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Analytical and Experimental Studies on Optimal Details of Orthotropic Steel Decks for Long Span Bridges

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Abstract

Orthotropic steel decks gradually come into general use in long span bridges due to various advantages such as dead weight reduction, easy quality control, and so on. On the other hand, stress concentration is often observed near the connection details, which may lead to fatigue problems and govern the design of the structure. Therefore, researches to understand the structural behaviors and examine stress distribution of details in orthotropic steel decks are required in order to develop an optimal connection detail. In this paper, optimal parameters regarding height, thickness and shear area of a cross-beam and efficiency and shapes of bulkhead plates are characterized by analytical studies and verified by experiments, so that fatigue cracks could be prevented.

Keywords: orthotropic steel deck, bulkhead plate, stress concentration, fatigue, optimal details, parametric study

1. Introduction

An orthotropic steel deck system strengthens the deck plate with longitudinal ribs, also known as U-ribs, and transverse cross-beams. The ribs make the deck system torsionally stiff by welding to the deck plate and cross-beams.

The orthotropic steel decks have become very frequently used in construction of long span bridges such as suspension bridges and cable stayed bridges. The increasing employment of the orthotropic steel deck system for bridge construction is due to its advantages such as (1) light weight compared with that of concrete decks, (2) guarantee of high quality by controlling its fabrication procedures at manufacturers, (3) convenience for in-situ assembling process which leads to reduction of construction period and cost, (4) aesthetic appearance provided by slender external shapes, etc (Mangus and Sun, 1999). Furthermore, development of new steel materials and improvement in welding skills widens its application fields.

On the contrary, the orthotropic steel deck system has the complicated structural configuration consisting of a thin steel deck plate incorporated with longitudinal U-ribs and transverse cross-beams, and it accordingly makes the stress states in the connection details complicated. In particular, around the lines of intersection between the longitudinal ribs and transverse cross-beams are vulnerable areas in terms of fatigue (Dexter and Fisher, 1997). It is local stress concentration caused by interaction between out-of-plane motion of U-ribs to adjacent cross-beams and in-plane distortion of the cross-beam that induces the secondary stresses, which may initiate fatigue cracks inside the cross-beams (Wolchuck, 1999).

Numerous researches demonstrated that the fatigue failure is one of the major factors in damage of steel bridges (Bocchieri and Fisher, 1998; Connor *et al*., 2003; Dexter and Fisher, 1997; Fuchs and Stephens, 1980; Rolf and Barsom, 1977; Tsakopoulos and Fisher, 2004). In Korea, however, studies on fatigue failure especially occurring at the orthotropic steel deck systems are yet insufficient and foreign design specifications have been just adopted without specific analyses and/or experiments. Even foreign specifications are also insufficient to design the details vulnerable to fatigue damage. Therefore, it is necessary to obtain reliable research results defining optimal structural details based on which appropriate design specification can be provided and quality monitoring becomes feasible.

In this paper, parametric studies are carried out to investigate the sensitivity of the parameters on the stress concentration inducing fatigue cracks. The parameters taken into consideration are height, thickness, and shear

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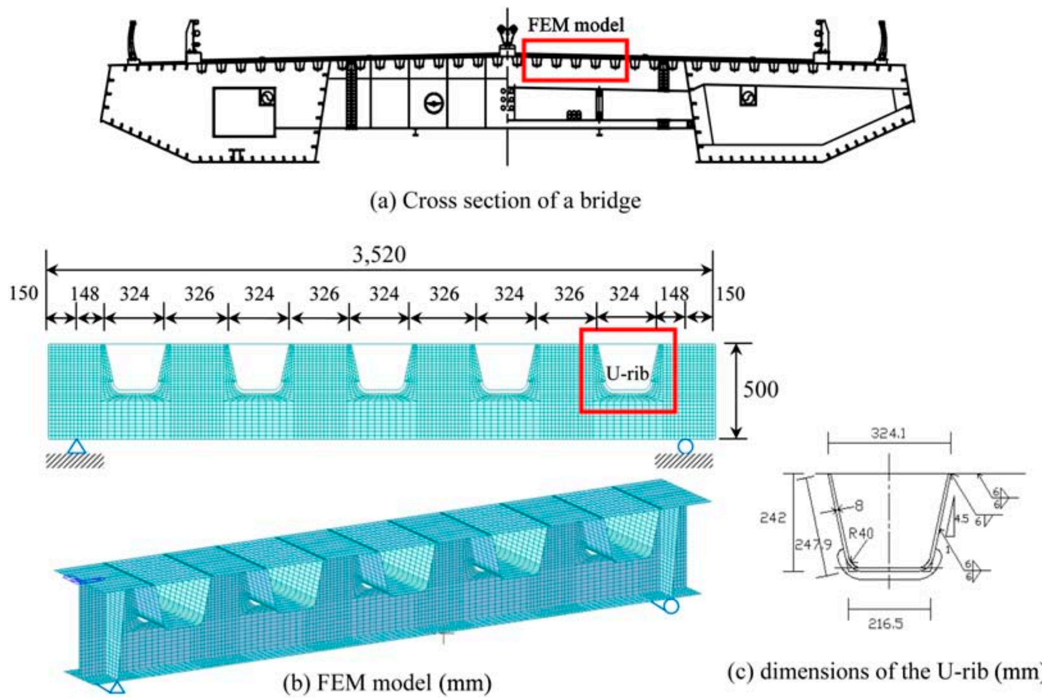


Figure 1. Configuration of a structural model.

area, i.e., multiplication of height and thickness, of a cross-beam. Then, bulkhead plates are proposed to demonstrate the capability of reducing magnitude of concentrated stresses. Optimal parameters to characterize dimensions of the cross-beam and shapes of bulkhead plates are decided based on computational analysis results using a commercial finite element method (FEM) program. Experimental verification utilizing a partial model of a bridge is also presented. Then, an optimal connection detail will be provided at the end. Note that the performance is estimated in terms of the maximum principal stress alone, since the utmost goal of the study is to propose the optimal connection detail to prevent fatigue failure.

This paper is organized as follows. Section 2 presents analytical results regarding effects of parameters of interest and proposes the optimal connection detail. In Section 3, experimental results are compared with those of analysis for verification based on the suggested optimal detail. Finally, the conclusion and discussions are provided in Section 4.

2. Parametric Study by Computational Analysis

2.1. Structural model

A structural model for the FEM analysis is constructed utilizing a portion of a bridge, i.e., a section inside a (red) box in Fig. 1(a), which has 5 U-ribs and 1 cross-beam with simple beam boundary condition as shown in Fig. 1(b). Detailed dimensions of a U-rib in Fig. 1(b) are shown in Fig. 1(c). Design of the U-ribs and scallop complies with Guide Lines for the Design of Details of Steel Highway Bridges (MOCT, 2006).

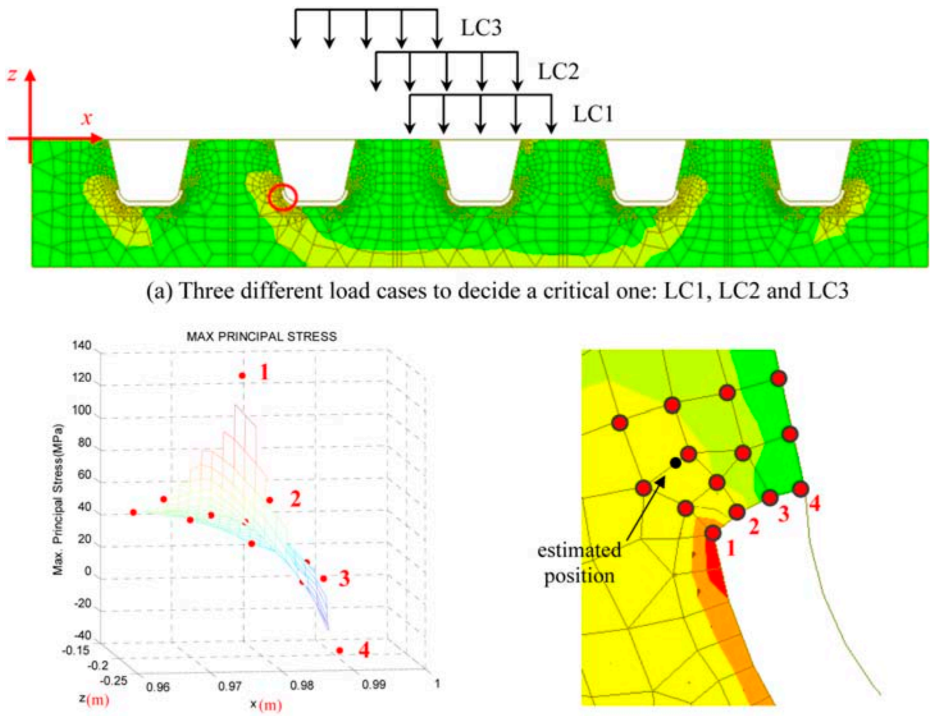
The structure is modeled in plate elements of a commercial FEM program MIDAS 2009 with SM490 steel having E=206 GPa. Thickness of entire structural members is 14 mm except 8 mm U-ribs. The length, height and width of the structural model are 3.52, 0.5, and 0.5 m, respectively. Note that a 3-dimensional bridge model containing four equally spaced cross-beams with U-ribs and a deck plate expanded will be deployed after this preliminary study on the 2-dimensional partial model.

The effects of parameters defining the structural model such as height, thickness and shear area of the cross-beam and installation of different shapes of bulkhead plates are examined under a load case that induces the maximum (tensile) principal stress near the connection details of interest. Then the optimal structural details are proposed, so that fatigue cracks caused by the maximum principal stress could be prevented.

2.2. Parametric study by varying load cases

Three critical load cases are taken into consideration: distributed loads denoted by LC1, LC2 and LC3 are applied to the center of the structure and two off-centered regions, respectively (See Fig. 2(a)). A DB24 load including impact effect, i.e., 124.8kN, is applied to a contact area of a wheel 0.58 m×0.23 m as specified on Design Specifications for Highway Bridges (KSCE, 2010).

For each case, the maximum tensile principal stress is



(a) Three different load cases to decide a critical one: LC1, LC2 and LC3

(b) Maximum principal stress distribution near the scallop under LC2

(c) Points of interest (red dots) and an estimated position (black dot)

Figure 2. Decision of a critical load case and the corresponding maximum principal stress distribution.

observed near the scallop of the second U-rib as marked with a (red) circle in Fig. 2(a). Due to the abrupt curvature change of scallop in this region, the principal stress becomes radically increased as location of the element is closer to the scallop. Furthermore, the intersection point between the U-rib and the scallop functions as an elastic support having relatively larger strength than that of the longitudinal U-rib.

In Fig. 2(b), the exponential increment of the maximum principal stresses computed at (red) dots in Fig. 2(c) is demonstrated. For example, the difference of the maximum principal stresses amounts to 172.62 MPa, i.e., 134.60 MPa-(-38.02) MPa, between two separate points being 15.7 mm apart from each other. The values of (red) dots in Fig. 2(b) represent maximum principal stresses at each node computed by the FEM model based on which a surface is estimated using triangle-based cubic interpolation in Matlab. Note that the estimated location to compute all the values of maximum principal stresses from analysis as well as experiment is about a thickness of the cross-beam away from the edge of the scallop and the end of welding because fatigue design is performed using the magnitude of nominal stresses (see Fig. 2(c)).

The parametric analysis for entire load cases addresses the LC2 is critical (See Fig. 3(a)) and the maximum principal stress under LC2 approaches 69 MPa which is a threshold stress in fatigue specified on AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) and Design Specifications for Steel Structures (KSSC, 2010) for a category C. The results demonstrate it is necessary to take a reliable and efficient stress-control measure, so that the concentrated stress could be alleviated.

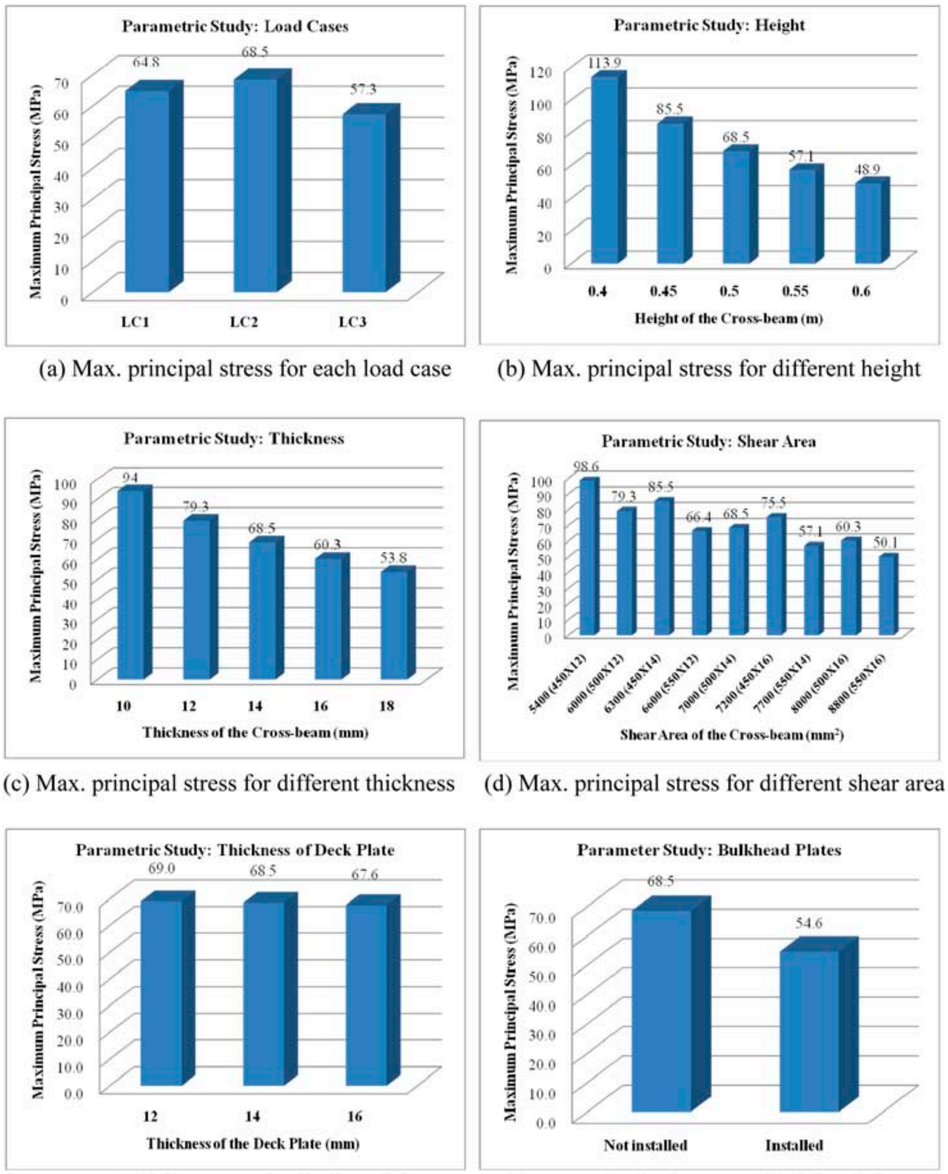
The stress concentration is mainly caused by in-plane distortion of the cross-beam resulting from external loads applied to the associated U-ribs. From the following subsection, therefore, dimensions of the cross-beam such as height, thickness and shear area are altered in order to investigate their influence on decreasing the concentrated stresses.

2.3. Parametric study by varying dimensions of the cross-beam

The factors to define the cross-beam are height, thickness and shear area, i.e., shear area=height×thickness. First of all, five different heights of 0.40, 0.45, 0.50, 0.55 and 0.60 m for the cross-beam are attempted. As shown in Fig. 3(b), the maximum principal stresses tend to decrease as the height increases. Of great interest is the fact that the rate of reduction becomes lower, so that at some point using higher cross-beam may be no longer advantageous considering the self weight.

The thickness of the 0.5 m high cross-beam is also changed from 10, 12, 14, 16 and 18 mm and the increasing thickness results in reduction of the maximum tensile principal stress as shown in Fig. 3(c).

Based on the results so far achieved, it is expected that



(a) Max. principal stress for each load case (b) Max. principal stress for different height

(c) Max. principal stress for different thickness (d) Max. principal stress for different shear area

(e) Max. principal stress to deck plate thickness (f) Max. principal stress to bulkhead plates

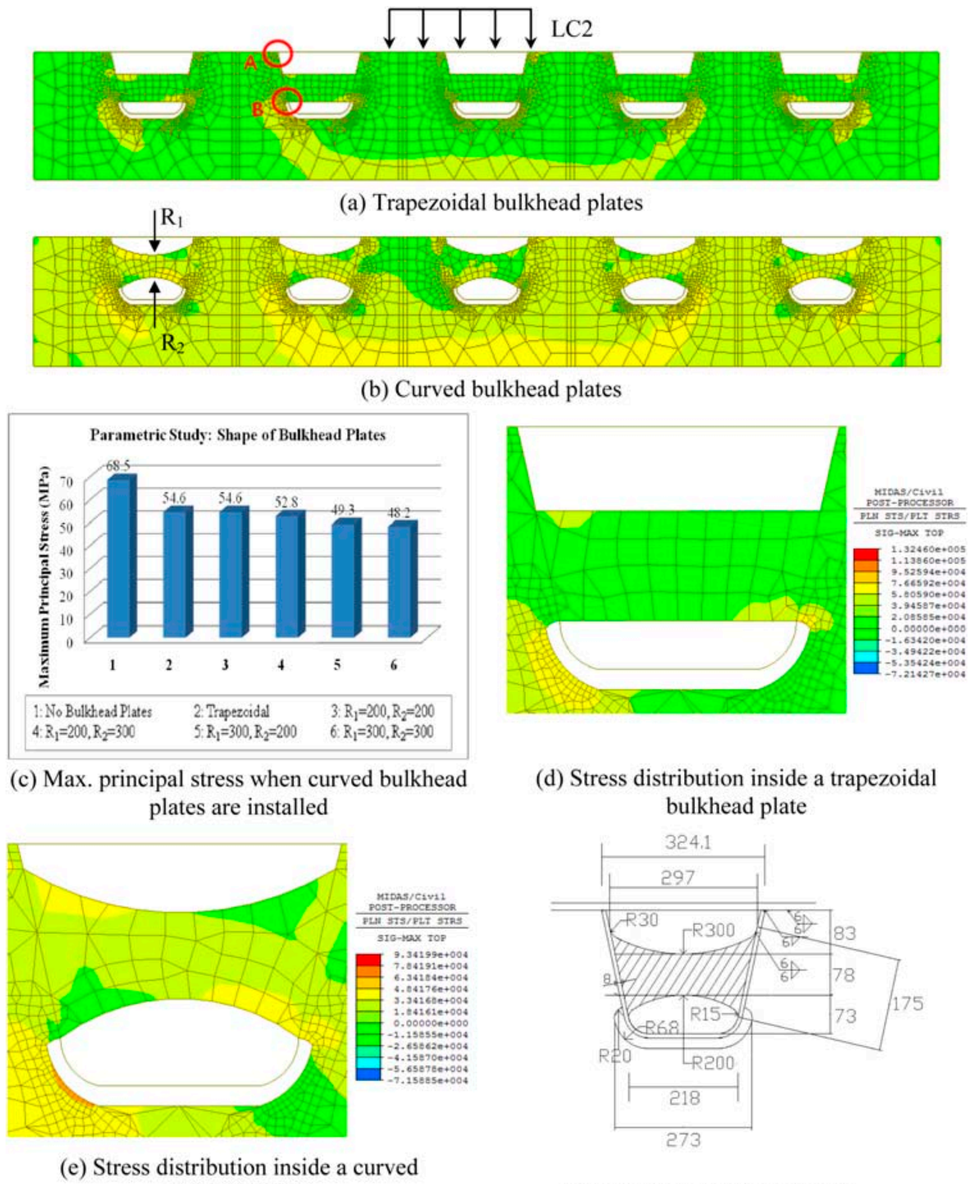
Figure 3. Parametric studies on load cases, height, thickness and shear area of the cross-beam, deck plate thickness and installation of bulkhead plates.

stress distribution around the scallop in the cross-beam is closely related with shear area of the cross-beam. FEM analyses are conducted using various shear areas of the cross-beam that change from 5,400 to 8,800 mm^2 and in general the maximum principal stresses are inversely proportional to the shear areas as shown in Fig. 3(d).

In addition to this fact, we can recognize that it is favorable to select a higher cross-beam if the difference of shear areas between candidates is insignificant. For example, a model with 6,000 mm^2 shear area is 0.6% lighter than another one with 6,300 mm^2 but the maximum principal stress falls by 12% (See Fig. 3(d)).

2.4. Parametric study by varying thickness of the deck plate

The thickness of the deck plate is often considered one of the major factors to reduce stress concentration. Three structural models having different deck plate thickness of 12, 14 and 16 mm are analyzed accordingly. Contrary to the expectation, the maximum principal stresses show a slight decrease of 2.03% as shown in Fig. 3(e). Therefore, it can be confirmed that the in-plane distortion taking place inside the cross-beam governs the stress states to produce stress concentration. In the future, 3-dimensional structural model will be employed to examine the



(a) Trapezoidal bulkhead plates

(b) Curved bulkhead plates

(c) Max. principal stress when curved bulkhead plates are installed

(d) Stress distribution inside a trapezoidal bulkhead plate

(e) Stress distribution inside a curved bulkhead plate

(f) Optimal connection details

Figure 4. Parametric study on bulkhead plates and the resulting optimal connection details.

influence of out-of-plane motion caused by U-ribs onto the cross-beam.

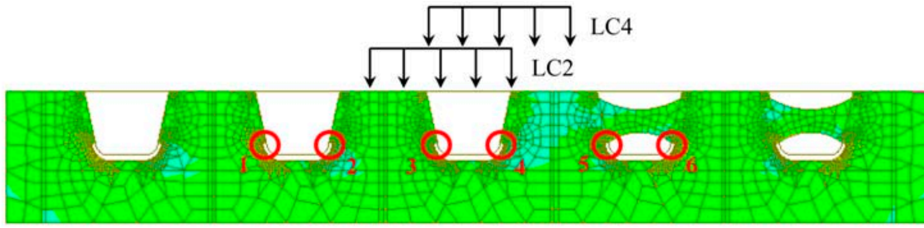
Deck thickness, however, is known to be a governing factor in deformation-induced fatigue cracks rather than force-induced fatigue cracks appearing at the root of welding where the U-rib, cross-beam and deck meet together, e.g., ‘A’ in Fig. 4(a). (Wolchuk, 1999)

2.5. Parametric study by installation of bulkhead plates

A lesson learned from the parametric studies in the previous subsections is that increasing the shear area of the cross-beam is very effective in reducing the maximum principal stress. In this subsection, a bulkhead plate, also known as an internal diaphragm, is taken into consideration as an alternative (See Fig. 4(a)).

The bulkhead plate is installed inside the U-ribs in order to resist distorsion of the associated U-rib under external loads. It was introduced on 1998 to retrofit Williamsburg Bridge in US (Bocchieri and Fisher, 1998), but rarely attracted Korean engineers due to densely concentrated welding. To avoid the heavy welding especially in regions ‘A’ in Fig. 4(a) where all components of the deck plate, U-rib and cross-beam meet together, the bulkhead plate is brought down as lower as possible until the tail end reaches the tip of scallop (See Fig. 4(a) ‘B’). Attaching the bulkhead plate to this end would restrict the deformation-induced fatigue near point ‘B’ in Fig. 4(a).

As for the shapes of the bulkhead plates, trapezoidal and curved shapes with radii of curvature of either 200 or



(a) FEM model for experimentation

(b) Elimination of the welding bead

(c) Bulkhead plate installed inside a U-rib

(d) Static load test

Figure 5. Details of the experimental model and a static load test.

300 mm for both R_1 and R_2 in Fig. 4(b) are attempted. Height of the trapezoidal bulkhead plate is 110 mm, because it makes the area of trapezoidal bulkhead plate identical with the largest area of curved one.

Installation of the bulkhead plates plays an important role in mitigating stress concentration by resisting shear forces in the cross-beam. The concentrated stresses near a scallop in the cross-beam are redistributed into the bulkhead plates, which leads to the stress reduction.

The maximum principal stress plummets from 68.5 to 54.6 MPa when the bulkhead plates are set up and it corresponds to 20.30% reduction as shown in Fig. 3(f). The magnitude of reduction acquired from bulkhead plates is 13.9 MPa, which is bigger than that obtained by expanding the height of the cross-beam from 500 mm into 550 mm, i.e., 11.4 MPa.

Four kinds of curved bulkhead plates having different R_1 and R_2 combinations, i.e., (R_1, R_2)=(200, 200 mm), (200, 300 mm), (300, 200 mm) and (300, 300 mm), are employed to test the effect of curvature. The maximum principal stresses for theses four cases are described in Fig. 4(c) along with that without bulkhead plates. The results address that (R_1, R_2)=(300, 200 mm) combination is optimal among four candidates. Notice although the maximum principal stress of a (R_1, R_2)=(300, 300 mm) case is 0.01 MPa smaller, the authors judge that this difference is of little importance and it is advisable to select lighter one.

Stress distribution inside the bulkhead plate in Fig. 4(d) and 4(e) demonstrates that both plates behave as diagonal bracings and it explains the curved bulkhead plate having variations of area, i.e., smaller area in the middle and larger one at the end, outperforms the trapezoidal one. So the advantages of bulkhead plates would be maximized when sufficient extent of area could be guaranteed by adjusting the curvature of the plates at the top and bottom.

The analysis results indicate that installation of the curved bulkhead plates is not only effective but also efficient in reducing the magnitude of the maximum

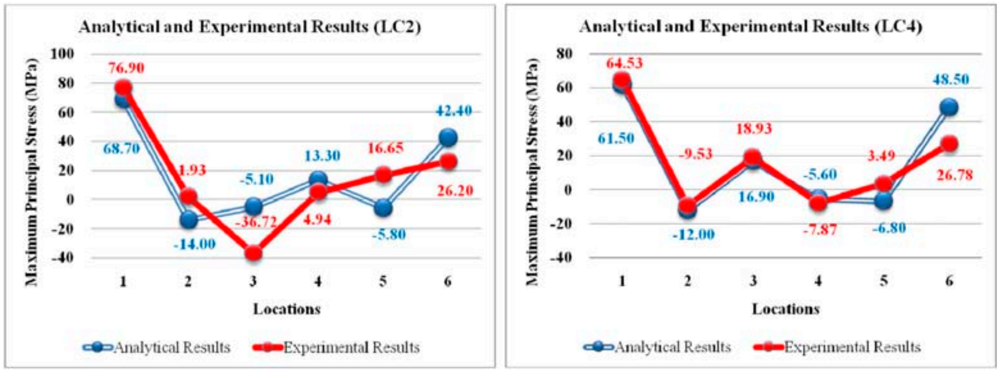


Figure 6. Comparison between analytical and experimental results for LC2 and LC4.

principal stress by a slight increment in self weight.

2.6. Optimal structural details based on parametric study

Based on the analytical results so far achieved, an optimal connection detail can be demonstrated in Fig. 4(f) and summarized as follows:

(1) the bulkhead plates are recommended instead of increasing shear area of the cross-beam;

(2) a curved bulkhead plate with 300 and 200 mm radii of curvature at the top and bottom of the plate, respectively, is suggested.

In the next section, entire analytical findings are verified by experimentation.

3. Experimental Verification

A laboratory investigation involving static load tests is performed to study the effectiveness of the modified connection details.

3.1. Experimental model

A model for experimentation is identical with the model for the FEM analysis in overall geometry as well as size except that only two consecutive U-ribs at one side contain bulkhead plates (See Fig. 5(a)). This asymmetrical model is employed to investigate the effects of stress reduction with and without the bulkhead plates using one experimental model. Note that the external load to induce the maximum principal stress is also asymmetrical, i.e., shifted to one side as LC2 and LC4 in Fig. 5(a).

The welding beads at the end of scallop are eliminated by grinding to prevent unexpected stress concentration as shown in Fig. 5(b). Figure 5(c) shows the bulkhead plate installed by fillet welding inside a U-rib. Attention was paid to remove welding beads to avoid stress concentration as before.

21 numbers of strain gauges including uniaxial and triaxial gauges are attached to the experimental model and static load tests are performed as shown in Fig. 5(d). Note that the strain gauges are not put on both sides of the cross-beam because an out-of-plane motion is not predicted. In the future, it is necessary to conduct additional experiments on a 3-dimensional model in order to investigate out-of-plane motions.

3.2. Comparison between analytical and experimental results

A static load test using the asymmetrical model is carried out to compare experimental results with those obtained from FEM analyses. Since the configuration of the experimental model is modified, the associated FEM model is also revised as shown in Fig. 5(a).

Magnitude of the external load is set to be 124.8 kN applied to the same area of 58 cm$\times$23 cm as before. The maximum principal stresses at the regions numbered 1 to 6 in Fig. 5(a) are measured under two critical load cases LC2 and LC4 and plotted in Fig. 6 for each load case.

As shown in Fig. 6, the analytical results agree qualitatively with the measured values. Tensile and compressive stresses are measured alternatively along the points of interest from 1 to 6. As for the maximum principal stresses, measured data differ from FEM results by 10.66 and 4.70% at most in case of LC2 and LC4, respectively. The largest percentage discrepancy is observed right beneath the external loading, because the stress distribution around the region is extremely complicated as mentioned before in subsection 2.2. However, when the bulkhead plate redistributes the concentrated stresses in the vicinity of loading, the observed difference becomes decreased. Considering the fact that the stress states around the scallop is very protean, the authors judge that this agreement is profound in meaning.

4. Conclusion

This paper presented an optimal connection detail of the orthotropic steel deck system by performing parameter studies. The parameters under investigation are height, thickness and shear area of a cross-beam and shapes of bulkhead plates. The optimal structural detail needs to maintain a balance between two primary objectives:

minimizing the principal stress leading to fatigue failure without increasing the self weight significantly.

The analytical and experimental results presented in the paper indicate that

(1) increasing the height, thickness or shear area of cross-beam is able to reduce the maximum principal stress;

(2) thickness of a deck plate is not closely associated with maximum principal stress reduction;

(3) bulkhead plates is capable of decreasing significant amount of the concentrated stress (20~28%) without sacrificing the self weight;

(4) curved bulkhead plates are more efficient than a trapezoidal one of the same area.

The aforementioned two primary goals are achieved by installing bulkhead plates inside U-ribs. The bulkhead plates prove to be effective as well as efficient in reducing the magnitude of maximum principal stresses without actually increasing the self weight. Based on the analytical results, the optimal parameters defining bulkhead plates are also proposed as shown in Fig. 4(f) and then FEM results are confirmed experimentally.

As for other parameters that are not dealt with in this study, it is advisable to follow current design specification at the moment. Additional studies are underway to investigate the effects of scallop curvature considering bulkhead plate curvature simultaneously, out-of-plane motion that can be observed from a 3-dimensional model, various surfacing such as traditional asphalt pavement, ultra high performance concrete, ductile fiber reinforced cementious composite, etc. A fatigue test of a 3-dimensional bridge model made out of high performance steel for bridges will be carried to gain further insight into the efficiency of the proposed connection detail.

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References

AASHTO (2007). *AASHTO LRFD Bridge Design Specifications.* American Association of State Highway and Transportation Officials, Washington D.C., USA.

Bocchieri, W. J. and Fisher, J. W. (1998). *Williamsburg bridge replacement orthotropic deck as-built fatigue test*. ATLSS Report No. 98-04, Lehigh University, Bethlehem, PA, USA.

Connor, R. J., Richards, S. O., and Fisher, J. W. (2003). *Long-term Monitoring of Prototype Orthotropic Deck Panels on the Bronx-Whitestone Bridge for Fatigue Evaluation in Recent Developments in Bridge Engineering.* A. A. Balkema, New York, NY, USA, pp. 257-268.

Dexter, R. J. and Fisher, J. W. (1997). "Fatigue cracking of orthotropic decks." *Proc. IABSE workshop: evaluation of existing steel and composite bridges*, International Association for Bridge and Structural Engineering, Innsbruck, Austria.

Fuchs, H. O. and Stephens, R. I. (1980). *Metal Fatigue in Engineering.* Wiley, New York, NY, USA.

KSCE (2010). *Design Specifications for Highway Bridges.* Korean Society of Civil Engineers, Seoul, Korea (in Korean).

KSSC (2010). *Design Specifications for Steel Structures.* Korean Society of Steel Construction, Seoul, Korea (in Korean).

Mangus, A. R. and Sun, S. (1999). "Orthotropic Deck Bridges." *Bridge Engineering Handbook*, CRC Press, Boca Raton, FL, USA.

MOCT (2006). *Guide Lines for the Design of Details of Steel Highway Bridges.* Ministry of Construction and Transportation, Kyonggi-do, Korea (in Korean).

Rolf, S. T. and Barsom, J. M. (1977). *Fracture and Fatigue Control in Structures-Application of Fracture Mechanics.* Prentice-Hall Inc., Englewood Cliffs, NJ, USA.

Tsakopoulos, P. A. and Fisher, J. W. (2004). "Full-scale fatigue tests of steel orthotropic deck panel for the Bronx-Whitestone Bridge rehabilitation." *Journal of Bridge Structures*, 1(1), pp. 55-66.

Wolchuk, R. (1999). "Steel orthotropic decks: developments in the 1990s." *Journal of the Transportation Research Board*, 1688, pp. 30-37.