# SAFETY EVALUATION OF STEEL BRIDGES BY MEANS OF AFFORDABLE MODELING OF CONNECTIONS

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## 1. ABSTRACT

The purpose of this work is to develop a practical methodology to detect potential structural safety problems on steel truss bridges before they could become fatal. Current design and maintenance software use Finite Element Analysis (FEA) in the form of beam elements connected by fixed joints. Nevertheless, in the case under consideration in this study, the collapse of the I-35W Bridge in Minnesota on August 1st 2007, the investigation only revealed the cause of the collapse after highly detailed and computationally intensive modeling of the joints using solid elements [1]. The main goal here proposed is therefore to develop accurate but computationally affordable connection models to improve global analysis and thus allow bridge owners to predict the effects of buckling in the gusset plates, joint deterioration, design deficiencies and to guide the requirements for structural monitoring. These models will propose a method of analysis which wants to be an improvement on what is currently used (such as the perfectly rigid connections) but not as onerous or precise as detailed model developed by means of finite element codes, which are way too difficult for routine design. Specifically, the purpose will be to reproduce the characteristics of the connections into non linear range, up to failure.

## 2. INTRODUCTION

The August 1st, 2007, catastrophic collapse of the I-35 W Bridge in Minnesota, United States, under ordinary traffic and construction loads, was triggered by a design flaw that had remained undetected for 40 years. It took very intensive Finite Element Analysis (FEA) to prove that the cause was the buckling of an undersized gusset plate [1]. Constructed in 1967, Bridge 9340 was designed in the early days of computer structural analysis, and did not include load-path redundancy. Connections (gusset plates) were designed with hand formulas that verified strength across critical sections, and were subsequently assumed to be stronger than the structural members they connected. The truss bridge was designed, most likely by matrix linear structural analysis, as an assemblage of rigidly connected beams. The bridge was inspected every two years, as recommended by the FHWA, [2], and was load tested in 2000. It is worth nothing that an inspector had actually photographed gusset plate U10 because of its bowed- out appearance (Fig.1), but did not judge that to be alarming [3].

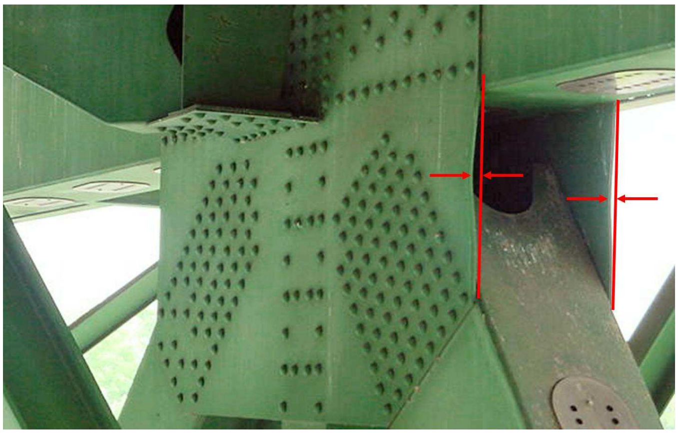


*Fig. 1: Picture of U10 W connection taken on June 2003 [1]*

The NTSB used massive computing to model the bridge and several connections with finite elements, but the Guidance issued by FHWA for the load rating of gusset plates relied very much on hand calculation methods, [2].

The literature review [3] provides information on the strength and the design of gusset plates but no guidance on the actual load displacement behavior of the connections. Both the Eurocodes and the American Codes are missing a way to assess the stiffness of those particular connections [4,5]. Current design and maintenance software use FEA in the form of beam elements connected by fixed joints. Yet, the investigation, made by FHWA and NTSB, only revealed the cause of the collapse after highly detailed and computationally intensive modeling of the joints using solid elements.

This work aims at filling the gap between advanced computing noted that can be brought to bear on a failure investigation, but is too expensive for routine design, and design methods that rely on highly approximate hand calculations to dimension gusset plates and then assume they behave rigidly in a global analysis.

The idea is to properly account for the behavior of connections in the global analysis with the use of equivalent springs. If a bridge contains similar joints, the same set of equivalent springs, suitably modified to account, for example, for changes in thickness of the gusset plates, can be used repeatedly. Moreover, computational savings can be gained in repeated load cases, as each analysis run makes use of the simplified connection. This technique can also be used for verifying the load rating of existing bridges and identify potential locations in need of strengthening. The method is therefore to perform nonlinear analyses on detailed joint model in order to assess the characteristics of non linear springs that will replace the gusset plate in a global model of the bridge and produce significant savings in computational effort at the cost of little loss of accuracy.

## 3. STRATEGY OF ANALYSIS OF SUBSTRUCTURING METHOD

From the literature review [3] it is seen that there are simple design methods based on equilibrium and elastic behavior and proven safe by experiments. There is, however, no simple way of calculating the actual behavior of a gusset plate, even in the elastic range.

Designers ensure that the connections are stronger than the members and then proceed with a structural analysis that assumes rigid connections. Such a structural analysis is incapable of predicting connection failure, or account for the flexibility of the connection in the global behavior of the structure. At the other extreme of structural analysis are detailed models such as the one analyzed by the National Transportation Safety Board (NTSB) as a result of the collapse of the I-35 W Bridge [1]. Forensic investigation had already pinpointed and preliminary analysis confirmed that the trigger of the collapse was the buckling of the undersized joint U10. So there was justification in performing a detailed FE analysis of joint U10 to replicate the collapse. In the present work, the NTSB detailed FE model (formulated in software Abaqus [6]) of gusset plate U10 is taken in consideration in order to establish the equivalent stiffness of springs that completely model the behaviour of the connection. The FE model has 5 stub members attached to a pair of gusset plates (Fig. 2 left side), and that model is connected to the appropriate members in the global model. For the simplified connection model, the stub members and gusset plates are replaced by 5 user-defined structural elements, called springs for short, that can each have up to a full 6 x 6 stiffness matrix for all 6 degrees of freedom (DoFs), Fig. 2 (right side). To establish the flexibility of the equivalent spring for member 1 for example, the ends of members 2 to 5 are fixed and a unit force is applied in order to obtain the displacements and rotations at the end of member 1. The repetition of the application of a unit force or a moment corresponding to all 6 DoFs produces the 6 x 6 flexibility matrix for member 1 in global coordinates (XYZ). This flexibility matrix is inverted to obtain the stiffness matrix, which is then transformed to local coordinates (123) and applied to the simplified spring model (Fig. 2 right side).

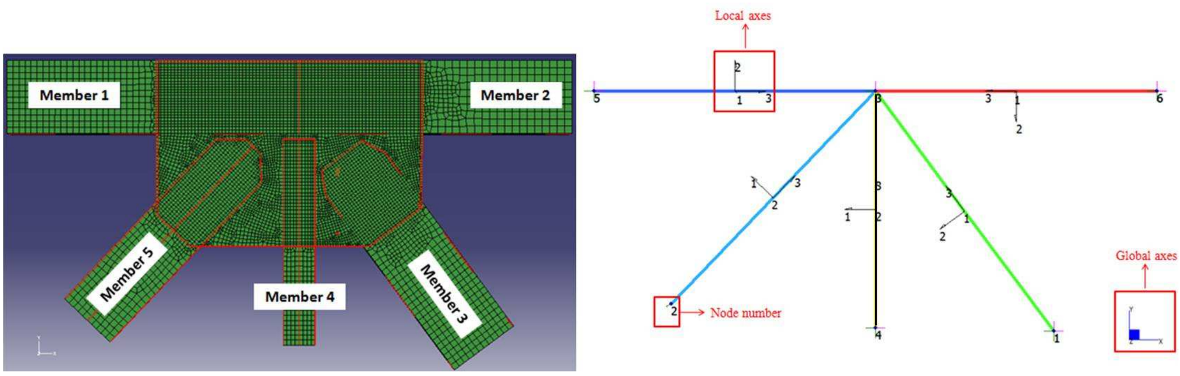



*Fig.2: Detailed FE model of gusset plate and spring element*

The spring stiffnesses are determined from FEA by means of finite element codes Abaqus [6] and Straus7 (Strand7) [7] and once they are installed in the global model, they can simulate the actual behavior of the structure. The results will not be as accurate as NTSB, but the cost will also be much less, especially if many load cases need to be run, and if the connection can be generalized to other locations. On the other hand, results will be more accurate than those obtained with rigid connection, linear analysis, as is currently done.

## 4. GLOBAL ANALYSES

The various connection models were placed then in a two-dimensional model of the I-35 W Bridge (Fig. 3), at a location corresponding to the U10 gusset plate that triggered the bridge collapse. The model was subjected to its own dead load, a uniform deck load of

74.92 kN/m, and a concentrated construction load near U10 of 115.7 kN, [8]. Liao et al. [9] showed by influence lines that the temporary construction loads placed near U10 significantly affected the forces imposed on it and may have triggered the collapse.

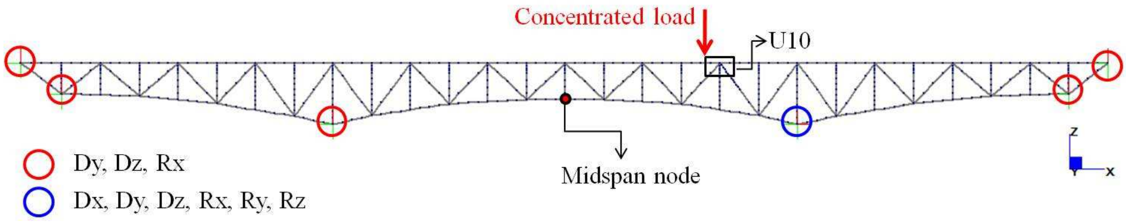



*Fig. 3: Two-dimensional model of I-35 W showing support conditions (restrained DoFs)*

### 4.1 Linear analyses

Four cases, shown in Fig. 4, were run, [8]:

1. All joints rigid;
2. U10 modeled with user-defined elements, all other joints rigid;
3. U10 modeled with detailed FE, all other joints rigid; and
4. All 5 member joints hinged and all other joints rigid.

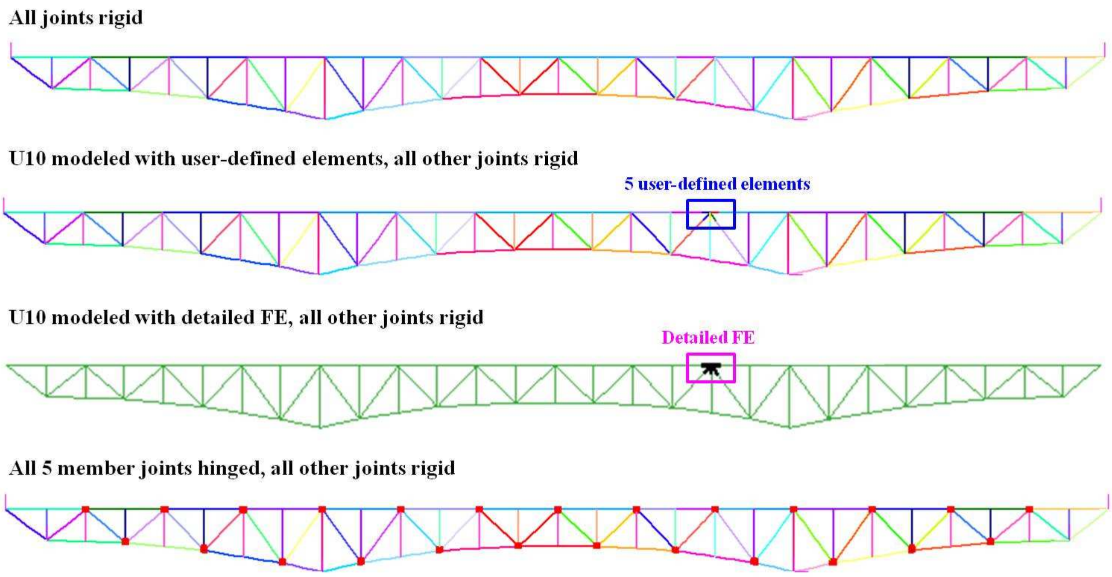


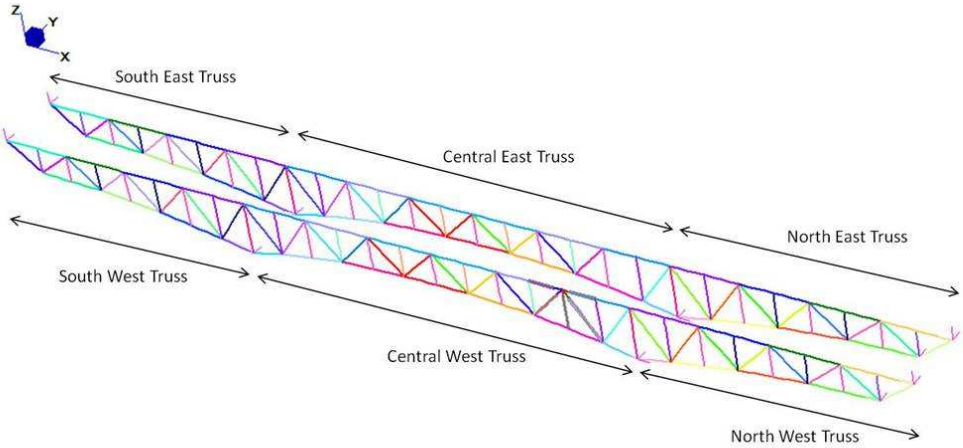

*Fig. 4: Four different 2D models of the bridge*

Results (Table 1) show that modifying the stiffness of one connection within the elastic range produces no noticeable effect on the maximum vertical deflection of the bridge (at midspan).

*Table 1: Midspan displacements for the four different models*

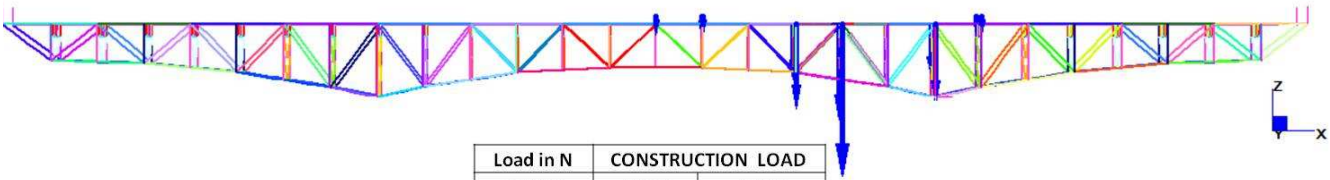
| Connection models | All Rigid | Rigid + 1 set of springs | Rigid + 1 detailed FE model | All 5-member joints hinged |
|---|---|---|---|---|
| (m) | -0.281 | -0.285 | -0.2845 | -0.286 |

The equivalent spring model produces a good approximation of the behavior of a gusset plate connection in the elastic range.
In order to run nonlinear analyses the three-dimensional model was considered; the introduction of the lateral bracing members was made to prevent the global buckling of the truss chord. The loads applied are shown in Figs. 5 and 6.




| Load in N/m | TRAFFIC LOAD | | STEEL LOAD | | CONCRETE LOAD | |
|---|---|---|---|---|---|---|
| ***TRUSS*** | *WEST* | *EAST* | *WEST* | *EAST* | *WEST* | *EAST* |
| *SOUTH* | 6183.77 | 3010.88 | 17931.14 | 17931.14 | 111390.27 | 111390.27 |
| *CENTRAL* | 8278.88 | 1571.38 | 17931.14 | 17931.14 | 111390.27 | 111390.27 |
| *NORTH* | 9099.75 | 2157.85 | 17931.14 | 17931.14 | 111390.27 | 111390.27 |

*Fig. 5: Traffic load*




| Load in N | CONSTRUCTION LOAD | |
|---|---|---|
| ***TRUSS*** | *WEST* | *EAST* |
| *SOUTH* | 7506.34 | 5838.26 |
| | 7506.34 | 5838.26 |
| | 1105494.25 | 663296.55 |
| | 2130697.50 | 1278418.50 |
| | 1025203.88 | 615122.33 |
| NORTH | 2502.11 | 1946.09 |

*Fig. 6: Concentrated load for the construction load*

Two different models are taken account:

1. Rigid joints, 5 members around the U10-W elasto-plastic and the others elastic;
2. Semi-rigid joint, elasto-plastic connection elements (only the diagonal terms of the stiffness matrix) and the 5 members around the U10W joint elasto-plastic.

### 4.2 Nonlinear analyses

The nonlinear behavior of the 5-member connections is now represented by means of 5 nonlinear elements using the method similar to the linear model [10]. There are two essential differences, though:

1. The behavior must be represented by a set of load-displacement or moment-rotation points calculated from the detailed FEM (using Abaqus);
2. The stiffness matrix of each nonlinear element is diagonal only, in order to use the connection elements available in Straus7 (Strand7). It is nevertheless expected that this simplification will introduce some errors compared to the detailed FEM.

Nonlinear material behavior is modeled using the Von Mises yield criterion and the isotropic hardening rule, with $\sigma_y$=355MPa, $\sigma_u$=611MPa, E=1.99E+11 Pa and $\varepsilon_u$=0.118.

A large strain-large displacement formulation, which is the default option for Abaqus, is used to carry out the nonlinear analysis. Fig. 7 shows the load-displacement and moment-rotation curves of each connection element, where XYZ refers to the global coordinate system.

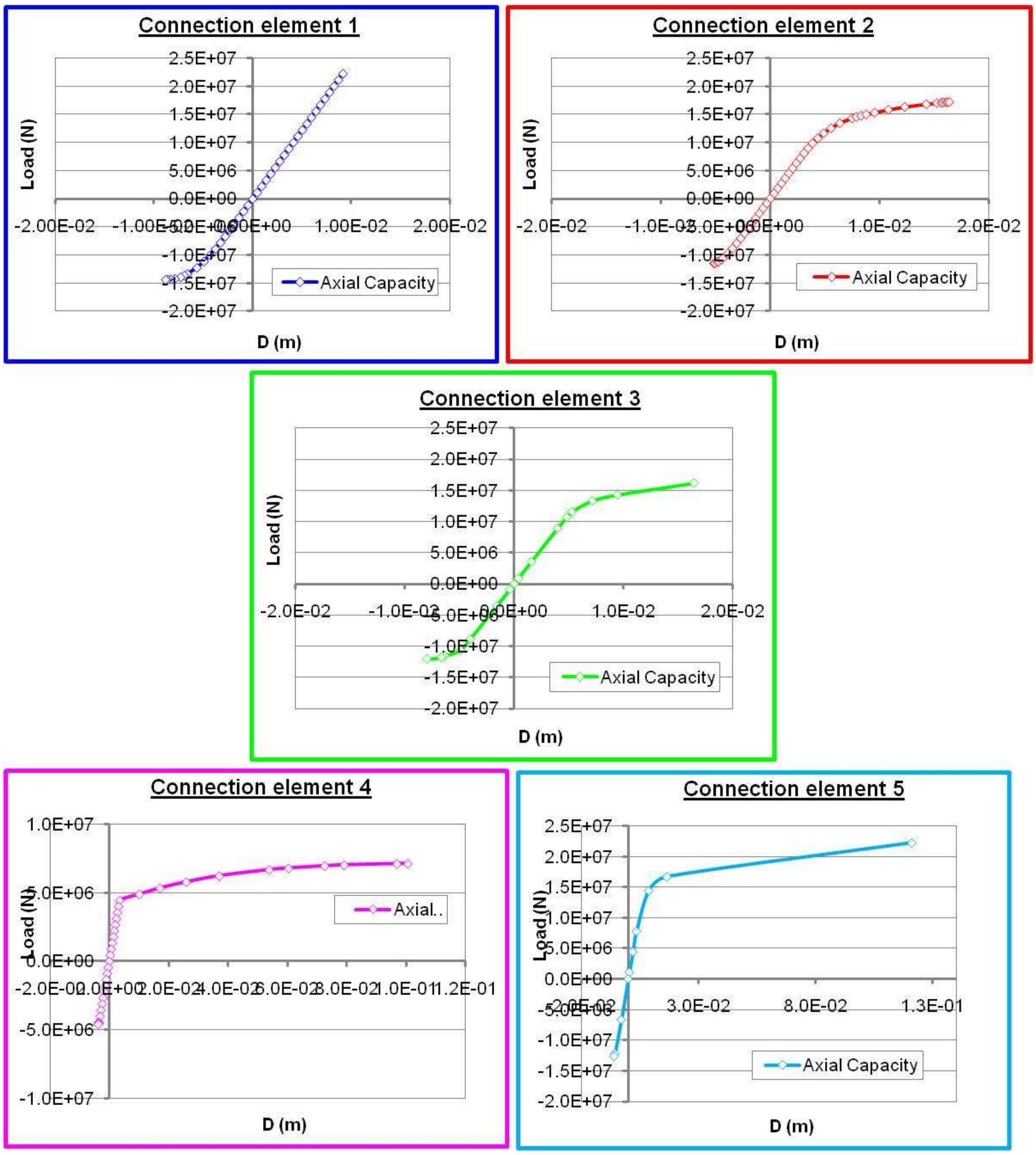



*Fig. 7: Tension and compression axial capacity in the five connection elements*

In Fig. 8 it is possible to assess how the semi-rigid connection provides a much better description of the actual behavior. Whereas the rigid joint model predicts member failure at a load factor of 6.36, the semi rigid joint model predicts that U10 begins to fail at load factor 0.92, and completely fails at load factor 1.7, leading to the collapse of the bridge.

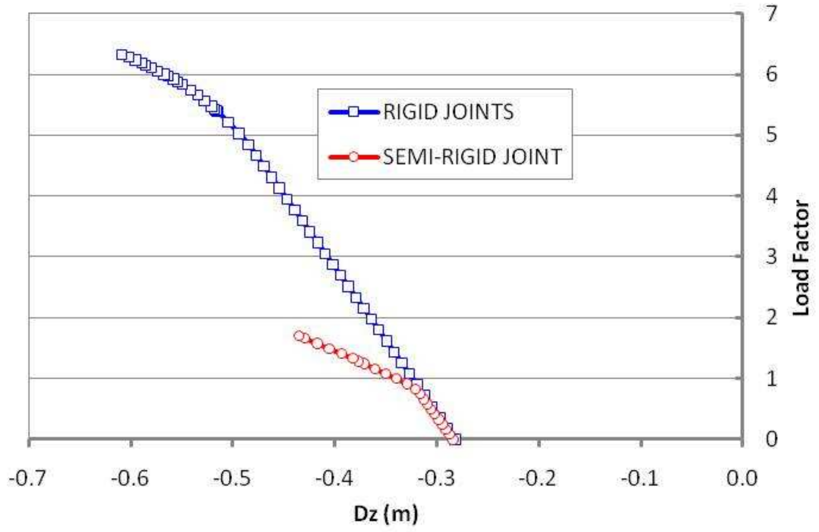



*Fig. 8: Load Factor-Vertical Displacement at the node in the midspan.*

In Fig. 9 the trend of the axial forces in the 5 connection elements modeling the joint U10-W is shown. This figure shows that collapse starts because of the achievement of the axial capacity in compression of the connection element 3, which is actually the main reason for the real collapse.
On the left side of Fig. 9 shows the deformation in the detailed model in Abaqus with scale factor of 10. This figure refers to the ultimate compression load the model can carry. It is obvious to see that the failure occurs because of the large displacements on the free edge of the gusset plate. This particular deformed shape could be made in comparison with the Fig. 1 where it easy to see the initial deformation that the gusset plate had before the collapse.

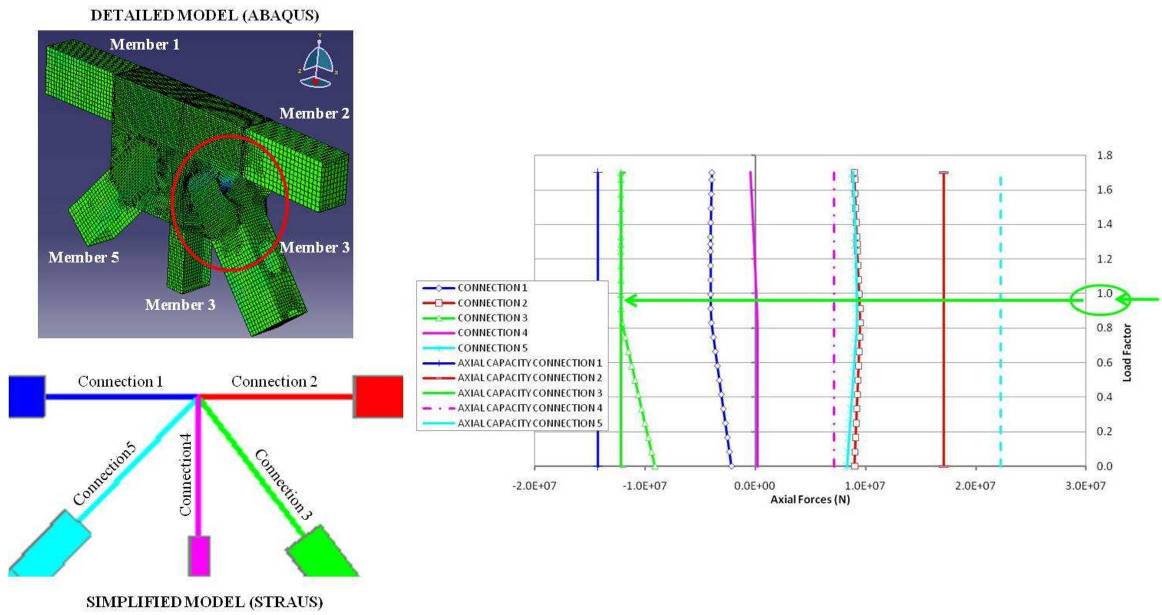



*Fig. 9: On the left the detailed and simplified model used to model the joint, on the right the trend of the axial force of each connection elements modeling the join under study*

## 5. CONCLUSION

From the nonlinear analyses on the 3 dimensional model of the bridge it has been possible to reproduce the collapse of the bridge. The correct modeling of the critical joint (U10-W) led the connection element (used to reproduce the gusset plate and the member) to fail that is just what happened on the real structure. After that, the west truss lost stiffness and resistance and after a while the local failure led the whole structure to fail.

All of that has been possible to achieve because of the correct way to model the connection, in fact if this would not have been done, modeling the joint as rigid for example, the ultimate load factor would have been about four times bigger.

Therefore this way to proceed is not only faster and less complicated to deal with but also sufficiently accurate to reproduce the real behavior of the structure under study.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