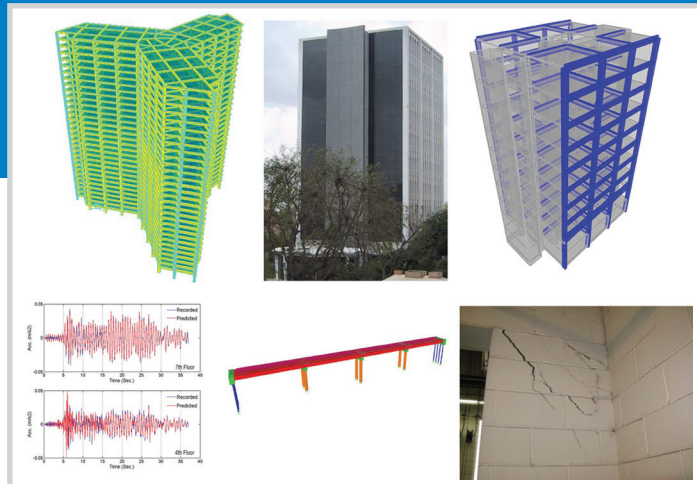


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Design and Performance of Concrete Bridges and Buildings when Interacting with Soils and Foundations

Editors:

Yail J. Kim and Nien-Yin Chang

SP-316



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Preface

Soil-structure interaction has been of interest over several decades; however, many challenging issues remain. Because all structural systems are founded on soil strata, transient and long-term foundation displacements, particularly differential settlement, can severely influence the behavior of structural members in buildings and bridges. This is particularly important when a structure is constructed in earthquake-prone areas or unstable soil regions. Adequate subsurface investigation, design, and construction methods are required to avoid various damage types from structural and architectural perspectives. Typical research approaches include laboratory testing and numerical modeling. The results of on-site examinations are often reported. Recent advances in the state-of-the-art of soil-structure interaction contribute to accomplishing the safe, reliable, and affordable performance of concrete structures. This Special Publication (SP) encompasses nine papers selected from two technical sessions held in the ACI Fall convention at Denver, CO, in Nov. 2015. All manuscripts submitted are reviewed by at least two experts in accordance with the ACI publication policy. The Editors wish to thank all contributing authors and anonymous reviewers for their rigorous efforts. The Editors also gratefully acknowledge Ms. Barbara Coleman at ACI for her knowledgeable guidance.

Yail J. Kim and Nien-Yin Chang
Editors
University of Colorado Denver

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Earthquake-Induced SSI Effects on High Rise Buildings

Nien-Yin Chang and Hien Manh Nghiem

Synopsis: Because of the complexity of the soil-structure interaction (SSI) effect on high-rise buildings, contemporary design codes allow the use of results from an advanced numerical analysis in the design of structures without providing further stipulation. The information on SSI effects, however, is only available for low rise buildings with simple analysis procedures. Two hypothetical 20-story buildings and one 30-story real building were subjected to seismic response analyses using SSI3D under the following conditions: rigid base, flexible base with linear foundation springs, flexible base with linear soil, flexible base with nonlinear springs, and the full SSI analysis with flexible base with nonlinear soils for two hypothetical buildings. For the real building, the calculated natural periods, base shears, and top-floor displacements were compared to the values evaluated using the recorded building motions. It was observed that the natural periods increase and the base shears decrease as the base becomes more flexible, but further study is needed to examine the top-floor side sway.

Keywords: Soil-structure-interaction, pile, models, modal earthquake, stiffness, high-rise building, seismic

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INTRODUCTION

A nonlinear finite element analysis computer code, SSI-3D was developed at the Center for Geotechnical Engineering Science at the University of Colorado Denver to investigate the SSI effect on structures including high rise buildings, bridges, etc.¹. In an analysis a ground motion was imposed along the bedrock-soil interface, propagated into soil, soil-pile system and finally into the building, where beams and columns were modeled by beam elements and slabs and shear walls were modeled by shell elements. Mat foundations, piles and soil media were modeled by solid elements. Nonlinear material models were only applied to solid elements. Pile foundation and soil were represented by linear or nonlinear springs. Modal, response spectrum, and nonlinear time history analyses were performed. Nonlinear soil-pile-building interaction (SPBI) analyses were performed using different constitutive soil models, including Mohr-Coulomb (MC), modified hyperbolic model (MH) and modified Ramberg-Osgood model (MRO). Viscous damping of soils needed for SPBI analyses was determined from the damping ratio of soils using matching transfer functions from one soil layer to another. To verify viscous damping so evaluated, the recorded free-field motion was deconvoluted to find the base rock motion using soil damping and then convoluted using viscous damping to find the corresponding surface motion. Excellent agreement was found between the calculated and recorded free-field motion on ground surface. This validated the soil damping used the analysis. The validated viscous damping of soils was then used in the SPBI analysis and both kinematic and inertial interactions were included with the use of transfer function to convert a free-field motion to a corresponding basement motion, equivalent stiffness, and damping of soil-pile system. For the real building, recorded free-field ground motion was used in the analysis and the results compared to those derived from the recorded strong motion in the strong motion instrumentation program².

RESEARCH SIGNIFICANCE

Safe and cost effective aseismic designs of high rise buildings on piles depend on accurate soil-pile-structure interaction analyses. While rigid base analyses is prevalent, full soil-pile-structure-interaction analyses of buildings have not become a common practice because of its complexity, particularly in mathematical modeling of soil and soil-structure interface behaviors. During strong seismic shaking, a building dances with foundations and soils and produces translational, torsional and rocking motions, and proper building designs require accurate evaluation of the distribution of base shear, axial and torsional forces, torsional and bending moments and building side sways. This needs the adoption of proper constitutive models of soils and soil-foundation contact model, which need further research in development and selection of soils' constitutive models and soil-pile contact models.

SEISMIC RESPONSE OF HYPOTHETICAL BUILDINGS

Description of Two Hypothetical Buildings

The first building is a 20-story reinforced concrete office building 75.6 m [248 ft] tall with rectangular base footprint, shear walls and concrete moment frames, four frames in the X direction and nine frames in the Z direction. The 0.15 m [5.8 in.] thick concrete slab is pre-stressed. The beam section dimensions depend on span length and do not change between floors. The column section dimensions remain the same for several floors and are decreased every few floors per design requirement from the bottom to the top of the building. The shear wall has a uniform thickness of 0.25 m [9.84 in.] throughout the full building height. All stories have the same height of 3.6 m [11.8 ft]. The length of each span in the Z direction is 4.8 m [15.75 ft]; in the X direction, the middle span is 2.4 m [7.87 ft] long and the others are 7.5 m [24.6 ft] long. The building is supported on 126 drilled shafts of 0.8 m [31.5 in.] in diameter, which are connected at top with a 1.5 m [4.92 in.] thick pile cap. The plan, elevation and isotropic views

of the building are given in Fig. 1. The second building is a 20-story office tower 82.2 m [269.69 ft] tall. It is modified from the building analyzed by Krishnan³. This building has a typical floor area of 180 m² [1937.5 ft²], and story height of 4.0 m [13.12 ft]. The lateral systems are steel moment frames along the perimeter of the building and diaphragms are assumed to be rigid. The 0.14 m [5.5 in.] thick concrete slabs rest on metal decks with 0.0254 m [1 in.] topping supported on interior steel beams supported by gravity and moment frame columns. All interior steel beams are assumed to have no contribution to the lateral system and are not considered in the building model. The building is supported by 80 1.0 m-diameter [3.28 ft-diameter] drilled shafts with 1.5 m [4.92 ft] thick pile cap. Plan, elevation and isotropic views of the building are given in Fig. 2.

Site Selection and Soil Properties

The site is located at latitude 34.069° and longitude 118.442° with a soil profile represented by weathered bed-rock underlying 19.8 m [64.96 ft] of surficial clayey silts and sands. Table 1 gives site name, sensor number, site class and peak ground acceleration for parametric studies. For the detailed descriptions of these sites below, refer to Stewart and Stewart (1997). The dynamic properties of soils including shear modulus, shear wave velocity and damping are summarized in Tables 2 and 3 for the subsoil of the site shown in Table 1. The properties of soil model are calculated from curves given by Seed and Idriss⁴ and Sun et. al.⁵. The undrained shear strength of the 4th layer is assumed to be 500 kN/m² [72.5 psi]. The MRO parameters are calculated by using shear strength at middle of soil layers. Properties for equivalent linear analysis of deconvolution are shown in Table 4. Natural site periods are shown in Table 5.

Analysis Assumptions and Procedures

The following assumptions are made in these analyses:

- Beams, columns and slab are elastic in all analyses
- 100 percent of dead load and live load are used in modal and time history analyses
- 5 percent model damping is used to determine the Rayleigh damping of beams, columns, shear walls and slabs.

The analyses of both buildings include: 1) Modal analysis, and 2) Time history analysis. In modal analysis, the periods, mode shapes, modal participation factors are determined. For time history analyses, the Rayleigh damping of beams, columns, slabs, walls and foundations are determined from 5 percent modal damping for all modes by matching two first natural frequencies in the X direction.

Dead Load and Live Load

The dead load and live load for buildings are based on the occupancy of floor according to Table 4-1 of ASCE 7-10. Dead load and live load applied on the floor are assigned on plate elements as vertical uniform loads. The self-weight of structural system is calculated internally using the program. The dead load and live load are input separately as two load cases. Dead load and live load are used to determine the consistent mass of structure.

Time History Functions

In seismic soil-structure interaction analysis, the ground motion is imposed on the bedrock below the pile tip. Thus, the recorded surface ground motion is deconvoluted through the soil layers to establish the associated bedrock motion. Input motion must be located deeper than pile tip or the bedrock-soil boundary. The deconvoluted motions at the bedrock level are shown in Fig. 3.

Foundation Properties

Foundation properties of the two buildings include the stiffness, damping values of pile and pile group that were used in modal, response spectrum, and time history analyses for flexible base cases. The real and imaginary parts of impedance functions of single pile and pile group are calculated by the method presented by Nghiem¹ and shown in Tables 6 to 9. Tables 6 through 8 show the soil model effect is not significant for the same building, however, the initial stiffness (both real and imaginary parts), is significantly different between the concrete and steel structures. The pile arrangements for Buildings 1 and 2 are shown in Figs. 4a and b, with dimensionless pile spacing $S/D=3$. All piles are connected by a 1.5 m [4.92 ft] thick reinforced concrete pile cap. Lateral load-displacement functions of pile head for single piles fixed at the pile top are from stand-alone SSI3D analyses. The soil-single pile system can be modeled by six equivalent springs (three translations and three rotations) at the top of the pile. The equilibrium equation is written as follows:

$$[K]\{U\} = \{P\} \quad (1)$$

where $[K]$ 6 x 6 is the stiffness matrix; $\{U\}$ is the displacement vector; and $\{P\}$ is the load vector. The stiffness matrix, displacement vector and load vector for a pile are given in Eqs. (2) and (3) and shown in Fig. 5.

$$[K] = \begin{bmatrix} K_{11} & 0 & 0 & 0 & K_{15} & 0 \\ 0 & K_{22} & 0 & K_{24} & 0 & 0 \\ 0 & 0 & K_{33} & 0 & 0 & 0 \\ 0 & K_{42} & 0 & K_{44} & 0 & 0 \\ K_{51} & 0 & 0 & 0 & K_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & K_{66} \end{bmatrix} \quad (2)$$

$$\begin{aligned} \{U\} &= [u_x \quad u_y \quad u_z \quad \theta_x \quad \theta_y \quad \theta_z] \\ \{P\} &= [P_x \quad P_y \quad P_z \quad M_x \quad M_y \quad M_z] \end{aligned} \quad (3)$$

where K_{11} , K_{22} , and K_{33} are the translational stiffness components; u_x , u_y , u_z are the translations; and P_x , P_y , P_z are the loads along the axis x, y, and z, respectively; K_{44} , K_{55} , and K_{66} are the rotational stiffness components, θ_x , θ_y , θ_z are the rotations and M_x , M_y , M_z are the moments about axis x, y, and z, respectively; and $K_{51} = K_{15}$ and $K_{24} = K_{42}$ are coupling stiffness. The coupling effects between vertical stiffness, torsional stiffness and others, and between stiffness in the directions x and y are ignored. Single piles are symmetric in the x-z and y-z planes in their local coordinates and soil and pile isotropic in the x and y directions. Under these conditions, some stiffness components are equal, $K_{11} = K_{22}$ and $K_{15} = K_{51} = K_{24} = K_{42}$.

Kinematic Analyses

Kinematic interaction results from the occurrence of stiff foundation elements on the surface of or embedded in a soil deposit causes foundation deformations to differ from those in free field. The kinematic interaction will occur whenever the stiffness of foundation obstructs development of the free-field motion. Kinematic analyses need to be performed to determine ground floor motions from far field motions.

Modal Analyses

In modal analyses, a total of 12 modes are calculated for fixed base, full soil-pile-building interaction (SPBI) and spring model of soil-foundation system. The natural periods and directions of vibrations of the first three modes are given in Tables 10 and 11. The SSI effects are represented by the comparison between the flexible base analysis and fixed base analysis. It was found the influence on natural period is not significant, and the natural period for Building 1 in the flexible base analysis is about 5 percent higher than that in the fixed base analysis, for Building 2 it is 2 percent higher, and the difference is affected by the stiffness of the soil-pile system. The natural periods computed using the spring model of soil-pile system are in good agreement to those of soil-structure interaction system with 1 percent maximum differences for first three modes. This shows that the initial spring stiffness given in Tables 6 and 7 are good approximations for stiffness of soil-pile system.

Time History Analyses

Time history analyses are performed for full soil-pile-structure interaction and simple model of soil-pile system. In analyses, the free field motion duration is shortened to 25 seconds and used in calculating the corresponding bedrock motions through deconvolution and then used in the time history analyses. For the full soil-pile-structure interaction analyses, three soil models: Mohr-Coulomb (MC), modified hyperbolic (MH) and modified Ramberg-Osgood (MRO) are used to evaluate the effect of soil models. The story shears for full soil-pile models are shown in Figs. 6 for Building 1 and Fig. 7 for Building 2, the maximum base shears and the total displacements at the 20th floor level are given in Table 12. There are no significant differences between results of analyses using the three soil models in analyses in terms of story shear as shown in Figs. 6 and 7. The total displacements in full soil-pile model analyses are compared to those from fixed-base analyses in Table 12. For the flexible base using linear and nonlinear springs,

the ground floor motions used in the analyses are determined from far field motion by using kinematic analyses. The comparisons of maximum base shear and total displacement at 20th floor between flexible base using linear and nonlinear spring models and fixed base model are also given in Tables 12. It can be observed that the maximum base shears do not differ significantly between linear and nonlinear spring cases. The percentages of base shear reduction in the Z direction are higher than those in X direction for Building 1 and also for Building 2. The total displacements of flexible base models change inconsistently when compared to those of fixed base models. For Building 1, total displacements increase in the Z direction and decrease in other cases. For all cases of analyses, the story drifts are detected less than the allowable story drift defined in design code as 0.054 m [2.13 in.] for Building 1 and 0.06 m [2.36 in.] for Building 2.

SEISMIC RESPONSE OF ACTUAL 30-STORY BUILDING

Description of 30-Story Building

The thirty story building is a reinforced concrete structure on soft soil deposit containing 583 condominium units. The building has a Y-shaped configuration with three wings and one axis of symmetry as shown in Fig. 8. The central core area houses elevators and staircases are located in each wing. The structure of the thirty story reinforced concrete building is ductile moment-resistant concrete frames. Reinforced shear walls are located in the central core and in each wing from foundation to second floor. The building has 60 columns with six in the core area and 18 in each wing. The building is supported by 828 square prestressed concrete piles of 0.36 m [14.1 in.] x 0.36 m [14.1 in.] penetrated 30.5 m [100 ft] into Site 1 subsoils. The piles are connected by a 1.524 m [60 in.] thick concrete mat. The site period varies from 0.5 seconds to 1.5 seconds from estimates and seems to be larger than the 1.0 second obtained from the data recorded during the Loma Prieta earthquake. Two types of concrete were used: for the columns of the 1st to 5th stories and the interior columns of the 20th and 21st stories, the design compressive strength of the concrete was 450 kg/cm² (6500 psi), and it is 346 kg/cm² (5000 psi) for all other columns (Anderson et al., 1991). Grade 60 reinforcement (4200 kg/cm² or 60 ksi strength) was used in all reinforced concrete components.

Instrumentation

The building was instrumented with 21 CRA-analog acceleration sensors located in all three wings and the central core on the ground, 13th, 21st, and 31st floors. Six additional components of CRA-1 were installed to measure the far field motions in which three were 40 m (131 ft) north of the building and three were 100 m (328 ft) south of the building. The measurement of the vertical component in the building was only at ground floor, so it does not allow the evaluation of the effect of vertical motion. The locations of sensors are shown in Fig. 9.

Analysis Assumptions

The following assumptions were made in the 3-D modeling and analyses of the 30-story building:

- Beam, column, and slab are elastic,
- 100 percent mass is used in modal, response spectrum and time history analyses,
- Soil-pile system is modeled by spring and dashpot elements that can represent both linear and nonlinear viscous behaviors.

Both modal and time history analyses were performed to determine the dynamic behaviors of the building and the effect of soil structure interaction.

Foundation Properties

Based on the properties of soils² and the properties of the building pile foundation, the equivalent impedance function of single piles were calculated by method presented by Nghiem¹ and results given in Table 13. While the pile geometric arrangement is not provided in the literature, it was assumed that the pile spacing of 2.5 x D (diameter of piles) and were arranged in 3x5 pile group for the foundation in each wing and 3x3 pile groups for the foundation in center core, as shown in Fig. 10. The pile size is 0.35 m [13.78 in.] and pile length is 30.5 m [100 ft].

Modal Analyses

In modal analyses, nine modes of vibration were computed including three modes each in the X and Z directions, respectively, and three torsional modes. The fixed and flexible base periods are shown in Table 14 with the periods determined from earthquake responses⁶ and ambient tests⁷. The ambient tests give the smallest periods caused by small displacements of building at the time of ambient test. It is believed that the ambient test shows that the building system is much stiffer than the analyses show. The periods of building are possibly calculated from the

Loma Prieta earthquake when the stiffness of building was reduced by some microcracking in the concrete members⁶. As can be seen in Table 14, the periods in first mode increase from 1.77 s to 2.69 s. The periods from modal analyses are in good agreement with those from the recorded earthquake response. The stiffness of the real building in the X direction (E-W) is slightly higher than stiffness in the Z (N-S) direction, while the model stiffness in both directions is almost the same. The mode shapes of vibrations are shown in Fig. 11 for first three modes. The translation mode in the Z (N-S) direction in Fig. 11a is not pure translation because of the geometric asymmetry of and mass distribution by the mezzanine slab in the west wing. Second mode is torsional as shown in Fig. 11b. Due to the symmetry in the X (E-W) direction, the mode shape of third mode is almost pure translation, which shown in Fig. 11c. The slight increase in natural periods of flexible base model of the foundation system modeled by spring elements over the fixed base model indicates that the soil-structure interaction is not significant in linear analysis.

Time History Analyses

The structural responses were evaluated from four time history analyses: (1) use the ground motion recorded on the ground floor as input (Fig. 12) with fixed base structure model, (2) use the far field motion with fixed base structure model, (3) use the acceleration calculated from far field acceleration using the kinematic interaction analysis with linear flexible base structure model, and (4) the same as analysis number 3, but with nonlinear flexible base structure model. In these analyses, the accelerations and equivalent forces in two orthogonal directions were applied simultaneously. Mode superposition method was used with damping considered as modal damping in the fixed base model and Rayleigh damping in flexible base model. The modal damping for structure is assumed as 5 percent in all modes to obtain better matches with the measured behavior⁶. This indicates that the damping used in the time history analyses is higher than that obtained from the earthquake response with 3 percent of damping. The Rayleigh damping of structure in soil structure interaction analyses given in previous section gives the best match to 5 percent of damping. The results of displacement in the central core are compared with the measured data in Figs. 13 and 14. Good agreement is observed between the calculated and recorded values with the maximum displacements of 0.124 m [4.88 in.] and 0.104 m [4.09 in.], respectively in X (E-W) direction in Fig. 13. Similar data in the Z (N-S) direction in Fig. 14 shows better agreement with the calculated displacements of 0.0445 m [1.75 in.] and the recorded displacement of 0.0517 m [2.04 in.]. The input motion for the flexible base model is determined from kinematic interaction analysis and considered as base motion. The comparisons of maximum displacements for different cases of analyses are given in Table 15. The calculated displacements in the X direction are not significantly different in all cases with average of 0.261 m [10.28 in.], and their difference compared with the measured displacement is 0.042 m [1.65 in.]. The displacement in the X direction in the fixed base analyses using far field motion over predicts the measured displacement by 0.045 m [1.77 in.]. The comparison of the base shears is given in Table 16. Significant reduction of base shear is observed in X direction in comparison to the flexible base case with fixed base using far field motion.

CONCLUSION

Major findings from this study are summarized as follows:

- The time domain analysis allowing full soil-pile-structure interaction must use equivalent viscous damping rather than traditional soil damping ratio. The measured ground motion is de-convoluted to any depth below pile tip in frequency domain analysis using the damping ratio of soil to obtain the input motion for the full soil-pile-structure interaction analysis. The equivalent viscous damping of soil in time domain analysis can be evaluated by matching the transfer functions, soil damping ratio and soil damping value in frequency domains.
- Numerous analyses of hypothetical buildings and an actual building were performed, and the natural periods, base shears, and the top floor displacements were compared. It was observed that the natural period increases and the base shear decreases as the base becomes more flexible by the inclusion of SSI effects.
- The inertial and kinematic effects can be combined by using equivalent springs and dashpots to model the soil and pile foundation system. The analysis of a structure using an equivalent spring and dashpot model to represent the soil-pile and foundation system yields good agreement with the analysis using the full soil structure interaction model.
- Load-displacement curves of single pile and pile group can be calculated from stand-alone analyses using geotechnical computer codes or from static load tests.

- In case of critical high rise buildings, it is advised to perform a realistic full SSI dynamic analysis to assess their performance under strong seismic shaking with SSI effects and under different base stiffness from multiple credible earthquakes.
- Analyses of the hypothetical buildings for full SSI effects showed 1) base shear is higher in rigid base analysis, 2) effect on 20th floor side sway is not consistent and not significant, and 3) base shear is much higher for the reinforced concrete Building 1 than for the steel Building 2.

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Table 1—Site locations and time history properties

Site Name	Sensors	Peak Acceleration (g)
LA 7 Story UCLA Math/Sci. Building	CSMIP 24231/24688	NS 0.47 EW 0.28 V 0.27

Table 2—Soil properties

Layer	Depth m (ft)	V_s m/s (ft/s)	S_u kN/m ² (psi)	γ kN/m ³ (pcf)	G kN/m ² (psi)	r	$\bar{\alpha}$
1	0-3 [0-9.84]	152.4 [500]	72 [10.44]	19.0 [120.9]	45057 [6534.97]	2.5	4.5
2	3-9.1 [9.84-29.86]	243.8 [800]	132 [19.14]	17.9 [113.9]	108456 [15730.21]	2.3	1.9
3	9.1-19.8 [29.86-64.96]	365.8 [1200]	223 [32.34]	18.6 [118.4]	253706 [36796.95]	2.2	1.0
4	19.8-29.8 [64.96-97.77]	609.6 [2000]	500 [72.52]	19.0 [120.9]	720902 [104558]	1.5	0.35

Table 3—Parameters for damping ratio

Depth (m)	$\xi_{\max}$ (%)	$\xi_{\min}$ (%)	γ_r (%)	a
0-3	21.0	1.30	0.17	0.75
3-9.1	21.0	1.30	0.17	0.75
9.1-19.8	15.0	1.30	0.20	0.90
19.8-29.8	5.1	0.44	0.07	0.60

Table 4—Equivalent linear soil parameters

Layer	Depth m (ft)	G kN/m ² (psi)	ξ (%)	η kN.s/m ² (lb.s/in ²)	f (Hz)	Rayleigh damping, β (s)
1	0-3 [0-9.84]	38391 [5568.15]	4.7	48.9 [7.09]	11.8	0.00127
2	3-9.1 [9.84-29.86]	91982 [13340.89]	4.9	247.4 [35.88]	5.8	0.00269
3	9.1-19.8 [29.86-64.96]	221974 [32194.67]	2.8	511.2 [74.14]	3.9	0.0023
4	19.8-29.8 [64.96-97.77]	641772 [93081.33]	1.53	913.9 [132.55]	3.4	0.0014

V_s = shear wave velocity; S_u = undrained shear strength; γ = soil unit weight; G = shear modulus; r , $\bar{\alpha}$, γ_r , and a = parameters of Ramberg-Osgood models¹; $\xi_{\max}$ and $\xi_{\min}$ = maximum and minimum soil damping factors, respectively; and η = equivalent viscous damping;

Table 5—Natural site periods (s)

Mode 1	Mode 2	Mode 3
0.285570	0.120135	0.073104

Table 6—Initial stiffness of single piles for Building 1

Site	Torsional Stiffness, K_{66} kN.m/rad (kip-in./rad)	Vertical Stiffness, K_{33} kN/m (kips/in.)	Lateral Stiffness, K_{11} kN/m (kips/in.)	Rotation Stiffness, K_{55} kN.m/rad (kip-in./rad)	Coupled Stiffness, K_{15} kN/rad (kips/rad)
2a	207744 [1801519.3]	1380640 [7724.29]	417915 [2338]	628754 [5452444]	362200 [79779.7]
2b	192962 [1673332.4]	1301671 [7282.48]	369949 [2069.76]	606247 [5257267]	335100 [73810.6]

Table 7—Initial stiffness of single piles for Building 2

Case	Torsional Stiffness, K_{66} kN.m/rad (kip-in./rad)	Vertical Stiffness, K_{33} kN/m (kips/in.)	Lateral Stiffness, K_{11} kN/m (kips/in.)	Rotation Stiffness, K_{55} kN.m/rad (kip-in./rad)	Coupled Stiffness, K_{15} kN/rad (kips/rad)
a	413049 [3581888]	1857184 [10390.4]	523436 [2928.5]	1256649 [10897438]	578194 [127355.5]
b	384694 [3335998.5]	1751932 [9801.6]	465014 [2601.6]	1213988 [10527490]	537103 [118304.6]

Table 8—Imaginary parts of impedance functions of single piles for Building 1

Torsional Stiffness kN.m/rad (kip-in./rad)	Vertical Stiffness kN/m (kips /in.)	Lateral Stiffness kN/m (kips /in.)	Rotation Stiffness kN.m/rad (kip-in./rad)	Coupled Stiffness kN/rad (kips/rad)
8406 [72895.3]	43240 [241.9]	32150 [179.9]	15937 [138202.9]	18675 [4113.4]

Table 9—Imaginary parts of impedance functions of single piles for Building 2

Torsional Stiffness kN.m/rad (kip-in./rad)	Vertical Stiffness kN/m (kips /in.)	Lateral Stiffness kN/m (kips /in.)	Rotation Stiffness kN.m/rad (kip-in./rad)	Coupled Stiffness kN/rad (kips/rad)
16183 [140336.1]	56947 [318.6]	39172 [219.2]	30247 [262296.6]	28345 [6243.4]

Note: Case a for MH and MRO models; cases b for MC model.

Table 10—Natural periods of Building 1 (s)

Direction	Fixed base	Flexible base	MC model	MH and MRO models
X	2.773231	2.87531	2.921124	2.915363
Torsion	2.037941	2.052666	2.057998	2.056809
Z	1.781740	1.945561	2.024808	2.018142

Table 11—Natural periods of Building 2 (s)

Direction	Fixed base	Flexible base	MC model	MH and MRO models
X	4.220462	4.281107	4.347977	4.341472
Z	2.970607	3.005187	3.041860	3.038459
Torsion	1.767567	2.189665	1.812939	1.810574

Table 12—Maximum base shears and displacements

Building	Case	Base shear in X Direction kN (kips)	Base shear in Z Direction kN (kips)	Displacement in X Direction m (in.)	Displacement in Z Direction m (in.)
1	MC model	12379 [2726.6]	15887 [3499.3]	0.219 [8.622]	0.147 [5.787]
	MH model	10785 [2375.6]	14983 [3303.2]	0.219 [8.622]	0.145 [5.709]
	MRO model	11050 [2433.9]	14736 [3245.8]	0.219 [8.622]	0.145 [5.709]
2	MC model	4862 [1070.9]	6164 [1357.7]	0.310 [12.204]	0.224 [8.819]
	MH model	4688 [1032.6]	5917 [1303.3]	0.308 [12.126]	0.223 [8.780]
	MRO model	4829 [1063.6]	5920 [1303.9]	0.308 [12.126]	0.223 [8.780]
1	Fixed base	13599 [2995.4]	18322 [4035.7]	0.318 [12.520]	0.142 [5.591]
	Linear spring	12395 [2730.2]	15279 [3365.4]	0.210 [8.268]	0.153 [6.024]
	Nonlinear spring	12380 [2726.9]	15269 [3363.2]	0.210 [8.268]	0.153 [6.024]
2	Fixed base	5462 [1203.1]	6150 [1354.6]	0.349 [13.740]	0.220 [8.661]
	Linear spring	5201 [1145.6]	5738 [1263.9]	0.315 [12.402]	0.219 [8.622]
	Nonlinear spring	5195 [1144.3]	5733 [1262.8]	0.315 [12.402]	0.219 [8.622]

Table 13—Initial impedance function of single piles

Component	Torsional Stiffness, K_{66} kN.m/rad (kip-in./rad)	Vertical Stiffness, K_{33} kN/m (kips/in.)	Lateral Stiffness, K_{11} kN/m (kips/in.)	Rotation Stiffness, K_{55} kN.m/rad (kip-in./rad)	Coupled Stiffness, K_{15} kN/rad (kips/rad)
Real part	42996 [372853.7]	572543 [3203.2]	473030 [2646.5]	101328 [878698.5]	154805 [34098]
Imaginary part	1396 [12105.9]	28618 [160.1]	26502 [148.3]	1902 [16493.8]	5799 [1277.3]

Table 14—Natural periods (s)

Mode	Direction	Fixed base	Flexible base	Recorded	Ambient test
1	Z	2.564	2.572	2.59	1.71
2	Torsion	2.557	2.566	-	-
3	X	2.553	2.560	2.69	-

Table 15—Maximum displacements on the roof

Case	X Direction m (in.)	Differences from measurement (%)	Z Direction m (in.)	Differences from measurement (%)
Measurement	0.219 [8.622]	0.0	0.065 [2.559]	0.0
Fixed base (Ground floor motion)	0.259 [10.197]	15.4	0.085 [3.346]	23.5
Fixed base (Far field motion)	0.264 [10.394]	17.0	0.146 [5.748]	55.5
Linear spring (Base motion)	0.261 [10.276]	16.1	0.101 [3.976]	35.6
Nonlinear spring (Base motion)	0.261 [10.276]	16.1	0.097 [3.819]	32.9

Table 16—Maximum base shears

Case	Base Shear in X Direction kN (kips)	Base Shear in Z Direction kN (kips)
Fixed base (Ground floor motion)	57449 [12653.96]	26859 [5916.08]
Fixed base (Far field motion)	60543 [13335.46]	29700 [6541.85]
Linear spring (Base motion)	56355 [12413]	29005 [6388.77]
Nonlinear spring (Base motion)	54033 [11901.54]	29124 [6414.98]

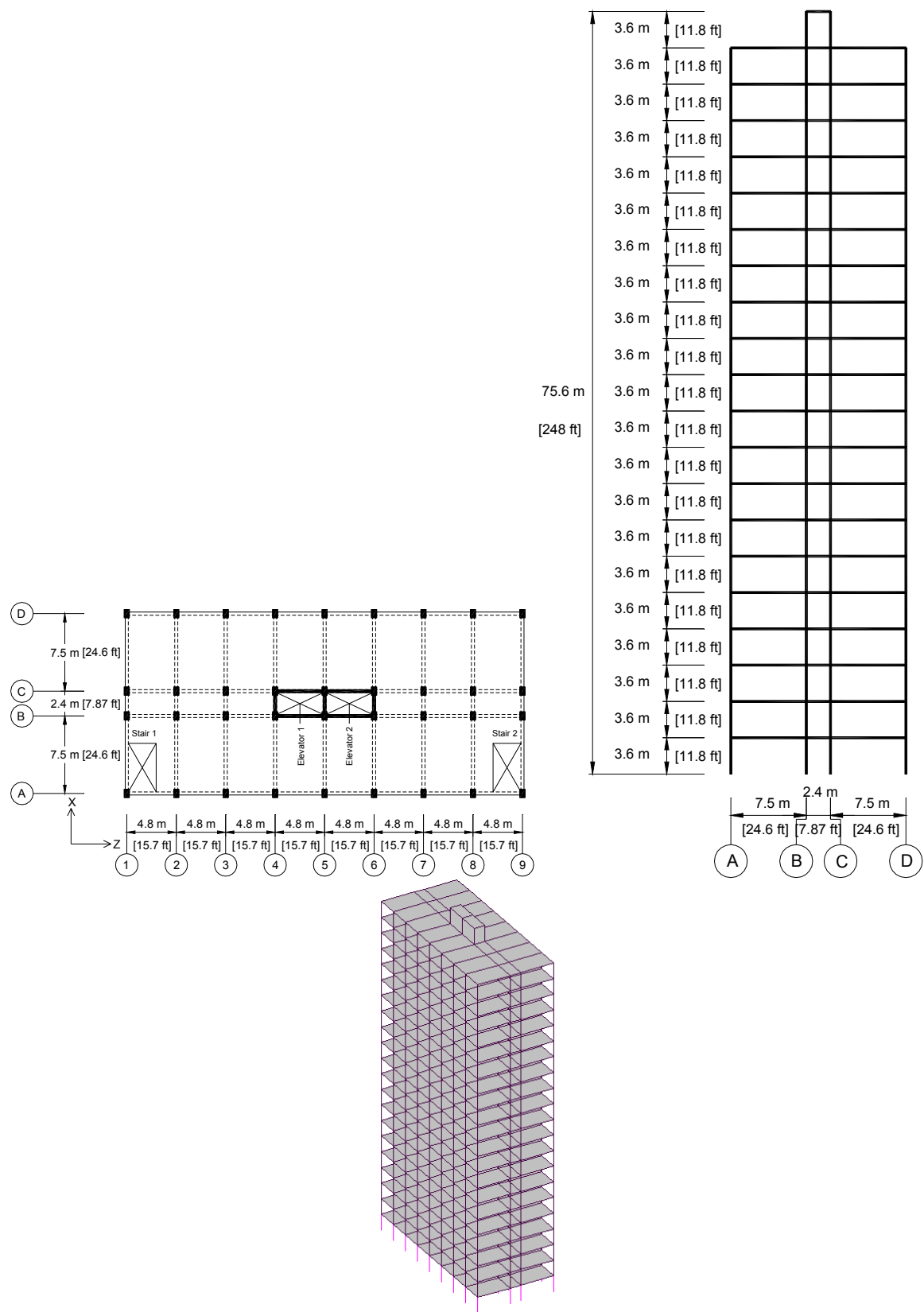


Figure 1—Plan view, Frame "1" and isotropic view of Building 1

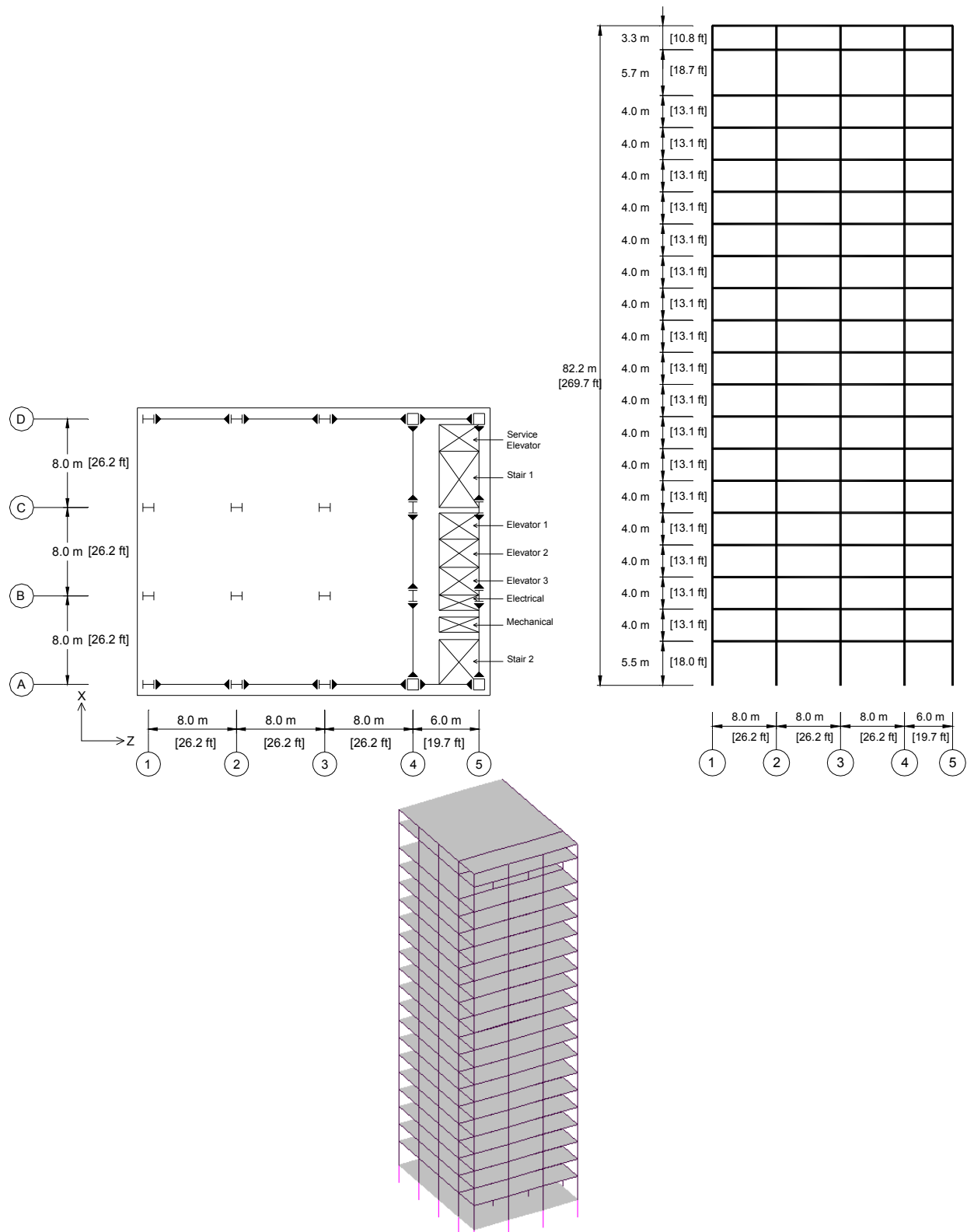
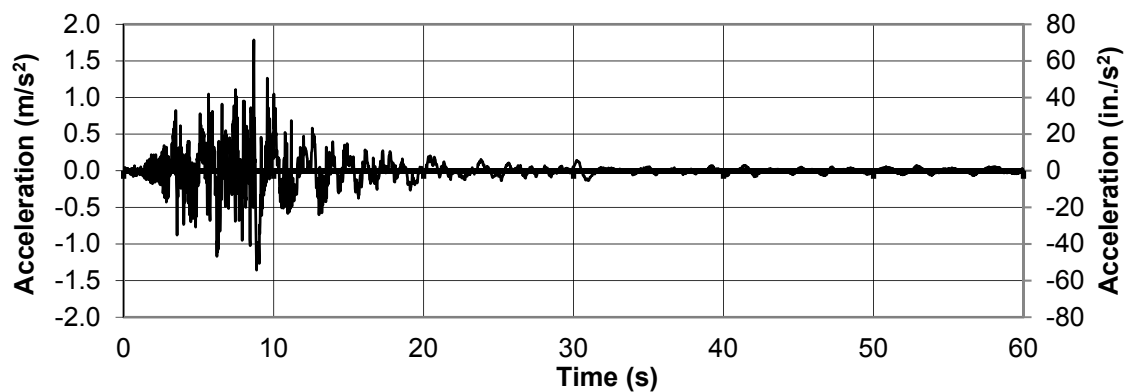
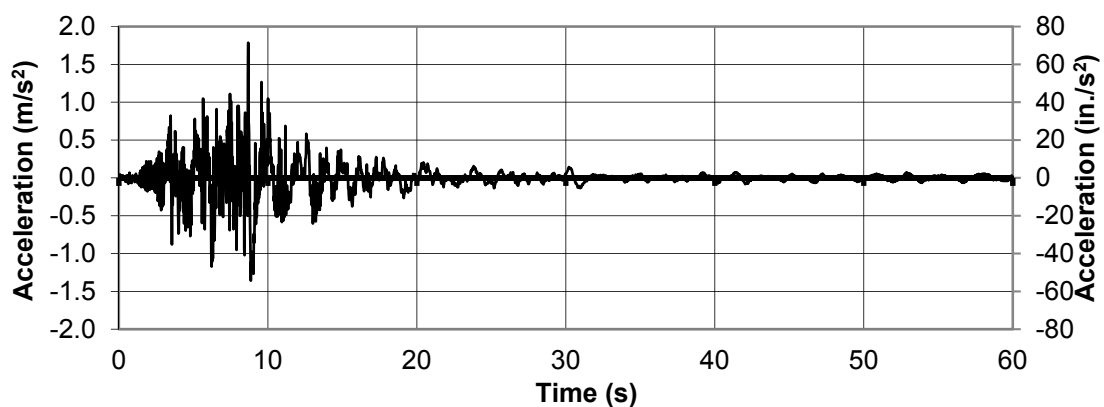


Figure 2—Plan view, frame "A" and isotropic view of Building 2



(a)



(b)

Figure 3—Bedrock motions: (a) in the X direction; (b) in the Z direction

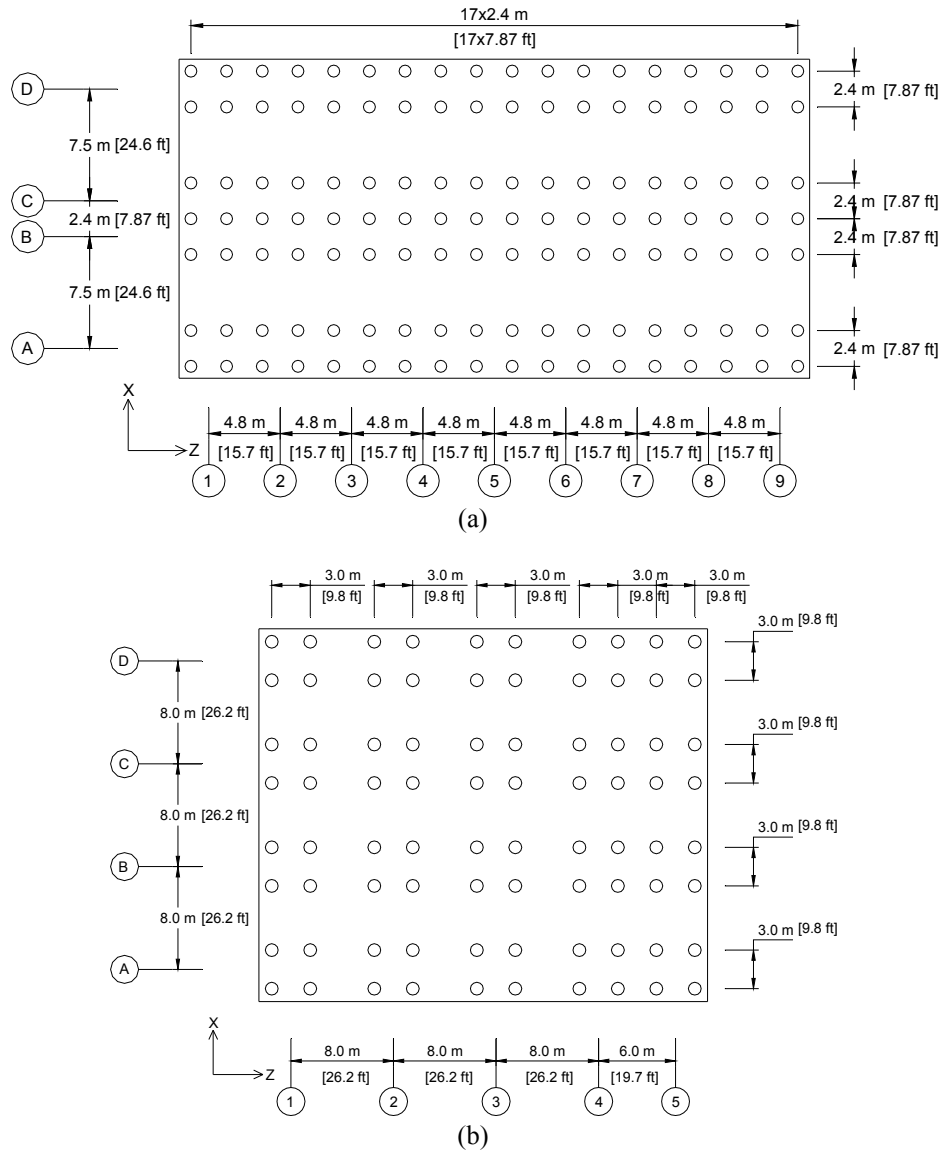


Figure 4—Pile foundation arrangement: (a) Building 1; (b) Building 2

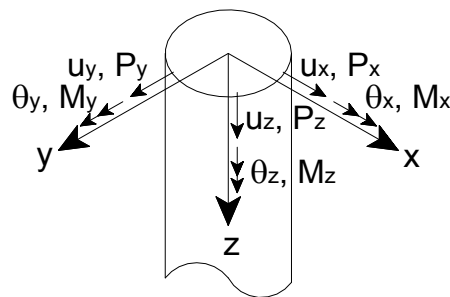


Figure 5—Displacement and load components in local coordinate system of a single pile

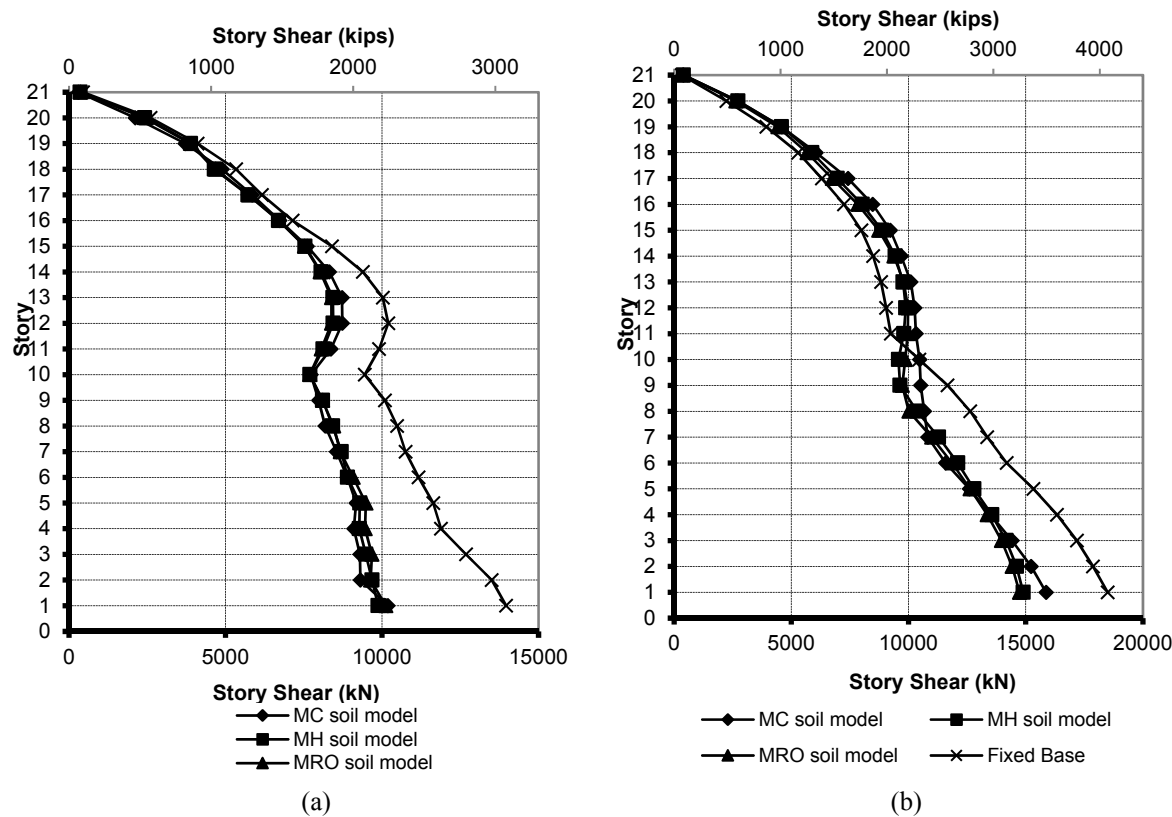


Figure 6—Maximum story shear (Building 1): (a) in the X Direction; (b) in the Z direction

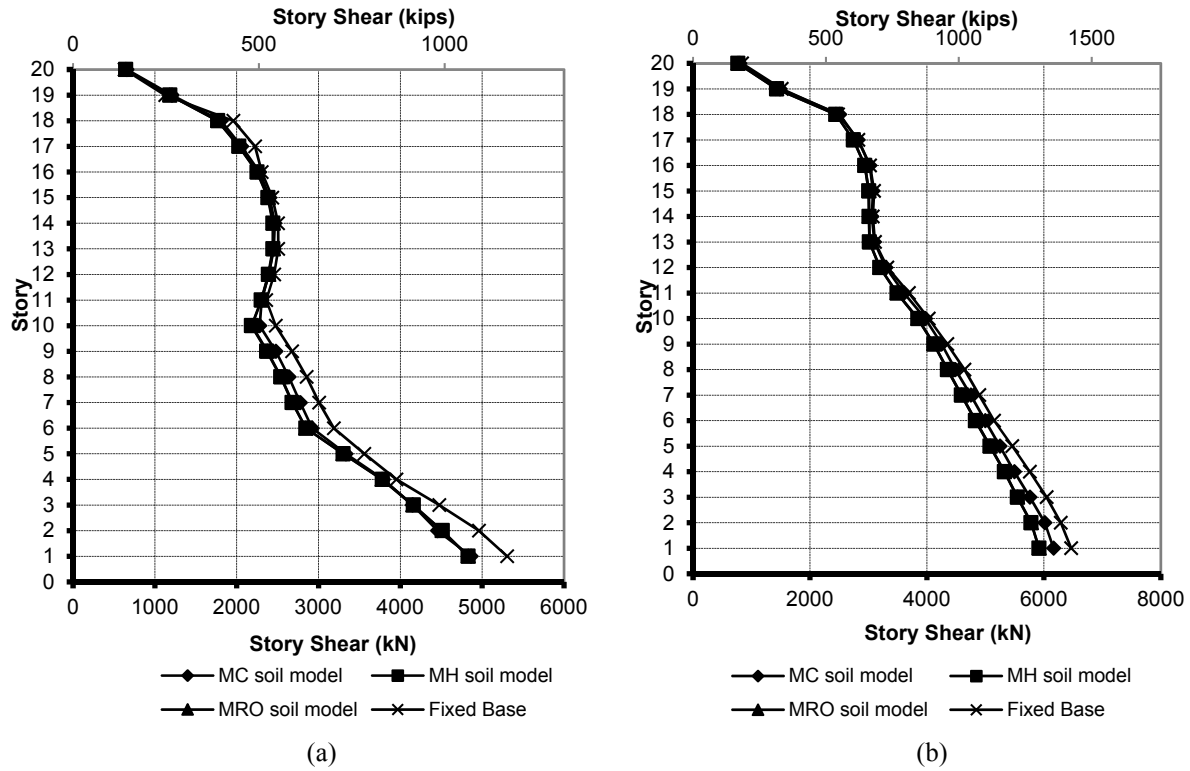


Figure 7—Maximum Story Shear (Building 2): (a) in the X direction; (b) in the Z Direction

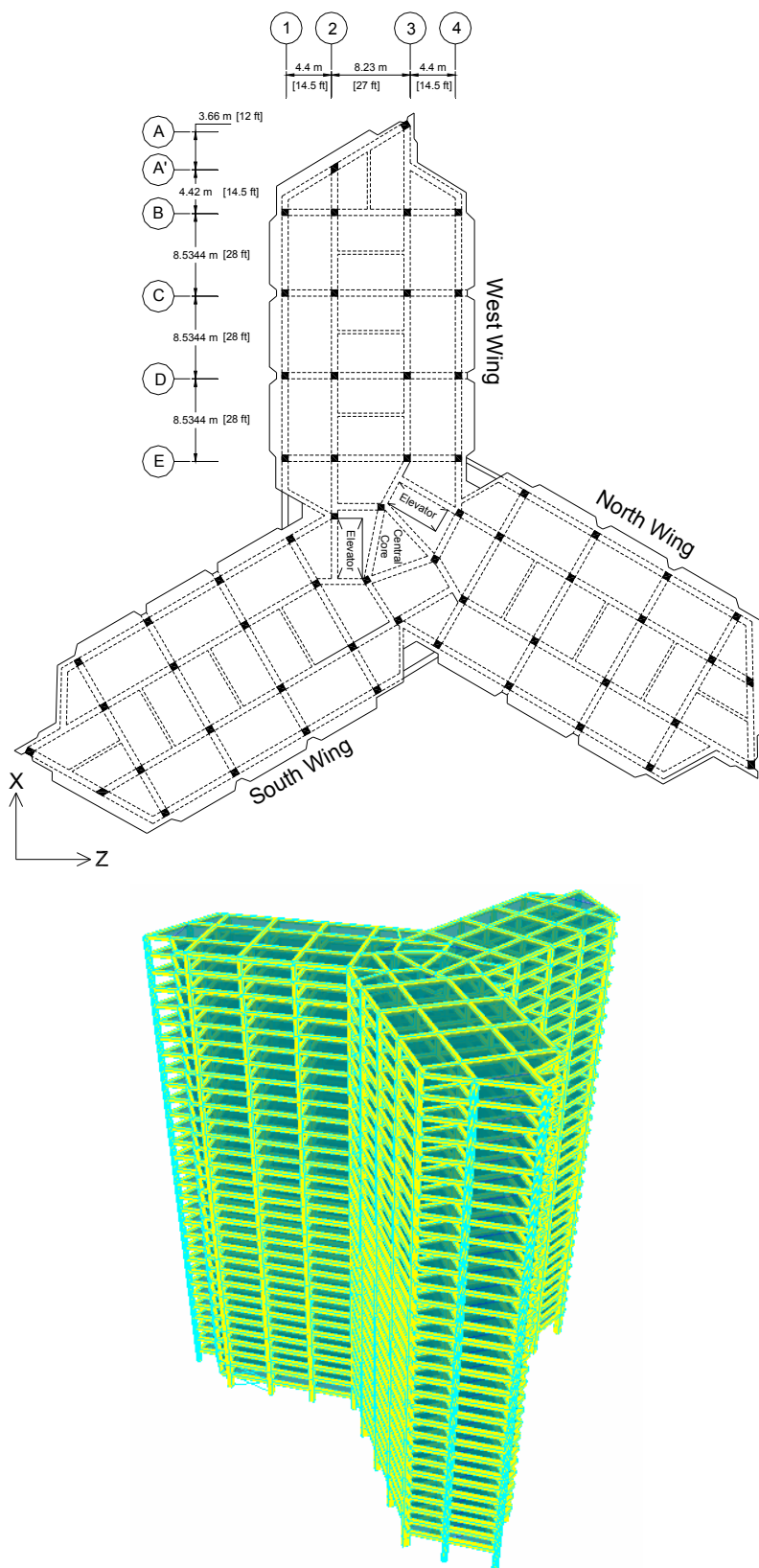


Figure 8—Plan view and isotropic view of 30-story building

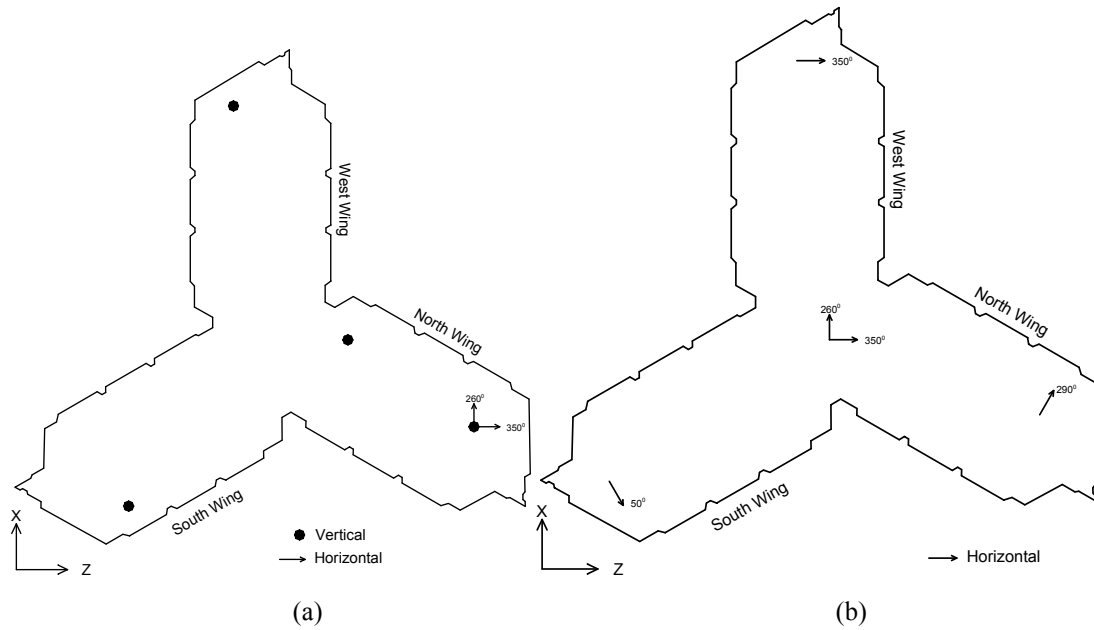


Figure 9—Instrument locations in 30-story building: (a) ground floor; (b) 13th, 21st, and 31st Floor

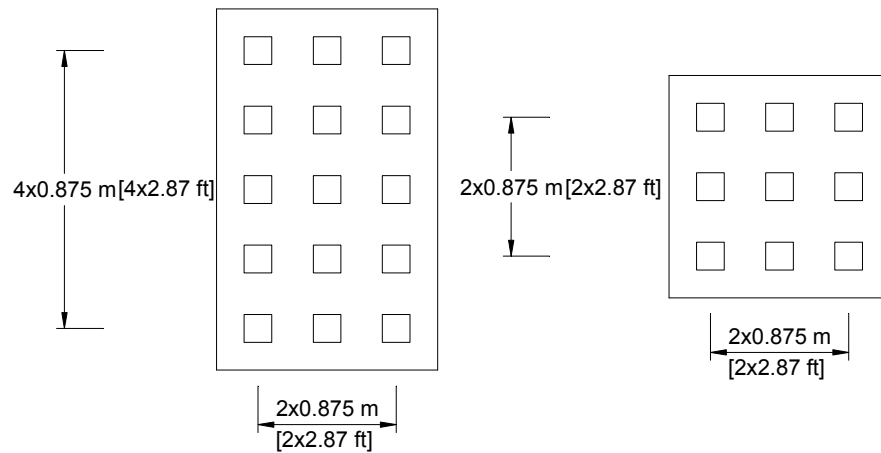


Figure 10—Pile arrangements

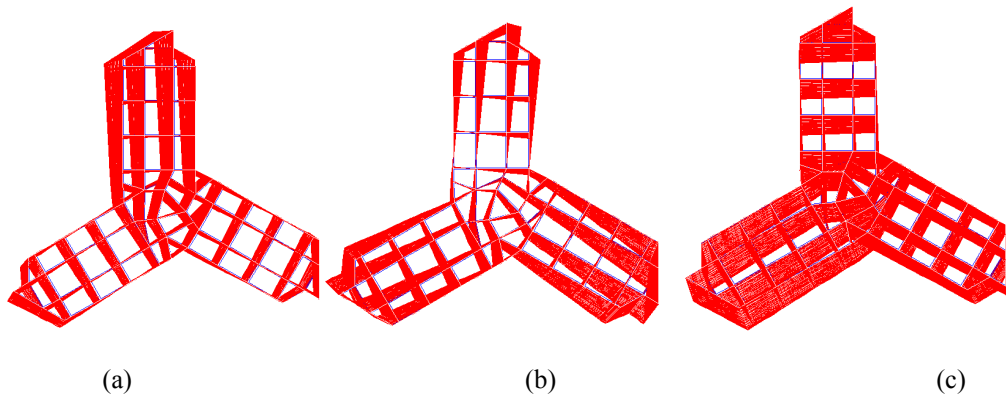
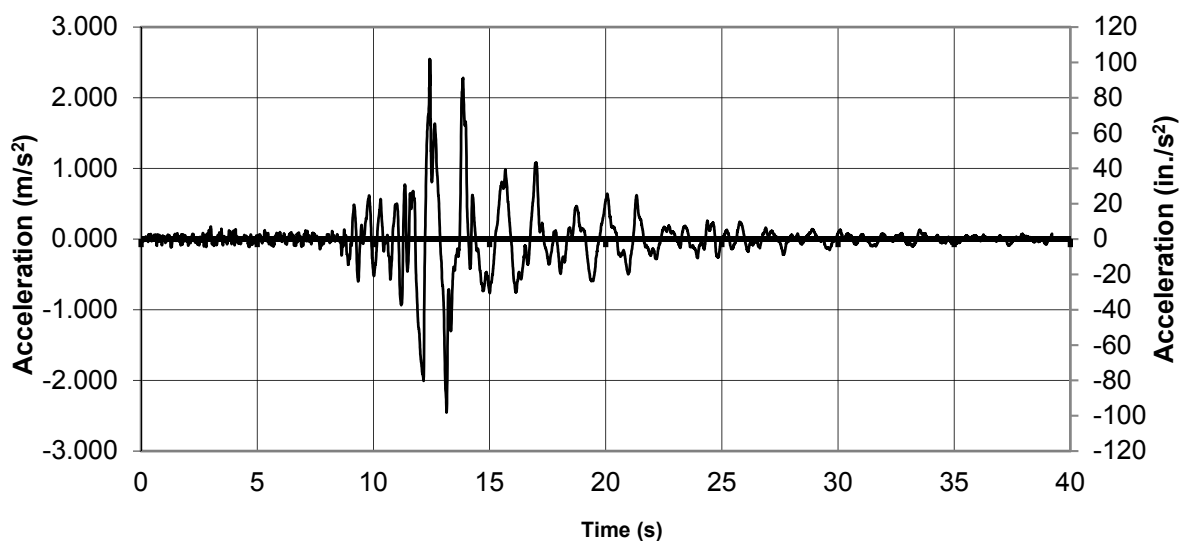
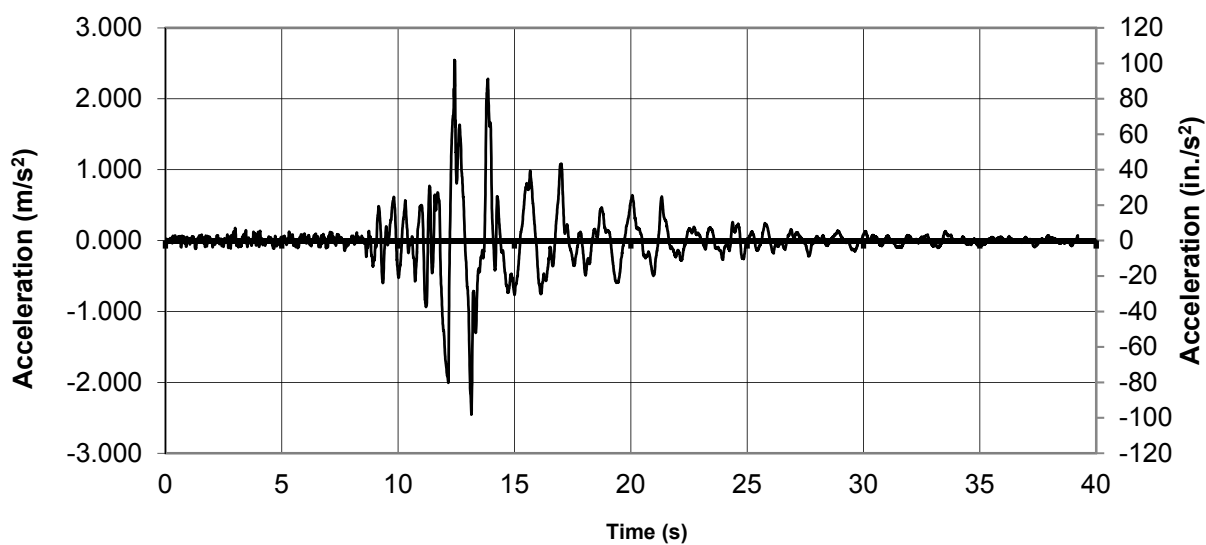


Figure 11—First three modes of vibration: (a) first mode; (b) second mode; (c) third mode



(a)



(b)

Figure 12—Ground Floor Input Motion: (a) E-W (X) direction; (b) N-S (Z) direction

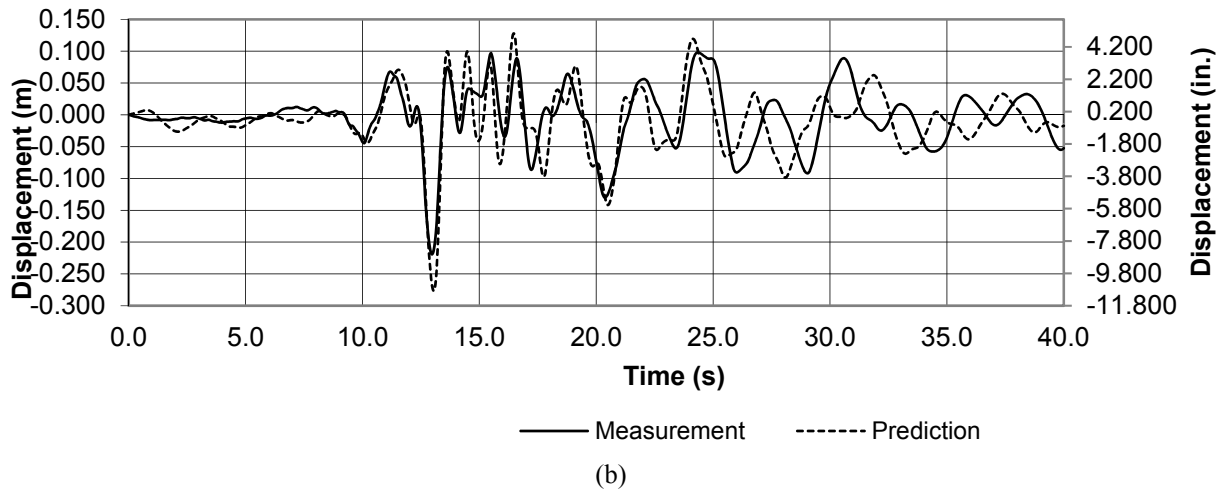


Figure 13—Displacements in E-W direction (31st floor)

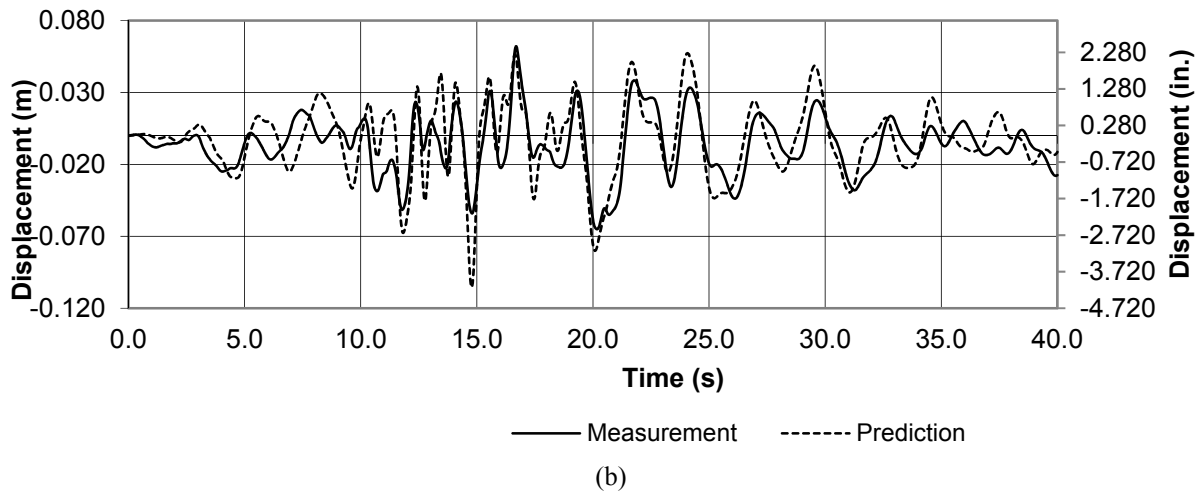


Figure 14—Displacements in N-S direction: (a) ; (b) 31st floor

Vibration Analysis for Dynamic Machine Foundation Using the Rapid Load Test of Pile

Yohei Tanaka, Keisuke Matsukawa, Naoya Kishi, and Genki Seo

Synopsis: In LNG plants and refineries, foundations for heavy rotating machines are required to be designed to limit the vibration induced by the unbalanced force and the moment of the machines to be within allowable amplitude for designated performance. When designing a pile-supported foundation for those dynamic machines, soil-pile spring constant to be used for dynamic analysis is calculated by conventional methods. However, there have been few studies of examining the accuracy of these methods and comparing them with the actual vibration data. The authors had an opportunity to carry out the vibration-proof design of reciprocating-type compressor foundation of which supporting PC piles were tested by rapid load test with a single mass model analysis. Moreover, vibration measurement was conducted at the Commissioning and multiple-point vibration data were obtained and studied by FFT analysis. Consequently, soil-pile spring constant was estimated from the rapid load test and the vibration analysis was conducted by 3D FEM software (STAAD. Pro)¹. In particular, the vibration amplitude and the mode of various wave components calculated based on the rapid load test and conventional methods, and those measured by actual vibration, were compared. From this result, we confirmed the validity of the analysis method based on the rapid load test result, which can be proposed for dynamic machine foundation design.

Keywords: Dynamic Machine Foundation, Rapid Load Test of Pile, Spring Constant of Pile, Vibration Analysis,

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INTRODUCTION

In LNG plants and refineries, heavy rotating machines such as compressors are required to carry out their foundation design with vibration analysis. Since heavy rotating machines induce unbalanced force and moment, amplitude of foundation shall be within allowable amplitude for designated performance. When designing pile-supported foundation for those dynamic machines, soil-pile spring constant to be used for dynamic analysis is calculated by conventional methods such as NOVAK's and Yamamoto's. In the field of plant engineering, there have been a lot of past records using NOVAK's and Yamamoto's methods, and it has been concluded that their vibration analysis is acceptable with a fair safety margin, thus there have been few studies of examining the accuracy of these methods and comparing them with the actual vibration data. On the other hand, highly accurate vibration analysis and vibration-proof design have come to be required in the actual projects, and at the same time, highly accurate and sophisticated pile load tests are also become available. Therefore, it is important to conduct a verification of the vibration analysis by the actual measurement of the vibration of the foundation, which would contribute to the enhancement of the methodology of such machine-induced vibration analysis. The authors had an opportunity to carry out vibration-proof design of a reciprocating-type compressor foundation of which supporting prestressed concrete (PC) piles were tested by rapid load test with a single mass model analysis. Moreover, vibration measurement was conducted at the Commissioning and multiple-point vibration data were obtained and studied by Fast Fourier Transform (FFT) analysis. Consequently, the soil-pile spring constant was estimated from the rapid load test and the vibration analysis was conducted by 3D FEM software (STAAD. Pro). In comparison with the vibration amplitude and the mode of various wave components between the analyses and the actual vibration, we confirm the validity of the analysis method based on the rapid load test result, which can be proposed for dynamic machine foundation design.

RESEARCH SIGNIFICANCE

In the field of plant engineering, there have been many past records of the machine-induced vibration analysis using design parameters of the piles calculated by conventional methods such as NOVAK's and Yamamoto's, and this methodology has been recognized as optimum design practice based on the fact that actual machine foundations were completed without any troubles. Few studies of examining the accuracy of these methods by measuring actual vibration of foundation have been conducted. In addition, more accurate and sophisticated pile load tests such as the rapid pile load test became available recently, but those data have been used only for confirmation of actual bearing capacity of a single pile basically, and not yet been used for assessment of the machine-induced vibration. In this paper, a new attempt to conduct vibration analysis by 3D FEM software using the soil-pile spring constant estimated from the rapid load test was presented and the verification of the results was performed by the measurement of actual vibration of the foundation. This study would lead to enhancement of the accuracy of the vibration analysis method and thus is considered significant.

DESIGN OF DYNAMIC MACHINE FOUNDATION

Outline of Dynamic Machine Foundation Design

In designing foundation for heavy rotating machine such as compressor, static analysis and dynamic analysis should be conducted. In static analysis, the shape and size of foundation, the length of piles, and the numbers of piles are designed by stability analysis and proportioning of section. In the stage of dynamic analysis carried out after the static analysis, the frequency ratio and allowable dynamic amplitude should be checked carefully to control vibration of foundation. To prevent resonance, frequency ratio of the equipment to the natural frequency of the

foundation is confirmed to be away from the unity (1) by sufficient margins. This limit value is usually decided by the owner's project specification. In addition, the maximum amplitude calculated based on the analysis model is confirmed not to exceed the allowable amplitude to maintain equipment operation safety. This limit value is usually specified by the manufacturers of the machine.

Analysis model

Dynamic analysis is usually conducted by 3D FEM software. A brief description of the 3D analysis is as follows:

3D FEM software: STAAD. Pro
Type of elements: solid element (Concrete) and spring element (Pile)
Boundary conditions: bottom of spring elements are fixed
Dynamic analysis type: modal analysis and frequency response analysis

As design data regarding the pile such as Young's modulus and spring constant of piles, numbers of piles, arrangement of piles, and loading data of rotating machine are necessary for analysis, these data should be determined by soil data and result of static analysis. Figure 1 shows the 3D FEM model of a boil off gas (BOG) compressor foundation used for this case study.

Spring Constant of Pile

In Japan, the method proposed by Yamamoto² (hereinafter referred to as Yamamoto's method), is widely used for calculating spring constant of pile. In Yamamoto's method, the vertical spring constant of a pile is defined by combining spring constant of the pile itself with spring constant of soil as shown in Eq. 1

$$K_v = \frac{K_s \cdot \bar{K}_z}{K_s + \bar{K}_z} = \frac{(E_p \cdot A/L) \cdot \bar{K}_z}{(E_p \cdot A/L) + \bar{K}_z} \quad (1)$$

where K_s is the spring constant of soil, E_p is Young's modulus of pile, A is the area of pile section, L is the length of pile and $\bar{K}_z$ is coefficient for static soil at pile toe determined based on Young's modulus of bearing stratum, radius of pile and Poisson's ratio of soil. In the US and other countries, Novak's method^{3,4} is well known for calculating the spring constant of pile. In this method, the vertical spring constant of pile is calculated by Eq. 2

$$K_v = (E_p \cdot A/r_0) \cdot f_2 \quad (2)$$

where E_p is Young's modulus of pile, A is the area of pile section, r_0 is the radius of pile, and f_2 is the coefficient relating to the ratio of shear wave velocity in soil to that in pile, and to the pile slenderness ratio calculated by its length and radius. Novak's method holds the same approach as Yamamoto's method in that using combination of soil and pile data. In this case study, the pile group effect is not considered as dynamic group effects are complicated as commented by ACI 351.3R⁵.

Loading Data of Dynamic Machine

In LNG plants and refineries, reciprocating type compressor such as this BOG compressor is one of the heavy rotating machine types that frequently adopted in their process. The reciprocating type compressor is characterized by a series of piston's motions moving backwards and forwards in a straight line, and usually brings vibrations by three types of unbalanced forces. Figure 2 shows the mode of three unbalanced forces, namely, direction of unbalanced horizontal force, rocking moment, and torsional moment. These unbalanced forces are assumed as the main and unique sources of vibrations in the dynamic analysis.

RAPID PILE LOAD TEST

Outline of Rapid Pile Load Test

Rapid pile load test (The Japanese Geotechnical Society 1815-2002)⁶ is classified into the dynamic loading tests. In this test, dynamic load is applied to the pile head through a cushion material. Judging from the load feature, this test can be defined as an intermediate method between static load test in which vertical load is added statically by hydraulic pressure jacks, and dynamic load test in which impact load is applied by a hammer. Although the influence of the wave propagation phenomenon in the pile is negligible, influences of velocities and accelerations of the pile as a single lumped mass on the surrounding soil are significant. In particular, the duration of the rapid load would typically lie in the range of $5 \leq Tr < 500$, where Tr is the relative loading duration, that is the ratio of the loading duration to the time necessary for the wave to travel along one pile length, defined as Eq. 3

$$T_r = \frac{t_L}{2L/c} \quad (3)$$

where L is the pile length, t_L is the loading time duration from the start of loading to the end of unloading, and c is the wave velocity given by the following Eq. 4

$$c = \sqrt{E/\rho} \quad (4)$$

where E and ρ are Young's modulus and density of the pile material, respectively.

Unloading Point Method

The objectives of the rapid pile load test are to obtain information on the vertical bearing capacity of a single pile and to confirm the predetermined design bearing capacity of a pile. In design of LNG plant and refineries, the latter is the main purpose of this method. In the former case, the procedure to calculate the static resistance component in the rapid pile load test is the unloading point method. Unloading point resistance is a force at the maximum pile displacement after correcting the inertia force of a pile, and is regarded as the maximum static resistance of the pile obtained from the rapid load test. Figure 3 shows the resistance components of the rapid pile load test, and the pile used in the test is modeled as a single lumped mass supported by spring and dashpot which represent the soil resistance as shown in Figure 4. From the equilibrium of loading force to pile, Eqs. 5-7 are derived as follows

$$F_{rapid} = R_a + R_{soil} \quad (5)$$

$$= M \cdot \alpha + R_{soil} \quad (6)$$

$$R_{soil} = R_w + R_v = R_w + C \cdot v = F_{rapid} - M \cdot \alpha \quad (6)$$

$$R_w = F_{rapid} - R_a - R_v = F_{rapid} - M \cdot \alpha - C \cdot v \quad (7)$$

where F_{rapid} is the loading force, R_a is the inertia force of the pile, R_{soil} is the resistance of the soil, R_w is the static resistance, R_v is the dynamic resistance of the soil, M is the mass of the pile, α is the acceleration of the pile, C is the damping ratio and v is the velocity of the pile. Figure 5 shows the static resistance of the unloading point method. The unloading point is the maximum displacement point of the curve line relating R_{soil} and the displacement of the pile. Therefore, dynamic resistance of the soil (R_v) does not work at the unloading point since the velocity of the pile is 0 at the unloading point. Then, R_w is equal to R_{soil} from Eq. 6. That is, R_{soil} corresponding to the unloading point is the same force as the maximum static resistance ($R_{w(max)}$) of the rapid pile load test, and is equal to the unloading point resistance (R_{ulp}). Ultimately, the spring constant of the pile is calculated as the ratio of the unloading point resistance (R_{ulp}) to the maximum displacement. Therefore, the static resistance can be calculated by measurement of loading force (F_{rapid}) (by a load cell) and maximum displacement of pile (by an accelerometer) in this test.

Differences between Static Axial Compressive Load Test and Rapid Pile Load Test

Figure 6 schematically shows the set up for static axial compressive load test. The test requires relatively heavy-duty equipment such as anchor piles and loaded beam firmly fixed to resist the jacking force. This costly setting and corresponding long schedule are drawback, however the test results are highly reliable because the loading condition is exactly the same as actual piles. On the contrary, Figures 7 and 17 show the set up for the rapid pile load test which consist of relatively simple test equipment. The advantages of the rapid load test are the less costly setting, and thus relatively short schedule, and that the test is able to be conducted in a narrow area.

ACTUAL VIBRATION ANALYSIS IN DESIGN PHASE

Actual Design Data

The case study is the foundation of reciprocating BOG compressor in LNG receiving plant located in Japan. The outline drawing and isometric view are indicated in Figure 8. This plant is constructed on a landfill site. And the ground is composed of relatively soft soil surface to 8m (26.2ft) depth, sand gravel in the subsequent deeper layer, and granite at bearing stratum of which N value is over 50. Then the designed foundation is pile-supported foundation using PC piles. This pile length is 17.2m (56.4ft) and diameter is 500mm (19.7in). This foundation design was conducted by static analysis first, then verified by vibration analysis of 3D-FEM model using general-purpose structural analysis software. Yamamoto's method was selected for calculating the vertical spring constant of the pile. This design resulted in a pile-supported foundation by 20 piles for 765kN (172kipf) of equipment and 4,536kN (1,020kipf) of foundation weight.

Loading data of equipment

Table 1 shows the unbalanced maximum force and the moments of the reciprocating compressor. Those unbalanced forces and moments given by the manufacturer consist of those at the primary frequency (5.92Hz) and the secondary frequency (11.84 Hz, double the primary frequency), which are used for the vibration analysis. Vibration at any higher frequencies that can be observed at the measurement is not considered in the analysis.

Result of vibration analysis

Table 2 summarizes the six lowest modes of natural frequencies of the designed foundation. According to this table, the primary natural frequency of the foundation is 8.1Hz. Then this foundation is able to be judged not to resonate because the ratio of the 1st vibration frequency of the equipment to the 1st natural frequency of foundation is 1.37, and the natural frequency is set over 30% away from the operating frequency. Figure 9 summarizes the single amplitudes (0 - peak) of three directions at eight points corresponding to the later measurement points shown in Figure 10. The allowable amplitudes of this compressor are 10 μ m (0.39mils) for X and Z-direction and 20 μ m (0.79mils) for Y-direction. As shown in this Figure, amplitudes of Y-direction are much higher than those of X-direction and Z-direction. The reason is that Y-direction is the reciprocation direction of the vibration from the compressor. All amplitudes of each direction are under the allowable amplitudes.

MEASUREMENT OF FOUNDATION AMPLITUDES

Measurement of Foundation Amplitudes

Actual amplitude was measured at the Commissioning by 100% performance testing for the BOG compressor by accelerometers. The measuring points were made at four corners of the pedestals for the compressor and the motor as shown in Figure 10, and three directions (the X-direction (perpendicular to the reciprocation direction), the Y-direction (the reciprocation direction) and the Z-direction (vertical direction)) were measured at every measuring point. These multiple-point measurements (eight measuring points) were made at the same time. This simultaneous measurement is the main feature of this study, of which purpose is to grasp the relative vibrational movement of each measuring point, i.e. to grasp modes of the vibration.

Result of Actual Amplitude

The result of actual vibrational amplitude is shown in Figure 11. This Figure shows that actual vibrational amplitudes of three directions were observed within the allowable amplitude limit designated by the machine manufacturer and there is no problem with the actual vibration.

FFT analysis

The result of measurement was resolved into every 1Hz component in the range of 1-100Hz by FFT analysis. Figure 12 shows the result of No.3 for Y-direction as the representative of the trend commonly seen for all measurements. It is evident from Figure 12 that the amplitude of the primary vibration frequency of compressor (6Hz) becomes the biggest. However, the amplitude of the secondary vibration frequency (12Hz) is small, and rather significant amplitudes of 3rd and 6th vibration frequency are observed. In the foundation design, only 1st and 2nd unbalanced force and moments of equipment are considered and the influence of higher vibration frequency is negligibly small and generally not considered in design practice. However, many pipes and instruments of control system are connected around the compressor in the actual construction. Therefore, the cause of this vibration at the high frequencies is considered to be induced vibrations of these miscellaneous attached pipes and instruments, not by vibration of the BOG compressor.

Result of Phase Differences

The state of the vibration wave of 6Hz component measured simultaneously in each direction is respectively indicated in Figure 13. These figures are indicating four corner points on the compressor. As figures indicate, No.3, 4 and No.7, 8 are vibrating in pairs, and the maximum amplitude of each pair alternately occurs. To visually grasp these phase differences, animation tool was developed. Figure 14 shows a sample picture of animation. The movement at each measuring point of three directions (X, Y and Z) can be simultaneously observed by inputting vibration wave data. By this animation, the comparison between actual vibration and the analysis model simulation was conducted. From this result, it is observed that the actual movement generally agrees with the movement of the analysis model, and it is thus confirmed that this vibration analysis method by using 3D FEM model can express the actual vibration with reasonable accuracy especially in terms of phase differences.

COMPARISON OF AMPLITUDE

To grasp accuracy of amplitude based on analysis model, actual and analytical amplitudes of each measuring point were compared. The analytical amplitude data was calculated by 3D-FEM model which uses spring constants of pile obtained from Yamamoto's method, Novak's method, and the rapid load pile test. Table 3 shows vertical spring constant of pile. The spring constant of pile obtained from Novak's method and the rapid pile load test is more than twice of Yamamoto's method. The comparison of total amplitude is shown in Figure 15. As Figure 15 indicates, the amplitude from Yamamoto's method are bigger than that of actual amplitude at almost all the points. Especially, the amplitude of Y-direction which is parallel to the main reciprocation direction is significantly bigger than that of the actual data. On the contrary, the amplitude obtained from the rapid pile load test is smaller than that of the actual data at most of the points. Comparison of amplitude in 6Hz which corresponds to the primary frequency (6Hz) of the equipment is indicated in Figure 16. In Figure 16, the simulated vibration amplitudes are almost always bigger than that of the actual data unlike the case of total amplitude in which the order of amplitudes for simulation and actual varies on a case-by-case basis. The differences between the actual data and the rapid pile load test are less than those of between the actual data and Yamamoto's or Novak's method. In dynamic analysis, it is not preferable if the actual amplitude surpasses amplitude predicted by the analysis model, not showing the safer side answer. The amplitude obtained from the rapid pile load test resulted in smaller amplitude than actual data in total vibrational wave. This could be due to the influence of high vibration frequency arising in foundation which is not included in loading data and identified by FFT analysis. This complication by high frequency vibration is an issue to be solved. However, 6Hz vibration component is the primary source of the compressor vibration and the largest unbalanced force in most cases. That is why simulation for the amplitude in 6Hz vibration component based on the rapid pile load test gives prediction less than (only one point exception) and close to the actual. It is also evident from the comparison of amplitude that the amplitude obtained from the rapid pile load test is closer to actual data than those calculated based on Yamamoto's method.

CONCLUSION

Vibration measurement of a real-scale reciprocating compressor foundation was conducted and compared with the vibration analysis results at the design stage. The following conclusions have been drawn from this study.

1. The vibration measurements at multiple points around the compressor were simultaneously conducted at a real-scale plant and phase difference among measurement points and consequently, the mode of actual vibrations were clarified for the primary mode frequency of the machine.
2. The vibration analysis by 3D FEM was conducted for the same foundation. For the primary mode vibration, the vibration mode obtained by the actual measurements and that by analysis agree with reasonable accuracy when visualized by animation.
3. The vertical static spring constant obtained by the rapid pile load test is considered to sufficiently simulate the real vibration. According to the comparison of the vibration amplitudes of the primary frequency component (6 Hz) by actual measurement and those calculated by design simulation analyses, the analysis using the spring constant by the rapid pile load test provided the closest amplitudes to the actual measurement than those using either Novak's method, or Yamamoto's method.

As for the remaining issues for the future, the study is for one foundation for a vibrating equipment on a particular soil condition, and data accumulation for more cases on different conditions is necessary, especially for the more reliable use of the rapid pile load test. It is hoped that the findings in this paper will contribute to a better understanding of the soil and pile testing and a more accurate vibration-proof design.

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Table 1—Unbalanced forces (max)

Unbalanced Force	Primary (5.92Hz)	Secondary (11.84Hz)
Fu : Unbalanced Force	4.73kN (1.06kipf)	0.46kN (0.10kipf)
Mh : Unbalanced Moment (Rocking)	97.48kN · m (71.90kipf · ft)	1.13kN · m (0.83kipf · ft)
Mv : Unbalanced Moment (Torsional)	29.28kN · m (21.60kipf · ft)	

Table 2—Natural frequencies of foundation

Mode	Frequency(Hz)
1st	8.1
2nd	8.2
3rd	11.3
4th	16.2
5th	17.4
6th	18.6

Table 3—Spring constant of pile (vertical)

Method	Spring Constant of Pile
Yamamoto's Method	180,199N/mm (1,029kipf/in)
Novak's Method	497,082N/mm (2,838kipf/in)
Rapid Pile Load Test	374,714N/mm (2,140kipf/in)

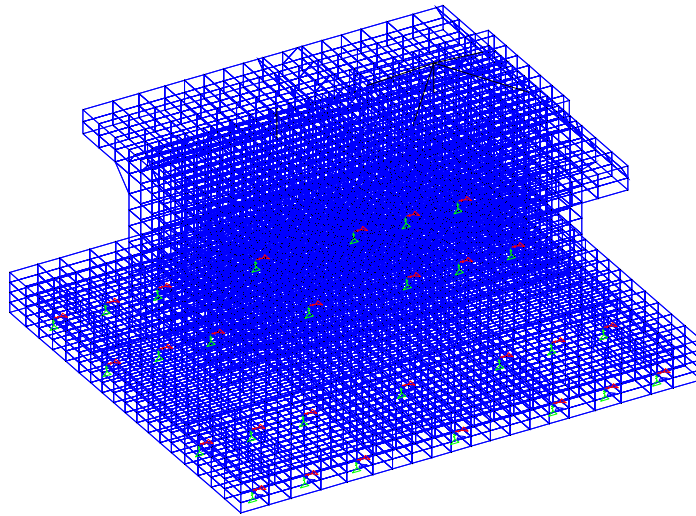


Figure 1—3D FEM model

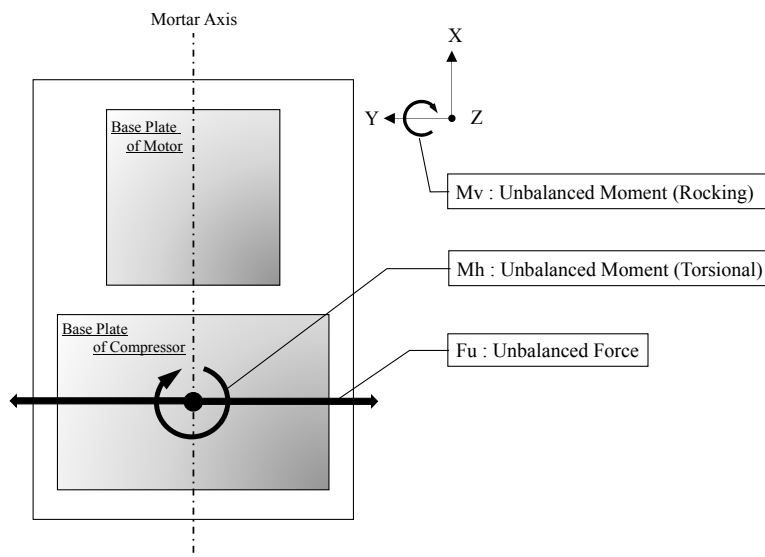


Figure 2—Unbalanced forces

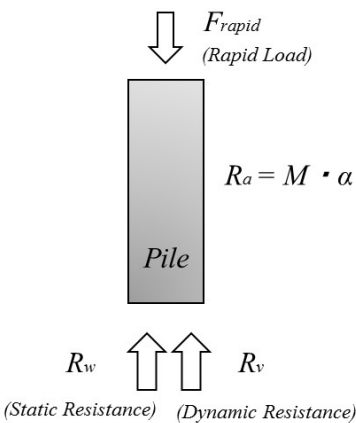


Figure 3—Resistance component

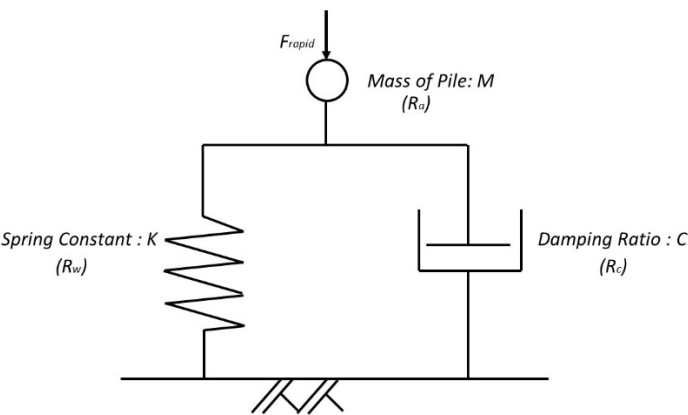


Figure 4—Single mass model

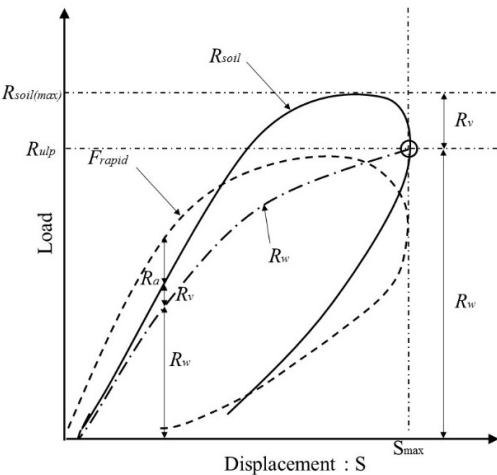


Figure 5—Curve line relating R_{soil} and displacement of pile (static resistance)

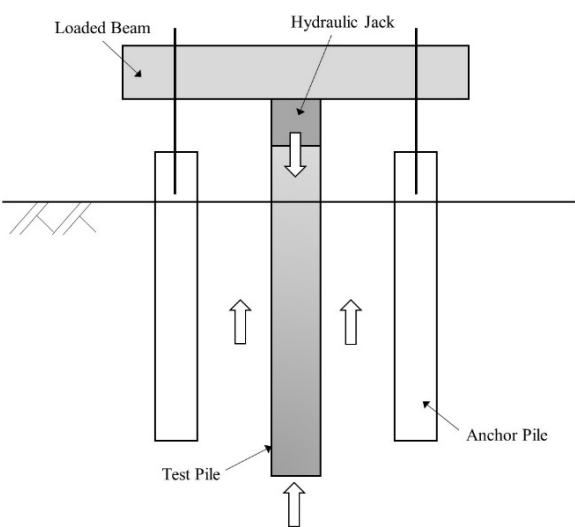


Figure 6—Static axial compressive load test

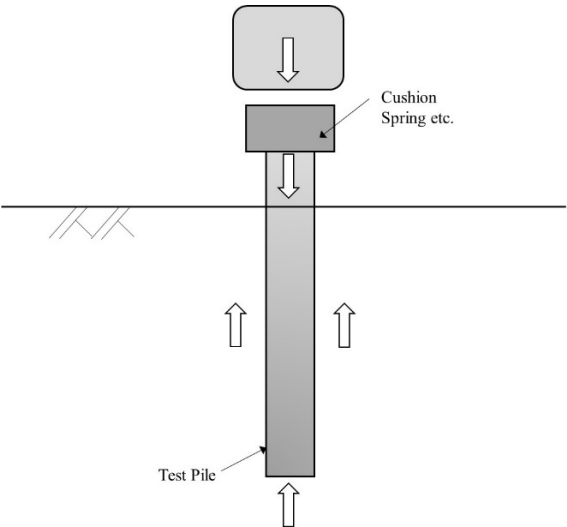


Figure 7—Rapid pile load test

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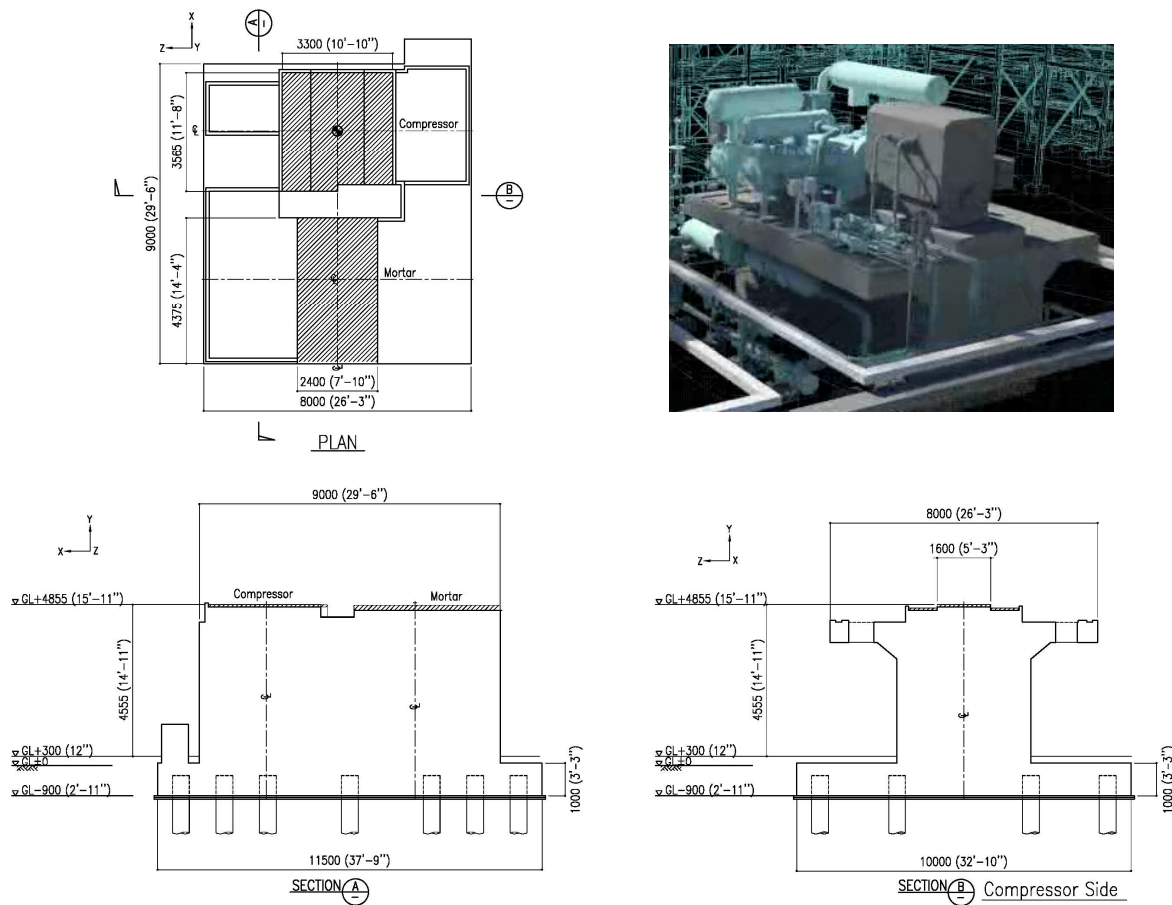


Figure 8—Outline drawing and isometric view of compressor foundation

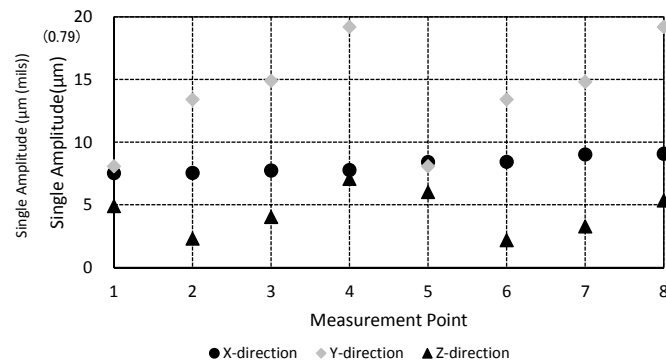
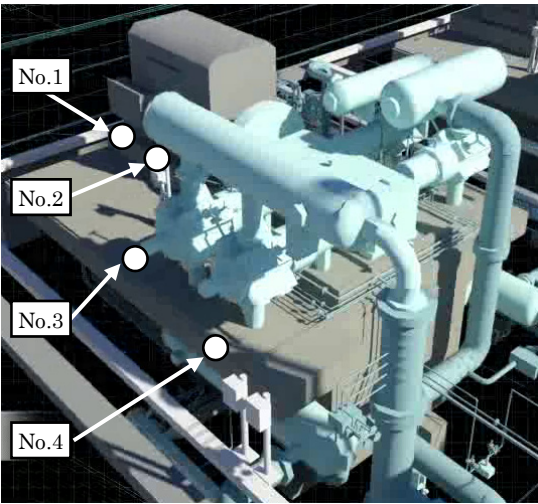
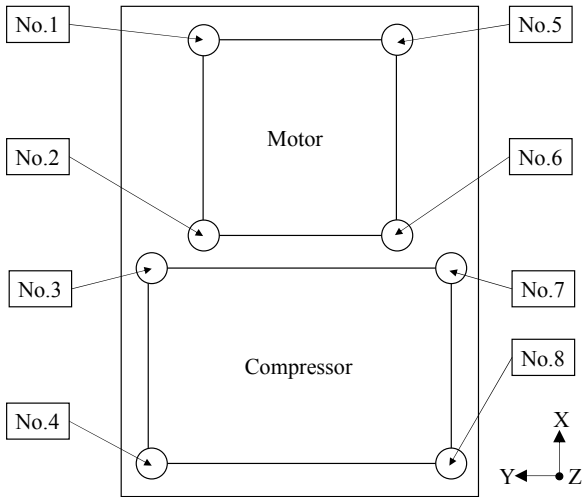


Figure 9—Amplitude of design (single)



Note: No.1 to No.4 and No.5 to No.8 are at symmetry position

Figure 10—Measuring point

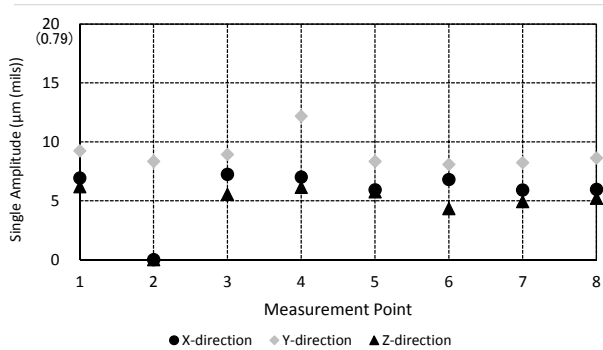


Figure 11—Actual vibrational amplitude (single)

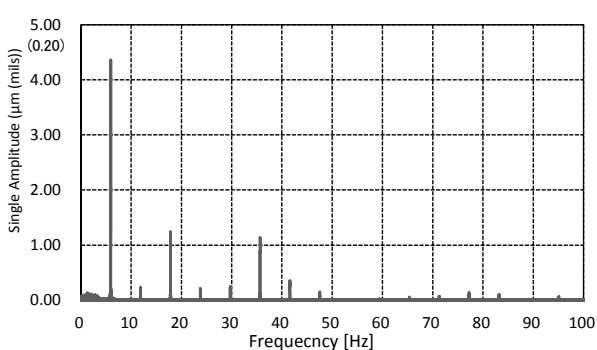
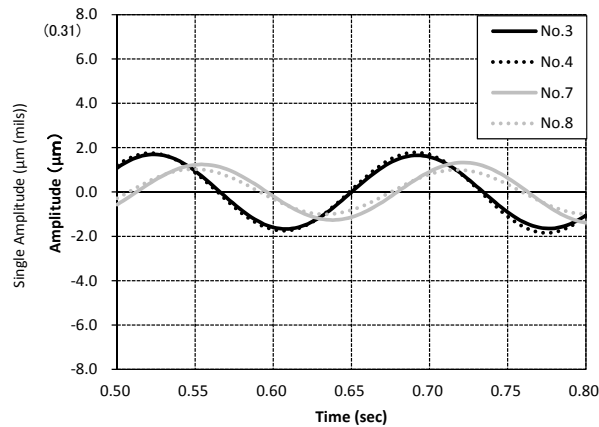
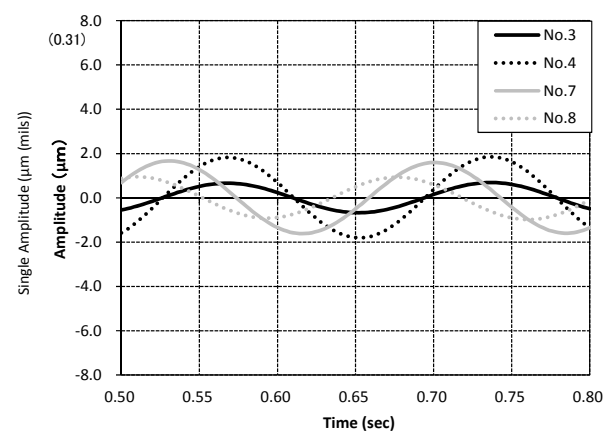


Figure 12—Result of FFT analysis (No.3-Y direction)

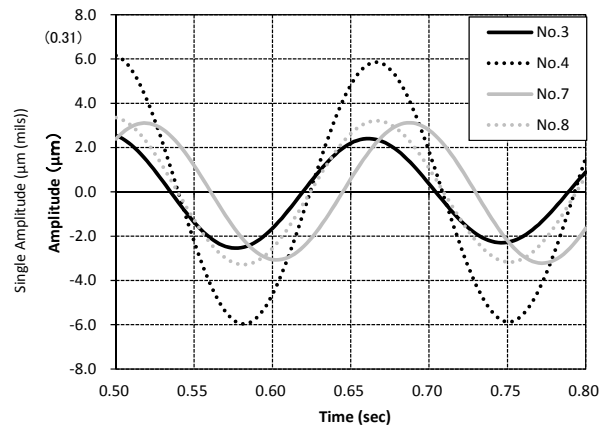
Vibration Analysis for Dynamic Machine Foundation Using the Rapid Load Test of Pile



(a) 6Hz-X direction



(b) 6Hz-Z direction



(c) 6Hz-Y direction

Figure 13—Time history vibrational wave

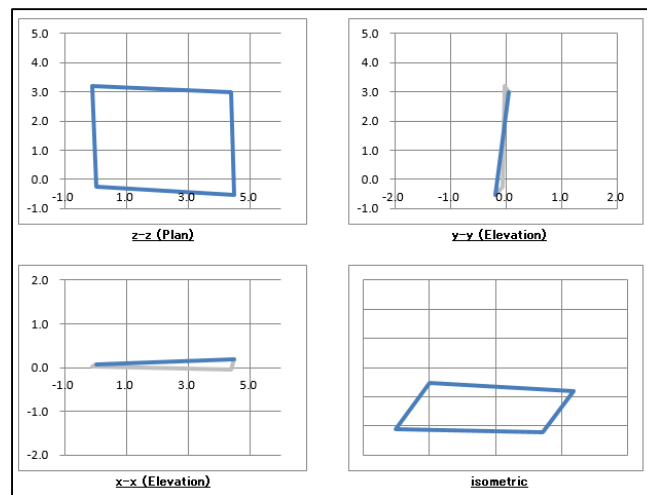
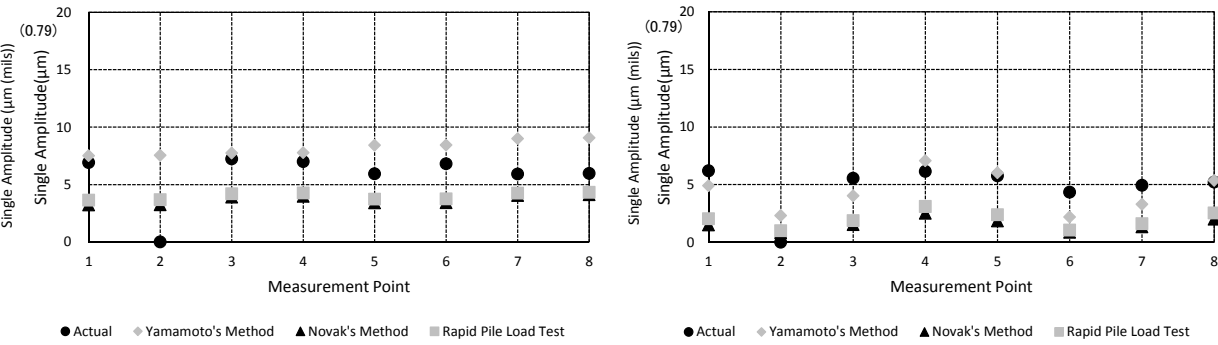
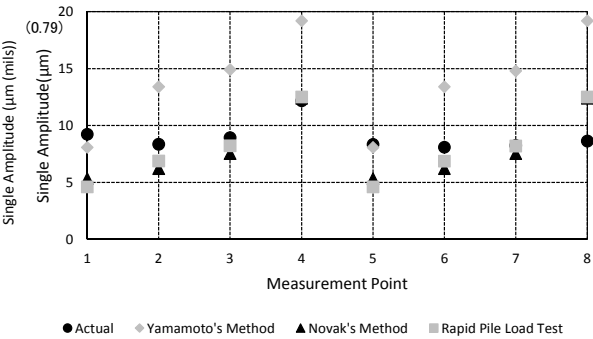


Figure 14—Sample picture of animation



(a) X direction

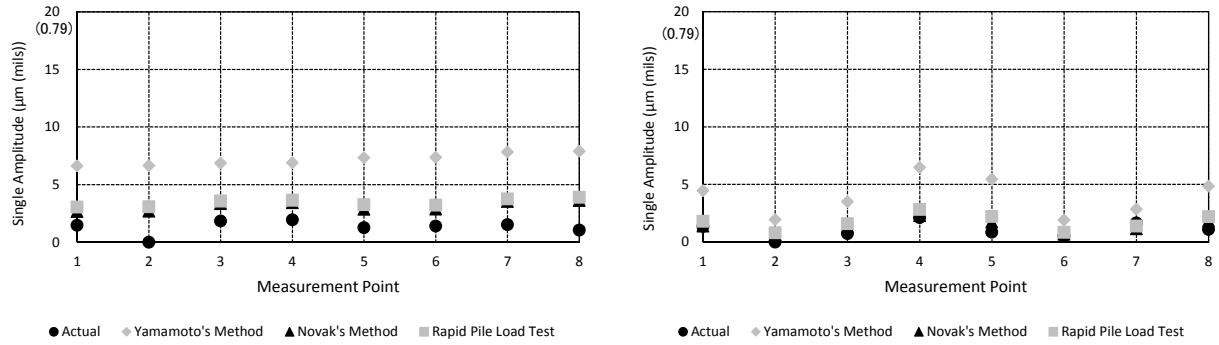
(b) Z direction



(c) Y direction

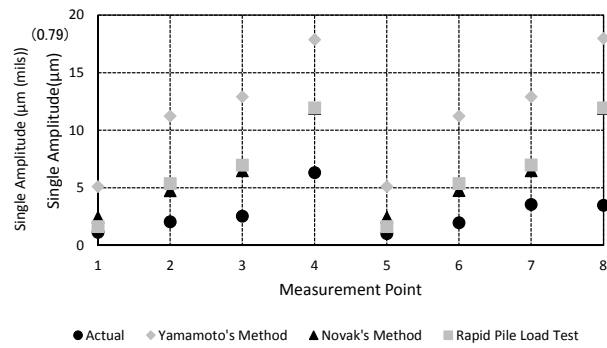
Figure 15—Comparison of total amplitude

Vibration Analysis for Dynamic Machine Foundation Using the Rapid Load Test of Pile



(a) X direction

(b) Z direction



(c) Y direction

Figure 16—Comparison of amplitude (6Hz)



Figure 17—Rapid pile load test (photograph courtesy: Jibanshikenjo Co. Ltd.)

A Tale of Two Buildings: Case Studies of Underpinning by Compaction Grouting

Frederick R. Rutz, Jennifer Harris, and James Robert Harris

Synopsis: Two concrete masonry buildings, at adjacent sites in Glenwood Springs, Colorado, are located atop existing collapsible debris fan soils. Both buildings were constructed on concrete foundations with spread footings, and both suffered serious and damaging differential settlements. Compaction grouting was utilized for underpinning and lifting both buildings. Compaction grout columns are comprised of a low slump and low strength grout made from a combination of sand, soil, pea gravel, cement, and water. When installed under pressure, the grout densifies the surrounding soils supporting the building foundation, and when carried to the underside of footings, the grout can offer direct support. The grout was also used to lift and partially level the buildings. But here the similarity ends; each had unique circumstances and the repair designs were custom tailored. One was underpinned with deep (100') compaction grout columns while the other received a much shallower underpinning treatment. Each had unique drainage problems. Both projects were challenging and required cooperation among the Owners, Structural, Geotechnical and Civil Engineers, and the Contractors. The geotechnical studies, the structural design for repair, the drainage provisions for each, and the construction are described, with a focus on structural damage, design of the underpinning to be compatible with the structural capacities, and control systems utilized during construction.

Keywords: Case Study, Hydro-Compressive Soil, Compaction Grouting, Debris Fan, Underpinning

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ACI member **James R. Harris** is Principal and Owner of J.R. Harris & Company, Denver, CO. Dr. Harris serves on numerous structural code committees and is a member of the National Academy of Engineering. He was Engineer-of-Record for the two compaction grouting projects.

INTRODUCTION

Debris fans are found throughout the Rocky Mountain West. Soils found in debris fans tend to be collapsible and can settle, sometimes significantly, when wetted. This paper presents case studies of two buildings sited on a particularly large debris fan in Glenwood Springs, Colorado. Both buildings were constructed with walls of concrete masonry units on reinforced concrete spread footing foundations. Both buildings settled where water ingress to the soil occurred and both suffered significant distress when subsequent settlement occurred. Both were underpinned using compaction grouting, and both had the lowest portions raised to an approximate level condition. Similarities and differences of the two cases are presented with commentary on our experiences using compaction grouting for both ground improvement and for lifting concrete masonry buildings.

RESEARCH SIGNIFICANCE

These case studies highlight remediation of concrete foundations suffering significant differential settlement caused by collapsible soils. Compaction grouting was designed to accomplish both soil improvement and direct underpinning to more stable layers. Future research could lead to improved quantitative procedures for both design philosophies.

PROBLEM SUMMARY STATEMENT

Both buildings suffered from severe settlements due to collapse of soil upon the introduction of water. Such soils are known both as hydro-compressive or hydro-collapsible, but the term “collapsible soil” will be used throughout this report. In both cases the introduction of water led to the soil compression. Serious structural distress had occurred with the potential for partial building collapse a real concern. The goal for each project was to:

- Arrest further settlement
- Lift the most severely settled portions of the buildings back to approximately level
- Include provisions to resist future settlement as the existing wetting front advanced, even if no further water entered the soil
- Improve drainage to guard against future water ingress
- Repair damage and reconstruct as necessary

LOCATION

The buildings are located atop a debris fan in Glenwood Springs, CO. Development has been ongoing on the debris fan since about 2000, around the time when the buildings were constructed. For purposes of this report, they are referred to as Building A and Building B. See Fig. 1.

DEBRIS FAN

A debris fan is a type of alluvial fan with steeply sloped sides. The fan is shaped like a cone which fans out at the base and is filled with debris in the form of boulders, cobbles, gravel, sand, and silt typically caused by geologic events such as a landslide, debris flow, or flooding. This soil matrix is often susceptible to collapse and it is the silty soil constituents within the overall debris fan materials that tend to be collapsible. The silty portion dries in a high void configuration with bonds between particles that dissolve when wetted. If load is present, the void ratio will reduce. At the subject site, the fan is particularly large, about ½ mile (0.8 km) wide at its base. The depth varies, with the deepest parts – on the order of 100 feet (30.5 meters) near the base, where it is underlain by river cobbles from the ancient Colorado River bed.

COLLAPSIBLE SOIL

Characteristics of collapsible soil are summarized: Collapsible soil is typically fine-grained, has a low Plasticity Index (usually $PI < 10$), has low moisture content, and has a low dry density. It is when the moisture content and/or load increases that collapse occurs, leading to increased density and reduced volume. Collapsible soil is sometimes called “metastable,” meaning that the soil matrix responds to changes in moisture rapidly [1]. It is sometimes found cemented with clay or gypsum.

BUILDING DESCRIPTIONS

Both buildings, built in close proximity to each other in the early 2000’s, were constructed with steel roof decks on open web steel joists supported by concrete masonry unit (CMU) bearing walls on reinforced concrete footings. The first floor of each building is a concrete slab-on-grade. Building A has stem walls three feet (one meter) deep on relatively small footings while Building B has full basement walls on wide, continuous footings. A two-story interior open bay meant that the top of the basement wall had no lateral bracing – it had been designed as a retaining wall with both lateral pressure from soil and surcharge and gravity load from the bearing wall of the building above it. Both buildings had been designed by the same design team and constructed by the same contractor.

DISTRESS

Distress in both buildings manifested itself in the form of severe cracks through CMU walls and cracks through cast-in-place concrete walls. Tearing of roof elements and differential movements of slabs-on-grade also occurred.

Building A

While the magnitude of the foundation movements prior to our involvement in the project is unknown, we measured a maximum differential settlement of 8 inches (203.2 mm) near the middle of the building. Diagonal cracks formed, some diagonally up to the east on one side of the low point and others diagonally up to the west on the other side of the low point. See Fig. 2.

Building B

The greatest differential settlement was at the building corners, on the order of 7 inches (177.8 mm), resulting in movements that tended to cause in-plane flexure in CMU walls. The typical cracks were vertical and widest at the top of the wall. See Fig. 2.

MONITORING

Building A

Anecdotal reporting of distress but no formal monitoring had been documented when we began to study this building. The first comprehensive structural condition assessment included a relative elevation survey of the second floor, main floor slab and foundation from which Fig. 3 was created. The low region is shown as a “bull’s eye” directly over a water collection pit at a bus washing facility. The pit was found to be leaking water into the underlying soil, presumably since the end of construction about twelve years prior to the mapped elevation of the floor slabs. Fig. 4 illustrates the differential settlement superimposed over a building elevation.

Building B

Building monitoring tied to a benchmark began in April, 2010. Initially, elevation surveys were conducted monthly and gradually increased to three month intervals. The survey data showed that a continued downward settlement was occurring at all monitored points around the building perimeter. From April 2010 until June 2014, the southeast corner of the building had settled 4.2 inches (106.7 mm). The southwest corner of the building had settled 4.0 inches (101.6 mm) over the same time interval. The amount of settlement prior to the start of monitoring in 2010 is unknown, but has been estimated at about 3 inches (76.2 mm); thus the total settlement was estimated at about 7.2 inches (182.9 mm), shown in Fig. 5.

GEOTECHNICAL INVESTIGATION

In order to make an informed evaluation of complex problems such as these, considerable soil data from borings up to 100’ in depth was considered. Three soil borings were conducted at Building A, while eight soil borings were conducted at Building B. The data obtained from these borings and tests conducted on the samples included moisture content of the soil, soil density, Atterberg Limits, gradation (including hydrometer tests), specific gravity, consolidation, and water soluble sulfate determination. [2].

Building A

Three borings were drilled to a maximum depth of 97 feet (29.6 meters), revealing debris fan alluvial soils underlain by river alluvium.

Building B

A total of eight borings were drilled. Boring depths varied from 15 to 80 feet (4.6 – 24.4 meters). The bedrock was shallow at the southeast corner of the building where refusal was met at 15 to 23 feet (4.6 – 7.0 meters) and deepest at the southwest corner. The relative elevations of the buildings and the bearing strata are illustrated in Fig. 6. Results from a typical consolidation test are shown in Fig. 7. A particular concern was that the wetting front, estimated from the soil boring information, would continue to advance with consequent additional soil collapse. This is illustrated in Fig. 8.

ALTERNATIVES

Approaches for solutions were limited and all had pros and cons. Three conceptual approaches were studied for Building A:

□ Mat Foundation

Pros: Would act as a single spread footing under the entire building. Pressure on the soil would be distributed over a large bearing area. Could be designed to bridge over soft spots in the soil. The building would “float” on the soil. The top of the mat could replace existing damaged slab-on-grade surfaces. Possibly the most cost-effective approach.

Cons: Would have to be installed incrementally under existing building footings. All interior slabs-on-grade would have to be removed and either replaced or the top of the mat be designed to become the new floor surface. A long construction period resulted in operational issues for the Owner.

□ Micro-piers

Pros: Would support the building directly from building foundation to a bearing stratum below. Down drag can be reasonably estimated. Loads that each micro pier would carry could be calculated with reasonable accuracy.

Cons: Would require a certain minimum degree of stiffness and strength in the concrete foundation elements. Would require a special connection from micropile to concrete foundation element. The existing foundation stem walls and basement walls would have to act as grade beams. Most costly repair alternative.

□ Compaction Grouting

Pros: Densifies soil around the grout columns *and* can act as “poor man’s” underpinning piers. Initially the building footings are supported on densified soil. Later, after the wetting front advances, the footings would be supported on the compaction grout columns themselves, acting as piers.

Cons: Would have to be installed to the depth of firm bearing strata – 100 feet (30.5 meters) for Building A. (See Fig. 6). Down drag could be high and difficult to estimate because of the irregular surfaces. As with micropiers, the existing foundation stem walls and basement walls would have to act as grade beams. After review of all concepts, compaction grouting was selected as the most likely to satisfy the technical requirements for underpinning and lifting the buildings, constructability concerns, and meet budget and schedule requirements. Building B benefited from these studies; compaction grouting was the only alternative considered.

COMPACTION GROUTING

Compaction grouting involves the injection of a low slump, low strength concrete with coarse aggregate up to 3/8 inches (9.5 mm) into “columns” in the ground such that the insitu soil is densified (compacted) around the grout

“columns” [3]. While alternatives such as mat foundations and micropiles were considered, compaction grouting was selected as the preferred method for underpinning and lifting the buildings. Compaction grout “columns” were up to 100 feet (30.5 meters) deep at Building A and 50 feet (15.2 meters) deep at Building B. (See Fig. 6). Compaction grouting is schematically illustrated in Fig. 9.

REPAIR DESIGN

The existing wetting front was determined from geotechnical boring data and a future wetting front was estimated. Compaction grout “columns” were extended below the depth of the wetting front to river gravels at Building A and to a gravel/cobble layer found by the borings at Building B. The structural design considered two conditions for each building:

- Early in the life of the repair the existing footings would still act as footings on densified (compacted) soil.
- Later in the life of the repair, as the wetting front advanced (see Fig. 8), we anticipated some of the newly wetted soil would settle, and the load paths would shift from soil-supported footings to “point supports” on the compaction grout columns themselves (we called this scenario a building “on stilts”).

Only after geotechnical studies had been completed, and in coordination with the construction concerns and schedule did the actual structural design begin. The first case was relatively simple to design: the footings would remain effective but the densified soil could accommodate a higher bearing pressure. The second case necessitated a load path into the compaction grout “columns” so the strength of the grout became important. Thus a “dual load path” approach, a unique design feature here, was utilized. Grout strength was specified and grout “column” areas were estimated. Further, the existing footings would act as beams and the existing walls would act as grade beams, all requiring a sharpened pencil for structural analysis. While many different design sections were developed for differing conditions, two basic scenarios are discussed below:

Building A

The building was founded on stem wall three feet (one meter) deep and spread footing system. The compaction grouting occurred beneath the footings; the stem walls became grade beams by this scheme. Spacing of the “grout columns” was based on geotechnical considerations, permissible spans and torsion considerations of the stem wall/grade beam, and constructability issues. See Fig. 10.

Building B

The building had basement walls and wide footings that served to support gravity loads and as retaining walls against soil lateral pressure where there were open bays, bays that had no structural diaphragms at the tops of the basement walls. The basement wall/footing was a retaining wall and carried vertical gravity loads. Here the compaction grout “columns” were placed directly below the wide footings. The basement/retaining walls became grade beam/retaining walls. The spacing and arrangement of “grout columns” also required consideration of the large footings to act as structural slabs that could transfer gravity load and lateral loads from the basement wall to the slab to the grout columns. See Fig. 11.

CONSTRUCTION

Compaction grouting operations necessitated considerable coordination among the Owner, the General Contractor, the Compaction Grout Contractor, and the Engineer. While these complex projects called for considerable structural work to address all the damage, we will focus on the compaction grouting construction activities.

Bottom-Up Grout Injection

Once the Compaction Grouting Contractor had mobilized on the job sites, drilling of holes through the existing footings was an early step. While holes can be drilled using rotary impact hammers, we became concerned about the damage such hammering could do to the underside of the existing footing. So we changed to core drilling through the footings to minimize footing damage. Then the rotary impact drills were inserted through the holes in the footings and drilled to the design depth – on the order of 100 feet (30.5 meters) for Building A and 50 feet (15.2 meters) for Building B. At this point, grout was injected under pressure up to 700 psi (4826 kPa) using the “bottom – up” technique. Grout bulbs were formed at the bottom of the drilled hole, the injection pipe raised one foot (0.3 meter) and another grout bulb injected. This procedure was repeated until the top of the grout column was near the surface of the ground. The upper portions of the grout columns presented a new dilemma: The conflicting

requirements of obtaining acceptable soil densification without heaving of slabs or pavements was cause for careful monitoring and attention to this critical operation.

Grout

Grout mixing was done on-site using locally obtained sand and aggregate for the grout mixture. The grout was quite stiff with a maximum of 1" slump. The grout specification called for a minimum of 1000 psi (6895 kPa) compressive strength. All drilled holes and grout columns were logged for Quality Control and monitoring purposes.

Lifting

Lifting of portions of the buildings demanded a change to the ordinary grout columns. For locations to be lifted, a portion of the grout column was intentionally left out. The omitted length was about 5 feet (1.5 meters) in height and about 10-15 feet (3 – 4.6 meters) down from the surface. After the grout columns were constructed with grout bulbs both above and below this region, the upper grout column was drilled and large volumes of grout injected into the regions where it was previously omitted. The large grout volume densified the adjacent soil more than at other places, so much that it lifted the soil, and thus the footing. Lifting was accomplished incrementally, going back and forth between the grout columns in the lift zones.

PROBLEMS

As Geotechnical Engineers will attest, subsurface work is often unpredictable and this is certainly true for working below a building with complex functions. A discussion of these case studies would be incomplete without some discussion of problems encountered.

Interferences

While the grout column layout was designed taking into account known obstructions, unforeseen interferences became apparent once excavations commenced. The seemingly simple solution of relocating grout column positions actually resulted in considerable difficulty in checking that the former stem walls – now grade beams – could handle all loads when their spans were changed. Likewise, large footings acted as two-way slabs on new support points at grout columns and the relocation of a single grout column could cascade into the need to reposition several nearby grout columns.

Grout Take

The specified volume of grout was typically 5 cubic feet/foot of depth (0.14 cubic meters/meter of depth), this to densify (i.e. compact) the adjacent soil. Grout injection pressure was limited to 700 psi (4826 kPa) for safety reasons. The specified volume of grout could not always be achieved, owing to probable boulders in the debris fan. While the holes could be drilled through boulders, the amount of grout was limited within them. This resulted in less soil densification at such points.

Grout Strength

Samples of the grout were taken to assure the specified strength was obtained. The subsequent testing revealed that some of the installed grout was under-strength. Once the grout is in the ground, it cannot be removed and replaced. Installation of sister grout columns was not practical. The solution was to drill into the understrength grout columns and grout-in a rebar of sufficient size to compensate for the reduced grout strength.

Lifting Limitations

Lifting of buildings cannot be accomplished with precision and lifting by soil densification even more so. Despite careful monitoring, including dual monitoring and close cross-checking of results, the buildings could be lifted to meet only approximate elevation goals. The lifts were to approximate level, not exact level. See Fig. 12.

Slab Blow-Out

A slab blow-out was caused by injecting grout under pressure near the surface. The slab involved was an interior floor slab-on-grade; it had to be demolished, removed and replaced.

Heaved Slab

An interior slab heaved and cracked but the Owner decided to leave it in place because the function of that floor space was not overly impacted.