13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3279

SEISMIC UPGRADE OF OAK STREET BRIDGE WITH GFRP

Yuming DING[1], Bruce HAMERSLEY[2]

SUMMARY

Vancouver's 2 km long Oak Street Bridge was constructed in the 1950's across the North Arm of the Fraser River. The south approach consists of continuous cast-in-place concrete girders founded in liquefiable soils. The structure required protection from movements caused by soil liquefaction, as part of a comprehensive upgrade.

Klohn Crippen was retained to examine structural alternatives for improving the girders' ability to accommodate the pier movements. Detail finite element analysis indicated that should the piers settle in an earthquake the bridge's under-reinforced girders would be expected to form single flexural cracks at the positive moment region of the girders, which would open wide enough to rupture the reinforcing.

Various strengthening methods were studied including steel plates, carbon fiber reinforced polymer wraps, and glass fiber reinforced polymer wraps. Composites provided cost and aesthetic advantages over steel plates. Glass fiber was chosen for design because it was considered the most flexible of the composites, which would accommodate the required level of settlement without brittle failure of the structure. The retrofit was completed in 2002 and resulted in cost savings in excess of $1 million over the ground improvement alternative.

INTRODUCTION

The Oak Street Bridge, connecting the cities of Vancouver and Richmond, British Columbia, is 1840 m in length and consists of a steel girder main span and cast-in-place concrete girder approaches (Figure 1). During the 1990s, the Ministry of Transportation of BC conducted a seismic retrofit with the primary objective being to prevent structural collapse of any part of the bridge under a 475 year return period earthquake [1,2]. Construction of the structural retrofit measures was performed under four separate contracts. A ground improvement retrofit of the south approach was also designed in 1994, however, construction was not implemented at the same time.

[1] Structural Engineer, Klohn Crippen Consultants, Vancouver, BC, Canada.
[2] Project Manager, Klohn Crippen Consultants, Vancouver, BC, Canada.

Figure 1 Overview of the Oak Street Bridge

Ground improvement to mitigate liquefaction was expected to be costly and further optimization studies were undertaken. Geo-technical study provided pier settlements induced by the ground movements from liquefaction. It was requested that Klohn Crippen would perform engineering to balance ground densification work with further structural upgrades to accommodate permanent ground displacements. The purpose was to provide a robust solution to the super-structure.

The current paper discusses the capacity and finite element analysis of the girders, and the preliminary design process of the Glass Fiber-Reinforced Polymer (GFRP) application.

CAPACITY ANALYSIS AND FINITE ELEMENT ANALYSIS

Pier Settlement Demand

The recent geotechnical study reduced the anticipated level of post-earthquake liquefaction ground movement with a mitigated ground densification scheme. The scheme focused more on understanding vertical deformations and in particular, differential movements between piers. Horizontal deformations were not as critical because previous structural retrofits provided significant reinforcing in lateral strength and ductility.

The ground movement estimates included a wide variety of scenarios for settlements of piers. The girders required analysis and checking for all the different combinations of pier movements, creating a large amount of data for interpretation. The maximum differential pier settlement after densification was evaluated as 100 mm, which was applied as a forced displacement in the structural analysis of girders.

Capacity Analysis of Girders

The south approach concrete girders consist of cast-in-place concrete haunched stringers composite with the concrete deck (Figure 2). There are five girders across a typical section. The three interior stringers are similar with 480 mm (1'7") thick sections with the negative moment top steel. The two exterior stringers have similar depths but are 400 mm (1'4") thick, with the negative moment top steel placed in the concrete curbs above deck level. Exterior stringers have much less shear reinforcement than interior ones, typically 10M (#3) or 15M (#4) stirrups at 450 mm (18") and 900 mm (36") spacing.

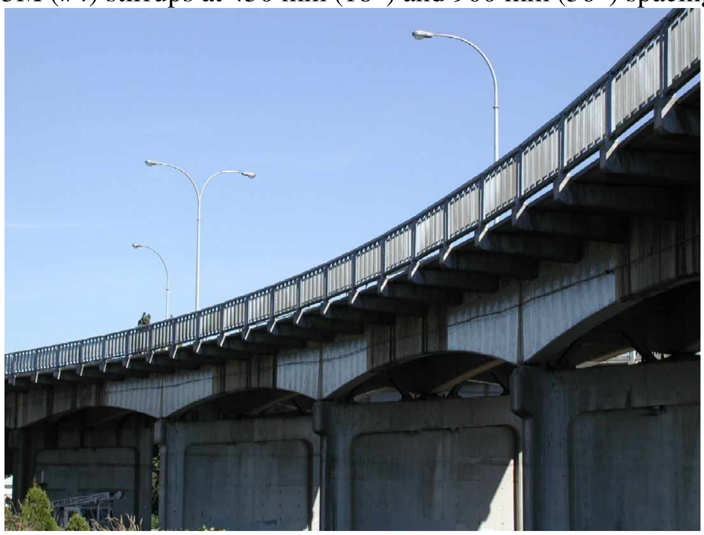

Figure 2 South Approach Girders of the Oak Street Bridge

Linear elastic analyses were performed using S-Frame, a structural analysis program. Beam element models were developed for a number of span configurations including: typical 4-span at 18 m (60') each span, 3-spans at various span length, and one 2-span at 18 m each span. Girder soffits were parabolic between spans, leading to non-prismatic section properties and a curved beam profile through the element centroids. Typical beam elements were 600 mm in length, and section properties varied accordingly. A 3-D view of one model is presented in Figure 3.

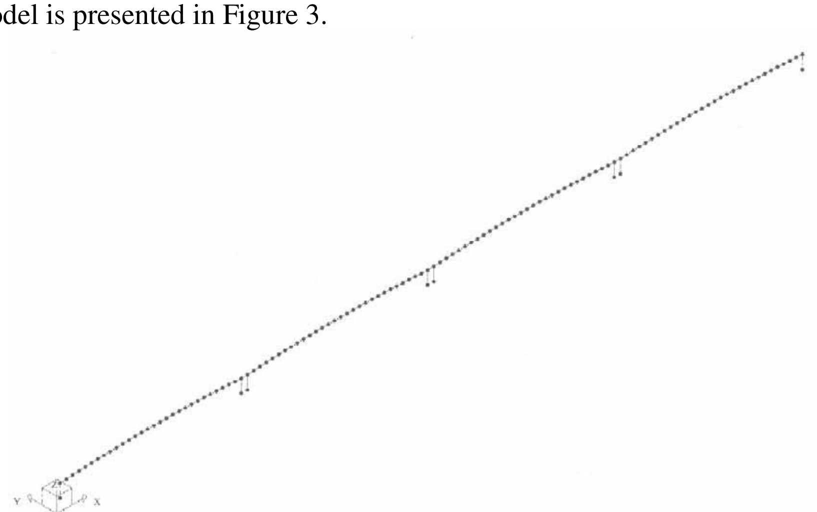

Figure 3 S-Frame Computer Model

Uncracked moments of inertia were initially used to determine the critical bending moments. Since the moment demands derived were generally higher than the cracking moments, effective moments of inertia were calculated according to the ACI Code [3]. Typically the effective moments of inertia were 60% of the gross moments of inertia. Research by Priestley [4] on reinforced concrete members indicated the level of reduction within normal ranges.

CS-600 truck loading from CAN/CSA-S6 [5] was considered for vehicle live load. Live load distribution factors and dynamic factors from the same code were applied. Girder dead weights were calculated internally using a concrete density of 2400 kg/m^3 (150 pcf). Support settlements were imposed at side supports, intermediate supports and center supports.

Capacity/Demand ratios for sections of the typical 4-span model are presented in Tables 1. In the table, section 1 is defined at side support, section 2 at mid-span between side support and intermediate support, section 3 at intermediate support, section 4 at mid-span between intermediate support and center support, and section 5 at center support. Md is the moment demand under loading(dead+live+100mm settlement), while Mdd is the moment demand under loading(dead+100mm settlement). Similar definitions apply to Vd and Vdd.

Table 1 C/D Ratios for Typical 4-span Girder Sections

Girder Section	Moment		Shear	
	Mc/Md	Mc/Mdd	Vc/Vd	Vc/Vdd
Section 1			0.60	1.17
Section 2	0.58	0.85		
Section 3	0.58	0.75	0.58	0.93
Section 4	0.61	1.11		
Section 5	0.52	0.69	0.53	0.92

A number of diagrams created in spreadsheets were also utilized to show the relationship between the moment demands and capacities along the length of the continuous girders. One typical diagram was shown in Figure 4.

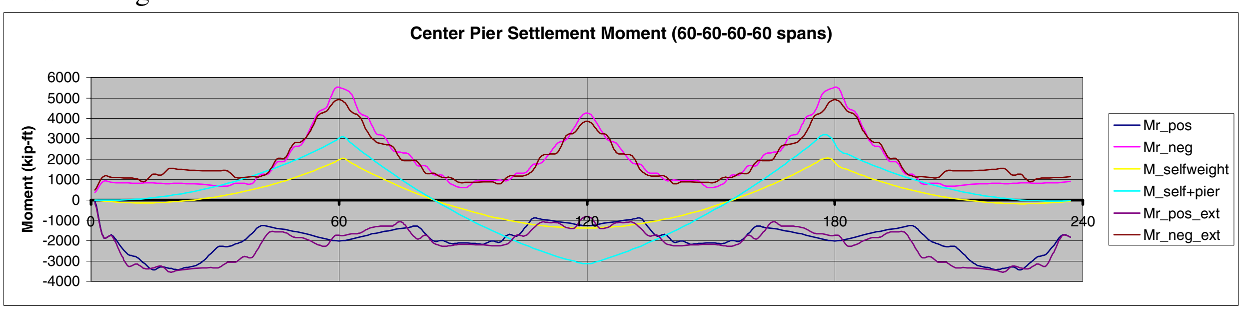

Figure 4 Capacity and Demand Diagram

For assessment of girder capacities and retrofit design purposes the existing concrete strength was evaluated at f'c = 34.5 MPa (5 ksi). The concrete strength specified for original construction was 20 MPa (3 ksi), however an average compressive strength of 40 MPa (6 ksi) was shown from core tests. Clause 12 of CAN/CSA-S6 [5] was referenced to derive the 34.5 MPa design strength.

Standard ACI methods of sectional analysis were used to calculate flexural strength, using an ultimate concrete compressive strain of 0.005. Strain hardening and section over-strength were not incorporated into the analysis because of the elastic nature of the behavior investigated

The capacity/demand (C/D) ratios indicated that many of the continuous concrete girders were deficient in positive moment flexural capacity when subjected imposed pier settlement. Deficiencies were found in exterior girders only, as interior girders reinforcing details were better.

The shear capacities were found to be acceptable in all of the girders.

Settlements at supports would increase positive bending moments at adjacent mid-spans. As the concrete sections were under-reinforced, a single crack was likely to develop through the section before well-spaced cracks occur. And rupture of the longitudinal reinforcement was predicted. Without the pier to stabilize within 100 mm of settlement, the girder could continue to deflect until the single cracks penetrate the deck, leading to collapse of the superstructure. The non-ductile failure mode prompted the study of crack patterns and appropriate retrofit design.

Finite Element Analysis and Crack Control

Simulation of crack development required advanced level of analysis and powerful finite element program. Dr. Perry Adebar at the University of British Columbia was consulted as a specialist in the field of crack control in reinforced concrete members. A sophisticated finite element model was developed for a typical 4-span at 18m (60') girder using the ADINA software. A 3-D view of the model with deformed shape is shown in Figure 5.

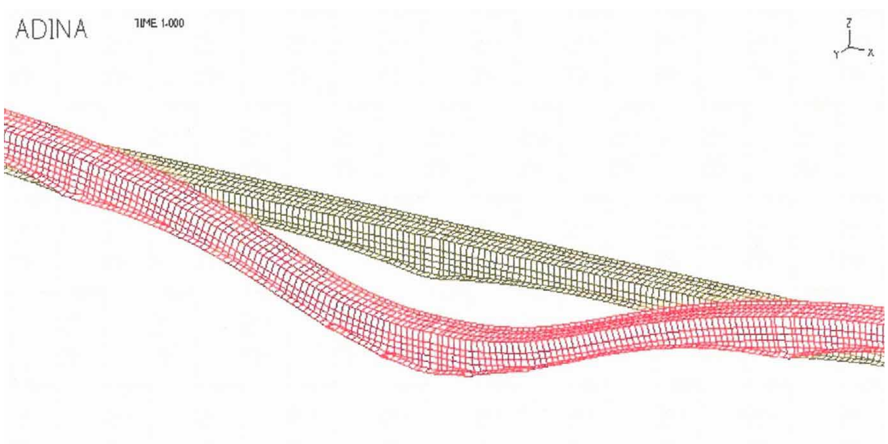

Figure 5 Finite Element Model in ADINA

Important features in the model include:

- The T-section of the interior girder was explicitly modelled by building geometry of both the deck and beam web section.
- The haunch at the bottom of the girder was explicitly modelled. Elements with a typical width of one foot were used to give a relatively smooth parabolic curve.
- The 3-D isoparametric displacement-based solid element was used to account for the three-dimensional state of stress and strain in the web.
- Concrete material in the ADINA program was applied to all elements. The basic material characteristics were:
  - Tensile failure at a maximum, relatively small principal tensile stress.
  - Compression crushing failure at high compression, following a typical nonlinear stress-strain relationship to allow for the weakening of the material under increasing compressive stresses.
  - The tensile and compression crushing failures were governed by tensile failure and compression crushing failure envelopes.

- Strain softening from compression crushing failure to an ultimate strain, at which the material totally fails.

- The tension-only truss element was applied in key areas to simulate the effect of reinforcement on the cracking pattern of the girder.
- The total number of geometry points in the model is 6982. The total number of solid elements is 2584.

The analyses of the model were able to provide cracking patterns and visualization of areas of concern. The analyses confirmed crack opening at the positive moment region around settled pier. The assessment of the girders indicated that strengthening of the positive moment regions of the continuous girders was necessary.

RETROFIT DESIGN

The analyses of the girders indicated that pier settlement would cause positive moment demands in excess of member capacity, and that there was potential for collapse of the superstructure. A total of 112 of the exterior girder spans were identified for strengthening. Girder strengthening design was thus performed. Three options for strengthening of the positive moment capacity were initially considered:
Option 1 – Steel Side Plates;
Option 2 – Carbon Fiber-Reinforced Polymer;
Option 3 – Glass Fiber-Reinforced Polymer.

Substantial advances in Fiber-Reinforced Polymer (FRP) technology and the technique's increased acceptance in the engineering community were considered in choosing a robust design. Option 2 and 3 would have less effect on bridge aesthetics, and the extensive drilling through the girders required for Option 1 could be avoided. Option 3 would also provide good economy and constructability for strengthening reinforced concrete structures. However the stiff material of FRP would not readily improve structural flexibility, and careful design was required to obtain crack control and flexibility.

A unique concept for the use of fiber FRP was developed (shown in Figure 6). This included the application of a debonding agent intermittently along the bottom longitudinal wrap to allow the fibre-glass to stretch. The design would force well-distributed narrow cracks in the girder, thus providing the required flexibility to accommodate differential pier settlements. For a more detailed description of the design , see Ding and Hamersley [6].

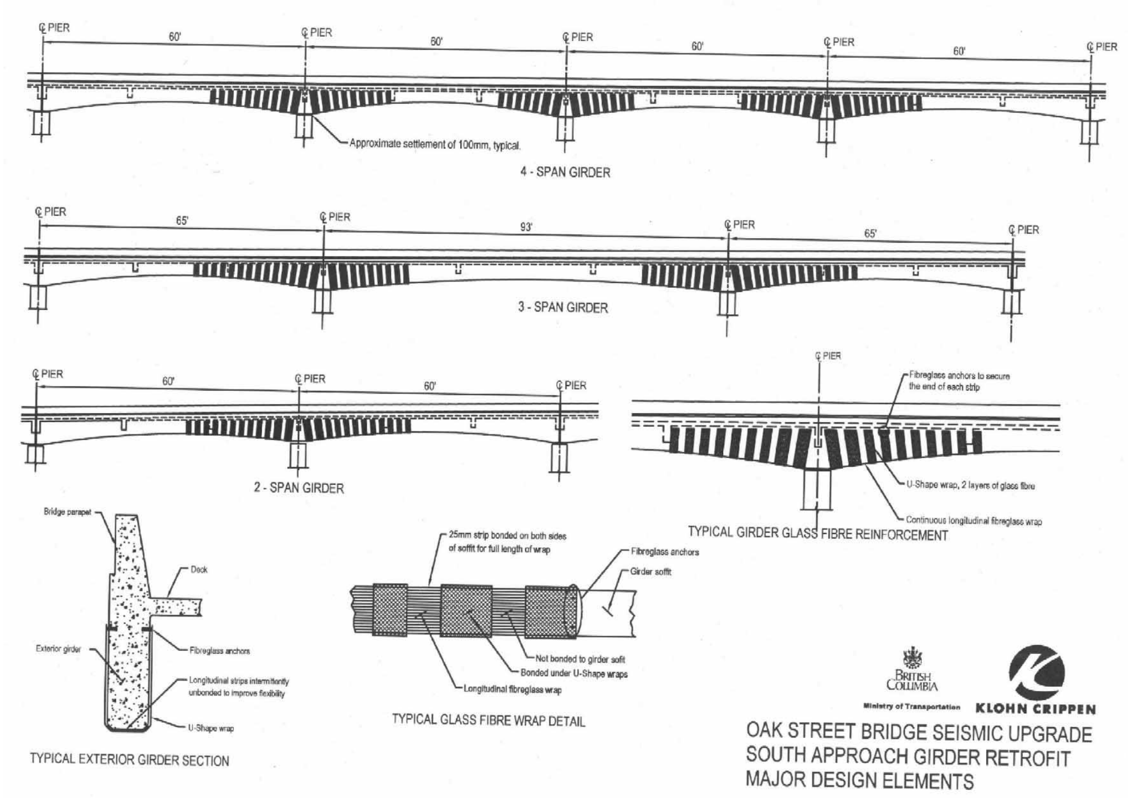

Figure 6 Details of the GFRP Design

The retrofit construction was finished in late 2002. The following picture (Figure 7) showed the final finish of the GFRP retrofit.

Figure 7 GFRP Construction

CONCLUSION

The continuous concrete girders of the Oak Street Bridge were not able to sustain the pier settlements caused by soil liquefaction under design level earthquakes. Analysis indicated that a single crack could extend through the depth of the girder leading to collapse of the superstructure. Various retrofit strategies were evaluated to strengthen the girders. GFRP was selected and successfully applied in the design.

Protection of the bridge from a major earthquake was achieved with savings in excess of $1million over ground improvements. Other advantages include reduction of risk to underground utilities, strengthening of the girders for live load traffic, and aesthetically pleasing upgrades to the bridge.

ACKNOWLEDGEMENTS

The authors would like to thank Mr. Bill Szto of the Ministry of Transportation for his continuous support throughout the project.

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