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Research Article

Simplified Seismic Evaluation of Aged Corrosion Damaged Reinforced Concrete Bridge Columns as Part of Simplified Semi-Quantitative Assessment Framework

Amina Mohammed[1*], Husham Almansour[2], Beatriz Martín-Pérez[1]

[1]Department of Civil Engineering/Faculty of Engineering, University of Ottawa, Ottawa, Canada
[2]Construction Research Centre/National Research Council Canada, Ottawa, Canada
Email: amohamme@uottawa.ca

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Abstract: Severe reinforcement corrosion significantly reduces the structural stiffness and load-carrying capacity of Reinforced Concrete (RC) columns. The interactive effects of corrosion-induced damage and repeated traffic load cycles further accelerate bridge columns' load-carrying capacity deterioration. When subjected to seismic excitation, corrosion-affected RC columns could show a dynamic response significantly different from non-affected columns. This paper proposes a Simplified Nonlinear finite element Seismic Analysis approach (SNLSA) based on enhanced inspection of corrosion-damaged RC columns and as a handy tool for evaluating their seismic response, which is a crucial step in a semi-quantitative assessment framework. The SNLSA integrates Nonlinear Sectional Analysis (NLSA), DRAIN-RC computer program for nonlinear time history analysis, and Takeda's hysteretic analysis. The approach provides three options: (i) establish the staged failure mechanism using express analysis simulating quasi-static loading up to failure; (ii) use a more comprehensive analysis simulating cyclic loading developing the hysteretic relationships; and (iii) conduct a nonlinear full time-history analysis. The SNLSA can estimate the significant contraction of the column interaction capacity when subjected to severe corrosion damage for all load-over-capacity ratios. The SNLSA quantitatively predicts the change in the seismic performance of corrosion-affected versus as-built bridge columns. The approach could also be used to select the appropriate design option for bridge columns in seismic-critical zones.

***Keywords*:** simplified nonlinear seismic analysis, aged reinforced concrete bridge columns, reinforcement corrosion, semi-quantitative assessment framework

1. Introduction

Chloride-induced reinforcement corrosion generated mainly from the application of de-icing salts in winter is the primary cause of damage in bridges located in cold regions. Reinforced Concrete (RC) bridge columns represent the most critical elements for bridge safety and stability. Hence, developing a cost-effective and simplified evaluation approach that enables a fast and accurate assessment of the structural capacity of corrosion-damaged RC bridge columns in seismic-critical areas presents a significant challenge [1]-[2].

Severe reinforcement corrosion significantly reduces the structural elements' stiffness and, hence, their load-

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carrying capacity. The interactive effects of corrosion-induced damage and repeated traffic load cycles could further accelerate critical structural elements' deterioration [3]. When subjected to seismic excitation, corrosion-affected RC columns could show a dynamic response significantly different from those of non-affected columns depending on the corrosion damage level. For instance, the locations of critical sections for structural evaluation could shift from the typical seismic-design locations to corrosion-damaged zones. Localized reductions in stiffness and strength of corrosion-damaged zones could result in a severe decrease in the structural elements' overall seismic capacity. The collapse mechanism of these elements could change from ductile failure to a more brittle one [3].

Estimating the structural capacities and the dynamic performance of bridges' critical elements when subjected to seismic loads is a major component of the assessment of aging bridges in seismic-critical regions. Many advanced software packages like OpenSees, DIANA, and IDARC, among others, can perform two- or three-dimensional seismic analysis. Advanced Finite Element Modeling (FEM) of existing bridges with different levels of deterioration/damage using commercial software is very challenging. The advanced modeling approaches require complex material and deterioration models, convergence and refinement studies, and advanced verification, validation, and calibration. On the other hand, using simplified analysis approaches based on field non-destructive tests and with a reasonable margin of error would accelerate the evaluation process, specifically when the infrastructure owner has many deficient bridges. The development of simplified FEM models with high accuracy and a friendly interface would enable a better cost-effective quantitative evaluation of the bridge. For everyday use by practicing bridge evaluation engineers, a simplified approach is highly required to give a reliable evaluation in a short time and with acceptable accuracy [1]-[2].

Given the subjectivity of existing qualitative assessment approaches currently used in most jurisdictions in North American states and provinces, there is a need to develop an accurate quantitative assessment approach. Such an approach would quantitatively evaluate bridges' structural performance when safety-critical elements (bridge columns, for instance) are partially damaged. Mohammed et al. [4] proposed a semi-quantitative assessment approach, introducing a Limit States Evaluation method (LSE). The LSE is analogous to the Limit States Design (LSD) method in North American bridge design codes [5]-[6]. The primary evaluation limit states are (i) Evaluation Ultimate Limit State (E-ULS), which includes the evaluation of earthquake ultimate capacity, and (ii) Evaluation Serviceability Limit State (E-SLS).

In order to estimate the instantaneous seismic capacity of a bridge beam column deteriorated by reinforcement corrosion, it is essential to identify the changes in the geometrical and material properties of the corrosion-damaged zones for different critical damage levels. Seismic analysis of corroded RC structures, particularly bridge piers, is very complex, and many uncertainties limit the use of numerical methods [7]. Kashani et al. [8] considered the impact of different corrosion damage models on the nonlinear flexural response of corroded RC columns. The authors provided modeling guidelines to model the nonlinear behavior of corroded RC bridge piers, including the effects of cyclic degradation up to complete collapse, and attempted to simulate and capture multiple failure modes of corroded RC columns. Their results highlighted what damage mechanisms are important to be considered in modeling the nonlinear behavior of corroded RC columns. Carlo et al. [9] also considered the behavior of corroded columns under cyclic loads using three-dimensional finite element analysis and taking into account steel and bond decay as well as potential longitudinal reinforcement buckling. The models developed by Kashani et al. [8] and Carlo et al. [9] can be used alone to only predict the seismic response of corroded circular RC bridge piers; however, (i) they cannot be used to evaluate all the ultimate and serviceability limit states of the corroded bridge elements; (ii) they have to be implemented using OpenSees or any other software, which might be challenging to use in the daily evaluation in a bridge asset management office; (iii) convergence and refinement studies, verifications, validation, and calibrations are not speedy tasks to be conducted for a large number of bridges.

Mohammed and Almansour [10] show that it is possible to simulate the staged failure mechanism developed in RC concrete columns subjected to quasi-static or cyclic loads up to failure. The collapse is further accelerated, and the structural capacities deteriorate more when severe reinforcement corrosion damage states occur. The staged failure mechanism mostly takes similar patterns to those staged failure patterns of non-corrosion-damaged elements loaded to failure. However, observations show significant changes in the strength, ductility, and hysteretic behavior of corrosion-damaged elements. Different types of reinforcement corrosion-induced damage in bridge columns, such as spalling of concrete cover, reduction in reinforcement section, fracture of one or more stirrups, etc., are identified as critical damage levels. As a transition stage preceding the development of a time-dependent quantitative assessment approach, the enhanced visual inspection, including precise measurements and some material tests on the corroded reinforcement, can be employed as a major source of input to the proposed semi-quantitative assessment framework.

Most bridge columns are conservatively designed (or over-designed), and their Service-Load-Over-Capacity Ratio (SLOCR) ranges between 25% and 50%. The SLOCR is increased in aging bridge columns due to the increase in the truckloads, the number of axles, traffic density, loading frequency, traffic speed, and the reduction of the corrosion-damaged column ultimate capacity. Higher design SLOCRs are targeted for newly constructed bridges mainly because (i) they could reduce the initial cost, and (ii) smaller column sections would be required for aesthetical needs.

Two possible scenarios exist for the interactive effects of reinforcement corrosion and seismic loads on RC columns: (i) an earthquake load applied where reinforcement corrosion is in progress (possibly in an advanced stage), or (ii) a bridge column survived a medium size earthquake with some damage in the form of cracks (perhaps initiating corrosion or further accelerating existing corrosion). Whereas in the first case, it is essential to evaluate the bridge column's safety and structural capacity as presented in this paper, in the second case, the post-disaster inspection and maintenance process has to occur.

While some studies have experimentally shown that reinforcing steel corrosion reduces the static capacity of RC structural elements (e.g. Rodríguez et al. [11] and Biswas et al. [12]), limited efforts have been directed to experimentally study the structural behavior of RC columns affected by reinforcement corrosion when subjected to seismic load (e.g. Kashani et al. [8], Meda et al. [13], Ge et al. [14], and Biswas et al. [15]).

Among the studies that considered the seismic performance of existing RC structures affected by corrosion are Oyado et al. [3] and Saito et al. [3], [16], who conducted several tests to evaluate RC columns' strength and deformation capacities with reinforcement corrosion subjected to static and cyclic loads. In their study, two sets of columns were tested: (i) columns subjected to only axial compression; and (ii) columns subjected to axial compression and lateral cyclic load. They observed that longitudinal bars and hoops corrosion reduces the column deformation capacity. The authors proposed a stress-strain model and corresponding deformation capacity by considering only the reduction of the rebar cross-section. The study did not consider the effects of corrosion on the steel ductility, core concrete compressive strength when one or more stirrups are damaged, and the reduction in stiffness due to concrete cover spalling. Choe et al. [17] developed probabilistic drift and shear force capacity models by integrating the effects of the deterioration of structural elements affected by reinforcement corrosion into a structural capacity estimation model. In their study, a numerical analysis was performed using the OpenSees software to obtain the fragility estimate of a given column, which was modeled by fiber-discretized cross-sections. The material of each fiber was modeled as a uniaxial inelastic material. Probabilistic modeling for reinforcement corrosion (corrosion initiation, corrosion rate, and loss of cross-section area) was also carried out to determine existing and new structures' service life. Berto et al. [18] conducted an analytical investigation to assess the corrosion effects on RC structures' seismic behavior. They considered two major parameters: the rebar section reduction and the concrete cover loss. Using MIDAS Gen software, nonlinear static analysis was performed considering gravitational and seismic loads. Choe et al. [17] and Berto et al. [18] studies considered only the reduction of reinforcement cross-sectional area, while they did not consider the impact of stirrups damage on the concrete confinement and hence the concrete strength. Xu et al. [19] presented a two-dimensional nonlinear FE model to simulate the behavior of corroded reinforced concrete columns. The model considered the shear capacity deterioration due to the corrosion suffered by the transverse reinforcement and the flexure-shear interaction behavior in a coupled manner. The OpenSees software was used for the simulation.

Mohammed et al. [20] proposed a nonlinear elastoplastic finite element analysis approach to simulate bridge columns under the combined effects of reinforcement corrosion and seismic excitation. The approach addressed the diversity in time between the two processes, the reinforcement corrosion's progress, and the sudden or "flash" attack of earthquake load. The instantaneous seismic load-carrying capacity of an aging column subjected to reinforcement corrosion as a time-dependent process was evaluated. Reinforcement corrosion was simulated in an external time-dependent cycle, while the seismic load was simulated in an internal cycle of elastoplastic time-history analysis. The seismic time-history analysis was activated at each primary corrosion damage level that was identified assuming a constant corrosion rate. The approach was based on assuming ideal conditions that resulted in a constant corrosion rate and did not include the effect of inspection and possible maintenance/rehabilitation interventions on the bridge columns.

This paper aims to present a Simplified Nonlinear finite element Seismic Analysis (SNLSA) of corrosion-affected RC columns developed as a tool for the evaluation of the seismic response. The SNLSA is part of a Semi-Quantitative

Assessment Framework (SQAF) proposed by Mohammed et al. [4] to evaluate the serviceability and ultimate limit states of RC bridge piers affected by reinforcement corrosion. The analysis approach involves an enhanced inspection and material testing as a major input source to evaluate the column's instantaneous structural performance. The proposed SNLSA presents a practical, simplified, and cost-effective evaluation approach of the residual seismic capacity and the seismic behavior of slab-on-girder bridge columns subjected to local damage due to reinforcement corrosion. The emphasis is on: (i) ensuring the numerical efficiency and stability of the SNLSA; (ii) simulating the staged failure mechanism effects on the hysteretic diagram envelope; and (iii) demonstrating the approach's capability to capture the characteristics of changes in the time history behavior.

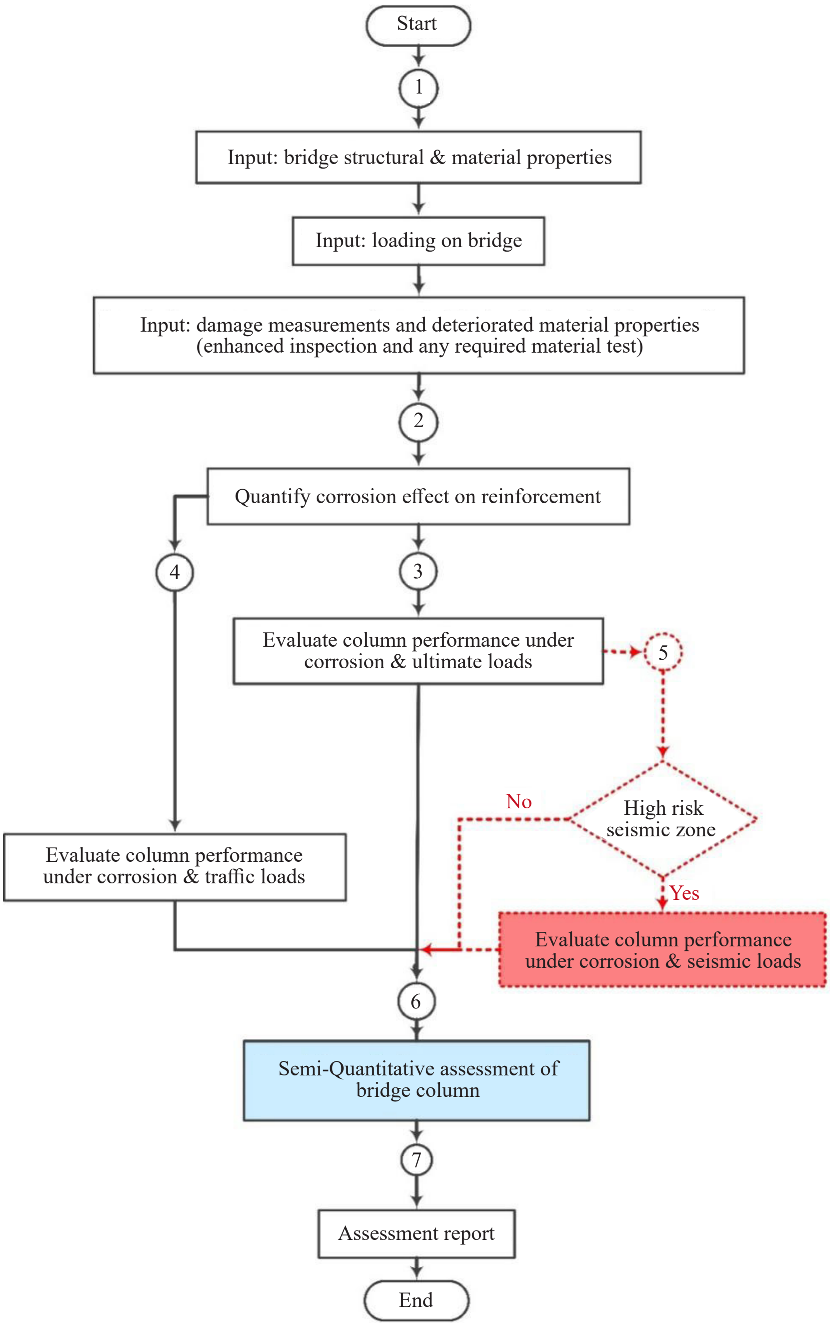

Figure 1. The proposed SQAF of aging RC bridge columns

2. Semi-Quantitative Assessment Framework (SQAF)

The SQAF previously proposed by the authors [4] (see Figure 1) has six components: (1) input data; (2) quantification of reinforcement corrosion and its effects on the damage zone and materials' properties; (3) evaluation-ULS: evaluation of columns' performance under combined corrosion and ultimate loads; (4) evaluation-SLS: evaluation of columns' performance under combined corrosion and traffic loads; (5) evaluation E-ULS-EQ evaluation of columns' performance under corrosion and ultimate seismic loads as part of evaluation-ULS only in high-risk seismic zones [4]; and (6) semi-quantitative assessment and reporting. Nonlinear Sectional Analysis (NLSA) is the basis for evaluating the columns' performance under corrosion and ultimate seismic loads as part of evaluation-ULS only in high-risk seismic zones [4]. 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 traffic loads are presented in Mohammed et al. [21], [22].

The first component of the proposed SQAF includes three data-input tasks: (I-a) the structural material and geometrical data, including boundary conditions; (I-b) the loading data; and (I-c) 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 from 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 is to be determined in step I-b.

In task I-c, the enhanced in-depth visual inspection (or enhanced inspection) is used in the proposed SQAF to provide detailed field measurements of the damaged concrete zone's dimensions and the volume of concrete spalling along the corrosion-affected zone [23]. This study employs a hybrid approach to find the steel rebars' properties. Out of all the columns in a bridge pier under assessment, the most damaged one is to be selected. The damage status, size, and material properties can be generalized over the other columns of the pier for a conservative structural capacity assessment. The most damaged rebars are then identified, and samples of suitable size and number are collected for testing. Accurate measurement of the maximum reduction of rebar diameter is then determined for each sample. If a constant corrosion rate is assumed, the time elapsed to generate the measured diameter reduction in the reinforcing bars is then found using the data of different samples and Faraday's law [22]. Using the evaluated reinforcement corrosion parameters and empirical formulae (for example, see Lay and Schiebl [24] and Cairns et al. [25]), the loss of strength and ductility of the reinforcement as a result of pitting corrosion 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 a suitable length. The deterioration of the structural ultimate and earthquake capacities is then quantified using the nonlinear finite element analysis and the simplified nonlinear seismic analysis, SNLSA, presented in this paper.

The SQAF proposed by Mohammed et al. [4] identifies four major damage cases due to reinforcement corrosion: (a) flexural cracking of concrete due to corrosion; (b) spalling of the concrete cover; (c) rupture of one or more stirrups; and (d) in a more advanced state of corrosion damage, structural failure of the column through complete loss of confinement and/or rebar(s) buckling. It is essential to mention here that the focus is on the last three damage cases, where damage case (a) can be considered as part of damage case (b), in which the impact on the structural behavior is more apparent. The Evaluation Limit States (ELS) are integrated with the proposed SQAF (see Figure 1), wherein quantifying the effects of reinforcement corrosion for each main case is taken into account. The flowchart shows the link to one of the two major evaluation limit states; E-ULS (including E-ULS-EQ for seismic-critical zones) and E-SLS. The focus of this paper is on presenting the E-ULS-EQ for seismic critical zones. The simplified nonlinear seismic analysis, SNLSA, is the basis of the ULS-EQ four tasks (see Figure 2), which are: (a) establishing the nonlinear load-displacement relationship for critical sections and different states of corrosion damage; (b) establishing the load-displacement hysteretic relationship based on states of corrosion damage; (c) establishing the time-history relationships of lateral displacement of critical sections; and, (d) establishing the interaction diagram for different states of corrosion damage. These four tasks of ULS-EQ end with preparing the required data for the final step of the SQAF [4].

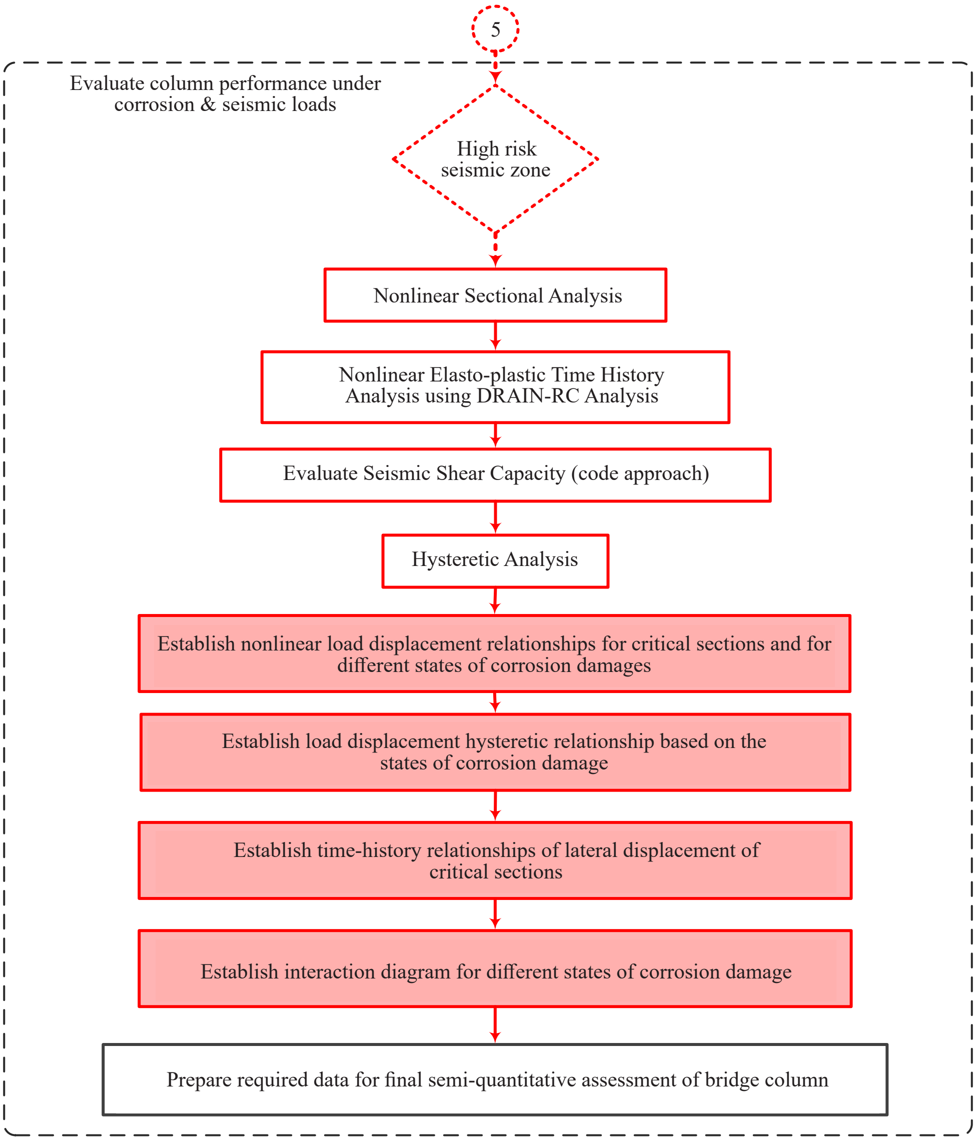

Figure 2. Evaluation of column performance under corrosion & seismic loads

3. Proposed Simplified Non-Linear Seismic Analysis (SNLSA) for corrosion damaged beam-columns

For bridges located in seismic regions, the main concerns are the level of damage before and after a significant earthquake, the reduction of the corrosion-affected RC column's residual capacity in terms of strength and ductility, and the collapse mechanisms compared to columns not affected by reinforcement corrosion. The seismic response of corrosion-damaged bridge columns depends on their original design, the level of corrosion damage/deterioration, and the characteristics of the earthquake load. This study focuses on the effects of local corrosion damage in RC columns of slab-on-girder bridges on their seismic response.

Figure 3 shows a highway overpass where a typical multi-column RC pier with an RC cap (substructure) beam supports a three-lane slab-on-steel-girder (superstructure). Figure 3 also shows the traffic directions over and under the bridge with a typical road configuration. The figure also shows the possible seismic excitation applied in the direction of the traffic over the bridge. The salty water splashes from moving traffic affect the bridge columns above the traffic level, where corrosion damage usually develops at approximately the middle third of the column height [26]. Figure

4 illustrates the most likely location of the column corrosion-affected zone relative to the seismic excitation, which corresponds to the middle third of the column height.

The proposed SNLSA is based on four main steps: (i) the Semi-Quantitative Assessment Framework (SQAF) [4]; (ii) the simplified Nonlinear Sectional Analysis (NLSA) [22]; (iii) the dynamic inelastic analysis of plane RC structures (DRAIN-RC) [27]-[28]; and, (iv) Takeda hysteretic model [29]. Corrosion-induced damage on the steel reinforcement is considered by decreasing the cross-sectional area and ductility as a function of corrosion, whereas damage in the concrete is incorporated by removing the concrete cover (spalling) and/or removing one or more ties, with the ensuing decrease in concrete confinement. These damage states in the critical zone are determined from the first component of the SQAF, as explained above. Once the damage state in the steel and concrete are determined, the nonlinear behavior of the corrosion-damaged section is evaluated using NLSA, which performs sectional analysis of aged beam-column elements subjected to service or ultimate loads combined with reinforcement corrosion. The model is a nonlinear iterative technique that uses numerical integration of the sectional stresses and satisfies force equilibrium in every load increment step. The NLSA model incorporates corrosion-induced damage by reducing the steel cross-section and ductility, removing the concrete cover (simulating spalling), accounting for the loss of local bond, removing lateral ties/stirrups (simulating tie fracture due to corrosion), and reducing concrete confinement as a result of tie loss in corrosion-damaged zones. The model can simulate the mechanical behavior of corrosion-damaged RC beam columns at the section level for all loading situations, including pure flexure, pure axial load, or any combination of axial and flexural stresses [22].

The time-history analysis of the RC columns based on the sectional analysis is then performed using DRAIN-RC. The column's hysteretic response at the critical section of the damaged zone is then established using Takeda's model.

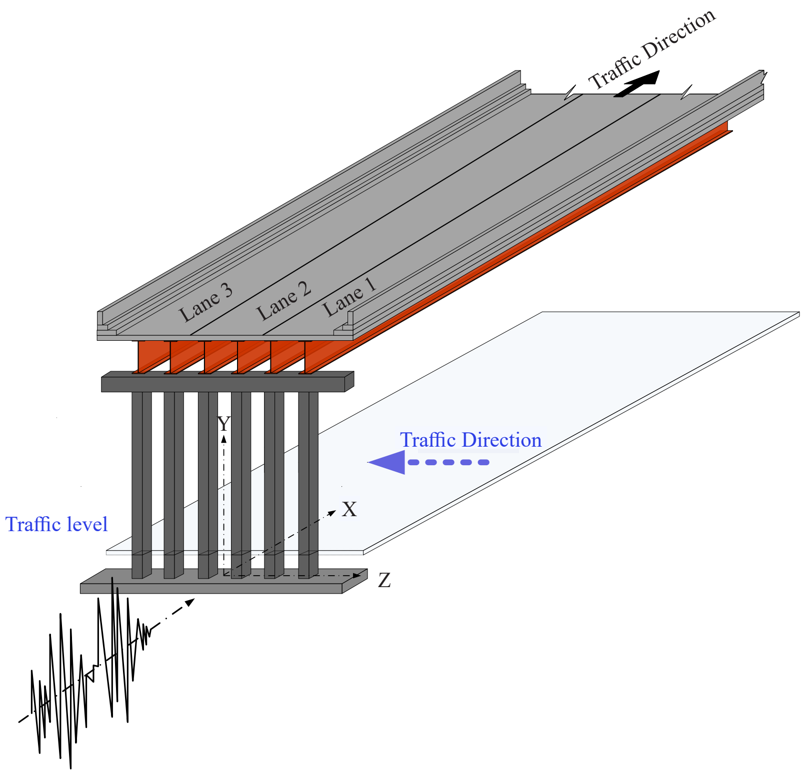

Figure 3. Slab-on-girder bridge under seismic load also shows traffic configuration

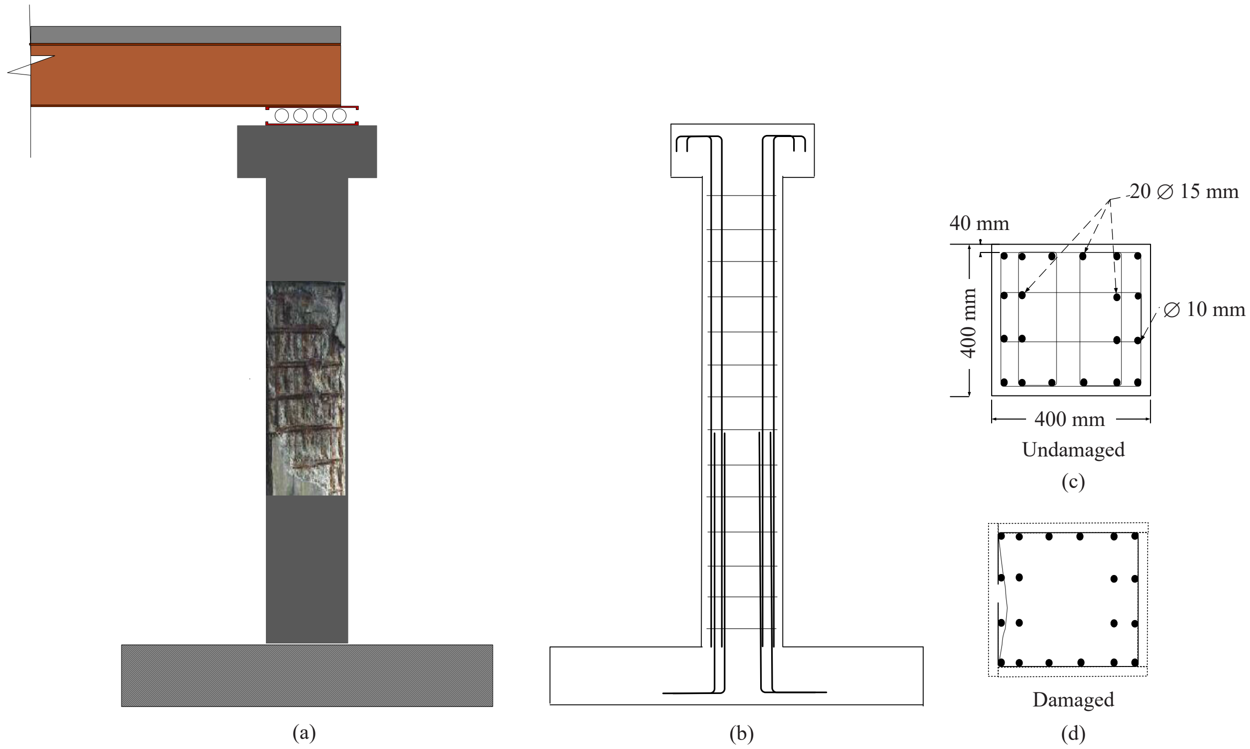

Figure 4. Most possible critical corrosion-damaged zone of slab on girder bridge column subjected to corrosion and seismic load (a), column design (b), and reinforcement details and damage (c & d) due to corrosion in critical corrosion zone

3.1 *Nonlinear modeling and time history analysis*

The computer program developed for SNLSA (using Visual FORTRAN) integrates three programs: Nonlinear Sectional Analysis (NLSA), inelastic dynamic time-history analysis (DRAIN-RC), and Takeda's model. The main program also includes a preliminary part of the SQAF that involves: (i) data preparation; (ii) enhanced inspection and any required tests; and (iii) the quantification of the corrosion effect on the steel and concrete sections. DRAIN-RC is a computer program (written using an older version of FORTRAN, which allows consistent integration with NLSA, and the main program) used to perform inelastic dynamic time-history analysis, as shown in Figure 2. Kanaan and Powell [30] developed the original version of DRAIN-RC (named DRAIN-2D) to conduct a dynamic analysis of inelastic two-dimensional structures. The program was further modified by Alsiwat [27] and Shoostari [28] by adding new features such as inelastic static analysis (push-over) and P-Δ effects. In this program, the structure is discretized into a number of elements with three degrees of freedom at each node: two translations and one rotation. The global stiffness of the structure is assembled from individual elements' stiffnesses using the direct stiffness method, where the structure's mass is assumed lumped at the joints resulting in a diagonal mass matrix. Damping can be specified as either mass-dependent or stiffness-dependent Raleigh damping. Ground motion is entered as accelerations specified at constant time intervals and can differ for horizontal and vertical directions.

The dynamic response is determined using the step-by-step integration technique, assuming constant average acceleration during each time step (t). In assembling the stiffness matrix, the tangent stiffness at the start of an integration step is assumed to remain constant throughout that step. The stiffness of each element is updated at the beginning of each step, and if the nonlinear analysis option is selected, errors due to possible overshooting are corrected by applying a corrective load at the beginning of the following step [30]. More details on how to perform the integration between the different programs are shown in Section 4.

3.2 *Modeling the effects of reinforcement corrosion on the steel rebars and the concrete*

As mentioned earlier, the proposed analysis approach employs enhanced inspection and any required material testing as the primary source of the input data characterizing corrosion damage and material properties. The focus is on quantifying the effects of corrosion on the material properties, the damage size, and the RC element's integrity in the damaged zone at the inspection/assessment time. In order to prepare the data for the analysis approach, a uniform rate of corrosion is assumed, and the observed level of damage at the time of the inspection can be matched. Hence, the instantaneous material properties can be estimated.

In the Nonlinear Sectional Analysis (NLSA), modeling the effects of corrosion on the reinforcement, the concrete section, and the concrete-and-steel reinforcement composite action is presented in detail by Mohammed et al. [22]. The three major effects of corrosion on the state of damage are (i) the reduction of cross-sectional area and ductility of the reinforcement; (ii) loss of concrete cover; and (iii) the local bond loss of tensile steel reinforcement. In addition, an advanced state of damage simulates tie fracture due to pitting corrosion by removing the ties and reducing the concrete core's confined strength.

3.3 *Load combinations*

The load combinations for the nonlinear seismic analysis in this study are considered according to the extreme possibilities of the traffic and seismic load combinations. The three considered scenarios of the load combination on an aging bridge during an earthquake are: (i) the earthquake occurs when there is no traffic on the bridge; (ii) the earthquake occurs during rush hour; and (iii) the earthquake occurs when one truck is moving on the bridge. The traffic load is considered a static load in all three load combination scenarios.

4. Integration and data flow between the SNLSA programs

In the proposed SNLSA, as illustrated in Figure 1 and Figure 2, the main program enables the data preparation and transfer from any analysis step to the following one. Each of the three "supporting" programs (NLSA, DRAIN-RC, and Takeda's model) runs inside the main program reading the required input data from an instantaneous data file issued in the preceding step. This technique avoids the effects of having huge files in the RAM during the analysis. After completing the running of each supporting program, the output of the step is saved in a separate file, and so on. The main program prepares the data input for the NLSA and controls the supporting programs' running. Then it extracts each program's results and re-circulates them if required in the seismic analysis. The main program accumulates the results needed for the final semi-quantitative assessment of the bridge column. The NLSA involves iterative cycles to find the most accurate location of the inelastic centroid and hence the instantaneous neutral axis location [22]. The approach is numerically efficient with steady convergence in all the studied cases [22]. Thanks to the high processing speed of recent computers, the high computation effort in terms of the number of arithmetic tasks required to perform the proposed NLSA results in short computing times. However, the high computational stability and simplicity enable effective integration of the procedures of the main program and supporting programs. This simplified analytical approach (with a reasonable margin of error and high accuracy) would enable a better cost-effective quantitative evaluation of the seismic performance of affected bridge columns.

As mentioned earlier, using available commercial software to perform such a seismic analysis of existing bridges with different levels of deterioration/damage is very challenging, given the need to integrate the analysis results with other evaluation steps. It requires complex material models, convergence and refinement studies, and advanced verification and calibrations. For example, using DIANA FEA for the analysis of bridge columns under concentric or eccentric loads requires significant mesh convergence time, massive run time, huge disk space, and other modeling efforts [31]. In this study, the concrete in the column is modeled using two types of isoparametric solid brick elements based on quadratic interpolation and Gaussian integration (a twenty-node "CHX60" and a fifteen-node "CTP45") with a meshing based on 50 × 50 × 50 mm elements volume.

5. Case studies, results, and discussions

5.1 *Staged failure mechanism under cyclic loads*

An RC bridge column is usually subjected to gravity loads when an earthquake event occurs. The loads applied on the bridge columns are a combination of the bridge superstructure, the column self-weight, and the traffic loads. For simply-supported slab-on-girder bridges, the loads of the superstructure are applied eccentrically. When the column suffers from corrosion damage, the load eccentricity could further be increased in the damaged zones, resulting in additional local and global flexural stresses. When an earthquake occurs, a large lateral seismic load is applied to the aging column. Figure 5 shows several states of localized corrosion damage and the internal forces acting on the damaged segments. The damage states are (a) flexural and corrosion cracks; (b) initial (or partial) spalling; (c) one stirrup failure (with partial spalling); (d) all-sides spalling; (e) two stirrups failure (with all-sides spalling); (f) loss of confinement and possible rebars buckling.

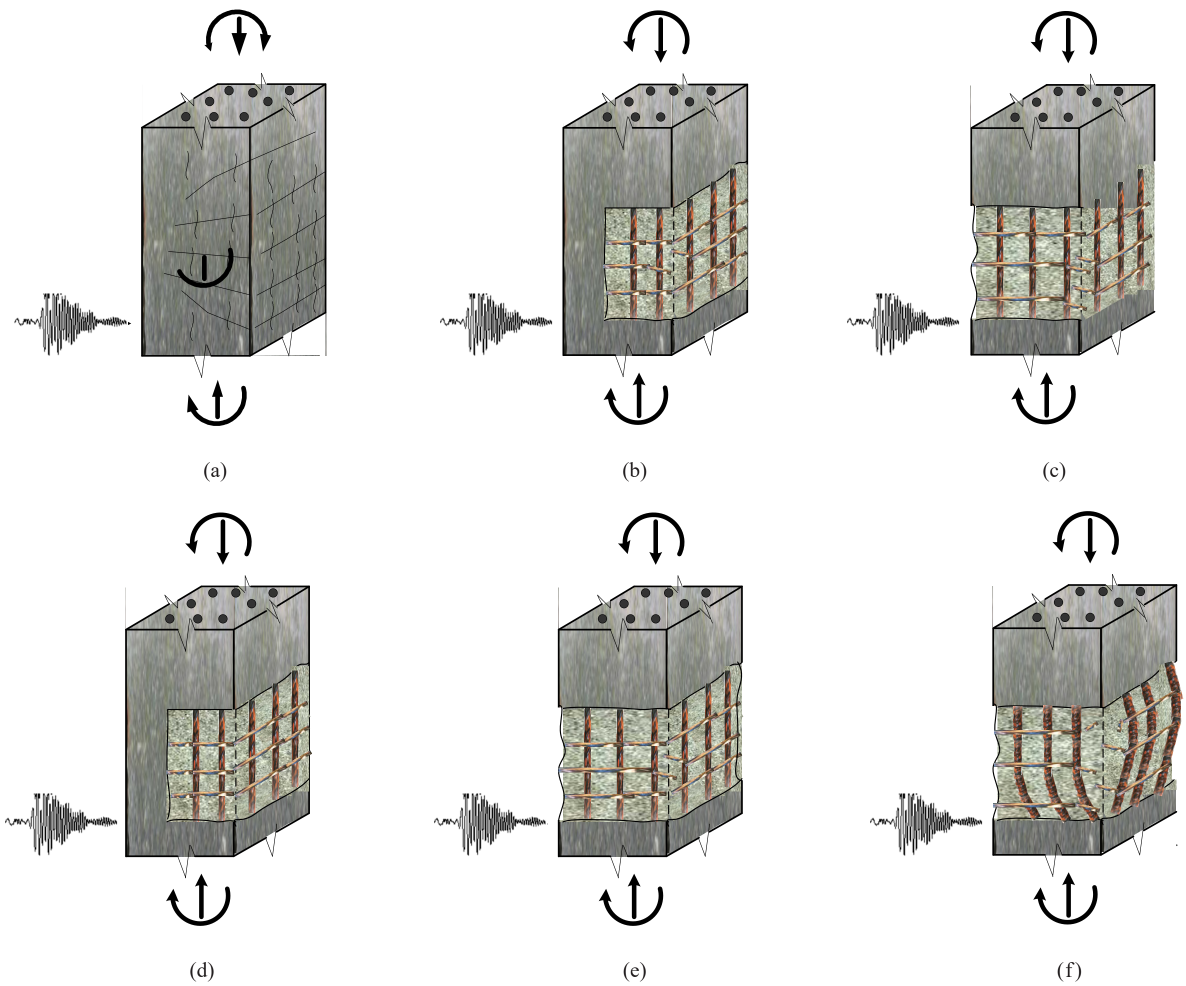

Figure 5. Possible damage and failure modes of RC columns due to combined gravity loads, seismic loads, and reinforcement corrosion; (a) flexural and corrosion cracks; (b) initial spalling; (c) one stirrup failure; (d) spalling on all sides; (e) two stirrup failure; (f) loss of confinement and possible buckling

5.2 *Model verification*

The SNLSA results are compared with the results of tests performed on RC columns by Oyado et al. [3]. Figure 6 shows the column specimen design and reinforcement details. The column is reinforced with 16.0-mm diameter longitudinal rebars, with a reinforcement ratio of $\rho_{st} = A_s/A_g = 2.5\%$, and with 10-mm diameter hoop reinforcement, with a reinforcement ratio of $\rho_{sh} = 2.5\%$, where ρ_{st} A_g is the total longitudinal reinforcement, and A_g is the gross area of the concrete column. The column was subjected to accelerated corrosion on two opposite faces over a length of 200 mm (starting at 350 mm and ending at 550 mm from the foundation). The comparison between the proposed model results and the test results focuses on the control specimen (no corrosion, specimen 1 of Oyado et al. [3]) and one of the corroded specimens (specimen 4).

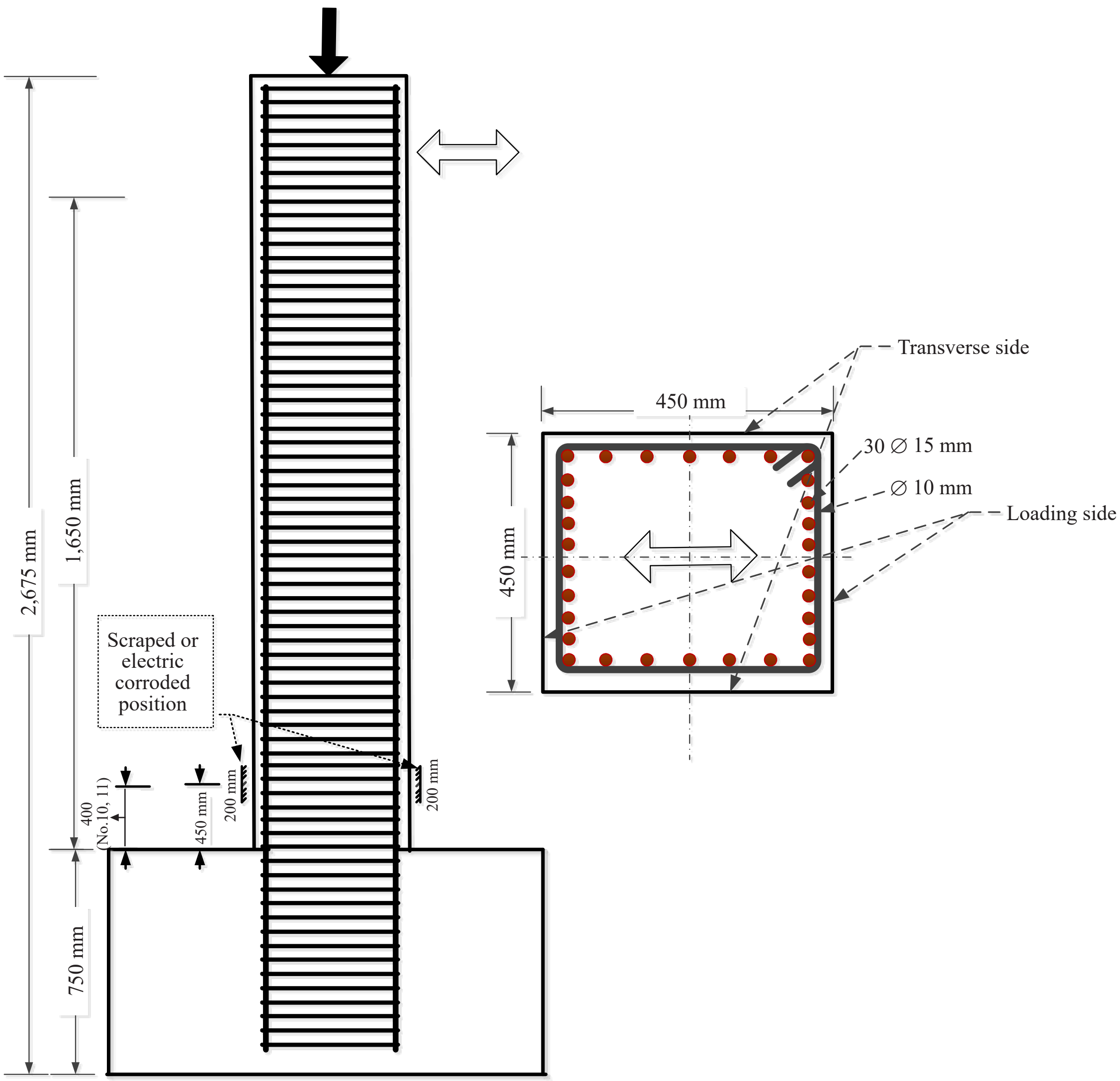

Figure 6. Case study verification: specimen details (reproduced from Oyado et al. [3])

Figure 7 shows the results of the NLSA for the main damage states of the non-damaged column loaded quasi-statically up to failure. The damage states during the analysis are identified as concrete cover spalling, one stirrup failure, two stirrup failure, and loss of confinement. The path of the staged failure is established (dotted line) with the guidance of the test results (red line). The NLSA results show a slight underestimation of the strength and stiffness. Figure 8 shows the envelopes of the experimental load-displacement hysteretic relationship and the load-displacement hysteretic relationship of Oyado's corroded specimen No.4 [3]), compared to the load-displacement relationships generated by the SNLSA. Different states of corrosion-induced damage were generated directly by the NLSA as part of

SNLSA. A good match is observed between the NLSA results and the staged failure path developed as an envelope by the SNLSA load-displacement hysteretic relationship. Hence, the NLSA could be used directly to establish the envelope of hysteretic relationships saving great efforts with an acceptable approximation. Comparing the envelopes of the load-displacement hysteretic relationship for column specimens No.1 and No.4 with the test results by Oyado et al. [3] shows that the different stages of corrosion-induced damage result in a large reduction of the column load and displacement capacities.

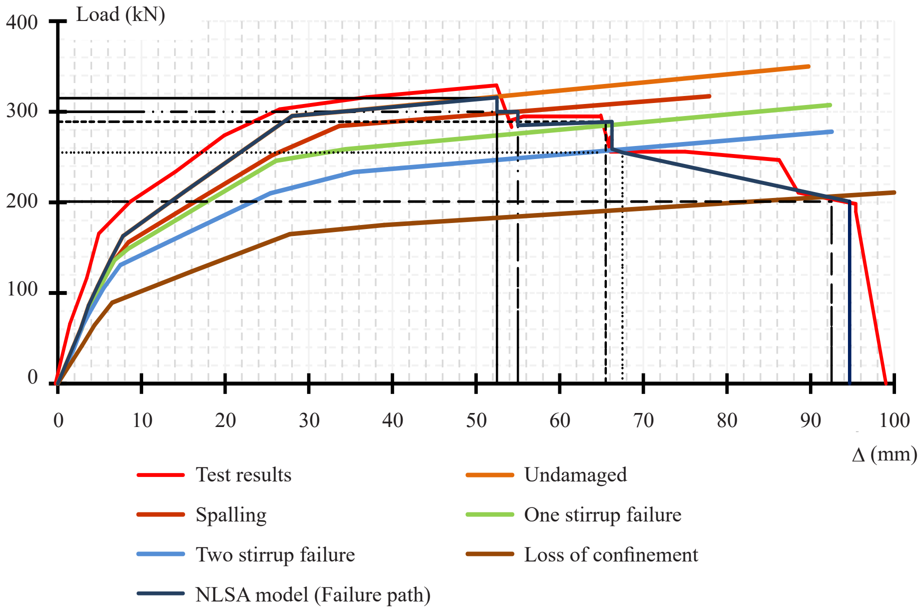

Figure 7. Envelope of load-displacement relationships for different damage levels using NLSA proposed in Mohammed et al. [18] versus test results by Oyado et al. [3], specimen No.1

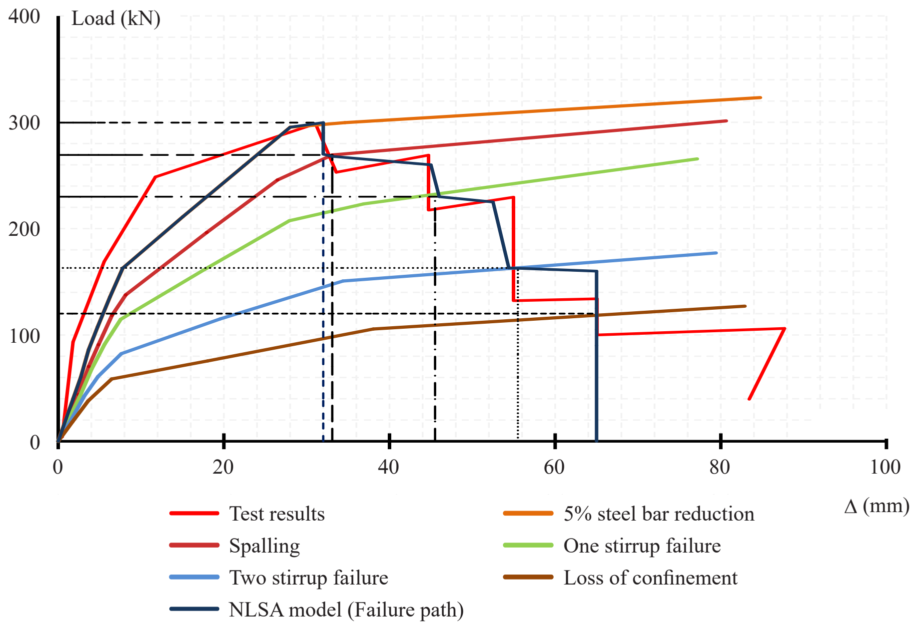

Figure 8. Envelope of load-displacement relationships for different damage levels using NLSA versus test results by Oyado et al. [3], specimen No.4

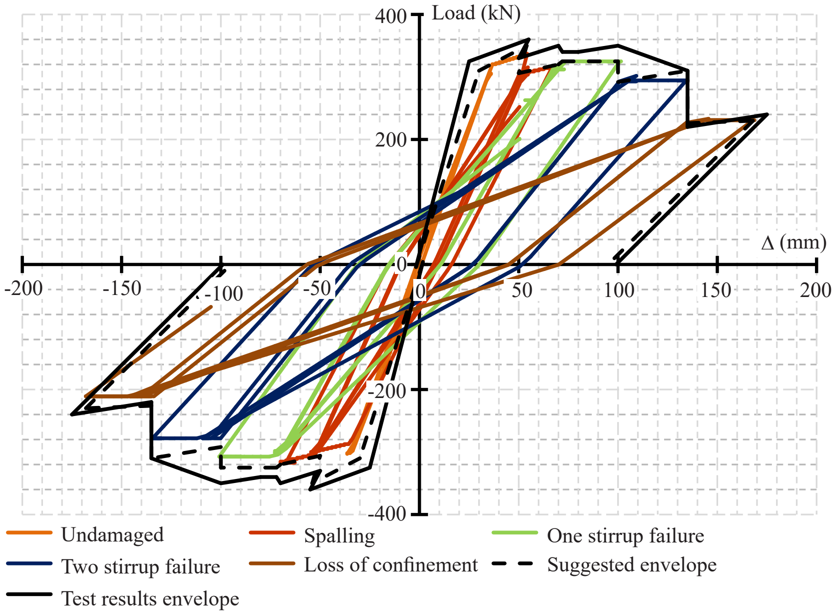

Figure 9. Envelopes of the load-displacement hysteretic relationship of a non-corroded specimen under cyclic load up to failure (specimen No.1 of Oyado et al. [3])

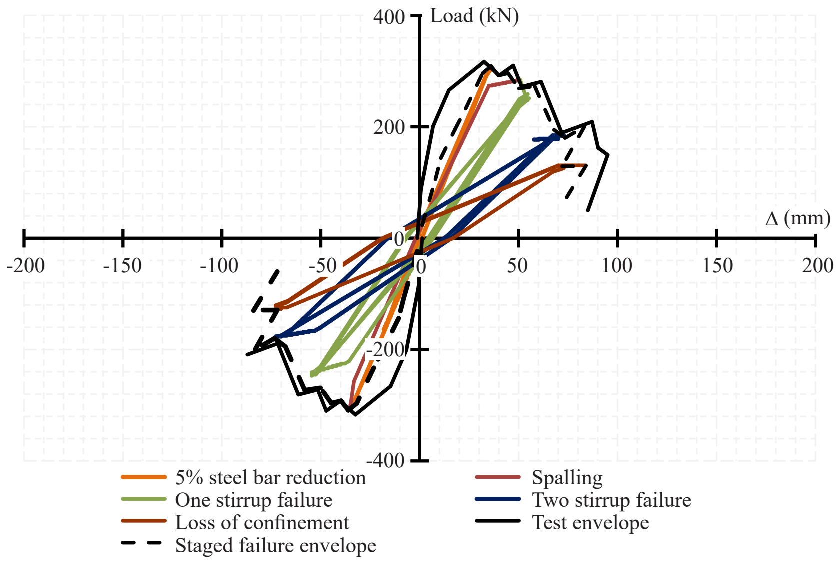

Figure 10. Envelopes of the load-displacement hysteretic relationship of a corroded specimen under cyclic load up to failure (specimen No.4 of Oyado et al. [3])

Following a similar approach, the staged failure path of the hysteretic load-displacement relationship of Oyado's non-corroded specimen (No.1) is established using the proposed SNLSA, which includes Takeda's model. In each cycle of the response, the damage state is changed from undamaged to spalling, to one and two stirrups failure, up to the loss of confinement (see Figure 9). The nonlinear load-displacement behavior is evaluated using the NLSA as part of the SNLSA, transferring the column's sectional behavior results in each damage state to Takeda's model. The SNLSA results again slightly underestimate the test results. The staged failure path of the hysteretic load-displacement relationship of Oyado's corroded specimen (specimen No.4) is also established (see Figure 10). The proposed SNLSA gives a larger underestimation of strength capacity than in the case of the non-corroded column. This underestimation can be explained by the conservative input data for material properties. Comparing the results in Figure 9 and Figure 10 shows a reduction in the hysteretic relationship, indicating a significant decrease in the column's energy absorption capacity due to corrosion damage.

5.3 *Slab-on-girder bridge column with variable load-over-capacity ratio*

This section discusses the second case study where the column design is varied according to an assumed load-over-capacity ratio. It aims to show the proposed SNLSA capability to estimate the structural behavior of bridge columns with various possible design alternatives. A slab-on-steel girder bridge with a center-to-center span of 61 m and width of 20.5 m constructed in 1977 [32] is selected. The bridge is simply-supported on RC piers formed from eight square columns. The 6.0-m height columns are assumed to have full fixity at the foundation level. The bridge superstructure consists of a concrete slab compositely cast on steel girders. The girder spacing is 2.64 m, their cross-sectional area is 0.05 m^2, and the concrete slab thickness is 0.22 m.

According to the Canadian Highway Bridge Design Code [5], the load combination for earthquake design assumes only 50% of the equivalent traffic load is acting on the bridge. Three design cases of the bridge column are discussed here based on three assumed Loads Over Capacity Ratios (LOCR), which are: (i) 25% (or low) for a conservative design case, (ii) 40% (or medium) for traditional design, and (iii) 60% (or high) for design controlled by aesthetic considerations. All three columns are designed for seismic resistance as per the Canadian Highway Bridge Design Code [5]. The concrete compressive strength (f_c') is assumed to be 35 MPa. For LOCR = 25%, the cross-sectional dimensions are 700 mm × 700 mm, the longitudinal reinforcement ratio is $\rho_{st} = A_s/A_g = 3.46\%$, with rebar diameter of 20 mm and hoops of 15 mm diameter at 30 mm c/c. For LOCR = 40%, the cross-sectional dimensions are 600 mm × 600 mm, the longitudinal reinforcement ratio is $\rho_{st} = A_s/A_g = 2.0\%$, with rebar diameter of 20 mm and hoops of 15 mm diameter at 30 mm c/c (see Figure 4). For LOCR = 60%, the cross-sectional dimensions are 500 mm × 500 mm, the longitudinal reinforcement ratio is $\rho_{st} = A_s/A_g = 2.5\%$, with rebar diameter of 20 mm and hoops of 10 mm diameter at 20 mm c/c.

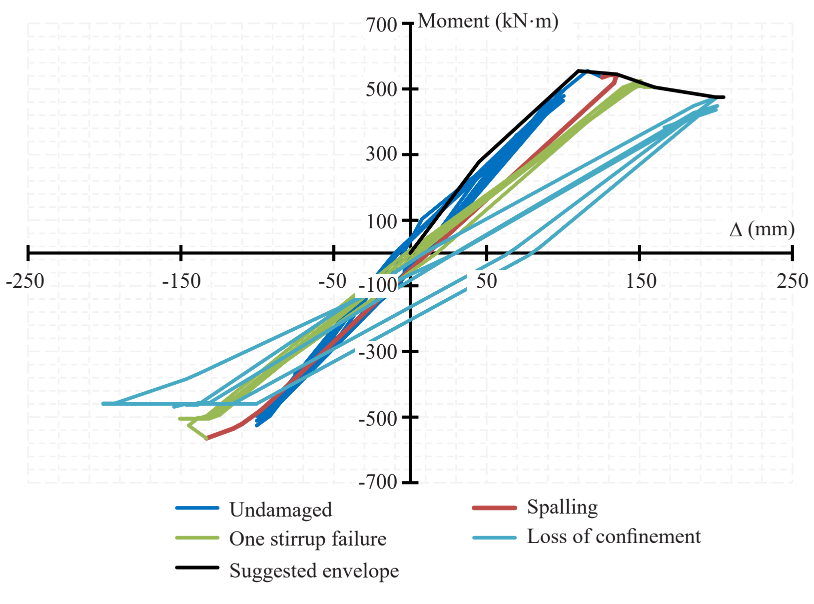

Figure 11. Modeled envelope of the load-displacement hysteretic relationship of non-corroded column (LOCR = 40%) under cyclic load up to failure

Corrosion damage is assumed to affect all three columns (see Figure 4). Figure 11 shows the load-displacement hysteretic relationship of the non-corroded column (LOCR = 40%) and the envelope when the column is subjected to cyclic load up to failure. The figure shows different states of damage, including cover spalling, loss of one stirrup, and the loss of confinement, for LOCR = 40% when no corrosion is applied (i.e., no loss of reinforcement cross-section and ductility), while Figure 12 shows the same relationships when corrosion is applied (assuming a corrosion current density of 1 μA/cm^2). In this case, it is assumed that a 5% reduction of steel cross-sectional area due to reinforcement corrosion would result in concrete cracks; while a 15% reduction would result in concrete spalling, a 20% reduction would result in one stirrup failure, and a 30% reduction would result in two stirrup failure. These assumptions are approximate and based on a generalization of the lab tests done by Oyado et al. [3]. The reason behind these assumptions is to illustrate how to apply the proposed analysis approach. In field investigations, accurate measurements of the steel and the concrete reduction are required; however, for approximate evaluation, using empirical equations could reduce the number of tests and samples needed to prepare the data input.

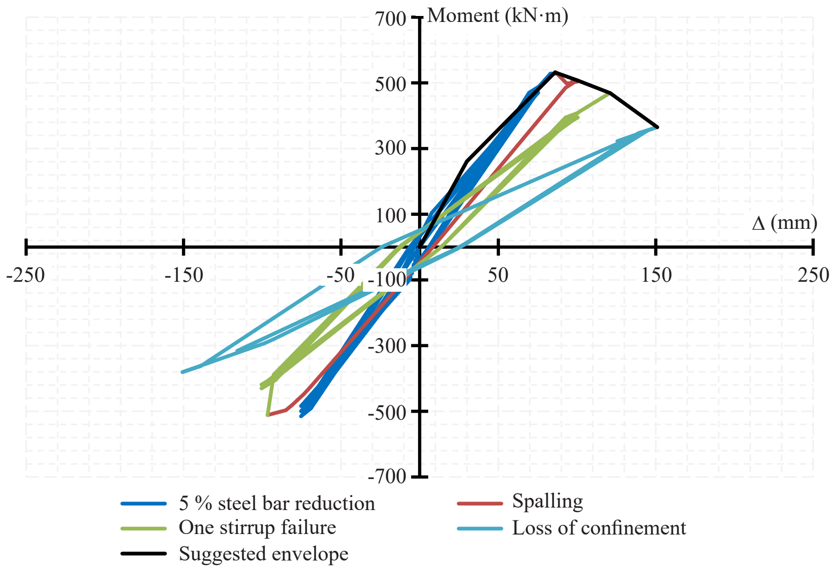

Figure 12. Modeled envelope of the load-displacement hysteretic relationship of corroded column (LOCR = 40%) under cyclic load up to failure

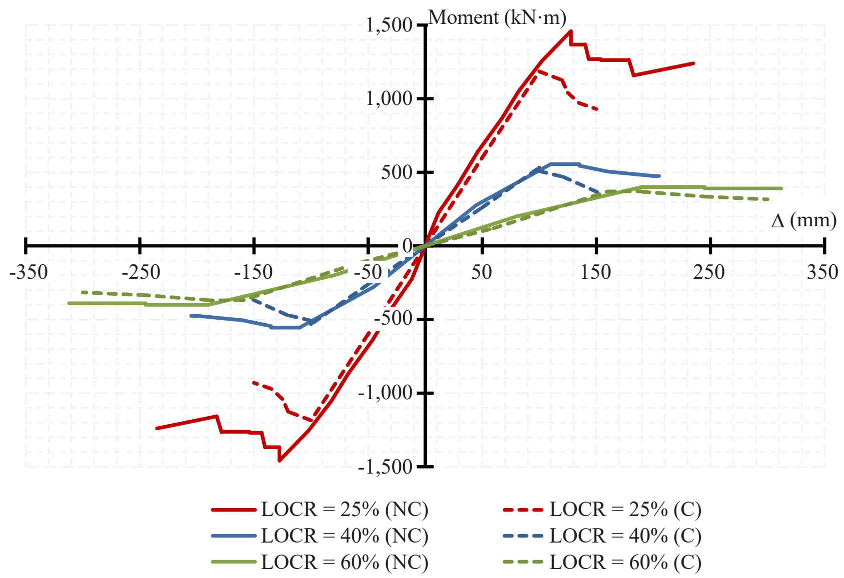

Figure 13. Modeled envelope of load-displacement hysteretic relationship of Non-Corroded (NC) and Corroded (C) columns (different LOCR percentages) under cyclic load up to failure

Figure 13 compares the envelopes of the moment-displacement hysteretic relationships for the three columns when no corrosion is accounted for and when corrosion is applied. Similar to the first case study, the comparisons of different cases show that corrosion-induced damage results in a considerable reduction of the column load and displacement capacities. The comparisons also show a decrease in the hysteretic relationship, which indicates a significant decline in the energy absorption capacity due to corrosion damage.

In this case study, the behaviors of corrosion-damaged versus non-damaged columns subjected to seismic loading are compared. The comparisons show that SNLSA can simulate the staged degradation of capacity up to collapse or the "staged failure mechanism" for all column design options.

5.4 *Time-history of displacement of non-corroded and corroded bridge columns*

The data of a short earthquake record in Ottawa, Ontario [28], is used to study the three columns' time history behavior of the last case study presented in Section 5.3. As the column's steel area and concrete cover at the critical cross-section are reduced due to corrosion and resulting damage, the column stiffness is reduced accordingly. With the reduction in the column flexural stiffness and the reduction in the reinforcing steel ductility, the time history of lateral displacement of the corroded section shows an increase when the bridge is subjected to seismic load. In the parametric study of the displacement time history presented in the following paragraph, the corrosion damage is considered to be located on the column's top and bottom sections as well, although field observations show that the middle height zone of the column is the most corrosion-affected zone [26].

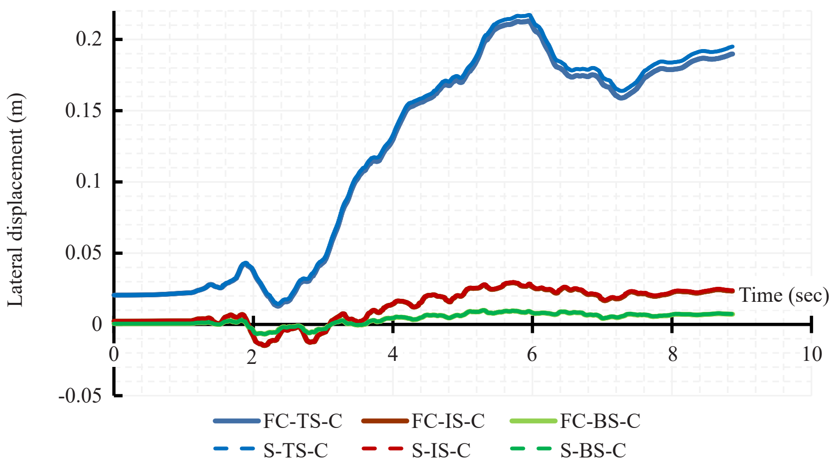

Figure 14. Time history of lateral displacement for different sections of the non-corroded column with LOCR = 40%

Figure 14 and Figure 15 show the time history of the lateral displacement for the top, mid-height, and bottom sections of the column with LOCR = 40% for the non-corroded and corroded sections, respectively; it is assumed that corrosion attacks one section at a time. Table 1 defines the acronyms used in the figures. As expected for the columns' given boundary conditions, the figures show that the top section has the maximum deformation compared to the other sections for all the cases. It is also observed that the non-corroded column can continue to deform up to and after the loss of one stirrup, while the corroded column develops the same level of deformation only up to the spalling of the concrete cover followed by the collapse of the column. In Figure 16, the staged failure path in the time history is established based on the level of deformation correlated to each damage state in the earlier analysis steps (NLSA and load-displacement hysteric analysis). Each state of damage is identified in the figure by a different colour. The figure shows that the corroded columns develop larger deformations at earlier stages of damage. For instance, with LOCR = 40% and at the same seismic excitation and time frame, the corroded column with only cover spalling deforms more than the non-corroded column with one stirrup broken. The same observation is recorded with other LOCR values as shown in Figure 17 and Figure 18. However, the columns with large LOCR have no ability to develop an acceptable time history of deformation when an advanced state of corrosion damage is applied. Figure 17 and Figure 18 compare the columns' time history for different LOCR values and the three selected sections (top, middle height, bottom). Based on Figures 14 through 18, it is concluded that for seismic-critical zones, a low load over capacity ratio (LOCR < 40%) or "overdesign" of the columns is recommended. On the other hand, it is found that a cost-effective structural evaluation can be achieved by only using the NLSA and/or the hysteretic analysis without the need to conduct the time-history analysis.

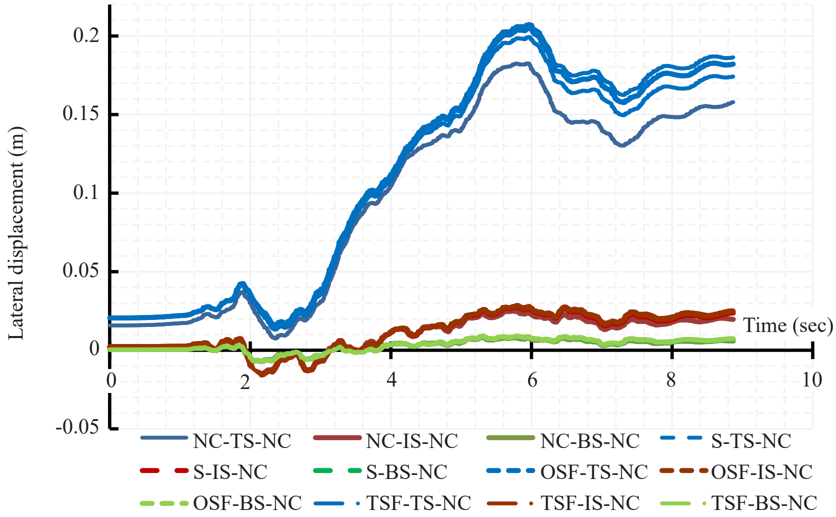

Figure 15. Time history of lateral displacement for different sections of the corroded column with LOCR = 40%

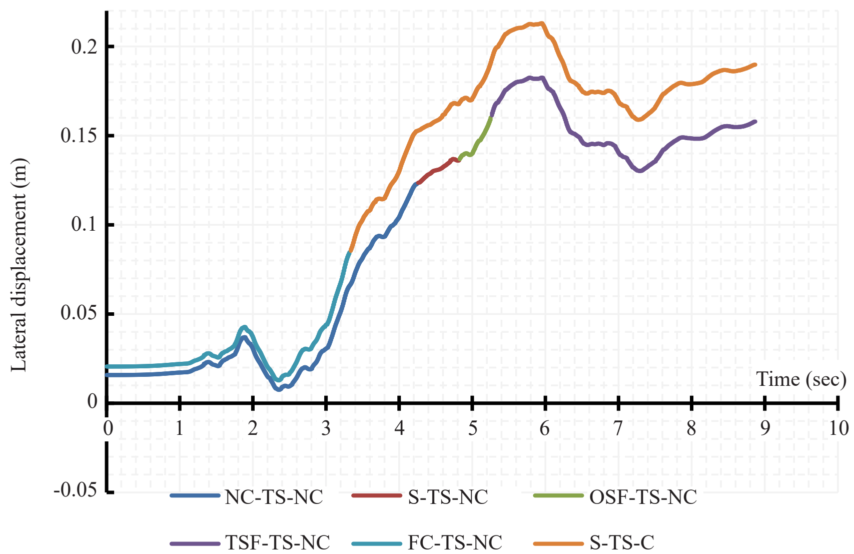

Figure 16. Time history of lateral displacement of top-section of the non-corroded and corroded column with LOCR = 40%

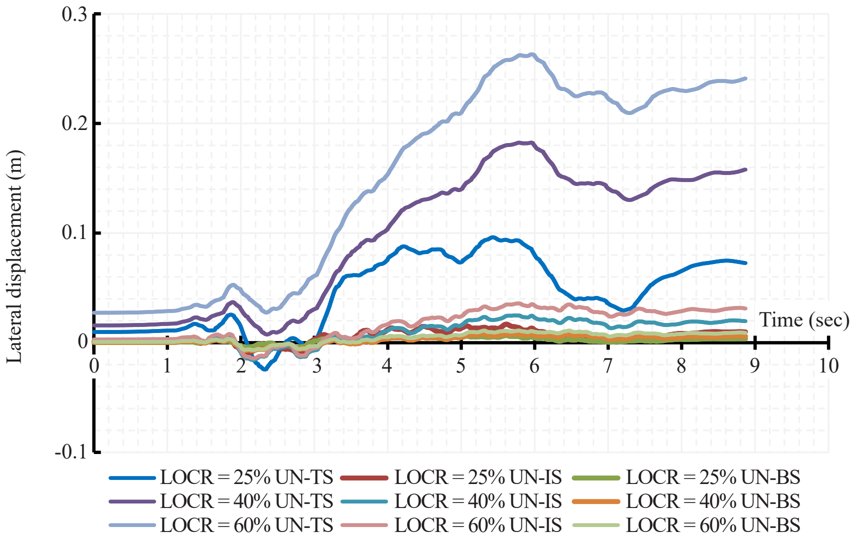

Figure 17. Time history of lateral displacement for different sections of the non-corroded columns with different LOCR values

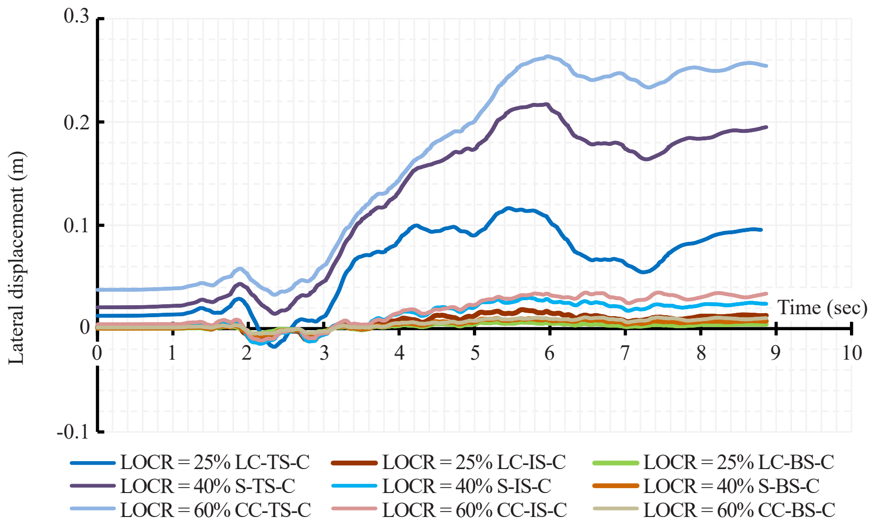

Figure 18. Time history of lateral displacement for different sections of corroded columns with different LOCR values

Figure 19 shows the interaction diagram of corrosion-damaged RC columns versus undamaged columns for different LOCR values. The state of corrosion damage is assumed to be severe, resulting in local loss of the concrete confinement after two stirrups' failures. The figure shows a significant contraction of the column interaction capacity when subjected to severe corrosion damage for all load-over-capacity ratios. It is observed that the percentage reduction is very high when the column is conservatively designed or designed with a high LOCR. For columns designed for a LOCR of 60%, it is found that the interaction relationship is reduced to highly unsafe levels when corrosion-induced damage is applied. The figure shows that for conservative or medium LOCR values (25% and 40%), the interaction envelope is higher than the applied service load and moment by a large margin. This significant impact of reinforcement corrosion on the axial load-moment interaction relationship confirms the previous recommendation on the need to conservatively design bridge columns with low to medium LOCR (LOCR ≤ 40%) in the critical seismic zones when reinforcement corrosion is expected.

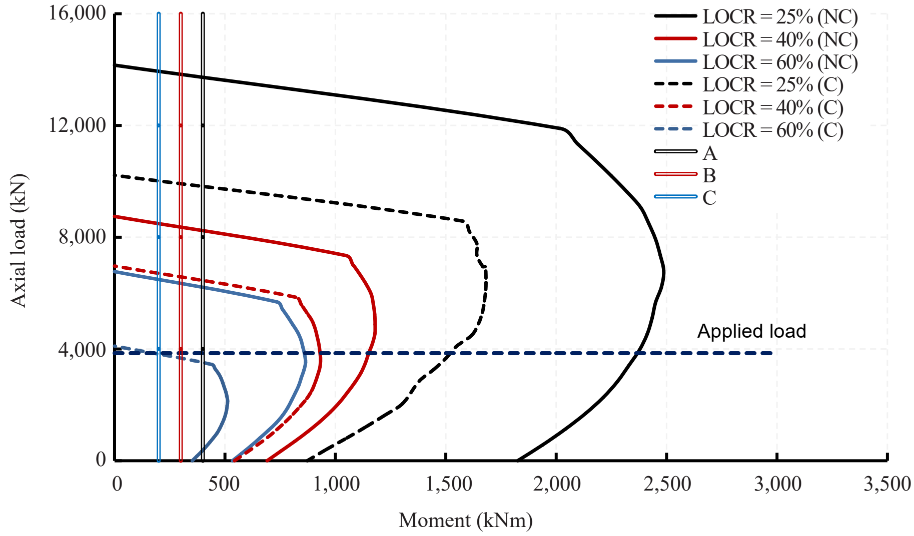

Figure 19. Interaction diagrams of non-corroded and corroded columns with different LOCR values; A: the applied moment of LOCR = 25%; B: the applied moment of LOCR = 40%; C: the applied moment of LOCR = 60%

Table 1. Acronyms definition for Figures 14-19

Definition	Acronyms	Definition	Acronyms
Load over capacity ratio	LOCR	Loss confinement-TS-NC	LC-TS-NC
Top section	TS	Loss confinement-IS-NC	LC-IS-NC
Intermediate section	IS	Loss confinement-BS-NC	LC-BS-NC
Bottom section	BS	Flexural cracks-TS-C	FC-TS-C
Non-corroded	NC	Flexural cracks-IS-C	FC-IS-C
Corroded	C	Flexural cracks-BS-C	FC-BS-C
Non-corroded-TS-NC	NC-TS	Spalling-TS-C	S-TS-C
Non-corroded-IS-NC	NC-IS	Spalling-TS-C	S-IS-C
Non-corroded-BS-NC	NC-BS	Spalling-TS-C	S-BS-C
Spalling-TS-NC	S-TS-NC	1 stirrup failure-TS-C	OSF-TS-C
Spalling-IS-NC	S-IS-NC	1 stirrup failure-IS-C	OSF-IS-C
Spalling-BS-NC	S-BS-NC	1 stirrup failure-BS-C	OSF-BS-C
1 stirrup failure-TS-NC	OSF-TS-NC	2 stirrup failure-TS-C	TSF-TS-C
1 stirrup failure-IS-NC	OSF-IS-NC	2 stirrup failure-TS-C	TSF-IS-C
1 stirrup failure-BS-NC	OSF-BS-NC	2 stirrup failure-TS-C	TSF-BS-C
2 stirrup failure-TS-NC	TSF-TS-NC	Loss confinement-TS-C	LC-TS-C
2 stirrup failure-IS-NC	TSF-IS-NC	Loss confinement-TS-C	LC-IS-C
2 stirrup failure-BS-NC	TSF-BS-NC	Loss confinement-TS-C	LC-BS-C
		Applied Moment	AM

6. Summary and conclusions

A simplified, cost-effective, and handy nonlinear seismic analysis (SNLSA) approach is proposed to evaluate columns' seismic response as part of a Semi-Quantitative Assessment Framework (SQAF). The approach is based on Nonlinear Sectional Analysis (NLSA), DRAIN-RC nonlinear time history analysis program, and Takeda's hysteretic analysis model. The SNLSA is capable of matching experimental results with high accuracy. The approach provides three options: (i) establish the staged failure mechanism using expressed analysis simulating quasi-static loading up to failure; (ii) use a more comprehensive analysis simulating cyclic loading developing the hysteretic relationship; and (iii) conduct a complete time-history analysis.

The SNLSA quantitatively shows the change in the structural performance of corrosion-affected bridge columns. It shows that corrosion-induced damage results in a considerable reduction of the column load and displacement capacities and a considerable reduction in the hysteretic relationship, which indicates a significant decrease in the different columns' energy absorption capacities. The time history analysis shows that the corroded column deforms more than the non-corroded column at earlier stages of damage at the same seismic excitation and time frame. The SNLSA could also be used to select the appropriate design option for bridge columns in seismic-critical zones. The study recommends that corrosion-damaged columns' time history analysis is too complex and not always needed for the evaluation process. The SNLSA can estimate the significant contraction of the column interaction capacity when subjected to severe corrosion damage for all load-over-capacity ratios.

Conflicts of interest

The authors confirm that there is no conflict of interest.

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