# Performance of Linkage Bars

### for

# Restraint of Bridge Spans in Earthquakes

Prepared for:

***New Zealand Transport Agency***

Consultancy Services Contract Number: NO 09-554

Final Revision: 20 March 2012

JOHN WOOD CONSULTING

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Prepared by: J H Wood
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# Summary

The project involved both the testing of typical linkage bar assemblies and a review of analysis procedures used to determine the earthquake induced forces in bridge span linkage systems.

Linkage assemblies formed from a number of different types of steel bar were tested for strength, ductility and fracture toughness. An initial test on one bar type was followed by three tests on each of 12 types and two tests on one further type.

The analysis procedures that are currently used to evaluate the earthquake induced forces in linkage assemblies that restrain bridge spans were reviewed. Recommendations are made on the procedures best suited for application in New Zealand.

## Bar Types Tested

The bar types tested are summarised in Table S1. The testing was carried out in three stages with evaluation of results carried out following each stage.

**Table S1. Bar Assemblies Tested**

| Bar Type | Bar Identifier | Test Stage | Nominal Shank Dia. mm | Turned - Down Dia. mm | Loaded Length Between Nuts mm | Specified Min Yield or 0.2% Proof Load[1] kN | Specified Min Ultimate Load[1] kN |
|---|---|---|---|---|---|---|---|
| Reidbar - Grade 500E galvanised | RB32 | 1 | 32 | - | 620 | 402 | 462 |
| Reidbar - Grade 500E galvanised | RB25 | 3 | 25 | - | 880 | 246 | 282 |
| Macalloy S650 - Grade 316 stainless | MS32 | 1 | 32 | - | 660 | 506 | 622 |
| Round - galvanised mild steel | RM36 | 1 | 36 | - | 808 | 253 | 417 |
| Round - Grade 316 stainless steel | RS38 | 1 | 38 | - | 810 | 399 | 526 |
| Round - Grade 316 stainless steel | RS36 | 2 | 36 | - | 810 | 266 | 493 |
| Fully threaded - Grade 316 stainless steel | US36 | 2 | 36 | - | 810 | - | 725 |
| Freyssibar - high tensile steel | FH27 | 3 | 27 | - | 820 | 549[2] | 1092 |
| Turned-down - galvanised mild steel | TM30 | 1 | 36 | 30 | 810 | 219 | 360 |
| Turned-down - Grade 316 stainless steel | TS30 | 1 | 38 | 30 | 810 | 345 | 455 |
| Turned-down - galv. high tensile steel | TH30 | 2 | 36 | 30 | 810 | 573 | 953 |
| Round - galvanised mild steel ex Kaiapoi | RMK | 2 | 38 | - | 720 | 249 | 440 |
| Turned-down - galv. mild steel ex Otaki | TMO | 3 | 40 | 30 | 1060 | 190 | 300 |

Notes: 1. From test certificates supplied with bar where available or from product technical manuals.
2. From specified 0.1% proof stress.

The RB and MS bars were cut into lengths and tested without further modification using the proprietary nuts and locknuts supplied with the bar.

The RM36 and RS36 assemblies had a shank diameter of 36 mm and the RS38 a shank diameter of 38.1 mm. All three of these assembly types were fabricated with 150 mm long sections of M36 machine cut thread at either end.

The US36 bar was supplied in 1.0 m fully threaded lengths with a M36 cut thread. The lengths were cut to 950 mm to fit the testing machine but otherwise unmodified.

The FH27 bar was supplied with a 190 mm length of rolled thread at both ends, and a lamellar zinc coating (applied during rolling).

The TM30 and TS30 assemblies had the same maximum shank diameters as the RM36 and RS38 assemblies but a 300 mm long section of the shanks was turned down to 30 mm diameter. They had the same threaded ends as the RM36 and RS38 assemblies. The TH30 was similar in section geometry and thread to the TM30 assembly but fabricated from high tensile bar instead of mild steel and with a turned down length of 500 mm instead of 300 mm.

The RMK bar assemblies were recovered from the State Highway (SH) 2 Kaiapoi Railway River Bridge after the 2010 Darfield Earthquake. They were span linkage bars that failed at their head ends during the earthquake, and were fabricated from round galvanised mild steel 38.1 mm (1.5 inch) diameter bar. The recovered lengths were 990 mm long with a 130 mm length of thread at one end. For testing, 40 mm was cut from the fractured end and this end threaded with a 150 mm length of 1.5 inch Standard British Whitworth thread to suite the nuts recovered with the bars.

The TMO bar assemblies were recovered from movement joints located within the spans of the SH1 Otaki River Bridge during seismic strengthening work on the bridge. They were fabricated from round galvanised mild steel 40 mm diameter bar with a 660 mm length of shank turned down to 30 mm diameter. They had an overall length of 1270 mm with a 200 mm length of M36 thread at either end.

### Testing Procedure

Stages 1 and 2 of the tensile testing of the linkage assemblies were carried out using the Opus Central Laboratories 2 MN Servotest Universal Testing Machine (UTM). Because of a failure in the electronic control system on the testing machine, Stage 3 of the tensile testing was carried out using a 1 MN centre hole hydraulic jack and a strain-gauge load cell set-up on a special purpose support frame.

Prior to testing, gauge lengths of about 100 mm were marked out along the bars using a small indentation formed with a centre-punch. Prior to and after each test, the gauge lengths were measured to the nearest 0.1 mm using vernier calipers.

Special purpose loading frames were fabricated from mild steel to hold the bar assemblies in the UTM. These allowed the load to be applied to the faces of the washers and nuts to simulate the application of load in a typical bridge situation where the bars are installed between concrete diaphragms or steel brackets. These loading frames were not required when using the hydraulic jack since it was possible to apply the load using washers bearing against the jack ram and the end of the load cell. The 280 mm long jack had a limited travel of 75 mm and for the bars where the extension exceeded this limit it was necessary to unload and reload after increasing the loaded length using a variety of large spacer washers ranging in thickness from 20 to 50 mm.

In Stage 1 and 2 of the testing the loaded length of the bars varied between 620 and 810 mm. This length was generally chosen to be the maximum that could be accommodated in the UTM fitted with loading frames, however it was reasonably typical of bridge linkage installations which have loaded lengths of 600 mm to 1200 mm. The RMK bars were tested at a loaded length of 720 mm compared to the length of about 900 mm used in Kaiapoi Railway River Bridge. The TMO bars were tested (Stage 3) at a loaded length of 1060 mm which was about the length used in the Otaki River Bridge movement joints.

## Summary of Testing Results

The measured minimum yield and ultimate loads and the ratio of the minimum measured ultimate load to the specified minimum ultimate load for each of the bar types are summarised in Table S2. Minimum elongations for each bar type expressed in both millimetres and percentage of the loaded length are also listed.

**Table S2. Measured Yield and Ultimate Loads and Elongations**

| Bar Identifier | Measured Minimum Yield or 0.2% Proof Load | Measured Minimum Ultimate Load | Ratio of Minimum Ultimate Load/ Specified Minimum Ultimate Load | Loaded Length | Minimum Elongation on Assembly Loaded Length | |
|---|---|---|---|---|---|---|
| | kN | kN | | mm | mm | % |
| RB32 | 430 | 530 | 1.15 | 620 | 107 | 17 |
| RB25 | 290 | 355 | 1.26 | 880 | 124 | 14 |
| MS32 | 550 | 850 | 1.37 | 660 | 40 | 6 |
| RM36 | 240 | 375 | 0.90 | 808 | 63 | 8 |
| RS38 | 450 | 580 | 1.10 | 810 | 32 | 4 |
| RS36 | 340 | 535 | 1.08 | 810 | 81 | 10 |
| US36 | 500 | 660 | 1.11 | 810 | 72 | 6 |
| FH27 | 620 | 665 | 1.09 | 820 | 27 | 3 |
| TM30 | 210 | 330 | 0.92 | 810 | 85 | 11 |
| TS30 | 400 | 510 | 1.12 | 810 | 72 | 9 |
| TH30 | 620 | 715 | 1.06 | 810 | 24 | 3 |
| RMK | 280 | 445 | 1.12 | 720 | 38 | 5 |
| TMO | 180 | 325 | - | 1060 | 141 | 13 |

- The RB32 and RB25 assemblies had the greatest plastic elongation at 17% and 14% of the loaded lengths respectively. Yielding followed by strain hardening in the failure section resulted in significant plastic elongation spreading over most of the loaded length.
- The MS32 and TH30 assemblies had the highest ultimate loads but their elongations of 6% and 3% respectively were much lower than the elongations of the RB32 and RB25 assemblies. A large part of the total elongation of the MS32 and TH30 assemblies (between 65% and 85%) occurred on the gauge length that necked-down and failed and in the two adjacent gauge lengths. Unlike the RB32 and RB25 bars, the medium tensile MS and high tensile TH30 bars do not have a distinct yield plateau followed by significant strain hardening. When present these features spread plastic strains over a large part of the loaded length.
- The RM36, RMK, RS38, RS36, US36 and FH27 assemblies failed in loaded sections of thread. The elongations in the failure gauge lengths were quite high at 30%, 28% and 43% for the RM36, RS38 and RS36 assemblies respectively showing that the cut thread did not greatly reduce the tensile ductility.
- There was not a large difference between the plastic elongations measured for the RM36 and TM30 assemblies (8% compared to 11%). However, obtaining good elongations from round mild steel assemblies requires the shank diameter to be no larger than the thread nominal diameter and since a large part of the elongation occurs in the threaded length (about 55% of the total elongation) the loaded sections of thread at both ends of the bar need to be quite long. (Comparison of the elongations for the RS38 and RS36 assemblies shows the influence of the shank diameter. A precise comparison cannot be made since although the bars in both

assemblies were Grade 316 stainless steel, they were fabricated from bar supplied by different manufacturers.)

- Turning the bars down to ensure that failure does not occur in the threads will generally increase the elongation and give greater reliability of performance since a turned down bar assembly is not sensitive to installation tolerances that could affect the length of thread in the loaded length.
- Except for an initial test on a RB32 assembly, the first test of the RS38 assemblies, the final test of the FH27 assemblies and the two tests of the RMK assemblies, all assemblies were set-up with locknuts or two standard nuts fitted at both ends. A nut failed before the bar in both the initial RB32 test and the first of the RS38 assembly tests. All proprietary bar linkage systems should be fitted with the proprietary nut and locknut. Fabricated linkage bar systems should be fitted with two standard nuts at each end.
- Apart from the FH27 and TH30 assemblies, all the bar types tested provided satisfactory ductile performance with plastic elongations between 4% (32 mm) of the loaded length for the RS38 assembly and 17% (107 mm) for the RB32 assembly. (The RS38 performance could be improved by having the shank diameter the same as the thread diameter.) Elongations of less than 30 mm (3%) for the FH27 and TH30 assemblies indicated that high tensile steel is unlikely to satisfactory for most bridge linkage bar applications.

### Linkage Bar Design Forces

The most satisfactory method of determining design forces in longitudinal linkage systems connecting the superstructure to abutments and piers is to carry out a static push-over analysis modelling the stiffness of all the substructure components. The analysis can be carried out using a spreadsheet approach with force versus displacements functions developed for the abutments and piers from published information on the passive soil resistance against abutment walls and the displacement response of pile foundations.

For short bridges of up to three spans and where the abutments are clearly a lot stiffer under longitudinal horizontal loads than the piers it would be reasonable to design the linkages at the abutments to carry the total superstructure longitudinal inertia load.

Analysis methods for calculating the design forces in linkage systems are complicated and liable to produce approximate results for a number of reasons including; uncertainty in ground conditions and input motions, soil-structure interaction, permanent ground deformations, travelling wave effects along the bridge length, and concurrency of longitudinal and transverse response of the superstructure.

Maximum likely forces in linkage systems can be estimated by using an envelope of parameter values. The problem with such an approach is that it can produce impractically large demands. The solution is therefore to provide well proportioned, practically sized linkages and reliable fixtures for them and to accept that the linkages will be required to yield. It is often more practical to provide more generous span/support overlaps than to provide greater linkage strength.

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## 1. Introduction

### 1.1 Background

Many New Zealand bridge superstructures consist of simply supported spans, which are interconnected with steel linkage bars. The main purpose of the bars is to restrain the bridge spans in the event of an earthquake and prevent the spans from falling. The prediction of the forces that may be imposed on the linkages is quite indeterminate because of the many variables that affect the responses of adjacent bridge spans during strong earthquake motions. For economic reasons it is also usually not practical to make the linkages so strong that they will never fail under the strongest likely shaking. Linkage bars are therefore designed for a reasonable and practical strength, and are then detailed to yield and stretch rather than fail at low extensions.

Seismic screening of the state highway (SH) bridges identified that there were 180 bridges that required improvement to their inter-span linkages. The work on the 140 bridges on Priority 1 and 2 routes is substantially complete and design work has commenced on some of the 40 Priority 3 route bridges. There are also thought to be a significant number of important bridges administered by Territorial Authorities that require improvements to inter-span linkages. Although not all of the 100 plus bridges requiring inter-span linkages will require linkage bars it is likely that more than half of them will.

Detailed seismic assessments have been completed on about 70 of the 336 bridges that the screening identified as requiring investigation. About 65% of the bridges assessed require improvements to their earthquake resisting elements and have been ranked for design and construction of the required work. Many of the bridges that have been assessed have been found to also require additional inter-span linkages or improvements (including replacement) to their inter-span linkages to meet the requirements of the New Zealand Bridge Manual (BM) (NZTA, 2003).

### 1.2 Uses of Linkage Bars and Reasons for This Project

Linkage bars were installed between the spans and at deck hinge joints on many SH bridges constructed from about 1960 onwards. Most of the linkages were fabricated from steel rod and galvanised for corrosion protection. In marine environments, or where water has leaked through the deck joints onto the bars, the galvanising has deteriorated to the point where corrosion of nuts and parts of the rods is significant and ongoing replacement of some of these linkages will be required.

In new long-span bridges there are often benefits in using a continuous deck to eliminate deck joints and the need for inter-span linkages. However, for shorter bridges the use of simply supported precast concrete superstructure components is economical. For this type of construction it may not be practical to use a continuous deck and linkage systems at the supports are required to prevent spans falling.

One method of improving the earthquake resistance of shorter bridges is to develop horizontal diaphragm action using the deck and beams to transfer some of the horizontal earthquake load normally carried by the piers to the abutments, which often have better transverse load resistance than the piers. To achieve satisfactory diaphragm action on bridges with simply supported spans it is necessary to strengthen the inter-span linkages on the outer beams.

To successfully make use of restraint from soil/structure interaction at the abutments it is necessary to use monolithic abutments or to have a robust linkage between the superstructure and abutments. If precast beams are used, it may be more practical to use a linkage bar system to make the connection than to form an abutment monolithic with the superstructure.

About thirty years ago a design of linkage bar was developed that included a machined-down length of bar, which would cause the yielding to develop within its length, rather than failure occurring in the threaded length of the bar near to the nuts. This design was developed by logic but has never been investigated by testing to evaluate the effect on performance of possible variables, such as the length of machining. In the intervening years a number of other options for linkage bars have become available, such as Reidbar and proprietary stainless steel bars with rolled threads. The main aim of the project was investigate and evaluate the various options for linkage bars now available, and to set out appropriate design methods.

The fracture toughness of materials used for linkage bolts is required to meet the criteria set out in the Steel Structures Standard NZS 3404:2009. This is particularly important for those used in areas of low temperature. The fracture toughness of the materials used in the sample linkage bolts tested therefore required investigation, as reported in Section 7.

Further background information on linkage bar use in New Zealand is given in Appendix A. Results from tensile tests carried out on 25.4 mm diameter galvanised mild steel linkage bars removed from the Ahuriri River Bridge on SH 8 after being in service for about 45 years are given in Appendix B. Bridges that have recently been assessed as needing linkage bars or have had linkage bars recently installed are listed in Table C1 in Appendix C.

## 2. Project Outline

The project involved three main components as described below.

### 2.1 Testing

Thirteen types of linkage assembly, formed from various types of steel bar, were tested for tensile strength and ductility. Three samples of each type of linkage assembly were tested, except for RB32 (4 tests) and RMK (2 tests). The tensile testing was carried out in three stages over a period of 18 months, each stage comprising all samples of each of several of the types of linkage assembly. Results were processed and reviewed at the completion of each stage before proceeding to the following stage. The linkage assemblies bar types and stages of testing are listed in Table 1.

The thirteen types of linkage assembly comprised:

- Three types of proprietary threaded bars in the as-supplied condition.
- Four types of bars with plain round shanks. Three of these had standard threads cut by machining and one had proprietary rolled threads.
- One type of bar with its complete length threaded by machining.
- Three types of bars with turned-down shanks.
- Two types of bars recovered from older bridges. One of these had a plain round shank and the other a turned-down shank.

The fracture toughness of the materials used in the test samples was evaluated from test certificates received with the sample materials, or from test results recently carried out by others.

Fracture toughness tests were carried out on bar type TH30 in the absence of available information for this type of steel. The evaluation and test results are reported in Section 7.

### 2.2 Analysis Procedures

The analysis procedures that are currently used to evaluate the earthquake induced forces in linkage assemblies that restrain bridge spans were reviewed. Recommendations are made on the procedures best suited for application in New Zealand and worked examples provided to demonstrate their application.

### 2.3 Dissemination of Results

This report has been written in the form of a design guideline that can be referenced by the Transit New Zealand Bridge Manual (BM) (NZTA, 2003). A paper describing Stage 1 of the tensile testing was presented at the Ninth Pacific Conference on Earthquake Engineering (Wood and Chapman, 2011). Further papers will be written for the Bulletin of the New Zealand Society for Earthquake Engineering (NZSEE) and the Journal of the Structural Engineering Society New Zealand (SESOC).

## 3. Linkage Assemblies Tested

### 3.1 Linkage Assemblies

Details of the 13 types of linkage assemblies tested are given in Table 1. The general arrangement of the assemblies showing threaded, turned-down (where used) and gauge lengths are shown in Figures 1 and 2.

**Table 1. Linkage Assemblies**

| Linkage Assembly | Linkage Identifier | Test Stage | Number Tested | Nominal Shank Dia. mm | Turned - Down Dia. mm | Loaded Length Between Nuts mm | Specified Min. Yield or 0.2% Proof[1] kN | Specified Min. Ultimate Load[1] kN |
|---|---|---|---|---|---|---|---|---|
| Reidbar - Grade 500E galvanised | RB32 | 1 | 4 | 32 | - | 620 | 402 | 462 |
| Reidbar - Grade 500E galvanised | RB25 | 3 | 3 | 25 | - | 880 | 246 | 282 |
| Macalloy S650 - Grade 316 stainless | MS32 | 1 | 3 | 32 | - | 660 | 506 | 622 |
| Round - galvanised mild steel | RM36 | 1 | 3 | 36 | - | 810 | 253 | 417 |
| Round - Grade 316 stainless steel | RS38 | 1 | 3 | 38 | - | 810 | 399 | 526 |
| Round - Grade 316 stainless steel | RS36 | 2 | 3 | 36 | - | 810 | 266 | 493 |
| Fully threaded - Grade 316 stainless steel | US36 | 2 | 3 | 36 | - | 810 | - | 725 |
| Freyssibar - high tensile steel | FH27 | 3 | 3 | 27 | - | 820 | 549[2] | 1092 |
| Turned-down - galvanised mild steel | TM30 | 1 | 3 | 36 | 30 | 810 | 219 | 360 |
| Turned-down - Grade 316 stainless steel | TS30 | 1 | 3 | 38 | 30 | 810 | 345 | 455 |
| Turned-down - galv. high tensile steel | TH30 | 2 | 3 | 36 | 30 | 810 | 573 | 953 |
| Round - galvanised mild steel ex Kaiapoi | RMK | 2 | 2 | 38 | - | 720 | 249 | 440 |
| Turned-down - galvan. mild steel ex Otaki | TMO | 3 | 3 | 40 | 30 | 1060 | 190 | - |

Notes: 1. From test certificates supplied with bar where available or from product technical manuals.
2. From specified 0.1% proof stress.

### 3.2 Bar Material and Tensile Strength

Specified yield and ultimate strengths of the bars were obtained as follows:

- Minimum yield and ultimate loads for the galvanised Grade 500E (micro alloyed) Reidbars (RB32 and RB25) and Macalloy S650 (Grade 316 Stainless Steel) (MS32) bars were taken from product technical manuals.
- The mild steel bar used for bar types RM36 and TM30 was manufactured as Grade300Plus, 39 mm diameter black bar. The yield and ultimate loads were calculated from yield stress and ultimate tensile stress (UTS) values given on the test certificate provided by the bar supplier.
- The Grade 316 stainless steel bar used for bar types RS38, RS36 and TS30 bar was manufactured as bright 38.1 mm diameter (1½ inch) bar. The yield and ultimate loads were calculated from yield stress and UTS values given on the test certificate provided by the bar supplier. The steel used in the RS38 and TS30 bars was from the same source but the steel used in the RS36 bars was from a different source.
- The specified ultimate load for the Grade 316 stainless steel in the fully threaded US36 bars was taken from the manufacturer's test certificate. No yield strength was given on the certificate.
- The yield load and ultimate loads for the Freyssibar (FH27) were calculated from the yield stress and the UTS given on a test certificate provided by the bar supplier. The yield stress was given as a 0.1% proof stress.
- The high tensile TH30 bars were from type SCM440 steel to Japanese Industrial Standard (JIS) G4053. (This steel is commonly referred to by local suppliers as 4140 steel or AISI 4140 steel.) Yield and ultimate loads were calculated from the yield stress and the UTS given on a test certificate provided by the bar supplier. The certificate indicated that the bar was cold drawn and quenched and tempered to 850-1000 MPa.
- The mild steel bar in the RMK assemblies recovered from the Kaiapoi Railway River Bridge was assumed to have a minimum yield stress of 275 MPa and UTS of 440 MPa. These are typical strength values for mild steel bar manufactured at the time of construction of the bridge (1970). No indication of the steel strength was given on the bridge drawings.
- The mild steel bar in the TMO assemblies recovered from the Otaki River Bridge was specified on the drawings as Grade 300 to NZS 3402 with a bar dependable yield force of 190 kN. The UTS for the bar was assumed to be 440 MPa as given in AS/NZS 3679.1 for Grade 300 steel. (The bridge linkage assemblies were retrofitted in 1991 or 1992 and NZS 3402: 1989, *Steel Bars for the Reinforcement of Concrete* used at the time of the design has since been superseded.)

### 3.3 Fabrication of Linkage Assembly Bars

- The RB32 and RB25 bars were supplied as galvanised deformed bar with the rolled deformations forming a coarse thread for engaging the nuts. The MS32 bar was supplied with a continuous rolled thread. All three types were cut into lengths and tested as-supplied. The assembly lengths shown in Figure 1 of 870 mm and 950 mm for the RB32 and MS32 bars respectively where essentially the maximum lengths that could be fitted into the universal testing machine used for Stage 1 and 2 of the tensile tests. Because of the longer nuts used on the RB32 and MS32 assemblies the loaded length between the nuts was a little shorter than for the other bars (see Table 1). The RB25 bars were tested in Stage 3 using a centre-hole jack, and as their length was not restricted by the testing equipment they were made longer than the RB32 bars.

- The black bar for both the round RM36 and turned-down TM30 mild steel assemblies was initially turned-down to 36 mm diameter over the full length of the bars. A 150 mm long M36 cut thread (ISO metric coarse pitch) was added at either end. The TM30 bars had a 300 mm long section turned-down to 30 mm diameter with 50 mm long tapers at both ends. The bars were galvanised after fabrication. It was intended that the complete lengths including the threads at both ends be galvanised but the initial bar lengths were found to be too long to fit into the testing machine and a section had to be cut-off with a new thread machined at one end. The bars were not regalvanised after this modification.
- The 150 mm threaded length on each end of the round RS38, RS36 and turned-down TS30 stainless steel bars was turned-down to 36 mm diameter to suite M36 nuts (ISO metric coarse pitch thread). The TS30 bars also had a 300 mm long section turned-down to 30 mm diameter with 50 mm long tapers at either end. The sections of the RS38 and TS30 bars that were not turned-down for the threads, or to the 30 mm reduced diameter and tapers, were left at the 38.1 mm diameter of the supplied bar. The RS36 bars were turned down from 38.1 mm to 36 mm diameter over their full lengths prior to threading. All three types had machine cut threads.
- The US36 bar was supplied fully threaded in 1.0 m lengths with a machine cut M36 thread (ISO metric coarse pitch). The lengths were cut to 950 mm to fit the testing machine but otherwise unmodified.
- The FH27 bars were supplied as 27 mm diameter round 950 mm long bars with a 190 mm length of rolled thread at both ends, and a lamellar zinc coating (applied during rolling).
- The TH30 bars were turned down from 38.1 mm cold drawn SCM440 high tensile bar to have similar section geometry and thread to the mild steel TM30 bars except that the turned down length was increased from 300 mm to 500 mm.
- The RMK bar assemblies were recovered from the SH2 Kaiapoi Railway River Bridge after the 2010 Darfield Earthquake. They were span linkage bars that failed at their head ends during the earthquake, and were fabricated from round galvanised mild steel 38.1 mm (1.5 inch) diameter bar. The recovered lengths were 990 mm long with a 130 mm length of thread at one end. For testing, 40 mm was cut from the fractured end and this end threaded with a 150 mm length of 1.5 inch Standard British Whitworth cut thread to suite the nuts recovered with the bars.
- The TMO bar assemblies were recovered from movement joints within the spans of the SH1 Otaki River Bridge during seismic strengthening work on the bridge. The bars were fabricated from round galvanised mild steel 40 mm diameter bar with a 670 mm length of shank turned down to 30 mm diameter and tapered as shown in Figure 2. They had an overall length of 1270 mm with a 200 mm length of M36 thread at both ends.

### 3.4 Nuts and Washers

Details of the nuts, locknuts and washers used in the test assemblies are summarized in Table 2.

### Table 2. Nuts and Washers Used on Test Assemblies

| Linkage Assembly | Linkage Identifier | Thread Type | Thread Pitch mm | Nut Property Class | Nut Height x Width Across Flats mm | Washer Thickness x Diameter or Width mm x mm | Washer Hole & Backing Hole Diameter mm | Lock Nuts |
|---|---|---|---|---|---|---|---|---|
| Reidbar - Grade 500E galvanised | RB32 | Ribbed bar | 15 | - | 82 x 54 | 10 x 75 sq | 38 / 42 | Half nut: 39 mm height[1] |
| Reidbar Grade 500E galvanised | RB25 | Ribbed bar | 12.5 | - | 65 x 46 | 6 x 75 sq | 28 / 40 | Half nut: 31 mm height |
| Macalloy S650 Grade 316 stainless | MS32 | Rolled continuous | 6 | - | 41 x 56 | 5 x 70 dia | 37 / 42 | Half nut: 20 mm height |
| Round galvanised mild steel | RM36 | M36 | 4 | 8 | 37 x 59 | 3 x 72 dia | 38 / 42 | Standard nut |
| Round Grade 316 stainless | RS38 | M36 | 4 | A4-70 | 28 x 54 | 5 x 70 dia | 37 / 42 | Standard nut[1] |
| Round Grade 316 stainless | RS36 | M36 | 4 | A4-70 | 28 x 54 | 5 x 70 dia | 37 / 38 | Standard nut |
| Fully threaded Grade 316 stainless | US36 | M36 continuous | 4 | A4-70 A4-80[2] | 28 x 54 | 5 x 70 dia | 37 / 38 | Standard nut |
| Freyssibar high tensile steel | FH27 | M27 rolled | 3.5 | 10 | 30 x 46 | 12 x 100 sq | 32 / 40 | Standard nut[3] |
| Turned-down galvanised mild steel | TM30 | M36 | 4 | 8 | 37 x 59 | 3 x 72 dia | 38 / 38 | Standard nut |
| Turned-down Grade 316 stainless | TS30 | M36 | 4 | A4-70 | 28 x 54 | 5 x 70 dia | 38 / 38 | Standard nut |
| Turned-down galvanised high tensile | TH30 | M36 | 4 | 8 | 37 x 59 | 5 x 70 dia | 38 / 38 | Standard nut |
| Round - galvanised mild steel ex Kaiapoi | RMK | $1^1/_2$ inch BSW | 4.23 | - | 36 x 56 | 12 x 100 sq | 40 / 40 | Half nut: 18 mm height[4] |
| Turned-down – galv. mild steel ex Otaki | TMO | M36 | 4 | - | 30 x 59 | 19 x 165 sq | 40 / 52 | Standard nut |

Notes:
1. Locknuts were not used in the first of the four RB32 and the first of three RS38 tests.
2. A4-80 nuts were used in the first test and A4-70 nuts used in the other two tests.
3. Locknuts were not used in the final test of the three tests.
4. Locknut only used on one end.

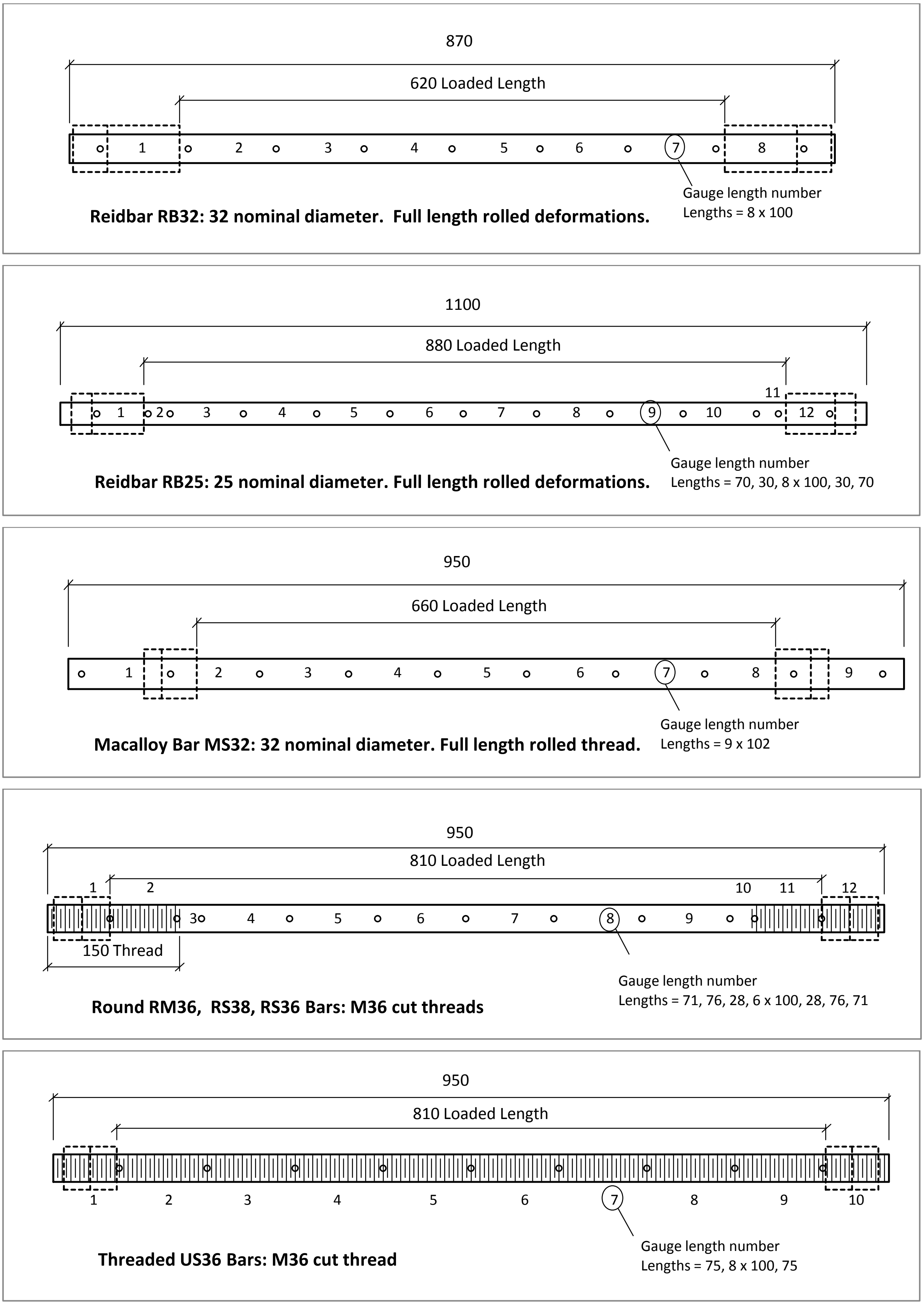



Figure 1.  Dimensions of RB32, RB25, MS32, RM36, RS38, RS36 and US36 linkage assemblies and location of gauge lengths.  All bars symmetrical.

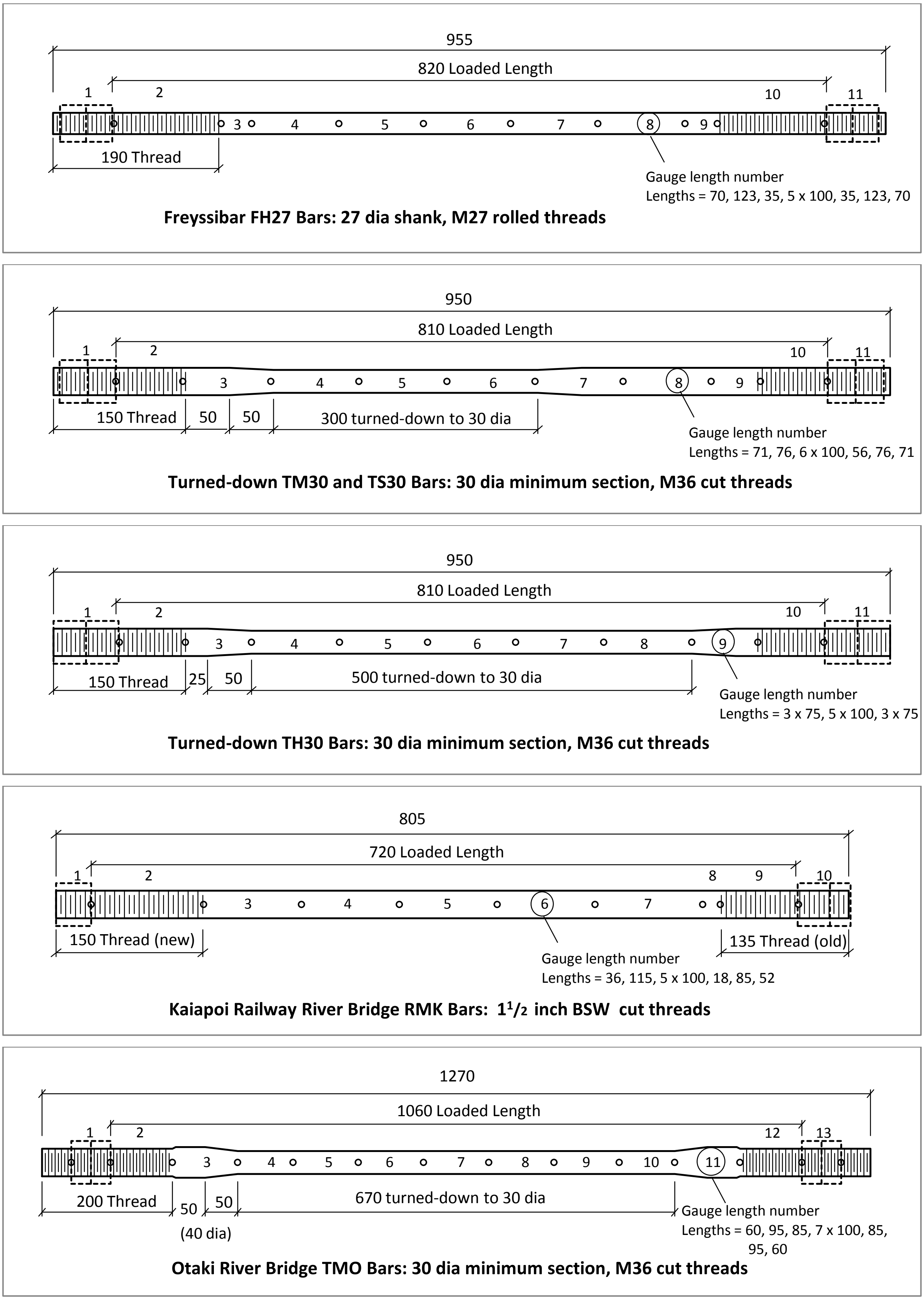



Figure 2. Dimensions of FH27, TM30, TS30, TH30, RMK and TMO, linkage assemblies and location of gauge lengths. All bars symmetrical except for TM30 and TS30 bars.

#### 3.4.1 Nuts

### *Nuts for Proprietary Bars RB32, RB25, MS32, and FH27*

Proprietary nuts and locknuts were used on the proprietary bars. The nuts supplied with the FH27 bars were stamped GM 10. The GM is an abbreviation for the manufacturer GTM, France, and the 10 indicates Property Class 10. This Property Class has a nominal proof load stress of 1,000 MPa. The nuts did not have a standard ISO metric thread (3.5 mm instead of 3 mm pitch) so it is not clear what their proof load should be but it is likely to be greater than the value of 487 kN given in AS/NZS 4291.2: 1995 for a Property Class 10 nut with an M27 ISO metric thread.

Locknuts were used on all the proprietary assemblies except in a preliminary test on an RB32 assembly, and the third test of the FH27 assemblies. A nut failed in the preliminary RB32 test but the single nuts on the FH27 bars performed satisfactorily.

### *Nuts for Mild Steel and High Tensile Bars RM36, TM30 and TH30*

Hot-dip galvanised M36 steel nuts of Property Class 8 were specified and used on the mild and high tensile bar assemblies. All nuts supplied for the assemblies were stamped with the AS/NZS 1252:1996 Property Class 8 identification marking. An inspection certificate was provided with the nuts supplied with the bars used in the Stage 2 testing. This indicated that the M36 nuts were hot dipped galvanised and complied with AS/NZS 1252: 1996 (Mechanical) and AS1252: 1983 (Dimensional). The certificate indicated that the nuts had passed a proof load of 951.8 kN and had a final hardness of 29.5 HRC. The required proof load for M36 galvanised nuts given in AS/NZS 1252:1996 is 951.8 kN. (This is the same value as given on the inspection certificate raising some doubts about the proof load certification procedure.) The dimensions of the nuts and hardness complied with AS/NZS 1252:1996. It is understood that the nuts were manufactured in China but the name of the manufacturer was not given on the inspection certificate or on the nuts.

The maximum ultimate load for the RM36 and TM30 assemblies was 395 kN and for the TH30 assemblies 752 kN. The Property Class 8 nuts used in the testing would not be expected to fail at these maximum loads. (Nuts are required to pass the specified proof load without stripping.)

All the RM36, TM30 and TH30 assemblies were tested with two standard nuts at both ends.

### *Nuts for Stainless Steel Bars RS38, RS36, US36 and TS30*

Nuts from three different manufacturers were used on the stainless steel Grade 316 bars and these were all supplied by EDL Fasteners Ltd, Lower Hutt. It is understood that the nuts were imported from China by James Glen Pty Ltd, Auckland. James Glen Pty Ltd provided a Certification Letter that stated that the nuts complied with DIN 934. However, this standard is superseded and has been replaced by DIN ISO 4032: 2000, which only covers the dimensional requirements for nuts. It refers to ISO 3506-2 for the property classes of stainless steel nuts although this standard only covers nuts for diameters less than or equal to 24 mm.

The nuts were stamped as follows: LE, A4-70; OL, A4-70 and WL, A4-80. The first two letters refer to the manufacturer and the last four to the steel grade. The A in the steel grade indicates austenitic stainless steel and the 4 indicates cold formed containing chromium, nickel and molybdenum (known as Grade 316). The 70 and 80 in the grade markings is a Property Class and indicates UTS's of 700 and 800 MPa respectively for the nut material. These grades have corresponding yield stresses of 450 and 600 MPa. A single test certificate was supplied indicating that the Property Class 70 nuts had a UTS of 711 MPa.

The A4-70 nuts were used on all the stainless steel assemblies except on one of the US36 assemblies where A4-80 nuts were used. The proof loads for M36 nuts to Property Classes 70 and 80 are 572 kN and 654 kN respectively (UTS x stress area). The maximum ultimate loads for the stainless steel assemblies were 525 kN, 542 kN, 599 kN and 672 kN for the TS30, RS36, RS38 and US36 assemblies respectively. Since the maximum ultimate loads of the RS38 and US36 assemblies exceeded the proof load for a single M36 nut of material Property Class 70, using two standard nuts at both ends of these assemblies was clearly necessary to avoid nut failures. (The proof loads for a single A4-80 nut would marginally exceed the maximum load recorded in the tests of the RS38 assemblies.)

Except for the first test of the RS38 assemblies all the RS38, RS36 and US36 assemblies were tested with two standard nuts at both ends. An A4-70 nut failed in the first test of the RS38 assembly at a load of 569 kN which is just below the proof load of 572 kN for the nut. However, the assembly test set-up did not follow the standard nut testing procedure, which uses a hardened and threaded test mandrel (for example, see AS/NZS 4291.2:1995) so the nut failure at less than the proof load did not necessarily indicate that it did not meet its specified strength.

***Nuts for Mild Steel Bars Recovered From Bridges RMK and TMO***

Nuts were recovered with the linkage bars from the two bridges and used in the test assemblies in a similar configuration to the bridge installations. On the Kaiapoi Railway River Bridge assemblies (RMK) a standard galvanised 1½ inch BSW nut (38.1 mm) with a half-length locknut was used on one end and a single standard nut on the other end (simulating the head end). On the Otaki River Bridge assemblies (TMO) two standard M36 galvanised nuts were used on both ends. The nut Property Class was not shown on the drawings.

#### 3.4.2 Washers

***Washers for Proprietary Bar Assemblies RB32, RB25, MS32 and FH27***

Grade 316 stainless steel washers supplied by the manufacturer were used on the Macalloy MS32 assemblies.

Mild steel washers of the dimensions listed in Table 2 were used for the RB32, RB25 and FH27 bars. The FH27 nuts indented and distorted the mild steel washers and this increased the plastic extensions of the assemblies by a small amount. Preventing this increased deformation is not usually necessary in bridge linkage systems but higher strength washers might need to be used in some applications.

***Washers for Mild Steel and High Tensile Bar Assemblies RM36, TM30 and TH30***

All washers used on the RM36, TM30 and TH30 assemblies were mild steel to the dimensions given in Table 2.

***Washers for Stainless Steel Bar Assemblies RS38, RS36, US36 and TS30***

The Grade 316 stainless washers supplied with the MS32 bars had 37 mm diameter holes and were used on all the other stainless steel assemblies which all had M36 threads.

***Washers for Mild Steel Bars Recovered From Bridge, Assemblies RMK and TMO***

The 165 x 165 x 19 mm washers recovered from the Kaiapoi Railway River Bridge (RMK assembly) installations were too large to fit within the testing frame used in the Stage 1 and 2 testing and smaller mild steel washers were used (see Table 2). Washers from the Otaki River Bridge (TMO assemblies, 200 mm diameter x 33 mm thick washers) could have been used in the jack set-up used in the Stage 3 testing but for convenience of testing (related to the deflection measurement system) the smaller mild steel washers recovered from the Kaiapoi Railway River Bridge were used.

*Bearing Washers*

In the Stage 1 and 2 test set-up the washers under the nuts were in contact with either the 50 mm thick loading plates of the test frame or packing washers located within the frames, which were used to increase the loaded length. Except for the RMK assemblies these packing washers were 25 mm thick. 12 mm thick packing washers were used for the larger diameter RMK bars.

In the Stage 3 test set-up washers under the nuts were in contact with the 19 mm thick washers bearing on the centre-hole load cell and jack ram (see Figure 5) except for the TMO assemblies where the nuts were in direct contact with the19 mm thick washers.

In bridge linkage assemblies it is usual to use rubber pads as well as steel washers at one or both ends to accommodate temperature contractions and expansions of the bridge spans. For example, 50 mm thick rubber pads were used on both ends of the Otaki River Bridge span linkage assemblies and 38 mm thick pads were used on one end of the Kaiapoi Railway River Bridge span linkage assemblies. No rubber pads were used in the present testing as the main objective was to determine the strength and deformation performance of the linkage bars.

### 3.5 Fabrication Costs

Fabrication costs for the linkage assemblies are summarized in Table 3. The prices for the Stage 1 bars were obtained in August 2010 and for the Stage 2 bars in May 2011. The RB25 assemblies tested in Stage 3 were purchased in August 2011. The Freyssibar assemblies are not included in the table because these bars were supplied free of charge. They are fabricated in France so prices in New Zealand would include a significant freight component.

With the exception of the proprietary bars (RB32, RB25, MS32 and FH27) all the linkage assemblies were fabricated and supplied by J & D McLennan Ltd in Lower Hutt. There appeared to be a significant increase in the fabrication unit rates between the assemblies supplied for Stages 1 and 2. For example, the TM30 and TH30 bars involved a similar amount of fabrication work and used the same nuts but cost $240 and $490 respectively per unit. The 4140 high tensile steel used in the TH30 assemblies cost about $70 per metre compared to about $20 per metre for mild steel bar. The 4140 steel is probably more difficult to machine than mild steel but the price difference appeared to reflect more than additional machining time.

The M36 galvanised nuts specified as Property Class 8 cost $6 and the M36 Grade 316 stainless nuts $18 (both Stages 1 and 2).

**Table 3. Fabrication Costs for Test Assemblies**

| Linkage Assembly | Linkage Identifier | Test Stage | Nominal Shank Dia. mm | Bar Supplied Length[1] mm | Supply and Fabrication Cost per Assembly Including Nuts and Washers (Exc GST) |
|---|---|---|---|---|---|
| Reidbar - Grade 500E galvanised | RB32 | 1 | 32 | 1200 | $100 |
| Reidbar - Grade 500E galvanised | RB25 | 3 | 25 | 1100 | $70 |
| Macalloy S650 - Grade 316 stainless | MS32 | 1 | 32 | 1200 | $780[2] |
| Round - galvanised mild steel | RM36 | 1 | 36 | 1200 | $165 |
| Round - Grade 316 stainless steel | RS38 | 1 | 38 | 1200 | $375 |
| Round - Grade 316 stainless steel | RS36 | 2 | 36 | 950 | $525 |
| Fully threaded - Grade 316 stainless steel | US36 | 2 | 36 | 950 | $405 |
| Turned-down - galvanised mild steel | TM30 | 1 | 36 | 1200 | $240 |
| Turned-down - Grade 316 stainless steel | TS30 | 1 | 38 | 1200 | $450 |
| Turned-down - galv. high tensile steel | TH30 | 2 | 36 | 950 | $490 |

Note: 1. Bars tested in Stage 1 were shortened prior to testing to fit the UTM.
2. Macalloy S650 cost includes $260 airfreight per assembly.

# 4. Testing Method

Stages 1 and 2 of the tensile testing of the linkage assemblies were carried out using the Opus Central Laboratories 2 MN Servotest Universal Testing Machine (UTM). This machine has a certified 1% load accuracy. Extension of the assemblies was measured using the electrical output from the machine. Calibration of the extension measurement was made at the time of testing to 1% accuracy using gauge rods and a steel ruler. Both the load and extension electrical outputs were recorded on a data logger. Following each test the logger data records were converted to Excel files for plotting.

Special purpose loading frames were fabricated from mild steel to hold the bar assemblies in the UTM. These allowed the load to be applied to the faces of the washers and nuts to simulate the application of load in a typical bridge situation where the bars are installed between concrete diaphragms or steel brackets. Tongues on the loading frames were gripped in the hydraulic grips of the UTM. The frames were constructed of thick plate and their extension during the testing was small in comparison to the assemblies. The frame extensions were both calculated and measured and corrections made to the total extensions measured on the UTM output to get the extensions of the test assemblies. Dimensions of the loading frames are shown in Figure 3 and the test set-up of an assembly in the UTM is shown in Figure 4.

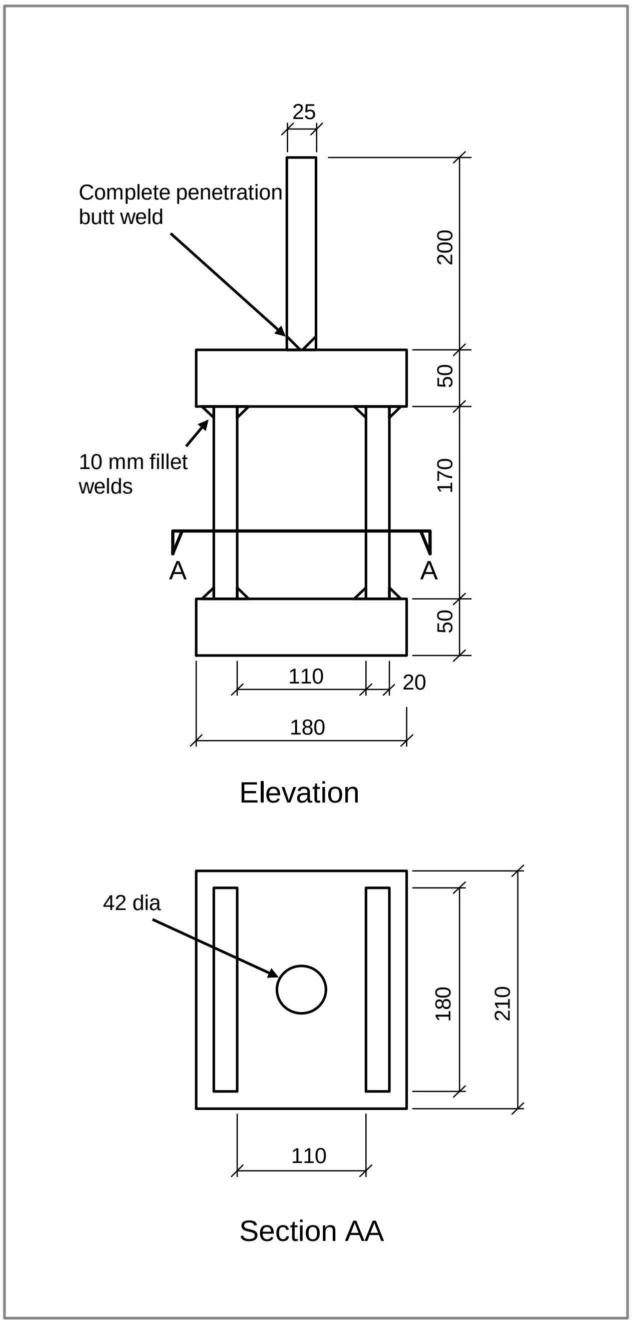
Because of the failure of the UTM electronic control system, Stage 3 of the testing (three assembly types) was carried out using a 1 MN capacity centre-hole hydraulic jack mounted on a support frame anchored to the concrete strong-floor at Opus Central Laboratories. Load was measured with a special purpose electric-resistance load cell and the extension deformations of the bar with a Kyowa DT-100 electric-resistance displacement transducer. The load cell was calibrated on a compression testing machine with a load calibration of 1% and the displacement transducer was calibrated to 1% using gauge rods. Both the load and extension electrical outputs were recorded on a data logger. The maximum travel of the jack was 75 mm which meant that three loading steps were required to load the RB25 and TMO bars to failure. When the jack travel limit was reached the bars were unloaded then reloading after steel plate packers had been placed between the jack ram and the 165 mm square x 19 mm thick washer applying load to the nuts at the jack end. Details of the jack loading arrangement are shown in Figure 5 and the test set-up for an RB25 assembly in Figure 6.

Pseudo-static loading was applied with a pre-yield loading rate of about 20 kN per second for the bars tested in Stage 1 and 2 (UTM) and between 22 to 38 kN per second for the bars tested in Stage 3 (hydraulic jack).

Prior to testing, gauge lengths of 100 mm nominal length were marked out along the bars using a small indentation formed with a centre-punch. Shorter gauge lengths were used on the threaded sections at the ends of the bars (see Figures 1 and 2). Prior to and after each test, the gauge lengths were measured to the nearest 0.1 mm using vernier calipers. This gave a reliable measurement of the total plastic elongations over each of the gauge lengths. As shown in Figures 1 and 2, the number of gauge lengths varied between eight and 13. The number was varied to suit the different lengths of bars and to capture separate extensions on sections under the nuts, and on the turned-down and threaded sections.

Three tests were carried out on each assembly type except for the RB32 and RMK assemblies. In addition to the three main tests a preliminary test was carried out on the RB32 assemblies. In this initial test, lock nuts were not used and the standard nut failed at a load between the yield level and the maximum ultimate load reached in the other tests. Only two RMK assemblies were recovered from bridge and both were tested.

The results of each test are reported in Section 5. Load versus extension plots show the overall bar extensions measured between the faces of the loaded nuts. Separate plots show the percentage elongation of each gauge length on completion of each test.



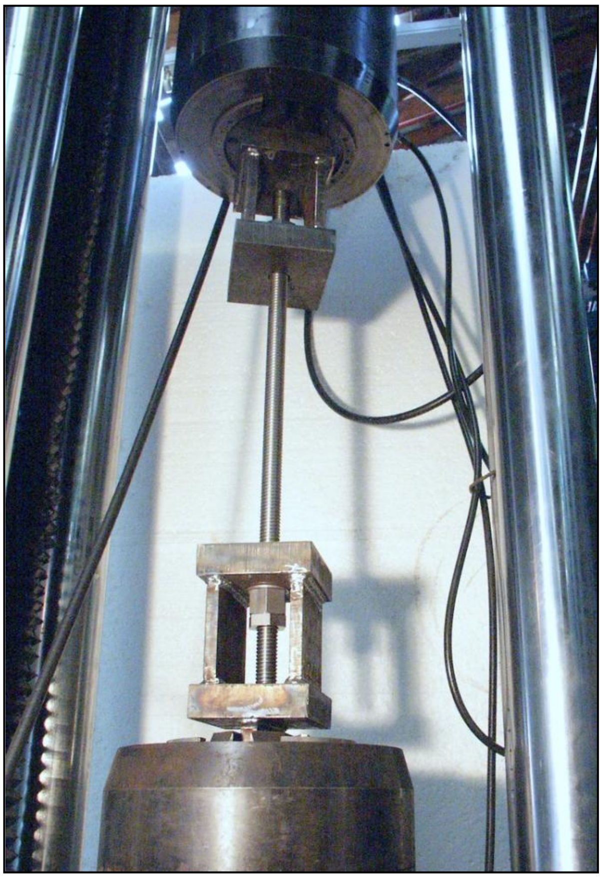



Figure 3.  Loading frame used in Stage 1 and 2.

Figure 4. MS32 assembly  in Servotest UTM.

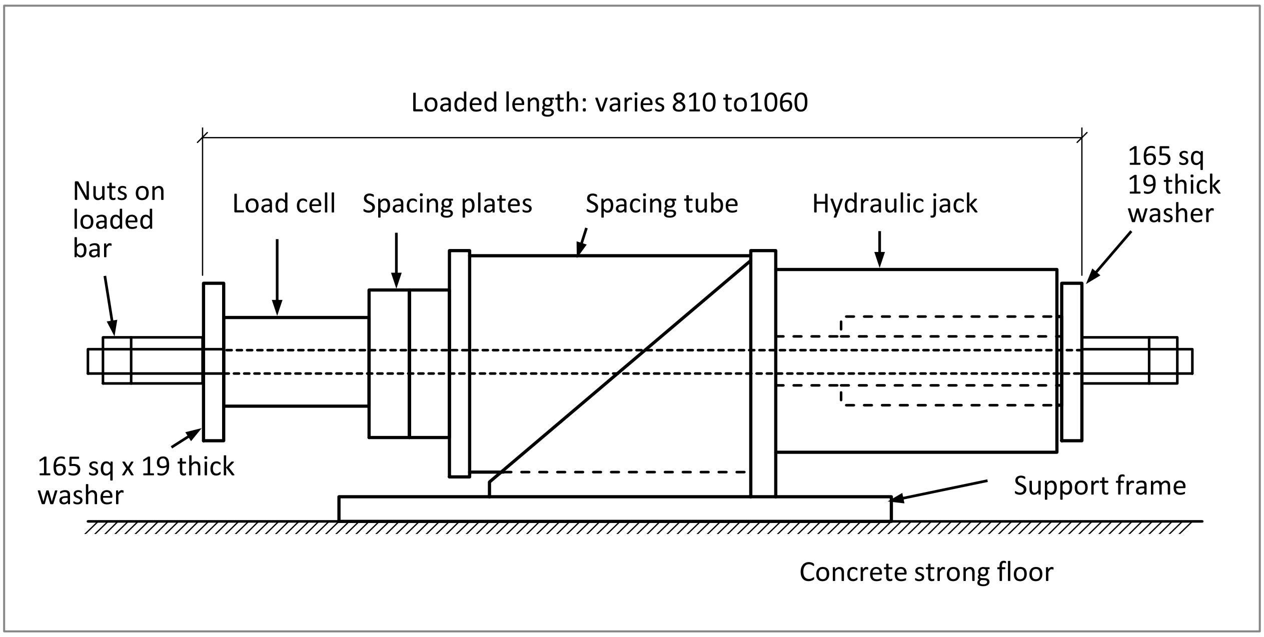



Figure 5. Elevation of hydraulic jack loading system and support frame used in Stage 3. (Additional smaller washers were used under the nuts for all tests except on the TMO assemblies – see Table 2)

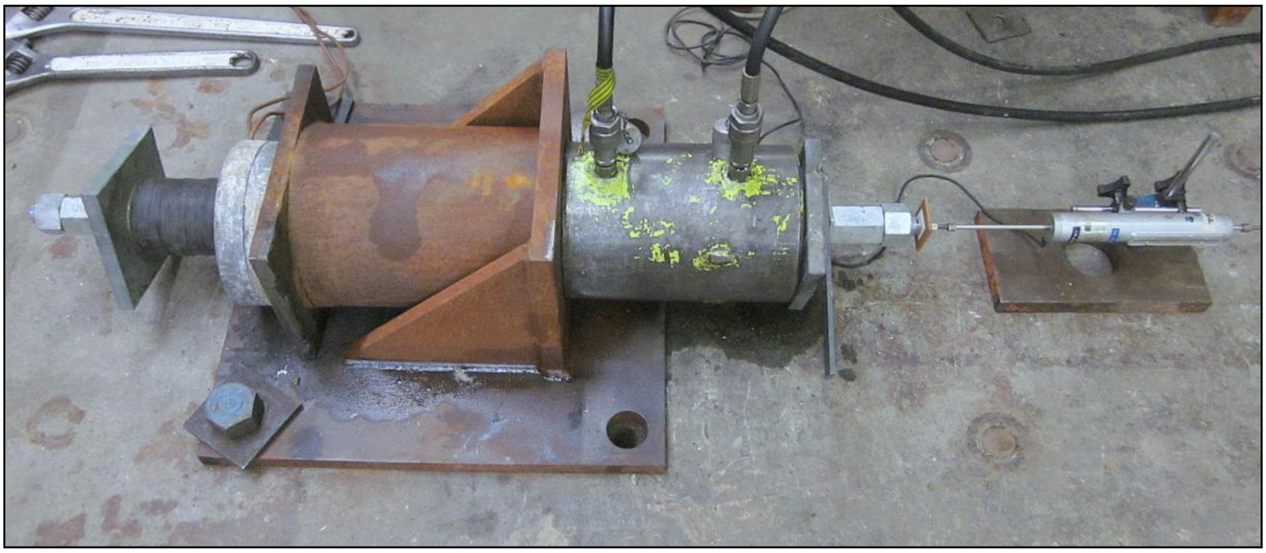


Figure 6. Hydraulic jack loading system set-up for Reidbar RB25 tests. Deflection measurement gauge is at the right end of the bar. Gauge was repositioned to be alongside the jack prior to bar failure.

# 5. Test Results

## 5.1 Reidbar RB32 Assemblies

Load versus extension plots are shown in Figure 7 for the four tests. With the exception of Test 1 (initial test), the performance curves were reasonably consistent with failure extensions (elastic plus plastic) of between 113 mm to 121 mm on the 620 mm loaded length. The minimum total extension (elastic plus plastic) expressed as a percentage of the loaded length was 18%. In Test 1, which was carried out without locknuts, a standard nut on one end failed at a total extension of 46 mm.

With the exception of Test 1, all the ultimate loads were at least 15% higher than the manufacturer's specified minimum ultimate load of 462 kN. The maximum ultimate load of 565 kN was close to the specified maximum ultimate load of 563 kN given in the manufacturer's *Product Catalogue and Design Guide* (Reid, 2008).

Figure 8 shows the failures in the bars and nut (Test 1). Although the failures in Tests 2 to 4 were in the lower section of the bar (as orientated in the testing machine) they were all well removed from the nuts.

Plastic elongations measured on the eight nominally 100 mm gauge lengths are plotted in Figure 9 for the final three tests. As shown in Figure 1, Gauge Lengths 1 and 8 were within the sections covered by the nuts. Significant elongation occurs over the total length between the nuts with typical values of 14% on five of the six gauge lengths outside the nut lengths. In the gauge lengths that necked-down and failed the elongation reached between 26% and 29%. Yielding following by strain hardening (see Figure 7) in the failure section results in significant plastic elongation spreading over most of the loaded length.

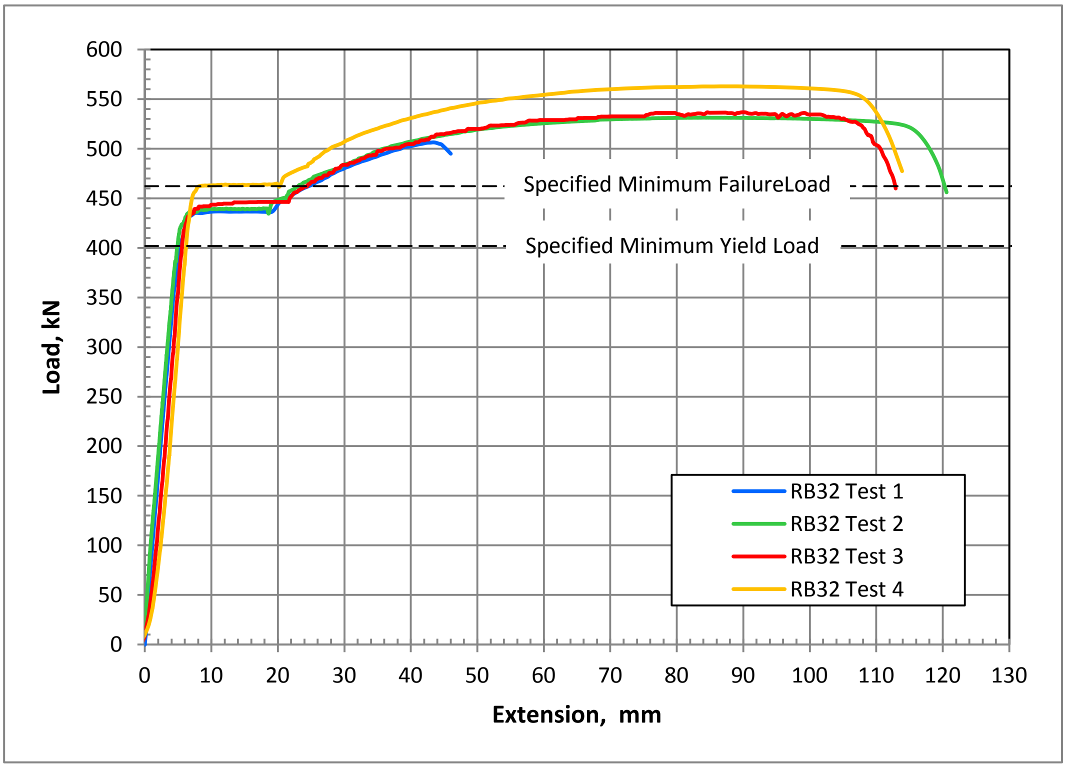



Figure 7. Load versus extension plots for Reidbar RB32 assemblies.

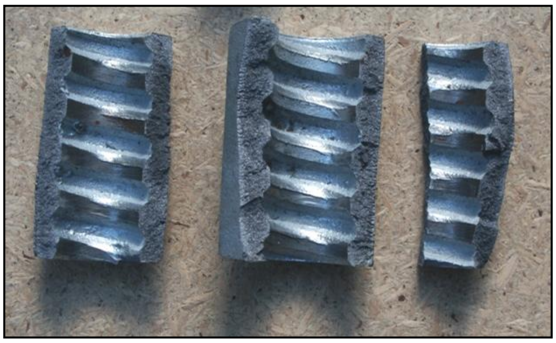


Figure 8. Failure locations in Reidbar RB32 bars (left) and in the nut (Test 1) (above).

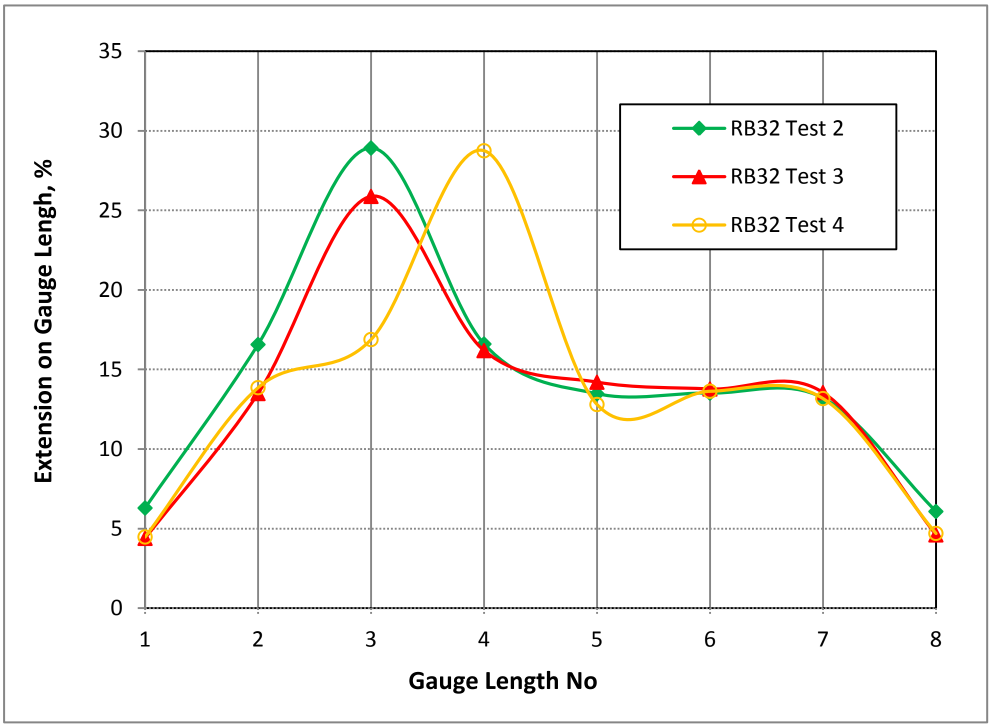



Figure 9. Elongations on Reidbar RB32 gauge lengths.

### 5.2 Reidbar RB25 Assemblies

Load versus extension plots are shown in Figure 10 for the three tests. The performance curves were very consistent with failure extensions (elastic plus plastic) of between 129 mm to 137 mm on the 880 mm loaded length. The minimum total extension (elastic plus plastic) expressed as a percentage of the loaded length was 15%. All the ultimate loads were at least 26% higher than the manufacturer's specified minimum ultimate load of 282 kN. The maximum ultimate load of 360 kN was about 5% greater than the manufacturer's specified maximum ultimate load of 344 kN *(Product Catalogue and Design Guide*, Reid, 2008). This level of over-strength may need to be considered in the design of bar anchor brackets and the supporting structure.

Figure 11 shows the failures in the bars. Although the failures were not centrally located they were all well removed from the nuts with the distance from the failure to the face of the nuts varying from 125 mm to 320 mm.

Plastic elongations measured on the 12 gauge lengths are plotted in Figure 12 for the three tests. As shown in Figure 1, Gauge Lengths 1 and 12 were within the sections covered by the nuts. Significant elongation occurs over the total length with values exceeding 10% on all gauge lengths outside the nut lengths. In the gauge lengths that necked-down and failed the elongation reached between 25% and 28%. Yielding following by stain hardening (see Figure 10) in the failure section results in significant plastic elongation spreading over most of the loaded length.

The total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded lengths varied from 14% to 16%. For the RB32 bars this plastic extension was a little greater at 17% to 18%. However, the loaded length for the RB32 bars was 620 mm which was significantly shorter than the 880 mm length used for the RB25 bars.

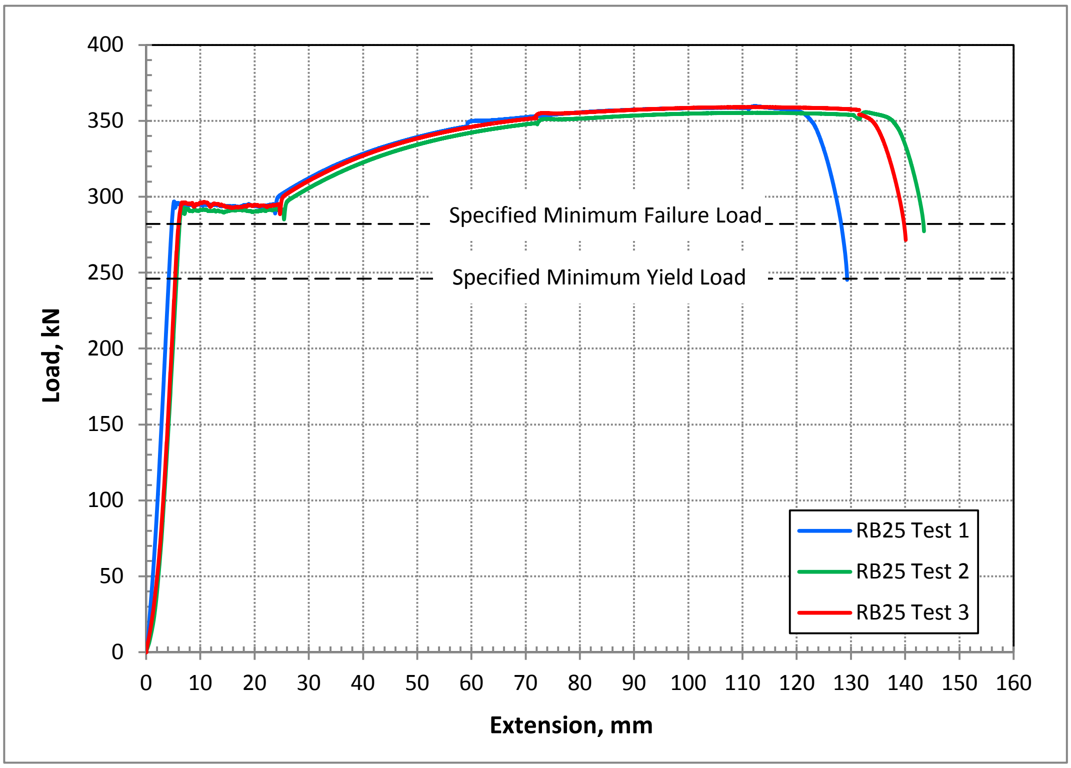



Figure 10. Load versus extension plots for Reidbar RB25 assemblies. Minor unevenness in curves at extensions greater than 58 mm resulted from unloading and reloading to extend the extension range of the jack extension

Figure 11. Failure locations in Reidbar RB25 bars.

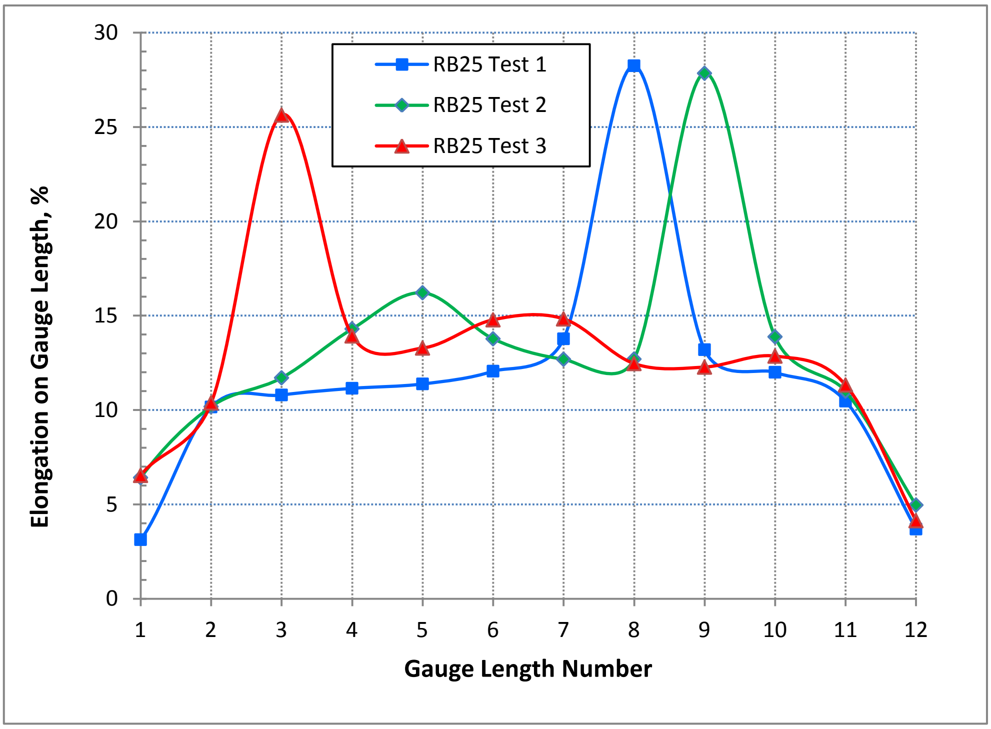



Figure 12. Elongations on Reidbar RB25 gauge lengths.

### 5.3 Macalloy MS32 Assemblies

Load versus extension plots are shown in Figure 13 for the three tests. These performance curves are very consistent with failure extensions (elastic plus plastic) of between 47 to 56 mm on the 660 mm loaded length. The minimum total extension (elastic plus plastic) expressed as a percentage of the loaded length was 7%. All the ultimate loads were about 37% higher than the manufacturer's specified minimum ultimate load of 622 kN. This level of over-strength may need to be considered in the design of bar anchor brackets and supporting structure.

Figure 14 shows the failures in the bars. Although the position of the failures varied over the length they were all some distance from the nuts. The closest distance of about 50 mm occurred in Test 1 which had the lowest overall extension.

Plastic elongations measured on the eight gauge lengths are plotted in Figure 15 for the three tests. As shown in Figure 1, Gauge Lengths 2 and 8 were partially within the sections covered by the nuts. Although significant elongation occurred over the total length between the nuts with typical values of 4% on four of the seven gauge lengths, about 65% of the total elongation occurred on the gauge length that necked-down and failed and the two adjacent gauge lengths. The plastic elongation was more concentrated in the failure section than was the case for the Reidbars. The medium tensile MS32 (S650) bar does not have a distinct yield plateau followed by significant strain hardening that is typical of the steel used in the Reidbars (see Figure 10). When present these features spread plastic strains over a large part of the loaded length.

The total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded lengths varied from 6% to 7%. This was a lot less than the 14% to 18% measured on the Reidbars.

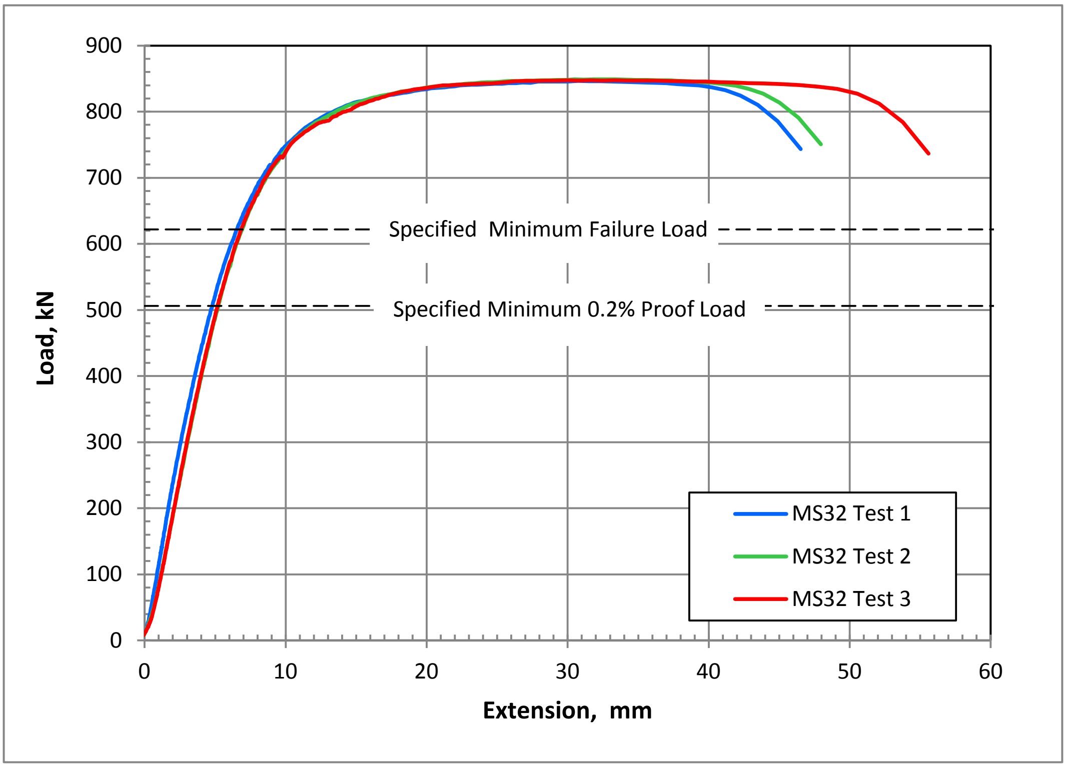



Figure 13. Load versus extension plots for Macalloy MS32 assemblies.

Figure 14. Failure locations on Macalloy MS32 bars.

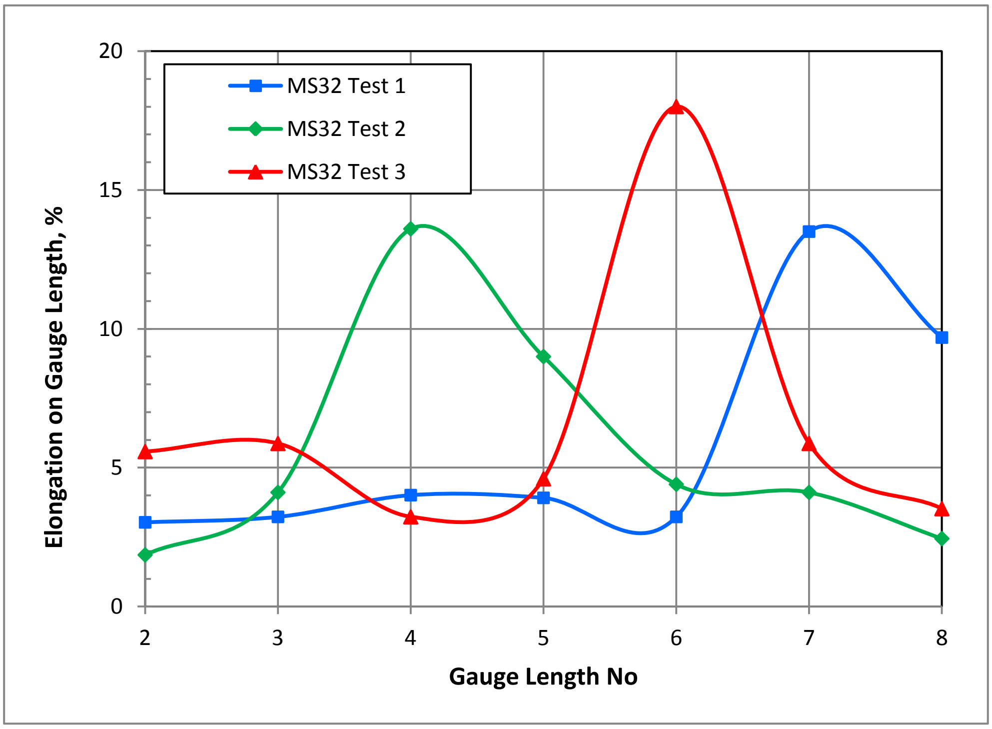



Figure 15. Elongations on Macalloy MS32 gauge lengths.

### 5.4 Round Galvanised Mild Steel RM36 Assemblies

Load versus extension plots are shown in Figure 16 for the three tests. These performance curves are reasonably consistent with failure extensions (elastic plus plastic) of between 65 mm to 70 mm on the 810 mm loaded length. The minimum total extension (elastic plus plastic) expressed as a percentage of the loaded length was 8%. The minimum ultimate load was 10% lower than indicated by the material UTS on the test certificate supplied for the bar. This same discrepancy was noted for the TM30 bars and so it was not related to the bars being threaded. Two yield points are evident on the curves. Yield initially occurred in the threaded lengths which then strain hardened leading to yielding and strain hardening spreading into the unthreaded shanks before failure occurred in a threaded end section.

Figure 17 shows the failures in the bars. They were all in the threaded sections. There was 80 mm of loaded length of thread at both ends of the bars and the failures occurred at about 30 mm from the face of the nuts, that is, about 50 mm from the end of the thread on the shank.

Plastic elongations measured on the 12 gauge lengths are plotted in Figure 18 for the three tests. As shown in Figure 1, Gauge Lengths 1 and 12 were fully within the sections covered by the nuts and locknuts. Although significant elongation occurs over the total length between the nuts, with typical values of 4% on the eight gauge lengths on the unthreaded shank, about 54% of the total elongation occurred in the two threaded lengths that were not covered by the nuts. Elongations in the threaded sections outside the nut zones at both ends differed by a factor of about two. Some of this difference was thought to be related to a fabrication error. Initially the bars did not fit into the testing machine and had to be shortened with a new thread cut on one end. This second thread was cut to a slightly larger root diameter than the original thread, which included a cutting depth allowance for the galvanising, and was therefore slightly stronger than would be the case for a galvanised thread.

Elongations of up to 33% occurred in the threaded failure sections. This elongation was similar to the percentage elongation on the unthreaded gauge lengths in the turned-down sections that failed in the TM30 assemblies (see Figure 33) indicating that the thread geometry did not reduce the elongation by a significant amount. The minimum total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded lengths was 7.9%.

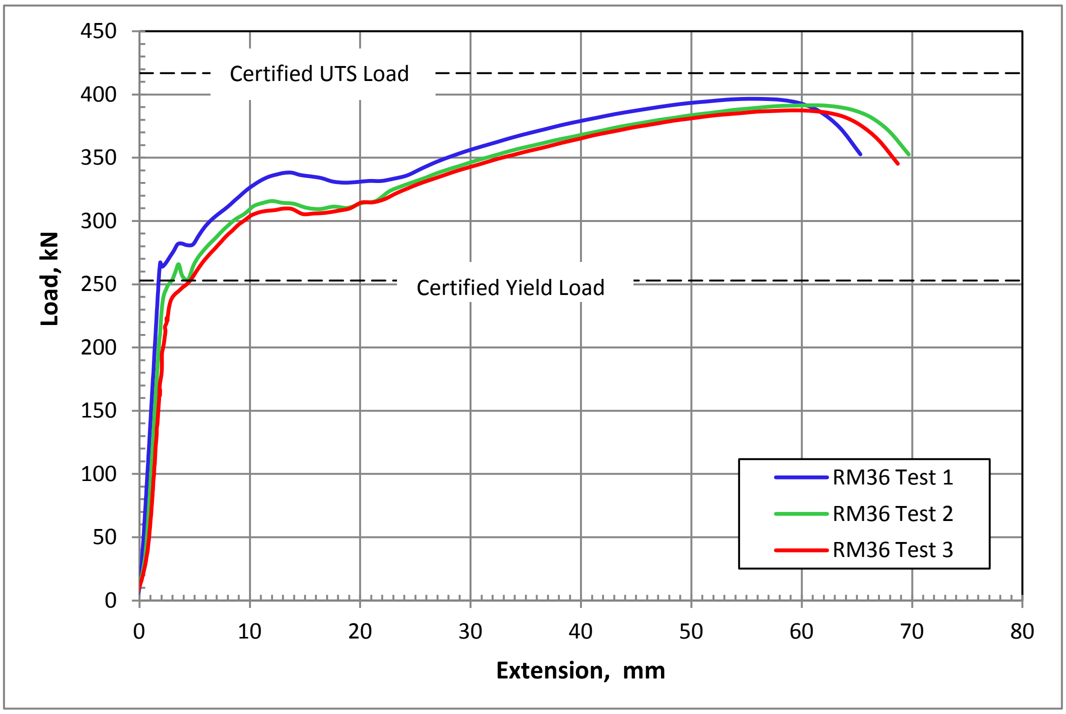



Figure 16. Load versus extension plots for Round Galvanised Mild Steel RM36 assemblies.

Figure 17. Failure locations on RM36 bars.
During testing two nuts were used at either end.

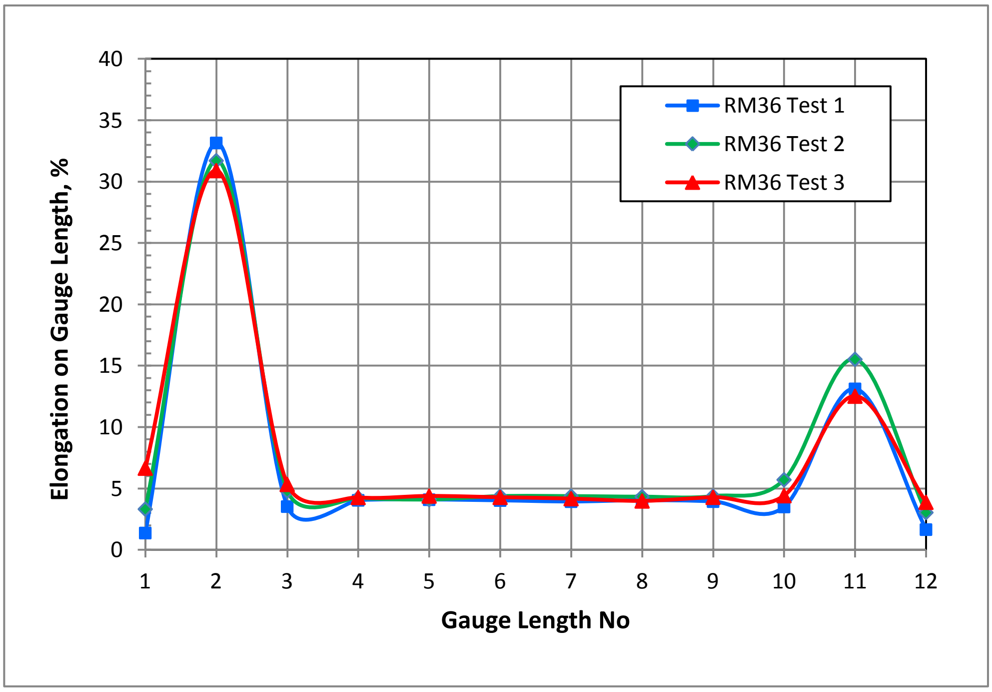



Figure 18. Elongations on Round Galvanised Mild Steel RM36 bar gauge lengths.

### 5.5 Round Grade 316 Stainless Steel RS38 Bars

Load versus extension plots are shown in Figure 19 for the three tests. The total failure extensions (elastic plus plastic) in Tests 2 and 3 were between 35 mm and 38 mm on the 810 mm loaded length. The low extension of 15 mm in Test 1 was related to threads stripping in the nut at one end. In this test the bar was not fitted with locknuts. The minimum total extension in Tests 2 and 3 expressed as a percentage of the loaded length was 4%. All the ultimate loads were about 10% higher than indicated by the material UTS given on the test certificate supplied with the bar.

Figure 20 shows the failures in the bars. They were all in the threaded sections. There was 80 mm of loaded length of thread at either end of the bars and the failures in Tests 2 and 3 occurred at about 30 mm from the face of the nuts, that is, about 50 mm from the end of the thread on the shank.

Plastic elongations measured on the 12 gauge lengths are plotted in Figure 21 for Tests 2 and 3. As shown in Figure 1, Gauge Lengths 1 and 12 were fully within the sections covered by the nuts and locknuts. Nearly all of the total elongation occurred in the two threaded lengths that were not covered by the nuts. There was no measurable elongation in the unthreaded shank. This was related to the shank being 38.1 mm in diameter compared to the 36 mm nominal diameter of the threaded sections. Following yield in the threaded sections there was insufficient strain hardening to force yield into the larger diameter unthreaded length. In contrast, the RM36 bars had the same nominal thread and shank diameters which resulted in strain hardening in the threaded sections forcing yield and strain hardening into the unthreaded sections before failure in the threads.

The elongations in the threaded section at either end outside the nut zones differed by a factor of about two. A similar behavior was observed for the RM36 bars, where it was thought that the difference was related to a fabrication error which led to a slightly larger root diameter at one end. However, for the RSS bars the threads at either end should have been identical so the same explanation does not apply. The testing of all assembly types showed that greater elongation occurs in the section that yields first.

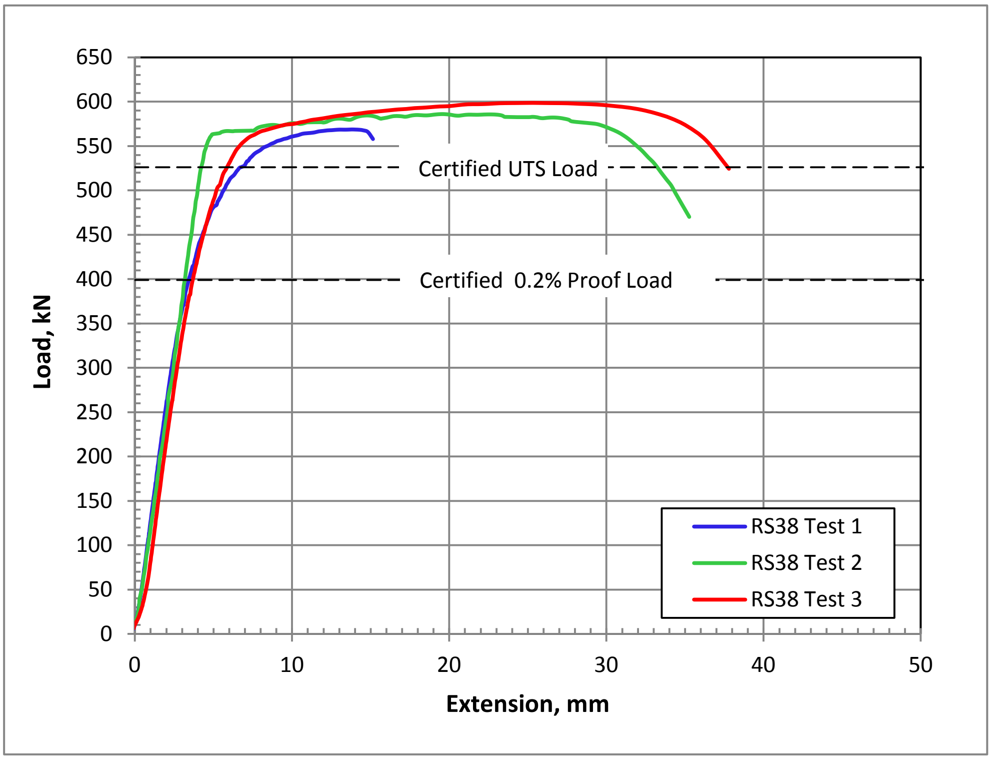



Figure 19. Load versus extension plots for Round Stainless Steel RS38 bar assemblies.

Elongations of up to 29% occurred in the threaded failure sections. This elongation was a little less than the 35% to 39% elongations in the turned-down sections that failed in the TS30 assemblies (see Figure 36) indicating that the thread geometry did not reduce the elongation by a large amount.

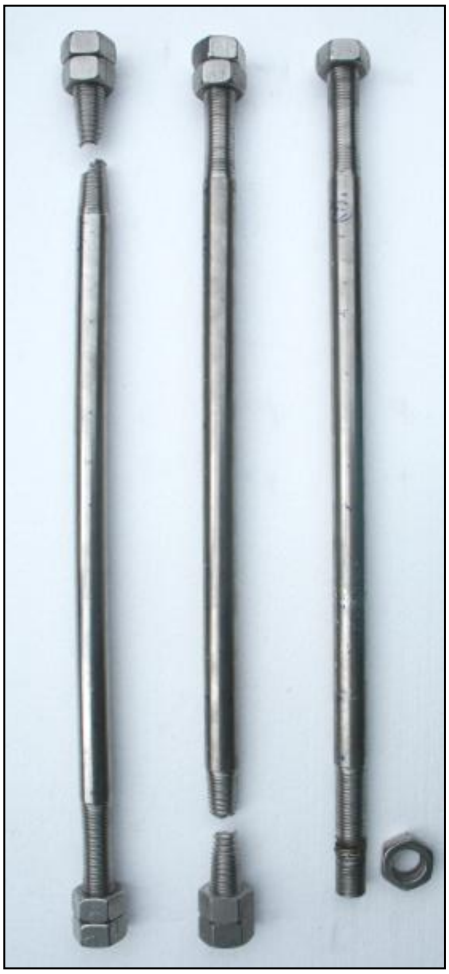


Figure 20. Failure locations on RS38 bars.
The nut failed on the bar on the right (Test 1) by thread stripping. This bar was cut near the failed nut to remove it from the testing machine.

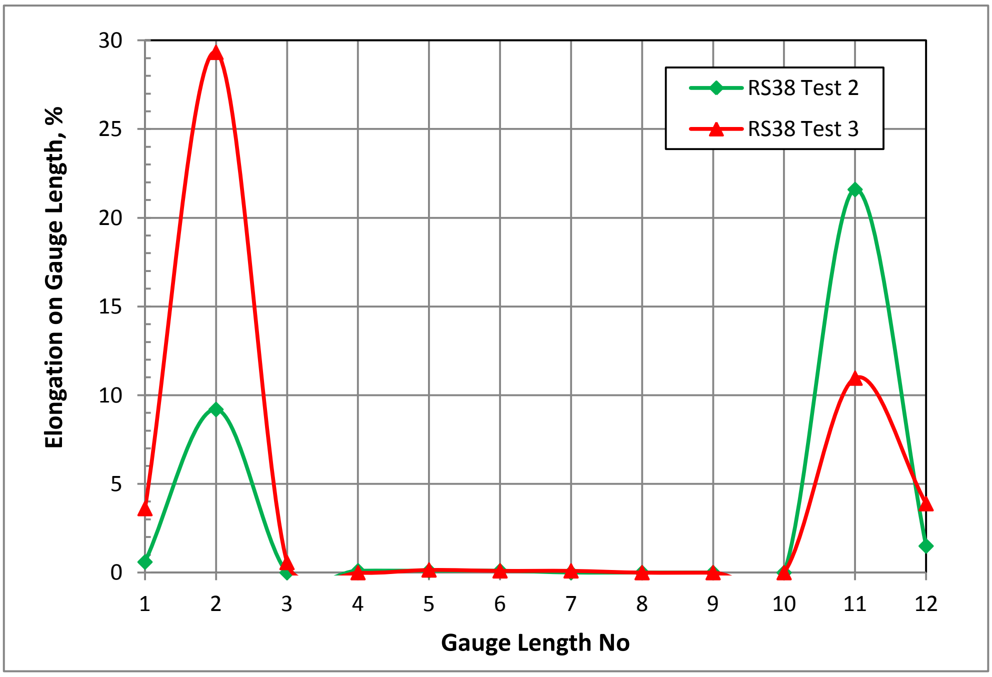



Figure 21. Elongations on Round Stainless Steel RS38 bar gauge lengths.

### 5.6 Round Grade 316 Stainless Steel RS36 Assemblies

Load versus extension plots are shown in Figure 22 for the three tests. These performance curves are very consistent with failure extensions (elastic plus plastic) of between 85 mm to 86 mm on the 810 mm loaded length. The Test 2 and 3 curves were almost identical. The minimum total extension (elastic plus plastic) as a percentage of the loaded length was 10%. All the ultimate loads were about 8% higher than indicated by the material UTS given on the test certificate supplied with the bar.

Figure 23 shows the failures in the bars. They were all in the threaded sections. There was 80 mm of loaded length of thread at either end of the bars and the failures occurred at about 55 mm from the face of the nuts, that is, about 35 mm from the end of the thread on the shank.

Plastic elongations measured on the 12 gauge lengths are plotted in Figure 24. As shown in Figure 1, Gauge Lengths 1 and 12 were fully within the sections covered by the nuts and locknuts. There was about 2.5% elongation in the gauge lengths on the unthreaded shanks. This was significantly greater than the almost negligible extension in the RS38 bar shanks but less than the 4.2% extensions in the gauge lengths on the shanks of the RM36 bars which had slightly smaller total extensions than the RS36 bars (about 65 mm for the RM36 bars compared to about 85 mm for the RS36 bars). The elongations in the threaded ends for the RS36, RS38 and MS36 bars expressed as a percentage of the total elongations were 80%, 99% and 57% respectively.

As was the case for the RS38 bars, there was a significant difference in the elongations in the threaded sections at either end outside the nut zones. This was consistent with the previous observation that the elongation is greater in the threaded section that yields first. (Failure also occurs in this section.) Elongations of up to 44% occurred in the threaded failure sections. The test certificate elongation for the bar used in the RS36 assemblies was 51%. Again the thread geometry did not reduce the elongation in the failure section by a large amount.

The minimum total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded length was 8.5%. This was significantly greater than the minimum plastic elongation of 4.1% recorded for the RS38 bars.

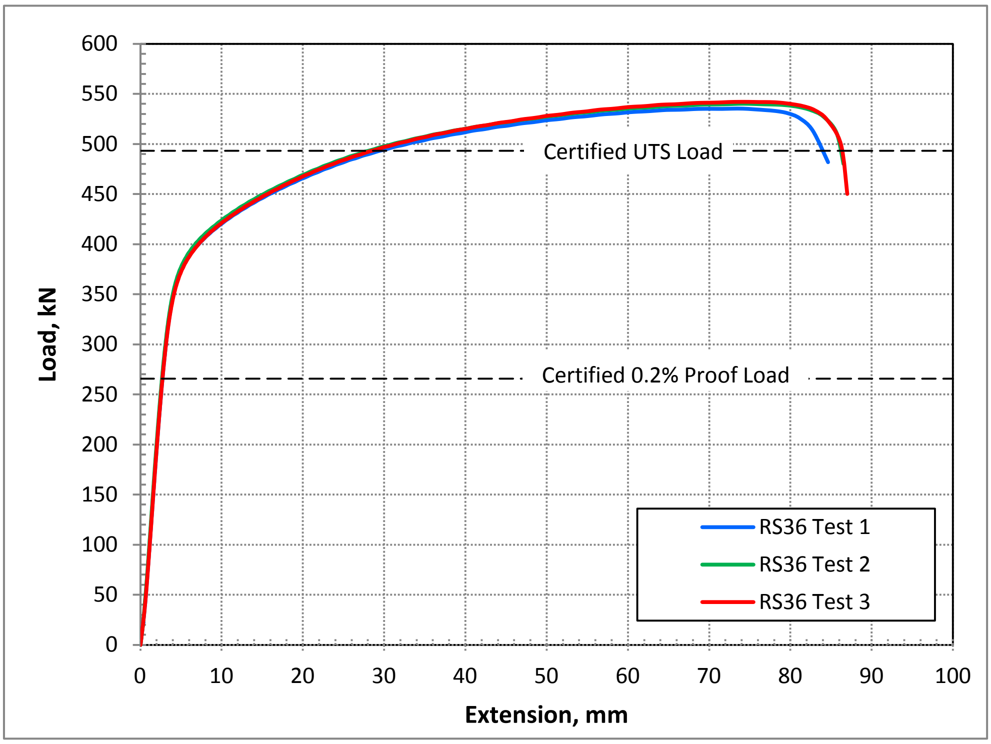



Figure 22. Load versus extension plots for Round Stainless Steel RS36 bar assemblies. Test 2 and 3 curves were almost identical.

Some of the greater extension was clearly attributable to having the shank diameter the same as the nominal diameter of the threads on the RS36 bars. The bar used for the RS38 assemblies had a certified elongation of 47% which was a little lower than the 51% certified for the steel used in the RS36 assemblies and this may have also contributed to the greater extensions of the RS36 assemblies.

Figure 23. Failure locations on RS36 bars. All bars were tested with two nuts at each end.

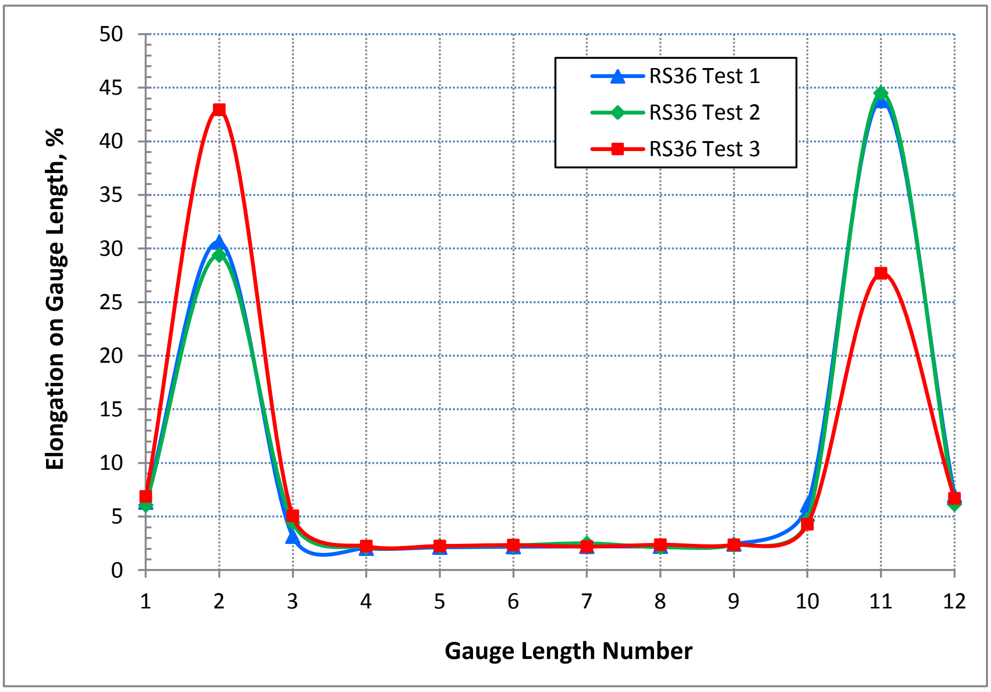



Figure 24. Elongations on Round Stainless Steel RS36 bar gauge lengths.

### 5.7 Fully Threaded Grade 316 Stainless Steel US36 Assemblies

Load versus extension plots are shown in Figure 25 for the three tests. There is a wide variation in the failure extensions (elastic plus plastic) of between 52 mm and 140 mm on the 810 mm loaded length. The minimum total extension (elastic plus plastic) as a percentage of the loaded length was 6.5%. All the ultimate loads were about 11% higher than indicated by the material UTS given on the test certificate supplied with the bar.

Figure 26 shows the failures in the bars. They were all in very different locations with two of the three failures towards one end of the bar. The distances of the failures from the nearest nut face varied from 50 mm to 380 mm. The failure closest to the nut face (Test 3) resulted in a much lower plastic extension than the failure towards the centre of the bar (Test 1).

Plastic elongations measured on the 10 gauge lengths are plotted in Figure 27. As shown in Figure 1, Gauge Lengths 1 and 10 were fully within the sections covered by the nuts and locknuts. Elongations of up to 28% occurred in the failure sections. The test certificate did not provide elongation results so a comparison could not be made with the measured values. In Test 1, yield commenced near the centre of the bar with strain hardening spreading the plastic elongations in both directions to give a large total elongation. In the other two tests initial yield occurred near one end. This limited the length of spread of the plastic elongation resulting in much smaller total elongations. The minimum total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded length was 5.8%.

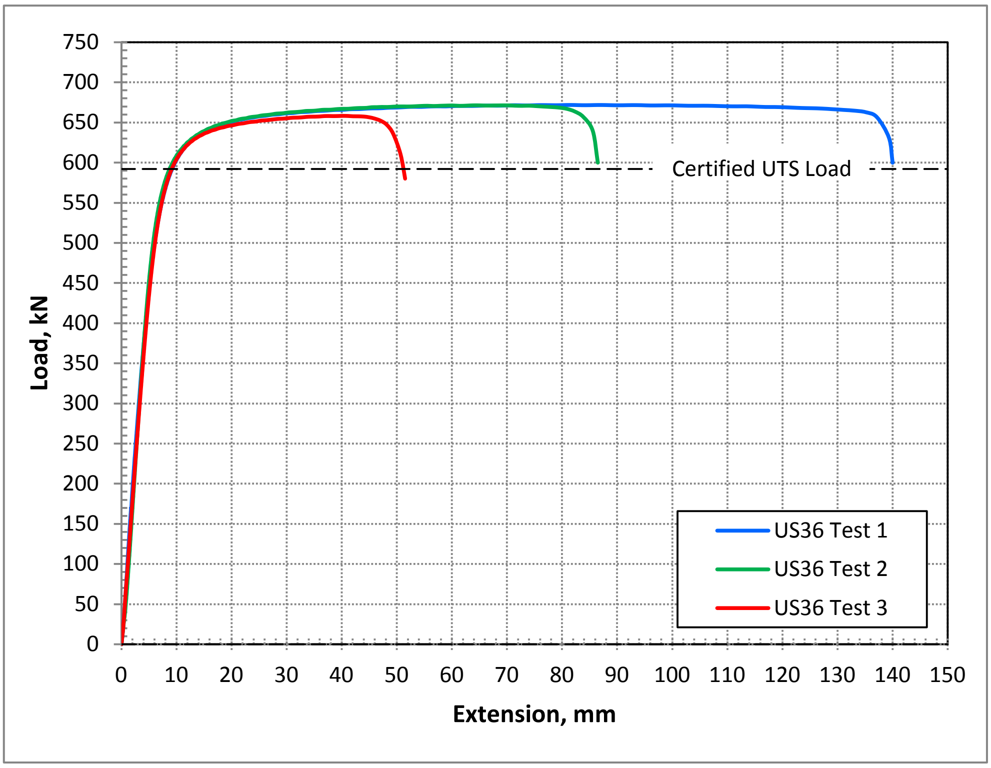



Figure 25. Load versus extension plots for Fully Threaded Stainless Steel US36 bar assemblies.

Figure 26. Failure locations on US36 bars.

The bar on the right (Test 1) was cut in the upper end to remove it from the testing machine.

The bar on the left (Test 3) was bent by a testing machine malfunction after the test was completed.

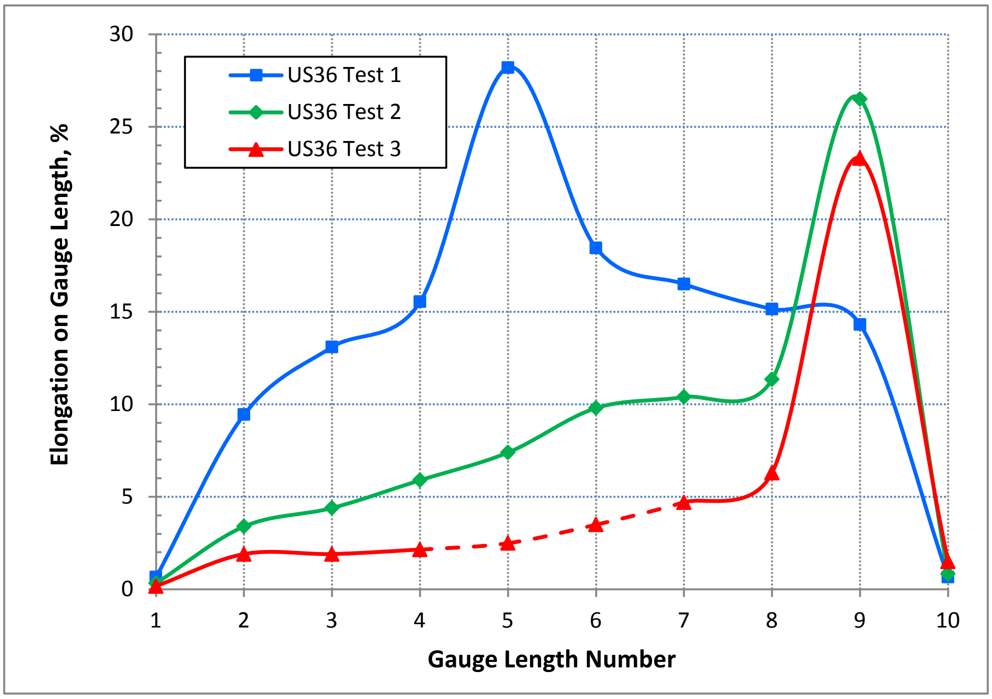



Figure 27. Elongations on Fully Threaded Stainless Steel US36 bar gauge lengths.

### 5.8 Freyssibar High Tensile Steel FH27 Assemblies

Load versus extension plots are shown in Figure 28 for the three tests. These performance curves are very consistent with failure extensions (elastic plus plastic) of just over 36 mm in each test. The minimum total extension (elastic plus plastic) expressed as a percentage of the loaded length was 4.4%. The minimum ultimate load was 6.6% higher than indicated by the material UTS on the test certificate supplied with the bar. The initial curvature in the plots at loads less than 50 kN was mainly caused by bedding-in of the various washers and spacers required in the hydraulic jack loading system used in Stage 3 of the testing.

Figure 29 shows the failures in the bars. They were all in the threaded sections. There was 130 mm of loaded length of thread at both ends of the bars and the failures occurred between 70 mm and 85 mm from the face of the nuts, that is, 60 mm to 45 mm from the end of the thread on the shank.

Plastic elongations measured on the 11 gauge lengths are plotted in Figure 30 for the three tests. As shown in Figure 2, Gauge Lengths 1 and 11 were fully within the sections covered by the nuts and locknuts. About 68% of the total elongation occurred in the two threaded lengths. This was a little less than the 80% of the total elongation that occurred in the threaded ends of the RS36 bars. As was the case for the other bars with threaded ends, the elongations in the threaded sections outside the nut zones at either end differed by a factor of about two with the greatest elongation occurring in the end that failed.

Elongations of up to 10.5% occurred in the threaded failure sections. This elongation was significantly less than the 16.7% elongation given on the test certificate supplied with the bars indicating that the rolled threads may have caused some reduction in elongation. The minimum total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded lengths was 3.3%.

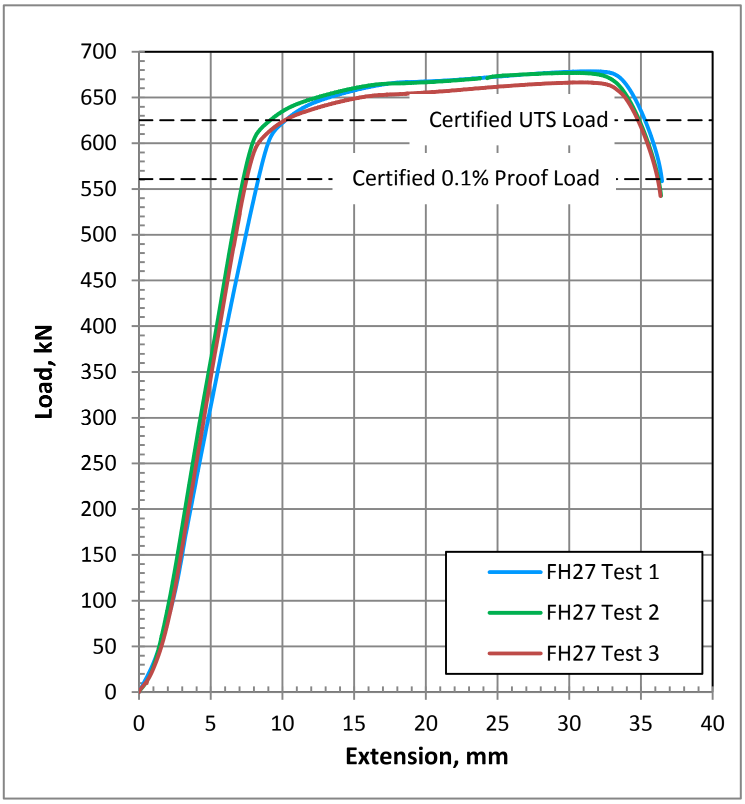



Figure 28. Load versus extension plots for Freyssibar FH27 assemblies.

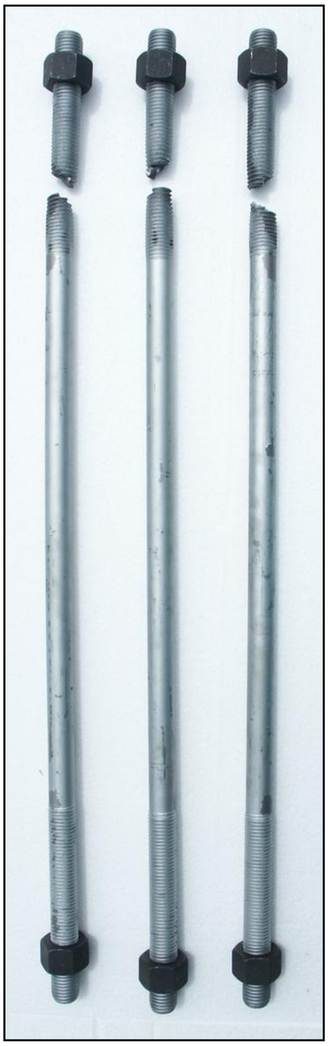


Figure 29. Failure locations on FH27 bars. Two of the bars were tested with two nuts at each end. The third test was with a single nut at each end.

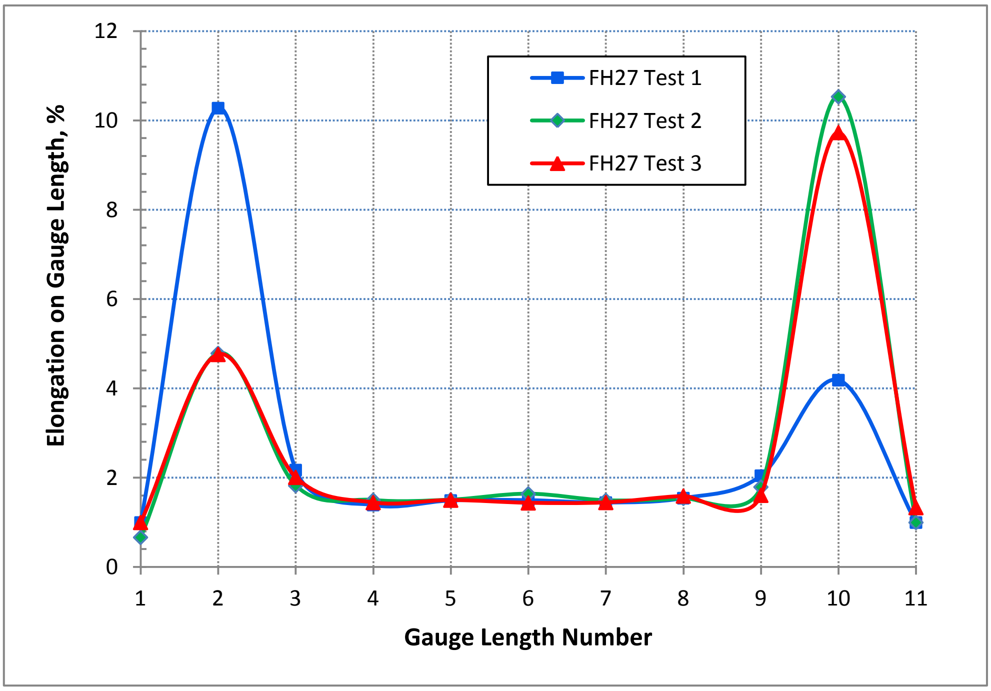



Figure 30. Elongations on Freyssibar FH27 bar gauge lengths.

### 5.9 Turned-Down Galvanised Mild Steel TM30 Assemblies

Load versus extension plots are shown in Figure 31 for the three tests. The plots are very consistent with failure extensions (elastic plus plastic) on the 810 mm loaded lengths of between 88 mm and 93 mm. The minimum overall extension expressed as a percentage of the loaded length was 11%. As was found for the RM36 bars, all the ultimate loads were about 7% lower than indicated by the material UTS given on the test certificate supplied with the bar.

Figure 32 shows the failures in the bars. As expected, the failures were consistently near the centre of the turned-down length.

Plastic elongations measured on the 11 gauge lengths are plotted in Figure 33 for the three tests. As shown in Figure 2, Gauge Lengths 1 and 11 were fully within the sections covered by the nuts and locknuts. A large part of the total elongation (about 75%) occurred in the 300 mm long turned-down length. There were small but significant elongations in the threaded sections. These occurred because the measured ratio of the UTS to yield stress of about 1.6 was significantly greater than the area ratio of 1.2 between the stress area of the thread and the area of the turned-down section. In some applications the turned-down section might be designed to have a smaller diameter although the performance was satisfactory with the 30 mm reduced diameter.

The results showed that plastic elongations of about 30% were available in the 100 mm gauge length that failed in the turned-down section. This is a little less than the 34% elongation given on the test certificate. The total elongation in the 300 mm long turned-down section expressed a percentage of the turned-down length was about 20%. The minimum total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded lengths was 10.5%.

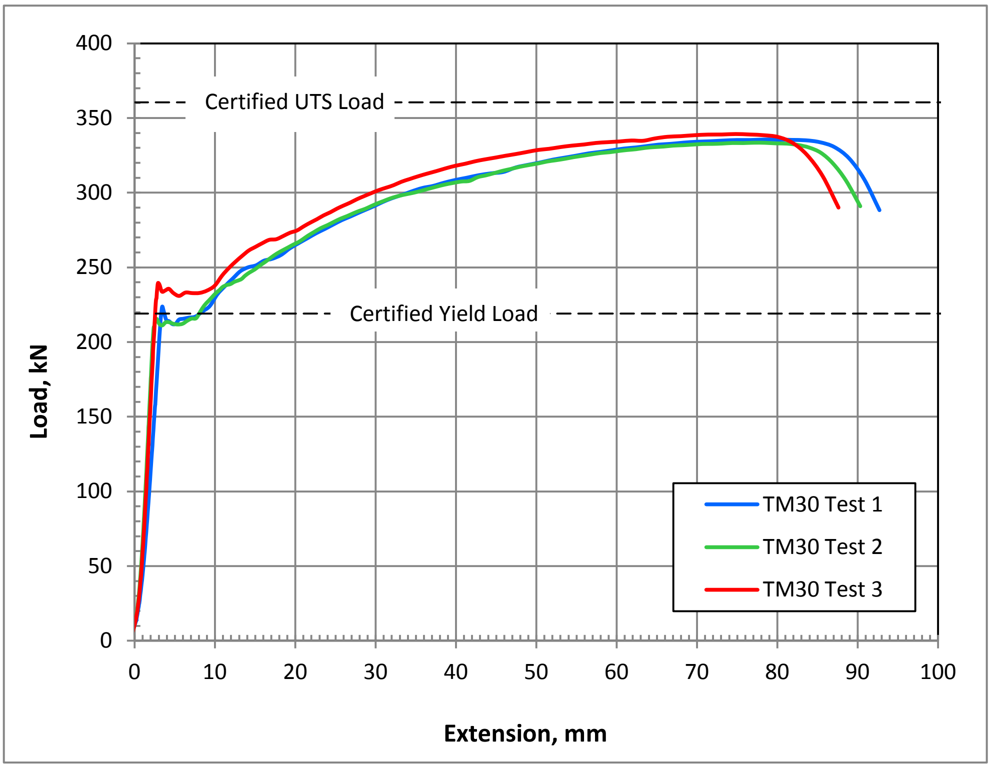



Figure 31. Load versus extension plots for Turned-Down Galvanised Mild Steel TM36 bars.

Figure 32. Failure locations on Turned-Down Galvanised Mild Steel TM30 bars. All bars were tested with two nuts on each end.

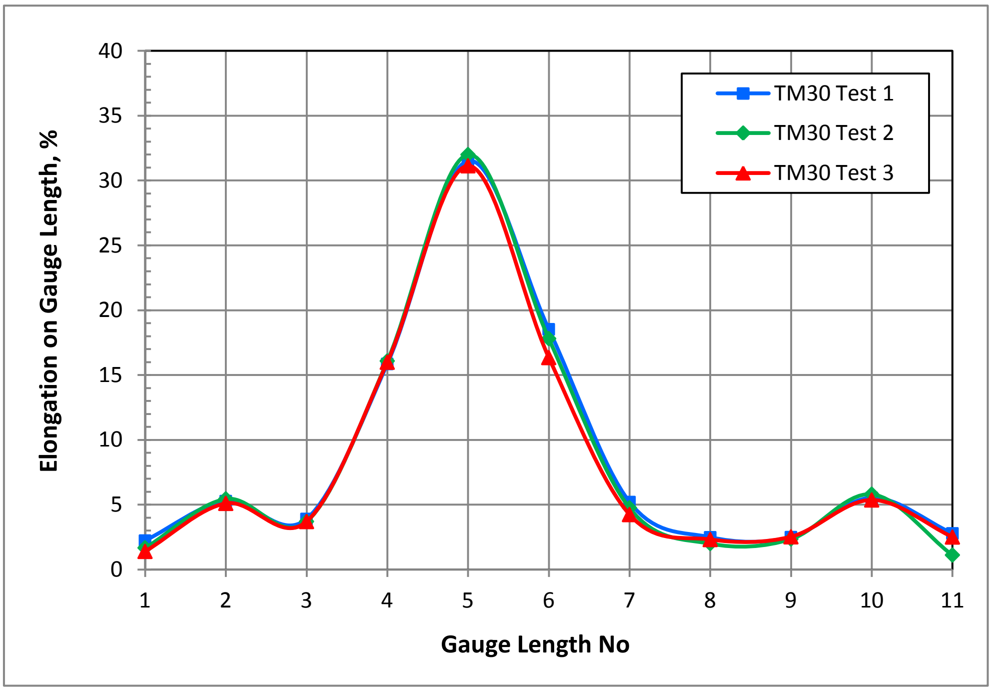



Figure 33. Elongations on gauge lengths for Turned-Down Galvanised Mild Steel TM30 bars. Gauge lengths 4, 5 and 6 were turned-down.

## 5.10 Turned-Down Stainless Steel TS30 Assemblies

Load versus extension plots are shown in Figure 34 for the three tests. These performance curves are very consistent with failure extensions (elastic plus plastic) of between 76 mm and 90 mm on the 810 mm loaded length. The minimum total extension expressed as a percentage of the loaded length was 9%. As was found for the RS38 assemblies, all the ultimate loads were about 10% higher than indicated by the material UTS on the test certificate supplied with the bar.

Figure 35 shows the failures in the bars. As expected, the failures were consistently near the centre of the turned-down length.

Plastic elongations measured on the 11 gauge lengths are plotted in Figure 36 for the three tests. As shown in Figure 2, Gauge Lengths 1 and 11 were fully within the sections covered by the nuts and locknuts. About 95% of the total plastic elongation occurred in the 300 mm long turned-down length. Because the measured ratio of 1.17 for the UTS over the yield stress (based on a 0.2% offset) was not significantly greater than the ratio of 1.15 calculated for the stress area of the thread over the turned-down section area, no significant yield occurred outside the turned-down length.

The plastic elongation results indicated that elongations of about 35% were available in the 100 mm turned-down gauge length that failed. However, the total elongation in the 300 mm long turned-down section expressed as a percentage of the turned-down length was about 25%. The minimum total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded lengths was 8.9%.

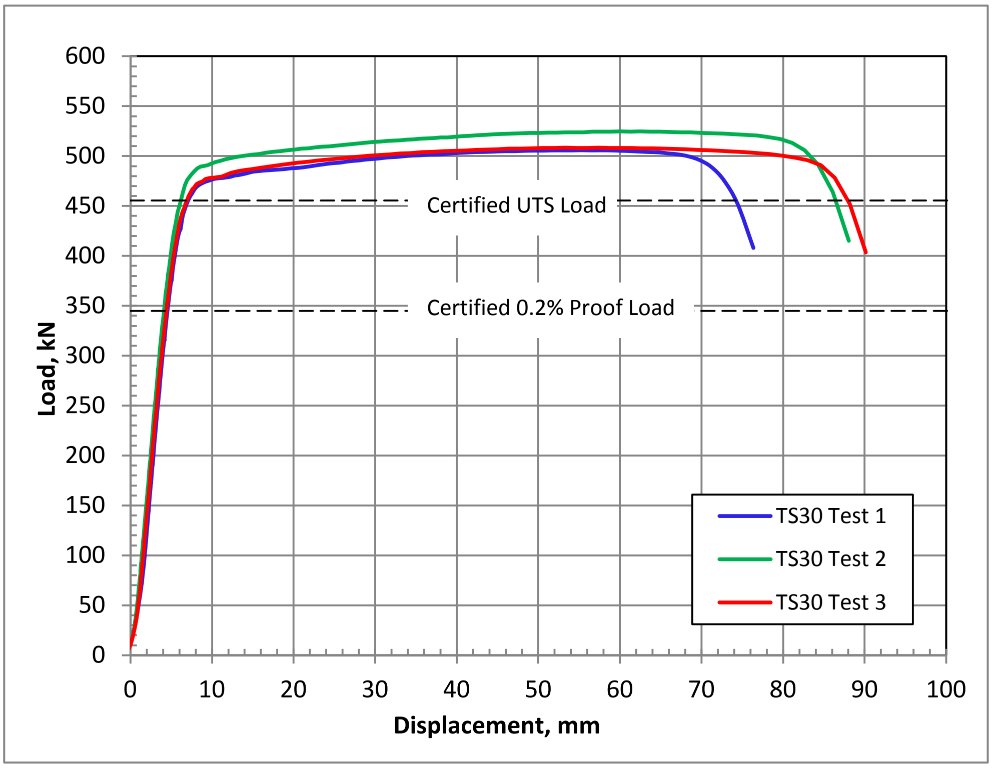



Figure 34. Load versus extension plots for Turned-Down Stainless Steel TS30 bars.

Figure 35. Failure locations on Turned-Down Stainless Steel TS30 Bars.

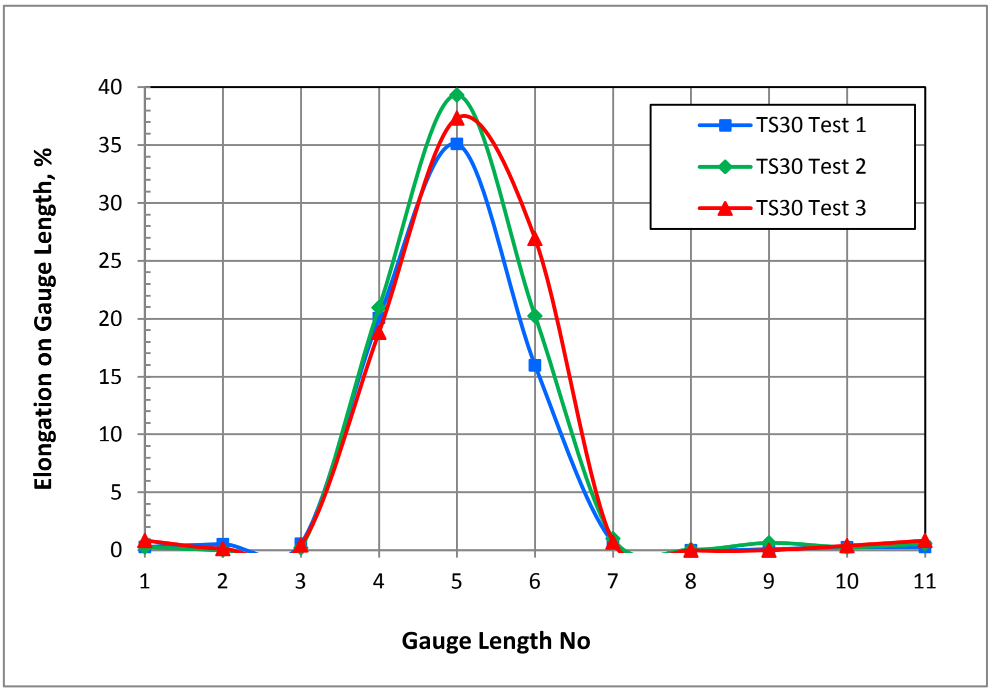



Figure 36. Elongations on Turned-Down Stainless Steel TS30 bar gauge lengths. Gauge lengths 4, 5 and 6 were turned-down.

### 5.11 Turned-Down High Tensile Steel TH30 Assemblies

Load versus extension plots are shown in Figure 37 for the three tests. The three curves show similar characteristics with failure extensions (elastic plus plastic) of between 30 mm and 32 mm on the 810 mm loaded length. The minimum total extension as a percentage of the loaded length was 3.7%. The minimum ultimate load was about 11% higher than indicated by the material UTS on the test certificate supplied with the bar.

Figure 38 shows the failures in the bars. As expected, the failures were consistently near the centre of the turned-down length.

Plastic elongations measured on the 11 gauge lengths are plotted in Figure 39 for the three tests. As shown in Figure 2, Gauge Lengths 1 and 11 were fully within the sections covered by the nuts and locknuts. About 98% of the total plastic elongation occurred in the 500 mm long turned-down length. Because the measured ratio of 1.16 for the UTS over the yield stress (based on a 0.2% offset) was not significantly greater than the ratio of 1.15 for the stress area of the thread over the turned-down section area, no significant yield occurred outside the turned-down length.

The plastic elongation results indicated that elongations of about 15% were available in the 100 mm turned-down gauge length that failed. This is significantly less than the elongation of 22% given on the test certificate supplied with the bars. The total elongation in the 500 mm long turned-down section expressed as a percentage of the turned-down length was about 5%. The minimum total plastic elongation expressed as a percentage of the loaded length was 2.9%.

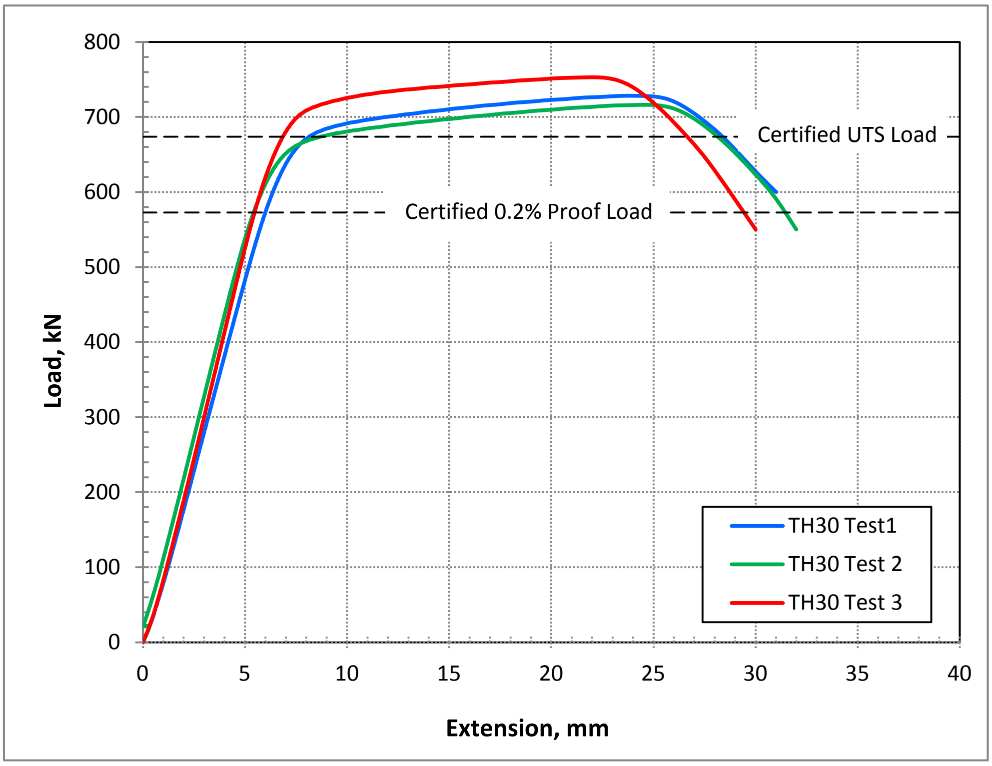



Figure 37. Load versus extension plots for Turned-Down High Tensile Steel TH30 bars.

Figure 38. Failure locations on Turned-Down High Tensile Steel TH30 Bars.

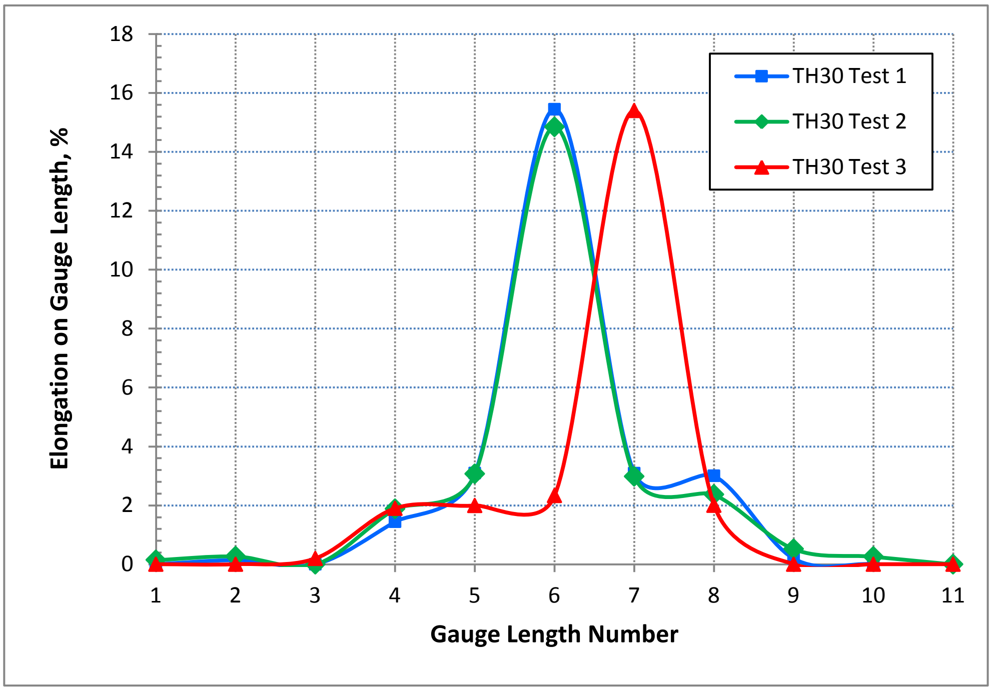



Figure 39. Elongations on Turned-Down High Tensile Steel TH30 bar gauge lengths. Gauge lengths 4 to 8 were turned-down.

### 5.12 Round Galvanised Mild Steel (RMK) Bars from Kaiapoi Railway River Bridge

Load versus extension plots are shown in Figure 40 for the two tests. The two curves show similar ultimate loads and extensions with failure extensions (elastic plus plastic) of 38 mm and 40 mm on the 720 mm loaded length. The minimum total extension (elastic plus plastic) expressed as a percentage of the loaded length was 5.3%. Yield initially occurred in the old threaded section with yielding spreading into the new threaded section and unthreaded shanks before failure occurred in the old threaded section.

Figure 41 shows the failures in the bars. There was 80 mm of loaded length of old thread and 115 mm of new thread. The failures occurred at about 25 mm from the face of the nuts on the old thread, that is, about 55 mm from the end of the thread on the shank.

Plastic elongations measured on the 10 gauge lengths are plotted in Figure 42 for the two tests. As shown in Figure 2, Gauge Lengths 1 and 10 were fully within the sections covered by the nuts and locknuts. Although significant elongation occurs over the total length between the nuts, with values of between 0.5% and 3% on the six gauge lengths on the unthreaded shank, about 67% of the total elongation occurred in the two threaded lengths that were not covered by the nuts. The elongations in the two threaded sections outside the nut zones differed by a factor of about three. This was partially due to the new thread being cut to a slightly larger root diameter than the old thread, which included a cutting depth allowance for the galvanising, and was therefore slightly stronger than would be the case for a galvanised thread. As noted in the tests of the RS38 and RS36 assemblies, which had nominally identical threads on both ends, the elongation was consistently greater in the threaded section that yielded first.

Elongations of up to 22% occurred in the failure gauge lengths on the old thread. This was less than the 33% that occurred in the threaded failure gauge lengths on the RM36 assemblies which were fabricated from different bar. The minimum total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded length was 5.6%.

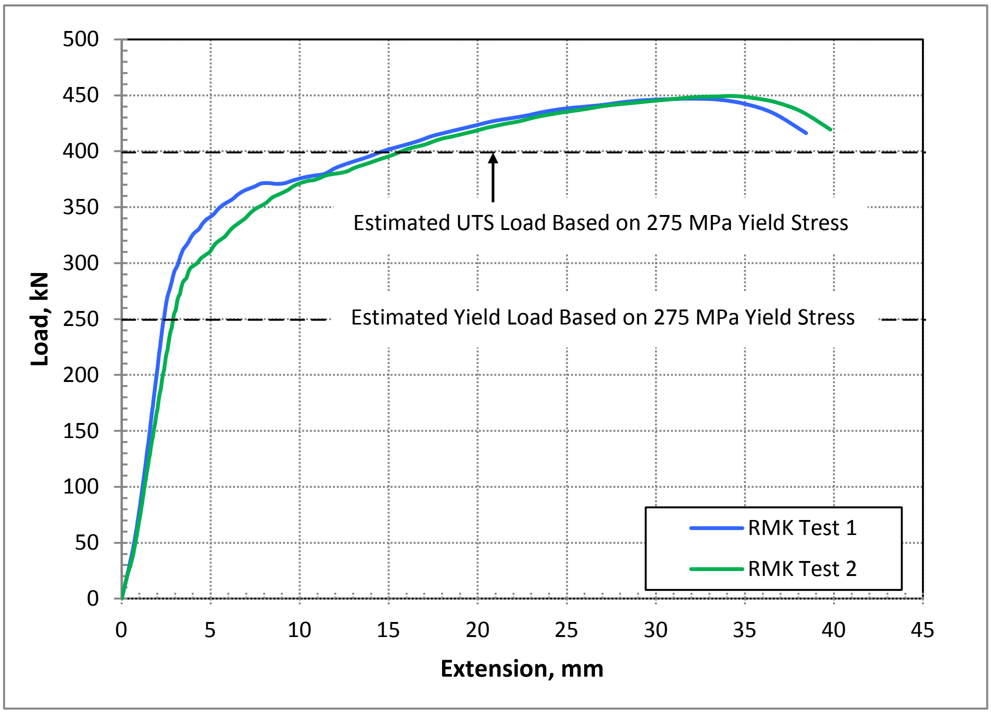



Figure 40. Load versus extension plots for Round Galvanised Mild Steel RMK assemblies from the Kaiapoi Railway River Bridge.

Figure 41. Failure locations on RMK bars.
(Recovered from Kaiapoi Railway River Bridge.)

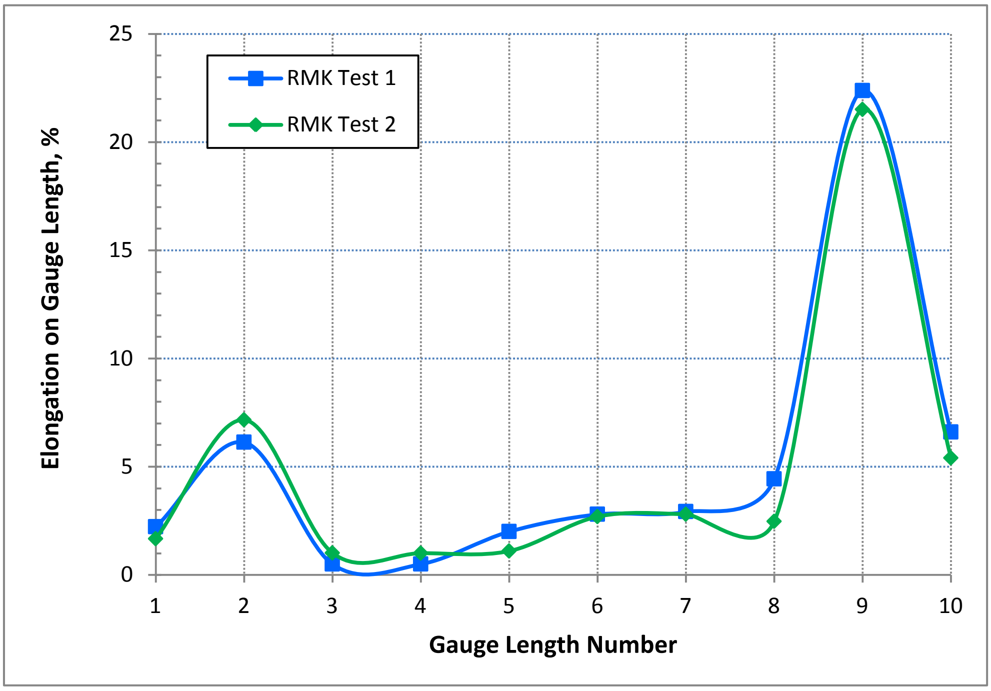



Figure 42. Elongations on Round Galvanised Mild Steel RMK bar gauge lengths. (Bars recovered from Kaiapoi Railway River Bridge.)

### 5.13 Bar Head Tests on Bars from Kaiapoi Railway River Bridge

After the initial tensile tests on the two bars recovered from the Kaiapoi Railway River Bridge a further two tensile tests were carried out to determine the strength of bar heads formed by installing nuts tight against a thread termination. Effectively a head of this type has no loaded length of thread except for the thread immediately under the nut. In the original installation on the Kaiapoi Railway River Bridge, the linkage bars were formed with a head of this type at one end, and a nut on a 135 mm long section of thread at the other end. At the head end the bars were cut flush with the end of the nut and a light weld applied to the end of the bar to prevent the nut turning. In the 1960's this was a common method of fabricating heads on large diameter linkage bars.

For the bolt head tests, two 250 mm long test specimens were cut from the initially tested bar shanks. Short sections of 1 ½ BSW threads were machined at each end to fit the nuts recovered from the bridge (also used in the initial testing). The specimens were set up for tensile testing using the loading frames used in the Stage 1 and 2 tensile tests (see Figures 2 and 3) and the Opus Central Laboratories 1 MN Shimadzu UTM. Nuts were installed on both ends and wound tightly to the ends of the threads but were not welded to the bars. Details of the testing set-up are shown in Figure 43.

Figure 43. Specimen arrangement in Shimadzu UTM for testing bar heads on bars recovered from Kaiapoi Railway River Bridge.

The bars were loaded in tension at a rate of approximately 250 kN/minute.

Failures occurred at the loaded face of the nut on one end of the bar as shown in Figure 44. The ultimate loads for the two bars were 522 kN and 513 kN. These loads were about 15% greater than the loads recorded in the initial tensile tests (see Figure 40). In both tests the location and appearance of the failures were similar to those that occurred in the bars during the 2010 Darfield Earthquake (see Figures 44 and 45).

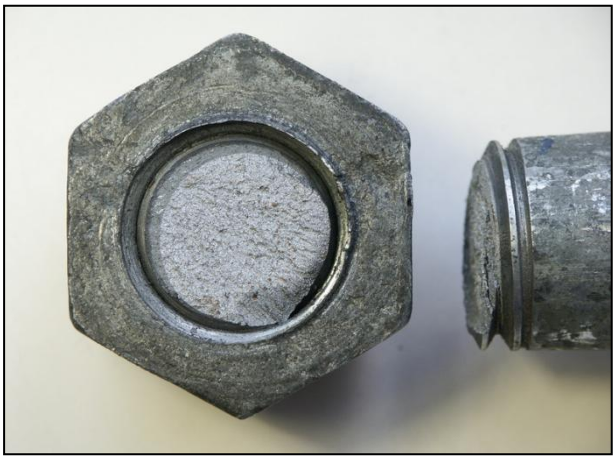


Figure 44. Failure in bar head test.

Figure 45. Failure at bar head on bridge.

One reason for the higher ultimate loads at the bar head is related to the thread run-out of the machine cut thread. Immediately at the failure section of the thread the root diameter is greater than at a distance of several thread pitches from the end of the thread. The stress area of the 1½ inch BSW thread is 80% of the shank area so at the failure section the area is likely to be significantly greater than the stress area. The presence of the nut and a secondary stress zone may also be a factor influencing the ultimate load.

Although the testing did not exactly simulate the linkage bar configuration on the bridge or the loading of the linkage bars during the Darfield Earthquake, the results indicated that failure of the head end was unlikely unless this end received higher loads than the threaded length at the nut end. Inequality of load at either end of the bars probably occurred because the bar shanks were clamped by large friction forces and distortion from relative transverse movements between the adjacent spans.

The total plastic elongation of the 155 mm long loaded length of the nut test specimens, including the sections of thread under the nuts, was about 6 mm. In a bridge installation this small elongation (less for a single head) may not be sufficient to allow the span inertia loads to become evenly distributed over a number of linkages positioned across the width of the bridge before failure occurs in the most heavily loaded linkages. Limited tensile ductility at linkage bar head ends would be less of an issue if the shanks did not become clamped by relative transverse movements between the spans.

### 5.14 Turned-Down Galvanised Mild Steel TMO Assemblies from Otaki River Bridge

Load versus extension plots are shown in Figure 46 for the three tests. The three curves are reasonably similar, with some variation in the ultimate loads and with total extensions (elastic plus plastic) on the 1060 mm loaded lengths of between 144 mm and 156 mm. The minimum total extension expressed as a percentage of the loaded length was 14%. The lowest yield load was 180 kN recorded in Test 3. This was about 5% below the minimum dependable yield load of 190 kN specified on the drawings. The yield loads recorded for the other two bars were just on or above the specified yield load.

Figure 47 shows the failures in the bars. As expected, the failures were in the turned-down lengths.

Plastic elongations measured on the 13 gauge lengths are plotted in Figure 48 for the three tests. As shown in Figure 2, Gauge Lengths 1 and 13 were fully within the sections covered by the nuts and locknuts. About 95% of the total elongation occurred in the 670 mm long turned-down length. There were small but significant elongations in the threaded sections (about 4%). These occurred because the measured ratio of 1.8 for the UTS to yield stress was significantly greater than the ratio of 1.2 between the stress area of the thread and the area of the turned-down section.

Plastic elongations of up to 38% measured as a percentage of the gauge length were recorded in the 100 mm gauge lengths that failed in the turned-down sections. This elongation was a little greater than the corresponding value of 32% for the TM30 bars indicating that both steels had similar elongation characteristics. The total elongation in the 670 mm long turned-down section expressed a percentage of the turned-down length was about 20%. The minimum total plastic elongation (including elongation under nuts) expressed as a percentage of the loaded lengths was 13.4%. The equivalent value for the TM30 assemblies with a 300 mm long turned-down section was 10.5%.

Comparison of Figure 31 (TM30 assemblies) with Figure 46 (TMO bars) illustrates the benefit of increasing the turned-down section from 300 mm to 650 mm in terms of increasing the total elongation. An elongation of greater than 13% occurred on each of the seven turned-down gauge lengths on the longer TMO assemblies. In comparison the elongation was greater than 15% on the three turned-down gauge lengths of the shorter TM30 assemblies. However, the total elongations of the shorter bars reached a minimum of 88 mm (compared with 144 mm for the TMO assemblies) which would be sufficient in most applications.

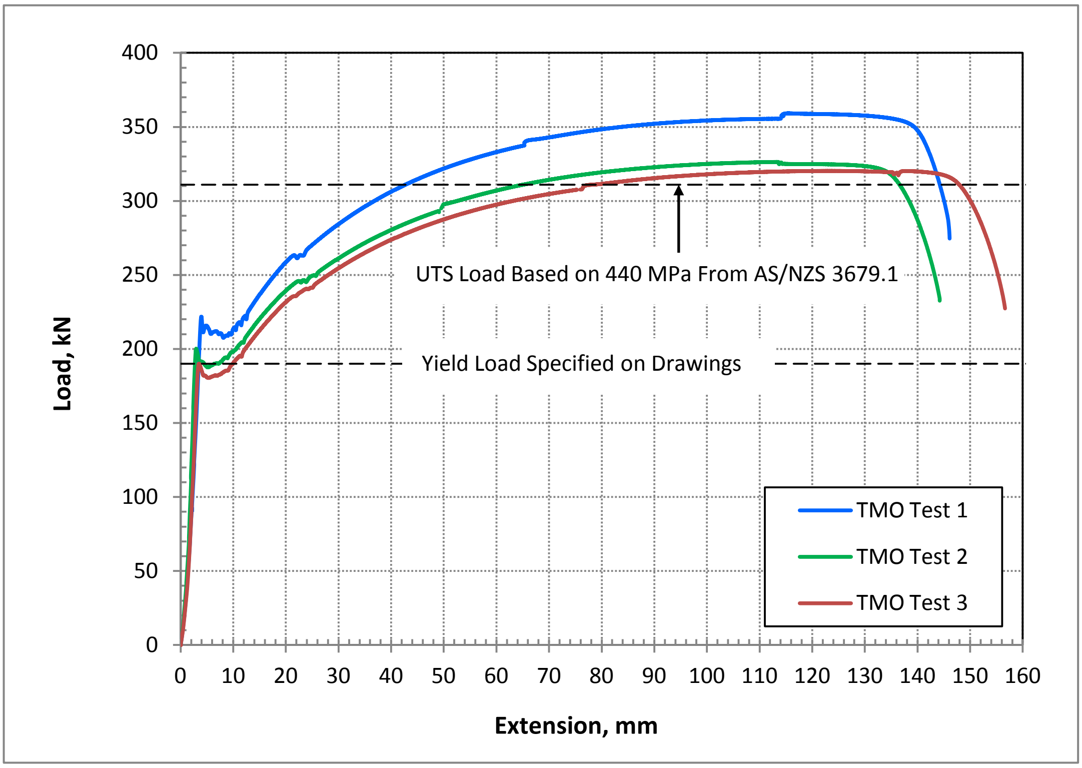



Figure 46. Load versus extension plots for Turned-Down Galvanised Mild Steel TMO assemblies recovered from the Otaki River Bridge. Minor unevenness in curves at extensions greater than 48 mm resulted from unloading and reloading to extend the extension range of the jack.

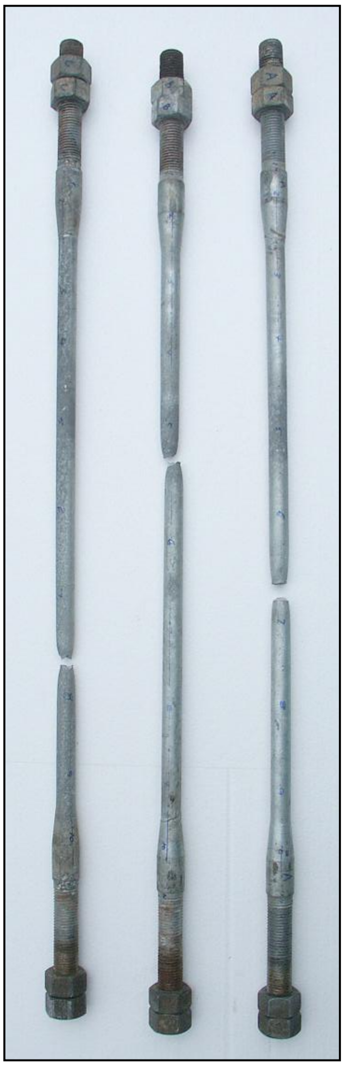


Figure 47. Failure locations on Turned-Down galvanised mild steel TMO bars. (Recovered from Otaki River Bridge.)

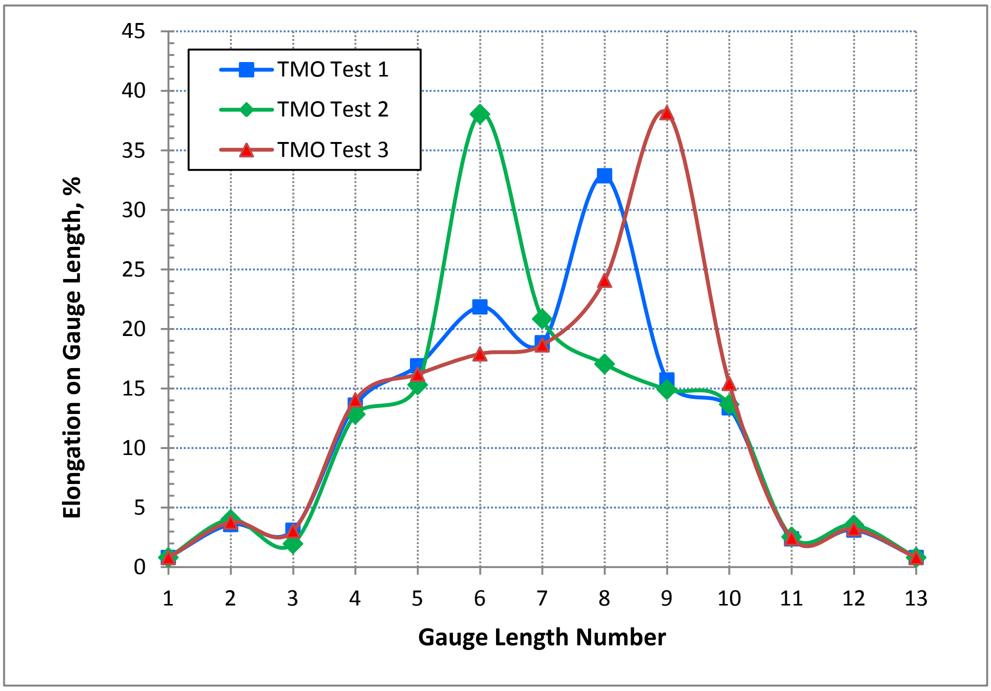



Figure 48. Elongations on gauge lengths for Turned-Down Galvanised Mild Steel TMO bars. Gauge lengths 4 to 10 were turned-down. (Bars recovered from Otaki River Bridge.)

## 5.15 Nut Testing

The strength of the nuts was assessed by both undertaking several of the linkage bar tests without locknuts and in a separate test of the nuts using 650 mm long sections of grade 4140 high tensile bar in the centre-hole jack loading system used in Stage 3 of the testing (see Figures 5 and 6). Results of these tests are summarised in Table 4.

**Table 4. Nut Strength Results**

| Nut Type | Nut Property Class (Manufacturer) | Nut Proof Load[1] kN | Assembly Types Nuts Used On | Maximum Load Applied in Assembly Tests[2] kN | Maximum Load in Assembly Tests Without Locknuts kN | Minimum Ultimate Load In Nut Test kN | Nut Failure Description |
|---|---|---|---|---|---|---|---|
| RB32 Galvanised | - (Reidbar) | - | RB32 | 563 | 508 | - | Nut split in RB32 assembly Test 1 without locknuts. |
| M27 High Tensile | 10 (GM) | > 487 | FH27 | 679 | 666 | - | Nuts undamaged. |
| M36 Galvanised | 8 (Not marked) | 952 | RM36, TM30, TH30 | 753 | - | 785 | Test bar failed in nut test. Nuts undamaged. |
| 1 ½ inch BSW Galvanised | - (Not marked) | - | RMK | 450 No locknut on one end | 522 Nut head test | - | Nuts undamaged. |
| M36 Stainless | A4-70 (LE) | 572 | RS38, TS30 | 599 | 569 | 590 | Nut stripped in RS38 assembly Test 1 without locknuts and in nut tests. |
| M36 Stainless | A4-70 (OL) | 572 | RS36, US36 | 671 | - | 690 | Nut stripped in nut test. |
| M36 Stainless | A4-80 (WL) | 654 | US36 (Test 1) | 672 | - | 615 | Nut stripped in nut test. |

Notes: 1. For stainless steel nuts proof load taken as Property Class UTS x Thread Stress Area. Value for M36 galvanised nuts taken from AS/NZS 1252:1996 and Inspection Certificate.

2. This is the maximum load applied in all assembly types listed using locknuts.

### *5.15.1 Proprietary Bar Nuts*

The Reidbar nuts were only tested in the initial test of a RB32 assembly which was set-up without locknuts. One of the nuts failed by splitting at a load of 508 kN which was about 10% lower than the maximum of the bar ultimate loads recorded in the testing.

In one of Freyssibar FH27 assembly tests the bar was loaded with only single nuts at both ends. The bar failed in the threaded section about 80 mm from the face of the nut without any apparent damage to the nut. After the test the nuts could still be moved freely on the threads outside the loaded length.

No specific tests were carried out on the Macalloy nuts. They performed satisfactorily in the assembly tests and were not distorted by the bar ultimate loads. After the tests they could still be moved freely on the threads outside the loaded length.

### *5.15.2 M36 Galvanised Steel Nuts*

Only one brand of nut was used on all of the mild and high tensile galvanised steel assemblies (see Section 3.4.1). Two of these nuts, one each end of a short bar, were test loaded in the jack set-up used for the nut testing. The high tensile 4140 steel bar failed in the threaded end without any apparent damage to the nut. The bar ultimate load of 785 kN exceeded the maximum load of

753 kN applied to the locknutted mild and high tensile galvanised steel bars in the assembly tests.

To fail the galvanised nuts by thread stripping it would be necessary to load them using 4140 bar that had been heat treated after threading to enhance its UTS. Unfortunately no fabricators carry out heat treatment in the Wellington area and because the nuts were satisfactory for the assemblies tested it was considered unnecessary to undertake more comprehensive destructive testing.

#### 5.15.2 1 ½ inch BSW Galvanised Steel Nuts

Locknuts were only used on one end of the two RMK assemblies recovered from the Kaiapoi Railway River Bridge. A single nut was used on the end that had failed on the bridge and had been rethreaded after removal from the bridge.

After the tensile tests on the two assemblies were completed, a further two tensile tests were carried out on short lengths of bar cut from the shanks to determine the strength of bar heads formed by tightening nuts against the end of a short length of thread (see Section 5.13). In the tests on the two assemblies the bars failed in the old threaded section about 25 mm from the face of the nuts without any apparent damage to the nut. In the bar head tests the bar failed very close to the face of one of the nuts, again without damage to the nuts (see Figure 44). After both sets of tests the nuts could still be moved freely on the threads outside the loaded length.

#### 5.15.3 Stainless Steel Nuts

Three different brands of stainless steel nut were used in the assemblies formed from Grade 316 stainless steel bar. Details of these nuts are summarised in Section 3.4.1 and in Table 4 above. All of the stainless steel nuts were tested on short lengths of high tensile steel bar in the nut testing jack set-up. Two of each of the A4-70 (OL) and A4-80 (WL) nuts were tested with a single nut on each end of a test bar. Four of the A4-70 (LE) nuts were tested using two test bars with a single nut on each end. In addition, this type of nut was tested in one of the RS38 assembly tests by using a single nut on each end of the bar instead of locknuttting.

The ultimate loads for the stainless steel nuts are summarised in Table 4. Only the A4-70 (OL) nuts had an ultimate load (690 kN) that exceeded the maximum ultimate load of 672 kN for all of the assemblies tested with Grade 316 stainless steel bar. Locknutting by using two nuts was clearly necessary to achieve satisfactory performance of these assemblies.

The ultimate load of the A4-80 nuts (615 kN) in the nut test was less than the nut proof load of 654 kN. However, the assembly test set-up did not follow the standard nut testing procedure, which uses a hardened and threaded test mandrel (for example, see AS/NZS 4291.2:1995) so the nut failure at less than the proof load did not necessarily indicate that it did not meet its specified strength.

All the nuts failed by thread stripping. Initially the nuts deformed by elongating across their width on the loaded side. This “opened” the threads on one end reducing the shear area of the thread at the contact points with the bar threads. Thread “opening” apparently led to shear failure in the nut threads on the loaded side followed by “unzipping” of the remaining threads. The loaded side of the nuts elongated by between 2 mm to 3 mm measured across the flats. The thread height in the nuts was about 2.6 mm so a width elongation of 3 mm results in a significant reduction in the thread shear area. The A4-70 (OL) nuts elongated less than the two other types and this may have contributed to their higher ultimate loads. The deformation in the nut width can clearly been seen in Figure 49 which shows the nut and a section of thread which failed in the RS38 assembly test.

The performance of the stainless steel nuts would clearly be improved by increasing their height.

The heights of all three brands of stainless steel nuts used in the testing varied from 27.6 mm to 28.2 mm. These heights are less than the minimum height of 29.4 specified in the current DIN EN ISO 4043 Standard: Hexagon Nuts, Style 1. The width across the flats of all three brands of nuts was between 54.3 mm and 54.7 mm and exceeded the minimum of 53.8 mm given in the Standard. The nuts were apparently manufactured to a superseded Standard DIN 934 and in this standard the minimum nut height was 27.4 mm.

Figure 49. Failure of A4-70 (LE) nut in RS38 assembly test. Note the distortion across the width of the nut

# 6. Comparison of Performance

## 6.1 Load – Extension Plots

Figure 50 compares load versus extension plots for the 13 different types of bar assemblies. To simplify the comparison, and because all the plots for each test within an assembly type were similar, only the test with the minimum extension measured for each type is plotted. (In some cases the bars with the minimum extensions did not have the minimum ultimate loads.) The plots for the RB32 and RS38 tests where nuts failed have been excluded and the curve shown for these assemblies is for the test with the next lowest extension.

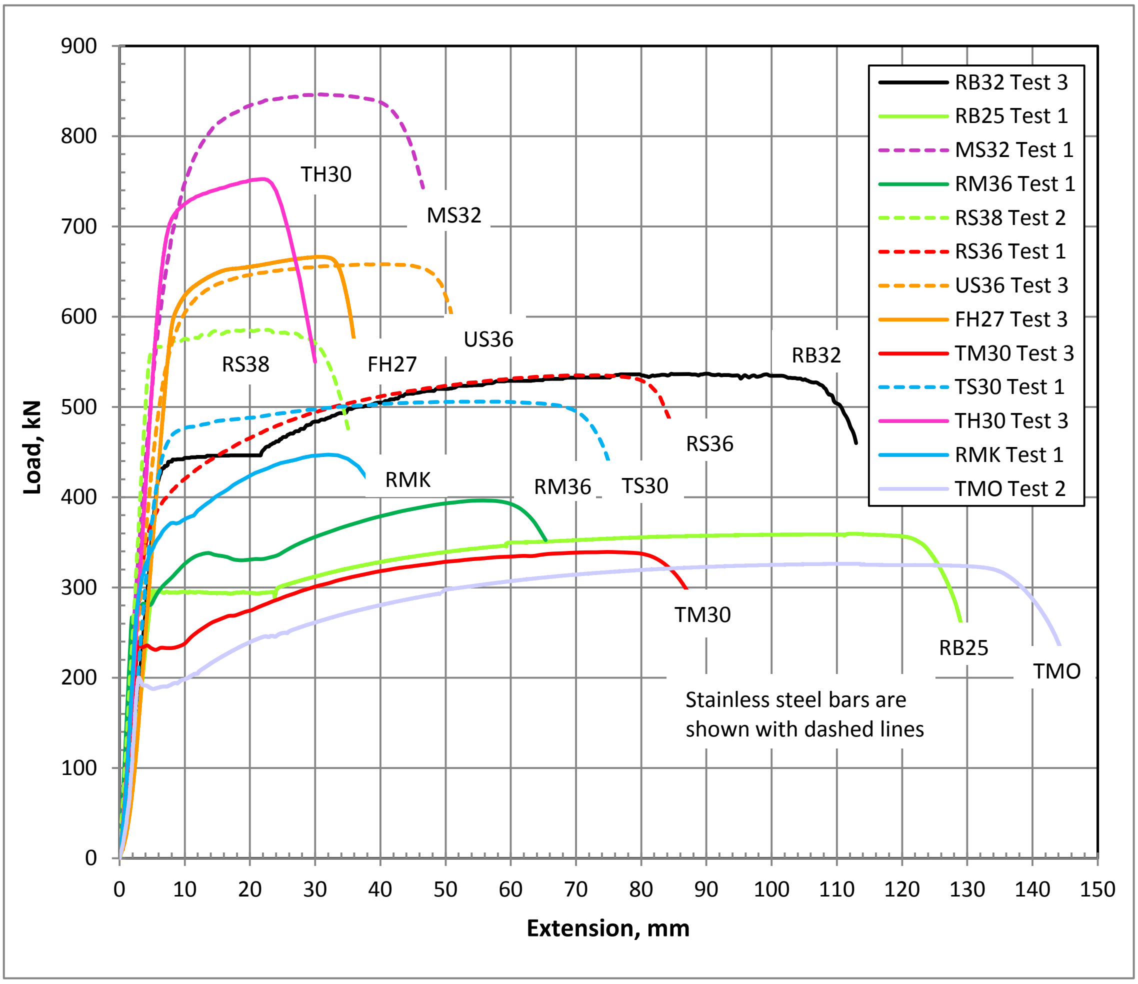



Figure 50. Comparison of load versus extension plots for the thirteen assembly types.
(The loaded lengths varied from 620 mm to 1060 mm – see Table 6)

## 6.2 Yield Stresses, Ultimate Stresses and Yield and Ultimate Loads

Comparisons of the specified and measured yield and ultimate stresses, and yield and ultimate loads are given in Table 5. The "specified" minimum yield and UTS values were based on test certificates where available. No certificates were obtained for the RB32, RB25 and MS32 bars and the specified minimum stresses were taken from published technical manuals. Certificates were also not available for the bars recovered from the two bridges (RMK and TMO). The contract drawings for the Otaki River Bridge specified Grade 300 steel and a minimum dependable yield load for the fabricated bars of 190 kN. Except for the turned-down and proprietary bars (RB32, RB25, MS32 and FH27) the "specified" yield and UTS loads were calculated from the specified stresses using the stress area of the M36 threads. Minimum yield

**Table 5. Specified and Measured Yield and Ultimate Stresses and Forces**

| Assembly Identifier | Specified Yield or 0.2% Proof Stress MPa | Specified Yield or 0.2% Proof Load kN | Test Minimum Yield or 0.2% Proof Load[1] kN | Specified Minimum UTS MPa | Specified Minimum Ultimate Load kN | Test Minimum Ultimate Load[1] kN | Ratio Test Ultimate Load/ Test Yield Load | Ratio Test Ultimate Load/ Specified Ultimate Load |
|---|---|---|---|---|---|---|---|---|
| RB32 | 500 | 402 | 436 | 575 | 462 | 531 | 1.2 | 1.15 |
| RB25 | 500 | 246 | 290 | 575 | 282 | 356 | 1.2 | 1.26 |
| MS32 | 650 | 506 | 650 | 800 | 622 | 847 | 1.3 | 1.36 |
| RM36 | 310 | 253 | 240 | 510 | 417 | 387 | 1.6 | 0.93 |
| RS38 | 488 | 399[1] | 490 | 644 | 526 | 587 | 1.2 | 1.12 |
| RS36 | 325 | 266 | 350 | 604 | 493 | 535 | 1.5 | 1.08 |
| US36 | - | - | 540 | 725 | 592 | 658 | 1.2 | 1.11 |
| FH27 | 980 | 562 | 615 | 1092 | 626 | 666 | 1.08 | 1.06 |
| TM30 | 310 | 219 | 212 | 510 | 360 | 334 | 1.6 | 0.93 |
| TS30 | 488 | 345 | 420 | 644 | 455 | 506 | 1.2 | 1.11 |
| TH30 | 810 | 575 | 650 | 953 | 674 | 716 | 1.1 | 1.06 |
| RMK | 275[2] | 249 | 280 | 440[2] | 399 | 447 | 1.6 | 1.12 |
| TMO | 280 | 190 | 180 | 440 | 339 | 324 | 1.8 | 0.98 |

Notes: 1. The minimum test yield and ultimate loads do not apply specifically to the results plotted in Figure 50, which are for the tests giving the minimum extensions.

2. Estimated value. No test certificates available.

and UTS loads (characteristic loads) for the RB32, RB25 and MS32 bars are given in the product technical manuals. The FH27 minimum loads were based on the stresses on the test certificate supplied with the bars and the full section area of the bar. The threads are rolled and the stress area is not significantly less than the bar section area.

Only the Reidbars and mild steel bars had distinct yield levels (see Figure 50). Estimates of the minimum yield levels were made for the other bars by adjusting the elastic part of the curves and using a 0.2% strain offset. Because the geometry of the bars and the load application method did not closely replicate standard tensile test specimen shapes and testing procedures, the test yield loads for the stainless and high tensile steel bars are approximate.

## 6.3 Elongations

A summary of specified and measured elongations is given in Table 6. The measured values in the tables are the minimum of the values measured in the three tests for each assembly type (excluding nut failures). The test elongations on the gauge length are only for the particular gauge length that failed and the values are expressed as a percentage of the gauge length. The tabulated total elongations measured over the loaded length include the elongations that occurred under the nuts and are given in both millimetres and as a percentage of the loaded length. Values of total elongation on the turned-down lengths, where present, are also tabulated as these are of interest in optimising the length of any turned-down section. Again these are given in both millimetres and as a percentage of the turned down length.

The "specified" elongation values given in Table 6 were from test certificates supplied with the bars except for the RB32, RB25 and MS32 bars and the TMO bars recovered from the Otaki River Bridge. The RB32, RB25 and MS32 specified values were taken from the product

**Table 6. Specified and Measured Elongations**

| Assembly Identifier | Assembly Loaded Length Between Nuts | Specified Elongation | Minimum Test Elongation on Failure Gauge Length | Minimum Test Elongation on Assembly Loaded Length | | Minimum Test Elongation on Total Length of Turned-Down Section | | Calculated Elastic Extension at Yield Load |
|---|---|---|---|---|---|---|---|---|
| | mm | % | % | % | mm | % | mm | mm |
| RB32 | 620 | 20 | 26 | 17 | 107 | - | - | 2.4 |
| RB25 | 880 | 20 | 26 | 14 | 124 | - | - | 3.1 |
| MS32 | 660 | 15 | 14 | 6.1 | 40 | - | - | 3.5 |
| RM36 | 810 | 34 | 31 | 7.9 | 64 | - | - | 1.3 |
| RS38 | 810 | 47 | 28 | 4.1 | 33 | - | - | 2.4 |
| RS36 | 810 | 51 | 43 | 10 | 81 | - | - | 1.8 |
| US36 | 810 | - | 23 | 5.8 | 47 | - | - | 3.2 |
| FH27 | 820 | 17 | 9.7 | 3.3 | 27 | - | - | 5.0 |
| TM30 | 810 | 34 | 32 | 11 | 85 | 21 | 64 | 1.3 |
| TS30 | 810 | 47 | 35 | 9.0 | 72 | 23 | 70 | 2.4 |
| TH30 | 810 | 22 | 15 | 3.0 | 24 | 4.7 | 24 | 4.2 |
| RMK | 720 | - | 22 | 5.8 | 39 | - | - | 1.2 |
| TMO | 1060 | 30 | 33 | 13 | 140 | 19 | 128 | 1.4 |

technical manuals. The TMO specified value is the minimum elongation specified for Grade 300 steel in AS/NZS 3679.1.

Calculated elastic extensions at the yield force level of each test assembly are given in Table 6. These values include an allowance for the extensions in the threads under the nuts. They have been corrected for the estimated extensions in the loading frames. Measured elastic extensions are not listed but were a factor of between 1.5 and 2.5 greater than the calculated values. This difference was thought to be mainly due to take-up in the washers and threads. It was influenced by the initial tightness of the assembly and the number and shape of the washers used within the loaded length. Measured values varied quite significantly between the three tests on most of the assemblies (see Figures 7, 10, 16, 19, 28, 31, 37 and 40). The elastic extensions are small in comparison to the plastic elongations so reducing them by greater initial tightening would not make a significant change to the load versus extension plots.

### 6.4 Location of Plastic Elongations

The location of the plastic elongations that occurred within the loaded lengths is summarised in Table 7. All the elongations in the table are expressed as a percentage of the total elongation over the loaded length and are the mean of results from the three tests except for the RB32 and RS38 assemblies where the nut failure tests were excluded.

For the *uniform bars* (RB32, RB25, MS32 and US36) assemblies the failure zone was assumed to extend over a nominal 100 mm section either side of the 100 mm nominal gauge length in which the failure occurred.

For the *bars with threaded ends and uniform shanks* (RM36, RS38, RS36, FH27 and RMK) the total elongation in the threaded sections, excluding the sections covered by nuts, is listed.

For the *bars with turned-down shanks* (TM30, TS30, TH30 and TMO) the failure zone was assumed to extend over a nominal 100 mm section either side of the 100 mm nominal gauge length in which the failure occurred. Where the turned-down length was greater than 300 mm,

**Table 7. Location of Measured Plastic Elongations**

| Bar Type | Measured Elongation on Nut Zone, % of Total Elongation | Measured Elongation on 300 mm Long Failure Zone, % of Total Elongation | Measured Elongation on End Threaded Lengths % of Total Elongation | Measured Elongation on Turned Down Length Outside Failure Zone % of Total Elongation | Measured Elongation on Other Sections % of Total Elongation |
|---|---|---|---|---|---|
| Uniform Bars | | | | | |
| RB32 | 9 | 53 | - | - | 38 |
| RB25 | 4 | 41 | - | - | 55 |
| MS32 | 4 | 65 | - | - | 31 |
| US36 | 1 | 60 | - | - | 39 |
| Bars With Threaded Ends and Uniform Shanks | | | | | |
| RM36 | 4 | Failure in threads | 54 | - | 42 |
| RS38 | 7 | Failure in threads | 92 | - | 1 |
| RS36 | 9 | Failure in threads | 71 | - | 20 |
| FH27 | 2 | Failure in threads | 66 | - | 32 |
| RMK | 9 | Failure in threads | 67 | - | 24 |
| Bars With Turned-Down Shanks | | | | | |
| TM30 | 2 | 75 | 10 | - | 14 |
| TS30 | 0.4 | 97 | 0.5 | - | 2 |
| TH30 | 0 | 81 | 1 | 16 | 2 |
| TMO | 1 | 53 | 5 | 39 | 2 |

the elongations on the turned-down length outside the failure zone are also presented (TH30 and TMO bars).

The nut zone referred to in the table is the length of bar or machined thread covered by the nuts.

## 6.5 Proprietary Bars and Fully Threaded Assemblies

The RB32 and RB25 assemblies had the greatest plastic elongations at 17% and 14% of the loaded length respectively. The MB32 assemblies had the highest ultimate loads but only about 37% of the elongation of the RB32 bars, reflecting the lower tensile ductility available in the higher strength steel.

The FH27 high tensile steel bars had high strength but low elongation at only 3.3% of the loaded length. They should only be used in special applications where strength is more important than tensile ductility.

One potential problem with the RB32, RB25, MS32 and US36 bars is that because of their uniformity over the loaded length the position of the failure section may occur almost anywhere within the length. When the failure occurred close to the nuts the elongations in the MS32 and US36 assemblies were significantly reduced (Figures 13 and 25). This was less of an issue for the RB32 and RB25 assemblies where a large part of the total elongation occurs outside the failure zone (see Table 7).

The greater elongation of the RB25 compared to the RB32 bars (124 mm compared to 105 mm) was mainly due to the longer loaded length used in the tests of the RB25 bars. Although only a single loaded length was tested on the MS32 and US36 bars, the elongations would not increase with increasing length as much as measured on the RB bars since a greater part of their elongation occurred in the failure zone (see Table 7).

### 6.6 Assemblies with Threaded Ends and Uniform Shanks

There was not a large difference between the plastic elongations measured on the loaded length of the RM36 and TM30 assemblies fabricated from the same mild steel bar (8% compared to 11%). However, obtaining good elongations from RM assemblies requires the shank diameter to be no larger than the thread nominal diameter and since a large part of the elongation occurred in the threaded length the loaded sections of thread at both ends of the bar need to be quite long. The tests on the mild steel bars showed that if the shank and nominal thread diameters are the same, strain hardening following yield in the threaded sections causes significant elongation in the shank. This is illustrated by comparing the elongations of the RS38 and RS36 stainless steel assemblies. Although these assemblies were not fabricated from the same stainless steel bar the elongations on the test certificates for the two steels were similar. The RS36 assemblies had about 2.5 times the elongation of RS38 assemblies. Significant yield strain did not develop in the larger diameter RS38 shanks before failure occurred in the M36 threaded sections.

On all of the five types of assemblies tested with threaded ends and uniform shanks, more than 54% of the elongation occurred in the threaded ends (greater than 66% except for the RM36 assemblies). The loaded length of thread at each end of the assemblies tested varied from 75 mm (RM36, RS38 and RS36 assemblies) to 125 mm (FH27 assemblies). The testing indicated that to achieve high elongations lengths of this order are required. Ideally, for 36 mm or smaller diameter bars, loaded thread lengths of at least 100 mm should be used at both ends of the assemblies.

### 6.7 Assemblies with Turned-Down Shanks

The TM30, TS30 and TMO assemblies with turned-down shanks all exhibited good elongation with the minimum recorded values ranging from 72 mm to 140 mm. The minimum elongation recorded for the TH30 assemblies was 24 mm (lowest of all assemblies). This low elongation was a consequence of using high tensile 4140 steel having low tensile ductility.

Comparison of the elongations for the TM30 and TMO assemblies shows that increasing the length of the turned-down section from 300 mm to 670 mm increased the total elongation by a factor of about 1.6 (85 mm to 140 mm). Although the assemblies were fabricated from different mild steel bar the materials probably had similar elongation characteristics. For optimum elongation, the 300 mm length used for the TM30 and TS30 assemblies appeared rather too short and ideally for a 30 mm turned-down diameter a length of at least 500 mm should be used.

Turning mild or stainless steel bar down to ensure that failure does not occur in the threads will generally increase the elongation and give greater certainty on performance since a turned-down assembly is not sensitive to installation tolerances that could affect the length of thread in the loaded length. With end threaded uniform shank assemblies there could be a risk of premature failure in the threaded sections, which is not an issue with turned-down assemblies. A particular problem could arise if the head of the bar is formed by installing a nut on just sufficient length of thread to accommodate the nut (see Section 5.13).

Turning the stainless and mild steel bars down increased the cost of the assemblies by about 20% and 50% respectively (see Table 3). For all the non-proprietary assemblies threads had to be machined onto the bars and in the case of the RM36 and TM30 assemblies the full length of the bar was turned-down from black bar. Costs of turning-down would be a greater percentage of the total assembly cost if bars were available that did not require any machining (for example, if standard bolts were used) but this would rarely be the case for the length of linkage assembly usually employed. Reidbar, Macalloy and fully threaded bars can be used for linkages without any machining. Reidbars cost significantly less than the mild steel RM36 bars but both types of assemblies are not very expensive. The Macalloy and fully threaded stainless bars are unlikely to offer any significant cost advantage over stainless steel bars requiring threads to be machined on their ends. However, 36 mm diameter stainless steel bar is not readily available in New Zealand and needs to be machined down from 38.1 mm diameter bar which increases the cost to about the same as a stainless steel bar with a turned-down section of shank.

### 6.8 Assemblies from Ahuriri and Kaiapoi River Bridges

Two 25.4 mm (1 inch) diameter round galvanised mild steel linkage bars removed from the Ahuriri River Bridge (SH 8 near Omaramara, Otago) were tensile tested at Opus Central Laboratories in July 2008. Their elongation performance was similar to that of the RM36 and RMK (Kaiapoi Railway River Bridge) bars tested in the present project. The Ahuriri bars were only threaded at one end with a head formed by a nut on a short length of thread similar to the Kaiapoi Railway River Bridge bars in their installed configuration.

The elongation in the threaded section of the Ahuriri bars was about 20% and the elongation in the shank about 3%. These values are a little lower than the corresponding values of 30% and 4% shown in Figure 18 for the RM36 bars but are very similar to the elongations shown in Figure 42 for the RMK bars. Further details and results from the tests carried out on the Ahuriri River Bridge bars are given in Appendix B.

# 7. Fracture Toughness

## 7.1 Steel Structures Standard NZS 3404: Part 1: 2009

Clause 2.6.3.1.2 of NZS 3404 requires the Design Service Temperature of steel structures to be the lowest one-day mean ambient temperature (LODMAT). In the selection of the Design Service Temperature an allowance is to be made for unusually cold conditions and Clause 2.6.3.2 states; "*For structures which may be subjected to especially low ambient temperatures, such as exposed bridges over inland rivers or structures located in alpine regions, a design service temperature 5 $^{o}$C less than the basic design temperature shall be used.*" The LODMAT isotherm for most inland regions (excluding alpine regions) in the South Island is -5 $^{o}$C and in the coastal regions 0 $^{o}$C.

Clause 2.6.4.3.2 of NZS 3404 requires members capable of sustaining structural displacement ductility demands sufficient to form plastic hinging into the strain hardening region under earthquake loading to be designed for a permissible service temperature 10 $^{o}$C greater than for members which comply with the fabrication and erection provisions of the Standard and with the provisions of AS/NZS 1554.1 *(Welding of Steel Structures)* or AS/NZS 1554.5 *(Welding of Steel Structures Subjected to High levels of Loading)* as appropriate. This requirement is equivalent to reducing the Design Service Temperature by -10 $^{o}$C.

Since it does not seem necessary to combine extreme low temperatures with design level earthquake loading the provisions of NZS 3404: 1997 can be interpreted as requiring linkage bars to be used in most of the inland regions of the South Island to have a minimum impact resistance of 27 joules at –15$^{o}$C (basic LODMAT isotherm of -5$^{o}$ lowered by 10$^{o}$ C to allow for high strain in linkage bars under earthquake loads and in most of the coastal South Island regions a resistance of 27 joules at –10$^{o}$C (Clifton, 2011).

## 7.2 Fracture Toughness of Steel Used in Test Assemblies

### *7.2.1 Reidbar*

Pacific Steel Group provided Charpy Test results for 20 mm diameter Reidbar (Grade 500E steel). These indicated a minimum impact resistance of 41 Joules at –20$^{o}$ C. The Reidbar Design Guide (2008 edition) when referring to Reidbar states; *".... All low temperature applications should be considered carefully, especially where impact loads are also present, even though Steel Reinforcing Materials, AS/NZS4671:2001 has no impact test requirement. Recent tests have shown values of Charpy impact resistance for Grade 500E RB32 at -15º C at around 17 joules. Grade500/7 SG Iron is not recommended for service at temperatures below freezing if impact loads are present."* The SG (spheroidal graphite) iron is presumably the material used in the cast nuts.

Although the information provided by Pacific Steel Group and Reid Construction Systems is inconsistent it appears that Reidbar and the proprietary nuts supplied with the bar are not suitable for cold temperature applications. There were fractures of nuts and cast fittings in the 4 September 2010 Darfield and 22 February 2011 Christchurch earthquakes, which occurred at temperatures well above freezing (Clifton, 2011).

### *7.2.2 Macalloy Bars*

Results of impact testing carried out on Macalloy bars have recently been reported by Opus International Consultants (Mandeno, 2010). The S650 Grade 316 bar had a minimum impact resistance of 70 Joules at –20$^{o}$ C. The Macalloy nuts are apparently manufactured from the same material as the bar so Macalloy linkage assemblies would be satisfactory in the coldest temperatures expected at South Island bridges located on the major routes. For comparison, Macalloy S1030 (high tensile stainless steel) had a minimum impact resistance of 3 Joules at -20$^{o}$ C and would not be suitable for linkage bars in cold temperatures. Even at 0$^{o}$ C the impact resistance was only 7 Joules and so this bar should not be used for linkages in New Zealand where a ductile performance is required.

#### *7.2.3 Grade 300 Mild Steel Bars*

Tables 2 and 5 of NZS 3404:Part 1:2009 indicate that the Grade 300 steel (AS/NZS 3679.1) used in the RM36 and TM30 galvanised mild steel assemblies would not be satisfactory at Service Temperatures less than 0º C. Clearly Grade 300 steel should not be used in the South Island and colder parts of the North Island. However, Grade 300 L15 steel has a minimum specified impact resistance of 27 Joules at -15$^{o}$ C and in practice is expected to be satisfactory in all but the coldest regions of the South Island.

Charpy impact tests were carried out on a 25.4 mm diameter galvanised mild steel bar removed from the Ahuriri River Bridge on SH8 in July 2008. The bridge is located on the LODMAT 5$^{o}$ C isotherm (see Figure 1 in NZS 3404:Part 1:2009). The impact resistance at -15$^{o}$C was 48 Joules and at -10$^{o}$C 65 Joules (mean values from three tests). These results indicated that bars fabricated from some mild steels could be satisfactory in cold regions. Further details of the bars, including tensile test results, are given in Appendix B.

Many older bridges in the South Island will have mild steel linkage bars so it would be useful to carry out further impact tests when bars are replaced as it is not clear at present whether there is a risk of unsatisfactory performance in earthquakes during cold weather.

#### *7.2.4 Stainless Steel Grade 316 Bars*

Because of the high Nickel content of Grade 316 stainless steel, it has very high impact resistance at low temperatures and linkage bars fabricated from this material will easily meet the NZS3404 requirements for the inland regions in the South Island. Technical information published by Sandvik (www.smt.sandvik.com) for a similar Grade 316 stainless steel to that used in the RS38, RS36 and TS30 linkage assemblies indicated an impact resistance of 60 Joules at - 196$^{o}$ C.

#### *7.2.5 Freyssibar*

The test certificate supplied with the 27 mm diameter Freyssibar used in the present project indicated an impact resistance of 49 Joules at -20 $^{o}$C so it appears suitable for use in the coldest regions of the South Island.

#### *7.2.6 High Tensile Bar SCM440 (AISI 4140)*

Charpy impact tests were carried out on the high tensile bar used in the TH30 assemblies. The average impact resistance values from three tests at each of -10 $^{o}$C and -20 $^{o}$C temperatures were 67 Joules and 74 Joules respectively. The corresponding lowest values from the three tests were 66 Joules and 72 Joules. These results indicate that the high tensile bar used in the tested assemblies would be suitable in the coldest regions in the South Island. However, this type of bar is supplied from many different sources and to different degrees of heat treatment and its impact resistance should be tested prior to application.

# 8. Design Issues

## 8.1 Durability

### *8.1.1 Ahuriri River and Kaiapoi Railway River Bridges*

Mild steel linkage bars recovered from both the Ahuriri River and Kaiapoi Railway River Bridge contained short sections where there was significant rust with the galvanising completely lost. In these areas pitting corrosion was present with pits about 1 mm deep. The appearance of the corroded areas is shown in Figures 51 and 52. On the corroded sections moisture and dirt from the deck joints had spilled onto the bars leading to galvanic action. The galvanising was in reasonable condition on the remaining lengths and on the washers and nuts. Because the bar shanks have a greater area than the threaded stress area, they are likely to perform satisfactory for a number of years. The Ahuriri River Bridge was constructed in 1965 and the Kaiapoi Railway River Bridge in 1970. Considering that the bars have been in service for more than 40 years they probably have at least a further 10 years of life.

### *8.1.2 Otaki River Bridge*

Several of the mild steel linkage bars recovered from the Otaki River Bridge contained short lengths of significant rust on the turned-down sections and on the threaded ends. The appearance of typical corroded areas is shown in Figure 53. Moisture and dirt from the deck joints would again have been a factor in causing the damage to the turned-down sections. Tight fitting nuts probably damaged the galvanising on the ends of the bars. There was no complete loss of galvanising on sections of thread that had not been subjected to nut tightening.

Loss of area on the turned-down sections of the Otaki River Bridge bars will obviously reduce the strength and ductility. However, the damage shown in Figure 53 appeared to be the worst damage on any of the 18 bars removed from the bridge and the loss of area on this section would have been less than 2%. The linkage bars were installed in 1991 or 1992 so they had been in service for about 20 years. The bridge is located 3.9 km from the coast and the marine environment would have been a factor in the relatively rapid deterioration of the bars. The life of galvanised bars on this bridge was less than the normally expected for bridge hardware.

### *8.1.3 Stainless Steel Linkage Bars*

Stainless steel linkage bars have only recently been used on New Zealand bridges. One of the earliest installations was on the Railway Avenue and Waterloo Road Overbridges on main arterial city streets in Lower Hutt. Macalloy Grade S650 (Grade 316 stainless steel) 30 mm diameter bars were retrofitted on these two bridges in 2005. There are no present issues with their durability but they were installed with stainless steel washers directly against galvanised steel brackets and are subjected to a marine environment so monitoring their long term durability performance would be informative.

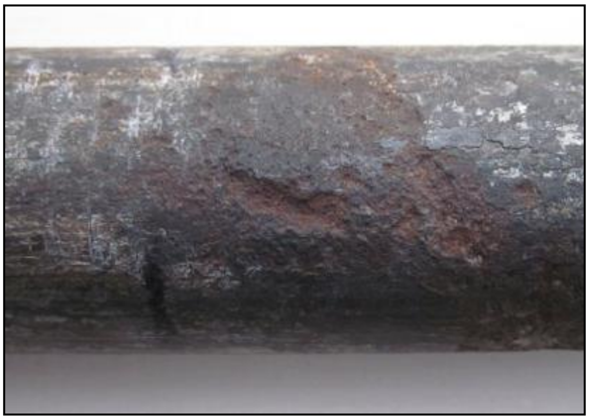


Figure 51. Corroded area on 25.4 mm diameter Linkage bars recovered from the Ahuriri River Bridge.

Figure 52. Corroded area on 38.1 mm diameter linkage bars recovered from Kaiapoi Railway River Bridge.

Figure 53. Corroded areas on linkage bars recovered from the Otaki River Bridge. The photo on the right shows an area on the 30 mm diameter turned-down section.

### 8.2 Bridge Manual Requirements

The Bridge Manual (BM) requires "tight linkages" to be used between spans where relative movement is not intended to occur under service loads or seismic loading. With regard to the design strength of tight linkages the BM states; *"...not less than the force induced therein under seismic conditions, nor less than that prescribed below for loose linkages."* Loose linkages are provided where relative horizontal movements are intended to occur. They are intended to act as a second line of defence when the design horizontal movements are exceeded. The strength of a loose linkage is required to resist a force equal to at least 0.2 times the dead load of the contributing length of superstructure.

The BM gives no guidance on how to calculate the forces in a tight linkage system and in the past for the design of smaller less important structures the minimum requirement of 0.2 times the dead load of the contributing length of the superstructure has been used or at least adopted as a reference to compare the results of any more detailed assessment. The contributing dead load might often be assumed to be the superstructure dead load reaction at the pier and abutment linkage location.

In the 2010 Darfield Earthquake two of the 38 mm diameter galvanised mild steel linkage bars at a pier adjacent to the northern abutment of the 148 m long six-span Kaiapoi Railway River Bridge failed. The total yield force capacity of the 24 bars at each pier was about 0.84 times the dead load superstructure reaction on the pier. Clearly for this bridge the forces in the linkages were much greater than given by the minimum requirement of the BM. High loads in the linkages in end spans resulted from very stiff abutments which carried most of the longitudinal load. In this case a detailed earthquake load analysis using a refined model of the bridge that includes the span bearings and all the substructure components is required to estimate the forces in the linkages.

### 8.3 Linkage Bar Design Forces for Long Multi-Span Bridges

The most rational method of determining design forces in longitudinal linkage systems connecting the superstructure to abutments and piers is to carry out a static push-over analysis modelling the stiffness of all the substructure components. The earthquake loads acting on the bridge should be estimated using the Bridge Manual design response spectrum (based on NZS 1170.5). Usually the analysis can be carried out using a spreadsheet approach with force versus displacements functions developed for the abutments and piers from published information on the passive soil resistance against abutment walls and the displacement response of pile foundations. The individual force displacement functions can be summed to give the overall stiffness function for the bridge and the fraction of the bridge inertia force transferred to each of the substructure components. These forces provide the design loads for the various linkage systems at the connections to the abutments and piers.

The SH2 Tauherenikau River Bridge is a typical two-lane state highway bridge which is currently being investigated for linkage improvements and is used here to illustrate the recommended analysis and design method for linkage systems. The bridge is a concrete structure with the superstructure formed from simply-supported precast prestressed double U beams with a cast insitu deck spanning between hammer-head type single circular column piers founded on single 1.39 m diameter bored piles. The bridge has eight equal spans of 12.7 m and two end spans of 12.5 m giving an overall length of 126.9 m. The overall deck width is 9.25 m. The general layout of the bridge is shown in Figures 54 and 55.

The reinforced concrete sill beam type abutments are founded on two 1.39 m diameter bored piles of similar detail to those at piers. At both the abutments and piers the piles have 9.5 mm thick steel casings extended to the underside of the sill beam at the abutments and to a typical height of 1.5 m above ground level at the piers.

At both the abutments and the piers the ends of the spans rest on 203 mm wide by 12.7 mm thick continuous neoprene strips. The ends of the spans are held down with eight pairs of 12.7 mm diameter dowels located between the beam units and sheathed with rubber over part of their embedment length into the pier caps and abutment seating. The ends of the spans are linked to the abutments and over the piers with eighteen 15.9 mm diameter galvanized mild steel bars passing through solid diaphragms at the ends of the beam units and are anchored within the hollow cells of the units. The bars are terminated with standard nuts, 38 mm square x 6.4 mm thick steel washers and 70 mm square x 12.7 mm thick rubber pads.

For the purpose of the illustrative calculations the column height of the 1.14 m diameter columns from the underside of the hammer-head pier caps to the top of the foundation pile was taken as the 4.88 m dimension shown on the drawings for all piers. As-constructed this height varied between about 1.3 m and 3.0 m.

The main steps and assumptions made in the application of the recommended analysis method are summarised below. A summary of the bridge details, assumed design parameters and calculated stiffness factors is given in Table 8.

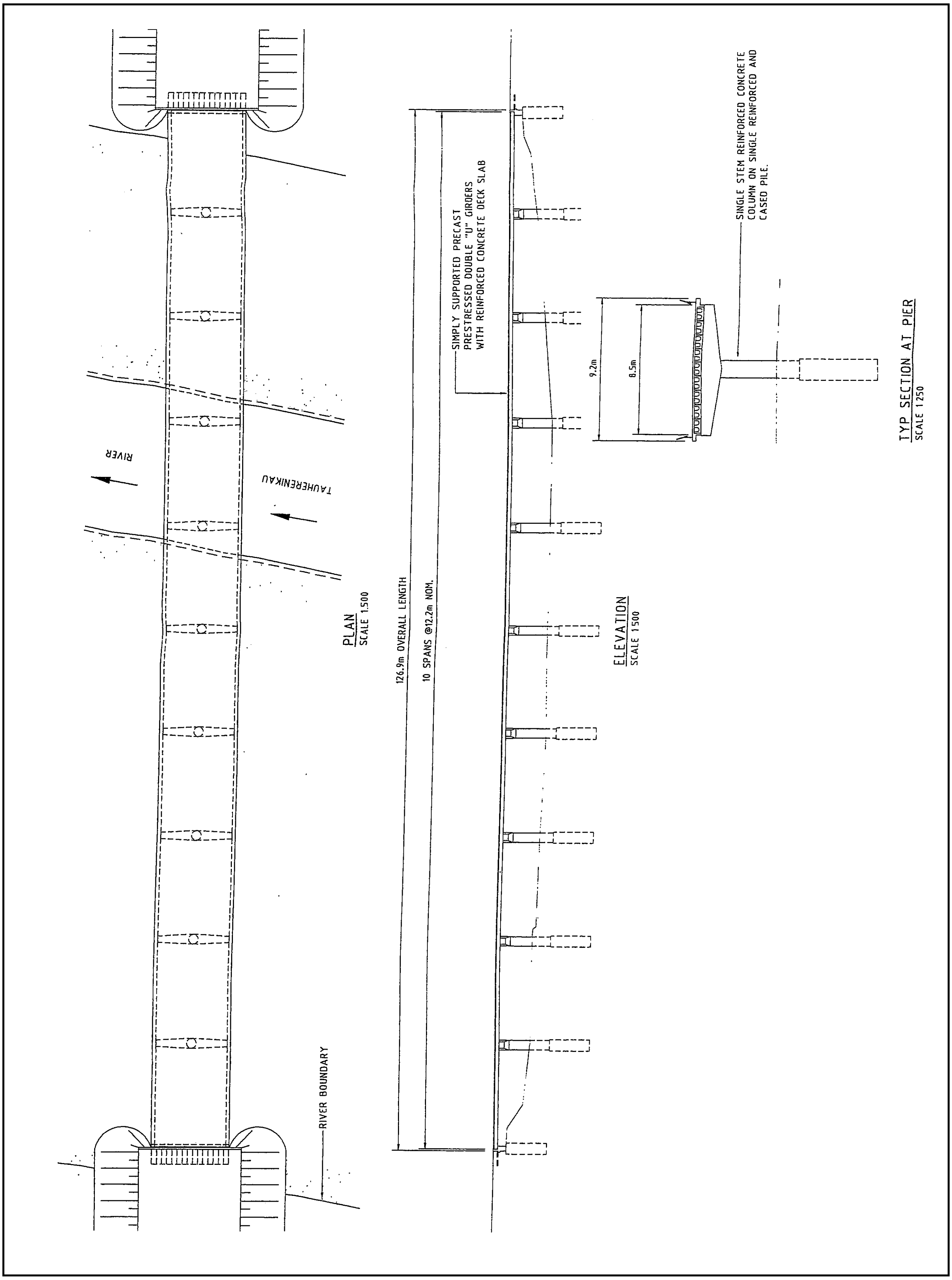



Figure 54. Tauherenikau River Bridge.

Figure 55. Tauherenikau River Bridge looking towards the east end.

### *Earthquake Loads*

The design earthquake for the linkage system was assumed to be the 1000-year return period event. (This level is commonly used for retrofitting bridges on the main SH's). Geotechnical investigation previously carried out at the site indicated a deep soil site with a Class D Site Subsoil Category (NZS 1170.5 classification). The river bed soils were described in the investigation report as medium dense to very dense bedded sandy gravels, gravels and cobbles and occasional interbedded silty sand lenses and minor to trace amounts of clay and silt. The abutment backfill material was described as medium-dense sandy gravels with traces of silt.

The NZS 1170.5 Hazard Factor, $Z$, at the site is 0.43 and the Structural Performance Factor, $S_p$, was taken as the 0.67 value specified in the Bridge Manual for soil Class D. In the longitudinal direction the response of the bridge was found to be essentially elastic so no ductility reduction was used to derive the earthquake demand spectrum. The $S_p$ reduction was considered to satisfactorily allow for damping from minor nonlinear interaction in the pier and abutment foundations. The design acceleration versus displacement demand spectrum shown in Figure 58 was calculated from the NZS 1170.5 spectral shape factor $Ch\ (T)$ using the usual assumptions. The design spectrum has spectral acceleration plateau value (periods between 0 to 0.5 seconds) of 1.12 g resulting in high loads on the substructure from the superstructure inertia forces.

The total longitudinal earthquake load was based on the mass of the superstructure (including superimposed mass) plus the mass of the nine pier caps. This gave a total dynamic weight of 12,800 kN and a longitudinal inertia force of 14,300 kN at the response acceleration of 1.12 g.

## Table 8. Linkage Analysis - Tauherenikau River Bridge

| Parameter | Value | Comment |
|---|---|---|
| **Bridge Geometry, Weights and Capacities** | | |
| Span length - typical | 12.7 m | End spans are 12.5 m |
| Span weight | 1090 kN | Includes superimposed dead load |
| Pier cap weight | 211 kN | |
| No of spans | 10 | |
| Pier height | 6.1 m | Top of pier cap to top of pile. Design value – not “as-constructed”. |
| Pier column diameter | 1.14 m | |
| Pier column reinforcing | 22 x 32 dia | 1 ¼ inch deformed bars |
| Pier column hoops | 12.7 dia | Spaced at 305 mm |
| Pile outside diameter | 1.39 m | |
| Concrete strength | 32 MPa | Probable strength |
| Reinforcement yield strength | 300 MPa | Probable strength based on specified strength of 275 MPa |
| Pile casing thickness | 9.5 mm | $^{3}/_{8}$ inch plate. No reduction in casing area for stiffness calculations |
| Column flexural capacity | 2900 kN m | No strength reduction |
| Column shear capacity | 1700 kN | No strength reduction. Uniaxial displacement ductility = 3 |
| Column cracked stiffness | 0.4 Ig | Reduction to gross second moment of area. |
| **Earthquake Analysis Parameters** | | |
| Hazard Factor, Z | 0.43 | NZS 1170.5 |
| Return Period Factor | 1.3 | NZS 1170.5 1000 year return period |
| Spectral Shape Factor | 3.0 | Plateau value for Site Subsoil Class D |
| Structural Performance Factor | 0.67 | Bridge Manual for Site Subsoil Class D |
| Design spectral acceleration | 1.12 | Plateau value (periods less than 0.5 seconds) |
| **Abutment Backfill Soil Properties** | | |
| Friction angle | $36^{o}$ | |
| Cohesion | 0 | |
| Soil unit weight | 18 kN/m$^{3}$ | |
| Strain at 50% ultimate stress | 0.004 | |
| Soil gap at abutments | 10 mm | Gap assumed developed by cyclic loading |
| **Pile Foundation Soil and Stiffness Properties** | | |
| Soil unit weight | 18 kN/m$^{3}$ | |
| Young’s modulus coefficient | 30 kPa/m | Medium-dense gravels below water table. Linear increase with depth. |
| Stiffness of abutment pile | 108 MN/m | Single pile measured at beam seating level. |
| Stiffness of pier - elastic | 11.4 MN/m | Stiffness of pile and column acting in series at beam seating level. |
| Stiffness of pier - post yield | 3.1 MN/m | Post-yield stiffness factor of column taken as 0.15. |
| **Beam Bearing Details and Stiffness** | | |
| Bearing pad width x length | 203 x 8230mm | |
| Bearing pad length | 8.23 m | |
| Bearing pad thickness | 12.7 mm | |
| Bearing pad shear modulus | 1.1 MN/m | Rubber: assumed IRDH = 60. Single pad stiffness. (Twin pads on piers.) |
| Coefficient of friction | 0.7 | Typical for dry concrete on rubber |
| Shrinkage gap at abutments | 5 mm | Assumed joint gap at abutments from creep and shrinkage |
| **Linkage Bar Details** | | |
| Effective length at abutments | 690 mm | |
| Axial stiffness – retrofitted bars | 300 MN/m | 8 x 32 dia Macalloy bars; 200 mm sq x 20 mm rubber washer IRHD = 60 |

### *Pier Stiffness*

The stiffness of the piers (assumed to be identical for each pier) was calculated using the *Elastic Continuum* method of modelling soil-pile interaction as described by Pender (1993). In this analysis procedure, closed-form analytical expressions are used to calculate the rotation and lateral displacement at the top of a pile for a horizontal load applied at a distance above ground level. The full stiffening effect of the steel casing was assumed and the concrete within the casing was taken be uncracked. The soil Young's Modulus was assumed to increase linearly from zero at the ground surface using a rate of increase coefficient m = 30 MPa per m. This m value is typical for dense sandy gravels below the water table. (There is usually considerable uncertainty in the numerical value for the modulus increase and in design the sensitivity of the results should be investigated over a range of m values.) The displacement at deck level from the pile ground level deformations and the deflection in the column were obtained by conventional beam analysis using the section properties for cracked concrete. For the column a bi-linear force displacement relationship was assumed with a post-yield stiffness reduction factor of 0.15. A more exact moment/curvature relationship could be computed for the column but this was unnecessary for the present example as the columns did not reach their flexural capacity under the 1000 year return period longitudinal loading.

Because the piers are effectively pinned at their tops the calculation of their stiffness is straightforward. If the piers are framed into the superstructure the analysis is more complicated and it may be necessary to carry out a simple frame analysis of a single pier representing the pile by the equivalent cantilever concept described by Pender. Inelastic behaviour of the soil is considered by Pender but was not used in the present analysis as the results were not sensitive to the soil assumptions for the pier piles.

A correction was made for the shear stiffness of the neoprene strip beam seatings but they are very stiff and did not appreciably alter the calculated overall stiffness of the piers. Any stiffening effect of the hold-down dowel bars was neglected.

### *Abutment Stiffness: Movement Outwards From Backfill*

The spring model shown in Figure 56 was used to evaluate a force displacement relationship for the abutment moving outwards from the retained fill ("pulled" abutment). A linkage tension spring and a bearing shear spring are assumed to act in parallel with a pile spring acting in series with this pair. The linkage was assumed to consist of bars with rubber washers under nuts and steel washers at both ends. The linkage system was assumed to be a new retrofitted system which would be stiffer than the existing linkage bar system. An overall stiffness for this system was calculated from the compression modulus for rubber and the conventional axial load stiffness for steel bar (*Young's Modulus x Area/Length*). The bearing stiffness (*Shear Modulus x Area /Thickness*) was calculated from the shear modulus for rubber (assumed to have an IRHD value of 60 for this example) with the bearing force limited to the slip load between rubber and the concrete beams. The slip load was estimated using a coefficient of friction between the rubber and concrete of 0.7. The stiffness of the abutment piles was calculated using the procedure described above for the pier piles.

The influence of the existing hold-down dowels was neglected. Initially they would stiffen the bearings but would fail soon after bearing slip commenced and therefore would not be effective except in the initial loading cycles.

For the case of outward movement of the abutment the abutment backwall is loaded with an active earthquake pressure increment. This pressure is in phase with the ground motion but is not necessarily in phase with the dynamic response of the bridge. Conservatively it can be taken to be in phase with the bridge inertia loads acting on the abutment. In the present example the outward movement of the abutment from the pressure increment acting alone was estimated to be about 0.7 mm for the 1000 year return period peak ground acceleration. Because of the small

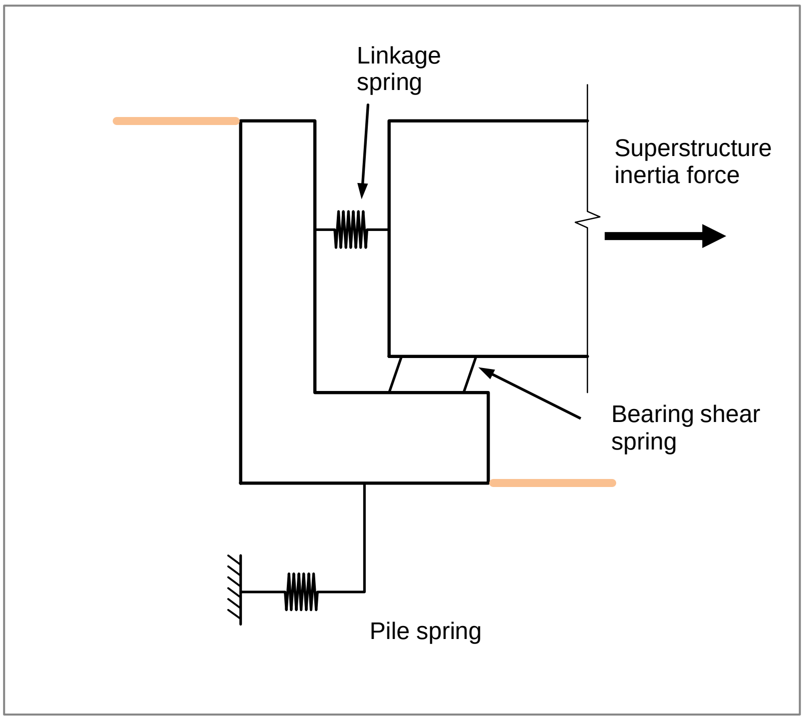



Figure 56. Abutment model for superstructure inertia load forces pulling abutment away from backfill.

magnitude of this displacement it was neglected in the assessment of the total loads transferred to the abutment from the bridge superstructure. For bridges with back walls significantly higher than the 1.43 m of the Tauherenikau River Bridge the earthquake pressure increment displacement of the abutment may need to be considered. It effectively reduces the stiffness of the abutment, reducing the component of load carried by the abutment.

### *Abutment Stiffness: Movement Towards Backfill*

The spring model shown in Figure 57 was used to evaluate a force displacement relationship for the abutment moving towards the retained fill. A bearing shear spring was assumed to act in series with the pile and passive soil pressure springs acting in parallel. As for the case of the abutment moving outwards from the backfill, the bearing stiffness was calculated from the shear modulus for rubber with the bearing force limited to the slip load between rubber and the concrete beams with the stiffening of the hold-down dowels neglected. For the present example, a clearance gap of 5 mm was assumed between the superstructure and the restraining shear key (or backwall). Initially the superstructure displacement is restrained by the bearings until slip occurs at a displacement of about 3 mm. The clearance gap then closes and the restraint from the piles becomes effective. It was assumed that cyclic loading had opened a gap of 10 mm between the backwall and the backfill. The soil passive pressure does not become effective until this gap closes.

The stiffness of the abutment piles was calculated using the procedure described above for the pier piles. The passive soil resistance was calculated using the hyperbolic force displacement relationship published by Kahalili-Tehrani et al (2010). This relationship has the following form:

$$F(y) = \frac{a_r\, y}{H + b_r\, y}\, H^n \qquad (1)$$

Where $F$ and $y$ denote the lateral force per unit width (kN/m) of the backwall and the lateral deflection (cm). $a_r$ and $b_r$ are coefficients calibrated by wall experiments and $H$ is the wall height (m).

To use the above expression for passive resistance it is convenient to select a displacement then compute the force. This same procedure can be conveniently used to calculate the total forces acting on both abutments and the piers.

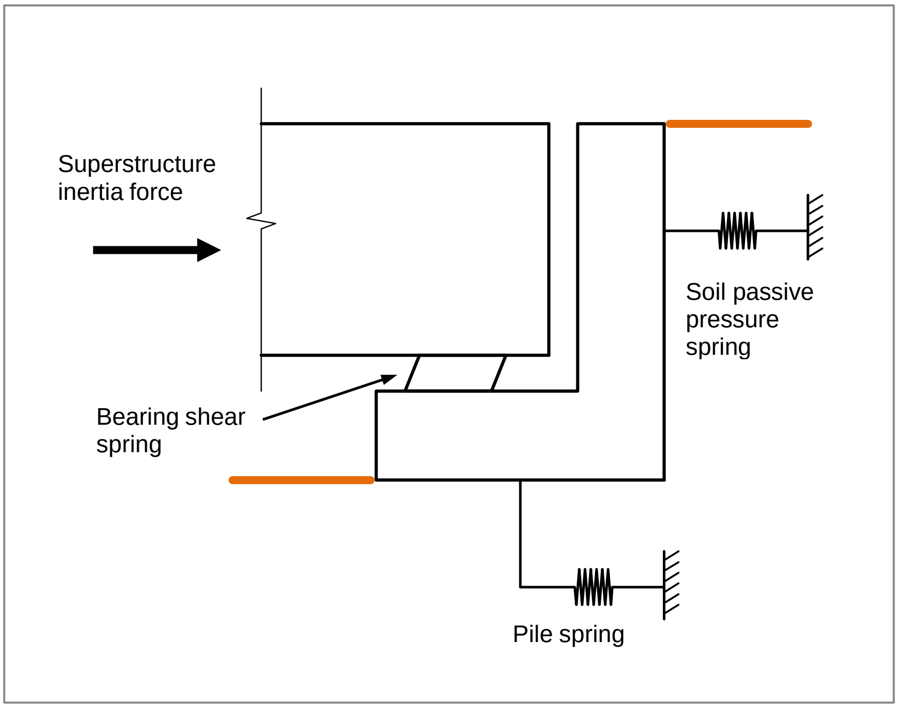



Figure 57. Abutment model for superstructure inertia load forces pushing abutment against backfill.

### *Results of Analysis*

Force displacement relationships for both abutments, the sum of the piers, and the sum of all the substructure components (total response curve) are shown in Figure 58 together with the 1000-year return period acceleration versus displacement spectrum. The total response curve intersects the demand spectrum at a displacement of 31 mm and at a response acceleration of 1.12 g (plateau response). At this point the percentages of the total longitudinal inertia load carried on the "pulled" abutment, "pushed" abutment and the sum of the piers are 28%, 51% and 21% respectively. The equivalent period of vibration is 0.33 seconds.

The 1000 year return period linkage force at the "pulled" abutment was 4050 kN. The combined capacity of the existing linkage bolts (yield capacity), hold-downs and bearing friction is about 1800 kN. To obtain full benefit of the abutment pile capacity it is therefore necessary to improve the linkage system. Suitable linkage could be provided by eight 32 mm diameter Macalloy S650 bars which would provide a minimum unreduced yield capacity of 4050 kN. The existing system should be left in place to provide a reserve strength to counter any underestimation of the forces or under-capacity in the retrofitted bar materials.

The 1000-year return period linkage force at the "pushed" abutment was 6900 kN. (An additional 380 kN is transferred through the bearing.) The combined capacity of the existing backwall (shear capacity), hold-downs and bearing friction is about 3200 kN. Again to obtain full benefit from the abutment pile capacity it is necessary to retrofit shear keys to transfer the superstructure load to the abutment piles.

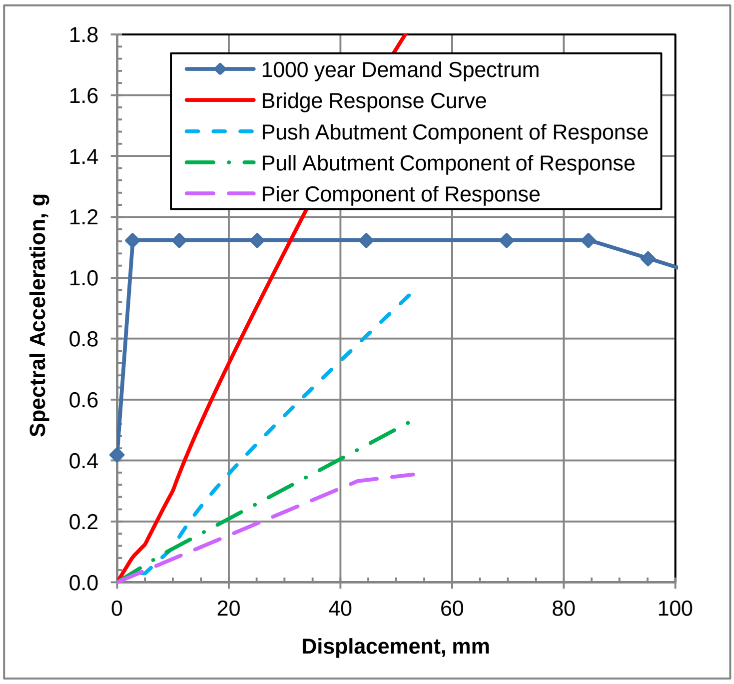



Figure 58. Bridge response curves and 1000 year demand spectrum.

Because the abutment seating beams are narrower than the piles the flexural/shear capacity of the connection between the two may be inadequate to resist the estimated superstructure load on the "pushed" abutment. The joint would need to be checked and strengthened if necessary.

The loads that need to be transferred between the spans through the linkage system at each pier are summarised in Table 9. Positive loads indicate that the linkage is in tension and negative loads indicate compression. At each pier a load of about 340 kN is required to displace the pier without slip occurring between the pier and the superstructure. This force can be transferred through the bearings and hold-down bolts or by the anchor brackets used in the linkage system if they are set with small gaps to the pier faces.

**Table 9. Linkage Loads at Each Pier**

| Abutment or Pier | **Linkage Force, kN** |
|---|---|
| Pull Abutment | 4050 |
| Pier B | 2960 |
| Pier C | 1860 |
| Pier D | 770 |
| Pier E | -330 |
| Pier F | -1420 |
| Pier G | -2520 |
| Pier H | -3620 |
| Pier I | -4710 |
| Pier J | -5810 |
| Push Abutment | -7240 |

The drawings show cast insitu diaphragms at the ends of the spans with a 4.8 mm thickness of hardboard separating the adjacent diaphragms and a 19 mm square chase in the deck filled with sealant. Because of shrinkage and creep a gap of about 3 mm is likely between the adjacent diaphragms. This gap could be greater if the hardboard has decayed. Since a bar linkage system does not usually act in compression, at the piers with significant linkage compression forces (Piers F to J) any joint gap needs to close to transfer the force. Alternatively compression can be transferred by linkage brackets with small gaps to the pier faces. Regardless of the method used to transfer the compression, slippage on the bearings of some of the piers is likely, resulting in the tops of the piers displacing different amounts. The analysis method described above assumes uniform displacement of the superstructure along the length of the bridge, so if slippage occurs at the joints the method does not provide accurate estimates of the linkage forces with greater force distributed to the "pull" abutment and to the piers where slip does not occur. However, if the joint gaps are small or the linkage brackets are set with small gaps to the pier faces the analysis methods gives satisfactory results. If large slip displacements occur at the bearing contacts then a more detailed analysis is necessary to accurately estimate the linkage forces but because there are many uncertainties in predicting the slip behaviour it is probably best to make adjustments based on judgement to the results of a simple analysis similar to the one described above.

The results shown in Table 9 indicate that the linkage system over the piers needs to be designed to carry a tension force of at least 3000 kN and a compression force of at least 5800 kN. The combined capacity of the existing linkage bolts, hold-down dowels and bearing friction for transferring the linkage tension force is 1800 kN which is not adequate for the 1000-year return period event. In this bridge the compression linkage force can be carried by closing of the joint gaps but if the gaps are large then the linkage system anchor brackets may need to be designed to transfer the compression force across the top of the pier. In theory, the existing linkages near the centre of the bridge are satisfactory but if retrofitting is carried out it would be good practise to provide a system with equal capacity at each of the piers.

A minimum requirement for the connection system at the top of the piers is that it should be sufficient to develop the strength capacity of the pier. The horizontal load necessary at the top of the pier caps required to develop the flexural capacity of the columns (plastic hinge at base of the column) is about 480 kN. Applying an overstrength factor of 1.5 to this value gives a load of 720 kN. The combined capacity of the existing hold-down bolts and bearing friction can produce the force required to develop plastic hinging at the base of the columns. The minimum requirement is obviously not critical for this bridge.

The calculated 3000 kN capacity required for the linkage system over the piers is about the weight of each span. This illustrates that simple rule-of-thumb linkage force estimates based on the weight of the contributing mass reacting on each pier are not adequate for long multi-span bridges.

### 8.4 Linkage Bar Design Forces in Short Bridges

For short bridges of up to three spans and where the abutments are clearly a lot stiffer under longitudinal horizontal loads than the piers it would be reasonable to design the linkages at the abutments to carry the total superstructure longitudinal inertia load. The inertia load should be computed using the design response spectrum as outlined above for long multi-span bridges. Unless more detailed stiffness calculations are carried out, 50% and 70% of the inertia force should be assumed to be carried on the "pull" and "push" abutments respectively. The connections between the superstructure and piers (if any) should be designed to have a minimum strength sufficient to develop the capacity of the pier increased by an overstrength factor of 1.5.

### 8.5 Linkage Bars at In-Span Movement Joints

A number of long bridges on state highways have been designed with in-span movement joints. Typically these joints are at ¼ span points near the centre of the bridge (for example SH1, Ohau River Bridge). In a number of very long bridges several movement joints were installed (for example SH1 Otaki River Bridge). Most of these joints in bridges designed in the 1950's and

1960's were designed without positive longitudinal restraint. In some cases a vertical articulated hanger was provided to transfer live load shear from the longer span section to the shorter section. Under large gap openings this would provide a component of some longitudinal restraint but the designs did not consider this action. Typical joint gaps are not sufficient to prevent pounding as the joint closes under longitudinal earthquake actions.

For both retrofitting movement joints in older bridges and designing movement joints in new bridges it is important to be able to estimate under earthquake loading the relative displacements at movement joints and the forces in linkage systems or "restrainers" installed at the joints. Two method of calculating these parameters are presented below.

***Priestley et al (1996) Method***

Relative displacement results from numerical analyses on typical bridge models are presented by Priestley et al (1996). Results of their analyses are reproduced in Figure 59. This information is useful for design but has limitations including:

- Details of the models are not fully described. The main parameter used to present their results is the relative stiffness between the frames either side of the joint – the Stiffness Ratio plotted in the Priestley et all Figures shown in Figure 59. (If the frames either side of the joint have the same stiffness and mass, in theory there is no relative displacement.)
- Apparently the frames either side of the movement joint were assumed to have equal mass.
- The results presented are for "moderate" seismic intensity which is not defined.
- Details of the restrainers and bearings assumed at the joint in the model are not presented. Stiff restrainers can influence the joint relative movements.
- Absolute stiffness and mass values for the models are not presented but these determine the periods of vibration and hence the displacement response.

Based on the results of their numerical model studies Priestley et al state that a simple method of predicting the relative longitudinal displacement at the movement joint with reasonable accuracy is found from the difference between the absolute magnitude of peak longitudinal displacements calculated for the two frames separated by the joint, where each frame was considered as a stand-alone element and the absolute magnitudes of peak displacement have the same sign (see Priestley et al Fig 5.91 (b) reproduced in Figure 59). To estimate design relative displacements they state that relative displacement calculated by this method should have components added for transverse response and travelling wave effects. They suggest the following equation for a conservative estimate in regions of high seismicity:

$$N_E = \Delta_L + 0.015W + 0.001L \qquad (2)$$

Where $\Delta_L$ is the longitudinal relative displacement calculated as described above, $W$ is the width of the bridge seating in the transverse direction at the movement joint, and $L$ is the average distance to the adjacent movement joints. Priestley et al indicate that the above equation gives "impossibly" large displacements for short bridges. However, short bridges are unlikely to have movement joints of the type being considered in this report.

Priestley et al Fig 5.94 reproduced in Figure 59 shows that the restrainer stiffness does not have a strong influence on the relative displacement at a movement joint. They do not define the "standard stiffness" referred to in the figure but state that restrainers are relatively ineffective at reducing the relative displacement unless the restrainer stiffness is at least as large as the stiffness of the more flexible of the two frames connected across the joint. They recommend designing restrainers by either one of two methods. The first of these is to use dynamic time-history analysis where the strength and stiffness of the restrainers is varied until an acceptable result is achieved.

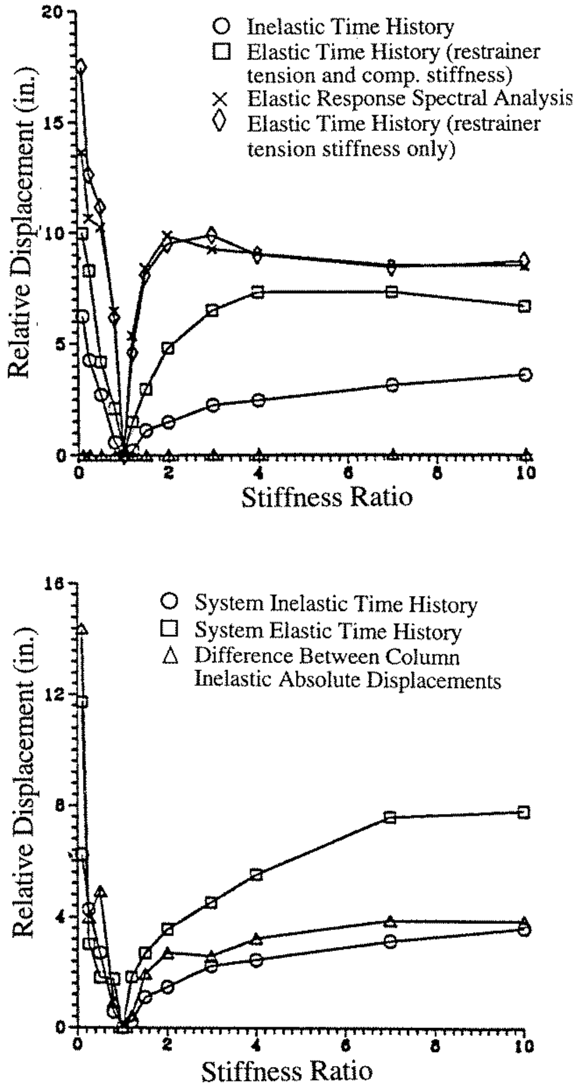



**FIG. 5.91** Relative longitudinal displacements between two frames connected across a movement joint. (*a*) Relative movements predicted by different analytical approaches; (*b*) Comparison between relative movements predicted from nonlinear time-history analysis and difference between absolute displacement of independent adjacent frames.

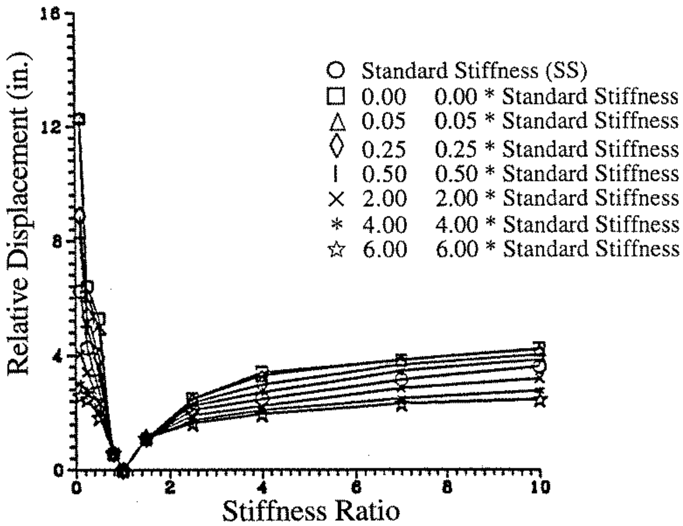



**FIG. 5.94** Influence of restrainer stiffness on movement joint opening (1 in.

Figure 59. Relative joint movements from computer model studies. Reproduced from Priestly et al (1996).

Their second approach is based on their observation that connected frames in the analysis model respond essentially in-phase during maximum response. As a consequence, the maximum tensile force transfer between frames should be equal to the difference between the frame overstrength shear capacities. Thus the maximum restrainer design force is given by:

$$F_R = V^o_{F1} - V^o_{F2} \qquad (3)$$

Where $V^o_{F1}$ and $V^o_{F2}$ are the overstrength capacities, found from summing the overstrength capacities of all columns in each frame.

### *Elastic Dynamic Analysis of Two-Degree-of-Freedom Spring Mass System*

Figure 60 shows a two-degrees-of-freedom dynamic system with two masses restrained by end springs and interconnected by a spring between them. This is essentially a simplified version of the bridge model investigated by Priesley et al with end springs $k_1$ and $k_2$ and the two masses $m_1$ and $m_2$ representing the stiffness and masses of the frames either side of a movement joint. The centre spring of stiffness $k_c$ represents the linkage system or "restrainer" and the relative displacement between the two masses gives the relative displacement across the movement joint. On the assumption of elastic behaviour this two-degrees-of-freedom system can be solved analytically to give closed form solutions for the displacements of the masses under earthquake loading. The main advantages that this approach offers over using the Priestley et al results are:

- A comprehensive set of joint movements can be calculated for a wide range of the basic parameters with minimum computational effort.
- The linkage force is readily calculated as part of the solution.
- Results can be obtained for any earthquake input intensity.
- The masses can be varied independently of the frame stiffnesses.

The main limitations of the method are:

- It assumes elastic response (likely to be conservative for inelastic response). However it would be possible to extend the method to consider inelastic action.
- The restrainer is assumed to have equal stiffness in compression and tension.
- Two modes of vibration are generated (masses in phase and out of phase) and to get the total relative displacement between the masses the responses in the two modes have to be combined by an approximate method.

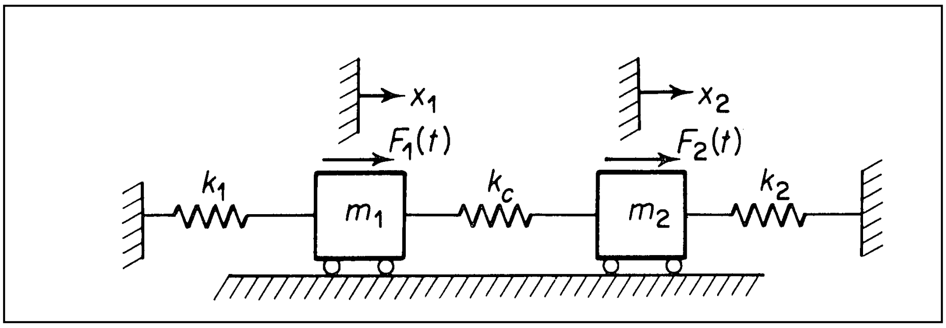



Figure 60. Two-degrees-of-freedom model. From Thompson (1965).

In the present investigation the displacement in the two modes of vibration were combined using the widely accepted square-root-of-the-sum-of-the-squares method (SRSS). It was also assumed that the earthquake response in the second mode was a linear function of the ratio of the first and second mode periods. This is a reasonable assumption because earthquake displacement response spectra are reasonably linear over the period range of interest for most bridges.

Closed form analytical solutions given in Thompson (1965) for the natural frequencies and mode shapes of the two-degrees-of-freedom model were adopted for the present study. Modal participation factors and the corresponding displacement response functions for earthquake inputs using the NZS 1170.5 elastic response spectrum (inelastic spectra could be used when inelastic response is significant) were calculated from the frequencies and mode shapes. Stiffness and mass ratios were varied over typical ranges for bridge structures to give graphical results in terms of the ratios of the two end spring stiffnesses (frame stiffness ratio), restrainer to end spring stiffness and the two masses.

Joint relative displacement solutions from the two-degrees-of-freedom model for a range of bridge and restrainer stiffness ratios are shown in Figure 61. The parameter c in the legend is the restrainer stiffness ratio defined by:

$$c = k_c / k_1 \qquad (4)$$

The displacement is plotted in dimensionless form. The absolute displacement, *D,* is obtained from:

$$D = D_d \text{ x } (d_R, \textit{the fist mode displacement response}) \qquad (5)$$

Where $D_d$ is the dimensionless displacement plotted in Figure 61.

The force in the restrainer is given by:

$$F = D\, k_c \qquad (6)$$

The first mode displacement response is calculated from the site hazard displacement response spectrum using the first mode period $T_1$ of the two-degrees-of-freedom model. The first mode period is calculated from the dimensionless period, $T_d$, plotted in Figure 62 by:

$$T_1 = T_d\, 2\pi \sqrt{\frac{m_1}{k_1}} \qquad (7)$$

Figure 63 shows the relative joint displacement plotted against the restrainer stiffness ratio, $c = k_c / k_1$, for typical structure stiffness and mass ratios, $k_2/k_1$ and $m_2/m_1$. A corresponding set of curves for the dimensionless restrainer force is shown in Figure 62. The absolute restrainer force F is obtained from Figure 64 by:

$$F = F_d\, d_R\, k_1 \qquad (8)$$

Alternatively the restrainer force can be determined directly from the relative joint displacement and the restrainer stiffness. However, Figure 63 illustrates how the restrainer force varies as a function of the restrainer stiffness ratio and the structure stiffness and mass ratios. The variation of the dimensionless first mode period of the two-degrees-of-freedom model with restrainer stiffness ratio is shown in Figure 65.

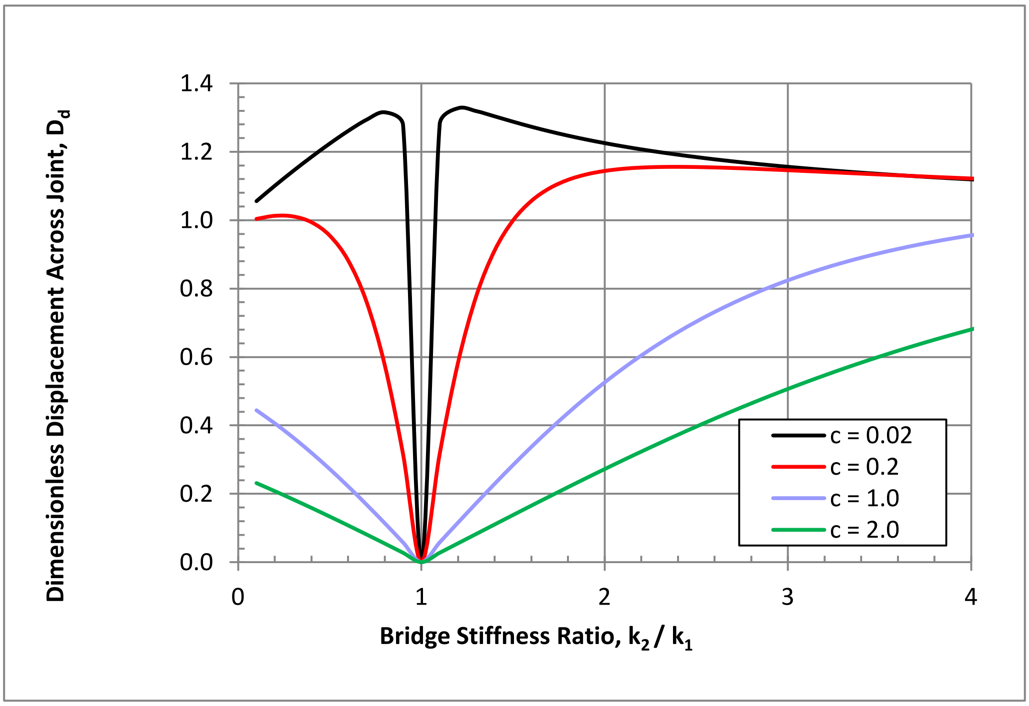



Figure 61. Joint dimensionless relative displacement from two-degrees-of-freedom model.

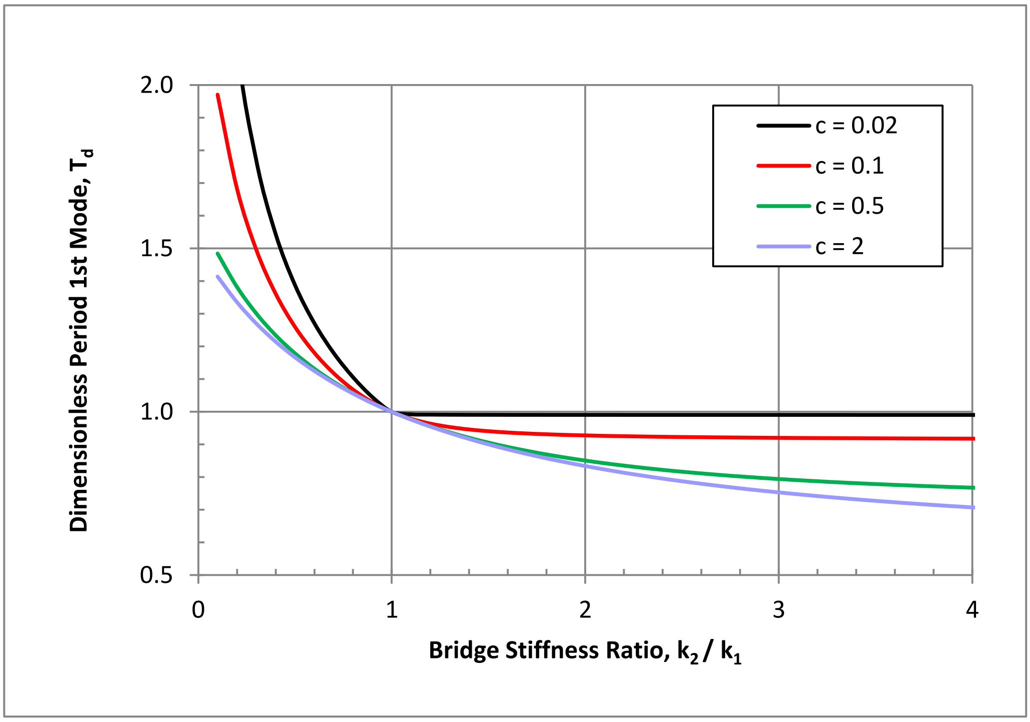



Figure 62. First mode period of vibration of two-degrees-of-freedom model.

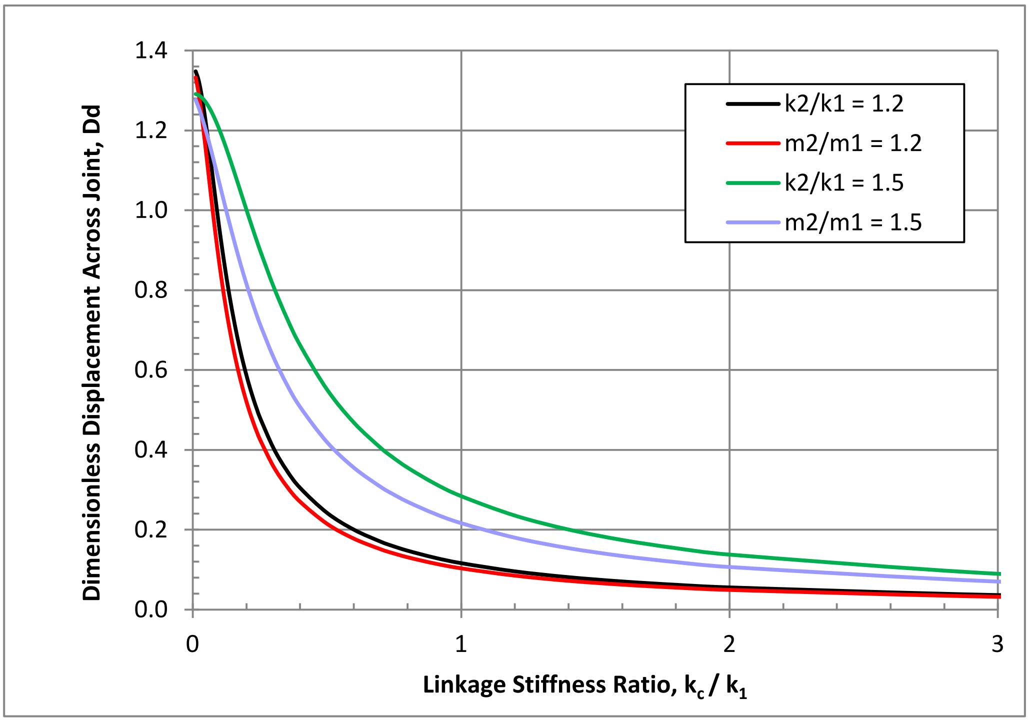



Figure 63. Joint dimensionless displacement from two-degrees-of-freedom model.

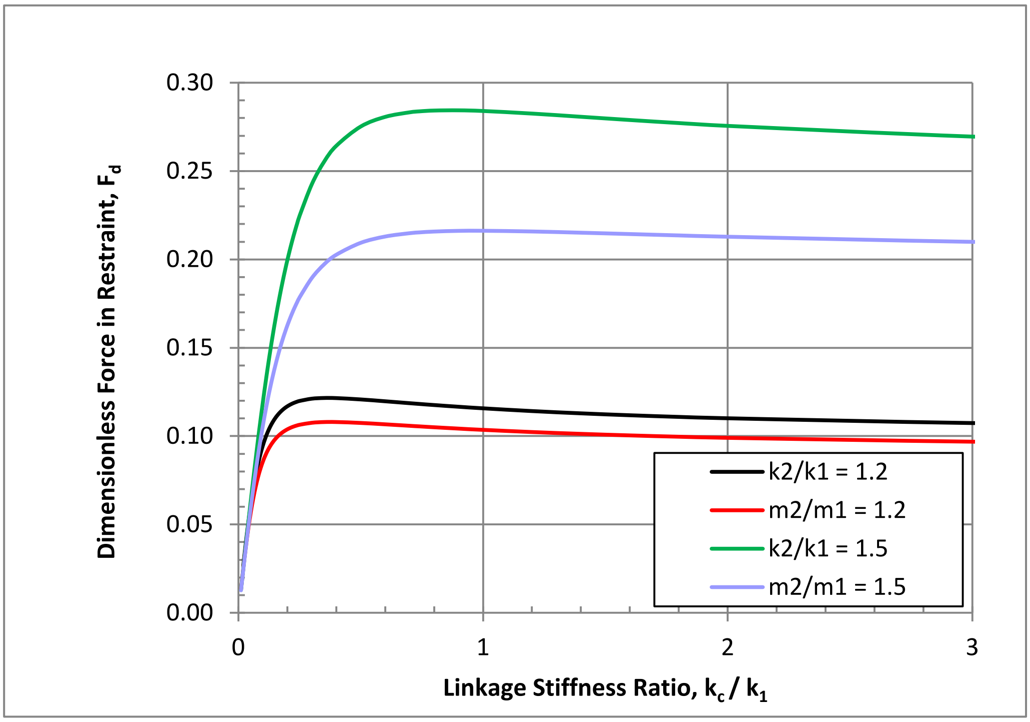



Figure 64. Restrainer dimensionless force from two-degrees-of-freedom model.

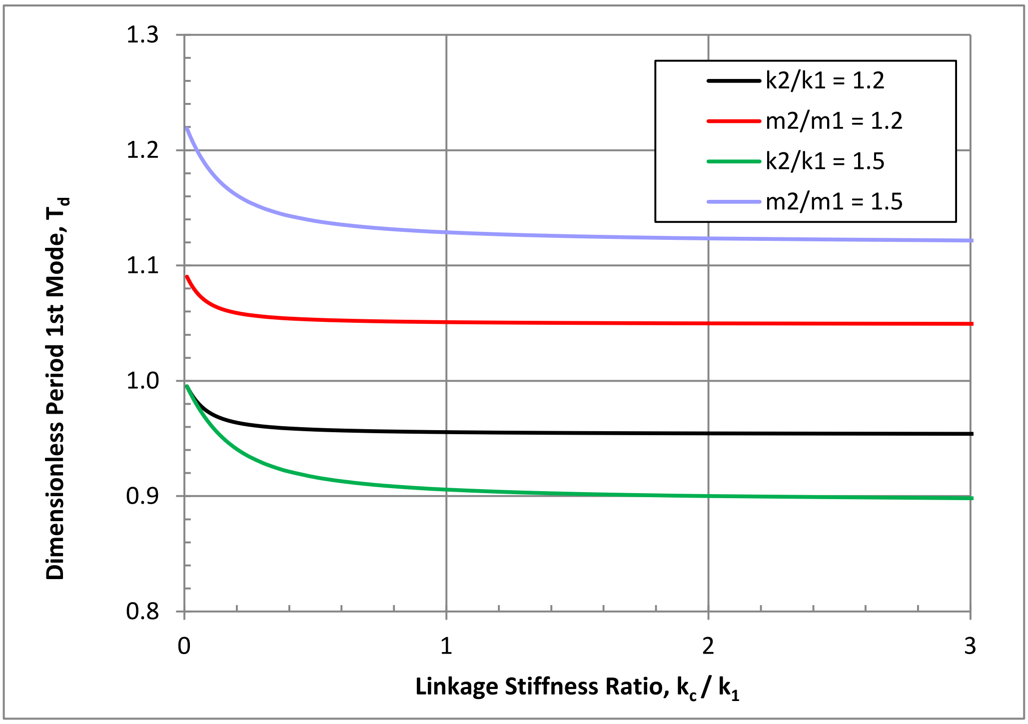



Figure 65. First mode period of vibration of two-degrees-of-freedom model.

### *Otaki River Bridge*

The application of the two-degrees-of-freedom model for estimating joint relative movements and restrainer forces is illustrated using the structural properties of the Otaki River Bridge, which has three in-span hinges as shown in Figures 66 and 67. To simplify the analysis only the southernmost section of seven spans, with a single hinge joint between the first section of three spans and the second section of four spans, was considered. Details of the bridge and the analysis results are summarised in Table 10. Two solutions are given in the table; a solution based on assuming the mass ratio is 1.0 and the second for the more correct ratio of 0.75. (The charts given above do not give an exact solution for the case when both the structure and stiffness ratios vary from 1.0.)

The displacement response spectrum used in the analysis was based on NZS 1170.5 for a 1000-year return period event assuming Site Soil Category D and a $S_p$ factor of 0.67. The site design displacement spectrum is shown in Figure 68.

The total minimum yield capacity of the six Macalloy 36 mm diameter S650 bars recently retrofitted to the bridge was 3800 kN. The total minimum yield capacity of the six mild steel bars with a turned-down diameter of 30 mm originally fitted to the bridge (installed in 1991) was 1080 kN which is less than the calculated design requirement of 1800 kN (Table 10; Analysis 2). The stiffness of the old linkage bars would be less than the replacement bars but the stiffness of the linkage system is dominated by the stiffness of the rubber pads which were of similar dimensions in the old and new installations. If no (or very flexible) stiffeners were installed the analysis indicates that the relative movement at the joint would be about 130 mm. (See Figure 61 for c = 0.02. Figure 62 shows the increase in first mode period and Figure 68 the increase in spectral displacement.) Although the above analysis is over simplified it does indicate that there was merit in installing stronger restrainers. They will clearly reduce the amount of relative movement at the in-span joints to acceptable limits.

Calculations for the hinge joint relative displacement using the simplified Priestley (Equation 2, Section 8.5) are also presented in Table 10. The displacement components from transverse

### Table 10. Otaki River Bridge Analysis of Hinge Joints

| | | | |
|---|---|---|---|
| **Bridge Geometry, Weights and Stiffness** | | | |
| Span length - typical | 14.3 m | End spans are 10.9 m. | |
| Span weight | 1200 kN | Includes superimposed dead load. | |
| Pier weight | 400 kN | Excluding pile cap. | |
| Total number of spans | 15 | | |
| Spans in Section 1 | 4 | | |
| Spans in Section 2 | 3 | | |
| Dynamic mass per span | 143 t | Span + 50% of pier. | |
| Mass of Section 1 | 560 t | | |
| Longitudinal stiffness of pier | 10.9 MN/m | From FEA frame analysis. Piers frame into superstructure. | |
| Longitudinal stiffness of abutment | 30 MN/m | From Pender analysis using Young's modulus increase of 30 MPa/m. | |
| Number and length of linkage bars | 6 x 1.0 m | Macalloy 36 mm diameter S650 stainless steel. (Retrofitted bars.) | |
| Linkage rubber washers | 250 sq x 50 mm | One each end. IRDH = 60. | |
| Overall stiffness of linkage per bar | 12.3 MN/m | | |
| **Earthquake Analysis Parameters** | | | |
| Hazard Factor, Z | 0.40 | NZS 1170.5. | |
| Return Period Factor | 1.3 | NZS 1170.5 1000 year return period. | |
| Site Subsoil Category | D | NZS 1170.5. | |
| Structural Performance Factor | 0.67 | Bridge Manual for Site Subsoil Class D. | |
| **Two-Degrees-of-Freedom Model Parameters and Solution** | | | |
| | **Analysis 1** | **Analysis 2** | |
| Stiffness of Section 1, $k_1$ | 43.6 MN/m | 43.6 MN/m | |
| Restrainer stiffness ratio $k_c / k_1$ | 1.69 | 1.69 | |
| Structure stiffness ratio $k_2 / k_1$ | 1.5 | 1.5 | |
| Structure mass ratio $m_2 / m_1$ | 1.0 | 0.75 | |
| First mode dimensionless period | 0.90 | 0.86 | From Figure 60 (Analysis 1). |
| First mode period | 0.64 s | 0.61 s | |
| Response spectrum displacement | 96 mm | 91 mm | Figure 68. (NZS 1170.5 at first mode periods.) |
| Dimensionless relative disp. | 0.16 | 0.27 | From Figure 61 (Analysis 1). |
| Absolute relative displacement | 16 mm | 24 mm | |
| Dimensionless restrainer force | 0.28 | 0.45 | From Figure 62 (Analysis 1). |
| Absolute restrainer force | 1200 kN | 1800 kN | |
| **Priestley et al (1996) Solution** | | | |
| Period of Section 1 | 0.71 s | | From mass and stiffness of Section 1 (Analysis 1). |
| Response spectrum disp. Sect. 1 | 110 mm | | From Figure 68. |
| Stiffness of Section 2 | 65.4 MN/m | | From structure stiffness ratio. |
| Period of Section 2 | 0.59 s | | From mass and stiffness. |
| Response spectrum disp. Sect. 2 | 86 mm | | From Figure 68. |
| Abs. relative disp. between frames | 24 mm | | Assuming frames act independently. |
| Width of hinge seating | 6.45 m | | |
| Transverse response component | 97 mm | | From Equation (2) |
| Dist. between movement joints | 57.3 m | | |
| Travelling wave component | 57 mm | | From Equation (2) |
| Total design displacement | 178 mm | | |

Figure 66. Otaki River Bridge. Details of in-span hinge are shown in the bottom photo.

response and travelling wave effects are large in relation to the longitudinal dynamic response component. These components should also be added to the displacements computed using the two-degrees-of-freedom model.

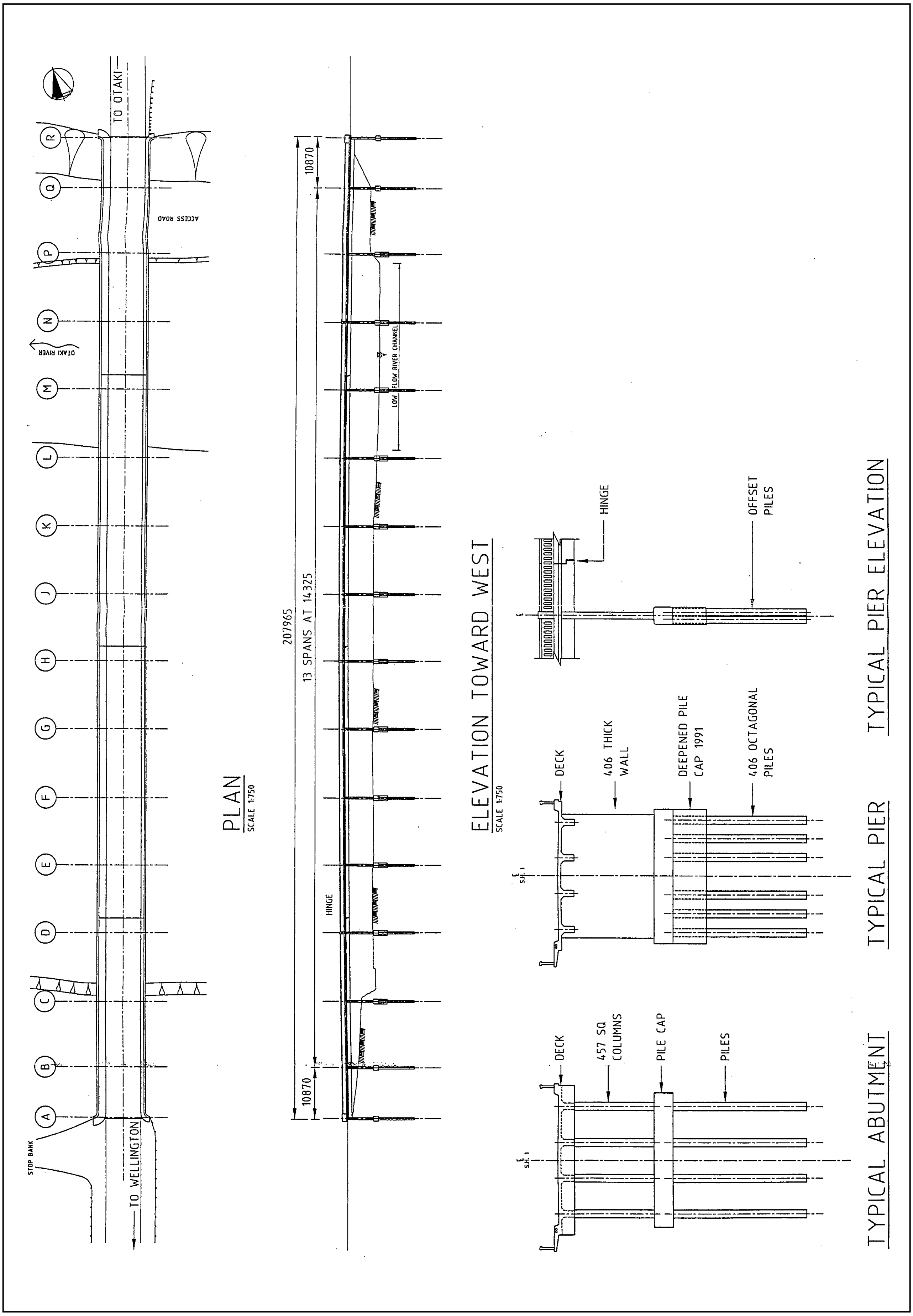



Figure 67. Otaki River Bridge.

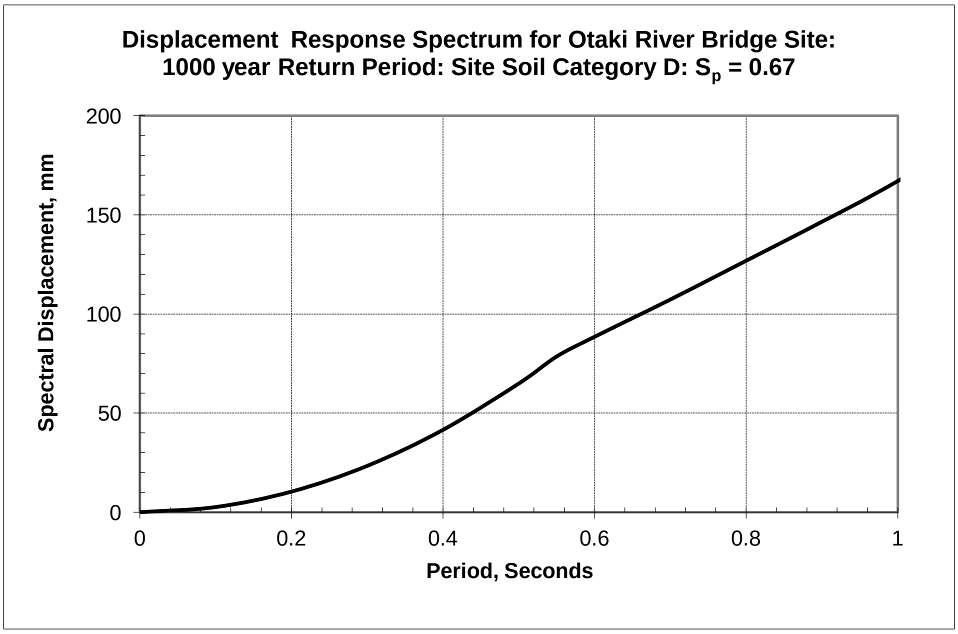



Figure 68. Site design displacement response spectrum for Otaki River Bridge site. (from NZS 1170.5)

## 8.6 Design Judgement

It is recognised that the theoretical approaches described in Sections 8.3 to 8.5 are complicated and liable to produce approximate results for the following reasons:

- The assumed properties of the structure and the supporting ground can be very different from those that actually apply.
- The assumed properties of the interaction between the ground and the foundations, at piers and abutments, are approximations only, and structural response can therefore differ significantly from the theoretical result.
- The ground motions driving the structure are not necessarily synchronous at each pier, particularly for long bridges. While this may reduce overall structure response it does not necessarily reduce the demands on linkages at structural joints.
- Ground motions are not uni-directional and, while the analyses may only consider the effects of longitudinal motions, concurrent transverse motions can introduce significant demands on, for example, linkages at the outer edges of bridge decks.
- Ground motions assumed in design can be significantly exceeded in practice and structures therefore must incorporate tolerance to such excess. For linkages this is usually achieved by ensuring the linkage has as much ductile capacity as can reasonably be provided.

It may be thought that, if theories are applied using an envelope of possible assumed parameter values, then the worst conditions for elastic demands on linkages can be determined. The problem with such an approach is that it can produce impractically large demands. The solution is therefore to provide well proportioned, practically sized linkages and reliable fixtures for them and to accept that the linkages will be required to yield. It is often more practical to provide more generous span/support overlaps than to provide greater linkage strength. The theoretical approaches described in Sections 8.3 to 8.5 provide a starting point from which practically sized structure solutions can be developed.

## 8.7 Design Standards

- Linkage assembly design should be based on the provisions of the Bridge Manual and relevant clauses in NZS3404 for connections and seismic design with the design force actions modified by the appropriate strength-reduction and over-strength factors specified for connection and seismic design. In assessing the performance of existing linkage systems, strength reduction factors can be neglected.
- A capacity design approach should be adopted for designing the anchoring brackets and diaphragms with yield in the linkage bars the controlling failure mode.
- Earthquake induced soil pressures on high abutment walls tend to cause the walls to slide or tilt towards the centre of the bridge. These wall deformations may be significant on single span and short two- or three-span bridges closing joint gaps and reducing the risk of spans falling. Where significant abutment wall movements towards the centre of the bridge are expected and the backwalls are robust, an acceptable minimum design level for abutment linkages is a 500-year return period event. However, when providing a stronger system than the minimum required is practical and likely to incur little additional cost this should be considered. If the backwall fails it is possible that an end span could unseat if the superstructure slides towards one end of the bridge. This possibility needs to be considered.
- The reduction of linkage forces by friction in span bearings may be considered in the design of linkage systems but reliable coefficients (based on reduced probable values) should be used and the effects of vertical accelerations considered.

## 8.8 Linkage Bar Design Details

In detailing bar linkage systems for both new and retrofitted bridges the following design issues should be considered:

- Achieving sound practical details is more important than achieving a high strength capacity with poor details which are likely to have low ductility. It is not possible to predict linkage forces to a good degree of precision and designing the system to be ductile and as strong as practical at a reasonable cost is better than placing undue reliance on analysis results.
- The bars should have good ductility (see test results in this report). This is to allow loads to become evenly distributed across the width of the bridge after yield in the most severely loaded linkage bars. Generally the initial “elastic” loads will be unevenly distributed.
- Improving horizontal diaphragm action in the superstructure by concentrating pier linkage system near the outer edges of the spans should be considered. Estimates of forces arising from diaphragm action should be made. In this case, the stiffness of the linkages in tension, as well as their strength, may be important for maintaining stiffness of the diaphragm action.
- Anchoring linkage bars and anchor brackets by drilling through the anchoring members and installing nuts is preferred to relying on anchoring bars and bolts with epoxy grout.
- Both the strength of the linkage system and the span seating length should be considered when assessing the risk of spans falling. With large seating lengths a less robust linkage system may be acceptable (see Bridge Manual).
- Durability of the linkage system is important. With higher strength steels now available it is possible to provide the design forces in small bridges with bars of quite small diameter. Galvanised steel bars have limited life and allowance for loss of section by corrosion should be considered. Thread damage, bending of bars during maintenance operations and corrosion of nuts due to galvanic action are considerations in the detailing of stainless steel bars. Diameters of less than 20 mm should not be used for either galvanised or stainless steel bars.

- Adequate clearances and linkage bar hole sizes should be specified to reduce the risk of damage to linkage bars under combined longitudinal and transverse displacements of the superstructure.
- When retrofitting linkage systems to bridges with existing linkages and holding-down bolts the new linkages should be sufficiently stiff (or tight) within their elastic range to minimise damage to the existing linkage and holding-downs components in earthquake events less than the design level.

# 9. Conclusions

### *Ductility of Proprietary Bar Systems*

- Both the Reidbar and Macalloy bar assemblies provided satisfactory tensile ductility with minimum plastic elongations of 6% (40 mm) of the loaded length for the MS32 assembly and 17% (107 mm) for the RB32 assembly.
- Provided locknuts are used it is not necessary to turn-down Reidbar or Macalloy bar assemblies since the ultimate strength of the nuts will exceed the ultimate strength of the bar by a satisfactory margin. About 50% of the elongation in these bars occurred in a 300 mm length near the failure section and the remaining elongation was distributed over the loaded length of the bar. Unless the bar is turned-down over most of its length, turning down would probably reduce the elongation as it would only occur in the turned-down length. Increasing the bar length will increase the elongation but because a large part of the elongation occurs in a relatively short section the elongation is not directly proportional to bar length.
- The Freyssibar FH27 assemblies had low tensile ductility with a minimum plastic elongation of 3.3% (27 mm) of the loaded length. They had the second lowest ductility of the assemblies tested and would not be suitable for use in most bridge linkage systems. Because of its high strength, Freyssibar may be useful on large bridges that have special span and abutment anchorage requirements. The minimum specified ultimate load of 873 kN for a 32 mm diameter bar is 40% greater than for the equivalent diameter Macalloy S650 bar.

### *Ductility of Round Bar with Threaded Ends*

- Although the minimum plastic elongations of the RM36 and RS36 assemblies at 8% (64 mm) and 10% (81 mm) of the loaded length respectively were not large, they would be satisfactory in many linkage applications where the total bridge length was less than about 50 m and for longer bridges on good foundations.
- The RS38 assemblies had low plastic elongations (4.1% of the loaded length). Comparison with the RS36 elongations illustrated the benefit of having the shank diameter the same as the nominal thread diameter. The elongation of the round bar assemblies is sensitive to the length of thread in the loaded length as over 50% of the elongation in the RM36 and RS36 assemblies occurred in the threaded sections. To achieve good ductility the loaded length of thread at either end of the assembly should be at least three times the nominal thread diameter.

### *Ductility and Geometry of Turned-Down Plain Bar*

- Comparison of the plastic elongation results for the RM36 and TM30 assemblies and for the RS36 and TS30 assemblies showed that turning down plain round bar did not necessarily improve the tensile ductility by a great amount provided adequate length of loaded thread was used on the plain round bars. However, turning down provides a more reliable system eliminating the possibility of nut failures or poor performance where the bars are installed with insufficient threaded length in the loaded section.
- The increase in cost from turning-down a plain round bar is unlikely to be large if the bar has to be machined to provide threads. For the TS30 assemblies, turning-down increased the cost of the assembly by about 20% compared to the RS38 assemblies which required no machining on their shanks. However, if the performance of the R38 bars had been improved by turning the shanks down to 36 mm diameter there would have been little cost advantage in using a plain round bar. In many cases turning-down may not increase the cost of a complete linkage retrofit by a significant amount and for important or long bridges a small cost increase would be justified on the basis of improved and more reliable ductility performance.

- Comparison of the elongations for the TM30 and TMO assemblies indicated that a significant increase in tensile ductility of mild steel bars could be obtained by increasing the turned-down length from 300 mm to 670 mm. However, the 300 mm turned-down length used for the TM30 and TS30 bars gave elongations of the loaded length of 11% and 9% respectively which should be sufficient in most applications. The turned-down length should be a minimum of 10 times and ideally 15 times the turned-down diameter.
- The turned-down diameter ratio of the TM30, TS30, TH30 and TMO bars of 0.83 times the nominal thread diameter was satisfactory. With this high ratio, significant yield occurred in the threaded lengths of the TM30 and TMO assemblies but the elongations in these sections were only about 5%, which was well below the minimum of 25% expected to cause failure in the threads. Although a smaller diameter ratio can obviously be used it requires the use of larger bars and anchor fittings.
- The minimum elongation of the TH30 high tensile steel turned-down assemblies was 3.0% (24 mm) of the loaded length. They had the lowest elongations of the tested assemblies. The turned-down length of 500 mm was chosen to match the length used on the recently retrofitted linkage bars on the SH1 Pukerua Bay Railway Overbridge. The retrofitted bars had a turned-down diameter of 28 mm but the other details including the steel material were similar. A large part (over 80%) of the elongation of the TS30 assemblies incurred in a 300 mm long failure zone so increasing the length of the turned-down section would not significantly increase the total elongation. The ultimate strength of the bars used on the bridge would be about 620 kN or about 13% lower than the tested assemblies. All the stainless steel assemblies tested, excluding the TS30 assemblies, had minimum strengths approaching this value and would have been acceptable alternatives with significantly greater tensile ductility and corrosion resistance. With the possible exception of the MS32 bars, the stainless steel assemblies would be unlikely to cost significantly more than the TH30 assemblies.

***Ductility of Fully Threaded Bar***

- Although the US36 fully threaded stainless steel bar assemblies may have a small cost advantage over some of the other stainless steel alternatives their minimum elongation of 5.8% of the loaded length was significantly less than for the RS36 and TS30 assemblies. The TS30 assemblies had a more predictable elongation performance and for most applications would be a better alternative.

***Nuts***

- It is essential that lock nuts be used on all linkage assemblies. Obviously they prevent accidental loosening but they also reduce the risk of a non-ductile nut splitting or stripping failure occurring. Had the first Reidbar assembly tested been fitted with locknuts it is unlikely that a nut failure would have occurred. In the case of the RS38 assembly nut failure, the bar had a specified UTS approaching that of the nut steel. Some Grade 316 stainless steels have very high UTS values and this needs to be considered in linkage assembly design. The Property Class required for the nuts should be included in any retrofitting specification.

***Fracture Toughness***

- Grade 316 and S650 (MB) stainless steel bars have good fracture toughness and can be used for linkage bars at any location in New Zealand. Reidbar assemblies are unlikely to be suitable in the South Island and if used elsewhere the site Service Temperature needs careful consideration. Reidbar nuts are not suitable for use in temperatures below 0 $^{o}$C and instances of brittle fracture in the two recent Canterbury earthquakes raise issues over their fracture toughness at higher temperatures.

- The fracture toughness of mild and medium tensile steels should be assessed before they are used in the South Island and colder regions in the North Island. Grade 300 L15 should be satisfactory in all but the coldest regions of the South Island.

***Linkage Bar Design Forces***

- The most satisfactory method of determining design forces in longitudinal linkage systems connecting the superstructure to abutments and piers is to carry out a static push-over analysis modelling the stiffness of all the substructure components. The earthquake loads acting on the bridge should be estimated using the Bridge Manual design response spectrum (based on NZS 1170.5). Usually the analysis can be carried out using a spreadsheet approach with force versus displacements functions developed for the abutments and piers from published information on the passive soil resistance against abutment walls and the displacement response of pile foundations. It is essential that the results of the analysis are tempered with judgement to ensure that practically sized, tolerant and economic solutions are adopted. It must be remembered that analyses are likely to be only an estimate of the likely performance of the structure.
- For short bridges of up to three spans and where the abutments are clearly a lot stiffer under longitudinal horizontal loads than the piers it would be reasonable to design the linkages at the abutments to carry the total superstructure longitudinal inertia load. Unless more detailed stiffness calculations are carried out, 50% and 70% of the inertia force should be assumed to be carried on the "pull" and "push" abutments respectively.
- The likely added forces that may be imposed on linkage bars due to horizontal diaphragm action of the deck under transverse response should be estimated by a transverse analysis and combined with the linkage forces estimated from the longitudinal response analysis. In the transverse analysis of multi-span bridges the relative stiffness of the deck and the sub-structure components should be considered. In short bridges with stiff abutments the superstructure can be assumed to span as a horizontal beam between the abutments.

***Relative Displacements at In-Span Hinge Joints***

- The computer model results published in Priestley et al (1996) and the two-degrees-of-freedom model results presented in this report can be used for preliminary design of movement joint seatings and restrainers. It is important to consider the relative longitudinal displacement resulting from transverse response and travelling wave effects (see Priestley et al). For large new structures verification and final design should be undertaken using specific computer model analyses.

***Linkage Bar Design Details***

Detailing linkage assemblies requires careful consideration. The design should take into account the detailing issues summarised in Sections 8.6 and 8.8.

# 10. Recommendation

In addition to the complexity of determining the loading and displacement conditions under which linkages may be required to perform, the design and selection of suitable linkage assemblies for application to bridge spans can be quite complex. As a successor to this testing project it would be valuable to bridge designers if a range of standard linkage assemblies were designed, taking account of the variables of length, diameter, geometry (e.g. turned-down) and materials.

## 11. Acknowledgements

The New Zealand Transport Agency funded this project.

Thanks are due to Howard Chapman who reviewed this report. His comments and revisions resulted in a much improved presentation.

Bob Stevenson and Russell Kean of Opus Central Laboratories assisted with the testing and data recording.


## 12. References


AS 1252:1983 *High-strength bolts with associated nuts and washers for structural engineering.*

AS/NZS 1252:1996 *High-strength bolts with associated nuts and washers for structural engineering.*

AS/NZS 4291.2: 1995 *Mechanical properties of fasteners. Part 2: Nuts with specified proof load values – Coarse thread.*

AS/NZS 4671: 2001 *Steel reinforcing materials.*

DIN EN ISO 4032 *Hexagon nuts, style 1.*

Kahalili-Tehrani P and Taciroglu E. 2010. *Backbone Curves for Passive Lateral Response of Walls With Homogenous Backfill.* Soil-Structure-Interaction; Orense, Chouw and Pender Editors. Taylor & Francis Group, London.

Mandeno W L. 2010. *Personal Communication.*

Macalloy. 2012. *Bar and Cable Systems. Macalloy 650 Stainless Specification.*

NZS 3402: 1983. *Steel Bars for the Reinforcement of Concrete.*

NZS 3404: Part 1: 2009. *Steel Structures Standard.* Standards New Zealand.

NZTA. 2003. *Bridge Manual, Second Edition, Including amendments to December 2004.* New Zealand Transport Agency, Wellington.

Pender M J. 1993. *Aseismic Pile Foundation Design Analysis.* Bulletin NZSEE, Vol 26, No 1, pp 49 – 161.

Priestley M J N, Seible F, and Calvi G M. 1996. *Seismic Design and Retrofit of Bridges.* John Wiley & Sons Inc.

Reid Construction Systems. 2008. *Product Catalogue and Design Guides.*

Roberts B. 2010. *Personal Communication.*

Thompson W T. 1965. *Vibration Theory and Applications.* Prentice-Hall Inc.

Wood J H, and Chapman H E. 2011. *Design of Linkage Bolts for Restraining Bridge Spans in Earthquakes.* Proceedings 9th Pacific Conference on Earthquake Engineering, Auckland, New Zealand.

# Appendix A

# Technical Background Supporting Project

It is not possible to reliably predict the loads that might develop in a linkage system when a bridge is subjected to severe earthquake ground shaking. Complications arise because of permanent ground deformations and the lack of coherence between the ground input motions at the piers of long bridges.

Should the linkage system become overloaded there are clearly benefits from having ductility or elastic resilience in the linkage system. Ductility or elastic stretch results in a better distribution of loads across the width of the bridge and results in load being transferred to the linkages on adjacent piers which may less heavily loaded. Transfer of load may prevent the overloaded bars from failing. Plastic deformation in linkage rods also absorbs energy reducing the build-up of the superstructure vibrations. Without adequate ductility, sudden fracture of the linkage may occur with complete loss of restraint leading to an unacceptable risk of a span sliding off the supporting pier or abutment.

Ductility or elastic resilience can be provided by one or more of the following methods:

(a) Ductile alloy steel that is not prone to reduced elongation because of stress concentrations in the thread.

(b) Proprietary bar with rolled thread which reduces the risk of yield over a short length close to the nut.

(c) Long bars which have significant elastic extension.

(d) A turned-down shank that yields away from the nuts reducing the effect of the reduced stress area in the threaded sections.

(e) Elastomeric pads in the linkage assembly to provide elastic extension.

All methods have shortcomings. Methods (c) and (e) may result in excessive deformation at design level loads that prevents diaphragm action being utilized and may result in holding down bars being damaged. Elastic extension does not provide the energy absorption available from a yielding bar.

In New Zealand it is common practise to use turned-down shanks (method (d)), to eliminate the possibility of the nut or threaded section reducing the plastic elongation. This is a satisfactory approach but machining down the shanks increases the cost and it is unclear how much additional ductility is achieved. When a bar commences to yield it strain hardens in the adjacent sections and necks down with plastic elongation in a short local area. Turning down over a long length is of little advantage and turning down over either a long or short length may not result in significantly better ductility than an unmodified bar with suitable threading. One obvious disadvantage is that a turned-down bar requires the use of a larger diameter to maintain the same strength as the unmodified section, and although this may not increase the cost very much it may add significant cost where the bars are anchored on fabricated brackets which need to be larger.

Mild steel galvanised linkage bars with standard cut threads (method (a)) were recently removed for testing from the Ahuriri Bridge (SH8 near Omaramara). Because of the reduced stress area yield commenced in the threaded section with plastic elongation mainly restricted to this length. The plastic deformation in the threaded section was almost as large as expected in an unthreaded section. The total plastic elongation appeared to be related to the length of thread available between the end of the shank and head of the nut. The stress concentration effect of the thread did not appear to be significant. It appeared that provided the bar is designed to have sufficient length of thread in the loaded section between the nut and shank, the ductility available might be satisfactory.

After being in service for approximately 45 years there was significant corrosion in the Ahuriri Bridge linkage bars where water had run through the joints removing the galvanising over a short length.

Proprietary bars with rolled threads (method (b)) have been used for linkages in New Zealand bridges. Reidbar is the most commonly used bar. It is manufactured from medium tensile steel and has surface deformations formed in the rolling process similar to those of reinforcing bar. Special nuts engage on the deformations. Macalloy prestressing rod has also been used. It can be supplied with a continuous rolled U shaped thread and matching nuts. A particular advantage is that it is supplied in Grade 316 stainless steel which has a medium to high yield or proof stress and good corrosion resistance.

Yield and ultimate strengths, and elongation data are available from the proprietary bar manufactures but there is no information available on the elongation (or ductility) performance of bar assemblies using the system nuts. It is not known whether the nuts reduce the elongation or over what length yield is likely to occur in a typical assembly. In some bridge linkage installations, Reidbar has been turned-down but it is unclear whether this offers significantly improved ductility over an unmodified assembly.

# Appendix B

## Linkage Bars Removed From Ahuriri River Bridge

(25.4 mm diameter galvanised mild steel bolts – loaded length 700 mm approx,)

## Emailed Summary of Test Results

| | |
|---|---|
| **From:** | John Wood [john.wood@xtra.co.nz] |
| **Sent:** | 04 July 2008 07:13 |
| **To:** | 'Gavin Gregg' |
| **Cc:** | 'Donald Kirkcaldie'; 'Dejan Novakov'; 'Howard Chapman'; 'Bob Stevenson' |
| **Subject:** | Ahuriri River Bridge: Linkage Bolts |
| **Attachments:** | PICT0276.JPG; PICT0278.JPG; PICT0284.JPG; PICT0286.JPG |

Gavin

Bob Stevenson of Opus Central Laboratories carried out tensile testing on two of three linkage bolts removed from the Ahuriri River bridge for inspection and testing. I was present during the test on the first bolt. A summary of the procedure and findings from the first bolt tested is as follows:

1. The test length was chosen to simulate the arrangement in the bridge as far as possible with the load applied through the nut and head bearing against heavy washers. The 38 mm thick rubber washer used on the bridge installation was excluded from the test setup. The overall length between the load faces was about 710 mm with about 40 mm of thread included in this length. Two standard nuts were present on one end (as on the bridge) and the head was a single nut tightened onto a short length of thread.
2. The bolt failed in the thread length at a load of about 180 kN representing a failure stress on the stress area of the thread of about 460 MPa (see attached photos).
3. The total extension at failure was about 31 mm equivalent to a 4.4% extension of the loaded length. This is well in excess of the minimum acceptable strain of 0.5% specified during the review work.
4. The yield stress in both the threaded and shank lengths was estimated to be about 320 MPa. This was about 30% higher than the nominal value of 250 MPa used in design and assessment.
5. Initially the bolt yielded in the threaded section and as this length strain hardened the yield moved into the shank length. Shortly after strain hardening commenced in the shank the bolt failed in the threaded section. The ratio of the shank area over the threaded stress area was about 1.3 so this behaviour was reasonably predictable. Of interest was the large strain that developed in the threaded section with the failure not influenced to any large extent by stress concentration from the thread. The nuts and bolt head appeared undamaged. At failure the estimated strain in the treaded length was about 15% with the strain in the shank of about 3% (rough estimates).
6. The bolts showed pitting corrosion with pits about 1 mm deep at their centre sections (see attached photos). Apparently moisture and dirt from the deck joints had spilled onto the bolts on this section leading to loss of galvanising and steel corrosion. The galvanising was in reasonable condition on the remaining lengths, washers head and nuts. Because the bolt shanks have a much greater area that the threaded stress area they are likely to perform satisfactory for a number of years. Considering they have been in the bridge for about 45 years they probably have at least a further 10 years of life.

At present the plan is to send the third bolt to Auckland for fracture toughness testing. Given the condition of the bolts it would seem best to replace them during the present upgrading work. If it is agreed that this is the best approach then a decision is required as to whether it is worthwhile to proceed with the fracture toughness testing which is expected to cost about $800. In my opinion it is worthwhile as the information would be relevant to other bridges located in cold areas. Already very useful information has been obtained from the investigation to date.

Bob Stevenson needs to know early next week whether to send the third bolt to Auckland for fracture testing. I understand that he is preparing a report on the tensile testing.

Regards
John Wood

## Table C1. Ahuriri River Bridge: Linkage Bolt Test Summary

Prepared: J Wood
Date: 6-Feb-2011

| Item | Symbol | Value Bolt 1 | Value Bolt 2 | Units | Comment |
|---|---|---|---|---|---|
| **Lab Measured Values** | | | | | |
| Measured yield force in shank | Pys | 163 | 158 | kN | |
| Measured yield force in thread | Pyt | 125 | 120 | kN | Estimated Bolt 2 |
| Measured failure load | Pu | 179 | 180 | kN | |
| Measured loaded length | Lg | 709 | 701 | mm | Len. between head and nut faces |
| Measured extension of loaded length | Du | 31 | 32 | mm | |
| Overall strain % on loaded length | eu | 4.37 | 4.56 | % | |
| **Measured Strain on Gauge Length** | | | | | |
| Extension shank on 500 long gauge length | Dgl | | 14 | mm | |
| Strain in gauge length | egl | | 2.8 | % | |
| **Calculated Stress Values** | | | | | |
| Shank area | As | 507 | | $mm^2$ | 25.4 mm diameter |
| Area at bottom of threads | At | 358 | | $mm^2$ | Assuming BSW 8 threads per in. |
| Stress area factor | Fsa | 0.606 | | | From internet |
| Stress area | Ast | 391 | | $mm^2$ | From internet |
| Check stress area using published formula | | 395 | | $mm^2$ | Reasonable agreement |
| Area ratio Shank Area / Stress Area | Ra | 1.30 | | | |
| Stress at yield in shank | fys | 322 | | MPa | |
| Stress in threads at yield | fyt | 319 | | MPa | Based on "stress" area above |
| Failure stress in threads | fut | 459 | | MPa | |
| Ratio failure to yield stress in threads | Rs | 1.44 | | | |
| Yield strain - based on shank | esy | 0.161 | | % | |
| **Calculated Extension** | | | | | |
| Number of free treads | Nft | 10 | 9 | | Approximate |
| Threaded len. - 12 threads free some in nut | Lt | 48 | 48 | mm | Approx - based on 15 threads |
| Estimated strain at UTS in thread | eut | 20 | | % | Best guess |
| Ultimate extension in threaded length | Dtu | 10 | | mm | Approximate |
| Ratio shank stress at ultimate/yield stress | Rsu | 1.10 | | | |
| Expected strain in shank at failure load | esye | 3.0 | | % | Priestley et al Fig 5.5 (p273) |
| Total extension calculated at failure | Dp | 29 | | mm | Good agreement with observed |
| Extension calculated at thread yield | Dty | 0.90 | | mm | |
| Measured extension was about | Dtym | 2.0 | | mm | |
| Estimated yield extension in thread | Dtye | 0.95 | | mm | 2% on yield plateau |

**Table C2. Bolts Removed From Ahuriri River Bridge: Charpy Impact Test Results**

| Test No | Impact Resistance, Joules | |
|---|---|---|
| | Temp = -10 C$^{o}$ | Temp = -15 C$^{o}$ |
| 1 | 54 | 48 |
| 2 | 68 | 48 |
| 3 | 74 | 48 |
| Mean | 65 | 48 |

Figure B1. Linkage bolt from Ahuriri River Bridge after tensile testing.

Figure B2. Failure in loaded thread of Ahuriri River Bridge linkage bolt.

Figure B3. Corrosion on Ahuriri River Bridge linkage bolt.

Figure B4. Corrosion on Ahuriri River Bridge linkage bolt.

# Appendix C

# Bridge Linkage Installations

A summary of bridges that have recently been assessed as requiring new linkage bars or have had linkage bars recently installed is given in Table C1.

**Table C1. Bridges Requiring Linkage Bars or With Recently Installed Linkage Bars.**
(Assessment since 2004. Construction since 2007.)

| Bridge | NZTA Reg-ion | SH | Comment |
|---|---|---|---|
| **Detailed Seismic Assessments: Linkages Proposed as Part of More Extensive Assessment and Retrofit Work** | | | |
| Market Rd Main Bridge | 2 | 1 | Linkage bars designed to connect new abutment piles. 32 dia. Reidbar. |
| Victoria Park Viaduct | 2 | 1 | In-span joints require linkages. |
| Pahurehure Inlet Bridges | 2 | 2 | Linkage bars proposed for abutments. Grade 316 SS specified. |
| Strand Rail Overbridge | 2 | 16 | Linkage bars retrofitted at piers. M30 turned-down to 25 dia - Grade 8.8. |
| Whau River Bridges | 2 | 16 | In-span linkages need replacement as part of widening project. |
| Waihou River Bridge | 3 | 2 | Improvements to linkages at piers and abutments will be required. |
| Hairini River Bridge | 4 | 2 | Linkage bars designed to improve abutment linkage. 32 dia. Reidbars. |
| Westshore Bridge | 6 | 2 | New linkage bars installed at piers. Grade300, M64, M56, turned-down. |
| Mohaka River Bridge | 6 | 5 | Linkage bars installed to anchor bridge to one abutment. M30 – Grade 8.8. |
| Frasertown Bridge | 6 | 38 | Improved linkages required for suspended spans. |
| Ohau River Bridge | 8 | 1 | Replacement of span hinge linkage bars proposed. |
| Mangaturuturu Stream Bridge | 8 | 4 | Improvements to linkages at abutments and piers may be required. |
| Pukerua Bay Overbridge | 9 | 1 | Linkage bars designed for piers. Galvanised AISI 4140 turned-down to 26 dia. |
| Otaki River Bridge | 9 | 1 | Span hinge linkage bars replaced with Macalloy 36 dia. S650 bar. |
| Waikanae River Bridge | 9 | 1 | Linkage bars retrofitted at abutments using Macalloy 30 dia. S650 bar. |
| Pakuratahi River Bridge | 9 | 2 | Linkage bars retrofitted at abutments using Macalloy 32 dia S1030 bar. |
| Tauherenikau River Bridge | 9 | 2 | Linkage bars proposed for abutments and piers. |
| Waima River Bridge | 10 | 1 | Linkage bars proposed at abutments and piers. Design forces specified. |
| Racecourse Creek | 10 | 6 | Linkage bars installed at piers and abutments. AISI 4140HT or SS specified. |
| Chaneys Road Underpass | 11 | 1 | Earthquake damaged linkages at one abutment need replacement. |
| Kaiapoi Railway River Bridge | 11 | 1 | Earthquake damaged linkages at piers & abutments need replacement. |
| Shale Peak Stream Bridge | 11 | 7 | Improvements to linkage system at piers and abutments proposed. |
| Hototane Valley Overpass | 11 | 74 | Linkage bars installed at piers. SS bar used. Design forces specified. |
| Port Hills Rd Overpasses | 11 | 74 | Linkage bars installed at piers & abuts. SS bar used. Design forces specified. |
| Port of Timaru Bridge | 11 | 78 | Linkage improvements proposed at piers and abutments. |
| Kawarau River Bridge | 13 | 6 | Linkage bars installed at abutments. 34 mm dia. SS bar turned-down. |
| Clutha River (Alexandra) | 13 | 8 | Linkage bars required at north abutment. Macalloy bars proposed. |

**Table C1 Continued………**

| Bridge | NZTA Reg-ion | SH | Comment |
|---|---|---|---|
| **Bridges Assessed for Linkage Retrofits Only** | | | |
| Haumi River Bridge | 1 | 11 | Shear keys designed with long fixing bolts through piers & abutments. |
| Tangiteroria Bridge | 1 | 14 | Linkage bars designed (susp. span hinges). Round SS 304L or 316 bar specified. |
| Waikawau River Bridge | 3 | 25 | Linkage bars installed at abutments and piers. 20 mm dia. Reidbar. |
| Maukoro Canal Bridge | 3 | 25 | Linkage bars installed at abutments and piers. 20 mm dia. Reidbar. |
| Waitakaruru Canal Bridge | 3 | 25 | Linkage bars installed at piers and abutments. 16 and 20 mm dia. Reidbar. |
| Whangamaroro River Bridge | 3 | 25 | Linkage bars installed at piers and abutments. 16 and 20 mm dia. Reidbar. |
| Waitoa River Bridge | 3 | 26 | Linkage bars installed at piers and abutments. 12 mm dia. Reidbar. |
| Warahoe Stream Bridge | 3 | 26 | Linkage bars installed at abutments and piers. 12 mm dia. Reidbar. |
| Puriri Stream Bridge | 3 | 26 | Linkage bars installed at abutments. 12 mm dia. Reidbar. |
| Ohinemuri River Bridge | 3 | 26 | Linkage bars installed at piers and abutments. 20 and 25 mm dia Reidbar. |
| Waiete Rail Overbridge | 3 | 30 | Linkages to be installed at abutments and piers. 20 mm dia. Reidbar. |
| Waimaha Stream No 2 Bridge | 3 | 30 | Linkages to be installed at abutments and piers. 20 mm dia. Reidbar. |
| Mangaongoki Stream Bridge | 3 | 32 | Linkages to be installed at abutments. 20 mm dia. Reidbar. |
| Waiopua Stream Bridge | 4 | 2 | Linkage bars installed at abutments. 16 mm dia. Reidbar. |
| Pekatahi Flood Control Bridge | 4 | 2 | Linkage bars installed at abutments. 16 mm dia. Reidbar. |
| Mangaone Stream Bridge | 8 | 56 | Linkages required but at low medium/low priority. |
| Wairoa River Bridge | 10 | 6 | Linkage bars installed on piers. Round SS or AISI 4140HT bar specified. |
| Kowhai No 1 Bridge | 11 | 73 | Linkage bars installed at abutments. SS rod turned-down? |
| Rakaia Gorge No. 2 Bridge | 11 | 77 | Linkage bars proposed at two main piers. Design forces specified. |
| Camping Gully Stream Bridge | 11 | 77 | Linkage bars proposed at abutments. Design forces specified. |
| Selwyn River Bridge | 11 | 77 | Linkage bars proposed at every second pier. Design forces specified. |
| Cadrona Bridge | 13 | 6 | Linkage bars installed at one abut. Round SS or AISI 4140HT bar specified. |
| Ahuriri Bridge | 13 | 8 | Replace or refurbish existing linkage bars at piers and abutments. |
| Silver Stream Bridge | 13 | 87 | Linkage bars proposed at abutments. Design forces specified. |
| Lee Stream Bridge | 13 | 87 | Linkage bars proposed at abutments. Design forces specified. |
| Kaiwera Stream Bridge | 14 | 93 | Linkage bars installed at abutments. |
| Aparima River Bridge | 14 | 99 | Linkage bars proposed at piers. Design forces specified. |
| Waiau River Bridge | 14 | 99 | Linkage bars proposed at one abutment. Design forces specified. |

| NZTA Region Number | Name |
|---|---|
| 1 | Northland |
| 2 | Auckland |
| 3 | Waikato |
| 4 | Bay of Plenty |
| 5 | Gisborne |
| 6 | Hawke's Bay |
| 7 | Taranaki |
| 8 | Manawatu Wanganui |
| 9 | Wellington |
| 10 | Nelson Marlbrough |
| 11 | Canterbury |
| 12 | West Coast |
| 13 | Otago |
| 14 | Southland |