Ultimate and residual strength assessments of intact and collision damaged columns of the Bjørnafjorden floating bridge

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ABSTRACT: The Norwegian Public Road Administration (NPRA) has initiated an ambitious project to replace the current ferry crossing solutions with floating bridges over the wide and deep Norwegian fjords. This will substantially reduce the travelling time among major cities in Norway. It is found that ship collisions represent major critical loads to the safety and integrity of floating bridges that governs the structural design. This paper investigates the ultimate and residual strength of columns in an initial design of the Bjørnafjorden floating bridge before and after ship collisions with an initial kinetic energy of 125 MJ and 150 MJ respectively corresponding to a 10,000-year event as estimated from risk assessment. The ultimate bending and torsional capacities are assessed. Three different approaches are adopted, (1) the nonlinear finite element method using LS-DYNA; (2) A semi-analytical code PULS based on energy methods; (3) DNV rules-based formulations. The results are compared and discussed.

1 INTRODUCTION

The Norwegian Public Road Administration has initiated an ambitious project to replace the current ferry crossing solutions over the Norwegian fjords with floating bridges to connect major cities in Norway. The fjords along E39 roads can be up to 5-6 km wide and 1300 m deep. This is extremely challenging for traditional fixed foundation bridges whereby floating bridges become promising (Moan and Eidem, 2020). A number of floating bridge concepts are proposed, and the end anchored, curved semi-submersible type floating bridge is chosen for crossing the Bjørnafjord as shown in Fig. 1.

Fig. 1. Artist's impression of a proposed floating bridge for the Bjørnafjord, from NPRA (2016)

Floating bridges across navigable waters are exposed to the risk of being collided by large commercial and passenger ships and small vessels (Sha et al., 2021). Ship collision loads are identified as one of the major critical loads to the safety and integrity of floating bridges that govern the structural design (Sha et al., 2019). Design of bridges against ship collisions according to NPRA-N400 (2017) requires that bridge structures shall be able to resist a collision energy of a 10,000-year event without progressive collapse in Accidental Limit States (ALS). Risk analysis studies by Moan and Jin (2023) have demonstrated the need to introduce additional "ULS type" requirements for the bridge to limit the road traffic disruption, especially due to the occurrence of relatively frequent low energy impacts, especially on the pontoons. The low energy impacts may also induce fracture and flooding of bridge pontoons and lead to increased risk of bridge downtime, which is an important design parameter in the evaluation of accident consequences.

After ship collisions, it is required in the bridge design handbook (NPRA-N400, 2017) that the structures shall maintain sufficient residual strength to be able to resist environmental loads of a 100-year event with safety factors of 1.0.

This paper evaluates the ultimate and residual strengths of intact and collision damaged columns of the Bjørnafjorden floating bridge with finite element methods, PULS simulation and DNV rule formulations. The collision event considered corresponds to the ALS design requirement of a 10,000-year event. The results are compared and discussed.

2 FINITE ELEMENT MODELING OF THE FLOATING BRIDGE COLUMN

2.1 *Finite element models*

A finite element model of the floating bridge column was established for ultimate strength analysis as shown in Fig. 2. The model is used for an initial design calculation and the dimensions do not represent the final design of the Bjørnafjorden floating bridge columns.

The bridge column is 41.86 m high, which connects the bridge girder at its top and the floating pontoon at the bottom. The interface with the pontoon is a square of 8 m × 8 m at the bottom of the column and a square of 9.6 m × 9.6 m at the top of the column. The middle part of the column measures 7.6 m × 7.6 m with chamfered corners. The outer plate shell thickness is 25 mm in general and is increased to 30 mm in the region close to the top as is marked in blue in Fig. 1. The plate thickness at the connection of the top-middle section is 40 mm. The plates are reinforced with flat bar stiffeners of 400 × 20 mm with a stiffener spacing of 0.6 m. The stiffeners are supported by transverse girders of T1000×10×300×20 mm. The girder spacing is 1.45 m in the transition region (blue part in Fig. 2) and is 2.90 m otherwise.

The four-node Belytschko-Lin-Tsay shell element is used. The shell element size is in general 100 mm. This yields generally four elements for the stiffener web, which is considered sufficient to develop buckling modes.

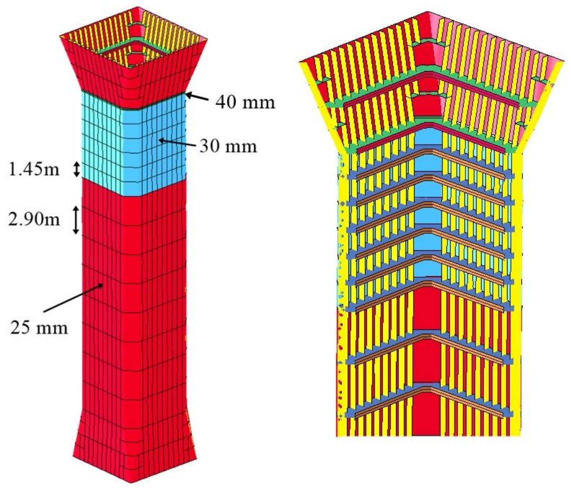

Fig. 2. Finite element models of the floating bridge column

2.2 *Material properties*

The power law hardening model with a yield plateau is used to model the steel material behaviors together with the BWH (Bressan-Williams-Hill) instability criterion (Alsos et al., 2008) for fracture. The BWH criterion considers that fracture occurs at the onset of local necking instability neglecting the post-necking regime, and this is conservative for structural safety (Yu and Amdahl, 2018). A high strength steel with a yield stress of 420 MPa is used for the entire column structure and the detailed material properties are given in Table 1.

Table 1. Properties of the steel material

Young modulus (MPa)	2.10×10^5
Yield stress (MPa)	420
Poisson ratio	0.3
Power law K (MPa)	860
Power law n	0.13
$\varepsilon_{plateau}$	0.012

2.3 *Boundary conditions and imperfections*

To check bending capacities of the column combined with shear, two types of boundary conditions are considered at the column top. The first boundary condition BC1 is by fixing the nodes at the column top and BC2 is by fixing all the nodes in the transition region from the column top to the main column with uniform cross sections, see Fig. 3. This is to check if the designed structures in the transition region can potentially be the weak link of the structure.

Ultimate strengths of marine structures are sensitive to initial imperfections. In the column FE model, sinusoidal shaped imperfections are introduced using the keyword *PERTURBATION with 5 half waves between transverse frames and 1 halfwave in the circumferential direction. The magnitude of the imperfection is taken as 0.25% of the member length.

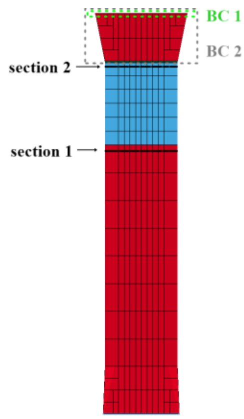

Fig. 3. Boundary conditions of the bridge column for ultimate strength analysis

3 PULS METHOD AND DNV RULES FOR BUCKLING

3.1 *PULS for ultimate strength analysis*

PULS (Panel Ultimate Limit States) (PULS, 2006) is a computational buckling code for thin-walled plate constructions. It assesses elastic buckling stresses and ultimate load bearing capacities under combined loads for stiffened and unstiffened plates used as building blocks in larger plated constructions such as ships and offshore constructions. The PULS buckling models apply the non-linear large deflection plate theory of Marguerre and von

Karman. Discretization of the buckling displacements follows the Rayleigh-Ritz method using Fourier series expansions across the plate and stiffener surfaces. Energy principles are used for establishing the algebraic non-linear equilibrium equations and incremental perturbation techniques are used for solving the equations.

For the column buckling assessment, one side of the column with 9 bays of stiffened panels was selected. The structural dimensions and material properties follow those described in section 2. The boundary conditions on the edges are that the edges are free to move in but shall remain straight. Imperfection shapes of the structures in PULS follow that of the first eigenmode from an eigenvalue analysis and the magnitude is according DNV rules (DNV-RP-C201, 2010).

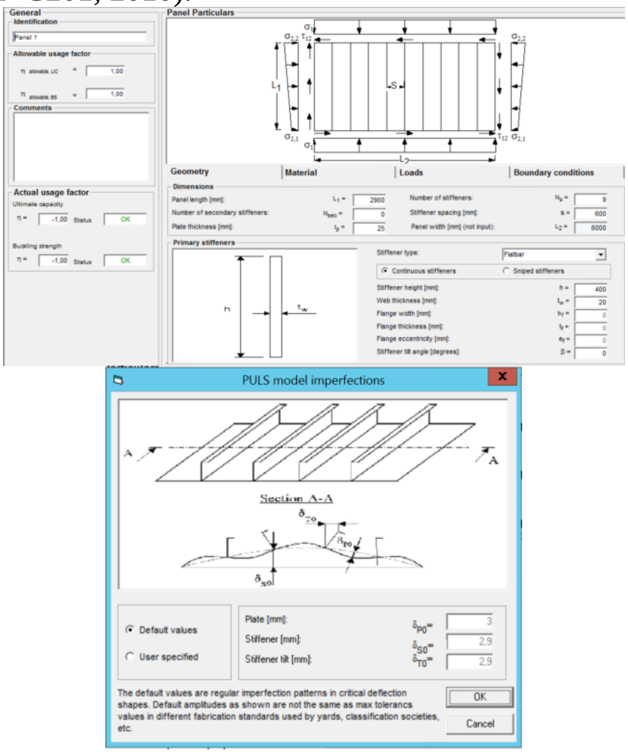

Fig. 4. PULS setup for buckling analysis

3.2 *DNV rules for design against buckling*

Relevant DNV rules for the buckling design of stiffened panels are DNV-RP-C201 (2010), DNVGL-CG-0128 (2015) and DNV-CG-0128 (2021). They provide somewhat different formulations for ultimate strength check of stiffened panels under combined loading.

4 ULTIMATE STRENGTH OF INTACT BRIDGE COLUMNS

4.1 *Ultimate strength under combined bending and shear*

- LS-DYNA results

In LS-DYNA FE analysis, the bottom end is given a low prescribed sideways velocity of 0.15 m/s to produce the loading condition of combined bending and shear. The resulting bending moments at cross section 1 and 2 are plotted in Fig. 5 with the two different boundary conditions. The buckling patterns are shown in Fig. 6.

It is found that the structural design at the connection between the uniform column and the column top structure is critical and governs the ultimate strength. Buckling occurs at the connection region, which represents a weak link of the column. If the whole top structure is fixed (i.e., boundary condition 2 with infinitely strong connection), the ultimate bending moment can increase from 950 MNm for BC1 to about 1270 MNm for BC2. The cross-section elastic section modulus is estimated to be 2.632 m^3. Assuming always elastic material, this gives an ultimate stress of 361MPa for BC1 and 482 MPa for BC2, which exceeds the material yield stress of 420 MPa.

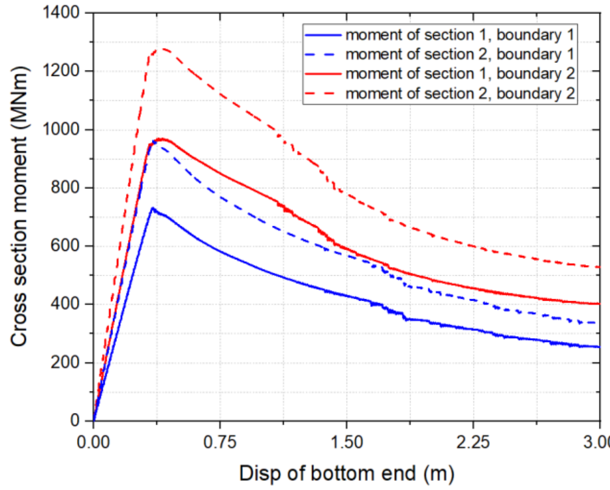

Fig. 5. Bending moment of column cross sections under combined bending and shear

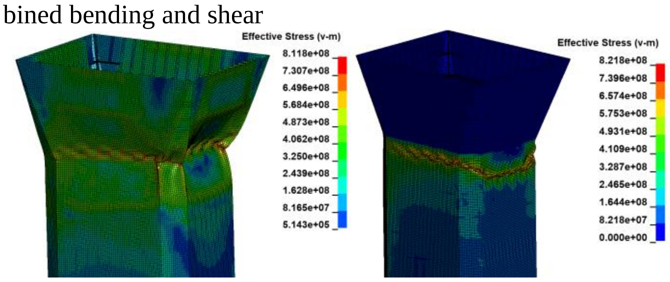

Fig. 6. Buckling modes of the column structure with different boundary conditions

- PULS and DNV rules results

The ultimate capacities for the panel with an outer shell thickness of 25 mm and a frame spacing of 2.9 m under combined compression and shear are examined using PULS and DNV rules of three different versions (DNV-CG-0128, 2021; DNV-RP-C201, 2010; DNVGL-CG-0128, 2015) with a safety factor of 1.15. The resulting interaction curves for the panel under biaxial loading are plotted in Fig. 7. Under axial compression without shear, the PULS simulation gives an ultimate strength of 376 MPa and the corresponding buckling pattern is shown in Fig. 8 with coupled local plate buckling and stiffener torsional buckling. The DNV rules predict generally consistent and conservative ultimate capacities compared with the PULS result.

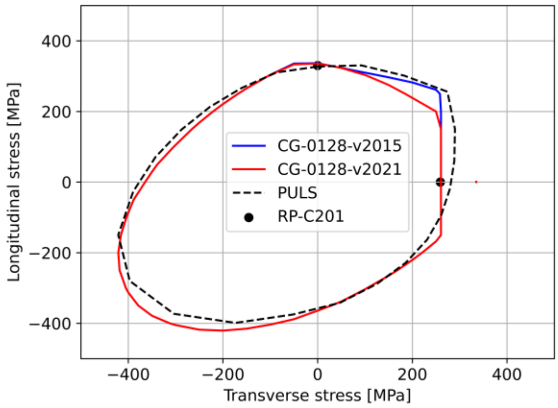

Fig. 7. Interaction curves for the ultimate strength of column stiffened panels under biaxial loading with a safety factor of 1.15.

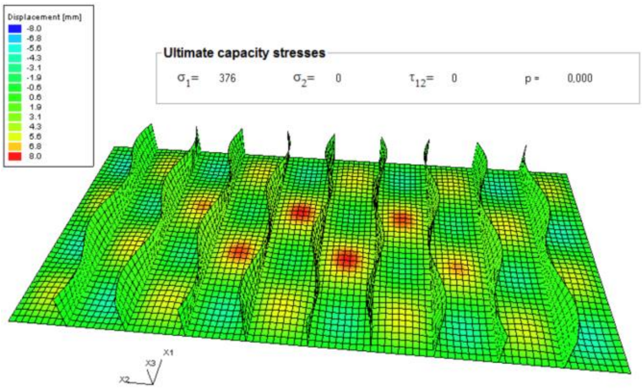

Fig. 8. Buckling of the column stiffened panel under axial compression using PULS.

4.2 *Ultimate strength in torsion*

In LS-DYNA FE analysis, the bottom end is given a prescribed rotational velocity of 0.15 rad/s about the column axis for torsional capacity check. The resulting torsional moments at cross section 1 and 2 are plotted in Fig. 9. The buckling patterns are shown in Fig. 10. Under torsion, torsional buckling occurs around the middle part of the column with a lower plate thickness of 25 mm. Torsional moment does not directly trigger elastic buckling of the column but material yielding at a moment of 676 MN. After that, considerable energy absorption capacity remains with increasing torsion before final plastic buckling collapse.

The stiffened panels capacities under combined axial loading and shear are checked using PULS and DNV RP C201 with a safety factor of 1.15. The resulting interaction curves are plotted in Fig. 11. In general, the results are consistent. Under pure shear, the ultimate shear stress can reach 210 MPa and this corresponds to a von mises stress of 363 MPa, which is close to the yield stress. The buckling pattern is shown in Fig. 12. The results show that the ultimate stress is close to the material yield stress in both compression and shear. The structure is in general compact against buckling.

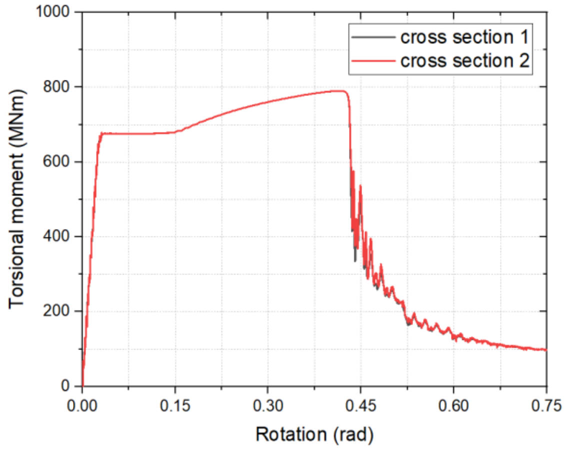

Fig. 9. Boundary conditions of the bridge column for ultimate strength analysis

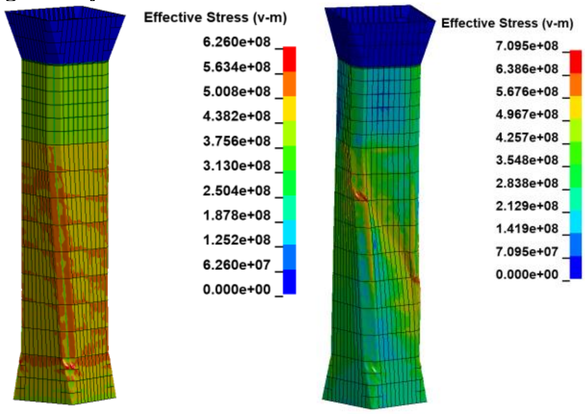

Fig. 10. Boundary conditions of the bridge column for ultimate strength analysis

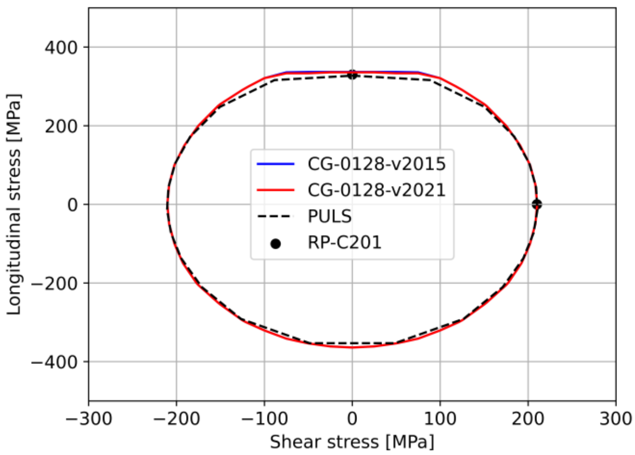

Fig. 11. Interaction curves for the ultimate strength of column stiffened panels under combined axial and shear loading with a safety factor of 1.15.

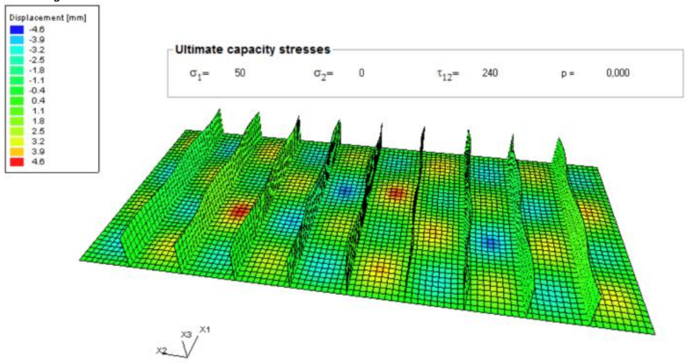

Fig. 12. Stiffened panel buckling under shear using PULS.

5 RESIDUAL STRENGTH OF COLLISION DAMAGED BRIDGE COLUMNS

5.1 *Ship collision damage*

Ship collision analysis is conducted in LS-DYNA with a global-local model for the floating bridge and nonlinear springs for the ship stiffness. The global-local bridge model adopts detailed shell elements for the pontoon-column system in the collision region and beam elements otherwise for the bridge. The striking vessel is a container ship with an overall length of 120-170m. Detailed description of the collision models and simulations can be found in Jin et al. (2021). The damaged bridge pontoons and columns are shown in Figs. 13 and 14, with two different initial impact energy of 125 MJ and 150 MJ, respectively.

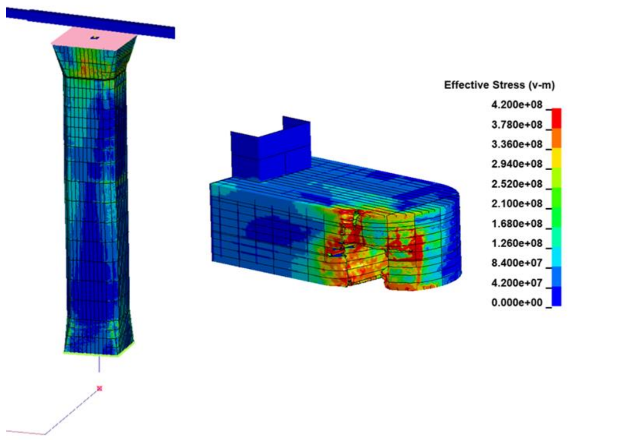

Fig. 13 Energy absorption of 36.65MJ by column deformation (left), 62.4 MJ by pontoon damage (right) for the 90-degree impact to pontoon A3 with an initial kinetic energy of 125 MJ.

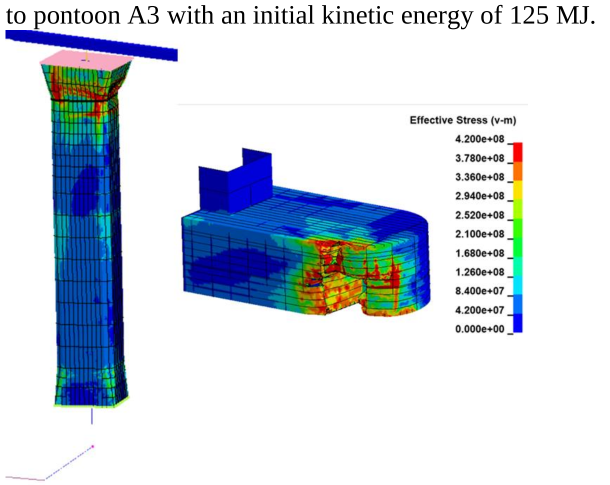

Fig. 14 Energy absorption of 74MJ by column deformation (left), 70 MJ by pontoon damage (right) for a 90-degree impact to pontoon A3 with a kinetic energy 150 MJ.

5.2 *Residual strength of collision damaged bridge columns*

A restart analysis with the ship collision damaged columns was carried out using LS-DYNA to check the residual strength in bending and torsion. The numerical settings follow those from Section 4. Fig. 15 plots the residual strength of the damaged bridge column under controlled lateral displacement. For the damaged column with an initial collision energy of 125 MJ, no fracture occurred on the column cross sections. The maximum bending capacity is calculated to be 700 MNm, which represents 26.3% reduction of the ultimate bending capacity of an intact column. For the damaged column with an initial collision energy of 150 MJ, large fracture was observed as shown in Fig. 14. This significantly reduces the residual ultimate bending capacity to 230 MNm for bending towards to deformed section and 332 MNm for bending normal to the deformed section, which represent 76% and 65% reduction.

Similar trends are also observed in the residual torsion capacity of damaged in columns in Fig. 16. Almost no reduction of column torsional capacity is observed in the case with an initial kinetic energy of 125 MJ, where no shell fracture was observed. The reduction is significant for the case with an initial kinetic energy of 150 MJ, where the outer shell is fractured. In that case, the crack propagates fast under torsion and the top section can be completely torn off as shown in Fig. 17.

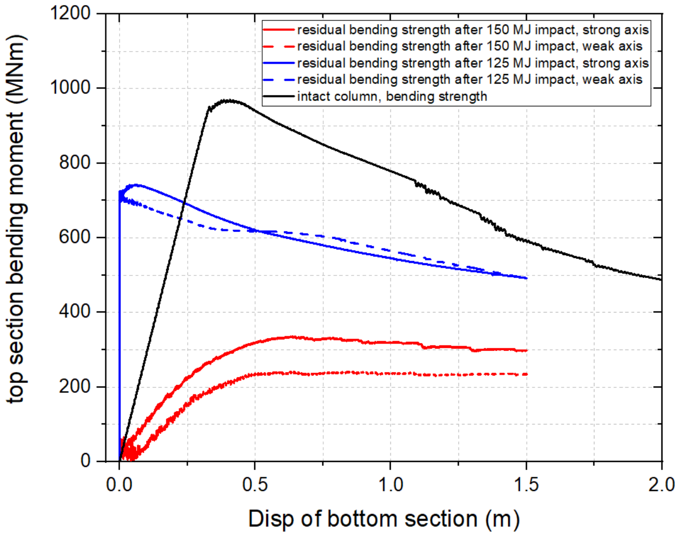

Fig. 15. Residual bending strength of the bridge column after ship collision damage.

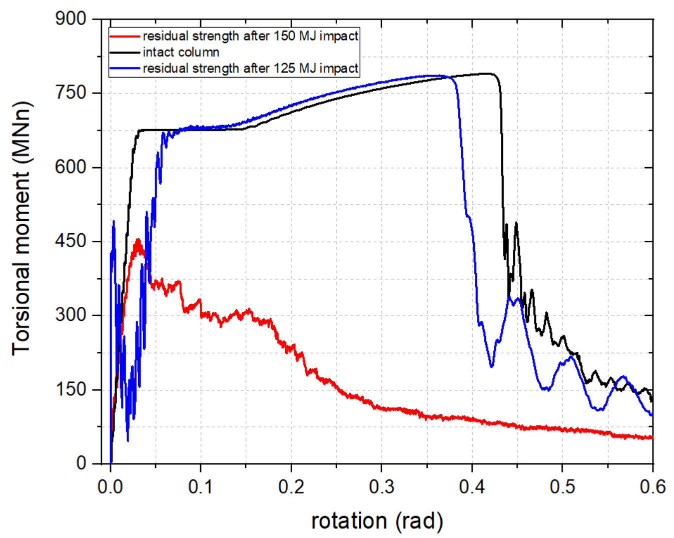

Fig. 16. Residual torsional strength of the bridge column after ship collision damage.

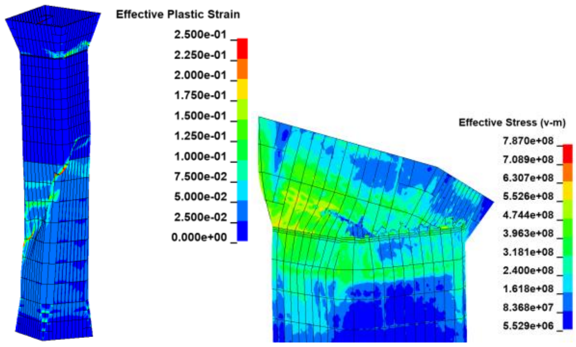

Fig. 17. Residual bending strength of the column after ship collision damage with (left) an initial kinetic energy 125 MJ. (Right) an initial kinetic energy 150 MJ

6 CONCLUSIONS

This paper conducted ultimate and residual strength assessments of columns of the bjørnafjorden floating bridge before and after ship collision damage. Three different approaches are used the nonlinear finite element methods, PULS analysis and DNV rules. The following conclusions are drawn:

1. The column structure is in general quite compact against buckling. Utilization of the ultimate stress with respect to material yield stress under different loading conditions is quite high. The connections between the column top and the uniform cross section represents a weak location and are recommended to be strengthened.

2. The three different approaches of NLFEA, PULS and DNV rules give in general quite consistent predictions of the ultimate strength for the present structure. DNV rules are on the conservative side.

3. Moderate reduction of the bending and torsional strength of the bridge column is observed after ship collisions with a total kinetic energy of 125 MJ. When the total collision energy is increased to 150 MJ, the reduction is significant due to the presence of fracture. The results are sensitive to the adopted fracture criterion.

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