KAITOKE FLUME BRIDGE REPLACEMENT: USING SEISMIC RESILIENT DUCTILE IRON PIPE

Hayden Pipe (Stantec New Zealand), Todd Randell (Hynds Pipe Systems)

ABSTRACT

The Kaitoke Flume Bridge conveys raw water 20 metres above the Te Awa Kairangi River on its way to Te Mārua Water Treatment Plant. This plant provides the Wellington Region with up to 50% of its drinking water supply. The bridge is located in a geographically, geological and seismically challenging location and crosses numerous shear zones. It is therefore at risk to significant seismic forces.

The 50 metre long flume bridge is a 50-year old leaky concrete structure in poor condition. The bridge has poor seismic resilience and does not meet post-earthquake serviceability requirements for a critical supply.

A new network arch bridge with a DN1500 Seismic Resilient Ductile Iron Pipeline (SRDIP) has been designed to replace the existing flume bridge.

This paper outlines what SRDIP is, why SRDIP was selected as the pipe material for the bridge crossing, and how it is designed to behave in conjunction with the bridge in the design seismic events.

This paper also covers the pipe support system and restraint required to the pipeline due to seismic acceleration.

SRDIP originated in Japan and has been used there for many years. It suffered no documented failures in the 1995 Great Hanshin (Kobe) earthquake or the 2011 Great East Japan Earthquakes. The key aspects of the pipe that make it worth considering is the ability for every joint to deflect as well as contracting and expanding. This allows for a certain amount of differential movement between pipes without putting strain along the pipe barrel.

SRDIP was selected as it offers a robust system that allows for the differential movement along the pipeline that is expected in a large seismic event without the need for any other seismic or thermal joints. This was particularly important at the bridge ends where the superstructure is designed to slide on the abutments resulting in large deflections. These large deflections were allowed for by using the joint deflection as well as allowing the pipe to move transversely near the abutments.

KEYWORDS

Seismic Resilient Ductile Iron Pipe, SRDIP, Seismic Resilience, large pipeline, water supply.

PRESENTER PROFILE

Hayden Pipe is a Principal Civil Engineer at Stantec New Zealand (Stantec) and has been heavily involved in the Kaitoke Flume Bridge Design, with a role as Pipe and Civils Design Lead.

Todd Randell is a Business Development Manager working at Hynds Pipe Systems (Hynds). His role involves locating unique trenchless and specialist pipeline solutions for customers and asset owners.

1. INTRODUCTION

In 2020, Stantec undertook the detailed design of a new pipe and bridge crossing that would replace the existing Kaitoke Flume Bridge and concrete pipeline located in the Kaitoke Regional Park. The Kaitoke Flume Bridge crosses the upper reaches of the Te Awa Kairangi/Hutt River, with the surrounding area consisting of steep forest clad hills. The existing Flume Bridge, shown below in Figure 1, consists of a concrete flume with concrete deck footbridge above.

Figure 1 Existing Kaitoke Flume Bridge

The existing Flume Bridge is in poor condition, with several leaks, and does not meet current seismic requirements. The new bridge and pipeline were designed in accordance with the Waka Kotahi (NZTA) Bridge Manual, the client's (Wellington Water's) service requirements and relevant New Zealand design standards.

2. BACKGROUND

The Kaitoke Flume Bridge is a critical asset for Greater Wellington Regional Council (GWRC), conveying approximately 50% of the raw water supply to the Wellington regional bulk water supply network.

The existing network at Kaitoke was installed by the Ministry of Works as part of the water conveyance and treatment system. It starts with an intake on the river and flows from the Kaitoke intake through Tunnel 1 to the gravel trap, across the flume and through a sand trap before reaching the Strainer House, located on the southern side of the Kaitoke Flume Bridge.

Prior to installing the Te Mārua Water Treatment Plant (WTP), a chlorination house was located after the Strainer House, to protect against waterborne diseases such as cholera and meningitis. In 1987, the WTP was built downstream of Tunnel 2, to meet increasingly stringent water standards, and led to decommissioning the chlorination house.

An overview to the system is shown in Figure 2.

Figure 2 Te Mārua Raw Water Intake

The existing Kaitoke Flume Bridge consists of a 3-span concrete flume box structure and a gravel trap connecting to the tunnel portal on the northern bank abutment. The bridge is supported on three high concrete piers, located either side of the Te Awa Kairangi River. The existing flume construction was completed in 1954 and strengthened in 1992 to improve the overall stability of the structure. Figure 3 shows the construction and current condition of the original Flume Bridge, including the strengthening works carried out in 1992. Figure 4 shows the gravel trap at the north abutment that connects to the Tunnel 1 portal, which conveys flows from the intake weir on the other side of Tunnel 1.

Figure 3: Left photo from the construction of the original bridge, Right photo current condition of the bridge

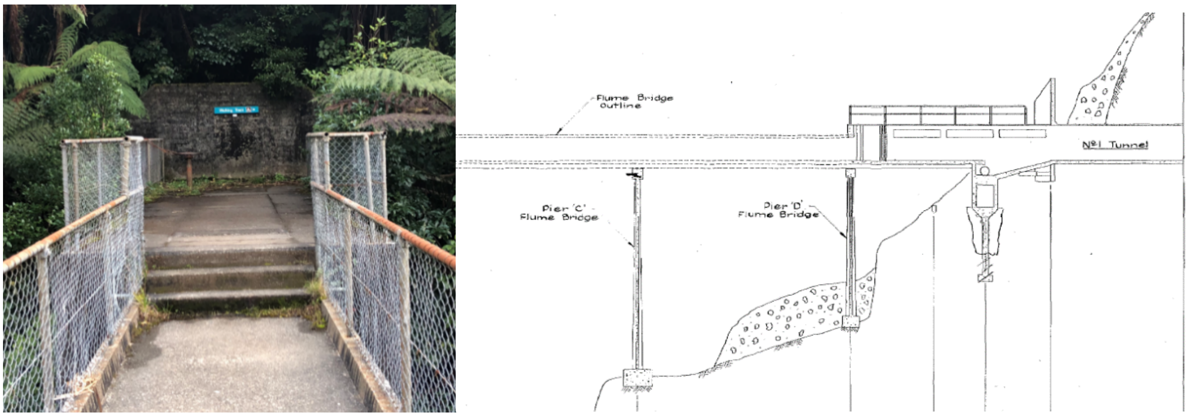

Figure 4: The gravel trap that connects the flume bridge to the Tunnel 1 portal

3. SEISMIC DESIGN REQUIREMENTS

The new bridge and components were designed to comply with the requirements of the Waka Kotahi Bridge Manual for a 100-year design life and IL3 structural designation. The bridge is designed to a Damage Control Limit State Earthquake associated with a return period of 1 in 1000 years. The seismic performance objectives are:

- Serviceability Limit State (SLS) Earthquake: Operation functionality of the pipe is maintained. No damage to the primary structural elements, and minimal damage to secondary elements.

- Damage Control Limit State (DCLS) Earthquake: Operational functionality of the pipe is maintained. Access might be compromised but is feasible to be recovered in three days. Structural damage might be significant but repairable to the extent sufficient to avoid collapse under a repeat design level earthquake. Permanent differential displacement between the abutments and the bridge deck is acceptable, provided the bridge can be relevelled and re-seated.

- Collapse Avoidance Limit State (CALS): Operational functionality of the pipe is compromised. Collapse is prevented but extensive damage to the structure is expected. Large permanent gap expected between the abutments and the bridge deck.

4. PROPOSED BRIDGE ALIGNMENT

The new Network Arch Bridge will carry a DN1500 pipeline on the bridge deck, with a walkway above the pipe, which will also provide access for a 2.5 tonne access hoist for maintenance and inspection for the bridge and pipeline. The new bridge is proposed to run parallel and upstream of the existing concrete flume bridge (which will be removed) as shown in Figure 5. An artist's impression of the final bridge design is shown in Figure 6.

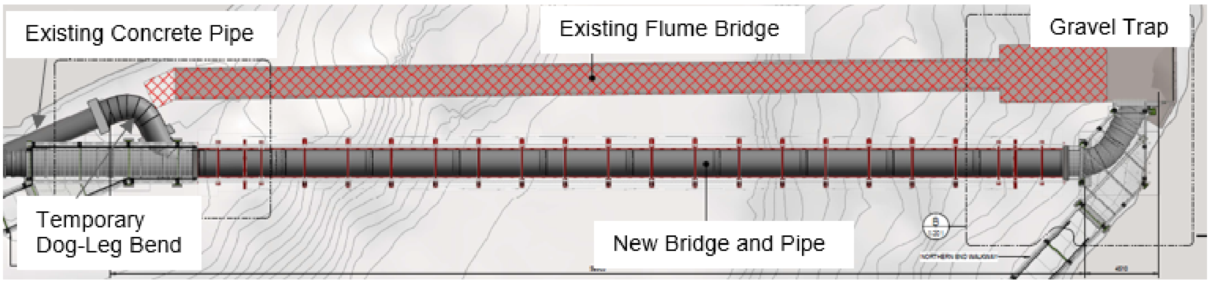

Figure 5 Proposed Bridge Alignment

Figure 6 Artist's Impression of the Final Bridge Design

5. SELECTING SEISMIC RESILIENT DUCTILE IRON PIPE

5.1 MATERIAL SELECTION

A materials assessment was undertaken to determine the material and jointing system for the pipe section crossing the bridge, considering seismic resilience, operational resilience, durability, longevity, ease of installation, maintenance, safety and cost. The assessment considered the following materials:

- Glass Reinforced Plastic (GRP)
- High Density Polyethylene (HDPE)
- Carbon Steel (CS)
- Seismic Resilient Ductile Iron (SRDIP)

SRDIP was selected as the main pipe material crossing the bridge due to the following reasons:

- Good seismic resilience properties due to the allowable elongation and contraction at each joint (+/- 60mm); and large allowable rotation at the pipeline joint (5.83 degrees) which can accommodate a large displacement between abutments.
- Good thermal expansion properties due to allowable movement in joints.
- The joints have high resistance to pull-out force.
- SRDIP eliminates the need of expensive mechanical movement joints at the abutments.
- Good rockfall and vandalism resistance

- Joint system does not require welding, except where custom made steel specials are used.

SRDIP is commonplace in Japan and more recently in seismically prone areas in the USA. It has been documented that during the 1995 Great Hanshin (Japan) earthquake, there were 1,757 pipe repairs within the City of Kobe water system. At the time of the earthquake, about 5% of the pipe inventory in Kobe were SRDIP, and these apparently suffered no damage (American Lifelines Alliance: Seismic Guidelines for Water Pipelines, 2005). There is also no evidence of failures during the 2011 Great East Japan Earthquakes where this material was used.

5.2 SEISMIC RESILIENT DUCTILE IRON PIPE

5.2.1 BACKGROUND

SRDIP is highly rigid and offers unique joint characteristics such as an expansion and contraction mechanism, large allowable joint deflection and slip-out resistance. When a large deformation occurs, after one joint extends to the limit, the next pipes are pulled one after the other to allow for more movement and to maintain water supply functions. This mechanism is known as "chain-link-structure"; Figure 7 shows a 6 link chain fully extended and the same chain compressed. "Chain-link-structure" SRDIP allows compression and extension to occur simultaneously during seismic wave ground-movement.

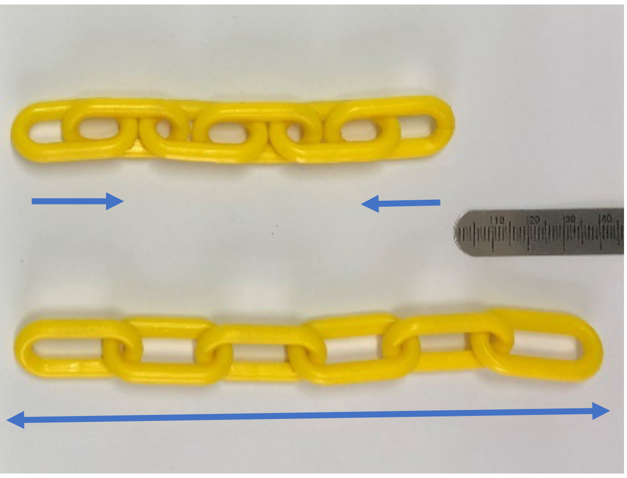

Figure 1: Plastic chains compression-extension showing "chain-link-structure" (Source: T. Randell)

SRDIP utilises this extension and compression feature within its centrally located pipe-spigot in its very deep pipe socket.

It has been 45 years since SRDIP was first installed in Japan, in 1975. The product has since experienced a large number and a wide range of natural disasters with no known failures during many catastrophic earthquakes up to M9.0 and the large liquefaction events that followed severe ground-movement.

SRDIP has proven to be the most effective in areas such as:

- Near the fault-lines
- Liquefaction grounds such as landfill
- Artificially deformed areas such as road embankment

- Pipelines to critical water supply points (hospitals, shelters, public facilities)
- Pipelines for fire hydrant water supply
- Pipelines that are difficult to repair, such as bridge-mounted pipelines

The product is manufactured to the Japanese Industrial Standards (JIS) JIS-G 5526 Ductile Iron Pipe and JIS G-5527 Ductile Iron Fittings, as well as the JWWA standards. Its seismic performance also sufficiently satisfies the International Organization for Standardization (ISO) criteria.

5.2.2 PRODUCT RANGE

Table 1 summarises the types of pipe/fitting and joint systems used for seismic resilient pipelines. Type-NS is widely employed for DN1000 or less. For DN1100 and larger, Type-S is used for straight lines and Type-UF at each end of the fittings.

Joint system	Size range and types of pipe/fitting	
Type NS	DN	75～1000
	Pipe	Class of thickness : 1, 3 (available for DN75～450) Class of thickness : S (available for DN500～1000)
	Fitting	Collar, bend, tee, reducer, etc.
Type S	DN	1100～2600
	Pipe	Class of thickness : 1, 2, 3
	Fitting	Collar
Type UF	DN	1100～2600
	Pipe	Class of thickness : PF
	Fitting	bend, tee, reducer, etc.

Table 1: Types of pipe/fitting and joint systems for SRDIP (Source: JDPA)

Expansion/contraction

Type-NS pipe joint (without liner) and Type-S pipe joint are of “chain-link-structure” utilising expansion/contraction, angular deflection and slip-out resistance functions. Figure 7 to Figure 8 show the profiles of joint structures of Type-NS and Type-S pipes.

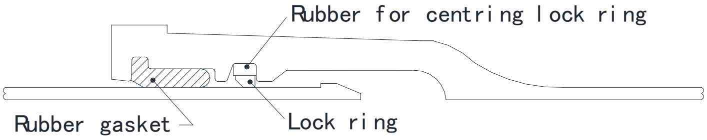

Figure 7 : Joint structure of Type NS pipe joint (DN75~450) (Source: JDPA)

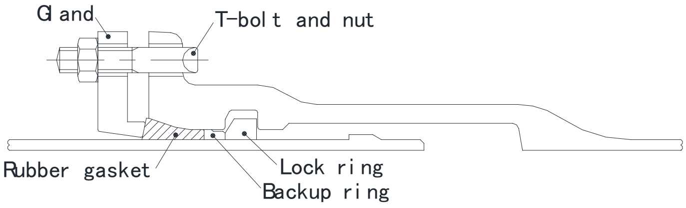

Figure 8: Joint structure of Type NS pipe joint (DN500~1000) (Source: JDPA)

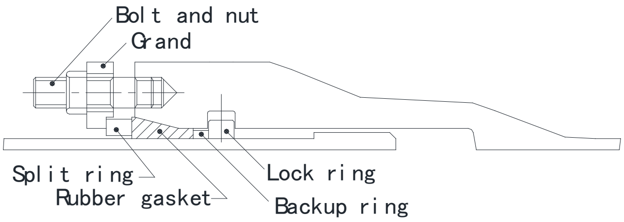

Figure 9: Joint structure of Type S pipe joint (DN1100~2600) (Source: JDPA)

Slip-out resistance

Type-NS fitting joint other than collar, type-NS pipe joint using liner, and type-UF joint also offer slip-out resistance capability to prevent the movement of fittings caused by water pressure thrust forces.

(1) Type NS fitting

Figure 10 to Figure 12 show the profiles of joint structures of fittings.

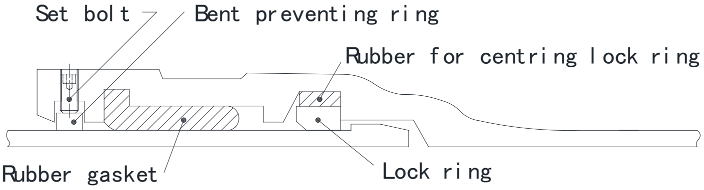

Figure 10: Structure of Type NS fitting joint (DN75~250) (Source: JDPA)

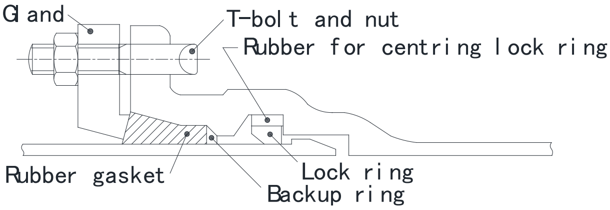

Figure 11: Structure of Type NS fitting joint (DN300~450) (Source: JDPA)

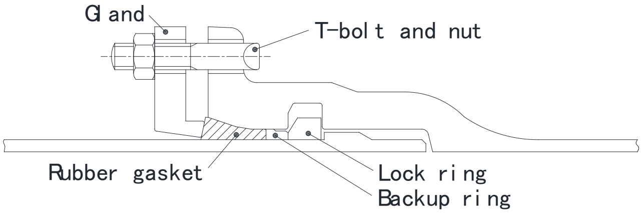

Figure 12: Structure of Type NS fitting joint (DN500~1000) (Source: JDPA)

(2) Type NS pipe joint using a liner

It is necessary to use a liner for the pipe socket to be joined with type-NS fitting spigot in-order-to maintain its slip-out resistance function. The joint structures using a liner are shown in Figure 13 and Figure 14.

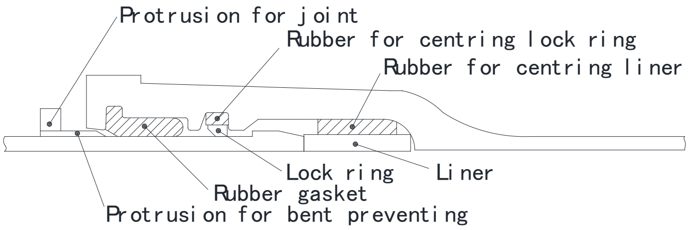

Figure 13: Structure of Type NS pipe joint using a liner (DN75~450) (Source: JDPA)

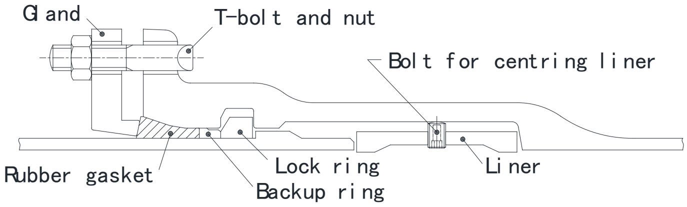

Figure 14: Structure of Type NS pipe joint using a liner (DN500~1000) (Source: JDPA)

(3) Type UF pipe and fitting

Figure 15 shows the joint structure of Type-UF pipes and fittings. Since Type-S does not have pipes using a liner or fittings other than collar, it is necessary to apply Type-UF pipes or fittings within the integrated length range.

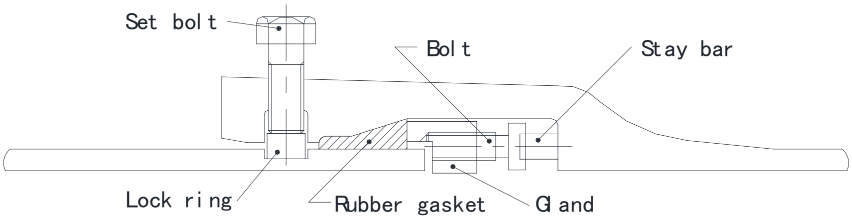

Figure 15: Structure of Type UF pipe and fitting (DN1100~2600) (Source: JDPA)

5.2.3 TECHNICAL ATTRIBUTES

The specifications of SRDIP joints are presented in Table 2. Its expansion/contraction and slip-out resistance functions are classified as class S-1 and class A respectively, based on ISO 16134: 2020 (Table 3); the highest ranked earthquake-resistant joints.

DN	Max. expansion/contraction amount when straight (mm)	Slip-out resistance (kN)	Allowable bending angle when piping	Max. bending angle during an earthquake or subsidence	Limit bending moment (kN·m)	Limit water pressure (bar)
75	±45.5	225	4°	8°	4.4	-
100	±45.5	300	4°	8°	7.4	-
150	±60	450	4°	8°	17	-
200	±60	600	4°	8°	24	-
250	±60	750	4°	8°	35	-
300	±69	900	3°	6°	64	-
350	±70	1050	3°	6°	81	-
400	±71	1200	3°	6°	130	-
450	±73	1350	3°	6°	170	-
500	±75	1500	3°20'	7°	360	-
600	±75	1800	2°50'	7°	540	-
700	±75	2100	2°30'	7°	820	-
800	±75	2400	2°10'	7°	1180	-
900	±75	2700	2°00'	7°	1630	-
1000	±80	3000	1°50'	7°	2010	-
1100	±78.5	3300	1°40'	7°	2600	72
1200	±78.5	3600	1°30'	7°	3140	71
1350	±78.5	4050	1°30'	6°30'	4360	66
1500	±81	4500	1°30'	5°50'	5150	56
1600	±72.5	4800	1°30'	5°	6670	60
1650	±72.5	4950	1°30'	4°50'	7310	60
1800	±75	5400	1°30'	4°40'	9270	59
2000	±77.5	6000	1°30'	4°20'	12600	58
2100	±80	6300	1°30'	4°10'	14000	56
2200	±80	6600	1°30'	4°	16100	55
2400	±82.5	7200	1°30'	3°50'	20300	55
2600	±82.5	7800	1°30'	3°40'	32300	68

Table 2: SRDIP joint specifications (Source: JDPA)

Parameter	Class	Component performance
Expansion/contraction performance	S-1	±1% of *L* or more
	S-2	±0.5 % to less than ±1 % of *L*
	S-3	Less than ±0.5 % of *L*
Slip-out resistance	A	3 *d* kN or more
	B	1.5 *d* kN to less than 3 *d* kN
	C	0.75 *d* kN to less than 1.5 *d* kN
	D	Less than 0.75 *d* kN
L is the component length, in millimetres (mm)		
d is the nominal diameter of pipe, in millimetres (mm)		

Table 3: ISO classification of seismic pipeline performance (Source: ISO16134)

6. DESIGN ELEMENTS

6.1 BRIDGE SUPPORTS

The pipe will be supported on the bridge deck in cradles at two points along each section of pipe to give the pipe the stability required as per the manufacturer’s recommendations. The supports have been designed to be located one metre away from each joint to ensure that if the pipes do move within the joints, the joint will not clash with any of the support brackets.

A low friction product (Ultra high molecular weight polyethylene (UHMWPE)) will be placed between the support cradles and the pipe to reduce the axial force, as much as possible, and the torsion exerted on the bridge members if the pipe moves longitudinally.

The bridge superstructure has been designed to be able to slide both in the longitudinal and transverse directions on the abutments. Because of this, the pipeline joints need to be able to move within these limits. To enable this, the supports under the pipe nearest the abutments at both ends are free to slide on the bridge. The superstructure transverse movement on the abutments is restricted by a lip on the abutment so that it does not go beyond the allowable deflection of the pipe joints.

6.2 THRUST RESTRAINT

The hydrostatic force on the thrust restraints is small as the pipeline is not under pressure, however the seismic force of the pipeline is much larger. The seismic force is generated by the pipes and water inside accelerating in the direction of the earthquake. As the joints are allowed to move and there is a lower friction interface between the pipe and the bridge, the anchor blocks have been designed to withstand the force of this pipe acceleration.

6.3 SEISMIC PIPE DESIGN

As discussed above, the DN1500 SRDIP joints are designed to contract and expand +/- 60mm and rotate up to 5.83°. This allowable movement and rotation results in the pipe being able to withstand both the differential movement between abutments but also the bridge superstructure moving independently of the abutments.

The following design scenarios have been allowed for assessing the suitability of the SRDIP for the following differential movements:

- Abutment movement – longitudinal
- Abutment movement – transverse
- Bridge superstructure movement (i.e. the abutments remain stationary and the bridge structure slides on the abutment pads)
- A combination of the above movements

6.3.1 ABUTMENT LONGITUDINAL MOVEMENT

Modelling of the 100% Collapse Avoidance Limit State (CALS) event results in the northern abutment moving south 180mm and the southern abutment moving south 31mm. The abutment movement is therefore expected to result in the two abutments becoming 149mm closer. There are 11 pipe joints on the bridge with an allowance to contract or expand up to 660mm in total. Therefore, the joints can accommodate this movement without putting large stresses on the pipe barrel and other structures. It is expected that the pipe joints at either end will accommodate most of this movement, the southern side will lengthen and the northern side contract, and the joints in the middle of the bridge will experience very little movement. The pipe supports have been designed to withstand the force induced by the friction between the pipeline and the supports.

6.3.2 ABUTMENT TRANSVERSE MOVEMENT

Modelling of the 100% CALS event results in the northern abutment moving 60mm in the northwest direction and the south abutment moving 170mm in the opposite direction resulting in 230mm differential transverse movement. This movement will need to be accommodated by the two pipe joints located at the very end of the bridge where the differential movement occurs between the bridge structure and the abutment. The pipe length between the joints at each abutment is 53m, the 230mm differential transverse movement between abutments will result in a rotation of the joints at each abutment of 0.25° which is well within the tolerance of the 5.83° allowable rotation of the joints.

6.3.3 SUPERSTRUCTURE MOVEMENT ON ABUTMENTS

Drilling the abutment piles to the diameter required to resist the design seismic load is not feasible. This is because the equipment required to drill these large piles is too heavy for the three existing bridges that access the site, even when dismantled. Therefore, the bridge has been designed so that the superstructure can slide on the abutments to limit the seismic forces applied to the smaller piles. To allow for this movement to occur, the supports under the pipe section nearest to each abutment are not fixed to the bridge deck which means they can slide on the bridge deck if the superstructure moves on the abutments as shown in Figure 16 and Figure 17.

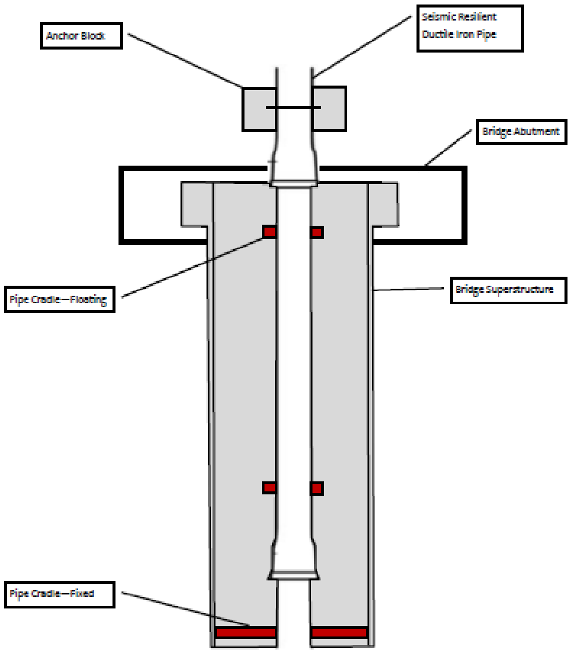

Figure 16 Superstructure Initial Position (NTS)

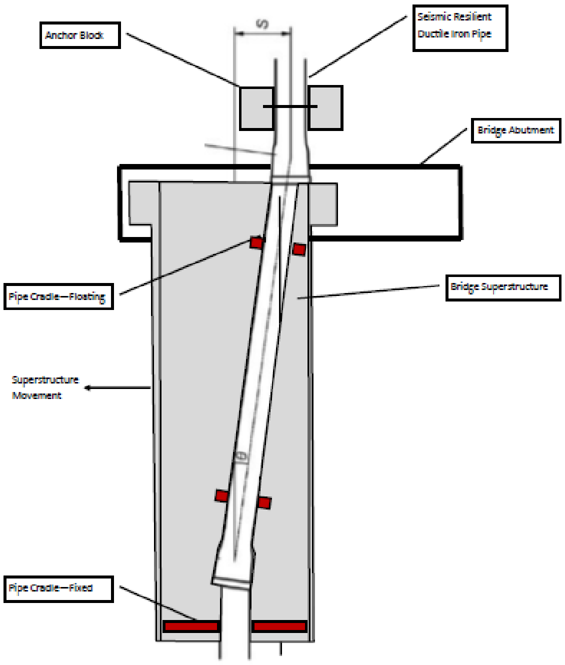

Figure 17 Superstructure Position following movement induced by large seismic event (NTS)

For the SLS event, the performance of the bridge is expected to be in the elastic range with a 91mm differential movement between the bridge superstructure and the abutment. This will result in a 0.98° rotation of the two pipe joints closest to the abutments which is well within the limit of the pipe joint allowable rotation.

For the DCLS event, the bridge might show some permanent deformation, but minimal damage is expected. There is expected to be 363mm of differential movement between the bridge superstructure and the abutment. This will result in a 3.92° rotation of the pipe joints closest to the abutment which is within the limit of what the pipe can rotation.

For the CALS event, where damage might be extensive but repairable, there is expected to be 545mm differential movement between the bridge superstructure and the abutment. This will result in a 5.87° rotation of the pipe joints closest to the abutment which is right on the limit of what the pipe can rotate so repairs may be necessary following the event. The floating pipe support is also expected to clash with the lip on the bridge deck which will also require repair. This is a non-critical structural element of the bridge. Allowing more pipes to float on the bridge to avoid this clash was considered but would generate high, unmanageable seismic forces on the bridge during a large event which could cause significant damage to the bridge structure.

6.4 CONNECTIONS TO EXISTING SYSTEM

The connections to the existing network at the project extents requires custom sized bends. It is only possible to get SRDIP bends and fitting in standard sizes and this sizing does not fit the requirements of this project. Therefore, custom made carbon steel specials are required as shown in Figure 18.

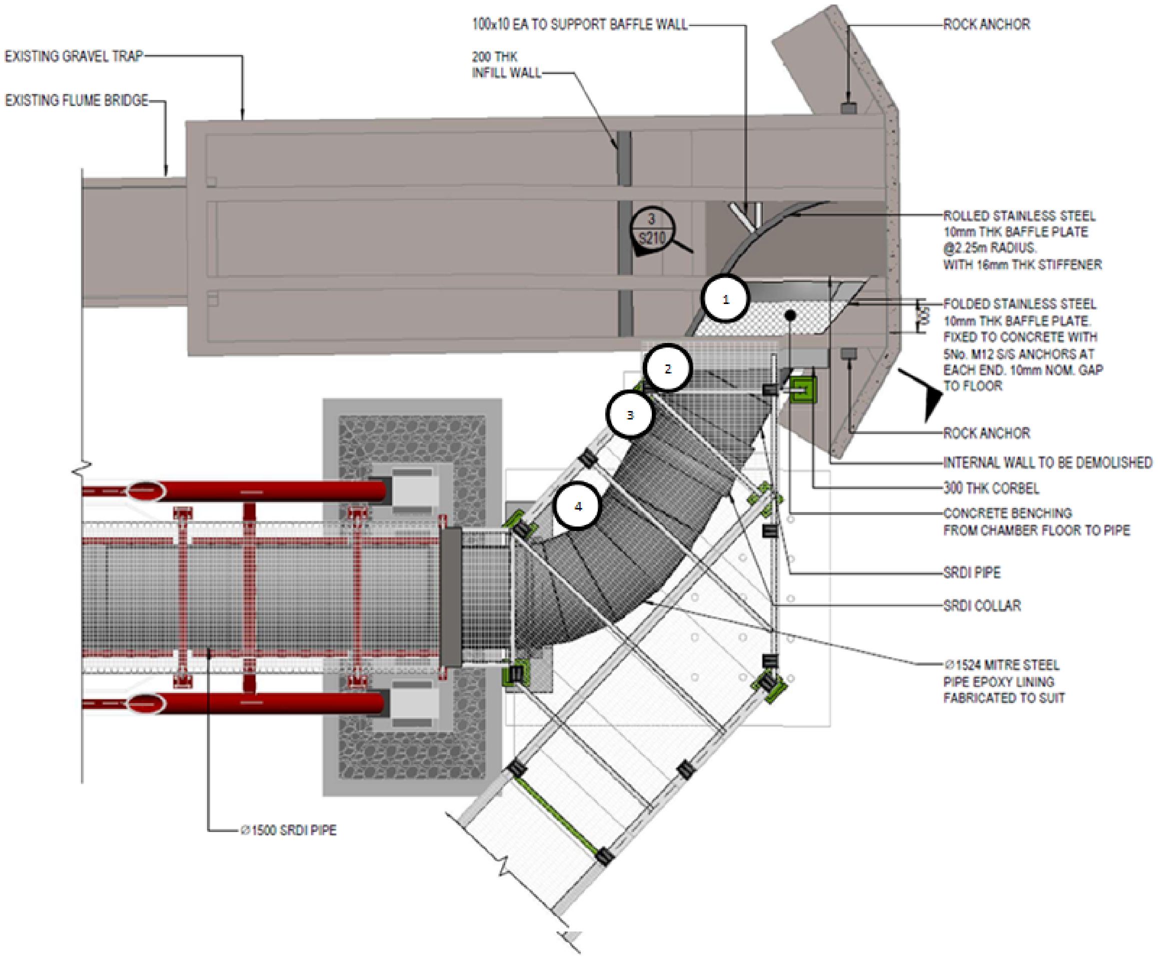

Figure 18 North Bank Connection

SRDIP is made in Japan to Japanese pipe standards. This means that it is not compatible with New Zealand steel diameters. There are two options to work around this issue:

- Manufacture the custom steel specials in Japan

- Use steel transition pieces that can connect to SRDIP at one end and can be welded to New Zealand made steel at the other end.

In conjunction with the contractor, the project team chose to use Japanese made steel due to:

- Cost savings
- Additional room to make a smoother bend
- Fewer fittings and joints

CONCLUSIONS

- The SRDIP system may be an effective pipe material to use on a bridge where differential seismic movement needs to be allowed for.

- For Kaitoke, the pipe system is designed to remain undamaged in a SLS and DCLS event and may suffer minor, repairable damage in a CALS event.

ACKNOWLEDGEMENTS

We would like to acknowledge all the members of the project design team and other parties that assisted in the design, including Wellington Water Ltd, Greater Wellington Regional Council, Holmes Consulting Ltd, Stantec, Hynds Pipe Systems, Kurimoto Ltd, Brian Perry Civil Ltd.

REFERENCES

American Lifelines Alliance: Seismic Guidelines for Water Pipelines, March 2005