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HYBRID AND PARALELL STATIC ELASTO-PLASTIC STRESS ANALYSIS FOR HIGH-RISE REINFORCED CONCRETE BUILDING COMBINED WITH PARTIAL ACTUAL EXPERIMENT

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Abstract . *It is necessary to evaluate the model about the material characters with acting forces for each member in the structures when the structural behaviors for the structures would be required by using the structural analysis. However, it is quite difficult to evaluate such models for each member exactly. In this paper, the object and summary about a developed parallel structural analysis system by using three computer systems and an effective loading system with small scale actual experiments partially for both RC external columns in a high rise RC plane frame building , that material characters would not be evaluated, are described. Two cases of parallel static elasto-plastic stress analysis for 20 story RC building was performed with this system. One was controlled with Al distributed loading by considering displacements at the top of the plane frame building and other was controlled with displacements made from dynamic response analysis by using lumped mass model for an earthquake wave. Experiment results for each case are described lastly.*

1. INTRODUCTION

1.1 Background for this study

It is general to perform the stress and response analysis with computers for understanding the behaviors of the buildings under much external disturbance, such as earthquakes and strong winds. It is necessary for the engineers to make the exact numerical models about the material characters, boundary conditions and others for members.

However, it is quite difficult to evaluate the exact models for the material characters about columns in which normal forces changes largely and the joint parts at that columns and beams are connected. As there have been many actual experiments by using members to evaluate the numerical models for materials, it has not been correct reappearance for the complex behaviors in the parts of joints that are composed with some members, because simple setting has been performed for the displacements and forces on the boundaries without considering all behaviors in the building about the scale, distribution, and history of assigned displacements and forces. There have been much pseudo-dynamic experiments to escape these problems. However, those experiments have been adapted for very simple frames, because dynamic experiments are very complex.

1.2 Study in the past

Many studies with computer-on-line experiments combined the experiments with numerical stress analysis had been performed. Dr. Hakuno[1)] proposed an experimental method. Members, that would be destroyed, were picked up in the structure. Actual experiment for those members were performed and those experimental results were reflected to the numerical analysis by applying the correct boundary conditions for supports and load conditions. The suitable non-linear response results for the increase of the periods about vibration, the decrease of magnification for response, history of response and etc. were acquired in the one-degree structure under a earthquake by using this method. Dr. Takanashi, et al.[2)] constructed the computer-on-line seismic response analysis system with small computer and electric oil actuators by using pseudo-dynamic technique. Fine results for non-linear seismic response analysis were gotten by applying to RC and S structures with one span and one story. In this study, the problems for sampling time of A/D and D/A converters and the stiffness matrix method with considering instability for the electric-oil actuators were referred. They performed the seismic response analysis for a simple steel frame with one span and one story that was destroyed in the beam at first. Proper results were acquired in the stage that the rotational angles in the response were small. Dr. Adachi and et al.[3)] described the

main cause for errors occurred in pseudo-dynamic experiments and proposed the method to correct errors. Dr. Kondo, Dr. Higashiura and et al.[4] declared restoring force characteristics of four columns that were at the first story of high-rise RC building and were under much change of normal forces by using computer-on-line experiments with oil actuators and the substructure method. Dr. Nishizawa and Dr. Tachibana and et al.[5),6)] pointed out faults for use of oil actuators to improve the precision. They developed an effective digital difference analyzer and a digital actuators and constructed a special computer-on-line system. They proved it was possible to analyze the steel structure with 2floors in the inelastic response analysis by using this system and by correcting errors at high precision. Dr. Nakashima, Dr. Ishida and et al. showed operator-splitting method were most suitable response analysis method for pseudo-dynamic response experiments. Dr. Calros, Dr. Ohi, Dr. Takanashi and et al.[8),9)] proposed the efficiency to treat the various estimated parameters as the networks for errors and showed availability with this method by applying them to the steel structures.

1.3 Object

Authors and et al. have been studying and proposed the static analysis method(call experimental analysis method, EAM after here) that combined the actual experiment for some members as scaled models with numerical stress analysis for other members by using a computer about the steel structures. In this paper, static analyses were tried to perform with actual experiment for external columns(call experimental members) where actual normal forces changes extremely in the high-rise RC building using numerical stress analysis for other members in Fig. 1. Both results were reflected for each other as a whole structure. It is a characteristics to analyze large and complex structures by combing the engineering work station system(EWS), electric actuators with stepping motors, and to apply the correct boundary conditions for the experimental members in high-precision. In this paper, this experimental analysis method and the results by using that method are described.

2. A TARGET STRUCTURES AND LOAD CONDITIONS

Detail members' sizes and frame scales of high-rise RC building with 20 floors and 4 spans as the target structure to analyze are showed in Fig. 2. It is a special feature that the building was designed to destroy at columns in the first floor. Actual experimental models for columns at the first floor were made with scale of 1 to 10 for the sections and quantity of reinforcement bars. The section for the experimental model is showed in Fig. 3. Main reinforcement bars are 24-D3, and stirrups are 4D1@10. Material characters for the used reinforcement bars and mortar was showed in Table 1. Two kinds of loading patterns were applied at those structures.

The vertical distribution for storey shear coefficient was performed with load-control in the first case. Displacements at the each floors that were taken from results in the time history at the response analysis by performing seismic analysis with considering bending and shear deformations were applied to this building in the case 2.

3. SYSTEM FOR PARALLEL EXPERIMENTAL ANALYSIS METHOD

3.1 Components and function in the system

Components of the system are showed in Fig. 4. The system was constructed with three computer systems, loading components, and a test frame. There were a static stress analysis system, a data transferring system, and data acquisition and load control system as three computer systems. One EWS system and two PC computer systems were used. EWS performed the static elasto-plastic stress analysis for the whole structure with including experimental members. Displacements at horizontal and vertical displacements and rotations at the top for experimental members and control commands were transferred to data transferring PC computer system by using network filing system(NFS) in the EWS. Data transferring PC transfers displacement and control data to data acquisition and load control PC with RS232C. Data acquisition and load control PC sent displacement data to pulse product equipment to satisfy the limit displacements. Those data were transferred to motor drivers and electric actuators were able to move to desired displacements. When target displacements were reached at two tops of experimental columns, displacements and strains were measured with displacement meters and load-cells set on two external experimental columns. These data were transferred to the data acquisition and load control PC through a switch box, data logger, and GP/IB. In the data acquisition and load control PC, horizontal and vertical forces and bending moments as the reactions at two tops of experimental columns by combining the measured data, basic material data, and geometrical data for experimental columns. These reactions were transferred to EWS through the data transferring PC. Comparison with analytical data and those experimental data was performed in the EWS. Numerical stress analyses and actual experiments were repeated again until the difference between analytical data and experimental data was 1/300. If both data were agreed with considering the precision in detail, numerical stress analysis was begun for the next time step boundary condition. The flow of each works in this system is showed in Fig. 5. Fine lines show the flow for each works and thick lines show the flow of data exchanged among the computer systems and equipment. It was necessary for a loading system to transfer displacements gotten from numerical stress analysis to the data acquisition and control PC. It is difficult for usual actuators with the oil pressure-type to control in applying the desired

displacements at two tops of experimental columns in the higher precision. In this study, electric actuators had been developed with stepping motors that rotate at very exact angles with corresponding to input pulse steps in the most high precision and had been used in this experiment. Capacity for electric actuators has plus and minus 2 ton as maximum loading and has plus and minus 3cm as maximum displacement. The relation with a test frame, electric actuators and specimen is showed in Fig. 6. As it would be the case to make loading over plus and minus 2 ton at tops of external 1st floor column, capacity for the maximum loading more than plus and minus 2 ton at tops of external columns was revised to apply loading more than plus and minus 2 ton by using the mechanism of levers. Load-cells at the tops of the electric actuators measured strains and converted applying forces. Pin supports were set at the both ends of electric actuators for the horizontal direction. However, electric actuators were very heavy about for each. Four strong springs were used to support an actuator with out applying be ding moment at two end of a actuator. Figures for test equipment are showed in Photo. 1 and 2.

3.2 Static elasto-plastic stress analysis method

Middle parts of beam members were treated as a elastic bar element with considering displacements affected from effect for bending moment and shear deformation, and plastic characteristics for beams was expressed from the facility of rigid plastic spring at the ends of members. A member model in Fig. 7 was used for column members. Parts on a-b and e-f of a member were finite size joints and a part on c-d was elastic by considering for a bending moment and a shear deformation. parts on b-c and d-e were treated with many elasto-plastic springs for only normal forces as the direction. Normal forces and bending moments were evaluated with displacements by dividing section of experimental columns as layers and setting the normal stress and strains constant for each layer. The method for the evaluation of stiffness matrices and the treatment for forces at member ends were used by the procedures that were described in our old references.[12] The relation with normal force and normal displacement about characters of mortar and steel bars in the columns was evaluated by using data in Table 1. The flow of the static elasto-plastic stress analysis is showed at the right hand side in Fig. 5. However, it was not available to evaluate the stiffness matrices for experimental members in the plastic zone at each step. It was necessary to evaluate the displacements including rotations at tops of experimental columns by considering that foots at the experimental members were fixed. However, as it was not possible to evaluate the element stiffness sized 3 by 3 for an experimental member at each loading step, it was necessary to assume element stiffness for experimental members. Displacements would be evaluated as more large than correct values when stiffness matrices for experimental members would be

evaluated more weaker than actual values, because total stiffness matrix would be low with comparison actual stiffness matrix. In this study, initial element stiffness matrices were used for the assumed element stiffness matrices at all times to evaluate always higher element stiffness matrices. The incremental displacements for all nodal points in the structure were evaluated, calculation total summation(T) for square numbers of those displacements and summation(S) for square numbers of total displacement at all nodal point were performed . It was a convergence condition that ratio of value(T/S) would be smaller than a assigned value. If value(T/S) would be larger than a assigned value, static numerical stress analysis was performed again after the calculation for unbalanced forces at member ends for the structure. Repeated calculations were performed 3 times as a minimum and 25 times as a maximum and it took from 45 minutes to 50 minutes for one analyzed step. As a vertical force, two horizontal forces at a top of experimental member, and a vertical displacement, a horizontal displacement and rotation at the boundary of experimental member and other parts in the structure were combined each other, it would be possible to evaluate normal forces, shear forces and bending moments at tops of experimental members exactly. At this stage, component of a shear force born by incline of axial bars set was considered to apply normal forces.

4. STATIC ELASTO-PLASTIC STRESS ANALYSIS AND EXPERIMENTAL ANALYSIS

At this stage, vertical loads were applied and horizontal loading with vertical distribution for storey shear coefficient was loaded by controlling the horizontal displacements at the top of a structure in Fig. 8.

4.1 Progress in the experimental analysis

At first, vertical loading was performed on the experimental columns with loading values divided into five equal values for total vertical load to check that the vertical loading bars were set at correct centers of experimental columns. After loading points on the center of experimental columns were recognized as correct points by comparing the normal forces in the experimental columns X1 and X5 and in the analytical columns X2, X3, X4 and with those from analysis only, horizontal loading with vertical distribution for storey shear coefficient by controlling displacement at the top of a structure. Experimental columns kept whole-some conditions without seeing hair cracks until horizontal displacements were over 40 cm at the top of a structure. The failure by compression stress and shear force was occurred suddenly at the bottom of X5 column when displacement at the top was 40cm. At that situation, there were 1041.8 ton as normal force, 114.49 ton as shear force and 444.08 tm as

bending moment at the bottom of X5 column with converting the values from model size to actual size. There were hair cracks at X1 column at that time. After that, horizontal loading control was performed to move the top of a structure for 40cm. Suddenly, failure by bending moment and shear force was occurred at the top of X1 column where there were hair cracks. Experimental analysis was stopped as the limit displacement of electric actuator was reached. The last state of destroyed column X1 is showed in Photo 3.

4.2 Comparison and consideration for results from two analyses

4.2.1 Relations between shear forces and relative storey displacements

There are the diagrams for the relations between shear forces and relative storey displacements from the results at the analysis only(dashed line) and the experimental analysis(solid line) in Fig. 9. Tendency for behaviors of both analyses in the elastic zone was almost similar, as behaviors at experimental analysis were very complex. However, behaviors in the horizontal displacement at X1 column on the experimental analysis were same as behaviors in the analysis only according to increase of displacements. Increase in the horizontal direction at X5 column on the experimental analysis had been bigger than these on the analysis only until the destruction at X5 column occurred. There were the actual shear force as 114.49 ton(1.6 times compared results in the analysis only) and axial force ratio was 0.59 at the destroyed time. Different behaviors were seen in X3 column on the experimental analysis. Though behaviors on the experimental analysis would be same as behaviors on the analysis only, there was some difference in X3 column, because behaviors in X5 column would give affection to X3 column. When positive horizontal forces were applied at X5 column, variable compressive axial forces came at X5 column. As X5 column was an external column that accept the variable compressive axial forces, increase of strength for bending moment was occurred. Large shear forces were acted on this column as results. However, there would be difference between experimental analysis and analysis only, because it was impossible to consider those behaviors in the analysis only. As X1 column was under the same situation, when the negative horizontal forces were applied at that column. However, X1 column was destroyed with bending moments and shear forces before X1 column was under those situations. There were equivalent shear forces on both analyses in the elastic zone. As elastic stiffness for shear force was used to the experimental columns in the experimental analysis, lower values were seen after destruction with bending moments and shear forces. There were negative slope zones at X3 column in the experimental analysis. In these negative slope zone analysis, it is not the case to release stresses, when compressive strains are increased and it is the case to release stresses when compressive strains are decreased in the model for relation between axial forces and axial displacements about concrete spring model.

In the analysis only, axial displacements at the extreme fiber parts in the concrete spring increases simply. However, compressive strains do not increase always on the way, as very complex repeated calculation in the plastic zones is performed. Negative slope zone was showed as results.

4.2.2 Relation between axial forces and bending moments

Relations between axial forces and mending moments at tops and bottoms in X1 and X5 column and N-M interaction curves with plotting results taken from the material tests are showed in Fig. 10. Interaction curves with thin lines are obtained by using e-function for stress-strain relation about concrete and perfect elasto-plastic relation for stress-strain about steel bars. Tendency is almost similar for results gotten from analysis only and experimental analysis at tops and bottoms in X1 column, though behaviors from experimental analysis are complex. In the analysis only, points with axial forces(N) and bending moments(M) are inside NM curve showed with thin lines that is the limit to fracture at top and bottom of X5 column. However, those points from experimental analysis are exceeded over NM curve a little at bottom of X5. Tendency for the characters of behaviors is similar in both analyses and data from experimental analysis are bigger than those from analysis only. It is the NM curves with thick line in Fig. 10 that Dr. Takiguchi, et al.[13),14),15)] proposed by considering restricted effect from setting big stubs. Compressive stresses for concrete were 1.5 times of normal stresses for concrete in those curves. Points with axial forces(n) and bending moment(M) are inner in this thick line curve in the experimental analysis.

4.2.3 Change of inflection points

The change of inflection points is showed for X1,X3 and X5 column in Fig. 11. There are different results in the change of inflection points at X1 and X5 column in both experimental analysis and analysis only. Tendency for change of inflection points is agreed in both analyses in X3 column, though there are influences in this column from behaviors in X3 and X5 column. It was obvious to come very complex behaviors that would not be possible to make model for material characters and geometrical configuration about members by using mathematical model. This means that behaviors in RC members are very complex. It has been possible to analyze stresses for all parts in the building by combining with experimental members and other members for analysis only after the building would be destroyed under complex loading cases including vertical loads and horizontal loads.

5. TWO STATIC ELASTO-PLASTIC STRESSES WITH TIME HISTORY

5.1 Characteristics of frame to perform dynamic analysis as multi-freedom model

Equivalent shear spring stiffness, strength of cracking and strength of yield stresses at each stories were derived from the relations between storey shear forces and relative storey displacement at each stories by the minimum mean square method with performing repeated static elasto-plastic stress analysis for all building. Parameters for lumped-mass model are showed in table-2 and Fig. 12.

5.2 Applied earthquake wave and displacements for control

Applied earthquake wave were accelerations in components for NS direction taken at El Centro in 1940. Maximum acceleration was 600cm/sec^2 and period for loading time was 15 second. Time step for experimental analysis was used as 0.002 second. Dynamic response analysis was performed for lumped mass model described in 5.1 under those earthquake and displacements at each story were taken in time history at 0.002 second as time step. Damping coefficient was 0.03 and damping matrices were made as the type of damping factor proportion to instantaneous stiffness. In the experimental analysis, displacements were applied at time step 0.02 second to shorten the experiment.

5.3 Experimental analysis procedure in displacement control

The same structure for a frame described before was used. After vertical loading for dead and live load was performed at first in equal 2 division with considering the convergence for the analysis, described horizontal displacements In time history in 5.2 were applied at each story statically in the analysis only. In the experimental analysis, there were actual experimental members in test model. At first, vertical loading was performed in equal 5 division with considering similar convergent error. After that, displacements at each story described in 5.2 were applied.

5.4 Progress of experimental analysis

After checking to set the centers of bars that apply the axial force to centers of both columns with same procedure as former test, horizontal displacements at each story were applied. There were not cracks for fractures in X1 and X5 column till maximum displacements at tops of columns were *3.516 cm. After that time, slope by relative storey displacement was 1/88 and analysis time step was 4.44 second. At the maximum displacement, fracture appeared at the top of X1 column due to bending moment and shear force. There were cracks at the bottom of X5 column. The top of X5 column was destroyed suddenly at reversed horizontal loading displacement at the top of X5 column was over described maximum displacement at that time. Experimental analysis was stopped, as capacity for the movement of electric actuator was exceeded to the limit displacement.

5.5 Comparison and consideration for results from both analysis

5.5.1 Relation between shear force and relative storey displacement

Relation between shear force and relative storey displacement is showed in Fig. 14 from results in both analyses. Results from analysis only is showed with dashed lines and results from experimental analysis are showed with thick lines. Axial compressive force was 657.10 ton, shear force is 119.13 ton in X1 column when conversion data from model size to actual scale structure. Fractures due to conversion of bending moment and shear force were came at the top of X1 column where bending moments at the top of X1 column was 20.56 tm and those at the bottom of X1 column was 351.16 tm. Ratio of actual axial force performed in X1 column and resistant axial force was 0.37. In general, tendency in behaviors for both analyses was similar. However, complex behaviors in the experimental analysis were seen and stiffness in the experimental analysis would be higher than that in analysis only. Behaviors in X3 column were almost similar, because those behaviors were calculated in the computer at both analyses. Values at maximum strength in X1 column under negative direction loading are quite different. In both cases, Fracture owing to combination with bending moments and shear forces was occurred. However, there is a little difference in results, because fracture at case 2 was occurred in the large displacement zone.

5.5.2 Relation between axial force and bending moment

Relations between axial forces and bending moments at tops and bottoms of X1 and X5 column are plotted in Fig. 15 with drawing NM interaction curves. Results from analysis only are showed with dashed lines and those from experimental analysis are showed with thick lines. Two NM interaction curves could be seen in Fig. 15. NM interaction curves with thin lines are evaluated from material test and those with thick lines that have not top parts are calculated with considering effect by the stubs. As behaviors at top and bottom in X1 column are very complex, tendencies in the experimental analysis are almost agreed with those in the analysis only. It is possible to explain also the fact that was described in 4.2.2.

5.5.3 Changes of inflection points

Changes of inflection points in X1, X3, and X5 column are showed in Fig. 16. Similar results that were described in 4.2.3 were come. It is useful to perform these experimental analysis in order to catch complex behaviors of non-linear members that have much plastic characters such as reinforced concrete members.

6. CONCLUSION

In general, industrial goods are produced on a large scale and sent to consumers after much testing would be performed for much trial pieces and safety for them would be made sure. However, buildings would be made as a unique product. It would be impossible to make them as trial pieces. It also would not be possible to test safety for the building against big earthquakes, because scales for a big earthquakes and strong winds in the design process would be used with considering expected values in the several hundred years as the occurred period. By these reasons, experiments for a piece of members and small frame with a very few members would be performed without doing test against all of trial pieces. From these results, theory to explain results would be composed and the process in the design would be proceeded on the desk by checking the safety for building that would be made actually with those theory. It would be very difficult to pre-capture exact actual behaviors in the building, because boundary conditions for loading and displacements would be made to simplify with packing up some members in the building as described in the introduction section. In this study, it is the object to investigate behaviors of external right and left columns in the first story, that are very difficult to understand them with stress analysis or experiment, in the high-rise RC building. Experimental analysis, in which some experiments for actual scale model against members that are difficult to make mathematical model are performed and behaviors for other members are analyzed by the computer, and those experimental results and analytical results are reflected with keeping the compatibility and the equilibrium each other, has been established. After testing by using this system for the steel structure, it has been recognized that efficient results had been brought by this method. In this study, this method has been applied to more complex high-rise RC building. Two loading cases had been performed for RC structure. The effective results for change of stiffness in the columns, complex relation between axial force and bending moment and change of inflection points in the column with moving bending moments between beams and columns at the second floor had been understood. In the future, it will be possible to use this system developed here for the more high speed experimental analysis with more many experimental members by the reason that capability of computers will be improved more and faculty of mechanical actuators and data acquisition equipment will also be enhanced.

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Table 1. Physical characteristics of concrete and steel bar

Main steel bar(D3)

section area (cm^2)	yield stress (kg/cm^2)	tensile strength (kg/cm^2)	ratio of elongation (%)
0.0731	3236	4496	40.7

Mortar

compressive strength (kg/cm^2)	strain of compressive strength( )	Young's modulus (kg/cm^2)	tensile strength (kg/cm^2)
361	2280	219	23.4

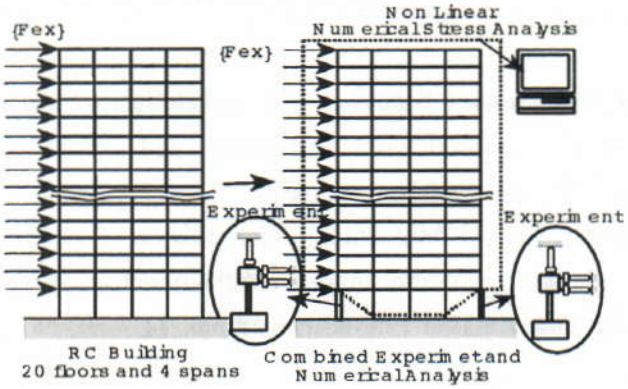


Fig. 1 Concept of Experimental Analysis

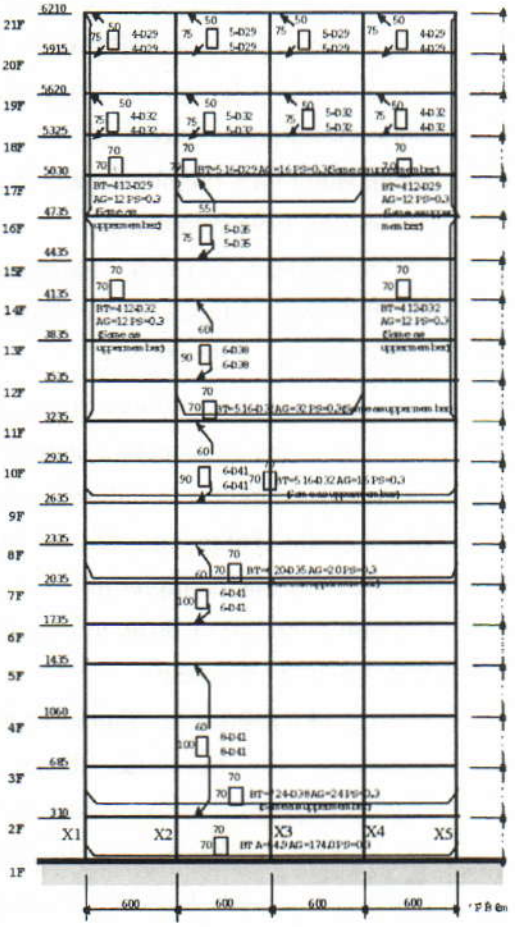


Fig. 2 RC Structure for Experimental Analysis

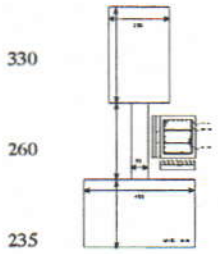


Fig. 3 Experimental Column

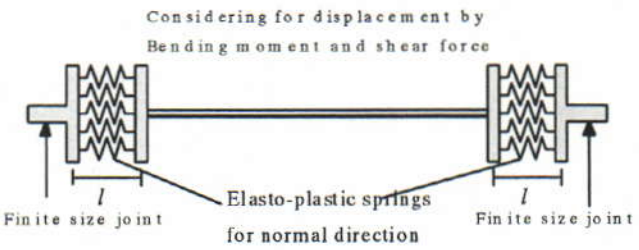


Fig. 7 Analysis Model

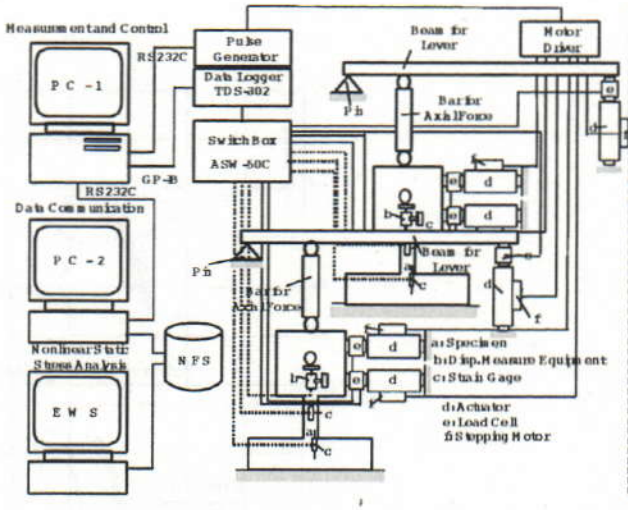


Fig. 4 System Configuration

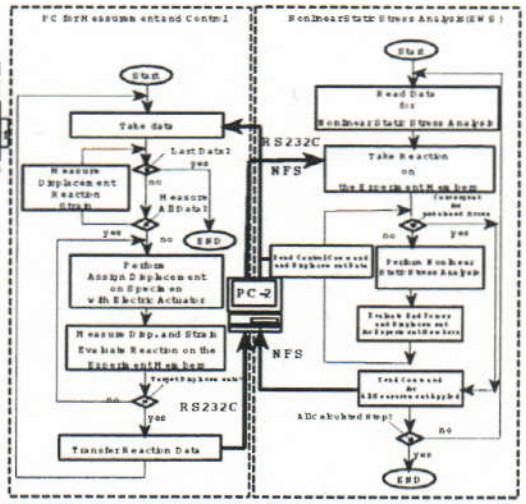


Fig. 5 Flow Chart for System

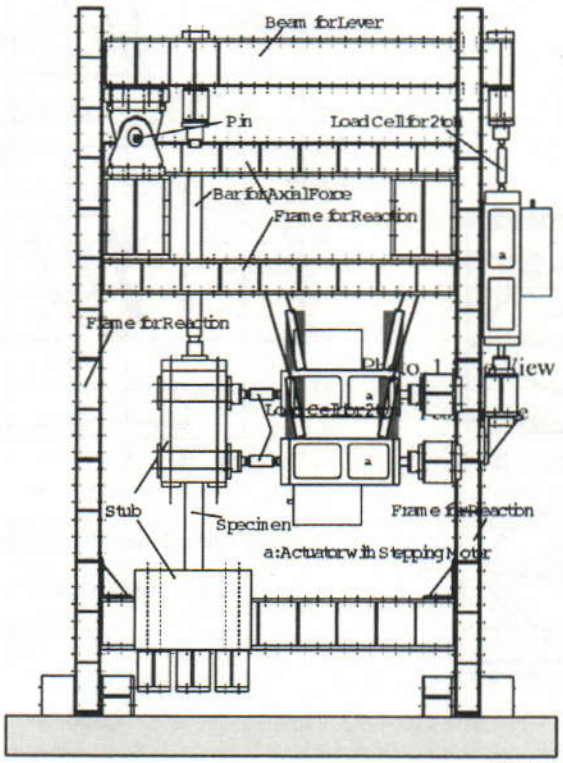


Fig. 6 Test Frame and Equipment

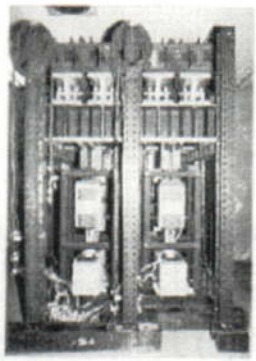
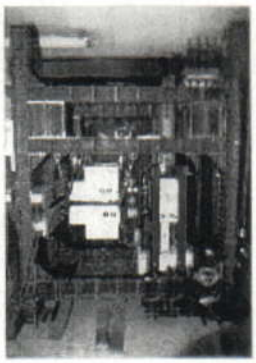


Photo. 2 Normal View



Of Test Frame

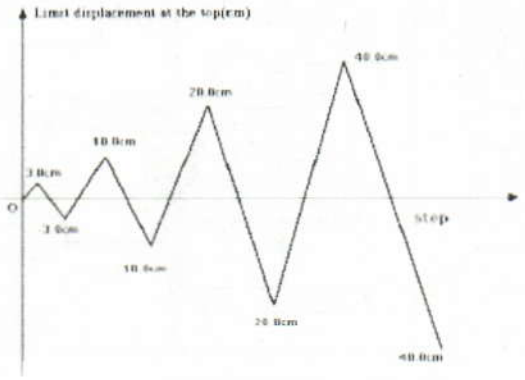


Fig. 8 Limit displacement at the top

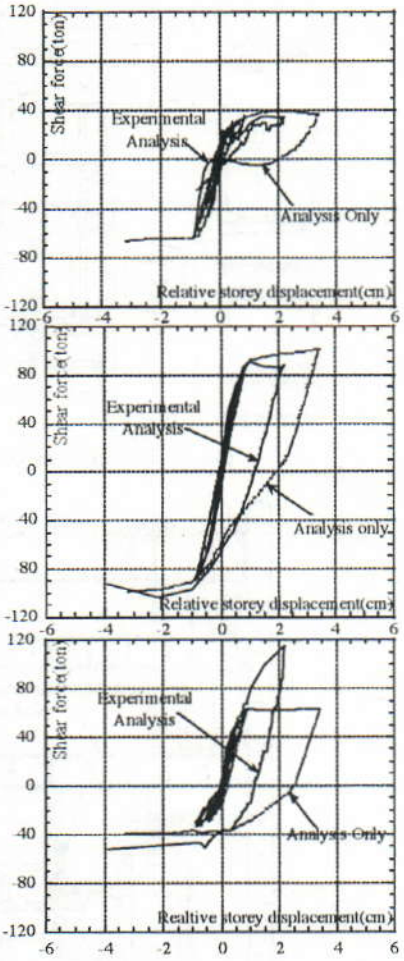


Fig. 9 Shear Force-Relative Storey Displacement (case 1)

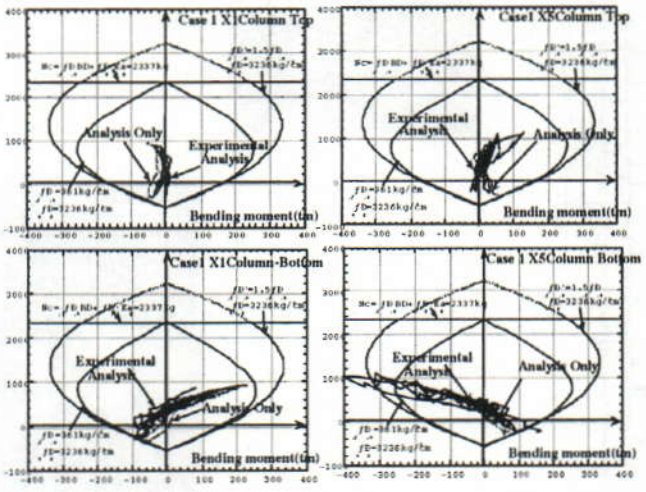


Fig. 10 NM Curve in X1 and X5(case 1)

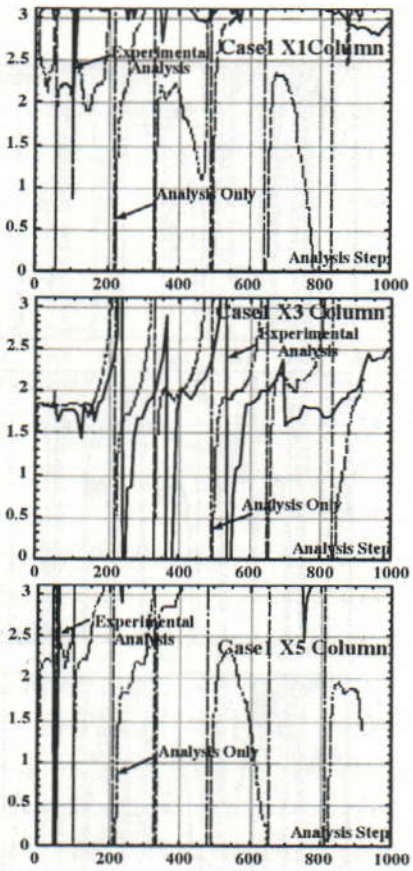


Fig. 11 Change of Inflection Point (case 1)

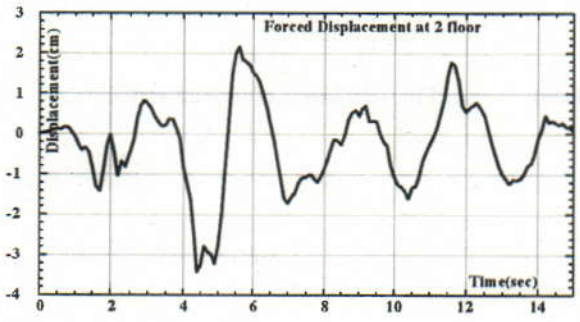


Fig. 13 Time History of Displacement at 2nd floor

s of Lumped Mass Model

ST.	K1(ton)	fi 2	fi 3	Qy1(ton)	Qy2(ton)	fi
20	1.75E+02	0.51	0.03	29.99	58.57	0.35
19	2.16E+02	0.48	0.03	48.46	88.16	0.35
18	2.53E+02	0.39	0.03	60.59	118.39	0.35
17	2.82E+02	0.27	0.04	99.03	141.06	0.35
16	3.06E+02	0.33	0.04	86.06	164.06	0.35
15	3.45E+02	0.32	0.03	89.28	189.78	0.35
14	3.77E+02	0.31	0.02	98.80	210.02	0.35
13	4.21E+02	0.30	0.02	116.96	228.55	0.35
12	4.43E+02	0.32	0.03	118.04	245.01	0.35
11	4.67E+02	0.33	0.02	137.79	265.86	0.35
10	4.90E+02	0.32	0.02	146.16	275.28	0.35
9	5.21E+02	0.32	0.02	148.51	297.12	0.35
8	6.21E+02	0.32	0.02	161.14	307.20	0.35
7	7.02E+02	0.31	0.01	158.05	320.07	0.35
6	7.62E+02	0.31	0.01	159.80	323.70	0.35
5	7.79E+02	0.29	0.01	168.88	341.99	0.35
4	5.13E+02	0.30	0.01	169.12	336.42	0.35
3	4.56E+02	0.30	0.01	181.69	339.89	0.35
2	4.81E+02	0.33	0.02	196.23	341.77	0.35
1	8.34E+02	0.32	0.01	199.04	357.64	0.35

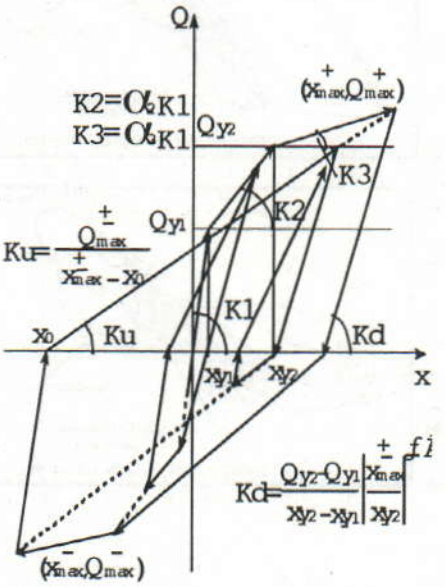


Fig. 12 Elasto-Plastic Model of RC Material

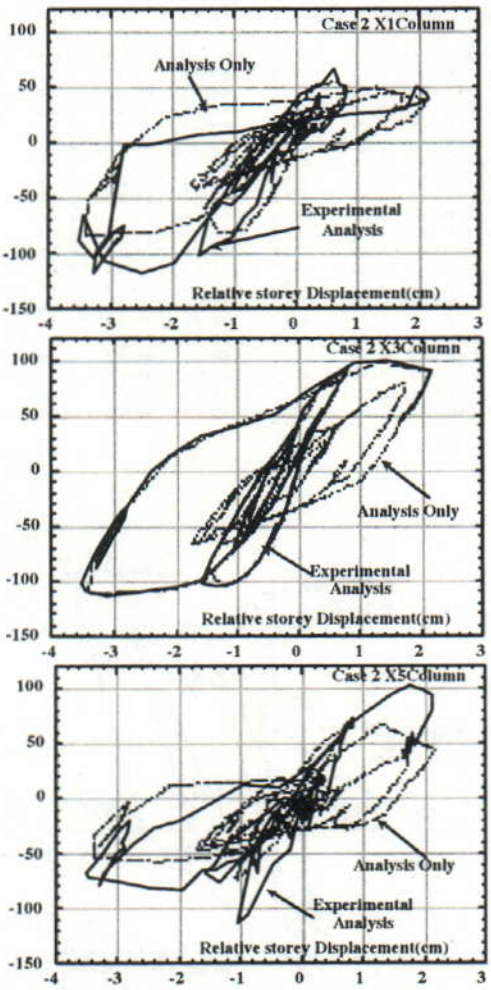


Fig. 14 Shear Force and Relative Storey Displacement (Case 2)

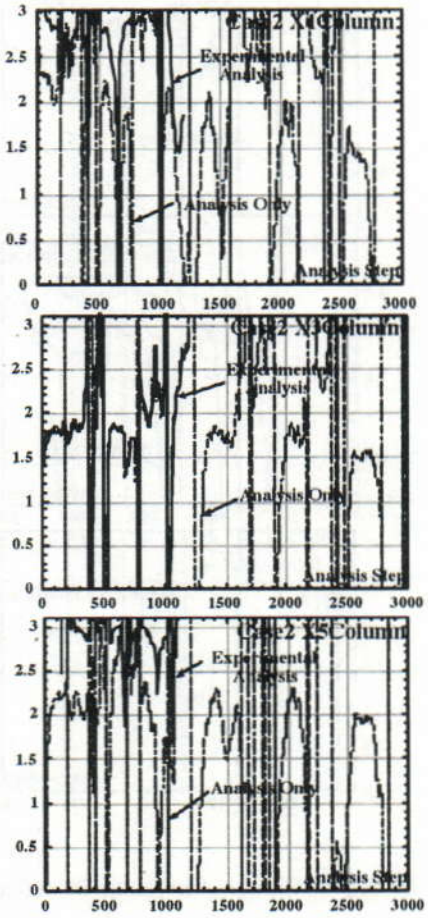


Fig. 16 Change of Inflection (Case 2)

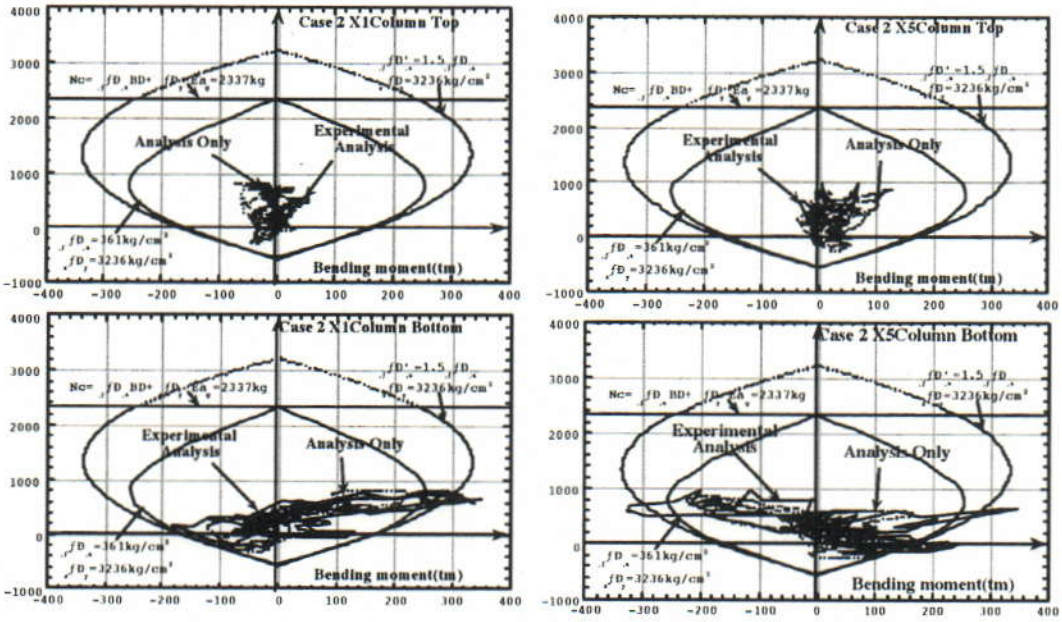
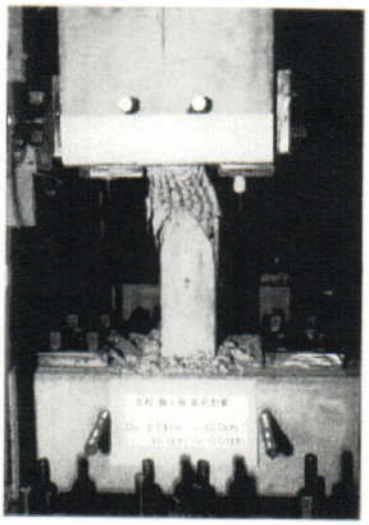



Photo. 3 State of Fracture at X1 Column