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Experimental evaluation of shear behavior of partially encased composite beams with replaceable bolted joints

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ABSTRACT

This study proposes a Replaceable Bolted Joint (RBJ) for Partially Encased Composite (PEC) beams to enhance reusability and support efficient precast construction. Five PEC beam specimens were tested under monotonic shear loading to evaluate the effects of three parameters: the number of slits in the connection plate, the bolt type, and the shear-span-to-effective-depth ratio (a/d). Slitted connection plates increased peak load by about 11% through stress redistribution but reduced energy dissipation (45–68%) due to a smaller effective area. High-strength bolts provided approximately 13% higher peak load and stiffness than threaded rods. Increasing the a/d ratio shifted the response from brittle shear-dominated behavior to more ductile flexure-dominated behavior with smoother post-peak degradation, while strain measurements confirmed reduced local stress concentration and early shear yielding in specimens with smaller a/d ratios. These findings demonstrate that the proposed RBJ provides a viable design strategy for replaceable connections, contributing to resilient structural design and efficient modular construction practices.

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KEYWORDS

Partially encased composite beam; modular construction; slit connection; dry joint; span-to-depth ratio

1. Introduction

The precast concrete (PC) construction method involves fabricating structural members in a controlled factory environment and assembling them on-site. This approach enables mass production of components while minimizing on-site work. Unlike conventional cast-in-place construction, it reduces variability associated with worker proficiency and weather conditions, leading to shorter construction periods and decreased construction waste, which enhances environmental sustainability. In the connection of PC members, the wet joint method – where mechanical splicing of reinforcing bars is followed by on-site casting – is most commonly employed. Although the wet method provides strong monolithic behavior, it has notable drawbacks, including the need for on-site casting and limited flexibility during construction and repair processes. In contrast, dry connection methods assemble members without casting, enabling efficient on-site assembly and improved constructability (Aninthaneni et al. 2018). C. Yang, Li, and Xie (2022) proposed a dry-connected rotational friction dissipative beam-to-column joint (DRFDBJ) applicable to PC frames and analyzed its structural behavior. They also suggested a design methodology for non-monolithic precast concrete frames (NPCF). Their results demonstrated that the NPCF exhibited superior seismic performance compared with

monolithic reinforced concrete (RC) frames and that the proposed design approach effectively achieved the intended seismic performance. Similarly, Hu, Hong, and Park (2016), proposed a dry-type column connection system using high-strength bolts and steel plates attached to the upper and lower column segments. Experimental results revealed that the proposed joint achieved a rigid connection exhibiting structural behavior comparable to that of monolithic columns while also allowing for convenient assembly and disassembly. In addition, torsional behavior has been investigated in connected reinforced concrete members and composite joints. Hamoda et al. (2023) studied reinforced concrete beam-to-steel column composite joints subjected to torsional moment, while Elkhoully et al. (2025) examined the torsional response of reinforced concrete stepped beams, highlighting the influence of geometric discontinuities on cracking behavior and stiffness.

A partially encased composite (PEC) beam consists of a steel H-section encased in concrete, which enhances the fire and corrosion resistance of the steel beam while the steel section provides confinement to delay concrete damage (Ahn and Lee 2017; Deng et al. 2025). Due to its high steel utilization efficiency, good assemblability, and reduced section depth, the PEC beam has been increasingly adopted

in recent years (Ye et al. 2022). More generally, composite action and design considerations in steel – concrete members have been extensively investigated at the member level, including thin-walled double-skin concrete-filled steel tubular columns (Ahmed et al. 2022). Despite its advantages, studies on PEC beam-to-beam dry joints remain limited, particularly regarding their shear performance. Numerous studies have been conducted to evaluate the mechanical behavior of PEC beams. Chen, Sudibyo, and Erwin (2019) experimentally compared the seismic performance of conventional steel beams and PEC beams, confirming that PEC beams exhibit superior resistance to lateral – torsional buckling and enhanced ductility. Their performance was further improved when coupled with external elements such as floor slabs. Jiang et al. (2016) compared the flexural performance of conventional steel – concrete composite beams and PEC continuous beams, showing that the latter effectively delayed flange yielding and prevented local buckling, thereby demonstrating excellent ductility. The application of corrugated webs to PEC beams can further improve the interfacial bonding between steel and concrete and enhance shear resistance. Chu et al. (2024) experimentally evaluated corrugated-web PEC beams (CWPEC) and confirmed their superior load-carrying and ductile performance. K. Yang et al. (2024) conducted cyclic loading tests on PEC beams with wavy webs, considering flange thickness and shear span ratio as variables. Their results indicated that PEC beams exhibited outstanding ductility, and thinner flanges helped limit crack propagation in concrete, resulting in improved energy dissipation capacity. In addition, studies have been conducted on hybrid systems combining PEC beams with cellular steel beams featuring web openings. This structural configuration effectively reduces self-weight and material usage and facilitates the integration of mechanical and electrical services, thereby improving spatial efficiency. Wang et al. (2025) Liang et al. (2025) experimentally investigated the effects of web-opening shapes and numbers, reporting that circular openings alleviated stress concentration and improved ductility (Wang et al. 2023). further verified that such structures could effectively transfer shear forces and maintain sufficient flexural performance even without shear connectors.

Given the superior composite action between steel and concrete, the Partially Encased Composite (PEC) beam offers improved stiffness, load-bearing capacity, and fire and corrosion resistance compared to conventional steel or reinforced concrete beams. Furthermore, its high steel utilization efficiency and reduced section depth make it particularly suitable for modular or precast structural systems, where both performance and constructability are critical. Accordingly, this study proposes a replaceable bolted

joint (RBJ) applicable to PEC beams, aiming to enhance the adaptability and sustainability of precast concrete (PC) systems. The proposed RBJ connects two PEC beams through a steel plate attached to each beam and an additional connecting plate, enabling dry assembly via simple bolting on-site. The joint is designed to allow controlled deformation and energy dissipation within the connection region, ensuring that the overall behavior of the member remains stable and undamaged while enabling easy replacement of the joint components and reuse of undamaged members (Zhang, Hou, and Wang 2025). To verify the structural performance of the proposed RBJ, five PEC beam specimens incorporating RBJs were fabricated and tested under shear loading.

2. Description of the Replaceable Bolted joint (RBJ)

2.1. Concept and configuration of the RBJ

As illustrated in Figure 1, the proposed Replaceable Bolted Joint (RBJ) consists of three main components:

- (1) a steel plate welded to both ends of the PEC beam,
- (2) a connection plate interlinking the adjacent beam modules, and
- (3) bolts fastening the two plate components together.

The steel plate is welded to the flange of the H-section steel within the PEC beam to ensure smooth load transfer to the connection plate. The connection plate is double-layered and positioned on both sides of the steel plate to provide adequate joint stiffness and strength. The plates are assembled on-site by bolting, enabling a dry-type connection that facilitates easy installation, disassembly, and reuse of PEC members.

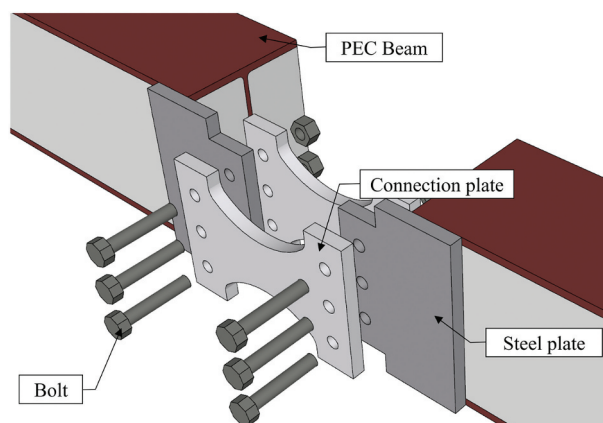


Figure 1. Concept of RBJ.

2.2. Assembly and joining sequence

The assembly procedure of the RBJ follows the sequence illustrated in Figure 2.

At the initial stage, steel plates welded to the PEC beam ends are accurately positioned and aligned with the internal H-section flange. Subsequently, connection plates are placed in front of and behind the steel plates to establish full contact for effective load transfer. Once the bolt holes of all components are aligned, high-strength bolts and threaded rods are inserted and tightened sequentially to form a stable dry-type connection. To accommodate typical fabrication and erection tolerances, the bolt holes were designed with a clearance of 1/8 in., in accordance with AISC provisions (AISC 2022), allowing minor alignment adjustments during installation. This joining sequence ensures even distribution of axial and flexural forces across the joint, minimizing assembly errors and

residual stresses. The entire process can be completed without welding or casting, enabling faster construction and safer field operations. In addition, the modular dry assembly allows damaged components to be easily replaced without dismantling the PEC beam, improving the system's maintainability and sustainability.

3. Experimental program

3.1. Specimen details

To evaluate the shear performance of Partially Encased Composite (PEC) beams incorporating the proposed Replaceable Bolted Joint (RBJ), five full-scale beam specimens were fabricated and tested under monotonic loading. Each specimen consisted of two PEC beam segments, each 1500 mm in length, joined at midspan through the RBJ, resulting in a total specimen length of 3480 mm. The PEC beam cross section was

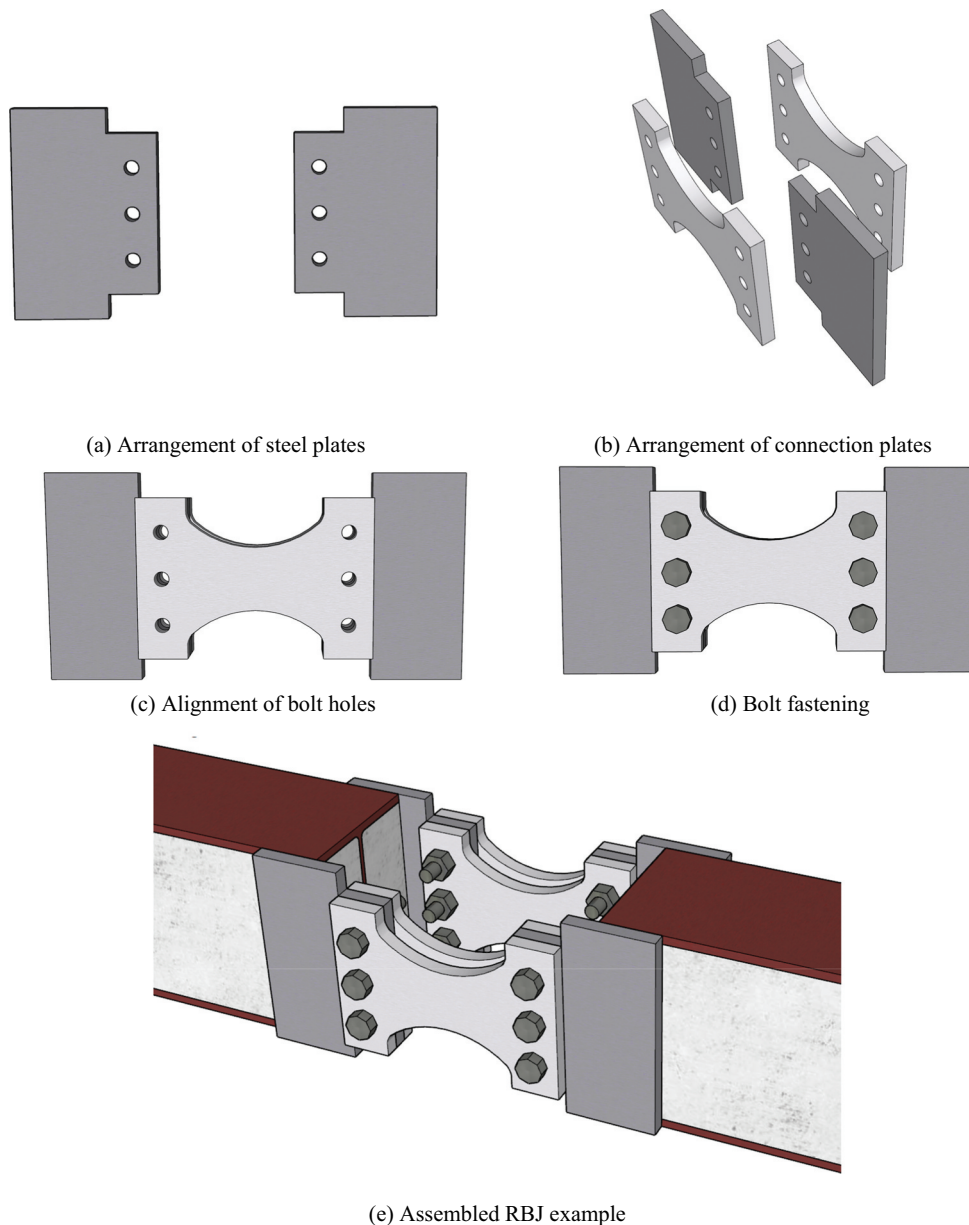


Figure 2. RBJ joining sequence.

250 mm wide and 340 mm deep. All PEC beams shared the same geometric configuration and reinforcement details. The internal H-shaped steel section (H-250 × 340 × 9 × 14, grade SS275) was embedded in the concrete core. Eight D19 deformed bars were arranged as longitudinal reinforcement, and D10 deformed bars were used as transverse reinforcement at 130 mm spacing. To prevent local crushing or splitting in the joint region, stirrup spacing was reduced to 50 mm around the RBJ. The concrete used for the PEC beams had a design compressive strength of 35 MPa, and the measured 28-day compressive strength was 34.3 MPa. Three parameters were selected as test variables:

- (1) the number of slits in the connection plate,
- (2) the type of bolt (high-strength bolt or threaded rod), and
- (3) the shear-span-to-depth ratio (a/d).

The number of slits was varied to investigate its influence on connection flexibility and shear ductility. The bolt type was considered to compare the load transfer mechanism between localized and through-type fastening. Finally, the a/d ratio was adjusted to examine the effect of shear demand on the global response and failure mode. The detailed configuration of the proposed steel module and reinforcement layout is illustrated in Figure 3. The specimen configuration and test variable matrix are summarized in Table 1, and the reinforcement details and material properties are listed in Table 2. All monotonic loading tests were conducted using a universal testing machine (UTM) at the Hybrid Structural Testing Center (HYSTEC) of Myongji University, Korea.

3.2. Detail of Replaceable Bolted joint (RBJ)

The detailed configuration of the Replaceable Bolted Joint (RBJ) used in the test specimens is shown in

Table 1. Experimental variables.

Specimen	Number of slits	Type of bolts	a/d
RT-2.7	1	Threaded rod	2.7
RH-2.7		High-strength bolt	
R-2.7	0		
R-2.5			2.5
R-2.2			2.2

Table 2. Properties of the reinforcing materials.

Concrete	Compressive strength (MPa)			
	34.3			
	Diameter (mm)	Cross-sectional area (mm ²)	Yield strength (MPa)	Modulus of elasticity (MPa)
Rebar	19.1	286.5	400	200,000
Stirrup	9.53	71.33		
High-strength bolt	20	245	900	200,000
	Type	Sectional dimension (mm)	Yield strength (MPa)	Modulus of elasticity (MPa)
H-beam	SRT275	250 × 340 × 9 × 14	275	205,000
Plate	SS275	25		

Figure 4. Each RBJ consisted of steel plates welded to both PEC beam ends and connection plates bolted on both sides. The steel plates were 25 mm thick and 130 mm wide, with three 12 mm-diameter bolt holes. The connection plates were 380 mm wide, 260 mm high, and 25 mm thick, containing six 12 mm-diameter bolt holes arranged symmetrically to allow bilateral bolting. Two types of connection plates were fabricated with different slit numbers, which served as the main experimental variable for assessing deformation behavior and shear ductility. High-strength bolts and threaded rods were used for fastening – the former connected each connection plate to the beam-end steel plate, and the latter passed through both sides to integrate the joint assembly into a single unit.

3.3. Manufacturing process

All PEC beams were fabricated following the same manufacturing procedure, as illustrated in Figure 5. First, longitudinal reinforcement and stirrups were arranged between the flanges and the web of the H-shaped steel section, after which concrete was poured to form the PEC beams. Following curing, steel plates were welded to the exposed flanges on both sides of the H-section at each beam end. Subsequently, two PEC modules were aligned, and connection plates were positioned on both sides of the joint. The joint assembly was then completed by tightening high-strength bolts and threaded rods in accordance with the RBJ configuration. The fastening procedures for each bolt type are illustrated in Figure 6.

3.4. Test setup

The experimental setup of the PEC beam specimens with the Replaceable Bolted Joint (RBJ) is illustrated in Figure 7. Each specimen was simply supported and subjected to monotonic single-point loading at midspan to evaluate its shear performance. The loading was applied under displacement control using a hydraulic actuator until the applied load dropped below 80% of the peak load after reaching the maximum resistance. The shear-span-to-depth ratio (a/d) was adjusted according to the specimen design

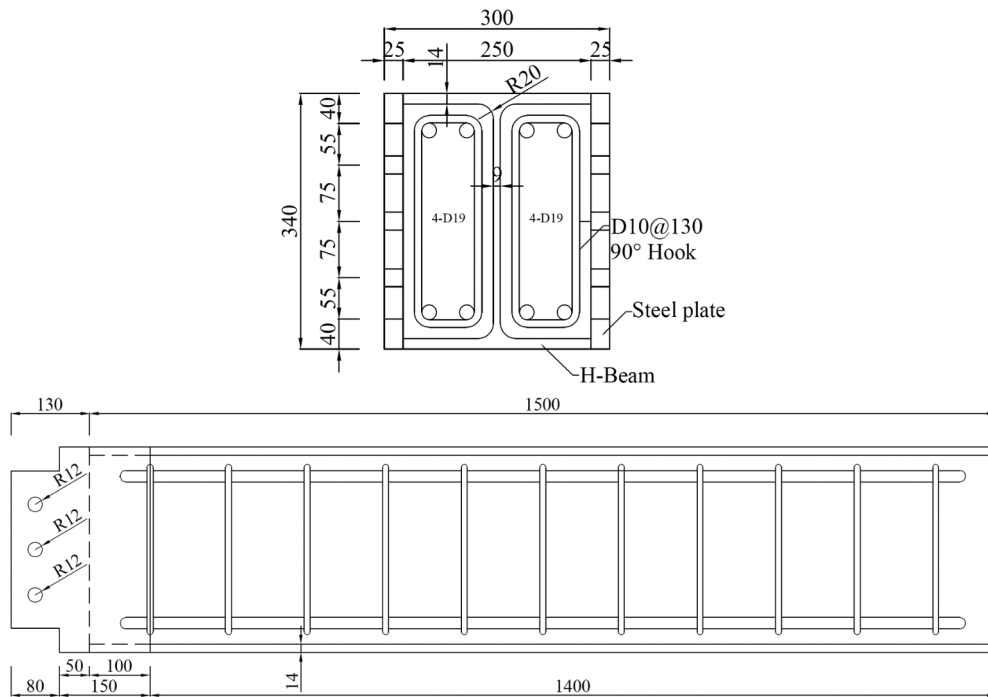


Figure 3. Details of the PEC beam specimen with Replaceable Bolted joint (RBJ).

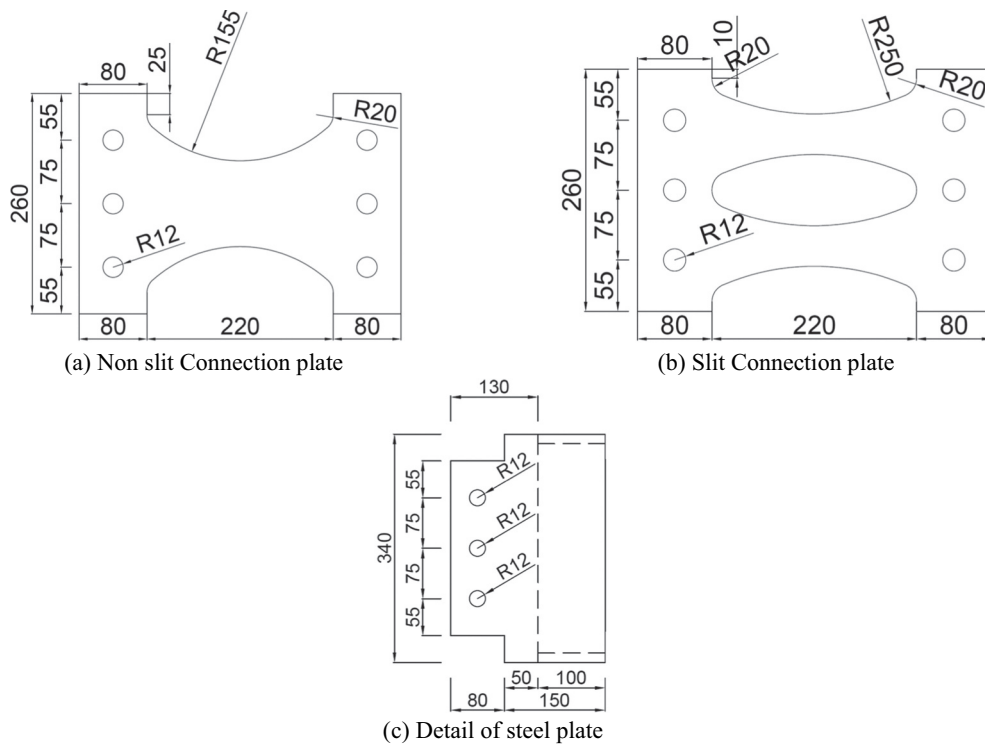


Figure 4. Detail of RBJ.

summarized in Table 1, and the loading and support positions were configured to achieve the intended shear span. Linear Variable Differential Transformers (LVDTs) were installed at the midspan and near the supports to record vertical displacements and beam-

end rotations. To monitor strain responses during loading, strain gauges were attached to the connection plates of the RBJ and to the longitudinal reinforcing bars in the joint region, as shown in Figure 8. All load, displacement, and strain data were

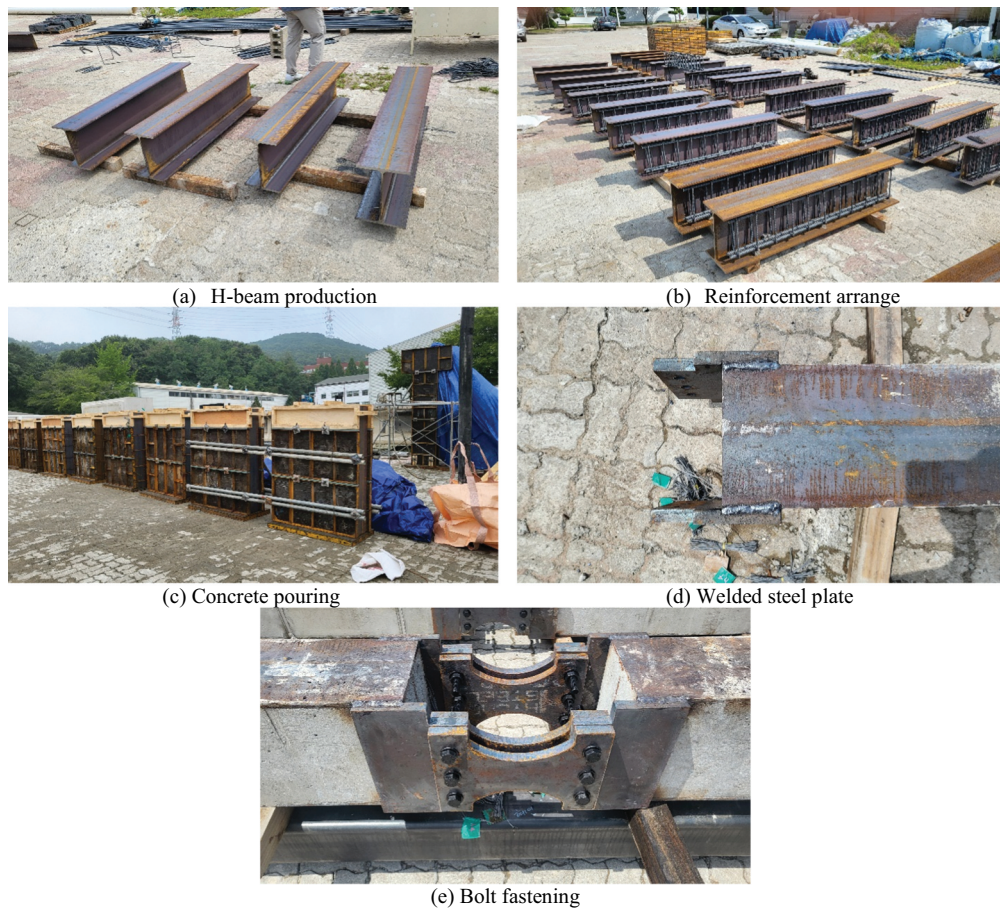


Figure 5. Specimen fabrication process.

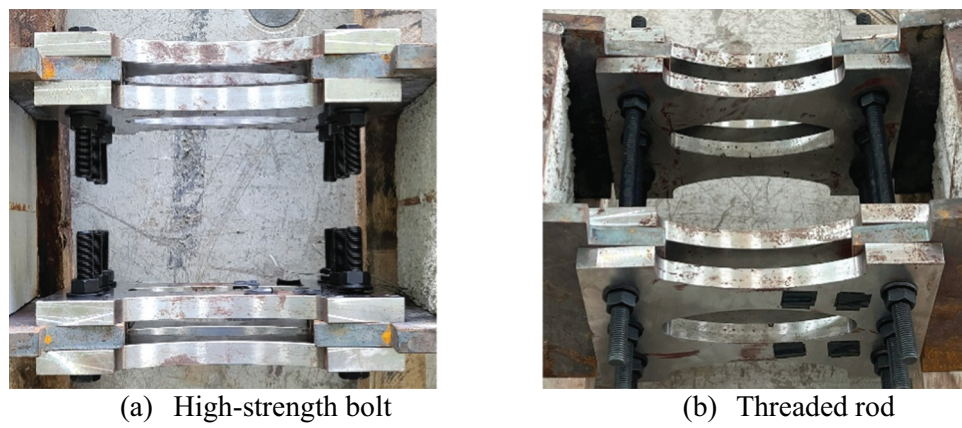
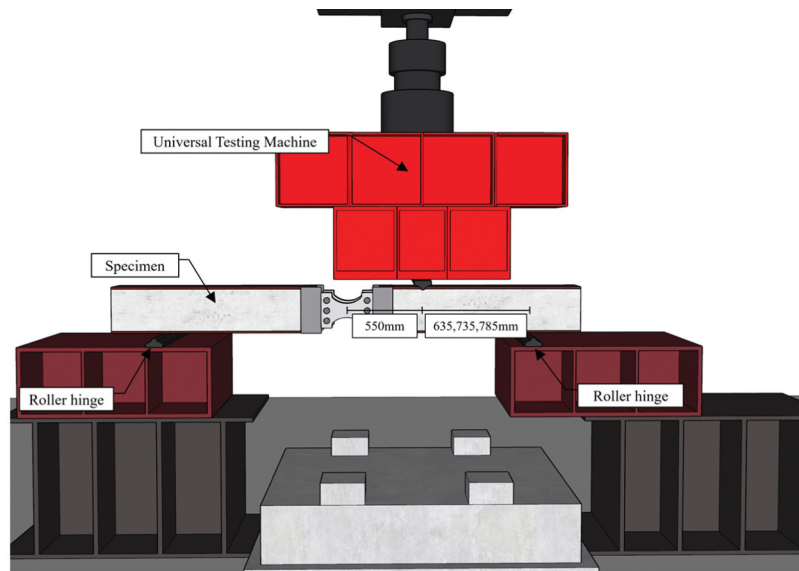


Figure 6. Type of bolt connection.



(a) Schematic of test set up



(b) Photograph of test set up

Figure 7. Experimental setup.

continuously recorded using a data acquisition system throughout the test.

4. Experimental results and analysis

4.1. Cracks and failure modes of specimens

Since the steel plates were welded to the flanges of the H-shaped steel sections in the PEC beams, the weld quality between the PEC beam and the steel plate plays a critical role in load transfer and joint performance. Throughout all tests, no cracks or deformations were observed at the welded interface, confirming that the welded steel plates effectively transferred loads to

the PEC beams until the end of the tests. The welding and bonding conditions are shown in [Figure 9](#).

[Figure 10](#) presents the crack patterns and final failure modes of all specimens observed during the tests. For Specimen RH-2.7, which incorporated slits in the connection plate and used high-strength bolts, the first flexural crack appeared at approximately 295 kN. As the load increased, crack propagation extended along the joint, accompanied by rotational deformation. Further loading induced bending deformation at the center of the connection plate, and the test was terminated after the high-strength bolts fractured. For Specimen RT-2.7, which also had slits but used threaded rods, the first flexural crack developed at

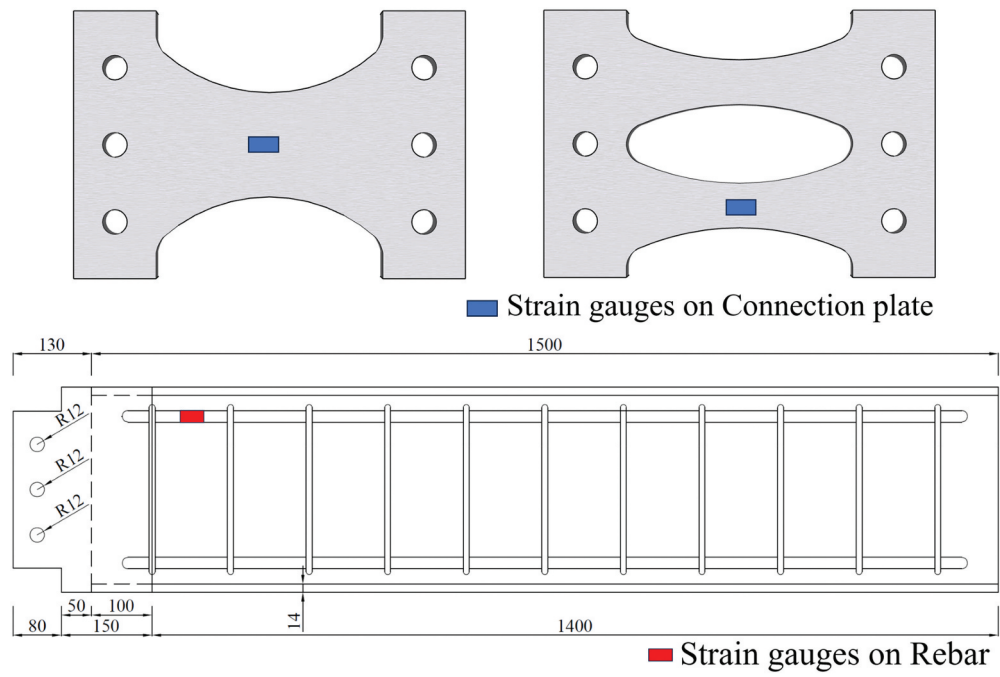


Figure 8. Locations of strain gauges on the connection plates and longitudinal reinforcement.

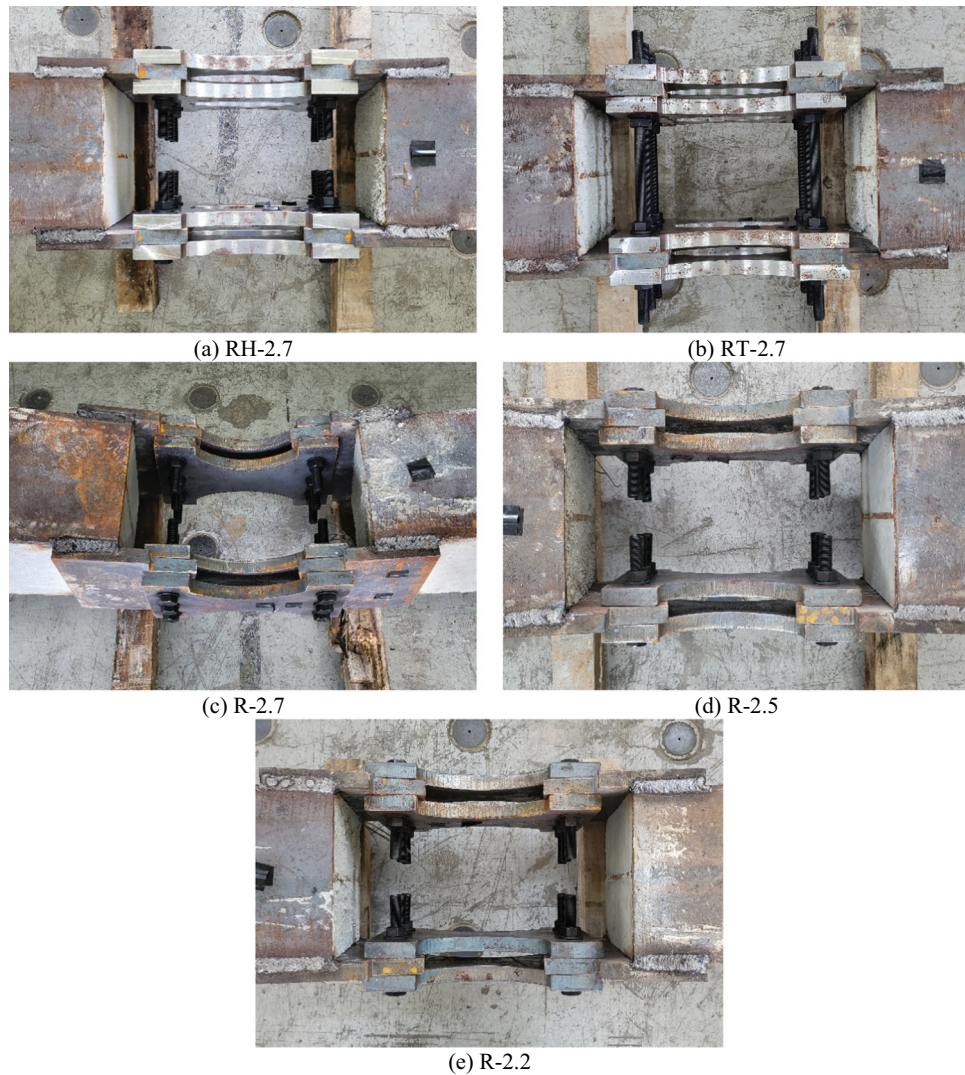


Figure 9. Welding area.

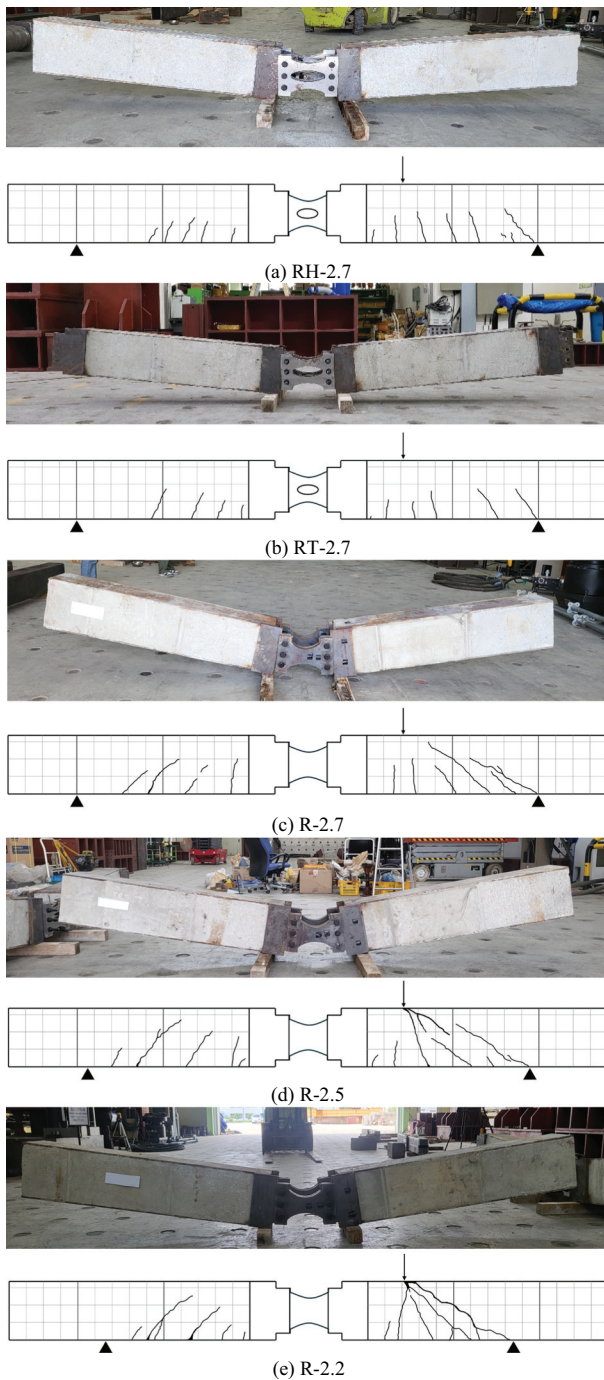


Figure 10. Crack pattern.

about 326 kN, followed by the formation of diagonal cracks and gradual rotation of the joint. No bolt fracture occurred, and the test was terminated when the load dropped below 80% of the peak value after reaching maximum strength. For Specimen R-2.7 (without slits, $a/d = 2.7$), flexural cracks appeared at around 270 kN, followed by shear cracks near 284 kN, which reduced the rate of load increase. Bending deformation developed at the mid-height of the connection plate, and failure occurred due to bolt fracture. In Specimen R-2.5, the first flexural crack appeared at approximately 300 kN, and joint rotation developed progressively. Shear cracks were observed at 330 kN,

after which the load-carrying capacity dropped rapidly following bolt fracture.

In Specimen R-2.2, the first flexural crack appeared at around 340 kN, followed by shear cracking at 371 kN. Although the specimen reached its peak load more quickly than others, it failed in a brittle shear mode without any bolt fracture. Comparatively, RH-2.7 and RT-2.7, which featured slits in the connection plate, exhibited less beam-end rotation than the non-slit specimens. This indicates that the slits effectively redistributed stress and mitigated localized rotation near the joint interface. As the shear-span-to-effective-depth ratio (a/d) increased, flexural effects became more dominant, leading to greater deformation and enhanced ductility. Conversely, the R-2.2 specimen with the smallest a/d exhibited shear-dominated brittle behavior with minimal rotation. The absence of bolt fracture in this specimen suggests that the shear demand was primarily carried by the concrete and internal steel section rather than the bolts. Representative bolt failure modes observed in other specimens are shown in Figure 11.

4.2. Load-displacement relationships

The load – displacement curves for all specimens are shown in Figure 12, and the corresponding maximum loads, displacements at peak load, and ultimate displacements are summarized in Table 3. All specimens incorporating the RBJ exhibited a distinct *tri-linear* response due to load transfer through the bolted joint assembly. During the initial loading stage, a short nonlinear segment appeared as minor slip occurred at the bolt interfaces, resulting in limited load increase with noticeable displacement. After the slip was eliminated, the response became nearly linear until cracking and yielding, followed by a gradual softening after the peak load. For Specimen RH-2.7 (with slitted connection plates and high-strength bolts), the maximum load was 502.10 kN at a displacement of 94.86 mm. After reaching the peak, the load gradually decreased to 80 % of its maximum at 101.22 mm displacement.

Specimen RT-2.7 (with slits and threaded rods) showed a slightly lower maximum load of 446.13 kN at 86.33 mm, with a similar gradual post-peak decline, dropping below 80 % at 92.87 mm.

For Specimen R-2.7 (without slits, $a/d = 2.7$), the maximum load reached 453.8 kN at 133.7 mm, followed by a gentle reduction until 149.92 mm, when the load fell below 80 %. Specimen R-2.5 reached a peak load of 483.39 kN at 126.81 mm, then experienced a relatively sharp strength drop to 80 % at 131.17 mm. Finally, Specimen R-2.2 exhibited the highest peak load of 525.5 kN at 117.92 mm, but its load decreased abruptly after the peak, reaching 80 % at

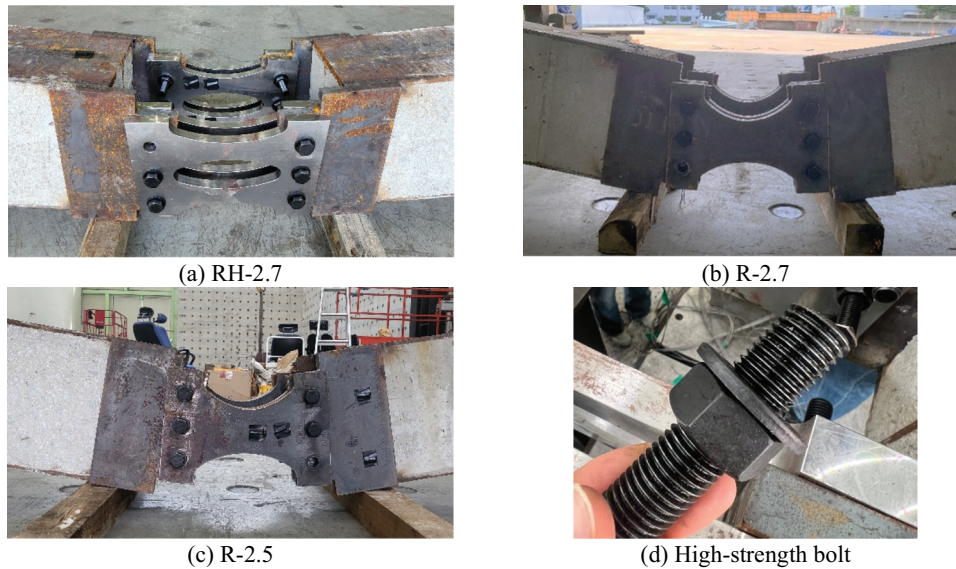


Figure 11. Bolt failure.

121.26 mm. The slitted specimens (RH-2.7 and RT-2.7) developed higher maximum loads compared with the non-slitted specimens. This improvement can be attributed to stress redistribution around the slits, which reduced local stress concentration and delayed the onset of major cracking. In particular, RH-2.7 achieved about 11 % higher peak load than R-2.7, demonstrating the beneficial effect of slit-induced load redistribution. Furthermore, the high-strength-bolt specimen (RH-2.7) showed approximately 13 % greater peak load than the threaded-rod specimen (RT-2.7), indicating that the superior clamping capacity of high-strength bolts enhanced joint stiffness and shear resistance.

Specimen R-2.2 with the smallest shear-span-to-effective-depth ratio (a/d) exhibited the highest load capacity among all specimens. The maximum loads of R-2.5 and R-2.7 were approximately 92 % and 86 %, respectively, of that of R-2.2, confirming that the ultimate shear strength decreases as a/d increases. This trend reflects the shift from shear-dominant brittle behavior at lower a/d to flexure-dominant ductile behavior at higher a/d , consistent with previously reported findings on composite beams. This trend is consistent with previous findings on PEC and composite beams under shear loading (Chu et al. 2024; Liang et al. 2025).

4.3. Strains

The measured strain histories of the connection plates for all specimens are presented in Figure 13. For Specimen RH-2.7, which had slitted connection plates and high-strength bolts, the strain increased almost linearly at the initial stage and then rose sharply at a displacement of approximately 90 mm. This abrupt increase coincided with the attainment of the peak

load, reflecting combined stress concentration in the bolt region and the propagation of diagonal cracks through the joint. At the end of loading, strain values spiked rapidly due to bolt fracture, confirming that the stress transfer path was lost once the bolts failed. For Specimen RT-2.7 (with slits and threaded rods), strain growth was initially nonlinear, which can be attributed to greater slip at the interfaces owing to the lower clamping stiffness of threaded rods compared with high-strength bolts. Once the slip was stabilized, the strain increased gradually until the specimen reached its maximum load. Specimen R-2.7 (without slits, $a/d = 2.7$) exhibited a moderate strain increase during the early stage, followed by a sudden rise near 70 mm displacement, corresponding to the expansion of shear cracks and resulting in local stress concentration in the connection plate. Similarly, Specimen R-2.5 showed almost no strain change at the beginning but a rapid increase after the first shear crack formed. At around 60 mm displacement, the strain accelerated sharply as crack propagation localized stress around the plate center. In Specimen R-2.2, strain initially increased gently but surged abruptly at about 60 mm, suggesting early damage initiation in the connection plate under higher shear demand. The measured maximum strain in R-2.2 approached approximately $1,800 \mu\epsilon$, close to the expected yield strain of the steel plate, indicating early yielding and shear-dominated behavior.

In general, specimens with slitted connection plates (RH-2.7 and RT-2.7) exhibited slower strain development compared with the non-slitted ones. This demonstrates that the slit configuration effectively redistributed local stresses and delayed strain concentration in the connection region.

Furthermore, specimens using threaded rods showed a more uniform strain distribution than those

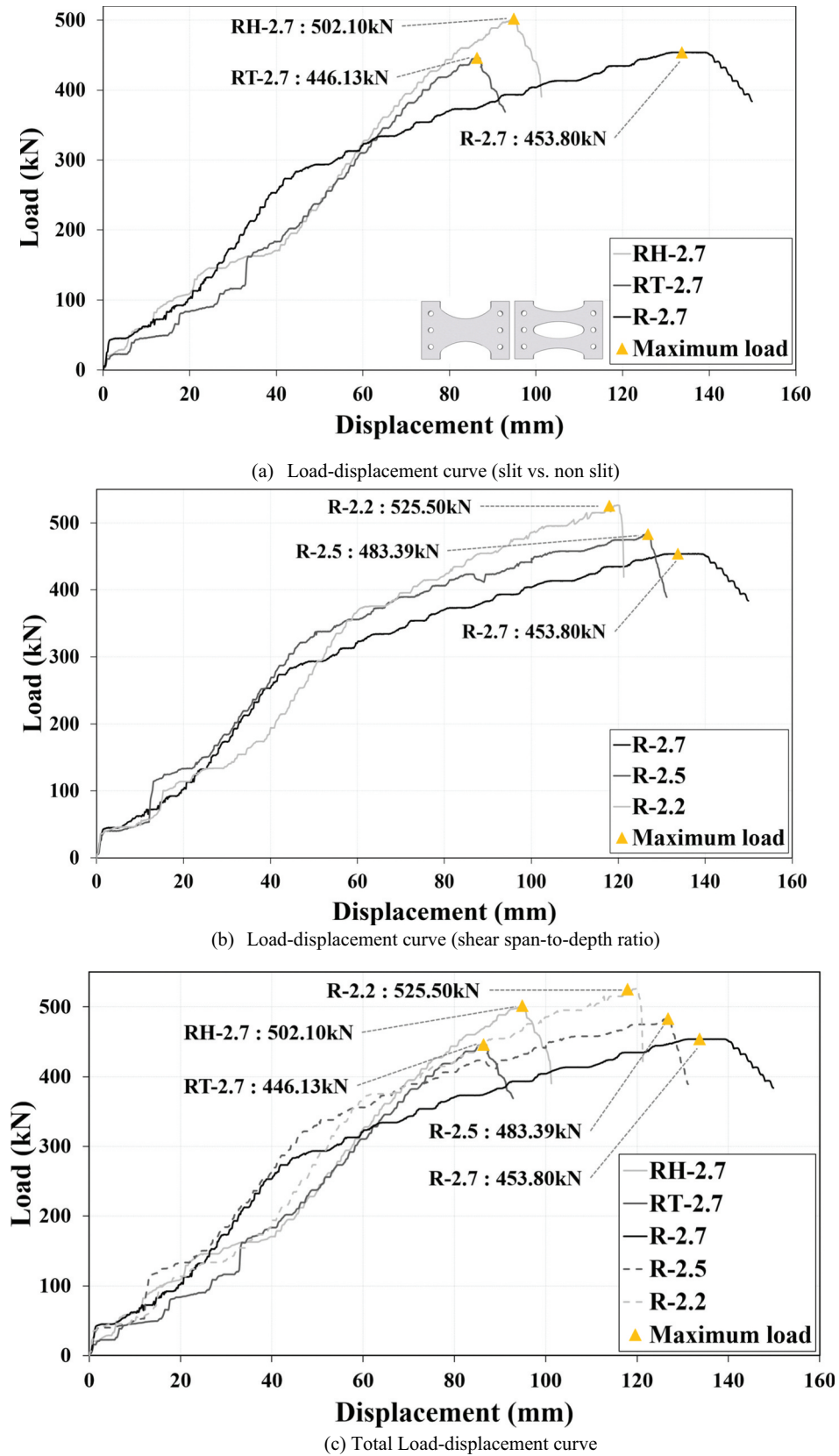


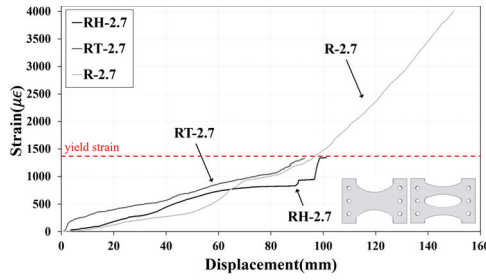
Figure 12. Load-displacement curve.

using high-strength bolts, implying that their lower clamping stiffness facilitated more even load sharing among bolts. Among the specimens with varying shear-span-to-effective-depth ratios (a/d), the smallest-

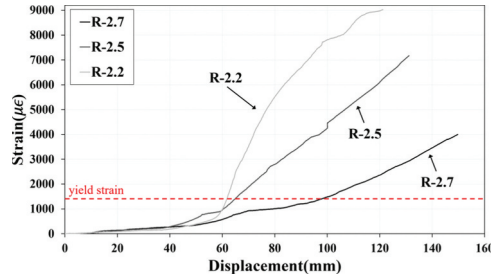
ratio specimen (R-2.2) exhibited the earliest and highest strain rise, verifying its shear-dominated response with early yielding and concentrated damage. As a/d increased, the strain evolution became more gradual

Table 3. Maximum load and maximum displacement.

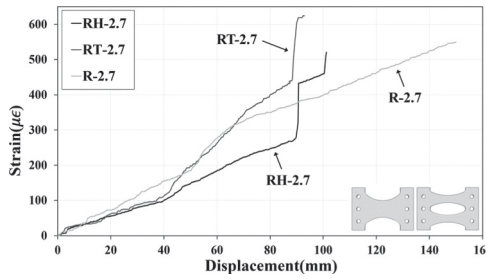
Specimens	Maximum Load (kN)	Displacement at	
		Maximum Load (mm)	Maximum Displacement (mm)
RH-2.7	502.10	94.86	101.22
RT-2.7	446.13	86.33	92.87
R-2.7	453.80	133.70	149.92
R-2.5	483.39	126.81	131.17
R-2.2	525.50	117.92	121.26



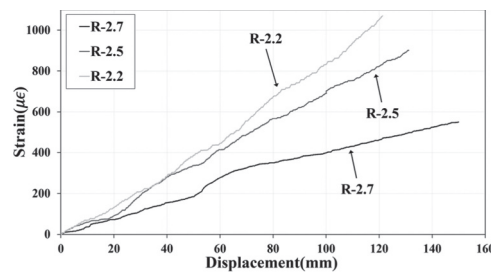
(a) Strain-displacement curve (slit vs. non slit)



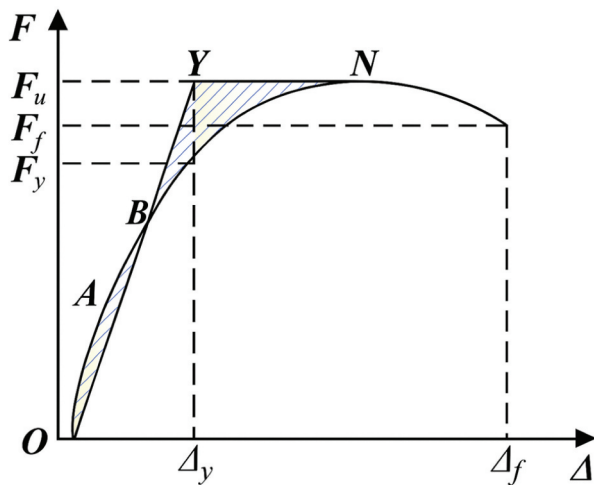
(b) Strain-displacement curve (shear span-to-depth ratio)

Figure 13. Strain-displacement curve at connection plate.

(a) Strain-displacement curve (slit vs. non slit)



(b) Strain-displacement curve (shear span-to-depth ratio)

Figure 14. Strain-displacement curve at rebar.**Figure 15.** Schematic diagram of shear ductility value (Liu et al. 2024).

and the maximum strain decreased, indicating a behavioral shift toward flexure-dominated response with distributed stress transfer.

The measured connection-plate strains indicate that the presence of slits does not significantly affect the pre-yield response of the joint. Differences observed at larger displacement levels are therefore discussed in conjunction with the load – displacement response and failure characteristics, rather than based on strain measurements alone.

The measured strain responses of the longitudinal reinforcement in each specimen are shown in Figure 14. For Specimen RH-2.7, which had slitted connection plates and high-strength bolts, the strain increased linearly with displacement during the early loading stage. As the load approached the peak, stress concentration in the connection plate caused a mild rise in bar strain, followed by a sharp increase near the maximum load. In Specimen RT-2.7 (with slits and threaded rods), the strain also increased linearly at first but rose abruptly at around 40 mm displacement. This was caused by slip at the bolt interfaces due to the lower clamping stiffness of the threaded rods, which allowed partial stress transfer from the connection plate to the longitudinal bars. A second sharp strain increase occurred near the peak load, similar to the behavior of RH-2.7.

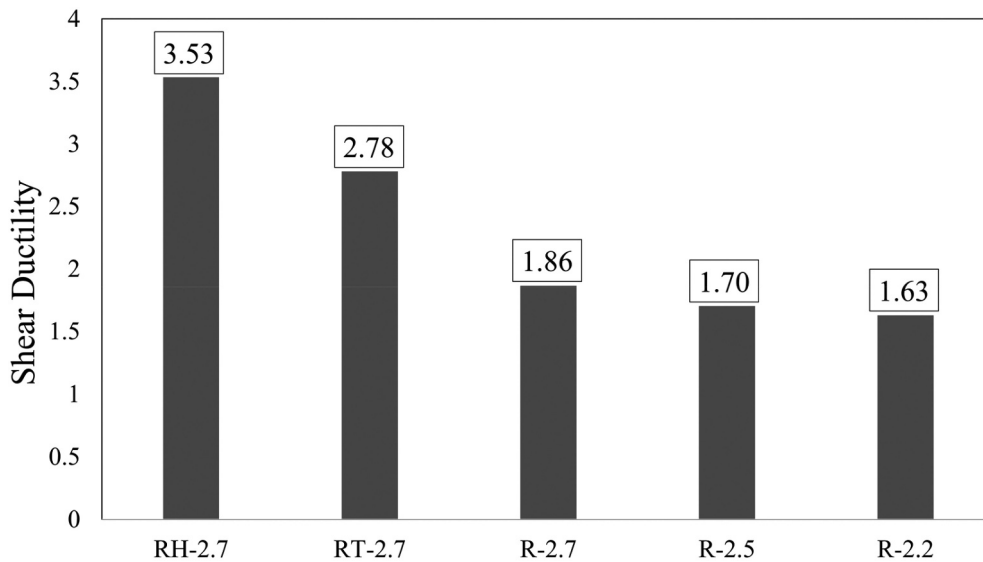


Figure 16. Shear ductility.

For Specimen R-2.7 (without slits, $a/d = 2.7$), the strain evolution was similar to RH-2.7 during the early stage, but a pronounced strain escalation was observed when shear cracks formed, as the absence of slits reduced energy dissipation and induced localized stress concentration.

After that, the strain continued to rise gradually until failure. Specimen R-2.5 showed mostly linear strain development, with a moderate rise following the formation of shear cracks.

In contrast, Specimen R-2.2 exhibited the highest final strain among all specimens; the strain rapidly increased after crack propagation, reaching approximately 1,800 $\mu\epsilon$, close to the yield strain of the longitudinal bars. This confirms that the R-2.2 specimen experienced early yielding and shear-dominated behavior due to concentrated stress transfer through the connection plate.

Specimens with slitted connection plates (RH-2.7 and RT-2.7) exhibited sudden strain surges at the peak load stage. This occurred because the slits had already reached their deformation capacity, after which additional stress was redirected to the longitudinal reinforcement. Among the specimens with different shear-span-to-effective-depth ratios (a/d), the smallest-ratio specimen (R-2.2) showed the highest ultimate strain, while the maximum strain decreased as a/d increased. In all cases, the strain in the longitudinal reinforcement developed more gradually than that in the connection plate, indicating that the applied load was primarily resisted and transferred through the connection plate before being carried by the beam reinforcement.

The strain measured at the mid-span longitudinal reinforcement remained well below the yield strain for all specimens. At comparable displacement levels, the slitted specimens (RT-2.7 and RH-2.7) exhibited higher

rebar strains than the non-slitted specimen (R-2.7), indicating that load transfer from the connection region to the surrounding beam was more readily facilitated in the presence of slits, rather than deformation being primarily confined within the joint. When the strain results are considered together with the load – displacement response and observed failure modes, it is evident that the introduction of slits does not significantly affect the pre-yield behavior of the joint. Instead, the influence of the slits becomes apparent at larger deformation levels through differences in deformation sharing between the connection region and the surrounding beam, as reflected by the strain responses and damage progression.

4.4. Shear ductility

Figure 15 illustrates the idealized force – displacement relationship used to evaluate shear ductility. F_y denotes the yield force corresponding to the yield displacement (Δ_y), F_u represents the maximum force, and F_f is defined as the residual force at the failure displacement (Δ_f). Shear ductility (μ_Δ) was evaluated as the ratio of the displacement at failure (Δ_f) to the yield displacement (Δ_y). The failure displacement was defined as the deflection recorded at the termination of loading, while the yield displacement was determined based on the equal-energy assumption, in which the areas of S_{OAB} and S_{YNB} shown in Figure 15 are considered equal (Liang et al. 2025). The computed shear ductility values of all specimens are presented in Figure 16. Among the five specimens, RH-2.7, which incorporated slitted connectors and used high-strength bolts, exhibited the highest shear ductility of 3.53. The other specimens showed ductility ratios of 2.78 for RT-2.7, 1.86 for R-2.7, 1.70 for R-2.5,

and 1.63 for R-2.2. Specimens with slitted connectors achieved up to 57 % higher shear ductility compared with those without slits. This improvement is attributed to the redistribution of stress induced by the slit geometry, which alleviates local stress concentration and allows larger inelastic deformation. A decreasing trend in shear ductility was observed with a reduction in the shear-span-to-depth ratio, indicating that shorter-span specimens experienced higher shear demand and earlier web-concrete crushing, thus exhibiting lower post-yield deformation capacity. These observations are consistent with the shear-ductility behavior reported for prestressed partially-precast steel-reinforced concrete (PPPSRC) beams (Jiang et al. 2016), confirming that confinement provided by steel or prestressing enhances diagonal-crack control and contributes to improved ductile shear response. All tests were carried out at the Key Laboratory of Structural Engineering and Earthquake Resistance, Xi'an University of Architecture & Technology, where the static loading setup and displacement control followed the same methodology as described in Liu et al. (2024).

5. Conclusion

In this study, a Replaceable Bolted Joint (RBJ) was proposed to facilitate the application of Partially Encased Composite (PEC) beams in precast concrete systems. The joint was designed to improve member reusability by allowing easy replacement after damage. Five specimens were fabricated to evaluate the shear performance of the proposed joint, with key parameters including the number of slits in the connection plate, the type of bolt, and the shear-span-to-effective-depth ratio (a/d). Based on the experimental results, the following conclusions were drawn:

- (1) Specimens with slitted connection plates showed approximately 11% higher peak load than non-slitted ones due to stress redistribution, although their energy dissipation was reduced to 45–68% of that of non-slitted specimens.
- (2) The use of high-strength bolts increased the maximum load by about 13%, providing better stiffness and energy dissipation than threaded rods.
- (3) As the shear-span-to-depth ratio (a/d) increased, the behavior transitioned from shear-dominated brittle to flexure-dominated ductile, with smoother post-peak strength degradation.
- (4) Slitted plates resulted in more gradual strain development, confirming stress-redistribution effects. The R-2.2 specimen exhibited early yielding with localized shear behavior.

- (5) The RH-2.7 specimen, incorporating both slitted plates and high-strength bolts, achieved the highest shear ductility among all specimens. Specimens with slitted connectors demonstrated improved deformation capacity due to the stress dispersion effect of the slit geometry. In contrast, specimens with lower shear-span ratios exhibited reduced shear ductility because of the higher shear demand and earlier web-concrete crushing.

The proposed RBJ enables mechanical assembly and disassembly of PEC beams through simple on-site bolting. This characteristic facilitates practical member replacement and modular construction processes without cast-in-place concrete. Within the scope of the present experimental program, the RBJ demonstrated stable shear resistance and deformation capacity under monotonic loading. Future studies will investigate the cyclic behavior and seismic performance of the RBJ under repeated loading to further assess its structural applicability. In addition, comparative numerical and experimental studies with conventional wet-connected PEC beam systems will be conducted to directly compare shear strength, stiffness characteristics, deformation capacity, and failure mechanisms between dry and wet connection methods.

Disclosure statement

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Author contributions

Young Hak Lee conceived and supervised the study and contributed to the interpretation of the results. Min Sook Kim designed and conducted the experiments, analyzed the data, and drafted the manuscript. Sung Woo Park contributed to the review of the experimental results and manuscript revision. All authors discussed the results, approved the final version of the manuscript for publication, and agree to be accountable for all aspects of the work.

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