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Design of Joint in Hybrid Girder
Combining Steel and PSC Members

2016년 8월

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Abstract

Design of Joint in Hybrid Girder
Combining Steel and PSC Members

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Hybrid structure is incorporating steel member and concrete member in longitudinal direction. Hybrid structure has some advantages. First, hybrid structure can make main span longer by adopting steel or steel-concrete composite girder as main span and concrete or PSC girder as side span. Second, in case that bridge has asymmetric span because of environmental condition or constructing causes, huge negative reaction may occur at side span so extra devices must be used to controlling it. In hybrid structure, negative reaction can be controlled by adopting concrete or PSC girder as side span without extra devices. On the other hand, some problems may take place at joint between steel girder and concrete girder. Angle refraction or
stress concentration because of rigidity difference may occur at the joint. But there are no design codes and standards about hybrid bridge joint so most design of joint were very conservative.

In this thesis, finite element analysis was conducted for design of hybrid girder joint. Girders for finite element analysis had different joint types, loading conditions and failure modes. As analysis results, analysis with linear tension softening model and parabolic compressive model is the most appropriate.

Based on finite element analysis results, parametric study was conducted. Girder section was from girder experimented by other researcher. To act only bending moment at joint part, 4-point load system was used. Parameters were spacing between shear stud connector, joint length, number of shear stud connector and area of prestressing tendon. As results of parametric study for spacing between shear stud connector and joint length, a factor which had influence on joint performance was not spacing between shear stud connector but joint length. If joint length was same, load-deflection relationship was almost same regardless of spacing between shear stud connector. However, if joint length was different, load-deflection relationship was also different. Generally the longer joint length, the larger maximum load is.

Number of shear stud connector had influence on degree of coupling between PSC girder and steel girder. If the number of shear stud connector is larger than specific number, behavior had little difference. From the results of
parametric study, the required minimum number of shear stud connectors for composite behavior of PSC girder and steel girder is when spacing between shear stud connectors is same with height of girder section.

Parametric study results for area of prestressing tendon showed that cracking moment maximum moment of hybrid girder were larger than those of PSC girder in every case. Cracking moments of girder with 3x2 and 3x3 shear stud connectors were similar and maximum moments of girder with 3x3 and 5x3 shear stud connectors were similar.

Keywords: Hybrid girder, Joint, Finite element analysis,

Parametric study

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Chapter 1. Introduction

1.1 Background

Composite structure consists of different materials to strengthen the strength and make up for the weakness. In civil engineering, generally steel and concrete are used for composite structure because they have almost same coefficient of thermal expansion. Composite beam, which is a representative composite structure, consists of I-shape steel girder at lower side, reinforced concrete slab at upper side and shear connection between them for composite behavior. Concrete may proof steel against rust, block out fire and prevent local buckling or lateral torsional buckling of steel. Steel may reinforce concrete tension load carrying capacity. Typical Composite beam is shown in Figure 1.1(a).

Mixed structure or hybrid structure is similar with composite structure in terms of consisting of different materials. But in hybrid structure steel member and concrete member are connected in longitudinal direction unlike composite structure. Hybrid structure has some advantages in comparison with other structure. First, hybrid structure can make main span longer by adopting steel girder as main span and concrete girder as side span. Generally steel is lighter than concrete but more expensive. So steel bridge can have a longer main span than concrete bridge but make cost more expensive. Hybrid bridge can have both advantages. Second, sometimes bridges should have asymmetric span because of environmental condition or constructing causes.
In this case, huge negative reaction may occur at side span so extra devices must be used to controlling it. However in hybrid structure, negative reaction can be controlled by weight difference between steel at main span and concrete at side span without extra devices. Typical hybrid structure is shown in Figure 1.1(b).

![Figure 1.1 Structure consisting of different materials: (a) composite structure; and (b) hybrid structure](image)

On the other hand, joint between steel girder and concrete girder of hybrid structure may be a vulnerable point because members with different materials are stuck. Angle refraction may occur at the joint because of rigidity difference between members and stress concentration also may occur if force flow between members does not go on smoothly. Therefore design of hybrid structure joint requires meticulous attention. However currently design code and standard about hybrid structure joint have not been established specifically and consequently most design of joint are very conservative or following previous construction design without extra verification. This is why research about hybrid bridge girder joint is required.
1.2 Objectives and scope

The final goal of this research is to propose design of hybrid girder joint. To do this, the specific objectives were defined.

- Development of finite element model for hybrid girder joint: Based on experimental data, finite element analysis has been conducted and its validity was verified.
- Parametric study for design of hybrid girder joint: Using the finite element model developed for hybrid girder joint, parametric study has been conducted.
- Proposal design of hybrid girder joint: Based on parametric study results, design of hybrid bridge girder joint is proposed.

1.3 Organization of the thesis

The research presented in this thesis consists of six main chapters. Brief explanation of each chapter is followed.

Chapter 1 introduces background, objectives and scope of the research. Basic concept of hybrid structure and necessity of this research are described. And specific objectives, scope of this research and structure of this thesis are also informed.

In chapter 2, literature reviews for hybrid structure were described.
Design code for hybrid structure in Korea and Japan, connection design of hybrid structure were described. After that, researches about joint of hybrid girder in Korea were described and those limitation and research topic were described.

In chapter 3, basic concepts of four types of hybrid girder joint are presented. After that, design procedures of joint, which include how to determine joint type and location and to design section of joint, are described.

In chapter 4, finite element modelling technique for hybrid girder joint is summarized. Finite element analysis has been conducted using commercial software program and analysis results have been verified by comparison with experimental results.

In chapter 5, parametric study for design of hybrid girder joint has been conducted based on finite element analysis results in chapter 4. Based on parametric study results, design of joint and design procedure have been proposed.

Chapter 6 summarizes the conclusions and major findings of this research and recommendations for further research are also proposed.
Chapter 2. Literature Review

2.1 Introduction

Construction cases of hybrid cable-stayed bridges are listed in Table 2.1. As seen in Table 2.1, cable-stayed bridges adopting hybrid structure have been constructed in Europe since 1970’s and recently constructed in Korea. However currently design code and standard about hybrid structure joint have not been established specifically.

Table 2.1 Construction cases of hybrid cable-stayed bridge

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Country</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Worldcup bridge</td>
<td>Korea</td>
<td>2015</td>
</tr>
<tr>
<td>Kimsimin bridge</td>
<td>Korea</td>
<td>2013</td>
</tr>
<tr>
<td>Cheongpoong bridge</td>
<td>Korea</td>
<td>2010</td>
</tr>
<tr>
<td>Kiso-gawa bridge</td>
<td>Japan</td>
<td>2002</td>
</tr>
<tr>
<td>Ibi-gawa bridge</td>
<td>Japan</td>
<td>2002</td>
</tr>
<tr>
<td>Tatara bridge</td>
<td>Japan</td>
<td>1999</td>
</tr>
<tr>
<td>Sun-marine bridge</td>
<td>Japan</td>
<td>1996</td>
</tr>
<tr>
<td>Normandie bridge</td>
<td>France</td>
<td>1995</td>
</tr>
<tr>
<td>Iguchi bridge</td>
<td>Japan</td>
<td>1990</td>
</tr>
<tr>
<td>Flehe bridge</td>
<td>German</td>
<td>1979</td>
</tr>
<tr>
<td>Bybrua bridge</td>
<td>Norway</td>
<td>1978</td>
</tr>
<tr>
<td>Kurt-Schumacher bridge</td>
<td>German</td>
<td>1972</td>
</tr>
</tbody>
</table>
2.2 Design code for hybrid structure

2.2.1 Hybrid structure design and construction manual (Japan)

Hybrid structure design and construction manual suggest that joint must not reach limit state ahead of other parts. In example, Eq. (2.1) is applied for ultimate limit state.

\[ F_{jd} > f_{d \text{ min}} \]  

(2.1)

where,

\( F_{jd} \) : ratio of resistance to load of joint part

\( f_{d \text{ min}} \) : minimum ratio of resistance to load of the other parts

The ratio of resistance to load is calculated by Eq. (2.2).

\[ F_d = \frac{R / \gamma_b}{S_d / \gamma_a} \]  

(2.2)

where,

\( R \) : resistance force

\( \gamma_b \) : resistance factor

\( S_d \) : design load

\( \gamma_a \) : load factor
2.2.2 Korean highway bridge design code (2010) (Korea)

In Korean highway bridge design code, specific design code for hybrid structure is not present. Just design codes for composite structure are present. Among them, design codes for shear stud connector may apply to hybrid structure.

The maximum distance between shear stud connectors is three times the thickness of concrete slab or 600 mm. The minimum distance between shear stud connectors for bridge longitudinal direction is five times the diameter of shear stud connector or 100 mm. The minimum distance between shear stud connectors for direction perpendicular to bridge is diameter of shear stud connector +30 mm.

The allowable shear force of shear stud connector is calculated by Eq. (2.3). Adhesion between slab concrete and steel girder flange is ignored.

\[
Q_a = 9.5d^2 \sqrt{f_{ck}} \quad (H/ d \geq 5.5)
\]
\[
Q_a = 1.74dH \sqrt{f_{ck}} \quad (H/ d < 5.5)
\]

(2.3)

where,

\( Q_a \): allowable shear force of shear stud connector

\( H \): total height of stud, 150 mm is a standard

\( d \): diameter of stud, 19 mm, 22 mm, 25 mm

\( f_{ck} \): concrete design compressive strength (MPa)
2.3 Connection design of hybrid structure

There are two methods to transfer bending moment when two girders are connected. The first one is to use prestressing tendon of PSC girder and the second one is to use shear stud connectors at the connection.

2.3.1 Connection using shear stud connector

In case of using shear stud connectors to resist bending moment as seen in Figure 2.1, compressive force and tensile force are transferred through shear stud connectors. Required numbers of shear stud connectors is calculated by Eq. (2.4).

\[ M_a = n_s Q_a d \] 

(2.4)
where,

\[ M_u : \text{design moment at joint} \]
\[ Q_a : \text{allowable shear force of shear stud connector} \]
\[ n_A : \text{number of shear stud connectors at upper flange or lower flange} \]
\[ d : \text{moment arm distance} \]

When steel girder and reinforced concrete column are connected, the connection has to be designed to resist bending moment and shear force as seen in Figure 2.2. Shear stud connectors are classified into two groups as their location and role. Shear stud connectors at concrete column resist bending moment. Required number of shear stud connectors at concrete column can be calculated by Eq. (2.5). Shear stud connectors at steel girder resist shear force. Required number of shear stud connectors at steel girder can be calculated by Eq. (2.6)

![Figure 2.2 Connection using shear stud connectors between girder and column](image-url)
\[ M_u = n_A Q_a d \]  \hspace{1cm} (2.5)

where,

- \( M_u \): design moment at joint
- \( Q_a \): allowable shear force of shear stud connector
- \( n_A \): number of shear stud connectors at each side of column
- \( d \): moment arm distance

\[ Q_u = n_B Q_a \]  \hspace{1cm} (2.6)

where,

- \( Q_u \): design shear force at joint
- \( n_B \): number of shear stud connectors at girder
- \( Q_a \): allowable shear force of shear stud connector

### 2.3.2 Connection using prestressed tendon

When PSC girder and steel girder are connected as seen in Figure 2.3, compressive force can be transferred through contacting from concrete to steel and tensile force can be transferred through prestressing tendon. Then ultimate bending moment that the connection can resist is calculated by Eq. (2.7).

\[ M_u = T_p \left( d_p - \frac{a}{2} \right) \]  \hspace{1cm} (2.7)
2.4 Researches about joint of hybrid girder

Researches about joint of hybrid girder are classified into two categories. The first category is researches using finite element method. Researches of this category have developed finite element methods for hybrid girder joint and investigated its validity. The other category is researches to improve joint performance. Researches of this category focus on modifying joint shape or using other shear connectors instead of shear stud connectors to improve joint performance.

2.4.1 Researches using finite element method

Kwon, H. J., et al. (2006, 2009) researched finite element analysis technique for steel-concrete hybrid girder. They used nonlinear interface element for describing partial-interaction behavior at the steel-concrete contact surface. Material property of interface element was gotten from push-out test. The analysis technique was verified by comparing with test results. This technique was very simple and easy to use but getting precise material
property of interface element was very hard.

Figure 2.4 Finite element model in Kwon, H. J., et al. research (Kwon, H. J., 2006)

Nguyen, H. T., and Kim, S. E., (2009, 2010 and 2012) studied behavior of hybrid girder joint using finite element method. Standard push-out test for shear stud connector was simulated using 3D finite element analysis. After that, they have conducted an analysis of hybrid girder joint and verified by comparing with test results. Based on the analysis results, parametric study was conducted to determine the effective connection type for three connection types used in hybrid girder.
2.4.2 Researches to improve joint performance

Yun, I. J., et al. (2008a, 2008b) developed a new advanced joint type. To develop more effective joint, finite element analysis was conducted for several types joint. Based on the analysis results, experiment for two specimens was conducted. The experimental results showed that proposed model had greater stiffness under flexural loading had better structural performance.
Kim, S. H., et al. (2011a, 2011b) suggested to use perfobond ribs instead of shear stud connectors at hybrid girder joint. To verify performance of hybrid girder with perfobond ribs, hybrid girders with perfobond ribs or shear stud connectors were tested. The test results showed that proposed girder had somewhat higher beam stiffness and strength prior to reaching ultimate strength and constructional convenience also can be anticipated.
2.5 Limitation of previous researches

As seen in previous, there is no design code and design procedure for hybrid girder joint in Korea. So designer could use design code for stud connector of composite structure and follow hybrid bridge joint design of foreign country.

Research using nonlinear interface element had no accuracy. There are
big differences between analysis results and experimental results.

Research using 3D finite element analysis had good accuracy. The analysis results showed good agreement with experimental results. However in parametric study, used section and loading condition had no relation. The section was designed from real cable-stayed bridge. But the loading condition was just for simple support girder.

In experimental researches about hybrid bridge, finite element analysis had been conducted and compared with experimental results. But there is no specific instruction for finite element analysis like element type, element number, loading method, contact behavior, and etc.

In this thesis, finite element analysis had been developed. The developed finite element analysis showed good agreement with experimental result. And section design and loading condition for parametric study had been selected for loading state of cable stayed bridge under construction.
Chapter 3. Design of Hybrid Girder Joint

3.1 Introduction

In this chapter, basic concepts of each joint type and design procedure are presented. There are four types of hybrid girder joint: front plate type, filling concrete and front plate type, filling concrete and back plate type and filling concrete and front, back plate type. As seen in the Table 3.1, these days front plate type adopted at first is not used, filling concrete and back plate type and filling concrete and front, back plate type are used.

Table 3.1 Joint type of constructed hybrid bridges

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Country</th>
<th>Year</th>
<th>Joint Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Worldcup bridge</td>
<td>Korea</td>
<td>2020</td>
<td>Filling concrete and back plate type</td>
</tr>
<tr>
<td>Kimsimin bridge</td>
<td>Korea</td>
<td>2013</td>
<td>Filling concrete and front, back plate type</td>
</tr>
<tr>
<td>Cheongpoong bridge</td>
<td>Korea</td>
<td>2010</td>
<td>Filling concrete and front, back plate type</td>
</tr>
<tr>
<td>Kiso-gawa bridge</td>
<td>Japan</td>
<td>2002</td>
<td>Filling concrete and front, back plate type</td>
</tr>
<tr>
<td>Ibi-gawa bridge</td>
<td>Japan</td>
<td>2002</td>
<td>Filling concrete and front, back plate type</td>
</tr>
<tr>
<td>Tatara bridge</td>
<td>Japan</td>
<td>1999</td>
<td>Filling concrete and back plate type</td>
</tr>
<tr>
<td>Sun-marine bridge</td>
<td>Japan</td>
<td>1996</td>
<td>Filling concrete and back plate type</td>
</tr>
<tr>
<td>Normandie bridge</td>
<td>France</td>
<td>1995</td>
<td>Front plate type</td>
</tr>
<tr>
<td>Iguchi bridge</td>
<td>Japan</td>
<td>1990</td>
<td>Filling concrete and back plate type</td>
</tr>
<tr>
<td>Flehe bridge</td>
<td>German</td>
<td>1979</td>
<td>Filling concrete and front plate type</td>
</tr>
<tr>
<td>Bybrua bridge</td>
<td>Norway</td>
<td>1978</td>
<td>Front plate type</td>
</tr>
<tr>
<td>Kurt-Schumacher bridge</td>
<td>German</td>
<td>1972</td>
<td>Front plate type</td>
</tr>
</tbody>
</table>
3.2 Joint type of hybrid girder

3.2.1 Front plate type

Figure 3.1 shows front plate type. Front plate type is the oldest type. Bending moment and axial force are transferred through front plate and shear force is transferred through shear connector at the front plate. Front plate type is simple and easy to incorporate steel member and concrete member so it was used at the early age of hybrid-bridge construction. However angle refraction and stress concentration might happen at the joint so the front plate needs to be thicker. It is no more used now because other efficient types were developed. Construction cases of front plate type are Kurt-Schumacher bridge(1972), Bybrua bridge(1978) and Normandie bridge(1995).

3.2.2 Filling concrete and front plate type

Figure 3.2 shows filling concrete and front plate type. Filling concrete and front plate type was adopted for making up for the weakness of front plate
type. Bending moment and axial force are transferred through front plate and shear connector in the filling concrete and shear force is transferred through shear connector at the front plate. However shear connector and filling concrete behind front plate hardly perform a role so it is no different from front plate type. That’s why this type was just used in Flehe bridge(1979) and have not been used ever since then.

Figure 3.2 Filling concrete and front plate type

3.2.3 Filling concrete and back plate type

Filling concrete and back plate type is shown in Figure 3.3. Filling concrete and back plate type was also adopter for making up for the weakness of front plate type. Bending moment and axial force are transferred through back plate and shear connector in the filling concrete and shear force is transferred through shear connector at the back plate. Unlike filling concrete and front plate type, filling concrete of this type is effective for stress flow. If PSC girder and steel girder are combined, prestressed tendon of PSC girder could be used for connecting PSC girder and steel girder without extra
prestressed tendon. However, low workability because of narrow space in joint part is a big weakness. Iguchi bridge(1990), Sun-marine bridge(1996), Tatara bridge(1999) and Worldcup bridge(2020) adopted this joint type.

![Figure 3.3 Filling concrete and back plate type](image)

3.2.4 Filling concrete and front, back plate type

Filling concrete and front, back plate type is shown in Figure 3.4. Bending moment and axial force are transferred through shear connector in the filling concrete, filling concrete and front, back plate and shear force is transferred through shear connector at the front plate. Filling concrete and front, back plate type has an advantage that stress flow between steel member and concrete member is the most smooth. But there is a drawback that non shrink mortar has to be filled in the closed narrow space in joint part and it needs extra prestressed tendon for connection. Ibi-gawa bridge(2002), Kiso-gawa bridge(2002), Cheongpoong bridge(2010) and Kimsimin bridge(2013) adopted this joint type.
3.3 Design procedure of joint

3.3.1 Determination of joint type

Joint type of hybrid-bridge constructed in Japan and Korea are categorized into two groups depending on its section form and listed in Table 2. As seen in Table 2, section form has nothing to do with determination of joint type. In case of girder type, if Kiso-gawa bridge and Ibi-gawa bridge are excluded, the bridges with PSC and steel girder adopted filling concrete and back plate type and the others with RC and steel girder adopted filling concrete and front, back plate type. Actually, Kiso-gawa bridge and Ibi-gawa bridge are Extradosed bridges while others are cable-stayed bridges. Joint of the two bridges located at the point where axial force is very small so it needs extra prestressed tendon for connection. Therefore girder type of the two bridges at the joint is pretty similar with RC and steel. Consequently it is concluded that joint type is related to girder type.
Table 3.2 Construction cases of hybrid bridge in Korea and Japan

<table>
<thead>
<tr>
<th>Joint type</th>
<th>Filling concrete and back plate type</th>
<th>Filling concrete and front, back plate type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cases</td>
<td>Iguchi bridge</td>
<td>Kiso-gawa bridge</td>
</tr>
<tr>
<td></td>
<td>Tatara bridge, Worldcup bridge</td>
<td>Ibi-gawa bridge</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cheongpoong bridge</td>
</tr>
<tr>
<td>Girder type</td>
<td>PSC + Steel</td>
<td>PSC + Steel</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RC + Steel</td>
</tr>
<tr>
<td>Section form</td>
<td>Closed section</td>
<td>Closed section</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Open section</td>
</tr>
</tbody>
</table>
Figure 3.5 Iguchi bridge: (a) Bridge side elevation; (b) Main span section-Steel; and (c) Side span section-PSC
Figure 3.6 Tatara bridge: (a) Bridge side elevation; (b) Main span section-Steel; and (c) Side span section-PSC
Figure 3.7 Worldcup bridge: (a) Bridge side elevation; (b) Main span section-Steel; and (c) Side span section-PSC
Figure 3.8 Kiso-gawa/Ibi-gawa bridge: (a) Bridge side elevation; (b) Main span section-Steel; and (c) Side span section-PSC
Figure 3.9 Cheongpoong bridge: (a) Bridge side elevation; (b) Main span section-Steel; and (c) Side span section-RC
Figure 3.10 Kimsimin bridge: (a) Bridge side elevation; (b) Main span section-Steel; and (c) Side span section-RC
3.3.2 Determination of joint location

In general, two main considerations for determination of joint location are zero bending moment and control of negative reaction. Firstly, it is chosen the location where bending moment is almost zero based on analysis of completed state. Secondly, it is also chosen the location where negative reaction could be controlled effectively. Generally within the framework of controlling negative reaction, it is the best location under the pylon but if the joint is located under the pylon it is too complex. So it is common that joint is located around the pylon. Besides there are many considerations depending on construction circumstance, i.e. girder construction method and girder segment length.

Figure 3.11 Joint location of hybrid bridge: (a) Cheongpoong bridge; and (b) Worldcup bridge
3.3.3 Section design of joint

Section design methods for each force type are arranged. Joint have to resist every construction stage and completed state. Therefore designer has to do construction step analysis and completed state analysis firstly and select the maximum section force at joint from the analysis results. For bending moment and axial force, axial stress for the section can be calculated. If prestressed tendon is used for connection, maximum tensile force at the section must be lower than allowable tensile force of the tendon. If shear connectors in the filling concrete are used for connection, required number of shear connectors for maximum tensile or compression force can be calculated. For shear force, required number of shear connector to resist maximum shear force can be calculated. For torsion, shear flow theory is used. From the maximum torsion maximum shear stress and shear force due to torsion can be calculated using shear flow theory and required number of shear connector is calculated from the shear force.
Chapter 4. Development of Finite Element Model
for Hybrid Girder Joint

4.1 Introduction

In this chapter, finite element model for hybrid girder joint was developed. Commercial package program TNO DIANA was used. TNO DIANA is a popular analysis program in civil engineering. It provides several element types and material models required when analysis in civil engineering.

Finite element model was developed and verified based on previous researcher’s experimental data. The first hybrid girder was experimented by Yun, Ik Jung et al. The joint type was filling concrete and front, back plate type. The second one was tested by Kim, Sang-Hyo et al. The joint type was filling concrete and back plate type.

4.2 Summary of experimental program

4.2.1 Yun, Ik Jung et al (2008)

Yun, Ik Jung et al (2008) proposed the modified joint type and investigated the quality of the proposed joint type. The typical joint type was used in this study.

Geometry of the hybrid girder is shown in Figure 4.1. The girder had 5,400 mm span length and 4,800 mm clear span. Left side of the girder was
PSC girder, right side was steel girder and center of the girder was joint part, which type was filling concrete and front, back plate type. The PSC girder had rectangular section which dimension was 300 mm x 600 mm and the steel girder had I-shape section. The joint length at the center of the girder was 300 mm. D10 reinforcing steel bars were used for both longitudinal and transverse reinforcements in the PSC girder and the PSC girder was prestressed using two prestressing tendon which section area was 555 mm². Eight shear stud connectors were used at the outside of the joint and twelve shear stud connectors were used at the inside of the joint. The height and diameter of shear stud connectors were 50 mm and 16 mm.

Concrete compressive strength ($f_c$) was 31 MPa. Yield strength of structural steel, reinforcing steel bar and shear stud connectors ($f_y$) were 400 MPa. Ultimate strength of prestressing tendon ($f_{pu}$) was 1,900 MPa.

The girder was tested under four-point static loading. Two loading plates were placed at 500 mm apart from the center of the girder. The load was applied with a displacement control using a hydraulic actuator of 2,000 kN capacity and the displacement rate was 0.02 mm/sec.

LVDT was used to record the midspan deflection and crack gauge was used to record opening width of the joint. Four strain gauges at the PSC girder and three strain gauges at the steel girder were used to record girder deformation.
Figure 4.1 The first hybrid girder: (a) Girder elevation; (b) PSC section; (c) Steel section; and (d) Joint part
4.2.2 Kim, Sang Hyo et al (2011)

Kim, Sang Hyo et al (2011) proposed a suitable joint type for hybrid girder consisting of steel girder at the midspan and PSC girders at the supports. Seven girders with shear stud connectors or perfobond rib were tested under static loading or cyclic loading. Among them, a girder with shear stud connectors tested under static loading was used in this study.

Geometry of the hybrid girder is shown in Figure 4.2. The girder had 3,900 mm span length and 3,600 mm clear span. Each side of the girder was PSC girder and steel girder was located at the center of the girder. The joint part, which type was filling concrete and back plate type, was located at the each end of the steel girder. The PSC girder had rectangular section which dimension was 200 mm x 400 mm and the steel girder had I-shape section. The joint length at the center of the girder was 457 mm. D10 and D16 reinforcing steel bars were used for longitudinal and transverse reinforcements in the PSC girder and the PSC girder was prestressed using four prestressing tendon which was SWPC7B 12.7 mm. Thirty four shear stud connectors were used at the inside of the joint. The height and diameter of shear stud connector were 70 mm and 16 mm.

Concrete compressive strength \( f_c \) was 50 MPa. Yield strength of structural steel, reinforcing steel bar and shear stud connectors \( f_y \) were 400 MPa. Ultimate strength of prestressing tendon \( f_{pu} \) was 1,900 MPa.

The girder was tested under three-point static loading. A loading plate was placed at the center of the girder. The load was applied with a
displacement control using a hydraulic actuator of 2,000 kN capacity. The
displacement rate was 0.05 mm/sec until a crack was visible and after that it
was decreased to 0.02 mm/sec.

Seven LVDT were installed to measure the displacements for the various
positions. Strain gauges were attached to the steel girder and the PSC girder.
Figure 4.2 The second hybrid girder: (a) Girder elevation; (b) PSC section; (c) Steel section; and (d) Joint part
4.3 Finite element analysis

4.3.1 General

Finite element analysis has been conducted using the commercial finite element program DIANA. For the first girder, only a half of the girder was modelled to reduce analysis time because it had a symmetry section. For the second girder, a quarter of the girder was modelled because it was symmetry not only in section but also in longitudinal direction.

Figure 4.3 Finite element model: (a) A half modelling of the first girder; and (b) A quarter modelling of the second girder
4.3.2 Finite element type and mesh

4.3.2.1 Concrete, structural steel and shear stud connector

The concrete was modelled using HX24L solid element available in DIANA. The HX24L element is an eight-node isoparametric solid brick element. It is based on linear interpolation and Gauss integration.

The structural steel and shear stud connector were also modelled using HX24L solid elements. The real shape of shear stud connector was a cylinder shape but it was modelled to be a square pillar shape for the simplicity and reducing analysis time.

Mesh size of the section was 25 mm per side. From center of span to a half of clear span length, mesh size of longitudinal direction was 25 mm and the mesh size of the other parts was 50 mm.

4.3.2.2 Reinforcing steel

Reinforcing steel was modelled by embedded element. By using embedded element, there is no need to consider node share between reinforcing steel and concrete because the program automatically generate necessary nodes of reinforcing steel. Concrete and reinforcing steel were assumed to be fully bonded.

4.3.2.3 Prestressing tendon

Prestressing tendon was modelled by L2TRU truss element and shared nodes with concrete elements. Prestressing tendon could be modelled by embedded element like reinforcing steel. However if prestressing tendon was modelled by embedded element, prestressing force for concrete and steel composite action wasn’t functioning properly because prestressing tendon
couldn’t penetrate contact element between concrete and structural steel. Therefore prestressing tendon was modelled by truss element and tendon and concrete were assumed to do full composite action by sharing node.
Figure 4.4 Generated meshes of the first girder: (a) Half of the girder; (b) PSC Girder; and (c) Steel girder
Figure 4.5 Generated meshes of the second girder: (a) Quarter of the girder; (b) PSC Girder; and (c) Steel girder
4.3.3 Material models

4.3.3.1 Concrete

The total strain crack model was used as concrete material model. The total strain crack model, which was developed by Vecchio and Collins (1986) along the lines of the Modified Compression Field Theory and extended to three dimensional theory by Selby and Vecchio (1993), is based on smeared crack model and described the stress as a function of the strain.

As tensile behavior of concrete, linear tension softening curve, hordijk tension softening curve and exponential tension softening curve were implemented because these models have the most similar behavior with actual tensile behavior of concrete. Softening curves are based on fracture energy as shown in Figure 4.6. Concrete tensile strength and fracture energy were calculated by CEB-FIP Model Code 1990(Eq. (4.1)). Nakamura and Higai (2001) suggested that concrete compressive fracture energy was 250 times tensile fracture energy. The calculated properties were shown in Table 4.1.

As compressive behavior of concrete, Thorenfeldt compression curve and parabolic compression curve were implemented. Compression curves are shown in Figure 4.7. Thorenfeldt compression curve was described by Eq. (4.2) and parabolic compression curve was described by Eq. (4.3).
Figure 4.6 Concrete tensile behavior: (a) Linear tension softening curve; (b) Hordijk tension softening curve; and (c) Exponential tension softening curve
\[ f_{cm} = f_{ck0,m} \left( \frac{f_{ck}}{f_{ck0}} \right)^{2/3} \]

\[ G_f = G_{f0} \left( \frac{f_{cm}}{f_{cm0}} \right)^{0.7} \]  

(4.1)

Table 4.1 Concrete material properties

<table>
<thead>
<tr>
<th>Properties</th>
<th>1\textsuperscript{st} Girder</th>
<th>2\textsuperscript{nd} Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength ($f_{ck}$)</td>
<td>31 MPa</td>
<td>50 MPa</td>
</tr>
<tr>
<td>Young’s Modulus ($E_c$)</td>
<td>26,701 MPa</td>
<td>31,314 MPa</td>
</tr>
<tr>
<td>Tensile Strength ($f_t$)</td>
<td>2.98 MPa</td>
<td>4.1 MPa</td>
</tr>
<tr>
<td>Tensile Fracture Energy ($G_f$)</td>
<td>0.101 N/mm</td>
<td>0.14 N/mm</td>
</tr>
<tr>
<td>Compressive Fracture Energy ($G_c$)</td>
<td>25.25 N/mm</td>
<td>35 N/mm</td>
</tr>
</tbody>
</table>
Figure 4.7 Concrete compressive behavior: (a) Thorenfeldt curve; and (b) Parabolic curve
\[ f = -f_c \frac{\varepsilon_i}{\varepsilon_p} \left( \frac{n}{n-1 + \left( \frac{\varepsilon_i}{\varepsilon_p} \right)^{nk}} \right) \]  

(4.2)

\[ n = 0.80 + \frac{f_{cc}}{17} \]

Where  \( k = 1 \)  \((\text{if } 0 > \varepsilon > \varepsilon_p)\)

\[ k = 0.67 + \frac{f_{cc}}{62} \]  \((\text{if } \varepsilon \leq \varepsilon_p)\)

\[ f = \begin{cases} 
- f_c \frac{1}{3} \frac{\varepsilon_i}{\varepsilon_{c/3}} & \text{if } 0 \leq \varepsilon_i < \varepsilon_{c/3} \\
- f_c \frac{1}{3} \left( \frac{\varepsilon_i - \varepsilon_{c/3}}{\varepsilon_c - \varepsilon_{c/3}} \right) - 2 \left( \frac{\varepsilon_i - \varepsilon_{c/3}}{\varepsilon_c - \varepsilon_{c/3}} \right)^2 & \text{if } \varepsilon_{c/3} \leq \varepsilon_i < \varepsilon_c \\
- f_c \left( 1 - \frac{\varepsilon_i - \varepsilon_c}{\varepsilon_u - \varepsilon_c} \right)^2 & \text{if } \varepsilon_c \leq \varepsilon_i < \varepsilon_u \\
0 & \text{if } \varepsilon_u \leq \varepsilon_i 
\end{cases} \]

(4.3)
4.3.3.2 Steel, shear stud connector and prestressing tendon

It was assumed that steel, shear stud connector and prestressing tendon had elasto-plastic idealized stress-strain relationships as shown in Figure 4.8.

![Figure 4.8 Idealized stress-strain relationship](image)

4.3.4 Constraint conditions

Steel girder and PSC girder are composed by frictional force at the joint surface and shear stud connector. The frictional force is relatively small and to measure it is too hard, so it is generally ignored. On the other hand, shear stud connector is generally assumed to be fully bonded with concrete.

In the analysis, shear stud connector and concrete were assumed to be fully bonded and shear stud connector and structural steel were also assumed to do be fully bonded through sharing nodes.

Friction between structural steel and concrete was ignored but contact between them was not ignored. The contact analysis was conducted by using
Q12CT contact element to model zones of possible contact. A contact zone must be modelled by two types of contact elements: a surface with contacter elements and a surface with target elements.

### 4.3.5 Boundary conditions

The girders were simply supported in the experiment. In the analysis, fixed end boundary conditions, for which the translational displacements $T_1$, $T_2$ and $T_3$ were restricted, were placed at steel girder supports and roller end boundary conditions, for which the translational displacements $T_2$ and $T_3$ were restricted, were placed at concrete girder supports. Symmetric conditions were placed at symmetric boundary.

### 4.3.6 Solution method

The Newton-Raphson method was selected as solution method for nonlinear analysis. In the Newton-Raphson method, a fixed amount of load is applied in each increment. The prestressing force and self-weight was loaded by five steps under load control. After that, the external load was loaded under displacement control.
4.4 Verification of finite element model

4.4.1 Load-deflection relationship

For verification of finite element model, analysis results were compared with experimental results. Load-deflection relationships at the center of the girder have been compared.

4.4.1.1 Effect of concrete tensile model

Figure 4.9(a) shows effect of concrete tensile model to the load-deflection relationships of the first girder. The used concrete compressive model was parabolic model and concrete tensile model were exponential and linear tension softening model. As seen in Figure 4.9(a), in the early part of analysis, deflections of analysis were smaller than that of experiment because analysis overestimated tangent stiffness. This made a difference between experiment and analysis. However, analysis results of both tensile model had no difference. The first girder had concrete crushing failure at the upper part of interface between PSC girder and steel girder. So the concrete tensile model has little influence on the analysis results.

Figure 4.9(b) shows effect of concrete tensile model to the load-deflection relationships of the second girder. The used concrete compressive model was parabolic model and concrete tensile model were hordijk and linear tension softening model. Unlike first girder, the concrete tensile model has a big influence on the analysis results for the second girder. In the early
part of analysis, there is little difference because those tensile models have same parameters until crack occurs. After cracking, in case of hordijk tension softening model, the analysis had stopped so the maximum load of analysis was 70% of experiment. However, in case of linear tension softening model, the analysis had continued so the maximum load of analysis was same with that of experiment.

### 4.4.2.1 Effect of concrete compressive model

Figure 4.10 show the load-deflection relationships which indicate effect of concrete compressive model. The used concrete tensile model was linear tension softening model which showed the most appropriate results. The used concrete compressive models were parabolic model and Thorenfeldt model. As seen in Figures, in case of Thorenfeldt model, good agreements were observed between analysis and experiment in both girders before cracking. However after cracking, the analysis had stopped immediately so there were big differences between analysis and experiment.

In case of parabolic model, the analysis results before cracking were similar to that of Thorenfeldt model because those compressive models have similar parameters until maximum compressive stress. After cracking, unlike Thorenfeldt model, in case of parabolic model, the analysis has continued so the maximum loads were same with that of experiment in both girders.

Cracking load and ultimate load of analysis and experiment are listed in Table 4.2 ~ 4.3. As seen in the Tables, analysis results with linear tension softening model and parabolic model are the most appropriate.
Figure 4.9 Load-deflection relationship: Effect of concrete tensile model
(a) The first girder; and (b) The second girder
Figure 4.10 Load-deflection relationship: Effect of concrete compressive model (a) The first girder; and (b) The second girder
Table 4.2 Analysis results of the first girder

<table>
<thead>
<tr>
<th>Properties</th>
<th>Cracking</th>
<th>Ultimate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load (kN)</td>
<td>Deflection (mm)</td>
</tr>
<tr>
<td>Analysis</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Exp-Para</td>
<td>225</td>
<td>3.21</td>
</tr>
<tr>
<td>Lin-Para</td>
<td>226</td>
<td>3.22</td>
</tr>
<tr>
<td>Exp-Thor</td>
<td>225</td>
<td>3.21</td>
</tr>
<tr>
<td>Lin-Thor</td>
<td>226</td>
<td>3.22</td>
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<tr>
<td>Experiment</td>
<td>270</td>
<td>5.56</td>
</tr>
</tbody>
</table>

Table 4.3 Analysis results of the second girder

<table>
<thead>
<tr>
<th>Properties</th>
<th>Cracking</th>
<th>Ultimate</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Deflection (mm)</td>
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<tr>
<td>Analysis</td>
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</tr>
<tr>
<td>Hor-Para</td>
<td>66</td>
<td>1.8</td>
</tr>
<tr>
<td>Lin-Para</td>
<td>73</td>
<td>2.0</td>
</tr>
<tr>
<td>Hor-Thor</td>
<td>66</td>
<td>1.8</td>
</tr>
<tr>
<td>Lin-Thor</td>
<td>73</td>
<td>2.0</td>
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<tr>
<td>Experiment</td>
<td>70</td>
<td>2.3</td>
</tr>
</tbody>
</table>
4.4.2 Crack patterns and failure modes

As seen in the load-deflection relationships, analysis results with linear tension softening model and parabolic model are the most appropriate. So the crack patterns and failure modes of analysis using linear tension softening model and parabolic model are compared with experimental results.

The failure mode and crack pattern of the first girder is shown in Figure 4.11. The first crack occurred at the interface between PSC girder and joint part. After that, the flexural crack occurred at the PSC girder. Finally concrete compressive failure occurred at the upper concrete of the interface between PSC girder and joint part. To observe the crack pattern and failure mode, the maximum principal total strain $E_1$ and the minimum principal total strain $E_3$ are shown in Figure 4.12. The first crack occurred at the interface between PSC girder and joint part and concrete compressive failure occurred at the upper concrete of the interface between PSC girder and joint part same as experimental result.

The failure mode and crack pattern of the second girder is shown in Figure 4.13. The first crack occurred at the interface between the joint and the PSC girder. The tensile cracks extended along the shear stud connector and joint failure had occurred. The cracking load was determined to be 70 ~ 80 kN. The maximum load was approximately 200 kN. To observe the crack pattern and failure mode, the maximum principal total strain $E_1$ and the minimum principal total strain $E_3$ are shown in Figure 4.14. As you can see in the figures, the first crack occurred at the shear stud connector. The crack at the
shear stud connector extended to the joint part and joint failure had occurred like experiment. Unlike the first girder, concrete compressive failure did not occur.
Figure 4.11 Experimental results of the first girder: (a) Failure mode; and (b) Crack pattern
Figure 4.12 Analysis results of the first girder: (a) Principal strain $E_1$ at cracking load; and (b) Principal strain $E_3$ at maximum load
Figure 4.13 Experimental results of the second girder: (a) Failure mode; and (b) Crack pattern
Figure 4.14 Analysis results of the second girder: (a) Principal strain E1 at cracking load; (b) Principal strain E1 at maximum load; and (c) Principal strain E3 at maximum load
4.5 Concluding remarks

In this chapter, finite element model for hybrid bridge girder joint was developed using commercial package program TNO DIANA. To verify correctness of the finite element model, analysis for experiments of previous researchers were conducted and compared with experimental results. The chosen experiments had different joint type and different failure modes. The first one had filling concrete and front, back plate type and concrete compressive failure. The second one had had filling concrete and back plate type and joint failure. The analysis with various concrete tensile models and concrete compressive models had conducted. As analysis results, linear tension softening model and parabolic model are the most appropriate. Analysis results with these concrete material models had good agreement with experimental results.
Chapter 5. Parametric Study for Design of Hybrid Girder Joint

5.1 Introduction

In this chapter, parametric study has been conducted to determine design of hybrid girder joint. Girder section design for parametric study comes from research of previous researcher Kim, Sang Hyo et al.

The girder has been modelled based on finite element analysis of previous chapter, and analyzed with some parameters which mainly influence joint behavior. After parametric study, effective design of hybrid girder joint was proposed based on parametric study results.

5.2 Design of hybrid girder

5.2.1 Geometry

Geometry of the hybrid girder for parametric study is shown in Figure 5.1. The girder section was a small-scale beam, a quarter-scale version of an actual bridge section. The girder had 6,300 mm span length and 6,000 mm clear span. Left side of the girder was PSC girder and right side was steel girder. Filling concrete and back plate type joint was located at the center of the girder.

The PSC girder had rectangular section which dimension was 200 mm x
400 mm and the steel girder had I-shape section. The joint length at the center of the girder was 800 mm. D10 and D16 reinforcing steel bars were used for longitudinal and transverse reinforcements in the PSC girder and the PSC girder was prestressed using four prestressing tendon which was SWPC7B. Thirty four shear stud connectors were used at the inside of the joint. The height and diameter of shear stud connector were 70 mm and 16 mm. The design service load for joint was 61 kN-m of moment and 38 kN of shear force.

5.2.3 Material properties

Concrete compressive strength \( (f_c) \) is 50 MPa and structural steel, reinforcing steel and shear stud connector yield strength \( (f_y) \) is 400 MPa. D10 and D16 reinforcing steel bars were used for longitudinal and transverse reinforcements in the PSC girder and the PSC girder was prestressed using four prestressing tendon which was SWPC7B. Yield strength \( (f_{py}) \) of prestressing tendon was 1,580 MPa.
Figure 5.1 Geometry of hybrid girder for parametric study: (a) Girder elevation; (b) PSC section; (c) Steel section; and (d) Joint part
5.3 Finite element model

5.3.1 General

Finite element modelling has been conducted based on the analysis results of previous chapter using the commercial finite element program DIANA. Only half of the girder was modelled to reduce analysis time because it had a symmetry section.

![Finite element model of the hybrid bridge girder](image)

Figure 5.2 Finite element model of the hybrid bridge girder

5.3.2 Finite element type and mesh

5.3.2.1 Concrete, structural steel and shear stud connector

The concrete was modelled using HX24L solid element available in DIANA. The HX24L element is an eight-node isoparametric solid brick element. It is based on linear interpolation and Gauss integration.
The structural steel and shear stud connector were also modelled using HX24L solid elements. The real shape of shear stud connector was a cylinder shape but it was modelled to be a square pillar shape for the simplicity and reducing analysis time.

Mesh size of the section was 25 mm per side. From center of span to a half of clear span length, mesh size of longitudinal direction was 30 mm and the mesh size of the other parts was 60 mm.

5.3.2.2 Reinforcing steel

Reinforcing steel was modelled by embedded element. By using embedded element, there is no need to consider node share between reinforcing steel and concrete because the program automatically generate necessary nodes of reinforcing steel bar. Concrete and reinforcing steel bar were assumed to be fully bonded.

5.3.2.3 Prestressing tendon

Prestressing tendon was modelled by L2TRU truss element and shared nodes with concrete elements. Prestressing tendon could be modelled by embedded element like reinforcing steel. However if prestressing tendon was modelled by embedded element, prestressing force for concrete and steel composite action wasn’t functioning properly because prestressing tendon couldn’t penetrate contact element between concrete and structural steel. Therefore prestressing tendon was modelled by truss element and tendon and concrete were assumed to do full composite action by sharing node.
Figure 5.3 Finite element model of the hybrid girder: (a) Girder elevation; (b) PSC girder; and (c) Steel girder
5.3.3 Material models

5.3.3.1 Concrete

The total strain crack model was used as concrete material model. Linear tension softening curve shown in Figure 5.4 was implemented as tensile behavior of concrete and parabolic compression shown in Figure 5.5 curve was implemented as compressive behavior of concrete. Concrete tensile strength and fracture energy were calculated by CEB-FIP Model Code 1990. Concrete compressive fracture energy was calculated 250 times tensile fracture energy as suggested by Nakamura and Higai (2001). The calculated properties were shown in Table 5.1.

![Figure 5.4 Linear tension softening curve](image1)

![Figure 5.5 Parabolic compression curve](image2)
Table 5.1 Concrete material properties

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength ($f_{ck}$)</td>
<td>50 MPa</td>
</tr>
<tr>
<td>Young’s Modulus ($E_c$)</td>
<td>31,314 MPa</td>
</tr>
<tr>
<td>Tensile Strength ($f_t$)</td>
<td>4.1 MPa</td>
</tr>
<tr>
<td>Tensile Fracture Energy ($G_f$)</td>
<td>0.14 N/mm</td>
</tr>
<tr>
<td>Compressive Fracture Energy ($G_c$)</td>
<td>35 N/mm</td>
</tr>
</tbody>
</table>

5.3.3.2 Steel, shear stud connector and prestressing tendon

It was assumed that steel, shear stud connector and prestressing tendon had elasto-plastic idealized stress-strain relationship as shown in Figure 5.6.

![Idealized stress-strain relationship](image)

Figure 5.6 Idealized stress-strain relationship

5.3.4 Constraint conditions

Steel girder and PSC girder are composed by frictional force at the joint
surface and shear stud connector. The frictional force is relatively small and to measure it is too hard, so it is generally ignored. On the other hand, shear stud connector is generally assumed to be fully bonded with concrete.

In the analysis, shear stud connector and concrete were assumed to be fully bonded and shear stud connector and structural steel were also assumed to do be fully bonded through sharing nodes.

Friction between structural steel and concrete was ignored but contact between them was not ignored. The contact analysis was conducted by using Q12CT contact element to model zones of possible contact. A contact zone must be modelled by two types of contact elements: a surface with contacter elements and a surface with target elements.

5.3.5 Boundary conditions

The girders were simply supported in the experiment. In the analysis, fixed end boundary conditions, for which the translational displacements $T_1$, $T_2$ and $T_3$ were restricted, were placed at steel girder supports and roller end boundary conditions, for which the translational displacements $T_2$ and $T_3$ were restricted, were placed at concrete girder supports. Symmetric conditions were placed at symmetric boundary.

5.3.6 Solution method

The Newton-Raphson method was selected as solution method for nonlinear analysis. In the Newton-Raphson method, a fixed amount of load is
applied in each increment. The prestressing force and self-weight was loaded by five steps under load control. After that, the external load was loaded under displacement control.

5.4 Parametric study

5.4.1 Parameters

Spacing between shear stud connectors, joint length, number of shear stud connectors and area of prestressing tendon were selected as parameters.

5.4.2 Spacing between shear stud connectors & Joint length

The minimum spacing based on ‘Highway bridge design specification (2005)’ is 80 mm and the maximum spacing is 600 mm. If the minimum spacing is used, joint length is 440 mm which is 1.1 times height of girder section.

In this research, $1.2h$, $1.5h$ and $2.0h$ of joint length were selected as parameters. 85 mm, 120 mm and 170 mm were spacing between shear stud connectors in each cases. In addition, to investigate the effect of spacing between shear stud connectors, 85 mm and 120 mm spacing between shear stud connectors with $2.0h$ of joint length were selected as parameters. These parameters are listed in Table 5.2 and joint figure with these parameters are shown in Figure 5.7.

Load-deflection at the interface between PSC girder and steel girder are
shown in Figure 5.8. As seen in the figures, the longer joint length is, the larger the maximum load is. Based on these analysis results, parameter which has effect on joint performance is not spacing between shear stud connectors but joint length.
Figure 5.7 Parameter for spacing between stud connectors and joint length:
(a) 1.2h & 85 mm; (b) 1.5h & 120 mm; (c) 2.0h & 170 mm; (d) 2.0h & 120 mm; and (e) 2.0h & 85 mm
Table 5.2 Parameters for spacing between stud connectors and joint length

<table>
<thead>
<tr>
<th>Name</th>
<th>Area of tendon (mm²)</th>
<th>Joint length</th>
<th>Stud spacing (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>219_1.2h_85</td>
<td>219</td>
<td>1.2h</td>
<td>85</td>
</tr>
<tr>
<td>219_1.5h_120</td>
<td></td>
<td>1.5h</td>
<td>120</td>
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<tr>
<td>219_2.0h_170</td>
<td></td>
<td>2.0h</td>
<td>170</td>
</tr>
<tr>
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<td>2.0h</td>
<td>120</td>
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<td>2.0h</td>
<td>85</td>
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<tr>
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<td>1.2h</td>
<td>85</td>
</tr>
<tr>
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<td>120</td>
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</tr>
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</table>

*1.2h = 460 mm, 1.5h = 600 mm, 2.0h = 800 mm
Figure 5.8 Load-deflection relationship for spacing between stud connectors and joint length: (a) $A_p = 219 \text{ mm}^2$; (b) $A_p = 359 \text{ mm}^2$; and (c) $A_p = 658 \text{ mm}^2$
5.4.3 Number of shear stud connectors

To investigate effect of number of shear stud connectors, analysis with various numbers of column and row of shear stud connector were conducted. The number of columns was changed from 5 to 2 and the number of row was changed 3 to 1. Location of shear stud connectors and spacing between shear stud connectors of each case are shown in Figure 5.9 ~ 5.10. Figure 5.9 shows joint figure when joint length is $2.0h$ and Figure 5.10 shows when joint length is $1.5h$. Parameters for number of shear stud connectors are listed in Table 5.3.

Load-deflection relationships for number of stud connectors when joint length is $2.0h$ and $1.5h$ are shown in Figure 5.11 ~ 5.14. In Figure 5.11 and 5.13, load-deflection ships are categorized according to number of columns. In Figure 5.12 and 5.13, load-deflection ships are categorized according to number of rows. Maximum loads are listed in Table 5.4 ~ 5.5. In Table 5.4, maximum loads are categorized according to number of columns. In Table 5.4, maximum loads are categorized according to number of rows. As seen in the Figure and Table, if joint had 3, 4 or 5 columns of shear stud connector, load-deflection relationships of girder with 2 and 3 rows of shear stud connector were almost same. The maximum deflections were different in some cases but these were just because of analysis convergence problem. The maximum loads were almost same. And if joint had 2 or 3 rows of shear stud connector, load-deflection relationships of girder with 3, 4 or 5 columns of shear stud connector were almost same. From these analysis results, shear stud connector had influence on degree of coupling between PSC girder and steel girder.
the number of shear stud connectors is larger than required number for composite behavior of PSC girder and steel girder, influences of the number of shear stud connectors on the performance of joint are very small. Spacing between shear stud connectors in case of 3 ~ 5 columns is 120 ~ 340 mm. These are smaller than height of girder section. On the other hand, spacing between shear stud connectors in case of 2 columns is 480 ~ 680 mm. These are larger than height of girder section and maximum spacing between shear stud connectors. In conclusion, the required minimum number of shear stud connectors for composite behavior of PSC girder and steel girder is when spacing between shear stud connectors is same with height of girder section.
Figure 5.9 Location of shear stud connectors and spacing between shear stud connectors when joint length is $2.0h$: (a) 5 columns & 1 ~ 3 rows; (b) 4 columns & 1 ~ 3 rows; (c) 3 columns & 1 ~ 3 rows; and (d) 2 columns & 1 ~ 3 rows.
Figure 5.10 Location of shear stud connectors and spacing between shear stud connectors when joint length is $1.5h$: (a) 5 columns & 1 ~ 3 rows; (b) 4 columns & 1 ~ 3 rows; (c) 3 columns & 1 ~ 3 rows; and (d) 2 columns & 1 ~ 3 rows
Table 5.3 Parameters for number of shear stud connectors

<table>
<thead>
<tr>
<th>Name</th>
<th>Joint length (mm)</th>
<th>Stud column (EA)</th>
<th>Stud row (EA)</th>
<th>Total number of stud (EA)</th>
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<tr>
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<td></td>
<td>3</td>
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</table>

*1.5h = 600 mm, 2.0h = 800 mm
Figure 5.11 Load-deflection relationship for number of stud connectors when joint length is $2.0h$: (a) 5 Columns; (b) 4 Columns; (c) 3 Columns; and (d) 2 Columns
Figure 5.12 Load-deflection relationship for number of stud connectors when joint length is $2.0h$: (a) 3 Rows; (b) 2 Rows; and (c) 1 Row
Figure 5.13 Load-deflection relationship for number of stud connectors when joint length is $1.5h$: (a) 5 Columns; (b) 4 Columns; (c) 3 Columns; and (d) 2 Columns
Figure 5.14 Load-deflection relationship for number of stud connectors when joint length is $1.5h$: (a) 3 Rows; (b) 2 Rows; and (c) 1 Row
Table 5.4 Analysis results of parametric study for number of stud rows

<table>
<thead>
<tr>
<th>Name</th>
<th>Maximum load (kN)</th>
<th>Name</th>
<th>Maximum load (kN)</th>
</tr>
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<tbody>
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<td>1.5h_2C_1R</td>
<td>95</td>
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<td>129</td>
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<tr>
<td>2.0h_2C_3R</td>
<td>145</td>
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<td>138</td>
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Table 5.5 Analysis results of parametric study for number of stud columns

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<th>Name</th>
<th>Maximum load (kN)</th>
<th>Name</th>
<th>Maximum load (kN)</th>
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<tbody>
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<td>2.0h_2C_2R</td>
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<td>145</td>
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<td>1.5h_5C_3R</td>
<td>150</td>
</tr>
</tbody>
</table>
5.4.4 Area of prestressing tendon

Magnitude of prestress seems to have the biggest effect on performance of joint. Analysis with various area of prestressing tendon had been conducted to investigate the effect of magnitude of prestress.

The area of prestressing tendon must satisfy Eq. (5.1) and (5.2) to prevent sudden failure or concrete crushing failure. The minimum and maximum area of prestressing tendon calculated based on Eq. (5.1) and (5.2) were 130 mm² and 750 mm². In case that SWPC 7BN prestressing tendon is used, available area of prestressing tendon are calculated and listed in Table 5.6.

\[ M_u \geq 1.2M_{cr} \]  \hspace{1cm} (5.1)

where,

\[ M_u : \text{Factored moment} \]
\[ M_{cr} : \text{Cracking moment} \]

\[ \omega_p \leq 0.32\beta_i \]  \hspace{1cm} (5.2)

where,

\[ \omega_p : \text{Reinforcement index} \]

In chapter 5.4.3, there was almost no difference between girder with 5x3, 3x3 and 3x2 shear stud connectors. So parametric study for area of
prestressing tendon have been conducted about girder with 5x3, 3x3 and 3x2 shear stud connectors. Parameters are listed in Table 5.7.

Load-deflection relationships for area of prestressing tendon are shown in Figure 5.15. Analysis results of parametric study for area of prestressing tendon are listed in Table 5.6. Cracking moment and maximum moment of PSC girder were also calculated and compared. Cracking moment and maximum moment were larger than that of PSC girder in every case. In case of cracking moment, girders with 3x2 and 3x3 shear stud connectors were almost same. The difference between these girders and girder with 5x3 shear stud connectors became larger if area of prestressing tendon is larger. In case of maximum moment, unlike cracking moment, girders with 3x3 and 5x3 shear stud connectors were almost same. The difference between these girders and girder with 3x2 shear stud connectors became larger if area of prestressing tendon is larger. This is why cracking moment is affected by number of column of shear stud connectors and maximum moment is affected by number of row of shear stud connectors.
<table>
<thead>
<tr>
<th>Name of tendon</th>
<th>Number of tendons (EA)</th>
<th>Area of tendon (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWPC 7BN 9.5 mm</td>
<td>4</td>
<td>219</td>
</tr>
<tr>
<td>SWPC 7BN 11.1 mm</td>
<td>4</td>
<td>297</td>
</tr>
<tr>
<td>SWPC 7BN 12.7 mm</td>
<td>4</td>
<td>395</td>
</tr>
<tr>
<td>SWPC 7BN 9.5 mm</td>
<td>8</td>
<td>439</td>
</tr>
<tr>
<td>SWPC 7BN 11.1 mm</td>
<td>8</td>
<td>594</td>
</tr>
<tr>
<td>SWPC 7BN 9.5 mm</td>
<td>12</td>
<td>658</td>
</tr>
<tr>
<td>Name</td>
<td>Area of prestressing tendon (mm²)</td>
<td>Stud column (EA)</td>
</tr>
<tr>
<td>--------</td>
<td>----------------------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>219_3C_2R</td>
<td>219</td>
<td>3</td>
</tr>
<tr>
<td>297_3C_2R</td>
<td>297</td>
<td></td>
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<tr>
<td>395_3C_2R</td>
<td>395</td>
<td></td>
</tr>
<tr>
<td>439_3C_2R</td>
<td>439</td>
<td></td>
</tr>
<tr>
<td>594_3C_2R</td>
<td>594</td>
<td></td>
</tr>
<tr>
<td>658_3C_2R</td>
<td>658</td>
<td></td>
</tr>
<tr>
<td>219_3C_3R</td>
<td>219</td>
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</tr>
<tr>
<td>297_3C_3R</td>
<td>297</td>
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<tr>
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<td>594_5C_3R</td>
<td>594</td>
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</tr>
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<td>658_5C_3R</td>
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</table>
Figure 5.15 Load-deflection relationship for area of prestressing tendon: (a) 3 Columns & 2 rows; (b) 3 Columns & 3 rows; and (c) 5 Columns & 3 rows
Table 5.8 Analysis results of parametric study for area of prestressing tendon

<table>
<thead>
<tr>
<th>Name</th>
<th>Cracking Moment (kNm)</th>
<th>Maximum Moment (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PSC</td>
<td>Hybrid Girder</td>
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<td>31</td>
<td>77</td>
</tr>
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<td>297_3C_2R</td>
<td>36</td>
<td>90</td>
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<td>395_3C_2R</td>
<td>41</td>
<td>111</td>
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<tr>
<td>439_3C_2R</td>
<td>44</td>
<td>113</td>
</tr>
<tr>
<td>594_3C_2R</td>
<td>53</td>
<td>145</td>
</tr>
<tr>
<td>658_3C_2R</td>
<td>56</td>
<td>156</td>
</tr>
<tr>
<td>219_3C_3R</td>
<td>31</td>
<td>78</td>
</tr>
<tr>
<td>297_3C_3R</td>
<td>36</td>
<td>91</td>
</tr>
<tr>
<td>395_3C_3R</td>
<td>41</td>
<td>113</td>
</tr>
<tr>
<td>439_3C_3R</td>
<td>44</td>
<td>124</td>
</tr>
<tr>
<td>594_3C_3R</td>
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<td>148</td>
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<tr>
<td>658_3C_3R</td>
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<td>159</td>
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<td>169</td>
</tr>
<tr>
<td>658_5C_3R</td>
<td>56</td>
<td>181</td>
</tr>
</tbody>
</table>
5.5 Design of hybrid girder joint

To design of hybrid bridge girder joint, joint length, spacing between shear stud connectors, number of shear stud connectors and area of prestressing tendon are determined.

The longer joint length, the larger girder performance is. The difference between $1.2h$ and $1.5h$ is small. But the difference between $2.0h$ and others is evidently large. Considering the efficiency of joint length, it is reasonable to determine $1.2h$ or $2.0h$ as joint length.

As a result of parametric study, spacing between shear stud connectors hardly had influence on joint performance if shear stud connectors were properly arranged. Spacing between shear stud connectors is not a main factor to design hybrid girder joint, but a secondary factor from joint length and number of shear stud connectors.

Generally the number of shear stud connectors at joint may be calculated by following procedure. In this thesis, design service moment ($M$) at joint was determined as 61 kNm. Center distance between the top and bottom shear stud connectors ($d$) was 0.3 m. From Figure 5.16, the tensile force acting on the top and bottom shear stud connectors ($T$) was calculated as 203 kN. The allowable shear force of shear stud connector ($Q_a$) calculated by Eq. (5.3) was 13.8 kN/EA. The required number of shear stud connectors calculated by Eq. (5.4) is 15 EA. The shear stud connectors were arranged by 5 columns and 3 rows at the top and bottom flange.
Figure 5.16 Connection using shear stud connectors between girders

\[ M_u = n_A Q_a d \]  \hspace{1cm} (5.3)

where,

- \( M_u \) : design moment at joint
- \( Q_a \) : allowable shear force of shear stud connector
- \( n_A \) : number of shear stud connectors at upper flange or lower flange
- \( d \) : moment arm distance

\[ Q_a = 9.5d^2 \sqrt{f_{ck}} \quad (H/d \geq 5.5) \]
\[ Q_a = 1.74dH \sqrt{f_{ck}} \quad (H/d < 5.5) \]  \hspace{1cm} (5.4)

where,

- \( Q_a \) : allowable shear force of shear stud connector
- \( H \) : total height of stud, 150 mm is a standard
- \( d \) : diameter of stud, 19 mm, 22 mm, 25 mm
- \( f_{ck} \) : concrete design compressive strength (MPa)
As results of parametric study, spacing between shear stud connectors must be smaller than height of girder section to be composite behavior. And number of column and row of shear stud connectors affect cracking moment and maximum moment respectively. Based on these results, at least, shear stud connectors at joint must be arranged by 3x2. If shear stud connectors is arranged by 3x3, girder behavior is almost same with girder arranged by 5x3.

It is not important how much area of prestressing tendon is used if it satisfy the minimum and maximum area of prestressing tendon of PSC girder because cracking moment and maximum moment are larger than those of PSC girder in every case.
Chapter 6. Conclusions and Recommendations

6.1 Summary and conclusions

Hybrid structure is combining steel and PSC members in longitudinal direction unlike composite structure. Hybrid structure has some advantages in comparison with other structure like longer main span or controlling negative reaction. However some problems like angle refraction or stress concentration may take place at joint part between steel girder and concrete girder.

In this thesis, effective design of hybrid girder joint was suggested. To find effective design, finite element model for hybrid bridge girder had been developed. To verify validity of finite element model, analysis for hybrid girders experimented by other researchers had been conducted. The girders had different joint type and failure mode. As analysis results, linear tension softening model as concrete tensile behavior and parabolic model as concrete compressive behavior are the most appropriate material models.

Based on finite element analysis, parametric study for effective design of hybrid bridge girder joint had been conducted. Girder section was designed from experiment by other researcher. 4-point loading system was adopted for only bending moment to act to girder joint part. Selected parameters were Spacing between shear stud connectors, joint length, number of shear stud connectors and area of prestressing tendon.

As results of parametric study for spacing between shear stud connectors and joint length, a factor which had influence on joint performance was not
spacing between shear stud connectors but joint length. If joint length was same, load-deflection relationship was almost same regardless of spacing between shear stud connectors. However, if joint length was different, load-deflection relationship was also different. Generally the longer joint length, the larger maximum load is.

The number of shear stud connectors had influence on degree of coupling between PSC girder and steel girder. If the number of shear stud connectors is larger than specific number, behavior had little difference. From the results of parametric study, the required minimum number of shear stud connectors for composite behavior of PSC girder and steel girder is when spacing between shear stud connectors is same with height of girder section.

Parametric study results for area of prestressing tendon showed that cracking moment maximum moment of hybrid girder were larger than those of PSC girder in every case. Cracking moments of girder with 3x2 and 3x3 shear stud connectors were similar and maximum moments of girder with 3x3 and 5x3 shear stud connectors were similar.

6.2 Further studies

In this study, design of hybrid girder joint was suggested based on finite element analysis. Although finite element model used in this study was verified by comparing experimental results by other researchers, parametric study result was not verified. So experiment for parametric study must be conducted.
Currently the required number of shear stud connectors is calculated following allowable stress design. However, as seen in parametric study results, it was confirmed that too many shear stud connectors were arranged. So it is needed how to calculate required number of shear stud connectors instead of allowable stress design.

Based on analysis results for girder under bending moment, it is needed how to apply to cable-stayed hybrid bridge. Generally cable-stayed hybrid bridge has a massive section and experiences various loading state. So it is impossible to experiment or conduct 3D finite element analysis for entire joint of cable-stayed hybrid bridge. Another research method for cable-stayed hybrid bridge must be prepared.
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초 록

하이브리드 구조는 이종재료로 이루어진 부재를 길이방향으로 접합하여 시스템을 구성하는 구조 형식이다. 이러한 하이브리드 구조를 채택한 사장교가 갖는 이점은 다음과 같다. 먼저 주간간 보강형인 가벼운 강거더 또는 강합성거더로 하고 측간간 보강형은 RC거더 또는 PSC거더로 구성함으로써 주간간의 길이를 보다 더 길게 할 수 있다. 두 번째로 주변 환경 또는 시공상의 이유로 부득이하게 단면 구성의 비대칭으로 해야 되는 경우 측간간에 부반력이 발생하게 되는데 측간간을 자중이 큰 RC거더 또는 PSC거더로 구성함으로써 부반력을 제어할 수 있다. 하지만 하이브리드 교량의 경우 서로 다른 재료로 이루어진 부재를 접합하는 형태이기 때문에 접합부가 구조적으로 가장 취약한 부분이 될 수 있다. 접합부에서 부재간의 강성이 급변함으로써 외력이나 차륜하중에 의해 골절각이 발생할 수 있고 서로 다른 부재를 접합함으로써 응력집중현상이 발생할 수 있다. 따라서 접합부 설계시 주의를 기울여야 한다. 현재 국내에는 이러한 하이브리드 교량 접합부에 대한 설계 기준이 마련되어있지 않기 때문에 접합부에 대한 과다설계가 이루어지고 있다. 본 연구에서는 하이브리드 거더 접합부의 설계를 위하여 유한요소해석을 실시하였다. 해석에 사용된 하이브리드 거더들은 기존 연구자들의 실험에 사용되었던 거더로서 서로 다른 접합부 형태와 하중재하, 그리고 파괴모드를 가지고 있다. 여러 가지
콘크리트 재료모델을 사용하여 해석결과와 실험결과를 비교한 결과, 선형인장연화모델과 포물선 압축모델을 사용한 해석결과가 실험결과와 가장 일치하는 것으로 나타났다.

다음으로 유한요소해석결과를 기반으로 하여 매개변수연구를 실시하였다. 매개변수연구에 사용된 거더의 단면은 다른 연구자의 실험에 사용된 거더의 단면이다. 그리고 중앙부에 위치한 접합부에 홀모멘트만 작용하도록 4점재하 방식을 채택하였다. 매개변수는 접합부 거동에 가장 큰 영향을 미치는 요소들로 접합부에 배치되는 전단연결재 간격, 접합부 길이, 전단연결재 개수, 긴장재량에 따른 긴장력의 크기이다. 매개변수연구 결과, 전단연결재 간격은 접합부 거동에 큰 영향을 미치지 않는 것으로 나타났다. 접합부 길이가 동일한 경우, 전단연결재 간격에 상관없이 하이브리드 거더의 거동이 비슷하게 나타났다. 반면에 접합부 길이는 하이브리드 거더 거동에 영향을 미치는데 접합부 길이가 길수록 하이브리드 거더의 최대하중이 증가하는 것으로 나타났다. 다음으로 전단연결재 개수는 강거더와 PSC거더의 합성정도에만 영향을 미치는 것으로 나타났다. 전단연결재가 합성에 필요한 개수보다 많이 배치될 경우 하이브리드 거더의 최대하중은 매우 작게 나타났다. 마지막으로 긴장재량이 증가함수록 하이브리드 거더의 균열하중과 최대하중이 증가하는 것으로 나타났다. 그리고 동일한 단면계원을 갖는 PSC 거더의 균열하중, 최대하중과 비교한 결과, 모든 경우에 대해서 하이브리드 거더의 균열하중, 최대하중이 큰 것으로 나타났다. 접합부에 배치된 전단연결재가 3열 2행, 3열 3행, 5열 3행인 경우에 대해서 해석을 실시한 결과, 균열하중의 경우 3열
2행과 3열 3행인 거더가 비슷하게 나타났고 최대하중의 경우 3열
3행과 5열 3행인 거더가 비슷하게 나타났다.

주요어: 하이브리드 거더, 접합부, 유한요소해석, 매개변수연구,
학번: 2009-30939