An Experimental Study of Shear Wall Units

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AN EXPERIMENTAL STUDY OF SHEAR WALL UNITS

by

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INTRODUCTION

In the medium sized usual steel buildings, steel brace members are often used as earthquake resisting elements in Japan. In this study prefabricated shear wall units consisting of corrugated steel sheets, which are usually used for supporting concrete floor slabs, are considered for earthquake resisting elements instead of brace members, because such prefabricated shear wall units are more suitable for a kind of building system than usual brace members. When those shear wall units are set into the structure, only the top and bottom of the shear wall units are connected to the structural frame through gusset plates (see Fig. 1). For one shear wall unit several steel sheets are used in the vertical direction and are enclosed in a rectangular frame, which consists of angle steel for horizontal members and light gage steel for the vertical members. The main purpose of this study is to investigate the static characteristics of such shear wall unit. In this study three types of connecting methods used between the connections of steel sheets and the horizontal members of the rectangular frame of the unit, are experimentally investigated. Two are "welded connections" and the other is "high strength bolted connection". The aspect ratio of the shear wall unit (defined by "h/W" in Fig.3) is considered to be another important parameter of this experimental study.

TEST SPECIMEN

In this investigation the program was divided into three test series mainly corresponding to the applied connecting details in the connections of steel sheets to the horizontal angle steel members. In the first test series, the welded connections with insert plates are applied (Type I, Fig.2). In this test series light gage square hollow sections are used as the vertical members of the rectangular frames of the units. In the second test series, the welded connections with sandwich plates are applied (Type II, Fig.2). In this test series light gage lipped channel members are used as vertical members. In the third test series, steel sheets are connected to the horizontal members by high strength bolts at only one side of flanges of the corrugated sheets(Type III, Fig.2). In this test series light gage channel members are used as vertical members. The details of each connecting method are shown in Fig.2. The corrugated steel sheets are joined to each other by means of welding in the first and second test series and by means of self-tapping screws in the third test series. The configuration of a specimen is shown in Fig.3 as an example. Each specimen and their principal parameters are listed in Table 1. In this investigation two types of steel sheets with defferent cross section are used. One is a steel sheet whose top and bottom flange widths are equal (Type A), and the other is a steel sheet with unequal flange widths

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(type B).
Each specimen is named as follows.

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W II-17012A
│ │  │  │ └─────────── Type of steel sheets
│ │  │  └──────────── The thickness of steel sheets
│ │  └───────────── The aspect ratio of specimen (x100)
│ └─Type of connecting method of steel sheets to the horizontal member
└── Symbolic character of shear wall test
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The tested mechanical properties of the steels, which are used in this experiment, are shown in Table 2.

TEST PROCEDURE

During testing, the bottom horizontal member of shear wall unit was fixed to the base beam (WF section) on the loading floor and the top horizontal frame member was connected to the loading beam by high strength bolts through gusset plates. The test setup is shown in Fig.4. Horizontal displacement of the top of the shear wall unit was measured by displacement transducers and principal stresses of steel sheets and axial stresses in the vertical frame member of the shear wall unit were measured by wire strain gages. The location of such strain gages is shown in Fig.5. By using the hydraulic acutuater, a gradually increasing cyclic horizontal load was applied to the top of the specimen through the loading beam.

BEHAVIOR OF SPECIMENS IN TEST

In each specimen of the first and second test series, several local bucklings suddenly occured at the center part of the corrugated steel sheet and applied load decreased at the same time, while the specimen behaved almost elastically. The maximum strength of each specimen of these test series was defined by the local bucklings, which occured in the flange of steel sheets. In some specimens slight local buckling occured at the corners, which was followed by the local buckling at center of the specimen. This slight local buckling, however, had little influence on the restoreing force characteristics. After the occurance of local buckling, out-of-plane buckling of the whole steel sheet precipitated a collapse such that adjoining the local bucklings of center part with increasing lateral deformation. When the lateral drift reaches nearly h/50, the load decrease stopped and the shear wall was stable at a certain load level until larger lateral deformation occured. On the other hand, each specimen of the third test series behaved differently from those of the first and second test series. These specimens' local deformation started to occur at the unconnected flanges of top and bottom parts of the corrugated steel sheets as shown in Fig.6 from an early stage of loading. This local deformation increased gradually with increases in the lateral load. In addition, slip phenomenon at each vertical joint between the steel sheets was observed. Finally, out-of-plane buckling of a whole steel sheet occured and this buckling mainly determined the maximum strength. The maximum strength of two specimens of the third test series was determined by the local buckling of flange of vertical member(C-100x50x3.2) of the shear wall unit.

TEST RESULTS

The relationship between the load and horizontal deformation measured at the top of each specimen are shown in Fig.7. The main test results, such as the maximum strength, the local buckling strength, the out-of-plane buckling strength of a whole steel sheet and the initial stiffness of the shear wall unit are listed in Table 3. Because the thickness of steel sheets and the width of shear wall unit are different from each other, direct comparison of test results about each specimen is impossible. Therefore, the apparently equivalent shear stress τ', which can be obtained by dividing the lateral load by product of the thickness of steel sheet and the width of shear wall unit ($\tau'=Q/t\cdot W$), and the lateral drift R are used as the indexes of the load and deformation relationship. The envelopes of τ' versus R relationship of each specimen are shown in Fig.8. The typical distribution of principal stresses in the steel sheets is shown in Fig.9. Two examples of the outline of bending stress distribution in the shear wall unit, based on the measured normal stresses at the vertical frame member, is shown in Fig.10. The typical appearance of specimens after testing are shown in Photo 1.

DISCUSSION

In the third test series simplification of the connection of steel sheets to the horizontal frame members and the mutual joint of steel sheets were checked by experimental study. Judging from the test results, it becomes clear that the high strength bolted and self-tapping screw connection was not so effective that the connected steel sheets could not make full use of their in plane shear capability against the shearing stress. Therefore in the discussion on the test results, the test results of the third test series have been omitted.

1) Initial Stiffness

Fig.11 shows the relationship between the initial stiffness Ko and the aspect ratio of the shear wall unit h/W. Here the initial stiffness is defined as the stiffness at the strength level of 1/3 of the maximum strength. In Fig.11 the initial stiffness is expressed as a non-dimensional value Ko/G, where G is the modulus of transverse elasticity of the steel. From this figure it is clear that the initial stiffness of the shear wall unit decreases with increasing of the aspect ratio. The main reason for this tendency is considered to be the influence of bending effect. This presumption is supported by the bending stress distribution in the shear wall unit shown in Fig.10. The relation between Ko/G and h/W is inversely proportional and is expressed roughly by following linear equation in the renge of $0.5\leq h/W\leq 2.0$.

$$Ko/G=-0.25(h/W)+0.73 \qquad (1)$$

2) Maximum Strength

The apparently equivalent shear stress at the maximum strength $\tau'm$ is plotted in Fig.12 against the aspect ratio h/W. From this figure it is clear that $\tau'm$ decreases with the increase of the aspect ratio. As described above, the maximum strength of each specimen is determined by the local shear buckling of the steel sheet. In general the buckling stress of a plate, which is subjected to shear stress, is calculated based on the width-to-thickness ratio and the aspect ratio of the plate. However, at the flange of the corrugated sheet, the aspect ratio is very large and it is thought that such a large aspect ratio has no significance in the calculation of the buckling stress. In addtion, the difference of the height of the corrugated steel sheet, which is one of the main parameters of this investigation, has little influence on the calculation of the buckling stress. Therefore, in this investigation the estimation of the maximum strength is done based on the bold assumption that each corrugated steel sheet is replaced by a pair of brace members. The cross section of the replaced brace member is assumed to be the same as m times the unit width of the corrugated steel sheet. The replaced brace model is shown in Fig.13. By means of reverse operation from test results, the assumed multiple factor $m = m1$, which corresponds to the maximum strength, is calculated for each specimen and they are shown in Table 4 and plotted in Fig.14 against the aspect ratio of the shear wall unit. From these test results the approximating linear equation between m1 and h/W is obtained as follows:

$$m1=0.68(h/W)+1.67 \tag{2}$$

Then, recalculation of maximum strength of each specimen cQm is done using m1, which is calculated by equation (2). The value of cQm is shown in Table 4 and plotted in Fig.15 against the experimental value Qm. cQm coincides with Qm very well. Therefore, it seems possible to estimate the maximum strength of the shear wall unit by replacing the brace member as shown in Fig.13. In that case the multiple factor m1, which is used for the determination of the width of assumed cross section of the replaced brace member, is obtained by equation (2).

3) The Ultimate Plastic Strength

From observing the restoring force characteristics, this type of shear wall maintains a certain level of shear strength at larger plastic deformation range. The strength, which corresponds to the horizontal drift of h/50, is called the ultimate plastic strength in this paper. The apparently equivalent shear stress, which corresponds to the ultimate plastic strength, is expressed $\tau'p$ and the value of $\tau'p$ is listed in Table 4. The relationship between $\tau'p$ and the aspect ratio of the shear wall unit h/W is shown in Fig.16. It is thought that the ultimate plastic strength is determined by the strength of the tension field, which is formed at the whole shear wall unit. Therefore, it is considered possible to estimate the ultimate plastic strength by replacing the whole shear wall unit with one tensile brace member. The replaced model is shown in Fig.17. In this model the cross section of the replaced brace member is assumed to be the same as m2 times the unit width of the corrugated steel sheet. Again, by means of reverse operation of the test results the value of multiple m2, which corresponds to the ultimate plastic strength, is calculated and listed in Table 4. Those values are plotted in Fig.18 against h/W. From the relationship between m2 and h/W, the approximating linear equation is obtained as follows:

$$m2=-0.96(h/W)+3.88 \tag{3}$$

Then recalculation of the ultimate plastic strength cQp is done using m2, which is obtained by the equation (3). The value of cQp are listed in Table 4 and plotted in Fig.19 against the experimental value Qp. cQp coincides well with Qp.

CONCLUSION

The main conclusions obtained from this experimental study are summerized as follows:

1) In order to use the full capacity of corrugated steel sheets in shear wall, it is very important to connect the steel sheets to the

surrounding rectangular frame at their vertical and horizontal edges. At these connections both flanges of the corrugated steel sheets must be fixed. For this purpose, welding them with insert plates or sandwich plates is effective.

2) When the above mentioned connection method is practically applicable,(e.g., in fabrication, installation and economy,) from the view point of the static characterristics the prefabricated shear wall unit consisting of corrugated steel sheets can be used well as an earthquake resisting element for medium size steel builings.

3) The main factors which feature the static chracteristcs of the shear wall unit are initial stiffness, maximum strength due to local buckling and ultimate plastic strength. As the analytical results show, it was found that the effect of the variation of the aspect ratio of the shear wall unit can be evaluated by a simple static model and also their characteristics can be expressed by function of the aspect ratio.

ACKNOWLEDGEMENTS

The whole test series was carried out in collaboration with the students in professor Tanaka's laboratory at Utsunomiya University. The authers would to thank them for their intense effort and collaboration, especially the late Mr.Kazuhito Kawata, who was a graduate student of the laboratory at the time. The authers would like to dedicate this paper to him.

APENDIX-NOTATION

- G : the modules of transverse elasticity
- h : the height of the shear wall unit
- Ko : the initial stiffness of the shear wall unit
- m : the multiple factor of the unit width of the corrugated steel sheet for calculating the sectional area of the assumed replaced brace member
- m1 : m value for the estimation of the maximum strength of the shear wall unit
- m2 : m value for the estimation of the ultimate plastic strength of the shear wall unit
- Q : the lateral load subjected to the top of the shear wall unit and the in plane shear strength of the shear wall unit
- R : the lateral drift $= \delta / h$
- t : the thickness of the corrugated steel sheets of shear wall unit
- W : the width of the shear wall unit
- δ : the rateral deformation of the top of shear wall unit
- τ' : the apparently equivalent shear stress in the shear wall $= Q/t \cdot W$

Table 1 List of Specimens

Specimen	Specimen height h (mm)	Specimen Width W (mm)	Thickness of steel sheets t (mm)	Aspect ratio h/W	Vertical and Horizontal member (mm)	Type of Connection
WI-08512B	2250	2662	1.2	0.85		I
WI-08516B	2250	2662	1.6	0.85	□-75x75x3.2	I
WI-07016B	2250	3276	1.6	0.69		I
WII-08516B	2225	2662	1.6	0.84	∟-125x75x7	II
WI-08512A	2250	2662	1.2	0.85		I
WI-08516A	2250	2606	1.6	0.86		I
WII-13512A	2300	1700	1.2	1.35		II
WII-17012A	2900	1700	1.2	1.71	[-100x50x20x4.5	II
WII-17016A	2900	1700	1.6	1.71		II
WII-19512A	3300	1700	1.2	1.95	∟-75x75x6	II
WII-19516A	3300	1700	1.6	1.95		II
WIII-13512A	2300	1700	1.2	1.35		III
WIII-13516A	2300	1700	1.6	1.35		III
WIII-17012A	2900	1700	1.2	1.71	⊏-100x50x3.2	III
WIII-17016A	2900	1700	1.6	1.71		III
WIII-25012A	3300	1310	1.2	2.52	∟-75x75x6	III
WIII-25016A	3300	1310	1.6	2.52		III
WIII-25016A'	3300	1310	1.6	2.52		III

Table 2 Mechanical Properties of Materials

Steel Sheet					Rectangular Frame			
Name of specimen	Thickness (mm)	Y.P. (MPa)	T.S. (MPa)	El. (%)	Vertical Member (mm)	Y.P. (MPa)	T.S. (MPa)	El. (%)
WI-08512B	1.2	273	382	32	□-75x75x3.2	339	377	25
WI-08516B WI-07016B WII-08516B	1.6	370	483	22				
WI-08512A	1.2	298	353	29				
WI-08516A	1.6	357	449	26				
WII-13512A WII-17012A WII-19512A	1.2	247	347	31	[-100x50x20x4.5	301	455	25
WII-17016A WII-19516A	1.6	329	459	29				
WIII-13512A WIII-17012A WIII-25012A	1.2	238	339	34	⊏-100x50x3.2	342	444	26
WIII-13516A WIII-17016A WIII-25016A WIII-25016A'	1.6	318	474	26				

Y.P. :Yield Point T.S. :Tensile Strength El. :Elongation

Table 3 Test Results

Specimen	Maximum strength (KN)	Local buckling strength	Out of Plane Buckling strength	Initial stiffness K_0 ($\times 10^4$ N/mm^2)	K_0/G
WI-08512B	235 -225	235 (KN) 0.0019 (rad)	172 (KN) 0.0105 (rad)	3.48	0.44
WI-08516B	390 -357	390 (KN) 0.0034 (rad)	-339 (KN) 0.0038 (rad)	3.55	0.45
WI-07016B	475 -453	475 (KN) 0.0033 (rad)	475 (KN) 0.0033 (rad)	4.99	0.63
WⅡ-08516B	392 -399	353 (KN) 0.0026 (rad)	353 (KN) 0.0026 (rad)	3.55	0.45
WI-08512A	323 -294	320 (KN) 0.0038 (rad)	240 (KN) 0.0034 (rad)	5.02	0.63
WI-08516A	475 -453	407 (KN) 0.0032 (rad)	304 (KN) 0.0076 (rad)	4.17	0.53
WⅡ-13512A	149 -130	149 (KN) 0.0045 (rad)	149 (KN) 0.0045 (rad)	2.79	0.35
WⅡ-17012A	208 -169	208 (KN) 0.0065 (rad)	208 (KN) 0.0065 (rad)	1.97	0.25
WⅡ-17016A	173 -172	158 (KN) 0.0041 (rad)	173 (KN) 0.0053 (rad)	1.54	0.19
WⅡ-19512A	253 -216	252 (KN) 0.0085 (rad)	253 (KN) 0.0086 (rad)	2.38	0.29
WⅡ-19516A	193 -170	193 (KN) 0.0057 (rad)	-128 (KN) 0.0061 (rad)	1.75	0.22
WⅢ-13512A	85 -99	83 (KN) 0.0100 (rad)	— —	0.21	0.03
WⅢ-13516A	186 -97	-78 (KN) 0.0120 (rad)	-78* (KN) 0.0040* (rad)	0.39	0.05
WⅢ-17012A	86 -86	86 (KN) 0.0100 (rad)	83 (KN) 0.0150 (rad)	0.39	0.05
WⅢ-17016A	144 -137	131 (KN) 0.0100 (rad)	-127* (KN) 0.0080* (rad)	0.68	0.09
WⅢ-25012A	65 -65	-56 (KN) 0.0070 (rad)	65 (KN) 0.0100 (rad)	0.39	0.05
WⅢ-25016A	78 -116	-108 (KN) 0.0070 (rad)	75 (KN) 0.0120 (rad)	0.39	0.05
WⅢ-25016A'	101 -96	101 (KN) 0.0100 (rad)	101 (KN) 0.0100 (rad)	0.45	0.06

G: Modulus of Transverse Elasticity (=7.94 $\times 10^4$ N/mm^2)

*: Corresponding Value for Local Buckling of Vertical Member

Table 4 Test Results and Calculated Values about Strength

Name of specimen	τ'_m (Mpa)	m_1	Qm (kN)	cQm (KN)	Qm/cQm	τ'_P (MPa)	m_2	Q_P (kN)	$_cQ_P$ (kN)	$Q_P/{}_cQ_P$
W I-08512B	79.8	1.96	235	270	0.87	47.3	3.00	137	140	0.98
W I-08516B	99.3	2.31	390	379	1.03	62.6	2.66	219	252	0.87
W I-07016B	96.7	2.26	475	451	1.05	54.2	3.01	236	253	0.93
W II-08516B	99.8	2.29	399	384	1.02	75.9	3.73	265	253	1.05
W I-08512A	109.8	2.40	323	304	1.06	49.3	2.86	143	154	0.93
W I-08516A	123.8	2.46	475	451	1.06	70.4	2.99	250	248	1.01
W II-13512A	73.0	2.96	149	151	0.98	36.6	2.24	74	67	1.11
W II-17012A	77.1	3.18	210	199	1.06	36.1	1.68	98	118	0.83
W II-17016A	84.6	2.73	172	179	0.96	43.2	2.46	88	80	1.10
W II-19512A	92.9	3.04	253	236	1.07	42.5	1.84	116	141	0.82
W II-19516A	92.7	2.19	193	229	0.84	59.6	3.08	122	102	1.19

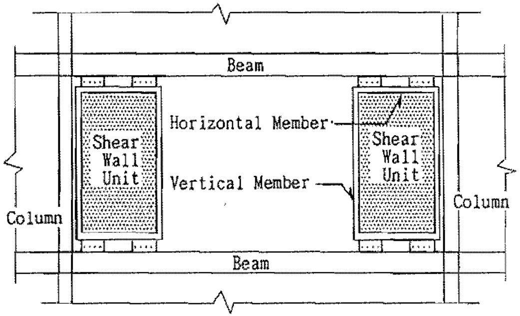


Fig.1 Setting of the Shear Wall Unit to the Structure Frame

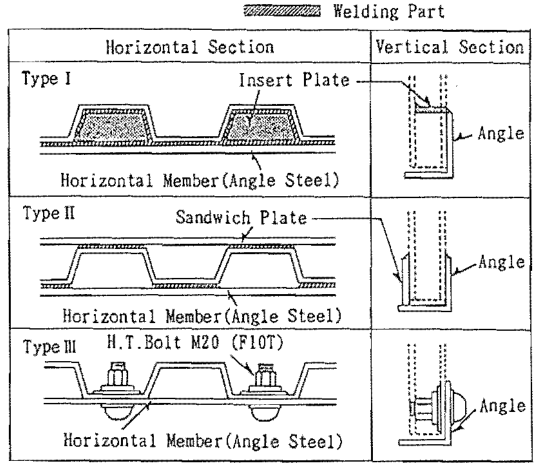


Fig.2 Detail of Connecton Methods

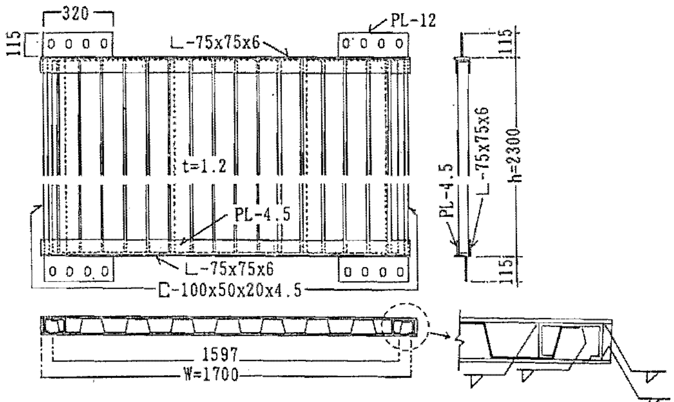


Fig.3 Configuration of Specimen (WII-13512A)

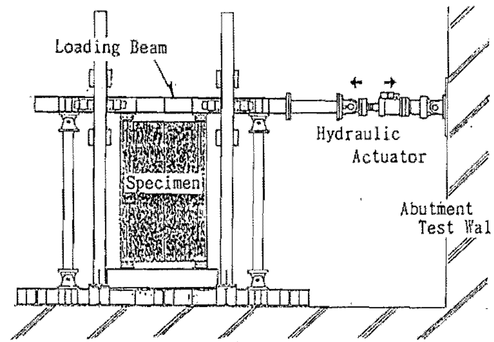


Fig.4 Loading Setup

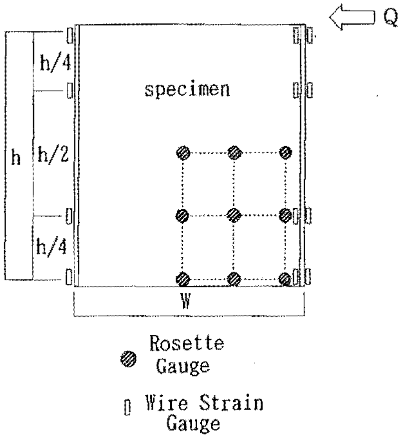


Fig.5 Location of the Strain Gauge

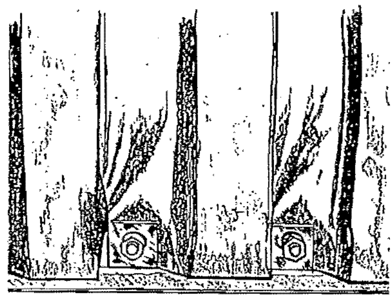


Fig.6 Local Deformation of Steel Sheet near H.S.Bolted Connection

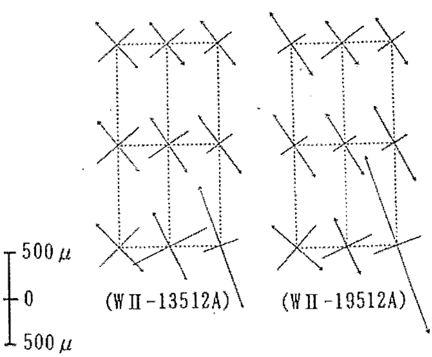


Fig.9 Distribution of Principal Stresses

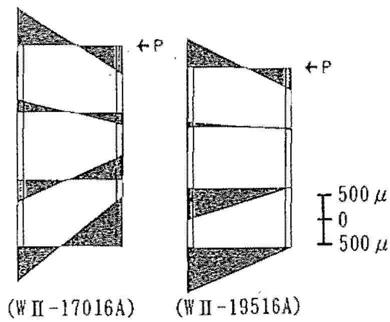


Fig.10 Distribution of Normal Stresses

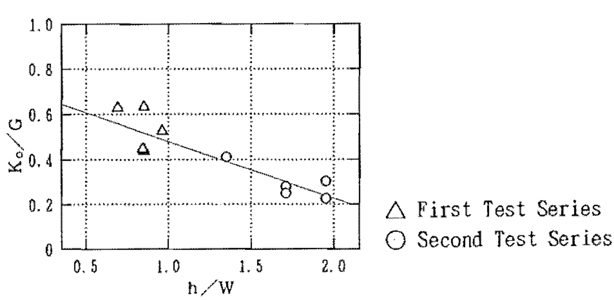


Fig.11 $K_0/G - h/W$ Relationship

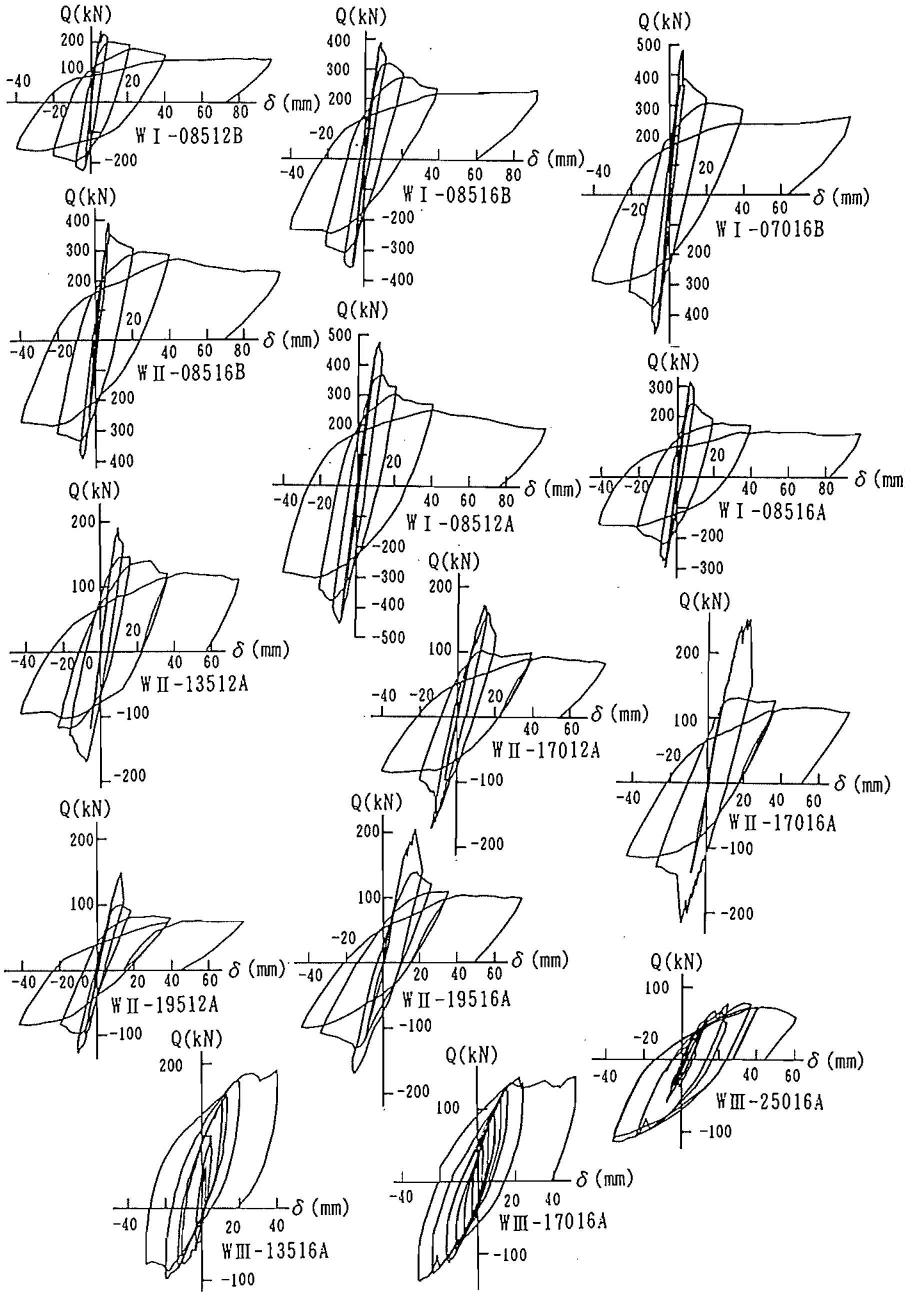


Fig.7 Load versus Displacement Relationship

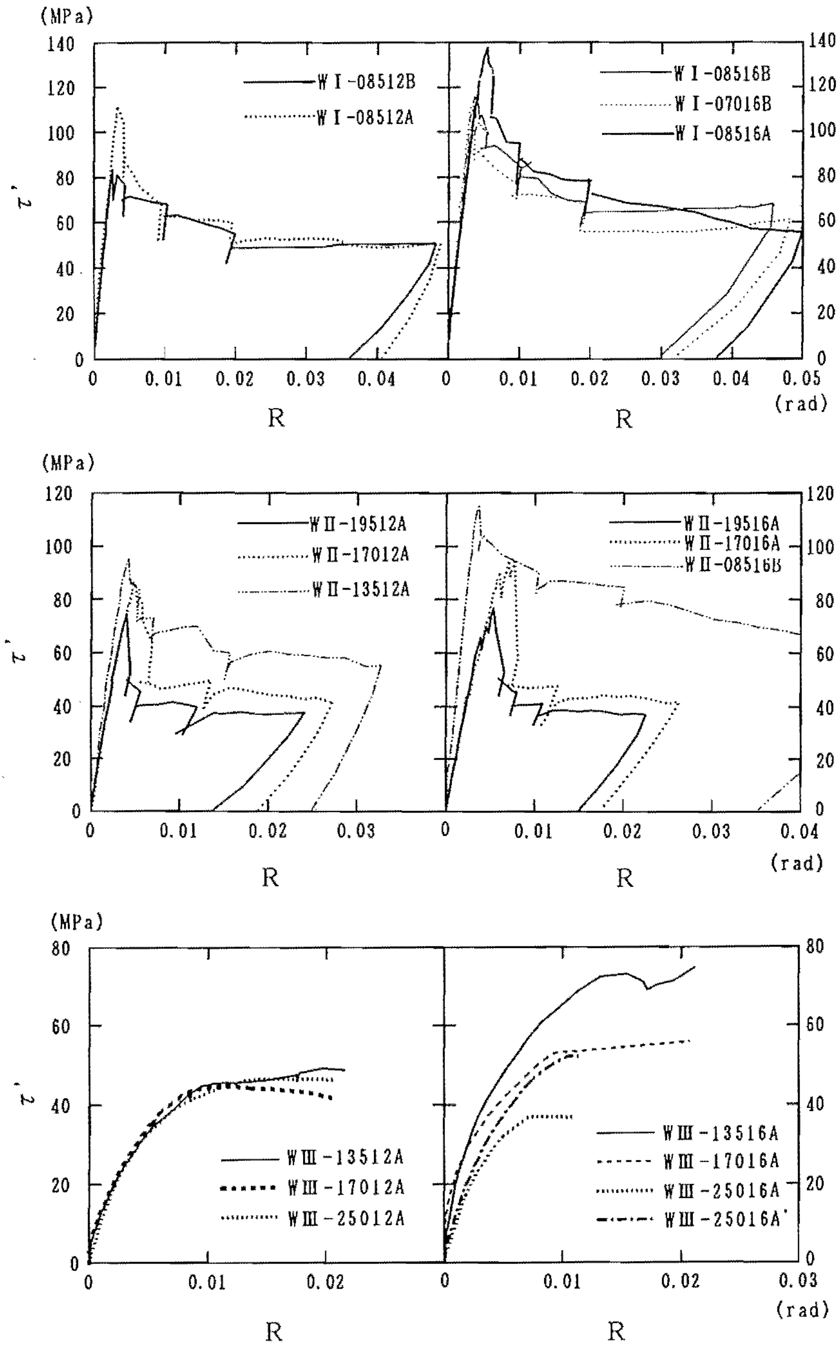


Fig.8 Envelopes of τ' versus R Relationship

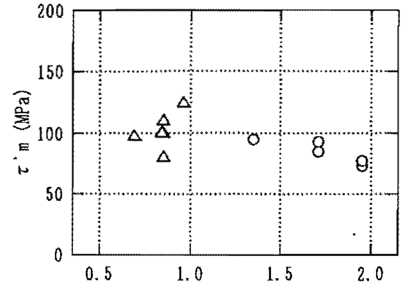


Fig.12 Maximum Strength

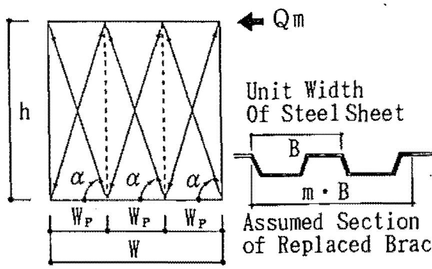


Fig.13 Brace Model for Estimation of Maximum Strength

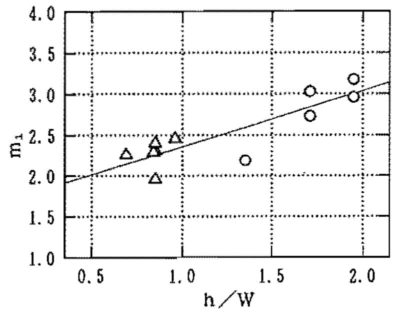


Fig.14 m_1－h/W Relationship

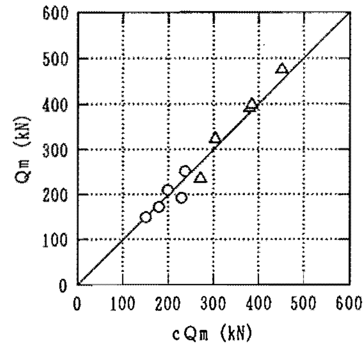


Fig.15 Comparision of Maximum Strength cQm and Qm

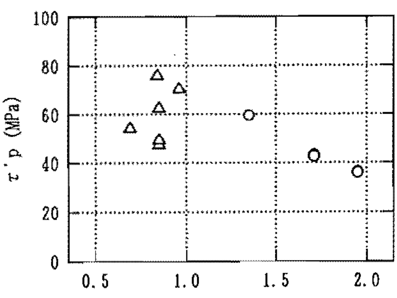


Fig.16 Ultimate Plastic Strength

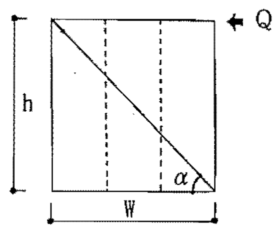


Fig.17 Brace Model for Estimation of the Ultimate Plastic Strength

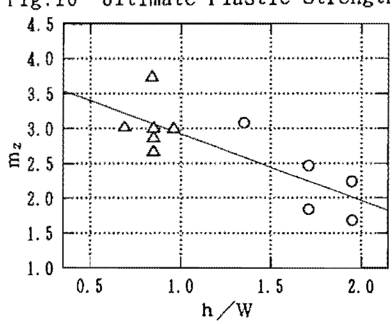


Fig.18 m_2－h/W Relationship

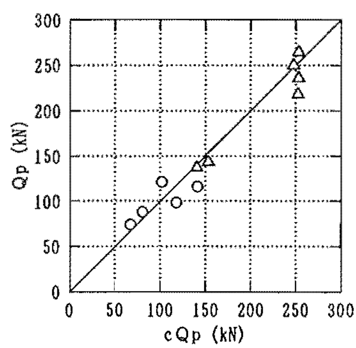
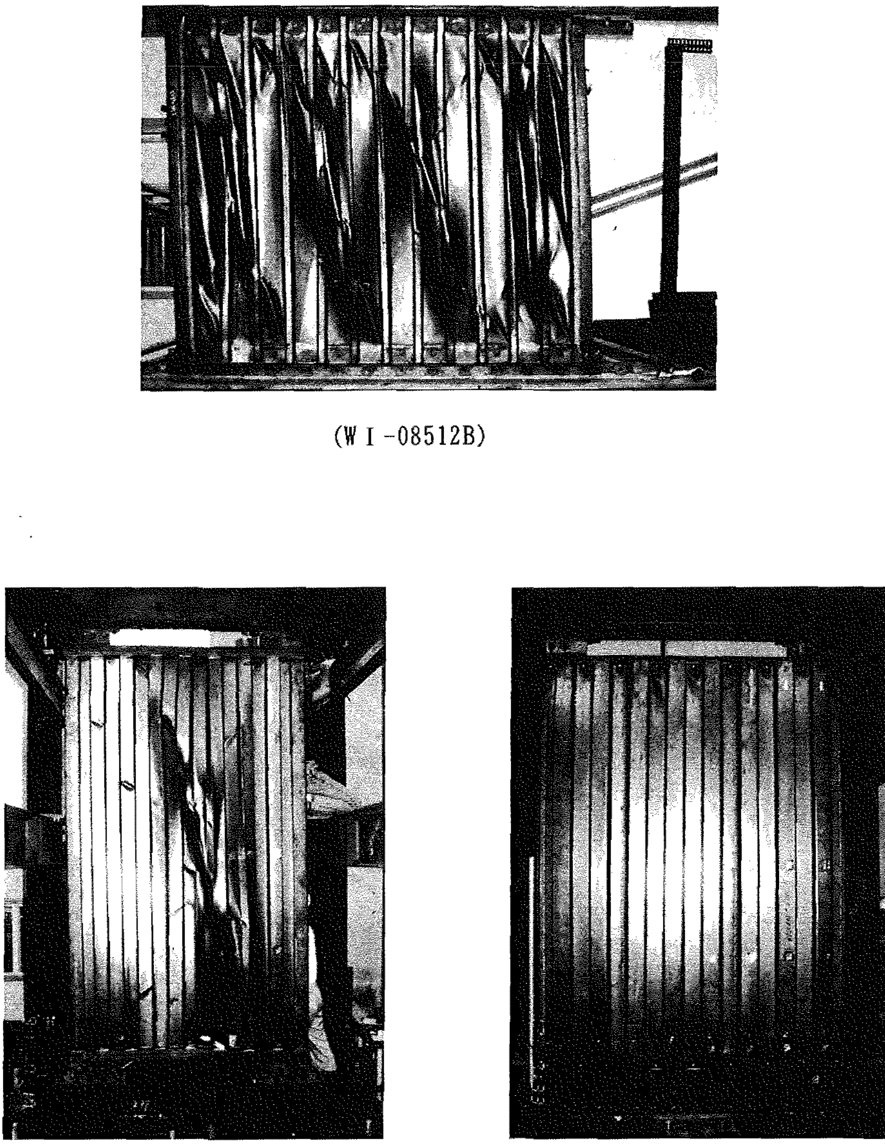


Fig.19 Comparision of Ultimate Plastic Strength cQp and Qp



(W I -08512B)

(W II -13516A)

(W III -17016A)

Photo 1 Buckling of Steel Sheets

AN EXPERIMENTAL STUDY OF SHEAR WALL UNITS

by

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SUMMARY

The static characteristics of prefabricated shear walls consisting of corrugated steel sheets are experimentally investigated. It becomes clear that the connecting method of corrugated steel sheets to the surrounding frame is very important in order to use the full capacity of steel sheets.