Research and tests of Steel-concrete-steel sandwich composite shear wall in reactor containment of HTR-PM

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Abstract: By quasi-static test of 8 specimens of steel-concrete-steel sandwich composite shear wall, the bearing capacity, hysteretic behavior, failure mode of the specimens was studied. So was the effect of the shear-span ratios, steel ratios and spacing of studs on the properties of the specimens. The failure patterns of all specimens with different shear-span ratios between 1.0 and 1.5 were compression-bending failure. The hysteretic curves of all specimens were relatively plump, which validated the well deformability and energy dissipation capacity of the specimens. When shear-span ratio less than 1.5, the shear property of the steel plate was well played, and so was the deformability of the specimens. The bigger the steel ratio was, the better the lateral resistance capacity and the deformability was. Among the spacing of studs in the test, the spacing of studs had no significant effect on the bearing capacity, deformability and ductility of the specimens. Based on the principle of superposition an advised formula for the compression-bending capacity of the shear wall was proposed, which fitted well with the test result and had a proper safety margin.

Keyword: steel-concrete-steel sandwich, composite shear wall, seismic behavior, ductility

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Introduction

Steel-concrete-steel sandwich composite shear wall is a new structure, its basic form is: pour the concrete between two steel plates, lay the studs in inside of steel plates, set up the tensile rebar, steel strips or partitions between two steel plates, make the steel plate and concrete into a whole to co-resist load, and no rebar in concrete.

This structure has the following significant advantages:

1. Convenient construction, construct the walls without any template and steel banding on site, and improve construction speed and quality.
2. Can achieve modular, factory plant.
3. High resistance capacity, good seismic behavior, can improve the safety of NI in HTR NPP.

In the field of civil construction at home and abroad, have conducted a series of in-depth research & tests on steel-concrete-steel shear wall, is mainly focused on embedded and one-sided steel plate composite shear wall. For example, the design
methods of such steel-concrete-steel shear wall have been stipulated in "Draft for approval of composite structure design specifications (JGJ138-2012)" newly revised by China. Have the similar research for double steel plate shear wall at abroad, including double steel plate composite shear wall inside set stiffening ribs and Bi-steel composite wall. In addition, china's Wenjun Zhu, etc., Jianguo Nie, etc., Jun Zhang, Jie Wu, etc., Fangfang Wei, etc. have also conducted in-depth tests and theoretical research, and proved that such structure has good resistance capacity and ductility energy-dissipation capacity.

According to the design experiences of HTR-PM, optimize the design of reactor containment. Using steel-concrete-steel composite structure module can simplify the construction design of reactor containment, reduce construction quality, and optimize the force characteristics of construction at the same time. However, up to now, only Japan has the corresponding specification of steel-concrete-steel structure in the world, but it cannot be used for the reactor containment structure by positive instructions. Compared with other types of reactor containment, the reactor containment of HTR has some differences, including no air tightness requirements, but its force features are substantially similar, such as it also loads some internal pressures and temperature effects, so the reactor containment of HTR is a force-complex construction member. It is necessary to expand the special research and develop the appropriate structural design regulations. Based on the design schemes of steel-concrete-steel composite shear wall of reactor containment in HTR-PM, this paper carried out a number of research & tests on steel-concrete-steel composite shear wall.

1 Overview of specimens

1.1 Design of specimens

Conduct the design of specimens, in the context of HTR preliminary design of the containment. Fig.1 shows the horizontal layout of reactor containment structure in HTR. Design the specimens according to the horizontal size of the straight section. The designed strength grade of concrete is C35, the material of steel plate is Q345, and the material of rebar is HRB335. The specimens have been welded and pouring at large structure laboratory of China Academy of Building Research.

Fig.1 Diagram of the horizontal layout of reactor containment structure in HTR

The form of specimen is monolithic composite shear wall of rectangular cross-section outside coated steel plates, the studs are laid in inside of steel plates, the tensile rebar are set up between two steel plates. The cross-section is 150mm×1000mm, shear-span ratio is set to 1.0 and 1.5. Other variable parameters of the specimen include: steel ratio of cross-section (2.7%, 4.0%, and 5.3%), corresponding thickness of steel plate (2mm, 3mm, and 4mm), spacing of studs in inside of steel plate (40mm, 60mm, and 80mm). The specific parameters of all specimens are shown in Tab.1.
<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Shear-span ratio</th>
<th>Steel ratio</th>
<th>Thickness of steel plate/mm</th>
<th>Spacing of studs/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCW1-1a</td>
<td>1.0</td>
<td>4.0%</td>
<td>3</td>
<td>40</td>
</tr>
<tr>
<td>SCW1-1b</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCW1-2a</td>
<td>1.5</td>
<td>4.0%</td>
<td>3</td>
<td>40b</td>
</tr>
<tr>
<td>SCW1-2b</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCW1-3</td>
<td>1.0</td>
<td>2.7%</td>
<td>2</td>
<td>40</td>
</tr>
<tr>
<td>SCW1-4</td>
<td>1.0</td>
<td>5.3%</td>
<td>4</td>
<td>40</td>
</tr>
<tr>
<td>SCW1-5</td>
<td>1.0</td>
<td>4.0%</td>
<td>3</td>
<td>60</td>
</tr>
<tr>
<td>SCW1-6</td>
<td>1.0</td>
<td>4.0%</td>
<td>3</td>
<td>80</td>
</tr>
</tbody>
</table>

All specimens were carried out the quasi-static tests of in-plane, and the design axial compression ratio of specimen is determined as 4.0 during the tests. The designed strength grade of concrete is C35, the material of steel plate is Q345, and the material of rebar is HRB335 (only for load beams and ground beams). The schematic diagram of a typical specimen is shown in Fig.2.

Measured cubic compressive strength of concrete is 42.9MPa, axial compressive strength is 28.7MPa, and elastic modulus is 33000MPa. Measured material properties of steel are shown in Tab.2.

<table>
<thead>
<tr>
<th>Thickness of steel plate/mm</th>
<th>Yield strength /MPa</th>
<th>Ultimate strength /MPa</th>
<th>Elastic modulus/MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>307</td>
<td>394</td>
<td>2.06×10^5</td>
</tr>
<tr>
<td>3</td>
<td>330</td>
<td>477</td>
<td>2.06×10^5</td>
</tr>
<tr>
<td>4</td>
<td>361</td>
<td>463</td>
<td>2.06×10^5</td>
</tr>
</tbody>
</table>

1.2 Loading device of tests

The quasi-static test methods are applied to exert the low-cycle reciprocating horizontal load at the top of the shear wall model, vertical load is kept constant during the tests. Load-deformation hybrid control method is used as the loading system. Load-control grades-loading is used before the specimen is yield, and deformation-control grades-loading is used after the estimated yield load is closing. Each grade of the load should loop twice, and try to ensure that the load can be loaded to more than 85% of peak load. The loading device is shown in Fig.3.
2. Test results

2.1 Failure features and shapes of specimens

Failure process and shape can basically be divided into three stages:

(1) Elastic stage. From the beginning of loading to the lateral strain of steel plate reaches 2000, the force of concrete is basically in elastic region, the relational expression of horizontal load and apical displacement is linear because of the small horizontal load. The appearance of the specimen is basically non-destructive, and has no significant damage.

(2) Flexion stage. With the horizontal deformation gradually increases, the compression-side concrete at the foot of specimen will be softened. Meanwhile, the steel plate will be yielded, bulged and bended, but the frontal steel plate has no significant phenomenon.

(3) Failure stage. With continuing the loading, the steel plate welds of other specimens except SCW1-2b and SCW1-6 have been torn in the corner of compression-side. The phenomenon of bulged and bended will be spread to the corners of the front steel plate. Horizontal bearing capacity will be reduced, the steel plate of tension-side will be pulled out, some tensile rebar will be broken and accompanied by sound, but above the bottom section of the specimen is basically in a non-destructive state. Ultimate bearing capacity of the specimen will be reduced to 85% or less. The failure modes are typical bending damage. The failure mode of specimen SCW1-3 of 2mm thick steel plate is different from other specimens. Full cross-section compression-bending of steel plate is appeared at the bottom of the whole specimen, but there is no significant bulge in the flank.

Final damage and detailed damage of typical specimen is shown in Fig.4.
2.2 Hysteretic curves of horizontal force and apical displacement

Hysteretic curves of horizontal force and apical displacement of the specimens under the reciprocating horizontal load is shown in Fig.5. In the early of loading, hysteretic curves of the specimens are the straight line, that means no residual deformation and the force is in elastic region. With continuing the loading, the concrete will crack, the stiffness of specimen will reduce, and the residual deformation after unloading will gradually increase. After reaching the peak load, the bearing capacity of specimen will reduce slowly. Hysteretic curves of the specimens are both relatively plump, that means steel-concrete-steel composite shear wall has relatively strong energy dissipation capacity. There is a certain folded phenomenon in hysteretic curve of specimen SCW1-3 that appears as "opposite S type", hysteretic curve of other specimens basically appears as "shuttle type".

2.3 Energy dissipation capacity of specimens

The equivalent viscous damped coefficient is used to analyze the energy dissipation capacity of specimens. The equivalent viscous damped coefficient and hysteretic loop area of each specimen in peak displacement (corresponding to the maximum horizontal bearing capacity) is shown in Tab.3. The table shows that:

(1) Hysteretic loop area of specimen SCW1-3 is significantly smaller than other specimens, that means its energy dissipation capacity is poor.

(2) The bigger shear-span ratio is, the sooner corner steel plate will be yield. So the smaller hysteretic loop area is, the worse absolute energy dissipation capacity is.

(3) The equivalent viscous damped coefficient of all specimens are greater than 0.13, that means the viscous energy dissipation capacity is strong.
Tab.3  Equivalent viscous damped coefficient of specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Hysteretic loop area/kN.mm</th>
<th>Equivalent viscous damped coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCW1-1a</td>
<td>15551</td>
<td>0.175</td>
</tr>
<tr>
<td>SCW1-1b</td>
<td>13722</td>
<td>0.172</td>
</tr>
<tr>
<td>SCW1-2a</td>
<td>10962</td>
<td>0.137</td>
</tr>
<tr>
<td>SCW1-2b</td>
<td>8689</td>
<td>0.128</td>
</tr>
<tr>
<td>SCW1-3</td>
<td>5941</td>
<td>0.167</td>
</tr>
<tr>
<td>SCW1-4</td>
<td>15181</td>
<td>0.156</td>
</tr>
<tr>
<td>SCW1-5</td>
<td>15379</td>
<td>0.157</td>
</tr>
<tr>
<td>SCW1-6</td>
<td>18310</td>
<td>0.180</td>
</tr>
</tbody>
</table>

2.4  Maximum horizontal bearing capacity, ultimate displacement angle and displacement ductility coefficient

Skeleton curves of horizontal load and apical displacement of specimens reflects different stages and its characteristics of force and deformation of the specimens. The skeleton curve of specimens is shown in Fig.5. Nominal yield point is obtained from skeleton curve by the method of energy equivalent area, ultimate displacement is taken to the displacement corresponding to 85% of ultimate bearing capacity, and thus displacement ductility coefficient is got.

Maximum horizontal bearing capacity, yield displacement, ultimate displacement angle and displacement ductility coefficient is shown in Tab.4. Skeleton curve shows that:

1. Stiffness of each specimen basically changes significantly when the horizontal load is up to 60% of ultimate bearing capacity.

2. Ductility coefficient of all specimens is bigger than 2.35, ultimate displacement angle is greater than 1/100, and the drop-segment of skeleton curve is relatively gentle. It means steel-concrete-steel composite shear wall has good deformability.

Tab.4  Maximum horizontal bearing capacity, ultimate displacement angle and displacement ductility coefficient of specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Direction</th>
<th>Load/kN</th>
<th>Apical displacement/mm</th>
<th>Apical displacement angle</th>
<th>Ductility coefficient</th>
<th>Load value Pmax/kN</th>
<th>Average value µ</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Yield</td>
<td>Peak</td>
<td>Yield</td>
<td>Peak</td>
<td>Yield</td>
<td>Peak</td>
</tr>
<tr>
<td>SCW1-1a</td>
<td>Push</td>
<td>1536</td>
<td>1821</td>
<td>3.43</td>
<td>10.40</td>
<td>1/292</td>
<td>1/96</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>-1468</td>
<td>-1745</td>
<td>-3.61</td>
<td>-9.67</td>
<td>1/277</td>
<td>1/103</td>
</tr>
<tr>
<td>SCW1-1b</td>
<td>Push</td>
<td>1371</td>
<td>1614</td>
<td>3.45</td>
<td>11.81</td>
<td>1/289</td>
<td>1/85</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>-1346</td>
<td>-1610</td>
<td>-3.92</td>
<td>-10.78</td>
<td>1/255</td>
<td>1/93</td>
</tr>
</tbody>
</table>
2.5 Impact analysis of each parameter for the capability of the wall

(1) The impact of concrete strength

The comparison of skeleton curves in different grades of concrete is shown in Fig.7. It shows that: with the improvement of concrete strength, the bearing capacity of specimen will be increased, the drop-segment of skeleton curve will even steeper, and the ductility will become poor. Therefore, the concrete strength is advised as no more than C40 in actually application.

![Fig.7](image)

(2) The impact of steel ratio

The steel ratio is 2.7%, 4.0% and 5.3% in the tests.

The comparison of hysteretic curve shows that: hysteretic curve of specimen which steel ratio is 4.0% and 5.3% basically appears as "shuttle type", hysteretic curve of specimen which steel ratio is 2.7% basically appears as "bow type" and has a certain folded phenomenon. It means that because of small steel ratio and consequent premature buckling of steel plate, the hysteretic behavior and energy dissipation capacity of specimen SCW1-3 is poor.

The comparison of the test data shows that: the bigger the steel ratio is, the higher the anti-side bearing capacity is, the bigger the yield displacement angle is, and the deformability is relatively strong. Conversely, the lower the anti-side bearing capacity is, the smaller the yield displacement angle is, and the deformability is relatively weak.

The comparison of skeleton curve of steel-concrete-steel composite shear wall under different steel ratios is shown in Fig.8. The sensitivity analysis of the steel ratio shows that: the bigger the steel ratio is, the higher the bearing capacity of wall is. The effect of steel ratio on the steep extent of drop segment is very small, that means the ductility of wall will not be impacted by steel ratio.
3) The impact of spacing of studs

The spacing of studs in the inside of steel plate is 40mm, 60mm and 80mm in the tests. The comparison of hysteretic curve shows that the hysteretic loops are both "long-shuttle type" within the above ranges of the spacing of studs. The comparison of the test data also shows that the impact of the spacing of studs for bearing capacity and yield displacement angle of specimens are not significant. The comparison of skeleton curve of specimens under different spacing-thickness ratios is shown in Fig.9. The sensitivity analysis of the spacing of studs shows that: skeleton curve corresponding to different spacing is basically coincident, that means the impact of the spacing of studs is very small.
3 Bearing capacity calculation

Fig.10 is the diagram of bottom strain distribution of a typical specimen in loading process. It shows that the results are basically fit with the flat-section hypothesis, when the strain of steel plate does not exceed the yield strain. If the horizontal load becomes larger, especially the steel plate has been buckling, the flat-section hypothesis cannot been met in the bottom section, because the strain measured by the measuring pieces can not reflect the actual deformation due to two-sides of steel plate has been bulged and bended.

![Fig.10 Strain distribution of a typical specimen in loading process](image)

The formula of bearing capacity is based on the observed bends of steel plate flank in the tests, considering the difference between the force of thin steel plate and profile steel, introducing the reduction coefficient of stability, and reflecting the impact of the bend on the edge of steel plate.

Bearing capacity calculation formula of the containment walls on the side of SG is:

\[
N \leq \left( a_1 f_{\xi} \xi b_{h_{w0}} + f_a A_a - \sigma_a A_a + N_{wp} \right) / \gamma_{RE}
\]

\[
N_e \leq \left( a_1 f_{\xi} \xi (1 - 0.5 \xi) b_{h_{w0}}^2 + \eta f_a A_a (h_{w0} - a_a) + M_{wp} \right) / \gamma_{RE} A_p
\]

\[
M_{wp} \leq \left[ 0.5 - \left( \frac{\xi - \beta_1}{\beta_1 \omega_p} \right)^2 \right] f_a A_p h_{wp}
\]

\[
e = e_0 + \frac{h_w - a_a}{2}
\]

\[
e_0 = \frac{M}{N}
\]

\[
\omega_p = \frac{h_{wp}}{h_{w0}}
\]

\[
h_{w0} = h_w - a_a
\]

The steel stress of tensile-side or relatively small compression-side can be calculated according to the following conditions:

While \( \xi \leq \frac{\xi}{\xi_h} h_0 \), it is large-eccentric compression member, take \( \sigma_a = f_a \)

While \( \xi > \frac{\xi}{\xi_h} h_0 \), it is small-eccentric compression member, \( \sigma_a \) can be calculated by the following formula:

\[
\sigma_a = \frac{f_a}{\xi - \beta_1} (\xi - \beta_1)
\]

In the formula:

\( A_a \), \( A_a \) —— the area of steel plate at the end portion of compression or tensile side, can be included in the partition of the end portion

\( a_a \), \( a_a \) —— the distance between total force point and the side edge of steel plate at the end portion of compression or tensile side

\( \alpha_1 \), \( \beta_1 \), \( \xi_b \) —— take the value according to China's "Concrete Structures Design Specification" (GB50010)

\( h_{w1} \), \( h_{w} \) —— the height of steel plate included in the end portion of compression or tensile side

\( A_{p} \) —— the total area of central steel plate except the steel plate at the end portion of compression or tensile side
The design value of compressive or tensile strength of steel plate is denoted by $f_a$. $f_u$.

The bearing axial force of central steel plate except the steel plate at the end portion of compression or tensile side, while $\xi > 0.8$, is given by $N_{wp} = f_a A_p$.

$M_{wp}$ is the moment produced by the total force and the point of tensile-side force of central steel plate except the steel plate at the end portion of compression or tensile side. While $\xi > 0.8$, take

$$M_{wp} = 0.5 f_a A_p h_{wp}$$

$
\omega_p$ is the configuration coefficient of central steel plate except the steel plate at the end portion of compression or tensile side.

The thickness of shear wall is denoted by $b_w$, the thickness of concrete in shear wall is $b_c$, and the height of section of shear wall is $h_w$. The distance between axial force point and tensile-side force point of shear wall is $e$. The eccentric moment produced by axial force and gravity-center of the section is $e_0$.

$M$ is the design value of moment of the end of shear wall. $N$ is the design value of axial pressure of shear wall.

The reduction coefficient of stability, denoted by $\eta$, can be taken as

$$\eta = \frac{1}{\lambda}$$

$\lambda$ is the calculated shear span ratio. The value is taken as the ratio of design value of the larger moment at the upper and lower end (M), corresponding design value of shear force (V) and the height of section ($h_w$), $M / V h_w$; while $\lambda < 1$, take 1, while $\lambda > 3$ [9], take 3.

Bearing capacity calculation formula of the containment walls on the side of RPV is:

(1) Use the following formula to check the strength of steel plate on both sides of concrete:

$$\sigma_{red} \leq f_a$$

$$\sigma_{red} = \sqrt{\sigma_{ax}^2 + \sigma_{ay}^2 - \sigma_{ax} \sigma_{ay} + 3 \tau_{xy}^2}$$

$$\tau_{xy} = \frac{\tau_{xy} A}{A_a}$$

While $\sigma_x$ is tensile stress, take $\sigma_{ax} = \frac{\sigma_x A}{A_a}$,

while $\sigma_x$ is compressive stress, take $\sigma_{ax} = \frac{E_a}{E}$,

while $\sigma_y$ is tensile stress, take $\sigma_{ay} = \frac{\sigma_y A}{A_a}$,

while $\sigma_y$ is compressive stress, take $\sigma_{ay} = \frac{E_a}{E}$

$$E = \frac{E_a A_u + E_c A_c}{A}$$

In the formula:

$\sigma_x$ — design value of X-direction tensile stress in the plane of the wall

$\sigma_y$ — design value of Y-direction tensile stress in the plane of the wall

$\tau_{xy}$ — design value of shear stress in the plane of the wall

$\sigma_{red}$ — conversion stress

$f_a$ — design value of tensile strength of steel plate

$A_u$ — cross-sectional area per length of two-sides steel plate

$A_c$ — cross-sectional area of concrete per length of steel-concrete-steel sandwich composite shear wall

$A$ — cross-sectional area per length of steel-concrete-steel sandwich composite shear wall

The combined force of the design value of compressive stress is checked according to multi-axial compressive strength of concrete in the following
formulas, and need to comply with the requirements of Chapter VII of this guide.

\[
\left| \sigma_{ci} \right| \leq f_{ci} (i = 1,2)
\]

\[
\sigma_{ci} = \frac{\sigma A}{A_c}
\]

In the formula:
- \( f_{ci} \) —multi-axial compressive strength of concrete, determine the value according to China’s "Concrete Structures Design Specification" (GB50010)
- \( \sigma_{ci} \) —the design value of principal compressive stress of the component according to elastic or elastic-plastic analysis methods in the structure

Tab.5 shows the comparison of test values and calculations of maximum horizontal bearing capacity of specimens (calculations are calculated by the recommended formula, the measured intensity values are used as material strength). The results show that the recommended formula is good fit with the measured values and has a certain safety margin (19%-37%) except the specimen SCW1-3. These results also show that avoid using too thin steel plate in actual application to avoid the damage of full cross-section compression-bending. It is recommended the thickness of steel plate is not less than 1/50 of the thick of shear wall.

<table>
<thead>
<tr>
<th>No.</th>
<th>Test value Pt/kN</th>
<th>Calculations Pc/kN</th>
<th>Relative difference /%</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCW1-1a</td>
<td>1783</td>
<td>1322</td>
<td>35</td>
</tr>
<tr>
<td>SCW1-1b</td>
<td>1612</td>
<td>1322</td>
<td>22</td>
</tr>
<tr>
<td>SCW1-2a</td>
<td>1031</td>
<td>750</td>
<td>37</td>
</tr>
<tr>
<td>SCW1-2b</td>
<td>950</td>
<td>750</td>
<td>27</td>
</tr>
<tr>
<td>SCW1-3</td>
<td>961</td>
<td>1033</td>
<td>-7</td>
</tr>
<tr>
<td>SCW1-4</td>
<td>1973</td>
<td>1596</td>
<td>24</td>
</tr>
<tr>
<td>SCW1-5</td>
<td>1569</td>
<td>1322</td>
<td>19</td>
</tr>
<tr>
<td>SCW1-6</td>
<td>1660</td>
<td>1322</td>
<td>26</td>
</tr>
</tbody>
</table>

4 Conclusions

Through a series of experimental phenomena and results analysis, the main conclusions are as follows:

(1) The failure modes of all specimens are compression-bending failure, skeleton curves are relatively plump, the drop-segment of skeleton curve are gentle, and ultimate displacement angles are basically more than 1/100. It means the specimens of steel-concrete-steel composite shear wall both have good deformability and energy dissipation capacity.

(2) The smaller the shear-span ratio of shear wall specimens is, the better the shear-bearing capacity of steel plate is, the bigger the anti-side bearing capacity is, and the better the ductility is. The bigger the steel ratio of specimen section is, the higher the anti-side bearing capacity is, the bigger the yield displacement angle is, and the deformability is relatively stronger.

(3) The impact of the spacing of studs for bearing capacity, deformation and ductility are not significant within the adopted ranges in the tests.

(4) The bending capacity calculation formula of Steel-concrete-steel composite shear wall proposed by this paper is good fit with the experimental results.

References


