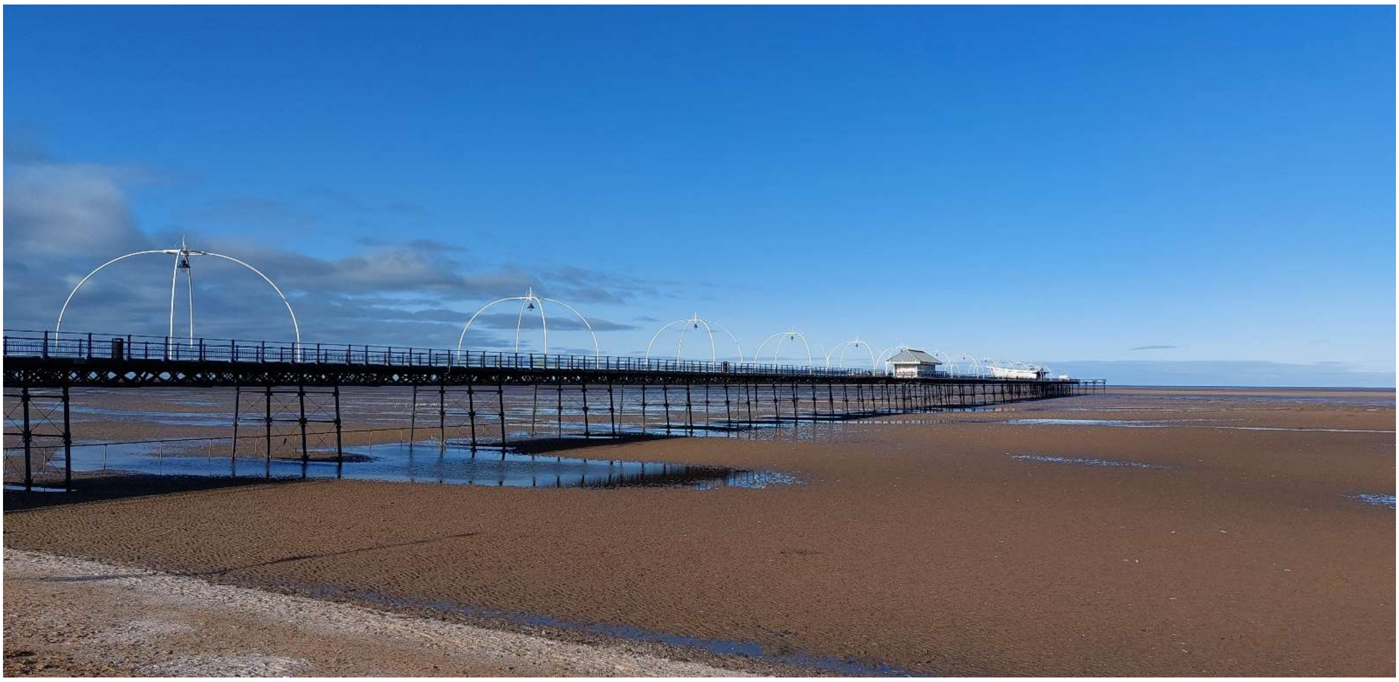
Structural Steelwork Inspection

at

Southport Pier
Promenade
Southport
PR8 1QX



for

Sefton Metropolitan Borough Council
Magdalen House
Trinity Road
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Contract No: LV1255
Dated: June 2023

Prepared by	Signature	Date
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Issue	Description	Date	Originator	Approved
01	First Issue for Comment	12/06/23	AJJ	IJB
02	Final Issue	08/08/23	AJJ	IJB
03	Updated Final Issue	25/08/23	AJJ	IJB

Whilst every effort has been made to fully inspect those parts of the building requested of us, no permanent or secured fixtures and fittings will have been removed. We will not have inspected woodwork or other parts of the structure which were covered, unexposed or inaccessible, and we are therefore unable to report that any such part of the property is free from defect.

Thomason Partnership Limited certify that they have carried out the work contained herein with due care and diligence to their best belief and knowledge based on the time and information available.
This report is made on behalf of Thomason Partnership Limited. By receiving it and acting on it, the client – or any third party relying on it – accepts that no individual is personally liable in contract, tort or breach of statutory duty (including negligence).

<u>THOMASONS</u>

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1.0 INTRODUCTION

1.1 Southport Pier is a Grade II listed structure originally opened in 1860. The pier has undergone significant changes due to damage and fires over the years and was fully refurbished over a four year period from 1998. The pier originally had a steam train running along the south side edge of the deck. During the refurbishment, the rails were moved to the centre of the deck to allow the running of a new electric tram along the full length of the pier, and the deck widened to one side. The tram has since been decommissioned and a surface running train has been utilised. In December 2022, the structure was closed to the public for health and safety concerns relating to the condition of the timber decking.

1.2 Thomasons have been instructed by Sefton Metropolitan Borough Council to undertake a visual inspection of the pier. The inspection has been broken down into two elements, the first includes the timber decking and supporting timber joists, which is covered in a separate report. The second element includes an inspection of the isolated sections of the supporting frame beneath the timber decking and joists. This supporting frame consists of a mix of cast iron and steelwork members.

1.3 The purpose of this report is to summarise the condition of the structural frame and advise on any remediation works required. Any degradation in the strength of the existing structure based on the appearance and corrosion present to the steel would be identified. The inspection was carried out by Thomasons starting 22nd February 2023 concluding on 2nd May 2023. This was a non-destructive visual inspection of the structural frame and incoming trusses at six locations along the landward side of the pier between the pier entrance and the sea wall.

1.4 The seaward section of the pier was visually inspected to the underside, from the beach. Deck boards were lifted at various locations to allow visibility of the steelwork frame from above. Steelwork over the lake was excluded from this survey due to lack of access during the survey. Thomasons were assisted with access by Davies Maintenance.

1.5 Documentation provided prior to the survey included a topographical survey carried out by Formby Surveys, and a feasibility study by Mott MacDonald for Programme of Major Maintenance Works dated December 2014. These documents and drawings are included in the Appendix Section at the end of this report.

2.0 OBSERVATIONS

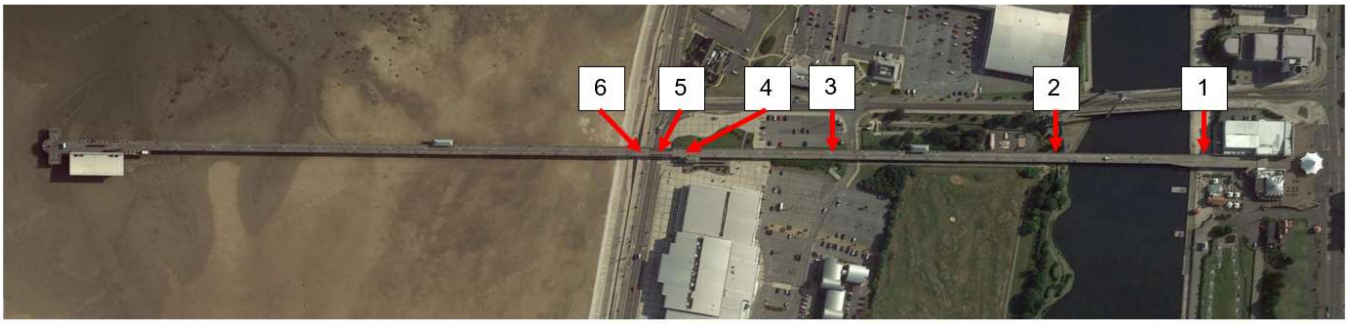
Photograph 1 - Aerial image of Southport Pier (Google maps)

2.1 The topographical survey produced by Formby Surveys is used as a reference for Thomasons survey notes and co-ordination with reference points on site. Along the pier, there are a number of overhead light frames that are numbered, see photograph 2, these were used as reference points for the timber deck and steelwork inspections.

Photograph 2 – Indicating light frame numbers (No. 4 shown)

2.2 Thomasons inspected the structural frame at the locations indicated on photograph 3 below. For clarity these areas are:

- Adjacent to lake on the Funland (East) side.
- Adjacent to the West side of the lake.
- Under Light Frame No. 10, adjacent to the access road into the leisure park.
- Underside of the Donut shop
- Frame adjacent to the East side of bridge over Marine Drive
- Frame adjacent to the West (Beach) side of bridge over Marine Drive



Photograph 3 – Plan on pier indicating focused areas of inspection

2.3 This deck report covers:

- The steel frame areas noted above.
- Overview of the steelwork over the beach

2.4 The following pages include Figures 1 to 4 to show the layout and indicative member sizes:

- Plan on a typical Frame Bay along the pier
- Typical section through the deck,
- Typical elevation of the longitudinal truss support frame,
- Elevation on a typical longitudinal truss.

2.5 Truss notation was added by Thomasons to provide clarity for referencing in photographs and figures in this report.

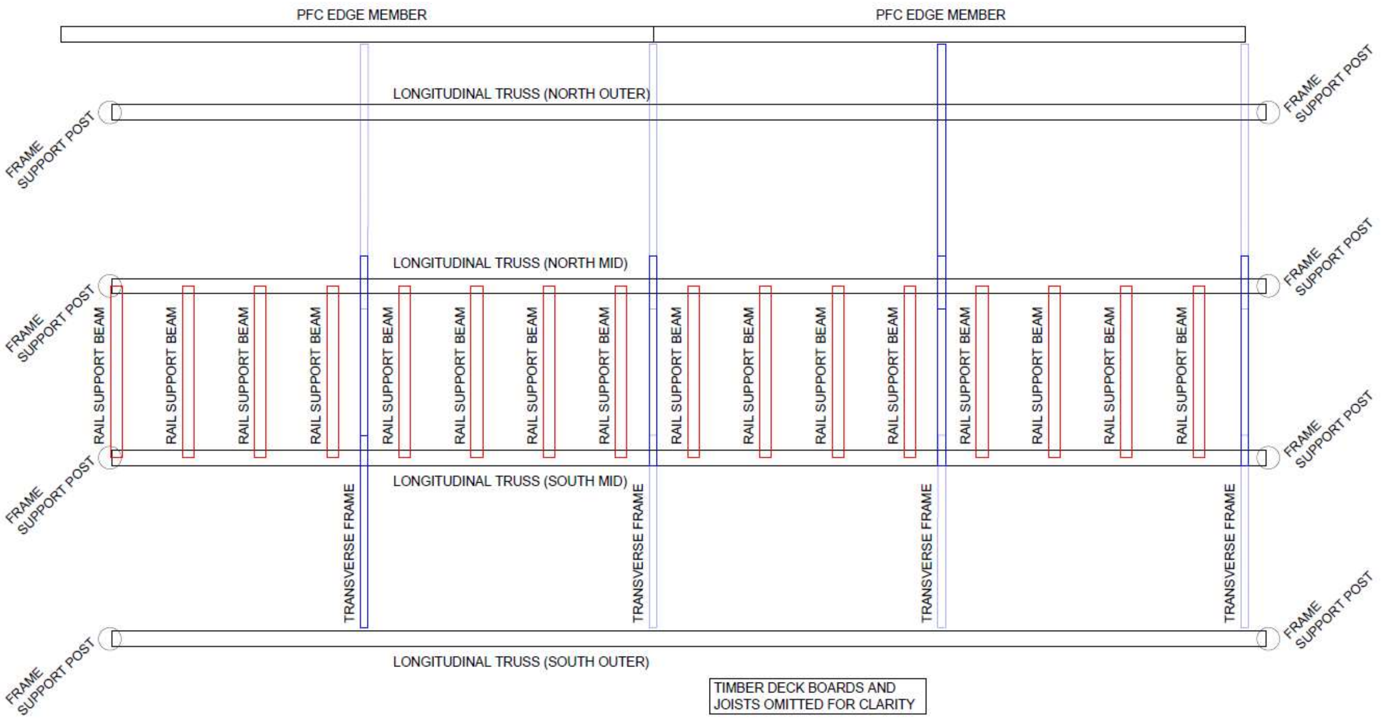


Figure 1 – Plan on Typical Frame Bay to Pier

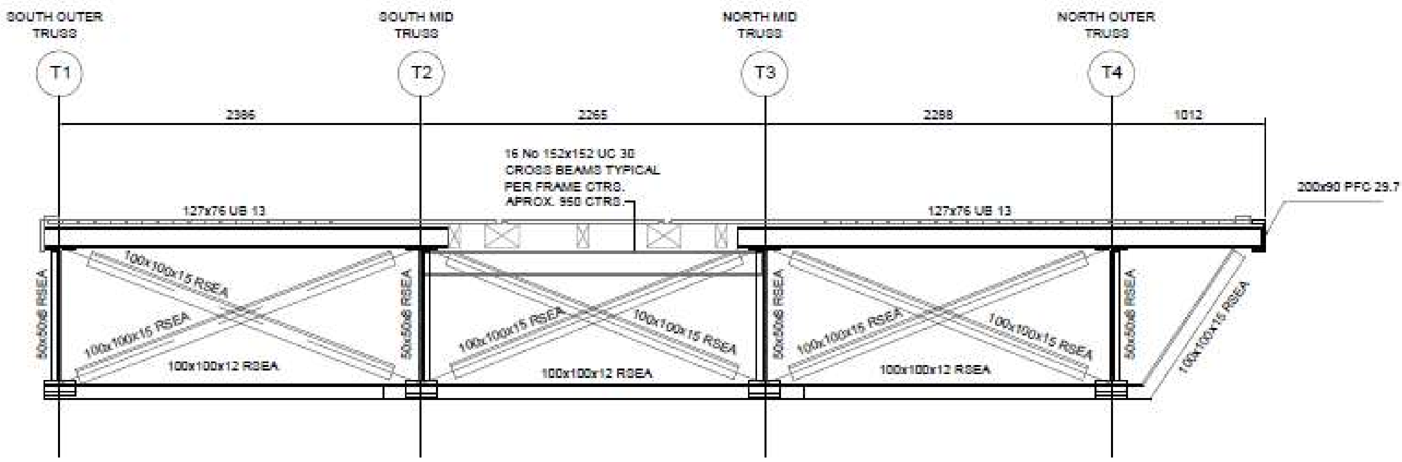


Figure 2 – Typical section through deck

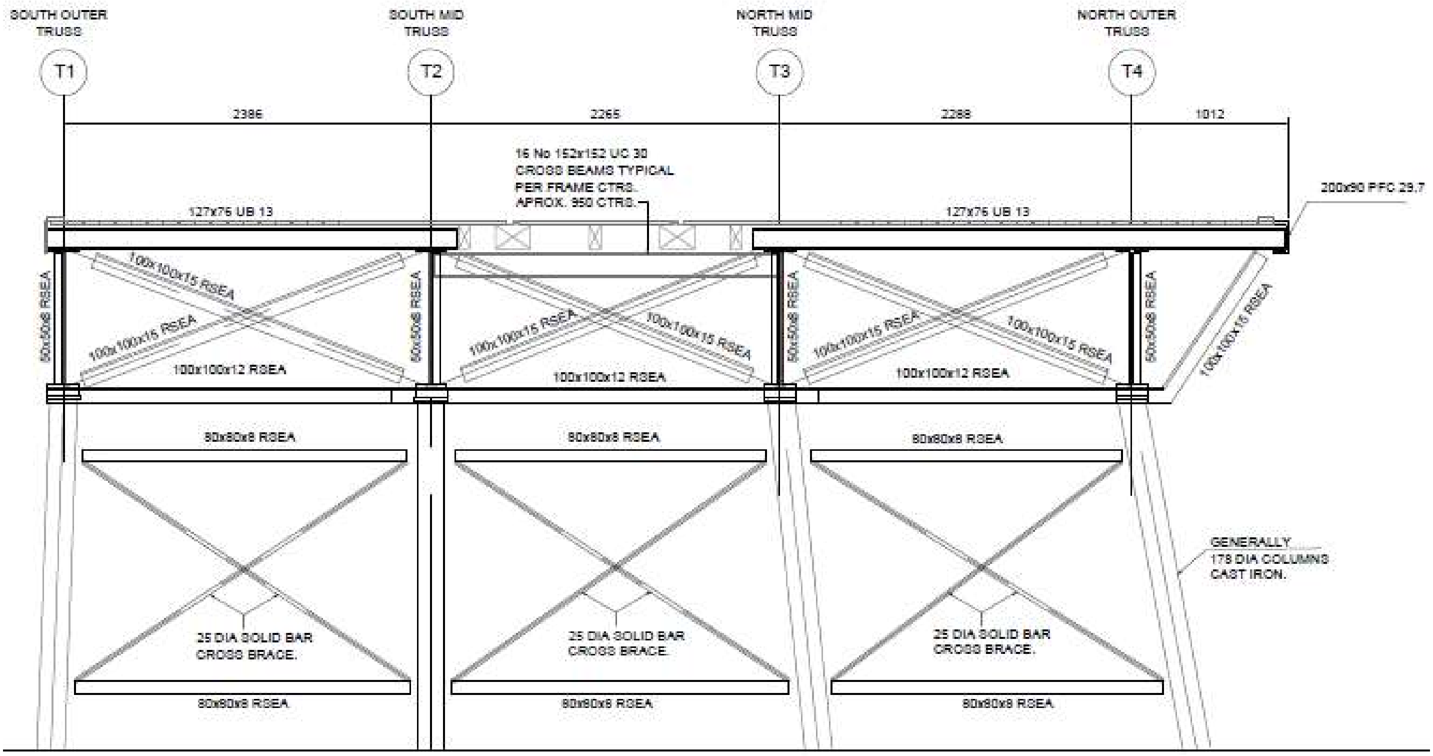


Figure 3 – Typical frame elevation

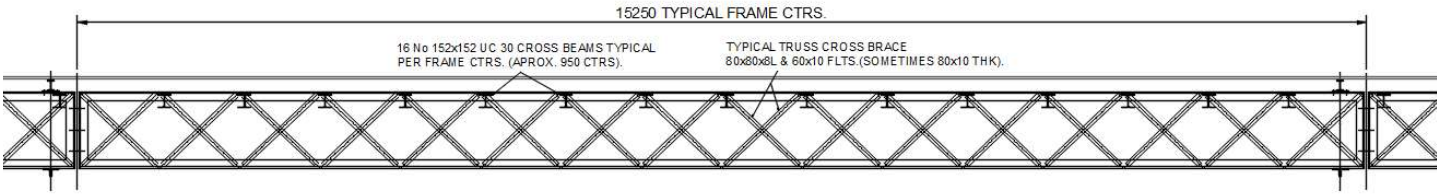


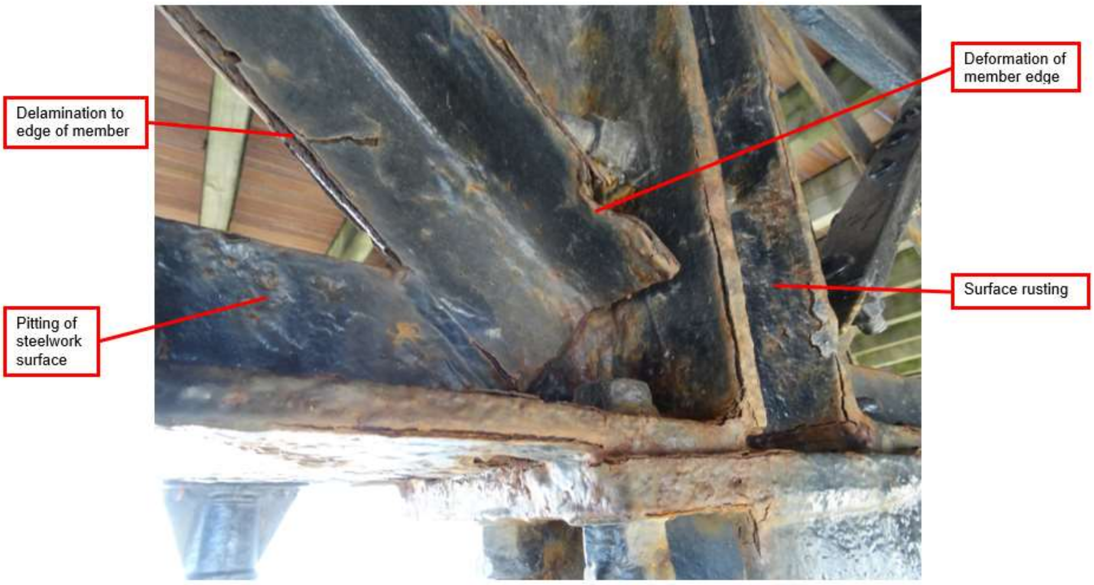
Figure 4 – Typical elevation on longitudinal truss

2.6 No intrusive or destructive testing has been carried out on the steelwork to determine the grade or composition of the materials used, there has been localised removal of loose paint and corrosion only. Photograph 4 below shows a typical support frame (as Figure 4) adjacent to the access road into the leisure park. Further investigation is required for the existing cast iron columns to determine if any cracking is present and depth of installation, foundation size, form and bearing strata to allow comment on the frame's capacity.

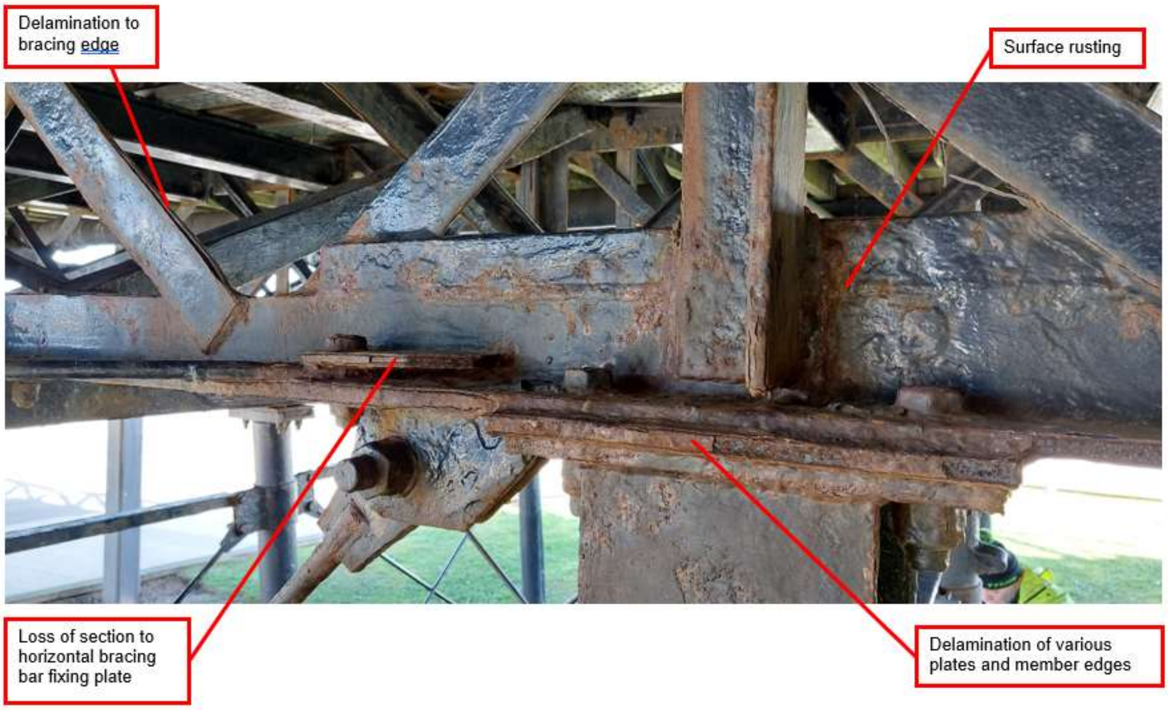
Photograph 4 – Typical support frame.

2.7 Due to the unevenness of the painted surface of the steel members, an accurate record of the original size was difficult to obtain. As no historical drawings were available at the time of the survey, Thomasons used digital callipers and NDT Systems – TG110-DL Ultrasonic Thickness Gauge to determine the size and thicknesses of the steelwork to relate to similar sections within the appropriate historical literature, (for ease the sections sizes are noted in metric equivalents) following which as assessment of any loss of section could be evaluated.

2.8 Examples of the typical defects found along the steel frame are shown below with the photo record sheets for each surveyed location included within Appendices C-I. Each of the areas inspected has a small selection of the photographs taken included to provide a snapshot of the defects observed.



Photograph 5 – Typical defects found along steel frame



Photograph 6 – Typical defects found along steel frame

2.9 Typically, the extent of any corrosion and delamination is noticeable at the edges of the steel members highlighted in Photographs 5 and 6 above. The thickness of the steelwork was difficult to ascertain due to the uneven surfaces, previously removed rust and pitting, and the paint thickness and unevenness, due to the paint and any corrosion under the paint, along the member.

2.10 The main issues with the steelwork observed in areas 1 - 6 inclusive were:

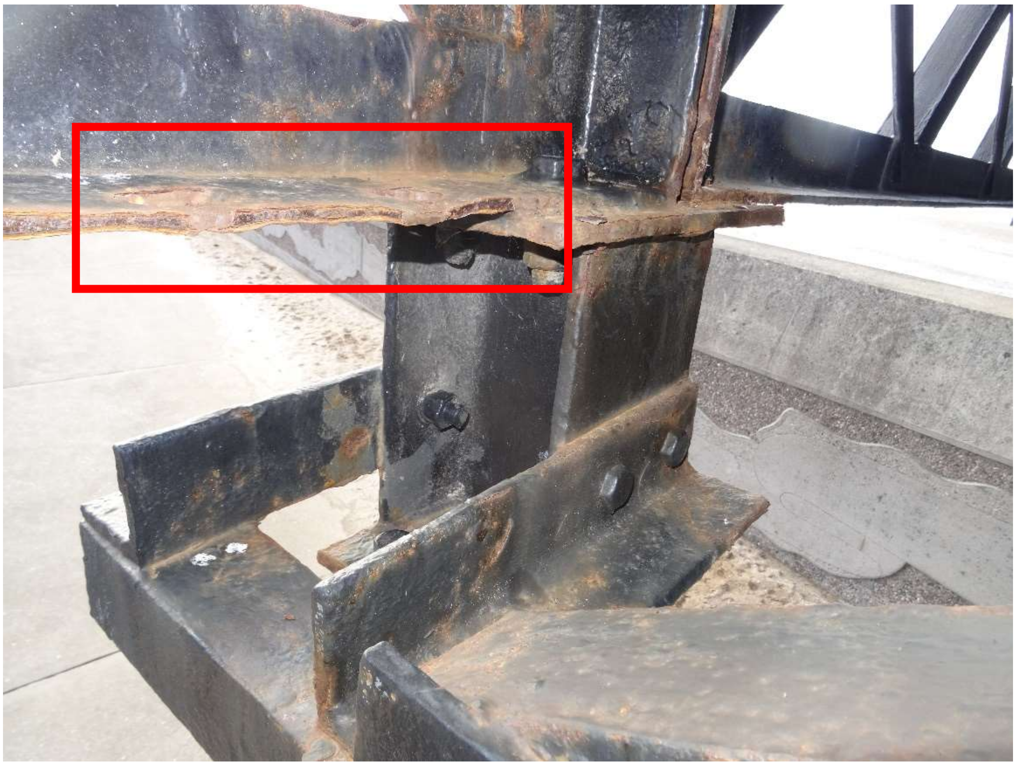
- Loss of protective finishes leading to surface rusting/pitting
- Loss of protective finishes leading to significant rusting, delamination and loss of section material
- Damaged, deformed and buckled member edges
- Broken and damaged bracing to main frames

2.11 Examples of the issues listed above can be seen in photographs 7-11

Photograph 7 – Exposed member edge and pitting to vertical web.

Photograph 8 – Rust present to the underside and around the bolt holes of the truss member.

Photograph 9 – Uneven paint application, and hole in web highlighted.



Photograph 10 – Buckling/deformation of member edge, highlighted for clarity.

Photograph 11 – Delamination to top and underside of flange and indication of varying thickness in section.

2.12 These defects were noted as typical to all areas of the survey. Additional defects are included under the following headed sections relating to the Areas surveyed.

2.13 Area 1: Steel Frame adjacent to lake on Funland side

2.14 Photograph 12 shows the column head that supports the incoming South Mid trusses, it highlights some of the typical defects that were observed to this frame.

Photograph 12 – Trusses on support post, indicating defects present.

2.15 The steelwork directly under the timber decking had the most significant areas of localised damage including reduction in section size, delamination and rusting. Photograph 13 shows an example of one of these beams this is a typical and reoccurring issue with numerous of other examples observed in other areas along the pier. The beam shown is a 127mm deep beam that spans between the top of the South Outer and South Mid trusses, and similarly between the top North Mid and Outer trusses. It would appear as part of the replacement boards this beam has been raised off the steelwork below and now supported off timber packing or has been a redundant members re-used in this area.

Photograph 13 – Damaged 127UB located to underside of new decking

2.16 The table below gives an indicative view on various measured items, and what section loss percentage was noted in localised areas of the beam or brace.

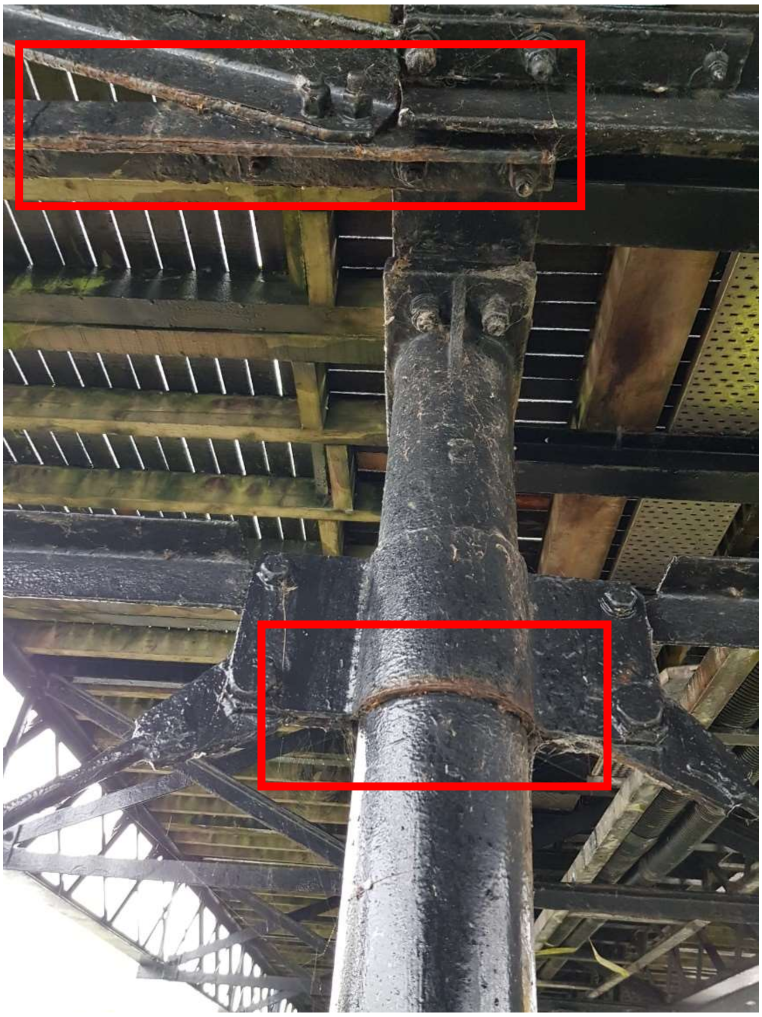
Element and measured section size (Width x Depth x Flange thickness)	Measured thickness – worst case (mm)	Percentage loss (%)
Transverse Frame – Bottom Member – 100x100x16 RSA	7.9	52
Transverse Frame – Bracing Angle – 100x100x16 RSA	9.0	42
Outer North Longitudinal Truss Bottom Chord – 205x95x16.2 TEE	9.13	44
Outer South Longitudinal Truss Bracing – 80x11.9 Flat	9.8	18

Table 1 – Selected Measured elements and Percentage section loss – Area 1

2.17 <u>Area 2: Steel Frame adjacent to West side of the lake:</u>

2.18 The frame in this area appears to be in better condition than other areas surveyed. Photograph 14 below, shows the frame adjacent to the lake edge, and the paintwork appears untarnished. Closer inspection revealed localised delamination to the bottom edge of the high-level bracing collars, and to the underside of the transverse frame angles, this is evidenced in Photograph 15. We cannot confirm if this section was replaced as part of previous remediation works.

Photograph 14 – View on support frame in Area 2, and incoming Trusses



Photograph 15 – Highlighting delamination to underside of members and bottom edge of the bracing collar

2.19 The table below gives an indicative view on various measured items, and what section loss percentage was noted in localised areas of the beam or brace.

Element and measured section size (Width x Depth x Flange thickness)	Measured thickness – worst case (mm)	Percentage loss (%)
Transverse Frame – Cross Bracing Bar – 25mm Dia.	13.6	46
Transverse Frame – Top Member – 127x76x8 UB Bottom Flange	4.2	48
Transverse Frame – Bottom Member – 100x100x16.3 RSA	11	32
Outer North Longitudinal Truss Vertical Angle – 50x50x9.3 RSA	5	46
Outer South Longitudinal Truss Bottom Chord – 205x102x12 TEE	10	17

Table 2 – Selected Measured elements and Percentage section loss – Area 2

2.20 <u>Area 3: Steel Frame under Light Frame No. 10, adjacent to leisure park access road:</u>

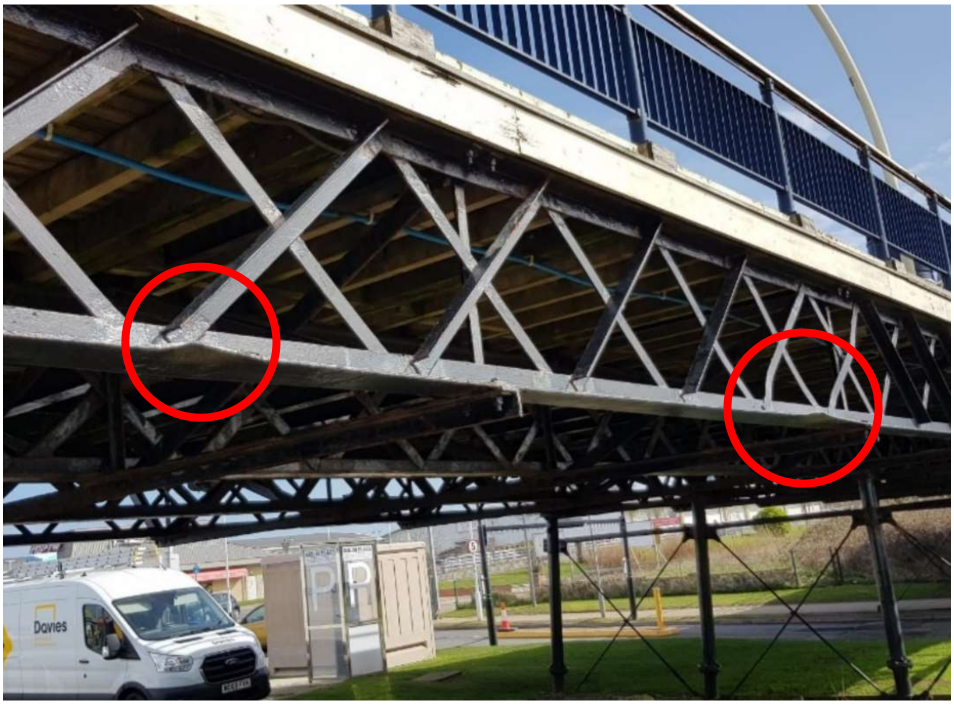
2.21 Photograph 16 shows a transverse frame adjacent to the survey area with highlighted evidence of delamination and rusting to the members. The paint on the underside of the rail support beam is peeling and delamination to the flange edges in this area are highlighted in blue.

Photograph 16 – A transverse frame adjacent to the support frame in the survey area

2.22 Some of the 152mm deep cross beams spanning between the Mid trusses, have visible evidence of corrosion, rusting and delamination to the top flange. In some locations, as shown in Photograph 17, additional corrosion localised around existing holes in the bottom flange is present.

Photograph 17 – Damaged 152UC located to underside rail support beams

2.23 Photograph 18 highlights where the base of the South Outer longitudinal truss may have been impacted causing the localised buckling of the bottom flange. Similarly this may have occurred during propping of the trusses during previous inspections/remedial works.



Photograph 18 – Impact damage (highlighted) to the underside of the South Outer Truss

2.24 The table below gives an indicative view on various measured items, and what section loss percentage was noted in localised areas of the beam or brace.

Element and measured section size (Width x Depth x Flange thickness)	Measured thickness – worst case (mm)	Percentage loss (%)
Transverse Frame – Bottom Member – 100x100x12 RSA	10.2 both Legs	15
Transverse Frame – Top Member – 127x76x8 RSA	5.8	29
Outer South Longitudinal Truss Top Chord – 205x95x13.5 TEE	10.1	25
Mid North Longitudinal Truss Bottom Chord – 205x95x14.6 TEE	12.8	12

Table 3 – Selected Measured elements and Percentage section loss – Area 3

2.25 <u>Area 4: Steel Frame under the Donut Shop:</u>

2.26 To the underside of the Donut shop, at approximately the midpoint along the pier, there is a double frame configuration. This provides cross bracing between the support frame posts in two directions, and horizontal cross bracing to the underside of the trusses. Photograph 19 provides a view from the side of this braced configuration.

Photograph 19 – View Braced Double Support Frame on left hand side

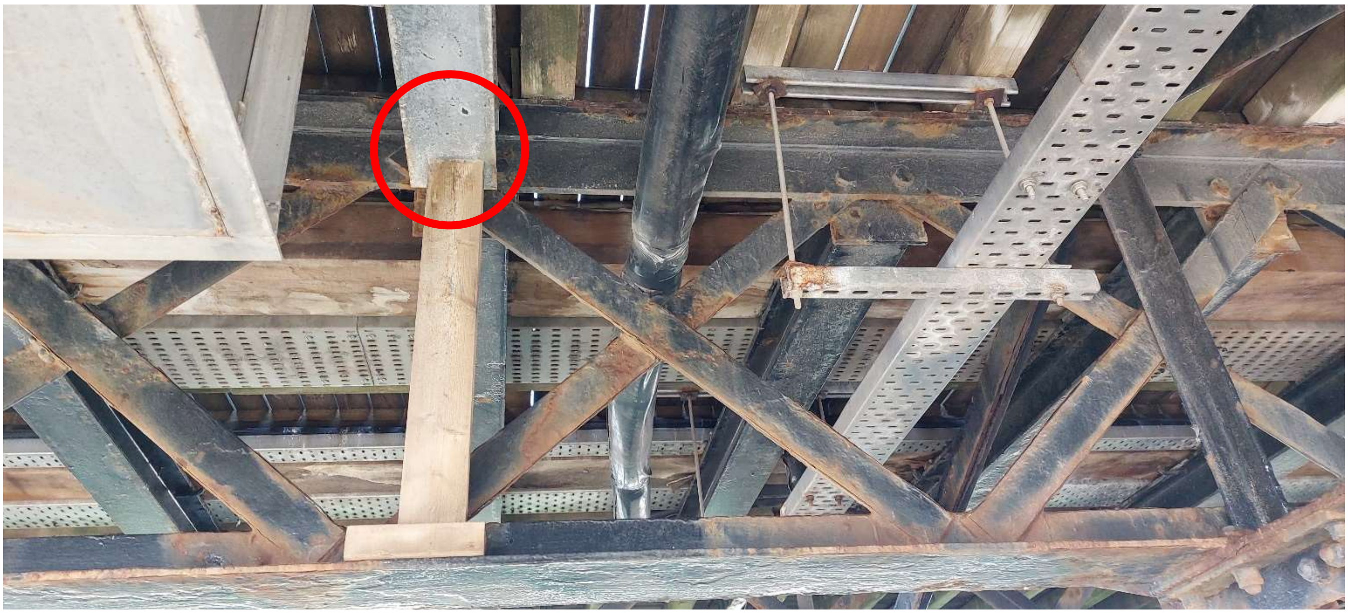
2.27 In Photograph 20 numerous defects, including the horizontal bracing bar connection plate thickness, highlight some of the similar problems observed to this area.

Photograph 20 – Various defects around surveyed frame

2.28 Photograph 21 indicates the severe rusting and corrosion around the welded connection between the diagonal bracing and vertical truss element.

Photograph 21 – Rusted welds between the bracing members and the vertical truss member

2.29 In the area directly below the Donut shop, mechanical equipment has been suspended from the underside of the deck. The supporting steelwork for this equipment has been post-fixed to the existing pier frame. One of these support beams, Photograph 22, has a single bolt connection in the highlighted area, the beam has been propped off the bottom boom of the truss with timber. It was noted that the beam has been drilled for a second bolt, Photograph 23, but not installed.



Photograph 22 – Steel beam with single bolt connection has been propped with timber

Photograph 23 – Propped steel beam with missing bolt

2.30 The table below gives an indicative view on various measured items, and what section loss percentage was noted in localised areas of the beam or brace.

Element and measured section size (Width x Depth x Flange thickness)	Measured thickness – worst case (mm)	Percentage loss (%)
Transverse Frame – Bottom Member – 100x100x16 RSA	11.2	31
Transverse Frame – Bracing Angle – 100x100x16 RSA	8.2	44
Outer North Longitudinal Truss Bottom Chord – 203x102x13.6 TEE	6	56
Mid North Longitudinal Truss Bracing Member – 80x11.0 TEE	7.9	28
Mid South Longitudinal Truss Bottom Chord – 203x102x13.6 TEE	7.5	45

Table 4 – Measured elements and Percentage section loss – Area 4

2.31 <u>Area 5 – Frame adjacent to the East side of Marine Drive Bridge:</u>

2.32 Due to restricted access, Inspection of the bridge deck steelwork was limited to the support frames either side of Marine Drive. Thomasons were advised by Sefton BC that the bridge and its supporting structure are maintained by the Local Authority Highways Department and does not form part of this report. The observation noted in photograph 24 is for information purposes only.

2.33 In order to raise the headroom on Marine Drive, the original pier structure was removed and replaced with the bridge. It was observed that there are plates protruding below the underside of the bridge deck members that have been struck by high vehicles, these plates have been highlighted in Photograph 24.

Photograph 24 – View on bridge over Marine Drive, highlighting damaged plates to underside

2.34 From a cursory review of the bridge framing above and below, the trusses spanning over the road could be seen to be corroding in areas of peeling paintwork, and examples can be seen in the photographs below. The columns either side of the bridge also had similar visible corrosion of the steelwork within holes in the paintwork.

Photograph 25 – Peeling paint and visible corrosion of frame highlighted on bridge truss member

Photograph 26 – Peeling paint and visible corrosion of frame highlighted on bridge truss member

Photograph 27 – Peeling paint and visible corrosion of frame highlighted on bridge truss member

Photograph 28 – Peeling paint and visible corrosion of frame highlighted on bridge truss members

Photograph 29 – Peeling paint and visible corrosion of frame highlighted on bridge truss column

2.35 The paintwork and steelwork to the underside of the bridge is in poor condition, with significant corrosion and delamination present, this can be seen in Photographs 30 and 31 below.

Photograph 30 – Condition of steelwork to the underside of Marine Drive bridge.

Photograph 31 – Peeling paintwork and delamination of steel members

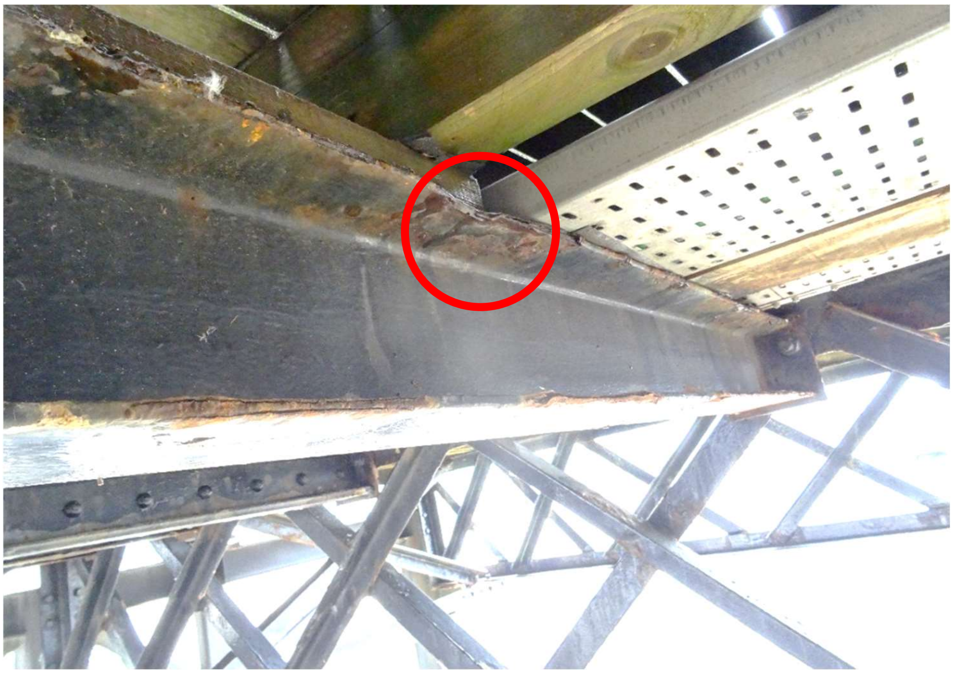
2.36 Area 6 – Frame adjacent to the West side of Marine Drive Bridge:

2.37 The inspection of the steel frame to the western side of Marine Drive, close to the sea wall, highlighted corrosion and delamination of the bracing members and high level post connection collars.

2.38 In Photograph 32, the horizontal leg of the angle, and the single bolt fixing to the collar bracket utilises the vertical leg. The notching of angles forming the horizontal element of the bracing systems at the support post locations, was a typical detail noted in similar fixing locations along the pier.

Photograph 32 – Brace, connection and collar to post are delaminated

2.39 Within the steel frame spanning over the sea wall, the rail support beams have corroded and as highlighted in Photograph 33 below, part of the top flange has eroded with approximately 15% loss of section width. The beam has a varying flange thickness due to delamination.



Photograph 33 – Damaged rail support beam

2.40 The steel frame in this area is in poor condition, with a significant amount of delamination noted, and numerous beams and truss elements with varying loss of section.

2.41 The table below gives an indicative view on various measured items, and what section loss percentage was noted in localised areas of the beam or brace.

Element and measured section size (Width x Depth x Flange thickness)	Measured thickness – worst case (mm)	Percentage loss (%)
Transverse Frame – Bottom Member – 100x100x16 RSA	8.2	51
Transverse Frame – Bracing Angle – 100x100x16 RSA	7	58
Transverse Frame – Bracing Bar – 17.6mm Dia.	15.6	11

Table 5 – Measured elements and Percentage section loss – Area 6

2.42 Section of Pier over the beach:

2.43 On the beach side of the pier, the support frames are naturally higher and thus require a double braced configuration shown in Photograph 34 and Figure 5.

Photograph 34 – View on typical support frame along the beach

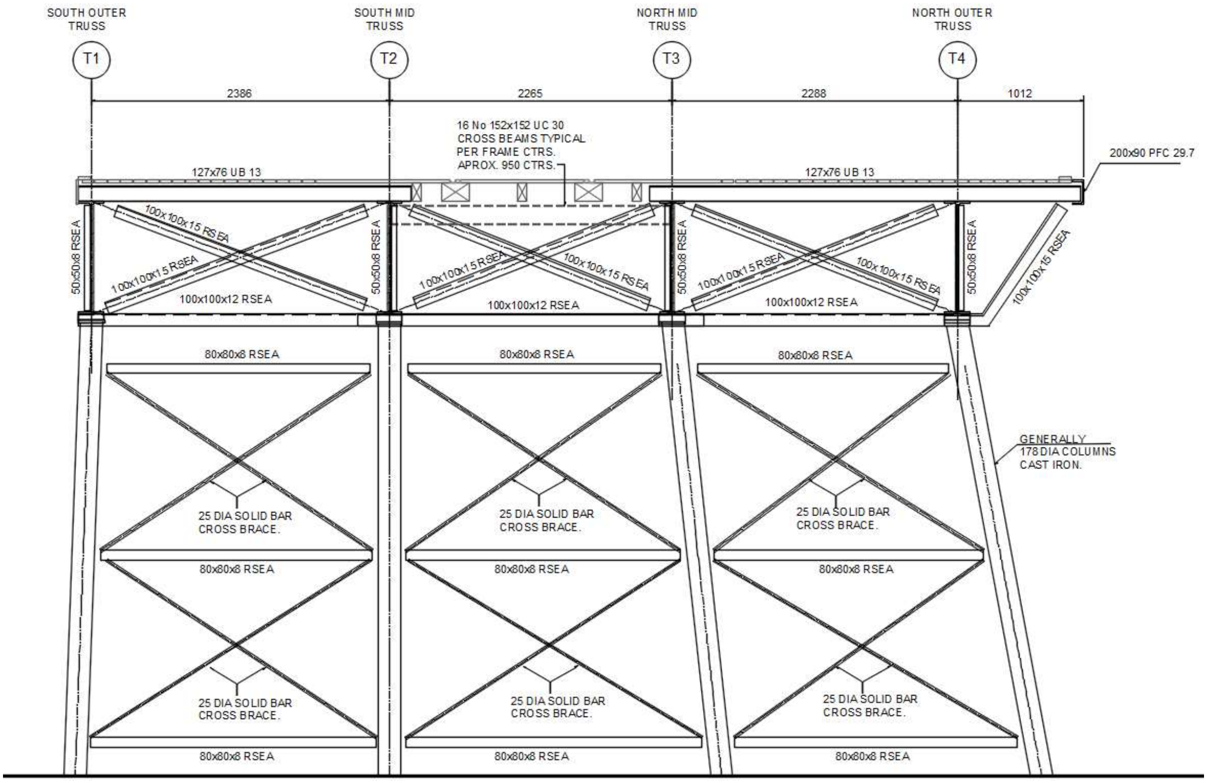


Figure 5 – Typical elevation on beach support frame

2.44 There are similar issues noted to other parts of the structure and this was apparent once decking boards were lifted and the steelwork exposed for review from above. Examples of the exposed beams can be seen in photographs 35 and 36.

Photograph 35 – Detail showing varying section thickness and corrosion to member.

Photograph 36 – Example of delamination to member surfaces and extent of corrosion.

2.45 Bracing within the support frames along the beach section of the pier has significant damage in numerous locations. Damage includes snapped and/or bent bracing bars. In some areas, the bracing turn buckles have sheared, evidence of this can be seen in photographs 37 and 38.

Photograph 37 – View on frame with bent and broken bracing bars

Photograph 38 – Sheared bracing turn buckle.

2.46 Some of the damage to the beach frames has been caused by the ebb and flow of the tide. Other areas, especially closer to the sea wall, have likely been damaged by members of the public climbing on the frames. This is highlighted by the twisted angle to the base of the frame in Photograph 39.

Photograph 39 – Support frame damaged braced member (arrow indicates direction of twist in member)

2.47 Corrosion of the exposed structural elements at the base of the columns was observed. The corrosion, seen in photograph 40, encompasses the perimeter of the cast iron column, and adjacent steel bracing members at this level. This damage is likely as a result of these members being under repetitive submersion by sea water and any movement in the sands that would also conceal the member.

Photograph 40 – Corrosion to base of seaward pier columns

2.48 No testing was carried out on the iron columns, these will need further investigation as part of the remediation scheme to clarify the structural condition, e.g., any internal cracking not externally visible.

2.49 The transverse frames that span across the pier deck have a 127mm deep beam that spans between the top flange of the outer and mid trusses either side of the central section. In Photographs 41 and 42, the extent of the corrosion to these members is so severe, there is delamination to all exposed surfaces of the beam making it difficult to determine what structural capacity remains.

Photograph 41 – View on elevation of damaged 127mm deep Beam

Photograph 42 – View on end of damaged beam in Photograph 36

2.50 The steelwork supporting the central section of the deck under the existing train rails and the oak beams, was inspected in numerous locations and there was severe damage present. The beams, 152mm deep column sections, can be seen to have corroded and delaminated to such an extent that there is significant loss of the beam flange, there is no structural capacity remaining, this can be seen in Photographs 43 and 44. These will need to be replaced during the next phase of works.

Photograph 43 – Severe delamination of the top flange there is no remaining steel section

Photograph 44 – Holes caused by delamination and corrosion in the top flange of the steel beam

2.51 A visual survey to the underside of the pier over the beach, recorded similar damage at a number of locations. The true extent of damage to the beams directly under the timber decking is difficult to quantify at this time and would require lifting of the decking along the full length of pier.

2.52 An issue observed at the base of the columns along the beach section was corrosion of the exposed structural elements. The corrosion, seen in photograph 45, encompasses the perimeter of the cast iron column, and adjacent steel bracing members at this level. This damage will likely be caused by these members under repetitive submersion by sea water and any movement in the sands that would also conceal the member.

Photograph 45 – Corrosion to base of seaward pier columns

2.53 No testing was carried out on the iron columns, these will need further investigation as part of the remediation scheme to clarify the structural condition, e.g. any internal cracking not externally visible.

2.54 <u>Corrosion of elements:</u>

2.55 Observations to the steelwork, previously indicated, note significant corrosion to the members, and around the connections. An explanation for this would be that the steelwork was not completely cleaned of all contaminants during the previous refurbishment and corrosion has re-appeared in these areas. Any voids or cracks in the paintwork will allow ingress of salt via the sea water at high tide or moisture in the sea air. This leads to a cyclic issue of damage due to increase in size of void, increases the exposure to the elements and allows greater ingress of contaminants.

2.56 Due to the marine environment, the risk of corrosion would be classed as C4 High or C5 Very High s taken from the table in figure 6 below. Clarification of the salinity in the area around the pier would allow confirmation of the corrosive category for the next phase of the remedial works.

Atmospheric corrosivity categories and examples of typical environments (BS EN ISO 12944-2[1])

Corrosivity category	Low-carbon steel Thickness loss (µm)[a]	Examples of typical environments (informative only)	
		Exterior	Interior
C1 very low	≤ 1.3	-	Heated buildings with clean atmospheres, e.g. offices, shops, schools, hotels
C2 low	> 1.3 to 25	Atmospheres with low level of pollution: mostly rural areas	Unheated buildings where condensation can occur, e.g. depots, sports halls
C3 medium	> 25 to 50	Urban and industrial atmospheres, moderate sulphur dioxide pollution; coastal area with low salinity	Production rooms with high humidity and some air pollution, e.g. food-processing plants, laundries, breweries, dairies
C4 high	> 50 to 80	Industrial areas and coastal areas with moderate salinity	Chemical plants, swimming pools, coastal ship and boatyards
C5 very high	> 80 to 200	Industrial areas with high humidity and aggressive atmosphere and coastal areas with high salinity	Buildings or areas with almost permanent condensation and high pollution
CX extreme	> 200 to 700	Offshore areas with high salinity and industrial areas with extreme humidity and aggressive atmosphere and sub-tropical and tropical atmospheres	Industrial areas with extreme humidity and aggressive atmosphere

Notes:

- 1µm (1 micron) = 0.001mm
- [a] The thickness loss values are after the first year of exposure. Losses may reduce over subsequent years.
- The loss values used for the corrosivity categories are identical to those given in BS EN ISO 9223[2].

Figure 6 – Classification of atmospheric corrosivity taken from BS EN ISO 12944-2

2.57 Based on the C4 and C5 categories, and a current lifespan of approximately 20 years, this would mean a loss of section for C4 category of between 1-1.6mm during this time frame, and for C5 category a loss of 1.6-4mm. From the measurements taken during the survey, reduction in section thickness in the transverse frame bracing angles (100x100x15RSA's) varies between 23-41% loss, which equates to 3-5mm. In some of the longitudinal frames, the 15mm thick flange has reduced down to approximately 6mm in locations, meaning a loss in section of 56%. The values for reduction in section thickness are higher than the typical thickness loss values given in Figure 6.

2.58 Subject to the specification of the coating applied during the refurbishment, Figure 7 gives an indication of the coating life for C4 and C5 risk categories. This gives a typical coating life of between 15-20 years. This means that the coating is approaching/past the expected lifespan and needs re-applying as soon as possible. From historical photographs online and streetview on Google Maps, the corrosion can be seen to start appearing on the steelwork either side of Marine Drive in approximately 2014. This likely due to inadequate cleaning/painting of the steelwork during the refurbishment in early 2000's.

Standard systems for C4 – High risk environment category

System number	E-C4-A	E-C4-B	E-C4-C
Coating life	15-20	20	20+
Nearest equivalent BS EN ISO 12944-5[3]	C4.11	-	-
Surface preparation to BS EN ISO 8501-1[4]	Blast clean to Sa 2½	Blast clean to Sa 2½	Blast clean to Sa 2½
Factory applied coatings	i) Zinc rich epoxy primer 40µm (note 6) ii) High build epoxy MIO 100µm	i) Zinc phosphate epoxy primer 80µm ii) High build glass flake epoxy 300µm	i) Zinc phosphate epoxy primer 25µm (note 7) ii) Elastomeric urethane 1000µm (note 8)
Site applied coatings	High build epoxy MIO 100µm (notes 4 & 5)	Recoatable polyurethane finish 60µm	Recoatable polyurethane finish 60µm

Standard systems for C5 – Very high risk environment category

System number	E-C5-A	E-C5-B	E-C5-C
Coating life	15	20	15
Nearest equivalent BS EN ISO 12944-5[3]	TSM5.01	C5.08	G5.04
Surface preparation to BS EN ISO 8501-1[4]	Blast clean to Sa 3	Blast clean to Sa 2½	-
Factory applied coatings	i) Sprayed aluminium to BS EN ISO 2063[8][9] 150µm (note 9) ii) Zinc phosphate epoxy sealer coat 50µm iii) High build epoxy MIO 100µm (note 4)	i) Zinc rich epoxy primer 40µm (note 6) ii) High build epoxy MIO 200µm total (one or two coats) (note 4)	i) Hot-dip galvanize to BS EN ISO 1461[5] (note 1) ii) Mordant wash iii) Etch primer 40µm iv) High build epoxy MIO 100µm (note 4)
Site applied coatings	Recoatable polyurethane finish 60µm	High solid aliphatic polyurethane finish 60µm	Recoatable polyurethane finish 60µm

Figure 7 – Standard protection systems for C4 & C5 categories from BS EN ISO 14713

3.0 CONCLUSIONS

3.1 Following the limited visual inspection of the six identified areas Thomasons conclude there is a significant amount of remedial works required to the six areas of the pier steelwork, it should be noted these areas represent less than 5% of the whole steel structure of the pier and are representative of most locations.

3.2 Consideration to the appropriate remedial options need to be discussed and assessed further by the stakeholders in order to clarify the sequencing pro’s and con’s, financial implications and operation of the pier. These need to be considered alongside the timber replacement proposals and in particular the ongoing use of the commuter train ride, which as noted in the timber deck report had a significant impact on the lifespan of the previous deck replacement work.

3.3 Thomasons further conclude that as part of any remedial works the road bridge over Marine Drive requires significant remedial works for it to be suitable for its intended use as a foot bridge but also the motorised commuter train to the end of the pier. Thomasons have provided comments on a very limited visual inspection and some of the bridge members are exhibiting unsuitable section loss to primary members.

3.4 It is further concluded that if the pier was limited to pedestrian access only then some of the required remedial measures may be left and form part of an ongoing maintenance strategy over the forthcoming 5 year period.

3.5 In reviewing the finidings observed within this report and a comparison to the Mott MacDonald report of 2014 it was observed that the areas of defect have still continued to deteriorate despite the remedial works undertaken and that they appear to be on an exponential increase in decline.

3.6 Following any remediation works to the pier, Thomasons would recommend a stringent maintenance strategy be set up to ensure the steelwork is inspected regularly and any defects in the paintwork or steelwork be rectified at an earlier stage to reduce the likelihood of a large pier refurbishment re-occurring. This should include a yearly walk over and photographic inspection of the pier from above and below, a 3 – 5 yearly inspection similar to the reported inspection including lifting of boarding to inspection timber to timber and timber to steel interfaces.

4.0 RECOMMENDATIONS

4.1 Due to the amount of corrosion, delamination and loss of section observed to the steelwork, Thomasons recommend consideration is given to a hierarchy of repairs for implementation to any remediation scheme, 1 being immediate. Items 1 – 3 must be completed prior to the pier being opened to the public.

1 – All steelwork in direct contact or providing direct support to timber elements

2 – All steelwork that exhibits a loss of strength/section greater than 15%

3 – All critical items of structure, bracing, principal members, stability members and members forming part of public safety – handrails, stairs.

4 – All steelwork that has a utilisation ratio of greater than 1.25 under full working load capacity but loaded area can be restricted to 60%.

5 All other steelwork can then be assessed for priority on a maintenance strategy but all works completed within a 5 year period.

4.2 Thomasons understand that previous remedial works have been undertaken to the cast iron column, no reports where available at the time of the inspection and Thomasons recommend that an appropriate further survey (X-Ray) is carried out to assess their current conditions prior to any remedial works commencing.

4.3 Due to the extent of work required Thomasons recommend that a phased programme is adopted that allows for one of the following sequence of work –

1 – One to Two bays (one bay being from overhead lamp to overhead lamp) have all the timber decking, joists, noggins and all other timbers removed

2 – All steelwork stripped of finishes with an immediate application of rust inhibiting paint or similar applied

3 – Engineer carries out a measured assessment of all exposed sections and provides annotated drawings of required remedial locations and repair details whilst second area of piers timbers are being removed

4 – Consideration should be given to repeating the above either in the following two bays, working back from Marine Drive or starting from Marine Drive towards the end of the pier.

This would then give three options to consider – 1st – Commence working from the start of the pier and complete all works to Marine drive allowing this section to opened to the public and the earliest available opportunity, or 2nd – Concentrate on the starting from Marine Drive complete all works to the end of the pier allowing this section to be opened to the public at the earliest available opportunity, or 3rd – having a number of teams to allow work to commence from the start of the pier towards Marine Drive and from seaward side of Marine Drive to the end of the pier to enable both sections to be worked on concurrently.

4.4 The extent of the remedial, replacement work will also be driven by the proposed sequence and budget constraints, due to the variable issues observed it is not possible at present to provide a definitive scope of repairs or quantities as until the existing decking is removed, together with an agreed scope of the piers future use and agreed allowable imposed loadings, can a full assessment of the members capacity and residual strength be determined.

When reviewing the remediation options for the pier, it is important to understand the difference between repair, strengthening and short-term temporary works.

Repairing - to restore the original element to its original strength, reinstating similar materials as per the original construction.
Strengthening – maintains the original section in-place while enhancing its capacity strength by adding supplemental elements (plates, angles or additional beams).
Short-term temporary works – those works necessary to provide a temporary solution whilst still maintaining the piers integrity – these works should have a permanent solution within 12-24months of being identified.

Due to the extent of the corrosion/delamination, where there is significant loss of section, i.e. over 50%, these elements would need to be replaced. For elements with between 15-50% loss of section, strengthening maybe suitable. This strengthening would need to be assessed after each member has been cleared of all delamination and rust, to determine the remaining section cross-section. This option would allow the main frame elements to remain in place, and any replacement works carried out over bay lengths.

Strengthening of the members requires fixing additional angles, plates or additional beams or PFC's to the existing steel members, this form will need to be agreed from Grade II Listing approach with engagement with Sefton BC Heritage Officers department. This approach would strengthen the frame and allow insitu works, without the need for removal of large portions of the frame. A full review of the revised steelwork configuration, and agreement on imposed loadings would be required to check the impact of additional weight is within acceptable limits for the cast iron columns.

A strengthening approach would be a simpler and cheaper solution for the diagonal bracing members as the plates could be welded to the inside face of the angle but again would need to be discussed/agreed with the heritage officer.

Acceptable and suitable strengthening of the main longitudinal truss members is considered more complicated due to the configuration of the cross-bracing members limiting the addition of strengthening elements while also affecting the piers appearance, the internal members can be plated and agreement just to strengthen weaken elements would again need agreement with the heritage officer to avoid having to add a sympathetic repair to all the members. Significant propping would also be required to replace members which would prove difficult to achieve over the lake and past Marine Drive over the sands.

5 The current condition of the pier has been impacted by various issues since the significant multi-million pound refurbishment works carried out in 1998-2002 which left Sefton Council with inherent latent defects, although a nominal compensation payment was received it was far too small to allow the required remedial works to be undertaken. Following a report in 2017 it was established that costs to carry out works to rectify the inherent defects and areas of deterioration created and escalated by those inherent defects was costed at £8.8m. At this time, it was decided to adopt a 'maintenance painting' solution at the cost of £1.5m while continuing maintaining and repairing defects as they materialised. This has continued to the present day when the extent of the deck deterioration and numerous areas of the structure have reached a point where the pier is now deemed unsafe for the public to be allowed access.

APPENDIX A

Formby Surveys Topographical Survey dated August 2022

APPENDIX B

Mott MacDonald Feasibility Study for Programme of Major Maintenance Works dated December 2014

APPENDIX C

Thomasons Photo Record Sheet – Area 1

APPENDIX D

Thomasons Photo Record Sheet – Area 2

APPENDIX E

Thomasons Photo Record Sheet – Area 3

APPENDIX F

Thomasons Photo Record Sheet – Area 4

APPENDIX G

Thomasons Photo Record Sheet – Area 5

APPENDIX H

Thomasons Photo Record Sheet – Area 6

APPENDIX I

Thomasons Photo Record Sheet – Beach Section