Comprehensive Evaluation and Seismic Retrofit of a Three-Story Non-Ductile Concrete Structure

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ABSTRACT

Performance-based earthquake engineering was utilized to ascertain the seismic performance of an existing two-story non-ductile reinforced concrete building with a proposed new third floor steel-framed addition. The structure, hereafter referred to as the Capital Unity Center, is located in downtown Sacramento, California, and was designed and constructed in early 1940s per the applicable building codes at the time of construction. Capitol Unity Center is a remodel and addition to an existing warehouse structure and is an integral part of the city's attempt to renovate the older downtown buildings. The new center will house a museum with space for exhibits, performing area, and workshops. Since the renovation constitutes major structural changes, the structure in the new configuration must comply with the current seismic provisions of the California Building Code [3]. In addition, since the facility and will be used by the California Department of Education, it must comply with the more stringent requirements of the California Division of State Architecture (DSA). Provisions of FEMA 356 [1], ACI 318 [2] and CBC 2001 [3] were used for a detailed investigation. Comprehensive material testing, detailed geotechnical investigations, site-specific seismic hazards, and state-of-the-art nonlinear structural analysis were performed to assess the seismic performance of the building, identify deficiencies, and evaluate efficacy of seismic retrofit techniques. Analysis showed that the existing structural elements with the architectural renovations did not meet the required performance goals. To obtain satisfactory behavior, both the superstructure and substructure have to be retrofitted. In its retrofitted configuration, the building would then meet the performance objectives.

INTRODUCTION

The scope of the evaluation reported herein is limited on ascertaining the seismic performance of the existing components and the effectiveness of the proposed retrofit methodology. Since significant changes to the lateral-load resisting system of the building are proposed, an analytical investigation of the renovated structure was performed. Such evaluation is required per CBC 2001 [3].

EXISTING STRUCTURE

The structure was constructed in the 1940s with two stories, each having a 12.5-ft story height, and measures 136 feet in the North South (y-) and 61 feet in the East West (x-) directions, respectively. The existing structure is composed of cast-in-place concrete elements with an 8.5-inch thick two-way cast-in-place concrete slab at each floor, reinforced by No. 5 bars. The slabs are supported by a system of interior circular columns and perimeter structural walls and square pilasters. The slab support on all columns include a 3 ft diameter column capitals and 7 ft square drop panels.

For the second story, the perimeter walls are 8 in. thick, pilasters are 18 in. square, and interior columns are 18 in. in diameter. For the first floor, the short walls (x-direction) are 12 in. thick, long walls (y-direction) are 8 in. thick, pilasters measure 22 in. square, and interior columns have a diameter of 22 in. The square pilasters are reinforced by four 1.25 in. square reinforcement and No. 3 ties at 12 in. on center. The interior columns are reinforced by ten No. 7 longitudinal bars and No. 5/16 spirals at 3 in. pitch. The slab on grade is 4.5 in. thick.

Columns are supported on 16 ft deep cylindrical shafts. The shafts have diamter of 28 to 32 in. and terminate in belled caissons measuring 4.5 ft deep and a have bearing diameter of 64 to 68 in. at the base. Figure 1 presents a photograph of the building and Figure 2 depicts the typical floor plan [2]. The contract plans [2] show that the building was designed to incorporate a third floor to be constructed later.

FIGURE 1
PHOTOGRAPH OF THE BUILDING

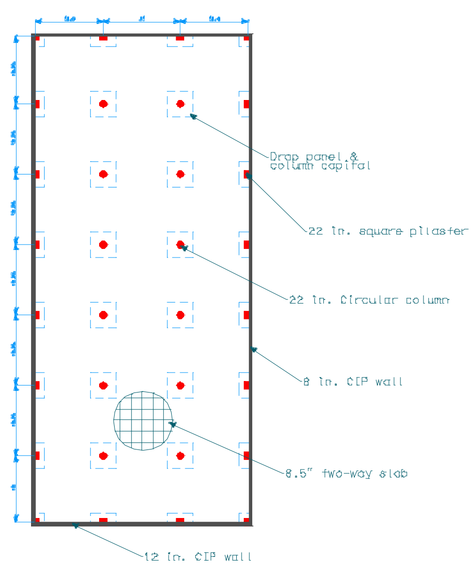

FIGURE 2
FLOOR FRAMING FOR THE STRUCTURE

PROPOSED BUILDING RENOVATION

It is proposed to add a steel-framed third floor to the building and to remove a number of the existing walls as part of the renovation. The proposed renovations to the building consist of: 1) add ordinary steel moment frames (OMRF) to support the new roof of the building above the third floor, 2) remove a number of perimeter concrete walls to provide additional openings, 3) enlarge the footprint of the building and add additional concrete structural walls, and 4) remove a 20 ft by 25 ft segment of existing slabs. Figure 3 presents schematics of the building after proposed renovations. The three-dimensional mathematical model of the building used in analysis and evaluation is shown in Figure 4.

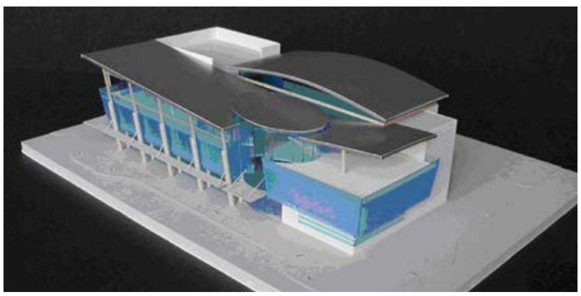

FIGURE 3
RENDERING OF THE REHABILITATED STRUCTURE

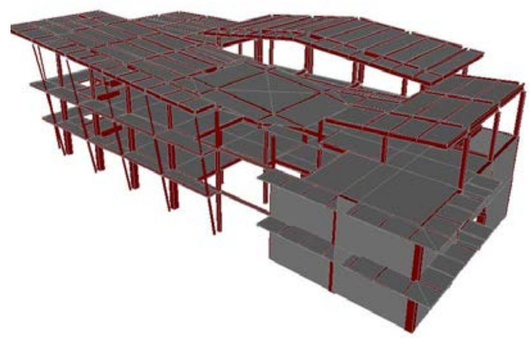

FIGURE 4
THREE-DIMENSIONAL MODEL OF THE BUILDING

PERFORMANCE OBJECTIVES

The provisions of FEMA 356 [1] were used to model the building and assess its seismic performance. Two seismic levels are defined: The 475-year event as the Design Basis Earthquake (DBE) and the 2,500-year event as the Maximum Credible Event (MCE). Three performance levels are considered: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). The reader is referred to Chapter 6 of FEMA 356 [1] for definition and pertinent factors for these performance levels as relating to reinforced concrete members. Nonlinear static (pushover) and nonlinear dynamic (response history) analyses were used in evaluation.

CBC 2001 [3] Method B evaluation procedure was utilized. This implies defining scope of rehabilitation, selecting an appropriate evaluation methodology, comprehensive material testing, and employing peer review. For DSA projects, CBC 2001 [3] requires meeting two performance targets: LS-0.33 (LS-IO) for the DBE level, and CP for MCE level. Since for this site, the seismic intensity at MCE level is approximately 1.5 times that of DBE event, and since the performance level at the MCE level is less than 1.5 times that of DBE performance event, if the structure meets the DBE requirements, then it would also meet those of the MCE level. As such, the remainder of this paper would only address the analysis and evolution result at the DBE level.

Geotechnical Investigation

Overview

An extensive and comprehensive geotechnical investigation of the site was conducted [8 and 9]. The objectives of the investigations were to: 1)assess the susceptibility of the site to liquefaction and differential settlement, 2) perform soil-pile interaction analysis to provide soil spring models and compute the vertical (bearing and uplift) and lateral capacity of shafts, and 3) prepare site-specific acceleration response spectrum and three sets of spectrum-matched acceleration histories for structural analysis. The investigation ruled out the liquefaction potential. The remaining findings are presented below.

Soil-structure interaction

Soil-pile interaction analysis of the existing foundation was carried out. The evaluation was based on the soil data obtained from borings and shaft data depending on the shaft measured material properties. The detrimental effect from lack of confinement and limited lap splice of longitudinal reinforcement were included in the model. Since the reinforcement terminated above the caisson bell-to-shaft intersection, the uplift capacity only depended on the skin friction at the perimeter of the shaft. Figure 5 presents the the idealized curve for the vertical response of the shafts for both uplift (tension) and bearing (compression). Lateral shaft capacity was computed and is shown in Figure 6. The solid line corresponds to the free head case, whereas, the dashed line is used when the top of shaft is assumed as fixed, a condition presented only when walls above grade provide rigidity in the direction perpendicular to shaft axis under consideration. The allowable soil bearing pressure at the base of bell caissons was estimated at 7 ksf for gravity loading effects. For the grade beams, a passive lateral pressure of 15 k/in per ft of the grade beam was estimated.

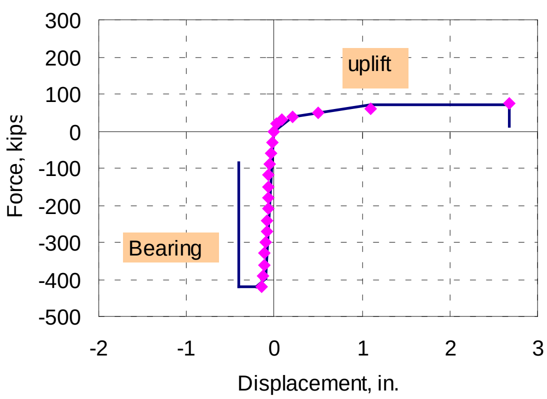

FIGURE 5
SHAFT AXIAL SOIL-SPRING RELATION

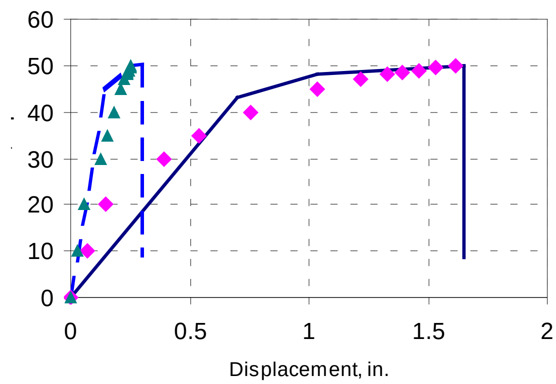

FIGURE 6
SHAFT LATERAL SOIL-SPRING RELATION

Site seismic hazard

Geotechnical investigations [9] were conducted to assess the seismic hazard at the site. The DBE and MCE spectra are presented in Figure 7. FEMA 356 [1] requires that at low periods the acceleration ordinates equal to that of the maximum (short period) acceleration. As such, the modified accelerations were used in evaluation. Three sets of spectrum-compatible motions were developed for analysis. Figure 8 presents the trace of one of these records.

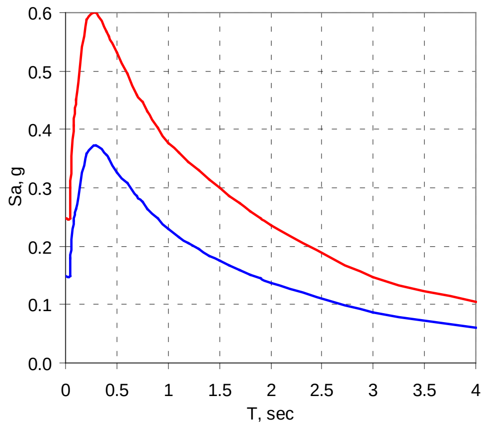

FIGURE 7
SITE RESPONSE SPECTRA (5% DAMPED)

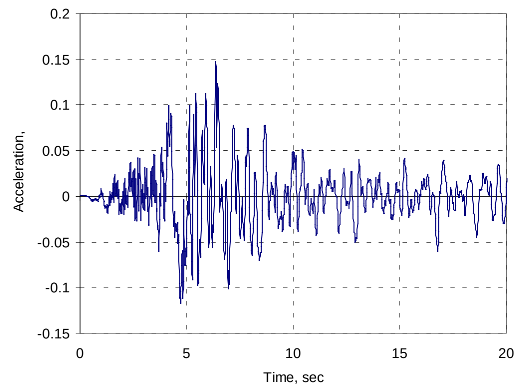

FIGURE 8
SPECTRUM-MATCHED ACCELERATION RECORD

COMPREHENSIVE MATERIAL TESTING

Per FEMA 356 [1] requirements, a minimum of three samples of concrete cores and reinforcing steel were extracted from the various structural components and tested. Table 1 presents the results of the material tests [6]. The values shown in Table 1 correspond well with FEMA 356 [1] recommended values for this period of construction. In addition to destructive tests, several site visits and condition assessments investigations were conducted. Concrete members appeared intact with minimal cracking or evidence of reinforcement corrosions. The in-situ dimensions, member sizes, reinforcement sizes, and concrete cover measured in the field closely correlated with the values shown in the contract documents [4].

Component		f'c, ksi	fy, ksi
Slab	Upper floor	2.9	42.5
	On Grade	6.2	
Walls*	Upper		
	Lower	3.3	51
Columns#		3.0	53
Shafts		6.0	51

* Pilasters were poured monolithically with walls
\# Circular inter columns

TABLE 1
MATERIAL PROPERTIES OF THE EXISTING BUILDING ELEMENTS

SEISMIC EVALUATION OF THE EXISTING STRUCTURE

Analytical model of the building

Program SAP [5] was used to prepare mathematical models for the building. Figure 4 presents an isometric view of the model. Member centerline dimensions were used in analysis. The geometry of the building was obtained from the existing contract documents. Nominal member sizes and material properties as presented in Table 1 were used. The proposed values for live load and additional non-structural dead load and seismic mass were applied to the building. This included the large 125 or 100 psf non-reducible live load for the storage and assembly areas, respectively. The upper floor roof was modeled as a series of slanted segments that matched the architectural drawings. After removal of slab segments would increase its nominal aspect ratio to approximately three. Per FEMA 356 [1] recommendations, in separate analyses, the slabs were modeled as rigid and flexible. The most severe case was selected.

Accidental torsion was included in analysis and this effect was amplified per requirements of FEMA 356 [1] when necessary. P-Δ was included in the analysis. No rigid offsets was modeled for members. To account for cracking, the flexural stiffness of walls and columns were reduced to 50% and 75% of gross properties, respectively [1, 3]. For the concrete structural walls, the out-of-plane flexural properties were conservatively ignored. The seismic mass of the building incorporated selfweight, additional dead load, and 25% of un-reduced live load of 125 psf.

As part of reconfiguration, the existing window openings will be shotcreted to match the thickness of the existing walls. As such, these elements were modeled as solid. All the proposed new openings are included in the model. When a section of existing wall is removed for an opening, a four-ft deep segment will be left in place. These remaining portions were modeled as concrete header beams. Soil structure interaction was included in the models by placing nodal nonlinear three-dimensional springs at the base of the shafts and linear distributed springs along the grade beams. Nonlinearity was modeled via user-defied plastic hinges in pushover analysis or using nonlinear elements for response history analysis.

Structural shortcomings

Figures 9 and 10 present the results of the pushover analysis of the existing building. The demand spectrum was obtained by selecting acceleration parameters, C_a and C_v, in order to match the standard CBC 2001 spectrum [3] to the site specific acceleration spectrum of Figure 7. Note that the building nonlinear behavior is concentrated at the soil springs and that the hinge demands exceed the allowable value for the performance level.

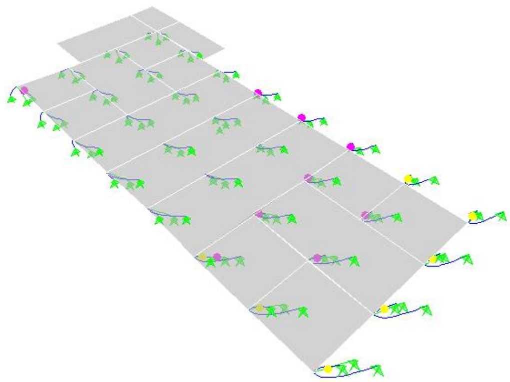

FIGURE 9
PLASTIC HINGE PATTERN, X-DIRECTION

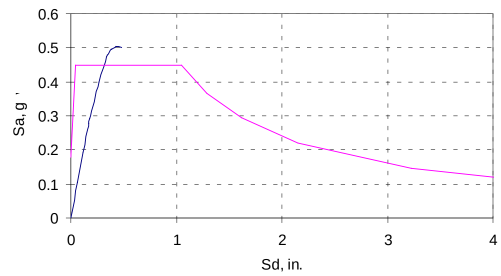

FIGURE 10
NONLINEAR FORCE-DISPLACEMENT CURVE

The results of analytical investigations and review of contract plans showed that the building in its current configuration possessed major seismic deficiencies. These deficiencies are listed below.

- Inadequate capacity of concrete walls: These walls have an aspect (height to length) ratio of less than 1.5 and are classified as short or squat per FEMA 356 [1]. Their behavior is governed by shear and hence possess limited ductility. In addition, reinforcement at the upper floor is spaced at 22 in., which is larger than the 18 in. upper limit of FEMA 356 [1], CBC 2001 [3], or ACI 318 [2]. The vertical and horizontal reinforcement ratios equal the minimum allowable reinforcement ratio of 0.25%.
- Inadequate anchorage of slab reinforcement into the existing walls to transfer shear.
- Inadequate splice of longitudinal reinforcement of the pilasters and large tie spacing.
- Inadequate lateral capacity of the shafts and grade beams.
- Inadequate vertical load capacity of the slabs to resist the proposed large vertical loading. Due to presence of drop panels, punching shear capacity of the slab is adequate. However, the slabs do not have adequate flexural capacity once the proposed section is removed.
- Inadequate means of transfer of slab lateral forces around the proposed opening.

Retrofit Methodology

To mitigate the above-mentioned seismic deficiencies, a comprehensive retrofit methodology was investigated. The components of the seismic retrofit are briefly discussed here.

- A six-in. thick shotcrete cover would be applied to the interior faces of the perimeter walls and pilasters. The shotcrete will be reinforced with a horizontal and a vertical mat of closely spaced No. 5 reinforcement. Sufficient dowels will be provided to attach the shotcrete to the existing walls to ensure that the completed unit works in unison as 18 or 14 in. walls. As a result, the effective reinforcement ratio and spacing of reinforcement would then meet the FEMA 356 [1] requirements.

- By extending the shotcrete vertical reinforcement mat through the slab, these bars would be used to provide sufficient anchorage of the slab.
- For the top level, additional No. 9 bars would be placed near pilasters to form a new column and transfer the existing column loads through the splice. At the ground level, the reinforcement would be exposed and direct splice would be provided by either welding or mechanical splices.
- Additional grade beams would be added to provide a mesh below grade and activate soil passive pressure. The existing grade beams would also be retrofitted as part of wall retrofit; see Figure 11.
- Fiber reinforced polymers (FRP) would be added to the top and bottom of the slabs to provide additional flexural capacity for the critical locations. The FRP strips would be required in both x- and y- directions and would be extended and anchored at locations where the moment demand is small. Figure 12 depicts the FRP zones as hatched areas.
- Steel tubular section would span the opening and mechanically anchored to the concrete walls. They would serve to collect and transfer the seismic forces across the opening; see the solid line of Figure 12.

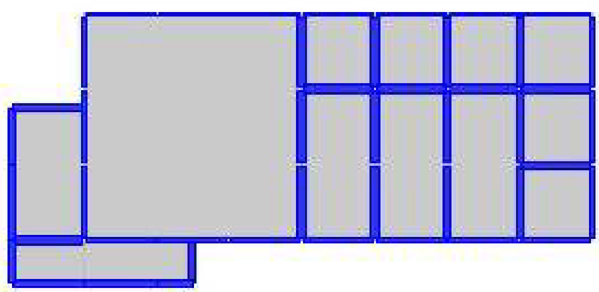

FIGURE 11
ADDITIONAL GRADE BEAMS

Figure 12
FRP AND COLLECTOR ELEMENTS

SEISMIC ANALYSIS OF THE RETROFITTED STRUCTURE

Modal analysis

The inertial weight of the building was estimated at 5,000 kips. Table 2 presents the dynamic properties of the first six modes. The first three modes correspond to the response of the new steel framing, and the last three modes are comprised of the response of the concrete lower floors. As shown in Figure 13, the concrete walls are much stiffer than the foundation and as such, the last three modes correspond to the nearly rigid motion of the superstructure above the flexible foundation.

Mode	Period, sec	Participating mass, %		
		x-	y-	θ-
S1	0.56	8	1	2
S2	0.49	1	11	0
S3	0.41	3	0	8
C1	0.20	60	4	21
C2	0.18	10	70	2
C3	0.15	14	7	63

TABLE 2

MODAL PROPERTIES OF THE BUILDING

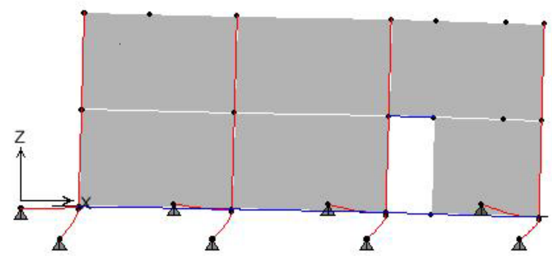

FIGURE 13

MONCRETE PORTION MODE SHAPE

Nonlinear static (pushover) analysis

Using the procedures of FEMA 356 [1] the building was preloaded with gravity loading and then subjected to incremental lateral loading. The control node was selected as the center of mass of the third floor. In each lateral direction, two load patterns were utilized: one proportional to the displaced shape of the building subject to response spectrum loading, and one proportional to the story mass. Figure 14 presents the displaced shape of the building substructure at the performance point. The performance point is obtained as the intersection of the capacity and demand spectra as shown in Figure 15. Note the building response is improved significantly.

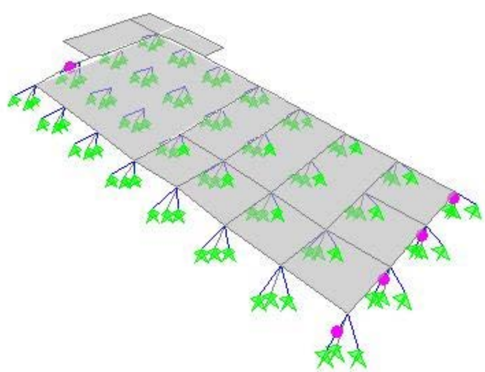

FIGURE 14

PLASTIC HINGE PATTERN, X-DIRECTION

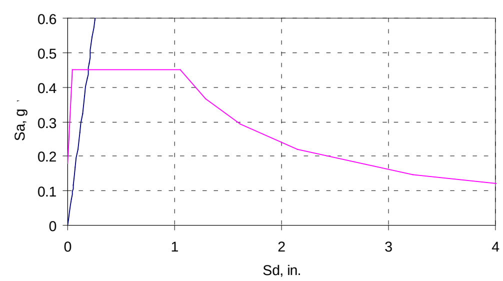

FIGURE 15

NONLINEAR FORCE-DISPLACEMENT CURVE

Nonlinear response history analysis

Nonlinear response history analysis was conducted utilizing the three two-component site-specific spectrum-matched records. Six analyses were conducted and the building response was evaluated using the most severe response. Figure 16 presents the displacement response of the slab-on-grade. Note that the maximum displacement is approximately 0.2 in. As shown in Figure 6, at this level, only slight nonlinearity in the lateral springs would be anticipated. Figure 17 presents the shear-lateral deflection hysteresis for one of the lateral springs. The springs did not experience any net uplift (no tension) and the maximum bearing on them was below their yield point.

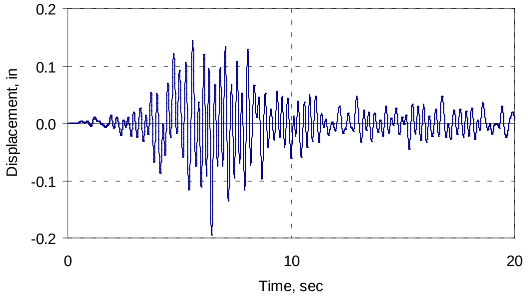

FIGURE 16
TOP OF SHAFT DISPLACEMENT, X-DIRECTION

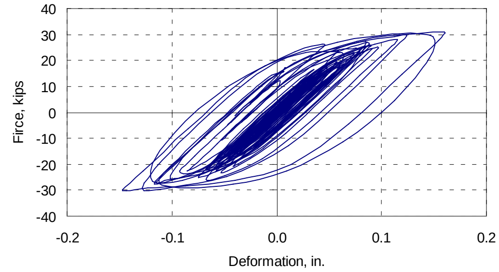

FIGURE 17
SPRING HYSTERESIS RESPONSE

Supplementary calculations

For the retrofitted walls, the maximum ratio of axial load to the wall capacity is below 0.35, and therefore these members can be counted as structural walls resisting seismic loading [3]. For the second floor, the effective reinforcement ratio and spacing and 0.4% and 16 in., respectively. For these walls, the maximum demand-to-capacity ratio (DCR) for shear is 0.24 and hence the walls would remain elastic at the DBE level. No boundary elements are required for the walls. For all the walls the flexural demand is well within the moment-interaction diagram.

For the existing columns, the DCR ratios are less than one. Since for all the columns the ratio of applied load to the column axial capacity is less than 0.5, these members are not force-controlled [1]. Since the axial loading from gravity effects is less than 0.8 of column nominal capacity, the vertical component of seismic loading need not be considered for columns.

The slabs have adequate shear capacity to transfer seismic loading. Steel HSS collectors, anchored to concrete walls and slabs, would be used to transfer the seismic forces along discontinuous wall segments.

SUMMARY AND CONCLUSIONS

Performance-based earthquake engineering was used to assess the seismic performance of an existing structure scheduled for renovation. The analysis indicated that building had some major deficiencies. In particular, the existing foundation shafts had very limited lateral capacity. As part of the retrofit grade beams were added, and the retrofitted structure had satisfactory performance.

- Performance based seismic analysis is an invaluable tool in performing comprehensive seismic investigation.
- For rigid superstructures, the seismic response is highly depended on the stiffness and capacity of substructure. This can be the critical area when examining retrofits.
- Comprehensive seismic retrofit can be utilized to enhance the seismic performance of the complete structural system. Such a retrofit is necessary to save an architecturally or historically significant building. The rehabilitated building then complies with all the current seismic requirements.

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