Materials Evaluation

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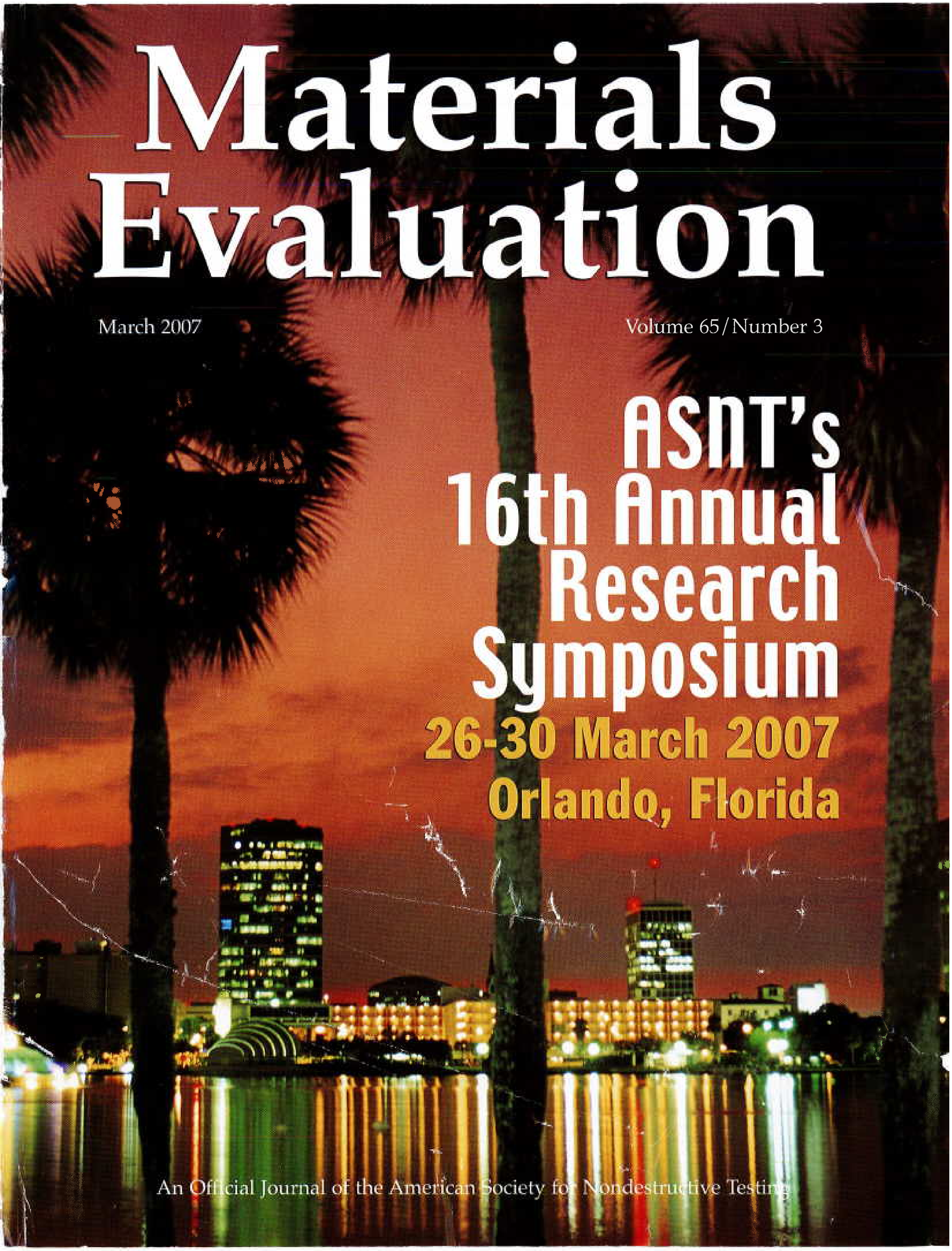

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Structural Conservation of Robert Stephenson's High Level Bridge across the River Tyne

by Graham Herdman,* Tim Abbott† and Jon Powell†

History

Newcastle and Gateshead, in northeastern England, are joined by 10 bridges. Newcastle stands on the northern bank of the Tyne Gorge, facing its smaller neighbor Gateshead on the opposite side of the river; both make up Tyneside. The Romans built the first bridge over the River Tyne, guarded by a fort on Hadrian's Wall, and named it Pons Aelius. They picked this site as it was the first point upstream at which the River Tyne could be bridged with their technology. The Normans, realizing the importance of the site, built a wooden castle in 1080, with the "New Castle" giving the area the name by which it continues to be known.

The abundance of local natural resources stimulated the growth of industry on Tyneside. Coal had been mined in the area since the 14th century and had directly stimulated the development of the world's earliest railways in the northeast during the 1800s. Also significant for Tyneside was the local availability of iron ore, which, in conjunction with coal, provided support for shipbuilding, locomotive engineering, civil engineering and armament manufacture.

Though Newcastle was considered a destination by the various companies that built lines to it, they felt that the city could not be a hub because the new railway could not descend to cross a low level bridge at the bottom of the gorge. Any connection would need to cross the gorge at a high level.

In 1845, the decision was taken to build a combined road and rail bridge. Robert Stephenson's High Level Bridge (completed 1849) was one of the world's first road and rail crossings, with the railway carried above and the roadway below. Stephenson's design followed at least 19 different proposals by the most prominent railway engineers of the time. The designs included both high and low level bridge crossings between Gateshead and Newcastle. The fact that foot and cart passage was also required led to Stephenson's adoption of a two-tier structure — the first time this form was used.

Contracts for the construction of the bridge were handed out in July 1846, and in February 1847 the casting of the ironwork began. By July 1847, the first span was ready for testing. It was intended to proof test the first span to 635 000 kg (700 tons), but during the testing a partner asked Stephenson to increase the load by another 90 000 kg (100 tons) to improve public confidence. Confidence in Stephenson's designs, and iron bridges in general, were at an all-time low after the failure of his River Dee bridge, which had collapsed under a train only eight weeks earlier.

Once completed, this restoration project will extend the useful life of the bridge for at least another 50 years.

The ironwork was completed in April 1849, and a full working load test was carried out in August of the same year using a test load of four tender locomotives and 18 wagons loaded with ballast weighing a total of 180 000 kg (200 tons). Queen Victoria officially opened the High Level Bridge on 28 September 1849 (Addyman and Fawcett, 1999). It now has Grade I Listed status (that is, it is a legally protected landmark), being recognized as among the world's finest ironwork structures (Figure 1).

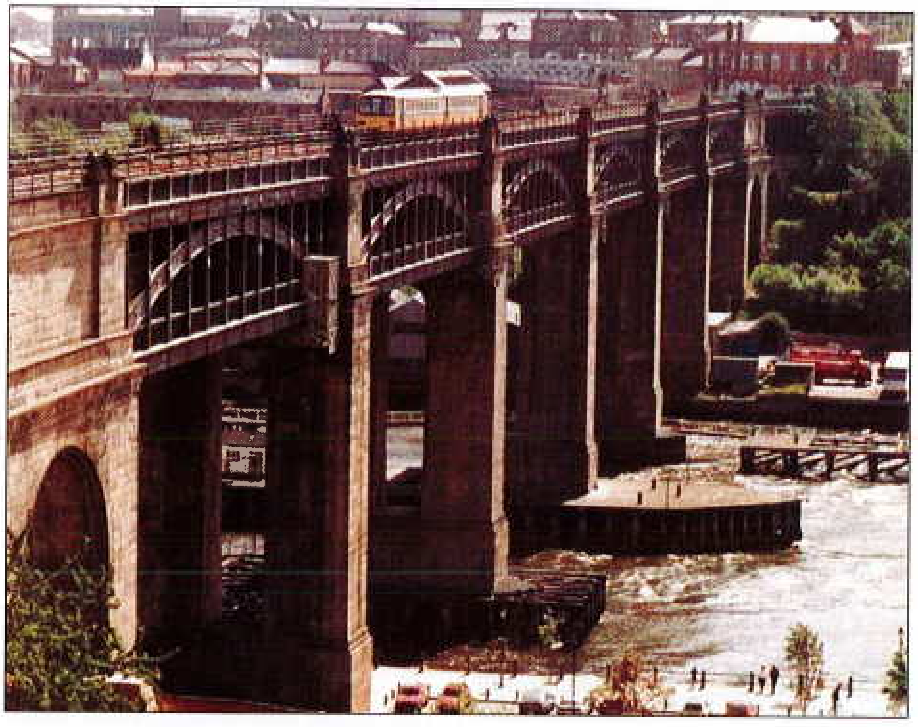

Figure 1 — Stephenson's High Level Bridge viewed from Newcastle.

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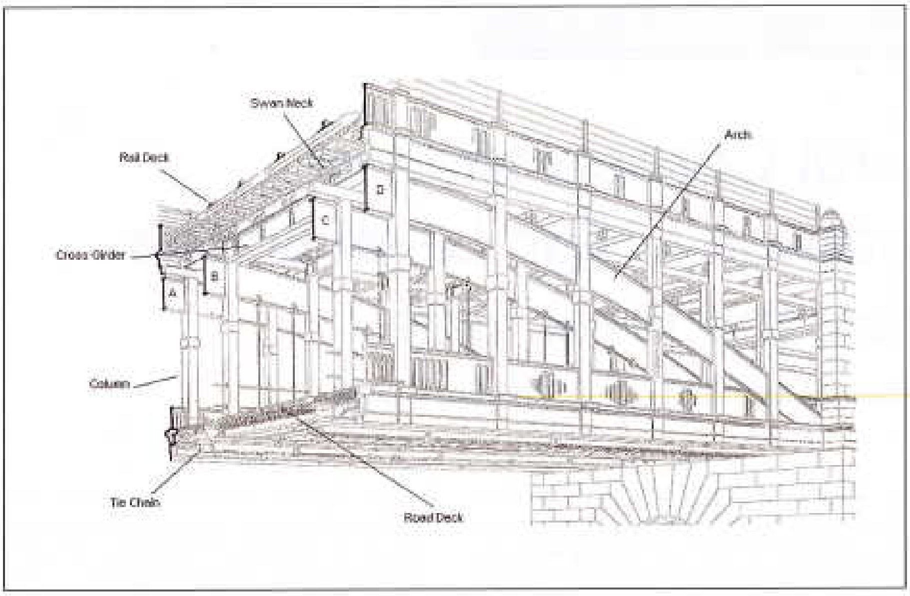

Figure 2 — Isometric cross-sectional drawing of a High Level Bridge span.

The Bridge Today

The current owner of the Bridge, Network Rail, also owns a number of cast iron arch underbridges, but these work predominantly in compression; the High Level Bridge is a rare example of an underbridge supporting a live railway with cast iron members in tension bending.

The use of cast iron as a structural material for arched bridges had been pioneered for the Ironbridge at Coalbrookdale in 1799 (Addyman and Fawcett, 1999). Although strong in compression, cast iron is known to be brittle in tension. Failures of a number of cast iron bridges in the late 1800s led to the decision to remove or strengthen all cast iron bridges subjected to live load bending. These failures have all, however, been attributed to the presence of major, internal casting anomalies.

Stephenson's High Level Bridge has six main spans, each comprising four cast iron arch ribs (tied arches) restrained by wrought iron tie chains anchored at the bearings (Figure 2). The road deck is suspended beneath the arch ribs by wrought iron bars, which support cast iron longitudinal girders and, originally, a timber deck. The rail deck is carried by cast iron U-shaped cross girders, which are supported above each arch rib by cast iron columns; longitudinal timber weigh beams supported in cast pockets of the cross girder then directly support the rails of the three tracks.

When the High Level Bridge was opened, maximum total train weights were around 445 000 kg (490 tons). Train weights can now be almost three times this value, although freight movements are restricted. Initially, the bridge with its three tracks carried an average of 25 trains per day; by 1890 there were 800 train movements per day.

Various modifications have been made to the bridge over its lifetime to accommodate these loading changes. The rail deck cross girders were strengthened in 1890 by the addition of steel "swan neck" girders. In 1922, the road deck was strengthened by replacing the timber cross beams with steel cross girders, to take double-track electric tramcars (previously, horse-drawn trams had used the bridge). In the 1950s, the timber weigh beams and rail decking were replaced by a heavier ballasted track. In 1990, the three train tracks were reduced to two.

Structural Health Assessment of the High Level Bridge

A principal inspection and structural assessment highlighted elements within the structure that were below strength or were in need of repair or replacement. Various strengthening and repair works have been designed over the last four years and have been implemented while the bridge was open to both road and rail traffic by the framework contractor, May Gurney. In February 2005, however, the road deck was closed to allow the replacement of the primary road deck elements to begin. Removal of the road deck paint (some 32 coats), resulted in the exposure of latent anomalies and it was subsequently decided that paint should be removed from the entire structure to allow detailed nondestructive testing (NDT) and subsequent repainting. Working closely with Bridges NDT and Dave Ackley NDT, Mott MacDonald are close to completing the NDT of 57,000 m^2 (614 000 ft^2) of metalwork.

What follows is a description of some of the more interesting problems encountered during the nondestructive testing of the High Level Bridge.

NDT of Metallic Elements

All metallic elements have been grit blasted to remove paintwork and corrosion to allow, where possible, 100% visual testing. Generally, a complete span has been blasted and handed over for NDT without holding primer. Magnetic particle testing with alternating current yokes, using black ink and white contrast paint, is used to confirm any visual indications. Once confirmed, depth sizing, using A-scan ultrasonic testing, was carried out. Where geometry or access to the anomaly inhibited ultrasonic testing, other NDT techniques were used. These included digital radiography, alternating current field measurement and alternating current potential drop.

Typical problems on the bridge can be split into six groups:

- original casting discontinuities
- discontinuities in the original dropforge welding
- fatigue induced anomalies
- corrosion
- weather damage
- heat damage.

Those anomalies classed as "original casting discontinuities" can be further divided into the following categories (Figure 3):

- gas holes and gas porosity caused by either hydrogen in the molten metal at

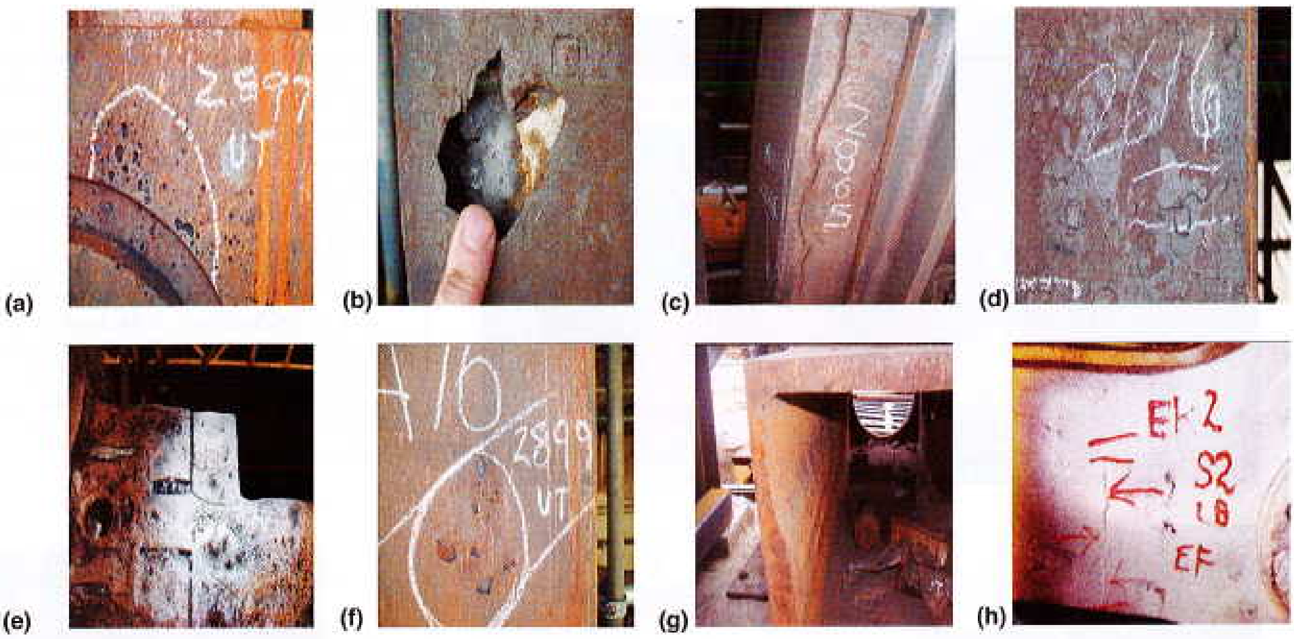

Figure 3 — Original casting and forge discontinuities: (a) gas porosity and holes; (b) gas entrapment; (c) cold shuts; (d) unfused chills; (e) hot tears; (f) sand inclusions; (g) uneven wall thickness; (h) lack of weld fusion.

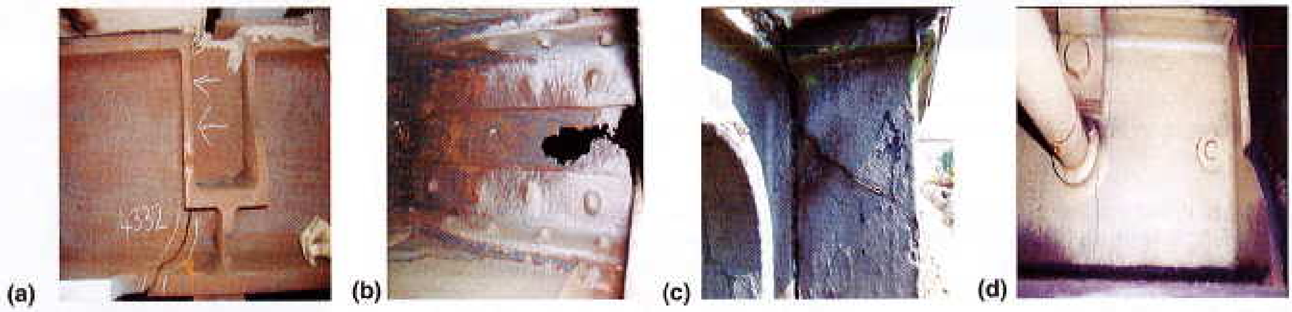

Figure 4 — Other forms of anomalies encountered on the bridge: (a) fatigue stress cracks; (b) girder corrosion; (c) weather damage; (d) heat damage.

tapping or dampness in the mold, resulting in gas holes or wormholes

- gas entrapment (airlock) caused by air that has been trapped in the mold during pouring, resulting in a large void
- cold shuts caused by low metal temperature splashes of the molten metal onto the mold or streams of molten metal not fusing in the mold, resulting in lack of fusion
- unfused chills and chaplets caused by low metal temperature or contamination of the insert
- hot tears caused by lack of mold/core breakdown or restriction during the solidification process
- inclusions caused by sand erosion at the ingate during pouring or poor mold closure, resulting in pieces of sand scattered on up-surfaces of the casting
- uneven wall thickness caused by misplaced cores.

The original dropforge welding discontinuities were caused by low metal temperature, contamination or lack of hammering on the joint, resulting in lack of fusion at the joint.

The other problems encountered on the bridge (Figure 4) include fatigue anomalies caused by cyclic loading, resulting in cracks in various structural members including tie chains, columns, cross girders and longitudinal girders. Corrosion, caused by wind and water erosion and salt corrosion, resulted in section loss. Weather damage, caused by freeze/thaw action, resulted in cracks and ruptures. Heat damage, caused by a warehouse fire beneath Span 6 in 1886 (Addyman and Fawcett, 1999), resulted in quench cracks.

Road Deck Longitudinal Girders

While dismantling the road decking and blast cleaning the retained members, cracking of the original cast iron longitudinal girders was exposed (Figure 5). The same areas were targeted for testing throughout the structure and magnetic particle and ultrasonic testing were utilized extensively. A single girder was also extracted to validate the ultrasonic testing results. The decision was made to investigate the feasibility of providing an alternative load path. The requirements for working on protected structures in the UK, however, stipulate that the original members must be retained and that any solution should be concealed from the bridge users.

The solution was a compact fabricated steel overbeam that suspended the cross girders of both the road deck and pedestrian walkway and removed all vertical loads from the original cast iron girder. The overbeam was designed to lie beneath a

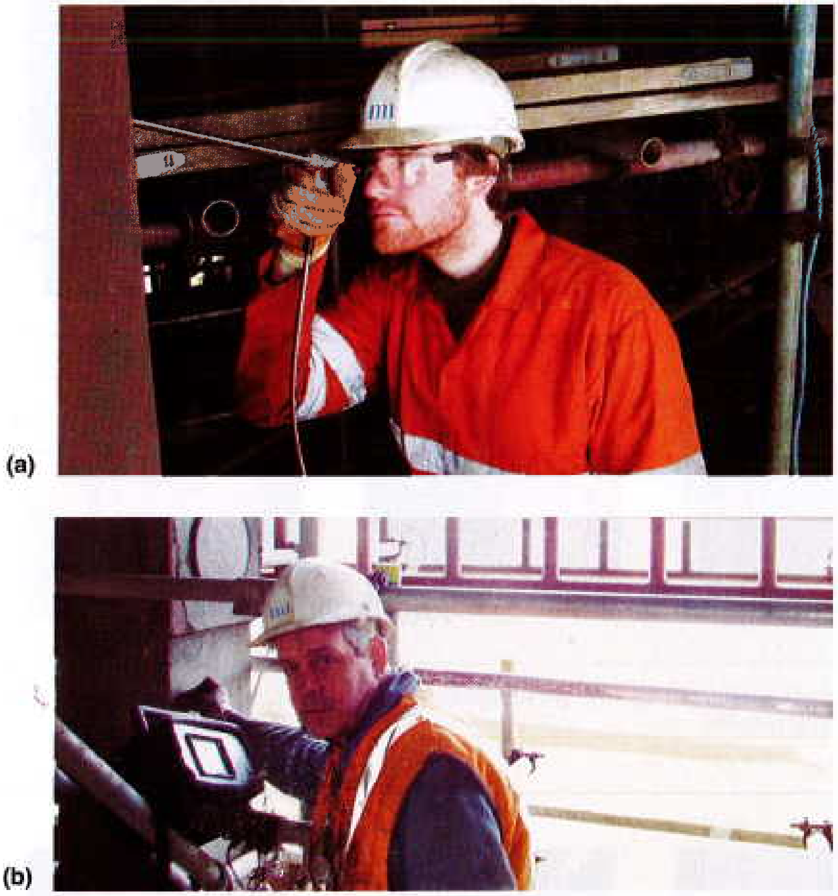

Figure 5 — Nondestructive testing of the columns: (a) visual testing; (b) ultrasonic testing.

maintenance cover plate at footpath level and with minimal intrusive fixings could, if required, be removed in the future.

Rail Deck Cross Girders

The cast iron rail deck cross girders were described within the assessment as being below strength. There are a total of 136 cast iron rail deck cross girders within the bridge, and a high percentage required strengthening. The chosen strengthening solution was an innovative post-tensioning system that required steel fabricated anchorages to be bonded and dowelled onto/into the cast iron of the cross girders such that the strands could be tensioned to reduce the tensile bending stresses. The solution was, however, not cheap.

In parallel with the detailed design of the strengthening solution, it was decided to undertake a study into the structure-specific fatigue performance of cast iron and, in particular, the cast iron rail deck cross girders. In order to do this, it was proposed to extract and remove three cross girders from an untrafficked area of the bridge for full scale testing. The testing was carried out in stages, each stage being more intrusive than the last but providing greater understanding of the material. The intention was to justify an improvement on the limiting stress approach stipulated within the assessment code and thereby reduce, or even eliminate, the need for the expensive strengthening.

Each stage relied heavily upon NDT. Initially, ultrasonic testing was carried out on a representative sample of cross girders throughout the bridge in order to satisfy those involved that the girders to be removed were representative of those in the main spans. MT, RT and UT were utilized to investigate the existence of surface breaking and internal casting discontinuities or stress cracks. In-situ replication and hardness testing were also utilized throughout the bridge to confirm the microstructure and provide an estimate of ultimate tensile strength. Ultrasonic testing was also utilized to verify material thicknesses. Brinell hardness results showed that the material had an average hardness value of 151 H_B, corresponding to an ultimate tensile strength of 125 MPa (18 100 lb/in.^{2}).

Small samples were then removed for hardness testing, and metallurgical examinations were carried out to provide information on the physical and chemical composition of the material. Chemical and microstructure analysis showed the cast iron microstructure was mainly large flake graphite in a matrix of pearlite and steadite.

Larger material specimens were then extracted from the cross girders, from which tensile test pieces were machined. These were used to confirm the ultimate tensile strength, modulus of elasticity and fatigue strength of the material. Fatigue testing was carried out to establish a statistically reliable fatigue performance spectrum (*S-N*) curve.

Following the small scale fatigue trial, three full cross girders were removed from an untrafficked part of the bridge rail deck and were sent for full scale fatigue trials (Figure 6). On arrival, all the girders were cut into test pieces. A total of 10 full scale fatigue specimens were tested at various stress ranges.

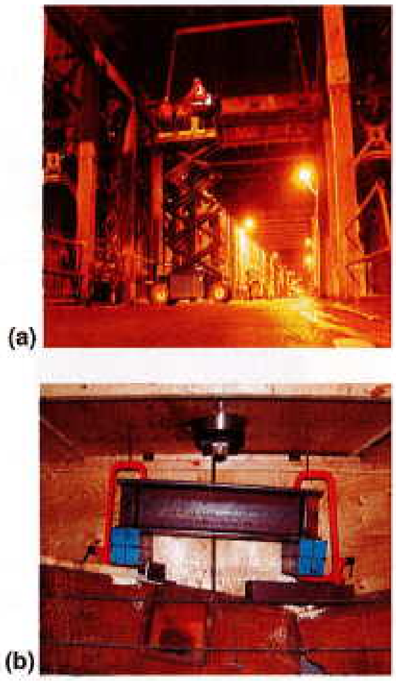

Figure 6 — Testing of the cast iron cross girders: (a) removal from the bridge; (b) full scale fatigue testing.

The first six samples were tested in a specially designed fatigue test rig and subjected to cyclic loads of predetermined range to a maximum of 2 million cycles.

The fatigue damage history was determined by detailed examination of historical records. Account was taken of modifications, such as change in dynamic effects, resulting from ballasting, relocating and reducing the number of tracks, as well as from other stress related aspects of the bridge's history.

Further testing was carried out utilizing the quarter sections, which were mounted within a smaller test rig and subjected to lower stress ranges at a higher number of cycles (up to 5 million), such that a nonpropagating stress limit could be established.

A condition of Listed Building consent was that all of the test pieces were to be reassembled, repaired and reinstated back into the bridge. The girders are therefore to be mechanically stitched and reinforced using bonded carbon fiber reinforced plates. Reporting and conclusion of the testing program is ongoing.

Tie Chains

The wrought iron tie chains between the arch bases are primary structural members as they act as a simple tie member restraining the longitudinal thrust of individual arches (Figure 7). Each span has six tie chains, two per inner arch and a single tie chain on each outer arch. Individual tie chains are made up of four links in either 3 or 6 m (9.8 or 19.7 ft) lengths that are connected together by fitted pins at a knuckle joint to form a tie chain 42 m (137.8 ft) in length.

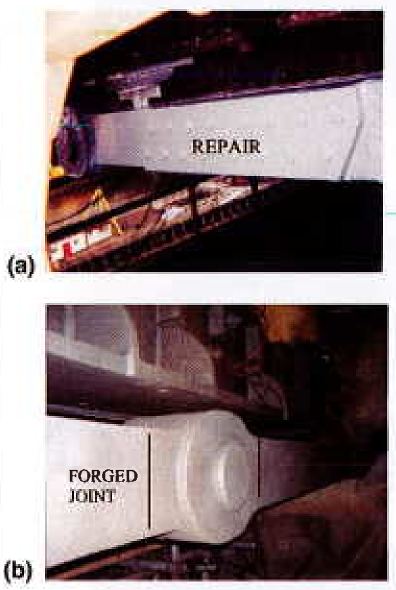

Figure 7 — Tie chain link assembly: (a) showing repair; (b) showing forged joints.

The links were manufactured in three sections, consisting of two forged eyes and a flat plate. The sections were welded together by drop forging to produce the finished link. The links were all assembled into chains, with an eight eye-connecting knuckle. Historical evidence shows that all the links were proof loaded to 57 000 kg (63 tons) before they were incorporated into the permanent works.

An examination of the structure in 1999, prior to any blast cleaning work, uncovered a single fracture of a tie chain link at the forged joint between the eye and main body of the link. This was attributed to rust forcing between adjacent tie chain links (imparting secondary load effects onto the forge weld), and the probability that the forge weld contained anomalies that had not been highlighted during the original proof loading. The fracture was repaired by a combination of welding and steel splice plates in order to prevent locked-in stresses due to welding.

In addition to visual and magnetic particle testing, various techniques of NDT were considered, with ultrasonic testing deemed the most appropriate. Radiography was not considered because of the isotope strength that would be required to penetrate the chains.

Ultrasonic testing of wrought iron can be difficult due to the microstructure. Siliceous slag (stringers) and graphite in wrought iron can give spurious indications or absorb the ultrasonic beam. Benchmarking trials were undertaken to determine the limitations of the ultrasonic method.

An eye of a tie chain link was forged to the same profile from currently available wrought iron of a similar metallurgical composition to the wrought iron found on the structure. This allowed the benchmarking of the anomalies around the knuckle eye and adjacent forge welded joints.

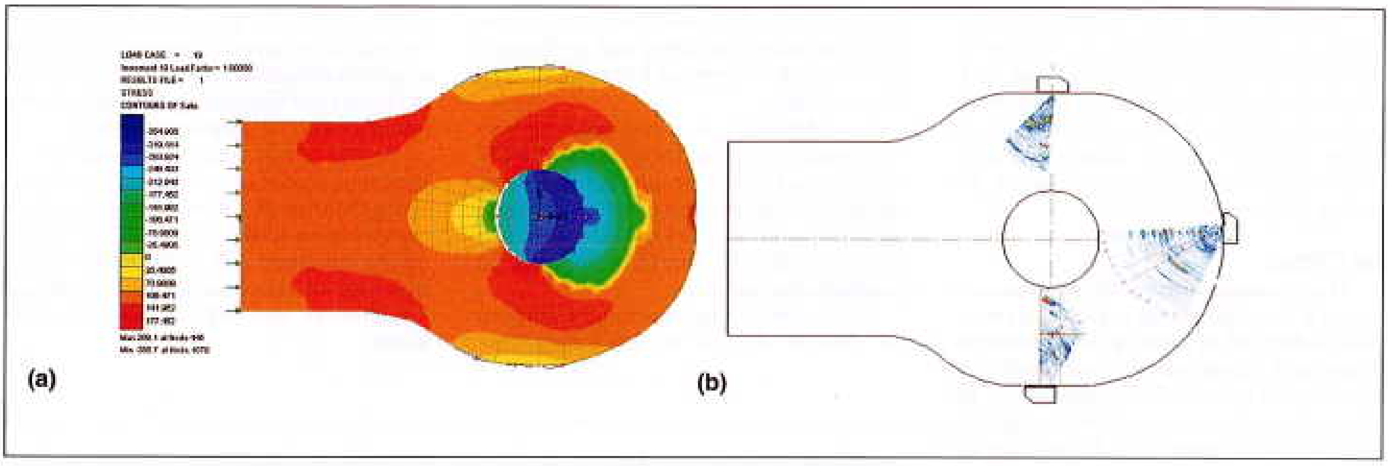

Figure 8 — Tie chain link data: (a) from finite element analysis; (b) from ultrasonic scanning.

Ultrasonic testing and digital radiography were carried out on the test piece to locate any anomalies inherent within the material. A 3 mm (0.12 in.) diameter hole, half through-thickness, was drilled in the main body of the link and used to calibrate the sensitivity of the ultrasonic equipment.

Benchmarking was undertaken by drilling a number of 3 mm (0.12 in.) diameter holes at known stressed areas determined from finite element analysis of the tie chain (Figure 8). The depth of each 3 mm (0.12 in.) hole was increased until the ultrasonic operator could consistently and accurately locate the hole. All benchmarking was undertaken replicating actual testing conditions on the structure, with holes concealed from the ultrasonic operator and all testing being limited to the accessible faces of the tie chains. Upon completion of the benchmarking, the test piece was used as a calibration block. Benchmarking has allowed the NDT of the tie chains to proceed with a greater degree of confidence.

Remedial Measures

Structural repairs to the bridge have all been subject to Listed Building consent and comply with agreed conservation principles so that they are sympathetic to and in keeping with this heritage structure. Where possible, repairs or replacements have utilized traditional materials, and where not possible they have been designed to be reversible. For all anomalies, careful consideration has been given to the cause, whether they have resulted during manufacture (as in the case of many of the casting discontinuities), or from local or global loading effects (whether from live loads or environmental loads). Careful consideration is also given to the compatibility of materials, so that local strain incompatibilities and bimetallic corrosion is avoided.

Two methods of repair that have been popular with conservation officers include carbon fiber and steel bonded plates and mechanical stitching due to their subtle visual effects (Figure 9). Mechanical stitching

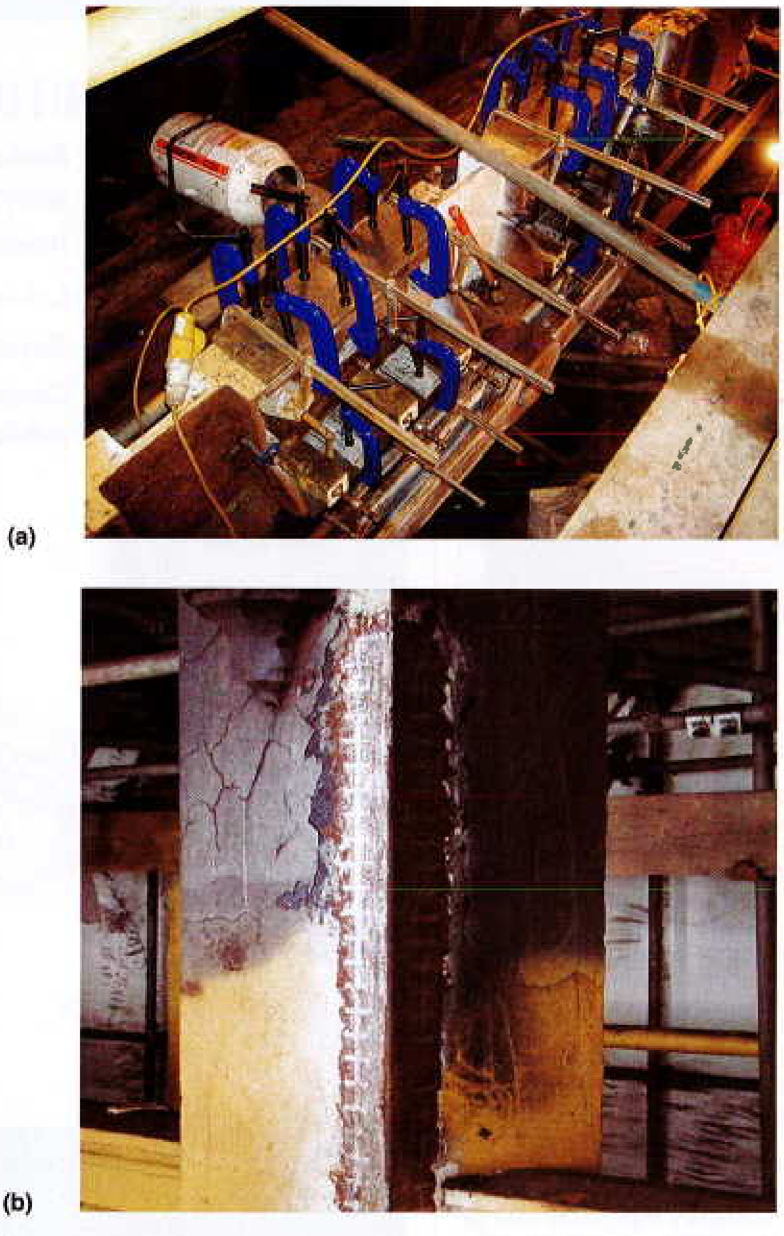

Figure 9 — Two methods of repairs: (a) plate bonding; (b) mechanical stitching.

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requires the incorporation of nickel alloy keys that are peened into jig-drilled holes across cracks.

When it was first built, the total cost of Stephenson's High Level Bridge was £491,000 — equivalent to £40 million in today's money. The current estimated cost of the refurbishment work is £27 million. Once completed, this restoration project will extend the useful life of the bridge for at least another 50 years.

Conclusion

The testing of the High Level Bridge has provided the opportunity to add to the information available on the fatigue performance and nondestructive testing of cast iron. This will enable industry to make better-informed decisions when dealing with the conservation of protected cast iron structures.

Acknowledgments

The authors would like to thank Alistair Cooke, of Mott MacDonald, for his help in preparing this paper.

References

Addyman, John and Bill Fawcett, *The High Level Bridge and the Central Station: 150 Years across the Tyne*, Stockton on Tees, England, North Eastern Railway Association, 1999.

What's in a Name?

To ensure proper usage of NDT terminology, ASNT would like to remind readers that the following terms should be used when referring to the certification status of NDT personnel.

- Personnel certified by examination by their employer (or through contracted examination agencies or training facilities other than ASNT) should be listed as being "certified in accordance with *Recommended Practice No. SNT-TC-1A*." In addition, employer certified Level IIIs may be called "Level IIIs."
- Personnel certified by ASNT examination may be referred to as being "ASNT certified" and may be called by the proper name "ASNT NDT Level III."
- Personnel certified through the ASNT Central Certification Program (ACCP) may be said to be "ASNT certified" or, more properly, to have been certified as an "ACCP Level II" or "ACCP Professional Level III."

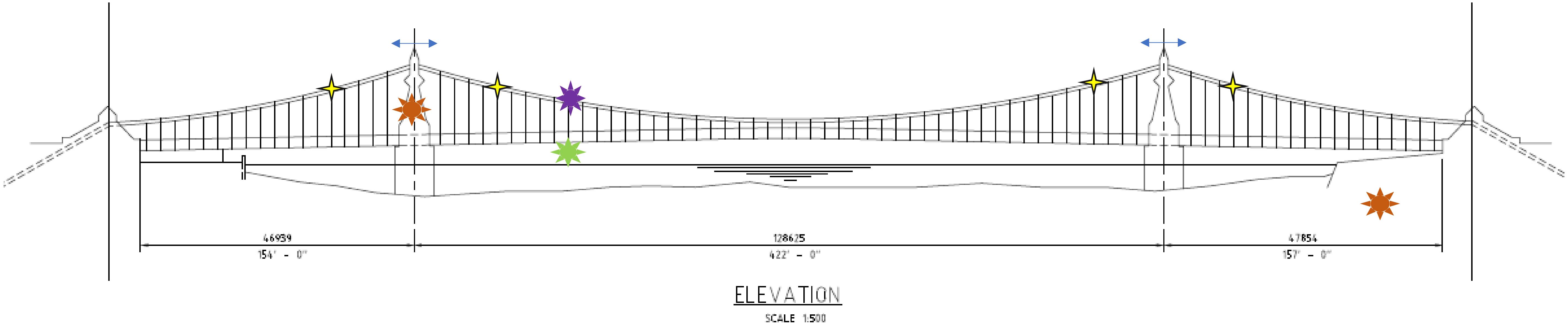
Adherence to these simple distinctions will reduce confusion in the industry.

C. Proposed Instrumentation

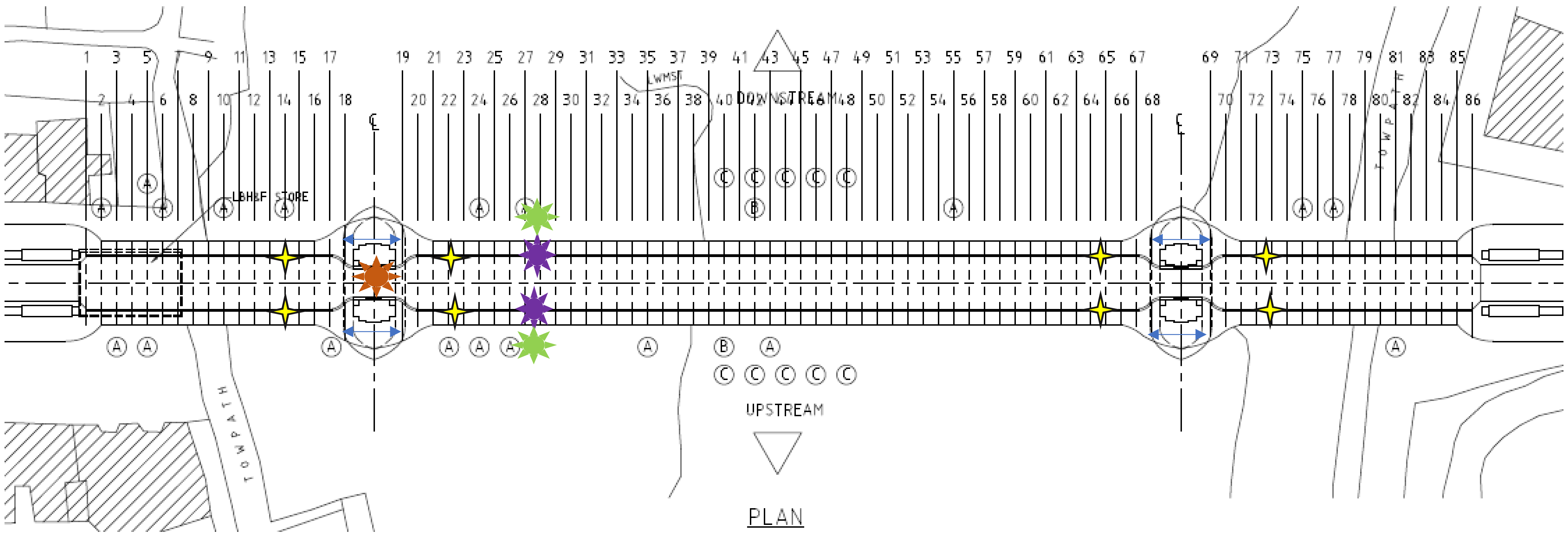
(referred to in Section 1.4)

Instrumentation for Long Term Monitoring – to end of Contract

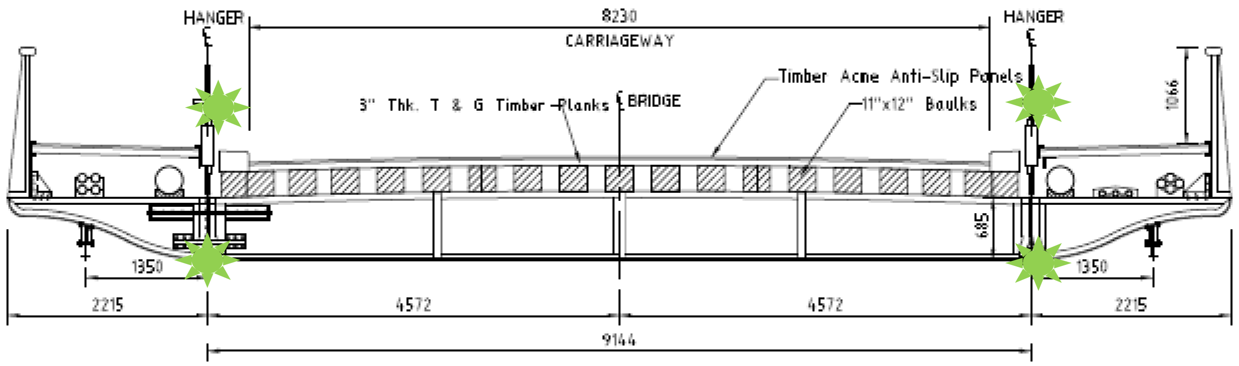
Elevation



Plan



Cross Section



CROSS SECTION LOOKING SOUTH

SCALE 1:50

Key:

LVDT at bearings

Prism for total station (100% on 1 face, 75% on the other face)

Thermocouple on truss – top and bottom flange

Thermocouple on chain – inner, outer and centre leaves

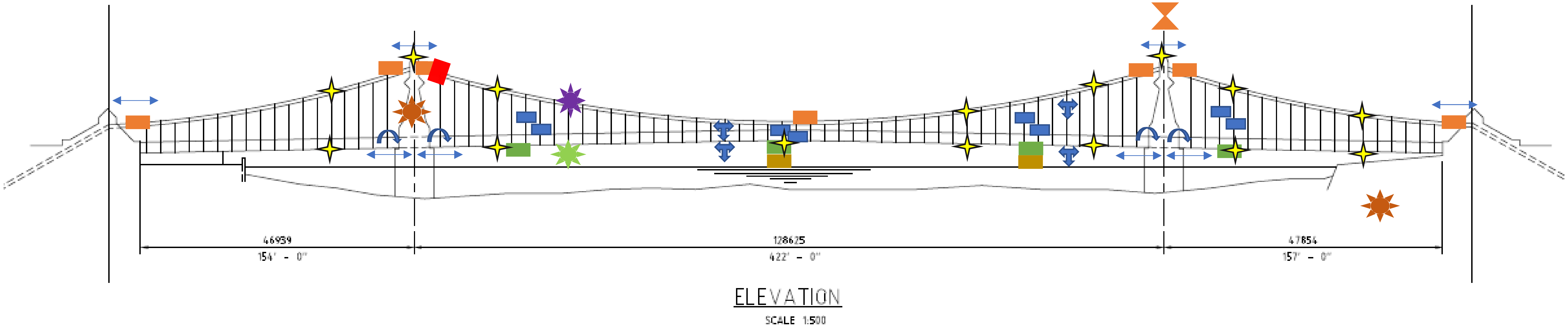
Thermocouple in tower and ambient thermocouple

Monitoring Equipment Schedule – Long Term

Equipment	Location	Number – Long Term (To end of Contract)
LVDT	4 tower bearings	4
Prisms	Cables and deck level + 2 control prisms	8
Thermcouples	6 on Cables, 4 on Main Truss, 1 Tower, 1 Ambient	12

Instrumentation for Short Term Monitoring – to end of Investigations Phase

Elevation



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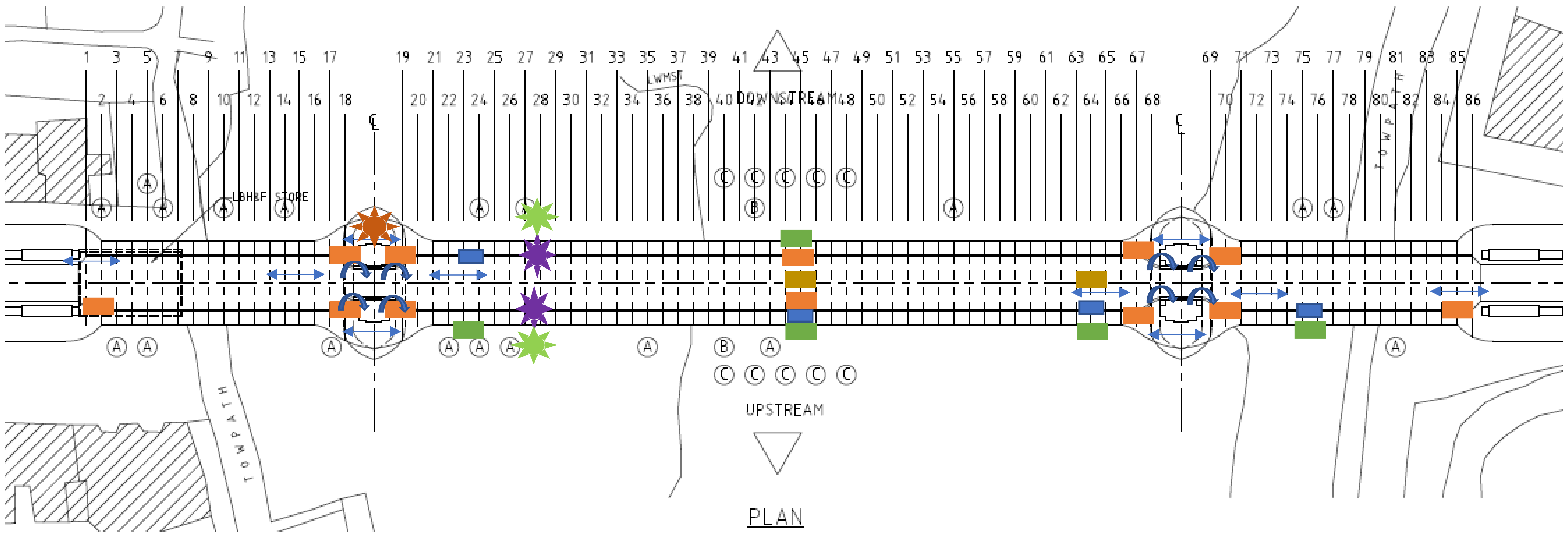
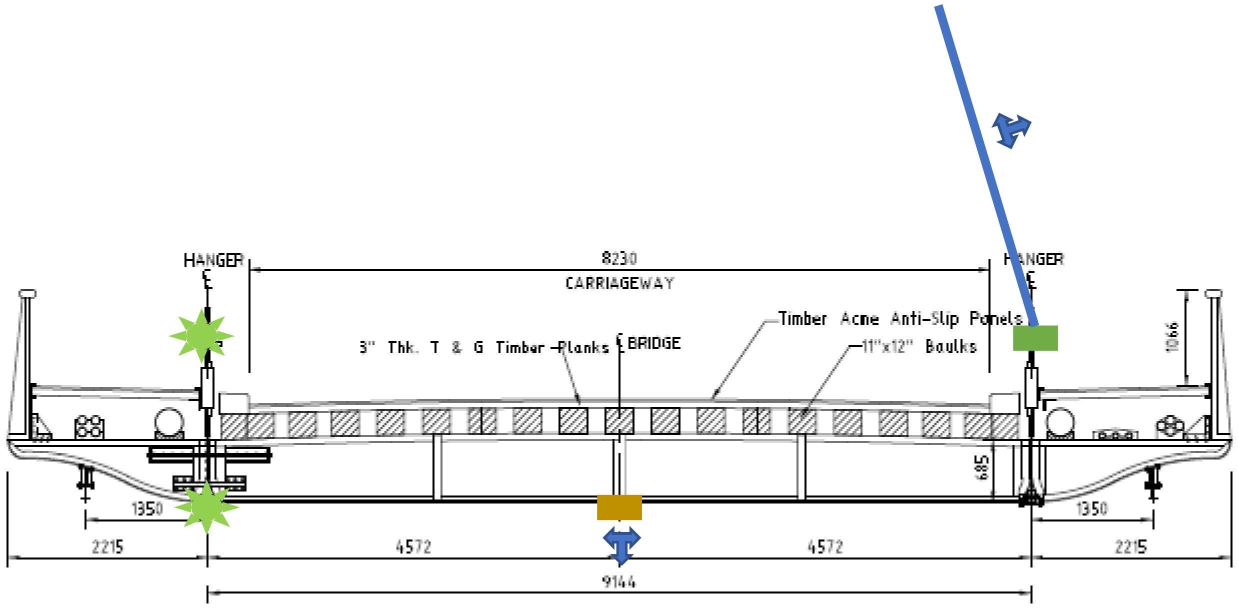


Cross Section

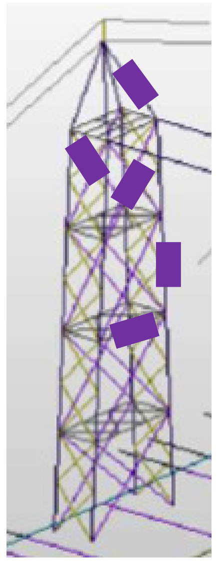


CROSS SECTION LOOKING SOUTH

SCALE 1:50

Tower

Strain measurements will be taken at 5 locations in 2 towers (diametrically opposite towers)



Key:

- Strain measurement on hanger (2 adjacent hangers per location – i.e. one to top chain and one to bottom chain)
- Strain measurement on longitudinal truss top chord
- Strain measurement of outer and inner leaves of chain
- Strain measurement of tower truss elements
- Strain measurement bottom flange of cross girder
- LVDT at bearings
- Inclinometers to pin bearings
- Inclinometers to hanger and corresponding cross-girder
- Prism for total station (100% on 1 face, 75% on the other face)
- Anemometers
- Strain measurement at 6 locations at eye of chain
- Thermocouple on truss – top and bottom flange
- Thermocouple on chain – inner, outer and centre leaves
- Thermocouple in tower and ambient thermocouple

Monitoring Equipment Schedule – Short Term

Equipment	Location	Number – Short Term (To end of Investigations Phase)
Strain Gauge	Hangers – at dead load strain monitoring locations	8
Strain Gauge	Longitudinal Truss - at dead load strain monitoring locations	5
Strain Gauge	Chains – inner and outer leaves – at dead load strain monitoring locations + chain saddles	24
Strain Gauge	Tower elements - at dead load strain monitoring locations	10
Strain Gauge	Cross girder	2
Strain Gauge	Chain eye	6
LVDT	4 tower bearings and at 2 anchorages	10
Inclinometers	Pin bearings	8
Inclinometers	Hangers and corresponding cross girder	4
Prisms	Cables and deck level + 2 control prisms	27
Anemometers	Top of towers	1
Thermcouples	6 on Cables, 4 on Main Truss, 1 Tower, 1 Ambient	12

D. M56 Helsby Junction Viaduct Assessment Report

(referred to in Section 1.5)

Appendix A. SLS Bearing Assessment Calculations

Appendix B. Jacking Options Drawings

Appendix C. Jacking Options Assessment Calculations

[redacted]

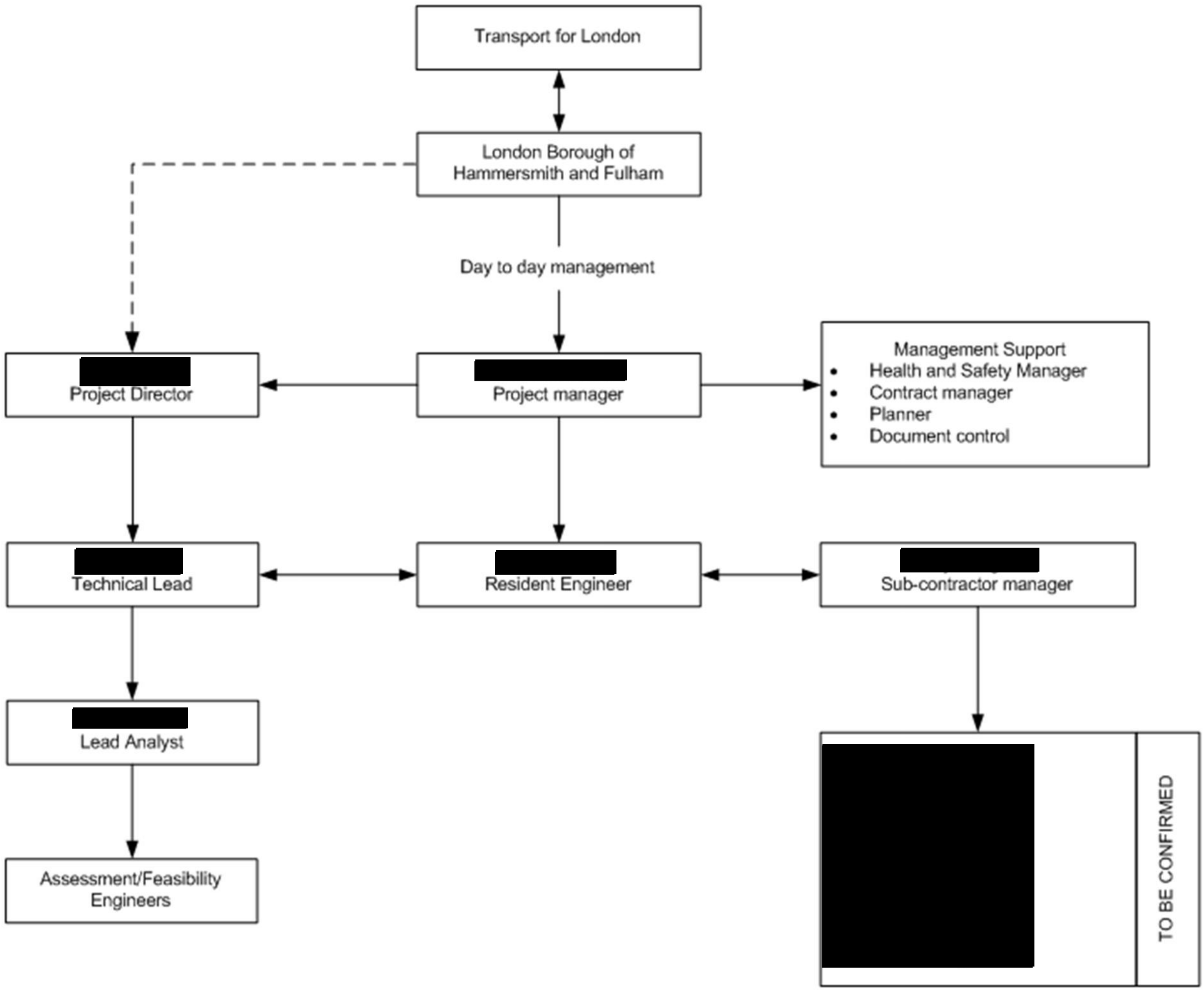
Appendix D. Seized Bearings Assessment Calculations

E. Checking and Approval Procedure

(referred to in Section 1.8)

Hammersmith Bridge

Mott MacDonald Organisation Chart



G. Templates for Factual and Interpretive Reports

(referred to in Section 1.4)

Hammersmith Bridge Strengthening and Refurbishment Works

Site Testing and Monitoring: Interpretive Report

10 May 2017

London Borough of Hammersmith and Fulham

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Hammersmith Bridge Strengthening and Refurbishment Works

Site Testing and Monitoring: Interpretive Report

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London Borough of Hammersmith and Fulham

Issue and Revision Record

Revision	Date	Originator	Checker	Approver	Description
P1	10/04/17	[redacted]	[redacted]	[redacted]	Draft for comments
P2	10/05/17	[redacted]	[redacted]	[redacted]	First Issue

Information class: Standard

Contents

Executive summary 1

1 Introduction 2

1.1 Scope of Report 2
1.2 Background 2
1.3 Methodology 2
1.4 Limitations 2

2 Site works 3

2.1 Visual inspection of general condition 3
2.2 Non-Intrusive Investigation 3
2.2.1 Photographic Survey 3
2.2.2 Asbestos Survey 3
2.2.3 Electrical and Lighting 3
2.2.4 Topography and Geometric Survey 3
2.2.5 Ecology 3
2.2.6 Fatigue involving NDT 3
2.2.7 Hanger vibration tests 3
2.2.8 Bridge monitoring 3
2.2.9 Load testing 3
2.3 Intrusive Investigation 3
2.3.1 Protective Coat Survey 3
2.3.2 Hanger Load Tests 3
2.3.3 Chain Load Tests 3
2.3.4 Material Tests 3
2.3.5 Schedule of test areas 3

3 Discussion 5

3.1 The investigation findings 5
3.1.1 Inspection observations 5
3.1.2 Overall conclusions 5

4 Factual Reports 6

5 References 7

Appendices 8

A. Site Layout

Executive summary

1 Introduction

1.1 Scope of Report

1.2 Background

1.3 Methodology

1.4 Limitations

2 Site works

2.1 Visual inspection of general condition

2.2 Non-Intrusive Investigation

2.2.1 Photographic Survey

2.2.2 Asbestos Survey

2.2.3 Electrical and Lighting

2.2.4 Topography and Geometric Survey

2.2.5 Ecology

2.2.6 Fatigue involving NDT

2.2.7 Hanger vibration tests

2.2.8 Bridge monitoring

2.2.9 Load testing

2.3 Intrusive Investigation

2.3.1 Protective Coat Survey

2.3.2 Hanger Load Tests

2.3.3 Chain Load Tests

2.3.4 Material Tests

2.3.5 Schedule of test areas

Table 1 Testing locations and scope

Sample Reference	Element / Location	Testing Scope

3 Discussion

3.1 The investigation findings

3.1.1 Inspection observations

3.1.2 Overall conclusions

4 Factual Reports

The factual data on which this report is based can be found in the following reports:

Report Title	Company	Reference	Link
Photographic Survey	CAN	383488-MMD-00-ZZ-RP-F4-0001	Photographic Survey
Asbestos Survey	XYZ Services	383488-MMD-00-ZZ-RP-F4-0002	Asbestos Survey
Electrical and Lighting	ABC Circuits	383488-MMD-00-ZZ-RP-F4-0002	Electrical and Lighting

Table 2 Schedule of Factual Reports

5 References

Appendices

A. Site Layout 9

A. Site Layout

MOTT MACDONALD

Hammersmith Bridge Strengthening and Refurbishment Works

Photographic Survey

10 May 2017

London Borough of Hammersmith and Fulham

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Contents

Executive summary 1

1 Introduction 2

1.1 Scope of Report 2
1.2 Background 2
1.3 Methodology 2
1.4 Limitations 2

2 References 3

Appendices 4

A. Site Layout 5

B. List of Photographs 6

C. Photographs 7

Executive summary

1 Introduction

1.1 Scope of Report

1.2 Background

1.3 Methodology

1.4 Limitations

2 References

Appendices

A. Site Layout 5
B. List of Photographs 6
C. Photographs 7

A. Site Layout

B. List of Photographs

C. Photographs
