Lap Length of Steel Bars Between Novel Prefabricated Wall and Foundation Floor Structures: Full-Scale Experiment

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Abstract. In the context of promoting environmentally friendly practices, prefabricated components are increasingly utilized in civil engineering design and construction. Prefabricated shear walls, in particular, have become prevalent in contemporary residential and office buildings. Following an economic analysis of the construction steps for prefabricated walls, Shanghai Hengxu Energy Conservation Company developed a novel type of prefabricated wall for use in building basements. This innovative wall consists of two thin plates joined by distributed tie pieces, with an internal steel bar cage positioned between them. The core concrete is cast in place between the plates. The wall is designed to withstand water and soil pressures from underground and connects to the basement floor through overlapping steel bars. The effectiveness of the wall's integration with the foundation floor depends on the lap length of the steel bars, a critical factor in structural design and construction. To optimize the lap length, three full-scale specimens were tested, differentiated by varying steel bar lap lengths: 0.8 lae, 1.0 lae, and 1.2 lae, where lae represents the basic anchorage length. The specimens were subjected to horizontal loading to assess failure modes, crack propagation, load-displacement behaviors, and stress-strain responses of the reinforcements. The experimental results revealed that increasing the lap length of the steel bars beyond 0.8 lae does not significantly enhance the bearing capacity or deformation potential of the specimens.

Keywords: Prefabricated assemble walls; prefabricated external shell and cast-in-place core; lap length of steel bar; crack propagation; load-displacement.

1. Introduction

Prefabricated building has a rich history spanning over a century. The concept of building industrialization was first pioneered in Germany in the 1920s, and by 1933, it had gained widespread application in residential construction in France, Russia, and other European countries. Japan introduced prefabricated housing in 1968, establishing a unified modular standard through a series of policies and norms that achieved standardization, mass production, and diversity in concrete prefabricated components and their corresponding buildings [1].

Advancements in science and technology have driven significant developments in the prefabricated industry. In 2003, China's Ministry of Construction issued the "Engineering Construction Standard System" to enhance the standardization of the construction industry. This initiative was further advanced in 2015 when the Ministry of Housing and Urban-Rural Development released the "Evaluation Standards for Industrialized Buildings," laying the groundwork for China's industrialized buildings and promoting the development of building industrialization. The "Evaluation Criteria for Prefabricated Buildings" marked a significant milestone in this evolution [2].

The use of prefabricated components is now expanding from auxiliary parts to structural stress components, such as frame columns and shear walls in primary structures, extending their application from conventional or minor structures to high-rise and super-high-rise constructions. While the prefabrication and assembly level of main underground structures remains relatively low compared to superstructures, prefabricated components are also widely used in underground structures, such as subway tunnel lining structures, foundation pit enclosure components during deep foundation pit excavation in soft soil areas, and foundational pile structures [3].
The construction industry is increasingly focusing on the diminishing demographic dividend. Economic analysis suggests that the highest economic benefit comes from casting core concrete at construction sites rather than in prefabricated factories. Leveraging its inherent advantages, this method aligns with advancements in modern construction techniques and technology. Building components are now mass-produced in factories and subsequently transported to construction sites for assembly. This industrialized workshop production enhances construction quality, production efficiency, and reduces environmental pollution [4].

The prefabricated composite external shell (Fig. 1) represents a novel approach in prefabricated structural systems. Comprised of two thin concrete plates connected by distributed tie pieces and an inner steel bar cage set between the plates, these components are prefabricated and assembled in factories. They are then transported to the construction site for on-site assembly. This wall structure combines an external shell with core concrete casting on-site, thereby augmenting the overall stiffness and load-bearing capacity of the wall [5].

(a) Prefabricated external shell (b) Prefabricated wall after pouring concrete on site

Fig. 1 Novel prefabricated wall

The novel prefabricated wall system presents several notable advantages over traditional reinforced concrete walls, including superior component quality, enhanced processing precision, reduced construction time, and improved performance characteristics [6]. This system, particularly adaptable to various building forms and functions, has become a preferred solution in modern construction. Specifically, the technology has been extended to prefabricated basement exterior walls, which are tailored to resist lateral soil pressures in underground spaces, differing from prefabricated shear walls designed mainly for in-plane seismic and wind loads.

A critical aspect of the novel system is the steel lap joint, which plays a vital role in the structural integrity, facilitating easier construction and transportation. Extensive research on lap forms and lengths in the prefabricated composite shear wall, as well as reinforcement of adjacent components, has led to the development and implementation of relevant codes and procedures [7-9]. However, using prefabricated walls as basement exterior walls and connecting with the basement bottom plate poses significant construction challenges. Designing with conservatively long lap lengths may not be practical on-site due to issues like exposed steel bars at the lower part of the external shell and potential misalignment at the top of the bottom plate. These issues could compromise construction quality and introduce safety hazards.

This paper investigates the feasibility of reducing the lap length of steel bars in the prefabricated external shell when subject to out-of-plane loads. This consideration is crucial for maintaining structural integrity and safety while enhancing construction efficiency. The effectiveness of stress transfer in lap steel bars is dependent on the lap length. The critical lap length is the minimum length necessary for the steel bar to yield without experiencing lap failure, while the limit lap length is the shortest length at which the steel bar can break without failure. For optimal design and safety, the chosen lap length should exceed the critical lap length but remain below the limit lap length [10].

Experimental testing and reliability analysis can establish the appropriate design value for lap length under various conditions [11-13]. Given that a lap joint is a form of anchorage, its length can be calculated by multiplying the anchorage length with a lap length correction factor. Commonly, increasing the lap length is used to enhance joint strength, a practice adopted by various countries to standardize and address reinforcement connection issues effectively.
In connecting a prefabricated external shell wall, which has a 100% reinforcement lap rate, to the foundation floor, optimizing the lap length is essential for ensuring that the structure meets construction standards and functions effectively under normal usage conditions, especially under out-of-plane stress.

2. Specimen preparation and testing scheme

2.1 Specimen preparation

As illustrated in Figure 2, the dimensions and reinforcement details of the test specimens are meticulously outlined. These specimens are constructed using C30 grade concrete and are reinforced with HRB400 grade rebar. The primary variable among the three specimens is the lap length of the foundation reinforcements, while all other parameters are consistent across the specimens. The specific design parameters for each specimen are detailed in Table 1.

![Specimen detail](image)

![DXQ1, DXQ2, and DXQ3 specimen strain gauge position and number](image)

Fig. 2 Different lap lengths of the steel bars of the specimen and the layout of the steel strain gauges

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>Basic plan</th>
<th>Vertical tension reinforcement</th>
<th>Vertical compression reinforcement</th>
<th>Horizontal reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>DXQ1</td>
<td>Foundation inserts 1.2l&lt;sub&gt;ae&lt;/sub&gt;</td>
<td>HRB400 14@200</td>
<td>HRB400 14@200</td>
<td>HRB400 10@100</td>
</tr>
<tr>
<td>DXQ2</td>
<td>Foundation inserts 1.0l&lt;sub&gt;ae&lt;/sub&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DXQ3</td>
<td>Foundation inserts 0.8l&lt;sub&gt;ae&lt;/sub&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.2 Test loading device and loading scheme

2.2.1 Loading device

The tests were conducted in the structural laboratory at Nanchang Hangkong University. To ensure precise measurements, five displacement gauges were strategically positioned on each test specimen. A schematic representation of the loading device and its arrangement is shown in Figure 3.
2.2.2 Load the scenario

Preload Phase: To mitigate factors such as specimen installation and the sensitivity of the instrument's response, each specimen was initially subjected to a preload. This involved applying a load of 8 kN, maintaining it for 1 minute, and then unloading. This process was repeated twice. After completing the preload, the load, displacement, and strain measurements of the actuator were reset.

Formal Loading Phase: This phase consists of two stages—load control and displacement control:(1) Load Control: The target load values are based on the estimated yield load, with increments at 10, 20, 30, 35, 40, 45, 50, 55, 60, and 70 kN. Upon reaching the peak load at each level, loading is paused and maintained for 5 minutes to observe crack development in the specimen.(2) Displacement Control: The loading level for this stage is determined by a multiple of the horizontal displacement ($\Delta$) at the specimen’s top, corresponding to the yield load. This continues until the horizontal displacement reaches a critical capacity marker, such as 1/25 of the cantilever length, or until the specimen collapses. At this point, loading is ceased, and the test is concluded. (3) Assessment of Key Technical Indicators:(1) Horizontal Load Value at the Loading End: Measured using the sensor integrated with the actuator.(2) Horizontal Displacement at the Loading End: Gauged by YHD-40 and YHD-100 displacement sensors.(3) Longitudinal Stress of the Underpin Wall Reinforcement: Monitored using multiple strain gauges attached to the reinforcement's surface.

Crack Development and Width: The specimen’s surface is coated with a layer of white lime slurry to enhance visibility. Cracks are observed visually, and their width is measured with a reading microscope.

Instruments used for each monitoring component are detailed in Table 2.

<table>
<thead>
<tr>
<th>Instrument name</th>
<th>Numbering</th>
<th>Instrument type</th>
<th>Range</th>
<th>Quantity</th>
<th>remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement meter</td>
<td>HDGs 1-3 shown in Fig.3</td>
<td>YHD-100</td>
<td>100 (mm)</td>
<td>3</td>
<td>Horizontal displacement of the loading end of the test piece</td>
</tr>
<tr>
<td></td>
<td>HDGs 4-5 shown in Fig.3</td>
<td>YHD-40</td>
<td>40 (mm)</td>
<td>2</td>
<td>Horizontal displacement of the base of the test piece</td>
</tr>
<tr>
<td>Strain gauges</td>
<td>Shown in Fig.2</td>
<td>120-3AA</td>
<td>2000 (mm)</td>
<td>124</td>
<td>Measure the strain on the surface of the reinforcement</td>
</tr>
</tbody>
</table>
3. **Laboratory tests and test results**

3.1 Specimen pouring and loading

![Figure 4. Concrete pouring in situ](image1)

![Figure 5. Diagram of the device at the test loading site](image2)

Figure 4 shows the scene of pouring concrete in the field, and the test loading field device is shown in Figure 5.

To accurately determine the mechanical properties of the model material, three 100×100×100 mm cubic concrete test blocks were produced concurrently with the model casting. Both the test blocks and the model specimens underwent identical curing conditions. Simultaneously, tensile strength tests were performed on the reinforcement, employing various types of stress bars. Three samples from each batch were tested to ascertain the mechanical properties of the materials. The mechanical properties of both the steel and concrete used are comprehensively detailed in Tables 3 and 4.

In this experiment, a microcomputer system was employed to automate the collection of data, recording both the displacement and strain in the reinforcement and generating the load-displacement curve. Throughout the loading phase, the load at which initial cracking occurred was noted, along with the progression of crack development and the measurement of the primary crack's width at various load levels. The loading direction for all three specimens was oriented towards the south side. Detailed records of deformation and crack evolution for these specimens were meticulously maintained during the tests. Additionally, the ultimate failure modes were observed to assess their functional performance. Detailed descriptions of each specimen's destruction during the test are provided in the subsequent section.

<table>
<thead>
<tr>
<th>Rebar specifications</th>
<th>Design yield strength</th>
<th>Yield strength</th>
<th>Ultimate strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured values</td>
<td>mean</td>
<td>Measured values</td>
</tr>
<tr>
<td>HRB400-10</td>
<td>400</td>
<td>430.83</td>
<td>427.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td>430.32</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>421.78</td>
<td></td>
</tr>
<tr>
<td>HRB400-14</td>
<td>400</td>
<td>416.05</td>
<td>416.42</td>
</tr>
<tr>
<td></td>
<td></td>
<td>417.02</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>416.18</td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Measured mechanical indexes of steel bars (Unit: MPa)

<table>
<thead>
<tr>
<th>Cube compressive strength</th>
<th>Axial compressive strength</th>
<th>tensile strength</th>
<th>Elastic modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured values</td>
<td>mean</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Measured mechanical indexes of concrete (Unit: MPa)
3.2 Evolution of specimen load performance

3.2.1 Crack development and load-displacement relationships

From the onset of loading to the point of failure, the three specimens underwent three distinct stages of structural response: the initial emergence of cracks, the formation of vertical cracks between the mold shell layer and the post-cast concrete, and the development of horizontal through-cracks. Figure 6 illustrates these vertical and horizontal cracks, capturing the progression of crack formation. Specimen DXQ1 and DXQ3 show the vertical crack on the east and horizontal crack on the south side. Specimen DXQ2 shows the vertical crack on the west and horizontal crack on the south side. Table 5 provides detailed information on the load/limit load ratios, as well as the specific locations and characteristics of each stage. The load-displacement curve, which visually depicts these phases, is presented in Figure 7.

![Fig. 6 Characteristics of vertical and horizontal cracks in the specimen](image1)

<table>
<thead>
<tr>
<th>Crack Type</th>
<th>Load/Limit Load Ratio (%)</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical cracks</td>
<td>30-40</td>
<td>Foundation Junction</td>
</tr>
<tr>
<td>Horizontal penetration</td>
<td>&gt;90</td>
<td>Width direction of the formwork wall/foundation</td>
</tr>
<tr>
<td>Vertical cracks</td>
<td>70-75</td>
<td>At the interface between the mold shell layer and the concrete</td>
</tr>
</tbody>
</table>

![Fig. 7 Load-displacement curve](image2)

3.2.2 Strain at the steel bar lap joint

The load-strain relationship between the novel prefabricated wall and the base reinforcement lap of the simulated foundation base plate is shown in Figure 8.
4. Discussion of the results of the trial

4.1 Mechanical performance of underground wall of prefabricated shell with different lap lengths

The load-displacement curves for the three specimens, as depicted in Figure 9, reveal that each exhibits typical bending failure characteristics, alongside substantial deformation capability. For specimen DXQ1, a horizontal crack appeared at a load of 98 kN, about 100 mm below the interface between the mold shell’s underground wall and the foundation, on the south side (loading side). Despite this, the specimen continued to support additional loads until failure, without a significant decrease in load capacity.

Conversely, specimens DXQ2 and DXQ3 displayed horizontal cracks at loads of 126 kN and 115 kN, respectively, also on the south side along the mold shell wall. Following these cracks, there was a noticeable and sudden drop in load capacity; however, the capacity began to gradually increase thereafter. This suggests that the mold shell started to share the load post-cracking, with stress redistributing to the concrete and onsite-poured steel bars. The stress on the steel bars increased rapidly, but there were no indications of bonding issues or slippage, allowing the specimens to maintain their load-bearing capacity.
The load-bearing capacities of the three specimens, ranked from highest to lowest, are as follows: DXQ2, DXQ3, and DXQ1. It was observed that extending the lap length beyond 0.8 lae did not significantly enhance the load-bearing and deformation capabilities of the specimens. Although DXQ1 exhibited the lowest load capacity, largely due to the horizontal cracks penetrating the foundation, there was no evidence of bonding slip in the areas where reinforcements overlapped.

![Fig. 9 Load-displacement curves of specimen](image1)
![Fig. 10. Failure diagram of the junction position](image2)

During the normal service limit state, the loading side of the specimen demonstrated strong bonding with the concrete in the prefabricated external shell, which in turn bore a portion of the load. Notably, before the occurrence of abrupt, one-time penetration cracking, no visible cracks were observed on the surface of the mold shell. The cracks that did appear on the loading side were primarily located at the interface between the mold shell and the foundation. Initially, the formation of microcracks at this location was imperceptible to the naked eye. The earliest cracking in all three specimens was observed along the thickness side, specifically at the junction where the mold shell’s underground wall meets the foundation. These side cracks were fewer in number but expanded rapidly in width, indicating that the intersection of the underground wall and the foundation was a weak point in each specimen.

Table 6 presents both the calculated and the experimentally measured values of the ultimate bending moment for the specimens. The values are specifically calculated from the loading point to the interface between the mold shell and the foundation. The experimentally measured bending moment values listed in Table 6 are derived from the ultimate load values obtained through experiments, while the calculated ultimate bending moments are determined based on the measured strengths of the reinforced steel and concrete materials.

The data in the table suggests that, in the extreme limit state, the minimum bending moment for DXQ1 is 1.46, indicating that all three specimens exhibit commendable bearing capacity on the surface.

<table>
<thead>
<tr>
<th>Plate number</th>
<th>Calculated value of the ultimate bending moment $M_u$-calculation (kN·m)</th>
<th>The measured value of the ultimate bending moment is $M_u$-real (kN·m)</th>
<th>$M_u$-real/$M_u$-calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>DXQ1</td>
<td>130.34</td>
<td>189.86</td>
<td>1.46</td>
</tr>
<tr>
<td>DXQ2</td>
<td>130.34</td>
<td>235.64</td>
<td>1.81</td>
</tr>
<tr>
<td>DXQ3</td>
<td>130.34</td>
<td>216.58</td>
<td>1.66</td>
</tr>
</tbody>
</table>

Figure 10 displays the failure pattern at the junction between the underground wall and the foundation, observed after removing the surface mold shell layer from the loaded and damaged specimen. The figure highlights that the foundation was cast first, and the concrete within the mold shell was poured only after the foundation had developed sufficient strength. This sequential casting process resulted in an interface of newly poured and already set concrete between the foundation and the mold shell layer. Additionally, a foam agent was applied at the end of the mold shell concurrently. This interface showed signs of cross-sectional weakening and poor bonding between the new and old concrete. During the tests, the final cracks in all three specimens predominantly propagated along this interface. Although there were fewer cracks in these areas compared to others, the width of these cracks at the interface expanded rapidly.
4.2 Load transmission of steel bars at lap joints

Lap joints function by transferring load between two reinforcing bars through the surrounding concrete. The force is conveyed from one rebar to the next via bonding with the adjacent concrete, and simultaneously to the other rebar within the joint. This force transmission can induce significant shear stresses and splitting forces within the concrete segment involved. The effectiveness of a lap joint depends on the cohesion along the reinforcement's surface and the concrete's ability to remain intact without cracking or excessive deformation.

The required lap length must exceed the individual reinforcement's exertion length to address potential weaknesses at the joints. Building codes also impose limitations to prevent the placement of joints in less favorable locations and require that the joint strength be at least 125% of the reinforcing bars' strength. This precaution is crucial when reinforcements are closely spaced, at limited transverse intervals, or are welded together, ensuring structural integrity and safety.

In demanding situations, such as sections under high tensile stress in a flexural member or tension members like tie rods, specific standards are necessary. According to the American Concrete Institute (ACI) standards, the required lap length for tension members should be twice the development length of the reinforcement, and the joint must be encased in spiral reinforcement. For bars larger than No. 4 (13 mm diameter), a hook installation is mandated.

For flexural members where the bending moment is highest, the code specifies varying lap lengths—either 1.7, 1.3, or 1.9 times the development length, depending on specific conditions. The code also suggests the use of staggered lap joints, strategically placed away from areas experiencing the greatest tension. This approach ensures enhanced structural integrity and resistance to stress concentrations, especially in critical load-bearing sections of the structure.

5. Conclusions

This chapter presents a comprehensive analysis based on a full-scale test of lap joints in the steel reinforcement of the underground wall within the prefabricated shell. The key findings are summarized as follows:

1. Bending Failure Characteristics: The three specimens tested demonstrated typical bending failure characteristics, occurring at the interface between the mold shell and the foundation. Despite this, the specimens exhibited high bearing capacity and deformation capability, with values exceeding 1.46.

2. Strain Measurement and Tensile Stress: Measurement points for the vertical and U-shaped steel bars were close to their corresponding foundation points, with the vertical steel bar's measurement point near the anchorage end. The strain at this point was lower than that of the U-shaped steel bar at the same location, indicating that the lap area predominantly experienced tensile stress, which was borne by the U-shaped steel bar.

3. Bond Strength and Crack Development: The bond between the concrete in the mold shell and the specimen's mold shell was strong, contributing to the load-bearing capacity. However, the junction of the underground wall and the foundation was identified as a weak point, where cracks developed primarily along this interface. These cracks were fewer but widened rapidly.

4. Bearing Capacities and Lap Length: The bearing capacities of the specimens ranked from highest to lowest were DXQ2, DXQ3, and DXQ1. It was observed that a lap length greater than 0.8l did not significantly affect the specimens' bearing capacity and deformation capability. Additionally, no bond-slip failure was observed in the lap area of the steel bars during the tests.

5. Lap Length of Steel Bars: A lap length exceeding 0.8 l did not significantly influence the bearing capacity or deformation ability of the specimens. Furthermore, no bond-slip failure occurred in the lap area of the steel bars during the tests.
Acknowledgements

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References


