Galena Creek Bridge: Structural-health Monitoring, Instrumentation and Finite-element Modeling

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INTRODUCTION AND BACKGROUND

The Galena Creek Bridge carries Interstate 580 and U.S. Route 395 between Reno and Carson City, Nevada. Shown in Figure 1, the seven-span reinforced concrete box-girder bridge, with a total length of 526.2 meters, was completed in 2012 and includes a 210-meter cathedral arch span. Internal hinges are located near the piers just outside of the arch, allowing for longitudinal movement and separating the structure into three frames. The middle frame is supported at the base of the arch and at the bottoms of the adjacent columns by thrust block foundations. The longitudinal post-tensioned two-cell box-girders rest on the six sets of single column piers, and the deck is post-tensioned transversely. The column and arch cross-sections are all hollow rectangular sections. The bridge consists of two separate structures tied together for lateral loading resistance using a link slab between the decks at the crown of the arch, and link beams connecting the thrust blocks at the base of the arch.

During construction of the bridge, the Nevada Department of Transportation (NDOT) collaborated with the University of Nevada, Reno (UNR) to install instrumentation and perform monitoring on the main arch span (Taylor & Sanders 2008). Nonlinear time history analysis was found to have comparable results with elastic response spectrum analysis. Strain and temperature data were collected between 2009 and 2010 (Vallejera & Sanders 2011). Analytical models attempted to consider the contribution of load, time, and temperature-dependent effects on the total strain experienced.

In 2012, NDOT and UNR performed a second study to characterize the dynamic properties of the completed bridge (Carr & Sanders 2013). Accelerometers were installed throughout the southbound main arch span. During the test, the structure was dynamically excited in the vertical direction using a construction vehicle and in the transverse direction using an eccentric mass shaker. Field data were compared to the results from analytical models. Following the controlled dynamic testing, traffic loading was monitored for a short duration. The experimental results agreed with the predicted results from the analytical models. The 2012 instrumentation system was intended to be a permanent structural-health monitoring (SHM) installation; however, at the conclusion of the project, the system was no longer maintained or monitored. NDOT has a renewed interest in establishing a permanent SHM system on the Galena Creek Bridge to monitor its response to seismic events and routine traffic.

Figure 1. Galena Creek Bridge.

OBJECTIVE AND SCOPE

The research objective is to develop and implement a permanent SHM system on the Galena Creek Bridge. The SHM system will measure the structural response to routine traffic, wind, seismic, and thermal loadings to provide proactive measures, such as analysis results and timely alarm notifications in the form of SMS and email messages. To facilitate communication and foster an information network to stakeholders, including NDOT traffic personnel and roadway users, the research will integrate the developed system into the NDOT Intelligent Traffic System (ITS).

The Galena Creek Bridge SHM system will be an entirely new installation on the northbound structure. Accelerometers will be used to capture the dynamic response. Sensor locations and orientations were selected to enable a full assessment of the bridge. A finite-element model will be used to compare the measured structural response from ground motion data to computational predictions. The model will also provide fundamental information, such as the natural frequencies, mode shapes, modal damping values, and general response of the structure. Ultimately, the goal of the project is to create a system NDOT is able to manage and maintain after the completion of the project. If successful, the monitoring system will lead to future SHM systems implemented on other bridges in Nevada.

DESCRIPTION OF THE GALENA CREEK BRIDGE

The Galena Creek Bridge consists of two separate structures, a northbound and a southbound, tied together laterally by two link beams and a link slab in the arch span. The superstructure is a two-cell box-girder with a width of 18.9 meters and a depth of 3 meters as displayed in Figure 2. Two expansion joint hinges separate each structure into three frames. The hinges are located 15 meters to either side of the arch span, measured from the centerline of the hinge to the centerline of the adjacent column. The diaphragms at the hinges allow for accommodation of enough conventional reinforcement and prestressed tendons in the hinge regions. Diaphragms are also located at both abutments, at the mid-span of each structural span, and in the arch-superstructure merging region. The depth of the superstructure only varies in the arch-superstructure merge region, where the total depth increases to 3.6 meters. The thickness of the soffit, or the bottom slab of the box-girder, increases near the piers.

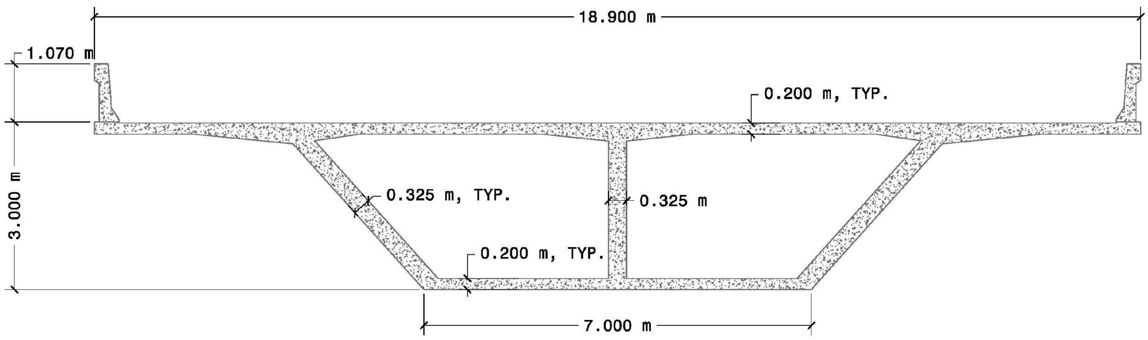

Figure 2. Box-girder superstructure typical section.

Each of the structures consists of seven spans, which are supported by single column piers and an arch. The twelve columns are hollow and rectangular with exterior dimensions of 3 meters by 6 meters and interior dimensions of 1.8 meters by 4 meters. The strong axis of the column is oriented to resist transverse bending. Due to site topography, the height of the columns widely varies, resulting in the northbound columns being taller than the southbound columns. A pedestal is located at the bottom of the southbound Pier 4 column due to strong winds that knocked over the original reinforcing bars before the concrete was poured during construction.

Each structure has a 210-meter cathedral arch in the middle frame. The bottoms of the arch are supported with the adjacent columns by the thrust blocks, and the crown of the arch merges with the superstructure. The cross-section of each arch is hollow and rectangular with exterior dimensions of 3.6 meters by 6 meters and inner dimensions of 2.8 meters by 5.2 meters. Similar to the column cross-section, the strong axis of the arch coincides with the transverse direction of the bridge.

A link slab and two link beams tie the northbound and southbound structures together to reduce seismic forces and displacements, specifically in the transverse direction. The 200-millimeter thick link slab connects the two structures along the arch frame between the cantilever overhangs of the two box-girders. A link beam was used to effectively rigidly connect each pair of arch thrust blocks. The connection forces the two foundations to act as one during a seismic event.

INSTRUMENTATION METHODOLOGY

The purpose of structural-health monitoring is to continuously assess the condition of a structure, typically either for long-term degradation or short-term impact from an extreme event. The traditional way of assessing structural condition is through manual, visual inspection, giving SHM practical advantages over common practice. In the long-term, a monitoring system can be more economical. SHM also has the benefit of continuously collecting data and checking on the condition of the structure, while traditional inspection occurs periodically, resulting in sporadic data collection and follow-up condition assessment. The monitoring of structures allows for the ability to detect structural damage, which can significantly reduce the cost and effort involved in the maintenance of the structures (Heo et al. 2018). Having a system that examines structural conditions can help ensure that the maintenance of a bridge is safe and effective.

When using accelerometers in a SHM system, location and orientation are crucial. As the main objective of SHM is to detect, locate, and inform of damage in a structure, optimal sensor networks are required to ensure a successful monitoring system (Azarbayejani 2010). For example, vertical acceleration data can determine relative displacements between different columns during a given event, and lateral acceleration data can obtain the relative displacement (drift) between the top and bottom of each column during that event. Optimal sensor placement is used in SHM to help identify the most effective locations and orientations of sensors, as well as the count of sensors necessary for a given purpose. A total of 33 uniaxial accelerometers will be installed on the northbound structure, as shown in Figure 3. Some of the accelerometers will be grouped to capture response in multiple directions, resulting in 15 monitoring locations. In addition, a free-field site consisting of a triaxial accelerograph, including a data recorder and a triaxial accelerometer, will be located approximately 50 meters from the bridge. The location will serve as a baseline for the actual ground motion in all three axes, while remaining unaffected by the response of the structure to the ground movement.

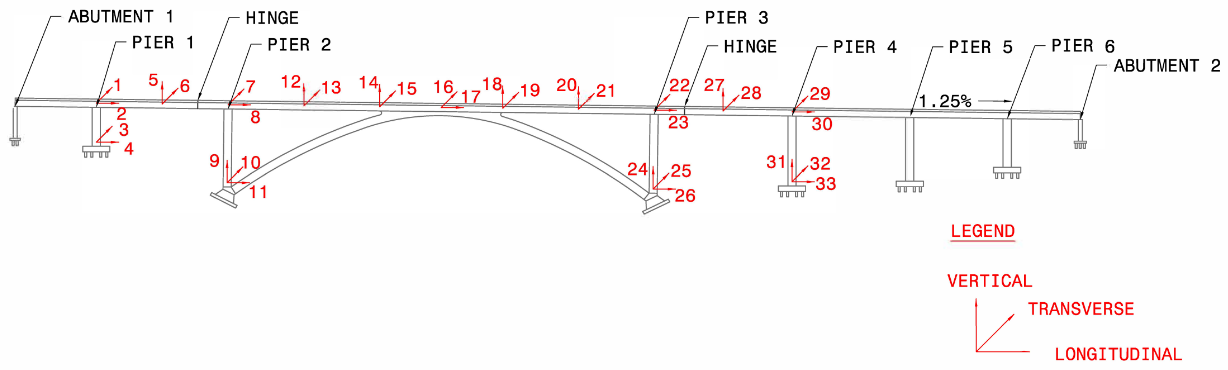

Figure 3. Instrumentation plan.

The 33 uniaxial accelerometers deployed in the northbound structure will provide the relative motion of the bridge between different points of interest during an earthquake. Longitudinal accelerometers are located at the top and bottom of four of the six piers of the bridge, as well as at the crown of the arch. Vertical accelerometers are located at the bottom of three piers, edges of the arch-superstructure merge region, between the merge region and adjacent piers, and between each set of piers adjacent to the hinges. Transverse accelerometers are located at all of the previously listed points of interest. The longitudinal and transverse accelerometers will be used to compute lateral motion, while the vertical accelerometers will allow for the calculation of relative displacements along the length of the bridge. The triaxial accelerograph located at a free-field site will monitor the three components of ground motion without having interference from the response of the structure.

A proven SHM system for buildings will be adapted for the Galena Creek Bridge. The SHM system is able to record data, perform computations, and send alarm notifications through various media. A key component of the system is the control software, the framework of which is shown in Figure 4. The SHM easily accommodates new sensor data for instrumentation beyond accelerometers. The SHM software was designed to rapidly alert appropriate authorities within minutes of a trigger event. In addition, uniquely tailored summary reports for emergency responders, structural engineers, and expert analysts can be generated automatically and accompany the alert. The reports enable management to make swift condition assessments, as well as to identify potential areas of structural damage.

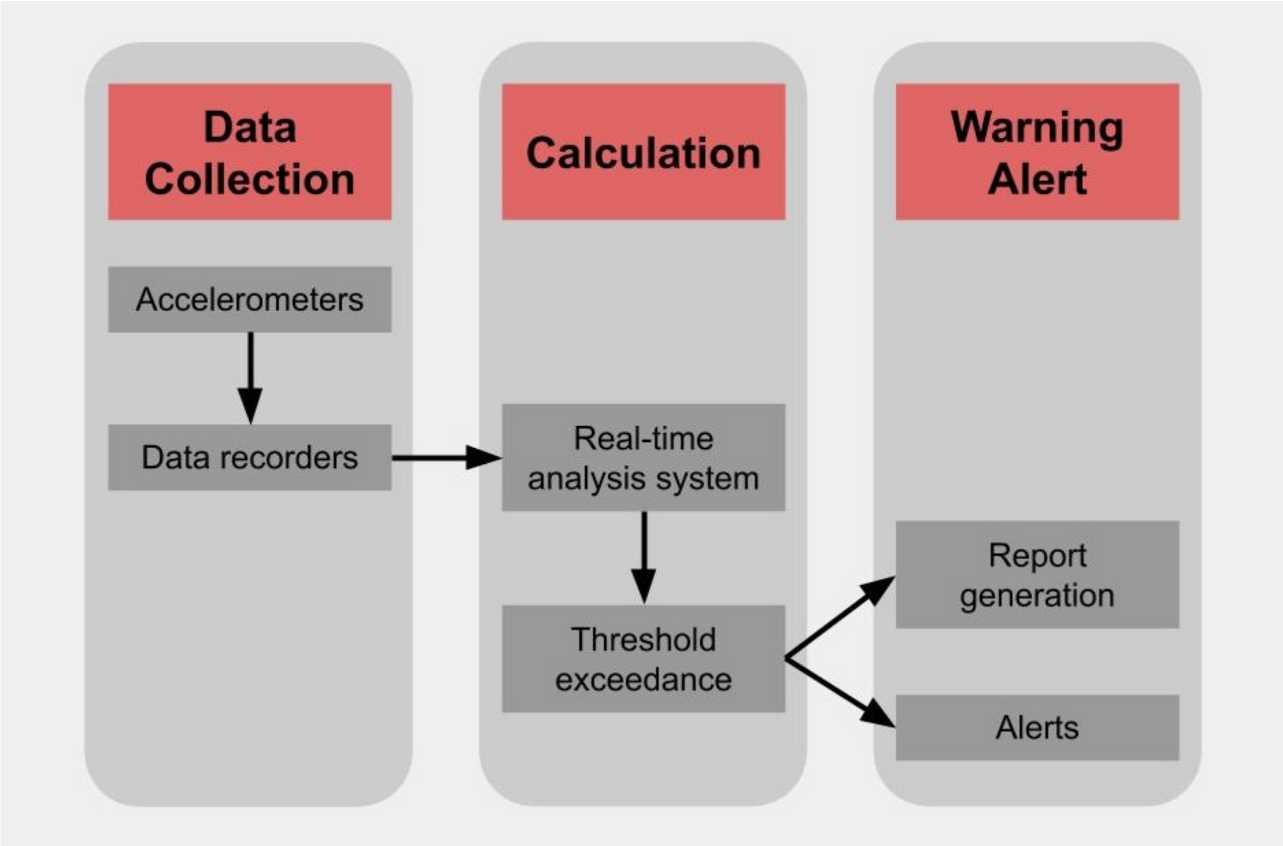

Figure 4. SHM system architecture.

FINITE-ELEMENT MODEL DEVELOPMENT

There are many finite-element analysis programs, and each typically has added benefits when used in particular situations. CSiBridge, by Computers and Structures, Inc. (CSi), is one of the latest and most powerful structural analysis programs used for finite-element models. The software allows bridges to be modeled either as spine models comprised of frames, or as three-dimensional (3D) finite-element (FE) models. Due to the wide variety of features included and the availability of many references and user manuals, CSiBridge was chosen to model the Galena Creek Bridge. The bridge was previously modeled using SAP2000, another CSi program, as well as using MIDAS Civil, in past work done by NDOT and UNR (Carr & Sanders 2013). CSiBridge has many additional features not included in SAP2000, specifically for bridges, enabling a more detailed and representative structural model than those previously created.

The development of the computer model was comprised of many steps. Material and sectional properties were referenced from the as-built plans and defined as close to the real structure as practical. Other main steps included modeling the bridge geometry, superstructure, substructure, link beams, link slab, foundations, post-tensioning, and inputting loads. The 1.25% grade in the longitudinal direction of the bridge was accounted for in the model, while the 2% superelevation in the transverse direction was neglected.

The 28-day specified compressive strength of various components varied from 28 to 35 MPa. The compressive strength was 28 MPa for the columns and link beams, 31 MPa for the superstructure, and 35 MPa for the arch. A Mander unconfined stress-strain curve was used for the concrete, essentially neglecting the effects of confining reinforcement on the axial strength of the components. The modulus of elasticity of each component was calculated using Equation 1, where E_c is the modulus of elasticity and f'_c is the 28-day compressive strength. The CSiBridge default coefficient of thermal expansion for concrete (9.9×10^{-6}/°C) was used. A Poisson's ratio of 0.2 was used for the concrete, and the unit weight of the normal weight concrete was taken as 23.56 kN/m^3. The longitudinal post-tension tendons were comprised of 27 strands of A416 Grade 270 steel, with a coefficient of friction of 0.2 and a wobble factor of 0.00066/m.

$$E_c = 4700\sqrt{f'_c} \text{ (MPa)} \quad (1)$$

The model is primarily comprised of frame elements assigned uniform sectional properties, essentially making it a spine model. However, certain elements differentiate it from a typical spine model; specifically, non-prismatic frame elements in segments of the superstructure, thin shells forming the link slab, and frame elements defining the cross-sections of the superstructure. Non-prismatic frame elements were used where the depth and soffit thickness varied linearly. The link slab is modeled as a set of connected thin shells. The superstructure cross-sections are broken apart into separate frames, instead of having the entire section be assigned to frames in the longitudinal direction connected by intermediate nodes.

The northbound and southbound structures were modeled using layout lines offset 20.92 meters apart with bridge objects assigned to those lines. All diaphragms in the superstructure were included in the structural model. The pedestal located at the base of the southbound Pier 4 column was neglected in the computer model, which will not significantly affect the overall structural response. The link slab was modeled as a set of thin-shells, which are area objects used to model both in-plane (membrane) and out-of-plane (plate-bending) behavior. The membrane and plate-bending thickness of the homogenous slab was defined from its geometry.

For modeling the region of the bridge where the arch and superstructure merge together, referred herein as the merge region, two main methods were initially considered. The first method was to model the arch and superstructure independently and then use rigid links to connect the two elements. The second method was to use a merged frame section, replacing the superstructure and arch in the merge region. Such a frame section would be linearly varied between the crown of the arch and the ends of the merge region. NDOT originally created two separate models to compare both methods and found the models had agreeable results, most likely due to the high rigidity of the merge region. The first of the two methodologies was chosen because it was more computationally efficient. The arch and superstructure merge in the middle 61.5 meters of the arch span, including the crown of the arch. Six rigid links for each structure were used to connect the arch nodes to the superstructure.

The pile caps and thrust blocks were modeled as fully fixed at the base of the columns and arch. The thrust blocks were modeled as fully fixed, as they were placed on competent rock. The pile caps were modeled as fully fixed because of the effects of having multiple cast-in-drilled-hole (CIDH) piles grouped together on the rigidity of the foundations. A total of 12 drilled shafts per cap were arranged in multiple rows. For the purpose of analyzing the bridge, modeling the pile caps as fixed would be more accurate than assuming a depth of fixity for each individual pile. Additionally, using soil springs to model the pile caps would require extensive information defining the soil properties, which was not available. For the abutments, translation was restrained in the transverse and vertical directions of the bridge, and rotation was restrained in the longitudinal direction. Rigid links were used to connect the abutments to the webs of the box-girders.

The prestressing of the structure includes longitudinal and transverse post-tensioning. The longitudinal post-tensioning consists of both internal and external tendons. The tendons were placed as elements in the model. All prestress losses were calculated by CSiBridge; as such, the losses were set to zero in the model input to prevent double consideration of the losses. The built-in parabolic calculator was used to input the geometry of the prestress tendons. Each frame has a set of internal tendons, which are equally distributed between the three girders of the superstructure. The anchorages are located in the two hinges and the two abutments. The internal tendons in the inner frame were jacked from both hinges simultaneously, while the internal tendons in the outer frame were jacked from the abutments. The inner frame also contains two sets of external tendons, located between each hinge and the adjacent pier, with jacking from the hinges. The transverse post-tensioning on the actual structure was ignored for the purposes of the structural computer model. The purpose of the transverse post-tensioning was to strengthen the deck slab; therefore, omitting the corresponding compressive force will not significantly change the structural response to dynamic loading.

The initial structural model, shown in Figure 5, is mostly linear for multiple reasons. The conventional reinforcement was initially ignored in the model due to the linearity of most components. The superstructure was modeled as linear due to the stiffening effects of the post-tensioning limiting the formation of cracks, while the arch was modeled as linear due to the high axial load experienced. The only part of the initial FE model considering non-linear effects was the definition of the effective moment of inertia of the columns.

A common assumption used when designing or analyzing reinforced concrete structures is that the concrete will not resist tensile forces. Concrete that has cracked due to tensile stresses will have a lower moment of inertia than the original moment of inertia of the given cross-section. To account for the reduced moment of inertia, structural models often use cracked section properties in the form of an effective moment of inertia. The effective moment of inertia can be estimated various ways, usually through using an effective moment of inertia equal to a given ratio of the gross moment of inertia. One such ratio recommended by Priestley, Seible, and Calvi is taking the effective moment of inertia as 40% of the gross moment of inertia (Priestley et al. 1996). The 40% ratio was used in the initial FE model for the columns in the bridge to account for the cracking in the concrete. This allowed to consider the non-linearity of the columns in the model, while maintaining the overall model’s linearity.

To perform the initial FE analysis, it was necessary to add any external loads not previously accounted for in the model. The dead loads consisted of the self-weight of the concrete, loads from the post-tension tendons, the weight of wearing surface, and the weight of the barrier rails. It was not necessary to input any live loads for the initial model. The self-weight of each major bridge component was accounted for when the model was created. The prestressing loads were added when the tendons were created in the model. The wearing surface load was applied as a 1.8 kPa area load everywhere except where the barrier rails were located. The barrier rail loads were applied as 6.52 kN/m line loads located at their centroids. All dead loads were then used when comparing the current model with previous models developed to simulate the bridge.

Separate from the model, hand calculations were used to check the reactions expected at the restraints. The rough estimates agreed well with the model output. For example, the sum of the vertical reactions was within 1% of the estimated total weight of the bridge. A modal analysis was also performed with the initial model, and the mass participation of each mode was used to determine which modes would be considered in the results of the analysis. The results were then compared to results from previous Galena Creek Bridge models to further improve and verify the initial model.

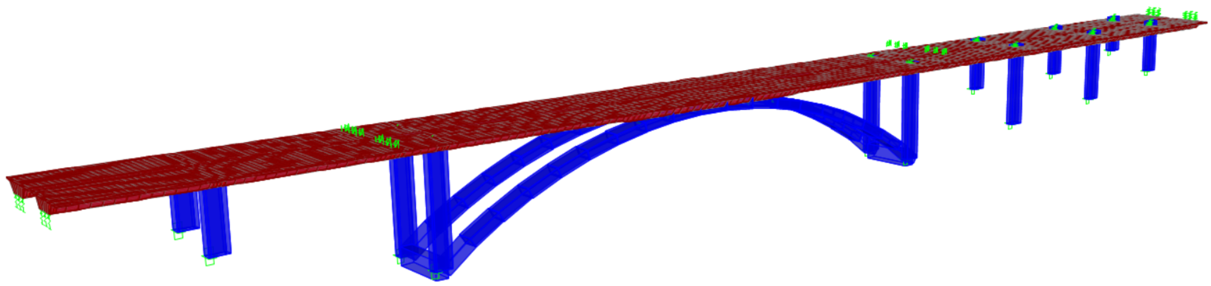

Figure 5. 3D view of completed structural model in CSiBridge.

CONCLUSIONS

Using the system consisting of the accelerometers, data recorders, and a robust SHM software, dynamic structural properties can be obtained. The system makes it possible to measure the response of the structure to various ground motions. A refined computational model will be used to simulate and predict the structural response to the same input ground motions. The model will be verified using a combination of short and long-term field data. Having such a monitoring system deployed on the Galena Creek Bridge will provide NDOT with a host of benefits. The most significant of which would be the ability to alert the appropriate authorities in the case of an extreme event, triggering the generation of technical reports to be delivered with the real-time notification.

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