INFLUENCE OF ANCHORAGE DETAILING ON SEISMIC BEHAVIOR OF PRECAST
CONCRETE WALLS REINFORCED WITH SBPDN REBARS

Chuxuan WEI*, Yuping SUN**, Takashi TAKEUCHI*** and Jiayu CHE*

Three precast concrete walls, reinforced by weakly bonded ultra-high strength (SBPDN) rebars within their edge zones, were fabricated and tested under reversed cyclic lateral load to investigate effects of anchorage detailing of SBPDN rebars in wall-base joint and embedment length of SBPDN rebars on seismic behavior of precast concrete walls. Experimental results have indicated that if the embedment length of sheath duct housing SBPDN rebars was five times its diameter, the precast concrete walls reinforced by SBPDN rebars could exhibit as high drift-hardening capability as the cast-in-site shear walls.

Keywords: SBPDN rebar, RC shear walls, precast concrete, drift-hardening capability, wall-base joint, screwed thread

1. INTRODUCTION
Earthquake is one of the most devastating natural hazards that cause great loss of human life and property. An average of 10,000 people is killed by earthquakes each year, while annual economic losses are in the billions of dollars and often constitute a large percentage of the GDP of the affected country [1]. In earthquake-prone regions such as Japan and China, ductile reinforced concrete (RC) structures have been adopted as the favorite seismic resistance solution in the last decades [2, 3]. However, the earthquake engineering community has recently begun to reassess the seismic design procedures, in the wake of several devastating earthquakes such as the Hyogo-ken Nanbu earthquake (17 January, 1995; $150 billion loss and 6,000 deaths), the Wenchuan earthquake (12 May, 2008; $150 billion loss and 69,000 deaths), and the Gorkha earthquake (25 April, 2015; $20 billion loss and 9,000 deaths) [4-6]. From these major earthquakes, structural engineering community has learnt that though most of ductile concrete buildings did not collapse during major earthquake, many of them might be left unreparable due to the large residual deformation caused by stronger earthquakes than the code-prescribed level.

RC shear walls have been recognized as cost-effective way of providing lateral force resistance to buildings in seismic areas around the world. On the other hand, as observed by Wood et al [7], ductile RC shear walls tend to experience large drift under design level earthquakes in order to absorb the input earthquake energy, which leads to significant residual drift and damage tends to accumulate at the wall base plastic hinge region after the earthquake. Furthermore, structural components with large residual drifts were difficult to be repaired after earthquakes, which inevitably leads to high reconstruction cost and business downtime [8].

Therefore, from the viewpoint of reducing the cost of recovery and reconstruction, and making sure that the buildings and infrastructures still maintain sufficient resistance to intense aftershocks after a major earthquake, a new solution is urgently necessary.

One of the authors and his colleagues have proposed an alternative solution, referred to as drift-hardening structures [9, 10]. The core point of the drift-hardening concrete components lies in the utilization of weakly bonded high-strength SBPDN rebars as the primary tensile reinforcement of concrete columns and walls instead of soundly bonded deformed rebars. As shown in Fig. 1, the drift-hardening components have two advantages over the conventional ductile components: (1) drift hardening capability and (2) significantly reduced residual deformation. The former implies stable response without degradation in lateral resistance up to large drift, while the latter means high reparability and low repairing cost.

After verifying effectiveness of the use of SBPDN rebars in enhancing drift-hardening capability of concrete columns [9-11], Sun et al have applied the SBPDN rebars to concrete walls with rectangular section in the light of the revision of AIJ Standard for RC Buildings[12], which permits the use of rectangular shear walls without boundary columns.

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Fujitani et al. [13] have experimentally verified that using the weakly bonded SBPDN rebars as the concentrated rebars placed in the edge zones of wall panel could enhance drift-hardening capability up to 3.0% drift for rectangular RC walls. Meanwhile, Fujitani et al. also confirmed that because the flexure strength of the walls can be greatly enhanced by SBPDN rebars, brittle shear failure is more likely to occur at large drift if the shear reinforcement is not sufficient.

To avoid premature shear failure of RC walls reinforced by SBPDN rebars, a new arrangement of distributed longitudinal (DL) bars in the wall panel was proposed by Wei et al. [14]. The DL bars were not anchored into the adjacent loading beams so that they do not directly resist the axial stress caused by bending moment and reduce the flexural strength of the wall section. Wei et al. have clarified that this arrangement method could also delay the local buckling of DL bars, mitigating the damage of concrete near the wall toes, and preventing premature shear failure of squat walls with shear span ratio of 1.5 and reinforced by SBPDN rebars.

Aiming at the reduction of the unit CO2 emissions of concrete structural component during the construction processes, Che et al. [15] have studied seismic behavior of precast concrete walls with shear span ratio of 2.0 and reinforced by SBPDN rebars. Their test results indicated that the embedment length of 20d (d is the nominal diameter) of SBPDN rebar into the base beams of precast concrete walls could neither effectively prevent the slippage of SBPDN rebars at large drift, nor effectively enhance drift-hardening capability of the precast walls.

The previous studies of concrete columns and walls have also confirmed that to fully take advantage of the ultra-high-strength of SBPDN rebars, each SBPDN rebar needs to be fixed at both ends to prevent the slippage of the SBPDN rebar, whose bond strength is about 3 MPa and only about one-fifth of the bond strength of deformed rebar [16].

The fixation method for SBPDN rebars in previous studies mentioned above is to fix them to steel ring or plate via high strength nuts at both ends as shown in Fig. 2 (a). This fixation method is effective to assure sufficient drift-hardening capability to cast-in-site concrete columns and walls, but may affect construction quality of the concrete around the end plates. Other effective anchorage methods are also desirable to improve constructability of concrete components with SBPDN rebars.

Funato et al. have experimentally verified that if the SBPDN rebar was screwed (see Fig. 2(b)), its bond strength could be enhanced up to 21.8 MPa [16]. Therefore, the lap joint with screwed thread can be considered as is a potential fixation method for SBPDN rebars and is expected to simplify fabrication of concrete components reinforced with SBPDN rebars.

Based on the above-mentioned background, in order to fully take both the advantages of SBPDN bars and the precast construction method, the primary objectives of this paper are, 1) to experimentally clarify the influence of embedment length of SBPDN rebar on seismic performance of the precast concrete walls with SBPDN rebars through cyclic testing of three concrete walls and comparison with the experimental results described in [14], and 2) to verify the effectiveness of anchorage by screwed threads at the ends of SBPDN rebars instead of combination of the end-plate and nuts.

![Fig. 1 Performance comparison between ductile and drift-hardening components](image)

![Fig. 2 Typical anchorage methods of SBPDN rebars](image)

### 2. EXPERIMENTAL PROGRAM

#### 2.1 Outlines of test specimens

To achieve the aforementioned goals of this paper, three cantilever rectangular concrete shear walls were designed, fabricated, and tested under reversed cyclic lateral loading while subjected to constant axial load. Assembly of all the specimens is shown in Fig. 3. To be specific, the wall panel with the concentrated SBPDN rebars and the bottom base were fabricated separately, and two sheath ducts were embedded into bottom base to house SBPDN rebars. The surfaces of concrete at the wall-base joint were made uneven to increase the bond between concrete and the grouting material. The experimental variables included the anchorage method of SBPDN rebars at wall-base joint and the embedment length of the sheath ducts.

![Fig. 4 displays the dimensions and reinforcement details of the specimens](image)

### 2.2 Test program

As can be seen from Table 1 and Fig. 4, all specimens had a rectangular section of 150mm in thickness and 600mm in depth. The specimens had a shear span of 900 mm to give a shear span ratio of 1.5 (position of the loading is shown in Fig. 8 later).

The steel amount of distributed longitudinal (DL) bars and distributed horizontal (DH) bars in wall panel as well as of SBPDN rebars are the same for all test walls. The DL bars consisted of twenty D6 deformed bars uniformly placed with a spacing of 59 mm to give a steel ratio of 0.70%, while the DH bars were comprised of D6 deformed bars with a spacing of 65 mm. The DL bars were anchored at wall panel ends with 180-degree hooks as shown in Fig. 4, and the DH bars were placed in a closed form to sustain shear force and provide effective confinement effect. Eight SBPDN rebars with nominal diameter of 12.6mm were placed at the edge...
Table 1 Primary experimental parameters and main test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$a/D$</th>
<th>$n$</th>
<th>$f_c'$ (N/mm²)</th>
<th>$f_y'$ (N/mm²)</th>
<th>Longitudinal rebars</th>
<th>Concentrated SBPDN rebars</th>
<th>Transverse rebars</th>
<th>Anchorage of SBPDN rebars</th>
<th>$Q_{exp}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WP15-D12H35P</td>
<td>1.5</td>
<td>0.075</td>
<td>34.95</td>
<td>52.95</td>
<td>20-D6</td>
<td>0.70</td>
<td>D6@65</td>
<td>120</td>
<td>262</td>
</tr>
<tr>
<td>WP15-D10H55P</td>
<td>1.5</td>
<td>0.075</td>
<td>34.68</td>
<td>64.96</td>
<td>Not Fixed</td>
<td>0.58</td>
<td>U12.6</td>
<td>100</td>
<td>337</td>
</tr>
<tr>
<td>WP15-D10H55N</td>
<td>1.5</td>
<td>0.075</td>
<td>43.65</td>
<td>66.16</td>
<td>Fixed</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$a/D$: shear span ratio; $n$: Young's modulus ratio; $f_c'$: concrete cylinder strength; $f_y'$: cylinder strength of grouting materials; $\rho_l$: reinforcement ratio of longitudinal rebars; $\rho$: reinforcement ratio of concentrated rebars; $\rho_{sv}$: volumetric ratio of transverse reinforcement; $D_s$: diameter of sheath ducts; $H$: embedment depths of rebars; $Q_{exp}$: measured maximum lateral force (average).

Fig. 3 Assembly of the precast walls

Fig. 4 Reinforcement details of test shear walls (Unit: mm)

zones of wall panel. For specimen WP15-D12H35P and WP15-D10H55P, each SBPDN rebar was anchored to a steel plate (having a thickness of 9mm) by nuts at both ends. As for specimen WP15-D10H55N, instead of the steel plate, each SBPDN rebar was screwed at bottom end along the splice length of 20d (d is the nominal diameter of SBPDN rebar), and the embedment depth of the screwed threads was 40d.

In addition, D6 deformed bars with a space of 65mm (less than 6 times
the diameter of SBPDN rebars) were applied as transverse confinement to prevent premature local buckling of the concentrated rebars.

### 2.2 Material properties

The ultra-high strength SBPDN rebar has a specified yield strength of about 1275 MPa and spiral grooves on its surface as shown in Fig. 5(a). Fig. 5(b) shows the SBPDN rebar with screwed thread.

Mechanical properties of the used steels are summarized in Table 2, and the measured tensile stress-strain relationships of SBPDN rebars are shown in Fig.6. As indicated in Fig.6., the SBPDN rebars maintained elastic behavior till the strain of 0.6%, and did not exhibit apparent yield plateau. Hence, the yield strength shown in Table 2 were determined by the 0.2% offset yielding method.

<table>
<thead>
<tr>
<th>Type</th>
<th>$f_y$ N/mm$^2$</th>
<th>$\varepsilon_y \times 0.01$</th>
<th>$f_u$ N/mm$^2$</th>
<th>$E_s$ kN/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D6 SD295A</td>
<td>387</td>
<td>0.21</td>
<td>517</td>
<td>184</td>
</tr>
<tr>
<td>U12.6 SBPDN</td>
<td>1379</td>
<td>0.85</td>
<td>1467</td>
<td>212</td>
</tr>
</tbody>
</table>

$f_y$: yield stress; $f_u$: ultimate stress; $E_s$: Young’s modulus; $\varepsilon_y$: yield strain (0.2% offset strain);

Ready-mixed concrete made of Portland cement and coarse aggregates with maximum aggregate size of 20mm was used to make the specimens. The concrete strengths were evaluated at the same day of loading by testing three standard cylinders (diameter: 100mm, height: 200mm), which were cured under the same condition as the shear walls. The measured concrete strengths are shown in Table 1 for each specimen.

A cementitious non-shrinkage mortar was used as the grouting material for this experiment. The cement was completely mixed up by a hand mixer more than 120 second as is recommended. The compression strengths of grouting material were evaluated at the same day of loading by testing three standard cylinders (diameter: 50mm, height: 100mm), which were cured more than 21 days under the same condition as the shear walls, and the measured compressive strengths are shown in Table 1.

### 2.3 Test setup and loading program

The loading program is shown in Fig. 7. To find out the first flexure or shear crack, the lateral loading was initially controlled by force before reaching drift ratio of 0.125%. After then, two complete loading cycles were applied at each specified drifts (0.25%, 0.375%, 0.5%, 0.75%, 1%, 1.5%, and 2%), and one cycle was applied at each level of targeted drift after drift ratio was beyond 2%.

The experiments were conducted using the setup shown in Fig. 8. The loading apparatus was designed to subject the shear wall to reversed cyclic lateral load and constant axial compression. A vertical hydraulic jack with a capacity of 1000 kN, which was connected to stiff loading frame via a roller, was used to apply constant axial compression. The reversed cyclic lateral load was applied by two 500 kN horizontal hydraulic jacks. The lateral loading was controlled by drift ratio (R), which is defined as the ratio of the lateral displacement (A) at the loading point of lateral force to the shear span (a) of walls.

<table>
<thead>
<tr>
<th>No.</th>
<th>Stress (N/mm$^2$)</th>
<th>Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No.3</td>
<td></td>
<td></td>
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</tbody>
</table>

Fig. 5 Surface of U12.6 (SBPDN1275/1420) bars
The mechanical properties of the used steels are summarized in Table 1. The ultra-high-strength ultra-high-performance concrete (UHPC) used in this experiment has a specified compressive strength of more than 1200 N/mm² and a water absorption of 0.6%, and the SBPDN rebar with screwed thread was used to apply constant axial compression. The reversed cyclic lateral load and constant axial compression. A vertical hydraulic jack with a loading apparatus was designed to subject the shear wall to reversed cyclic lateral load and constant axial compression. The cements were completely mixed up by a horizontal displacement of the bottom loading stub.

The experiments were conducted using the setup shown in Fig. 6. To find out the first flexure or shear crack, the lateral loading was initially controlled up. Finally, at the drift ratio of 5.2%, obvious expansion of flexure and shear crack was dominant for all specimens, and all test walls exhibited similar crack pattern finally.

As shown in Fig. 11, since the shear span ratio of these specimens are relatively low (a/D=1.5), shear crack was dominant for all specimens, and all test walls exhibited similar crack pattern finally.

For specimen WP15-D12H35P, its first flexure crack run through the web side of each specimen. In these figures, the grids have a spacing of 50 mm, the red lines and blue lines represent the cracks that were drawn at the peak drifts of the targeted levels in both push and pull direction of lateral loading, respectively, while the blacked portions express the spalled-off cover concrete.

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shear cracks were observed. Due to the significant degradation in lateral resistance, the test was terminated at the drift ratio of 5.5%. On the other hand, when the drift ratio of specimen WP15-D10H55N reached 3.5%, less damage than that of specimen WP15-D10H55P was observed at the wall-base joint. Finally, at the drift ratio reached 5.0%, the SBPDN rebar was pulled out from the grouting materials.

### 3.2 Lateral force – drift ratio hysteretic behaviors

The measured lateral resistance force \( (V) \) versus drift ratio \( (R) \) relationships of all specimens are shown in Fig. 13., while the measured lateral capacities that averaging the peak lateral forces in both directions are shown in Table 1.

As can be seen from Fig. 13, for specimen WP15-D12H35P with shorter embedment length of sheath ducts, its lateral force reached peak at the drift ratio of 1.5% where the damage of concrete was observed at the wall-base joint. From that drift ratio on, the lateral resistance of the specimen began to degrade gradually due to the pulling out of sheath ducts.

It is noteworthy that after the damage of concrete was observed at the wall-base joint, the unloading curves for all test walls began to become irregular. Specifically, during unloading, the lateral force decreased to zero fast and leave large deformation, but applying a small reverse lateral load (about 10 kN) pushed the specimen back to its original position, which implies the recovery of the pull-out sheath ducts.

For the specimens with longer embedment length of sheath ducts, the lateral forces of them stably increased along with drift, and two specimens exhibited drift-hardening capability up to the drift ratio of 3.0%. In particular, the lateral resistance of specimen WP15-D10H55P with SBPDN rebars being anchored to steel plates by nuts exhibited no obvious decrease till the drift ratio of 4.8%.

The irregular unloading curves were also observed in specimens with longer embedment length sheath ducts during unloading from the drift ratio of 2.0% on, which indicated that longer embedment length could not completely prevent the pulling out of the sheath ducts, but did provide sufficient bond strength to assure the precast walls a drift-hardening capability till drift ratio of 3.0%.

To better see the influence of the anchorage detailing on the seismic performance, the envelope curves in both pull and push direction of hysteretic loops are compared in Fig. 14. The reference specimen W15 was previously tested by the authors [14], having the same material strength, geometric dimensions, shear span ratio, and axial load level as the
specimens described in this paper, but was cast-in-site wall.

It is obvious from Fig.14 that embedment length of 25d couldn’t prevent the pulling-out of sheath ducts, resulting in degradation in lateral resistance of the precast wall at large drift. However, embedment length of 40d (about five times of the diameter of sheath duct) could delayed the pulling-out of sheath ducts and ensure nearly the same drift-hardening capability up to 3.0% drift as the cast-in-site wall.

Moreover, Fig 14 also shows that if the embedment length is long enough, the fixation of SBPDN rebars by steel plate and nuts and by screwed thread would provide the same drift-hardening capability to the precast concrete walls reinforced by SBPDN rebars.

3.3 Strains measured in reinforcements

The measured axial strains of SBPDN rebars for all specimens on the initially tensile side are shown in Fig.15. The superimposed red dashed horizontal lines represent the yield strain.

It is apparent that the axial strains of SBPDN rebars exhibited similar behaviors to the lateral resistance (see Fig. 13). The axial strain measured in specimen WP15-D12H35P exhibited stable increase till the drift ratio of 1.5%. But from that drift on, the axial strains remained constant due to the pulling-out of sheath ducts and decrease gradually along with drift ratio.

The gradual decrease of steel strain implies that the stress sustained by SBPDN rebars does not develop along with drift, hence resulting in degradation in the lateral resistance by SBPDN rebars at large drift.

On the other hand, as can be seen from Fig. 15(b) and (c), the axial strain of SBPDN rebars in these two specimens exhibited stably increase till drift ratio of 3.0%, and the SBPDN rebars in specimen WP15-D10H55P and WP15-D10H55N yielded at the drift ratio of 3.8% and 3.2%, respectively. In addition, even after the yielding of SBPDN rebars, the lateral resistance of specimen WP15-D10H55P maintained increasing until the drift ratio of 5.2%. As for specimen WP15-D10H55N with SBPDN rebars being anchored by the screwed threads, at the last loading cycle after drift ratio beyond 3.5%, the measured axial strain remained the same along with the increased drift ratio, which indicated commencement of the slippage between SBPDN rebars and the grouting material from drift of 3.5%.

The distributions of strains of SBPDN rebars along height of walls are shown in Fig. 16. The strains measured in SBPDN rebars of precast specimens showed similar behaviors to those of the cast-in-site specimen W15 till drift ratio of 0.75%. After that drift level, they all exhibited a nearly uniform distribution along the wall height, but the strains of SBPDN rebars in WP15-D12H35P stop increasing along with drift ratio when the damage of wall-base joint was confirmed. As for the precast walls WP15-D10H55P and WP15-D10H55N, their concentrated rebars indicated almost the same distribution as the conventional fabricated specimens, but the strain of SBPDN rebars in wall WP15-D10H55N increased faster than those in specimen WP15-D10H55P after drift ratio reached 2% as can be seen in Fig. 16. This could be suspected that due to the screwed thread of SBPDN rebars started at the position of 20d (where d is the diameter of SBPDN bars) under the bottom end of wall panel, while the steel plate in
3.4 Residual drift ratio

Fig. 17 summarized the average residual drift ratio of the plus and minus directions measured at each targeted drift ratio. Since the hysteretic behaviors of the precast specimen became irregular during unloading, the residual drift ratios were measure when the lateral resistance force of the specimen became zero. As can be seen from Fig. 17, the residual drift ratios of all specimens were controlled under a very low level till 1.5% drift level. However, from that drift ratio on, the residual drift ratio of the specimen WP15-D10H55P increased sharply because the sheath ducts were lifted up or pulled out as shown in Fig. 12. The residual drift ratios of specimens WP15-D10H55P and WP15-D10H55N exhibited gradual increase till drift of 3.0%, and the increase slope was a little shaper than that of the reference cast-in-site wall W15 due to the damage around the sheath ducts at large deformation. Besides, for the specimen WP15-D10H55N, whose SBPDN rebars were anchored by screwed threads, its residual drift ratio increased more sharply from drift ratio of 2.0% on than that of the specimen WP15-D10H55P with WBPDN rebars being anchored by steel plates and nuts. As shown in Fig.16, the axial strain of SBPDN rebars in specimen WP15-D10H55N increased faster and beyond its elastic stage (about 0.6%) at the drift ratio of 2%, therefore even the SBPDN rebars in specimen WP15-D10H55N did not reach its 0.2% offset yield strain (0.85%), it left a little more residual deformation than that of specimen WP15-D12H35P.

Fig. 15 Measured strains-drift ratio relationships of concentrated rebars

WP15-D10H55P were embedded 40d under the wall base (details shown in fig.4), so even the concentrated rebars had the same overall embedment depth in wall WP15-D10H55NP and WP15-D10H55N, the SBPDN rebars fixed by the screwed thread was easier to slip and as can be noted that, even in the wall-base joint zone, the strains of SBPDN rebars could still indicate an almost uniform distribution.

Fig. 16 Strains distribution of concentrated rebars
4. ASSESSMENT OF SEISMIC PERFORMANCE OF PRECAST CONCRETE WALLS with SBPDN REBARS

4.1 Description of analytical method

Previous research [14] had proved that, due to the neglect of the slippage of SBPDN bars at large deformation, analytical methods prescribed in current design codes [12, 17] could not accurately evaluate ultimate capacity of concrete walls reinforced by SBPDN rebars. Therefore, to reasonably evaluate the seismic behavior and ultimate lateral capacity of precast concrete walls reinforced by SBPDN rebars, analytical method that can take account of the effect of slippage of SBPDN rebars is necessary. In this study, the finite springs method (FSM) refined by Kitajima [18] will be applied to predict the cyclic response of the precast concrete walls.

To evaluate the overall seismic behavior of precast concrete walls reinforced with SBPDN rebars, the following assumptions are made: 1) concrete only resist compression stress, 2) only the concrete plane remains plane after bending, 3) the stress-strain relation of concrete follows the model proposed by Sakino and Sun [19, 20], which can take the confinement effect of stirrups into consideration, 4) the bond-slip relationship of the SBPDN rebars follows the model proposed by Funato et al [16], 5) the modified Menegotto-Pinto model [21] is utilized as the constitutive model of SBPDN rebars, 6) the axial strain and stress of the longitudinal rebars are uniformly distributed in the plastic hinge region, while the proportion of flexure deformation and the length of the potential plastic hinge region are determined by the method proposed by Fukuhara et al. [22], 7) the concentrated rebars are completely fixed at both ends without slippage, 8) based on the previous research [23], only the four D6 distributed longitudinal bars located at the boundary zone of wall panel participate in resistance of flexure.

4.2 Verification of the numerical analytical method

To verify the reliability and accuracy of the presented analytical method for evaluating the seismic performance of drift-hardening precast concrete walls, the results calculated by the analytical method will be compared with the experimental ones. Comparison will be made in three important aspects, the lateral force $V$ versus drift ratio $R$ relationship, the strains in concentrated rebars, and the residual drift ratio.

For comparison, the theoretical results calculated by ignoring the slippage of SBPDN rebars are also shown in Fig. 18 and expressed by the legend “An.NS”, while the theoretical predictions considering the effect of slippage of SBPDN rebars are expressed by the legend “An.S” and shown in Fig. 18. One can see by comparing Fig. 18 (a) that if the slippage of SBPDN rebars is neglected, the predicted ultimate lateral resistances for all specimens are about 40% higher than the results in which the slippage of SBPDN rebars are considered. The reason for this significant discrepancy shown in Fig. 18 (b), which displays comparison between the measured strains of SBPDN rebars and the analytical results. Ignorance of the slippage tended to promote the development of longitudinal tensile strain in SBPDN rebars, and resulting in much larger stress and flexural resistance provided by SBPDN rebars at smaller drift level.

One the other hand, the theoretical predictions calculated by the presented method all exhibited very good agreement with the experimental results till the drift ratio of 2.0%. From that drift ratio on, since it is assumed that the concentrated SBPDN rebars are completely fixed at both ends without slippage, the analytical results showed a little higher lateral resistance than the test results. The lateral resistance of specimen WP15-D12H35P reached peak at the drift ratio of 1.5%, where the damage of concrete was obviously observed at the wall-base joint. From that drift ratio on, the lateral resistance of the specimen began to degrade gradually, and the discrepancy between the analytical and measured results of specimen WP15-D12H35P became larger got later, implying that the shear ducts with embedment length of 310mm could not completely be fixed within the wall-base join, and preventing the SBPDN bars from providing more lateral resistance to the precast walls at large drift ratio.

As for specimens WP15-D10H55P and WP15-D10H55N with longer embedment length of 500mm for SBPDN rebars, after drift ratio of 2.0%, the analytical results tend to overestimate the experimental loops about 10% at drift ratio of 3.0%. This difference between the theoretical results and the measured ones could be attributed to the negligence of the damage of concrete at large deformation in the presented analytical methods.

One can also see from Fig. 18(c) that if the slippage of SBPDN rebars were ignored, the yielding of concentrated SBPDN rebars would cause larger residual drift than experimental results. Meanwhile, the predicted residual drift ratio by the presented method seemed to underestimate the residual drift ratios because of the ignorance of damage of concrete around the wall-base joint and the shear ducts, which resulted in the attenuation of lateral resistance of the precast walls at large drift and causing relatively large residual deformation.

5. CONCLUSIONS

This paper is intended to present a simple but efficient way to make drift-hardening precast concrete walls, and to experimentally investigate the influence of anchorage detailing of primary longitudinal rebars on seismic performance of precast walls reinforced with SBPDN rebars. Three precast concrete walls were fabricated and tested under combined reversed lateral loading and constant axial compression, with the anchorage detailing of SBPDN rebars as the primary experimental variable. Based on the experimental results and analytical work described in this paper, the following conclusions can be drawn:

1) If the embedment length of shear ducts that accommodate SBPDN
Regardless of the difference of anchorage detailing, no severe damage was observed around the anchorage portions of specimen WP15-D10H55P and WP15-D10H55N with embedment length of 510 mm. Both specimens exhibited excellent deformability up to about 5.0% drift without obvious degradation in the lateral resistance. The residual deformation of the precast walls was identical to that of the cast-in-site walls till the drift level of 2.0%. After that drift level, 3) 

2) Regardless of the difference of anchorage detailing, no severe damage was five times the duct diameter, giving an embedment depth of 510 mm, both anchorage methods (by combination of the steel plate and nuts or by screwed threads at end portion) could provide sufficient bond strength to SBDPN rebars, and ensure the precast concrete walls drift-hardening capability till drift ratio of 3.0%.

Fig. 18 Comparisons between tested and analytical results
the residual drift of precast walls became 30% larger than the cast-in-site wall due to the slippage of anchorage portions at large deformation. The anchorage by nuts and steel plate was more effective than that by screwed threads at ends of SBPDN rebars in aspect of controlling the residual deformation at larger drift than 2.0%.

4) Analytical work showed that if the slippage between SBPDN rebars and concrete is reasonably considered, the presented analytical method could give relatively accurate prediction of the lateral resistance of the precast drift-hardening concrete walls up to drift ratio of about 3.0%.

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REFERENCES
APPENDIX A

Fig. A1 displays the concept of FSM proposed by Katajima et al [18]. As can be seen from Fig. A1, in the FSM, a concrete shear wall under seismic loading will be divided into three zones: 1) the elastic zone where the curvature is linearly distributed but ignored as calculating the displacement, 2) the plastic hinge zone with a length of 0.45D (D is the depth of wall section) [23], and 3) the joint zone that simulates the wall-base joint. The elastic and joint zones are further divided into finite spring segments along the height of wall panel.

For a given curvature $\phi$ and the concrete strain $\varepsilon_{ce}$ at the center of wall section, the concrete strain $\varepsilon_{ci}$ located at the center of the i-th row of SBPDN rebar in the plastic zone can be obtained following the plane-remain-plane assumption. The actual axial strain $\varepsilon_{si}$ for each row of SBPDN rebars can then be obtained as follows to take the slip effect into account:

1) Give an initial slip $S_{0i}$ and stress $\sigma_0$ to the upper end of the joint zone, and represented them as $S_0$ and $\sigma_0$, respectively.

2) Calculate the stress $\sigma_{i1}$ and slip $S_{i1}$ of the adjacent segments by Eq. (1) through Eq. (4) until $k = n$ (the number of segments). In these equations, $\tau(S)$ is the bond stress corresponding to the slip $S$, and $f'_{si}$ is the inverse function of stress-strain relation of SBPDN rebar to calculate the strain $\varepsilon_{si}$ corresponding to the given stress $\sigma_{si}$ (see Fig. A2).

3) If the boundary condition $S_{i1} = 0$ is met, the rebar stress $\sigma_{i1}$ is taken as $\sigma_0$, and then proceed to step 5).

4) If the boundary condition $S_{i1} = 0$ is not met, assume a new rebar stress for $\sigma_{i1}$ and return to step 2).

5) Obtain the rebar strain $\varepsilon_{si}$ in the hinge zone by $\varepsilon_{si} = f'_{si}(\sigma_{si})$ for the rebar at the bottom end segment of the elastic zone.

6) Calculate the stress $\sigma_{si}$ corresponding to $S_{i1}$ following the procedures as step 2) through step 4).

7) If $\sigma_{si} = \sigma_0$, the $\varepsilon_{si}$ obtained at step 5) is the actual axial strain of the i-th row of SBPDN rebars, then proceed to the calculation of the actual strain for the other SBPDN rebars; if not, assume a new slip $S_{0i}$ and repeat the step 1) through step 8).

After obtaining the actual strains for all SBPDN rebars, the steel stress can be obtained through stress-strain model, and the resultant axial force and moment sustained by concrete and SBPDN rebars can be obtained by summing up the resistances by concrete fibers and SBPDN rebars layers. Then applying the lumped hinge model to the wall, the lateral drift $R$ and the lateral resistance $V$ corresponding to the given curvature $\phi$ and the concrete strain $\varepsilon_{ce}$ can be obtained simply [18].

APPENDIX B

The bond stress ($\tau$) - versus slip ($S$) model for SBPDN rebar was proposed by Funato et al. [16], and is shown in Fig. B1 and Fig. B2. The originally proposed $\tau$-$S$ model was developed on the basis of pull-off tests of SBPDN rebars embedded in concrete with compressive strength of 40MPa on average. When applying this model to the specimens described in this study, effect of concrete strength on the bond strength ($\tau_{max}$) is taken into account in form of

$$\tau_{max} = 3.0 \sqrt{\frac{f'_{c}}{40}}$$

where $f'_{c}$ is strength of unconfined concrete and $K$ is strength enhancement ratio, which represents the confinement effect by transverse reinforcement on the core concrete [20].