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Inelastic Dynamic Analysis of a Precast Concrete Office Building Subjected to Moderate Seismicity

S.-G. Low[1], X. Huo[1], A. Einea[2], M.K. Tadros[3] and D.C. Salmon[2]

ABSTRACT

An inelastic dynamic analysis of a precast concrete office building using the computer program DRAIN-2DX is performed. The analytical models of the building include a monolithic cast-in-place and four precast systems. Four ground acceleration records are used to simulate earthquake ground motions.

Comparison of the analysis results shows that the base shear of the precast systems is consistently smaller than that of a monolithic system. Smaller base shear results in less forces in the precast members. In addition, the results also indicate that the stiffness of precast concrete systems is reduced considerably by the presence of the joints. This behavior is reflected by larger story drift and roof displacement. However; the critical value of the drift is well below acceptable limits for this type of structures.

Based on this study, it is concluded that a precast concrete system can be designed to withstand moderate level earthquake with a ductile behavior which results in satisfactory drift limits and potentially low level of irreparable damage.

INTRODUCTION

To better understand the seismic behavior of precast concrete structures, the National Science Foundation (NSF), collaboratively with Japan, has initiated the three-phase Precast Seismic Structural Systems (PRESSS) research program. As a part of the research sponsored under PRESSS program, phase II, an inelastic dynamic analysis of a precast concrete building system is conducted. The study consists of analyzing models of five structural models with identical geometry. More detailed description of the analytical studies is presented by Low (1995).

The studied models include a monolithic cast-in-place system, and four precast systems. The monolithic and precast models contain elements which respond inelastically beyond their proportionality limits. The purpose of this study is to investigate the seismic performance of precast systems and to seek connections which have characteristics suitable for use in moderate seismic regions. The models are subjected to four ground acceleration records. Peak ground acceleration of each record is scaled appropriately to represent the Uniform Building Code (UBC) Zone 2 events (ICBO, 1991). The time-

[I] Ph.D. Candidates, Dept. of Civil Engineering, University of Nebraska-Lincoln, Omaha, NE 68182
[II] Assistant Professors, Dept. of Civil Engineering, University of Nebraska-Lincoln, Omaha, NE 68182
[III] Cheryl Prewett Professor, Dept. of Civil Engineering, University of Nebraska-Lincoln, Omaha, NE 68182

history analyses are performed using the program DRAIN-2DX (Prakash, 1993). Due to space limitation, only the results obtained from the El Centro earthquake acceleration record is presented in this paper.

DESCRIPTION OF THE BUILDING AND ANALYTICAL MODELS

The study focuses on a six-story office building with a typical floor-to-floor height of 4.0 m. Plan dimensions are 31.1 x 68.3 m. The building consists of interior gravity load resisting frame and a "dual system" for lateral load resistance. The lateral load resisting "dual system" consists of shear walls surrounding the stair cases and elevator shafts, and moment resisting spandrel frames at the perimeter of the building. Detailed description of the structural system is presented by Tadros et. al (1994).

Figure 1 shows the typical two-dimensional model layout for one half of the building. The model is built for one half of the building only to benefit from symmetry. The frame beams and columns are modeled using beam-column elements while the shear walls are modeled using elastic panel element. All the models are identical except for the joints. Precast connection joints are modeled using connection elements which have no physical dimensions. In the monolithic model, no descrete joints are assumed, and the frames and shear walls are assumed fixed at the foundation. The precast concrete models, however, are assumed to have semi-rigid fixidity at the foundation in addition to the descrete joints which simulate the field constructed joints in the locations shown in Fig. 1. Diaphragm action is modeled by slaving the joints of each level, and thus, forcing them to move equally in the horizontal direction.

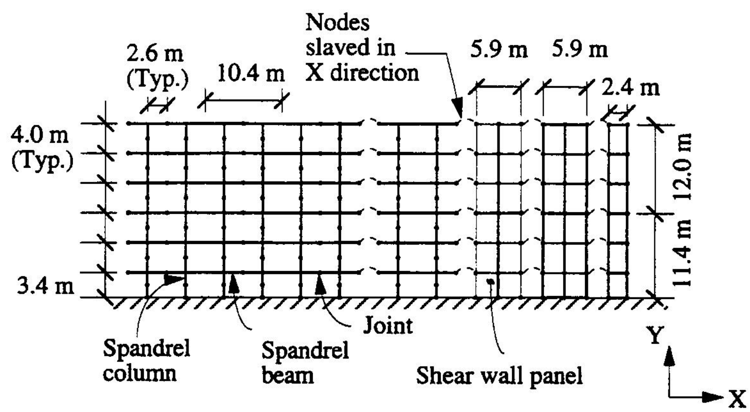


Fig. 1 - Typical model layout of the lateral load resisting system.

Member sizes and reinforcement were obtained by designing the building in accordance with the 1991 UBC with some modifications (Magana et al., 1994).

CONNECTION PROPERTIES AND BEHAVIORAL MODELS

The characteristics of the connection elements used for the various precast models are varied to monitor the corresponding changes of the overall models' behavior. Each element is described by its initial stiffness, ultimate capacity, and its hysteretical behavior. The properties of the connection elements for Precast System 1 are estimated based on published test results and available guidelines and is used as the baseline for comparison. In Precast System 2, the strength of the panel horizontal joint element is reduced by 25% compared to that of Precast System 1. Strength and stiffness of the frame joints in Precast System 3 are increased by about 300% while the strength and stiffness of the shear wall connections are reduced to 25% of Precast System 1. Precast System 4 is identical to System 1 with the exception that two elements with different behavioral models are used for each joint to more accurately simulate the joint behavior assumed in Precast System 4. Only the properties of the joints in Precast System 1 are presented in Table 1, the properties of joints in the other precast systems are presented by Low (1995).

Table 1. Properties of joints in Precast System 1.

	Direction	Initial Stiffness[1]	Yield force/moment (kN/kN.m)	Elasticity code[2]
FRAME JOINTS				
	X	0.194	2600/2600	2
Column-column and	Y	1.751	1,920(T)/11,500(C)[3]	2
column-foundation	Rotation	0.222	12,870/12,870	2
PANEL JOINTS				
Vertical joint	X	1.751	1,110(T)/1,110(C)	2
	Y	0.033	100/100	0
Horizontal joint and	X	0.590	680/680	2
panel-foundation	Y	1.751	940(T)/8,620(C)	1

(1) kN/m for force and kN/rad for moment
(2) 0 = Unload inelastically; 1 = Unload elastically; 2 = Unload inelastically with gap
(3) T = tension; C = compression

Column-Column and Column-Foundation Joints

All column-column and column-foundation joints are assumed to have the same properties, and each joint consists of three type 4 elements: X translational, Y translational, and a rotational element.

The initial stiffness of the X-translational element is extrapolated from a test reported presented by the Splice Sleeve Japan, Ltd. (1990). Its ultimate capacity is determined using the equation proposed by Foerster, et al. (1989). The initial stiffness of the column-column connection in the vertical (Y) direction is assumed to be large since the column is supporting a large amount of gravity loads. Tension and compression capacity of the connection is assumed equal to that of the column.

The assumed hysteretic behavior of column-column and column-foundation joints is shown in Fig. 2. These elements are assumed to unload inelastically with a gap and dissipate some energy in the process.

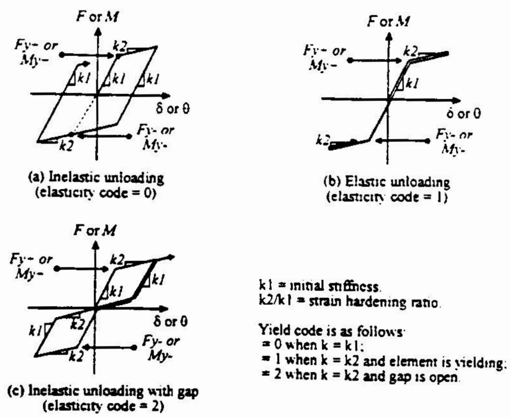


Fig. 2 - Behavioral models of connection elements (Prakash et al., 1993).

Panel-Panel and Panel-Foundation Joints

The panel-panel joints are assumed the same as the panel-foundation joints. Two types of joints elements are used to model each joint along the horizontal and vertical joints between the wall panels: X-translational and Y-translational elements. The initial stiffness and strength of the Y-translational element in the vertical joint are obtained from test results reported by Schultz et al. (1994). The initial stiffness of the X-translational element is assumed to be very large, and the yield force is estimated based solely on the tensile strength of the connector that is used in the panel tests.

The initial stiffness of the panel connection elements in the horizontal joint is estimated according to the results presented by Splice Sleeve Japan, Ltd. (1990). The initial stiffness is assumed to be directly proportional to the concrete area. Yield strength of the joint is computed using the equation proposed by Foerster et al. (1989). Yield force in tension and in compression of the element are obtained by treating the wall panel as a compression member.

GENERAL ASSUMPTIONS

Stiffness of the element in rigid end zones of the frames and second-order or "P-Δ" effects are taken into account in the analysis. Gravity loads are applied to the model as nodal forces. The inertial mass of the structure is assumed to be lumped at the nodes, and the total seismic weight at each floor is distributed equally to all nodes in that floor. A five percent viscous damping ratio is assumed for beam-column elements to account for miscellaneous energy losses. Connections and shear wall panels are assumed to have negligible viscous damping. Effects of vertical acceleration are not accounted for in the analysis, and the capacity of the foundation is assumed adequate to prevent uplift of the structure.

TIME-HISTORY ANALYSIS

Earthquake Records

The ground acceleration records used for this study are 1940 El Centro NS, 1952 Taft S69E, 1966 Parkfield N65E, and 1986 San Salvador (CIG Station). The time increment for all records is 0.02 seconds.

Fundamental Periods

Fundamental periods of all models are computed prior to time-history analysis. In general, the periods of precast models are higher than that computed from Eq. 34-3 of the 1991 UBC and that of the monolithic system model, as much as 70 percent larger (see Table 2). A larger period indicates a reduction in stiffness due to the presence of the joints.

Table 2. Fundamental periods.

System	(UBC), 1991	Monolithic	Precast 1	Precast 2	Precast 3	Precast 4
Period (seconds)	0.4598	0.4572	0.7854	0.7854	0.7909	0.8076

Base Shear

Base shear values of all systems are summarized in Table 3. These values represent half of the total shear experienced by the building, as the mathematical model represents only half of the structure. The noticeable difference between the base shear computed by the UBC 1991 equation and that obtained for the monolithic model is due to the reduction factor R_w applied to the former. The ratio between the two base shear values, however, is 1:4 which is half the 1:R_w ratio. This difference indicates that the El Centro acceleration record is not the most severe earthquake for this building (after being scaled to 0.2 g). It does not excite the model to generate maximum base shear. In addition, the base shear values for the precast models are considerably lower than that of the monolithic system in all cases. This is due to the fact that the stiffness of the precast models is less than that of the monolithic system. A system with larger stiffness attracts more inertial force.

Varying the properties of the connection elements results in the following behavioral differences: 1) reducing the strength of the panel-panel horizontal joint decreases the total base shear and increases the percentage of shear resisted by the frames (see Precast 2); 2) appropriate adjustment in the properties of the panel-panel and column-column joints can redistribute the base shear between the frames and the shear walls (see Precast 3); and 3) changing the behavioral model of the panel-panel horizontal joints can significantly affect the response of the system (see Precast 4). In general, the magnitude of the base shear is significantly affected by the strength and stiffness of the panel-panel horizontal joint connections.

Time-history plots of the base shear due to El Centro acceleration record for the Monolithic System and Precast System 1 are illustrated in Fig. 3. The Monolithic System experiences higher base shear at about two seconds into the record followed by smaller shear. On the other hand, the precast systems experience a lower base shear, but at a relatively large magnitude, for three consecutive cycles. This might cause greater damage in the precast systems than the monolithic systems.

Table 3. **Base shear for all systems.**

System	UBC 1991	Monolithic	Precast 1	Precast 2	Precast 3	Precast 4
Total (MN)	2.49$^{(1)}$	10.31$^{(2)}$	8.02$^{(2)}$	7.26$^{(2)}$	7.78$^{(2)}$	10.08$^{(2)}$
% resisted by frames	55	54	15	31	46	6
% resisted by panels	45	46	85	69	54	94

$^{(1)}$ This is calculated assuming $R_w = 8$ and $S = 1.0$
$^{(2)}$ Subjected to El Centro 1940 NS record

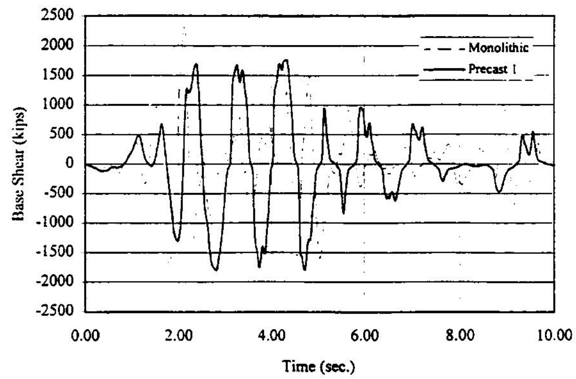


Fig. 3 - Base shear response to El Centro acceleration (1 kip = 4.45 kN).

Roof Displacement and Drift Ratio

Table 4 shows the roof displacement and the maximum interstory drift ratio of all systems. The roof displacement of the monolithic system is about 2.8 times greater than that calculated by the UBC 1991 equation, and the maximum drift ratio is approximately 2.6 times greater.

Table 4. Displacement at the roof and maximum drift ratio of all systems.

Type of analysis/system	Displacement at roof level (mm)	Maximum drift ratio (%)
UBC 1991	6.6	0.040
Monolithic*	18.6	0.105
Precast 1*	61.8	0.499
Precast 2*	84.0	1.121
Precast 3*	78.3	0.849
Precast 4*	60.6	0.326

*Subjected to El Centro 1940 NS record

Due to the reduction in stiffness, the roof displacement and story drift of precast systems are greater than those of the monolithic system. However, the maximum drift ratio is well below the recommended limits for precast systems (1.5% for shear walls and 2% for moment frames). The roof displacement response of the monolithic system and Precast System 1 is depicted in Fig. 4.

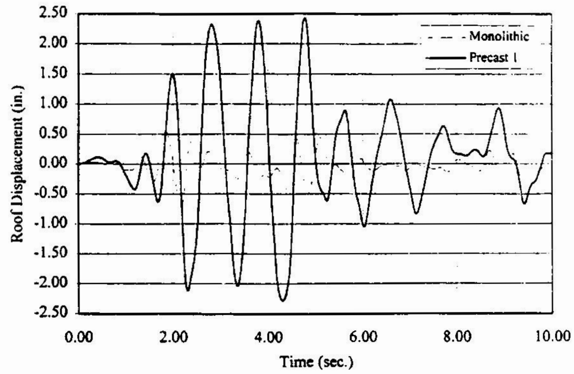
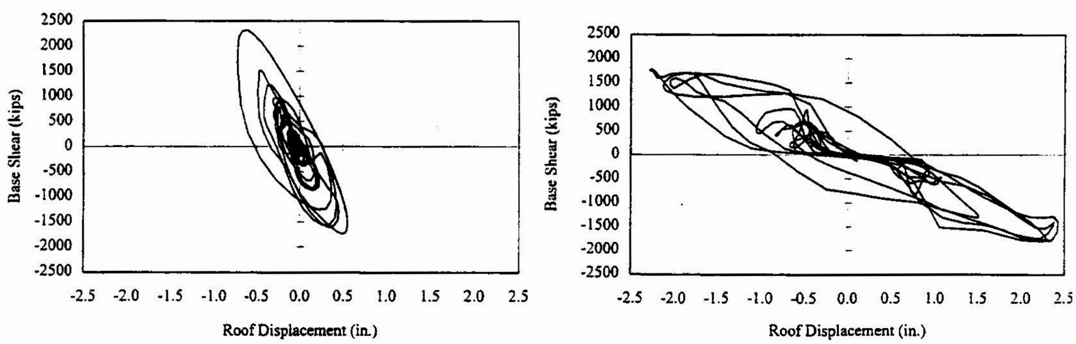


Fig. 4 - Roof displacement response to El Centro record (1 in. = 25.4 mm).

SYSTEM RESPONSE

Roof displacement versus base shear plots of the Monolithic and Precast System 1 models subjected to El Centro record are shown in Fig. 5. Among the two systems, the monolithic system exhibits a larger strength and stiffness. However, the energy dissipation of the monolithic system is less than that of the precast system, as can be seen from the total area within the hysteresis loops.



(a) Monolithic System (b) Precast System 1

Fig. 5 - System response to El Centro record (1 kip = 4.45 kN, 1 in. = 25.4 mm).

Because an greater amount of the base shear is resisted by the walls, the behavior of Precast System 1 is identical to the behavior of the X-translational horizontal panel joint element. The hysteresis loops are pinched during inelastic loading.

CONCLUSIONS

Dynamic analysis of a few lateral load resisting systems is performed as a part of the PRESSS phase II research program. The analytical models simulate a monolithic, and four precast concrete systems. The results of the analysis of the monolithic and precast systems show that the base shear of a monolithic system is higher and the maximum interstory drift ratio is lower than that of precast systems. Changing the properties of the panel horizontal joints can affect the behavior of the system.

Field joints in precast concrete buildings reduce their overall stiffness compared with cast-in-place monolithic buildings. The increased interstory drift ratios, however, can still be within the acceptable limits by building codes and practical recommendations. The reduction in stiffness, however, results in lower base shear which allows these buildings to be designed and behave satisfactorily in earthquakes.

REFERENCES

Foerster, H.R., Rizkalla, S.H., and Heuvel, J.S., Behavior and Design of Shear Connections for Loadbearing Wall Panels, PCI JOURNAL, Jan.-Feb. 1989, pp 102-119.

International Conference of Building Officials, Uniform Building Code, Whittier, CA, 1991.

Low, S-G., Behavior of a Six-Story Office Building Under Moderate Seismicity, a dissertation submitted in partial fulfillment of requirements for the degree of Doctor of Philosophy, University of Nebraska, Lincoln, Dec. 1995.

Magana, R.A., Tadros, M.K., Einea, A., Low, S-G., and Schultz, A.E., Preliminary Seismic Design of a Six-Story Precast Concrete Office Building, submitted for possible publication in the PCI Journal.

Prakash, V., Powell, G.H., and Campbell, S., DRAIN-2DX, Base Program Description and User Guide, Version 1.10, University of California, Berkeley, California, Nov. 1993.

Schultz, A.E., Magana, R.A., Tadros, M.K., and Huo, X., Experimental Study of Joint Connections in Precast Concrete Walls, Proceedings, Fifth U.S. National Conference on Earthquake Engineering, Chicago, IL July 10-14, 1994.

Splice Sleeve Japan, Ltd., Tests on Re-Bar Splices In Reinforced Concrete Columns Using NMB Splice Sleeves, Test Report, Tokyo, Japan, 1990.

Tadros, M.K., Einea, A., Low, S-G., Magana, R.A., and Schultz, A.E., Seismic Behavior of a Six-Story Precast Office Building, Proceedings, FIP 94 XII Congress, Washington D.C., May 29-June 2, 1994, Vol. 1, pp. E16-22.