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SEISMIC RETROFIT OF A MAJOR HOSPITAL BUILDING BASED ON NONLINEAR ANALYSIS CONSIDERING SOIL-STRUCTURE INTERACTION

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SUMMARY

This paper focuses on the seismic retrofit of a six-story 600,000 sq. ft. cast-in-place reinforced concrete building designed in 1968 that is one of the largest medical centers operated by Los Angeles County. The lateral force-resisting system consists of reinforced-concrete shear walls supported by cast-in-drilled hole piles. The seismic performance of the existing building and retrofit scheme was primarily evaluated by the nonlinear static analysis procedure. Although a nonlinear analysis of a large building requires an intensive engineering effort, the owner had the foresight to invest in such an analysis that is widely considered to be state-of-the-art predicting seismic performance, in order to leverage greater savings in construction costs compared to that required based on traditional linear analysis procedures and to provide higher confidence in the retrofit solution. Two-dimensional nonlinear models were developed utilizing elements capable of capturing the axial-flexure-shear interaction in perforated shear walls and the axial-flexure interaction in frame members. Three-dimensional linear elastic models were also developed to evaluate the torsional response of the building. Several seismic deficiencies in the building structure were identified from the analyses, but the primary deficiency was found to be an inadequate lateral capacity of the foundation. Lateral force demands on the foundation obtained from traditional fixed-base analyses were found to be conservative and inaccurate, therefore, a more realistic estimate of the demands on the foundation system were obtained by explicitly considering the nonlinear strength and stiffness behavior of the piles and surrounding soil at each pile group along with the nonlinear building superstructure. Each pile group was modeled using fiber elements that capture the axial-flexural interaction effect on the strength and stiffness of the piles. The soil was modeled using nonlinear horizontal springs along the length of the piles based on soil p-y curves and nonlinear vertical springs to represent the upward and downward capacity of the piles. Analysis of these models allowed for an evaluation of the structural integrity of the foundation system based on displacements, which resulted in a significant reduction in the extent of foundation retrofit work.

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INTRODUCTION

The State of California requires that all general acute care hospital buildings comply with the regulations developed by the Office of Statewide Health Planning and Development (OSHPD) as mandated by Senate Bill (SB) 1953. SB 1953 was introduced and became a law in 1994 as an amendment to the Alfred E. Alquist Hospital Seismic Safety Act of 1983. The Alquist Act establishes seismic safety building standards for hospitals and emphasizes that hospitals should remain operational after an earthquake. SB 1953 addresses the seismic retrofit of existing general acute care hospital buildings under the Division VI-R regulations [1]. The goal of these regulations is to ensure retrofit designs for existing hospital buildings that result in predictable seismic performance. One of the main provisions of SB 1953 is the development of classifications for seismic performance of structural and non-structural components, namely Structural Performance Categories (SPC) and Nonstructural Performance Categories (NPC). A seismic performance evaluation of all general acute care hospital buildings must be performed in order to establish SPC and NPC ratings for the buildings. SB 1953 regulations require that buildings that are classified as SPC-1 and NPC-2 must be brought to SPC-2 and NPC-3 compliance by January 1, 2008, in order to remain operational until the year 2030. According to SB 1953, buildings that have SPC-2 and NPC-3 ratings do not significantly jeopardize life, but may not be repairable or functional following a design level seismic event. The design level seismic event according to the Division VI-R regulations is the smaller of the 475-year return period earthquake at the building site and two-thirds of the maximum considered earthquake, which approximately the 2475-year return period earthquake.

A seismic performance evaluation of the buildings at Martin Luther King, Jr./Drew Medical Center (MLK) was completed in 2000 (Dames & Moore/URS, 2000) and established that the Main Hospital Building is classified as SPC-1. Due to the strategic importance of this facility to the citizens of Los Angeles County, the Los Angeles County Department of Public Works (DPW) issued a Request for Proposal to obtain the services of a design team that could provide a cost effective, timely and minimal operation impact approach to upgrading the seismic performance of the MLK Main Hospital building to be in compliance with the SB 1953 regulations by January 2008. To comply with the County's process to select a design consultant, an RFP was issued, and through the Evaluation Selection Committee, DPW selected the GKK Corporation to serve as the architect for the project, with Saiful/Bouquet Consulting Structural Engineers, Inc. as the Structural Engineer of Record.

The traditional approach to seismic evaluation and retrofit is founded upon new building design where limited damage and collapse prevention objectives are presumably assured through satisfaction of code provisions. Use of such code provisions has allowed engineers to focus on the behavior of the building in the elastic range only using linear analyses. The main shortcoming of a linear analysis is that it provides little or no insight into the post-yield behavior of the building that actually dictates its survivability in the event of a major earthquake. For new building design, ignoring these effects is generally acceptable because the structural engineer can control the load path and locations of energy dissipation through proportioning and detailing of the structural members and connections. However, for existing buildings, the strengths and details of the structural members are predetermined, and therefore, using a linear analysis to predict the seismic performance may be overly conservative or provide misleading information about the deficiencies in the lateral force resisting system and as a result, has led practicing engineers to either recommend retrofit where none is required or recommend retrofit measures which do not address the actual seismic deficiencies. In light of these facts, the design team proposed the use of a nonlinear static pushover analysis procedure (NSP), as provided by the Division VI-R regulations, to evaluate the existing building and the proposed seismic retrofit scheme in order to gain a better insight into the seismic performance of the building, and to determine the amount of retrofit work required. Although the effort of performing a nonlinear analysis is significantly greater than performing a linear analysis, the DPW supported this method, understanding that the additional effort and expense of a nonlinear analysis may result in a more cost efficient retrofit solution.

LOS ANGELES COUNTY DECISION-MAKING

The DPW project management team's participation was important in the seismic retrofit design of the MLK Main Hospital building. To retrofit an operating facility, various restrictions during the construction phase create design challenges. DPW worked closely with the design team and the hospital occupants to assist in the decision-making process of the project. The major challenge was to minimize the disruption to the hospital during construction and still provide a cost-effective solution that meets the year 2008 deadline, without compromising the quality, integrity, and safety of the building. At 50% completion of the schematic design phase, the design team was required to submit three structural options. The County selected the option that was most cost-effective, but more importantly, least disruptive to the facility. The implementation of a structural design retrofit solution that minimizes the hospital's operation during construction was critical. The majority of the impact to the retrofit solution occurred at the basement level. Several areas sensitive to noise, dust, vibration, and infection control requirements are located in the basement, such as, Central Sterile Processing, Pharmacy, and Radiology Departments. In order to minimize the foundation work, DPW authorized extensive structural analysis, in order to develop retrofit solutions that minimized impacts to the interior of the basement. During design, DPW orchestrated numerous meetings between OSHPD and the design team, which helped develop a close working relationship with OSHPD. Quality review and comments on the design were obtained early on in the project in order to streamline the plan check review process. Continuous communication with the hospital occupants assisted the design team's coordination of move management phasing plans with the departments impacted by the retrofit work. Multiple meetings were scheduled to obtain information about the operations of the facility. Furthermore, the information provided by the hospital users assisted the design team to reduce the number of move management phases, thereby, reducing the construction duration.

BUILDING DESCRIPTION

The Main Hospital building of the Martin Luther King, Jr./Drew Medical Center is located in the Compton area of the County of Los Angeles. The building structure was designed in 1968 using the 1965 Los Angeles County Building Code. The building is a six-story cast-in-place reinforced concrete structure with a mechanical penthouse. Grade level is at the first floor on the north and east sides and at the basement level on the south and west sides. The Main Hospital building consists of a central Main Building, a North Wing, and a South Wing. The Main Building is separated from the North and South Wings by a 4-inch seismic joint. Figure 1 shows a layout of the buildings. The reinforced concrete two-way slabs that are supported by beams and columns carry the gravity loads in each building. The seismic lateral force resisting system in both directions of each building consists of reinforced concrete shear walls. The buildings are supported by a deep foundation system consisting of 24-inch diameter cast-in-place drilled piles. An existing canopy at the entrance of the building consists of a reinforced concrete waffle roof slab and beams supported by reinforced concrete columns.

MATERIALS TESTING

As part of the seismic retrofit design for the Main Hospital building, the design team sought to utilize the actual strengths of the materials, which are inherently higher than the specified strengths, in order to reduce the required amount of retrofit work. The design strengths for the concrete and steel reinforcement were specified on the record drawings for this project; however, reports from materials tests performed during construction are not available. As a result, a material testing program was prepared by the structural engineer and performed by a testing laboratory contracted by DPW. Concrete cores were tested primarily to determine the maximum compressive strength of the concrete and steel reinforcement samples were tested primarily to determine the tensile yield strength. Sample locations for concrete and steel reinforcement were selected to provide a wide distribution throughout the building. In

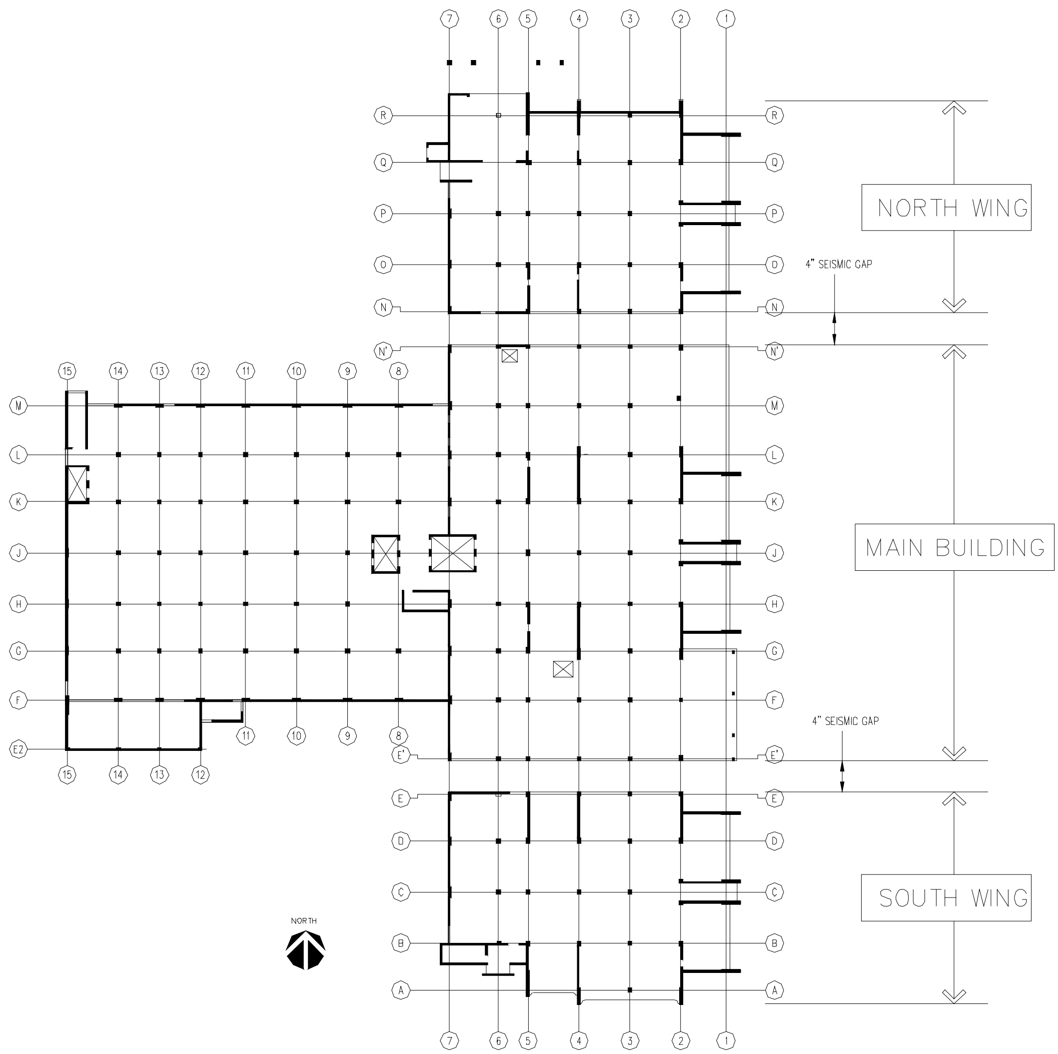

Figure 1. 1st Floor plan view of Main Hospital buildings

addition, the test locations were selected such that the following criterion is met: (1) Minimum of 3 tests per element type (e.g. walls, beams, and slabs.), (2) Minimum of 3 tests per specified strength, and (3) Minimum of 3 tests per floor. The results of the material tests showed that the expected strength of concrete is from 16% to 40% greater than the specified strength and the expected yield strength of steel is 11% and 63% greater than the specified strength for 60 ksi and 40 ksi steel, respectively.

GEOTECHNICAL & SEISMIC HAZARD INVESTIGATION

A geotechnical investigation [3] was performed at the building site in order to explore the subsurface conditions, evaluate the existing foundation and develop recommendations for the seismic analysis and retrofit design of the existing foundation and any potential new foundation elements. Based on the subsurface investigation, it was concluded that alluvial deposits underlie the building site, with groundwater at depths ranging from 34 to 52 feet. It was also concluded that the building site will not be subject to liquefaction under upper-bound earthquake loading conditions and that seismic settlement at the building site is not a concern. A seismic hazard analysis [4] was performed to estimate the site-specific ground motion response spectra for the Main Hospital building. The spectra were computed for the horizontal ground motion corresponding to average return periods of 475 years, 950 years, and 2475 years. These spectra were used in the derivation of the design level earthquake response spectrum, as required by the SB 1953 regulations. In order to develop the response spectra, a probabilistic seismic hazard analysis was performed using a characterization of the seismic sources that could potentially generate strong ground motion at the site. The design response spectra were developed for both 5% and 7% damping and are shown in Figure 2.

NONLINEAR ANALYSIS MODEL

Nonlinear analyses of the MLK Main Hospital building were performed and the results of these analyses form the basis for the proposed seismic retrofit scheme. The nonlinear analyses were performed using a commercially available computer software program called RAM Perform-2D [5]. The nonlinear computer model of the building is a two-dimensional representation of the strength and stiffness of the lateral force resisting elements. A separate computer model was created for the north-south and east-west directions of each of the three buildings, resulting in a total of six models. Using each computer model, a NSP analysis was performed according the recommendations described in FEMA-356 [6]. In the NSP analyses, the building is slowly pushed and when the shear walls reach their capacity, the seismic forces are redistributed to other shear walls that still have the capacity to resist seismic forces. The pushing of the building continues until the roof level of the building reaches a target displacement. The target displacement is calculated using the Coefficient Method procedure described in the FEMA-356 document, which is an iterative procedure requiring the natural period of the fundamental mode of vibration, the design earthquake ground motion response spectrum and the pushover curve (base shear versus roof displacement) from the NSP analysis. In order to account for torsional motions, a target displacement is calculated for each wall line in the two- dimensional nonlinear models. The target displacement for a given wall line is calculated by amplifying the target displacement at the roof level by the ratio of the wall displacement to the average floor displacement obtained from linear analyses of a three-dimensional elastic model of the building using the effective stiffness of the shear walls based on the recommendations in the FEMA-356 document. At the target displacement, the performance of the building is evaluated and compared with the acceptance criteria. The buildings were analyzed using two load patterns, Uniform and Triangular. The Uniform load pattern is a set of lateral forces applied to each floor in proportion to the seismic floor weight. The Triangular load pattern is based on the typical building code distribution where the lateral force distribution along the height of the building is triangular in nature and is a function of the floor height above the base and the seismic floor weight. For each of

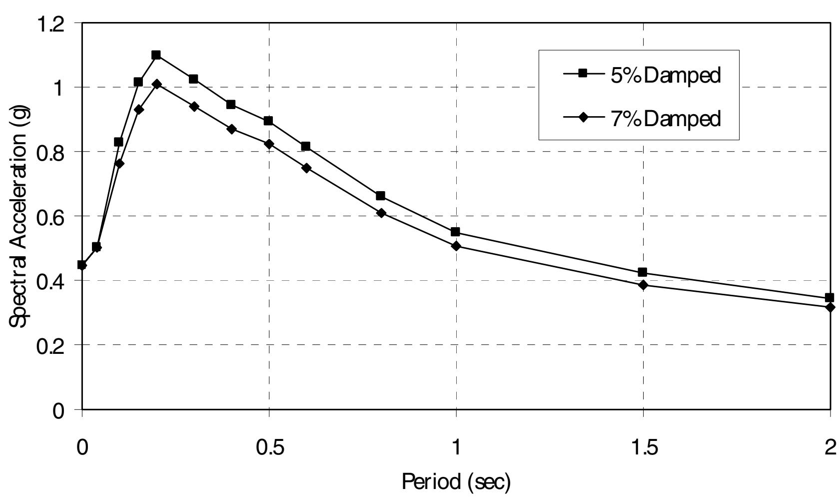

Figure 2. Design response spectrum

these lateral load patterns, the forces were applied in both directions of the building, resulting in "Push" and "Pull" load cases.

The concrete shear walls were modeled using the Perform-2D General Wall element. The General Wall element is a four-node element capable of representing the nonlinear shear and axial-flexure interaction behavior of reinforced concrete shear walls. The General Wall elements account for the concrete in the walls and the distributed reinforcement in the walls by assigning the appropriate stress-strain relationship based on the materials testing results. Nonlinear elements having only axial stiffness and strength were incorporated into the model to account for concentrations of steel and concrete, such as in boundary elements and added reinforcement around openings. These elements were also modeled at the locations of perpendicular wall flanges to account for an effective area of concrete and steel in the wall flange. Columns supporting discontinuous shear walls were modeled using nonlinear beam-column elements that have moment–rotation hinges at the element ends and can account for P-M interaction and have hinge properties as recommended in FEMA-356. Coupling beams were modeled using the General Wall elements, without the effect of the diagonal compression layer. The shear layer was assigned a shear stress-shear strain backbone relationship based on the FEMA-356 guidelines for coupling beams.

EXISTING BUILDING FIXED-BASE ANALYSIS

NSP analyses were performed using the nonlinear models of the existing building assuming that the base of the structure is fixed. Figures 3 and 4 show the pushover curves for each of the six building models and the range of target displacement demands due to the 5% damped design earthquake response spectrum. The nonlinear analyses indicate that the existing building performs well in an overall global sense, however several localized deficiencies were identified. These deficiencies include inadequate lateral resistance of the foundation, vertical discontinuities in shear walls, inadequate strength of some columns supporting discontinuous shear walls, weak coupling beams in shear walls, local stress concentrations in the shear walls, and a lack of seismic bracing system in east-west direction of Main Building in the 5th story. These deficiencies have been addressed in the preliminary seismic strengthening scheme. The proposed retrofit scheme was developed by the design team with the intent of correcting the structural deficiencies while having the utmost concern for minimizing disruption to the operation and function of the facility during and after construction. This goal resulted in a focus of the design team's effort towards developing a solution that minimizes interior impact. Significant vertical shear wall discontinuities will be eliminated by infilling the discontinuity with new reinforced concrete shear walls. In those less significant locations, the columns supporting the discontinuous shear walls are strengthened by concrete jacketing or wrapping with fiber reinforcement. It was found that many of the shear wall coupling beams are weak and may exhibit strength degrading behavior. However, even with the failure of many of the coupling beams, the structure generally has enough reserve lateral strength to provide adequate performance. Of particular interest is the shear wall along gridline Q between gridlines 5 & 7 (see Figure 1), which is one of the primary North Wing shear walls resisting seismic forces in the east-west direction. The wall is interrupted by significantly large openings at each floor level creating coupling beams at each floor level that have inadequate shear strength. This problem is further exacerbated by the fact that the west half of this wall is located adjacent to an existing stairwell and is not connected to the floor slab. As shown in Figure 4, failure of these coupling beams causes a significant loss in lateral load carrying capacity. This deficiency will be mitigated in the proposed retrofit scheme by increasing the thickness of a significant portion of the shear wall and reinforcing the coupling beams such that the integrity of the existing coupling beams is maintained.

In addition to the seismic deficiencies in the superstructure, the NSP analyses of the three buildings revealed significant issues with the adequacy of the foundation system to resist the lateral load demands. The existing foundation system consists of 24" diameter reinforced concrete cast-in-place drilled piles with either 4-#9 or 4-#10 longitudinal reinforcement and #3@12" square ties. Pile groups are formed

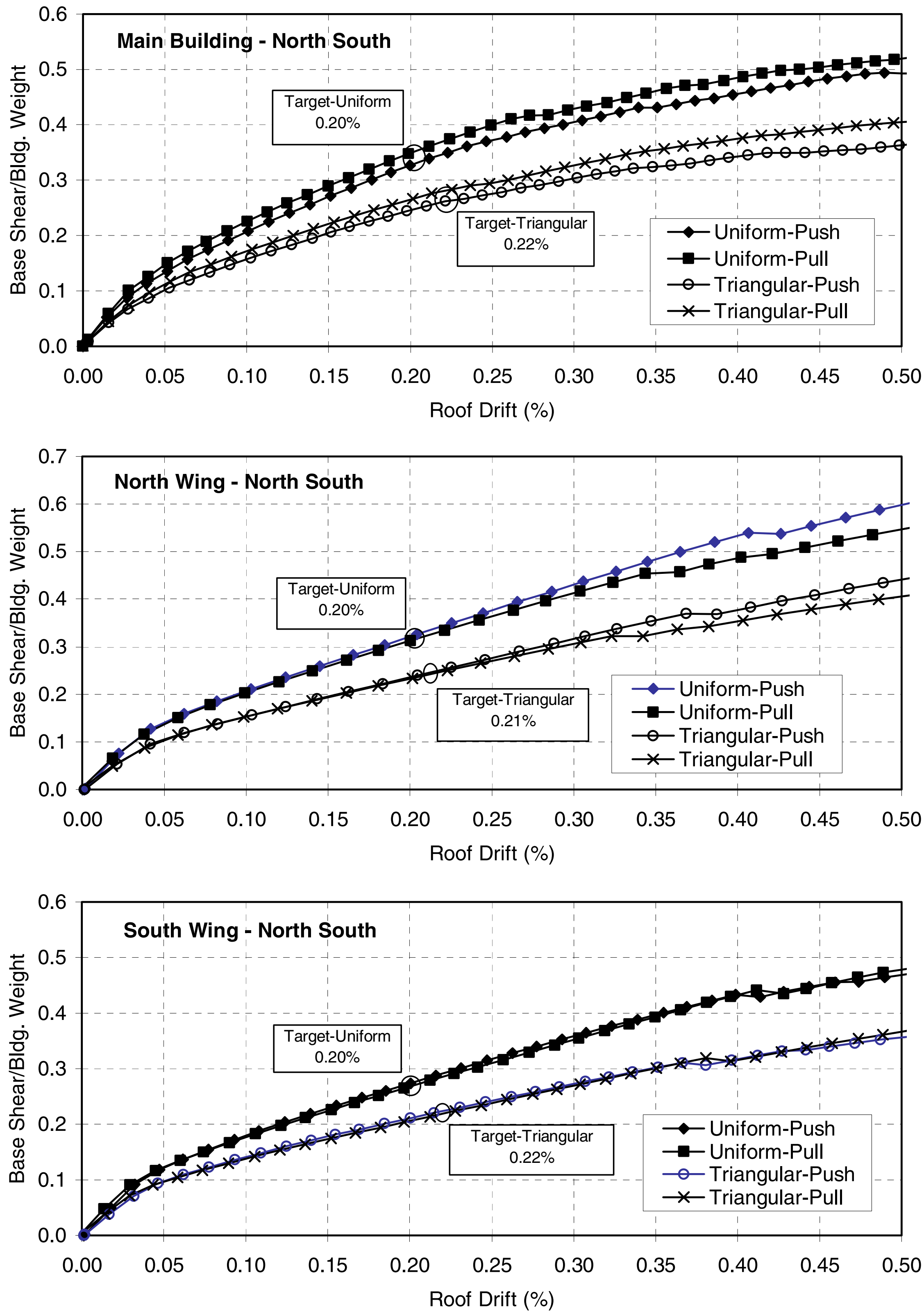

Figure 3. Fixed-base pushover curves for North-South direction

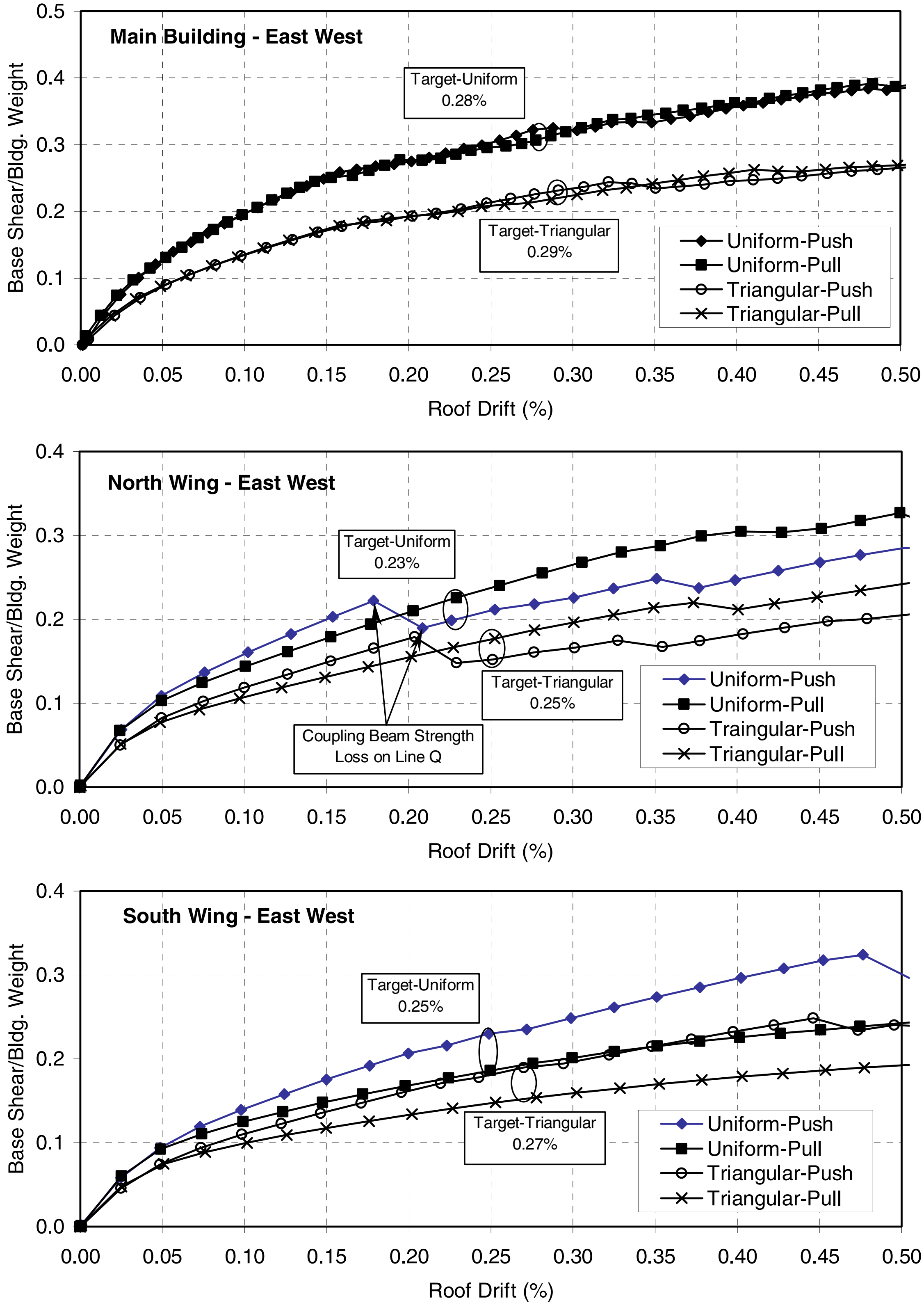

Figure 4. Fixed-base pushover curves for East-West direction

using 5-ft thick pile caps linked together with tie beams. A slab-on-grade at the Basement Level is connected to the shear walls with minimal dowel reinforcement and has construction joints regularly spaced at 30 feet intervals in the north-south and east-west directions. As such, the slab-on-grade is unable to transfer shear from the walls to adjacent lines of piles and only the piles along the lines of the walls could be reliably considered to resist the lateral forces. The geotechnical engineer provided the ultimate lateral capacity of the piles as a function of the lateral displacement of the pile head based on a pile-soil interaction analysis where the soil p-y curves have a nonlinear force-deformation relationship and the piles are linear elastic with assumed 50% cracked flexural stiffness. In addition, the geotechnical engineer provided recommendations for passive pressure resistance against the pile caps and pile group efficiency factors that reduce the lateral capacity of the piles to account for shadowing effects.

The foundation lateral load demands from the NSP fixed-base analysis and the corresponding lateral capacities are shown in Tables 1 through 6. The lateral load demands shown in these tables are the maximum demands obtained from the four applied lateral load cases at the appropriate target displacements. In all cases, the demands from the analyses using the uniform load pattern dominate the design by 20% to 25% more than the demands from the triangular load pattern. The lateral capacities are based on the assumption that the piles would be capable of displacing 3/4" at the top of the pile before the structural integrity of the piles is diminished. The tables show that for some walls, considerable additional lateral capacity is required. One of the primary concerns is the lack of lateral foundation capacity under the basement walls along the east side of all three buildings at gridline 1, and at the north side of the North Wing along gridline R and the south side of the South Wing along gridline A. The basement walls introduce a significant stiffness at the 1st floor, thus attracting a large amount of shear to these walls that the foundation system is unable to accommodate. The lack of foundation lateral capacity is also a concern for the primary lateral load resisting walls in the Main Building at gridlines 7 & 15 in the north-south direction and gridlines F & M in the east-west direction. The geotechnical engineer offered several suggestions for providing additional lateral foundation resistance, including adding new 24" or 36" diameter piles, adding massive shear lugs below grade to mobilize a large volume of soil through passive pressure, or improving the capacity of the existing soil by injecting slurry or other materials. It was found that the most effective method for providing additional foundation capacity would be to add new 36" diameter piles. Tables 1 through 6 show the additional lateral capacity required for each direction of each building and the number of new piles needed to achieve this capacity. Considering the piles required for all three buildings, a total of 82 new piles would be needed to supplement the existing foundations for lateral loads in the north-south direction and 74 new piles would be needed for the east-west direction. A foundation retrofit scheme of this magnitude would require a substantial amount of construction work to be performed in the interior of the building, causing severe interruptions to the operation of the facility, in addition to the fact that it seemed overly conservative to the design team.

In Section 4.4.3.3 of the FEMA-356 document, the foundation element acceptance criteria for NSP analyses using a fixed-base assumption requires that the demands be considered "force-controlled" and that the earthquake component of the demand can be divided by a "J-factor," as specified in Section 3.4.2.1.2. However, the intent of the "J-factor" is to reduce the load demands that are calculated from a linear analysis to account for ductility of the elements that deliver load to the foundation. In the case of a nonlinear analysis, one may argue that the "J-factor" is not appropriate since the foundation load demands have already been reduced due to the nonlinear behavior of the superstructure. However, in reality, the fixed-base analysis does not account for the strength and flexibility of the soil and the piles, which would reduce the load demands. Furthermore, the uniform load pattern distribution assumption provides a conservative upper bound demand on horizontal shear force that can be delivered to the foundation, as much as 25% more than the triangular load pattern. As in the case of the MLK Main Hospital building, the combined effect of these two factors can lead to a massive and costly retrofit of the existing foundation. Therefore, it would seem appropriate to apply a reduction factor such as the "J-factor" to reduce the foundation loads obtained from the pushover analysis using the uniform load pattern, however,

Table 1. Foundation results for North Wing N-S Dir. (NSP)

Gridline	Fixed Base								Flexible Base Preliminary Retrofit	
	Max Shear Demand (k)	Lateral Capacity (k)	Unreduced		J=1.5		J=2			
			Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Max Pile Disp (in)	# of New Piles Added
1	3205	1475	1730	13	662	5	128	1	0.59	4
2	1409	2271	0	0	0	0	0	0	0.25	0
4	1303	2052	0	0	0	0	0	0	0.20	0
5	1930	2384	0	0	0	0	0	0	0.23	0
7	2571	2660	0	0	0	0	0	0	0.28	2
7.6	24	189	0	0	0	0	0	0	0.04	0
			Total =	13		5		1		6

Table 2. Foundation results for South Wing N-S Dir. (NSP)

Gridline	Fixed Base								Flexible Base Preliminary Retrofit	
	Max Shear Demand (k)	Lateral Capacity (k)	Unreduced		J=1.5		J=2			
			Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Max Pile Disp (in)	# of New Piles Added
1	3199	1475	1724	8	658	3	124	1	0.52	4
2	1654	2271	0	0	0	0	0	0	0.20	0
4	1571	1738	0	0	0	0	0	0	0.19	0
5	1486	1903	0	0	0	0	0	0	0.17	0
6	182	748	0	0	0	0	0	0	0.08	0
7	2703	2619	84	1	0	0	0	0	0.24	2
			Total =	9		3		1		6

Table 3. Foundation results for Main Building N-S Dir. (NSP)

Gridline	Fixed Base								Flexible Base Preliminary Retrofit	
	Max Shear Demand (k)	Lateral Capacity (k)	Unreduced		J=1.5		J=2			
			Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Max Pile Disp (in)	# of New Piles Added
0.5 & 1	8303	2108	6195	14	3427	8	2043	5	0.36	9
1.4	91	180	0	0	0	0	0	0	0.23	0
1.6	115	177	0	0	0	0	0	0	0.37	0
2	4589	2664	1925	5	395	1	0	0	0.29	0
4	2488	2319	169	1	0	0	0	0	0.19	0
5	1778	3110	0	0	0	0	0	0	0.12	0
7	7961	4759	3202	15	548	3	0	0	0.31	9
6.5	330	590	0	0	0	0	0	0	0.35	0
7.5	409	692	0	0	0	0	0	0	0.34	0
7.9	121	2371	0	0	0	0	0	0	0.04	0
8.5	333	977	0	0	0	0	0	0	0.06	0
11	135	846	0	0	0	0	0	0	0.03	0
12	363	976	0	0	0	0	0	0	0.12	0
13	2635	1027	1608	8	730	4	291	2	0.66	0
15*	10829	7449	3380	17	0	0	0	0	0.60	0
			Total =	60		16		7		18

* In both Fixed and Flexible Base cases, additional lateral resistance is provided by adding new foundation elements to engage the pile caps and piles on gridline 14.

Table 4. Foundation results for North Wing E-W Dir. (NSP)

Gridline	Fixed Base								Flexible Base	
	Max	Lateral	Unreduced		J=1.5		J=2		Preliminary Retrofit	
	Shear Demand (k)	Capacity (k)	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Max Pile Disp (in)	# of New Piles Added
O.8	694	886	0	0	0	0	0	0	0.17	0
N.4	545	858	0	0	0	0	0	0	0.22	1
P.3	612	886	0	0	0	0	0	0	0.15	0
P.6	74	155	0	0	0	0	0	0	0.15	0
Q	913	1901	0	0	0	0	0	0	0.32	0
Q.3	47	189	0	0	0	0	0	0	0.25	0
Q.6	326	858	0	0	0	0	0	0	0.18	0
R & R.5	2489	1585	904	7	74	1	0	0	0.52	3
N.1	1519	4221	0	0	0	0	0	0	0.22	0
			Total =	7		1		0		4

Table 5. Foundation results for South Wing E-W Dir. (NSP)

Gridline	Fixed Base								Flexible Base	
	Max	Lateral	Unreduced		J=1.5		J=2		Preliminary Retrofit	
	Shear Demand (k)	Capacity (k)	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Max Pile Disp (in)	# of New Piles Added
E	1233	1979	0	0	0	0	0	0	0.13	0
D.6	459	858	0	0	0	0	0	0	0.08	1
C.3	319	886	0	0	0	0	0	0	0.06	0
B.8	318	886	0	0	0	0	0	0	0.06	0
A.4	230	858	0	0	0	0	0	0	0.04	0
B	930	1416	0	0	0	0	0	0	0.14	0
A.6	537	1207	0	0	0	0	0	0	0.13	0
A.3	30	177	0	0	0	0	0	0	0.08	0
A	2699	1384	1315	6	415	2	0	0	0.24	3
			Total =	6		2		0		4

Table 6. Foundation results for Main Building E-W Dir. (NSP)

Gridline	Fixed Base								Flexible Base	
	Max	Lateral	Unreduced		J=1.5		J=2		Preliminary Retrofit	
	Shear Demand (k)	Capacity (k)	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Additional Capacity (k)	# of New Piles Required	Max Pile Disp (in)	# of New Piles Added
N	1299	3214	0	0	0	0	0	0	0.70	0
K.8	118	1346	0	0	0	0	0	0	0.08	0
K	144	884	0	0	0	0	0	0	0.23	0
M	13274	4901	8373	40	3948	19	1736	9	0.80	11
L.4	1246	177	1069	3	654	2	446	1	0.34	0
K.6	440	908	0	0	0	0	0	0	0.22	0
G.1	1062	785	277	1	0	0	0	0	0.20	0
J8	695	822	0	0	0	0	0	0	0.07	0
J4	755	1494	0	0	0	0	0	0	0.41	0
H.6	729	1362	0	0	0	0	0	0	0.43	0
J3 & H.8	2705	1648	1057	3	155	1	0	0	0.08	0
G.4	21153	872	20281	45	13230	29	9704	22	0.23	1
G.8	416	1327	0	0	0	0	0	0	0.26	0
F	9057	4383	4674	20	1655	7	146	1	0.80	6
E6	128	185	0	0	0	0	0	0	0.53	0
E3	344	398	0	0	0	0	0	0	0.43	0
E1	812	874	0	0	0	0	0	0	0.76	0
D.2	493	368	125	1	0	0	0	0	0.78	0
			Total =	113		58		33		18

FEMA-356 is unclear in this regard. As a result, the OSHPD review team concluded that if a fixed-base analysis is used to predict load demands on the foundation, then the demands should remain unreduced. Since the lateral capacity of the foundation is not an issue of strength of the piles or the soil, but instead an issue of displacement capacity of the system, the design team proposed and OSHPD accepted an alternate approach in order to mitigate the need to extensively retrofit the existing foundation. The alternate approach consisted of a NSP analysis incorporating the strength and flexibility of the soil and piles, providing a more accurate insight into the seismic performance of the foundation system. This would require additional information from the geotechnical engineer regarding the soil p-y curves and additional modeling effort to incorporate this information into the nonlinear analysis models. DPW strongly supported the design team and agreed to make a greater investment in the design phase that could potentially produce a significant construction cost savings and minimize disruption to the hospital operations.

SOIL-STRUCTURE INTERACTION MODELING

The geotechnical engineer provided additional soils information to the structural engineer in order to incorporate the soil-structure interaction effects into the nonlinear models. Based on the variation in the soil samples extracted during the original geotechnical investigation, the soil beneath all three buildings was subdivided into six zones named as follows: Tower West, Tower East, South Tower, North Tower, Elevator Pits and Medical Records soil zones. Soil p-y curves were provided for the existing 24" diameter piles in each soil zone at 12" intervals along the depth of the piles. For three out of the six soil zones, p-y curves were provided for both typical pile caps and for pile caps that are significantly lower than the typical pile caps. Downward and upward capacities were provided for piles in each soil zone. Although the piles are typically 40 feet in length, the longitudinal reinforcement in the piles does extend throughout the complete length of the piles. The piles supporting walls are typically reinforced 25 feet down from the top of the pile, whereas all other piles are reinforced only 17 feet down from the top of the pile. The upward capacities of the piles were adjusted by the geotechnical engineer to consider that frictional resistance can be developed only along the reinforced length of the pile.

The nonlinear fixed-base models of the buildings were further developed to incorporate soil-structure interaction effects. The flexibility of the existing pile foundation was incorporated into the Perform-2D models of the MLK buildings by explicitly modeling the piles and the soil springs along the length of the piles. The pile-soil interaction model was incorporated at each pile cap location providing support for a wall. Pile groups were modeled as a fixed head condition and single piles were modeled as a free head condition. The piles were modeled using fiber elements capable of capturing axial-flexural interaction effects on the strength and stiffness of the piles. Expected material strengths obtained from the materials testing program were used to develop the stress-strain curves for concrete and steel. The properties of the pile elements were scaled to account for the number of piles at each support. Horizontal inelastic springs having a trilinear force-deformation backbone curve were modeled at 12" intervals along the reinforced length of the piles based on the soil p-y curves obtained from the geotechnical engineer. The properties of the soil springs were scaled to account for the number of piles and the efficiency of the pile group. In addition, a horizontal inelastic soil spring was modeled at the top of the pile to represent the strength and stiffness provided by passive soil resistance against the pile cap. A vertical inelastic spring was modeled at the bottom of the pile to represent the downward and upward capacity of the soil.

RETROFIT BUILDING FLEXIBLE-BASE ANALYSIS

The proposed foundation retrofit scheme consists of adding new 36" diameter piles around the perimeter of the building as shown in Figure 5. The new piles are connected to each other with deep grade beams in order to provide rotational restraint at the top of the piles and are attached to the existing foundation system by a thick slab to transfer the lateral loads. NSP analyses were performed using the nonlinear models of the buildings incorporating soil-structure interaction effects in models that include the preliminary superstructure and foundation retrofit scheme. The target displacement demands were calculated using the 7% damped design earthquake response spectrum shown in Figure 2, in order to account for the additional energy dissipation by the soil during a seismic event. By including the strength and flexibility of the soil and piles in the nonlinear models of the buildings, the acceptability of the existing foundation system now becomes an issue of displacement demands rather than force demands of the piles. The goal of the foundation retrofit scheme is to limit the displacements on the existing foundation system such that structural integrity of the existing piles is maintained. Single pile models were studied to relate displacements to the structural behavior of the piles with sensitivity of the behavior to soil zone type, pile group efficiency factors and a range of axial loads. It was found that the acceptable displacement of the piles is governed by the case where the pile is subjected to the maximum compression force that the pile will experience as limited by the downward capacity of the soil. The pile concrete is essentially unconfined, since the transverse reinforcements consists of only #3@12" square ties. Therefore, for large magnitudes of compression axial load, the pile experiences a significant loss in lateral load carrying capacity after the formation of a flexural hinge at the top of the pile. In general, the pile strength degrading occurs at approximately 1" of pile head displacement. Tables 1 through 6 show the average displacement of the foundation system at the base of each wall line and the number of new 36" diameter piles added to supplement each wall line. The walls along gridline F and M in the east-west direction of the Main Building experience the largest foundation displacement demands, however, these displacements are still within the acceptable limits. In addition to evaluating the displacement, pile shear

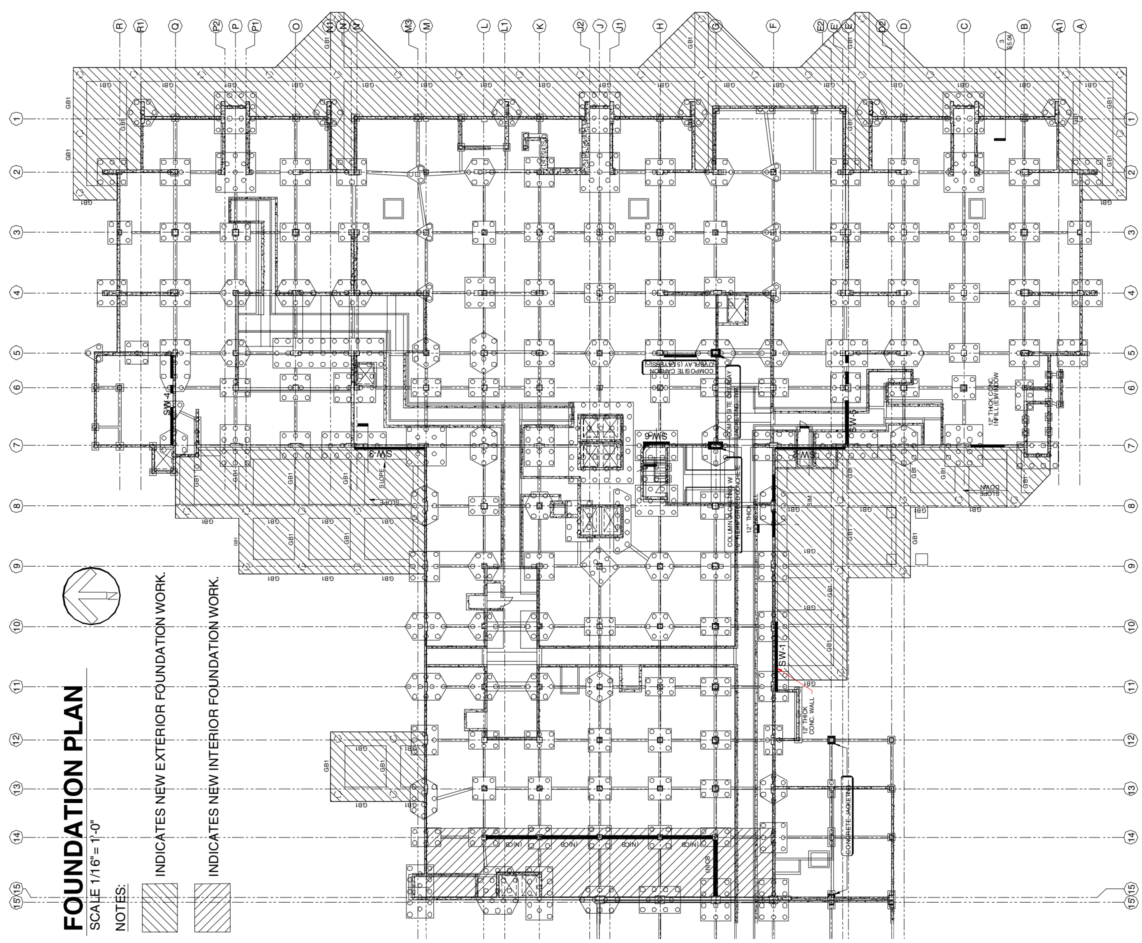

Figure 5. Foundation plan showing proposed foundation retrofit scheme

demands were checked after the analysis to determine whether pile shear failures would occur. Pile capacities were calculated according to the guidelines provided in the FEMA-356 document that include the effects of axial load on the concrete contribution to the shear capacity. In all cases, it was found that no pile shear failures would occur using the proposed foundation retrofit scheme.

The NSP analyses incorporating soil-structure interaction has demonstrated that the preliminary foundation retrofit scheme shown in Figure 5 provides acceptable seismic performance by maintaining the structural integrity of the existing foundation system. With the proposed foundation retrofit, only 30 piles were needed to supplement the north-south direction compared to the 82 piles that would have been required based on the fixed-base analysis. In the east-west direction, the proposed foundation retrofit requires only 26 piles compared to 74 using the fixed base analysis. This translates into a significant savings in construction costs, move-management efforts and disruption to hospital operations. In retrospect, the extent of foundation retrofit work required by the more accurate NSP analysis incorporating the soil-structure interaction effects was used to hypothetically determine an appropriate "J-factor" that could be used to reduce the demands from the NSP fixed-base analysis and achieve a similar foundation retrofit scheme. The lateral foundation demands obtained from the fixed-base analysis shown in Tables 1 through 6 were reduced by "J-factors" of 1.5 and 2. These tables show the additional lateral capacity that would be needed using these "J-factors" and the number of new 36" diameter piles that would be needed. The results indicate that by using a "J-factor" of 1.5, the number of piles required using the fixed-based analysis is comparable to the retrofit resulting from the flexible-base analysis. Note that in both cases, the foundation retrofit is governed by foundation demands obtained from the uniform load pattern. As previously demonstrated, the uniform lateral load pattern provides upper bound demands on the foundation, which may be as much as 25% higher than the foundation demands experienced during the design-level earthquake ground shaking.

CONCLUSIONS

The Main Hospital building of the Martin Luther King, Jr./Drew Medical Center currently does not meet the life-safety performance objectives required by SB 1953 regulations to maintain the operation of the facility beyond the year 2008. The seismic performance of the existing building was evaluated by the nonlinear static pushover analysis procedure using two-dimensional nonlinear models of the building. The analyses showed that the overall global seismic performance of the existing building is satisfactory and requires only minor strengthening of the superstructure to correct localized deficiencies, in contrary to the more conservative linear analysis that would require major strengthening of the superstructure. However, unreduced lateral force demands on the foundation obtained from nonlinear fixed-base analyses, showed that extensive foundation retrofit work would be required at a considerable expense to the County of Los Angeles and disruption to hospital operations, potentially threatening the future of the project. Since the nonlinear behavior of the piles and soil is not considered in a fixed-base analysis, the lateral force demands on the foundation should be reduced to account for this behavior with the use of a "J-factor" similar to that defined for linear analysis procedures in the FEMA-356 document. However, since there is no guidance on this issue in existing documents, OSHPD provided the design team with the alternative of considering the flexibility of the foundation directly in the nonlinear analyses in order to predict displacement demands on the piles and to evaluate their structural integrity. The result was a more accurate prediction of the seismic performance of the building superstructure and foundation system during the design level earthquake. The number of new piles required to supplement the existing foundation system based on the flexible-base analysis is considerably less than that required by the fixed-base analysis using unreduced lateral force demands. Although the explicit consideration of the soil-structure interaction represented an intensive effort on behalf of the structural engineers and an additional investment in the design phase by DPW, the reduction in construction and move-management costs made this effort extremely beneficial to the citizens of Los Angeles County.

In this study, it was found that the unreduced lateral force demands from a nonlinear fixed-base analysis would require an unrealistic amount of foundation retrofit work in order to maintain the structural integrity of the existing foundation. This is caused by the assumption of a fixed-base, which does not account for the nonlinear behavior of the piles and soil, and also by the use of the uniform lateral load pattern, which provides conservative upper bound lateral force demands on the foundation and ultimately governs the foundation retrofit design. Future research should focus on the development of appropriate values for the “J-factor” for foundations evaluated using a nonlinear fixed-base analysis. Based on the buildings, soil conditions and seismic hazard at this particular site, it was found that if the existing foundation was evaluated using the lateral force demands from a nonlinear fixed-base analysis reduced by a “J-factor” of 1.5, then the amount of foundation retrofit work required would be similar to that currently provided in the preliminary foundation retrofit scheme designed based on the results of the nonlinear flexible-base analysis. Since there is little guidance for the use of a “J-factor” in the calculation of lateral force demands on the foundation, the final seismic retrofit scheme for this project will be based on a nonlinear model that includes explicit consideration for soil-structure interaction effects.

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