DESIGN METHOD OF REINFORCED CONCRETE FRAMED SHEAR WALLS
TO SUSTAIN VERTICAL LOADS AFTER SHEAR FAILURE

M. Tomii (I)
F. Esaki (II)
Presenting Author: M. Tomii

SUMMARY

This paper presents a design method of reinforced concrete monolithic framed shear walls whose columns do not fail in shear by an earthquake and whose horizontal shear capacity is dominated by slip failure of their infilled panel wall. This type of shear failure of reinforced concrete framed shear walls is not dangerous, because even if the relative story displacement of the framed shear walls increases after the slip failure the columns can sustain vertical loads of the framed shear walls and fairly good ductility can be expected.

INTRODUCTION

In the experiments of isolated slender multistory framed shear walls subjected to lateral forces, many of the shear walls fail in flexure. In Japan, however, where because of frequent severe earthquakes most of the reinforced concrete frame structures are less than seven stories high, the framed shear walls (hereafter called "shear wall"), not only squat one-story shear walls but also slender multistory ones, arranged in reinforced concrete buildings of the three-dimensional frame structure, frequently fail in shear by an earthquake since the flexural deformations of the shear walls are restrained by the surrounding structure which sustains both dead loads and live loads.

Besides "slip failure of the panel wall" mentioned in the summary, there is another type of shear failure of shear walls which is induced by "shear failure of the edge columns". If the columns fail in shear, bearing capacity of the shear wall decreases, and the upper stories supported by the shear walls are in danger of falling.

In order to prevent such dangerous brittle failure, the authors propose a design method of shear walls in which the shear failure is induced by slip failure of the panel wall.

DESIGN METHOD

In regard to one-bay one-story shear walls as to which sufficient data of lateral shear tests were obtained, the authors proposed semi-analytical formulae to calculate the following lateral shear capacities:

$Q_{uo(ws)}$ = lateral shear capacity dominated by slip failure of panal wall

(I) Professor of Structural Engineering, Kyushu University, Fukuoka, Japan

(II) Research Assistant of Structural Engineering, Kyushu University, Fukuoka, Japan

$Q_{uo(cs)}$, $Q_{uo(bs)}$ = lateral shear capacity dominated by shear failure of edge columns and that of edge beams

and proved that if the shear walls are designed to satisfy Eq. 1, shear failure is induced by slip failure of the panel wall (see Fig. 1) and can easily be repaired[1].

$$Q_{uo(ws)} \leqq 0.8\,min(Q_{uo(cs)},\ Q_{uo(bs)}) \quad \cdots\cdots\cdots\cdot(1)$$

When shear cracks occur in the panel wall of continuous shear walls frequently used in actual buildings, the flexural deformation of the intermediate columns (beams) between the panel walls is smaller than that of the edge columns (beams)[2]. Therefore, lateral shear capacity per lateral length, $Q_u/\Sigma l$, of continuous shear walls is larger than that of one-bay one-story shear walls in either mode of shear failure, since more effective confinement of the boundary frame to the expansion of the shear cracked orthotropic infilled panel wall which behaves as compression diagonal field by shear can be expected in continuous shear walls than in one-bay one-story shear walls[3],

where l = distance from center to center of columns of each unit shear wall.

But in regard to continuous shear walls the data of lateral shear tests are not sufficient to determine the increase of lateral shear capacity.

The authors conducted lateral shear tests of eight specimens of two-bay one-story shear walls assumed to be 1/10-scale models of two-bay multistory shear walls by making their edge beams rigid (see Fig. 2). The mechanical behavior and figures of these false two-bay multistory shear walls are defined as follows.

i Each unit shear wall satisfies the critical condition of Eq. 1 (see Table 2). The external forces which are polar-symmetrical with respect to the center of the panel wall are assumed as in Fig. 3, where

N = the sum of vertical loads applied to unit shear wall,

and the angle between each shear crack and horizontal direction is assumed as 45°.

ii Each column section satisfies the requirement mentioned in the commentary of A.I.J. Code[4]:

$$b_c D_c \geqq \frac{st}{2} \quad \text{and} \quad min(b_c,\ D_c) \geqq min\left(\sqrt{\frac{st}{3}}\ ,\ 2t\right),$$

where b_c = width of each column of shear wall,

D_c = depth of each column of shear wall,

$s = min(l',\ h')$,

l' = clear length of panel wall fixed to the column,

h' = clear height of panel wall fixed to the column,

t = thickness of panel wall fixed to the column,

and is close to the minimum requirements (see Table 1 and Fig. 2).

By experimental results of all the specimens, it is proved that:

i Shear failure is induced by slip failure of the panel wall.

ii When the angular shear distortion, R (= δ_s/h', where δ_s is relative lateral displacement between the inside faces of the upper and bottom continuous rigid edge beams of the shear wall and given by the mean value of δ_s measured at the midspan of each bay and at the joints of the intermediate column.), reaches approximately 4×10^{-3} rad, the lateral shear force, ${}_{ex}Q$, applied to the framed shear wall becomes maximum, ${}_{ex}Q_{u(ws)}$.

iii Even when R is approximately doubled after ${}_{ex}Q$ reaches ${}_{ex}Q_{u(ws)}$, that is, R reaches 8×10^{-3} rad, each shear wall can sustain the vertical load safely and carry more than 90% of ${}_{ex}Q_{u(ws)}$.

iv When R reaches over 8×10^{-3} rad, Q decreases remarkably since the panel wall is crushed, but each shear wall can still sustain the vertical loads.

From these experimental results it is proved that in multistory shear walls whose unit shear walls are designed to satisfy the condition given by Eq. 1, the shear failure is induced by the slip failure of the panel wall and the damage can be repaired easily. Besides, fairly good ductility can be expected if R is less than 8×10^{-3} rad.

When, besides the design condition mentioned above, the condition that the design lateral shear force of shear walls, Q_D, is to be below $Q_{uo(ws)min}$ which is the lower bound of $Q_{uo(ws)}$ is given, the thickness of the panel wall and the total sectional area of longitudinal reinforcing bars of each column must satisfy the conditions given by Eq. 2.

$$t_o \leqq t \leqq \frac{\alpha_{2c}/l'}{3\sqrt{F_c}+4250p_{s\,min}-\alpha_{1c}\,p_h\,\sigma_{yh}-0.262\alpha_{2c}\,p_v\,\sigma_{yv}}(17.16b_cD_c+0.748a_g\,\sigma_{yg}+0.262N) \qquad \text{..........(2)}$$

where
$$t_o = \frac{1.25Q_D/\Sigma l}{(1+\alpha_{(N')(ws)})\,(2.4\sqrt{F_c}+3400p_{s\,min})} \quad \text{if } 2.4\sqrt{F_c}+3400p_{s\,min} \leqq 60\text{kg/cm}^2$$

$$t_o = \frac{Q_D/\Sigma l}{(1+\alpha_{(N')(ws)})\,(2.4\sqrt{F_c}+3400p_{s\,min}-12)} \quad \text{if } 2.4\sqrt{F_c}+3400p_{s\,min} > 60\text{kg/cm}^2$$

$$\alpha_{1c} = \left(\frac{h'}{l'}-\frac{2D_c}{l'}\right)\alpha_{2c}$$

$$\alpha_{2c} = \frac{1}{0.262-0.476\frac{D_c}{l'}+0.738\frac{h'}{l'}}$$

a_g, σ_{yg} = total sectional area (cm^2) and yield strength (kg/cm^2) of longitudinal reinforcing bars of each column, where $a_g\,\sigma_{yg} \leqq 80b_c\,D_c$ (kg)

p_v, σ_{yv} = shear reinforcement ratio and yield strength (kg/cm^2) of shear reinforcement in vertical and horizontal direction in panel wall, where $p_v\,\sigma_{yv} \leqq 30kg/cm^2$

p_h, σ_{yh} = shear reinforcement ratio and yield strength (kg/cm^2) of shear reinforcement in vertical and horizontal direction in panel wall, where $p_h\,\sigma_{yh} \leqq 30kg/cm^2$

$p_{s\,min}$ = the smaller of p_v and p_h

$\alpha_{(N')(ws)}$ = the rate of increase of $Q_{uo(ws)}$

F_c = compressive strength of concrete (kg/cm^2)

The factors α_{1c} and α_{2c} can be given in the design chart of Fig. 4. The allowable lateral shear capacity of the shear walls which satisfies the condition of Eq. 2, $Q_{uo(ws)min}$, can be given in the design chart of Fig. 5.

LATERAL SHEAR TESTS OF FALSE MULTISTORY SHEAR WALLS WITH TWO BAYS

Details of specimens are shown in Table 1 and Fig. 2. The shear walls were tested by the test setup shown in Fig. 6. Test results of all specimens are shown in Table 2. The crack patterns at $R=8\times10^{-3}$ rad and hysteresis curves of all specimens are shown in Fig. 7.

CONCLUSION

It is recommended to design those framed shear walls in which the thickness of each panel wall and the total sectional area of longitudinal reinforcing bars of each column satisfy the conditions given by Eq. 2, since

i the shear failure of such shear walls is induced by slip failure of the panel wall and the unfailing columns can sustain the vertical loads when bearing capacity of the slipped and crushed panel wall remarkably decreases,

ii the damage can be repaired easily,

iii fairly good ductility can be expected.

ACKNOWLEDGEMENTS

This investigation was carried out at Kyushu University under the support of the Grant-in-Aid for Scientific Research from Japanese Ministry of Education.

The authors wish to thank Mr. F. Fujiwara, engineer of Kyushu University, and Mr. M. Fujita, a graduate student of Kyushu University, who assisted in this experiment and Miss Kiyoko Furuta, secretary, who typed the manuscript of this paper.

REFERENCES

1. Tomii, M. and Esaki, F., "Predicting Method for Shear Failure Modes of Reinforced Concrete Framed Shear Walls," Transactions of the Japan Concrete Institute (Trans. of J.C.I.), Vol. 4, 1982.
2. Tomii, M., Esaki, F. and Funamoto, K., "Mechanical Behavior of Two-Story or Two-Bay Duplex Framed Shear Walls Having Shear Cracked Panel Walls," Trans. of J.C.I. Vol. 4, 1982.
3. Tomii, M., Sueoka, T. and Hiraishi, H., "Elastic Analysis of Framed Shear Walls by Assuming Their Infilled Panel Walls to Be 45-degree Orthotropic Plates, Part I and II," Transactions of the Architectural Institute of Japan (Trans. of A.I.J.), No. 280, June 1979 and No. 284, Oct. 1979.
4. A.I.J. Standard and Commentary for Structural Calculation of Reinforced Concrete Structures, Revised in June, 1982 (in Japanese). The Article and Commentary of earthquake resistant shear walls (framed shear walls) are the same as those in the 1979 revision which is translated into English.

Table 1 Properties of specimens

SPECIMEN	l (cm)	EACH COLUMN b_c (cm)	D_c (cm)	p_g (%)	EACH PANEL WALL t (cm)	p_s (%)	F_c ($\frac{kg}{cm^2}$)	ϕ
D2 -6/2 -H	70.0	6	6	2.68	2.0	0.61	189	1.29
D2 -7/2.5-M	70.0	7	7	2.95	2.5	0.61	185	1.40
D1.5-6/2.5-H	52.5	6	6	3.13	2.5	0.61	199	1.03
D1.5-6/2 -M	52.5	6	6	3.13	2.0	0.61	266	1.29
D2 -6/1.5-M	70.0	6	6	2.23	1.5	0.62	243	1.60
D2 -7/2 -L	70.0	7	7	2.95	2.0	0.61	257	1.62
D2 -8/2.5-L	70.0	8	8	2.76	2.5	0.61	169	1.60
D1.5-6/1.5-L	52.5	6	6	2.68	1.5	0.62	210	1.60

Notes: 1) $p_g = a_g/(b_c D_c)$, $\sigma_{yg} = 2510 kg/cm^2$
2) $p_s = p_v = p_h$, $\sigma_y = \sigma_{yv} = \sigma_{yh} = 2290 kg/cm^2$
3) $\phi = min\left[\frac{b_c D_c}{st/2}, \frac{min(b_c, D_c)}{\sqrt{st/3}}, \frac{min(b_c, D_c)}{2t}\right]$

Table 2 Test results

SPECIMEN	$\frac{N}{2b_c D_c}$	$_{ex}\bar{\tau}_{cr}$ (kg/cm²)	$_{ex}\bar{\tau}_{u(ws)}$ (kg/cm²)	$\frac{_{ex}\bar{\tau}_{u(ws)}}{\bar{\tau}_{uo(ws)}}$	$\bar{\tau}_{uo(ws)}$ (kg/cm²)	$\bar{\tau}_{uo(cs)}$ (kg/cm²)
D2 -6/2 -H	$0.36F_c$	21.4 -21.4	76.1 -72.9	1.42 1.36	53.7	67.1
D2 -7/2.5-M	$0.22F_c$	24.9 -20.0	72.5 -70.6	1.36 1.32	53.4	66.8
D1.5-6/2.5-H	$0.37F_c$	32.4 -28.6	79.8 -80.4	1.46 1.47	54.6	68.2
D1.5-6/2 -M	$0.20F_c$	45.7 -45.2	98.8 -91.9	1.65 1.53	59.9	74.9
D2 -6/1.5-M	$0.21F_c$	31.3 -25.4	89.1 -83.9	1.52 1.43	58.2	73.1
D2 -7/2 -L	$0.09F_c$	30.4 -17.9	90.8 -81.0	1.53 1.37	59.2	74.0
D2 -8/2.5-L	0	15.7 -15.7	67.5 -60.8	1.30 1.17	51.9	65.3
D1.5-6/1.5-L	$0.03F_c$	25.4 -10.4	83.1 -73.0	1.49 1.31	55.9	69.8

Notes: 1) $_{ex}\bar{\tau}_{cr} = {}_{ex}Q_{cr}/(t\Sigma l)$, where
$_{ex}Q_{cr}$ = experimental lateral shear force of the shear wall at first shear cracking
2) $_{ex}\bar{\tau}_{u(ws)} = {}_{ex}Q_{u(ws)}/(t\Sigma l)$
3) $\bar{\tau}_{uo(ws)} = Q_{uo(ws)}/(t\Sigma l)$
4) $\bar{\tau}_{uo(cs)} = Q_{uo(cs)}/(t\Sigma l)$

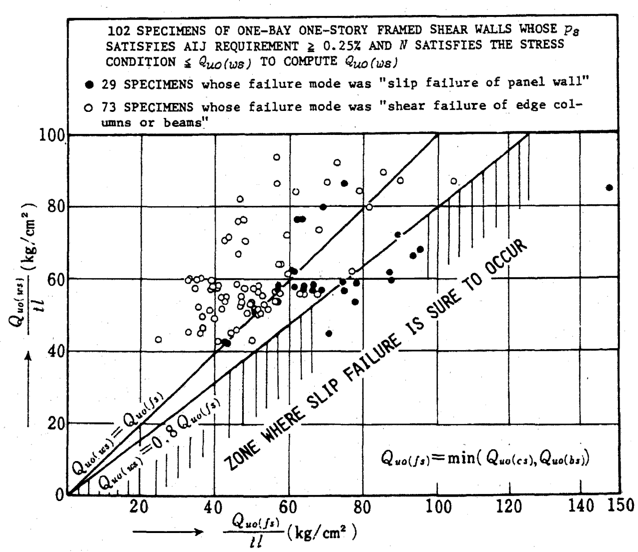


Fig. 1 Relation between $Q_{uo(ws)}/Q_{uo(fs)}$ and shear failure modes

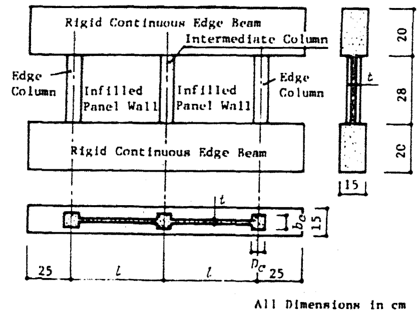


Fig. 2 Definitions of test specimens

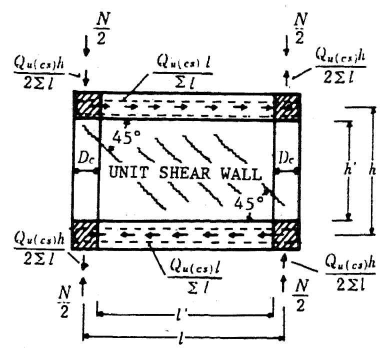


Fig. 3 External forces assumed to apply to unit shear wall in derivation of Eq. 2

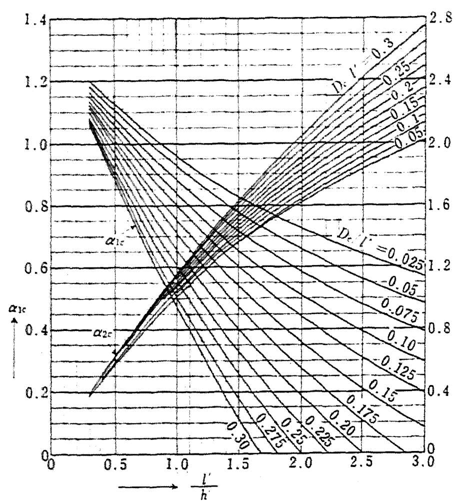


Fig. 4 Design chart for shape factors α_{1c} and α_{2c}

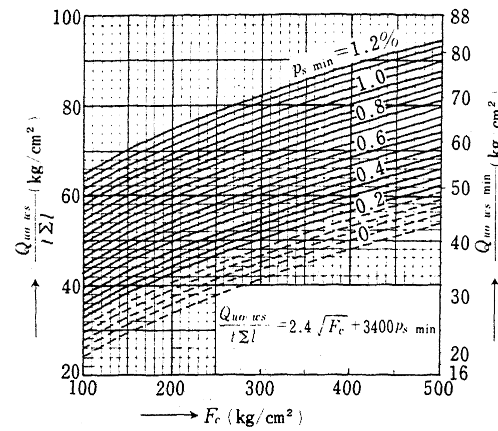


Fig. 5 Design chart for allowable lateral shear capacity $Q_{uo(ws)min}$

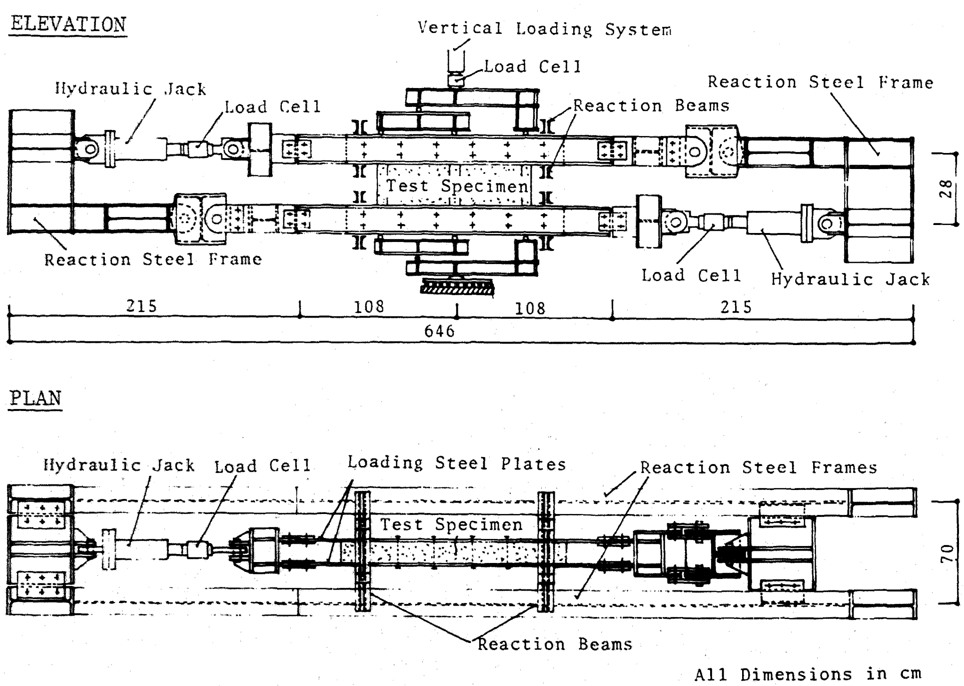
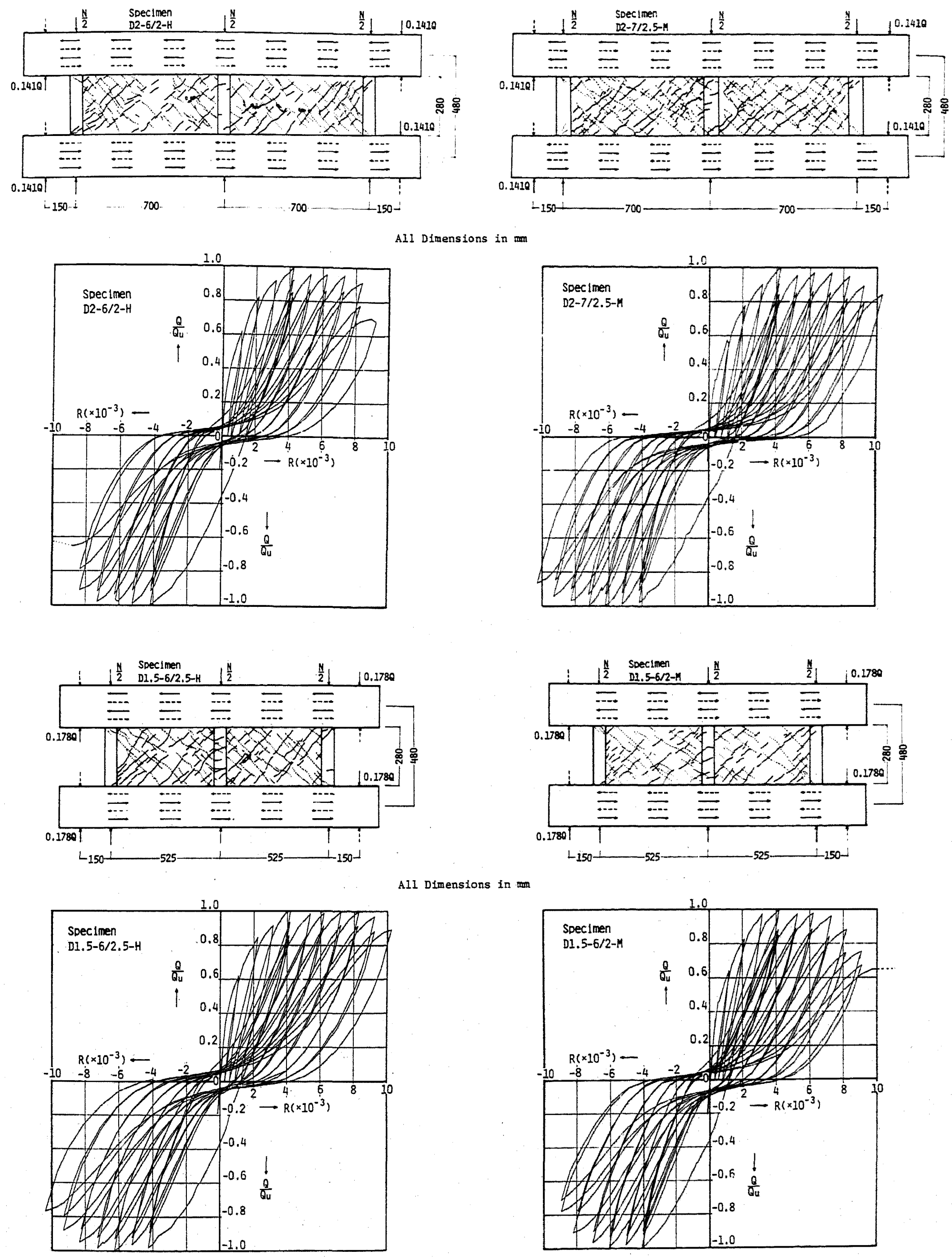
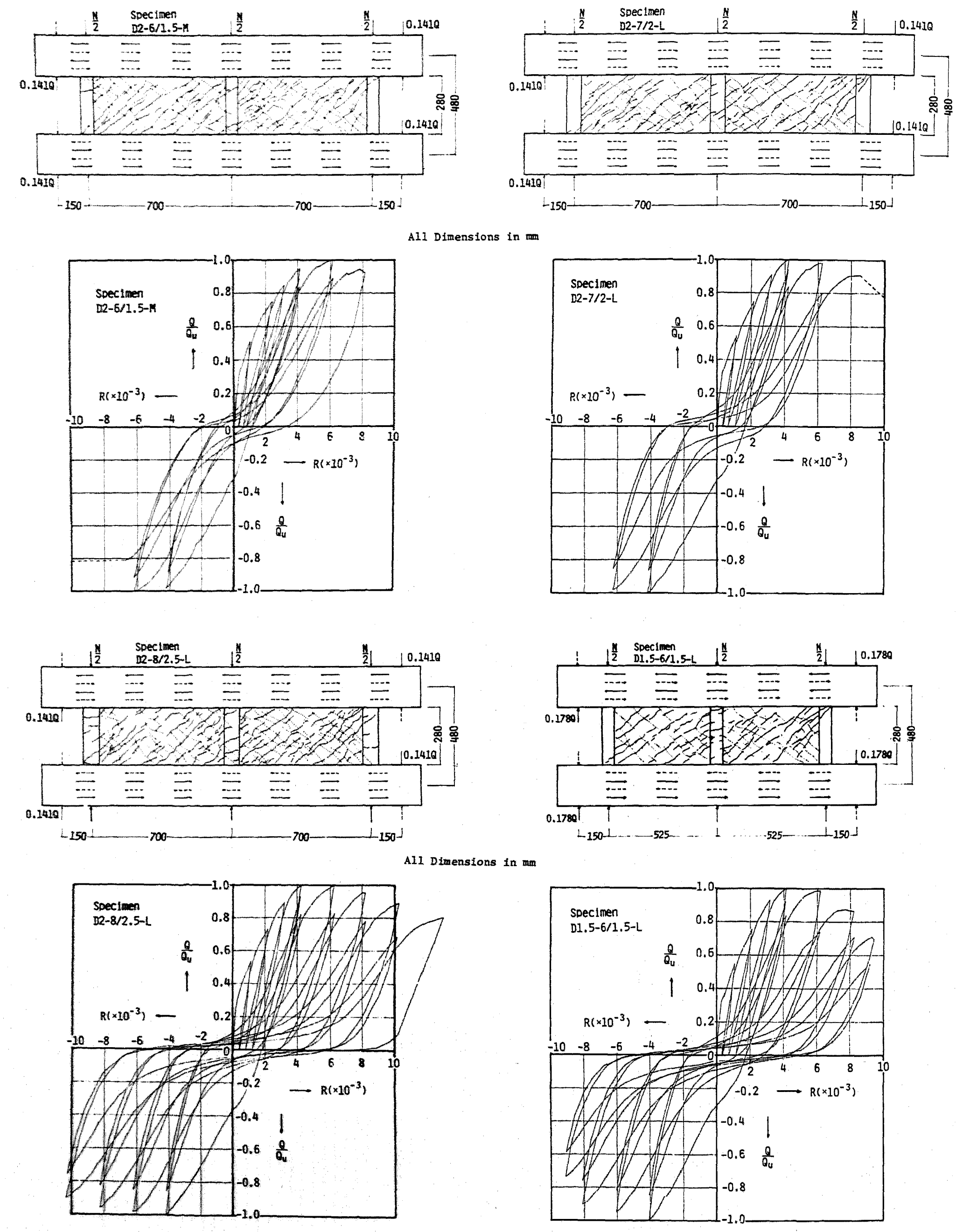


Fig. 6 Test setup



Q = total lateral shear force

Fig. 7a Crack patterns and hysteresis curves



Q = total lateral shear force

Fig. 7b Crack patterns and hysteresis curves