PERFORMANCE OF LINKAGE BARS FOR RESTRAINT OF BRIDGE SPANS IN EARTHQUAKES

John H. Wood[1] and Howard E. Chapman[2]

SUMMARY

Many New Zealand bridge decks consist of simply supported spans, which are interconnected with steel linkage bars. The main purpose of the bars is to restrain and prevent the bridge spans falling in an earthquake. The prediction of the forces imposed on the linkages is quite indeterminate because of the many variables that affect the response 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 have large plastic extensions before failing in tension.

The paper presents the results of laboratory tensile testing of a range of linkage bar types and conclusions are made regarding the most suitable bar assemblies, taking into account tensile ductility and cold temperature fracture toughness.

Recommendations are made regarding methods of predicting earthquake loads in linkage system and on design detailing for linkage assemblies.

1. INTRODUCTION

Seismic screening of the New Zealand state highway (SH) bridges identified that there were 170 bridges that required improvement to their inter-span linkages (Chapman *et al*, 2005). Retrofit work on the 134 bridges on Priority 1 and 2 routes is substantially complete and design work has commenced on some of the 36 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.

Detailed seismic assessments have been completed on about 70 of the 335 bridges that the screening identified as requiring investigation (Chapman *et al* 2005). 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 them.

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 transverse 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 and damping 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.

The design of the first linkage bars used in the 1960s was based on strength considerations without specific attention to ductility. 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 less ductile 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.

2. PROJECT OUTLINE

The aims of the project were to investigate and evaluate the various options for linkage bars now available, and to set out appropriate linkage system design methods.

[1] *Principal, John Wood Consulting, Lower Hutt (Life member)*

[2] *Retired, Porirua (Life member).*

2.1 Testing

Thirteen types of linkage assemblies, formed from various types of steel bar, were tested for tensile strength and ductility.

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 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 one bar type in the absence of available information for the type of high tensile steel used.

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.

3. LINKAGE ASSEMBLIES TESTED

Details of the 13 types of linkage assemblies tested and the specified minimum yield loads for the bars are given in Table 1. Table 3 lists the specified minimum yield and ultimate loads of the bars and the values achieved by the tests. The general arrangement of the assemblies showing threaded, turned-down (where used) and gauge lengths are presented in Figures 1 and 2.

3.1 Bar Material and Tensile Strength

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 (Reid, 2008; Macalloy, 2012).

The mild steel bar used for bar types RM36 and TM30 was manufactured as Grade 300Plus, 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 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 manufacturer.

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 850-1,000 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).

Table 1. Linkage Assemblies

Linkage Assembly	Ident-ifier	Nom Shank Diam	Loaded Length Between Nuts	Spec Min Yield or 0.2% Proof[1] Load
		mm	mm	kN
Reidbar - Grade 500E galvanised	RB32	32	620	402
Reidbar - Grade 500E galvanised	RB25	25	880	246
Macalloy S650 - Grade 316 stainless	MS32	32	660	506
Round - galvanised mild steel	RM36	36	810	253
Round - Grade 316 stainless steel	RS38	38	810	399
Round - Grade 316 stainless steel	RS36	36	810	266
Fully threaded - Grade 316 stainless steel	US36	36	810	-
Freyssibar - high tensile steel	FH27	27	820	549[2]
Turned-down - galvanised mild steel	TM30	36	810	219
Turned-down - Grade 316 stainless steel	TS30	38	810	345
Turned-down - galv. high tensile steel	TH30	36	810	573
Round - galvanised mild steel ex Kaiapoi	RMK	38	720	249
Turned-down - galv. mild steel ex Otaki	TMO	40	1060	190

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

3.2 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 the tensile tests. The RB25 bars were tested 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 machined 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 suit 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.

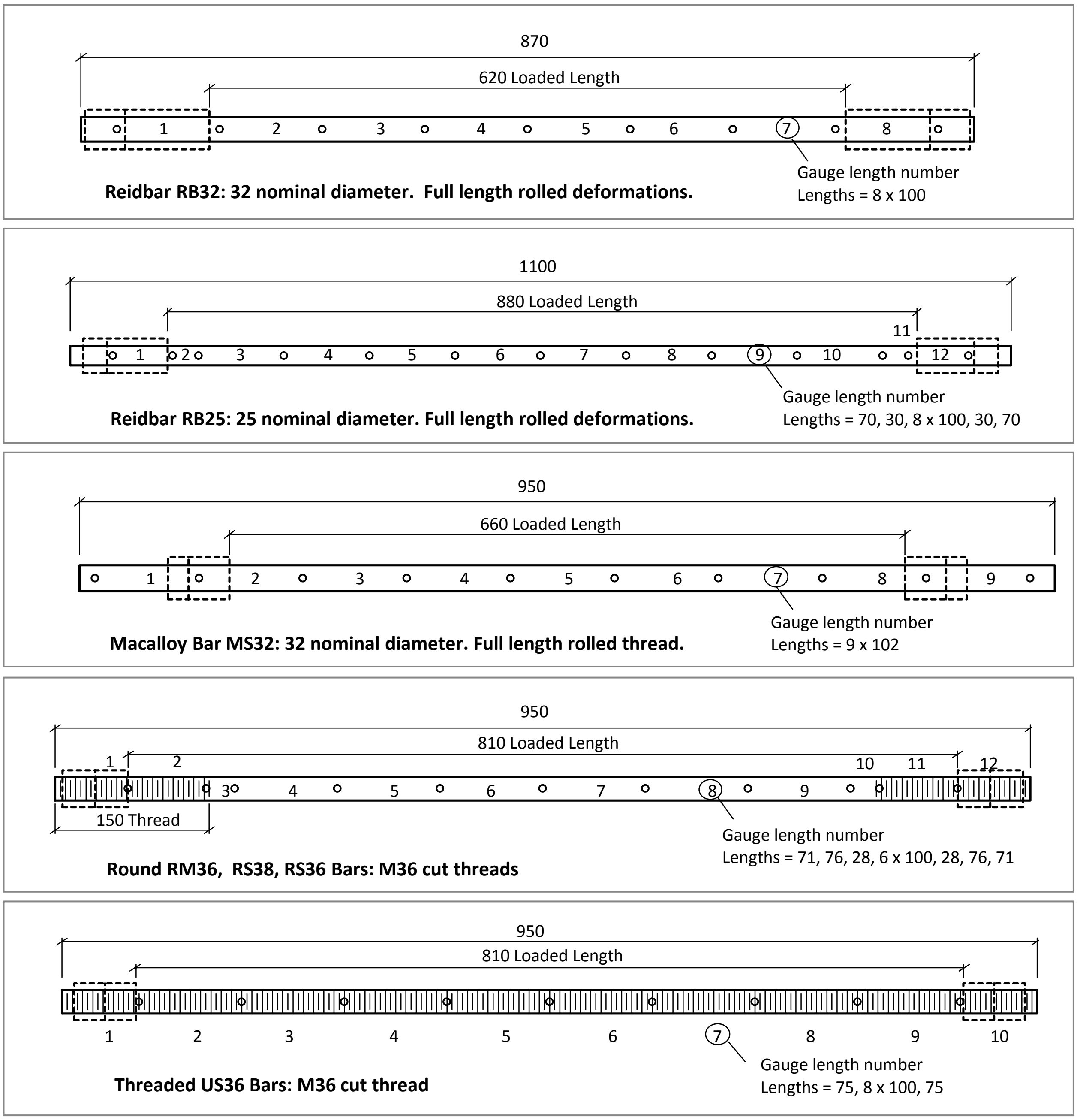

Figure 1: Dimensions of RB32, RB25, MS32, RM36, RS38, RS36 and US36 linkage assemblies and location of gauge lengths. All bars symmetrical about centre point.

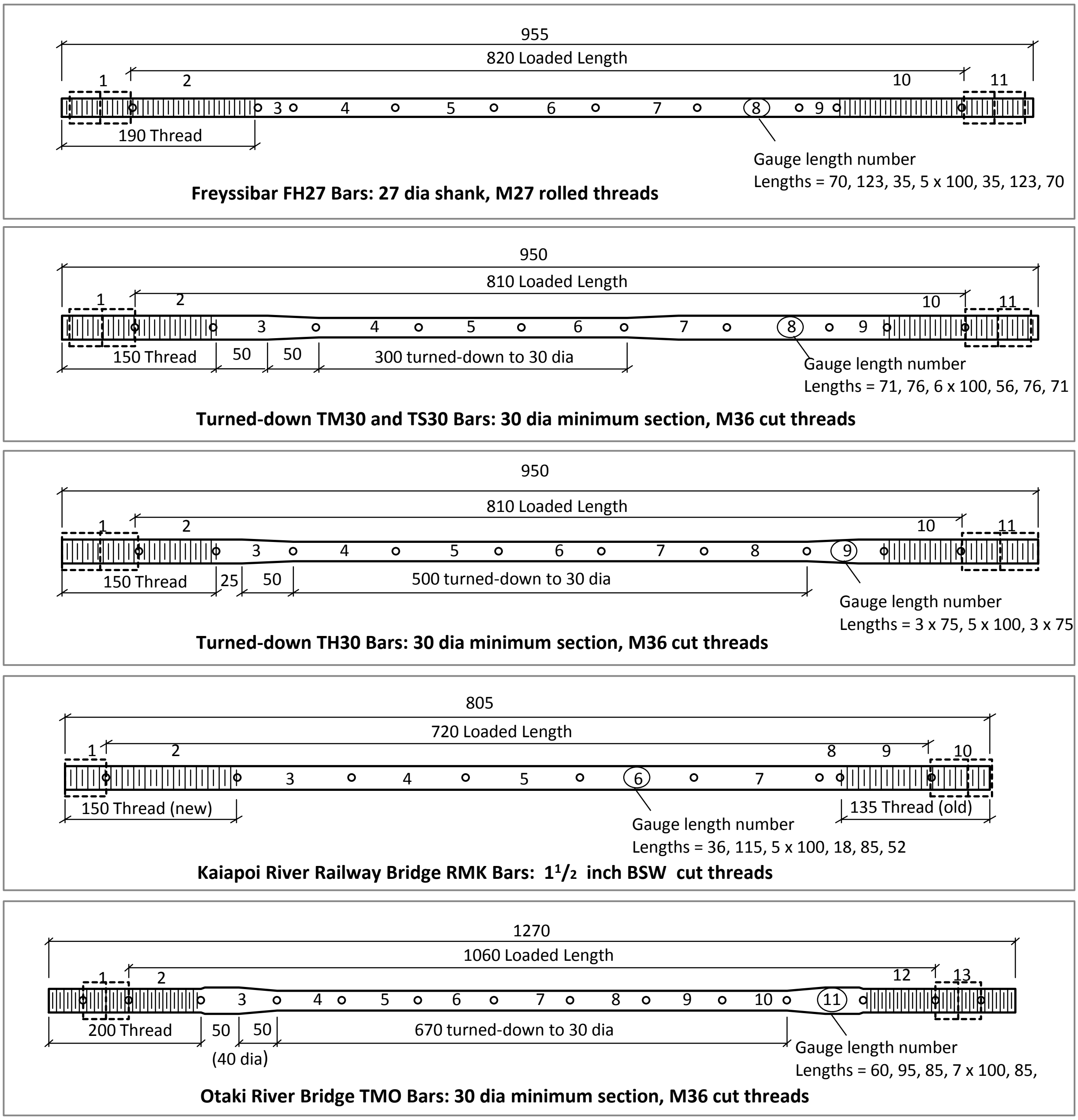

Figure 2: ***Dimensions of FH27, TM30, TS30, TH30, RMK and TMO, linkage assemblies and location of gauge lengths. All bars symmetrical about centre point except for TM30 and TS30 bars.***

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 British Standard Whitworth (BSW) cut thread to suit 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 1,270 mm with a 200 mm length of M36 thread at both ends.

3.3 Nuts and Washers

Details of the nuts and washers used in the test assemblies are summarised in Table 2.

Proprietary Bars: Proprietary nuts and locknuts were used on the four proprietary bar types. 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.

Table 2. Nuts and Washers Used on Test Assemblies

Identifier	Thread Type and (Pitch)	Nut Property Class	Nut Height x Width Across Flats	Washer Thickness x Diameter or Width
			mm	mm x mm
RB32	Ribbed bar (15 mm)	-	82 x 54	10 x 75 sq
RB25	Ribbed bar (12.5 mm)	-	65 x 46	6 x 75 sq
MS32	Rolled cont. (6 mm)	-	41 x 56	5 x 70 dia
RM36	M36 (4 mm)	8	37 x 59	3 x 72 dia
RS38	M36 (4 mm)	A4-70	28 x 54	5 x 70 dia
RS36	M36 (4 mm)	A4-70	28 x 54	5 x 70 dia
US36	M36 cont. (4 mm)	A4-70 A4-80[1]	28 x 54	5 x 70 dia
FH27	M27 Rolled (3.5 mm)	10	30 x 46	12 x 100 sq
TM30	M36 (4 mm)	8	37 x 59	3 x 72 dia
TS30	M36 (4 mm)	A4-70	28 x 54	5 x 70 dia
TH30	M36 (4 mm)	8	37 x 59	5 x 70 dia
RMK	$1^1/_2$ in. BSW (4.23 mm)	-	36 x 56	12 x 100 sq
TMO	M36 (4 mm)	-	30 x 59	19 x 165 sq

Note: 1. A4-80 nuts were used in the first test and A4-70 nuts used in the other two tests.

Mild and High Tensile Bars: Hot-dip galvanised M36 steel nuts of Property Class 8 were specified and used on the three 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 some of the tests. 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. 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 measured ultimate load for the RM36 and TM30 assemblies was 395 kN. The maximum measured ultimate load for the TH30 assemblies was 752 kN. The Property Class 8 nuts used in the testing would not be expected to fail at these maximum loads. All the RM36, TM30 and TH30 assemblies were tested with two standard nuts at both ends.

Stainless Steel Bars: Nuts from three different manufacturers were used on the stainless steel Grade 316 bars. It is understood that the nuts were imported from China and they were supplied with 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.

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.

Bars Recovered from Existing Bridges: Nuts were recovered with the linkage bars from the two bridges and were 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 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.

4. TESTING METHOD

4.1 Tensile Testing

The tensile testing of ten of the linkage assembly types was carried out using a 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.

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.

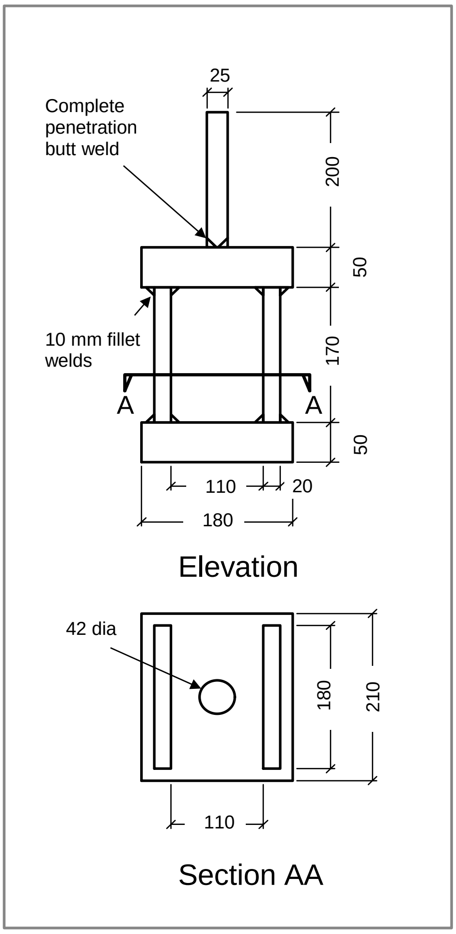

Figure 3: Loading frame for UTM testing.

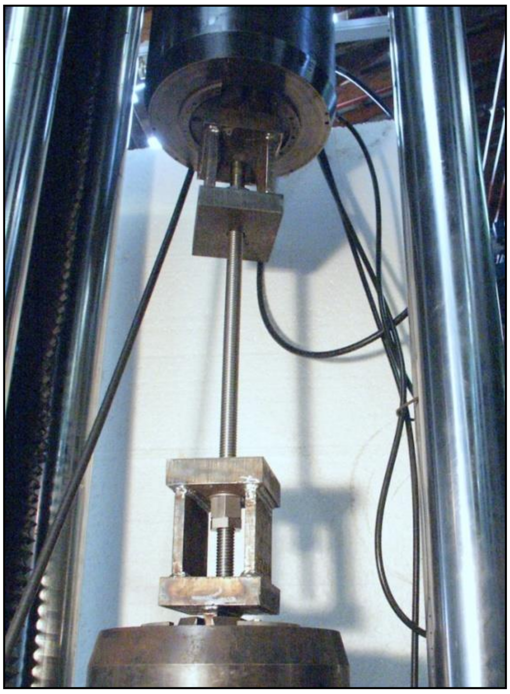

Figure 4: MS32 assembly in Servotest UTM.

Tensile testing of three assembly types was carried out using a 1 MN capacity centre-hole hydraulic jack mounted on a support frame anchored to a concrete strong floor. Load was measured with a special purpose electric-resistance load cell and the extension deformations of the bar with an 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. 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 using the UTM and between 22 to 38 kN per second for the bars tested with the 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 callipers. 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 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.

4.2 Bolt Head Tests

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 1960s 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 tensile tested using a 1 MN Shimadzu UTM and the loading frames previously used for the assembly tests in the Servotest 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 Figures 7 and 8.

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

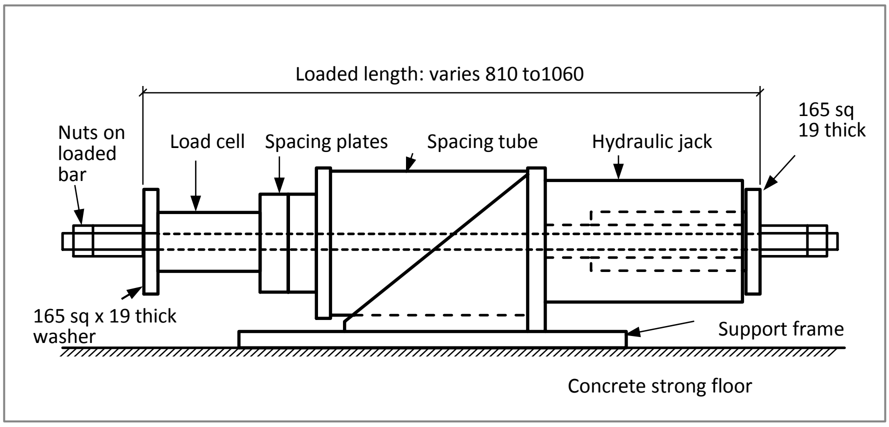

Figure 5: Elevation of hydraulic jack loading system and support frame.

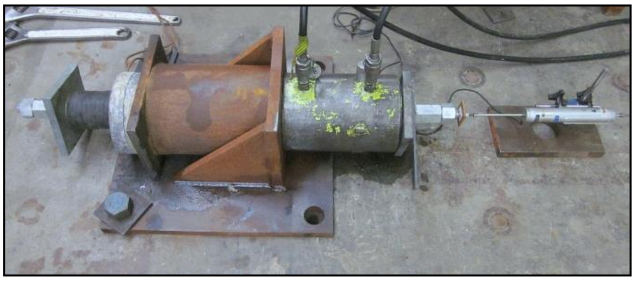

Figure 6: Hydraulic jack loading system set-up for Reidbar RB25 tests. Deflection measurement gauge is at the right end of the bar. Gauge was repositioned to be alongside the jack prior to bar failure.

4.3 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 (see Figures 5 and 6).

Figure 7: Specimen arrangement in Shimadzu UTM for testing bar heads on bars recovered from Kaiapoi Railway River Bridge.

Figure 8: Bar head test set-up in Shimadzu UTM.

5. TEST RESULTS

5.1 Load – Extension Plots

Figure 9 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. 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.

5.2 Yield and Ultimate Loads

Comparisons of the specified and measured yield and ultimate loads are given in Table 3.

TMO	190	180	339	324	1.8	0.96

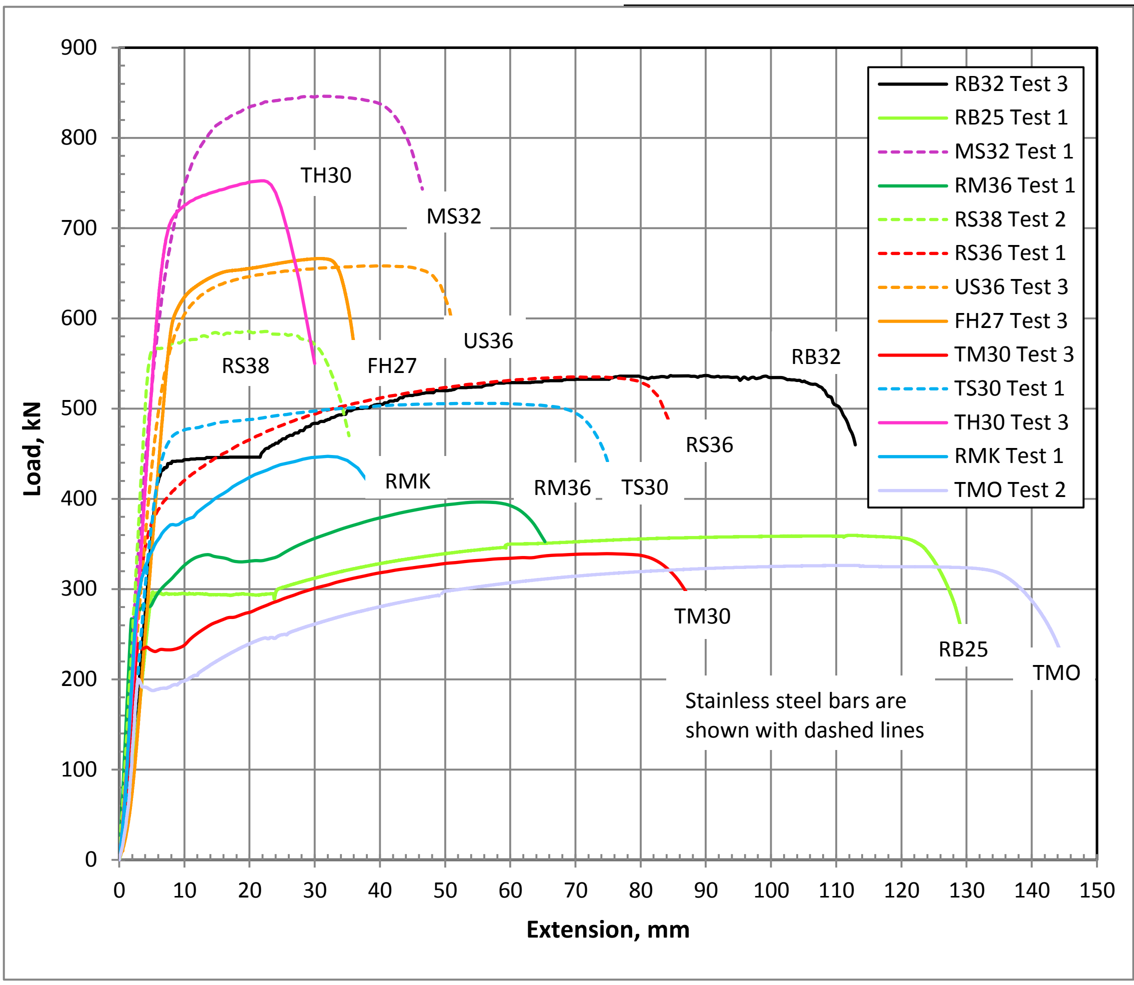

***Figure 9:** Comparison of load versus extension plots for the thirteen assembly types. (The loaded lengths varied from 620 mm to 1,060 mm – see Table 1.)*

Table 3. Specified and Test Yield and Ultimate Loads

Identi-fier	Minimum Yield or 0.2% Proof Load		Minimum Ultimate Load		*Test Ult. /Test Yield Load*	*Test Ult. /Spec Ult. Load*
	Spec kN	Test kN	Spec kN	Test kN		
RB32	402	436	462	531	1.2	1.15
RB25	246	290	282	356	1.2	1.26
MS32	506	650	622	847	1.3	1.36
RM36	253	240	417	387	1.6	0.93
RS38	399	490	526	587	1.2	1.12
RS36	266	350	493	535	1.5	1.08
US36	-	540	592	658	1.2	1.11
FH27	562	615	626	666	1.1	1.06
TM30	219	212	360	334	1.6	0.93
TS30	345	420	455	506	1.2	1.11
TH30	575	650	674	716	1.1	1.06
RMK	249	280	399	447	1.6	1.12

Only the Reidbars and mild steel bars had distinct yield levels (see Figure 9). 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.

5.3 Elongation Summary

A summary of specified and measured elongations is given in Table 4. 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. The “specified” elongation values given in Table 4 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 technical manuals. The TMO specified value is the minimum elongation specified for Grade 300 steel in AS/NZS 3679.1.

Table 4. Specified and Test Elongations

Identi -fier	Load-ed Length	Spec Elong -ation	Test Elong-ation on Failure Gauge Length	Test Elongation on Assembly Loaded Length Between nuts	
	mm	%	%	%	mm
RB32	620	20	26	17	107
RB25	880	20	26	14	124
MS32	660	15	14	6.1	40
RM36	810	34	31	7.9	64
RS38	810	47	28	4.1	33
RS36	810	51	43	10	81
US36	810	-	23	5.8	47
FH27	820	17	9.7	3.3	27
TM30	810	34	32	11	85
TS30	810	47	35	9.0	72
TH30	810	22	15	3.0	24
RMK	720	-	22	5.8	39
TMO	1060	30	33	13	140

Calculated elastic extensions at the yield force level of each test assembly varied from 1.2 to 5.0 mm with values less than 3.5 mm for all assemblies except for the assemblies fabricated from high tensile bar. These values include an allowance for the extensions in the threads under the nuts and were corrected for the estimated extensions in the loading frames. Measured elastic extensions 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. 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.

5.4 Location of Plastic Elongations

The location of the plastic elongations that occurred within the loaded lengths is summarised in Table 5. 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, the elongations on the turned-down length outside the failure zone are also presented (TH30 and TMO bars).

Table 5. Location of Measured Plastic Elongations

Bar Type	Elong. on Nut Zone	Elong. on 300 mm Long Failure Zone	Elong. on Thread Lengths	Elong. on Turned Down Length Outside Failure Zone	Elong. on Other Leng.
	% of Total Elong.	% of Total Elong.	% of Total Elong.	% of Total Elong.	% of Total Elong.
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 for all uni-form shank bars.	54	-	42
RS38	7		92	-	1
RS36	9		71	-	20
FH27	2		66	-	32
RMK	9		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 nut zone referred to in the table is the length of bar or machined thread covered by the nuts.

5.5 Comments on Elongations

5.5.1 *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 MS32 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. 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 5).

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 5).

5.5.2 *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 thread 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.

5.5.3 *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 down mild or stainless steel bar 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 5.6).

5.6 Bar Head Tests - Kaiapoi Railway River Bridge Bars

Failures occurred at the loaded face of the nut on one end of the bar as shown in Figure 10. 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 9). 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 10 and 11).

Figure 10: Failure in bar head test.

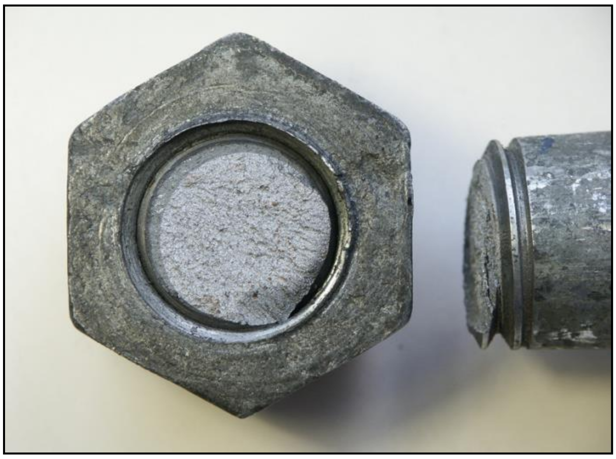

Figure 11: Failure at bar head on bridge.

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.

5.7 Nut Test Results

Results of the nut tests are summarised in Table 6.

5.7.1 *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 the 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.

5.7.2 *M36 Galvanised Steel Nuts*

Only one brand of nut was used on all of the mild and high tensile galvanised steel assemblies. Two of these nuts, one each end of a short bar, were test loaded in the centre-hole jack set-up used for the nut testing (see Figure 5). 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. Because the nuts were satisfactory for the assemblies tested it was considered unnecessary to undertake more comprehensive destructive testing.

5.7.3 *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. 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.

Table 6. Nut Strength Results

Nut Type	Nut Property Class (Manufacturer)	Nut Proof Load[1]	Assembly Types Nuts Used On	Max. Load Applied in Assembly Tests[2]	Max. Load in Assembly Tests Without Locknuts	Min. Ultimate Load In Nut Test	Nut Failure Description
		kN		kN	kN	kN	
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.

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 10). After both sets of tests the nuts could still be moved freely on the threads outside the loaded length.

5.7.4 *Stainless Steel Nuts*

All of the three different brands of stainless steel nuts were tested on short lengths of high tensile steel bar using 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 6. 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. Lock-nutting 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 12 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.

Figure 12: Failure of A4-70 (LE) nut in RS38 assembly test. Note the distortion across the width of the nut.

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.

6. FRACTURE TOUGHNESS

6.1 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).

6.2 Fracture Toughness of Steel Used in Test Assemblies

6.2.1 *Reidbar*

Pacific Steel Group (Roberts, 2010) 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.

6.2.2 *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 where a ductile performance is required.

6.2.3 *Grade 300 Mild Steel Bars*

Tables 2 and 5 in 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.

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.

6.2.4 *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.

6.2.5 *Freyssibar*

The test certificate supplied with the 27 mm diameter Freyssibar 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.

6.2.6 *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 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.

7. DESIGN ISSUES

7.1 Bridge Manual Requirements

The Bridge Manual (BM), (NZTA, 2003) 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 locations.

The two linkage bolts in the Kaiapoi Railway River Bridge that failed in the 2010 Darfield Earthquake were at a pier adjacent to the northern abutment of the 148 m long six-span bridge. 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.

7.2 Design Requirements

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 BM design response spectrum.

Linkage assembly design should be based on the provisions of the 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.

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.

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.

7.3 Linkage Bar Design Forces in Long Multi-Span Bridges

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.

Further details of this recommended analysis procedure for multi-span bridges can be found in the project report (Wood, 2012).

7.4 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 strength capacity of the pier increased by an overstrength factor of 1.5.

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.

7.5 Linkage Bar Design Details

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

1. 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.
2. The bars should have good ductility. 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.
3. Improving horizontal diaphragm action in the superstructure by concentrating pier linkage systems 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.
4. 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.
5. Both the strength of the linkage system and the span seating length should be considered when assessing the risk of spans falling. It is often more practical to provide more generous span/support overlaps than to provide greater linkage strength. With large seating lengths a less robust linkage system may be acceptable.
6. 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.
7. 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.
8. 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-down 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.

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.

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 and the threaded length should have the same nominal diameter as the shank.

Ductility and Geometry of Turned-Down Plain Bar

Turning down did not necessarily improve the tensile ductility of the bars by a large amount; 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 turned-down length should be a minimum of 10 times and ideally 15 times the turned-down diameter.

The turned-down diameter ratio 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.

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 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.

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. 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.

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