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SENSOR CLUSTERING BASED DAMAGE DETECTION FRAMEWORK FOR RAILWAY BRIDGES USING ACCELERATION RESPONSE

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Abstract: Bridges are critical components of the railway infrastructure system and the majority of these bridges are approaching their estimated design life. Moreover, the demands on the bridges have been burgeoning both in terms of increased axle loads and operational frequency. It is paramount that; these systems are maintained effectively. Therefore, crafting powerful Structural Health Monitoring (SHM) systems for railroad infrastructure is relied upon to help the proprietors with their decision-making policies. Hence, developing damage investigation strategies specifically tailored for railroad bridges is the principle goal of this on-going study. In this paper, we present our preliminary findings to build up a damage identification framework based on acceleration measurements for railroad bridges. Initially, a Finite Element Model (FEM) of an open-deck railway bridge is developed. The model is then utilized to conduct numerical studies and gather acceleration response under moving train for both baseline and damaged conditions. This info is then further scrutinized by a sensor clustering based damage identification technique using time-series modeling. The damage in the bridge is investigated by observing the damage features of the damaged and undamaged bridge. The investigation demonstrates the damage features by comparing the fit ratios of locations of interest so that damage could be identified and located. The relative severity of the damage can also be assessed by comparing the magnitude of the damage features. Assessing the condition of our railway bridges continuously in this manner and early detection of potential structural changes are deemed very valuable for the infrastructure owners for developing more economical and effective maintenance strategies.
Keywords: - Structural health monitoring, railway bridges, damage identification and localization, damage quantification, acceleration measurement, sensor clustering.

1. INTRODUCTION

Existing infrastructures are subjected to various potential risks such as aging, fatigue, corrosion and also vulnerable to natural disasters like hurricanes, earthquakes etc. In addition, modern bridges are also subjected to an ever-increasing demand for heavy axle loads and operational frequency. These potential risks result in different levels of damage which may cause the failure or even collapse of the infrastructures. Therefore, the need for better condition assessment of these structures has become more paramount (Choi et al. 2010). As bridges are vital components of the railway transportation systems and federal and provincial bridges passed the halfway mark of their design life in Canada (Gaudreault 2006), paying attention to the health of these bridges is essential.

Several researchers have reviewed structural health monitoring concepts during the past decades for different bridge type structures (Catbas et al. 2012, Wiberg 2006, Banerji and Chikermane 2012, Scott et al. 2013). The main focus was on bridge monitoring for identifying damage existence as well as condition, safety, and serviceability assessment. As a result, more complex SHM concepts were developed in order to expand their evaluation capacities for bridge conditions and decision making pertaining to the use, repair, strengthening of damage and deteriorated structures.

The methodology used in this paper is based on sensor clustering based time series analysis of acceleration response, which was introduced by Gül (Gül et al 2011). The time series model order for this method is arbitrarily chosen through trails and experience. The methodology was tested using experimental results obtained by on-site impact loading test on a frame. This paper presents our ongoing efforts for improving this methodology and its application to real-life structures i.e., railway bridges. This method offers clear advantages to detecting and localizing structural changes and estimates relative severity under unknown loading conditions. The methodology allows the use of bridge acceleration response monitored at critical locations with operating traffic on a bridge, which is usually the most economical and convenient measurement method for bridges.

The basic requirement for damage detection and localization using sensor clustering based time series analysis is the availability of spatially distributed acceleration response on critical elements of the structure. A change in the structural behavior will be shown as a change in the Damage Feature (DF) between sensors. The responses retrieved from all sensor measurements is employed for the data analysis to obtain fit ratios and finally damage features.

The objective of this article is to demonstrate that this methodology can be economical and efficient for long-term monitoring of railway bridges. This methodology also facilitates an opportunity to develop effective instrumentation plans and data analysis strategies for common types of railway bridges tailored to the specific needs and requirements of the respective utilization. In long term, this method can be a critical component of a more economical maintenance and rehabilitation strategy for different railway bridges.

2. METHODOLOGY

The dynamic responses (accelerations, velocities, and displacements) of a structure are governed by the Equation of Motion (EOM). This equation, with which the linear dynamic response of a structure with N Degrees of Freedoms(DOFs) complies with can be written in simple form as **Error! Reference source not found.**Eq. (1). Here, *M, C, K* represent mass, damping and stiffness matrices of the system respectively. The vectors $\ddot{u}$, $\dot{u}$ and u are accelerations, velocities, and displacements respectively. The external forcing function is denoted by *P*. If the external force is zero, Eq. (1) can be simplified to obtain a response for the 1st DOF, $\ddot{u}_1$ as in Eq. (2).

$$\begin{pmatrix} m_{11} & \cdots & m_{1n} \\ \vdots & \ddots & \vdots \\ m_{N1} & \cdots & m_{NN} \end{pmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \vdots \\ \ddot{u}_N \end{Bmatrix} + \begin{pmatrix} c_{11} & \cdots & c_{1n} \\ \vdots & \ddots & \vdots \\ c_{N1} & \cdots & c_{NN} \end{pmatrix} \begin{Bmatrix} \dot{u}_1 \\ \vdots \\ \dot{u}_N \end{Bmatrix} + \begin{pmatrix} k_{11} & \cdots & k_{1n} \\ \vdots & \ddots & \vdots \\ k_{N1} & \cdots & k_{NN} \end{pmatrix} \begin{Bmatrix} u_1 \\ \vdots \\ u_N \end{Bmatrix} = \begin{Bmatrix} p_1 \\ \vdots \\ p_N \end{Bmatrix} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(1)}$$

$$\ddot{u}_1 = -\frac{(m_{12}\ddot{u}_2(t) + \ldots + m_{1N}\ddot{u}_N(t)) + (c_{12}\dot{u}_2(t) + \ldots + c_{1N}\dot{u}_N(t)) + (k_{12}u_2(t) + \ldots + k_{1N}u_N(t))}{m_{11}} \quad \ldots\ldots\ldots\ldots \text{(2)}$$

Eq. (2) contains velocity and displacement terms. The time-series model used in the study only incorporates acceleration response assuming that assuming that the time-series model inherently accounts for velocity and displacement responses. In real-life bridges, obtaining velocity and displacement responses under moving train can be very difficult.

In this study, time series models are used to fit the above dynamic response of a structure. The ARMAX time series model to represent the relationship of input, output and error terms of a system can be written as Eq. (3).

$$y(t) + a_1 y(t-1) + \cdots + a_{n_a} y(t-n_a) = b_1 x\left(t-1\right) + \cdots + b_{n_b} x(t-n_b) + e(t) + d_1 e\left(t-1\right) + \cdots + d_{n_d} e(t-n_d) \quad \ldots\ldots \text{(3)}$$

where y(t), $x(t)$ and $e(t)$ are output, input and error terms of the model, respectively. The unknown parameters of the model are shown with $\boldsymbol{a_i}$, $\boldsymbol{b_i}$ and $\boldsymbol{d_i}$. The model orders are n_a, n_b and n_d. In this study, the ARX model orders of n_a=0, n_b=1 are considered adequate for the simple bridge model without noise.

3. DESCRIPTION OF THE BRIDGE

The finite element model of the railway bridge for this study is developed using CSiBridge software (CsiBridge, 2014). The model is shown in **Error! Reference source not found.**. This open-deck bridge is 36 m long 2-span carrying single lane railway track. The width of the bridge is 3.0 m. The abutments are modeled as simple supports which are restrained against horizontal and vertical translation, but free to rotate in any direction. For simplicity, railway track is not modeled and only a lane is modeled for the train to move. The main girders and diaphragms are made of structural steel ASTM A709 GR50 with a modulus of elasticity of 200 GPa, the yield stress of 345 MPa, the ultimate tensile stress of 448 MPa and a Poisson ratio of 0.3. There are two main I-girders in the bridge, 2300 mm in depth. The top and bottom flange widths are 500mm. The bridge also consists of equally spaced 8 diaphragms with two of them at the abutments. The purpose of the diaphragm is to provide lateral stability of the bridge and do not essentially carry vertical loads. The connection point between main girder and diaphragms are considered as critical locations where the accelerometers are placed. It is assumed that the bridge inherent damping ratio for dynamic analysis is very small (1%). To represent train load, AREMA COOPER E80 train is chosen. It is a standard train load used to design railway bridges in North America. For this study, it is assumed that the train crosses the bridge at 20, 30,40,50,60 and 70 kph speed. The relevant properties of the bridge are summarized in **Error! Reference source not found.**.

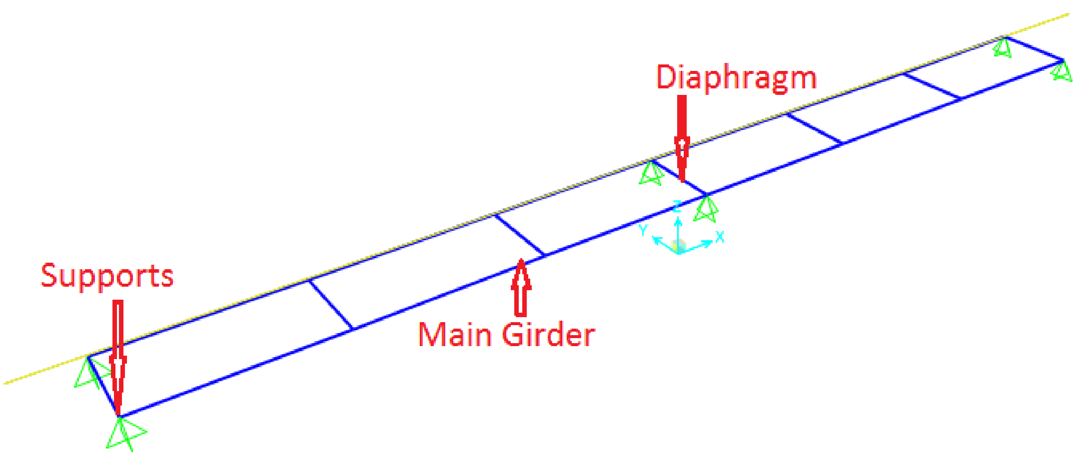

Figure 1. Finite element model of the bridge under study

Table 1. Relevant properties of the bridge model

Bridge Component	**Properties**
Span length	36 m 2 equal span
No. of lane(s)	1
Total deck width	3.0 m
No. of main girders	2
No. of diaphragms	8
Diaphragm spacing	6 m
Deck thickness	150 mm
Steel material	ASTM A709 Gr50
Main girder depth	2300 mm
Flange width	1000 mm
Flange and web thickness	50 mm

Diaphragm section height	2000 mm
Diaphragm flange width	1000 mm
Diaphragm flange and web thickness	25 mm
Damping Ratio	1%

4. SENSOR CLUSTERING FOR THE BRIDGE

From the above discussion, it is observable that the response of a particular DOF can be predicted from the response of its adjacent DOFs. This methodology can be utilized to develop different sensor clusters. These models can then be used to extract damage related features in order to identify, locate and quantify the damage. For this bridge, all the joints between main girder and diaphragms are considered locations of interests where accelerometers are placed to obtain bridge response. The accelerations of the support nodes are excluded from the model since these are very close to zero and cause instability in the ARX model. The position of accelerometers in the bridge is shown in Figure 2. Each node act as reference channel (output of ARX model) and adjacent channel for other reference channels (input of ARX model) depending on their connectivity. Eight different sensor clusters were created for each reference channel and these are shown in **Error! Reference source not found.**.

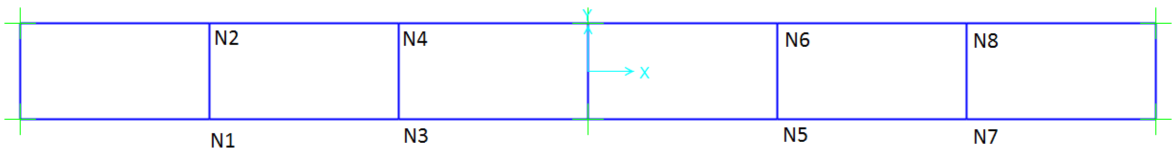

Figure 2. Node numbers for the bridge

Table 2. Sensor clustering network for the bridge

Sensor Cluster	*Output of ARX Model (reference Channel)*	*Input of ARX Model*
1	N1	N1, N2, N3
2	N2	N1, N2, N4
3	N3	N1, N3, N4, N5
4	N4	N2, N3, N4, N6
5	N5	N3, N5, N6, N7
6	N6	N4, N5, N6, N8
7	N7	N5, N7, N8
8	N8	N6, N7, N8

Acceleration responses were obtained from each node of the bridge as the train move over it. The train loading used in the study is AREMA COOPER E80. The response is obtained in the form of vertical acceleration for the entire duration the train is over the bridge plus additional 4 seconds of free response after the train fully crossed the bridge. For the study, only the last 4 seconds of free response is considered. This is due to the fact that, duration of full response depends on the speed and length of the train and can complicate results if different trains are combined to obtain results. After creating the ARX models for both undamaged and damaged condition utilizing the sensor clustering framework, Damage Features(DF)s are extracted from the ARX models to detect damage. For this study, DF is defined as the difference of Fit Ratios(FR) obtained using Eq. (4). Here, $u, \hat{u}$ and $\overline{u}$ represents measured output, predicted output and mean of measured output respectively. The DF is calculated using Eq. (5). Here FR_h is the fit ratio of the healthy data and FR_d is the fit ratio of the damaged data. By comparing the differences in values of DFs between different channels, the presence of damage, its location and severity can be assessed.

$$FR = \left(1 - \left|\frac{u-\hat{u}}{\left|u-\overline{u}\right|}\right|\right) \times 100 \quad (4)$$

$$DF = \frac{|FR_h - FR_d|}{FR_h} \times 100 \quad (5)$$

5. DAMAGE SIMULATIONS

In this study, different real-life damage case scenarios on the railway bridge are considered. These damage cases involve changes in the main girder stiffness, the formation of hinges at joints and changes in boundary conditions. Changes in member stiffness can be caused by corrosion, fatigue, and damage due to accidents or collisions. This problem is simulated on the railway bridge model by reducing the modulus of elasticity of the affected member. Sometimes connections can lose its moment carrying capacity due to accidental removal of bolts, decaying of welds etc. This is represented by forming a hinge at a connection. The changes in boundary conditions are caused by corroded or blocked bridge supports which develop unintended fixity at the abutments. This problem is simulated by fixing the support. Also, sometimes supports settle due to scouring of river floor, resulting in inactive support. These different damage cases are shown in Figure 3. The cases are named in Table 3.

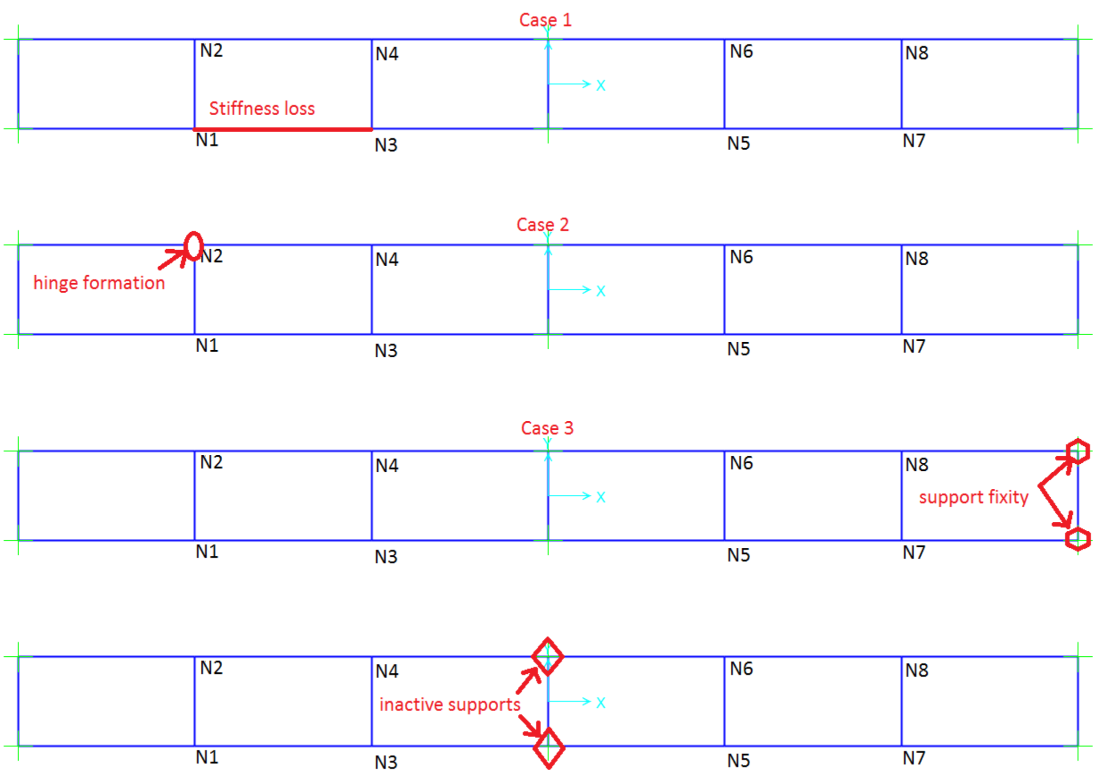

Figure 3. Studied damage cases on the railway bridge

Table 3. Damage cases under study

Damage Case	Damage location
Case 1	Main girder section between N1 andN3: reduction of the elasticity modulus by
a	50 %
b	90 %
Case 2	Connection node 2, hinge formation

Case 3	Bearings on the right abutment become fixed
Case 4	Intermediate support removal due to scouring

6. ANALYSIS & RESULTS

The analysis results are discussed in the subsequent sections. The results are displayed in terms of DFs vs Datasets plots, where each data set corresponds to DF obtained at a particular speed. In this study, it is assumed that the obtained responses are noise free. Therefore, any damage feature greater than 0 is indicative of damage occurrence.

6.1 Damage Case 1: Main girder stiffness loss in section 1-3

In this damage case, the main girder section between joint 1 and 3 is assumed to have lost its stiffness by a)50% and b) 90%. The Damage Feature (DF)s for case (a) and (b) are shown in Figure 4.

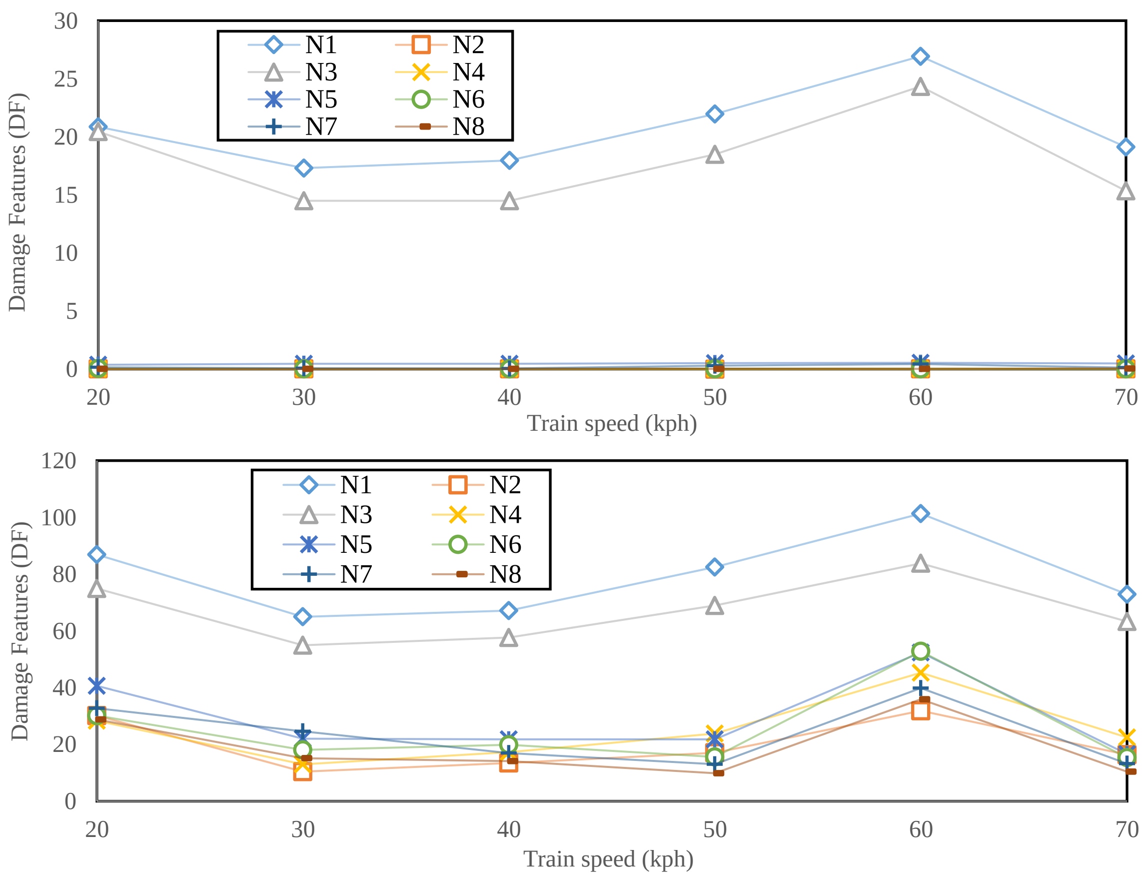

Figure 4. Damage Features (DFs) for Damage Case 1: (top) case a, (bottom) case b

It can be observed that the highest DFs are obtained for N1 and N3 for both case (a) and case (b). This is expected since this are the two joints closest to the location of damage. DFs for all other nodes are significantly smaller than N1 and N3, indicating that damage has occurred and its location is likely between N1 and N3. It is also noticeable that the corresponding DFs are significantly higher for case (b) than for case (a). This quantifies that case (b) is a more severe damage than case (a). For case (a), all the DFs other than N1 and N3 are close to zero, indicating it is more of a local damage and only section 1-3 is affected. For case (b), all the DFs other than N1 and N3 exhibit DFs at least 10. This is because 90% stiffness reduction in the portion of the main girder considerably affects the other parts of the bridge and thus is more of a global damage.

6.2 Damage Case 2: Hinge formation at node 2

In this damage case, the connection at N2 is assumed to have lost its moment capacity (i.e., a hinge has formed). The Damage Feature (DF)s for this case is shown in Figure 5. In this case, very high DFs are obtained for N2 and N4, while DFs for other nodes are close to zero. N2 and N4 are both parts of the same main girder, it is expected that these two nodes will have high DFs since the loss of moment capacity at a joint affects the entire girder. N2 exhibits significantly higher DFs than N4. Overall, this indicates that damage has occurred which is likely to be located between N2 and N4 with a higher chance of location at N2. It is to be noted though that, despite being directly connected to N2, N1 does not show DFs. This may be because, N1 is connected to N2 through the diaphragm, which does not carry the vertical load, and hence does not transfer moment. Comparing the DFs with Case 1, it can be also concluded that, Case 2 while a local damage case, is more severe than case 1(a) and less severe than case 1(b).

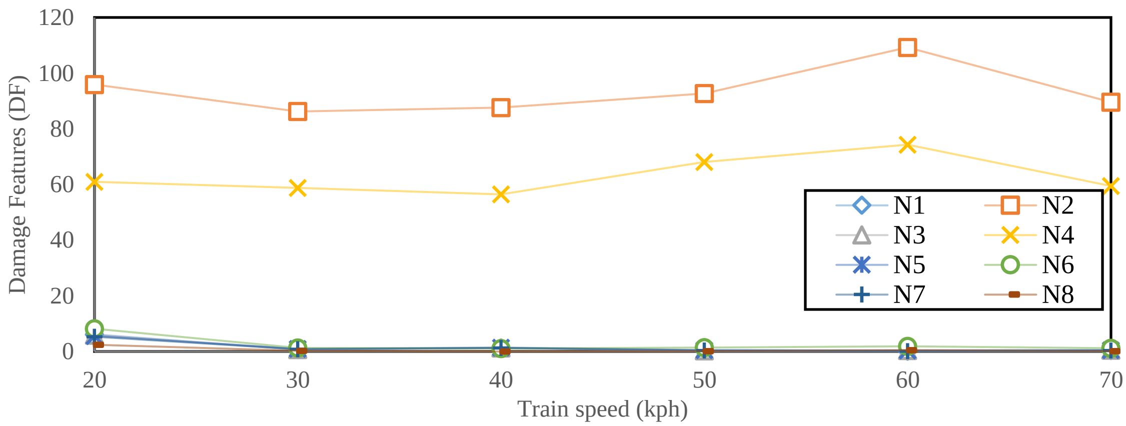

Figure 5. Damage Features (DFs) for Damage Case 2

6.3 Damage Case 3: Support fixity at the right abutment

In this damage case, the bearings at the right abutment are assumed to develop rotational fixity. (i.e., abutment becomes fixed support. The DFs for this damage case is shown in Figure 6. In this damage case, it is observed that all the nodes indicate damage since this is a global damage case which affects the entire structure. The highest DFs (magnitude around 100) are obtained at N7 and N8 which are closest to the right abutment. N5 and N6 also show high DFs (maximum around 70), as these two nodes are neighbors to N7 and N8. The other nodes (which are on Span 1) show lower DFs compared to those on span 2. Comparing the magnitudes of DFs with previous cases, it is noticeable that this is a severe global damage case with higher damage features than case 1a and 2 and comparable to case 1b.

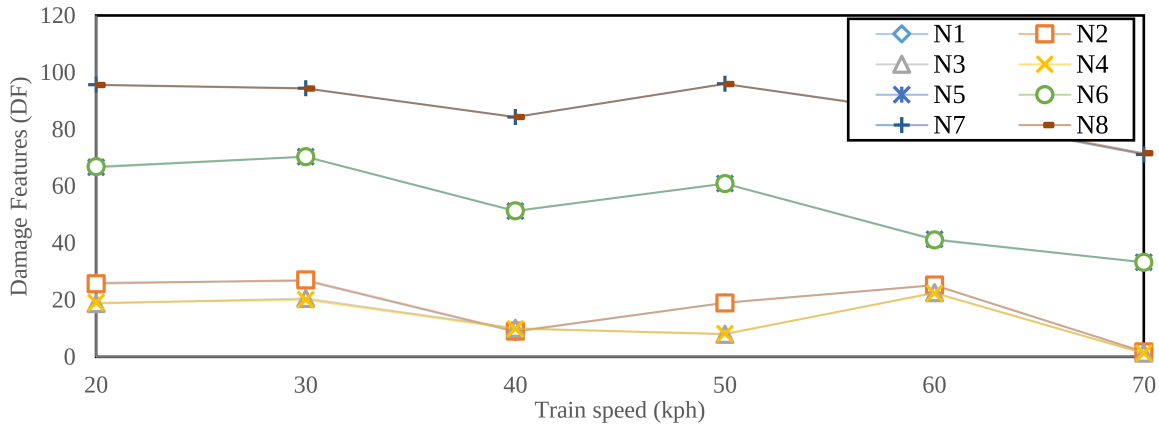

Figure 6. Damage Features (DFs) for Damage Case 3

6.4 Damage Case 4: Intermediate support removal due to scouring

In this damage case, it is assumed that the intermediate supports have become inactive due to scouring of the pier. The results for this case is shown in Figure 7. Similar to damage case 3, this is a global damage, which is expected to affect the entire bridge. This is because removal of the pier of a 2-span bridge effectively makes it a single span bridge and this will change the force transfer mechanism of the entire bridge structure and redistribution of forces will occur. The results show that the entire structure is highly damaged. The nodes N3, N4, N5 and N6 exhibit very high DFs (magnitude around 140), as these are directly connected to the intermediate support which has been removed. So, these nodes are most severely affected. For the remaining nodes, average DF of around 80 is observed which is still very high. Overall, this is the most severe damage case among all discussed.

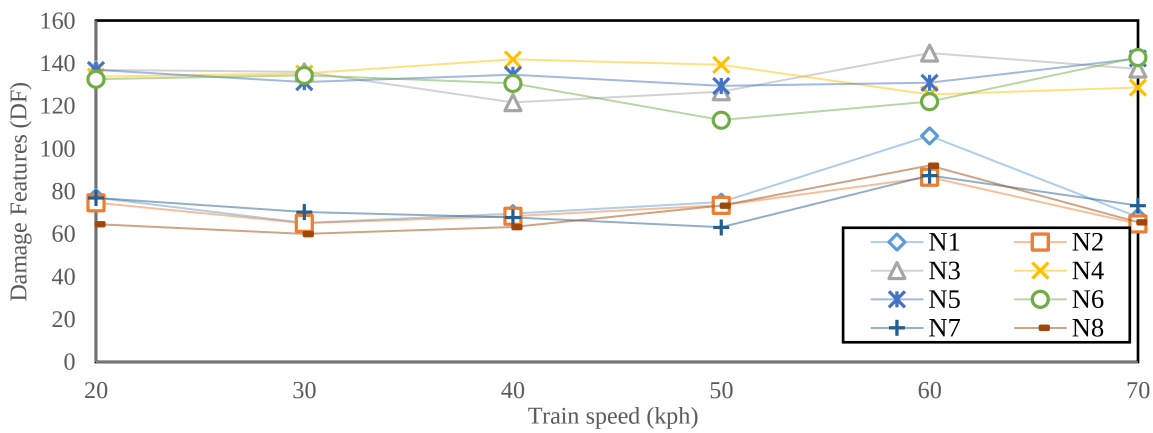

Figure 7. Damage Features (DFs) for Damage Case 4

7. SUMMARY & CONCLUSION

A robust and effective sensor clustering based damage detection methodology for structural health monitoring of railway bridges utilizing time series analysis of acceleration response from different accelerometer locations is presented. This proposed method is demonstrated by a numerical bridge model under undamaged and damaged conditions to detect structural changes and damage.

The results presented in this study show good agreement between predicted and expected damage features. Thus, the results demonstrate the potential of the proposed methodology is for railway bridges. The major advantage of the method is that damage is detected and localized and quantified using bridge response under operational train loading. Therefore, the need for on-site testing could be reduced. This method could be especially uselful to detect damage in inaccessible bridge locations where on-site testing could be difficult. Moreover, once the system is installed, continuous real-time monitoring is possible. While this method is developed for steel railway bridges, the method is robust enough to be applied for condition monitoring and maintenance to different kinds of bridge and other infrastructures subjected to dynamic loading. Consequently, this methodology of damage detection can be implemented as part of SHM system for many different railway bridge structures. It offers the opportunity to detect damage at an early stage to develop economical maintenance strategies and address the problem before they become too costly to repair. Future plans for this on-going study includes investigation of operational loads with varying configurations and noise effects as well as experimental investigations in the laboratory and real-life bridges.

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