EVALUATING CURRENT PROCEDURES AND MODELING FOR SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS

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SUMMARY

A seven-story reinforced concrete building was built as a hotel in Van Nuys, California in 1966. The structure has been instrumented since its construction and records are available for three earthquakes (1971 San Fernando, 1989 Whittier, and 1994 Northridge), after the last of which shear failures were observed in the columns of the south face of the fourth floor. As a regular structure with structural and nonstructural details that are typical of building construction prior to the 1970's, this building offers an ideal opportunity to compare analytical response with recorded response.

The scope of the project, which was conducted over a period of one and a half years, involved performing two- and three-dimensional analyses of the building with the intent of modeling the observed damage and recorded structural response. The analyses included both structural and nonstructural elements. The nonlinearity of both structural and nonstructural elements were modeled where appropriate.

The different non-structural components that possibly participated in, and influenced the dynamic response of the seven-story building, were identified. Examples of nonstructural components that were studied included masonry infill walls, partitions, exterior non-structural walls, stairs, and stair and elevator enclosures. The effects of these components will be assessed by comparing the analytical response of the building, both with and without nonstructural components, to the recorded response.

INTRODUCTION

Performance-based engineering of a structure typically requires structural analysis and performance assessment of the various participating structural and nonstructural elements. Current analysis methods may lack the desired level of accuracy and reliability needed to make an effective evaluation of a structure due to the simplifications made in modeling the structure and to the limitations imposed by currently available analysis platforms. Simplifications in capacity evaluation include: material behavior and interaction (i.e. reinforced concrete), exclusion or inappropriate modeling of nonstructural elements, ignoring the three dimensionality of structural response, and ignoring soil structure interaction. Simplifications in the demand evaluation, particularly as related to earthquake response, are another contribution to a loss of accuracy.

The building studied has been instrumented since its construction in 1966, and records exist for the 1971 San Fernando, 1987 Whittier Narrows and 1994 Northridge earthquakes. The structure was heavily damaged and subsequently closed for repair and retrofit after the Northridge earthquake. Under earthquakes of varying demand the building has exhibited several damage states and thus presents a good example for comparison of actual response to the analytical response predicted by various evaluation procedures and analysis platforms.

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This paper describes the studied building, how it was modeled using the various analysis platforms, the type of analyses performed and the current results of the performance assessments. The main goal is to identify the strengths and weaknesses of existing analysis platforms.

BUILDING DESCRIPTION

The building is located in Van Nuys, California. The site is near the center of the San Fernando Valley, approximately 15 miles from the epicenter of the 1971 San Fernando earthquake, 25 miles from the epicenter of the 1987 Whittier Narrows earthquake, and 4.5 miles from the epicenter of the 1994 Northridge earthquake. Originally designed in 1965, the approximately 65,000 square foot structure was built in 1966 [Murphy 1973]. Its primary use has been as a hotel, with restaurants, lobby and services on the first floor.

Building Configuration/Structure

The building has a seven-story, reinforced concrete structure with a rectangular floor plan of approximately 60 ft by 148 ft. The first floor is designed for support services such as lobby space, restaurant, banquet/conference rooms, etc. The remaining six floors are hotel rooms arrayed off of a central corridor running the length of the building. The penthouse is a metal-stud framed structure constructed on the roof level for housing the hotel's mechanical system. Applicable standards in place at the time of design include the 1961 Uniform Building Code [ICBO 1961] and the American Concrete Institute's ACI 318-63 [ACI 1963].

The structural system comprises reinforced concrete perimeter beam-column framing and interior slab-column framing supported on reinforced concrete piles. The rectangular plan contains eight bays in the East-West longitudinal dimension and three bays in the North-South transverse direction (Figure 1). The plan of the structure is regular and symmetric with the exceptions of an exterior canopy on the east side of the first floor, and an external stair tower on the east side at the northern corner. Four bays of the first floor framing on the east side of the North face are infilled with lightly reinforced brick walls. Expansion joints separate the sides of the brick wall from the surrounding columns and overhead spandrel beam.

The gravity load path is simple and direct, involving two-way slabs with exterior spandrel beams that transmit gravity loads to columns that are continuous from roof to foundation level.

The lateral load path is also relatively simple. Primary masses are the floor slabs, which act as diaphragms transmitting floor-level inertial loads to the supporting columns. It is likely that the design was based on the assumption that the perimeter frames acted as the main structural elements to resist lateral loads. Some lateral resistance is also available from the interior slab-column frames. Exterior walls on the East-West faces are covered with 1 in. thick plaster panels. All bays on the North and South faces are windowed (excepting one bay on the east side of the north face that is filled with plaster panels over the building height, similar to those on the east and west faces; and lightly reinforced masonry walls fill four bottom story bays of the North face).

Interior partitions are gypsum board over metal-stud framing. In the transverse, or North-South direction, the structure is divided into rooms separated by the interior partitions at approximately 12 ft centers. Although the interior partitions could be (and in these analyses, were) assumed to be non-load bearing, reconnaissance teams after both the 1971 San Fernando and 1994 Northridge earthquakes noted extensive damage to the interior partitions, suggesting possible interaction with the structural system.

Concrete and reinforcing steel design strengths were varied over the height of the structure. Normal-weight aggregate concrete was used throughout, with design strengths ranging from 3000 to 5000 psi. Grade 40 reinforcing steel was used in the beams and slabs, and grade 60 steel was used in the columns.

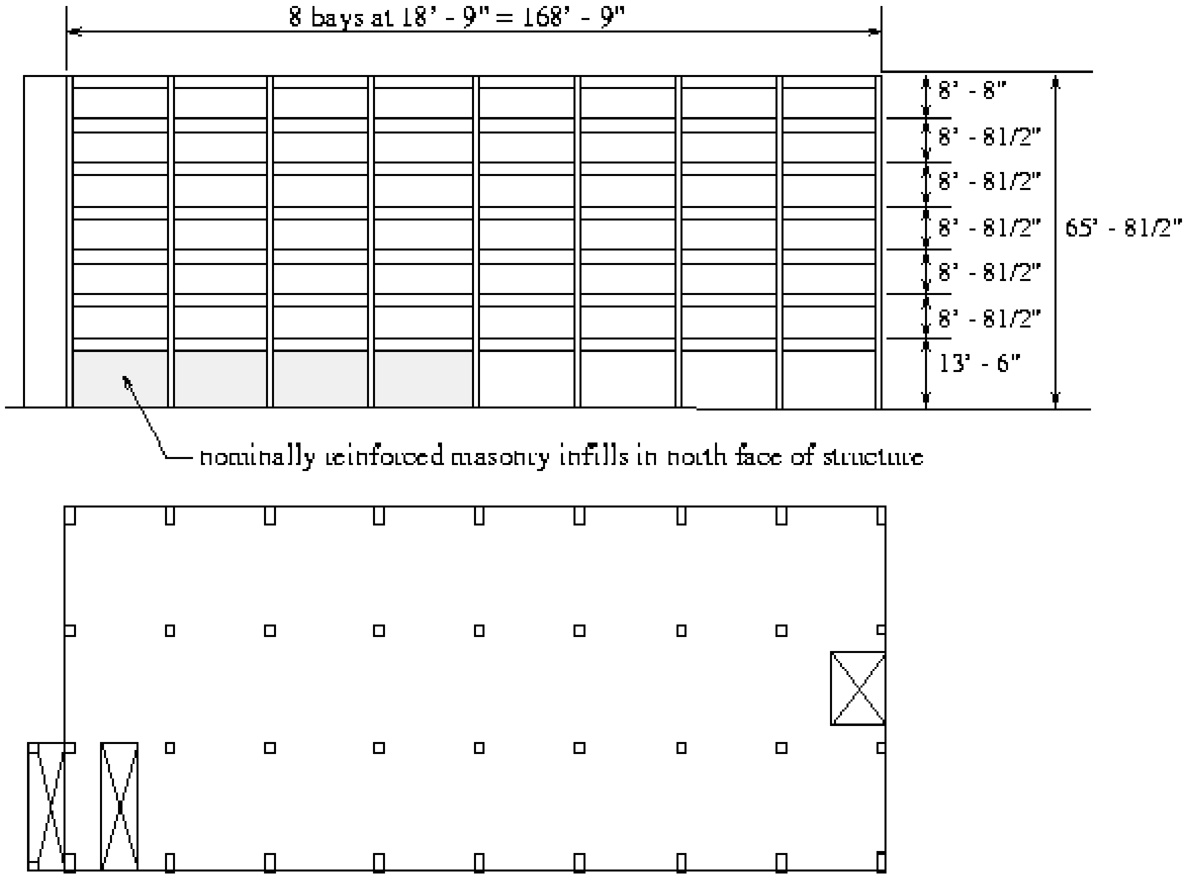

Figure 1: Plan and elevation of existing structure

Building History

Since its construction in 1966 the structure has been exposed to three earthquakes of notable magnitude: the 1971 San Fernando, 1987 Whittier Narrows, and 1994 Northridge earthquakes. The building's response and the observed damage from the three earthquakes are summarized in the following sections.

Accelerometers were designed and incorporated into the building's construction for the purpose of recording the building's response to earthquakes. Triaxial accelerometers were in place on three levels of the structure for the 1971 San Fernando earthquake. Prior to the 1987 Whittier Narrows earthquake, the structure was instrumented with a total of 16 sensors: 5 measuring longitudinal accelerations, 10 measuring transverse accelerations, and 1 to measure vertical accelerations at the ground floor.

1971 San Fernando Earthquake - Approximately 40 seconds of motion were recorded for the San Fernando earthquake by Earth Sciences AR-240 strong-motion accelerographs. In the longitudinal direction, the peak acceleration at the ground floor was 0.13 g, reached at 9.0 sec. Peak longitudinal acceleration at the roof level was 0.33 g at 9.2 sec. In the transverse direction, the peak acceleration at the ground floor was 0.25 g at 12.5 sec. Peak transverse acceleration at the roof level was 0.41 g at 9.9 sec. Vertically, the peak acceleration at the ground floor was 0.18 g at 3.6 sec. Peak vertical acceleration at the roof level was 0.24 g at 3.6 sec [CDMG 1994].

Analyses of the data indicate a fundamental period in the longitudinal direction of 0.6 to 0.7 sec during the initial 7 sec of the earthquake record. A transition occurred from 7 to 13 sec in the record leading to a fundamental period of approximately 1.5 to 1.6 sec [Murphy 1971].

Structural damage was limited to the beam-column joints on the east side of the north face. Some cracking was noted between the spandrel beams and the columns on the exterior.

Observed non-structural damage was extensive. Damage was observed on all floors, but was most severe on the second and third floors. About 80 percent of the repair cost was spent on drywall partitions, bathroom tile, and plumbing fixtures. Spalling of architectural concrete and the exterior plaster panels also required repairing. Although no doors and windows required replacement, many required alignment and/or caulking [Murphy 1973].

1987 Whittier Narrows Earthquake - Forty seconds of motion were recorded for the Whittier Narrows earthquake. Peak ground floor accelerations for the 1987 Whittier Narrows earthquake were 0.17 g in the transverse direction, reached at 3.6 sec. Peak accelerations at the top floor were 0.20 g, reached at 9.2 seconds in the transverse direction, and 0.17 g in the longitudinal direction. The period of the structure at the end of the record was 1.2 sec. There was no observed damage to the structure from the 1987 Whittier Narrows earthquake [Abdel-Ghaffar 1990].

1994 Northridge Earthquake - Sixty seconds of motion were recorded for the Northridge earthquake. In the longitudinal direction, the peak acceleration at the ground floor was 0.47 g, reached at 9.0 sec. Peak longitudinal acceleration at the roof level was 0.59 g at 9.0 sec. In the transverse direction, the peak acceleration at the ground floor was 0.40 g at 5.5 sec. Peak transverse acceleration at the roof level was 0.58 g at 4.5 sec. Vertically, the peak acceleration at the ground floor was 0.28 g at 2.0 sec. Peak vertical accelerations at the roof level were not measured.

Analysis of the 1994 Northridge earthquake records [CDMG 1994] indicates an initial fundamental period in the longitudinal direction of approximately 1.0 sec during the first 2 sec of the record. There is a transition from 4.0 to 5.0 sec into the record, after which the fundamental period is approximately 1.7 sec [Stewart 1996].

Observed structural damage was concentrated in the bottom floor and the top of the fourth floor columns (Figure 2). Horizontal cracking at the column bases of the north face was also observed. The failures at the tops of the south face exterior columns at the fourth floor show inclined cracking, buckling of longitudinal steel, and splitting of the concrete along the longitudinal reinforcement (Figure 2). The damage suggests distress due primarily to shear and bond splitting.

Figure 2: South face of structure (left), and detail of damaged column (right) - damage after the 1994 Northridge earthquake [EERC 1994]

STRUCTURE MODELING

The building was analyzed in the longitudinal direction. The four column lines were used as the basis for creating four discrete frames that were rigidly linked at each floor using the assumption that the slab floors acted as rigid diaphragms. The exterior frames consisted of the columns and the spandrel girders with the participating amount of slab based on tributary width. Interior slab frames consisted of the interior columns with equivalent beams using the modified strengths and stiffnesses specified in ATC 40 [Comartin 1996] and FEMA 273 [FEMA 1997]. There was no attempt made to model the foundations, and the columns were assumed to be fixed at the base.

Building masses were simplified as lumped masses at the nodes. The full structure was included in the mass calculation with non-structural mass estimated as ten percent of the structural contribution. Calculations were made to estimate the mass of the penthouse structure and the accompanying HVAC equipment on the roof. The gross building weight was calculated to be 9480 kips.

The specified concrete strengths were used to calculate member strengths and stiffnesses. Specified reinforcement strengths were increased by 25 percent, as was recommended in ATC 40 and FEMA 273.

DRAIN-2DX MODELING

Structural Elements

Beams and columns were modeled using the plastic hinge beam-column element (type 02), which allows for the FEMA 273-recommended procedure of specifying member stiffness values. Version 1.10 of DRAIN-2DX [Prakash 1993] allows only a bilinear stiffness to be specified - a limitation when stiffness and/or strength degradation is desired.

Flexural strengths of the beams, columns and slabs were determined using the ACI procedure for calculating nominal strengths. The value for fy specified in ACI 318 was substituted with 1.25 fy. Column strengths were calculated accounting for axial load-moment (P-M) interaction, and the envelope for the P-M curves was simplified for the DRAIN-2DX elements.

Gross-section stiffnesses were used and modified using the values provided in the draft FEMA 273. For the beams in the perimeter frames, fifty percent of the gross section stiffness of the combined beam and effective width of the slab (as defined by ACI-318, Section 8.10) was used. Seventy percent of the gross section stiffness was used for the columns.

The equivalent widths of the slabs for the interior frames were tributary widths of one half of the adjacent span. The draft FEMA 273 indicated the slab flexural stiffness was to be 0.35EcIgKfp , where Ec is the modulus of elasticity for concrete as defined by ACI, Ig is the gross moment of inertia, and Kfp is a factor accounting for whether the slab support is exterior, interior or corner.

Nonstructural Elements

The architectural walls of the building were included using a model developed specifically for architectural walls (Smith and Vance, 1996) that is incorporated into a modified version of DRAIN-2DX. The model is based on a plane-stress quadrilateral element with four nodes and eight degrees-of-freedom; mass is lumped at the nodes. A pinched hysteresis model is used to represent the load-deformation characteristics of the walls. The pinched hystersis model accounts for both stiffness degradation and strength deterioration.

It was necessary to use a modified version of DRAIN-2DX to model the architectural walls since the structural panel element available in the commercially available version of the program may be used to represent linear behavior only and the behavior of architectural walls is highly nonlinear. While a nonlinear frame element (which could be used as an equivalent strut) is available in the commercially available version of the program, it may be used to represent bilinear elastic-plastic load-deformation behavior only – with no stiffness degradation or strength deterioration.

Assumptions made in the analyses including nonstructural elements are listed below.

- Only interior partitions and masonry infill walls were included – exterior partitions in the longitudinal direction were not included since they have numerous perforations;
- Corridor walls are assumed to be full-height while walls in rooms are assumed to be partial-height (to the bottom of the suspended ceiling);
- Wall stiffness was reduced to account for openings;

- Wall configurations were assumed to be 25-gage metal studs at 24 inches o.c. with a single layer of 5/8 inch type X gypsum wallboard;

- The following properties were assumed for the analytical model based on experimental data from Rihal and Granneman (1985) and Freeman (1966): Elastic stiffness = 4.5 k/in; Strain hardening = 0.015; Pinching stiffness = 0.3 kips; Peak displacement = 0.75 inches; and Failure displacement = 1.5 inches.

SAP 2000

Structural Elements

The structure was modeled in SAP 2000 in much the same way it was modeled in DRAIN-2DX. In SAP 2000, non-linear frame elements with two nonlinear hinges per element (one at each end of the element) were used to model the structural frame in three dimensions. The flexural strengths of the columns were determined by using the BIAX program [Wallace 1989]; the value for the reinforcement yield strength, fy specified in the ACI was substituted with 1.25 fy. Beam and slab element moment curvature relationships were obtained using BIAX, again replacing fy with 1.25 fy. The equivalent width of the slabs for the interior frames was 50 inches. The primary differences in the modeling are listed below.

- Non-structural mass was estimated as 20 psf (partition loading).

- The gross building weight was calculated to be 9484 kips.

- SAP allows for strength degradation only.

Non-Structural Elements

Masonry infill walls were modeled as equivalent struts using the FEMA 273 recommendations.

Interior partitions were typically steel-stud framed with gypsum wall-board finish. Only the full-height partitions were included in the SAP2000 model. The full-height partitions were modeled as equivalent struts, based on an average lateral resistance of 50 plf and a corresponding lateral displacement of 0.25 inch. These equivalent struts were assumed to be placed at the centerline of column lines.

The exterior façade at each end of the building, and at the stair and elevator bays on the longitudinal (south) face of the building, consists of one inch thick cement plaster applied over double 16 gage metal-stud framing. This exterior façade enclosure was modeled using one inch thick shell elements. The modulus of elasticity was assumed to be 1.5×10^6 psi.

RESULTS AND DISCUSSION

Figure 4 plots the top floor displacement versus time of both the DRAIN-2DX model and the actual structure. The peak calculated displacement was 6.0 inches, as compared with the actual peak displacement of 9.2 inches. Additionally, beyond t = 4 seconds, the computed response and period is notably less than that for the actual response. This is most likely due to the fact that stiffness and strength degradation characteristics were not considered in the structural model.

Browning et al [Browning 1997] note that including stiffess degradation or both stiffness and strength degradation provides a much closer approximation of actual response in terms of story displacement and structure period. Although, the analytical model of the structure in this report was as complete and as accurate as possible, including all possible interacting elements, the lack of strength and or stiffness degrading elements greatly impacts the modeled response.

Modal analysis of the SAP 2000 model provided a first modal period of 1.604 seconds with the infill walls but without the interior partitions, and 1.067 seconds with the infill walls and the interior partitions.

Potential problems are being addressed with the nonlinear pushover capabilities of SAP2000. At present the nonlinear pushover runs do not converge when user-defined hinge properties are utilized. The results will be published at a later date.

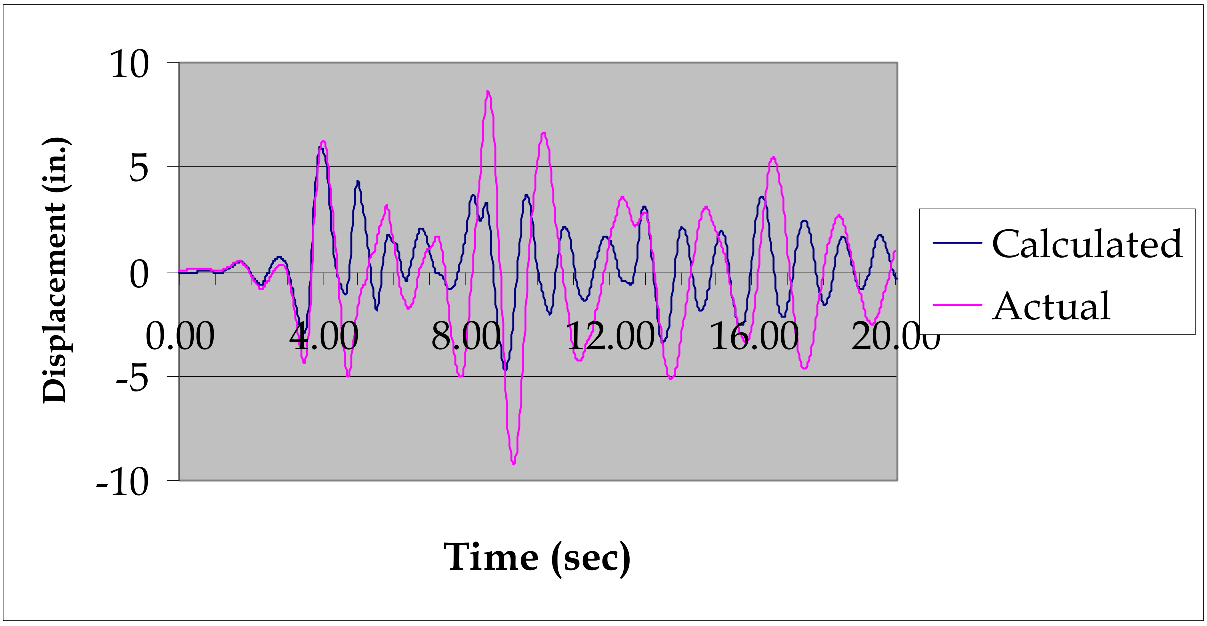

Figure 4: Calculated and actual top floor displacement versus time

CONCLUSIONS AND RECOMMENDATIONS

Without the capability of modeling stiffness and strength degradation at the element level the response history analysis using DRAIN-2DX provides unsatisfactory results. Peak displacements are substantially under estimated and changes in the building's natural modal period are not possible.

Additional potential problems/issues with the DRAIN-2DX program are noted below.

Three dimensional analysis with the lumped plasticity beam column elements is not possible.

Stiffness or strength degradation in either the lumped plasticity beam-column element or the strut element cannot be modeled.

The program is in need of a pre- and post-processor.

The analyses using the SAP 2000 Nonlinear Push program are still in progress. Some issues encountered with the version of the program are noted below.

There is no capability for nonlinear response history analysis.

The strut and plate elements provided in the program have only linear properties, which will potentially decrease the effectiveness of modeling torsional response of the structure with assymetric wall placement.

ACKNOWLEDGEMENTS

The research for this report was funded by the Pacific Earthquake Engineering Research Center (PEER - an engineering center funded by the National Science Foundation) under the UCB Subaward Number SA1820JB. Additional support by a number of students through the PEER Undergraduate Summer Intern program is gratefully acknowledged.

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