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Madeira Terrace
Brighton

[redacted]
CEng FIMechE MIOA
Director

Dynamic Appraisal Trial

For

Brighton & Hove
City Council

Project: **11983**

Date: **May 2023**

Prepared By: [redacted] *BSc(Hons) CEng MIStructE IHBC*
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1.0 Introduction

1.01 Mann Williams were commissioned by Brighton and Hove City Council to carry out an initial inspection of the historic Madeira Terrace located on Brighton Seafront.

1.02 Concerns had been raised in relation to the level of confidence that existed in the ability of the existing historic ironwork structures to be retained, refurbished and, where necessary repaired.

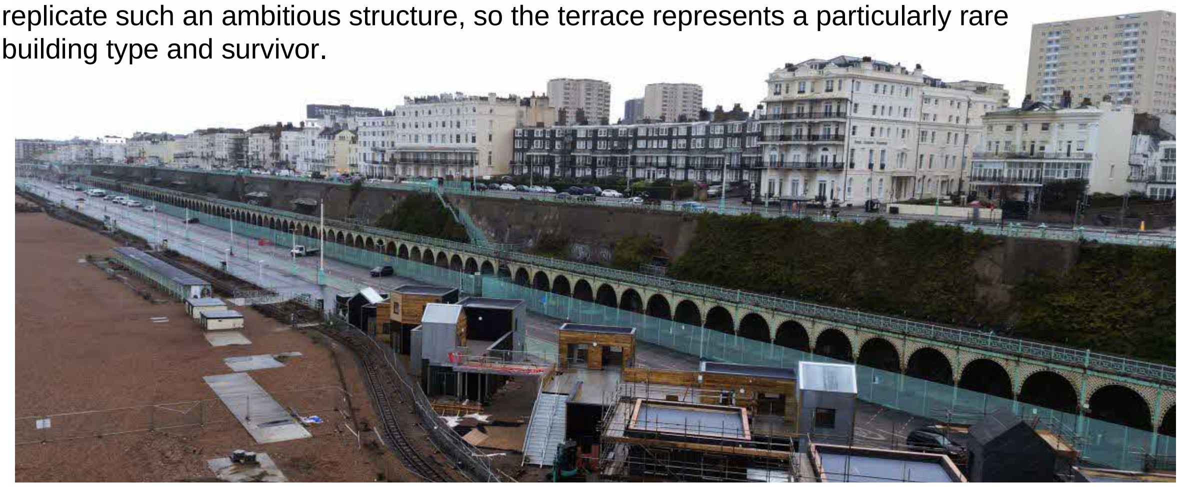
1.03 The inspection was carried out on 17th to 19th April 2023. The inspection consisted of a stage of trial testing using dynamic excitation techniques with the objective of gaining a better understanding of the performance characteristics of the Madeira terrace structure.

1.04 The aim of the trial was to assist with and enable informed decisions to be made by the project team on the proposed renovation and conservation works, and to have a better understanding of how Dynamic Testing might benefit the Madeira Terrace project going forward in providing further understanding of the performance and capabilities of the existing structure.

1.05 The objective of the testing is to establish the predicted performance of the structure under design loadings of 5 kN/m^2 imposed deck load and 3 kN/m imposed line load on the balustrades.

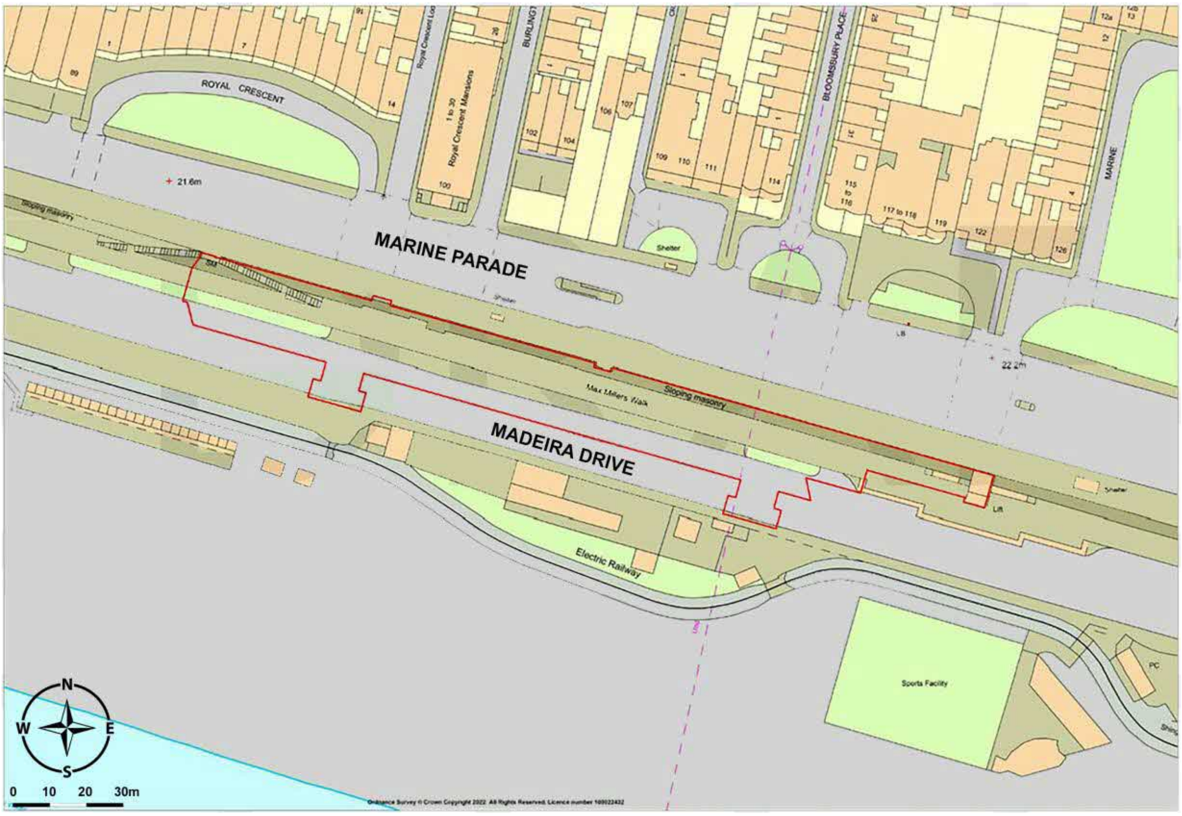
2.0 Madeira Terrace Structure

2.01 Madeira Terrace, illustrated in the arial view looking west, was designed in the 1880s by the Brighton Borough Surveyor Philip Causton Lockwood as a sheltered walkway with an integral shelter hall and three stage lift which connected Madeira Drive and the beach with the upper walkway and Marine Parade. It was extended in the 1890s by Lockwood's successor, Francis May, and at over 865 metres it is thought to be the longest and oldest continuous cast iron structure in the country. Late Victorian and Edwardian seaside resorts chose not to attempt to replicate such an ambitious structure, so the terrace represents a particularly rare building type and survivor.



2.02 The structure is statutorily listed at Grade II* placing it in the top 5.8% of the nation's listed building stock. The retention of its historic fabric would be an expectation of any proposed renovation and replacement of any or all of the key structure would be most-likely resisted by Historic England.

2.03 The location of the first phase of proposed renovation and conservation works is illustrated on the key plan below. This initial phase of dynamic assessment of the structure has focused on this area of the site.



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3.0 Current Condition and Scope of Assessment

3.01 Concern has been raised that the extent of corrosion and local failures of some elements of the historic ironwork structure is of a magnitude that requires extensive replacement of structural elements.

3.02 The scope of this report is to carry out an independent assessment of a sample area of the Madeira Terrace structure to determine if Dynamic Assessment is capable of demonstrating an adequate level of confidence that the structural ironwork can be retained and reused.

3.03 It is an additional objective to identify any areas of structural performance that are not characteristic of the general structure, as such anomalies may indicate local defects, failures or weaknesses that require further investigation.

3.04 It is noted that this dynamic assessment is a trial of limited site time (3 days) with the objective of establishing the potential for the methodology and technique to justify retention and reuse of the existing structure.

4.0 Dynamic Load Testing Methodology

4.01 The process of appraisal of existing structures is conventionally carried out by either a theoretical calculation based on known structural properties or by load testing. Both methods have disadvantages. A theoretical appraisal requires knowledge of the structural elements within the structure and the current condition of those components. A load test is disruptive and expensive to carry out with particular difficulties in accurately measuring deflections. It also has potential to cause damage to historic fabric. It is also generally a method that is only able to test a sample area of the structure, which is then assumed to be representative of the whole structure. This can have risks of '*missing'* areas that have hidden weaknesses.

4.02 To overcome these disadvantages Mann Williams and Eatec Dynamics have successfully developed and utilised an innovative non-invasive dynamic assessment method that is faster to carry out and provides additional information about the actual performance of the structure being assessed. The images opposite show the two test rigs utilised on the trial phase of testing at Madeira Terrace. The balustrade rig$_{(1)}$ provides a lateral excitation to the structure and the deck/floor rig$_{(2)}$ provides vertical excitation.

4.03 The methodology consists of the application of low magnitude sinusoidal loading into the structure at a known frequency and measuring the response amplitude from the structure at key locations. From the data obtained a finite element model is produced that accurately matches the recorded characteristics of the actual structure. The completed computer model is then able to be used to establish and accurately predict performance and load carrying capacity.

4.04 The technique has been tested and proved on a range of structures for clients including Historic England, Cadw, National Trust and many other national organisations.

National Trust   Historic England   DfC Department for Communities Historic Environment Division   OPW Oifig na nOibreacha Poiblí Office of Public Works   Cadw

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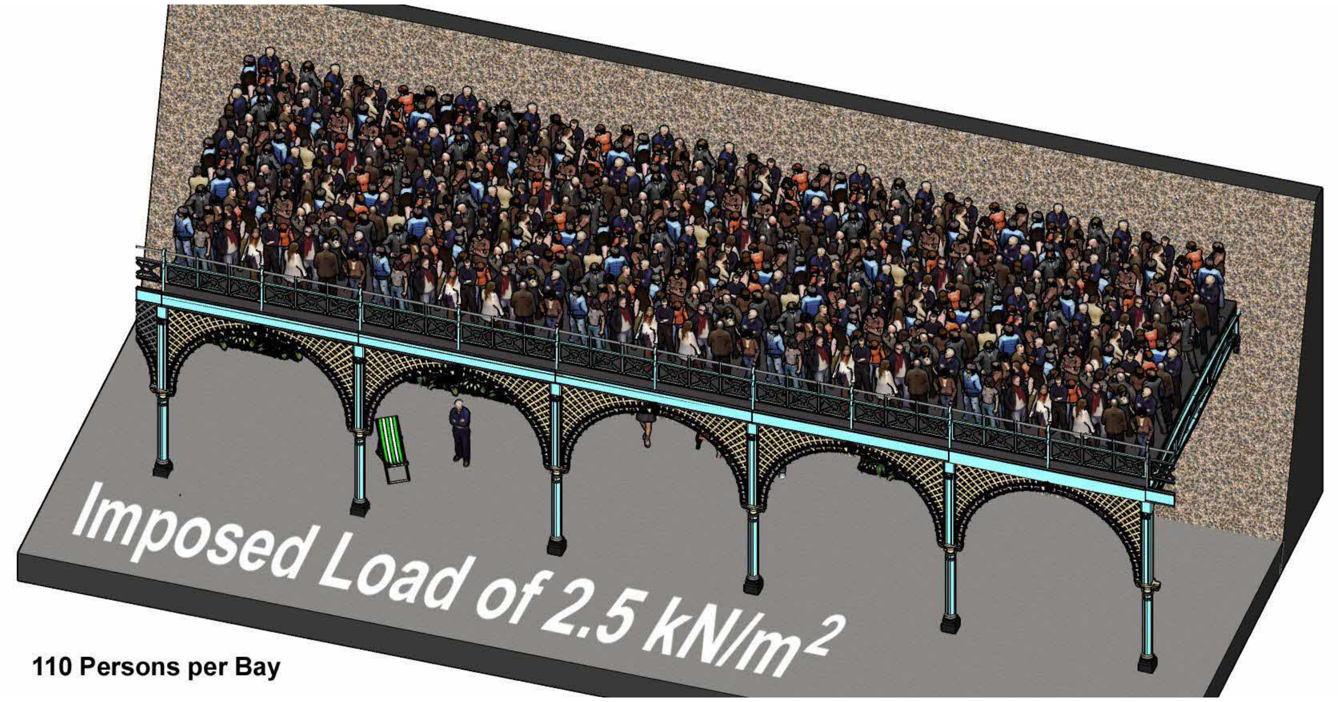
5.0 Loading

5.01 The Terrace structure loading has been stated as requiring an imposed deck load of 5 kN/m^2 and for balustrade handrails to be capable of sustaining a horizontal imposed load of 3 kN/m

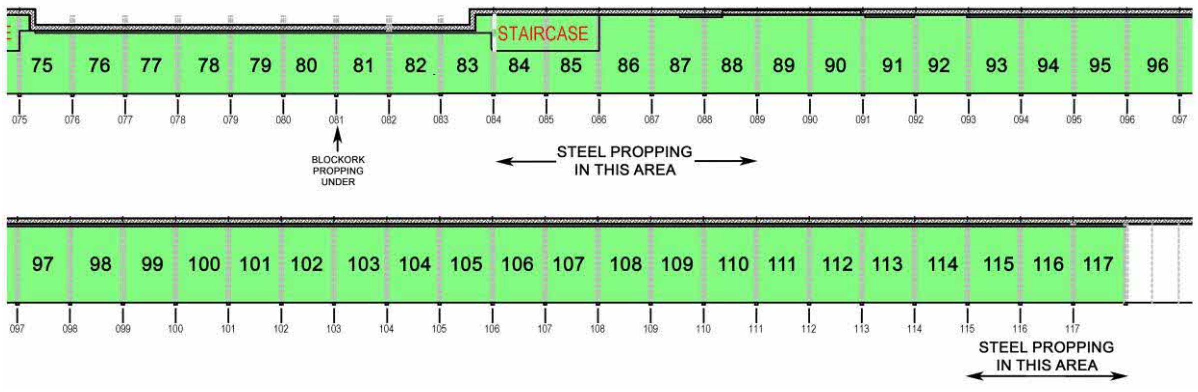
5.02 When considering any existing historic structure there is a need to understand the basis for design loading. Existing structures will inevitably have been exposed to 'working loads' over their working lives, and although consideration of any areas of deterioration is important and necessary they will have proved themselves 'fit-for-purpose' through service loading. Design loadings provided in current codes-of-practice should be considered for guidance purposes and not automatically a mandatory requirement. They should simply form part of the overall design appraisal process.

5.03 The image below illustrates a typical crowd loading that equates to 2.5 kN/m^2 on the Madeira Terrace deck structure. This equates to approximately 110 persons on each bay and in excess of 16,000 persons on the terrace as a whole. Crowd densities in excess of this figure would be considered a risk to the public from crushing. Whilst the appraisal of the structure within the scope of this dynamic assessment has progressed based on the full 5 kN/m^2 imposed load it is considered to be a particularly onerous and possibly excessive design criteria.

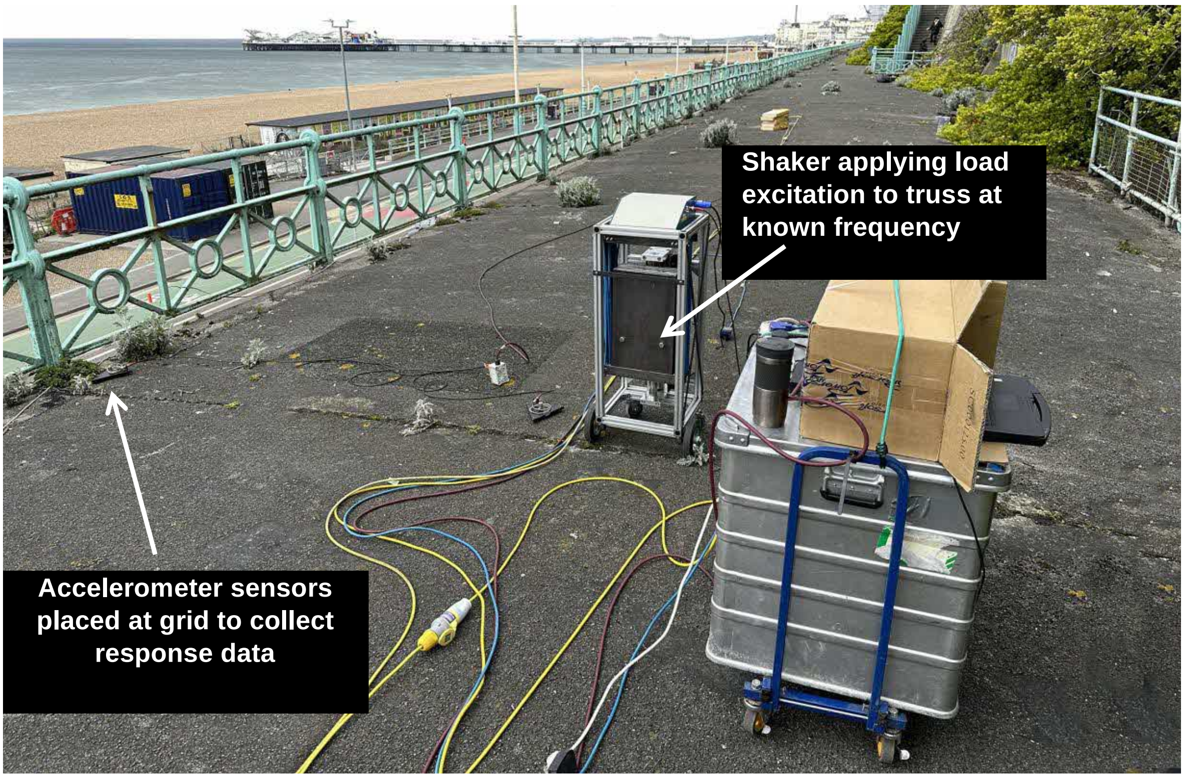


6.0 Assessment of Main Trusses

6.01 The dynamic testing trial consisted of a three day phase of assessments, collecting data to enable initial understanding of the structural performance to be developed. The key plan below provides the grid and bay referencing used for testing and reporting.



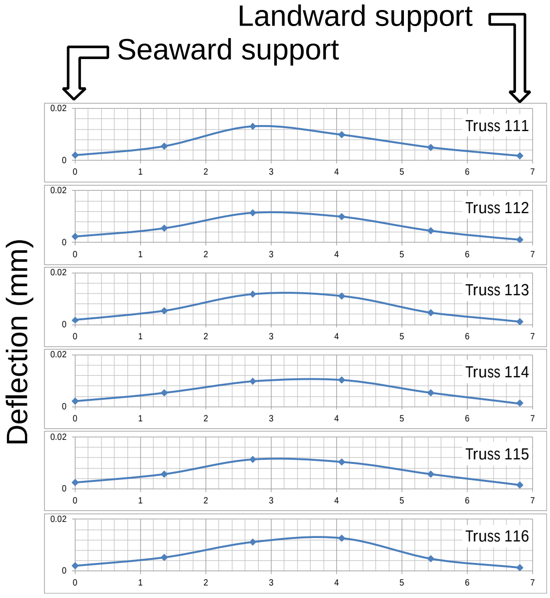
6.02 To assess the main trusses the deck structure was excited on each truss line as shown in the image below with response data collected at regular spacings along the line of the truss.



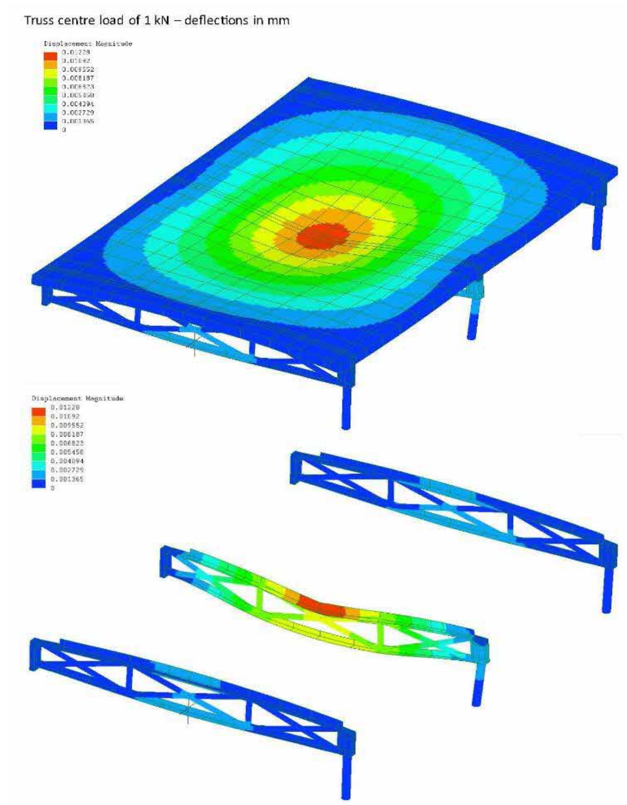
6.03 Data was collected from Trusses 87-116

6.04 The image opposite shows the configuration of principal cast iron truss supported on the cliff wall to the right and cast-iron column to the left (seaward) side. The current iron filler joist/concrete slab deck spans over the trusses.

6.05



The testing provided direct stiffness characteristics at each truss line for 1kN unit point load at mid span, with a sample of the output shown opposite. The full output of stiffness results for the tested trusses is provided in appendix B.



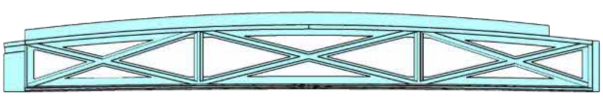
6.06 The data obtained enabled a Finite Element model of a typical truss to be produced with E values adjusted to match the actual site performance data obtained.

6.07 Initial modelling of a bare truss as an isolated element showed significantly greater predicted flexibility than the data gained from the site trial dynamic testing.

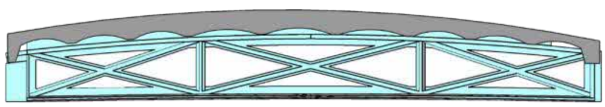
6.08 Modelling of the truss with an allowance for a zone of concrete decking providing composite action produced a truss simulation that matched the data obtained for most of the sampled trusses.

Bare Truss
Modelling showed significantly greater flexibility and was not representative of site conditions revealed by testing.

Truss with composite action from slab
Modelling with deck slab included was able to match stiffness obtained from site testing



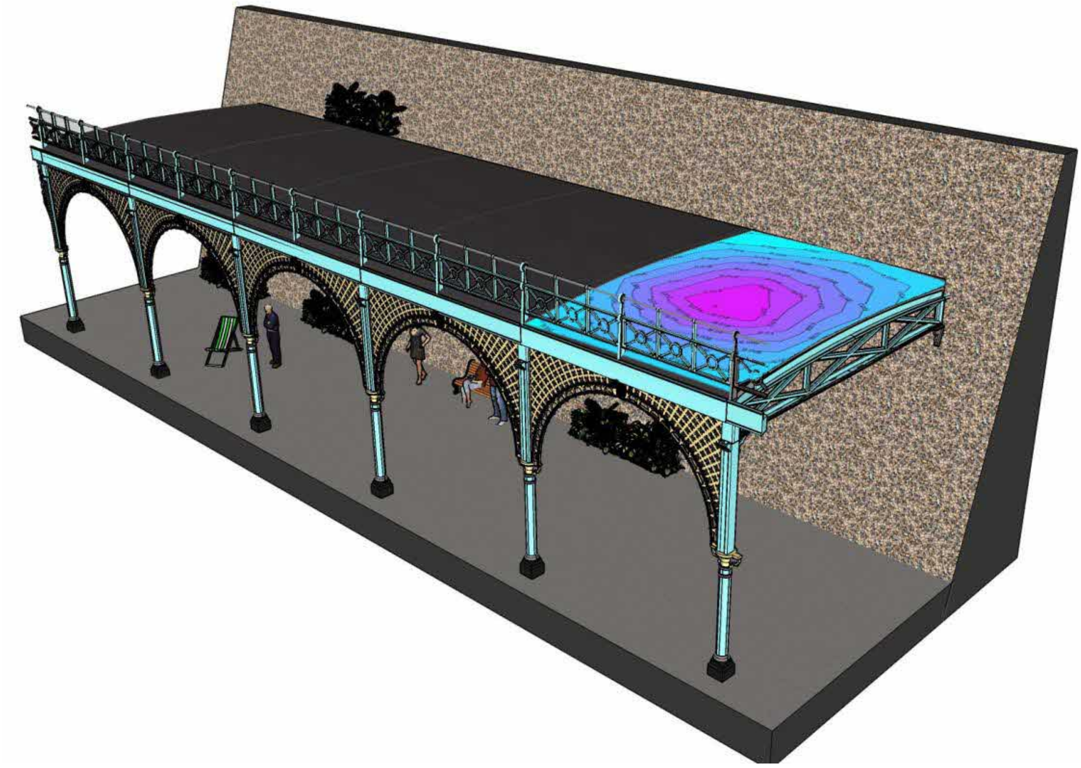
0.06mm



0.012mm

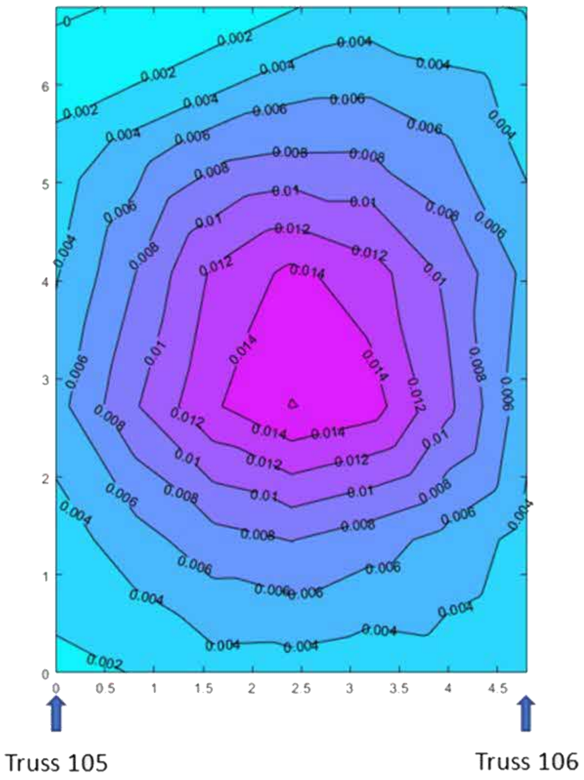
Deflections for 1 kN mid-span unit point load

6.09 The Finite Element model was further verified by carrying out a test loading in the centre of a span and measuring the deflected shape between two adjacent trusses. A similar load was applied to the Finite Elemsnt model so that the predicted shape could be compared with the measured shape.

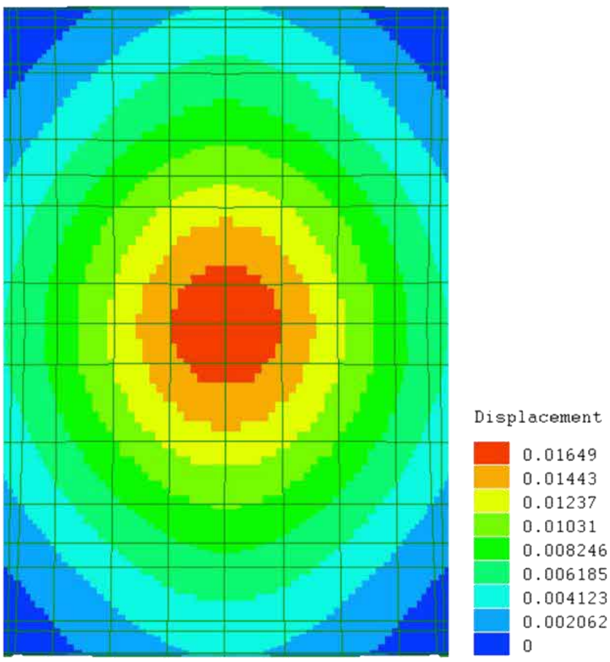


6.10 The results showed excellent correlation between FE Model predictions and the actual performance data obtained from site testing as shown in outputs below.

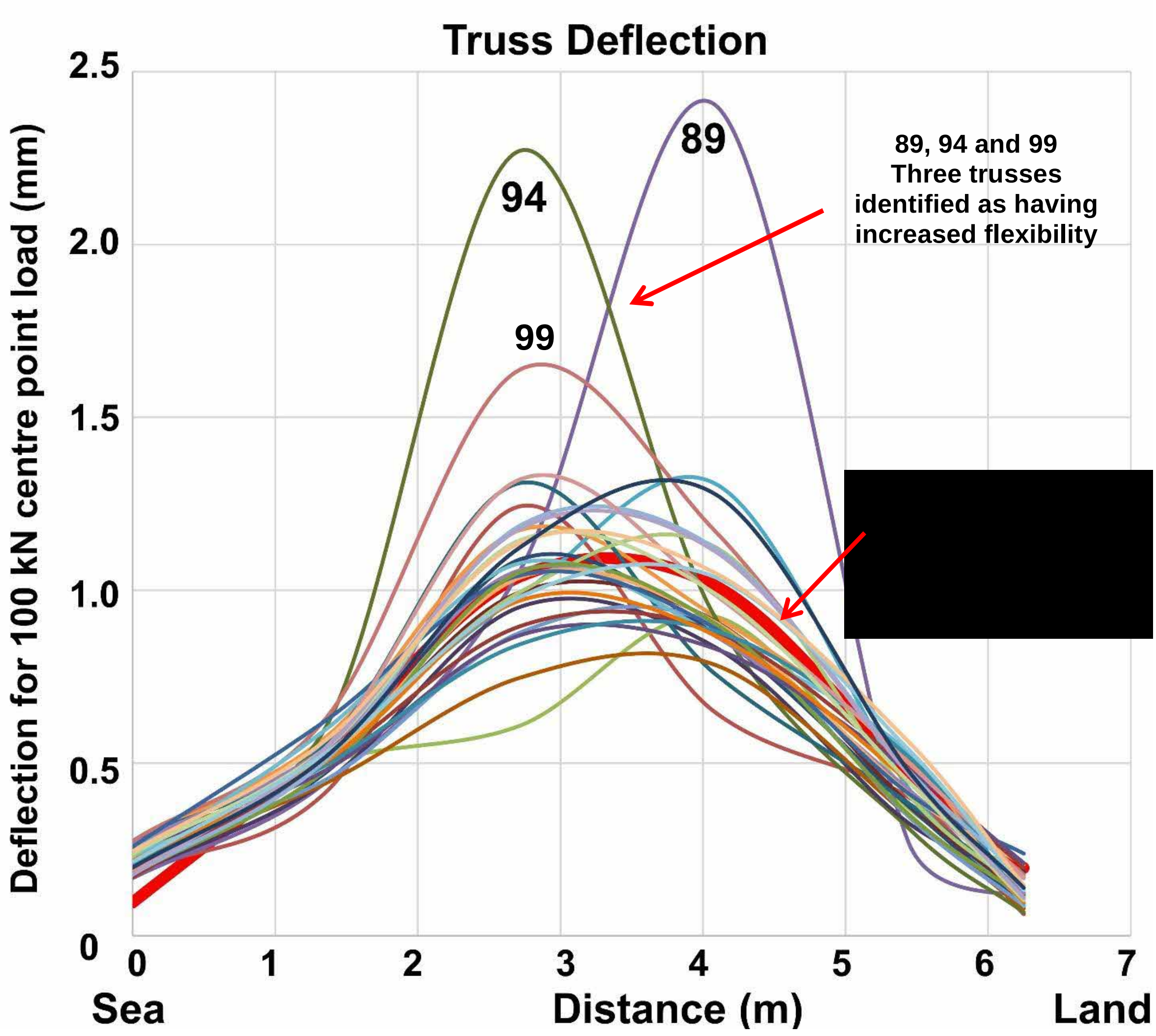
Site Test Results



Finite Element Model Predictions



6.11 The graph below shows the performance predicted by the Finite Element model with the majority of trusses closely matching. In the trial testing trusses 89, 94 and 99 showed increased flexibility characteristics suggesting a potential weakness or deficiency.

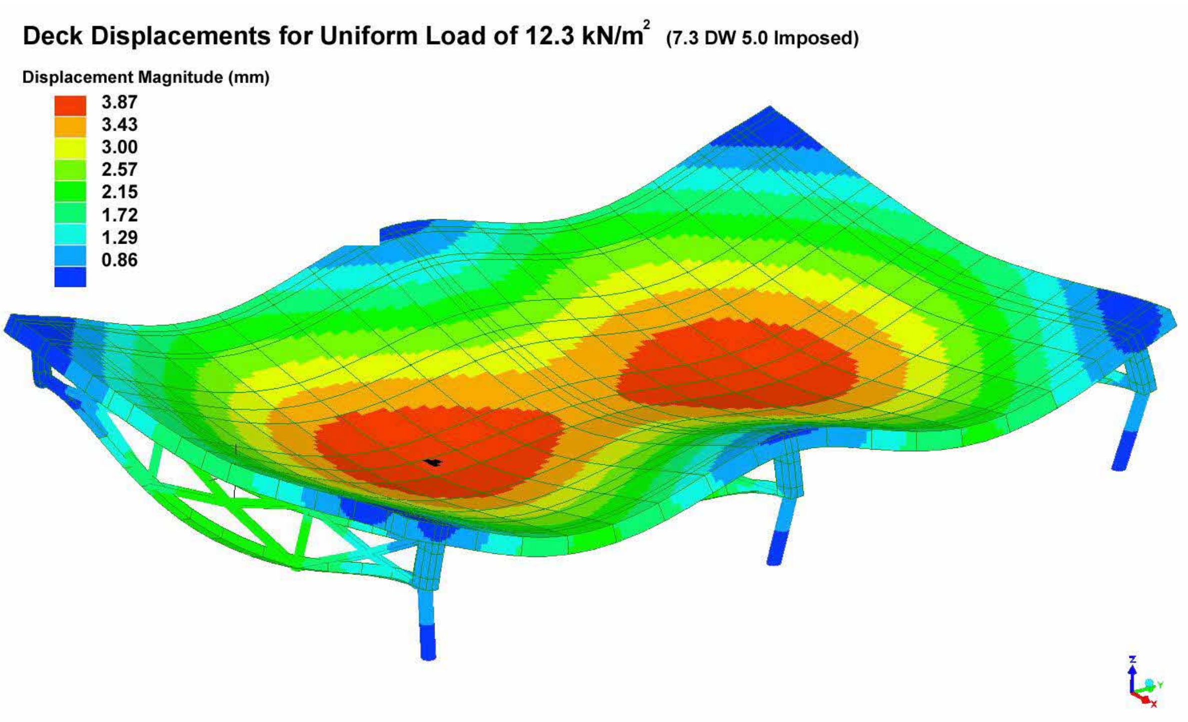


6.12 The trial testing demonstrates the ability to identify areas of potential weakness or defects and enable further testing or inspections to be focused and targeted. For trusses shown to be performing to a more consistent response characteristic the level of confidence in their integrity is enhanced.

6.13 The anomalies revealed in truss positions 89, 94 and 99 may be a result of a number of factors that may include corrosion, element failure in the truss or loss of composite action due to a deficiency in the deck. Further investigation would be required in these areas.

6.14 With a verified Finite Element model produced the structure was analysed for the stated imposed deck load of 5 kN/m^2.

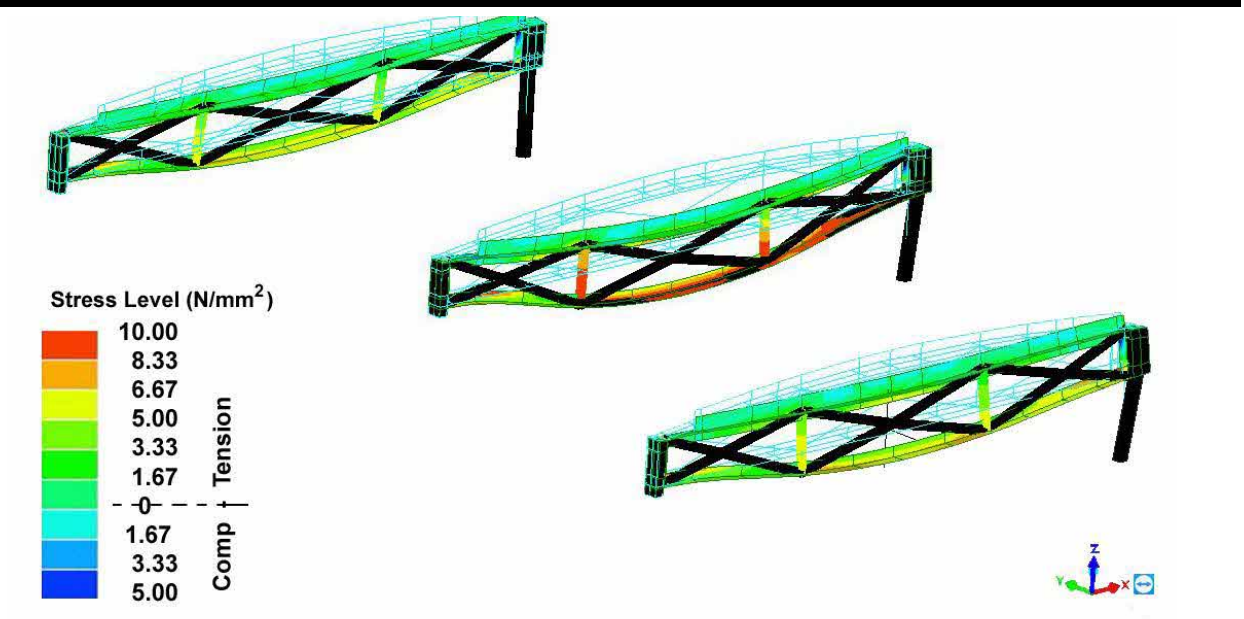
6.15 The output below shows the displacement characteristics predicted under full imposed load. The figures predicted of under 4mm deflection were considered acceptable.



6.16 Under full design load applied across two adjacent bays the stress levels for the truss elements were established and illustrated in the following sections. Bracing members were modelled as beam elements and the stress levels recorded was a maximum of 23.12 N/mm^2 tension and 38 N/mm^2 compression.

6.17 Under full design load applied across two adjacent bays the stress levels for the truss elements were established and illustrated in the following sections. Principal members were modelled as shell elements and the stress levels recorded was a maximum tension of 10 N/mm^2.

[redacted]



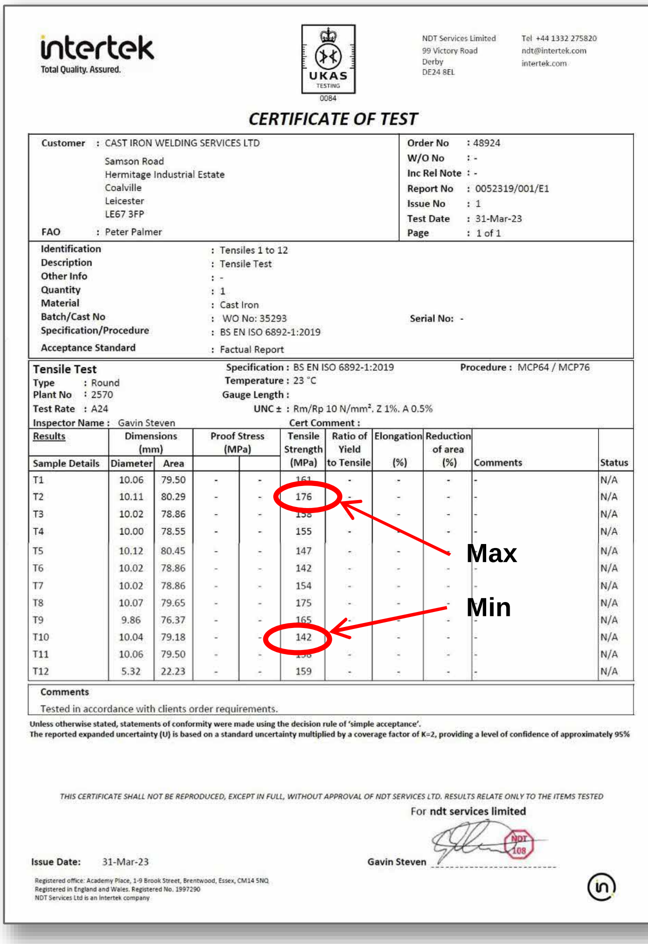
6.19 Samples from the cast iron available on site has been tested (by others) and the results are provided in appendix A.

6.20 The results quote a tensile strength range from 142N/mm^2 to 176 N/mm^2.

6.21 Based on the evaluated maximum working stress of 23.12 N/mm^2 under full imposed load of 5 kN/m^2 this safety factor range is between 6.1 and 7.6 as noted in the table below. The table also records the increased safety factor that would be achieved with the more realistic maximum loading of 2.5 kN/m^2, giving a range of safety factors from 7.7 to 9.6.

6.22 For the majority of normal 'day to day' use the terrace would rarely see imposed loads in excess of 1.5 kN/m^2. At this loading the safety factor range would be 8.6 to 10.6.

6.23 The reasonable conclusion to be drawn from the dynamic assessment trial on the principal truss structures is that they are inherently fit-for-purpose and it is only isolated trusses that require more detailed inspection and potentially further testing.



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CERTIFICATE OF TEST

Customer : CAST IRON WELDING SERVICES LTD
Samson Road
Hermitage Industrial Estate
Coalville
Leicester
LE67 3FP
FAO : Peter Palmer

Order No : 48924
W/O No : -
Inc Rel Note : -
Report No : 0052319/001/E1
Issue No : 1
Test Date : 31-Mar-23
Page : 1 of 1

Identification : Tensiles 1 to 12
Description : Tensile Test
Other Info : -
Quantity : 1
Material : Cast Iron
Batch/Cast No : WO No: 35293   Serial No: -
Specification/Procedure : BS EN ISO 6892-1:2019
Acceptance Standard : Factual Report

Tensile Test
Type : Round
Plant No : 2570
Test Rate : A24
Inspector Name : Gavin Steven
Specification : BS EN ISO 6892-1:2019
Temperature : 23 °C
Gauge Length :
UNC ± : Rm/Rp 10 N/mm². Z 1%. A 0.5%
Cert Comment :
Procedure : MCP64 / MCP76

Results	Dimensions (mm)		Proof Stress (MPa)		Tensile Strength	Ratio of Yield	Elongation	Reduction of area		
Sample Details	Diameter	Area			(MPa)	to Tensile	(%)	(%)	Comments	Status
T1	10.06	79.50	-	-	161	-	-	-	-	N/A
T2	10.11	80.29	-	-	176	-	-	-	-	N/A
T3	10.02	78.86	-	-	158	-	-	-	-	N/A
T4	10.00	78.55	-	-	155	-	-	-	-	N/A
T5	10.12	80.45	-	-	147	-	-	-	-	N/A
T6	10.02	78.86	-	-	142	-	-	-	-	N/A
T7	10.02	78.86	-	-	154	-	-	-	-	N/A
T8	10.07	79.65	-	-	175	-	-	-	-	N/A
T9	9.86	76.37	-	-	165	-	-	-	-	N/A
T10	10.04	79.18	-	-	142	-	-	-	-	N/A
T11	10.06	79.50	-	-	158	-	-	-	-	N/A
T12	5.32	22.23	-	-	159	-	-	-	-	N/A

Max
Min

Comments
Tested in accordance with clients order requirements.

Unless otherwise stated, statements of conformity were made using the decision rule of 'simple acceptance'.
The reported expanded uncertainty (U) is based on a standard uncertainty multiplied by a coverage factor of K=2, providing a level of confidence of approximately 95%

THIS CERTIFICATE SHALL NOT BE REPRODUCED, EXCEPT IN FULL, WITHOUT APPROVAL OF NDT SERVICES LTD. RESULTS RELATE ONLY TO THE ITEMS TESTED

For **ndt services limited**

Issue Date: 31-Mar-23

Gavin Steven

NDT 108

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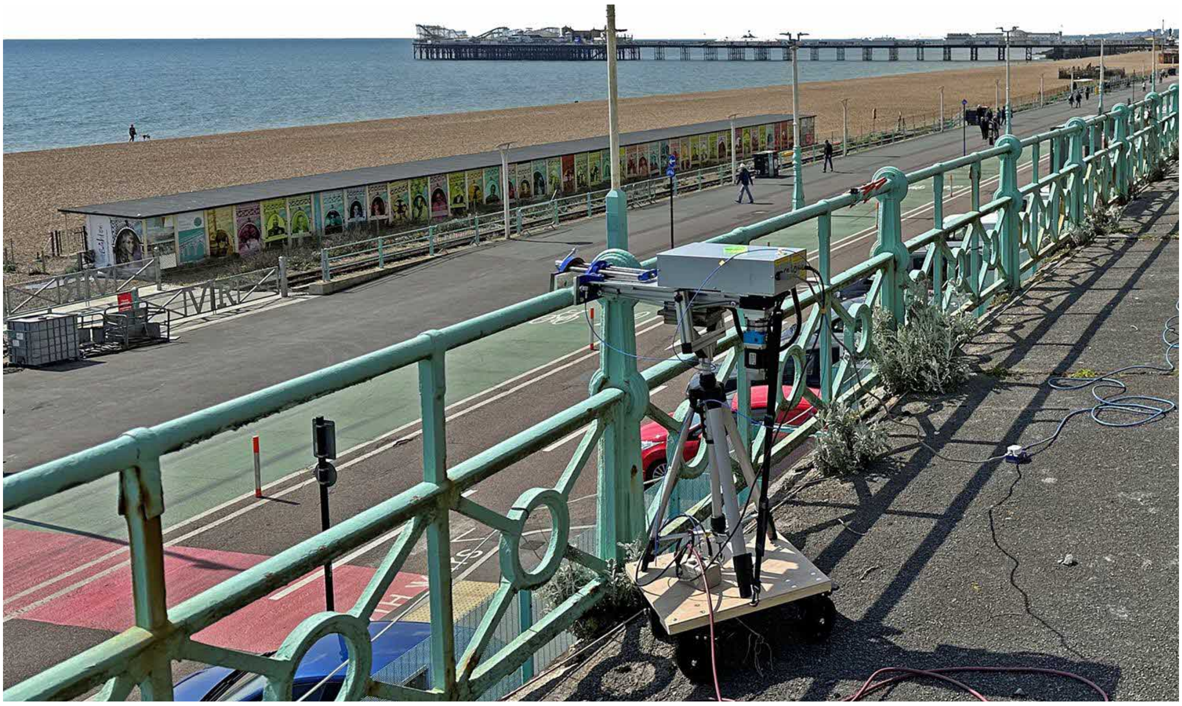
6.24 The table below provides an assessment of the range of safety factors applicable to the existing cast iron trusses supporting the Madeira Terrace deck at grids 87-116. (Note anomalies identified to bays 89, 94 and 99 require further inspection and assessment).

	Dead Load	kN/m²	7.30	7.30	7.30
	Imposed Load	kN/m²	5.00	2.50	1.50
	Total	kN/m²	12.30	9.80	8.80
	Maximum Stress	N/mm²	23.12	18.42	16.54
Min	Tensile Strength	N/mm²	142.00	142.00	142.00
	Safety Factor		6.1	7.7	8.6
Max	Tensile Strength	N/mm²	176.00	176.00	176.00
	Safety Factor		7.6	9.6	10.6

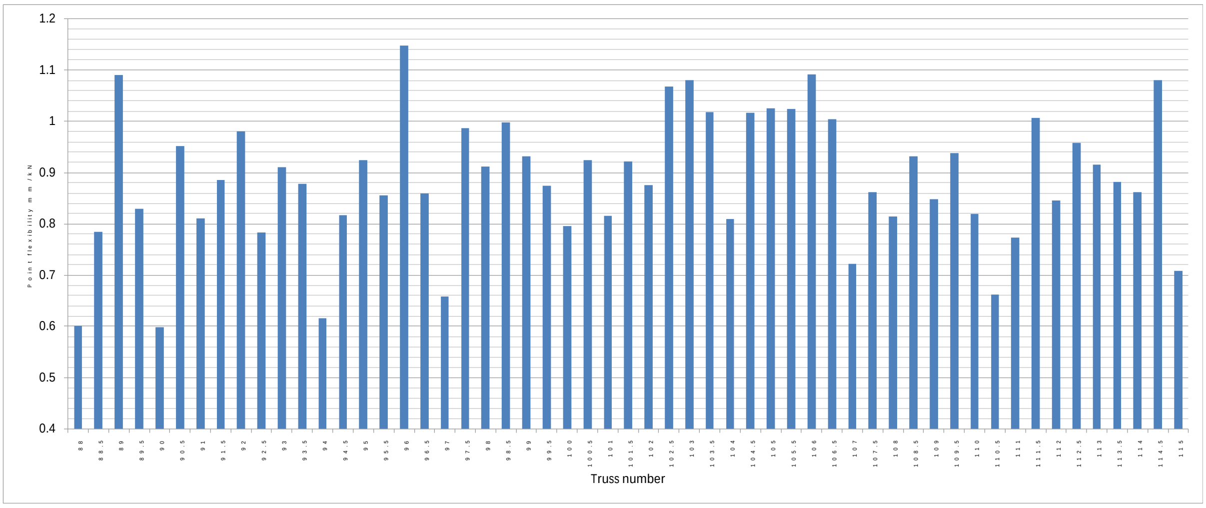
6.25 For compression loads the maximum value recorded was 38 N/mm2. This is well withing the guidance of 125 N/mm^2 as an acceptable compression load.

7.0 Assessment of Balustrades

7.01 The balustrade to Madeira terrace is shown in the image below. It consists of cast iron, vertically cantilevering, sections located on each main grid line and at mid positions giving a structural spacing of approximately 2.4m. Tests were carried out from grid 88 to 115 at both primary and intermediate locations, with lateral excitation imposed using the rig illustrated below.



7.02 The graph below shows the lateral stiffnesses measured with variations from approx. 0.6 mm/kN to 1.15 mm/kN recorded along the length from grid 88 to 115. No significant variation was noted between primary (on grid) and secondary balustrades (between grids).
With approx. 2.4m between balustrades a 3kN/m line load would predict around 6.5mm maximum deflection.

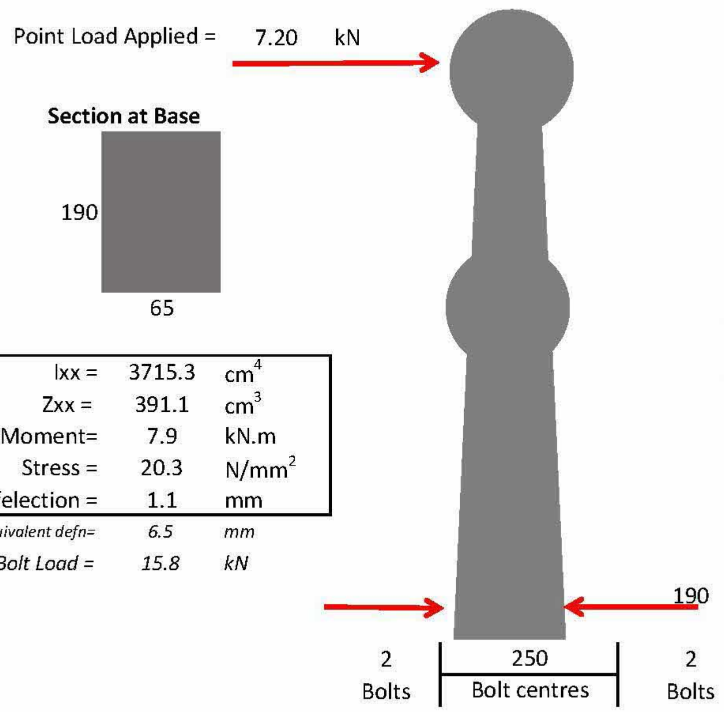


7.03 When comparing the dynamic test results to a simple static analysis (shown below) the deflection/flexibility predicted was significantly less, suggesting an anomaly in the structure.

Balustrade Assessment

Principle Balustrade centres	2400	mm
Line load applied	3	kN/m
Height of application	1100	mm

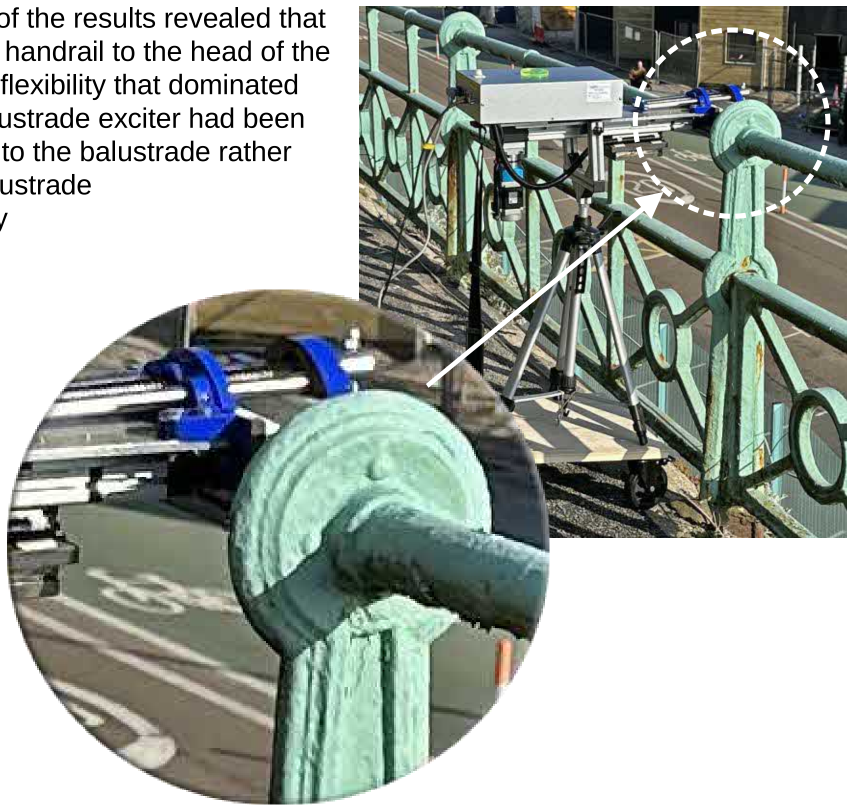
Allowable Stress =	25	N/mm^2
Youngs Modulus, E =	80	kN/mm^2
Dynamic test stiffness =	0.9	mm/kN



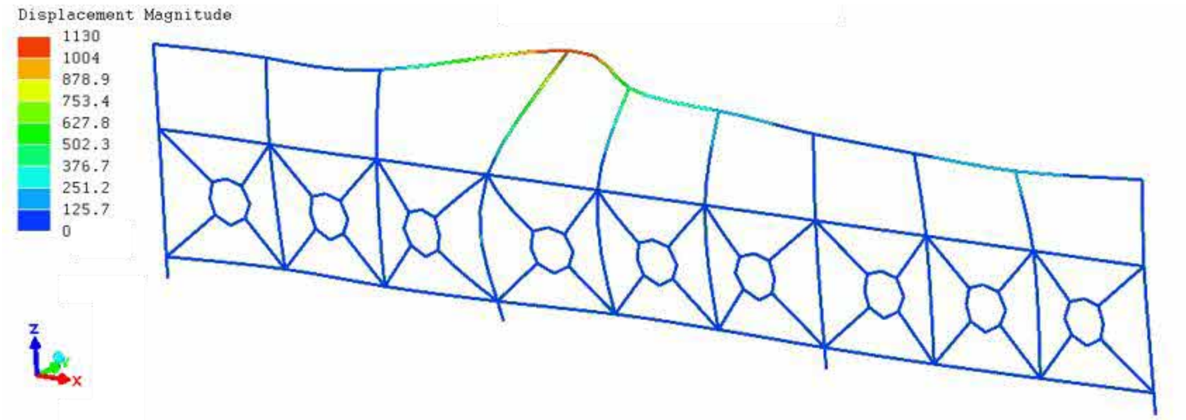
Ixx =	3715.3	cm^4
Zxx =	391.1	cm^3
Moment=	7.9	kN.m
Stress =	20.3	N/mm^2
Defelection =	1.1	mm
Insitu equivalent defn=	*6.5*	*mm*
Max Bolt Load =	*15.8*	*kN*

7.04 A more detailed inspection of the results revealed that the connection of the upper handrail to the head of the balustrade had a high local flexibility that dominated the readings. Since the balustrade exciter had been secured to the rail adjacent to the balustrade rather than directly fixed to the balustrade head the additional flexibility was recorded.

The image opposite shows the loading configuration used for the dynamic test

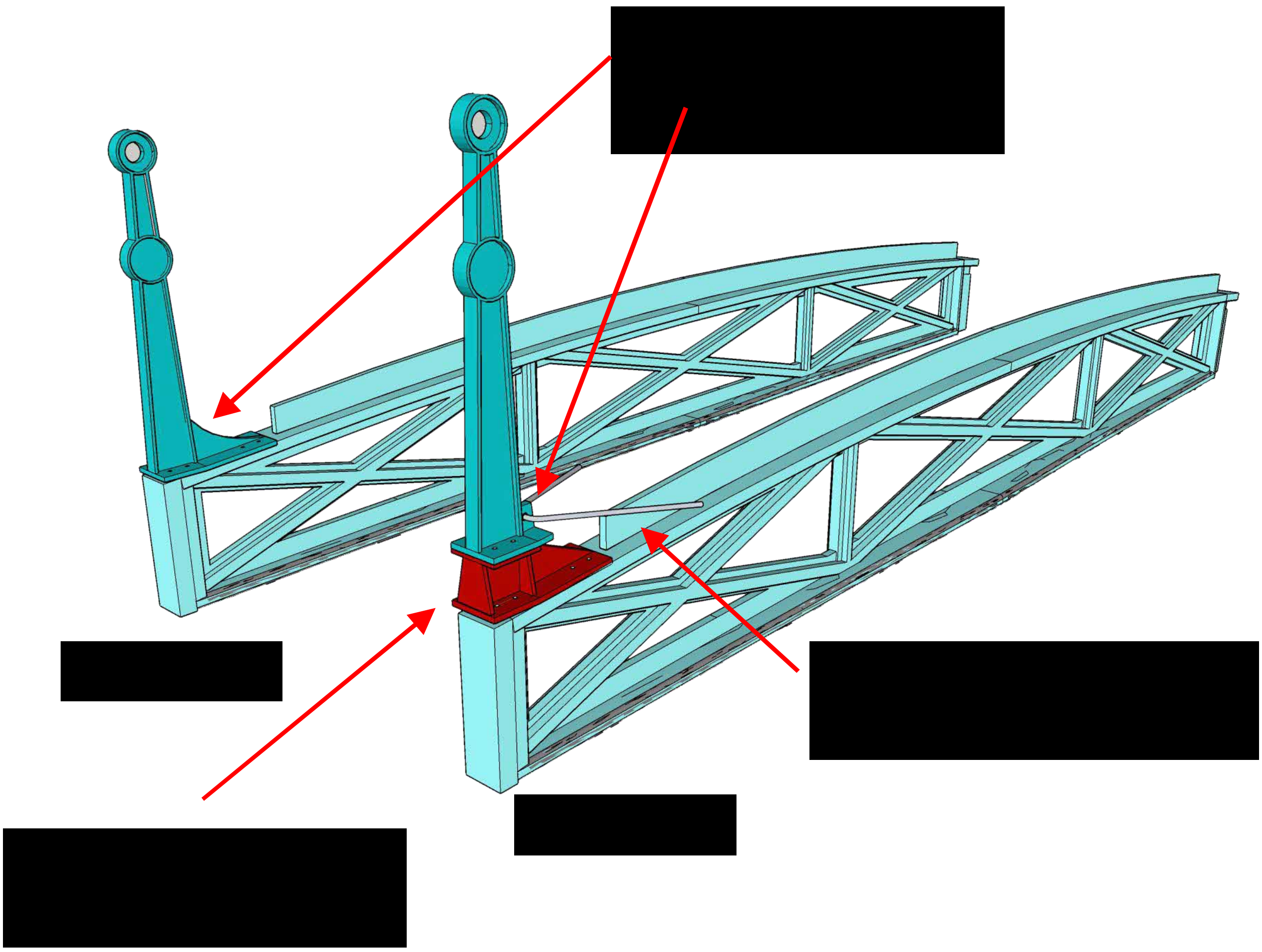


7.05 The detailed inspection of a sample balustrade panel confirmed that the lower section of balustrade were well restrained and had stiffness characteristics more aligned to the simple static analysis.



7.06 The dynamic testing has confirmed good stiffness characteristics are attributable to the original cast iron balustrades, however there is some local rotational flexibility in the current handrail (not original structure) where it connects to the original cast iron balustrade head.

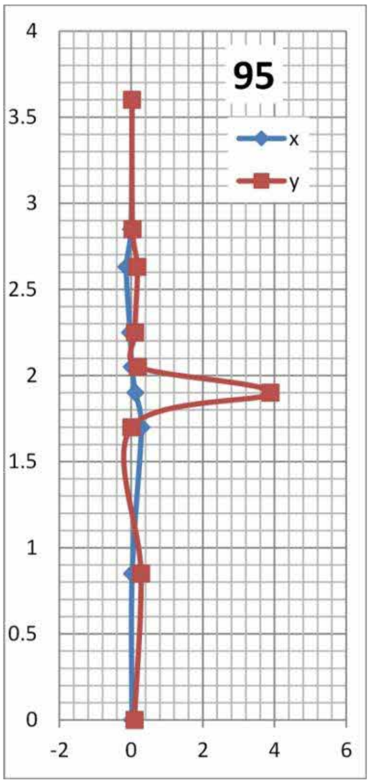
7.07 The requirement to raise the handrail height will provide an opportunity to provide improvements in the connectivity of handrail to balustrade. The raising of the existing balustrade is considered to be relatively simply achieved with the use of steel shoe section as illustrated below.

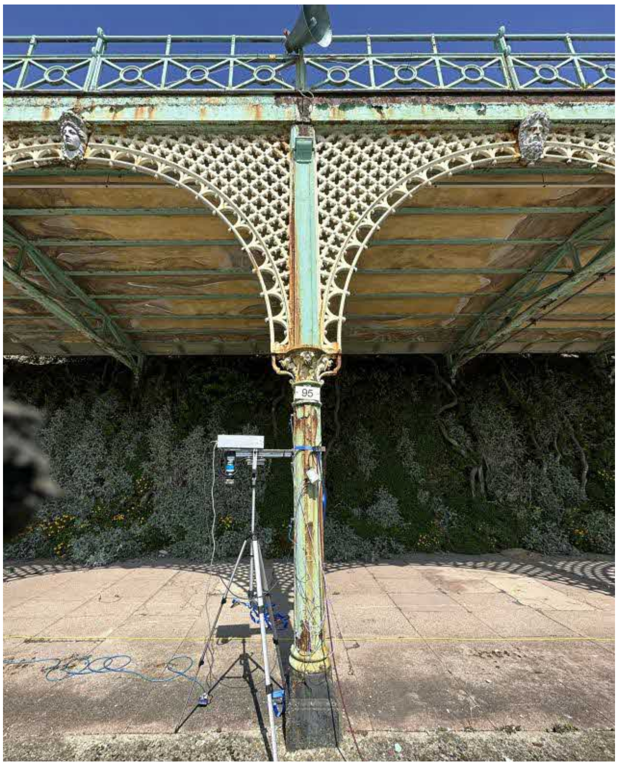


8.0 Assessment of Columns

8.01 The trial dynamic assessment of the Madeira Terrace structure was primarily focused on the balustrades and main cast iron trusses, however two columns were briefly assessed during the available time allocated to the trial testing with lateral excitation applied to the column as illustrated in the image opposite.

8.02 The testing clearly identified the presence of a joint and discontinuity at mid height as illustrated in the stiffness plot below. This discontinuity relates to the socketed connection between upper and lower column sections and would be an expected response. Further testing and modelling of the columns would be required should any performance concerns exist.





9.0 Conclusions

General Summary

9.01 The initial trial dynamic testing of samples of the Madeira Terrace structure yielded good data of consistent quality that enabled performance characteristics of the structures to be established with a high degree of confidence.

9.02 The testing enabled aspects of the construction that were important to the performance to be understood in greater detail than previously existed. The information gained will assist in design decisions moving forward to the construction phase of the project.

9.03 Although there is a potential need for additional testing and analysis to occur as works progress the conclusion of the trial phase of dynamic testing was that the existing cast iron elements of the structure are capable of retention, refurbishment and reuse.

9.04 The optimised computer model which was tuned to match actual site performance enabled the actual performance of the structure to be determined under specified design loads. This showed working loads to be within acceptable limits and demonstrated ‘fitness for purpose’ in the structure where deficiencies were not revealed.

9.05 Where deficiencies were revealed in a small number of elements the opportunity is provided to focus further scrutiny to establish the reasons and what localised remedial works are appropriate.

Balustrades

9.10 The dynamic testing trial showed good consistency along the length of balustrade tested.

9.11 The trial testing revealed localised flexibility in the handrail section at the junction with the balustrade head. Whilst this characteristic revealed by dynamic testing initially suggested a more flexible assembly than would be predicted by static analysis of estimated balustrade section, a more detailed inspection of a balustrade bay confirmed the original cast iron balustrade sections were suitable for reuse.

9.12 The need to raise the handrail height requires geometry and detailing to be considered, however this is not considered to present any significant design challenges and the introduction of plinths or stools to achieve the desired height is considered to be a practical approach.

Main Trusses

9.21 The main cast iron trusses were revealed as acting compositely with existing deck structure which provided a significant level of enhancement to capacity.

9.22 Stress levels under full working design load of 5 kN/m^2 revealed predicted stress levels of around 23 N/mm^2, which is within recognised safe working stress level of 24 N/mm^2 (ref appendix C). Whilst this is recognised as being around 95% of the safe working stress it is important to note that the allowable stress value recognises and takes account of all appropriate safety factors so even being at 100% utilisation is acceptable as no additional safety margins need to be applied.

9.23 If the more realistic maximum imposed load of 2.5 kN/m^2 is considered then there is an additional margin of safety of between 25% and 30% provided, and further confidence in the ability of the existing trusses to be retained and refurbished.

9.24 The peak compression stress predicted in the main truss is around 38 N/mm^2, which is considerably below the guidance acceptable compressive stress of 125 N/mm^2 (ref appendix C).

Columns

9.31 Two columns were only briefly assessed during the available time allocated to the trial testing (the majority of time spent on the balustrades and main trusses as proposed).

9.32 The testing clearly identified the presence of a joint and discontinuity at mid height. It is unclear, at present, if there are any concerns in relation to column condition or capacity. The use of further dynamic assessment can be used to increase the level of confidence in the capacity of columns if required.

Foundations

9.41 No testing was carried out on foundation flexibility or resistance. This may be a future option to elevate confidence in foundation support/resistance.

10.0 Next Stage

10.01 Whilst the trial phase of dynamic testing has yielded significant confidence in the ability for the existing structural elements to be retained it is noted that Madeira Terrace is an extensive structure. During future phases of renovation and conservation there will inevitably be areas of weakness and defects revealed that will need to be addressed. It is, however considered clear from the initial phase of dynamic assessment that wholesale replacement of existing principal structural ironwork elements is neither required, or justified on the basis of existing information.

10.02 The objective of minimising the carbon footprint of the proposed refurbishment of Madeira Terrace would add further weight to the need to retain and reuse.

10.03 With confidence in the ability to retain, refurbish and reuse existing trusses then a methodology and approach would be required to ensure a mechanism for identifying any local issues or defects in structural ironwork elements. This may be a combination of visual assessments and further dynamic testing.

10.04 Truss position 89, 94 and 99 require anomalies to be considered. This may simply be a deck deficiency and not a truss deficiency, resulting in reduced composite action, or it may be a truss defect that may require repair.

10.05 It would be considered beneficial to test a bare truss ***(as exists on site, image opposite)*** as this would add refinement to assessment methodology and to allow a better understanding and modelling of composite action of deck.

10.06 Where there are trusses with known defects present there would be an option to dynamically test these. The Finite Element model can also be run with the defect simulated. This would enable an understanding of the level of structural redundancy that exists in the system to be assessed. The benefit would be a greater understanding of the risk of disproportional collapse or sudden unpredicted failure.

10.07 For balustrades it would be possible to refine testing to remove the influence of local handrail flexibility. There is also the option available to test a bare balustrade section by securing to a workshop rig or other fixed point to determine the performance characteristics in isolation from slabs.

10.08 If concerns exist in relation to column reuse, then consider dynamic assessment of these structural elements to raise confidence in capacity and identify any areas of potential concern.

10.09 If concerns exist in relation to foundation performance, then consider dynamic resistance assessment of these to raise confidence in capacity and identify any areas of potential concern.

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Appendices

A Cast Iron Test Results

B Truss Stiffness Data

C Historic Ironwork

A. Cast Iron Test Results

A.01 The following test results were provided, and relate to samples taken from an existing balustrade section of the terrace.

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NDT Services Limited
99 Victory Road
Derby
DE24 8EL

Tel +44 1332 275820
ndt@intertek.com
intertek.com

CERTIFICATE OF TEST

Customer : CAST IRON WELDING SERVICES LTD
Samson Road
Hermitage Industrial Estate
Coalville
Leicester
LE67 3FP
FAO : Peter Palmer

Order No : 48924
W/O No : -
Inc Rel Note : -
Report No : 0052319/001/E1
Issue No : 1
Test Date : 31-Mar-23
Page : 1 of 1

Identification : Tensiles 1 to 12
Description : Tensile Test
Other Info : -
Quantity : 1
Material : Cast Iron
Batch/Cast No : WO No: 35293 **Serial No:** -
Specification/Procedure : BS EN ISO 6892-1:2019
Acceptance Standard : Factual Report

Tensile Test
Type : Round
Plant No : 2570
Test Rate : A24
Inspector Name : Gavin Steven

Specification : BS EN ISO 6892-1:2019 **Procedure** : MCP64 / MCP76
Temperature : 23 °C
Gauge Length :
UNC ± : Rm/Rp 10 N/mm², Z 1%, A 0.5%
Cert Comment :

Results	Dimensions (mm)		Proof Stress (MPa)		Tensile Strength	Ratio of Yield	Elongation	Reduction of area		
Sample Details	Diameter	Area			(MPa)	to Tensile	(%)	(%)	Comments	Status
T1	10.06	79.50	-	-	161	-	-	-	-	N/A
T2	10.11	80.29	-	-	176	-	-	-	-	N/A
T3	10.02	78.86	-	-	158	-	-	-	-	N/A
T4	10.00	78.55	-	-	155	-	-	-	-	N/A
T5	10.12	80.45	-	-	147	-	-	-	-	N/A
T6	10.02	78.86	-	-	142	-	-	-	-	N/A
T7	10.02	78.86	-	-	154	-	-	-	-	N/A
T8	10.07	79.65	-	-	175	-	-	-	-	N/A
T9	9.86	76.37	-	-	165	-	-	-	-	N/A
T10	10.04	79.18	-	-	142	-	-	-	-	N/A
T11	10.06	79.50	-	-	158	-	-	-	-	N/A
T12	5.32	22.23	-	-	159	-	-	-	-	N/A

Comments
Tested in accordance with clients order requirements.

Unless otherwise stated, statements of conformity were made using the decision rule of 'simple acceptance'.
The reported expanded uncertainty (U) is based on a standard uncertainty multiplied by a coverage factor of K=2, providing a level of confidence of approximately 95%

THIS CERTIFICATE SHALL NOT BE REPRODUCED, EXCEPT IN FULL, WITHOUT APPROVAL OF NDT SERVICES LTD. RESULTS RELATE ONLY TO THE ITEMS TESTED

For **ndt services limited**

NDT 108

Issue Date: 31-Mar-23 **Gavin Steven**

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Registered in England and Wales. Registered No. 1997290
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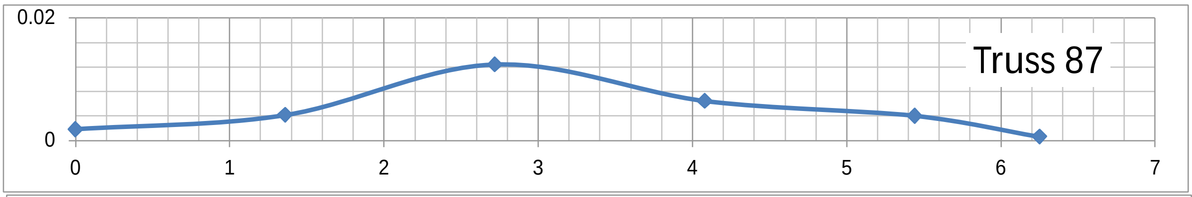
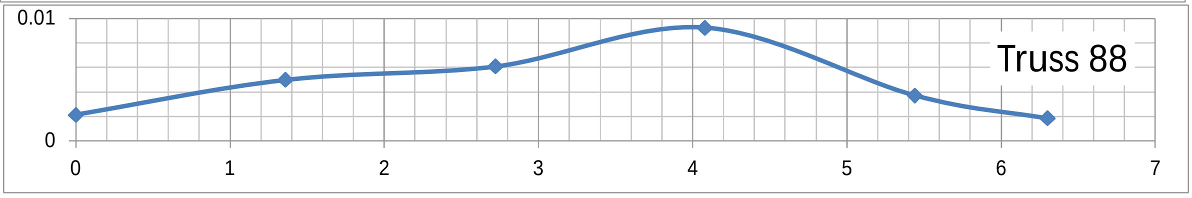
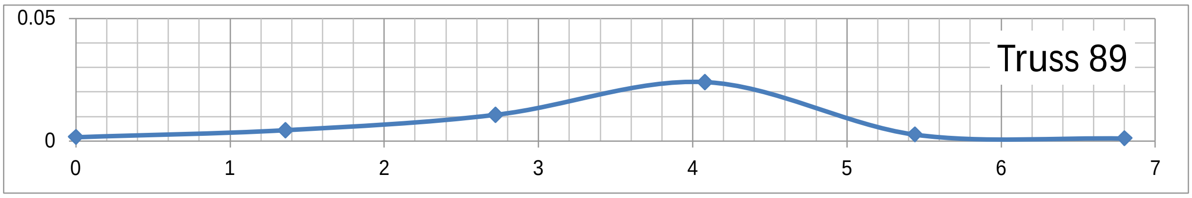
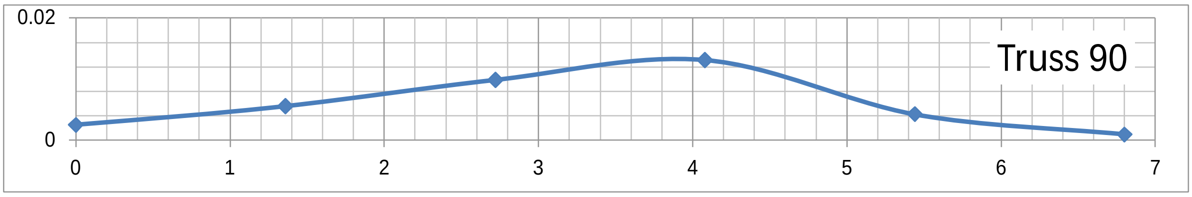
B Truss Stiffness Data

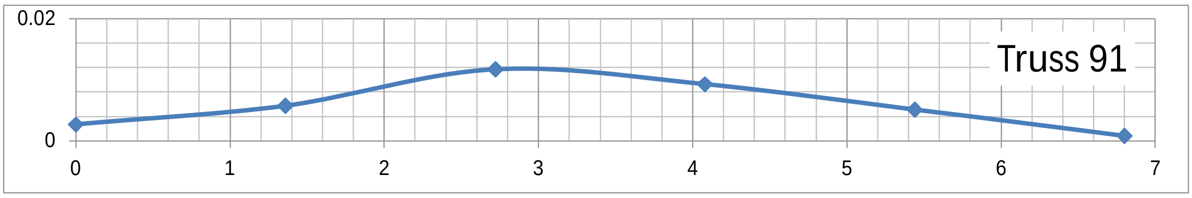
The following plots provide the stiffness characteristics of each truss line with results in mm based on a 1 kN unit central point load on the truss grid line.

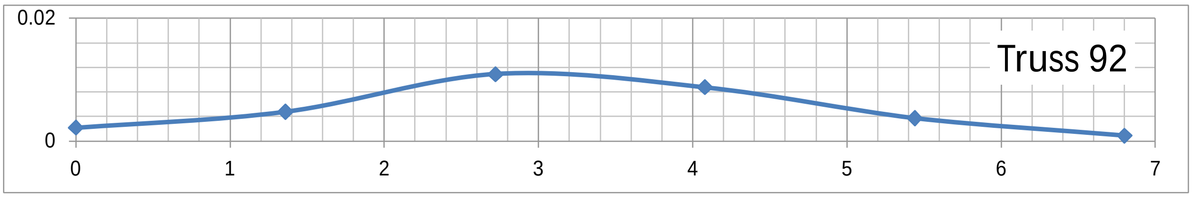


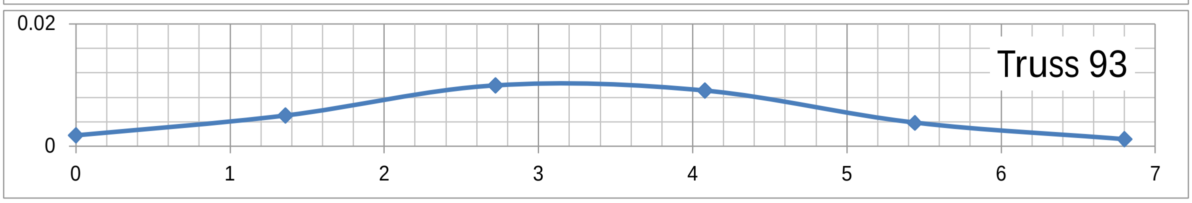


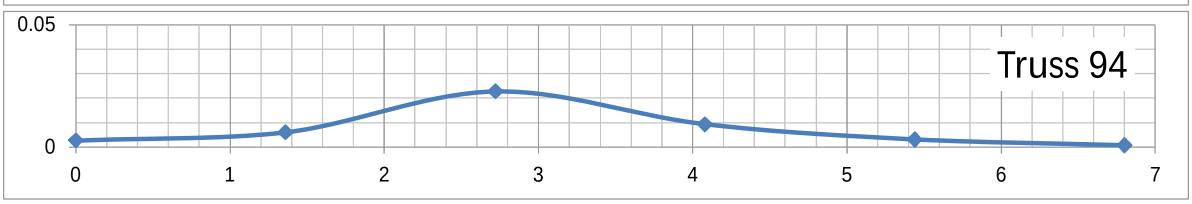












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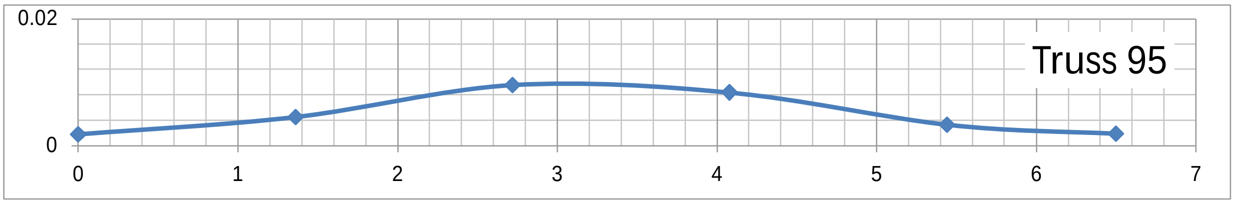

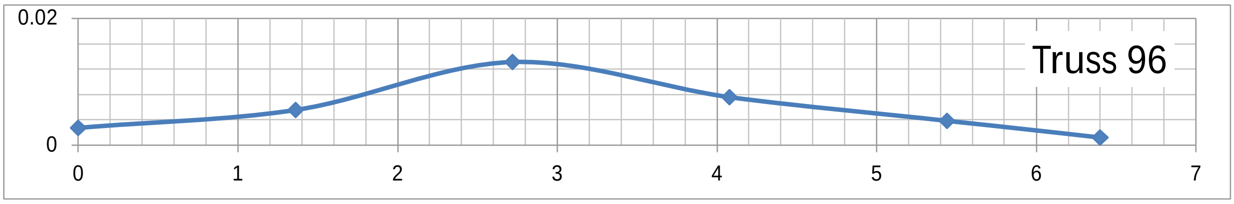

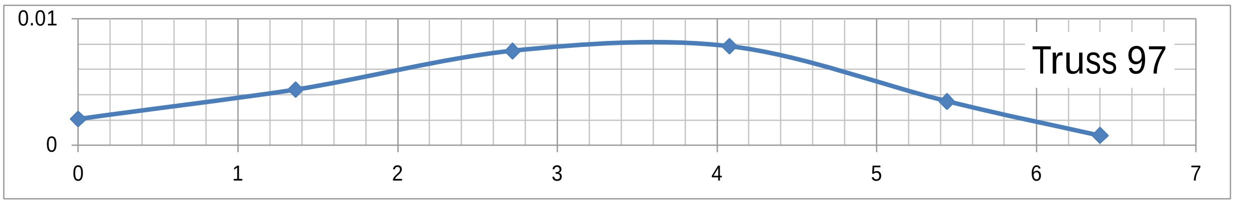

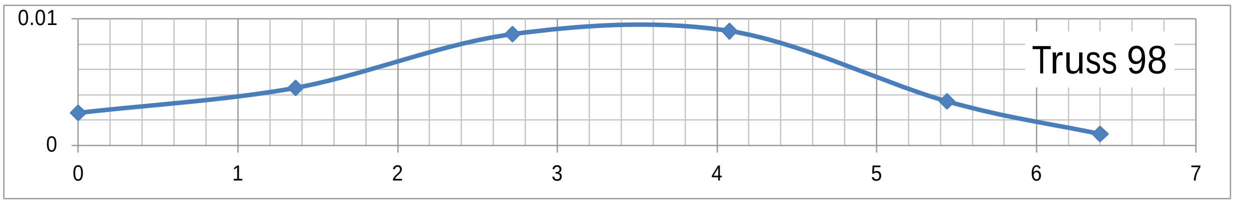

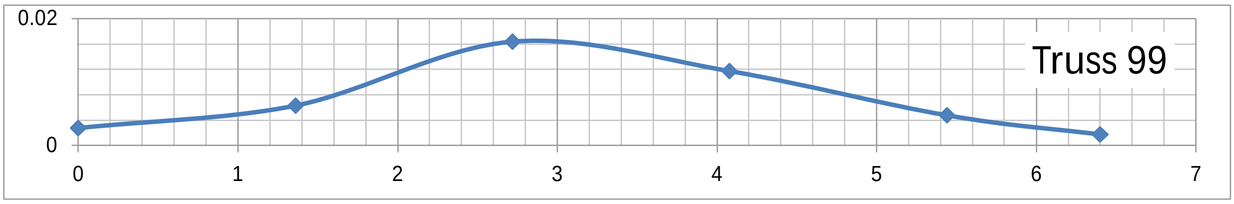

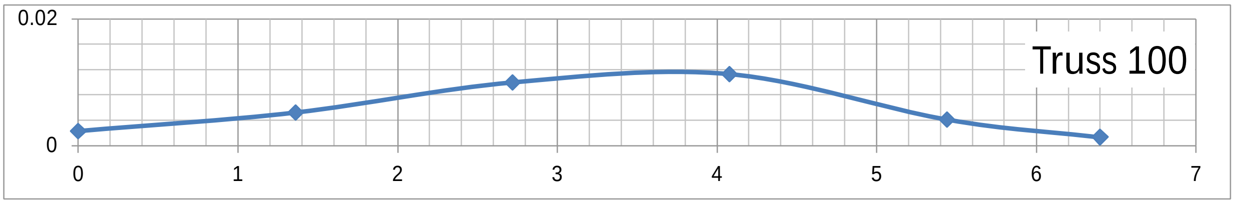

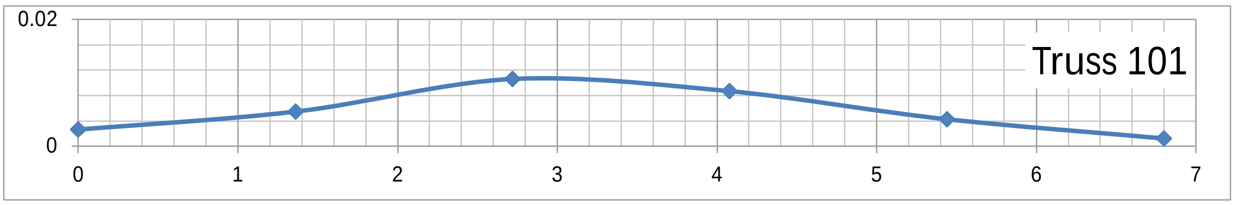

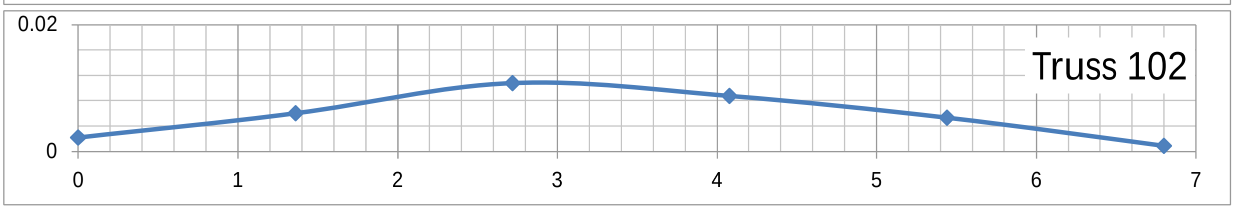

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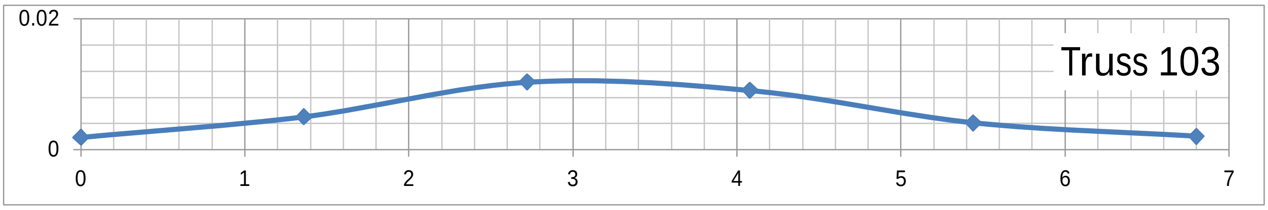

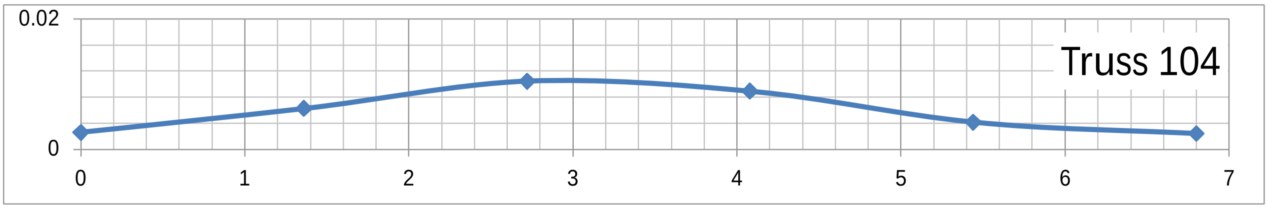

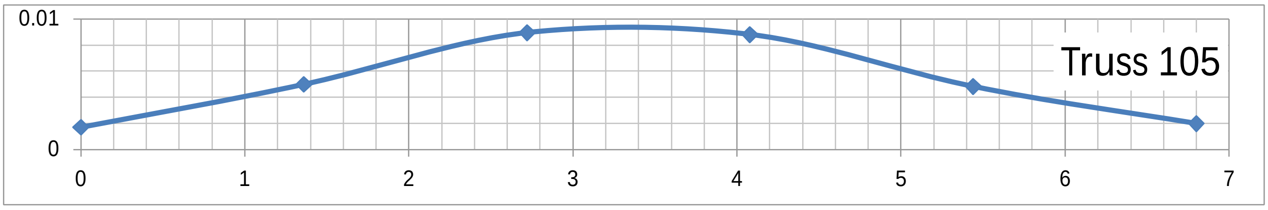

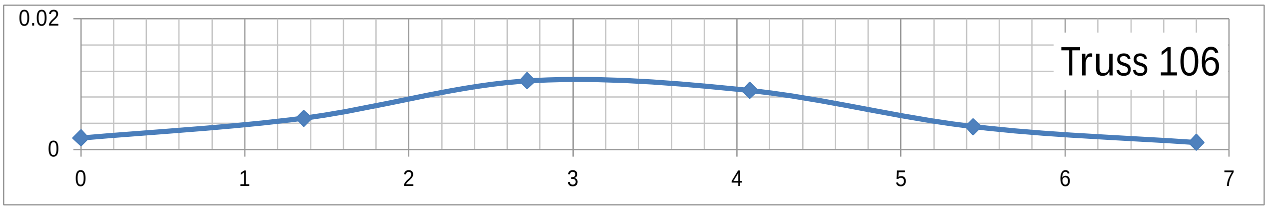

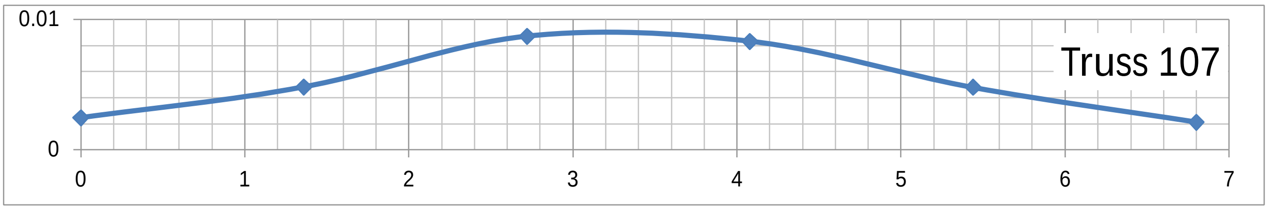

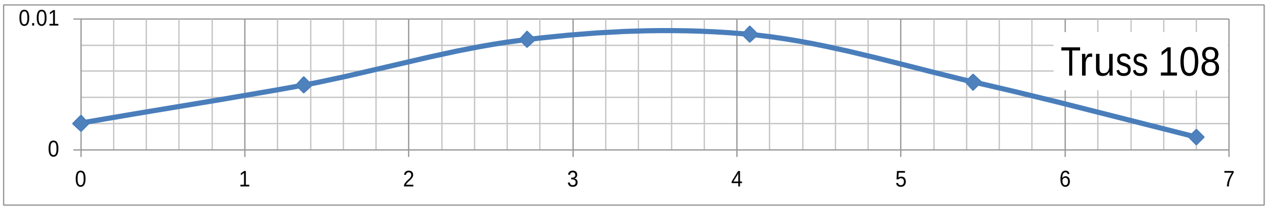

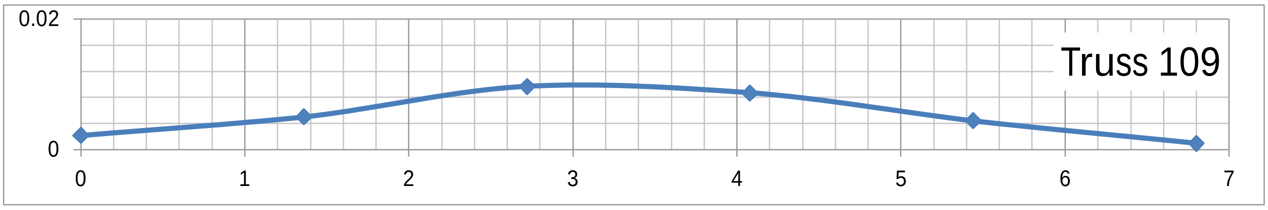

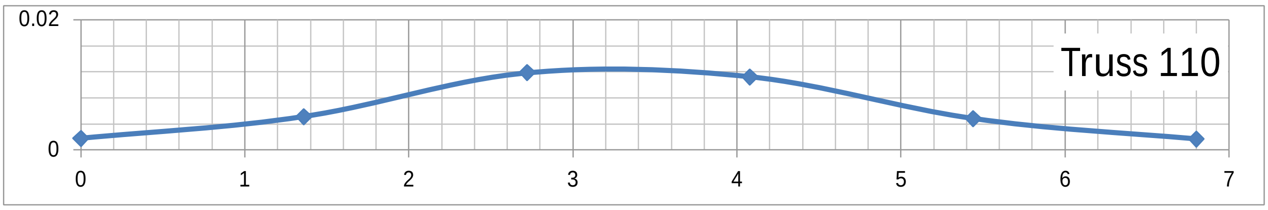

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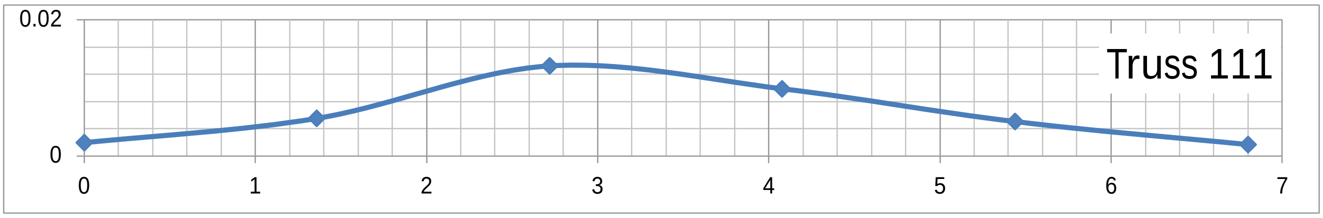

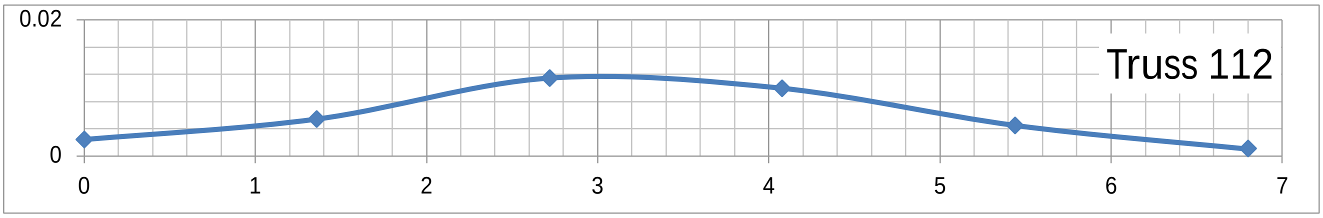

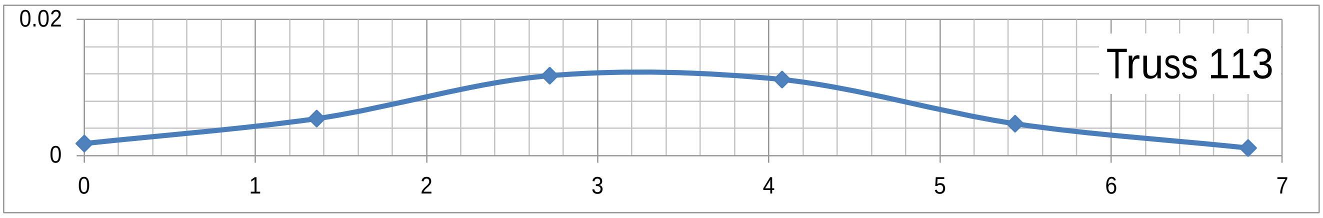

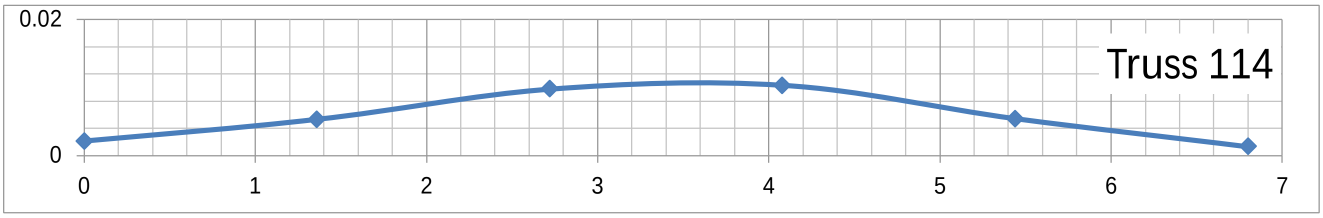

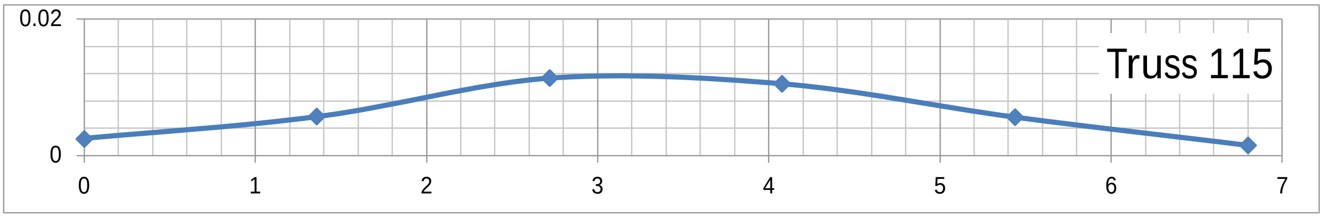

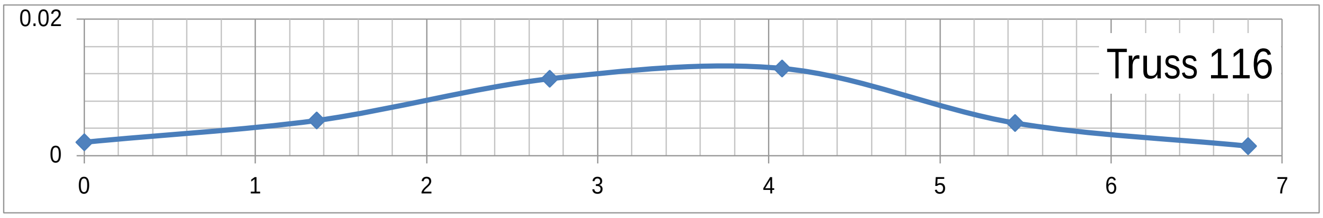

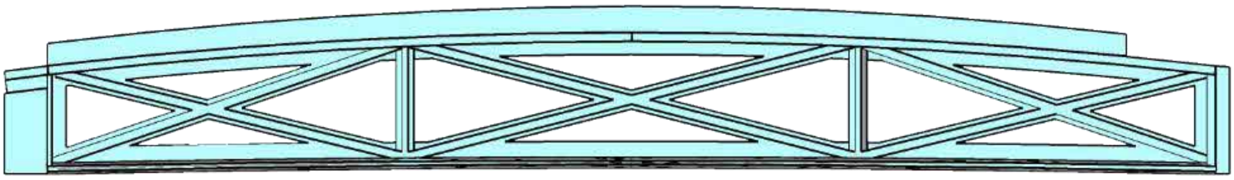

C Historic Ironwork

Conservation compendium

Part 3: Historic wrought iron, cast iron and mild steel

This article forms part of the Conservation compendium, which aims to improve the way engineers handle historic fabric through the study of historic materials, conservation philosophy, forms of construction and project examples. Articles in the series are written by Conservation Accredited Engineers. The series editor is James Miller.

John E Ruddy BEng, MA(Conservation), CEng, MICE, MIStructE, Conservation Accredited Engineer and Director, Capstone Consulting Engineers Ltd

As structural engineering students, we learn about mild steel, modern design and construction methods. However, historic structures often do not fit into this mould. Whether you work in conservation or are a general practitioner, you are likely to come across cast iron, wrought iron, as well as early mild steel structures. The historic ironwork could be as small as a strap, providing tension across a joint, or more dramatically, the whole structure.

The first major all steel bridge – the Forth Bridge – was famously called "the supremest specimen of all ugliness" by William Morris (co-founder of the Society for the Protection of Ancient Buildings). Yet it went on to become not only listed in the UK (on the Statutory List of Buildings of Special Architectural or Historic Interest), but is currently being considered for designation as a UNESCO World Heritage Site.

Key properties

Cast iron, wrought iron and mild steel are chemically speaking very similar to each other, being alloys of iron and carbon. It is the carbon content which gives them their distinctly separate properties (Table 1)[1,2].

Cast iron

The lower tensile strength of cast iron in comparison to compression is due to the carbon within it. On cooling, the carbon forms into 'plates' of graphite throughout the iron. These plates are able to transfer compressive stresses, but because they are not bonded to the iron they represent planes of weakness under tensile loads. To overcome this issue, cast iron beams, for example, typically have larger tension flanges.

Figure 2
Cast iron column, complete with cast Corinthian capital

Figure 1
Repair to corroded steel frame

However, the resistance of cast iron to corrosion is excellent. This is partially attributed to the 'fire skin' which develops on the surface of a casting, the fusion of iron and silicon (from the sand mould), during production.

The production process of cast iron greatly influences its properties such as strength, ductility and resistance to fatigue. If a casting cools quickly, the graphite plates do not form, resulting in a stronger but more brittle alloy of iron. Cast iron was specified in terms of the origin of the pigs used, each having its own slightly different characteristics which affected the overall properties. Imperfections incorporated during the casting process act as stress concentrations, lowering the capacity of the section. Due to its brittle nature, cast iron is not suited to rivet connections which are driven through punched holes. Also cast iron cannot be 'welded in the fire' like wrought iron. Connections tend to be mechanical, such as bolts (using cast holes).

Wrought iron

The advantage of wrought over cast iron is that it exhibits ductile characteristics, deflecting under impact and shock loads. Importantly, when overstressed, it gives a clear warning of an approaching collapse by permanently deforming.

Wrought iron is made up of almost pure iron and an inert silicate 'slag' material. The iron is worked, or 'wrought', under heat, lining up the slag layers and iron into strands, which are better able to resist the passage of microscopic cracking. Another consequence of this aligning of slag layers is that wrought iron is weaker in the perpendicular direction to the aligned layers. This is not a problem with wrought iron sections in service, as the process of forming them (by hammer blows or rolling) aligns the iron and slag the right way. However, this is why electric arc (fusion) welding to wrought iron is not advisable.

Wrought iron can be forge-welded together, a process where the two pieces are heated and squashed into one piece

Data for cast Iron

Table 1: Key properties of cast iron, wrought iron and steel

Key properties	Cast iron	Wrought iron	Steel
Carbon content (%)	2.5–4.0	<0.2	0.3
Tensile strength	Poor	Good	Good
Ultimate stress (N/mm^2)	65–280	278–593	386–494
Allowable stress (N/mm^2)	24	78	117
Compressive strength	Good	Good	Good
Ultimate stress (N/mm^2)	587–772	247–309	386–494
Allowable stress (N/mm^2)	125	78	117
Ductility	Poor	Good	Good
Young's modulus (kN/m^2)	66–94	154–220	200–205
Corrosion resistance	Excellent	Good	Poor
Fatigue resistance	Poor	Good	Good

The ultimate stress values for the metals were obtained from the 19th century experiments of Hodgkinson, Twelvetrees and others, as described by Swailes[1] and Bussell[2].
The allowable stress values are from the London Building Act 1909.

under hammer blows, using the slag as flux. Also, as wrought iron is ductile, it can be punched to accept rivets. Wrought iron is more susceptible to rusting than cast iron. The rust delaminates from the body of the iron along the slag veins of weakness.

Mild steel
Mild steel is stronger than wrought iron and also exhibits greater ductile characteristics. This has contributed to it currently being the most widely used structural metal. Connections can be made to steel in a variety of ways. Having no slag, steel is isotropic in strength and can be fusion welded. As with wrought iron, it can be bolted, and its ductility allows it to be punched to receive rivet connections.

Steel readily rusts in atmospheric conditions, and steel therefore needs to be protected, using a barrier such as paint to separate it from the atmosphere.

Development of iron

Wrought iron has been smelted into a bloom from iron ore over charcoal since before 2000BC, primarily to be used for tools and weapons.

Cast iron has been produced in quantity since the invention in the 1300s of the blast furnace, in which the iron is liquefied out of the ore. The molten iron could then be cast into a variety of mould shapes. Various advances were made over the centuries, improving the production process. In the 1860s Bessemer and Siemens invented processes to produce significant quantities of steel cheaply.

The continuous rolling mill invented by George Bedson in 1862 is one of a number of advances in this revolution from small-scale craftsmanship to mass production. With it came improved quality control, so that the material properties could be assumed with confidence. In 1880 Siemens invented the electric arc furnace. As no combustible fuel is present, the steel cannot be contaminated by the combustion products and a pure steel is produced. This heralded the birth of modern structural steel.

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Figure 3 Example of cold stitching

Figure 4 Wrought iron strap within 15th century church tower

Examples of repair

Corrosion
Atmospheric corrosion is an electro-chemical process that takes place in the presence of oxygen and water. The reaction transforms the strong useful metal into weak rust. Not only does this result in reduced strength, the rust itself expands as it forms. This occurs with high molecular force, and the forming rust can cause considerable damage to surrounding work, particularly where the iron is built into masonry. A small dowel or cramp can jack up a surprising weight of stonework above it. Where iron plates are riveted together, these forces can be enough to snap the rivets holding the plates together.

Figure 1 shows a steel frame from the early 1900s which was exposed when refurbishing a shop front in Glossop, Derbyshire. The area of steelwork was severely reduced due to corrosion in places, particularly where there had been long-term contact with damp, such as where columns pass into the ground, or close to failing flashings. Here the solution was to dress the area back to sound metal and weld on new pieces to compensate for the material lost, before protecting with paintwork.

Cast iron
A redundant church in Leeds was recently converted into a community performance space. This involved a new infill floor, adding

load to the existing slender balcony columns. Unlike modern (and historic) rolled steel sections, these columns were not made to set dimensions. An important step in assessing the load capacity of exiting cast iron columns is finding out how thick the casting wall is. This is done by drilling small holes. Three holes are needed as the void within may be off centre. One of the columns is shown in Figure 2.

Due to the brittle nature of cast iron, fractures can occur. One possible cause can be impact damage, or localised thermal shock. Cast iron cannot be readily welded; however, a mechanical 'cold stitching' technique can be used. This is where nickel-steel stitches are inserted into tight-fitting drilled slots at regular intervals, running across the fracture line, knitting the two sides together again (Figure 3).

Wrought iron

Figure 4 shows a wrought iron strap repair to the bell frame within a 15th century Lincolnshire church. This has been carefully crafted to fit the oak frame. Despite hundreds of years of relative exposure, all that is needed is the removal of the surface rust, followed by painting (although it is suspected that it would manage many more years unpainted).

Conclusion

I am from a 'steel town' and am reminded of that heritage when I see 'Dorman Long' or 'Middlesbrough' stamped on steel sections across the country and across the globe. I have had the privilege of seeing castings being poured at Longbottoms iron foundry near Huddersfield, wrought iron worked in the furnace at the Topp and Co. blacksmith's works in Yorkshire, and steel beams being formed in the rolling mill at Redcar. There is real craftsmanship in iron and steelwork. It is often easier to see it in older structures, and it is this craftsman's input that to my mind gives historic work its 'value', making it worth conserving. I suspect it is recognising and respecting this value in older structures that draws engineers towards 'conservation engineering' as a career.

References and further reading

1) Swailes T (1995) '19th century cast iron beams: their design, manufacture and reliability', *Proc. ICE Civ. Eng.*, 114 (1), pp. 25–35

2) Bussell M (1997) *P138: Appraisal of existing iron and steel structures*, Ascot, UK: Steel Construction Institute

Further reading

Bussell[2] provides further information on uses and dates of iron and steel structures, guidance on analysis and the estimation of load capacity.