13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 64

THE SIMPLIFIED ELASTO-PLASTIC ANALYSIS MODEL OF REINFORCED CONCRETE FRAMED SHEAR WALLS

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SUMMARY

In this paper, the load-displacement relations of reinforced concrete shear walls are determined by analysis. The simplified elasto-plastic analysis models that the author proposed to analyze the maximum strength are used for the analysis. The models are simple as compared with other elastio-plastic analytic models. In order to determine load-displacement relations using these models, the stress-strain relations of the struts are improved through many experimental results. The struts constitute the wall panels of shear walls. Therefore, the stress-strain relations of struts are associated with the compressive strengths of concrete, and the displacement of shear walls. And those relational equations are determined. By using these relational equations for the simplified elasto-plastic analysis model, the load-displacement relations of shear walls are analyzed accurately.

INTRODUCTION

In the studies to analyze the maximum strength of reinforced concrete framed shear walls (it is abbreviated to shear walls henceforth), it is possible to analyze accurately using the approach by the macro models adapting the limit analysis method. Also in the studies to analyze the load-displacement relations of shear walls, the load-displacement relations to the maximum strength are analyzed accurately. However, the load-displacement relations after the maximum strength are not analyzed exactly. In the structural design of buildings, it is important to evaluate the load-displacement relations of earthquake resisting elements exactly. Since shear walls are resisting the great seismic force particularly, they are very important.

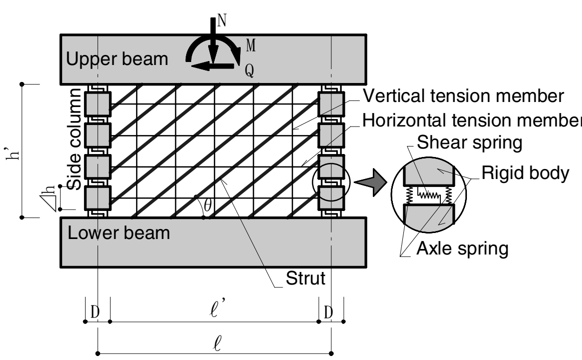

Fig.1 Elastio-plastic analysis model of maximum strength

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ANALYSIS FOR THE LOAD-DISPLACEMENT RELATIONS OF SHEAR WALLS

In order to analyze the maximum strength of shear walls, Fig. 1 shows the simplified elasto-plastic analysis model. The authors showed it in reference [1]. In the reference, the analysis results of the maximum strength of 518 specimens of Japan analyzed by the models are shown. The analysis accuracy is as follows. In examine of the value of the experimental value / analysis value of the maximum strength, the mean value

Table 1 Data of specimens

No.	Specimen	Column						Wall						$r \cdot h'$	N	σ_B	$+Q_{exp}$	$-Q_{exp}$	$+{}_eR_m$	$-{}_eR_m$	$+{}_eR_u$	$-{}_eR_u$	Reference
		b	D	p_g	${}_g\sigma_y$	p_w	${}_w\sigma_y$	ℓ'	h'	t	p_{sv}	p_{sh}	${}_s\sigma_y$										
		mm	mm	%	N/mm^2	%	N/mm^2	mm	mm	mm	%	%	N/mm^2	mm	kN	N/mm^2	kN		$\times 10^{-3}$rad		$\times 10^{-3}$rad		
1	87SWII-1	150	150	1.27	344	0.81	270	600	600	30	0.90	0.89	186	750	111	17	182	188	7.1	6.8	10.5	10.3	[2]
2	87SWII-2	150	150	1.27	344	0.81	270	900	600	32	0.90	0.84	186	750	-96	13	124	143		10.8	*20*	*20*	
3	87SWII-4	150	150	1.27	344	0.81	270	900	600	32	0.90	0.84	186	750	14	13	172	186	8.3	8.1	15.4	14.6	
4	87SWII-5	150	150	1.27	344	0.81	270	1200	600	34	0.80	0.79	186	750	-100	16	156	158	8.8	10.9	*20*	19.8	
5	87SWII-6	150	150	1.27	344	0.81	270	1200	600	31	0.87	0.86	186	750	-38	16	176	206			*20*	*20*	
6	87SWII-7	150	150	1.27	344	0.81	270	1200	600	35	0.77	0.77	186	750	26	15	236	226	8.2	9.0	17.0	16.8	
7	88SW-01	120	120	2.47	370	1.16	468	880	500	31	1.05	1.05	515	700	78	29	257	269	6.4	6.0	9.0	8.4	[3]
8	88SW-02	120	120	1.38	376	1.16	468	880	500	35	0.93	0.93	515	700	78	22	255	236	10.4	9.5	15.3	14.9	
9	88SW-03	120	120	1.87	382	1.16	468	880	500	37	0.88	0.88	515	700	78	25	314	279	6.8	6.5	10.6	10.0	
10	88SW-04	120	120	4.02	341	1.16	468	880	500	34	0.96	0.96	515	700	78	25	373	350	6.1	5.3	8.7	7.0	
11	88SW-05	120	60	4.94	370	1.16	468	940	500	34	0.96	0.96	515	700	78	22	223	228	4.8	4.7	6.7	6.1	
12	88SW-06	120	90	3.30	370	1.16	468	910	500	33	0.98	0.98	515	700	78	21	257	267	6.2	6.2	8.0	8.0	
13	88SW-07	120	150	1.98	370	1.16	468	850	500	34	0.96	0.96	515	700	78	20	298	323	5.9	5.7	9.2	8.9	
14	88SW-08	120	180	1.65	370	1.16	468	820	500	32	1.02	1.02	515	700	78	20	310	300	5.2	4.8	7.5	6.9	
15	88SW-09	120	120	2.47	370	1.16	468	380	500	31	1.05	1.05	515	700	78	21	162	151	9.0	7.4	12.0	10.0	
16	88SW-10	120	120	2.47	370	1.16	468	1130	500	33	0.98	0.98	515	700	78	22	310	357	6.2	5.8	9.2	8.4	
17	88SW-11	120	120	2.47	370	1.16	468	1380	500	33	0.98	0.98	515	700	78	24	396	382	5.6	5.4	7.8	7.0	
18	89SW-01	150	150	6.19	367	0.93	457	600	1200	35	0.93	0.93	490	1400	6	25	416	392	5.6	5.5	7.5	7.5	[4]
19	89SW-06	150	150	4.42	367	0.93	457	600	1200	37	0.88	0.88	490	1400	6	27	204	202	9.9	9.9	13.3	11.3	
20	89SW-09	150	150	2.82	374	0.93	457	600	1200	39	0.83	0.83	490	1400	6	27	145	151	9.3	10.2	16.7	16.2	
21	90SW-01	150	150	4.52	354	1.07	352	1000	800	31	1.05	1.05	299	930	0	31	355	339	4.9	5.2	7.2	7.0	[5]
22	90SW-02	150	150	4.52	354	1.07	352	1000	800	37	0.88	0.88	299	930	0	31	380	361	8.6	8.8	11.4	11.0	
23	90SW-03	150	150	1.27	354	1.07	352	1000	800	32	1.02	1.02	299	930	0	29	199	192	6.9	6.9	11.4	11.3	
24	90SW-04	150	150	1.27	354	1.07	352	1000	800	33	0.98	0.98	299	930	0	29	191	188	8.2	7.5	13.9	12.3	
25	90SW-05	150	150	2.26	354	1.07	352	1000	800	34	0.96	0.96	299	930	0	33	264	260	7.7	6.4	12.7	11.2	
26	90SW-06	150	150	2.26	354	1.07	352	1000	800	34	0.96	0.96	299	930	0	33	243	257	7.3	7.9	11.9	10.8	
27	90SW-07	180	180	1.57	354	0.89	352	800	550	32	1.02	1.02	299	1430	0	35	154	164		9.1	*20*	18.6	
28	90SW-08	180	180	2.46	370	0.89	352	800	550	35	0.93	0.93	299	1140	0	35	296	270	10.1	10.9	17.8	16.2	
29	90SW-09	180	180	1.57	354	0.89	352	800	550	33	0.98	0.98	299	1080	0	35	188	215	12.1	10.0	*20*	17.9	
30	90SW-10	180	180	0.88	354	0.89	352	800	550	33	0.98	0.98	299	999	0	35	153	172	11.7		*20*	*20*	
31	91SW-01	120	120	0.89	377	2.90	377	600	400	26	1.25	1.25	284	510	0	34	115	129	5.3	7.9	*20*	11.6	[6]
32	91SW-02	120	120	1.98	362	2.90	377	600	400	26	1.25	1.25	284	510	0	42	213	199	6.3	6.2	10.3	9.5	
33	91SW-03	120	120	5.52	356	2.90	377	600	400	25	1.29	1.29	284	510	0	39	330	338	4.8	4.0	6.1	4.8	
34	91SW-04	120	120	1.98	362	2.90	377	600	400	26	1.23	1.23	284	510	0	42	220	196	5.0	6.3	12.6	9.9	
35	91SW-05	120	120	3.53	344	2.90	377	600	400	28	1.16	1.16	284	510	0	40	274	261	5.8	6.5	10.5	10.2	
36	91SW-06	120	120	5.52	356	2.90	377	600	400	26	1.24	1.24	284	510	0	43	363	364	5.4	4.7	7.3	6.2	
37	91SW-07	120	120	1.98	362	2.90	377	600	400	25	1.32	1.32	284	510	0	43	189	218	5.9	5.6	10.7	10.6	
38	91SW-08	120	120	3.53	344	2.90	377	600	400	23	1.44	1.44	284	510	0	38	280	270	5.5	5.4	8.1	7.8	
39	91SW-09	120	120	5.52	356	2.90	377	600	400	25	1.33	1.33	284	510	39	45	326	353	4.7	4.9	6.5	6.2	
40	93SW-01	120	120	0.89	387	1.33	387	650	400	27	1.15	1.15	494	510	0	50	133	137		6.5	*20*	17.1	[7]
41	93SW-03	120	120	3.53	388	1.33	387	650	400	30	1.05	1.05	494	510	0	51	295	281	6.8	6.3	10.3	9.9	
42	93SW-04	120	120	3.53	388	1.33	387	650	400	26	1.18	1.18	494	510	0	50	296	283	5.3	4.5	7.3	6.6	
43	93SW-05	120	120	3.53	388	1.78	387	650	400	28	1.11	1.11	494	510	147	52	383	377	6.4	6.0	9.1	8.4	
44	93SW-06	120	120	3.53	388	1.78	387	650	400	26	1.18	1.18	494	510	147	61	364	377	5.0	4.9	7.1	6.8	[9]
45	94SW-01	150	150	1.27	359	1.07	307	600	880	30	1.06	1.06	488	990	0	31	109	106	9.9	11.3	18.6	18.0	[8]
46	94SW-02	150	150	2.26	356	1.07	307	600	880	28	1.12	1.12	488	990	0	34	162	152	7.7	8.5	10.7	10.5	
47	94SW-03	150	150	2.26	356	1.07	307	600	880	28	1.12	1.12	488	990	0	35	159	157	7.9	9.4	11.5	10.9	
48	94SW-04	150	150	3.39	356	1.07	307	600	880	27	1.15	1.15	488	990	196	34	216	220	6.5	6.8	8.0	8.0	
49	94SW-05	150	150	1.90	359	1.07	307	600	880	27	1.17	1.17	488	990	0	55	162	156	9.0	9.1	15.2	13.6	
50	94SW-06	150	150	2.26	356	1.07	307	600	880	27	1.15	1.15	488	990	196	55	250	240	6.8	7.4	9.2	8.5	
51	94SW-07	150	150	2.26	356	1.07	307	600	880	27	1.17	1.17	488	990	196	54	231	224	6.6	7.4	8.1	8.1	
52	95SWH-01	150	150	2.26	819	1.14	404	600	400	34	0.96	0.96	520	510	0	51	482	518	8.2	8.9	10.8	10.4	[9]
53	95SWH-02	150	150	2.26	819	1.14	404	600	400	28	1.17	1.16	520	510	0	50	453	506	7.2	7.1	8.7	8.5	
54	97sw-01	150	150	2.26	345	1.07	377	1000	800	54	0.60	0.60	381	920	0	28	257	263	10.0	10.5	*20*	19.9	[10]
55	97sw-02	150	150	3.39	345	1.07	377	1000	800	52	0.63	0.63	381	920	0	30	329	347	9.3	9.2	15.4	13.6	

[Symbol]

R_u *20* show the values exceeding 20.

b : Width of column
D : Depth of column
p_g : Longitudinal reinforcement ratio
${}_g\sigma_y$: Yield strength of longitudinal reinforcement
p_w : Hoop reinforcement ratio
${}_w\sigma_y$: Yield strength of hoop reinforcement

ℓ' : Length of wall
h' : Height of wall
t :Thickness of wall
p_{sv} : Vertical reinforcement ratio of wall
p_{sh} : Horizontal reinforcement ratio of wall
${}_s\sigma_y$: Yield strength of reinforcement of wall

$r \cdot h'$: Height of point of contraflexure
N : Axial force
σ_B : Unconfined compressive strength of concrete
Qexp: Maximum strength of experiment
${}_eR_m$: Displacement at Qexp
${}_eR_u$: Displacement of 0.8Qexp in downward region

is 1.013, a standard deviation is 0.124, and coefficient of variation is 0.124. The models are more accurate than other ones to analyze the maximum strength of shear walls. And in the models proposed as elasto-plastic analysis models, they are very simple models. Hereafter, the simplified elastio-plastic analytic models analyze the load-displacement relations of shear walls.

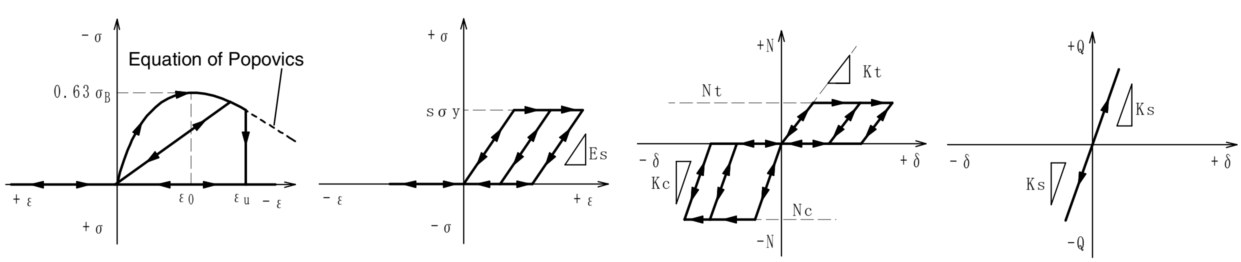
SPECIMENS FOR ANALYSIS

The specimens chosen for analysis are 55 among authors' experiments. Those experiments were executed from 1987 to 1997. Those results are shown in Reference [2] - [10]. The data of all specimens are shown in Table 1. The loading of the displacement increment repeated in the direction of plus and minus controlled by displacement acted on all specimens. Thus, the number of samples is 110 including plus and minus loads. Furthermore, the limit displacement shown in Table 1 is the displacement at the strength falling to 80% of the maximum strength. In Japan, the limit displacement is generally used to estimate the deformation capacity of earthquake resisting elements.

ANALYTIC MODELS

The simplified elastio-plastic analytic models were assumed as follows to analyze the maximum strength of shear walls. Multi-story framed shear walls are the objects to analyze by the models. Therefore, the upper beam and footing beam of shear walls are substituted for rigid bodies in the models. The wall panel is substituted for inclined multiple struts and the tension members of the vertically and horizontally direction. Struts are concrete wall plates, and tension members are reinforcing bars. The details of the stress-strain relations of struts are mentioned later. Tension members are perfect elasto-plasticity, which has the stiffness and strength of reinforcing bars and does not resist the compressive force. Rigid body-spring models substitute for the columns of each side. The springs, which connect rigid bodies, are the combination of axle springs and a shear spring. Axle springs substitute for the stiffness and strength of concrete and longitudinal bars. The stress-strain relations of the concrete are the perfect elasto-plasticity with which secant stiffness is used for elastic stiffness and the compressive strength is used for yield strength. The stress-strain relations of the reinforcing bars are the perfect elasto-plasticity with which yield strength was used for the maximum strength. The sectional area of an axle spring is the half of a column cross section, and the position is the center of gravity of the longitudinal bar in the half of a column depth. However, axle springs have only the properties of reinforcing bars in the state of the tensile force. The substitution length is equal to the length of a rigid element. A shear spring is substituted for the shear stiffness of concrete equal to the length of a rigid element. Struts, tension members, axle springs, and shear springs are connected to the surface of rigid elements. Fig. 2 shows the hysteresis rule of a strut, a tension member, an axle spring, and a shear spring.

As examples of analysis, Fig.3 shows the load-displacement relation curves of an experiment and analysis. Also, the analysis result of other specimens is almost the same. Though analysis values are almost the same as experimental values about the maximum strength, the relation between a load and displacement is not the same. In the stress-strain relations of Fig.3, especially struts control the load-displacement relation of a shear wall. The stress-strain relations of the other elements seldom affect the load-displacement relations of



Strut Tension member Axle spring Shear spring

Fig.2 Hysteresis rule of composition members shear walls. Therefore, in order to determine the load-displacement relations of shear walls, it is necessary to reconsider assumption of the stress-strain relation of the struts currently used in the analysis of the maximum strength. Furthermore, the inclination angle θ of struts shown in Fig.1 is the direction of the principal stress of the wall panels at the maximum strength of shear walls. The direction of the principal stress is determined as follows. When struts change various inclination angles θ of and the maximum strength is analyzed, principal stress is the angle which obtains the greatest maximum strength. However, this approach requires many calculations. The authors proposed the macro models [11] applying a limit analysis method, in order to determine the maximum strength other than the simplified elasto-plastic analysis models. Macro models were constructed based on elasto-plastic analysis models, and it excels in determining the inclination angle θ of struts. Hereafter, the inclination angles θ determined by macro models are used for an angle of inclination of the struts of the simplified elasto-plastic analysis model.

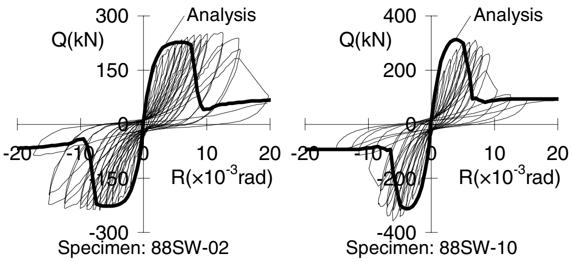


Fig.3 Load-displacement relations of experiment and analysis

STRESS-STRAIN RELATION OF STRUTS

Stress-strain relation of the struts for the maximum strength analysis

The stress σ-strain ε relations of the struts used for the analysis of the maximum strength of shear walls apply the equation of Popovics [12] shown in an equation (1).

$$\sigma=\frac{n\cdot\varepsilon/\varepsilon_0}{n-1+(\varepsilon/\varepsilon_0)^n}\times\sigma_B \tag{1}$$

Where, ε_0: Strain at the maximum stress, n: Experiment constant, σ_B: Unconfined compressive strength.

It is confirmed that the equation of Popovics is flexible including a downward region of stress-strain curves of concrete. Although the equation of Popovics means the character of cylinder concrete, it does not mean the character of the concrete of the wall panels of shear walls. Therefore, when this equation is used to consider the biaxial stress states of wall panels, the concrete compressive strength σ_B is multiplied by 0.63 (Fig.2). 0.63 is an effectiveness factor of the wall panel concrete. It is determined by regression analysis from the experimental data of 36 specimens, which occurred slip failure all over the wall. The authors showed it in reference [13]. Also, it was observed in the experiment that failure of the wall panels after the maximum strength is very brittle. And it is not tough as the equation of Popovics shows. Therefore, the downward region of this equation was corrected. The correction is a simple method of setting the limit strain ε_u, as shown by Fig.2. As mentioned above, the maximum stress was thoroughly compared with experimental results. However, strain ε_0 the maximum stress, the experiment constant n, and the limit strain ε_u were not thoroughly compared.

Effect of the compressive strength of concrete

In order to determine exact ε_0, and n and ε_u, the stress-strain relations of struts are examined by the experimental result. First, it is examined to determine ε_0. The specimens for the examination must be chosen from the specimens that have comparatively small limit displacement for the following reasons. The wall panels of specimens with great limit displacement are acted by many repeated loads until they achieve limit displacement. Because repeated loads deteriorate wall panels, it is thought that the stress-strain relation of the struts of specimens changes with the sizes of limit displacement. Therefore, to examine strain of struts should be targeted at specimens that are acted by limited repeated loads, and they are specimens with limited limit displacement. Then, the limit displacement of the target specimens is provided under 8×10^{-3}rad. In Table 1, the number of the target specimens is 12.

In each specimen, the strain ε_0 at the maximum strength with the highest suitability is determined by comparing with the envelope curves of an experiment and analysis. The envelope curves of the analysis are obtained by the analysis changing the value of ε_0 of the stress-strain curved line of struts. However, there are cases that the maximum strength of specimens is difficult to read correctly by the scattering in an experimental value. To consider such cases, the displacement at the maximum strength of experiments is determined as an average of displacement of the upturned region and downward region at 90% of the maximum strength on an envelope curve. Fig. 4 shows relations of strain ε_0 of the struts at the maximum strength obtained by such an approach and concrete compressive strength σ_B. It is understood that ε_0 decreases with the increase in σ_B. The curved line in Fig. 4 is a regression curve of distribution, and is shown with the following equation.

$$\varepsilon_0 = 0.0065/\sigma_B + 0.0020 \qquad (2)$$

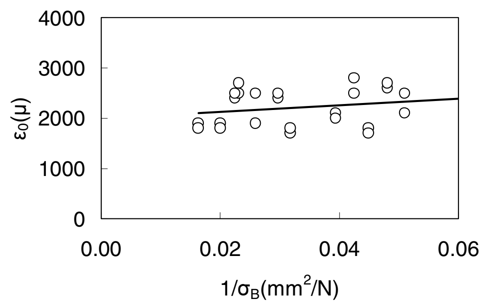


Fig.4 Relation between ε_0 and σ_B of struts

The experiment constant n of the equation of Popovics transmutes the configuration of a stress-strain curved line. When the value of n is 1, the property of perfect rigid plasticity is shown. When the value of n is infinity, the property of perfect elasto-brittleness is shown. In each specimen, the suitable n is determined by comparing with the envelope curves of an experiment and analysis. The envelope curves of analysis are obtained by the analysis changing the value of n of the stress-strain curved line of struts. Here, the values of ε_0 determined for each specimen are used as a value of ε_0. Fig. 5 shows relations of n obtained by such an approach and concrete compressive strength σ_B. It is understood that n increases with σ_B. The curved line in Fig. 5 is a regression curve of distribution, and is shown with the following equation.

$$n = 0.057\sigma_B + 0.79 \qquad (3)$$

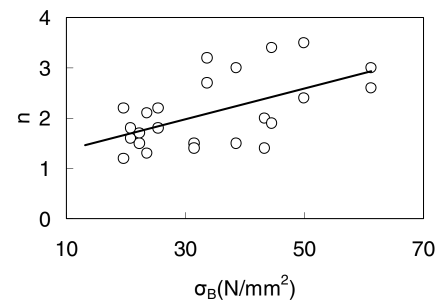


Fig.5 Relation between n and σ_B of struts

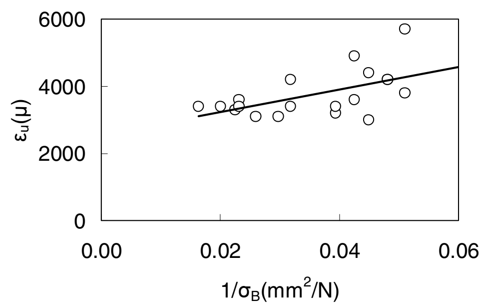


Fig.6 Relation between ε_u and σ_B of struts

The suitable ε_u is also determined by comparing with the envelope curves of an experiment and analysis. The envelope curves of analysis are obtained by the analysis changing ε_u value of the stress-strain curved line of struts. Here, the values of ε_0 and n determined each specimen are used as values of ε_0 and n. Fig. 6 shows the relation between limit strain ε_u obtained by such an approach and concrete compressive strength σ_B. It is understood that ε_u decreases with the increase in σ_B same as ε_0.
The curved line in Fig. 6 is a regression curve of distribution, and is shown with the following equation.

$$\varepsilon_u = 0.043/\sigma_B + 0.0022 \qquad (4)$$

As for the result above, ε_0, n, and ε_u are derived as a function of compressive strength σ_B of concrete, respectively. They are shown in an equation (2), an equation (3), and an equation (4). All the specimens of Table 1 are analyzed using these equations. Fig. 7 shows the relation of the analysis value ${}_cR_u$ and experimental value ${}_eR_u$ of limit displacement. The analysis values of the specimens more than limit deformation 8×10^{-3}rad are less accurate than the analysis values of the specimens used to determine these equations. Those analysis values are larger than experimental values. As stated previously, when limit displacement is large, a wall panel is deteriorated under the effect of repeated loads. Therefore, it needs to consider that the stress-strain curved line of struts changes according to the quantity of displacement.

Effect of degradation by a repeated load

By using the specimens that exceed limit deformation 8×10^{-3}rad, improved ${}_i\varepsilon_u$ is determined by same operations as equation (4). However, the samples of limit displacement exceeding 20×10^{-3}rad are excluded because the maximum displacement observed in the experiment is 20×10^{-3}rad. Fig. 8 shows the relation between ${}_i\varepsilon_u/\varepsilon_u$ obtained by this approach and limit displacement ${}_eR_u$. It is understood that the limit strain of struts decreases with the increase in limit displacement. This is considered to be a degradation phenomenon accompanying the increase in displacement. Thus, reduction ratio ${}_i\varepsilon_u/\varepsilon_u$ of limit strain is associated by displacement R. It is shown with the following equation as a regression curve of distribution of Fig. 8.

$$ {}_i\varepsilon_u/\varepsilon_u = \min(0.094\,{}_eR_u^{-0.48},1) \tag{5}$$

$$ {}_i\varepsilon_u = \min(0.094\,{}_eR_u^{-0.48},1)(0.043/\sigma_B + 0.0022) \tag{6}$$

Fig.9 changes an equation (4) to an equation (6), and shows the result of having reanalyzed all the specimens. Furthermore, Fig.10 shows the relation of the analysis values ${}_cR_m$ and experimental values ${}_eR_m$ of the displacement at the maximum strength. The analysis accuracy of Figs.9 and 10 has the value of an experimental value / analysis value as follows. A mean value is 1.008 from 1.034, and a standard deviation is 0.221, and coefficient of variation is 0.219 from 0.214. Accurate analyses of the load-displacement relations of shear walls are enabled by the simplified elasto-plastic analysis models.

Simple calculation of the inclination angle of struts

In reference [2], authors calculated 518 specimens using the macro model. And the close correlation between the inclination angle θ of a strut and the aspect ratio h'/ℓ of a wall panel became obvious. An equation (7) shows the relation.

$$ {}_s\theta = \tan^{-1}(0.72\, h'/\ell + 0.40) \tag{7}$$

Furthermore, reference [2] shows the equation (7) can be used for the inclination angle θ of the struts of the elasto-plastic analysis for the maximum strength. Fig.11 shows relation of experimental values and the analysis values that used ${}_s\theta$ of an equation (7). Since there is no great difference in the analysis accuracy of Figs.9, 10, and 11, the practicality of an equation (7) is clear.

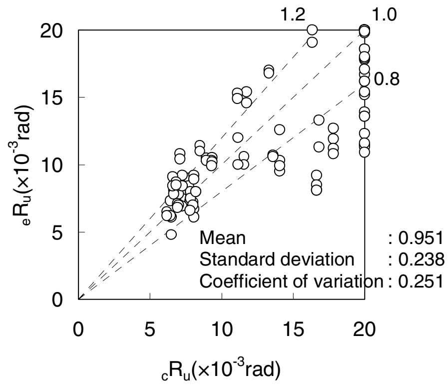


Fig.7 Relation of ${}_eR_u$ and ${}_cR_u$

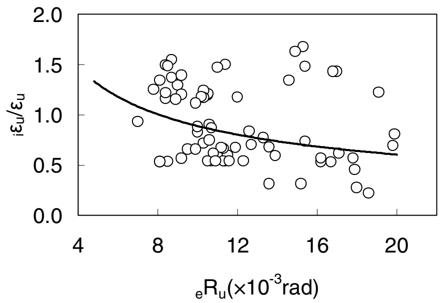


Fig.8 Relation between ${}_i\varepsilon_u/\varepsilon_u$ and ${}_eR_u$

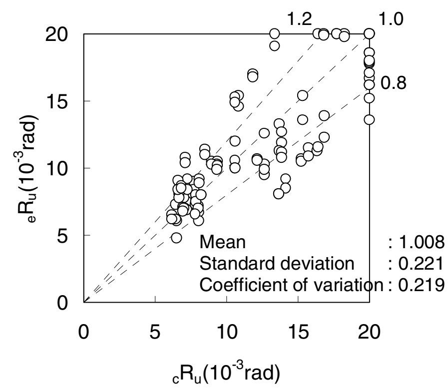


Fig.9 Relation of ${}_eR_u$ and modified ${}_cR_u$

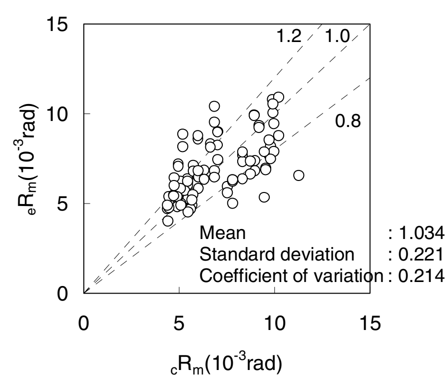
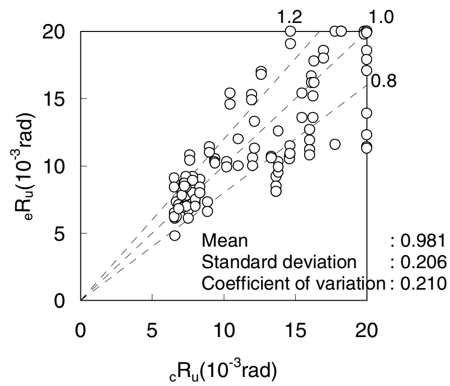
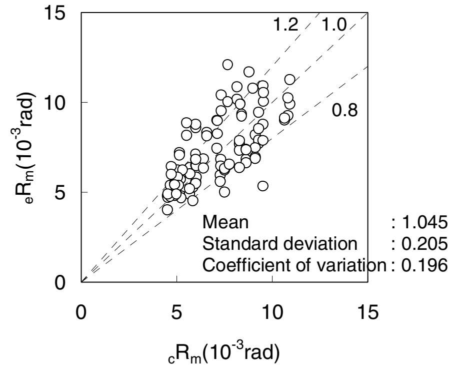


Fig.10 Relation of ${}_eR_m$ and ${}_cR_m$



Relation of $_eR_u$ and $_cR_u$



Relation of $_eR_m$ and $_cR_m$

Fig.11 Relation of experimental values and analysis values using $_s\theta$

CONSIDERATION

The analysis accuracy of the simplified elasto-plastic analysis models was verified using the experimented specimens by the authors. And it was shown sufficient analysis accuracy among these specimens. However, all of these specimens are the experiments of the same loading. Different load doesn't guarantee sufficient accuracy. Especially dynamic load and one-way load may not be unable to calculate an experimental value correctly. However, it is expected that dynamic load and one-way load have larger strength and displacement than static load. The number of repeated loads is more than an experiment of other researchers because experiments of the authors are repeated twice each for the increment of 1×10^{-3}rad to displacement 10×10^{-3}rad. Therefore, it is expected that the limit displacement of the authors' experiment is smaller than other experiments. Thus, the analysis approach determined by the authors' experiments gives safety evaluation to other experiments.

CONCLUSION

Simplified elasto-plastic analysis models for analyzing the load-displacement relations of shear walls were shown. The analysis result of the model was able to catch the load-displacement relations of 55 specimens in sufficient accuracy. The analysis accuracy of the model is controlled by the stress-strain relation of struts, and the feature of struts can be summarized as follows.

1) The strain ε_0 at the maximum strength of a strut decreases with the increase in compressive strength σ_B of concrete. The relational equation is shown as an equation (2).
2) In order to use the equation of Popovics for the stress-strain curved line of struts, the experiment constant n increases with the increase in concrete compressive strength σ_B. The relational equation is shown as an equation (3).
3) In order to use the equation of Popovics for the stress-strain curved line of struts, it is necessary to prepare limit strain ε_u. The value of limit strain $_i\varepsilon_u$ decreases with the increase in concrete compressive strength σ_B and displacement R of a shear wall. The relational equation is shown with an equation (6).

REFERENCES

[1] M.Mochizuki, N.Onozato, M.Takehara, Y.Kanehira: Analytical Accuracies of Maximum Strengths of Reinforced Concrete Framed Shear Walls, Concrete Research and Technology, Japan Concrete Institute, Vol.14, No.2, 2003, pp.11-22

[2] M.Mochizuki, M.Umeda, N.Onozato: On Ductility of Layered Shear Wall Frame System Subjected to Cantilever Typed Loading, Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan 1988.10, pp.567-568

[3] N.Onozato, M.Mochizuki: Effects to column's strength and wall's aspect ratio on maximum strength of multi-story framed shear walls, Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan 1989.10, pp.625-626

[4] T.katagiri, M.Mochizuki, N.Onozato: Strength and deformation of multistory framed shear walls with small aspect ratio, Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan 1990.10, pp.579-580

[5] M.Nakamura, M.Mochizuki, N.Onozato: Verification on Macro Model of Multistory Framed Shear Walls, Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan 1991.9, pp.423-424

[6] K.Tanimoto, M.Mochizuki, N.Onozato, K.Takeuchi: Failure Behaviors of Multistory Framed Shear Walls with Concrete of σ_B=400kgf/cm^2 Level, Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan 1993.9, pp.297-298

[7] T.Fjiwara, M.Mochizuki, N.Onozato, D.Watanabe: Strength and Deformation of Framed Shear Walls with Concrete of σ_B=500kgf/cm^2 Level, Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan 1994.9, pp.259-260

[8] T.Nishizawa, M.Mochizuki, N.Onozato, T.Fujiwara: Failure behaviors and strength estimation of high rise framed shear walls, Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan 1995.8, pp.187-188

[9] M.Takeuchi, M.Mochizuki, M.Takehara: Failure Behavior of Framed Shear Walls with High Strength Concrete and Estimation of Strength and Deformation by Macroscopic Models, Proceeding of the Japan Concrete Institute Vol.19, No.2, 1997, pp.1047-1052

[10] K.Koseki, M.Mochizuki, N.Onozato, H.Goto: Failure Behavior of Framed Shear Walls Aimed to Control Shear Failure of Side Columns, Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan 1998.9, pp.813-814

[11] M.Mochizuki, N.Onozato: Macro Model of multistory Framed Shear Walls, Concrete Research and Technology, Japan Concrete Institute, Vol.1, No.1, 1990, pp.121-132

[12] S.Popovics: Factors Affecting the Elastic Deformations of Concrete, Mechanical Behavior of Materials, Proceeding of the International Conference on Mechanical Behavior of Materials, Vol.Ⅳ, Society of Materials Science, Japan, 1972, pp.172-183

[13] M.Mochizuki, M.Takehara, N.Onozato: Slop Shear Strength of Shear Walls Surrounded with Reinforced and Stiffend Frame, Journal of Struct. Constr. Engng, AIJ, No.416, Oct., 1990, pp.79-89