SEISMIC DESIGN OF EARTHQUAKE RESILIENT COUPLED SHEAR WALL WITH REPLACEABLE COUPLING BEAM AND NON-DAMAGED WALL FOOT

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ABSTRACT

Traditional structures could not meet the new demand of sustainable earthquake resistance, whereas earthquake resilient structure is an effective way to meet this demand. The research of earthquake resilient coupled shear wall is an important part of the earthquake resilient structure research, however, the research nowadays focuses on the replaceable coupling beam, whereas the damage in wall foot has not been solved, so it’s difficult to realize the earthquake resilience in current research. Based on the previous study, earthquake resilient coupled shear wall with replaceable coupling beam and non-damaged wall foot was designed. Damage in the wall foot was avoided, so the damage would be concentrated in the replaceable device of coupling beam under earthquakes, and the repair would be finished rapidly after earthquakes. Theoretical analysis and experimental verified numerical simulation were used to study the damage control mechanism, damage concentration capacity, earthquake resilient capacity of the earthquake resilient coupled shear wall. The simulation and design method were proposed and verified, providing the application approach of earthquake resilient coupled shear wall to sustainable earthquake resistant structures.

Keywords: Earthquake resilience; replaceable coupling beam; self-centering; coupled shear wall; design method

1. INTRODUCTION

The structure design method to prevent the structural collapse under earthquakes is necessary. However, the damage after earthquakes is hard to repair or the repair time would be so long that to disturb the normal operation of the building. A new demand of sustainable earthquake resistance is proposed in current structural seismic design. Earthquake resilient structure (Lu et al. 2013) is an innovative type of structure developed to meet this demand, in the purpose to restore the structure immediately after strong earthquakes. Little repair is needed and the repair time is very short in the way of self-centering, rocking or damage replacement. Previous research proved the structure with replaceable components could achieve the earthquake resilience (Fortney et al. 2004 and 2007; Fortney 2005; Tang and Manzanarez 2001; Madaniel et al. 2003; Shen et al. 2011; Mansour et al. 2011). When parts of the structural component are replaced by replaceable devices, the structural component will act as a structural “fuse”. The damage concentrates in the replaceable device during an earthquake, and the damaged part could be replaced conveniently after the earthquake, which could have less repair time than the traditional structure.

As for the reinforced concrete (RC) shear wall structures, the structure with replaceable coupling beams was proposed by the authors (Lu et al. 2013; Chen and Lu 2015a and 2015b; Chen et al. 2016). The replaceable coupling beam consists of the central replaceable device and the non-yield connecting beams at the two ends. The replaceable device is allowed to undergo plastic deformation and the end-

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plate bolt connection is used to implement replacement of the damaged device after strong earthquakes. The planar coupled shear walls and the spatial tube with replaceable coupling beams were tested and a real engineering project with replaceable coupling beams were analyzed. The results showed that the plastic deformation of the replaceable coupling beam mainly concentrated on the replaceable device.

However, as observed in previous researches, severe damage at wall foots was still observed in the shear wall structure with replaceable coupling beams (Chen et al. 2016; Chen and Lu 2015b). The wall foot damage is hard to repair, which influences the quick recover capacity of the coupled shear wall. To prevent the damage from occurring in the wall pier, a non-damaged wall foot was proposed in this paper, and the seismic performance of the earthquake resilient coupled shear wall with replaceable coupling beam and non-damaged wall foot was analyzed. Damage in the wall foot was avoided, so the damage would be concentrated in the replaceable device of coupling beam under earthquakes, and the repair would be finished rapidly after earthquakes. Theoretical analysis and experimental verified numerical simulation were used to study the damage control mechanism, damage concentration capacity, earthquake resilient capacity of the earthquake resilient coupled shear wall. The simulation and design method were proposed and verified, providing the application approach of earthquake resilient coupled shear wall to the sustainable earthquake resistant structures.

2. DESIGN CONCEPT

The proposed earthquake resilient coupled shear wall is illustrated in Figure 1. The center part of the RC coupling beam is removed and replaced by a replaceable device, in the aim to concentrate the damage and dissipate earthquake energy. The plastic zone of the wall foot is also removed, which is replaced by the rubber base. The post-tensioned tendons (PTs) are inserted in the wall pier from the top floor, through the removed wall foot and anchored in the base, providing the capacity compensation and self-centering ability. To prevent the local failure of concrete, wall corners at top of the removed wall foot are armored by steel jackets, and steel plates are casted on the top of the concrete base. Under strong earthquakes, the interface between the steel jacket and the steel sheet would be separated and the rocking happens. The system is capable of preventing the wall foot from damage and concentrating the damage in the replaceable device when subjected to strong earthquakes, whereas the damaged device could be conveniently replaced by a new one and the system recover to its normal operation quickly.

Figure 1. Description of the earthquake resilient coupled shear wall
2.1 Design assumption

For the simplification of the earthquake resilient coupled shear wall design, some assumptions are defined:

(1) Strong lateral bracing prevents out of plane deformation of the shear wall, so the design method is conducted for the in plane deformation.
(2) The distribution rebar forces contribute to the shear wall capacity, however, the rebar is small and easy to buckle under compression, so the rebar compression force contributed to capacity is neglected. The rebar tension force near the neutral axis is also very small, so the calculated rebar tension force zone is assumed in the area outside $1.5x$ (MHUPRC 2010a), while $x$ is the length of concrete equivalent compressive zone.
(3) To control the concrete compression stress in filled steel base and make it in the elastic state, the concrete compression stress $f_{cb}$ corresponding to the limit state of the shear wall is supposed to be $0.27f_c$ (MHUPRC 2010a), which is less than the measured elastic range of the concrete (the elastic range limit strength $0.3f_c$ divided by safety factor 1.1 (MHUPRC 2010a)), and $f_c$ is the concrete compression strength. Also, the limit compression force contributed by the shear wall concrete is calculated using the stress $0.27f_c$, preventing the damage occurred in the shear wall’s RC parts.
(4) The steel compression stress $f_{ys}$ in the limit state of the shear wall is defined as 0.5 times the steel yield strength $f_y$, promising no plastic behavior occurred in the concrete filled steel base. Also, the limit distributed rebar stress $f_{ysd}$ is defined as $0.5f_y$.
(5) To concentrate the plastic behavior in the replaceable device and prevent any damage in the area outside it, the force contributed by PTs in the limit state of the shear wall is calculated using initial prestress $f_{yp}$.

2.2 Design method

Providing the compression force $N$ and the moment $M$ of the wall piers, and the moment of the coupling beam $M_b$ are obtained from the structure through the forced-based or displacement-based method, design of the earthquake resilient coupled shear wall could be conducted as follows.

2.2.1 Removed RC area

![Figure 2. Dimension mark of the combined replaceable devices in shear wall](image)

For the earthquake resilient coupled shear wall with replaceable coupling beam and non-damaged wall foot, the proper removed RC area should be considered firstly. As in the previous research by authors (Chen and Lu 2015a) height of the non-yield connection beam $h_b$ and the span of the replaceable device $l_r$ are defined according to the strength and stiffness equivalence of the replaceable coupling beam to the conventional coupling beam (see Figure 2) (Chen et al. 2016). To concentrate the plastic damage, the height of the removed wall foot $h_r$ should be larger than the length of the plastic hinges.
The width of the removed wall foot \( l_c \) was determined by the region where the compression strain of the concrete exceeds \( \varepsilon_c \), in order to avoid the concrete crush. The minimal height and width of the removed wall foot are calculated as follows:

\[
h_c \geq 0.2h_u + 0.044H
\]  

\[
l_c \geq 1.5(1 - \frac{\varepsilon_c}{\varepsilon_{c,\text{max}}})\xi h_u = 1.5(1 - \frac{\varepsilon_c h_p}{\theta_c h_h})\xi h_h
\]  

where \( h_u, H \) are the cross-section depth and effective height of the shear wall respectively (Paulay and Priestley 1992), \( \varepsilon_{c,\text{max}} \) is the maximum compressive strain at the extreme compression edge, which equals the ultimate curvature plus the equivalent compression zone height \( \xi h_h \) of the bottom section of the shear wall. The ultimate curvature of the section is defined as the ratio of the maximum story drift \( \theta_c \) to the height of the plastic hinge \( h_p \).

### 2.2.2 Wall pier design

The flexural capacity of the wall pier \( M_p \) is decided by Equation 3, and the moment capacity contributed by Post-tensioned tendon \( M_p \) and distributed rebar \( M_w \) are given by Equation 4 and 5. The PT force \( T_p \) and the tension force of the distributed rebar \( T_w \) can be calculated using Equation 8 and 9. The concrete contributed moment are \( M_c \) (Equation 6) of the shear wall web concrete and \( M_b \) (Equation 7) transferred by the rubber base, where \( C_b \) is the compression force of rubber base. In the equations, \( f_y \) and \( A_y \) are the initial prestress and area of PT, \( \rho_w \) is the reinforcement ratio of distribution rebar, \( b_w \) is the shear wall thickness, \( t \) is the outside steel tube thickness of the concrete filled steel base, and \( h_u, L, L_p, p \) are shown in Figure 3.

\[
M_p = M_p + M_w + M_c + M_b \geq M \quad (3)
\]

\[
M_p = T_p (p + 0.5(h_u - 2L_p)) \quad (4)
\]

\[
M_w = T_w (1.5x - 0.5(h_u - 2L_p)) + 0.5(h_u - 2L_c - 1.5x)) = 0.75T_w x \quad (5)
\]

\[
M_c = 0.27\alpha_f f_y b_w(x(h_u - 2L_c - 0.5x)) \quad (6)
\]

\[
M_b = C_b(c + 0.5(h_u - 2L_c + L_b)) \quad (7)
\]

\[
T_p = f_y A_y \quad (8)
\]

\[
T_w = (h - 2L_c - 1.5x)b_w f_y \rho_w = 0.5(h - 2L_c - 1.5x)b_w f_y \rho_w \quad (9)
\]

In this state, the vertical capacity of the wall pier should satisfy Equation 10:

\[
C + C_b - T_p - T_w \geq N \quad (10)
\]

### 2.2.3 Coupling beam design

As stated in the previous study by authors (Chen et al. 2016), the moment capacity of the non-yield section \( M_n \) is calculated using Equation 11, and the yield shear force \( V_y \) of the replaceable device is
given by Equation 12, where $\xi$ is the amplified coefficient recommended by the design code (MHUPRC 2010a), $L$ is the length of the coupling beam.

\[
M_e = \xi M_b 
\]

(11)

\[
V_r = \frac{M_b}{0.5L} 
\]

(12)

2.2.4 Other details

The steel sheet plates and the steel jackets should be in the elastic range, so the steel stress in the limit state should be verified not to exceed the yield strength.

![Equilibrium of sectional forces of the earthquake resilient coupled shear wall](image)

Figure 3. Equilibrium of sectional forces of the earthquake resilient coupled shear wall

2.3 Design example

2.3.1 The original coupled shear wall W1

A 29-story residential building with the height of 99.7m was selected for the new shear wall design. The structural system is a frame-shear wall with the typical plan layout shown in Figure 4. The designed PGA of basically occurring earthquakes with the mean return period of 475 years is 0.2g and the site soil belongs to Type II in China (MHUPRC 2010b). The seismic design group is classified as Group 1 in China (MHUPRC 2010b), which means the near shock earthquakes were selected for the seismic design. Each floor gravity dead load is 3.0kpa and the live load is 2.5kpa, while the representative value of gravity load for the seismic design is 1.0 times the dead load and 0.5 times the live load. Choose the outer wall W1 (shown in Figure 4) for the earthquake resilient coupled shear wall design. The design details of the conventional RC shear wall W1 is shown in Figure 5. The calculated compression force $N$ and moment $M$ of the wall pier are 3000kN and 237.5kN·m, respectively, and the calculated moment in the end of the coupling beam $M_b$ is 386.8kN·m.
The same material as the prototype structure were used for concrete and rebars, and \( \Phi 15.2 \) steel strand was chosen as the post-tensioned tendons for its high strength and low relaxation (MHUPRC 2010a). The mechanical properties of the material were shown in Table 1.

**Table 1. Material Properties.**

<table>
<thead>
<tr>
<th>Material</th>
<th>Strength grade</th>
<th>Location in structural part</th>
<th>Concrete compression strength ( f_c ) (Mpa)</th>
<th>Steel yield strength ( f_y ) (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>C60</td>
<td>Shear wall, concrete infilled steel base Longitudinal reinforcement in shear wall’s boundary element and coupling beam’s non-yield section, vertical distributed shear wall rebar Stirrup in shear wall’s boundary element and coupling beam’s non-yield section, horizontal distributed shear wall rebar Web of coupling beam’s replaceable device</td>
<td>27.6</td>
<td>-</td>
</tr>
<tr>
<td>Rebar</td>
<td>HRB400</td>
<td></td>
<td>-</td>
<td>360</td>
</tr>
<tr>
<td>Rebar</td>
<td>HRB300</td>
<td></td>
<td>-</td>
<td>270</td>
</tr>
<tr>
<td>Steel</td>
<td>BLY225</td>
<td></td>
<td>-</td>
<td>205</td>
</tr>
<tr>
<td>Steel</td>
<td>Q235</td>
<td></td>
<td>-</td>
<td>215</td>
</tr>
</tbody>
</table>
device, connection plate, steel jacket, steel plate, steel tube of concrete infilled steel base

| Steel strand | A$^3$15.2 | Post-tensioned tendon | - | 1320 |

2.3.2 The earthquake resilient coupled shear wall RW1

As stated above, the height of the non-yield connection beam $h_b$ and the span of the replaceable device $l_b$ were chosen as listed in Table 2 to make the stiffness of the replaceable coupling close to the original coupling beam (the stiffness is 0.93 times the original coupling beam). From Equation 1 and 2, the minimal height and width of the removed wall foot is 649mm and 675mm, respectively. From Equation 3 to Equation 11, the wall foot is designed. The replaceable coupling beam is designed following the principle of equivalent strength and similar stiffness as stated in previous studies (Chen and Lu 2015; Chen et al. 2016). Table 2 shows the detailed design of the earthquake resilient coupled shear wall RW1.

Table 2. Dimension of the removed RC areas.

<table>
<thead>
<tr>
<th>Design parameter</th>
<th>Calculated value</th>
<th>Adopted value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of the non-yield connection beam $h_b$</td>
<td>-</td>
<td>600</td>
</tr>
<tr>
<td>Span of the replaceable device $l_b$</td>
<td>-</td>
<td>200</td>
</tr>
<tr>
<td>Height of the removed wall foot $h_c$</td>
<td>$\geq$649</td>
<td>650</td>
</tr>
<tr>
<td>Width of the removed wall foot $l_c$</td>
<td>$\geq$675</td>
<td>675</td>
</tr>
</tbody>
</table>

2.3.3 The compared earthquake resilient coupled shear walls RW1-NSP and RW1-Slit

The wall RW1 described above are considered as the baseline case for the earthquake resilient coupled shear wall. However, the function of the steel plate should be investigated, and to eliminate the damage at the wall base, slit could be throughout the wall base. Therefore, the shear wall RW1-NSP without the 4mm steel plate embedded in the wall piers was created, and the wall RW1-Slit with the slit throughout the wall base (slit length of each wall pier is 1150mm) was proposed.

3. NUMERICAL SIMULATION

3.1 Numerical model

The finite element software ABAQUS was used to develop numerical simulation of the designed coupled shear walls. The steel rebars and the unbonded PT tendons were simulated by the space truss element T3D2. Linear reduced integration three-dimensional solid element C3D8R was used to simulate the shear wall concrete, the rubber base, the steel plates, the steel jackets, and the replaceable device. Prestress is added by lowering the temperature and apply an initial strain before the actual analysis. The top of the removed wall foot of the wall RW1, RW1-NSP and RW1-Slit are expected to open at the interface between the steel jacket and the steel sheet under strong earthquakes. Also, the wall base of the wall RW1-Slit would be open at the interface between the wall and the base beam. The opening/closure behavior is modeled by “Hard contact” in the normal direction, whose behavior is illustrated in Figure 7. No force exists when the opening happens and pressure is generated when closed. In the tangential direction, “Rough” behavior is defined, which means no sliding motion occurs in the interface.

The plasticity damage model based on continuum damage mechanics is adopted to describe the nonlinear behavior of the concrete. Tensile cracking and compressive crushing are considered and the
damage level is measured by tensile damage factor $d_t$ as well as compressive damage factor $d_c$. The damage factors can be expressed by Equation 13 and 14, where $\varepsilon_{pl}^t$ and $\varepsilon_{pl}^c$ are the plastic compression strain and the tension strain, respectively, $b_t = 0.9$, $b_c = 0.7$ (Hibbitt 2000). Steel and PTs are characterized by the bilinear metal plasticity model.

\[
d_t = 1 - \frac{\sigma_t E_t^{-1}}{\varepsilon_{pl}^t (1/b_t - 1) + \sigma_t E_t^{-1}}
\]  
\[
d_c = 1 - \frac{\sigma_c E_c^{-1}}{\varepsilon_{pl}^c (1/b_c - 1) + \sigma_c E_c^{-1}}
\]  

Figure 6. Design of the earthquake resilient coupled shear wall RW1
3.2 Pushover analysis

The original coupled shear wall was loaded by the axial forces firstly. After the initial step to add
prestress, axial forces were applied to the earthquake resilient coupled shear wall. With the applied
axial force, pushover analysis was carried out. Figure 8 demonstrates comparisons of the lateral force-
top displacement skeleton curves of the original coupled shear wall W1 and the earthquake resilient
coupled shear walls. It can be seen that the shear wall RW1 with strengthened steel plates embedded in
the wall piers demonstrates stronger capacity. The shear wall RW1-NSP without steel plates and the
wall RW1-Slit with slit throughout wall bottom achieve similar strength, which is because when the
wall pier was separated from the base beam, the steel plate embedded in it lose the function.
Compared to the original shear wall W1, the earthquake resilient coupled shear walls exhibits better
deformation capacities.

![Figure 8: Lateral force-top displacement skeleton curves](image)

Figure 9 shows the damage variable $d_i$ of the shear wall concrete, which reflects the concentration
locations of the plastic deformation. Compared to the original shear wall W1, the earthquake resilient
shear walls have lower damage index, which shows a relieved damage in the earthquake resilient shear
walls. A typical shear deformation and diagonal damage were found in the original shear wall W1.
The span-depth ratio of the coupling beam in W1 is small, so shear failure might occur and no obvious
plastic deformation was observed in it. In the shear wall RW1, plastic deformation concentrated in the
wall area upon the embedded steel plates, which indicates that the steel plates have strengthened the
wall bottom but the other parts still experience damage. Plastic concentration in wall RW1-NSP was
similar as the original wall W1, which demonstrated diagonal form but with reduced bottom parts. As
for the wall RW1-Slit, the damage variable was much lower, and the plastic distribution was more
uniform.

![Figure 9: Damage variable $d_i$](image)
Figure 9. Compressive damage variable $d_c$ (DAMAGEC) of the shear walls

Compressive equivalent plastic strain (PEEQ) reflects the accumulation of plastic in the shear walls. As shown in Figure 10, the plastic accumulated in the replaceable devices in all earthquake resilient shear walls. With steel plates embedded in the wall piers, flexural deformation was obvious, whereas slight plastic deformation was found in the reduced wall bottom corners without steel plates, which shows the strengthened function of the steel plates. Opening occurred between rubber foots and the wall piers in all earthquake resilient shear walls, and when the slit was through the wall bottom, opening occurred also in the wall bottom.
4. CONCLUSIONS

Based on the previous study, earthquake resilient coupled shear wall with replaceable coupling beam and non-damaged wall foot was designed. Following the design and simulation method proposed in this paper, an earthquake resilient coupled shear wall was designed. The original wall W1, the baseline earthquake resilient shear wall RW1, the earthquake resilient shear wall RW1-NSP without embedded steel plates and the wall RW1-Slit with slit through the wall bottom were theoretical analyzed. The results show that the damage of the wall concrete was relieved in the earthquake resilient shear walls, and if the slit throughout the wall bottom, wall damage was much lower. In accord with the design objective, plastic deformation was concentrated in the replaceable devices, and opening was occurred in the rubber foots in the earthquake resilient shear walls, as well as the wall bottom of the shear wall RW1-Slit.

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6. REFERENCES


