

# Détection d'endommagement dans les ponts par recalage de modèles numériques

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## Résumé

Cet article présente un volet d'un projet de recherche global mené par l'Université de Sherbrooke et le Ministère des Transports du Québec visant à détecter de l'endommagement dans les ponts à partir de mesures de vibrations ambiantes induites par le trafic routier. La méthode repose sur la mise à jour automatisée d'un modèle 3D très détaillé par éléments finis de la structure à partir de ses propriétés modales expérimentales. Le cas d'étude du pont Est de la Rivière-aux-Mulets situé sur l'autoroute 15 à Ste-Adèle est présenté. L'exposé se concentre sur le développement d'une procédure efficace, pratique et systématique d'application des algorithmes de mise à jour de modèles numériques à des fins de détection d'endommagement dans les ponts, basées sur l'évolution des fréquences et modes propres de la structure entre deux périodes de temps où des mesures ont été effectuées. Le logiciel commercial FEMtools est utilisé pour résoudre ce problème inverse fortement sous-déterminé, où le nombre d'inconnues surpasse largement le nombre d'équations expérimentales.

Le pont à l'étude est un pont de type poutre-caisson, construit par encorbellements successifs en 1964 et qui a connu un endommagement graduel depuis sa construction. Un modèle du pont par éléments finis est d'abord brièvement présenté, qui résulte d'un travail ardu de recalage très précis sur les propriétés dynamiques de la structure mesurées expérimentalement en 2013. Quatre cas d'endommagements sont ensuite simulés numériquement en réduisant la rigidité de groupes d'éléments du modèle situés au niveau des travées, des piles ou des appuis du pont. Les effets de ces dommages sur les propriétés modales de la structure sont analysés. Ils montrent que les déformées modales ne sont pas des indices d'endommagement pertinents et que des dommages mêmes élevés ne sont pas nécessairement synonymes de réduction critique des fréquences propres de la structure. La méthode de détection utilisée, basée sur l'évolution des propriétés modales, doit donc être très sensible.

L'application directe des algorithmes de recalage de modèles pour les cas de dommages simulés n'en fournit pas d'estimation satisfaisante. Une procédure d'application optimisée est donc proposée, qui repose sur trois éléments principalement : (i) le contrôle du nombre d'équations par rapport au nombre de variables de recalage dans une plage optimale en utilisant pour objectifs de recalage additionnels les points individuels d'une déformée modale; (ii) l'utilisation d'une procédure itérative de détection dans laquelle des variables de recalage sont éliminées progressivement en fonction des résultats obtenus de l'itération précédente dans le but de réduire le nombre d'inconnues du problème sous-déterminé; (iii) empêcher les algorithmes de recalage d'augmenter la rigidité des éléments en béton qui sont supposés s'endommager à travers le temps. Les résultats de cette procédure appliquée aux quatre cas de dommages simulés donne d'excellents résultats tant pour localiser que pour quantifier le dommage à la structure. Un de ces cas combine deux types de dommages aux effets opposés sur les propriétés modales de la structure et montre la robustesse de la procédure à détecter des endommagements complexes.

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## INTRODUCTION

Damage detection methods using model updating are based on the modification of mass, stiffness and damping matrices of finite element (FE) models to minimize the discrepancies between experimentally measured responses of the structure and the same responses of the numerical model. The application of the method between two periods of time allows for damage identification, localization and quantification by comparing the updated matrices to the original

correlated matrices. Although update calculations are performed automatically by algorithms, the application of the model updating method is not straightforward and depends on many user-defined parameters [1]. The user should also be aware of numerous sources of errors [2, 3], including modelling errors and experimental errors in the target modal responses.

Most civil engineering case studies of model updating in the literature have been applied for damage detection purposes only on small laboratory structures [4], on real-scale structures that can be idealized with stick models [5] or were used for reduced problems using simplified damage functions [6, 7]. In this paper, the commercial software FEMtools [8] is used for direct damage detection by FE model updating on a real bridge using a detailed 3D FE model where the number of update parameters of the model largely exceeds the number of experimental data. The study presented is part of a global research project lead by the University of Sherbrooke and the Ministry of Transportation of Quebec aiming to detect damage in bridges with modal analyses of traffic-induced ambient vibrations using FE model updating. The paper presents the case study of the Rivière-aux-Mulets Bridge located on Highway 15 in Ste-Adèle north-west of Montreal (QC, Canada). The bridge is first briefly presented together with modal analysis results. Then, the reference FE model used for damage detection is briefly presented, resulting from a challenging work of tuning based on experimental modal analyses. Four numerically-simulated damage cases to either deck regions, piers or bearings are considered by modifying the stiffness of the elements in the model. The effects of these damages on the vibration frequencies and mode shapes are studied. The basic application of the FE model updating algorithms of FEMtools is shown not to provide good estimates of the simulated damages. Thus, an improved step-by-step application procedure of the model updating methods dedicated to bridge damage detection with detailed 3D FE models is then established to obtain systematically reliable damage detection results for all damage cases, including combined damage cases.

## RIVIÈRE-AUX-MULETS BRIDGE

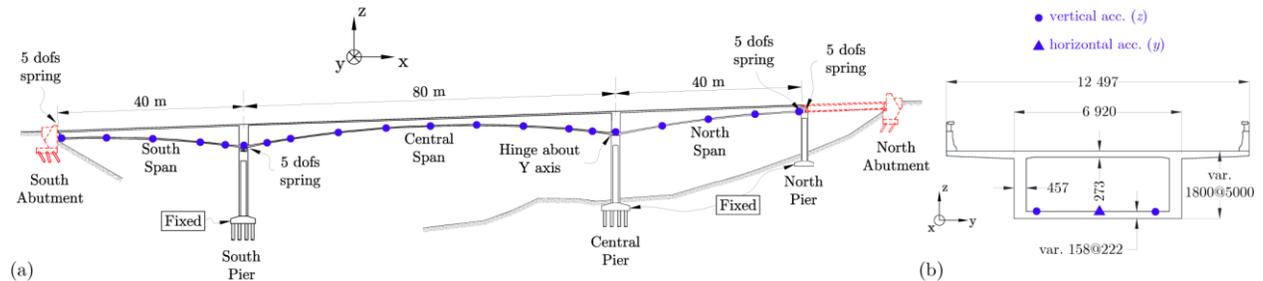


Figure 1. Rivière-aux-Mulets Bridge and measurement locations: (a) elevation view; (b) cross-section of the box girder.

The bridge under study in this paper is the Rivière-aux-Mulets Bridge located on Highway 15 (East direction) in Ste-Adèle, north-west of Montreal (Quebec, Canada). The bridge was built in 1964 together with a twin bridge in the West direction and was one of the first post-tensioned concrete box girder bridge built by balanced cantilever method in North America. Figure 1 presents an elevation view of the bridge and the typical cross-section of the concrete box girder. The twin bridges underwent several increasing damage mainly related to this erection method. The west bridge was replaced in 2006 and major repairs were done on the east bridge including addition of post-tensioning cables and of monitoring systems.

Several modal identification tests under traffic-induced ambient vibrations were performed on the bridge by the Ministry of Transportation of Quebec at regularly spaced intervals since 2004,

including a much detailed modal analysis in November 2013 performed with the University of Sherbrooke. This former test was used as a basis for the damage detection study presented in this paper. Figure 1a shows the 20 measurement locations instrumented with accelerometers along the deck of the bridge for this test. Figure 1b shows the location of the three accelerometers used at each location in the box girder (1 in  $y$  direction, 2 in  $z$  direction), resulting in 60 recorded degrees of freedom (DOF) on the whole structure. Table 1 presents the first eight natural frequencies identified using the Frequency Domain Decomposition method (FDD) for the bridge in 2004 and 2013 together with the type of each mode (flexural, lateral or torsional). Figure 2 illustrates the mode shapes of three modes.

Table 1. Natural frequencies of the bridge.

Exp. mode	Type*	2004	2013	$\Delta f_{\text{exp}}$	FEM	$\Delta f_{\text{num}}$	MAC <sub>num</sub>
1	1 <sup>st</sup> F	1.534 Hz	1.563 Hz	+1.89 %	1.568 Hz	+0.30 %	0.998
2	1 <sup>st</sup> L	1.987 Hz	2.036 Hz	+2.47 %	2.035 Hz	-0.04 %	0.954
3	2 <sup>nd</sup> F	3.335 Hz	3.330 Hz	-0.15 %	3.337 Hz	+0.23 %	0.985
4	3 <sup>rd</sup> F	4.559 Hz	4.616 Hz	+1.25 %	4.595 Hz	-0.45 %	0.925
5	2 <sup>nd</sup> L	5.005 Hz	5.209 Hz	+4.08 %	5.206 Hz	-0.04 %	0.974
6	4 <sup>th</sup> F	5.583 Hz	5.644 Hz	+1.09 %	5.640 Hz	-0.06 %	0.957
7	1 <sup>st</sup> T	6.252 Hz	6.185 Hz	-1.07 %	6.185 Hz	+0.00 %	(0.432)
8	5 <sup>th</sup> F	8.067 Hz	8.182 Hz	+1.43 %	8.194 Hz	+0.16 %	0.882

\*Note: F = Flexural L = Lateral T = Torsional

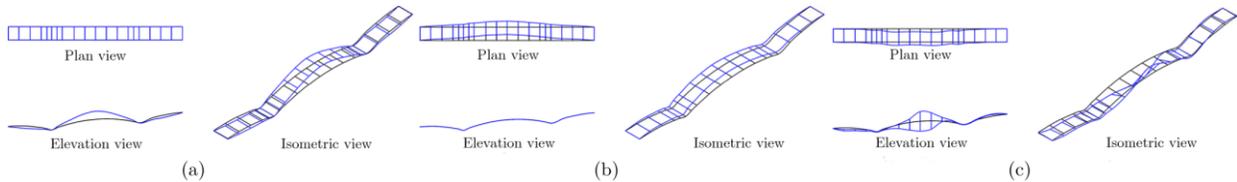


Figure 2. Mode shapes of the bridge: (a) 1<sup>st</sup> flexural (mode 1); (b) 1<sup>st</sup> lateral (mode 2); (c) 1<sup>st</sup> torsional (mode 7).

## UPDATED FINITE ELEMENTS MODEL OF THE BRIDGE

A 3D detailed numerical model of the bridge was developed using the FE software Abaqus [9] and it was then directly exported into FEMtools to perform model updating tasks. Elements in red in Figure 1 were not modeled and their interaction with the rest of the bridge was considered by using spring elements. Figure 3a illustrates the 3D FE model. The final detailed model is made of 4484 hexahedral C3D20R elements for the concrete box girder (length of the elements is similar to the mean length of the segments), 414 hexahedral C3D20R elements for the piers and 20 spring elements for the boundary conditions. The boundary conditions used are shown in Figure 1a and mainly consist in fixed conditions at the base of the piers and five-DOF spring elements (all DOF except rotation about  $y$ -axis that is free) either for piers-to-deck or abutments-to-deck interactions.

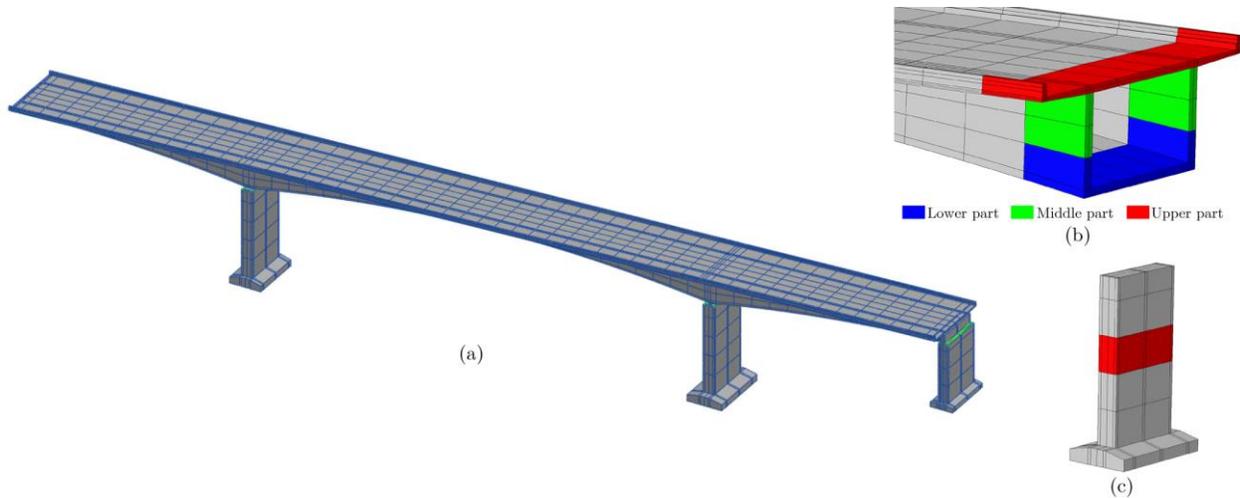


Figure 3. Detailed 3D FE model of the bridge: (a) mesh; (b) box girder update parameters; (c) piers update parameters.

Model updating algorithms of FEMtools were used to update the model with the experimental modal properties of the bridge extracted in 2013. By doing this, the authors proposed a systematic practical updating procedure dedicated to bridge structures that leads to a well-tuned FE model in five steps. This methodology is out of the scope of this paper and only information needed for the rest of the paper is presented. Update parameters were defined as the Young's moduli of the elements of the segments and of the piers grouped in layers as illustrated in Figure 3b and 3c, resulting in 199 update parameters including the stiffness of the 20 springs for the boundary conditions. A sensitivity analysis performed on the natural frequencies allowed to discard not-detectable parameters (middle part of the box girder and several springs) to finally keep 133 update parameters (including 8 springs). Natural frequencies and mode shapes of the eight identified modes in Table 1 (except the inaccurate shape of the torsional mode 7) were used as target responses of the structure for the model updating process. Thus, the unknowns-to-experimental equations ratio in the model updating inverse problem is  $r_p = 133/15 = 8.87$ , corresponding to a largely underdetermined problem. Mode shape target responses were considered through MAC values. Figure 4 and last columns of Table 1 illustrate the accuracy of the updated FE model, which will be used as a reference for the damage detection study.

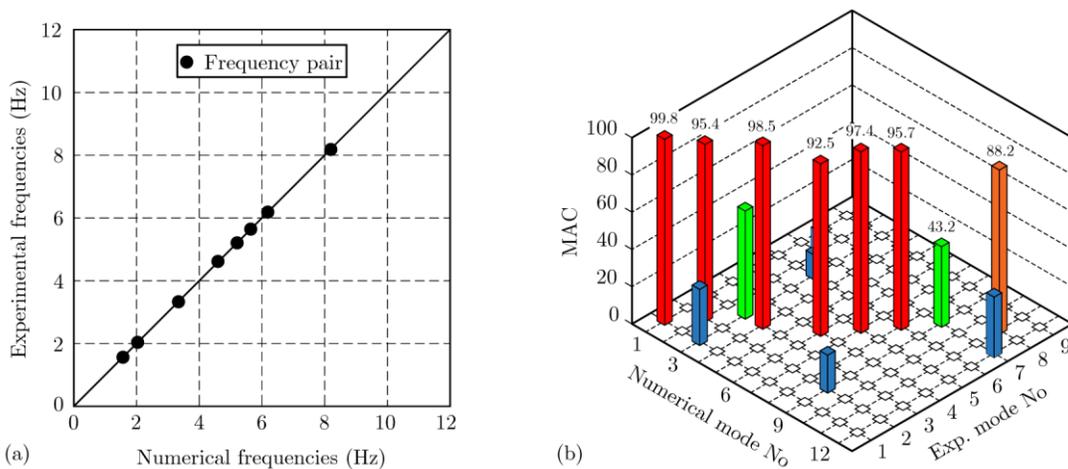


Figure 4. Correlation between updated numerical and 2013 experimental (a) natural frequencies; (b) mode shapes (MAC)

## EFFECTS OF SIMULATED DAMAGE CASES ON MODAL PROPERTIES

Numerically simulated damages to the bridge were considered in this damage detection study by model updating rather than real damages. Apart from the fact that real cases of significantly damaged structures with modal properties available before and after the damage occurred are difficult to find, the main reason for simulated damage cases is that the damage detection process using FEMtools with a detailed 3D FE model had to be first evaluated on reference cases where the solution is known exactly. Simulated cases were thus required to develop a practical, systematic and efficient methodology able to provide reliable results in such largely underdetermined model updating problems.

Four damage cases are presented in this paper and are illustrated in Figure 5. First case (Fig 5a) consists in a loss of stiffness up to -50% spread in a realistic manner through several elements of the underside of the north span of the bridge deck. In a similar manner, the second case (Fig 5b) consists in a progressive stiffness reduction at the base of the south pier up to -50%. The third case (Fig 5c) corresponds to the deterioration of the south pier-to-deck bearing in the lateral direction with an increase of stiffness of +50%. The last case (Fig. 5d) is a combination of damage to the central span of the bridge deck (-50% loss of stiffness) and to the rotation bearing around  $x$ -axis at the south pier (+50% gain of stiffness).

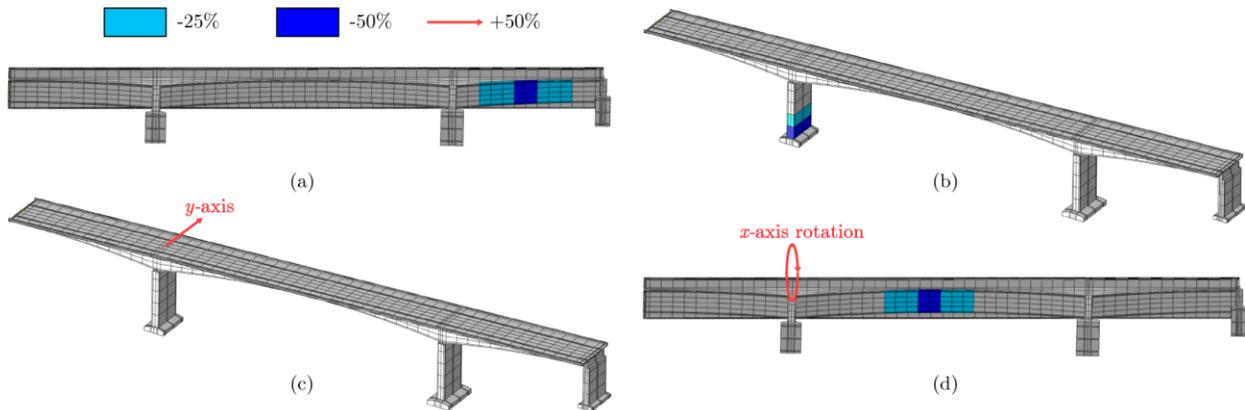


Figure 5. Simulated damage cases: (a) north span; (b) south pier; (c) lateral bearing at south pier; (d) combined damage.

When using model updating damage detection methods based on damage-induced evolutions of natural frequencies and mode shapes, it matters to understand the orders of magnitude of these modifications related to the kind and intensity of the damages. Table 2 compares the natural frequencies and mode shapes (through MAC values) of the original model to those of the damaged models for each of the four damage cases considered. MAC values between mode shapes were computed by using only the same DOF available in the experimental modal analysis presented in Figure 1. For synthesis purpose a cumulative damage index is presented in the last line of Table 2 for frequencies and MAC that corresponds to the square root of the quadratic sum of the evolutions over all modes. Table 2 shows that local damages such as those presented have only slight effects on the mode shapes of the structure, which translates into MAC values always near or equal to unity. Thus, MAC cannot be considered as valuable damage indices. If we focus on natural frequencies, it is important to note that even significant damages do not necessary lead to large evolutions. The order of magnitude of the cumulative frequency damage index between 3% and 5% (up to 7% for the case of combined damage) makes any concerned engineer appreciate that the generally low evolutions of experimentally measured frequencies on a real

structure (as those obtained between 2004 and 2013 in Table 1) can be in fact related to non-negligible damages. This result also points out the need for model updating damage detection methods to be very sensitive and the obvious problem caused by experimental uncertainties.

Table 2. Damage-induced evolutions of the modal properties for each damage case.

Exp. Mode	$f_{num}$	North span		South pier		y-south bearing		Combination	
		$\Delta f$	MAC	$\Delta f$	MAC	$\Delta f$	MAC	$\Delta f$	MAC
1	1.568 Hz	-3.29 %	0.999	-0.18 %	1.000	+0.00 %	1.000	-2.74 %	1.000
2	2.035 Hz	-0.14 %	1.000	-3.57 %	0.999	+3.21 %	0.997	+0.24 %	1.000
3	3.337 Hz	-2.38 %	0.996	-0.04 %	1.000	+0.00 %	1.000	-1.91 %	0.999
4	4.595 Hz	-1.60 %	0.993	-0.34 %	1.000	+0.00 %	1.000	-0.84 %	0.999
5	5.206 Hz	-0.37 %	1.000	-0.27 %	1.000	+0.15 %	0.999	-0.31 %	0.999
6	5.640 Hz	-1.34 %	0.995	-0.51 %	0.999	+0.00 %	1.000	-1.34 %	0.999
7	6.185 Hz	-0.00 %	1.000	-0.36 %	1.000	+2.66 %	0.998	+5.26 %	0.988
8	8.194 Hz	-0.68 %	0.998	-0.65 %	0.999	+0.00 %	1.000	-3.30 %	0.996
Cumulative damage index		4.63 %	1.0 %	3.71 %	0.2 %	4.17 %	0.4 %	7.24 %	1.3 %

## DETECTION OF SIMULATED DAMAGE CASES USING MODEL UPDATING

FEMtools model updating algorithms were used to detect the simulated damage cases presented in Figure 5. The study consisted in extracting for each damaged model the eight natural frequencies and mode shapes corresponding to those available through an experimental modal analysis and to use them as target responses to update the undamaged reference model. The same 133 update parameters defined to tune the reference model and the same target modal properties were used in the damage detection study, i.e. eight natural frequencies and seven mode shapes defined only at DOF available in the experimental modal analysis.

### Direct application of the model updating process

Like most model updating algorithms, FEMtools allows the user for setting different computing options that can greatly influence the results of the analysis. Particularly, several convergence criteria can be used and their acceptance value can be chosen, and weighting factors (namely, *scatters* in FEMtools) on both target responses and update parameters can be set. All of these parameters (*scatters* particularly) are not easy to define at optimal values even for experienced users, and they often relies on non-evident considerations that depend on each case study or on trial-and-error analyses. A direct model updating calculation was first performed to detect the simulated damages by using standard computing options of FEMtools. By experience, the authors set the convergence criterion to  $CC_{TOT} = CC_{ABS} + CC_{MAC} < 0.2\%$  (this low limit value is justified by the numerical nature of the study without uncertainties) or maximum 20 update iterations. Criteria  $CC_{ABS}$  and  $CC_{MAC}$  correspond to the weighted mean relative differences of all natural frequencies and mode shapes (through MAC values), respectively, between the target responses and the FE model responses. Values of *scatters* were set to FEMtools default values, which give more importance to the tuning of the natural frequencies than to mode shapes (due to higher uncertainties in usual experimental studies) and which consider that all update parameters have the same uncertainty.

Figure 6a presents in a graphical form the stiffness modification applied to the update parameters by the model updating analysis for the case of damage in the north span. Even if the analysis identifies non negligible damage mainly to the bridge deck, the results of such an analysis are obviously not useful to localize accurately the concerned span of the bridge nor the upper or lower part of the girder, nor to quantify the damage. For academic purpose, another analysis was performed by using all natural frequencies and mode shapes of the first 20 modes of the FE

model (Fig. 6b). In such conditions, localization and quantification results are highly improved, although results are still not perfect with non-negligible damage identified in both other spans and on the vertical bearing of the north pier. These last results are of course practically not available since only eight modes were identified experimentally. However, they show that the main challenge of damage detection by updating detailed 3D FE models is the lack of experimental data compared to the number of unknowns.

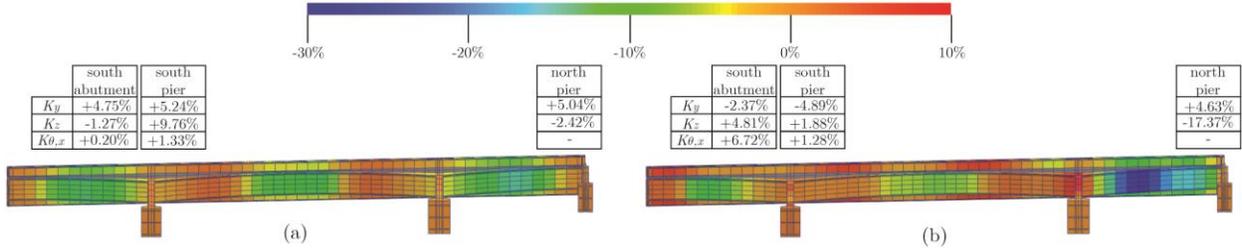


Figure 6. Direct model updating results for damage detection of the north span case using: (a) 8 modes; (b) 20 modes.

### Efficient model updating procedure for damage detection in bridge structures

Considering the results of the previous section, the authors focused on the development of a practical, systematic and efficient model updating procedure dedicated to bridge structures that allows for reliable results when a detailed FE model is used that makes the inverse problem largely underdetermined. Since the objective was to get a user-friendly procedure, the authors developed the methodology to be efficient with FEMtools default values of all weighting factors. The proposed methodology relies on three points aiming to control the unknowns-to-equations ratio  $r_p$  and to guide the analysis to the most probable solution:

1. Use of selected individual DOF of one mode shape as target modal responses in the model updating process in addition to the natural frequencies and mode shapes MAC. By doing this, the determination ratio  $r_p$  of the inverse problem can be controlled in a recommended range from 1.0 to 2.0 (slightly underdetermined problem) defined according to the authors' experience. The first flexural mode is a good candidate for these additional responses because it is generally experimentally well-defined and it is sensitive to almost all update parameters in a bridge. The choice of the DOF used should ensure that the mode shape is globally well-defined over the whole structure.
2. Proceed with iterative analyses by progressively discarding the update parameters that remain unmodified and that are obviously not involved in the sought damage. Whole regions of structure should be discarded instead of individual update parameters. By doing this, the excessive number of unknowns can be reduced at each step. At each iteration, the updating process should start from the undamaged FE model and the user should also choose adequately the additional target DOF in order not to get a largely overdetermined problem, which generally causes the algorithms to diverge.
3. Avoid concrete elements from stiffening over the update process by imposing a zero upper limit to the allowed variation of the related update parameters. Even if not essential, this strategy proved to be useful to get more accurate results. The authors acknowledge that it is a kind of bias assuming that concrete components can only loose stiffness over the time, which is an acceptable assumption at least for non-brand-new concrete structures.

## Step-by-step application of the developed procedure for the north span damage case

For illustration purpose, the previous model updating procedure is applied and described in details in this section using the north span damage case, which proved to be the one that required the largest number of iterations. Although the procedure implies subjective decisions of the user concerning the parts of the structure to discard from one iteration to the other, the authors' choices always relied on clear evidences.

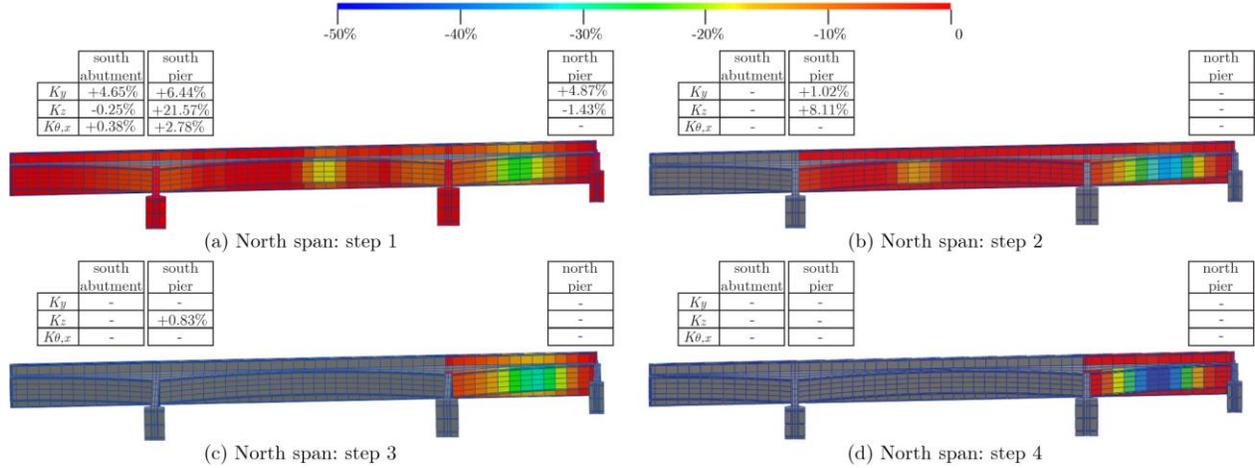


Figure 7. Application of the proposed model updating iterative procedure to detect damages for the north span case.

Figure 7a presents the modifications to the update parameters after the first iteration, which was performed by using all 133 update parameters and for targets the eight natural frequencies, seven mode shape MAC and all experimentally available DOF of the first flexural mode. Considering instrumented DOF shown in Figure 1, 60 new target responses were added by using individual DOF of mode 1, which results in a reduction of the unknowns-to-equations ratio  $r_p$  from 8.87 to 1.77. The comparison of Figure 7a with Figure 6a shows a great improvement of the detection results, even before any iteration is done. The probable main damage is clearly identified on the lower part of the north span with -25% stiffness reduction (compared to -15% in Fig 6a), but some doubts remain about eventual damage to the central span and to at least the vertical bearing of the south pier. All piers, the south span of the deck and most of the bearings are obviously not affected by the analysis. Hence, a second iteration was performed, which started from the undamaged model and used only update parameters from the central and north spans and two remaining bearing springs that were modified by more than 5% during iteration 1. In order to ensure a slightly underdetermined problem instead of an overdetermined one, only additional target DOF of mode 1 from the downstream side of the bridge were used (which does not affect the global definition of this flexural mode), resulting in a ratio  $r_p = 1.49$ . Figure 7b presents the detection results (discarded update parameters in gray). The results obtained are already sufficient for damage detection purpose and the analyses could have been stopped at iteration 2. The localization of the damage is obvious in the underside only of the north span and the quantification is valuable with up to -35% in stiffness. Nevertheless, a third iteration was performed in order to get the most accurate updated model suitable for eventual further analysis of the bridge. Figure 7c presents the results of iteration 3, where only update parameters of the north span and the vertical bearing of the south pier were kept and no additional target DOF were required ( $r_p = 1.93$ ) in order not to get a largely overdetermined problem. The results show that the remaining spring is unaffected, but the stiffness reduction in the north span is surprisingly not

as accurate as for iteration 2. Considering the fact that springs elements at boundary conditions have a huge sensitivity for the bridge modes, a final iteration was performed by only discarding the now useless spring parameter ( $r_p = 1.87$ ). Figure 7d presents the results that give a nearly perfect detection of the damage location and intensity.

### Results of other damage cases

Model updating damage detection analyses were performed on the three other damage cases presented in Figure 5 by using the proposed iterative procedure. Since the proposed methodology is systematic, the first model updating iteration always consisted in updating all 133 parameters by using 75 experimental target data (eight natural frequencies, 7 mode shape MAC and 60 individual DOF of mode 1) resulting in a ratio  $r_p = 1.77$ .

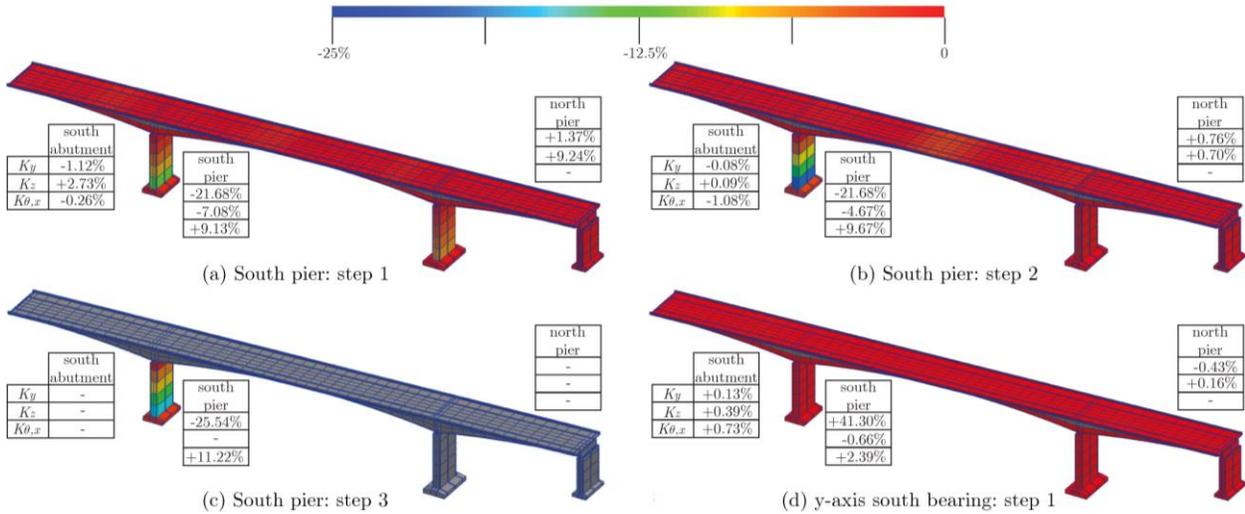


Figure 8. Damage detection results for cases: (a) to (c) south pier; (d) lateral y-axis south bearing.

Figure 8a to 8c present the three iteration analyses required to detect the damage case on the south pier. This case is particularly interesting in the fact that the authors had to introduce some sophistication in the method to solve it. Iteration 1 (Fig 8a) shows clearly that the damage comes from either the south or central piers or from some bearings. A new iteration using only these parameters and no additional DOF as required by the procedure led to inaccurate localization of the damage to the wrong pier. That is why in Figure 8b, iteration 2 reproduced the same analysis as iteration 1 ( $r_p = 1.77$ ) but by substituting additional DOF of mode 1 (flexural mode) by those of mode 2 (lateral mode). Such a substitution should indeed always be done when the first iteration shows that the whole deck is not involved in the damage, considering the fact that lateral modes are much more sensitive to piers stiffness than flexural modes. Results in Figure 8b provide now an accurate localization of the damage to the base of the south pier or to the lateral or rotational springs of the south bearing. The final iteration in Figure 8c was then performed with only one pier, two bearing springs and without additional target DOF ( $r_p = 0.53$ ), but it did not improve the results. It matters to note that even if the location of the damage was accurately identified, the accuracy of the damage intensity is not as good in this case. The algorithms had obviously some problem to differentiate the effects of the pier and those of the corresponding bearing. The overdetermination of the problem at iteration 3 ( $r_p = 0.53$ ) is another probable explanation.

Figure 8d presents the detection results for the damage case on the y-axis spring of the south bearing, which only required a single iteration to localize and to quantify perfectly the damage.

Damage to the bearings is in fact the most detectable type of damage because of the high sensitivity of the modes to the boundary conditions of the model.

Figure 9 presents the detection results of the combined damage case, involving damage to both the lower part of the central span and the rotation spring about  $x$ -axis of the south bearing. This combined case was studied to challenge the robustness of the proposed methodology when several parts of the bridge are damaged simultaneously. In addition, both applied damages have opposite effects on the main target responses (natural frequencies) of the model updating process. Also, to get an objective review of the proposed model updating procedure for damage detection, this case was developed by one of the authors, but the analysis was performed blindly by another one. After iteration 1 (Fig. 9a), only update parameters related to the central span and three spring elements were kept, and additional DOF of mode 1 from only the downstream side of the bridge were used for iteration 2 ( $r_p = 1.12$ ). Figure 9b shows that two iterations were enough to identify both damages accurately (location and intensity), hence validating the procedure for bridge structures.

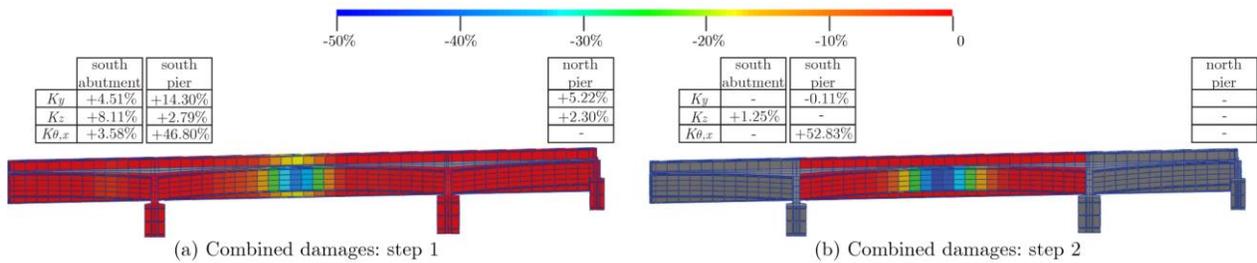


Figure 9. Damage detection results for the combined damage case (central span and  $x$ -axis rotation of the south bearing)

## CONCLUSIONS

This paper presented a damage detection study based on the case study of the Rivière-aux-Mulets Bridge located on Highway 15 in Ste-Adèle north-west of Montreal (QC, Canada). The commercial software FEMtools was used to perform FE model updating analyses to detect simulated damage cases by using a detailed 3D FE element model of the bridge. Such a study results in a largely underdetermined inverse problem where the number of update parameters is much higher than the number of target natural frequencies and mode shapes available from experimental modal analyses.

Four numerically-simulated damage cases to either deck regions, piers or bearings were considered by modifying the stiffness of the elements of the reference 3D FE model, which was originally updated on experimental modal analysis results. The effects of these damage on the vibration frequencies and mode shapes were studied. It appeared that the mode shapes are very poor indices for local damage and that severe damage are not necessarily linked to high evolutions of the vibration frequencies. The basic application of the FE model updating algorithms did not provide good estimates of the simulated damages. Thus, an improved procedure was proposed, which uses the default values of all FEMtools weighting parameters and that has three main elements: (i) the addition of selected individual DOF from an adequately selected mode shape to control the unknowns-to-equations ratio of the inverse problem within a recommended range from 1.0 to 2.0; (2) iterative analyses in which update parameters are progressively discarded if they are not modified in the previous iteration; (3) the prevention of the algorithms from increasing the stiffness of concrete elements. This practical and systematic methodology was illustrated in details with the damage case on the north span of the bridge and

results of the other cases were more briefly presented. All results obtained showed good accuracy in localizing and quantifying the simulated damage in only two or three iterations. The damage to the pier was the most difficult case to identify, because the model updating algorithms seem to be confused by the high sensitivity of the spring elements of the corresponding bearing. On the contrary and for the same reason, the damage case performed on a bearing spring element was the easiest case, requiring no iteration. The robustness of the proposed procedure was finally blind-tested by the authors with a combined damages case, where opposite modifications to both a span stiffness and a spring element stiffness were applied. Even in this case the model updating procedure showed very good performance to localize and to quantify both damages.

Real damage cases that include experimental uncertainties on the target modal properties were not discussed in this paper. Such studies would obviously require additional improvements in the proposed methodology, because individual DOF of mode shapes always contain significant experimental errors. Different approaches have to be studied for this problem, including the adequate use of the available weighting factors or the use of convergence data from the model updating process as a source of evidences to trust the results obtained or not.

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