MECHANICAL BEHAVIOR OF JOINT CONNECTION IN STEEL-CONCRETE HYBRID LANGER BRIDGE

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ABSTRACT

Steel-concrete hybrid PC Langer bridge, which consisted of RC arches, vertical steel tube hangers and PC stiffening girders, was constructed. Three dimensional numerical simulation of steel-concrete connection between the RC arch and the vertical steel tube hanger were conducted for the future establishment of verification technique of joint connection in hybrid structures using finite element method. The assembled finite element model was verified with sufficient accuracy for the mechanical behavior of the test specimens of the connection subjected to pull-out load in the steel tube. Furthermore, according to the result of numerical simulation with the partial model of joint connection in the actual bridge, it was numerically confirmed that the connection had a sufficient resistance to the pull-out load through the vertical steel tube.

KEYWORDS: less than 5 words

1. INTRODUCTION

The steel-concrete hybrid PC Langer bridge, which consisted of reinforced concrete (RC) arches, vertical steel tube hangers and prestressed concrete (PC) stiffening girders as shown in **Fig.1**, was constructed in Hokkaido, Japan. Many existing Langer bridges were constructed using single material such as steel or concrete, and no other examples of similar type of structures to the current target bridge exist. Three-dimensional frame analysis was conducted in the design, in which train load was applied as a moving load to the frame model of the bridge, and the component members of the bridge were designed for the computed sectional forces. Much attention was paid to the design of joint connection between steel tube hanger and RC arch, of which resistance to the pull-out force through the hanger to the arch was dominant in the design. Loading tests of model specimens of joint connection were also conducted to verify the design result.

Fig.1 Steel-concrete hybrid PC Langer bridge

The structural performance verification of the bridge was made according to the design standard for railway structures (RTRI 2007). Standard Specifications for Hybrid Structures was also published by the Japan Society of Civil Engineers (JSCE 2009), which describes an integrated performance verification method using finite element analysis (FEA) and nonlinear material constitutive laws. However, there are still few examples of verification of special joint connection design. Based on such background, three-dimensional FEA was conducted for loading test of model specimen and finite element modeling concept was verified. Furthermore, numerical investigations were also conducted on the influences of span length of RC arch on the mechanical behavior and failure pattern of the joint connection.

2. JOINT CONNECTION IN LANGER BRIDGE AND TEST SPECIMENS

2.1 Profile of the target bridge and its joint connection

The target Langer bridge, having 65m span length, consists of two RC arches and PC stiffening girders connected each other with fourteen steel tube hangers. Span-to-rise ratio is 1/5.8 and the rise is about 9.6m. The overall configuration of the bridge is shown in **Fig.2**. The detail of the arch member and the center joint connection are shown in **Fig.3**. The longitudinal and lateral reinforcing bars in the arch and the main bars in the strut are penetrated through the steel tube embedded into the arch. The bearing (anchor) plates and the stiffeners are also provided at the end of the steel tube. SM490YB (JIS) plate having 16mm thickness is used for the steel tube, and SM400A (JIS) plate with 20mm thickness is adopted for both the anchor plates and the stiffeners. In the RC arches and struts, SD345-D32 (JIS) and SD345-D16 (JIS) deformed bars are provided as longitudinal and lateral reinforcement. The longitudinal bars in the strut are anchored to the concrete inside the steel tube in the joint connection by the ϕ80mm anchor plates welded at the end of the bars. Design strength of concrete is 60N/mm^2 for arches and 40N/mm^2 for struts. Concerning the design of joint connection at the center of the bridge, the design pull-out force in the steel tube hanger and the design axial force in the RC arch are 949.7kN and 7477.3kN, respectively.

2.2 Test Specimens

The loading tests of joint connection specimens were conducted in order to verify the damage level of the connection under design load (Kawamura et al. 2007, Doi et al. 2009). The capacity of joint connection against pull-out force through the steel tube varies with the axial force level in the arch. Therefore, the applied axial force to the arch was selected as an experimental parameter, as 0.0 (No.1), 0.19 (No.2) and 0.3 (No.3) of the axial force ratios defined as a ratio of the applied axial force to the ultimate axial force of the arch, in consideration of the design axial force ratio of 0.19 in the actual bridge. As shown in **Fig.4**, the specimen with a span length of 2.2m, in which a shear span-to-depth ratio was lower than the actual bridge, was designed to fail in shear in the arch, instead of the failure of connection itself. The target of the specimen was the center connection in the bridge because the pull-out shear force in the arch was a critical parameter at the center compared to the other connections in the design. The compressive strength of concrete was 64.9 to 68.5N/mm^2 for three specimens, and the yield strength and Young's modulus of reinforcing bars were 430N/mm^2 and 196kN/mm^2 for D19 bars, and 432N/mm^2 and 197kN/mm^2 for D10 bars,

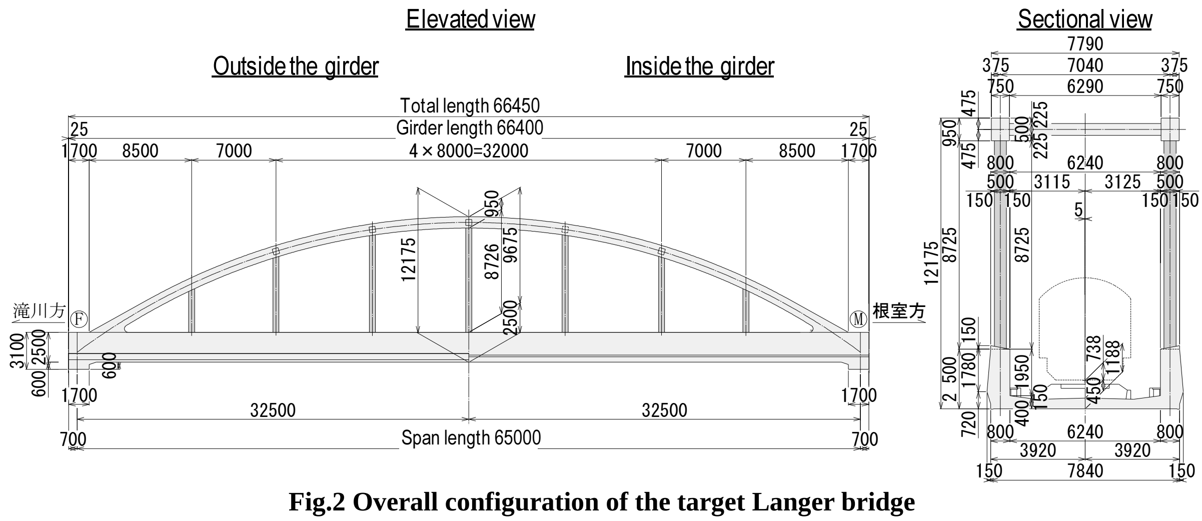

Fig.2 Overall configuration of the target Langer bridge

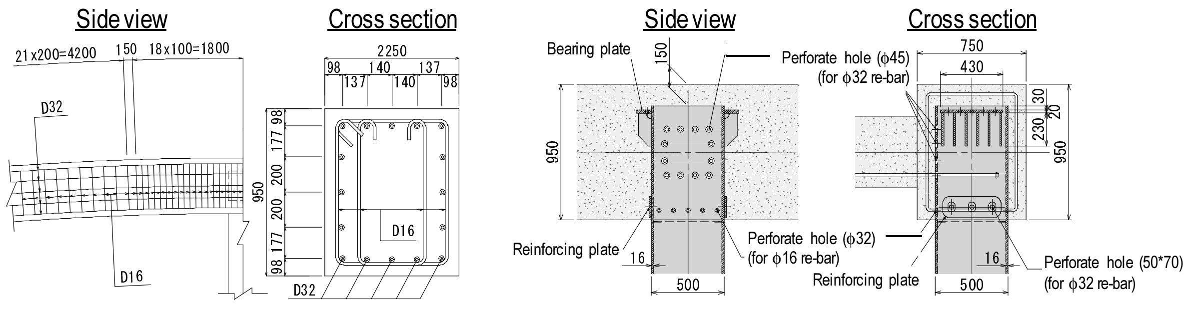

Fig.3 Detail of center joint connection

respectively. The left stub was fixed to the floor by PC tendons, whereas the right stub was rigidly fixed to the floor during the loading. Pull-out load was applied to the top end of steel tube under constant axial load to the left stub of the specimen. DAH-1 to DAH-5 in **Fig.4** indicate the numbers of displacement transducers.

2.3 Summary of Test Result

Fig.5 shows the crack diagrams of the specimens after loading. All the specimens failed in shear in the right span, due to the asymmetric fixity of the left and right stubs in the test. The angle of critical diagonal crack lowered with an increase of axial load level.

Fig.6 shows the relationships between the applied pull-out load and the arch displacement of the specimens. The maximum capacity was higher in the specimen under higher axial load level, as similar to the tendency observed in the ordinary RC members. The longitudinal reinforcements in all the specimens were yielded in tension only at a location just close to the steel tube, due to the influence of relative displacement of the tube to RC arch. Some of the lateral reinforcements were also yielded at the intersections of the bars and the critical diagonal crack that led to the diagonal shear failure in the shear span of the specimens.

3. FEA SIMULATION OF TEST SPECIMENS

3.1 Modeling

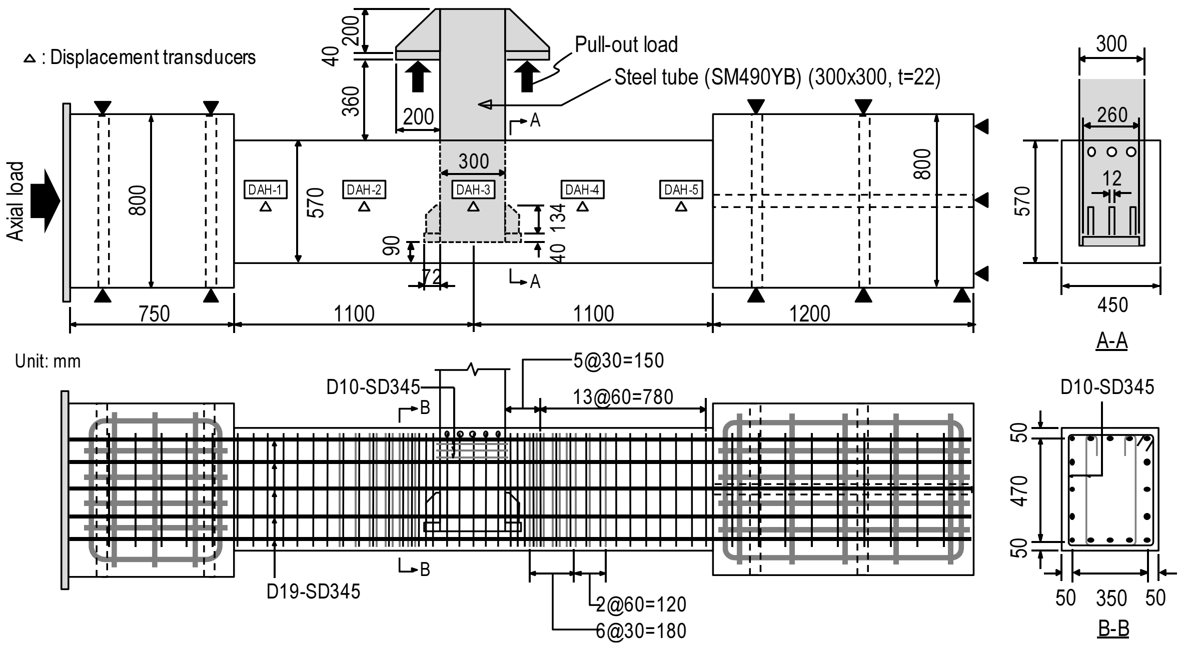

Fig.4 Dimensions and reinforcement arrangement of test specimen

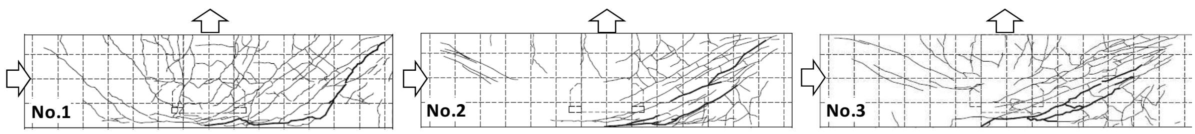

Fig.5 Crack diagrams after loading

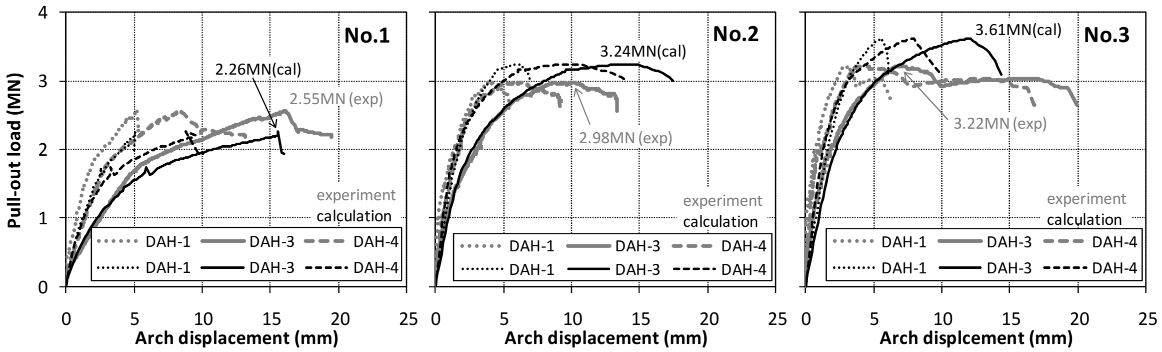

Fig.6 Relationships between pull-out load and arch displacement

Numerical simulations of the specimens tested in the previous section were conducted using the three-dimensional nonlinear finite element analysis program COM3 (Okamura et al. 1991, Maekawa et al. 2003), developed in the University of Tokyo, Japan. The finite element mesh used in the simulations is shown in **Fig.7**. The half of the specimen was modeled in consideration of the symmetry of the problem, and three-dimensional 20-node solid (brick) elements were used for both RC and steel parts.

The existing example of FEA to the hybrid structures (Hauke et al. 1999) was referred in the assembling of the model, based accurately on the reinforcement arrangement in the specimen. The effective zone of bond between concrete and deformed bar (An et al. 1997), as well as the element size in relation to the control volume of the applied constitutive laws (JSCE 1997), was carefully considered. The localized deformation of penetrated reinforcing bars through the perforated holes on the steel tube was considered by providing RC joint elements (Okamura et al. 1991) at the exact location between steel and concrete elements, and no shear slip resistance was assumed at the other portion of the steel tube surface.

The boundary conditions in the both stubs were adjusted to the test condition; i.e. the right stub was rigidly fixed whereas the left stub was fixed by two truss elements in which 5000μ of initial prestressing strain was introduced.

3.2 Material Models and Parameters

The nonlinear path-dependent constitutive law of reinforced concrete, proposed by Okamura and Maekawa (Okamura et al. 1991), was applied to the RC elements in the arch part. The applied model consists of multi-directional compression-tension-shear model of concrete and reinforcement, with non-orthogonal four-way fixed crack model based on the smeared crack assumption, and it has already been verified for nonlinear behavior of RC structures under various types and combinations of external loads as well as the seismic excitations. The values of material parameters shown in the previous section were used as input parameters to the constitutive laws. The post-peak relation in the stress-strain curve of concrete in the non-effective zone of bond with reinforcement was carefully defined in consideration of fracture energy of the material (An et al. 1997, El-Kashif et al. 2004, Nakamura et al. 2001). Note that such attention shall be paid in the modeling of steel-concrete hybrid structures because the element size highly

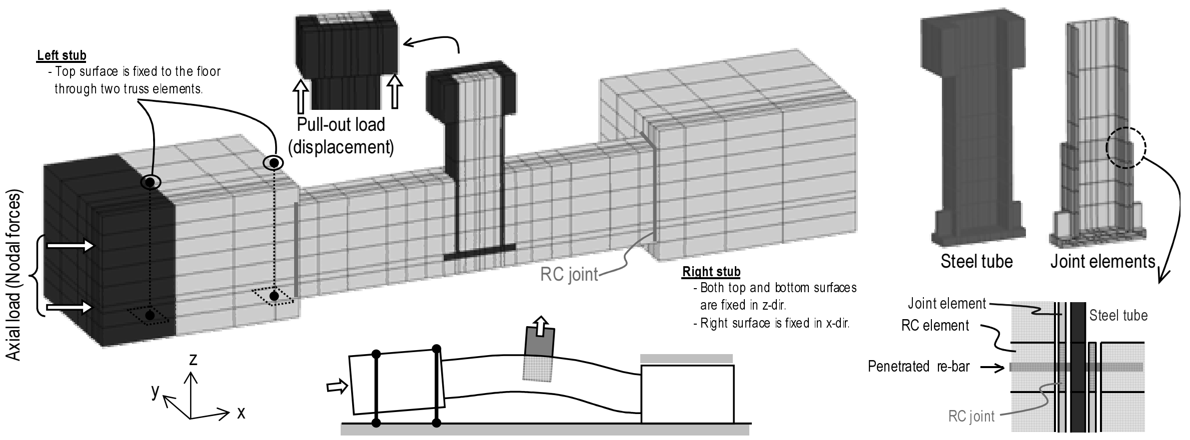

Fig.7 Finite element model used in the simulation of pull-out test

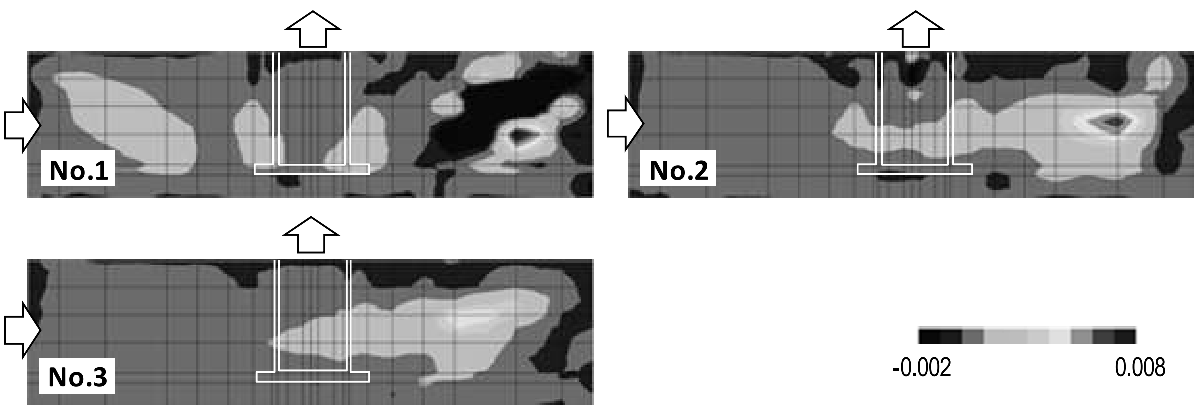

Fig.8 Vertical strain contour plots of the specimens

depends not only on the reinforcement arrangement but also the dimensions of steel members and elements.

3.3 Results and Discussions

The relationships between pull-out load and displacement obtained by the numerical analysis are shown in **Fig.6** together with the test results. Although the maximum pull-out load in No.2 and No.3 are slightly overestimated, numerical result of the deflection at the center of the specimen, as well as that at the point in the shear span, has a good agreement with the test results. In the vertical strain contour plots of the specimens at their maximum loads, shown in **Fig.8**, high strains with inclined distribution are observed in the right span rather than in the left span, as similar to the crack diagrams shown in **Fig.5**. Therefore, it is concluded that the proposed FE model can predict the damage and failure of the target joint connection due to the pull-out load in the steel tube.

4. FEA SIMULATION OF JOINT CONNECTION IN ACTUAL BRIDGE

4.1 Modeling

The numerical simulation of the pull-out behavior of center joint connection in the actual bridge shown in **Fig.2** was conducted with the FE model assembled with the same modeling policy stated in the previous section. According to the bending moment distribution diagram obtained by the preliminary frame analysis, the total span length of the arch in the partial model, as shown in **Fig.9**, was determined as 5m, and the both ends of the arch were simply supported at the mid height on the end surface. The effective depth in the joint connection was 751mm, given as the distance between the edge of the anchor plate and the bottom end of the arch. Consequently, the nominal shear span-to-depth ratio was 2.863. The actual arch member had a curved shape with 54m radius at its mid height; however, the arch was modeled as a straight member be-cause the span length was not so long.

Only the pull-out load was applied to the vertical steel tube because the bending moment and shear force in the steel tube under the design load were negligible. Because the Langer bridge is a statically-indeterminate structure, the sectional forces in the component members depend on the deformation level of the bridge. Especially, both the pull-out force in the steel tube and the axial force level in the arch are de-pendent on the applied load on the bridge deck. In order to consider this fact, the incremental pull-out load was applied to the steel tube, with the ratio of the applied axial load to the pull-out load kept constant

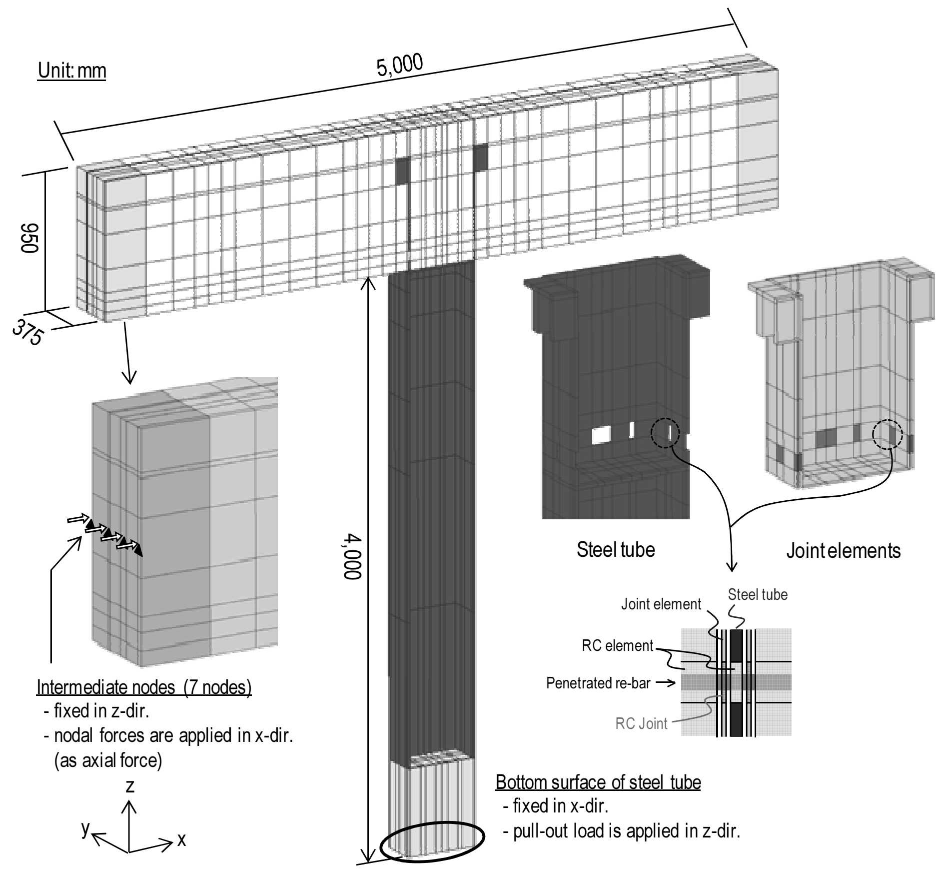

Fig.9 Finite element model used in the simulation of joint connection in the actual bridge

as 7.87, given as the ratio of design sectional forces (N_d/P_d =7.48/0.95=7.87, where, N_d and P_d are design axial force and design pull-out force, respectively.).

4.2 Results and Discussions

The overall deformation and the longitudinal strain contour plot are shown in **Fig.10**. Here, deformation is magnified by 10 times as large as the original deformation. No relative displacement of the steel tube to the arch (pull-out displacement) could be observed even at the maximum pull-out load (about 4 times as large as the design pull-out load), and the bottom end of the arch still remained in compression due to the high axial force in the arch. Because the localized high compressive strain occurred at the top end of the arch just above the steel tube, the ultimate state of the joint connection was governed by the flexural compression failure of the arch. **Fig.11** shows the pull-out load and displacement relation and the interaction curve of the arch section. P_{mu} is the calculated pull-out load equivalent to the ultimate bending moment of the arch section under the maximum axial force N'_{max} equal to $7.87P_{max}$. In the interaction curve, the bending moment was converted to the pull-out load through the shear span length. The fact that the maximum pull-out load in the simulation was reached P_{mu} proved the sufficient capacity in the joint so that it can transfer the pull-out force in the steel tube to the arch as bending moment. Furthermore, it was confirmed that the joint had a sufficient safety capacity against the design sectional forces used in the verification of the ultimate limit state in the practical design process.

5. SUMMARY

Numerical simulations using three-dimensional finite element method were conducted on the pull-out loading test specimens and the joint connection in the actually-constructed steel-concrete hybrid Langer bridge. The conclusions are summarized as follows:

(1) The assembled finite element model was verified for the pull-out loading test specimens under different axial loads, which gave the simulated result having a good agreement with the test results.
(2) The parametrical investigations on the influence of shear span-to-depth ratio on the damage and failure pattern of the joint connection were also conducted. Within the range of the parameters in this study, it was clarified that the joint with the nominal shear span-to-depth ratio over 1.0 exhibits the ductile flexural failure in the arch under the constant axial load.
(3) Based on the proposed modeling policy, the pull-out simulation of the joint connection in the actual hybr-id Langer bridge was conducted under the incremental axial load to the arch member. It was confirmed that the designed joint connection had a sufficient capacity to transfer the pull-out force in the steel tube hanger to the arch as bending moment.

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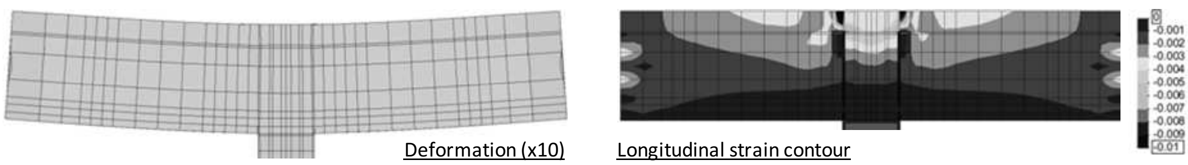

Fig.10 Deformation and longitudinal strain contour plot when the maximum load was reached

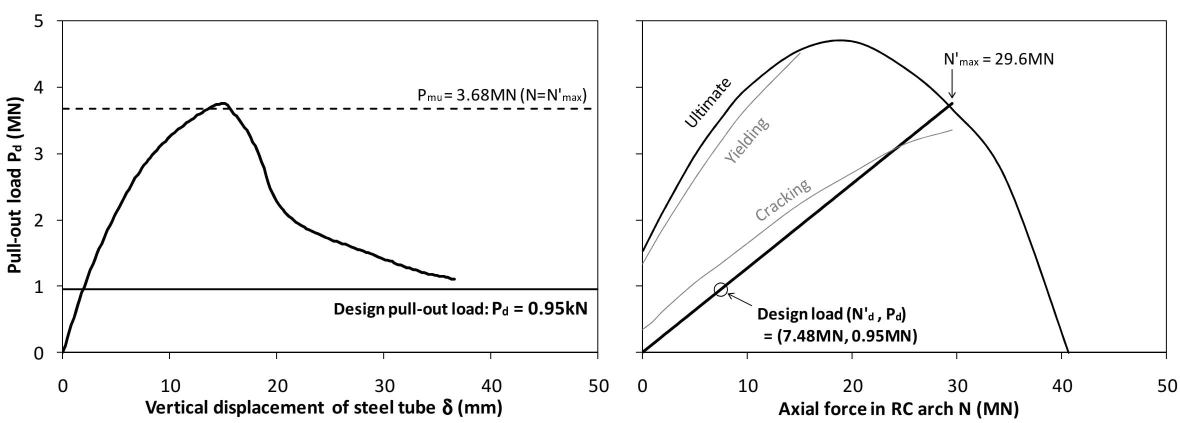

Fig.11 Pull-out load and displacement relationship and the interaction curve of arch section

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