SEISMIC EVALUATION AND RETROFIT OF A HOSPITAL BUILDING USING NONLINEAR STATIC PROCEDURE IN ACCORDANCE WITH ASCE/SEI 41-06

Y. Wang

Design Engineer, PhD , TMAD Taylor & Gaines, Pasadena, California, USA
Email: ywang@tmadtg.com

ABSTRACT

California Hospital Safety Acts require that, after January 1, 2008, any general acute care hospital buildings which continue acute care operation be upgraded to a minimum SPC-2, the Life Safety performance level. The building studied herein is a seven-story tower of an acute care complex in southern California. Following evaluation procedures, the building has an SPC-1 rating, which may significantly jeopardize life. Retrofit strategies are proposed following the ASCE/SEI 41-06 guidelines. To minimize retrofit cost, Nonlinear Static Procedure is adopted because it provides a more realistic estimate of the seismic demands under a large seismic event. Geometric, material, and soil-supporting nonlinearities are properly defined in establishing the pushover analysis model. Building performance and retrofit goals are examined and illustrated by analyses from various patterns of pushover loads.

KEYWORDS: Performance-based Design, Nonlinear Static, Pushover, ASCE 41, Seismic, Retrofit

1. INTRODUCTION

Many essential facilities such as hospital buildings are located in high seismic zones in the United States and throughout the world. Some of them were designed and built at a time without sufficient earthquake knowledge and are consequently susceptible to earthquakes. *California Hospital Seismic Safety Act* (SB1953) requires hospitals to evaluate and rate all their general acute care buildings for seismic resistance. It employs standards developed by the California *Office of Statewide Health Planning and Development* (OSHPD) to measure a building's ability to withstand a major earthquake.

Developed from FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, ASCE/SEI 41-06, *Rehabilitation of Existing Buildings*, is the latest generation of a performance-based seismic rehabilitation methodology and the consensus national standard for seismic rehabilitation. California Building Standards Commission has approved to adopt the ASCE/SEI 41-06 as a reference standard and effective from January 1, 2008. OSHPD has permitted to use ASCE 41-06 as a means of compliance on a case-by-case basis for the rehabilitation of hospital buildings. Some old hospital buildings may have high potential of significant damage and may endanger lives. These buildings fall into Structural Performance Category I (SPC-1) (SB1953). The OSHPD requires any hospital buildings be at a minimum SPC-2 level to continue acute care operation, which is equivalent to the Life Safety (LS) performance level in ASCE 41 (§3403A.2.3.4.3 of CBC' 07, Part 1, Title 24, California Code of Regulations).

The hospital building studied herein is a seven-story concrete building constructed before 1970. Based on the previous pseudo nonlinear static analysis, i.e., the nonlinearity is limited only to the foundation vertical stiffness using vertical nonlinear spring/link elements, retrofit measures are proposed to fix structural deficiencies. This study does a nonlinear static analysis for the retrofit building following ASCE 41 provisions, aiming to provide a more realistic estimate of the seismic demands and economic-effective retrofit strategy.

2. BUILDING DEFICIENCIES AND REHABILITATION MEASURES

The building is a seven story building of an acute care complex in southern California. The gravity load resisting system is reinforced concrete frame and nonbearing walls. The lateral load resisting system consists of the concrete roof and floor slabs, reinforced concrete shear walls and perimeter masonry piers, with concrete frame and internal concrete shear walls at the first story.

Typical concrete floor is 4.5" two-way and one-way slabs supported on concrete beams and columns, 4" slab on grade. Floor of mechanical penthouse area is 6" one-way slab. Reinforced concrete walls are typically 10 inch thick, with a few 12" and 15" wall at some lines. Perimeter reinforced masonry brick walls have two types: 8.5" fully grouted and 7.5" partially grouted. There are three types of footings: spread footings for walls, isolated pad footings for columns, and mat footing for stair walls. SAP building models are shown in Figure 1.

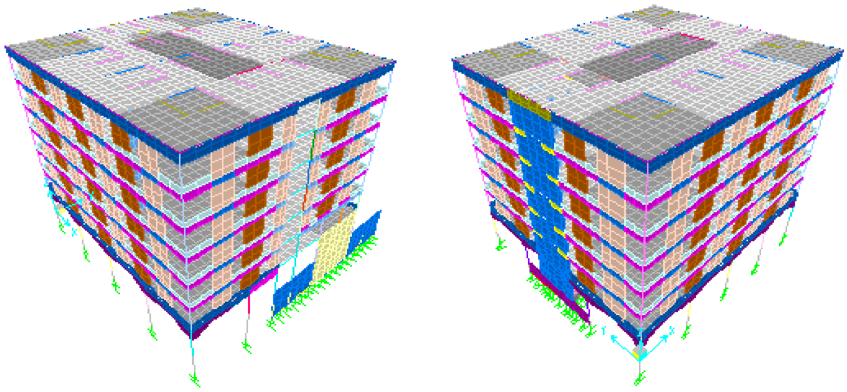


Figure 1 (a) Southeast Elevation (b) Southwest Elevation

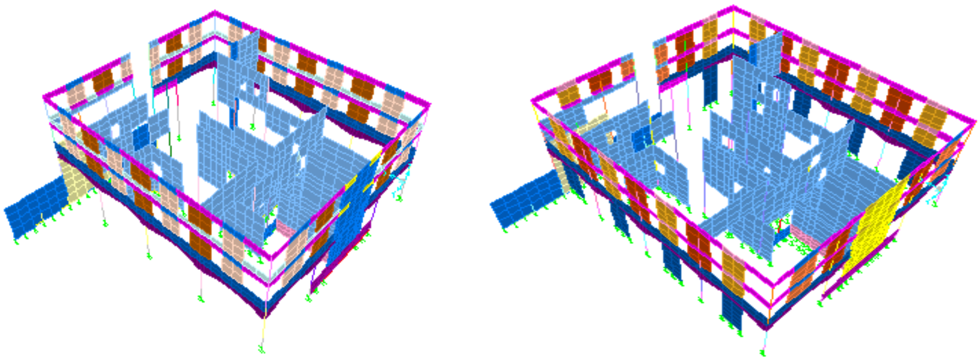
Per the Seismic Evaluation Procedure for Hospital Buildings in primary study, the as-built building is non-conforming with an SPC-1 rating. Its major structural deficiencies include:

1. Vertical Discontinuity: Some shear walls in the upper stories did not extend down to the lower stories.
2. Deflection Incompatibility: Gravity load resisting systems are not capable of accommodating imposed building drifts.
3. Shear Stress Check: Some shear walls do not have adequate shear capacity. Shear failure may occur.
4. Weak stories: A quick check based on linear static analysis shows that, the 2nd story in x-direction and the 3rd in y-direction story are weak. Ratios of strength to shear are 68% and 79.8% of the above floor.
5. Some walls do not have adequate total reinforcement. Coupling beams have too large spacing stirups.

These features are unfavorable for seismic resistance. Rehabilitation measures that may be effective to fix the above deficiencies include:

1. Infill between columns supporting discontinuous shear walls to make walls continuous and strengthen weak stories.
2. Increase shear strength of wall by casting additional reinforced concrete adjacent to the web and add new walls.
3. Use confinement jackets to improve deformation capacity of coupling beams and columns supporting discontinuous shear walls.
4. Add confinement jackets at wall boundaries.

Demand/Capacity ratios (many are >1.5) indicated that some components might have responded inelasticly. They shall be modeled as nonlinear inelastic elements in accordance with Tables 6-18 and 6-19 of ASCE 41 (§3413A.1.4 of CBC'07). Here, the rehabilitation model that includes measures 1 and 2 is to be studied to accurately estimate the seismic demand and thus decrease retrofit cost. The lower three stories of the as-built and the rehabilitation SAP models are shown in Figure 2 (diaphragms are hidden for easier comparison).



(a) (b)

Figure 2 Lower Three Stories of (a) Building As-built Model, (b) Rehabilitation Model

3. MODELLING FOR NONLINEAR PUSHOVER ANALYSIS

The fundamental periods in *x*- and *y*-directions are 0.3993 sec. and 0.445 sec., respectively. A modal response spectrum analysis to the retrofit model shows that the shear in any story considering modes required to obtain 90% mass participation only exceeds 106% maximum of the corresponding story shear considering only the first mode, so higher modes are not significant. Nonlinear static analysis is appropriate for this building (§2.4.2.1, ASCE 41).

3.1 Site-Specific Seismic Hazards and Modeling Parameters

ASCE 41 defines two seismic levels: Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2). BSE-1 corresponds to 10% probability of exceedance in 50 years. This paper only considers one performance goal, the Life Safety (S-3) at BSE-1 earthquake. The site-specific ground motion seismic hazard analysis and the soil seismic design parameters were performed by the geotechnical engineers in accordance with ASCE 41. The site-specific BSE-1 acceleration response spectrum is shown in Figure 3.

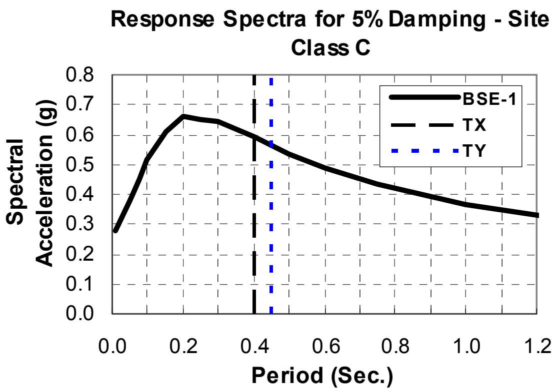


Figure 3 Site-Specific BSE-1 Acceleration Response Spectrum

3.2 Nonlinear Parameters:

In the structural models for seismic evaluations of the buildings, a flexible base assumption shall be used that incorporates foundation flexibility. The force-displacement relationship is idealized as a series of uncoupled elastic, perfectly plastic springs (translational and rotational) that are attached to the node at the bottom of the columns, walls, or mats. For this project, only the compressive vertical translational springs are considered. The stiffness of the foundation springs is based on a Modulus of subgrade reaction and reduced for the width of the footings. The force-deformation failure behavior of the springs takes into account only the soil failure (ultimate bearing capacities). The soil spring model is shown in Figure 4(a).

Generalized force-deformation relationship (Figure 4(b)) of building materials and components are described in related chapters in ASCE 41. Curve parameters are given in tables for different components. Deformation limit corresponding to these components are also given for performance levels of IO (Immediate Occupation), LS(Life Safety), CP(Collapse Prevention).

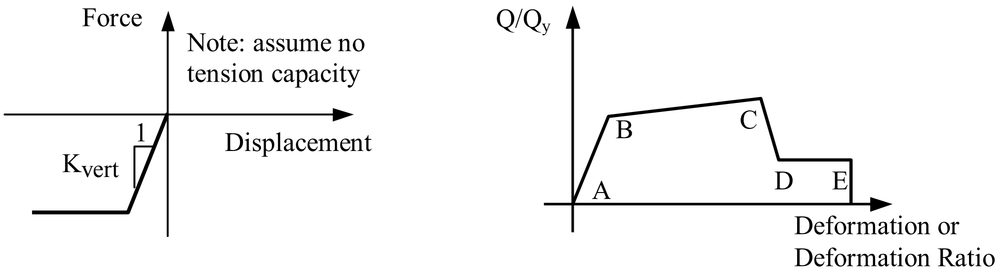


Figure 4 (a) Stiffness and Capacity of Vertical Foundation Springs
(b) Generalized Force-Deformation Relation

Curve (Figure 4b) parameters for plastic hinges are determined per Chapters 6 and 7 of ASCE 41. In this study, three types of hinges are defined for masonry coupling beam, frame beam, and column. Parameters are shown in Table 3.1:

Table 3.1 Frame Plastic Hinge Modeling Parameters for General Force-Deformation Curve

Column				Beam				Coupling Beam			
a	0.025	IO	0.01	a	0.025	IO	0.01	a	0.02	IO	0.006
b	0.05	LS	0.02	b	0.05	LS	0.02	b	0.04	LS	0.013
c	0.2	CP	0.025	c	0.2	CP	0.025	c	0.5	CP	0.018

Frame hinges ($M-\theta$ relation) in SAP are defined as in Figure 5. Hinge is not developed before yielding, so AB is a vertical line. The curve is normalized by yielding stress and yielding rotation, so point B is always (0, 1). In this study, stress of point C is 1.25 times of the yielding stress for all hinges, considering strain hardening and over strength. Ratio of remaining stress to yielding stress of point E is given in ASCE 41.

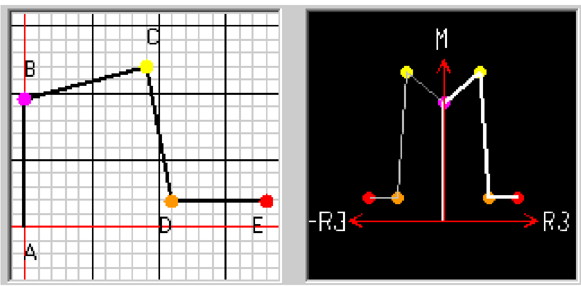


Figure 5 Moment-Rotation Relation of Plastic Hinge

Yielding moment M_y is obtained from frame member $P-M$ interaction analysis. Yielding rotation θ_y is taken to be equal to the story drift. It is calculated using Eqns 5-1 and 5-2 in ASCE 41, which are shown here as Eqns 3.1 and 3.2:

$$\theta_y = \frac{M_y l_b}{6E_b I_b} \tag{3.1}$$

$$\theta_y = \frac{M_y l_c}{6E_c I_c}(1 - \frac{P}{P_{ye}}) \tag{3.2}$$

where E is the elastic modulus, I is the section moment of inertia, and l is frame length. Subscripts *b* and *c* represent beam and column, respectively. P is the axial force, and P_{ye} is the expected yield axial strength.

In SAP, shell elements could not define plastic hinges, so the flexural failure of walls is not considered. This is rational because most walls in this building are shear controlled short walls. Wall with high h/l ratios will be replaced with frame elements to account for flexural failure. To ensure the walls to fail in the right patterns, stress-strain curves of concrete and masonry are defined by materials' shear capacities for shell components and by materials' compressive capacities for frame elements. For masonry frame components, masonry is converted to equivalent concrete by equivalent elastic modulus, as masonry does not have nonlinear definition option.

3.3 Pushover Load patterns:

Two pushover load patterns are compared: 1). story shear from the vertical distribution of static base shear; 2). story shear from the BSE-1 spectrum and scaled up to the static story shear. Load values are given in Table 3.2.

Table 3.2 Pushover loads

Level	Weight	Ex-static	Ey-static	Ex-dynamic	Ey-dynamic
ROOF	1778	1539	1435	1221	1185
7TH	1969	1452	1354	1006	933
6TH	1968	1222	1140	855	762
5H	1985	1002	934	709	614
4TH	2045	795	741	568	488
3RD	2137	583	543	443	394
2ND	2473	387	361	334	279
Σ	14354	6980	6507	5136	4655

Loading can be either force-controlled or displacement controlled. In this study, four load cases are defined as force-controlled based on static loads (EX-S, EY-S), four analysis cases are defined as force-controlled based on dynamic loads (EX-Df, EY-Df), and four analysis cases are defined as displacement-controlled based on dynamic loads (EX-Dd, EY-Dd). The target displacement δ_t for displacement-controlled loading is determined by Eqn 3.3 (§3.3.3.3.2 of ASCE 41):

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{3.3}$$

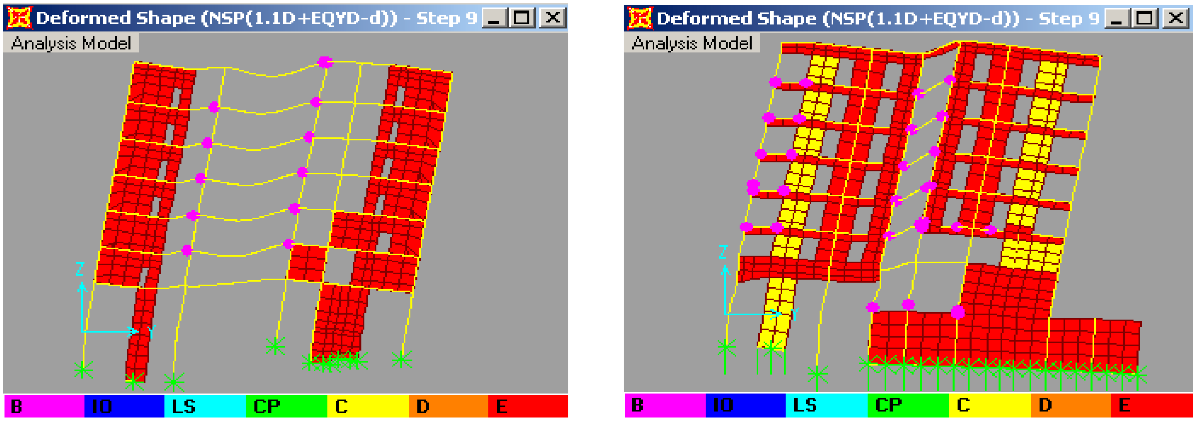
where C_0, C_1, C_2 are modification factors, S_a is the spectral response acceleration, T_e is the effective fundamental period, and g is the acceleration of gravity. For this building model, $\delta_{t,x}$ is 1.15 in. and $\delta_{t,y}$ is 1.45 in. Analysis shall be carried out to 150% target displacement to investigate the likely performance of the model under extreme conditions as suggested by ASCE 41.

In accordance with §3.2.8 of ASCE 41, actions due to gravity loads, Q_G, shall be considered as follows: $Q_G = 1.1(Q_D + Q_L)$ if gravity loads and earthquake loads are in the same direction; $Q_G = 0.9Q_D$ if gravity loads and earthquake loads are in the opposite direction; For all actions, design action (demand), Q_{UD}, shall be obtained from both gravity and earthquake actions per Eqn 3-18, §3.4.2.1.1: $Q_{UD} = Q_G + Q_E$. So in this study, 6 types of push loads are applied at the end of 2 types of gravity loads. As a result, 12 load cases are analyzed:

1.1D+EX-S, 1.1D+EX-Df, 1.1D+EX-Dd, 0.9D+EX-S, 0.9D+EX-Df, 0.9D+EX-Dd,
1.1D+EY-S, 1.1D+EY-Df, 1.1D+EY-Dd, 0.9D+EY-S, 0.9D+EY-Df, 0.9D+EY-Dd.

4. PUSHOVER ANALYSIS RESULTS AND EVALUATION

Analysis shows that most masonry coupling beams of exterior masonry wall piers have yielded and the hinge status is between points B and C (Figure 4(b)). Though many hinges' status is in LS performance level, some of them go beyond CP stage. This fact indicate that masonry coupling beams are overall too weak to effectively transfer seismic loads between shear walls and may fail and endanger lives. They must be upgraded. Figure 6 shows the building drifts and hinges of two elevations at step 9 of load case 1.1D+EY-Dd.



(a) Line 7 Elevation (N-S) (b) East Elevation

Figure 6 Drifts and Hinges to Load Case 1.1D+EY-Dd

The second finding of this analysis is that, gravity load resisting systems (the reinforced concrete frames) are not capable of accommodating the imposed seismic drifts. Some columns yielded in early steps and calculation does not converge for displacement-controlled load cases when P-Delta effect is considered. If P-Delta effect is not considered, cases based on larger gravity loads (1.1(D+L)) do not converge in the last steps and could not reach $1.5\delta_t$ in y-direction (N-S), which means, the gravity systems might have lost stability at large drift.

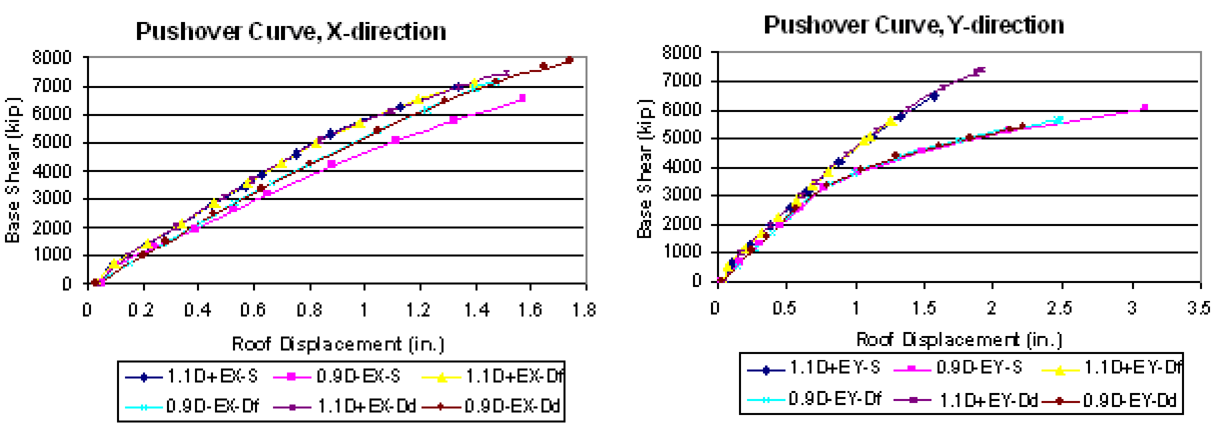


Figure 7 Pushover Curves

Figure 7 show the pushover curves without considering P-Delta effects. Case 1.1D+EX-Dd pushes up to 1.52 in. and case 1.1D+EY-Dd pushes up to 1.95 in. Though both displacements exceed the design value δ_t, drift and interstory drifts could possibly exceed LS performance requirements. Design drift ratios for IO, LS, and CP performance levels could be found in original CBC'07 (the section is removed in later revisement.).

Also observed is that the static load pattern pushes the model farther than the dynamic load pattern without losing stability. This fact indicates that dynamic load pattern matches the actual mode shape more than the static load pattern and is thus more unfavorable to the model. The last step building profile to load cases based on 0.9D confirms the conclusion (Figure 8).

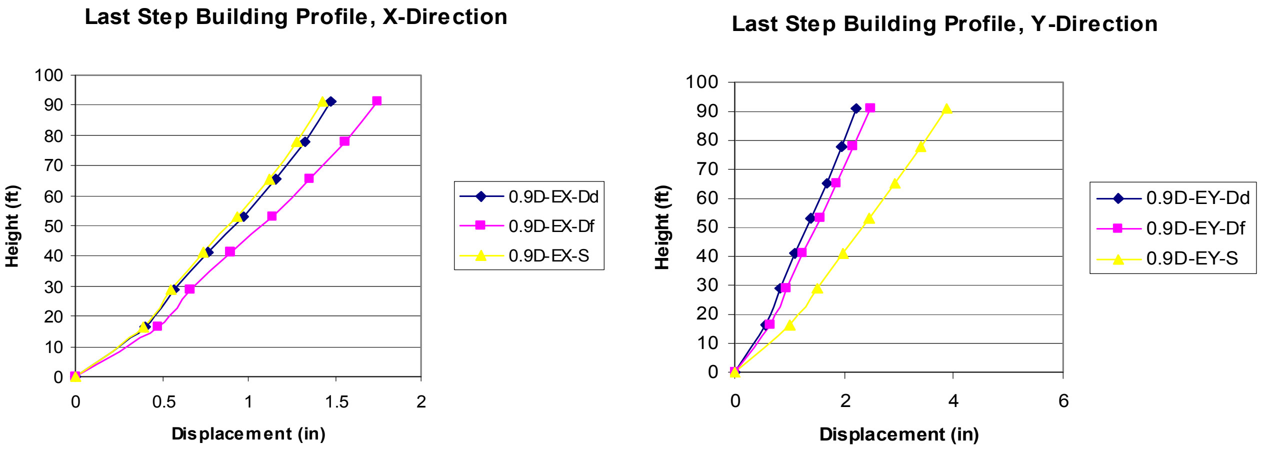


Figure 8 Building Profiles

To investigate the hinge status of frame members, column 3773 (20x20 with 8#11 reinforcement) and beam 3341(24x16.5 with 8#8 reinforcement) are selected to show the formation and development of plastic hinges. Their moment-rotation relations are shown in Figure 9.

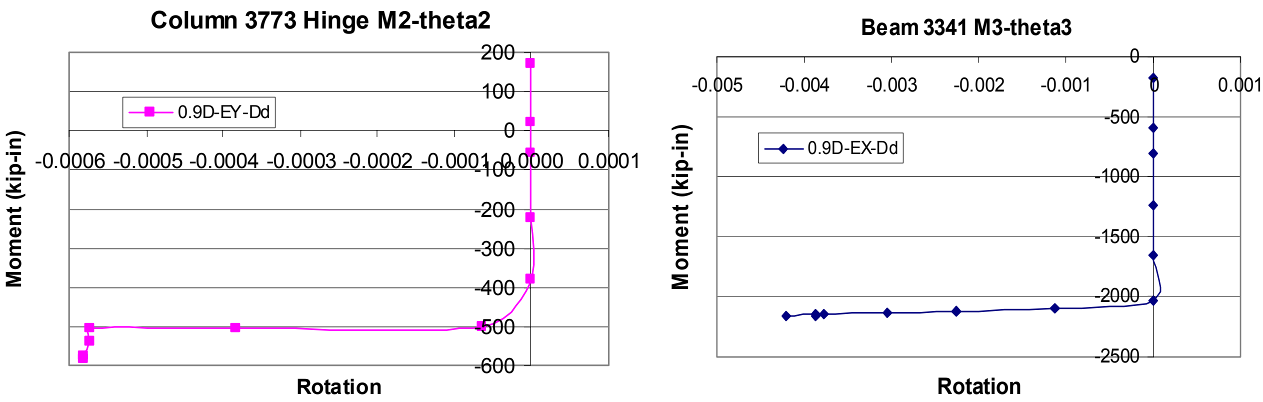


Figure 9 Development of Column and Beam Plastic Hinges

Column 3773 is of concern because of its relatively small size (many other columns in low stories are 24x24), large axial force (between 1^{st} and 3^{rd} floors) and is supporting a discontinuous shear wall (Figure 6B). Figure 9 shows that it yields at low moment (400 kip-in) and loses capacity at 7^{th} step. From pushover curve for this load case (Figure 7), we can see that the target displacement (1.45 in.) is reached just a little before step 7, and hinges about both axes are between points C and D (Figure 5) and are in the ">CP" status from step 6. Similarly, there are two more columns in the interior elevations have similar situation. These columns must be strengthened.

Beam 3341 is in an interior elevation that is support a discontinuous wall. Although it does not lose all capacity, the hinge formed in early steps and the state shows that it has been in LS states early. Measures need to take to strengthen such beams, too.

Primary components demands shall be within the acceptance criteria at the selected structural performance level. The nominal shear strength of shear walls shall be computed based on the provisions of ACI 318-99. Lower bound yield strength shall be used for the calculations. The strength reduction factor shall be 1.0. Shear forces of two wall piers are taken from section cut outputs to show the wall's shear demands: 2^{nd} story, #5 piers of north wall (2A-J-w5) and south wall (2A-U-w5). Figure 10 (a) shows that both walls are yielded in shear. Their calculated demand/capacity ratios are 3.2 and 2.4, respectively, indicating that the retrofit measure to these walls is not sufficient.

Figure 10(b) gives the soil spring forces of joint U9, which is located at the bottom of the south-west corner of the building. It illustrates the spring force changing during pushing: in the first 6 steps, the spring is under compression; from the 7^{th} step, the joint is lifted and spring force drops to zero (soil cannot be tensioned. See Figure 4(a).).

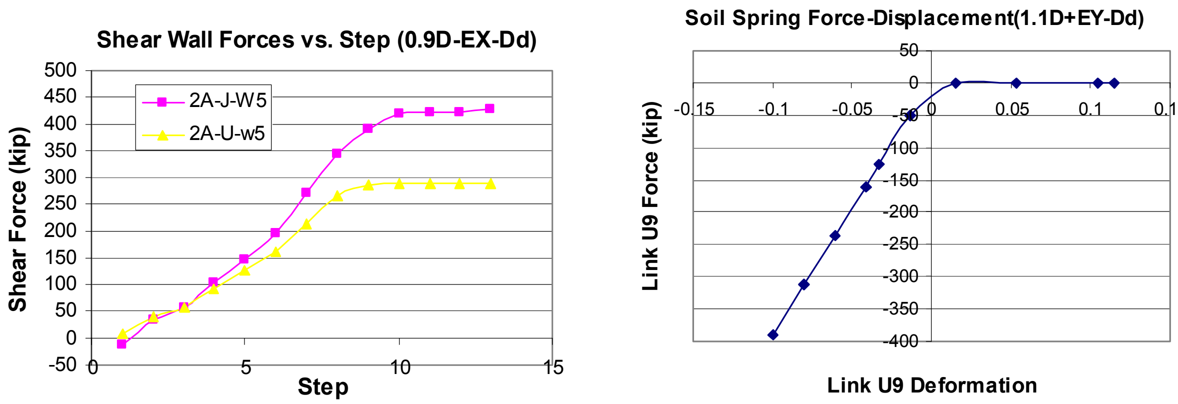


Figure 10 (a) Shear Wall Forces vs. Step (b) Soil Spring Force-Displacement

CONCLUSIONS

Some conclusion are made based on the above analyses: 1). Exterior masonry walls and masonry coupling beams need stronger retrofit measures; 2). N-S direction gravity load resisting system needs to be strengthened, especially the columns supporting discontinuous shear walls; 3). Dynamic load pattern is more appropriate for this nonlinear analysis than the static load pattern.

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