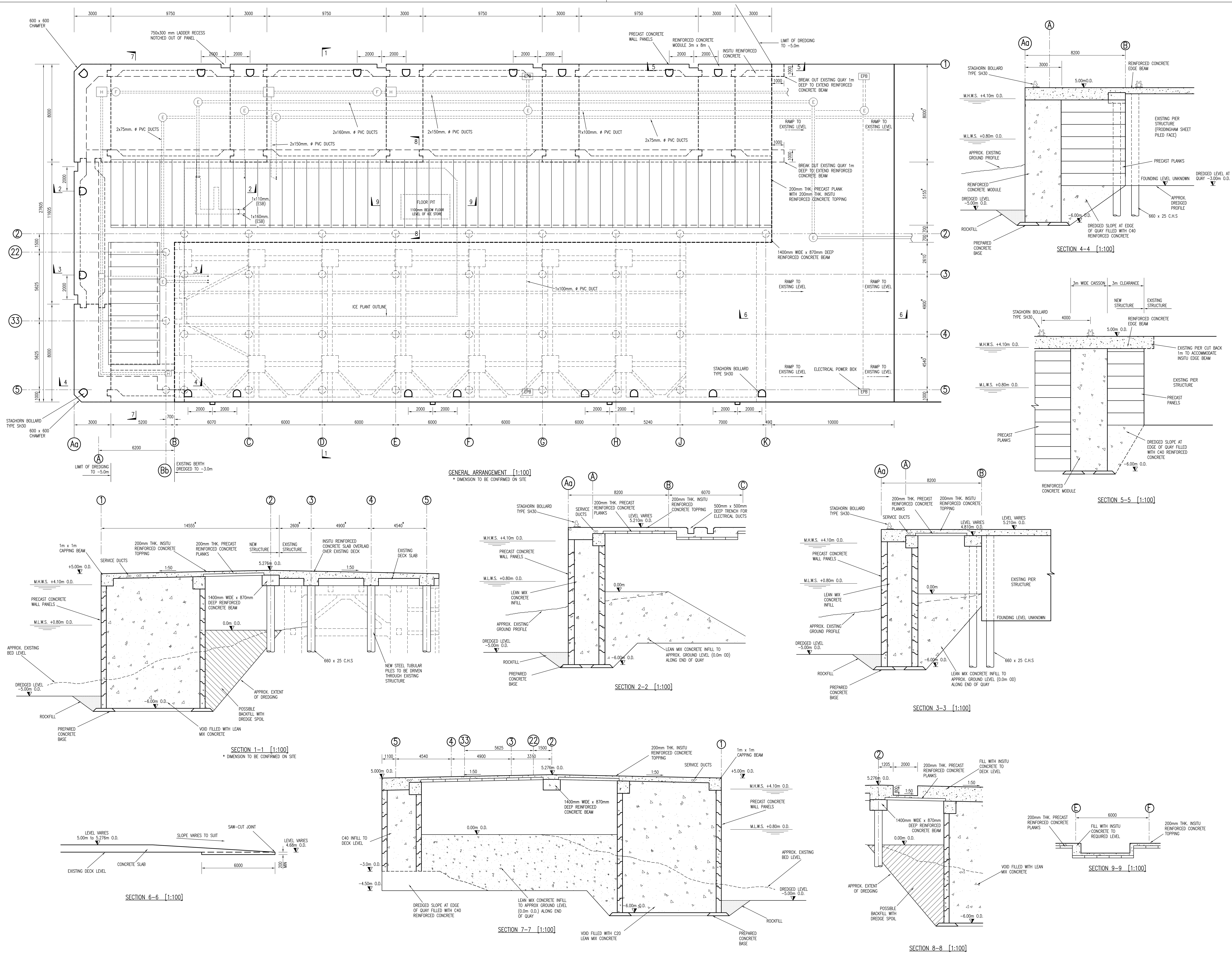
APPENDIX 6

DURABILITY PROBLEMS

IN

REINFORCED CONCRETE


PROPOSED 1998 PIER EXTENSION
1990 PIER EXTENSION
OLD ORIGINAL PIER
1927 PIER EXTENSION
1970's PIER EXTENSION
A 19.6.98 ISSUED FOR INFORMATION
Rev. Date Description by ch'd app
Project
DINGLE PIER DEVELOPMENT.
Title
SCHEMATIC \ VARIOUS ADDITIONS.
Client
DEPARTMENT OF THE MARINE
ENGINEERING DIVISION
LEESON LANE, DUBLIN 2
Malachy Walsh and Partners
Consulting Engineers
Boreenmanna Rd. Cork.
tel. 021 962866 fax. 021 962929
Park House, 21 Denny Street Tralee
tel. 066 23404 fax. 066 28586
54 Old Street, London ECIV941
tel. 0044 71 2530893 fax. 0044 71 2532502
Scales NOT TO SCALE.
Drawn
Ch'd(D.O.)
Ch'd(Eng.)
Approved
Drg. No.
3165/P1
rev. A



NOTES

1. VERIFYING DIMENSIONS. THE CONTRACTOR SHALL VERIFY DIMENSIONS AGAINST SUCH OTHER DRAWINGS OR SITE CONDITIONS AS PERTAIN TO THIS PART OF THE WORK.

2. SERVICES. APPROVED OPENINGS FOR SERVICES THROUGH THE STRUCTURE ARE INCORPORATED ON THE DRAWING. ANY ADDITIONAL OPENINGS OF A MINOR NATURE REQUIRED BY THE MAIN CONTRACTOR OR HIS SUBCONTRACTORS MUST BE SUBMITTED ON A DRAWING FOR APPROVAL BEFORE WORK COMMENCES.

3. DATUM. ORDNANCE DATUM

4. CONCRETE	MIX TYPE	COVER	FINISH
R.C. MODULES	C40	70mm	F3 (steel formwork)
PRECAST PLANKS	C40	60mm	F3
INSITU CONCRETE	C40	60mm	F3
LEAN MIX INFILL	C20	/	/

5. ALL CONCRETE EDGES TO HAVE 25 x 25mm CHAMFERS

6. ALL PRECAST DECK PLANKS TO BEAR 200mm ONTO SUPPORTING BEAMS

7. EPB = ELECTRICAL POWER BOX
E = ELECTRICAL DRAW PIT
F = FUEL POINT
H = WATER HYDRANT

REV	DESCRIPTION	DRAWN DATE	CHECKED DATE
B	MINOR ADMENDMENTS TO DECK BEAMS AND VERTICAL PLANKS AT HEAD OF PIER AND JUNCTION TO OLD QUAY		
A	EDGE BEAMS ADDED, SERVICES AMENDED AND ADDED, DECK LEVELS SHOWN CROSS-SECTIONS 6-6 TO 9-9 ADDED AND DETAILS AMENDED	RMG 20/03/00	Do'L 20/03/00

DRAWN BY M.H. DATE 16/02/2000 | CHECKED BY Do'L DATE 16/02/2000 | APPROVED BY Do'L DATE 16/02/2000

BASE DRAWING SCALE 1:100 | SCHEDULE SHEETS –

CLIENT
IRISHENCO CONSTRUCTION Ltd.

PROJECT
DINGLE HARBOUR DEVELOPMENT
PIER EXTENSION

TITLE
GENERAL ARRANGEMENT OF
MODULAR CAISSON SYSTEM
AND PIER EXTENSION

KIRK McCLURE MORTON
TEL: 01232 667914 CONSULTING ENGINEERS FAX: 01232 668286
ELMWOOD HOUSE 74 BOUCHER ROAD BELFAST BT12 6RZ

ARCHITECT

DRAWING No. 4100.45/C/01

REVISION A B

DWG. STATUS: PRELIMINARY / TENDER / CONSTRUCTION ● / RECORD