**13th World Conference on Earthquake Engineering**
**Vancouver, B.C., Canada**
**August 1-6, 2004**
**Paper No. 3160**

# SEISMIC SAFETY RETROFITTING DESIGN FOR THE SOOKE RIVER BRIDGE, VANCOUVER ISLAND, BC

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## SUMMARY

The Sooke River Bridge is located at the southern tip of Vancouver Island and on the north shore of Juan de Fuca Strait. The bridge site is in Seismic Performance Zone 4, with a Peak Horizontal Ground Acceleration (PHA) of 0.303g for the 1-in-475-year design earthquake. The bridge carries the two traffic lanes of Highway 14 and two sidewalks, and is classified by the British Columbia (BC) Ministry of Transportation as a Disaster Response Route Bridge. The existing bridge structure is a hybrid construction, comprising a steel tied-arch 61.5 m long crossing over the main channel of the river on the west, and an approach structure of three-span-continuous steel plate girders on the east. With the exception of the West abutment which is supported on a spread footing, all spans are supported on reinforced concrete piers and abutments which in turn are supported on battered steel H-piles. The bridge was designed and constructed in the middle 1960s with no consideration of earthquake-induced loads. As part of the BC Seismic Risk Reduction Program, this bridge is currently being seismically rehabilitated to safety level standards. The initial seismic assessment was carried out by performing an elastic dynamic analysis using a three-dimensional SAP 2000 finite element model. The effects of soil-pile interaction were investigated by incorporating upper and lower bound foundation spring stiffnesses at piers and the West abutment, and non-linear spring stiffness for the East abutment. Non-linear static push-over analysis was carried out to obtain the displacement ductility capacity of the pier walls in the weak direction. The analysis takes into account a number of key factors, including non-linear constitutive models for concrete and steel reinforcement, details of lap splices, and foundation flexibility. Significant analysis and design efforts were undertaken to optimise the overall bridge response and the distribution of earthquake effects to various bridge components, in order to achieve a cost-effective retrofit strategy. Geotechnical challenges included pier foundation soils that were susceptible to liquefaction to a depth of 5 m, and irregular support conditions at one of the piers. This paper examines both design and construction challenges of the seismic retrofitting which consists of strengthening two of the pier foundations, replacing the bearings on all piers, installing seismic restrainers on both abutments and employing ground densification around all three piers.

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## INTRODUCTION

The Sooke River Bridge No. 396 was designed by the British Columbia (BC) Department of Highways in 1965, and the construction was completed in 1967. The existing bridge was not designed to the current seismic design standards.

The bridge carries the two 4.27 m (14 ft) wide traffic lanes of Highway 14, and two 1.5 m wide sidewalks, over the river near the Town of Sooke, which is located to the west of the City of Victoria on the southern tip of Vancouver Island. The bridge is a key link between the Town of Sooke and the City of Victoria. The BC Ministry of Transportation (MoT) designated this bridge as a Disaster Response Route Bridge in February 2000. In the fall of 2002, ND LEA Consultants Ltd. was retained by MoT to carry out a detailed seismic assessment and, if required, to provide a final design for seismically upgrading the structure to a "safety" level of retrofit such that the bridge can withstand the design earthquakes without collapse.

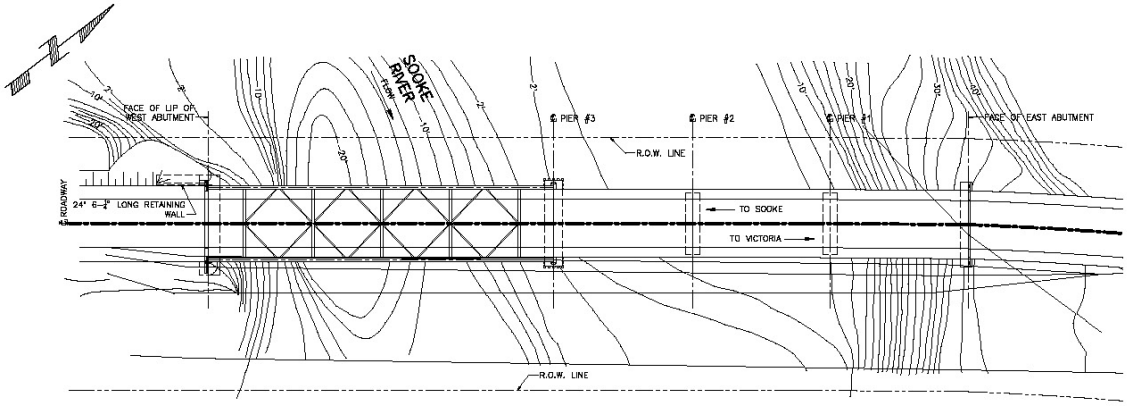
The seismic assessment and retrofit design was carried out in accordance with the current MoT Seismic Retrofit Design Criteria [1] with the following prescribed levels of performance to be met:

- **Retrofit Level**: Safety – to prevent the bridge from collapsing during the design earthquake (an earthquake with a return period of 475 years);
- **Service Level**: Significantly Limited – it is expected, but not guaranteed, that limited access to light emergency traffic is possible within days following the earthquake but no public access is possible until repairs are completed;
- **Damage Level**: Significant – no risk of collapse, but damage that would require closure to repair. Partial or complete replacement may be required depending on the extent of damage.

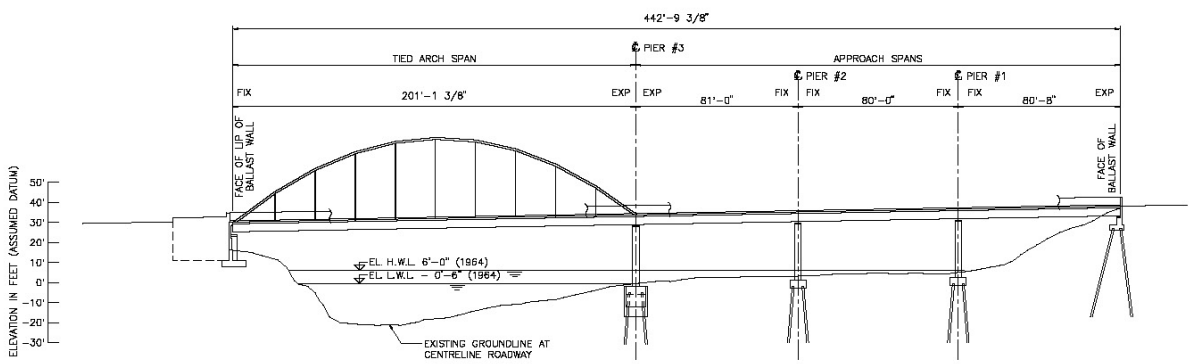
For this project, the MoT's geotechnical engineers have provided the geotechnical investigation and foundation assessment. The results of the geotechnical investigation are documented in the MoT Geotechnical Report for the Sooke River Bridge [2], and the findings of the seismic assessment of the existing bridge including various retrofit options investigated are presented in the Seismic Safety Retrofit Final Report for the Sooke River Bridge [3]. The engineering design for the seismic retrofitting has been completed and the construction is currently underway.

## DESCRIPTION OF EXISTING BRIDGE

The Sooke River Bridge, as shown in Figure 1, is a four-span structure, comprising a steel tied-arch crossing over the main channel of the river on the West, and an approach structure constructed of steel plate girders on the East. The steel tied-arch has a single span of 61.5 m (201 ft) length, and the approach structure has three spans of 24.7 m (81 ft), 24.4 m (80 ft), and 24.5 m (80.6 ft), respectively. All spans are supported on reinforced concrete piers and abutments, which in turn are supported on steel H-piles, except for the West abutment, which is supported on a spread footing. After more than 36 years in service, the bridge super- and sub-structures are still in relatively good condition. No major structural rehabilitation has been done, other than the re-surfacing of the bridge deck in 1977, providing increased concrete cover to the deck reinforcement.




**(a) Plan**




**(b) Elevation**

**Figure 1: General Arrangement of the Sooke River Bridge**

The tied-arch span is supported on four steel bearings: two steel fixed bearings on the West abutment and two rocker expansion bearings on Pier 3. Both types of bearings have performed poorly during past earthquakes. Transverse restrainers or shear keys were not originally provided at the bearing locations.

The approach structure consists of three spans of continuous steel plate girders with a cast-in-place concrete deck. Steel rocker bearings are used at the ends (the East abutment and Pier 3), and steel fixed bearings are used on intermediate supports (Piers 1 and 2). Again, no transverse restrainers or shear keys were originally provided at the bearings.

The West abutment is a reinforced concrete cantilever wall on a spread footing, which appears to be constructed on an excavated surface of bedrock. It also has a monolithic retaining wall on the north side. The East abutment is a reinforced concrete bank seat type and is supported on two rows of battered steel H-piles.

All three piers were built as reinforced concrete pier walls supported on two rows of steel H-piles. Piers 1 and 2 are nearly identical except in terms of the as-built pile locations and lengths. Pier 2 has a total of 14 piles installed as per design with pile length varying from 15.85 m (52 ft) to 16.46 m (54 ft). However, due to an outcropping of bedrock in the vicinity of Pier 1, one corner of the Pier 1 pile cap was founded

directly on bedrock and a total of 15 piles were driven to bedrock with pile length varying significantly, from shorter piles of 2.59 m (8.5 ft) in length to longer piles of 14.94 m (49 ft) in length.
Pier 3, supporting both arch and steel stringer spans, has a thicker and wider pier wall than Piers 1 and 2. The piled foundation was built using a cofferdam of steel sheet piles, which were cut off at the top of pile cap elevation with the remainder left in place. The foundation was excavated to 1.83 m (6 ft) below the base of the pile cap and this over-excavation was filled with tremie concrete. A total of 28 steel H-piles were installed with pile length varying from 10.97 m (36 ft) to 13.11 m (43 ft).

Under existing conditions, the tied-arch and the approach spans are both supported on the rocker bearings on top of Pier 3, without any structural connection to the pier. Consequently, the bridge superstructure is very vulnerable to unseating at this tall pier, and seismically the Pier 3 substructure makes little or no contribution to the overall seismic resistance of the bridge.

**Figure 2: Rocker Bearings on Pier 3**

It should be noted that although all steel H-piles were driven to bedrock, the pile driving records seem to indicate that the pile driving was stopped as soon as the pile tip had reached bedrock. Therefore, the uplift capacity of the piles is very low.

## GEOTECHNICAL ASSESSMENT AND SOIL SPRING MODELS

Geotechnical investigation and assessment of the sub-soil conditions for the Sooke River Bridge was carried out by the MoT's geotechnical engineers as part of the current seismic safety retrofitting assignment. It was found that there is a high risk of liquefaction of the recent sand deposits in the flood plain of the Sooke River. Liquefaction around the piers may result in flow slide forces exerted on the pier foundations. The existing pier foundation piles were also found to have insufficient capacity to support earthquake induced forces.

Because of uncertainty in predicting the soil-pile interaction under the liquefiable soil conditions, it was decided that analyses using upper and lower bound soil stiffnesses should be performed to determine the sensitivity of seismic response to the soil conditions. The upper bound soil springs at pier locations represent the initial (prior to liquefaction) soil condition and the lower bound values reflect conditions

when pore pressures have risen to approximately 75% of liquefaction levels. In both cases, foundation spring coefficients were calculated for both translation and rotation in all three axes.

It should also be noted that the asymmetrical configuration of the West abutment has complicated the behaviour of the abutment under earthquake loadings. To more realistically capture the dynamic response of the bridge, it was necessary to consider three different sets of foundation springs developed in consideration of physical geometry of the foundation, direction of earthquake motions, and interface conditions between the foundation and underlying rock.

For the East abutment, on the other hand, a non-linear foundation spring model was developed in order to capture the post-failure behaviour of the East abutment.

## ASSESSMENT OF EXISTING STRUCTURE

### Design Response Spectrum

A site-specific peak horizontal ground acceleration was obtained from the Geological Survey of Canada. The design earthquake with a return period of 475 years that was used for the safety level of retrofitting has a peak horizontal ground acceleration of 0.303g. The design response spectra were based on the standard spectra curves of the Canadian Highway Bridge Design Code (CAN/CSA-S6-00) [4] with the following site-specific parameters: (a) site coefficient: S = 1.5; (b) importance factor: I = 1.0.

### Elastic Dynamic Analysis

Elastic modal spectral dynamic analysis of the bridge was carried out using SAP 2000 computer program [5]. A three-dimensional finite element model used for the analysis is shown in Figure 3.

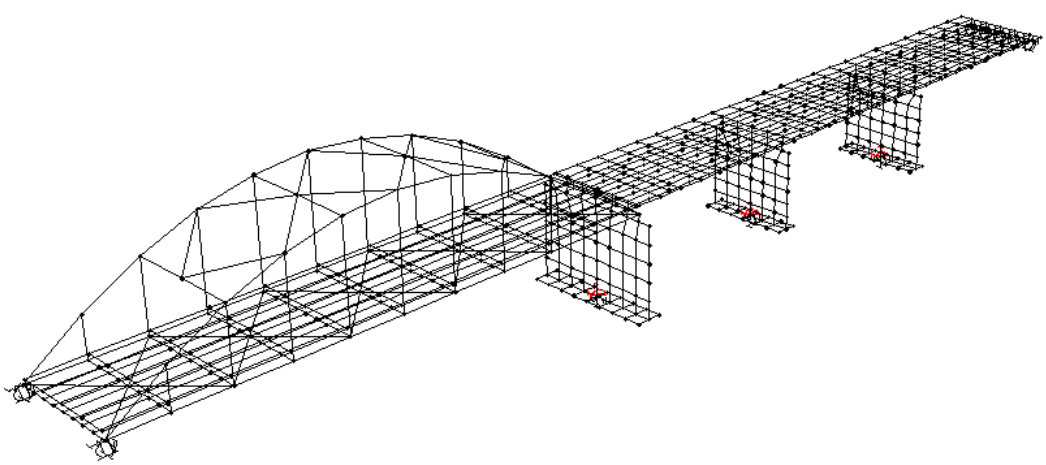


**Figure 3: Bridge Finite Element Model**

For the seismic analysis, 3-D beam elements were used for the arch ribs, hangers, floor beams, stringers, and bracing. The perforated beams in the bracing were modelled by averaging the section properties of the perforated and solid section. The composite bridge deck was modelled using shell elements rigidly constrained to the floor-beams. The deck elements had an effective stiffness of 50% of gross stiffness, accounting for the effects of cracking.

Modelling of the pier wall effective stiffness in the weak direction was not as simple as in the strong direction. The pier walls can be reasonably modelled as shear walls in the strong direction, which can be readily achieved by using shell elements with a reduced elastic modulus to account for the effects of cracking. For the response in the weak direction, both AASHTO design specifications [6] and Caltrans Seismic Design Criteria [7] currently require pier walls to be designed as columns. It appears that this simplification is primarily derived from theoretical considerations, rather than from actual structural response. The recent experimental studies conducted on the representative existing bridge pier walls by Abo-Shadi [8] have revealed that the displacement ductility of pier walls in the weak direction is largely influenced by the vertical steel ratios, but is only marginally affected by the confinement steel ratios. It was also observed that the seismic performance of pier walls is considered satisfactory even with confinement steel ratios 60% lower than the minimum required by AASHTO and Caltrans. These observations seem to indicate that the lateral response of pier walls in the weak direction should not be simplified as columns. Based on this conclusion, we decided that it was necessary to perform a non-linear moment-curvature analysis of the pier walls in order to evaluate their effective flexural stiffness and displacement ductility. The analysis was carried out using the RESPONSE 2000 computer program [9] together with application of the equivalent plastic hinge length method given by Paulay [10]. In addition, the foundation flexibility effects were considered in the assessment of the global displacement ductility of the piers. For the Sooke River Bridge, it was calculated that the effective flexural stiffness in the weak direction is approximately 15% of the gross stiffness for Piers 1 and 2, and is about 13% for Pier 3. These effective stiffness ratios are significantly lower than the typical range of 30% to 60% for regular pier columns. In order to confirm this result, further investigation of the analytical and experimental results of Abo-Shadi [8] was made, showing that for seven of the pier wall specimens tested, a similar lower range of effective flexural stiffness was found.

As the seismic acceleration may come from any direction, two different load cases are examined, one combining 100% of the demands in the longitudinal direction with 30% in the transverse; and the other load case combining 30% of the demands in the longitudinal direction with 100% in the transverse. Spectral demands were combined with dead load demands in such a way as to create the most critical case. In conformance with the MoT Seismic Retrofit Design Criteria [1], load factors on dead load and seismic load were taken as 1.0.

**Assessment Results**

A visual inspection of the bridge was carried out to confirm the existing conditions of the main bridge structural elements. The inspection confirmed that the steel superstructure elements and the concrete deck are in relatively good condition, and that there is no significant deterioration of structural steel that might affect the structural strength. Accordingly, the capacities of structural elements were calculated based upon the as-built drawings and using the Canadian Highway Bridge Design Code [4]. No structural deficiencies were identified for arch ribs, hangers, floor beams, stringers and cross bracings. Except for the bearings, no retrofit is required for the existing superstructure elements of both the tied-arch span and the steel plate girder approach spans.

The flexural and shear capacity/demand (C/D) ratios for the pier walls are presented in Table 1. The shear demands in the weak direction of the pier walls were calculated from the over-strength moments, whereas in the strong direction they were obtained from the elastic response spectral analysis. The shear capacities in the strong direction were based on the ACI [11] provisions for shear resistance of structural walls under seismic shear loading. The maximum shear stress was also checked as recommended by Caltrans [7] for the seismic shear loading in the strong direction.

**Table 1: Pier Load Capacity/Demand Ratios**

| Location | Forces | Strong Direction | | | Weak Direction | | |
|---|---|---|---|---|---|---|---|
| | | Demand | Capacity | C/D | Demand | Capacity | C/D |
| **Pier 1** | Moment [kN*m] | | | | 19 500 | 6 400 | *0.33* |
| | Shear [kN] | 4 550 | 12 700 | *2.79* | 870 | 5 200 | *5.98* |
| **Pier 2** | Moment [kN*m] | | | | 14 800 | 6 400 | *0.43* |
| | Shear [kN] | 2 810 | 12 700 | *4.52* | 850 | 5 200 | *6.12* |
| **Pier 3** | Moment [kN*m] | | | | 15 400 | 15 400 | *0.73* |
| | Shear [kN] | 1 360 | 19 400 | *14.6* | 1 500 | 7 930 | *5.29* |

The global displacement capacity/demand ratios of the pier walls in the weak direction are given in Table 2 for two levels of the ultimate concrete strain. The lower limit of 0.2% concrete strain is for pier walls with inadequate lap-splice length for the vertical reinforcement at the base where the lap-splice failure occurs before the full flexural capacity is reached. The upper limit of 0.4% concrete strain is a conservative estimate for practical design where the full flexural capacity is reached before the lap-splice failure is initiated. In the case of the Sooke River Bridge, the dowels from the pile caps of all three piers have a splice length of 914 mm (36 times bar diameter), which exceeds the most stringent requirement of 740 mm for the lap splices in plastic hinge zones proposed by Priestley [12]. The anticipated ductility factors of the Sooke River Bridge piers will be between the C/D ratios of these two limits. Therefore, all three pier walls are expected to perform satisfactorily for a longitudinal earthquake.

**Table 2: Global Displacement Ductility Factors of Pier Walls**

| Location | Demand [mm] | 0.2% Concrete Strain | | 0.4% Concrete Strain | |
|---|---|---|---|---|---|
| | | Capacity [mm] | C/D | Capacity [mm] | C/D |
| **Pier 1** | 295 | 436 | *1.48* | 751 | *2.54* |
| **Pier 2** | 295 | 524 | *1.78* | 839 | *2.84* |
| **Pier 3** | 279 | 472 | *1.69* | 679 | *2.43* |

The rocker bearings on Pier 3 and the East abutment are vulnerable to toppling during earthquakes. Their retrofit is necessary. The fixed bearings on the West abutment are deficient in shear resistance for both longitudinal and transverse earthquakes, and retrofit or replacement is required. The fixed bearings on Piers 1 and 2, on the other hand, have sufficient resistance for a longitudinal earthquake, but are deficient transversely. Providing suitable transverse restraint is required at these piers.

There were a few major structural deficiencies identified in the existing substructure and foundations including:

- The West abutment wall and footing have insufficient flexural capacity for the longitudinal earthquake. Furthermore, if the footing is not bonded to sound rock, it may be susceptible to overturning failure;
- The East abutment and Pier 3 have insufficient seat length, possibly resulting in unseating of the approach end spans;
- Piles of all three pier foundations have insufficient capacity for uplift force and bending in the weak direction.

## RETROFIT STRATEGY INVESTIGATION

The majority of structural elements of the existing bridge have an acceptable structural capacity for the seismic demand. Apart from the retrofit work for deficient bearings and inadequate seat length, the most expensive and difficult retrofit tasks are the mitigation of liquefiable soils and the strengthening of pier foundations. The objective of our retrofit strategies is to avoid retrofitting the West abutment and the foundation of Pier 1. The basic concept for the retrofit strategy investigation is to change the existing support articulation in order to utilise the stiffness and resistance of Pier 3 and the East abutment (both with expansion bearings prior to retrofit), thereby minimising lateral loads on the other supports.

Four feasible retrofit strategies were identified and evaluated in detail. A Seismic Retrofit Strategy Report was prepared based on the results of the analysis. Retrofit strategy meetings were held with MoT to review and finalize the recommended seismic retrofit strategy. The final retrofit strategy recommended included implementation of the following modifications to the existing bridge structure:

- Connection of both the tied-arch span and the steel plate girder span to Pier 3 by replacing the existing expansion bearings and installing tie-downs;
- Replacement of all deficient fixed bearings on Piers 1 and 2. The new bearings on Pier 1 will be unidirectional bearings oriented in the transverse direction allowing for free movement transversely, and thereby releasing any transverse force transfer to the foundation;
- Strengthening the piled foundations of Pier 2 and 3 by installing additional piles and enlarging the pile caps;
- Installation of longitudinal and transverse restrainers on both the West and East abutments;
- Soil densification for all three piers.

## RETROFIT DESIGN DETAILS

### Soil Densification at Piers

There were two feasible schemes to mitigate the potential for liquefaction of foundation soils: installation of stone columns to drain the pore pressure, or installation of timber compaction piles. Decision to choose one over the other depends upon the requirements of sustainability and constructability for a particular site. As the Sooke River is a salmon-bearing stream, there are very stringent restrictions on in-stream works. Driving in untreated timber piles to compact the soil is much less likely to stir up sediment than the drilling of holes to place stone columns. Even so, there will be requirements for berms lined with geotextile to prevent sediment release into the downstream waters.

From a constructability perspective, pile driving equipment will be brought to site to retrofit the foundations of Piers 2 and 3, and the same equipment can be used for the soil densification program as well. Furthermore, there is only 6 m headroom beneath the bridge deck, so it would not be possible to use large drilling equipment for the stone columns. In order to fit underneath the deck, spliced timber piles will be used.

Since Pier 1 was partially built upon an outcropping of bedrock, the upstream side was founded on rock and the downstream side founded on deep piles in liquefiable soils. Under earthquake motions, the pier may lead to a rotational response. Such behaviour could seriously undermine the pier foundation. In order to reduce the tendency to rotate, timber piles are to be installed on the downstream side of the pier foundation.

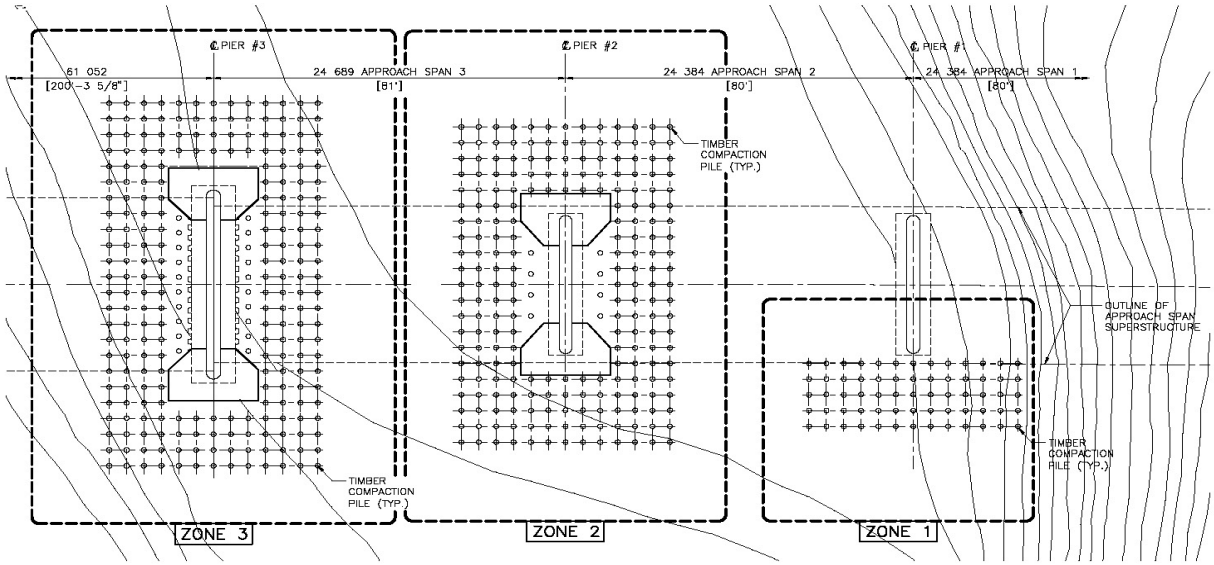



**Figure 4: Timber Densification Plan**

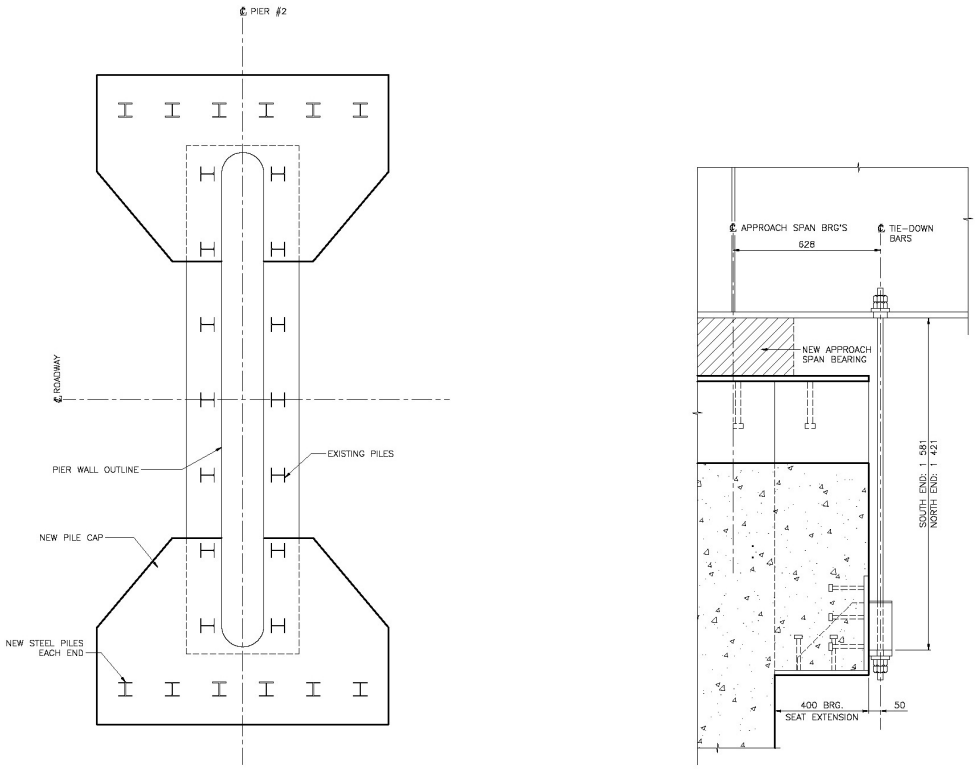
The timber piles used for soil densification are 355 mm (14 inches) butt diameter untreated piles. The piles length varies in order to suit the site conditions. In particular, 12.0 m (40 ft) long piles are used beneath the superstructure, 13.7 m (45 ft) to 15 m (50 ft) long piles are used elsewhere. Over 300 piles will be required.

**Pier Foundation Strengthening**

The installation of unidirectional bearings removes the need to retrofit the foundation of Pier 1. However, pier foundation strengthening is required for Piers 2 and 3. The retrofit work required includes installation of additional piles and strengthening of reinforced concrete pile cap as shown in Figure 5 (a).

Decision as to selecting an optimal solution for the foundation strengthening is largely dependent upon consideration of constructability and environmental impact. Steel caissons were considered, but not used, as the presence of boulders in the ground will have made them very difficult to construct and the installation process for a large caisson will have significant impact on the river bed. The H-piles are ultimately selected for the retrofit due to consideration that the existing piles were of the same type and were successfully driven to bedrock during original construction. The new piles are about twice as heavy though, thereby minimising the number of new piles that have to be installed. The existing piles have limited tension capacity, so the new piles have been designed to provide all the required resistance to uplift by driving into the bedrock. All the new piles will be embedded into extended concrete pile caps that will be monolithically connected to the existing pier caps.

At Pier 3, the pier wall will be widened at its top to provide sufficient seat length and more working space for the replacement of the bearings. In addition, new vertical tie-downs will be installed to the approach span beams only as shown in Figure 5 (b).




**(a) Pier Foundation Retrofit** **(b) Vertical Tie-Downs**

**Figure 5: Pier Retrofit Details**

**Restrainers at West Abutment**

The West abutment has inadequate longitudinal capacity to resist the seismic loads of the design earthquake. Rather than devising a complicated modification to the abutment itself, it was decided to pass through the abutment and bring the longitudinal loads directly into the ground behind it. Longitudinal restraint is provided by connecting tension rods from underneath the main tie girders of the bridge to a built-up steel column. The steel column is anchored on the abutment footing and restrained by soil anchors, thereby transferring longitudinal forces to the ground directly. Soil anchors will have to extend through drilled holes in the ballast wall into the bedrock behind the abutment. Figure 6 shows engineering details of the longitudinal restrainer developed.

Transversely, the abutment has adequate capacity. Two shear restrainer brackets will be mounted onto the underside of the end diaphragm (not shown in Figure 6). In the event of an earthquake, these restrainers will engage the abutment by transferring shear forces onto the existing concrete pilasters.

Although the steel fixed bearings on the West abutment are expected to be damaged during earthquake, the replacement option was considered, but not adopted in the detailed design. The tied-arch bridge is currently supported on two large steel bearings at each end. The rocker-bearings on Pier 3 must be replaced in order to prevent the bridge from collapsing under an earthquake event. Since the Sooke River Bridge is not permitted to shut down traffic for an extended period to facilitate bearing replacement, it was decided that the bearing replacement should be considered as the last resort. Rather than replacing the existing fixed bearings on the West abutment, which still function well under current live and dead loads,

the top of the built-up steel column is designed to act as a catcher, which will provide a support to the bridge in case it loses support from the existing bearings.

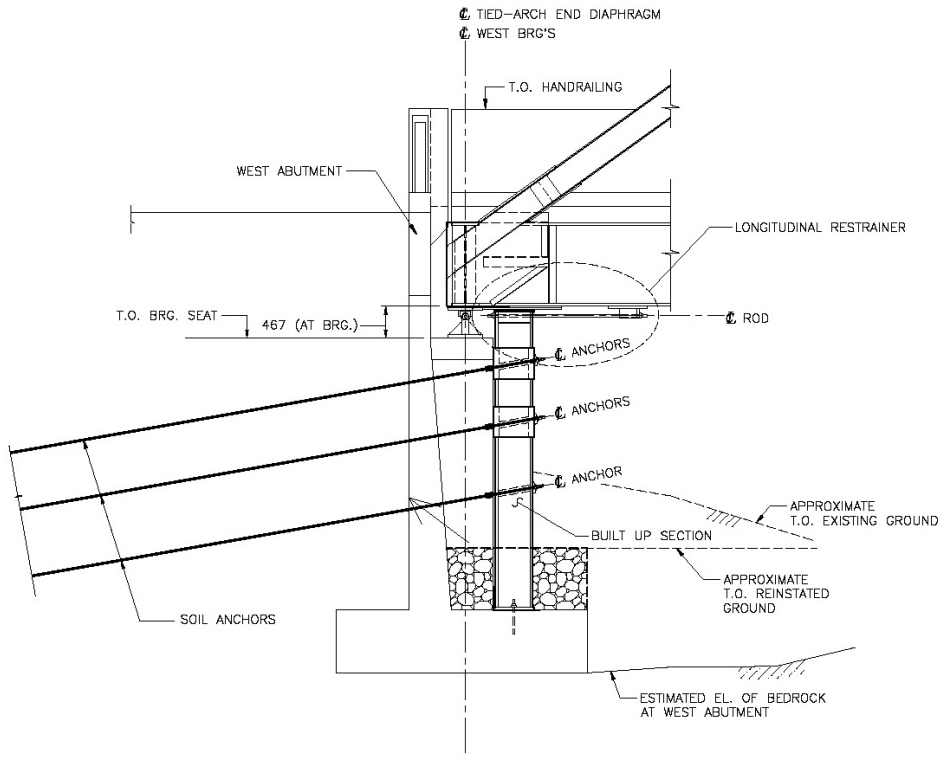



**Figure 6: West Abutment Longitudinal Restrainers**

**Restrainers at East Abutment**

The East abutment presents similar challenges as the West abutment. Again, in an attempt to avoid replacement of the existing bearings, built-up steel columns will be installed underneath the bridge girders, acting as catchers for the superstructure if the bearings should topple in an earthquake. The longitudinal capacity of the abutment is adequate to resist the seismic demands, so restraint is provided by means of tension rods attaching the built-up steel columns to the underside of the girders, with the steel columns anchored to the abutment. Figure 7 shows engineering details of the longitudinal restrainers developed for the East abutment. In addition, transverse restrainers are required and they are mounted onto the face of the abutment, rather than onto the underside of the superstructure as used in the West abutment.

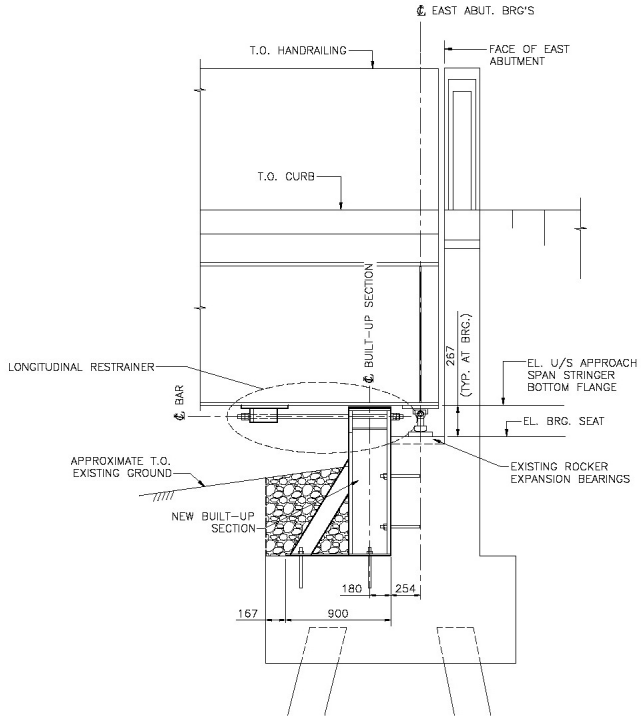



**Figure 7: East Abutment Longitudinal Restrainers**

**Bearing Replacement**

The retrofit design has made an effort to minimise cost and bridge shutdowns by keeping the existing bearings wherever possible. Nevertheless, in order to efficiently distribute seismic demands to the foundations, it is necessary to replace the bearings at all the piers as illustrated in Figure 8. The existing rocker bearings on Pier 3 will all be replaced, using new bearings that will transmit seismic lateral forces. The bearings on Pier 2 will be replaced with standard fixed bearings with the required transverse capacity.

At Pier 1, the retrofit strategy is to release the seismic forces in the transverse direction only, so that the Pier 1 foundation will not require expensive structural and geotechnical strengthening. To achieve this goal, a standard unidirectional bearing can be used. The bearing will have to be oriented at 90º to the standard direction, thereby providing the desired freedom for transverse motion.

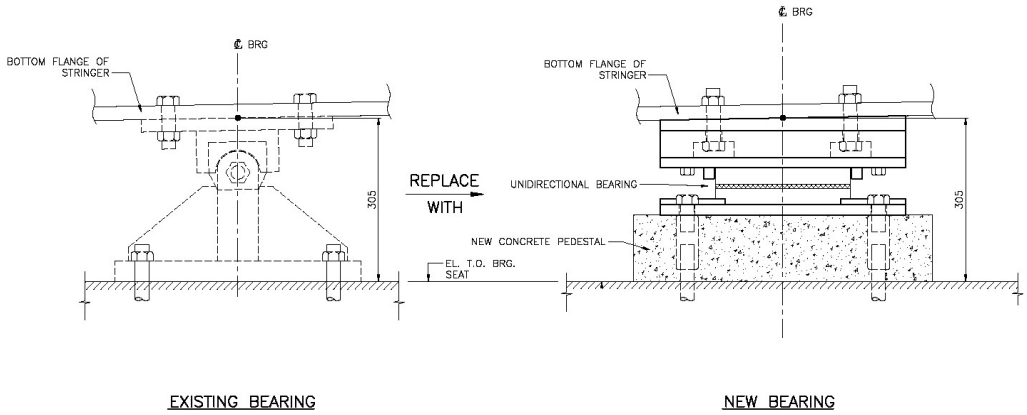



**Figure 8: Bearing Replacement Details**

## CONCLUSIONS

### General

In bridges, the greatest seismic demands are frequently experienced by the substructures. When evaluating existing bridges built decades ago, it is often the case that substructures are not detailed for ductility in accordance with current code requirements. It is fortunate that research is presently being undertaken to understand the seismic behaviour of poorly detailed reinforced concrete pier walls, as the large inventory of existing structures needs to be evaluated and often retrofitted. In the case of the Sooke River Bridge specifically, a non-linear static push-over analysis was carried out to adequately determine displacement ductility and effective stiffness of the pier walls. This analysis showed that, even though there were no lateral ties in the plastic hinge zone, the pier wall themselves were expected to perform adequately for the design earthquake. Nevertheless, the piled foundations still required upgrading.

By identifying and evaluating several sets of boundary conditions, corresponding to different bearing articulations and foundation responses, an optimal retrofit strategy was developed. This strategy optimises the retrofit of the bridge, by sharing the lateral seismic loads among all piers and abutments. With careful consideration of the constructability challenges, the environmental sensitivity of the river crossing, and the need of maintaining traffic for the public, construction details were developed for retrofitting the bridge. The construction work is currently underway. After completing the required retrofitting work, the Sooke River Bridge should have a level of safety comparable to that of ordinary bridges designed to the current bridge codes.

### Acknowledgements

The authors gratefully acknowledge the Ministry of Transportation of BC for its co-operation in authorising the publishing of this paper. Mr. Bill Szto, P.Eng., and Mr. Don Gillespie, Ph.D., P.Eng., of the MoT made essential contributions to the project, with Mr. Gillespie responsible for geotechnical analysis and design.

## REFERENCES

1. Ministry of Transportation of British Columbia. “Seismic retrofit design criteria”. July 2000.
2. Ministry of Transportation of British Columbia. “Geotechnical report – Sooke River bridge No 396”. January 2004.
3. ND LEA Consultants Ltd. “Safety retrofit final report of the Sooke River bridge No 396”. April 2003.
4. CSA International. “Canadian highway bridge design code”. CAN/CSA-S6-00. 2000.
5. SAP 2000. Version 7.10. Berkeley, CA: Computers and Structures Inc., 1998.
6. American Association of State Highway and Transportation Officials. “Standard specifications for highway bridges”. 16th Edition. 1996.
7. California Department of Transportation. “Caltrans seismic design criteria”. Version 1.2. 2001.
8. Abo-Shadi N, Saiidi M, Sanders D. “Seismic response of reinforced concrete bridge pier walls in the weak direction”. Technical Report MCEER-00-0006. Buffalo, NY: MCEER, 2000.
9. Bentz EC, Collins MP. RESPONSE-2000. Toronto, ON: Department of Civil Engineering of University of Toronto, 2000.
10. Paulay T, Priestley MJN. “Seismic design of reinforced concrete and masonry buildings”. New York, NY: John Wiley and Sons, 1992.
11. ACI International. “Building code requirements for structural concrete”. ACI 318-02.
12. Priestley MJN, Seible F, Calvi GM. “Seismic design and retrofit of bridges”. New York, NY: John Wiley and Sons, 1996.