ANALYTICAL STUDY ON REMAINING CAPACITY OF CORRODED GUSSET PLATE CONNECTION IN TRUSS BRIDGES

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Recently, numerous instances of severe corrosion damage to the gusset plate connections of steel truss bridges have been widely reported across the world. The corrosion of gusset plate connections has been confirmed to decrease load-carrying capacity, and it can lead to the collapse of the entire bridge. In this study, the remaining load-carrying capacity of a corroded gusset plate connection was evaluated using load testing and finite element method (FEM) analysis. Two different types of gusset plate corrosion were investigated: the corrosion loss of the lower chord flange-to-gusset weld and the corrosion loss of the gusset plate thickness. The loading tests and FEM analyses were conducted on an approximately half-scale model of a real bridge. Corrosion effects were evaluated for an assumed disconnection of 50% of the weld length, and for the loss of 50% and 75% of the gusset plate thickness in a selected region. This study then implemented parametric FEM analyses of the effect of the degree of corrosion on the remaining load-carrying capacity of the gusset plate connection. Finally, based on the results of the parametric FEM analysis in cases with corrosion loss of the gusset plate thickness, a method for evaluating the local buckling strength of the corroded section was proposed.

Key Words: steel truss bridge, corroded gusset plate connection, FEA, remaining load-carrying capacity

1. INTRODUCTION

The truss bridge is a popular type of bridge that primarily consists of a truss system of main members connected by gusset plates. The truss structure has been so widely used because of its light weight, large load-carrying capacity, and a high in-plane-stiffness. Steel truss bridges have been applied from early on in the development of modern bridges worldwide as their efficient use of materials enabled lower construction costs. However, the application of steel truss bridges to highway systems did not generally occur as early or as rapidly, nor were they as crucial as they were in railroad systems. Indeed, the truss
bridge is frequently associated with the development of railway bridge systems worldwide. According to statistics from the International Database for Civil and Structural Engineering (IDCSE)\(^1\), many existing steel truss bridges are considered “old” at ages from 50 to over 100 years, mostly having been built in the period before 1950, and between 1950 and 1970. Numerous 100-year-old truss bridges remain in service today\(^2\), \(^3\), and the number of steel truss bridges over 50 years old will increase significantly in the coming years. With this considerable increase in old steel truss bridges, severe damage due to corrosion has become a serious problem. Numerous studies have shown that in steel truss bridges, corrosion is frequently found on the gusset plates that connect members, particularly where the plate connects to the upper flange of the lower chord member. This corrosion is simply due to the complex shapes in this region, which readily accumulate debris and water. These locations are shown in Fig. 1.

The gusset plate connections of a truss bridge are considered to be structurally critical components of the truss structure system because they connect all main members. It has been confirmed that the reduction in the load-carrying capacity of a corroded gusset plate connection can lead to the collapse of the entire truss bridge. A typical example of this problem occurred in the USA in 2007, when the I-35W steel truss bridge\(^4\)-\(^6\) collapsed because of insufficient gusset plate thickness, resulting in a connection having a lower load-carrying capacity than necessary. Therefore, evaluation of the remaining load-carrying capacity of gusset plate connections, accounting for the corroded section of the gusset plate, has become a critical subject of research.

In this study, laboratory loading tests and FEM analyses were conducted using approximately half-scale models of real bridges on two different forms of corrosion of a critical gusset plate: the corrosion loss of the welded connection between the gusset plate and the upper flange of the lower chord member in the compressive direction, and the corrosion loss of the gusset plate thickness. Both the loading tests and the FEM analyses were implemented with the objective of determining the deformation performance, failure behavior, and the rate of reduction in the load-carrying capacity of the corroded gusset plate connection. Additionally, this study conducted parametric analyses of the effects of the size of the corroded sections on the gusset plate using FEM, in order to determine the relationship between the remaining capacity and the extent of dimensional reduction due to corrosion. Finally, based on the result of the parametric FEM analysis of cases with the corrosion loss of the gusset plate thickness, an evaluation method for determining the local buckling strength of the corroded section was proposed.

![Fig.1 Frequently corroded locations on steel truss bridges.](image1)

![Fig.2 Specimen geometry.](image2)
2. EXPERIMENTAL OVERVIEW

(1) Specimen shape

The gusset plate specimens used in this study were monolithic with the chord members: the projected web plates of the lower chord members were projected to serve as the gusset plates. The dimensions of all members in the models were approximately 50% the size of those on the subject bridges (the S-truss-bridge (1974) and the T-truss-bridge (1983) in JAPAN used bolted-connections), which were chosen because of the severe corrosion damage found on their gusset plate connections. The length, width, and thickness of the model gusset plate connection were 1200 mm, 216 mm, and 8 mm, respectively, as shown in Fig. 2. The angle between the lower chord member and the two diagonal members connecting into the gusset plate was 56 degrees. Further, because the axial force of the diagonal members was much larger than that of the vertical member in the subject bridges and the existing bridges, the influence of the vertical members was omitted in the specimens. The loss of the weld between the flange plate and the gusset due to corrosion was represented by introducing a disconnection between the gusset plate and the upper flange of the lower chord member in the compressive direction (see Fig. 2(a)). The corrosion loss of the gusset plate thickness was represented as a cross-sectional loss by cutting a groove (called the “Groove”) of height $h_z$ and width $t_z$ at the location where the gusset plate connected to the upper flange of the lower chord member (see Fig. 2(b)). The base metal used in this study was SS400 steel with a yield stress of 317 MPa and an elastic modulus of 200 GPa, determined from the mill sheet certificate and the tensile experiments of the SS400 steel in the laboratory.

Because of the limited capacity of the experimental equipment (3000 kN), the loading tests were conducted using two frames: a truss frame system and a link frame system with the size of the specimen used 50% that of the actual bridges, as shown in Fig. 3. The dimensions of the members of the two frames were designed such that they would operate within their elastic phase during the loading test process. The length and height of the truss frame was 4000 mm and 1500 mm, respectively, and 1790 mm and 2000 mm, respectively, for the link frame. The frame loading system was connected to the specimen using high-tension bolts through the connecting plates.

(2) Experimental parameters

The experimental parameters used in this study are shown in Table 1. With the basic aim of determining the deformation performance, failure behavior, and load-carrying capacity of the gusset plate connection, a loading test was first conducted on the connection without any simulated corrosion loss. Because the excessive loss of section in the welded connection in the compressive direction of the gusset plate could lead to large out-of-plane deformation, a loading test was then conducted on a specimen with one-half of the flange-to-gusset weld removed, as shown in Fig. 4(a). To account for the failure behavior of corroded gusset plate thickness, approximately 50% and 75% of the gusset plate thickness was assumed to be corroded (see Fig. 4(b)) in two additional specimens. The 50% and 75% corrosion levels were simulated by introducing a Groove in the gusset plate of $h_z = 25$ mm and $t_z = 4$ mm, and $h_z = 50$ mm and $t_z = 6$ mm, respectively.

In total, four specimens were tested: one control with no simulated corrosion loss (Specimen N), one simulating the loss of the flange-to-gusset weld (Specimen W), and specimens simulating small and large cross-sectional corrosion (Samples S and L).
Table 1 Experimental parameters.

| No. | Specimen | Corrosion level                        | Dimension of Groove section | Test frame |
|-----|----------|----------------------------------------|-----------------------------|------------|
| 1   | N        | Without corrosion                      | h_z (mm)                   | --         |
| 2   | W        | Welding corrosion                      | t_z (mm)                   | Link frame |
| 3   | S        | Small cross-sectional corrosion        |                            | Link frame |
| 4   | L        | Large cross-sectional corrosion        |                            | Truss frame|

Fig. 4 The specimens prior to the loading tests.

Fig. 5 Finite element analysis model.

(3) Loading method

The loading tests were conducted in the laboratory using the link frame system for the intact Specimen N, the welding loss Specimen W, and the small cross-sectional loss Specimen S; and using the truss frame system for the large cross-sectional loss Specimen L.

Shimadzu experimental equipment with a capacity of 3000 kN was used to test all specimens. The formal loading test process was conducted after about two or three initial load cycles within the elastic phase of the material.

3. FINITE ELEMENT ANALYSIS OVERVIEW

Finite element analysis (FEA) was conducted to reproduce the experimental results and observed failure behavior of the loaded gusset plate connections. The analysis software used in this study was DIANA 9.67.

(1) Analysis model

A three-dimensional geometric nonlinear analysis was implemented to model the gusset plate connections both with and without the simulated corrosion losses using a displacement load as shown in Fig. 5. The gusset plate connection and loading members were constructed of curved shell elements (the eight-node CQ40S and six-node CT30S) and the three-dimensional beam element (the two-node L13BE), respectively. In the case with the welding loss, the loss of the weld was modeled by introducing a disconnection as shown in Fig. 5(a). In cases with a Groove section, the Groove section itself was simulated using the solid brick element (the twenty-node CHX60) (see Fig. 5(b)). The sections connecting this solid element to the shell element were considered to be in the central plane of the cross-section of the gusset plate. The boundary conditions of the right and left support of the truss frame were simulated as a roller support and a pinned support, respectively.
Furthermore, the movement of the right support of the truss frame was limited by the linear translation spring element (the two-node SP2TR), as shown in Fig. 5(b). The linear spring stiffness was determined as the average slope of the relationship between the load and displacement of the right support in the horizontal direction, as measured during the loading test with a value of approximately 24000 N/mm.

With the major aim of focusing on the failure behavior, deformation performance, and strength of the gusset plate when corrosion damage occurred on the gusset plate, the dimensions of the specimens were designed such that the failure conditions due to block shear, tensile fracture, and slippage of the gusset plate would not appear during the loading tests. In addition, like the actual gusset plate connections on the subject bridges, to improve the eccentric bending moment, which could occur owing to the eccentricity between the original plane of the gusset plate and that of the flange of the diagonal member, four additional plates were attached to the outside of the gusset plate with their thickness similar to the thickness of the flange of the diagonal members, as shown in Fig. 4. Therefore, to enable easier simulation of the connecting sections between the gusset plate and the diagonal members, these connections were modeled as being monolithic. This means that the high-tension bolts were not modeled in the FEM analysis, and the gusset plate and the flange of the diagonal member were the same plane. The resolution of the finite element mesh in all of the models was 1 mm in the Groove section and 5 mm in all other members. Therefore, the total number of nodes and elements was 78223 and 27378, respectively, in the intact case, and 101403 and 28476, respectively, in cases with cross-sectional loss.

(2) Material and initial imperfection
a) Steel

The stress-strain curve relationship of the SS400 steel used in this analysis was bilinear, in which the primary Young’s modulus was 200 GPa, and the secondary modulus after yield was E/100 = 2 GPa. The Poisson ratio was 0.3, the yield stress and tensile strength were 317 MPa and 436 MPa, respectively, as declared on the mill sheet certificate. In addition, the yield stress and tensile strength of the SS400 steel were also reconfirmed through the tensile experiment in the laboratory. The Von Mises yield condition was applied to simulate the steel material, and geometric nonlinearity was considered. In this analysis, all members of the gusset plate connection were simulated as a multilinear material, and the loading members and connecting plates were considered to be elastic materials.

b) Initial imperfection (Initial deflection and residual stress)

In this study, before conducting the loading test on all specimens, the initial deflection of the gusset plates was measured directly. The measured value indicated that the average inclination level of the gusset plates was approximately 1 mm, and the initial deflection shape was considered as the SIN-shape. Therefore, the initial deflection of the gusset plate was considered in all of the analysis models. The maximum initial deflection of the gusset plate was \( h_w/K \) for the intact case, the cross-sectional loss cases S, and L; and \( h_d/K \) for the weld loss case W, as determined using Equations (1) and (2), respectively.

\[
\Delta y = \frac{h_w}{K} \sin \left( \frac{\pi z}{h_w} \right) \quad (1)
\]

\[
\Delta y' = \frac{h_d}{K} \sin \left( \frac{\pi z}{h_d} \right) \quad (2)
\]

where, \( h_w \) is the height of the gusset plate, \( h_d \) is the height of the gusset plate connection, and \( K \) is a factor determining the maximum initial deflection.

In addition, because the final failure shape of the experimentally tested specimens was asymmetric in the plane of the loading frame, asymmetry was also considered in the initial deflection of the gusset plate in this analysis. Finally, an additional 1 mm of inclination was added to the initial deflected shape of the gusset plate to reproduce conditions measured in the field, as presented at the beginning of this section. The complete initially deflected shape of the gusset plate can be seen in Fig. 6.

The residual stress distributions on each surface of the gusset plate connection and the diagonal members were considered as shown in Fig. 7. However, in this analysis, the effect of the residual stress on the loading frame members was ignored. The shape and amplitude of the residual stress were determined by reference to the Specifications for Highway Bridges (JSBH)\(^b\), with a stress of \( \sigma \) for the tensile region and \(-0.25\sigma \) for the compressive region. The width of the tensile and compressive stress portions on each surface of the gusset plate connection were calculated from the self-balanced condition of the stress in the cross-section.

The initial deflection and the residual stress were inputted into the data file of the FEA models, which was used directly to implement the calculation. The displacement load was applied in 0.1-mm steps during the second phase of the analysis once the self-balanced condition of the residual stress was achieved.
Red dashed line: Specimen W, Black dashed line: the other specimens’

Fig.6 Initial deflection of the gusset plate.

(a) Gusset plate connection  (b) Diagonal member

Fig.7 Residual stress distribution. $\sigma$: Yield stress

Fig.8 Load-displacement relationship, subject to initial deflections of the gusset plate (Specimen N).

Fig.9 Contour drawing of the out-of-plane deformation of the Specimen N intact series (at maximum load).

Table 2 Analysis parameters.

| No. | Case | Corrosion level          | Dimension of Groove section | Residual stress $K (\frac{h}{h_0} \text{ and } \frac{t}{t_0})$ | Initial deflection $\Delta y$ | Test frame   |
|-----|------|--------------------------|-----------------------------|---------------------------------------------------------------|------------------------------|--------------|
| 1   | N    | Without corrosion        | $h_0$ (mm) $t_0$ (mm)      | --                                                            | --                           | 500          |
|     | N    | N500                      |                             | --                                                            | --                           | Link frame   |
|     | N    | N250                      |                             | --                                                            | --                           | Link frame   |
|     | N    | N150                      |                             | --                                                            | --                           | Link frame   |
|     | N    | N_NR                      |                             | --                                                            | --                           | Link frame   |
|     | N    | N_R                       |                             | --                                                            | --                           | Link frame   |
| 2   | W    | Welding corrosion         | $h_0$ (mm) $t_0$ (mm)      | --                                                            | --                           | 250          |
| 3   | S    | Small cross-sectional corrosion | $h_0$ (mm) $t_0$ (mm) | --                                                            | --                           | 250          |
| 4   | L    | Large cross-sectional corrosion | $h_0$ (mm) $t_0$ (mm) | --                                                            | --                           | 250          |

Unit: mm
(3) Analysis parameters

The parameters used to evaluate the FEM analysis presented in this section are shown in Table 2. In the case of Specimen N, which had no simulated corrosion loss, in order to clarify the influence of the initial deflection of the gusset plate on the maximum load-carrying capacity and the deformation performance of the gusset plate connection, a parametric analysis was conducted on the parameter K in Equation (1) for determining the initial deflection, in which the K factor was assigned values of 500, 250, and 150. The original case N_ND, in which the initial deflection was not considered, was also included in this parametric analysis. Note that the residual stress is commonly accepted to be a major influence on the load-carrying capacity and the load-displacement relationship of normal steel members under compressive force. To confirm the effects of this parameter, an additional analysis of the intact Specimen N was conducted with and without the effects of residual stress.

As the main purpose of this section was to reproduce the experimental results and failure behavior of the gusset plate connection under the loading test, an FEM analysis of the three corroded specimens (W, S, and L) was conducted. The effect of the residual stress was not considered in these cases, and the K factor was taken with 250 for all three.

4. ANALYSIS RESULTS AND DISCUSSION

As mentioned in Section 2, in order to reach the complete failure condition of the specimens with the limited capacity of the experimental equipment, the link frame and the truss frame were used to conduct the loading tests. Because of the difference in the loading frames, in this study, the following data from the experimental and analytical results were evaluated using the average axial force in the two diagonal members of either frame connecting directly to the specimen.

(1) Intact model (Specimen N)

a) Influence of initial imperfection on max. load

To investigate the influence of initial imperfections on the maximum load and the load-displacement relationship of the gusset plate connection under the FEM analysis, two parametric analyses were implemented by varying the initial deflection of the gusset plate and the residual stress in the gusset plate connection.

Figure 8 depicts the relationship between load and vertical displacement at the highest point of the tensile link member of the link frame for the Specimen N test. In Fig. 8, the K factor in Equation (1) determining the initial deflection of the gusset plate was varied, and the parametric analysis results were obtained as follows. The enlarged view attached to Fig. 8 is provided to clarify the changes toward the end of the load-displacement curve. Based on the information shown in Fig. 8, when the initial deflection was changed, the initial stiffness of the gusset plate connection was completely unchanged. However, the change in the maximum load was quite small. The out-of-plane deformation of each initial deflection case evaluated in the FEM parametric analysis is shown in Fig. 9, which depicts the deformation contours at maximum load. The out-of-plane deformation at the free edges of the gusset plate did not appear when the initial deflection was not considered in the case N_ND (Fig. 9(a)). However, in the cases where the initial deflection was considered, the largest out-of-plane deformation was observed at the free edges of the gusset plate, and this deformation decreased as the initial deflection decreased (Fig. 9(b), 9(c), and 9(d)). As before, this change was not considerable.

Figure 10 illustrates the relationship between the load and vertical displacement at the highest point of the tensile link member, in the intact Specimen N tests, both with and without residual stress on the gusset plate connection, as determined by the FEM analysis. Clearly, the inclusion of residual stress resulted in a slightly reduced gusset plate stiffness. However, there was almost no change in the maximum load carried by the gusset plate connection: the maximum load was 1735 kN without residual stress and 1731 kN with residual stress. This small difference is due to the limited size of the compressive stress regions of the gusset plate connection, and the small residual stress levels in these areas.

From the FEM analysis results when the initial deflection of the gusset plate and the residual stress in the gusset plate connection were varied, it is confirmed that the initial imperfections had very little effect on the maximum load and the load-displacement relationship of the gusset plate connection. However, the out-of-plane deformation at the free edges of the gusset plate in all cases will not appear, if the initial deflection of the gusset plate is not considered. Therefore, in the FEM analysis of all following specimens, the K factor was taken as 250, and the residual stress was not considered.
b) Load-vertical displacement relationship

With the basic aim of determining the deformation performance, failure behavior, and load-carrying capacity of the gusset plate connection, the loading test and FEM analysis of the intact Specimen N (without any simulated corrosion loss) were implemented. The relationship between load and vertical displacement at the highest point of the tensile link member of the link frame for Specimen N are shown in Fig. 11 to provide comparison between the analytical and experimental results.

From the analysis, it is clear that the load-displacement curve begins to change at a load of 1226 kN (the buckling load) due to buckling of the plate region underneath the compressive diagonal member, a trend consistent with the behavior obtained in the experiment. Furthermore, the experimental and analytical initial stiffness of the gusset plate connection was almost in agreement. After overcoming the buckling load, the load-displacement curve of the analysis diverged slightly from the experimental result; however, the difference was quite small. The maximum load carried by the gusset plate connection was 1634 kN from the experimental result, and 1735 kN from the analytical result, a difference of approximately 6%. This difference is explained by the influence of the variations in the thickness of the actual gusset plate. The measured thickness of the actual gusset plates tested at the laboratory varied from 7.6 mm to 8 mm, as did the gusset plate used for Specimen N. As in Fig. 11, a gusset plate thickness of 7.6 mm and 7.8 mm in the analysis results in a 1% and 4% difference in maximum load, respectively, compared to the experi-
mental result.

These comparisons confirm that the load-displacement curve and the maximum load provided by the analytical result agreed well with those of the experimental result in the intact Specimen N case.

c) Failure conditions

The initial failure condition observed in the link frame tests of Specimen N was the buckling of the plate region underneath the compressive diagonal member at a load of 1184 kN. This was confirmed by the relationship between the load and bending strain of this plate region, shown in Fig. 12. Furthermore, the load-carrying capacity of Specimen N reached the maximum value after large out-of-plane deformation occurred both in this plate region and at the free edges of the gusset plate, due to buckling.

The comparisons of the analytical results with the experimental results in Figs. 12 and 13 clearly demonstrate that the simulated and observed relationships of load-bending strain and out-of-plane deformation in the plate region underneath the compressive diagonal member and at the free edges of the gusset plate in the compressive direction are consistent. The overall shape of the gusset plate connection is compared in Fig. 14: the out-of-plane deformation shown in the simulated contours is quite similar to the final shape of the experimental specimen after achieving maximum load.

As a result of these comparisons, the failure behavior of the gusset plate connection observed in the loading test can clearly be accurately reproduced using an FEM analysis.

(2) Corroded models (Specimens W, S, and L)

a) Load-vertical displacement relationship

Damage resulting in the loss of some portions of the gusset plate is likely to reduce the load-carrying capacity of the gusset plate connection. Therefore, to clarify the relationship between damage and capacity reduction, loading tests and FEM analyses were conducted to evaluate the effects of two types of losses on the gusset plate: the loss of the flange-to-gusset weld and the loss of plate cross-section. The relationship between load and vertical displacement, determined by loading tests and FEM analyses of the weld loss Specimen W, and the small and large cross-sectional loss Specimens S and L, respectively, are shown in Fig. 15. The black dashed lines (1634 kN) indicate the load-carrying capacity of intact Specimen N, obtained by loading test and described in Section 4.1b.

From the experimental results, the load-carrying capacities achieved by Specimens W, S, and L were 1547 kN, 1303 kN, and 415 kN, respectively. This indicates that the loss of the weld in Specimen W resulted in a load-carrying capacity decrease of only 5.3% from that of the intact Specimen N. On the other hand, the cross-sectional loss simulated in Specimens S and L resulted in a decrease in load-carrying capacity of 20.3% and 74.6%, respectively, from that of the intact Specimen N.

The relative impacts of weld loss and cross-section area reduction on the load-carrying capacity of the gusset plate connection have thus been clarified by the experimental results: the corrosion loss of the flange-to-gusset weld had only a slight effect, while the corrosion loss of the gusset plate cross-section had a significant effect.

The comparison of the experimental and FEM analysis results shown in Fig. 15 indicates consistent agreement between the two results. The load-displacement curve in the case of Specimen W begins to change at a load of 1200 kN as a result of the buckling of the plate area underneath the compressive diagonal member. In the case of Specimen S, the load-displacement curve begins to change at a load of 800 kN due to local buckling in the Groove section, and in the case of Specimen L, this occurs at a load of 240 kN due to shear buckling in the Groove section.

The maximum load determined by the analysis was 1639 kN for Specimen W, 1334 kN for Specimen S, and 441 kN for Specimen L. The maximum loads determined by experiment and analysis for all specimens are compared in Fig. 16, in which it can be observed that the difference in maximum load between the analytical and experimental results was between -6% and -2%, indicating that the analytical model provides a high level of accuracy. The small differences observed can be mainly attributed to the influence of the varying thickness of the actual gusset plate, as described in Section 4.(1) b).

b) Failure condition

The results of the loading test conducted on Specimen W indicate that the initial failure condition was buckling in the plate region underneath the compressive diagonal member and near the lost portion of the weld at a load of 1200 kN. This load is determined from the relationship between the load and the bending strain of the plate region underneath the diagonal member shown in Fig. 17. The load-carrying capacity of Specimen W reached its maximum value after large out-of-plane deformation developed in the region underneath the compressive diagonal member as a result of buckling. Unlike in the intact Specimen N, Specimen W exhibited the largest out-of-plane deformations not only at the free edges of the gusset plate, but also in the region near
the weld loss because losing the weld increased the effective buckling length in this area of the plate. Figure 18 illustrates the relationship between the load and the out-of-plane deformation in the subject plate region, indicating that the analytical results are consistent with the experimental results.

In the small cross-sectional loss Specimen S, local buckling initially appeared during the experiment in the Groove section under a load of 800 kN. This is understood to be the result of eccentricity due to the decreased thickness of the Groove section, causing a significant increase in the bending moment in the compressive direction. This is illustrated by the relationship between the load and bending strain in the Groove section as shown in Fig. 19. Additionally, the next failure condition was marked by the appearance of buckling in the plate region underneath the compressive diagonal member at a load of 1200 kN, as determined by reference to the relationship between load and bending strain shown in Fig. 20. The final failure condition for Specimen S was the large out-of-plane deformation resulting from buckling in the plate region underneath the compressive diagonal member, and shear fracture at the Groove section. The analytically and experimentally determined relationships of load-bending strain and out-of-plane deformation in this plate region and at the free edges of the gusset plate in compression are shown in Fig. 20 and 21, indicating that the analytical result is consistent with the experimental result.

(a) Weld loss Specimen W

(b) Small cross-sectional loss Specimen S

Max load of the intact case N from EXP: 1634 (kN)

(c) Large cross-sectional loss Specimen L

Fig.15 Load-vertical displacement relationship (W, S, and L).

Fig.16 Maximum load between FEA and EXP.

(a) Underneath the diagonal member

(b) Free edge of the gusset plate

Fig.17 Load-bending strain relationship (W).

Fig.18 Out-of-plane deformation in plate region underneath diagonal member (W).
Fig. 19 Load-bending strain relationship of Specimen S.

(a) Location A
(b) Location B
(c) Location C

Fig. 20 Load-bending strain relation (S).

(a) Underneath the diagonal member
(b) Free edge of the gusset plate

Fig. 21 Out-of-plane deformation in plate region underneath diagonal member (S).

(a) Weld loss Specimen W
(b) Small cross-sectional loss Specimen S
(c) Large cross-sectional loss Specimen L

Fig. 22 Analytical contours of out-of-plane deformation and physical deformation of the gusset plate (at maximum load).
In the case of the large cross-sectional loss Specimen L, unlike Specimen S, shear buckling occurred at the Groove section. This is understood to be a result of the comparatively large height/thickness ratio of the Groove section, which reduced its shear buckling strength. The final failure in this case was the result of large deformation due to shear buckling and shear fracture. Figure 22 provides a comparison of the simulated contours describing the out-of-plane deformation and the residual deformation of the gusset plate connection specimens after the conclusion of the loading tests. In this figure, the white dashed-line indicates the weld loss and Groove section areas on the gusset plate.

As a result, in the gusset plate with simulated corrosion loss of the weld, the largest out-of-plane deformations appeared not only at the free edges of the gusset plate (as in the case of Specimen N), but also in the area of the corroded weld. In the gusset plates with simulated corrosion loss of plate cross-section, the failure behavior of the corroded gusset plate depended on the severity of the corroded section: local buckling was observed in the small corrosion case of Specimen S, while shear buckling was observed in the large corrosion case of Specimen L. Notably, the failure behavior of the corroded gusset plate connections observed in the loading test could be accurately reproduced using the FEM analysis.

Moreover, from the results shown in Section 4, the influential level of not modeling the bolts of the connecting sections in the FEM analysis was confirmed to be inconsiderable through comparison of the analytical and loading test results: the difference in maximum load being between -6% and -2%, the same in the load-bending strain relationship, and the same in the load-out-of-plane deformation relationship and the physical deformation.

5. PARAMETRIC ANALYSIS

Once the FEM analysis had been confirmed as accurate by comparison with the experimental results, two parametric FEM analyses were conducted to investigate the relationship between the remaining capacity of the corroded gusset plate connection and the degree of corrosion in the two forms evaluated.

(1) Analysis parameters

The loading tests and the FEM analysis conducted in Section 4 both concluded that the damage due to corrosion could lead to a reduction in the load-carrying capacity of the gusset plate connection. The most significant reductions were observed in the cross-sectional corrosion cases. The failure conditions of the cross-sectionally corroded region depended on the corrosion level, either manifesting as local buckling under a small degree of corrosion or as shear buckling under a large degree of corrosion. To obtain further information on the remaining load-carrying capacity, failure behavior, and deformation performance of the gusset plate connection with even more severe corrosion, a parametric analysis was implemented by varying the degree of cross-sectional loss in the gusset plate. The analysis cases conducted are listed in Table 3, consisting of remaining thickness of 87.5%, 75%, 62.5%, 50%, 43.75%, 37.5%, and 25% of the original gusset plate thickness. For each thickness, the height of the corroded section was evaluated for approximately 50% and 100% of the maximum height of the potentially damaged area, defined as 50 mm in this study. Note that the link frame model was applied in all these FEM analysis cases.

An additional parametric FEM analysis was conducted by varying the length of the flange-to-gusset weld loss, detailed in the cases listed in Table 4, in which the length of the corroded weld was assumed to be either 50% or 100% of the length of the gusset plate, which was defined as 600 mm in this study.

(2) Results of parametric analyses

a) Load and vertical displacement relationship

Figure 23 depicts the relationship between the applied load and vertical displacement of the gusset plate connection as the cross-sectional loss parameter was varied from remaining thicknesses of 87.5%, 75%, 62.5%, 50%, 43.75%, 37.5%, to 25%. The inset diagram in each figure depicts an enlarged view of the initial stiffness of the gusset plate connection. When looking at Fig. 23, it is immediately obvious that for remaining thicknesses of 87.5%, 75%, 62.5%, and 50%, the initial stiffness of the gusset plate connection exhibited nearly no change with the reduction rate in the range of 2% to 5%. However, for considerable cross-sectional losses (remaining thicknesses of 43.75%, 37.5%, and 25% of the plate), a significant decrease in the initial stiffness can be clearly observed with the reduction rate in the range of 9% to 13%. Moreover, in all cases with the cross-sectional loss, the load-displacement curve began to change after the first signs of failure appeared in the Groove section. The details of the observed initial failure condition in the Groove section in each case were as described in Section 5.(2) c).

The relationships between load and vertical displacement of the gusset plate connection as the length of the flange-to-gusset weld loss was varied.
are shown in Fig. 24. As the enlarged view of the initial stiffness shown inset in Fig. 24 illustrates, there was almost no change in the initial stiffness of the gusset plate connection with the reduction rate of 3% to 5% as the length of the corroded weld increased. Additionally, the load-displacement curve of the gusset plate connection did not significantly change until buckling occurred in the plate region underneath the compressive diagonal member.

The results of the FEM parametric analyses indicate that a significant reduction in the initial stiffness of the gusset plate connection occurs in cases when the thickness of the gusset plate has been reduced by more than 50%. In cases with less than 50% loss of thickness, there is no significant reduction in stiffness. Finally, there was no change observed in the initial stiffness of the gusset plate connection as the length of the corroded portion of the flange-to-gusset weld was increased.

b) Remaining load-carrying capacity

The maximum loads determined using the parametric FEM analysis of the change in cross-sectional loss are shown in Fig. 25(a), in which the horizontal axis and vertical axis depict $P_{\text{max}}/P_{\text{0 max}}$ and the remaining thickness (%), respectively. In this figure, $P_{\text{max}}$ and $P_{\text{0 max}}$ are the maximum load carried by the cross-sectional loss case and the maximum load of the intact Specimen N, respectively. As shown in Fig. 25(a), the loss of the gusset plate cross-section significantly decreased the load-carrying capacity of the gusset plate connection. For both Groove section heights evaluated, the relationship between $P_{\text{max}}/P_{\text{0 max}}$ and the remaining thickness rate were approximately linear, while the value of $P_{\text{max}}/P_{\text{0 max}}$ exhibited a sharp decrease when the failure condition in the Groove section changed from local buckling to shear buckling.

For the same remaining thickness, as the height of the Groove section was increased, a reduction in the maximum capacity of the gusset plate connection was only notable for cases in which local buckling appeared in the Groove section, while only a slight reduction was observed for cases in which shear failure or shear buckling occurred in the Groove section. The final failure of the gusset plate in cases failing by shear buckling was due to shear failure (i.e., insufficient shear strength) of the Groove section, which is independent of the height of the cross-sectional loss. On the other hand, the final failure of the gusset plate in cases failing by local buckling in the Groove section was due to the large out-of-plane deformation that appeared in the plate region underneath the compressive diagonal member.

The parametric FEM analysis determined that as the length of the corroded flange-to-gusset weld increased, the maximum loads were as shown in Fig. 25(b), which reveals that the loss of the weld only slightly reduced the load-carrying capacity of the gusset plate connection, in the range of 5% to 7%. This is understood to be a result of the compressive strength of the diagonal member, which determines the load-carrying capacity of the gusset plate connection, and thus depends greatly on the dimensions and strength of the free edges of the gusset plate in the compressive direction. Obviously, the behavior of these free edges was not at all affected by the loss of the flange-to-gusset weld.

Table 3 Parametric analysis parameters for cross-sectional corrosion loss cases.

| No. | Case    | Remaining thickness of corroded section / 8 mm (%) | Height of corroded section / 50 mm (%) | Dimension of Groove section $t_z$ (mm) | $h_z$ (mm) | Test frame | Note |
|-----|---------|-----------------------------------------------|-------------------------------------|------------------------------------|------------|------------|------|
| 1   | t87.5h50| 87.5                                           | 50%                                 | 1                                  | 25         |            |      |
| 2   | t87.5h100| 100%                                          | 100%                                | 2                                  | 50         |            |      |
| 3   | t75h50  | 75%                                           | 50%                                 | 2                                  | 25         |            |      |
| 4   | t75h100 | 100%                                          | 100%                                | 3                                  | 50         |            |      |
| 5   | t62.5h50| 62.5%                                         | 50%                                 | 3                                  | 25         |            |      |
| 6   | t62.5h100| 100%                                        | 100%                                | 4                                  | 25         |            |      |
| 7   | t50h50  | 50%                                           | 50%                                 | 4                                  | 50         | Link frame| EXP  |
| 8   | t50h100 | 100%                                          | 100%                                | 5                                  | 50         | Link frame| EXP  |
| 9   | t43.75h50| 43.75%                                        | 50%                                 | 5                                  | 25         |            |      |
| 10  | t43.75h100| 100%                                        | 100%                                | 6                                  | 50         |            |      |
| 11  | t37.5h50| 37.5%                                         | 50%                                 | 5                                  | 25         |            |      |
| 12  | t37.5h100| 100%                                        | 100%                                | 6                                  | 50         |            |      |
| 13  | t25h50  | 25%                                           | 50%                                 | 6                                  | 25         |            |      |
| 14  | t25h100 | 100%                                          | 100%                                | 6                                  | 50         | EXP        |      |

Table 4 Parametric analysis parameters for flange-to-gusset weld loss cases.

| No. | Case        | Location of corroded weld               | Length of corroded weld / 600 mm (%) | Test frame | Note |
|-----|-------------|----------------------------------------|---------------------------------------|------------|------|
| 1   | W50         | Compressive direction                  | 50%                                   | Link frame | EXP  |
| 2   | W100        | Compressive direction + Tensile direction | 100%                                  |            | EXP  |
Fig. 23 Load-displacement relationship as the cross-sectional loss parameter is varied.

Fig. 24 Load-displacement relationship as the flange-to-gusset weld loss parameter is varied.
Fig. 25 Relationship between corrosion loss and remaining load-carrying capacity.

Fig. 26 Analysis contours of out-of-plane deformation as the cross-sectional loss is varied (at maximum load).
c) Failure condition

Figure 26 shows the contours of the residual out-of-plane deformation of the gusset plate connection as the degree of cross-sectional loss was varied. The results obtained from the FEM analyses confirmed that the initial failure condition of the gusset plate occurred in the Groove section in all cases. For the cases with a Groove height of 25 mm (50% of the maximum damaged section height), local buckling appeared in the Groove section when the remaining plate thickness was greater than 50% of the original (see Figs. 26(a1)-26(d1)). For the cases with a Groove height of 50 mm (100% of the maximum damaged section height), local buckling in the Groove section was observed when the remaining thickness was greater than 37.5% of the original (see Figs. 26(a2)-26(e2)). In all these cases, at the peak load value the gusset plates failed from large out-of-plane deformation due to the buckling in the plate region underneath the compressive diagonal member and at the free edges of the gusset plate.

In the other cases, the initial failure condition of the gusset plates was observed to be shear failure or shear buckling in the Groove section (see Figs. 26(e1)-26(g1), and 26(f1)-26(g2)), and at their peak load-carrying capacity the gusset plates failed from...
large deformation of the Groove section due to shear failure.

The contours describing the out-of-plane deformation of the gusset plates at maximum load as the length of the flange-to-gusset weld loss was increased can be seen in Fig. 27. The FEM analysis indicates that in all cases, the initial failure condition region underneath the compressive diagonal member, nearest to the lost weld, and the final failure condition was large out-of-plane deformation in the plate region nearest to the lost weld, and at the free edges of the gusset plate. The size of the region exhibiting large out-of-plane deformation near the lost weld can be observed to increase considerably as the length of the weld loss increased.

6. EVALUATION EQUATION FOR LOCAL BUCKLING STRENGTH

(1) Proposed evaluation equation

This section proposes an evaluation method for determining the local buckling strength of the plate region underneath the compressive diagonal member in cases of the corrosion loss of the gusset plate thickness. For the specimens tested in the experiments, the diagonal members were connected to the gusset plate using bolts through the connecting plates. Therefore, the effective width of the buckling plate area was determined in accordance with the Whitmore method (Fig. 28(a)). The local buckling strength of the plate region with the cross-sectional corrosion was then calculated from three component fixed-end-columns (l01, l02, and l03) with the sudden change in cross-section, as shown in Fig. 28(b). In this evaluation method, the eccentricity of the cross-sectional loss section and the initial deflection of the local buckling plate region is neglected. Thus, the buckling load condition of each “component column” with sudden change in cross-section can be described by Equation (3).

\[
\frac{\lambda_1}{\lambda_2} \tan(\lambda_1 l_1) + \tan(\lambda_2 l_2) = 0 \quad (3a)
\]

\[
\lambda_1 = \sqrt{\frac{P}{EI_1}} \quad \lambda_2 = \sqrt{\frac{P}{EI_2}} \quad (3b)
\]

where, \(E\) is the elastic modulus of steel; \(l_1\) and \(l_2\) are the length of the plate region without and with corrosion, respectively; and \(I_1\) and \(I_2\) are the moment of inertia of the plate region without and with corrosion, respectively (see Fig. 28(b)).

The process of calculating the local buckling strength of the plate region underneath the compressive diagonal member is as shown in the flowchart in Fig. 29. More specifically, it is described as follows.

In Step 1, the effective buckling length (\(l_{01}, l_{02}, \) and \(l_{03}\)) of each component column was calculated by using the local buckling strength (\(P_1, P_2, \) and \(P_3\)) of each component column, calculated using Equation (3a). Then, in Step 2, the slenderness ratio \(\bar{\lambda}_c\) of the plate region in which the local buckling occurred was determined as the average value of the three component slenderness ratios, by Equation (4b).

The slenderness ratio is determined by:

\[
\left\{
\begin{align*}
I_{01} \Rightarrow \bar{\lambda}_{01} &= \pi \sqrt{\frac{EI_{01}}{P_1}} \\
I_{02} \Rightarrow \bar{\lambda}_{02} &= \pi \sqrt{\frac{EI_{02}}{P_2}} \\
I_{03} \Rightarrow \bar{\lambda}_{03} &= \pi \sqrt{\frac{EI_{03}}{P_3}}
\end{align*}
\right. \quad (4a)
\]

\[
\Rightarrow \bar{\lambda}_{c} = \frac{\lambda_{c1} + \lambda_{c2} + \lambda_{c3}}{3} \quad (4b)
\]

where, \(l_{01}, l_{02}, \) and \(l_{03}\) are the moment of inertia of each component column; \(r_{11}, r_{22}, \) and \(r_{33}\) are the radius of gyration of each component column; \(\lambda_{c1}, \lambda_{c2}, \) and \(\lambda_{c3}\) are the slenderness ratio of each component column; and \(\sigma_y\) is the yield stress of the steel.

Finally, the local buckling strength of the compressive plate area was determined using the standard buckling equations specified in Japanese Design code (JISHB) with the previously calculated slenderness ratio value, as shown in Equation (5).

\[
P_{cr} = \sigma_y A_{\text{average}} \begin{cases} 
\bar{\lambda}_c & (\bar{\lambda}_c \leq 0.2) \\
(1.109 - 0.545\bar{\lambda}_{c})\sigma_y A_{\text{average}} & (0.2 < \bar{\lambda}_c \leq 1.0) \\
\frac{1}{0.773 + \bar{\lambda}_c}\sigma_y A_{\text{average}} & (1.0 < \bar{\lambda}_c)
\end{cases} \quad (5a)
\]

where, \(A_{\text{average}}\) is the average cross-sectional area of the plate.

(2) Calculated result and discussion

The details of the process used to calculate the local buckling strength of the plate region underneath the compressive diagonal member with cross-sectional corrosion are shown in Table 5 and Table 6. To confirm the accuracy of the proposed evaluation method, the local buckling strength value obtained from the parametric FEM analyses, the loading tests, are also listed in Table 6. Further, a comparison of the local buckling strength predicted by the loading test and analysis and by the proposed calculation is shown in Fig. 30(a). To evaluate the differences between the proposed calculation method and a method using another standard buck-
ling strength curve, another set of results, calculated with AASHTO, was included, as shown in Fig. 30(b). The information shown in Fig. 30 indicates that when using the buckling strength curve obtained by JSHB, the difference in strength between the calculated and observed results was in the range of -10% to 0% on the safe side, indicating that the proposed calculated result had a high level of accuracy. However, when using the buckling strength curve of AASHTO, the difference in strength between the calculated and observed results was in the range of -10% to +10%, with only some of the results on the safe side. This is due to the fact that for the same slenderness ratio, the value of the buckling strength given by the strength curve in JSHB is typically lower than that given by AASHTO, as shown in Fig. 31. Therefore, to safely evaluate the local buckling strength of the compressive plate area, the use of the standard buckling strength curve given by JSHB is strongly preferred.

**Figure 32** illustrates the calculated local buckling strength of the cross-sectionally corroded section using the proposed method for various corrosion heights and remaining thicknesses. In this figure, the shear strength of the cross-sectionally corroded section is expressed by the value of the shear yield strength. It is immediately obvious that for both corrosion heights, the intersections between the local buckling strength curve and the shear yield strength line are consistent with the point at which the mode of failure of the cross-sectionally corroded section (obtained in Section 5.2(c)) changes from local buckling to shear buckling. This demonstrates that using the relationship between the local buckling strength curve and the shear yield strength line can effectively determine the change in the failure condition of a corroded gusset plate cross-section.

![Local buckling plate area](image)

(a) Local buckling plate area  
(b) A column with fixed ends

**Fig.28** Local buckling underneath the compressive diagonal.

**Step 1**

1. Calculate the local buckling strength of each component column.

2. Calculate the effective buckling length of each component column.

3. Calculate the slenderness ratio of each component column.

4. Calculate the average slenderness ratio.

5. Calculate the local buckling strength of the compressive plate region.

**End**

**Table 5** Calculating the local buckling strength of the three component columns of the gusset.

| Case | $l_{i1}$, $l_{i2}$ and $t_i$ (mm) | $h$ (mm) | $l_1$ (mm) | $l_2$ (mm) | $t_1$ (mm) | $t_2$ (mm) | $t_{average}$ (mm) | $b$ (mm) | $I_1$ (mm$^4$) | $I_2$ (mm$^4$) | $E$ (MPa) | $f_u$ ($f_i = 1$) (MPa) | $P_c$ ($f_i = 1$) (kN) |
|------|---------------------------------|---------|------------|------------|-----------|-----------|------------------|---------|-------------|-------------|-----------|--------------------------|-----------------|
| S50h50 Column 1 | 75 | 25 | 45.0 | 30.0 | 8 | 4 | 6.4 | 218 | 9301 | 1163 | 200000 | 4757 | 1412 |
| Column 2 | 148 | 25 | 118.0 | 30.0 | 8 | 4 | 7.2 | 218 | 9301 | 1163 | 200000 | 6747 | 385 |
| Column 3 | 44 | 25 | 44.0 | 0.0 | 8 | 8 | 8.0 | 218 | 9301 | 9301 | 200000 | 9301 | 37934 |
| N Column 1 | 75 | 25 | 45.0 | 30.0 | 8 | 8 | 8.0 | 218 | 9301 | 9301 | 200000 | 9301 | 37934 |
| Column 2 | 148 | 25 | 118.0 | 30.0 | 8 | 8 | 8.0 | 218 | 9301 | 9301 | 200000 | 9301 | 3353 |
| Column 3 | 44 | 25 | 44.0 | 0.0 | 8 | 8 | 8.0 | 218 | 9301 | 9301 | 200000 | 9301 | 37934 |
7. CONCLUSION

This study evaluated the effects of two different forms of gusset plate corrosion on the load-carrying capacity of the gusset plate connection: the loss of the flange-to-gusset weld and the loss of the gusset plate cross-section. Loading tests performed in the laboratory and an FEM analysis were conducted on an existing bridge configuration using an approximately half-scale model. The gusset plate connections were tested for 50% and 75% corrosion of the gusset plate thickness, and 50% corrosion of the length of the flange-to-gusset weld in compression. This study then conducted parametric FEM analyses by changing the size of the corroded sections to verify the relationship between the remaining load-carrying capacity and corrosion levels. Finally, based on the results of the parametric FEM analysis of cases with the corrosion loss of the gusset plate thickness, an evaluation method for determining the local buckling strength of the corroded section was proposed. The results obtained from this study are summarized as follows:

(1) The reduction in the load-carrying capacity of the gusset plate connection resulting from corrosion was determined using the experimental results. Specifically, only a slight reduction in the load-carrying capacity was observed in the case of flange-to-gusset weld corrosion, while significant reduction in capacity was observed in the case of cross-sectional corrosion.

(2) The experimental results revealed that in the case of flange-to-gusset weld corrosion, large out-of-plane deformation appeared not only at the free edges of the gusset plate, but also in the area of the corroded weld. In cases with cross-sectional corrosion, local buckling was observed to be the failure mechanism of a plate with a small degree of corrosion and shear buckling was verified as the failure mechanism of a plate with a large degree of...
corrosion.

(3) The deformation performance, failure behavior, and load-carrying capacity of the gusset plate connection observed in the loading tests was reproduced with high accuracy using an FEM analysis in all cases.

(4) The results of the parametric FEM analyses confirmed that there was a significant reduction in the initial stiffness of gusset plate connections in cases of corroded gusset plates with less than 50% of the original thickness remaining. Corroded gusset plates with greater than 50% remaining thickness showed little change in stiffness. No change was observed in the initial stiffness of the gusset plate connection when the length of the flange-to-gusset weld corrosion was extended.

(5) Based on the results of the parametric FEM analysis, as the dimensions of the corroded sections were increased, the load-carrying capacity of the gusset plate connection decreased. Specifically, for each cross-sectional corrosion height evaluated, the load-carrying capacity of the gusset plate connection exhibited nearly the same linear decrease with the increasing thickness of the cross-sectional corrosion of the section. Furthermore, the load-carrying capacity of the gusset plate connection was found to sharply drop when there was a change in the failure condition of the corroded section. Only a slight reduction in capacity, in the range of 5% to 7%, was found as the length of the flange-to-gusset weld corrosion increased from 50% to 100% of the gusset plate width.

(6) The parametric FEM analysis results revealed that under the effects of cross-sectional corrosion, local buckling in the corroded section occurred in cases with more than 50% of the original gusset thickness remaining for a corrosion height of 25 mm (50% of the potential corroded section), and in cases with more than 37.5% of the original gusset thickness remaining for a corrosion height of 50 mm (100% of the potential corroded section). Shear failure or shear buckling in the corrosion section was observed in the other cases. In cases of flange-to-gusset weld corrosion, large out-of-plane deformation was observed in the area near the corroded weld during failure.

(7) In this study, it was proposed that the local buckling strength of the cross-sectionally corroded section was evaluated as a column with suddenly changing cross-section properties. The calculated result obtained from this proposed evaluation method was on the safe side in the range of -10% to 0% when compared to the results of the FEM analysis and loading tests.

(8) Using the proposed method to evaluate the local buckling strength of the cross-sectionally corroded section, it was confirmed that the relationship between the local buckling strength curve and the shear yield strength line can be used to easily determine the failure conditions of a cross-sectionally corroded section.

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