HOP
CONSULTING CIVIL AND
STRUCTURAL ENGINEERS

MADEIRA TERRACES REFURBISHMENT, BRIGHTON

IRON REUSE STRATEGY REPORT

31 OCTOBER 2023

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1.0 INTRODUCTION

1.1 General

1.1.1 Instructions were received from Brighton and Hove City Council (B&HCC) to provide Civil and Structural Engineering services to inform the strategy for iron reuse associated with the refurbishment of Madeira Terraces, Brighton (The Terraces).

1.1.2 This report was prepared by Nigel Hosker, a Chartered Civil Engineer, Director at HOP Consulting Ltd (HOP), Fellow of the Institution of Civil Engineers (FICE) and a member of the ICE Southern Maritime Committee. I confirm I have over 20 years in the industry and have particular experience in coastal buildings and dealing with large Victorian iron structures..

1.1.3 I have visited, inspected and been involved with a number of maintenance campaigns to The Terraces in the past and have visited the proposed site a number of times in 2022/23.

1.1.4 Archive information associated with The Terraces has been provided by the owner; Brighton and Hove City Council. I have reviewed and make reference to various historical information throughout this report. Much of this information dates back to The Terraces original construction as well as periodic inspections and surveys throughout its service. Due to the volume of material available it is inappropriate to reproduce the entire archive in this report. I have however identified and reproduced what is considered to be the most relevant and focussed information in Appendices referred.

1.1.5 The scope of this report is limited to iron elements that contribute to structural stability. Decorative components such as cover plates, rainwater goods and masks etc are to be dealt with separately. The existing jack arch deck has planning approval to be replaced in reinforced concrete and is outside the scope of this report.

1.1.6 No other aspects of the site have been investigated by HOP. Similarly, no detailed inspections of any concealed voids, floor voids, foundations and the like have been carried out. The results of this appraisal do not therefore constitute the results of a full structural survey. We have not inspected woodwork or other parts of the structure that are covered, unexposed or inaccessible and are therefore unable to report that any such part of the structure is free from defect.

1.1.7 This report is intended to be updated as and when more information becomes available as the project progresses. Due to the nature of the dangerous structure, restrictions on the ability to practically remove and inspect the various components in detail and the need to conduct Non-Destructive Testing (NDT) the approach has been to collate as much background information and provide preliminary conclusions on the best way forward. These preliminary conclusions should be validated as the project progresses and more information becomes available following processing and inspecting iron elements.

1.2 Background

1.2.1 It is assumed the reader is familiar with the particulars of The Terraces and as such these are not described in any great detail. Reference is made to the current Listed Building & Planning Consents which, at the time of writing, had been decided with conditions associated with cast iron reuse...

1.2.2 Briefly, The Terraces were originally constructed in two phases; with Phase 1 constructed circa 1888 and comprising of the section East of Royal Crescent Steps. This initial section was later extended to the west, circa 1894. There are subtle differences in the detailing of the two original phases which deviate slightly in elements such as rainwater goods, column detailing, surfacing and the like. The primary structural arrangement is however broadly similar. It is not clear if the same foundry was used either throughout or between phases hence some variability of original ironwork may be expected.

1.2.3 The Terraces were closed to the public circa 2012 due to structural safety concerns following a period of demise, The Terraces have remained closed since.

1.2.4 The existing deck structure is formed from short span transverse spanning jack arches formed typically in unreinforced mass concrete. Jack arches span between nominally 4.8m longitudinal spanning ‘I’ section beams. It is understood that the original jack arch beams were formed in wrought iron, however there is some variation in construction either dating from the original construction or and later episodes of interventions where some original wrought iron beams have been replaced with more modern steel sections and some jack arches recast with reinforced concrete entirely.

1.2.5 Jack arch beams are supported on trusses (or lattice beams) formed in grey cast iron spanning transversely. These lattice beams were formed in a single casting with a nominal span of 7.3m. The northern bearing is built into the Madeira Drive retaining wall and the southern bearing is supported on ornate cast iron columns.

1.2.6 The southern elevation of the structure is partially infilled with ornate and intricate spandrel panels formed in mirrored pairs with a ‘key’ connecting the two panels and form an arch or portal frame.

1.2.7 At eaves level, the southernmost jack arch is received by a cast iron edge beam which is concealed from public view by decorative rainwater goods and cover plates.

1.2.8 Above deck level is a cast iron balustrade, again formed in Grey Cast Iron and provides edge protection. This departs from current standards and is low, climbable and is unlikely to have the containment capacity to resist modern crowd loading requirements.

1.2.9 Importantly all original cast iron members are understood to be formed in Grey Cast Iron which was commonly specified at the time of original construction.

1.2.10 Samples from cast iron columns, were taken for testing to ascertain metallurgical and mechanical properties, including tensile strength in the 1980’s. Laboratory reports are available in Appendix B.

1.2.11 HOP Consulting Ltd were commissioned in 2001 to assess Cast Iron railings to the entire seafront frontage as well as to direct materials testing to other areas of seafront railings. These date from a similar period and are formed in similar materials. These older tests have been updated with site specific tensile test results undertaken in 2023. The population of tensile test results is however small and variable currently. It is considered current testing results are likely representative of the cast iron at The Terraces; this should be verified through supplementary, site-specific testing, as and when elements are available for close inspection in due course of this project. It is noted that no tensile testing of trusses has at the time of writing been possible. These could potential be formed from a higher grade of iron hence this should be confirmed.

1.2.12 A sketch of The Terraces is provided in Figure 1.

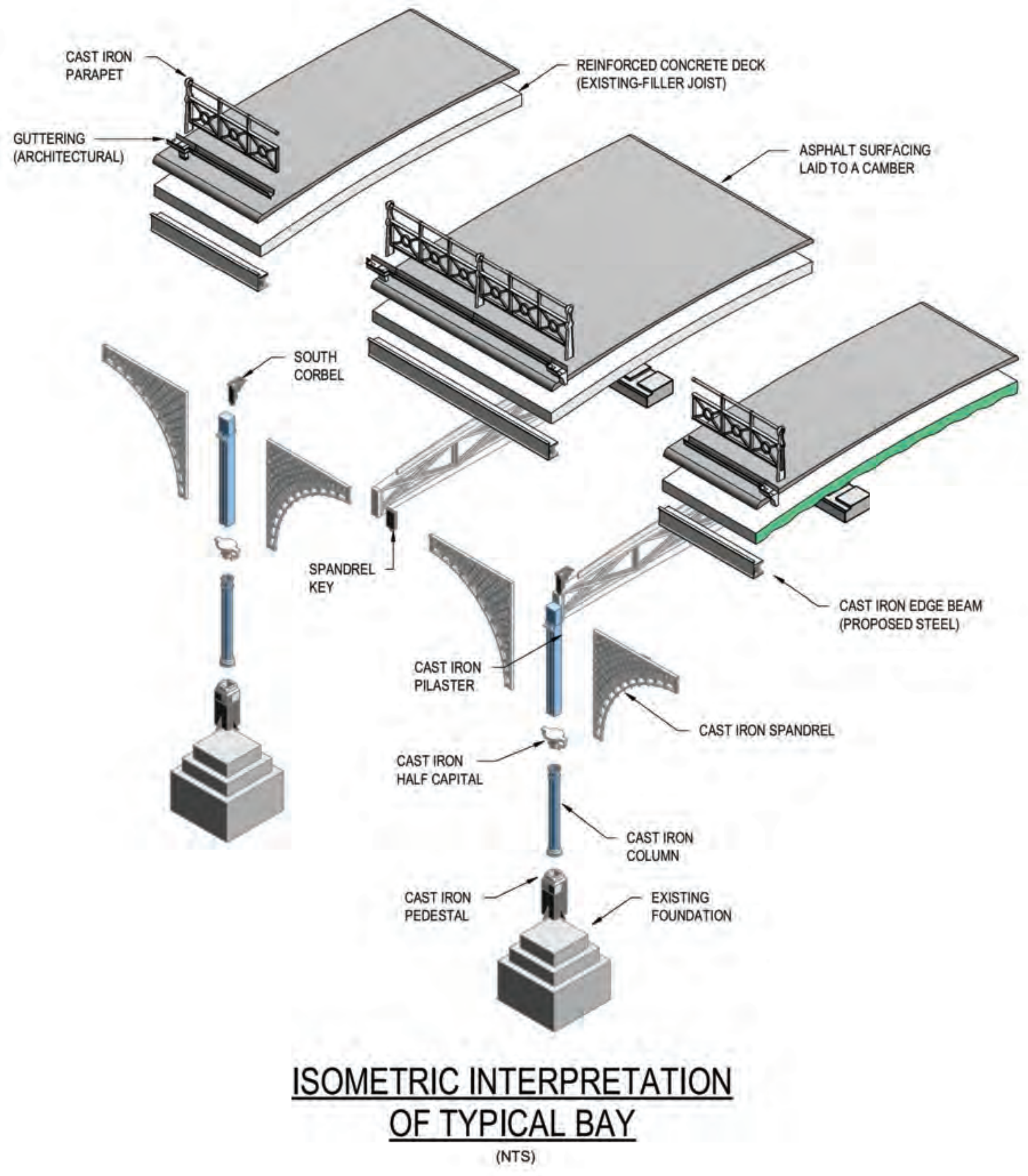


Figure 1 – Isometric Sketch of a Typical Bay indicating components and articulation

2.0 APPRAISAL OF CAST IRON STRUCTURES

2.1 General

2.1.1 The Steel Construction Institute (SCI) Publication 138 '*Appraisal of Existing Iron and Steel Structures' (SCI 138)* provides comprehensive guidance on the appraisal of cast iron structures. This guidance is well respected in the industry and is referred to throughout this report. A short abridgement of the core processes, technical aspects and guidance of upgrading and restoring structural capacity follows.

2.2 Cast Iron Material Properties

2.2.1 Cast iron has good compressive strength characteristics, good corrosion resistance and can be moulded to achieve intricate shapes.

2.2.2 The principal disadvantages of Grey Cast Iron are low tensile or flexural strength, brittle (sudden and catastrophic) failure mode and the frequency of flaws such as inclusions and blow holes which can significantly reduce strength.

2.2.3 Cast iron components are manufactured by pouring molten iron into a mould which was traditionally formed from often ornately carved timber patterns. The pattern surround was packed with a sand and the pattern removed to form a negative. The mould was normally formed in two halves enabling the removal of the patten and subsequent pouring. As the iron cools shrinkage occurs and for this reason pattens are normally slightly larger than the intended component. For hollow sections, such as cylindrical columns, the internal core of the patten can often distort during the manufacturing process resulting in an asymmetrical cross section, which can influence the structural properties of the manufactured component. Typical defects include blow holes, which are formed by superheated air escaping from the molten iron as it cools, or 'Inclusions' which are caused by sand contaminates in the surface of the cast iron component.

2.2.4 Other common casting defects include cracks due to restrained shrinkage especially at corners and cold spots where the molten iron didn't cool monolithically.

2.2.5 A truss is a structural component formed from individual members in either tension or compression. The structural properties of cast iron are unlike steel which behaves in a consistent linear elastic fashion, that is it has similar compressive and tensile strengths. Grey Cast Iron however is non-linear elastic with its compressive strength of the order of 5 times that of its tensile strength. For this reason, trusses formed in cast iron are rare and susceptible to tensile or flexural failure of the tension chord. One of the few examples of cast iron trusses in buildings was in Crystal Palace 1851 where SCI138 notes proof testing before erection was employed to reduce the risk that no defective castings with catastrophic consequences were incorporated into the works.

2.3 Building Regulation, Quality Control and Statutory Regulation

2.3.1 It is generally accepted that assessments for existing structures should be completed giving due regard to current statutory regulation. For The Terraces relevant statute includes but is not limited to:

i) The Building Regulations 2010.

ii) The Construction (Design and Management) Regulations 2015

iii) The Planning Act 2008

iv) Planning (Listed Buildings and Conservation Areas) Act 1990

2.3.2 At the time of original construction (circa 1888), there was very little in the way of formal regulation or guidance regarding design of iron and steel structures, with the introduction of 1909 London County Council (General Powers) Act forming the first major piece of regulation in the industry. It was therefore common at the time to validate that actual strength by proof testing or load testing, such methods are not generally accepted today. Indeed *CS 463 'Load Testing for Bridge Assessment'* notes that load testing shall only be used if analysis shows that there is a realistic possibility of improving the assessed capacity to a level which can satisfy the assessment standard.

2.3.3 Modern codes of practice generally adopt the principle of limit state design, which limits the applied stress that a material experiences and typically expresses structural utilisation in terms of 'Ultimate Limit State' and 'Serviceability Limit State'

2.3.4 Conversely, some older design codes (prior to the introduction of Limit State Design) or first principal approaches utilise a 'Permissible Stress' approach. That is, the applied loading which induces stress or actions on the material, is limited to the theoretical strength of the material reduced by a 'Factor of Safety' such that the material was unlikely to yield or fail.

2.3.5 Modern materials are manufactured to strict quality controls and as such strength and other properties can be relied upon with a higher degree of confidence.

2.3.6 The project involves refurbishment of iron manufactured in the late 1800's and hence the assessment process should give due regard to a lower level of confidence in representative strengths and material properties.

2.3.7 The strength of cast iron is affected by carbon content, rate of cooling, size and shape of member, quality of foundry and the presence of casting flaws. Voids, cold joints and spatters of slag iron can act as crack inducers. It is not practical to identify all of these for a structural component that is over a century old. For an Engineer to safely justify a component for continued service, it is normal to make an allowance to provide a comfortable margin of safety to account for the many uncertainties involved.

2.4 The Process of Appraisal

2.4.1 The project involves reopening of the structure for use by the general public and as such will need to be able to safely and reliably support pedestrian loading. Modern pedestrian loadings are likely to have increased marginally from those used for the original design and are taken as a minimum of 5kN/m^2 (0.5 tonnes per square metre). Alterations or interventions needed to the structure comprise of replacement foundations that have been deamed unsuitable for reuse and the contingency to recast cast iron elements where these are justified by inspection and testing to be inappropriate to support the loads involved. These are to ensure that structural stability is achieved and justified for a reasonable service period. Some slight changes to waterproofing detailing and associated surfacing depths is required to incorporate a waterproof membrane noting that the structure as existing has a history of the waterproofing system becoming compromised, particularly around movement joints.

2.4.2 Consultation with access groups have highlighted the need to address the aggressive (steep) camber at the edge of the deck to improve accessibility for wheelchair users, etc. As such, the camber of the proposed deck has been reduced whilst remaining respectfully the original architectural intent. These necessary changes have resulted in a nominal increase in superimposed dead load.

2.4.3 Some original design deficiencies need to be tackled in the works, particularly:

i) Failure of existing inherent movement provisions which has, in places, compromised waterproofing over trusses and contributed to fracturing of various Grey Cast Iron elements. (Note the original design contained little or no provision to accommodate thermal movement).

ii) Recorded fractures to main trusses - particularly associated with tension / flexural members.

iii) A history of intervention of serviceable components with various elements having been replaced in the past. These include, but are not limited to, columns spandrels, balustrades and deck beams. Archive records further indicate that in places, different materials may have been used in previous interventions, with records of some Grey Cast Iron components having been recast in Aluminium. Furthermore, use of Metalock cold stitching repairs to cast iron elements is recorded. Metalock is not now considered appropriate for a structural repair. (Refer to SCI 138)

2.4.4 SCI 138 notes the appraisal process requires engineering judgement to carefully consider these, sometimes competing, aspects drawing to well considered conclusions to balance structural adequacy and intended use.

3.0 HISTORICAL PERFORMANCE AND PROBLEMS

3.1.1 As part of the appraisal process, a review of existing condition surveys and known historical issues has been completed.

3.1.2 Briefly, The Terraces have been closed to the public since 2012 with the Royal Crescent steps remaining open through implementation of a localised temporary back propping scheme. This provides a 'crash deck' under the Terrace at the bottom of the stair, to mitigate against the overhead risk of debris detachment in order to permit safe pedestrian passage. Temporary works have been installed in several locations in the form of masonry walls and steel back propping to alleviate failed or fractured trusses. These works have been focused mostly on areas of high risk identified by recent periodic special inspections, the latest of which are available in Appendix A.

3.2 Structural Report on the Middle Terrace, Madeira Drive, Brighton 1983

3.2.1 This report was located within archives and provides a useful insight into historical issues and previous interventions; a copy is reproduced in Appendix A. Key observations and conclusions of the report are set out as follows:

i) Surfacing differs between the two original construction phases with a history of integrity issues that impact on durability.

ii) Square balustrade posts are found intermittently on the original phase 1 works and appear to have been omitted in the later phase 2 works.

iii) 7No. columns have been replaced.

iv) Some Spandrel panels have been replaced in segmental aluminium at the junctions of the columns replaced as noted above.

v) Some original cast iron masks and gutters have been replaced in aluminium.

vi) The deck was resurfaced in 1952 with this having deteriorated by 1978.

vii) The weathering wall to the cliff to the north of the structure was at least partially refurbished with 'Gunite' (sprayed with concrete) in 1971.

viii) Jack arch deck beams have been replaced primarily to the northern side of the deck with the whole deck width between columns 3 and 12 recorded as having been replaced.

ix) Various Metalock cold-stitching repairs have been carried out.

x) Movement joints over lattice beams are noted to have been remediated with a fibreglass reinforced surface dressing.

xi) Wide cracks are noted to the lattice beams (trusses) with minor secondary cracks at right angles over the encased jack arch deck beams. The wide cracks appear attributed to hogging of the deck over lines of lattice beam support. Secondary cracks are recorded as extending for the full deck width and are attributed to expansive corrosion product on the top flange of the embedded 'I' section beams.

xii) An assessment of the deck concluded that this could only support a nominal live load of $0.9kN/m^2$. Subsequent, investigations deemed 214 beams to be inadequate which are recorded as having been replaced in 1981-82 with a further 48 additional beams added.

xiii) 1No. edge beam was identified as having failed.

xiv) Balustrade posts were inspected in 1977-78 with several found to be loose. The base of a post was exposed which revealed a crack at the junction with the 'I' section post and its base, these are recorded as having been repaired with Metalock cold stitching in 1981-82.

xv) A section of panelling (assumed spandrel) at the top of the west ramp is recorded as having collapsed onto a car in 1983 prompting an inspection of all spandrels with several being found to have cracked and several deemed dangerous. Cracking, thought to be recent, is attributed to the proximity of the balustrade posts.

xvi) In 1977-78, 6No. lattice beams were found to be cracked with a 7th crack identified during works. The issue was investigated and factor of safety noted as having reduced from 3.1 to 2.0.

xvii) Various Metalock repairs are noted to have been undertaken to lattice beams. It should be noted that this type of repair is not suitable for structural applications such as on lattice beams. It would be reasonable to conclude then, that historical repairs exist that are considered inappropriate by modern standards.

xviii) Lattice beams are reported to have been assessed via finite element analysis in the 1980's concluding a factor of safety of 3.1-3.07.

xix) In 1981-82, columns were jetted and flushed with water with several columns found to be cracked either in the upper square section or lower circular section, occasionally both. Metalock repairs are also noted as having been employed.

xx) Tensile test samples were obtained from column 70 and tested in a laboratory, results are available in Appendix B.

xxi) Columns were analysed in 1980 which concluded a factor of safety of 6.34 and that this was less than the recommended factor of safety of 8.5.

xxii) Inherent weaknesses in the structure are noted to necessitate continued structural inspections and maintenance work to be carried out on a regular basis.

xxiii) A recommendation to provide secondary support over recast aluminium spandrels is made.

xxiv) The failure mode of existing balustrade posts is noted to have resulted in cracking of the adjacent jack arch.

xxv) Multiple episodes of painting balustrades along with other interventions are noted to have reduced the balustrades' ability to deal with expected thermal expansion resulting in fracturing.

xxvi) Reference is made to a 19th century text by Professor Rankine not recommending the use of open type beams (Trusses) in cast iron due to weakness induced by openings in the web.

xxvii) It is concluded that previous column failures are "undoubtedly" the result of freezing water within blocked columns, HOP have had experience of this in other structures and agree this seems a reasonable conclusion save column 152 which is sheared.

xxviii) The factor of safety against failure of the structure is noted as being below current (1983) standards. In addition, cast iron members are noted as being prone to cracking. The need for increased maintenance liability is highlighted to compensate for the reduced factor of safety.

xxix) Maintenance liabilities are estimated for the period beyond 25 years (around 2006) it was predicted that all surfacing would require removal and resurfacing, that most 'I' section beams will need replacement, further that if maintenance was not kept up with recommendations, then wider refurbishment would likely come with additional cost.

3.3 Special Inspection March 2021

3.3.1 A Special inspection was completed in March 2021 and is reproduced in Appendix A. Key observations and conclusions are noted as follows:

i) The structure was found to have insufficient capacity to support pedestrian loading.

ii) 2No. Danger zones were highlighted on the ground.

iii) Column 153 to the east was noted as being visibly sheared below decorative detailing; this was not recorded in the previous 2019 Special inspection and movement is noted as increasing.

iv) Waterproofing to the entire deck is noted to have failed.

v) Parapet fixings should be investigated further to assess condition and risk of collapse.

vi) Brittle failure mode of lattice beams is noted with section loss to truss nodes.

vii) Fractures noted to lattice beam 2, 41, 51 (2No.), 149 (2No. fractures and back propped).

viii) Separation between lattice beams and edge beams in bays 12, 17, 28, 37, 93, 103, 111, 113, 119, 128, 132, 138 is recorded.

ix) Pitting to trial bays where paint has been exposed to bays 22 & 23.

x) Column 152 was noted as having sheared by a significant amount. Column 42 is noted as being cracked at its connection with the edge beam. Column 150 suffers a diagonal fracture.

xi) Other columns generally noted to be in a good condition with no significant critical defects noted.

xii) The condition of spandrel panels varies from good to poor with a small number suffering from fractures.

xiii) Separation between arch spandrel and edge beam noted at bays 14, 30, 31, 35, 39, 44, 49, 50, 55, 59, 65, 74, 117, 119, 131, 136, 137, 142, 146, 150 and 151. Separation being classified as either large, medium or slight.

xiv) Fractures to arch spandrels noted to bays 37, 91, 95, 148, 149 with two fractures in by 151.

xv) Fractures to edge beams noted in bays 1, 31 and 151 which had sheared completely.

xvi) Cracks to parapet posts in bays 26, 29, 31, 33, 36, 37, 38, 41, 58, 62, 64, 69, 78, 79, 91, 103 are noted.

xvii) The deck waterproofing system is noted to have failed causing seeping at almost all joints in materials.

xviii) It is noted multiple repainting episodes makes it difficult to examine the structure with paint giving the potential to hide defects.

xix) The condition of parapet fixings should be investigated to determine overall condition of parapets, assess risk of collapse and identify further repair works.

3.4 Summary of Known Interventions

3.4.1 Historical records for the structure refer to a number of historical interventions to replace or recast various components. These are summarised in the annotated Historical Defect Log in Appendix F and are not to be considered as an exhaustive record of defects that may exist. Whilst there are significant sources of historical information, perhaps unsurprisingly this is limited and incomplete and not likely to capture **all** historical interventions. The historical defect log should be interpreted with care with the expectation that further defects are present. A summary of readily auditable interventions, locations, dates and elements involved is provided in table 1a below.

YEAR	GRID REF	ELEMENT	DESCRIPTION
78	3	Column	1No. replaced
78	4	Column	1No. replaced
78	27	Column	1No. replaced
78	88	Column	1No. replaced
78	117	Column	1No. replaced
78	119	Column	1No. replaced
78	148	Column	1No. replaced
92-04	37-39	Panels	4No. Replaced
92-04	42	Panels	1No. Replaced
99	44	Panels	1No. replaced
94-95	45	Panels	1No. replaced
94-95	54	Panels	1No. replaced
94-95	60	Panels	1No. replaced
94-95	64	Panels	1No. replaced
94-95	67	Panels	1No. replaced
94-95	78	Panels	1No. replaced
96	95	Panels	1No. replaced
96	109	Panels	1No. replaced
92-04	117a	Panels	Replaced
94-95	46-53	Panels & Posts	14No. replaced
96	86	Panels & Posts	2No. replaced
96	87-88	Panels & Posts	3No. Panels, 2No. Posts replaced
96	98-100	Panels & Posts	4No. replaced
91-92	1-5	Panels & Posts	8No. replaced
94-95	12	Panels & Posts	1No. replaced
94-95	14	Panels & Posts	2No. replaced
94-95	16	Panels & Posts	2No. replaced
94-95	17	Panels & Posts	2No. replaced
94-95	21-28	Panels & Posts	15No. replaced
91-92	28	Panels & Posts	1No. replaced
92-04	32	Panels & Posts	2No. Replaced
92-04	33-35	Panels & Posts	2No. Replaced
94-95	70-77	Panels & Posts	14No. replaced

94-95	81-85	Panels & Posts	14No. replaced
78	3	Spandrel	Aluminium replacement
92-04	3/4	Spandrel	CI replacement
92-04	4/5	Spandrel	CI replacement
78	27	Spandrel	Aluminium replacement
92-04	26-28	Spandrel	CI replacement
92-04	87	Spandrel	refers to replacements in 4 sections
92-04	88	Spandrel	refers to replacements in 4 sections
99	116	Spandrel	1No. replaced
99	117	Spandrel	1No. Half spandrel replaced
92-04	118-119	Spandrel	CI replacement

Table 1a, indicating summary of elements known to have been replaced and referred to in historical reports not considered exhaustive. Note current consented scheme bound gridline 78 to 117.

3.4.1 In summary a significant number of elements are known to have been replaced historically and as a minimum the relative proportion of interventions are indicated in terms of the entire structure in table 1b below:

Element	Quantity	Relative Proportion of entire terrace
Half Spandrels	19No.	6%
Columns	7No.	5%
Balustrade Posts	85No.	28%
Balustrade Infills	104 Linear m	34%

Table 1b, summary of known recast / replacements.

3.4.2 The above defects are not to be considered as exhaustive. Other consultant's report other defects are likely concealed by multiple layers of paint and suspected to be present.

3.5 Pilot Grit Blasting and NDT of 2014 Trial components

3.5.1 A pair of spandrels and a pair of Balustrade sections including posts and infills were available on the deck having been disassembled during the 2014 Trials. The opportunity was taken to remove these from site for grit blasting and non-destructive testing by using Magnetic Particle Inspection to look for defects and fracturing that would otherwise be concealed by multiple layers of paint. A record of this site visit 2nd May 2023 is available in Appendix H.

3.5.2 Observations are summarised as follows:

i) The contractor seemed to have a robust proposed method in place to label and track artifacts.

ii) Fusion welding techniques were demonstrated with a selection of repaired components available for review suggesting that the fusion welding process remains appropriate.

iii) A half spandrel was available for MPI inspection, this one of a pair having been recovered from the 2014 trials. MPI was conducted during the visit to CIWS yard. MPI revealed 7No. previously concealed defects, none of which were noted to be associated with bolt hole corrosion-initiated fracturing. Some fracturing was noted in the haunch area which would be structurally significant in terms of spandrels delivering effective portal action to stabilise the south elevation. It could not be determined if fracturing was due to service, original construction defect or handling / storage however the former seems most likely.

iv) Fracturing to the voussoir (Key) was evident and associated with corrosion, this was noted as potentially structurally significant where arching action develops.

v) The half spandrel not yet grit blasted exhibited significant fracturing to the haunch zone through the full depth of section, this unrelated to bolt hole related initiation.

vi) 1No. infill panel was inspected for MPI. Historical attempts to weld repair the infill was noted on low quality materials and workmanship.

vii) Geometry of the infill's original casting was noted to vary significantly with some sections varying in geometry of between 12mm and 6mm or so. This suggests that the original quality of workmanship / consistency employed during the casting process is variable.

viii) Significant casting defects were noted to infill transoms at approximately 1/3 span. The holes were probed to gauge the significance of the defects which indicated that 20mm of the 35mm section was not present. This would have a significant impact on the structural resistance of the Transom to span between balustrade posts. It is noted that these defects could not reasonably be identified visually prior to grit blasting. Further it is understood that subsequently these defects have been repaired by the gas fusion welding process satisfactorily. Both these facts perhaps provide confidence in the overall projects approach to the identification and remediation of casting defects.

4.0 STRUCTURAL SENSITIVITY ANALYSIS

4.1 General

4.1.1 This section presents the results of a 'Sensitivity' Analysis. The purpose of this analysis is to identify the most critical elements by theoretical analysis. The results can then be compared with actual known problems to assist in the development of detailed design of any interventions required to bring the structure back into service.

4.1.2 Structural elements are considered separately along with checks on specific failure modes with a narrative summary highlighting the outcome of each of the analysis.

4.1.3 Dimensions and parameters used in the assessment are a combination of those measured on site or extracted from record drawings. Material parameters are informed by established published practice corroborated with site specific tensile testing obtained from cast iron columns and railings at The Terraces and other local sampling to Brighton Seafront Iron structures of a similar date.

4.1.4 The analysis initially considers the modes of failure as 'mutually exclusive' scenarios (i.e. not as combinations) with discussion on the results provided later.

4.1.5 The Sensitivity Analysis assumes that the elements assessed are in a good condition, thus assessing the theoretical sensitivity of the original design.

4.1.6 The actual condition of structural elements is noted as generally poor or very poor in places with a rich history of interventions recorded to remediate elements. Ultimately, condition cannot be definitively determined until the structure has been grit blasted and carefully inspected. Extensive corrosion to some iron components is noted also. Other potential damage to the components may exist, for example some components are recorded as having been repaired with Metalock cold stitching techniques. This technique does not achieve a full structural repair hence known weaknesses exist that will reduce the actual capacity of a member when compared to a purely theoretical assessment.

4.1.7 In general, the sensitivity analysis analyses the proposed loading conditions resisted by grey cast iron members.

4.1.8 Cast iron design has traditionally adopted a permissible stress design approach. This approach reduces the theoretical strength of the cast iron by a Factor of Safety. Advice on what factor of safety to adopt varies depending on individual circumstances and published guidance.

4.1.9 SCI 138 notes the need to adopt higher factors of safety for brittle cast iron compared with ductile wrought iron reflecting the more serious consequences of cast iron failure. SCI 138 goes on to summarise basic permissible stresses for cast iron.

4.1.10 SCI 138 recommends the use of a minimum Factor of Safety of 3.0 applied to 95% confidence to ultimate tensile strength for a 'Stage 3' assessment. A stage 3 assessment adopts site specific tensile testing results to inform the assessment. Whilst at the time of writing a relatively small population of tensile test results is available it is anticipated that these will be ratified by supplementary sampling and testing as and when the structure is carefully dismantled and inspected. For the currently available laboratory testing refer to Appendix B.

4.1.11 The Historical Structural Steelwork Handbook, extract provided in Appendix C notes it is desirable to adopt a factor of safety of 5.0.

4.1.12 A statistical analysis of currently available samples is provided in Figure 2 below:

Element	Tensile Strength (N/mm2)	Probability Density
MT Column (historical)	148	0.021
Brighton Seafront Rails	119	0.006
	119	0.006
	115	0.004
MT Railing Tests 2023	161	0.018
	176	0.008
	158	0.019
	155	0.021
	147	0.021
	142	0.020
	154	0.021
	175	0.008
	165	0.015
	142	0.020
	158	0.019
	159	0.019

Mean:	149.6	N/mm2
Standard Distribution:	18.6	N/mm2
5th Percentile:	119.0	N/mm2

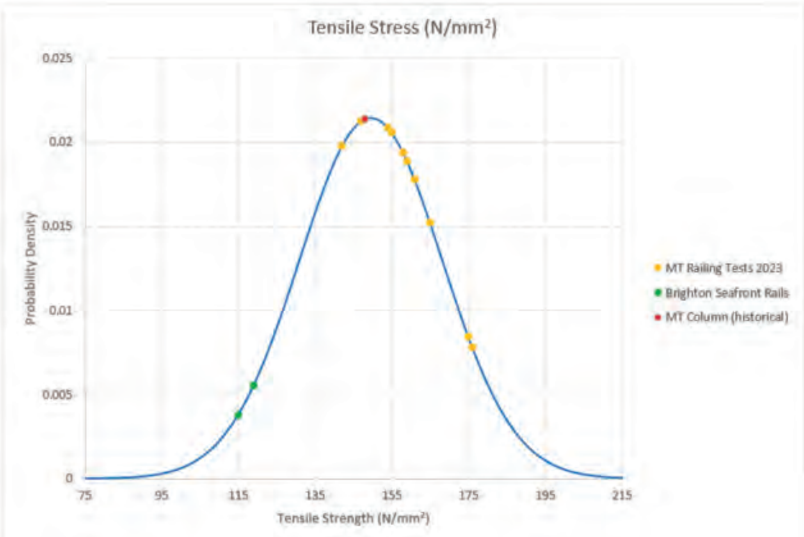


Figure 2 - statistical analysis of Iron tensile strength of iron sampling results.

4.1.13 The permissible tensile stress adopted for Grey Cast Iron in the sensitivity analysis is therefore considered to be 23.8N/mm^2 based on the 5th percentile, with a minimum applicable factor of safety of 5.0.

4.1.14 Coincidently, use of a FoS of 5.0 results in a permissible stress that is similar to that published in the 'The Structural Engineer, Conservation Compendium Part 3: Historic wrought iron, cast iron and mild steel' which adopts a permissible stress of 24N/mm^2 hence some confidence this can be taken as at or around the right order of things.

4.2 Spheroidal Graphite Cast Iron

4.2.1 SCI 172 Castings in construction describes Spheroidal Graphite Cast Iron (SG Iron) as follows:

4.2.2 "SG Iron, also known as nodular cast iron or ductile iron, is a type of iron developed 50 years ago which displays improved strength and ductility due to carbon being coagulated into spheres. It is formed by adding magnesium to the melt in the ladle. The tensile strength is typically 75% of its compressive strength, and elongations are between 2 and 22%, depending on the grade. SG iron has found application as standards for crash barriers or bollards because it can be cast to the preferred shape and provides adequate protection against vehicle impact by bending and absorbing energy before breaking."

4.2.3 "…. The type of cast iron used in historical structures was almost always Grey Cast Iron. A number of failures of these structural iron castings occurred in the nineteenth century before the limitations of the material were fully understood (i.e. lack of strength in tension and the *risk of brittle fracture as a result of notches or cracks in the material). Poor foundry practice also contributed to the failures."*

4.2.4 *"… In recent years, cast iron has been making a comeback in conservation work, largely because of the need for castings to replace damaged or lost components. Guidance on appraisal techniques for existing structures and the design of Grey Cast Iiron is given by Bussell(')"*

SCI 172 provides permissible stress for SG iron base on the 0.1% proof stress which are reproduced in Figure 3 below.

Table 4.1 *Permissible stresses in tension and compression for SG iron*

(Adapted from Gilbert, G N J, Engineering data on nodular cast irons -SI units, BCIRA, 1986)

Stress condition	Ferritic SG iron: Grades 350/22, 350/22L40, 400/18, 400/18L20	Ferritic SG iron: Grade 420/12	All other grades of SG iron: Grade 450/10	All other grades of SG iron: Grades 500/7, 600/3, 700/2, 800/2, 900/2
Direct tension	$0.75 \times 0.75\ R_{p0.1}{}^{t} = 0.56 \times R_{p0.1}{}^{t}$	$0.75 \times 0.70\ R_{p0.1}{}^{t} = 0.52 \times R_{p0.1}{}^{t}$	$0.75 \times 0.693\ R_{p0.1}{}^{t} = 0.52 \times R_{p0.1}{}^{t}$	$0.75 \times 0.6 \times R_{p0.1}{}^{t} = 0.45 \times R_{p0.1}{}^{t}$
Direct compression	$0.75 \times 0.8 \times R_{p0.1}{}^{c} = 0.6 \times R_{p0.1}{}^{c}$			

Notes: Permissible stress = 0.75 × limit of proportionality
$R_{p0.1}{}^{t}$ = 0.1% proof stress in tension
$R_{p0.1}{}^{c}$ = 0.1% proof stress in compression

Figure 3 - extract from SCI 172 Castings in Construction providing permissible stresses for SG Iron

4.2.5 ISO 1083-2018 tabulates the Mechanical properties for SG iron which are reproduced below:

Table 1 — Mechanical properties measured on test pieces machined from cast samples for ferritic to pearlitic grades

Material designation	Relevant wall thickness t mm	0,2 % proof strength $R_{p0,2}$ MPa min.	Tensile strength R_m MPa min.	Elongation after fracture A % min.
ISO1083/JS/350-22-LT[a]	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	220 210 200	350 330 320	22 18 15
ISO1083/JS/350-22-RT[b]	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	220 220 210	350 330 320	22 18 15
ISO1083/JS/350-22	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	220 220 210	350 330 320	22 18 15
ISO1083/JS/400-18-LT[a]	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	240 230 220	400 380 360	18 15 12
ISO1083/JS/400-18-RT[b]	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	250 250 240	400 390 370	18 15 12
ISO1083/JS/400-18	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	250 250 240	400 390 370	18 15 12
ISO1083/JS/400-15	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	250 250 240	400 390 370	15 14 11
ISO1083/JS/450-10	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	310 to be agreed upon between the manufacturer and the purchaser	450	10
ISO1083/JS/500-7	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	320 300 290	500 450 420	7 7 5
ISO1083/JS/550-5	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	350 330 320	550 520 500	5 4 3
ISO1083/JS/600-3	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	370 360 340	600 600 550	3 2 1
ISO1083/JS/700-2	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	420 400 380	700 700 650	2 2 1
ISO1083/JS/800-2	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	480 to be agreed upon between the manufacturer and the purchaser	800	2
ISO1083/JS/900-2	$t \leq 30$ $30 < t \leq 60$ $60 < t \leq 200$	600 to be agreed upon between the manufacturer and the purchaser	900	2

NOTE 1 Elongation values are determined from $L_0 = 5\ d$. For other gauge lengths, see 9.1 and Annex B.

NOTE 2 The mechanical properties of test pieces machined from cast samples do not necessary reflect exactly the properties of the casting itself. Values for tensile properties of the casting are given in Annex D for guidance.

NOTE 3 The data apply to separately cast samples, cast on samples and side-by-side cast samples; therefore the suffix "S" is not included.

[a] LT for low temperature. [b] RT for room temperature.

Figure 4 - Extract from ISO 1083-2018 indicating mechanical properties of SG iron

4.2.6 Depending on the grade, SG iron has a minimum tensile strength of between 320-900N/mm^2 (see Figure 4 above). It is this increased tensile strength that significantly enhances SG Cast Iron when compared to traditional Grey Cast Iron. Many grades are available, a permissible stress is estimated based on the 0.2% proof stress rather than 0.1% and reduced by factor of 0.85. Should SG iron be used, the final permissible stress will be subject to detailed design, chosen grade of material used and will need to be developed with the foundry if required. A relatively low and commonly available grade of SG iron has been adopted to test this option for its appropriateness as a recasting material.

Tensile and Compressive Permissible Stress = 0.85 x 0.56 x 450 =214N/mm^2.

4.3 Lattice Beams (Trusses)

4.3.1 A graphical summary of the proposed decks 'live load' capacity is provided in Appendix D. Truss Sensitivity Analysis Summary. Live load means the weight of people or other variable loads that may be needed to service the deck. Results have been annotated Red, Amber and Green (RAG Analysis) with the following definitions used.

RAG Analysis Colour	Description of failure
Red	Lattice Truss Expected to Fail
Amber	Lattice Truss considered Unsafe
Green	Lattice Truss Considered Safe

4.3.2 To interpret the RAG analysis, it is important to understand the principle of permissible stress design and assessment. The assessment process involves evaluating applied load and comparing this against capacity. Although the applied loads can be evaluated with a reasonable degree of confidence, evaluating the residual capacity of 150-year-old trusses is not an exact science.

4.3.3 To evaluate capacity, it is normal practice to apply Factors of Safety (FoS) to provide a margin of safety. The FoS is representative of the magnitude of risk, to account for variations in structural condition, geometry, quality of available information, materials and loading during service. For example, if little information is available and no site-specific testing has been undertaken, then the FoS is often taken as 5.0 or greater. Acceptance of a reduced FoS means an acceptance of a greater degree of risk, however an FoS 3.0 might be reasonably argued if comprehensive information and test data is available to mitigate this, especially if supplemented by sampling and / or load testing to prove assumptions as reasonable. To erode a FoS significantly increases the risk of failure and a FoS 1.0, for example, would represent a completely unacceptable risk and acceptance that failure under service is to be expected.

4.3.4 Much has been said of FoS in the historical reporting for the structure. The 1983 report noted the structure was working at an unacceptable FoS. More recently an Amey report recommended load mitigations not to exceed an applied live load limit of 3.9kN/m^2. This is lower than the standard assessment loading of 5.0kN/m^2, perhaps suggesting that client has been accepting to a risk managed approach in the past. It is however considered that this situation was largely unsustainable in terms of practical management with various previous maintenance campaigns reacting to problems encountered during the works. To some degree this is considered to have led to the structures demise and hence is an important consideration.

4.3.5 Amey's permissible stress figure should be treated with care also as their report was written before site specific tensile test results were available and hence adopted a higher permissible stress that is not currently supported by recent site-specific test results, albeit at the time of writing no tensile tests were available sourced from trusses.

The assessment of the overall capacity of a truss requires identification of the various failure modes. Figure 5 below indicates the principal failure modes of consideration with the Diagonal Strut failure mode identified as governing overall truss load carrying capacity. This conclusion is also supported by others that have modelled the structure historically.

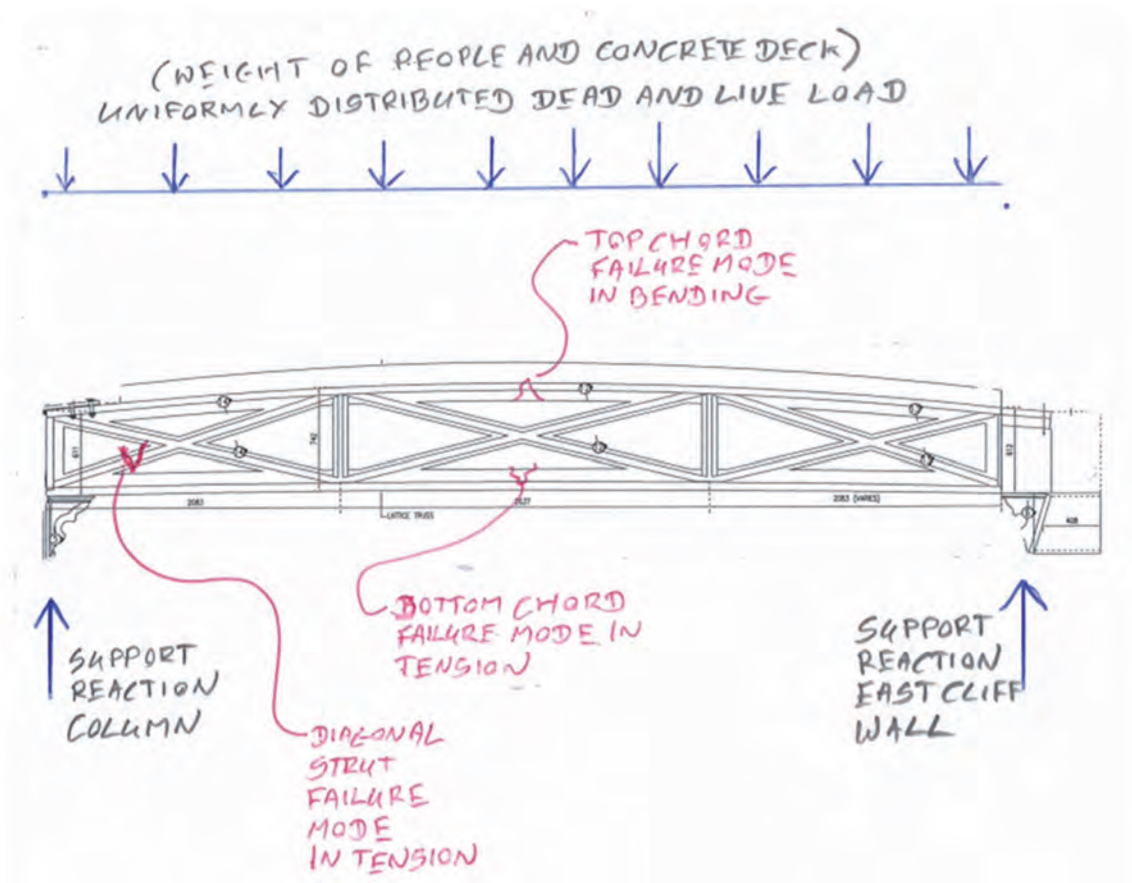


Figure 5 – Indicating principal failure modes of consideration with the Diagonal Strut mode identified as governing overall truss capacity.

Ultimately until more confidence in the tensile strength of the material from which trusses are formed (actual truss specific testing), the assessment process of a truss cannot be concluded.

The best outcome is that grade of iron used for trusses is higher than other cast iron site specific elements that have been tested. It is hoped this is the case as the engineering knowledge at the time of construction is documented as knowing that this form of construction is particularly sensitive to tensile strength. (Ref 3.2.1 (xxvi) above).

4.4 Should truss specific testing not yield adequate tensile strength is available to justify truss reuse then consideration could be given to building up individual members

with fusion welding. More likely favoured is recasting in SG iron to eliminate the brittle failure mode for trusses. Assessment of Lattice Beams at Maderia Terrace

4.4.1 The existing trusses are formed in Grey Cast Iron which is known to suffer a brittle (or sudden) failure mode. Design and assessment codes promote the use of robust Factors of Safety to prevent this from ever happening. There are multiple examples of existing trusses being distressed i.e. fracturing and back propping; indeed recently (2023) a number of new trusses have been back propped due to concern over adequacy.

4.4.2 Uncertainties remain in that no Truss specific tensile strength data is currently available. Currently available site-specific data is derived from 12No. samples obtained from a single balustrade infill panel. The actual tensile strength of the Cast Iron used for Trusses could therefore be higher, lower or perhaps similar. Trusses may have been formed from a higher quality material, or to a higher quality of workmanship (fewer casting defects). Similarly, actual loads will vary throughout the structure, where stiffer areas would reasonably attract increased load to individual bays, compared to that derived in a purely static analysis. Uncertainty always remains in actual material densities and dimensions due to time dependent processes such as corrosion, no matter how detailed the survey information for the structure.

4.4.3 Appendix D presents a graphical representation of the governing tensile stresses (Diagonal struts). This adopts a FoS of 5.0 which HOP are of the opinion is an appropriate FoS to use given the residual level of uncertainly. When this is applied to the available test data, a permissible stress can be derived and sets a threshold which is deemed unacceptable to exceed.

4.4.4 To further inform the engineering judgement a trial of Dynamic Testing to the structure was commissioned in 2023. Briefly this involved the oscillating a 20kg mass on the existing structure with 3No. accelerometers to measure response. The process informed the likely stiffness characteristics of the existing deck. This testing concluded that there may well be some hidden strength in the existing deck / trusses which may provide some confidence that truss reuse could be realised following full and final assessment.

4.4.5 The Dynamic testing process however cannot be fully relied in full at this stage for the following reasons:

4.4.6 i) CS 463 'Load Testing for Bridge Assessment' cl 4.5 states: "The test load is not likely to be sufficient to develop non-linear behaviour where the failure mode is brittle in nature (e.g. in a shear load test)….".

4.4.7 ii) the study identified some anomilies which would require further investigation.

4.4.8 iii) The study was small and would require significant extension to reliably model the deck. Further the existing deck is to be removed and hence the replacement deck requires modelling.

4.4.9 Current conclusions, based on all of the above, is that there remains uncertainty over the suitability that existing trusses may be incorporated back into the refurbished scheme. .Consideration must therefore be given to the prospect to recast trusses in Spheroidal Graphite (SG) Iron. We understand that there is pressure to balance conservation and the need to prevent the loss of historic building fabric where reasonable and practical. The strategy under development recommends testing of trusses to inform the final decision of whether trusses can be safely adopted back into the works. Following this testing (and depending on the results) confidence in the existing ironwork may be increased, and there may be scope to reduce the FoS used to 4.0. When a FoS of 4.0 is used (and assuming it is justified by a suitably wide population of actual test results) this might indicate a deck live load capacity in the order of 2.0kN/m^2. This order is consistent with previous advice and opinion. Likewise, a FoS of 3.0 might indicate a deck capacity of the order of 4.7kN/m^2. This is based on current tensile testing data available which it is hoped would be conservative to be substantiated by further testing.

To narrow the unknown variable of actual tensile strength it is recommended that trusses are sampled as soon as practically possible. This might be completed in-situ with samples taken from several of the back propped trusses which would be relatively straightforward in terms of site works. Alternatively, and a higher risk approach might be to leave sampling until the first dismantled trusses are available. If lower factors are supported by increased confidence informed by sampling, then consideration could be given to relaxing the FoS supplemented by physical load testing of trusses to confirm they are safe to reincorporated into the works.

4.4.10 Should asset owner want to adopt a lower FoS or consider a managed approach by the application of a weight limit to the deck then this would be needed to be accepted as a departure from generally accepted design advice hence this is not recommended.

4.4.11 An approach as to how the above might practically implemented is developed in section 5 of this report.

4.5 Recasting Should Existing Trusses Fail Assessment

4.5.1 SCI 138 notes for replication of Grey Cast Iron members the use of spheroidal graphite cast iron offers a less brittle and an economical structural alternative and that this approach is commonly used in conservation today.

4.5.2 Detailed design will need to select the final grade of SG iron in conjunction with the appointed foundry. Due to SG irons significantly increased strength, inherent resistance to brittle failure and in the knowledge that the existing Grey cast iron elements are working at or around capacity SG iron; is recommended for replacement of existing components which fail assessment for reuse.

4.6 Balustrades

4.6.1 Brighton and Hove City Council have an extensive length of seafront railings serving in excess of 4km of frontage, a fraction of which includes The Terraces. Seafront railings were reported on for the entire frontage in 2001/02 which identified a number of deficiencies. The Seafront railing investigations included iron sampling and trial pitting to balustrade standards (Posts). Deficiencies were noted in the 4No. types of balustrading that make up the frontage including:

- i) Inadequate height;
- ii) Inadequate infilling that would allow a 100mm sphere to pass;
- iii) Departing from anti-climb requirements of current codes;
- iv) Inadequate containment capacity of infills;
- v) Inadequate containment capacity of standards.

4.6.2 The balustrades at The Terraces are of a similar type to those reported on in 2001/02. The issues associated with the wider seafront railings are therefore relevant.

4.6.3 It is understood the Council Highways Department have maintenance responsibility for most of the seafront railings. Over time, it is understood a number of balustrade components have failed and have been replaced with recast components. The Highways team maintain casting patterns and a stock of replacement components that are called upon as an when failures occur during service. The railings are therefore a serviceable asset and have been for many years.

4.6.4 Many areas of the seafront frontage are supported on suspended structures. The evolution of the seafront developed significantly during the 1800's and 1900's. In the mid 1800's a number of arch structures were formed, typically during the late 1800's there was a further extension southward pushing the boundary between upper and lower esplanade south further. Later in the 1930's there was a further extension southward in places. The end result is that a number of generations of suspended structures support the A259. Many of these structures have approached the end of their serviceable life and have been replaced or upgraded; notably:

i) The Aquarium Colonnade, approximately 35 linear meters of 1930's reinforced concrete structure deemed inadequate in terms of load carrying capacity replaced circa 2001.

ii) The West Pier West, approximately 80 linear meters of Arch structures upgraded circa 2013.

iii) West Pier Arches East, approximately 130 linear meters of arch structure upgraded circa 2014.

iv) The West Pier frontage, approximately 75 linear metres of frontage developed on the footprint of the West Pier circa 2015.

v) West Street Shelter, approximately 75 linear meters of frontage upgraded circa 2020.

4.6.5 Photograph 1 below indicates the Aquarium Colonnade frontage and serves to indicate how the frontage has developed south over the years, noting the original old sea wall (Pre-Circa 1930) can be seen right of frame.

Photograph 1 - Indication the Aquarium Colonnade frontage immediately east of Brighton Palace Pier circa 2001 during reconstruction. Note original sea wall right of frame with associate steps and more modern on current southern sea frontage alignment steps left of frame.

4.6.6 Whilst precedent is not considered entirely relevant, all of these projects have enabled lessons to be learned in terms of what is practically achievable to save historic fabric associated with the seafront railings.

4.6.7 The existing railing at The Terraces are formed from two types of standards. A large moment resisting foot connection to the main balustrade standards on truss gridlines. A simpler intermediate post is provided between truss grids. One of the lessons learned from previous projects has been the practical limitation of justifying the balustrade moment resisting holding down connection into the developed works.

4.6.8 Due to a combination of constraints, principally to extend balustrade height to current standards and to balance this with preservation of building fabric, the overarching practical solution has been to stretch the original pattern to achieve adequate height by recasting. This approach has also enabled simplification of the holding down moment resisting connection to incorporate it into various deck arrangements similar to The Terraces. This approach enabled recasting in Spheroidal Graphite Cast Iron to remove the brittle failure mode associated with Grey Cast Iron and improve containment capacity. All above ground detailing and connections were similar to existing thereby resulting in visual continuity.

4.6.9 In practical terms, The Terraces balustrade should be carefully inspected to confirm these preliminary informed assumptions.

4.6.10 This approach is considered consistent with Historic England preapplication advice dated 6th October 2022. " … *Any harm caused by increasing the height of the balustrade railings could be minimised by ensuring that the same pattern of railings and connections is adopted as that used elsewhere along the seafront where other railings have been increased in height for health and safety reasons*".

4.6.11 Photograph 2 below indicates the completed works to the Aquarium Colonnade and the junction of new and refurbished seafront railings.

Photograph 2 - indicating Junction of Seafront railings at the Aquarium Colonnade. Left are the original seafront railings, right extended railings adopting similar detailing and connections, baseplate connection concealed.

4.6.12 Specific to The Terraces and with reference to section 3.4, it is noted that a significant proportion of balustrade standards and infill panels are recorded as having been historically recast or replaced mostly due to suspected fracturing. Whilst detailed inspection has not been possible save the Infills NDT tested off site currently, there is clear visual evidence of current and existing fracturing evident. Refer to Appendix G Balustrade Assessment.

4.6.13 Where limited and detailed inspection by grit blasting to infill panels recovered from the 2014 trials have been possible, this has revealed casting defects and variability in casting geometry / original quality control which reasonably has a negative impact on strength and therefore containment capacity or reliability of service in terms of safety.

4.6.14 A theoretical assessment of containment capacity has been completed and the following departures noted for client consideration with mitigations considered and summarised in table 4 below.

Element	Failure mode	Pass / Fail	Potential Mitigation
Balustrade Standard	Height	Fail	Increase in height to achieve 1.1m supported by HE preapplication advice.
	Base connection	Fail, if reseated to achieve 1.1m height this results in a significant tripping hazard at deck level which is not considered acceptable in terms of serviceability for deck users.	Consideration has been given to cutting the foot however this negatively impacts on bending resistance resulting in unjustifiable bending resistance to resist applied loads.
Strategy	**Stanchions proposed to be recast to new height and new base connection detail, incorporating new lower rail to reduce gap below the balustrade infill panel. As per detailed drawings. See Cast Iron Heritage and Design Summary Note (Purcell).**		
Infill Pannel	Anti climb / Permeability	Fail, the existing arrangement departs from current standards where a 100mm spear can pass and horizontal members promote ease of climbing.	Significant changes to an infill panel geometry are considered to 'cut against' conservation practice in terms of character to the elevation. Client to consider if a departure from current standards is acceptable or otherwise and check with their insurers with regards to public safety duty. It is noted that this seems to have been accepted elsewhere on the frontage historically. Further if an increase in height is adopted, a permeable area will increase hence Architect should consider mitigating this by introduction of supplementary kick plate or similar such that the situation is no worse than existing in terms of permeability. Particular attention should be given to the camber and tendance for debris to role off the deck resulting in an overhead risk to the public.

	Top Transom capacity	Fail (Brittle failure) Pass (Deflection)	Theoretical checks conclude top transoms fail to adequately contain the applied pressure derived from service loading. Whilst some hidden strength could be available, historical information suggests a considerable proportion of infill panels have failed and have required recasting. Variable geometry and casting defects are also noted as prevalent. Client could consider further quality and testing which could reasonably be piloted with the 2No. infill panels derived from the 2014 trials it is not normally recommended to proceed to load testing unless theoretical checks conclude adequate capacity is likely present however such a supplementary round of testing and perhaps physical load testing could be considered to exhaust options for reuse or justify as suitable for reuse. Theoretical deflection check pass however this is somewhat academic as the brittle failure mode governs.
	Bottom Transom Capacity	Pass (Brittle failure) [Marginal Pass] Pass (Deflection)	Theoretical calculations suggest that Bottom transoms pass brittle failure checks albeit this is marginal with 93% utilisation or so. It is noted actual capacity will be highly dependent of actual tensile strength and the presence of casting defects which are known to be present as well as highly variable geometrical deviations in the original castings which adversely impact on actual strength to be realised. Theoretical deflection check pass however this is somewhat academic as the element being assessed is a single element of a wider infill Pannel consisting of other elements hence Top Transom governs overall containment capacity of an infill.

	Diagonal strut	Fail (Brittle failure) Pass (Deflection)	Theoretical checks conclude diagonal struts fail to adequately contain the applied pressure derived from service loading. Whilst some hidden strength could be available historical information suggests a considerable proportion of infill panels have failed and have required recasting. Variable geometry and casting defects are also noted as prevalent. Client could consider further quality and testing including physical load testing which could reasonably be piloted with the 2No. infill panels derived from the 2014 trials it is not normally recommended to proceed to load testing unless theoretical checks conclude adequate capacity is likely present however such a supplementary round of testing could be considered to exhaust options for reuse or justify as suitable for reuse. Theoretical deflection check pass however this is somewhat academic as the brittle failure mode governs.
Strategy	**The infill panels from the terrace are to be tested in section 1 works, including physical load testing.**		
Top Rail	Top Rail (existing 48.3CHS3.2)	Fail (Ultimate limit state) Fails (Serviceability limit state / Deflection)	Assessment of the existing top rails fails assessment in ultimate containment capacity and deflection and are not considered suitable for reuse unless client accepts a departure from established best practice)

	Top Rail (replacement alternative 48.3CHS6.0)	Pass (Ultimate limit state) Fail (Serviceability limit state / Deflection) albeit the solution is likely to form a practical strengthening measure which is and improvement on the existing)	Consideration has been given to replacing the existing top rail with the same size tube with increased wall thickness to seek to pass assessment. Ultimate limit state passes the assessment. Deflection is estimated as 33.8mm with a limit of deflection 13mm. in reality intermediate transom upstands and partial fixity at supports are likely to yield a slightly reduced deflection. Considered a reasonable and practical enhancement on existing situation working within the constraints of the existing geometry (6mm wall thickness is the largest available wall thickness commonly available to maintain external diameter to match existing. Client would need to accept a deflection departure for current codes perhaps taking comfort that the situation is improved from existing.
	Top Rail (Teak option)	Fail (strength) Fail (Deflection)	Section significantly fails assessment and is not suitable for adoption.
Strategy	**New top rails are specified, complete with new fixing detail to re-cast stanchions.**		

Table 2 - summary of balustrade departures for client consideration in developing strategic approach.

4.7 Disproportionate Collapse and Robustness

4.7.1 Building codes and standard note the importance of disproportionate collapse and sway stability. Whilst some confidence might be taken from the fact the structure has stood for over a century, historical reporting records a number of structural problems and interventions associated with the Spandrel (south) elevation.

4.8 Sway Stability to South Elevation (Spandrel Elevation)

4.8.1 The existing structure relies on the stiffening of spandrel bays by portal / arching action. This being particularly important as it limits the bending that would otherwise be applied to cast iron columns which are ill-placed to resist bending action due to the nature of the brittle cast iron from which they are formed.

4.8.2 A review of the available historical reports and the grit blasting trial has identified the following problems:

i) Fracturing evident to existing spandrels at the structurally sensitive haunch area not associated with bolt hole corrosion initiation. Refer to Appendix F, Record of Grit Blasting Inspection to trial components 2/5/23.

ii) Voussoirs (key stones) not present, missing or damaged rendering arch / portal action to the south elevation ineffective. This results in adjacent bays 'taking up' additional load compounding the wider fracturing problem.

iii) Minimum 19No. half spandrels are recorded as having been replaced in Aluminium or iron due to suspected brittle fracturing as is evident elsewhere.

iv) Joints are recorded as suffering excessive movement compromising the waterproof system over time.

v) A panel (assumed Spandrel) is recorded to have collapsed onto car 1983.

vi) Columns are reported to have lower than favoured factor of safety (1983) hence are sensitive to bending / eccentric loading. Column 152 is reported as having sheared (Horizontal Load) with 2No. spandrel fractures in the associated bay i.e. portal / arch action appears to have been lost and column has entered bending which results in brittle failure mode.

vii) Separation between Spandrel and edge beams are commonly reported rendering arch / portal action ineffective / unreliable.

4.8.3 These existing structural problems are summarised in Figure 3 below.

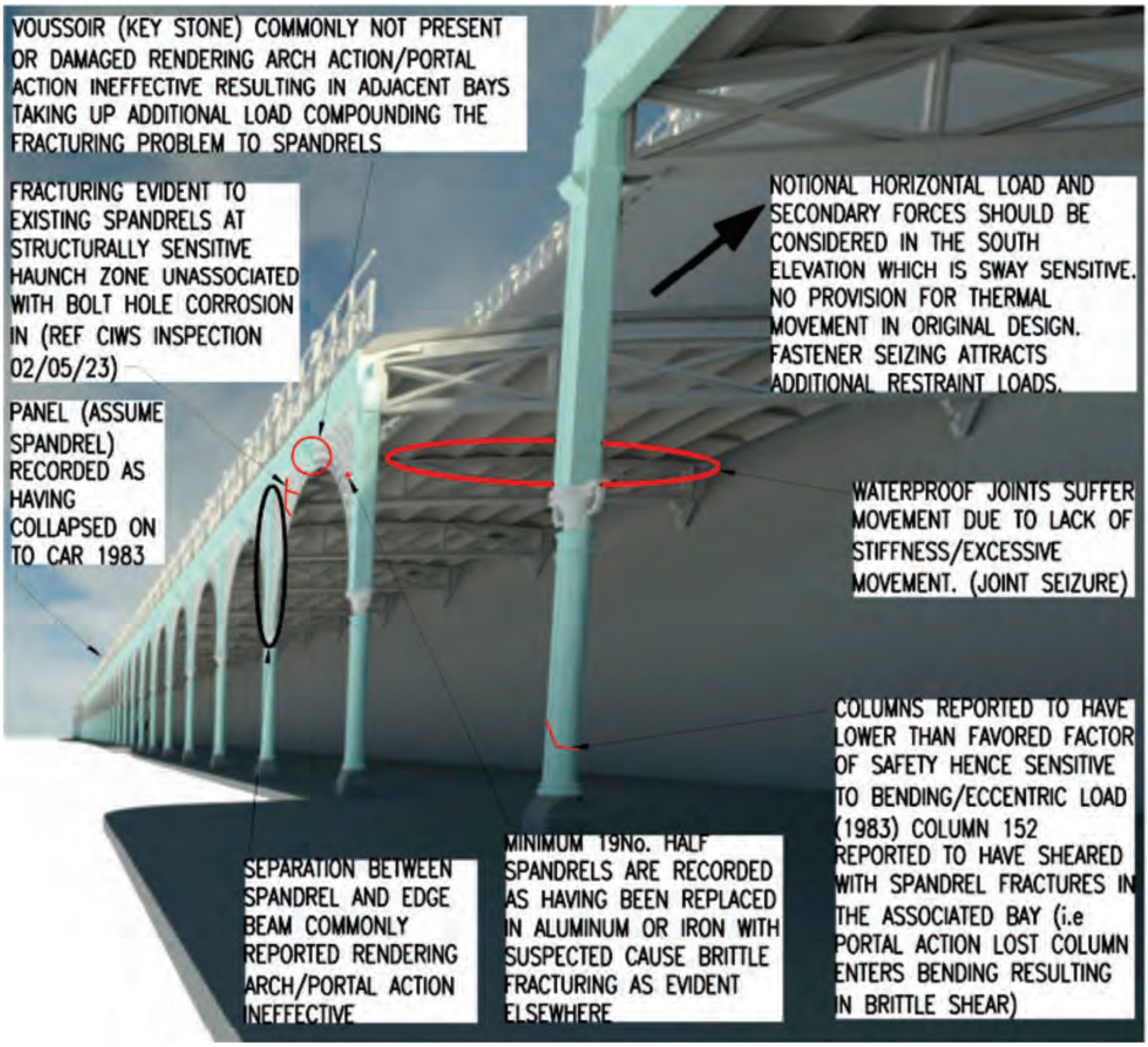


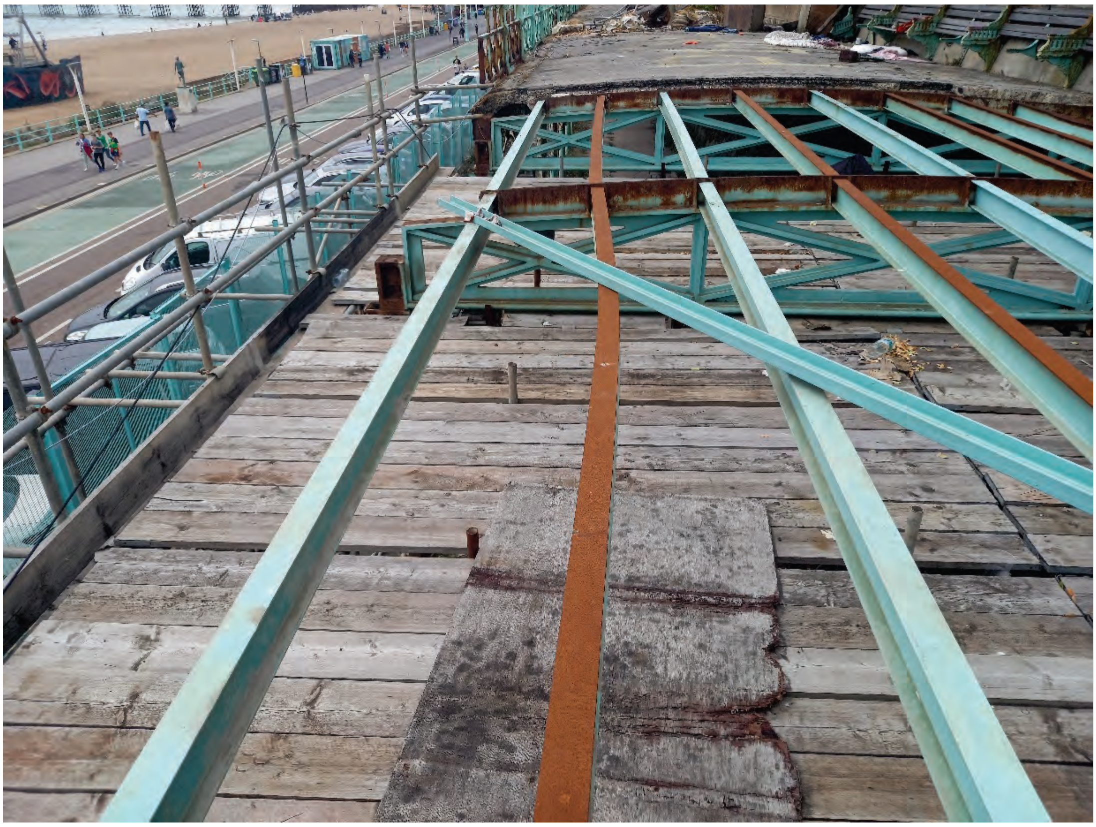
Figure 6 – Sketch summarising existing structural problems with the south (Spandrel) elevation.

4.8.4 All materials expand and contract with changes to ambient temperature. The environmental exposure of the structure is such that it is exposed to a temperature range of the order of -5°C in the winter months and +35°C in the summer. The structures overall length is 865m and as such is reasonably expected to extend by the order of $12\times10^{-6}\times865\times40=410$mm or so.

4.8.5 A contributing factor to the causation of south elevation structural distress is the provision for thermal movement incorporated into the original design. If designed today it would be normal to incorporate movement joints into the structure at periodic intervals up to 70 metres or so. The existing structure has no formal movement joints and would appear to rely on nominal tolerances to bolted connections which have a tendency to seize over time due to corrosion.

4.8.6 The restraining action of this nominal provision for movement attracts load. Further when a spandrel bay releases (yields and fractures) these restraint forces are released and the adjacent bay will tend to ‘take up’ the load. These processes compound the fracturing problem which may be the reason so many interventions have been required in the past.

4.8.7 As and when historical interventions have been implemented in the past e.g. replacement of columns or spandrels, it is considered that these interventions are likely to change the movement regime again i.e. a previously seized connection may be renewed or replaced resulting in increased provision for movement locally to the intervention. Photograph 3 below indicates this process where 2No. bays had been removed in 2014 releasing the area and attracting movement.



Photograph 3 - indicating 2014 trial bays following removal of spandrels forming a 'release' to the structure. Note how steel beam has buckled in the weak axis where insufficient capacity is available to resist restraint forces to horizontal movement.

4.8.8 Likewise, where aluminium spandrels (and perhaps SG Iron) have been used as a replacement these might be stronger / stiffer and attract load.

4.8.9 Due to the historical nature of the structure, there is limited opportunity to introduce movement joints or radically alter the existing structural articulation without significant intervention.

4.8.10 SCI 138 notes where structures are anchored (stabilised in this case) at infrequent intervals there is less 'self-containment' in each bay and that in such cases additional tying elements may be introduced.

4.8.11 The rich history of defects to spandrels leads to the conclusion that some are ineffective, and others cannot be fully relied on for continued service. To mitigate the defects to spandrels and enhance stability (put less reliance on distressed spandrels) it is proposed to Introduce supplementary tying action into the deck diaphragm by positive connection into the east cliff wall. This intervention will be largely concealed from view and incorporated into the reinforced concrete deck at truss bearing blocks. This approach maximises the opportunity to reuse existing spandrels. Further it reduces bending action applied to columns thereby enhancing opportunities for column reuse also.

4.8.12 Figure 7 below summarises the tying proposal:

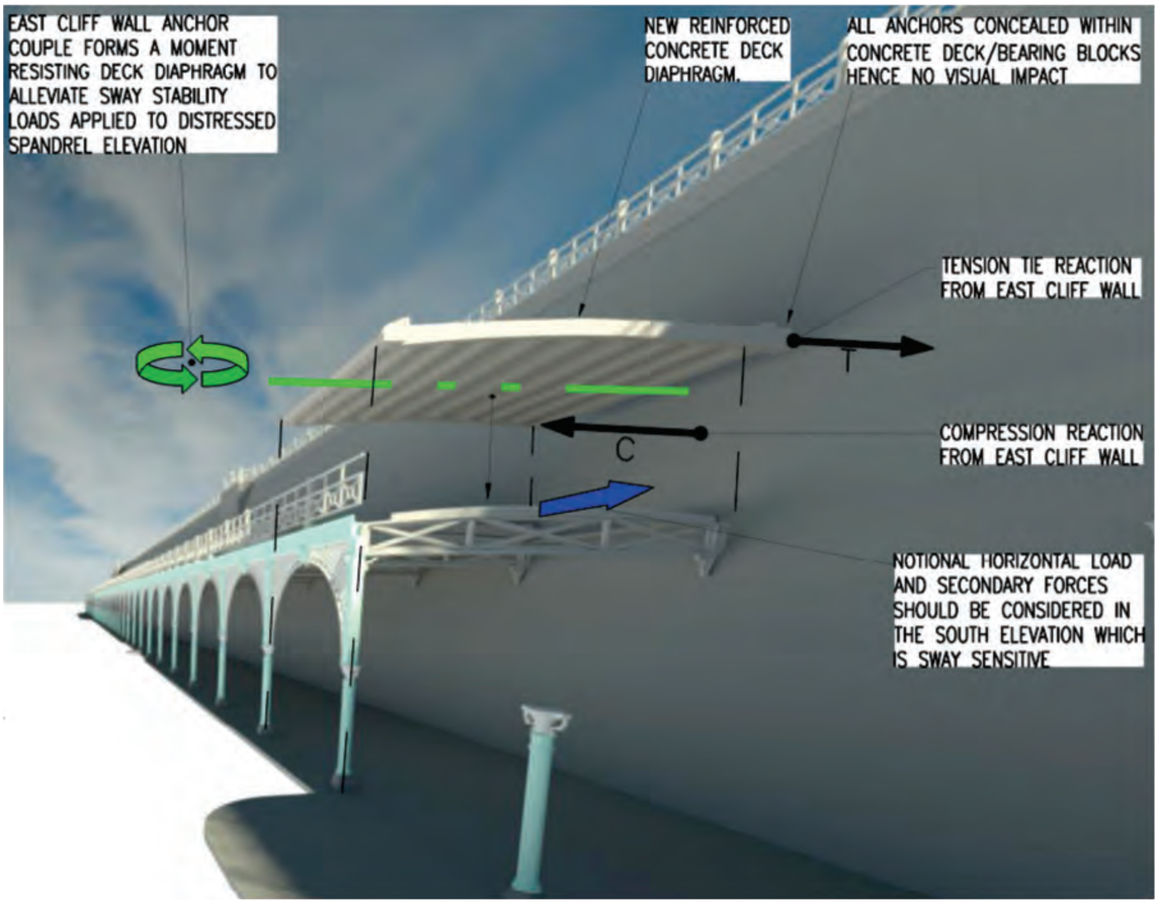


Figure 7 – Sketch indicating structural reasoning for introduction of deck diaphragm tying action

4.9 Columns & Spandrels

4.10 Tying Action Over Supports

4.10.1 Tying action across the transverse bearings within the depth of the replacement slab is also considered necessary to more evenly distribute the slab loading from a series of point loads to a near uniform distributed load to alleviate bending induced into the top chord of the trusses in the event of fracturing.

4.10.2 Accidental impact loading is also a consideration. Madeira Drive is a live highway and occasionally hosts special events, including speed trials. The design team have identified the risk of a vehicle strike to a column. Due to the nature of brittle cast iron failure, these columns have little ability to resist such a strike. The public realm nature of Madeira Drive doesn't lend itself to the incorporation of heavy-duty barrier type protection. The landscape architectural design has however, incorporated mitigations that offer some protection to columns in the form of verges and kerbing. Where level thresholds are required, it is understood localised bollard protection is being considered.

4.10.3 In the interest of providing enhanced robustness to mitigate the risk of a vehicle strike to a single column resulting in development of a progressive collapse scenario, it is proposed to incorporate continuity to the eaves beam over columns to provide alternative load paths for the slab to span over the loss of a column in the accidental case.

4.10.4 Both these tying provisions are incorporated into the new reinforced concrete deck and are consistent with advice to incorporate tying action and redundancy given in SCI 138.

4.10.5

4.10.6 Deck diaphragm tying action alleviates sway stability demands put on spandrels, which are known to suffer problems. It is anticipated that provided existing spandrels can be made good by fusion welding they can be incorporated back into the works on an as existing basis. There are several spandrels recorded as having been replaced in aluminium and perhaps more modern iron casting. It is recommended as careful dismantling progresses that the interface between Aluminium and Iron components is inspected for evidence of localised galvanic corrosion associated with dissimilar metals. Should this be found to be a problem, consideration should be given to recasting theses aluminium spandrels in SG iron.

4.10.7 Deck diaphragm tying provides secondary moment resistance to eccentrically loaded columns and hence it is anticipated that column reuse will be possible following refurbishment of individual members. Care should be taken to identify any historical cold stitching repairs (Metalock) and to remediate these with fusion welding to full parent metal strength. Similar to spandrels some columns are recorded as having been replaced in Aluminium. These might reasonably be reused where no bimetallic corrosion is identified during dismantling alternatively recast.

5.0 DISCUSSION (REUSE STRATEGY)

5.1 General

5.1.1 Given the possible heritage impact of removing and replacing substructure elements, a positive strategy for the conservation of elements is required. Reference the National Planning Policy Framework (NPPF), February 2021, Paragraph 200. "…*Any harm to, or loss of, the significance of a designated heritage asset (from its alteration or destruction, or from development within its setting), should require clear and convincing justification…*"

5.1.2 This section documents the assessment strategy to be implemented in determining the potential reuse of structural elements removed as part of the works. It is also proposed that the strategy discussed could be adopted for other areas of The Terraces with regard to future refurbishment works. It is noted that The Terraces have a history of interventions as is to be expected and the continuation of this strategy into its future is mitigated to some degree by the interventions proposed, however continued servicing throughout the residual life of the structure is to be expected.

5.2 Reuse Assessment

5.2.1 To assist in determining if an element is suitable for reuse, an 'Iron Reuse Flow Diagram' is provided in Appendix E. This has been developed to highlight a 'step-by-step' process which can be applied on site works to enable interested parties to understand the processes at hand. Ultimately the structure must perform reliably at its primary function to sustain the loads or actions to be applied during service.

5.2.2 The assessment process is led by three main contributing factors, these being to:

i) Maintain the serviceable life of the structure and principally structural load carrying capacity and stability,
ii) Ensure safe public access,
iii) Conserve and prevent loss or harm of significant heritage assets.

5.2.3 Each element or component is recovered for detailed assessment. The reuse process, as indicated in the Process Flow Diagram is defined by assessment stages / work items. It is important to note that for a full assessment to be practical, this will involve multiple disciplines including specialist foundry assistance, non-destructive testing and cast iron repair specialists. At the end of each stage, a simple 'Yes' or 'No' question is asked as to whether the element passed or failed the respective level of assessment.

5.2.4 There follows a summary of works to be undertaken for each Assessment Level:

Note that all elements contained within Section 1 of 6 bays of arches are to be dismantled and tested. The outline process below is to be refined for the remainder. Section 1 assessments will be carried out in conjunction with the Design Team, Client Team and the Statutory Authorities. Please refer to HOP Refurbishment of Existing Ironwork report for details and Purcell Heritage and Design Summary Note.

Level 1 Assessment – Visual on site

5.2.5 Upon recovery, a close visual inspection and initial assessment will be undertaken. The Level 1 Assessment will identity significant and obvious structural defects such as: holes, fractures, casting defects, loss of section, necking etc. A 'Reuse check sheet' corresponding to the component is to be completed along with photographing, cataloguing and labelling.

5.2.6 Data collected will inform an initial Engineering assessment to determine the residual structural capacity that is likely to exist and confirm if repair is likely viable. For this assessment, reference will be made to the structural appraisal works discussed previously. Details to be shared with the fusion welding specialist to seek confirmation that repair remains viable.

5.2.7 The site test should also differentiate between likely Aluminium and Iron components by a practical weight test on site undertaken during handling in conjunction with hand magnet test. Where aluminium is identified carefully inspect the junction of dissimilar metals for localised bimetallic corrosion.

5.2.8 The conclusion of the assessment is either 'Pass' and move to the Level 2 Assessment or 'Fail' and move to the Re-purpose/Re-cycle, Disposal section.

5.3 Level 2 Assessment - Engineering Appraisal at Offsite Facility

5.3.1 Assuming a component progresses to Level 2 assessment, due to the mixture of materials used throughout the structure, and the need for the fusion welding process to match the welding electrode to the parent metal, the element is to be tested to determine metal composition. This will determine if it is formed from Grey Cast Iron, SG Iron or Aluminium where not identified from level 1 Assessment.

5.3.2 Tensile and compressive test samples will be recovered and samples tested to BS EN ISO 6892-1:2019 for tensile and compressive strength. Tensile test results shall be added to the site wide testing population and statistical analysis updated such that confidence in the respective strengths can be improved as works progress, to maximise the opportunity for reuse. Tensile test results from trusses should be analysed separately to determine if they were originally formed from a higher-grade iron than say balustrade infills, noting these are particularly sensitive to tensile strength due to the vulnerability to brittle failure mode. Anecdotally trusses seem of higher quality casting, hence could be a higher grade material also.

5.3.3 Tensile and metallurgic composition results are to be used to determine which type and grade of iron is used for the component.

5.3.4 Surface corrosion and finishes are to be removed by grit blasting and cleaning. This exposes the surface and enables more detailed assessments by Magnetic Particle Inspection to identify fracturing, potential casting defects, previous cold metal stitching repairs, and accurately record geometry.

5.3.5 The results of the Level 2 Assessment are to be fed back to the Engineer to assess whether test results are more or less onerous than expected.

5.3.6 Initially, due to the variability and unknown nature of material properties, testing frequency is to initially comprise of 100% of components. Depending on the results, if a trend is identified that material properties are consistent then testing frequency may be revised. This will need to be informed by actual results at the discretion of the Engineer / Architect / Project Manager, however forecasting the likely outcome at this stage might be say 100% inspection for the first 5No. bays. Where consistent results are obtained this might be relaxed to say 50% of components. Should consistency of test results continue then this might be reduced to 20%-30% of components, all subject to review. Should results yield inconsistency then 100% testing may not be justified to depart from.

5.3.7 The conclusion of the assessment is either ‘Pass’ and move to the Level 3 Assessment or ‘Fail’ and move to the Re-purpose/Re-cycle, Disposal section.

5.4 Level 3 Works - Engineering Appraisal

5.4.1 Update engineering assessment for the component informed by actual material test results to determine if the component has adequate theoretical capacity to justify reuse.

5.4.2 The concluding dilemma of the assessment process is ‘does the element have capacity to support expected loading’ (actions). ‘Yes’, element can pass to next level of assessment 'No', concludes the element is unsuitable for reuse in the structure and alternative reuse options are to be considered.

5.4.3 The conclusion of the assessment is either 'Pass', element suitable for reinstatement, move to the Level 4 Assessment or 'Fail' and move to the Re-purpose / Disposal section.

5.5 Level 4 Works – Repair

5.6 Where deemed suitable component to proceed to repair by cast Iron Fusion welding with associated quality checks and warrantees, any problems or issues with the repair process to be reported to the Engineer / Architect / Project Manager for consideration.

5.6.1 The conclusion of the assessment is either 'Pass' and move to the Level 5 Assessment or 'Fail' and move to the Re-purpose/Re-cycle, Disposal section.

5.7 Level 5 Works Physical Load Test

5.7.1 Depending on the results of previous levels of Assessment the component may be theoretically be justified for reuse. However due to the lack of confidence in the original materials and workmanship initially 100% of components shall be physically load tested to verify adequacy at an offsite facility using kentledge or water ballast.

5.7.2 Strain gauges and instrumentation to be used in the normal way with the component taken up to working load.

5.7.3 Depending on the results, if a trend is identified that material properties are consistent then testing frequency may be revised. This will need to be informed by actual results at the discretion of the Engineer / Architect / Project Manager however forecasting the likely outcome at this stage might be say 100% load testing for the first 5No. bays. Where consistent results are obtained this might be relaxed to say 50% of components. Should consistency of test results continue then this might be reduced to 20%-30% of components, all subject to review. Should results yield inconsistency then 100% testing may not be justified to depart from.

5.7.4 The conclusion of the assessment is either 'Pass', element suitable for reinstatement or 'Fail' and move to the Re-purpose/Re-cycle, Disposal section.

5.8 Repurpose / Disposal Options

5.8.1 Below are tabulated options considered available for removed structural elements that fail the assessment process and hence are unsuitable for incorporation into the refurbishment.

Proposal	Description	Advantage	Disadvantage
Re-Purpose (Non-structural)*	Consider alternative usage such as exhibiting or incorporation in landscaping works.	Preserves the historical heritage of the element and improves / enhances the local community.	Element is not retained in its original location.
Re-Purpose (Structural)*	If local failure of the element restricts it ability for reuse, consideration should be given to cutting down or shortening the element to enable reuse.	Seeks to minimise the possible requirement for full element disposal off site.	The full element is not retained and there is limited opportunity to incorporate into the works.
Disposal*	Disposal of element off site	Can be managed into appropriate waste streams and recycled.	Element is not retained in its original location or preserved.
Dispose (Artefact)*	Elements sold as whole artefacts or broken down into small parts prior to sale.	Encourages public support for the refurbishment of the structure and promotes interest in the conservation of listed structures / buildings.	Element is not retained in its original location and/ or may not be preserved on site.
Reuse as stock	Refurbished element made available as a stock item for reuse in future phases of refurbishment works.	Preserves and mitigates the requirement for replacement elements to be installed.	Refurbished element unlikely to meet dimensional requirements of future phases and hence no guaranties for reuse can be made.

Table 3 – Options to be considered if elements do not pass assessment levels.

*Note: Prior to removal from site ALL elements shall be photographed and recorded as part of a Historical Building Record.

6.0 PRELIMINARY CONCLUSIONS & RECOMMENDATIONS

6.1 General

6.1.1 This report forms a compendium of the various historical interventions and relevant investigations available at the time of writing. It sets out to collate the rich history of inspections and reporting on condition of The Terraces to test the approach to refurbishment.

6.1.2 As works progress opportunities exist to update the preliminary conclusions drawn in this report. It is anticipated that this report will therefore require periodic update as works progress, and more objective evidence becomes available to inform iron reuse options.

6.1.3 There are limitations in what can practically be achieved in terms of identifying the various known and suspected unknown historic repairs, defects, material properties and original workmanship. There remains a need to carefully dismantle, inspect, test and assess individual iron components to maximise opportunity for reuse.

6.1.4 There is consistency in the various engineering reports and opinions available that have both implemented and predicted a requirement for periodic interventions to The Terraces over many years as well as the need for continued maintenance to maintain integrity and future service. As ever there is always the prospect that hidden strength is available that cannot be reasonably determined through theoretical calculation.

6.1.5 The structure is complex and has a high dependency on the reliability of Grey Cast Iron members with a well-documented history of failures and interventions particularly those associated with sudden and brittle failure of Grey Cast Iron. Whilst some testing is available the population of tensile testing results should be improved as refurbishment progresses.

6.1.6 Bringing the structure back into service is considered to achieve significant substantial benefit and optimum viable use that outweighs the minimal harm proposed, thereby providing clear and convincing justification and balanced judgement of proposals.

6.1.7 The factor of safety against failure of the structure is well documented as being below that required by current standards for many years. Refurbishment of the deck was forecast as far back as 1983. This forecast and planned intervention provide the proposed works a significant milestone in the structures life to incorporate stability enhancements to both reopen the structure and improve the sustainability of maintenance for continued service in to the future.

6.1.8 Proposals are considered consistent with recommendations in SCI 138 ‘Appraisal of Existing Iron and Steel Structures’.

6.2 Lattice Beams (Trusses)

6.2.1 Historical evidence suggests that trusses have a factor of safety below that generally accepted as adequate.

6.2.2 There is a history of brittle failure of elements in tension fractures to bottom chord and internal struts un-associated with bolt hole corrosion initiation. There is also extensive bolt hole initiated fracturing to the top compression chord. Some of these failures have been repaired with inappropriate structural repair techniques historically such as cold stitching.

6.2.3 From the limited tensile testing results currently available, sensitivity analysis concludes it is marginal that an adequate factor of safety can be achieved by reuse of Grey Cast

Iron trusses. At the time of writing, it is understood asset owner is considering if load limitations are reasonable and practical to consider in terms of management. There are limitations on what can practically and actually be achieved in terms of reliable management and enforcement of load limitations in such an external exposed structure.

6.2.4 Ultimately trusses require careful inspection and assessment to validate these preliminary conclusions as works progress to maximise potential for the reuse of components where safe to do so.

6.2.5 It is recommended that the prospect that trusses fail the assessment process is borne in mind and that contingency is made for the recasting of trusses in SG Iron where reuse is unviable.

6.3 Balustrades

6.3.1 Brighton's Seafront railings are to a historic design that does not meet current standards. Where railings have been replaced in the past, opportunity to make improvements have been taken, though in a manner sympathetic to the original design.

6.3.2 There is a need to enhance the safety of the seafront railings including those at The Terraces and that it is practical to incorporate such necessary enhancements into a capital refurbishment project. The Terraces project provides this opportunity.

6.3.3 It seems reasonable that the iron pattern for balustrades is stretched to achieve 1.1m above adjacent paved surfacing level and that these are recast in SG iron to eliminate the foot detail which forms a tripping hazard and achieve adequate containment strength.

6.3.4 Theoretical assessment of infill panels informed by site specific tensile testing highlights some departures in terms of strength and containment capacity of balustrades which will need to be accepted by the asset owner if existing infills are to be incorporated back into the works. This is not recommended as safety is considered paramount.

6.3.5 It is recommended that pilot physical load testing of infill panels is completed to maximise the potential for infill reuse. Physical load testing has therefore been incorporated into section 1 works to seek to exploit any hidden strength that may be available to justify reuse. The prospect of recasting infill panels in SG iron should not be ruled out at this stage.

6.4 Columns

6.4.1 Little is available currently in terms of columns and these require careful assessment. There are some columns that are recorded as having been recast in aluminium and some that are fractured or repaired.

6.4.2 Proposed structural tying enhancements incorporated into the new deck to deal with disproportionate collapse and robustness will reduce the applied bending to columns.

6.4.3 Columns are predominately compression members and therefore if bending (tension) is reduced this goes some way to improve the likelihood that they can be justified for reuse following assessment.

6.4.4 At this stage and prior to more detailed assessment, as and when additional information becomes available, column reuse seems likely.

6.5 Spandrels

6.5.1 Proposed structural tying enhancements incorporated to deal with disproportionate collapse and robustness will reduce the contribution spandrels make to stability in terms of portalisation, thereby maximising potential for reuse.

6.5.2 Initial grit blasting trials have identified a number of fractures. The projects cast iron repair specialist is currently confident these can be repaired to original parent metal strength.

6.5.3 Given load on spandrels is reduced slightly at this stage and prior to more detailed assessment as and when additional information becomes available spandrel repair and reuse seems likely.