Risk Review, Risk Assessment & Risk Management Plan

40062 Moylinn East Footbridge

CO402011-AMEY-SBR-XX-RP-CB-000001 Rev 1

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amey consulting

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Executive Summary

Amey Consulting were commissioned by The Department for Infrastructure (DfI) to undertake a Risk Review, Risk Assessment and Risk Management Plan of Structure No. 40062 Moylinn East Footbridge in accordance with the DMRB codes of practice – CS 465, *The Management of Post-Tensioned Concrete Bridges* and CS 466, *Risk management and structural assessment of concrete half-joint deck structures.*

This report combines a Risk Review and Risk Assessment as set out in both DMRB CS 465 and CS 466 and makes recommendations on the risk management of Moylinn footbridge.

The structure has been classified as **Very High Risk** with the half joints being considered the critical element. In addition, the uncertainty in the cause of the apparent 'sag' in the structure raises some uncertainty in regards the structural behaviour of the footbridge. Although considered 'very high risk', immediate intervention is not required – the structure does not meet the criteria to be classed as an 'Immediate Risk Structure' at this time – however this may be subject to change following the outcomes of further site investigation and structural analysis.

Given the very high-risk classification of the structure, the uncertainty surrounding the deflected shape and inherent risks associated with half joint and post-tensioned structures; this report agrees with the findings of the 2021 DfI Options Report in that the structure should be demolished as a long-term risk management option.

Until such time that the structure is demolished and replaced and is required to remain in service; it is recommended that the Risk Management Plan as outlined in Section 8 of this report is implemented. The overall risk rating of the structure will be updated following the findings of the Risk Management Plan. Following completion of the structural assessment outlined within the recommended Short Term Risk Management Plan the structure may then be classified as Immediate Risk and should be managed in accordance with CS470 for Substandard Structures.

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1. Introduction

1.1. Scope

1.1.1 Amey Consulting were commissioned by The Department for Infrastructure (DfI) to undertake a Risk Review, Risk Assessment and Risk Management Plan of Structure No. 40062 Moylinn East Footbridge in accordance with the DMRB codes of practice – CS 465, *The Management of Post-Tensioned Concrete Bridges* and CS 466, *Risk management and structural assessment of concrete half-joint deck structures.*

1.1.2 This report combines a Risk Review and Risk Assessment as set out in both DMRB CS 465 and CS 466 and makes recommendations on the risk management of Moylinn footbridge.

1.2. Structure Details

Construction

1.2.1 Moylinn East Footbridge was designed by the Craigavon Development Commission as Footbridge S73 in 1971 and constructed in 1973 as one of three footbridges within Project ER6D as part of the A3 Distributor Contract. The structure provides pedestrian access over the Lake Road A3 dual carriageway between Tullygally Roundabout and Lakes Roundabout. The footbridge has a Southeast – Northwest Alignment with the underlying A3 dual carriageway running Southwest – Northeast.

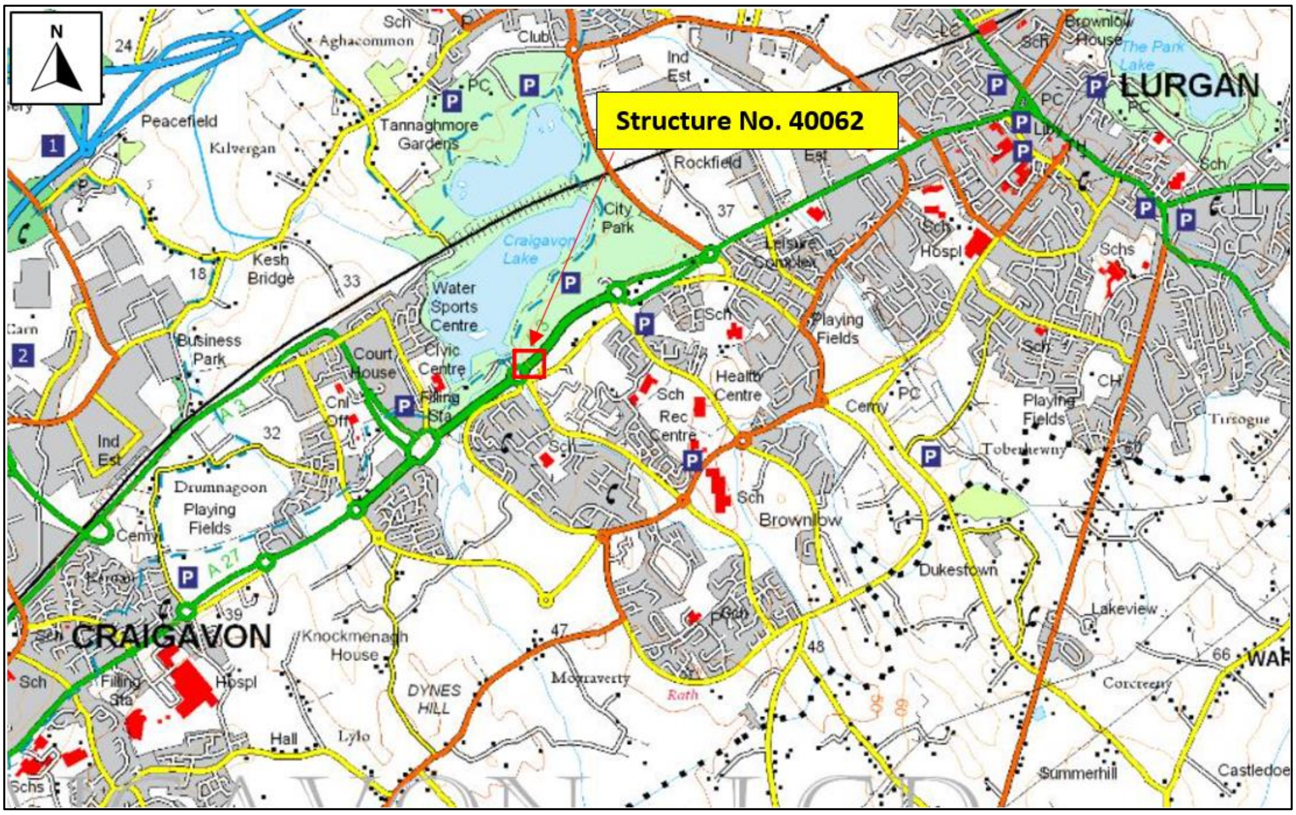


Figure 1: Structure Location

General Arrangement

1.2.2 The structure comprises variable depth cellular cast in-situ post-tensioned concrete cantilever decks simply supported on reinforced concrete abutments and continuous over reinforced concrete intermediate piers, forming a cantilever for the 'drop in span'. The suspended or 'drop in' span is a reinforced concrete deck of varying depth; it is simply supported upon half-joints by the cantilevered ends of the post-tensioned decks.

1.2.3 The footbridge comprises 3 spans. The central and outer spans measure 31.45m and 31.0m respectively. The footbridge has an overall square width of 4.115m, carrying a 3.655m footway as shown in Figure 2 below.

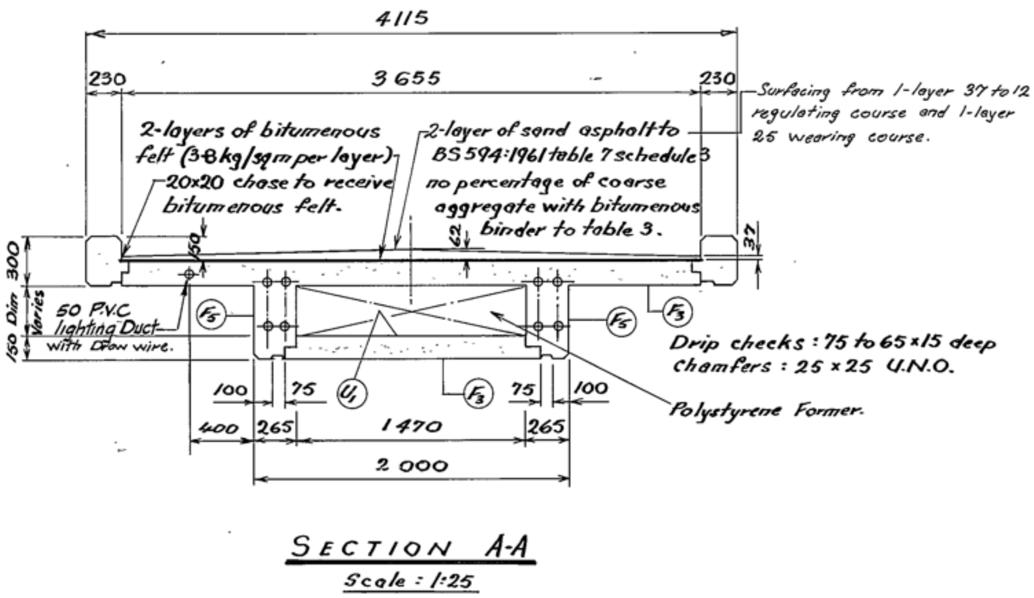


Figure 2: Deck cross section (Extract from Record Drawing ER6D/S73/1)

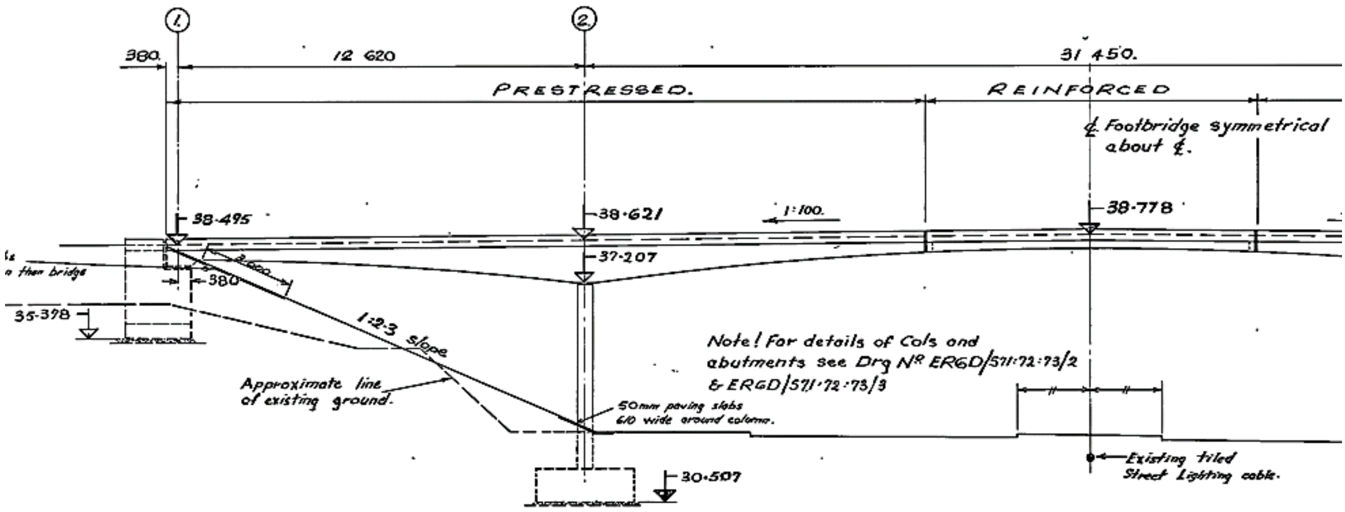


Figure 3: Elevation of bridge (Extract from Record Drawing ER6D/S73/1)

1.2.4 The A3 Lake Road below the structure has dual carriageways in both directions. The minimum headroom measured to the deck soffit during previous inspections was 5.153m as measured along the outer lane of the Northbound carriageway.

Sub-structure

1.2.5 Each end of the deck is simply supported by half-joints on reinforced concrete bank-seat abutments with elastomeric rubber bearing pads. The post-tensioned cantilevers are continuously supported over the cast in-situ reinforced concrete intermediate piers with 2 No. elastomeric rubber bearing pads. The suspended reinforced concrete deck is supported by half-joints on reinforced concrete bank-seat abutments with 2 No. elastomeric bearing pads positioned on the nib of the half-joint.

1.2.6 The reinforced concrete abutments are 1.2m x 1.98 wide and supported by 3.2m wide x 0.45m thick pad foundation as shown in Figure 4 below. There are no records of the ground conditions providing bearing the abutments.

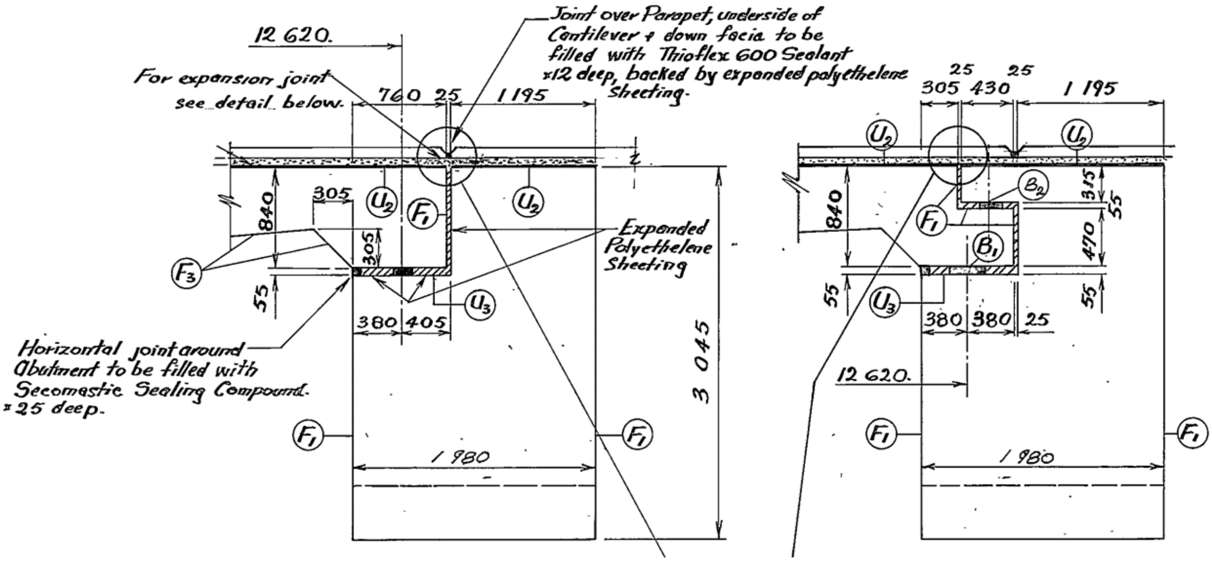


Figure 4: Reinforced concrete abutment (Extract from Record Drawing ER6D/S73/1)

1.2.7 The reinforced concrete intermediate piers have a depth of 0.5m and taper in width from 1m wide at the base to 2m at the top. The piers are supported on reinforced concrete pad foundations. There are no records of the ground conditions providing bearing the pier foundations.

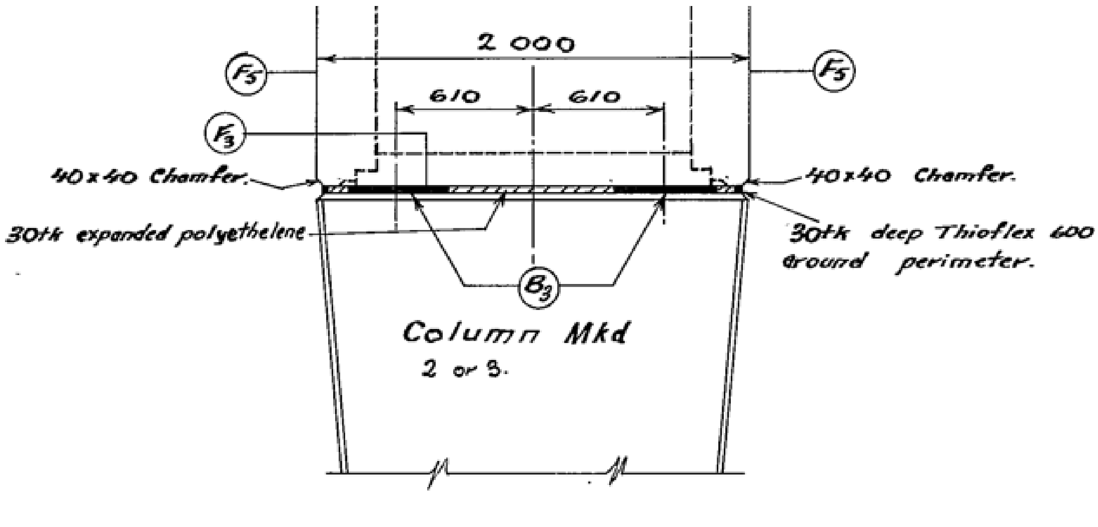


Figure 5: Intermediate Pier (Extract from Record Drawing ER6D/S73/1)

Bearings and Articulation

1.2.8 At the abutments, intermediate piers and half-joints, laminated rubber bearings are provided at supports to facilitate translation and rotation. The bearings are specified in record drawing ER6D/S73/1 as supplied by Andre Rubber Co. Ltd. At abutments there are 4 No. 229x203x55mm bearing pads. At the intermediate piers there are 2 No. 432x165x30mm bearing pads. At half-joints there are 2 No. 229x152x12mm bearing pads.

1.2.9 A half joint arrangement is also present at the centre of each abutment. This top 'nib' provides a tie down for the end span and prevents the post-tensioned deck from rotating about the piers.

Movement Joints

1.2.10 The expansion joints at abutments are ETAG 032 Part 2 buried expansion joints (Formerly Type 1 BD 33) as shown in Figure 6 below. The joint comprises a 500mm wide by 3mm thick elastomeric neoprene pad and a 230mm wide x 3mm thick steel plate placed across the 25mm wide expansion gap to support the surfacing.

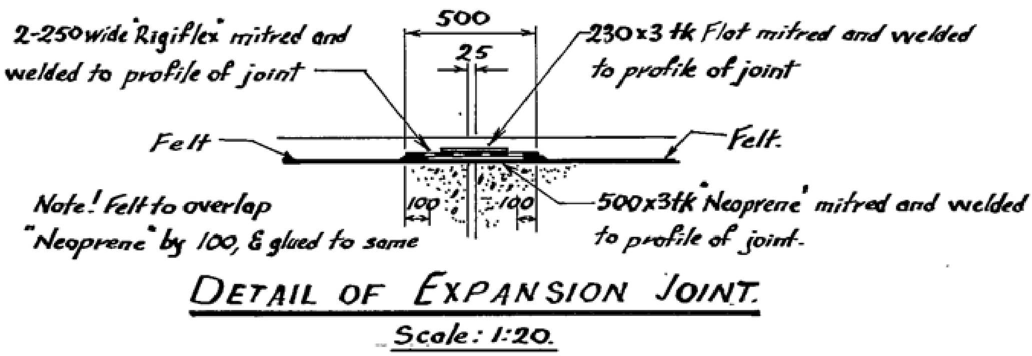


Figure 6: Expansion joint at abutments (Extract from Record Drawing ER6D/S73/1)

1.2.11 At the half-joints there are construction joints comprised of a 12mm gap with compressible filler board. The joint is sealed with Thioflex 600 to a depth of 12mm as shown in Figure 7 below.

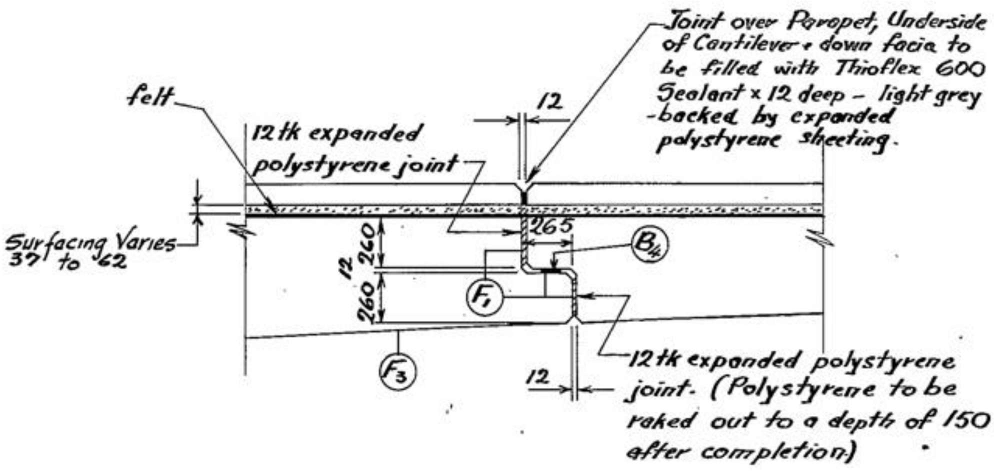


Figure 7: Expansion joint at half-joints (Extract from Record Drawing ER6D/S73/1)

Deck Drainage

1.2.12 There is no internal drainage system installed within the deck itself. The deck was originally designed with a fall longitudinally towards each abutment from a high point halfway along the length of the deck. The as-constructed deck has a single fall continuous fall from North to South.

Surfacing and Waterproofing

1.2.13 The deck waterproofing consists of 2 layers of bituminous felt applied to the top surface of the deck. The originally surfacing consisted of 2 layers of sand asphalt cambered with a high-point along the centreline of the deck with thickness varying from 37mm to 62mm. There are no records of the deck waterproofing or surfacing being replaced.

Access to the deck interior

1.2.14 There is currently no access to inspect the interior elements of the voided deck.

1.3. Post Tensioned Deck Description

1.3.1 The post-tensioned concrete deck is a cast in-situ voided box section with cast in-situ reinforced concrete cantilevers. The deck is simply supported at each abutment and continuous over intermediate piers; cantilevering to support a drop-in span supported on half joints. The voided deck varies in cross section to a maximum depth at the intermediate supports.

1.3.2 The soffit and top slab of the deck are 150mm thick. The two main webs which contain the post-tensioned tendons, are 265mm thick. The overall depth of the deck varies from 0.535m at abutments and half-joints to a maximum of 1.35m at the pier supports.

1.3.3 The top slab is cantilevered past either side of the webs by 1.050m, with a reinforced concrete upstand rising 150mm above the top of deck. Construction joints between the webs and the soffit slab suggest that the webs, top slab, and cantilevers were poured monolithically.

1.4. Post-Tensioning Details

Longitudinal Post-Tensioning and stressing sequence

1.4.1 Each cantilevered deck is longitudinally pre-stressed with post-tensioning specified in the record drawings as a CCL "Strandsaver" system. Each of the two deck webs contain 4 No. tendons in a 2x2 arrangement spaced 216mm horizontally. The record information does not specify the duct type or diameter.

1.4.2 The jacking force to each tendon is stated in the record drawings as 152.6 kN (34,300 lbf) This gives an initial jacking force of 1068 kN per tendon. Each tendon comprises 7 No. strands. Record information does not specify the strand or duct diameter, however, based on the stated jacking force per tendon, the tendons are likely 12mm diameter, 7 wire drawn strands. The tendons were stressed individually following a set sequence as shown on the record drawings with each strand likely anchored individually using external barrels and wedges – as is common at the time for a CCL system.

1.4.3 The tendons were stressed continuously from the 'dead end' at the half joint nib to the 'live' stressing end at the abutments. Details of the 'live end' anchorage is shown below in Figure 8.

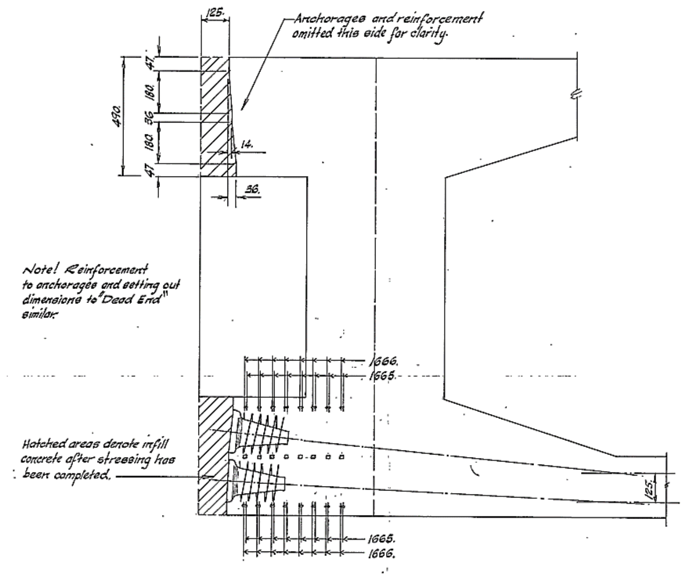


Figure 8: Plan view of end anchorage (Extract from Record Drawing ER6D/S73/7)

1.5. Protection to Post-Tensioning System

1. **Deck waterproofing**

   As there are no records of re-waterproofing projects completed to the deck, it is assumed that the current deck waterproofing system consists of 2 layers of bituminous felt applied to the top surface of the deck, as per the original construction details.

2. **Concrete deck**

   Concrete protection is provided via the in-situ concrete deck surrounding the duct. cracking of the concrete can provide a direct path for water and chlorides to penetrate the ducts and tendons.

3. **Tendon ducting**

   The tendon strands are contained within steel ducts; however, the record drawings do not contain information regarding the specific type or diameter of ducting, but corrugated steel would have been likely. The ducts act as a line of protection to the tendons from external sources of corrosion.

4. **Grouting**

   The tendon strands would have been grouted within the tendon ducts following stressing. Although no details are included regarding the type of grout mix used; it is assumed that the grout contained a suitable expanding agent which was standard practice at the time of construction. The grout around the tendons is considered the last line of protection from internal or external sources of corrosion. It is noted that previous PTSI's noted that the grouting condition was considered very poor with voids noted.

1.6. Half-joint Description

1.6.1 The half joints at each end of the post-tensioned cantilever deck consist of a lower bearing nib within the central portion of the deck, 970mm wide and projecting 265mm to form a bearing shelf to support a matching upper nib of the suspended span. The arrangement is as per Figure 9. It is noted that the post-tensioned end anchors are not located within the half joint, thus not contributing to its structural capacity.

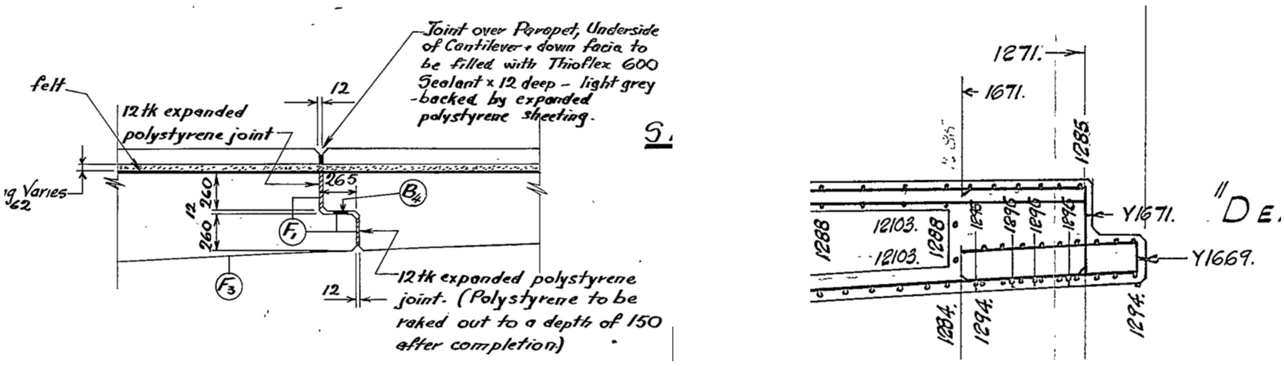


Figure 9: Half-joint details (Extract from record drawing 40062 general arrangement)

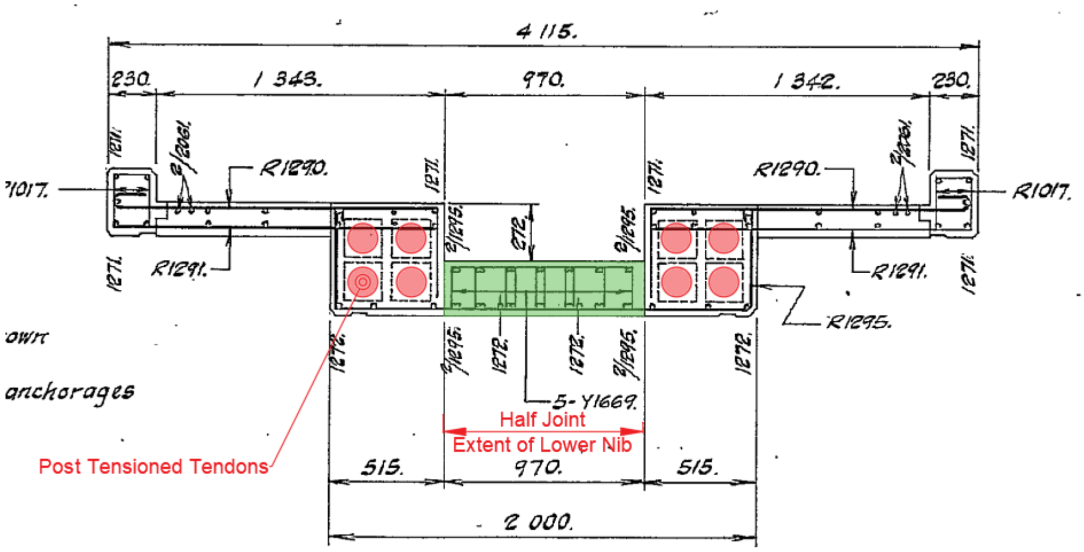


Figure 10: Deck cross section at half joint location

1.7. Materials

1.7.1 Table 1 below summarises the construction materials as per record drawings.

Material	Details
Prestressed concrete beams	52.5 N/mm^2
Reinforced concrete beams	30 N/mm^2
Anchorage infill	30 N/mm^2
Steel reinforcement	Round mild steel BS4666:1969: 250N/mm^2
Tendon wires	Not specified. Assumed strand Characteristic strength (f_{pu}) = 1645 N/mm^2
Grouting	Not specified.
Ducting	Not specified.

Table 1: Material summary

1.8. Construction Sequence

1.8.1 In the absence of records outlining the deck construction sequence, the exact construction sequence of the structure is not known. It is assumed that the post-tensioned was cast in-situ and supported on falsework prior to stressing. Construction joints between the webs and the soffit slab suggest that the webs, top slab and cantilevers were poured monolithically. Following this, the suspended in-situ concrete deck would have been 'dropped in' and supported on the half joints.

1.9. Design

1.9.1 The structure was design in the early 1970's; therefore, it would have been designed before the advent of limit state design, and in accordance with BS 153 and CP115. This process designed for service with no partial factors applied to the loads and all the safety allowance was incorporated in reduced permitted stresses. The design for shear in this structure would have been in accordance with CP 115.

2. Post-Tensioning Risk Review & Half-joint Initial Review

2.1. Adequacy of Previous Risk Assessments

2.1.1 There are no previous PTSI Risk Assessments available for the structure in line with the current codes of practice.

2.2. As-built Information and Construction Records

2.2.1 8 No. as built drawings were made available at the time of writing of this report. Other structure records that were available for review included the historic inspection documents.

Drawing No.	Title
-	Adjustments to footway levels
ER6D/S71, S72, S73/12	Cable stressing sequence
ER6D/SK73/2A	West elevation of footbridge showing headroom
ER6D/S73/1	General Arrangement
ER6D/S73/8D	Layout and Details of Handrailing
ER6D/S73/7	Details of Ramps
ER6D/S71, S72, S73/7	Details of Reinforcement to Post-tensioned Decks
ER6D/SK73/1	Survey of Footbridge

Table 2: Available record drawings

2.3. Inspection and Testing History

2.3.1 General Inspection (GI), Principal Inspection (PI) and Special Inspection (SI) reports were made available and are listed below.

Inspection Date	Inspection Type	Inspection Reason
January 2004	Principal Inspection	
May 2006	Special Inspection	Special Inspection Report - half-joints
November 2020	General Inspection	

Table 3: Inspection and Testing History

2.4. Review of Previous Post-Tensioned Special Inspection Reports

Phase 1-3 PTSI Reports, Doran Consulting, 1997-1998

2.4.1 In 1997/1998 Doran Consulting carried out Phases 1-3 of PTSI of the footbridge in accordance with BD54/93, the findings of which were presented within three reports. The summary of reports is as follows:

(1) **Phase 2 Report (Preliminary Site Inspection), Feb 1998:** The report concluded that the post-tensioned deck was in generally good condition visually, although areas of damp and efflorescence were evident to the deck soffit. Rust staining was present on the southeast half-joint and transverse cracking was present on the soffit of the post-tensioned deck close to the northwest pier. The expansion joints at both half-joints and abutments were noted to be in poor condition with active water leakage present. The surfacing was in poor condition with

severe cracking at the abutments and ponding present at the mid-point region. A level survey showed considerable deflection throughout the centre span, which the report suggested was possibly evidence of long-term loss of prestress and exacerbated due to a deeper cantilever slab and further layer of surfacing on the suspended slab. The risk assessment taking into account structural form and defects concluded that the risk of sudden collapse was moderate with a rating of 41%. However, considering the presence of a holding down nib at both abutments; the bridge was considered a high-risk category for brittle collapse.

(2) **Phase 3 Report (Site Investigation), July 1998:** The Phase 3 PTSI included 3No. Duct and Tendon Exposures (DTE's) in each post-tensioned cantilever deck. In the southeast deck all DTE's showed significant voiding in the tendons, damp grout and waterlogged ducts. In the northwest deck, partial voiding was encountered at all DTE's with soft damp grout present. Despite the voiding and presence of water, the tendons were found to be in generally good condition, with only moderate corrosion evident in 5 DTE's. Materials testing on samples of the post-tensioned deck concrete and grout from ducts showed that chloride ion content of the concrete and grout was low. However, given the significant voiding and presence of water in the ducts the chloride ion contents may be sufficient to initiate corrosion of the tendon strands. Petrographic analysis of core sample from the deck concrete indicated that there was no evidence of alkali-silica reaction. The final risk assessment taking into account probable loss of residual prestress, the poor conditions in the ducts and the ongoing water ingress concluded that the post-tensioned elements of the footbridge was of medium risk of sudden failure. The risk was refined considering the presence of the holding down nib at the abutments and given a high-risk category for brittle collapse.

Phase 1-3 PTSI Report, Parsons Brinckerhoff, 2002-2004

2.4.2 In 2002-2004 Parsons Brinckerhoff completed Phases 1-3 of a PTSI on the footbridge, the results of which are contained within three reports. A summary of Parsons Brinckerhoff PTSI is as follows:

(1) **Phase 1-2 PTSI Review Report, 2002**; this report was a review of the adequacy of the previous PTSI undertaken by Doran's in 1997. The reports concluded that the Phase 1 and 2 reports completed by Doran's were accurate and contained adequate information. The report also noted the amount of DTE's undertaken in the previous PTSI was sufficient to demonstrate the significant issues with the post-tensioning. A number of recommendations was subsequently made by Parsons Brinkerhoff:

(a) Additional testing and sampling of selected areas of the structure and limited further DTE's and EAE's to clarify the extent of defects;

(b) Additional laboratory testing and sampling;

(c) Defect survey of the super-structure and sub-structure;

(d) In-situ stress measurements and a level survey of the top of deck;

(e) Sampling and testing of the concrete to determine the cause and extent of ettringite crystal formation identified during the previous PTSI;

(f) Special inspection of the reinforced concrete holding down nibs at the abutments;

(g) Special inspections of the deck waterproof system and transverse joints.

(2) **Phase 3 PTSI Report, 2004**; The report outlines the results of further testing as recommended from the 2002 review. In total 25 exposures were undertaken focussing on the end anchorages and tendon high point over the piers. The number of DTE'S and EAE's was not specified. The report states that the ducts, tendons and anchorages were in fair condition, although the degree of voiding and presence of moisture in the ducts was considered a concern.

Testing of the deck concrete and grout for chloride ions indicated that the chloride ion content in the grout was low but higher levels were found in the top of deck and deck soffit. The chloride ion levels in the deck concrete were found to be high enough to cause corrosion of the tendons. No evidence of sulphate levels in excess of 4.0% were found. Corrosion testing showed that there was a low risk of widespread corrosion due to ingress of water rich in de-icing salts. Breakouts of the reinforcement found the mild steel reinforcement to be in good condition. The report stated that it was necessary to prevent further water ingress in order to avoid sulphate induced deterioration of the concrete and to prevent the occurrence of alkali silica reaction.

2.5. Review of Previous Half-Joint Reports

Special Inspection Half Joints Report, Doran Consulting, 2006

2.5.1 In 2004, Director of Engineering Memorandum (DEM) 71/04 was introduced as an interim management strategy for half-joint structures. Doran Consulting were appointed in May 2006 to undertake a special investigation of the half-joints in accordance with DEM 71/04.

2.5.2 The report concluded that the slabs and webs of both the post-tensioned and the suspended reinforced concrete decks at the half-joints were in reasonable condition. There were no significant defects present other than minor hairline cracking to the rear of the half-joint nibs and some isolated areas of shutter debris. The chloride ion content of the deck soffit at the half-joint was low to moderate with no visual evidence of ongoing reinforcement corrosion. Half-cell potential testing of the deck soffit near the half-joints indicated that probability of corrosion was unlikely.

2.5.3 The top surface of the deck at the half-joints was in good condition although there was some minor edge cracking to the North side of the suspended deck. However, no evidence of reinforcement corrosion was present. Half-cell potential testing of the top of deck indicated that potential difference results were higher than that of the soffit. The majority of the half-cell results indicated that likelihood of corrosion was uncertain, and no results indicated corrosion was likely. The chloride ion content indicated that there was no significant chloride content and there was no evidence of corrosion in the top of the deck slab near the half-joints.

2.5.4 No intrusive testing was undertaken during the special inspection. However, there was no evidence of significant ongoing corrosion of the internal surfaces of the half-joint and intrusive testing was not recommended. The report recommended that the condition of the half-joints be monitored on an ongoing basis and that the joints should be resealed to eliminate the risk of further water ingress.

Half Joints Review and Feasibility Report, Mouchel, 2010

2.5.5 In 2010 Mouchel were appointed to undertake a review of the half-joints in accordance with DEM 71/04. The review was used to determine the requirements for further inspections/investigation and provide prioritisation schedules, costs, and programmes for any recommendations.

2.5.6 The review identified the main defects to the half-joints as water ingress and damp staining, presence of hairline cracking, poor quality mastic sealing and sagging deflection of the suspended deck.

2.5.7 The review recommended that the cause of the water ingress through the joint needed to be investigated and works undertaken to address the ingress in order to prevent further deterioration to the half-joints. Measurement of the cracking and installation of Demec studs to monitor crack movement during future inspections were recommended. A level survey of the top of deck to monitor sagging was also recommended. The report stated that the general condition of the half-joints need to be monitored on a continuous basis in future inspections.

Management Strategy Report, Mouchel, 2012

2.5.1 In 2012 Mouchel were appointed to develop an overall management strategy for all DfI structures with half-joints. The management strategy made the following conclusions and recommendations in relation to Moylinn footbridge.

2.5.2 The priority of the half-joints was given a component rating of 2, which were considered to be non-urgent in nature and not affecting the function or the structural stability of the half joint component. The report determined that assessment of the half-joints should only be undertaken ahead of programme if deterioration increases.

2.5.3 The report recommended the following additional testing works to the half joint region:

- Detailed hammer and visual survey of half joint and cover meter survey
- Half-cell survey
- Material testing (chloride samples, cement content, alkali/sulphate content, carbonation testing).

2.6. Review of other reports

Headroom Risk assessment, DfI Highways Structures Unit, 2021

2.6.1 In 2019 DfI Highways Structures Unit carried out a headroom survey in accordance with TD27/05. The structure was found to have inadequate clearance as per the requirements of TD27/05 Table 6.1 by 164mm at the Northeast face and 257 mm when vertical deflection due to full pedestrian loading is considered.

2.6.2 The risk assessment found that Moylinn bridge had a low priority risk classification, and the degree of risk was acceptable. Given the lack of history of strikes and the high cost of raising the structure, the report recommended no mitigation measures be implemented and a departure from standard for sub-standard headroom be submitted for technical approval.

Structure Options Report, DfI Highways Structures Unit, 2021

2.6.1 In November 2020 DfI Highway Structures Unit were appointed by DfI Southern Division Structures to undertake an options report for remedial works. The report considered that the previous defects to the post-tensioning were of immediate concern and recommended that the bridge be monitored on a monthly basis to gauge movement of the deck and crack formation. The report recommended that the structure should be replaced as a safe long-term solution and that in the event that the bridge is repaired but not replaced within 2 years; a risk review and risk assessment be completed in accordance with DMRB CS465.

2.7. Main Defect Summary

Water/soft grout and voids in post-tensioned tendons

2.7.1 The main defect noted to the post-tensioning system was the poor quality of grouting. In both the 1998 and 2004 PTSI Site Investigations, the majority of duct and tendon exposures noted voiding, presence of water and in some cases, soft wet grout. Despite the voiding and presence of water the tendons were found to be in generally good condition, with only moderate surface corrosion evident and not section loss.

2.7.2 It can be assumed that the issue of soft damp grout with voids is widespread throughout the structure and that the grout does not provide any protection to the tendon strands. Furthermore, following the guidance of DMRB CS 455, tendons with poorly grouted ducts shall be treated as unbonded. The bending and shear capacity of the deck at ULS may be reduced as a result of the tendons behaving as unbonded. Soft, voided grout will also not allow re-bonding of a tendon should it break or slip from the anchor point, resulting in an abrupt loss of pre-stress over the stressing length.

Deflected shape of deck

2.7.3 The suspended span of the of the footbridge exhibits a pronounced 'sag' which has been a cause for concern. The May 2006 Half Joint Special Inspection Report referenced an inspection completed in 1980 to investigate the sag in the centre span. A series of level surveys were carried out by Roads Service in the early 1980's confirmed that the whole of the centre span, including the post-tensioned cantilevers, was sagging by 175 mm below design level at midspan. In 1998, a level survey of the central span was completed as part of the PTSI Phase 2 and confirmed the sagging profile of the cantilever and in situ reinforced concrete suspended span and indicates that the sag at midspan has increased from a maximum of 175 mm in 1980/82 to 200 mm presently in 1998. In addition, the report stated that the gradients along the structure increase rapidly from the piers along the cantilevers to the scarf joints, suggesting there is a loss of residual prestress over the piers.

Figure 11: Looking across the deck from the abutment (2002 PBI)

Figure 12: Overall view of parapets with visible deflection noted - November 2021

2.7.4 In November 2021, Amey Consulting conducted a point cloud survey of the bridge deck to assess the deflected shape – See Appendix A. It was noted from the point cloud that the curvature of the soffit of the main spine beam (from pier to pier) generally follows a uniform arc, which suggests that the main spine is not sagging locally at mid-span. The scan shows that the profile of the parapet upstand/cantilevers is irregular, however, and sags at mid-span, causing water ponding on the top of the deck. It is possible that the top slab and cantilevers were cast with a sag, possibly due to poor workmanship or deflection of the formwork during placement. Previous report has stated that loss of prestress is the likely issue; however, it would be likely if the cantilever deck started to deflect vertically due to loss in prestress, the suspended span would rotate about the half joint. There is no clear signs of rotation about the half joint and the deflection appears to be concentrated to the parapet upstands. Furthermore, loss of prestress would likely lead to cracking of the prestressed elements. The guard rail and base plates do not exhibit signs of buckling, twisting, or cracking due to deck deflection which raises further uncertainty of the cause of the deflection.

2.7.5 There is much uncertainty surrounding the cause of the sagging. It is clear however, that it is a long-standing issue since at least 1980, therefore it is not of immediate concern.

Leakage through half joint

2.7.6 The majority of previous inspection reports noted active leakage with stalactites from the suspended span half joints. The 2006 Half Joint Special Inspection Report noted that active leakage at both half joints was recorded as far back as the 1998 visual inspection, therefore, it is a longstanding issue. The formation of stalactites suggests that de-icing salts has been used on the footbridge which increases the risk of chloride induced corrosion to the half joint. Leakage at the half joint location also increases the risk of chloride induced corrosion of the post-tensioned end anchorages which are located within the ends of the cantilever deck. There is also a risk of chloride rich water entering the poorly grouted post-tensioned ducts via the anchorages, increasing the risk of corrosion to the tendon strands.

2.7.7 A site visit undertaken by Amey Consulting in November 2021 confirmed that the leakage through the half-joint over the Southbound carriageway was active, with evidence of ponding on the carriageway below, thus confirming the leakage as significant.

Figure 13: Active leakage from half-joint at Southbound carriageway

Leakage at abutment/tie down nib

2.7.8 The site visit undertaken by Amey Consulting in November 2021 confirmed active leakage at both abutment movement joints. Stalactite formation was evident which suggest the presence of de-icing salts from the deck run-off. Prolonged leakage at the abutment joints which chloride laden deck run off increases the risk of chloride induced corrosion of the reinforced concrete half joint nib; which is critical in preventing rotation and subsequent collapse of the post-tensioned deck.

2.7.9 Leakage at the deck ends also increases the risk of chloride induced corrosion of the post-tensioned end anchorages which are located within the ends of the deck. There is also a risk of chloride rich water entering the poorly grouted post-tensioned ducts via the anchorages, increasing the risk of corrosion to the tendon strands.

Figure 14: Active leakage at abutment joints – November 2021

Minor cracking to deck soffit

2.7.10 Fine horizontal cracks have been noted to the soffit of the box beam at the corners of both half joint lower nibs – this is evident in Figure 15. Evidence of effloresce has also been noted along the cracks. Furthermore, vertical cracking to the to the sides of the deck box section has been noted adjacent to the half joint. The cracks appear to be long standing as they were noted in a 1998 inspection and have not appeared to have increased in extent or severity at the time of writing this report.

2.7.11 It is possible that the fine cracking to the corners of the half joint are a result of high localised tensile stresses at the re-entrant corner.

Figure 15: Fine cracks to half joint

2.8. Maintenance History

2.8.1 Table 4 summarizes the know major maintenance and repair records undertaken to date on Moylinn Footbridge.

Bridge Element	Repair/Maintenance Details	Date
Deck soffit	No records of concrete repairs to deck soffit.	-
Deck (Topside - surfacing/waterproofing)	No records of re-waterproofing of deck. Resurfacing works had taken place in the past to achieve camber.	Circa 1980.
Piers	No records of concrete repairs to piers.	-
Abutments	No records of concrete repairs to abutments.	-
Movement Joints	No records of replacement, maintenance, or repair of movement joints.	-
Bearings	No records of replacement, maintenance, or repair of bearings.	-

Table 4: Summary of repair and maintenance

2.9. Load Assessments

Post-Tensioned Deck

2.9.1 There are no records of a previous structural assessment determining the load capacity of the post-tensioned cantilever deck.

Half-joint assessment – Doran Consulting 2008

2.9.2 A structural assessment of the half-joints in accordance with BA39/93 was undertaken in 2008 by Doran Consulting. The footway live loading for the assessment was applied in accordance with BD 21/01 and BD 37/01.

2.9.3 The half-joints were assessed at the Serviceability Limit State (SLS) to determine the calculated crack widths in accordance with BA 39/93 and compared to the allowable crack widths in BS 5400 Part 4, Table 1. The results of the assessment are summarized in table 5 below.

Half joint section	BA 39/93; calculated crack width	BS 5400 Part 4, Table 1; Allowable crack width
Reinforced concrete suspended deck (Upper section)	0.31 mm	0.25 mm
Post-tensioned cantilever deck (Lower section)	0.44 mm	0.15 mm

Table 5: Half-Joint assessment results 2008

2.9.4 The upper section of the half-joint exceeded the allowable crack width for a "severe" exposure class. The lower section of the half-joint exceeded the allowable crack width for a "very severe" exposure class. The assessment recommended that further internal investigations of the half-joint should be undertaken.

2.9.5 There are no records of a ULS lower bound analysis (strut-and-tie analysis) checks or higher bound mechanism analyses completed on the half-joint, thus the safe load rating is not known.

Pier impact assessment - Parsons Brinckerhoff 2004

2.9.6 A Pier impact assessment in accordance with BD 48/93 and BD 44/95 was completed by Parsons Brinkerhoff in 2004. The assessment found that the piers were inadequate for impact loading with the applied moment 257% of the pier capacity in bending and applied shear force 646% of the pier capacity in shear. The assessment recommended interim installation of a VRS with a view to strengthening the pier columns in the longer term.

2.9.7 There are currently no high containment VRS systems installed adjacent to the piers.

2.10. Actions taken in response to recommendations of previous reports

2.10.1 The previous PTSI Report undertaken in 2002 made the following recommendations:

Recommendation	Year	Actioned
Additional testing and sampling and further duct, tendon, and anchorage exposures.	2002	An additional 25 duct, tendon and anchorage exposures were undertaken in 2004 as part of Phase 3 of a PTSI.
Undertake a defect survey of the super-structure and sub-structure	2002	Structure was inspected during 2004 PBI.
In-situ stress measurements to identify the sections of the post tensioning system in need of remedial measures	2002	No record of in-situ stress measurement. No record of a level survey of the deck.
Special inspection of the reinforced nibs at the abutments	2002	No record of a Special inspection of the nibs at the abutments.

Recommendation	Year	Actioned
Special inspections of the deck waterproof system and transverse deck joints	2002	Special Inspection of the half joints including exposure of the deck top side completed in May 2006.

Table 6: Previous PTSI recommendations

2.10.2 The previous half-joint reports undertaken in 2006 and 2010 made the following recommendations:

Recommendation	Year	Actioned
The deck joints at the end of each cantilever should all be resealed to eliminate the risk of further water penetration into the half-joint regions	2006	No record of the joints being resealed.
Investigation of the cause of leakage through the half-joints	2010	No record of investigation of the leakage through the half-joint.
Installation of Demec studs to monitor crack widths in the deck soffit	2010	No record of installation of Demec studs to monitor cracking.
Undertake a level survey should be conducted to monitor the sagging of the suspended span	2010	No record of a level survey of the deck.

Table 7: Previous half-joint report recommendations

2.11. Critical Sections

2.11.1 Critical sections are areas where yield points may form a collapse mechanism and an area that is at high risk from water ingress causing corrosion to the post-tensioned tendons. Typical critical sections are as follows:

- Regions where voids may form preferentially in tendon ducts;
- End anchorage regions;
- Regions over intermediate supports and other duct high points;

2.12. Identification of Critical Points

2.12.1 A critical point is any location where a post-tensioning system intersects a critical section. Table 8 summarises the critical points for each identified critical section as outlined in Figure 16 below. The number of critical points requiring investigation has also been calculated as per Equations A.1 and A.2 of CS 465. The calculated critical points are used to determine the adequacy of previous investigations.

Critical Section	Description	Ref	Number of Critical Points
North Abutment	Longitudinal end anchorage region	CS-L1	8 (2 webs x 4 tendons)
North Pier	Longitudinal post-tensioning intermediate support region (Tendon high point)	CS-L2	8 (2 webs x 4 tendons)
North Cantilever	Longitudinal end anchorage region	CS-L3	8 (2 webs x 4 tendons)
South Cantilever	Longitudinal end anchorage region	CS-L4	8 (2 webs x 4 tendons)
South Pier	Longitudinal post-tensioning intermediate support region (Tendon high point)	CS-L5	8 (2 webs x 4 tendons)
South Abutment	Longitudinal end anchorage region	CS-L6	8 (2 webs x 4 tendons)
		Total number of critical points C_{PL}	**48**
Recommended number of critical points requiring investigation C_L		$C_L = 3.57 . C_{PL}^{0.36}$	**15**

Table 8: Identification of critical points

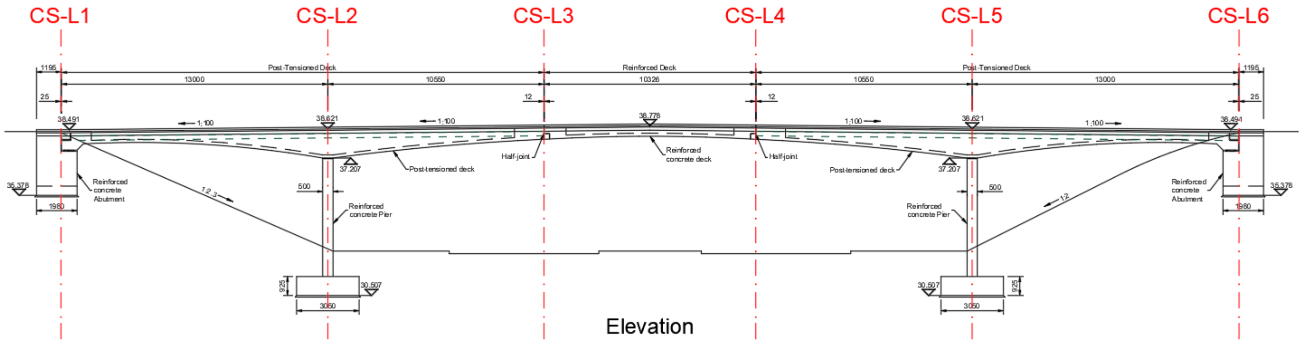


Figure 16: Critical sections - Longitudinal post-tensioning

2.12.2 The recommended number of critical points requiring investigation has been calculated as **15** as per the guidance of CS 465.

2.12.3 It is noted that the previous PTSI in 2004 completed 25 No. DTE's & EAE's, thus conforming that the number of previous investigations conform to the current codes of practice.

2.13. Adequacy of Previous PTSI

2.13.1 There are no PTSI Risk Assessments available for Moylinn East Footbridge in line with CS 465 or formally BD54/15, as such a Risk Assessment will be completed as part of this report for the post-tensioned elements.

2.13.2 Copies of the previous PTSI Reports completed in 2004 were not available for review at the time of writing this report. This Risk Review has based the outcomes of the previous PTSI on a synopsis included within a 2008 Half Joint SI Report. It was reported that the 2004 PTSI Phase 3 investigations completed approx. 25 No. DTEs and EAEs to the post-tensioned structure, exceeding the requirements as stated in Table 8 thus considered adequate at the time of the investigation.

2.13.3 Given the time elapsed from the previous PTSI (18 years) and the fact that significant grouting defects were noted; it is likely that further investigation will be required to determine the current condition of the tendons and to inform a Risk Management Plan. This will be confirmed following a Risk Assessment Process completed in Section 3 of this report.

2.14. Summary of Risk Review

2.14.1 In absence of any PTSI Risk Assessments available in accordance with either BD54/15 or CS 465, the Risk Review recommends that a PTSI Risk Assessment and Risk Management Plan be completed as per CS 465. A Risk Assessment will assess the hazards relating to the structure's post-tensioning system based upon the critical sections highlighted within this report. A bridge specific Risk Management Plan will then be developed, which will manage and mitigate the specific risks identified to the post-tensioning.

2.14.2 A PTSI Site Inspection is not considered necessary in this instance as previous inspections have verified the required objectives of a Site Inspection, namely:

- the type of bridge construction is per record drawings and record any variations;
- the form of articulation, geometry, type, and locations of construction joints;
- Identification of post-tensioned elements showing signs of distress and/or deterioration;
- Identification any access constraints at critical sections.

2.14.3 In accordance with CS 466 Section 3, the Risk Review has determined that the information available on the half-joints is current and valid and sufficient to enable a risk assessment to be carried out. As such, a risk assessment is recommended to be completed on the half-joint in accordance with CS 466 Section 4.

3. Risk Assessment of Post-tensioned desk

3.1. Introduction

3.1.1 This section consists of a qualitative Risk Assessment of Moylinn East Footbridge in accordance with CS 465. No previous Risk Assessment has been completed for this structure in accordance with the current codes of practice.

3.1.2 The purpose of the Risk Assessment is to ensure that asset owners and managers understand the risks associated with their post-tensioned bridge stock, enabling selection and implementation of appropriate risk management over the life of the structure.

3.1.3 Hazards identified from the Risk Review are discussed and are grouped under; Age, Structural Form, Vulnerable Details, Condition (External and Internal) Hazards, History Hazards and Assessment Hazards. The likelihood and consequences of the risk events occurring, and proposed risk management measures are also discussed.

3.2. Age

3.2.1 The bridge was built in 1971 (51 years old) therefore it is in the second highest risk band as per CS 465 Table B.1. Knowledge, experience, design standards and specifications for methods and materials have improved significantly since Moylinn Footbridge was constructed.

3.3. Structural Form Hazards

3.3.1 The deck is a monolithic box girder, composite and continuous over intermediate supports with longitudinal post-tensioning. This structural form is considered to have a low risk of brittle failure mode.

3.3.2 CS 465 Table B.2 considers cantilevered spans, with anchor spans tied down for dead loads and live loads as a very high risk of brittle failure mode. Moylinn footbridge is tied down at the abutment with a reinforced concrete half-joint 'nib' rather than stressed bars forming the tie down; therefore, it is not considered to fall into the stated very-high category as covered in CS 465. Nonetheless, the reinforced concrete holding down nib is a hidden critical element, and its failure could lead to a brittle collapse of the bridge.

3.3.3 It is therefore considered appropriate to assign the post-tensioned bridge an overall 'high' risk of brittle failure mode based on its holding down feature.

3.4. Vulnerable details and material hazards

Vulnerable details and material hazards							
Hazard		**Critical Section Ref:**	**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
1	**Half Joints**	CS-L1 CS-L3 CS-L4 CS-L6	Risk of deck run off penetrating failed surfacing, waterproofing and joint sealant at half joint locations where the post-tensioned anchorages are located. This increases the risk of chloride ion penetration to the post-tensioned end anchorages and potentially leading to water ingress to the tendon ducts causing corrosion of the post-tensioned tendon strands and anchorages.	**High** Post-tensioned end anchorages of the cantilever deck correspond to half joint locations. It is clear from recent and historic inspections, that all half joints (both at the abutments and suspended span locations) are actively leaking with signs of stalactite formation. The structure would not be subjected to routine salting. However, it possible that there may have been some level of chloride ingress to the ends of the deck over time. Given the concealed nature of the joints, inspection of the deck ends is not possible therefore a high likelihood is considered prudent.	**High** Water/chloride ingress causing corrosion to tendon strands and anchorages. Anchorage failure could result in excessive prestress along the length of the deck – especially as previous PTSI's have determined that the grouting is poor thus unlikely that broken tendon strands could re-anchor.	**High**	**Short Term** Undertake a PTSI Site Investigation to determine: - Extract concrete dust samples from top and bottom of deck corresponding to anchor locations to determine depth and concentration of chlorides. - Examine condition of tendon ducts, strands and grouting towards the ends of the deck, close to end anchorage regions. - Undertake pressure testing voids to determine extent of potential grouting voids at end anchorage regions. **Longer Term** Full deck refurbishment including full deck waterproofing, re-surfacing, and new deck joints.

Vulnerable details and material hazards							
Hazard		Critical Section Ref:	Risk Event	Likelihood	Consequence	Hazard Risk Level	Risk Management Measures
2	**Tendons located close to the upper surface of the deck.**	CS-L2 CS-L5	The deck tendon profile is arranged so the tendons run close to the footway surface over interior support locations (Max Hogging Moments). Risk of water penetrating failed deck waterproofing and subsequently the concrete deck – leading to potential chloride induced corrosion of the tendon strands.	**Medium** The previous PTSIs in 1998 and 2004 noted significant voiding within the longitudinal tendon ducts containing soft wet grout. This presents a significant durability issue as good quality grout is considered the last line of protection to the tendon strands. The tendons were found to be in good condition however with no significant signs of corrosion noted.	**High** Water/chloride ingress through cracks in the concrete deck causing corrosion to tendon ducts and eventually tendon strands. Localised tendon failure could lead to prestress loss along the length of the deck – especially as previous PTSI's have determined that the grouting is poor thus unlikely that broken tendon strands could re-anchor.	**Medium**	**Short Term** Undertake a PTSI Site Investigation: - Trial hole removing surfacing and waterproofing from top of deck at pier locations to determine presence of flexural cracks which may suggest loss of pre-stress and rotation of the deck over the piers. - Extract concrete dust samples from top deck top slab at pier locations to determine the depth and concentration of chlorides. - Re-examine condition of tendon ducts, strands and grouting at previous exposure locations where voids were noted. - Undertake pressure testing voids to determine extent of potential grouting voids at end anchorage regions. **Longer Term** Full deck refurbishment including full deck waterproofing, re-surfacing, and new deck joints.

Vulnerable details and material hazards							
Hazard		**Critical Section Ref:**	**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
3	**Tendon ducts with hogging and sagging profiles with a vulnerability to void formation and water ponding**	CS-L2 CS-L5	Risk of voids forming at tendon high points over intermediate supports and at low points at mid-span. Risk of water containing salts ponding within deck voids corresponding to tendon low points.	**Medium** Refer to Hazard No. 2	**High** Refer to Hazard No. 2	**Medium**	**Short Term** Undertake a PTSI Site Investigation to determine: - Extract concrete dust samples from top deck top slab at pier locations to determine the depth and concentration of chlorides. - Re-examine condition of tendon ducts, strands and grouting at previous exposure locations where voids were noted. - Undertake pressure testing voids to determine extent of potential grouting voids at end anchorage regions. **Longer Term** Full deck refurbishment including full deck waterproofing, re-surfacing, and new deck joints.
4	**Longitudinal anchorages located close to upper surface of the deck and at deck movement joints**	CS-L1 CS-L3 CS-L4 CS-L6	Refer to Hazard No. 1	**High** Refer to Hazard No. 1	**High** Refer to Hazard No. 1	**High**	Refer to Hazard No. 1

Vulnerable details and material hazards							
Hazard		**Critical Section Ref:**	**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
5	**Absent drainage system.**	CS-L1 CS-L2 CS-L3 CS-L4 CS-L5 CS-L6	Risk of water ponding on top of deck and penetrating concrete deck through failed/inadequate deck waterproofing resulting in the corrosion of deck reinforcement and the post-tensioning system.	**Medium** There is no drainage system present within the deck. The structure was designed for surface water runoff to drain from a high point in the centre of the deck towards the abutments. However, the bridge was constructed with a single crossfall increasing the drainage path for surface water on the deck. There are no records of the original bridge deck waterproofing having been replaced. It is clear that due to the sag in the bridge at surface water does pond at mid-span. As the ponding is not close to the PT elements of the structure, the likelihood has been downgraded to a medium.	**High** Chloride induced corrosion of the post-tensioning at intermediate support regions. Severe corrosion of the post-tensioning could result in structural failure, particularly as the previous PTSI noted significant voiding in the tendon ducts.	**Medium**	Refer to Hazard No. 2
6	**Old deck waterproofing system.**	CS-L1 CS-L2 CS-L3 CS-L4 CS-L5 CS-L6	Water penetrating concrete deck through failed/inadequate deck waterproofing resulting in the corrosion of deck reinforcement and the post-tensioning system.	**Medium** Refer to Hazard 5.	**High** Refer to Hazard 5.	**Medium**	Refer to Hazard No. 2

Vulnerable details and material hazards							
Hazard		**Critical Section Ref:**	**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
7	**Grouting problems.**	CS-L1 CS-L2 CS-L3 CS-L4 CS-L5 CS-L6	Grouting voids in tendons due to poor workmanship. Voids in grouting significantly increase the risk of tendon strand corrosion when water and chlorides penetrate and corrode the tendon ducting.	**High** The 1998 and 2004 PTSIs found that the majority of tendon ducts exposed were inadequately grouted. The report also noted that in some cases the grout was soft and damp. It is noted that the tendon strands did not exhibit any signs of severe corrosion at the voided locations, nonetheless, the presence of voids increases the likelihood of corrosion to the tendon strands from external sources of corrosion.	**High** Un-grouted or poorly grouted tendons are susceptible to corrosion following subsequent ingress of water and chlorides from de-icing salts. Tendon corrosion could lead to loss of prestress and eventual local or global structural failure. Unbonded tendons result in a lower bending and shear capacity of the deck at the ultimate limit state.	**High**	**Short Term** Undertake a PTSI Site Investigation to: - Re-examine condition of tendon ducts, strands and grouting at previous exposure locations where voids were noted. - Undertake pressure testing voids to determine extent of potential grouting voids at end anchorage regions. **Longer Term** Full deck refurbishment including full deck waterproofing, re-surfacing, and new deck joints.

3.5. Condition Hazards

Condition hazards						
Hazard		**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
8	**Cracking in post-tensioned concrete elements.**	Risk of further cracking/spalling of the deck leading to deterioration of the underlying post-tensioned strands due to freeze/thaw action from water penetrating the cracks.	**Medium** Fine cracking was noted to the rear of the half-joint nibs during the 2006 SI. The dead-end anchorages of the longitudinal tendons are located either side of the nibs. The extent of cracking of the PT deck at hogging regions over the piers is not known. The structure would not be subjected to routine salting. However, it possible that there may have been some level of chloride ingress to the ends of the deck over time.	**High** Water containing de-icing salts could penetrate through cracks leading to chloride induced corrosion of the tendon ducts and post-tensioned strands. This could eventually lead to loss of prestress and a reduction in capacity of the deck due to severe spalling/delamination.	**Medium**	Refer to Hazard No. 2

Condition hazards						
Hazard		**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
9	**Cracking in footway surfacing.**	Water penetrating concrete deck through failed/inadequate deck waterproofing resulting in the corrosion of deck reinforcement and the post-tensioning system.	**Medium** A November 2021 Site Inspection by Amey concluded that the footway surfacing was in poor condition with cracking note There are no records of the original bridge deck waterproofing having been replaced.	**High** Water could penetrate through cracks in surfacing and failed waterproofing. Water containing de-icing salts could penetrate through cracks in concrete deck leading to chloride induced corrosion of the tendon ducts and strands. This could eventually lead to loss of prestress and failure of the PT deck.	**Medium**	Refer to Hazard No. 2
10	**Water ponding on deck surface.**	Risk of water ponding on top of deck and penetrating concrete deck through failed/inadequate deck waterproofing resulting in chloride induced corrosion of deck reinforcement and the post-tensioning system.	**Medium** Refer to Hazard 9.	**Medium** Ponding at mid-span downgraded to Medium as it is not adjacent to PT elements.	**Low**	Refer to Hazard No. 2
11	**Damaged or missing deck joint seals.** **Water leaks and staining at movement joints.**	Risk of water penetrating through joints perpendicular to longitudinal post-tensioned tendons at half-joints and abutments where end anchorages are located. Water and de-icing salts can lead to chloride induced corrosion of the tendons.	**High** Previous inspections have noted that joint seals have perished and are not adequate. A November 2021 Site Inspection noted that both the suspended span half joints and abutment half joints were actively leaking.	**High** Water/chloride ingress causing corrosion to tendon strands and anchorages. Anchorage failure could result in excessive prestress along the length of the deck – especially as previous PTSI’s have determined that the grouting is poor thus unlikely that broken tendon strands could re-anchor.	**High**	Refer to Hazard No. 1

Condition hazards						
Hazard		**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
12	**Excessive deflection**	Excessive deflection of the deck leading to overstress and potential structural collapse. Excessive deflection reduces the available headroom of the structure and increases the risk of accidental collision of high loads.	**Medium** A notable 'sag' has been present in the structure as far back as 1980, possibly occurring during construction. It is not known if the sag is a result of prestress loss or if it occurred during construction.	**High** Excessive deflection of the deck could result in overstress and cracking of the deck at pier locations, increasing the risk of a global collapse. Cracking of the PT deck at tendon high points increases the risk of chloride induced corrosion of the post-tensioned tendons.	**Medium**	Undertake trial holes to remove surfacing and waterproofing from top of deck at pier locations to determine presence of flexural cracks which may suggest loss of pre-stress and rotation of the deck over the piers. It is also recommended to complete a structural assessment of the post-tensioned deck and suspended span to calculate the deflection of the structure after long term prestress losses.
13	**Water present in ducts**	Water in poorly grouted ducts can lead to corrosion of the tendon strands – resulting in loss of prestress and structural failure of the deck.	**High** The 1998 and 2004 PTSIs found that the majority of tendon ducts exposed were inadequately grouted. The report also noted that in some cases the grout was soft and damp. It is noted that the tendon strands did not exhibit any signs of severe corrosion at the voided locations, nonetheless, the presence of voids increases the likelihood of corrosion to the tendon strands from external sources of corrosion.	**High** Water in poorly grouted ducts can lead to corrosion of the tendon strands – resulting in loss of prestress and structural failure of the deck.	**High**	**Short Term** Undertake a PTSI Site Investigation to: - Re-examine condition of tendon ducts, strands and grouting at previous exposure locations where voids and moisture were noted. **Longer Term** Full deck refurbishment including full deck waterproofing, re-surfacing, and new deck joints.
14	**Voided/Un-grouted ducts.**	Grouting voids in tendons due to poor workmanship. Voids in grouting significantly increase the risk of tendon strand corrosion when water and chlorides penetrate and corrode the tendon ducting.	**High** Refer to Hazard 13	**High** Un-grouted or poorly grouted tendons are susceptible to corrosion following subsequent ingress of water and chlorides from de-icing salts. Tendon corrosion could lead to loss of prestress and eventual local or global structural failure. Unbonded tendons result in a lower bending and shear capacity of the deck at the ultimate limit state.	**High**	Refer to Hazard 13.

Condition hazards						
Hazard		**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
15	**Soft, moist grout**	Soft and voided grout significantly increases the risk of tendon strand corrosion when water and chlorides penetrate and corrode the tendon ducting.	**High** The 1998 and 2004 PTSIs found that the majority of tendon ducts exposed were inadequately grouted. The report also noted that in some cases the grout was soft and damp. It is noted that the tendon strands did not exhibit any signs of severe corrosion at the voided locations, nonetheless, the presence of voids increases the likelihood of corrosion to the tendon strands from external sources of corrosion.	**High** Refer to Hazard 14	**High**	Refer to Hazard 7.

3.6. History Hazards

History hazards						
Hazard		**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
16	**Maintenance actions identified in previous inspection reports have not been completed.**	Risk of further deterioration of post-tensioned tendons due to water penetrating through failed deck waterproofing and expansion joints	**Medium** The 2004 PBI recommended that's the movement joints sealant be replaced. There are no records of the movement joint sealant being replaced.	**High** Refer to Hazards No. 1 and 2.	**Medium**	Refer to Hazards 1 & 2 for short term risk management measures.
17	**Use of de-icing salts**	De-icing salts contaminate the surface water with chlorides and sulphates which, when ponding or flowing over the concrete elements, penetrate the cover concrete, causing deterioration of the concrete, chloride induced corrosion of reinforcement and potentially the post-tensioning system.	**Medium** Previous inspections have confirmed active water leakage and stalactite formation at the half joints. This leaking water may not be chloride rich if the path above does not get regularly salted during winter maintenance operations. However, the exact extent of salting over the bridge is unknown and localised salting could be taking place, so it is felt that a medium likelihood is appropriate.	**High** Ponding water containing salts could penetrate the deck top slab, and end anchorages at joints and corrode the post-tensioned tendons and anchorages. Severe corrosion could lead to loss of prestress and eventual structural failure of the deck.	**Medium**	Refer to Hazards 1 & 2 for short- and long-term risk management measures.

3.7. Assessment Hazards

Assessment hazards						
Hazard		**Risk Event**	**Likelihood**	**Consequence**	**Hazard Risk Level**	**Risk Management Measures**
18	**Structure has not been assessed for current condition, assessment standards or operational loading.**	Risk of PT structure having inadequate load capacity for current operational loading in its current condition (unbonded tendons)	**Medium** There are no available load assessments for the post-tensioned deck in accordance with CS 454 - Assessment of highway bridges and structures. The previous PTSI noted significant voids to the post-tensioning. CS 455 states that for any PT structure with poorly grouted ducts; it shall be treated as unbonded. It is noted that the bending and shear capacity of the deck at ULS will likely decrease under this assumption.	**High** PT deck with unbonded tendons may be sub-standard for current loading, leading to overstress.	**Medium**	**Short Term** It is advised to complete a structural assessment of the post-tensioned deck in accordance with current DMRB standards. The structural assessment shall also assume that the post-tensioning is unbonded due to the significant voids noted previously. A sensitivity analysis should be undertaken to confirm the extent of corrosion required to change the loading rating of the structure.
19	**Structure has not been assessed for sensitivity to loss of prestress**	Risk of structure having inadequate load capacity for current operational loading. Undetected tendon corrosion or broken wires may attribute to a significant reduction of capacity.	**Medium** The previous PTSI did not record evidence of significant corrosion of the tendons or broken strands/wires. However there remains a possibility of some corrosion to the un-grouted tendons occurring in the 18 years since the previous PTSI.	**High** The bridge deck may be found to be sub-standard for live loading when considering loss of prestress.	**Medium**	Refer to Hazard No. 17

4. Prioritisation of post-tensioned structure

4.1. Risk rating

4.1.1 The purpose of a risk rating is to allow bridge owners and operators to prioritise their post-tensioned bridge stock.

4.1.2 The following Risk Rating has been calculated using Equation B.1 of CS 465.

$$R\% = \frac{100\left[\left((4F_Y + F_F + F_V + F_C)F_Q\right) - 6\right]}{254}$$

Year of Construction Factor (F_Y) – The bridge was built in 1971, therefore in the second highest age risk category: **F_Y = 4**

Bridge Form Factor (F_F) – As per Section 3.3 of this report, a high risk of brittle failure is considered appropriate for the bridge due to the tie down detail at the abutments. **F_F = 10**

Vulnerable Details and Materials Factor (F_V) - The following vulnerable details have been identified with reference to Table 4.7 CS 465.

1. Construction joints intersecting post-tensioned ducts;
2. Half-joints;
3. Tendons located close to the upper surface of the deck;
4. Tendon ducts with hogging and sagging profiles;
5. Anchorages concealed within joints or on upper surfaces of decks;
6. Absent or malfunctioning drainage system
7. Old deck waterproofing system;
8. Grouting problems;

F_V = 8

Condition Factor (F_C) - The following condition hazards have been identified with reference to Table 4.8 CS 465.

1. Cracking in post-tensioned concrete elements;
2. Cracking in footway surfacing;
3. Water ponding on deck surface;
4. Damaged or missing deck joint seals & water leaks and staining at movement joints;
5. Excessive deflection;
6. Water present in ducts;

7. Voided and un-grouted ducts;
8. Soft grout, moist grout

F_C = 8

Consequence Factor (F_Q) – Data from the Northern Ireland Traffic Count Data website determined that the Annual Average Daily Traffic (AADT) flow on the A3 Moira Road was 19,460 (One-way AADT was 9,730) With reference to Table B.3 from CS465 this gives a consequence factor **F_Q = 3**

R% = 100[((4 x 4 + 10 + 8 + 8) x 3) – 6]/254 = 47%

4.2. Hazard risk level score

4.2.1 A Hazard Risk Level has also been calculated based on the hazard risk levels identified in the risk assessment and as per CS465 Appendix B3.

Hazard Ref	Hazard Risk Level	Hazard Score (H_s)
1	High	5
2	Medium	4
3	Medium	4
4	High	5
5	Medium	4
6	Medium	4
7	High	5
8	Medium	4
9	Medium	4
10	Low	3
11	High	5
12	Medium	4
13	High	5
14	High	5
15	High	5
16	Medium	4
17	Medium	4
18	Medium	4
19	Medium	4
	SUM	82
	MEAN HAZARD RISK SCORE	**4.3**

4.2.2 The mean hazard risk level calculated is **4.3**, placing the post-tensioned deck of Moylinn East Footbridge in a Medium Risk Level.

5. Risk assessment for management of Suspended Span Half Joints

5.1. Introduction and scope

5.1.1 This section of the report will assign both a primary and secondary risk rating to the suspended span half joints. Where secondary risks are used to increase the primary risk rating, this is referred to as the 'refined primary risk rating.' The risk assessment process outlined in this section follows the guidance as set out in DMRB CS 466 Section 4 and Appendix C. The primary risk rating is calculated as per Table C.1 of CS 466 using a combination of the half joint condition risk and structure risk.

Table C.1 Matrix for determining primary risk rating of a half-joint structure

		Structural risk R_D			
		R_D **Very high**	R_D **High**	R_D **Medium**	R_D **Low**
Condition risk R_C	R_C **Very high**	Very high (1)	Very high (1)	High (1)	High (1)
	R_C **High**	Very high (1)	High (1)	High (1)	Medium (1)
	R_C **Medium**	High (1)	High (1)	Medium (1)	Medium (1)
	R_C **Low**	High (1)	Medium (1)	Medium (1)	Low (1)

Note (1) = The primary risk rating determined from the combination of R_C and R_D .

Figure 17: Primary Risk Rating Matrix – CS 466 Appendix C (Table C1)

5.2. Condition Risk

5.2.1 The condition risk R_C is determined using the method outlined in CS 466 Appendix C.

Half Joint Defect Identification		
Half Joint Defect	**Description**	**Photo**
Cracking	Fine cracking to lower nib soffit, re-entrant corner. Width of crack has not been verified but assumed to be <0.3mm based on description in pervious reports.	
Active water leakage	Longstanding active water leakage and stalactite formation from half joints.	

Defect Decision Diagram

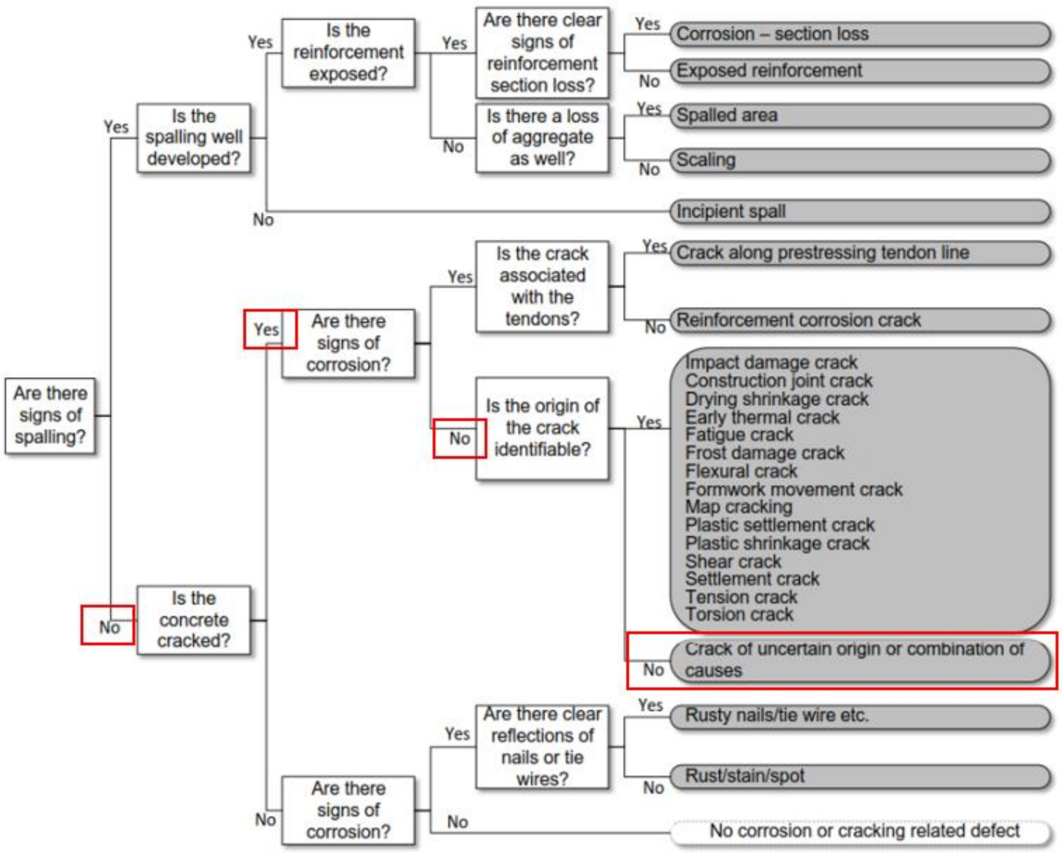


Figure 18: Defect Decision Diagram - CS 466 Figure C.1

5.2.2 From the Defects decision diagram Figure 18, the origin of the cracking at the half-joints is of uncertain origin.

Defect Zones

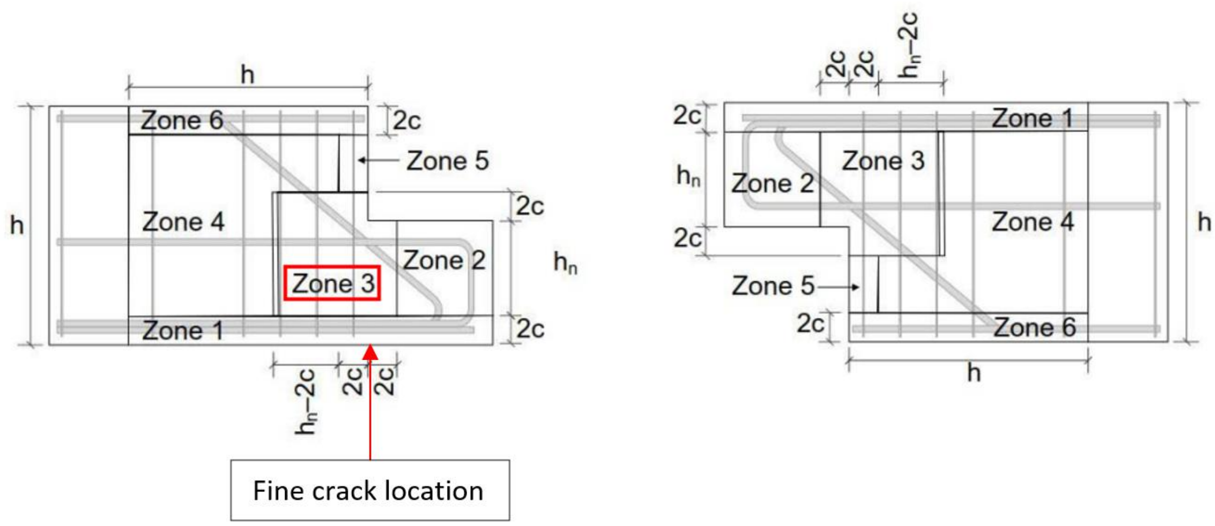


Figure 19: Defect Zone - CS 466 Figure C.2

5.2.3 The cracking to the suspended span half joint falls within Zone 3 as per CS 466 Figure 6.2.

Damage Rating

Damage	Very slight	Slight	Moderate	Severe	Very severe
Cracks in prestressed concrete	width <0.05 mm	width 0.05-0 .1mm	width 0.1-0.3mm	width 0.3-1. 0mm	width 1-3mm with some spalling
Cracks in reinforced concrete	width <0.1m m	width 0.1-0. 3mm	width 0.3-1.0mm	width 1-3m m with some spalling	width >5mm with widespread spalling
Effects of reinforce-ment corrosion	light rust stains	heavy rust stains	heavy rust stains and cracking along line of bar	heavy rust stains and spalling along line of bars	heavy rust stains and spalling along line of bars, in more than one location or for a number of bars.
Joint leakage/ water staining	clean/clear water, small patches of dampness	clean water seepage to large areas of the joint.	water seepage with discolouration at edge beam/footpath locations	30-70% of joint with discoloura-tion gritted road/ coastal environment	affecting >70% of joint with discolouration with signs of historic presence. gritted road/coastal environment
Spalling/ delamina-tion (1)	cover <50x5 0mm	100x100mm	area up to 150x 15 0mm across	area larger than 150 x 1 50mm	multiple areas >250 x 250mm; single areas more than 1000 x 1000m m
Note (1) - The size of a concrete spall should be compared to relative to the size of the half joint. The criteria is for guidance purposes and is not prescriptive.					

Figure 20: Damage Rating - CS 466 Table 6.2

5.2.4 The cracking to the half joints have been previously recorded as 'fine' therefore assumed to be less than 0.3mm in width, thus classified as **Moderate.**

5.2.5 The joint leakage at the half joint is assumed to be **Severe** as it covers 30% – 70% of the half joint and it is assumed that the footbridge is gritted/salted given the signs of stalactite formation.

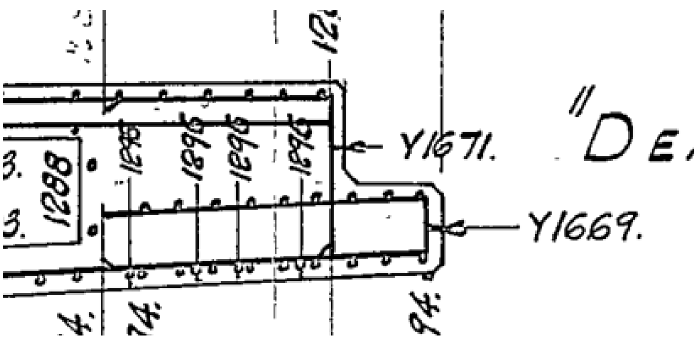
Condition Risk Rating

5.2.6 Following CS 466 Table C.3, the condition risk of the suspended span half joints is **Very High** as a single severe condition defect has been recorded.

5.3. Structural risk

Detailing Risk RD1

5.3.1 Detailing risk from CS 455 Table C.5 is **Medium** as detailing allows a single clear load path through the half-joint with adequate anchorage – no diagonal bar. Refer to Figure 21 below.



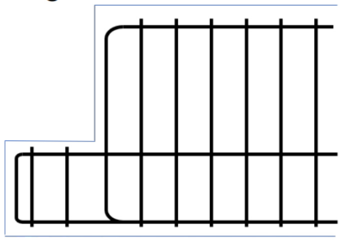
Medium	**Well-anchored but no diagonal bar** 

Figure 21: Half-joint reinforcement detailing

Structural Risk R_{D2}

5.3.2 The structure has not been assessed at the Ultimate Limit State using a lower bound or upper bound method of analysis. The 2008 Doran Consulting assessment assessed the half joint at SLS to calculate crack widths under full pedestrian live loading. The assessment found that the calculated crack widths of both the upper and lower sections of the half joint exceeded the allowable cracks widths as stated in BS 5400.

5.3.3 Given the lack of assessment information it is considered prudent to assign the Structural Risk rating as Medium. Note Table C.6 of CS 466 considered elements with reduced capacities on footpaths as Medium Risk.

5.4. Primary risk rating

5.4.1 The Half-joint Condition risk R_C is considered **Very High,** and the Structural risk R_D is considered **Medium**. Table C.1 gives the Primary risk rating as **High.**

		Structural Risk R_D			
		Very High	High	**Medium**	Low
Condition Risk RC	**Very High**	Very High	Very High	**High**	High
	High	Very High	High	High	Medium
	Medium	High	High	Medium	Medium
	Low	High	Medium	Medium	Low

Table 9: Primary risk rating matrix

5.5. Secondary risks

5.5.1 The Consequential risk R_Q has been determined using the method detailed as per CS 466 Section C.3:

1. Using CS 466 Table C.7 (derived from CIRIA C778 [Ref 8.N]) to select consequential risk scores;

2. Adding together the risk scores to determine the total score 'Q' for the structure;

3. Using CS 466 Table C.8 for Q scoring bands to assign a consequential risk level

Consequential Risks (CS 466 Table C.7)		
Risk category	**Consequence**	**Risk Score Q**
Number of people killed or seriously injured	**Very high** Potential for 1 or more people to be killed or seriously injured.	**10**
Potential damage vehicles	**Very high** Potential for severe damage to one or more road vehicles on the A3 below footbridge.	**10**
Potential damage to utilities and other public or private services	**High** Buried 33kVA NIE cable runs along carriageway verge on Lake Road below the structure. Potential for severe disruption to this service in the event of a collapse.	**3**
Nature of route	**Medium** Urban footpath	**1**
Diversion route	**High** Pedestrian diversion route > 0.5 miles and ≤ 1mile.	**3**
Volume of traffic	**High** Pedestrian only: generally used.	**3**
Length of time to restore normal network operation	**Very High** >1 Month	**10**
Potential environmental pollution	**Low** No contamination of land or watercourses	**0**
Political and reputation damage	**Very High** National media coverage	**10**
Financial impact	**High**	**3**
	Sum	**53**

Table 10: Risk Score Q

5.5.2 From Table C.7 the total consequential risk score Q is **53**. From Table C.8 this gives the consequential risk R_Q level as **Very High.**

5.5.3 The vulnerable details risk R_V has been determined from Table C.9 as **High**. The Half-joint is a single vulnerable detail with multiple defects; fine cracking and active water leakage.

5.5.4 The half-joint form risk R_F has been determined using the method detailed below:

1. Select a half-joint type from CS 466 Figures C.3, joints of unknown type should be assumed as type A;

2. Score the joint type from CS 466 Table C.10;

3. Score the ease of access from CS 466 Table C.11; and,

4. Select the half-joint form risk rating from CS 466 Table C.12, which is determined by adding the score from Table C.10 and Table C.11.

5.5.5 From Figure C.3 the half-joint is Type A, solid or box slab with no access to the bearing shelf.

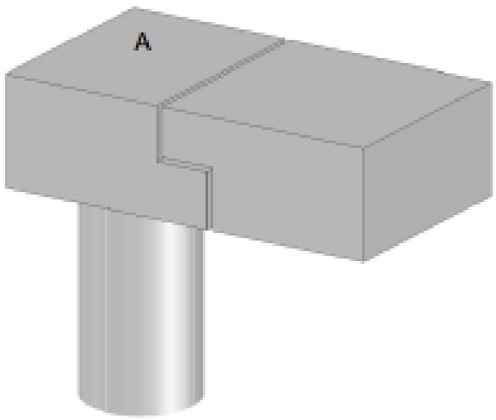


Figure 22: Type A half-joint (Extract from CS 466 Figure C.3)

5.5.6 From Table C.10; Type A joints are a solid or box slab with no access to the bearing shelf with a score of **7**.

5.5.7 From Table C.11; the ease of access to half-joint level is moderate with a score of **0**. Access to joint below can be accessed by MEWP and road/lane closures.

5.5.8 From Table C.12; the half-joint form risk R_F is **High**. Based upon the total (C.10 + C.11) score of **7**.

5.5.9 From Table C.13; the risk rating for other risks R_O is **low** as there are no other factors that will affect the risk of the structure.

5.6. Overall Risk Summary – Suspended span half joint

Primary risks			Primary risk rating	Secondary risks				Combined secondary risk rating	Refined Primary risk rating
Condition Risk	**Structural Risk**			**Consequential risk**	**Vulnerable details risk**	**Half joint form risk**	**Other risks**		
R_C	**R_{D1}**	**R_{D2}**		**R_Q**	**R_V**	**R_F**	**R_O**		
High	Medium	Medium	High	Very High	High	High	Low	High	**Very High**

5.6.1 The most severe risk from the list of Secondary Risks forms the Secondary Risk Rating. As the Consequential risk is considered Very High; the suspended span half joints are considered as Very High Risk overall.

6. Risk assessment for management of Abutment Half Joints

6.1. Introduction and scope

6.1.1 This section of the report will assign both a primary and secondary risk rating to the abutment half joint which is considered critical for the stability of the post-tensioned cantilever span. Where secondary risks are used to increase the primary risk rating, this is referred to as the 'refined primary risk rating.' The risk assessment process outlined in this section follows the guidance as set out in DMRB CS 466 Section 4 and Appendix C. The primary risk rating is calculated as per Table C.1 of CS 466 using a combination of the half joint condition risk and structure risk.

Table C.1 Matrix for determining primary risk rating of a half-joint structure

		Structural risk R_D			
		R_D **Very high**	R_D **High**	R_D **Medium**	R_D **Low**
Condition risk R_C	R_C **Very high**	Very high (1)	Very high (1)	High (1)	High (1)
	R_C **High**	Very high (1)	High (1)	High (1)	Medium (1)
	R_C **Medium**	High (1)	High (1)	Medium (1)	Medium (1)
	R_C **Low**	High (1)	Medium (1)	Medium (1)	Low (1)
Note (1) = The primary risk rating determined from the combination of R_C and R_D .					

Figure 23: Primary Risk Rating Matrix – CS 466 Appendix C (Table C1)

6.2. Condition Risk

6.2.1 The condition risk R_C is determined using the method outlined in CS 466 Appendix C.

Half Joint Defect Identification		
Half Joint Defect	**Description**	**Photo**
Active water leakage	Longstanding active water leakage and stalactite formation from half joints.	

Defect Decision Diagram

Figure C.1 Defects decision diagram

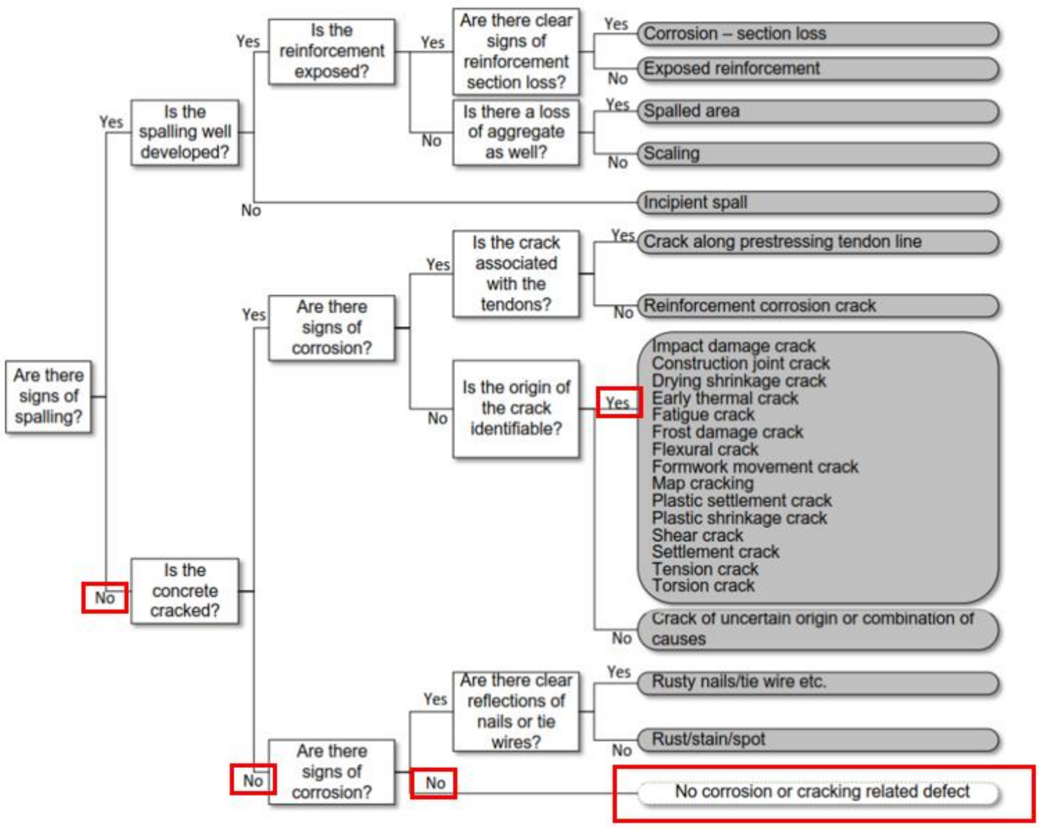


Figure 24: Defect Decision Diagram - CS 466 Figure C.1

6.2.2 From the Defects decision diagram Figure 24, there are no signs of defects to the abutment half joint. However, as the half joint is concealed, the presence of concrete defects is uncertain.

Damage Rating

Damage	Very slight	Slight	Moderate	Severe	Very severe
Cracks in prestressed concrete	width <0.05 mm	width 0.05-0.1mm	width 0.1-0.3mm	width 0.3-1.0mm	width 1-3mm with some spalling
Cracks in reinforced concrete	width <0.1mm	width 0.1-0.3mm	width 0.3-1.0mm	width 1-3mm with some spalling	width >5mm with widespread spalling
Effects of reinforcement corrosion	light rust stains	heavy rust stains	heavy rust stains and cracking along line of bar	heavy rust stains and spalling along line of bars	heavy rust stains and spalling along line of bars, in more than one location or for a number of bars.
Joint leakage/ water staining	clean/clear water, small patches of dampness	clean water seepage to large areas of the joint.	water seepage with discolouration at edge beam/footpath locations	30-70% of joint with discolouration gritted road/ coastal environment	affecting >70% of joint with discolouration with signs of historic presence. gritted road/coastal environment
Spalling/ delamination (1)	cover <50x50mm	100x100mm	area up to 150x150mm across	area larger than 150 x 150mm	multiple areas >250 x 250mm; single areas more than 1000 x 1000mm
Note (1) - The size of a concrete spall should be compared to relative to the size of the half joint. The criteria is for guidance purposes and is not prescriptive.					

Figure 25: Damage Rating - CS 466 Table 6.2

6.2.3 The joint leakage at the abutment half joint is assumed to be **Severe** as it covers 30% – 70% of the half joint and it is assumed that the footbridge is gritted/salted given the signs of stalactite formation.

Condition Risk Rating

6.2.4 Following CS 466 Table C.3, the condition risk of the suspended span half joints is **Very High** as a single severe condition defect has been recorded.

6.3. Structural risk

Detailing Risk RD_1

6.3.1 The reinforcement details of the holding upper abutment holding down nib are not known. The detailing of the lower nib is shown in Figure 26 below and shows a well anchored horizontal nib bar. Detailing risk from CS 455 Table C.5 is **Medium** as detailing allows a single clear load path through the half-joint with adequate anchorage – no diagonal bar. It must be noted that the half joint at the abutment is a holding down nib, therefore it is likely that the forces are not as onerous as a suspended span.

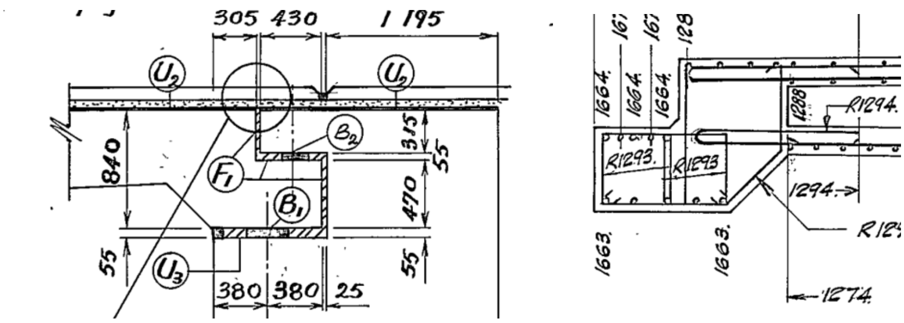


Figure 26: Half-joint reinforcement detailing

Structural Risk RD_2

6.3.2 There are no records of any structural assessment undertaken on the abutment holding down nib. However, the uplift forces are likely not significant, and even a poorly detailed half joint would likely provide sufficient resistance to support the uplift forces.

6.4. Primary risk rating

6.4.1 The Half-joint Condition risk R_C is considered **Very High,** and the Structural risk R_D is considered **Medium**. Table C.1 gives the Primary risk rating as **High.**

		Structural Risk R_D			
		Very High	High	**Medium**	Low
Condition Risk RC	**Very High**	Very High	Very High	**High**	High
	High	Very High	High	High	Medium
	Medium	High	High	Medium	Medium
	Low	High	Medium	Medium	Low

Table 11: Primary risk rating matrix

6.5. Secondary risks

6.5.1 The Consequential risk R_Q has been determined using the method detailed as per CS 466 Section C.3:

1. Using CS 466 Table C.7 (derived from CIRIA C778 [Ref 8.N]) to select consequential risk scores;
2. Adding together the risk scores to determine the total score 'Q' for the structure;
3. Using CS 466 Table C.8 for Q scoring bands to assign a consequential risk level

Consequential Risks (CS 466 Table C.7)		
Risk category	**Consequence**	**Risk Score Q**
Number of people killed or seriously injured	**Very high** Potential for 1 or more people to be killed or seriously injured.	**10**
Potential damage vehicles	**Very high** Potential for severe damage to one or more road vehicles on the A3 below footbridge.	**10**
Potential damage to utilities and other public or private services	**High** Buried 33kVA NIE cable runs along carriageway verge on Lake Road below the structure. Potential for severe disruption to this service in the event of a collapse.	**3**
Nature of route	**Medium** Urban footpath	**1**
Diversion route	**High** Pedestrian diversion route > 0.5 miles and ≤ 1mile.	**3**
Volume of traffic	**High** Pedestrian only: generally used.	**3**
Length of time to restore normal network operation	**Very High** >1 Month	**10**
Potential environmental pollution	**Low** No contamination of land or watercourses	**0**
Political and reputation damage	**Very High** National media coverage	**10**
Financial impact	**High**	**3**
	Sum	**53**

Table 12: Risk Score Q

6.5.2 From Table C.7 the total consequential risk score Q is **53**. From Table C.8 this gives the consequential risk R_Q level as **Very High.**

6.5.3 The vulnerable details risk R_V has been determined from Table C.9 as **High**. The Half-joint is a single vulnerable detail with active water leakage.

6.5.4 The half-joint form risk R_F has been determined using the method detailed below:

1. Select a half-joint type from CS 466 Figures C.3, joints of unknown type should be assumed as type A;
2. Score the joint type from CS 466 Table C.10;
3. Score the ease of access from CS 466 Table C.11; and,
4. Select the half-joint form risk rating from CS 466 Table C.12, which is determined by adding the score from Table C.10 and Table C.11.

6.5.5 From Figure C.3 the half-joint is Type A, solid or box slab with no access to the bearing shelf.

6.5.6 From Table C.10; Type A joints are a solid or box slab with no access to the bearing shelf with a score of **7**.

6.5.7 From Table C.11; the ease of access to the abutment half joints is moderate with a score of **0**. Moderate has been considered appropriate, as localised de-vegetation and excavation of revetments completed to achieve access to the holding down nib.

6.5.8 From Table C.12; the half-joint form risk R_F is **High**. Based upon the total (C.10 + C.11) score of **7**.

6.5.9 From Table C.13; the risk rating for other risks R_O is **low** as there are no other factors that will affect the risk of the structure.

6.6. Overall Risk Summary – Abutment half joint

Primary risks			Primary risk rating	Secondary risks				Combined secondary risk rating	Refined Primary risk rating
Condition Risk	**Structural Risk**			**Consequential risk**	**Vulnerable details risk**	**Half joint form risk**	**Other risks**		
R_C	**R_{D1}**	**R_{D2}**		**R_Q**	**R_V**	**R_F**	**R_O**		
High	Medium	Medium	High	Very High	High	High	Low	High	**Very High**

6.6.1 The most severe risk from the list of Secondary Risks forms the Secondary Risk Rating. As the Consequential risk is considered Very High; the abutment span half joints are considered as Very High Risk overall.

7. Conclusion

7.1.1 Moylinn East Footbridge is a structure consisting of both post-tensioned and half joint elements. This report has combined the Risk Review and Risk Assessment procedures as set out in DMRB CS 465 and CS 466 for Post-Tensioned and Half Joint structures respectively.

7.2. Post-Tensioned Structure

7.2.1 Following the CS 465 Risk Review, a number of hazards to the post-tensioning have been identified that have the potential to cause deterioration to the post-tensioning system if not properly investigated and managed. The CS 465 Risk Assessment has been completed within Section 3 of this report, whereby a number of identified hazards that present a risk to the integrity of the structure and its post-tensioning system have been assessed. The likelihood of events occurring varied from medium to high, however none of the hazards were considered to indicate an immediate risk to the integrity of the structure.

7.2.2 The Risk Assessment concluded that the structure falls within the second most vulnerable age group and that the structural form suggests a high risk of brittle failure mode, which is dependent upon the abutment holding down nib.

7.2.3 In order to prioritise a PTSI Site Investigation and/or maintenance works to DfI post-tensioned structures; a Risk Rating score of 47% and a Mean Hazard Risk Level of 4.3 has been calculated in accordance with CS 465 Appendix B. This Hazard score places the post-tensioned elements of the structure in a medium hazard risk level.

7.2.4 The previous PTSI undertaken was in 2004 (18 years ago) were significant voiding and soft, wet grout was noted, however the tendons did not exhibit signs of severe corrosion. Given the time elapsed from the previous PTSI Site Investigation and the ongoing signs of leakage through joints and lack of maintenance; it is possible that some deterioration to the structures post-tensioning system may have taken place in the 18 years since the previous PTSI. As such, this report recommends completing a Technical Plan in line with CS 465 for a PTSI Site Investigation to help confirm the long-term adequacy of the post-tensioning with an appropriate degree of confidence.

7.3. Half-Joint Elements

7.3.1 The half joint which supports the suspended 'drop in' span and the holding down half joint at the abutment ends of the footbridge have both been risk assessed in accordance with CS 466. This report has determined that the refined Primary Risk Rating for all half joints is considered as Very High. The very high risk has been dictated by the consequence factor and the significant long standing active leakage with signs of stalactite formation at all half joints. Furthermore, in the absence of a lower bound half joint structural assessment; the safe ultimate load capacity of the half joints is currently uncertain.

7.3.2 In order to ascertain the long-term adequacy of the structure half-joints with an appropriate degree of confidence; a number of risk management measures are recommended and listed in Section 8 of this report.

7.4. Overall Risk and Conclusion

7.4.1 The structure has been classified as Very High Risk with the half joints being considered the critical element. In addition, the uncertainty in the cause of the apparent 'sag' in the structure raises some uncertainty in regards the structural behaviour of the footbridge. Although considered 'very high risk', immediate intervention is not required – the structure does not meet the criteria to be classed as an 'Immediate Risk Structure' at this time – however this may be subject to change following the outcomes of further site investigation and structural analysis.

7.4.2 Given the very high-risk classification of the structure, the uncertainty surrounding the deflected shape and inherent risks associated with half joint and post-tensioned structures; this report agrees with the findings of the 2021 DfI Options Report in that the structure should be demolished as a long-term risk management option.

7.4.3 Until such time that the structure is demolished and replaced and is required to remain in service; it is recommended that the Risk Management Plan as outlined in Section 8 of this report is implemented. The overall risk rating of the structure will be updated following the findings of the Risk Management Plan. Following completion of the structural assessment outlined within the recommended Short Term Risk Management Plan the structure may then be classified as Immediate Risk and should be managed in accordance with CS470 for Substandard Structures.

8. Risk Management Plan

8.1. Risk Management Plan – Short Term (within 1 year)

Site Investigation

8.1.1 In order to ascertain the long-term adequacy of the post-tensioning and half joints with an appropriate degree of confidence; it is recommended that a Technical Plan is developed for a combined PTSI and Half Joint Site Investigation as a risk management measure within one year. The focus of the Site Investigation is to determine the following:

- The level of deterioration of post-tensioned tendons strands since the previous PTSI where voided and soft grout was noted.
- Level of deterioration to Half joint elements.
- Signs of structural distress associated with possible deflection issues.

8.1.2 The Technical Plan for the Site Investigation shall include the following:

1. Duct and Tendon Exposures (DTEs) at the following critical sections:
   a. Intermediate Support Regions: Top of deck (4 No.)
   b. Mid-span regions: Both from the soffit and elevations (4 No.);
   c. Deck end spans adjacent to end anchorages: Elevations of deck (4 No.);
2. Corrosion Test Areas (CTAs) at selected DTE locations;
3. Residual Tendon Stress Tests at selected DTE locations to ascertain long pre-stress loss;
4. Extraction of 2No. 100mm diameter cores from the post tensioned deck soffit to facilitate a Go-Pro camera inspection of the deck voids. This inspection is focused on identifying the presence of ponding water and any subsequent concrete deterioration of hidden critical elements.
5. Removal of surfacing and waterproofing from top of deck corresponding to intermediate pier locations. This is required to determine the presence of flexural cracking of the deck top slab over the piers which may suggest loss of pre-stress and in turn help ascertain the cause of the deflection issue. It is recommended to complete concrete in-situ stress tests (Slotting or core test) to determine in-situ concrete stresses in the deck over the piers.
6. Removal of surfacing and waterproofing from top of deck corresponding to abutment and suspended slab half joint locations to check for signs of cracking or concrete spalling. Corrosion Test Areas (CTAs) to be completed at these regions corresponding to the top nib of both half joints.

7. Corrosion Test Areas (CTAs) to be completed to the soffit of the lower half joint nib at suspended span half joints.

8. Verify the width of re-entrant cracking to half-joint nib. Consider installation of crack width monitors. This review assumes these cracks are no greater than 0.3mm. Should the cracking exceed 0.3mm then the Damage Rating of these defects must be updated following CS466 Table C.2. This may in turn alter the overall risk rating.

Structural Assessment

8.1.3 In addition to the Site Investigation works, the following structural assessment work is recommended:

1. Structural Assessment of Post-Tensioned Deck

   The previous PTSI confirmed that significant voiding and poor quality of grout was evident throughout the structure. CS 455 Section 8.2.1 states that tendons with poorly grouted ducts should be assessed to be unbonded. Reference is also made to CS 455 Section 8.1, Where structures have unbonded prestressing, the assessment shall verify that there is sufficient resistance to prevent collapse under permanent loads at the ULS in both of the following circumstances:

   i. Failure of any two tendons at a cross section; and,

   ii. Failure of 25% of the tendons at a cross section.

   It is therefore recommended to complete a structural assessment of the post-tensioned deck considering the effects of the unbonded tendons on the safe load carrying capacity of the deck. The structural assessment will also identify any potential issues with excessive long term pre-stress loss.

   The structural assessment of the half joint deck will determine the uplift force at the end abutments. in order to mitigate any potential issues with the abutment holding down nib – consideration could be given to installing kentledge blocks at the deck ends to counteract the uplift force and thus make redundant the holding down nib. The feasibility of this proposal would be confirmed following the calculation of the uplift force and kentledge requirements.

2. Half Joint Structural Assessment – ULS

   As there has been no previous lower bound assessment undertaken to the suspended span half joints for the Ultimate Limit State; it is recommended that a ULS strut-and-tie analysis is completed on the half joints to determine the safe load carrying capacity.

Budget Costs

Recommendation	Estimated Cost	
Development of Technical Plan for Site Investigation works, inc. PCI (Consultant Fees)	£	15,000
Site Investigation Works (Trial holes, DTEs, EAEs, CTAs and in-situ testing)	£	50,000
Site Investigation Report (Consultants Report)	£	10,000
Post-Tensioned Structural Assessment (inc. Check)	£	40,000
Half Joint Structural Assessment (inc. Check)	£	30,000
Total	£	**145,000**

8.2. Long term Risk Management Plan

8.2.1 A long-term risk management strategy including replacement of the structure shall be developed following the outcomes of the short-term plan as outlined in Section 8.1.

Appendix A: Point Cloud Survey – Deflection Check

File ref - c:\users\153635\onedrive - amey plc\desktop\co402011-amy-200 deflection check.dwg

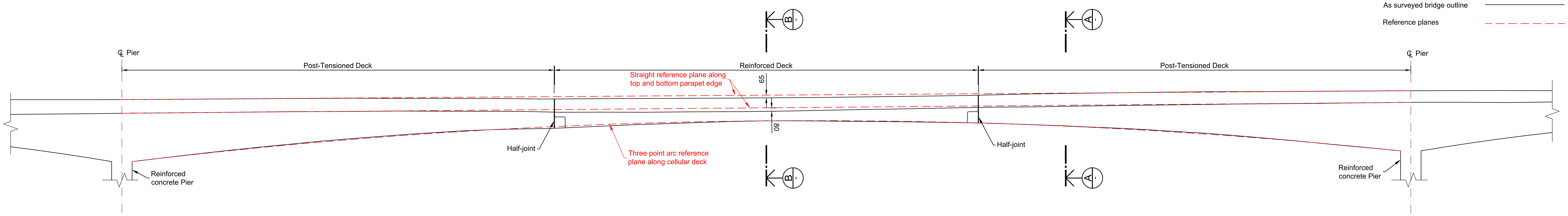
NOTES

1. All dimensions are in are in millimetres unless stated otherwise.
2. All existing geometry provided is based on bridge record drawings.

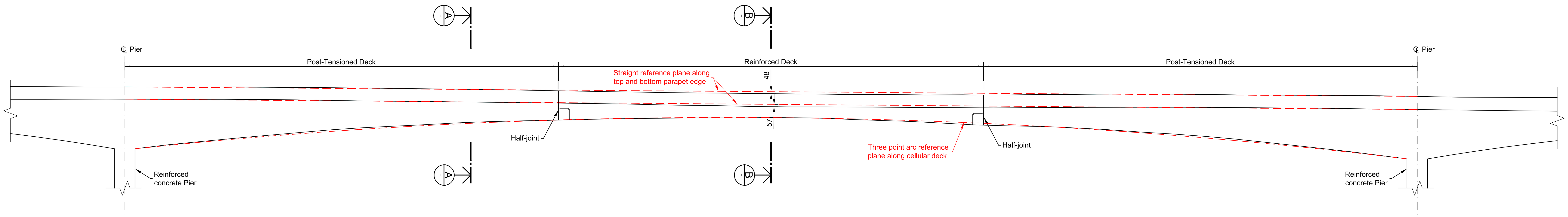
KEY

As surveyed bridge outline

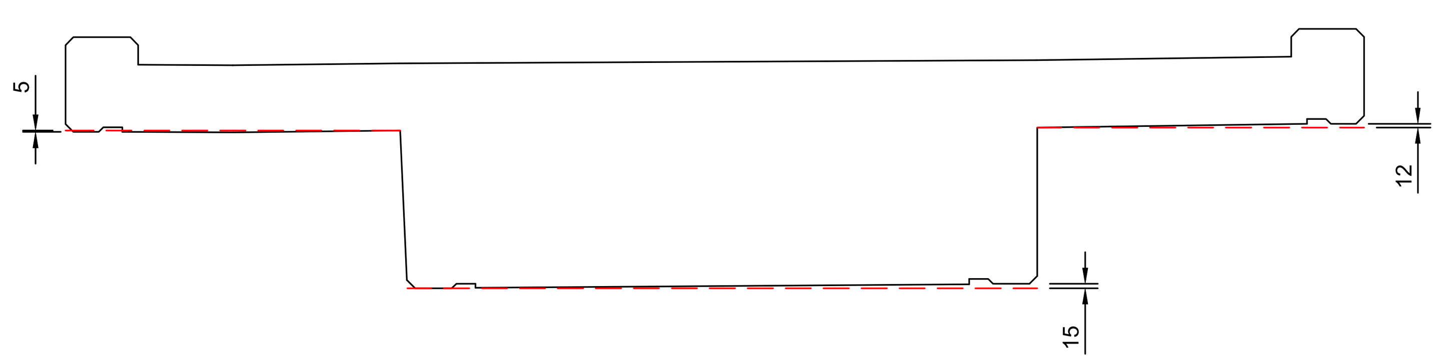
Reference planes



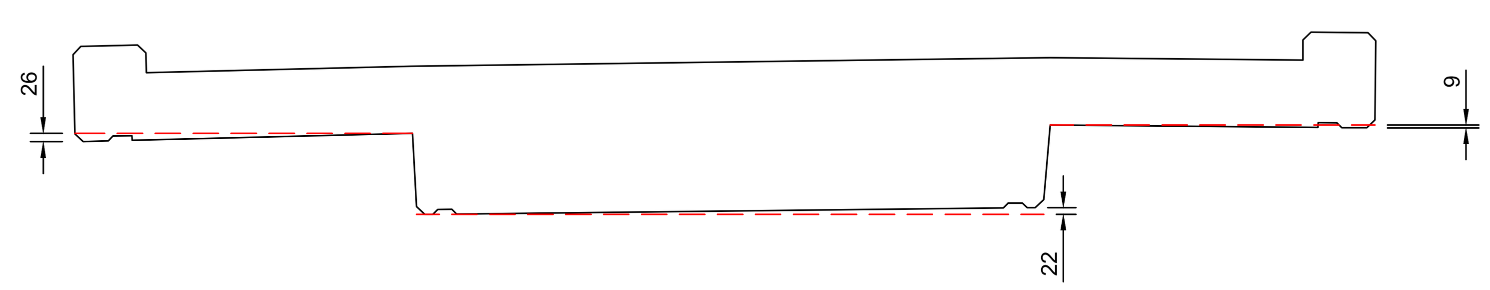
South Elevation
Scale 1:50



North Elevation
Scale 1:50



Section A-A Through cantilever Deck
Scale 1:20



Section B-B Through Drop in Deck
Scale 1:20

Rev	Revision details	Drwn	Chkd	Appd	Date

Designed: [redacted]		Date: .
Drawn:		Date: .
Snr Tech Check:	.	Date: .
Checked: [redacted]		Date: .
Approved:	.	Date: .

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Client
Text or Logo

Project Name
Moylinn East Footbridge
Structure no. 40062

Drawing Title
Point cloud survey
Deck Deflection check

Original Drawing Size : A1 | Scale : As Shown
Dimensions : -

Drawing Status
Suitable for Information

Suitability
S2

Drawing No
CO402011-AMEY-A3-DR-X-200

Rev
P01