16th World Conference on Earthquake Engineering, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 2417
Registration Code: S-N1464640635

DESIGN PROCEDURE FOR SEISMIC RETROFIT USING STUD-TYPE DAMPERS CONSIDERING THE STRENGTH AND STIFFNESS OF SURROUNDING FRAMES

M. Ito[1], K. Hayashi[2], Y. Taniguchi[3], M. Kurata[4], M. Nakashima[5]

[1] *Assistant Professor, Organization of Advanced Science and Technology, Kobe University, mai.ito@gold.kobe-u.ac.jp*
[2] *Assistant Professor, Graduate School of Engineering, Toyohashi University of Technology, hayashi.kazuhiro@forest.ocn.ne.jp*
[3] *Nippon Steel & Sumitomo Metal Co., Ltd, taniguchi.ag4.yuudai@jp.nssmc.com*
[4] *Associate Professor, Disaster Prevention Research Institute, Kyoto University, kurata.masahiro.5c@kyoto-u.ac.jp*
[5] *Professor, Disaster Prevention Research Institute, Kyoto University, nakashima@archi.kyoto-u.ac.jp*

Abstract

Stud-type dampers that are installed in a limited length of beam span are useful for building structures with many openings, but its performance is affected significantly by the deformation of surrounding frames. This study proposes a seismic retrofit design procedure for existing steel frames using stud-type dampers in consideration of the interaction between the damper and surrounding frame. As a stud-type damper, slitted steel shear wall stiffened by plywood panels is used. It is thinner than a shear wall with conventional stiffeners, light enough to carry by hand, easy to construct, and highly cost efficient. Furthermore, the strength and stiffness of the shear wall can be controlled independently by the adjustment of slit arrangement, namely spacing and length.

The procedure of proposed seismic retrofit design is as follows:

(a) The damper size is determined based on the location of openings and installation position applicable for a damper in the existing frame.
(b) The maximum strength of the damper is derived to prevent shear yielding, bending yielding or lateral buckling of existing steel beam connected to the damper. When the damper width increases, the upper limit of damper strength also increases.
(c) The number and layout of the dampers are determined in consideration of the desired base shear coefficient of the frame retrofitted with dampers.
(d) The required stiffness of the damper is estimated to obtain sufficient energy dissipation. When the damper strength is fixed, the equivalent viscous damping ratio increases with the damper stiffness and then reaches a constant value. The stiffness that has 0.8 times the maximum achievable damping ratio is chosen as the minimum value.
(e) The steel plate thickness and slit arrangement of the damper are determined considering the above design conditions.
(f) If a feasible slitted steel plate is not obtained, the damper size, layout or strength should be re-examined.

Cyclic loading test is conducted to examine the validity of proposed design procedure and the performance of retrofit strategy. The specimen was a single-story, single-bay steel frame in which a slitted steel shear wall was installed as a stud-type damper in the middle of beam span, and with a scale ratio of 0.6. The damper yielded at a shear angle of 0.24 %, then the beam ends yielded near the beam-to-column connection at 1.0 %. The test shows that the design procedure is proven effective, because the beam did not yield or buckle at sections close to the damper. The damper had stable hysteresis behavior until the shear angle of 9 % (equivalent to the story drift angle of 3%) and sufficient energy dissipation with the equivalent viscous damping ratio of approximately 0.18.

Keywords: Stud-type shear wall; Seismic retrofit; Design method; Steel structure

1. Introduction

As large-scale earthquakes are serious concern for the near future, it has become critical to advance and spread seismic retrofitting technology. There are ways of installing dampers that can absorb earthquake energy in existing buildings by seismic retrofitting. Among them, a stud-type damper that has considerable flexibility in its installation position is suitable for existing buildings that have several openings. A brace-type damper that is the most common earthquake-resistant device is diagonally installed across an entire span of buildings. Therefore, mutual effects between the brace-type damper and attached frame are small and the design of the damper is relatively simple. In contrast, a stud-type damper is installed in a limited area of beam spans; thus, the performance of the damper is affected by the strength and rigidity of the existing frame. If a stud-type damper with excessive strength is installed on an existing frame, the surrounding beam yields; consequently, the damper rotates or deforms in the out-of-plane direction. Therefore, the shear deformation and energy dissipation of the damper might decrease. In recent papers regarding stud-type dampers, the elastic stiffness of the entire system in consideration of the deformation effect of the surrounding frame[1], dynamic behavior of the frame with stud-type viscoelastic damper[2], and its simplified estimation[3] have been studied. However, a simple and useful design method for stud-type dampers that takes into consideration the mutual effects between the existing frame and damper has not been developed.

In this study, the design procedure for a stud-type damper for seismic retrofitting was derived in consideration of the strength and stiffness of the existing frame. A slitted steel shear wall, which has been studied by the authors[4, 5], was applied as a stud-type damper. The steel shear wall is sandwiched between two wood panels to suppress steel plate buckling and to increase energy dissipation. It is significantly thinner than a steel damper having conventional stiffeners, light enough to be portable, easy to construct, and highly cost efficient. In addition, the strength and stiffness of the damper can be controlled fairly independently of one another by varying the slit spacing and length; therefore, a damper can be optimally designed for the seismic capacity requirements of an existing building.

In this study, the basic performance of a slitted steel shear wall is described first. Then, the design conditions are derived from the interaction between a stud-type damper and connecting beam and required energy dissipation for the shear wall. The design procedure for installing the damper in the middle of the span of an existing steel frame is also investigated. Moreover, a series of shear loading tests are conducted for a retrofitted one-story one-span steel frame to confirm the validity of the design procedure.

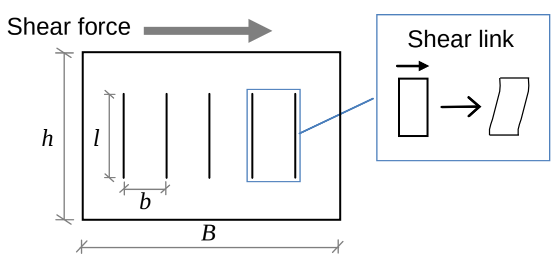


Fig. 1 – Slitted steel plate

2. Basic Performance of Slitted Steel Shear Wall

The steel plate segments between the slits behave as a series of flexural links, which dissipate the energy by yielding at both ends of the shear links, which have a large flexural deformation. The in-plane strength (design strength) and initial stiffness of the slitted steel plate are calculated as follows[6]:

$$Q_{wt} = \frac{ntb^2}{2l}\sigma_y \tag{1}$$

$$K_{wt} = \frac{1}{\frac{\kappa(h-ml)}{GBt} + \frac{\kappa l}{Gbt}\cdot\frac{m}{n} + \frac{l^3}{Etb^3}\cdot\frac{m}{n}} \quad (2)$$

where E is Young's modulus, G is the shear modulus of the steel plate, κ is the shear deformation shape factor for a rectangular section (=1.2), σ_y is the yield stress of the steel plate, t is its thickness, l is the length of the shear link, b is its width, B is the width of the steel plate, m is the number of row for links (=1), and n is the number of links (n =6). The in-plane (design) strength is the maximum strength assuming a non-buckling steel plate. It is obtained from the total shear force to form plastic hinges at the ends of the shear links. The initial stiffness is the sum of the elastic bending deformation of the shear links and shear deformation of the wall part divided by the horizontal force P. The strength and stiffness of the damper can be controlled fairly independently of one another by varying the slit spacing and length. However, if lateral buckling occurs in a steel plate subjected to shear deformation, the strength and stiffness decrease. In a reference paper[4], methods for suppressing the degradation of strength and dissipated energy were proposed by sandwiching the slitted steel plate between two plywood panels, and the effects of stiffening and hysteretic performance of the shear wall were confirmed. In this paper, the slitted steel plate restrained by wood panels was assumed to be used as a stud-type damper.

3. Design for Shear Wall

3.1 Modeling of shear wall

In mid- or low-rise steel buildings, there are many window or door openings and nonstructural walls between them in the longitudinal direction. Slitted steel shear walls are considered to be installed in the nonstructural walls and window backs next to windows are regarded as rigid members. The target model is a frame with a shear wall and rigid part at the center of a span, as shown in Fig. 2. In this figure, α is the ratio of shear wall width B and span length L and β is the ratio of shear wall height h and story height H.

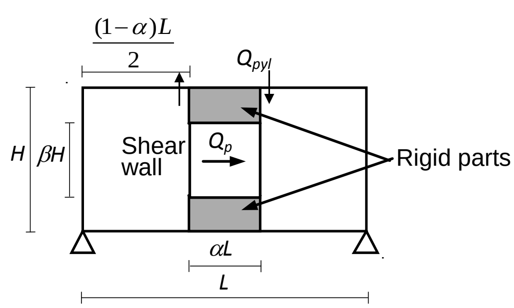


Fig. 2 – Modeling of stud-type shear wall

3.2 Derivation of yielding angle

The story drift for the yielding of a beam at the connection between a column and beam is shown as follows, where the shear wall yields before the beam:

$$\theta_{fy} = \frac{1}{6}M_{bp}\left(\frac{L}{EI_b} + \frac{H}{EI_c}\right) - \frac{Q_{wt}HL(1-\alpha^2)}{48EI_b} \quad (3)$$

where M_{bp} is the full plastic moment of the beam, EI_b is its bending stiffness, and EI_c is the bending stiffness of the column. The first term on the right side of Eq. (3) represents the story drift for yielding at the end of the beam when the frame is subjected to shear force, and the second term represents the angle of the beam being turned back by the vertical force that is transferred from the shear wall.

On the other hand, the story drift of the entire structure at the time of yielding of the shear wall is calculated by the principle of virtual work for the rigid connection model. However, as this is obtained based on the idealized one-story one-span model, it is not used for the generalized model, which is multi-story and multi-span. The simple model shown in Fig. 3, which simulates a rigid connection model, is proposed in this study to more easily estimate the story drift of yielding of the shear wall. The left frame in this model represents the hysteresis behavior of the shear wall, the right frame represents that of the frame, and the entire frame represents that of the overall structure with the shear wall. The left frame is a model that has pin connections between the columns and beams. It has a shear wall at the middle of a span. The effect of the rigid connection on the shear wall is simulated by increasing the apparent stiffness of the connecting beam. However, as the left frame itself does not have shear strength, the influence of the frame is added by connecting the rigid frame with rigid part on the right. As shown in Fig. 4, the ratio of increasing stiffness for the beam is defined to obtain the same deflection angles at the left end of the beam, which is subjected to bending moment M_b in (a) the rigid connection model and (b) the pin connection model with increasing stiffness of the beam. The ratio of increasing stiffness for the beam, and the story drift angle at the time of yielding of the shear wall are calculated as follows:

$$\varphi = 1 + \frac{I_c}{I_b}\frac{L}{H}(1-\alpha^3) \tag{4}$$

$$\theta_{fpy} = \frac{Q_{wt}HL}{24EI_b\varphi}(1-\alpha)^3 + \beta\frac{Q_{wt}}{K_{wt}l} \tag{5}$$

The first term on the right side of Eq. (5) represents the story drift angle of the frame based on the vertical force transferred from the shear wall to the beam, and the second term represents the story drift angle based on the shear deformation of the wall. When the first term increases, the shear deformation of the shear wall decreases, and the dissipation energy degrades because the rotation angle of the beam increases due to the vertical force transferred from the shear wall. For Eq. (5), only the interaction between the beam and shear wall should be considered; therefore, it can be used for a multi-story multi-span frame.

When both the shear wall and frame are elastic, the story drift angle is obtained by replacing the yielding strength (Q_{wt}) by the shear force of the wall (Q_p).

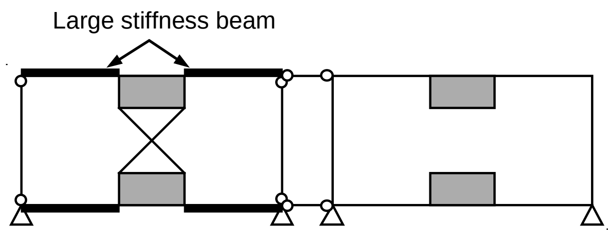
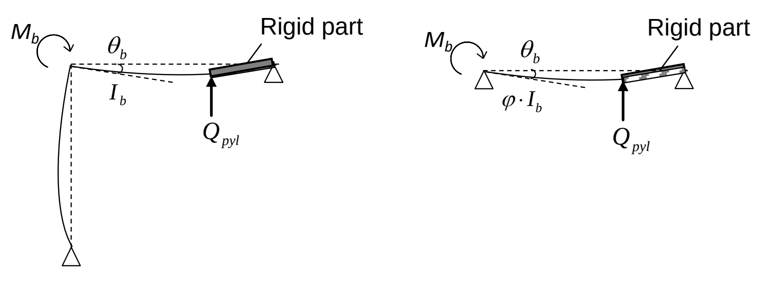


Fig. 3 – Simple model



(a) Rigid connection model (b) Pin connection model

Fig. 4 – Simulation model

3.3 Design conditions of strength for the shear wall

If the shear wall is subjected to shear deformation, vertical force is transferred to the beam from the shear wall, as shown in Fig. 2. In this section, the conditions of not yielding or lateral buckling of the beam at the connection ends of the shear wall are considered. The vertical force acting on the beam at the time of yielding of the shear wall (Q_{pyl}) is shown by Eq. (6), using the yielding shear strength of the wall (Q_{wt}).

$$Q_{pyl} = \frac{Q_{wt}H}{2\alpha L} \tag{6}$$

The upper limit on the strength of the shear wall in order for the adjacent beam not to yield from the bending moment and shear force acting due to this vertical force is calculated below. The strength obtained from yielding due to the bending moment is shown in Eq. (7), and that obtained from yielding due to shear force is shown in Eq. (8). The safety factor is set to 0.8.

$$Q_p = (Q_{p1} + Q_{p2}) \le 0.8 \cdot \frac{1+\alpha}{1-\alpha} \cdot \frac{4M_{bp}}{H} \tag{7}$$

$$Q_p = (Q_{p1} + Q_{p2}) \le 0.8 \cdot \frac{2\alpha L}{\sqrt{3}H} \cdot \sigma_{by} A_w \tag{8}$$

In Eqs. (7) and (8), Q_{p1} is the shear strength of the shear wall in the lower story, Q_{p2} is the shear strength of the wall in the upper story, σ_{by} is the yielding stress of the beam, and A_w is the cross-sectional area of the beam web. If the width of the shear wall increases compared with the span length to get larger α, the limit on the strength of the shear wall is relaxed.

Next, the conditions for suppressing lateral buckling of the beam are derived. The elastic lateral buckling strength is calculated as[7]

$$M_{cre} = C_1 \frac{\pi^2 EI_y}{(k_u L_b)^2} \left[C_2 g + \sqrt{(C_2 g)^2 + \frac{I_w}{I_y} \left\{ \left(\frac{k_u}{k_\beta} \right)^2 + \frac{GJ(k_u L_b)^2}{\pi^2 EI_w} \right\}} \right] \tag{9}$$

where L_b is the beam length, EI_y is the bending stiffness of the beam in the weak axis direction, EI_w is the bending torsion rigidity of the beam, GJ is the Saint-Venant torsional rigidity of the beam, g is the distance between the acting point and shear center subjected to a lateral force, k_u, k_β is the coefficient of buckling length for evaluation of the boundary condition, C_1 is the effect of the moment distribution, and C_2 is a correction factor of g associated with the lateral force. In Eq. (9), L_b represents the distance between the beam end and the center of the shear wall. The beam is simply supported in the horizontal and vertical directions to consider safe side, and k_u and k_β are set to 1.0. Although the point that is acted upon by the vertical (lateral) force from the shear wall depends on the width of the shear wall, it is assumed that the concentrated load acts on the center of the simpley supported beam; C_1 is set to 1.4, and C_2 is 0.55.

The condition on the strength of the shear wall is obtained from Eq. (10) in order not to buckle the beam adjacent the the shear wall. The safety factor is 0.8.

$$Q_p = (Q_{p1} + Q_{p2}) \le 0.8 \cdot \frac{1}{(1-\alpha)} \cdot \frac{4(\alpha M_{bp} + M_{cre})}{H} \tag{10}$$

Considering these conditions, a design that can concentrate shear deformation on the shear wall will be feasible by installing a shear wall with strength satisfying Eqs. (7), (8), and (10) in order to suppress yielding and lateral buckling of the connecting beam at both ends of the shear wall.

3.4 Design condition of stiffness for shear wall

In this section, it is assumed that the hysteresis performance of an existing frame and shear wall are perfectly elasto-plastic, and that of the frame with the shear wall is their sum. The following equation is obtained from Fig. 5[8]:

$$\gamma Q_f = Q_{wt} = K_{wt}\theta_{fpy} \tag{11}$$

$$Q_f = K_f\theta_{fy} \tag{12}$$

$$\theta_{fpy} < \theta_{fy}$$

$$Q_f = 4M_{bp} / H$$

where Q_f is the frame strength, γ is the strength ratio between the shear wall and frame, and K_f is the elastic stiffness of the frame. Once a target frame is determined, the strength Q_f and stiffness K_f of the frame can be calculated, and the strength and stiffness of the shear wall are variables. The equivalent viscous damping ratio (h_{eq}) is shown by

$$h_{eq} = \frac{E_w}{4\pi E_e} \tag{13}$$

where E_w is the area of one cycle for the hysteresis loop of the frame and shear wall, and E_e is the crosshatched area of Fig. 6.

$$E_w = 4\left[(\mu_f\theta_{fy} - \theta_{fpy})\gamma + (\mu_f - 1)\theta_{fy}\right]Q_f \tag{14}$$

$$E_e = \frac{1}{2}\mu_p\theta_{fpy}(1+\gamma)Q_f = \frac{1}{2}\mu_f\theta_{fy}(1+\gamma)Q_f \tag{15}$$

μ_f is the ductility factor of the frame, and μ_p is the ductility factor of the shear wall. As μ_f is assumed to be 1, the equivalent viscous damping ratio (h_{eq}) is obtained from Eqs. (11) to (15) as follows:

$$h_{eq} = \frac{2\gamma}{\pi(1+\gamma)} \cdot \left(-\frac{K_f}{K_{wt}}\gamma + 1\right) \tag{16}$$

Fig. 7 shows the relationship between the stiffness ratio of the shear wall and frame, strength ratio, and equivalent viscous damping ratio. The damping ratio increases and then remains constant with increasing stiffness of the shear wall to the frame. The value of the convergent damping ratio increases with increasing strength of the shear wall and reaches the maximum value of the equivalent viscous damping ratio (h_{eqmax}) when the stiffness of the shear wall is set to infinity.

$$h_{eq\max} = \frac{2\gamma}{\pi(1+\gamma)} \tag{17}$$

The stiffness of the shear wall is obtained from Eq. (18) in order to achieve more than η times the maximum damping ratio (h_{eqmax}), where η is the damping arrival factor.

$$K_p \geq \frac{\gamma}{1-\eta}K_f \tag{18}$$

$$\gamma < 1.0$$

The strength of the shear wall has already been determined by the condition shown in Section 3.3; therefore, the strength ratio γ is known. If the strength ratio is constant, the energy dissipation of the shear wall becomes large as the damping arrival factor increases. In this study, the minimum damping arrival factor is set to 0.8.

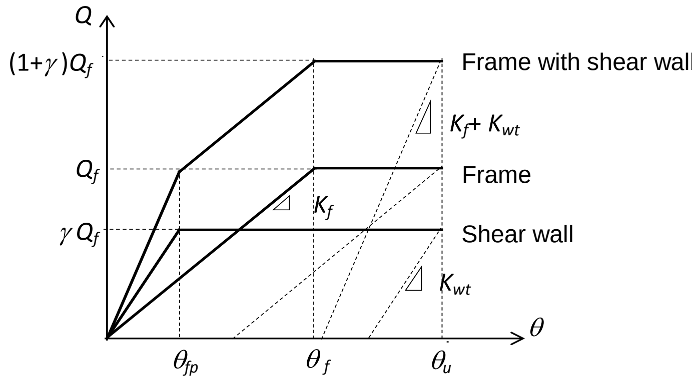


Fig. 5 – Hysteresis behavior of frame with shear wall

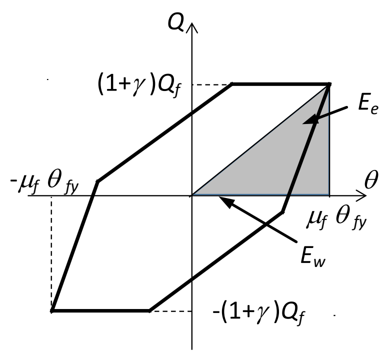


Fig. 6 – Calculation of equivalent viscous damping ratio h_{eq}

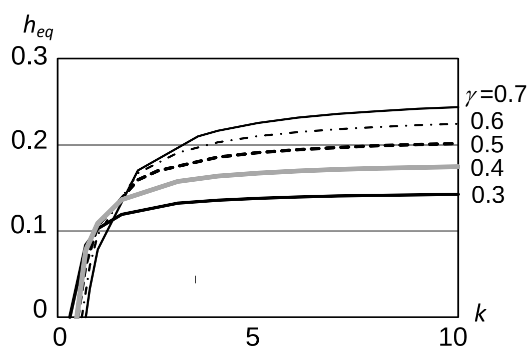


Fig. 7 – Relationship between stiffness ratio k, strength ratio γ and equivalent viscous damping ratio h_{eq}

3.5 Design procedure for seismic retrofit

Fig. 8 shows the design procedure for a stud-type damper for an existing steel frame. In the conventional design procedure for an energy-dissipated device, the response is predicted using a time history analysis of the lumped mass model, and the required degree of stiffening for the device is determined from the required reduction ratio. In this method, it is presupposed that the interaction between the frame and shear wall is not considered and that the energy-dissipated device has sufficient performance. By contrast, this study proposes the design procedure for a shear wall with an efficient damping effect, which is applicable to an existing beam to consider the interaction between the frame and shear wall.

The design procedure for the proposed seismic retrofit is as follows: The damper width and height are determined based on the location of openings and applicable position for a damper in the existing frame (Fig. 8 i). Then, the maximum strength of the damper is derived to prevent shear yielding, bending yielding, or lateral buckling of the existing steel beam connected to the damper (Fig. 8 ii). The number and layout of the dampers are determined in consideration of the desired base shear coefficient of the frame retrofitted with the dampers (Fig. 8 iii). The required minimum stiffness of the damper is estimated to obtain a sufficient equivalent viscous damping ratio that has 0.8 times the maximum achievable damping ratio (Fig. 8 iv). The steel plate thickness and slit arrangement of the damper are determined considering the above design conditions (Fig. 8 vi). If a feasible slitted steel plate is not obtained, the damper size, layout, or strength must be re-examined.

Select target steel frame

1. Damper size is determined based on the location of openings and applicable position for a damper.

2. The maximum strength of the damper is derived to prevent shear yielding or lateral buckling of the existing steel beam connected to the damper.

$$Q_p = \left(Q_{p1} + Q_{p2}\right) \leq 0.8 \cdot \frac{1+\alpha}{1-\alpha} \frac{4M_{bp}}{H}$$

$$Q_p = \left(Q_{p1} + Q_{p2}\right) \leq 0.8 \cdot \frac{2\alpha L}{\sqrt{3}H} \sigma_{by} A_w$$

$$Q_p = \left(Q_{p1} + Q_{p2}\right) \leq 0.8 \cdot \frac{4\left(\alpha M_{bp} + M_{cre}\right)}{H\left(1-\alpha\right)}$$

3. The number and layout of the dampers are determined in consideration of the desired base shear coefficient of the frame retrofitted with the dampers.

4. The required stiffness of the damper is estimated to obtain sufficient energy dissipation. The stiffness that has 0.8 times the maximum acievable damping ratio is chosen as the minimum value.

$$K_p \geq \frac{\gamma}{1-\eta} K_f \qquad \left(\gamma = Q_{wt}/Q_f, \quad 0.8 \leq \eta < 1.0\right)$$

5. The steel plate thickness and slit arrangement of the damper are determined considering the above design conditions.

$$t = \frac{2l}{nb^2\sigma_y} Q_{wt}, \quad K_{wt} = \frac{1}{\dfrac{\kappa(h-ml)}{GBt} + \dfrac{\kappa l}{Gbt} \cdot \dfrac{m}{n} + \dfrac{l^3}{Etb^3} \cdot \dfrac{m}{n}}$$

NO

YES

6. Strength and stiffness of the damper are determined.

Fig. 8 – Design procedure of stud-type shear wall

4. Verification Test of Design Procedure

4.1 Test plan

In this section, a shear wall was designed based on the procedure shown in Fig. 8, and the validity of the design procedure was checked through a frame test. The target structure for the test was a one-story one-span frame of a three-story six-span steel structure with a 3 m height and 6 m span length. The frame was scaled down to 0.6 times the original model because of the size of our loading setup. The frame had columns with a section of H-200 × 100 and beams with a section of H-175 × 175. The column-to-beam stiffness ratio was 1.5. A shear wall was installed in the center of the frame with rigid parts above and below the shear wall. The shear wall was 600 mm height and width, so the panel width ratio α was 0.17, and the panel height ratio β was 0.33. The frame is illustrated in Fig. 9.

The strength and stiffness of the shear wall were determined from Sections 3.3 and 3.4. Using Eqs. (7), (8) and (10), the maximum strength of the shear wall was 141, 85.7, 211 kN by considering of the bending yielding, shear yielding, or lateral buckling of the beam, respectively. The lateral buckling strength of the beam was 1.6 times larger than the full plastic moment, so it was considered that lateral buckling did not occur in the beam. The frame stiffness was 7.1 kN/mm by using the frame strength, Eqs. (3), and (12). The stiffness of shear wall was 18 kN/mm, and the equivalent viscous damping ratio was 0.15, which was 0.8 times as large as the maximum equivalent viscous damping ratio. Therefore, the maximum strength of the shear wall was 85.7 kN, and its minimum stiffness was 18.0 kN/mm. The right side of Fig. 9 shows the slitted steel shear wall that was designed based on this condition. The steel plate had three slits with a shear link aspect ratio of 2.2. The strength of this shear wall was 60.0 kN, and its stiffness was 57.0 kN/mm; therefore, it satisfied the design condition. The steel plate was restrained by 24 mm thick plywood panels with nine bolts in consideration of the design calculations of a previous study[9].

Fig. 10 shows the loading setup and measurement position. Loading was applied using a hydraulic jack with a maximum force of 1.5 MN and was controlled by displacement. Columns were pin connected at their tops and bottoms. A shear force was applied to the two columns through a loading beam that was connected to the jack. The out-of-plane deformation was suppressed by rollers at the connection point of the jack. Cyclic loading up to a story drift angle of 0.03 rad was applied. The shear displacement of shear wall was measured by transducers that were set diagonal to the steel plate. The shear force of the column was calculated using the data from strain gauges that were set on the column flange, and the shear force of the wall was calculated from the difference between the jack force and shear force of the column.

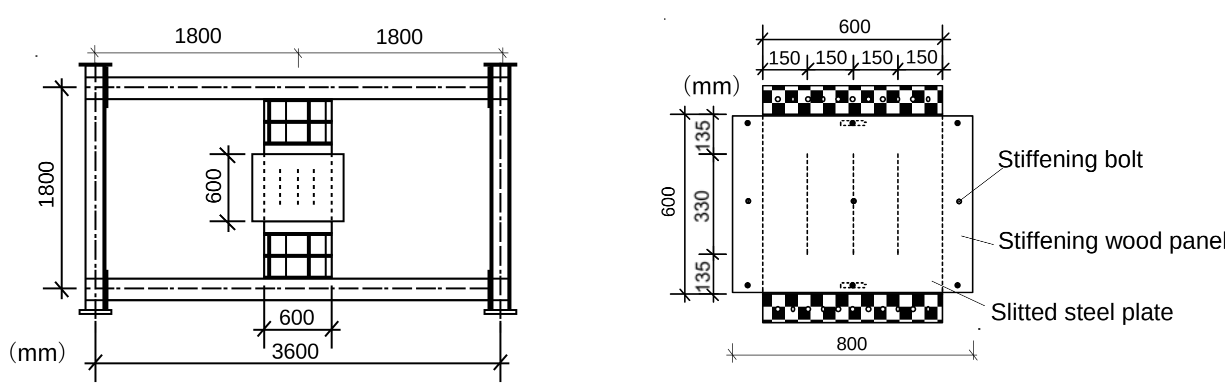


Fig. 9 – Specimen

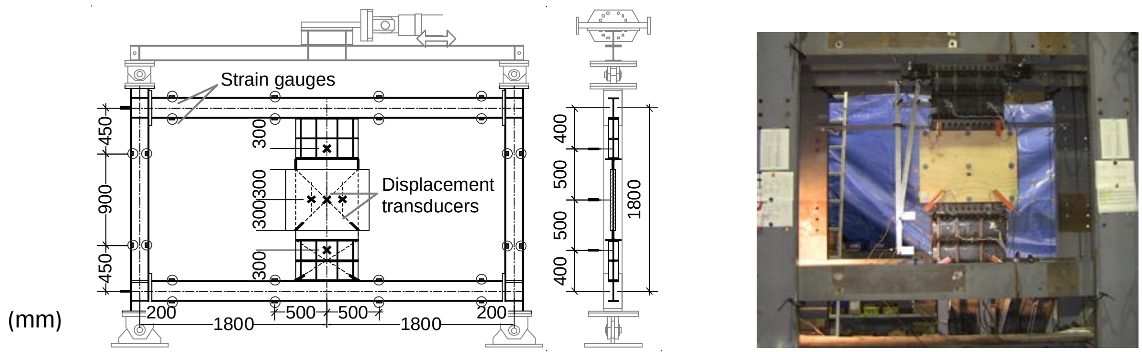


Fig. 10 – Loading setup

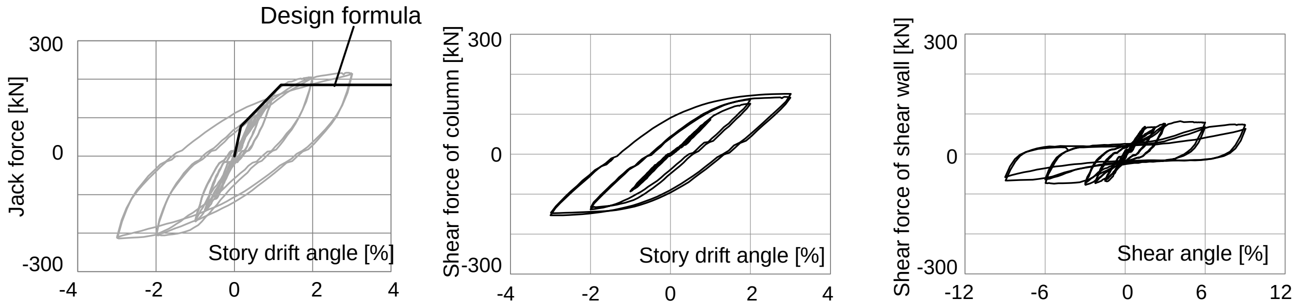
4.2 Test results

Fig. 11 shows the relationship between the jack force and story drift angle, the relationship between the shear force of the column and story drift angle, and the relationship between the shear force of the shear wall and shear angle. The value of the horizontal axis in Fig. 11(c) is the shear angle of the shear wall, which is the shear deformation of the wall divided by its height (600 mm). As the shear wall yielded before the frame, and the beam yielded near the connection of column and beam without yielding near the connection of the shear wall, it was found that the specimen performed as intended. Fig. 11(a) also shows the relationship between the force and deformation angle of the entire frame obtained using the design procedure. Although the maximum strength in the test was somewhat larger than that of the design procedure, the design procedure showed good agreement with the test results. The cause of the increased maximum strength was assumed to be the effect of material properties, such as strain hardening of the steel plate. The maximum strength of shear wall was 80 kN in the test. This was larger than the theoretical value (60 kN), which did not considere strain hardening, but smaller than the maximum design value (85.7 kN).

Table 1 shows a comparison between the yielding angle obtained from the test and the calculation value shown in Section 3.2. The story drift at the time of frame yielding in the test was almost the same value as that in the calculation, but the story drift at the time of yielding of the shear wall in the test was 1.4 times larger than that in the calculation. There were two causes for this difference: First, Eq. (5) was based on the simple model shown in Fig. 3. Second, the rigid parts above and below the shear wall were not stiff enough in the test. In the first case, the rigid connection was simulated by increasing the stiffness of attached beam in the left frame of the simple model in Fig. 3, and the initial stiffness corresponded to that in Fig. 2. Therefore, the shear force acting on the shear wall was larger than that in rigid connection model, so the shear wall yielded sooner, and the stiffness of shear wall increased. Even though the yielding angle of the shear wall in the calculation of the simple model was smaller than that in the test, the difference was less than 10%. In the second case, the rigid parts that were simulated as perfectly stiff in the calculation deformed slightly in the test, which lead to a delay in the shear wall yielding and a larger story drift. This is considered to be the main cause for the difference in this test.

The equivalent viscous damping ratio for the entire frame was approximately 0.18 when the frame yielded at a story drift angle of 1.0% (see Fig. 12). This was larger than the calculated value of 0.15, so the design was appropriate. This also showed sufficient energy dissipation compared to previous tests[4] for a slitted steel plate or other steel shear wall. Fig. 13 is a photo of the slitted steel plate after the test. The steel plate had out-of-plane deformation in the vicinity of the slit ends and horizontal cracks from the slit ends. However, these did not lead to significant degradation of the damping performance, and the shear wall had stable behavior up to a shear angle of 0.09 rad.

In this paper, the validity of the design procedure was confirmed using a model of a one-story one-span steel frame. In the future, the generality of the design procedure and the damping effect of shear wall will be checked using various multi-story multi-span frames.



(a) Frame with shear wall (b) Column (c) Shear wall

Fig. 11 – Hysteresis behavior

Table 1 – Yielding angle

	Story drift angle at the time of yielding of shear wall (%)	Story drift angle at the time of yielding of frame (%)
Test result	0.24	1.00
Calculation	0.17	0.98

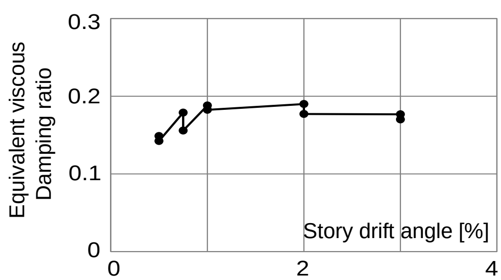


Fig. 12 – Equivalent viscous damping ratio

Fig. 13 – Photo of steel plate after test

5. Conclusions

The design procedure for a stud-type damper was proposed for the retrofit of a steel frame. A shear loading test of a steel frame with a slitted steel plate restrained by wood panels was also conducted to validate the design procedure. The major conclusions are as follows:

1) The maximum strength and the minimum stiffness of the shear wall were obtained to prevent yielding and lateral buckling of the attached beam and to achieve an equivalent viscous damping ratio that was more than

0.8 time the maximum. In consideration of these conditions, the design procedure for a stud-type damper was proposed.

2) In the shear loading test, a specimen designed based on the proposed design procedure displayed the intended behavior: The shear wall yielded before the frame, and the beam yielded near the connection of the column and beam without yielding at either end of the shear wall.

3) The stud-type shear wall showed stable behavior up to the shear angle of 0.09 rad. The equivalent viscous damping ratio of the frame was approximately 0.18, and the shear wall provided sufficient energy dissipation.

6. References

[1] Kamura H, Inoue K, Kuwahara S (2002): Elastic stiffness of shear panel hysteretic damper supported with inner column. *Summaries of technical papers of annual meeting Architectural Institute of Japan*, 603-604.

[2] Kuramoto H, Kohira W, Kato S, Matsui T, Nakamura H, Tokoro K, Uematsu K, Kasai K, Wada A (2006): Dynamic behavior of steel frame installing stud column with viscoelastic damper. *Journal of Structural and Construction Engineering, Architectural Institute of Japan*, 606, 97-104.

[3] Kasai K, Jodai A (2002): Dynamic property, behavior, and their simplified estimation rules for a passive control system with stud-type viscoelastic damper. *Journal of Structural and Construction Engineering, Architectural Institute of Japan,* 558, 125-132.

[4] Ito M, Taniguchi Y, Tsuboyama N, Hoki K, Nakashima M (2012): Stiffening methods for enhancement of hysteretic performance of slitted steel shear wall. *Proceedings of the 15th World Conference on Earthquake Engineering*, Lisbon, No.1750.

[5] Ito M, Taniguchi Y, Hayashi K, Kurata M, Nakashima M (2013): Influence of Design Variables on Performance of Slitted Steel Shear Wall Sandwiched by Wood Panels. *Proceedings of the 10th Pacific Structural Steel Conference*, Singapore, No.49.

[6] Hitaka T, Matsui C (2003): Experimental study on steel shear wall with slits. *Journal of Structural Engineering*, ASCE, 129(5), 586-595.

[7] Architectural Institute of Japan (2009): Recommendations for stability design of steel structures.

[8] Inoue K, Kuwahara S (1998): Optimum strength ratio of hysteretic damper. *Earthquake Engineering and Structural Dynamics*, 26,577-588.

[9] Ito M, Hayashi K, Nakashima M (2015): Design Procedure for Stiffening Panels of Slitted Steel Shear Wall, *Proceedings of International Association for Bridge and Structural Engineers Conference*, Nara, Japan.