16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 336
Registration Code: S-K1461196677

NONLINEAR MODELING OF MAT FOUNDATION DIFFERENTIAL SETTLEMENTS IN SEISMIC EVALUATION OF HIGH-RISE BUILDINGS

A. Aviram[(1)], A. Dutta[(2)], R.O. Hamburger[(3)]

[(1)] *Structural Engineering Independent Consultant, Ph.D., P.E. (CA), ady.aviram@gmail.com*
[(2)] *Senior Project Manager, Ph.D., S.E. (CA), Simpson Gumpertz & Heger Inc., adutta@sgh.com*
[(3)] *Senior Principal, S.E. (CA and others), SECB, Simpson Gumpertz & Heger Inc., rohamburger@sgh.com*

Abstract

Differential foundation settlement can impose significant demands on a building's seismic force resisting system and reduce the structure's seismic capability. This paper describes an inventive evaluation methodology of the effects of such settlement on a tall reinforced concrete shear wall building. Seismic evaluation of building structures experiencing large differential foundation settlement is a complex task given current limitations of commercially-available analysis software, as well as many geotechnical and structural uncertainties. Under static conditions, the effect of differential settlement on structural adequacy can be assessed by modeling the structure and its foundation, imposing a deformed shape on the foundation model or base supports to match measured or estimated settlement contours, and evaluating structural demands on the structure and foundation system. However, for seismic analysis, the foundation deformation should adjust as the mathematical representation of the building structure is subjected to simulated ground motions and as mat nonlinearity develops. The evaluation of the high-rise building and its foundation was carried out in several stages using CSI Perform-3D. The pile cap and mat foundation supporting the building were represented by a nonlinear beam grillage model with flexural and shear hinges, which was validated through finite element analysis. Hinge properties were estimated based on multiple layers of top and bottom flexural reinforcement, as well as vertical dowels used as shear reinforcement, provided throughout the foundation. Externally applied vertical point loads in select locations of the mat and underlying vertical soil spring properties were iteratively determined to produce a settlement profile matching measured deformations, while allowing additional mat deformation under ground motion excitation. The building model included explicit representation of basement retaining walls, first floor slab, building core walls, moment resisting frames, and outriggers to capture the relative stiffness of these elements and their effect on the foundation deformation. A suite of 7 ground motion records was imposed on the combined superstructure-foundation model to assess the effect of settlements on the building's seismic adequacy using Performance-Based acceptance criteria.

Keywords: nonlinear grillage; differential settlements; mat foundation; pile cap; seismic evaluation

1. Introduction

Seismic evaluation of building structures experiencing large foundation settlement is a complex task given current limitations of commercially-available analysis software, as well as many geotechnical, structural and modeling uncertainties. To assess differential settlement-related demands on the superstructure, typical engineering practice involves imposing a deformed shape on the mat or base supports for the structure to match measured or estimated settlement contours. However, this modeling approach has several shortcomings. First, the foundation deformations should adjust as the mathematical representation of the building structure is subjected to simulated ground motions. Second, as nonlinear behavior develops in the mat due to anticipated flexural or shear failures, the deformed mat shape should adjust accordingly to capture nonlinear effects. Finally, settlement contours are developed based on a finite number of settlement measurement points, which typically neglect the stiffness and nonlinear response of the building superstructure, basement walls, or soil-structure interaction. Thus, strictly imposing deformation contours on a building mat foundation while disregarding these effects can lead to severe inaccuracies in force demand estimates on the supporting mat, as well as connecting superstructure. This paper presents the details of an inventive modeling approach that overcomes the above-mentioned difficulties. This methodology was used for nonlinear modeling and seismic performance assessment of a pile-supported high-rise building that underwent differential settlement.

To evaluate anticipated building behavior under seismic demands, a nonlinear computer model of the building and its foundation was developed using CSI Perform-3D V5.0.0 (Perform) [1]. The building is supported on a continuous, thick, cast-in-place, reinforced concrete pile cap connecting evenly-spaced precast concrete piles. Initial mat evaluation was carried out using an elastic shell model with effective cracked cross section properties. Imposed deformations on this model coinciding with a finite number of measured settlement locations were obtained by specifying support spring loads. The model included explicit representation of basement retaining walls, first floor slab, building core walls and outrigger frames to capture the relative stiffness of these elements and their effect on the mat's deformation contours.

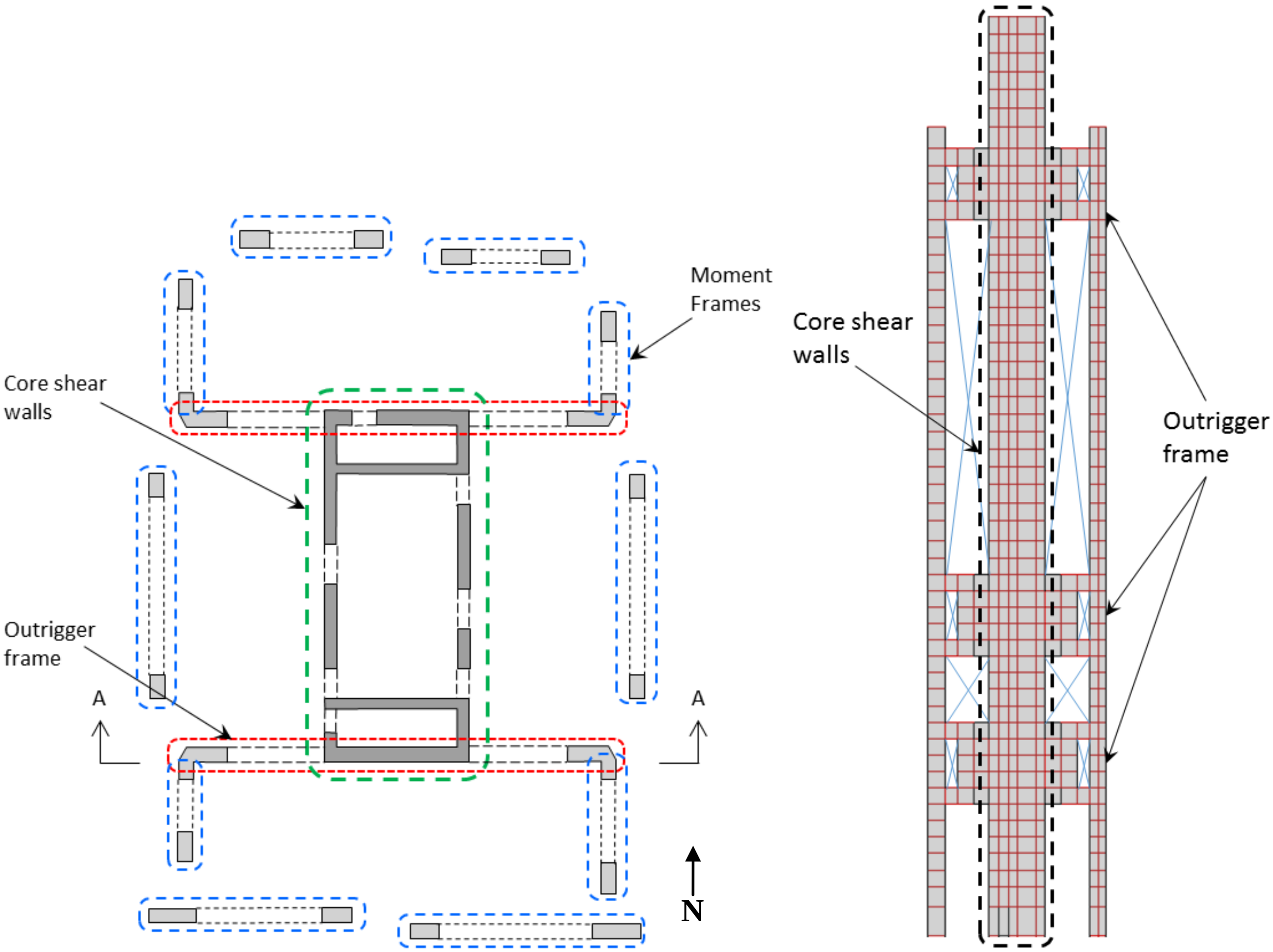
A second modeling stage involved removal of the support springs and replacement with externally applied vertical point loads on the mat and underlying vertical soil springs to allow mat deformation under ground motion excitation. Since net tensile demands were not anticipated on the piles in the current assessment due to high gravity loads, compression-only soil springs were defined throughout the mat. An iterative process involving adjustment of the distributed soil spring stiffness at different areas of the mat and the magnitude of external loads applied at select locations was used to match the smooth deformed shape under gravity loads and differential settlements previously calculated.

A third modeling stage consisted of replacing the elastic shell elements with a nonlinear beam grillage that includes flexural and shear hinges and which was incorporated into a global model of the building. Hinge properties were estimated based on the multiple layers of top and bottom flexural reinforcement, as well as vertical dowels used as shear reinforcement, provided throughout the mat. Grillage beam model assumptions are validated through finite element analysis. A suite of 7 ground motion records was imposed to assess the effect of settlements on the building's seismic adequacy using Performance-Based acceptance criteria.

2. Building Model Overview

2.1 Superstructure

The building is a high-rise structure with one basement level supported on a pile cap foundation. The main lateral and gravity load-resisting system consists of a reinforced concrete core and reinforced concrete outrigger frames (see Fig. 1). To evaluate anticipated building behavior, a non-linear computer model of the building was developed in Perform following the PEER Tall Building Initiative guidelines [2]. The mathematical representation of the structure was then subjected to a suite of 7 ground motions to simulate seismic demands on structural elements. The Perform model included representation of the core and outrigger shear walls and spandrel beams, reinforced concrete and embedded steel coupling beams, basement retaining walls, concrete slabs at the bottom two levels, diaphragm constraints in remaining levels, and several representations of the pile cap, discussed below.



(a) Plan (b) Elevation

Fig. 1 – Plan view and elevation of main lateral force-resisting system

2.2 Pile Cap Details

The building is supported on a single, continuous 10 ft thick cast-in-place reinforced concrete pile cap connecting precast concrete piles spaced at an average 5’-0” on center. A 3 ft thick slab cantilevers off of the 10 ft thick pile cap on one end of the cap, as shown in Fig. 2(a). This portion of the mat is directly supported on soil. Multiple layers of top and bottom flexural reinforcement are provided throughout the mat foundation. Shear reinforcement is provided in the 10 ft thick portion, consisting of headed bars at spacing of 24 to 36 in, as shown in Fig. 2(b).

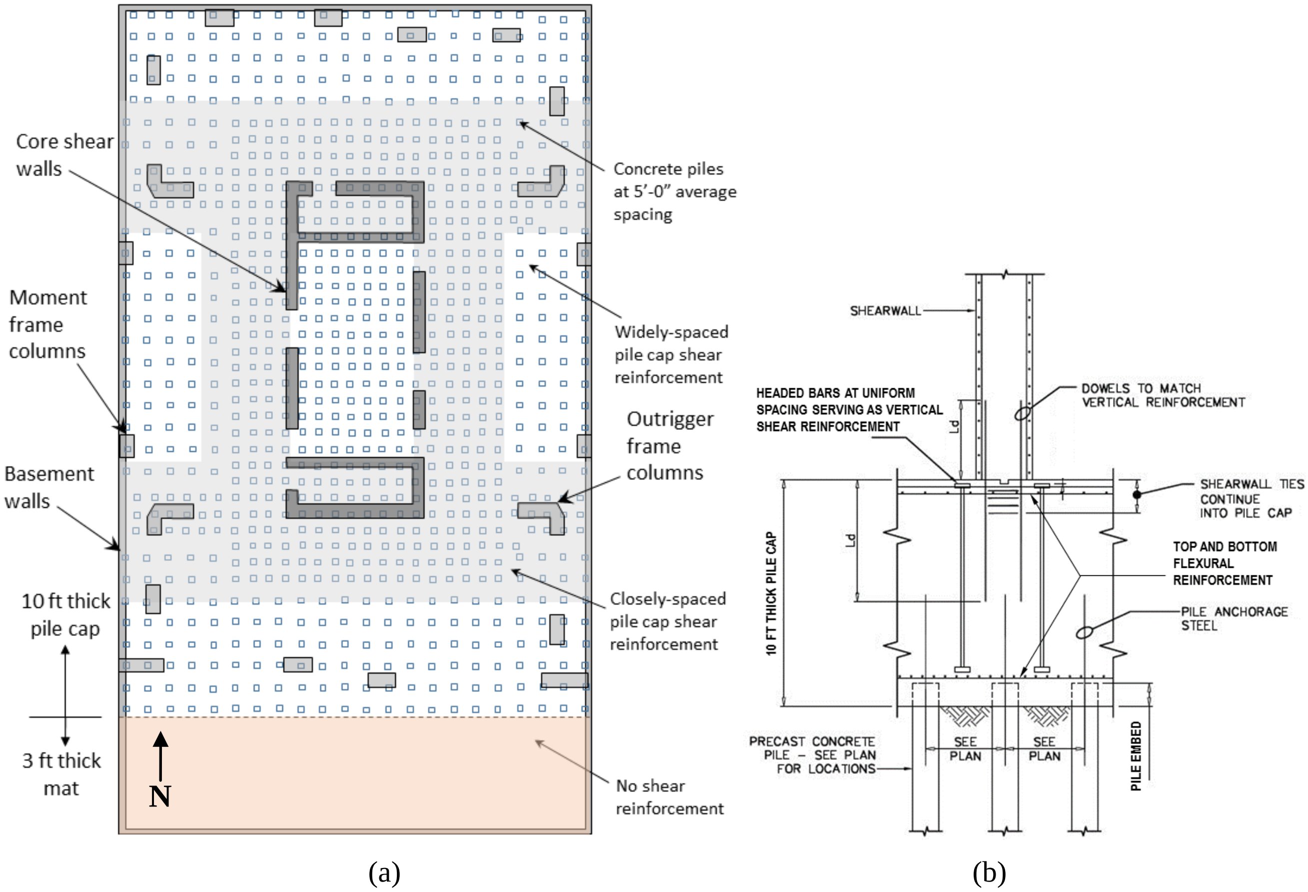
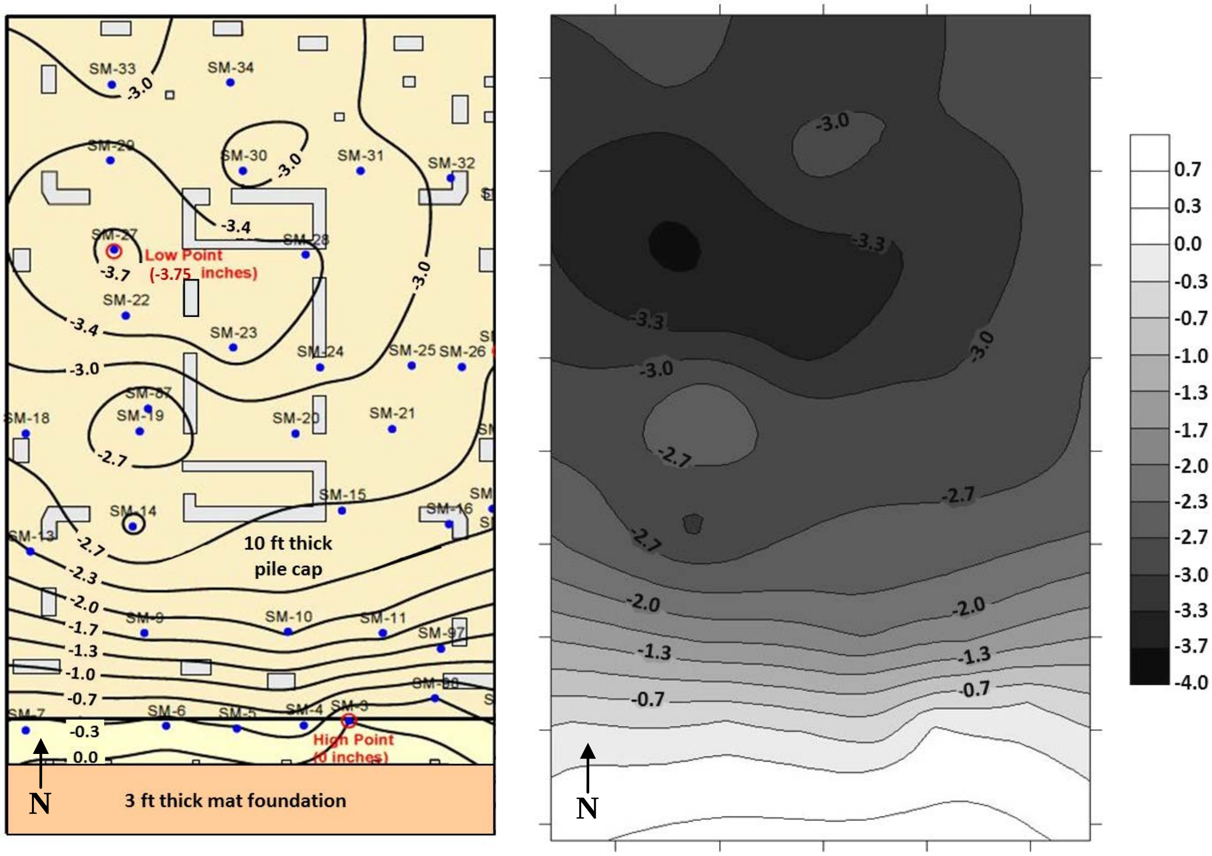


Fig. 2 – (a) Pile cap foundation plan; (b) Typical pile cap cross section and reinforcement details.

2.3 Settlement Measurement

The deformation profile of the pile cap foundation is based on 31 settlement measurement points shown in Fig. 3(b). These measurements were recorded throughout the mat over time, with a maximum estimated differential settlement at the time of this study of 3.75 in. Based on the measurements, a smooth settlement map was created (contours shown in Fig. 3(a) and 3(b)) disregarding the stiffening effect of the superstructure, core, and basement walls. This is evidenced in Fig. 3(b) where smooth contours are shown with no clear indication of the location of core walls, outrigger frames, or moment frames throughout the pile cap foundation. In particular, the high stiffness of core shear walls extending several stories above the foundation system is expected to promote rigid body rotation of individual wall segments connected through coupling beams and significantly alter the smooth settlement contours around the core location. It is important to note that imposing a strict deformed shape on the foundation slab matching settlement contours can lead to an erroneous overestimation of superstructure and foundation stress demands.

Despite the inconsistency of the settlement contours with the rigid body constraints imposed by the superstructure, the foundation's general deformation pattern can still be appreciated from Fig. 3. The 3 ft thick soil-supported mat foundation to the South of the building, which does not directly support the superstructure, experienced very small settlements. The relatively rigid tower core settled nearly uniformly about 3 in with a slight inclination towards the North-West corner of the building. Around the tower core the location of relatively more flexible outrigger and moment frame columns fell between various contour lines indicating possible deformation of frame beams. The closely-spaced contour lines between the mat foundation to the South and the tower core Southern walls indicate a sharp slope and deformation of the pile cap foundation in double curvature.



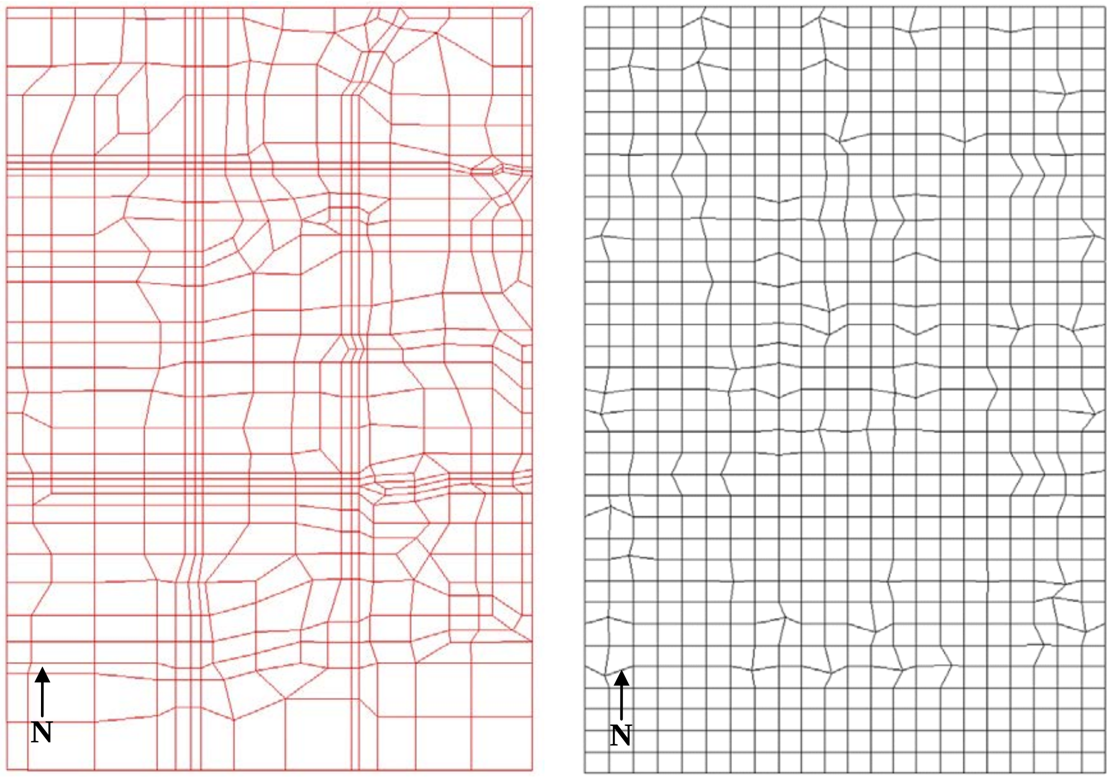
(a) Settlement measurement point (b) Computed contours

Fig. 3 –Settlement measurements and computed contours

3. Foundation Model

3.1 Elastic Shell Model

The initial modeling of the foundation employed elastic shell elements in Perform. The thick pile cap and mat foundation were assumed to be uncracked and unreduced gross section properties were used. This relatively simple model was meshed to accommodate shear wall and column layout, as well as settlement measurement points. Figure 4(a) presents a plan view of this model. As can be seen, mesh size was refined near locations of superstructure vertical elements. Due to Perform modeling limitations, mesh regularity was drastically distorted to match shear wall and column layout as well as the irregular location of the 31 measurement points. Fig. 4(b) presents the nonlinear grillage model used in later analyses. The elastic shell foundation model was used for preliminary assessment of differential settlement effects on superstructure seismic performance. This assessment included preliminary demand-to-capacity ratios and degree of nonlinear action in the superstructure and foundation system due to settlement and ground motion. Although these results may be highly inaccurate due to force redistribution and nonlinearity, this initial modeling stage allowed for quick determination of problem areas and possible seismic deficiencies.



(a) Elastic shell b) Nonlinear grillage

Fig. 4 –Foundation Model Plan Views

In the preliminary study, applied differential settlements consisted of imposed vertical displacement at each pile cap node using linearly interpolated values between mapped settlement contours. The pile cap deformation remained constant throughout the response history analysis. This model did not account for the relative stiffness and nonlinear response of the superstructure and foundation, nor the effect of ground motion excitation on pile cap deformation. Analysis results for the settlement only case were found to be inconsistent with the building condition, predicting severe shear and flexural failures in areas that do not have any observed cracking or distress. Demand-to-capacity ratios were also highly sensitive to very small variations of applied settlements and pile cap meshing.

Given inconsistencies between analysis results and physical observations, a revised analysis was performed in which the differential settlements were applied only at the 31 measurement points. Differential settlements were imposed by defining very stiff support springs at the 31 locations. Support spring element loads were applied as vertical deformations, where a negative sign represents a downward settlement while a positive sign represents an upward displacement. In this analysis, no other soil springs were present. Since only the 31 measurement points were restrained, the rest of the pile cap was free to deform, constrained only by the stiffness of the foundation system and superstructure. Pile cap corners were restrained against lateral displacements. Fig. 5 shows the resulting pile cap deformation. Accuracy of this modeling approach was limited by the essentially rigid supports at the 31 measurement points.

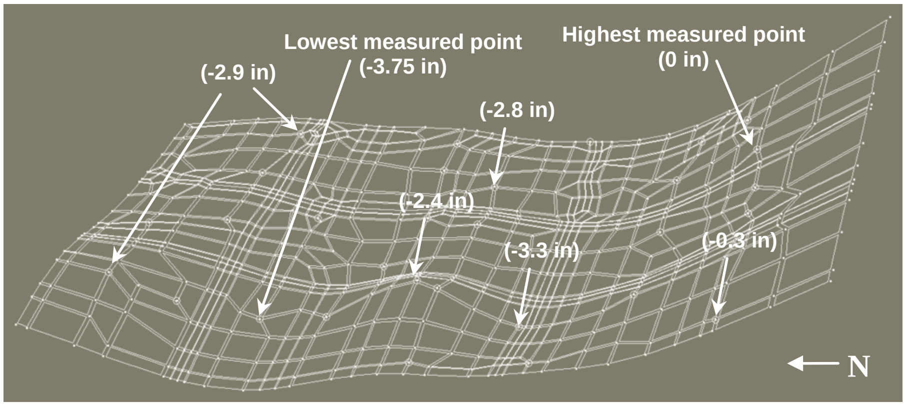


Fig. 5 – Deformed shape of elastic pile cap model (amplification factor=100)

3.2 Elastic Grillage Model

Since the preliminary seismic analysis using the elastic pile cap model indicated probable nonlinear behavior in flexure and shear in certain areas of the foundation, a nonlinear grillage model of the entire foundation system was developed. A relatively regular and orthogonal layout of beam elements representing segments of the pile cap was modeled at an approximate spacing of 5 ft on center in both longitudinal (North-South) and transverse (East-West) directions. This spacing corresponded to the spacing of precast piles. As shown in Fig. 4(b), the beam layout was slightly distorted to match shear wall and column layout, and measurement point locations.

Since grillage beam spacing was approximately 5 ft and the rectangular cross section was also defined with a width of 5 ft, overlap occurs at every grillage intersection. To match the elastic pile cap stiffness from the shell model, the deformed shape of both models was compared at different locations throughout the mat and the concrete elastic modulus adjusted to produce comparable shapes. A similar modeling approach was used to impose different settlements in both models (i.e., simply using stiff support springs and element loads corresponding to scaled vertical displacements at the 31 measurement points). To match the deformed shape throughout the pile cap, as well as the roof lateral displacements a 0.60 multiplier was applied to the concrete elastic modulus ($EI_{eff}=0.6EI_{gross}$). Fig. 6 shows the resulting elastic grillage model deformed shape. Imposed settlements in either model can be equivalently applied as an internal element deformation on stiff support springs or as an external force load on a finite stiffness soil spring. The load demand on each of the 31 nodes were computed as a finite spring stiffness times the desired settlement, i.e., $F_{spring}=K_{spring} \times D_{settlement}$.

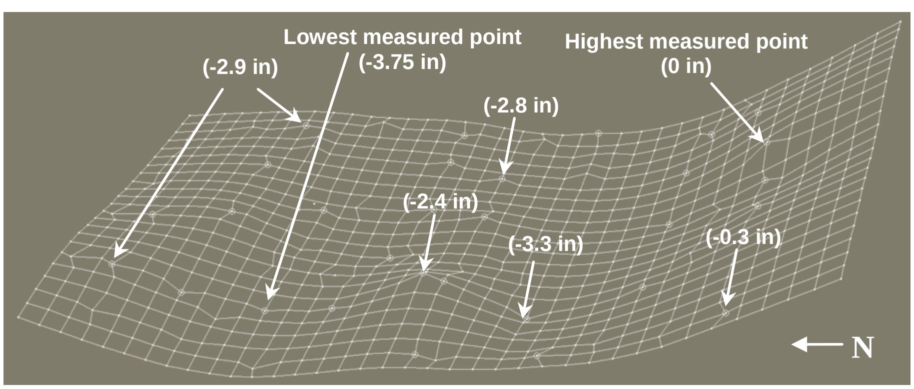
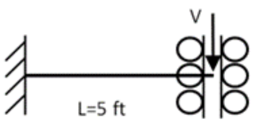
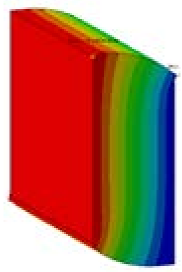
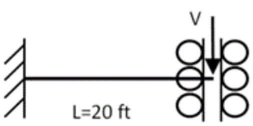
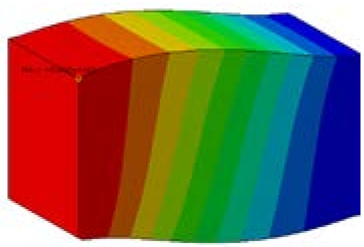
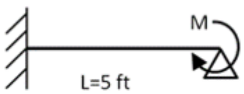
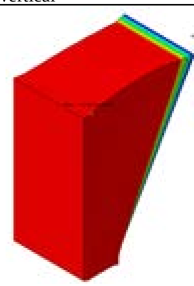
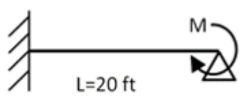
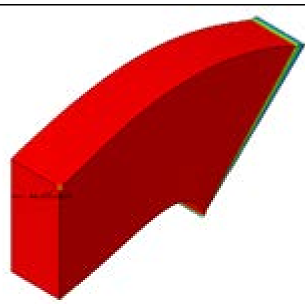


Fig. 6 – Deformed shape of elastic grillage model with $EI_{eff}=0.6EI_{gross}$ (amplification factor=100)

The selected grillage beam spacing resulted in very deep rectangular beams measuring 60 in width (equal to beam spacing at 5 ft on center) by 120 in deep (corresponding to a 10 ft thick slab). The adequacy of this modeling approach was validated by comparing the flexural and shear force-deformation relationship of simple elastic beam models in Perform to thick finite element solid models in Abaqus v6.13.1 (see Table 1).

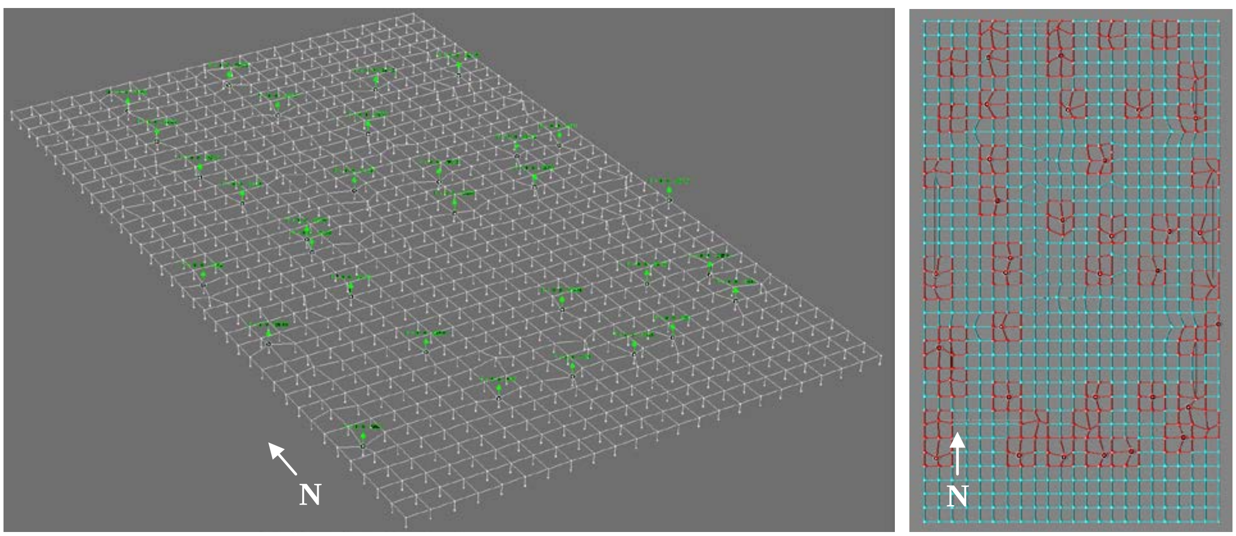
Table 1 – Validation of beam grillage modeling approach

Model Schematics	**Deformation Results**		
	Finite Element Model (Abaqus)	**Beam Model (Perform)**	**Difference**
 V=1200 kip	 $\delta_{vertical}$=1.099e-2 in.	$\delta_{vertical}$=1.18e-2 in.	7%
 V=300 kip	 $\delta_{vertical}$=2.542e-2 in.	$\delta_{vertical}$=2.60e-2 in.	2%
 M=70,000 kip-in	 θ =1.432e-4 rad	θ =1.477e-4 rad	3%
 M=70,000 kip-in	 θ =2.634e-4 rad	θ =2.673e-4 rad	1%

Two beam lengths were assessed: 60 in. (5 ft. beam spacing in orthogonal direction) and 240 in. (4 beams in series arbitrarily selected to represent a longer beam span). In the Abaqus beam/slab model a very fine mesh size of 5x5x5 in. of solid 3D deformable elements with 8 integration points were used. Table 1 shows the loads and boundary conditions used in both Perform and Abaqus models. Comparison of analysis results displays small differences in shear and flexural force-deformation relationships, thus demonstrating the adequacy of this modeling approach.

3.3 Distributed Soil Springs and Applied Settlements

Once the grillage model elastic stiffness was calibrated, distributed compression-only soil springs were added at a spacing of approximately 5 ft. on center representing the vertical stiffness provided by precast piles. The Perform nonlinear Elastic Gap-Hook Bar element was used to model the soil springs where initially a constant spring stiffness was applied throughout the pile cap. The initial spring stiffness was defined as 1800 kip/in corresponding to a high subgrade modulus, $k_{s,initial}$ of 500 pci approximating stiff support conditions [3] provided by the precast piles and a tributary area of 5x5 ft. Settlement loads were applied as external downward vertical forces at the 31 measurement points, as shown in Fig. 7(a). Initially, settlement loads were proportionally scaled to the settlement measurements at the 31 points and then were amplified to match the maximum settlement displacement measured at a single location in the foundation system. The elastic stiffness of beam elements adjacent to the point of load application was increased by a factor of 100 to simulate the force distribution at a 45^o angle from the point of load application that would occur in a thick mat. Fig. 7(b) shows the location of stiffened beams in the 10 ft thick pile cap (highlighted in red). Soil damping effects were not included in the model since significant radiation damping was not expected.



(a) Grillage model with vertical springs (b) rigid beam elements

Fig. 7 – Grillage model with vertical soil springs and stiffened beams (highlighted in red)

The preliminary use of constant soil spring stiffness and an initial set of settlement loads did not result in the desired deformed shape. To approximate this shape it was necessary to iteratively adjust the spring stiffness and applied loads, initially using elastic grillage beam elements, then a second time, after adding nonlinear grillage beam hinges, and finally after adding the basement retaining walls around the pile cap perimeter and an explicit representation of the first floor slab. Throughout these iterations the soil spring values were reduced where high settlements were recorded and increased where measured differential settlements were relatively low. The final soil spring values ranged from 170-1550 kip/in, which correspond to a range of subgrade modulus, k_s of 50-430 pci, with an average $k_{s,base}$ of 200 pci, as shown in Fig. 8. Similarly, the applied settlement loads were scaled to achieve the desired level of deformation. The dead load and expected live load (25% of the unreduced live load) in the entire superstructure were included in these iterations to obtain the final deformed settlement shape. The resulting deformed shape, shown in Fig. 9 matched within 10% accuracy the settlements at the 31 measurement points and approximately matches the settlement contours of Fig. 3.

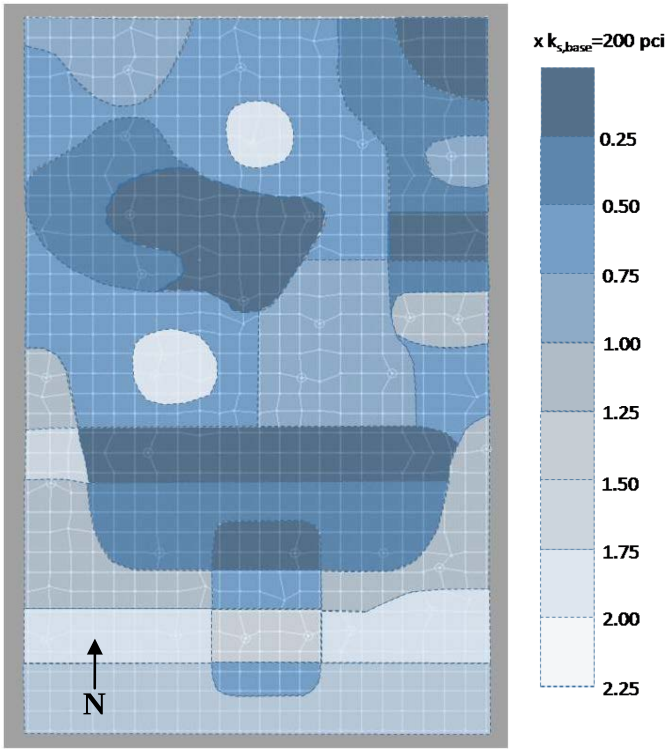


Fig. 8 –Soil subgrade modulus used in the final Perform model.

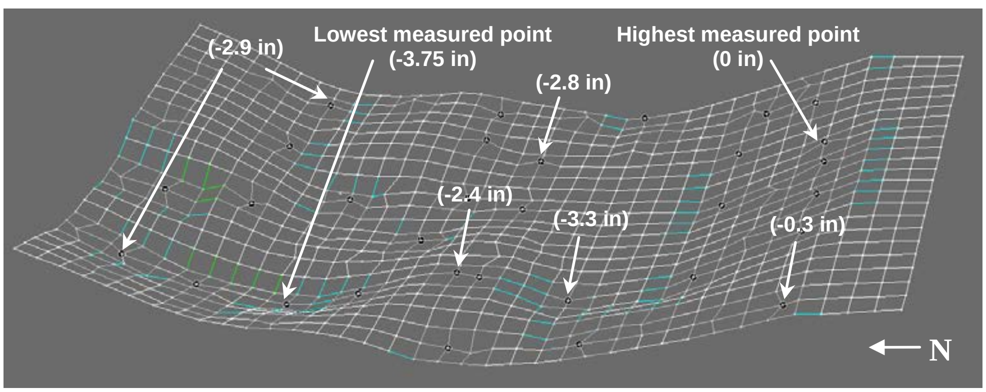


Fig. 9 – Deformed shape of final nonlinear grillage model with distributed soil springs, retaining walls and first floor slab (amplification factor=100).

3.4 Grillage Beam Nonlinear Shear and Flexural Hinges

Each grillage beam was modeled using shear and flexural hinges at each end, defined according to the reinforcement details and calculated capacities using ACI 318-2011 criteria [4]. The *Shear Hinge*, *Displacement Type* was used to define expected nonlinear shear behavior. Nonlinear parameters for beams controlled by shear were defined according to Table 6-7 of ASCE 41-06 Standard [5], assuming the stirrup spacing was greater than d/2 since only vertical bars (not closed stirrups) were used in the current study throughout the pile cap. Expected material properties were used to calculate the shear capacity of the slab (as one-way beam shear) in the area around the core and outriggers (with closely-spaced shear reinforcement), the remaining areas of the 10 ft thick pile cap (with widely-spaced shear reinforcement), and the 3 ft thick mat to the South of the building (where shear reinforcement was not specified).

The *FEMA Beam, Concrete Type* element was used to model nonlinear flexural behavior of the grillage. Nonlinear flexural parameters were defined per Table 6-7 of ASCE 41-06 [5], assuming a relatively symmetrical flexural reinforcement layout and high shear demands. To achieve convergence of the settlement loading and deformed shape, strength degradation was not included in the material model. Instead, the performance of the beams was monitored using a limiting plastic hinge rotation. The positive and negative flexural capacities of the pile cap were calculated (as beam segments measuring 5 ft wide) throughout the floor area according to reinforcement details in Fig. 2 using expected material properties for concrete and reinforcing. Three main areas of the pile cap and mat foundation were identified, i.e., around the core and outriggers (where high flexural reinforcement ratios are used), the remaining areas of the 10 ft thick slab (where intermediate reinforcement ratios are used), and the 3 ft thick mat (with a relatively low reinforcement ratio).

4. Superstructure and Foundation Seismic Evaluation

The superstructure seismic evaluation was carried out in this study using only a fraction of the measured settlements at the foundation level assuming force redistribution, stress relaxation, and cracking are expected to occur during the extended construction process of a high-rise building. Additionally, in the building model dead and live loads are applied as a single step whereas in reality gravity loads are amassed gradually, allowing for force redistribution throughout the structure. Conversely, the foundation system is evaluated using 100% of the measured differential settlements at that level. The seismic assessments of both systems consisted of nonlinear response history analysis of the building model using a suite of 7 pairs of orthogonal horizontal ground motion records. The analyses were carried out with and without initial differential settlements applied to the foundation, to evaluate the effect of settlements on the building seismic performance. Gravity loading consisting of 100% dead and 25% live load was also applied prior to the dynamic analysis and settlement loading, in accordance with the PEER Tall Building Initiative criteria [2]. A maximum of 50 vibration modes was used for the dynamic analysis together `with a constant 3% modal damping for all modes and a negligible Rayleigh damping. P-Delta effects were disregarded since relatively low story drift demands were expected in this core wall building, even with initial differential settlements, an assumption verified through preliminary runs.

Table 2 – Seismic performance criteria defined for superstructure and foundation system

Element	Deformation	Limit (CP)
Building global behavior	Story drift, Δ/h (%)	3.0% average
Core shear wall	Concrete compressive strain, $\varepsilon_{cu,Compr}$	0.005 (in/in)
	Reinforcing steel compressive strain, $\varepsilon_{su,Compr}$	0.02 (in/in)
	Reinforcing steel tensile strain, $\varepsilon_{su,Tens}$	0.05 (in/in)
	Shear strain (drift ratio, Δ/h)	1.0%
Outrigger coupling beams	Shear strain (drift ratio, Δ/h)	6.0%
Outrigger concrete spandrel beams	Plastic hinge rotation, θ_{pl}	4.0%
Reinforced concrete frame beams	Plastic hinge rotation, θ_{pl}	4.0%
Reinforced concrete columns	Plastic hinge rotation, θ_{pl}	0.8-1.8%
Embedded steel coupling beams	Shear hinge displacement, $\Delta=\theta_p L$	θ_p=3.0%
Pile cap foundation	Slab plastic rotation at point of strength loss, θ_{pl}	1.0%
	Slab plastic shear hinge displacement, $\Delta=\theta_p L$	θ_p=0%

Superstructure seismic performance criteria corresponding to Collapse Prevention (CP) level was defined in accordance with PEER's Tall Building Initiative [2] and ASCE 41-06 [5] for different elements and material types (see Table 2 above). For the foundation system, nonlinear flexural and shear performance criteria was taken from the ASCE 41-06 [5] limits for reinforced concrete beams, also shown in Table 2.

Usage ratios (ratios between maximum deformation and limiting deformation criteria) were computed for all 7 ground motion records for each element in the model. The compliance with performance limits (i.e., usage ratios smaller than 1), as well as the percent increase in maximum usage ratios due to differential settlements were determined for all elements. The assessment results are not within this paper's scope. However, this methodology allows identification of requirements for local or global strengthening and seismic retrofit of the high-rise superstructure elements.

5. Conclusions

This paper presents an inventive methodology that was successfully used to assess the effect of large differential settlements on a high rise building's seismic capability. The main difficulty in this assessment was imposing an initial state of settlements on the foundation system while allowing it to deform freely under ground motion excitation of the building. Another obstacle was incorporation of the stiffness and nonlinearity of the superstructure, foundation system, and underlying soil support.

In this study, the common yet extremely limiting use of elastic shell elements to model foundation slabs was replaced with a versatile grillage beam model, which assumptions were validated through finite element analysis. Severe elastic shell element distortion was resolved through use of a regular grillage beam layout defined with nonlinear shear and flexural hinges. The elastic stiffness of the grillage beams was calibrated to match that of a thick mat or pile cap foundation. Nonlinear vertical soil springs with asymmetric compressive and tensile behavior were defined underneath each node of the grillage model, with a preliminary uniform elastic stiffness value throughout the foundation.

Rather than imposing a constant deformed shape on the foundation slab to exactly match incorrect settlement contours that ignore the superstructure stiffness and may therefore result in significant over or underestimation of superstructure and foundation demands, a compound iterative process was developed. Initial differential settlements were applied using vertical loads on select points of the grillage model corresponding to the locations of settlement measurements at the building site. The magnitude of these loads as well as the soil spring stiffness distribution throughout the foundation were determined iteratively to closely match measured settlement values and generally approximate the overall deformed shape of the foundation slab to settlement contours. As the building model was subjected to ground motion simulations using response history analysis, the methodology allowed the mat to deform while accounting for the relative stiffness and nonlinearity of the superstructure and foundation system, as well as soil-structure interaction.

6. References

[1] CSI (2011): User Guide – Perform 3DTM Nonlinear Analysis and Performance Assessment for 3D Structures, Version 5, Computers & Structures, Inc., Berkeley, California, USA.

[2] PEER (2010): Tall Buildings Initiative – Guidelines for Performance-Based Seismic Design of Building Structures, Version 1.0. Technical Report No. 2010/05, Pacific Earthquake Engineering Research Center, Berkeley, California, USA.

[3] Terzaghi, K. (1955): Evaluation of coefficients of subgrade reaction. Géotechnique 4, 297-326.

[4] ACI (2011): Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary, American Concrete Institute Committee 318.

[5] ASCE (2006): Seismic Rehabilitation of Existing Buildings, ASCE/SEI Standard 41-06, American Society of Civil Engineers, Reston, Virginia, USA.