

Imperial College London
Department of Civil and Environmental Engineering

ASSESSING GROUND INTERACTION EFFECTS AND
POTENTIAL DAMAGE ON EXISTING TUNNELS BEFORE
AND AFTER NEW EXCAVATION WORKS

Jessica Bing Yan Yu

PhD Thesis
March 2014

I declare that this submission is my own work and to the best of my knowledge it contains no materials previously published or written by another person, except where due reference is made in the thesis. Any contribution made to the research by others, with whom I have worked at Imperial College, is explicitly acknowledged in the thesis.

The copyright of this thesis rests with the author and is made available under a Creative Commons Attribution Non-Commercial No Derivatives licence. Researchers are free to copy, distribute or transmit the thesis on the condition that they attribute it, that they do not use it for commercial purposes and that they do not alter, transform or build upon it. For any reuse or redistribution, researchers must make clear to others the licence terms of this work.

Abstract

The need for the research project is driven by the Crossrail project in London for which new 7 m diameter tunnels are to be constructed close to numerous existing operational tunnels of the London Underground (LU) network. The main aim of this research is to investigate the impact of new tunnel excavations on existing tunnels.

This research component is based on field instrumentation and experimental work conducted on half-scale grey cast iron (GCI) tunnel lining segments with chemical composition similar to the Victorian age GCI segments in the LU network. Currently, there is great uncertainty about the behaviour of segmental linings. General belief is that the behaviour of the lining is influenced by the behaviour of the joints, but there has been little experimental work to investigate this relationship.

The laboratory experiments aim to find out the deformation behaviour of the bolted segmental lining and the influence of parameters such as overburden pressure, bolt preload and presence of grommets at small distortions. The measured behaviour of the segmental lining is compared against the theoretical behaviour of a continuous lining based on the assumption of linear elasticity. The laboratory results are used to assess the validity of the tunnel assessment methods used by the industry. The results from the parametric tests will form the basis for future experimental investigations taking the half-scale test ring to large deformations and ultimately to failure.

The field component involved taking measurements of existing bolted segmental grey cast iron tunnels. Small sections of tunnels constructed at the LU Acton Depot, at Tottenham Court Road Station and in the Central Line running tunnel were monitored to gain an understanding of the deformation of tunnels from construction and self-weight, from ground loading and finally from the influence of adjacent tunnelling works. The thesis proposes recommendations for future in-tunnel monitoring based on the findings obtained in this research.

Acknowledgements

I wish to acknowledge the financial support from the EPSRC, the financial and in-kind contributions made by our industry collaborators Crossrail Ltd., Morgan Sindall and London Underground Ltd. I am deeply grateful for the guidance and insights from my supervisors - Dr. Jamie Standing, Dr. Robert Vollum, Prof. David Potts and Prof. John Burland. The most sincere thank you to research technicians Mr. Duncan Parker, Mr. Steve Ackerley and Mr. Leslie Clark. Thank you to Mr. Mike Black, Mr. Neil Moss, Dr. Keith Bowers, Mr. Colin Eddie, Mr. David Harris, Mr. Peter Wright, Mr. Sotiris Psomas, Dr. Barry New, Mr. Tim Morrison, Mr. Mick Maloney, Mr. Chris Barnes, Mr. Jeremy Pallant and Ms. Anu Balasubramanian.

Thank you to my husband for joy and laughter.

Thank you Lord Jesus for giving my life eternal purpose.

Table of Contents

Abstract.....	3
Acknowledgements.....	3
1 Introduction.....	11
1.1 Background.....	11
1.2 Aims of the overall research project.....	12
1.3 Methodology.....	12
1.4 Layout of the thesis	15
2 Literature review	17
2.1 Use of grey cast iron as a tunnel lining material.....	17
2.2 Grey cast iron material properties.....	18
2.3 Existing deformations of tunnel linings.....	21
2.4 Earth pressure acting on tunnel lining	24
2.5 Deformations of tunnel linings due to adjacent excavations	26
2.6 Previous laboratory tests.....	26
2.7 Tunnel lining assessment.....	27
2.8 Discussion	32
3 Field instrumentation	34
3.1 In-situ measurements at LU Acton Depot	34
3.2 In-situ measurements at Tottenham Court Road platform tunnel	34
3.3 In-situ measurements in the Central Line running tunnel near Lancaster Gate	41
3.4 Fibre optics installations by ETH Zurich.....	47
3.5 Discussion	48
4 Material composition and testing	50
4.1 Production of grey cast iron segments.....	50
4.2 Mechanical tests on grey cast iron specimens	51
4.3 Previous wrought iron bolt test data	54
4.4 Testing of wrought iron bolt specimens	55
4.5 Use of grommets.....	57
4.6 Discussion	57
5 Revisiting the two segment test.....	59
5.1 Initial estimate of segment and joint moment capacities at zero hoop force	59
5.2 The two segment test setup.....	62
5.3 Loading of two segment test.....	63
5.4 Finite element modelling	64
5.5 Comparison of FE prediction with laboratory results.....	65
5.6 Influence of bolt preload.....	66
5.7 Comparison with Thomas' tests	68
5.8 Change of loading direction	69
5.9 Mode of bending in the longitudinal flange.....	69

Table of contents

5.10	Revised estimate of joint moment capacity for full-ring test.....	70
5.11	Results from strain gauges.....	70
5.12	Distortion of the skin during positive bending	71
5.13	Discussion	71
6	Development of full-ring setup and planning of tests	74
6.1	Design of test rig components	74
6.2	Instrumentation.....	76
6.3	Loading procedure.....	80
6.4	2D finite element modelling.....	84
6.5	Discussion	86
7	Behaviour of bolted segmental cast iron ring.....	89
7.1	Rationale for testing regime	89
7.2	Measurements and corrections for change in radius.....	90
7.3	Measurements of strain and calculations for bending moment.....	92
7.4	Using linear elastic solutions to predict the onset of joint opening	95
7.5	The measured behaviour at the joints	97
7.6	Distortion of circumferential flanges.....	99
7.7	Measurements from strain gauge rosettes.....	100
7.8	Discussion	100
8	Conclusions and future research	105
8.1	Conclusions	105
8.2	Future research	110
Figures		114
2	Literature review	114
3	Field instrumentation	124
4	Material composition and testing	143
5	Revisiting the two segment test.....	161
6	Development of full-ring setup and planning of tests	204
7	Behaviour of bolted segmental cast iron ring.....	226
References.....		255
Appendix.....		261
A.	Earth pressure acting on tunnel lining.....	261
B.	Deformation of existing tunnels due to adjacent excavations	268
C.	Calculations for grey cast iron segment bending capacities.....	276
D.	Estimation of bending moment in 2-segment test.....	278
E.	Calculations of change in bending moment from statics	280
F.	Design of reaction ring.....	282
G.	Hanging frame design	284
H.	Loading sequence to bring all actuators under equal load	287
I.	Measurements from strain gauge rosettes from full ring tests	294
J.	Figures	301

Table of Figures

Figure 2-1. Terms relating to positions in a ring (Thomas, 1974).....	114
Figure 2-2. Effect of section on strength (Angus, 1976).....	114
Figure 2-3. Stress/strain characteristics of a grey iron to BS1452 Grade 17 tested on a 2.1in bar. (a) Longitudinal stress/strain curve in tension; (b) lateral stress/strain curve in tension; (c) Elastic modulus E (longitudinal) at different stress levels (Angus, 1976).....	115
Figure 2-4. Elastic modulus of cast iron. Variation with tensile stress for irons of different strength (Angus, 1976).	116
Figure 2-5. Stress/strain relationship in (top) tension and (bottom) compression (Angus, 1976).....	117
Figure 2-6. Typical stress/strain curves for the 1st application of stress of 170 MPa and the stabilised stress/strain curve after 10 applications of 170 MPa tensile stress (Angus, 1976).	118
Figure 2-7. Variation in Poisson's ratio with stress (a) tensile (b) compressive (Angus, 1976).	119
Figure 2-8. Definition of 'ovalisation' parameter.	120
Figure 2-9. Summary of survey results for Northern Line running tunnel between Tottenham Court Road and Leicester Square. Majority of survey points fall between ovalisation of $\pm 1\%$	120
Figure 2-10. Bending test on SGI tunnel segment (Thomas, 1977).	121
Figure 2-11. Schematic diagram showing distortion associated with positive bending (distortion is exaggerated).	121
Figure 2-12. Schematic diagram showing distortion associated with negative bending (distortion is exaggerated).	122
Figure 2-13. Rubber bag strapped to the back of a SGI segment for the application of a uniform pressure (Thomas, 1977).	123
Figure 2-14. Bending test on the horizontal joint between two half segments of GCI lining (Thomas, 1977)... ..	123
Figure 3-1. A section of GCI tunnel constructed above ground at the LU Acton Depot.	124
Figure 3-2. Directions of circumferential and longitudinal strains.	124
Figure 3-3. The DEMEC gauge.	125
Figure 3-4. Rosette type electrical resistance strain gauge.	125
Figure 3-5. P3 Strain Indicator.	125
Figure 3-6. LVDT displacement transducer in metal holder.	126
Figure 3-7. Dataloggers for LVDT and temperature gauge housed in plastic box.	126
Figure 3-8. Northern Line northbound platform tunnel looking north.	127
Figure 3-9. Approximate locations of strain gauges. (S=DEMEC span. R=strain gauge rosette).	127
Figure 3-10. Strain gauges on segment 1. Without protective coating (left) and with protective coating (right).	128
Figure 3-11. Strain gauges on segment 2.	128
Figure 3-12. As built photo showing locations of LVDTs and boxes housing loggers.	129
Figure 3-13. Wire bracket to protect strain gauge rosettes.	129
Figure 3-14. Development of strains on the circumferential flange.	130
Figure 3-15. Comparison of LVDT trends.	130
Figure 3-16. Plan view of Crossrail interface with Central Line east of Lancaster Gate Station.	131
Figure 3-17. Location of instrumented segments and joints on ring 336 and 360.	131
Figure 3-18. Location of strain gauges and displacement transducers on Ring 336 and Ring 360.	133
Figure 3-19. Photo of potentiometric displacement transducer installation by CMCS and tape extensometer eyebolt installation by Imperial College in Central Line.	134
Figure 3-20. Bracket for securing dataloggers to tunnel segments.	134
Figure 3-21. Additional datalogging unit on the eastbound platform headwall.	135
Figure 3-22. Typical strain gauge response for TBM1 passage showing segment bending (top), segment twisting (middle), and erratic gauge readings (bottom).	136
Figure 3-23. Typical strain gauge responses for TBM2 passage.	137
Figure 3-24. Changes in Central Line lining segment due to TBM1 (top) TBM2 (middle) and combined TBM 1 and TBM 2 (bottom).	138
Figure 3-25. Results from potentiometer displacement transducers at the joints for Ring 336 (top) and Ring 360 (bottom).....	139
Figure 3-26. Tape extensometer readings covering the passage of TBM1 for Ring 336 (top) and Ring 360 (bottom).....	140
Figure 3-27. Tape extensometer readings covering the passage of TBM2 for Ring 336 (top) and Ring 360 (bottom).....	141

Table of figures

Figure 3-28. Fibre optics fixed to the key pieces along the crown of Central Line.	142
Figure 4-1. Bottom pattern for cast iron segment.	143
Figure 4-2. Pouring sand into bottom mould.	143
Figure 4-3. Bottom part of sand mould.	144
Figure 4-4. Assembling the top and bottom halves of the sand mould for a segment.	144
Figure 4-5. Melting the charge mix.	145
Figure 4-6. Pouring the charge mix into the ladle.	145
Figure 4-7. Pouring the charge mix into the sand mould.	146
Figure 4-8. Half-scale cast iron segment prior to machining.	146
Figure 4-9. Stress-strain curves for 20mm diameter GCI test specimens.	147
Figure 4-10. Stress-strain curves for 10mm diameter GCI test specimens.	147
Figure 4-11. Total strain, plastic strain and elastic strain.	148
Figure 4-12. Tangent modulus for 20mm diameter GCI specimens.	148
Figure 4-13. Tangent modulus from 1 st loading curve.	149
Figure 4-14. Secant modulus for 20mm diameter GCI specimens.	149
Figure 4-15. Secant modulus from 1 st loading curve.	150
Figure 4-16. Secant and tangent moduli of 20mm dia. GCI specimens from 1 st loading curve.	150
Figure 4-17. Secant and tangent moduli of 10mm dia. GCI specimens from 1 st loading curve.	151
Figure 4-18. First wrought iron specimen tested with strain gauges and LVDT displacement transducers attached.	152
Figure 4-19. Measurements using strain gauges (top) and LVDTs (bottom).	153
Figure 4-20. Stress-strain curves of wrought iron specimen using average strains.	154
Figure 4-21. Stress-strain curves of all wrought iron specimens.	154
Figure 4-22. Stress-strain curves of wrought iron specimens at small strain range.	155
Figure 4-23. Pre-yield loops for specimen wrought iron bolt1811_3.	156
Figure 4-24. Stress-strain response of wrought iron specimens (with area reduction).	157
Figure 4-25. Examples of stress-strain response for the pre-yield unload-reload loops of wrought iron bolt specimens.	158
Figure 4-26. Examples of stress-strain response for the post-yield unload-reload loops of wrought iron bolt specimens.	159
Figure 4-27. Stress-strain data with best fit, upper bound and lower limit curves for GCI specimens.	160
Figure 5-1. Half-scale grey cast iron ring cross section. All dimensions in millimetres.	161
Figure 5-2. Half-scale grey cast iron ring internal elevation. All dimensions in millimetres.	162
Figure 5-3. Half-scale grey cast iron longitudinal joint.	163
Figure 5-4. Half-scale grey cast iron circumferential joint.	163
Figure 5-5. Schematic showing single curvature and double curvature bending.	164
Figure 5-6. Increase in bolt load against joint opening at intrados (Thomas, 1977).	164
Figure 5-7. Schematic diagram of the two segment test setup at Imperial College.	165
Figure 5-8. Instrumentation for the two segment test at Imperial College.	166
Figure 5-9. The two segment test setup at Imperial College with roller support at RHS end.	166
Figure 5-10. Line loads applied to the skin on the extrados of the segments.	167
Figure 5-11. One support of the jointed arch is pinned. A dial gauge was used to measure potential translation/rotation.	167
Figure 5-12. Locations of LVDTs at intrados: “middle”, “edge bolt” and “outer edge”.	168
Figure 5-13. LVDTs for measuring the deflection of the circumferential flanges and displacement transducer for measuring the skin deflection.	168
Figure 5-14. Grade 4.6 steel bolts instrumented with strain gauges.	169
Figure 5-15. Grommets were used in the final set of tests.	169
Figure 5-16. Close up view of the roller support.	170
Figure 5-17. FE meshes for segment with details for the joint and bolts.	171
Figure 5-18. Boundary conditions for FE mesh.	172
Figure 5-19. Application of loading as line load or equivalent point loads.	173
Figure 5-20. Comparison of point load and line load application using FE.	174
Figure 5-21. Comparison of ICFEP and laboratory results.	178
Figure 5-22. The joint opening magnitude is taken to be ‘x’ in the FEA.	179
Figure 5-23. ICFEP results for different joint element stiffness.	180
Figure 5-24. Different bolt stiffness moduli.	180
Figure 5-25. Decrease in bolt load with time after initial preload – grommets in place.	181
Figure 5-26. Comparison of segment behaviour with and without restriction to movement in the longitudinal direction.	184

Figure 5-27. FE prediction compared with measured response.	186
Figure 5-28. Bolted arch behaviour under different preload.	189
Figure 5-29. Bending moment plotted against measured joint opening (Thomas, 1977).	190
Figure 5-30. Comparison of Thomas' results with ICSEP prediction.	190
Figure 5-31. Theta ' θ ' denotes angular opening at intrados (c.f. Figure 5-22).	191
Figure 5-32. Using vertical and horizontal displacements to calculate rotation.	191
Figure 5-33. Comparison of joint rotation calculated using joint opening and rigid body movement.	191
Figure 5-34. Comparison of joint stiffness calculated using joint opening and rigid body movement.	192
Figure 5-35. Theta ' θ ' denotes angular opening at extrados.	192
Figure 5-36. Comparison between positive and negative moments.	193
Figure 5-37. Deformation of longitudinal flange along the intrados.	196
Figure 5-38. Horizontal flange showing locations of comparison on the intrados.	197
Figure 5-39. Inferred bending mode in horizontal flange: the red dashed lines indicate where the flange bends into the page.	197
Figure 5-40. Flange deformation and maximum tensile and compressive stresses around bolt hole on the longitudinal flange as predicted by FEA.	199
Figure 5-41. Measurements from t-gauges - change in microstrain.	200
Figure 5-42. Stress estimated using $E=100$ GPa and best fit stress-strain curve.	200
Figure 5-43. Results from strain gauge rosettes capturing change in strain when the moment at the joint increased from 0.25 kNm to 0.45 kNm. Tensile strains positive. Numbers denote strain magnitude in microstrain.	202
Figure 5-44. Distortion under positive loading.	203
Figure 6-1. Schematic drawing of full-ring setup.	204
Figure 6-2. Details of loading rig components.	205
Figure 6-3. Actual full-ring test setup. Locations where the actuators are removed are circled in red.	206
Figure 6-4. Air drying system.	206
Figure 6-5. Pressurised water supply for air/water interface.	207
Figure 6-6. Air/water interface.	207
Figure 6-7. Tangential stabilising rod.	208
Figure 6-8. Magnified images of strain gauges.	209
Figure 6-9. Strain relief wires connecting the rosette to terminals.	209
Figure 6-10. Coloured leadwires attached to terminals.	210
Figure 6-11. Strain gauge locations for segment H.	210
Figure 6-12. Strain gauge locations for segment I.	211
Figure 6-13. Strain gauge locations for segment J.	212
Figure 6-14. Strain gauge locations for segment G.	213
Figure 6-15. Strain gauged displacement transducer.	214
Figure 6-16. The inside of a strain gauged displacement transducer, with hand holding onto plunger rod.	214
Figure 6-17. Micrometer for calibrating strain gauged displacement transducers.	215
Figure 6-18. Strain gauged displacement transducers placed around the test ring.	215
Figure 6-19. Strain gauged displacement transducer (circled in red) for detecting rotation of the ring.	216
Figure 6-20. Strain gauged displacement transducer for monitoring convergence/divergence of circumferential flanges.	217
Figure 6-21. Strain gauged pressure transducer.	217
Figure 6-22. Budenberg pressure gauge for calibrating pressure transducer.	218
Figure 6-23. Pressure transducer connected to air/water interface. A load cell is located in front of the	218
Figure 6-24. Pancake style load cell.	219
Figure 6-25. LVDT for measuring joint opening.	219
Figure 6-26. Locations of LVDTs.	219
Figure 6-27. Instron loading rig.	220
Figure 6-28. Instrumented bolt used on test ring.	220
Figure 6-29. Comparison of segment and joint capacities at different depths.	221
Figure 6-30. Loading regime for laboratory tests.	221
Figure 6-31. Schematic diagram showing how the loading was analysed using Castigliano's second theorem.	222
Figure 6-32. Difference in bending moment between point load and loading pad analyses for unloading case (top) and loading case (bottom).	223
Figure 6-33. Difference in displacement between point load and loading pad analyses for unloading case (top) and loading case (bottom).	224
Figure 6-34. 2D structural FE model with normal stresses applied at discrete 'pad' locations.	225
Figure 7-1. Sign convention for test ring.	226
Figure 7-2. Calculation of radius change.	227

Figure 7-3. The test ring is distorted after the application of equal loads at all actuator locations.	227
Figure 7-4. Normalised hoop force from Stage One loading.	228
Figure 7-5. Measured displacement for unloading tests. Results for two different bolt preloads shown.	229
Figure 7-6. Measured displacement for loading tests. Results for two different bolt preloads shown.	230
Figure 7-7. ‘Ellipse’ superimposed over unloading displacements (top) and loading displacements (bottom). .	231
Figure 7-8. Comparison of bending moments on segment I (100 degrees) and segment G (260 degrees). Unloading case.	232
Figure 7-9. Comparison of bending moments on segment I (100 degrees) and segment G (260 degrees). Loading case.	232
Figure 7-10. Unloading Case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 5kN (top). Bending moments compared with joint capacities at 40% (bottom).	233
Figure 7-11. Unloading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 10kN (top). Bending moments compared with joint capacities at 40% (bottom).	234
Figure 7-12. Unloading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 5kN with grommets (top). Bending moments compared with joint capacities at 40% (bottom).	235
Figure 7-13. Loading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 5kN (top). Bending moments compared with joint capacities at 40% (bottom).	236
Figure 7-14. Loading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 10kN (top). Bending moments compared with joint capacities at 40% (bottom).	237
Figure 7-15. Loading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 5kN with grommets (top). Bending moments compared with joint capacities at 40% (bottom).	238
Figure 7-16. Bending moments calculated from Morgan’s Equation compared with bending moments calculated from strain measurements and analytical predictions based on the deformed shape. Unloading case (top). Loading case (bottom).	239
Figure 7-17. Unloading case. Changes in actuator load required to deform the ring by the same magnitude. ...	240
Figure 7-18. Loading case. Changes in actuator load required to deform the ring by the same magnitude.	240
Figure 7-19. Unloading case. Change in bolt load from Stage One to Stage Two.	242
Figure 7-20. Reduction of bolt preload in all the instrumented bolts when grommets were in place.	243
Figure 7-21. Loading case. Change in bolt load from Stage One to Stage Two.	245
Figure 7-22. Unloading case. Local displacement at joint locations. All-round normal load prior to distortion = 10 kN.	246
Figure 7-23. Unloading case. Local displacement at joint locations. All-round normal load prior to distortion = 20 kN.	247
Figure 7-24. Unloading case. Local displacement at joint locations. All-round normal load prior to distortion = 40 kN.	248
Figure 7-25. Loading case. Local displacement at joint locations. All-round normal load prior to distortion = 10 kN.	249
Figure 7-26. Loading case. Local displacement at joint locations. All-round normal load prior to distortion = 20 kN.	250
Figure 7-27. Loading case. Local displacement at joint locations. All-round normal load prior to distortion = 40 kN.	251
Figure 7-28. Joint movement along intrados and extrados for different bolt preloads. Normal actuator load = 10 kN.	252
Figure 7-29. Unloading case. Change in distance between circumferential flanges.	253
Figure 7-30. Loading case. Change in distance between circumferential flanges.	253
Figure 7-31. Comparison of strains on Segment H. Tensile strains are positive. C=circumferential strain. R = radial strain. Tensile strains are positive.	254

Table of Tables

Table 2-1. Tensile Strength of grey cast iron (BS 1452:1961).....	19
Table 2-2. Chemical composition of cast iron.	21
Table 2-3. Historical lining stress measurements.....	25
Table 3-1. DEMEC results for circumferential flanges.....	38
Table 3-2. Strain gauge results for segment 1.....	39
Table 3-3. Strain gauge results for segment 2.....	40
Table 3-4. Comparison of DEMEC and strain gauge rosette results.....	40
Table 3-5. Intermediate strain gauge results for segment 1.....	41
Table 3-6. Intermediate strain gauge results for segment 2.....	41
Table 3-7. Locations of installed instrumentation.....	42
Table 3-8. Locations where principal strains could not be calculated due to erratic gauge measurements.	46
Table 4-1. Grey cast iron charge mix for half-scale test segments.....	50
Table 4-2. Chemical composition of cast iron.	51
Table 4-3. Details of as-cast test bars.....	51
Table 4-4. Test specimen dimensions.....	52
Table 4-5. Estimation of shear strength for wrought iron bolts.....	54
Table 4-6 Properties of Grade 4.6 bolt and wrought iron.....	56
Table 4-7. Grey cast iron properties for use in conjunction with laboratory experiments.	57
Table 5-1. Initial estimate of joint bending capacity.....	62
Table 6-1. Strain gauged locations (see Figure 6-11 to Figure 6-14).....	78
Table 6-2. Estimated BM capacity for joint and segment under zero hoop load.	81
Table 6-3. Estimated BM capacity for joint including the contribution from hoop load.	81
Table 6-4. Linear elastic equations for ring under any number of equal radial forces equally spaced (Young and Budynas, 2002).	82
Table 6-5. Unloading regime for main series of tests.....	83
Table 6-6. Loading regime for main series of tests.....	83
Table 7-1. Predicted onset of joint opening for unloading case based on $E = 100$ GPa.....	96
Table 7-2. Predicted onset of joint opening for loading case based on $E = 100$ GPa.....	96
Table 7-3. Expected joint movements along joint extrados and intrados.....	99

1 Introduction

1.1 Background

As urban environments become more developed in most major cities, the underground space is also becoming more congested. Construction works are often carried out in close proximity to existing near- and subsurface structures such as basements, tunnels (for railways and services) and cut-and-cover openings. In particular, the construction of new tunnels and deep excavations can cause significant ground movements which change the stress regime around the existing structures. These inevitably cause structural deformations. There is always concern that the deformations will interfere with the running of trains (e.g. when acceptable kinematic envelopes are exceeded), the function of services (particularly brick-lined sewers and tunnels) and ultimately lead to failure of the structure.

Because of these concerns, stringent tolerances are often specified. Experience within London has found that frequently the specified tolerances have been exceeded even before any new excavation takes place, with most existing tunnels having an elliptical form with the horizontal diameter greater than the vertical diameter, i.e. squatting. This form results from a combination of deformations due to imperfect construction, the self-weight of the ring when it was assembled causing it to squat and ground stresses that developed subsequently. There is little guidance and knowledge relating to how much movement a tunnel can sustain before it approaches a yield point or ultimately failure. There are two issues: one relating to serviceability, the other structural stability: clearly both aspects have important safety implications.

The research described in this thesis relates to the conditions in and around existing tunnels before and after new nearby excavation works, in particular new tunnelling operations. The need for the project was driven by the Crossrail project in London for which new tunnels are to be constructed beneath 43 existing operational tunnels.

After detailed discussions with industry collaborators including Crossrail Ltd. (CRL), London Underground Ltd. (LU), and Morgan Sindall, a decision was taken to focus mainly on existing tunnels lined with grey cast iron (GCI). These are frequently encountered as GCI segments have been used extensively for lining tunnels in London from 1869 when the Tower Subway was built (Copperthwaite, 1906, Hewitt and Johannesson, 1922) until the introduction of less brittle spheroidal graphite iron (SGI) about a century later. Worldwide, GCI linings were used in New York, Budapest, Hong Kong, and Beijing, to name a few. In practice there is great uncertainty about the behaviour of GCI segmental linings.

In the past three decades extensive knowledge has been gained about ground response induced by the construction of tunnels and deep excavations. This has primarily been achieved through detailed field monitoring. However, although much information has been gained in terms of ground displacements around tunnels and to a lesser degree the magnitude of excavation-induced movement in existing tunnels, there is still great uncertainty over the state of stress acting on and within tunnel linings in-situ.

1.2 Aims of the overall research project

There are three main aims of the overall research project. One is to gain a better understanding of the range and type of deformations and stresses within existing tunnels prior to them being affected by any external influences. Another aim is to identify limits of deformation (strain) and stress that an existing tunnel can sustain before exceeding serviceability and ultimate limit states. These two aims are achieved through field measurements, including reviewing existing records, large-scale laboratory model studies and numerical analysis. Finally field measurements within and around existing tunnels are made during construction of the Crossrail tunnels, or other works interacting with LU's buried assets, to confirm the findings from the laboratory studies. Numerical analyses are performed to simulate the same Crossrail conditions and comparisons made with the monitoring data as part of a calibration and validation exercise.

The ultimate cumulative aim is to provide better understanding and guidelines, based on a combined scientific-empirical approach, about how much movement an existing GCI lined tunnel can safely sustain without adversely affecting its serviceability. This will clearly have great benefit to future infrastructure projects where it is vital that existing underground transport systems are kept running smoothly and safely.

A three-path approach was planned for the overall research project. Large-scale laboratory testing was performed in the Structures Laboratory on a half-size (6 foot diameter) instrumented GCI segmental ring. Connections between the elements, e.g. bolts, gaskets, along with the way they articulate were monitored. Field monitoring of existing tunnels and adjacent ground influenced by new tunnelling and excavation works were undertaken. Advanced numerical analysis was performed to model conditions in and around the structures both in the laboratory and field.

The author's remit was the laboratory study, supplemented by structural instrumentation and monitoring of existing tunnels. The field monitoring of ground response (Wan, 2014) and the advanced numerical analysis (Avgerinos, 2014) were the remit of other PhD and post-doctoral researchers dedicated to this research project.

1.3 Methodology

1.3.1 Laboratory study

This thesis focuses on the experimental component of the overall research project. The primary thrust of the laboratory study was to load a large-scale model tunnel to simulate in-situ conditions, monitoring a number of variables as loading was applied. The tunnel was represented by a single ring (of GCI segments) simulating a plane section through a tunnel.

The model lining was to 1:2 scale, with a tunnel diameter of about 2 m (6 feet). This was the smallest size that could be manufactured with cast iron while maintaining true proportionality of all dimensions, particularly the skin of the segment (established from discussions between Morgan Sindall and their tunnel lining manufacturers). The linings were manufactured to have a composition that matches that of existing linings - the intention being to replicate as closely as possible the existing linings.

Traditional linings were typically made from grade 10 GCI and a foundry process was followed to match this as closely as possible.

Tests were performed on a single ring comprising six GCI segments (with an arc length of ~1m), bolted together and placed on the structural floor in the Structures Laboratory so that the radial plane of the ring was horizontal. It was surrounded by a steel reaction ring. Radial loading of the ring was via eighteen computer-controlled actuators installed between the reaction ring and extrados of the test lining so that full control was achieved. This would not be possible if loading was through a soil medium. Loading of the ring was by a combination of load and displacement control applied so as to deform the ring into an elliptical form similar to that observed in-situ. Each segment was loaded on its extrados by three actuators spaced at 20° intervals via connections to spreader pads to distribute the loads from the actuators evenly over the full width of the segment. The force applied to each spreader pad was monitored using load cells so that an estimation of the stress distribution around the extrados of the ring could be made.

Around the intrados of the lining displacement transducers were set up directly in line with each actuator to monitor segment deformations. For the first stage of each test all actuators were set to apply the same load to the spreader pads such that the ring was under hoop stress but remained circular. Using computer feedback-control software, very small incremental changes in the diameter along one axis was applied until the ring distorted into the form of an ellipse. Very careful thought and experimentation with the computer-feedback control was needed in choosing the appropriate sequence of loading to deform the ring to the required elliptical shape. A key point was to impose only very small displacements at each stage. The research control software TRIAX, initially developed at Imperial College by Prof. David Toll was used as it is very versatile and the research group have extensive knowledge and experience with it conducting tests on soil samples in the Soils Laboratory. Additionally there are very strong links between Prof. Toll (now at Durham University) and the group and if necessary modifications to the software could be made. The actuators were individually controlled using stepper motors controlling manostats and hence air pressures (fed from a central compressor) which in turn was transmitted to hydraulic pressure, through air-water interfaces, to achieve fine control of the water-filled actuators.

The idea was to replicate typical shapes observed in the field, as determined from the field measurements (and existing records) or shapes observed from the numerical analyses. Key variables changed between tests were the tension in the bolts, inclusion of grommets and initial hoop stress.

The main variables monitored during loading included changes to the internal tunnel radius at various locations. Forces in the bolts connecting segments were measured using instrumented bolts. Stresses and bending moments within the segments themselves were determined from conventional strain gauge measurements at the intrados and extrados of each segment. This technique is both relatively cheap and reliable and bending moments can be determined indirectly from these measurements in conjunction with the deformed shape of the ring (i.e. from the radial displacement transducers). During the course of the loading, output from all transducers was automatically data-logged. The local

displacements at the joint locations were monitored using linear variable differential transformer (LVDTs) transducers.

The philosophy was that provided the displacements induced did not generate stresses that exceeded the elastic limit, the exact manner in which the ring was loaded was not important. With such a versatile laboratory load-displacement control set-up it was possible to perform a parametric study to investigate realistically, using real joints and observing the mechanical response of the ring, various 'patterns' of ring deformation.

The main deliverables from this component of research were:

- Design and set-up all necessary loading and monitoring components for the test ring.
- Trial and select appropriate sequence of loading to deform the ring to the required elliptical shapes.
- Perform parametric study at low stress levels to determine the influence of hoop stress, bolt preload and inclusion of grommets to the response of the bolted segmental ring.
- Comment on the validity of the current tunnel lining assessment methods.
- Estimate the onset of joint opening as a guide to the onset of reduction in the tunnel lining stiffness from that of a continuous ring.
- Comment on the relationship between ground stresses and tunnel shapes.
- Provide clear objectives for the limited number of future tests taking the GCI test ring to plastic deformation and ultimately to failure.

1.3.2 Field measurements and monitoring

The laboratory work was supplemented by in-situ monitoring of existing tunnels.

At the first site the deformation of a small section of lining that was constructed in a frame above ground was monitored. The aim of this exercise was to find out the magnitude of distortion that is possible in tunnel rings due to initial out-of-built and self-weight. The subsequent exercises focussed on tunnels affected by adjacent ground excavations. Two different sites were monitored. The monitoring in the Northern Line station tunnel at Tottenham Court Road aimed to deduce the current stresses in existing linings by measuring the unloading strains when the tunnel lining segments were removed from the ground. The monitoring in the Central Line running tunnel east of Lancaster Gate station aimed to deduce the *changes* in in-situ lining stresses as a result of adjacent Crossrail tunnel excavations and relate the changes in stress to the measured deformations of the existing tunnel. Attempts were made to capture any joint movements in the diametral plane of the instrumented tunnel rings.

Deformation was also measured longitudinally within the tunnel. Once again attempts were made to monitor segment connections to assess whether the majority of the overall deformation takes place at the segment junctions. This component of work was carried out by a research team from ETH Zurich under the supervision of Professor Sasha Puzrin, allowing them to test their state-of-the-art monitoring systems using optical fibre.

1.3.3 Numerical analysis

While the numerical analysis was under the remit of another PhD (Avgerinos, 2014) and a post-doctoral researcher, it is worth mentioning here as ultimately it is envisaged that numerical analysis

will draw together all the findings such that comprehensive recommendations for the assessment of existing tunnel linings can be produced by the overall research team.

The Imperial College Finite Element Program (ICFEP) was being used to perform the numerical analyses. This program has a long pedigree and has been used extensively to analyse soil-structure interaction problems involving excavations and tunnels. It can analyse two- and three-dimensional geometries, has an extensive library of advanced constitutive models for both soils and structural materials and has a number of specialised boundary condition options for performing analyses of tunnel construction.

Numerical analysis was a key tool in this overall research project. The analysis aided the design of the laboratory set-up as different loading and boundary conditions could be simulated and assessed. This saved valuable time and minimised the need for trial and error in the laboratory. Numerical analysis was used to simulate the conditions around existing tunnels to provide an insight into the transfer of ground stresses to the tunnel lining and the resulting deformed tunnel shapes.

Numerical modelling required input from the laboratory and field investigations. Numerical modelling allowed site-specific input parameters to be used and the laboratory and field investigations provided the necessary measurements at medium to full scale with which to calibrate and validate constitutive models and the analyses themselves. The aim was to become confident that the numerical simulations could replicate what was measured in the laboratory under controlled conditions and also capture what was measured in the field. It could then be used to extend and supplement the limited database of laboratory and field measurements to predict tunnel response.

1.4 Layout of the thesis

Chapter 2 summarises the current knowledge on grey cast iron and its use in the LU network. The historical measurements of ground stresses acting on tunnel linings and the deformation of tunnels over time after construction are presented. Chapter 2 also discusses the current methods used in the industry for the assessment of tunnel linings and the assumptions and limitations in the current methods. Finally, two case studies in Chapter 2 highlight the current lack of consistency in the mitigating measures adopted to protect existing tunnel linings from adjacent excavations.

The in-situ tunnel monitoring work is described in Chapter 3. As mentioned, three field-based instrumentation and monitoring exercises are carried out by the author and the research team. Chapter 3 gives the details of the types of instruments used in the field, the installation of the instruments, data collection and subsequent analysis. Reference is made to the work carried out by ETH Zurich and the importance of their preliminary findings is highlighted.

In Chapter 4 the details of cast iron segments manufactured specifically for the experimental investigations are described. Chapter 4 discusses the results of the tensile tests performed on the cast iron specimens and provides the mechanical properties used in the design of the test rig and in the analysis of laboratory tests. Chapter 4 also compares the properties of wrought iron specimens, machined from wrought iron bolts recovered from LU tunnels, with the properties of mild steel and discusses the validity of using mild steel bolts in the experimental work.

Chapter 5 revisits some laboratory investigations carried out in 1970s studying the behaviour of a bolted joint under zero hoop stress. The previous laboratory work is reviewed and the new experiments are carried out in conjunction with three-dimensional finite element analysis. The findings provide great insight to the behaviour of a bolted joint and all the results are presented in detail. The knowledge gained from this statically determinate problem is used to guide the planning and analysis of the full-ring investigations.

The design of the full-ring experimental set-up is explained in Chapter 6. All the structural components and all the instrumentation are described. Furthermore, this chapter explains the rationale and limitations behind the loading regime and presents the adopted sequence of loading.

The results of the full-ring parametric experiments are presented in Chapter 7. Both the global bolted segmental ring behaviour and the local response at the joint locations are studied in detail. The influence of hoop stress, bolt preload and the presence of grommets on the ring behaviour at small distortion is discussed. The validity of the assessment methods favoured by the industry is discussed in light of the laboratory findings.

Chapter 8 is the final chapter in this thesis, where the main conclusions from the research are drawn and the author's recommendations for future research are outlined in detail.

2 Literature review

2.1 Use of grey cast iron as a tunnel lining material

Since the successful use of grey cast iron as a tunnel lining material in 1869 by Greathead for the Tower subway, a considerable number of shield-driven tunnels constructed in Great Britain, in America and in Europe were lined with cast iron. From 1894 to 1937 bolted grey iron linings were always specified for the deep tunnels in the London Underground network (Groves, 1943).

The size and weight, as well as the thickness of the segments forming the tunnel rings were determined more by practical considerations of facility of casting and of placement, than on any theoretical grounds (Copperthwaite, 1906).

Refer to Figure 2-1 for terms relating to positions in a tunnel lining.

Cast iron segments were bolted together in rings to form a cylinder. Since the longitudinal flanges were machined there should be only small bending moments associated with the tightening of the bolts. However, 'birdmouthing' occurs in practice due to incorrect ring segment installation and considerable bending moments may be built into the flanges. During the later stages of erection, bolting, grouting and caulking, stresses may be built into the lining thus affecting the stress distribution within the ring. Therefore it has been difficult to correlate stresses measured in a lining with those calculated by theoretical methods (Craig and Muir Wood, 1978).

The first tunnels constructed with cast iron linings were built with continuous longitudinal joints but from the early 1900's it was general practice to stagger, or break, joints, i.e., rolling one ring compared with the next. While the method was introduced to help the building of the lining, the bending stiffness in the longitudinal direction was thought to be enhanced as a consequence (Craig and Muir Wood, 1978).

When a ring of cast iron lining was being erected within a shield in the tunnel there was no space available in front of the ring. The erection space was limited to the width of the ring and each segment was moved radially into place. Therefore the space left for the key segment must be at least as wide at the inside as the outside of the lining. Generally for ease of construction it was made wider at the inside. The longitudinal flanges of the key tapered towards each other at the outside. The two adjacent segments had one longitudinal flange conforming to the key and one standard radial longitudinal flange. The remaining segments had radial longitudinal flanges. To avoid confusion, the segments adjacent to the key were made distinguishably shorter. For going around horizontal and vertical curves, tapered rings were used with the planes forming a slight angle, so that the width of the rings varied from point to point.

Rust caulking was the principle of introducing into a groove finely divided and tightly packed steel or iron mixed with some compound that produced a rapid and thorough rusting of the metal. Rusting caused the metal particles to form a solid mass which was watertight and which pressed against the sides and bottom of the groove. Bolt holes, being outside the caulking zone, needed to be

waterproofed individually. This was usually done by placing a grommet of hemp, dipped in red lead and oil, under the washers at the head and nuts of the bolts (Hewitt and Johannesson, 1922).

According to Bates (1984), wrought iron, being very costly to produce, was replaced by steel from about 1850 and very little wrought iron was used after 1890. However, investigations by Tube Lines (2008) showed that a large number of bolts recovered from the LU tunnels were wrought iron bolts.

From 1937 other types of linings including concrete bolted linings, expanded concrete linings and expanded grey iron linings were introduced. However, linings for escalators, machine chambers, concourse and station tunnels were still constructed using bolted grey cast iron. Bolted grey cast iron was also the lining used in water-bearing non-cohesive ground where adequate waterproofing of the tunnel was essential (Craig and Muir Wood, 1978).

In 1947 London Transport used the first reinforced concrete bolted lining on the Central Line extension (Winterton, 1999). The techniques for the manufacture of ductile cast iron were also first developed in 1947 when the flake graphite form of grey cast iron was changed to spheroidal graphite (Lyons and Reed, 1974). The first use of bolted spheroidal graphite iron (SGI) lining was in June 1968 for a pilot tunnel later enlarged into a crossover tunnel for the Victoria Line extension to Brixton (Craig and Muir Wood, 1978).

2.2 Grey cast iron material properties

This section on the structural and mechanical properties of grey cast iron is largely based on the work by Angus (1976). Other references are stated as necessary.

The constituents of graphite and phosphorus are responsible for the main characteristics of cast iron. The effect of graphite on the stress-strain response of cast iron is substantially that of a void. A steel matrix carrying a substantial number of voids distributed as graphite in cast iron would show very similar stress-strain characteristics. Phosphorus tends to segregate and form brittle networks and its content is generally kept as low as possible. Grey cast iron exhibits brittle failure in tension.

In tension, graphite acts as a discontinuity in the matrix. The tensile strength of unalloyed cast iron ranges from 93 to 309 MPa, depending both upon the section size and the amount of free graphite present. Usually cast iron is composed structurally of ferrite, pearlite, cementite, graphite and phosphide eutectic (a mixture of ferrite and iron phosphide which solidifies at relatively low temperatures of around 950°C).

The mechanical strength and hardness for castings are higher at the more rapidly cooled exterior than in the more slowly cooled centre. The slower cooling parts of a casting are under residual tensile stress whereas the faster cooling parts are under residual compressive stress at room temperature. Grey irons maintain their room temperature values of tensile properties up to 400°C. Above this temperature the tensile strength decreases rapidly, and the elongation at failure increases to 2% at 600°C (Scholes, 1979), compared with failure strains below 1% for grey cast irons at lower temperatures.

Standards specify grey iron by reference to its strength when cast into test bars of fixed diameter and not by reference to the strength of the casting itself. The tensile test is regarded as the standard test by which cast iron is specified. The tensile strength is the stress required to pull apart a test piece by an axially applied load. The strength of grey cast iron in BS 1452:1961 is defined in relation to five standard diameters of as-cast bars from which standard tensile specimens can be machined. Each diameter is regarded as representative of a range of section thicknesses. See Table 2-1.

Nominal diameter of test bar as-cast in (mm)	Main cross-sectional thickness of casting represented in (mm)	Minimum tensile strength ton/in ² (N/mm ²)						
		Grade 10	Grade 12	Grade 14	Grade 17	Grade 20	Grade 23	Grade 26
0.6 (15)	Not exceeding $\frac{3}{8}$ (10)	11.0 (170)	13.0 (201)	16.0 (247)	19.0 (293)	22.0 (340)	25.0 (386)	28.0 (432)
0.875 (22)	Over $\frac{3}{8}$ and not exceeding $\frac{1}{2}$ (10–19)	10.5 (162)	12.5 (193)	15.0 (232)	18.0 (278)	21.0 (324)	24.0 (371)	27.0 (417)
1.2 (30)	Over $\frac{1}{2}$ and not exceeding $1\frac{1}{8}$ (19–29)	10.0 (154)	12.0 (185)	14.0 (216)	17.0 (263)	20.0 (309)	23.0 (355)	26.0 (402)
1.6 (41)	Over $1\frac{1}{8}$ and not exceeding $1\frac{3}{4}$ (29–41)	9.5 (147)	11.5 (178)	13.5 (208)	16.0 (247)	19.0 (293)	22.0 (340)	25.0 (386)
2.1 (53)	Over $1\frac{3}{4}$ (41)	9.0 (139)	11.0 (170)	13.0 (201)	15.0 (232)	18.0 (278)	21.0 (324)	24.0 (371)

Table 2-1. Tensile Strength of grey cast iron (BS 1452:1961).

It is common knowledge in the foundry industry that test bar results cannot be used to give the strength of the casting itself where section thicknesses and cooling rates may vary appreciably. The notional specifications therefore merely define the metal in the ladle and do not give a direct indication of the strength of the casting itself. However, when the strength of the metal cast under standard conditions is known, it becomes possible to estimate with some degree of accuracy the probable tensile strength to be found in a grey iron casting of a given thickness.

A statistical survey of a large number of industrially cast grey iron test bars of different sizes has been made (Angus, 1976), and relationships were established between the strength of various section sizes and the standard bar as shown in Figure 2-2. There is a falling off in strength with increasing test bar size. The line for each grade is the centre zone of uncertainty amounting to ± 15 to 23 MPa due to variation in grade from differences in composition, pouring technique etc. The total axial strain of grey iron tensile test pieces at fracture usually varies between 0.5% for high tensile iron and 0.75% for low tensile iron. The yield point after which non-elastic strain occurs varies between 0.2% for high tensile iron and 0.6% for low tensile irons.

Compression tests were carried out on cast iron specimens with a length:diameter ratio of 2:1. The compression strength of grey cast iron is approximately three to four times its tensile strength. Grey cast iron fractures at its maximum compressive load (Angus, 1976).

Conventionally the stress-strain behaviour is plotted as per Figure 2-3a and b, where the total strain is shown along with the permanent and recoverable strains. The permanent strain is determined at several points by returning the load to zero, measuring the permanent set and plotting the curve of the permanent strain. The curve of the recoverable strain is the difference between the total and the permanent strain. The elastic modulus is obtained by plotting the slope of the curve of the recoverable strain against stress as shown in Figure 2-3c and extrapolating to zero stress. The value of E at zero stress is E_0 , whereas secant modulus is denoted E_x , with x giving the stress level. Figure 2-4 indicates some determined values in tension of E_0 and E_x for various grades of iron which can be regarded as typical.

As shown in Figure 2-5a, the behaviour of grey cast iron in tension begins to deviate from linear elasticity at low stress levels. The cavities which contain the graphite flakes open up slightly under tension and local plastic deformation of the matrix occur even at low applied stresses. The total tensile strain is made up of four components:

1. Elastic deformation of the matrix
2. Plastic deformation of the matrix at points of high stress
3. Recoverable strain due to the opening of the graphite cavities
4. Permanent strain associated with the opening of the cavities

On unloading, the elastic deformation of the matrix is recoverable, but partial closure of the graphite cavities adds a further recoverable component so that the recoverable strain is not a straight line, but curved. If the specimen is stabilised by repeated application of a load equal to or higher than that to be ultimately applied, a curve of recoverable strain is obtained that will apply to any lower load (Figure 2-6). These effects are small under normal service conditions if the working stress does not exceed 25% of the tensile strength.

In compression the strains are less affected by volume changes associated with graphite flakes and the curve of the recoverable strain is an approximate straight line at least up to 154-232 MPa. Within the normal stress range the modulus of elasticity in compression is approximately constant with increases in stress and its value equal to E_0 , as can be seen from Figure 2-5b (Scholes, 1979).

Poisson's ratio varies with stress and grade of iron. It tends to fall as the tensile stress increases, but remains constant in compression until high stresses are imposed – more than 185 MPa (12 ton/in²) – when the value will rise. Grey iron decreases in density under tensile stress owing to opening up of graphite cavities and this may also occur with graphite at a certain orientation even in compression. Figure 2-7a & Figure 2-7b show the variation of Poisson's ratio with stress for a BS1452:1961 Grade 14 iron.

Higher phosphorus irons show higher tensile strengths, greater hardness, lower total strain at failure and little change in E_0 values. Increase in phosphorus content of a grey iron from 0.2% to 1.0% increases the tensile strength in 30 mm bars by 15-31 MPa. The increase in tensile strength is accompanied by lower tensile strain to failure as phosphorus content increases. The curves of strength versus carbon equivalent values (see Note 1, Table 2-2) were originally based on test bars industrially produced before 1949 in which phosphorus values in Great Britain were predominantly above 0.5%. In the 1960s and 70s, the phosphorus values dropped below 0.5%.

The chemical composition of Victorian cast iron linings for LU tunnels was obtained from LU archive reports and is given in Table 2-2.

Main Component (%)	Literature survey by Thomas (1997) for Northern Line tunnel segments dated from 1890 to 1910	Test results reported by Fallon (1998) for six cast iron segments dated from 1900 to 1925	Test results reported by Tube Lines (2008) for LU Jubilee/Northern/Piccadilly lines segments dated from 1890 to 1986
Carbon	3.30 – 3.50	3.12 – 3.54	3.3
Silicon	1.00 – 1.50	1.65 – 3.08	2.3
Manganese	0.60 – 0.80	0.29 – 1.34	0.4
Sulphur	0.08 – 0.12	0.05 – 0.13	0.1
Phosphorus	0.30 – 0.60	1.07 – 1.59	1.4
CE ¹		4.47 – 4.66	4.5
Average Ultimate Tensile Strength ² (MPa)	125 – 185	142 – 175	155 (pre-1928 specimens) 190 (post-1928 specimen)
Average tensile strain at failure (%)			0.46

Table 2-2. Chemical composition of cast iron.

Notes:

1. $CarbonEquivalent, CE = TotalCarbon\% + \frac{Silicon\% + Phosphorus\%}{3}$.

2. For comparison, the ultimate tensile strength for grade 10 Cast Iron is 150 MPa (Gilbert, 1977)

2.3 Existing deformations of tunnel linings

2.3.1 Current distorted shapes of LU tunnels

The case studies reporting the development of tunnel lining distortion highlighted two main modes of deformation for tunnel rings. In this thesis, the distortion mode corresponding to a reduction in the diameter at tunnel axis level and increase in length from invert to crown will be termed ‘egging’. The distortion mode corresponding to an increase in the diameter at tunnel axis level and a reduction in vertical distance between the crown and invert will be termed ‘squatting’.

Circularity surveys using a Leica 3000 laser scanner were undertaken in 45 km of LU running tunnels in 2004 and 2005, which was approximately 25% of the total length of tube tunnels in the LU network. It was found that the tunnels had squatted, that is, the horizontal diameter is greater than the vertical diameter. Figure 2-8 shows the definition of the ovalisation parameter calculated using the major axis of the elliptical shape. It is the change in radius divided by the original radius.

Of all the rings surveyed, 92% had ovalisations less than 1%, leading to Tube Lines’ recommendation of assuming 1% ovalisation for tunnels in cohesive ground. The remaining 8% of rings surveyed consisted of linings at a greater depth than 40m, linings constructed in non-cohesive or mixed ground, and other linings with special features such as rebuilt crown. An ovalisation of 2% is more appropriate for these linings and vertical shafts (Tube Lines, 2007).

LU standard 1-055 (2007) nominated an ovalisation of 0.6% to be used in the initial analytical assessment should the actual ovalisation be unavailable.

Tube Lines used the laser scanner to take about 40-50 discrete measurements for one ring, and then best fitted the surveyed data to an imaginary circle using the least squares method. Surveys were taken at 10 m intervals for straight sections, and 5 m intervals for curved sections.

An example of survey results is given in Figure 2-9 for the Northern Line running tunnels between Tottenham Court Road and Leicester Square. Each red dot represents a survey point, with the black circle being the best fit circle and the dashed circles representing different magnitudes of ovalisation.

In the report 'Assessment of ground movement effects: LU/05 Central Line east of Lancaster Gate' produced by Framework Design Consultant Contract 122 (C122) for Crossrail Ltd. (2011), the existing condition of the portion of Central Line within the zone of influence of the new Crossrail tunnels was described in detail. The Zone of Influence (ZOI) was taken as the estimated 1 mm vertical ground movement contour at asset level. For the eastbound running tunnel of the Central Line, rings 260 to 480 lie within this zone. C122 estimated the existing ovalisation for the 11' 8¼" internal diameter sections based on data processed at 10 m intervals. An average of 0.6% ovalisation was assessed for the Central Line eastbound tunnel based on the survey data. The data showed that the tunnel was generally squatting (i.e. the distance between crown and invert was less than the horizontal diameter of the tunnel at axis level) prior to the arrival of the Crossrail tunnel boring machines (TBM). The instrumentation and monitoring of this section of Central Line are discussed in Chapter 3.

2.3.2 Development of lining distortion over time

Tattersall, Wakeling and Ward (1955) described an investigation into the behaviour of a hydraulic pressure tunnel at Ashford Common in Middlesex, UK. The tunnel was lined with DonSeg concrete segments giving an internal diameter of 2.54 m and an external diameter of 2.84 m. The tunnel axis level was approximately 27 m below ground level. Ground investigation in the area indicated the London Clay layer extended from approximately 5 m to 6 m below ground to over 100 m below ground level.

Diametral changes were measured using a micrometer screwed onto a steel gas pipe. Diametral measurements on three rings indicated that immediately after construction the tunnel rings 'egged', with the vertical diameters being greater than horizontal diameters by about 12 mm.

Ward and Chaplin (1957) reported the measurement of the changes in diameter in several London Underground tunnel rings that were unloaded due to removal from the ground. The tunnels were around 50 years old. New construction required the complete removal of several rings in the existing tunnels. The tunnels were at 15.2 m to 33.5 m below ground level.

Measurements of changes in diameter associated with the removal of overburden load were made in the vertical and horizontal direction with a steel tape with an accuracy of 1/32 inch or 0.8 mm. At one site both the vertical and horizontal diameters increased upon unloading. At another site the vertical diameter decreased and the horizontal diameter increased. At a site where the twin tunnels were less than one diameter apart, the vertical diameter increased and horizontal diameter reduced.

Ward and Thomas (1965) presented the gradual development of earth loading, the bending moments in tunnel segments and the associated change in the shape of the linings up to six years following construction of two cast-iron lined tunnels and one precast concrete tunnel in London Clay.

One of the cast iron tunnels was constructed in 1957 as part of the Post Office railway system. It had an internal diameter of 2.1 m, with six segments bolted together to form one ring. The other cast iron

tunnel, built in 1960 as part of the Victoria Line, was of 3.9 m internal diameter, and also consisted of six segments. The segments were thrust directly against the clay by jacks inserted at the segment joints at approximately 60° on either side of the invert. Metal wedges were inserted into the joints adjacent to the jacks, which were then removed. The segments were not bolted. The precast concrete tunnel also formed a section of the Victoria Line, and it was formed by expanding the ring directly against the clay by means of a wedge. The concrete tunnel had an internal diameter of 3.8 m.

Changes in the vertical and horizontal diameters of the tunnels were measured using screw micrometer rods. Measurements could be reproduced to ± 0.0254 mm ($\approx \pm 10^{-3}$ inch).

After construction, the smaller diameter Post Office tunnel had squatted by 12.7 mm while the Victoria Line tunnel was approximately circular. The paper did not clarify whether the squat was deduced from vertical or horizontal measurements. Subsequent measurements showed that the increase in horizontal diameters was less than the decrease in the vertical diameters. During the second week after construction the change of diameters was up to 1 mm per week in the vertical direction and 0.5 mm per week in the horizontal direction for the larger tunnel. For the smaller tunnel the rate of change of diameters in the vertical and horizontal directions were 0.5 mm/week and 0.4 mm/week, respectively. The rates of change decreased with time but were still measurable six years after construction.

Thomas (1976) reported measurements taken in the Victoria Line extension to Brixton which took place between 1968 and 1970. Close to Brixton station, two 3.81 m internal diameter pilot tunnels were built to allow the construction of a 9 m internal diameter cross-over tunnel at a depth of 22 m below ground level. Both pilot tunnels were constructed from conventional GCI segments. However, in the first pilot tunnel, an experimental length of 30 rings was constructed using spheroidal graphite iron (SGI) segments. The first pilot tunnel had a total of three instrumented rings. One ring consisted of six segments of GCI with a key piece at the top (Ring One). The second ring consisted of 12 segments of SGI with a key piece at the top (Ring Two). Lastly, a ring consisting of 12 SGI segments of reduced stiffness plus key piece was instrumented (Ring Three). The stiffness was reduced by elongating the centre bolt hole of the circumferential flange to the tip of the flange.

Diametric changes were measured with a comparator consisting of a screw micrometer set in the end of a 3.81 m long tube.

Measurements of the horizontal diameters of Ring One and Ring Two immediately after construction and several days later showed that the horizontal diameters decreased. In the five weeks between grouting in the rings and the passing of the second pilot tunnel, the horizontal diameters increased and the vertical diameters decreased for each of the instrumented rings. The second tunnel passing the instrumented rings resulted in an increase of 10 mm in the horizontal diameter in Ring One and an increase of 15 mm in the horizontal diameter of Ring Two. The vertical diameters of Ring One and Ring Two both reduced by approximately 10 mm. In the five months following the passing of the second tunnel, squatting continued, but the vertical diameter was decreasing more than the horizontal diameter was increasing.

Attewell and El-Naga (1977) provided information on two 3.2 m diameter tunnels at depths of 13.4 m and 14.0 m. The tunnels were driven by hand using clay spades without a shield through stiff, stony, laminated clay and lined with bolted segmental concrete linings.

Diametral changes were measured in four directions using a 0.01 mm dial gauge fixed to a 50 mm diameter aluminium bar having a conical end for positive location in metal studs set on metal plates fixed in the concrete lining. The maximum squat (vertical shortening) of 0.041% was reached at 14 days after installation. The maximum horizontal increase of the tunnel was 0.027%, reached at 18 days after installation.

Nyren (1998) monitored the diametral changes of several rings of the eastbound and westbound Jubilee Line Extension running tunnels between Green Park and Westminster stations. Open face shields with mechanical backhoes were used to excavate tunnels of 4.85 m diameter which were lined with expanded segmental concrete linings. The westbound tunnel axis was approximately 31 m below the surface. The eastbound tunnel axis was 20.5 m below the surface and this tunnel was excavated eight months after the westbound tunnel.

Measurements of diametral changes were made using a digital tape extensometer with a steel tape. Installation of the monitoring points (eyebolts) for the westbound tunnel was carried out after the shield train had cleared the rings, which was about 210 days after the rings were erected. The first set of readings for the westbound tunnel was taken shortly after the installation of the eyebolts and this was used as the base readings. The two sets of measurements which were performed within 30 days after the base set showed increase in the horizontal span of less than 0.1 mm. The final set of measurements was taken approximately 400 days after the base readings were taken. The horizontal span was 4.8 mm to 5.6 mm longer than the base reading.

For the eastbound tunnel, measurements were made across the two specially designed instrumented rings. Vertical and horizontal base readings were taken 24 hours after the rings were installed. Within 12 days of the base readings, the horizontal diameter increased by 4.2 mm. This elongation increased to 11 mm after 120 days, and to between 17.2 mm and 20.4 mm after 1 year.

2.4 Earth pressure acting on tunnel lining

A summary of the historical tunnel lining stress measurements is given in Table 2-3. The main point to note is that for GCI bolted segmental tunnel linings, the stress acting on the lining deduced from the various measurement techniques expressed as a percentage of the overburden pressure ranged from 45% to over 100%. Refer to Section A in the Appendix for details.

Lining Type	Tunnel size (m)	Depth (m)	Soil type	Stress in lining as % of full overburden pressure	Time period between installation and measurement	Measurement technique	Reference	Location
Bolted Grey Cast Iron Segments	9.5	8.0 to 9.0 (below the river bed)	Hudson River Silt	72 -100	2 months	Hydraulic pressure gauges	Rapp and Baker (1936)	New York, USA
Bolted Grey Cast Iron Segments	3.9 O.D.	31.0 (crown)	(Blue) London Clay	100	14 days	Whittemore strain gauge	Skempton (1943)	UK
Bolted Grey Cast Iron Segments	7.6	30.5 (axis)	London Clay	60 - 100	3 months	Vibrating wire strain gauge	Cooling and Ward (1953)	UK
Donseg Concrete Segments	2.5 I.D.	27.0	London Clay	<100	1 year	Vibrating wire type load gauges	Tattersall, Wakeling and Ward (1955)	Ashford Common, UK
Bolted Grey Cast Iron Segments	3.8 to 6.9 O.D.	15.2 to 33.5	London Clay and Lambeth Group	~100 in general >100 when adjacent tunnel is less than 1 tunnel diameter away	50+ years	Vibrating wire strain gauge	Ward and Chaplin (1957)	UK
Bolted Grey Cast Iron Segments	2.1 I.D.	16.8 (crown)	London Clay	75	6 years	Vibrating wire strain gauge	Ward and Thomas (1965)	UK
Expanded Grey Cast Iron Segments	3.9 I.D.	24.1 (crown)	London Clay	100	3.5 years	Vibrating wire strain gauge	Ward and Thomas (1965)	Victoria Line, London, UK
Expanded Concrete Segments	3.8 I.D.	25.9 (crown)	London Clay	65	21 months	Vibrating wire strain gauge	Ward and Thomas (1965)	Victoria Line, London, UK
Expanded Concrete Segments	3.8 I.D.	16.8 to 39.2	London Clay	42 to 62	1 year	Photo-elastic stress gauges	Smith Osborne (1970)	Victoria Line, London, UK
Bolted Grey Cast Iron Segments	3.8 I.D.	22.0 (crown)	(Blue) London Clay	45	5 weeks	Vibrating wire strain gauge	Thomas (1976)	Victoria line extension to Brixton Station, London, UK
Bolted Spheroidal Graphite Iron Segments	3.8 I.D.	22.0 (crown)	(Blue) London Clay	45	5 weeks	Vibrating wire strain gauge	Thomas (1976)	Victoria line extension to Brixton Station, London, UK
Bolted Concrete Segments	3.2 O.D.	11.8 (crown)	Laminated Clay/Stony Clay	52 at crown 25 at knee	2-3 days	Hybrid electro-hydraulic earth pressure cell	Attewell and El-Naga (1977)	Tyneside Sewerage Scheme Tunnel, Howdon, UK
Bolted Concrete Segments	3.2 O.D.	12.4 (crown)	Laminated Clay/Stony Clay	53 at crown 43 at knee	8 days	Hybrid electro-hydraulic earth pressure cell	Attewell and El-Naga (1977)	Tyneside Sewerage Scheme Tunnel, Howdon, UK
Donseg and Wedge Block Concrete Segments	2.5 to 2.7 I.D.	19.0 to 54.0	London Clay	45 to 67	1 year	Vibrating wire type load gauges	Cooley (1982)	London, UK
Donseg and Wedge Block Concrete Segments	2.5 to 2.7 I.D.	19.0 to 54.0	London Clay	<89	17 years	Vibrating wire type load gauges	Cooley (1982)	London, UK
Bolted Grey Cast Iron Segments	3.8 I.D.	22.0 (crown)	(Blue) London Clay	45	15 years	Vibrating wire strain gauge	Thomas (1983)	Victoria line extension to Brixton Station, London, UK
Bolted Spheroidal Graphite Iron Segments	3.8 I.D.	22.0 (crown)	(Blue) London Clay	45	15 years	Vibrating wire strain gauge	Thomas (1983)	Victoria line extension to Brixton Station, London, UK
Wedge Block Concrete Tunnel	3.8 I.D.	20.0 (axis)	London Clay	40 at crown 60 at axis	19.5 years	Load cells	Barratt et al (1994)	Jubilee Line between Baker Street and Bond Street, London, UK
Wedge Block Concrete Tunnel	2.5m I.D.	15.5 (axis)	Oxford Clay	35 to 50	7.5 years	Load cells	Barratt et al (1994)	Oxford Trunk Outfall Sewer, Oxford, UK
Expanded Concrete Segment	4.85	20.5 (axis)	London Clay	45 at crown and invert 60 at axis	1 year	Load cells	Nyren (1998)	Jubilee Line Extension between Green Park and Westminster, London, UK
Expanded Concrete Segment	4.85	20.5 (axis)	London Clay	60 at crown 55 at invert 75 to 90 at axis	7 years	Load cells	Dimmock (2003)	Jubilee Line Extension between Green Park and Westminster, London, UK

Table 2-3. Historical lining stress measurements.

2.5 Deformations of tunnel linings due to adjacent excavations

Case studies where the deformations of existing tunnels due to adjacent excavations have been reviewed by the author include:

- The Jubilee Line extension project at London Bridge and Waterloo (JLE)
- The Heathrow Express project at the Central Terminal Area and at Terminal 4
- The Northern Line replacement project at Old Street
- The Thameslink2000 project
- Channel Tunnel Rail Link (CTRL)

However, the deformation of existing tunnels were influenced by factors such as the geometry of the crossing, the clearance between the new and existing tunnels, the type of construction used for the new tunnel to name but a few, these data are not immediately useful to the laboratory component of this research project. It is noted that the previous maximum recorded distortion in bolted cast iron tunnels due to new tunnel excavations is approximately $1000\mu\epsilon$ (0.1%) of diametral strain.

Cooper et al (2002) postulated that the relief of stress due to adjacent tunnelling causes ovality in the existing tunnel, with the axis of diametral increase corresponding to the direction of stress relief. There is also a concomitant diametral shortening across the orthogonal diameter of the tunnel. In other words, the diameter of greatest elongation rotates depending on the relative position of the face of the excavated tunnel.

In the JLE case study reported by Kimmance et al (1996), lining bolts of the existing Northern Line cast iron tunnel were tightened as a precautionary measure against deformations caused by the new JLE tunnels. Conversely, in the CTRL case study reported by Moss and Bowers (2006), the release of selected longitudinal joint bolts, in consultation with the metro tunnel owners, was undertaken as a mitigating measure.

Details of the case studies are given in Section B in the Appendix.

2.6 Previous laboratory tests

Thomas (1977) investigated the structural behaviour of bolted grey cast iron and spheroidal graphite iron (SGI) tunnel segments in the laboratory. The segments were dimensioned such that six GCI segments plus a key piece (see Figure 2-1) would join to form a ring of 3.83 m ID while 12 SGI segments plus a key piece would join to form a ring of 3.86 m ID. Since GCI behaves hysteretically upon loading while SGI exhibits little hysteresis, Thomas' experiments were mostly performed on the SGI segments.

In the bending tests the SGI segments were tested singly and loaded at two locations across the segment width as shown in Figure 2-10. Thomas found that a central circumferential strip along the skin hardly contributed to the stiffness of the SGI segments and concluded that strain gradients should be measured by gauges along the depth of the circumferential flanges. In Thomas' tests positive circumferential bending referred to the mode of bending tending to straighten the segment (Figure 2-11).

For the SGI segments tested, the flexural rigidity (EI) reduced as the applied bending moment increased. For GCI this could be attributed to decreasing modulus, E, as stress increases. However, for SGI this change in bending stiffness was more likely a result of the change in shape of the segment under load, since the material is only approximately twice as hysteretic as mild steel. The change in shape was a bulging out of the skin during positive bending (Figure 2-11) and a dishing in of the skin when bending was negative (Figure 2-12). Thomas used a rubber bag to investigate the effect of uniformly loading the skin of a segment as shown in Figure 2-13. The segments bent negatively with increasing pressure. Measurements showed that if a pressure was applied which caused dishing-in of the back during negative bending, the dishing is increased and the segment becomes even less stiff.

Bending tests on both the GCI and SGI segments were used to investigate the behaviour of the longitudinal joints in the absence of hoop force. Two half segments were bolted together at the longitudinal joint and the experimental setup is shown in Figure 2-14. At large deflections of about 1° to 2° at the joint, the joint of the GCI segment was approximately twice as stiff as the SGI segment. The SGI flanges were 20% longer and half as thick as those of the GCI segments. Tests showed that the flexibility of the longitudinal flanges reduced the efficacy of the bolts. When a pair of segments was bent in a manner tending to straighten them, hinging occurred at the joint not because of the bolts stretching but because the flange distorted. When a ring was subjected to hoop thrust, the load would be carried by the line of contact at the extrados. However if there was negative bending, the corners would come together and the line of contact would be at the bottom of the caulking groove. It was also shown that, if the three bolts were in line, the central bolt on the longitudinal flange carried substantially less loading than the two outside bolts when the segment was subjected to bending.

The two segment test conducted by Thomas (1977) on GCI segments will be revisited in Chapter 5 in greater detail. The main uncertainties with the tests completed in the 1970s are highlighted and the boundary and loading conditions for the test setup for this research are carefully described. An important point to make here is that this most recent two segment test setup at Imperial College was done in conjunction with the development of a three-dimensional finite element model using ICFEP. Post-doctoral researcher Dr. Aikaterini Tsiampousi was responsible for running the finite element analyses. The findings from the finite element analysis and the laboratory test results were used to refine and improve each other. Ultimately the finite element model was used to project the behaviour of the two segment test outside the range tested in the laboratory.

2.7 Tunnel lining assessment

Morgan (1961) set out the basis of the elastic continuum method for the analysis of a circular tunnel in elastic ground. Assuming that the circular lining distorted into an ellipse and neglecting shear stresses between the tunnel extrados and the ground, the maximum bending moments would occur at the tunnel axis and at the crown and invert and be given by the following equation:

$$M = \frac{3\delta EI}{a^2} \quad \text{Equation (1)}$$

Where δ = maximum distortion

a = tunnel radius

E = Young's modulus of the lining

I = second moment of area of lining per unit width of ring

For a tunnel in ground with mean elastic modulus E_c and Poisson's ratio ν , the symmetric loading in the ground set up by the distortion of the tunnel lining may be represented by a stress function (Timoshenko, 1934). Morgan (1961) used the stress function to formulate a direct relationship between the maximum bending moments in the lining and the assumed active loading:

$$M = \frac{p_0 a^2 EI (1 + \nu)}{6EI(1 + \nu) + 2a^3 E_c}, \text{ with } p_0 = p_{\text{vert}} - p_{\text{horz}}. \quad \text{Equation (2)}$$

Where p_{vert} = vertical stress acting on lining

p_{horz} = horizontal stress acting on lining

Where a continuous lining of constant section is used for a tunnel, Morgan suggested replacing E by

$$\frac{E}{(1 - \nu^2)} \text{ where } \nu \text{ is the Poisson's ratio for the lining.}$$

Muir Wood (1975) corrected a basic error in Morgan's method, the incorrect assumption that plane strain entailed plane stress, and also updated the method to reflect a 12 year gain in tunnelling experience. The maximum bending moment in the lining is related to the passive resistance of the ground according to:

$$M = \frac{p_0 \eta^2 r_0^2 EI (1 + \nu)(5 - 6\nu)}{6EI(1 + \nu)(5 - 6\nu) + 2\eta^3 r_0^3 E_c} \quad \text{Equation (3)}$$

Where η is the ratio of radius of lining centroid to extrados, and r_0 is the radius to lining extrados.

Muir Wood also allowed for reduction in lining stiffness for segmental linings. The effective second moment of area for the lining, I_e , is given by:

$$I_e = I_j + \left(\frac{4}{n}\right)^2 I \quad \text{Equation (4)}$$

Where I_j = second moment of area of the joint, and

n = number of segments (for $n > 4$; Muir Wood assumed that when $n < 4$ the joints have negligible effect on ring stiffness)

Curtis (1976) extended Muir Wood's method to consider the effects of frictional shear between the lining extrados and the ground, now termed 'full bond'. The maximum hoop load, N , is given by:

$$N = \frac{p_0 r_0}{2} \left/ \left\{ 1 + \left[\frac{2\nu Q}{(3-4\nu)(1+Q)} \right] \right\} \right. \quad \text{Equation (5)}$$

And the maximum bending moment is given by:

$$M = \frac{p_0 r_0}{4} \left/ \left\{ 1 + Q \left[\frac{3-2\nu}{3-4\nu} \right] \right\} \right. \quad \text{Equation (6)}$$

$$\text{Where } Q = \frac{E_c}{E} \frac{1}{1+\nu} \frac{r_0^3}{12I} \quad \text{Equation (7)}$$

Duddeck and Erdman (1985) compared the continuum model, the design model by Muir Wood and the bedded beam model. The first two models were intended for deep tunnels while the bedded beam approach was intended to be applied for shallow tunnels with small overburden loads. The continuum model was considered either with full bond between lining and ground or with tangential slip. The continuum model by Muir Wood assumed an elliptical deformation.

Duddeck and Erdman provided explicit formulae for a consistent continuum model, taking into account both the tangential ground stresses and the part of the radial deformations due to tangential stresses. Formulae for maximum bending moment in the lining, the hoop forces in the lining, and radial displacements were provided. They compared the bending moments obtained from the consistent continuum model with the results according to Muir Wood's assumption of elliptical deformation. Muir Wood's results were found to be much smaller, leading to their conclusion that the intuitive assumption of an elliptical deformation mode was an incomplete solution.

In order to obtain a squat in the tunnel, the ratio of horizontal stress to vertical stress in the continuum models needs to be less than 1.0, and a value of 0.7 is often adopted in the industry. However, profiles for the variation of the at rest earth pressure coefficient with depth produced for the Crossrail project for sites across Central London gave the ratio of horizontal stress to vertical stress of between 1.0 to 1.5 for depths of 20 m BGL to 30 m BGL (GCG, 2009a & 2009b). This depth range is most relevant to the bolted segmental cast iron tunnels in the LU network. The inconsistency between the deformed shape of the existing tunnels and the ground pressures is well known in the industry but remains unaddressed.

In 1999 Haswell was commissioned by LU to undertake an assessment of parameters for estimating stresses in existing tunnel linings (Haswell, 1999). The assessment was to be based on results from finite element studies undertaken by GCG (1999) using the finite element package developed at Imperial College by Prof. David Potts, ICFEP. GCG's study was intended to consider the effects of a potential reduction in lining stiffness due to articulation between lining segments under specified loading conditions. Ideally modelling of the cast iron linings used in the LU tunnels would incorporate three-dimensional effects of bolt action on the lining structure and the non-linearity of the ground

stiffness and the lining structure stiffness under load. However, in the study two-dimensional methods incorporating a number of simplifying assumptions were employed.

The ICFEP model was modified to simulate the effects of hoop load acting at an eccentricity to represent the capacity of the joint under the action of both hoop load and bending moment. An additional modification allows an appreciation of the effects of increased capacity afforded by bolt tension.

Where bending moments at the joints resulting from continuous (i.e., not segmented) ring analyses are not greater than the moment capacity of the joints, based on the associated hoop loads, articulation was deemed not to occur. Consequently the assumption of continuous ring behaviour applied and there was no joint rotation that would reduce the flexural stiffness of the lining. Conversely, if the bending moment from the continuous ring analyses was greater than the moment capacity of one or more joints, based on associated hoop load, articulation was assumed and the flexural stiffness of the ring was reduced from that of a continuous ring.

Where joint articulation occurred, the circular lining was assumed to deform into an ellipse and Morgan's equation (1961) was used to relate the bending moment from the finite element analysis to the apparent stiffness of the articulated lining:

$$EI_{reduced} = \frac{a^2 M_{FE}}{3\delta} \quad \text{Equation (8)}$$

Where M_{FE} = maximum bending moment from finite element (FE) analysis
 a = internal radius
 δ = 0.5×total deflection
 E = Young's modulus for the ring material
 $I_{reduced}$ = apparent I from articulated ring under distortional loading

Haswell (1999) then proposed an articulation factor, f_a , to apply to the second moment of area of the ring section for analysis by the closed-form mathematical solution of Muir Wood (1975) or Curtis (1976) which was given as:

$$f_a = \frac{EI_{reduced}}{EI} \quad \text{Equation (9)}$$

Using the results from the limited number of FE analysis, f_a for use in the Muir Wood/Curtis analyses should be restricted to 0.6 to 0.7. However, it was noted that the number of cases investigated where there was joint articulation was limited.

In the industry, whether the bending moment is initially estimated using distortion or an assumption of ground stresses, the flexural rigidity of the bolted ring is reduced by adopting Muir Wood's (1975) reduction formula for the second moment of area for the ring to take into account the presence of the joints. The validity of the reduction formula for cast iron bolted segmental lining is yet to be proven experimentally. In LU standard 1-055 (2007), the reduction equation by Muir Wood (1975) was simplified to:

$$I_e = \left(\frac{4}{n}\right)^2 I \quad \text{Equation (10)}$$

This means that for a typical running tunnel ring consisting of six segments plus a key piece, $n=7$ and the effective second moment of area used to compute bending moment is 33% of the original second moment of area. The laboratory experiments described in Chapter 6 and 7 are designed to measure the magnitude of reduction in stiffness in a bolted segmental lining when compared to a continuous lining of the same cross section.

In 2002, GCG (2002) was commissioned to produce a methodology for the assessment of running tunnels linings used in the LU network specifically for bolted cast iron linings. GCG (2002) noted that simple analytical methods relying on elastic closed-form solutions, such as Muir Wood/Curtis method, were not applicable to most of the situations encountered across the Underground network. It was not possible to develop simple correlations between the predictions made using the Muir Wood/Curtis method and those made using a non-linear elastic perfectly plastic finite element approach.

The method adopted by GCG (2002) to make predictions of stresses in linings was to use finite element models with non-linear elastic plastic soil behaviour and tunnel lining models which allow for joint articulation. Resistance to joint rotation associated with joint geometry in conjunction with hoop load is accounted for in the finite element model. Resistance to joint rotation associated with bolt tension draws on the laboratory test results from Thomas (1976) where the joint was subjected to flexure, and no hoop load. Thomas (1976) measured the joint rotation with increasing applied bending moment. When joint articulation occurred in the FE analyses, the bending moment transferred across the joint was subsequently enhanced by an amount corresponding to the joint rotation.

334 FE analyses were undertaken to generate a database of the likely stresses in tunnel linings. GCG (2002) produced figures of bending moments and associated hoop stresses plotted against the ratio of ground cover to separation between twin tunnels, with the intention that the database would be useful for calibration of future assessments.

Tube Lines (2007) reported findings from 3D structural modelling of a cast iron ring using STRAND7. The ring was longitudinally restrained at the rear face of the segments, and supported externally by no tension springs of stiffness E/R , where E is the Young's modulus of the ground and R is the tunnel radius. K_0 was assumed to be 0.7 and the longitudinal joints were filled with a thin layer of "no-tension" cast iron, so that the longitudinal joints could open under increasing load.

The analyses indicated that the ground load acting on the back of the lining caused a convex deflection (Figure 2-12) to occur to the inside of the skin, in turn causing the circumferential flanges to splay outwards slightly (possible due to a deep caulking groove). The convex deflection of the skin led to increased compression in the centre of the skin. Stresses at the junction between the longitudinal flanges and the skin are lower than in the centre. The stresses between joints are transmitted through solid contact between skin and skin and circumferential flange to circumferential flange. The longitudinal flanges did not transmit compression, and were in fact slightly in tension due to the circumferential flange distortion.

One aim of the research is to investigate independently, under carefully controlled conditions, the validity of the methods currently adopted to estimate bending moments induced in segmental tunnel linings. Because of the possibility to control and measure the loading applied onto the test ring in the laboratory environment, it was considered to be more appropriate and accurate to analyse the results of the full-ring test using Castigliano's second theorem, a set of linear elastic solutions. Formulae for the calculation of hoop stress, bending moments and deflections are available for a continuous circular ring under any number of equal radial forces equally spaced (Young and Budynas, 2002). By assuming linear elastic behaviour, superposition could be used to cover the loading conditions imposed in the laboratory tests.

2.8 Discussion

The literature review on the historical use of grey cast iron as a tunnel lining material provided valuable input in the planning of the laboratory tests described in Chapters 5 and 6. Considerations were given to ensure the experimental tunnel ring reflected as much as practicable the character of a tunnel ring in-situ. The experimental segments were manufactured to be at half the scale of a typical ordinary segment used in the Central Line running tunnel with an outer diameter of 12'6". This decision was based on discussions with tunnel lining manufacturers. A half-scale test ring was the smallest size that could be manufactured with cast iron while maintaining true proportionality of all dimensions, particularly the skin of the segment.

Details including caulking grooves and elliptical bolt holes were reproduced in the half-scale experimental segments. Grommets were specifically produced for the half-scale segment and the inclusion of grommets was a test variable. The test ring consisted of six ordinary segments bolted together, the key piece was omitted. This was done to reduce the complexity of the test system so that the behaviour of the joints around the ring would be more readily comparable.

While the literature review highlighted that in some tunnel sections the rings would be staggered in the longitudinal direction, the section of tunnel which was instrumented in Central Line and discussed in Chapter 3 consisted of key pieces all aligned along the tunnel crown.

The case studies reporting the development of tunnel lining distortion over time highlighted two main modes of deformation for tunnel rings. These two modes of distortion will be investigated in the full-ring laboratory test described in Chapter 6 & 7. It should be noted that in order to exercise the required level of control over the laboratory setup, 'egging' was obtained by loading the test ring along one axis, while 'squatting' was obtained by unloading the ring along the same axis. In real life, egging and squatting could be a result of a myriad of loading combinations.

Historically, between 45% and 100% of the overburden pressure has been reported to act on bolted grey cast iron tunnel linings. The field instrumentation exercise conducted at Tottenham Court Road was another attempt to measure the magnitude of overburden acting on the tunnel lining. This is described in Chapter 3. In the laboratory tests, the test ring was subjected to different magnitudes of hoop stress before the distortion was imposed with the aim to capture the ring behaviour under a range of overburden pressures.

The laboratory investigations by Thomas (1977) provided a good basis for planning the instrumentation for the laboratory tests conducted for this research and for understanding the three-dimensional behaviour of the segment as it undergoes bending.

The current practice in the industry with relation to the assessment of bending moment in tunnel linings is to use Morgan's equation (1961) to calculate the bending moment induced in the tunnel ring from a certain distortion. Some consultants would choose the complete continuum model and calculate the bending moment without presuming the elliptical distortion. However, the complete continuum model requires an assumption of the ratio of horizontal to vertical stress in the continuum, which in the case for tunnels, is soil. Currently there is a lack of understanding on the relationship between the stresses applied to the tunnel lining and its deformed shape, and this research will attempt to shed some light on the matter.

The literature review also highlighted the lack of experimental data to substantiate the use of Muir Wood's (1975) reduction formula for the second moment of area for a tunnel lining ring to take into account the presence of the joints. The laboratory experiments described in Chapter 6 and 7 are designed to measure the magnitude of reduction in stiffness in a bolted segmental lining when compared to a continuous lining of the same cross section.

The two case studies presented in Section 2.5 of this Chapter suggested that mitigating measures to reduce the impact of new tunnel excavations on existing cast iron tunnels have included loosening bolts in one case (Moss and Bowers, 2006), and tightening bolts in another case (Kimmance et al, 1996). Therefore, the laboratory experiments will also investigate the effect of different bolt loads on the behaviour of bolted segmental cast iron rings.

3 Field instrumentation

3.1 In-situ measurements at LU Acton Depot

Original grey cast iron segments from the City and South London Line were used to construct a section of tunnel of 12 rings above ground at the London Underground Acton Depot about ten years ago. The 5.25 km (3.26 mile) City and South London Railway, later becoming part of the Northern Line, was the world's first electric deep-level tube line. The City and South London Line opened in 1890 and ran from King William Street in the City to Stockwell.

The lower part of the tunnel ring was supported by a frame. Each ring consisted of four 'Ordinary' segments, two 'Top' segments and a key piece (Figure 3-1). Steel blocks were inserted between the segments in order to enlarge the 11'6" (3.505m) ID tunnel to 11'8¼" (3.562m) ID according to the inscription on the segments and the steel blocks. However, according to figures in the TRRL Supplementary Report (Craig and Muir Wood, 1978), the original City and South London Line was of 10'2" to 10'6" internal diameter.

The aim of this exercise was to give an indication of the magnitude of ovalisation that tunnel rings could display due to initial out-of-built and self-weight. Measurements of the segments, steel blocks and key piece were taken by the Imperial College research team on the 21st April 2010 using a tape measure to determine the internal circumference of the tunnel ring. The internal diameter was calculated from the measured internal circumference. The internal diameter was estimated to be 3.55 m. The distance from the top of the concrete infill to the soffit was measured using a Hilti Laser Range Meter with a precision of 1 mm. The distance from the top of the concrete infill to the invert was measured using a tape measure. The distortion in the vertical axis was estimated to be a shortening of 1.63%. The horizontal diameter was sighted in and three measurements were taken with the Hilti Laser Range Meter. The distortion in the horizontal axis was estimated to be a lengthening of 1.23%.

This exercise clearly showed that even in the absence of ground loading, bolted segmental tunnel linings could have a shape that is far from circular. The out-of-circularity could be a result of the self-weight of the segments, and the fact that the ring was expanded with steel blocks.

3.2 In-situ measurements at Tottenham Court Road platform tunnel

The redevelopment of Tottenham Court Road Underground Station started in 2011 as part of the Tube Upgrade Plan to improve and increase the capacity of the existing facility. The plan is to upgrade the station by 2016 to meet an estimated demand of more than 200,000 journeys per day once Crossrail is built.

During April to November 2011, major structural work was carried on the Northern Line platform tunnels as part of the station upgrade. This included removing tunnel lining segments on the platform side to allow construction of new cross passages to improve access to the platforms.

The works presented an opportunity for the Imperial College research team to trial their field instrumentation and to measure the hoop stress in real life tunnel segments.

The exercise involved:

- Making discrete measurements of the strains in selected platform side segments before and after the segments are removed from the ring.
- Making continuous measurements of the opening and closing of circumferential and longitudinal joints on trackside segments which are left in-situ and affected by adjacent excavations and tunnel segment removal.

The terms relating to tunnel lining are presented in Figure 2-1. Circumferential and longitudinal strains are defined as shown in Figure 3-2. Positive bending refers to the mode of bending having a straightening effect on the segment, and negative bending increases the curvature of the segment.

3.2.1 The instrumentation

DEMEC gauge

The DEMEC gauge is a portable device (Figure 3-3) which gives measurements of relative movement over a short gauge length. The studs were affixed to the surface of the structure using composite mortar. Once the studs were fixed and the mortar was set, the two points of the DEMEC gauge were carefully placed into the studs to take a zero reading. Further readings were taken subsequently to give the relative movement.

The selected gauge length was 150 mm. The gauge used was a digital gauge with a resolution of 0.001 mm. The accuracy under field conditions was given to be ± 25 to $50 \mu\epsilon$, compared to an accuracy of $\pm 5 \mu\epsilon$ under ideal laboratory conditions (Dunnicliff, 1988).

Electrical resistance strain gauges

350 Ω rosette type electrical resistance strain gauges with leadwires attached (Figure 3-4) were bonded directly onto the prepared cast iron surface using cyanoacrylate. Each strain gauge rosette was 16.5 mm by 20.3 mm in plan and less than 1 mm thick. After the strain gauges were installed, an initial reading was taken with a portable, battery-powered datalogger, the P3 strain indicator (Figure 3-5).

A quarter-bridge network with three wires was used in strain monitoring. This, along with the higher resistance 350 Ω resistance gauges, was selected to reduce lead wire effects. The strains caused by temperature change at the gauge were corrected during data processing. An AC excitation voltage was used for the portable strain indicator. Temperature on the surface of the cast iron segments was measured with a laser thermometer probe to facilitate adjustment of strain gauge measurements.

Displacement transducers

LVDT (Linear Variable Differential Transformer) type displacement transducers were used to measure movement across the circumferential and longitudinal joints of trackside segments. The displacement transducers were fixed onto the segments using a brass holder (Figure 3-6) which was initially positioned using magnets and then fixed in place using composite mortar.

The measurements were taken using mini-dataloggers. The mini-dataloggers were housed in a 120 mm x 120 mm x 90 mm ABS plastic box (Figure 3-7) which were fixed onto the skin of the trackside segments using composite mortar. There were two boxes in total, one box with two transducers set to

log at 3 minutes intervals and the other box with two transducers set to log at 10 minutes intervals. There was one temperature gauge with its own datalogger located in each plastic box (Figure 3-7).

3.2.2 The installation

Platform-side installations

Mechanical and electrical resistance strain gauges were installed on the top (Segment 2) and axis (Segment 1) segments of one ring on the northbound Northern Line platform tunnel. The positions of the two segments and the approximate locations of the strain gauges are shown on Figure 3-8 and Figure 3-9.

Figure 3-10 and Figure 3-11 show the as-built installations. The protective coating seen in the figures consisted of a base layer of Teflon to insulate all bare connections, a layer of butyl rubber sealant, a layer of neoprene rubber and a top layer of aluminium foil tape.

On the inside of the circumferential flange, a pair of DEMEC studs was placed close to the intrados where the maximum strains are expected. Another pair of DEMEC studs was placed close to the skin to allow bending strains to be calculated. Two pairs of DEMEC studs were located on the inside of the skin, to measure circumferential and longitudinal strains and allow comparison with the electrical strain gauge rosette.

Trackside installations

Figure 3-12 shows the location of the two data logging boxes on the trackside segment. Each box housed three data loggers, one for a temperature gauge and two for the LVDTs. One LVDT from each box was placed across the longitudinal joint and the other LVDT was placed across the circumferential joint. The loggers in Box 1 were set to log every three minutes and the loggers in Box 2 were set to log every 10 minutes (refer to Figure 3-12).

3.2.3 Estimation of unloading strain

It was assumed that when the platform tunnel segments were removed from the tunnel lining they were completely unloaded. The circumferential unloading strain assuming the full ground overburden was estimated to provide a comparison with the strains measured using the mechanical and electrical strain gauges.

The ground level was estimated to be at 25.0 m OD obtained from spot heights shown on drawing 1D0101-G0G00-G00-P-01016 Rev B from Geotechnical Sectional Interpretative Report 1&2: Royal Oak to Liverpool Street, Volume 3: Drawings, Report No. 1D0101-G0G00-00551 (GCG, 2009c).

The tunnel axis for the LUL Northern Line platform tunnel was estimated to be at -5.0 m OD from drawing HAG-N105-8742-TUN-D-SEC-5-03426 by Halcrow for the Tottenham Court Road Station Upgrade project. The axis level coincided with the London Clay and Lambeth Group interface (Halcrow, 2008).

Information on the geometry of the lining segments was obtained from LU Archive Drawing No. 146 for a 21'2½" (6.46 m) internal diameter tunnel. The section properties were calculated and used to estimate the tensile strain in the segment from the release of the hoop stress. The hoop load was

estimated assuming 30.0 m of overburden. The resulting strain was calculated to be about 400 $\mu\epsilon$ when the Young's modulus was taken to be 100 GPa for the grey cast iron.

3.2.4 Results from DEMEC measurements

Unloading strains due to removal of segments from ground

Using the baseline and post-segment-removal DEMEC measurements on spans 1, 2 and spans 5, 6 on the circumferential flanges (Figure 3-9), the axial and bending stresses due to unloading were estimated. In the calculations, the strain was assumed to vary linearly between spans 1 and 2 and spans 5 and 6. A constant modulus of 100 GPa was adopted. The results are presented in Table 3-1, along with the stress blocks, presented as variation in stress with distance away from the extrados, for each circumferential flange.

The measurements from all four circumferential flanges indicated circumferential tension and a positive bending moment upon unloading, which was expected since the segments would have experienced circumferential compression and negative bending moment while underground. The expected mode of deformation was based on Thomas' (Thomas, 1977) experiments as described in Chapter 2. Figure 2-12 illustrates the behaviour under negative bending. It was not clear why the circumferential tension was higher on span 1-2 for both segments. Assuming full overburden unloading, the change in axial force was estimated to be 940 kN. The change in axial force estimated using the DEMEC measurements ranged from 63% below to 30% above the value for the full overburden.

On the inside of the skin, DEMEC span 3 measured tensile circumferential strains from unloading, and DEMEC span 4 measured compressive longitudinal strains, for both segments. Referring to Thomas' test (1977) where a uniform pressure was applied on the skin of a segment (Figure 2-13) and negative bending was measured (Figure 2-12); the removal of a segment from the ground should measure positive bending. The strains measured from DEMEC spans 3 and 4 indicated the dishing outwards of the skin related to positive bending (Figure 2-11). The ratio of the longitudinal strain to circumferential strain was 1.2 for segment 1 and 1.0 for segment 2. Given that grey cast iron has a Poisson's ratio of approximate 0.26, this distortion of the skin was not due to Poisson's effect.

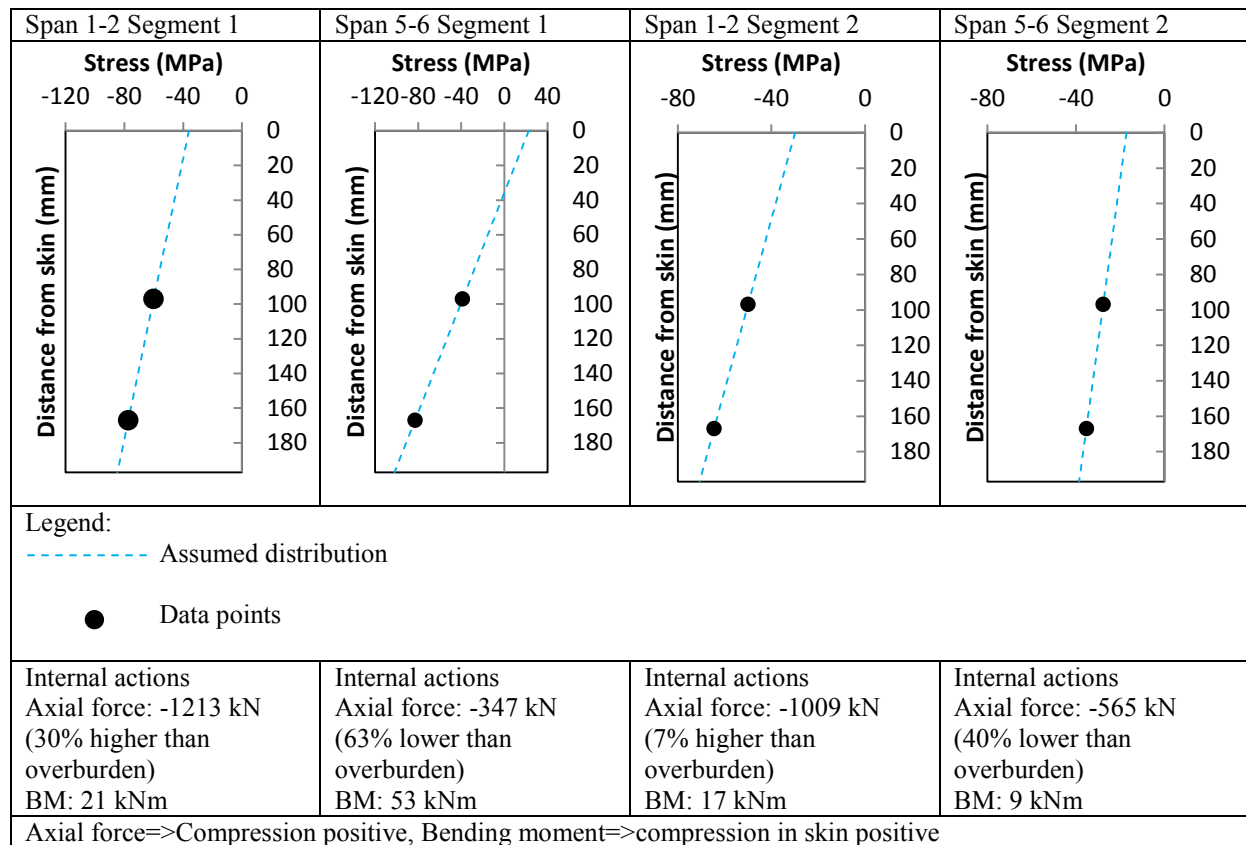


Table 3-1. DEMEC results for circumferential flanges.

Intermediate DEMEC measurements

A wire bracket was attached onto the skin to protect strain gauge rosettes from mechanical damage (Figure 3-13). Due to the presence of the wire bracket, only DEMEC spans 1 and 6 (see Figure 3-9) were available for intermediate monitoring between day of installation and the day of segment removal. The results are given in Figure 3-14.

For segment 1, span 1 and 6 measured increasing tensile strain, which is in line with the segment experiencing positive moment as the excavation works around the segment releases the ground loading. However, for segment 2, span 1 and 6 measured increasing compressive strains during the excavations works around the segment, while the segment was still part of the lining ring.

3.2.5 Results from strain gauge rosettes

Unloading strains due to removal of segments from ground

Using the baseline and post-removal strain measurements from each strain gauge in a rosette, the principal major and minor direct strains and the orientation of the principal direct strain directions on the surface of the cast iron were calculated for each rosette location. The calculations included corrections for strain gauge thermal output and gauge factor variation with temperature.

Refer to Figure 3-9 for the locations of the rosettes. The results for segment 1 are presented in Table 3-2. The results for segment 2 are presented in Table 3-3. Each plot shows the direction and magnitude of the major and minor principal direct strains on the surface of the cast iron. The y-axis

represents the circumferential direction and the x-axis represents the longitudinal direction as illustrated in Figure 3-2.

The strain gauges measured tensile strains slightly deviating from the circumferential direction and compressive strains slightly deviating from the longitudinal direction. This is consistent with measurements from DEMEC spans 3 and 4. The circumferential strain and longitudinal strain at the locations of rosette 2 and rosette 4 have been resolved from the major and minor principal direct strains to allow comparison with the DEMEC measurements. This is given in Table 3-4.

From Table 3-4, it can be seen that there was less than 5% difference in the circumferential unloading tensile strains measured using the electrical and mechanical strain gauges. However, the difference between the longitudinal strains measured by the two instruments differed by well over 20%. This larger difference corroborates with the positive bending mode of distortion the segment was expected to experience upon unloading, with the skin dishing radially outwards, compressive strains were expected on the inside of the skin in the longitudinal direction. Since the DEMEC instrument had a longer gauge length compared with the rosette, the compressive strain measured by the DEMEC was also larger because the chord length, rather than the arc length, was measured. In the circumferential direction, axial tensile strain dominated over bending strains, and the difference in gauge lengths had lesser effects on the measured strains.

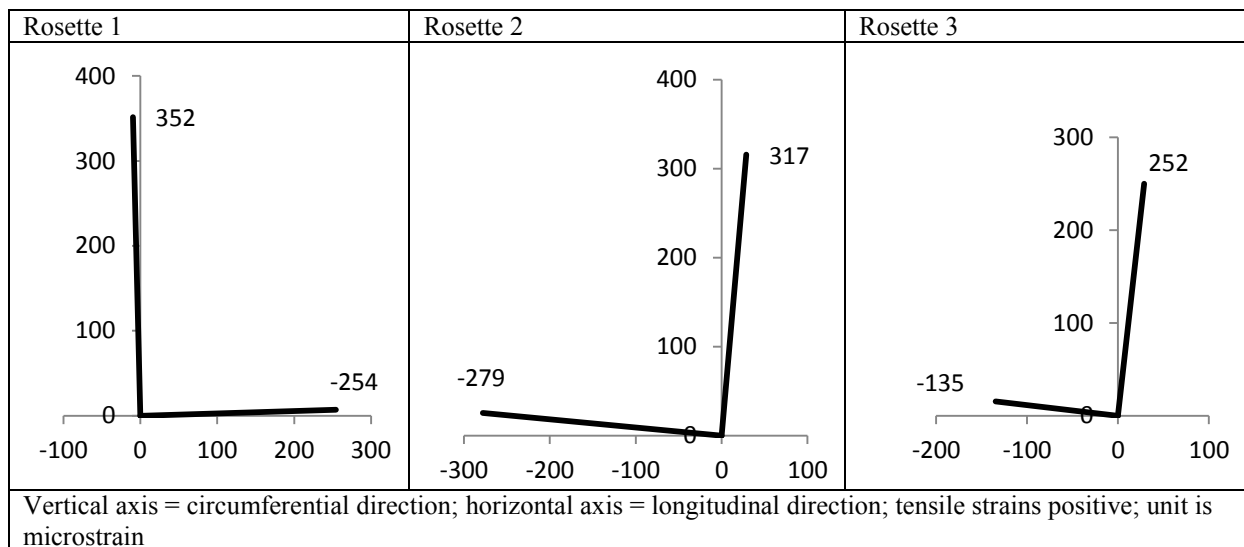
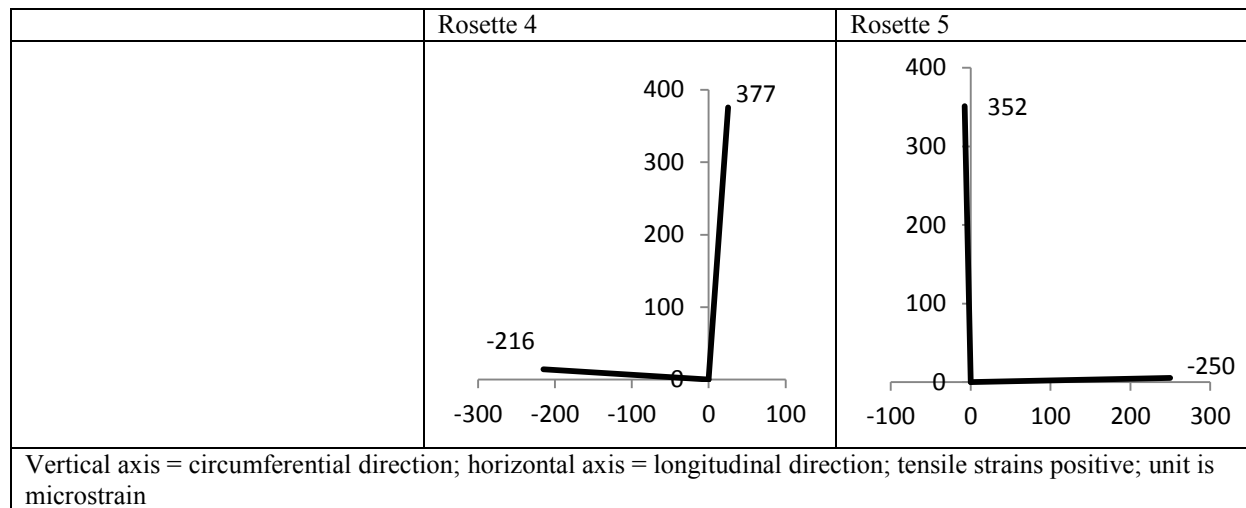


Table 3-2. Strain gauge results for segment 1.

**Table 3-3. Strain gauge results for segment 2.**

Segment		Circumferential strain ($\mu\epsilon$)	Longitudinal strain ($\mu\epsilon$)
1	Rosette 2	312	-274
	DEMEC	300	-353
2	Rosette 4	374	-213
	DEMEC	365	-370
Tensile strains positive			

Table 3-4. Comparison of DEMEC and strain gauge rosette results.

Intermediate strain gauge rosette measurements

The baseline readings and the final set of readings taken before segment removal were used to calculate the changes in principal major and minor strains in the skin. The intermediate DEMEC strains near the intrados of the circumferential flanges suggest that excavation works around the tunnel lining caused positive bending in segment 1 and negative bending in segment 2, before the segments were removed from the tunnel. The results for segment 1 are presented in Table 3-5. The results for segment 2 are presented in Table 3-6. The plots show the magnitude and direction of the principal major and minor direct strains on the surface of the cast iron. The strains in the circumferential and longitudinal directions have been calculated and are given in the tables.

For segment 1, the measured strains are predominantly slightly compressive in the longitudinal direction, and slightly tensile in the circumferential direction. The compressive strains in the longitudinal direction are consistent with the assumed mode of positive bending as suggested by the DEMEC readings on the circumferential flange.

For segment 2, the measured strains are tensile in the longitudinal direction, consistent with the negative mode of bending suggested by the DEMEC readings and the associated dishing inwards of the skin.

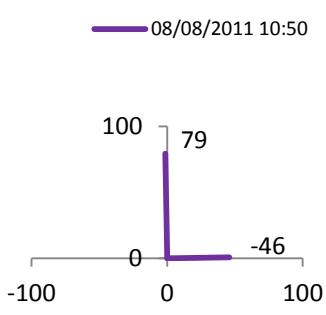
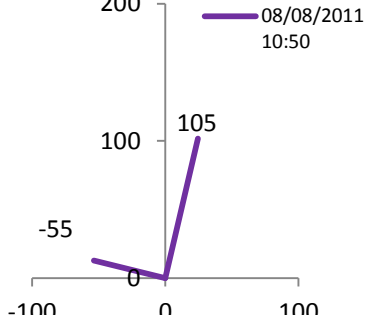
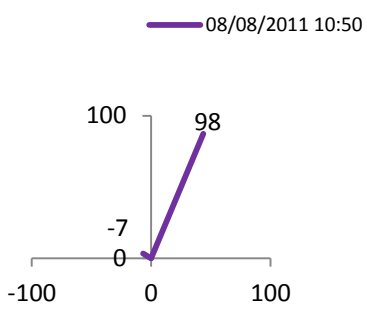
Rosette 1	Rosette 2	Rosette 3
 08/08/2011 10:50 100 79 0 -46 -100 0 100	 08/08/2011 10:50 200 105 100 -55 0 0 -100 0 100	 08/08/2011 10:50 100 98 -7 0 -100 0 100
Circumferential strain: 79 $\mu\epsilon$ Longitudinal strain: -46 $\mu\epsilon$	Circumferential strain: 96 $\mu\epsilon$ Longitudinal strain: -46 $\mu\epsilon$	Circumferential strain: 76 $\mu\epsilon$ Longitudinal strain: 13 $\mu\epsilon$
Vertical axis = circumferential direction; horizontal axis = longitudinal direction; tensile strains positive		

Table 3-5. Intermediate strain gauge results for segment 1.

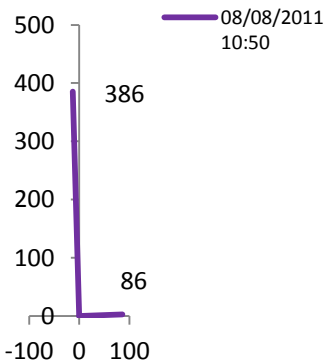
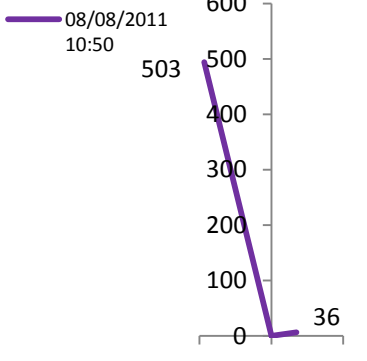
	Rosette 4	Rosette 5
	 08/08/2011 10:50 500 386 400 86 300 0 200 0 100 0 0 0 -100 0 100	 08/08/2011 10:50 600 503 500 36 400 0 300 0 200 0 100 0 0 0 -100 0 100
	Circumferential strain: 385 $\mu\epsilon$ Longitudinal strain: 87 $\mu\epsilon$	Circumferential strain: 487 $\mu\epsilon$ Longitudinal strain: 52 $\mu\epsilon$
Vertical axis = circumferential direction; horizontal axis = longitudinal direction; tensile strains positive		

Table 3-6. Intermediate strain gauge results for segment 2.

3.2.6 Results from displacement transducers

Continuous measurements of joint movement were taken by the displacement transducers over a period of six weeks. Figure 3-15 shows the changes in displacements measured by the four LVDTs with the effects of diurnal temperature fluctuations smoothed out by using 24 hour running averages. While the estimated displacements were clearly influenced by the overall temperature trends, there was also a downward shift in displacements for three of the four LVDTs. This downward trend was steeper before the platform side segments were removed (11th August 2011) and flattened out post removal. The LVDTs were calibrated so that an increase in displacement equalled the joint opening up. Also, because the measurements made by the LVDT displacement transducers were affected by the diurnal temperature fluctuations in the tunnel, a different system was selected for measuring joint movements in Central Line.

3.3 In-situ measurements in the Central Line running tunnel near Lancaster Gate

In October and November 2012, Imperial College installed electrical resistance strain gauges, potentiometric displacement transducers and tape extensometer eye bolts at selected locations in the

Central Line eastbound running tunnel between Lancaster Gate and Marble Arch. The instrumentation was installed shortly before the first of the Crossrail tunnel boring machines (TBM) passed below the Central Line tunnel at this interface.

The Crossrail twin tunnels begin at Royal Oak Portal in Westbourne Park, West London, and traverse westwards towards Farringdon. The Crossrail tunnels had a clearance of 4.2m to 4.9m under the Central Line twin tunnels east of Lancaster Gate station running at approximately 40 degrees skew (Figure 3-16). The Crossrail westbound tunnel passed under the Central Line eastbound tunnel at Crossrail chainage 2085m. The Crossrail eastbound tunnel passed under the Central Line eastbound tunnel at Crossrail chainage 2111m.

According to daily progress update from a personal communication with a CRL engineer (Oyenuga, 2013), the westbound tunnel TBM (TBM1) cutter head passed under Central Line eastbound tunnel at around 2:00pm on the 19th November, 2012. The cutter head of the eastbound tunnel TBM, (TBM2), passed under Central Line eastbound tunnel at around 19:30pm on the 5th February, 2013.

Continuous measurements of the strain in selected tunnel segments were made before, during and after the passage of the TBM at this interface. Similarly, continuous measurements were made of any opening and/or closing of longitudinal joints of the tunnel rings before, during and after the passage of the TBM at this interface. Finally, manual measurements were made of tunnel convergence and diameter/chord length changes at several rings of the eastbound Central Line tunnel before, during and after the passage of the TBM at this interface.

Table 3-7 summarises the locations of the instrumentation installed by Imperial College in the Central Line eastbound running tunnel. Figure 3-16 shows the locations of the instrumented tunnel rings in plan. Figure 3-17 shows the location of the instrumented segments and joints. Figure 3-18 shows the locations of the strain gauges on Rings 336 and 360.

Instrument	Location
Five no. strain gauges per segment	Axis, and two shoulder segments on R336 and R360
Displacement transducers at three horizontal joints	At the crown and at two shoulder level joints on R336 and R360
Seven no. temperature sensors	R336 and R360 - within each datalogger unit in the tunnel
Tape extensometer eye bolts at shoulder and knee levels.	R305, R336, R360, R378 and R402

Table 3-7. Locations of installed instrumentation.

3.3.1 The instrumentation

Electrical resistance strain gauges

The gauges used for this installation were the same as those used at Tottenham Court Road. However, instead of manual measurements the strain readings were continuously logged by dataloggers. Each gauge in the rosette was pre-wired with a 330-DFV leadwire. The leadwire was a flat cable of tinned stranded copper, insulated with vinyl.

Displacement transducers

Potentiometric displacement transducers were used to measure any opening/closing of the longitudinal joints. The transducers were supplied by Construction Monitoring Control Systems Ltd (CMCS) who were also responsible for the installation of the displacement transducers.

Temperature gauges

There was one temperature sensor placed within each datalogger unit in the tunnel. There were seven temperature sensors in total and the temperature was logged continuously.

Tape extensometer

An Ealey tape extensometer was used for taking manual measurements. The digital extensometer was able to provide a tenth of a millimetre accuracy. All the readings were taken by the same operative to ensure consistency.

The digital extensometer was hooked into eyebolts secured to the tunnel using the U-clamp system to measure the distance between two eyebolts. This allowed the tunnel convergence and diameter/chord length changes to be monitored. The tension sensor ensured the same tension was applied to the tape each time.

When the correct tension was reached, a green light switched on, and a red light indicated too much tension was applied.

Datalogger for strain gauge, displacement transducer and temperature gauge

The downloading unit on the platform consists of a power supply and an RS485 to RS232 converter through which the data from the in-tunnel loggers are transmitted.

3.3.2 The installation

Electrical resistance strain gauges

A total of 29 rosettes, i.e. 87 strain gauges, were installed. The installation of the rosette type electrical resistance strain gauges followed the same procedure as adopted for the Tottenham Court Road installations.

The leadwires were bundled together and brought down to the dataloggers by running the bundles through cable ties which were looped around and anchor glued onto the circumferential bolts. It took eight shifts during engineering hours to install the twenty-nine strain gauge rosettes and it took two additional shifts during engineering hours to connect the leadwires to the dataloggers. The installation at Central Line was labour intensive.

Displacement transducers

A total of six displacement transducers were installed and clamped onto the segments. The clamping bracket weighed about 1 kg. A total of four springs was used to hold the bracket in place. Each spring could apply 2.1 N/mm of force giving a total range of 100 N-150 N for all four springs. These springs pushed pointed rods onto the flange to hold it in place securely. The installation was done by CMCS. Figure 3-19 shows the installation by CMCS for Imperial College in Central Line.

The cable for each displacement transducer was brought down to the dataloggers by running the cables through cable ties which were looped around and anchor glued onto the circumferential bolts.

The main advantage of the potentiometric displacement transducers was that they were relatively easy to install. It took two shifts during engineering hours to install all six displacement transducers and have the associated cabling in place. However, the design of the bracket for the displacement transducer limited where the instrument could be placed on the flange. Unfortunately it was not possible to install the instrument in line with the edge bolt or to install the instrument close to the circumferential flange.

Eyebolts for tape extensometer measurements

The procedure for the installation of the eyebolts was efficient. Even though the discrete, manual tape extensometer readings could only be taken during engineering hours, nightly readings around the time of the TBM arrival were able to capture the elongation of the Central Line tunnel ring towards the new tunnel excavation.

Dataloggers

Datalogging units for the strain gauges, displacement transducers and temperature gauges were secured onto rings 336, 337 and rings 360, 361. The units were fixed to the segment using a bracket as shown in Figure 3-20. The logger box had counter sunk screws fed through the back and M6 nuts were used to secure the box on to the bracket. The weight of the logger with batteries is 3.93 kg.

Initially the dataloggers were powered via two wires in a six core cable. However, because of the distance between the power supply and the dataloggers, there was significant voltage drop and resulted in data corruption. Therefore batteries were placed in the dataloggers to provide a stable power supply during the passage of the first TBM. Subsequently, a second power cable was installed to power the loggers via six wires, thus reducing the voltage drop. The batteries were no longer necessary and were removed. The voltage requirement for the loggers is 9V.

An additional datalogger unit was secured onto the headwall at the east end of the eastbound platform at Lancaster Gate (Figure 3-21). This unit allowed the readings stored on the in-tunnel dataloggers to be downloaded onto a laptop during non-engineering hours.

3.3.3 Measurements and calculations

Electrical resistance strain gauges

Prior to the arrival of TBM1, 24 hours of stable baseline monitoring was obtained for all but five gauges. The data are presented as change in strain since the 18th November 2010 after 0100 hours in Figure 3-22. The measurements were initially unstable due to inadequate power supply. This was corrected by installing batteries at around 1am on the 18th November.

The vertical lines on the graphs mark the time when the cutting face of the TBM was directly beneath the Central Line tunnel. For subsequent discussions and calculations, the change in strain between the initial baseline and the final stabilised readings will be termed 'final change'.

Figure 3-22 gives an example of each of the three responses that was captured by the strain gauges. The title on the graph gives the location of the strain gauge. R336 or R360 refer to Central Line ring number 336 or 360. A1, A2 and B refer to the location of the segment in the ring as specified in Figure 3-17. 'Outer flange' refers to the strain gauge located near the intrados of the circumferential flange and 'Inner flange' refers to the strain gauge located near the extrados of the circumferential flange. R1, R2 and R3 refer to the strain gauge locations 1, 2 or 3 in the rosette (as seen in the inset in Figure 3-22). Finally, the 'E' or 'W' for the series on each graph indicates whether the gauge is on the west circumferential flange or the east circumferential flange of the tunnel ring.

The top graph in Figure 3-22 gives an example of the gauges on the east and west flanges measuring the same 'mode' of change, i.e. gauges at the same location but on opposite circumferential flanges would both measure either a tensile change or a compressive change.

The middle graph in Figure 3-22 gives an example of the readings from the gauges on the east and west flanges measuring opposite 'modes' of change. Three pairs of gauges displayed this behaviour. The bottom graph in Figure 3-22 gives an example of erratic measurement from a strain gauge (L341W). The unstable gauges are located on Ring 336 and their results are not processed further.

Figure 3-22 also shows that the strain gauges were detecting changes in strain in the segments prior to the TBM cutter face being directly below Central Line.

Figure 3-23 gives an example of the strain measurement over the period when TBM2 passed under Central Line. Unfortunately the power supply to the dataloggers was accidentally switched off for approximately 24 hours, resulting in no data when the TBM was passing under the Central Line. However, using the 48 hours of data prior to the power outage and the stabilised data collected after the TBM had passed; a 'final change' in strain in the segments as a result of TBM2 could still be obtained.

Using the 'final change' strain measurements from each strain gauge in a rosette, the principal major and minor strains and the principal strain directions were calculated for each rosette location. The calculations included corrections for strain gauge thermal output and gauge factor variation with temperature. Due to the erratic gauges described above, it was not possible to calculate the principal strains at several locations on Ring 336. These locations are listed in Table 3-8.

Refer to Figure 3-18 for the locations of the rosettes. Circumferential strain and longitudinal strain directions are defined as shown in Figure 3-2. Positive bending refers to the mode of bending having a straightening effect on the segment, and negative bending increases the curvature of the segment.

Once the magnitude and direction of the principal direct strains on the surface of the cast iron segment were determined, the direct strains on the circumferential flange along the circumferential direction could also be calculated. The circumferential strains were used to derive the bending moment induced in the segment at the instrumented locations. The calculations assumed a linear variation in strain from the circumferential flange intrados to extrados (i.e. between inner and outer measurements), and a constant modulus of 100 GPa. Due to the erratic measurements on ring 336, only the results for ring 360 are presented in Figure 3-24. The curved arrows indicate the mode of bending in the segment. In

all cases the calculated bending moments in the east and west circumferential flanges are in the same direction and have similar magnitudes.

It should be noted that the change in strains in the segments was more severe from the passage of the second TBM than from the combined effects of both TBM1 and TBM2.

Gauge designation	Location
L222	Ring 336 Segment A2 inner flange west
L242, L243	Ring 336 Segment A2 inner flange east
L341	Ring 336 Segment A1 inner flange west

Table 3-8. Locations where principal strains could not be calculated due to erratic gauge measurements.

Displacement transducer

The locations of the displacement transducers are shown on Figure 3-17. There were three displacement transducers on each ring. On ring 336 the displacement transducers were installed midway between the edge bolt and the middle bolt on the horizontal flange. On ring 360 one displacement transducer was installed between the edge bolt and middle bolt and two displacement transducers were installed at the middle bolt location. Displacements were recorded every fifteen minutes.

The displacement readings are given in Figure 3-25. The vertical axis shows the change in displacement. An increase in reading indicated the joint closing. The red vertical line indicates when TBM1 passed under Central Line and the black vertical line indicates when TBM2 passed under Central Line.

The displacement transducers on ring 336, located above the first Crossrail tunnel, registered joint opening magnitudes of less than 5 microns from the first Crossrail tunnel excavation beneath. Negligible change in displacement was measured from the second Crossrail tunnel excavation. The displacement transducers on Ring 360, located approximately 12m in front of the interface between the Crossrail tunnel and the Central Line tunnel, registered negligible changes due to the TBMs passing.

The gap in the data between 17/12/2012 and 06/01/2013 for five out of the six displacement transducers was due to data corruption during the data retrieval process. After the passage of TBM1, the logging interval was set to 4 hours and attempts were made to download the data once every few days. It was during this time that the data came back corrupted. At the beginning of February 2013, the logging interval was reset to 20 minutes and the data were retrieved once a day.

Tape extensometer

Tape extensometer measurements were corrected for temperature and the results are shown in Figure 3-26 for the passage of TBM1 and Figure 3-27 for the passage of TBM2. The tape extensometer eyebolts for ring 336 were secured across the circumferential flanges of ring 335 and 336. Similarly, the tape extensometer eyebolts for ring 360 were secured across the circumferential flanges of ring 360 and 361. The designations A, B, C and D shown on the graphs are for looking east along the tunnel. The final change in span for each case is shown in Figure 3-24 along with the results

from the strain gauges. The maximum changes were $\pm 0.1\%$ of the original span. This occurred as a result of the passage of TBM2.

Figure 3-26 shows that the tape extensometer readings stabilised quickly after the passage of TBM1. The baseline value for calculating the change in span as a result of TBM2 was taken as the measurement recorded on the 28/11/2012. As seen in Figure 3-27, ring 360 underwent more deformation as a result of the second Crossrail tunnel excavation because it lies closer to the second Crossrail tunnel than ring 336.

3.4 Fibre optics installations by ETH Zurich

The Imperial College research team facilitated optical fibres installation in the Central Line running tunnel by a team of researchers from ETH Zurich. Their preliminary results are of particular relevance to the experimental work described in this thesis. All the details for the optical fibres installation have been taken from the draft reports by ETH Zurich titled, “Results from the measurements of the strains development along the London Underground tunnel longitudinal axis during the passage of the Crossrail TBM by means of fibre optics” (ETH, 2013b) and “Results from the measurements of the strains development along the London Underground tunnel circumference during the passage of the Crossrail TBM by means of fibre optics” (ETH, 2013a).

An optical fibre cable 0.9 mm in diameter was fixed to the crown of the Central Line westbound running tunnel between Lancaster Gate and Marble Arch, for a distance of 65.6 m from ring 242 to ring 369. The Crossrail interface with Central Line was approximately 44 m from one end of the optical fibre cable. The cable was attached at every longitudinal flange location along the crown as shown in Figure 3-28. In theory, the cable should be able to measure the strains along a segment as well as the strains across the circumferential joints when the tunnel deforms longitudinally due to Crossrail excavations.

The installation involved first cleaning the surface of the cast iron of dust and cement, and sticking duct tape onto the surface to even out the surface further. The optical fibre cable was positioned onto the segment and secured by magnets with a holding force of 19 kg. The cable was prestrained to 0.5% to 0.8% to allow for compressive strain measurements. The monitoring was done manually during engineering hours between the 14th November 2012 and the 28th November 2012, covering the passage of Crossrail TBM1.

A sensor spacing of 2 mm was used to process the data. The strain measurements were affected by temperature and humidity in the Central Line tunnel, as well as structural elongations and contractions. Corrections to erase the environmental effects were attempted. However, because of the lack of baseline measurements, it was not possible to distinguish the humidity errors from the temperature errors. As a preliminary analysis of the data, the assumption that humidity changes were negligible was made, and only temperature effects were corrected.

The preliminary analysis suggested that the longitudinal deformation of the Central Line tunnel took place mainly as deformation of the lining with negligible strains observed across the joints. During the installation, the mortar at the joints between adjacent key pieces was in almost perfect condition, with

no sign of deterioration. With time, after the passage of TBM1, the strains in the joints appeared to increase to match the strains in the segments. One cause of this could be the redistribution of strains along the cable as it slipped beneath the magnets.

ETH Zurich also attached an optical fibre around the internal circumference of ring 332 to measure the changes in circumferential strain in Central Line due to Crossrail tunnelling. The cable was attached in such a way to monitor changes in strain along the segments as well as across the longitudinal joints. The optical fibre cable was positioned between the middle bolt and edge bolt. Once again attempts were made to correct the environmental errors. Preliminary analysis of the data suggested that joint movements across the longitudinal joints were observed as the ring deformed. The most relevant tape extensometer measurements would be that measured in ring 336 in Figure 3-26, which recorded changes in span of no greater than $\pm 0.05\%$ due to the passage of TBM1.

3.5 Discussion

The measurements taken of the section of tunnel constructed above ground at the LU Acton Depot have shown that the ovalisation of tunnel rings due to initial out-of-built and self-weight could be significant.

The main objectives of the instrumentation exercise at Tottenham Court Road were to measure the level of hoop stress in a tunnel segment and to trial the instruments and installation procedures.

The strain release when cast iron segments were removed from a tunnel ring was measured by electrical and mechanical strain gauges. The hoop force calculated from the magnitude of change in strain ranged from being 63% lower to 30% higher than that predicted assuming full overburden unloading and a constant modulus of 100 GPa. This exercise and the literature review in Chapter 2 highlighted the difficulty in ascertaining the hoop stress in existing tunnel linings.

The use of electrical strain gauge rosettes allowed the principal major and minor direct strains on the surface of the cast iron segment to be estimated, and provided insight into the mode of deformation of the segment.

Since it was noted that the measurements made by the LVDT displacement transducers were affected by the diurnal temperature fluctuations in the tunnel at Tottenham Court Road, a different system was selected for measuring joint movements in Central Line.

The main objective of the instrumentation exercise at Central Line was to measure the deformation behaviour of discrete rings as a tunnel excavation passed beneath.

The strain gauges responded with great sensitivity to the passages of the TBMs. Each rosette consisted of three strain gauges and only four out of the 87 strain gauges gave erratic response. This was testament to the care taken during all stages of the installation process. The readings began to deviate from the baseline levels approximately 12 hours before the TBM cutter head was beneath Central Line, when the cutter head was roughly 20 m away from the interface. The readings reached a final stable value no more than 48 hours after the TBM had passed. The most useful information provided by the strain measurement was to enable the calculation of the change in bending moment

and to ascertain the mode of bending. The main limitation was that because the external loading on the tunnel lining is unknown, it is not possible to calculate the bending moment distribution around the lining as a result of the excavations nearby.

The tape extensometer measurements were invaluable in providing straightforward information on the overall deformed shape of the tunnel lining. As seen in Figure 3-24, the deformed shape of the tunnel lining determined by the tape extensometer measurements corroborated well with the mode of bending in the lining segments captured by the strain gauge measurements. It is strongly recommended to include tape extensometer readings for future in-tunnel monitoring. Given the relative ease of installation and confidence in data interpretation of the tape extensometer relative to the strain gauges, it is suggested that where tape extensometer readings are taken, strain gauge installation would not provide significant additional useful information.

The lack of long-term stability of the potentiometric displacement transducer rendered it unsuitable in determining the magnitude of joint movement. The drift in the readings was several magnitudes higher than any potential joint movement. As mentioned previously, when the logging rate was set to a lower frequency and the data retrieval attempted once every few days, the data was corrupted. The reason is unknown but it appeared that daily data retrieval was necessary to ensure reliable data collection. Unless the stability of the system could be proven, and the bracket design modified to allow more flexibility in terms of installation location, it is not recommended to use this system for in-tunnel joint movement monitoring.

The work carried out by ETH Zurich was of great interest to the current research. The preliminary strain data measured during the passage of the first Crossrail TBM suggested that longitudinal deformation along the Central Line tunnel crown occurred along the segment and not at the circumferential joints. The optical fibre installed circumferentially on the internal surface of a tunnel ring number 332 was able to detect movement across the longitudinal joints as the Central Line tunnel ring deformed.

4 Material composition and testing

4.1 *Production of grey cast iron segments*

On the 10th of March, 2010, twelve half-scale grey cast iron segments were produced at Russell Ductile Castings (RDC) foundry for Imperial College. Figures 4-1 to 4-8 show some steps in the casting process. The test segments have an external arc length of approximately 1 m, giving a ring diameter of about 2 m (6 feet). This is the smallest size that can be manufactured with cast iron while maintaining true proportionality of all dimensions, particularly the skin of the segment (established from discussions between Morgan Sindall and their tunnel lining manufacturers).

The foundry was given the chemical composition for grey cast iron as determined by Fallon (1998) from disused cast iron segments dated between 1900 and 1925. This was done in order to replicate the properties of the Victorian cast iron lining in the LU network as closely as possible. The range given by Fallon (1998) was considered to be appropriate as the test results reported by Tube Lines (2008) for segments recovered from the Jubilee, Northern and Piccadilly Lines of the LU network fell within that range also (Table 2-2). The final mix used by RDC is given in Table 4-1. The specified and actual chemical composition of the grey cast iron segments are given in Table 4-2.

It can be seen that apart from the actual phosphorus content of the test mix being lower than the specified phosphorus content, all other chemical components were within the specified range. According to Section 2.2 in Chapter 2, the lower phosphorus content means the half-scale test segments are probably less brittle than the GCI tunnel segments in the LU network. In Table 4-2, the ultimate tensile strength given for the test mix was based on one tensile test performed by the foundry on a separately cast test bar. Additional tests were performed at Imperial College and this is discussed in the following section.

Component	kg	% of total weight
Pig Iron	1000	74.9
Graphite	3	0.2
Ferrophosphorus	50	3.7
Ferrosilicon	28	2.1
Ferromanganese	4	0.3
Sulphur	1	0.1
Steel Scrap	250	18.7

Table 4-1. Grey cast iron charge mix for half-scale test segments.

Main Component (%)	Test results reported by Fallon (1998) for six cast iron segments dated 1900 to 1925	Test results of half-scale grey cast iron mix by Russell Ductile Castings (2010)
Carbon	3.12 – 3.54	3.5
Silicon	1.65 – 3.08	2.2
Manganese	0.29 – 1.34	0.55
Sulphur	0.05 – 0.13	0.06
Phosphorus	1.07 – 1.59	0.88
CE ¹	4.47 – 4.66	4.5
Average Ultimate Tensile Strength ² (MPa)	142 – 175	167

Table 4-2. Chemical composition of cast iron.

Notes:

$$1. CE = TotalCarbon\% + \frac{Silicon\% + Phosphorus\%}{3} . \quad \text{Equation (11)}$$

2. For comparison, the ultimate tensile strength for grade 10 Cast Iron is 150 MPa (Gilbert, 1977)

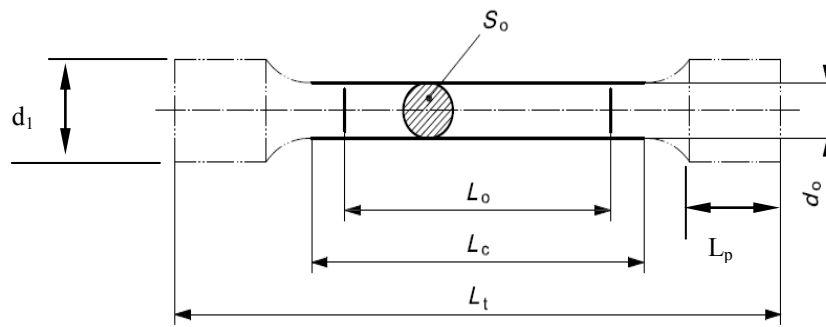
4.2 Mechanical tests on grey cast iron specimens

The use of grey cast iron was on the decline as a structural material when the BSI British Standards were first established. Therefore little interest was shown in producing a detailed specification for the use of the material in structures (Bates, 1984). There are however a number of standards for cast iron as a material, with the current standard being BS EN 1561:2011, published in October 2011. However, the version of British Standard that was available at the time of the casting and mechanical testing of GCI segments between March 2010 and February 2011 was BS EN 1561:1997. During the casting of the half-scale GCI segments on 10 March 2010, three standard test bars of 30mm diameter were cast in accordance with BS EN 1561:1997. Additionally, four circular rods of approximately 600 mm length and 30 mm diameter were cast.

The details of the as-cast test bars are given in Table 4-3. The test specimen dimensions are given in Table 4-4. In total, twelve cast iron test specimens were machined, with two different diameters being tested.

Quantity	Diameter (mm)	Length (mm)	Comments	Test
1	30	230	As-cast test piece. Used by RDC for standard strength and chemical tests.	Standard tensile strength and chemical composition tests
2	30	230	As-cast test piece. Machined and tested at Imperial College.	Tensile test with load and unload loops
4	30	600	Four additional test bars. Machined and tested at Imperial College.	Tensile test with load and unload loops

Table 4-3. Details of as-cast test bars.



Test Bar – Circular Cross-Section
Dimensions according to BS EN 1561:1997

No. of specimens	Diameter, d_0 (mm)	Parallel length, L_c (mm)	Grip end length, L_p (mm)	Diameter for grip ends, d_1 (mm)	Transition radius between grip end and parallel length, R (mm)
5	10 ± 0.1	30	40	12	25
7	20 ± 0.1	60	40	23	25

Table 4-4. Test specimen dimensions.

Tensile testing by RDC of one specimen gave a tensile strength of 167 MPa. For Grade 10 grey cast iron, the tensile strength is stated as 150 MPa, with the total strain at failure being approximately 0.6% when phosphorus content is greater than 0.4% (Gilbert, 1977). The total strain at failure comprises approximately 0.15% elastic strain and 0.45% plastic strain. Test results of the half-scale grey cast iron mix by RDC indicated a phosphorus content of 0.88%.

The remaining test specimens were tested at Imperial College. In order to capture the hysteretic stress-strain response and to construct the elastic modulus degradation curve, successively increasing strains were applied to the specimen, after each step returning to zero stress. The resulting stress-strain diagram should be of the type shown in Figure 2-5.

By returning to zero stress after each total strain increment, the magnitude of plastic strain, and therefore the elastic strain, corresponding to the total strain could be determined. Angus (1976) reported the method of obtaining the elastic modulus was by plotting the slope of the curve of elastic strain against stress and extrapolating to zero stress to obtain the modulus.

The tensile testing rig was controlled by the global extension rate. A linear variable differential transformer (LVDT) was attached to the cast iron specimen to measure the local extension for the calculation of local axial strain. The relationship between the global extension and the local strain of one specimen was determined in a conventional tensile test without unload-reload loops. The data were used to determine the unload-reload test programme for the subsequent GCI specimens.

Tensile testing was conducted with reference to BS EN ISO 6892-1:2009. The specimen was under displacement control at a rate of 0.006mm per second for both the loading and unloading loops. The rate corresponded to an internal strain rate of approximately 0.004% to 0.005%/s.

Of the seven 20mm diameter specimens available for testing, one was damaged because the clamping pressure on the jaws was too high. After that incident the jaw pressure was lowered to

800 psi. Of the remaining six specimens, one was tested as a conventional tensile test, while five were loaded and unloaded several times before reaching failure.

The diameters were measured at three locations along the parallel length of the specimen, and three measurements were taken at each location. The stress was calculated using the diametral measurements located closest to the failure plane. The resulting stress-strain curves are given in Figure 4-9. Two specimens failed outside of the LVDT gauge length and they are annotated on Figure 4-9. Excluding these two specimens, the failure stress ranged from 128 MPa to 148 MPa. These were lower than the tensile strength of 167 MPa reported by the foundry. The total strain at failure ranged from 0.57% to 0.93%, mostly higher than the failure strain of 0.6% reported by Gilbert (1977).

Unfortunately, the 10 mm diameter test specimens were probably slightly too small for the jaws of the tensile test rig to grip properly. The stress-strain curves in Figure 4-10 show a number of kinks where slippage might have occurred, especially during unloading. This has cast uncertainty over the reliability of the data. Excluding the results of the two 10mm diameter specimens which failed outside the gauged length, the failure stress ranged from 119 MPa to 170 MPa, with failure strains between 0.42% and 0.82%.

An average ultimate tensile strength for grey cast iron of 125 to 185 MPa was reported by Thomas (1997) for Northern Line Tunnel Segments dated 1890 to 1910. Test results reported by Fallon (1998) for six cast iron segments dated 1900 to 1925 gave ultimate tensile strengths between 142 and 175 MPa based on test specimens ranging from 11 mm to 14 mm in diameter. Test results on 20 mm diameter specimens reported by Tube Lines (2008) for lining segments dated 1890 to 1986 from the Jubilee, Northern and Piccadilly lines gave an average ultimate strength of 155 MPa for pre-1928 specimens and 190 MPa for post-1928 specimens. The tensile strengths of the 20 mm diameter specimens tested at Imperial College fell in the lower end of the previously reported tensile strengths.

The components of total and plastic strains at each load-unload cycle were measured and the elastic strain was calculated for the 20 mm diameter specimens. These values are presented in Figure 4-11. Up to a stress of 60 MPa, which was approximately 40% to 50% of the ultimate tensile strength of the specimens tested, the total strain was less than 0.1%, and the plastic strain was approximately constant at 0.02% or below.

The tangent modulus was calculated from the stress-strain data in Figure 4-9 using a 5-point running average to reduce the scatter in the plot. Figure 4-12 shows the results from all the loading curves. Figure 4-13 presents the tangent modulus for the first loading curves, i.e., not including the data from the unload-reload loops. While there is a large scatter when the strain is smaller than 0.1%, it can be seen that the tangent modulus values at small strain generally fell below 100 GPa.

The secant modulus was also calculated from the stress-strain data in Figure 4-9. Figure 4-14 shows the results from all the loading curves. Figure 4-15 presents the secant modulus for the first loading curves. At strains smaller than 0.1%, the secant modulus generally fell below 100 GPa.

The secant and tangent moduli curves for the 20 mm diameter specimens are shown together for the first loading paths in Figure 4-16. For completeness, the secant and tangent moduli curves for the

10 mm diameter specimens are also shown together for the first loading paths in Figure 4-17, despite the uncertainty in the quality of the test data as explained above. There is clearly more scatter in the data for the 10mm diameter specimens. However, from these plots, it seems reasonable to assume an elastic modulus of between 80 and 100 GPa for the grey cast iron test segments manufactured for the laboratory experiments.

4.3 Previous wrought iron bolt test data

Tube Lines (2008) reported that during 2004 and 2005 a large number of bolts were removed from tunnel linings and replaced with modern steel bolts from the following sections of LU tunnels:

- Tooting Bec to Tooting Broadway
- Oval to Stockwell
- Hampstead to Golders Green
- Covent Garden to Holborn
- Arsenal to Finsbury Park
- Swiss Cottage to St Johns Wood
- Green Park to Charing Cross
- Heathrow T4 to T123
- Leicester Square to Charing Cross
- Warren Street to Goodge Street

The removed bolts were all found to be wrought iron bolts. Wrought iron was produced from cast iron by the puddling process which consisted of raising the iron to a high temperature in a reverberatory furnace where the carbon and other impurities were removed by strong air blast, the iron being kept from direct contact with fuel (Bates, 1984).

Tube Lines (2008) reported that the removed bolts were tested according to BS EN 10002-1:2000 and the average ultimate tensile strength was 369 MPa. The characteristic strength was 342 MPa. Tube Lines (2008) adopted the strength calculations from BS 5950-1:2000 as shown in Table 4-5 to estimate the shear strength for the wrought iron bolts tested.

Bolt Grade	BS 5950-1:2000 Table 34 Tension Strength of Bolts (N/mm ²)	BS 5950-1:2000 Table 30 Shear Strength of Bolts (N/mm ²)
Other ($U_b \leq 1000 \text{ N/mm}^2$)	$0.7U_b < Y_b$	$0.4U_b$
Wrought Iron	$0.7 \times 342 = 239$	$0.4 \times 342 = 137$
U_b is the specified minimum tensile strength of the bolt Y_b is the specified minimum yield strength of the bolt		

Table 4-5. Estimation of shear strength for wrought iron bolts.

Sandberg (1990) also reported the tensile test results from bolts removed from tunnels at Angel Tube Station. Five bolts were tested according to BS 18:1987 and the average yield and ultimate stresses were 285 MPa and 348 MPa, respectively. It was noted that all the tested bolts showed a fibrous fracture face, typical of wrought iron structures.

Bates (1984) quoted generally accepted values from 1879 for the ultimate strength of wrought iron as being 324.3 MPa in tension, 247.1 MPa in compression and 308.9 MPa in shear. A factor of safety of 4 was considered as satisfactory which gave allowable stresses of 81.1 MPa for tension, 61.8 MPa for compression and 77.2 MPa for shear.

4.4 Testing of wrought iron bolt specimens

In 2011, fourteen wrought iron bolts were retrieved from the LU Waterloo and City Line and machined for tensile testing at Imperial College. Seven specimens were monotonically loaded to tensile failure; noting the tensile load at yield. The remaining seven specimens were tested with unload-reload loops.

The first wrought iron specimen was machined with threaded ends and was instrumented with three post-yield strain gauges and three local LVDTs. This exercise was done to compare the performance of the different instrumentation so that the remaining tests could be carried out with the most appropriate local instrumentation. This first specimen was monotonically loaded to tensile failure. Figure 4-18 shows the test setup. The specimen was machined with threaded ends so that it could be screwed into two stainless steel end-pieces to ensure the wedge jaws from the tensile testing rig could clamp onto the test piece properly, without slippage and damage to the ends of the wrought iron specimen.

Figure 4-19 shows six graphs comparing the strains measured using the strain gauges with those calculated from the LVDTs. The results are separated into three strain ranges so that the behaviour at small strains can be seen more clearly. The strain gauges show more variability at yield (around $1000\ \mu\epsilon$) and two of the gauges failed before the specimen failed. The LVDTs were better able to capture the entire loading response.

Figure 4-20 compares the average strain calculated from the strain gauges and the LVDTs. The average measurements were very similar. Taking into account the shortcomings of the strain gauges mentioned above, it was decided to use LVDTs to measure the local strains for the tensile tests on the remaining wrought iron specimens.

After the seven monotonically loaded specimens were tested, the testing regime for the tensile tests with unload-reload loops was finalised. The test regime was to load the specimens monotonically to a point pre-yield, then unload completely before subjecting the specimens to 10 cycles of load-unload at a pre-yield state. The specimens were then monotonically loaded to post-yield, then unloaded completely before being subjected to 10 cycles of load-unload at a post-yield state. Finally the specimens were loaded to failure.

Results from the wrought iron specimens which have been tested at Imperial College are shown in Figure 4-21 and Figure 4-22. Figure 4-21 shows the full strain range while Figure 4-22 zooms in on the small-strain range. The yield strength of 239 MPa calculated from the Tube Line (2008) data was higher than the stress at which the wrought iron specimens began to yield in the laboratory tests, which was closer to 200 MPa. The ultimate tensile strength, not taking into account area reduction, ranged from 340 MPa to 370 MPa. The specimens failed at between 15% to over 25% axial strain. The strain was measured using LVDTs with a linear range up to approximately 25% strain ($250000\ \mu\epsilon$). The initial gauge length was 30 mm.

Figure 4-23 shows the behaviour of a specimen during the pre-yield load cycles. When the specimen was unloaded at 30kN (150 MPa) to 0kN, there was a permanent strain of between $100\ \mu\epsilon$ and $200\ \mu\epsilon$. Subsequent load cycles were non-linear hysteretic elastic, with the hysteretic loop becoming

progressively narrower as the number of cycles increased. Also note that each loop shifted along the strain axis, indicating that plastic strains were developing in the specimen from the loading cycles.

The post-yield load cycles also showed non-linear hysteretic elastic behaviour.

The initial and final diameters were measured for each specimen and the reduction in area ranged from 18% to 36%. From visual observations during the tests the diameter of the specimen began to reduce after the yield point was reached. If the diameter was assumed to reduce linearly between the yield point and the ultimate load point then stress-strain curves as shown in Figure 4-24 are obtained. This assumption leads to larger differences in the ultimate tensile strength of the wrought iron specimens compared to the case where area reduction was not taken into account. If the area reduction at failure was taken into account the ultimate tensile strength ranged from 420 MPa to 560 MPa, far higher than the values cited in the existing literature.

The Tube Lines (2008) stress-strain data at 0.1% proof strength suggested a pre-yield modulus of approximately 270 GPa. Examining the slopes of the initial load path of the specimens tested at Imperial College gave a range of 200 GPa to 700 GPa for the pre-yield modulus.

The pre-yield secant modulus was also calculated using the pre-yield unload-reload loops. Examples of the loops are shown in Figure 4-25. The range of values was large, from approximately 300 GPa to 1250 GPa.

It was not possible to determine a specific pre-yield modulus for the wrought iron. However, 200 GPa and 1250 GPa could be taken as lower and upper limit values.

Secant modulus was calculated from the post-yield unload-reload loops. Examples are shown in Figure 4-26. This value was consistently between 150 GPa and 160 GPa.

Since it was not possible to manufacture wrought iron bolts for the purpose of this research, grade 4.6 mild steel bolts were used instead. The properties of grade 4.6 mild steel bolt from BS EN 1993-1-8:2005 are compared with the properties of wrought iron in Table 4-6. These values are also shown on Figure 4-22 to compare with the stress-strain response of wrought iron specimens.

The biggest difference is the pre-yield modulus. This was mainly because there was a large range for pre-yield modulus for the wrought iron specimens tested. Tube Lines (2008) reported a value of 270 GPa for the pre-yield modulus. The stiffer stress-strain response of wrought iron compared to mild steel will be considered when interpreting the laboratory tests on the bolted segments.

	Grade 4.6 mild steel	Wrought iron
Tensile strength (MPa)	400	340 to 370
Elongation at fracture ($\mu\epsilon$)	220000	150000 to over 250000
Yield strength (MPa)	240	220 to 250
Pre-yield modulus (GPa)	210	200 to 1250

Table 4-6 Properties of Grade 4.6 bolt and wrought iron

4.5 Use of grommets

In Chapter 2 it was mentioned that the bolt holes within some tunnel linings were waterproofed individually by placing a grommet of hemp, dipped in red lead and oil, under the washers at the head and nuts of the bolts (Hewitt and Johannesson, 1922). It has not been possible to recover grommets from the LU tunnels and test for their properties. Instead, modern-day gel grommets were ordered from Tunnelling Accessories for inclusion into the experimental setup. These gel grommets were designed specifically to fit the half-scale bolt holes. They were manufactured from jute yarn impregnated with a waterproof gel.

4.6 Discussion

After consideration of the published grey cast iron properties in the existing literature and the mechanical tests performed on the cast iron specimens, the parameters given in Table 4-7 were selected to be used in the planning of the laboratory experiments and in the subsequent analysis of results.

Parameter	Design value
Ultimate tensile strength (MPa)	120
Ultimate compressive strength (MPa)	480
Elastic modulus in tension and compression (GPa)	100
Best fit tensile stress-strain relationship	See Figure 4-27.
Poisson's Ratio	0.26

Table 4-7. Grey cast iron properties for use in conjunction with laboratory experiments.

The ultimate tensile strength was selected based on the lower limit of the tensile test results from the 20 mm diameter grey cast iron specimens. Results from the less reliable tests on 10 mm diameter specimens were also taken in account. For experimental purposes, it was decided to restrict the bending moment such that the tensile stress in the extreme fibre at the most critical locations were less than 40% of the ultimate tensile strength determined in this Chapter. Reference to Figure 4-11 will show that up to that stress level, the magnitude of plastic strain was fairly constant at below 0.02%. At this stress level, it was still considered to be reasonable to analyse the strain measurements based on the assumption of linear elasticity, and to be able to carry out a series of tests to compare the effects of variables such as overburden pressures, bolt preload, and grommets on the deformation behaviour of the cast iron segments.

The ultimate compressive strength of the grey cast iron was not tested, and was taken to be four times the ultimate tensile strength based on literature review.

The secant and tangent moduli shown in Figure 4-16 indicate that 100 GPa would be a reasonable upper limit for the test data. An upper limit was selected because in the laboratory tests strains would be measured and then subsequently be converted to stress using the elastic modulus. Since a limit was placed on the stress value as discussed above, choosing an upper limit for the elastic modulus was also conservative. Should a more detailed stress-strain relationship be required, the best fit, upper and lower limit curves have been provided in Figure 4-27. The curves are fifth order polynomials fitted to the test data. They are considered to be appropriate up to a total strain of 1%.

The Poisson's ratio was taken from the literature (Angus, 1976).

Since it was not possible to manufacture wrought iron bolts for the purpose of this research, grade 4.6 mild steel bolts were used instead. The stiffer stress-strain response of wrought iron compared to mild steel was considered when interpreting the laboratory tests on the bolted segments.

Grommets were included in the two segment and full-ring tests after the system has been tested at different bolt preloads without the inclusion of grommets. In the section of Central Line east of Lancaster Gate, grommets were not observed during the instrumentation and monitoring exercises described in Chapter 3. However because the use of grommets is recorded in literature, they have been included as a test variable in the experiments.

5 Revisiting the two segment test

5.1 Initial estimate of segment and joint moment capacities at zero hoop force

Before any laboratory experiments were carried out, the ultimate bending moment capacity of the grey cast iron segment was calculated and the ultimate bending moment capacity for the joint was estimated. The main aim of this was to obtain an upper limit to the applied loading so that a series of parametric tests could be carried out. It was also important to limit the stresses in the segment so that the data could be analysed assuming elastic behaviour. A series of tests with the bolts at the joint preloaded to different magnitudes was conducted. Limiting the stresses allowed the same two segments to be used for all the tests. Only twelve segments were cast for this project and six were required for the full-ring test. In order for multiple tests to be performed on the same segments, care was taken not to prematurely break the segments. As mentioned in Chapter 4, the ultimate strength of the grey cast iron was taken to be 120 MPa. It was decided to limit the bending moment imposed on the segments and joints such that the stress at the extreme fibre would not exceed 40% of its ultimate tensile strength. As shown in Figure 4-11, the plastic strain was approximately constant at 0.02% or below when the cast iron specimens were stressed up to approximately 40% to 50% of the ultimate tensile strength.

The bending capacity of the half-size segment was estimated based on the section geometry. Figure 5-1 and Figure 5-2 give the dimensions of the segments forming a ring at half the scale of a typical 12ft 6in GCI tunnel ring in the LU network. Figures 5-3 and 5-4 give the details at the joints. The calculations for the bending capacity of the segment are given in Appendix C. When the hoop force is zero and the mode of bending is positive, i.e., the segment straightens and the extreme fibre at the tip of the circumferential flange (intrados) is in tension, the bending moment capacity is 3.45 kNm. When the mode of bending is negative, i.e., the segment increases in curvature and the extreme fibre at the top of the skin (extrados) is in tension, the bending moment capacity is 9.54 kNm. These moment capacities were conservatively calculated assuming a linear stress distribution with failure occurring when the stress in the extreme tensile fibre reached 120 MPa.

The bending moment capacity of the joint was estimated based on the flexural capacity of the longitudinal flange. As mentioned in Chapter 2, Thomas (1977) found that when a pair of segments were bolted together at the longitudinal joint and tested such that there was positive bending at the joint, the middle bolt on the longitudinal flange carried substantially less load than the two outside bolts when the segment was subjected to bending. Keeping this in mind, only the contribution from the two edge bolts was taken into account when estimating the bending capacity of the joint.

The calculations are outlined in Table 5-1. The schematic view in the table shows the longitudinal flange with three bolt holes in line. As an initial estimate, the area A of the flange was assumed to deform as a cantilever under the action of bolt load. During positive bending, the area A is assumed to be effective in transferring tension from the bolt onto the circumferential flange. The maximum bolt load was estimated based on the flexural strength of the cantilever (Part 1, Table 5-1). Initially,

deformation of the cantilever in single and double curvature were considered (Figure 5-5). The bending moment capacity of the joint was then estimated by multiplying this load with the respective lever arms for positive bending or negative bending of the joint.

The data from Thomas (1977) was assessed to determine whether an assumption of single or double curvature is more realistic in the estimation of the joint bending capacity for the two segment test. A review of the test data from Thomas (1977) led to the assumption of double curvature in estimating the bending strength of the longitudinal joint. Thomas (1977) measured the increase in bolt load above initial preload as he increased the bending moment at the joint in his two segment test using full-scale GCI segments. Chapter 2 described the two segment test on 12ft 7in GCI segments and the test setup was shown in Figure 2-14. Thomas' bolt load measurements were plotted against the angle of opening measured at the intrados of the joint. The results are shown in Figure 5-6. One of the edge bolts had an increase in load of about 80 kN.

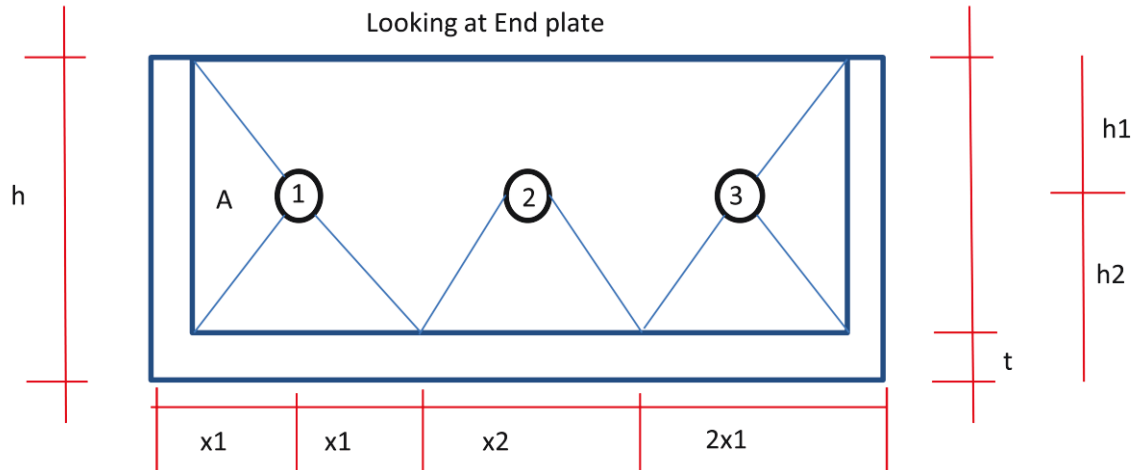
Using similar calculations as shown in part 1 in Table 5-1, the maximum bending moment of a 'cantilever' part of the longitudinal flange resulting from a bolt load of 80 kN could be estimated. Apart from the flange thickness, the dimensions of longitudinal flange were not explicitly given in Thomas' paper. Since Thomas conducted his tests on 12ft 7in segments, and the dimensions used in Table 5-1 for the half-scale test ring were based on 12ft 6in segments, it is considered that doubling the dimensions used in Table 5-1 would be sufficiently accurate for the purpose of determining the mode of bending.

Therefore, the cantilever length was assumed to be twice that calculated in Table 5-1, i.e. $2 \times 33.5 = 67$ mm. The bending moment assuming single curvature would be $80 \text{ kN} \times 67 \text{ mm} = 5.4 \text{ kNm}$. The width of the cantilever was assumed to be $2 \times 51 = 102$ mm. The thickness of the longitudinal flange was given by Thomas (1977) to be $1\frac{1}{4}$ in, or 32 mm. The resulting stress in the longitudinal flange would exceed 310 MPa. The lining Thomas tested was of Grade 12 according to BS1452: 1961 (Thomas 1977). The section thickness ranged from 1 in to $1\frac{1}{4}$ in. Referring back to Table 2.1, the tensile strength would be around 170 MPa to 190 MPa. The calculated stress of 310 MPa under the assumption of single curvature was far above the reported tensile strength of the Grade 12 cast iron. An assumption of double curvature would appear more reasonable for the initial estimation of the joint moment capacity.

Assuming double curvature bending, the positive bending capacity of the joint was calculated to be 1.12 kNm and the negative bending capacity was calculated to be 0.49 kNm. The bending capacity at the joint is much lower than the segment bending capacity.

In the two segment experiments, the positive bending at the joint was limited to 0.45 kNm, 40% of the capacity estimated in Table 5-1. In the full-ring experiments, the joint capacities will also take into account the contribution from the hoop stress as shown in Part 3 of Table 5-1.

Schematic view of longitudinal flange for the half-scale segment

**Dimensions from Drawing No. UnPS/ICR/GN/D/0004**Depth of segment, $h=62\text{mm}$ $x1=51\text{mm}$ Intrados to bolt hole, $h1=25.5\text{mm}$ $x2=76\text{mm}$ Bolt hole to extrados, $h2=36.5\text{mm}$ Thickness of longitudinal flange, $t1=15.8\text{mm}$ Thickness of skin, $t=11\text{mm}$ Thickness of circumferential flange, $t2=17.5\text{mm}$ **Part 1. Calculations based on flexural capacity of longitudinal flange**

Tensile strength of cast iron	σ_T	120	MPa	
b_A		51.0	mm	Width of cantilever assumed to be distance $x1$
d_A		15.8	mm	Depth of cantilever assumed to be $t1$
I_A	$=bd^3/12$	16763.3	mm^4	
Section modulus, Z_A	$=I/(d/2)$	2121.9	mm^3	
Maximum moment for cantilever, $M_{\max A}$	$=\sigma_T Z$	254632.8	Nmm	
Cantilever length, L	$= x1 - t2$	33.5	mm	
<i>Assuming that the maximum moment is caused by the bolt force</i>				
Maximum tensile force at bolt, $T_{\max \text{ double curvature}}$	$=2M/L$	15.2	kN	Limited by flexural capacity of longitudinal flange
$T_{\max \text{ single curvature}}$	$=M/L$	7.6	kN	

Part 2. Bending moment capacity of the joint under zero hoop load*Positive bending*

Lever arm from bolt to extrados, $h2$	36.5	mm	
$M_{\max}$ at joint from $T_{\max}$ bolt (double curvature) = $(15.2 \times 0.0365) \times 2$	1.12	kNm	Assumed 2 bolts; extrados compression
$M_{\max}$ at joint from $T_{\max}$ bolt (single curvature) = $(7.6 \times 0.0365) \times 2$	0.56	kNm	Assumed 2 bolts; extrados compression

Negative bending

Lever arm from bolt to intrados, $h1$ -caulking groove	16.0	mm	Caulking groove is 9.5mm deep
$M_{\max}$ at joint from $T_{\max}$ bolt (double curvature)	0.49	kNm	Assumed 2 bolts; extrados tension
$M_{\max}$ at joint from $T_{\max}$ bolt (single curvature)	0.24	kNm	Assumed 2 bolts; extrados tension

Part 3. Additional bending moment capacity of the joint with hoop load
Hoop force, N
Bending moment = Ne , where $e = 16.5\text{mm}$ for positive bending (centroidal axis to extrados) $e = 36.0\text{mm}$ for negative bending (centroidal axis to caulking groove)

Table 5-1. Initial estimate of joint bending capacity.

5.2 The two segment test setup

This section describes the two segment test setup at Imperial College. A schematic drawing showing the setup of the two segment test is given as Figure 5-7. The instrumentation layout for the test is shown in Figure 5-8.

Two half-scale cast iron segments were bolted together to form an 'arch' with a joint at the axis of symmetry as shown in Figure 5-7 and Figure 5-9. A line load was applied to the extrados of each segment as shown in Figure 5-10, 110 mm from the axis of symmetry. Loading was applied using an air-bellow actuator. Five load cells were used in the experiment. One load cell measured the total load applied to the segments, and four load cells were used to check that the loading was applied equally on the two segments and also evenly along the brass rod. Small adjustments could be made to the small 'foot' that extends below the load cell so that the loads were more evenly spread (Figure 5-10). The brass rod distributed the loads as line loads on the segment.

One segment was instrumented with strain gauges and rested on roller support as seen in Figure 5-9. The displacement of the roller support was measured with a strain gauged displacement transducer. The other segment simply rested on a steel plate as shown in Figure 5-11 and was considered to be a pin support. A dial gauge was placed at the pinned end. The resolution of the dial gauge was $1/1000^{\text{th}}$ of an inch. No movement was observed at the pinned end through all the loading and unloading cycles during the experiments. A displacement transducer was used to measure the vertical displacement at the intrados of the joint.

LVDTs were located on the intrados of the joint to measure joint opening, with the point of measurement located several millimetres below the actual segment (for example see outer edge LVDT in Figure 5-10). The locations of the LVDTs at the intrados are labelled as 'Middle', 'Edge bolt' and 'Outer edge'. The locations are shown in Figure 5-12.

Several instruments were installed to check if the segment behaved as described in Thomas (1977), namely that there was a bulging out of the skin during positive bending and a dishing in of the skin when bending was negative. This distortion for positive bending is illustrated in Figure 2-11. A displacement transducer was positioned to measure any deflection of the skin when the system was loaded. The tip of the displacement transducer was held in position with a magnet attached to the skin so that the tip always measured the same point on the intrados of the skin. Two LVDTs were used to measure any deflection of the circumferential flanges. Figure 5-13 shows the instruments used for measuring this change in the segment upon positive bending.

The axial bolt loads for the three bolts were measured directly using instrumented bolts. An example of an instrumented bolt is shown in Figure 5-14. Three 12 mm diameter grade 4.6 steel bolts were strain-gauged at Imperial College for this series of experiments.

The calibration of the instrumentation used in the two segment experiments and the full-ring experiments will be discussed in detail in Chapter 6.

5.3 Loading of two segment test

The geometry of the two segment test meant that the hoop load at the joint is zero. Due to the weight of the loading components, including the air-bellow actuator, the load cells, and the brass bars used to spread the load onto the skin of the segment, and also the self-weight of the segments themselves, there was a positive moment of approximately 0.25 kNm at the joint before any loading was applied by the actuator. The loading was increased by manually controlling the actuator until the positive moment at the joint reached approximately 0.45 kNm which is 40% of the estimated joint capacity. There were two intermediate loading steps, one at approximately 0.30 kNm and one at approximately 0.36 kNm.

The setup of the two segment test was as follows:

- The two segments were roughly placed in position with arbitrary low bolt forces. Self-weight of segments was acting at the joint.
- The segments were then supported by straps so that the self-weight was not acting at the joint.
- The bolts were tightened to the desired preload. Straps still in place to support the two segments.
- After the bolts were tightened, it was then possible to install the LVDT displacement transducers at the joint. There was not sufficient access to have the LVDTs in place before tightening the bolts. Straps still in place to support the two segments.
- The straps were removed and the self-weight of the segments acted at the joint.
- Positioned loading components onto the segments.
- Attempted to evenly distribute the loading across the four load cells.

It proved to be very difficult to evenly distribute the load. The loading component was lifted up and lowered several times, and at certain points the two segments shifted and had to be repositioned. When the loads were finally evenly distributed, it was decided to consider the 'start' of the test with the self-weight of the segments and the loading components already acting at the joint, i.e., a bending moment of approximately 0.25 kNm.

When discussing the results, all the graphs showing 'joint opening' give the INCREASE in joint opening from a baseline moment of 0.25 kNm.

A personal communication with a LUL engineer (Bowers, 2011) suggested that the general practice in tunnel ring construction was to have the bolts finger-tight and finished off with a quarter turn using a torque wrench. In November 2011, a one-inch diameter wrought iron bolt recovered from the LU Waterloo and City Line was strain gauged in order to investigate the magnitude of bolt preload resulting from the described tunnelling practice. The bolt was tightened to finger tight and finished off with a quarter turn. This was repeated over thirty times. The resulting average bolt preload was 30 kN, with a 95% confidence interval of 17 kN to 44 kN. Scaling down for the 12 mm diameter mild steel bolts used in the two segment laboratory tests, the preload range would be 4 kN to 10 kN. In the laboratory tests, preload values of 5 kN, 7.5 kN and 10 kN were adopted.

Four sets of tests were performed on the two segment setup. Each set consisted of seven to ten loading cycles to 0.45 kNm. The bolt preload was changed between each set of tests. The preload magnitudes were 5 kN, 7.5 kN and 10 kN as explained. The first three sets of tests were carried out without grommets. Grommets were introduced in the final set of tests and the preload was 5 kN in this case (Figure 5-15).

The horizontal movement at the roller support was measured at the top edge of the longitudinal flange as shown in Figure 5-16. This was taken into account when extracting results from the FE analyses for comparison with the laboratory measurements. Likewise, it was noted that the LVDTs used to measure joint opening were located several millimetres below the actual segment. This was taken into account when comparing with the FE predictions and the most appropriate displacements were extracted from the FE analyses so that like-with-like comparisons were made.

5.4 Finite element modelling

As mentioned in Section 1.4, three-dimensional FE analysis of the two segment test was being carried out as part of this research using ICFEP, the Imperial College Finite Element Program developed by Professor David Potts. The generation of the FE mesh and all the FE analyses were completed by post-doctoral researcher Dr. Aikaterini Tsiampousi. The author was responsible for post-processing the output from ICFEP such that the FE predictions could be compared with the laboratory test results

The geometries of the components modelled in the FE analysis were based on Figure 5-1 to Figure 5-4. The various FE meshes generated for the analyses are shown in Figure 5-17. The model takes advantage of two axes of symmetry and consisted of half of one segment (i.e., $1\frac{1}{2}$ bolt holes), and the appropriate boundary conditions. The boundary conditions are shown in Figure 5-18. At the vertical axis of symmetry, which is where the joint is, the segment was allowed to move vertically up and down but horizontal movements were restricted. At the horizontal support the segment was allowed to move horizontally but vertical movements were restricted. The third boundary condition was applied at the second axis of symmetry circumferentially along the middle of the segment. At this boundary out-of-plane movements in the z-direction were restricted, but movements in the x- and y-directions were not restricted.

In the FE analyses, linear elastic solid elements were used to model the segment, bolts and washers. The stiffness of the grey cast iron was assumed to be 100 GPa and the stiffness of the bolt and washers were also assumed to be 100 GPa. A Poisson's ratio of 0.26 was adopted for the linear elastic elements, based on literature review (Angus, 1976). The joint elements were assigned a Mohr-Coulomb type constitutive model with shear and normal stiffness moduli of 10^9 kN/m³ (or kN/m/m²). The shear and normal stiffness moduli are only relevant when the joint elements are in the elastic region.

In the FE analyses, the bolt preload was applied in increment one and gravity was switched on in increment two. The load on the segment was applied from increments three to twelve.

Initial FE analyses were carried out to compare the response of the segment when the loading was applied as two point loads rather than a line load (Figure 5-19). This was to check the implications if

the brass bar was not able to distribute the loading evenly as a line load to the skin of the segment. Figure 5-20 compares the FE results for the increase in bolt load, the opening at the joint, and the global movement at the roller support for the line load and the point load applications. There were negligible differences in the response of the segment and the joint from the two loading conditions. Consequently it was considered reasonable to simulate the loading as line loads in the FE predictions for comparison with the laboratory results.

5.5 Comparison of FE prediction with laboratory results

In this section, the laboratory results are compared with the FE predictions. All the relevant graphs are presented in Figure 5-21. The graphs are presented in three columns, with each column showing the results for a particular bolt preload. For the 5 kN preload, the laboratory results with and without grommets are shown on the graphs. Grommets were not used for the tests carried out at 7.5 kN and 10 kN preloads. The graphs on rows 1 to 6 focus on the behaviour at the joint, and the graphs on rows 7 and 8 show the global behaviour of the bolted arch. On the graphs, the ICFEP prediction is the red dashed line and the laboratory measurements are the other lines or markers.

As mentioned in the previous section, the joint opening magnitude was extracted from the FE analyses in a way to match where the measurements were taken in the laboratory tests as illustrated in Figure 5-22.

Row 1 in Figure 5-21 shows that as the moment at the joint increased from approximately 0.25 kNm to 0.45 kNm, the measured increase in joint opening at the edge bolt location was greater than the FE prediction. This suggests that the stiffness assumed for the joint elements in this case, 10^9 kN/m³, might have been too high. Row 2 shows that for the same increase in moment, the measured increase in joint opening at the outer edge location was similar to the FE prediction. Row 3 suggests that there was negligible joint movement at the location of the middle bolt.

The influence of the stiffness assigned to the joint elements on the joint behaviour was investigated. Figure 5-23 compares the laboratory results with a set of FE parametric studies using normal and shear stiffness moduli ranging from 10^6 to 10^9 kN/m³ for the joint elements. The bolt preload value used in the FE parametric study was 7.5 kN. At the outer edge location, the different joint stiffness values gave very similar response curves, all of which were similar to the measured response. At the middle bolt location, laboratory results showed that a joint stiffness of 10^6 kN/m³ would be too low. At the edge bolt location, laboratory results indicated that a joint stiffness between 10^6 and 10^7 kN/m³ seemed appropriate. Figure 5-23 suggests that the shear and normal joint stiffness moduli decreased as the joint opened up, i.e. the stiffness was high at the middle bolt location where there was negligible opening, the stiffness reduced at the edge bolt location as the joint opened up. At the outer edge location the various stiffness moduli gave similar results.

The Mohr-Coulomb model for the joint was given a small cohesion but a large angle of shearing resistance. When sufficient Gauss points within a joint element go into tension, the joint will open and while it is open it has no stiffness. This could explain the behaviour at the outer edge location, where the magnitude of joint opening achieved has rendered the joint stiffness moduli irrelevant.

Row 4 and row 5 in Figure 5-21 show that the measured increase in bolt load as the moment at the joint increased from 0.25 kNm to 0.45 kNm was lower than the FE prediction. However, the modulus used for the bolt in the FE prediction was 100 GPa, which was already lower than the modulus of 210 GPa generally adopted for mild steel. Figure 5-24 compares the response using different stiffness moduli for the bolt and it indicates the modulus of the bolt might need to be reduced to 50 GPa in order to match the measured behaviour. This could mean that the 'bolted connection', which included the bolts and the washers, had a lower stiffness than the stiffness of the bolts alone.

Row 7 in Figure 5-21 shows the horizontal movement at the roller support and row 8 shows the vertical displacement at the joint, when the bending moment at the joint increased from approximately 0.25 kNm to 0.45 kNm. FE predicted slightly higher horizontal and vertical displacements than the laboratory measurements, even though locally at the joint FE predicted smaller joint openings. The smaller global movements measured in the laboratory could be due to friction at the roller support.

Looking at the results for the 5 kN preload, the presence of the grommets appeared to result in an even 'softer' bolted connection. Row 4 in Figure 5-21 shows that for a given increase in bending moment, the increase in bolt load was smaller when the grommets were in place. This could imply that the grommets render the bolt tension less sensitive to the joint opening. When attempts were made to preload the bolts above 5 kN when the grommets were in place, the load would decrease with time as seen in Figure 5-25. The bolts had to be re-tightened several times in order for the preload to stabilise around 5 kN. For a real tunnel, it would be unlikely that the tunnellers would go back and re-tighten all the bolts. This suggests that the 5 kN preload might be more representative of the in-situ preload in real tunnels than the 7.5 kN or 10 kN preload if grommets were used.

One limitation of the current experimental study in capturing the behaviour of real tunnels is the lack of restraint in the longitudinal direction. The circumferential flanges of bolted segmental rings in a real tunnel would be limited in the extent to which displacement in the longitudinal direction could occur due to the presence of adjacent rings. The gap of several millimetres between circumferential flanges as seen in Figure 5-4 would generally be filled with waterproofing material as discussed in Chapter 2.

In an attempt to address this issue, a FE analysis was done where the circumferential flanges were restricted from any longitudinal movement. In Figure 5-26 all the series with a postscript 'z' refer to the cases where the circumferential flanges were not allowed to move in the longitudinal direction. Under the same loading conditions, the case with this additional restraint gave lower increase in bolt load, smaller joint opening at the outer edge location, smaller global horizontal movement measured at the roller support and lower principal tensile stresses in the longitudinal flange. These results, along with the in-tunnel measurements described in Chapter 3, will be used to guide the interpretation of the full-ring experimental data given in Chapter 7.

5.6 Influence of bolt preload

Figure 5-27 compares the influence of different magnitudes of bolt preload on the joint behaviour as predicted using FE analyses (LHS column) and as measured in the laboratory (RHS column). Despite the discrepancies between the predicted and measured responses which were mentioned in the

previous section, the overall influence of different preload on the local and global behaviour as predicted by the FE analyses captured the measured response reasonably well.

As mentioned previously and shown in Figure 5-21, in the laboratory tests the segment response was measured when the bending moment at the joint increased from 0.25 kNm to 0.45 kNm. Since the FE prediction matched the measured response reasonably well in this range, the FE prediction for local response and global response starting from zero bending moment is given in Figure 5-28 to help understand the influence of bolt preload.

Row 1 in Figure 5-28 shows that for the 5 kN and 7.5 kN preloaded edge bolts, the bolt load increased as soon as the segment self-weight was activated when the bending moment at the joint was 0.12 kNm. An increment of applied loading led to the greatest increase in bolt load for the bolt with the lowest initial preload. For the middle bolt the increase in bolt load was negligible for all three preload values.

Row 2 in Figure 5-28 shows that the preload magnitude affected the moment at which the joint began to open, as expected. However, comparing the graphs for the edge bolt in row 1 and row 2 would indicate that the bolt load started to increase *before* the joint began to open at the edge bolt location. One explanation could be that the joint started opening from the outer edge and therefore the load in the edge bolts increased even though the LVDT at the intrados in line with the edge bolt centreline had not measured any displacement yet.

The behaviour of the joint at the edge bolt location was investigated. The results are presented in Row 3 in Figure 5-28. The first graph in Row 3 plotted the bolt load in excess of the preload value against the joint opening at the edge bolt location and the three curves appeared to become parallel as the joint opening increased. This was further investigated by plotting the slope of each preload curve against the moment at the joint, also presented in Row 3. This confirmed that as moment increased, the slope of the preload curves trended towards the same value. The final graph in Row 3 plots the incremental joint opening divided by the incremental moment against the moment at the joint. This final graph in Row 3 of Figure 5-28 suggests that a moment greater than 1.2 kNm was needed before the preload ceased to have an effect on the magnitude of joint opening.

Row 4 in Figure 5-28 focuses on the influence of preload on the horizontal movement measured at the roller support, and also provides the predicted response of the arch without a joint for comparison. The first graph plots the roller movement against bending moment at the joint and the second graph shows the slope of the curves from the first graph against bending moment. It is clear that the *presence* of a bolted joint had far greater influence on the behaviour of the 'arch' than the *level of preload* in the bolted joint. With no joint in place, and assuming linear elastic material properties for the segment, ICPEP predicted a linear relationship between roller movement and bending moment. With a joint in place, this relationship became non-linear, with increasing roller displacement for each load increment; i.e., the segmental arch 'stiffness' reduced with increasing moment at joint. In the FE prediction using elastic material properties, this stiffness reduction appeared to taper off as the moment increased to near 0.45 kNm. However, the limited laboratory data indicated that the stiffness reduction did not taper off.

Row 5 in Figure 5-28 looks at the vertical displacement at the joint location. The vertical behaviour was similar to the horizontal movement in that ICFEP predicted a linear relationship between the vertical displacement and the bending moment when the joint was not modelled. The relationship became non-linear with the joint in place.

5.7 Comparison with Thomas' tests

In Thomas' two segment test (1977), the bolts used were 22 mm diameter and they were preloaded to 30 kN. This would be the equivalent of preloading a 12 mm diameter bolt to approximately 9 kN. Figure 2-14 shows Thomas' laboratory set up. Thomas provided results graphically for the moment at the joint in relation to the angle at the joint. This is given as Figure 5-29. However, it was not clear whether the joint opening was measured at the outer edge, or at the edge bolt location, or at the middle bolt location. As shown in the previous sections, the magnitude of joint opening is dependent upon the measurement location along the joint intrados.

Thomas' M-Theta graph was used to estimate a 'joint stiffness'. The moment was divided by the angle of opening at the joint and this was plotted against the moment at the joint, given by the solid black line in Figure 5-30. The moment vs joint opening relationship predicted by ICFEP at the outer edge location was also used to calculate a joint stiffness. Figure 5-31 shows how the angular opening was calculated from the ICFEP results.

Row 2 in Figure 5-21 shows that the ICFEP prediction at the outer edge location matched the laboratory measurements well for bending moment between 0.25 kNm and 0.45 kNm. The stiffness predicted by ICFEP in this range is shown as solid coloured lines on Figure 5-30. The stiffness predicted by ICFEP for bending moments exceeding the range tested in the laboratory is indicated by dashed coloured lines on Figure 5-30. Since the ICFEP prediction was for half-scale segments based on 12'6" tunnel lining, the 'full-scale' rotational joint stiffness expressed as kNm/rad from Thomas' tests on segments from 12'7" tunnel lining was multiplied by a factor of $\frac{1}{2}$ to be roughly comparable to ICFEP. The stiffness curves based on ICFEP results tended towards 180 kNm/rad as bending moment increased, approximately 3.5 times the stiffness estimated from Thomas' results.

The global horizontal and vertical displacements of the jointed arch partly arise through the deformation of the segments and partly through the rotation at the joint. The contribution of the joint rotation was isolated by performing a finite element analysis with the joint modelled as continuous i.e. with the end plates at the joint fully connected to each other. This analysis was compared with the previous analyses with the joint modelled and the differences in horizontal displacement at the roller and the vertical displacement at the joint between the two cases were attributed to the joint rotation.

Figure 5-32 shows that the segment modelled in the finite element analysis was assumed to undergo rigid body rotation. And since the angle of rotation was small, the angle of rotation, θ , was approximated to be $\tan^{-1}\left(\frac{dy}{L}\right) \approx \tan^{-1}\left(\frac{dx}{H}\right)$. The magnitudes for dy and dx were extracted from the FE analysis. In fact, calculations show that angles estimated from $\tan^{-1}\left(\frac{dx}{H}\right)$ were 10% larger than that

estimated using $\tan^{-1}\left(\frac{dy}{L}\right)$. The angle of rotation, θ , assuming rigid body rotation, was subsequently taken to be the average of $\tan^{-1}\left(\frac{dy}{L}\right)$ and $\tan^{-1}\left(\frac{dx}{H}\right)$.

Figure 5-33 compares the angle of joint rotation based on the assumption of rigid body rotation (Figure 5-32) with that estimated using the joint opening extracted at the outer edge location (Figure 5-31). The rotation estimated using joint opening was at most 20% higher than that estimated using rigid body rotation. However, this difference reduced to less than 10% as the bending moment increased to 0.46 kNm.

The rotational stiffness at the joint was calculated by dividing the moment at the joint by the angle of rotation. Figure 5-34 compares the stiffness values calculated using the two joint rotation estimates. This gives confidence that the method of using the joint opening to calculate the joint rotation was reasonable.

5.8 Change of loading direction

In the laboratory tests, the line load could only be applied vertically downwards. However, an ICPEP analysis was run to apply the line load upwards so that the joint would open at the extrados as shown in Figure 5-35.

The main differences in the joint behaviour between a positive and a negative moment are shown in Figure 5-36. For a negative moment, as the moment at the joint increased, the increase in load for the middle bolt was only marginally less than that for the edge bolt. For a positive moment, the middle bolt was barely loaded as the bending moment increased. This is corroborated with the predicted joint opening. For a negative moment, the three locations along the extrados, the outer edge, the edge bolt location and the middle bolt location, all opened to the same extent. For the positive moment, laboratory results confirmed the ICPEP prediction that the joint began to open at the outer edge and negligible movement was measured in the middle.

5.9 Mode of bending in the longitudinal flange

The results presented in rows 3, 4 and 5 in Figure 5-27 for joint opening at the intrados are used to help understand the deformation behaviour of the longitudinal flange as the moment at the joint increased. Figure 5-37 presents the increase in joint opening from a moment of 0.25 kNm to a moment of 0.45 kNm for the three preload values. The column on the left is the FE prediction and the column on the right is the measured response. The joint opening is plotted against the distance from the centreline on the intrados. As seen in Figure 5-38, the middle bolt location on the intrados coincides with the centreline of the longitudinal flange, i.e., $x=0$ on the graphs in Figure 5-37. M1 equals a moment of 0.25 kNm on all the graphs. On the graphs for laboratory tests, M4 equals the final moment of 0.45 kNm. On the graphs for FE analyses, M7 equals the final moment of 0.45 kNm. This is simply because the loading increments were smaller in the FE analyses.

The graphs in Figure 5-37 clearly show that the measured 'pinching' effect, i.e., the edge bolt inhibiting the opening of the joint, at the edge bolt was less pronounced than that predicted by FE. In fact, when

the initial preload was 5 kN there was hardly any pinching effect at the edge bolt location on the intrados. As the preload increased, laboratory measurements showed that this pinching effect increased.

Figure 5-39 takes into account the measured response and suggests the mode of bending in the longitudinal flange. In Figure 5-39 the longitudinal flange is drawn such that the red dashed lines indicate where the flange 'bends' into the page. For a given moment, a higher initial preload in the bolts encourages bending along the curved red dashed line. For a preload of 5kN, the bending is roughly along the straight dashed line.

5.10 Revised estimate of joint moment capacity for full-ring test

While a non-linear analysis would be necessary to find the true capacity of the joint as the stresses on the end plate are greater than the elastic limit, the available FE results could be used to provide a guide. Figure 5-40 shows the contours of displacement, maximum principal tensile and maximum principal compressive stresses in the longitudinal flange for cases of positive and negative bending from the FE predictions.

Using the strength of the grey cast iron obtained from unconfined uniaxial tensile tests and a tension strength cut-off criterion, the peak tensile stress in the dark pink area for positive moment would equal the ultimate tensile strength of 120 MPa at a moment of $120/150 \times 1.82 = 1.45$ kNm. This value is only slightly higher than the original estimate for a joint bending capacity of 1.12 kNm in Table 5-1.

For negative moment the peak tensile stress of 195 MPa in the dark pink area could only be relieved by plastic deformation. The capacity would scale down to $120/195 \times 1.66 = 1.02$ kNm if the stress in the dark pink area was restricted to 120 MPa. The original calculation, taking into account the contribution of three bolts rather than two (c.f. Section 5.8), would give a capacity of 0.74 kNm.

The strength of the joints would be enhanced by hoop stress in the bolted segmental ring when it is being loaded radially. In the planning of the full-ring laboratory tests, joint capacities of 1.12 kNm for positive bending and 0.74 kNm for negative bending were adopted, and the contribution of the hoop stress was included. The joints in the full-ring test were not subjected to bending moments beyond 40% of the enhanced joint capacities.

5.11 Results from strain gauges

The locations of the strain gauges on the segments are shown in Figure 5-8. Eight T-gauges were located on the circumferential flanges and the skin at an angle of 20 degrees clockwise away from the vertical joint. Six strain gauge rosettes were located close to the joint location.

Figure 5-41 gives the changes in strain as measured by the T-gauges corresponding to the change in bending moment at the joint from 0.25 kNm to 0.45 kNm. The strains measured by the gauges on both circumferential flanges were almost identical, giving confidence that the circumferential axis of symmetry assumed along the middle of the segment in the ICFEP model was reasonable.

In Appendix C the centroidal axis for the segment was calculated to be 16.5 mm away from the extrados. It was considered to be most appropriate to calculate the bending moment using the gauges at the furthest positions from the centroid of the section, i.e. the strain gauge on the skin near the junction with the circumferential flange and the strain gauge near the intrados of the circumferential flange. This is the series labelled 'Side A flange and skin' in Figure 5-41, which is the red line with the open square markers. A linear variation of strain from the extrados to the intrados was assumed. The resulting stress distribution for the series 'Side A flange and skin' is given in Figure 5-42. Figure 5-42 shows that there was very little difference between a stress distribution estimated assuming a constant modulus of 100 GPa, and a stress distribution using the best fit stress-strain curve as shown in Figure 4-27. The bending moment at 20 degrees from the joint where the strain gauges were located (see Figure 5-8) was estimated to be 0.12 kNm in both cases. The calculations for the bending moment are given in Appendix D.

The bending moment at 20 degrees away from the joint location could also be calculated using statics. When the moment at the joint increased from 0.25 kNm to 0.45 kNm, the change in bending moment at 20 degrees away from the joint was calculated to be 0.14 kNm. This is similar to the change in moment of 0.12 kNm estimated from strain measurements, giving confidence to the method of interpreting the strain gauge readings. The statics calculations are given in Appendix E.

The results from the strain gauge rosettes are presented in Figure 5-43. Using the change in strain from each strain gauge in a rosette, the principal major and minor direct strains and the orientation of the principal direct strain directions on the surface of the cast iron were calculated for each rosette location. As indicated on the graphs in Figure 5-43, the circumferential direction was assigned to be the x-axis. Out of the strain gauge rosette locations, the rosette located on the internal corner of the circumferential flange close to the joint measured the greatest tensile strain in the circumferential direction.

5.12 Distortion of the skin during positive bending

The distortion of the skin and circumferential flanges during positive loading was measured using two LVDTs and one displacement transducer as shown in Figure 5-13. The results are given in Figure 5-44. Positive values indicated the core of the LVDT or the displacement transducer moving outwards from the instrument. For the LVDTs, positive changes indicated the circumferential flanges moving towards each other, while for the displacement transducer positive changes indicated the skin bulging outwards. Therefore the measurements showed that Thomas' hypothesis was correct, and the segment deformed according to the schematic in Figure 2-11.

5.13 Discussion

A revisiting and careful review of the experimental data provided by Thomas (1977) contributed to the research at Imperial College in several ways. First, the measurements of the increase in bolt loads in Thomas' tests were taken into consideration when estimating an initial joint bending moment capacity in the planning of the two segment test for this research, namely, the inclusion of the action of only two of the three bolts and the assumption of double curvature of the longitudinal flange. This initial joint

capacity estimate allowed a series of parametric tests of varying preloads to be conducted safely and for the results to be interpreted under the assumption of elasticity.

Secondly, the type and location of the monitoring instrumentation were selected to build on the knowledge gained from Thomas' tests. The use of three LVDTs along the intrados of the joint, as well as the use of two LVDTs and one displacement transducer to monitor the distortion of the circumferential flanges and skin, both aimed to capture and quantify previously unmeasured displacements. It was shown conclusively that under positive moment and zero hoop stress, the joint moved apart starting at the circumferential flange. The upward bulging of the skin and inward movement of the circumferential flanges towards each other under positive bending, as described in Thomas (1977), was confirmed and measured during the testing at Imperial College.

Thirdly, the two segment test provided an opportunity to compare the bending moments estimated using strain measurements with analytical solutions. This gave an indication of the magnitude of error in the use and interpretation of the strain gauge readings. This finding is relevant to the full-ring test because the strain measurements will be used to determine whether the bending moment is reduced in a bolted segmented ring compared to a continuous ring when both are under the same loading.

Fourthly, revisiting the two segment test allowed a three-dimensional finite element model to be developed in conjunction with the laboratory experiment. In the early stages of laboratory testing and FE model development, the initial experimental results highlighted where refinement was necessary in the FE mesh, as well as the level of details required in the FE mesh, for example, the inclusion of the caulking groove in the mesh. On the other hand, the FE simulation gave confidence that it would be reasonable to analyse the loading on the segment as line loads. The main series of tests showed that the stiffness of the bolting system assumed in the FE model was too high. The actual bolting system of bolts, nuts, washers, and the contact between them rendered the stiffness of the system several times lower than the stiffness of the mild steel bolt. Since the stiffness of the bolting system affected how the FE model predicted the deflection of the longitudinal flange, this was a valuable finding for future FE modelling of segmental ring behaviour.

In general the FE predictions compared well with the laboratory measurements. The FE model proved helpful in giving a guide to the behaviour of the two segment arch beyond the loading conditions tested in the laboratory. An important prediction was that when the joint was subjected to negative bending in the FE model, all three bolts had similar increases in bolt load and the joint opened uniformly along the extrados, i.e. the displacement at the outer edge was predicted to be similar to the displacement at the middle of the segment. This led to a revision of the joint moment capacity estimation for the full-ring test where joints were subjected to both positive and negative bending.

Additionally, the FE analyses allowed an interrogation of the principal tensile and compressive stresses in the longitudinal flange. Since the analyses were linear elastic the results could only provide a guide to the capacity of the joint. Nonetheless by limiting the tensile stresses in the longitudinal flange as predicted by the FE simulations to below the ultimate tensile capacity of cast iron, it was shown that the initial estimate of joint moment capacity was reasonable. Non-linear analysis would be necessary to determine the true capacity of the joint. The joint bending capacity values based on

elasticity are lower limit estimates. Non-linear deformation would allow a higher joint moment capacity to be achieved.

Another benefit of revisiting the two segment test was the chance to study the effect of adding grommets to the bolting system. The effect of this was not previously quantified and also difficult to model in the FE simulation, since the properties of the grommets were largely unknown. The laboratory experiments were able to show that the main effect of including the grommets was that the bolt preload would decrease over time. This led to the conclusion that if grommets were used in real tunnels then the level of preload in the tunnel bolts would be significantly lower than the bolt preload in tunnels without grommets.

6 Development of full-ring setup and planning of tests

6.1 Design of test rig components

Since the inception of the research project, it was envisaged that the laboratory tests would be performed on a single test ring comprising of six GCI segments, bolted together and placed on a specially prepared structural floor in the Structures Laboratory so that the radial plane of the ring is horizontal. The ring would be supported in such a way so as to minimise friction between the ring and the floor. It would be surrounded by a 'reaction ring'. Radial loading of the ring would be via 18 computer-controlled actuators installed between the reaction ring and extrados of the test lining so that full control can be achieved. This would not be possible if loading were through a soil medium.

The advantages to this approach are seen as follows:

- There is always great uncertainty about how well stresses applied to boundaries in small-/medium-scale models replicate conditions at prototype scale when the stresses are applied using soil. Much depends on the soil type, method of placement, homogeneity, consolidation, applied boundary stresses to name a few.
- Using the approach described allows the conditions within the ring for different deformed shapes to be investigated clearly without ambiguity.
- The tests can be performed relatively quickly compared with experiments involving soil, especially at this large scale.

The various components of the testing rig were developed and fine tuned based on the results from numerous trial tests. The initial trial tests involved three actuators only. As confidence with the control system increased, the complexity of the trial tests increased. More components were added and the loading procedure was being continuously modified until the test ring could be controlled with the desired stability and the applied deformations were repeatable.

The following sections describe the various components. A schematic drawing showing the full-ring test setup is given in Figure 6-1. Figure 6-2 shows the details of the loading rig components. A photo of the actual experimental setup is given in Figure 6-3.

6.1.1 Actuators

The test ring was placed horizontally and the distortions were achieved by actuators evenly spaced around the circumference of the test ring.

Sixteen 14.5" diameter Norgren stainless steel air bellows were used as actuators. Each actuator weighed approximately 20 kg. Each actuator was fitted to a hanging frame and positioned onto the reaction ring at 20° spacing to simulate distributed load similar to ground stresses. Once in position they were filled with water since preliminary tests indicated that better control could be achieved with a water-filled system.

The maximum operating pressure for the air bellows was 8 bar, corresponding to an applied load of approximately 80 kN. During the experiments the operating air pressure was kept at 6.5 bar. A manostat was used to control the maximum pressure entering the system.

Figure 6-4 shows how the air supply from Imperial College mains was de-humidified so that dry air was used in the test system. Figure 6-5 shows the system devised to fill and empty the water sump which supplied water to all the actuators. Figure 6-6 gives details of the air/water interface for each of the sixteen actuators.

At two locations around the ring, the actuators were removed and replaced with reaction rods each fitted with a load cell. This was done to prevent rigid body motion of the test ring. This was the reason why in the final setup there were only 16 actuators. At the location opposite to the two reaction rods, a tangential member was fitted to the test ring to increase the ring stability. The long tangential member is shown in Figure 6-7. It did not restrict the radial movement either towards or away from the centre of the ring but it prevented the ring from rotating.

As mentioned, the philosophy behind testing without soil was that ambiguity relating to magnitude and sense of applied loading would be removed. It allowed for more rapid test setups compared to tests with soil loading which could take several months for each loading stage. Loading by actuators improved repeatability and enabled better control through the use of stepper motors.

6.1.2 Stepper motor

The pressure in each actuator was controlled by a stepper motor linked to the control program, Triax. Pressure transducers monitored the pressure the stepper motors were feeding into the actuators.

The actuators were controlled by specifying the target load or the target displacement or by specifying the rate of increase or decrease in load or displacement.

6.1.3 Reaction ring

The 3.0 m internal diameter reaction ring was fabricated by Morgan Sindall from grade S235 steel to replicate a 254x254x107 universal column section bent about the major axis.

Some very preliminary calculations were performed to determine a suitable section for the reaction ring. The reaction ring was designed so that it is able to resist a bending moment 17 times the bending moment capacity of the GCI test ring at the joint assuming an overburden depth of 20 m. For a continuous circular ring under any number of equal radial forces equally spaced, linear elastic solutions could be used to calculate the hoop stress, bending moments and deflections of the ring. The theorem indicated that the bending moment in a ring under any number of equally spaced radial forces was directly proportional to its radius. All the relevant formulas were found in Roark's Formulas for Stress and Strain (Young and Budynas, 2002), and outlined in Table 6-4. Given the ratio of the radii of the reaction ring to the test ring is 1.7; there was a factor of safety of 10 in terms of strength for the reaction ring.

The mechanical testing on GCI specimens was not yet performed at this stage and therefore in the calculations the ultimate tensile strength for the grey cast iron was assumed to be 150 MPa. The methodology of estimating the bending capacity of the joint was the same as that outlined in Table 5-1. It was also assumed that the loading from approximately 20 m of overburden pressure would be applied to the test ring which would enhance the moment capacity of the joints and

segments. Therefore the reaction ring was sized taking into account this larger capacity of the test ring. The calculations suggested that the lightest 254UC was more than adequate. However, it was more convenient for our supplier to source a heavier section. All calculations are found in Appendix F.

6.1.4 Roller support

The test ring rested on a roller system shown in Figure 6-7. Preliminary tests highlighted that a roller system was better than using PTFE sheets in providing a more frictionless support. When the test ring was supported on rollers, there was negligible static friction, and the ring moved upon contact with an actuator.

6.1.5 Spreader pads

The spreader pads transferred the load from the actuator to the extrados of the test segment. The spreader pads consisted of a piece of hardwood 25 mm thick machined to have the same external radius as the test ring. The hardwood was attached to a mild steel plate (250 mm x 250 mm x 15 mm thick). Each actuator had its own spreader pad. A layer of 3 mm thick butyl was glued onto the machined hardwood surface to ensure full contact between the spreader pad and the test ring even as the test ring was distorted.

6.1.6 Hanging frame for actuators

Hanging frames were used to secure the actuators onto the reaction ring (see Detail A of Figure 6-2). The frames provided the flexibility to adjust the positions of the actuators. The structural capacities of different components of the hanging frame were checked. All calculations are found in Appendix G.

6.2 Instrumentation

The instrumentation used in the full-ring test included electrical resistance strain gauges, strain gauged displacement transducers, pressure transducers, load cells, linear variable differential transformers (LVDTs), instrumented bolts and a temperature sensor. Each of these will be discussed in the following sections. The descriptions below also apply to the instrumentation used in the two segment test described in Chapter 5.

6.2.1 Electrical resistance strain gauges

Two types of universal general-purpose strain gauges were used to measure changes in strain on the surface of the cast iron segments. The rectangular rosette gauge consisting of three elements at 45° single plane, and the tee rosette consisting of two elements at 90° single plane. Each element, or gauge, has a resistance of $350\Omega \pm 0.4\%$ and a strain range of $\pm 5\%$. The Self-Temperature-Compensation (STC) of the gauges was 06 (in ppm/°F) to match the thermal expansion coefficient of grey cast iron which is around 10 to 12 ppm/K, i.e., 5.5 to 6.6 ppm/°F (Gilbert, 1977). The rectangular rosette matrix dimension is 16.5 mm by 20.3 mm; the tee rosette matrix dimension is 14.0 mm by 18.8 mm. Figure 6-8 gives magnified images of the strain gauge rosettes.

The three element rosettes were selected for installation close to the junctions of the skin and the flanges of the segment. At these locations, the orientation of the direct principal strains on the cast iron surface is not obvious, and the use of three element rosettes allows the orientation of the direct

principal strains to be calculated. On the circumferential flanges, tee rosettes were used and the principal direct strains were assumed to be circumferential and radial (see Figure 6-11 for the orientations).

Figure 6-9 and Figure 6-10 show the use of strain relief wires to prevent the leadwires pulling onto the rosettes directly. The leadwire was a flat cable of tinned stranded copper, insulated with vinyl.

The strain gauged locations are given in Table 6-1 and Figure 6-11 to Figure 6-13. As mentioned above, the three element rosettes were positioned near the junctions of the flanges and skin so that the orientations of the direct principal strains could be calculated. The tee rosettes were used to measure the changes in circumferential strains to facilitate the calculation of bending moments at the strain gauged locations. The segment located opposite to segment I was also strain gauged, but to a lesser extent, primarily to confirm if the ring was behaving symmetrically. This segment was labelled segment G and the strain gauge locations are given in Figure 6-14.

The leadwire for each strain gauge in a rosette was connected to the relevant channel on the datalogger to be used in the laboratory tests. Each datalogger channel was calibrated by connecting the leadwire to the '1550A Strain Indicator Calibrator'. The 1550A Calibrator is a true Wheatstone-bridge simulator, and can present known and repeatable resistance changes to the input of the indicator. Push buttons were used to produce incremental resistance changes. Each channel was calibrated up to 5000 $\mu\epsilon$. The manufacturer stated an accuracy of $\pm 1 \mu\epsilon$ for the calibrator.

After the channels were all calibrated, the leadwires were soldered onto the terminals on the segments. The stability of all the strain gauges over a time period of three minutes was checked. During the laboratory testing, the strains were measured for two to three minutes after an appropriate 'stage'. Extensive baseline checks were performed prior to the commencement of the tests. It was found that for all but three gauges, the stability was well within $\pm 1 \mu\epsilon$. The remaining three gauges had stability of $\pm 3 \mu\epsilon$, $\pm 7 \mu\epsilon$ and $\pm 9 \mu\epsilon$. However, linear trendlines for all the stability data showed a drift of less than $\pm 1 \mu\epsilon$ over the measurement period. It is considered that using the average strain measurement over three minutes would provide an accurate strain value to $\pm 1 \mu\epsilon$.

Baseline readings were taken before any tests were conducted and it was during this time that certain strain gauge channels were discovered to be logging erratic behaviour, while the ring was at rest. Exhaustive investigations were undertaken to identify the cause, and checks were made on the strain gauges, the dataloggers and the leadwire connections. Finally the dataloggers were sent back to the manufacturer to have the circuit boards cleaned. As a precaution, further baseline scans were performed after the circuit boards have been cleaned to confirm that all the strain gauge channels were logging correctly.

Segment	Rosette type	Locations
H	Rectangular	Skin extrados: 4 no.s Skin intrados: 4 no.s Circumferential flange external: 2 no.s Circumferential flange internal: 2 no.s
	Tee	Skin extrados: 2 no.s Skin intrados: 2 no.s Circumferential flange external: 2 no.s
I	Rectangular	Skin extrados: 4 no.s Skin intrados: 4 no.s Circumferential flange external: 2 no.s Circumferential flange internal: 2 no.s
	Tee	Skin extrados: 2 no.s Skin intrados: 2 no.s Circumferential flange external: 2 no.s
J	Rectangular	Skin extrados: 4 no.s Skin intrados: 4 no.s Circumferential flange external: 2 no.s Circumferential flange internal: 2 no.s
	Tee	Skin extrados: 2 no.s Skin intrados: 2 no.s Circumferential flange external: 2 no.s
G	Tee	Skin extrados: 2 no.s Skin intrados: 2 no.s Circumferential flange external: 2 no.s

Table 6-1. Strain gauged locations (see Figure 6-11 to Figure 6-14).

6.2.2 Strain gauged displacement transducers

Strain gauged displacement transducers were used to measure the radial displacements around the cast iron ring. The top diagram in Figure 6-14 shows how each displacement transducer is aligned with the midpoint of a loading pad. Of the eighteen displacement transducers, fourteen had a full range of 25 mm and four had a full range of 50 mm. Figure 6-15 shows a typical displacement transducer and Figure 6-16 shows the strain gauges making up the four-arm Wheatstone bridge responsible for giving linearly proportional voltage output in relation to the movement of a captivated spindle (plunger rod).

The strain gauged displacement transducers were individually calibrated with a micrometer (Figure 6-17). The 25 mm range displacement transducers were calibrated for 12.5 mm of forwards and backwards travel and the 50 mm range displacement transducers were calibrated for 25 mm of forwards and backwards travel.

The manufacturer quoted non-linearity and repeatability of less than $\pm 0.1\%$ of the full range, i.e., 25 microns for the 25 mm range transducers and 50 microns for the 50 mm range transducers. An analysis of the calibration data revealed that the maximum error for the 25 mm range transducers was 14 microns, 0.06% of the full range. The maximum error for the 50 mm range transducers was 28 microns, also 0.06% of the full range.

Figure 6-18 shows the tips of the strain gauged displacement transducers aligned with the intrados of the lower circumferential flange of the test ring.

In total eighteen displacement transducers were used to monitor the radial displacement of the test ring at 20 degree intervals. One displacement transducer was used to detect any rotation of the test

ring so the rotation could be corrected and prevent the test ring from spinning out of alignment during the initial loading steps when not all the actuators were under computer control (Figure 6-19). There was an additional displacement transducer to monitor the convergence/divergence of the top and bottom circumferential flanges (Figure 6-20).

6.2.3 Pressure transducers

Pressure transducers as shown in Figure 6-21 were used to measure the water pressure applied to each actuator. Inside the pressure transducer is a ceramic diaphragm that has been strain gauged to form a four-arm Wheatstone bridge. The selected pressure transducers can measure from zero to 10 bar pressure.

During calibration, pressure was fed to the pressure transducer via the control of a manostat. Each pressure transducer was calibrated against a Budenberg gauge (Figure 6-22). The calibrated range was up to 750 kPa.

The manufacturer quoted non-linearity, hysteresis and repeatability of $\pm 0.25\%$ of the full range, i.e., 2.5 kPa. An analysis of the calibration data revealed that the actual errors were larger than that quoted by the manufacturer. The maximum error was 7.9 kPa, or approximately 1.0% of the full range. The average error was 5.0 kPa.

Sixteen pressure transducers were used, one for every actuator. The pressure transducer was located downstream of the air/water interface and monitored the water pressure entering the actuator (Figure 6-23).

6.2.4 Load cells

A load cell was fitted to each actuator and to the two reaction rods to measure the load applied to the segment. Pancake load cells with up to 100 kN capacity were used (Figure 6-24). The load cells were strain gauged internally, and the gauges were configured to operate based on measurement of shear strain.

The 100 kN load cells were subjected to 50 cycles up to full load prior to being calibrated. The load cells were calibrated with a Budenberg dead weight tester up to 20 kN. The calibration within this range was extrapolated to higher loads.

The manufacturer quoted non-linearity, hysteresis and repeatability to be less than $\pm 0.05\%$ or the rated capacity, i.e., 0.05 kN. Analysis of the calibration data showed that the maximum error was 0.076 kN, or 0.46% of the calibrated range.

In total, eighteen load cells were used. The location of the load cell on an actuator is shown in Figure 6-23.

6.2.5 Linear variable differential transformers

Type D5/400K LVDT displacement transducers were used to measure the opening and closing of the horizontal joints. A typical LVDT displacement transducer is shown in Figure 6-25.

The manufacturer quoted non-linearity of less than $\pm 0.5\%$ of full scale, i.e., 50 microns.

The LVDTs were calibrated for 2mm of forwards and backwards travel with the micrometer shown in Figure 6-17.

The theoretic resolution of the LVDT was calculated as follows:

- Resolution of data logger = 320 μ V
- Volts per mm output: 2V
- $320\text{e-}6 / 2000 = 160\text{e-}9\text{ m}$ i.e. 0.16 microns.

Analysis of the calibration data showed that the maximum error was ± 9 microns, or 0.24% of the calibrated range.

Figure 6-26 shows the locations of the LVDTs at the intrados and extrados of an instrumented longitudinal joint. The LVDTs were aligned with the middle bolt, the edge bolt and the outer edge.

6.2.6 Instrumented bolts

Eight Grade 4.6 M12 bolts were instrumented at Imperial College for the full-ring test. Each instrumented bolt had two small sections of the unthreaded portion planed off and two strain gauges were attached to each planed surface to form a four-arm Wheatstone bridge as seen in Figure 5-14.

The instrumented bolts were cycled up to 25 kN ten times each before calibration. Calibration was done in tension in the 250 kN Instron testing rig but using a 25 kN Instron load cell as reference (Figure 6-27). Calibration data showed that the maximum error was 0.085 kN. Eight instrumented bolts were used to measure the changes in bolt load at the middle bolt and edge bolt locations on segments H, I and J (Figure 6-28).

6.2.7 Temperature sensor

The temperature sensor was kept in contact with the cast iron test ring to continuously monitor the changes in temperature as the tests progressed. As mentioned, a scan of 2 to 3 minutes would be taken as a snapshot of the relevant loading stages, and the temperature fluctuations within the scan duration would be negligible to cause any drift in data. The other concern was that tests carried out on different days with differences in temperature would not be directly comparable. Data from the tests conducted on the days with the maximum temperature deviation from the operational temperature of the strain gauges were studied. Applying temperature corrections to the data in these instances did not lead to any difference in the final computed values of the bending moments to two decimal places. Therefore it was deemed unnecessary to apply temperature corrections to the strain gauge data.

6.3 Loading procedure

It was discussed in Chapter 4 and Chapter 5 that any loading would be limited so that the cast iron bolted segmental ring was not stressed beyond 40% of its estimated bending moment capacity. In the two segment test, the moment capacity at the joint was not enhanced by hoop stress. The estimated joint and segment capacities calculated in Chapter 5 are given again in Table 6-2.

Joint capacity, BM_J^+ (kNm)	Positive bending	1.12
Joint capacity, BM_J^- (kNm)	Negative bending	0.74
ult Section capacity (kNm)	Positive bending	3.45
ult Section capacity (kNm)	Negative bending	9.54
40% Section capacity (kNm)	Positive bending	1.38
40% Section capacity (kNm)	Negative bending	3.82

Table 6-2. Estimated BM capacity for joint and segment under zero hoop load.

The variation in the bending moment capacity of the segment as the hoop load increases is shown in Appendix C. The bending moment capacity of the joint also increases as the hoop load increases. This additional moment capacity at the joint is afforded by the hoop load in compression, N , applied at maximum eccentricity, e , from the centroidal axis of the segment to the extreme compression fibre. Table 6-3 shows the total moment capacity at the joint under different depths of overburden pressure.

Depth below ground	Load per jack	Hoop load, N	Moment capacity from hoop load, Ne		Total Moment Capacity at the Joint, $BM_J + Ne$		40% Total Moment Capacity at the Joint	
m	kN	kN	kNm		kNm		kNm	
			+ve	-ve	+ve	-ve	+ve	-ve
2	3.4	9.7	0.16	0.35	1.28	1.07	0.51	0.43
6	10	29.0	0.48	1.05	1.60	1.77	0.64	0.71
12	20	58.1	0.96	2.09	2.08	2.81	0.83	1.12
24	40	116.1	1.91	4.18	3.03	4.90	1.21	1.96
Positive bending. Eccentricity, $e = 16.5$ mm. Centroidal axis to extrados (extreme compression fibre).								
Negative bending. Eccentricity, $e = 36.0$ mm. Centroidal axis to tip of caulking groove.								

Table 6-3. Estimated BM capacity for joint including the contribution from hoop load.

Figure 6-29 compares the bending capacities of the segment and the joint calculated at different depths using 40% of the ultimate tensile strength of grey cast iron. It shows the changes in the bending capacities as the hoop load increases with increasing overburden depth. A depth range of 20 m to 30 m is the most relevant to the bolted segmental grey cast iron tunnels in the LU network. From Figure 6-29, it can be seen that in this depth range, the joint bending capacity is lower than the segment bending capacity. The joint capacities have been calculated for positive (see Figure 2-11) and negative bending (see Figure 2-12)

In the full-ring laboratory experiment the test ring was loaded from eighteen locations evenly distributed around the ring to bring it under hoop stress corresponding to the depths given in Table 6-3. Only the loads at axis level were then adjusted to unload the ring (P^-) and increase the radius, or to load the ring (P^+) and decrease the radius, as shown in Figure 6-30. All subsequent reference to the increase or decrease in radius refers to the change in radius at axis level, i.e., at 90 degrees. As seen in Figure 6-30, the normal loads in all the actuators away from the axis remained unchanged from Stage One to Stage Two after the Stage One normal load magnitudes were achieved.

The application of equal normal loads in Stage One was to attain a certain magnitude of hoop force. In Stage Two, increasing or decreasing the load along the axis produces bending moment in the ring. The rationale behind this loading regime will be further discussed at the end of this chapter.

In order to plan the loading and unloading regime, the linear elastic analytical equations for a continuous ring were used to determine the extent to which the radius of the bolted segmental ring should be increased or decreased such that the bending moment was kept below 40% of the bending capacity of the segment or the joint, for the most critical case, i.e. when the hoop force was the lowest. It was also decided to decrease or increase the radius to the same magnitude while under different hoop stresses; so that the impact of the different hoop stress levels on the ring behaviour could be compared. The planned distortion of 0.1% to 0.2% for the test ring is similar to the distortion measured in Central Line as a result of the Crossrail excavations as well as the historical maximum distortion of 0.1% cited in Chapter 2 for cast iron tunnels affected by adjacent excavations.

Given the assumption of linear elastic behaviour and uniform stiffness of the ring, the bending moments corresponding to a given change in radius are independent of hoop force, and only dependent on the change in load at the axis level. The application of load to the segment via a loading pad instead of as a point load was taken into account by superimposing the bending moment distributions from a number of point loads over the arc of 15° subtended by the loading pad. This is illustrated in Figure 6-31. The equations used to calculate the bending moment are given in Table 6-4.

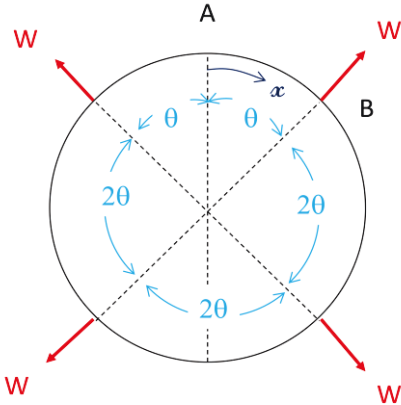
W = load; E = modulus of elasticity; A = cross sectional area; R = radius to the centroid of the cross section; I = second moment of area of the ring; s = sin θ; c = cos θ; z = sin x; u = cos x. Correction for thick rings not necessary, correction factors $k_1 = k_2 = 1$.			
	For $0 < x < \theta$	$M = \frac{WR(u/s - k_2/\theta)}{2}$	Equation (12)
	Max +M = M_A	$M = \frac{WR(1/s - k_2/\theta)}{2}$	Equation (13)
	Max -M at each load position	$M = \frac{-WR}{2} \left(\frac{k_2}{\theta} - \frac{c}{s} \right)$	Equation (14)
	Radial displacement at each load point = ΔR_B	$\Delta R_B = \frac{WR^3}{EI} \left(\frac{k_1(\theta - sc)}{4s^2} + \frac{k_2c}{2s} - \frac{k_2^2}{2\theta} \right)$	Equation (15)
	Radial displacement at $x=0, 2\theta, \dots = \Delta R_A$	$\Delta R_A = \frac{-WR^3}{EI} \left(\frac{k_1(s - \theta c)}{4s^2} - \frac{k_2}{2s} + \frac{k_2^2}{2\theta} \right)$	Equation (16)

Table 6-4. Linear elastic equations for ring under any number of equal radial forces equally spaced (Young and Budynas, 2002).

Figure 6-32 compares the bending moment distributions obtained using a point load analysis with that obtained by distributing the point load over the arc subtended by the loading pad. Both the loading and unloading cases are shown. Figure 6-33 shows the corresponding displacements. By distributing the load, the 'peak' at the axis location has been smoothed over. This is considered to be more representative of the loading conditions in the laboratory. The 40% bending moment capacities at the joint locations are also shown on Figure 6-32. These capacities corresponded to normal loads of 3.4 kN applied in Stage One. The estimated bending moments for the joints at zero and 180 degrees under the imposed distortion of 1 mm are close to the 'elastic' capacity for the joints. The distortion for

all the loading cases was set to 1 mm to allow for comparison for the effects of overburden and bolt preload. However, for the cases with higher Stage One normal loads, this imposed distortion would result in bending moments at the joints that are well within the elastic range.

During the laboratory testing period, analysis of field data collected from the Central Line instrumentation described in Chapter 3 indicated that the increase in span in the real tunnel was up to 4 mm, i.e. 2 mm over the radius. Realising this, checks were made to see whether this larger distortion could be attained in the laboratory for the unloading tests with initial normal loads greater than 3.4 kN, without stressing the cast iron beyond 40% of its tensile strength. This was found to be possible and therefore the unloading tests conducted at normal loads above 3.4 kN in the laboratory were allowed 2 mm of distortion over the radius.

Table 6-5 outlines the unloading regime for the full-ring tests and Table 6-6 outlines the loading regime for the full-ring tests. It is considered that the unloading tests would provide more relevant information to the tunnels in-situ, because excavations adjacent to the existing tunnels would generally cause the existing tunnels to unload.

Decrease in load => Increase in radius		Displacement away from centre of ring (mm)			
All-round actuator load (kN)	Approx depth below ground (m)	Bolt 7.5kN	Bolt 5kN	Bolt 10kN	Bolt 5kN Grommet
3.4	2.0	1.0	1.0	1.0	1.0
10.0	6.0	2.0	2.0	2.0	2.0
20.0	12.0	2.0	2.0	2.0	2.0
40.0	24.0	2.0	2.0	2.0	2.0

Table 6-5. Unloading regime for main series of tests.

Increase in load => Decrease in radius		Displacement towards centre of ring (mm)			
All-round actuator load (kN)	Approx depth below ground (m)	Bolt 7.5kN	Bolt 5kN	Bolt 10kN	Bolt 5kN Grommet
3.4	2.0	1.0	1.0	1.0	1.0
10.0	6.0	1.0	1.0	1.0	1.0
20.0	12.0	1.0	1.0	1.0	1.0
40.0	24.0	1.0	1.0	1.0	1.0

Table 6-6. Loading regime for main series of tests.

A combination of preliminary tests and analysis was used to identify the best procedure to bring all the actuators to equal loads before distorting the ring. All the details are provided in Appendix H. In short, one loading increment involved increasing the loads on all three actuators on three alternate segments, while setting the actuators on the remaining segments to hold their current displacement. The load and displacement controls were then swapped over for the next loading increment, so that the actuators previously under load control would be set to hold their current displacement while the remaining actuators would increase load. This would continue until the desired all-round load was reached. The magnitude of each increase in loading increment depended on the resultant bending moment induced in the segment and joints. This was estimated using linear elastic solutions as explained above. Since linear elasticity was assumed, the principle of superposition was used to attain the required loading configurations. Once again care was taken to ensure the tensile stress in the extreme fibre would not exceed 40% of the ultimate tensile strength of the cast iron.

It should be mentioned that 40 kN was selected to be the maximum normal load for the series of tests for two main reasons. First, this load roughly corresponds to an overburden depth of 24 m, which is representative of the depth where bolted cast iron tunnels of the LU network lie. The other reason relates to the practicality of the test duration. It took roughly 5.5 hours to bring the test ring up to this uniform load. A further 2 hours was required to perform the three unloading tests and another 1.5 hours for the three loading tests. A 9-hour day with constant supervision was required for the set of experiments. It was considered too risky to leave the actuators loaded unsupervised, let alone overnight. Therefore normal loads higher than 40 kN were not attempted.

In the unloading tests, after the all-around actuator loads were achieved, the actuator at the axis level on segment I (c.f. Figure 6-30) would be placed under displacement control to move away from the centre and the actuator load would decrease. The remaining actuators would continue to hold the previously specified load, thus allowing the ring to distort. The actuator at axis level on segment G (c.f. Figure 6-30) was under load control to be holding the average of the measured reaction loads on either side of it on segment G. As the loading actuator on segment I decreases its load, it was assumed that the reaction loads measured at the two reaction rod locations on segment G would also decrease, and therefore the actuator at the axis level on segment G would decrease its load. Similarly, for the loading tests, the actuator at the axis level on segment I (c.f. Figure 6-30) was put under displacement control to move towards the centre and the actuator load would increase.

For the first series of tests, the bolt preload in all eight of the instrumented bolts were adjusted with a torque wrench to be approximately 7.5 kN. The average torque measured was used as a guideline to adjust all the remaining bolts around the ring. The all-round load was increased from zero to 3.4 kN. When the all-round load was reached, a scan of all the instrumentation was taken for 2 to 3 minutes to obtain a 'snapshot' of the readings. The radius would then be increased by the magnitude given in Table 6-5. Once the desired distortion was reached, a scan of all the instrumentation would be taken for another 2 to 3 minutes. After this the ring was returned to the uniform state by setting all the actuators back to hold 3.4 kN. Another scan was taken and the ring was distorted once more. When three sets of 'unloading' readings were taken, three sets of 'loading' readings were taken. When all six sets of readings were taken for a specific hoop stress and a specific bolt preload, the ring was unloaded back to zero load.

The next series of tests conducted with the bolts preloaded to approximately 7.5 kN involved reaching a different hoop stress prior to ring distortion. When distortion under all four magnitudes of hoop stress had been completed, the ring was unloaded to zero and the bolts re-tightened to a different preload. Similar to the two segment tests, the bolt preload magnitudes were 5 kN, 7.5 kN, 10 kN and finally, 5 kN with grommets.

6.4 2D finite element modelling

While the author was responsible for the experimental structural investigation of the effects of adjacent excavations on existing tunnels, another PhD researcher at Imperial College, Vasileios Avgerinos, was

investigating the topic with finite element modelling using ICFEP. This section highlights some findings from his FE analyses and the implications for laboratory testing.

In the 2D FE analysis discussed in his PhD thesis, Avgerinos (2014) modelled the existing Central Line near Lancaster Gate, at the location described in Chapter 3. Avgerinos modelled the excavation of the Central Line and allowed 100 years of ground consolidation in his model to compare his simulations with the present day condition of the Central Line. The soil stratigraphy, soil properties and groundwater conditions input were based on the latest findings from the Crossrail ground investigations.

He noted that when no interface elements were introduced between the soil and the tunnel lining, i.e., when there was full bond between the soil elements and the lining shell elements, the deformed shape of the tunnel lining was governed by the shear stresses acting on the lining from the surrounding ground. In this scenario, the variation in hoop force around the lining was up to 30% when the lining was subjected to soil stresses arising from an overburden of approximately 22 m. When interface elements with zero shear stiffness were introduced, the deformed shape of the tunnel lining was governed by the normal stresses acting on the lining. The hoop force distribution was also much more uniform with the maximum difference around the lining being approximately 4%.

The historical measurements described in Chapter 2 do not provide sufficient evidence to determine how the ground stresses are transferred to tunnel linings. The FE modelling indicated that this unknown factor affects tunnel behaviour. However, the experimental investigation was limited to the application of normal stresses on the half-scale test lining.

It is also worthy to note that in the FE analyses, the normal stresses acting on the tunnel lining were always found to be lower than the full overburden pressure after the 100 year consolidation period. When the lining was modelled as impermeable by applying a no flow boundary around its perimeter, the normal stresses around the tunnel lining after excavation and a period of 100 years of consolidation were approximately 75% of the overburden pressure.

The reduction in normal stresses was more pronounced when the tunnel lining was modelled as permeable. When the lining was considered fully permeable the precipitation boundary condition in ICFEP was applied. The precipitation boundary condition monitored the pore water pressures at the excavation boundary nodes. If the soil pore water pressures became tensile then the boundary was prescribed as no flow, thus preventing the unrealistic situation where water from the excavation would flow into the soil. If the pore water pressures at the boundary were compressive, the boundary pressure was set to zero allowing water flow into the tunnel. With the lining modelled as permeable the normal stresses calculated at the tunnel boundary were only around 50% of the overburden pressure after a consolidation period of 100 years.

Another very important finding from the 2D FE modelling was the maximum distortion of the existing tunnel was found to be 0.2% after 100 years of consolidation, for the case of the lining modelled as fully permeable. When the tunnel lining was modelled as impermeable the distortion was significantly lower. This would suggest that tunnel deformation as a result of ground loading alone is significantly

lower than the 1% ovalisation measured in the LU tunnels as cited in Chapter 2. It is possible that for existing tunnels the out-of-circularity would have occurred during the construction of the tunnel rings, and under self-weight, prior to any ground loading being transferred to the lining. In Chapter 3 it was discussed that a section of tunnel constructed above ground at the LU Acton Depot had vertical shortening of 1.63% and horizontal lengthening of 1.23%.

Further 2D FE analyses by Avgerinos (2014) were run to check the impact of applying the normal stresses onto the lining via discrete loading pads as opposed to continuously around the ring. The FE model consisted of structural elements and the appropriate boundary conditions only, no soil elements were used. Only a quarter of the ring was modelled as shown in Figure 6-34. The magnitudes of normal stresses applied to the structural FE models were based on the normal stresses acting on the tunnel lining in the soil/structure interaction FE model where the shear stiffness of the interface elements at the extrados of the tunnel was set to zero. The total load over each 20 degree arc of the lining was calculated and re-distributed over 15 degrees, the arc span covered by each loading pad. The FE results showed that modelling the normal stresses at discrete pad locations only modified the response of the lining slightly. Under the soil loading, the soil/structure interaction model gave a squat of 0.24% of the tunnel radius. The structural model with normal loads at loading pad locations gave a squat of 0.28% of the tunnel radius (Avgerinos, 2014).

6.5 Discussion

The design of the test rig was informed by findings from the two segment test. All the components were sized up based on the requirements of the full-ring test. Numerous trial tests were performed to optimise the design of the system and to determine the most appropriate method to control the loading of the test ring, namely the decision to fill the actuators with water and the use of roller pads to reduce the friction between the ring and the supports. The decision to apply load with actuators and to eliminate soil from the setup at the outset was crucial to the success of the experiments. It would not have been possible to conduct the same number of parametric tests within the time-frame if soil was involved. The control of the loading would also have been a lot less precise if soil was involved.

However, during preliminary testing it became obvious that some compromises had to be made to the setup to allow better loading control. Two actuators were removed and replaced with reaction rods to increase the stability of the test ring during loading and to prevent free body movement. An additional tensile member was installed tangentially to the ring to enhance the stability. The removal of two actuators limited the number of ways the ring could be deformed in a symmetrical manner.

Results from the two segment test provided useful input with regards to the placement and interpretation of the strain gauge measurements. LVDT displacement transducers were positioned on both the intrados and the extrados of the instrumented joints based on the information gleaned from the two segment tests. Great care was taken during the installation and calibration of the instrumentation prior to commencing the full-ring test. Baseline readings of all the instruments were also taken every couple of weeks during the testing period to make sure that all the channels were functioning properly.

The loading procedure was designed to bring the test ring under uniform normal loads without subjecting the ring to high distortions. The imposed bending moments were restricted so that the tensile stresses in the extreme fibres in the ring were kept to below 40% of the ultimate value for the cases with the lowest Stage One normal loads. Linear elastic solutions were used to estimate the behaviour of the bolted segmental ring based on the analytical response of a continuous ring. Keeping the tensile stresses to within an 'elastic' range (c.f. Chapter 4) allowed parametric studies to be done. The full-ring tests aimed to investigate the influence of varying normal stresses and varying bolt preloads on the loading and unloading responses of the bolted segmental ring. A consequence of limiting the tensile stresses in the test ring was that the ring was only subjected to very small distortions. For the unloading cases distortion was up to 0.2% of the internal radius and for the loading cases distortion was only up to 0.1% of the internal radius. In fact Chapter 7 will show that the actual distortion achieved in the laboratory was even smaller.

The loading regime was designed to obtain as much information as possible from using one ring. The results from the parametric experiments performed at small deformations will help direct the planning of future tests so that the usefulness of the very limited number of tests taken to large deformations will be maximised.

However, it bears repeating that the 2D FE simulations by Avgerinos (2014) of the situation at Central Line east of Lancaster Gate show that after allowing 100 years of consolidation, the maximum distortion in the existing tunnel was only 0.2% of squatting. This implies that although on average 1% of distortion is measured in LU tunnels, it may be due to imperfect construction and self-weight rather than ground loading.

In Chapter 2, the range of hoop stresses measured in existing tunnels was summarised. However, there is a lack of reliable field measurements concerning how the loading from the ground is transferred to the tunnel lining. The two main families of distorted tunnel shapes observed in the field, that of a tunnel 'egging' (vertical diameter greater than horizontal diameter) and 'squatting' (horizontal diameter greater than vertical diameter), could be the result of a myriad of loading combinations. Furthermore, FE analyses showed that the action of interface shear stresses on the tunnel lining could control the deformation shape of the lining, but in the laboratory it was only possible to apply normal stresses.

Due to the uncertainty regarding the in-situ loading conditions, the loading regime presented in Section 6.3 was selected for its simplicity, stability and repeatability, established through trial tests as the loading setup was developed. The aim was to achieve the same distortion magnitude under different normal loads and bolting regime to investigate the influence of these variables on the behaviour of a bolted segmental ring in a rigorous manner. As mentioned the highest normal load applied by the actuators was limited to 40 kN due to the testing duration and the decision not to leave the actuators loaded unsupervised. This load is representative of the depth of cast iron tunnels in the LU network.

Another interesting aspect highlighted by the FE modelling was that the normal stresses acting on the tunnel lining was always lower than the overburden, and the loading as a proportion of the overburden

load differed depending on the relative permeability between the lining and the soil. Therefore the loading and unloading responses of the test ring were tested under a range of normal stresses which approximated different depths below ground. Finally the FE simulations gave confidence that the application of normal stresses via the use of loading pads was reasonable approximation to a uniformly distributed load around the circumference.

In summary, one main contribution of this research to the understanding of segmental lining behaviour was the design and development of the bespoke loading facilities from scratch. Many trial tests were conducted to optimise the stability of the loading system and to fine tune the loading procedure. This loading setup allowed very controlled and repeatable tests to be conducted on the half-scale cast iron tunnel lining. For the first time, the global and local joint response of a bolted segmental cast iron lining was studied experimentally and the internal actions relating to a particular deformed shape was measured.

Furthermore, because the loading and boundary conditions for the full-ring test in the laboratory was well defined, the set of results obtained from the small deformations tests would provide valuable calibration data for the future development of any FE models for bolted segmental cast iron rings. Numerical modelling will allow the interrogation of stress re-distribution at the longitudinal flanges, as well as the simulations of loading/deformation combinations that are not practicably achievable in the laboratory. Numerical modelling will be an essential component of any future research work in this area if the ultimate aim of providing comprehensive recommendations for the assessment of existing tunnel linings was to be met.

7 Behaviour of bolted segmental cast iron ring

7.1 *Rationale for testing regime*

The development and reasoning behind the loading procedures for the full-ring tests have been extensively discussed in Chapter 6. The main points are reiterated here.

Uniform Stage One normal loads were applied to sixteen actuators to bring the test ring under compressive hoop stress with negligible bending moment induced in the test ring (see Figure 6-30). The majority of the cast iron tunnels in the LU network lie between 20 m and 30 m below ground level. However, the FE analysis described in Chapter 6 suggested that in reality reduced overburden pressures may be acting on the tunnel lining. Different magnitudes of normal loads were selected for Stage One to investigate the effect of varying hoop stress on the bolted segmental ring behaviour.

During Stage Two, the loads in the actuators along one diameter were adjusted in order to achieve the prescribed deformations for the test ring (see Figure 6-30). The deformed shapes were elliptical, as current field measurements of distorted tunnels would generally report the maximum changes along two orthogonal diameters. The deformed tunnel shapes are normally described as ‘squatting’ or ‘egging’ as discussed in Chapter 2. Stage Two loading produced bending moment in the test ring. This bending moment in the test ring was estimated using the strain measurements in the circumferential direction at four sections around the test ring (see Figure 7-1), assuming a constant modulus. The deformations in the ring were kept low such that the bending capacity at the joint corresponding to the lowest Stage One normal loads was not utilised above 40%. The deformation magnitudes were kept constant even as the hoop stress increased. This allowed test results under different hoop stresses to be compared. The magnitude of deformation for the unloading tests was selected to replicate the measured deformation in Central Line which was presented in Chapter 3.

In Chapter 2, the industry practice for tunnel assessment was discussed. It is currently assumed that the presence of joints in the bolted segmental tunnel lining reduces the stiffness of the lining. As a result, for the same deformations and the same lining geometry, the bending moment produced in the segmental lining is assumed to be lower than that in a continuous lining. One of the objectives of the laboratory experiments is to test this assumption. Therefore, the bending moments calculated from the laboratory tests on a bolted segmental lining are compared with the analytically obtained bending moments of a continuous lining under the same deformation as the test lining.

For a continuous ring made from linear elastic material, the stiffness of the ring is uniform and the bending moments related to an imposed change in radius are independent of the hoop stress. The effect of having joints, as in the case of the bolted segmental ring, is a potential reduction in the ring stiffness. This reduction in ring stiffness would depend on the hoop stress (from the overburden) and the change in radius. Another indication of differences in ring stiffness due to joint behaviour, still under the assumption of linear elasticity, is differences in the loads required to impose a specific change in radius.

Finally, it was also mentioned in Chapter 2 that currently there appears to be a lack of consensus on the mitigation procedure for tunnels at risk of being overstressed. In the past, some engineers have opted for tightening all the bolts while others have decided to loosen the bolts. Therefore another aim of the laboratory experiments is to ascertain the influence of the bolt preloads on the segmental lining behaviour. Hence, the tests are repeated for different initial bolt preloads.

In the following sections the experimental results are presented.

7.2 Measurements and corrections for change in radius

This section explains how the change in radius of the test ring was computed. In Chapter 6, two components of the setup, implemented specifically to minimise free body movement of the test ring, were described. However, even by replacing two actuators with reaction rods and the installation of the tangential tension member, there was still measurable free body movement.

When fully unloaded, the test ring was assumed to be perfectly circular. After the test ring has reached the desired level of hoop stress through the application of Stage One loads (Figure 6-30), it was deformed into an ellipse through an increase or decrease of loading along one axis.

Figure 7-2 illustrates how the change in radius from a circle into an ellipse at the location of each of the 18 displacement transducer placed around the test ring was calculated from the laboratory measurements. Let points ABCDEF lie on the 'original' solid black circle representing the test ring at rest, with centre at (0,0) and a radius equal to the internal radius of the test ring, R . dR is the measured radial displacement at the location of each of the 18 displacement transducers placed at 20° intervals around the intrados of the test ring, in line with an actuator as shown in the top diagram of Figure 6-14, i.e., each displacement transducer was initially zeroed at points ABC, etc. And let points A'B'C'D'E'F' be the final displaced locations. The method of least squares is used to best fit a new circle based on points A'B'C'D'E'F', drawn in dashed red line, with centre (p,q) and the same radius as before, R .

The expression for the summation of the squares of deviation, E^2 , of the data points to the best fit circle is given by:

$$E^2 = \sum_{A'}^{F'} (\sqrt{(x_i - p)^2 + (y_i - q)^2} - R)^2 \quad \text{Equation (17)}$$

Where x_i , y_i are the coordinates of points A' etc and R is the internal radius of the test ring. For example, $x_{A'} = (R + dR)\cos(\theta_A)$, and $y_{A'} = (R + dR)\sin(\theta_A)$.

The Solver function in Excel is used to find (p,q) such that the derivatives $\frac{dE^2}{dp}$ and $\frac{dE^2}{dq}$ are close to zero.

The coordinates (p,q) give the magnitude and direction of the rigid body movement. The coordinates (p, q) are subtracted from the coordinates of points A'B'C'D'E'F' to correct for this rigid body movement. For example, C' has been corrected to C". The change in radius of the test ring at $\theta_{C''}$ equals $(R_{C''} - R)$.

In the experiments, the maximum free body displacement was approximately 1 mm. Most of the free body movement occurred when the ring was sequentially loaded up to simulate a particular overburden pressure. Theoretically, when the ring is under uniform loading around its extrados its shape should not deviate from circular. However, as shown in Figure 7-3 that was not the case during the experiments. Figure 7-3 captures the displaced shape of the test ring after all the actuators were brought to approximately 40 kN. In Figure 7-3, the locations of the original points of measurements around the intrados of the test ring under zero loading are represented by the red square markers. The green circular marker gives the coordinates of the centre of the circle that has been best fitted to the displacement data. The blue markers show the positions after the free body movement has been corrected. The change in radius was enhanced 250 times to make the new locations more distinct in the figure. It is obvious the test ring did not remain circular when it was under equal loading from all the actuators. However it needs to be noted that although efforts were made to build a perfectly circular ring prior to testing, this was done with tape measurements and an initial out-of-circularity may exist before any loading was applied. Furthermore, while the actuators were defined to apply equal loads, the load tolerance was set at $\pm 10\text{N}$. This in combination with the errors of the load cell measurements meant that Stage One loading may not have been identical at all loading positions.

Figure 7-4 compares the hoop forces estimated using linear elastic solutions with equal radial loads applied at 20 degrees around the ring with the hoop forces calculated using strain measurements obtained when the ring was under approximately equal all-round loads. In order to facilitate comparison between the hoop forces under different Stage One all-round actuator loads, the hoop forces have been normalised against the average hoop force calculated from the linear elastic solutions. It was expected that the hoop force in the segmental test ring should be the same as the estimated hoop force for a continuous ring using analytical equations. The differences between the two could be a result of the out-of-circularity of the test ring as mentioned above as well as errors in the strain gauge measurements. Overall, Figure 7-4 provides confidence that the measured strains are reasonable. In order to avoid the cumulative effect of errors, this chapter generally presents and discusses the change in measurements between Stage One and Stage Two loading.

For both Stage One and Stage Two loading as specified in Figure 6-30, the corrected displacement from baseline was calculated. This enabled the calculation of the change in shape from Stage One to Stage Two. In the subsequent sections where the experimental results are presented, the term 'change in radius' at 20° intervals around the intrados of the test ring refers to the change between Stage One and Stage Two in Figure 6-30. Many of the subsequent graphs are plotted against degrees around the test ring. Efforts were made to load the ring symmetrically about the diameter drawn through zero and 180 degrees. The unloading and loading distortions were governed by the actuator at 90 degrees (Figure 7-1).

Figure 7-5 and Figure 7-6 show the measured displacement around half the ring after the free body movement correction has been applied. Both figures give the change in radius between Stage One and Stage Two. On the graphs, the displacements under different all-round normal loads are shown, with each normal load magnitude represented by the different colours. As mentioned in Section 6.3,

for a combination of Stage One normal loads and bolt preloads, a set of three unloading tests was performed followed by a set of three loading tests. So, for example, the legend label “40kN_3 (5kN bolt)” on Figure 7-5 refers to the results of the third unloading test carried out under normal loads of 40 kN and with the bolts initially preloaded to 5 kN. The legend label “40kN_2 (5kN bolt)” on Figure 7-5 refers to the results of the second unloading test carried out under normal loads of 40 kN and with the bolts initially preloaded to 5 kN, and so on.

In Figure 7-5, the Stage One normal loads applied to the test ring ranged from 10 kN to 40 kN. In Stage Two the radius at 90 degrees was set to increase by 2 mm by reducing the actuator load at that location (c.f. Table 6-5). The lines represent the results obtained from the tests where all the bolts around the ring were bolted up to 5 kN preload initially. The markers represent the results obtained from the tests where all the bolts were bolted to 10 kN preload initially. The results highlight two main points. First, the control system worked consistently across all the tests such that practically the same distortion was achieved in all the tests. Secondly, the corrected displacement at 90 degrees was just over half of the magnitude that was specified, 1.2 mm instead of 2.0 mm. This was because the 2 mm of specified displacement was in fact applied over the diameter instead of the radius, as segment G only provided reactions.

Figure 7-6 tells a similar story for the loading test cases. The Stage One normal loads ranged from 3.4 kN to 40 kN. For the loading cases, the actuator at 90 degrees was set to move towards the centre by 1 mm by increasing its load. Once again the control system worked consistently across all the loading tests and the deformed shapes under different Stage One normal loads and bolt preloads were the same.

A check was made to see how closely the deformed shapes in the laboratory conformed to an ellipse. The measured radial displacements at 90 degrees were used as a starting point to create the ellipsoidal displacements in Figure 7-7. It can be seen that the combination of load and displacement control adopted for Stage Two of the loading procedure was able to deform the test ring to an ellipse.

7.3 Measurements of strain and calculations for bending moment

The bending moments induced in the ring as a result of the unloading and loading deformations were calculated from strain measurements at four locations around the ring. These locations are at 20 degrees, 100 degrees, 140 degrees and 260 degrees on the ring as shown in Figure 7-1. The method of calculating the bending moment from strain measurements has already been described in Section 5.11. The circumferential strains measured at the T-gauge locations furthest away from the centroidal axis of the segment cross section were used in the calculations. The change in strain between Stage One and Stage Two loading was considered. Elasticity was assumed and a modulus value of 100 GPa was used in the calculations to obtain the stress. Stress block calculations were used to estimate the change in bending moment from Stage One to Stage Two.

Figure 7-8 and Figure 7-9 compare the bending moment calculated from strain gauge measurements on segment I and segment G from Stage One. As seen in Figure 7-1, these two strain gauged locations should measure the same changes in strain if the ring was behaving symmetrically.

However, as shown in Figure 7-8 and Figure 7-9, the bending moment calculated for segment G was lower than the bending moment in segment I in all the cases. The difference became greater as the pre-distortion normal load increased.

Unsurprisingly, replacing the actuators with fixed length reaction bars affected the behaviour of segment G (c.f. Figure 6-3 and Figure 6-19). In light of this finding, only the measured response of segments H, I and J was compared against the bending moment distribution of a continuous ring using linear elastic solutions. The bending moment distribution for the continuous ring was obtained by adjusting the out-of-balance load at 90 degrees until the analytical equations gave the same displacement as that measured in the experiment.

The results for the unloading case with an initial bolt preload of 5 kN are given in Figure 7-10. In the top graph, the markers give the experimental response calculated from strain gauge measurements and the lines give the bending moment for a continuous ring subjected to the same deformation calculated using linear elastic solutions. The bottom graph compares the experimental and analytical results with the estimated 40% bending capacities of the joint as shown in Table 6-3. Figure 6-29 has shown that within the range of normal loads applied to the test ring in the laboratory, the joint capacity is below the segment capacity, and for clarity the segment capacities are not included on the figures. The bending capacities for each joint under different normal loads are shown on Figure 7-10. It is clear that the joint capacity calculated for normal loads of 10 kN per actuator was the most critical. Figure 7-11 to Figure 7-15 are presented in a similar format.

The most critical locations are the joints at zero and 180 degrees, for both the unloading and loading cases.

It is interesting to note how the measured response of the segmental ring at the discrete locations deviated from the response of a continuous ring. When the segmental ring was subjected to high normal loads of approximately 40 kN before it was distorted, the measured response at the strain gauged locations gave bending moments that were the same, or even slightly higher, than the moment for a continuous ring. For the cases with lower normal loads of 20 kN and 10 kN prior to distortion, the bending moments calculated from the measured change in strain were lower than the bending moment for a continuous ring. When the normal loads were approximately 20 kN before distortion, the reduction in bending moment compared to the continuous ring was up to 15% for the strain gauged section at 100 degrees. When the normal loads were approximately 10 kN before distortion, the reduction in bending moment compared to the continuous ring was about 20% for the strain gauged section at 100 degrees. Out of the three strain gauged sections from zero to 180 degrees, the strain gauged section at 100 degrees was where the measured moment was the greatest, and the results expected to be least affected by measurement errors.

This implies a reduction in the stiffness of the bolted segmental ring when the hoop stress was lowered. As discussed at the start of the chapter, this reduction in ring stiffness is due to the presence of joints, because for a continuous ring made from linear elastic material, the stiffness of the ring is uniform and the bending moments related to an imposed change in radius are independent of the hoop stress. The small imposed displacements were adopted so that the cast iron material would

respond elastically. This reduction in stiffness happens when the joints are in tension (start to open) and the joint stiffness affects the ring stiffness.

The results for the unloading case with an initial bolt preload of 10 kN are given in Figure 7-11. Once again, when the normal loads were approximately 40 kN before distortion took place, the behaviour of the bolted segmental ring was very similar to that of a continuous ring subjected to the same deformation. When the normal loads were 20 kN prior to distortion, the reduction in bending moment compared to the continuous ring was up to 15% for the strain gauged section at 100 degrees. When the normal loads were approximately 10 kN prior to distortion, the reduction in bending moment compared to the continuous ring was up to 20% for the strain gauged section at 100 degrees.

Figure 7-12 shows the results for the unloading case where the initial bolt preload was 5 kN and grommets were used at all the bolt locations. As before, when the initial all-round normal loads were approximately 40 kN, the bending moment calculated from measured strains upon distortion were similar to the bending moment in a continuous ring. When the normal loads were 20 kN prior to distortion, the reduction in bending moment compared to the continuous ring was up to 15% for the strain gauged section at 100 degrees. When the normal loads were approximately 10 kN prior to distortion, the reduction in bending moment compared to the continuous ring was up to 20% for the strain gauged section at 100 degrees.

The results for loading cases are given in Figure 7-13 to Figure 7-15. Figure 7-13 gives the results with an initial bolt preload of 5 kN. Figure 7-14 gives the results with an initial bolt preload of 10 kN. Figure 7-15 gives the results with an initial bolt preload of 5 kN with grommets. All the figures tell a similar story to the unloading results. When the normal loads reached approximately 40 kN prior to distortion, the behaviour of the bolted segmental ring was similar to the behaviour of a continuous ring. As the normal loads reduced, the bending moment in the segmental ring deviated further away from the bending response of a continuous ring. When the normal loads were 20 kN prior to distortion, the reduction in bending moment compared to the continuous ring was up to 15% for the strain gauged section at 100 degrees. When the normal loads were approximately 10 kN prior to distortion, the reduction in bending moment compared to the continuous ring was up to 20% for the strain gauged section at 100 degrees. When the normal loads were approximately 3.4 kN prior to distortion, the reduction in bending moment compared to the continuous ring was up to 30% for the strain gauged section at 100 degrees.

It was discussed in Chapter 2 that it is common industry practice to use Morgan's equation (1961) to calculate the bending moment in the tunnel linings, but with a reduced second moment of area according to Muir Wood (1975). Morgan's equation assumes elliptical deformation. Figure 7-16 shows the results given in Figure 7-10 and Figure 7-13, and also includes the bending moment calculated using Morgan's equation based on the measured distortion at 90 degrees. A reduction factor was not used. It would appear that even without any reduction, using Morgan's equation would underestimate the bending moment at 90 degrees for the cases with initial normal loads of 40 kN.

7.4 Using linear elastic solutions to predict the onset of joint opening

In the laboratory experiment the test ring was approximately equally loaded from eighteen locations evenly distributed around the ring to bring it under hoop stress. The load at 90 degrees was then adjusted to unload the ring (P^-) and increase the radius, or to load the ring (P^+) and decrease the radius at 90 degrees as shown in Figure 6-30. In the following sections the increase or decrease in radius refers to the change in radius along the loading axis at 90 degrees.

In the unloading experiments, all the tests, with the exception for the tests with normal loads of 3.4 kN, achieved an increase of 0.13% of the inner radius along the loading axis at 90 degrees. In the loading experiments all the tests achieved a decrease of 0.07% of the inner radius along the loading axis at 90 degrees.

The following procedure was used to check whether the joints would start to open at these magnitudes of deformations. Once again elastic behaviour was assumed. Also, the level of preload in the bolt was not taken into account. Section 5.6 has shown that the preload magnitude affected the moment at which the joint began to open, in the absence of hoop force at the joint. This exercise only takes into account the level of hoop force at the joint. First, the bending moment was divided by the hoop force to calculate the eccentricity of the line of thrust at the joints. The limiting value for the eccentricity based on the geometry of the section was calculated by dividing the section modulus by the cross-sectional area. These values for eccentricity were compared to determine if the extreme fibre stress at the joint location has reached zero. This is the point at which the joint was assumed to open and the bolts come into play.

From the mechanical tests conducted on cast iron specimens as described in Section 4.2 in Chapter 4, a stiffness value of 100 GPa for cast iron was selected in the analytical analysis. Obviously the outcomes for the predicted joint opening would be dependent on this stiffness value. Keeping the hoop stress in the ring the same, adopting a lower E value would mean that a greater distortion was required before the joint would start to open.

The load at 90 degrees, P^- or P^+ , was then adjusted to determine the deformation magnitude when the extreme fibre stress at the joint *first* reached zero. The maximum moment in the ring, which may or may not coincide with the joint location, was also calculated. This was used to check if the extreme fibre stress in the ring was within the elastic limit. If so, it suggests the joints will affect the deformability of the ring before nonlinear behaviour develops within the segments. If not, the entire ring will start to deform inelastically before the joints start to properly open.

For the unloading cases, the check was done for actuator loads corresponding to overburden pressures relating to depths of 6m, 12m and 24m. For the loading cases, the check was done for actuator loads corresponding to overburden pressures relating to depths of 2m, 6m, 12m and 24m. Refer to Table 6-5 and Table 6-6 for the relationship between actuator loads and overburden depths.

The findings are summarised in Table 7-1 for the unloading case and Table 7-2 for the loading case.

Depth of overburden (m BGL)	Deformation magnitude when joint first tend to open (% increase of inner radius)	Location of first joint opening	Max BM in ring (kNm)	Corresponding tensile stress in extreme fibre from max BM as % of ultimate tensile strength	% Utilisation of critical joint
6	0.05	0 and 180 degrees intrados	0.18	5	11
12	0.11	0 and 180 degrees intrados	0.37	11	18
24	0.21	0 and 180 degrees intrados	0.74	22	24

Table 7-1. Predicted onset of joint opening for unloading case based on E = 100 GPa.

Depth of overburden (m BGL)	Deformation magnitude when joint first tend to open (% decrease of inner radius)	Location of first joint opening	Max BM in ring (kNm)	Corresponding tensile stress in extreme fibre from max BM as % of ultimate tensile strength	% Utilisation of critical joint
2	0.04	60 and 120 degrees intrados	0.25	7	5
6	0.14	60 and 120 degrees intrados	0.79	23	12
12	0.29	60 and 120 degrees intrados	1.60	46	19
24	0.58	60 and 120 degrees intrados	3.24	94	26

Table 7-2. Predicted onset of joint opening for loading case based on E = 100 GPa.

It can be seen that for the unloading experiments where all the tests achieved 0.13% of deformation, the joints at zero and at 180 degrees would be opening at the intrados for the cases of overburden depths relating to 6m BGL and at 12m BGL. For an overburden depth relating to 24m BGL, the joints would only begin to open when the extreme fibre stress was reaching 22% of the ultimate tensile strength of the grey cast iron. It is generally regarded in the industry that 25% of the ultimate tensile strength is the elastic limit of grey cast iron.

It ought to be reiterated that the unloading cases are more relevant to the research problem involving new tunnel excavations close to existing tunnels. The results in Table 7-1 suggests that, for a typical LU tunnel at a depth of 24 m BGL, assuming full overburden acting on the lining, the joints would come into play to reduce the overall lining stiffness when the extreme fibre stress was still within 40% of the ultimate tensile strength of the cast iron.

For the loading case the joints would tend to open at the intrados at 60 and 120 degrees. Since in the experiments the inner radius was only decreased by 0.07%, joint opening would only occur for the lowest overburden load corresponding to 2m BGL. It should be noted that for an overburden load corresponding to 12 m BGL or deeper, significant inelastic deformation would occur before the joints would open. This suggests that for certain engineering works where additional loading will be experienced by existing tunnels, for example piling close to existing tunnels, extreme caution needs to be exercised, because joint opening cannot be relied upon to reduce the bending moment in the lining ring.

Assuming perfect contact between segments, the stiffness of the joints start affecting ring behaviour once tension develops at the joints. What has been shown above is that the bending moment, and the displacement, required to first open the joints increase with overburden. It was mentioned at the start of the chapter that for linear elastic behaviour and uniform stiffness as is the case for a continuous ring of uniform cross section, the bending moments resulting from an imposed change in radius are independent of the hoop stress. The presence of joints has the potential to reduce the ring stiffness. What has been shown in this section is that as hoop stress increases, greater displacements are necessary to open the joints, meaning that greater displacements need to be reached before the stiffness of the ring is reduced.

Another way to show the variation in the stiffness of the bolted segmental ring under different hoop stress is to compare the forces required to deform the ring by the same magnitude under different hoop stress. As discussed in Chapter 6, the forces in all the actuators were set to remain constant between Stage One and Stage Two, except for the actuators on the loading axis. The stiffer the ring the greater the force required to deform by a prescribed amount.

Figure 7-17 shows the reduction in load required at the loading axis in order to increase the radius by the same amount under different hoop stress. It is clear that as the all-round actuator loads, which gave rise to hoop stress, increased, a bigger change in load at the loading axis was needed to achieve the same deformation. At each level of hoop stress, three tests were done. The markers on Figure 7-17 show the results of each test and the lines show the average of the results. Once again the trends given by different initial bolt preloads were very similar. Figure 7-18 shows that greater and greater loads were necessary in order to reduce the radius at 90 degrees by the same amount as hoop stresses increased. These results provide more evidence that the bolted segmental ring became stiffer as the hoop stress increased.

7.5 The measured behaviour at the joints

Figure 7-19 provides the response of the bolts at zero, 60, 120 and 180 degrees around the test ring for the unloading tests. The measured change in bolt load between Stage One and Stage Two for the edge bolts and middle bolts are given. A positive change in load indicates increase in tension in the bolt. The bolt response under different initial all-round actuator loads are given on separate graphs. The different coloured lines on each graph represent the bolt behaviour under varying initial bolt preload and the inclusion of grommets. The legend label “First 10kN (5)” refers to the first test conducted at 10 kN normal loads and 5 kN bolt preload, and the legend label “Second 10kN (5)” refers to the second test conducted at 10 kN normal loads and 5 kN bolt preload, and so on. As discussed in Chapter 5 and shown in Figure 5-25, when grommets were in place the bolts needed to be re-tightened several times before a relatively constant preload value was achieved. This was also found to be the case for the test ring. Figure 7-20 shows the reduction in bolt load for all the instrumented bolts around the ring when grommets were included. The bolts had to be re-tightened several times before a stabilised preload of approximately 5 kN was reached.

An increase in Stage One normal loads resulted in smaller changes in bolt loads upon distortion. When the ring was loaded up to 40 kN prior to distortion the change in bolt loads was negligible. Unloading distortion should cause positive (straightening) moments at zero and 180 degrees and negative moments at 60 and 120 degrees.

Figure 7-19 shows that a positive moment was associated with an increase in bolt load and a negative moment was associated with a decrease in bolt load in the edge bolt. The results did not indicate any clear relationship between the initial preload and the subsequent change in bolt load upon distortion. The inclusion of grommets appeared to have lessened the changes in bolt load. The instrumented middle bolts registered negligible changes for both positive and negative moment.

Figure 7-21 provides the response of the bolts at zero, 60, 120 and 180 degrees around the test ring for the loading tests. These results tell a similar story to the unloading results. Given the same magnitude of distortion, the higher the initial all-round actuator normal loads, the smaller the change in bolt loads upon distortion. When the ring was loaded up to 40 kN prior to distortion the change in bolt loads was negligible. The inclusion of grommets did not significantly alter the behaviour of the bolts, apart from lessening the changes in bolt load. The instrumented middle bolts registered negligible changes for both positive and negative moment.

The next series of figures (Figure 7-22 to Figure 7-27) present the results of the local joint movements as measured by the LVDTs installed along the extrados and intrados of the joints. Figure 7-22 to Figure 7-24 provide the results for the unloading tests (actuator load at 90 degrees decreased after Stage One) at different normal loads prior to distortion. Figure 7-25 to Figure 7-27 provide the results for the loading tests (actuator load at 90 degrees increased after Stage One). Movement was measured at the outer edge, in line with the location of the edge bolt and in line with the location of the middle bolt. For the locations of the LVDTs refer to Figure 5-12. The change in LVDT measurements between the all-round load stage (Stage One) and the distortion stage (Stage Two) are presented. A positive change is indicative of the joint opening at that location.

Before the tests were carried out, the joints were expected to behave as outlined in Table 7-3. A review of Figure 7-22 to Figure 7-27 shows that the measured results matched the expected behaviour. Furthermore, the magnitude of joint movement decreased as the initial all-round normal load increased. The different initial bolt preload magnitudes, as well as the presence or absence of grommets, had very little effect on the joint movement. Along both the extrados and intrados, the greatest movement was recorded at the outer edge. The joint movement along the intrados was up to an order of magnitude greater than the movement along the extrados.

Degrees around ring	Unloading			Loading		
	Bending mode at joint	Joint movement from Stage One to Stage Two		Bending mode at joint	Joint movement from Stage One to Stage Two	
		Extrados	Intrados		Extrados	Intrados
0	Positive	Close	Open	Negative	Open	Close
60	Negative	Open	Close	Positive	Close	Open
120	Negative	Open	Close	Positive	Close	Open
180	Positive	Close	Open	Negative	Open	Close

Table 7-3. Expected joint movements along joint extrados and intrados.

In order to investigate the influence, if any, of the initial bolt preload more closely, the measured joint movements for the unloading case where the initial all-round normal loads were 10 kN are presented in Figure 7-28 as distance away from the middle bolt centreline, in a similar manner to Figure 5-37 for the two segment test. On the graphs, $x = 0$ is in line with the middle bolt centreline, $x = 76$ is in line with the edge bolt centreline, and $x = 127$ is in line with the outer edge of the longitudinal flange. The measured change in displacement along the extrados and intrados for each joint are presented. Once again, a positive value for joint movement is indicative of the joint opening between Stage One and Stage Two. The results corresponding to bolt preloads of 5 kN, 10 kN and 5 kN with grommets are given.

When the preload was 5 kN, having the grommets in place made negligible difference to the displacement at the joint. While having a higher preload did result in marginally smaller displacements, the effect is insignificant when compared to the influence of the all-round normal loads on joint movement as seen in Figure 7-22 to Figure 7-27. Recall that in Figure 5-37, the ‘pinching’ effect at the edge bolt location was not obvious at preloads of 5 kN and became more pronounced at preloads of 10 kN. The hoop force at the joint was zero. However, Figure 7-28 indicates that when the hoop force at the joint was not zero, the pinching effect of all the preload values was similar. This gives more weight to the argument that when a segmental ring is under hoop stress, the magnitude of preload in the bolts is not a significant factor in determining the behaviour of the joints.

Furthermore, Figure 7-28 shows that the pinching effect along the intrados at the edge bolt location occurred for both joint opening and joint closing.

7.6 Distortion of circumferential flanges

It was mentioned in Chapter 6 that a displacement transducer was positioned vertically in segment I to measure any displacement between the two circumferential flanges of the segment as it distorts (Figure 6-20). The measurements from this displacement transducer are given in Figure 7-29 for the unloading case and Figure 7-30 for the loading case. The markers represent individual measurements and the lines give the average displacement at different bolt preloads.

Segment I was expected to undergo negative bending, i.e., increase in curvature, during the unloading of the test ring. Conversely, segment I was expected to undergo positive bending when the test ring was loaded. As illustrated by Figure 2-11 and Figure 2-12, the circumferential flanges were expected to come together during positive bending and move apart during negative bending. The displacement transducer was calibrated such that a positive change indicates the circumferential flanges moving towards each other.

Figure 7-29 and Figure 7-30 confirmed the predicted behaviour, showing negative changes in displacement for negative bending and positive changes in displacement for positive bending. As the all-round actuator load increase, the relative movement between the flanges reduced. Interestingly, a higher initial bolt preload led to greater distortion of the circumferential flanges.

7.7 Measurements from strain gauge rosettes

The measurements from the 3-element strain gauge rosettes were analysed in a similar manner to that described in Chapter 5. Using the change in strain from each strain gauge in a rosette, the principal major and minor direct strains and the orientation of the principal direct strain directions on the surface of the cast iron were calculated for each rosette location.

The main findings from analysing the strains at discrete locations are given below, using the strains for the unloading case on segment H as an illustration. The strains are indicated on Figure 7-31. Tensile strains are positive. The diagram on the left hand side relates to an overburden of approximately 6 m BGL and the diagram on the right hand side relates to an overburden of approximately 24 m BGL. Reference back to Table 7-1 indicates that the extreme fibre stress at the intrados of the joints at zero and 180 degrees would be zero at this magnitude of deformation for the overburden loads corresponding to 6 m BGL. The main points are:

- When the joint is starting to open, the circumferential tensile strain at the external corner (location 1 in this case), is lower ($+5 \mu\epsilon$) than if the joint is closed ($+83 \mu\epsilon$).
- When the joint is starting to open, the circumferential tensile strain at the internal corner (location 5 in this case), is higher ($+132 \mu\epsilon$) than if the joint is closed ($-32 \mu\epsilon$).
- On the extrados of the skin away from the horizontal flanges (locations 9 and 10 in this case), the change in strain agrees with the expected behaviour for segments as presented in Figure 2-12 and Figure 2-11 for negative and positive bending.
- At the junction between the horizontal flange and the intrados of the skin (locations 13 and 14 in this case), the tensile circumferential strains are higher when the overburden is low and the joint is tending to open.
- The highest tensile strains are measured at the internal corner on the circumferential flange adjacent to the joint undergoing positive bending.

The detail results are given in Appendix I.

7.8 Discussion

The experimental setup and control system have enabled a set of parametric experiments to be completed successfully. The influence of initial normal loads, which gives rise to hoop forces at the joints, and the influence of bolt preloads, were investigated. The bending moments imposed on the bolted segmental ring were limited such that the cast iron segment was not stressed beyond 40% of its ultimate tensile strength at the extreme fibres for the most critical cases when the hoop stress was lowest. Within this range the stress-strain behaviour of the cast iron was essentially elastic as discussed in Chapter 4. This allowed numerous tests to be conducted on the same segments. Linear elastic solutions were used to provide the analytical response of a continuous ring having the same cross sectional area as the bolted segmental ring.

The different levels of normal loads applied in Stage One roughly corresponded to different depths of overburden. For a typical LU tunnel, the normal load of 40 kN, equating roughly to an overburden

depth of 24m, was the most appropriate. This assumes that 100% of the overburden pressure acts on the tunnel lining. At this level of normal load, the strain measurements from both the unloading tests and loading tests indicated that the bolted segmental behaved as a continuous ring under the imposed distortion.

However, while some case studies in Chapter 2 indicated that the load acting on the tunnel lining could be up to the overburden pressure, many case studies recorded pressures lower than the overburden. The field instrumentation exercise conducted by the Imperial College research team at Tottenham Court Road station described in Chapter 3 measured hoop stresses in the lining that ranged from 63% below to 30% above the hoop stress that would result from the overburden pressure. A further complexity was highlighted by the FE analysis discussed in Chapter 6, that depending on how the interface between the soil and the tunnel was modelled, the distortion of the lining could be related to the shear stresses acting on the lining and not the normal stresses.

The laboratory experiments also showed that, for the same amount of distortion, reducing the magnitude of the normal load prior to distortion had the effect of reducing the measured bending moment in the bolted segmental ring. The experimental results showed that whether the bolted segmental test ring was distorted by unloading or by loading, the reduction in the measured bending moment when compared with the bending moment estimated for a continuous ring was similar for a given magnitude of all-round normal load. The greatest difference between the bending moment estimated for a continuous ring and the bending moment calculated from measured strains was 30% when the normal load was 3.4 kN per actuator, roughly equating to 2 m of overburden pressure. This comparison between the estimated moment for a continuous ring and the experimentally obtained bending moment was made for the strain gauged section at 100 degrees. It is considered that the significance of errors in measurements would be least at this location where the measured change in strain was the largest.

As discussed in Chapter 5 Section 5.11 for the statically determinate two segment test, the bending moment calculated from strain measurements assuming a constant modulus of 100 GPa for the cast iron was 0.12 kNm compared to the theoretical bending moment of 0.14 kNm obtained from statics at the same location. The moment calculated from strain measurements was approximately 15% lower than the theoretical bending moment. This suggests that for the full-ring results, deviations of the experimentally obtained bending moment for the segmental ring from the analytically obtained bending moment for a continuous ring of 15% or less could be due to errors.

As discussed in Chapter 2, using the simplified Muir Wood's (1975) formula as presented in LU standard 1-055 (2007) to estimate the bending moments for the case of the test ring with six joints would result in a reduction of over 50% when compared with the bending moment for a continuous ring.

As expected, increasing the all-round normal load prior to distortion increased the stiffness of the joint. The experiments showed that having a higher all-round normal load resulted in smaller joint movements and also smaller changes in bolt loads upon distortion.

In Chapter 2, it was discussed that sometimes the bolts in the LU tunnels would be tightened or loosened as a measure to reduce negative impacts to existing lining segments from new excavations. In the full-ring experiments, the influence of varying bolt preloads on the behaviour of the bolted lining was studied on a global scale and on a local scale.

On the global scale, it was shown that under different magnitudes of bolt preload, the bending moment calculated from measured strains deviated from the bending moment distribution of a continuous ring in a similar manner. There was no evidence to indicate that an initial higher or lower preload had an impact on the reduction of bending moment for the small distortion imposed. Even after the inclusion of grommets, the observed pattern of behaviour did not change. The main effect of including grommets was that the bolt preload reduced with time. This implies that in tunnels where grommets were used, the bolts would be less tight than in tunnels without grommets.

Looking at the behaviour of the joints more closely, it was established that there was no clear relationship between the initial magnitude of bolt preload and the subsequent change in bolt load upon distortion. Similarly, the different magnitudes of bolt preload, and the presence or absence of grommets, had very little impact on the magnitude of joint movement upon distortion.

The main impact of including grommets agreed with the finding from Chapter 5, namely that the bolts had to be re-tightened several times before a stable preload was achieved.

In Chapter 5, the two segment FE analysis with negative bending at the joint showed that the middle bolts contributed as much as the edge bolt when the hoop force is zero. The full-ring experiments showed that the joint behaviour was different when there was hoop force across the joint. The instrumented middle bolts measured negligible changes in load for both positive and negative moment at the joint.

Overall, for the small distortions imposed, the presence of compressive hoop force renders the magnitude of bolt preload insignificant in terms of influencing bolted segmental lining behaviour.

However, the distortions achieved in the experiments were very small compared to the ovalisation measured in the field, which was mentioned in Chapter 2. The two segment test results discussed in Chapter 5 Section 5.5 indicated that joint stiffness decreased with increasing joint opening. Further experimental work is necessary to determine the reduction in stiffness of a bolted segmental tunnel lining as distortion of the tunnel ring increases. In the meantime, analytical calculations were used to predict the onset of joint opening.

By neglecting the influence of bolt preload, equations derived for the calculation of bending moment, hoop stress and deflection of a continuous ring subjected to equally distributed loading were used to determine the distortion necessary to first cause the joints of a bolted segmental ring to open.

For a tunnel at a depth of 24 m BGL, a typical depth for LUL tube tunnels, the inner radius needs to increase by 0.21% by unloading before the joints would start to open and contribute to the reduction in the overall stiffness of the ring. At such deformation, the extreme fibre stress in the ring is 22% of the ultimate tensile strength. This is within the elastic limit of grey cast iron, which in practice is usually taken as 25% of the ultimate tensile strength.

At this same depth, the inner radius needs to reduce by 0.58% by increasing the load before the joints would start to open. At such deformation, the extreme fibre stress in the ring is 94% of the ultimate tensile strength. For this case, the ring segments would deform inelastically before the joint would play a part in the distortion of the ring. This implies caution needs to be exercised when engineering works which would increase the loading on the tunnel lining, for example piling works, are carried out close to existing tunnels.

Interestingly, the experimental results suggested that there were reductions in the bending moment when compared to a continuous ring even when the distortion had not yet reached the magnitude for the joints to open. According to the calculations in Table 7-2, when the applied distortion was 0.07% reduction in radius, only the tests conducted at the lowest all-round normal load of 3.4 kN would measure joints opening. Yet the bending moment calculated from strain measurements in the test ring showed reductions in moment for the tests with normal loads of 10 kN and 20 kN. In Figure 7-21, small changes in the bolt loads were measured when the normal loads were at 10 kN and 20 kN. A change in bolt force also indicates that the joint was playing a part in the ring behaviour.

Studying the discrete strain measurements from the 3-element strain gauge rosettes did not provide much insight into the behaviour of the segmental ring. The measurements may provide some information if used in conjunction with the development of finite element models for cast iron segmental rings. However, experimentally, it would be more beneficial instead to install more 2-element T-gauges circumferentially around the ring to obtain a clearer understanding of the bending moment distribution around the ring when distorted.

A limitation of the full-ring test was that it provided no restriction to movement in the longitudinal direction, made evident by the measured deflections of the circumferential flanges during distortion. As discussed in Chapter 5, in real tunnels the circumferential flanges would be restricted from movement. The FE analysis described in Chapter 5 where the circumferential flanges were restricted from any longitudinal movement showed that the additional restraint resulted in lower increase in bolt load, smaller joint opening at the outer edge location, and smaller global horizontal movement measured at the roller support. All of the above indicates that restricting the circumferential flange stiffens the joint behaviour. This means that the LU tunnels would have a stiffer response than the experimental ring. The biggest implication is that in real tunnels the reduction in bending moment upon distortion when compared to a continuous ring may be even less than the experimental findings.

Based on the experimental findings from the full-ring tests taken to squatting distortion of 0.13% and egging distortion of 0.07%, which have been considered in light of the findings from 2D FE simulations and literature review, several recommendation are made.

Based on the FE findings conducted by Haswell (1999) described in Chapter 2 and the laboratory results, the reduction factor for the lining stiffness should be limited to 0.6 to 0.7 for the cases when the existing tunnel is being unloaded due to adjacent excavation works. The experimental work to date has not shown that the factor should be lower. Applying the simplified Muir Wood's formula of

$I_e = \left(\frac{4}{n}\right)^2 I$, as suggested in LU standard 1-055 (2007), could lead to the reduction factor being as low as 0.3 for a typical tunnel ring consisting of 6 segments plus a key piece, where the number of joints, n , equals 7.

The experimental results suggest that using Morgan's equation (1961) to calculate the maximum bending moment for a tunnel ring assuming elliptical distortion based on a maximum magnitude of radial displacement under-predicts the maximum bending moment in the bolted segmental ring, even before the application of any reduction factor. Linear elastic solutions from Castigliano's second theorem would be better suited to estimating the bending moment distribution in a tunnel ring from a measured maximum radial displacement.

The experimental results suggest that changing the bolt loads does not affect the behaviour of a tunnel under hoop stress. Therefore it is not recommended to either tighten or loosen bolts as mitigating measures in LU tunnels. However, the level of bolt preload would probably affect the initial, as-built circularity of the tunnel lining, with higher preloads giving better contact of the surfaces of the longitudinal joints.

While not related to the case of an existing tunnel being unloaded due to adjacent excavations, the estimations for the onset of joint opening suggest that in situations where existing tunnel linings at depths of 20 m to 30 m are subjected to increased loading, extreme caution needs to be exercised. The calculations suggest that the joints will not be mobilised to reduce the lining stiffness until the segments are very highly stressed.

8 Conclusions and future research

8.1 Conclusions

8.1.1 Experimental set-up and materials

The literature review on the historical use of grey cast iron as a tunnel lining material provided valuable input in the planning of the laboratory tests. Considerations were given to ensure the experimental tunnel ring reflected as much as practicable the character of a tunnel ring in-situ. Details including caulking grooves and elliptical bolt holes were reproduced in the half-scale experimental segments. Grommets were specifically produced for the half-scale segment to be included as a test variable. The cast iron mix for the casting of half-scale grey cast iron segments for laboratory testing was specified using the composition obtained from the LU archives. Separately cast test bars were produced using the same mix so that tests were performed to determine the mechanical properties of the half-scale test segments.

Tensile tests were done at Imperial College on wrought iron bolts recovered from the Waterloo and City Line. The results were compared to the known behaviour for mild steel to confirm that it was appropriate to use mild steel bolts in the laboratory investigations for this research.

A revisiting and careful review of the experimental data provided by Thomas (1977) facilitated an initial estimate of joint bending moment capacity. This initial joint capacity estimate allowed a series of two segment tests of varying bolt preloads to be conducted safely and for the results to be interpreted under the assumption of elasticity.

Since the loading arrangement and boundary conditions rendered the two segment test statically determinate, the interpretation of the instrumentation and estimation of the bending moment could be compared against analytical equations. This was very useful in giving confidence that the methods used to interpret the instrumentation, especially the strain gauge readings, were appropriate. The same methods of interpretation were used in the full-ring test.

The design of the full-ring test rig was informed by findings from the two segment test. All the components were sized up based on the requirements of the full-ring test. Numerous trial tests were performed to optimise the design of the system and to determine the most appropriate method to control the loading of the test ring. The decision to apply load with actuators and to eliminate soil from the setup at the outset was crucial to the success of the experiments. It would not have been possible to conduct the same number of parametric tests within the time frame if soil was involved. The control of the loading would also have been a lot less precise if soil was involved.

Preliminary testing identified issues with the full-ring setup and as a result, two actuators were removed and replaced with reaction rods to increase the stability of the test ring during loading and to prevent free body movement. An additional tensile member was installed tangentially to the ring to enhance the stability.

Great care was taken during the installation and calibration of the instrumentation prior to the commencement of the full-ring test. Baseline readings were taken to ensure that all the channels were logging correctly.

This loading setup allowed very controlled and repeatable tests to be conducted on the half-scale cast iron tunnel lining ring. For the first time, the global and local joint response of a bolted segmental cast iron lining was studied experimentally and the internal actions relating to a particular deformed shape could be measured. The design and development of the bespoke loading facilities from scratch is one main contribution of this research to the understanding of bolted segmental lining behaviour.

8.1.2 Ground stresses and tunnel shapes

Circularity surveys of 45 km of LU running tunnels conducted in 2004 to 2005 found that the tunnels had squatted (horizontal diameter is greater than the vertical diameter). Of all the rings surveyed, 92% had ovalisations less than 1%. LU standard 1-055 (Civil Engineering – Deep Tube Tunnels and Shafts) nominated an ovalisation of 0.6% to be used in tunnel assessments should the actual ovalisation be unavailable.

In order to obtain a squat in tunnels, the ratio of horizontal stress to vertical stress in the soil needs to be less than 1.0. However, profiles for the variation of the at rest earth pressure coefficient with depth produced for the Crossrail project for sites across Central London gave the ratio of horizontal stress to vertical stress of between 1.0 to 1.5 for depths of 20 m BGL to 30 m BGL (GCG, 2009a & 2009b). This depth range is most relevant to the bolted segmental cast iron tunnels in the LU network. The inconsistency between the deformed shape of the existing tunnels and the ground pressures is well known in the industry but remains unaddressed.

The 2D FE simulations by Avgerinos (2014) of the situation in Central Line east of Lancaster Gate using initial earth pressure at rest coefficient of 1.3 to 1.4 showed that after allowing 100 years of consolidation, the maximum distortion in the existing tunnel was 0.2% squat. This implied that although the measured distortion is generally greater than 0.2% in the LU tunnels, it may be due to imperfect construction and self-weight distortion rather than ground loading. The field exercise conducted at the LU Acton Depot certainly showed that tunnel rings constructed above ground could display greater than 1% of ovalisation from self-weight and initial out-of-built.

Furthermore, FE analyses showed that the action of shear stresses on the tunnel lining could control the deformed shape of the lining, but in the laboratory it was only possible to apply normal stresses.

Historically, between 45% and 100% of the overburden pressure have been reported to act on bolted grey cast iron tunnel linings. At the Tottenham Court Road station tunnel, the strain release when cast iron segments were removed from a tunnel ring was measured by electrical and mechanical strain gauges. The hoop force calculated from the magnitude of change in strain ranged from being 63% lower to 30% higher than that predicted assuming full overburden unloading and a constant modulus of 100 GPa. This exercise and the literature review highlighted the difficulty in ascertaining the hoop stress in existing tunnel linings.

In the laboratory tests, the test ring was subjected to different magnitudes of hoop stress before the distortion was imposed with the aim to capture the ring behaviour under a range of overburden pressures.

8.1.3 Deformation due to adjacent tunnel excavations

From a review of existing literature, it is noted that the previous maximum recorded distortion in bolted cast iron tunnels due to new tunnel excavations is approximately 0.1% of diametral strain. Field monitoring conducted by the Imperial College research team at the Central Line running tunnel also recorded, using tape extensometer measurements, a maximum change in span of $\pm 0.1\%$ due to the Crossrail tunnel excavations.

The preliminary strain data measured by ETH Zurich with optical fibres during the passage of the first Crossrail TBM suggested that longitudinal deformation along the Central Line tunnel crown occurred along the segment and not at the circumferential joints. The optical fibres installed circumferentially on the internal surface of a tunnel ring was able to detect movement across the longitudinal joints when the Central Line tunnel ring experienced changes in diametral span of no greater than $\pm 0.05\%$ due to the passage of the first Crossrail TBM. LVDT displacement transducers and potentiometric displacement transducers were unable to measure reliably any joint movements in the field.

8.1.4 Loading regime for full-ring test

Since there is a lack of reliable field measurements concerning how the loading from the ground is transferred to the tunnel lining, a loading regime considered to be simple, stable and repeatable was selected for the full-ring test.

Because of the possibility to control and measure the loading applied onto the test ring in the laboratory environment, and because the extreme fibre stresses was limited to below 40% of the ultimate tensile strength of the cast iron, it was considered appropriate to analyse the results of the full-ring test using linear elastic solutions. Formulae for the calculation of hoop stress, bending moments and deflections are available for a continuous circular ring under any number of equal radial forces equally spaced (Young and Budynas, 2002). By assuming linear elastic behaviour, superposition could be used to cover the loading conditions imposed in the laboratory tests.

The ultimate tensile strength of the cast iron of 120 MPa was selected based on the lower limit of the tensile test results from the 20 mm diameter grey cast iron specimens. The tensile tests conducted on cast iron test specimens also showed that at stress levels below 40% of the ultimate tensile strength, the magnitude of plastic strain was fairly constant at below 0.02%.

The loading regime was designed to allow a series of parametric tests to be carried out on the same half-scale test ring so that as much information as possible was obtained from using one ring. One way to achieve that was to ensure that the maximum tensile stress in the ring for the most critical case with the lowest hoop stress was within 40% of the ultimate strength of cast iron since it was shown that within this range the behaviour of the cast iron was essentially elastic.

In the tests, a circular ring at rest is loaded up until a desired level of uniform hoop stress is reached. The highest normal load applied by the actuators was limited to 40 kN due to the testing duration and the decision not to leave the loaded actuators unsupervised. This load is representative of the depth of cast iron tunnels in the LU network.

Given the restriction placed on the tensile stress in the extreme fibre, the ovalisation imposed on the test ring was not as large as the ovalisation measured in real life. However, the ovalisation in the real tunnels could be a result of the initial out-of-built, lining self-weight, the transfer of ground loading onto the segments, or deformation due to subsequent enlargement of certain tunnel sections. In the laboratory, ovalisation of the ring can be attributed wholly to the loading and unloading of segments.

It was considered prudent to build up an understanding of the bolted segmental ring behaviour at small deformations of 0.1% to 0.2% before taking the ring to larger, inelastic, deformations. The results from the parametric experiments performed at small deformations will help direct the planning of future tests so that the usefulness of the very limited number of tests taken to large deformations will be maximised.

8.1.5 Parametric studies

The experimental setup and control system have enabled a set of parametric experiments to be completed successfully. The influence of initial normal loads, which gives rise to hoop forces at the joints, and the influence of bolt preloads, were investigated.

Linear elastic solutions were used to compare the measured behaviour of the bolted segmental ring with the analytical response of a continuous ring.

For a typical cast iron LU tunnel at 20 m BGL to 30 m BGL, the tests conducted at all-round normal loads of 40 kN, were the most appropriate. At this level of normal load, the strain measurements from both the unloading tests and loading tests indicated that the bolted segmental ring behaved as a continuous ring under the imposed distortion.

The laboratory experiments also showed that, for the same amount of distortion, reducing the hoop stress prior to distortion had the effect of reducing the stiffness of the bolted segmental ring. The experimental results showed that whether the bolted segmental test ring was distorted by unloading or by loading, the reduction in bending moment was similar for a given magnitude of all-round normal load. The maximum reduction of 30% was measured when the normal load was 3.4 kN per actuator, roughly equating to 2 m of overburden pressure.

Experiments showed that decreasing the all-round normal load prior to distortion decreased the stiffness of the joint, as indicated by larger joint movements and also larger changes in bolt loads upon distortion.

8.1.6 Influence of bolts

In the past, mitigating measures adopted to reduce the impact of nearby tunnel construction on existing cast iron tunnels have included loosening bolts in one case (Moss and Bowers, 2006), and tightening bolts in another case (Kimmance et al, 1996). In the full-ring experiments, the influence of

varying bolt preloads on the behaviour of the bolted lining was studied on a global scale and on a local scale.

On the global scale, it was shown that under different magnitudes of bolt preload, the bending moment calculated from measured strains deviated from the bending moment distribution of a continuous ring in a similar manner. There was no evidence to indicate that an initial higher or lower preload had an impact on the reduction of bending moment for the small distortion imposed. Even after the inclusion of grommets, the observed pattern of behaviour did not change. The main effect of including grommets was that the bolt preload reduced with time. This implies that in tunnels where grommets were used, the bolts would be less tight than in tunnels without grommets.

Looking at the behaviour of the joints more closely, it was established that there was no clear relationship between the initial magnitude of bolt preload and the subsequent change in bolt load upon distortion. Similarly, the different magnitudes of bolt preload, and the presence or absence of grommets, had very little impact on the magnitude of joint movement upon distortion.

Overall, for the small distortions imposed, the presence of compressive hoop force renders the magnitude of bolt preload insignificant in terms of influencing bolted segmental lining behaviour. Therefore it is not recommended to either tighten or loosen bolts as mitigating measured in LU tunnels. However, the level of bolt preload would probably affect the initial, as-built, circularity of the tunnel lining, with higher preloads giving better contact of the surfaces of the longitudinal joints.

8.1.7 Onset of joint opening

The distortions achieved in the experiments were very small compared to the ovalisation measured in the field. The two segment test results indicated that joint stiffness decreased with increasing joint opening. Further experimental work is necessary to determine the reduction in stiffness of a bolted segmental tunnel lining as distortion of the tunnel ring increases. In the meantime, analytical calculations were used to predict the onset of joint opening.

Equations derived for the calculation of bending moment, hoop stress and deflection of a continuous ring subjected to equally distributed loading were used to determine the distortion necessary to first cause the joints of a bolted segmental ring to open.

For a tunnel at a depth of 24 m BGL, a typical depth for LU tube tunnels, the inner radius needs to reach a 0.21% increase by unloading at 90 degrees before the joints would start to open and contribute to the reduction in the overall stiffness of the ring. At such deformation, the extreme fibre stress in the ring is 22% of the ultimate tensile strength. This is within the elastic limit of grey cast iron, which in practice is usually taken as 25% of the ultimate tensile strength.

At this same depth, the inner radius needs to reach 0.58% reduction by increasing the load at 90 degrees before the joints would start to open. At such deformation, the extreme fibre stress in the ring is 94% of the ultimate tensile strength. For this case, the ring segments would deform inelastically before the joint would play a part in the distortion of the ring. This implies that caution needs to be exercised when engineering works which would increase the loading on the tunnel lining, for example piling works, are carried out close to existing tunnels.

8.1.8 Current lining assessment methods

One of the methods adopted in the industry with relation to the assessment of bending moment in tunnel linings is to use Morgan's equation (1961) to calculate the bending moment induced in the tunnel ring from a certain magnitude of distortion. The bending moment obtained using linear elastic solutions are compared against that estimated using Morgan's equation as well as the experimental measurements. The experimental results suggest that using Morgan's equation (1961) to calculate the maximum bending moment for a tunnel ring assuming elliptical distortion based on a maximum magnitude of radial displacement underpredicts the maximum bending moment in the ring, even before the application of any reduction factor. The linear elastic solutions appear to be better suited to estimating the bending moment distribution in a tunnel ring from a measured maximum radial displacement.

Another method adopted in the industry with relation to the assessment of bending moment in tunnel linings is to use the complete continuum model and calculate the bending moment without presuming the elliptical distortion. However, the complete continuum model requires an assumption of the ratio of horizontal to vertical stress in the continuum. It is common practice in the industry to assume a ratio of 0.7 for tunnel assessment purposes to obtain the 'squat' observed in real tunnels, despite the knowledge that at the depths relevant to LU cast iron tunnels the ratio should be closer to 1.0 or higher.

In the industry, whether the bending moment is initially estimated based on distortion or an assumption of ground stresses, the flexural rigidity of the bolted ring is reduced by adopting Muir Wood's (1975) reduction formula for the second moment of area for the ring to take into account the presence of the joints. In LU standard 1-055 (Civil Engineering – Deep Tube Tunnels and Shafts), the

reduction equation by Muir Wood (1975) is simplified to $I_e = \left(\frac{4}{n}\right)^2 I$, meaning that for a typical running tunnel ring consisting of six segments plus a key piece, $n=7$ and the effective second moment of area used to compute bending moment is only 33% of the original second moment of area.

The experimental findings from the full-ring tests taken to squatting distortion of 0.13% and egging distortion of 0.07% did not achieve a lining reduction factor lower than 0.7. The FE study conducted by Haswell (1999) described in Chapter 2 also recommended the reduction factor for the lining stiffness to be limited to 0.6 to 0.7. In both the experimental work and the FE simulations the tunnel was assumed to be perfectly circular and any deformations were the result of external loading. Future work needs to look into the effect of starting with an out-of-circular tunnel ring prior to the application of loading and tracking the reduction in lining stiffness as the deformation increases.

8.2 Future research

8.2.1 Experimental work

The full-ring experiments to date have investigated the influence of the joints on lining stiffness at small distortions. The objective of future experiments is to measure the reduction in ring stiffness as a result of the contribution of the joints once they begin to open as distortion increases. Given that

currently there are only sufficient half-scale cast iron segments for two tunnel rings, great care needs to be taken in the planning of the tests taking the cast iron beyond 40% of its ultimate tensile strength.

The most relevant and critical case to investigate for tunnels affected by nearby excavations would be to bring all the actuators to 40 kN in Stage One followed by unloading along one diameter until the ring is taken to failure. It is suggested that the bolts should be preloaded to 7.5 kN. This magnitude of bolt preload is probably most relevant to LU tunnels where grommets are not used (see Section 5.3 in Chapter 5).

After reaching 40 kN in all the actuators, the actuator at 90 degrees should be set to displacement control so the ring distorts to an ellipse incrementally. The author suggests taking a scan of all the instruments for each 0.5 mm of outward displacement as measured by the displacement transducer at 90 degrees (refer to Figure 6-30). This should continue until failure is reached. It is expected that the cast iron ring will fail in a brittle manner.

The following tasks should be performed prior to taking the cast iron ring to failure:

- Repeating the tests outlined in Table 6-5 for 10 kN, 20 kN and 40 kN but this time restricting the movement of the circumferential flanges. Bars could be inserted vertically into alternate pairs of circumferential bolt holes and locked so that relative movement between pairs of the circumferential flanges is restricted. This is to simulate the situation in tunnels in-situ where the circumferential flanges are inhibited from moving because they are bolted to the circumferential flanges in the adjacent tunnel rings. The three-dimensional FE analysis for the two segment test described in Chapter 5 showed that restricting the circumferential flange movement has a stiffening effect. Comparing this set of results with the current available full-ring test results will allow the increase in stiffness in real tunnels to be quantified.
- It was mentioned in Chapter 6 that the current loading sequence takes 5.5 hours to bring all the actuators to 40 kN. It is worth re-visiting the sequence and speed up the loading to 40 kN if possible to ensure the unloading test to failure could be completed in a day.
- It was shown in Chapter 7 that three-element strain gauge rosettes do not provide much additional insight to the lining behaviour. It is worth considering removing some three-element strain gauge rosettes to free up a number of logging channels and installing more two-element strain gauges onto the segments. It would be particularly interesting to measure the changes in strain at 90 degrees and compare with the bending moment calculated using Morgan's formula (1961).
- A safety frame will need to be designed and installed.

Six segments would be required for the first full-ring unloading test to failure as described above. The remaining six segments should be used to build another test ring. It will be necessary to re-use the segments from the two segment test. It is suggested that the bolts should be preloaded to 7.5 kN.

Instead of trying to achieve an initially perfectly circular assembled shape, the second test ring should be assembled with an initial ovalisation of 0.4% if possible, with the diameter at 90-270 degrees being greater than the diameter at 0-180 degrees. The FE analysis described in Chapter 6 simulating the existing condition in the Central Line running tunnel prior to Crossrail excavations showed that when ground loading was applied to an initially perfectly circular tunnel ring, 0.2% ovalisation was achieved. However, Chapter 2 reported that survey data from the Central Line suggesting an average ovalisation of 0.6%. This suggests an initial ovalisation of 0.4% as a result of out-of-built and lining self-weight.

The test ring with the initial out-of-built should then be taken to 10 kN all-round loading and then the actuator at 90 degrees should be placed under displacement control to move 2 mm away from the centre as outlined in Table 6-5. A scan of all the channels should be taken when the initial out-of-built

was achieved, when the 10 kN all-round loading was reached and also when the 2 mm displacement was attained. The ring should then be fully unloaded and the initial out-of-built should be checked and re-adjusted to 0.4% ovalisation. The test ring should then be taken to 20 kN and then the actuator at 90 degrees set to displace 2 mm away from the centre. This should be repeated for all-round loads of 40 kN to compare with current set of results with an initially circular ring. Ultimately, the test with the 40 kN all-round loads should be taken to failure by setting the actuator at 90 degrees to move away from the centre incrementally.

When estimating the bending moment from the strain measurements as the ring is deformed beyond the current range presented in this thesis, it is recommended to adopt the best fit stress-strain relationship presented in Chapter 4 instead of using $E=100$ GPa as was adopted in this thesis.

8.2.2 Field work

The author identifies two areas of focus for future field work. One area of focus relates to the transfer of ground stresses onto tunnel linings, and the second area of focus relates to the measurement of tunnel distortion with special attention paid to movement at the joints.

The field work in relation to ground stresses should address the following uncertainties:

- The proportion of overburden load transferred from the ground onto the lining acting as normal stresses acting on the lining.
- The proportion of overburden load transferred from the ground onto the lining acting as shear stresses acting on the lining.
- The onset of ground stresses transfer onto tunnel lining.

The laboratory work has shown that the behaviour of the tunnel lining is very dependent on the level of hoop stress in the lining. For assessment purposes it is recommended to assume 100% overburden acting on the tunnel lining. However, finite element modelling as well as previous field measurements indicate that only a proportion of the overburden load is transferred onto the lining. Also, implicit in the experimental work is the assumption that ground loading is transferred to the lining as normal stresses. The numerical work described in Chapter 6 has highlighted the difference in the behaviour of the tunnel ring depending on whether the ground stresses are acting as normal stresses or shear stresses. In particular, the variation in hoop stress around the tunnel lining due to the action of shear stresses at the extrados of the lining would have implications on the assessment of joint and segment capacities and response.

However, since it is notoriously difficult to obtain reliable measurements of ground stresses, it may be more feasible to instrument new tunnel segments with strain gauges before installation and monitor the changes in strain and calculate the magnitude of the hoop stress and bending moment developing in the lining segments.

The field work in relation to tunnel distortion should address the following uncertainties in new tunnels:

- The initial magnitude of tunnel distortion due to imperfect construction and self-weight of lining and the subsequent distortion due to ground loading.
- The magnitude of longitudinal joint movement in relation to the tunnel ring distortion.

In newly constructed tunnels, it may be difficult to gain access to the tunnel rings immediately after construction as the machinery may still be in the way, and by the time the tunnelling machine has passed most of the initial tunnel distortion might have already occurred. In the cases where it is possible to arrange for access, it is recommended that the tunnel ring distortion measurements be made with a tape extensometer. Discrete manual readings would have to be taken, but this has proved to be a most reliable instrument. The results are also easy to interpret. It is envisaged that further development is required for a suitable instrument for measuring longitudinal joint movements in-situ. The instruments used at Tottenham Court Road station and in Central Line did not perform very well. Furthermore, new tunnels are generally constructed from precast concrete segments rather than grey cast iron.

The field work in relation to tunnel distortion should address the following uncertainties in existing tunnels affected by adjacent construction works:

- The magnitude of longitudinal joint movement in relation to the tunnel ring distortion.
- The magnitude of circumferential joint movement in relation to the tunnel distortion in the longitudinal direction.

The preliminary results provided by ETH Zurich on the monitoring of longitudinal strains with an optical fibre in Central Line have proved to be very interesting. The main advantage of the ETH system was the ability to measure the strain across a very small distance, thus being able to differentiate between the strains in the segment and the strains across the circumferential joints.

8.2.3 Numerical modelling

The development of a three-dimensional FE model to simulate the full-ring test would be of great benefit to the overall research project. This is especially the case since the number of laboratory experiments is limited. The first challenge is to extend the three-dimensional model developed for the two segment test to simulate the full-ring test at low deformations where the material properties of the cast iron could be approximated with its elastic properties.

The second challenge is to develop and apply a new constitutive model to simulate the full-ring test taken to failure. Interrogation of the FE results will allow the re-distribution of stresses around the joints to be understood. This will be particularly helpful because it is very difficult to install strain gauges on the longitudinal joints and to interpret the results in a meaningful way in the laboratory.

Advanced numerical modelling has the potential to reveal the relationship between displaced shapes and stresses in existing tunnel linings so that realistic and acceptable tolerances can be applied safely in practice.

Figures

2 Literature review

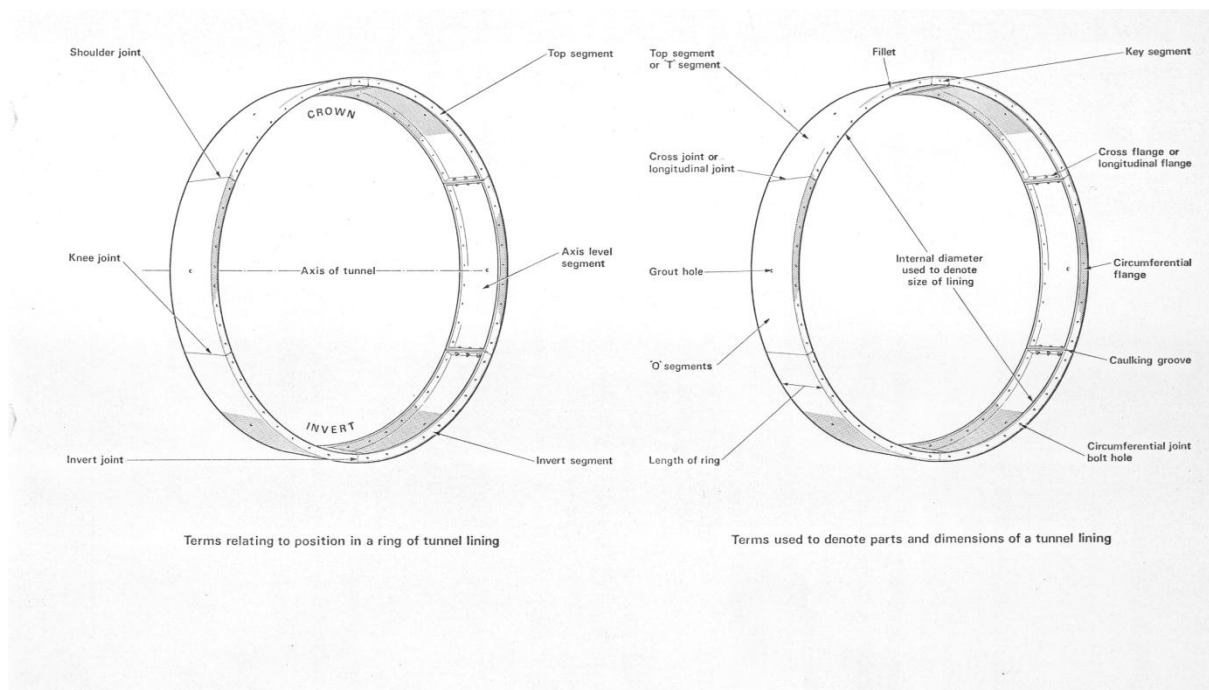


Figure 2-1. Terms relating to positions in a ring (Thomas, 1974).

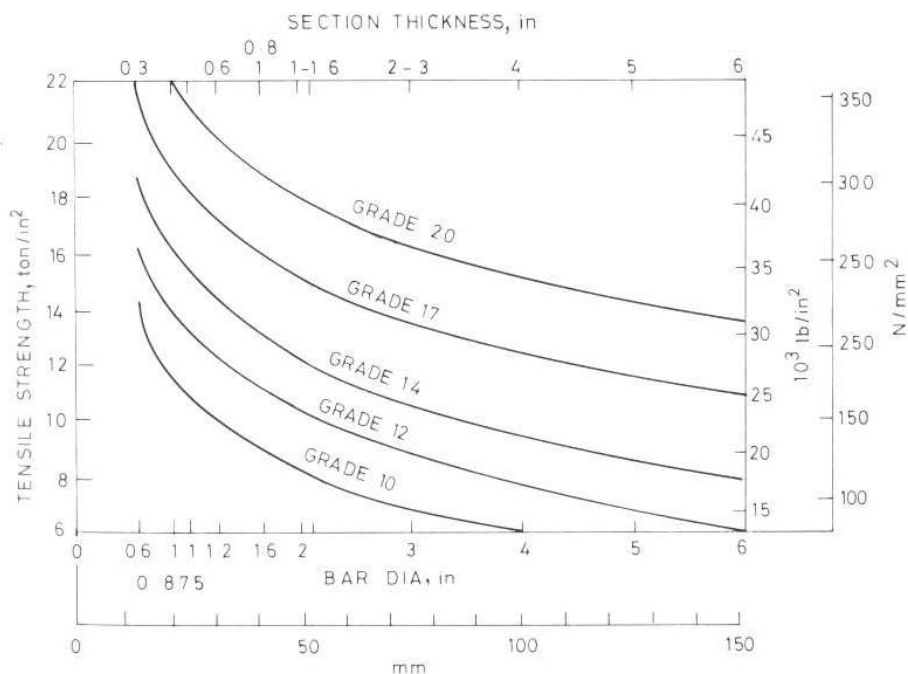


Figure 39. Effect of section on strength

Figure 2-2. Effect of section on strength (Angus, 1976).

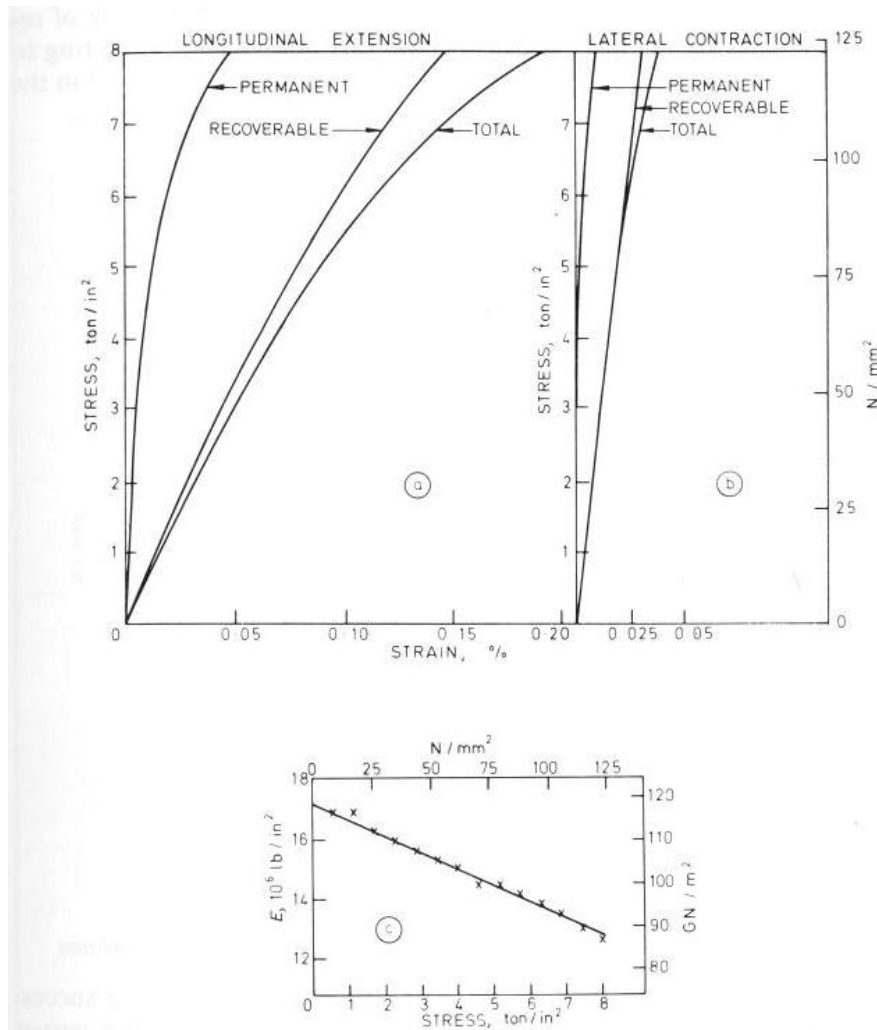


Figure 46. Stress/strain characteristics of a grey iron to BS 1452 Grade 17 tested on a 2.1 in bar: actual tensile 13.8 ton/in² (213 N/mm²) (a) Longitudinal stress/strain curve in tension; (b) Lateral stress/strain curve in tension; (c) Elastic modulus E (longitudinal) at different levels of stress

Figure 2-3. Stress/strain characteristics of a grey iron to BS1452 Grade 17 tested on a 2.1in bar. (a) Longitudinal stress/strain curve in tension; (b) lateral stress/strain curve in tension; (c) Elastic modulus E (longitudinal) at different stress levels (Angus, 1976).

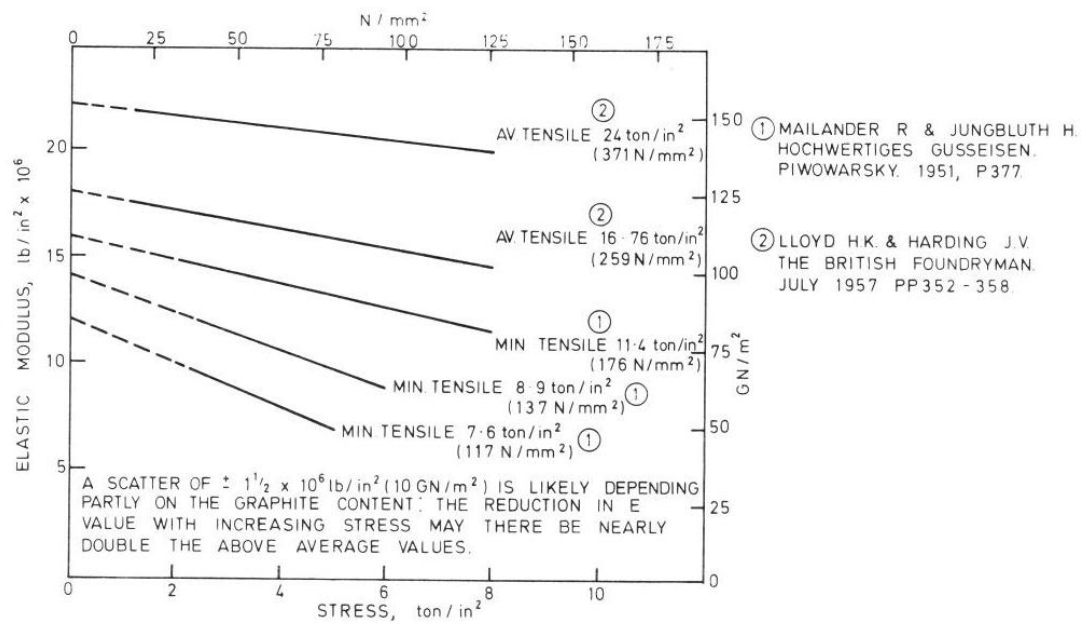


Figure 43. Elastic modulus of cast iron. Variation with tensile stress for irons of different strength (1.2 in bar)

Figure 2-4. Elastic modulus of cast iron. Variation with tensile stress for irons of different strength (Angus, 1976).

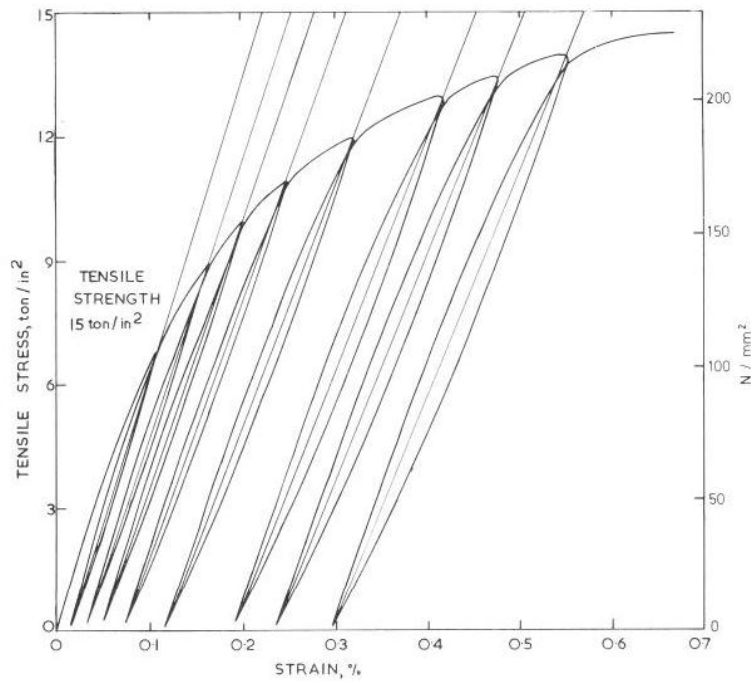


Figure 47(a). Tensile stress/strain curve of a flake graphite cast iron—as-cast condition

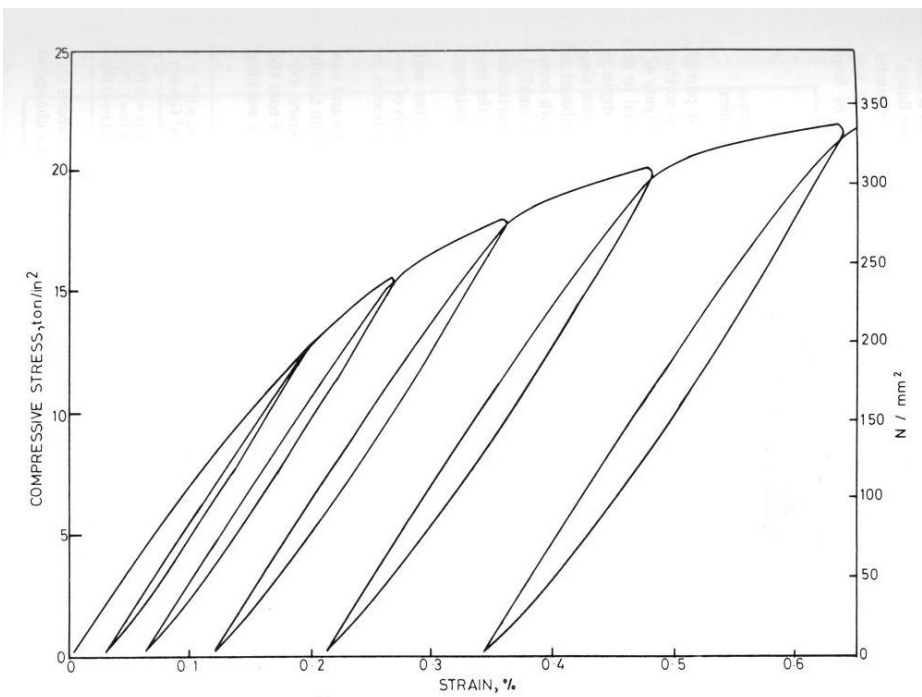


Figure 47(b). Compressive stress/strain curve of a flake graphite cast iron—as-cast condition

Figure 2-5. Stress/strain relationship in (top) tension and (bottom) compression (Angus, 1976).

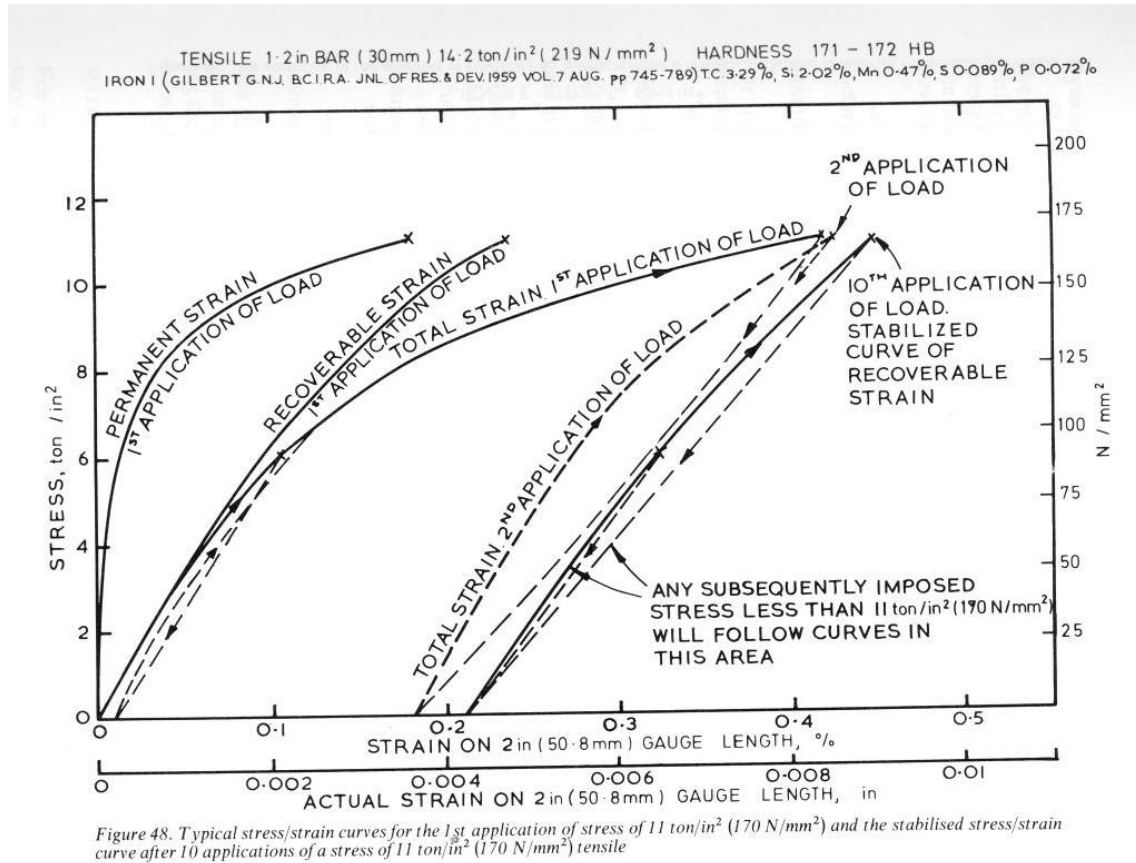


Figure 2-6. Typical stress/strain curves for the 1st application of stress of 170 MPa and the stabilised stress/strain curve after 10 applications of 170 MPa tensile stress (Angus, 1976).

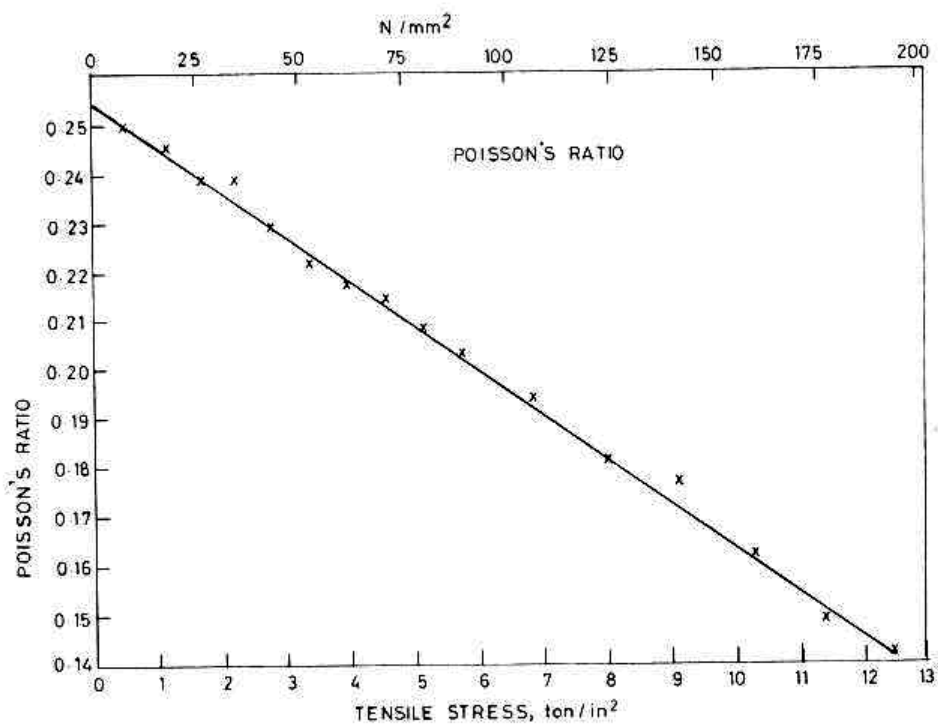


Figure 54(a)

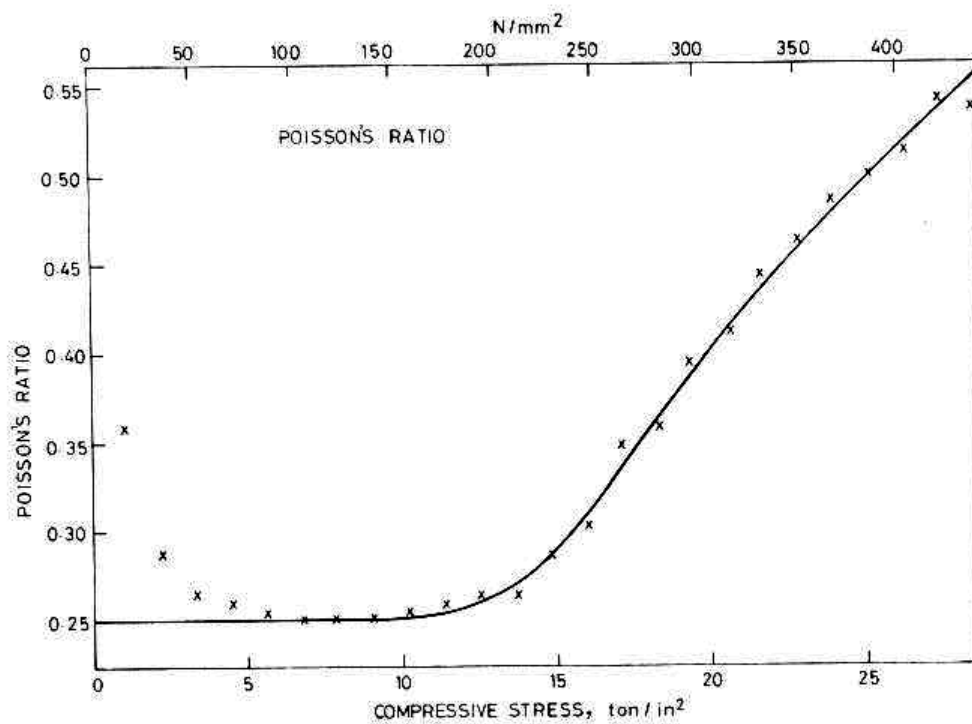


Figure 54(b)

Figure 2-7. Variation in Poisson's ratio with stress (a) tensile (b) compressive (Angus, 1976).

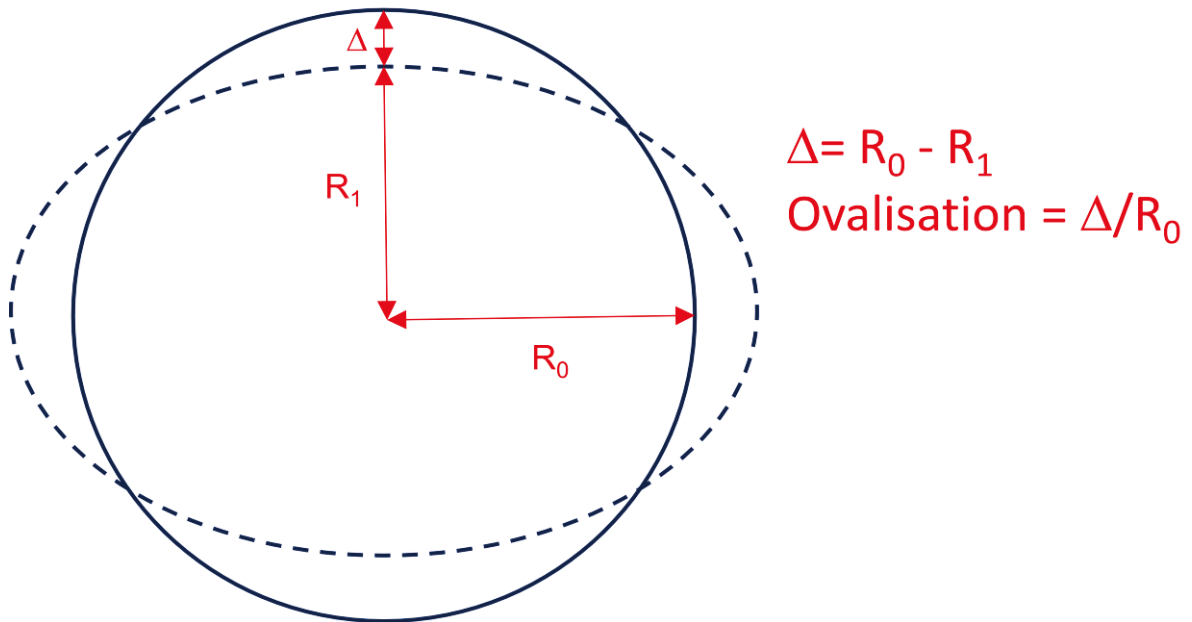


Figure 2-8. Definition of 'ovalisation' parameter.

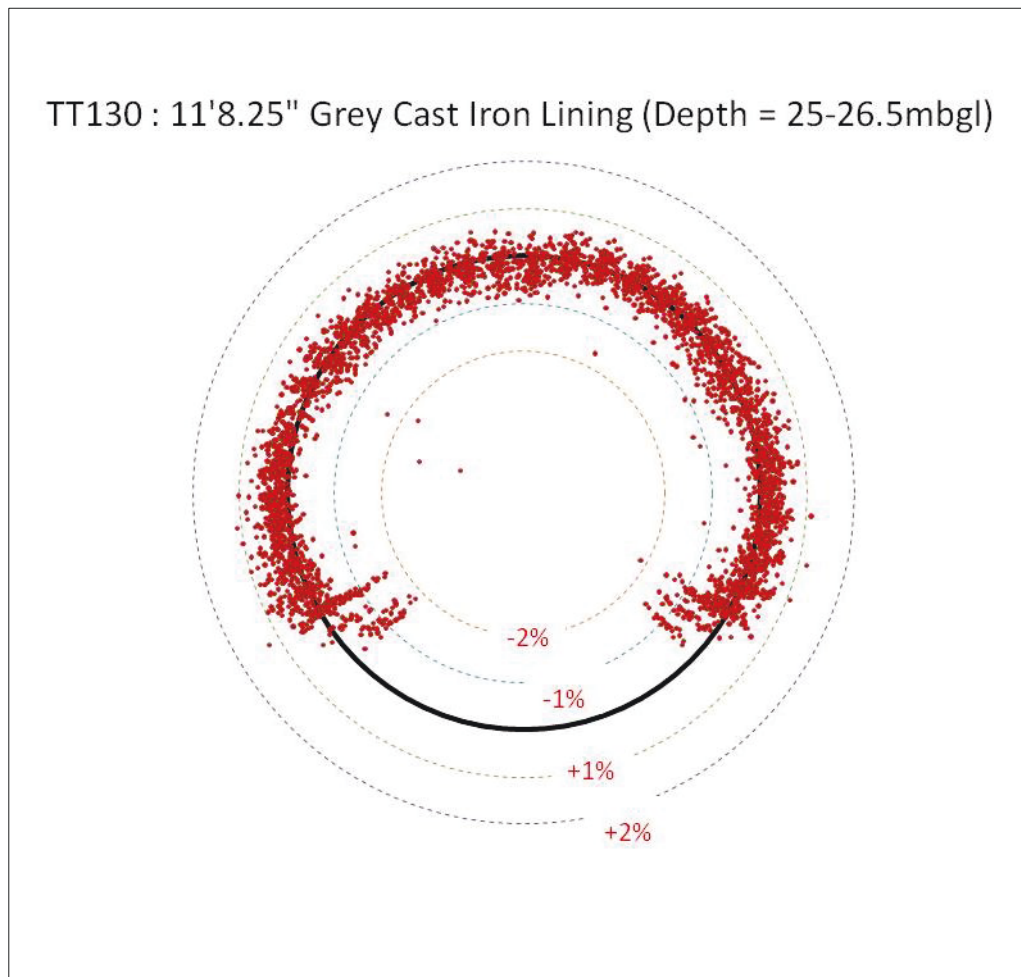


Figure 2-9. Summary of survey results for Northern Line running tunnel between Tottenham Court Road and Leicester Square. Majority of survey points fall between ovalisation of $\pm 1\%$.

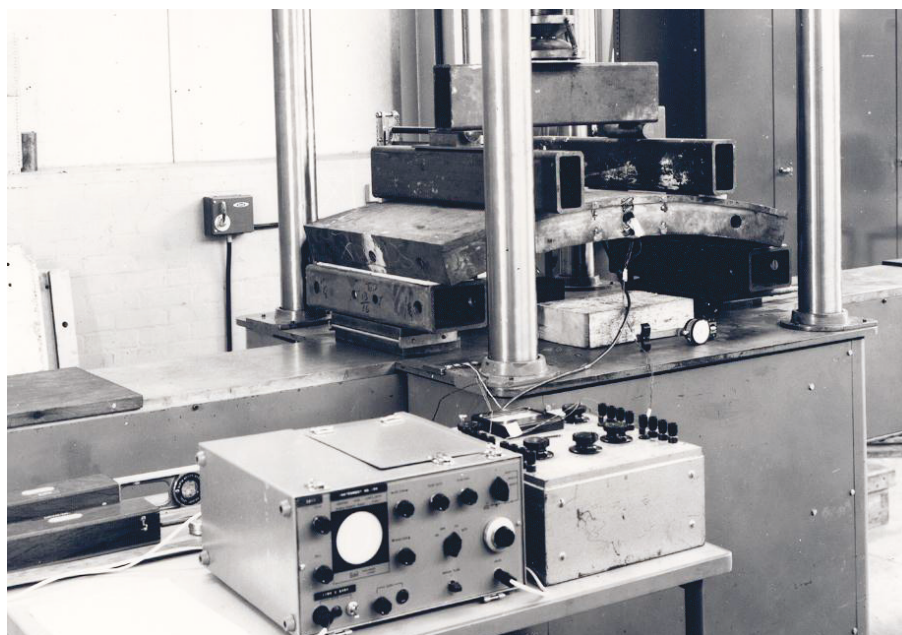


Figure 2-10. Bending test on SGI tunnel segment (Thomas, 1977).

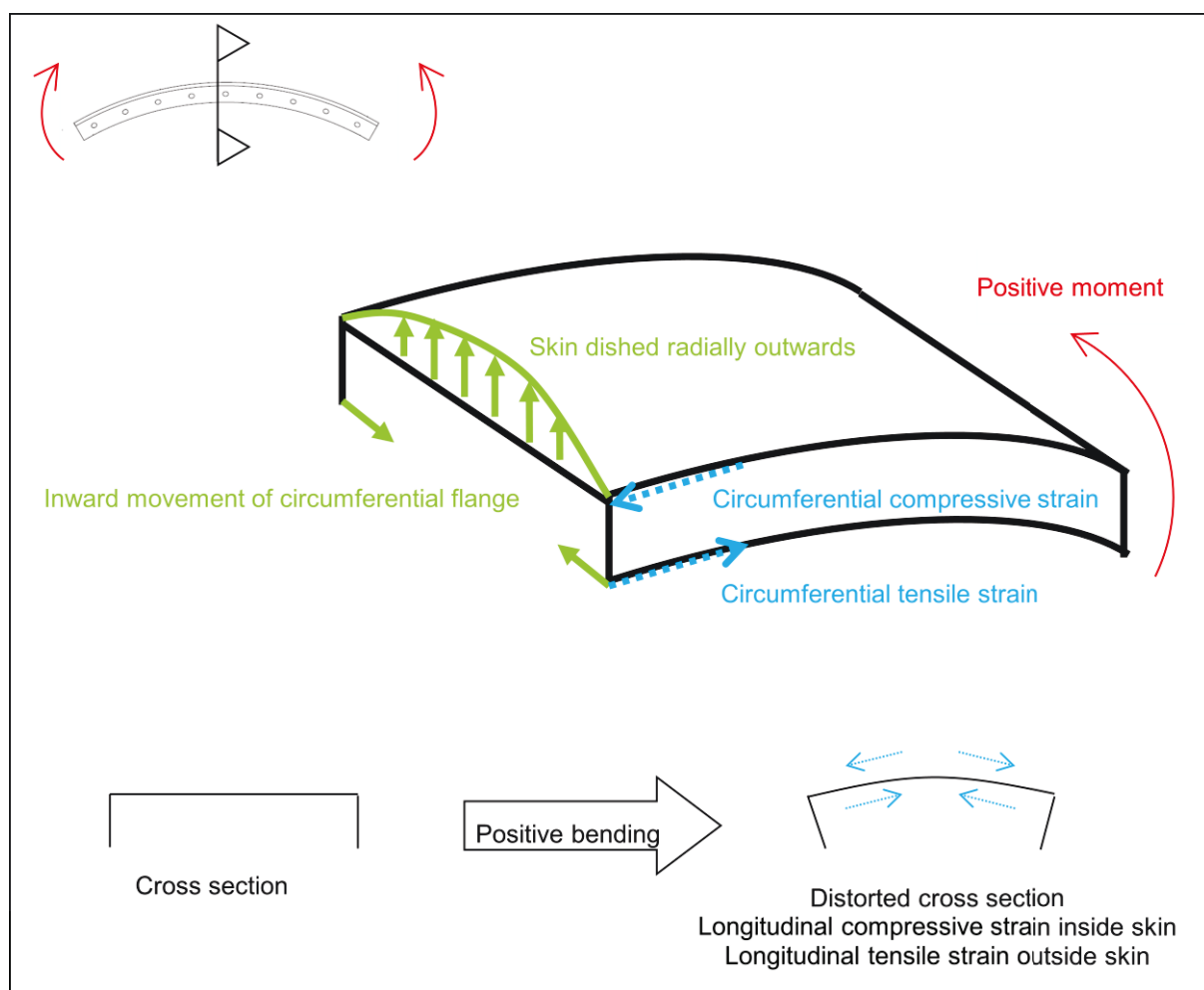


Figure 2-11. Schematic diagram showing distortion associated with positive bending (distortion is exaggerated).

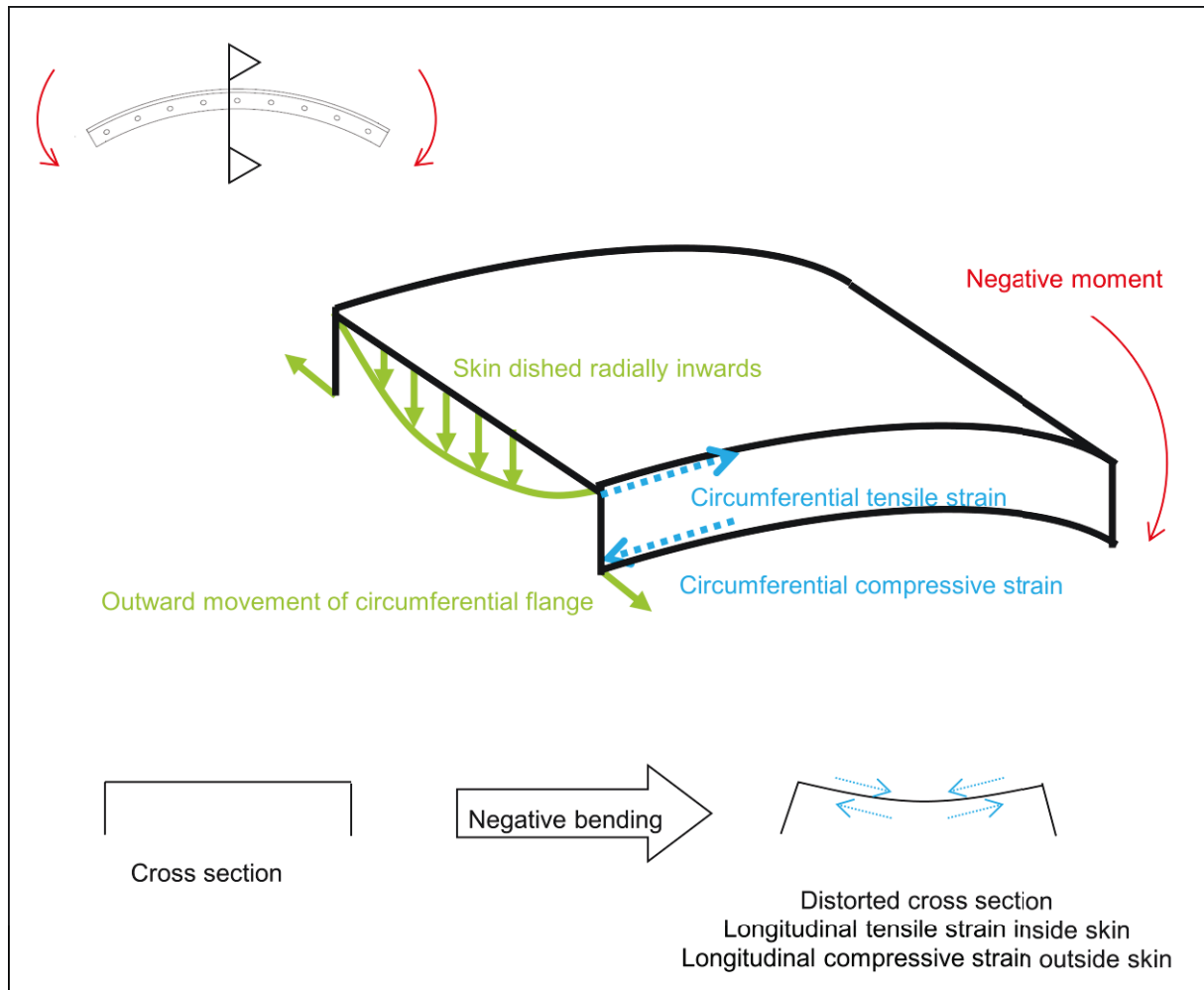


Figure 2-12. Schematic diagram showing distortion associated with negative bending (distortion is exaggerated).

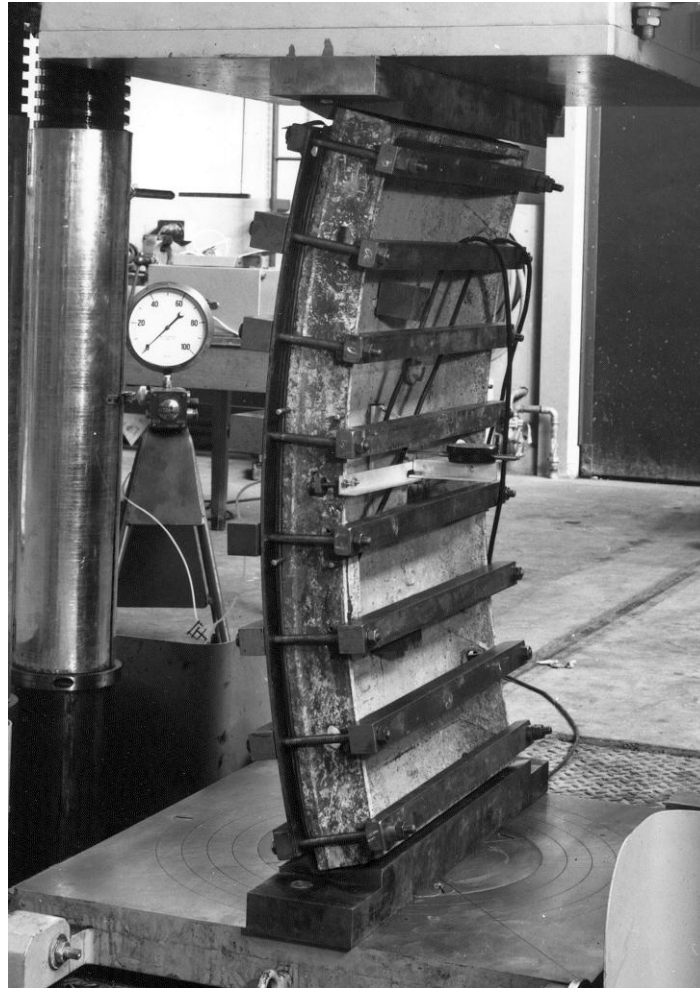


Figure 2-13. Rubber bag strapped to the back of a SGI segment for the application of a uniform pressure (Thomas, 1977).

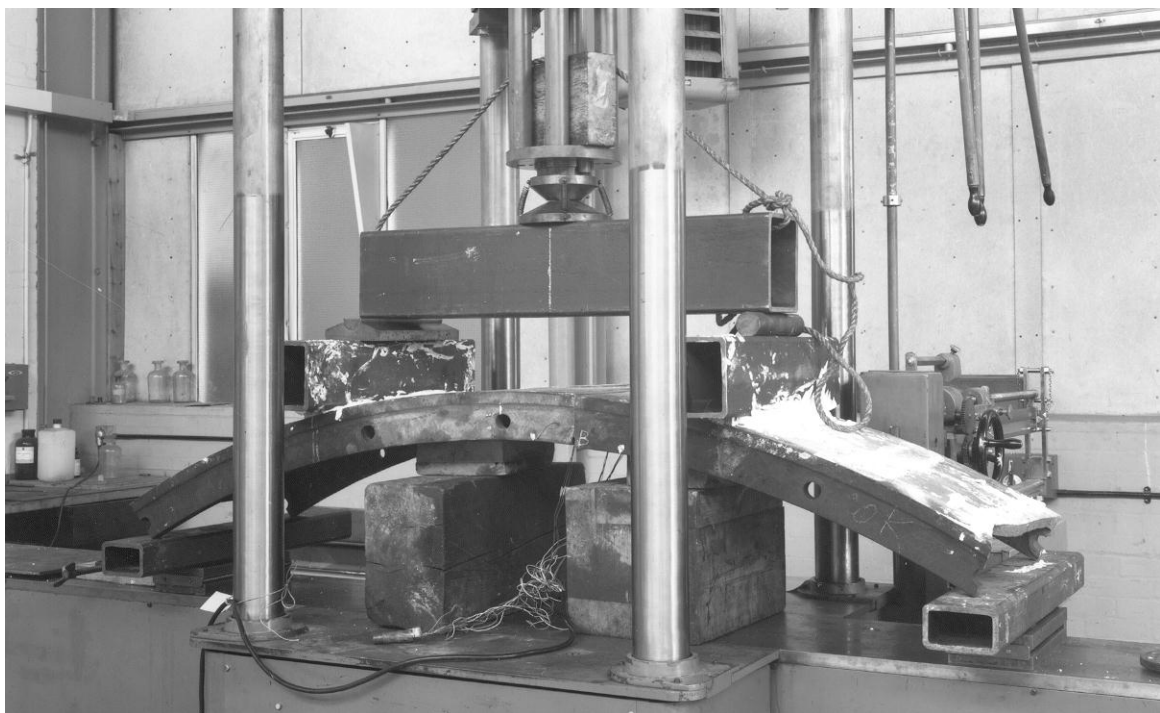


Figure 2-14. Bending test on the horizontal joint between two half segments of GCI lining (Thomas, 1977).

3 Field instrumentation



Figure 3-1. A section of GCI tunnel constructed above ground at the LU Acton Depot.

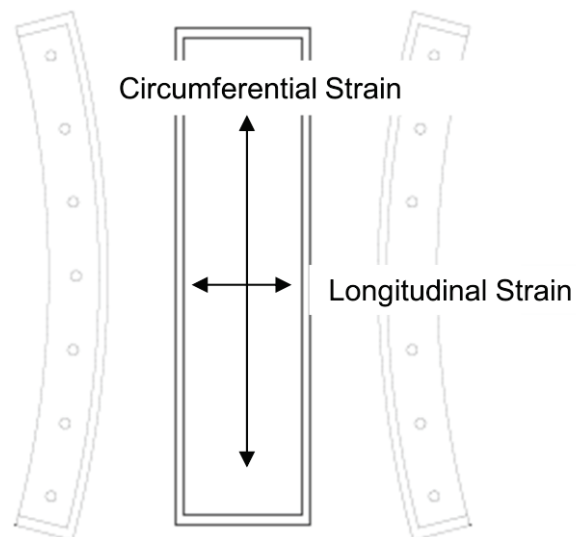


Figure 3-2. Directions of circumferential and longitudinal strains.

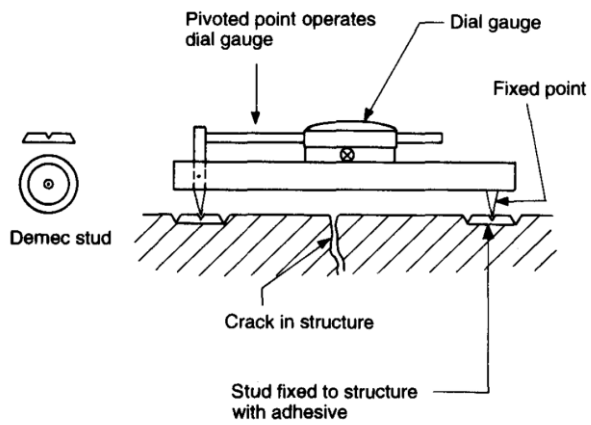


Figure 3-3. The DEMEC gauge.

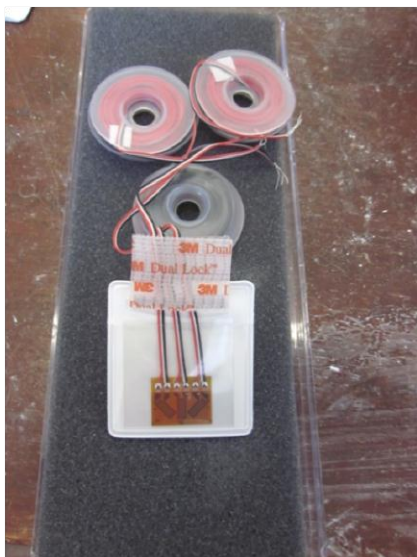


Figure 3-4. Rosette type electrical resistance strain gauge.



Figure 3-5. P3 Strain Indicator.



Figure 3-6. LVDT displacement transducer in metal holder.

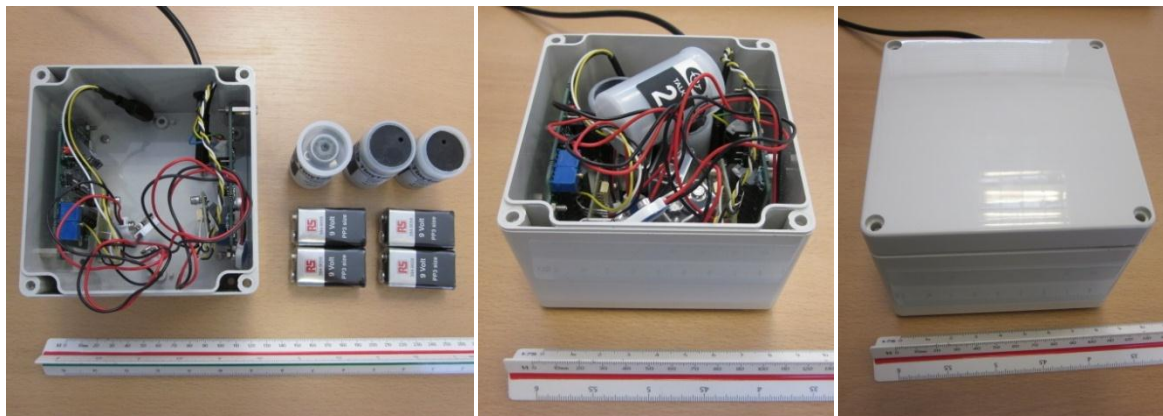


Figure 3-7. Dataloggers for LVDT and temperature gauge housed in plastic box.

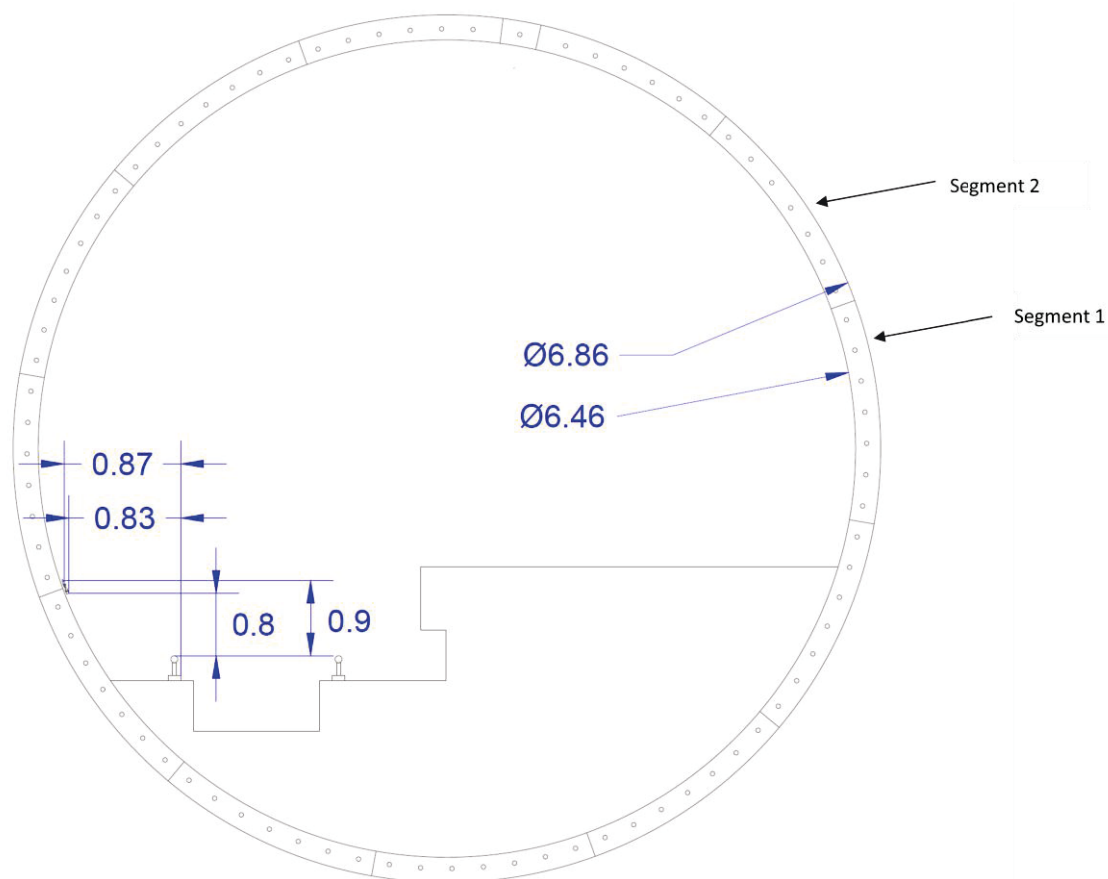


Figure 3-8. Northern Line northbound platform tunnel looking north.

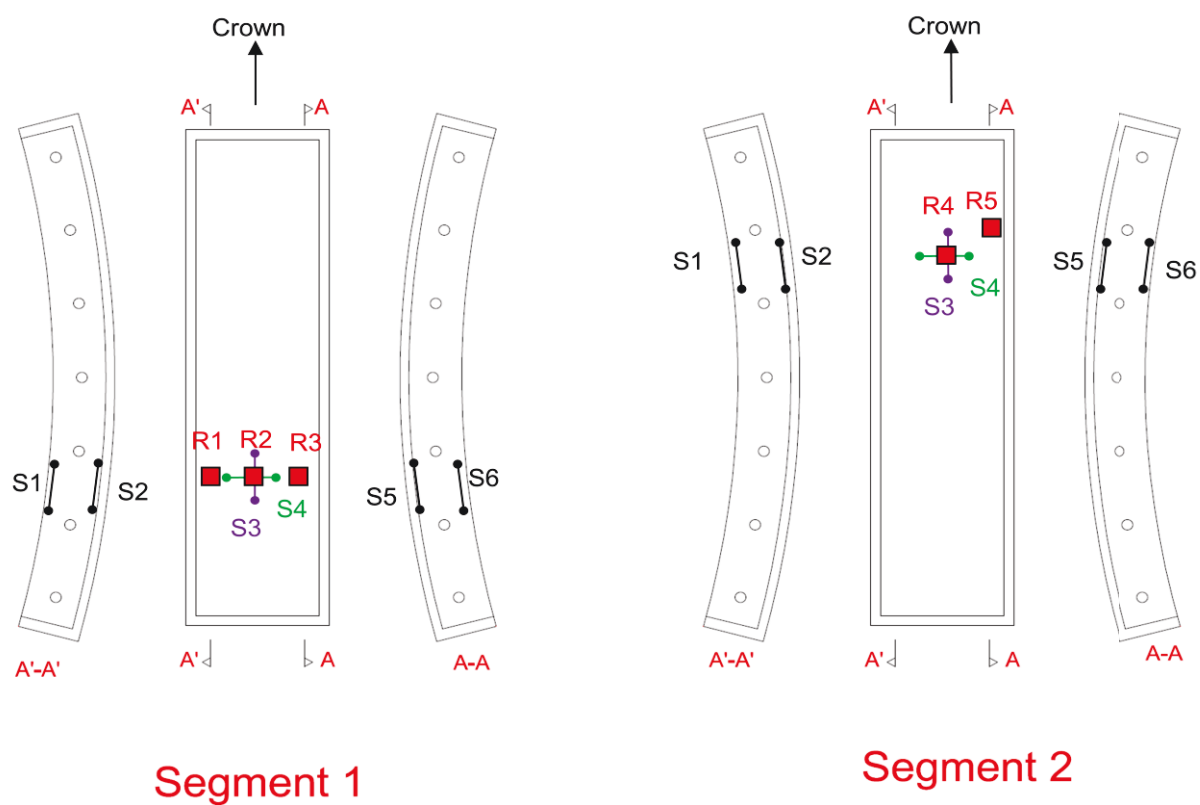


Figure 3-9. Approximate locations of strain gauges. (S=DEMEC span. R=strain gauge rosette).



Figure 3-10. Strain gauges on segment 1. Without protective coating (left) and with protective coating (right).



Figure 3-11. Strain gauges on segment 2.

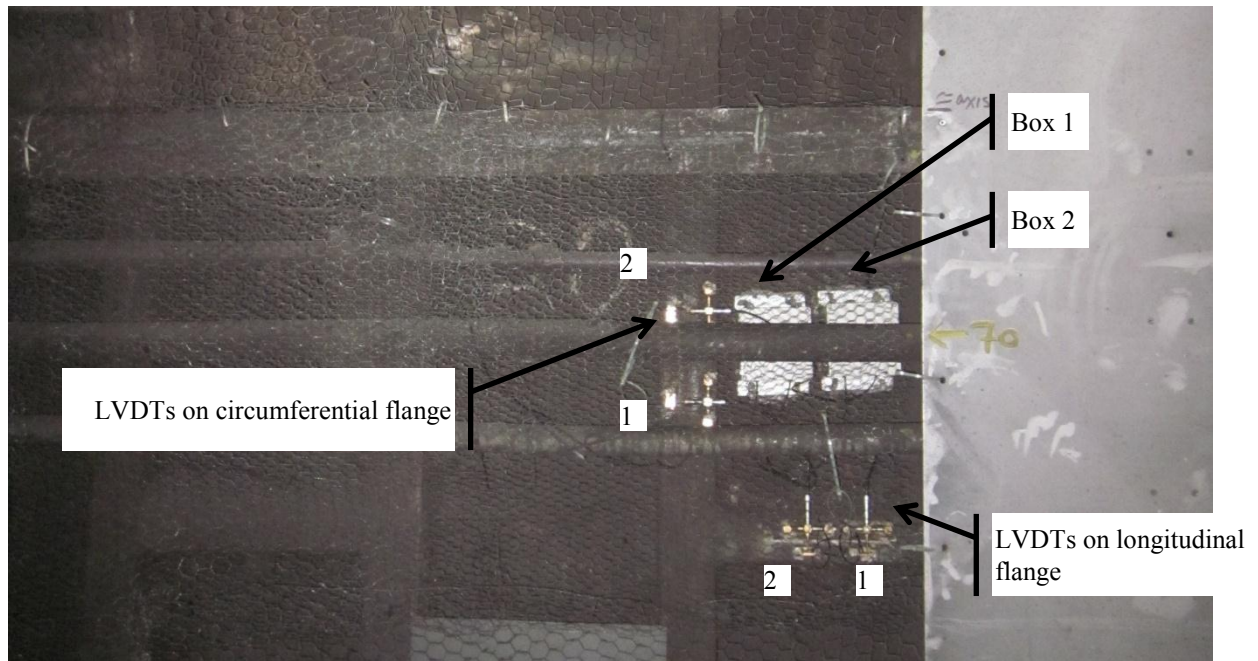


Figure 3-12. As built photo showing locations of LVDTs and boxes housing loggers.



Figure 3-13. Wire bracket to protect strain gauge rosettes.

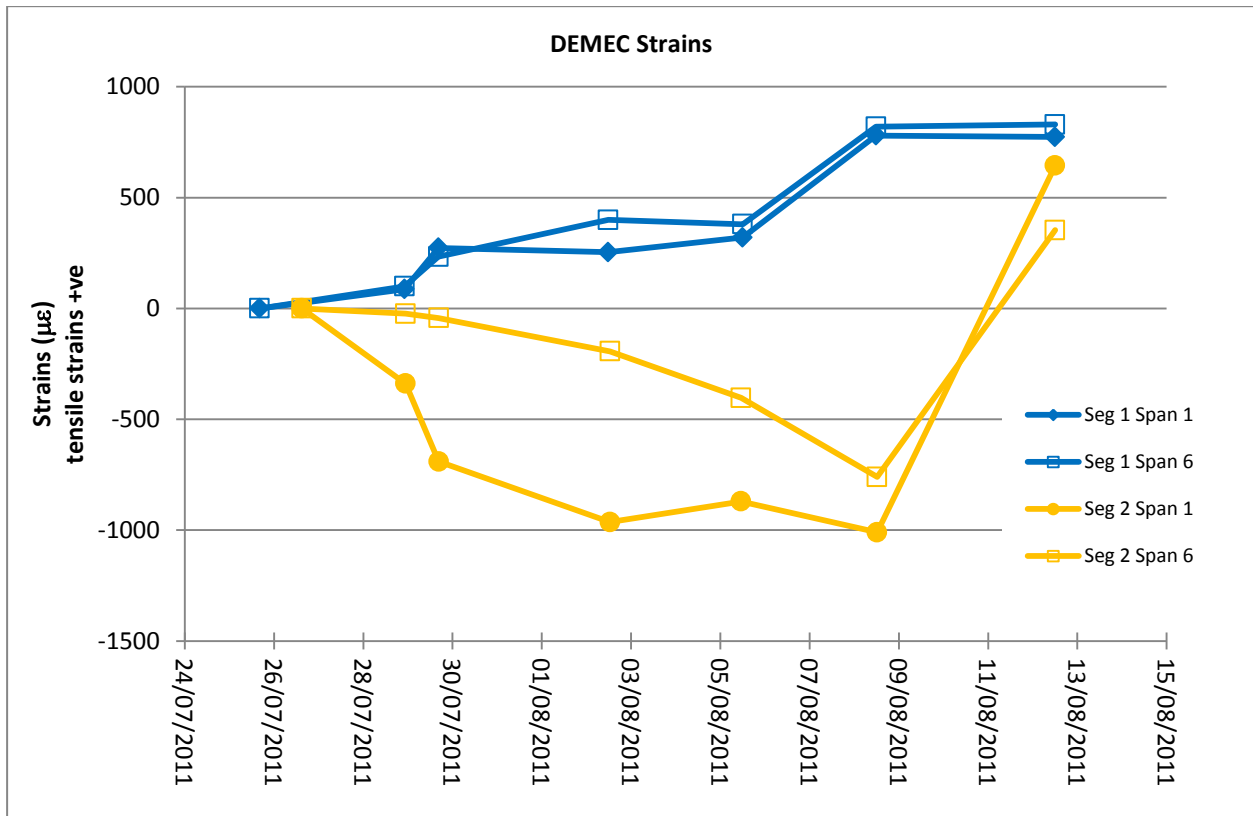


Figure 3-14. Development of strains on the circumferential flange.

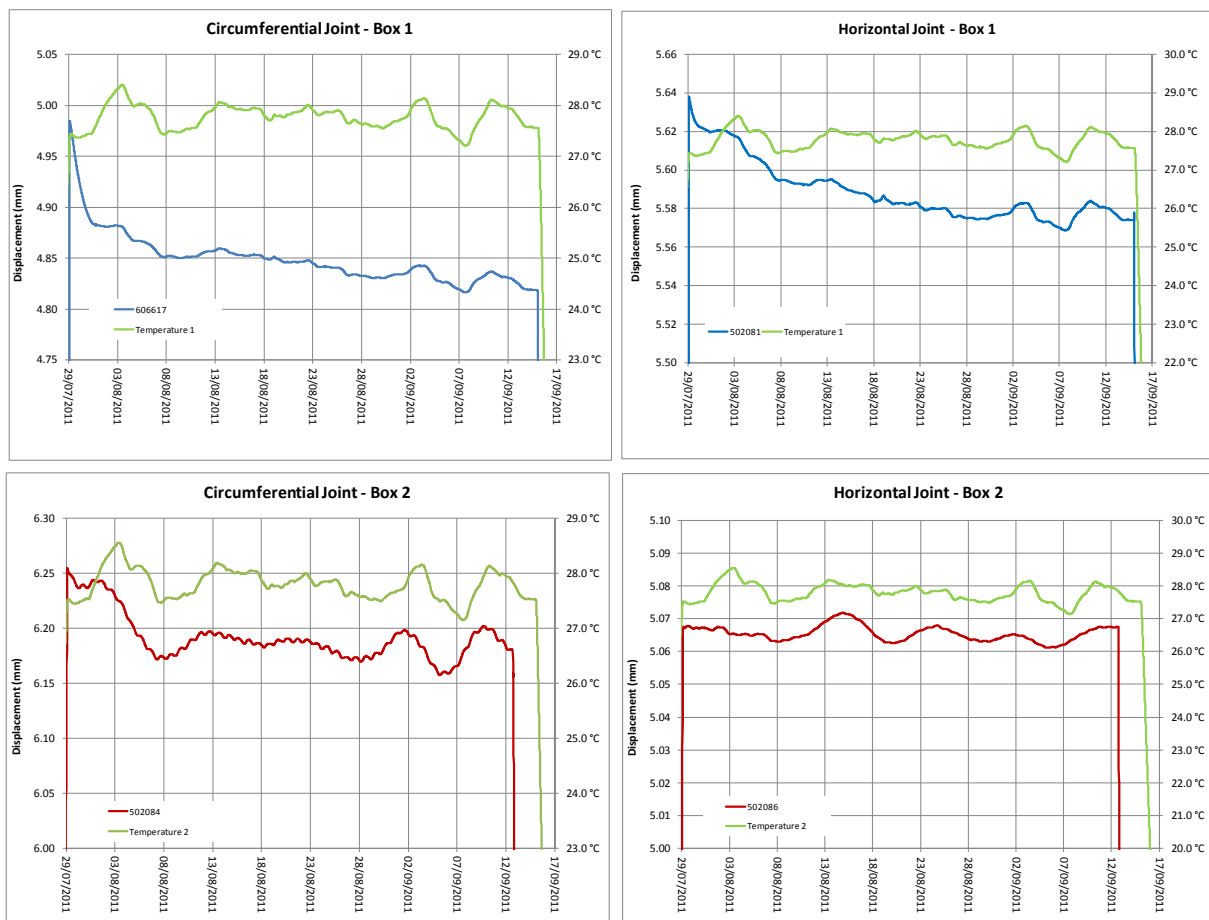


Figure 3-15. Comparison of LVDT trends.

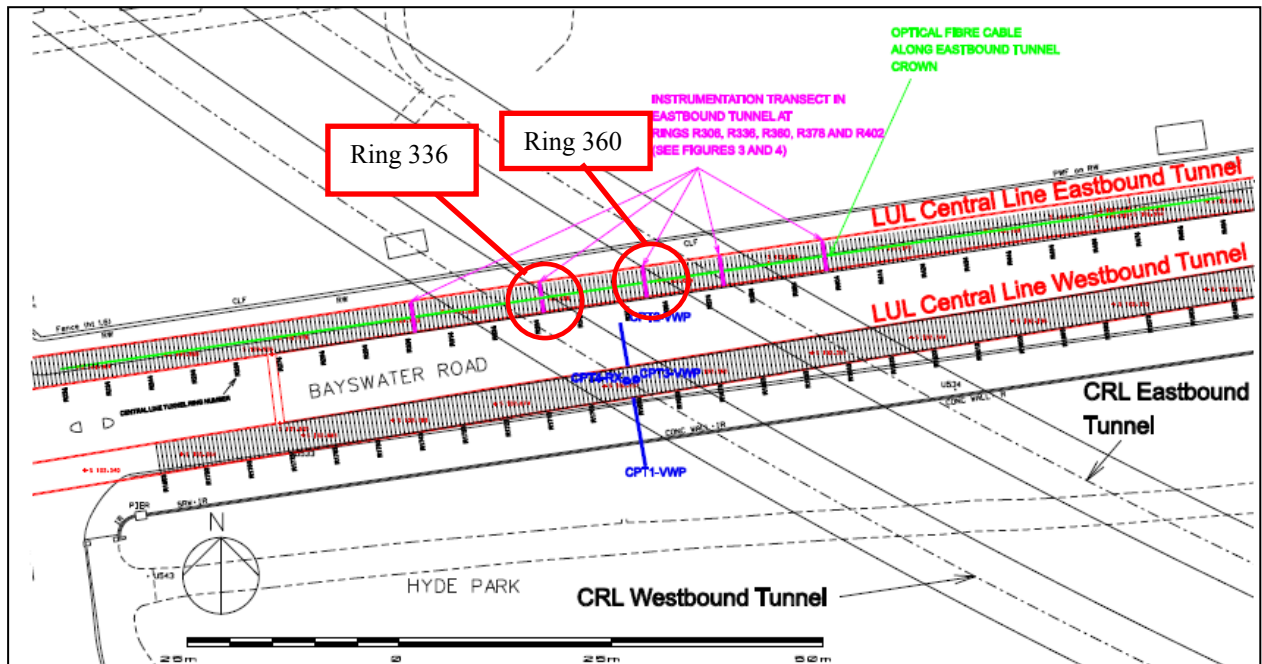


Figure 3-16. Plan view of Crossrail interface with Central Line east of Lancaster Gate Station.

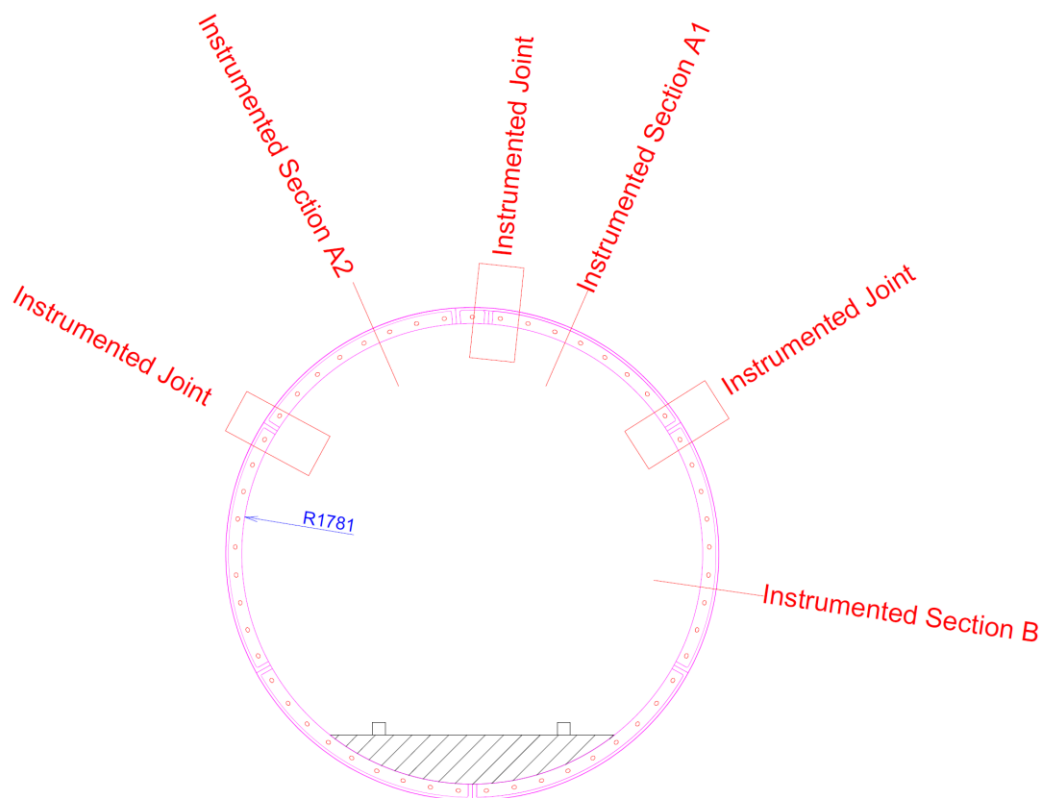


Figure 3-17. Location of instrumented segments and joints on ring 336 and 360.

Figure 3-18. Location of strain gauges and displacement transducers on Ring 336 and Ring 360.

Segment A2

Ring 336

Distance from Intrados:

Gauge 1 = 13 mm

Gauge 2 = 73 mm

Gauge 4 = 73 mm

Gauge 5 = 12 mm

Ring 360

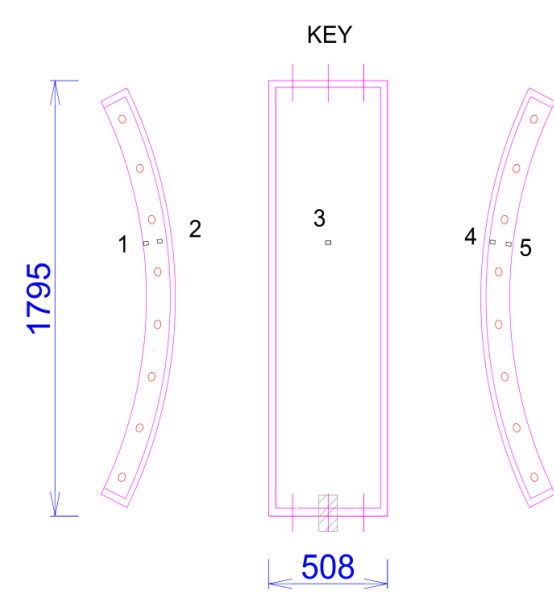
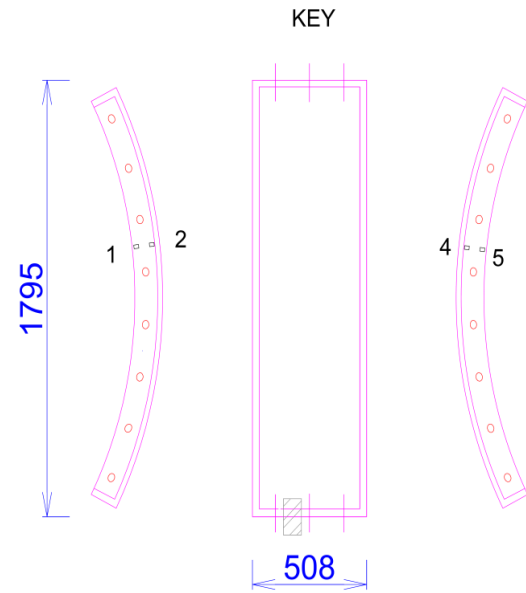
Distance from Intrados:

Gauge 1 = 12 mm

Gauge 2 = 72 mm

Gauge 4 = 76 mm

Gauge 5 = 11 mm



Segment A1

Ring 336

Distance from Intrados:

Gauge 1 = 14 mm

Gauge 2 = 73 mm

Gauge 4 = 75 mm

Gauge 5 = 15 mm

Ring 360

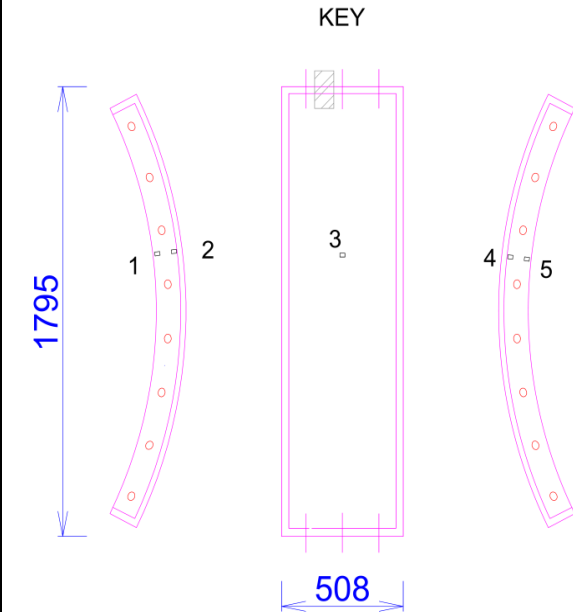
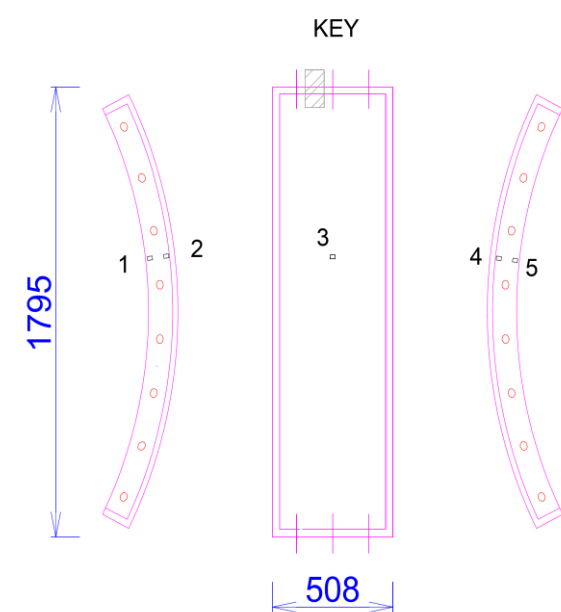
Distance from Intrados:

Gauge 1 = 15 mm

Gauge 2 = 75 mm

Gauge 4 = 73 mm

Gauge 5 = 14 mm



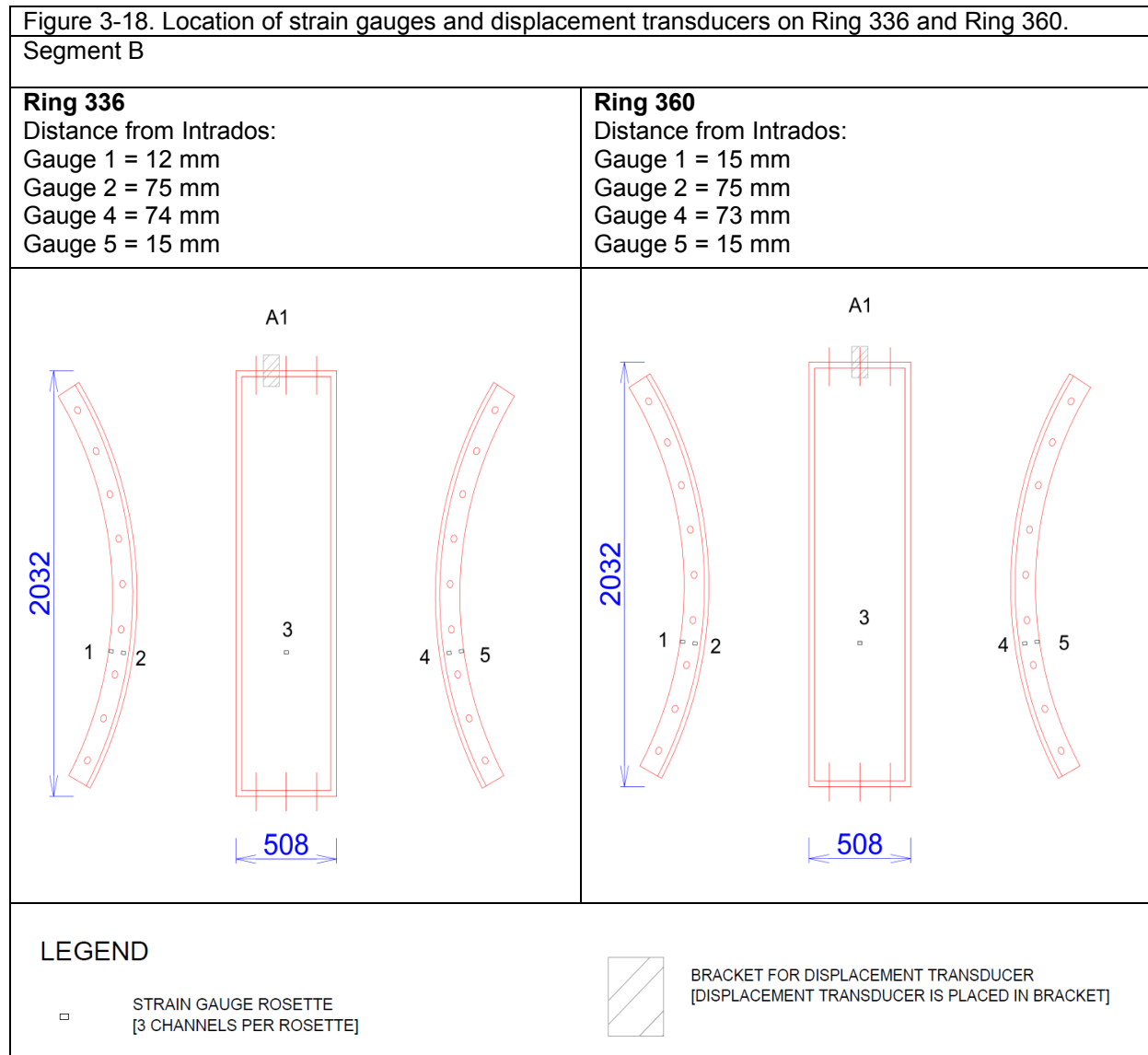


Figure 3-18. Location of strain gauges and displacement transducers on Ring 336 and Ring 360.

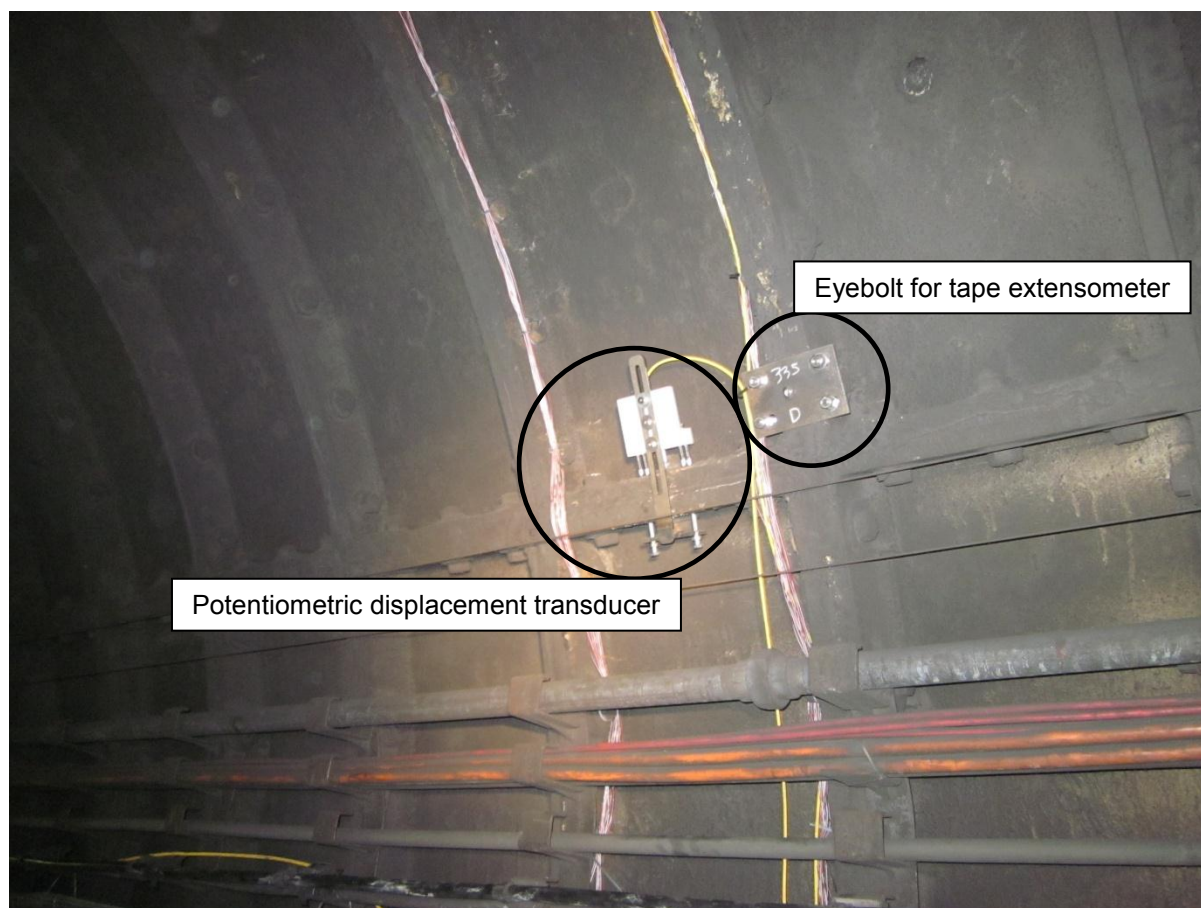


Figure 3-19. Photo of potentiometric displacement transducer installation by CMCS and tape extensometer eyebolt installation by Imperial College in Central Line.

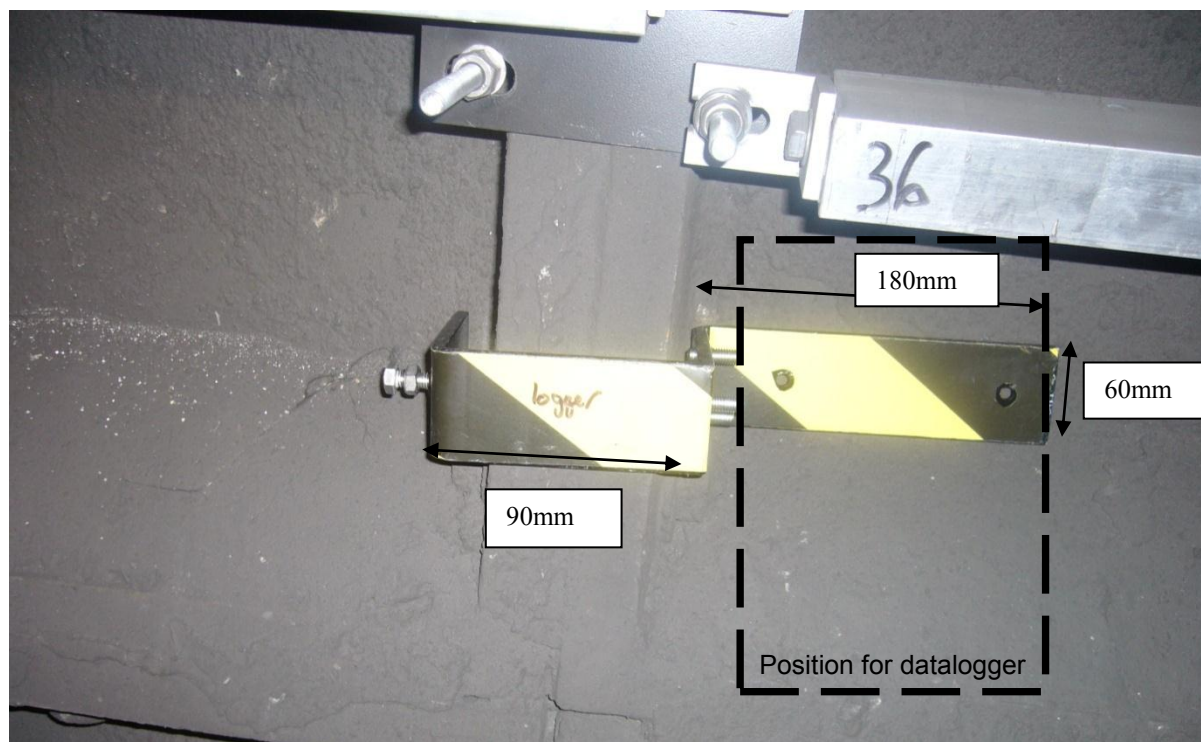


Figure 3-20. Bracket for securing dataloggers to tunnel segments.



Figure 3-21. Additional datalogging unit on the eastbound platform headwall.

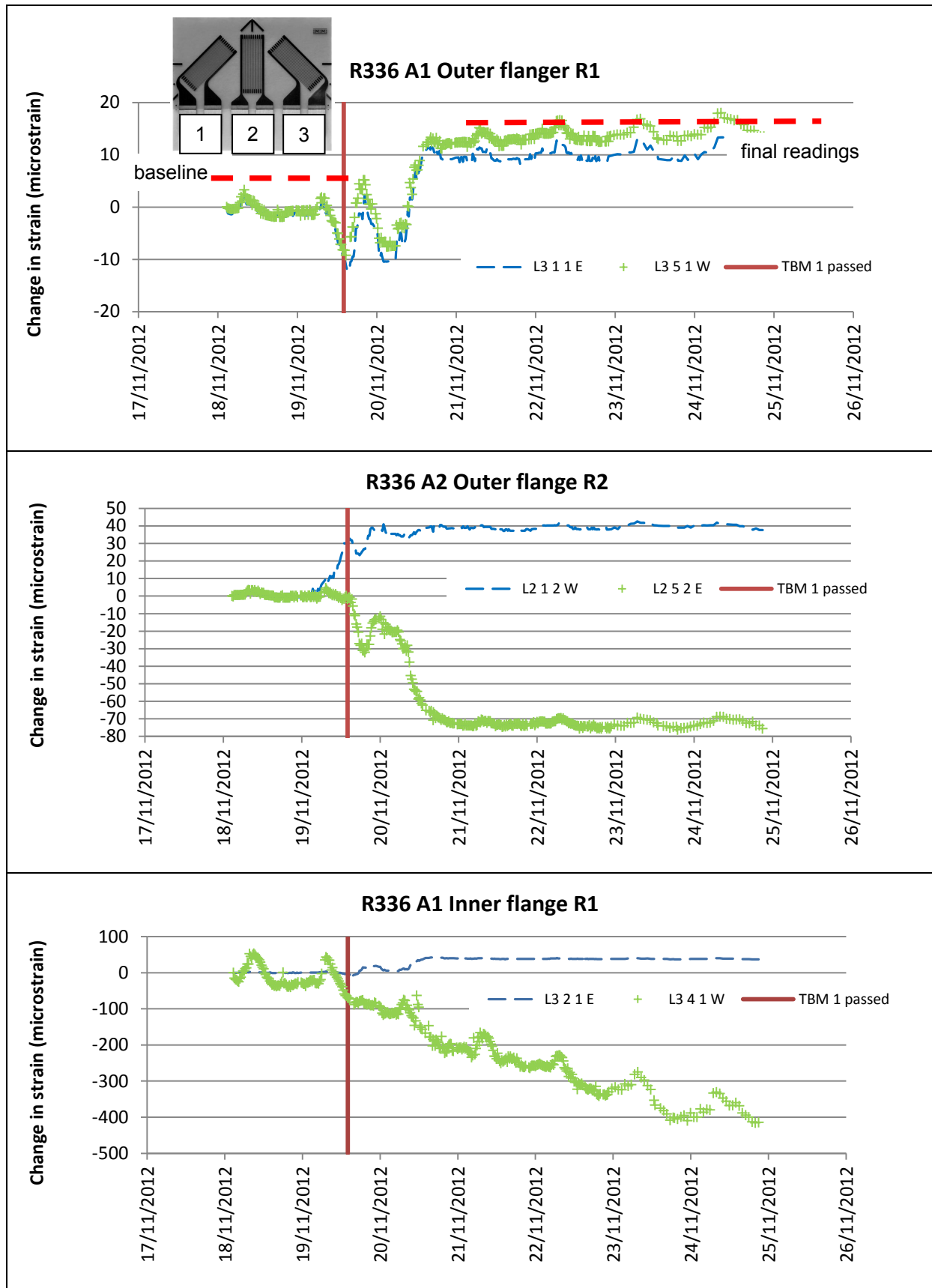


Figure 3-22. Typical strain gauge response for TBM1 passage showing segment bending (top), segment twisting (middle), and erratic gauge readings (bottom).

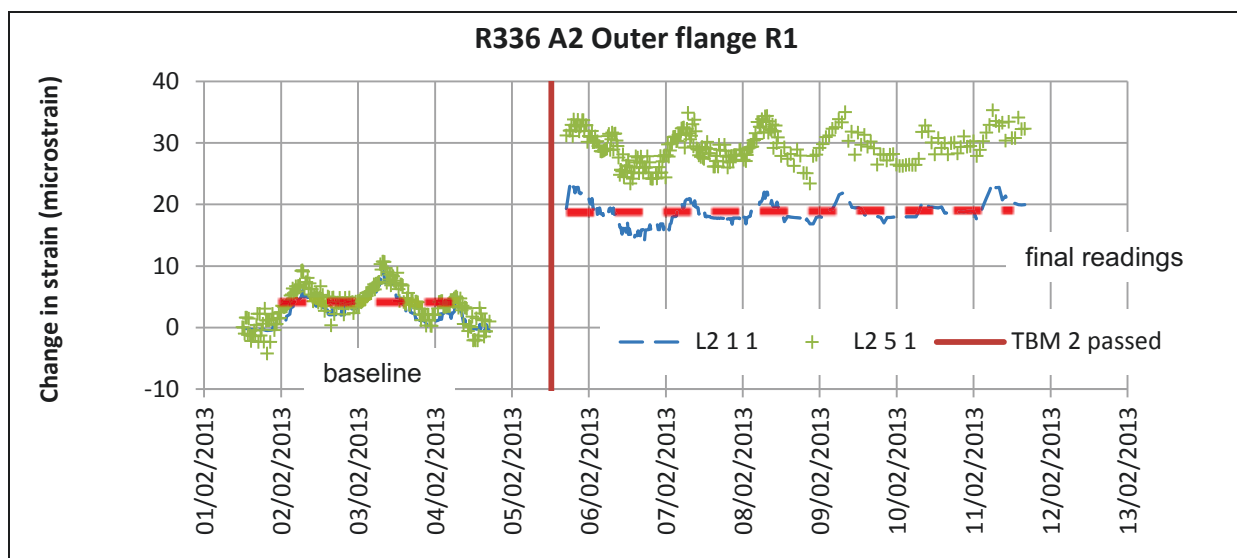


Figure 3-23. Typical strain gauge responses for TBM2 passage.

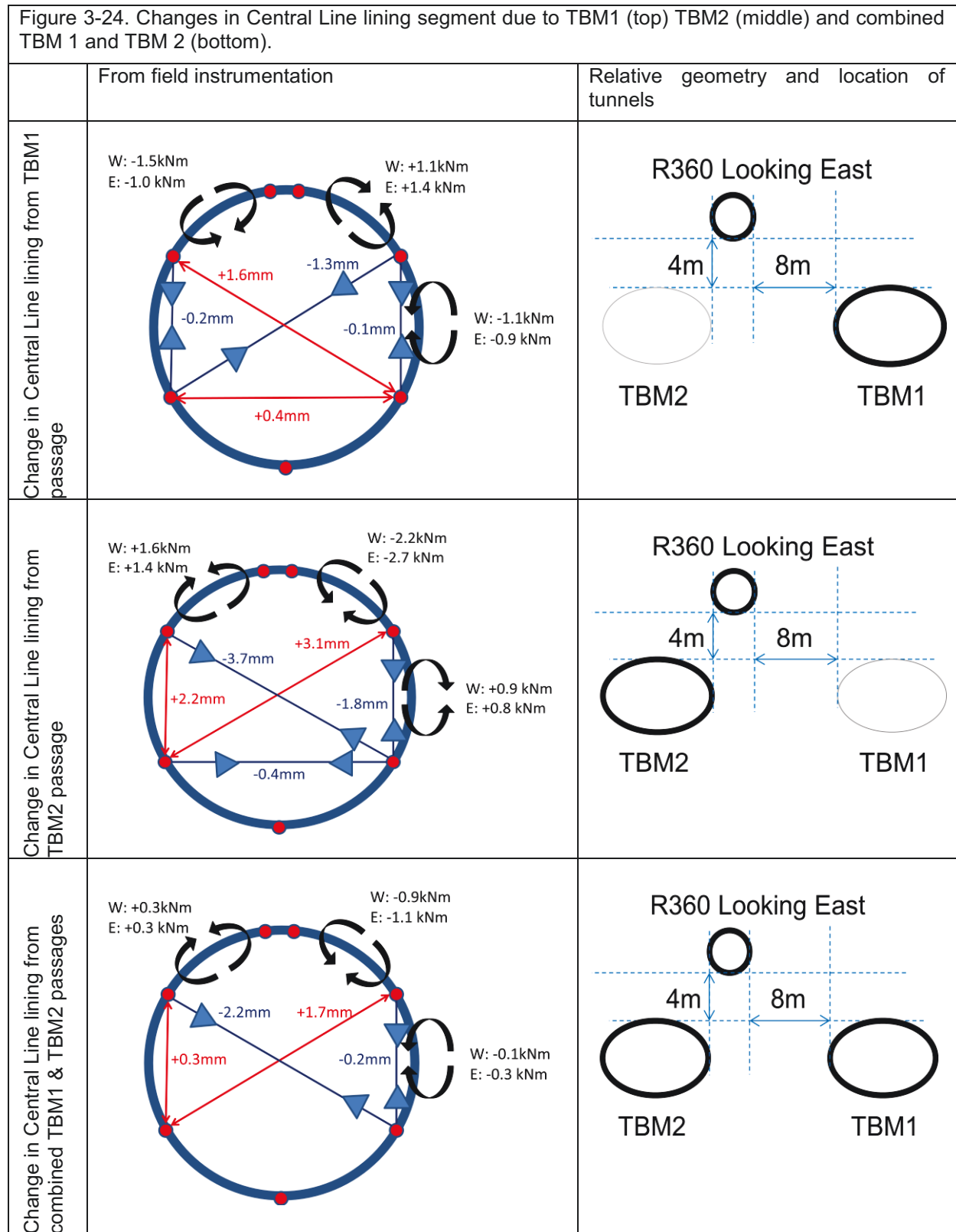


Figure 3-24. Changes in Central Line lining segment due to TBM1 (top) TBM2 (middle) and combined TBM 1 and TBM 2 (bottom).

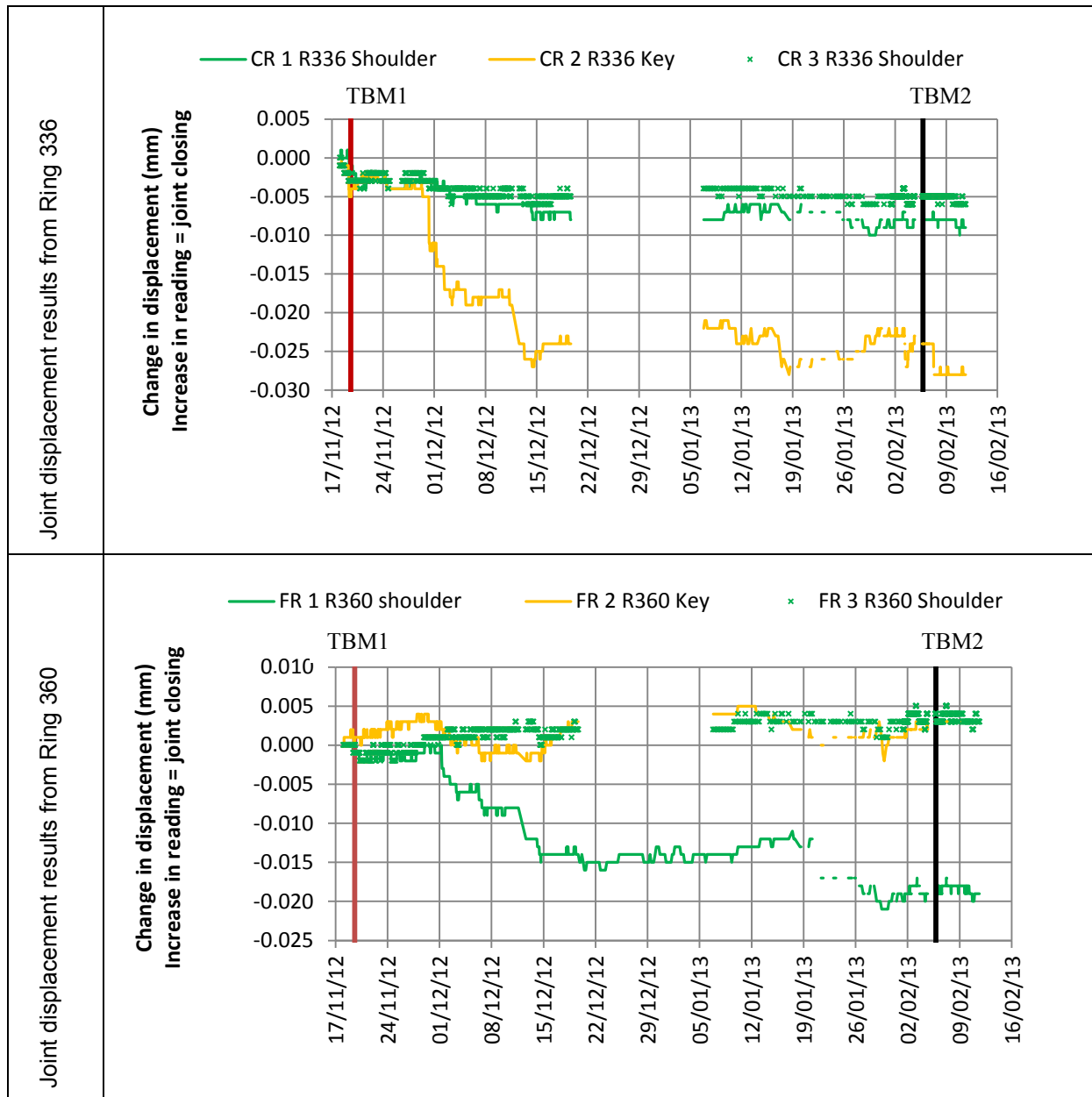


Figure 3-25. Results from potentiometer displacement transducers at the joints for Ring 336 (top) and Ring 360 (bottom).

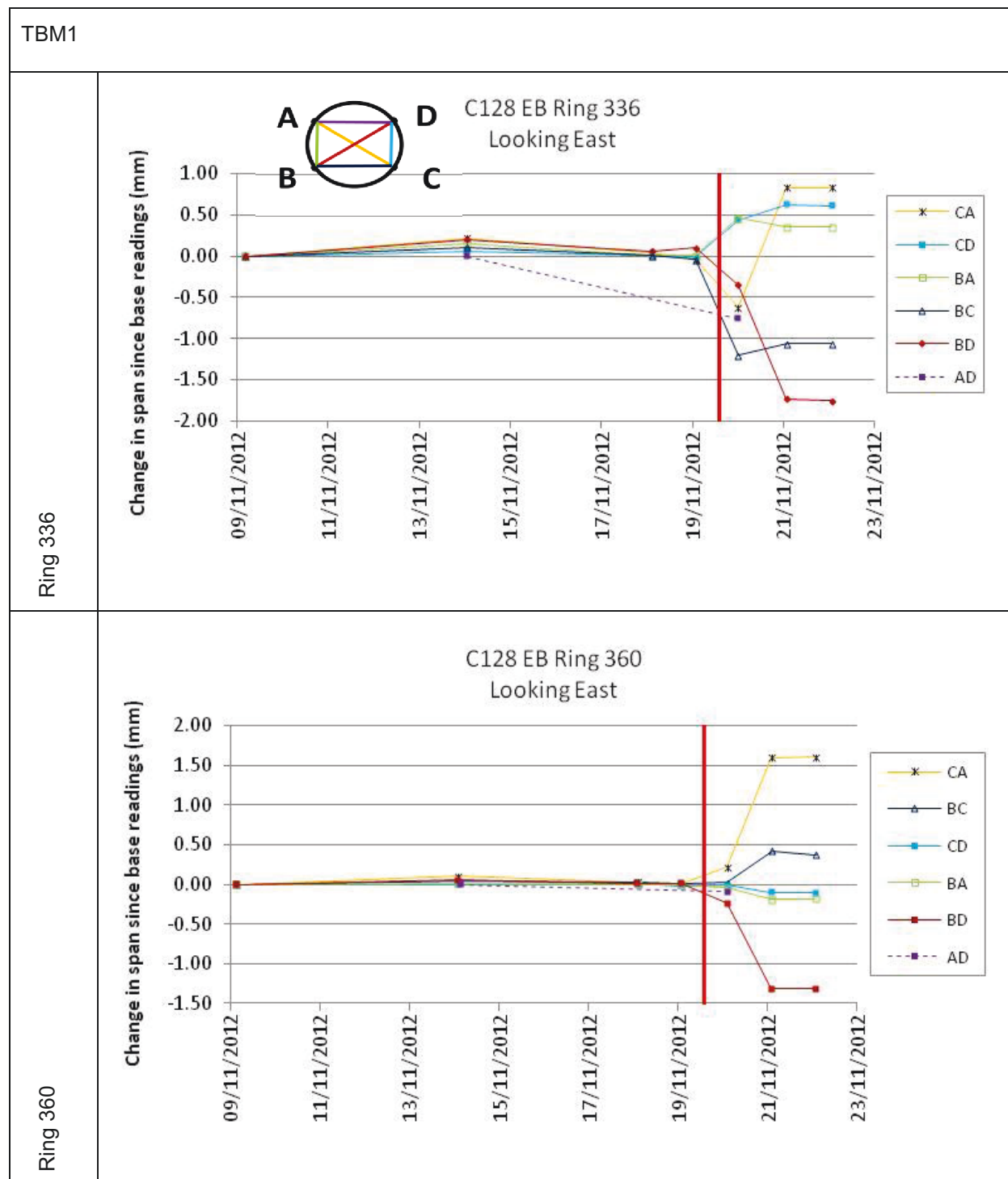


Figure 3-26. Tape extensometer readings covering the passage of TBM1 for Ring 336 (top) and Ring 360 (bottom).

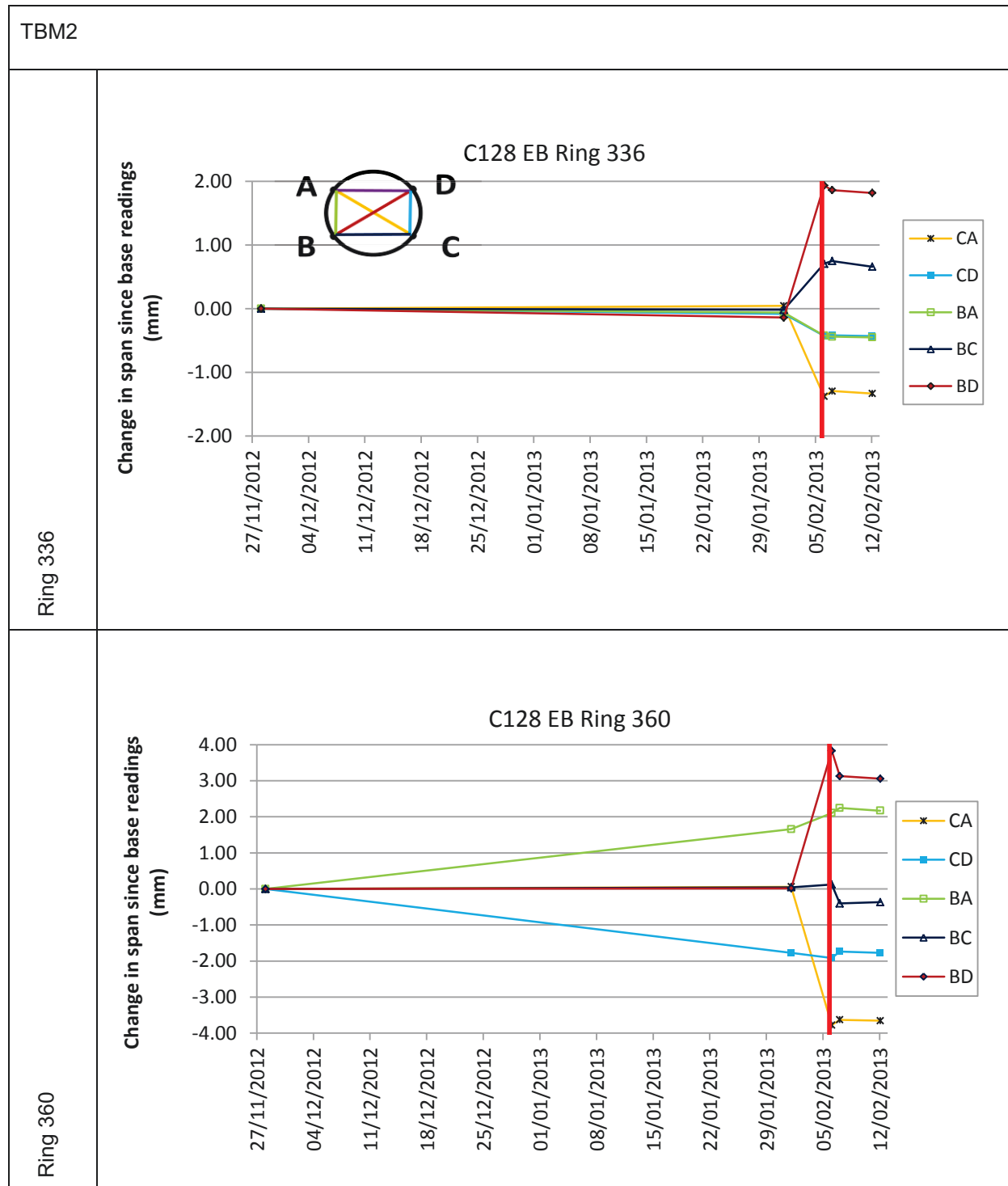


Figure 3-27. Tape extensometer readings covering the passage of TBM2 for Ring 336 (top) and Ring 360 (bottom).



Figure 3-28. Fibre optics fixed to the key pieces along the crown of Central Line.

4 Material composition and testing



Figure 4-1. Bottom pattern for cast iron segment.



Figure 4-2. Pouring sand into bottom mould.



Figure 4-3. Bottom part of sand mould.



Figure 4-4. Assembling the top and bottom halves of the sand mould for a segment.



Figure 4-5. Melting the charge mix.



Figure 4-6. Pouring the charge mix into the ladle.



Figure 4-7. Pouring the charge mix into the sand mould.



Figure 4-8. Half-scale cast iron segment prior to machining.

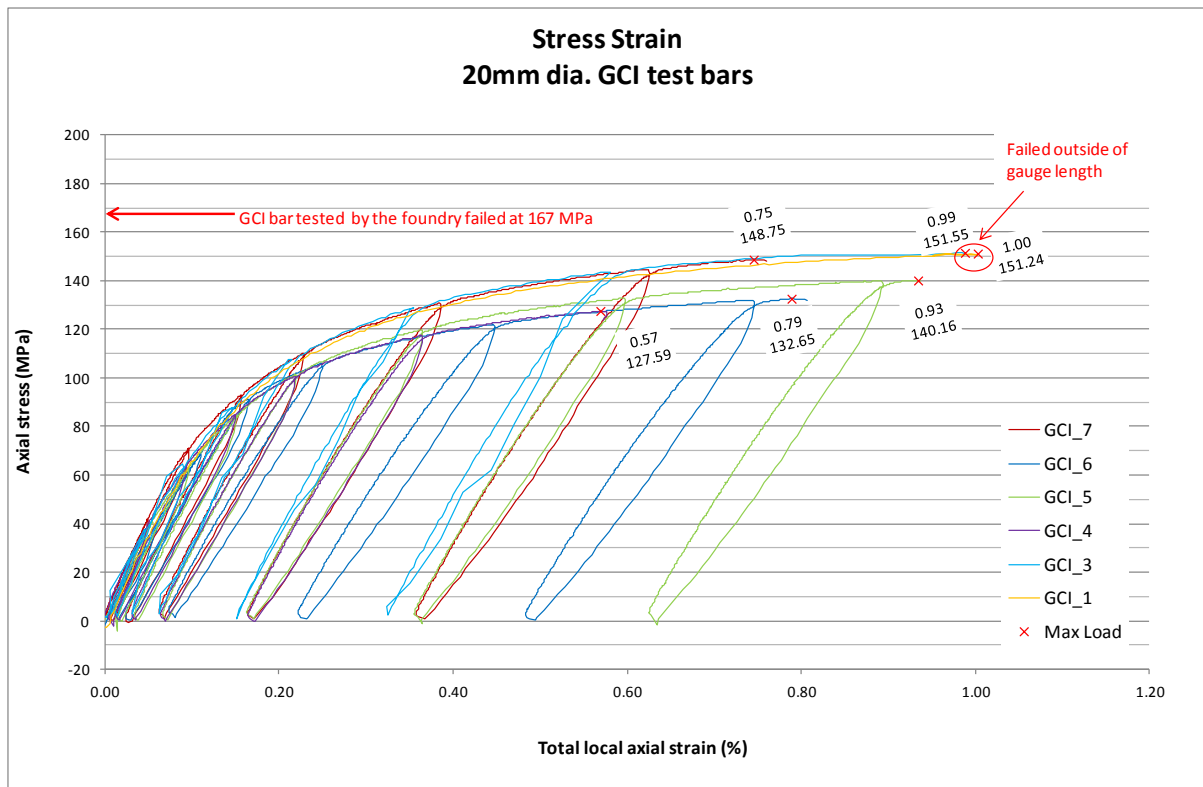


Figure 4-9. Stress-strain curves for 20mm diameter GCI test specimens.

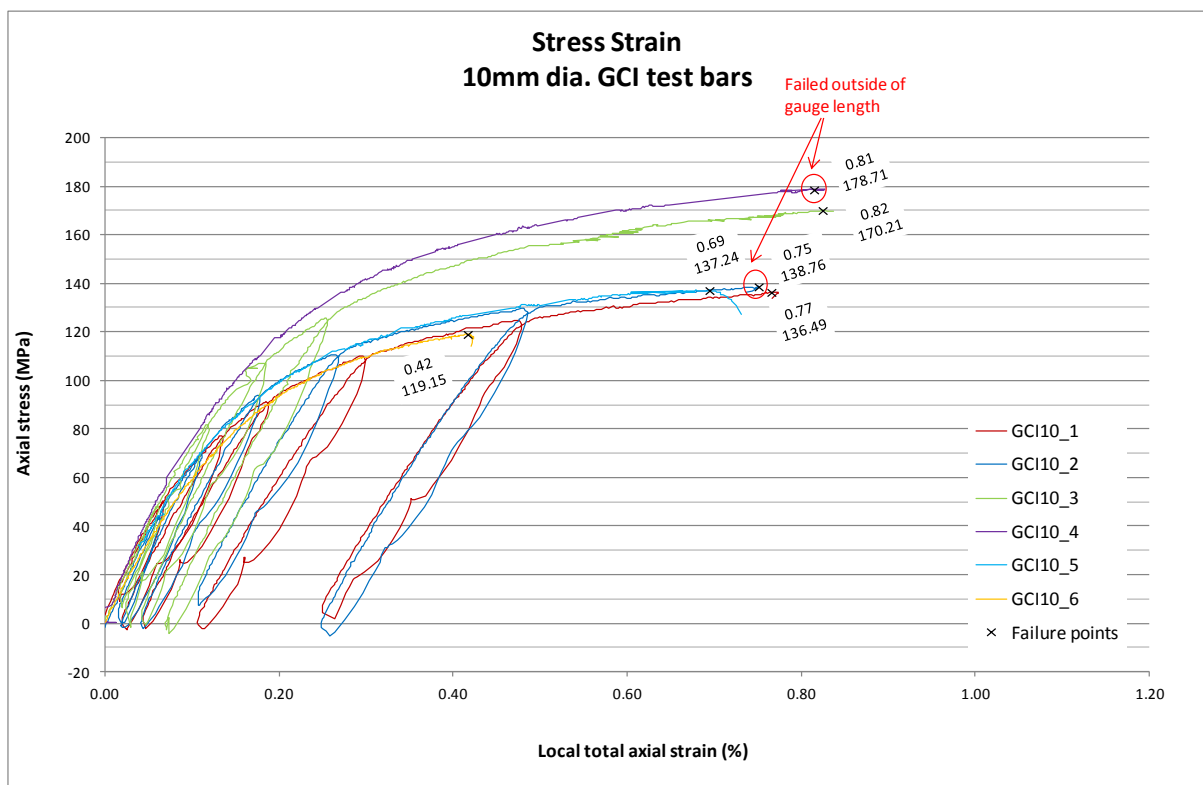


Figure 4-10. Stress-strain curves for 10mm diameter GCI test specimens.

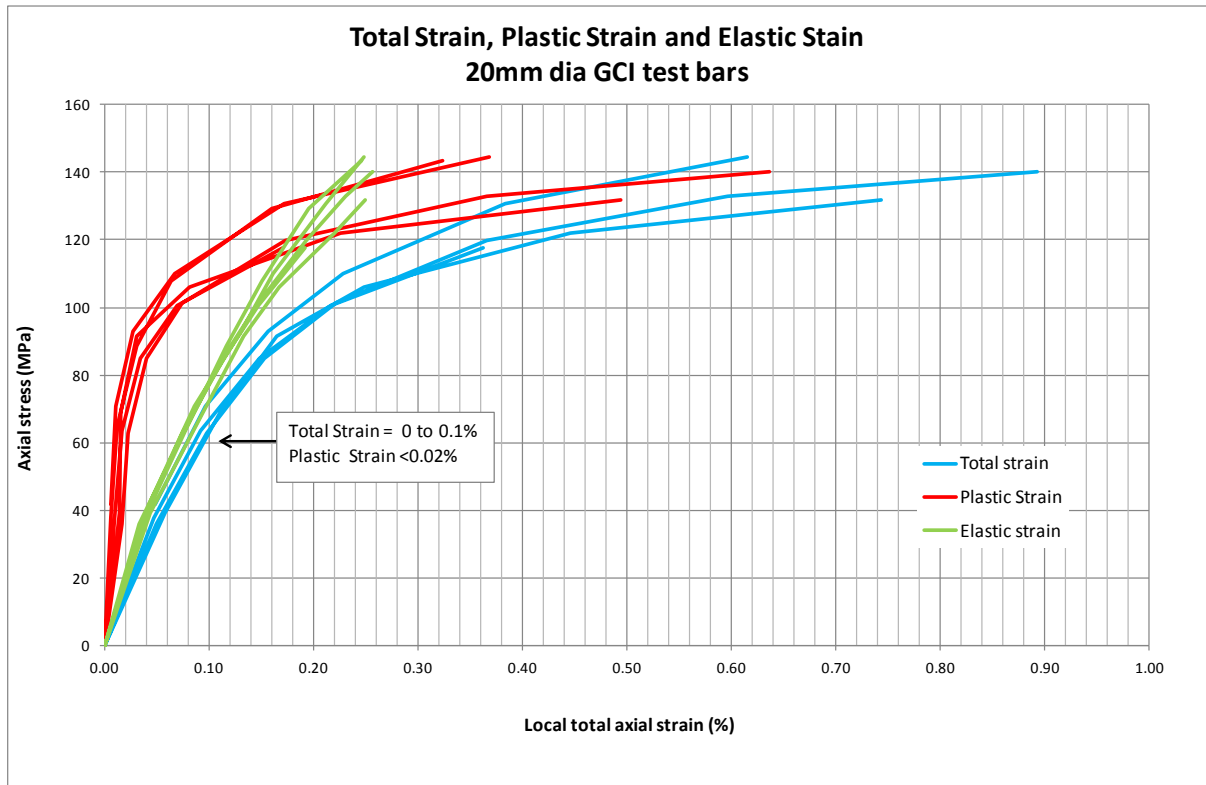


Figure 4-11. Total strain, plastic strain and elastic strain.

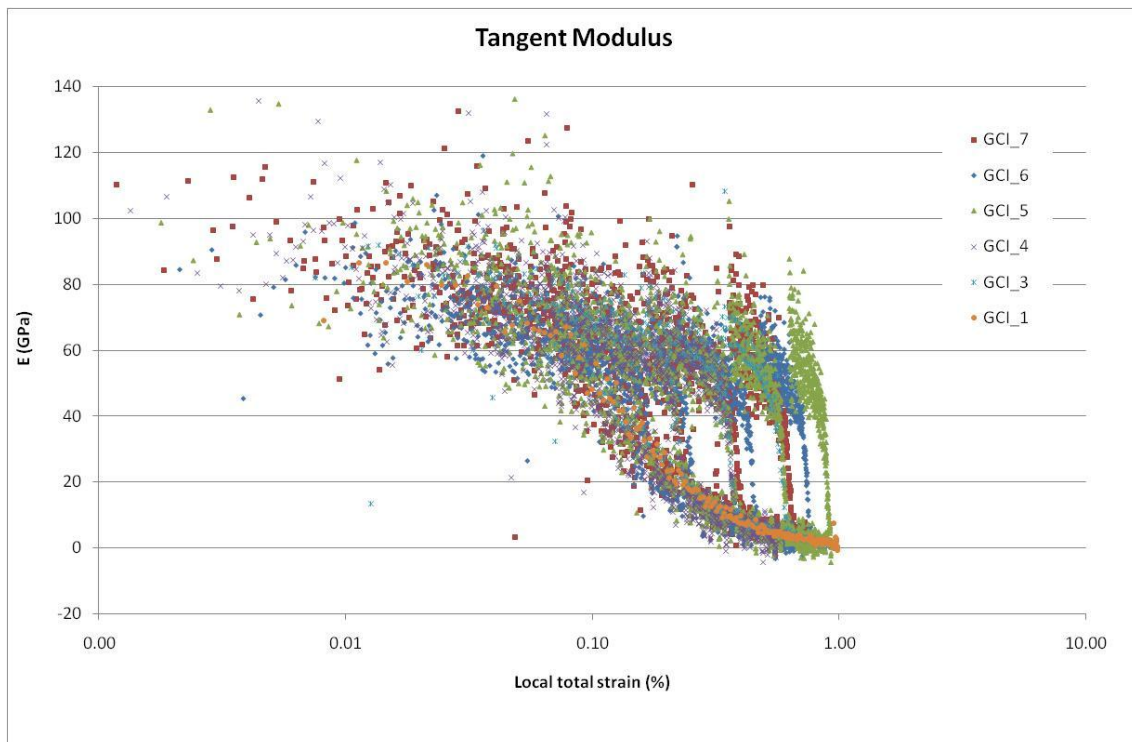


Figure 4-12. Tangent modulus for 20mm diameter GCI specimens.

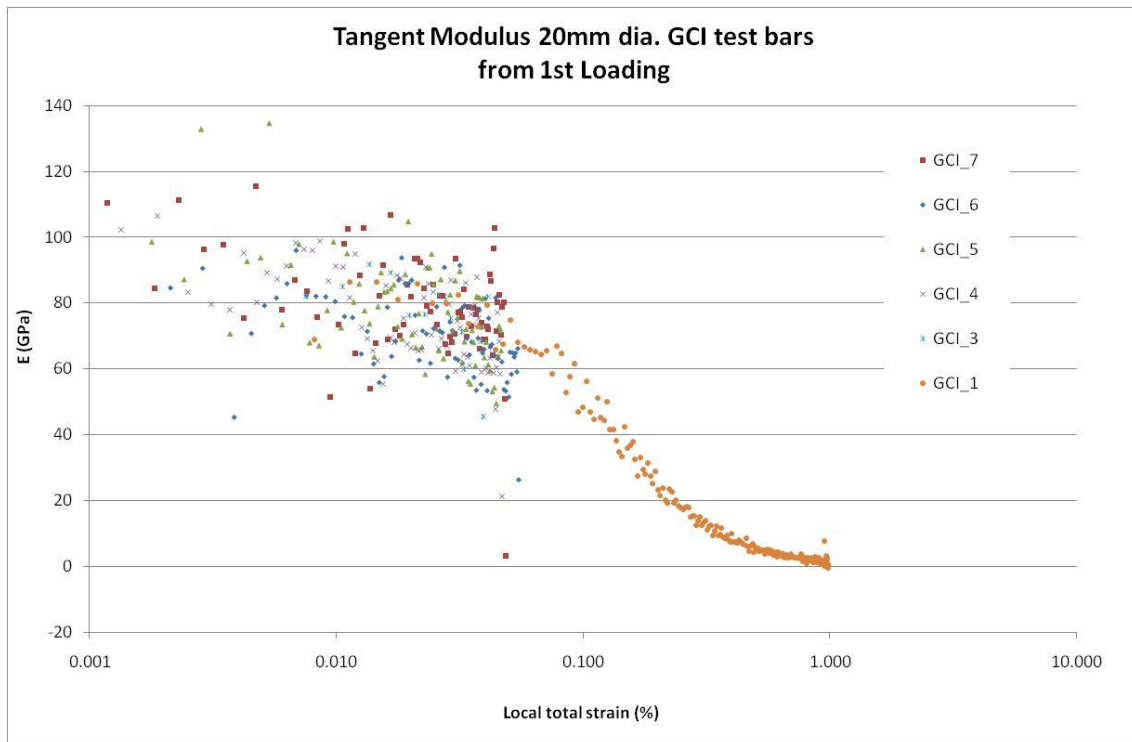


Figure 4-13. Tangent modulus from 1st loading curve.

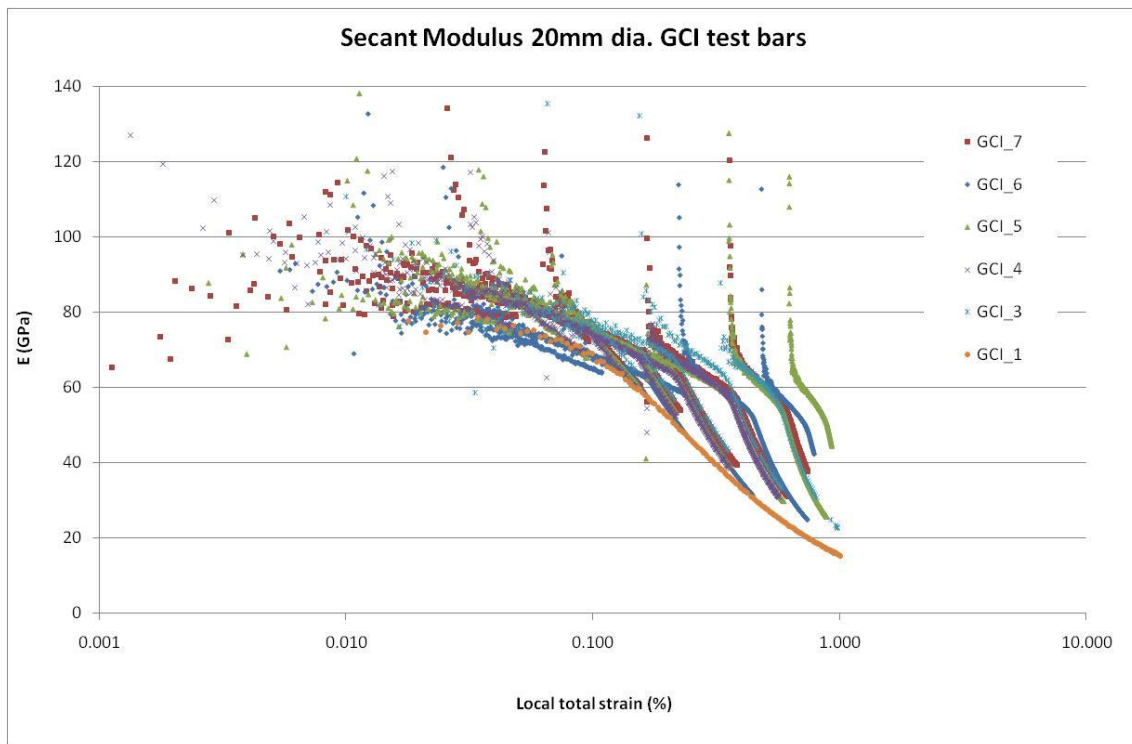


Figure 4-14. Secant modulus for 20mm diameter GCI specimens.

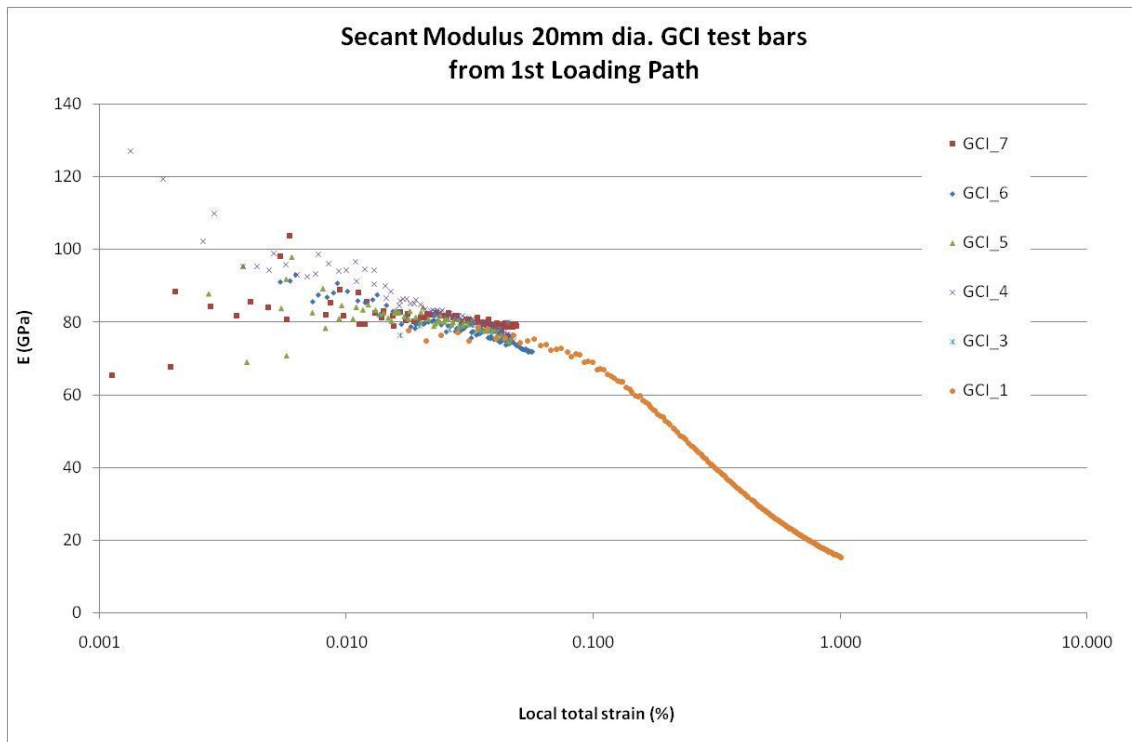


Figure 4-15. Secant modulus from 1st loading curve.

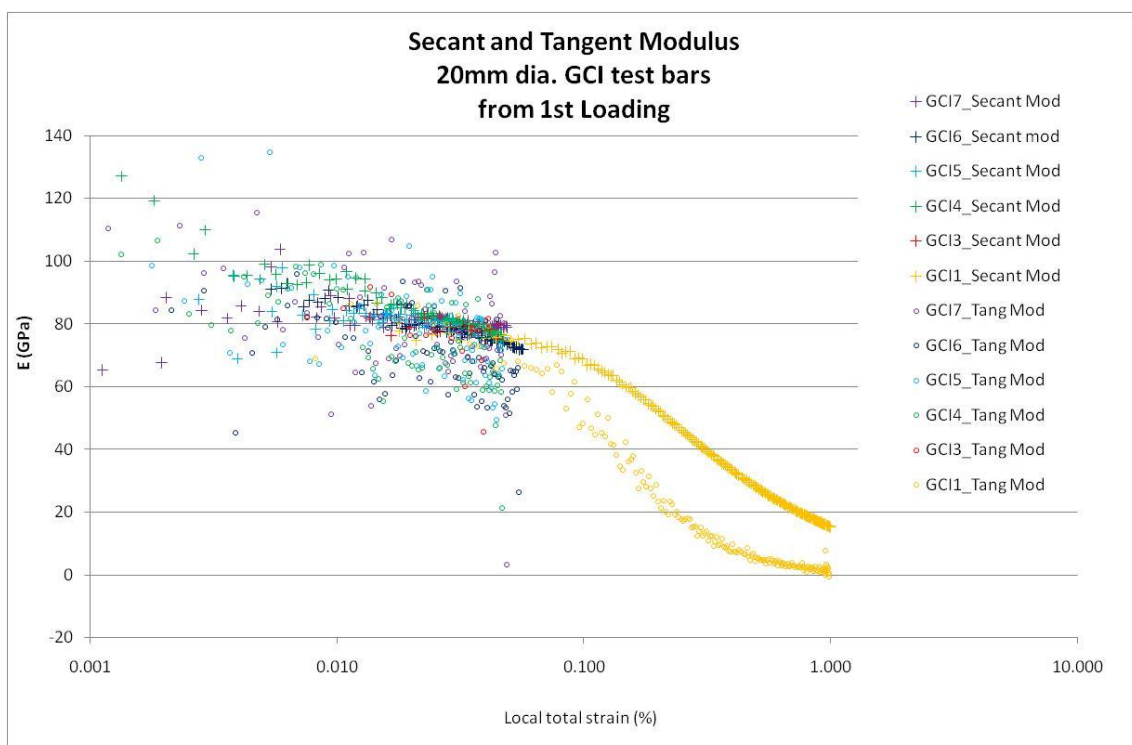


Figure 4-16. Secant and tangent moduli of 20mm dia. GCI specimens from 1st loading curve.

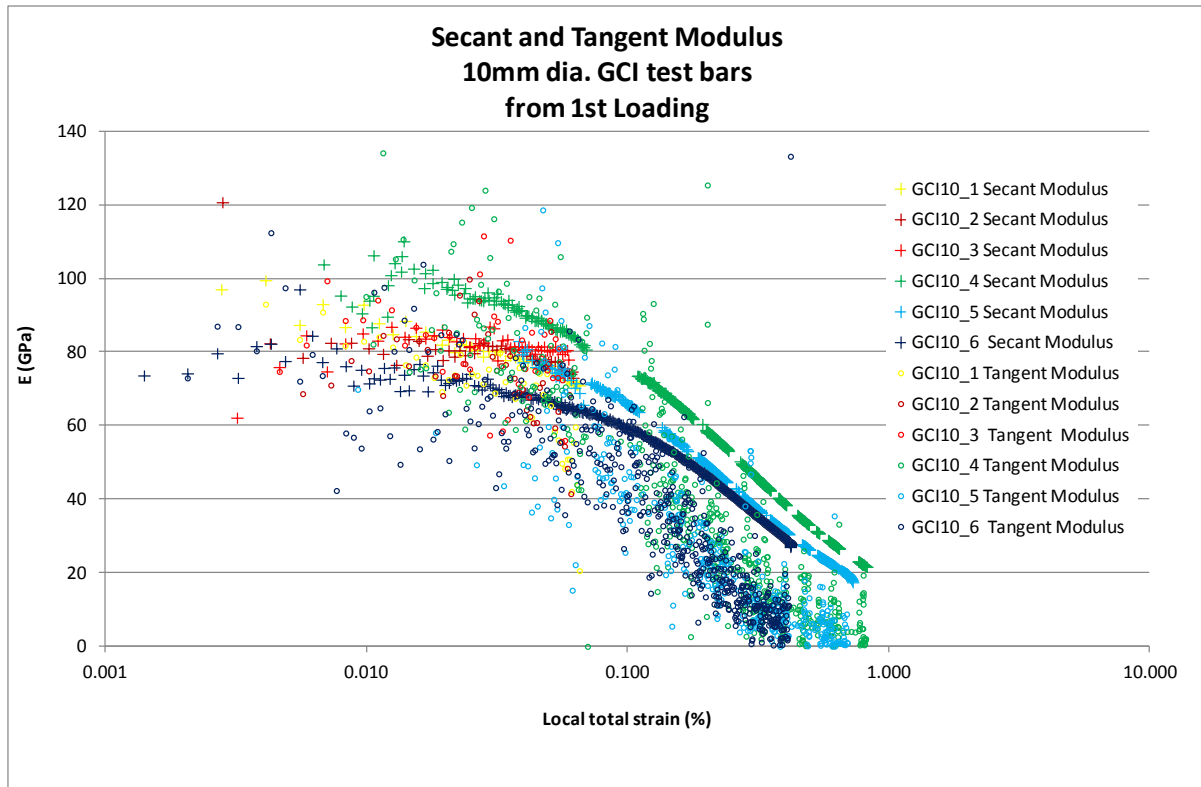
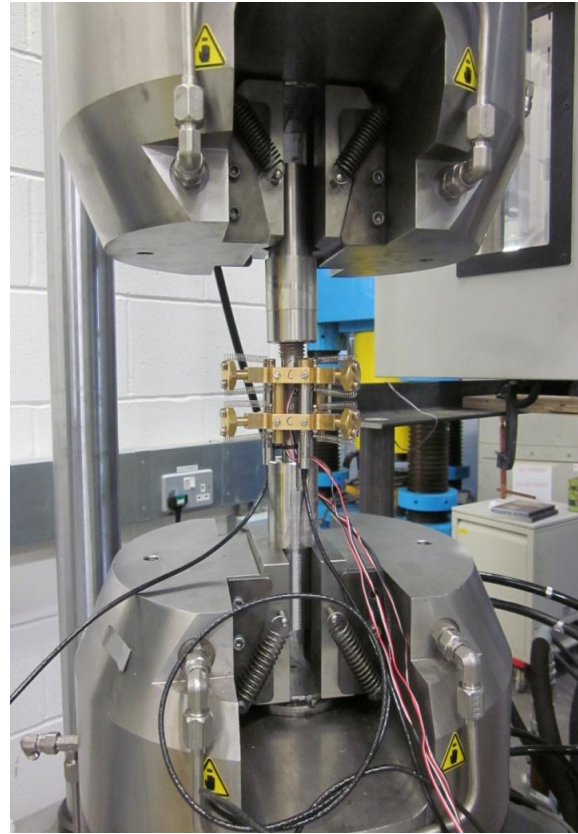


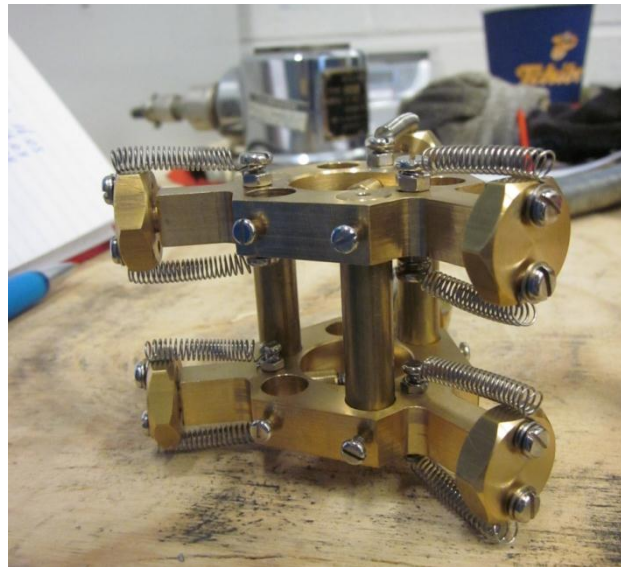
Figure 4-17. Secant and tangent moduli of 10mm dia. GCI specimens from 1st loading curve.



Post yield strain gauges



In test rig with LVDTs



Close up of spring –loaded LVDT holder

Figure 4-18. First wrought iron specimen tested with strain gauges and LVDT displacement transducers attached.

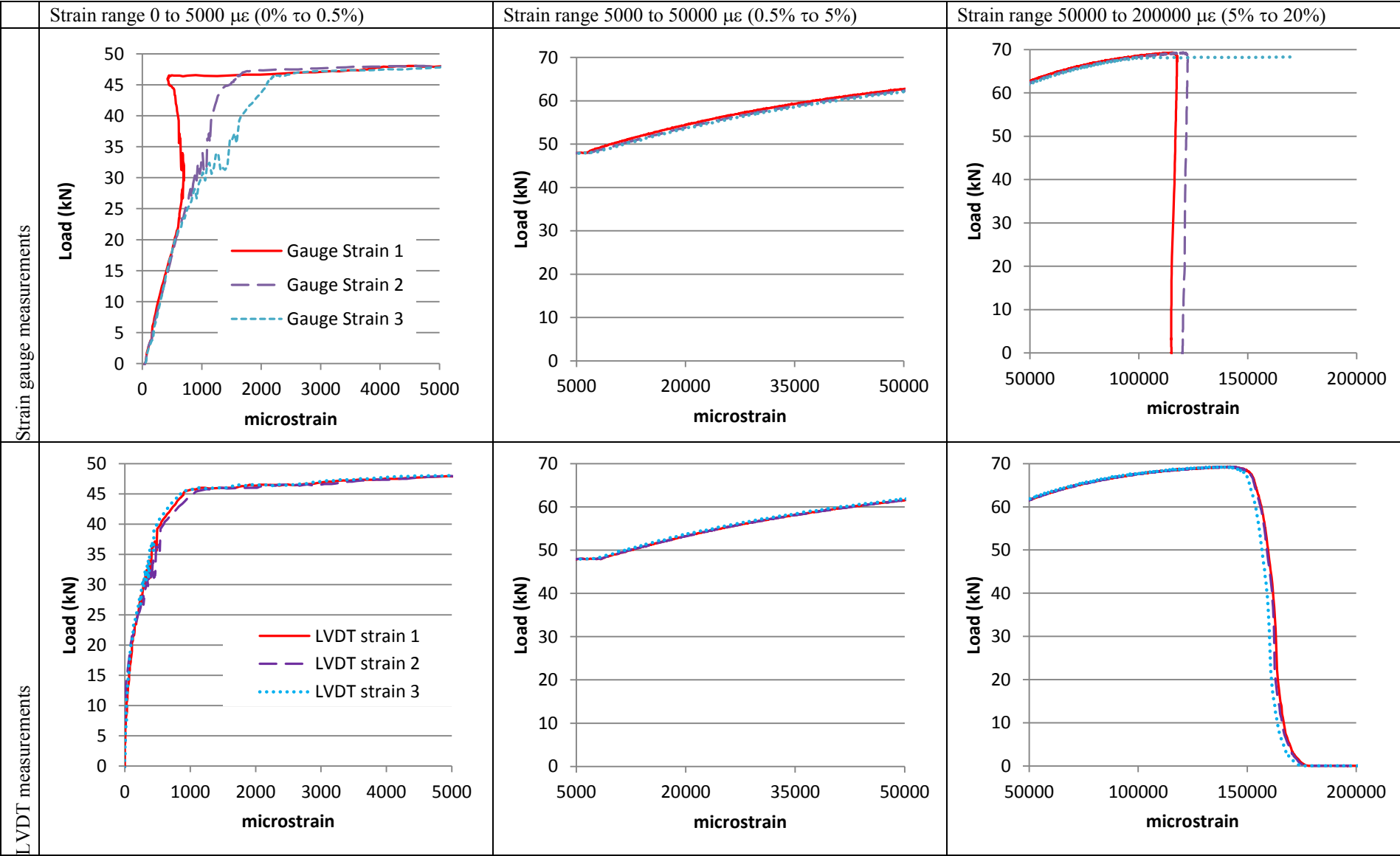


Figure 4-19. Measurements using strain gauges (top) and LVDTs (bottom).

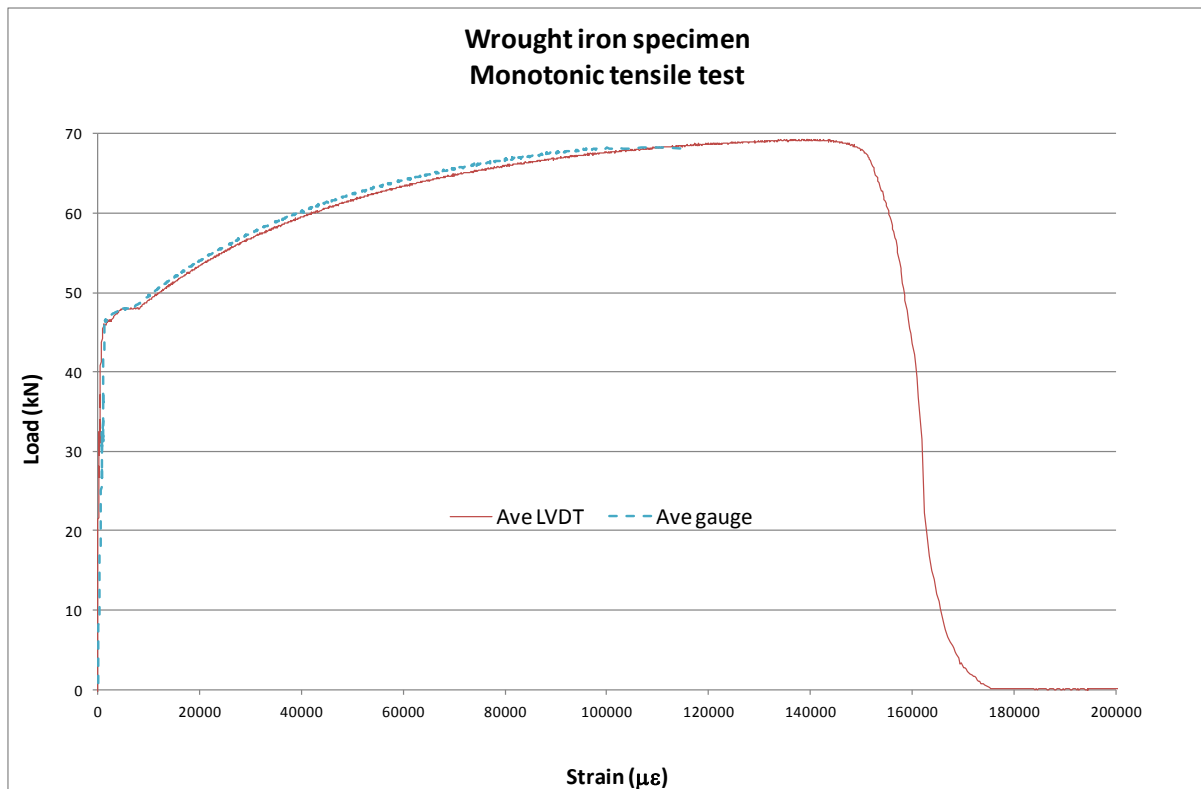


Figure 4-20. Stress-strain curves of wrought iron specimen using average strains.

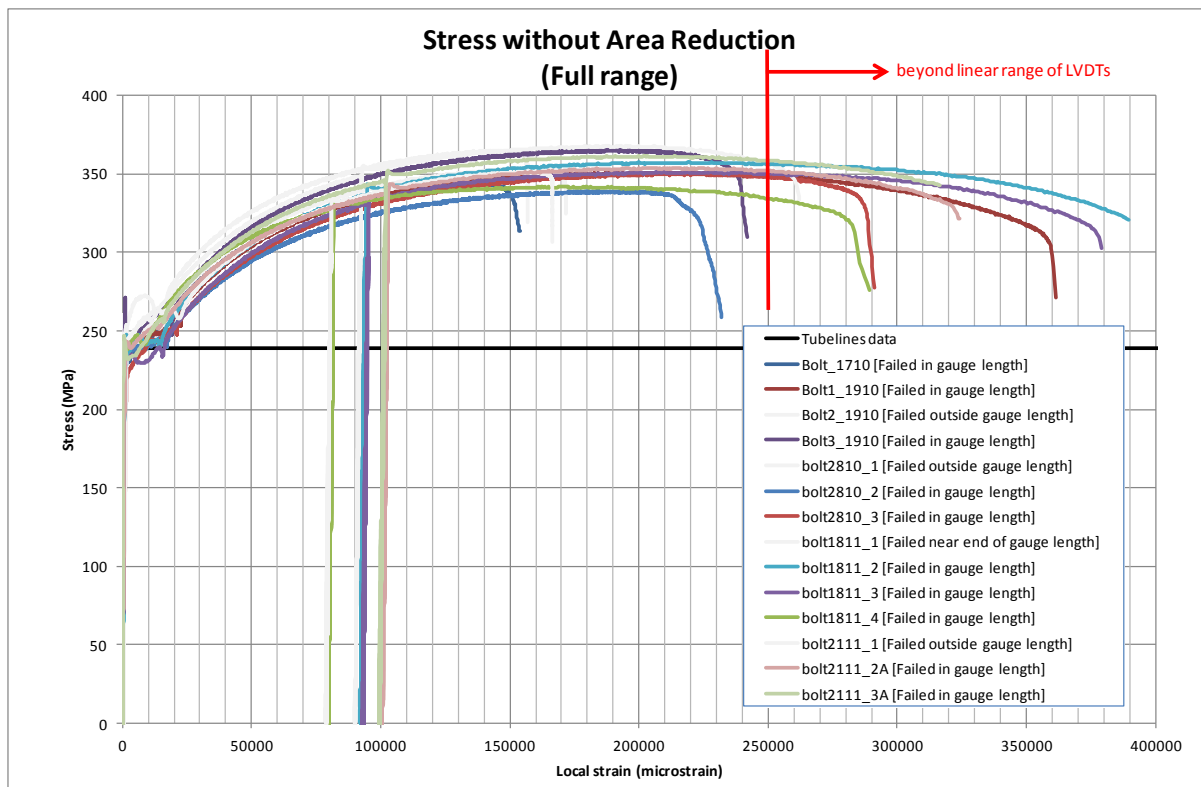


Figure 4-21. Stress-strain curves of all wrought iron specimens.

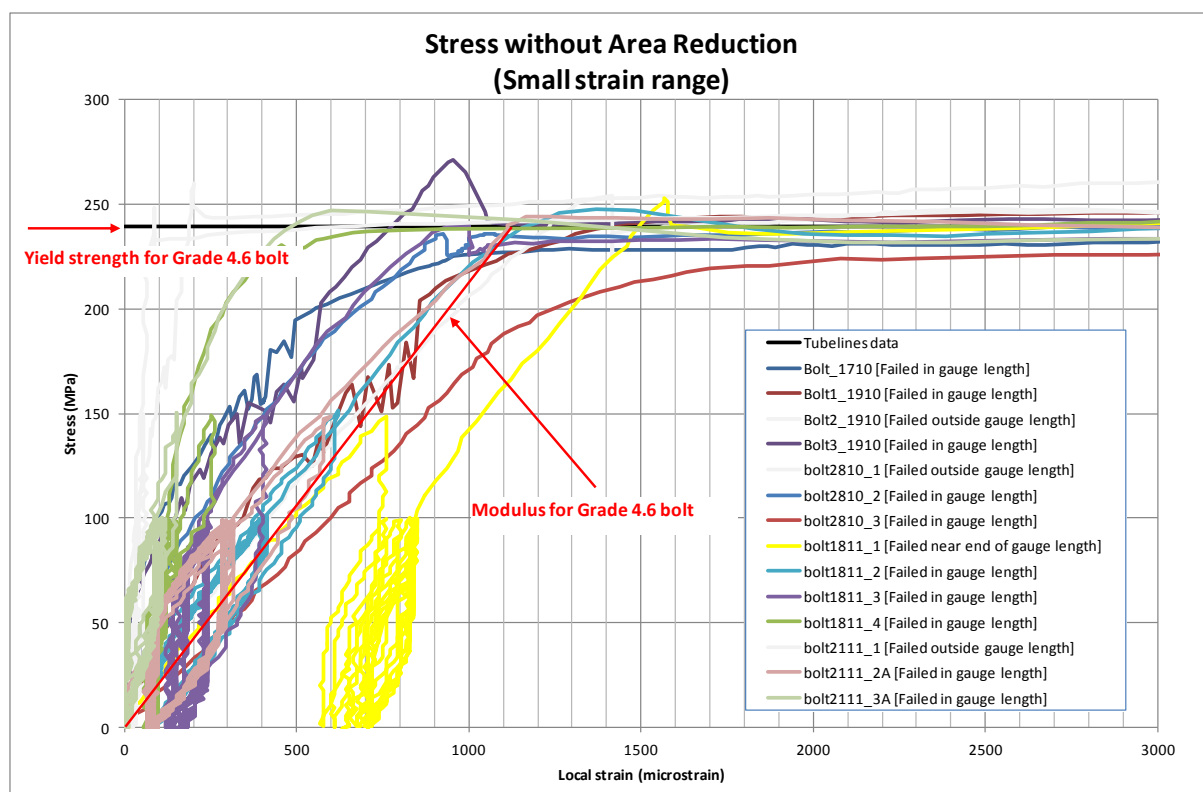


Figure 4-22. Stress-strain curves of wrought iron specimens at small strain range.

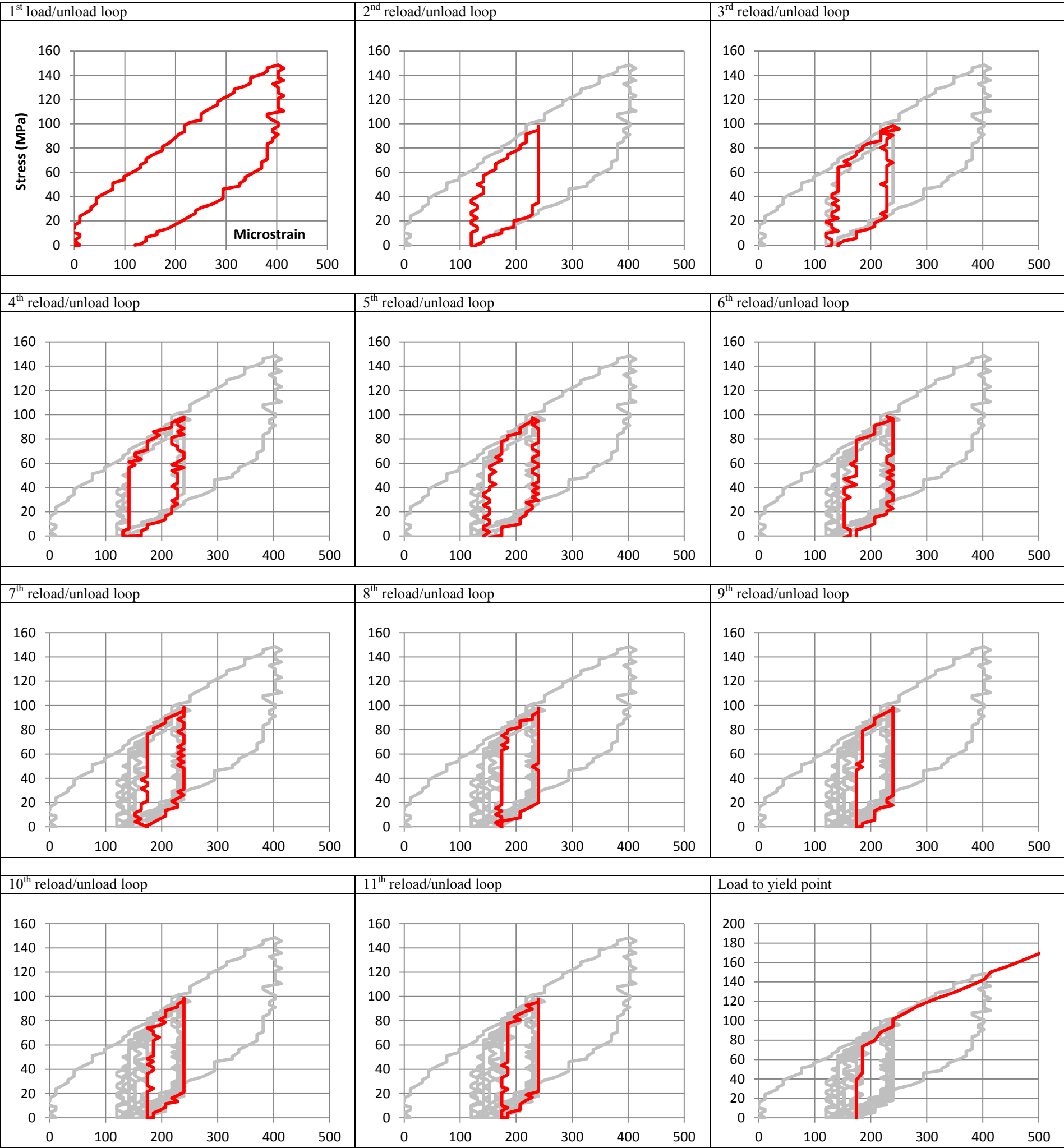


Figure 4-23. Pre-yield loops for specimen wrought iron bolt1811_3.

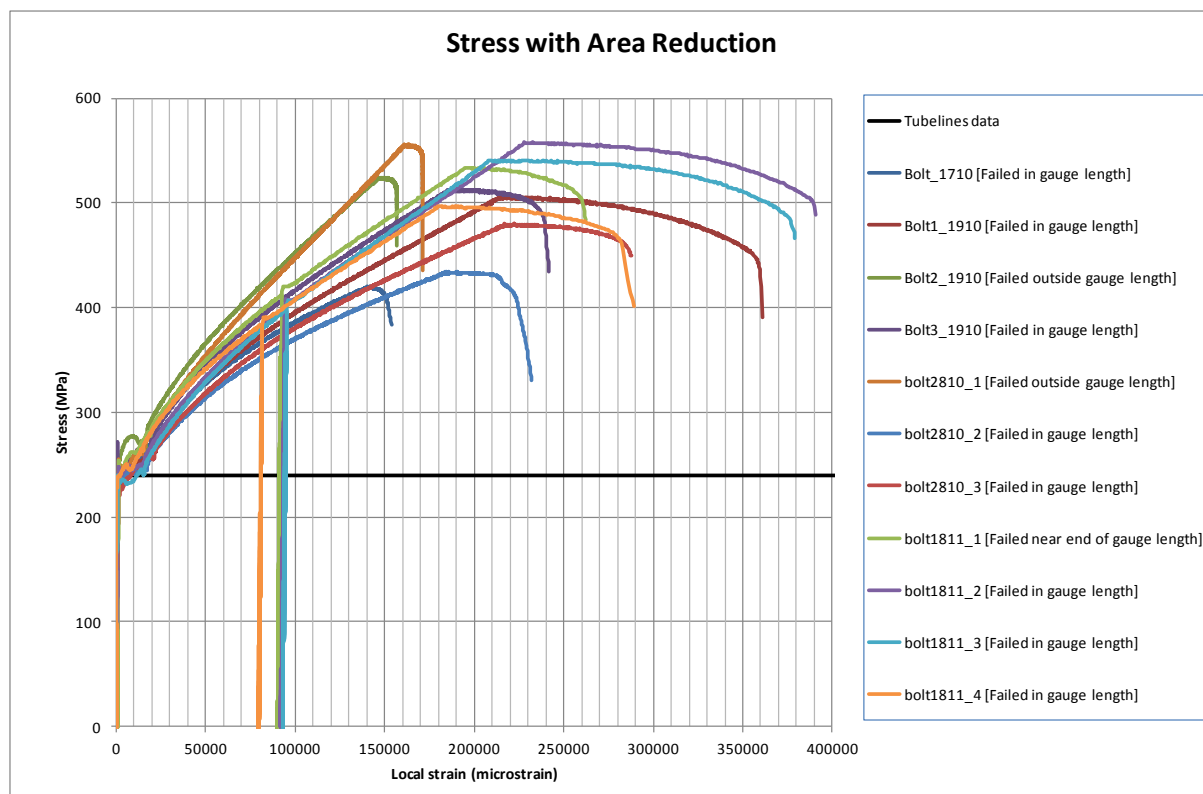


Figure 4-24. Stress-strain response of wrought iron specimens (with area reduction).

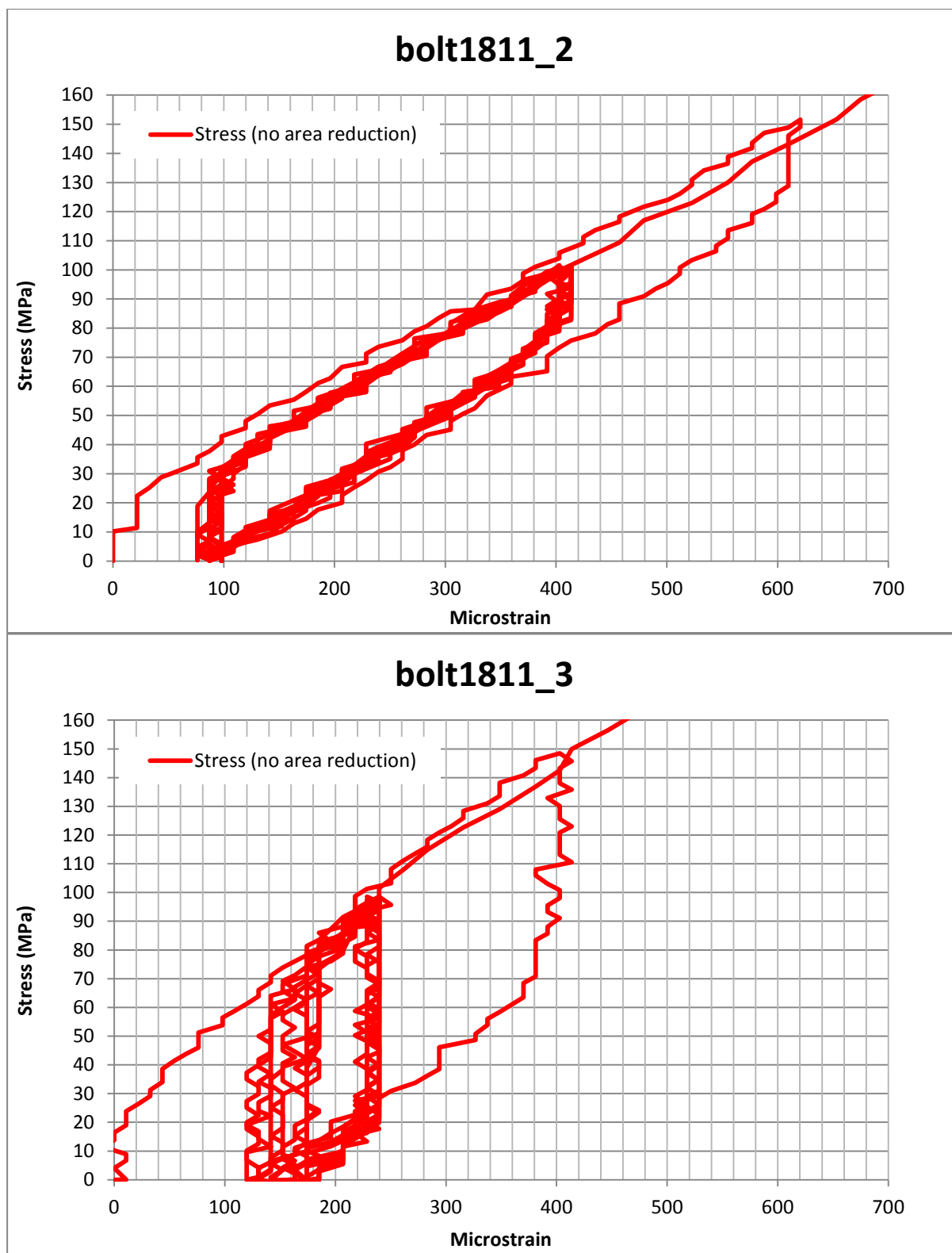


Figure 4-25. Examples of stress-strain response for the pre-yield unload-reload loops of wrought iron bolt specimens.

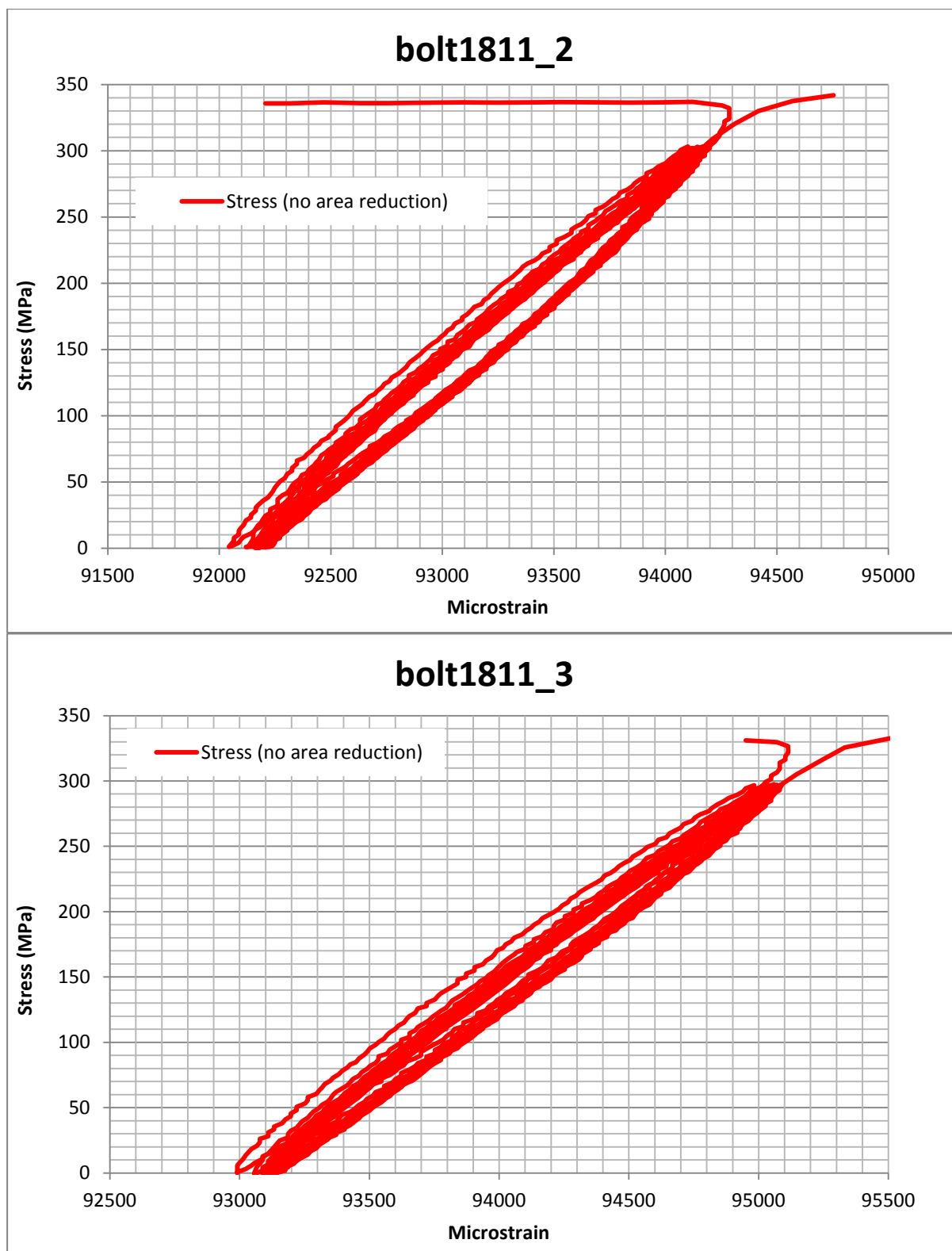


Figure 4-26. Examples of stress-strain response for the post-yield unload-reload loops of wrought iron bolt specimens.

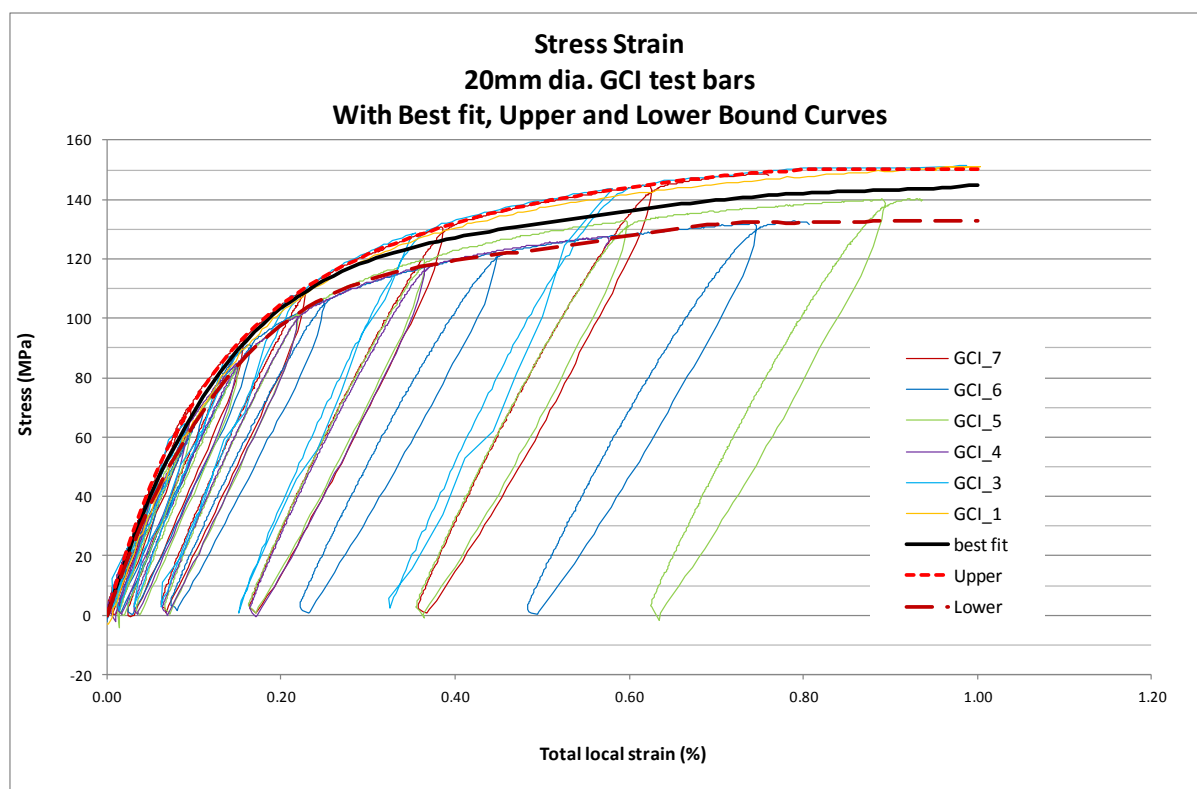


Figure 4-27. Stress-strain data with best fit, upper bound and lower limit curves for GCI specimens.

5 Revisiting the two segment test

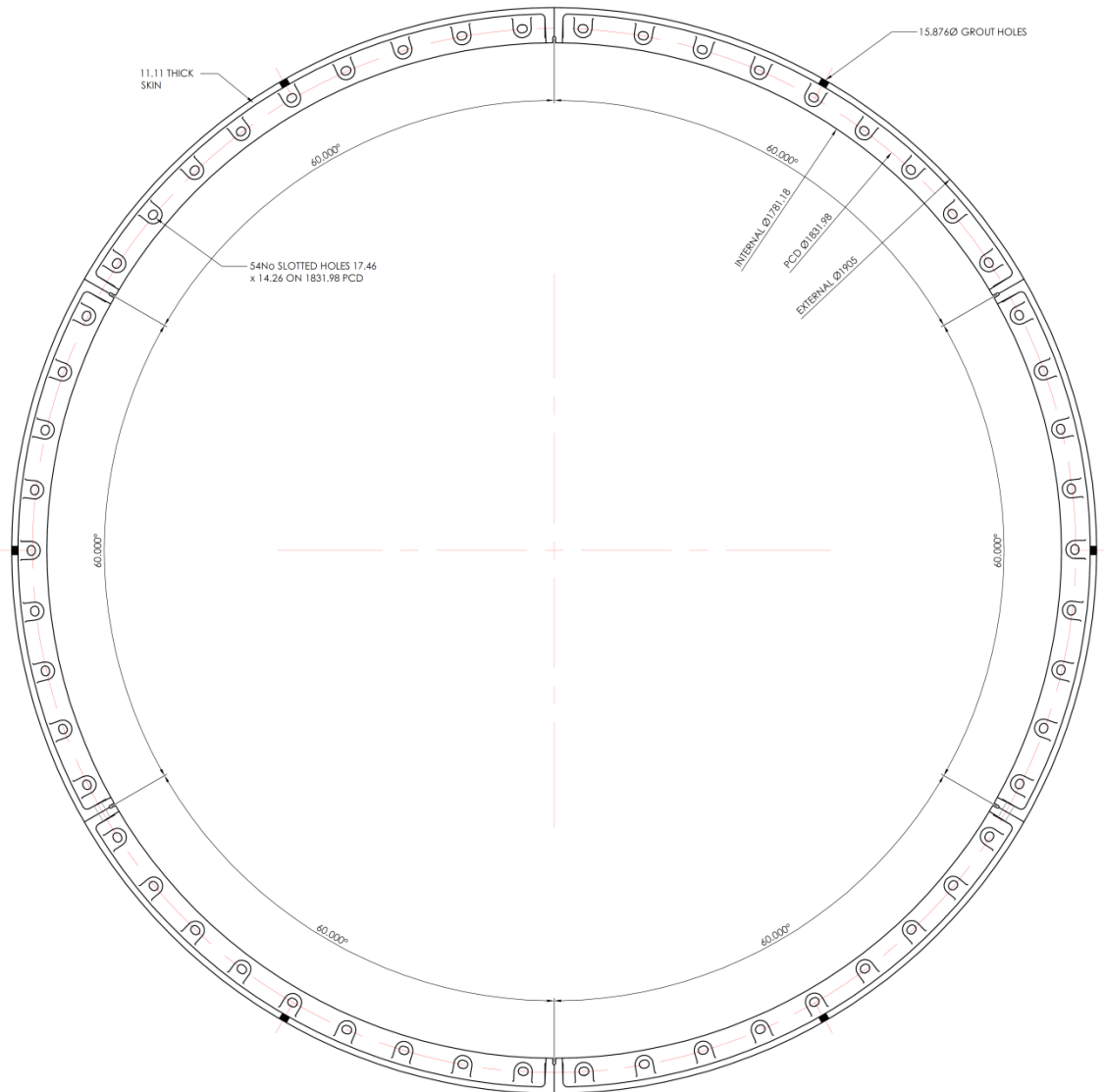


Figure 5-1. Half-scale grey cast iron ring cross section. All dimensions in millimetres.

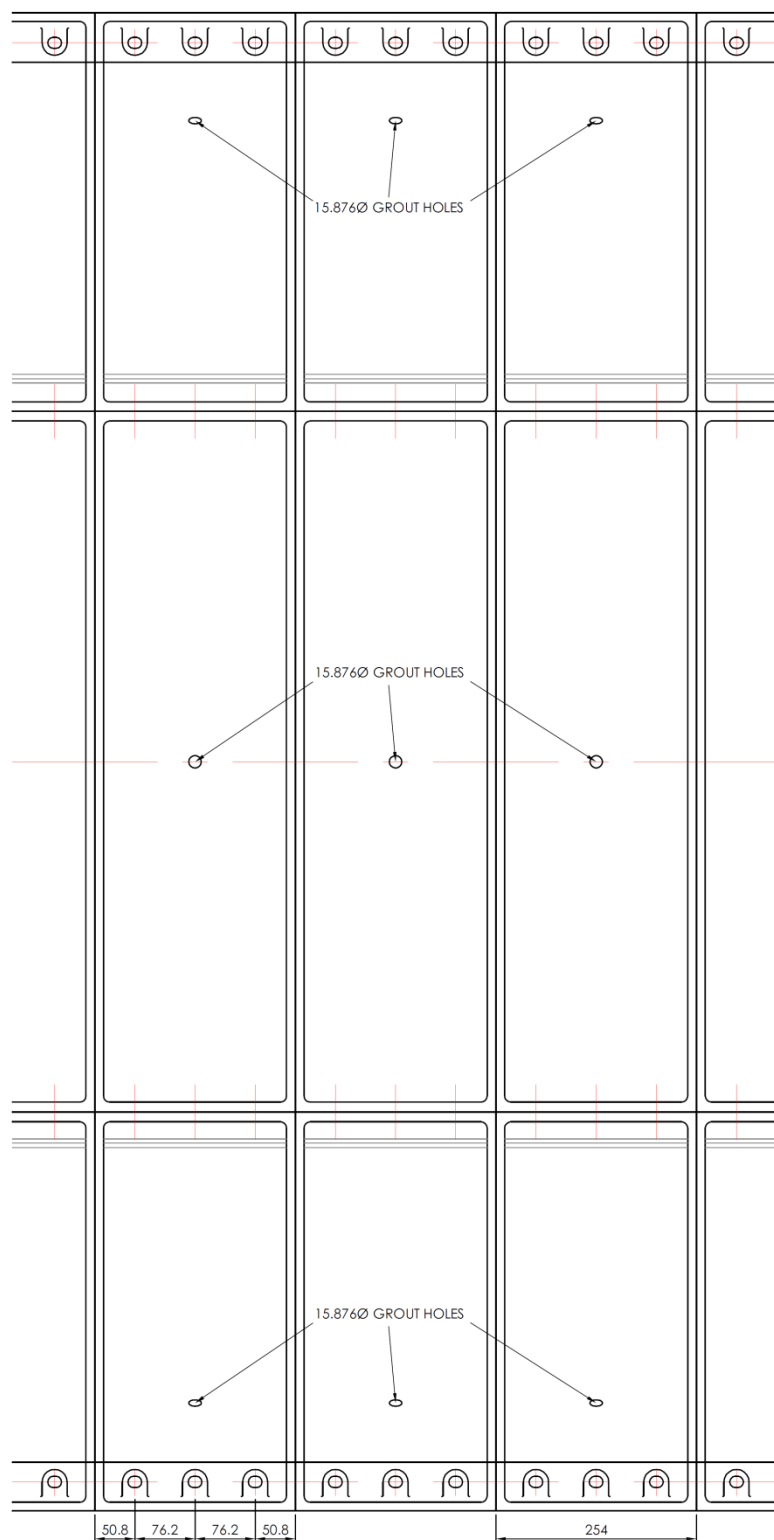


Figure 5-2. Half-scale grey cast iron ring internal elevation. All dimensions in millimetres.

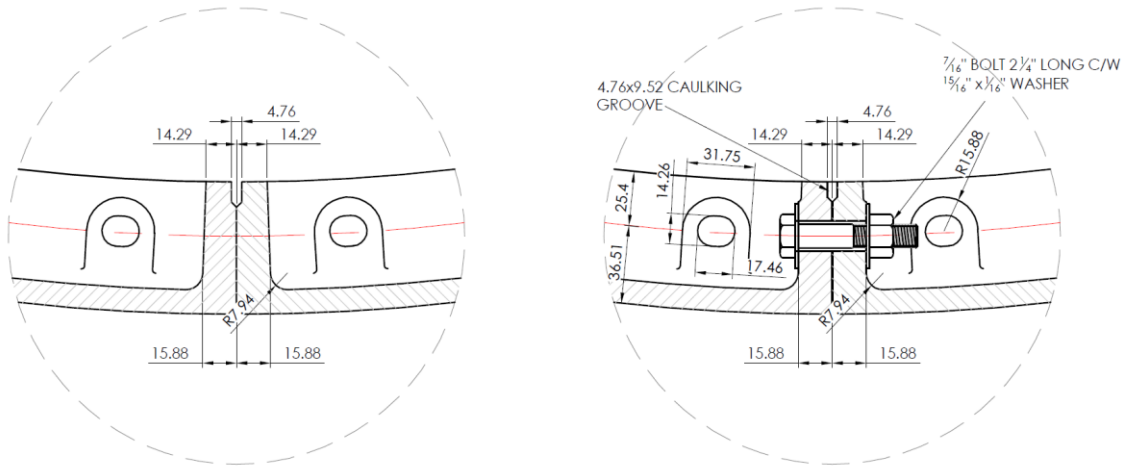


Figure 5-3. Half-scale grey cast iron longitudinal joint.

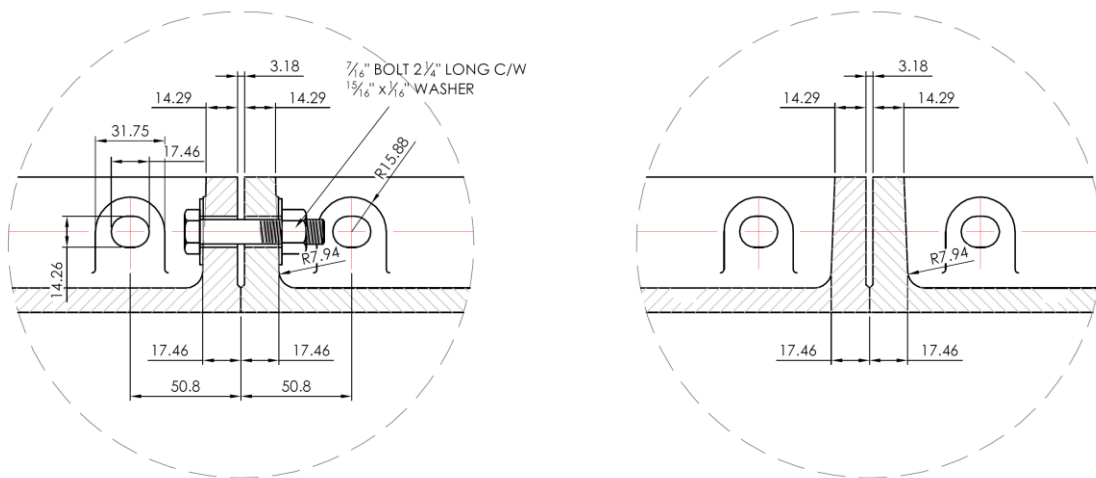


Figure 5-4. Half-scale grey cast iron circumferential joint.

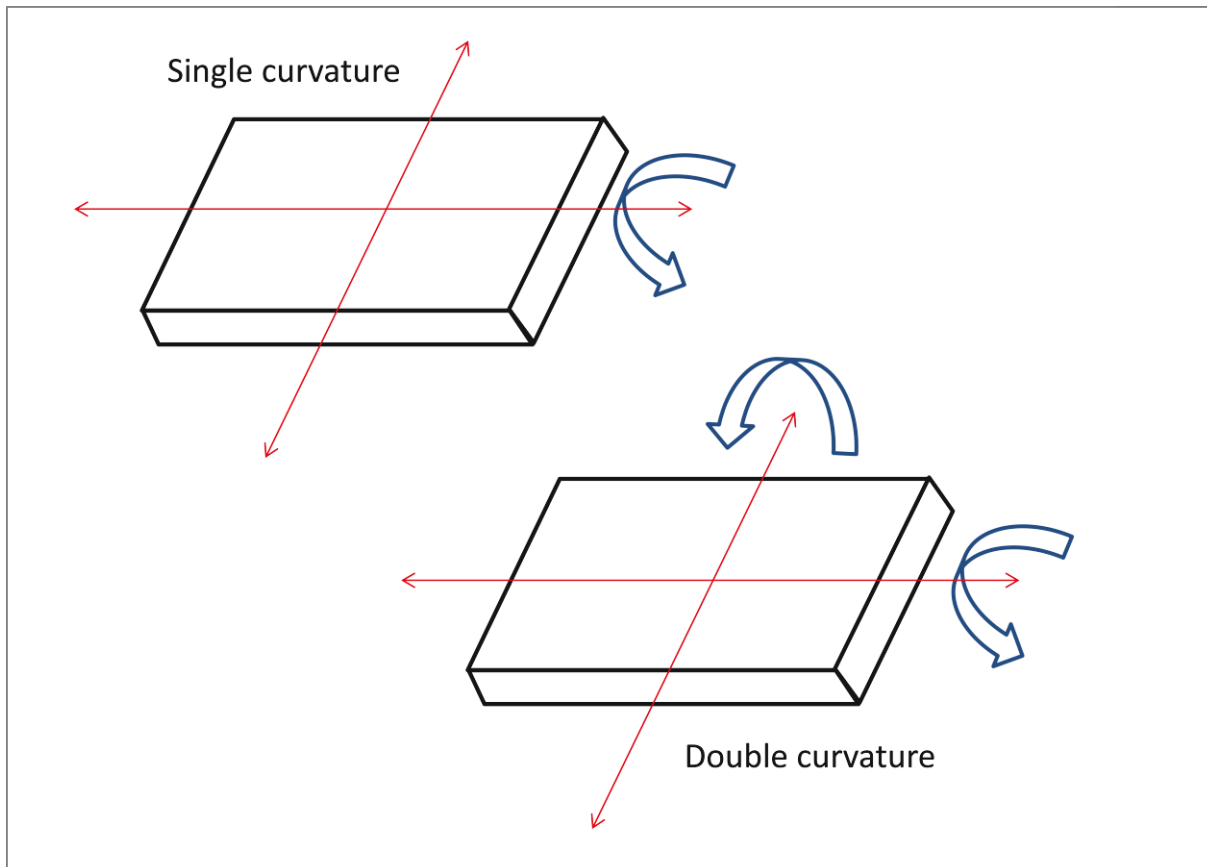


Figure 5-5. Schematic showing single curvature and double curvature bending.

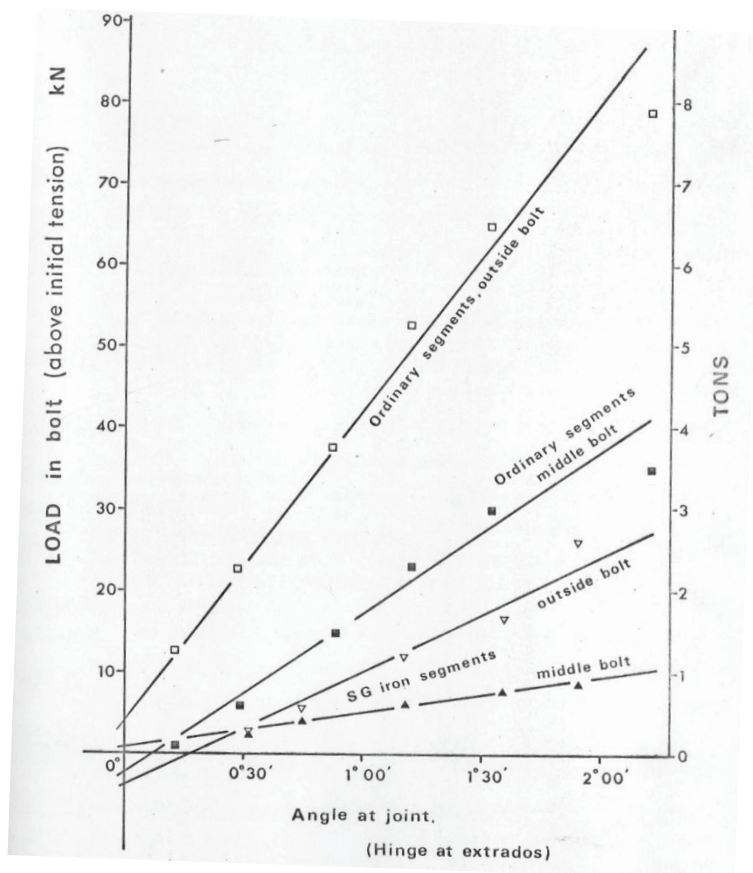


Figure 5-6. Increase in bolt load against joint opening at intrados (Thomas, 1977).

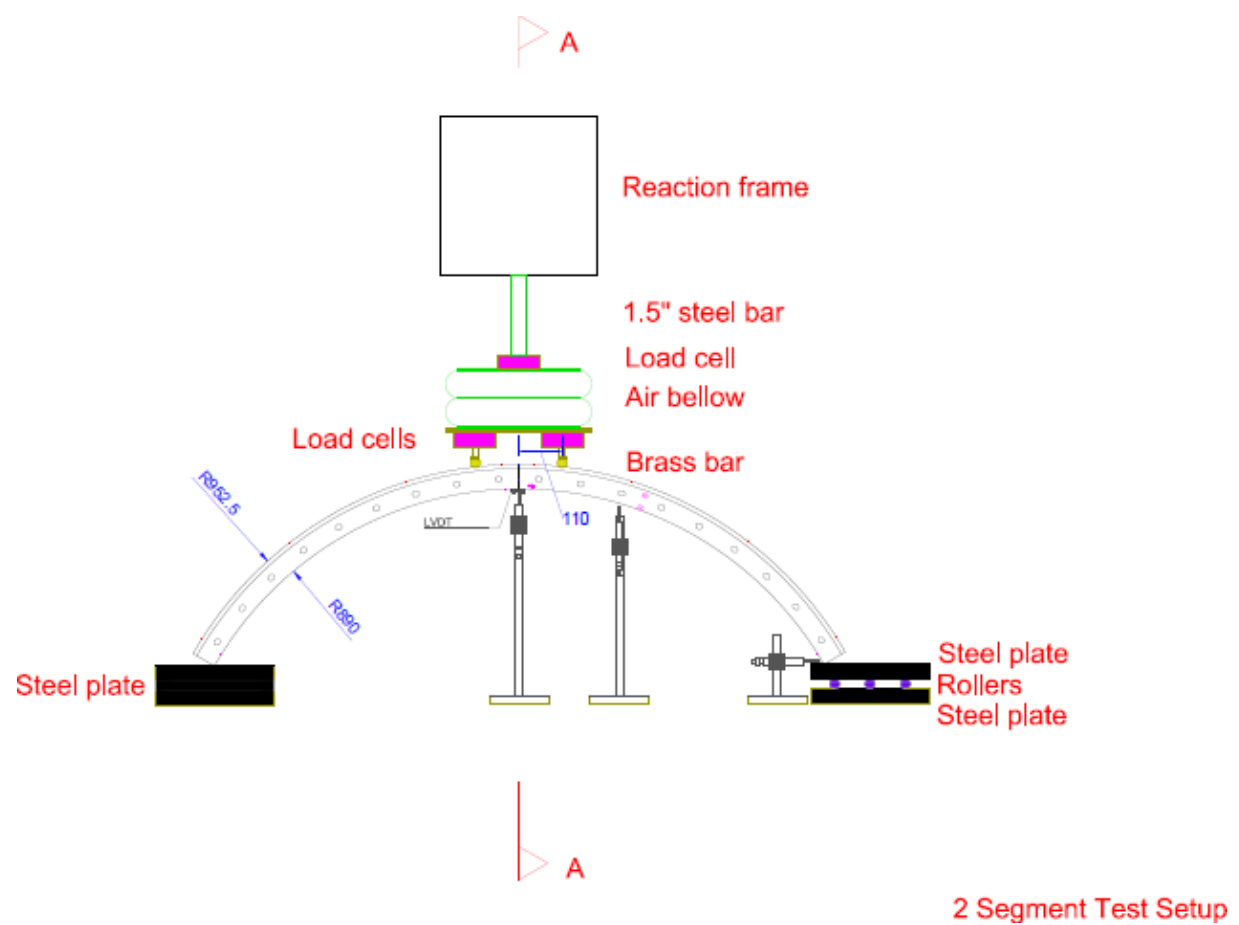


Figure 5-7. Schematic diagram of the two segment test setup at Imperial College.

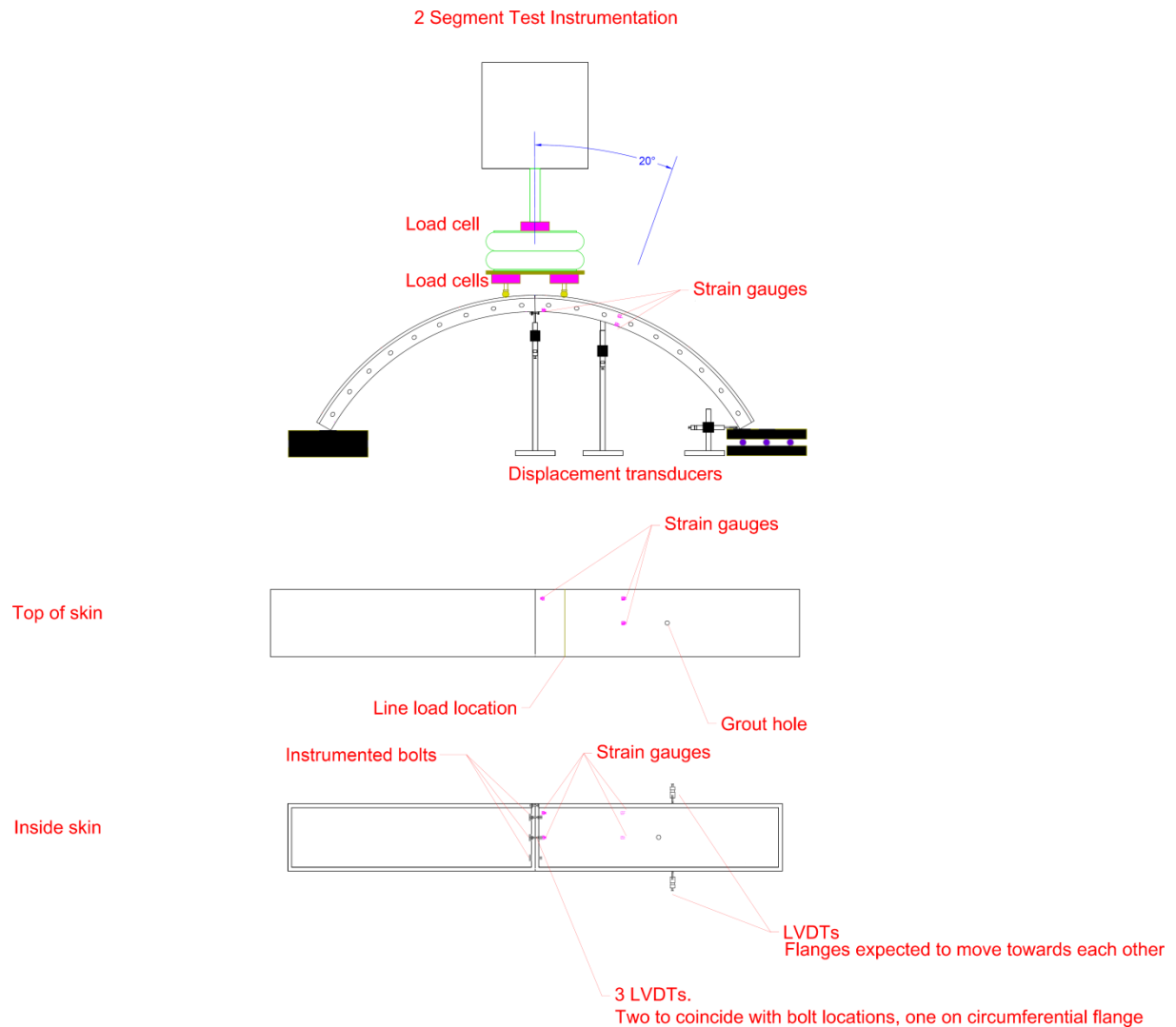


Figure 5-8. Instrumentation for the two segment test at Imperial College.



Figure 5-9. The two segment test setup at Imperial College with roller support at RHS end.

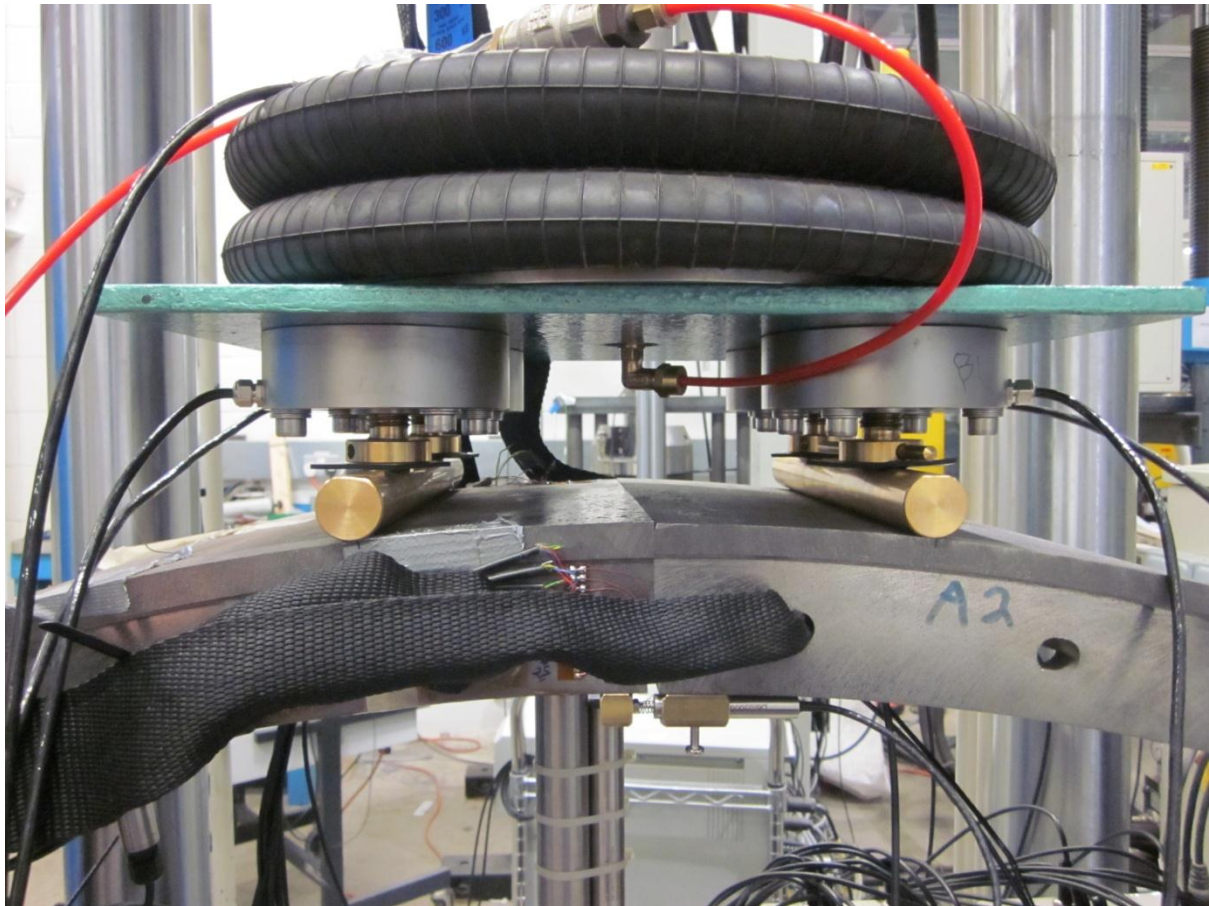


Figure 5-10. Line loads applied to the skin on the extrados of the segments.

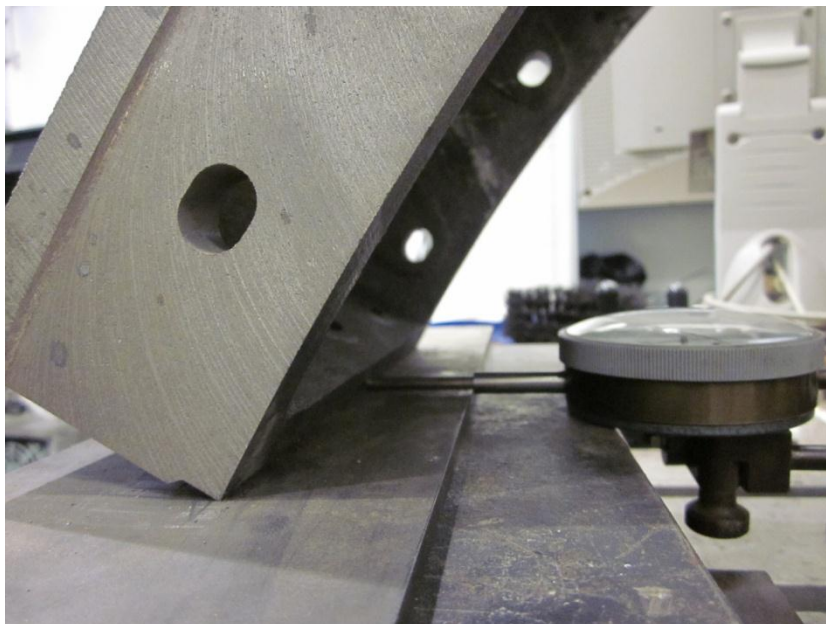


Figure 5-11. One support of the jointed arch is pinned. A dial gauge was used to measure potential translation/rotation.

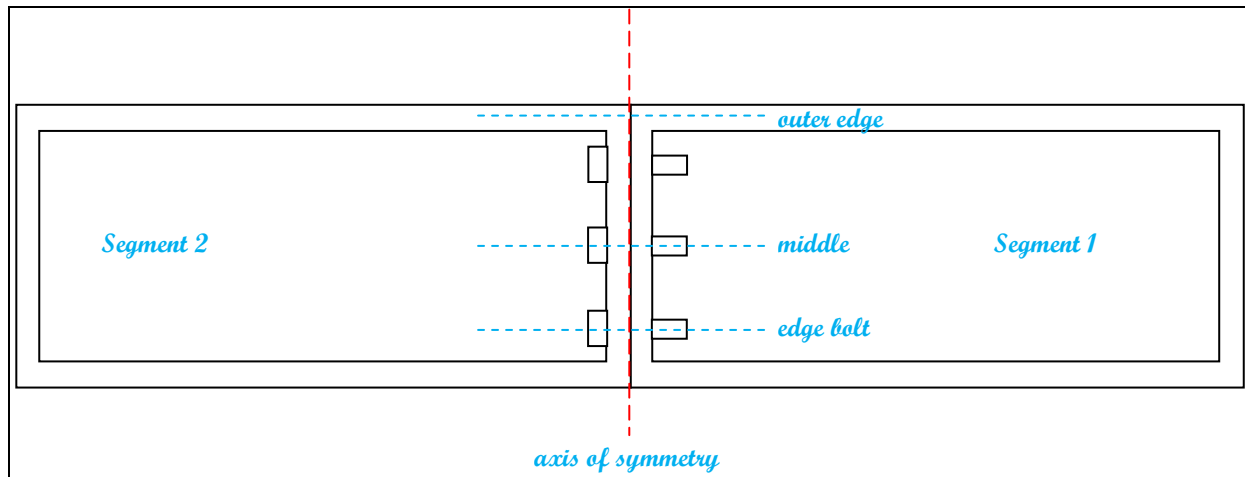


Figure 5-12. Locations of LVDTs at intrados: “middle”, “edge bolt” and “outer edge”.

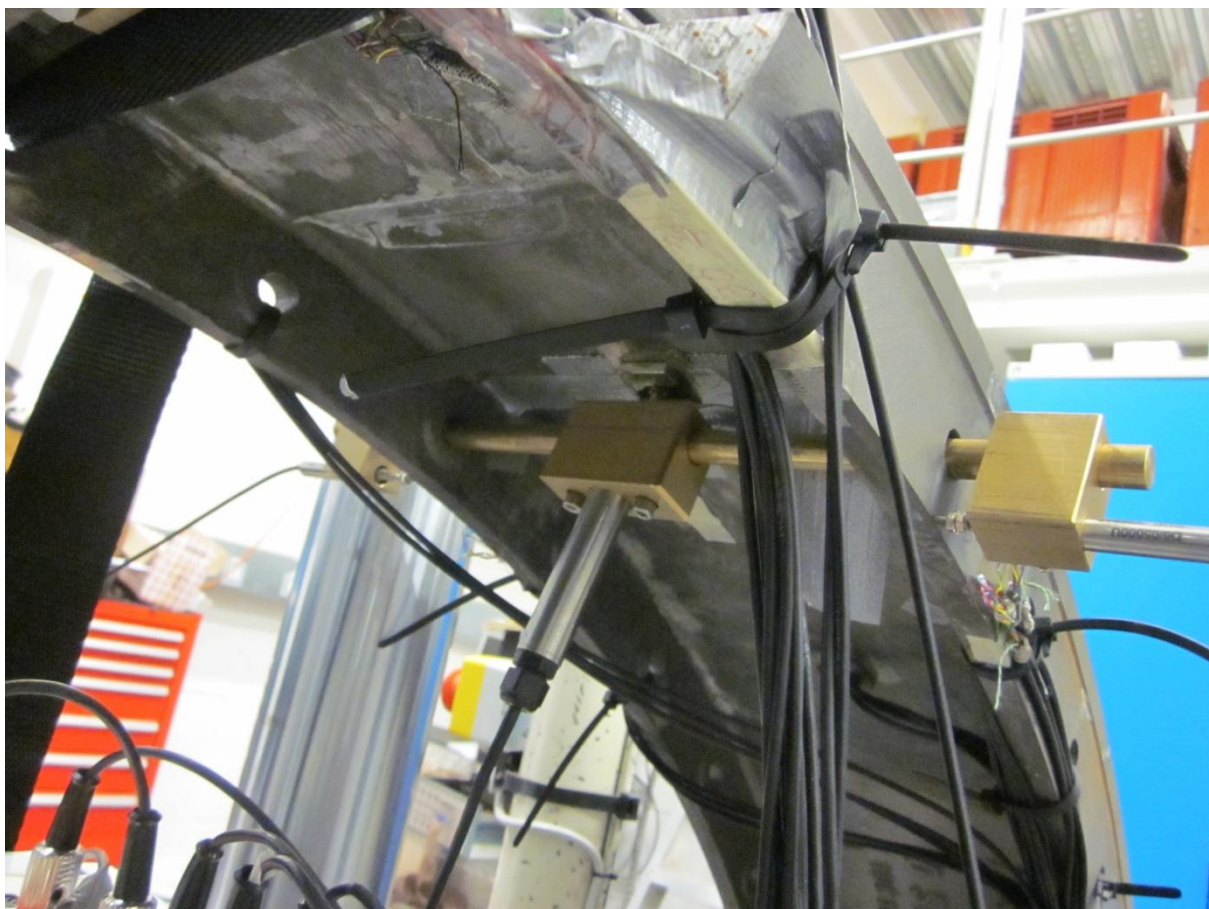


Figure 5-13. LVDTs for measuring the deflection of the circumferential flanges and displacement transducer for measuring the skin deflection.



Figure 5-14. Grade 4.6 steel bolts instrumented with strain gauges.



Figure 5-15. Grommets were used in the final set of tests.



Figure 5-16. Close up view of the roller support.

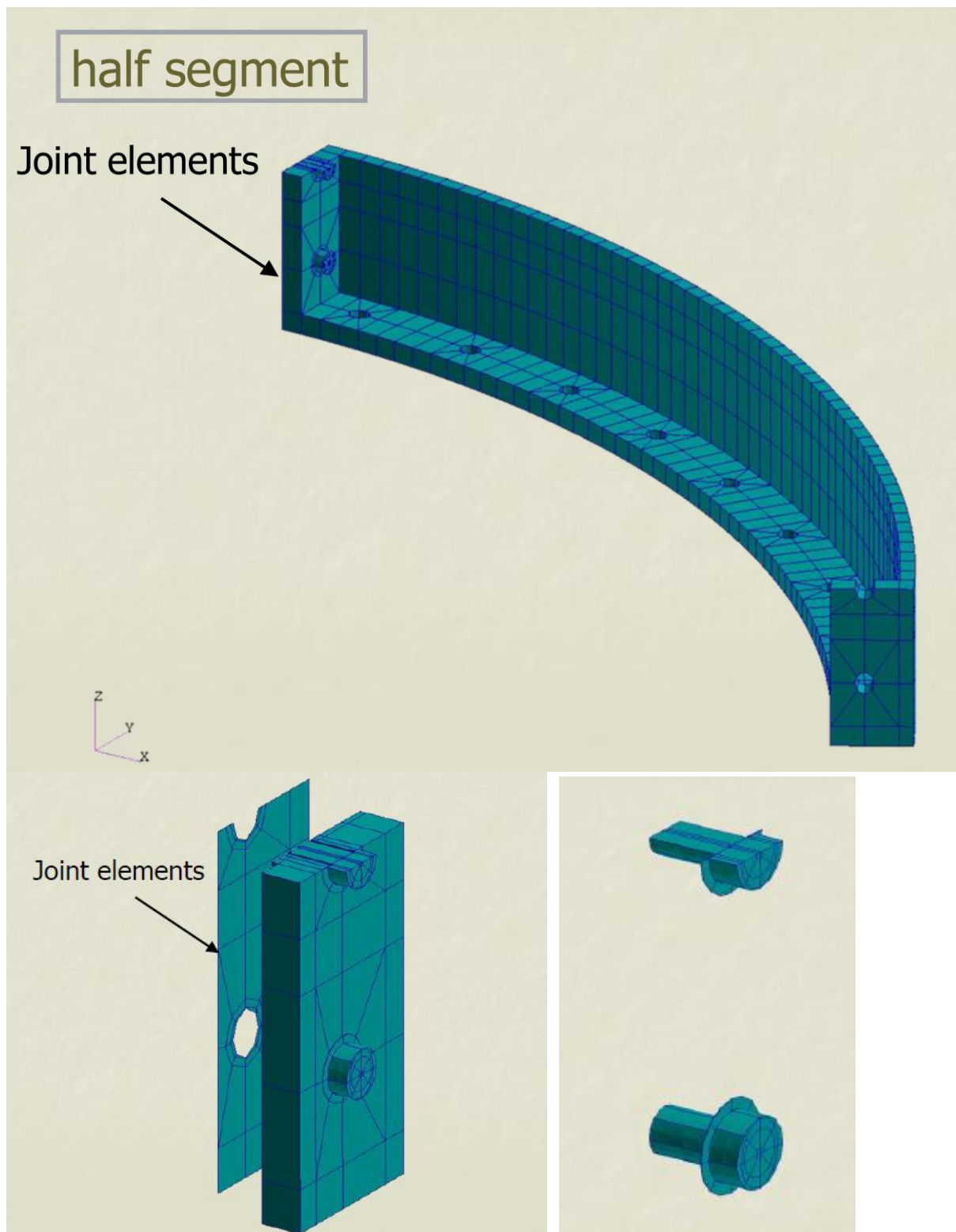


Figure 5-17. FE meshes for segment with details for the joint and bolts.

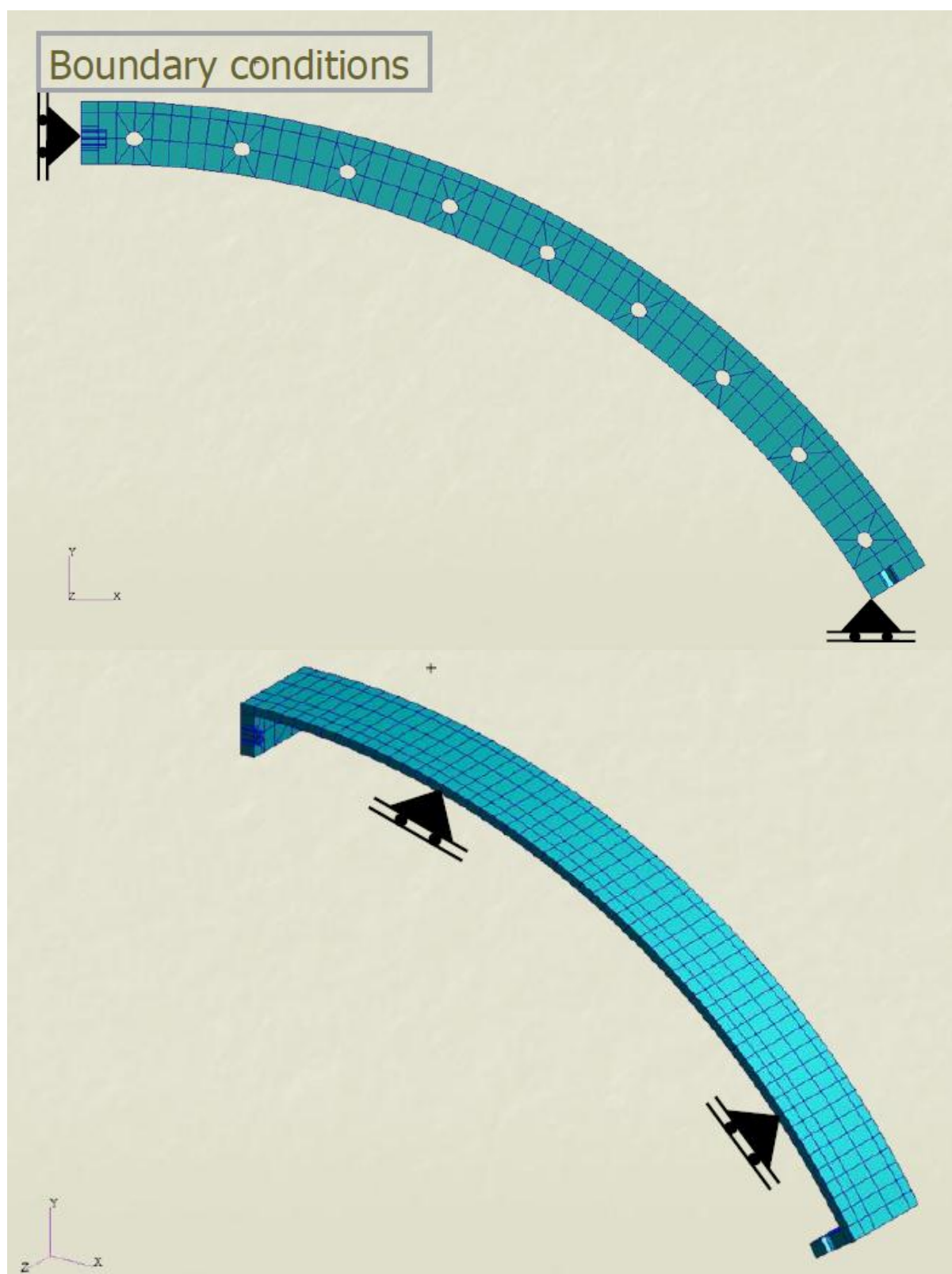


Figure 5-18. Boundary conditions for FE mesh.

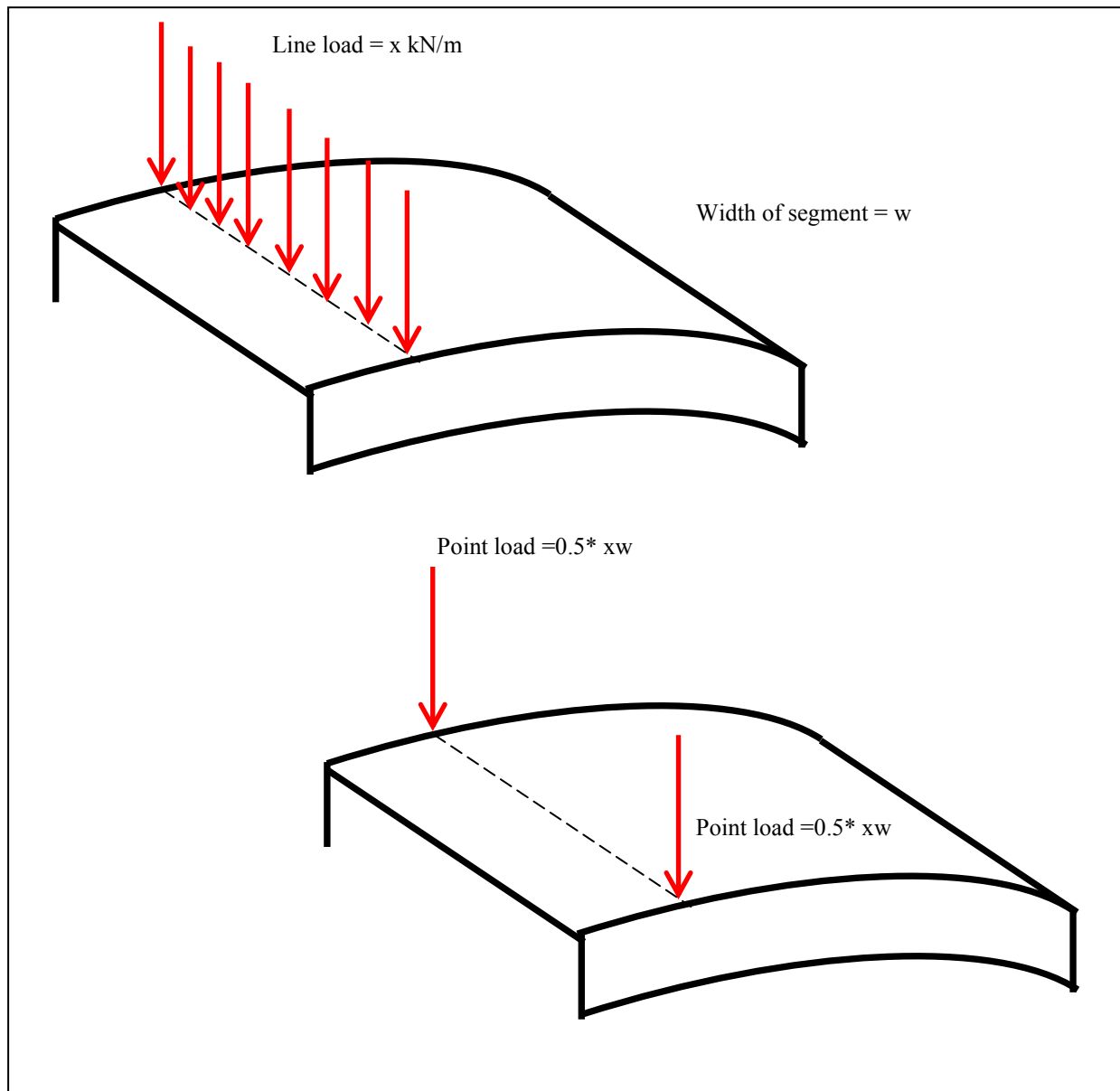


Figure 5-19. Application of loading as line load or equivalent point loads.

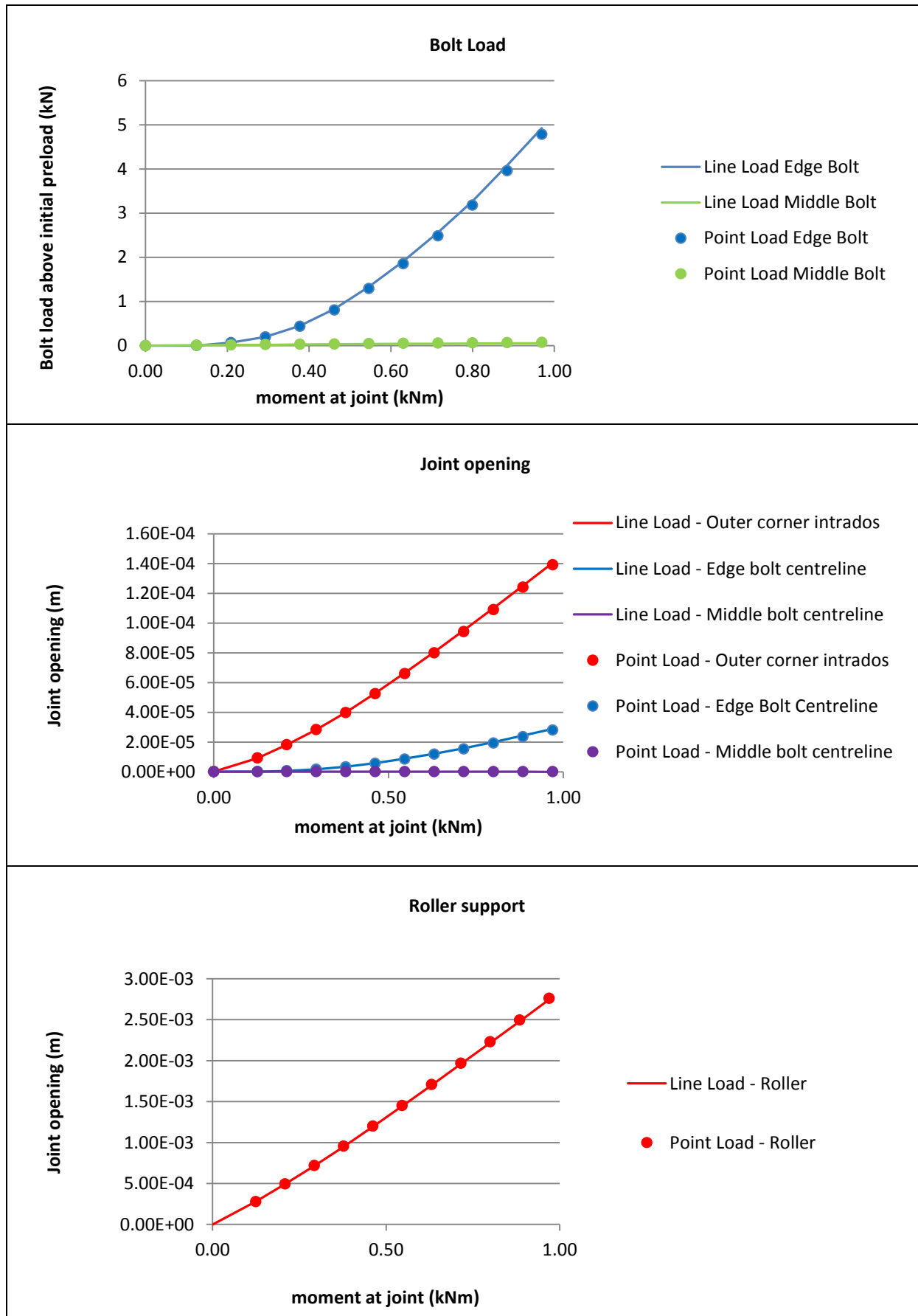
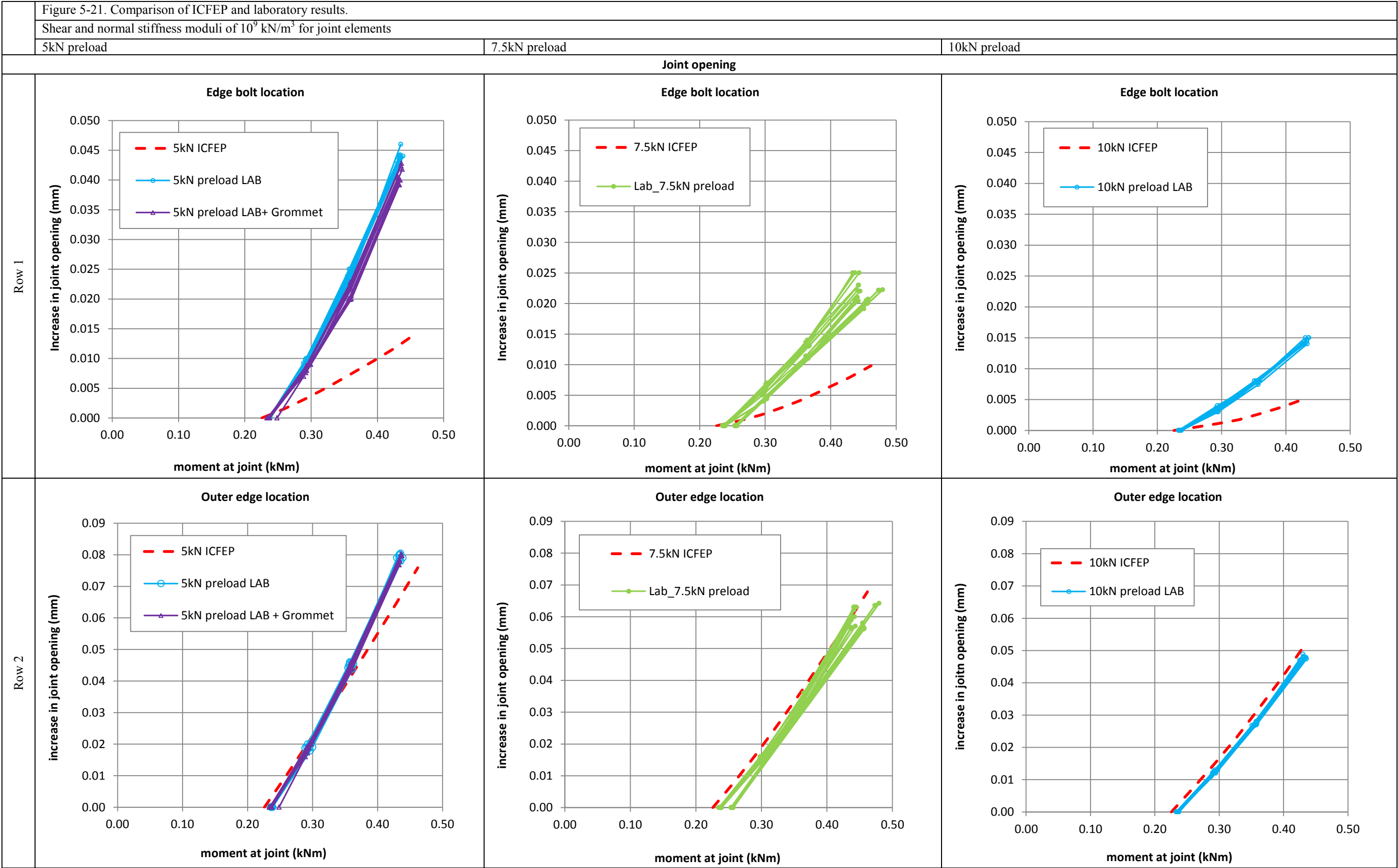
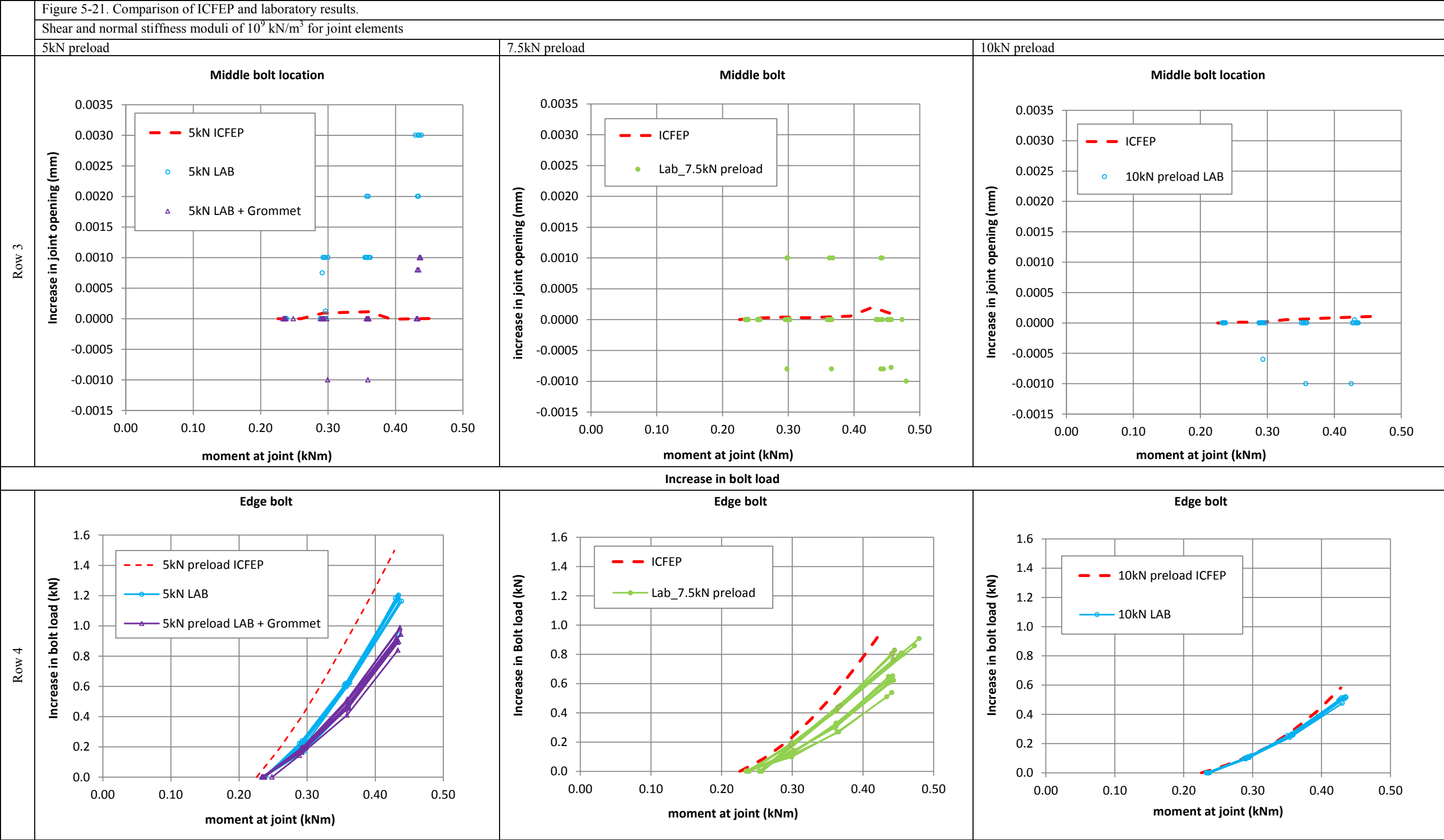
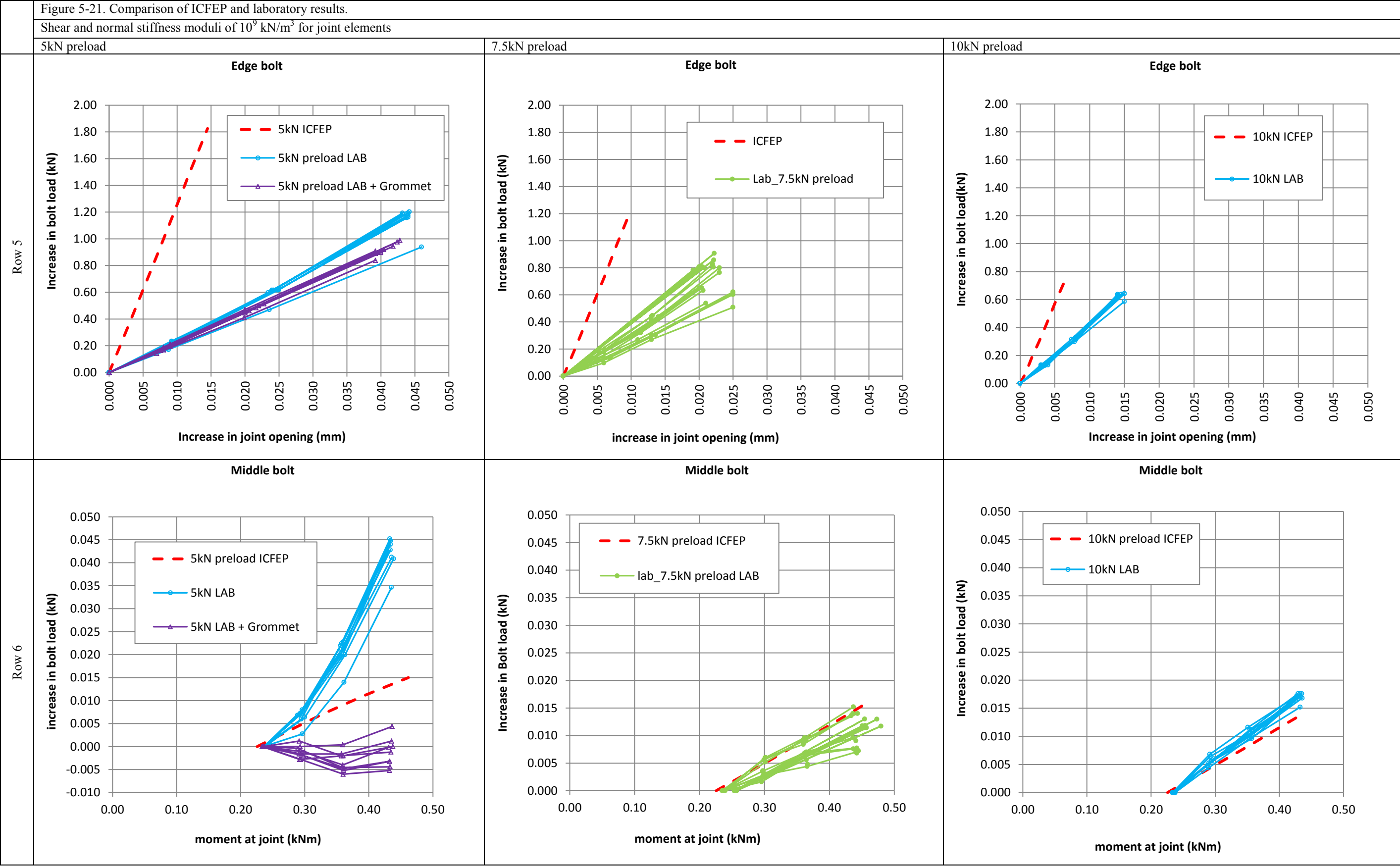


Figure 5-20. Comparison of point load and line load application using FE.







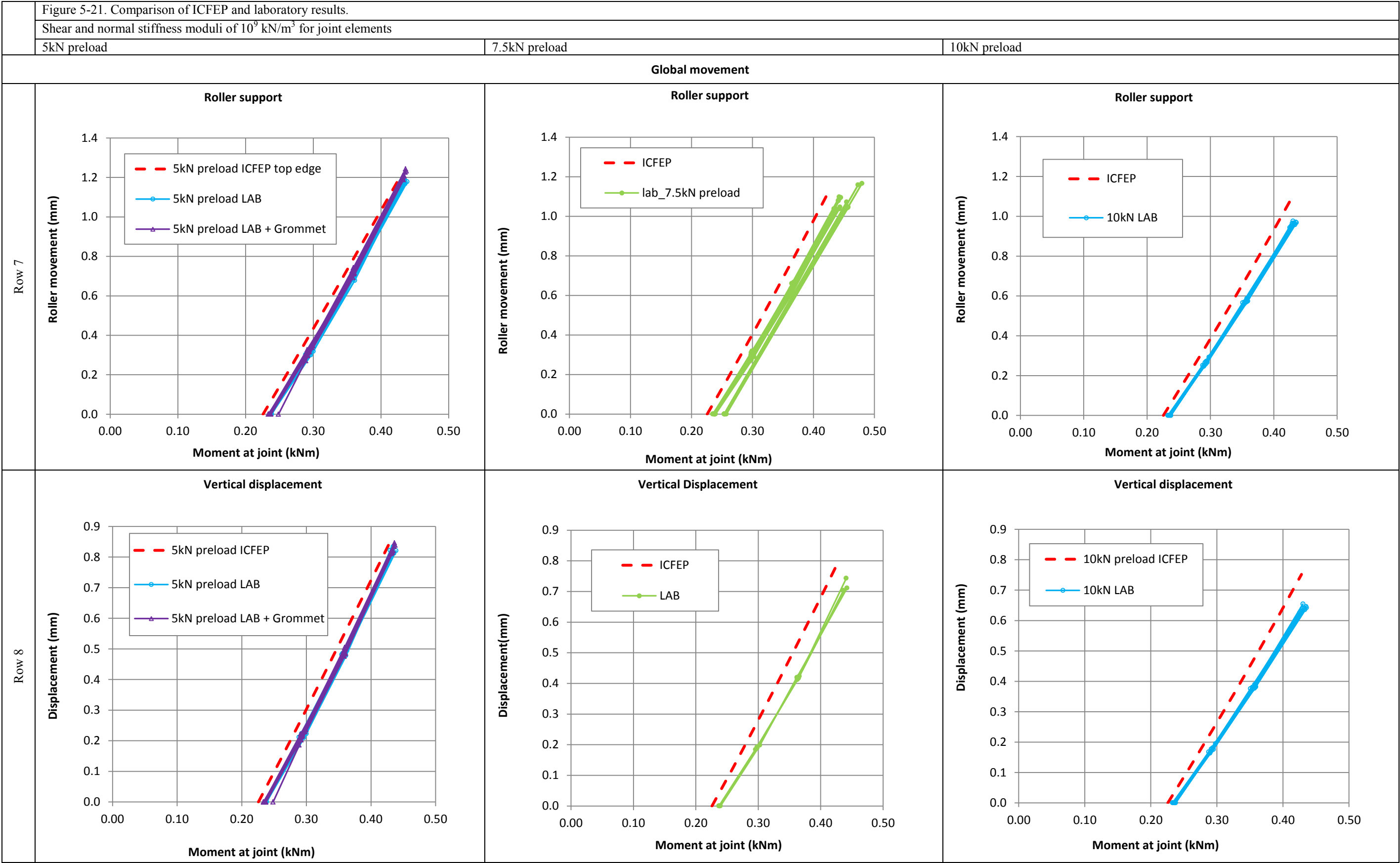


Figure 5-21. Comparison of ICFEP and laboratory results.

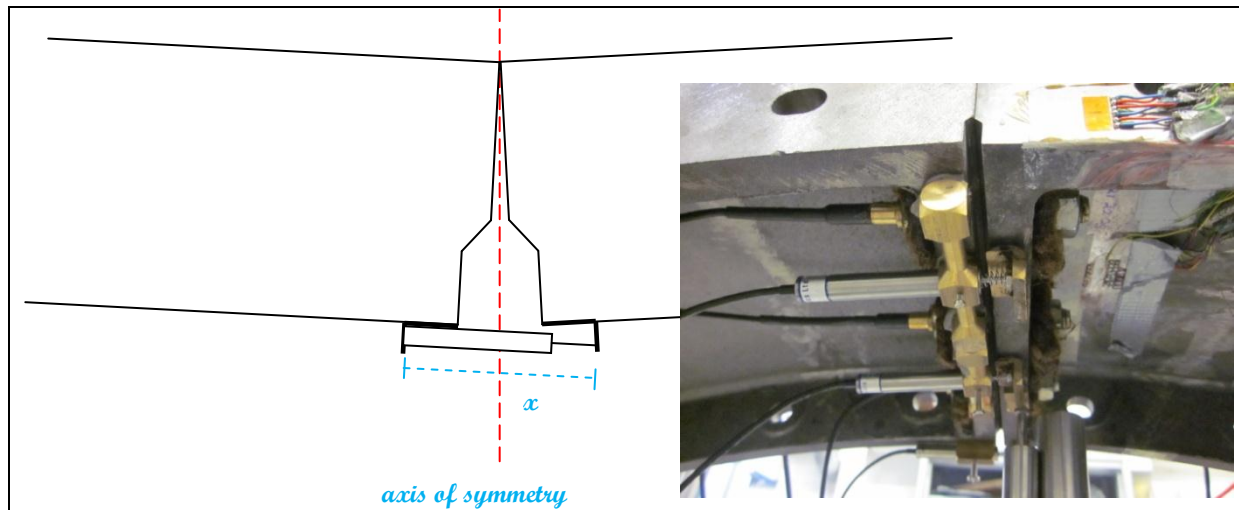


Figure 5-22. The joint opening magnitude is taken to be 'x' in the FEA.

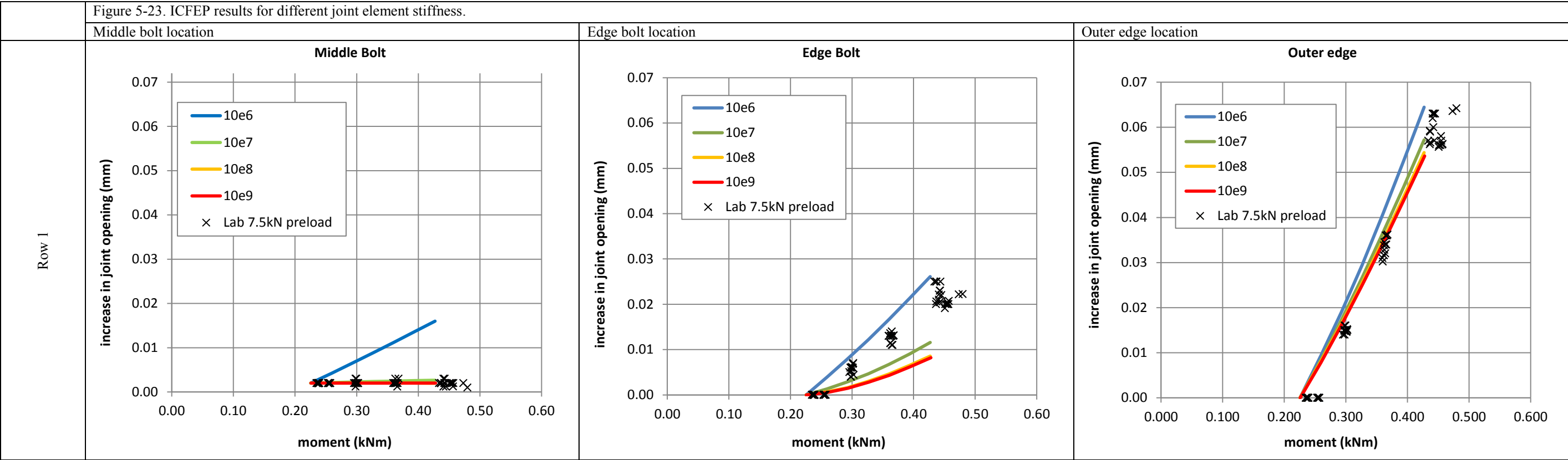


Figure 5-23. ICFEP results for different joint element stiffness.

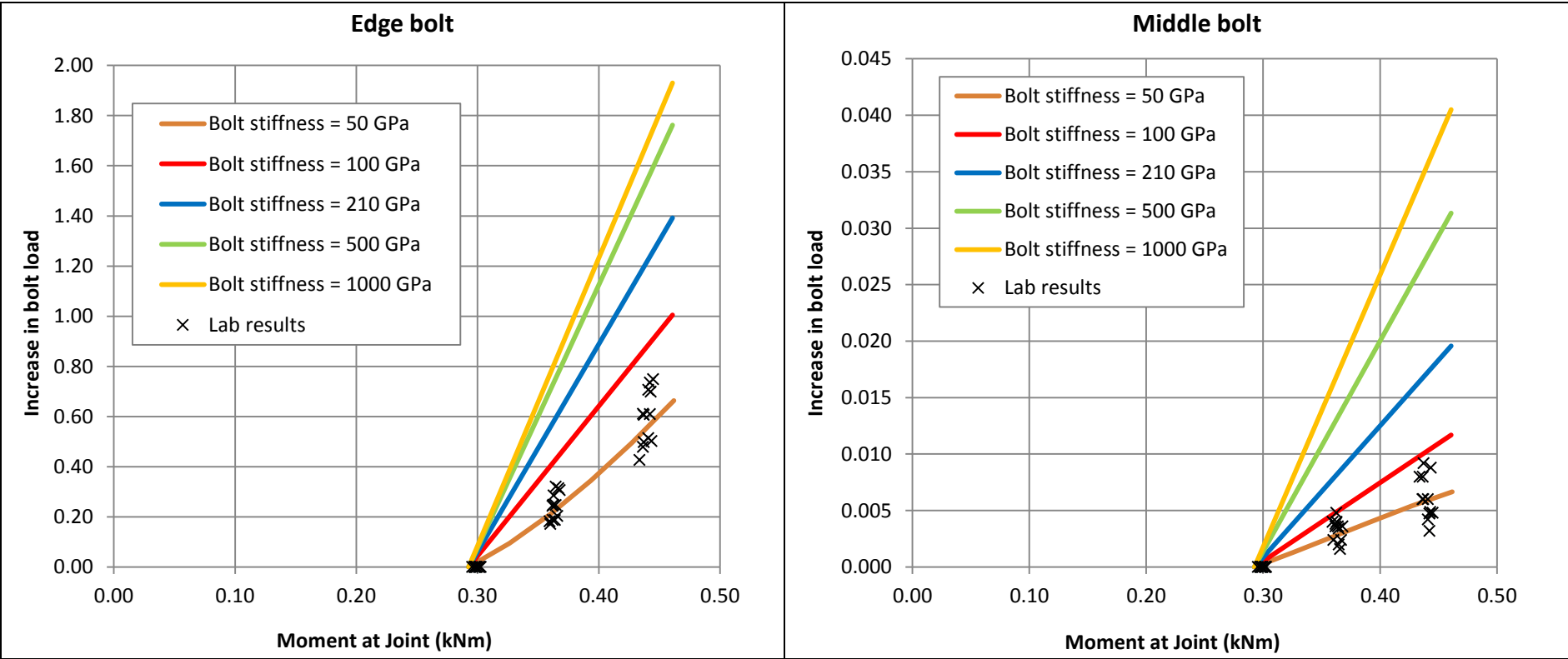


Figure 5-24. Different bolt stiffness moduli.

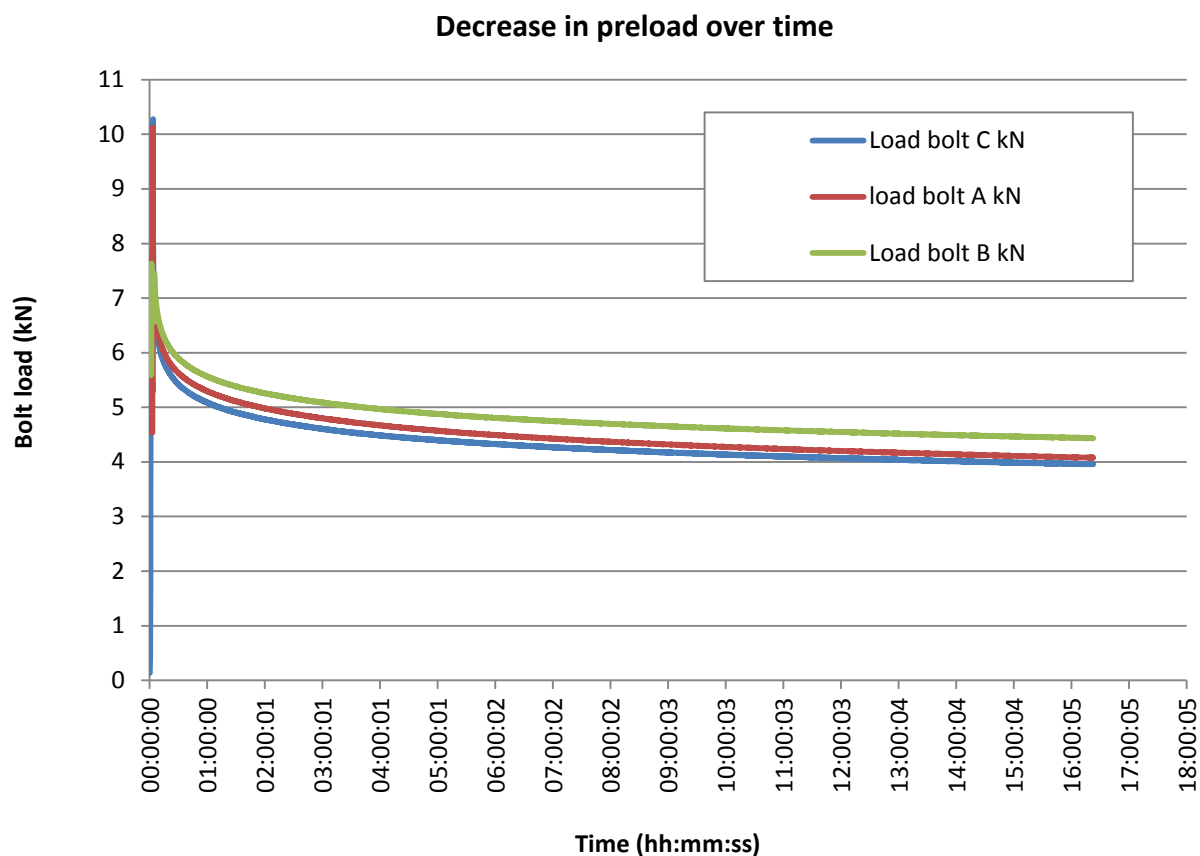
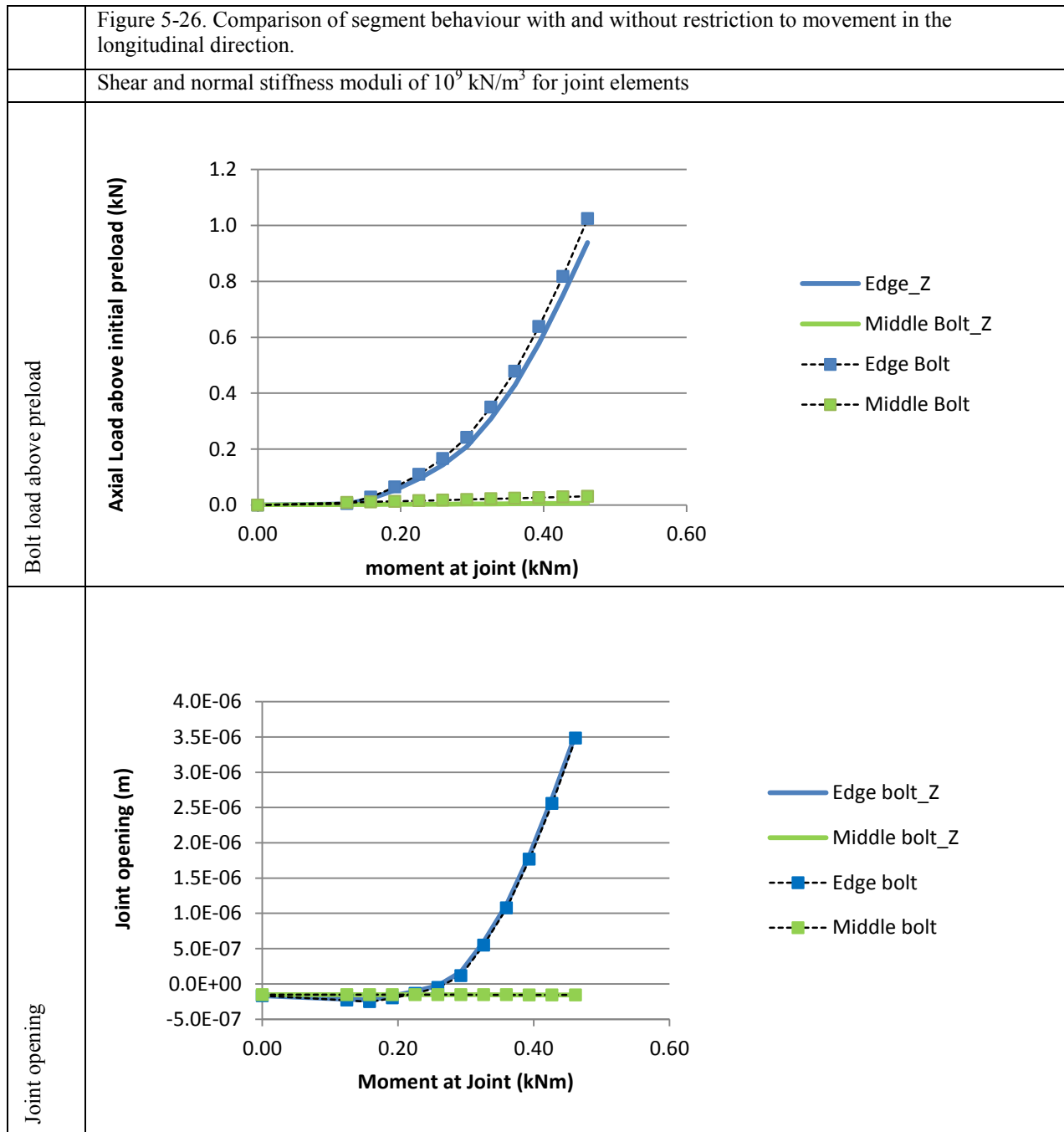


Figure 5-25. Decrease in bolt load with time after initial preload – grommets in place.



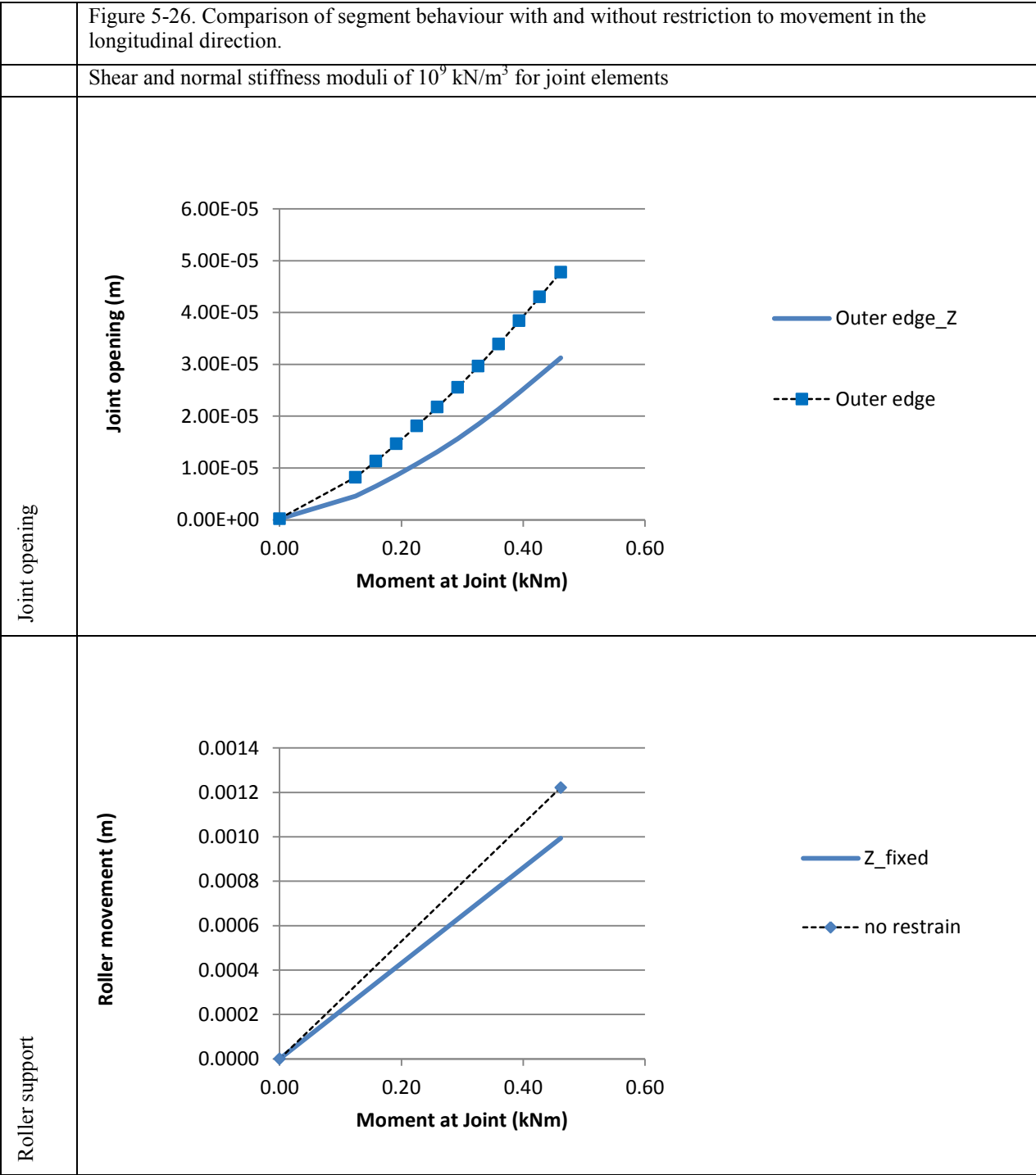


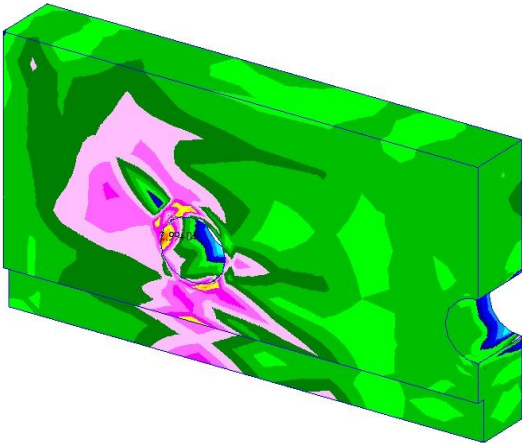
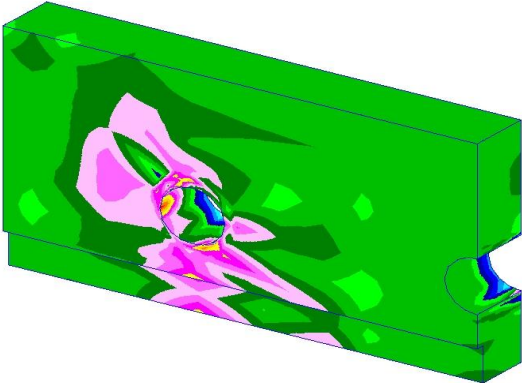
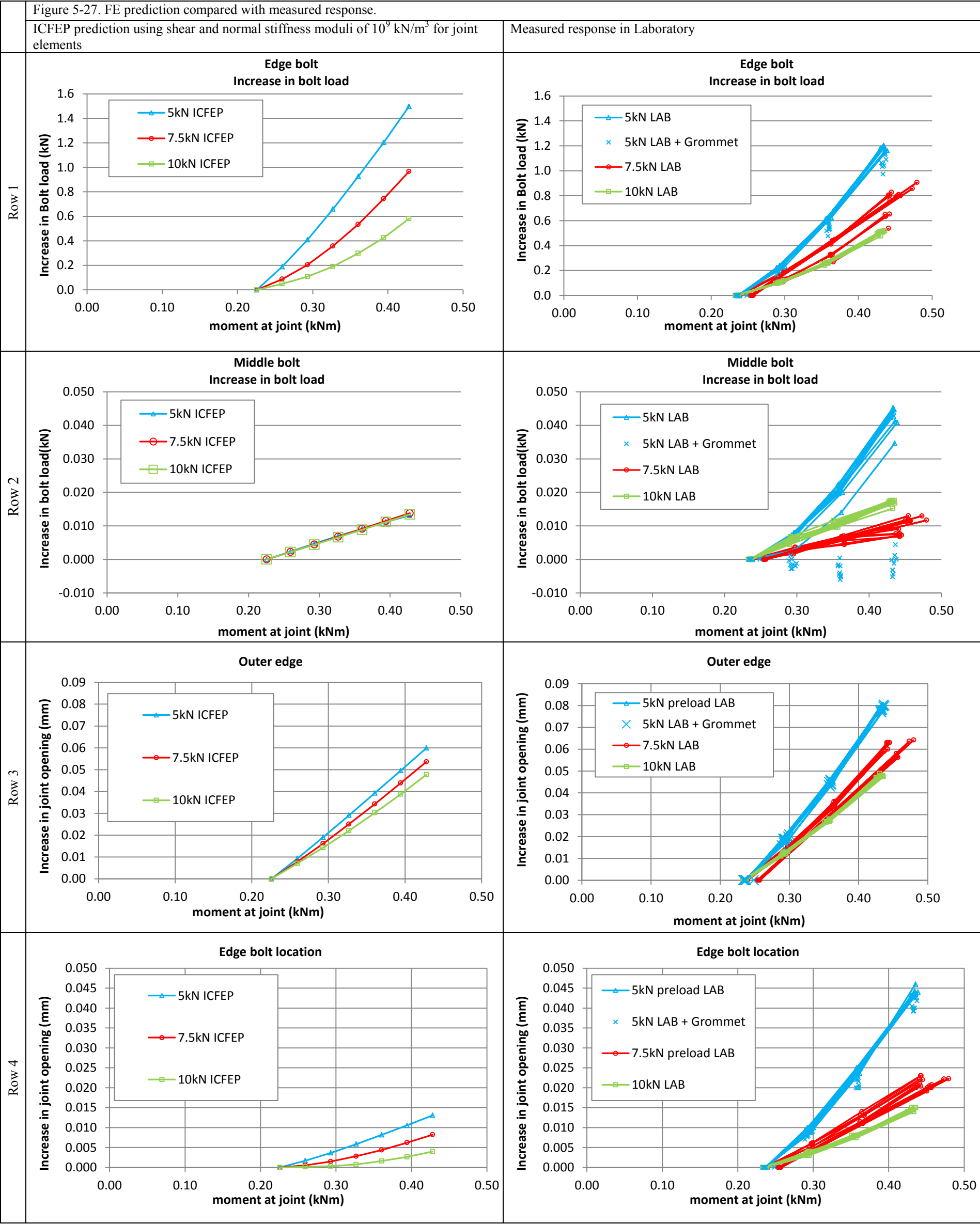
	Figure 5-26. Comparison of segment behaviour with and without restriction to movement in the longitudinal direction.
	Shear and normal stiffness moduli of 10^9 kN/m ³ for joint elements
Circumferential flanges NOT restrained Contours of stresses (kPa) Tensile stress positive	Patran 2007 r2 27-Oct-11 16:29:30 Fringe: LINE1, INCREMENT 12, CONTOURS OF ACC. TOTAL STRESS: S1,...(NON-LAYERED)  A 3D stress contour plot showing the distribution of tensile stress (S1) on a rectangular block with a central hole. The plot is color-coded, with green representing lower stress and yellow/red representing higher stress. A color bar on the right indicates the stress scale from -5.20e+04 to 7.99e+04 kPa. A small 3D coordinate system (X, Y, Z) is shown in the bottom left corner. default_Fringe: Max 7.99+04 @Nd 4374 Min -5.20+04 @Nd 3925
Circumferential flanges RESTRICTED Contours of stresses (kPa) Tensile stress positive	Patran 2007 r2 10-Nov-11 19:33:39 Fringe: Z_FLANGE_E09, INCREMENT 12, CONTOURS OF ACC. TOTAL STRESS: S1,...(NON-LAYERED)  A 3D stress contour plot showing the distribution of tensile stress (S1) on a rectangular block with a central hole, similar to the one above but with restricted movement. The plot is color-coded, with green representing lower stress and yellow/red representing higher stress. A color bar on the right indicates the stress scale from -5.13e+04 to 7.16e+04 kPa. A small 3D coordinate system (X, Y, Z) is shown in the bottom left corner. default_Fringe: Max 7.16+04 @Nd 3928 Min -5.13+04 @Nd 3925

Figure 5-26. Comparison of segment behaviour with and without restriction to movement in the longitudinal direction.



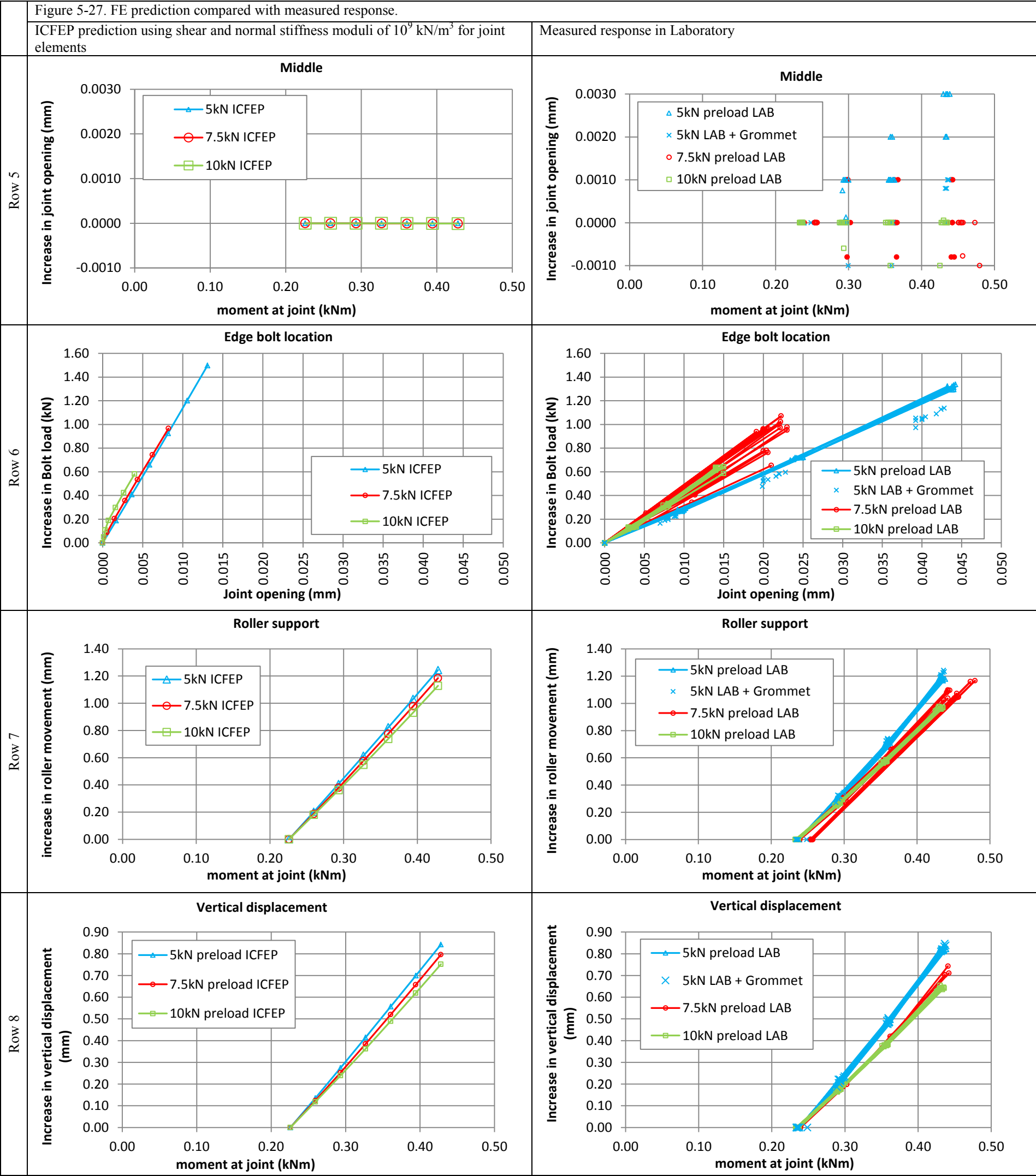
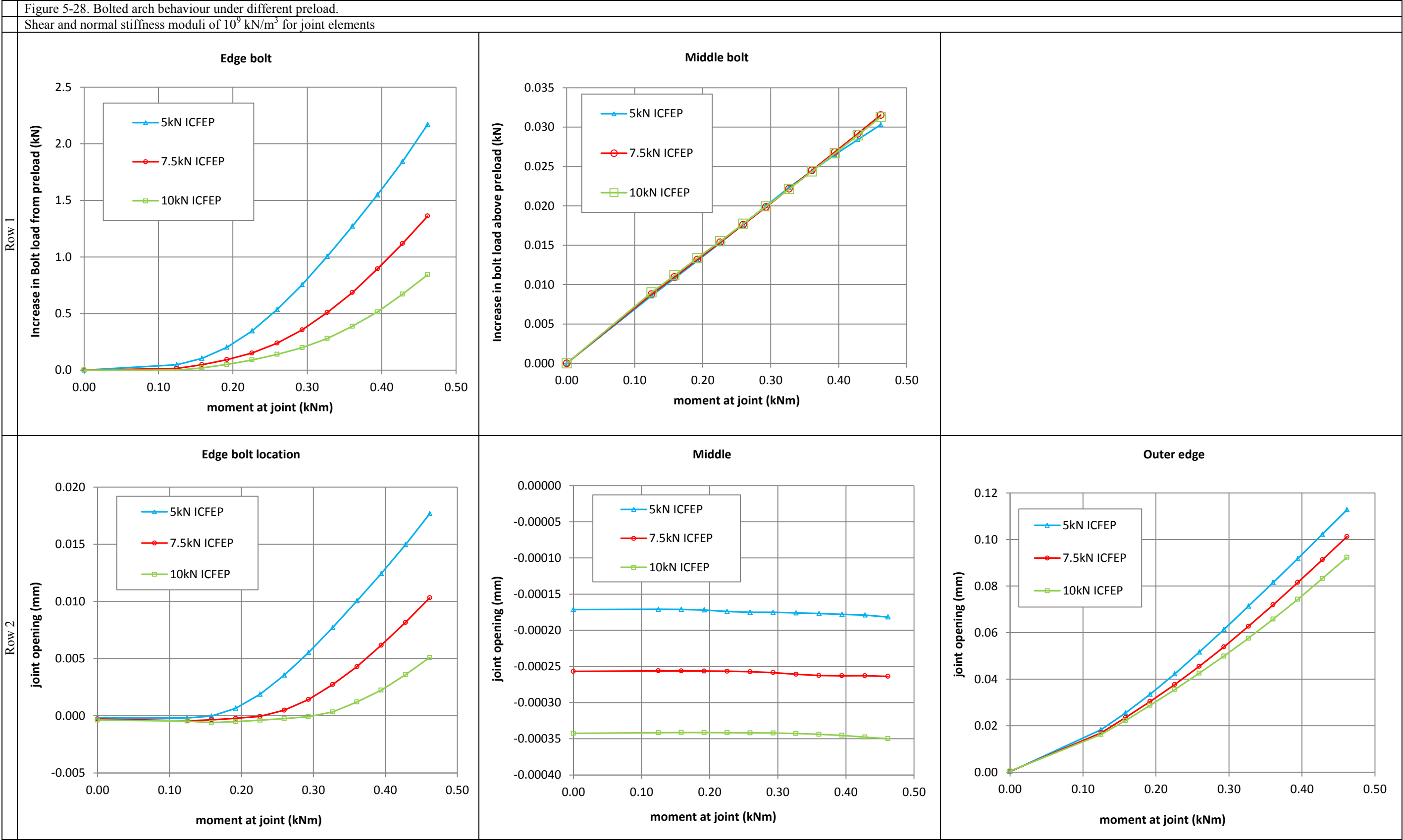
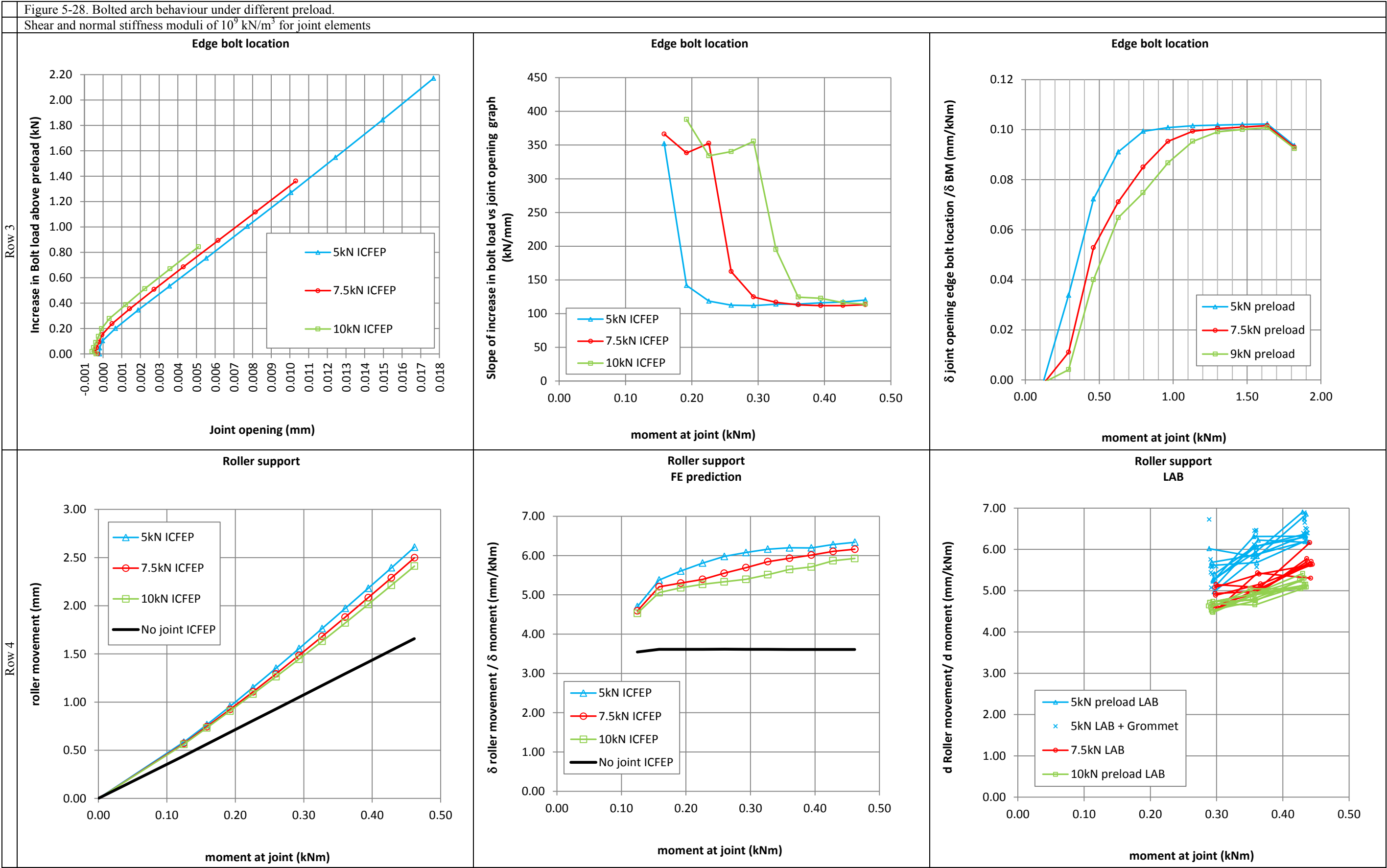


Figure 5-27. FE prediction compared with measured response.





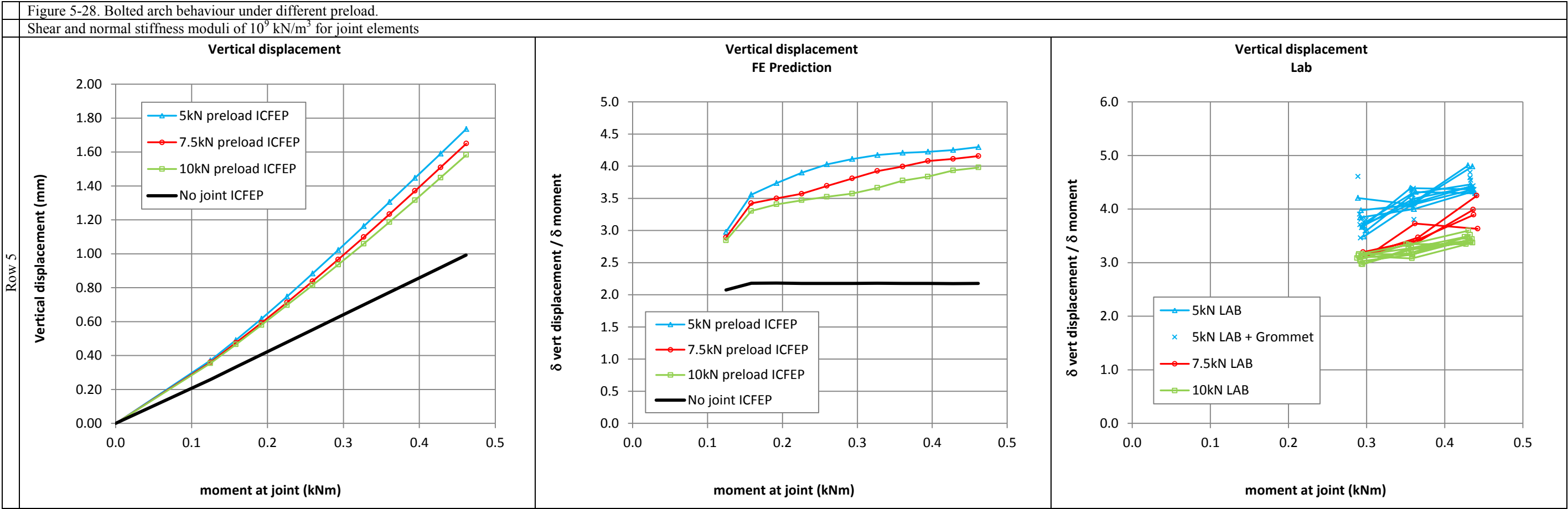


Figure 5-28. Bolted arch behaviour under different preload.

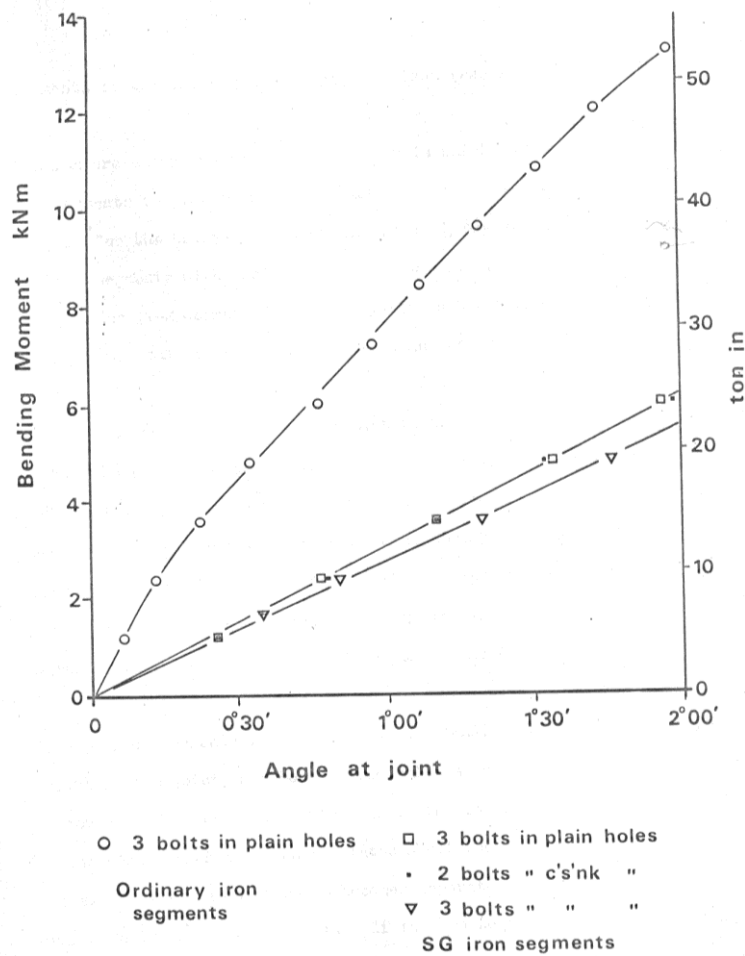


Figure 5-29. Bending moment plotted against measured joint opening (Thomas, 1977).

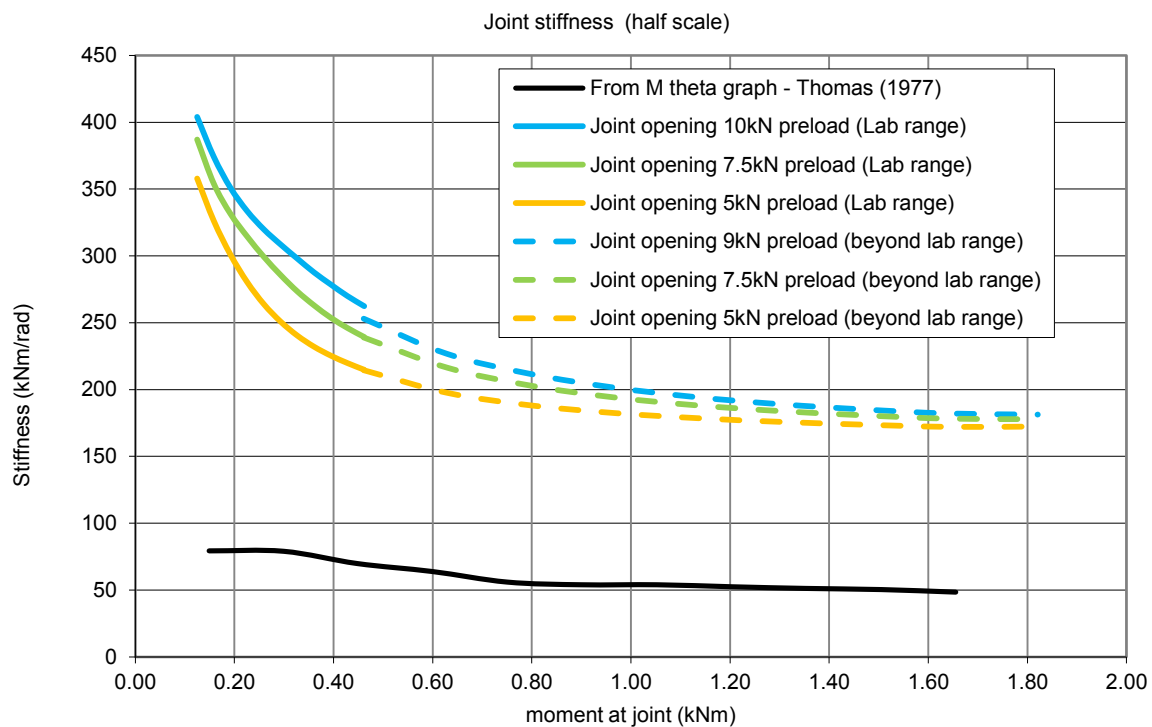


Figure 5-30. Comparison of Thomas' results with ICFEP prediction.

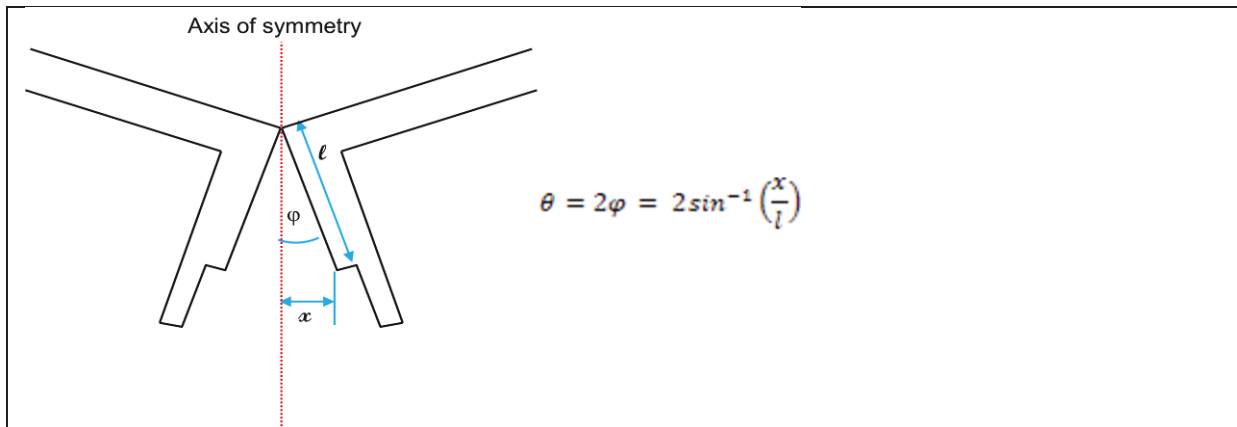


Figure 5-31. Theta 'θ' denotes angular opening at intrados (c.f. Figure 5-22).

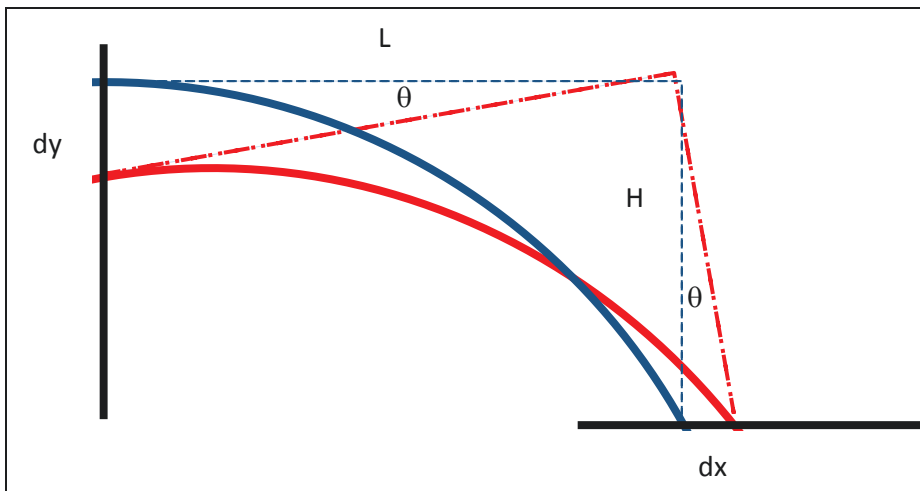


Figure 5-32. Using vertical and horizontal displacements to calculate rotation.

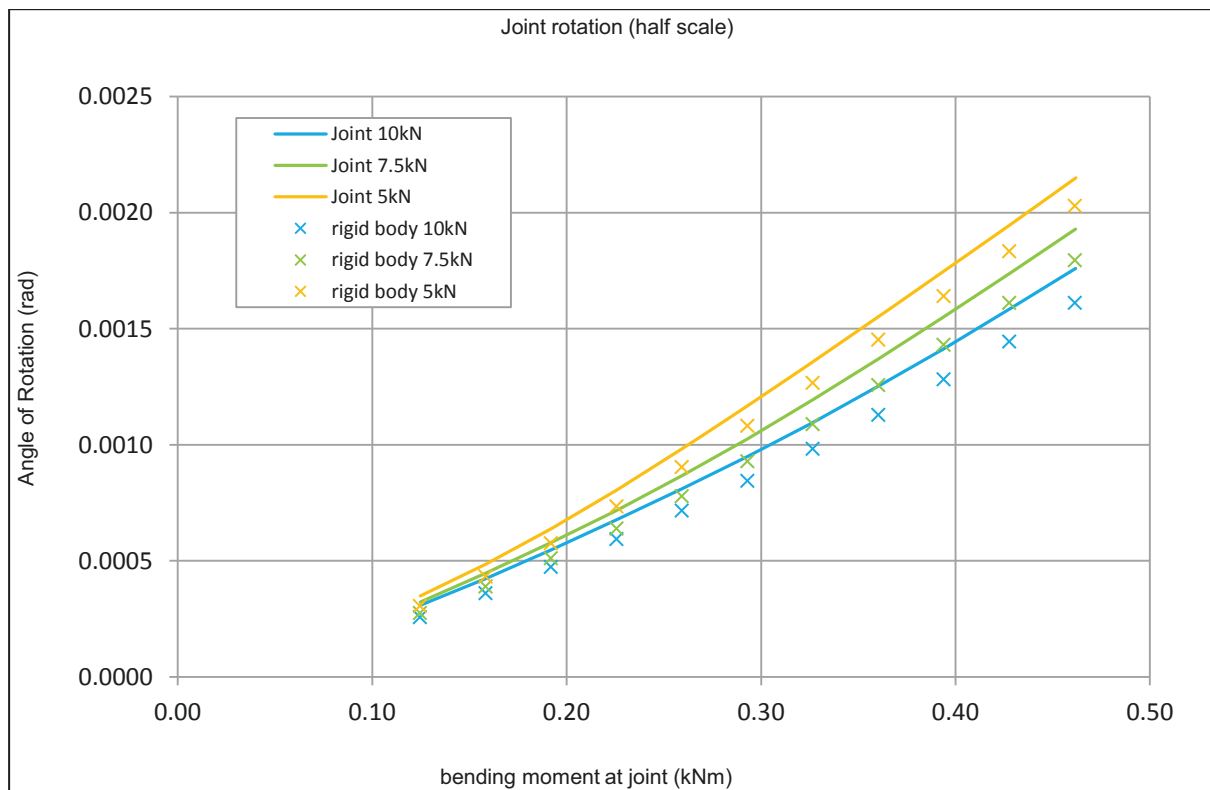


Figure 5-33. Comparison of joint rotation calculated using joint opening and rigid body movement.

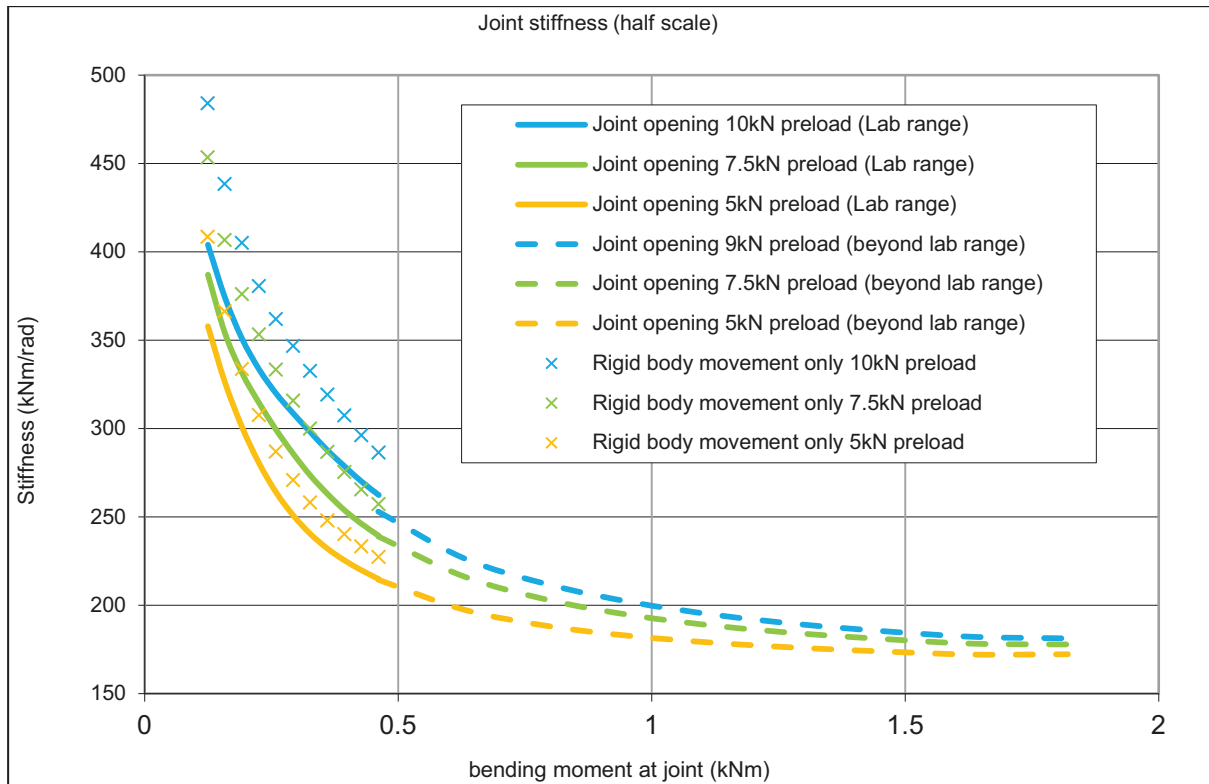


Figure 5-34. Comparison of joint stiffness calculated using joint opening and rigid body movement.

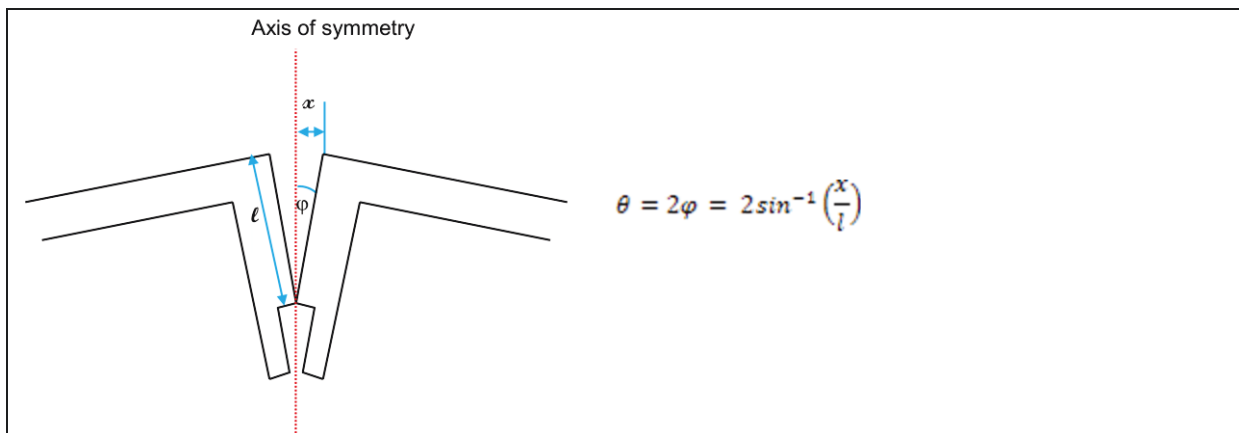


Figure 5-35. Theta 'θ' denotes angular opening at extrados.

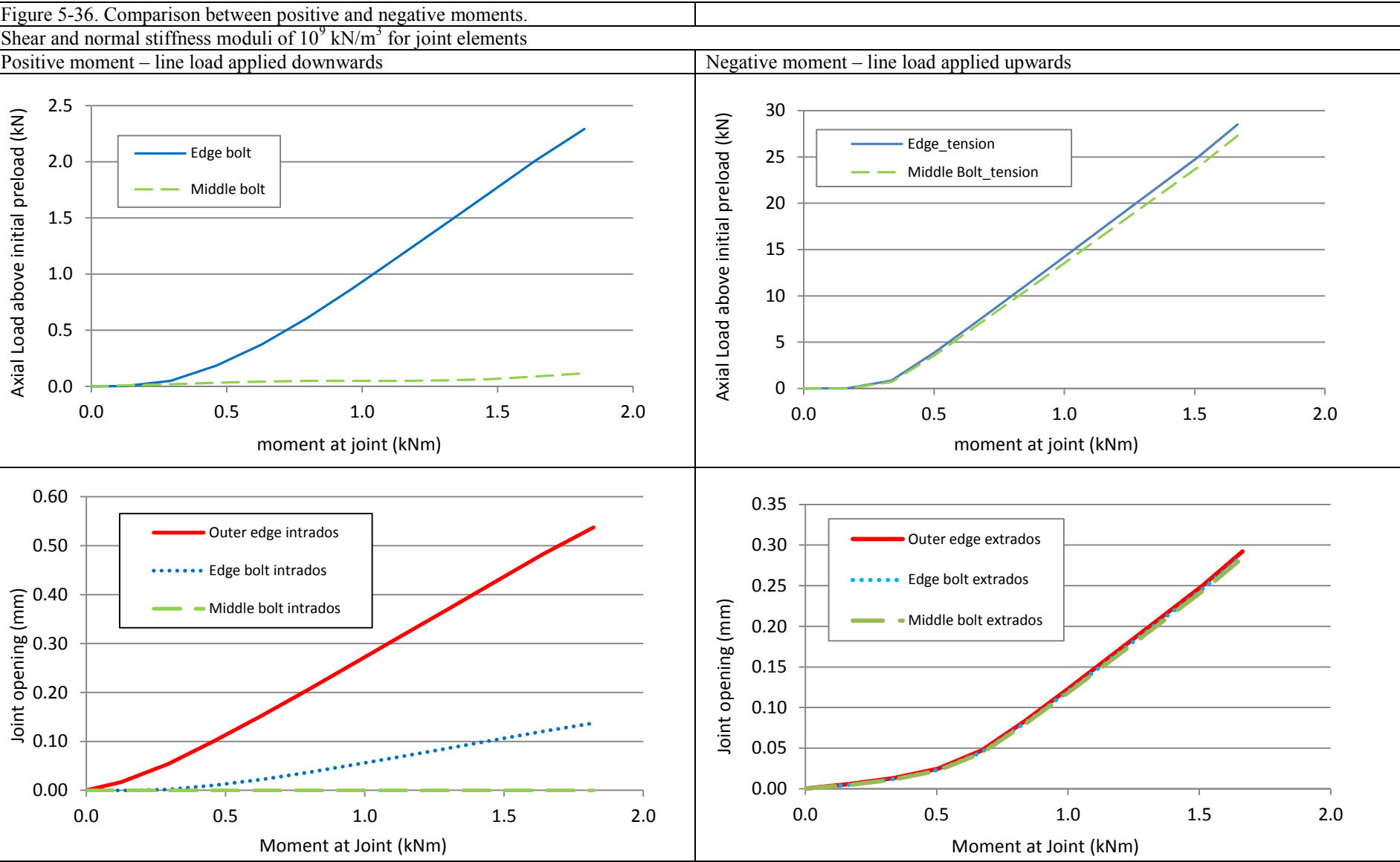
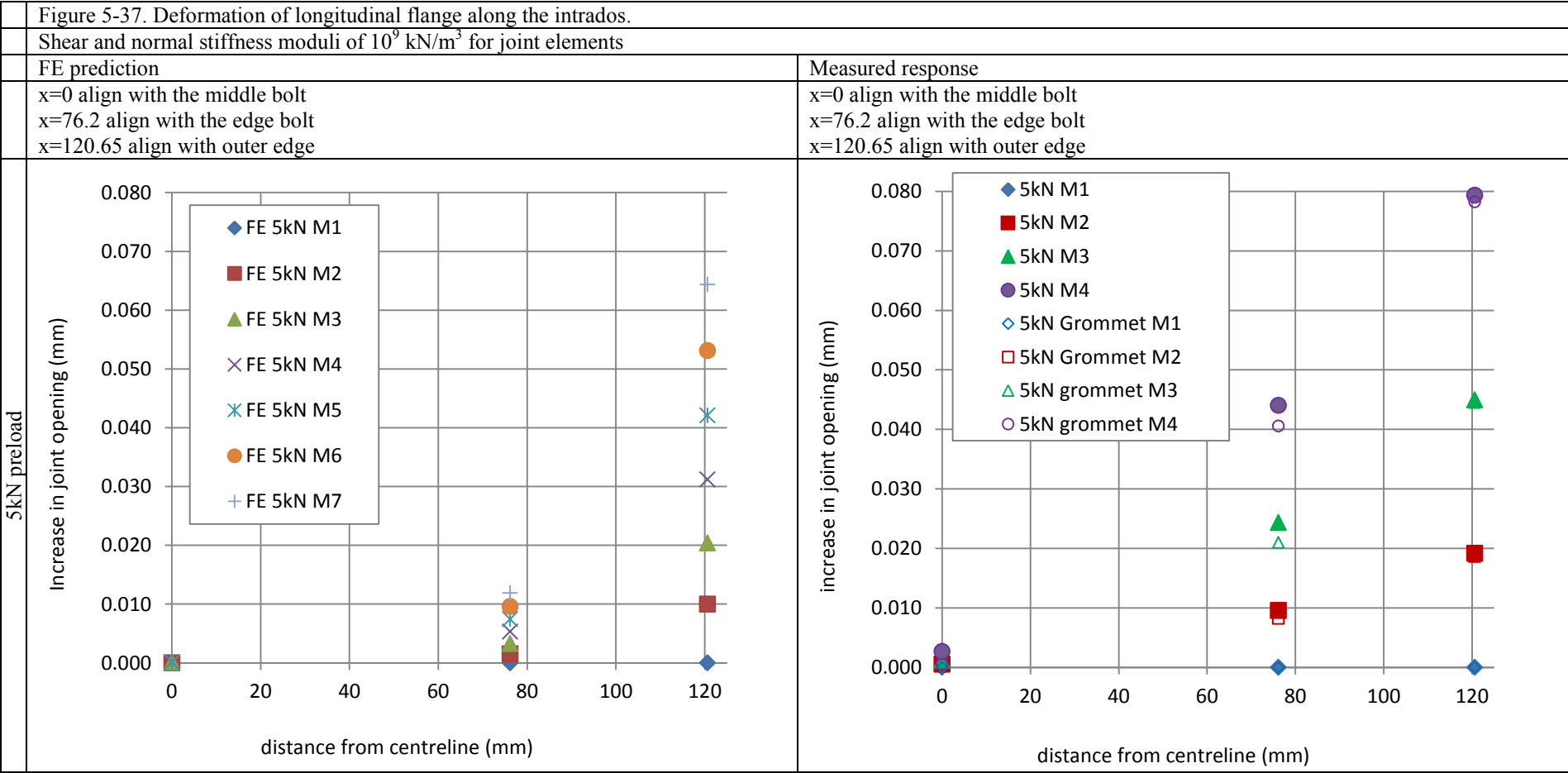
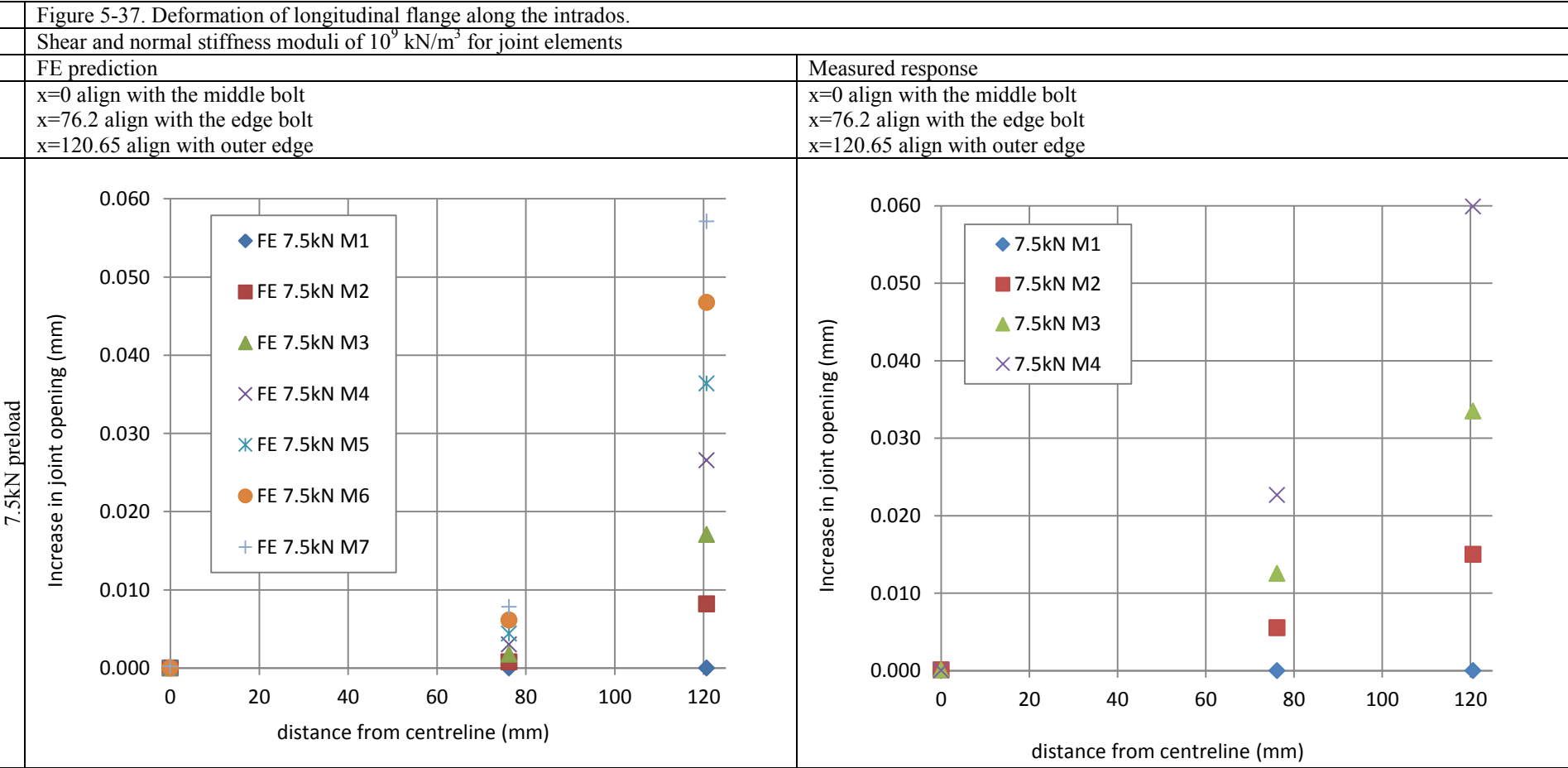


Figure 5-36. Comparison between positive and negative moments.





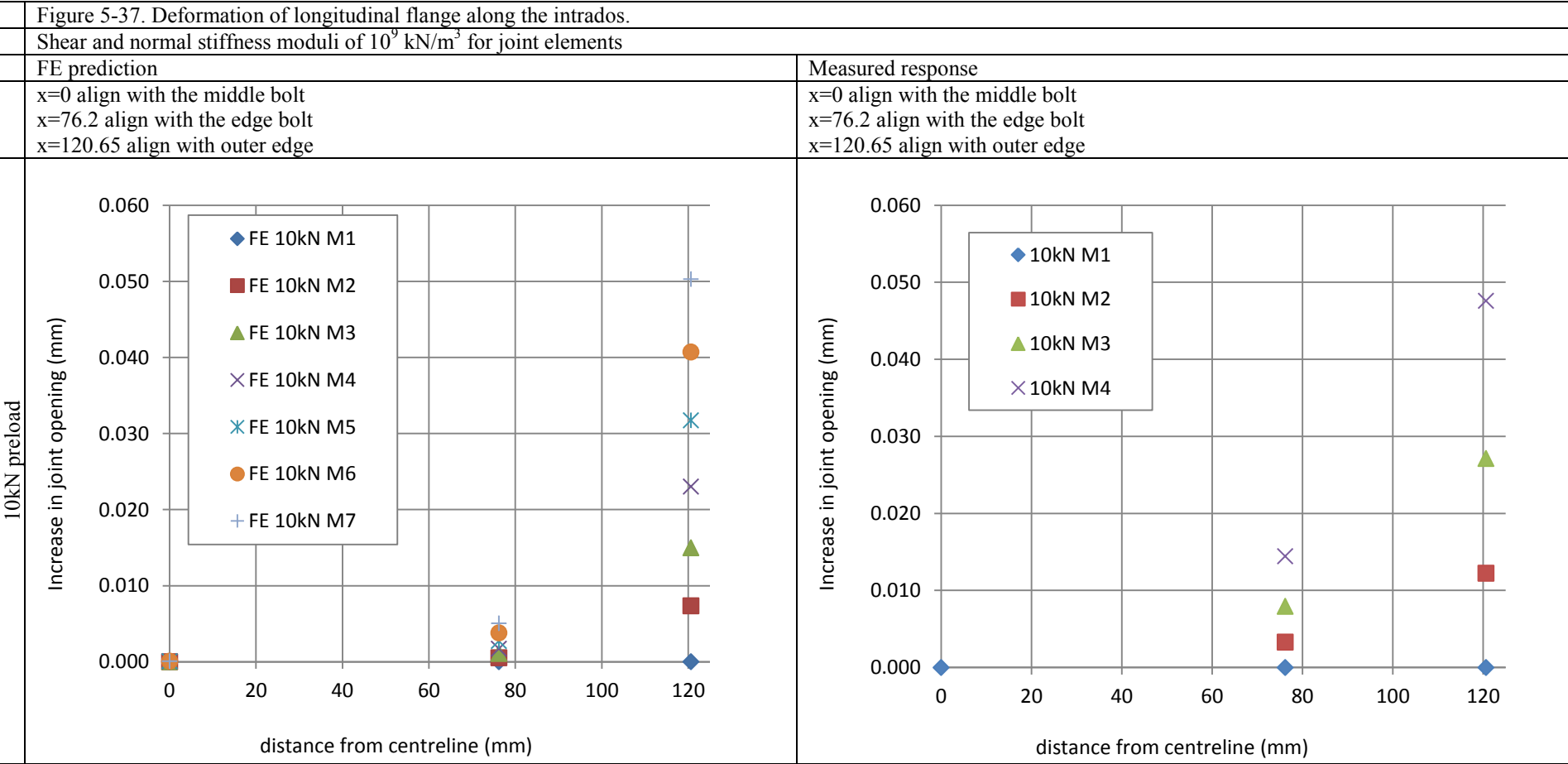


Figure 5-37. Deformation of longitudinal flange along the intrados.

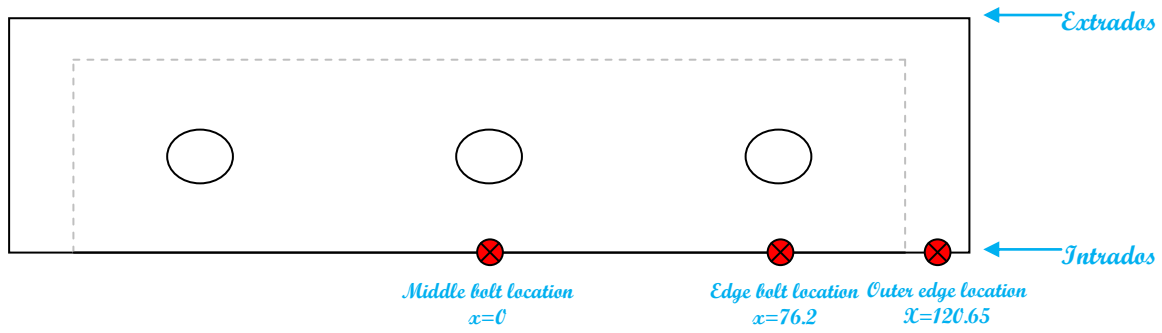


Figure 5-38. Horizontal flange showing locations of comparison on the intrados.

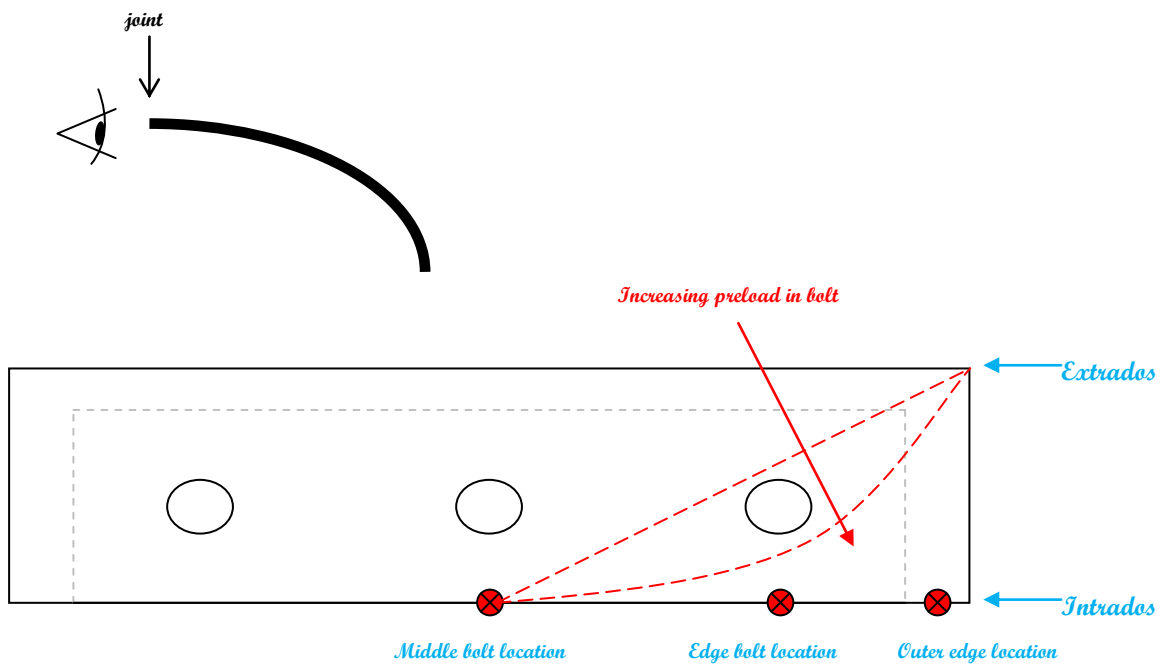
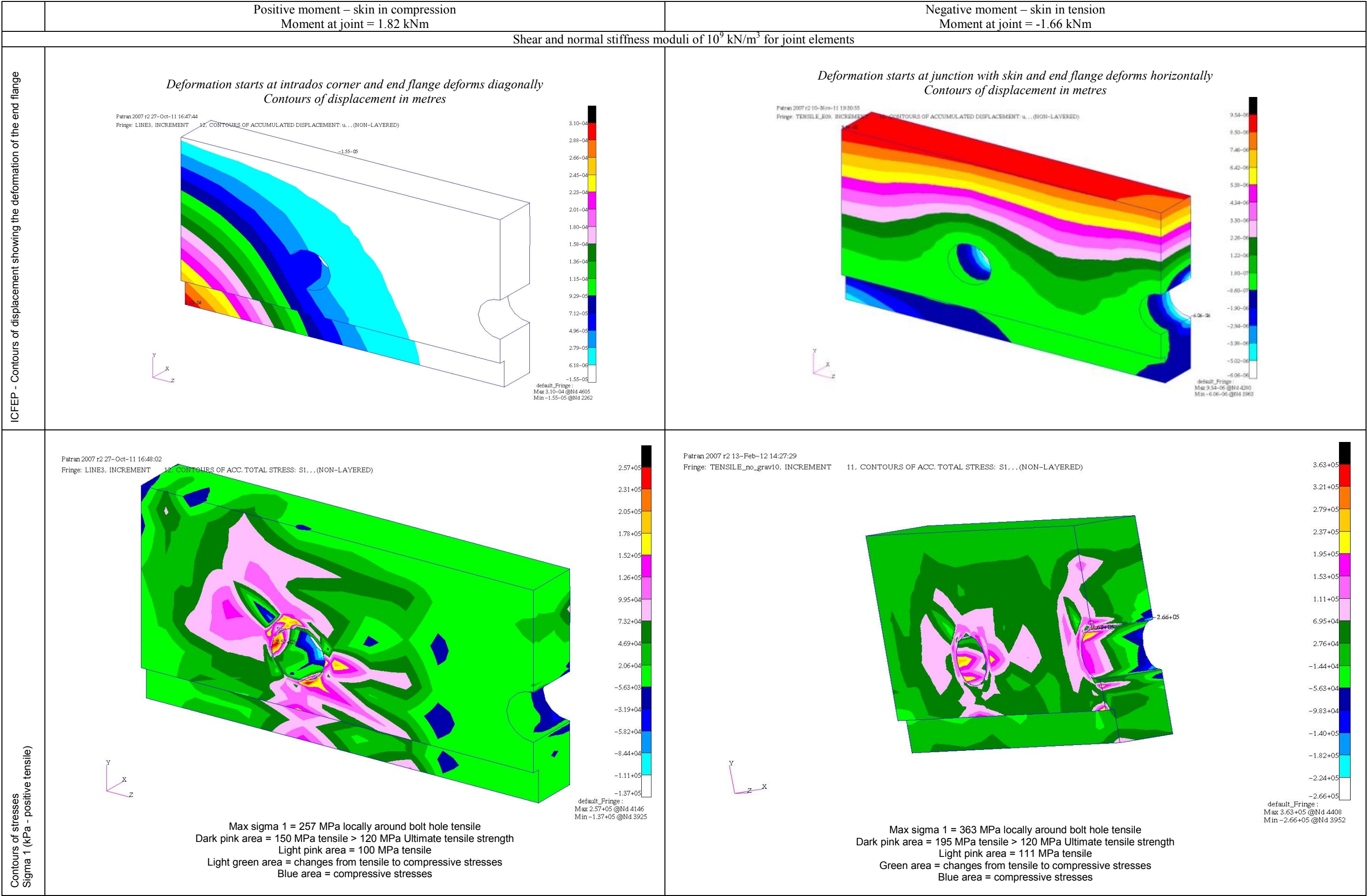


Figure 5-39. Inferred bending mode in horizontal flange: the red dashed lines indicate where the flange bends into the page.



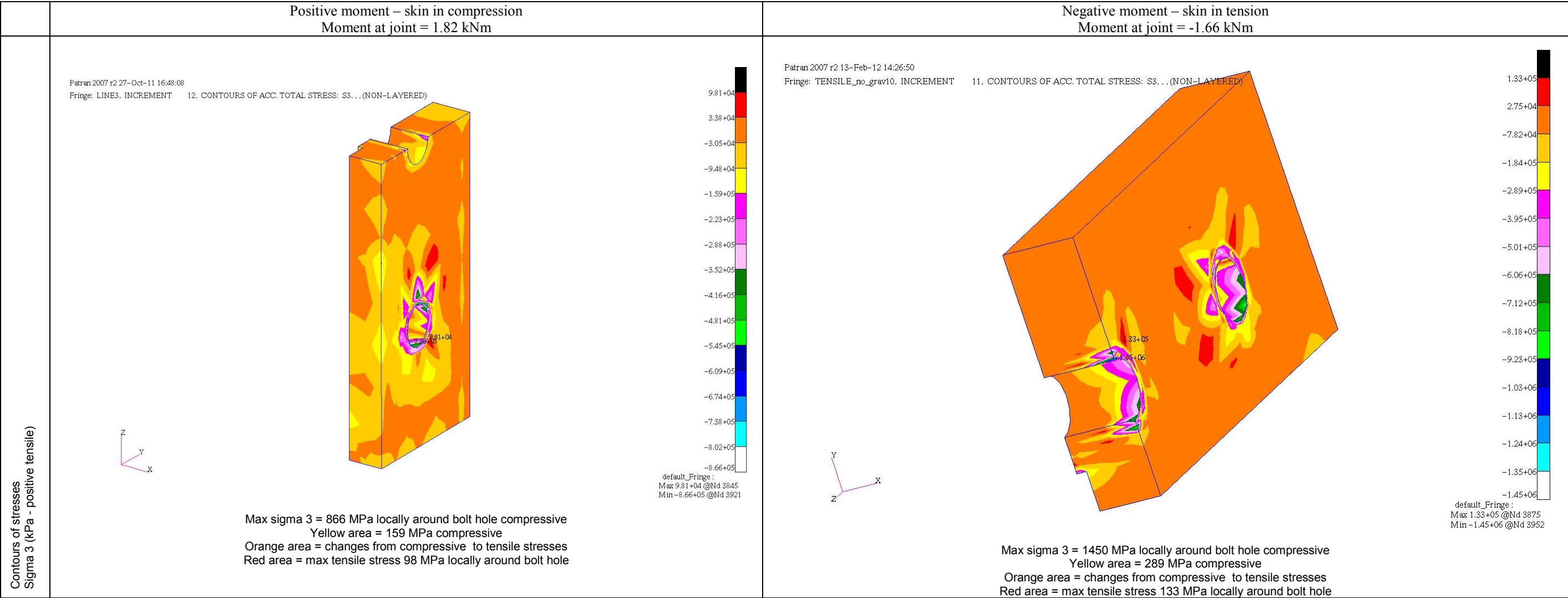


Figure 5-40. Flange deformation and maximum tensile and compressive stresses around bolt hole on the longitudinal flange as predicted by FEA.

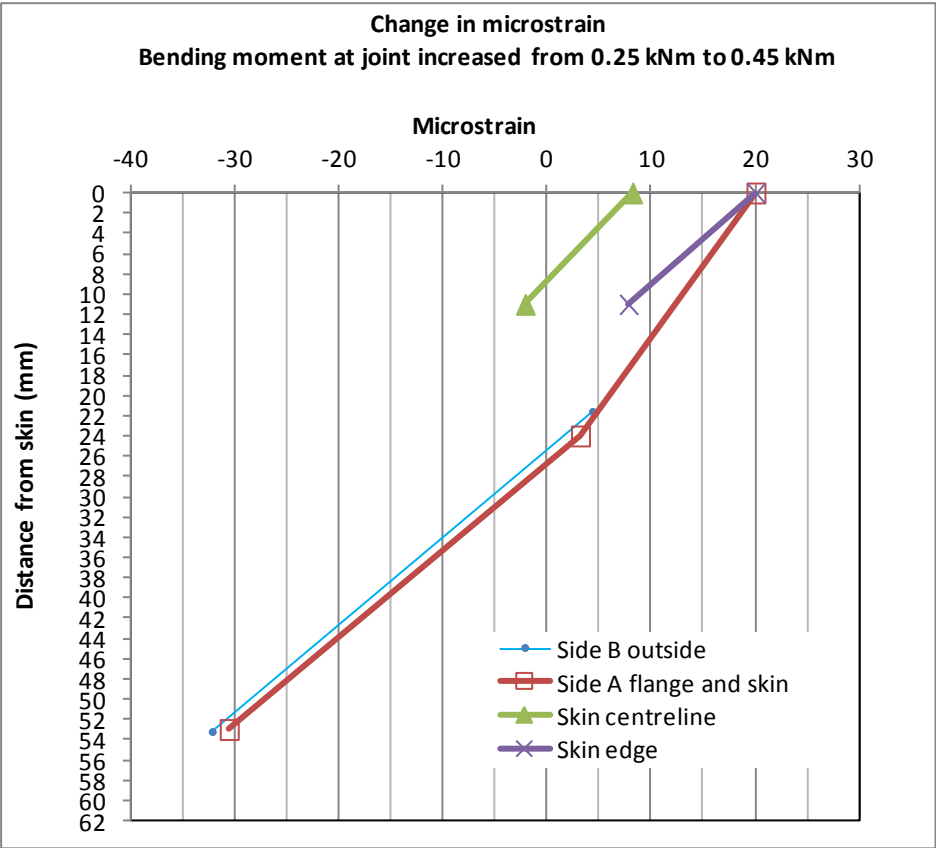


Figure 5-41. Measurements from t-gauges - change in microstrain.

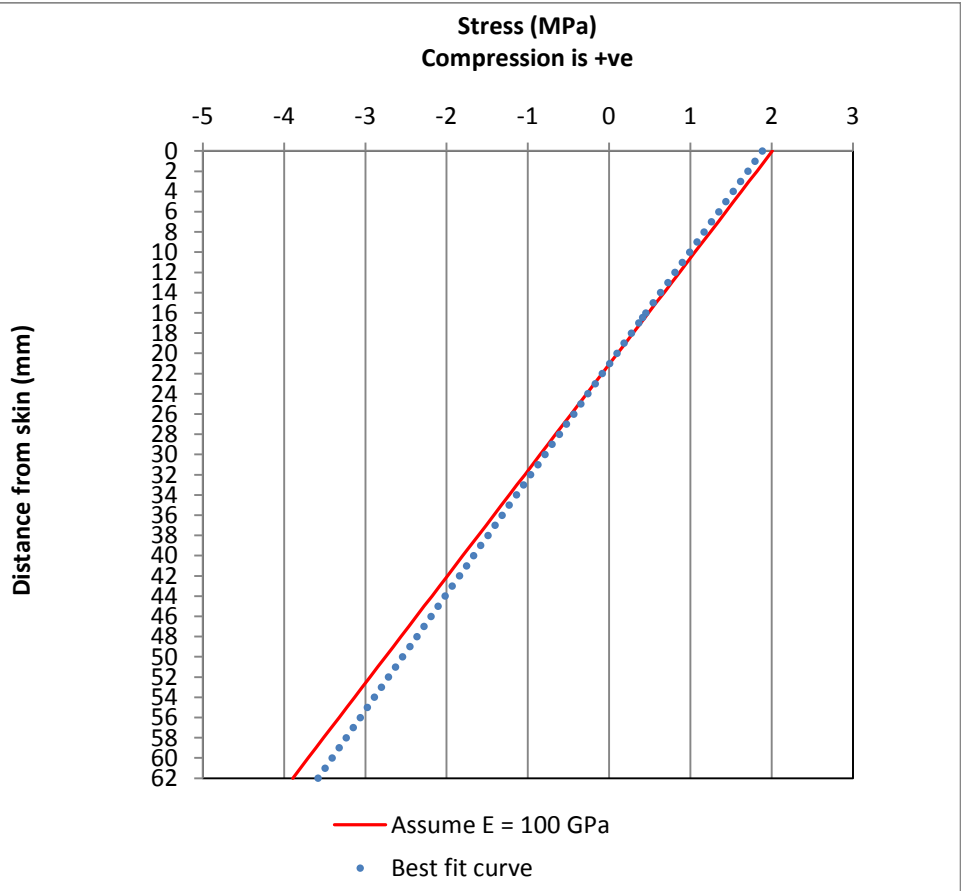
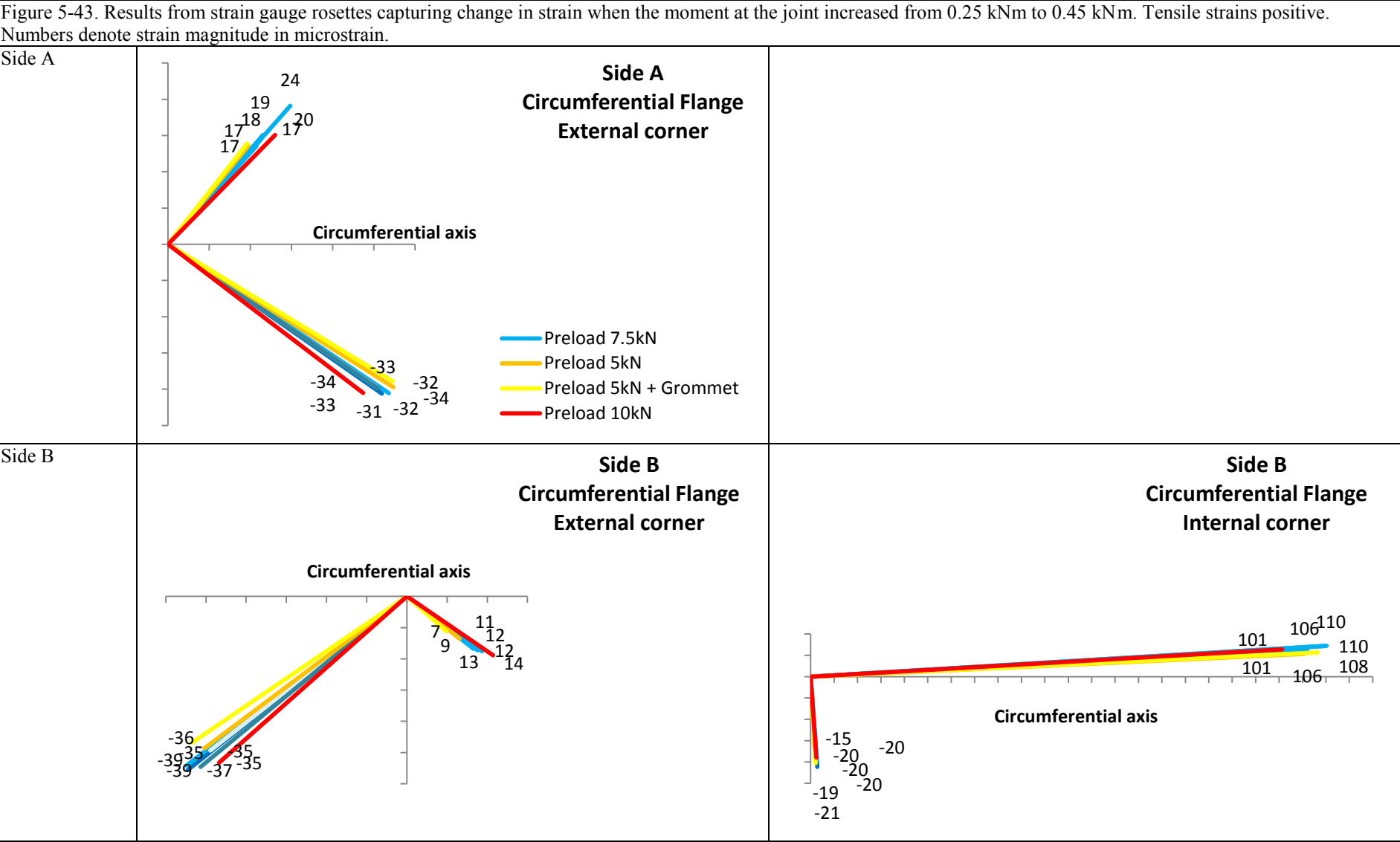


Figure 5-42. Stress estimated using E=100 GPa and best fit stress-strain curve.



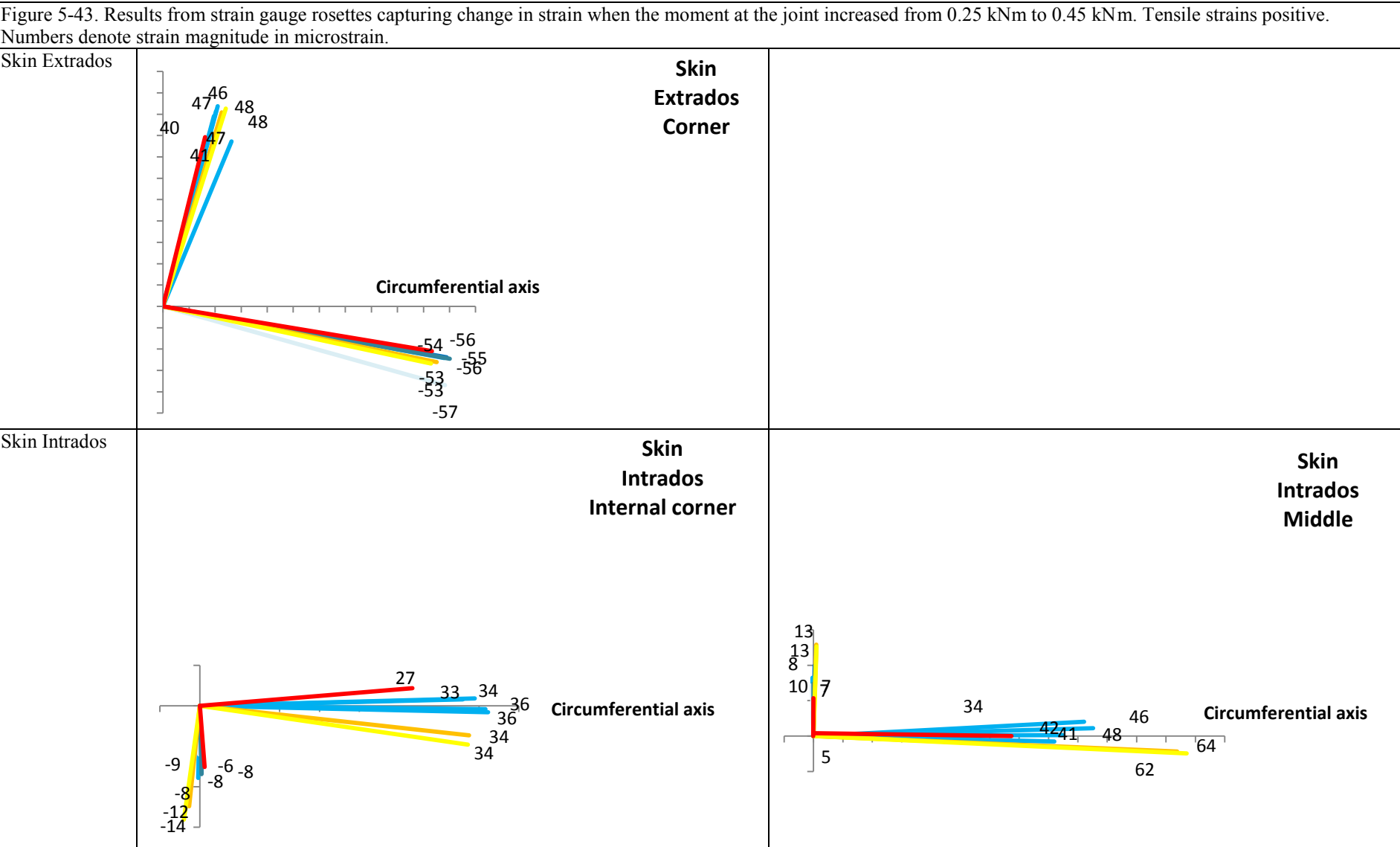


Figure 5-43. Results from strain gauge rosettes capturing change in strain when the moment at the joint increased from 0.25 kNm to 0.45 kNm. Tensile strains positive. Numbers denote strain magnitude in microstrain.

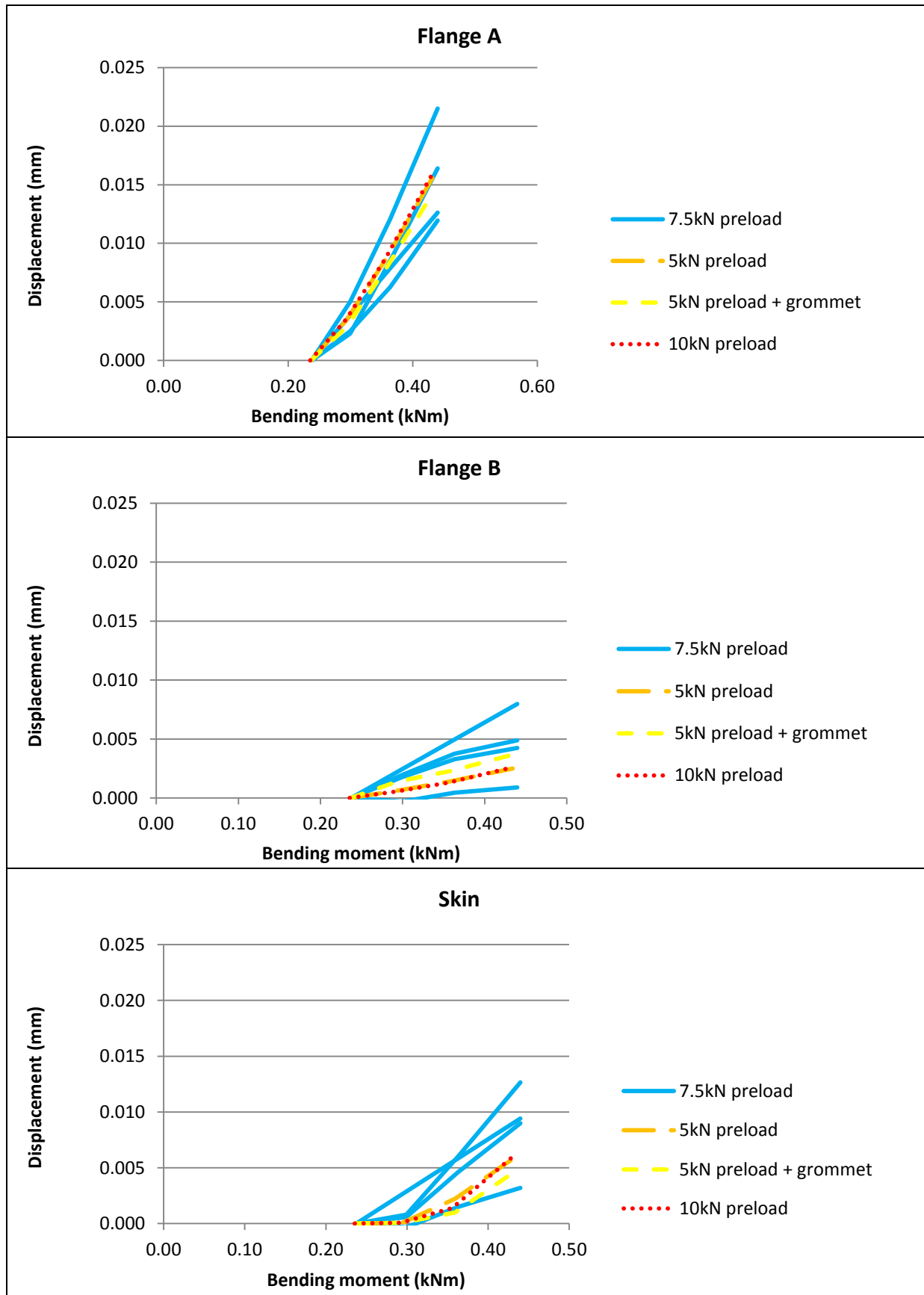


Figure 5-44. Distortion under positive loading.

6 Development of full-ring setup and planning of tests

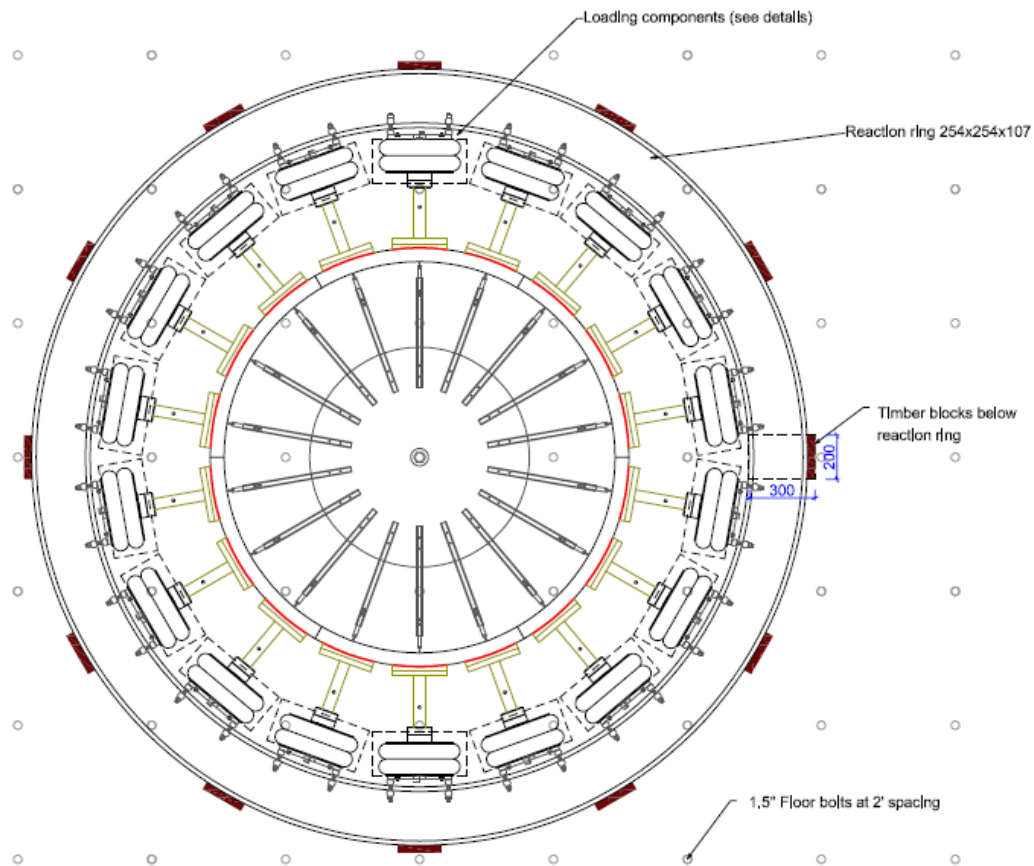


Figure 6-1. Schematic drawing of full-ring setup.

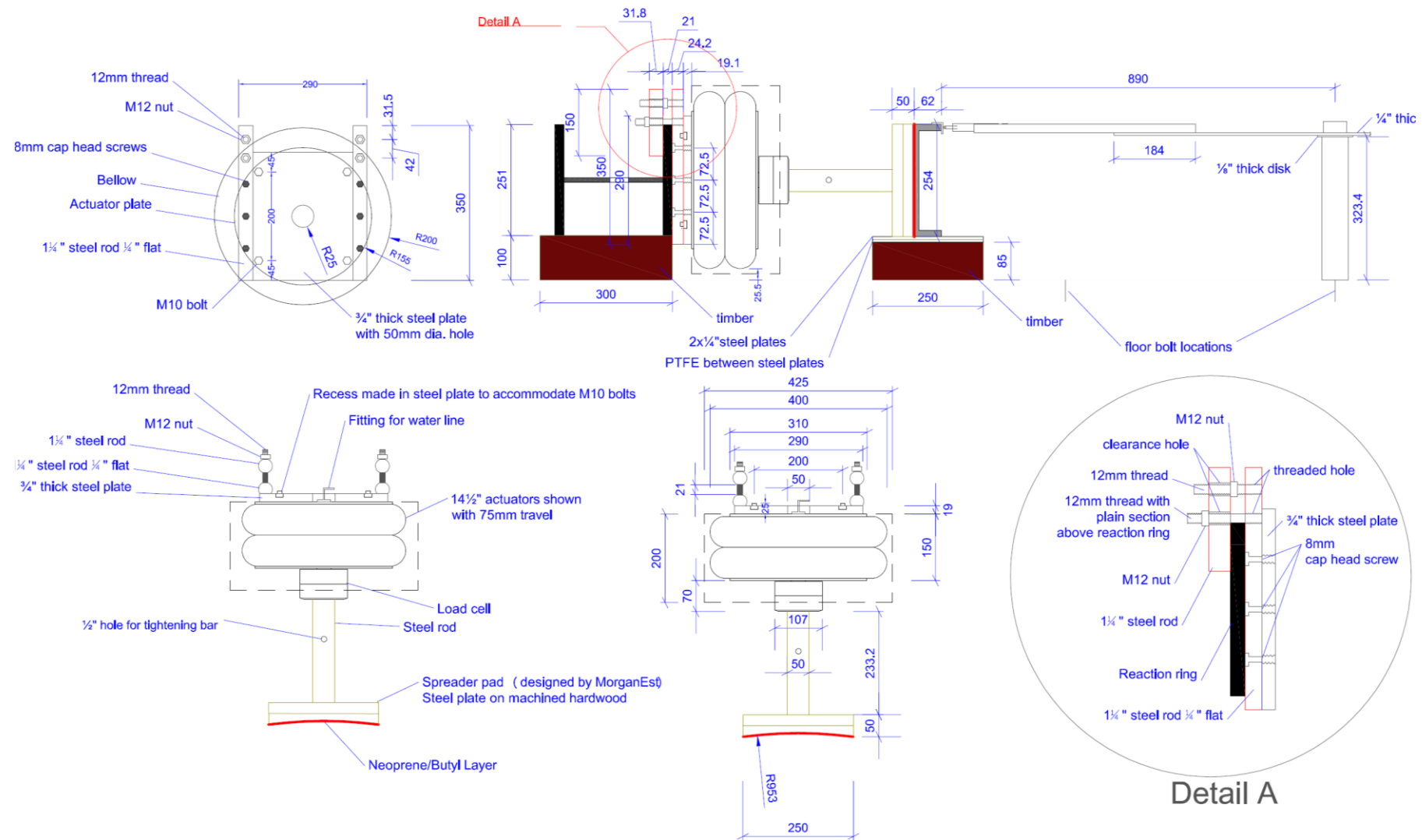


Figure 6-2. Details of loading rig components.



Figure 6-3. Actual full-ring test setup. Locations where the actuators are removed are circled in red.

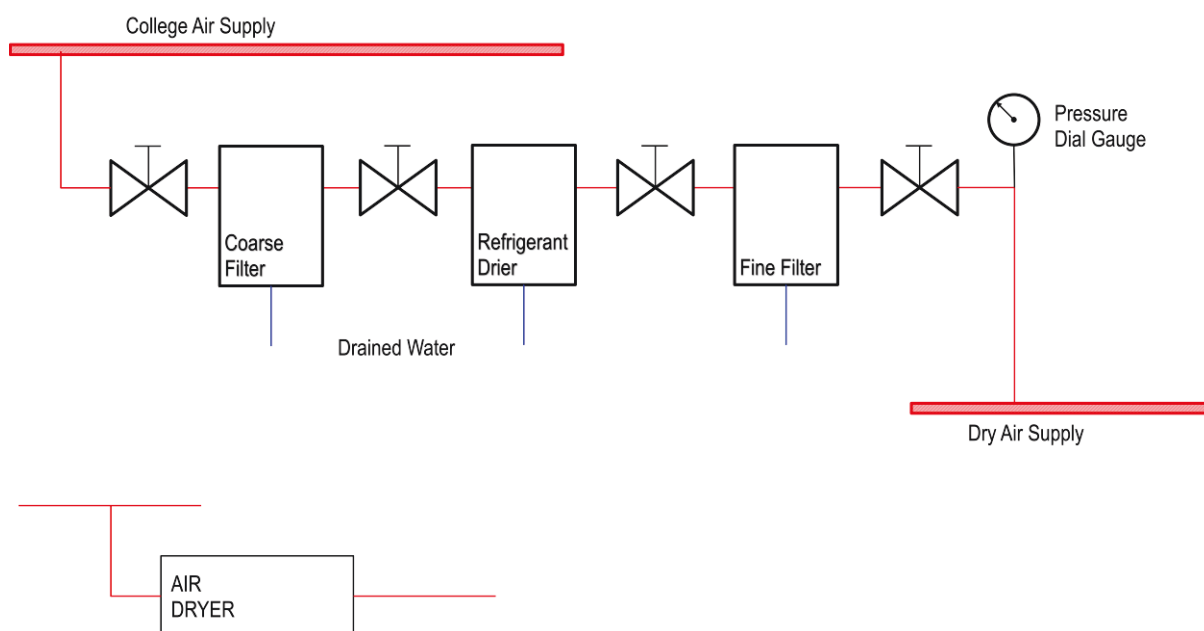


Figure 6-4. Air drying system.

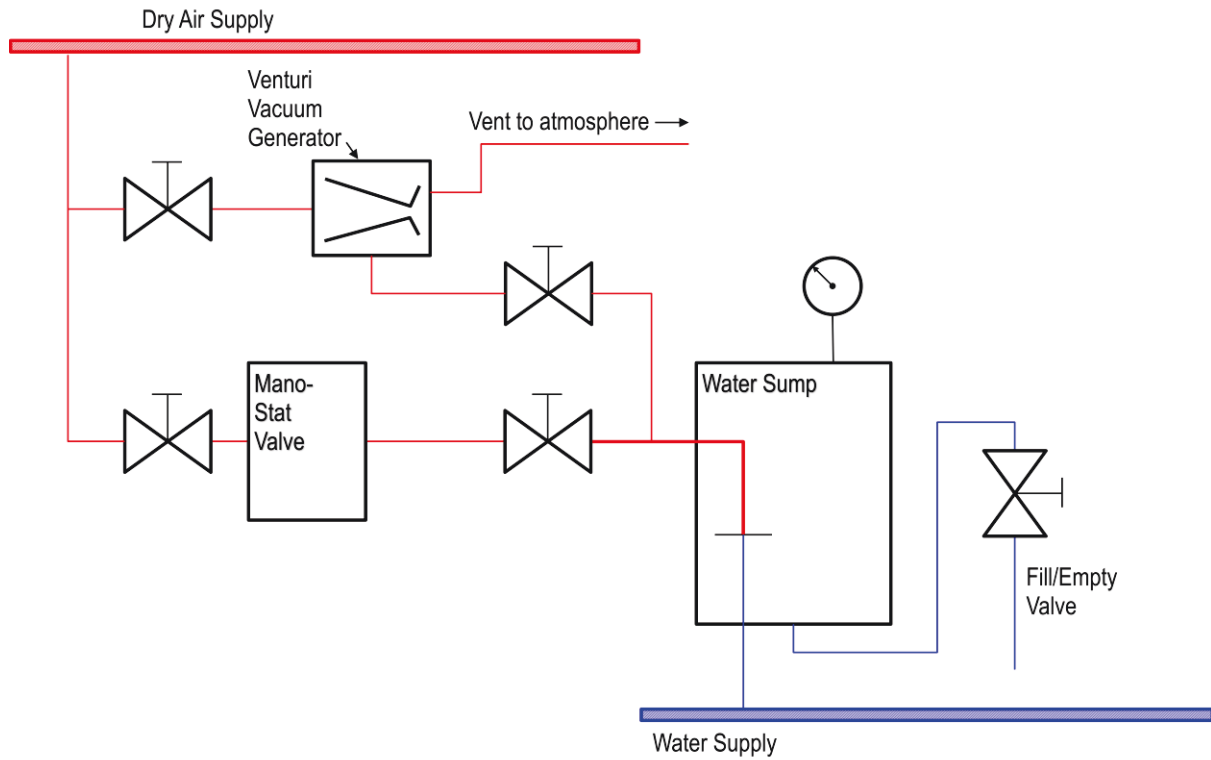


Figure 6-5. Pressurised water supply for air/water interface.

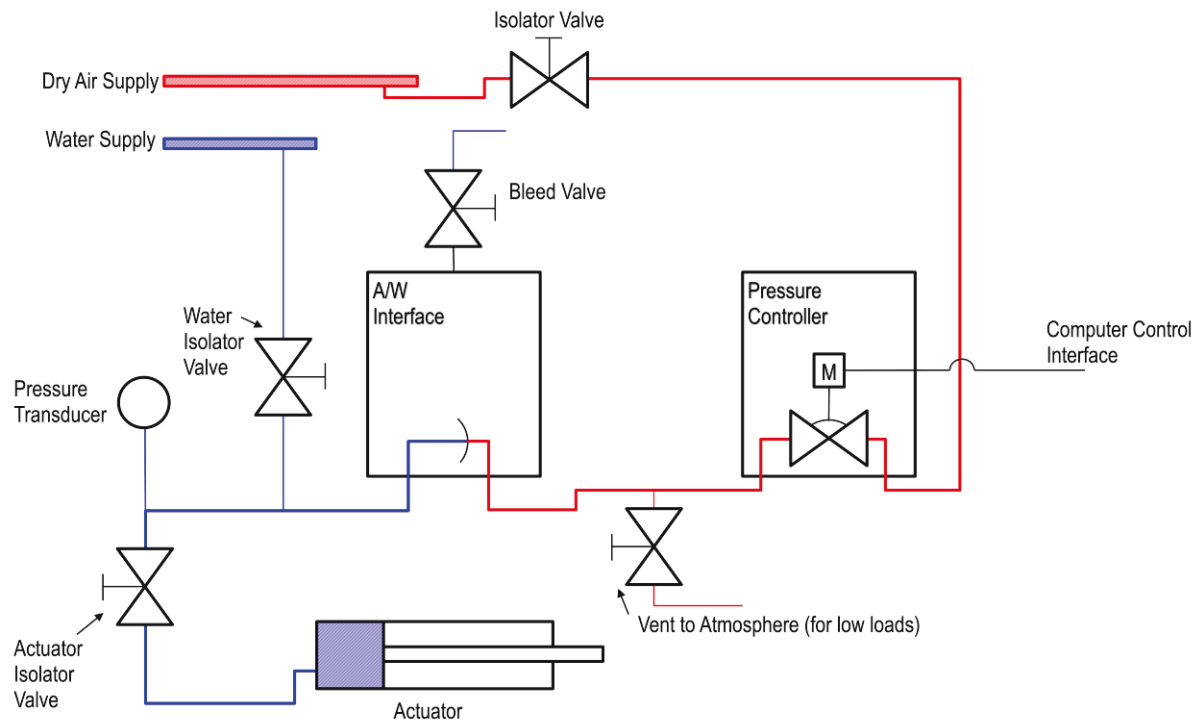


Figure 6-6. Air/water interface.



Figure 6-7. Tangential stabilising rod.

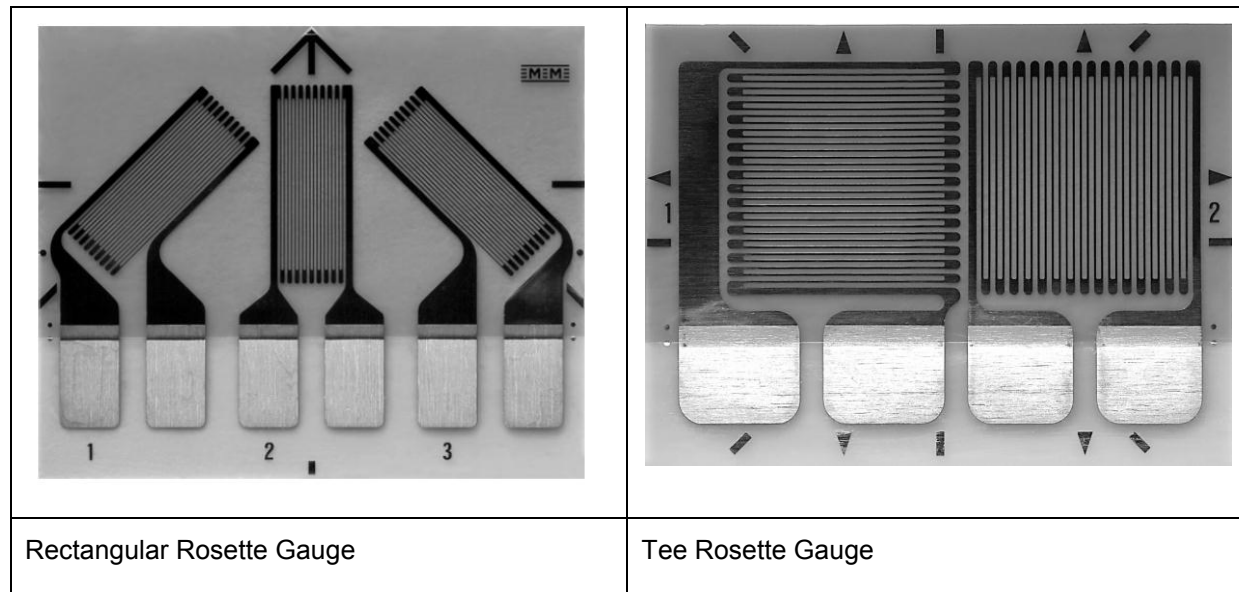


Figure 6-8. Magnified images of strain gauges.

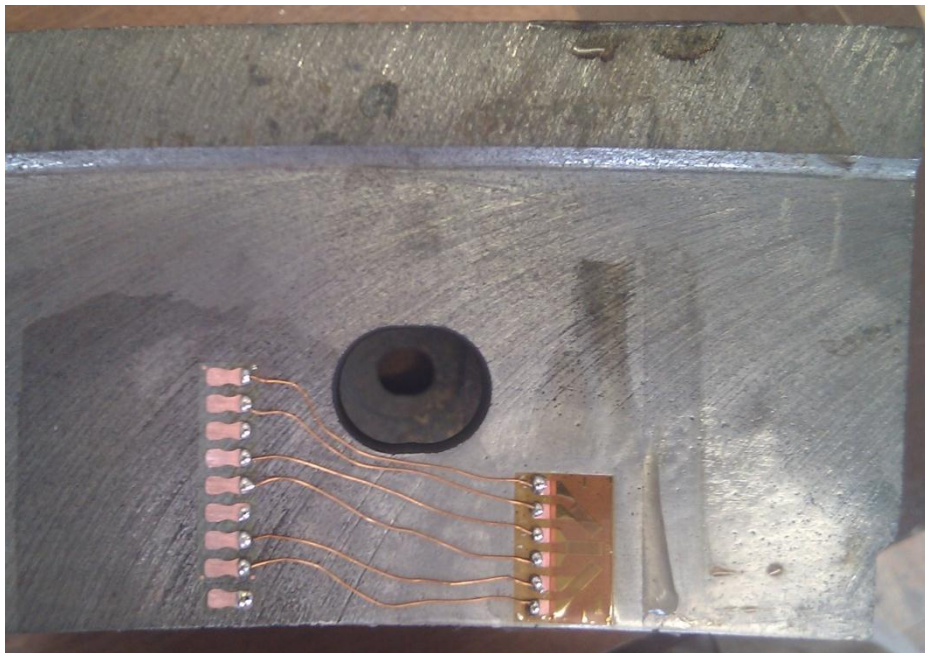


Figure 6-9. Strain relief wires connecting the rosette to terminals.

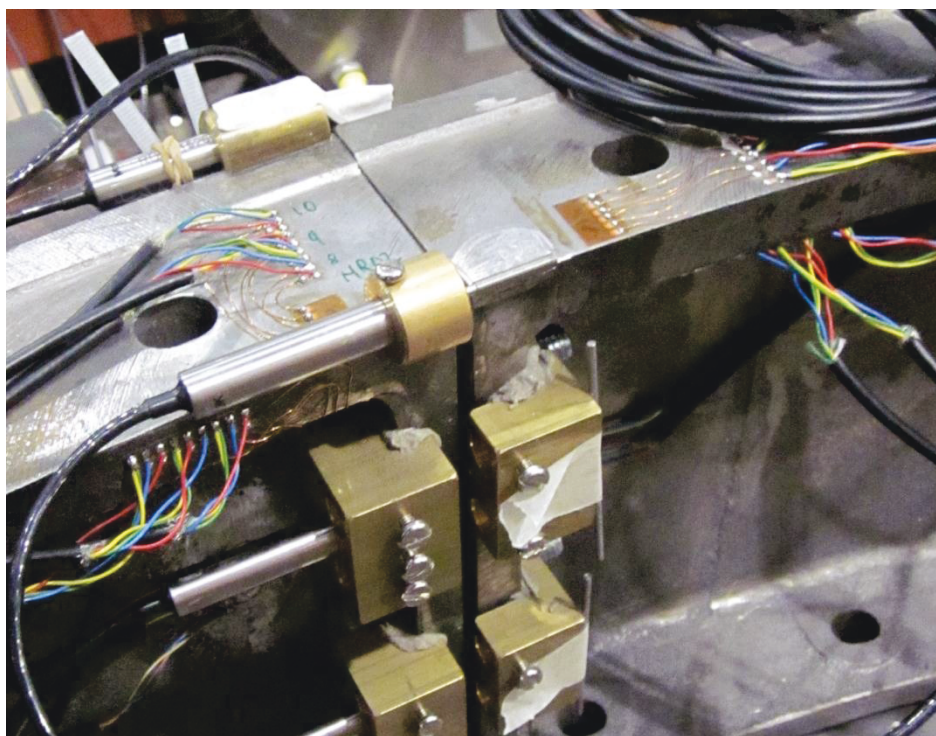


Figure 6-10. Coloured leadwires attached to terminals.

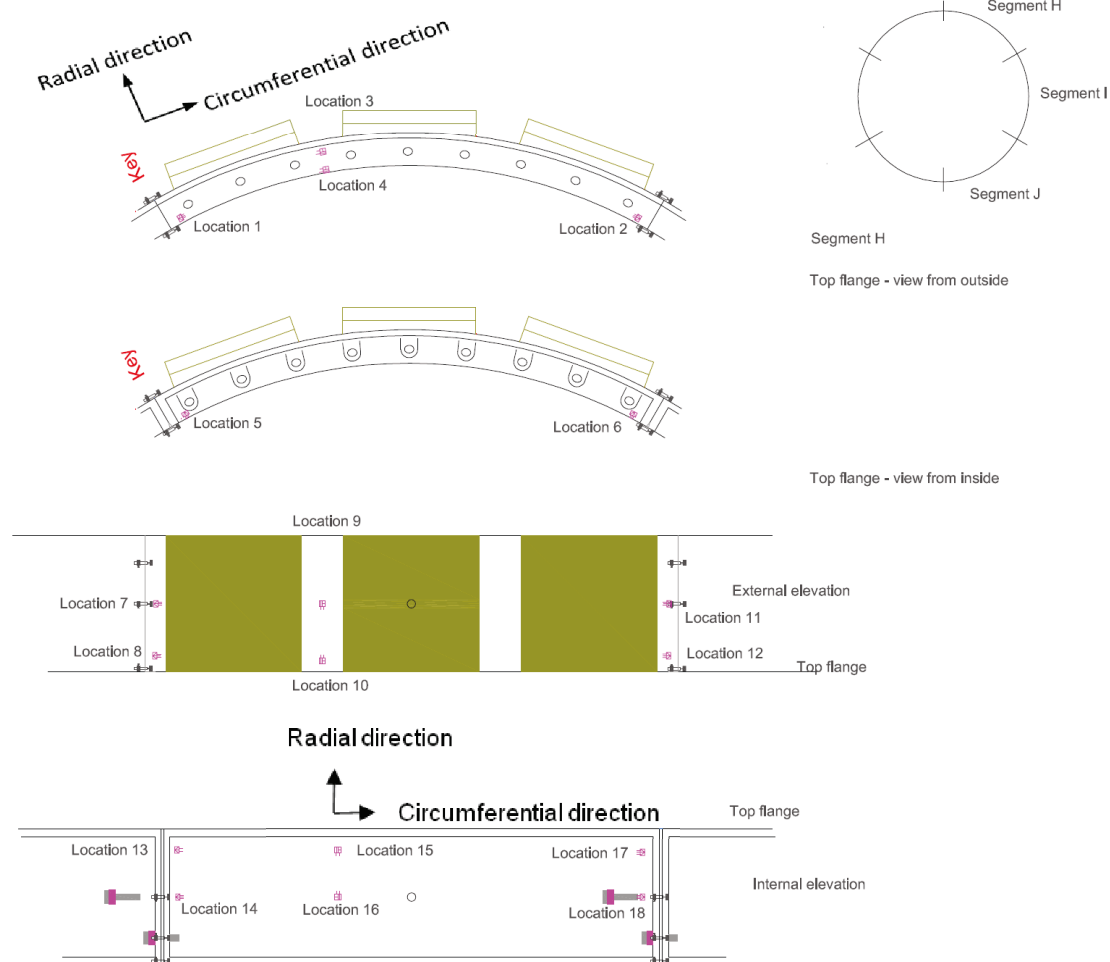


Figure 6-11. Strain gauge locations for segment H.

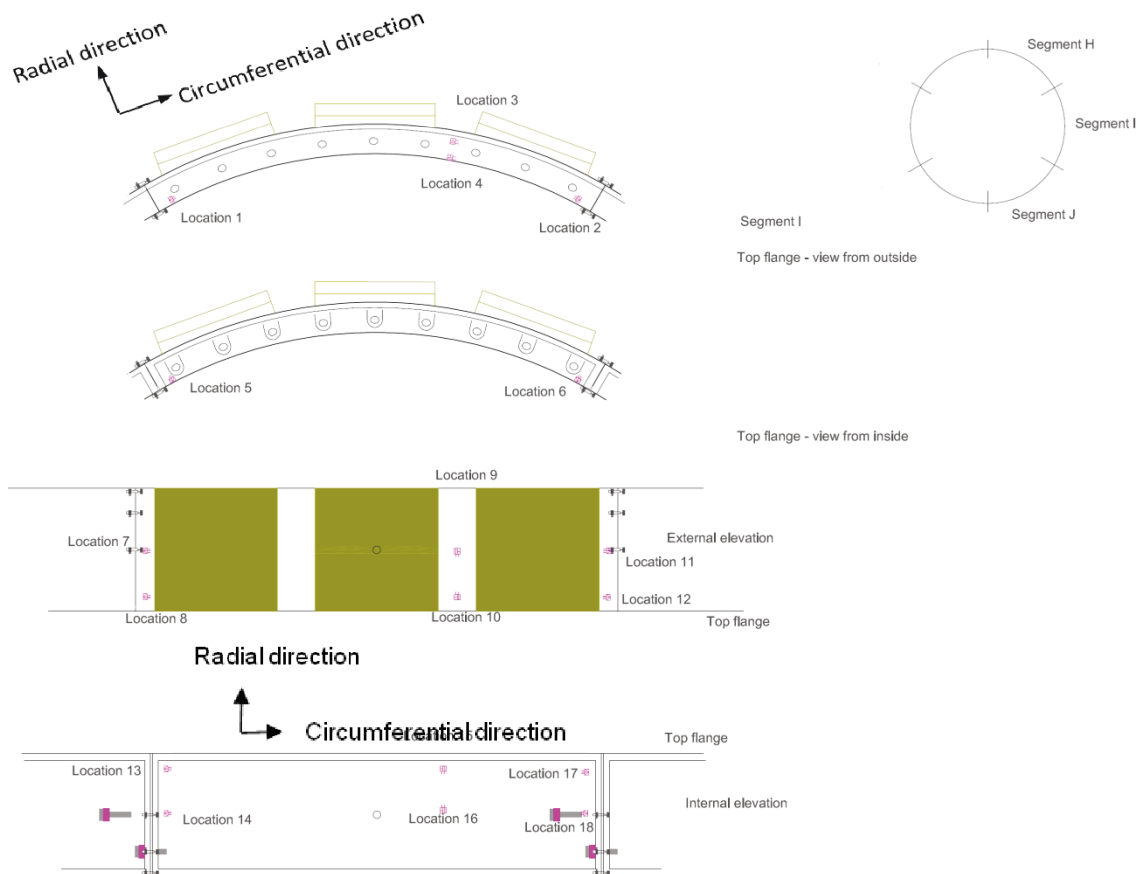


Figure 6-12. Strain gauge locations for segment I.

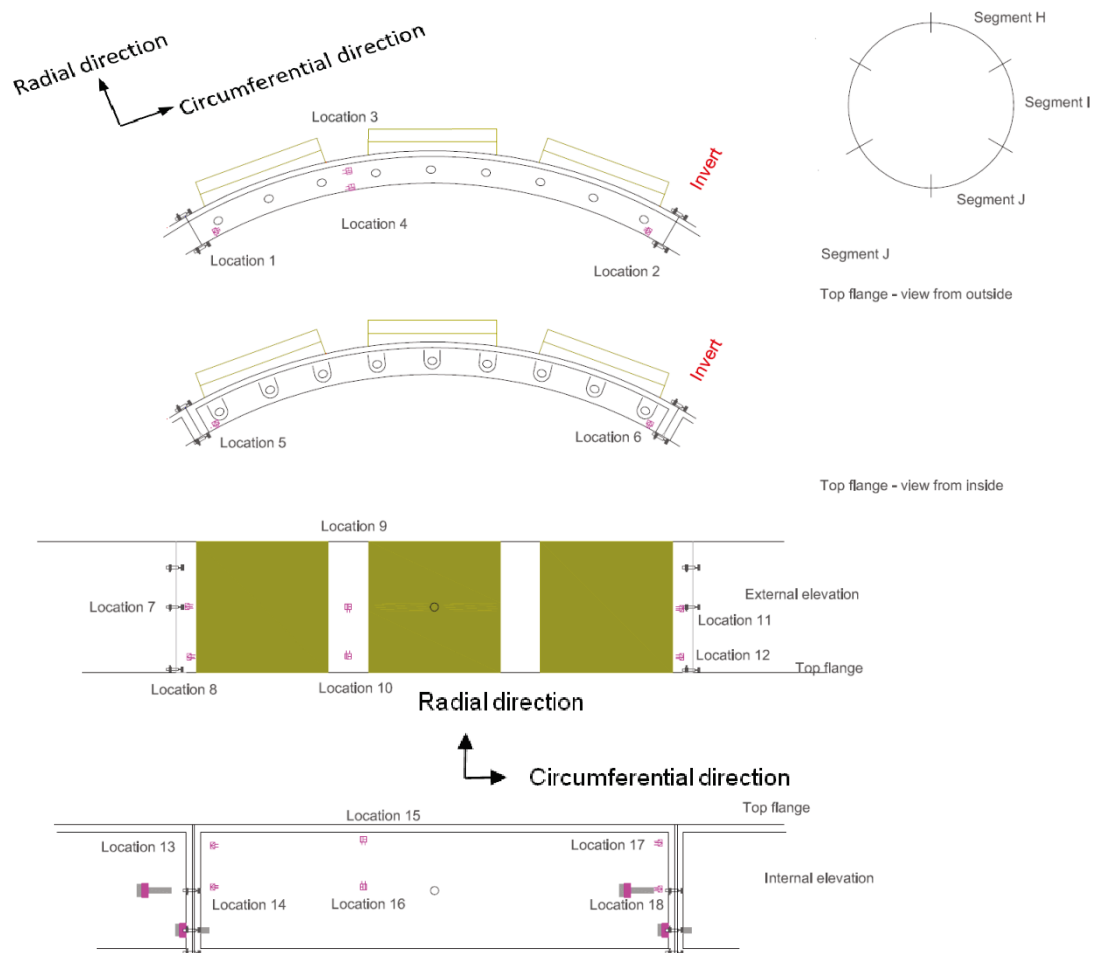


Figure 6-13. Strain gauge locations for segment J.

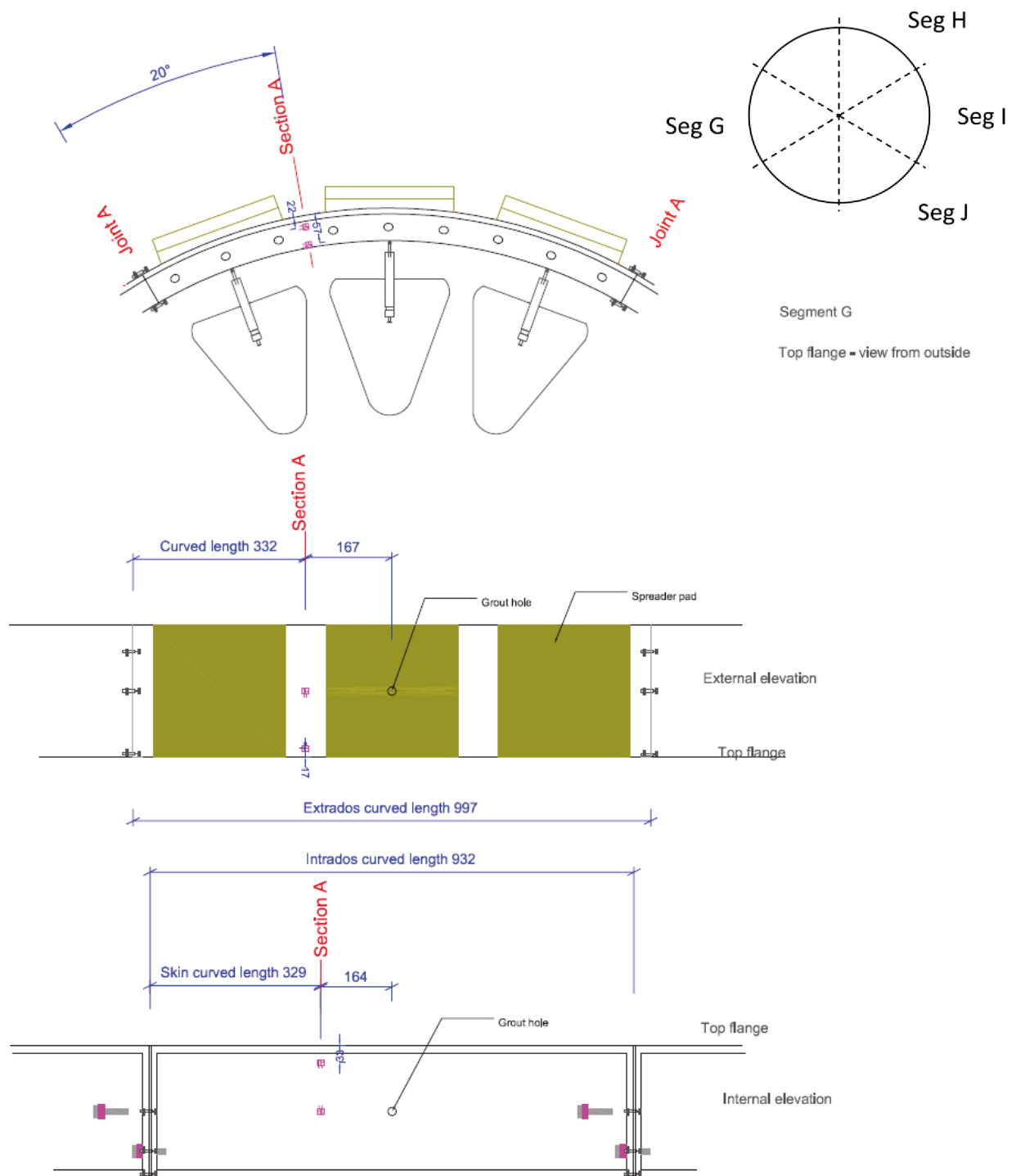


Figure 6-14. Strain gauge locations for segment G.



Figure 6-15. Strain gauged displacement transducer.



Figure 6-16. The inside of a strain gauged displacement transducer, with hand holding onto plunger rod.

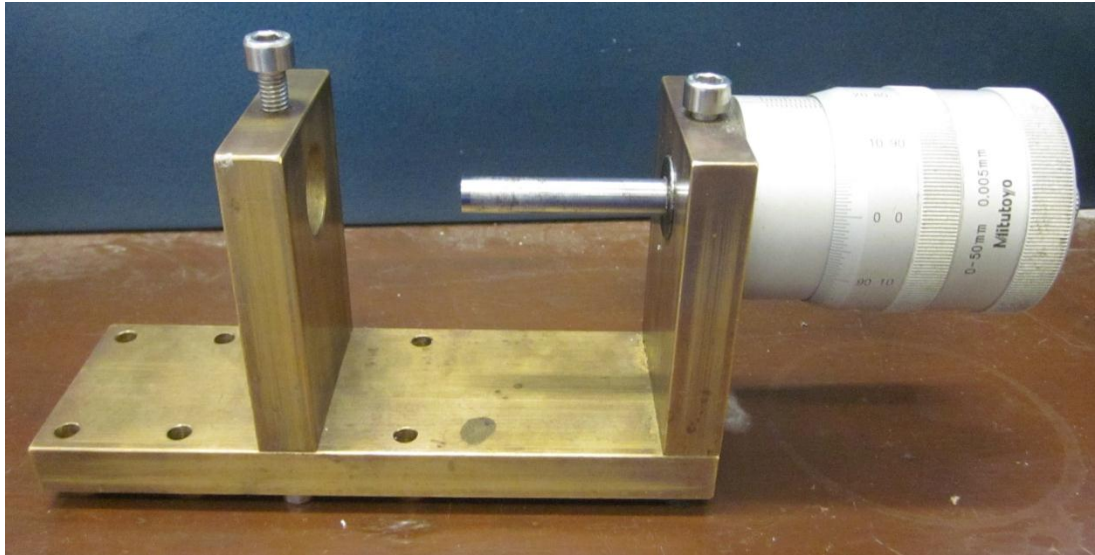


Figure 6-17. Micrometer for calibrating strain gauged displacement transducers.



Figure 6-18. Strain gauged displacement transducers placed around the test ring.

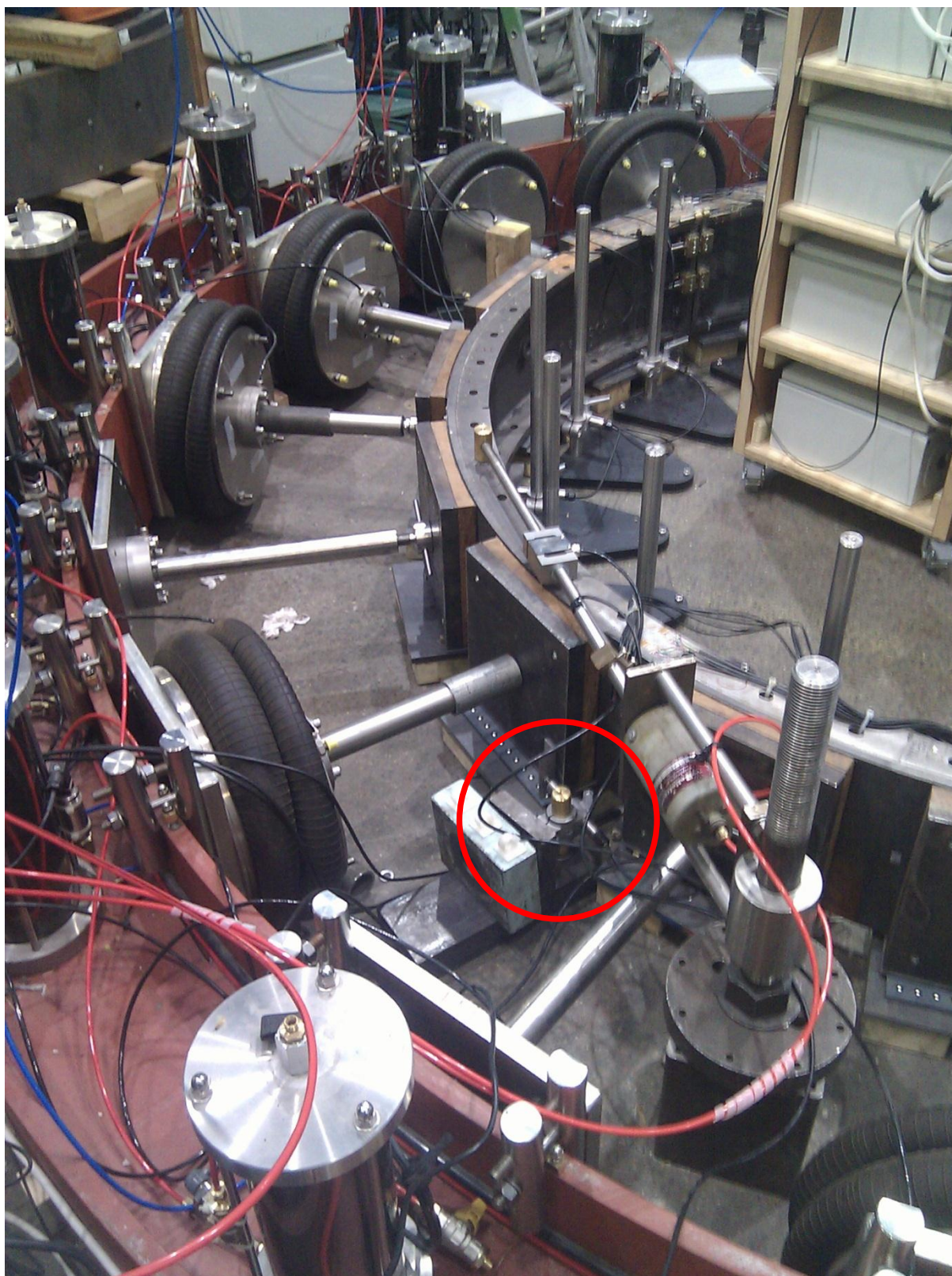


Figure 6-19. Strain gauged displacement transducer (circled in red) for detecting rotation of the ring.

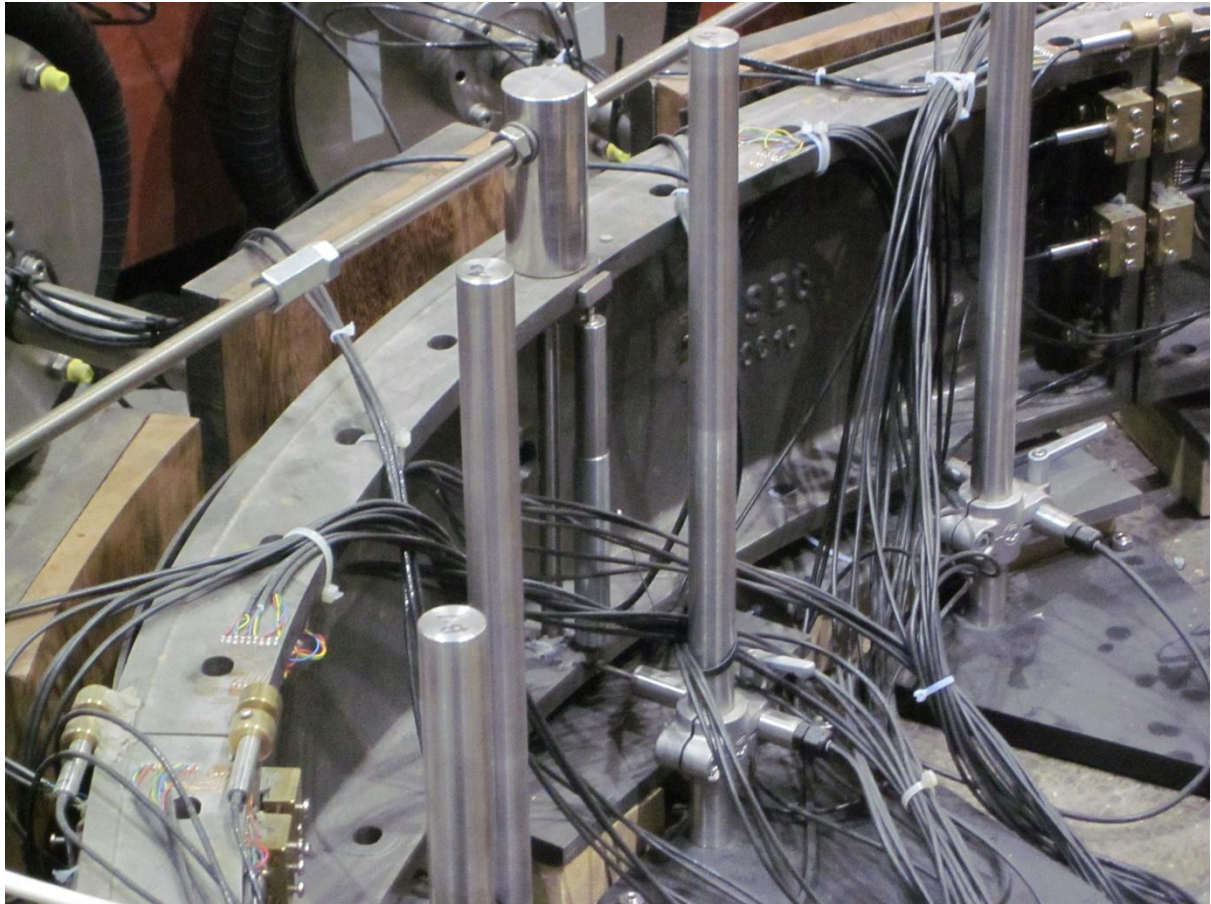


Figure 6-20. Strain gauged displacement transducer for monitoring convergence/divergence of circumferential flanges.



Figure 6-21. Strain gauged pressure transducer.

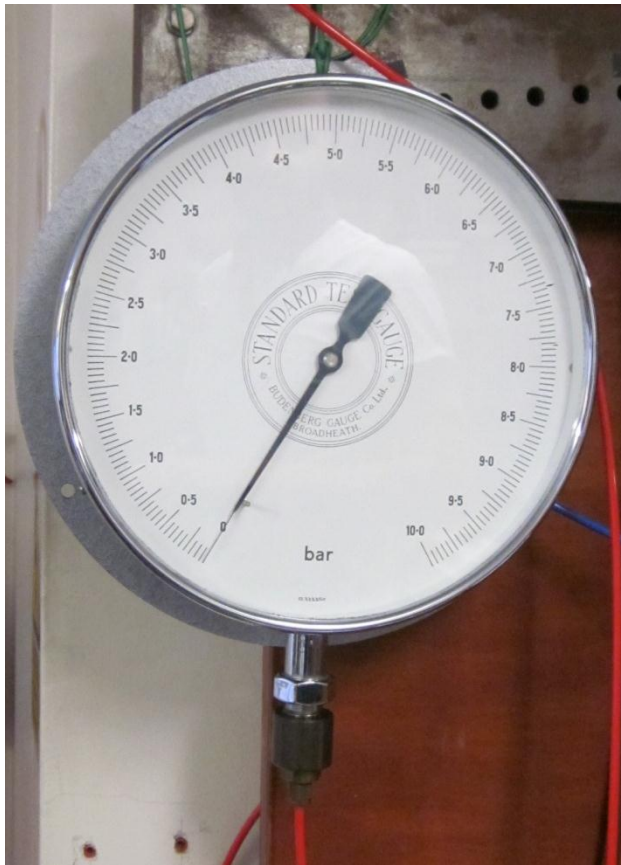


Figure 6-22. Budenberg pressure gauge for calibrating pressure transducer.

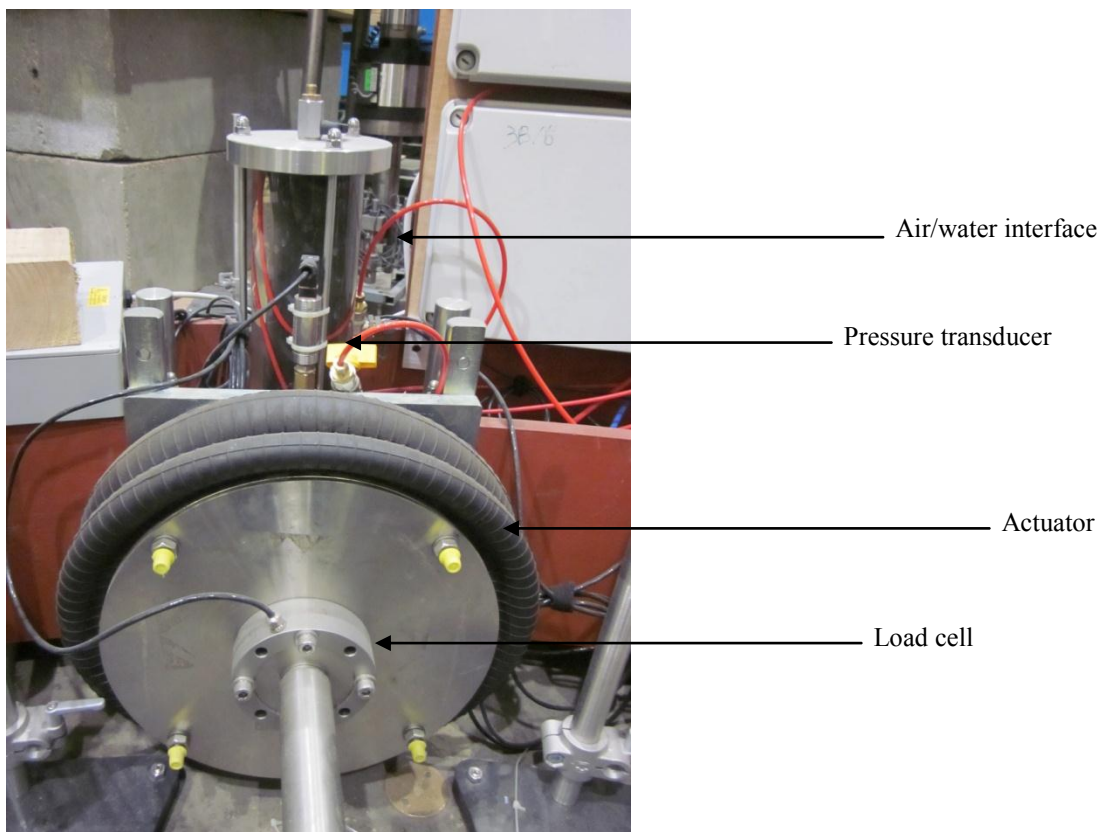


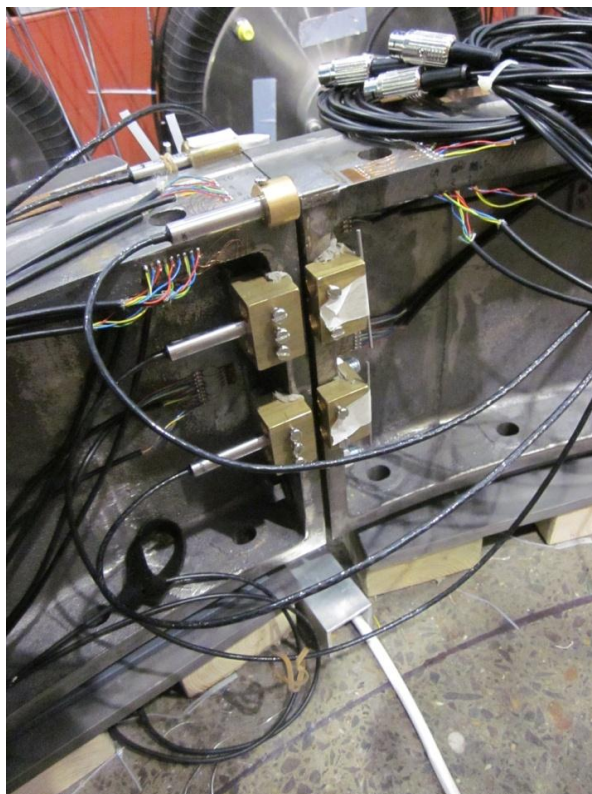
Figure 6-23. Pressure transducer connected to air/water interface. A load cell is located in front of the actuator at the base of the loading rod.



Figure 6-24. Pancake style load cell.



Figure 6-25. LVDT for measuring joint opening.



LVDTs on intrados



LVDTs on extrados

Figure 6-26. Locations of LVDTs.



Figure 6-27. Instron loading rig.

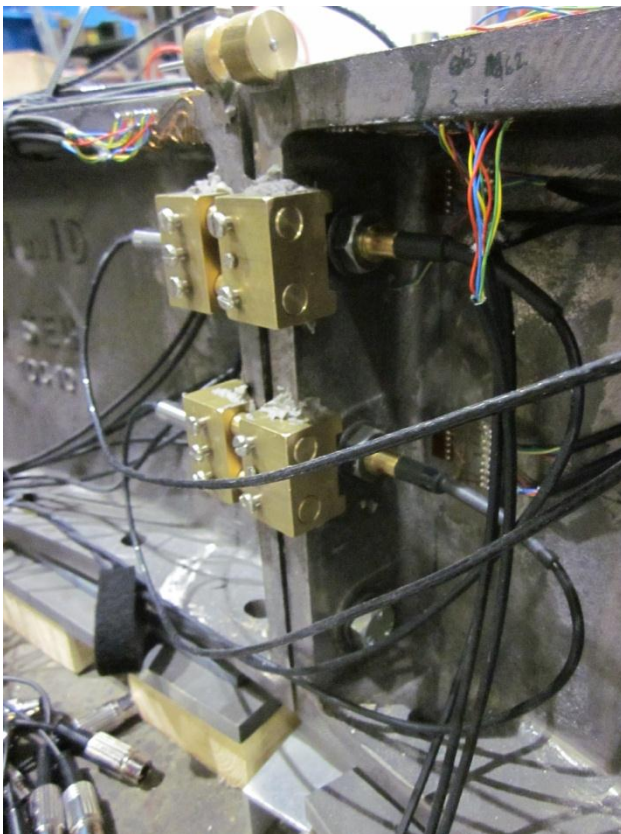


Figure 6-28. Instrumented bolt used on test ring.

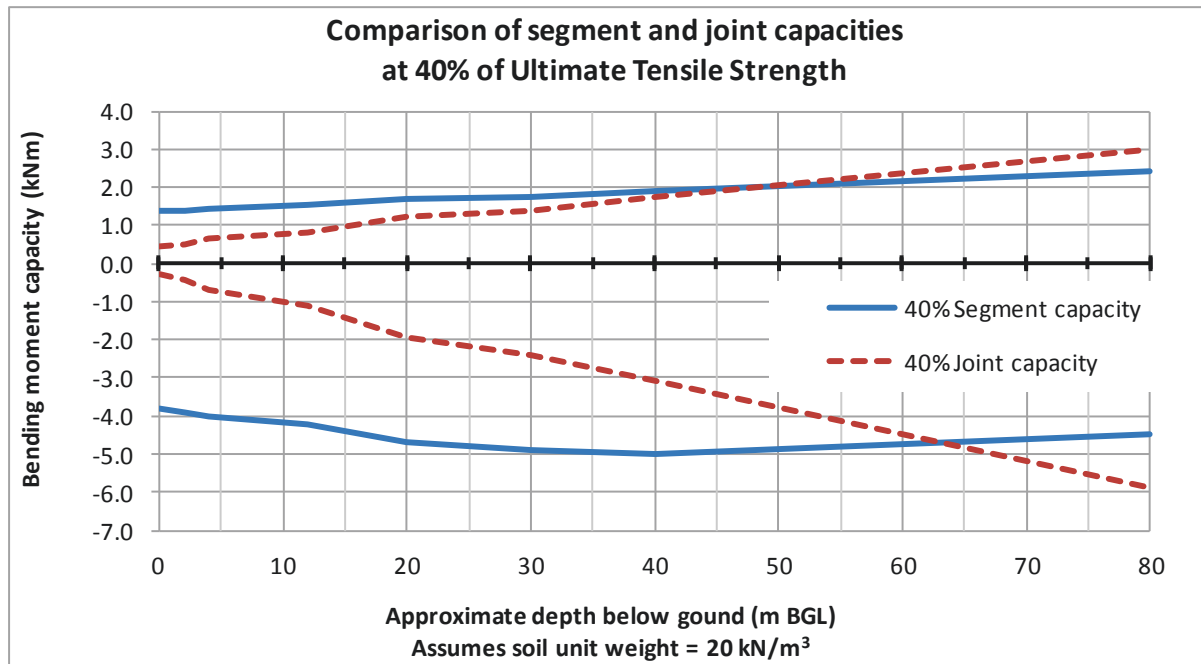


Figure 6-29. Comparison of segment and joint capacities at different depths.

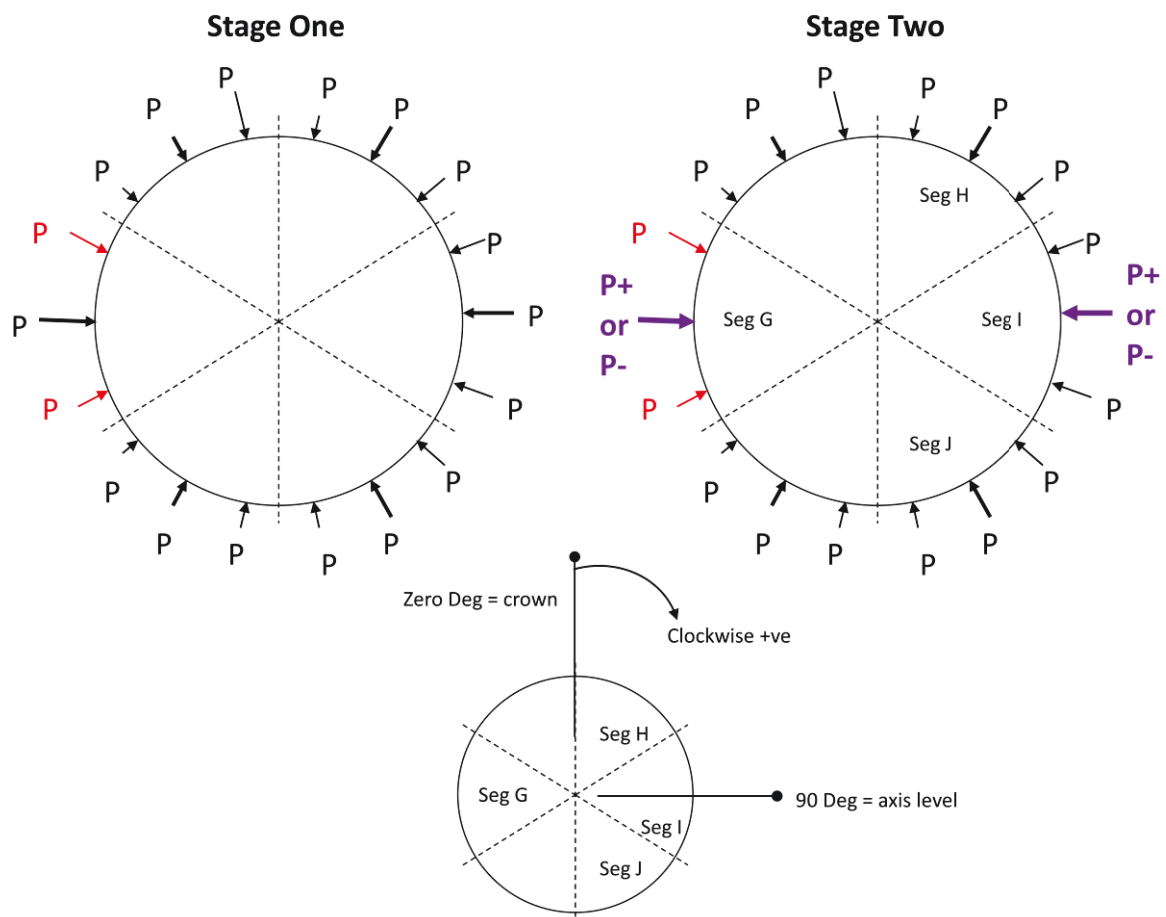


Figure 6-30. Loading regime for laboratory tests.

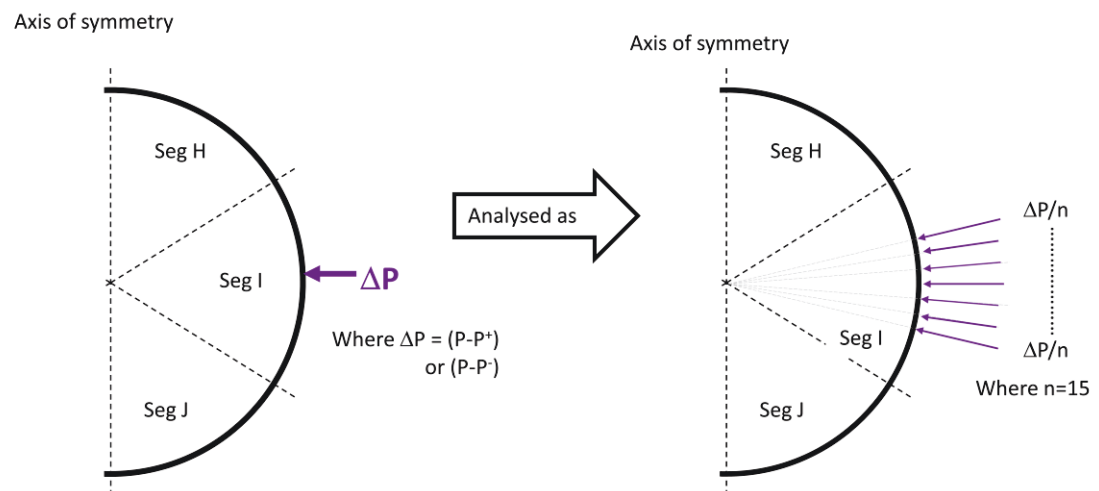


Figure 6-31. Schematic diagram showing how the loading was analysed using Castigliano's second theorem.

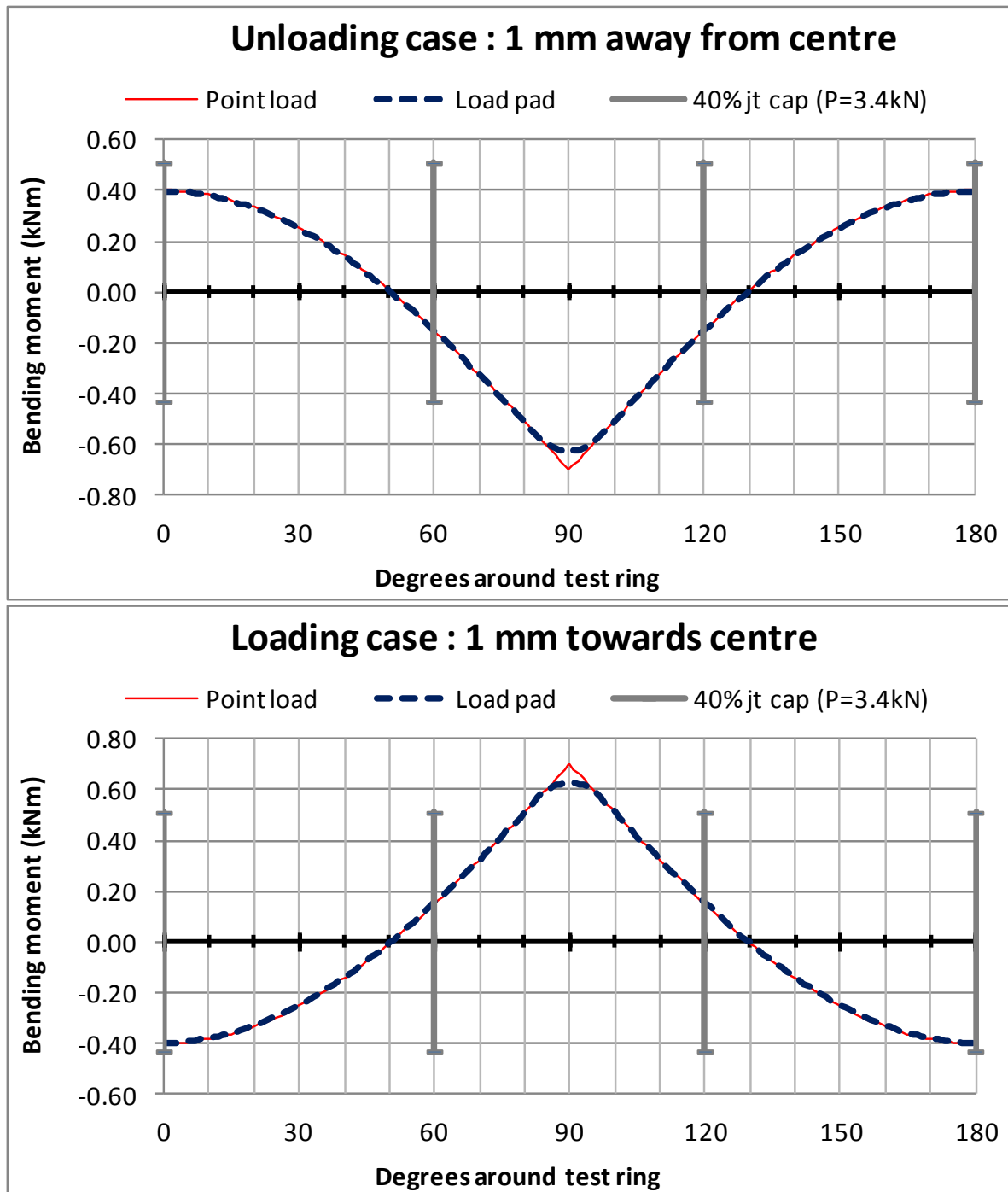


Figure 6-32. Difference in bending moment between point load and loading pad analyses for unloading case (top) and loading case (bottom).

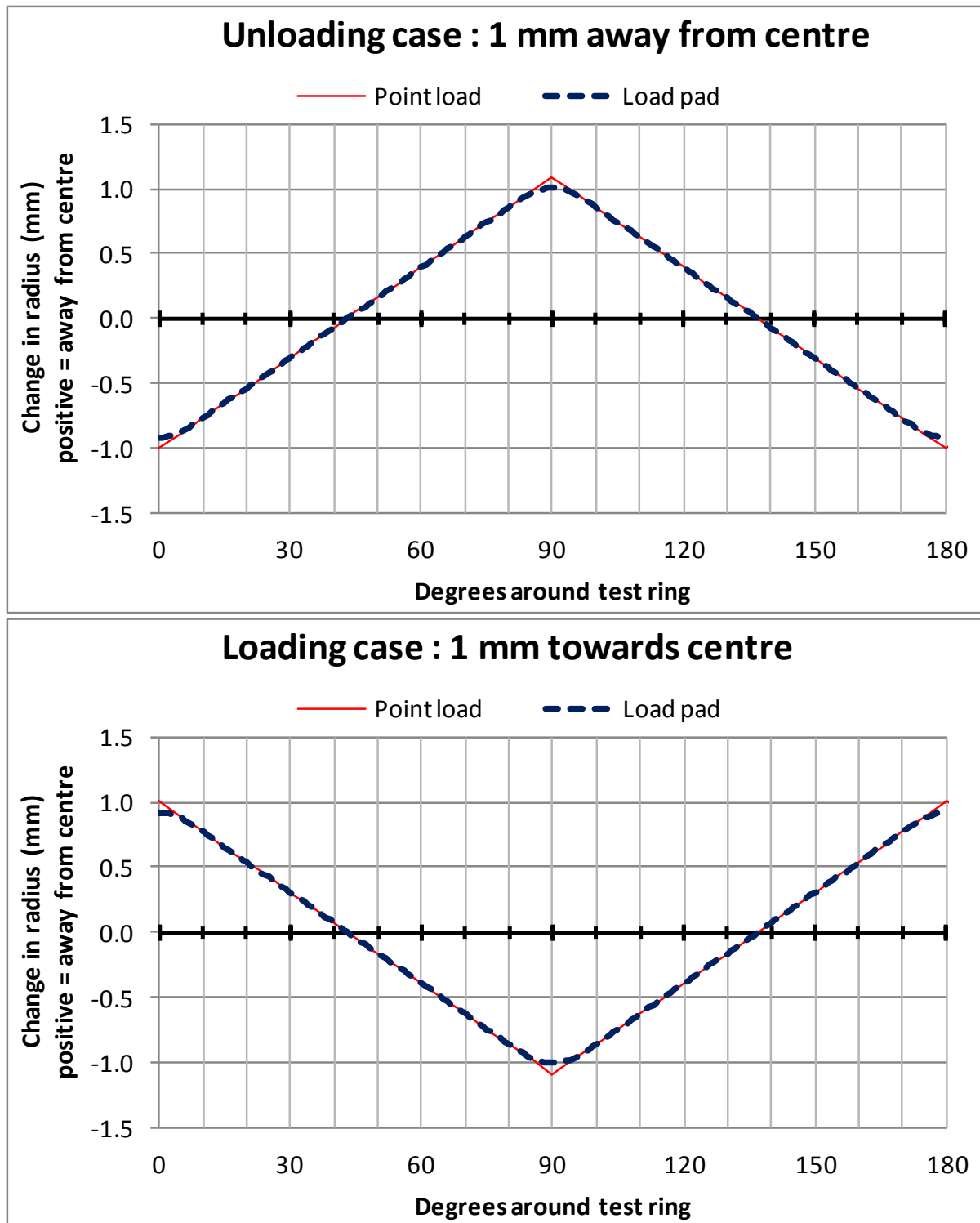


Figure 6-33. Difference in displacement between point load and loading pad analyses for unloading case (top) and loading case (bottom).

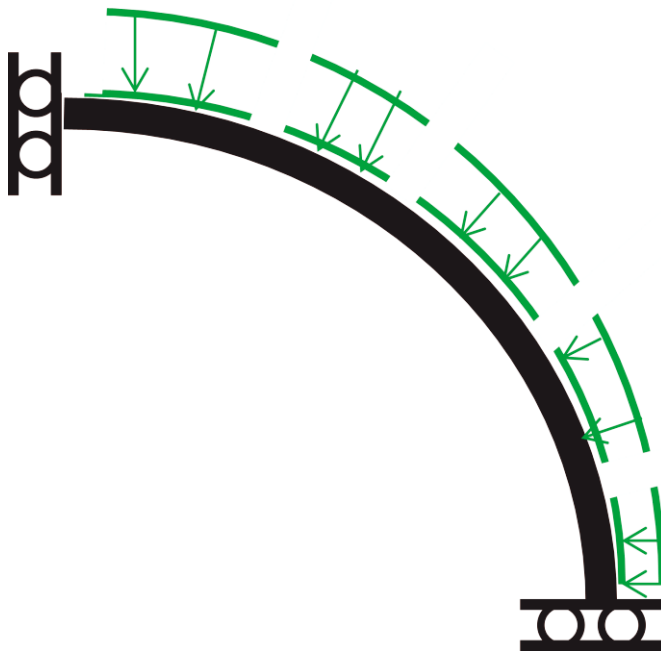


Figure 6-34. 2D structural FE model with normal stresses applied at discrete 'pad' locations.

7 Behaviour of bolted segmental cast iron ring

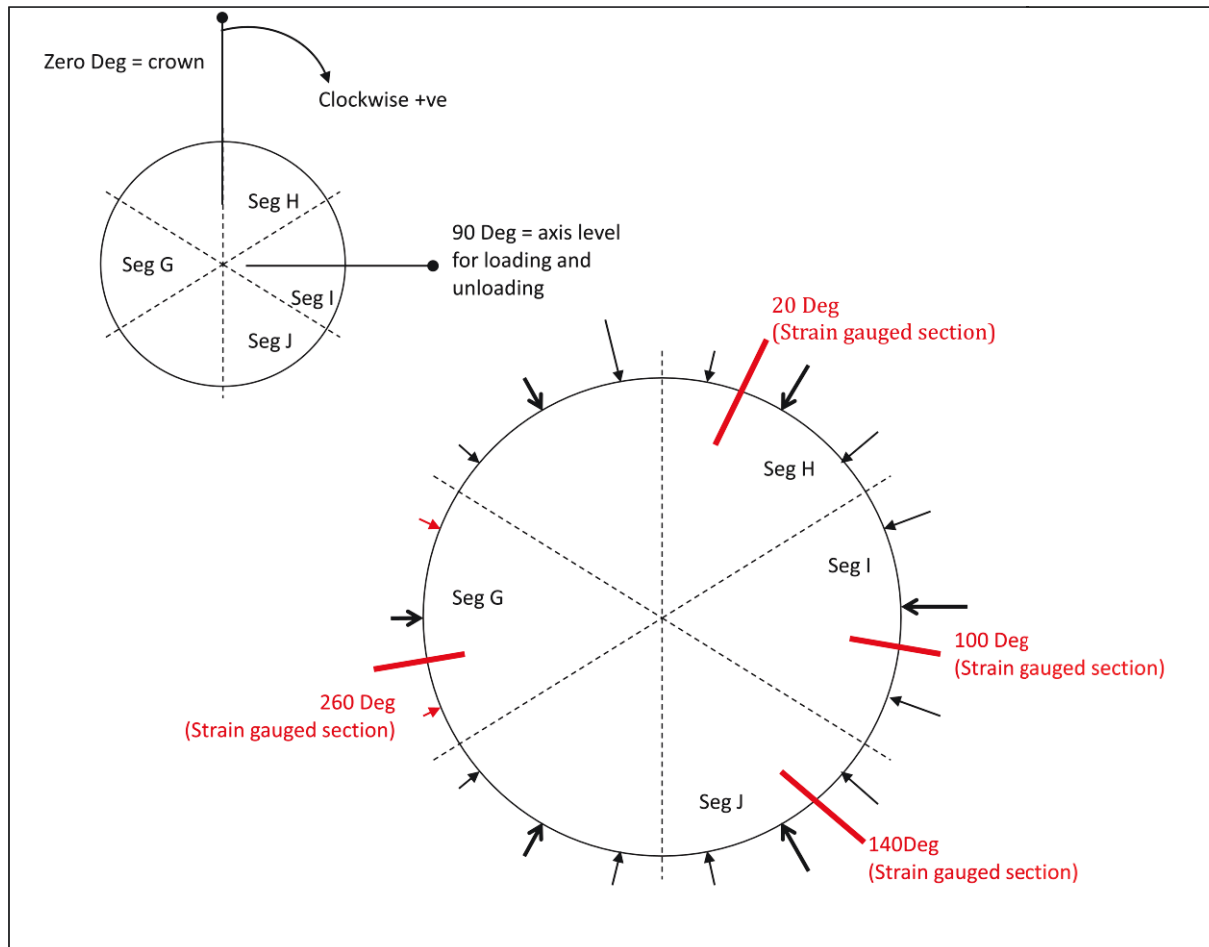


Figure 7-1. Sign convention for test ring.

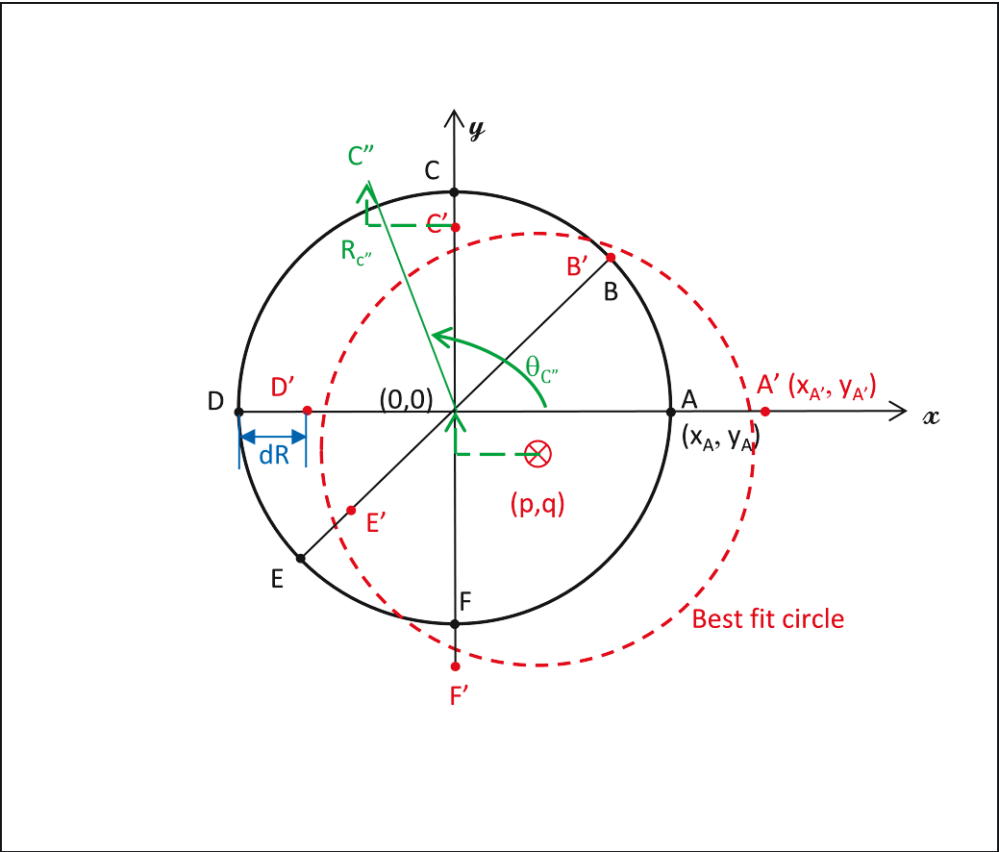


Figure 7-2. Calculation of radius change.

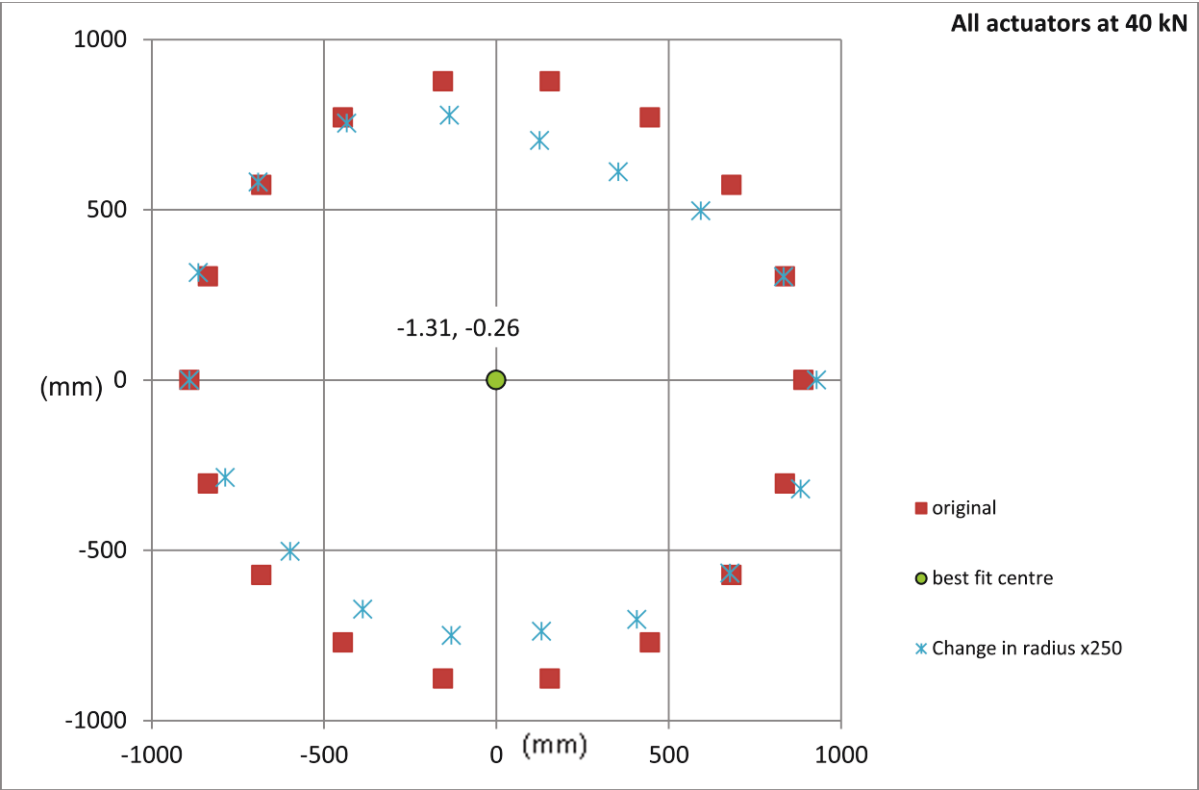


Figure 7-3. The test ring is distorted after the application of equal loads at all actuator locations.

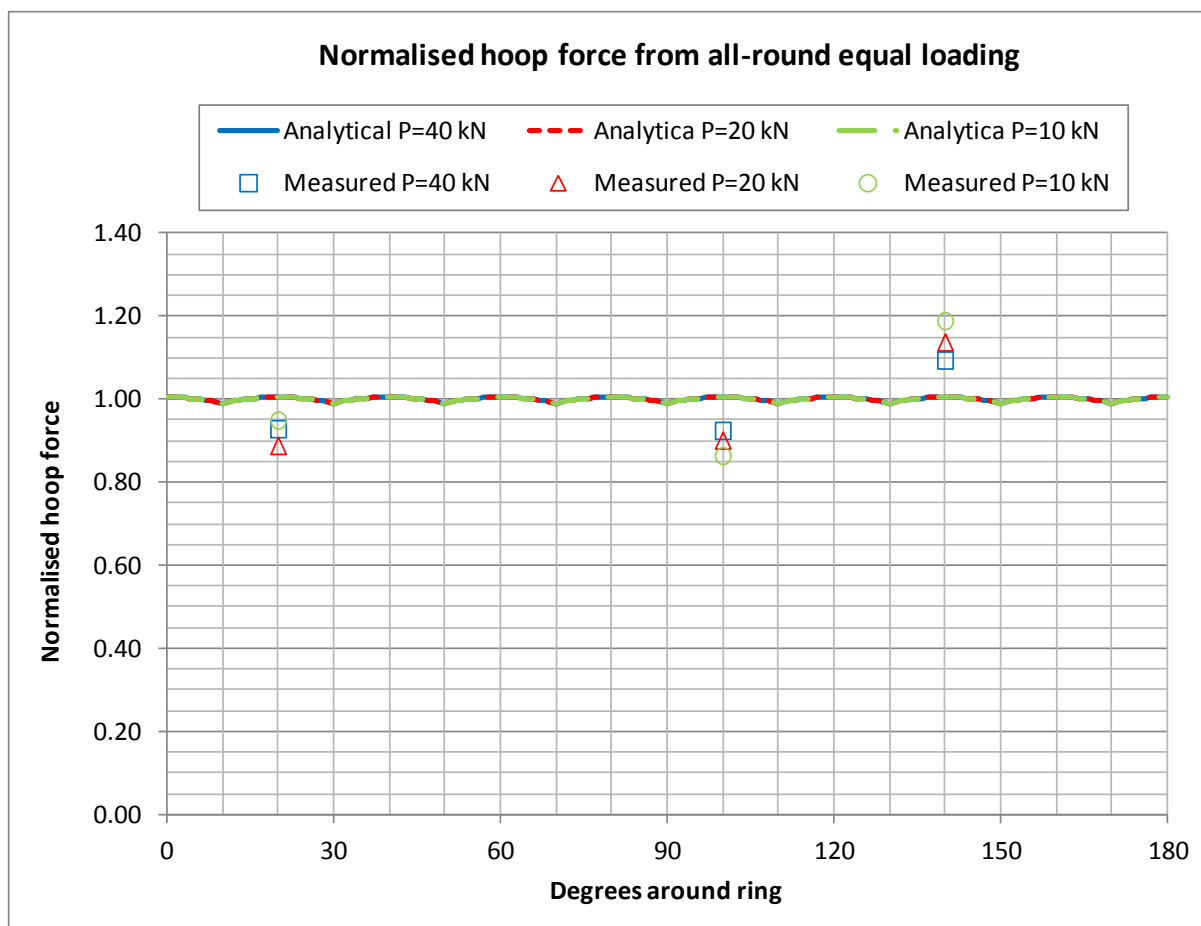


Figure 7-4. Normalised hoop force from Stage One loading.

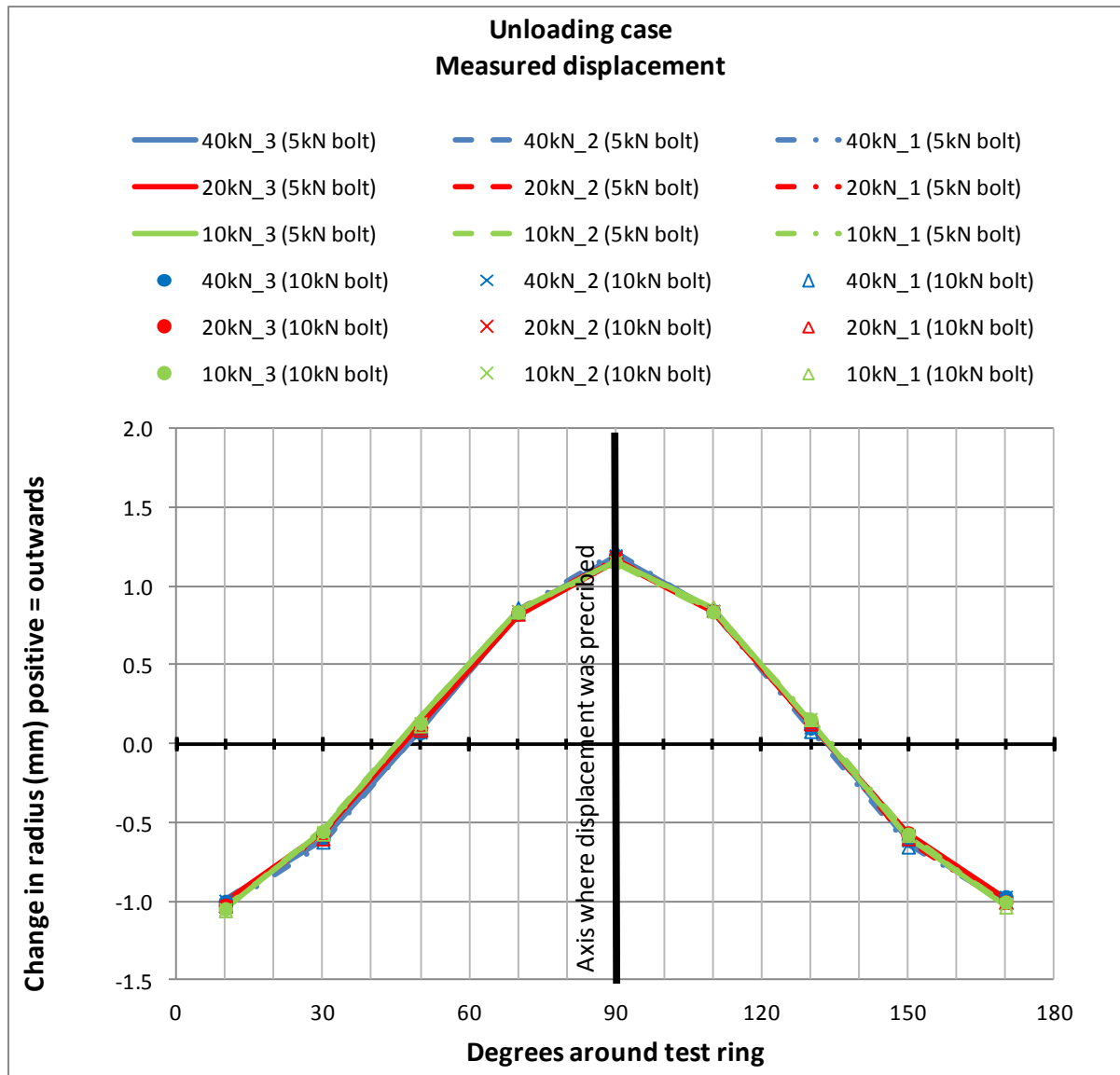


Figure 7-5. Measured displacement for unloading tests. Results for two different bolt preloads shown.

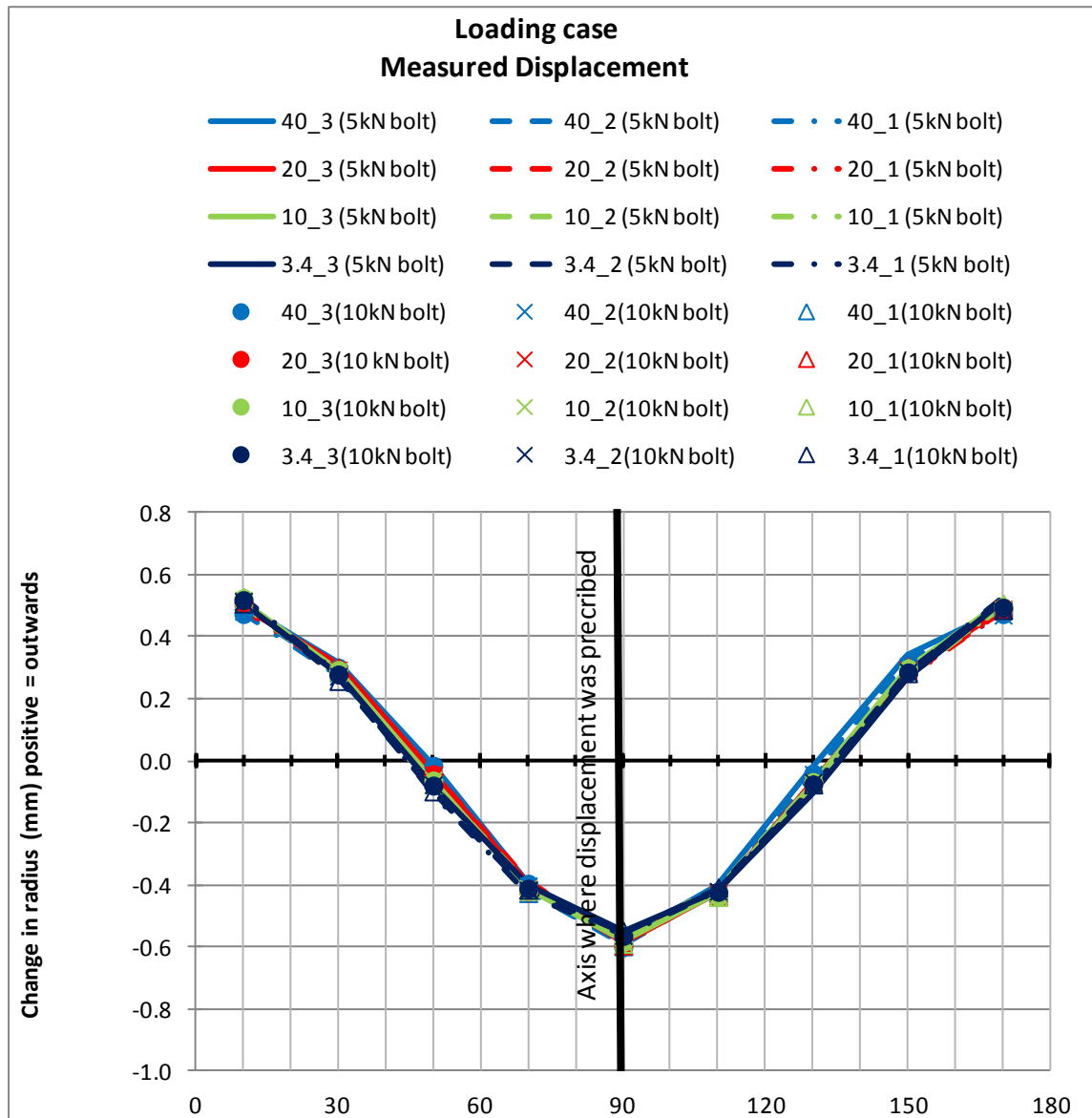


Figure 7-6. Measured displacement for loading tests. Results for two different bolt preloads shown.

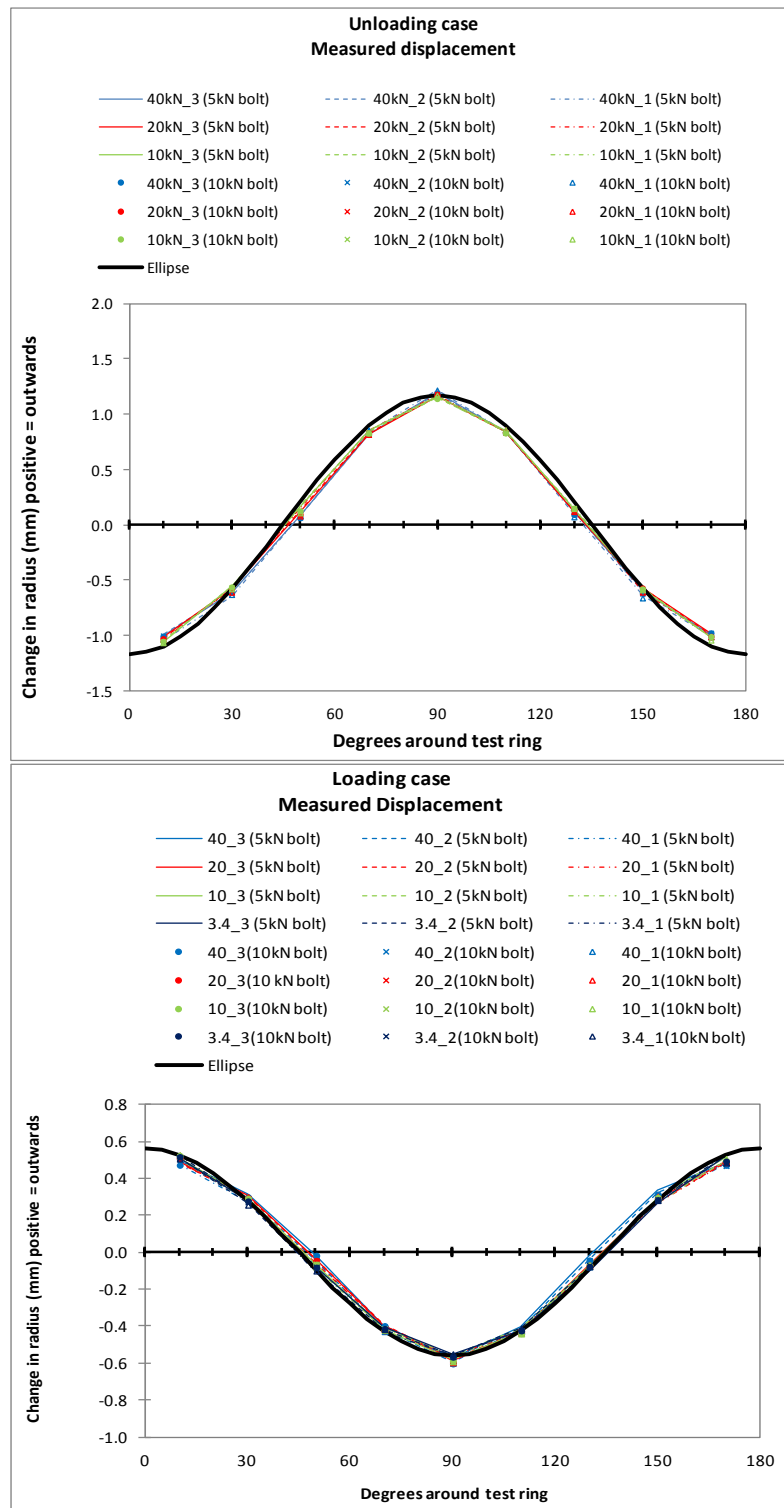


Figure 7-7. ‘Ellipse’ superimposed over unloading displacements (top) and loading displacements (bottom).

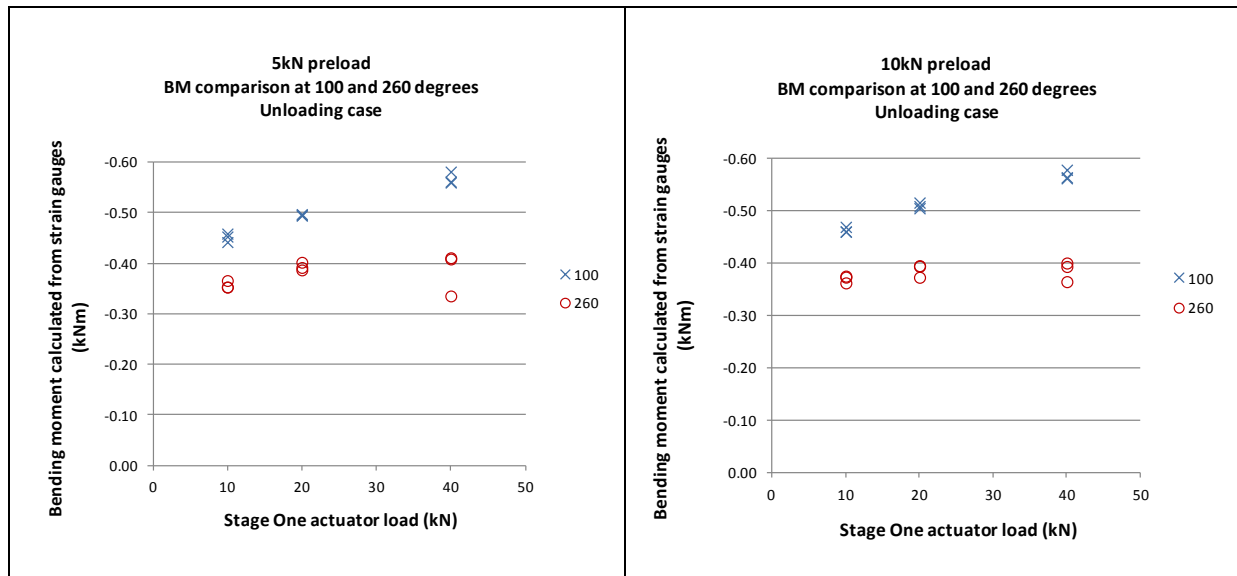


Figure 7-8. Comparison of bending moments on segment I (100 degrees) and segment G (260 degrees). Unloading case.

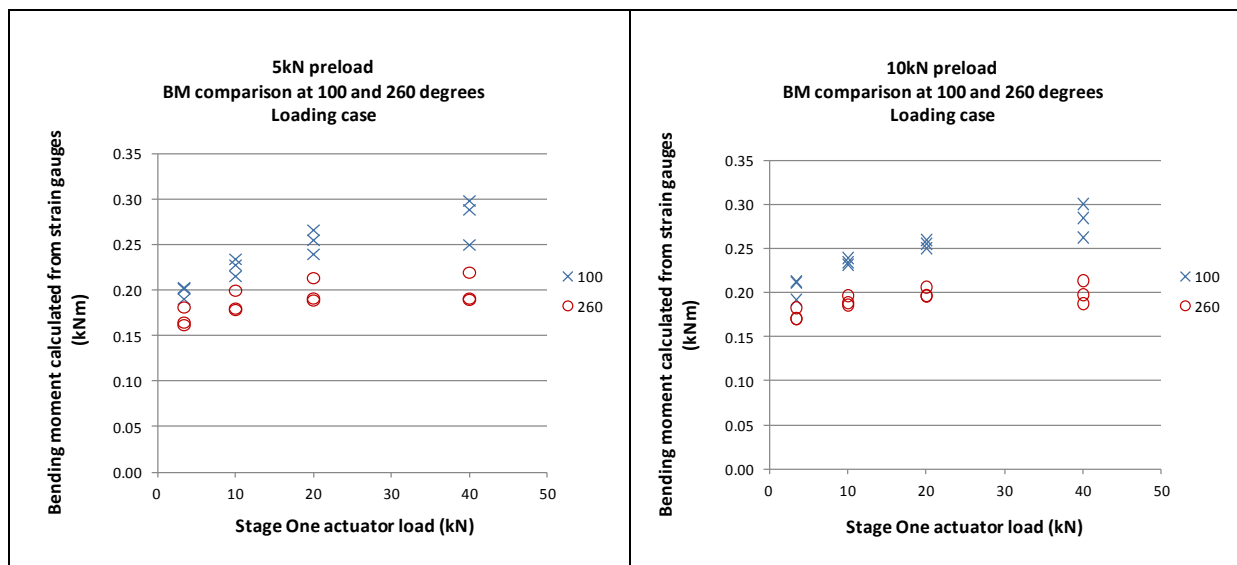
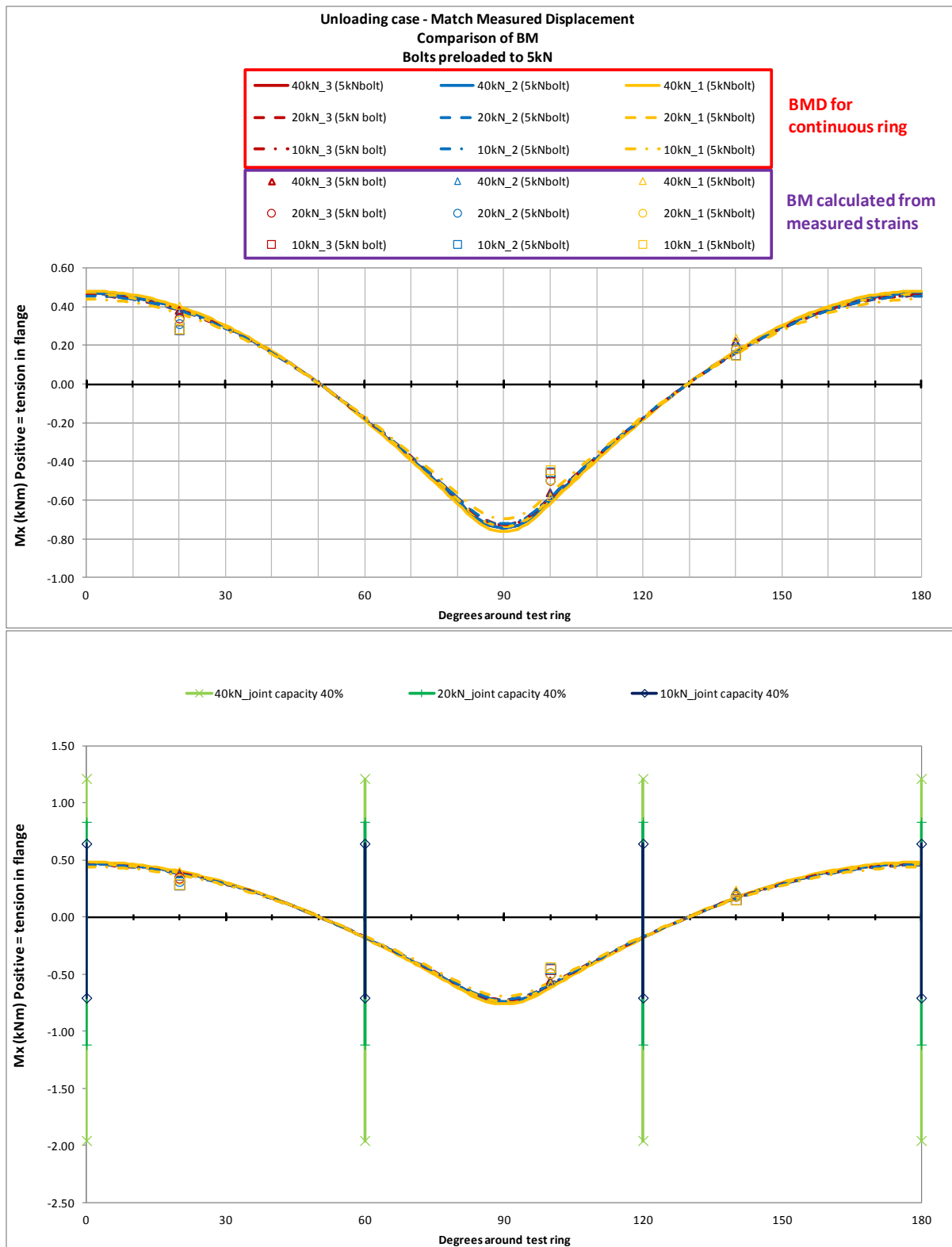


Figure 7-9. Comparison of bending moments on segment I (100 degrees) and segment G (260 degrees). Loading case.



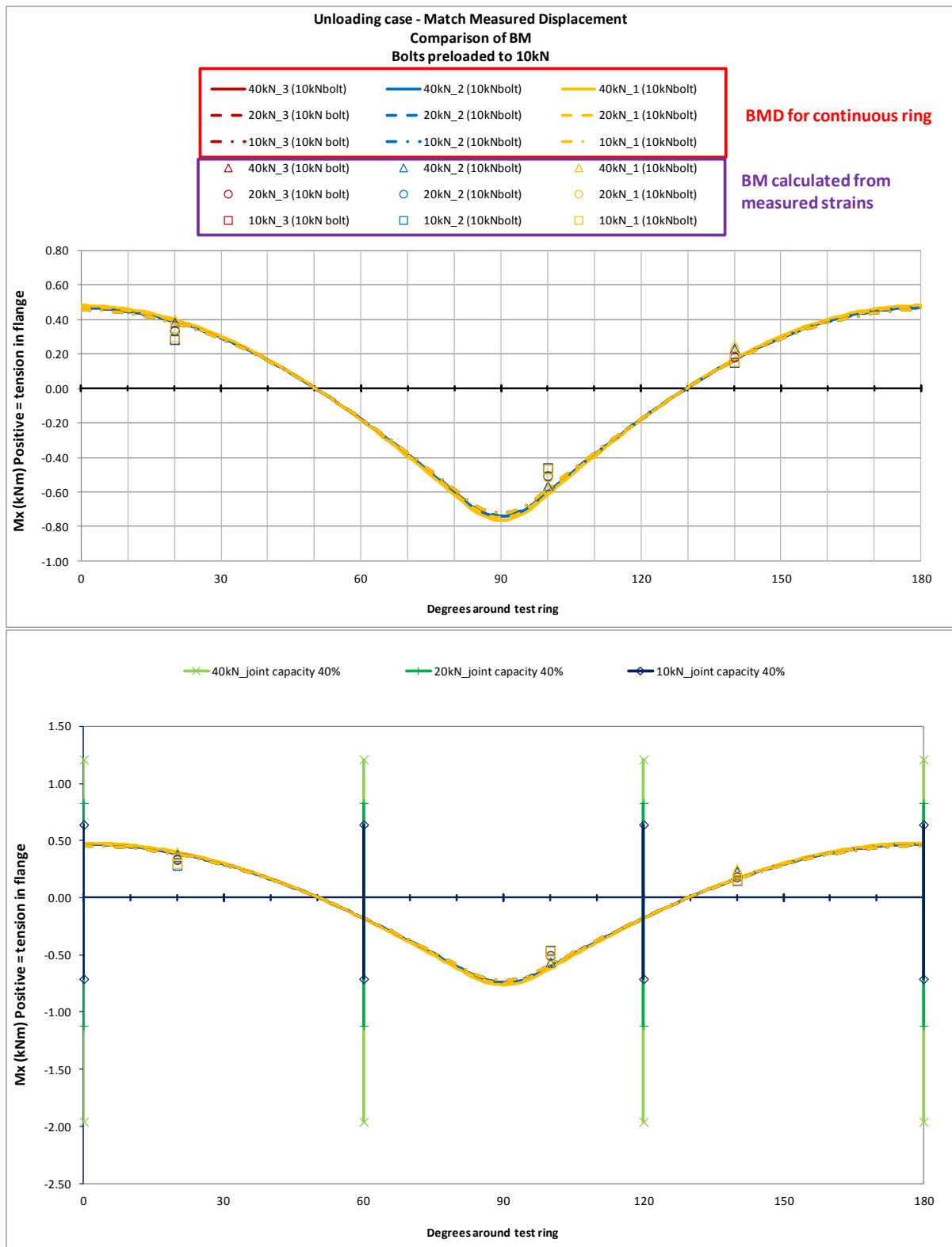


Figure 7-11. Unloading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 10kN (top). Bending moments compared with joint capacities at 40% (bottom).

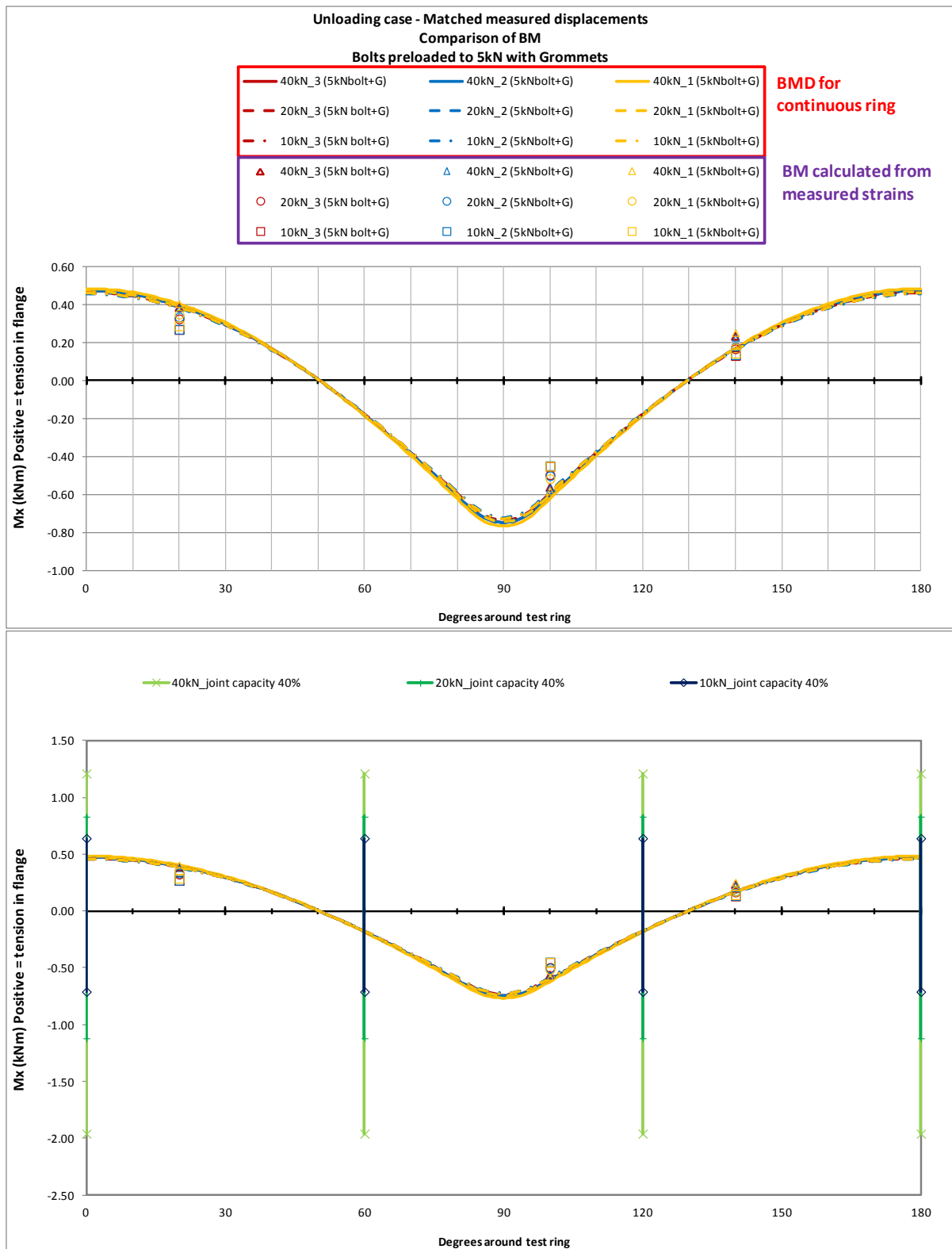


Figure 7-12. Unloading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 5kN with grommets (top). Bending moments compared with joint capacities at 40% (bottom).



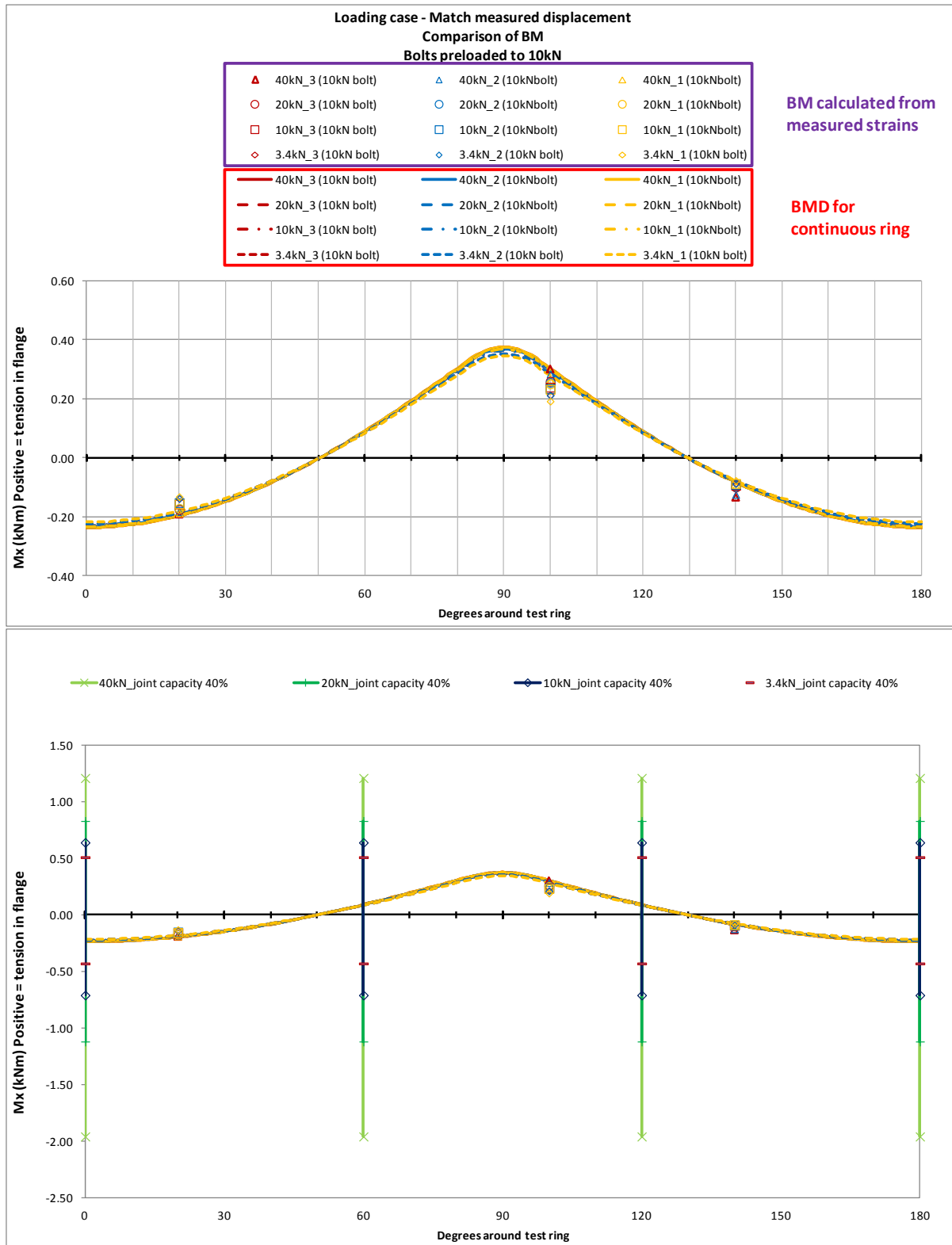


Figure 7-14. Loading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 10kN (top). Bending moments compared with joint capacities at 40% (bottom).

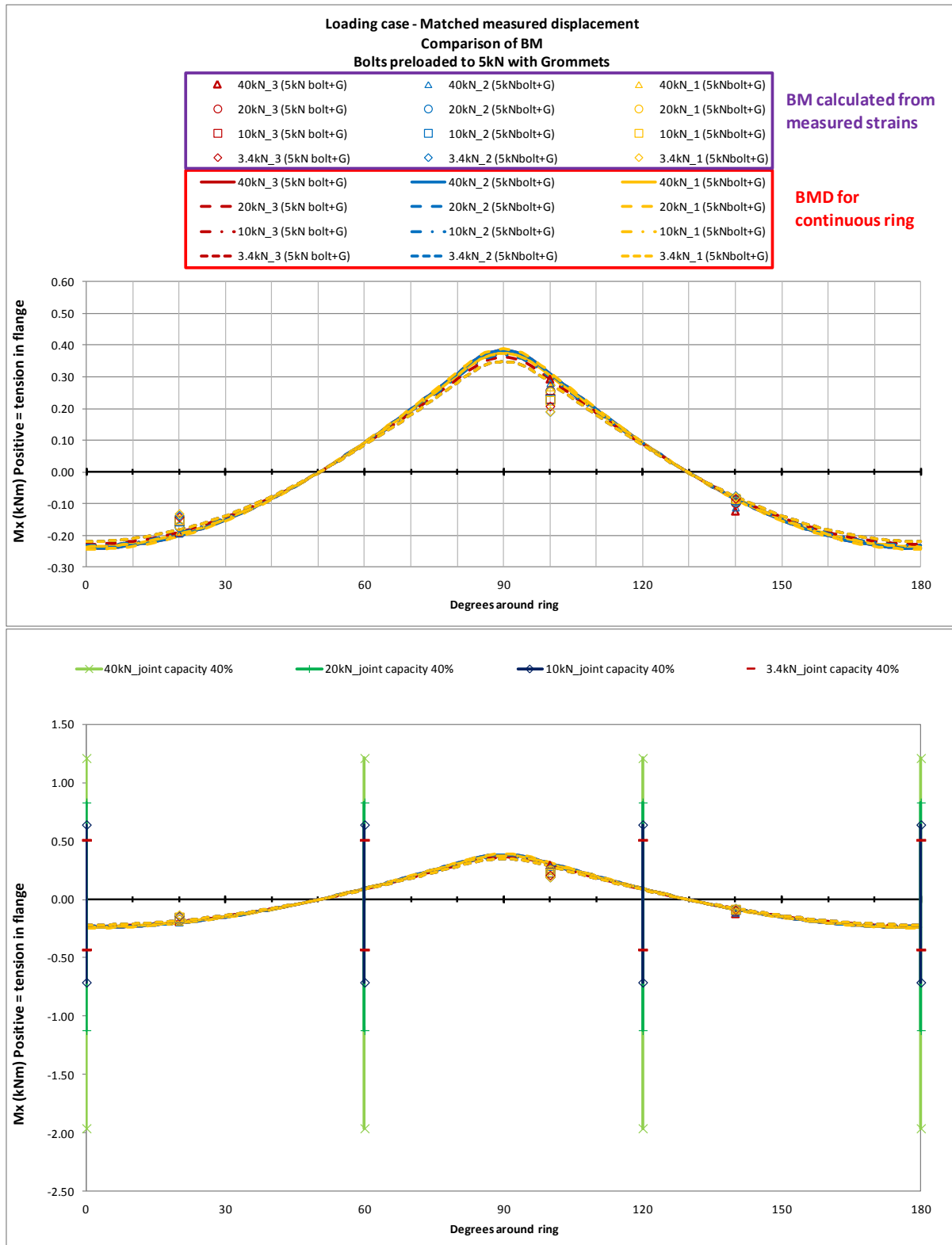


Figure 7-15. Loading case. Bending moment calculated from strain measurements compared with analytical predictions based on the deformed shape. Bolt preload 5kN with grommets (top). Bending moments compared with joint capacities at 40% (bottom).

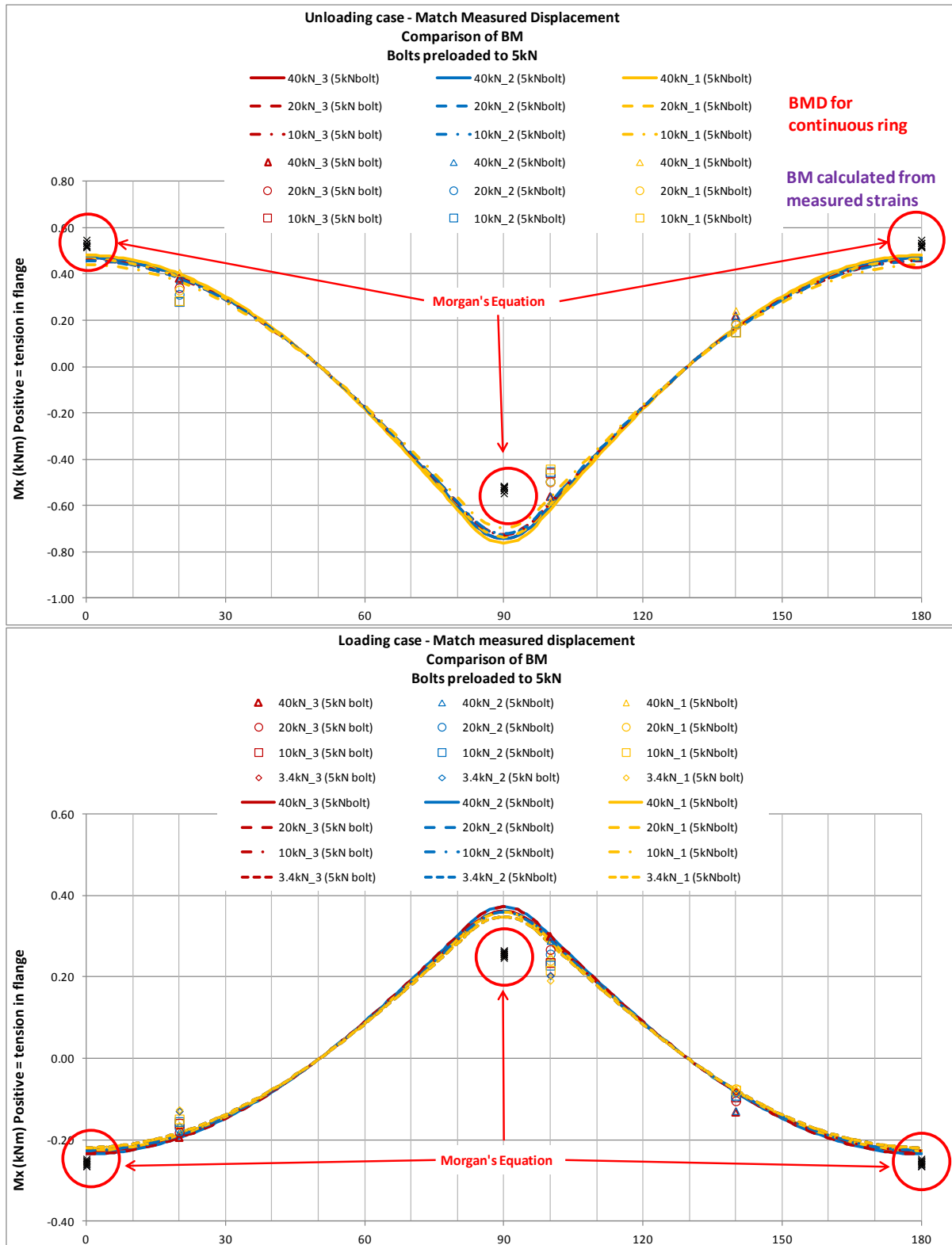


Figure 7-16. Bending moments calculated from Morgan's Equation compared with bending moments calculated from strain measurements and analytical predictions based on the deformed shape. Unloading case (top). Loading case (bottom).

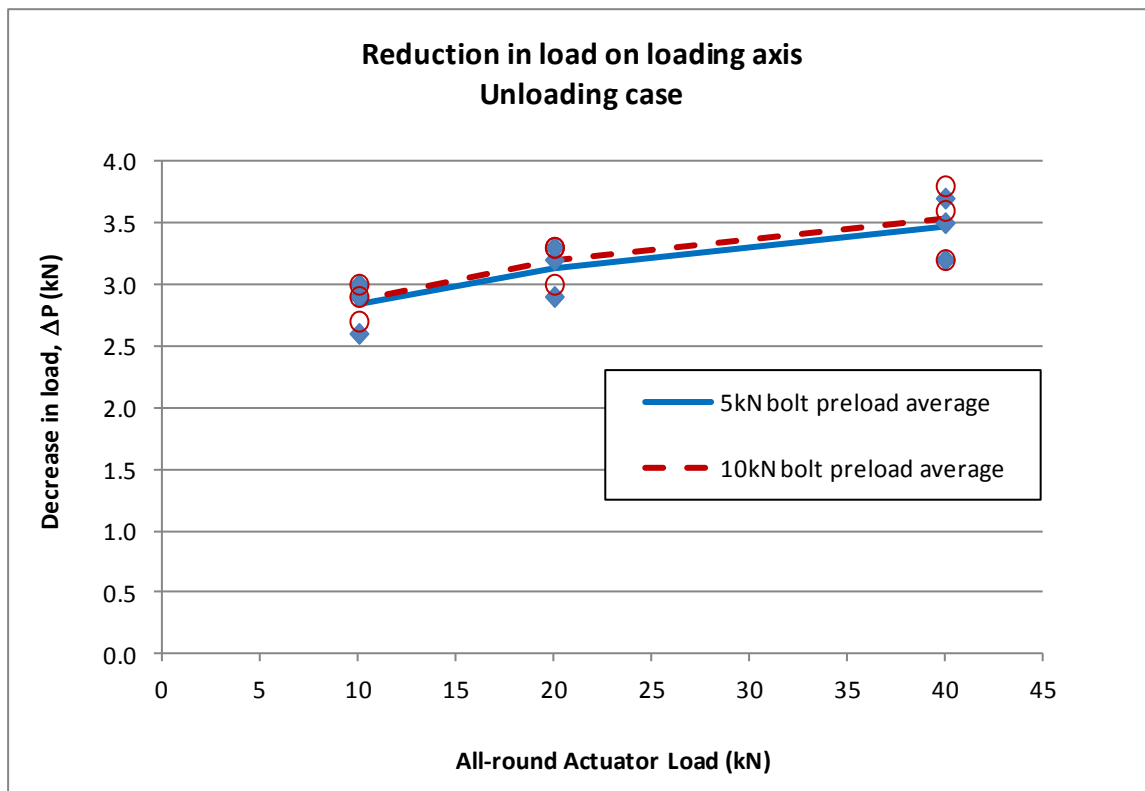


Figure 7-17. Unloading case. Changes in actuator load required to deform the ring by the same magnitude.

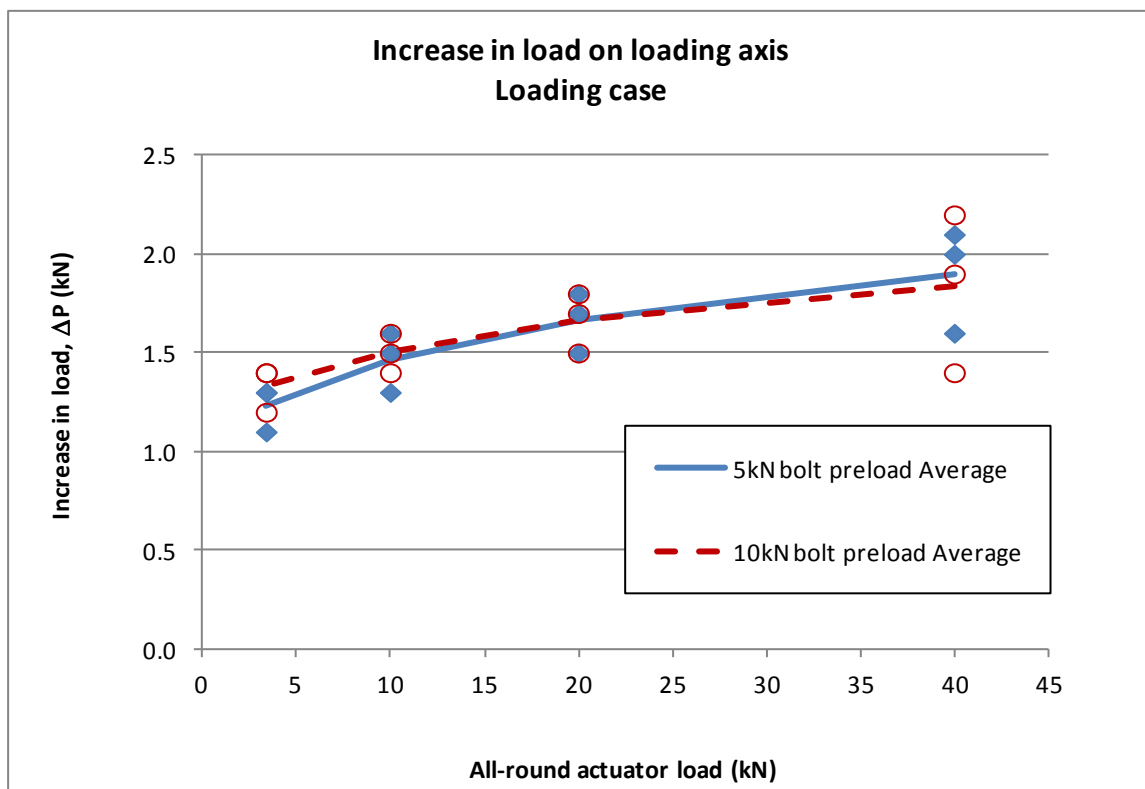


Figure 7-18. Loading case. Changes in actuator load required to deform the ring by the same magnitude.

Figure 7-19. Unloading case. Change in bolt load from Stage One to Stage Two.

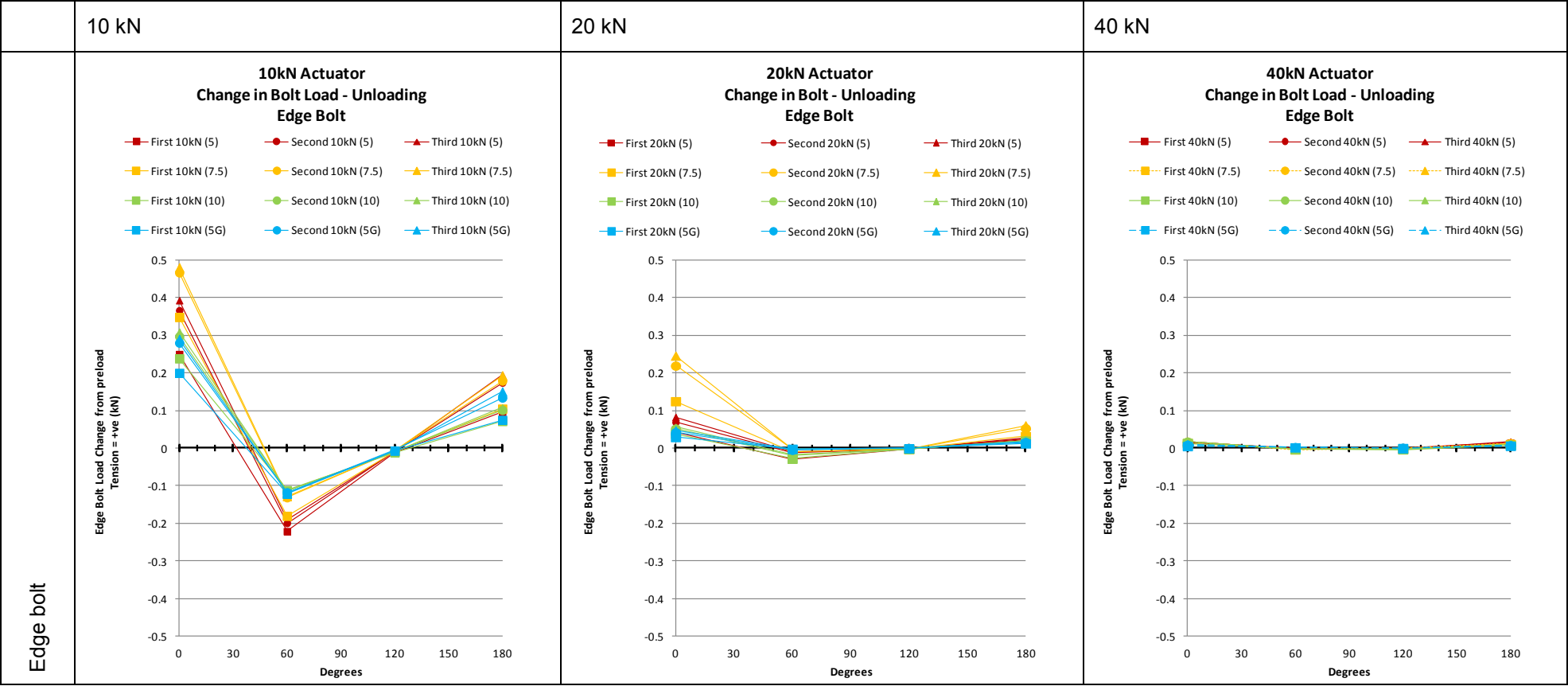


Figure 7-19. Unloading case. Change in bolt load from Stage One to Stage Two.

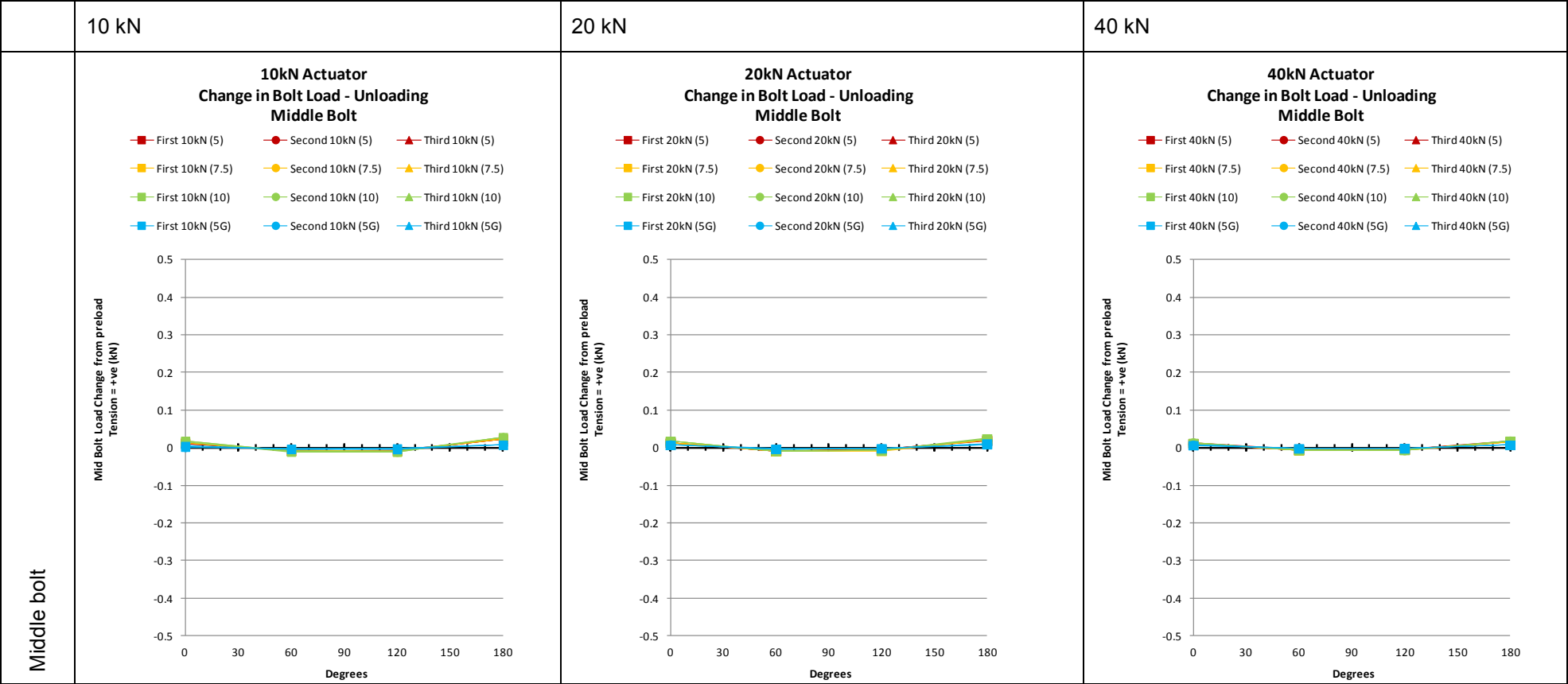


Figure 7-19. Unloading case. Change in bolt load from Stage One to Stage Two.

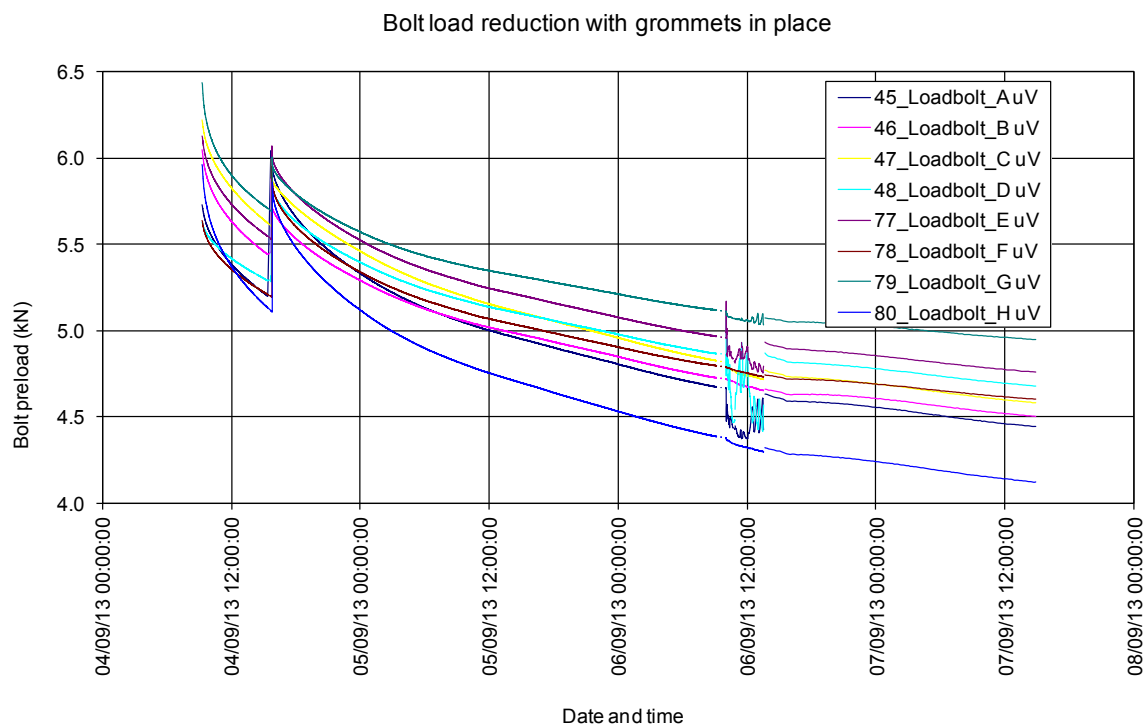


Figure 7-20. Reduction of bolt preload in all the instrumented bolts when grommets were in place.

Figure 7-21. Loading case. Change in bolt load from Stage One to Stage Two.

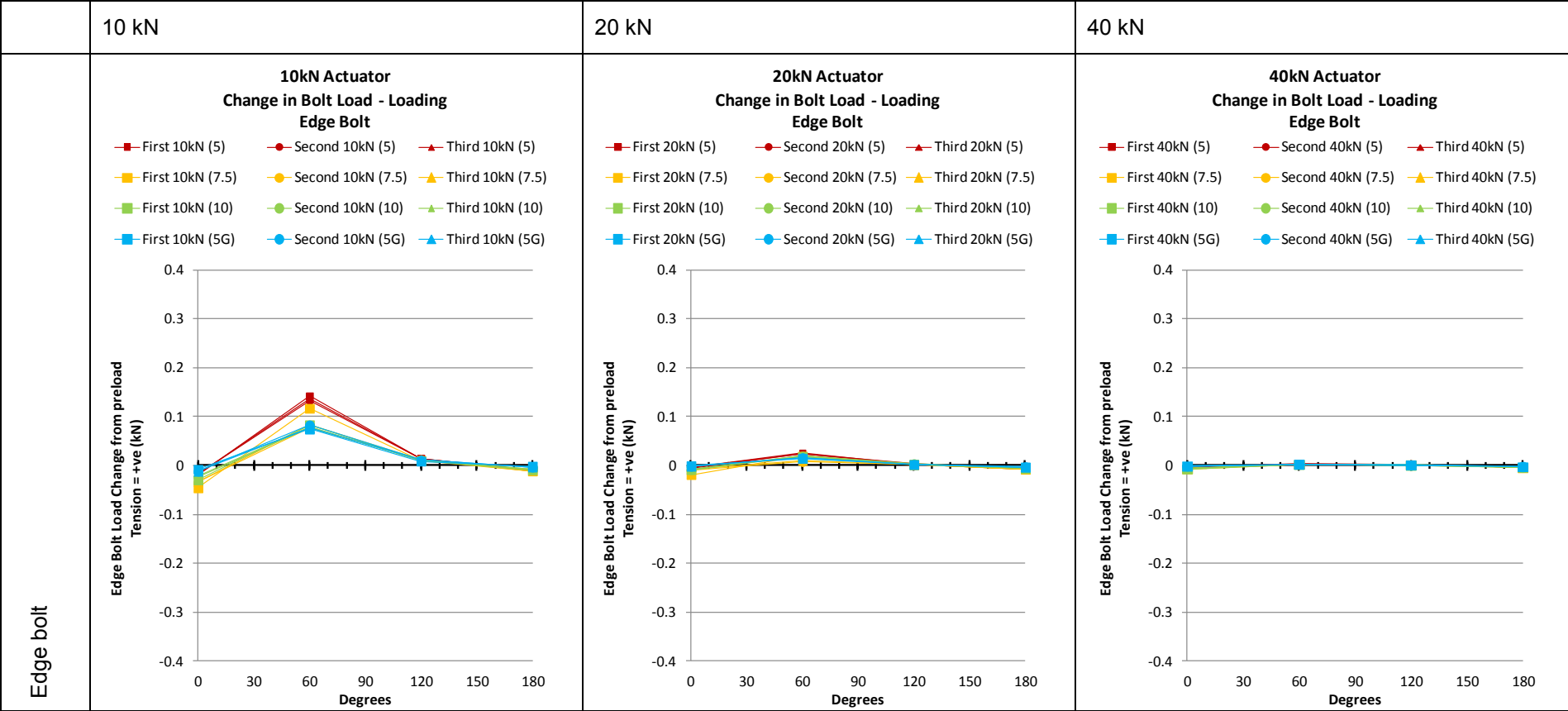


Figure 7-21. Loading case. Change in bolt load from Stage One to Stage Two.

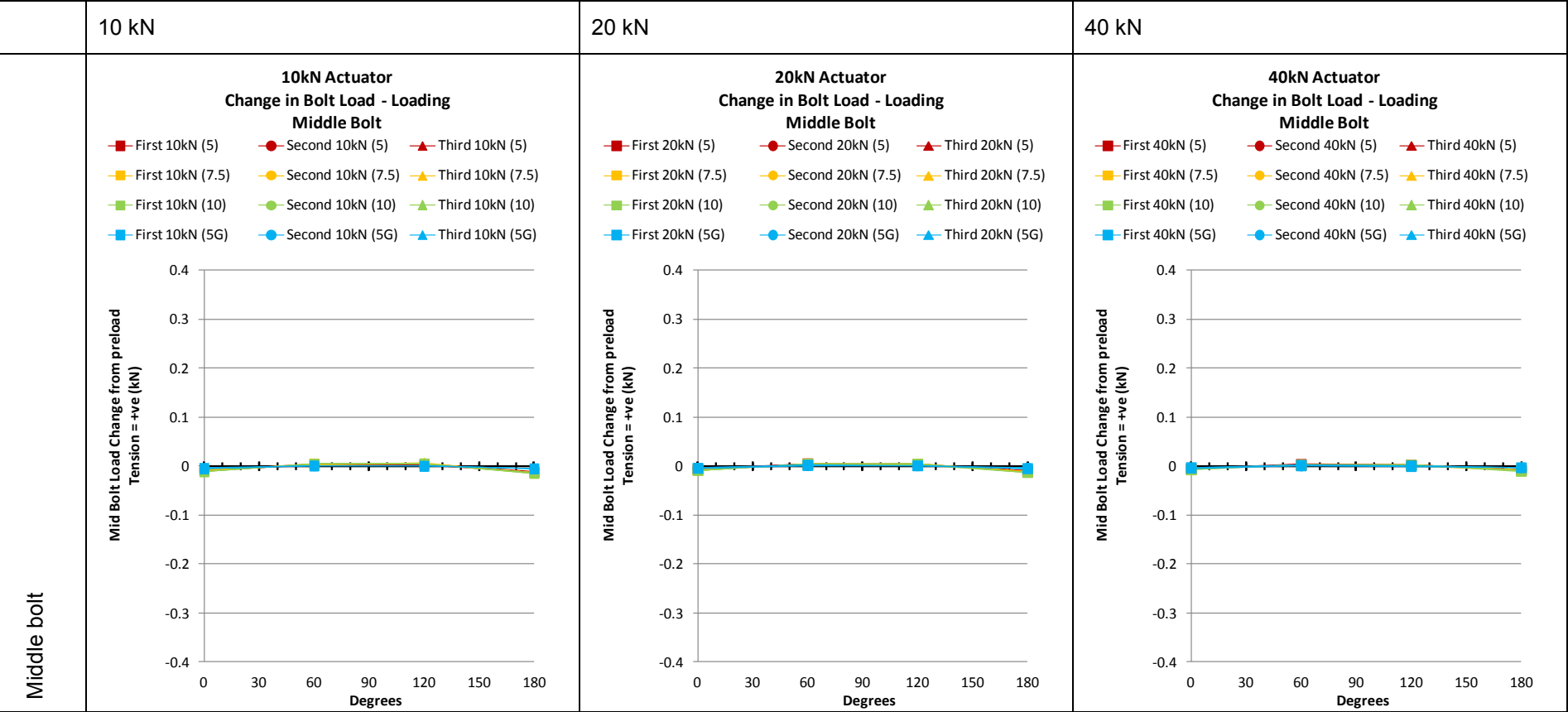


Figure 7-21. Loading case. Change in bolt load from Stage One to Stage Two.

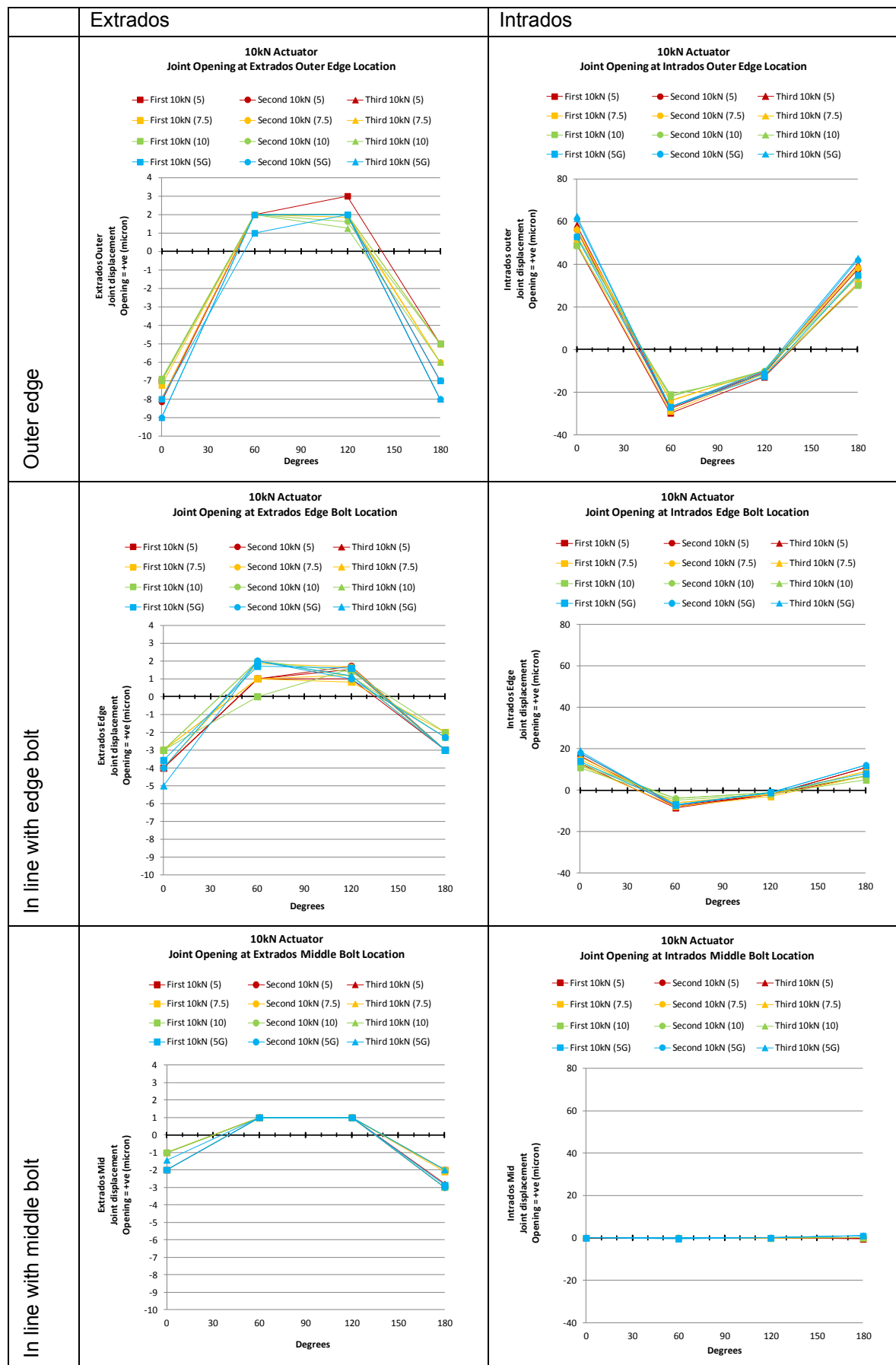


Figure 7-22. Unloading case. Local displacement at joint locations. All-round normal load prior to distortion = 10 kN.

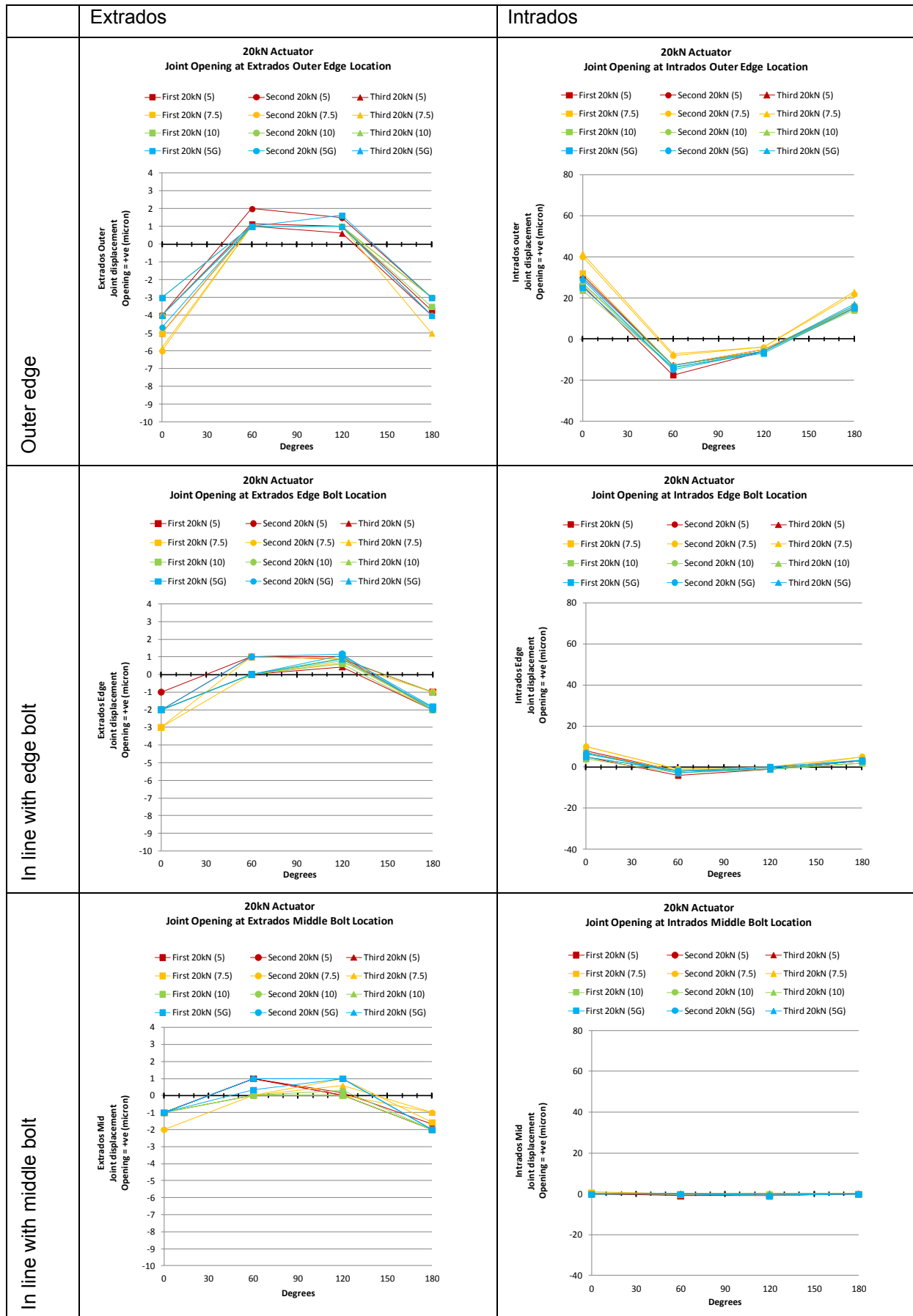


Figure 7-23. Unloading case. Local displacement at joint locations. All-round normal load prior to distortion = 20 kN.

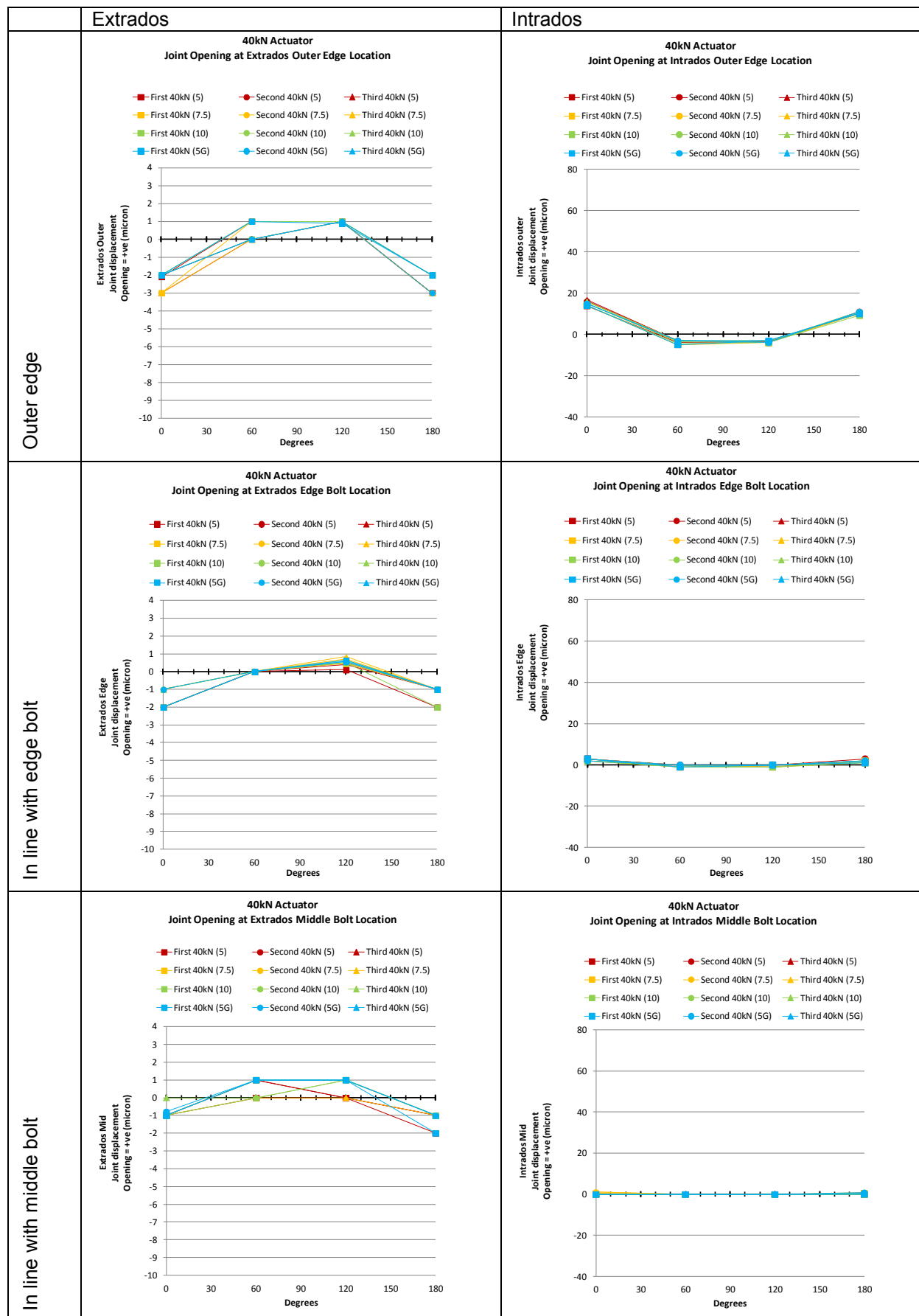


Figure 7-24. Unloading case. Local displacement at joint locations. All-round normal load prior to distortion = 40 kN.

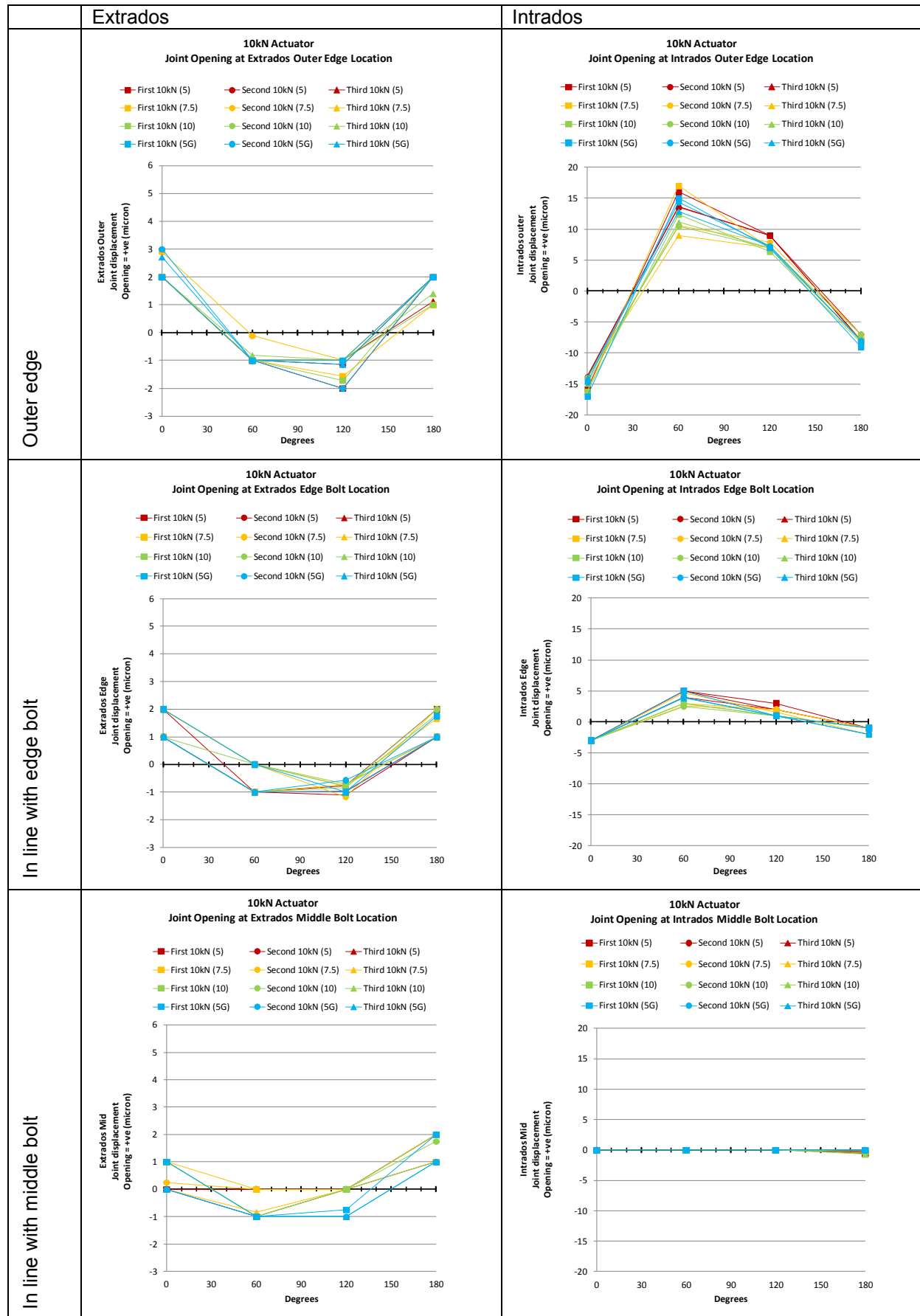


Figure 7-25. Loading case. Local displacement at joint locations. All-round normal load prior to distortion = 10 kN.

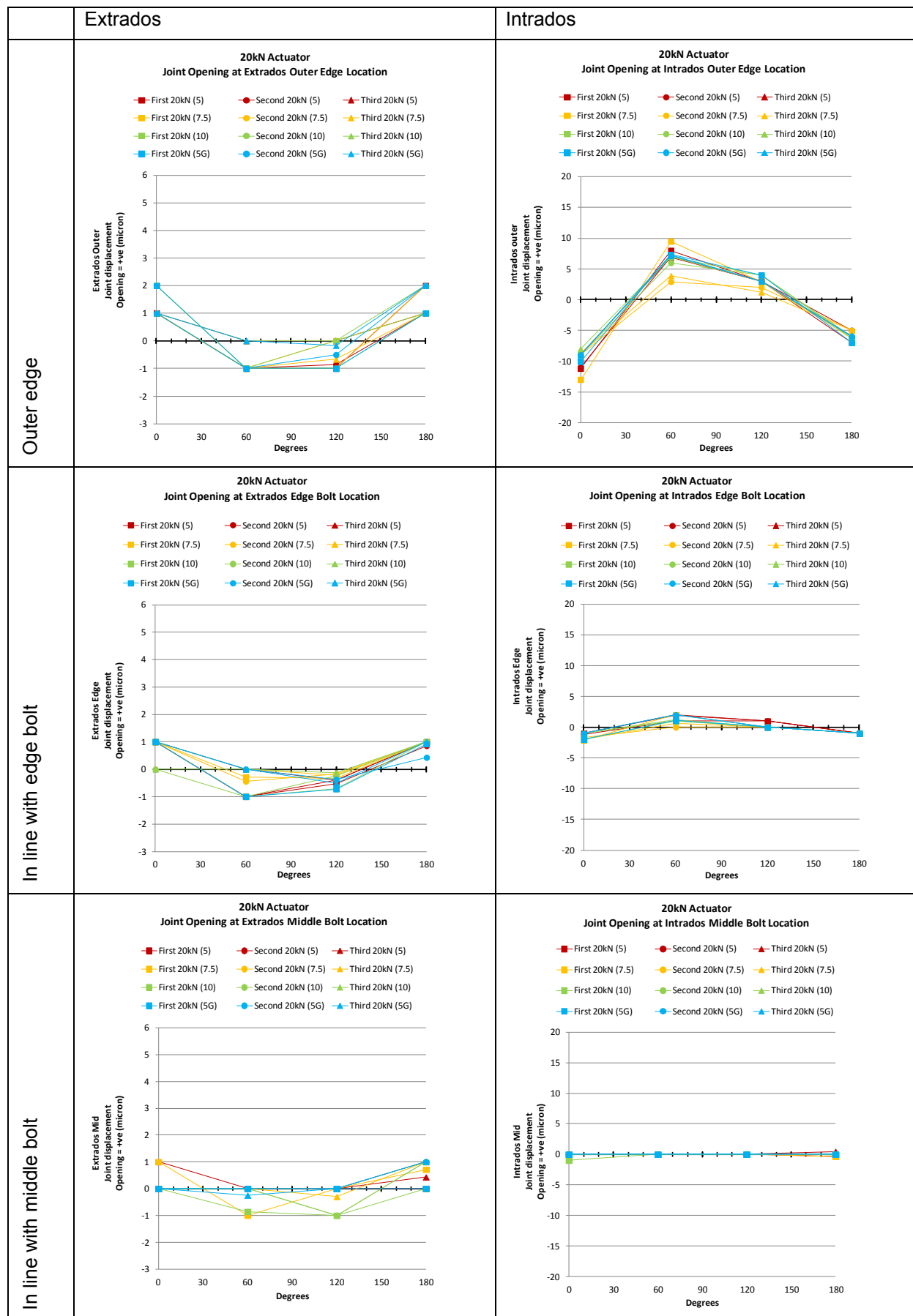


Figure 7-26. Loading case. Local displacement at joint locations. All-round normal load prior to distortion = 20 kN.

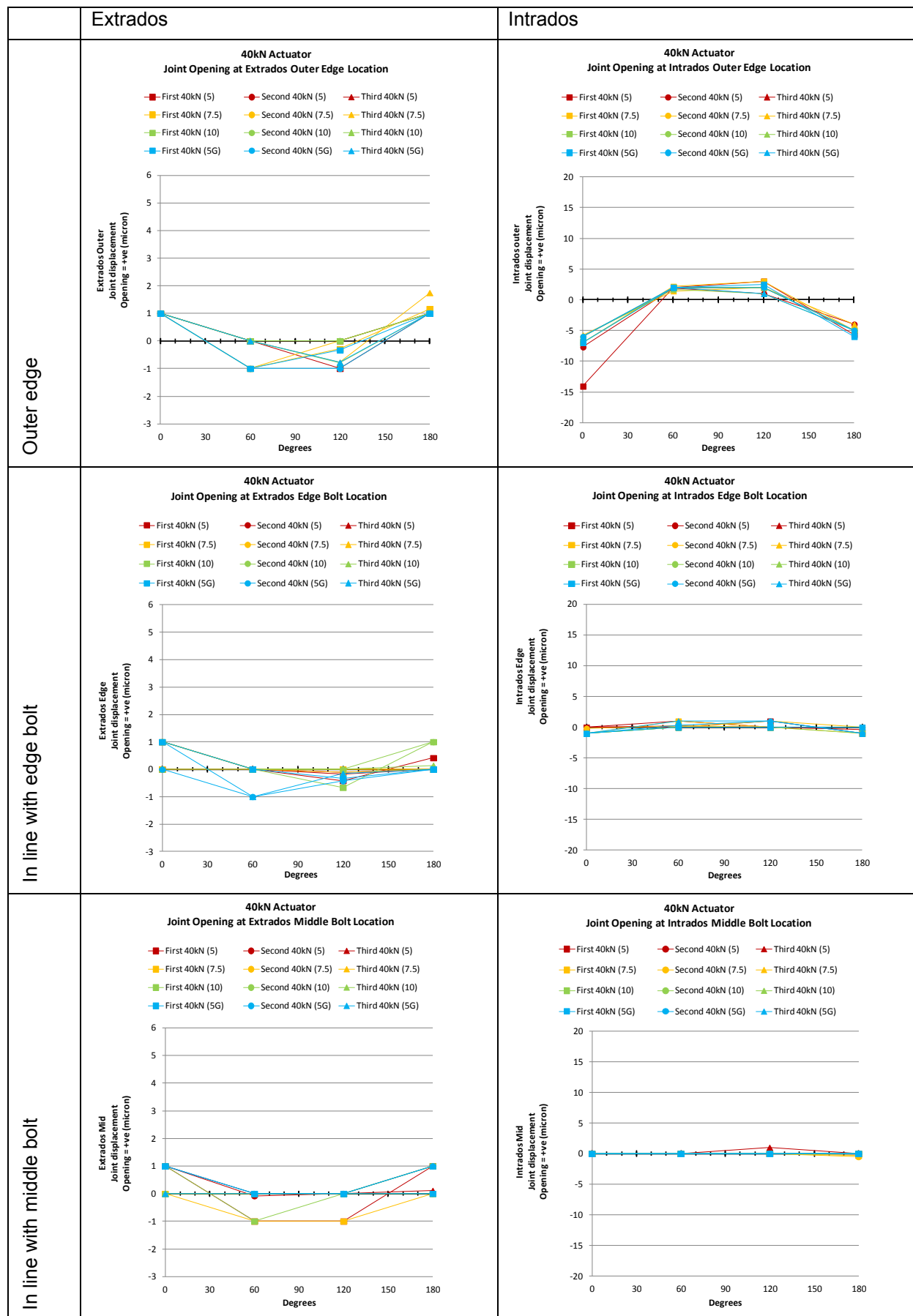


Figure 7-27. Loading case. Local displacement at joint locations. All-round normal load prior to distortion = 40 kN.

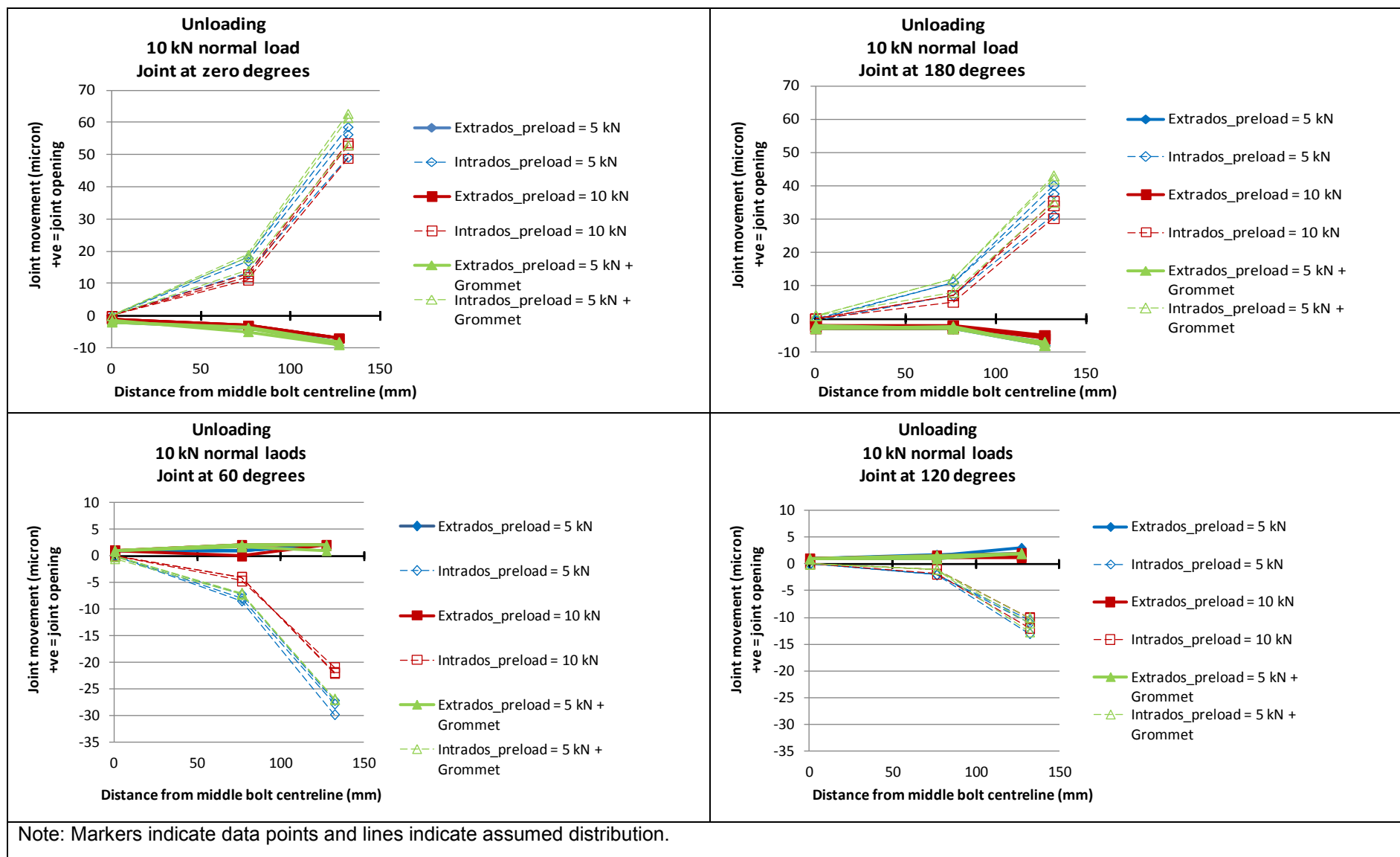


Figure 7-28. Joint movement along intrados and extrados for different bolt preloads. Normal actuator load = 10 kN.

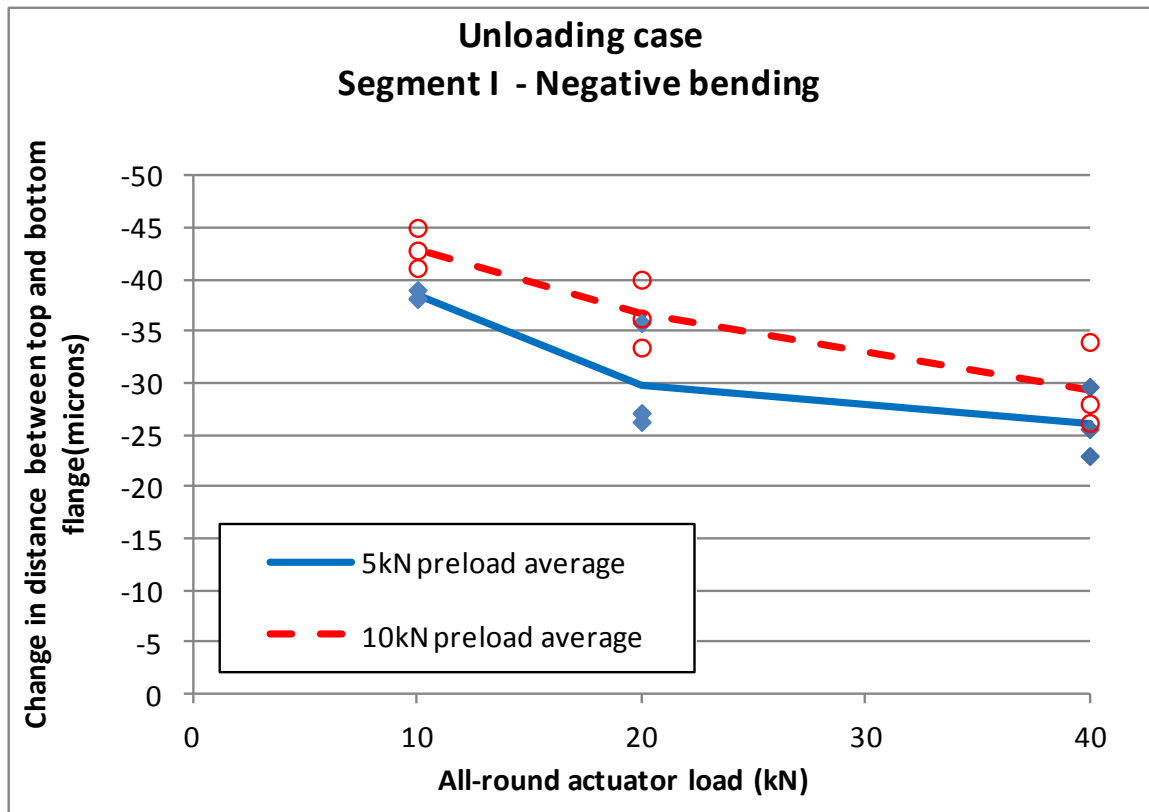


Figure 7-29. Unloading case. Change in distance between circumferential flanges.

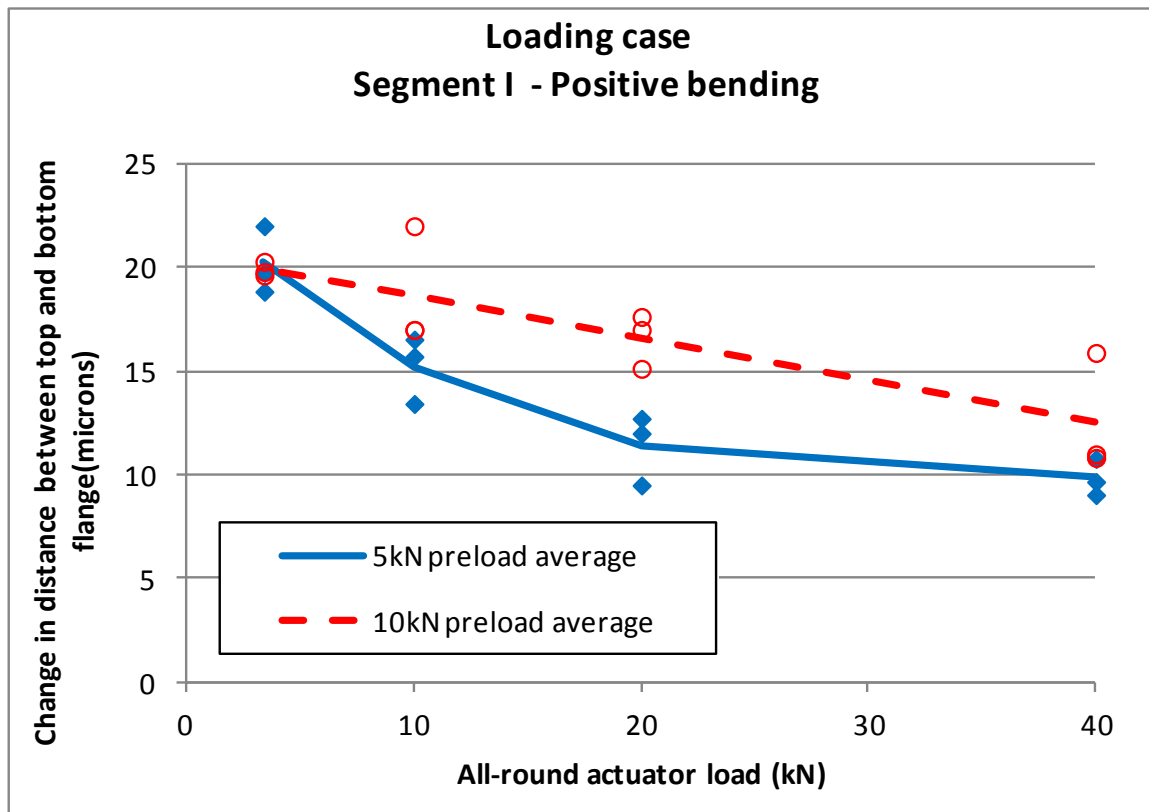


Figure 7-30. Loading case. Change in distance between circumferential flanges.

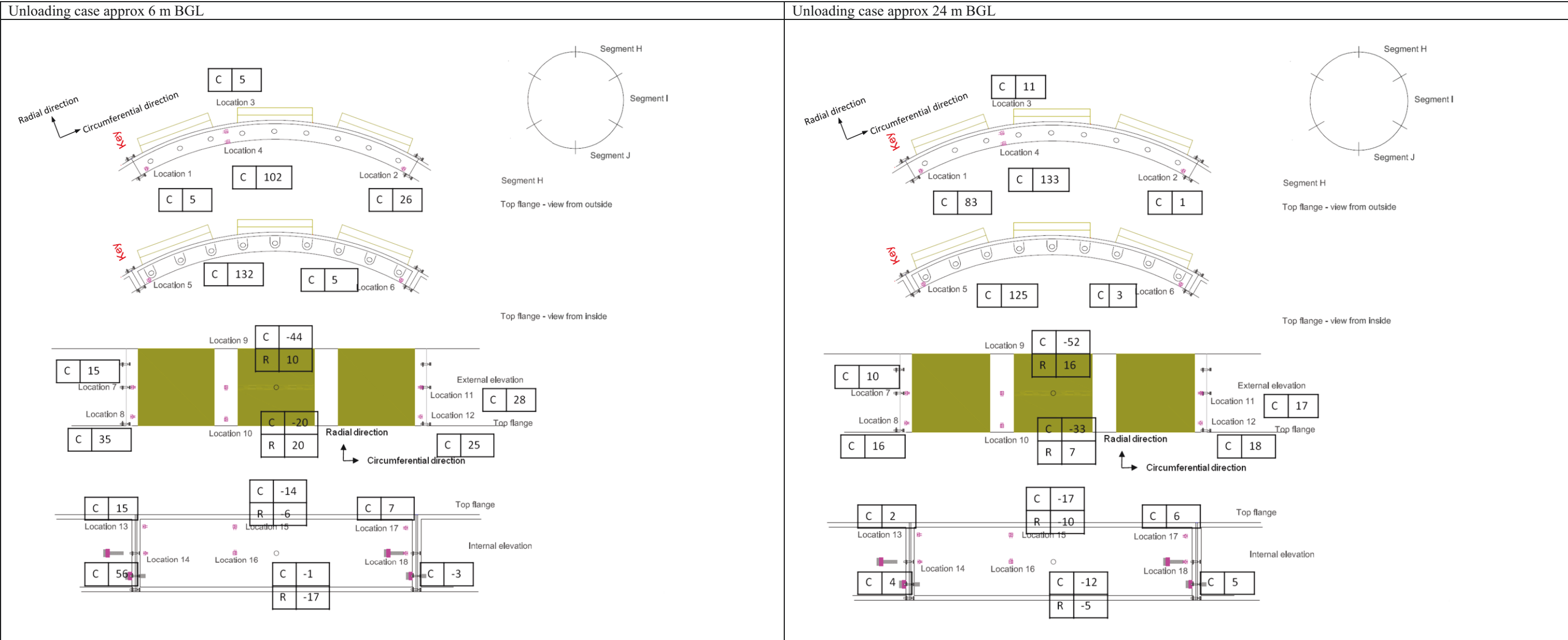


Figure 7-31. Comparison of strains on Segment H. Tensile strains are positive. C=circumferential strain. R = radial strain. Tensile strains are positive.

References

- Angus, H.T. (1976) *Cast Iron: Physical and engineering properties*. Butterworth and Co. Ltd, London, England, 1976.
- Attewell, P. B. & El-Naga, N. M. A. (1977) Ground-lining pressure distribution and lining distortion in two tunnels driven through stiff, stony/laminated clay. *Ground Engineering*, 10 (3), 28-35.
- Avgerinos, V. (2014) *Numerical investigation of tunnelling beneath existing tunnels*. PhD Thesis. Imperial College London.
- Barratt, D. A., O'Reilly, M. P. & Temporal, J. (1994) London. Long-term measurements of loads on tunnel linings in overconsolidated clay. *Tunnelling '94*. London, Chapman and Hall. pp. 469-481.
- Bates, W. (1984) *Historical structural steelwork handbook. Properties of U.K. and European Cast Iron, Wrought Iron and Steel Sections including Design, Load and Stress Data since Mid 19th Century*. The British Constructional Steelwork Association Ltd. London.
- Bilotta, E., Russo, G. & Viggiani, C. (2006) Ground movements and strains in the lining of a tunnel in cohesionless soil. In: Bakker, K. J., Bezuijen, A. & Broere, W. (eds.) *Geotechnical aspects of underground construction in soft ground: Proceedings of the 5th International Conference of TC28 of the ISSMGE*. London, Taylor Francis Group. pp. 705-710.
- Bowers, K.H. & Redges, J.D. (1996) Discussion: Observations of lining load in a London Clay tunnel. In: Mair, R.J., Taylor, R.N. (eds) *Proceedings of the International Symposium on Geotechnical aspects of underground construction in soft ground*. A A Balkema Publishers. p.335.
- Bowers, K.H. Principal Tunnel Engineer at London Underground (Personal communication, 25th August, 2011)
- BS 1452:1961. *British Standard Specification for flake graphite cast iron*. (1961) London, British Standards Institution.
- BS 18:1987. Method for tensile testing of metals (including aerospace materials). (1987) London, British Standard Institution.
- BS 5950-1:2000. *Structural use of steelwork in building. Code of practice for design. Rolled and welded sections*. (2001) London, British Standard Institution.
- BS EN 10002-1:2000. *Tensile testing of metallic materials. Method of test at ambient temperature*. (2001) London, British Standard Institution.
- BS EN 10025-1:2004. *Hot rolled products of structural steels — Part 1: General technical delivery conditions*. (2004) London, British Standards Institution.
- BS EN 1561: 1997. *Founding – Grey cast irons*. (1997) London, British Standards Institution.
- BS EN 1561: 2011. *Founding – Grey cast irons*. (2011) London, British Standards Institution.

- BS EN 1993-1-8:2005. *Eurocode 3 – Design of Steel Structures – Part 1-8: Design of Joints*. (2006) London, British Standard Institution.
- BS EN ISO 6892-1:2009. *Metallic materials. Tensile testing. Method of test at ambient temperature*. (2009) London, British Standard Institution.
- Carey, E.G. (1896) Cast Iron Segments for Railways and other Tunnels. Paper read before the Institution of Engineers and Shipbuilders in Scotland. *Industries and Iron* 1896, pp. 86-89.
- Cooley, P. (1982) Wedge-block tunnels in water supply. *Journal of the Institution of Water Engineers and Scientists*, 36 (February), 9-29.
- Cooling, L. P. & Ward, W. H. (1953). Measurements of loads and strains in earth supporting structures. *Proceedings of the third International Conference on Soil Mechanics and Foundation Engineering*. Zurich. pp. 162-166.
- Cooper, M. L., Chapman, D. N., Rogers, C. D. F. & Chan, A. H. C. (2002) Movements in the Piccadilly Line tunnels due to the Heathrow Express construction. *Géotechnique*, 52 (4), 243-257.
- Cooper, M.L. & Chapman, D.N. (2000) Settlement, rotation and distortion of Piccadilly Line tunnels at Heathrow. In: O. Kusakabe, K. Fujita & Y. Miyazaki (eds.) *Geotechnical Aspects of Underground Construction in Soft Ground*, Rotterdam Balkema, pp. 213-218.
- Cooper, M.L. (2001) *Tunnel-induced ground movements and their effects on existing tunnels and tunnel linings*. PhD thesis. The University of Birmingham.
- Cooper, M.L., Chapman, D.N., Chan, A.H.C. & Rogers, C.D.F (2003) prediction of effects of new tunnelling on existing tunnels. In: Jardine, F.M. (eds) *Response of buildings to excavation-induced ground movements*. CIRIA SP199, London, pp.233-243.
- Copperthwaite W.C. (1906). *Tunnel shields and the use of compressed air in subaqueous works*. Archibald Constable & Co Ltd., London.
- Craig R.N. and Muir Wood A.M. (1978). A review of tunnel lining practice in the United Kingdom. *TRRL Supplementary Report 335*, Transport Road Research Laboratory.
- Curtis, D.J (1976) Correspondence on Muir Wood (1975) *Géotechnique*, Volume 26, pp231-7
- Davies, H.R. & Bowers, K.H. (1996) Design and installation of special tunnel rings to monitor long term ground loading. In: Mair, R.J., Taylor, R.N. (eds) *Geotechnical aspects of underground construction in soft ground: Proceedings of the International Symposium*. A A Balkema Publishers, pp. 257-261.
- Design Consultant Framework Contract C122 – Bored Tunnels. (2011) *Assessment of ground movement effects: LU/05 Central Line east of Lancaster Gate*. Revision 4.0 Report number: C122-OVE-C2-RAN-B071_WS077-00001.
- Dimmock, P.S. (2003) *Tunnelling-Induced Ground and Building Movement on the Jubilee Line Extension*. PhD thesis. Cambridge University.

- Duddeck, H. and Erdman, J. (1985) On Structural Design Models for Tunnels in Soft Soil. *Underground Space*, vol. 9, No. 5-6, pp 246 - 253.
- ETH Zurich. (2013a) *Results from the measurements of the strains development along the London Underground tunnel circumference during the passage of the Crossrail TBM by means of fibre optics*. Report number: Draft.
- ETH Zurich. (2013b) *Results from the measurements of the strains development along the London Underground tunnel longitudinal axis during the passage of the Crossrail TBM by means of fibre optics*. Report number: Draft.
- Fallon, M.J. (1998) *Evaluation of the Properties and Integrity of Six Tunnel Segments and a Bridge Support Section used by London Underground*. The Castings Development Centre. March 1998.
- Field, C.F. (2001) JLE construction works at London Bridge station. In: Burland, J. B., Standing, J. R. & Jardine, F. M. (eds.) *Building response to tunnelling. Case studies from the Jubilee Line Extension, London, Projects and Methods*. CIRIA and Thomas Telford. London. pp. 229-248.
- GCG (1999) *Tunnel Lining Study*. Revision 0. June 1999.
- GCG (2002) *The Assessment of Running Tunnel Linings. Prepared for INFRACO JNP Ltd. Draft Final Report*. August 2002. Tube Lines Document Reference: GCG-L001-N369-DTAAWP1-TUN-RPT-00001.
- GCG (2009a) *Geotechnical Sectional Interpretative Report 1&2: Royal Oak to Liverpool Street Volume 1: Text. Revision A*. January 2009. Report No. 1D0101-G0G00-00549.
- GCG (2009b) *Geotechnical Sectional Interpretative Report 1&2: Royal Oak to Liverpool Street Volume 2: Figures. Revision A*. January 2009. Report No. 1D0101-G0G00-00550.
- GCG (2009c) *Geotechnical Sectional Interpretative Report 1&2: Royal Oak to Liverpool Street. Volume 3: Drawings. Revision A*. January 2009. Report No. 1D0101-G0G00-00551
- Gilbert, G. N. J. (1977) *Engineering Data on Grey Cast Iron – SI Units*. Alvechurch, BCIRA. Report number: Report 1285.
- Groves G.L. (1943). *Tunnel linings with special reference to a new form of reinforced concrete lining*. Proc. Inst. Civ. Engrs. Vol. 20, No. 5, March, pp 29-64.
- Halcrow (2008) *Northern Line cross passages/staircases long sections through staircase No.10*. Drawing no. HAG-N105-8742-TUN-D-SEC-5-03426 Rev 00. Dated 15/12/2008.
- Haswell (1999) *Report on Assessment of Parameters for Estimating Stress*. Report No. B3505-1253-02. August 1999.
- Hewitt B.H.M. and Johannesson S. (1922). *Shield and compressed air tunnelling*. McGraw, London.
- HSE (2000). *The collapse of NATM tunnels at Heathrow Airport. A report on the investigation by the Health and Safety Executive into the collapse of New Austrian Tunnelling Method (NATM) tunnels at the Central Terminal Area of Heathrow Airport on 20/21 October 1994*. Health and Safety Executive.

- Kimance, J. P., Lawrence, S., Hassan, O., Purchase, N. J. & Tollinger, G. (1996) Observations of deformations created in existing tunnels by adjacent and cross cutting excavations. In: Mair, R. J. & Taylor, R. N. (eds.) *Geotechnical Aspects of Underground Construction in Soft Ground: Proceedings of the International Symposium*. Taylor Francis Group. Rotterdam. pp. 707-712.
- London Underground. (2007) *Civil Engineering - Deep Tube Tunnels and Shafts. Standard. A1*. Report number: 1-055.
- Lyons, A. C. & Reed, A. J. (1974) *Modern Cast Iron Tunnel and Shaft Linings*. 1974 RETC (Rapid Excavation and Tunnelling Conference) Proceedings. pp.669-689.
- Mair, R. J. & Jardine, F. M. (2001) Tunnelling methods. In: Burland, J. B., Standing, J. R. & Jardine, F. M. (eds.). *Building response to tunnelling. Case studies from the Jubilee Line Extension, London, Project and methods*. London, CIRIA and Thomas Telford. pp. 127-134.
- Mair, R. J. (2008) Tunnelling and geotechnics: a new horizon. *Géotechnique*. 58 (9), 695-736.
- Morgan, H. D. (1961) A Contribution to the Analysis of Stress in a Circular Tunnel. *Géotechnique*. Volume 11, Issue 1, pp 37-46.
- Moss N.A. and Bowers K.H. (2006). The effect of new tunnel construction under existing metro tunnels. *Proc. Int. Symp. Geotechnical Aspects of Underground Construction in Soft Ground*, Bakker et al. (eds). Taylor Francis Group, London, pp. 151-157.
- Muir Wood, A.M. (1975) The circular tunnel in elastic ground, *Géotechnique*, Volume 25, Issue 1, pp 115-27.
- Nyren, R. J. (1998) *Field measurements above twin tunnels in London Clay*. PhD. Imperial College, University of London.
- Oyenuga, O. (2013) Data Controller IMDS C701 at Crossrail Ltd. (Personal communication, 15th February, 2013).
- Rapp, G. M. & Baker, A. H. (1936) The measurement of soil pressures on the lining of the Midtown Hudson Tunnel. *Proceedings of the 1st International Conference on Soil Mechanics and Foundation Engineering*. Harvard. pp. 150-156.
- Russell Ductile Castings (2010) *Test Certificate No. 36959*. Dated 17 March 2010.
- Sabine, G. D. & Skelton, E. (1985) Extension of the Piccadilly Line to Terminal 4, Heathrow. *Proceedings of the Institution of Civil Engineers*. 78 (6), 1261-1278.
- Sandberg LLP. (1990) *Test Results for Bolts from Existing Tunnel at Angel*. Facsimile dated 14 December 1990. Client: J. Murray and Sons Ltd.
- Scholes, J. P. (1979) *The selection and use of cast irons*. Oxford University Press, Oxford.
- Skempton, A. W. (1943) Discussion. Tunnel linings with special reference to a new form of reinforced concrete lining. *Proceedings of the Institution of Civil Engineers*, 20 (5), 53-56.

- Smith Osborne, R. (1970) Discussion: The Victoria Line. *Proceedings of the Institution of Civil Engineers.*, Supplement xiii (Paper 7270S), 319-322.
- Standing, J. R. & Selman, R. (2001) The response to tunnelling of existing tunnels at Waterloo and Westminster. In: Burland, J. B., Standing, J. R. & Jardine, F. M. (eds.) *Building response to tunnelling. Case studies from the Jubilee Line Extension, Case studies*. London, CIRIA and Thomas Telford, London. pp. 509-564.
- Sutherland, H. B. (1955). Discussions. *Conference on the Correlation between Calculated and Observed Stresses and Displacements in Structures*. London, Institution of Civil Engineers. pp. 370-486.
- Tattersall, F., Wakeling, T. R. M. & Ward, W. H. (1955) Investigations into the design of pressure tunnels in London Clay. *Proceedings of the Institution of Civil Engineers*, 4 (4), 400-455.
- Thomas, C.H. (1997) *Production and Properties of Grey Cast Iron Tunnel Segments Circa 1900*. The Castings Development Centre. December 1997.
- Thomas, H. S. H. (1974) *Tunnelling in London Clay: A study with field investigations including the long term structural behaviour of two linings*. M.Phil. University of Surrey.
- Thomas, H. S. H. (1976) Structural performance of a temporary tunnel lined with spheroidal graphite cast iron. *Proceedings of the Institution of Civil Engineers.*, 61 (2), 89-108.
- Thomas, H. S. H. (1977) Measuring the structural performance of cast iron tunnel linings in the laboratory. *Ground Engineering*, 10 (5), 29-36.
- Thomas, H. S. H. (1983) Observations on the behaviour of a pilot tunnel in London clay as the main tunnel was driven. *Proceedings of the Institution of Civil Engineers.*, 74 (1), 15-24.
- Timoshenko, S. (1934) *Theory of Elasticity*. McGraw Hill, New York.
- Tube Lines (2006) *Strain Measurement Report. Deep Tube Tunnel Knowledge and Inspection Programme Annual Works Plan 2*. 2. London, Tube Lines. Report number: Document No. TLL-L001-N416-DTAAWP2-TUN-RPT-00015.
- Tube Lines (2007) *Cast Iron Tunnels and Shafts Report. Deep Tube Tunnel Knowledge and Inspection Programme Annual Works Plan 2*. Document No. TLL-L001-N416-DTAAWP2-TUN-RPT-00002. Revision 4.0 dated 03/03/2007.
- Tube Lines (2008) *Cast Iron Coring Report. Deep Tube Tunnel Knowledge and Inspection Programme Annual Work Plan 2*. Revision 4 issued on 22/09/2008. Document No. TLL-L001-N416-DTAAWP2-TUN-RPT-00019.
- Underground Professional Services. *Drawing number UnPS/ICR/GN/D/0004 revision C1. Half size model of 12'6" cast iron tunnel lining*. (20/11/2009).
- Wan, M.S.P. (2014) *Field monitoring of ground response to EPBM tunnelling close to existing tunnels in London Clay*. PhD Thesis. Imperial College London.

Ward, W. H. & Chaplin, T. K. (1957). Existing stresses in several old London Underground tunnels. *Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering*. London. pp. 256-259.

Ward, W. H. & Thomas, H. S. H. (1965). The development of earth loading and deformation in tunnel linings in London Clay. *Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering*. Montreal, University of Toronto Press. pp. 432-436.

Winterton, T. (1999) Lining trends in the UK. *Tunnels and Tunnelling International*. January 1999. pp. 44-46

Young, W. C. & Budynas, R. G. (2002) Curved Beams. In: *Roark's Formulas for Stress and Strains*. 7th edition. New York, McGraw Hill Companies, Inc. pp. 267-380.

Appendix

A. Earth pressure acting on tunnel lining

Rapp and Baker (1936)

Rapp and Baker (1936) provided the results of the monitoring works undertaken on the Midtown Hudson Tunnel in New York, USA. The tunnel was about 9.5m in diameter, and was lined with bolted cast iron segments under compressed air. The tunnel was constructed in the formation known as Hudson River Silt with the tunnel crown approximately 8m to 9m below the river bed. Radial pressure acting on the lining was measured using hydraulic pressure gauges housed in 'plugs' which were inserted at 2m centres around the circumference of the ring immediately after lining installation.

When the internal air pressure was zero, the measured radial pressure on the lining ranged from 100% of the overburden pressure at the crown to 72% of the overburden pressure at the invert. This was approximately 2 months after the segments were installed.

Skempton (1943)

Skempton (1943) reported the measurement of earth pressures acting on a 3.66m internal diameter bolted cast iron tunnel using Whittemore strain gauges. The tunnel crown was approximately 31.0m below ground and the tunnel was entirely in London Clay. Tunnelling was carried out without the use of a shield. A second tunnel with the same diameter ran parallel to the instrumented tunnel. The distance between the external diameters was 1.5m.

Monitoring of the instrumented ring began 10 days after the adjacent tunnel was driven. Measurements were taken on the intrados of the two circumferential flanges and on the inside of the skin in the middle of the segment. A second set of readings were taken three days later and subsequently readings were taken over the following six week.

The stress was calculated from the strain measurements assuming a Young's modulus of approximately 100 GPa. It was found that the strains measured in the leading flange, the skin and the trailing flange were different, and an average value was taken in the calculation of the stress. It was assumed that the bending was negligible. Based on these assumptions, Skempton (1943) noted that it took a period of fourteen days for the full overburden pressure to be exerted on the tunnel.

The accuracy obtained by the Whittemore strain gauge was 0.3 ton/in², or approximately 4.5MPa.

Cooling and Ward (1953)

Cooling and Ward (1953) described an investigation which commenced in 1952 to measure the stresses in the cast iron linings of a group of 7.6m (25 ft) diameter tunnels with axis level 30.5m (100ft) below ground level in London Clay. One ring in each of the two adjacent tunnels was instrumented with vibrating wire gauges to measure the strains in the skin and circumferential flanges. The average stress was calculated assuming only plane bending took place parallel to the skin and a Young's modulus of 117 GPa (17×10^6 lb/in²). In certain segments the hoop stress corresponding to full

overburden pressure was reached 3 months after installation, in other segments the calculated compressive stress was 60% of the overburden pressure after a period of 3 months.

The accuracy of the vibrating wire gauges was 0.03 tons/in^2 , or about 0.5MPa, in compression.

Sutherland (1955)

Sutherland (1955) gave details of the instrumented tunnel constructed under the River Clyde in Glasgow. It was 3.66m (12ft) in diameter and constructed using grey cast iron. The upper face of the tunnel was in sand and the lower part was in very stiff clay and shale. At one location twelve vibrating-wire strain gauges were mounted on each of three consecutive rings, and at a second location twelve gauges were mounted on each of two consecutive rings. The gauges were mounted on the segments at ground surface and the segments were built in the tunnel under compressed air.

For a period of 30 hours, hourly readings were taken on all the gauges to measure stresses in tunnel segments due to tidal variations. The effects of tightening and loosening the bolts in the segments, as well as stresses induced by driving lead caulking between the segments, were measured. It was found that the caulking would induce substantial stress in the tunnel lining.

Tattersall, Wakeling and Ward (1955)

Tattersall, Wakeling and Ward (1955) investigated the ground pressure acting on a hydraulic pressure tunnel at Ashford Common in Middlesex, UK. The ground pressure acting on the tunnel lining was measured using both vibrating wire load gauges and hydraulic pressure gauges. Vibrating wire load gauges were also used to measure the hoop load in the lining, but due to the method of installation the load gauges were only installed in the lower half of the tunnel.

The use and interpretation of the hydraulic pressure gauge proved to be difficult. The vibrating wire load gauges measured radial pressures between 96kPa to 479kPa. When the vibrating wire load gauges were used to measure hoop load, the radial pressures calculated from the hoop loads were between 269kPa and 288kPa. Full overburden pressure was approximately 540kPa.

It was observed that within one month of erecting a segment, water from the clay began to appear in places on the roof segments.

Ward and Chaplin (1957)

Ward and Chaplin (1957) reported the measurement of the existing stresses in several London Underground tunnels. Vibrating wire strain gauge measurements were taken first when the surrounding clay was completely removed and subsequently when the linings were dismantled. In this way the stresses due to earth loading and that due to bolting and dismantling were measured. At the fourth site several segments from one ring were removed.

Four strain gauges were fixed to every accessible segment at the three sites (Figure A-1). Two were fixed onto the skin at third points and one on each circumferential flange at bolt level. These tunnels were the typical 3.8m outer diameter running tunnels. At the fourth site one strain gauge was fixed to the centre of the skin and two were fixed to the tip of the circumferential flanges (Figure A-2). This tunnel had an external diameter of 6.9m and consisted of 12 segments and a key.

For the locations where the entire ring was removed, the deduced hoop stresses in the skin and flange differed in magnitude and in several cases also in sign. However, the magnitude of the stress was generally no higher than that due to the full overburden load. Hoop loads substantially higher than that due to overburden were deduced in segments where the adjacent tunnel was less than a diameter apart. There was consistency in the stress distribution in consecutive rings and observations on one ring alone would give a good guide to the general situation.

For the location where only several segments were removed, the stresses deduced in the skin and flanges were higher than that exerted by full overburden. The adjacent tunnels were also less than one diameter apart at this location.

Ward and Thomas (1965)

Ward and Thomas (1965) presented the gradual development of earth loading, the bending moments in tunnel segments and the associated change in the shape of the linings up to six years following construction of two cast-iron lined tunnels and one precast concrete tunnel in London Clay.

One of the cast iron tunnels was constructed in 1957 as part of the Post Office railway system. It had an internal diameter of 2.1m, with six segments bolted together to form one ring. The other cast iron tunnel, built in 1960 as part of the Victoria Line, was of 3.9m internal diameter, and also consisted of six segments. The segments were thrust directly against the clay by jacks inserted at the segment joints at approximately 60° on either side of the invert. Metal wedges were inserted into the joints adjacent to the jacks, which were then removed. The segments were not bolted. The precast concrete tunnel also formed a section of the Victoria Line, and it was formed by expanding the ring directly against the clay by means of a wedge. The concrete tunnel had an internal diameter of 3.8m.

Vibrating wire strain gauges were fitted to the segments (Figure A-3).

The average circumferential stress at axis level of the smaller tunnel was about 75% of the full overburden stress 6 years after construction. For the larger tunnel, the average stress 3.5 years after tunnel construction was close to full overburden value. For the concrete ring, the average circumferential stress in the lining after a lapse of approximately 21 months was 65% of the theoretical overburden pressure.

Smith Osborne (1970)

Smith Osborne (1970) described some of the investigations and instrumentation Halcrow carried out on the expanded concrete rings of the Victoria Line in the 1960s. The positions of contact in the joints were measured using strain gauges. It was shown that the bending strains resulted from the positions of contact in the joints, i.e., the bending strains were determined by the building procedure rather than by the ground loading. Photo-elastic stress gauges were also installed in the concrete lining. The hoop loads measured at 4 locations in the lining were found to be between 42% and 62% of the overburden load a year after the construction of the lining.

Expanded steel linings were used at Oxford Circus in a station tunnel and at King's Cross in a station tunnel and in the concourse. These linings were expanded directly against the clay with no grouting

space to take up any lack of fit. Several segments were instrumented for measurement with a Demec gauge before installation. The gauge points were set on the intrados of the circumferential flanges and on the inside faces of the skin (Figure A-4). Readings were taken three times after construction. The bending strains measured four months after construction were similar to those measured shortly after installation. It seemed that the bending strains resulted from the installation process rather than from ground loading. It was also noted that the gauges on the skin measured negligible strains or very small compressive strains, while the gauges on the circumferential flanges measured tensile strains. Higher tensile strains were measured on the lower half of the ring.

Thomas (1976)

Thomas (1976) reported hoop stress measurements of the Victoria Line tunnel taken during the Victoria Line extension to Brixton. Local strains of the segments were measured using pairs of vibrating wire strain gauges located near the middle of the circumferential flanges on the inside face (Figure A-5).

It was observed that during the installation of the 12 segment SGI tunnel, the bottom eight segments were well fitted to the tail plate of the shield but the top four segments and the key were built as an arch springing from the shoulder joint faces of the segments supported by the shield. These faces were inclined at 1.5° more than their correct angle.

The hoop stress was taken to be the average stress calculated from strain measurements from each pair of gauges on a flange. The results for individual flanges varied considerably, sometimes being compressive in one flange of a segment and tensile in the other.

Prior to the second tunnel passing, approximately 5 weeks after installation of the instrumented rings in the first tunnel, the hoop stress in the ring corresponded to 45% of the overburden pressure. Nearly all the disturbances of the experimental rings in the first tunnel occurred when the face of the second tunnel was between one tunnel diameter behind and one diameter ahead of each ring. An increase in hoop stress equal to about 10% of the overburden pressure (or 23% of the hoop stress reached before the passing of the second tunnel) corresponded with the passing of the second pilot tunnel.

Attewell and El-Naga (1977)

Attewell and El-Naga (1977) provided information on the ground pressures acting on two 3.2m diameter tunnels at depths of 13.4m and 14.0m. The earth pressures were measured using hybrid electro-hydraulic type earth pressure cells (Figure A-6). The tunnels were driven by hand using clay spades without a shield through stiff, stony, laminated clay and lined with bolted segmental concrete linings.

The pressure cell is installed through a pre-formed grout hole in the lining segment, with the sensing diaphragm in direct contact with the soil. The cells were installed at six locations in the rings being monitored. The rings were monitored for 50 days after installation.

Measurements showed that the radial pressure developed first at the crown and invert and then progressively in the side walls as squatting of the lining mobilised lateral passive pressures. The

distribution of pressure after 50 days was essentially isotropic, implying that the lining was subjected to uniform hoop stress and was not required to support substantial bending moment. Pressure changes in the monitored rings ceased approximately 8 days after installation. The maximum pressure measured was almost 50% of the theoretical overburden pressure.

Cooley (1982)

Cooley (1982) presented the short-term hoop loads measured from instrumented Donseg and Wedge-Block rings of the Ashford Common, Thames-Lee, Queen Elizabeth II reservoir and Staines-Kempton water tunnels. The internal diameters of the tunnels ranged from 2.54m to 2.73m and they were all constructed in London Clay at depths of 19m to 54m below ground level. The hoop loads were measured using vibrating wire type load gauges at 12 months or thereabouts after building of the tunnel ring. The average short-term hoop loads were between 45% and 67% of the overburden load.

Long-term data were collected from the Thames-Lee tunnel over a period of eight years at five locations and over seventeen years at four of the five locations. The hoop load after eight years was greater than or equal to 45% of the overburden. The maximum recorded hoop load after seventeen years was 89% of the overburden load.

Thomas (1983)

Thomas (1983) reported long-term measurements from the Victoria Line tunnels described in Thomas (1976) and the effects on the instrumented pilot tunnel when a larger diameter tunnel was constructed by breaking out the pilot tunnels and encircling them. The instrumented pilot tunnel had been in the ground for about 15 years. Strain gauges showed that bending and hoop strain changes had virtually ceased approximately 7 months after the pilot tunnel was constructed. The value of the hoop strains reached in the instrumented pilot tunnel 7 months after its construction corresponded to 45% of the overburden.

As the 9m internal diameter crossover tunnel approached, measurements showed that the horizontal diameter of the instrumented pilot tunnel reduced and its vertical diameter increased. The hoop stress in the pilot tunnel also increased from the time the crossover tunnel face was 23-26m away, which was approximately the depth to the invert of the crossover tunnel. When the crossover tunnel was less than 4.6m away, the hoop stress in the pilot tunnel began to drop until it was completely unloaded.

Thomas suggested that the measured behaviour was due to the locked-in stresses in the London Clay. It was noted that stresses induced in the first pilot tunnel from the second pilot tunnel passing were not released until the crossover tunnel was very close to the instrumented ring in the first pilot tunnel.

Barratt et al (1994)

Barratt et al (1994) reported the results of long-term monitoring of loads developed in the instrumented rings of the segmental concrete linings in two tunnels. One tunnel was the northbound Jubilee Line tunnel between Baker Street and Bond Street station beneath Regent's Park which was erected in 1974. The depth to the tunnel axis was about 20m and the surrounding ground was London Clay. The

second tunnel was the Oxford Trunk Outfall Sewer which was constructed in 1981. The depth to the tunnel axis was about 15.5m and the surrounding ground was Oxford Clay.

For the instrumented ring of the Jubilee Line tunnel, the segments were cast in standard moulds and the instrumentation was fixed to the shuttering prior to casting. The instrumented ring was installed in the same way as a standard ring. The instrumentation consisted of load cells to measure the total loads in the ring and earth pressure cells and Glotzl cells to measure ground pressure around the ring (Figure A-7). Vibrating wire strain gauges were also embedded into the concrete segments. The Glotzl cells were damaged at an early stage and the interpretation of the vibrating wire strain measurements proved difficult and tenuous. Data from these two types of instruments were not presented in the paper.

The load cell was of the single vibrating wire type and one end of the cylindrical casing was domed to ensure that the load was transmitted axially through it. The effect of each load cell was to introduce a further hinge point at the centre of a segment. The overall stiffness of the instrumented rings was less than 10% lower than for standard rings. The earth pressure cells used a vibrating wire gauge to determine the change in deflection of a 140mm diameter circular diaphragm in contact with the ground.

The instrumented wedge-block ring of the Oxford Trunk Outfall Sewer also contained load cells and earth pressure cells (Figure A-8). The instruments were inserted into the segments during production therefore enabling the installation of the lining to proceed as normal.

Measurements were taken in the Jubilee Line Tunnel over a period of 19.5 years. The hoop load at axis level was built up to about 60% of the load under full overburden. The hoop load at the crown was at 40% of the load under full overburden. The earth pressure cells measured higher loads and indicated pressures up to 100% of the full overburden and slightly in excess of the calibrated range for the instruments. The earth pressure cells were still showing an upward trend, whereas the loads cells were trending towards asymptotic values.

Measurements were taken in the Oxford Trunk Outfall Sewer over a period of 7.5 years. Loads in the lining had built up erratically to between 35% and 50% of the load under full overburden. The earth pressure cells recorded negative pressures up to three months after installation; with final pressures being 5% to 30% of the total overburden value. Barratt et al (1994) discussed the difficulty in mounting the flat face of the earth pressure cell against the curved surface of the tunnel ring segment. Contact with the ground was unlikely to be uniform and the calibration would be different to that observed in the laboratory. It was suggested that direct measure of load was a more reliable method for tunnel linings.

Nyren (1998)

Nyren (1998) reported the monitoring data for the lining loads in the eastbound tunnel for the Jubilee Line Extension project between Green Park and Westminster. The monitoring was conducted by the Transport Research Laboratory (TRL) through two instrumented rings as described by Davies and Bowers (1996). One ring comprised vibrating wire strain gauges fitted to the reinforcement cage of

four segments (Figure A-9). The adjacent ring had four load cells fitted with a gap between adjacent segments so that all hoop stresses were transferred to the load cells (Figure A-10).

The lining loads measured by the load cells in the eastbound tunnel rings recorded an initial jump due to the lining expansion, then increased steadily to reach the equivalent of 40% and 50% of the total vertical overburden load after 100 days. After 1 year, the vertical loads, measured by load cells at axis level, increased to 60% of the vertical overburden stress. The horizontal loads measured at the crown and invert had only increased to 45% of the vertical overburden stress. It was noted by Bowers and Redgers (1996) that the initial ring expansion caused localised asymmetric loading which persisted over time.

Dimmock (2003)

Dimmock (2003) took readings of the load cells installed in the Jubilee Line Extension eastbound tunnel 7 years after lining installation. The load cells at the axis level were measuring vertical loads equivalent to 75% and 90% of the vertical overburden load. The load cells at the crown and invert were measuring horizontal loads between 55% and 60% of the overburden load. The ratio of the horizontal to vertical loads was approximately 0.75.

Tube Lines (2006)

As part of the Deep Tube Tunnel Knowledge and Inspection Programme Annual Works Plan 2, Tube Lines (Tube Lines, 2006) produced a report of the development and validation of strain measurement techniques that rely on the in-situ removal of cast iron samples by coring.

Most measurement techniques measure changes in strain in order to calculate the stress. In a tunnel lining a change in strain occurs if a section of the tunnel is removed. Theoretically, if that section of tunnel is instrumented, the change in strain can be measured and will give an indication of the in-situ stress of the tunnel section.

The basic technique employed by Tube Lines was to place instrumentation on a section of the skin of the cast iron, collect the base readings, then coring out the section of pan on which the instrumentation was fitted (Figure A-11 and Figure A-12). Measurements made after the coring process was compared with the initial measurement and the change was a measure of the in-situ strain within the pan of the segments forming the lining.

Both the dual vibrating wire (VW) gauge and the digital Demec gauge were selected to provide strain measurements. The VW strain gauges were installed on rings of the disused Piccadilly running tunnel between Holborn and Aldwych and on rings of the Aldwych station tunnel. Even after correction for residual stresses from the casting process, the measured changes in strains in the cores gave stresses that were erratic, unexpectedly high (180% of overburden pressure) or negative. Demec gauges were installed at the bottom of lift shafts at Hampstead station, within the ventilation passageways at Kennington Station and in the cable shafts at Stockwell. However, the Demec gauges were not able to provide consistent or repeatable results. The device was not able to measure strain changes at the ten thousandth of a millimetre required.

The Tube Lines report concluded that alternative methods should be investigated for the measurement of in-situ strain.

Bilotta et al (2006)

Bilotta et al (2006) described the monitoring done on the newly constructed segmental concrete lining of the Naples metro extension in Italy. The tunnel was excavated using Earth Pressure Balance TBM in cohesionless soils below the water table. Vibrating wire strain gauges were embedded into the concrete segments during casting (Figure A-13). There were five strain gauges in each instrumented segment, four aligned perpendicular to the tunnel axis and one aligned parallel to the tunnel axis which was used as a dummy gauge.

Each segment was individually calibrated in the laboratory in a triangular loading frame. Normal forces were applied to the end faces of the segment by means of two hydraulic jacks. The forces were balanced by two reactions on the convex surface of the segment. All tested segments were installed in the tunnel. The calibration information was used to deduce the hoop force and bending moments in the segments after installation in the tunnel. The values of circumferential force were obtained using the increment of strains measured after the pushing stage was completed and the soil was acting on the ring. The strains stabilised about one week after the installation of the segments.

B. Deformation of existing tunnels due to adjacent excavations

The Jubilee Line Extension Project at Waterloo

The response of existing tunnels including the Shell Centre cooling tunnels, and the Bakerloo and Northern Line tube tunnels were monitored during the construction of the Jubilee Line Extension (JLE) tunnels at Waterloo (Standing and Selman, 2001). The Shell Centre cooling tunnels were constructed in the late 1950s and were most likely constructed by conventional open-face shield techniques. The Bakerloo Line tunnels were excavated by hand between 1913 and 1915 using shield tunnelling techniques with an open face. The Northern Line tunnels were constructed in the 1890s using conventional open-face shield techniques. Construction of the 5.6m outer diameter JLE tunnels was carried out using the SCL method at this location (Mair and Jardine, 2001). All the tunnels were located within London Clay. More details of the tunnels are given in Table A-1.

Monitoring of the Shell Centre water cooling tunnel, the Bakerloo Line station tunnels and the Northern Line station and running tunnels was performed using precise levelling and tape extensometer measuring techniques. Typical spans measured within sections of existing tunnels using a tape extensometer are illustrated in Figure A-14.

Monitoring data from Standing and Selman (2001) showed that the existing tunnel rings tended to elongate vertically into an 'egg' shape when located directly above the centreline of the newly constructed tunnel. However, beyond 5m to 10m away from the centreline of the new tunnel the existing tunnels tended to elongate horizontally, i.e. to 'squat' rather than 'egg'. The measured deformations of the existing tunnels are presented in Table A-1. The smaller diameter bolted SGI Shell

Centre cooling tunnel gave a maximum strain of $500\mu\epsilon$. The London Underground tunnels were made from bolted GCI rings and the maximum strain recorded was $325\mu\epsilon$.

The Jubilee Line Extension Project at London Bridge

The construction works associated with the Jubilee Line Extension project at the London Bridge area was described in detail by Field (2001). Apart from the works for the new Jubilee Line station, there was also major upgrade works of the Northern Line passenger facilities and the interchange with the National Railway station. A three-dimensional plan of the JLE London Bridge station is given in Figure A-15.

Kimmance et al (1996) reported on the following interaction between the new excavations works and the existing underground infrastructure:

1. Northern Line works consisting of a new station tunnel being constructed adjacent to the existing cast iron segmental station tunnels and cross passages. Refer to Figure A-16 for geometry.
2. JLE westbound and eastbound running and station tunnels using SCL techniques, undercutting the Northern Line existing and new tunnels. Refer to Figure A-17 for geometry.

In creating a new station and central concourse arrangement for the Northern Line, a new 8m diameter SGI station tunnel was excavated parallel to and 5.8m away from the existing 7m diameter grey cast iron station tunnel (Figure A-16). Both tunnels were in London Clay. The vertical squat in the existing tunnel due to the adjacent excavation was 5 to 7mm, while the corresponding axis spread was 4 to 6mm. In terms of strain over the diametral length this was 0.06% to 0.10% ($571\mu\epsilon$ to $1000\mu\epsilon$).

The JLE westbound and eastbound running tunnels passed under the newly constructed SCL Northern Line tunnels and also the existing segmental cast iron Northern Line tunnels (Figure A-17). There was a skew of approximately 40° between the JLE tunnels and the Northern Line tunnels. Instrumentation including electrolevels, tape extensometers, precise levelling and stress cells in shotcrete linings were placed in the undercut tunnels. As a precautionary measure in the cast iron tunnels, the lining bolts were tightened, and where the rings were capable of independent movement invert concrete was reinforced to provide a single longitudinal unit. The observed deformations in the undercut tunnels are presented in Table A-2.

Appendix

Name	Diameter	Invert Level	Lining Type	Clearance to new tunnel ¹	Max. Settlement of existing tunnel	Above centreline of new tunnel	
						Max. horizontal strain ²	Sub-vertical strains ³
	(m)	(mBGL)		(m)	(mm)	($\mu\epsilon$)	($\mu\epsilon$)
Shell Cooling Tunnel ⁴	3.0	22	Bolted SGI segment	8.2	11	-400	+500
Bakerloo Line NB ⁵ Station Tunnel	6.8	20	Bolted GCI segment	7.5-7.8	10	-240	Strains too small
Bakerloo Line SB ⁵ Station Tunnel	6.8	20	Bolted GCI segment	10.0-10.3	8	No data	Strains too small
Northern Line NB ⁵ Station Tunnel	6.7	18	Bolted GCI segment	5.4	12	No data	No data
Northern Line NB ⁵ Running Tunnel	3.6	22	Bolted GCI segment	5.5	15	No data	No data
Northern Line SB ⁵ Station Tunnel	6.7	18	Bolted GCI segment	5.3	10	+125	No data
Northern Line SB ⁵ Running Tunnel	3.6	22	Bolted GCI segment	4.9	12	-325	+325

Table A-1. Measurement data from Standing and Selman (2001).

Notes.

1. Clearance equals distance between the invert of the exiting tunnel and the crown of the JLE tunnel.
2. Positive values indicate expansion. Negative values indicate contraction.
3. Tape extensometer measurements between crown and knee locations.
4. The strain measurements for the Shell Centre cooling tunnel were taken over a period of a week during the construction of the new tunnel.
5. The strain measurements for the Bakerloo and Northern Lines were based on 2 sets of measurements – one set before tunnelling and another set more than 2 years later.

Name	Diameter	Invert Level	Lining Type	Clearance to new tunnel	Max. Settlement of existing tunnel	Above centreline of new tunnel	
						Max. horizontal deformation	Sub-vertical strains
	(m)	(mBGL)		(m)	(mm)	(mm)	(mm)
Northern Line SCL tunnel	5.35	28	Sprayed concrete	2.0	10.5	7	7
Northern Line SB Running Tunnel	3.25	21	Bolted GCI segment	0.8	20.0	5	8
Northern Line NB Running Tunnel	3.25	21	Bolted GCI segment	0.4	15.0	5	8

Table A-2. Measurement data from Kimmance et al (1996).

Thameslink 2000 Project

Mair (2008) reported the use of Brillouin optic time domain reflectometry (BOTDR) fibre optic technology to monitor the response of the 8.5m OD masonry Thameslink Tunnel during the construction of the 6.5m OD Thameslink 2000 Tunnels. The existing Thameslink Tunnel was constructed between 1865 and 1868 using the cut and cover method. The new tunnels were excavated with a backhoe using open-face shield tunnelling technique and lined with concrete segments.

The minimum clearance between the crown of the new tunnel and the invert of the masonry tunnel was 3.6m. The maximum settlement of the masonry tunnel was 35mm. When the new tunnel was directly below the west wall of the masonry tunnel, the maximum tensile strain measured in the masonry was 0.25% (2500 $\mu\epsilon$).

Heathrow Express at Central Terminal Area (CTA)

Distortions in the Piccadilly Line tunnels due to the construction of Heathrow Express tunnels were reported in Cooper and Chapman (2000) and Cooper et al (2002) with further details given in Cooper (2001). The Piccadilly Line running tunnels had an internal diameter of 3.81m and were lined with unbolted concrete linings with convex radial joints. They were located 13m below ground level within London Clay. The tunnels were originally constructed between 1971 and 1976 using hand shields. The vertical clearance between the new and existing tunnels was 7m and there was a skew of 69° between them (**Error! Reference source not found.**Figure A-18).

The new Heathrow CTA concourse tunnel was constructed using SCL techniques at approximately 26m BGL. It had a flattened invert with the outside dimensions being 9.1m wide and 7.8m deep. A major collapse occurred when the face of the concourse excavation had progressed about 20m beyond the Piccadilly tunnels. As a safety precaution, the concourse tunnel was filled with concrete until a recovery plan was developed. The remaining upline and downline Heathrow Express tunnels were re-designed and constructed as 9.1m outer diameter shield driven tunnels, lined with bolted precast concrete segments using concentric 3.9m diameter pilot tunnels. Recovery works on the concourse tunnel included breaking out the concrete and building a circular 8m internal diameter bolted SGI secondary lining inside the previously constructed SCL (HSE, 2000).

Settlement of the existing Piccadilly Line tunnels was monitored using arrays of three precise levelling points at 6m intervals. As shown in Figure A-19, there was one levelling point on each side of the running rails, and a third point in the tunnel crown. Baseline readings began more than six months in advance of the tunnelling works. The mean of the measurements taken at each of the three precise levelling points in each array was used to calculate settlement. The maximum settlements of the 'inner' Piccadilly tunnel (Figure A-18) at significant construction stages and at 38 months after completion are shown in Table A-3. Results from the 'outer' Piccadilly tunnel were substantially similar.

Construction Stage	Maximum Settlement	Distortion Along Line C	Distortion Along Line F
	(mm)	(mm)	(mm)
After concourse construction	29	-3.5	+3
After upline pilot	32		
After downline pilot	37		
After upline enlargement	46	+7	-5
After downline enlargement	63	+7	-6
After ancillary works	68		
38 months after tunnel completion	80	+10	-8

Table A-3. Settlement in the southern Piccadilly Line due to Heathrow Express Construction at CTA.

Notes.

1. Sign convention for distortion: positive outwards

The monitoring system for distortion consisted of arrays of five tape extensometer points coinciding with the levelling arrays. Eyebolts were installed at the at the tunnel crown, shoulder and knee locations (Figure A-19). Using this system, six distortion measurements could be taken at each cross section. However, it was impossible to determine the absolute movements of the points. The maximum distortions occurred at the diagonals joining shoulder and knee locations at approximately 45° , i.e., lines C and F on Figure A-19. The distortion along the 45° lines is given in Table A-3.

The maximum distortion associated with the concourse tunnel SCL works was extension of 3-4mm along the 45° diagonals, and this was measured about 15m away from the centreline of the new tunnel. Distortion of 6 to 7mm elongation was observed along the 45° diagonals in association with the enlargement of the pilot tunnels. Data taken 38 months after tunnel completion indicated distortion of 8mm to 10mm along the diagonals (Cooper, 2001). Assuming the distortion occurred over the diameter, the distortion associated with SCL works was approximately 0.10% ($1049\mu\epsilon$), while the distortion 38 months after tunnel completion increased to 0.26% ($2625\mu\epsilon$).

Cooper et al (2002) postulated that the relief of stress due to adjacent tunnelling causes ovality in the existing tunnel, with the axis of diametral increase corresponding to the direction of stress relief. There is also a concomitant diametral shortening across the orthogonal diameter of the tunnel. In other words, the diameter of greatest elongation rotates depending on the relative position of the face of the excavated tunnel.

Heathrow Express at Heathrow Terminal 4

Cooper (2001) described several case studies of tunnel movement caused by new tunnelling works, one of which was the movement of Piccadilly Line running tunnels at Terminal 4 (T4) during the construction of the Heathrow Express vent tunnels. The Northeast (NE) and Southwest (SW) 7m diameter vent tunnels crossed under the Piccadilly Line at 45° skew.

The Piccadilly Line tunnels were constructed between 1983 and 1984 using shields. At shallow depths, the tunnels were lined with 3.85m diameter bolted cast iron segments (Sabine and Skelton, 1985). At the locations of the crossings, there was about 3m to 4m of cover to the crown of the Piccadilly Line tunnels. The vertical clearance between new and existing tunnels was approximately

5.4m on the NE side and 12.0m on the SW side. Plans and schematic sections of the crossings are shown in Figure A-20 and Figure A-21.

The ventilation tunnels were driven using SCL techniques and had an external span of 7.7m with a flattened invert. The NE tunnel was constructed using the 'half face' sequence, while the SW tunnel was subsequently built with the 'full face' technique. The NE tunnel was driven at a gradient of 1 in 14 and the SW tunnel was driven at a gradient of 1 in 8. The monitoring array for the Piccadilly Line tunnels at T4 was the same as that adopted at the CTA as shown in Figure A-19.

Excavation of the NE vent tunnel caused the Piccadilly line tunnel to elongate by a maximum of 4mm to 5mm. The maximum distortion along the extensometer lines was $\pm 4\text{mm}$. The maximum distortion along the extensometer lines caused by the SW vent tunnel was similar to that caused by the NE tunnel, approximately $\pm 4\text{mm}$. Assuming the distortion occurred over the diameter, the distortion associated with the NE and SW tunnels was approximately 0.10% ($1039\mu\epsilon$).

Old Street

During the enlargement of the Northern Line tunnels near Old Street Station in 1995 and 1996, extensive monitoring was undertaken in the mainline railway tunnels running overhead and approximately parallel to the Northern Line tunnels. Cooper et al (2000) described the work and presented monitoring data related to settlement and rotation. Data for distortion was presented in Cooper (2001).

The original Northern Line tunnels were built between 1899 and 1901, having internal diameter of 3.2m and were lined with bolted segmental grey cast iron rings. The mainline railway tunnels were built in 1902 with an internal diameter of 4.88m and were lined with bolted segmental grey cast iron rings. The vertical clearance between the two sets of tunnels was about 5m.

Between 1922 and 1924, the Northern Line tunnels were enlarged to an internal diameter of 3.56m by dismantling the cast iron segments of each ring and re-assembling with cast iron packers between the segments following excavation to the larger diameter. In the decades following this enlargement, cracking of the cast iron segments was observed in conjunction with water leakage and presence of chemical deposits. In 1995 work was undertaken to remove the 3.56m internal diameter bolted grey cast iron lining and replace with new 4.68m internal diameter bolted stainless steel rings over a length 80m. All excavation was done by hand. A typical section of the tunnels is given in Figure A-22.

For the northbound Northern Line tunnel, the enlargement was done during engineering hours and it took over five months to enlarge and replace 80m of tunnel lining. For the southbound tunnel, replacement work was allowed to proceed day and night and it took two months to complete. The monitoring system installed in the mainline rail tunnels consisted of precise levelling and tape extensometer points. All monitoring was carried out during engineering hours. A typical monitoring array is given in Figure A-23.

The squat was calculated as the difference between readings at A and the mean of D and E. The results indicate that enlargement of the Northern Line tunnels caused the overlying mainline railway tunnel to elongate vertically. The horizontal diametral changes were measured by tape extensometer

and showed the axis width reduced. The magnitude of the vertical elongation and the horizontal reduction was similar, reaching a maximum of 4mm. For a diameter of 4.88m this related to a strain of 0.08% (820 $\mu\epsilon$).

CTRL (Moss and Bowers, 2006)

Section 2 of the Channel Tunnel Rail Link (CTRL) included 36km of 8.15m diameter running tunnels constructed using precast concrete segments. The running tunnels passed under 6 LU tunnels. The construction techniques for the LU tube tunnels included traditional cast iron segmental rings, tapered cast iron rings, modern precast concrete as well as a composite lining with a masonry invert and a cast iron crown.

The assessment procedure included three stages. Stage 1 was the production of Greenfield settlement contours based on estimates of volume loss and trough width parameter in order to identify the LU tunnels that were at risk from the construction of the CTRL tunnels. Stage 2 was damage categorisation, classifying risk of damage to the structure from negligible to very severe. Metro tunnels were however considered to be extraordinary structures and a third stage of assessment was warranted.

Stage three first required detailed understanding of the existing tunnels though gathering archive drawings as well as inspection and condition survey. The aim was to generate a robust model of the likely behaviour of the structure. The model could then be used to determine the point where the structure would fail in terms of serviceability, and subsequently used to develop monitoring and mitigation measures and finally used in back-analysis.

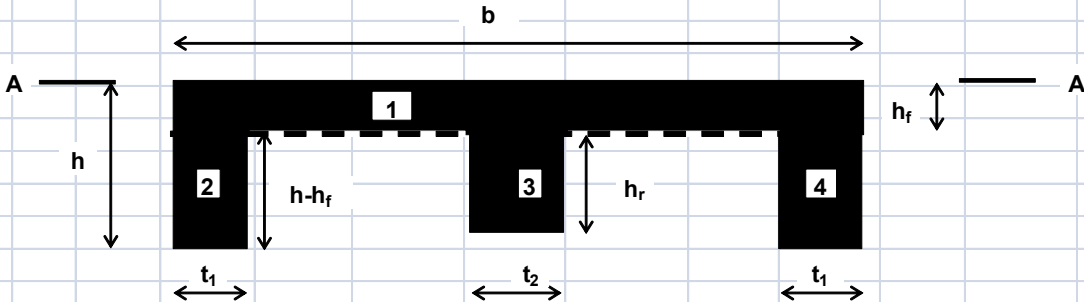
Ground movement predictions were applied to the existing tunnels using simple beam and spring models. The capacity of the metro tunnel was defined in terms of a limiting bending curvature. Since bending curvature was difficult to measure directly trigger levels were defined in terms of vertical settlement.

Defining the limiting bending curvature was complicated for bolted cast iron segments because the combination of cast iron and steel bolts resulted in a complex strain system. Once the limiting curvature was defined then an iterative process was used to find the matching volume loss and the associated tolerable vertical settlement. In cases where the calculated capacity of the metro tunnel was considered to be insufficient to withstand the predicted settlement, the release of selected circle joint bolts, in consultation with the metro tunnel owners, was undertaken as a mitigating measure.

Deflection of the existing tunnels was monitored in real time using electrolevels supplemented by manual levelling during engineering hours. However, at one location (Highbury) the difficult conditions faced during engineering hours meant that the precision from manual levelling was $\pm 2\text{mm}$. Horizontal inclinometers were used at one location (Central line cross-over) to monitor the ground movements at the horizon of the existing tunnels. At another location (Highbury), extensometers were installed prior to the crossing so that a target volume loss could be confirmed.

C. Calculations for grey cast iron segment bending capacities

The section capacity of the segment was calculated using an ultimate tensile strength of 120 MPa.

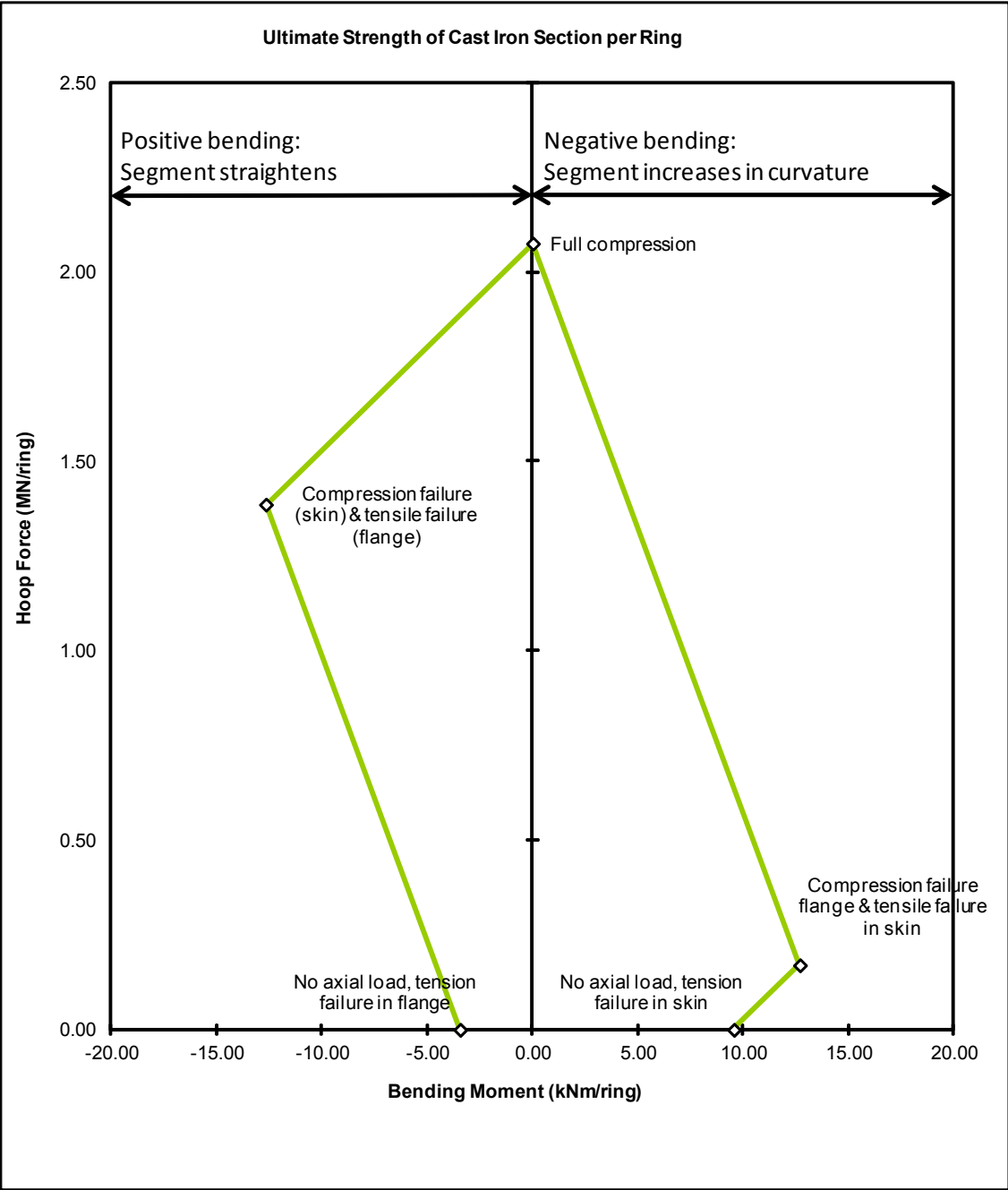
Grey cast iron segment bending capacity									
Input									
overall depth of section, h =				62	mm				
depth of skin, h_f =				11	mm				
length of centre rib, h_r =				0	mm				
overall width of section, b =				254	mm				
total width of ribs, b_w =				30	mm	from:	rib 1, t_1 =	15	mm
							rib 2, t_2 =	0	mm
							rib 3, t_3 =	15	mm
Section Properties									
									
Area of each element = $B.D$									
First moment of area of each element about the axis A-A = $B.D.y$									
Centroid of section is at a distance $y^* = \Sigma(B.D.y) / \Sigma(B.D)$ from the axis A-A									
First moment of area of the section about an axis through its centroid = $\Sigma(B.D.y^*)$									
Second moment of area of the section about an axis through its centroid = $\Sigma\{B.D.(y^*)^2 + (B.D^3/12)\}$									
where : B is dimension of element parallel to A-A									
D is dimension of element perpendicular to A-A									
y is distance of the centroid of an element from A-A									
y^* is distance of the centroid of the element from centre of gravity of section									
Area no.	y	$b.d$	$b.d.y$	$b.d.y^*$	$(b.d^3)/12$	$b.d.(y-y^*)^2$			
	(mm)	(mm ²)	(mm ³)	(mm ³)	(mm ⁴)	(mm ⁴)			
1	5.5	2 794	15 367	30 647	28 173	336 172			
2	36.5	765	27 923	15 324	165 814	306 949			
3	11.0	0	0	0	0	0			
4	36.5	765	27 923	15 324	165 814	306 949			
	Σ	4 324	71 212	61 295	359 800	950 070			
							$\therefore$ centroid of section is at a distance of $\Sigma(bdy) / \Sigma(bd)$,		
							or 16.47 mm from A-A		
							radius to centroid 936.03 mm		
Cross-sectional area of section = $\Sigma(b.d)$ =				4 324	mm ²	for width of	254	mm	
First moment of area of section = $\Sigma(b.d.y_1)$ =				61 295	mm ³	for width of	254	mm	
Second moment of area = $\Sigma(b.d^3/12) + \Sigma(b.d.(y-y^*)^2)$ =				1309 870	mm ⁴	for width of	254	mm	
Results									
Cross sectional area of section, A =				17024	mm ² /m				
Depth of centroidal axis from A-A, x' =				16.47	mm				
Second moment of area of section, I_f =				5.16E+06	mm ⁴ /m				
Effective Second Moment of Area, I_e =				2.29E+06	mm ⁴ /m	$=I_f (4/n)^{\zeta}$ (LU 1-055,3.4.7.3)			
Ultimate compressive stress, $f_{c\text{ ult}}$ =				480.00	N/mm ²				
Ultimate tensile stress, $f_{t\text{ ult}}$ =				-120.00	N/mm ²				

Ultimate strength - analysis of section (per RING)

	Axial Force (MN/ring)	Bending Moment (kNm/ring)
1. No axial load, tension failure in flange	0.00	-3.45 = $f_t \cdot I_f / y$
2. Compression failure (skin) & tensile failure (flange)	1.39	-12.68
3. Full compression	2.08 = $f_c \cdot A$	0.00
4. Compression failure (flange) & tensile failure (skin)	0.17	12.68
5. No axial load, tension failure in skin	0.00	9.54

Moment for the 5th point taken as the lesser of

$I_{fc} / (h - x') = 13.81$ or $I_{ft} / (x') = 9.54$



D. Estimation of bending moment in 2-segment test

Assuming constant modulus of 100 GPa.

Measured Strains	input	Side A flange and skin	Channel	Assume E = 100 GPa
Gauge 1 axis from skin	53 mm		7	
Gauge 2 axis from skin	0 mm		11	
Gauge 1 measurement	-31 $\mu\epsilon$	0.00 % (compression +)	7	
Gauge 2 measurement	20 $\mu\epsilon$	0.00 % (compression +)	11	

Stress-Strain relationship

$$\sigma = 1085e^5 - 3550e^4 + 4520e^3 - 2840e^2 + 930e$$

Best fit TENSION

a1	0
a2	0
a3	0
a4	0
a5	1000 (a5=1000, a1 to a4=0 means E=100 GPa)

$$\sigma = -1525e^5 + 3960e^4 - 3270e^3 + 360e^2 + 940e$$

best fit COMPRESSION

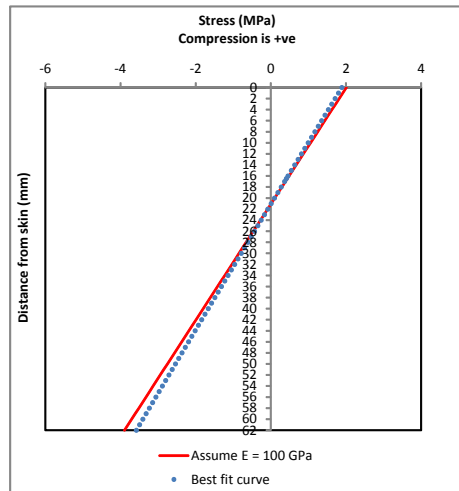
a1	0
a2	0
a3	0
a4	0
a5	1000 (a5=1000, a1 to a4=0 means E=100 GPa)

Internal Actions

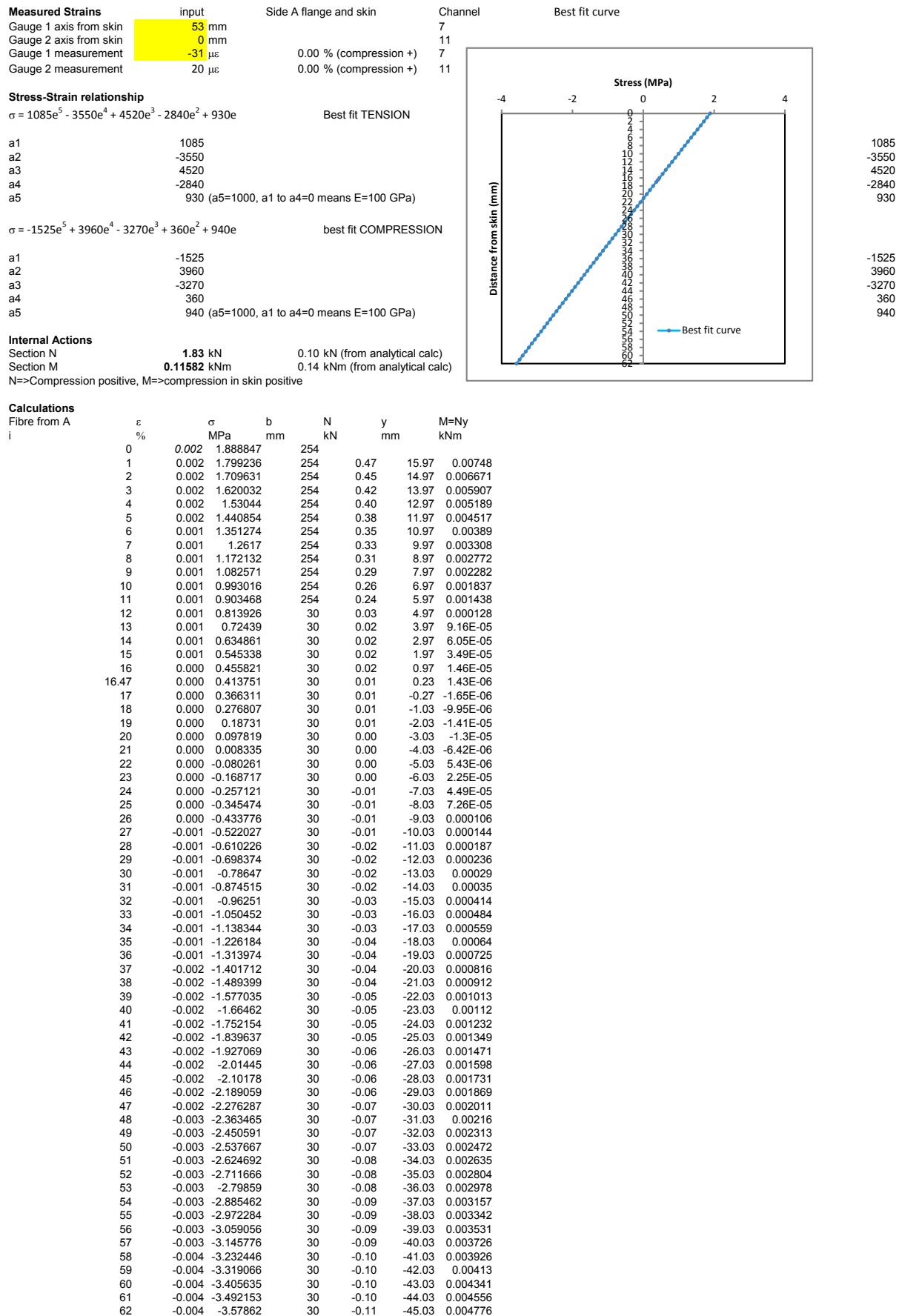
Section N	1.90 kN	0.10 kN (from analytical calc)
Section M	0.12465 kNm	0.14 kNm (from analytical calc)
N=>Compression positive, M=>compression in skin positive		

Calculations

Fibre from extrados	ϵ %	σ MPa	b mm	N kN	y mm	M=Ny kNm
i						
0	0.002	2.007896	254			
1	0.002	1.912704	254	0.50	15.97	0.007951
2	0.002	1.817512	254	0.47	14.97	0.007091
3	0.002	1.72232	254	0.45	13.97	0.00628
4	0.002	1.627128	254	0.43	12.97	0.005517
5	0.002	1.531937	254	0.40	11.97	0.004802
6	0.001	1.436745	254	0.38	10.97	0.004136
7	0.001	1.341553	254	0.35	9.97	0.003518
8	0.001	1.246361	254	0.33	8.97	0.002948
9	0.001	1.151169	254	0.30	7.97	0.002426
10	0.001	1.055977	254	0.28	6.97	0.001953
11	0.001	0.960786	254	0.26	5.97	0.001529
12	0.001	0.865594	30	0.03	4.97	0.000136
13	0.001	0.770402	30	0.02	3.97	9.74E-05
14	0.001	0.67521	30	0.02	2.97	6.44E-05
15	0.001	0.580018	30	0.02	1.97	3.71E-05
16	0.000	0.484826	30	0.02	0.97	1.55E-05
16.47	0.000	0.440086	30	0.01	0.23	1.53E-06
17	0.000	0.389635	30	0.01	-0.27	-1.75E-06
18	0.000	0.294443	30	0.01	-1.03	-1.06E-05
19	0.000	0.199251	30	0.01	-2.03	-1.5E-05
20	0.000	0.104059	30	0.00	-3.03	-1.38E-05
21	0.000	0.008867	30	0.00	-4.03	-6.83E-06
22	0.000	-0.086325	30	0.00	-5.03	5.85E-06
23	0.000	-0.181516	30	0.00	-6.03	2.42E-05
24	0.000	-0.276708	30	-0.01	-7.03	4.83E-05
25	0.000	-0.3719	30	-0.01	-8.03	7.81E-05
26	0.000	-0.467092	30	-0.01	-9.03	0.000114
27	-0.001	-0.562284	30	-0.02	-10.03	0.000155
28	-0.001	-0.657476	30	-0.02	-11.03	0.000202
29	-0.001	-0.752667	30	-0.02	-12.03	0.000254
30	-0.001	-0.847859	30	-0.02	-13.03	0.000313
31	-0.001	-0.943051	30	-0.03	-14.03	0.000377
32	-0.001	-1.038243	30	-0.03	-15.03	0.000447
33	-0.001	-1.133435	30	-0.03	-16.03	0.000522
34	-0.001	-1.228627	30	-0.04	-17.03	0.000603
35	-0.001	-1.323818	30	-0.04	-18.03	0.00069
36	-0.001	-1.41901	30	-0.04	-19.03	0.000783
37	-0.002	-1.514202	30	-0.04	-20.03	0.000881
38	-0.002	-1.609394	30	-0.05	-21.03	0.000985
39	-0.002	-1.704586	30	-0.05	-22.03	0.001095
40	-0.002	-1.799777	30	-0.05	-23.03	0.001211
41	-0.002	-1.894969	30	-0.06	-24.03	0.001332
42	-0.002	-1.990161	30	-0.06	-25.03	0.001459
43	-0.002	-2.085353	30	-0.06	-26.03	0.001591
44	-0.002	-2.180545	30	-0.06	-27.03	0.00173
45	-0.002	-2.275737	30	-0.07	-28.03	0.001874
46	-0.002	-2.370928	30	-0.07	-29.03	0.002023
47	-0.002	-2.46612	30	-0.07	-30.03	0.002179
48	-0.003	-2.561312	30	-0.08	-31.03	0.00234
49	-0.003	-2.656504	30	-0.08	-32.03	0.002507
50	-0.003	-2.751696	30	-0.08	-33.03	0.00268
51	-0.003	-2.846888	30	-0.08	-34.03	0.002858
52	-0.003	-2.942079	30	-0.09	-35.03	0.003042
53	-0.003	-3.037271	30	-0.09	-36.03	0.003232
54	-0.003	-3.132463	30	-0.09	-37.03	0.003427
55	-0.003	-3.227655	30	-0.10	-38.03	0.003628
56	-0.003	-3.322847	30	-0.10	-39.03	0.003835
57	-0.003	-3.418039	30	-0.10	-40.03	0.004048
58	-0.004	-3.51323	30	-0.10	-41.03	0.004266
59	-0.004	-3.608422	30	-0.11	-42.03	0.00449
60	-0.004	-3.703614	30	-0.11	-43.03	0.00472
61	-0.004	-3.798806	30	-0.11	-44.03	0.004955
62	-0.004	-3.893998	30	-0.12	-45.03	0.005196



Assuming the best fit stress-strain relationship from cast iron test specimens.



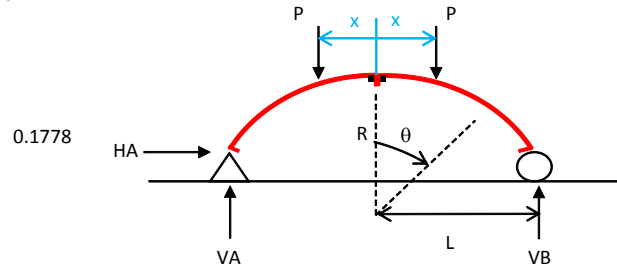
E. Calculations of change in bending moment from statics

When the moment at the joint equals 0.25 kNm.

2 Segment test

LINE LOAD

R	0.94	m	to centroid
P	0.70	kN/m	
x	0.11	m (see drawing)	
θ_B	60.00	Deg	
seg width	0.25	m	
Actuator Load	0.36	kN	
L	0.81	m	
θ_p	6.75	Deg	
H_A	0.00	kN/m	
V_A	0.70	kN/m	
V_B	0.70	kN/m	



Symmetry about centreline => θ positive from centreline clockwise

M_θ => positive if tension on intrados

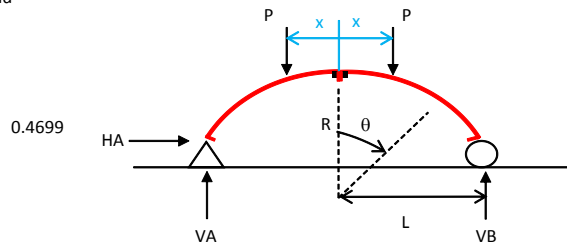
θ	M_p (kNm/m)	M_{p+sw} (kNm/m)	M_p (kNm)	M_{p+sw} (kNm)
0	0.490438631	0.9581951	0.12	0.24
0.5	0.490438631	0.957340428		
1.0	0.490438631	0.956407564		
1.5	0.490438631	0.955396887		
2.0	0.490438631	0.954308784		
2.5	0.490438631	0.953143656		
3.0	0.490438631	0.951901916		
3.5	0.490438631	0.950583988		
4.0	0.490438631	0.949190307		
4.5	0.490438631	0.947721323		
5.0	0.490438631	0.946177493		
5.5	0.490438631	0.944559291		
6.0	0.490438631	0.942867197		
6.5	0.490438631	0.941101707		
7.0	0.487587194	0.93641189		
7.5	0.481915039	0.928828982		
8.0	0.476249396	0.921180742		
8.5	0.470590699	0.913468143		
9.0	0.464939376	0.905692168		
9.5	0.459295859	0.897853809		
10.0	0.453660578	0.88995407		
10.5	0.448033962	0.881993966		
11.0	0.442416438	0.873974522		
11.5	0.436808436	0.865896774		
12.0	0.431210381	0.857761766		
12.5	0.425622701	0.849570556		
13.0	0.420045821	0.841324209		
13.5	0.414480165	0.833023801		
14.0	0.408926157	0.824670419		
14.5	0.403384221	0.816265158		
15.0	0.397854778	0.807809124		
15.5	0.39233825	0.799303432		
16.0	0.386835056	0.790749207		
16.5	0.381345616	0.782147585		
17.0	0.375870348	0.773499707		
17.5	0.370409668	0.764806728		
18.0	0.364963993	0.756069809		
18.5	0.359533737	0.747290122		
19.0	0.354119314	0.738468847		
19.5	0.348721136	0.729607172		
20.0	0.343339614	0.720706296	0.09	0.18

When the moment at the joint equals 0.45 kNm.

2 Segment test

LINE LOAD

R	0.94	m	to centroid
P	1.85	kN/m	
x	0.11	m (see drawing)	
θ_B	60.00	Deg	
seg width	0.25	m	
Actuator Load	0.94	kN	
L	0.81	m	
θ_P	6.75	Deg	
H_A	0.00	kN/m	
V_A	1.85	kN/m	
V_B	1.85	kN/m	



Symmetry about centreline => θ positive from centreline clockwise

M_{θ} => positive if tension on intrados

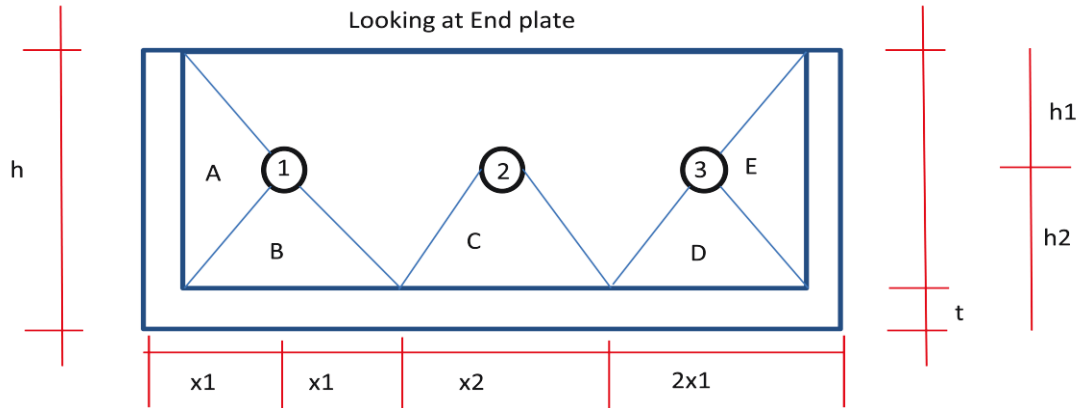
θ	M_P (kNm/m)	M_{P+SW} (kNm/m)	M_P (kNm)	M_{P+SW} (kNm)
0	1.296159239	1.763915708	0.33	0.45
0.5	1.296159239	1.763061036		
1.0	1.296159239	1.762128173		
1.5	1.296159239	1.761117495		
2.0	1.296159239	1.760029392		
2.5	1.296159239	1.758864264		
3.0	1.296159239	1.757622524		
3.5	1.296159239	1.756304596		
4.0	1.296159239	1.754910916		
4.5	1.296159239	1.753441931		
5.0	1.296159239	1.751898102		
5.5	1.296159239	1.750279899		
6.0	1.296159239	1.748587806		
6.5	1.296159239	1.746822316		
7.0	1.288623297	1.737447994		
7.5	1.273632602	1.720546545		
8.0	1.258659119	1.703590465		
8.5	1.243703989	1.686581434		
9.0	1.228768351	1.669521143		
9.5	1.213853343	1.652411292		
10.0	1.1989601	1.635253591		
10.5	1.184089756	1.61804976		
11.0	1.169243444	1.600801528		
11.5	1.154422294	1.583510632		
12.0	1.139627436	1.566178821		
12.5	1.124859995	1.54880785		
13.0	1.110121097	1.531399485		
13.5	1.095411864	1.5139555		
14.0	1.080733415	1.496477677		
14.5	1.06608687	1.478967806		
15.0	1.051473343	1.461427688		
15.5	1.036893946	1.443859128		
16.0	1.022349792	1.426263943		
16.5	1.007841986	1.408643954		
17.0	0.993371634	1.391000993		
17.5	0.978939837	1.373336897		
18.0	0.964547696	1.355653512		
18.5	0.950196305	1.33795269		
19.0	0.935886758	1.320236291		
19.5	0.921620144	1.302506181		
20.0	0.907397551	1.284764233	0.23	0.33

at 20 degrees from joint (SG location)		
P (kN/m)	BM (kNm)	Change in BM (kNm)
0.7	0.18	
1.8	0.33	0.14

F. Design of reaction ring

Firstly the moment capacity of the test ring at the joint was estimated as outlined below.

Schematic view of longitudinal flange for the half-scale segment



Dimensions from Drawing No. UnPS/ICR/GN/D/0004

Depth of segment, $h=62\text{mm}$	$x1=51\text{mm}$
Intrados to bolt hole, $h1=25.5\text{mm}$	$x2=76\text{mm}$
Bolt hole to extrados, $h2=36.5\text{mm}$	Thickness of longitudinal flange, $t1=15.8\text{mm}$
Thickness of skin, $t=11\text{mm}$	Thickness of circumferential flange, $t2=17.5\text{mm}$

Part 1. Calculations based on flexural capacity of longitudinal flange

Tensile strength of cast iron	σ_T	150	MPa	
b_A		51.0	mm	Width of cantilever assumed to be distance $x1$
d_A		15.8	mm	Depth of cantilever assumed to be $t1$
I_A	$=bd^3/12$	16763.3	mm^4	
Section modulus, Z_A	$=I/(d/2)$	2121.9	mm^3	
Maximum moment for cantilever, $M_{\max A}$	$=\sigma_T Z$	318291	Nmm	
Cantilever length, L	$= x1 - t2$	33.5	mm	

Assuming that the maximum moment is caused by the bolt force

Maximum tensile force at bolt, $T_{\max \text{ double curvature}}$	$=2M/L$	19.0	kN	Limited by flexural capacity of longitudinal flange
$T_{\max \text{ single curvature}}$	$=M/L$	9.5	kN	

Part 2. Bending moment capacity of the joint under zero hoop load

Positive bending

Lever arm from bolt to extrados, $h2$	36.5	mm	
$M_{\max}$ at joint from $T_{\max}$ bolt (double curvature) = $(19.0 \times 0.0365) \times 2$	1.4	kNm	Assumed 2 bolts; extrados compression
$M_{\max}$ at joint from $T_{\max}$ bolt (single curvature) = $(9.5 \times 0.0365) \times 2$	0.7	kNm	Assumed 2 bolts; extrados compression

Negative bending

Lever arm from bolt to intrados, $h1$ -caulking groove	16.0	mm	Caulking groove is 9.5mm deep
$M_{\max}$ at joint from $T_{\max}$ bolt (double curvature)	0.6	kNm	Assumed 2 bolts; extrados tension
$M_{\max}$ at joint from $T_{\max}$ bolt (single curvature)	0.3	kNm	Assumed 2 bolts; extrados tension

Part 3. Additional bending moment capacity of the joint with hoop load
Hoop force, N = 96.5 kN (20m overburden pressure)
Bending moment = Ne, where e = 16.5mm for positive bending (centroidal axis to extrados) e = 36.0mm for negative bending (centroidal axis to caulking groove) Ne = 1.6 kNm (positive bending) Ne = 3.5 kNm (negative bending)
Part 3. Total bending moment capacity of the joint with hoop load
Positive bending: 1.6 + 1.4 = 3.0 kNm
Negative bending: 3.5 + 0.6 = 4.1 kNm
Design bending capacity of reaction ring = 4.1x17 = 70 kNm.

Secondly the section required for a design moment of 70 kNm was estimated. The lightest 254 UC would have been adequate.

Reference:1. BS EN 1993-1-1:2005			
Reference:2. BS EN 10025-1:2004			
Design Loads			
Tensile Force [kN]		230.00	
Moment [kNm]		70.00	
Shear [kN]		40.00	
Assume Steel Grade S235			
Yield strength [N/mm ²]	f_y	235	(thickness less than 40mm)
Ultimate tensile strength [N/mm ²]	f_u	360	(thickness less than 40mm)
Young's Modulus [N/mm ²]	E	2.10E+05	
Material Factor	γ_{M0}	1.05	
Density of steel [kg/m ³]		7850	(BS EN 10025-1:2004 CI 7.7.2)
Poisson's ratio		0.3	(BS EN 1993-1-1 2005 CI 3.2.6)
Assume Section 254x254x73			
Cross section Area [cm ²]	A	93.10	Section Classification ε 1.00
Cross section Area [mm ²]	A	9.31E+03	Web buckling
Height of section [mm]	h	254.1	c 200.30
Width of section [mm]	b	254.6	t 8.60
thickness flange	t_f	14.20	c/t _ε 23.29 class 1
thickness web	t_w	8.60	Flange buckling
root radius	r	12.70	c 110.30
depth between fillets	d	200.30	t 14.20
Moment of Inertia of Ring [cm ⁴]	I	11407	c/t _ε 7.77 class 1
Moment of Inertia of Ring [mm ⁴]	I	1.14E+08	
Tensile Capacity (CI 6.2.3)			
Design plastic resistance [kN]	N_{pl}	2083.67	> 230.00
Moment Capacity (CI 6.2.5) for class 1 or 2 sections			
Design plastic resistance [kNm]	M_{pl}	200.94	> 70.00
Shear Capacity (CI 6.2.6)			
Shear area = max (A _v , A _w)	A_v	2562.16	
	A_w=d_tt_w	1722.58	

Design plastic resistance [kN]	V _{pl}	331.07	>	40.00	<	0.5*V _{pl}
no need to check						
Bending and Shear (CI 6.2.8)	check when Shear force > Half Plastic Shear Resistance					
Reduction	ρ	0.58				
Reduced design plastic resistance moment [kNm]	M _v	189.84	>	70.00	ok	
Bending and Axial (CI 6.2.9)	check when:					
N>0.25 N _{pl} AND	0.25N_{pl}	520.92	>	230.00		
N>0.5h _w t _w f _y /γ _{m1}	0.5h_wt_wf_y/γ_{m0}	192.76	<	230.00	no need to check	
n	N/N _{pl}	0.11				
a min(0.5, (A-2btf)/A)	(A-2btf)/A	0.22	i.e	0.22		
Reduced Moment Capacity	M _N	201.24	>	70.00	ok	

G. Hanging frame design

The hanging frame details are given in Figure 6-2. Structural capacity checks were made for the following components:

3. M12 bolts for hanging the actuators onto the reaction ring.
4. Bending capacity of the ¾" thick backing plate.
5. Buckling capacity of the connection rod.
6. Deflection of the steel plate for the spreader pad.
7. Compressive capacity of the machined timber plate for the spreader pad.

M12 Bolt

Density of Steel=	7850.00 kg/m3 77.009 kN/m3	BS EN 1993-1-8:2005 EN 1993-1-8:2005 (E)	
Weight of Actuator=	19.600 kg 0.192 kN		
Dimensions of bellows			
Diameter	0.400 m		
Height	0.150 m		
Volume	0.02 m3	18.85 L	
Number of bellows	2		
Unit weight water	10 kN/m3		
Weight of water in bellows	0.370 kN	38 kg	
Dimension 1.25" rod			
diameter	0.032 m		
length	0.350 m		
Weight	0.021 kN		
x2	0.043 kN		
Dimension 3/4" back plate=			
width	0.019 m		
breadth	0.290 m		
length	0.290 m		
Weight	0.123 kN		
Connection rod			
Diameter	0.05 m		
Length	0.26 m	conservative assumption	
Weight	0.04 kN		
Load cell			
Diameter	0.11 m		
Length	0.07 m		
Weight	0.05 kN	conservative assumption	
Spreader pad			
width	0.25 m		
breadth	0.05 m		
length	0.25 m		
Weight	0.24 kN		
Total weight	1.06 kN	108 kg	
Shear Force, $F_{v,Ed}$ per bolt	0.53 kN		
Bolt capacity=			
M12			
Diameter	12.000 mm		
Grade	8.8		
Yield strength, f_{yb}	640 N/mm2		
Ultimate tensile strength, f_{ub}	800 N/mm2		
Clearance from edge, 1.2D	14.400 mm		
Shearing resistance, $F_{v,Rd}$			
α_v	0.60		
f_{ub}	800.00 N/mm2	375	
A	113.10 mm2		
γ_{m2}	1.25		
$F_{v,Rd} =$	43.43 kN	42.4115 ok	

Table 3.1: Nominal values of the yield strength f_{yk} and the ultimate tensile strength f_{tk} for bolts

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yk} (N/mm ²)	240	320	300	400	480	640	900
f_{tk} (N/mm ²)	400	400	500	500	600	800	1000

NOTE: The National Annex may exclude certain bolt classes.

Table 3.3: Minimum and maximum spacing, end and edge distances

Distances and spacings, see Figure 3.1	Minimum	Maximum ¹⁾²⁾³⁾		
		Structures made from steels conforming to EN 10025 except steels conforming to EN 10025-5		Structures made from steels conforming to EN 10025-5
		Steel exposed to the weather or other corrosive influences	Steel not exposed to the weather or other corrosive influences	Steel used unprotected
End distance e_1	$1,2d_b$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Edge distance e_2	$1,2d_b$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Distance e_3 in slotted holes	$1,5d_b$ ⁴⁾			
Distance e_4 in slotted holes	$1,5d_b$ ⁴⁾			
Spacing p_1	$2,2d_b$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm
Spacing $p_{1,2}$		The smaller of $14t$ or 200 mm		
Spacing $p_{1,2}$		The smaller of $28t$ or 400 mm		
Spacing p_2 ⁵⁾	$2,4d_b$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm

¹⁾ Maximum values for spacings, edge and end distances are unlimited, except in the following cases:

- for compression members in order to avoid local buckling and to prevent corrosion in EN 10025-5 exposed members (the limiting values are given in the table) and; EN 10025-5
- for exposed tension members EN 10025-5 to prevent corrosion (the limiting values are given in the table). EN 10025-5

²⁾ The local buckling resistance of the plate in compression between the fasteners should be calculated according to EN 1993-1-1 using $0,6 p_{cr}$ as buckling length. Local buckling between the fasteners need not to be checked if p_{cr}/b is smaller than 9ϵ . The edge distance should not exceed the local buckling requirements for an outstand element in the compression members, see EN 1993-1-1. The end distance is not affected by this requirement.

³⁾ t is the thickness of the thinner outer connected part.

⁴⁾ The dimensional limits for slotted holes are given in 1.2.7 Reference Standards: Group 7.

⁵⁾ For staggered rows of fasteners a minimum line spacing of $p_2 = 1,2d_b$ may be used, provided that the minimum distance, l , between any two fasteners is greater or equal than $2,4d_b$, see Figure 3.1b).

Section 3.6

$$F_{v,Rd} = \frac{\alpha_v f_{yb} A}{\gamma_{m2}}$$

- where the shear plane passes through the threaded portion of the bolt (A is the tensile stress area of the bolt A):

- for classes 4.6, 5.6 and 8.8:

$$\alpha_v = 0,6$$

- for classes 4.8, 5.8, 6.8 and 10.9:

$$\alpha_v = 0,5$$

- where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt): $\alpha_v = 0,6$

3/4" thick backing plate

Density of Steel=	7850 kg/m ³		
	77.01 kN/m ³		
E_{steel}	210000 N/mm ²		
	2.E+08 kPa		
Max actuator load	80.00 kN		
Actuator load for 20m overburden	33.78 kN		
Dimension 3/4" back plate=			
width	0.019 m		
breadth	0.290 m		
length	0.290 m		
Steel Grade	S235	t<40	
Yield strength, f_y	235.00 N/mm ²		
Ultimate tensile strength, f_u	360.00 N/mm ²		
γ_{m0}	1.00		
I	1.67071E-07 m ⁴		
Y	0.009525 m		
W	1.75403E-05 m ³		
Bending moment resistance	4.12 kNm		
Midspan deflection			
w (max)	275.86 kN/m		
w (20m)	116.48 kN/m		
$\delta = 5wL^2/384EI$ (max)	-0.72 mm		
$\delta = 5wL^2/384EI$ (20m)	-0.31 mm		
BM= $wL^2/8$ (max)	2.90 kNm	ok	
BM= $wL^2/8$ (20m)	1.22 kNm	ok	

$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}}$ for class 1 or 2 cross sections

Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Nominal thickness of the element t [mm]			
	t ≤ 40 mm		40 mm < t ≤ 80 mm	
	f_y [N/mm ²]	f_u [N/mm ²]	f_y [N/mm ²]	f_u [N/mm ²]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	490 (G5)	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	490 (G5)	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

Connection rod

Connection rod			
Diameter	0.05 m		
Length	0.33 m		Conservative assumption. Length pending design of spreaders.
Weight	0.04 kN		Gap 0.5475 m
Area	1.96E-03 m ²		Actuator 0.15 m
I	3.07E-07 m ⁴		Load cell 0.07 m
Esteeel	210000 N/mm ²		Clearance 0.3275 m without spreader
	210000000 kPa		
Steel Grade	S235	t<40	
Yield strength, f_y	235.00 N/mm ²		
Ultimate tensile strength, f_u	360.00 N/mm ²		
γ_{m0}	1		
γ_{m1}	1		
Axial			
Axial capacity, $N_{c,Rd}$	461.42 kN		
Max load from actuator	80.00 kN		
Buckling			
$N_{c, pin ends} = \pi^2 EI/L^2$	5.93E+03 kN		
$N_{c, fixed ends} = \pi^2 EI/Le^2$	2.37E+04 kN		
λ (pin)	0.28		
λ (fixed)	0.14		
a	0.49		
Φ (pin)	0.56		
Φ (fixed)	0.49		
$\bar{\lambda}$ (pin)	0.96		
$\bar{\lambda}$ (fixed)	1.00		
$N_{b,Rd}$ (pin)	442.90 kN		
$N_{b,Rd}$ (fixed)	461.42 kN		

$N_{c,Rd} = \frac{A f_y}{\gamma_{M0}}$ for class 1, 2 or 3 cross-sections

BS EN 1993-1-1:2005
Cl6.2.4

BS EN 1993-1-1:2005
Cl6.3.1.1

6.3.1.1 Buckling resistance

(1) A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

where N_{Ed} is the design value of the compression force,
 $N_{b,Rd}$ is the design buckling resistance of the compression member.

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$$
 for Class 1, 2 and 3 cross-sections
$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1.0$$

where $\Phi = 0.5 \left[1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right]$

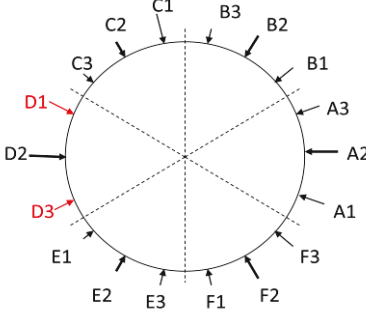
$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}}$$
 for Class 1, 2 and 3 cross-sections

Steel plate for the spreader pad

Steel plate	
Dimension	
length	250 mm
width	250 mm
thickness	25 mm
Grade	S275
Yield strength, f_y	275.00 N/mm ²
Ultimate tensile strength, f_u	430.00 N/mm ²
γ_{m0}	1.00
I	325520.8333 mm ⁴
y	12.5 mm
W	26041.66667 mm ³
Bending moment resistance	7.16 kNm
Connection rod	
Diameter	0.05 m
Length	0.3275 m
Weight	0.039313517 kN
Area	0.001963495 m ²
I	3.08798E-07 m ⁴
Esteel	210000 N/mm ²
Max actuator load	80 kN
Actuator load for 20m overbun	33.78 kN
Load on Lever arm	32 kN
Lever arm length	50 mm
Moment	1.60 kNm ok
Deflection of plate	
distance from rod to corner	151.78 mm
fixed end Cantilever with UDL	
$y = \frac{-\omega L^4}{8EI}$	
ω	320 N/mm
y	0.31 mm
	y
15mm thk	1.44
20mm thk	0.61
25mm thk	0.31

H. Loading sequence to bring all actuators under equal load

All actuators to 3.4 kN

			
Stage	Manual control	Triax control	Scan
101	All actuator isolator valves closed. (c.f. Figure 6-6.) Vents to atmosphere are open for low pressures.	All pressure controllers set to hold 7.5 kPa.	Baseline scan.

102	Open actuator isolator valves for all A, C, and E actuators.	Set all actuators A, C and E to hold 1.5kN.	
106	Open actuator isolator valves for all B and F actuators.	Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 1.5kN.	
109	Open actuator isolator valve for D2.	Set all actuators A, C and E to hold 3.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
110		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 3.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
113	Close all vents to atmosphere.	Set all actuators A, B, C, E and F to hold 3.4kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	Scan
114		As for 113 but set A2 to move away from centre by 1.0mm.	Scan
Repeat Stages 113 and 114 three times.			
113		Set all actuators A, B, C, E and F to hold 3.4 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	Scan
114*		As for 113 but set A2 to move towards the centre by 1.0mm.	Scan
Repeat Stages 113 and 114* three times.			

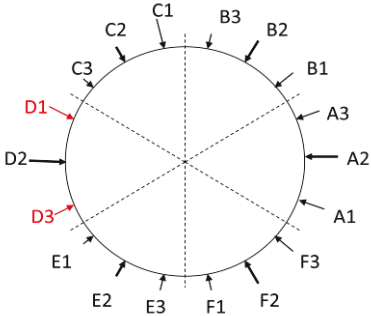
All actuators to 10 kN

Stage	Manual control	Triax control	Scan
101	All actuator isolator valves closed. (c.f. Figure 6-6.) Vents to atmosphere are open for low pressures.	All pressure controllers set to hold 7.5 kPa.	Baseline scan.
102	Open actuator isolator valves for all A, C, and E actuators.	Set all actuators A, C and E to hold 1.5kN.	
106	Open actuator isolator valves for all B and F actuators.	Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 1.5kN.	
109	Open actuator isolator valve for D2.	Set all actuators A, C and E to hold 3.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
110		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 3.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
116	Close all vents to atmosphere.	Set all actuators A, C and E to hold 5.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
117		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold	

Appendix

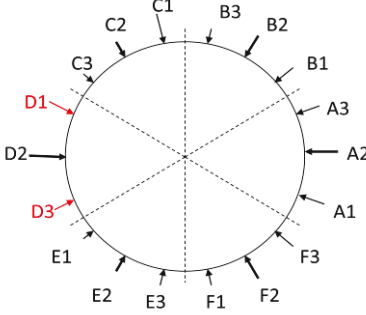
		5.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
118		Set all actuators A, C and E to hold 7.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
119		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 7.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
120		Set all actuators A, C and E to hold 9.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
121		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 9.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
113		Set all actuators A, B, C, E and F to hold 10.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	Scan
114		As for 113 but set A2 to move away from centre by 2.0mm.	Scan
Repeat Stages 113 and 114 three times.			
113		Set all actuators A, B, C, E and F to hold 10.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	Scan
114*		As for 113 but set A2 to move towards the centre by 1.0mm.	Scan
Repeat Stages 113 and 114* three times.			

All actuators to 20 kN

			
Stage	Manual control	Triax control	Scan
101	All actuator isolator valves closed. (c.f. Figure 6-6.) Vents to atmosphere are open for low pressures.	All pressure controllers set to hold 7.5 kPa.	Baseline scan.
102	Open actuator isolator valves for all A, C, and E actuators.	Set all actuators A, C and E to hold 1.5kN.	
106	Open actuator isolator valves for all B and F actuators.	Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 1.5kN.	
109	Open actuator isolator valve for D2.	Set all actuators A, C and E to hold 3.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
110		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 3.0 kN. Set actuator D2 to hold the average load	

		measured from reaction rod D1 and D3.	
116	Close all vents to atmosphere.	Set all actuators A, C and E to hold 6.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
117		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 6.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
118		Set all actuators A, C and E to hold 10.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
119		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 10.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
120		Set all actuators A, C and E to hold 14.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
121		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 14.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
104		Set all actuators A, C and E to hold 18.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
105		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 18.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
108		Set all actuators A, C and E to hold 20.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
107		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 20.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
113		Set all actuators A, B, C, E and F to hold 20.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	Scan
114		As for 113 but set A2 to move away from centre by 2.0mm.	Scan
Repeat Stages 113 and 114 three times.			
113		Set all actuators A, B, C, E and F to hold 20.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	Scan
114*		As for 113 but set A2 to move towards the centre by 1.0mm.	Scan
Repeat Stages 113 and 114* three times.			

All actuators to 40 kN

			
Stage	Manual control	Triax control	Scan
101	All actuator isolator valves closed. (c.f. Figure 6-6.) Vents to atmosphere are open for low pressures.	All pressure controllers set to hold 7.5 kPa.	Baseline scan.
102	Open actuator isolator valves for all A, C, and E actuators.	Set all actuators A, C and E to hold 1.5kN.	
106	Open actuator isolator valves for all B and F actuators.	Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 1.5kN.	
109	Open actuator isolator valve for D2.	Set all actuators A, C and E to hold 3.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
110		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 3.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
116	Close all vents to atmosphere. Increase volume of water in all air/water interfaces.	Set all actuators A, C and E to hold 6.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
117		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 6.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
118		Set all actuators A, C and E to hold 10.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
119		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 10.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
120		Set all actuators A, C and E to hold 14.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
121		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 14.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
104		Set all actuators A, C and E to hold 18.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
105		Set all actuators A, C and E to hold current displacement.	

		Set all actuators B and F to hold 18.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
108		Set all actuators A, C and E to hold 21.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
107		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 21.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
103		Set all actuators A, C and E to hold 24.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
111		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 24.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
112		Set all actuators A, C and E to hold 27.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
115		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 27.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
122		Set all actuators A, C and E to hold 30.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
123		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 30.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
124		Set all actuators A, C and E to hold 32.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
125		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 32.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
126		Set all actuators A, C and E to hold 34.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
127		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 34.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
128		Set all actuators A, C and E to hold 36.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	

Appendix

129		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 36.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
130		Set all actuators A, C and E to hold 38.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
131		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 38.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
132		Set all actuators A, C and E to hold 40.0 kN. Set all actuators B and F to hold current displacement. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
133		Set all actuators A, C and E to hold current displacement. Set all actuators B and F to hold 40.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	
113		Set all actuators A, B, C, E and F to hold 40.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	Scan
114		As for 113 but set A2 to move away from centre by 2.0mm.	Scan
Repeat Stages 113 and 114 three times.			
113		Set all actuators A, B, C, E and F to hold 40.0 kN. Set actuator D2 to hold the average load measured from reaction rod D1 and D3.	Scan
114*		As for 113 but set A2 to move towards the centre by 1.0mm.	Scan
Repeat Stages 113 and 114* three times.			

I. Measurements from strain gauge rosettes from full ring tests

Table I-1, Table I-2 and Table I-3 summarise the change in strain in the circumferential and radial directions for segment H, segment I and segment J, respectively when the load for the actuator at 90 degrees was reduced during the unloading tests. The table gives the change in strain from Stage One to Stage Two as shown in Figure 6-30. Refer to Figure 6-11 to Figure 6-14 for the location of the strain gauge rosettes. The circumferential and radial directions are also indicated on the figures. Each table also presents the results for Stage One normal loads corresponding to depths of 6 m BGL and 24 m BGL. Reference back to Table 7-1 indicates that the joints at zero and 180 degrees would open when the radius increases by 0.05% at 90 degrees for the overburden loads corresponding to 6 m BGL, while for overburden loads corresponding to 24 m BGL the joints would not open until the increase in radius was 0.21% at 90 degrees. Since the actual increase in radius at 90 degrees was 0.13% for the unloading tests, it is expected the joints were affecting the segmental ring behaviour for the case with Stage One normal loads corresponding to depths of 6 m BGL.

A decrease in inner radius was achieved through increasing the load on segment I. Table I-4, Table I-5 and Table I-6 summarise the change in strain in the circumferential and radial directions when the inner radius decreased by 0.07% for segment H, segment I and segment J, respectively. Each table presents the results for distortion under two different overburden loads, 2 m BGL and 24 m BGL. Reference back to Table 7-2 indicates that the extreme fibre stress at the intrados of the joints at 60 and 120 degrees would be zero at this magnitude of deformation for the overburden loads corresponding to 2 m BGL.

Unloading Case – Segment H straightens (positive moment)

The strain gauge locations for segment H are given in Figure 6-11.

For segment H, location 1 measures the strain at the external corner of the circumferential flange close to the intrados of the joint at zero degree. The circumferential tensile strain at the location was significantly smaller when the overburden was low and the joint was beginning to open. Interestingly, the reverse happened on the internal corner of the circumferential flange, location 5. At this location the tensile circumferential strain was slightly higher when the overburden was low and the joint was tending to open.

For locations 9 and 10 on the extrados of the skin away from the horizontal flanges, the change in strain agreed with the expected behaviour for segments undergoing positive moment, with compressive circumferential strains and tensile radial strains.

For locations 13 and 14, the junction between the horizontal flange and the intrados of the skin, the tensile circumferential strains were once again higher when the overburden was low and the joint was tending to open.

Unloading Case – Segment H straightens	Circumferential strain ($\mu\epsilon$) +ve strain = tensile		Radial strain ($\mu\epsilon$) +ve strain = tensile	
	Approx 6m BGL	Approx 24m BGL	Approx 6m BGL	Approx 24m BGL
Location 1	5	83	19	-38
Location 2	26	1	-2	-8
Location 3	5	11	2	4
Location 4	102	133	-62	-19
Location 5	132	125	-32	-36
Location 6	5	3	-38	-13
Location 7	15	10	-115	-73
Location 8	35	16	-114	-78
Location 9	-44	-52	10	16
Location 10	-20	-33	20	7
Location 11	28	17	-4	-3
Location 12	25	18	-11	-4
Location 13	15	2	-8	-19
Location 14	56	4	7	1
Location 15	-14	-17	-6	-10
Location 16	-1	-12	-17	-5
Location 17	7	6	-11	-3
Location 18	-3	5	-9	-2

Table I-1. Measured change in strain in segment H between Stage One and Stage Two when the test ring was unloaded for overburden loads corresponding to 6m BGL and 24m BGL.

Unloading Case – Segment I increase in curvature (negative moment)

The strain gauge locations for segment I are given in Figure 6-12.

Table 7-1 shows that the joints at either end of segment I (60 and 120 degrees around the ring) do not open when the increase in inner radius is 0.13%. Comparing the strains for segment I with the strains for segment H, the first thing to note is that the strains for segment I were more symmetrical.

For locations 9 and 10 on the extrados of the skin away from the horizontal flanges, the change in strain agreed with the expected behaviour for segments undergoing negative moment, with tensile circumferential strains and compressive radial strains.

For locations 7, 8, 11 and 12, the junction between the horizontal flange and the extrados of the skin, the tensile circumferential strains were higher when the overburden was lower.

Unloading Case – Segment I increase in curvature	Circumferential strain ($\mu\epsilon$) +ve strain = tensile		Radial strain ($\mu\epsilon$) +ve strain = tensile	
	Approx 6m BGL	Approx 24m BGL	Approx 6m BGL	Approx 24m BGL
Location 1	-13	19	14	-43
Location 2	21	19	-40	-48
Location 3	16	13	10	1
Location 4	-127	-160	29	38
Location 5	10	9	-49	-42
Location 6	10	11	-48	-56
Location 7	50	26	-9	-3
Location 8	42	28	-20	-8
Location 9	37	51	-39	-32
Location 10	73	85	-19	-16
Location 11	44	30	-7	-5
Location 12	43	32	-14	-10
Location 13	6	12	-13	-1
Location 14	-3	-3	-29	-1
Location 15	33	40	2	-3
Location 16	17	32	28	18
Location 17	4	10	1	0
Location 18	-2	-3	-18	-4

Table I-2. Measured change in strain in segment I between Stage One and Stage Two when the test ring was unloaded for overburden loads corresponding to 6m BGL and 24m BGL.

Unloading Case – Segment J straightens (positive moment)

The strain gauge locations for segment J are given in Figure 6-13.

The strain response for segment J is a mirror image of that for segment H. Location 2 measures the strain at the external corner of the circumferential flange close to the intrados of the joint at 180 degrees. The circumferential tensile strain at this location was significantly smaller when the overburden was low and the joint was beginning to open. The reverse happened on the internal corner of the circumferential flange, location 6. At this location the tensile circumferential strain was slightly higher when the overburden was low and the joint was tending to open.

For locations 9 and 10 on the extrados of the skin away from the horizontal flanges, the change in strain agreed with the expected behaviour for segments undergoing positive moment, with compressive circumferential strains and tensile radial strains.

For locations 17 and 18, the junction between the horizontal flange and the intrados of the skin, the tensile circumferential strains were once again higher when the overburden was low and the joint was tending to open.

Unloading Case – Segment J straightens	Circumferential strain ($\mu\epsilon$) +ve strain = tensile		Radial strain ($\mu\epsilon$) +ve strain = tensile	
	Approx 6m BGL	Approx 24m BGL	Approx 6m BGL	Approx 24m BGL
Location 1	5	3	-7	-15
Location 2	46	122	-40	-53
Location 3	6	10	1	1
Location 4	47	69	-7	-11
Location 5	8	7	-35	-27
Location 6	154	140	-36	-39
Location 7	29	23	-5	-4
Location 8	29	25	-4	-4
Location 9	-9	-16	12	20
Location 10	-19	-26	7	10
Location 11	19	12	-101	-75
Location 12	24	18	-112	-83
Location 13	9	12	-2	-3
Location 14	-2	7	1	-2
Location 15	-3	-4	-6	-9
Location 16	0	-6	-8	-4
Location 17	20	-1	-8	-15
Location 18	67	19	8	1

Table I-3. Measured change in strain in segment J between Stage One and Stage Two when the test ring was unloaded for overburden loads corresponding to 6m BGL and 24m BGL.

Loading Case – Segment H increases curvature (negative moment)

The strain gauge locations for segment H are given in Figure 6-11.

For segment H, location 2 measures the strain at the external corner of the circumferential flange close to the intrados of the joint at 60 degrees. The circumferential tensile strain at the location was only very slightly smaller when the overburden was low and the joint was beginning to open. The reverse happened on the internal corner of the circumferential flange, location 6. At this location the tensile circumferential strain was higher when the overburden was low and the joint was tending to open.

For locations 7 and 8, the junction between the horizontal flange at zero degree and the extrados of the skin, the tensile circumferential strains were higher when the overburden was low. However, for locations 13 and 14, the junction between the horizontal flange at zero degree and the intrados of the skin, the tensile circumferential strains were lower when the overburden was low. At the junction between the horizontal flange at 60 degrees and the skin, the difference in behaviour between the low overburden case and the high overburden case was unclear.

For locations 9 and 10 on the extrados of the skin away from the horizontal flanges, the change in strain agreed with the expected behaviour for segments undergoing negative moment, with tensile circumferential strains and compressive radial strains.

Loading Case – Segment H increases curvature	Circumferential strain ($\mu\epsilon$) +ve strain = tensile		Radial strain ($\mu\epsilon$) +ve strain = tensile	
	Approx 2m BGL	Approx 24m BGL	Approx 2m BGL	Approx 24m BGL
Location 1	-4	24	-11	-48
Location 2	0	2	-13	0
Location 3	0	-6	-1	-2
Location 4	-49	-68	27	5
Location 5	16	18	-61	-62
Location 6	21	4	-4	0
Location 7	58	32	-8	-4
Location 8	42	34	-11	-9
Location 9	23	27	-5	-8
Location 10	7	17	-13	-4
Location 11	1	2	-15	-8
Location 12	1	2	-11	-8
Location 13	4	10	-5	-1
Location 14	-5	0	-22	-2
Location 15	8	9	2	6
Location 16	-1	5	12	3
Location 17	-4	2	2	-3
Location 18	7	1	3	-3

Table I-4. Measured change in strain in segment H between Stage One and Stage Two when the test ring was loaded for overburden loads corresponding to 2m BGL and 24m BGL.

Loading case – Segment I straightens (positive moment)

The strain gauge locations for segment I are given in Figure 6-12.

The joints at the ends of segment I open at the intrados when the overburden is 2m BGL. At locations 1 and 2, the external corner on the circumferential flange, the tensile strains were lower when the overburden was low. In fact, at location 2, the strain at the external corner was compressive. However, at the internal corner of the circumferential flange, locations 5 and 6, the tensile strain was slightly higher for the low overburden case.

For locations 9 and 10 on the extrados of the skin away from the horizontal flanges, the change in strain agreed with the expected behaviour for segments undergoing positive moment, with compressive circumferential strains and tensile radial strains.

For locations 13, 14, 17 and 18, the junction between the horizontal flange and the intrados of the skin, the tensile circumferential strains were higher when the overburden was lower.

Loading case – Segment I straightens	Circumferential strain ($\mu\epsilon$) +ve strain = tensile		Radial strain ($\mu\epsilon$) +ve strain = tensile	
	Approx 2m BGL	Approx 24m BGL	Approx 2m BGL	Approx 24m BGL
Location 1	7	20	-7	-9
Location 2	-8	19	7	-7
Location 3	-7	-6	1	0
Location 4	57	85	-11	-20
Location 5	23	20	-7	-5
Location 6	26	24	-5	-5
Location 7	2	2	-28	-12
Location 8	2	4	-19	-13
Location 9	-20	-31	18	12
Location 10	-32	-45	6	11
Location 11	5	2	-29	-13
Location 12	6	4	-25	-14
Location 13	7	2	-5	-7
Location 14	16	0	6	0
Location 15	-15	-21	-1	0
Location 16	-5	-18	-14	-5
Location 17	5	1	-3	-5
Location 18	16	2	2	2

Table I-5. Measured change in strain in segment I between Stage One and Stage Two when the test ring was loaded for overburden loads corresponding to 2m BGL and 24m BGL.

Loading Case – Segment J increases curvature (negative moment)

The strain gauge locations for segment J are given in Figure 6-13.

For segment J, location 1 measures the strain at the external corner of the circumferential flange close to the intrados of the joint at 120 degrees. The circumferential strain at the location was compressive when the overburden was low and the joint was beginning to open. The reverse happened on the internal corner of the circumferential flange, location 5. At this location the tensile circumferential strain was slightly higher when the overburden was low and the joint was tending to open.

For locations 11 and 12, the junction between the horizontal flange at 180 degrees and the extrados of the skin, the tensile circumferential strains were higher when the overburden was low. However, for locations 17 and 18, the junction between the horizontal flange at 180 degrees and the intrados of the skin, the tensile circumferential strains were lower when the overburden was low. At the junction between the horizontal flange at 120 degrees and the skin, the difference in behaviour between the low overburden case and the high overburden case was unclear.

Loading Case – Segment J increases curvature	Circumferential strain ($\mu\epsilon$) +ve strain = tensile		Radial strain ($\mu\epsilon$) +ve strain = tensile	
	Approx 2m BGL	Approx 24m BGL	Approx 2m BGL	Approx 24m BGL
Location 1	-2	2	-9	2
Location 2	23	24	-47	-56
Location 3	-2	-6	-1	-1
Location 4	-24	-42	2	7
Location 5	22	7	-5	-1
Location 6	19	17	-70	-63
Location 7	2	3	-20	-11
Location 8	3	0	-17	-8
Location 9	4	10	8	1
Location 10	11	16	-4	6
Location 11	42	31	-10	-5
Location 12	43	34	-12	-8
Location 13	-4	2	3	-5
Location 14	7	2	3	-4
Location 15	2	2	3	5
Location 16	-5	3	6	2
Location 17	1	6	-3	1
Location 18	-4	0	-28	-6

Table I-6. Measured change in strain in segment J between Stage One and Stage Two when the test ring was loaded for overburden loads corresponding to 2m BGL and 24m BGL.

J. Figures

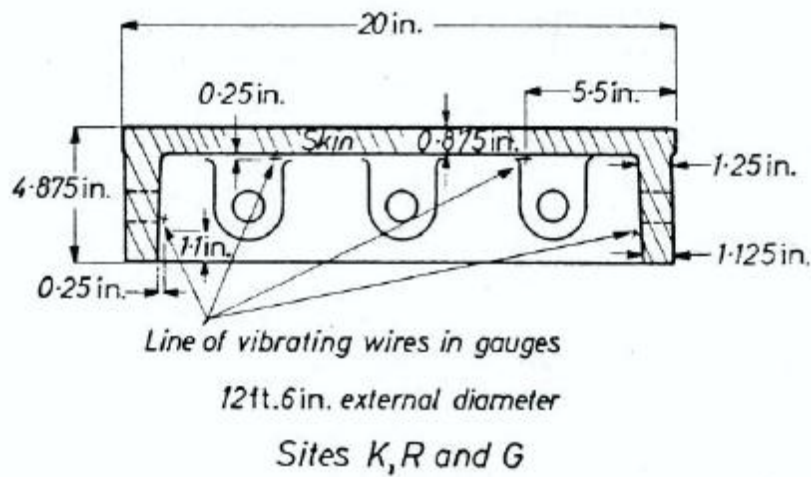


Figure A-1. Locations of vibrating wire strain gauges in 3.8m OD tunnels (Ward and Chaplin, 1957).

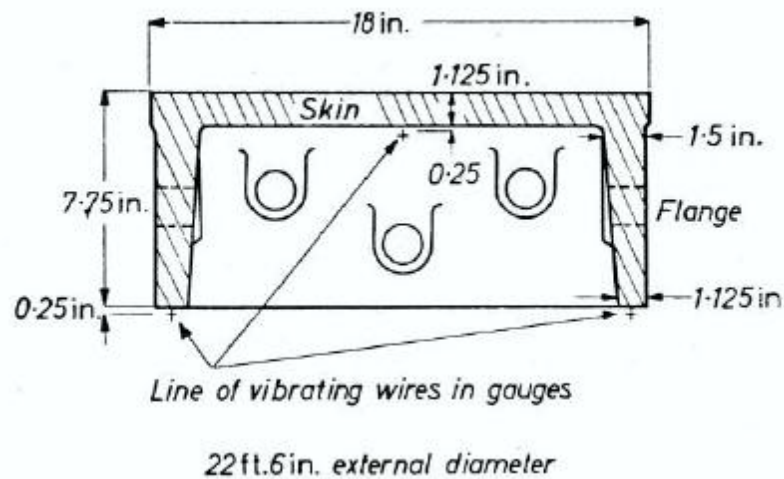


Figure A-2. Locations of vibrating wire strain gauges in 6.9m OD tunnels (Ward and Chaplin, 1957).

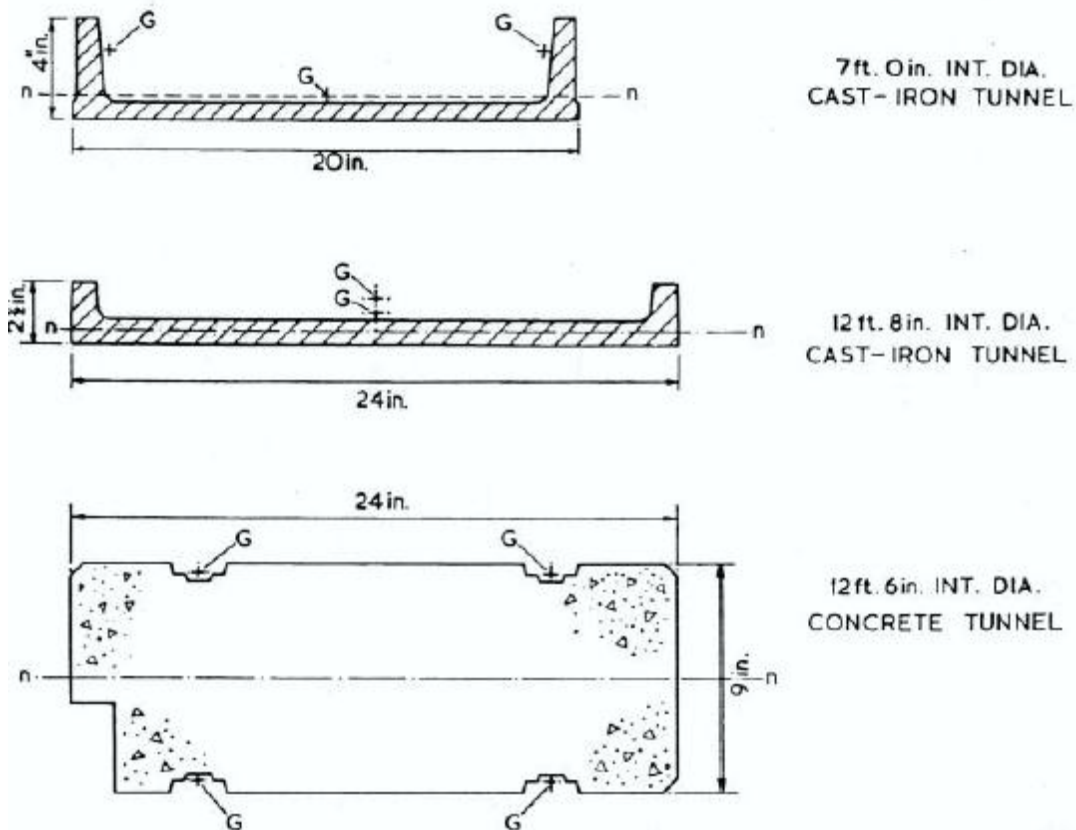


Figure A-3. Locations of vibrating wire strain gauges for Post Office tunnel (top), and the Victoria Line tube tunnels (middle and bottom). Gauge axis denoted by 'G' (Ward and Thomas, 1965).

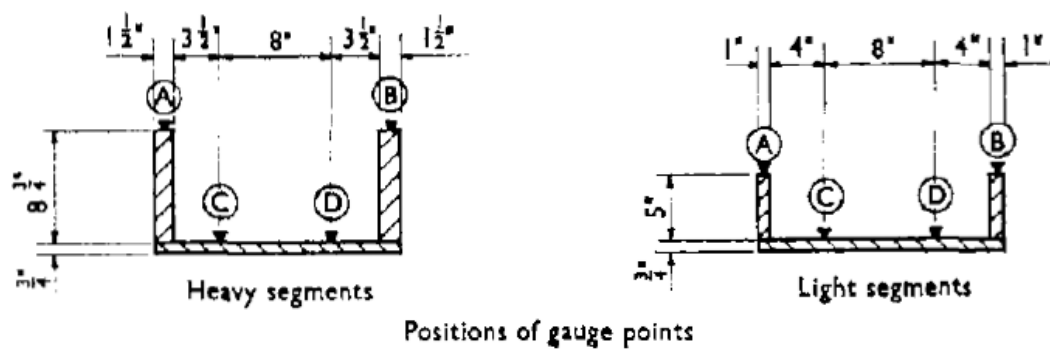


Figure A-4. Locations of Demec gauge at King's Cross station tunnel. (Smith Osborne, 1970).

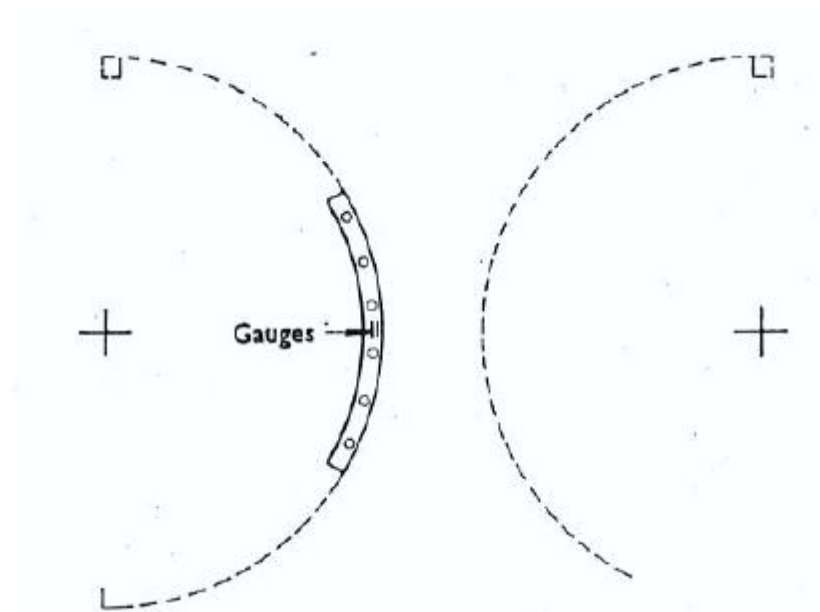


Figure A-5. Location of vibrating wire strain gauge on the Victoria Line extension to Brixton. (Thomas, 1976).

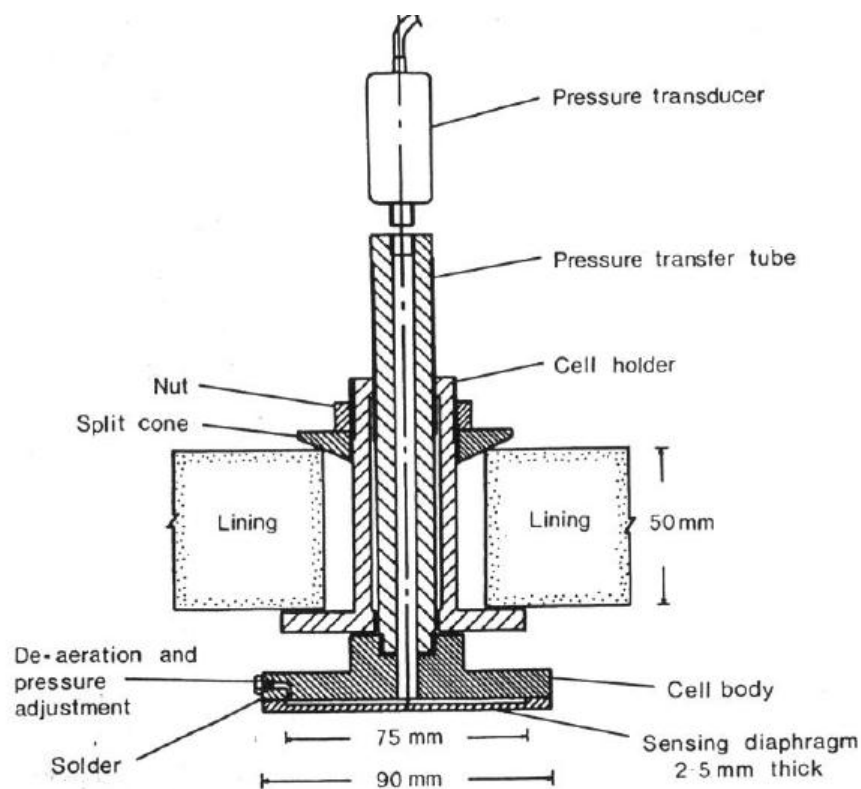


Figure A-6. Electro-hydraulic type earth pressure cell (Attewell and El-Naga, 1977).

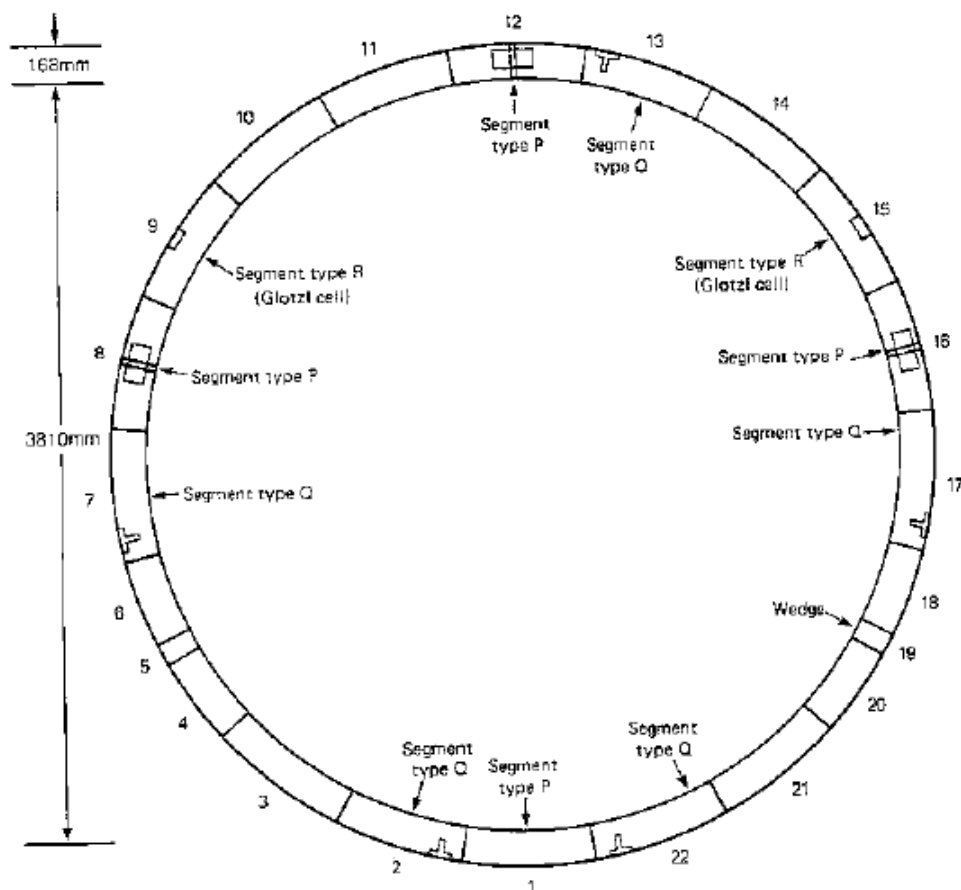


Figure A-7. Instrumented ring of the Jubilee Line (Barratt et al, 1994).

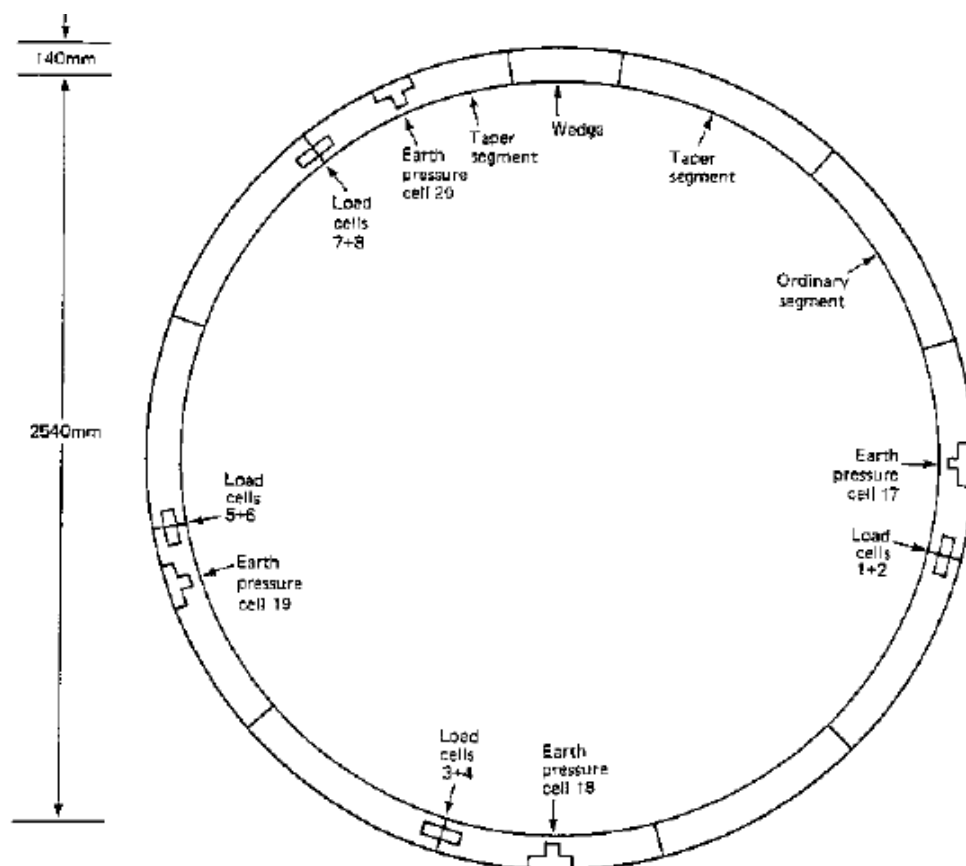
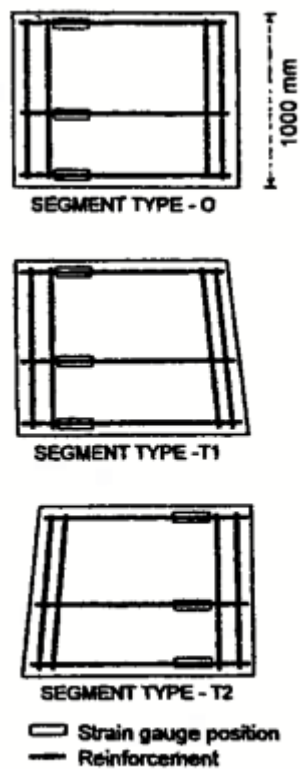
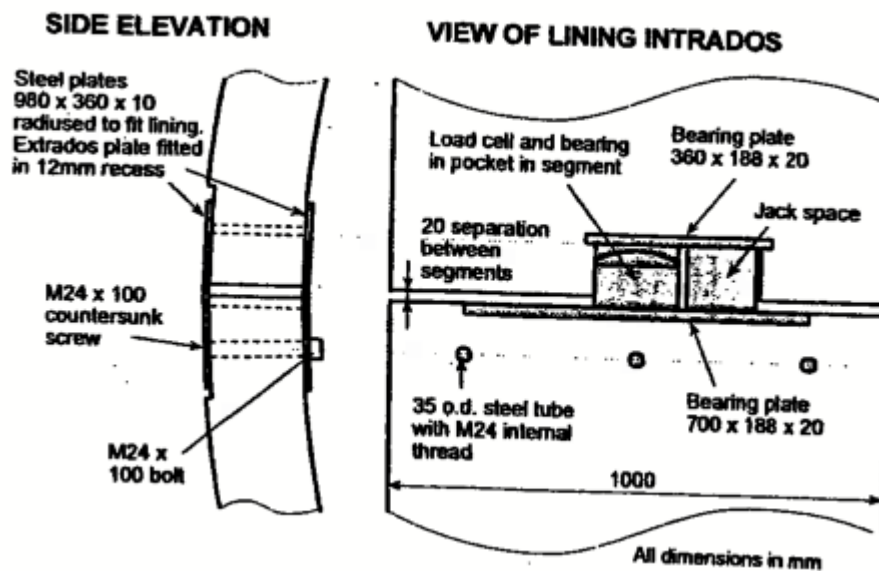


Figure A-8. Instrumented ring of the Oxford Trunk Outfall Sewer (Barratt et al, 1994).



(b) arrangement of strain gauges

Figure A-9. Vibrating wire strain gauge locations within segment. Jubilee Line Extension Project. (Davies and Bowers, 1996).



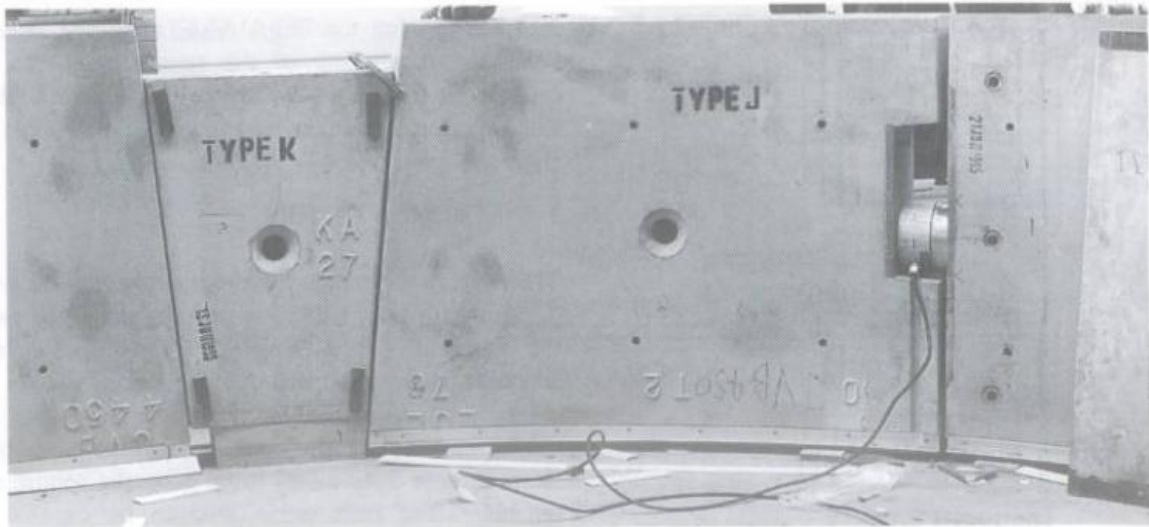


Figure A-10. Load cell arrangement within tunnel segment. Jubilee Line Extension Project. (Davies and Bowers, 1996).



Figure A-11. Vibrating wire strain gauge installed on skin of tunnel segment (Tube Lines, 2006).



Figure A-12. Instrumented portion of skin removed from tunnel segment (Tube Lines, 2006).

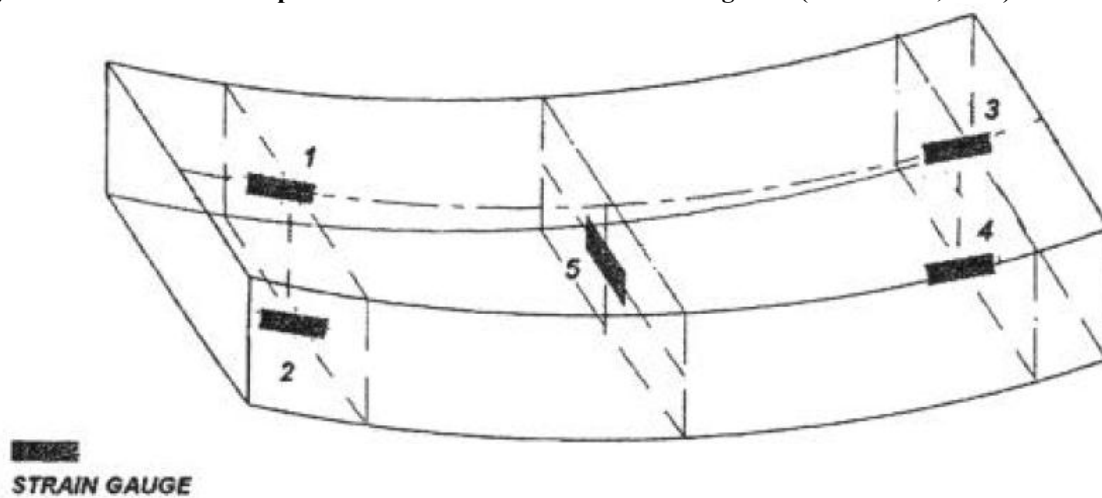


Figure A-13. Naples Metro Line 1 - strain gauge arrangement in concrete segments (Bilotta et al, 2006).

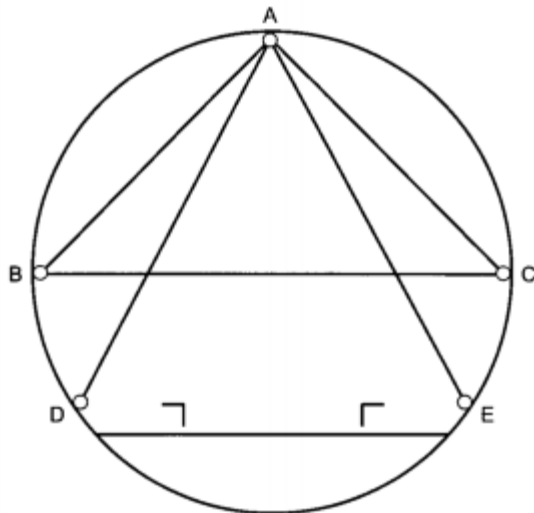


Figure A-14. Typical spans measured within sections of existing tunnels using a tape extensometer (Standing and Selma, 2001).

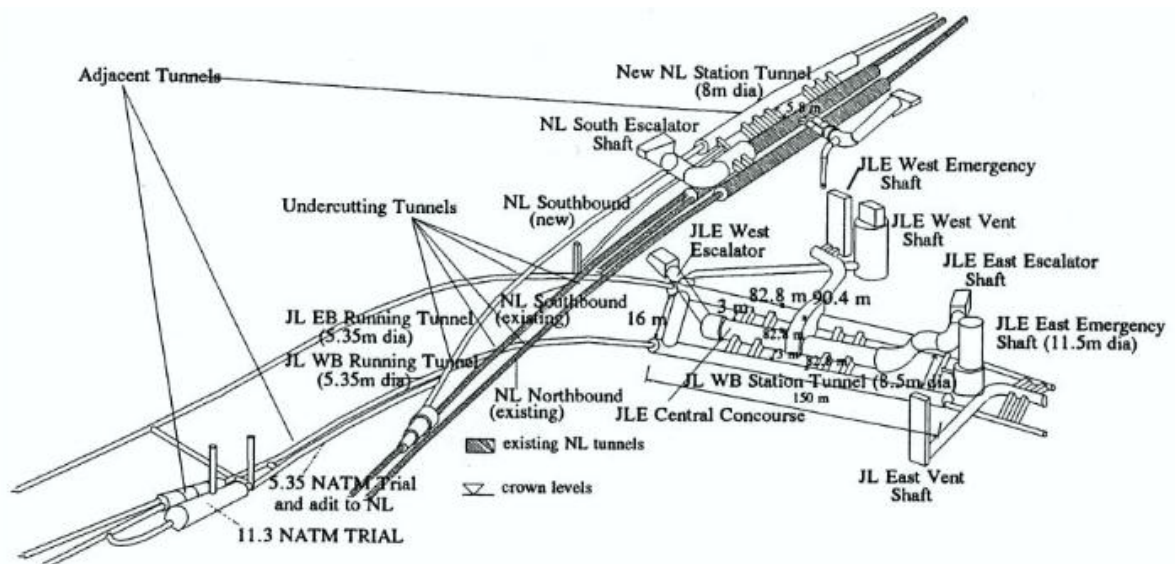


Figure A-15. A 3 dimensional plan of the tunnel construction for JLE London Bridge station (Kimmance et al, 1996).

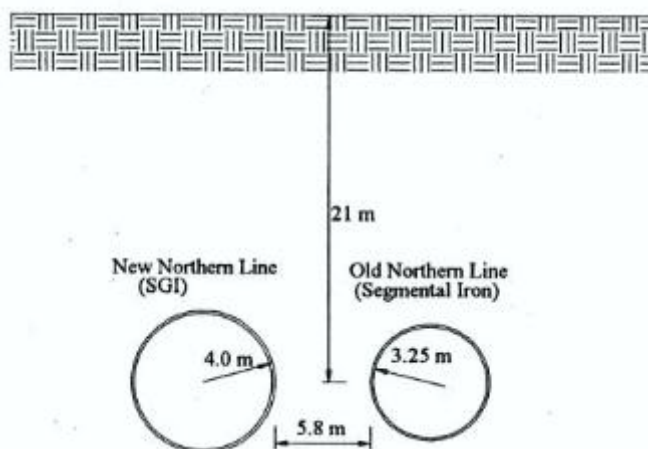


Figure A-16. Adjacent Northern Line tunnels (Kimmance et al, 1996).

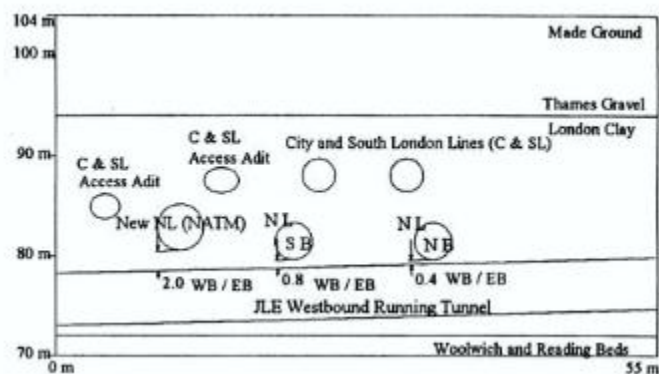


Figure A-17. Cross section of undercutting tunnels (Kimmance et al, 1996).

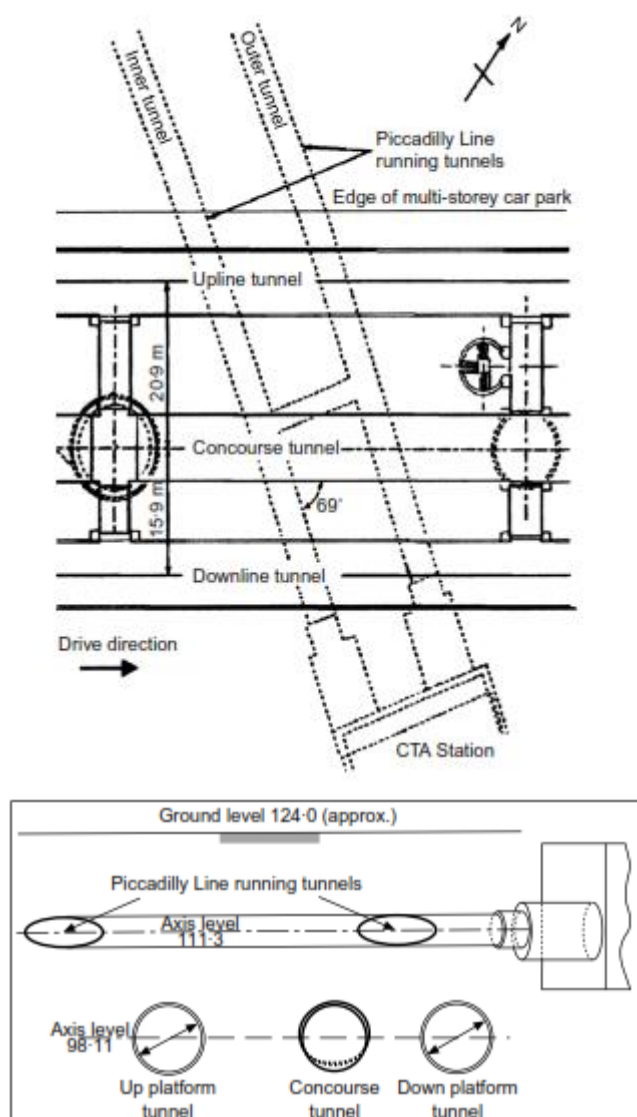


Figure A-18. Plan and cross-section at Heathrow Express Central Terminal area (Cooper et al, 2002).

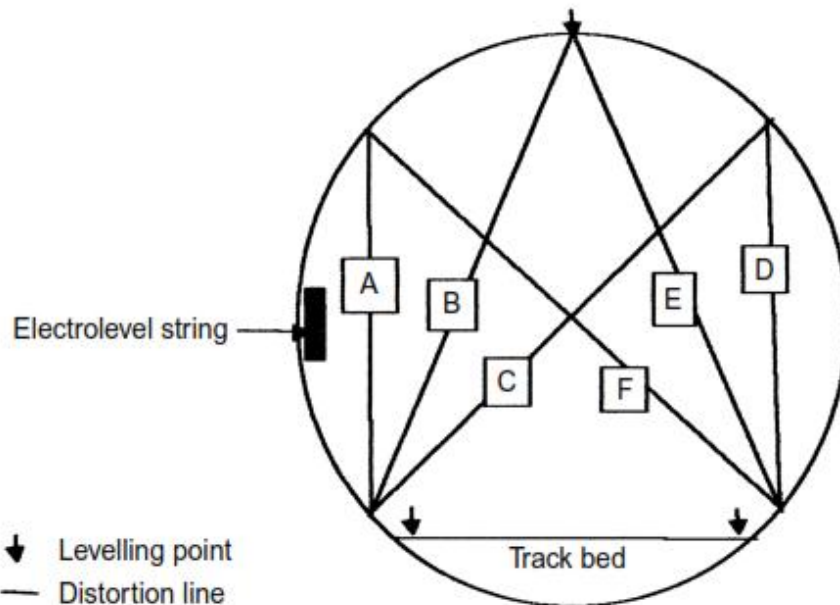


Figure A-19. Typical section of monitoring array for the Piccadilly Line tunnels (Cooper et al, 2002).

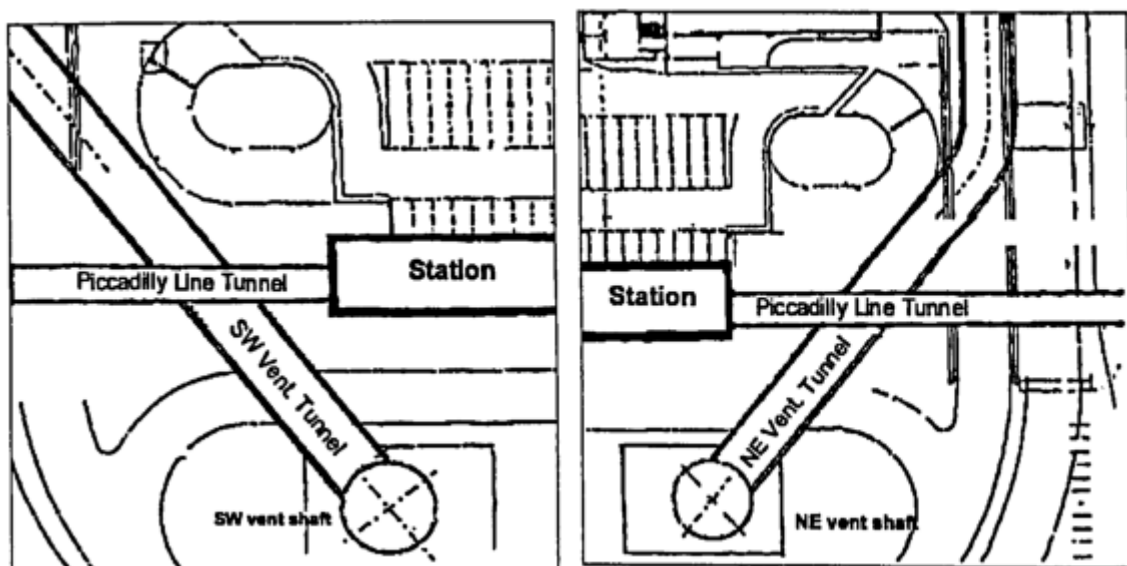


Figure A-20. Plan of SW and NE Heathrow Express vent tunnels at Heathrow T4 (Cooper, 2001).

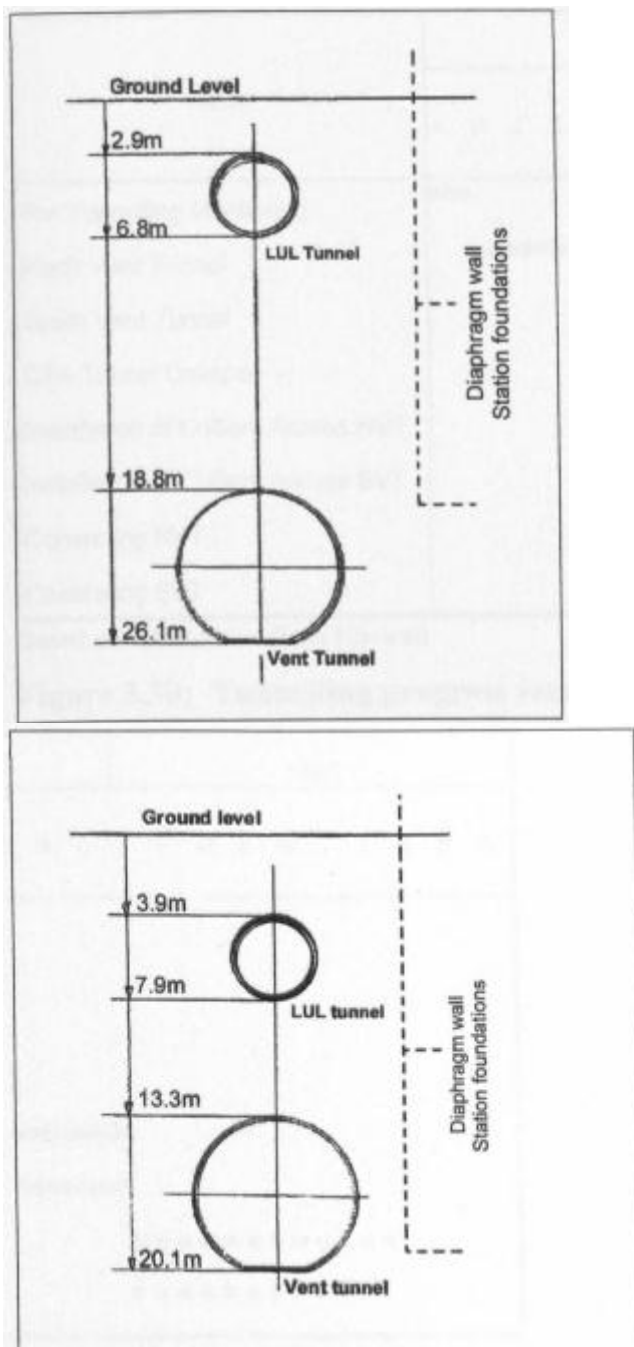


Figure A-21. Schematic sections at SW (left) and NE (right) vent tunnels, Heathrow T4 (Cooper, 2001).

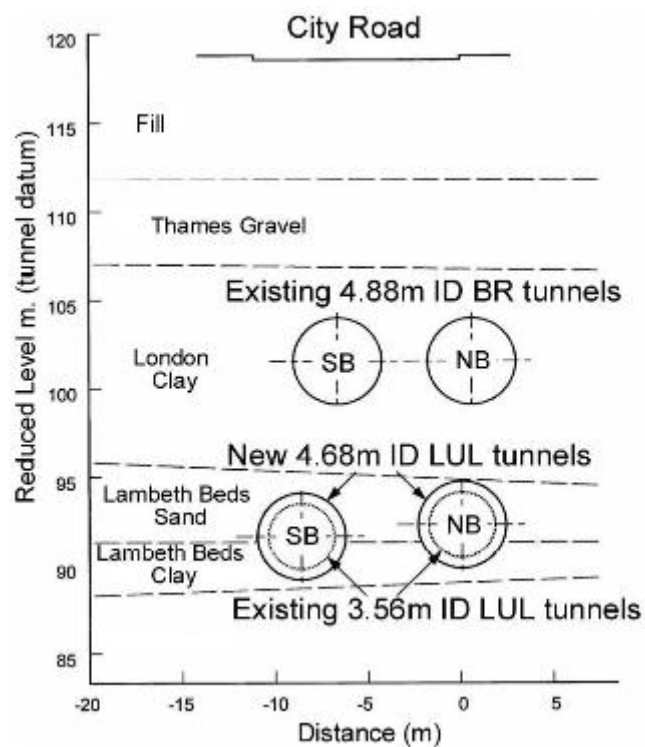


Figure A-22. Old Street reconstruction, typical section (Cooper et al, 2003).

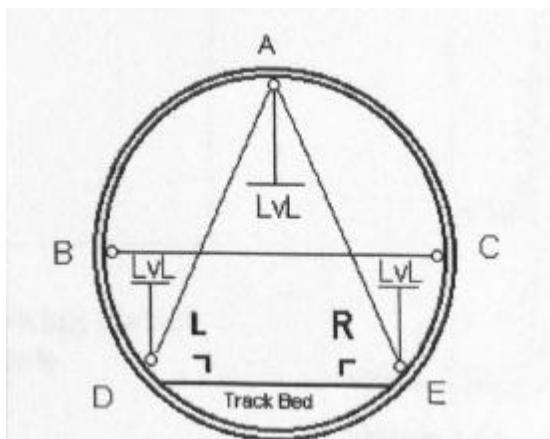


Figure A-23. Typical monitoring array for the mainline railway tunnel (Cooper, 2001).