Seismic Performance Evaluation of an Existing Precast Concrete Shear Wall Building

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SUMMARY:
Nonlinear analysis has become a popular approach for the assessment of new and existing buildings. Proper assessment of a building depends on several factors that include modeling, material properties, degradation models and applied loads. This study presents results from a comprehensive evaluation of an existing hospital in California after an earthquake occurred at the building location. This study is composed of three main components: obtaining a recorded ground acceleration at the building site, conducting a nonlinear analysis of the structure and an onsite damage evaluation of the building after an earthquake occurred near the building site. Results from this study include nonlinear static and nonlinear dynamic results for the earthquake that occurred at the building location. These results are compared with the observed level of damage in the building. The nonlinear analysis results from the existing building demonstrate a good correlation between the nonlinear analysis and the observed damage in the building.

Keywords: Seismic evaluation, nonlinear analysis, precast concrete shear wall

1. INTRODUCTION

In response to the 1994 Northridge Earthquake, the State of California enacted the Senate Bill 1953 (SB 1953). SB 1953 served to establish the Hospital Seismic Retrofit Program, which aims to prevent hospital collapse and ensuing loss of life, as well as continuing operation of acute care facilities during and following earthquakes (Huang, Wei et al, 2008). Under the SB 1953 Structural Performance Categories (SPC-1 thru 5) were introduced for building seismic classification.

The hospital building presented in this study was classified per the SB 1953 with a Structural Performance Category 1. Senate Bill 1953 requires hospitals rated SPC-1 (those that are considered hazardous and at risk of collapse or significant loss of life in the event of an earthquake) to be replaced or retrofitted to higher seismic safety standards by 2013, or later with an approved extension. Otherwise, acute care services may no longer be provided in such buildings.

2. BUILDING DESCRIPTION

The building hospital is located in the northern part of State of California, USA. The building was constructed in 1953 and designed according to the Uniform Building Code of 1949 (UBC 1949). It has 4 stories above grade with a partial roof top mechanical room and a partial single level below grade basement. The building has a “T” shaped footprint measuring approximately 280 feet by 145 feet. Figure 1 shows the first floor plan of the hospital building. The typical floor size is 14,500 square feet and the typical floor to floor height is 12’-6”. The construction of the building consists of cast in place concrete spread foundations, columns, slabs and precast concrete wall panels.. The east wall and south wall elevation are shown in Figure 2 and Figure 3.

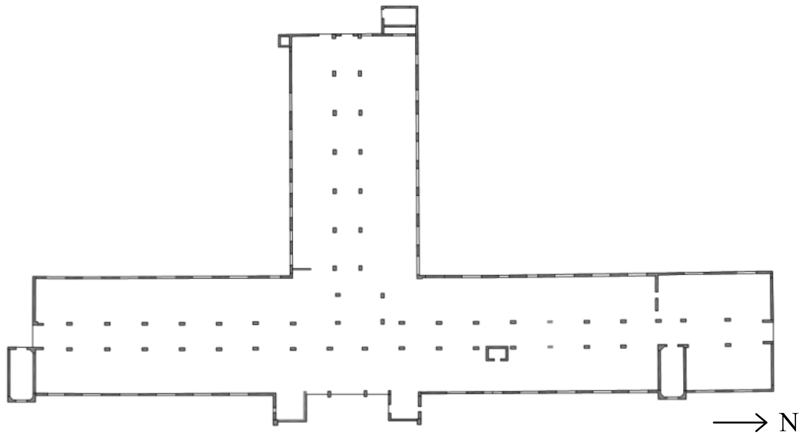


Figure 1. First level plan view

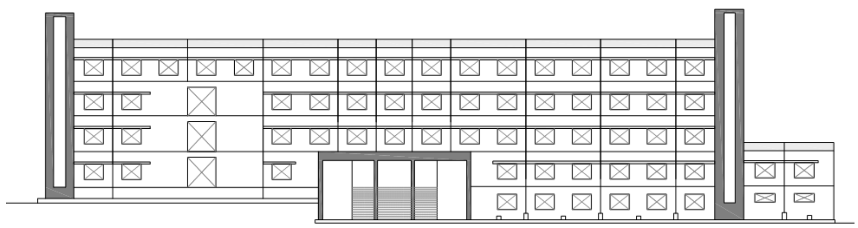


Figure 2. East wall elevation view

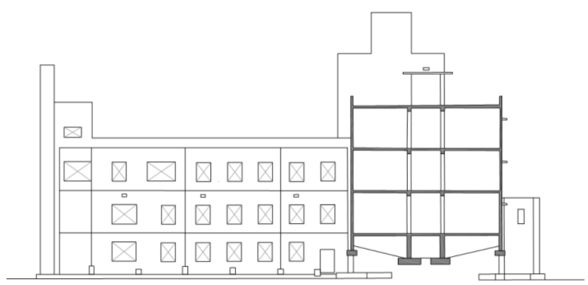


Figure 3. South wall elevation view

The lateral force resisting system of the building is composed of a tilt-up precast concrete shear wall panel system connected with welded steel embedded plate connectors. Figure 2 and Figure 3 presents the east and south elevations of the building, where the lateral force resisting system can be observed. These panels have panel to panel and panel to floor slab connections using welded splice plate connections. Three different types of steel splice plate connections were identified in the building as potential weak links, named as G2, H2, and B2a. Figure 4 shows the configuration of a typical precast concrete panel with the location of the steel connections. Connection G2 is a precast panel to precast panel connection between the top and bottom panels with a total of 8 connections of this type per panel. Connection B2a is a precast panel to precast panel connection between the left and right sides of the panels; each panel has a total of 6 connections of this type. Connection H2 connects the precast panel to the floor slab and each panel has 2 connections of this type. The precast concrete shear wall configuration extends over the periphery of the building, defining the lateral system of the building.

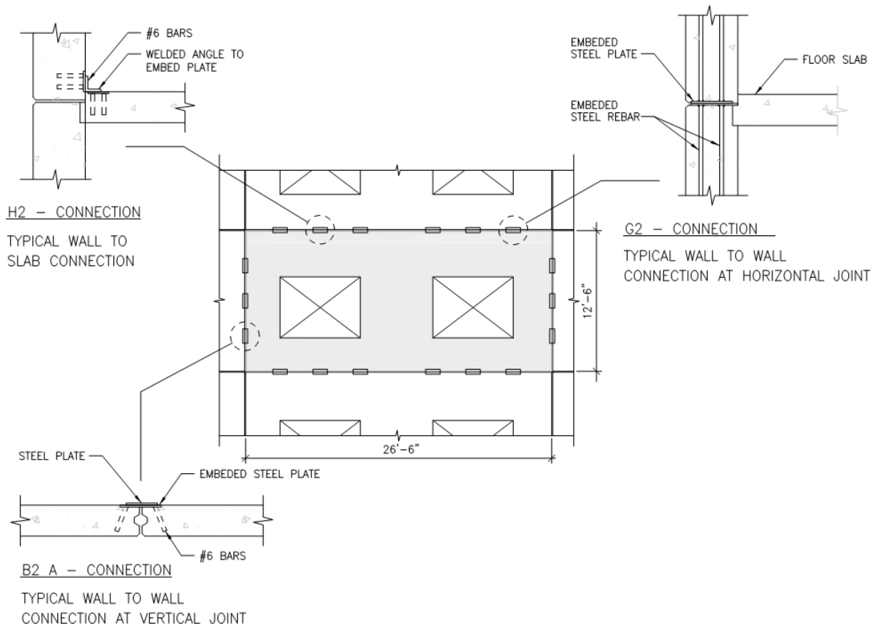


Figure 4. Typical precast concrete panel and steel connections configuration

In 1995 the hospital was expanded and an additional three story seismically separated building was constructed next to the east wall. As part of this expansion the precast concrete panels at the South of the east wall were modified to create an opening to connect the two structures. Modifications to the existing building included local thickening of the concrete wall thickness from 8" to 18", additional reinforcement, boundary elements and concrete infill for some of the window openings. The modified panels are those with the large openings at the South side of the east wall elevation shown in Fig. 2.

3. BUILDING EVALUATION

In 2000, the hospital building in this study was evaluated per Senate Bill 1953 guidelines and was classified with a Structural Performance Category 1 (SB 1953 Evaluation, 2000). This classification categorizes the building as a significant risk of collapse and danger to the public. The main observations per the SB 1953 evaluation were as follows:

- The precast concrete shear wall panels had inadequate strength and absence of reinforcing details that provide for ductile behavior
- The apparent brittle nature of welded panel-to-panel steel connections.
- Building diaphragms lack of additional reinforcing at the re-entrant corners and insufficient diaphragm chord reinforcing.
- Geometric irregularities in the building due to the setback of the west wall at the third level

In early 2010 a 6.5 magnitude earthquake occurred at 30 miles from the building location. A post earthquake visual assessment of the building was conducted to indentify if any damage occurred to the hospital building. No major damage was found, few seismic joints were damaged and some minor cracks in window corners were observed.

After the earthquake, the hospital management decided to perform a more detailed study to evaluate the building integrity. As an initial phase of the building evaluation, a linear model was developed in SAP2000 and analyzed, using a response spectrum analysis. Results from the response spectrum analysis concluded that a large number of the steel connections between the precast concrete panels could have been overstressed during the earthquake. However these results contradicted the apparent building condition where rather limited damage was observed. The fixed base of the linear elastic model, which prevented any rocking at the base, and the lack of redistribution of the internal forces once connections reached their capacity, were thought to be some of the reasons for the disagreement. A nonlinear based analysis was therefore implemented to overcome these modeling deficiencies. Details and results of the non-linear based evaluation are presented herein.

3.1 Nonlinear Model

A nonlinear model was developed based on the original structural drawings from 1952 and existing material testing documentation of the building. The computer software Perform 3D (CSI, 2007) was utilized to develop a three dimensional model as shown in Figure 5. The three dimensional nonlinear model incorporated all the primary lateral force elements and specific gravity members connected to the lateral system. The primary lateral force system that compose the model are the precast concrete panels and the steel connection joints as presented in Figure 4. A rigid diaphragm was utilized due to the size of the nonlinear model and software memory limitations.

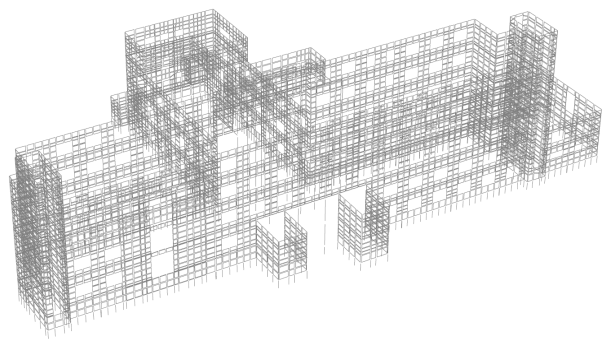


Figure 5. Three dimensional model in Perform 3D

Material and element nonlinearity were accounted in all the primary lateral force elements in the model. Nonlinear material concrete properties were calculated based on the compressive strength of the concrete. A specified concrete compressive strength of 2500 psi was used in the model according to the structural drawings and material testing evaluations. The elastic and post-elastic concrete material behavior was determined using the Trilinear Kent Park Model as shown in Figure 5. For the steel material, a bilinear force deformation behavior was utilized with corresponding steel rebar yield strength of 40 ksi per the material testing evaluation. Expected concrete compressive strength and steel yield strength were determined based on the ASCE 41-06. The precast shear wall strength and the load deformation behavior was determined as shown in Figure 6 per the ASCE 41-06.

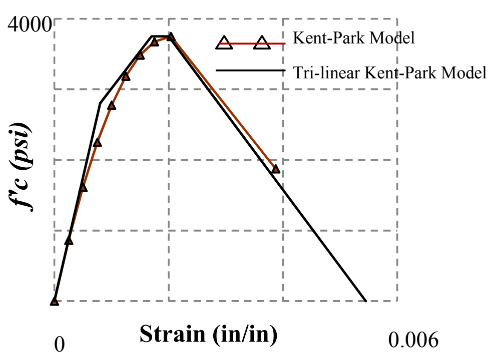


Figure 5. Kent Park Model used for the concrete material

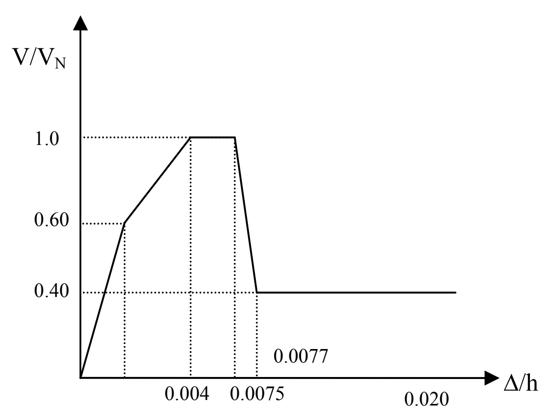


Figure 6. Shear Load-Deformation Relation for the precast concrete walls

The steel joints between the precast concrete panels were modeled as shear and axial elements with their associated strength capacity and deformation limit. The precast panel to panel connectors were modeled using two elastic beam elements, an axial hinge and a shear hinge as shown in Figure 7.

Two different types of steel joints connections named as G2, and B2a were modeled. Connection H2 connecting the precast concrete panels with the floor diaphragms was evaluated independently since a rigid diaphragm was utilized in the model.

The friction force between the precast concrete panels was accounted for in the nonlinear model and incorporated in the connection G2 which is a connection in the horizontal plane between top and bottom panels. The friction force was modeled as an additional shear strength force in the steel joints as shown in Figure 8. By incorporating the shear friction in the steel joint the initial shear resistance in the joint is provided by the friction resistance ($V_{friction}$) of concrete panel against concrete panel until the axial force in the joint is less that the precast panel weight (W). Consequently, once the axial force in the connection goes into tension, the shear resistance is reduced to the value of the steel joint capacity (V_{conn}) only. Connection B2a was modeled without considering the additional shear friction resistance since the connection is located in the vertical plane.

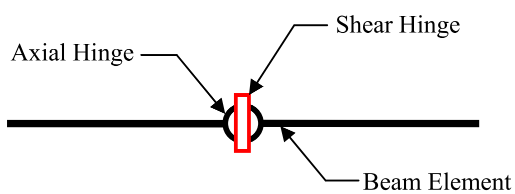


Figure 7. Steel joint compound element

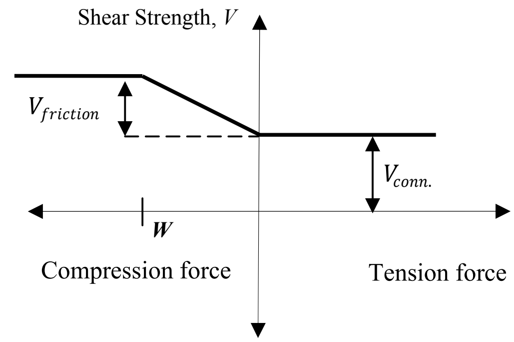


Figure 8. Friction force model for conn. G2

Building soil structure interaction was accounted for in the nonlinear model. For this purpose a soil spring element with a maximum compression capacity and zero tension capacity was utilized. Figure 9 presents the soil spring element composed of two elastic beam elements and an axial hinge with a maximum compression strength.

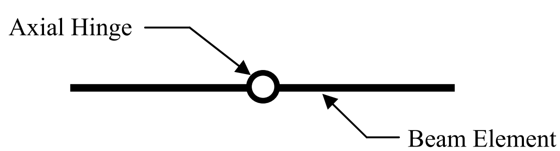


Figure 9. Foundation spring element

3.2. Analysis and Results

Two types of nonlinear analyses were performed in the hospital building: a nonlinear pushover analysis and a nonlinear time history analysis. For the nonlinear pushover analysis the procedure described in ASCE 41-06 was conducted to determine the forces and deformations in the building. Four demand levels were established to determine the performance level of the building in different earthquake scenarios. The demand levels were selected based on the peak ground acceleration for 0.30g, 0.50g, 0.85g and 1.28g. Where the peak ground acceleration is taken at a period T=0 seconds. The 0.85g and 1.28g correspond to the design base earthquake and the maximum considered earthquake. Figure 10 shows the response spectra for each of the demand levels were the building was evaluated. In addition, the response spectra for the two orthogonal earthquake components are included.

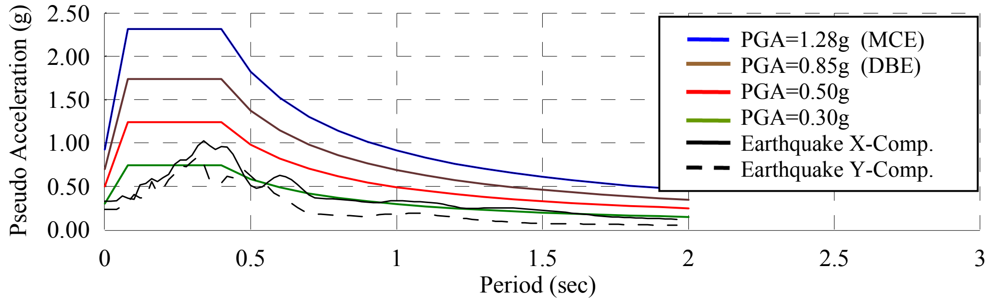


Figure 10. Response spectra and earthquake ground motion components (5% Damping)

Results from the pushover curves for each of the building orthogonal direction are presented in Figure 11. As expected a higher force capacity exists in the building along the North-South direction than in the East-West direction due to the building length along the North-South direction is longer and consequently stiffer and stronger.

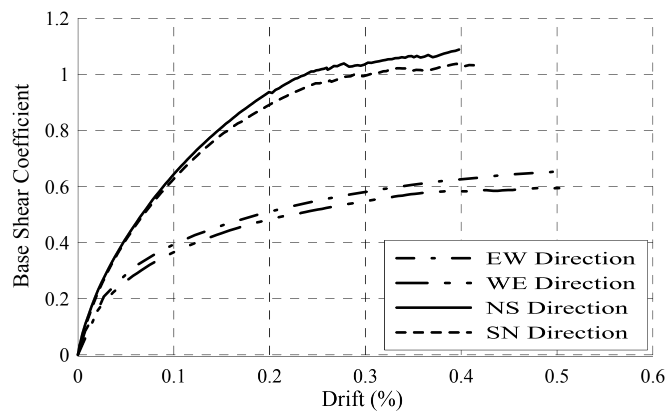


Figure 11. Capacity curves for each orthogonal direction of the building

Once the capacity curves were obtained, the building behavior was evaluated under each of the demand levels previously discussed. Figure 12 is a graphical representation of each of the precast panel steel connection usage ratios in the east walls that reached 0.7 to 1.0 usage ratio for each demand level. A legend that relates each usage ratio with a symbol is included to describe the level of damage in the steel connection.

For the nonlinear time history analysis the earthquake ground motion was utilized. The two orthogonal earthquake components were recorded at an annexed building that is equipped with seismic instrumentation. The ground motion had a total duration of 90 seconds with a maximum ground acceleration of 0.33 (g) and 0.23 (g) for the north-south and east-west components respectively.

The results from the nonlinear pushover and time history analysis are as follows:

A. Pushover Analysis Results:

1. Low damage to the building was observed for a seismic demand corresponding to a PGA of 0.3g and 0.4g as shown in Figure 12a and Figure 12b.

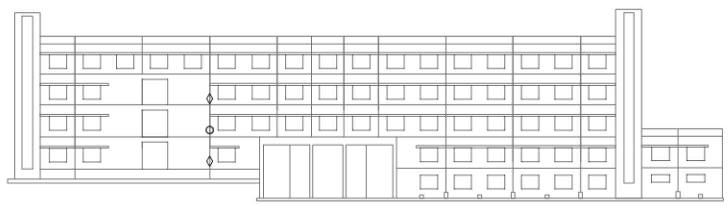
2. For the design based earthquake (DBE) seismic demand level specific areas are susceptible to suffer damage in the structure as presented in Figure 12c.

3. For the maximum considered earthquake the building is expected to suffer considerable damage and could potentially collapse due to the number of steel connections that reached a usage ratio (demand versus capacity) of 1.0 as shown in Figure 12d.

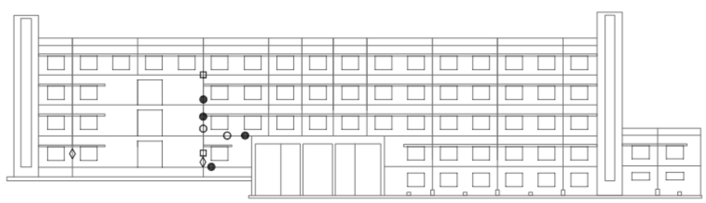
B. Time History Analysis Results:

4. Based on the Perform 3D Model minor damage due to the 2010 earthquake was predicted in the building. Figure 13 shows only one connection that could reach a usage ratio of 0.9 and a second connection with a 0.7 usage ratio.

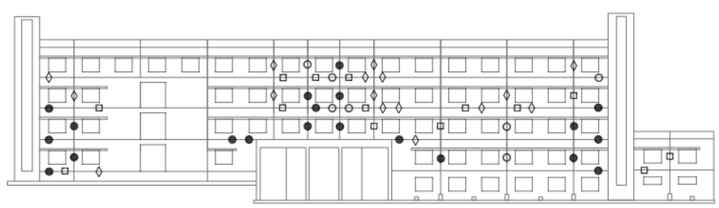
5. Good correlation between the nonlinear pushover analysis and the nonlinear time history analysis was observed. Pushover analysis results and time history analysis results agree on the locations that the building is susceptible to suffer damage as shown on Figure 12a and Figure 13.



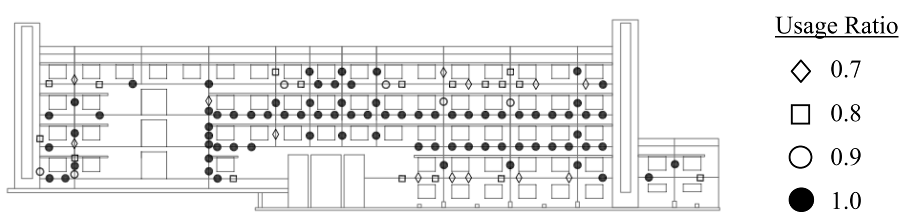
a. PGA=0.30g



b. PGA=0.50g



c. PGA=0.85g (DBE)



d. PGA=1.28g (MCE)

Figure 12. Steel joint connection usage ratio from pushover analysis results in the east elevation

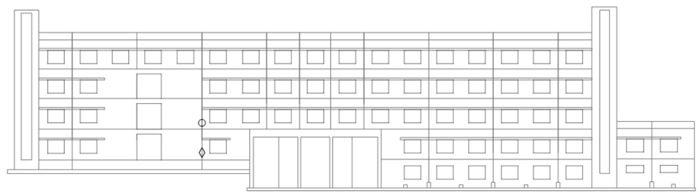


Figure 13. Steel joint connection usage ratio from nonlinear time history results in the east elevation

Table 1 presents the summary of the total number of steel connections that would be damaged or would fail in the building due to the different demand levels from the pushover analysis and the time history analysis. A connection was considered to be damaged if its demand versus capacity reached to a 90% of its strength capacity and fail if the connection reached 100% of its capacity.

Table 1. Number of steel connections with damage or failed

Load Case	Demand Level	Damage D/C=0.9	Fail D/C=1.0
Pushover Analysis	PGA=0.30 g	1	0
	PGA=0.50 g	2	4
	PGA=0.85 g (DBE)	13	33
	PGA=1.28 g (MCE)	16	172
Time history	PGA=0.30 g	1	0

3.2.1 Comparison with previous analyses:

The results of this study were compared to the studies done during the first SB1953 evaluations which estimated the capacity of the lateral force resisting system. This analysis corresponds well with the previous analyses in the locations where the steel joints reach their capacity. This study identifies additional locations where the building is susceptible to suffer damage for different earthquake demand levels. In addition, this study concludes that due to the earthquake in 2010, the building experienced minimum structural damage and coincides with the post earthquake visual observation as discussed in Section 3.

CONCLUSIONS

This paper discussed the results of a nonlinear static pushover analysis and a nonlinear time history analysis implemented for an existing hospital building in California. Results from the pushover analyses and the time history analyses were found to be comparable, with both analyses showing good correlation with the level of damage observed in the building after the 2010 earthquake. It was found that for a design basis earthquake (DBE) the hospital building may suffer significant damage while for a maximum considered earthquake the building could potentially collapse.

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