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EXPERIMENTAL INVESTIGATION OF P-DELTA EFFECTS TO COLLAPSE DURING EARTHQUAKES

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ABSTRACT:
This paper presents experimental results generated through a program of shake-table testing of simple frames through collapse. Fifteen four-column frame specimens were subjected to progressive unidirectional ground shaking as structural response was measured. The specimens were subdivided into groups of three different column slenderness ratios: 100, 150, and 200. Within each group, column dimensions and the mass used were varied with the specimens, so that ground motions of varying magnitudes are required to collapse each structure tested. A literature review found no other tests of a similar nature.

The experimental setup is described and some typical results are presented. However, focus is on thorough documentation of the experimental data (geometry, material properties, initial imperfections, detailed test results, etc.) such that the tests can be used as benchmarks to which analytical models can be compared. The intent is for immediate and free online access to the results to assist other researchers in the development or validation of analytical tools to model the inelastic dynamic behavior of structures up to collapse.

Keywords: Collapse; Columns; P-Delta Effects; Shake Table Testing

INTRODUCTION

The arbitrary lateral drift limits prescribed by modern design codes to limit non-structural damage also indirectly ensure that structural performance is minimally affected by the effect of gravity on the lateral force resistance of structures (a.k.a. P-Δ effect). However, these conventional drift limits are progressively being eliminated and replaced by other performance-based limits. As inelastic behavior is relied upon to a greater extent in the dissipation of seismic input energy, accurate quantification of the destabilizing effect of gravity is becoming more significant in structural design. As a result, it may be desirable to investigate the behavior of those structures in order to enhance our understanding of the condition ultimately leading to their collapse, and to ensure public safety during extreme events.

This research attempts to provide some of that data through a program of shake-table testing of fifteen simple frame specimens through collapse. Every effort was made to ensure that the experimental data is fully documented (geometry, material properties, initial imperfections, detailed test results, etc.) such that the test results can be used as a benchmark to which analytical models can be compared.

Towards that end, this paper reviews relevant P-Δ concepts, the specimen fabrication and the documentation of their pretest condition, the shaking table and ground motion used in testing, and various important aspects of the experimental setup. Dynamic properties measured from free vibration tests and selected shake table test results are presented. The peak response parameters extracted from each test are described. A preliminary investigation of behavioral trends of the shake table results is presented.

P-Δ CONCEPTS

Some concepts for characterizing P-Δ effects in inelastic SDOF structures under lateral load are described below, along with an overall view of the fundamental structural behavior. Figure 1(a) shows a column from a single bay, single story structure, with an infinitely stiff beam, thereby resulting in a lateral stiffness of the column, ignoring P-Δ, of $K_o = 12EI / L^3$. A bilinear, SDOF model is shown in Figure 1(b). Elastic-perfectly plastic structural response (neglecting P-Δ) is shown, as well as the response modified by the influence of P-Δ. MacRae, Priestley and Tao [1] provided a summary of additional concepts on P-Δ effects on simple structures during earthquakes.

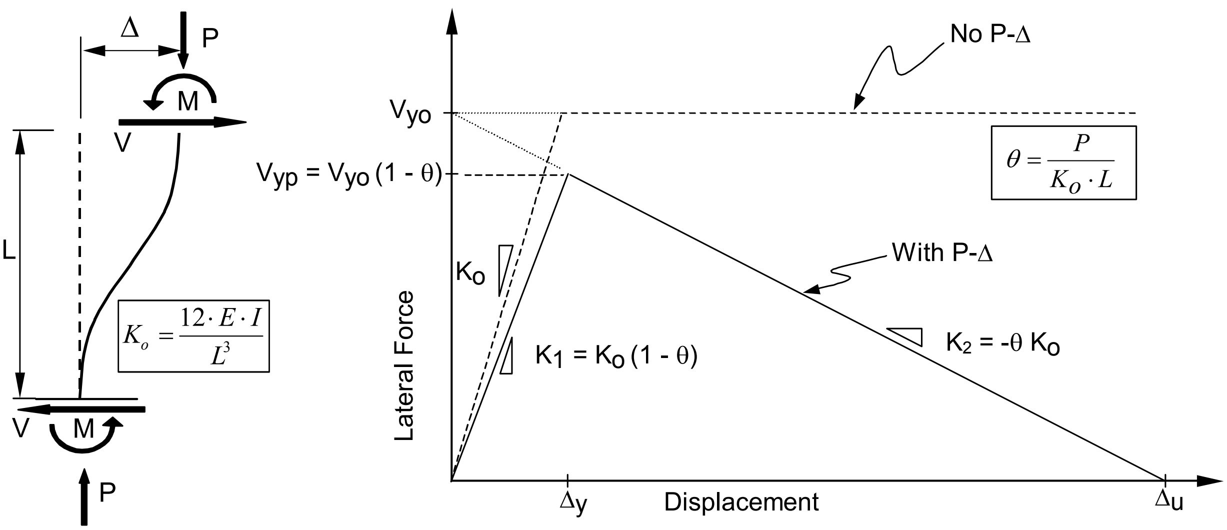


Figure 1: Free Body Diagrams of Typical SDOF structure

OVERVIEW OF EXPERIMENTAL PROGRAM

A SDOF shaking table located at the University at Buffalo Structural Engineering and Earthquake Simulation Laboratory (UB SEESL) was used to conduct the testing program detailed in this paper. Technical specifications of the table are given elsewhere [2].

The ground acceleration time history for the El Centro S00E Imperial Valley earthquake of May 1940 was used in this study. A displacement record was generated from this time history for use as input to the displacement-controlled actuator. Note that unscaled ground motions were used as the specimens were designed to fit actual parameters of interest, and not intended to be scaled models of actual structures.

Description Of Specimens

Fifteen specimens, each having four columns, were tested to failure in the course of this research. These fifteen specimens were subdivided into three groups of five with slenderness ratios of 100, 150, and 200. Sizes of specimens and masses used are listed in Table 1 along with bilinear behavioral properties for the average dimensions, according to the SDOF model described previously. A range of values for axial capacity versus demand, P_u/P_n, was chosen for each slenderness ratio, where P_u is the weight of the mass plates used in the test, and P_n is the axial capacity of all columns in the specimen, calculated using the AISC-LRFD specifications [3]. This ratio ranged from 0.09 to 0.79 for all specimens.

TABLE 1
GENERAL AND DYNAMIC PROPERTIES OF TESTED SPECIMENS

Spec	**Col. Ht. (mm)**	**Col. Width (mm)**	**Mass (kg/col)**	K_o **(N/mm)**	θ	K_1 **(N/mm)**	K_2 **(N/mm)**	T_{no} **(s)**	T_{np} **(s)**	$T_{n\text{-}spectrum}$ **(s)**	ΔT_n
(a) kL/r = 100											
1	137.2	4.8	36.63	40.27	0.065	37.65	-2.62	0.189	0.196	0.200	1.93%
2	137.4	4.9	72.23	41.79	0.123	36.64	-5.16	0.261	0.279	0.272	-2.50%
4	137.5	4.8	96.03	39.12	0.175	32.27	-6.85	0.311	0.343	0.323	-5.86%
5b	91.7	2.9	96.03	23.60	0.435	13.33	-10.27	0.401	0.533	0.698	30.93%
(b) kL/r = 150											
6	412.4	9.4	96.03	22.56	0.101	20.28	-2.28	0.410	0.432	0.430	-0.55%
7	343.7	7.7	96.03	17.70	0.155	14.96	-2.74	0.463	0.503	0.490	-2.66%
8	274.5	6.0	96.03	12.88	0.266	9.45	-3.43	0.543	0.634	0.655	3.39%
9	205.8	4.8	96.03	11.75	0.390	7.17	-4.58	0.568	0.727	0.760	4.52%
10	137.0	3.1	48.58	7.54	0.461	4.06	-3.48	0.504	0.687	0.662	-3.68%
10b	137.4	2.8	48.58	6.88	0.504	3.41	-3.47	0.528	0.750	0.727	-3.01%
(c) kL/r = 200											
11	549.5	9.4	72.23	9.34	0.138	8.05	-1.29	0.552	0.595	0.597	0.32%
12	458.2	7.7	72.23	7.21	0.214	5.67	-1.55	0.629	0.709	0.682	-3.85%
13	366.1	6.0	72.23	5.40	0.359	3.46	-1.93	0.727	0.908	0.959	5.61%
14	275.2	4.7	72.23	4.84	0.532	2.26	-2.57	0.768	1.123	1.200	6.90%
15	182.8	3.1	36.63	3.14	0.627	1.17	-1.97	0.679	1.111	1.004	-9.63%

The specimens were fabricated at the University of Ottawa. Individual columns were cut from hot-rolled steel plate and then milled to size, with a sample layout shown in Figure 2(a). A number of methods were used to analyze each specimen prior to testing. This data was used to determine the scaling factors for the earthquake excitation in developing a testing schedule for each specimen.

Measurement Of Initial Imperfections

Imperfections can have a significant impact on behavior, and must be documented appropriately for subsequent analysis. Thus, each column in each specimen was measured in a variety of ways prior to testing. One base plate for each column of a specimen was designated as the top and marked with an arrow, establishing a reference orientation from which all measurements are related in each orthogonal direction, noted as “U-D” and “L-R” (for “up-down”, and “left-right”).

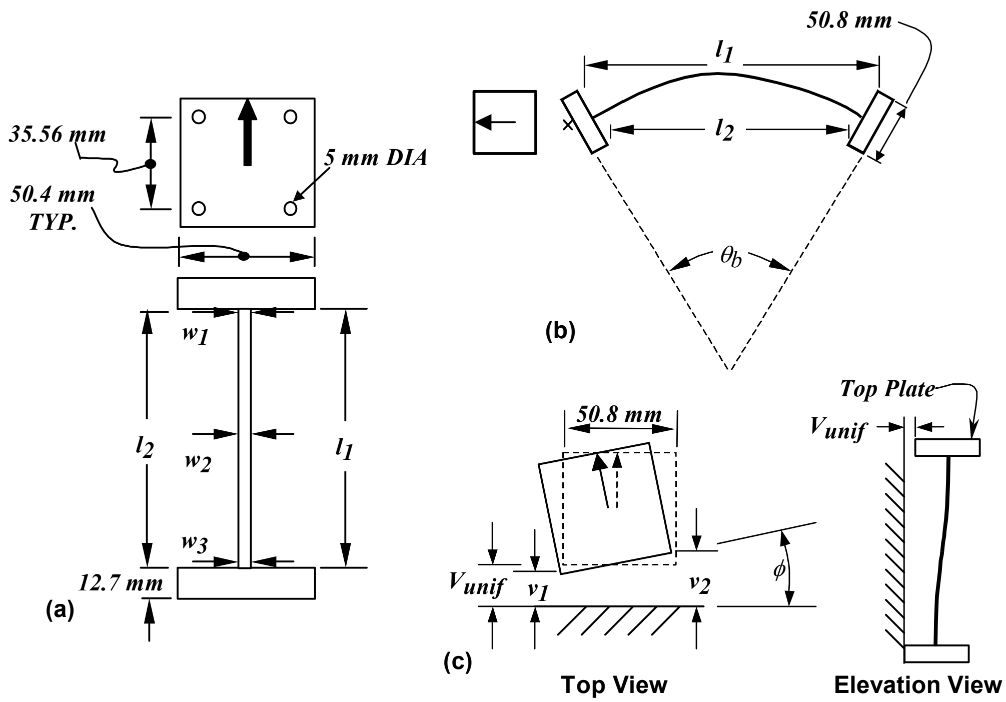


Figure 2: (a) Specimen Measurements in the U-D Orientation
(b) Angle of Bowing in U-D Orientation
(c) Lateral Shift in U-D Orientation

Figure 2 schematically shows general column measurements, and associated imperfections for the "U-D" orientation. The width of each column was measured at the top, middle, and bottom, in each direction, and noted as w_1, w_2, and w_3, respectively. The free height between base plates, l_1 and l_2 is used to calculate the angle of bowing, θ_b. The top lateral shifts of the column, v_1 and v_2, were measured at each corner of the top base plate for each specimen column. These measurements allow for calculation of the average uniform lateral shift, V_{unif}, as well as the angle of twist, ϕ. Column orientations were chosen to minimize the net sum of V_{unif} for all columns parallel to, and perpendicular to, the direction of shaking. Measured dimensions and resulting imperfections are provided elsewhere [4].

Lateral Bracing

Thin metal strips were used as cross bracing to prevent out-of-plane movement and torsion of the test structures. The strips were sufficiently thin to add only a negligible stiffness in the direction of shaking. This was verified analytically, as well as by free vibration tests which showed no change in the period of the structure with and without the metal strip bracing. The free vibration tests showed no significant change in structural damping between the bare and braced specimen.

Instrumentation Of Specimens

Instrumentation was designed to record structural response in a number of ways. A schematic of the test setup and instrumentation is shown in Figure 3.

One accelerometer was mounted on the shaking table to measure the ground acceleration exerted on the model structure. Two were mounted on the top mass plate of the test structure to measure the total acceleration of the mass, from which the inertial force acting on it can be calculated. This mounting procedure ensured the instruments would remain undamaged following each test.

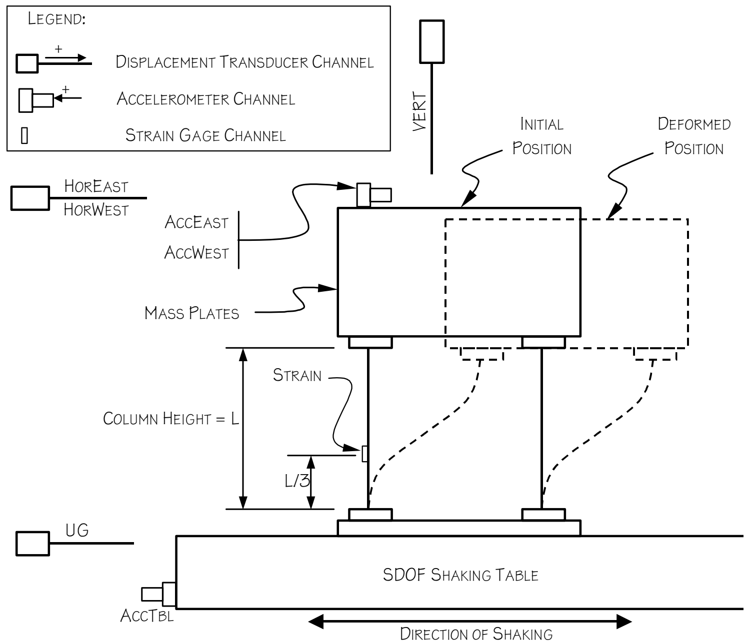


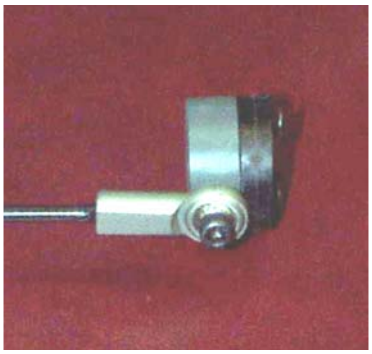
Figure 3: Schematic of Test Setup and Instrumentation (Looking West)

A strain gage was mounted on one column of each specimen, and located at a distance of one-third of the column height from its bottom base plate. Data from this gage can be used to calculate structural forces during testing as discussed below. Furthermore, as the gages were mounted in a region that was assumed to remain elastic throughout the shake table tests, and that was cut out afterwards to conduct a tensile test on the material to determine its stress-strain characteristics.

Linear variable displacement transducers (LVDT's) were used to measure displacement of the table (labeled "UG"), vertical displacement of the mass ("Vert"), and total horizontal displacement at the East and West side of the mass ("HorEast" and "HorWest").

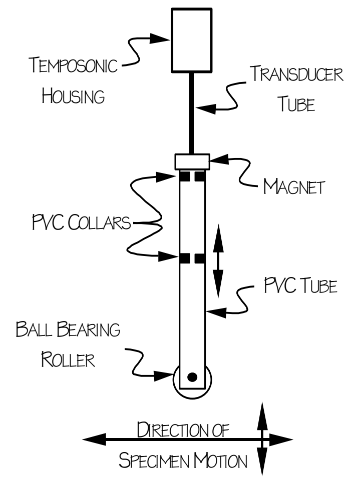
Measurement of the horizontal displacements of the structural mass during the entire structural response of the specimens, including throughout much of their collapse, required special modifications to the instrument setup. Collars were machined from round PVC stock to maintain the perpendicular position of the permanent magnet with respect to the transducer tube of the LVDT's. The PVC collars were attached to a fishing rod via a ball joint end connection as shown in Figure 4(a). The rod was attached to the structural mass by an additional ball joint, allowing rigid body movement of the rod. The fishing rod for this purpose was selected such that, should the test structure collapse in the direction of the LVDT, the out-of-plane flexibility of the rod would protect the instrument by flexing to absorb the impact, and do so without breaking, to allow re-use in later tests.

Another PVC collar assembly, shown in (b) and (c) of Figure 4, was made for the LVDT measuring vertical displacements. A ball bearing roller was attached to the base of an additional PVC tube to minimize friction between the assembly and mass plates. This roller made contact with a stainless steel plate that was epoxied to the top mass plate. The vertical measurements are used to correct error in horizontal measurement as described below.



(a) Horizontal LVDT Collar

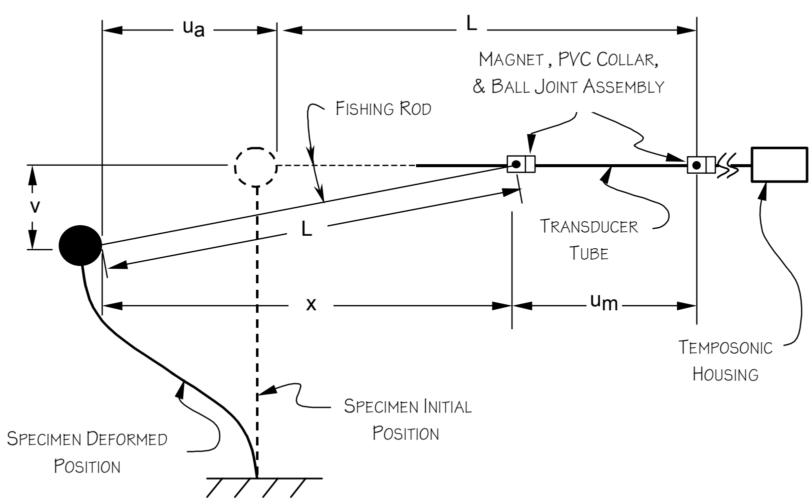
(b) Vertical LVDT Device



(c) Vertical Device Schematic

Figure 4: Displacement Transducer Modification Details

Figure 5 shows the measured vertical and horizontal displacements, and the relations to the actual displacement of the structural mass, at time *t*. In this figure, *L* is the length of the rod used on the horizontal LVDTs, and x, v, u_m, u_a, are the horizontal projection of *L*, the measured vertical displacement or vertical projection of *L*, the measured horizontal displacement, and the actual horizontal displacement, respectively, shown as variables of time, *t*, in the box on the right side of the figure.



Displacement relationships at time, *t*:

$$x(t)^2 + v(t)^2 = L^2$$

$$u_a(t) = x(t) + u_m(t) - L$$

$$u_a(t) = \sqrt{L^2 - v(t)^2} + u_m(t) - L$$

Figure 5: Definition of Variables used in Horizontal Displacement Correction

TESTING OF SPECIMENS

Shake table test schedules were established for each specimen, creating a series of progressively more severe shake table tests until the structure collapsed. Input signals for the testing schedules for the shake table tests were established as follows:

- Estimate the peak level of ground motion for elastic response, using NONLIN [5] to find the 1.5% damped elastic response spectrum for the El Centro ground motion and calculating the maximum PGA for which the maximum displacement of the specimen is equal to the yield displacement.

- Estimate the peak level of ground motion for collapse of bilinear model using the inelastic spectrum technique [6] and SDOF bilinear time history analyses (in NONLIN) to find the point at which the displacement of the system is equal to the ultimate displacement, Δ_u.

- Pre-select approximately five levels of ground motion to be applied to the specimen, progressively and proportionally increasing in magnitude from approximately two-thirds of the estimated peak elastic response to the estimated peak inelastic response.

EXPERIMENTAL RESULTS

Free Vibration Tests

A free vibration test was performed on each specimen prior to the initiation of its respective schedule of shake table tests. This was done by manually pushing the mass of the test structure in the direction of shaking, inducing free vibration response. Results from this test were used to determine the fundamental period of vibration and damping properties of the specimens.

The rightmost columns of Table 1 list, for each specimen, predicted fundamental periods of vibration excluding and including P-Δ, T_{no} and T_{np}, respectively, using average column dimensions. The experimentally obtained period, $T_{n\text{-}spectrum}$, calculated from time history data using Fourier Spectrum Analysis. The percent difference in the measured from the predicted value (including P-Δ), ΔT_n, is listed in the rightmost column.

The percentage of critical damping due to inherent damping in the structure, ξ, is estimated from the free-vibration time history data for each specimen using the logarithmic decrement method [7]. Each response curve was divided into three approximately equal intervals and estimates of the damping ratio were made using the first and last peaks of each interval. To illustrate a typical data set for this study free vibration test results for Specimen 1 are shown in Figure 6.

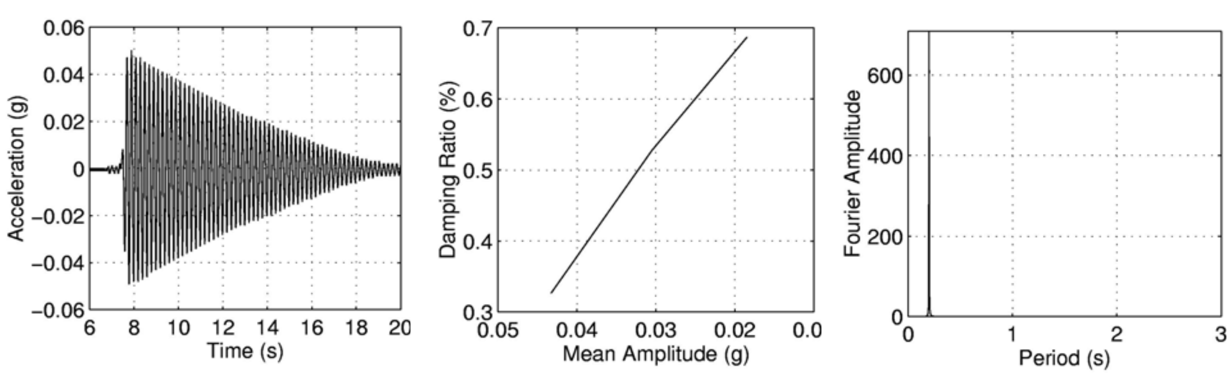


Figure 6: Free Vibration Test of Specimen 1 – AccWest Channel

When the mean amplitude of vibration of a given interval, u^*_i, is plotted versus the estimated damping ratio for that interval, ξ_i, it is observed as a general trend that the two variables are inversely related. Considering all of the individual data channels (displacement and acceleration) used in the free vibration analyses, approximately 80% of all specimens display this behavior.

SDOF Shake Table Tests

As an example, a summary of the shake table test results for Specimen 11 is provided in Table 3, where PGA is the measured peak ground acceleration of the table, "Max. Drift" is the maximum relative displacement of the structure in terms of displacement and inter-story drift (i.e. normalized by the average column height of the specimen), $\ddot{u}_{Tmax}$ is the maximum total acceleration of the structural mass, V^*_{max} is the maximum estimated base shear, not corrected for P-Δ, and u_r is the residual displacement at end of test.

TABLE 3

SHAKE TABLE TEST RESULTS – SPECIMEN 11

Test	**PGA (g)**	**Max. Drift (mm)**	**(%)**	**$\ddot{u}_{Tmax}$ (g)**	**V^*_{max} (N)**	**u_r (mm)**
1	0.132	23.7	4.32	0.271	184.2	0.7
2	0.166	32.2	5.86	0.324	206.3	3.1
3	0.201	45.7	8.32	0.352	206.3	12.8
4	0.262	58.1	10.57	0.346	221.0	24.3
5	0.244	63.7	11.59	0.316	221.0	35.7
6	0.248	82.4	15.00	0.303	228.4	63.2
7	0.242	∞	∞	--	--	∞

Inelastic behavior of specimens typically observed during testing is presented in Figs. 7 and 8. The final four shake table tests of specimen 11 are shown in time history plots of relative displacement (Figure 7) and plots of estimated base shear, V_p^*, described below, normalized by the plastic base shear, V_{yo}, versus displacement ductility, μ, (Figure 8).

The base shear force on the test structure (shown in Figure 8) is estimated from measured strains and simple static equilibrium at an instant of time. The strain gages located at the third point of one column in each specimen, and described previously, are used to calculate an estimate of base shear on the structure due to the earthquake loading. Assuming the moment along the height of the column varies linearly from top to bottom with the point of inflection at mid-height (i.e. neglecting P-Δ and P-δ moments), the base shear on that column can be estimated by:

$$V_o^* = \frac{6M_m}{L} = \frac{6}{L} \cdot \left(\frac{\varepsilon_m \cdot E \cdot I}{h/2} \right) \tag{1}$$

Where V_o^* is the estimated column base shear ignoring P-Δ, M_m is the moment calculated from the strains measured at the third point of column height, ε_m is the strain measured by strain gage at third point of column height, E is the elastic modulus of steel, taken as 200,000 MPa, I is the moment of inertia of column, and h is the width of column.

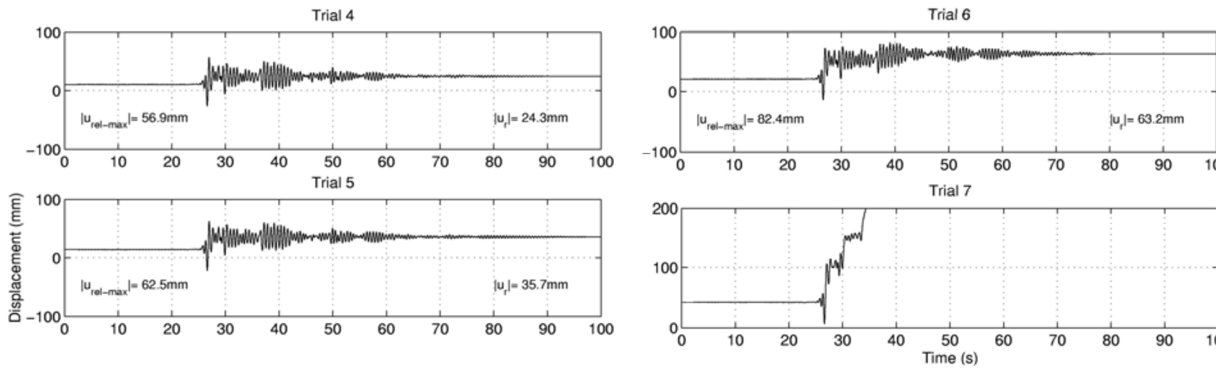


Figure 7: Displacement Response of Specimen 11

The estimated base shear force is further improved by the inclusion of P-Δ. The previous result is modified by an additional term: $V_p^* = V_o^* - (P/L) \cdot \Delta$, where Δ is the relative displacement at time, t, and given by $u_a(t) - u_g(t)$.

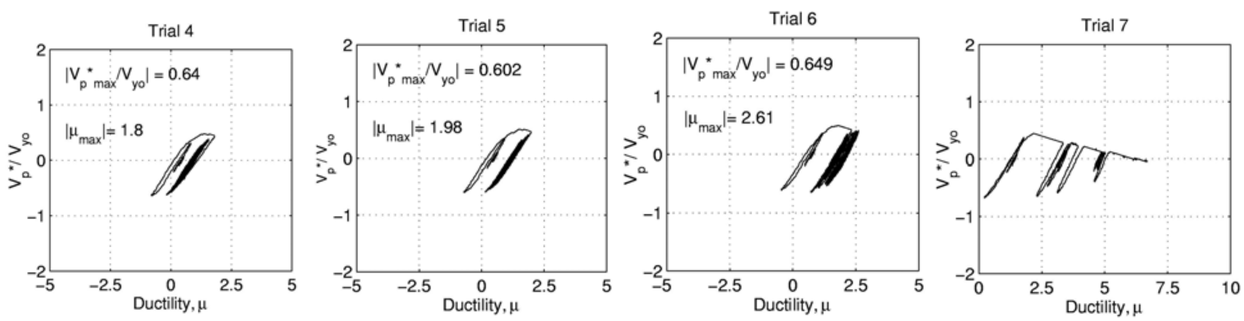


Figure 8: Force vs. Displacement Response of Specimen 11

The reader is cautioned that the assumed linear variation of moment along the height is incorrect, the inaccuracies introduced by this simplification being a function of the magnitude of axial force and of columns deformations, δ.

CONCLUSION

The experimental data generated by this project provides a well-documented set of shake table test results of a SDOF system subjected to earthquakes of progressively increasing intensity up to collapse due to instability. To make these results broadly and easily accessible to researchers who may wish to validate or develop algorithms capable of modeling inelastic behavior of steel frame structures up to and including collapse, the complete data from this study has been located on the world-wide-web for immediate access by interested researchers, (with all intermediate data files) at http://civil.eng.buffalo.edu/users_ntwk/experimental/case_studies/vian/.

Specimens' sizes were chosen to allow testing to complete collapse on a small-scale SDOF shaking table. Testing procedures were standardized, allowing tests to be performed in rapid succession while ensuring personal safety, as well as safety of the instruments recording data.

Unscaled ground motions were used as the specimens were designed to fit actual parameters of interest, and not intended to be scaled models of actual structures.

Fabrication quality, as in every structure, varied for the various columns tested here, even among those making up the same specimen. Imperfections were therefore measured in a number of ways to allow for appropriate consideration in subsequent analytical modeling. A procedure was also developed to correct the displacement time histories accounting for angle changes at large displacements.

Inherent damping of the specimens tested was measured to be non-linear. As a general trend, during free vibration testing, the damping ratio was observed to increase as the free vibration response amplitude decreased.

For similar test programs in the future, specimens could be tested under other conditions to further quantify the nonlinear inelastic behavior of columns under earthquake loads up to collapse. For example, a ground motion, measured or synthesized, with a more uniform response spectrum over the entire frequency range under consideration may be more desirable in removing the impact of ground motion as a variable affecting the behavior of the specimens. Alternatively, the effects of large pulses (near-fault effects) versus more regular cyclical excitations could be considered. Finally, rather than welding a specimen column to its base plate, clamping the column base by the plate itself may be worthy of consideration.

ACKNOWLEDGEMENTS

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