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SEISMIC REHABILITATION OF THE HERON ROAD BRIDGE

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ABSTRACT

The Heron Road Bridge (Ottawa, Ontario), consists of twin 7 span structures, each approximately 275m long, which carries Heron Road over the Rideau River and Canal. The bridge was constructed in 1966/67 and is composed of a combination of continuous cast-in-place post-tensioned voided concrete slabs with suspended precast girder segments.

The structural and seismic evaluations revealed that piers will be overstressed and are not adequately confined at the plastic hinge region to resist seismic loads. Pier foundations will not provide sufficient resistance to overturning moments associated with seismic forces, and abutments are inadequate to withstand the longitudinal seismic forces. Unseating and full failure of the suspended spans during a significant seismic event is likely.

Given the condition of the piers and foundation as determined by the seismic evaluation and the extent of overstressing experienced, the simplest and most cost effective solution was the seismic isolation approach. Such an approach would significantly reduce the forces transferred to the piers and the associated overturning moments transferred to the footings and it would also avoid a costly strengthening scheme for the piers and foundations. Lock-up devices were proposed to stabilize the response of the suspended spans.

Introduction

The Heron Road Bridge consists of 275 meter long twin structures which carry Heron Road over the Rideau Canal, the NCC pedestrian pathway, Colonel By Drive, the Rideau River, and Vincent Massey Park access road. The bridge was constructed in 1966/67 and consists of two separate structures – one carrying the three westbound lanes (North Bridge) and one carrying the three eastbound lanes (South Bridge).

Each of the two structures has seven (7) spans with semi-continuous articulation and

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drop-in suspended spans. The structures are composed of semi-continuous cast-in-place post-tensioned voided concrete slabs, 1321 mm deep, which are cantilevered to support three (3) suspended segments. These suspended segments are composed of nine (9) simply supported A.A.S.H.T.O. girders, type III, with a 178 mm thick reinforced concrete deck. Each bridge is supported on six (6) piers and abutments at the ends. The abutments and the three west piers are supported on H-piles driven to bedrock, whereas the three east piers are supported either directly on bedrock or on dense to very dense gravely sand underlain by bedrock.

In 2005, the City Ottawa retained Delcan to undertake a detailed condition assessment, structural and seismic evaluation of the bridge, as well as the preparation of a Structure Renewal Options Report, to identify any deficiencies and to determine a long-term plan of action for this bridge. Once this work (Phase 1) was completed, Delcan proceeded with the preliminary and detailed design for the full rehabilitation and seismic retrofit of these twin structures.

The seismic evaluation and preliminary design of the retrofit of this bridge was carried out using a Multi-Modal Response Spectral Analysis in accordance with the Canadian Highway Bridge Design Code CAN/CSA S6-06 (CHBDC). A site-specific time-history analysis was carried out, at the detailed design phase, to validate the retrofit design. The seismic retrofit scheme for this bridge involved the introduction of seismic isolation bearings at all supports and the installation of lockup devices at the expansion end of the suspended spans to prevent unseating. This paper will concentrate on the different seismic analysis that was undertaken as part of this project, as well as some of our key findings.

Analysis Background

Strategies for mitigating seismic demands on the Heron Road Bridge(s) were established during Phase 1. Results indicated that the combined use of high damping isolators and displacement dependent lockup devices successfully reduced substructure loads.

Hysteretic isolators and lockup devices are highly nonlinear devices not always accurately modeled using linear methods such as response spectrum analysis. CHBDC requires a nonlinear analysis for structures with damping ratios in excess of 30%. In addition, site conditions ranged from Soil Type I (rock) to Soil Type III (soft soil), suggesting the need for different excitations for each substructure. Given the above, along with the relatively low lateral strength of the substructures, it was determined that a nonlinear time history analysis using site specific excitations should be incorporated in order to check the responses used for design found using multi-mode analysis.

An in-house two degree of freedom program capable of modeling custom nonlinear elements was first used to evaluate forces across the displacement dependent lockup devices. Individual excitations were applied to each substructure, accommodating the markedly different soil overburden site conditions. Using site specific excitations generated by the geotechnical consultant (Golder Associates), some key observations were made;

1. Very little movement was occurring across the isolator interface.

2. Despite the different soil conditions, structure response was virtually the same whether unique excitations were applied to the substructure, or if they were all substructures were excited using the same signal.
3. The lockup devices were not displacing enough to lockup.

The time history response conflicted with the Phase 1 linear analysis results, the difference was traced to the use of site-specific excitations. It was found that though by one code definition the soils at the various substructures could be classified as widely varying, though at longer structural periods the excitations were nearly identical. These results suggested that a simpler, more cost effective alternative existed, using bumpers (impact bearings) instead of lockup devices. The impact bearings are uniaxial devices that resist in both compression and tension after a predetermined gap is traversed.

Multi-Mode Analysis

Design response was based upon multi-mode analysis per CHBDC. The isolation system was modeled using the effective stiffness of the isolators and a composite spectrum (AASHTO Guide Specifications for Seismic Isolation Design). A composite spectrum is a combination of the project design spectrum (Soil Type 1, 0.20g compatible) for use in the shorter period range, and the isolated spectrum Sa = (AS)/(TB) for use at longer periods, with a linear ramp connects the two. When using this method a necessary constraint is that the majority of the displacement across the isolators occurs at a period in the isolated portion of the spectrum. This was verified with modal (eigenvalue) analysis. Longitudinal and transverse direction analyses have different effective stiffnesses and composite spectra associated with them.

Both the effective stiffness and the composite spectrum change with response displacement. If a response displacement for a particular isolator is different than the one used to calculate the effective stiffness (and composite spectrum), the analysis should be re-run. Convergence criteria for this analysis are 1 mm for isolator displacements, and 0.02 for the spectrum damping parameter, B. Convergence to within 1mm achieved with all isolators except for those at the east abutment for the transverse run, for which it was 10mm. Composite spectra convergence was achieved in both directions.

The isolation system was designed to have a nominal elastic period of 2.0 seconds, substructure stiffnesses and soil compliance tend to lengthen the effective periods, while the hystertic damping from the isolotors tend to shorten the effective periods. The first mode was found to have period of 1.84 seconds, a torsional mode the result of the free rotation at joint locations (Figure 1). The second mode at 1.75 seconds was the first longitudinal mode. 30 modes were used in the analysis, representing 93% of the structural mass in both the longitudinal and transverse directions.

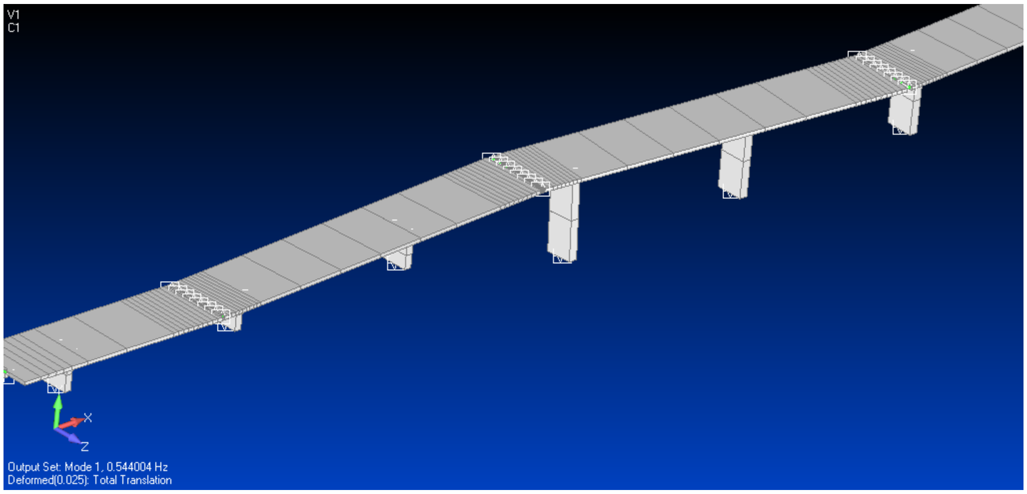

Figure 1. First mode, (torsion dominant)

Time History Analysis

Nonlinear time history analysis was run using OpenSees software, written by the PEER Center (Pacific Earthquake Engineering Research). The program has the advantages that it can model nonlinear elements accurately, with a fast, highly stable time history solver. Isolators were modeled as idealized bilinear hysteretic elements using OpenSees zero-length elements with parallel materials. Each parallel material was composed of elastic a perfectly plastic (Qd) material, and an elastic (Kd) material. Each isolator was modeled with its unique Qd, Ku (preyielding stiffness), & Kd material property identifiers. Impact bearings were modeled using the elastic perfectly plastic gap material, with a gap setting of 35 mm.

Time histories were supplied by the geotechnical consultant (Golder Associates), they were scaled to be compatible with the CHBDC Soil Type I Isolation spectrum. Results indicate that the time history results are in agreement with the multi-mode analysis results. In general the time history isolator displacements (Figure 2) were less than those found through multi-mode analysis.

Joint forces include those across the elastomeric bearings as well as the impacting force at the bumpers. The sudden rise in force from impact can be seen clearly in the time history traces (Figure 3). Figure 4 plot force versus displacement, where impacting can be seen to be occurring at Joints 1 & 2. The impact force is the height of the steep slope on the force-displacement plots, with a maximum of about 500 kN at Joint 1, significantly less than the 2500 kN design force.

OpenSees worked well for this analysis because of problem simplicity and its ability to accommodate nonlinear behavior. Pre/post processing is handled via tcl (a programming language pronounced “tickle”) scripts. This approach was cumbersome and added significant time to the analysis. To circumvent this, a translator was written integrating a commercial pre/post processor, utilizing simpler tcl scripts mainly to drive OpenSees. Recently introduced to OpenSees is the capability for the user to write their own materials and elements (in C++), especially valuable for nonlinear analysis.

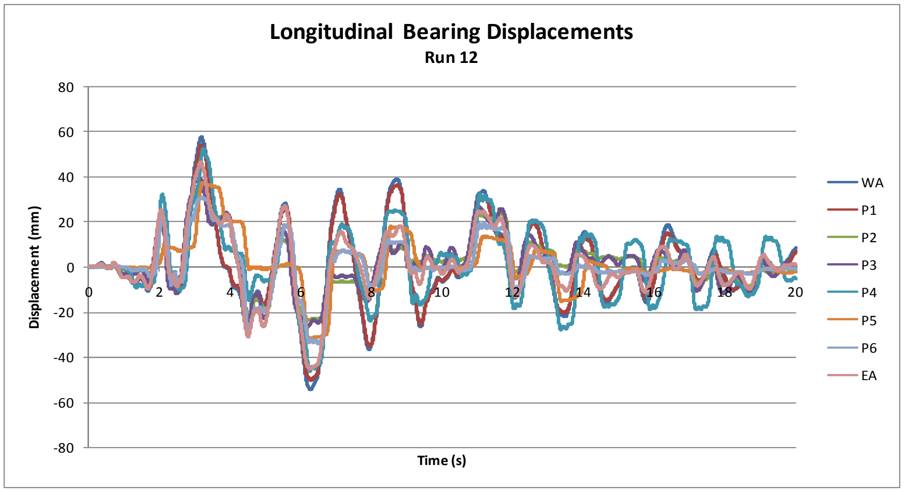

Figure 2. Longitudinal bearing displacements, Run 12

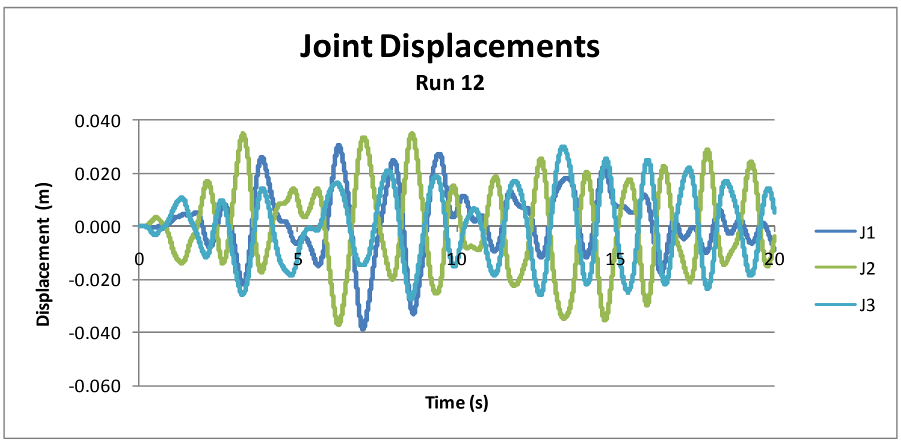

Figure 3. Joint displacements, Run 12

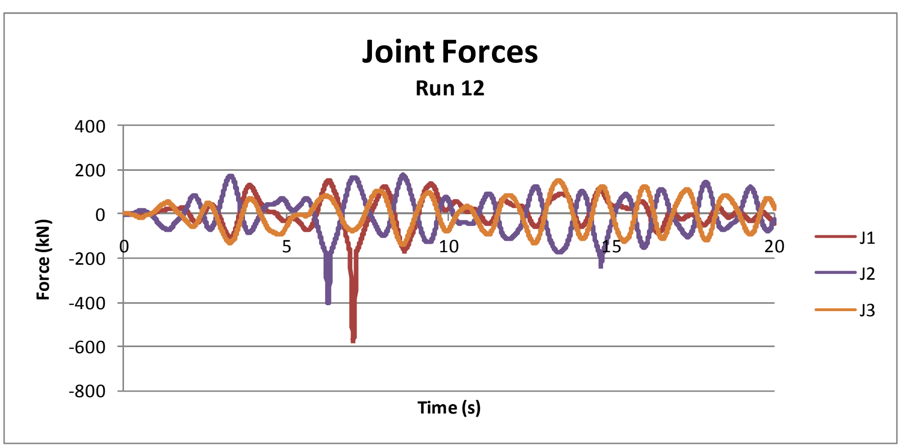

Figure 4. Joint forces, Run 12

Conclusions

Aside from structural response, some key findings of the analysis included;

- Current CHBDC time history analysis requirements call for usage of excitations compatible with conventionally fixed response spectra. This presents a problem with isolated structures.
- Soil classifications based on the fundamental periods of the structure result in better correlation with response found using site specific excitations.
- The initially proposed lock-up devices could be replaced with simpler bumper bearings.
- Shear key sizes could be greatly reduced from those resulting from multi-mode analysis.
- Restrainer forces are sensitive to assumed stiffnesses. A solution to both the analytical problem (force prediction) and functional problem (high forces) is to include a spring in series with the restrainer.
- Convergence issues exist with multi-mode analysis using effective stiffnesses to model nonlinear elements.
- Though OpenSees is typically used in a research setting, it is a valid medium for project use as well though pre/post processing is awkward.
- Response spectrum and time history analyses complement each other well, and a design process using both has many advantages.

Use of nonlinear time history analysis provided much insight into the response of the structure, assisted in isolation system optimization, and allowed for significant hardware reductions/simplifications.