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Evaluation of Dynamic Deformations of Slab-on-Girder Bridge under Moving Trucks with Corrosion-Damaged Columns

Abstract: A simplified hybrid linear/nonlinear dynamic finite element analysis (SHDFEA) based on enhanced inspection is introduced as part of a proposed semi-quantitative assessment approach of aged bridge columns. The focus is on the evaluation of the dynamic characteristics and behaviour of slab-on-girder bridges under moving trucks when their columns are subjected to severe corrosion damage. The proposed SHDFEA uses nonlinear finite element analysis to evaluate the damaged columns stiffness, mass and damping throughout the step-by-step time history analysis. The efficiency, accuracy and stability of the proposed SHDFEA are verified through different case studies, where very high numerical stability and fast convergence are achieved. The results show that the nonlinear dynamic analysis presents the most general approach to evaluate the effects of any level of corrosion damage in the column for any case of loading, boundary conditions and any progressive change in the column properties. However, linear dynamic analysis is found to be the most economical alternative when the columns are overdesigned. It is found that with severe local corrosion damage in traditionally designed slab-on-girder bridges columns, their static behaviour mostly remains in the elastic range, and the traffic load magnitude remains below their ultimate static capacity. It is also found that the changes in the bridge dynamic performance parameters are marginal when over-designed columns have severe corrosion damage and their safety and stability are critical.

Keywords: semi-quantitative assessment approach, corrosion damage, slab-on-girder bridge, nonlinear dynamic analysis, finite element method.

1. Introduction

Evaluating the dynamic performance of a bridge structure under service loads is a major component of the bridge design requirements as well as the assessment of aging bridges. At present, qualitative assessment approaches are used in most of North American states and provinces. With the lack of accuracy of existing assessment approaches, it is required to develop a more efficient assessment approach that can quantitatively evaluate the bridges structural performance when safety critical elements (the bridge columns, for instance) are partially damaged. The authors have proposed a semi-quantitative assessment framework (see Mohammed et al. [1]) introducing a limit states evaluation (LSE) approach in parallel to the limit states design (LSD) approach currently in practice in North American bridge design codes [2 & 3]. The major evaluation limit states are: evaluation ultimate limit state (ULS), evaluation serviceability limit state (SLS) and evaluation earthquake limit state (ELS). The focus of this paper is on the evaluation serviceability limit state, which is mainly based on evaluating the maximum static and dynamic deformations when the aging structure is under service loads.

The requirements for a quantitative assessment of the serviceability of aging bridges address the importance of estimating, measuring and monitoring the changes in bridges dynamic characteristics for different critical levels of corrosion damage. The damages that result from reinforcement corrosion of the bridge column, such as spalling of concrete cover, reduction in reinforcement section, fracture of one or more stirrups, etc., can be defined as critical damage levels. As a transition stage preceding the development of a time-dependent quantitative assessment approach, enhanced visual inspection (or

simply enhanced inspection) can be employed as the major source of input to the proposed quantitative assessment framework.

It is thought that the changes of basic dynamic characteristics such as the natural frequencies of the bridge systems (or bridge elements) can be used as an accurate measure to identify and quantify their state of damage. This is based on the observed reductions of the stiffness and the mass of the affected bridge members. However, the natural frequencies are proportional to the square root of the structural element stiffness-to-mass ratio, and hence they are not apparently affected by the discretization approach or to the variation of the finite element characteristics. Most bridge columns are conservatively designed (over-designed) and their service-load-over-capacity ratios (SLOCR) are usually low or sometimes very low (usually 25% to 50%). For aging bridges, the increase in the truck load, traffic density, frequency and speed in addition to the reduction in the load capacity of corrosion-affected columns could apparently increase their SLOCR. Higher design SLOCRs are targeted for newly constructed bridges to reduce their initial cost or to satisfy aesthetical needs, for the increasing use of high performance (HP) materials, and for performance-based design and structural optimization approaches. On the other hand, the design truck load, traveling velocity, number of truck axles, and the average number of trucks passing over bridges have been extensively increased in the past four decades. This has raised the concern about the possible increase of the vibration and dynamic deformation of the bridges, which could exceed the acceptable limits in the bridge design codes.

Bridge design codes in North America [2 & 3] limit the vibration amplitude to an acceptable level as part of the serviceability limit state checks. Depending on the

slenderness, damping, and continuity of the bridge superstructure and its integration with the bridge substructure, the vibration of the bridge could be sensitive to the dynamic excitation of the traffic load. For short and medium-span bridges, the significant changes of the superstructure or substructure stiffness and mass due to reinforcement corrosion of the bridge columns can result in distinguished changes of the vibration amplitude and/or mode shape. Corrosion-related damages to bridge substructures could result in significant reduction in their structural capacities and safety. However, it is not reported whether the changes in bridges substructures (columns) capacity to a critical level result in a similar critical change in the dynamic characteristics of the bridge superstructure under traffic load. Such changes in the bridge dynamic characteristics are of major interest to the bridge owners and maintenance industry as a basis for the development of health monitoring and/or non-destructive evaluation techniques. On the other hand, many observations of the change of vibration amplitude of the bridge superstructure related to reinforcement corrosion of the deck slab or girders or both are reported [4].

With the progress of reinforcement corrosion in the affected zones, the bridge column capacity is reduced, and hence the load-over-capacity ratio of the substructure (bridge columns) is increased. Although the service load is lower than ultimate capacity and the structural behaviour is mostly in the elastic range when subjected to traffic loads, the resulting damage of reinforcement corrosion and its effects on the concrete strength and confinement could shift the elastic behaviour of the column to the plastic range. Many researchers have observed that the ductility of the reinforcement is also reduced with the progression of corrosion [5]. Typically, linear dynamic finite element analysis is used to evaluate the bridge vibration. However, with the shift of the structural behaviour of the

bridge columns to the plastic region, the material nonlinearity should be taken into account when estimating the instantaneous element stiffness in the corrosion affected zones. Hence, nonlinear modelling of the substructure should be essential.

The objective of this paper is to develop a simplified hybrid linear/nonlinear dynamic finite element analysis (SHDFEA) that enables an accurate evaluation of the vibrations of aging bridges when the bridge columns are subjected to severe reinforcement corrosion, as part of a proposed semi-quantitative assessment framework. The emphases are on: (i) the numerical efficiency and stability of the analysis approach; (ii) the approach capability to capture the characteristics changes in dynamic deformation distributions over the superstructure and the substructure; (iii) the approach ability to determine the changes in the dynamic load allowance (or the impact factor); (iv) the validation of cases where the simplified linear dynamic finite element analysis (SLDA) is adequate.

2. Simplified hybrid linear/nonlinear dynamic finite element analysis (SHDFEA) as a part of semi-quantitative assessment framework (SQAF)

Figure 1 shows the proposed SQAF, which has six major parts: (I) data input; (II) quantifying the reinforcement corrosion and their effects; (III) evaluating columns performance under combined corrosion and ultimate loads (evaluation-ULS); (IV) evaluating columns performance under combined corrosion and service (or traffic) loads (evaluation-SLS); (V) evaluating columns performance under corrosion and seismic loads (evaluation-ELS) (only in high risk seismic zones); and (VI) semi-quantitative assessment and reporting. In North American bridge codes [2 & 3]; the serviceability limit state is a major limit state to be verified for bridge design. The nonlinear dynamic finite

element analysis is the basis for the evaluation serviceability limit state of the proposed semi-quantitative assessment framework (SQAF). The evaluation of the column structural performance under combined reinforcement corrosion and ultimate loads, and the evaluation of the column structural performance under combined reinforcement corrosion and seismic loads are presented in two separate papers (Mohammed et al. [1 & 6]).

The first part of the proposed SQAF includes three data-input tasks: (i) the structural material and geometrical data including boundary conditions; (ii) the loading data; and (iii) the enhanced inspection and reinforcement corrosion data. In the first task, the data are collected from the original design information/report and shop drawings (if available), and the field tests on the materials in their current state (if possible). The difference between the original design loads and the present loads on the bridge column under consideration are to be determined.

In task (iii), the enhanced in-depth visual inspection (or enhanced inspection) is used in the proposed semi-quantitative assessment framework to provide detailed field measurements on the dimensions of the damaged concrete zone, the volume of concrete spalling of the corrosion affected zone (FHWA-RD-01-020, 2001). Determining the state of corrosion damage of the rebars and their mechanical properties that used in the model is more complicated. In this study, a hybrid approach to find the steel rebars properties is suggested. Out of all the columns of the bridge pier under assessment, the most damaged one is selected, and the damage status and size and the material properties can be generalized over the other columns of the pier for a conservative assessment of the substructure. The most damaged rebars are then identified, and samples of suitable size

and number are collected. In the lab tests of the steel rebars, a precise measurement of the maximum reduction of rebar diameter or the reduction in the reinforcing steel thickness is found for each sample (using different available techniques ranged from hand cleaning and measurement to the use of recent technology of 3D laser scanner). Assuming a constant rate of corrosion, the corrosion current density and the time elapsed to generate the measured reduction in the reinforcing steel thickness are then found using the data of different samples and the empirical equations (see Mohammed et al.[1]). Using the evaluated reinforcement corrosion parameters and empirical formulae (for example, see Lay and Schiebl 2003 and Cairns et al. 2005) the loss of strength and ductility of the reinforcement subjected to corrosion attack can be estimated. On the other hand, the strength and ductility of the reinforcement can be evaluated by tensile tests if the corroded rebar samples can be available in suitable length.

The proposed SQAF identifies four major damage cases due to reinforcement corrosion, which are: (a) cracking of concrete due to the corrosion; (b) spalling of concrete cover; (c) failure of one or more stirrups; and (d) a more advanced state of corrosion progress, which may lead to the structural failure of the column through complete loss of the confinement or rebar(s) buckling. The details of each of these major possible deterioration states are shown in Fig. 2.

The evaluation SLS is integrated with the proposed SQAF, where at each major case of quantifying the effects of the reinforcement corrosion, the flowchart shows the link to one of the three major evaluation limit states: ULS, SLS (which is discussed in details in this paper) and ELS. For instance, the simplified hybrid linear/nonlinear dynamic finite element analysis (SHDFEA) is the basis of four tasks of the evaluation-SLS, which are:

(a) establishing the static mid-span deflection when the truck is moving very slowly over the bridge length; (b) establishing the dynamic deflection of the bridge superstructure under front axle for maximum truck speed and for extreme corrosion damage; (c) establishing the dynamic load allowance DLA as per CHBDC or the impact factor (IF) as per AASHTO for different states of damage of the column; and (d) establishing the dynamic lateral displacement of the bridge columns versus the truck location for different cases of damaged and undamaged columns. These four tasks of evaluation-SLS end with the preparation of the required data for the SQAF. The details of the proposed SQAF are presented in a separate paper (Mohammed et al. [7]).

3. Appropriate dynamic analysis approach of slab-on-girder bridge subjected to traffic combined with reinforcement corrosion

The focus of this paper is on the effects of local corrosion damage in slab-on-girder bridge columns on the vibrations of the bridge superstructure and columns themselves. It is believed that for the purpose of the development of the dynamic analysis approach, the bridge columns represent the most general members under axial load and moment in addition to their boundary conditions. Columns also represent the most critical elements for the bridge safety; developing a tool to quantify their damage effects on the bridge vibrations is of high interest to bridge owners, practicing engineers and researchers. In order to avoid the interaction of the corrosion damage effects of different zones of the bridge superstructure and columns on the bridge vibration, the bridge superstructure is assumed to be in a good condition with no corrosion damage. Figure 3-a shows a highway overpass where a three-lane slab-on-steel-girder (superstructure) is supported by a typical multi-column reinforced concrete (RC) pier with a cap RC beam (substructure).

Aging slab-on-girder bridges are affected by corrosion aggressive environment due to the use of de-icing-salt over the long winter in Canada and Northern states of the United States. Figures 3-a and 3-b show the traffic directions over and under the bridge, and they show a typical road configuration in the highway overpass. The splash of salty water affects the bridge columns above the traffic level, where corrosion damage develops in approximately the middle third of the column height (see Fig. 3-b).

When a heavy truck passes over the bridge superstructure as shown in Fig. 3-b, the truck load is transferred to the bridge substructure based on the location of the truck with respect to the two piers. As the truck passes over the left column (assuming the traffic direction is from left to right), the load transferred to that column is the maximum possible load. When the truck moves away from the left column, the load on that column is decreased and reaches its minimum level as the truck leaves the bridge. As shown in Fig. 3-c, for example, the truck front axle at point B_i gives lower load on the left column than when the truck front axle is at point A_i . As the bridge is designed to satisfy ultimate and serviceability limit states, the bridge column is assumed to have enough safety at ultimate limit state, and it would always behave in the elastic range when subjected to service load. As shown in Fig. 3-d, and right after the bridge construction is completed, the ultimate load in the left column when the front axle is at B (B_{iu}) results in elastic deformation, while the ultimate load in the left column when the front axle is at A (A_{iu}) results in plastic deformation well below the failure load (as per typical conservative design of bridge columns). With the onset and propagation of reinforcement corrosion and related damage, the strength, stiffness and deformability range of the columns are reduced as shown in Fig. 3-d. For severe level of corrosion damage, it is expected that

when an overloaded truck is at a critical location of the bridge, the bridge load capacity could be lower than the applied load. Even under typical truck load (service level), the column deformation could be in the non-elastic range. Hence, nonlinear analysis has to be included in the development of the dynamic analysis of the bridge column subjected to traffic load combined with reinforcement corrosion. In case a bridge column is overdesigned and continues to behave elastically under all levels of service loads even with severe levels of corrosion, then elastic dynamic analysis can give an economical solution.

4. Background of modelling the bridge structure and traffic load

Over the past three decades, several attempts have been conducted to model the effects of dynamic interaction between vehicles and a bridge system. It is usually considered as two dynamic systems that are introduced by deriving the equation(s) of motion of the interaction system. In almost all previous studies (except Mohammed et al. [8]), only the bridge superstructure is modelled, while the substructure and the superstructure-to-substructure connection (or internal boundary conditions) are ignored.

Lin and Trethewey [9] presented a finite element formulation of beams subjected to different types of moving loads (a moving concentrated force and a moving one and two-foot long dynamic "system"). Their objective was to derive the governing equations for the beam and moving dynamic system interaction, wherein cubic Hermitian polynomials were employed as interpolation functions of the finite element formulation. The Runge-Kutta method was used to solve the governing equations and hence obtain the dynamic response of both, the beam and the moving load (or "system"). Law and Zha [10],

Zambrano et al. [11], and Asnachinda et al. [12] presented analytical solutions for simply-supported and continuous beams with uniform cross sections under moving loads. The main objective of Law and Zha [10] and Zambrano et al. [11] studies was to determine the dynamic damage of the concrete bridge under moving vehicular loading. Asnachinda et al. [12] performed analytical and experimental studies to investigate the dynamic axle load identification of multiple vehicles from the bridge bending strain response. In these studies, the vehicle is modelled as a moving single or multi-degrees of freedom systems, and the bridge is modelled as a simply-supported Euler-Bernoulli beam. In these studies and in order to analyse the interaction of vehicle-bridge systems, two sets of equations of motion are presented: one is for the vehicle and the other for the bridge structure. Then the equations of motion of the interactive-system are solved to obtain its dynamic response.

Ali [13] has extended the application of the finite strip method to the dynamic analysis of bridge structures under moving trucks. The author studied single-span slab-on-girder bridges, box-girder bridges, and multi-span slab bridges. The study considered several modelling approaches of the vehicles, such as moving load, moving mass, sprung mass and un-sprung mass connected together by elastic springs and a dash-pot, and a two-axle vehicle.

Gong and Cheung [14] have studied the dynamic response of long-span box-girder bridges under moving loads using the finite strip method. Their study concluded that (i) the dynamic response of a bridge depends mainly on the bridge superstructure dynamic characteristics, which include the natural frequencies and vibration mode shapes, and (ii)

the vehicles-bridge structure interaction might be influenced by vehicle mass, traveling speed, damping ratio of the vehicles, and the ratio of the vehicle mass to the bridge mass.

Mohammed et al. [8] proposed a linear dynamic finite element analysis approach for slab-on-girder bridges under a moving vehicle when the columns are subjected to a time-dependent corrosion process. The bridge structure is modelled as a static and dynamic system using the direct stiffness method. Three types of analysis are performed: (i) static analysis, wherein the traffic load is defined by a static truck load positioned to induce maximum static response; (ii) free vibration analysis; and (iii) linear time-history analysis. The approach enables evaluating the effects of the current deterioration of bridge concrete columns on the dynamic performance of the bridge superstructure compared to its dynamic performance when no corrosion takes place.

5. Proposed simplified hybrid linear/nonlinear dynamic finite element analysis (SHDFEA)

The proposed SHDFEA approach is developed to estimate the vibration of different components of the slab-on-girder bridge: the superstructure as well as the damaged columns themselves. The effects of corrosion damage in the bridge columns on the vibration of the superstructure are more interesting than the vibration of the columns themselves because: (i) the superstructure vibration amplitude is limited by codes requirements; (ii) the direct vehicle-superstructure contact results in the highest possible dynamic response to be in the superstructure; and (iii) the hinge and roller connecting the superstructure to the substructure (in addition to the dampers) reduce the dynamic load transferred to the columns. It is important to mention that the proposed dynamic analysis approach is developed for two-axle trucks; however, it is extendable to any multi-axles

truck with any configuration when the basic dynamic characteristics of the truck are available.

Figure 4 shows a brief flowchart of the proposed SHDFEA, where both the static and the linear/nonlinear time history analysis of traffic load are performed. The input data of SHDFEA are transferred from earlier stages of the semi-quantitative assessment framework (SQAF), which include enhanced inspection and quantifying the effects of reinforcement corrosion as stated in Section 2 (see Figs. 1 & 2). As mentioned in Section 3, the bridge superstructure is assumed to behave in the linear elastic range under the traffic load (service load), as no corrosion damage is assumed in the superstructure. The bridge columns, on the other hand, are subjected to high axial load, and their behaviour could change from pure compression to combined axial compression and bending depending on the level of applied load and the location of the corrosion-damaged zone. The elements stiffness, mass and damping matrices of the corrosion-damaged columns are formulated using the nonlinear finite element analysis (NLFEA) approach proposed by the authors [1].

The SHDFEA procedure proposed in this paper employs a non-linear time-history analysis of the bridge under traffic load, where the stiffness, mass and damping matrices of the column elements are successively updated at each truck new location. As the truck moves, the loads on the columns vary, and hence the material properties in the damaged and no-damage zones of the columns also vary. The sectional rigidities, the elements stiffness, mass and damping matrices are all variable and dependent on the load and state of corrosion damage as observed by enhanced inspection. It is assumed that at each truck location, a nonlinear static analysis is performed on the column to estimate the element

stiffness and mass, and hence estimate the damping as it is assumed proportional to the stiffness and mass. The instantaneous nonlinear global dynamic matrices are then assembled into the equation of motion in every integration step of the time history analysis.

At each step of the time history analysis, a large amount of data is generated in the sub-processes of the nonlinear analysis to derive the instantaneous stiffness and mass matrices of the different elements. An interactive data filing system is developed and used to manage the data transfer between the Random Access Memory (RAM) and the external memory. The data filing system enables fast access to the data when needed in the instantaneous assembly process as part of the time history analysis. The data for each element matrix are transformed into a vector of higher order and saved as parallel records. A file-name is generated for each record and saved for reference; the record is saved randomly on the external drive. The symmetry of the matrices is taken into account to minimize the mathematical operations, the memory use and the time required for data processing and transfer. The convergence at three different levels (sectional, element and dynamic-time-history analysis) is checked and ensured. The filing system together with the displacement field tuning convergence used in the NLFEA [1] enable the generalization of the approach to be used in more complicated structural systems that have many structural elements.

As an example, at any step of the time history analysis, the instantaneous global stiffness matrix has the following general form, where it includes parts derived from the linear formulation ($[K_{\text{superstructure}}]L$; linear stiffness of superstructure) and other parts derived

from the nonlinear formulation ($[K_{Left\ column}]_{NL}$ & ($[K_{Right\ column}]_{NL}$; nonlinear stiffness of substructure) :

$$\begin{bmatrix} [K_{Left\ column}]_{NL} & & \\ & [K_{superstructure}]_L & \\ & & [K_{Right\ column}]_{NL} \end{bmatrix} \quad (1)$$

5.1 Modelling the bridge structure

A typical slab-on-girder bridge is simply supported on columns (piers) or abutments (Figs. 3-a and 3-b). The focus of this study is on developing the SHDFA based on the bridge structure-traffic dynamic interaction and exploring the effects of columns' reinforcement corrosion damage on the dynamic behaviour of the bridge. Hence, it is essential to model both the superstructure and substructure as one structural system. The SHDFA is required to integrate the components of the slab-on-girder bridge, the superstructure and the columns, into one structural system involving an internal hinge and roller. Since there is no load or expected response/deformation in the bridge lateral direction orthogonal to the traffic direction (or along the Z-direction as shown in Fig. 3-a), the bridge structure can be modelled as a two-dimensional frame (in the X-Y plane as shown in Fig. 3-b). The frame in the SHDFA is formed of a beam, representing the superstructure, and two columns representing the substructure. The connections between the beam and the columns are represented by an "internal" hinge on one side of the beam and an "internal" roller on the other side. The model also accounts for any possible eccentricity in both connections. The bases of the two columns are assumed fully fixed.

The columns sections are considered as reinforced concrete "composite" sections for the purpose of the nonlinear analysis. The sectional rigidities that lead to the estimation of

elements stiffness, mass and damping matrices of the corrosion-damaged columns are formulated using the nonlinear sectional analysis and the nonlinear finite element analysis (NLFEA) approaches proposed by the authors [1&15]. The superstructure is formed from either slab-on-steel girders or slab-on-prestressed concrete girders. The slab is assumed to be compositely integrated to the girders. The distribution of the traffic loads (truck or lane load) in the lateral direction (Z) is assumed to follow the CHBDC load distribution approach. In the proposed SHDFA, the composite beam and the columns are modelled as two-node frame elements with three degrees of freedom at each node. The self-weight of the bridge elements is included as a uniform load. The instantaneous damping matrices are derived assuming that the damping is linearly proportional to the elements mass and stiffness (Mohammed [16]).

As mentioned earlier, the beam-column joints are modelled as an internal hinge in one side and as an internal roller in the other side of the bridge system. The fulfilment of the boundary conditions in the beam-to-the-column joints is considered as follows:

Hinge: (i) continuity in the vertical displacement V; i.e., the beam and column have the same V; (ii) continuity in the horizontal displacement U; (iii) discontinuity in the rotations, i.e., the beam and column have independent rotations, or $\theta_{beam} \neq \theta_{column}$.

Roller: (i) continuity in the vertical displacement V; (ii) discontinuity in the horizontal displacement U, or $U_{beam} \neq U_{column}$; (iii) discontinuity in the rotations, or $\theta_{beam} \neq \theta_{column}$.

Incorporating the above internal boundary conditions (McGuire et al. [17]), the overall stiffness matrix of the modelled frame is:

$$\begin{Bmatrix} F_e \\ F_o \end{Bmatrix} = \begin{bmatrix} K_{ee} & K_{eo} \\ K_{oe} & K_{oo} \end{bmatrix} \begin{Bmatrix} \Delta_e \\ \Delta_o \end{Bmatrix} \quad (2)$$

From Eq. (2), the overall stiffness matrix and load vector of the modelled frame can be expressed by:

$$[K] = [[K_{ee}] - [K_{eo}][K_{oo}]^{-1}[K_{oe}]] \quad (3)$$

$$\{F\} = \{ \{F_e\} - \{[K_{eo}][K_{oo}]^{-1}\{F_o\}\} \} \quad (4)$$

where $\{F_e\}$ is the vector of applied forces on the free nodes; $\{F_o\}$ is the vector of forces on the roller and hinge nodes; $[K_{ee}]$ is the stiffness matrix associated to degrees of freedom of free nodes; $[K_{oe}] = [K_{eo}]$ are the interaction stiffness matrices; $[K_{oo}]$ is the stiffness matrix associated to degrees of freedom of roller and hinge nodes; $\{\Delta_e\}$ is the vector of deformations associated to free nodes; $\{\Delta_o\}$ is the vector of deformations associated to the roller and hinge nodes; $[K]$ is the overall stiffness matrix of the bridge structure; and $\{F\}$ is the overall load vector of the bridge structure.

5.2. Modelling the vehicle

The vehicle is modelled as a dynamic system with two degrees of freedom: the vertical displacement (y) and the rotation (θ), as shown in Fig. 5. It is assumed that the two axles of the vehicle remain in contact with the surface of the bridge superstructure at all times. Only vertical vibration is considered. The governing equations of the moving dynamic system and the bridge are adopted from Lin and Trethewey [9], where the overall vehicle-bridge system is presented as follows:

$$\begin{bmatrix} [M] & \sum_i [N_i]^T f_i & \sum_i [N_i]^T f_{oi} \\ [0] & m_1 & 0 \\ [0] & 0 & I_a \end{bmatrix} \begin{Bmatrix} \{d\} \\ \{y\} \\ \{\theta\} \end{Bmatrix} + \begin{bmatrix} [C] & [0] & [0] \\ \sum_i -c_i [N_i] & \sum_i c_i & \sum_i (-1)^{i+1} c_i b_i \\ \sum_i (-1)^i b_i c_i [N_i] & \sum_i (-1)^{i+1} c_i b_i & \sum_i c_i b_i^2 \end{bmatrix} \begin{Bmatrix} \{d\} \\ \{y\} \\ \{\theta\} \end{Bmatrix} + \begin{bmatrix} [K] & [0] & [0] \\ \sum_i (-c_i s[N_i]_a - k_i [N_i]_l) & \sum_i k_i & \sum_i (-1)^{i+1} k_i b_i \\ \sum_i (-1)^i (b_i c_i s[N_i]_a + b_i k_i [N_i]_l) & \sum_i (-1)^{i+1} k_i b_i & \sum_i k_i b_i^2 \end{bmatrix} \begin{Bmatrix} \{d\} \\ \{y\} \\ \{\theta\} \end{Bmatrix} = \begin{Bmatrix} \sum_i [N_i]^T f_i \\ 0 \\ d_F(t) \end{Bmatrix} \quad (5)$$

where $[M]$, $[K]$, $[C]$ are the structural mass, stiffness, and damping matrices of the bridge, $\{\ddot{d}\}$, $\{\dot{d}\}$, $\{d\}$ are the nodal acceleration, velocity and displacement vectors, respectively, f_i is the magnitude of a concentrated force, $[N]$ is the shape function, m_1 , k_i , c_i are the spring mass, stiffness, and damping of an axle of the moving system, respectively, $\ddot{y}$, $\dot{y}$, y are the vertical acceleration, velocity and displacement of the spring mass, respectively, and b_i is the vehicle axle position from its centre of gravity.

The vehicle dynamic system is first integrated with the bridge superstructure, and then it is integrated with the proposed beam-and-columns system. The combined equations of the vehicle-bridge system given by Eq. 5 are solved by the step-by-step integration method. The dynamic response of the bridge under the moving vehicle is expressed as a function of the transverse displacement of the structure $w(x,t)$, i.e., $w = [N]\{d\}$.

6. Modelling of corrosion load

Field surveys have shown that RC bridge columns in cold climates are among the bridge elements with highest observed corrosion-related damage. The most critical column zones that suffer from corrosion are the top (superstructure drain zone) and mid-height "splash zone" (Andersen [18]). Reinforcement corrosion is a continuous long-term process, leading to reduction in cross-sectional area of the affected steel bars and loss of

concrete section as a result of longitudinal cracking and spalling. Hence, reinforcement corrosion may cause significant changes in the bridge column capacity and safety. One of the most detailed experimental studies has been performed by Rodriguez et al. [5], who examined the effects of steel corrosion on the RC column behaviour by considering the concrete section reduction, concrete cover cracking and spalling, and the corrosion of longitudinal and tie reinforcement. From their results, it was observed that the ultimate load was reduced due to the effect of corroded reinforcement on the concrete column performance, and the compressive stiffness of the column at service load was reduced as well. Also, the ultimate axial force value was estimated by considering the reduced steel section and decreasing the concrete section by assuming that one or more concrete covers had delaminated. Further, it was observed that reinforcement corrosion introduced a load eccentricity due to spalling of the concrete cover and led to buckling of the longitudinal reinforcement due to premature loss of one or more columns ties.

Tapan and Aboutaha [19] have presented a methodology to evaluate the load-carrying capacity of deteriorated concrete bridge pier columns by construction of the full P-M interaction diagrams. The presented methodology incorporated the actual material properties of deteriorated columns, and accounted for amount of corrosion and exposed corroded bar length, concrete loss, loss of concrete confinement and strength due to stirrups deterioration, bond failure, and type of stress in the corroded reinforcement.

Mohammed et al. [1&15] have introduced a NLFEA approach based on enhanced inspection to identify the state of damage from which the nonlinear behaviour of aged bridge columns is simulated. The approach is developed to take into account the corrosion of both longitudinal and lateral (stirrups/ties) reinforcement, concrete spalling,

the effect of stirrups loss on the core concrete confinement, reinforcement ductility, progressive change in load eccentricity, and the effects of local loss of bond on the structural behaviour at both the sectional and element levels. Mohammed et al. [1] have shown that the critical evaluation zones for shear and flexure could be different from the critical design zones based on the location of the corrosion affected zone and the state of damage. The NLFEA approach estimates, with high accuracy, the current sectional and element (finite elements) instantaneous characteristic properties based on the observed state of damage (from enhanced inspection) and the required material tests. Some tests that follow the enhanced inspection are to estimate the corrosion effect on the steel and the material properties if detailed testing program is very expensive. The NLFEA approach is integrated with the SHDFEA approach, as mentioned earlier, where it enables evaluating engineers to have integrated tools in their everyday bridge assessment work.

7. Validation of the proposed model

7.1. Case study: Moving truck on slab-on-steel girder bridge

The proposed SHDFEA is validated through a case study of a concrete slab-on-steel girder bridge (called Bridge A throughout this study) with centre to centre span of 61.31 m selected from the Virginia State Bridge Inventory (Hevener [20] and Nutt et al. [21]). The bridge is simply supported on RC piers formed from eight square 600×600 mm columns, connected from the top to a cap-beam where the bridge superstructure girders rest on hinge or roller joints (see Fig. 6 (a), roller side). The 6-m height columns are assumed to have full fixity at the foundation level, and the reinforcement is detailed as shown in Figs. 6 (b) & 6 (c). The bridge superstructure consists of a concrete slab compositely casted on steel girders, as shown in Table 1. The bridge was built in the

1970's, and the current concrete compressive strength f_c for all the bridge components, the superstructure and the columns, is assumed equal to 35 MPa. Details of the slab thickness, girder geometrical properties, girder spacing, bridge width and number of girders are given in Table 1.

The present case study involves studying the structural dynamic performance when the bridge is subjected to different static and dynamic loads using a two-axle truck of a total mass of 34,400 kg, where the body mass is 30,189 kg, the front axle mass is 2,806 kg, and the rear axle mass is 1,403 kg. The truck mass moment of inertia is 263,052 kg.m², the stiffness of each axle is 5,363,163 N/m and the axle spacing is 6.19 m [13]. Since the proposed model is a two-dimensional model and the variation through the bridge width perpendicular to the traffic direction is ignored, the reported results (stiffness, mass, and damping) are average values per traffic lane/per column. It should be noticed in this case study that the slab-on-steel girder has been selected for its low stiffness and mass per lane, so its stiffness to mass ratio is relatively high and the bridge should be then more sensitive to vibrations.

7.2. Convergence study

To verify the validity of the proposed SHDFEA approach when performing static and/or dynamic analysis, it is necessary to ensure the correct implementation of the internal hinge and roller at the beam-column joints. A static analysis of the slab-on-steel girders bridge under the two-axle truck (see Section 6.1 for details) is performed where the front axle location changes on the superstructure, representing the truck movement from one of the bridge sides to the other in static mode. The analysis is performed for two cases: (i) simply-supported beam (SSB); and (ii) beam on a two-column system (BOTC) with

internal hinge and roller. Depending on the column stiffness, the deformations of the bridge superstructure are expected to increase in the case of the beam on two columns (BOTC) frame compared to the case of the simply-supported beam. Figure 7 shows that modelling the bridge as BOTC gives slightly higher deflections than modelling the bridge superstructure.

Two convergence studies are conducted on the dynamic analysis part of the proposed SHDFEA. For both convergence studies, the truck speed is equal to 100 km/h. In the first convergence study, the number of elements (NE) is assumed to be fixed (NE = 60), and the number of integration points (NI) varies from 5 to 100. Figure 8 (a) shows the dynamic deflection under the front axle of the truck along the span of Bridge A for different NI. It is found that after NI = 24, the change in the deflection becomes negligible. In the second convergence study, the number of integration points is set constant (NI = 50) and the number of elements varies from 6 to 80. It is found that after NE = 30 the results becomes stable and the change in the dynamic deflection due to the increase in the number of elements is negligible (see Fig. 8 (b)).

For nonlinear modelling of the columns, fast and consistent convergence is achieved through optimizing different variables that lead to a stable solution at the section and element levels [1 & 15], such as: (i) load step; (ii) element size; and (iii) number of sections per element. By compromising the accuracy within an acceptable range and the computation time of the step-by-step nonlinear dynamic time history analysis, the convergence can be further accelerated by eliminating the secondary correction cycles of the "displacement field tuning convergence" used in the NLFEA [1], provided that equilibrium is satisfied at both sectional and structural levels.

In order to show the proposed SHDFEA capability, a severe state of reinforcement corrosion damage in the columns is assumed in this case study. Corrosion-induced damage is estimated assuming a steel mass loss of 30%, and assuming that the concrete cover of the columns has spalled off in addition to local loss of the lateral reinforcement. For the two-axle truck moving over the bridge at a speed of 100 km/h, Fig. 9 shows a marginal increase in the dynamic deflection of the superstructure for all studied damage scenarios reinforcement corrosion of both columns.

8. Results and Discussion

8.1. Static deflection versus dynamic deflection of the bridge superstructure

Figure 10 shows the dynamic deflection of the superstructure of Bridge A under the front axle for different truck speeds. The figure shows that the dynamic deflection increases as the truck speed increases. The maximum deflection zone is also shifted towards the roller side with increasing truck speeds. The dynamic load allowances (DLA) (or the impact factors in AASHTO LRFD) are calculated from the ratio of the maximum dynamic deflection to the maximum static deflection, which range between 1.013 to 1.53 as shown in Table 2.

8.2. Effect of using nonlinear versus linear analysis of columns on dynamic deformations of bridge components

For the bridge in this case study (Bridge A), Figs. 11 (a) and 11 (b) show the dynamic deflection of the bridge superstructure and maximum lateral displacement of the bridge columns when the given two-axle truck passed over the bridge superstructure with a speed of 100 km/h. It is observed that the axial deformations of the columns are very small for the load of the moving traffic in this study. Linear and nonlinear analysis

alternatives are performed using the proposed SHDFEA approach. It is clear that an insignificant difference in the dynamic deflections of the superstructure is observed when either linear or nonlinear analysis is used, Fig. 11 (a). However, Fig 11 (b) shows that the nonlinear option of SHDFEA gives higher lateral displacement of the left column (with internal hinge) than that with the linear analysis, while the nonlinear analysis gives slightly higher lateral displacement of the right column (with internal roller) than that with the linear analysis.

Figure 12 shows the load-displacement curve of one column of the bridge pier using the NLFEA, where the corrosion results in high reduction of the load capacity in the two common loading cases: (i) concentric loading ($e = 0$), and (ii) eccentric loading with eccentricity equal to 15% of the column cross-section depth. The figure shows that the maximum un-factored load applied on the column (which equals to 634 kN for Bridge A) is very low compared to the load capacity of the column even in the elastic range. Also, the figure shows that the axial load capacity of the column is reduced by around 30% when the load eccentricity is increased to 15% of the column cross-section depth (H). This shows that the bridge columns are overdesigned. When a severe corrosion state is considered (well below the collapse level), the bridge load capacity is reduced, and the reduction in strength is very large when the applied load approaches the ultimate load capacity of the columns (in both cases: with or without eccentricity). Since the applied traffic load is very low compared to the load capacity, even in the case of severe corrosion, the changes in the performance of major parameters show very low sensitivity to corrosion. At the service (traffic) loading level, the differences between the linear and nonlinear analysis are negligible in most of the structural performance parameters.

8.3. Effect of the bridge span length on dynamic deflection of bridge superstructure

As mentioned earlier, the focus of the current study is on the effect of traffic load combined with reinforcement corrosion applied on the bridge columns at highly exposed zones on the dynamic behaviour of the bridge. The proposed SHDFA is used to investigate three more bridges with spans shorter than the span of Bridge A. The major difference between the selected bridges (Bridges A to D in Table 1) is the unsupported span length; however, the girder spacing, skew angle and transverse width are similar. All the four bridges columns are designed with load over capacity of 50% (i.e. the applied ultimate load divided by the bridge load capacity is equal to 0.5). Corrosion-induced damage is assumed to present the worse scenario of the column deterioration (but not a full damage or close to collapse state), where stirrups and confinement are partially lost. For the investigated bridge spans of 15, 30, 36, and 60 m, Fig.13 shows that the dynamic deflection is different from one bridge to other depending on its length. It is observed that with a truck speed of 100 km/h (usual design speed for highway bridges), the maximum dynamic deflection under the front axle is located close to the bridge mid-span, similar to the case of static analysis, while the maximum dynamic deflections in the cases of longer span lengths (36 m and 60 m) are shifted towards the column with an internal roller. It is also clear that the maximum dynamic deflection is significantly increased with the increase of the span. The influence of having corrosion damage in the bridge columns has negligible effects on the dynamic deflection of the superstructure for all bridge spans. It should be noticed that the bridge with a span length of 30 m has prestressed concrete girders instead of steel girders; however, this has not a large impact on the trend of the dynamic deflection of the superstructure. This can be explained by the fact that the higher

mass and higher stiffness of the prestressed concrete girders over the steel girders together with the girders spacing have not shifted the natural frequency of the composite superstructure of Bridge C from the range of natural frequencies of the other three bridges in the group.

8.4. Nonlinear static versus nonlinear dynamic analysis of various bridge span lengths.

In this section, comparisons of nonlinear static analysis using the NLFEA [1] versus nonlinear dynamic analyses using the SHDFA for undamaged and damaged bridge columns of various span lengths are presented. The nonlinear static behaviour of the columns when loaded up to ultimate static load and their nonlinear dynamic behaviour when they are loaded dynamically by a moving truck (traffic load) are investigated. All the bridges columns are assumed to be exposed to the same states of damage as in Section 6.2. The purpose of this investigation is to explore whether safety critical damage states of the columns due to reinforcement corrosion can be captured by the change of any of the bridge dynamic parameters that is easy to be observed and measured. Hence, it would be possible to develop a sensing technique that can predict any critical safety and stability states of the columns from the characteristic changes of the bridge dynamic behaviour under traffic. Figures 14 (a) and 14 (b) show the load versus axial deformation relationship for the columns of the four bridges (Bridge A through Bridge D) for the case of concentric axial load and the case of 0.15H eccentricity of the axial load, respectively. When the load is concentric, both the load capacity and the ductility (measured as the ratio of the ultimate displacement divided by the yield displacement) are decreased when the column is subjected to local corrosion damage (for instance at the column mid-height). When the load is eccentric by 0.15H, the load capacity is decreased when the

columns are subjected to severe reinforcement corrosion; however, the ductility is increased in some cases (for instance, $L = 60$ m and $L = 15$ m). Figure 14 (c) shows the applied moment versus the lateral displacement relationship, in which it is observed that the moment capacity is reduced for the columns of all bridges when they are subjected to severe corrosion damage.

Figures 15 (a) and 15 (b) show the distribution of the static axial displacement and the static lateral displacement, respectively, over the column height for the four bridges, Bridges A through D. Both figures show the change of the location of the maximum displacement when the columns are subjected to severe reinforcement corrosion. It is important to mention that the bents (or kinks) of the axial displacement distribution over the damaged-column height in Fig. 15(a) are located at the end of the damaged zone (4m from the base). The reduction of axial stiffness and column boundary conditions and the location of the damaged zone contributed to this change in the axial displacement.

As mentioned earlier, the service (or traffic) load magnitude is well below the ultimate static capacity of the column, and hence even with severe local corrosion damage in the columns, their static behaviour remains in the elastic range or in the early plastic range. This may explain the low sensitivity of the dynamic parameters of the bridge structures to the corrosion damage of the columns, in spite of the fact that they could be in a state of critical safety and stability. Figures 16 (a) and 16 (b) show that for all studied bridges (Bridges A through D) and when both the right and left columns of each bridge are subjected to severe local corrosion damage, there are no observed changes in the columns dynamic lateral displacement related to the truck front axle location on the bridge superstructure. Different extreme loading and design alternatives have shown that the

sensitivity of the dynamic response of the bridge superstructure or substructure remains low [22].

9. Conclusions

A major part of the evaluation serviceability limit state of bridges is to ensure that the dynamic deflections of the bridge superstructure when the bridge is under service (or traffic) loads are within the allowable limits defined by North American Highway Bridge design codes (CHBDC and AASHTO LRFD). In this paper, a simplified hybrid linear/nonlinear dynamic finite element analysis (SHDFEA) is proposed to evaluate the dynamic characteristics and behaviour of slab-on-girder bridges under moving trucks when their columns are subjected to severe corrosion damage. The proposed SHDFEA uses NLFEA to evaluate the damaged columns stiffness, mass and damping throughout the step-by-step time history analysis.

It is found that the proposed SHDFEA is numerically efficient and stable in all the studied cases. Although the nonlinear analysis presents the appropriate selection to evaluate the possible effects of the corrosion damage of the columns on the dynamic behaviour of the bridge, linear dynamic analysis is found to be a more economical alternative when the columns are overdesigned. It is also found that when the traffic load magnitude is well below the ultimate static capacity of the columns, even with severe local corrosion damage in the columns, their static behaviour remains mostly in the elastic range. This leads to a low sensitivity of the dynamic parameters of the bridge structure to the corrosion damage of the columns, even though they could be in a state of critical safety and stability.

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Figure Captions

Fig. 1: Framework assessment of aging RC bridge columns.

Fig. 2: Evaluation of column performance under corrosion & ultimate loads.

Cont'd. Fig. 2: Evaluation of column performance under corrosion & ultimate loads.

Fig. 3-a: Typical slab-on-steel girder bridge columns.

Fig. 3-b: A truck moving on an aged slab-on-girders bridge over a highway. Possible damaged zones and boundary conditions are shown.

Fig. 3-c: Variation of load on each column related to the location of front axle of two-axle moving truck.

Fig. 3-d: Applied ultimate or service loads vs. deformation for new and aged bridge columns.

Fig. 4: Proposed simplified hybrid linear/nonlinear dynamic analysis of slab-on-girder bridge structures under traffic load.

Fig. 5: Vehicle model (after Lin and Trethewey [9]).

Fig. 6: Column design, reinforcement details and damage due to corrosion in critical corrosion zone.

Fig. 7: Mid-span deflection of BOTC compared to SSB under truck load for nonlinear model.

Fig. 8-a: Convergence study 1: number of elements NE = 60 and truck speed = 100 km/h for different number of integration points (NI).

Fig. 8-b: Convergence study 2: number of integration points NI = 50 and truck speed = 100 km/h for different number of elements (NE).

Fig. 9: Comparison of dynamic deflections of superstructure under traffic load only and combined traffic and corrosion loads.

Fig. 10: Dynamic deflection of BOTC under front axle for different truck speeds using the proposed SHDFA.

Fig.11-a: Comparison of dynamic deflection due to linear and nonlinear analysis of the columns.

Fig. 11-b: Comparison of maximum linear and nonlinear lateral deformation of the bridge columns.

Fig. 12: Applied axial load vs. axial displacement of the top section of the bridge column (right before static failure load).

Fig. 13: Comparison of dynamic deflections of superstructure under traffic load only and combined traffic and corrosion loads for different bridge lengths.

Fig. 14-a: Applied axial load vs. axial displacement of the top section of the bridge column (right before static failure load) with zero eccentricity; UC: undamaged column; DC: damaged column.

Fig. 14-b: Applied axial load vs. axial displacement of the top section of the bridge column (right before static failure load) with 0.15H eccentricity; UC: undamaged column; DC: damaged column.

Fig. 14-c: Applied moment vs. lateral displacement of the top section of the bridge column right before static failure load; UC: undamaged column; DC: damaged column.

Fig. 15-a: Column height vs. axial displacement of the bridge column at failure load; UC: undamaged column; DC: damaged column.

Fig. 15-b: Column height vs. lateral displacement of the bridge column at failure load; UC: undamaged column; DC: damaged column.

Fig. 16-a: Truck position vs. maximum lateral displacement of the right column for different bridge lengths; URC: Undamaged right column; DRC: Damaged right column.

Fig. 16-b: Truck position vs. maximum lateral displacement of the left column for different bridge lengths; ULC: Undamaged left column; DLC: Damaged left column.

Table Captions

Table 1: Properties of selected bridges (Hevener [20], based on Nutt et al., NCHRP Report 12-26 [21]).

Table 2: Impact factors under two-axle moving truck.

Name	Span (m)	Type of girder	Year Built	Skew Angle (°)	Girder Spac- ing (m)	Girder Properties			Trans- verse width (m)	Slab Thick- ness (m)	No. of Girders
						A (m ²)	I (m ⁴)	H (m)			
A	61.3	Steel	1977	0.0	2.64	.0462	.01739	2.898	20.49	.2159	8
B	36.9	Steel	1978	0.0	2.46	0.0071	0.0017	1.119	19.59	0.229	8
C	30.8	Prestressed	1981	7.0	2.11	0.359	0.0782	1.601	22.57	0.158	7
D	15.8	Steel	Unknown	6.4	1.69	0.0323	0.0044	0.915	21.35	0.191	8

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Velocity (km/h)	Impact Factor Nonlinear
20	1.013
40	1.011
60	1.38
80	1.53
100	1.51

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