

Technical Memorandum of PWRI No.1801

REVERSED CYCLIC LOADING TEST OF BRIDGE PIER MODELS

October 1981

Earthquake Engineering Division
Earthquake Disaster Prevention Department
Public Works Research Institute
Ministry of Construction

Preface

Reinforced concrete members decrease their stiffness with the development of cracks, which change response characteristics of the structure. In spite of earthquake excitations larger than design loads, many structures could avoid severe damage because of the non-linear characteristics of their reinforced concrete members. Understandings of the non-linear characteristics of structural members during earthquakes are needed to introduce a rational earthquake-resistant design.

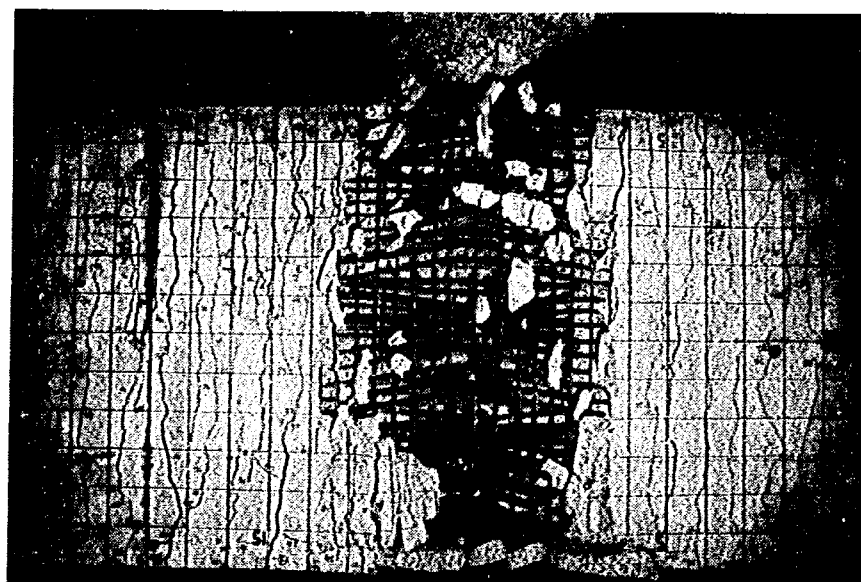
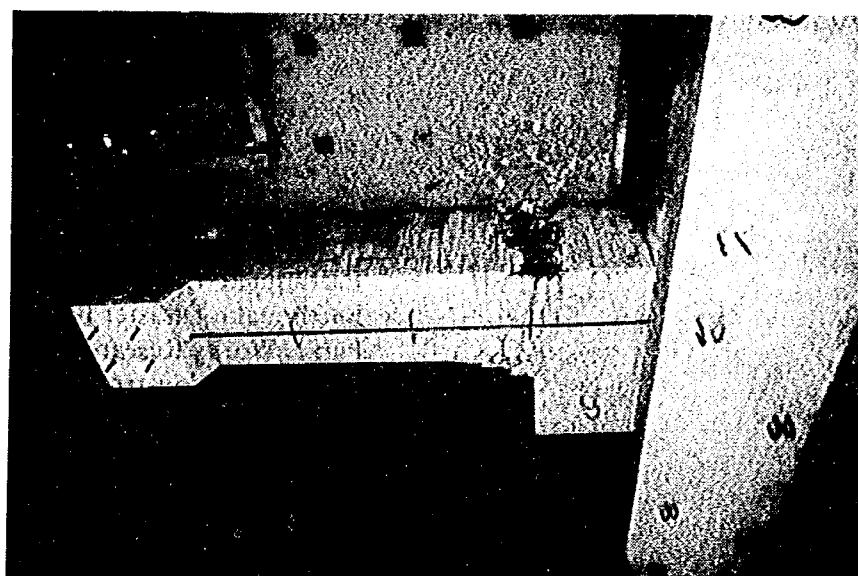
As a basic principle in the earthquake-resistant design, structures should maintain their functions subjected to a moderate earthquake as they experience a few times during their lifetimes, and they may allow some damage but should not collapse subjected to a rare and large earthquake. To design structures on this principle, their dynamic behavior to the ultimate stage should be verified.

This report describes the results of reversed cyclic loading tests of reinforced concrete bridge pier models to the ultimate stage using newly completed earthquake engineering facilities in Tsukuba.

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1. Outlines of the Experiment

1.1 Objects of the Experiment

Reversed cyclic loading tests of the models of a Ban-no-su Bridge pier in Kojima-Sakaide Route of Honshu-Shikoku Bridge were conducted. Ban-no-su Bridge is a double deck bridge for a highway and a railway, and its pier columns are divided into upper and lower columns as shown in Fig. 1-1. For this kind of a structure, detailed studies on dynamic behavior of the upper column subjected to earthquake excitations in the transverse direction are needed in the earthquake-resistant design. For this reason, model loading tests of the upper column of the reinforced concrete pier were conducted.

1.2 Facilities for the Experiment

Model loading tests were conducted using the newly completed earthquake engineering facilities in Tsukuba by Public Works Research Institute, Ministry of Construction. The facilities were developed to study dynamic behavior of structural members and soils during earthquakes. Loading tests are conducted in a test pit (20m x 15m x 5m) using actuators with performances shown in Table 1-1 and Fig. 1-2. The actuator can make excitations up to 125 tons with displacement amplitude up to 125 mm and frequency range of 0 to 30 Hz of arbitrary waves. With the actuators, static and dynamic loading tests of large-scale specimens can be performed (Appendix - 1).

1.3 Outlines of the Experiment

Scales of the models used in the experiment were 1/4 for one specimen (Fig. 1-3) and $1/(4\sqrt{2})$ for six specimens (Fig. 1-4). Conditions of the models were determined as shown in Table 1-2 from below mentioned points of view to study dynamic behavior of the actual bridge column and effects of the assumed structural conditions on aseismic performances of the column.

- (1) 1/4 model and one of $1/(4\sqrt{2})$ models have the same structural conditions as the actual bridge column has.
- (2) To study effects of loading conditions (one-directional loading, reversed loading).
- (3) To study effects of the hunch at the base of the upper column.
- (4) To study effects of the diameter of main reinforcements and the existence of side reinforcements.
- (5) To study dynamic behavior of a SRC (steel frame reinforced concrete) column to compare with that of a RC column.
- (6) To study effects of the studs attached to steel frames at the base of a SRC column.

A lateral load was applied to the column models at the gravity center of the superstructure. The input was controlled by load up to the yield point of reinforcements and by displacement after that. Though the displacement of the loaded point of each column model at the yield point differs relating to its structural conditions, the displacement at the loaded point corresponding to the yield strain ϵ_0 given by eq. (1.1) occurred in outer longitudinal reinforcements of No. 1 model was defined as basic displacement δ_0 ($\approx 18\text{mm}$). For $1/(4\sqrt{2})$ models, input was controlled by load until the displacement at the loaded point became δ_0 and by displacement of $n\delta_0$ ($n=1,2,3, \dots$) after that. For 1/4 model, input was controlled by load until the displacement at the loaded point became $\sqrt{2}\delta_0$ and by displacement of $n(\sqrt{2}\delta_0)$ ($n=1,2,3, \dots$) after that.

$$\epsilon_0 = \frac{\sigma_{sy}}{E_y} = \left. \begin{array}{l} 1,760 \times 10^{-6} \text{ (steel bar)} \\ 1,670 \times 10^{-6} \text{ (steel frame)} \end{array} \right\} \quad (1.1)$$

where, σ_{sy} : yield stress obtained by the tensile test of test pieces (steel bar: 3,700 kg/cm², steel frame: 3,500 kg/cm²)
 E_y : Young's modulus of steel members (2.1×10^6 kg/cm²)

In the case of load-controlled input, load was increased step by step, and load was returned to zero for each step. In the case of displacement-controlled input, the repeat of the same displacement input was three times for No. 1 model (one-directional loading) and ten times for other models (reversed loading). Loading velocity was 2 ~ 3 t/sec in the case of load-controlled input and 0.2 ~ 0.3 cm/sec in the case of displacement-controlled input. It took 2 ~ 20 minutes to observe cracks at the peak load or displacement.

As the height of models was larger than the depth of a test pit, models were set horizontally for the loading test as shown in Fig. 1-5. Outlines of the models are shown in Appendix - 2.

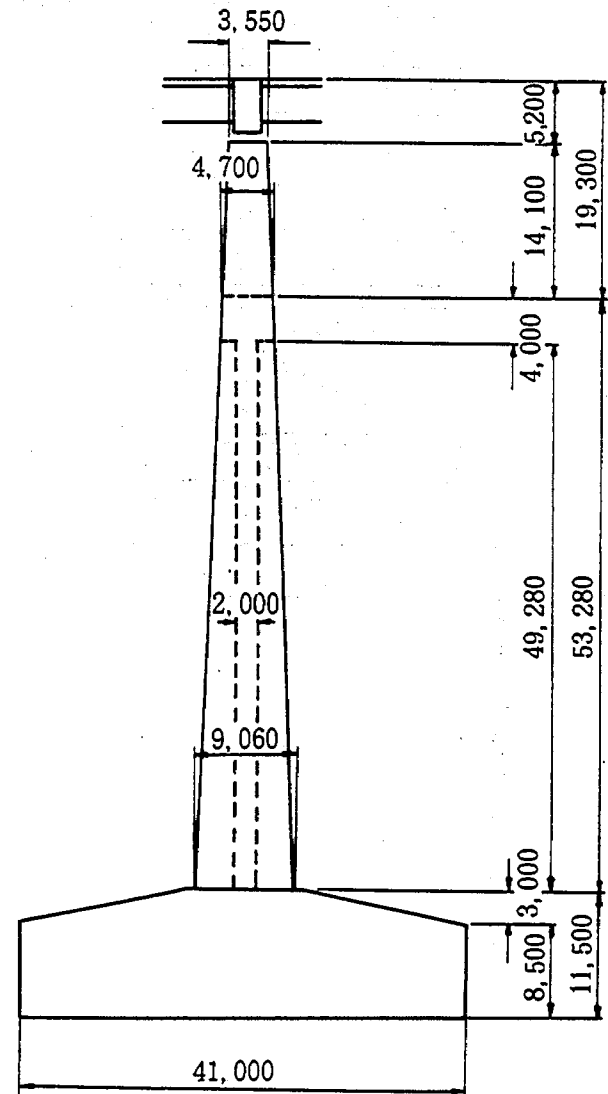
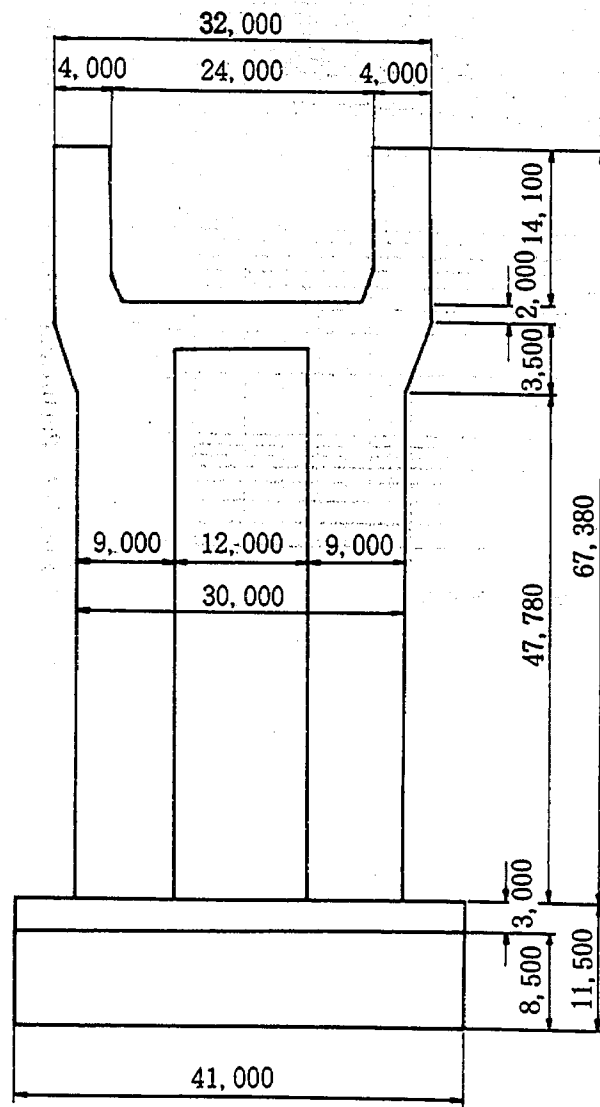


Fig. 1-1 Pier of Ban-no-su Bridge

Unit: mm

Table 1-1 Main Specifications of the Test Apparatus

Item	Performances
(1) Maximum Excitation	± 125 t/1 Actuator
(2) Maximum Stroke	± 125 mm
(3) Maximum Speed	± 1 m/sec.
(4) Frequency Range	DC $\sim$ 30 Hz
(5) Excitation Motions	Sinusoidal Waves Triangular Waves Rectangular Waves Ramp Waves Earthquake Motions
(6) Maximum Excitation Time	10 sec. (At Maximum Velocity)
(7) Control of Input Motions	Analog Control and Digital Control
(8) Control Item	Acceleration, Velocity or Displacement
(9) Exciter	Electro-hydraulic Servo System
(10) Hydraulic Power Supply	Main Pump 300 ℓ /min $\times$ 2 with 200 ℓ /10 sec $\times$ 6 Accumulators (3 Accumulators /1 Actuator) $\times$ 2

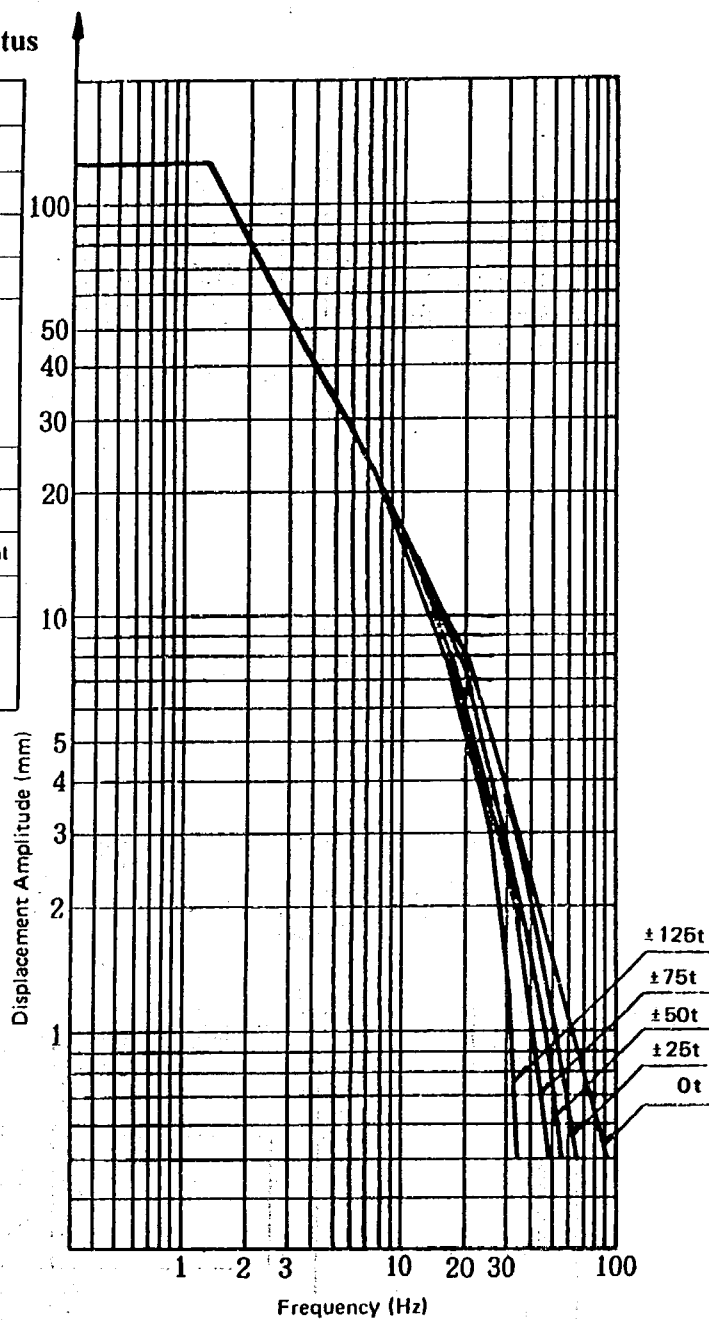


Fig. 1-2 Performance Curve of the Actuator

Table 1-2 Test Cases

Model	Test Conditions
No. 1 $1/(4\sqrt{2})$ one-directional loading model	one-directional loading, other conditions are the same as basic model.
No. 2 $1/(4\sqrt{2})$ without hunch model	without hunch, other conditions are the same as basic model.
No. 3 $1/(4\sqrt{2})$ basic model	same structural conditions as the actual bridge (main reinforcement—D13, reversed cyclic loading)
No. 4 $1/(4\sqrt{2})$ large-diameter reinforcement, without side reinforcement	large-diameter reinforcement and without side reinforcement (main reinforcement—D29, reversed cyclic loading)
No. 5 $1/(4\sqrt{2})$ SRC-without stud model	SRC without stud (CT shape steel + D13 reinforcement, reversed cyclic loading)
No. 6 $1/(4\sqrt{2})$ SRC-with stud model	SRC with stud (CT shape steel + D13 reinforcement, reversed cyclic loading)
No. 7 $1/4$ model	same structural conditions as the actual bridge (main reinforcement—D13, reversed cyclic loading)

Note: Axial load of 118t corresponding to the dead load of the superstructure was applied to $1/4$ model, but no axial load was applied to $1/(4\sqrt{2})$ models.

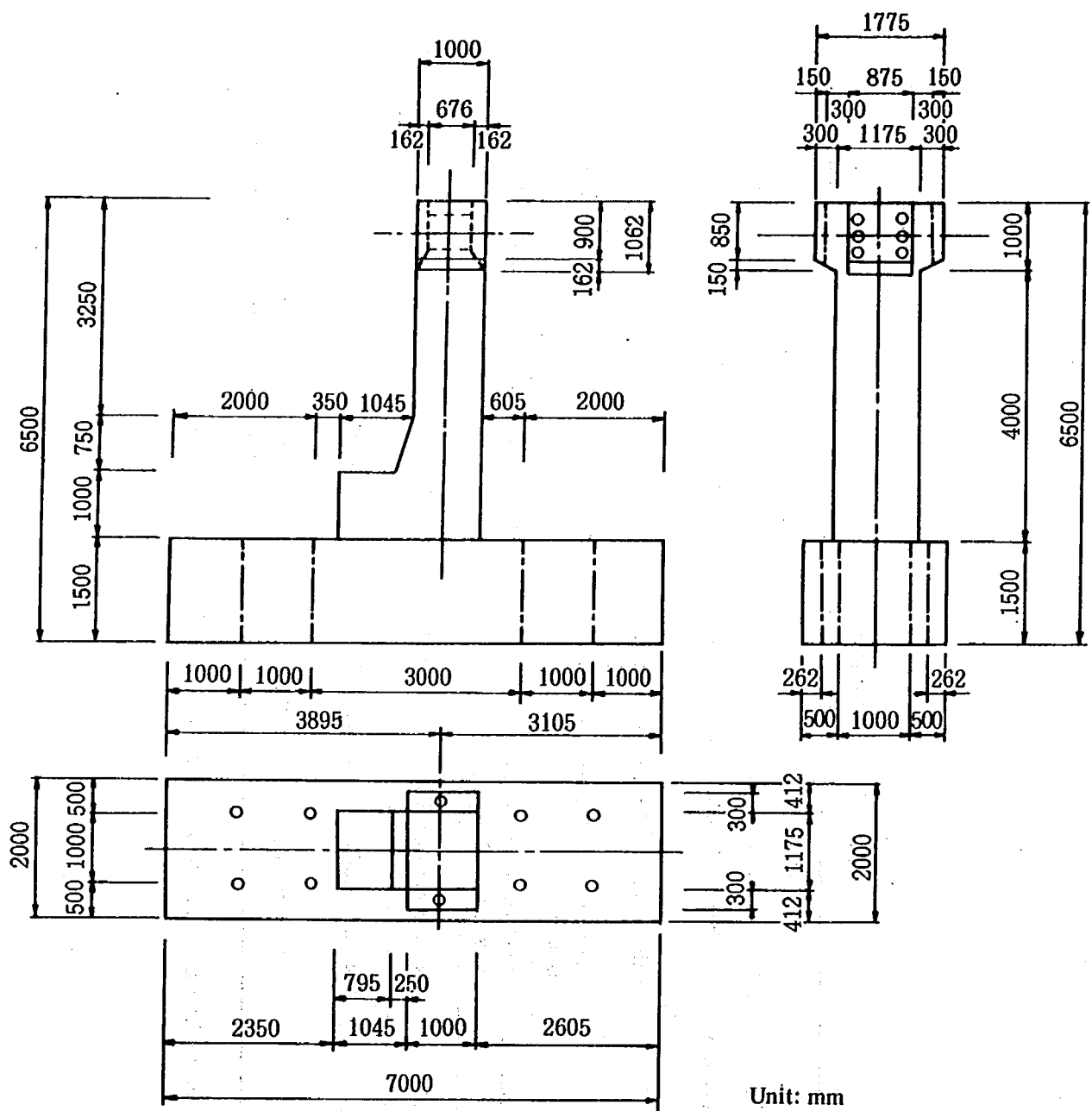
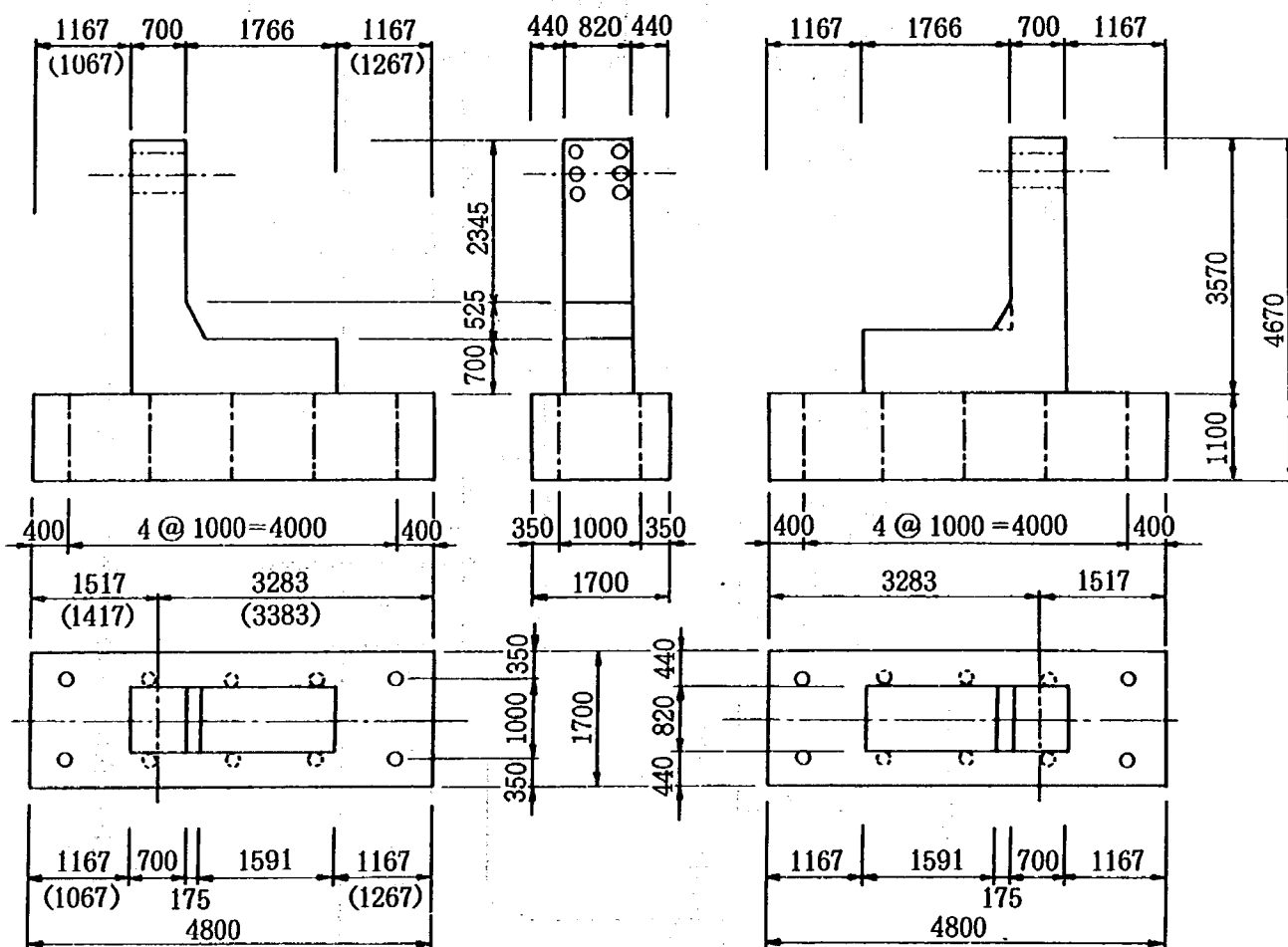


Fig. 1-3 Dimensions of 1/4 Model

Type A

Type B

() shows dimensions of one-directional loading model



Unit: mm

Fig. 1-4 Dimensions of 1/(4√2) Model

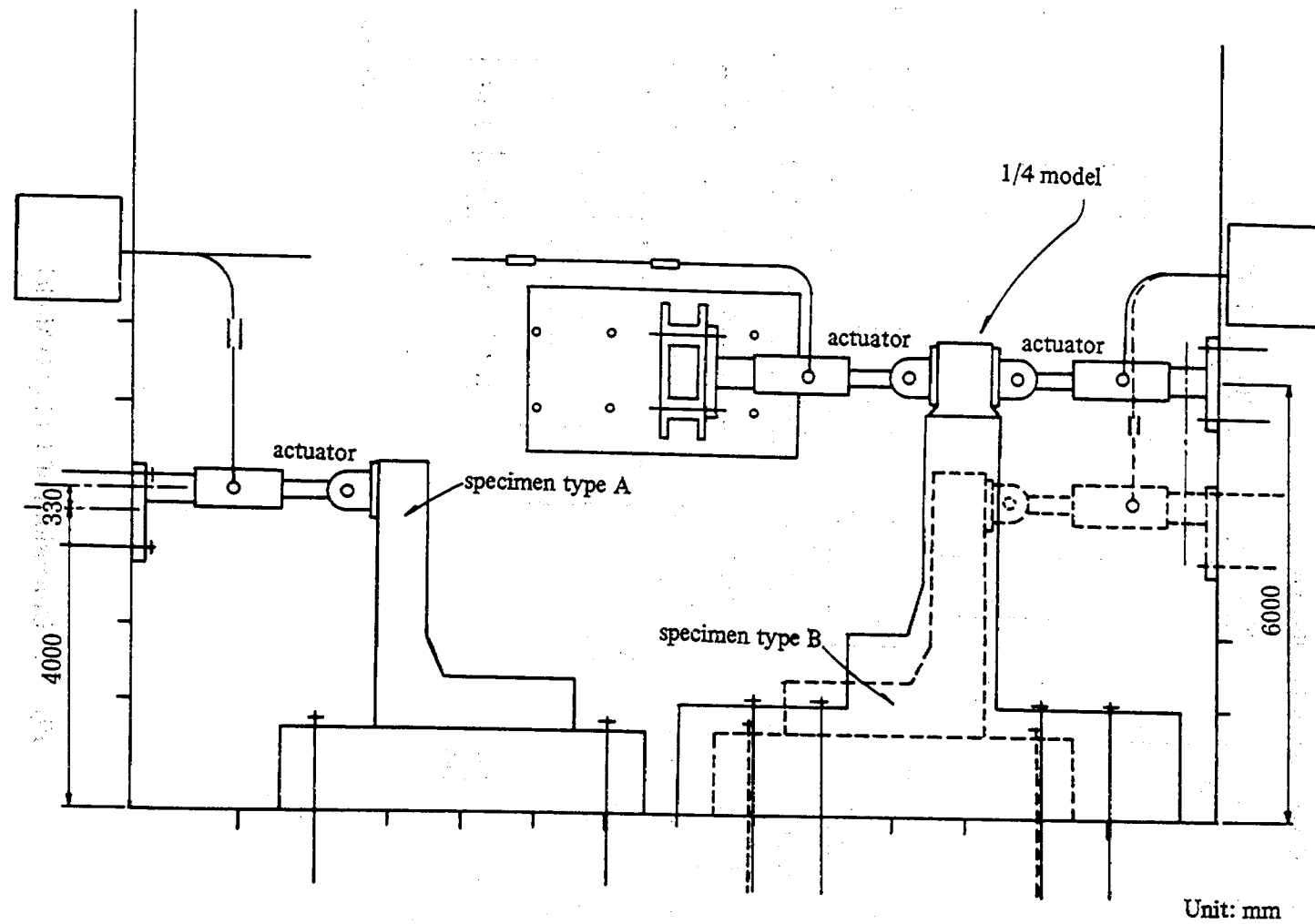


Fig. 1-5 Setting of Specimens and Actuators

2. Results of the experiment

Load-displacement curves obtained by the loading tests are shown in Appendix — 3. Yield load, ultimate load, yield displacement, ultimate displacement, and potential energy of each model obtained by the tests are shown in Tables 2-1 and 2-2. Yield point is defined here as the point that the strain of the longitudinal reinforcements becomes $2,050 \times 10^{-6}$. This strain of $2,050 \times 10^{-6}$ is the yield strain of the test piece which was planned to put a strain gauge as similar to the longitudinal reinforcement. In the analysis of test results, yield load and yield displacement were determined corresponding to the strain of outer longitudinal reinforcements of $2,050 \times 10^{-5}$.

In the load-displacement curves, load increased even after longitudinal reinforcements had yielded, and it became maximum (ultimate load) when input displacement was $2 \sim 3 \delta_0$. The ultimate load of the basic model (No. 3 model) was $20 \sim 30\%$ larger than the yield load. After that, load decreased to the level of yield load when input displacement was $4 \sim 5 \delta_0$ (ultimate displacement). When loading was repeated with the same displacement, load decreased remarkably by the first loading cycle and did gradually by the following loading cycles.

Final destruction modes of each model are shown in Fig. 2-1. In the case of one-directional loading (No. 1 model), cracks occurred at the base of the column at first, but the specimen finally shear-failed by the crack developed diagonally from the hunch. The specimen with large-diameter longitudinal reinforcements and without side reinforcements (No. 4 model) also shear-failed. Other models bending-failed at the base of the column.

Load-displacement envelope curves until longitudinal reinforcements yielded are shown in Fig. 2-2. The stiffness at the yield point decreased to $1/3 \sim 1/4$ of the initial stiffness. The yield load of the specimen without hunch (No. 2 model) and that of the specimen with large-diameter longitudinal reinforcements and without side reinforcements (No. 4 model) were $10 \sim 20\%$ smaller than that of the basic model (No. 3 model). When an axial load was applied (No. 7 model), the yield load became about 20% larger than that of the basic model.

Load-displacement envelope curves of the first loading cycle ($n=1$) and the tenth loading cycle ($n=10$) until the ultimate displacement are shown in Figs. 2-3 and 2-4. The load-displacement curves until the yield point in the figures are envelope curves for $n=1$ curves and skeleton curves for $n=10$ curves. Measured rotation angles at the base of the column are shown in Fig. 2-5. Comparisons of aseismic performances of the models are shown in Tables 2-3, 2-4 and Figs. 2-6 $\sim$ 2-10.

The yield load and the ultimate load of the one-directional loading model (No. 1 model) and the reversed-loading basic model (No. 3 model) were about the same. But the ultimate displacement of No. 3 model was 40% of that of No. 1 model. By reversed loading, displacement ductility ratio (ultimate displacement δ_u /yield displacement δ_y) decreased to $1/2 \sim 1/3$ of that by one-directional loading (Fig. 2-6).

As for the effects of the hunch at the base of the upper column, the yield load and the ultimate load of the model with a hunch (No. 3 model) were $20 \sim 30\%$ larger than those of the model without a hunch (No. 2 model). The ultimate displacement of the both models were about the same. The displacement ductility ratio of the both models in the hunch-compression direction were about the same, too. But the displacement ductility ratio of the model with a hunch was $10 \sim 20\%$ smaller than that of the model without a hunch in the hunch-tension direction because of larger yield displacement of the former model than the latter model (Fig. 2-7).

Comparing the test results of the model with large-diameter longitudinal reinforcements and without side reinforcements (No. 4 model) and those of the basic model (No. 3 model), the former shear-failed at the upper part of the column, while the latter bending-failed at the base of the column. No. 4 model was about 10% smaller in yield load, about 20% smaller in ultimate load, and about 40% smaller in displacement ductility ratio than No. 3 model.

As for the effects of an axial load, the model with an axial load corresponding to the dead load of the superstructure (No. 7 model) was about 20% larger in yield load, about 10% larger in ultimate load, but 10 ~ 20% smaller in displacement ductility ratio than the model without an axial load (No. 3 model) (Fig. 2-8).

Comparing the test results of the SRC model (No. 6 model) and the RC model (No. 3 model), the former was 10 ~ 20% larger in yield and ultimate loads than the latter. This is considered to be because that the actual strength of a steel frame was larger than the nominal strength, while the SRC model was designed based on the nominal strength. The SRC model and the RC model can be said to have had similar aseismic performances after all (Fig. 2-9).

In the case of SRC models, the model with studs on steel frames below the base of the column (No. 6 model) and the model without studs (No. 5 model) had similar aseismic performances (Fig. 2-10). This is because of small influences of the pulled-out of steel frames.

Measured crack width at the time of the loading corresponding to the design load are shown in Fig. 2-5. Estimated residual crack width of the actual bridge from the results of the basic model (No. 3 model) and the 1/4 model (No. 7 model) were smaller than 0.2 mm and than 0.15 mm respectively. A crack smaller than 0.2 mm in width is considered to be harmless for durability.

Estimated aseismic performances of the prototype upper column from the results of the experiment are shown in Fig. 2-6.

Table 2-1 Results of the Experiment (n=1 P- δ envelope curve)

loading direction model item	hunch-tension							hunch-compression						
	No. 1 one- directional loading	No. 2 without hunch	No. 3 basic model	large- diameter- reinforce- ment, without side re- inforcement	No. 5 SRC without stud	No. 6 SRC with stud	No. 7 1/4 model	No. 1 one- directional loading	No. 2 without hunch	No. 3 basic model	large- diameter- reinforce- ment, without side re- inforcement	No. 5 SRC without stud	No. 6 SRC with stud	No. 7 1/4 model
yield load P_y (t)	—	-55	-75	-69	-72	-79	-175	75	61	72	66	80	85	177
ultimate load P_u (t)	—	-69	-89	-73	-94	-96	-187	92	73	89	68	100	101	200
P_u/P_y	—	1.25	1.19	1.06	1.31	1.22	1.07	1.23	1.20	1.24	1.03	1.25	1.19	1.13
yield displacement δ_y (cm)	—	-2.3	-2.6	-2.6	-2.2	-2.3	-4.1	2.0	2.1	2.1	2.3	1.9	2.2	3.2
ultimate displacement δ_u (cm)	—	-7.8	-7.5	-4.5	-8.5	-8.3	-9.6	19.6	7.6	7.9	4.9	8.5	7.6	10.9
δ_u/δ_y	—	3.4	2.9	1.7	3.9	3.6	2.3	9.8	3.6	3.8	2.1	4.5	3.5	3.4
potential energy E_u (t·cm)	—	354	442	201	529	540	1,190	1,610	366	407	210	579	505	1,430
δ_o/δ_y	—	0.78	0.69	0.69	0.82	0.78	0.63	0.90	0.86	0.86	0.78	0.95	0.82	0.81

notes: P_y : load applied at the top of the column at the time of yield of outer reinforcements
 P_u : maximum load in n=1 P- δ envelope curve
 δ_y : displacement at the top of the column corresponding to P_y
 δ_u : displacement corresponding to P_y after δ_y in n=1 P- δ envelope curve
 E_u : integration of P- δ envelope curve up to the ultimate displacement.

Table 2-2 Results of the Experiment (n=10 envelope curve)

item	loading direction model	hunch-tension						hunch-compression							
		No. 1 one-directional loading	No. 2 without hunch	No. 3 basic model	Large-diameter reinforcement, without side reinforcement	No. 5 SRC without stud	No. 6 SRC with stud	No. 7 1/4 model	No. 1 one-directional loading	No. 2 without hunch	No. 3 basic model	Large-diameter reinforcement, without side reinforcement	No. 5 SRC without stud	No. 6 SRC with stud	No. 7 1/4 model
yield load P_y (t) $n=1$		—	—55	—75	—69	—72	—79	—175	75	61	72	66	80	85	177
ultimate load P_u (t) $n=10$		—	—61	—80	—60	—86	—85	—158	—	65	79	58	90	89	168
$\frac{P_u (n=10)}{P_y (n=1)}$		—	1.11	1.07	(0.87)	1.19	1.08	(0.90)	—	1.07	1.10	(0.88)	1.13	1.05	(0.95)
yield displacement δ_y (cm) $n=1$		—	—2.3	—2.6	—2.6	—2.2	—2.3	—4.1	2.0	2.1	2.1	2.3	1.9	2.2	3.2
ultimate displacement δ_u (cm) $n=10$		—	—5.3	—4.4	—	—5.3	—4.7	—	17.3	4.4	4.8	—	5.3	4.4	—
$\frac{\delta_u (n=10)}{\delta_y (n=1)}$		—	2.3	1.7	—	2.4	2.0	—	8.7	2.1	2.3	—	2.8	2.0	—

notes: P_y : load applied at the top of the column at the time of yield of outer reinforcements
 P_u : maximum load in $n=10$ P- δ envelope curve
 δ_y : displacement at the top of the column corresponding to P_y
 δ_u : displacement corresponding to P_y after δ_y in $n=10$ P- δ envelope curve

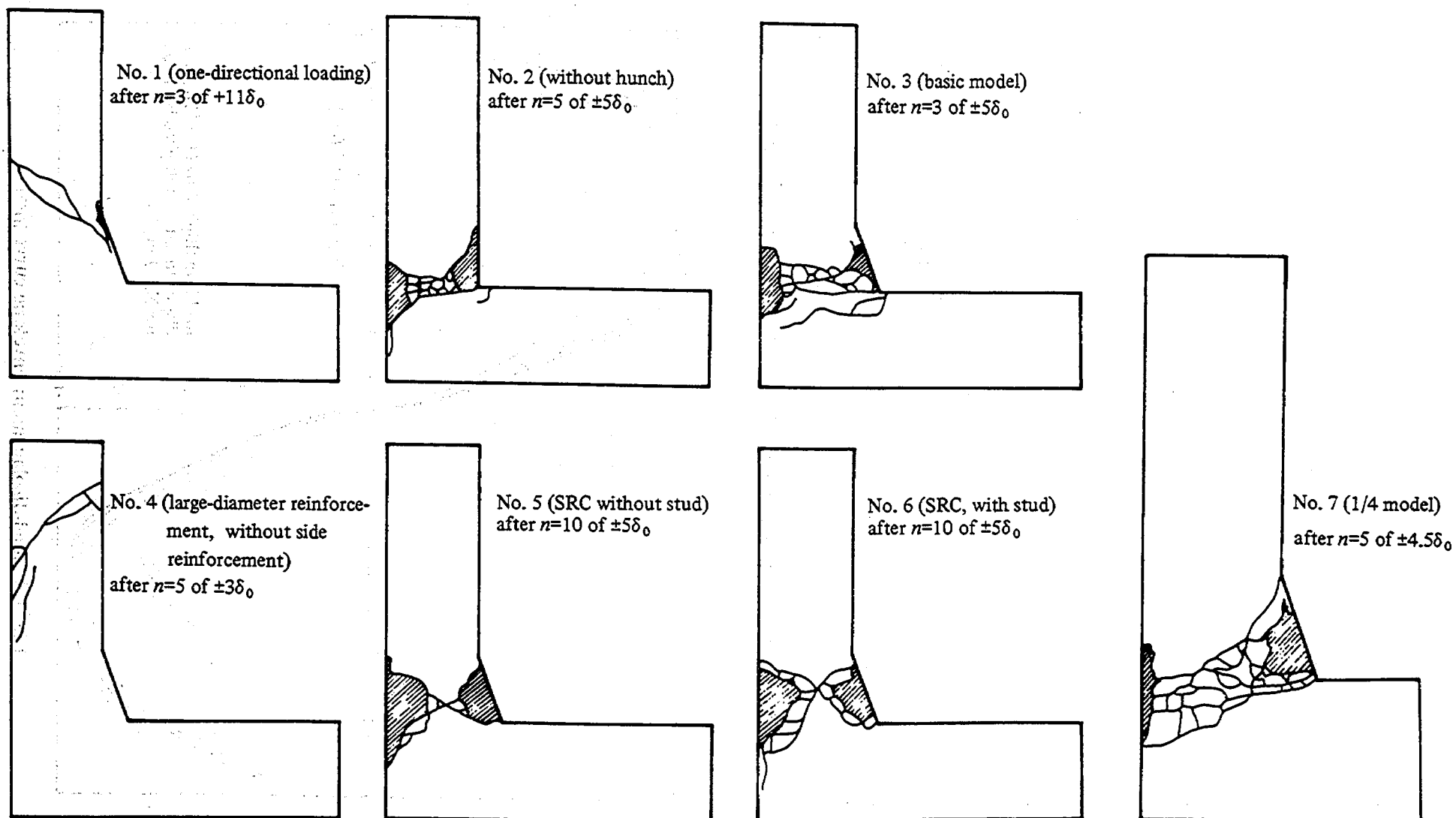


Fig. 2-1 Final Destruction Modes of Specimens

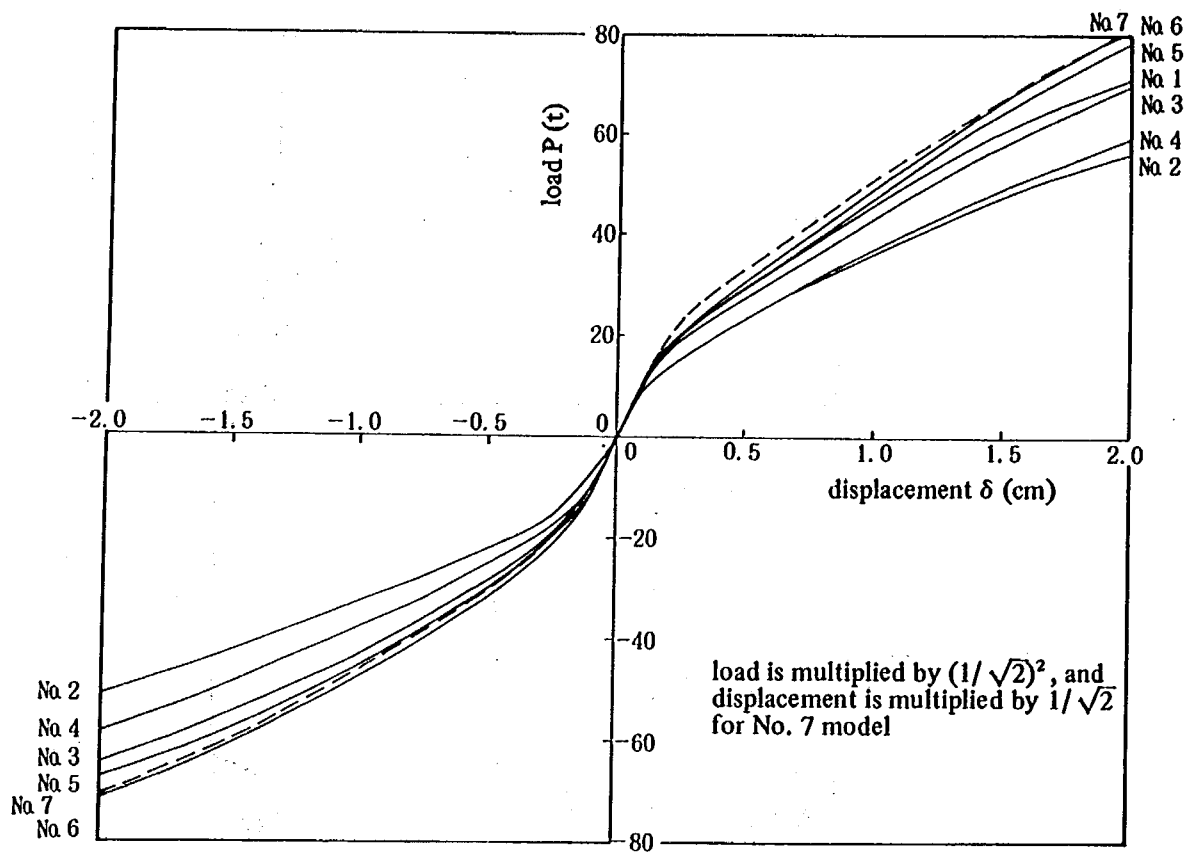


Fig. 2-2 load-displacement envelope curve (load-controlled part)

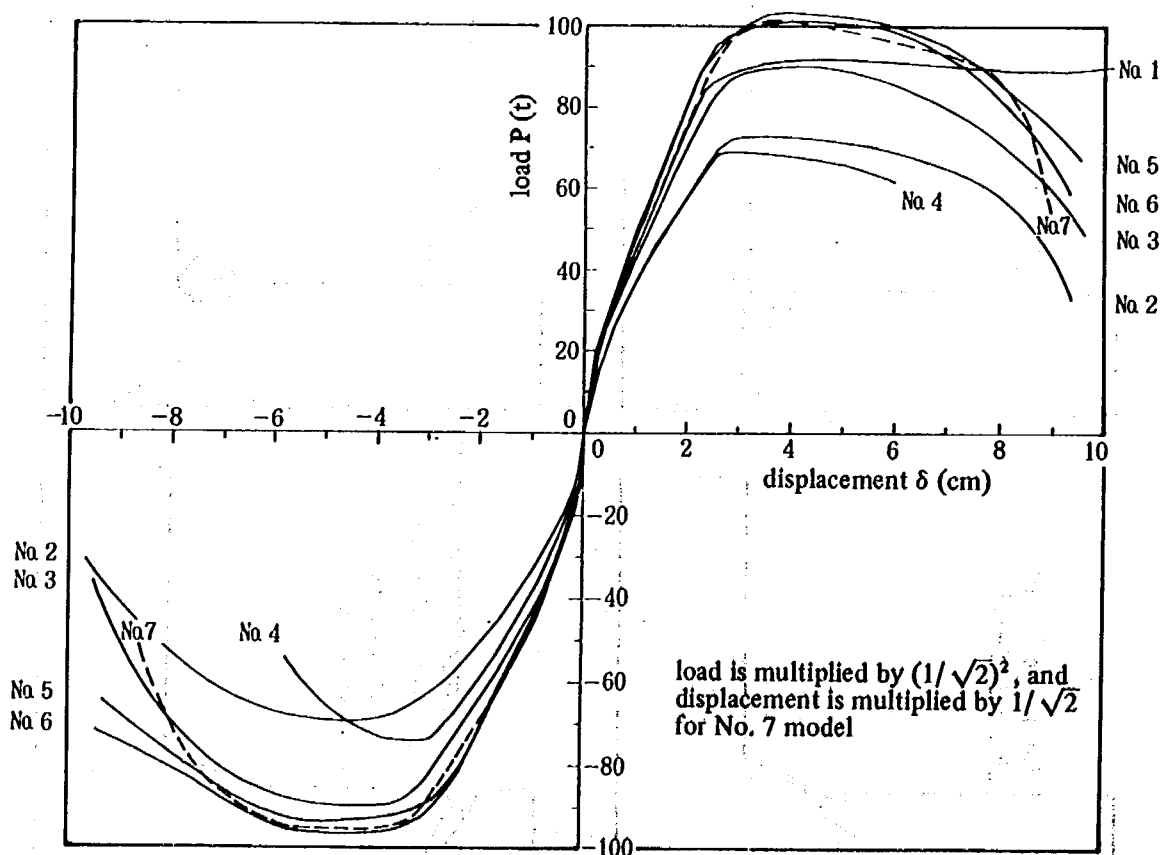


Fig. 2-3 load-displacement envelope curve ($n=1$)

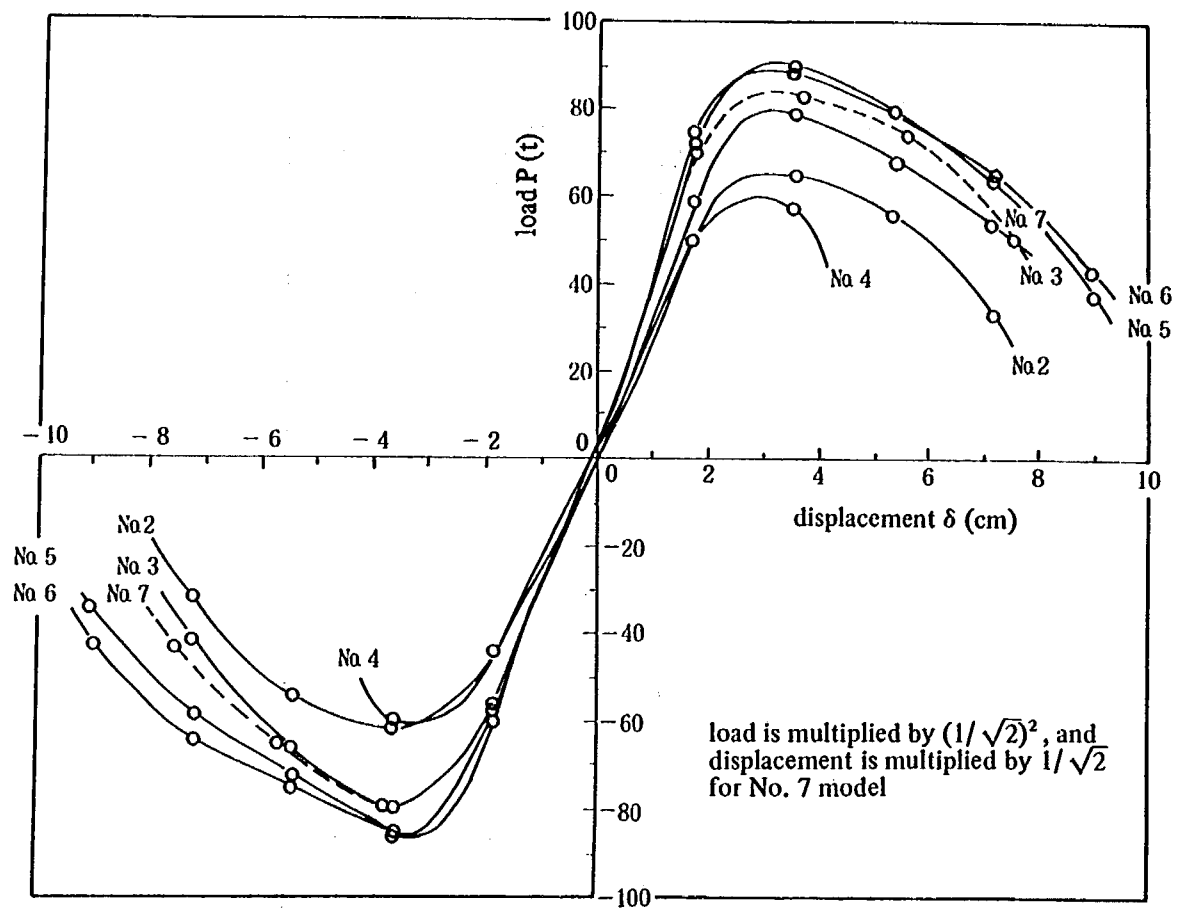


Fig. 2-4 load-displacement envelope curve ($n=10$)

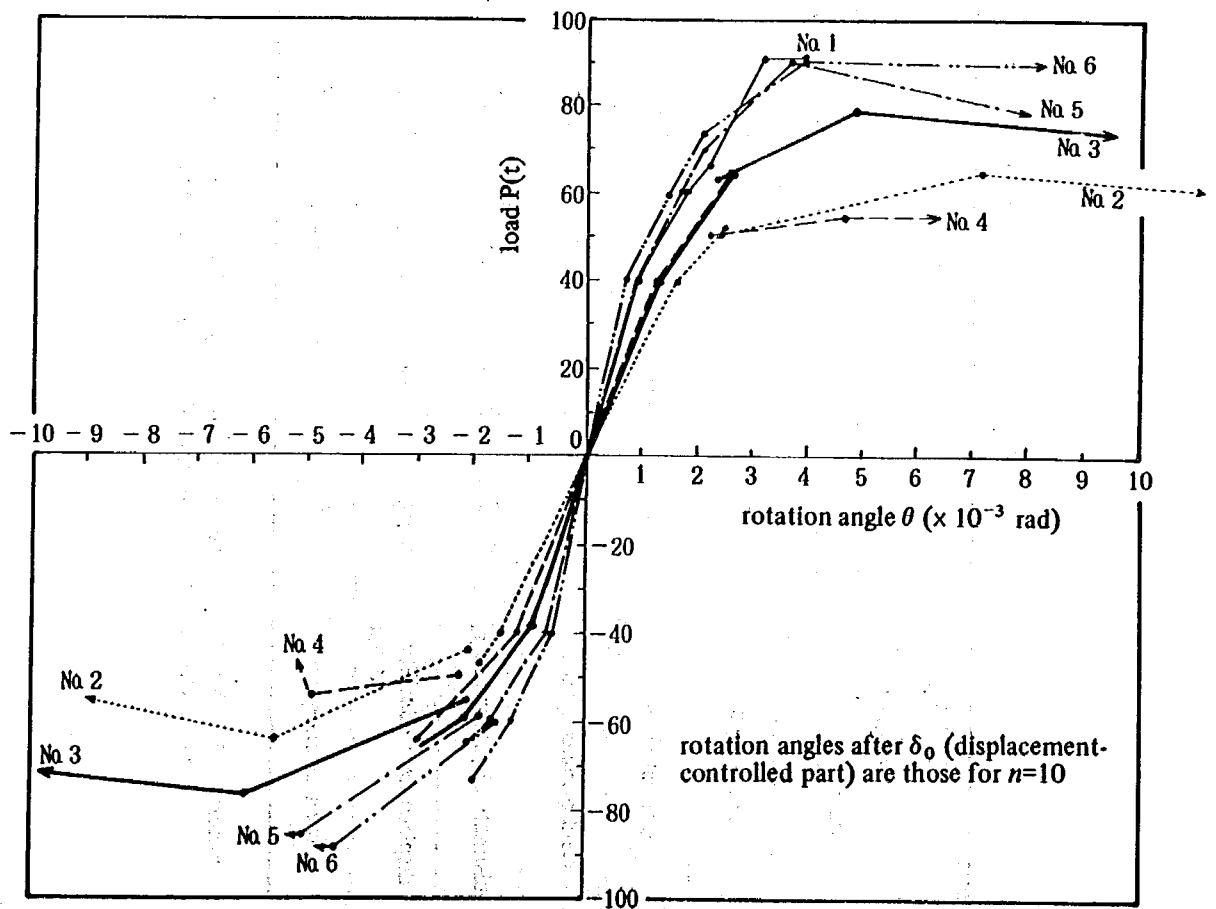


Fig. 2-5 Measured rotation angle at the base of the upper column

Table 2-3 Comparison of Aseismic Performances ($n=1$ P- δ envelope curve)

item	loading direction model	hunch-tension					hunch-compression					
		$\frac{Na\ 3}{Na\ 2}$	$\frac{Na\ 4\ *}{Na\ 3}$	$\frac{Na\ 6}{Na\ 5}$	$\frac{Na\ 6}{Na\ 3}$	$\frac{Na\ 7\ **}{Na\ 3}$	$\frac{Na\ 3}{Na\ 1}$	$\frac{Na\ 3}{Na\ 2}$	$\frac{Na\ 4\ *}{Na\ 3}$	$\frac{Na\ 6}{Na\ 5}$	$\frac{Na\ 6}{Na\ 3}$	$\frac{Na\ 7\ **}{Na\ 3}$
yield load P_y		1.36	0.92	1.10	1.05	1.17	0.96	1.18	0.91	1.06	1.18	1.23
ultimate load P_u		1.29	0.82	1.02	1.08	1.05	0.97	1.22	0.76	1.01	1.13	1.12
yield displacement δ_y		1.13	1.00	1.05	0.88	1.12	1.05	1.00	1.10	1.16	1.05	1.08
ultimate displacement δ_u		0.96	0.60	0.98	1.11	0.91	0.40	1.04	0.62	0.89	0.96	0.98
displacement ductility ratio μ		0.85	0.59	0.92	1.24	0.79	0.39	1.05	0.55	0.78	0.92	0.89
potential energy E_u		1.25	0.45	1.02	1.22	0.95	0.25	1.11	0.52	0.87	1.24	1.24

notes: * No. 4 model shear-failed at the upper part of the column.
 ** Values of No. 7 model ($1/4$ model) are converted into those of a $1/(4\sqrt{2})$ model by similitude.

Table 2-4 Comparison of Aseismic Performances ($n=10$ P- δ envelope curve)

item	loading direction model	hunch-tension					hunch-compression					
		$\frac{\text{No. 3}}{\text{No. 2}}$	$\frac{\text{No. 4}^*}{\text{No. 3}}$	$\frac{\text{No. 6}}{\text{No. 5}}$	$\frac{\text{No. 6}}{\text{No. 3}}$	$\frac{\text{No. 7}^{**}}{\text{No. 3}}$	$\frac{\text{No. 3}}{\text{No. 1}}$	$\frac{\text{No. 3}}{\text{No. 2}}$	$\frac{\text{No. 4}^*}{\text{No. 3}}$	$\frac{\text{No. 6}}{\text{No. 5}}$	$\frac{\text{No. 6}}{\text{No. 3}}$	$\frac{\text{No. 7}^{**}}{\text{No. 3}}$
yield load P_y		1.36	0.92	1.10	1.05	1.17	—	1.18	0.92	1.06	1.18	1.23
ultimate load P_u		1.31	0.75	0.99	1.06	0.99	—	1.21	0.73	0.99	1.13	1.06
yield displacement δ_y		1.13	1.00	1.05	0.89	1.12	—	1.00	1.10	1.15	1.05	1.08
ultimate displacement δ_u		0.83	—	0.89	1.07	—	—	1.09	—	0.83	0.92	—
displacement ductility ratio μ		0.74	—	0.83	1.18	—	—	1.09	—	0.71	0.87	—

notes: * No. 4 model shear-failed at the upper part of the column.
 ** Values of No. 7 model (1/4 model) are converted into those of a $1/(4\sqrt{2})$ model by similitude.

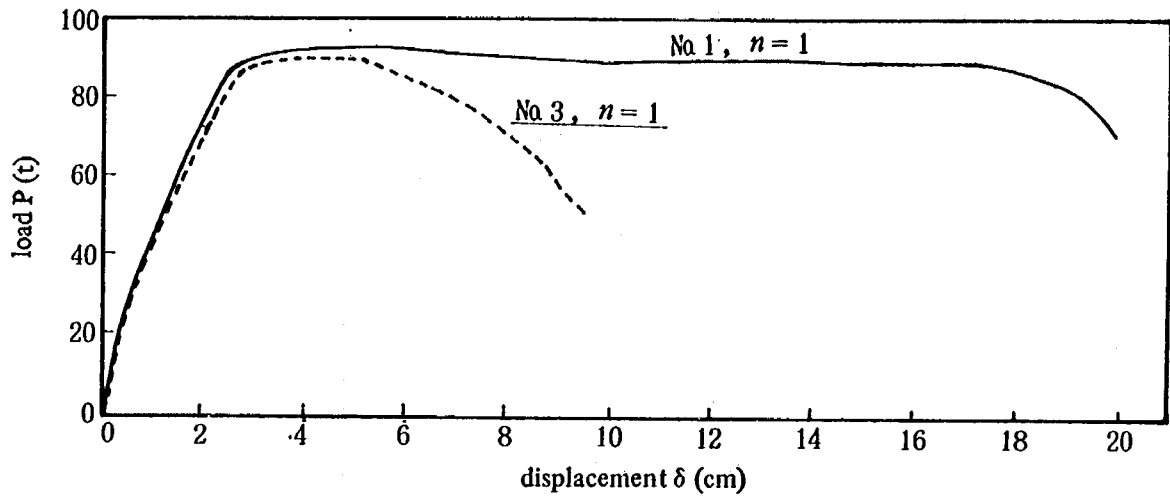


Fig. 2-6 Effect of Reversed Loading (Comparison of No. 1 and No. 3 Models)

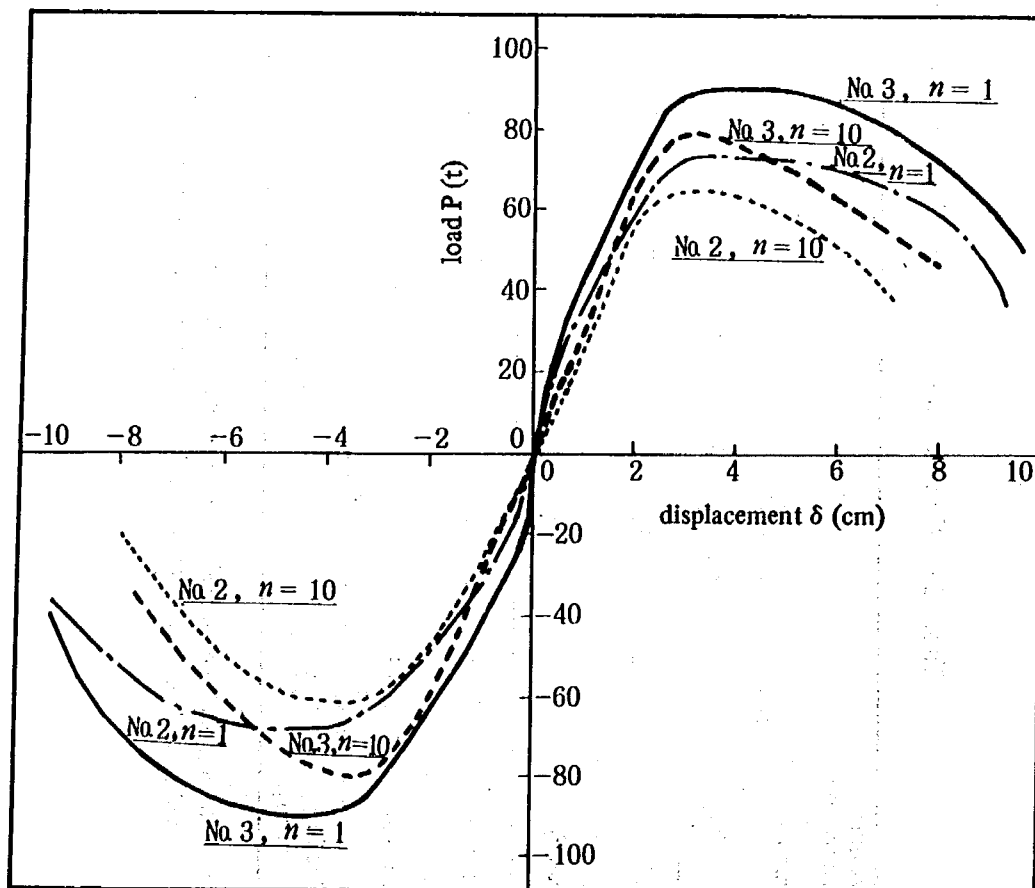


Fig. 2-7 Effect of the Hunch (Comparison of No. 2 and No. 3 Models)

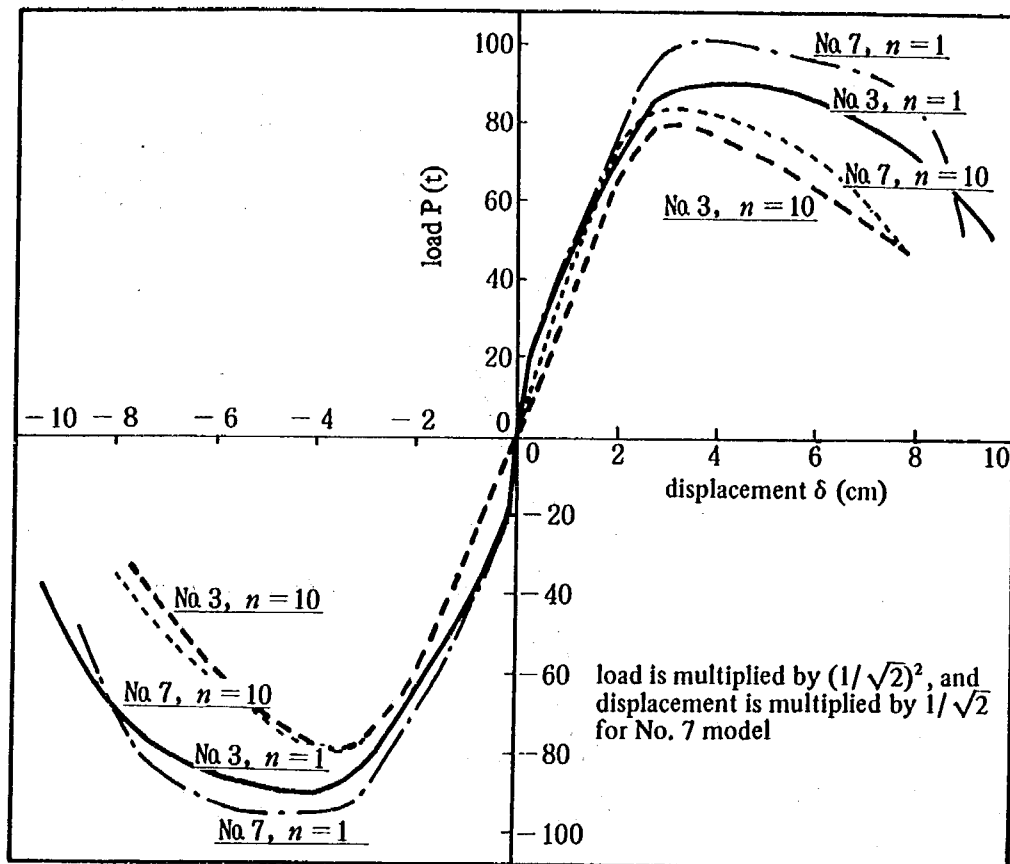


Fig. 2-8 Effect of an Axial Load (Comparison of No. 3 and No. 7 Models)

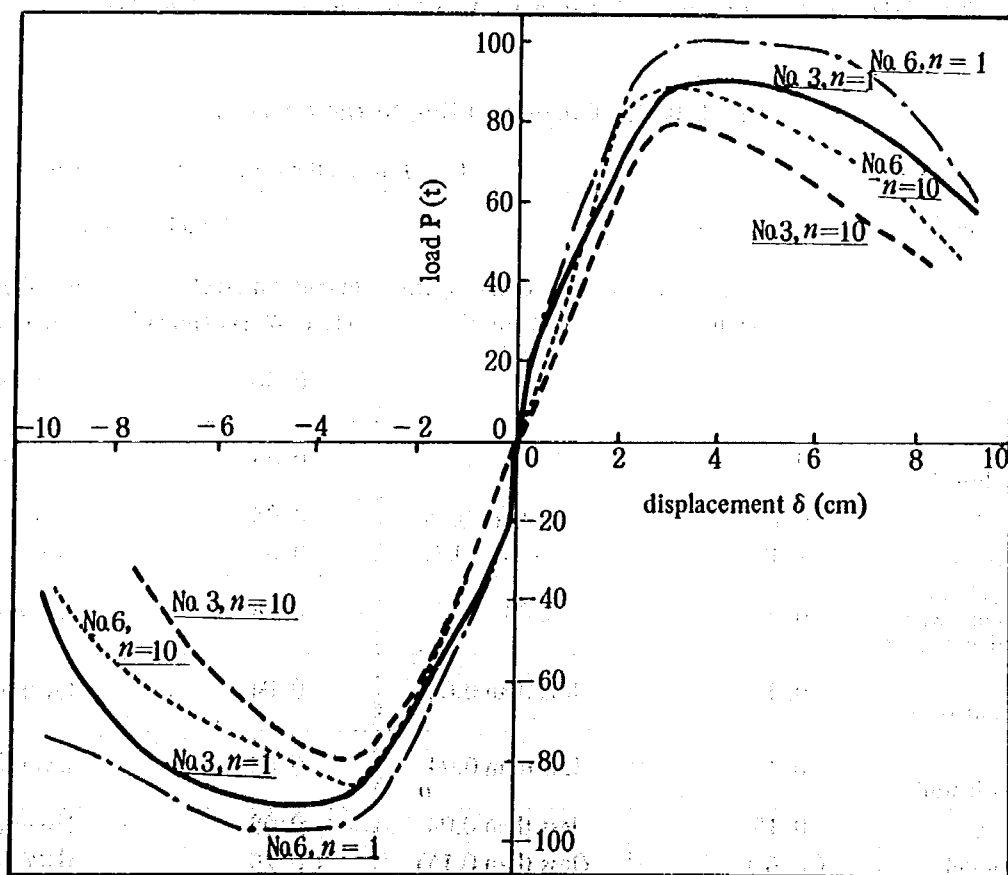


Fig. 2-9 Comparison of RC and SRC Structures (Comparison of No. 3 and No. 6 Models)

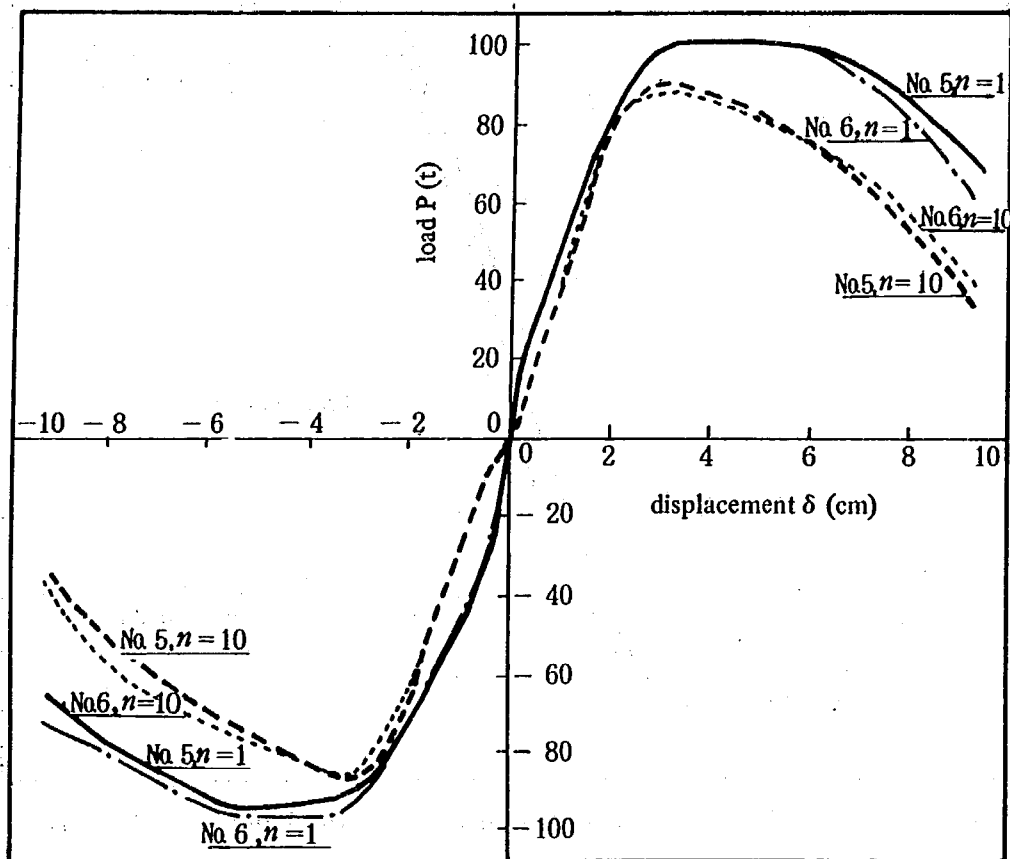


Fig. 2-10 Effect of Studs (Comparison of No. 5 and No. 6 Models)

Table 2-5 Crack Width Corresponding to the Design Load

() shows the estimated width of the actual bridge

loading direction item model	hunch-tension		hunch-compression	
	maximum crack width Wmax (mm)	residual crack width Wres (mm)	maximum crack width Wmax (mm)	residual crack width Wres (mm)
No. 1 one-directional loading	—	—	0.08	less than 0.04
No. 2 without hunch	0.3	0.04	0.08	less than 0.04
No. 3 basic model	0.2 (1.1)	less than 0.04 (less than 0.2)	0.08 (0.5)	less than 0.04 (less than 0.2)
No. 4 large-diameter reinforcement, with- out side reinforcement	0.3	0.04	0.08	less than 0.04
No. 5 SRC, without stud	0.1	less than 0.04	0.04	less than 0.04
No. 6 SRC, with stud	0.1	less than 0.04	0.06	less than 0.04
No. 7 1/4 model	0.15 (0.6)	less than 0.04 (less than 0.15)	0.06 (0.25)	less than 0.04 (less than 0.15)

Table 2-6 Estimated Aseismic Performances of the Upper Column of the Actual Bridge

Item		Loading direction		Hunch-tension		Hunch-compression	
		Loading cycle n		1	10	1	10
Strength	Yield load P_y (t)			-2,800	-2,800	2,830	2,830
	Ultimate load P_u (t)			-2,990	-2,530	3,200	2,690
Deformability	Yield displacement δ_y (cm)			-16.4	-16.4	12.8	12.8
	Ultimate displacement δ_u (cm)			-38.4	—	43.6	—
	Displacement ductility ratio μ			2.3	—	3.4	—
Potential energy E_u (t·cm)				76,200	—	91,500	—

3. Analytical Investigations

3.1 Estimation of strength

Measured loads by the experiment and calculated loads assuming a RC section corresponding to the resistant bending moment at the yield and ultimate states of the base section of the upper column are shown in Table 3-1 and Fig. 3-1. The measured loads were larger than the calculated ones at the yield state. This is because that a cracking point and a strain-measuring point were not necessarily the same. At the ultimate state, the measured and calculated loads were about the same for the first loading cycle ($n=1$), while the measured loads were smaller than the calculated ones for the tenth loading cycle ($n=10$) because of the effect of repeated cyclic loading. When an axial load was applied, the measured load was larger than the calculated one at the yield state, while the former was smaller than the latter at the ultimate state.

3.2 Estimation of a Load-displacement Curve

Calculated and measured load-displacement envelope curves of $n=1$ for the one-directional loading model (No. 1 model) and the basic model (No. 3 model) were compared as mentioned below. The following is the calculation procedure of a load-displacement curve.

Divide the upper column into m -elements in the axial direction. Calculate bending moment M_c , M_y , M_u and curvature ϕ_c , ϕ_y , ϕ_u of each element at the cracking, yield, and ultimate states to obtain M - ϕ relationships as shown in Fig. 3-2. M_c and ϕ_c can be calculated by eq. (3.1).

$$\left. \begin{aligned} M_c &= W \left(\sigma_{bt} + \frac{N}{A_g} \right) \\ \phi_c &= \frac{M_c}{E_c I_g} \end{aligned} \right\} \quad (3.1)$$

where, σ_{bt} : bending tensile strength of concrete
 N : axial load to the element
 A_g, I_g : equivalent sectional area and equivalent moment of inertia of area
 W : section modulus
 E_c : Young's modulus of concrete

For the calculation of M_y , ϕ_y , M_u , and ϕ_u , divide each element into n -small elements and obtain the strain of each small element assuming the stress and strain to be constant within the small element. Using the obtained strain, obtain the corresponding stress by the stress-strain curves as assumed in Fig. 3-3 and the neutral axis satisfying eqs. (3.2) and (3.3).

$$N = \sum_{i=1}^n \sigma_{ci} \cdot \Delta A_{ci} + \sum_{i=1}^n \sigma_{si} \cdot \Delta A_{si} \quad (3.2)$$

$$M = \sum_{i=1}^n \sigma_{ci} \cdot x_i \cdot \Delta A_{ci} + \sum_{i=1}^n \sigma_{si} \cdot x_i \cdot \Delta A_{si} \quad (3.3)$$

where, N, M : axial load and bending moment to the element
 σ_{ci}, σ_{si} : stress of concrete and reinforcements in the small element
 $\Delta A_{ci}, \Delta A_{si}$: sectional areas of concrete and reinforcements in the small element
 x_i : distance from concrete or reinforcements of the small element to the neutral axis

Calculate bending moment by eq. (3.3) and curvature by eq. (3.4).

$$\phi = \frac{\epsilon_{co}}{x_o} \quad (3.4)$$

where, ϵ_{co} : extreme fiber strain
 x_o : distance from the compressive edge of concrete to the neutral axis

Obtain M_y , ϕ_y assuming the yield strain of reinforcements as $\epsilon_s = \sigma_{sy}/E_s$ (σ_{sy} : yield strength of a reinforcement, E_s : Young's modulus of a reinforcement) and M_u , ϕ_u assuming the ultimate strain of concrete as $\epsilon_c = 0.0035$.

Calculate lateral displacement caused by the bending deformation of the upper column by eq. (3.5).

$$\left. \begin{aligned} \delta_i &= \int_0^{y_i} \theta \, dy \approx \sum_{j=2}^i \frac{\theta_j + \theta_{j-1}}{2} \cdot \Delta y_{j-1} \\ \theta_j &= \int_0^{y_j} \phi \, dy \approx \sum_{k=2}^j \frac{\phi_k + \phi_{k-1}}{2} \cdot \Delta y_{k-1} \end{aligned} \right\} \quad (3.5)$$

where, δ_i : deflection of section-i of y_i -distant from the base of the upper column
 θ_j, θ_{j-1} : rotation angles of j th and $j-1$ th sections from the base of the upper column
 Δy_{j-1} : distance between $j-1$ th and j th sections from the base of the upper column

Rotation at the base of the upper column was measured as shown in Fig. 2-5. Analytic models as shown in Table 3-2 were used for the calculation of a P- δ curve to take the rotation into consideration. The following are the outlines of the analytic models.

Case A: A model in which the base of the upper column is fixed and only bending deformation of the column is taken into consideration. Rotation at the base of the upper column is not taken into consideration.

Case B: A model in which the hunched column is assumed to be extended to the footing and bending deformation of the upper column including the extended part is taken into consideration. Rotation at the base of the upper column is taken into consideration by the bending deformation of the extended part.

Case C: A model in which the rotation at the base of the upper column is taken into consideration by pulled-out of the longitudinal reinforcements (Fig. 3-4). A rotation angle at the base of the upper column is estimated by eq. (3.6)

$$\left. \begin{aligned} \theta &= \frac{\Delta}{x} \\ \Delta &= \frac{\epsilon_s \cdot \ell}{2} \\ \ell &= \frac{A_s \cdot \sigma_s}{U \cdot \tau_{bs}} \end{aligned} \right\} \quad (3.6)$$

where, Δ : pulled-out length of the longitudinal reinforcement
 x : distance from the longitudinal reinforcement to the neutral axis
 ϵ_s : strain of the longitudinal reinforcement at the base of the upper column

- l : length of the longitudinal reinforcement from the base of the upper column to the point where the strain of the longitudinal reinforcement becomes zero
 A_s : sectional area of the longitudinal reinforcement
 σ_s : stress of the longitudinal reinforcement
 U : perimeter of the longitudinal reinforcement
 τ_{bs} : average bond stress

Case D: (a) In the case of hunch-compression

A model in which the rotation at the base of the upper column is taken into consideration by plastic deformation of concrete by pulled-out of the longitudinal reinforcement below the base of the column. In this case, the rotation angle is estimated by eq. (3.7) where ϵ_s is constant from the base of the column to the footing.

$$\left. \begin{aligned} \theta &= \frac{\Delta}{x} \\ \Delta &= \epsilon_s \cdot l \end{aligned} \right\} \quad (3.7)$$

(b) In the case of hunch-tension

A model in which the rotation at the base of the upper column is taken into consideration by a similar method as Case C. In this case, strain-zero point is assumed to be at the top of the footing considering measured distribution of strains and cracks.

Estimated load-displacement curves at the loaded point and rotation angles at the base of the upper column by the analytic models above mentioned are shown in Figs. 3-6 ~ 3-11 where measured ones are also represented.

In Case A in which the upper column was fixed at the base, the estimated yield displacement was 45% of the measured one in the hunch-compression direction and 40% of that in the hunch-tension direction. Displacement caused by the rotation at the base of the upper column was 35% of the measured total displacement in the hunch-compression direction and 30% of that in the hunch-tension direction in the experiment. This means the importance of taking the rotation into consideration in the estimation of the displacement. When the estimated displacement was corrected by the measured rotation, it became 80% of the measured total displacement in the hunch-compression direction and 70% of that in the hunch-tension direction. Estimated load-displacement curves after the yield point differed from the measured ones. The estimated ultimate displacement of the basic model (No. 3 model) is about a half of the measured one (Fig. 3-6).

In Case B in which the hunched column was extended to the footing, the estimated yield displacement was small in the hunch-compression direction and large in the hunch-tension direction as compared with the measured yield displacement. In this model, members below the base of the upper column yielded before those at the base of the column did by the hunch-tension load (Figs. 3-7, 3-8).

In case C and Case D, the estimated rotation angle θ subjected to the one-directional load and 2θ considering the effect of reversed loading were used to estimate the displacement. As for a load-rotation angle curve, Case D (+ θ) in the hunch-compression direction and Case D (- θ) and Case C (- 2θ) in the hunch-tension direction gave good agreement with the measured curve (Fig. 3-9). As for a load-displacement curve, Case D (+ θ) in the hunch-compression direction and Case D (- 2θ) in the hunch-tension direction gave good agreement with the measured curve (Figs. 3-10, 3-11).

For more precise estimation, it is needed to consider shear deformation and the effect of the concentration of cracking in the calculation of displacement. More over, it is needed to investigate stress-strain relationship of concrete and reinforcements, bending moment-curvature relationship, and effects of side reinforcements and confining reinforcements to estimate a load-displacement curve after the yield point more precisely.

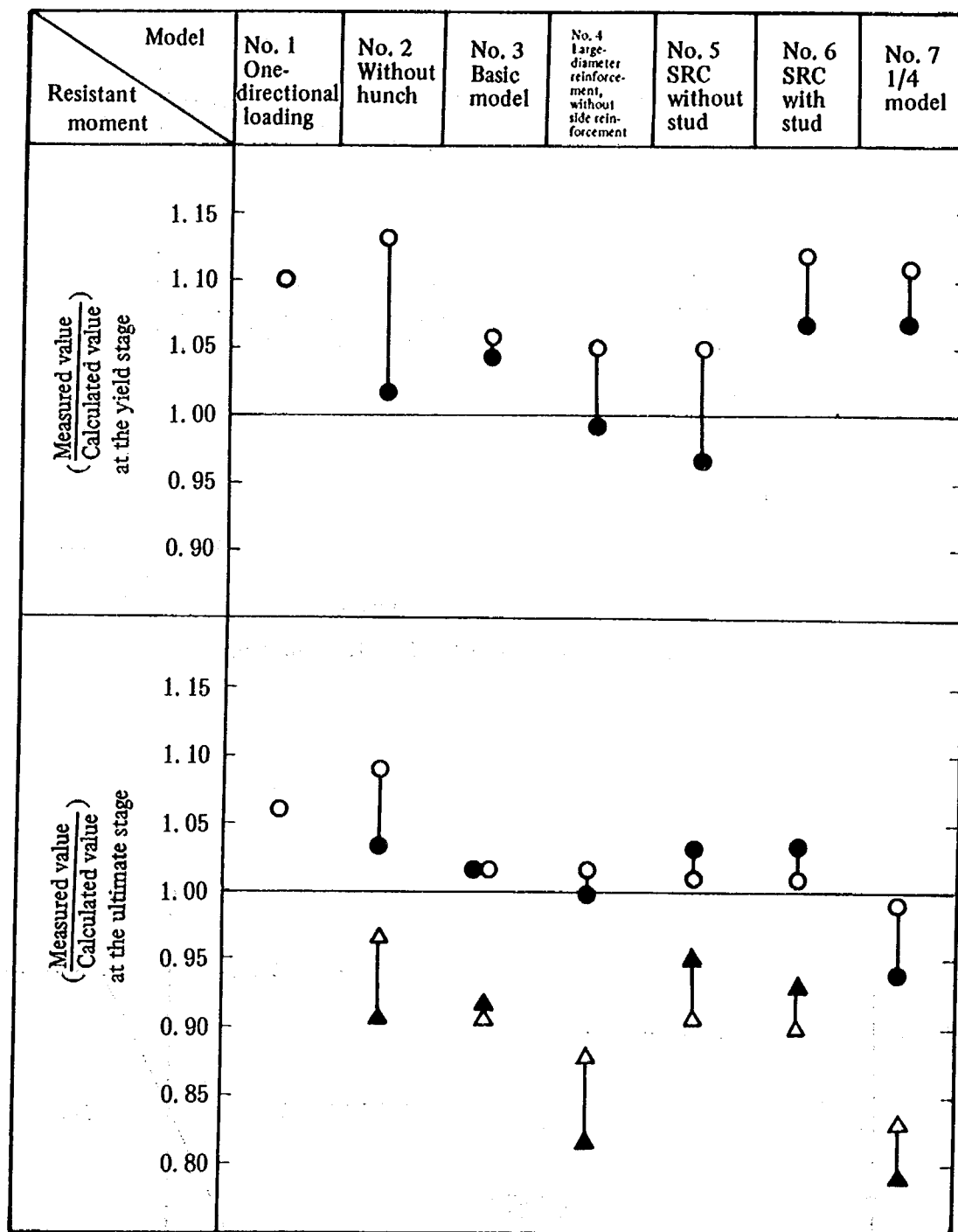
3.3 Hysteretic Characteristics

Hysteresis loops of load-displacement curves obtained by the experiment are shown in Appendix-3. The shape of the hysteresis loop was spindle-shape at the initial stage and shifted to inverse-S shape as the displacement became large and the cycle of loading increased. In the load-displacement envelope curves, the load increased after the yield point to the maximum load and decreased after that. The load decreased by reversed cyclic loading of the same displacement. As no analytic model to represent these hysteretic characteristics has been proposed so far, formulation of the characteristics is needed to develop a new model.

Equivalent damping factors (h) obtained from the hysteresis loops were $h = 0.02 \sim 0.04$ at the displacement amplitude of δ_0 , $h = 0.06 \sim 0.10$ at that of $2\delta_0$, and $h = 0.10 \sim 0.13$ at that of larger than $3\delta_0$.

Table 3-1 Comparison of Measured and Calculated Loads at the Yield and Ultimate Stages of the Base Section of the Upper Column

Loading direction Model Load (t)		Hunch-tension							Hunch-compression							
		No. 1 One-dire- ctional loading	No. 2 Without hunch	No. 3 Basic model	No. 4 Large- diameter reinforce- ment, without side rein- forcement	No. 5 SRC without stud	No. 6 SRC with stud	No. 7 1/4 model	No. 1 One-dire- ctional loading	No. 2 Without hunch	No. 3 Basic model	No. 4 Large- diameter reinforce- ment, without side rein- forcement	No. 5 SRC without stud	No. 6 SRC with stud	No. 7 1/4 model	
Yield	Measured	—	55	75	69	72	79	175	75	61	72	66	80	85	177	
	Calculated	—	54	72	70	74	74	163	68	54	68	63	76	76	160	
	Measured Calculated	—	1.02	1.04	0.99	0.97	1.07	1.07	1.10	1.13	1.06	1.05	1.05	1.12	1.11	
Ultimate	Measured	n = 1	—	69	89	73	94	96	187	92	73	89	68	100	101	200
		n = 10	—	61	80	60	86	85	158	—	65	79	58	90	89	168
	Calculated		—	67	87	73	91	91	199	87	67	87	66	99	99	203
	Measured Calculated	n = 1	—	1.03	1.02	1.00	1.03	1.05	0.94	1.06	1.09	1.02	1.03	1.01	1.02	0.99
		n = 10	—	0.91	0.92	0.82	0.95	0.93	0.79	—	0.97	0.91	0.88	0.91	0.90	0.83



Note: cycle of loading $n = 1$ { $\bigcirc$ Hunch-compression
 $\bullet$ Hunch-tension

$n = 10$ { $\triangle$ Hunch-compression
 $\blacktriangle$ Hunch-tension

Fig. 3-1 Comparison of Measured and Calculated Resistant Moments

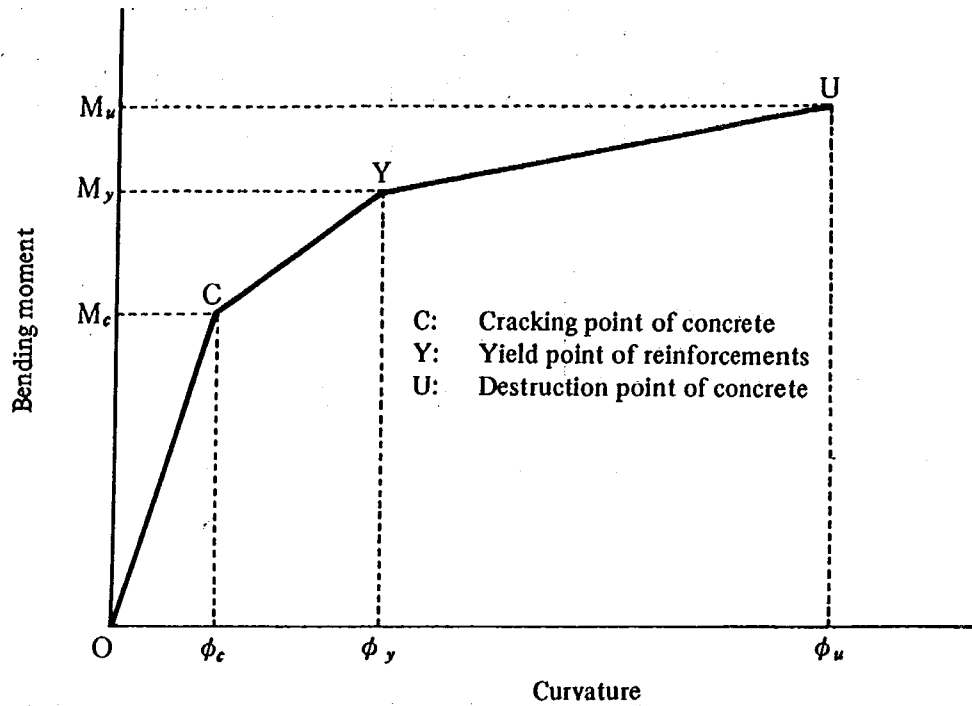


Fig. 3-2 Assumption of the M- ϕ Relationship

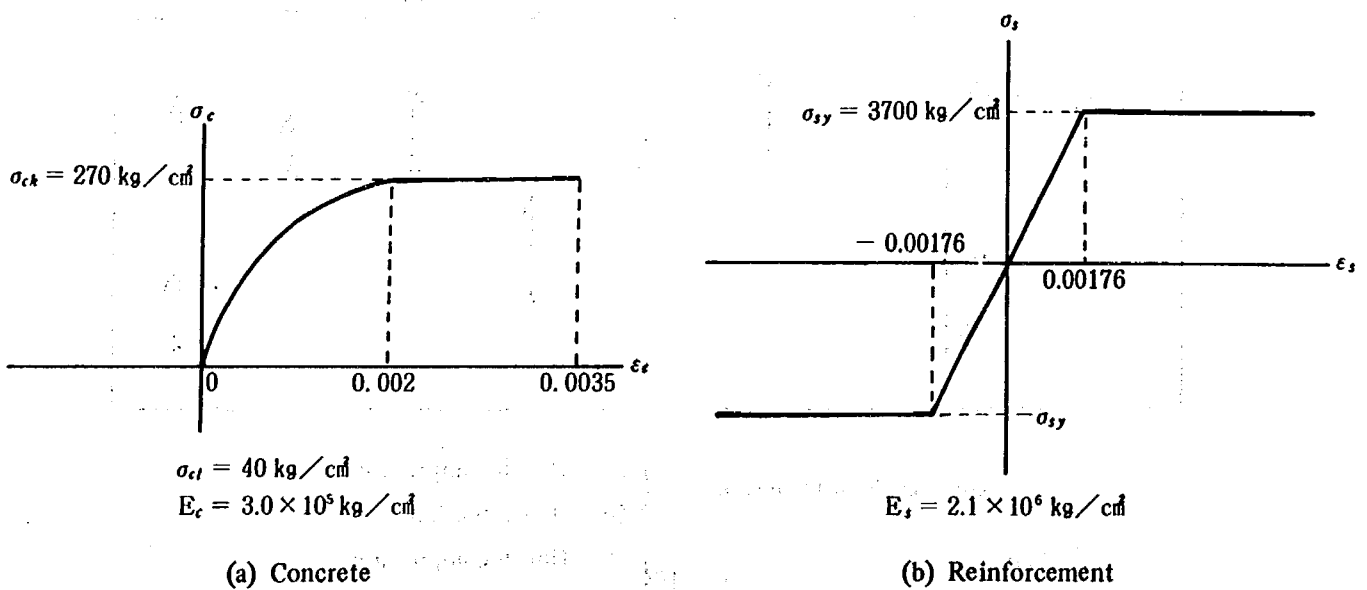
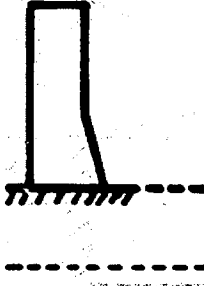
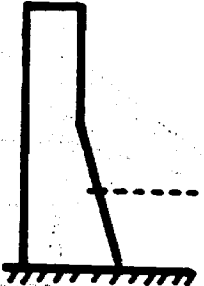
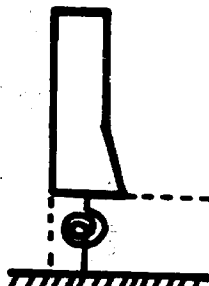
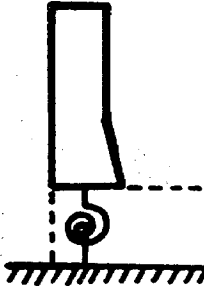
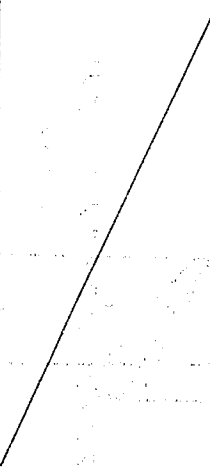
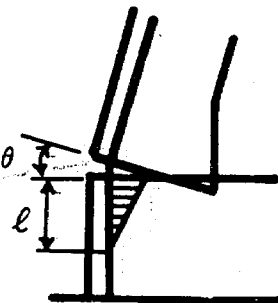
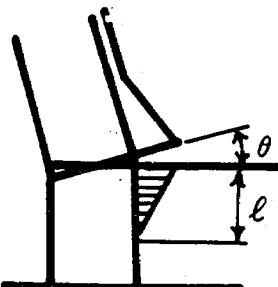
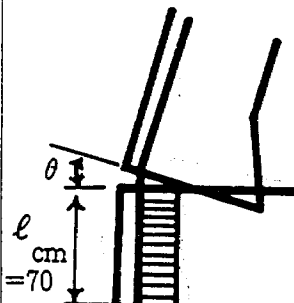
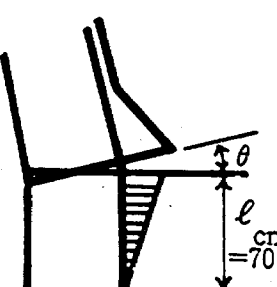
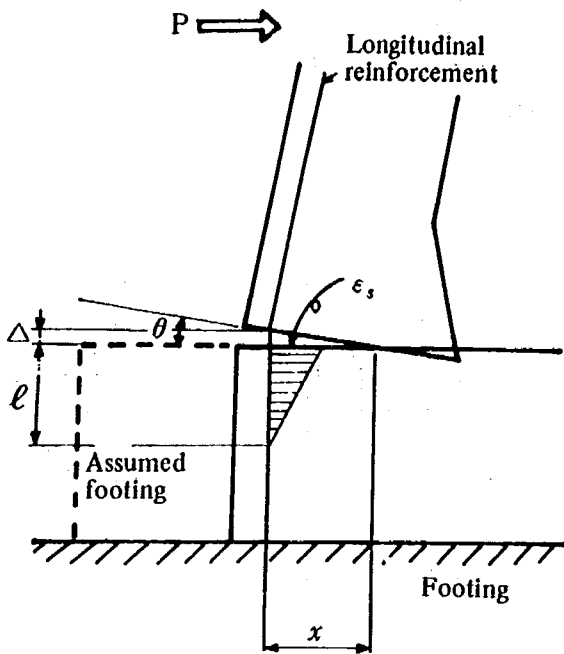


Fig. 3-3 Assumption of the Stress-strain Relationship of Materials

Table 3-2 Analytic Model for the Estimation of a P-δ Curve

Case	Case A	Case B	Case C		Case D	
Model						
Loading direction	Hunch-compression, hunch-tension	Hunch-compression, hunch-tension	Hunch-compression	Hunch-tension	Hunch-compression	Hunch-tension
Consideration of rotation		Rotation is considered by the deformation of the extended part of the column.	 θ and 2θ	 θ and 2θ	 θ and 2θ	 θ and 2θ
Remark	Column is fixed at the base.	Hanched column is extended to the footing.	Rotation is considered by the pulled-out of reinforcements. $\ell = \frac{A_s \cdot \sigma_s}{U \cdot \tau_{bs}}$		Rotation is considered by the extension of reinforcements below the base of the column to the footing.	Rotation is considered by the pulled-out of reinforcements.

(a) Hunch-compression



(b) Hunch-tension

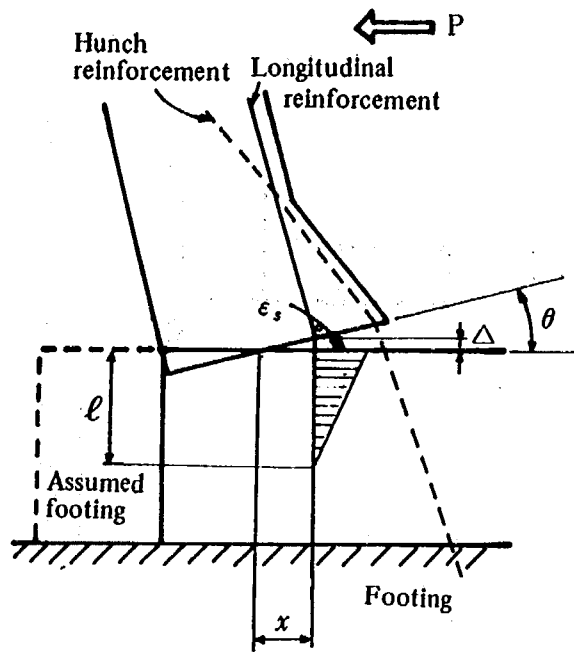
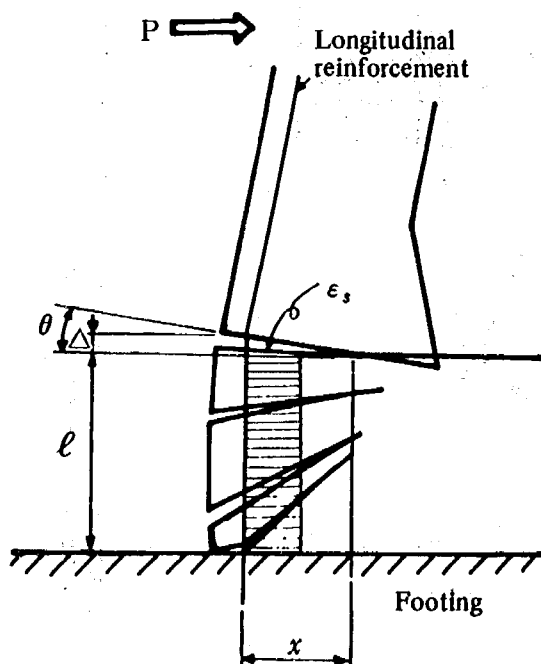


Fig. 3-4 Case C-Analytic Model

(a) Hunch-compression



(b) Hunch-tension

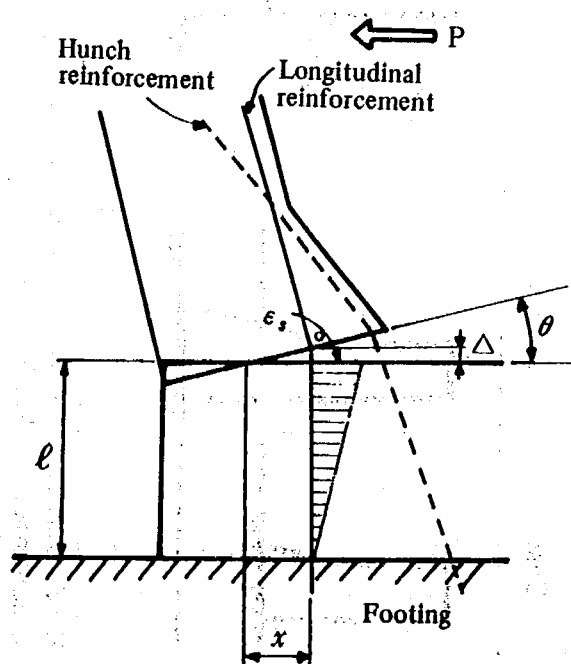
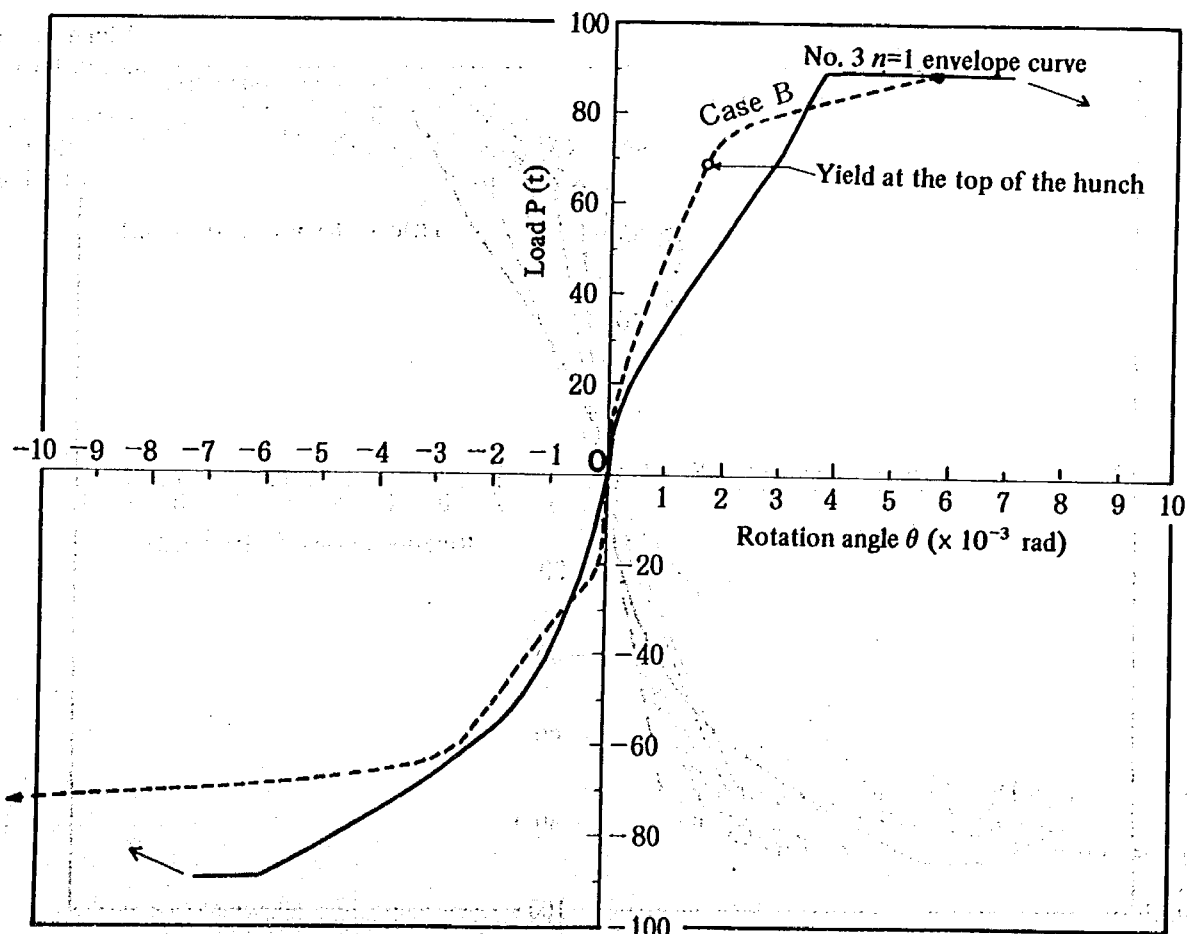
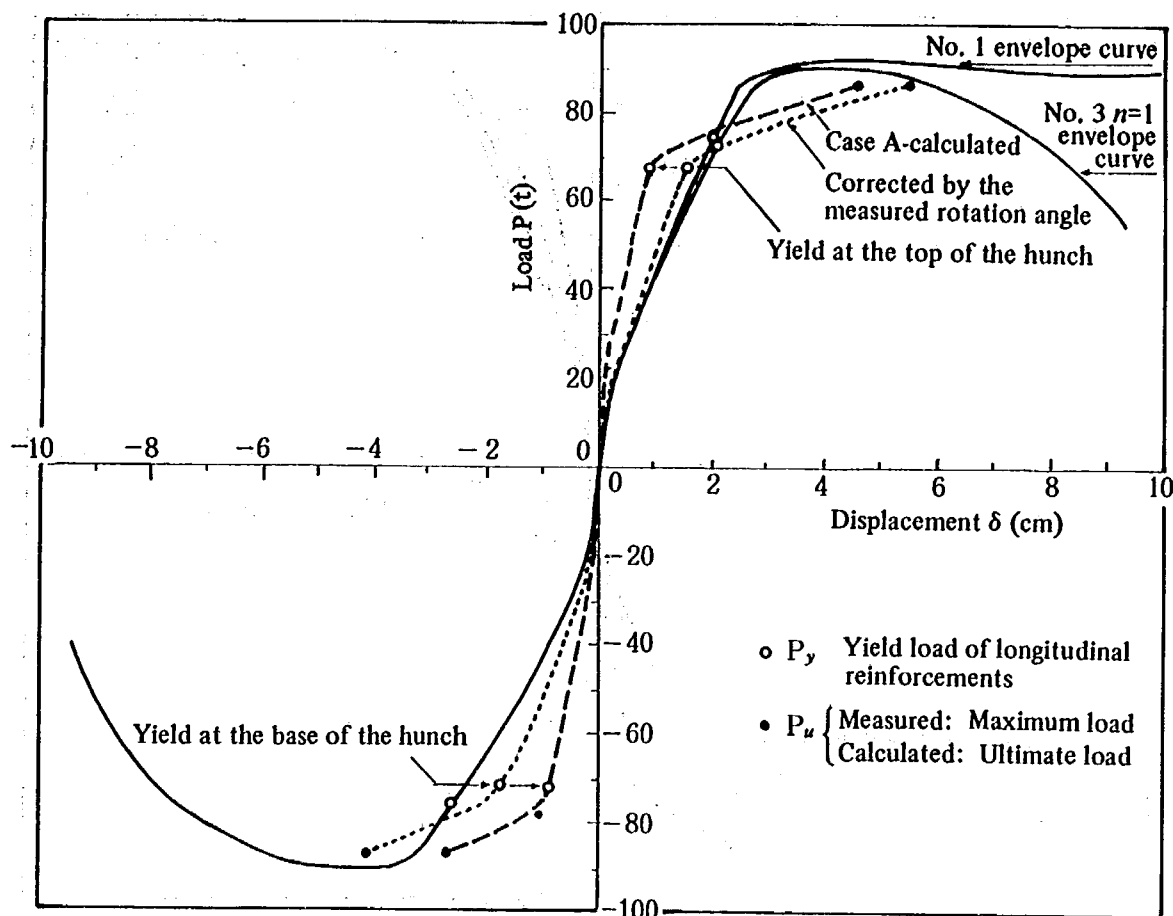


Fig. 3-5 Case D-Analytic Model



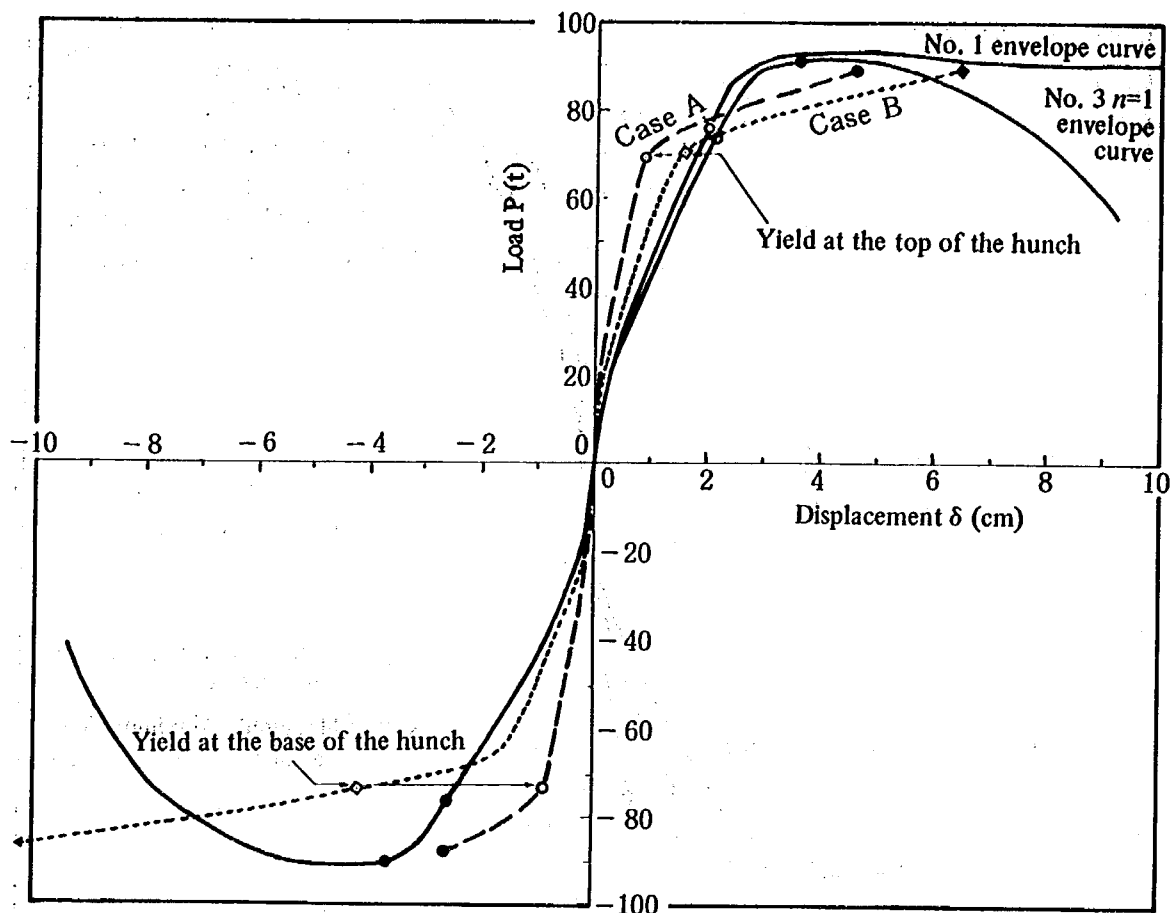


Fig. 3-8 Comparison of Case B – calculated and Measured P - δ Curves

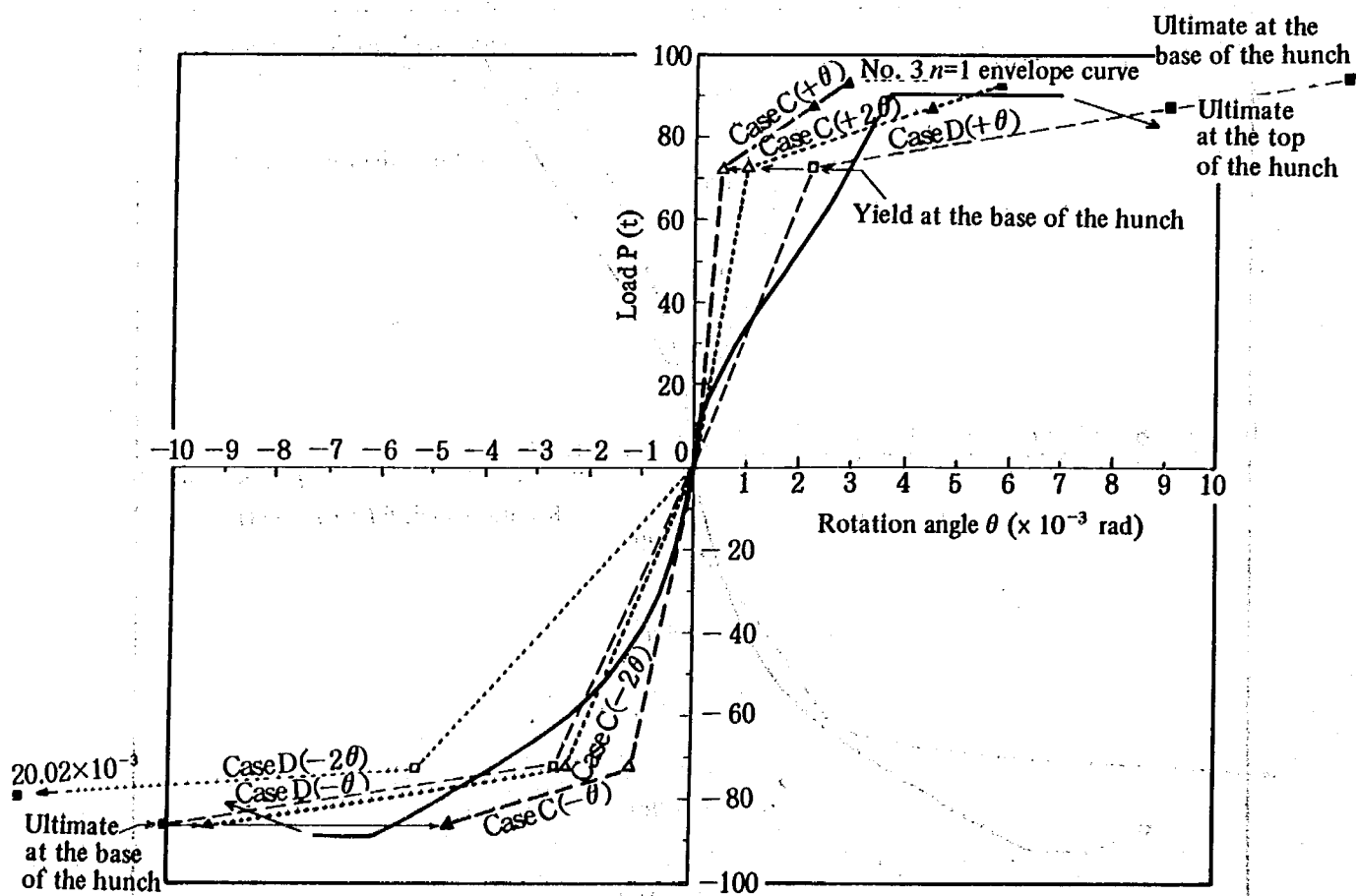


Fig. 3-9 Comparison of Case C and Case D – calculated and Measured P - θ Curves

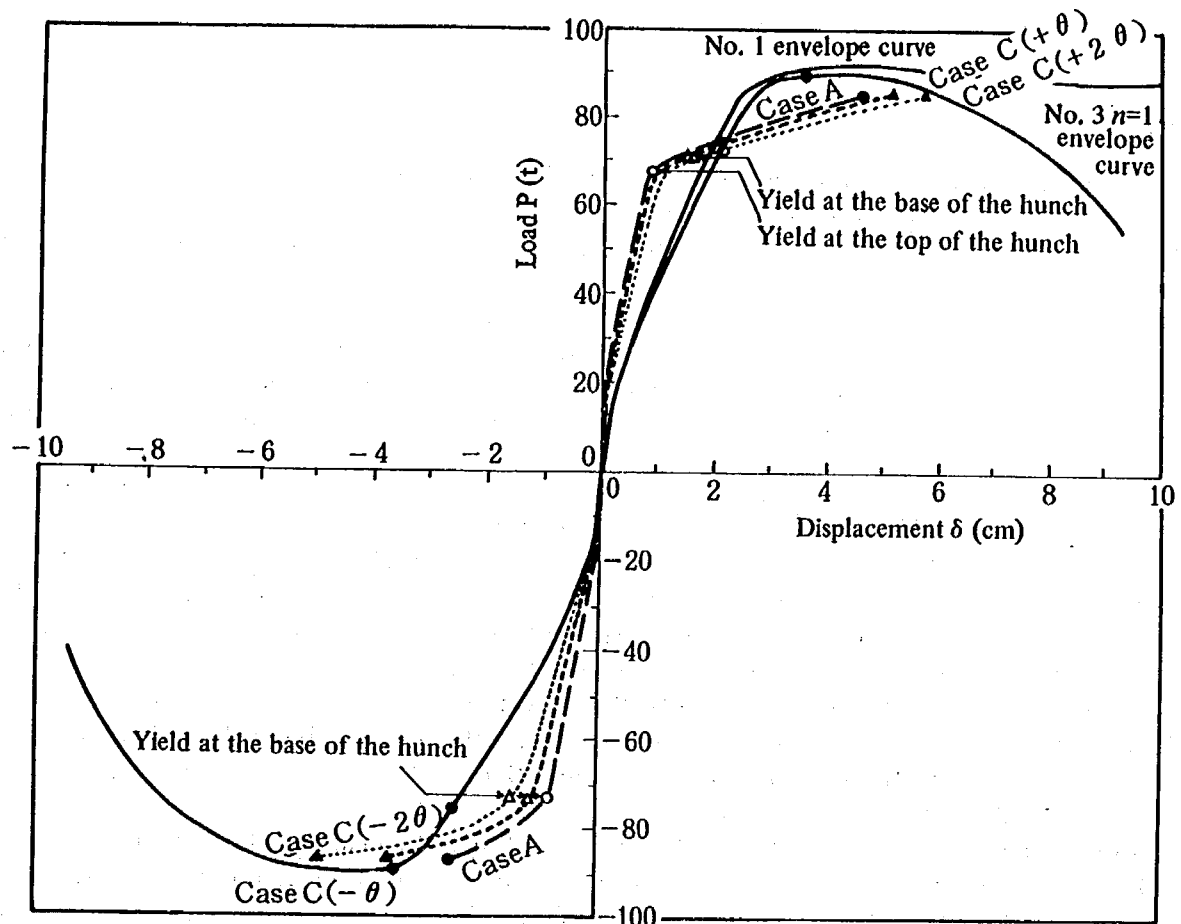


Fig. 3-10 Comparison of Case C — calculated and Measured P- δ Curves

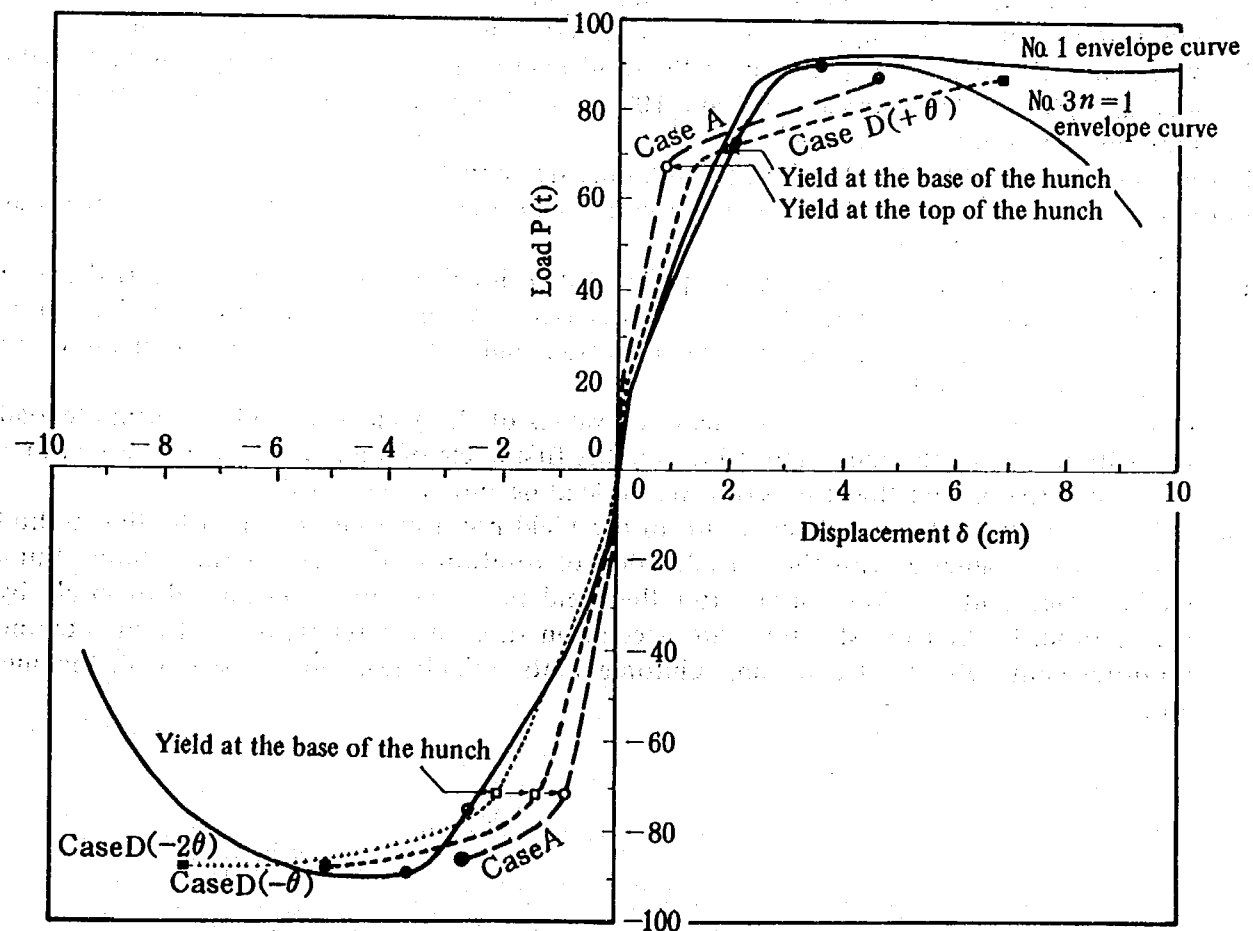


Fig. 3-11 Comparison of Case D — calculated and Measured P- δ Curves

4. Conclusions

The following results were obtained by the model loading tests of an upper column of the Ban-no-su Bridge pier of Honshu-Shikoku Bridge.

- (1) Stiffness of the reinforced concrete column decreases by the cracking. Stiffness at the yield stage is $1/3 \sim 1/4$ of the initial stiffness.
- (2) Maximum strength of the column is $10 \sim 30\%$ larger than the yield strength.
- (3) Displacement ductility ratio (ultimate displacement/yield displacement) of the column is 10 subjected to one-directional (hunch-compression direction) loads and 4 subjected to reversed loads. Reversed loading makes the ductility ratio decrease to $1/2 \sim 1/3$ of that by one-directional loading.
- (4) In the case of reversed cyclic loading with the same displacement, load decreases remarkably by the first cycle of loading and decreases gradually by the following cycle of loading.
- (5) Shape of the hysteresis loop is spindle-shape at the initial stage and shifts to inverse-S shape as the input displacement becomes large and the cycle of loading increases. The equivalent damping factor of hysteresis damping of the column is $h = 0.02 \sim 0.05$ at the yield stage and $h = 0.10 \sim 0.13$ after the yield of reinforcements.
- (6) Load-displacement envelope curves until the maximum load in the cases of one-directional loading and reversed loading are almost the same. After the maximum load, load decreases more remarkably in the case of reversed loading than in the case of one-directional loading as the input displacement increases.
- (7) A hunch at the base of the upper column makes the yield load and the ultimate load $20 \sim 30\%$ larger than those of the column without a hunch. The hunched column was destroyed by bending-failure in the experiment, but it should be noted that the possibility of shear-failure becomes large by the hunch.
- (8) The column with large-diameter longitudinal reinforcements and without side reinforcements shear-failed at the upper part, and its strength and ductility were rather small.
- (9) When an axial load corresponding to the dead load of the superstructure is applied, yield load and ultimate load increase 20% and 10% respectively, and ductility decreases $10 \sim 20\%$ than those without an axial load.
- (10) Aseismic performances of RC and SRC columns are almost the same.
- (11) Effect of studs at the base of the upper column to prevent pulled-out of steel frames was small in the experiment.
- (12) Rotation of the upper column by the pulled-out of longitudinal reinforcements at the base of the column was observed. This rotation caused 35% of the displacement at the top of the column in the hunch-compression direction and 30% of that in the hunch-tension direction in the experiment.
- (13) Difference between estimated and measured values of the yield load and the ultimate load was within 15% of the measured values for the first cycle of loading. Further investigations are needed to consider the effects of reversed loading and an axial load.
- (14) A load-displacement envelope curve up to the yield point can be estimated by the method described in chapter 3 with the consideration of rotation at the base of the column. But a load-displacement envelope curve after the yield point can not be estimated precisely by the method. Further investigations are needed on stress-strain relationship of concrete and reinforcements, effects of confining reinforcements, axial loads, and reversed cyclic loading, etc.

Acknowledgements

Authors express their deep thanks for the cooperation of Honshu-Shikoku Bridge Authority in the experiment.

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Appendix — 1 Outlines of the Earthquake Engineering Laboratory

(1) Buildings

The testing building (main building) has a test pit (L.20m x W.15m x H.5m) embedded below the ground level. The actuators are to be set on any part of the walls of the pit. It also has operation room, data recording room, data processing room, data communication terminal, instrument storeroom, exhibition room, storeroom, utility room, and workshop.

The hydraulic power generation annex has a hydraulic power supply and electric power supply.

(2) Actuator

Two actuators are installed in the pit. Each actuator is capable of ± 125 t of maximum excitation, ± 125 mm of maximum stroke and ± 1 m/sec of maximum velocity with the frequency range of DC $\sim$ 30 Hz. An actuator is 3 m long and weighs 4 t.

The hydraulic power is provided by the hydraulic power supply in the annex. The excitation is controlled independently for two actuators by the control system. It is excited by the analog or digital control system.

(3) Hydraulic Power Supply

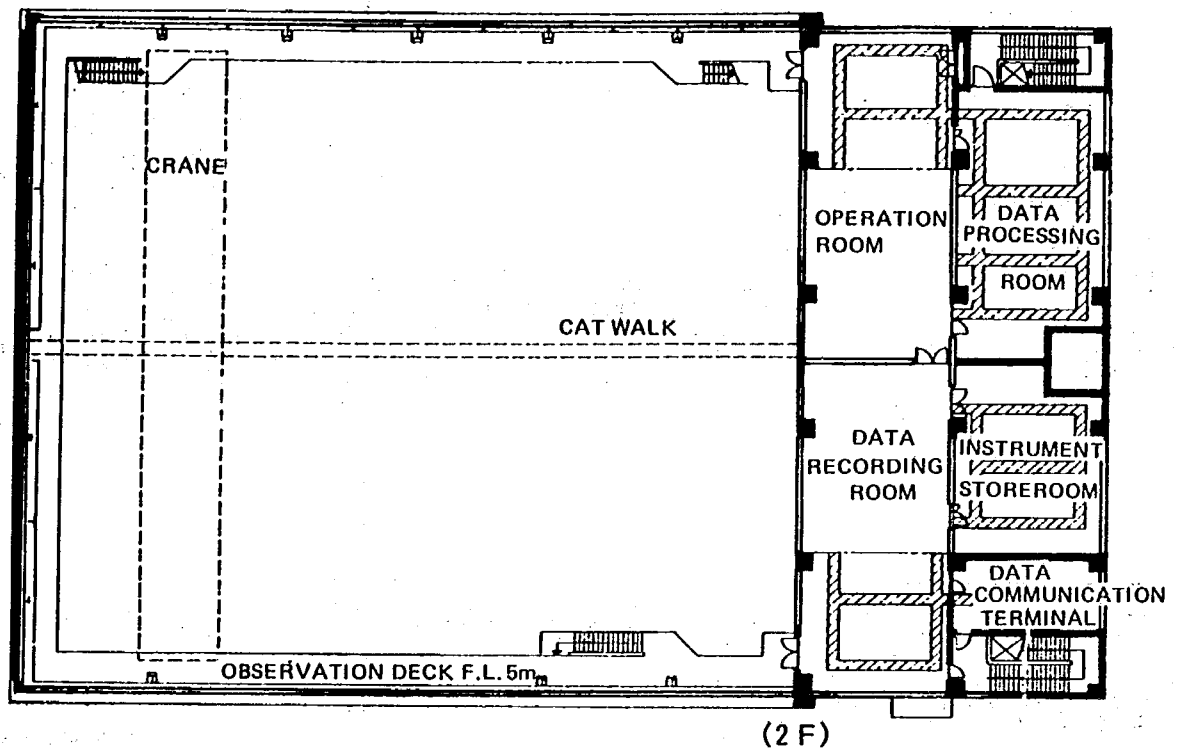
The hydraulic power supply is installed in the hydraulic power generation annex. The accumulators (6 x 200 l) are used for high velocity test (maximum excitation time is 10 sec.). The accumulators are released for the long-time durability and fatigue test in which only the oil pumps (2 x 300 l/min) are used. The capacity of the hydraulic power supply was determined to make the big velocity test possible. It is operated by the remote control in the operation room.

(4) Control System

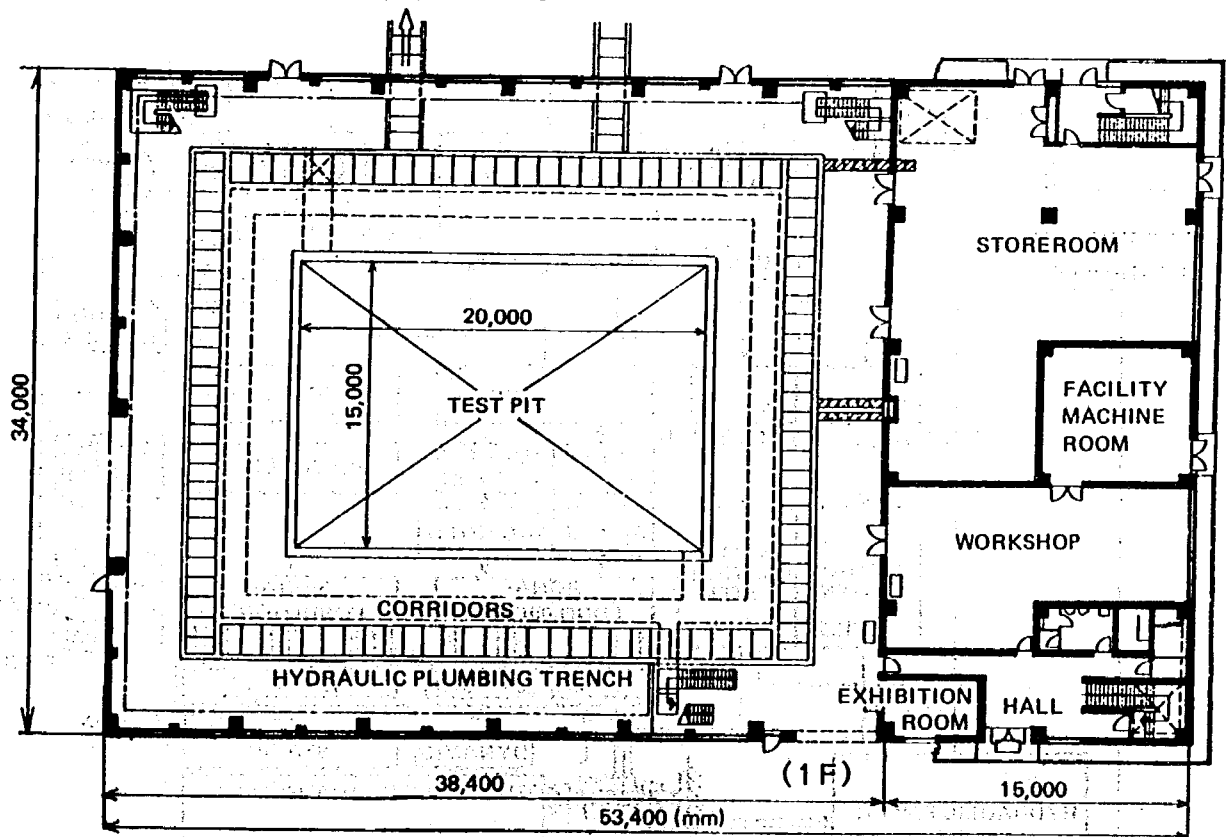
Control system is to control the actuators by the analog or digital control. The excitation is sinusoidal, triangular, rectangular, ramp waves and earthquake motions. Sweep excitation and the excitation with the designated power spectrum density are also possible. The drive signal is corrected, if necessary, to agree the excitation with the expected input motion.

(5) Data Acquisition

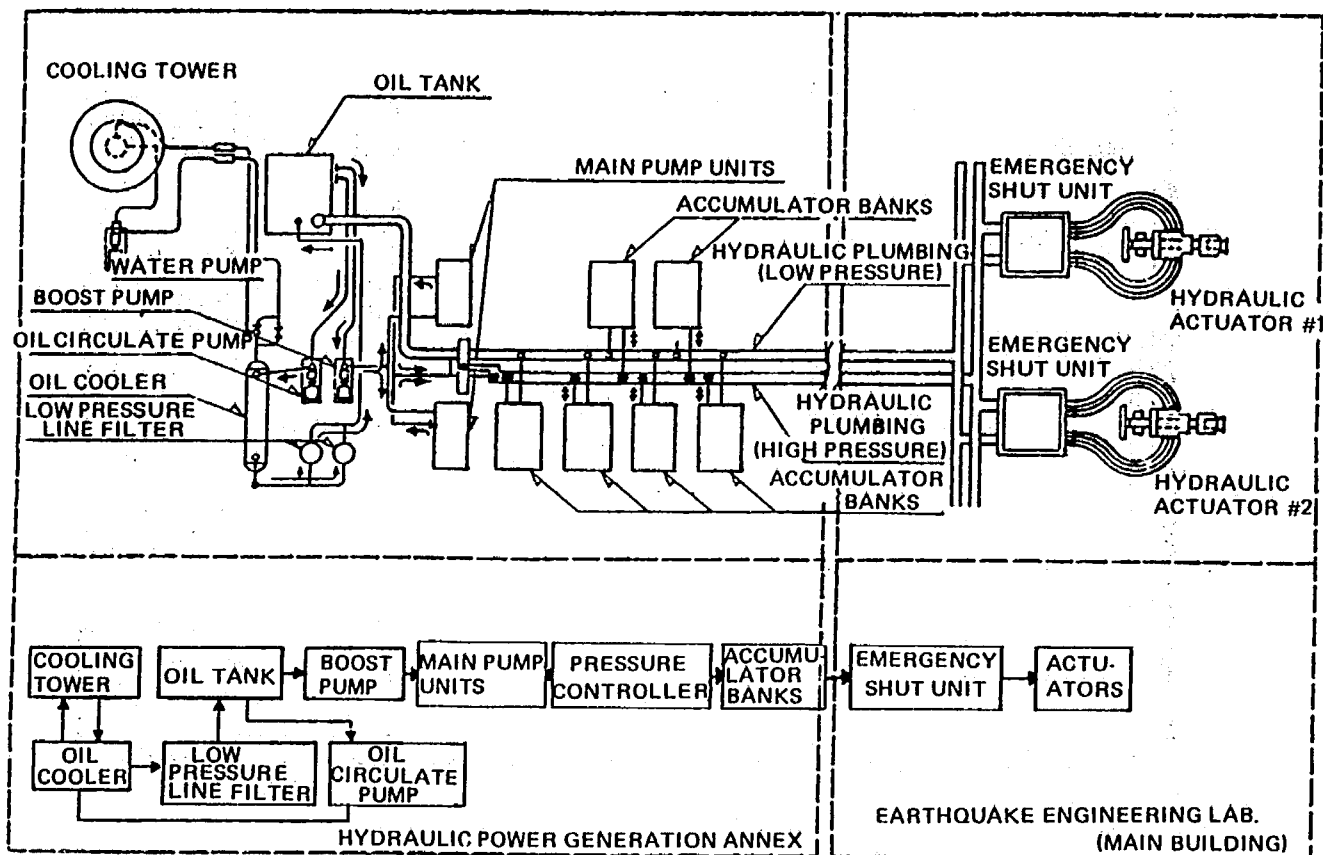
The data acquisition system has 32 channels for dynamic data and 128 channels for static data. The data are stored in the disk system through the A/D converter and the digital computer. The data are compiled and analysed later to get the dynamic characteristics of the specimens.



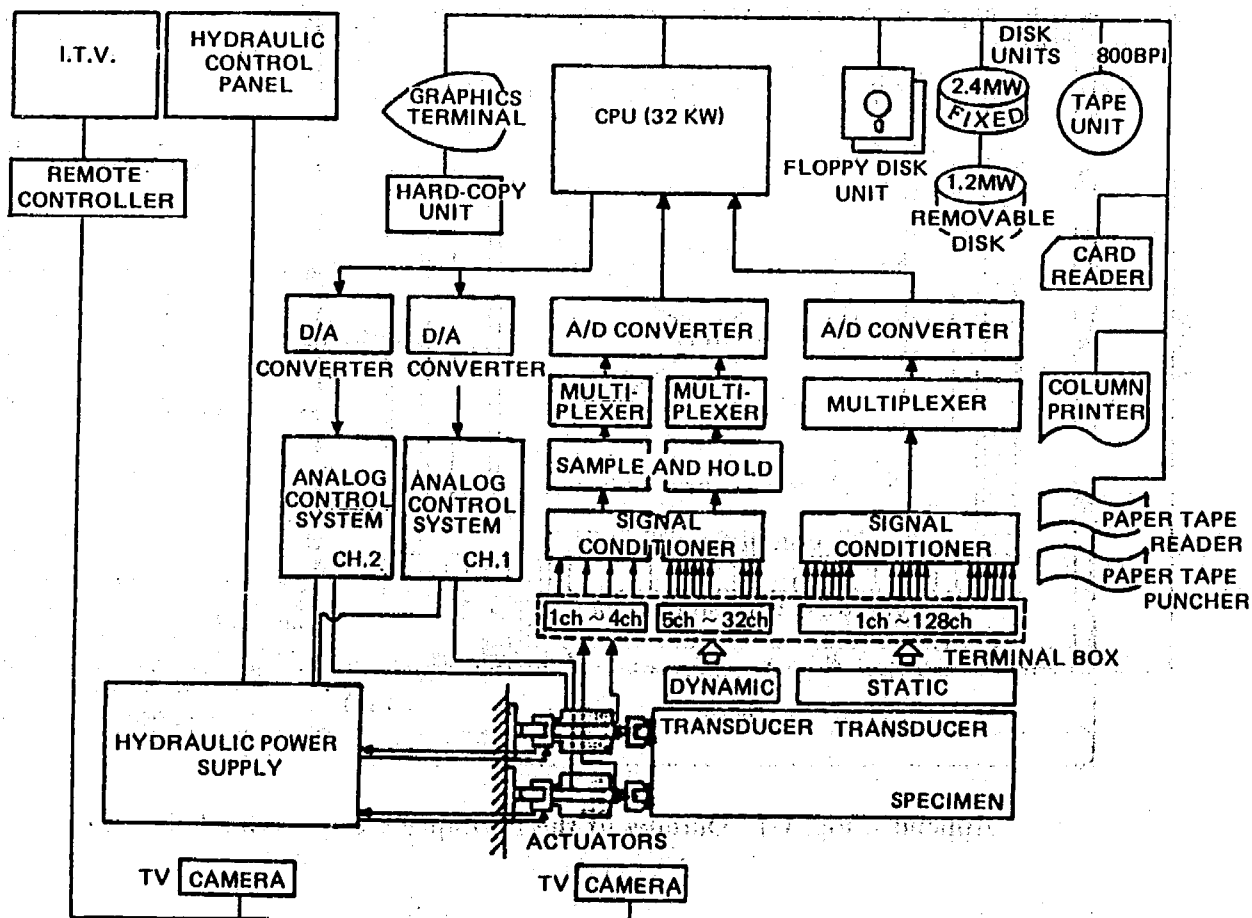
To hydraulic power generation annex



Appended Fig. 1-1 Outlines of the Earthquake Engineering Laboratory



Appended Fig. 1-2 Hydraulic Power Supply System



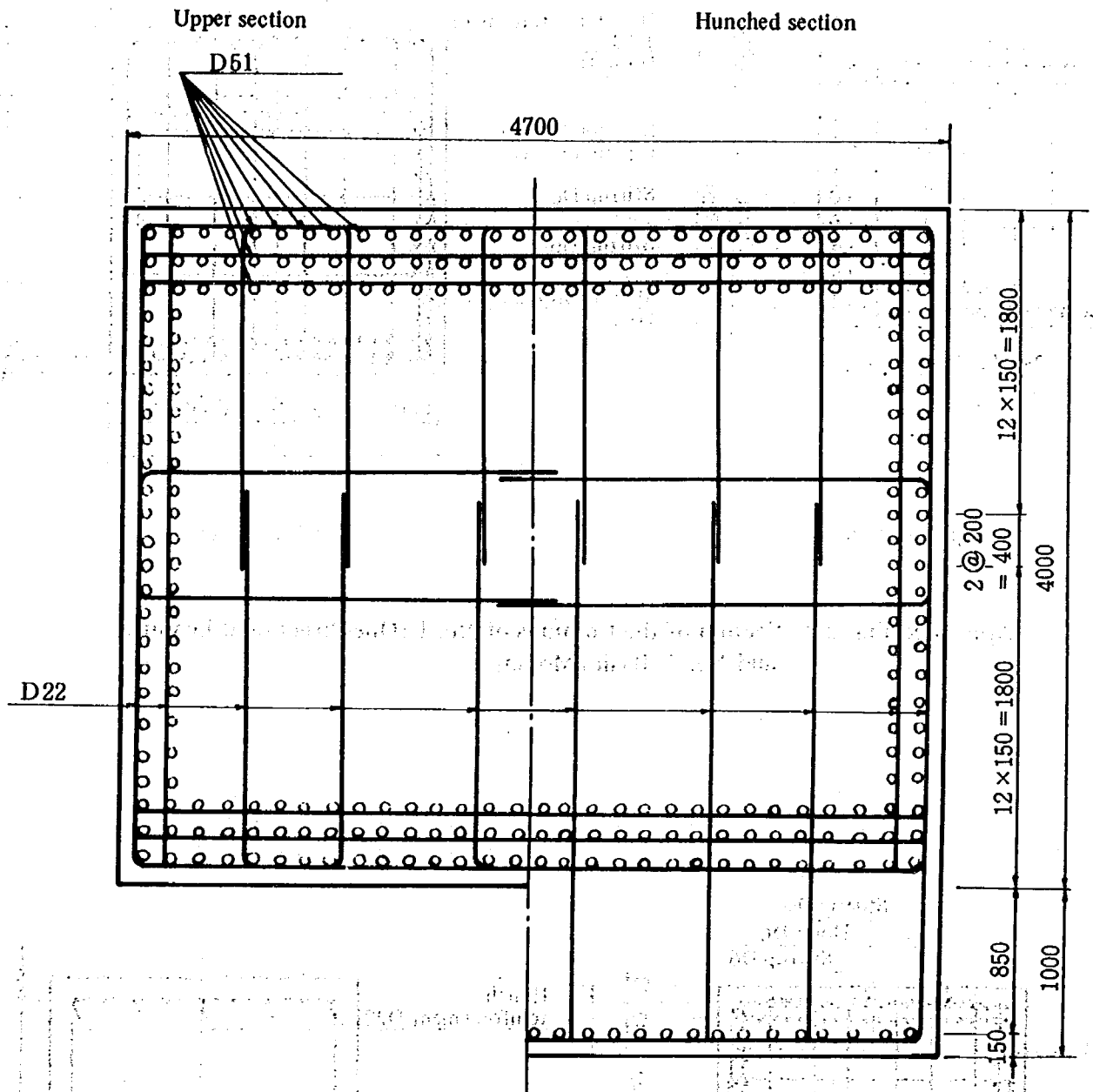
Appended Fig. 1-3 Control System Diagram

Appended Table 2-1 Outlines of Models

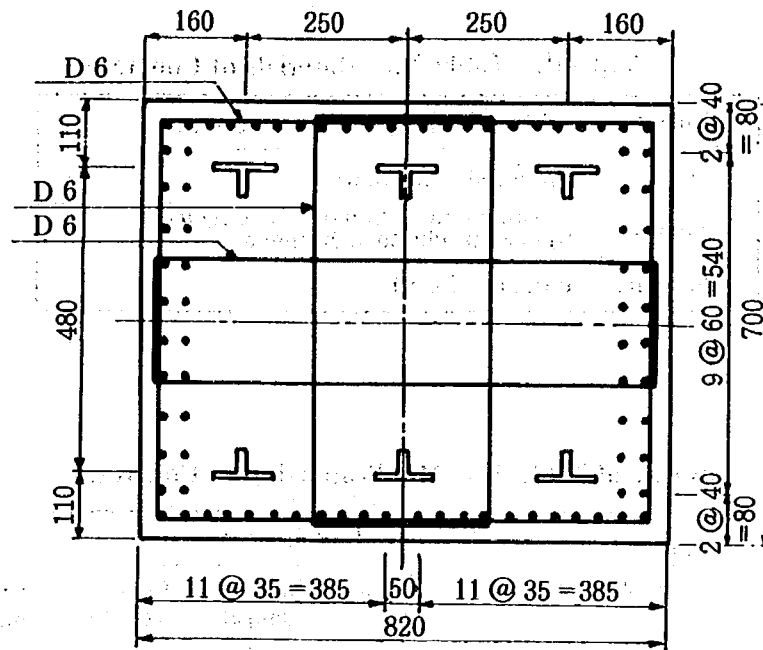
No.	Scale	Column height	Model shape	Hunch	Arrangement of reinforcement	Hoop		Spacing of stirrup	Axial load	Lateral load				Order of construction-		Provisional name												
						Reinforcement at the base of the column	Spacing			Velocity	Direction	Step	Cycle	Type	Order													
1	$\frac{1}{4\sqrt{2}}$	70cm	L-shape	○	Both sides, double arrangement-D13	○	D 6 $\begin{cases} 14\text{cm} \\ 7\text{cm} \end{cases}$	D 6 14cm	—	Static	One-directional loading (hunch-compression)	$n \delta_o$ $\leq \delta_u$	Three cycles	A ₁	1	One-directional loading												
2				—													B ₁	2	Without hunch									
3				○																One side, single arrangement-D29	D 13 $\begin{cases} 14\text{cm} \\ 11\text{cm} \end{cases}$	—	A ₂	3	Basic model			
4					B ₂		4	large-diameter reinforcement without side reinforcement																				
5																	Both sides, CT steel frame + single arrangement-D13	D 6 $\begin{cases} 14\text{cm} \\ 7\text{cm} \end{cases}$	D 6 14cm Diagonal member; ϕ 13 16 19 25							A ₃	5	SRC, without stud
6																												
7	$\frac{1}{4}$	100 cm	L-shape	○	Both sides, triple arrangement-D13	○	D 6 $\begin{cases} 10\text{cm} \\ 5\text{cm} \end{cases}$	D 6 10cm	○ (10 kg/cm ²)	Static	Reversed loading	$n \delta_o$ $\leq \delta_u$	Ten cycles	C	7	1/4 model												

Appended Table 2-2 Steel Materials of the Upper Column Model

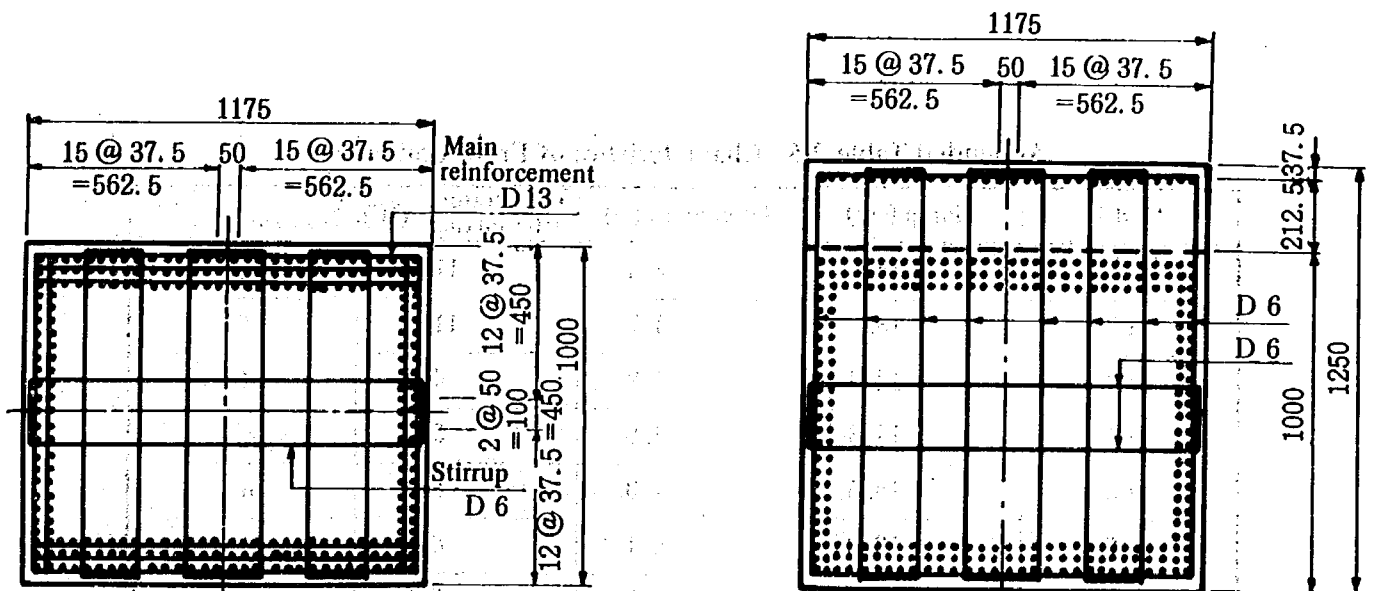
Model	(Width·h) × (Effective depth·d)	Longitudinal reinforcement				Hoop				Stirrup		
		Assort- ment	φ (a _s)	Number, row, set	$\frac{(\text{number})(\text{row}) \cdot a_s}{b \cdot d}$	Rein- forced section	φ (a _s)	Spacing·S	$\frac{2 a_s}{b \cdot s}$	φ (a _s)	(number)(spacing) n s	$\frac{n \cdot a_s}{b \cdot s}$
1.	(1/4√2)	Main reinforcement	D13 (1.267 cm)	22 × 2 × 2	1.062 %	—	D 6 (0.3167 cm)	14 ^{cm}	0.055 %	D 6	6 , 14cm	0.166 %
2.	Model	Side reinforcement		10 × 2 × 2	0.483							
3.		Hunch reinforcement		15 × 1 × 1	(0.362)			7	0.110			
4.	(1/4√2) Large-dia- meter rein- force- ment	Main reinforcement	D29 (6.242 cm)	9 × 1 × 2	1.102	—	D13	14	0.221	—	—	—
		Side reinforcement	—	—	—							
		Hunch reinforcement	D22 (3.871 cm)	5 × 1 × 1	(0.359)			11	0.281			
5.	(1/4√2)	Main reinforcement	D13	26 × 1 × 2	0.609	—	D 6	14	0.055	D 6	2 , 14cm	0.055
6.	S R C	Side reinforcement		10 × 2 × 2	0.483							
		Hunch reinforcement		15 × 1 × 1	(0.362)							
		Main steel frame	CT 100 × 50 (10.95 cm)	3 × 1 × 2	0.679	○		7	0.110	Diagonal member: φ 13, 16, 19, 25	3 , 48cm	0.110
7.	(1/4)	Main reinforcement	D13	30 × 3 × 2	1.049	—	D 6	10	0.054	D 6	6 , 10cm	0.162
Model	117.5 × 92.5	Side reinforcement		19 × 2 × 2	0.443							
		Hunch reinforcement		30 × 1 × 1	(0.350)			5	0.108			
Prototype	470 × 365	Main reinforcement	D51 (20.27 cm)	30 × 3 × 2	1.065	—	D22 (3.871 cm)	30	0.055	D22	6 , 30cm	0.165
		Side reinforcement		19 × 2 × 2	0.449							
		Hunch reinforcement		30 × 1 × 1	(0.354)			15	0.110			



Appended Fig. 2-1 Section of the Upper Column of Ban-no-su Bridge Pier



Appended Fig. 2-5 Section of the Columns of No. 5
(SRC-Without Stud) and No. 6 (SRC-With Stud) Models



Appended Fig. 2-6 Section of the Column of No. 7 (1/4) Model

Appended Table 2-3 Materials of Concrete

Cement	High-early-strength Portland cement
Sand	Produced at Kashima
Gravel	Maximum dimension of coarse aggregate is 10 mm, produced at Kashima
Admixture	Pozzolis, No. 5 L

Appended Table 2-4 Mix Proportion of Concrete

Air content (%)	Water-cement ratio W/C	Sand percentage S/a (%)	Unit content (kg/m ³)					Applied model
			Cement C	Water W	Fine aggregate S	Coarse aggregate G	Admixture pozzolis No. 5 LA3	
4	65	47.6	242	157	907	1,004	0.605	No. 1, No. 2
4	60	46.6	262	157	881	1,014	0.655	No. 3 ~No. 7

Appended Table 2-5 Characteristics of Fresh Concrete

Model	Slump (cm)	Air content (%)	Temperature after mixing (°C)	Atmospheric temperature at placing time (°C)
No. 1	14.5	4.4	11.0	10
No. 2	13.9	4.7	11.0	10
No. 3	14.3	4.9	4.0	4
No. 4	14.3	4.8	5.0	4
No. 5	13.5	4.3	8.0	6
No. 6	13.0	4.1	8.0	6
No. 7	13.8	4.3	13.0	8

Appended Table 2-6 Characteristics of Hardened Concrete

(average of three test pieces, unit: kg/cm²)

Age Strength Curing	Loading test time		28th day			
	Compressive strength		Compressive strength		Tensile strength	Bending strength
	Field water curing	Standard water curing	Field water curing	Standard water curing	Standard water curing	Standard water curing
No. 1	230	283	230	277	23	49
No. 2	234	285	—	270	—	—
No. 3	223	273	238	278	24	38
No. 4	230	270	—	271	—	—
No. 5	280	315	295	325	25	48
No. 6	300	320	—	323	—	—
No. 7	269	292	301	326	26	44

Appended Table 2-7 Characteristics of Steel Materials

Member	Model, steel material			Diameter, etc.	Quality	
Column	1.	$(\frac{1}{4}\sqrt{2})$ Model	Main reinforcement	D 13	S D 30	
	2.		Side reinforcement			
	3.		Hunch reinforcement			
		Hoop	D 6			
		Stirrup				
		Base reinforcement	D 13, D 16			
	4.	$(\frac{1}{4}\sqrt{2})$ Large-diameter reinforcement model	Main reinforcement	D 29		
			Hunch reinforcement	D 22		
			Hoop	D 13		
			Base reinforcement	D 22		
5.	$(\frac{1}{4}\sqrt{2})$ SRC model	Main reinforcement	D 13			
		Side reinforcement				
		Hunch reinforcement				
		Hoop	D 6			
		Stirrup				
6.		Base reinforcement	D 13, D 16			
		Main steel frame	C T 100 × 50	S S 41		
		Diagonal reinforcement	φ 13, φ 16 φ 19, φ 25	S R 24		
Footing	1 ~ 6		Longitudinal reinforcement	D 13, D 22	S D 30	
			Transverse reinforcement	D 16		
Column	7.	$(\frac{1}{4})$ Model	Main reinforcement	D 13		
			Side reinforcement			
			Hunch reinforcement			
				Hoop		D 6
				Stirrup		
		Base reinforcement	D 13, D 16			
Footing	7.		Longitudinal reinforcement	D 16, D 22		
			Transverse reinforcement	D 22		

Appended Table 2-8 Mechanical Characteristics of Steel Materials (Standard Value)

Type	Symbol	Yield point	Tensile strength	Elongation
Hot rolled deformed bar	S D 3 0	Larger than 30 kg/mm ²	49~63kg/mm ²	No. 2; larger than 14% No. 3; larger than 18%
Structural rolled steel bar	S S 4 1	Larger than 25 kg/mm ²	41~52kg/mm ²	No.1 A; larger than 17%
Hot rolled steel bar	S R 2 4	Larger than 24 kg/mm ²	39~53kg/mm ²	No. 2; larger than 20%

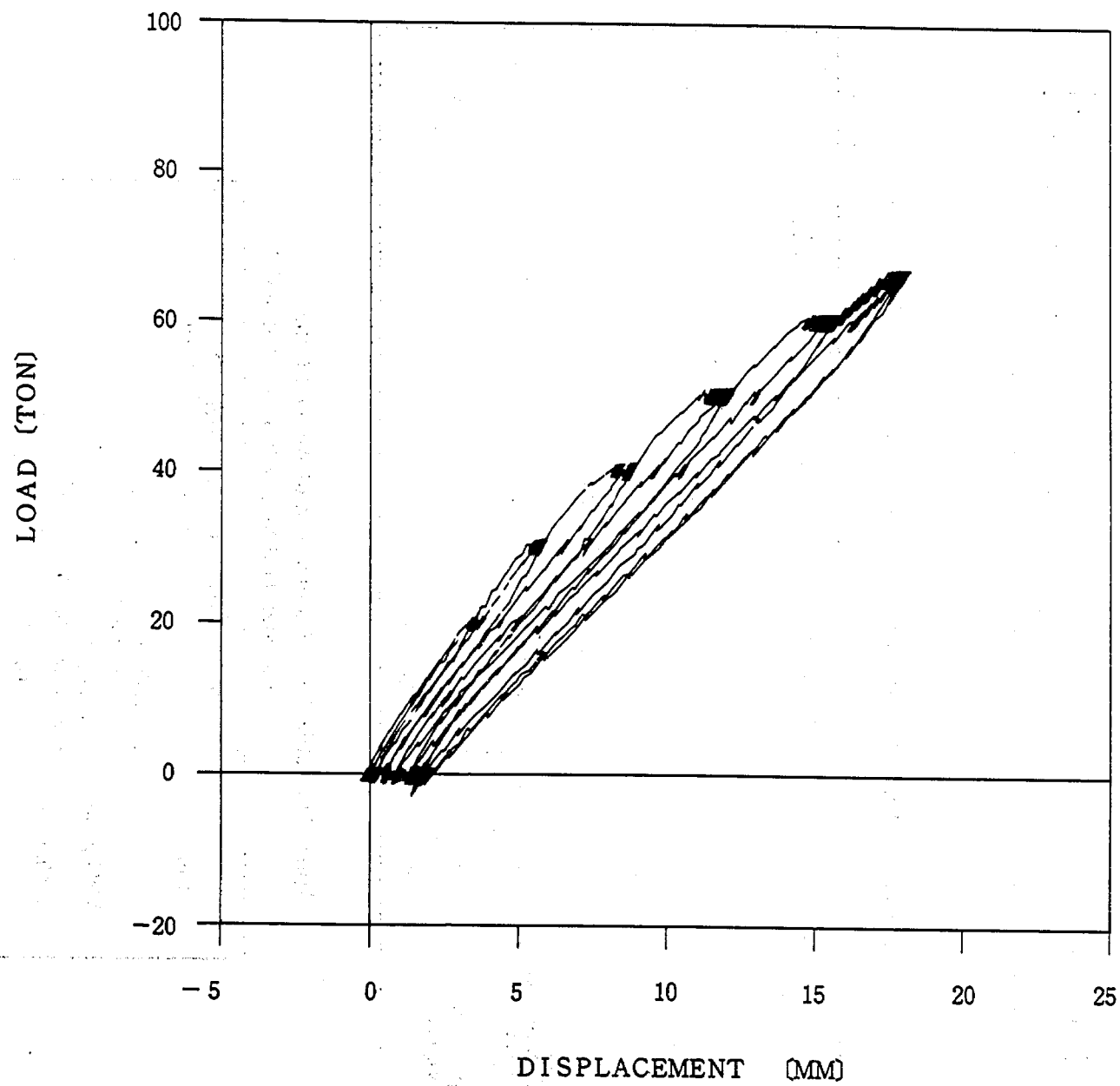
Appended Table 2-9 Results of the Tensile Test of Steel Materials

(average of three test pieces)

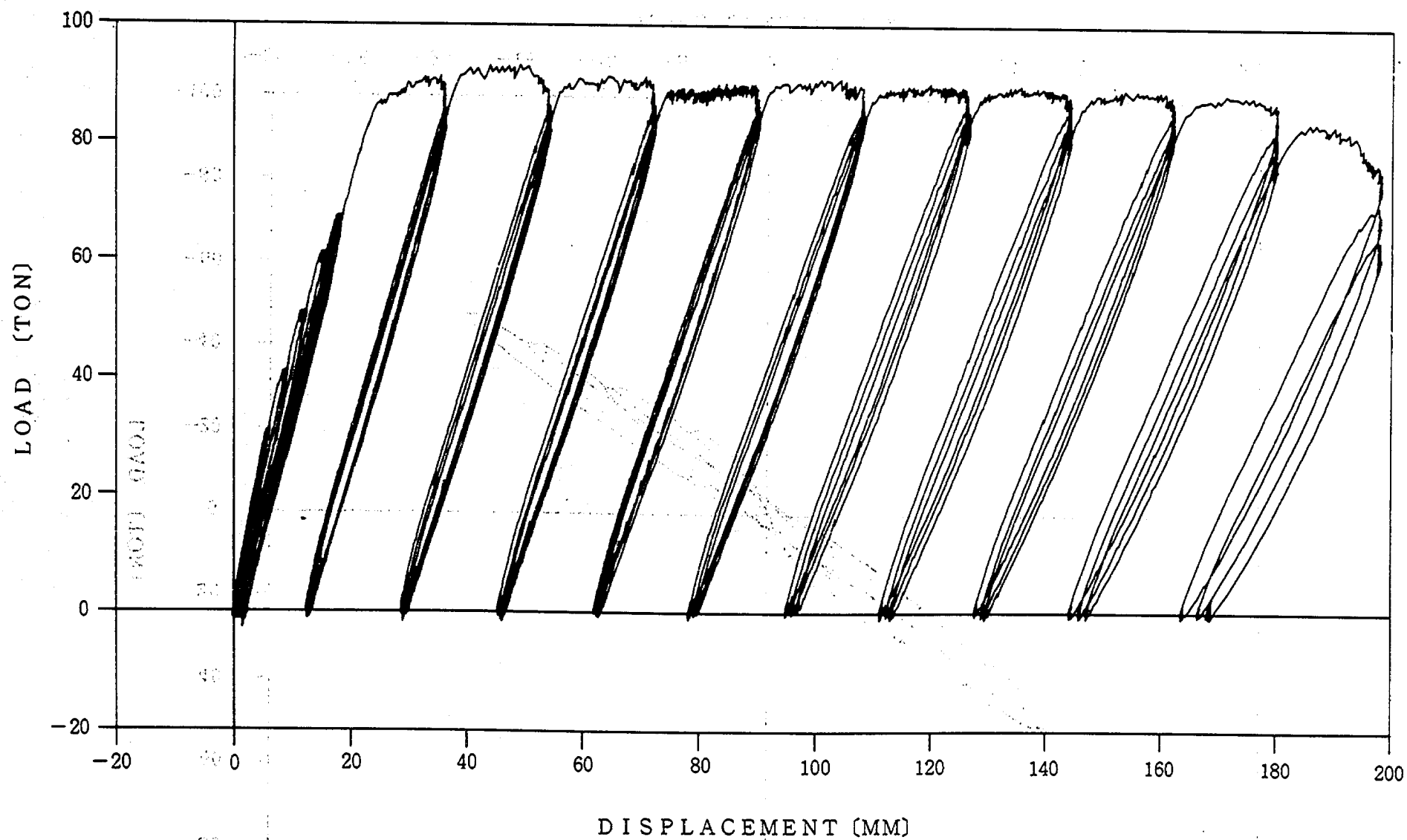
Materials		Test piece	Yield point(kg/mm ²)	Tensile strength (kg/mm ²)	Elongation (%)
Deformed bar (SD30)	D 6	No. 2	41	55	24. 2
	D 13	No. 2	37	55	24. 3
	D 22	No. 2	36	57	21. 3
	D 29	No. 3	36	58	28. 5
CT shape steel (SS41)	100 × 50 × 6 × 8	No. 1A	35	52	22. 7
Round bar (SR24)	φ 6	No. 2	38	52	29. 5
	φ 13	No. 2	33	46	33. 0

Appended Table 2-10 Comparison of Actual Strength and Standard Strength of Steel Materials

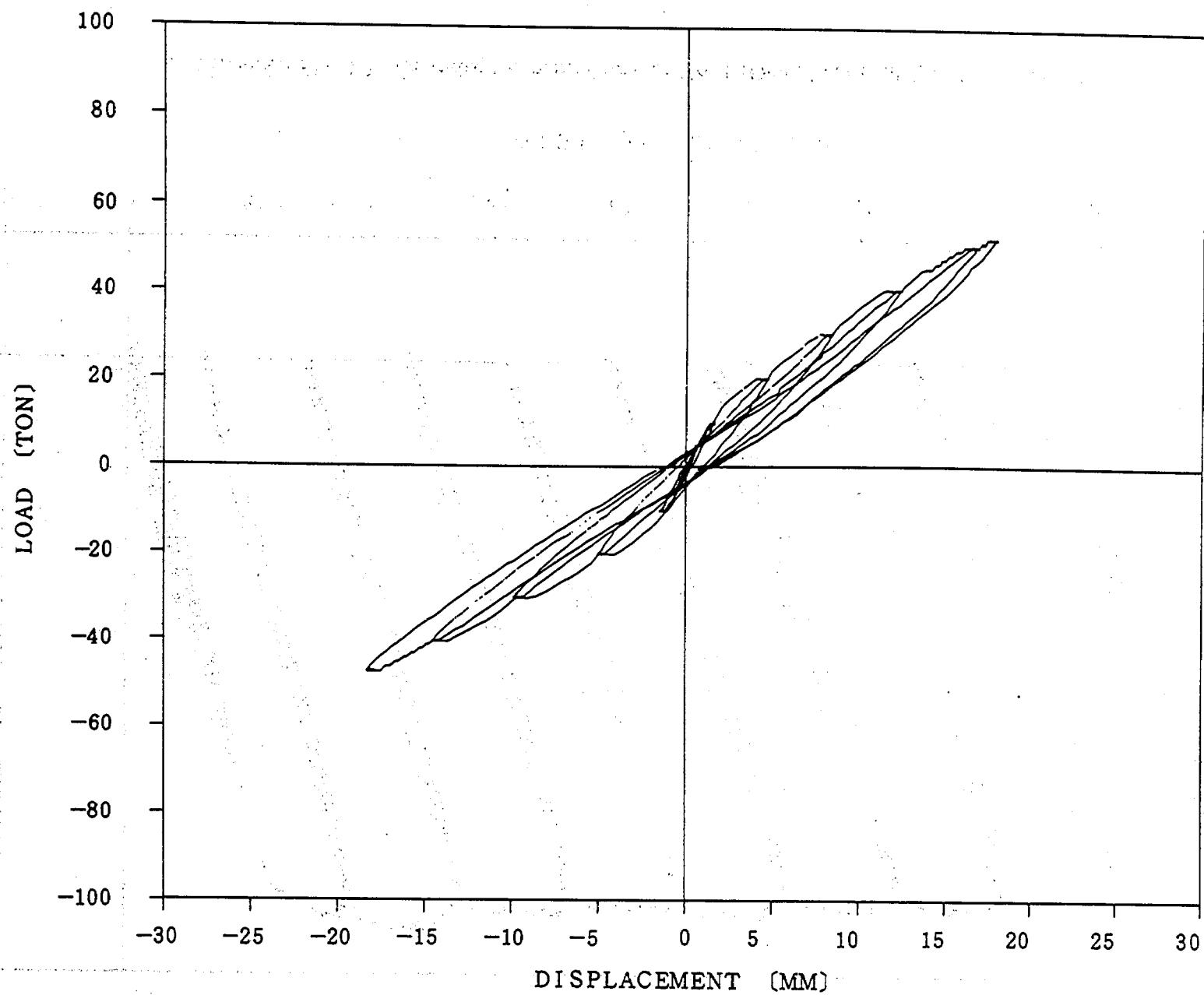
Steel material		CT shape steel SS41	D13 reinforcement SD30
Item			
Yield point (kg/mm ²)	① Result of tensile test	35	37
	② Standard value (lower limit)	25	30
	①/②	1. 40	1. 23
Tensile strength (kg/mm ²)	① Result of tensile test	52	55
	② Standard value (lower limit)	41	49
	①/②	1. 27	1. 12



Appended Fig. 3-1 Load-displacement Curve of No. 1 (One-directional Loading) Model (Up to δ_0)



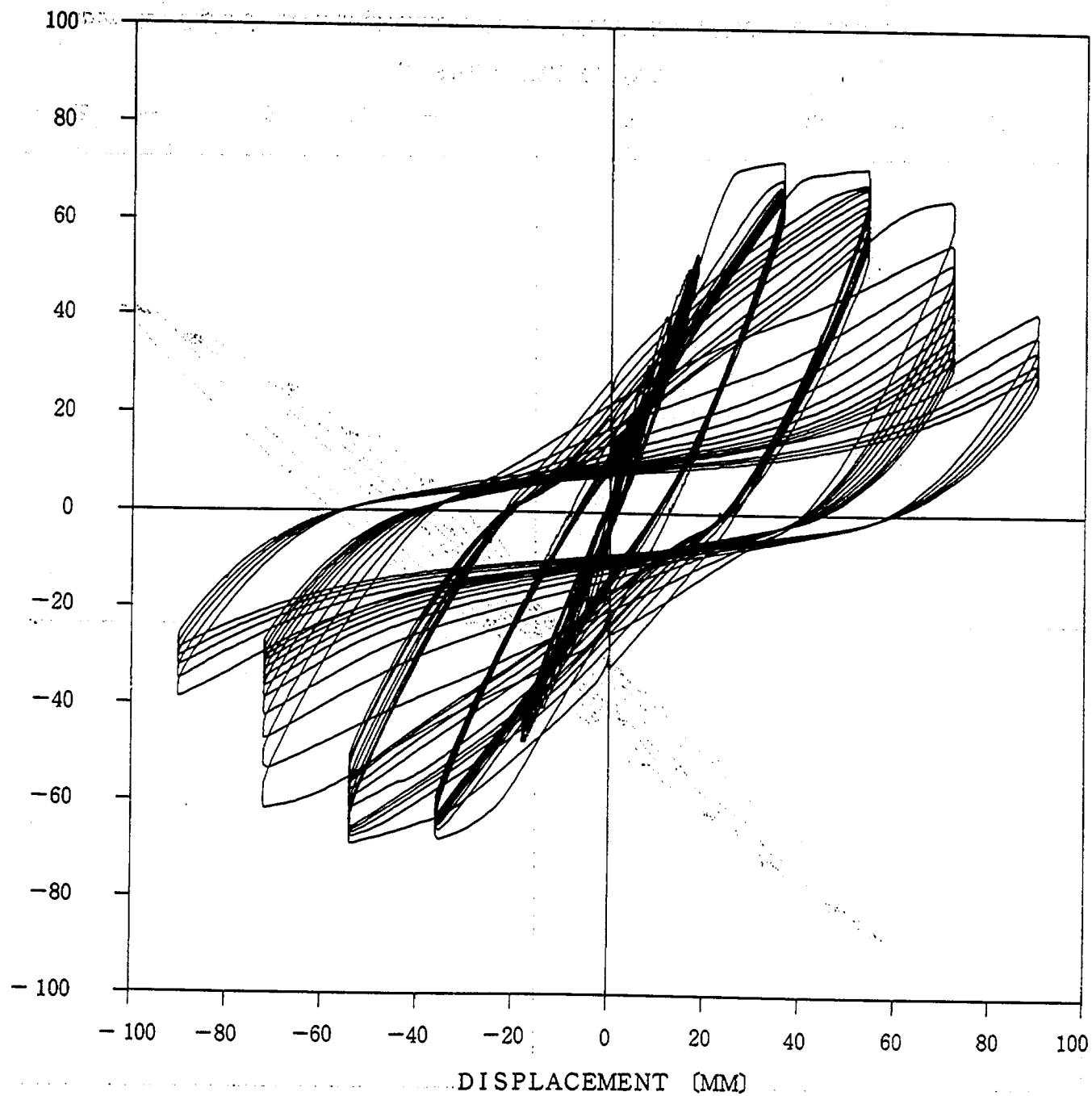
Appended Fig. 3-2 Load-displacement Curve of No. 1 (One-directional Loading) Model



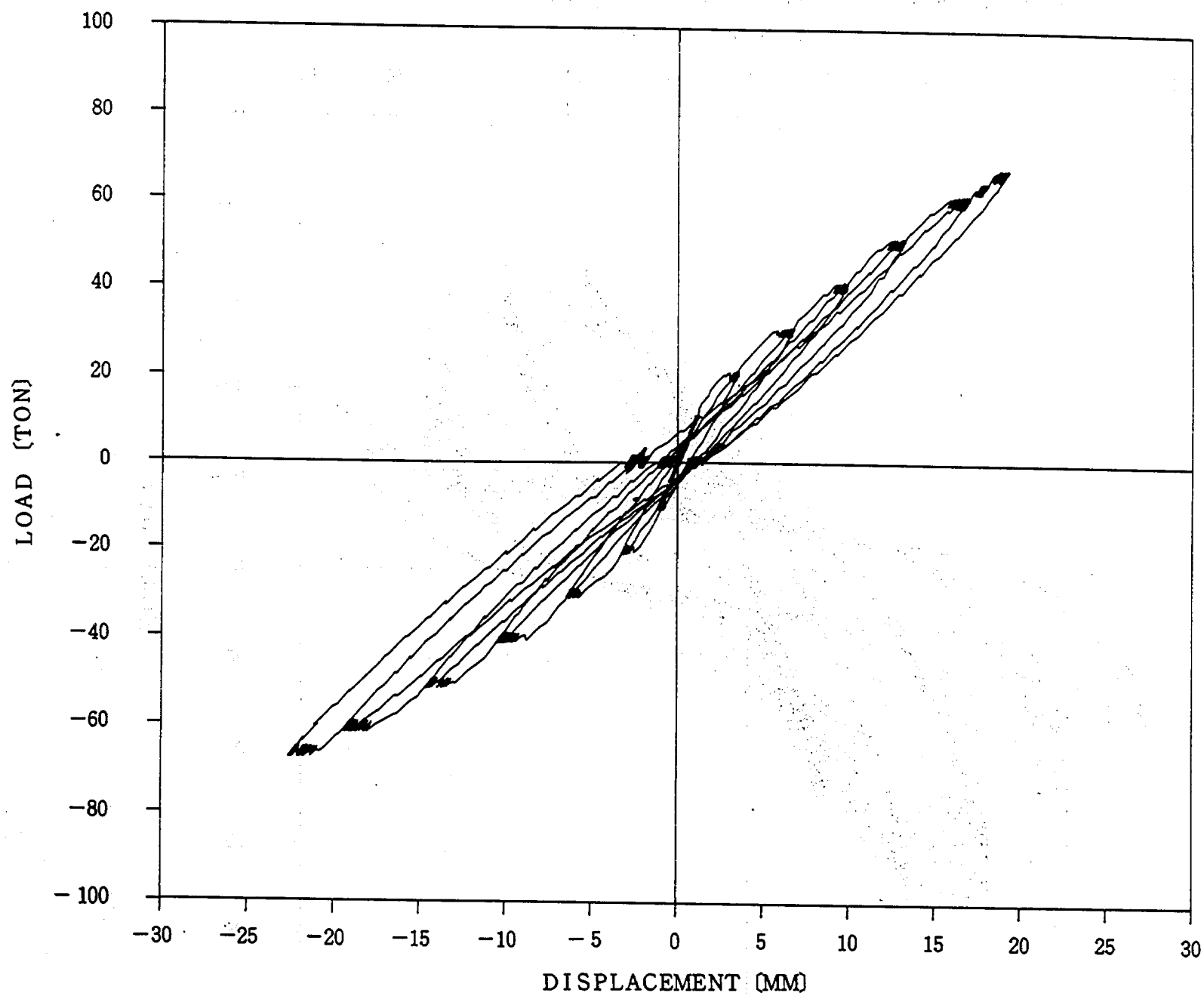
Appended Fig. 3-3 Load-displacement Curve of No. 2 (Without Hunch) Model (Up to δ_0)

LOAD (TON)

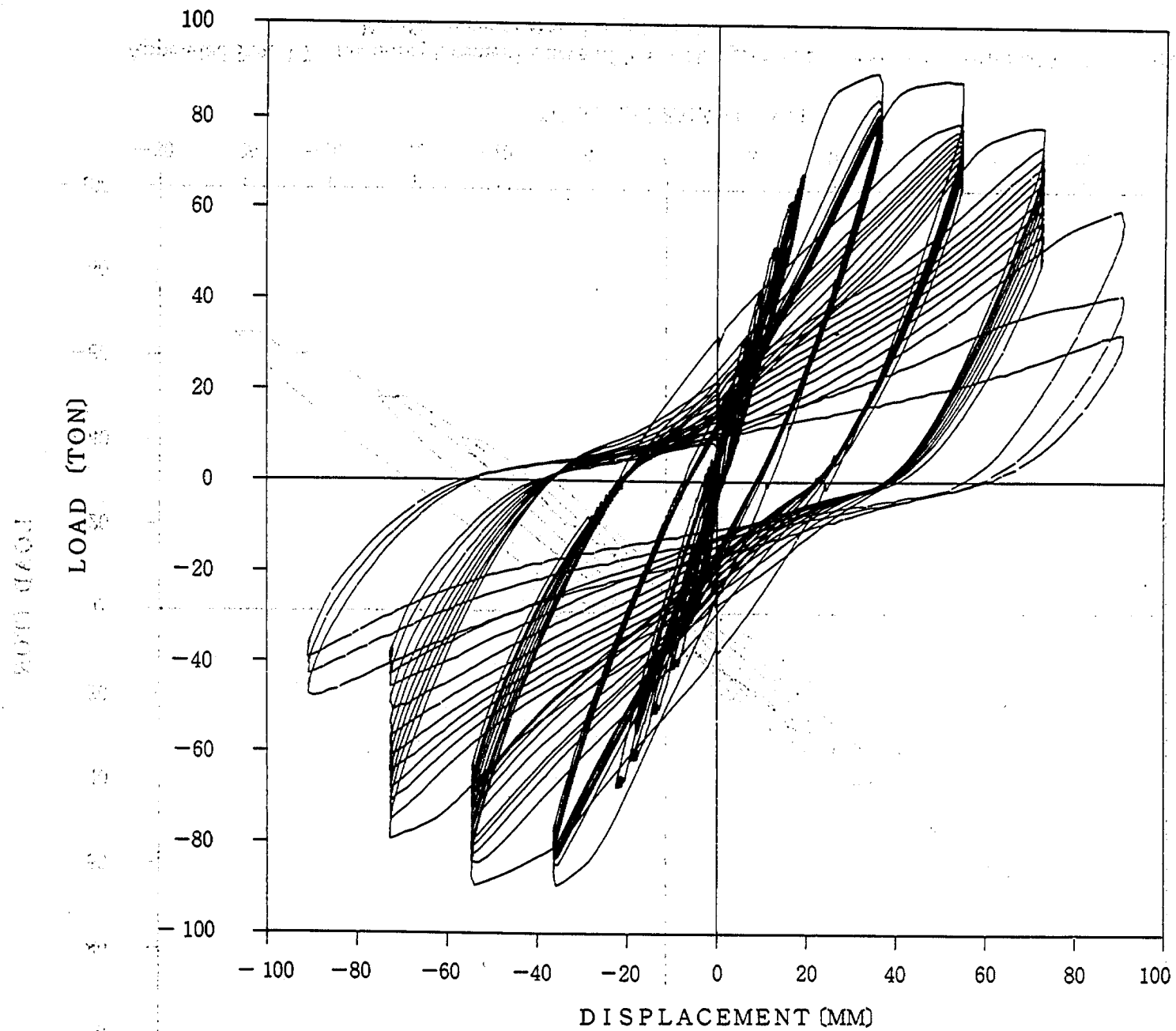
LOAD (TON)



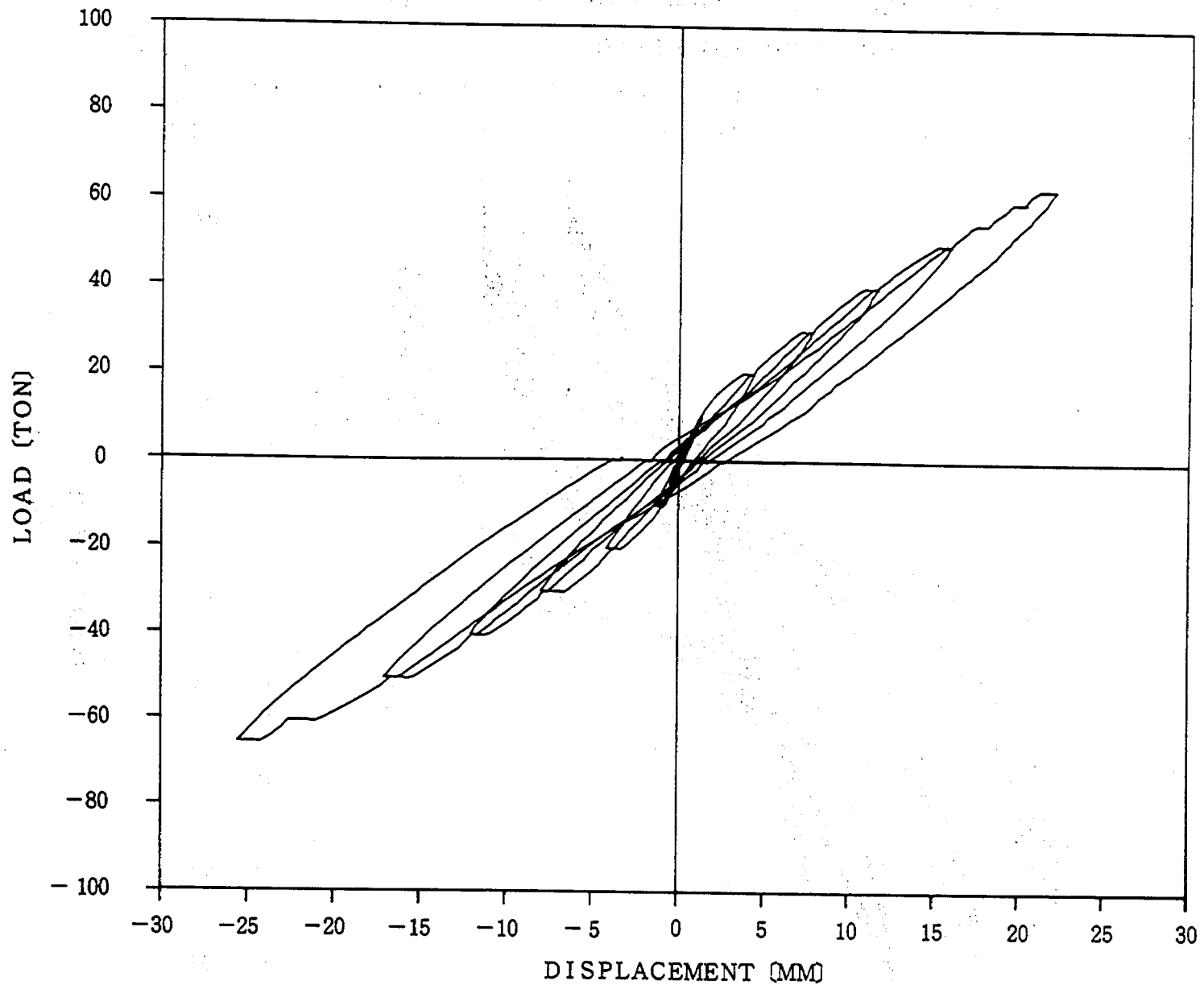
Appended Fig. 3-4 Load-displacement Curve of No. 2 (Without Hunch) Model



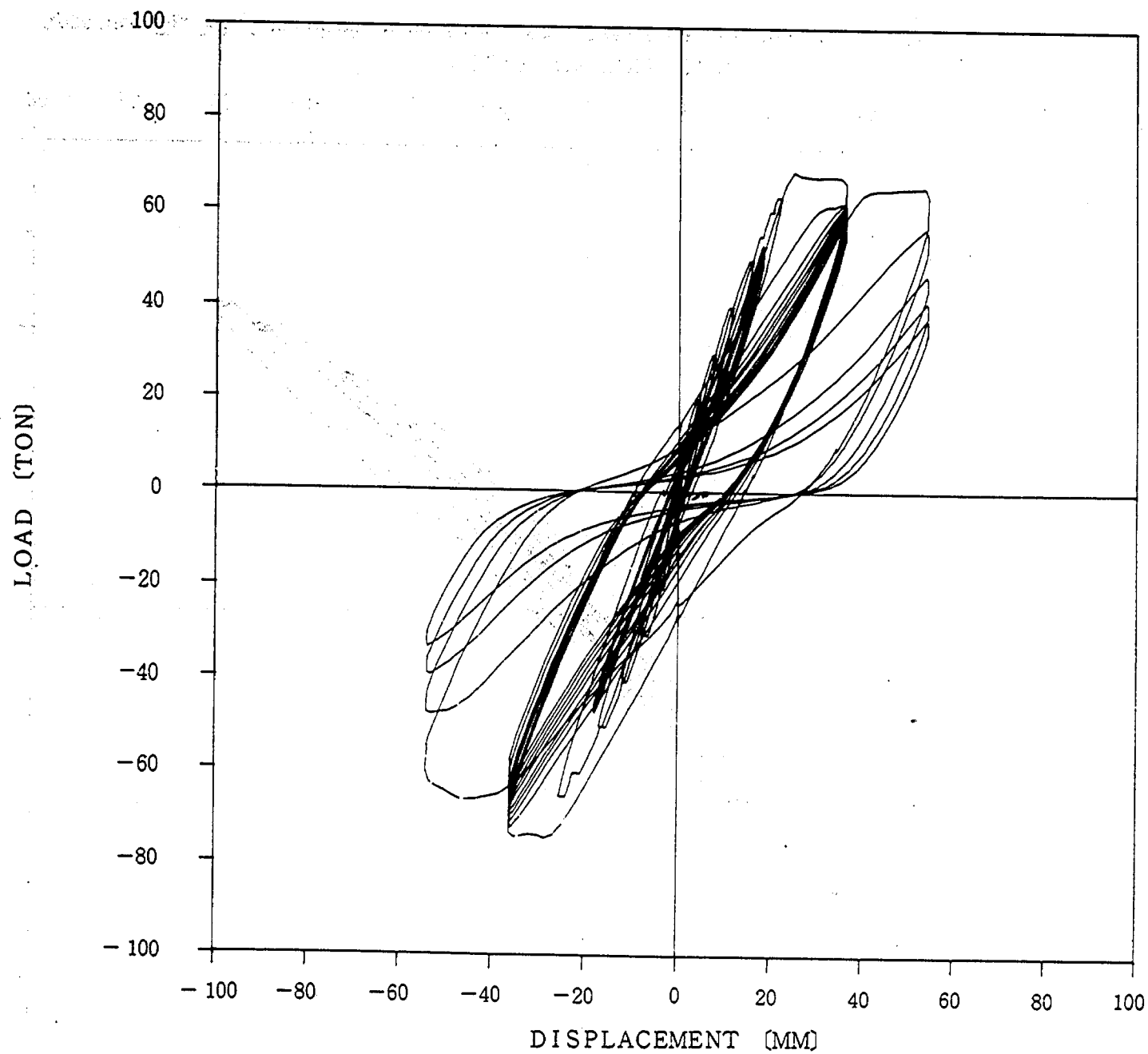
Appended Fig. 3-5 Load-displacement Curve of No. 3 (Basic) Model (Up to δ_0)



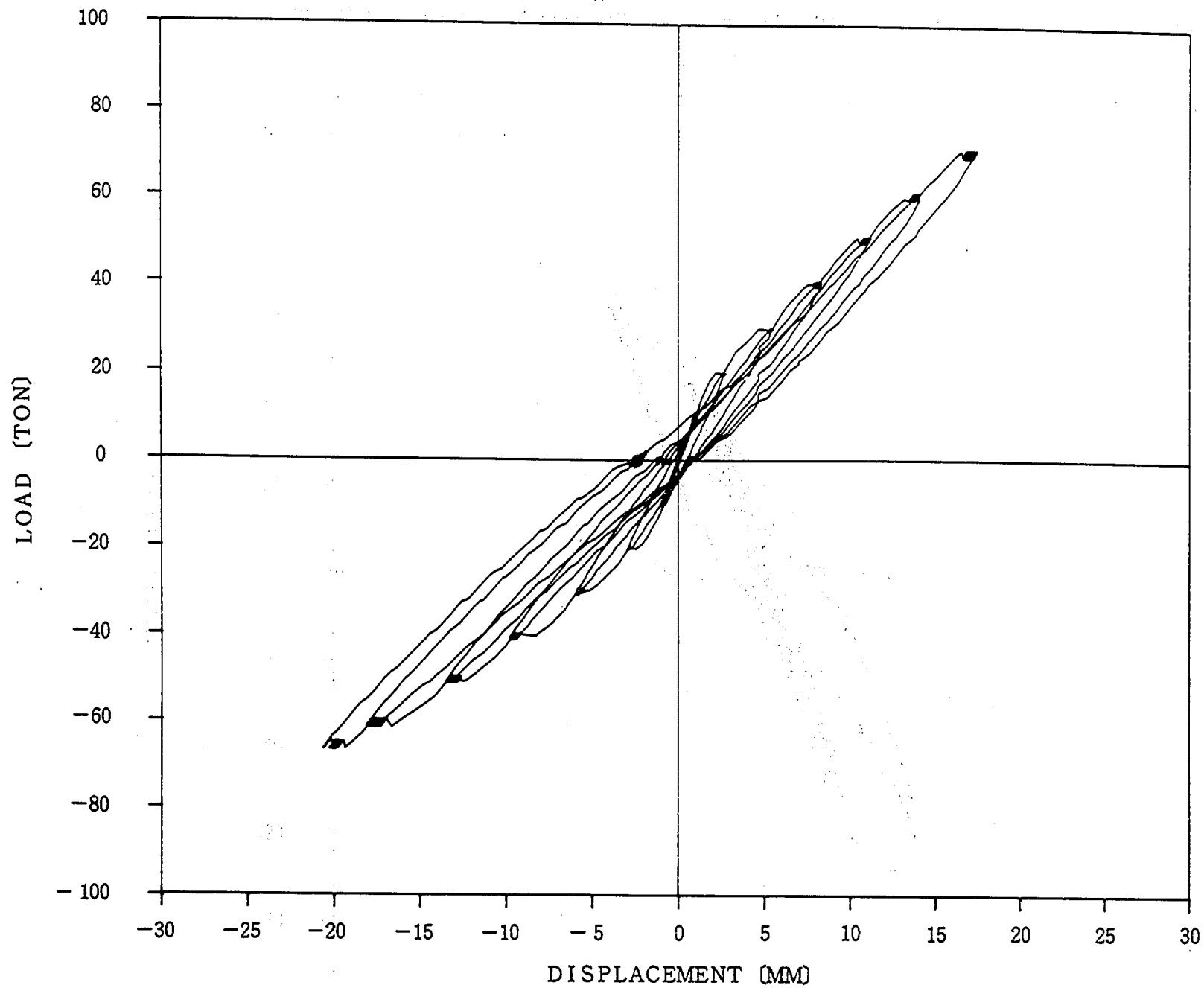
Appended Fig. 3-6 Load-displacement Curve of No. 3 (Basic) Model



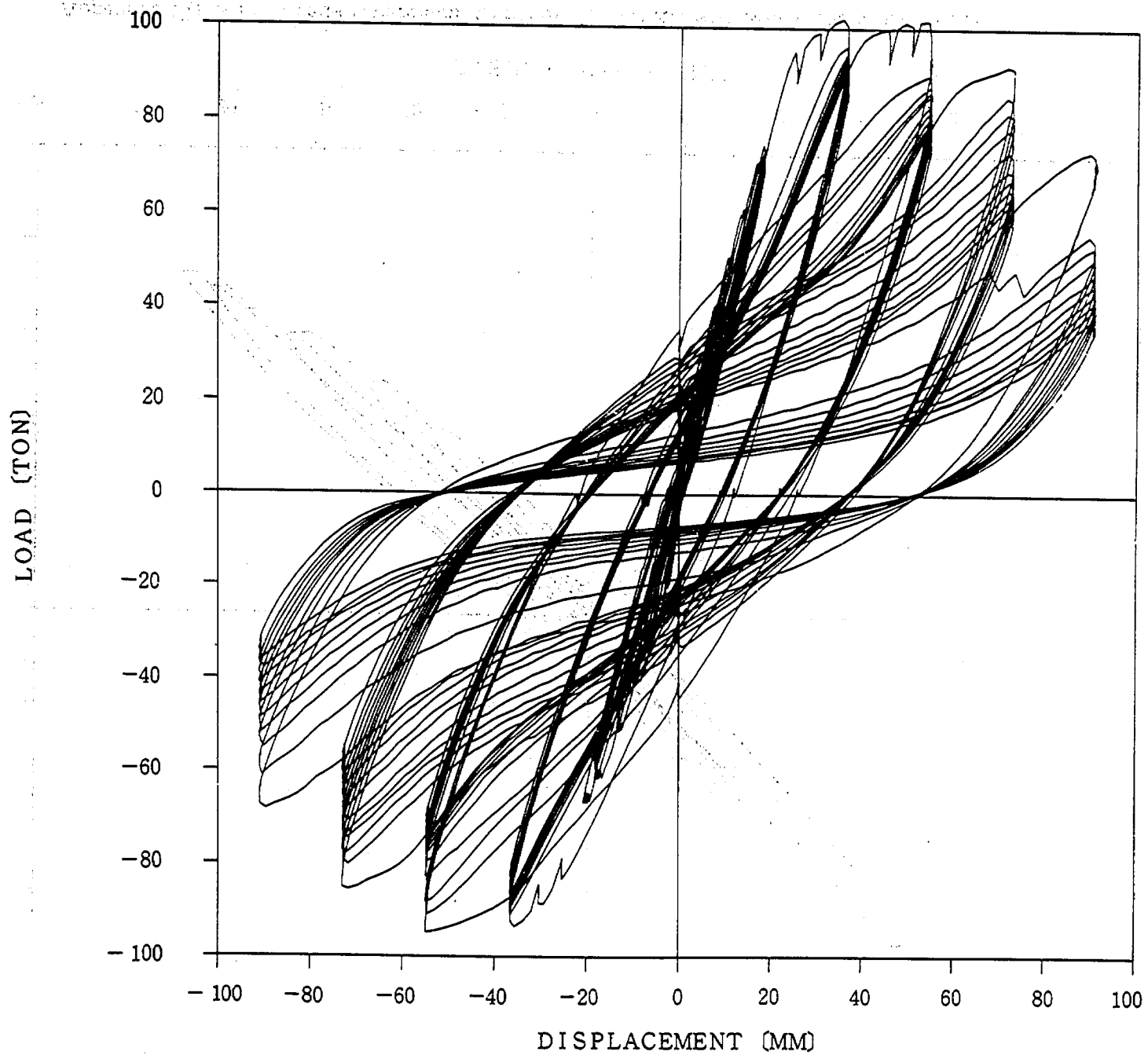
Appended Fig. 3-7 Load-displacement Curve of No. 4 (Large-diameter Reinforcement and Without Side Reinforcement) Model (Up to δ_0)



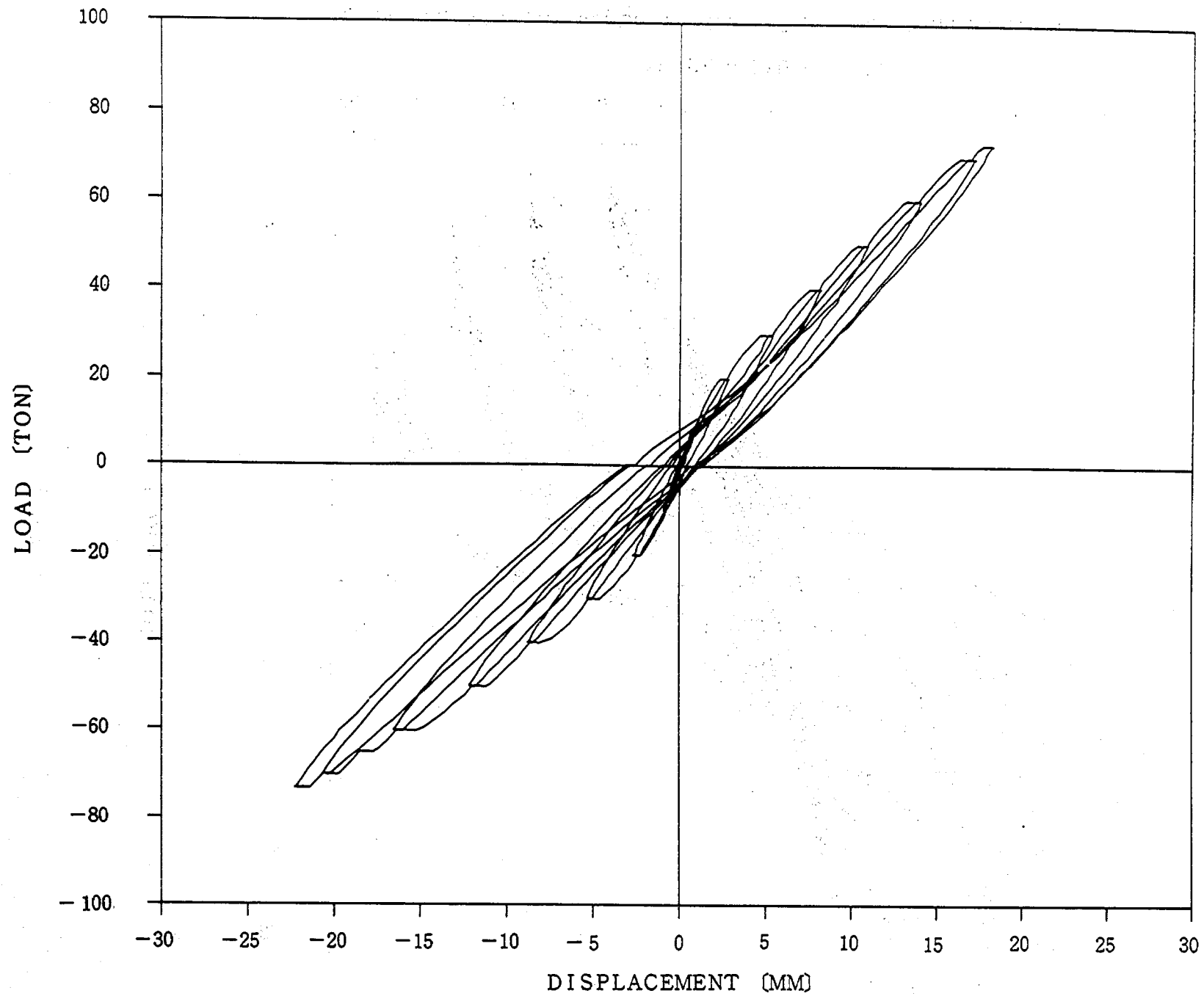
Appended Fig. 3-8 Load-displacement Curve of No. 4 (Large-diameter Reinforcement and Without Side Reinforcement) Model



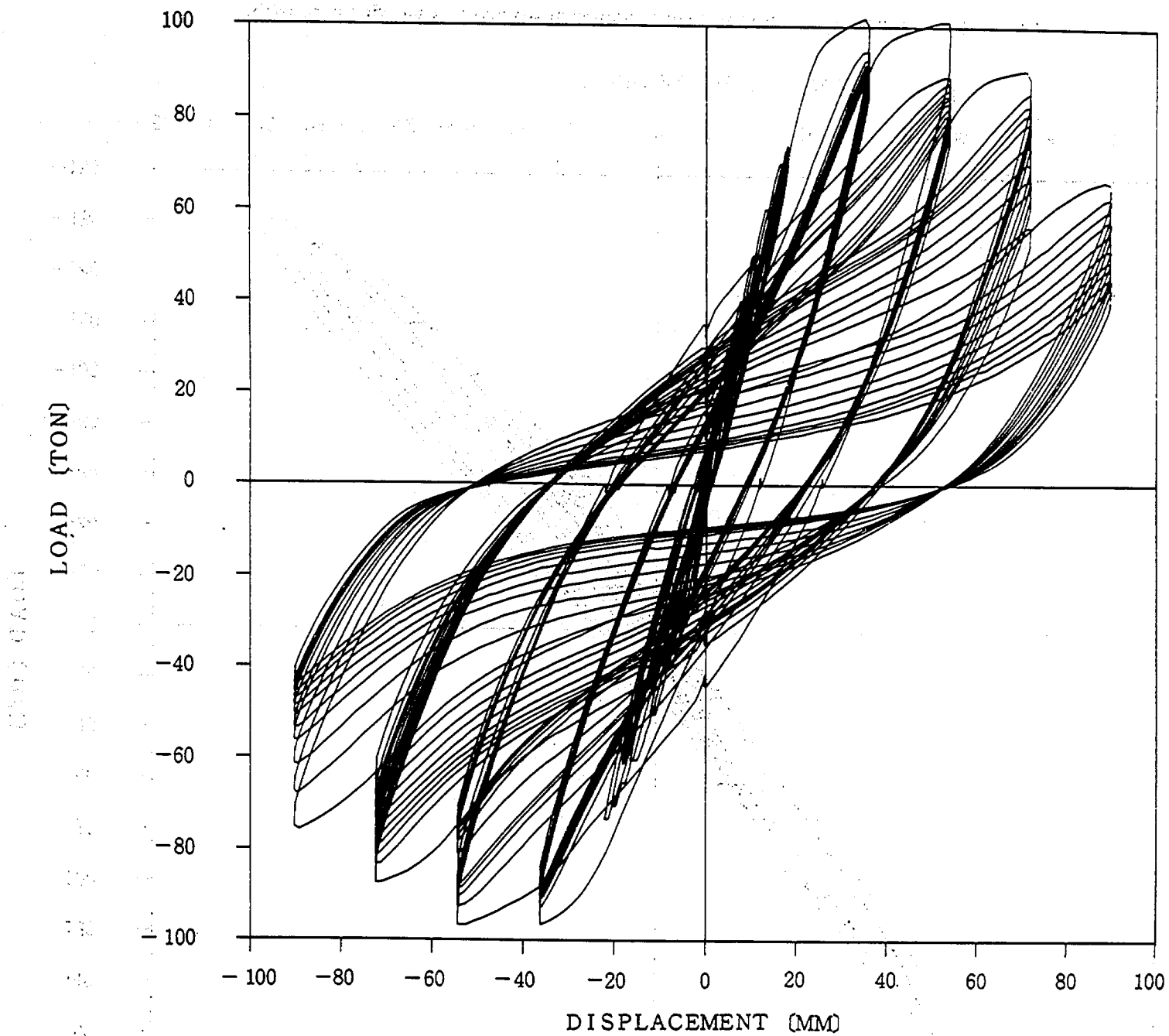
Appended Fig. 3-9 Load-displacement Curve of No. 5 (SRC-Without Stud) Model (Up to δ_0)



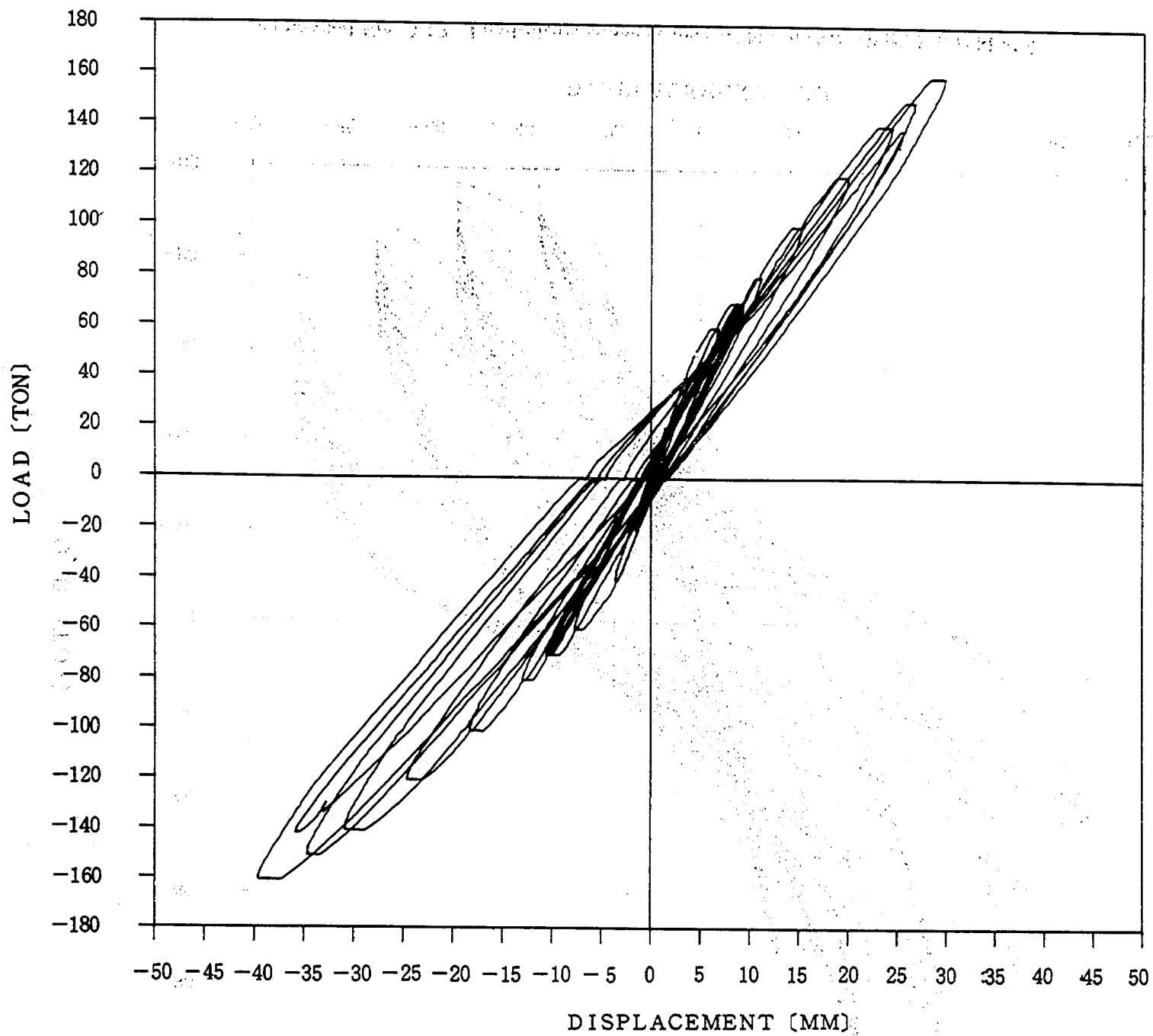
Appended Fig. 3-10 Load-displacement Curve of No. 5 (SRC-Without Stud) Model



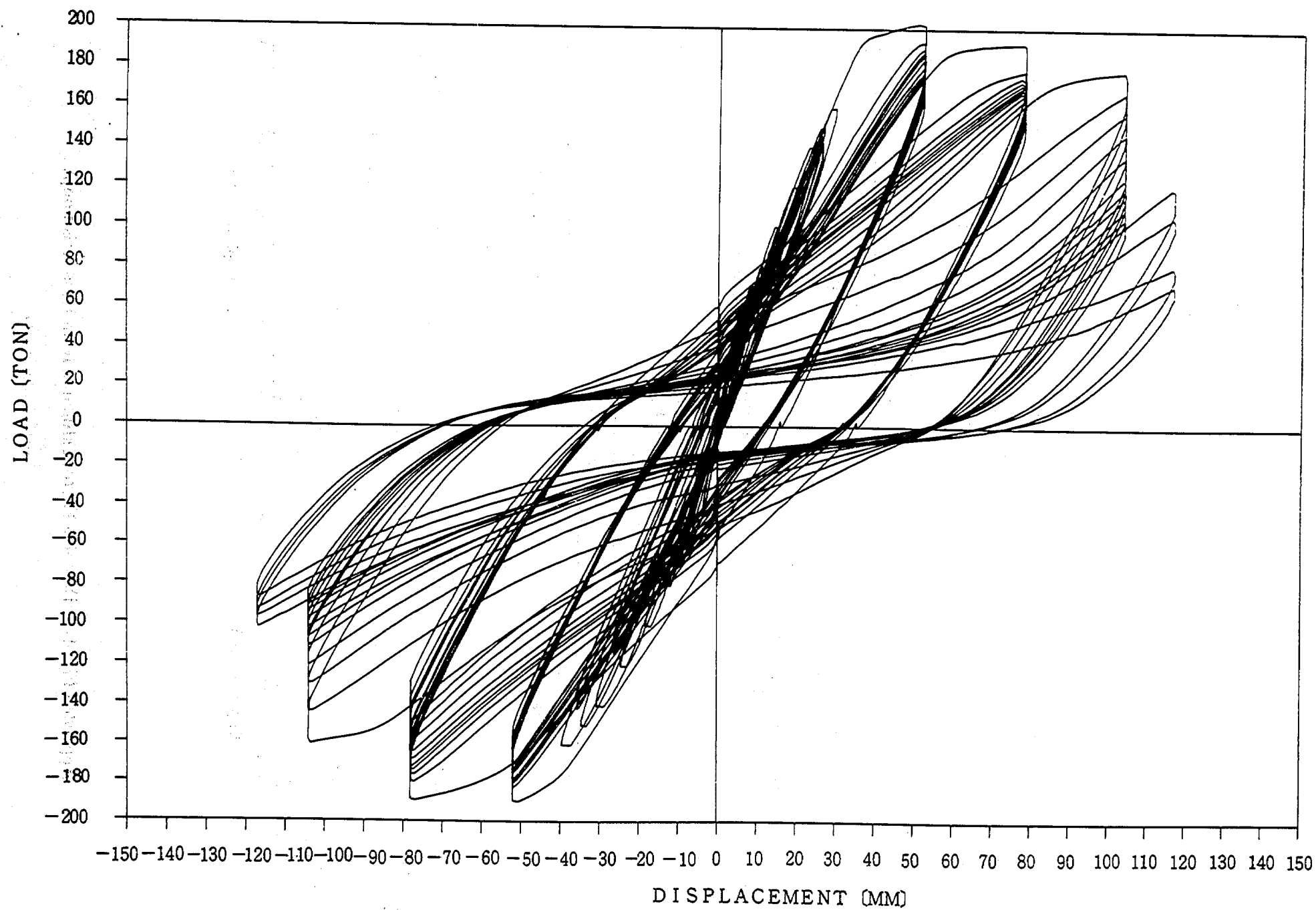
Appended Fig. 3-11 Load-displacement Curve of No. 6 (SRC-With Stud) Model (Up to δ_0)



Appended Fig. 3-12 Load-displacement Curve of No. 6 (SRC-With Stud) Model



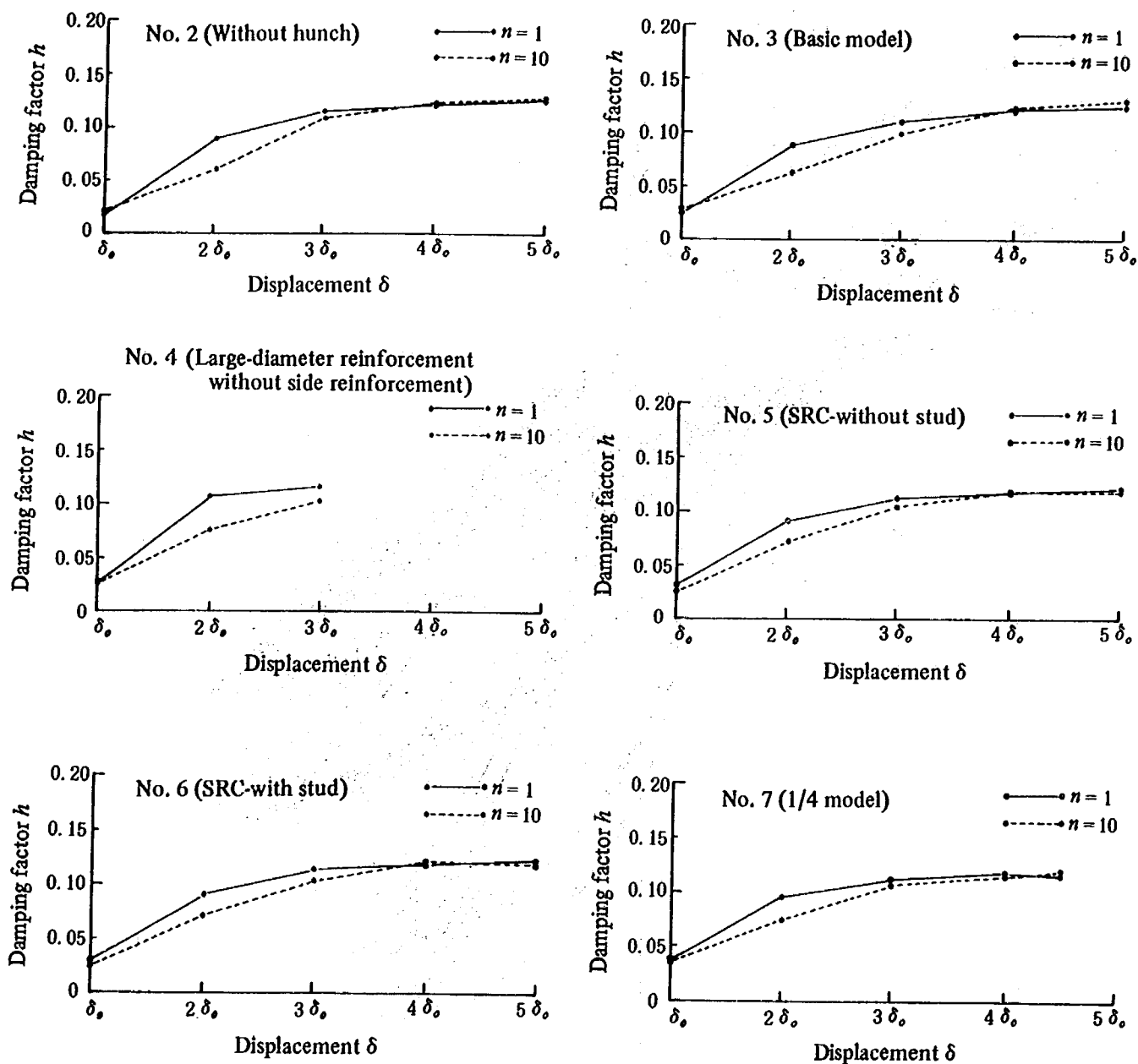
Appended Fig. 3-13 Load-displacement Curve of No. 7 (1/4) Model (Up to δ_0)



Appended Fig. 3-14 Load-displacement Curve of No. 7 (1/4) Model

Appendix – 4 Damping Factors Obtained by the Experiment

Equivalent Damping factors estimated from the hysteresis loops as shown in Appendix–3 obtained by the reversed cyclic loading tests are shown in Appended Fig. 4-1.



Appended Fig. 4-1 Damping Factor Estimated from Hysteresis Loops

REVERSED CYCLIC LOADING TEST
OF BRIDGE PIER MODELS

PWRI TECHNICAL MEMORANDUM
No. 1801
Oct. 1981
Public Works Research Institute

Key Words:

RC·SRC Bridge Pier, Aselsmicity,
Model Reversed Cyclic Loading Test

Synopsis:

The report describes the results of reversed cyclic loading tests of RC and SRC bridge pier models.

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