NONLINEAR EARTHQUAKE RESPONSE OF EQUIPMENT SYSTEM ANCHORED ON R/C BUILDING FLOOR

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SUMMARY

The objective of this study was to investigate nonlinear behavior of support structure of equipment structural systems installed on reinforced concrete buildings during an earthquake. For that purpose, an idealized structural system was chosen and nonlinear response under severe earthquake condition was simulated by the IIS Computer-Actuator On-line System.

INTRODUCTION

Existing equipment systems are of various kinds. The support structures of such systems have also wide varieties. Here an attempt was made to idealize such support structures into a simple comprehensive model structure. An equipment system supposed as a prototype is shown in Fig. 1. The system is installed on the reinforced concrete floor of a building. The equipment itself is considered only a rigid mass for simplicity. The simulation is carried out twofold; firstly,cyclic load tests were done along predetermined column-top displacement loading program and secondly, load tests were conducted tracing the response displacements which were calculated on the basis of restoring forces measured. The latter simulation system is called "IIS Computer Actuator Online System (Ref. 1-6)"

TEST STRUCTURES AND TEST SETUP

Test Structures

The test structures investigated were canti-lever type columns fixed on the reinforced concrete thick slabs. In the tests, a lateral load was applied in the direction normal to the column axes so that bending moment and shear force were produced at the column base. Test variables were the size, the embedment depth and the location arrangement of anchor bolts. As to the size, the rod diameters of all bolts were 22mmϕ, but the major thread diameter was specified as 27mmϕ for some of test structures which were distinguished by U in the specimen codes. The embedment depth was chosen to be 128mm and 208mm. These length-to-diameter ratios are 5.8 and 9.5 and designated by S and L in the code, respectively. The number of bolts was 4 or 6, which can be noticed in the code. The anchor head ring plates of 35mmϕ 12mm, confirming to the Japanese Industrial Standards for studs, were welded to the bolts. The details of the test structures are shown in Fig. 2.

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Test Setup

The test structures were fixed on the reaction wall in the laboratory by high tensile bolts as shown in Fig. 3. Therefore, the load was applied to the column top by the hydraulic actuator (jack). The displacement of the actuator head was always controlled, according to the pre-determined displacement program in the cyclic tests and the simultaneously calculated response displacement in the on-line simulation. The load applied to the column top was measured electrically by the load cell mounted on the actuator head. The displacement of the column top denoted by X_2 was measured by the differential transformer. These data were converted into the digital form and recorded in magnetic tapes.

CYCLIC LOADING TESTS

The cyclic load tests was carried out for four structural model specimens along the predetermined column-top displacement sequence. The loading programs are shown in Fig. 4. For specimen (I-1-4S), one cycle static load test was done by the load control loading at the amplitude of 2.5tons. Then cyclic loading was continued in five cycles at the same amplitude. Thereafter, fifteen cycles were followed at the displacement amplitude of 3.0cm. As for specimen (II-1-4L), (IIU-1-4L) and (Ⅲ-1-6L), the same procedure as (I-1-4S) was repeated till the amplitude of 3.0cm, but 5c., 10c. and 50c. cyclic tests were followed at the displacement amplitude of 5.0cm. One cycle static load test was preceded each cyclic test. The lateral load Q versus the displacement X_2 relationships at the tops of the column specimens (I-1-4S) and (Ⅲ-1-6L) are shown in Fig. 5(a) to (b). The maximum displacements X_{max} attained, the yield lateral load Q and the maximum Q_{max} observed in the tests are summarized in Table 1.

ON-LINE TESTS

In the simulations by "IIS Computer-Actuator On-line System" the natural period of an assumed equipment system and the intensity of an input acceleration can be arbitrarily determined in reference to the stiffness and the strength of a test structure. The period of the fictitious building is 0.4 sec as described later. Then, two periods of 0.8 sec and 0.3 sec were considered in order to interpose 0.4 sec between them. The period of 0.8 sec was assigned to the first group of I-2-4S, Ⅲ-2-6L and the period of 0.3 sec to the second group of IU-3-4S, ⅢU-3-6L as shown in Table 1. The input acceleration used in the simulation must be the floor acceleration. It was assigned the elastic response acceleration of a single story building with the period of 0.4 sec and the dumping ratio of 2% to the EW component of HACHINOHE record in 1968 TOKACHI-OKI earthquake. The response shear force-displacement diagram are shown in Fig. 6. In the figures the values of yield strength Q_y observed are also indicated.

FAILURE MECHANISM AND ULTIMATE STRENGTH OF SUPPORT STRUCTURES

Failure Mechanism

Failure mechanisms oberved in the tests were;
(1) Pull out failure of slab concrete (Type S)

(2) Yielding in tension of anchor bolts (Type B)
(3) Yielding in bending of H-shaped column (Type H)
(4) Bearing failure of concrete at anchor head (Type C)

However, the bearing failure was not predominated, but associated with the flexural yielding of H-shaped column. Among their failure types, as shown in Fig. 7 and 8 the pull out failure (Type S) was most brittle and the bending yielding of H-shaped column was most ductile.

Calculation of Yield Strength

Yield strength for each failure type was calculated by a full plastic theory. An analytical model and a stress distribution are shown in Figs. 9(a) and (b). Assumptions in the calculation are;

a) Base plate is rigid,
b) Stress-strain relationships of concrete in tension and compression and anchor bolts are rigid-plastic,
c) Anchor bolt does not work in compression,
d) When one type of failure is predominated, other types of failure do not occur, and
e) Shape factors of concrete stress block (k_1,k_2,k_3) are; $k_1=0.85$, $k_2=k_1/2$, $k_3=0.85$ (Ref. 7)
f) The yield strength of H-shaped column is calculated by the full plastic moment concept.

Relation between Failure Mechanism and Strength

Calculated yield strength and observed strength in the tests were compared in Figs. 10 and 11. As seen in the table and the figures, the calculated values showed a good agreement with the observed value. The discrepancy between them was within 20%. For each specimen, the failure mode having the smallest strength is corresponding to the failure types observed in the test.

EARTHQUAKE RESPONSE OF EQUIPMENT SYSTEM

In Fig. 12 the elastic-plastic response spectra of column-top displacement is shown, where a bi-linear type restoring force characteristics model with 2% of elastic stiffness after yielding is adopted. The test results plotted in the figure is compared with the spectrum curves. It is clear that almost all test results are larger than the corresponding spectrum curves. Such tendency was significant in the specimens failed by tensile fracture of concrete (Specimens-4S). Even though concrete slab did not fail, the same tendency was observed, because of the pinched force-displacement curves due to plastic deformation in anchor bolts.

CONCLUDING REMARKS

The concluding remarks obtained by this work are summarized as follows:

(1) Earthquake response of equipment systems installed on reinforced concrete building is influenced sensitively by failure mechanism of the system.
(2) The observed failure mechanisms were
    a) Pull out failure of slab concrete
    b) Yielding in tension of anchor-bolts, and
    c) Yielding in bending of H-shaped column

Among them, pull out failure showed an extremely brittle failure under earthquake condition.

(3) The calculation based on the full plastic concept can predict fairly well the strength and the corresponding failure mechanism.

(4) For practical design purpose, a simple analytical model representing the complicating restoring force characteristics observed in the test was developed.

ACKNOWLEDGEMENTS

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REFERENCES

[1] TAKANASHI,K., UDAGAWA,K., SEKI,M., OKADA,T. and TANAKA,H., "Non-linear Earthquake Response Analysis of Structures by a Computer-Actuator On-line System," Bulletin of ERS, No. 8, Dec. 1974.

[2] OKADA,T. and SEKI,M., "A Simulation of Earthquake Response of Reinforced Concrete Buildings," Proc. of 6-WCEE, New Delhi, India, Jan. 1977.

[3] TAKANASHI,K., UDAGAWA,K. and TANAKA,H., "A Simulation of Earthquake Response of Steel Buildings," Proc. of 6-WCEE, New Delhi, India, Jan. 1977.

[4] OKADA,T., "The Experimental Investigation on ERCBC with Emphasis on the use of Earthquake Response Simulators in Japan," Proc. of Workshop on Earthquake-Resistant Reinforced Concrete Building Construction (ERCBC)," University Extension, Univ. of California, Berkeley, Ca. U.S.A., July 1977.

[5] OKADA.T., SEKI,M. and PARK,Y.J., "A Simulation of Earthquake Response of Reinforced Concrete Building Frames to Bi-directional Ground Motion by IIS Computer-Actuator On-line System," A Paper submitted to 7-WCEE, Istanbul, Turkey, Sept. 1980.

[6] TAKANASHI,K., UDAGAWA,K. and TANAKA,H., "Pseudo-Dynamic Tests on a 2-story Steel Frame by Computer-Load Test Apparatus Hybrid System," A Paper submitted to 7-WCEE, Istanbul, Turkey, Sept. 1980.

[7] AMERICAN CONCRETE INSTITUTE, "Building Code Requirements for Reinforced Concrete" (ACI 318-77).

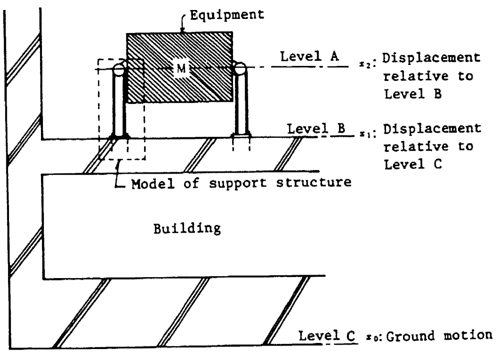


Fig.1 The eqipment system assumed in the simulations

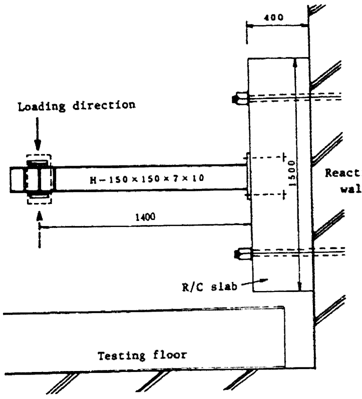
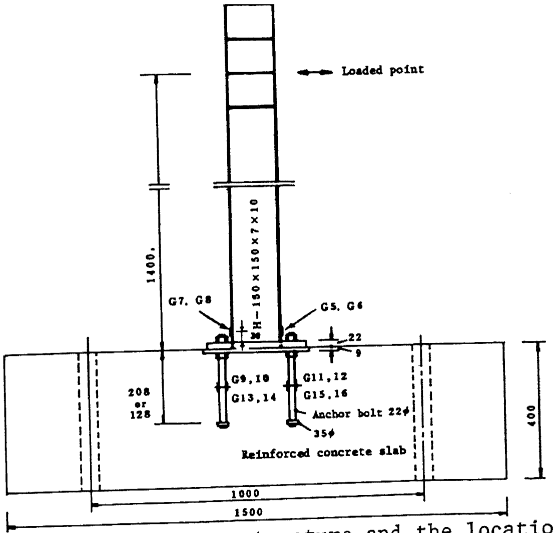
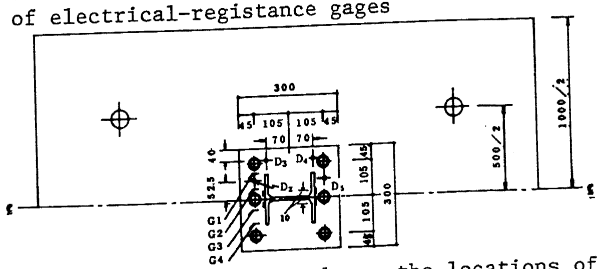


Fig.3 Schematic view of test setup



(a) Section of the test structure and the locations of electrical-registance gages

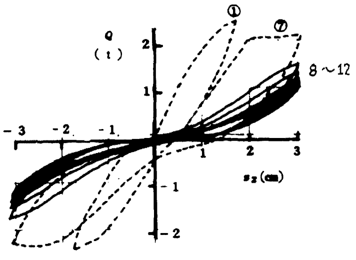


(b) Details of the column base, the locations of gages and displacement meters

Fig.2 Test structure and locations of instruments

Table 1 Test structures and test results

No.		Periods		Specimen code	Anchor bolts		Maximum displacement x_{max} cm	Yield strength Q_Y ton	Maximum strength Q_{max} ton	Q_{max}/Q_Y	Dominant failure mechanism observed
		Nominal	Measured		n	Length (mm)					
1	Cyclic test			I -1-4S	4	128 (5.8d)	+3.0 -3.1	2.4	+2.5 -2.3	+1.04 -0.96	Type-S
2				II -1-4L	4	208 (9.5d)	+5.0 -5.0	3.9	+4.3 -4.5	+1.10 -1.15	Type-B
3				II U-1-4L	4	208 (9.5d)	+5.1 -5.1	4.2	+4.2 -4.7	+1.00 -1.12	Type-B
4				III -1-6L	6	208 (9.5d)	+5.1 -5.0	4.6	+5.3 -5.6	+1.15 -1.22	Type-H
5	On-line test	T =0.8 (sec)	1.01	I -2-4S	4	128 (5.8d)	2.38 (4.92sec)	2.6	2.69	1.03	
6			1.04	II -2-4L	4	208 (9.5d)	2.38 (4.84sec)	–	3.40	–	
7			0.95	II U-2-4L	4	208 (9.5d)	2.03 (4.90sec)	–	2.96	–	
8			0.98	III -2-6L	6	208 (9.5d)	2.36 (6.52sec)	–	4.02	–	
9		T =0.3 (sec)	0.40	I U-3-4S	4	128 (5.8d)	9.20 (4.80sec)	2.6	2.90 (2.07sec)	1.12	Type-S
10			0.41	II -3-4L	4	208 (9.5d)	7.54 (5.84sec)	3.6	4.65 (4.80sec)	1.29	Type-B
11			0.37	II U-3-4L	4	208 (9.5d)	7.18 (5.84sec)	4.0	4.94 (2.27sec)	1.24	Type-B
12			0.35	III -3-6L	6	208 (9.5d)	6.58 (3.35sec)	5.1	6.22 (6.28sec)	1.22	Type-H
13			0.35	IIIU-3-6L	6	208 (9.5d)	7.08 (5.78sec)	5.7	6.45 (3.32sec)	1.13	Type-H and -S



(a) I-1-4s

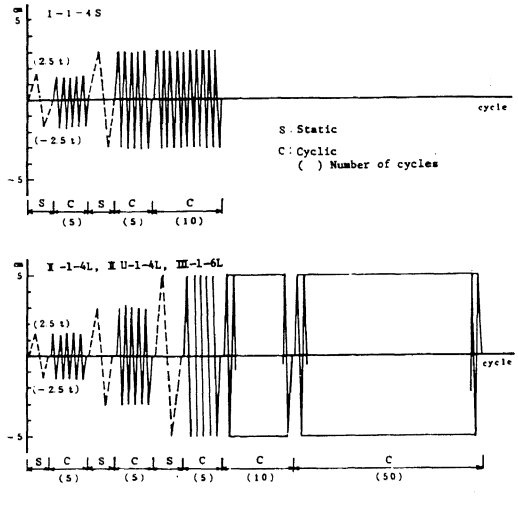
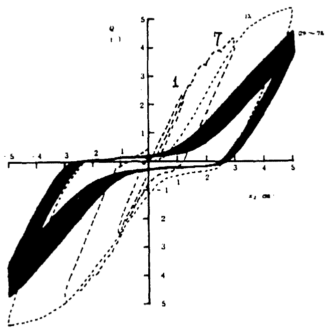
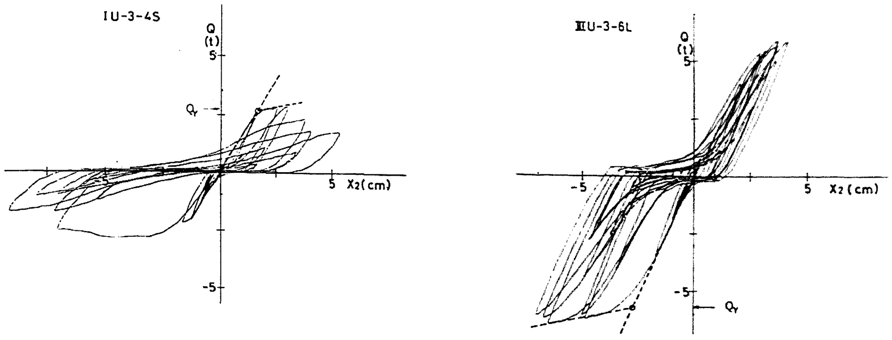


Fig.4 Loading programs in cyclic tests



(b) III-1-6L

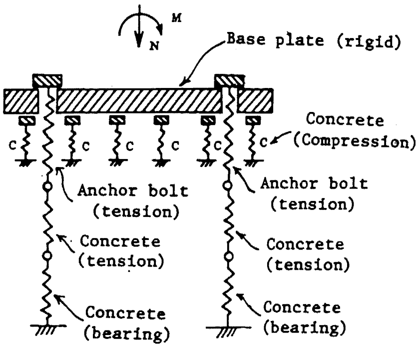
Fig. 5 Lateral load Q vs displacement X_2



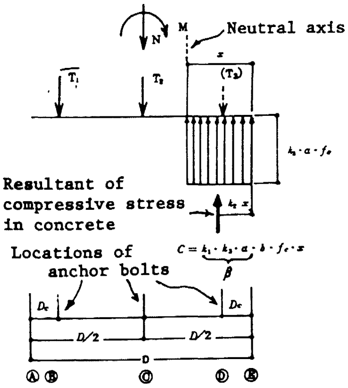
(a) IU-3-4S (To=0.3 sec)

(b) IIIU-3-6L (To=0.3 sec)

Fig.6 Response shear force vs displacement



(a) An analytical model



(b) Forces and stress distribution assumed

Fig.9 Analytical model and forces and distibuted stress acting on the column base

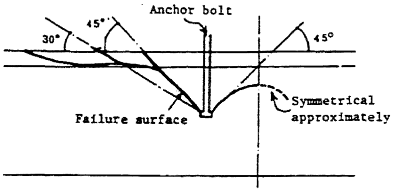


Fig.7 Schematic view of the pull out failure mechanism

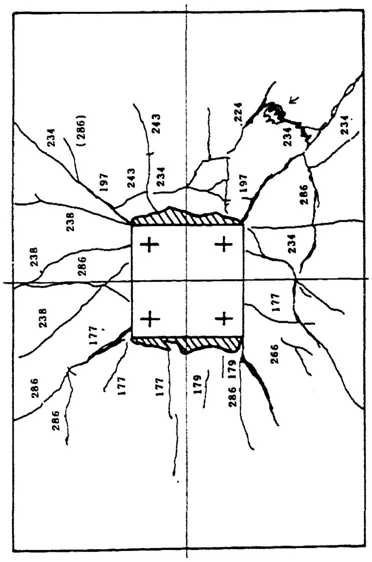


Fig.8 Sketch of the cracks on the slab surface caused by pull out failure (IU-3-4S)

Tests
- ● : Pull out failure
- ◐ : Pull out failure
- ○ : Yielding in tension of anchor bolts
- ◆ : Yielding of the column and pull out failure
- ◇ : Yielding of the column and pull out failure
- △ : No failure observed

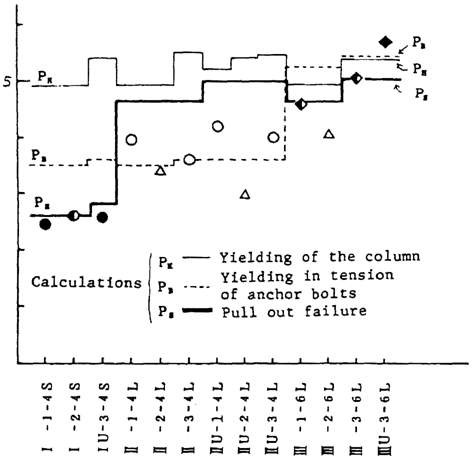


Fig. 10 Yield strength observed and calculated

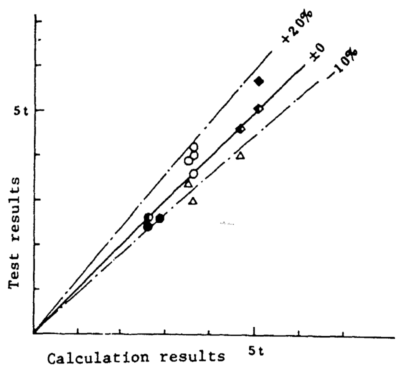


Fig. 11 Comparison in yield strength observed and calculated

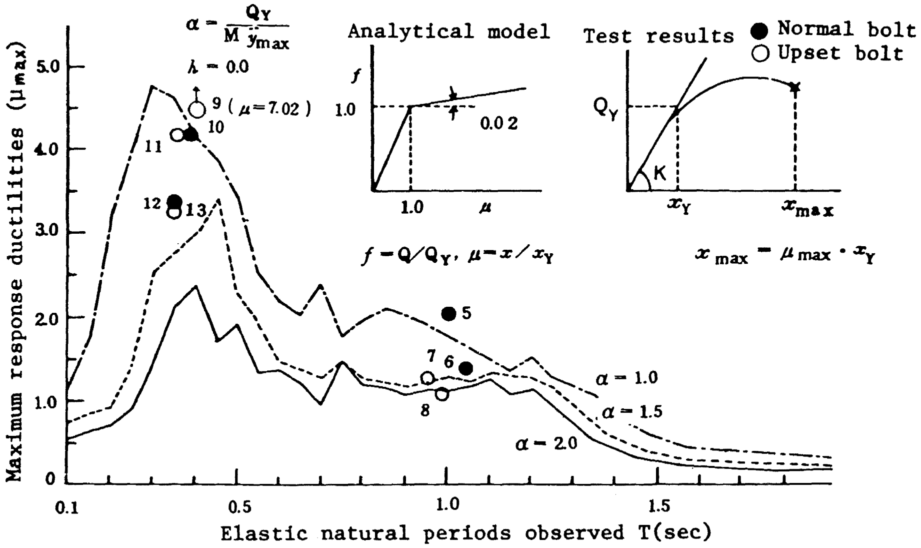


Fig. 12 Maximum response ductilities