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MODELLING OF THE NONLINEAR PERFORMANCE OF THE BEAM CONNECTION OF AN EXISTING STEEL BRIDGE

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Research area: *mechanics*

ABSTRACT

In the present article an analysis of the behaviour of a steel bridge girder connection is made. The appearance of cracks in the links between different beams has led to a great interest to researchers to perform various tests, which examine the joints and the current carrying capacity of the structure. In this article a study of the nonlinear behaviour of a beam connection is presented. It uses the methods of the nonlinear fracture mechanics. A comparison between the results from the different non-linear models and analysis are made, apart from that, conclusions of the obtained results are also made. Thus, the foundations for further future studies of the structure are laid.

1. Introduction

The present work is part of a research, made by the authors, on the existing steel bridge. The Storström bridge is a steel riveted structure. It consists of a two narrow road lanes and a rail track, which suggests extremely heavy traffic through the whole year. The

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bridge has been put into service in 1937, which gives clarification to its small size and to the traffic for which it was designed. The facts, mentioned above, explain to some degree the appearance of the cracks, which are located in the joints. In a longitudinal direction the bridge is a compound beam, which indicates that if there is a deformation of a hinge caused by cracking, it will lead to a complete destruction of the structure.

In the present paper a nonlinear finite element modelling in ANSYS software is performed. To simulate crack growth the incorporated in ANSYS nonlinear fracture mechanics model is used. This model is developed for simulating crack opening in concrete structures. However in this paper the model is used only as a first approximation of the bridge joint behaviour. Only static load is applied without taking into account dynamic behaviour.

2. Nonlinear fracture mechanics model of Xu and Needleman

The cohesive zone model, which is incorporated in the ANSYS program, was developed by Xu and Needleman (1994) [1].This model is based on Hillerborg's idea of cohesive stress distribution in the crack tip. The main theoretical base of the model is as follows, the cohesive zone laws can be *uncoupled* or *coupled*. In an uncoupled cohesive zone law the normal traction is independent of the tangential opening. In a coupled cohesive zone law, both normal and tangential tractions depend on both the normal and the tangential opening displacement. Uncoupled laws are developed to be used when debonding process occurs under only one mode. The majority of cohesive zone laws have a (partial) coupling between normal and tangential direction, which is achieved by introducing coupling parameters in the model. Further information can be found in [1], [2], [4]. In the present paper uncoupled law is used, thus neglecting the tangential component.

3. Numerical modelling

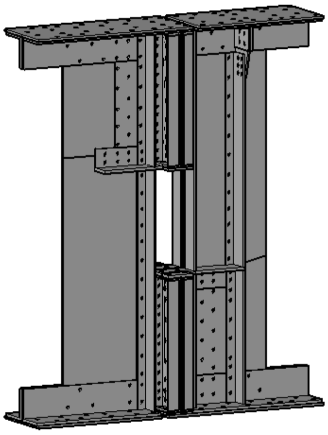


Fig. 1. 3D model with cohesive elements

Originally the geometry was designed for Tekla Structures, but after some processing it has been successfully imported in AutoCad. Then from AutoCad the coordinates of all the points that belong to a middle plane have been identified and then inserted into ANSYS.

With the use of the bridge drawings the coordinates and the radii of the rivet holes have been determined. The model, which is incorporated in the ANSYS program, is shown in fig. 1.

Main material data is: Young's modulus E = 195000 N/mm^2; fracture energy G = 70 N/mm; yield stress σ_y = 352 N/mm^2 [3]. The shear stress is not included. For this model SOLID185 and INTER205 finite elements from the ANSYS library are used [3]. Exponential law for traction-crack opening is used. SOLID185 is a prismatic volume element with 8 nodes and 24 DOF. INTER205 is an area element with 8 nodes and 24 DOF. The interface cohesive crack element works together only with SOLID185. Every stiffener and L-profile in this model are presented as a separate plate in the finite element model. All of the rivet holes are modelled according to the bridge drawings. The connections between the plates, the stiffeners and the L-profiles are made only by the rivet holes by placing a couple DOF. The hinge connection is also modelled by using a couple DOF, but in this case the couple is only in vertical direction. The existing cracks are placed in the middle web plate. In the previous work [4] the crack elements were placed on the full length of the middle web plate, while in the present paper the crack opening elements are placed only on the observed crack path.

Two different solutions are presented. In the first solution the model has a fixed support in the places where it is cut from the main beams, while in the second one spring constants are calculated and then used for a spring support.

For the calculation of the spring constants the bridge geometry in longitudinal direction, which is presented in fig. 2, is used (see [3]).

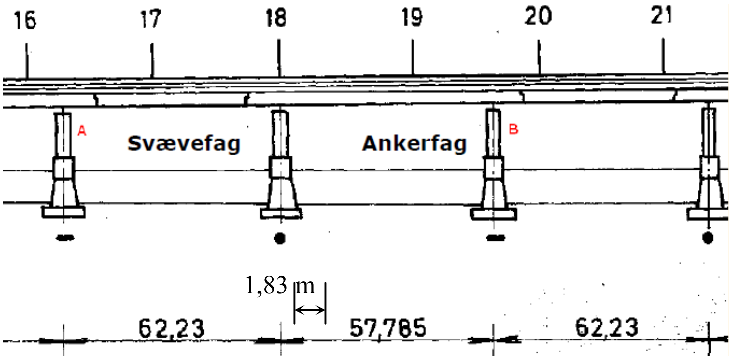


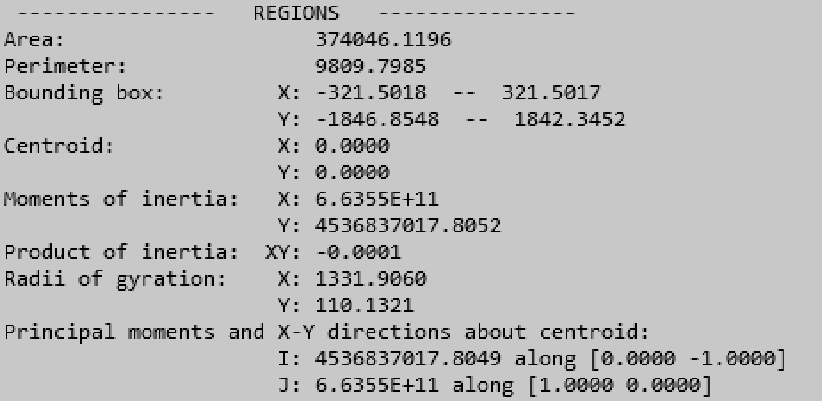
Fig. 2. Longitudinal geometry of the bridge

From the bridge design drawings the cross section of the beams is taken and the geometry parameters are then calculated. The area, the moment of inertia and other geometry parameters are shown in fig. 3. The cross section is an I-section, it is used for both the main and the secondary beams.

In order to calculate the spring constant a two-dimenssional model is used. The spring constants are calculated for the hinge connection point. The results for the spring constants are presented in tab. 1:

Table 1. Spring constants

Sprigs constant	kN/m	N/mm
C_y=	1944338	1944338,124
C_x=	626865,8	626865,7986
C_m=	6134757	6134757013



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 ----------------   REGIONS   ----------------
Area:                    374046.1196
Perimeter:               9809.7985
Bounding box:         X: -321.5018  --  321.5017
                      Y: -1846.8548  --  1842.3452
Centroid:             X: 0.0000
                      Y: 0.0000
Moments of inertia:   X: 6.6355E+11
                      Y: 4536837017.8052
Product of inertia:  XY: -0.0001
Radii of gyration:    X: 1331.9060
                      Y: 110.1321
Principal moments and X-Y directions about centroid:
                      I: 4536837017.8049 along [0.0000 -1.0000]
                      J: 6.6355E+11 along [1.0000 0.0000]
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Fig. 3. Geometry parameters of the beams' cross section

4. Numerical results

In the previous work [4] it was observed a significant tangential separation in the crack line and at the end of the solution the displacement was up to 12 mm. These results have led us to perform new solutions, which are presented here. First of all, the cohesive elements are placed only on the line of the observed cracks. By doing so we expect to obtain a bigger displacement from the solution. In the figures below the results from the model are shown.

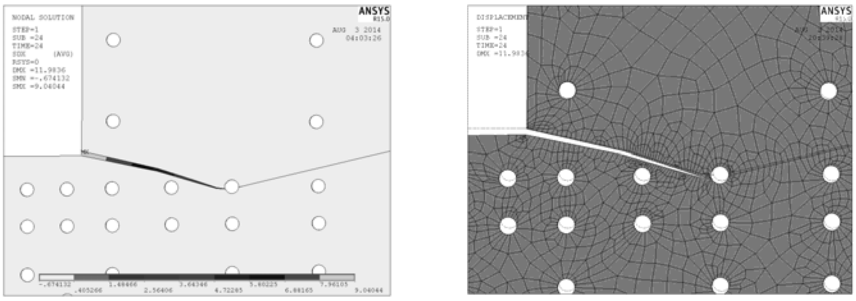

Fig. 4. Crack separation (left) and displaced shape (right) for the down crack

From the displacement shape shown in fig. 4 one may see that in the present solution there are no tangential deformations. This result shows that in the previous solution [4] the tangential displacement was a result of a fully opened crack along the whole middle web and it probably took place at the last stage of the solution. For the crack with the observed length, the tangential displacements are not significant.

One may see in fig. 6 that in the present solution the ultimate displacement is again 12 mm as in the previous one (presented in [4]). This result shows that the opening from the rivet hole to the end of the web happens very fast – indication of a brittle collapse of the bridge connection. The other result, which is shown in fig. 6 and in fig. 5 is the crack opening. In the present solution (fig. 5 and fig. 6 on the left) the crack opening is near 9 mm while in the previous one the crack opening is near 12 mm. This result shows us that the collapse is expected to occur somewhere in between 9 and 12 millimetres.

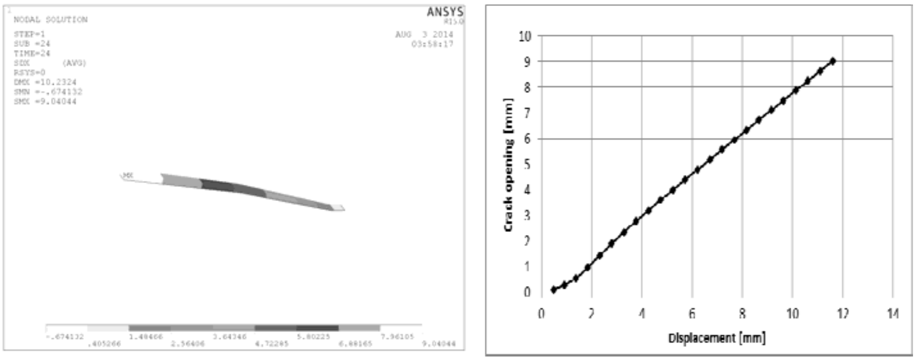

Fig. 5. Crack opening displacement of the down crack (left). Crack mouth opening vs displacement as a load (right)

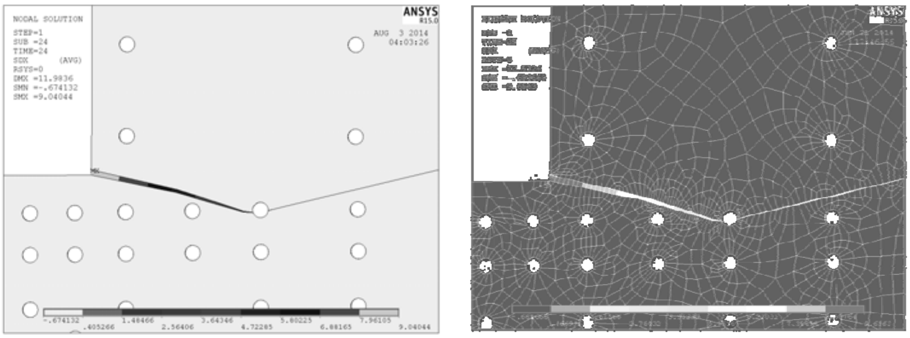

Fig. 6. Solution with cohesive elements up to the rivet hole (left). Solution with cohesive elements on the full length – previous solution [4] (right)

The next solution is done by the use of springs as it was explained earlier. This solution has the aim to take into account the beam stiffness over the beam connection. The spring constants are shown in tab. 1 from the previous section. The applied load on the bridge connection is again a displacement with a downward direction of 40 mm. The springs made the solution time consuming and difficult for convergence. The last converged load step is for 8 mm displacement. One of the reasons for the non-converging solution is the immediate loss of stiffness of the beam joint or brittle failure. Another possibility is the numerical problems with the non-linear solution. Additional analysis is needed.

In fig. 7 the crack opening and springs displacements are shown. One may see in fig. 7 that the crack is fully opened when the displacement has reached 8,11 mm. In the previous solution with the cohesive elements up to the rivet hole 9 mm displacement was reached. This result shows, as expected, the loss of stiffness in the beam connection.

In fig. 8 the force-displacement curve and the crack opening over the displacement curve are shown. One may see in fig. 8 that in the first step of the solution a displacement of 3 mm is reached. The reason for this is the additional springs in the model. In the next two steps appear large displacements, after that the solution continues with small steps. The same result for the crack opening is shown on the right in fig. 8. It is seen also that in the first three steps most of the displacement and the crack opening are occurring. This means that the displacement approaches its ultimate value.

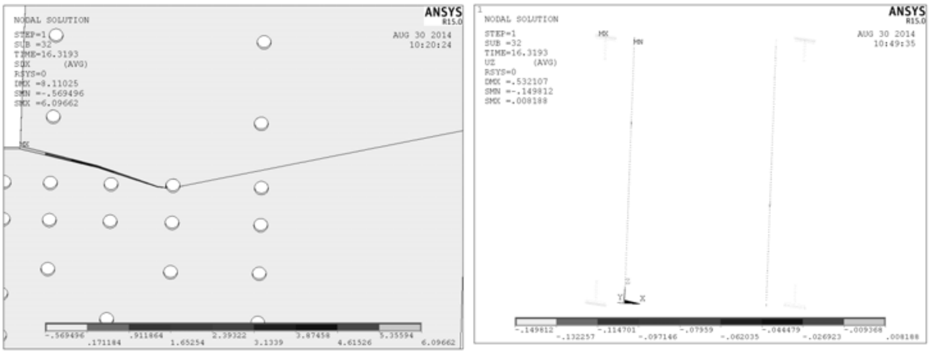

Fig. 7. Crack opening (right) and displacement of the spring supports (left)

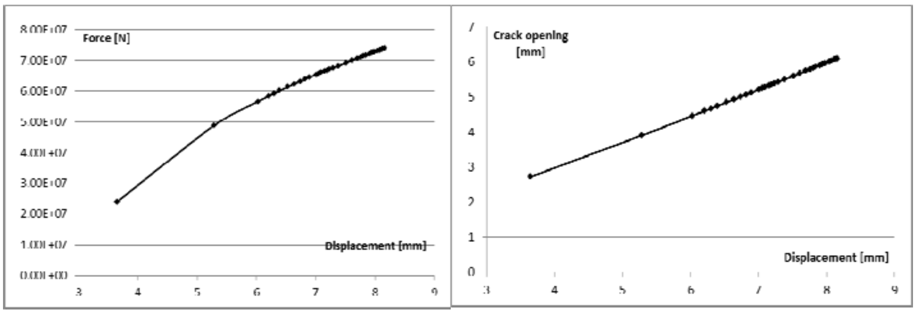

Fig. 8. Force-displacement (left) and crack-opening-displacement (right) curves

In fig. 9 a comparison between the solutions including springs and the solution from the previous research (see [4]) is done. By a grey line the previous solution is shown and by the black line the spring solution is shown.

One may see in fig. 9 the difference of the initial stiffness between the two solutions. The reason for this is the presence of springs. After the third step the two solutions are practically the same. This result shows again that in the initial steps the springs have significant influence and after that the cohesive elements begin to work. The reason why the spring solution crashes at some point is that the cohesive elements are only on part of the web.

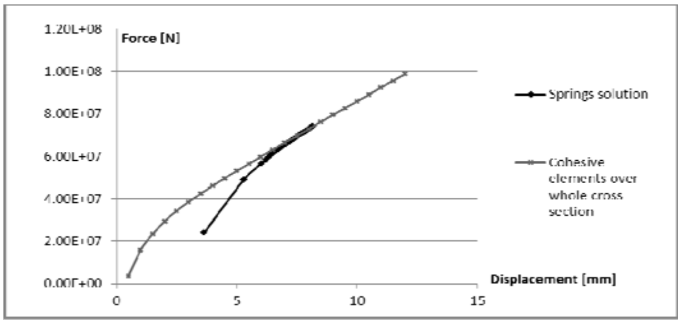

Fig. 9 Force-displacement curve for the spring solution and solution with cohesive elements over the whole cross section (see [4])

The presented results show that the stiffness of the whole beam has no significant influence on the ultimate load of the beam connection. As the solution including springs is more time consuming, it is not necessary to include springs for a further analysis.

5. Concluding remarks

- Shear stresses and sliding fracture mode are significant only for the fracture stage;
- The solution including spring supports is not significantly different and does not give any additional information for further analysis of the beam connection.
- Additional solutions taking into account the fatigue are needed.

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МОДЕЛИРАНЕ НА НЕЛИНЕЙНОТО ПОВЕДЕНИЕ НА ГРЕДОВА ВРЪЗКА НА СТОМЕНЕН СЪЩЕСТВУВАЩ МОСТ

И. Керелезова[1], Л. Малезанов[2], B. Laustsen[3], M. P. Nielsen[4]

Ключови думи: *механика на разрушението, умора, формула за развитие на пукнатина*

Научна област: *механика*

РЕЗЮМЕ

В представената статия е извършен анализ на поведението на съществуващ стоманен мост и по-точно на гредовата връзка на моста. Появата на пукнатини във връзките между отделните греди предизвиква множество изследователи да започнат разнообразни изследвания за причината за появата на съществуващите пукнатини както и за настоящата носимоспособност на съществуващата конструкция. В настоящата статия е представено изследване на нелинейното поведение на гредовата връзка с методите на нелинейната механика на разрушението. Направено е сравнение на резултатите от различни нелинейни модели и са направени анализи и заключения на резултатите, получени от тях. Дадени са насоки за бъдещи изследвания на конструкцията.

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