Finite Element Modelling of Bolt Shear Connections in Prefabricated Steel Lightweight Aggregate–Concrete Composite Beams

Wei Wang 1, Xiedong Zhang 1, Yu Ren 1,*, Fanglong Bai 1, Chaohui Li 2 and Zhiguo Li 2

1 School of Transportation and Logistics Engineering, Wuhan University of Technology, Wuhan 430063, China; wwei5633@whut.edu.cn (W.W.); zhangxd@whut.edu.cn (X.Z.); baifl@whut.edu.cn (F.B.)
2 Hubei Qiaoxiao Expressway Management Co., Ltd., Phase II, Wuhan 432020, China; lichaohui2022123@126.com (C.L.); Lizg711104@126.com (Z.L.)
* Correspondence: renyu@whut.edu.cn; Tel.: +86-15527038181

Abstract: Steel lightweight aggregate-concrete composite beams (SLACCBs) with bolted shear connections provide several advantages, such as reducing the overall self-weight, shortening the construction period, and improving the structural seismic performance. However, current research on the mechanical behavior of bolted connections is primarily concentrated on composite structures with normal concrete (NC), and there is no investigation focused on the shear performance of bolt connections embedded in lightweight aggregate concrete (LAC) slabs. Therefore, this paper developed a three-dimensional (3D) numerical model to study the shear properties of the bolt connections embedded in SLACCBs by utilizing ABAQUS software. Nonlinear geometric effects and material nonlinearities were considered in the finite element (FE) modelling. The accuracy and reliability of the FE modelling were initially calibrated and validated against the push-off tests described in the literature. Subsequently, the basic shear properties of the bolted connection embedded in SLACCBs were studied and compared with those of the bolted connection embedded in the NC slab by applying the verified FE modelling. Meanwhile, the effects of the concrete strength, concrete density, bolt diameter, and bolt tensile strength on the shear behavior of the bolt connections embedded in SLACCBs were also investigated using extensive parametric studies. Finally, some design formulae were proposed to predict the bolt connection shear strength.

Keywords: bolt shear connector; lightweight aggregate concrete; push-off test; finite element model; shear bearing capacity

1. Introduction

Steel–concrete composite beams (SCCBs) combining the full advantages of two materials (the tensile strength of steel and the compressive strength of concrete) have long been used in buildings and bridges [1–3]. Reliable shear connectors that resist the relative slip between the steel–concrete interface are the critical feature to guarantee the composite behavior [4–6]. Traditional connectors (e.g., steel studs and perfobond ribs) with outstanding mechanical performance and the ease of construction have been adopted and investigated extensively in SCCBs in recent decades [7,8]. However, disassembling and reusing the elements (i.e., concrete plates, shear keys, as well as steel girders) is challenging when conventional SCCBs reach the end of their service life because the connections are not only welded to steel girders, but embedded into the cast in the in situ concrete slabs. To achieve this challenging goal and to improve construction sustainability, high-strength bolts are applied in SCCBs to substitute for traditional connections as the shear connection.
Dallam [9] first reported the shear behavior of high-strength bolts embedded in the NC slab using push-off tests. Test results showed that the shear resistance of high strength bolts was twice that of steel studs with the same dimensions. Subsequently, Dedic and Klaiber [10] elucidated the possibility of utilizing the high-strength bolts in rehabilitation work with older bridges using push-out tests. In that study, the experiments concluded that the high-strength bolt could be applied as the shear connection with strength and stiffness comparable to steel studs. Then, in order to strengthen the existing non-composite bridges, three types of post-installed bolted connections were experimented with under static and fatigue loading by utilizing the single shear connection test setup [11,12]. Their studies indicated that the fatigue performance of the bolted shear connection was higher than that of welded studs. In addition, Pavlović et al. [13] undertook push-off tests and corresponding FE analysis to better understand the difference between the shear mechanism for bolts and studs embedded in prefabricated SCCBs. Their investigation revealed that the bolt connections applied in SCCBs can enhance the competitiveness and the level of prefabrication in composite structures. Moreover, different types of demountable bolt connections machined from steel studs or commonly used bolts were presented by scholars to improve the construction convenience of prefabricated composite structures [14–18]. Recently, the basic shear properties of a high-strength friction-grip bolt (HSFGB) connection embedded in NC slab [19–22], geopolymer concrete (GPC) slab [23–25], steel-fiber reinforced concrete (SFRC) slab [26], and ultra-high-performance concrete (UHPC) slab were explored by push-off tests and FE analysis [27]. It was observed that the HSFGB connection displayed outstanding mechanical performance in SCCBs. Additionally, the multiple bolt effects and the shear behavior of grouped bolt connectors embedded in NC slab were reported [28,29], and the results indicated that the shear strength per bolted connection was stronger than that of a single bolt embedded in grouped connectors due to the multiple bolts effect.

Compared with NC, LAC provides better properties, such as low density, good frost resistance, and impermeability [30,31]. Employing it to composite bridges can significantly reduce the overall self-weight, increase bridge spans, and improve the structural seismic performance [32]. Previous investigations mainly pertained to the shear behavior of bolted connections embedded in NC, and it appears that there are no studies on bolted connectors embedded in SLACCBs. This study will focus on this area. A 3D, non-linear FE modelling, considered the geometric and material nonlinearities, was initially presented, calibrated, and verified against the push-off tests. Subsequently, the basic shear performance of bolted connections embedded in SLACCBs was investigated and compared with NC slabs by utilizing the verified FE modelling. Moreover, the effects of several key variables, including the concrete strength, concrete density, bolt diameter, and bolt tensile strength on the shear behavior of the bolted connection, were explored by extensive parametric studies. Finally, some design recommendations were proposed to evaluate the per-bolt connection shear strength.

2. FE Modelling
2.1. Model Geometry

ABAQUS/Standard [33] was used to develop detailed 3D FE models to simulate the basic mechanical behavior of bolt connections in push-off tests. Figure 1 presents the configuration and details of push-out tested specimens in Refs. [18,19,26]. Four members, i.e., the concrete plate, steel girder, bolt connector, and steel bars, were all considered in the numerical model to reflect the actual tests. Furthermore, the non-linear geometric effects, material nonlinearities, and complicated contact properties involved in FE modelling were all taken into account in FE analysis.
2.2. FE Type and Mesh

To reduce the possibility of convergence problems and prevent shear locking, all parts were meshed by the 8-node linear brick element with reduced integration and hourglass control (C3D8R), except for the reinforcement cages. A two-node 3D truss element (T3D2) was selected to represent the rebar. The meshing details and element types of the FE modelling are outlined in Figure 2. Because of the symmetry of geometry and loading, 1/4 of the real tested specimens were modelled. The hexagonal head of the high-strength bolt was reduced to a round rod with the same cross-sectional diameter. In addition, mesh sensitivity analysis (Section 3.3) was conducted to capture simulation accuracy and improve analysis efficiency. The bolt connections, area around the hole in the concrete slab, and the steel holes were specified as 3 mm. At the same time, the mesh size of the concrete plates and steel girders was set as 30 mm.

![Mesh details and element types](image)

**Figure 2.** Mesh details and element types of the numerical model: (a) concrete block C3D8R; (b) steel girder C3D8R; (c) bolt shear connection C3D8R; (d) reinforcing bars T3D2.

2.3. Material Properties

The material constitutive models proposed by Ding et al. [34,35] were used to represent the nonlinear behavior of NC and LAC. The stress–strain curve of these two types of concrete under uniaxial stress can be expressed using the non-dimensional formula:

\[ y = \begin{cases} 
\frac{Ax + (B - 1)x^2}{1 + (A - 2)x + Bx^2} & x \leq 1 \\
\frac{x}{\alpha_1(x - 1)^2 + x} & x > 1
\end{cases} \]  

where: \( y = \sigma / f_c \) is the stress of the core concrete to uniaxial compressive concrete and \( x = \epsilon / \epsilon_c \) is the corresponding strain; \( \sigma \) is the stress of the core concrete, and \( \epsilon \) is the corresponding strain. \( f_c \) denotes the uniaxial compressive strength of the concrete (in MPa), where \( f_c = 0.4f_{cu}^{0.7}, 0.88f_{cu} \) for NC and LAC, respectively. \( \epsilon_c \) denotes the strain corresponding to \( f_c \).
where \( \varepsilon_c = 383f_{cu}^{1/3} \times 10^{-5} \), \( 730f_{cu}^{1/3} \times 10^{-5} \), for NC and LAC, respectively. \( f_{wu} \) represents the compressive cubic strength of the concrete (in MPa). \( A \) denotes the ratio of the initial tangent modulus to the secant modulus at peak stress, where \( A = 9.1f_{wu}^{1/9} \), \( 1.68 \times 10^{-3} \rho f_{wu}^{1/16} \), for NC and LAC, respectively. \( \rho \) is the apparent density of LAC (in kg/m\(^3\)). \( B \) denotes a variable that controls the decrease in the elastic modulus along the ascending branch of the axial stress–strain curve, where \( B = 1.6(A-1)^2 \). \( \alpha_1 \) is a parameter that controls the descending section of the stress–strain curve, where \( \alpha_1 = 0.15 \). CDP (concrete damaged plasticity) model, with the key plastic parameters comprising the flow potential eccentricity (\( e \)), biaxial/uniaxial compressive strength ratio (\( f^0/f^\text{p} \)), and the ratio of the tensile to the compressive meridian (\( K \)) set as 0.1, 1.225, and 2/3, respectively \([36]\). The dilation angle (\( \psi \)) and the viscosity parameter (\( v \)) were analyzed to calibrate the FE modelling described in Section 3.2.

The following function \([34]\) (Equation (2)), which considered the von Mises yield criteria, the Prandtl–Reuss flow rule, and isotropic strain hardening, was chosen to express the stress–strain relationship of steel beam and steel bars.

\[
\sigma_i = \begin{cases} 
E\varepsilon_i & \varepsilon_i \leq \varepsilon_y \\
 \frac{f_y}{f_y + 0.46\% E_y (\varepsilon_y - \varepsilon_i)} & \varepsilon_y < \varepsilon_i \leq \varepsilon_u \\
 \frac{f_y}{\varepsilon_u} & \varepsilon_i > \varepsilon_u 
\end{cases} \tag{2}
\]

where: \( \sigma_i \) is the equivalent stress of steel (in MPa) and \( \varepsilon_i \) is the corresponding strain, \( f_y \) is the yield strength, \( f_u (=1.5f_y) \) is the ultimate strength (in MPa), \( \varepsilon_y \) and \( \varepsilon_u \) is the yield strain, \( \varepsilon_u (=12\varepsilon_y) \) is the hardening strain, and \( \varepsilon_u (=12\varepsilon_y) \) is the ultimate strain of the steel; \( E_y \) is the elastic modulus (in MPa).

The material stress-strain relationship of the high-strength bolt proposed by Loh et al. \([37]\) was used in this paper, which was also utilized by Chen et al. \([18]\) and Liu et al. \([24]\).

\[
\sigma_{bt} = \begin{cases} 
E_{bt}\varepsilon_{bt} & \varepsilon_{bt} \leq \varepsilon_{try} \\
0.94f_{bui} + 0.86\% f_{bui} (\varepsilon_{bt} - \varepsilon_{try}) & \varepsilon_{try} < \varepsilon_{bt} \leq 8\varepsilon_{try} \\
8\varepsilon_{try} & \varepsilon_{bt} > 8\varepsilon_{try} 
\end{cases} \tag{3}
\]

where: \( \sigma_{bt} \) is the equivalent stress (in MPa), \( \varepsilon_{bt} \) and \( \varepsilon_{try} \) are the corresponding equivalent strain and yield strain, respectively; \( f_{bui} \) is the ultimate strength (in MPa); \( E_{bt} \) is the elastic modulus (in MPa).

### 2.4. Boundary and Loading Conditions

Figure 3 presents the symmetric boundary conditions and the loading surface of the FE modelling. All nodes lying in the symmetric Y-Z plane (surface 1, colored by red) were prevented at the X direction \((U1 = 0)\) and rotating in the Y and Z axes \((UR2 = UR3 = 0)\). All nodes lie on the symmetric X-Z plane (surface 2, colored by orange) were restrained from translating in the Y-direction \((U2 = 0)\) as well as rotating in the X and Z axes directions \((UR1 = UR3 = 0)\). Moreover, the bottom part of the concrete block (surface 3, colored by cyan) was fully fixed. The FE analysis of the model consisted of two steps similar to the real experiments. First, the BOLT LOAD function was chosen to simulate the pretension force applied in tests. Then, the push-down loads were imposed on the top section of the steel beam, as described in Figures 3 and 4.
2.5. Interaction and Constraint Conditions

The interaction of numerical models comprised four contact pairs: concrete-to-steel, concrete-to-bolt, concrete-to-bars, and bolt-to-steel. Except for the concrete-to-bars contact pair, all contact pairs were applied to the surface-to-surface contact interaction procedure available in ABAQUS. The HARD contact and PENALTY opinion were assumed for the normal response and tangential direction, respectively. Considering that the friction property in the contact pair of concrete-to-steel was not mentioned in the experiments, the friction coefficients ranging from 0.1 to 0.4 were investigated and calibrated (see Figure 5, take the T4-2 specimen [19] as an example). It was observed that the friction coefficient between the steel girder and concrete plate of 0.3 was closer to the results of the experiments than the other values. Meanwhile, the friction coefficient of 0.25 was specified for the other contact pairs [20]. The embedded constraints were used to describe the concrete-to-bars contact pair, in which the concrete was set as the host region. In this method, the relative slip and debonding of the reinforcements were ignored.
Figure 5. Load–slip relationship of T4-2 specimen for different contact friction coefficients.

3. Calibration and Validation of FE Model

3.1. Calibration of FE Model

To propose a more accurate and reasonable FE model, the effects of the critical plastic CDP parameters, the mesh dependency, and the damage variables were analyzed. In this research, the numerical models were calibrated based on the T1-1 specimen tested by Zhang et al. [19] (the main parameters are summarized in Table 1).

### Table 1. Comparison of the shear resistance obtained from FE results and experiments.

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Specimen</th>
<th>Pretension (kN)</th>
<th>Hole Diameter (mm)</th>
<th>Bolt Diameter (mm)</th>
<th>Compressive Strength of Concrete (MPa)</th>
<th>$P_u^0$ (kN)</th>
<th>$P_u^f$ (kN)</th>
<th>$P_u^f/P_u^0$</th>
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<td>[18]</td>
<td>T1-10-01</td>
<td>30</td>
<td>-</td>
<td>10</td>
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<td>45.1</td>
<td>36.2</td>
<td>0.803</td>
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<td></td>
<td>T1-10-02</td>
<td>30</td>
<td>-</td>
<td>10</td>
<td>33.7</td>
<td>39.2</td>
<td>36.2</td>
<td>0.923</td>
</tr>
<tr>
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<td>45</td>
<td>-</td>
<td>12</td>
<td>33.7</td>
<td>45.4</td>
<td>46.9</td>
<td>1.033</td>
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<td>45</td>
<td>-</td>
<td>12</td>
<td>33.7</td>
<td>49.1</td>
<td>46.9</td>
<td>0.955</td>
</tr>
<tr>
<td></td>
<td>T1-16-01</td>
<td>80</td>
<td>-</td>
<td>16</td>
<td>33.7</td>
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<td>94.6</td>
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<tr>
<td></td>
<td>T1-16-02</td>
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<td>-</td>
<td>16</td>
<td>33.7</td>
<td>89.8</td>
<td>94.6</td>
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<tr>
<td>[19]</td>
<td>T1-1</td>
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<td>24</td>
<td>20</td>
<td>50</td>
<td>207.0</td>
<td>209.4</td>
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<td></td>
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<td>212.6</td>
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<td>T1-4</td>
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<td>50</td>
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<td>T2-1</td>
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<td>16</td>
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<td>26</td>
<td>22</td>
<td>50</td>
<td>231.3</td>
<td>208.7</td>
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<td>230.6</td>
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<td>207.8</td>
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<td>207.9</td>
<td>1.205</td>
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<td>20</td>
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<td>178.5</td>
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<td></td>
<td>T4-2</td>
<td>155</td>
<td>24</td>
<td>20</td>
<td>45</td>
<td>172.8</td>
<td>193.7</td>
<td>1.121</td>
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<tr>
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<td>130</td>
<td>25</td>
<td>22</td>
<td>58.8</td>
<td>243.1</td>
<td>237.4</td>
<td>0.977</td>
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<tr>
<td></td>
<td>NC-M22-G8.8-γ-2</td>
<td>130</td>
<td>25</td>
<td>22</td>
<td>58.8</td>
<td>240.5</td>
<td>237.4</td>
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<td>NC-M27-G8.8-γ-1</td>
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<td>30</td>
<td>27</td>
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<td>298.8</td>
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<td>58.8</td>
<td>312.1</td>
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| Mean ($μ$) | 0.987 |
| Coefficient of variation ($η$) | 0.087 |
3.2. Effect of Key Plastic CDP Parameters

The effect of the variation of the dilation angle (ψ) and the viscosity parameter (v) was evaluated to represent the tested behavior. Previous studies [17,38] have shown that it is suitable to use the value of 30°, 36°, or 40° for the simulation in composite slabs and columns. Figure 6 shows the influence of the change of ψ on the load–slip curves of the T1-1 specimen. It can be found that the value of ψ equals 40° yielded the calculation results closest to the tested results. Figure 7 illustrates the effect of the viscosity parameter (v = 0, 0.0005, and 0.01) on the load–slip curves of the tested specimen. For v = 0, the FE model showed the convergence problems, and v = 0.0005 can simulate the tested specimens reasonably well.

Figure 6. Effect of dilation on load-slip relationship of T1-1 specimens.

Figure 7. Effect of viscosity parameter on load-slip relationship of T1-1 specimens.

3.3. Effect of Mesh Dependency

Three different mesh sizes (2 mm, 3 mm, and 5 mm) of the bolted connector were evaluated to calibrate the mesh sensitivity of the FE modelling. Figure 8 presents the influence of the mesh size on the load-slip response of the T1-1 specimen. It can be seen that the strength obtained from numerical models with the mesh size of 5 mm was lower than the test result. The result of the model with 3 mm was accurate enough to represent the shear behavior of the tested specimen.
3.4. Effect of Damage Variables

No damage, only the compressive damage variable, only the tensile damage variable, and both compressive and tensile damage were considered to analyze the influence of the damage variables on the shear behavior of FE models. Figure 9 shows the effect of damage variables on the shear response of FE modelling. It was observed that only the compressive damage or the tensile damage considered were not consistent with experimental results. The shear strength of FE modelling with no damage was close to the specimen, but the shear stiffness in the plastic stage was significantly different from the tested specimen. In general, both the compressive and tensile damage considered in the numerical model can simulate the tested specimen reasonably well.

3.5. Verification of FE Modelling

To verify the accuracy of the developed numerical models, push-out experiments completed by Chen et al. [18] and Zhang et al. [19,26] were selected and analyzed. Table 1 shows the push-off test parameters and the comparison of shear strength between the experimental and FE analysis. \( P_{\mu} \) and \( P_{\phi} \) are respectively denoted as the ultimate resistance obtained by the tested and numerical models. In this study, the determination of \( P_{\phi} \) was based on the Ref. [29]. It was seen that the shear strength captured by numerical simulation was consistent with the experimental results. The mean value (\( \mu \)) of the ratio \( P_{\phi}/P_{\mu} \) was 0.987, with the corresponding coefficient of variation (\( \eta \)) being 0.087.

The force-slip curves predicted by the numerical models were also compared with those recorded in the tests [18,19,26], as depicted in Figure 10. The trend of numerical curves obtained by FE models (Figure 10b–e) was the same as for the tested curves obtained by Zhang et al. [19]. The numerical curves consist of four distinct stages: (i) the friction transferring force stage (OA)—the natural bonding friction at the steel-to-concrete
interface and the pretension force exerted on the bolt connections were overcome by the external loading at the early stage. The slip at the interface was minimal, and the bolt connections displayed a high stiffness in this stage. The friction loads (point A) can be mainly determined by the bolt pretension; (ii) the slipping stage (AB)—noticeable slippage occurred due to the construction holes fabricated in the specimens; thus, the theoretical slip value equals the sum of the steel-to-bolt clearance and the concrete-to-bolt clearance [24,29]; (iii) the bolt shank transferring stage (BC)—the bolt shank simultaneously bears against the shear, tension, and bending force. The stress state of the bolt is similar to that of the traditional stud; (iv) the failure stage (CD)—the slips increased with the load until the specimens failed. As for tested specimens in Refs. [18,26], the trend of load-slip curves obtained from the FE models was consistent with the relationship predicted by the models in Ref. [19], except that there is no slipping stage (AB) (see Figure 10a,f). This could be explained by the simplified FE modelling without considering the construction holes. In these comparisons, good agreement has been achieved between the numerical and experimental load-slip response in the initial loading stage (OA and AB stage). The stiffness of the bolt connectors calculated by the FE models was slightly higher than the counterparts of the specimens in the BC stage. This may be attributed to the simplification made in the screw threads. Moreover, the theoretical curvature is different from the experimental curvature, due to the slip which occurred at the steel–concrete interface [19].

![Figure 10. Comparison of force-slip curves between numerical models and experiments results](image)

The numerical failure models were also verified against the experimental results [18,19,26], as illustrated in Figure 11. Two failure models were observed in the FE models and experiments: (i) bolt shear-off occurred in the case of the bolt diameter being relatively small (see Figure 11a,b); (ii) concrete failure with the bolt bending when the bolt diameter was relatively large (see Figure 11c,e). It was observed that the failure models predicted...
by the FE models resembled the push-off tests quite well. In general, the numerical analyses and the tested results agreed reasonably well, with only slight discrepancies. Therefore, the developed numerical models can be applied to accurately evaluate the fundamental behavior of bolt connections in the push-off test.

Figure 11. Comparison of the failure modes predicted by numerical models and experimental results: (a) bolt shear failure (T1-16 specimen); (b) bolt shear failure (T2-1 specimen); (c) bolt bending yield (NC-M27-G8.8 specimen); (d) concrete cracks (NC-M27-G8.8 specimen); (e) concrete failure (T4-1 specimen).

4. Shear Behavior of Bolted Connector in LAC

4.1. FE Modelling

The shear behavior of the bolt connection embedded in LAC slab was predicted based on the verified FE modelling shown above. The geometry of the numerical model was the same as in the push-off test conducted by Zhang et al. [26], as shown in Figure 1c. Previous studies [3,24] showed that the construction holes primarily affected the slippage stage and the corresponding slip and displayed little contribution to the shear bearing capacity. In this section, the construction holes were not taken into account in the numerical model. The nominal LAC strength, theoretical density, bolt diameter, and bolt strength grade were set as 40 MPa, 1900 kg/m³, 16 mm, and G8.8 ($f_u = 800$ MPa, $f_y = 640$ MPa), respectively. Meanwhile, the comparison of shear performance in load-slip response, shear bearing resistance, and failure modes between the FE models with NC (FE-NC) and models with LAC (FE-LAC) was also performed.

4.2. Results and Discussion

The load-slip relationship and peak load capacity of the bolted connectors embedded in LAC were compared with those of the bolted connectors embedded in NC, as presented in Figure 12. The green and blue bars represent the shear strength of the bolted connector embedded in NC and LAC, respectively. It was seen that the evolution of the force-slip response in FE-LAC was similar to that in FE-NC, which consists of OB, BC, and CD stages (see Figure 12a). The peak load capacity of the bolted connector in FE-LAC was 84.5 kN, 95.6 kN, 99.4 kN, and 106.4 kN for concrete strength grades of LC30, LC40, LC50, and LC60, respectively, which was 74.3, 75.1, 75.4, and 79.2% of the bolted connector in FE-NC. Therefore, the shear bearing resistance of the bolted connector in FE-LAC was usually less than that of the bolted connector in FE-NC. This is because the LAC possesses a lower
elasticity modulus than the NC with similar compressive strength. Meanwhile, the bolt connector embedded in LAC sustained a weaker confinement than in NC [4].

Figure 12. Comparison of shear behavior between the bolted connector embedded in LAC and NC: (a) force-slip relationship; (b) shear resistance.

Figure 13 describes the comparison of the failure modes between FE-NC and FE-LAC (take M16-C30 and M16-LC30, for example) when the same slip occurred. The output of compressive damage given as DAMAGEC was used to describe the damage failure of the concrete slab. The DAMAGEC variables value ranges from 0 to 1, indicating 0% to 100% damage. The closer the DAMAGEC value is to 1 (red), the more serious the slab damage is. Meanwhile, the equivalent plastic strains (PEEQ) were used to represent the failure of the bolted shear connector. The response of the bolted connector is also limited by a fracture of 0.15 [39].

As can be observed, the external load applied in M16-C30 was 91.4 kN, 106.8 kN, 109.1 kN, and 111.7 kN when the relative slip was 2 mm, 6 mm, 8 mm, and 12 mm, respectively, while in M16-LC30, it was 74.1 kN, 84.2 kN, 84.5 kN, and 81.9 kN, respectively. Thus, it can be concluded that the shear resistance of FE-LAC was lower than that of FE-NC when the compressive strength was the same. Moreover, the concrete slab damage in M16-C30 and M16-LC30 occurred first in the region around the bolted connector, and the damage value and damage area of the slab increased with the increase in the external load. The slab’s damage value and damage area of M16-LC30 were also more significant than that of M16-C30, indicating that the cracks created in FE-LAC appeared earlier and much more frequently than those in FE-NC. As for bolted shear connector, the value of PEEQ in M16-LC30 was lower than that in M16-C30, and its bending deformation in the former slab was more minor than that in the latter when the same relative slip occurred. When the relative slip was 8 mm, M16-LC30 sustained the maximum bearing capacity (see Figure 13c), in which the bolted connector had not reached the fracture strain. Hence, the failure mode of M16-LC30 was a concrete failure, with bolt shear bending. The value of PEEQ for the bolted connector in M16-C30 exceeded the fracture strain (in grey) when relative slip reached 12 mm, manifesting that the failure mode was the bolt shear fracture.
Figure 13. Comparison of failure modes between FE-NC and FE-LAC (M16-C30, M16-LC30): (a) $s = 2$ mm; (b) $s = 6$ mm; (c) $s = 8$ mm; (d) $s = 12$ mm.

4.3. Parametric Study

Parametric studies were conducted to further explore the basic shear behavior of bolted connections embedded in LAC. The design variables, such as concrete strength, concrete density, bolt diameter, and bolt tensile strength, are displayed in Table 2.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Range of Variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive cubic strength ($f_{cu}$)</td>
<td>$f_{cu} = 22.8, 30.5, 36.8, 43.8$ (MPa)</td>
</tr>
<tr>
<td>Concrete density ($\rho_c$)</td>
<td>$\rho_c = 1600, 1700, 1800, 1900$ (kg/m$^3$)</td>
</tr>
<tr>
<td>Bolt diameter ($d_b$)</td>
<td>$d_b = 16, 18, 22, 27$ (mm)</td>
</tr>
<tr>
<td>Bolt tensile strength ($f_{bzu}$)</td>
<td>$f_{bzu} = 800, 900, 1000, 1200$ (MPa)</td>
</tr>
</tbody>
</table>

4.3.1. Effect of Concrete Strength

In this section, $\rho_c$ and $f_{cu}$ were specified as 1900 kg/m$^3$ and 800 MPa, respectively. Figure 14 depicts the influence of the force-slip response and shear strength for models having different concrete strength grades ($f_{cu}$ ranging from 22.8 to 43.8 MPa). It can be observed that the increase in $f_{cu}$ would lead to a significant increase in the shear strength. The load resistance per bolt was improved by 29.5% for 16 mm, 33.7% for 18 mm, 35.5% for 22 mm, and 37.0% for 27 mm, when $f_{cu}$ was increased from 22.8 to 43.8 MPa.
4.3.2. Effect of Concrete Density

Four different ρc, ranging from 1600 kg/m³ to 1900 kg/m³, were considered in this investigation. Figure 15 shows the influence of this variable on the load-slip curves and load-carrying capacity. The change in ρc has no significant effect on the load-slip relationship and shear strength, similar to the conclusions drawn by Ollgaard et al. [4]. When ρc was added from 1600 kg/m³ to 1900 kg/m³, the shear capacity per bolt was increased by only 3.8% for 16 mm, 3.9% for 18 mm, 3.0% for 22 mm, and 2.7% for 27 mm.

4.3.3. Effect of Bolt Diameter

Figure 16 plots the force-slip response and load capacity for models having the same ρc (1900 kg/m³), f_{cub} (800 MPa), and different db, including 16 mm, 18 mm, 22 mm, and 27 mm. It can be seen that noticeable improvements in shear capacity can be observed as db was enhanced. As the increase in db from 16 mm to 27 mm, the shear capacity per bolt exhibited an increase of 70.0% for specimens with LC30, 67.8% for specimens with LC40, 88.0% for specimens with LC50, and 84.9% for specimens with LC60. This increased shear strength after increasing db is due to the enhancement of the bolt effective shear area.
Figure 16. Influence of bolt diameter \((d_b)\): (a) force-slip relationship; (b) shear capacity.

4.3.4. Effect of Bolt Tensile Strength

The influence of \(f_{btu}\) on the load-slip relationship and bearing capacity is illustrated in Figure 17. Four different \(f_{btu}\), including 800 MPa, 900 MPa, 1000 MPa, and 1200 MPa, were considered in this investigation. It can be seen that an increase in \(f_{btu}\) led to an improvement in the shear strength, when \(d_b\) was less than 18 mm. For example, approximately 19.9% and 13.6% increases in the shear capacity can be obtained in specimens with 16 mm and 18 mm, respectively, when \(f_{btu}\) increased from 800 MPa to 1200 MPa. However, when \(d_b\) exceeded 18 mm, minor enhancement in the shear capacity was detected as \(f_{btu}\) increased. This is because the concrete crushing and splitting were the primary failure modes, and the material tensile strength of the bolt was underutilized when \(d_b\) exceeded 18 mm.

Figure 17. Influence of bolt tensile strength \((f_{btu})\): (a) force-slip relationship; (b) shear capacity.

5. Design Recommendations

Currently, there are some design formulae for estimating the bolt shear capacity \((P_u)\) in steel-concrete composite beams, as summarized in Table 3. It was observed that the theoretical design model could be categorized into two methods. The simple equation \((aA_e f_{btu})\) considers the bolt effective cross-sectional area and its material tensile strength as the main parameters for shear capacity (see Equations (4), (7) and (9)). In reality, the shear behavior of the bolt connector in steel-concrete composite beams is affected by the concrete strength, as shown in Equations (5), (6), and (8). It should be noted that these design formulae were developed for bolted connectors embedded in NC, GPC, SFRC, and
UHPC slabs. However, it appears that no guidance is available for the design of the bolted connector embedded in the LAC slab.

Table 3. Design formulae for shear strength per bolt connector.

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Design Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kwon et al. [11]</td>
<td>$P_u = 0.5A_{sc}f_{bhu}$</td>
</tr>
<tr>
<td>Chen et al. [18]</td>
<td>$P_u = 0.23d^{1.76}f_{cu}^{0.29}(0.0007f_c + 0.53)$</td>
</tr>
<tr>
<td>Zhang et al. [19]</td>
<td>$\min\left{P_{us,c} = 0.7A_{sc}\left(f_{ck}E_c\right)^{0.5}, P_{us,b} = 0.62A_{sc}f_{bhu}\right}$</td>
</tr>
<tr>
<td>Liu et al. [24]</td>
<td>$P_u = 0.66A_{sc}f_{bhu}$</td>
</tr>
<tr>
<td>Zhang et al. [26]</td>
<td>$\min\left{P_{us,c} = 0.5A_{sc}\left(f_{ck}E_c\right)^{0.5}, P_{us,b} = 0.76A_{sc}f_{bhu}\right}$</td>
</tr>
<tr>
<td>Yang et al. [28]</td>
<td>$P_u = 0.6\alpha A_{sc}f_{bhu}$</td>
</tr>
</tbody>
</table>

The comparison of shear strength between the predicted results and those tested [9,11,14,18,19,21,26,40] are illustrated in Figure 18, in which $P_{u0}$ denotes the values obtained by different design formulae (Equations (4)–(9)). It was observed that the simple design models with $\alpha$ of 0.5 and 0.6 were relatively conservative ($\alpha = 0.5: \mu = 1.405, \eta = 0.207; \alpha = 0.6: \mu = 1.170, \eta = 0.207$). The formula proposed by Liu et al. [24], where $\alpha$ equals 0.66, was consistent with the test data. However, the value of $\eta$ for this equation ($\eta = 0.207$ for Equation (7)) was larger than other design formulae, indicating that the shear capacity predicted by this design recommendation has a large degree of dispersion. In addition, the design formulae that considered the concrete strength (Equations (5), (6), and (8)) predicted the bolt shear strength well, especially in Equation (5). The values of $\mu$ and $\eta$ for Equation (5) were 1.002 and 0.147, respectively, implying that Equation (5) offered the most reliable predictions among those design formulae.

Figure 18. Comparison of shear strength between the predicted and tests: (a) Equation (4); (b) Equation (7); (c) Equation (9); (d) Equation (5); (e) Equation (6); (f) Equation (8).
According to the comparison results of the design formulae mentioned above, design rules were suggested to predict the ultimate shear capacity of bolted connections embedded in SLACCBs. The expression of the design method is described as:

\[ P_u = \begin{cases} 0.24a_f \beta \rho^{0.29} (0.0007f'_c + 0.53) & d_b \leq 18\text{mm} \\ 0.28d_b \rho^{0.25} & d_b > 18\text{mm} \end{cases} \] (10)

where \( f'_c \) is the yield strength of the bolt connector (in MPa), \( f'_c = 640 \text{ MPa for Gr 8.8, 720} \text{ MPa for Gr 9.8, 900} \text{ MPa for Gr 10.9, and 1080} \text{ MPa for Gr 12.9.} \)

Figure 19 compares the shear strength obtained from the parametric studies with the Equation (10) predictions. A good correlation has been achieved between the FE analysis results and predicted values (\( \mu = 0.971, \eta = 0.074 \)). Therefore, the design recommendation (Equation (10)) can be used to evaluate the shear strength of the bolt connections embedded in SLACCBs.

![Figure 19. Comparison of shear strength between the FE results and Equation (10) predictions.](image)

6. Conclusions

This paper studied the mechanical shear performance of the bolted shear connection embedded in LAC slab by using three-dimensional nonlinear finite-element modelling. The accuracy of the 3D numerical model was calibrated and verified with the existing push-off test results from the literature. Then, the effects of the concrete strength, apparent concrete density, bolt diameter, and tensile strength on the shear behavior of the bolted connector embedded in LAC slab were explored. Based on the present investigation, the main conclusions are drawn as follows:

1. The developed 3D FE modelling considering both the compressive and tensile damage resembled the push-off tests reasonably well for basic shear performance, such as the shear carrying capacity, load–slip relationship, and failure modes.
2. The cracks created in the LAC slab appeared earlier and were more numerous than those in the NC slab, and the connector deformation that occurred in the LAC slab was smaller than that in the NC slab.
3. The evolution of the load–slip response of the bolt connection embedded in LAC slab was similar to that of the bolt connection embedded in NC slab. The shear strength of the bolted connector embedded in the LAC slab was usually lower than that of the bolted connector embedded in the NC slab.
4. The shear resistance of the bolted connector embedded in the LAC slab increased with an increase in the concrete strength, bolt diameter, and bolt tensile strength. The shear strength of the bolt connection was not materially influenced by concrete density.
5. Based upon the extensive parametric studies and the comparison of existing design formulae, an empirical formula was developed for the shear strength prediction of the bolt connectors embedded in LAC. Moreover, good agreement was observed between the numerical analysis results and the predictions.
Further experimental study of the mechanical properties of bolted connectors in LAC plates is of great significance to promote the application of bolted connectors in SLACCBs.

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**Conflicts of Interest:** The authors declare no conflict of interest.

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