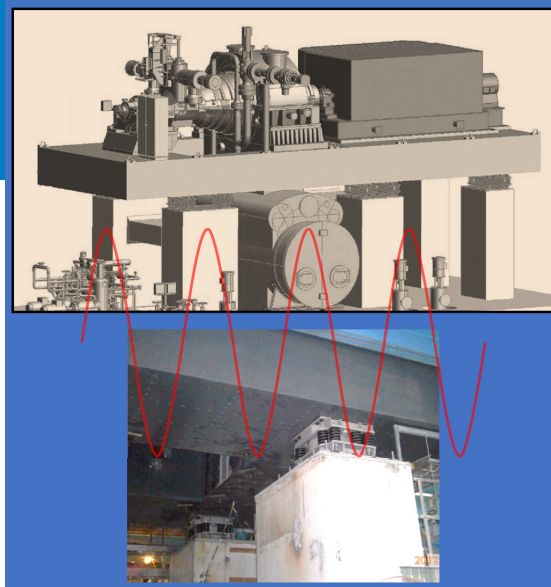


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SYMPOSIUM VOLUME



Foundations for Dynamic Equipment

SP-348

Editor:
Carl A. Nelson



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Foundations for Dynamic Equipment

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PREFACE

Foundations for Dynamic Equipment

This special publication grew out of the Technical Session entitled “Application of ACI 351-C Report on Dynamic Foundations,” held at the ACI Spring 2019 Convention in Québec City, Québec. Following this event, Committee 351 decided to undertake a special publication with contributions from those session participants willing to develop their presentations into full-length papers. Three papers included in the current publication were contributed by these presenters and their coauthors, with six additional papers provided by others. All but one of the papers deal with the subject matter of ACI 351.3—Foundations for Dynamic Equipment—updated in 2018. The one exception (the paper of Wang and Fang on wind turbine foundations) provides valuable information to engineers dealing with a lack of consistent design criteria among various codes for reinforced concrete foundations subjected to high-cycle fatigue loads.

I would like to thank the members of ACI Committee 351 for their support, in particular the current main Committee and Subcommittee C Chairpersons Susan Isble and Dr. Mukti L. Das, respectively. I also wish to express my gratitude to the authors for their perseverance through the difficult circumstances of 2020, and to the reviewers who generously contributed their time and expertise to this publication.

Last, but not least, I want to thank my wife Cindy for tolerating me (and the growing piles of paper) over the past several months as the deadline approached.

Carl A. Nelson
On behalf of ACI Committee 351

Minneapolis, December 2020

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Predicting Near and Far Field Ground Vibration for Equipment Foundations

David L. Pederson, Anthony J. Baxter and Carl A. Nelson

Synopsis: This paper discusses steps for both computing vibration from equipment foundations using the elastic half-space theory and then computing the decrease in vibration amplitude from the foundation to receivers. The steps are demonstrated on an existing foundation at a project site in Ohio that was subjected to dynamic loading from a hydraulic vehicle test rig. Several approaches are discussed to estimate the dynamic shear modulus of different soils, along with a methodology to establish an equivalent dynamic shear modulus for soils with varying shear wave velocities. Vibration transmission through the soil can affect people and sensitive equipment both near and far from the source. This paper shows a hybrid method and an SRSS method to compute the vibration attenuation through the near field and far field. The calculated results for this site were found to be very close to the measured values. Finally, vibration levels are compared for variations in stiffness, damping and attenuation to evaluate the sensitivity to calculations and/or field measurements. Variations in stiffness result in a nearly proportional change in vibration level while variations in damping and attenuation produce relatively small changes in the results.

Keywords: attenuation, damping, dynamic, elastic half-space, equipment, far field, foundations, frequency, material damping, near field, response, shear modulus, shear wave, stiffness, vibration

David L. Pederson is a consulting engineer at ESI Engineering, Inc., Minneapolis, Minnesota, working in structural engineering, stress analysis, dynamic analysis and vibration control. He holds a Bachelor of Science in Architectural Engineering from Iowa State University, and Master of Science and Doctor of Philosophy Degrees from the University of Minnesota in Civil Engineering. He is a registered civil engineer in Minnesota and a member of the American Society of Civil Engineers.

Anthony J. Baxter is a principal at ESI Engineering, Inc., Minneapolis, Minnesota, working in the areas of dynamic analysis, structural dynamics, measurements, vibration control and noise control. He holds a Bachelor of Science in Mechanical Engineering from Iowa State University, and Bachelor of Arts in Business Administration / Finance from the University of St. Thomas. He is a registered mechanical engineer in Minnesota and Wisconsin.

Carl A. Nelson is a consulting engineer at ESI Engineering, Inc., Minneapolis, Minnesota, working in the design of special structures and foundations, structural dynamics, and finite element analysis. He holds a Bachelor of Science in Civil Engineering, and Master of Science and Doctor of Philosophy Degrees from the University of Minnesota in Structural Engineering. He has been a voting member of ACI Committee 351, Foundations for Equipment and Machinery, since 2009. He is a registered civil engineer in Minnesota, Wisconsin, Iowa, Nebraska, and Virginia.

INTRODUCTION

Structural engineers are often asked to design large concrete foundations supporting heavy, vibrating equipment like that shown in Figure 1, and then to analyze the effects of this vibration throughout a building and/or a neighborhood. In addition, they are often asked to do it under a tight budget and short timeframe. A calculation template is a good way to accomplish this task with some slight modifications tailored to each individual project. This paper discusses the practical aspects of using the elastic half-space theory to compute the vibration of an existing foundation in Ohio subjected to dynamic loading. Mathcad [1], a popular engineering calculation software package, was used to develop the template.

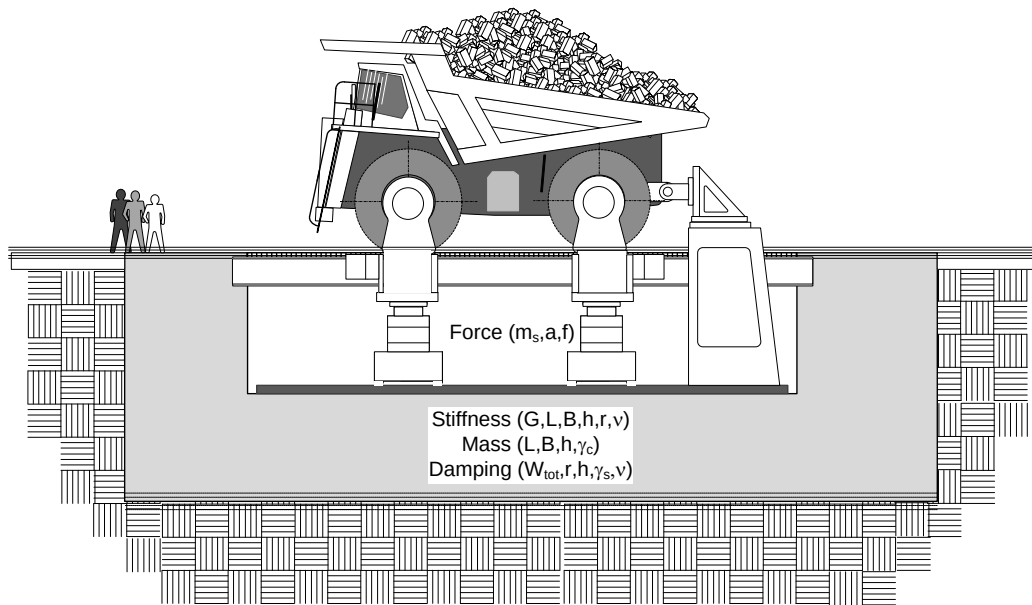


Figure 1—Large rigid concrete equipment foundation supporting a hydraulic test rig for heavy equipment

The dynamic shear modulus is the primary term driving the computations for stiffness of the elastic half-space and is a function of the measured shear wave velocity and soil density. Soils are usually stratified, and the dynamic shear modulus typically increases with depth. Various approaches are discussed to estimate values along with a methodology to establish a single equivalent value for soils with varying shear wave velocities.

Ground or floor vibration at some distance away from the foundation is almost always the main objective of the computation. Excessive vibration can be bothersome to people who are working and living nearby, as well as affect sensitive equipment near the foundation. The energy from a vibrating source disperses in the form of waves that travel

both through the ground and across the surface. The vibration decreases by spreading energy over a greater volume as it travels outward. As the vibration radiates from a foundation, the body waves (compression and shear waves) decay, while the surface wave (Rayleigh wave) develops in what is termed the near field zone. As the surface waves move farther outward into the far field zone, the overall vibration levels continue to decay. This decay, or attenuation of vibration over distance, can be attributed to two components: geometric attenuation and material damping. The material damping coefficient is specific to the soil type and frequency of the dynamic loading. Methods to estimate the coefficient by testing and curve-fitting results are discussed in this paper.

This paper discusses two methods of computing the vibration amplitude as it propagates through the near field into the far field. The first method uses the square root of the sum of the squares (SRSS) of the response at each one-third octave band and an attenuation coefficient at each frequency band, while the hybrid method uses the maximum value of the response and single attenuation coefficient. Results of the analysis compared with floor vibration measurement results are presented for a test-rig foundation project in Ohio.

ELASTIC HALF-SPACE MODEL OF FOUNDATIONS SUBJECTED TO DYNAMIC LOADING

The theory for the elastic half-space model assumes that the soil strata below a rigid foundation is a homogeneous, isotropic elastic body of infinite size, and that the stress-strain properties can be defined by the dynamic shear modulus (G) and Poisson's ratio (ν). Theoretical solutions for the elastic half-space model have been developed by numerous researchers including Lysmer, Richart, Whitman, Hall and Woods, which compute the vibration response of foundations to dynamic loading [2], [3], [4], [5], [6]. The model can be used to predict the response of the foundation to vertical and horizontal forces and rocking and twisting moments applied to the rigid foundation. The model typically has properties of stiffness and damping in five directions that are used to form five uncoupled, single degree of freedom systems with five associated equations of motion (rotation about the vertical axis is commonly ignored). The elastic half-space model dissipates energy, primarily through geometric attenuation, but also due to material damping. In this model the mass is taken to be that of the foundation and equipment vibrating together in phase. None of the mass of the soil is included. Figure 2 shows a model of the multi-degree of freedom system showing the coordinate system, center of force (CF) and center of gravity (CG).

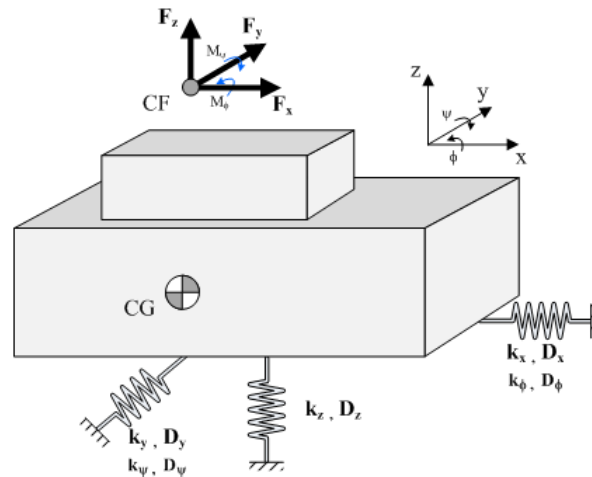


Figure 2—Model of the multi-degree of freedom system (refer to Tables 1 and 2 for symbols)

Most of the equations for stiffness and damping shown in Table 1 and Table 2 for rectangular foundations of length (L), width (B) and depth (h) were adapted from reference [7]. The β -coefficients are geometry factors based on the ratio of L/B as shown in Figure 3. The η -factors are embedment coefficients for the stiffness terms and the α -factors are embedment coefficients for the damping ratios. E_f is the effective embedment of the foundation in the soil, expressed as a ratio of the depth.

W_{tot} is the total weight of the foundation block and equipment, γ_s is the unit weight of the soil, I_ψ and I_ϕ are the mass moments of inertia of the foundation and equipment in the x and y direction with respect to the bottom of the foundation.

Table 1—Stiffness Values for Rigid Rectangular Foundations

Direction of Motion	Equivalent Radius	Embedment Coefficient	Soil/Foundation Stiffness
Horizontal (X-Direction)	$r_x = \left(\frac{BL^3}{3\pi} \right)^{.25}$	$\eta_x = 1 + .55(2 - \nu) \left(\frac{E_f h}{r_x} \right)$	$k_x = 2(1 + \nu) G \beta_x \left(\sqrt{BL} \eta_x \right)$
Horizontal (Y-Direction)	$r_y = \left(\frac{LB^3}{3\pi} \right)^{.25}$	$\eta_y = 1 + .55(2 - \nu) \left(\frac{E_f h}{r_y} \right)$	$k_y = 2(1 + \nu) G \beta_y \left(\sqrt{BL} \eta_y \right)$
Vertical (Z-Direction)	$r_z = \sqrt{\frac{BL}{\pi}}$	$\eta_z = 1 + .6(1 - \nu) \left(\frac{E_f h}{r_z} \right)$	$k_z = \frac{G \beta_z}{1 - \nu} \left(\sqrt{BL} \eta_z \right)$
Rotation - XZ Plane (pitching)		$\eta_\psi = 1 + 1.2(1 - \nu) \left(\frac{E_f h}{r_x} \right) + .2(2 - \nu) \left(\frac{E_f h}{r_x} \right)$	$k_\psi = \frac{G \beta_\psi}{1 - \nu} \left(\sqrt{BL} \eta_\psi \right)$
Rotation - YZ Plane		$\eta_\phi = 1 + 1.2(1 - \nu) \left(\frac{E_f h}{r_y} \right) + .2(2 - \nu) \left(\frac{E_f h}{r_y} \right)$	$k_\phi = \frac{G \beta_\phi}{1 - \nu} \left(\sqrt{BL} \eta_\phi \right)$

Table 2—Damping Values for Rigid Rectangular Foundations

Direction of Motion	Mass Ratio	Damping Embedment Factor	Geometric Damping Ratio
Horizontal (X-Direction)	$B_x = \frac{7 - 8\nu}{32(1 - \nu)} \frac{W_{tot}}{\gamma_s r_z^3}$	$\alpha_x = \frac{1 + 1.9(2 - \nu) \left(\frac{E_f h}{r_x} \right)}{\sqrt{\eta_x}}$	$D_x = \frac{.288}{\sqrt{B_x}} \alpha_x$
Horizontal (Y-Direction)	$B_y = \frac{7 - 8\nu}{32(1 - \nu)} \frac{W_{tot}}{\gamma_s r_z^3}$	$\alpha_y = \frac{1 + 1.9(2 - \nu) \left(\frac{E_f h}{r_y} \right)}{\sqrt{\eta_y}}$	$D_y = \frac{.288}{\sqrt{B_y}} \alpha_y$
Vertical (Z-Direction)	$B_z = \frac{1 - \nu}{4} \frac{W_{tot}}{\gamma_s r_z^3}$	$\alpha_z = \frac{1 + 1.9(1 - \nu) \left(\frac{E_f h}{r_z} \right)}{\sqrt{\eta_z}}$	$D_z = \frac{.425}{\sqrt{B_z}} \alpha_z$
Rotation - XZ Plane	$B_\psi = \frac{3(1 - \nu)}{8} \frac{I_\psi g}{\gamma_s r_x^5}$	$\alpha_\psi = \frac{1 + .7(1 - \nu) \left(\frac{E_f h}{r_x} \right) + .6(2 - \nu) \left(\frac{E_f h}{r_x} \right)^3}{\sqrt{\eta_\psi}}$	$D_\psi = \frac{.15}{(1 + \eta_\psi B_\psi) \sqrt{\eta_\psi B_\psi}} \alpha_\psi$
Rotation - YZ Plane	$B_\phi = \frac{3(1 - \nu)}{8} \frac{I_\phi g}{\gamma_s r_y^5}$	$\alpha_\phi = \frac{1 + .7(1 - \nu) \left(\frac{E_f h}{r_y} \right) + .6(2 - \nu) \left(\frac{E_f h}{r_y} \right)^3}{\sqrt{\eta_\phi}}$	$D_\phi = \frac{.15}{(1 + \eta_\phi B_\phi) \sqrt{\eta_\phi B_\phi}} \alpha_\phi$

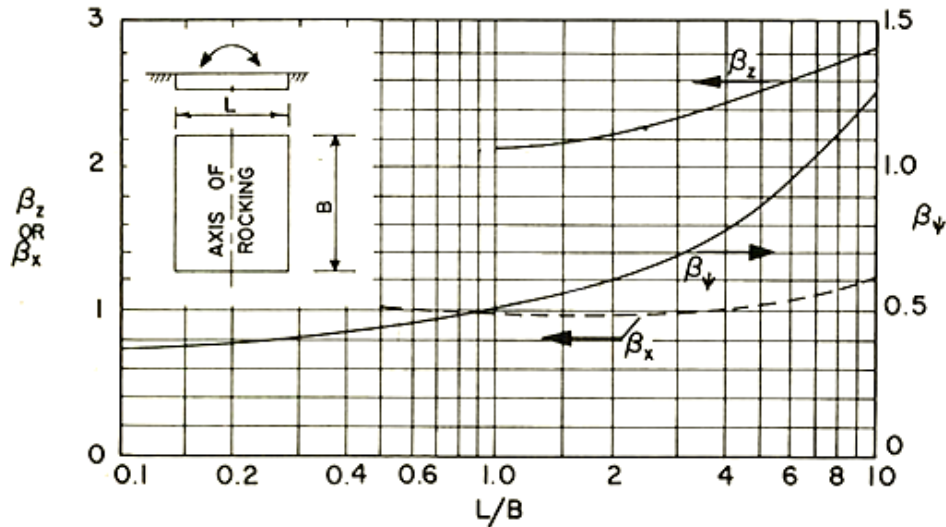


Figure 3—Geometry coefficients [2]

ALTERNATE DAMPING VALUES FOR TRANSLATIONAL MOTION

The damping ratios in Table 2 tend to over-estimate the translational damping of large, rigid foundations. Whitman [5] developed Eq. 1 for horizontal motion and Eq. 2 for vertical motion which combine both geometric and material damping and may be more representative of the total damping.

$$D_x = 0.31\alpha_x \left(\frac{W_{tot}}{r_z^3 \gamma_s} \right)^{-0.5} \quad (1)$$

$$D_z = 0.49\alpha_z \left(\frac{W_{tot}}{r_z^3 \gamma_s} \right)^{-0.5} \quad (2)$$

The presence of rigid rock or very stiff strata at shallow depths beneath the foundation is known to produce a low value of geometric damping, in some cases much lower than those given by Eqs. 1 and 2 because these stiffer layers can reflect body waves back towards the foundation. See Reference [6].

METHODS FOR DETERMINING THE SOIL DYNAMIC SHEAR MODULUS

The dynamic shear modulus (G) is the primary term used in the elastic half-space model to predict the response of foundations subjected to dynamic loading. G is a stiffness term and a function of soil shear wave velocity (v_s) and soil density (γ_s). Often the shear wave velocity and the dynamic shear modulus are unknown and must be estimated or obtained from field testing. The dynamic shear modulus can be calculated directly if the shear wave velocity is known.

Different approaches can be used to either estimate or determine values of G from on-site testing depending on the foundation size, budget and need for accuracy. Some of the considerations are as follows:

- In most cases there has not been a geotechnical survey and “textbook” values need to be used for G during a preliminary analysis.
- If there has been a geotechnical survey but the budget cannot afford tests to determine shear wave and compression wave velocities, then G can be estimated based on the soil types.
- If the foundation is large and the budget allows, a ReMi test, Crosshole test or SCPT test should be conducted to obtain shear wave and compression wave velocities of different soil layers or strata down to 100 feet (30 m) if possible. If both the shear wave and compression wave velocities are known, then Poisson’s ratio can also be calculated directly.

The refraction microtremor (ReMi) test is performed at the ground surface with no need for a borehole and is a means to estimate shear and compression wave velocity profiles. Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (V_s) can then be calculated over a specified interval with depth. Crosshole testing determines the shear wave velocity of soils using two special probes that are pushed into the soil using small machines with hydraulic rams. One probe works as an emitter of the shear waves, the other as the receiver of the waves.

- Shear wave velocities and dynamic shear moduli vary with depth into the soil, generally increasing with depth, and thus an effective, single value must be determined, unless “textbook” values or estimates are used.

EFFECTIVE SHEAR WAVE VELOCITY

If shear wave velocities have been determined for different soil layers, one could use the method of springs in a series to establish an effective shear wave velocity for the analysis. Since the dynamic shear modulus is a stiffness term and a function of shear wave velocity squared and of soil density, then the analogy for the effective shear wave velocity would be of the same form as the effective stiffness k_{eff} for springs in a series as shown in Eq. 3. This approach is similar to the development by Hadjian and Ellison [8] without the weighting of the influence coefficients for stress distribution versus depth.

$$\frac{1}{k_{\text{eff}}} = \frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_3} + \frac{1}{k_4} + \dots \quad (3)$$

The analogy of shear stiffness in a short beam can be applied, which is of the form GA/L where G is the shear modulus, A is the area and L is the beam length. Since area A is the same for all soil layers and soil density is similar for all layers, these can be factored out of the formulation. The depth of each layer or test interval (L_i) is used to calculate the shear wave velocities at each layer or test interval and since the depth of the soil layers or test interval are analogous to the beam lengths, the equation for effective shear wave velocity in a series is as shown in Eq. 4.

$$v_{s,\text{eff}} = \sqrt{\frac{\sum L_i}{\sum \frac{L_i}{v_{s_i}^2}}} \quad (4)$$

And the effective dynamic shear modulus may be computed as shown in Eq. 5.

$$G = \frac{\gamma_s}{g} v_{s,\text{eff}}^2 \quad (5)$$

EXISTING FOUNDATION WITH REMI TEST DATA

Shear wave velocity data was obtained using the ReMi Test on the soil beneath a large foundation in Ohio supporting a hydraulic test rig for vibration testing of large off-road vehicles. The data was collected at one-meter intervals down to 21 m, or 69 ft. Two locations (ST 101 and ST 102) were tested, and the effective shear wave velocity ($v_{s,\text{eff}}$) was computed for each location using Eq. 4. The average of the two was used to compute a single, effective dynamic shear modulus for the foundation analysis. The unit weight of the soil (γ_s) was 120 pcf (18.9 kN/m³). Shear wave velocities at one-meter intervals for both locations are shown in Table 3. The compression wave velocities ($v_{p,\text{eff}}$) were not recorded and thus the Poisson's ratio (ν) was not obtained from testing directly, but was assumed to be 0.4 based on the soil type. The calculated effective dynamic shear modulus was 17,205 psi (117 MPa).

SHEAR WAVE VELOCITY BASED ON SOIL TYPE

In the beginning stages of most building/foundation projects, a geotechnical survey will be completed for the building enclosure foundation and slab-on-grade. Generally, the report will include no information on the shear wave velocity or the dynamic shear modulus. The values shown in Table 4 can be used to estimate the dynamic shear modulus for the soil type characteristics described in the geotechnical report.

Table 3—Shear Wave Velocities in Ohio From Remi Tests

Depth		Shear Wave Velocity - feet/sec			
		Location ST 101		Location ST 102	
feet	(m)	ft/sec	(m/s)	ft/sec	(m/s)
3.28	1.0	570	174	500	152
6.56	2.0	795	242	710	216
9.84	3.0	960	293	720	219
13.12	4.0	905	276	1150	351
16.41	5.0	825	251	960	293
19.69	6.0	750	229	880	268
22.97	7.0	640	195	780	238
26.25	8.0	905	276	715	218
29.53	9.0	805	245	775	236
32.81	10.0	880	268	820	250
36.09	11.0	900	274	825	251
39.37	12.0	950	290	940	287
42.65	13.0	860	262	905	276
45.93	14.0	790	241	760	232
49.22	15.0	800	244	760	232
52.50	16.0	820	250	805	245
55.78	17.0	740	226	745	227
59.06	18.0	990	302	860	262
62.34	19.0	965	294	930	283
65.62	20.0	875	267	925	282
68.90	21.0	1200	366	1350	411
V_{eff}		823		802	
Average v_{eff}		813			

Table 4—Range of Values for Dynamic Shear Modulus for Soils

General Soil Types	Shear Modulus		Poisson's Ratio	Density		Ref.
	ksi	(MPa)		pcf	(kN/m ³)	
Soft Clay	3 - 5	21-34				[7]
Stiff Clay	10 - 20	68-136				[7]
Very Stiff Clay	> 20	>136				[7]
Medium Dense Sand	5 - 15	34-103	0.30 - 0.40	120	18.9	[7]
Dense Sand	10 - 20	68-136	0.40 - 0.50			[7]
Medium Dense Gravel	15 - 25	103-172	0.30 - 0.40			[7]
Dense Gravel	20 - 40	136-272	0.40 - 0.50			[7]
Specific Cases						
Med Round-Grained Sand	12.5 - 14.1	86-97		117	18.4	[7]
Med Dense Silty Sand w/Gravel	14	97	0.35	117	18.4	[7]
Medium Firm Clay	6.5	45	0.45	125	19.6	[7]
Soft Silty Clay	3.5	24	0.35	110	17.3	[7]
Saturated Clay	14.3 - 16.3	99-112	0.45 - 0.50	125	19.6	[7]
Project Site Tests						
Poorly Graded Sands	15.7	108	0.21	115	18.1	[9]
Poorly Graded Silty/Sands	27.6	190	0.32	115	18.1	[10]

DYNAMIC CHARACTERISTICS OF THE FOUNDATION

The dimensions of the rectangular concrete foundation in Ohio were: $L = 36$ ft (11 m), $B = 30$ ft (9.1 m) and $h = 6$ ft (1.8 m). Using the value of 17,105 psi (117 MPa) for the dynamic shear modulus and a Poisson's ratio of 0.40, the stiffness and damping values in each direction using the equations in Tables 1 and 2 are summarized in Table 5. An effective embedment factor (E_f) of 0.95 was assumed. Soil density (γ_s) was estimated at 120 pcf (18.9 kN/m³). The total foundation plus equipment weight was taken equal to 960 kips (4270 kN). The geometry coefficients were determined by means of interpolation in Figure 3.

Whitman's alternate damping formula [5], Eq. (2), was used to determine the value of the damping ratio ($D_z = 0.561$) shown in Table 5. This is an important point to note, as the equation for this quantity given in Table 1 yields a damping ratio greater than 1.0. This represents a highly overdamped system, and a system response that may be unconservatively low.

Table 5—Stiffness, Damping and Frequency Values of the Ohio Foundation

Direction of Motion	Equivalent Radius (r)		Geom. Coeff. (β)	Embed Coeff. (η)	Stiffness (k)		Damping Embed Factor (α)	Geom. Damping Ratio (D)	Natural Freq. Hz (f_n)
	feet	(m)			kip/ft	(kN/m)			
Horiz.- X-Axis	18.54	5.65	0.983	1.271	283,000	4,130,000	1.716	0.992	15.50
Horiz.- Y-Axis	18.54	5.65	0.999	1.271	287,600	4,197,000	1.716	0.992	15.50
Vertical - Z-Axis	18.54	5.65	2.164	1.111	324,300	4,732,000	1.281	0.561	16.59
Rotation - XZ Plane	19.63	5.98	0.537	1.217	104,309,000 kip-ft/rad	141,424,000 kN-m/rad	1.038	0.504	28.55
Rotation - YZ Plane	17.92	5.46	0.484	1.239	79,824,000 kip-ft/rad	108,227,000 kN-m/rad	1.046	0.473	29.79

FOUNDATION DYNAMIC RESPONSE

The vibration response of the foundation to dynamic loading can be computed in the frequency domain by means of the well-known relationship shown in Eq. 6 [18].

$$\mathbf{Z}(\omega_j) = \mathbf{F}(\omega_j)\mathbf{H}(\omega_j) \quad (6)$$

where $\mathbf{F}(\omega_j)$ is the input force amplitude as a function of frequency, $\mathbf{H}(\omega_j)$ is the complex frequency response function (FRF), and $\mathbf{Z}(\omega_j)$ is the response, a complex function of frequency that includes both magnitude and phase. Since the vibration criteria are evaluated in the frequency domain, the phase information is not relevant and only the FRF magnitude is of interest.

For the Ohio project, the input forces were provided by the owner for each hydraulic actuator. Figure 4 shows an example of the input force for one of the vertical actuators. A section of the data where the forces were most severe was selected and converted into the frequency domain using the fast Fourier transform (FFT) method with a bandwidth of 1 Hz. The resultant spectrum for the total vertical force is shown in Figure 5. A sharp, dominant peak was evident at 10 Hz.

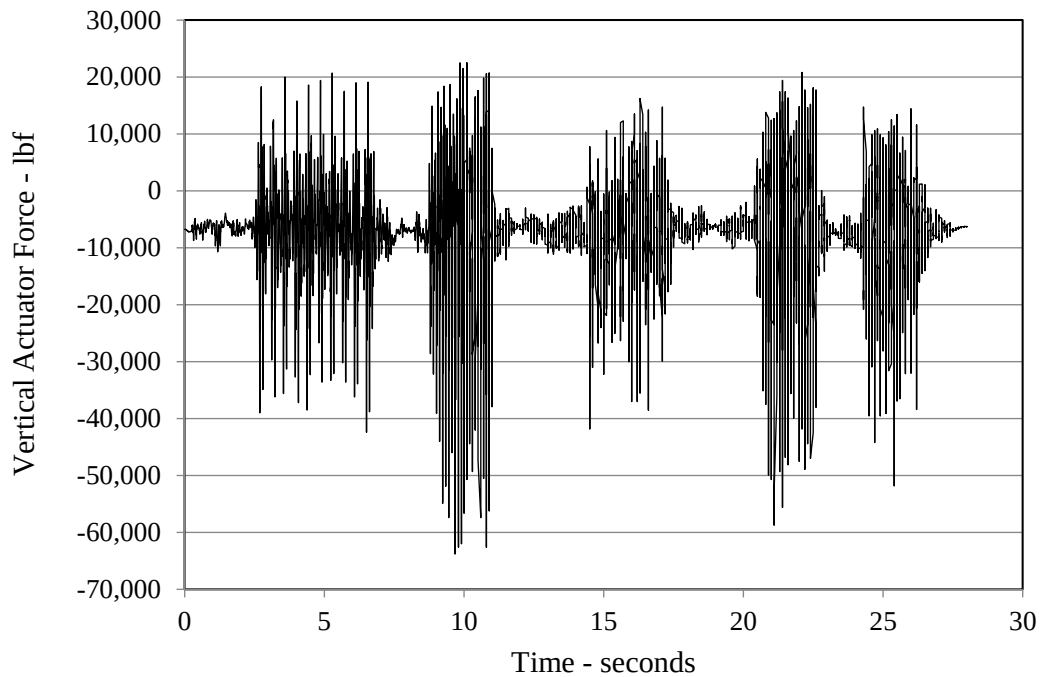


Figure 4—Example of force data for one of the vertical hydraulic actuators (1 lbf = 4.448 N)

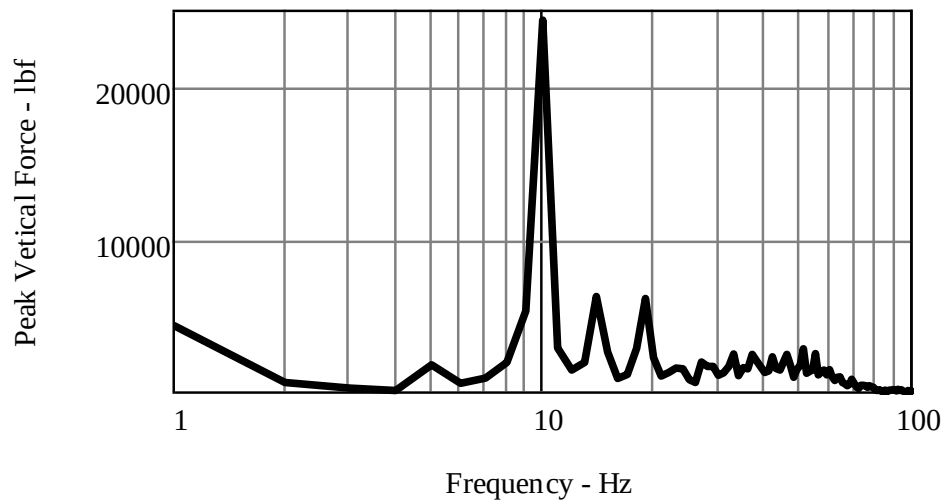


Figure 5—Narrow band spectrum of total vertical force reacted by the foundation (1 lbf = 4.448 N)

The magnitude of the frequency response function (H_j), i.e., $|H(\omega_j)|$ is denoted by $|H_j|$. Assuming the equations of motion for translation and rotation are uncoupled, the general frequency response function magnitude is shown in Eq. 7 for a single degree of freedom system [18]. Horizontal and rocking motions are usually coupled and thus an investigation should be completed to determine if an uncoupled model is justified.

$$|H_j| = \frac{1}{k \sqrt{\left[1 - \left(\frac{f_j}{f_n}\right)^2\right]^2 + \left[2D \frac{f_j}{f_n}\right]^2}} \quad (7)$$

A plot of the frequency response functions for the three translational axes for the Ohio case are shown in Figure 6. The three curves were obtained by application of Eq. (7) for the two translational and one rotational degrees of freedom considered, with the use of the values shown in Table 5, where f_n refers to the natural frequency. The amplitudes have been normalized by multiplying by the applicable stiffnesses. A small amplification of the vertical response is observed around 10 Hz. The horizontal x and y response functions (both translational and rotational) were very similar because of their nearly identical frequency and damping values. A two degree-of-freedom model of coupled horizontal and rocking motion in each direction was investigated. The results showed the uncoupling assumption to be justified for this case because of the wide frequency separation and high degree of damping for each mode.

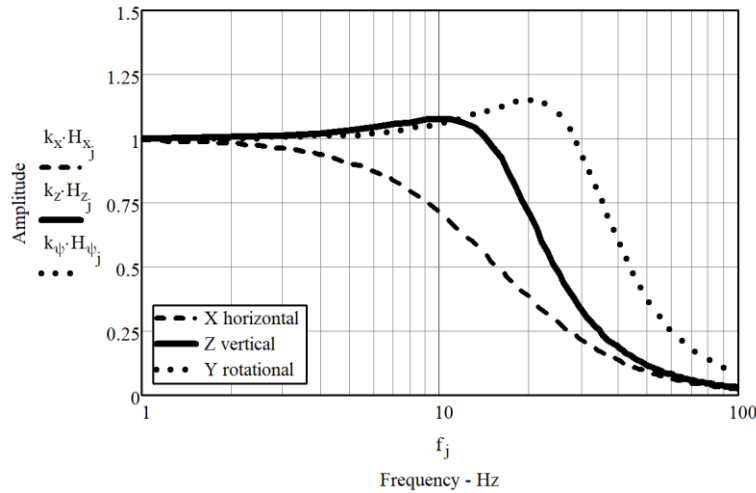


Figure 6—Frequency response functions for Ohio foundation

Eq. 8 through Eq. 12 below show the implementation of Eq. (6) for computing displacement and rotation due to vertical forces and moments.

Displacement:

$$Z_{zd_j} = P_{z_j} |H_{z_j}| \quad (8)$$

Rotation (pitching):

$$\psi_j = M_{\psi_j} |H_{\psi_j}| \quad (9)$$

Edge displacement from rotation:

$$\zeta_j = 0.5L\psi_j \quad (10)$$

Total displacement at edge:

$$Z_j = Z_{d_j} + \zeta_j \quad (11)$$

Total velocity at edge:

$$V_{cz_j} = 2\pi f_j Z_j \quad (12)$$

In the above equations, H_z and H_ψ are the translational and rotational frequency response functions evaluated at each frequency band (j). P_z and M_ψ are the input vertical force and pitching moment acting on the foundation. In the case of the Ohio foundation, the input forces were mainly in the vertical direction and aligned with the center of stiffness of the foundation. Thus, the displacement and velocity response were mostly vertical with a very small amount of rotation.

The human body is a good integrator of vibration, and people sense frequency changes in the broader one-third octave bands as opposed to narrow bands. People also tend to perceive average or rms levels as opposed to peak values. Hence the narrow band vibration responses obtained from Eq. 8 through Eq. 12 were differentiated (multiplied by ω_j) to obtain velocity and converted to one-third octave band rms levels. The resulting velocity response is compared to common vibration criteria for human exposure in Figure 7. As expected, the dominant response occurs in the 10 Hz one third octave band.

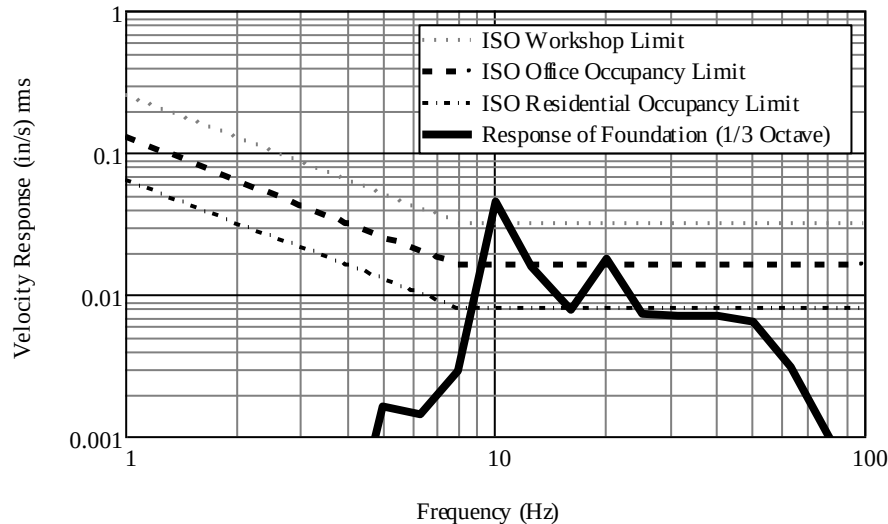


Figure 7—RMS velocity response in vertical direction at edge of Ohio foundation (1 in/sec = 25.4 mm/s)

ACCEPTABLE CRITERIA FOR HUMAN COMFORT

Subjective assessment of floor vibration in a home or office environment is difficult to characterize because people receive vibration through their feet, their seat, from visual cues and sounds all at the same time. In extreme cases the vibration level may be exacerbated by objects shaking and rattling. Furthermore, tolerance levels vary widely from person to person. Someone working in an environment with lots of movement and sound around them will tolerate more vibration than someone working or living in a normally still environment.

Research organizations have established various comfort criteria for people subjected to vibration. Results of work on an international level led to development of a set of standards published by the International Organization for Standardization (ISO). The standards ISO 2631/1 [11], 2631/3 [12] and ISO 6897 [13] give an evaluation of human exposure to whole-body vibration. The American Standards Institute (ANSI) in conjunction with the Acoustical Society of America have developed and published a similar set of standards, ANSI S3.29-1983 [14]. The general recommendations from these standards for various occupancies are summarized in Table 6 for frequencies above 8 Hz. Permissible levels are two times greater at 4 Hz and decrease linearly (log-log scale) to the levels shown in the table between 4 and 8 Hz.

The criterion values may be used to evaluate the effects of mechanical disturbances that occur at a single frequency, or for several widely separated frequencies, and for disturbances at multiple, closely spaced frequencies. However, the criterion values apply to disturbances observed in one-third octave bands, rather than at single frequencies [15]. As such, the values in the table are rms velocity values, not 0-peak values.

Table 6—Vibration Criteria for Human Comfort

Facility Use	Vibration Sensitivity	Permissible Vibration	
		$\mu\text{in/sec rms}$	$(\mu\text{m/sec}) \text{ rms}$
Workshops (ISO/ANSI Criteria)	Distinctly felt by people	32,000	800
Offices (ISO/ANSI Criteria)	Vibration is felt by people	16,000	400
Residential Daytime (ISO/ANSI Criteria)	Barely felt by people	8,000	200

VIBRATION ATTENUATION WITH DISTANCE

The vibration emitted from the foundation at some distance away is almost always a concern to building owners, occupants and neighbors. The vibration can affect people who are living and working nearby or sensitive equipment near the foundation.

When a foundation vibrates due to equipment or process forces acting on it, the vibration energy disperses in a form of waves that propagate through the ground and into the building. The vibration amplitude decreases by spreading energy over a greater volume of the ground. It is known that the waves that travel along the surface of the ground are due to Rayleigh wave (R-wave) energy which starts at zero at the source of the vibration and grow in the near field zone as body waves (P-wave and S-wave) are transferring energy to the R-waves. After some distance in the far field, the R-wave becomes fully developed, the body waves diminish, and the R-wave amplitude decreases as it moves farther outward. This concept is shown in the Figure 8.

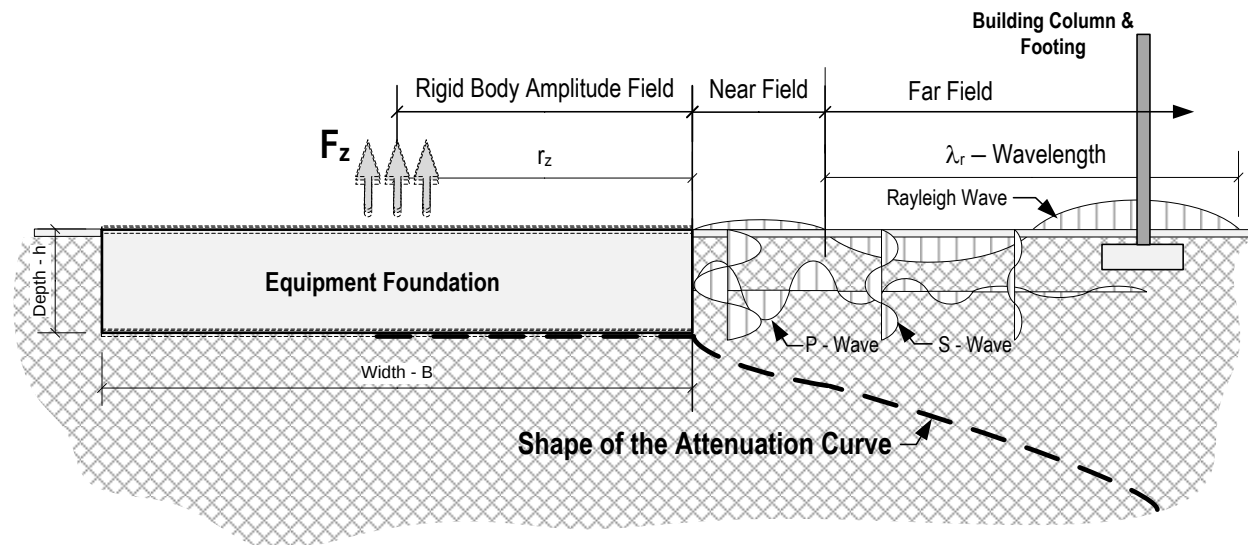


Figure 8—Vibration attenuation through near and far fields

Near the vibrating foundation, amplitudes do not follow the same attenuation law as they do farther away from the source in the far field. The far field is defined as the distance r from the vibration source beyond which ground vibrations are mainly dominated by R-waves. The near field is the distance between the vibrating source (edge of foundation) and the inner boundary of the far field (r_{ff}).

A study completed by Gao, Song and Yang [16] concluded that for the case of a point load, the boundary is located at the distance (r) of $5-6 \lambda_r$ for $\nu \leq 0.30$ and at the distance of $2-3 \lambda_r$ for $\nu \geq 0.35$, where λ_r is the Rayleigh wavelength and ν is Poisson's ratio. For the case of an infinite line load, the boundary is located at the distance $5.5-6.5 \lambda_r$ for $\nu \leq 0.30$ and at the distance $2.5-3.5 \lambda_r$ for $\nu \geq 0.35$. Other researchers (Macijauskas [17]) have proposed the start of the far field at distances of $0.5-1.0 \lambda_r$. In the Ohio case the far field appeared to start at $1.0 \lambda_r$ based on field testing.

It is important to distinguish these regions since geometric damping accounts for most of the attenuation in the near field, while both geometric and material damping account for vibration attenuation in the far field. The general shape of the attenuation curve is shown in the Figure 8 above. In the near field the attenuation is mostly attributed to geometric damping described by Eq. 13 [2], [16], [17] where V_{ref} is the vibration at the edge of the foundation (distance r_{ref}) and V_{gm} is at a distance r from the source or center of the foundation. The geometric damping coefficient (n) varies between 0.5-1.0 for sources like a single foundation, which can be approximated as a point source.

$$V_{gm}(r) = V_{ref} \left(\frac{r_{ref}}{r} \right)^n \quad (13)$$

The attenuation curve (in velocity units) used to represent geometric and material damping is shown in Eq. 14 [2], [16], [17]. The r_{ff} term was not factored out of Eq. 14 to illustrate similarity to Eq. 13.

$$V_{gm}(r) = V_{ref} \left(\frac{r_{nf}}{r_{ff}} \right)^n \left(\frac{r_{ff}}{r} \right)^n \exp[-\alpha(r - r_{ff})] \quad (14)$$

The material damping is represented by the coefficient of attenuation term (α) which is specific to the site, soil type and frequency of the dynamic response. One method to determine the coefficient by field testing is to use dropped weights, measuring vibration at radial distances away from the drop and then curve-fitting the results to determine the coefficient. Figure 9 shows a photograph of a concrete mass being dropped from forks of a skid-steer and an example of resulting vibration measured in the soil at distances from 40 ft (12 m) to 200 ft (61 m). The rms response for multiple drops can be averaged at each distance, and the coefficient determined by curve fitting Eq. 14 to the data using the method of least squares.

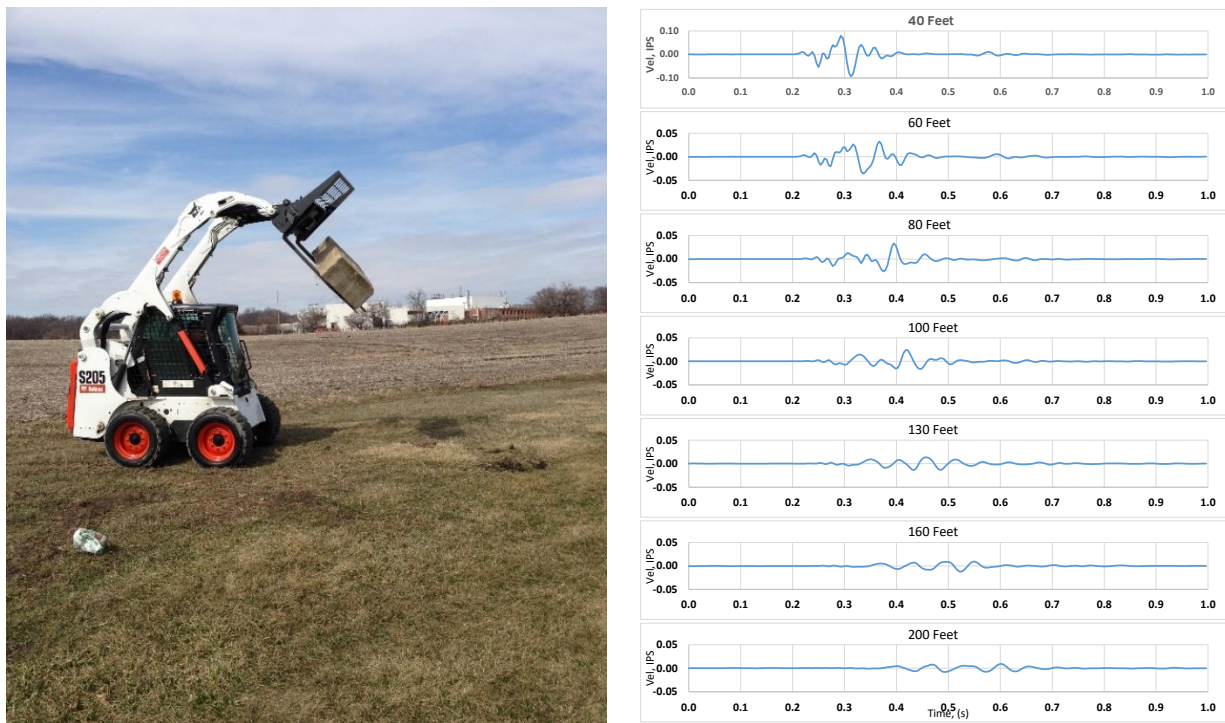


Figure 9—Photograph showing one drop test method and the measured ground vibration at various distances (1 in/sec = 25.4 mm/s, 1 ft = 0.305 m)

Forced vibration data can also be used to determine the material damping coefficient. This was done for the Ohio case for the existing equipment foundation. The α -term was determined to be 0.0043/ft (0.014/m) at 9.1 Hz and assumed to be directly proportional to frequency. Thus, higher frequency vibration has greater material damping.

Two methods of computing the attenuation through the near field into the far field can be used. In general, when the vibrating source (forcing function) has broad band frequency content, a method using square root of the sum of the squares (SRSS method) of the response at each one-third octave band frequency should be used as shown in Figure 10. When the vibrating source has a dominant narrow band frequency, a hybrid method using the α -term at the dominant frequency can be used as shown in Figure 11. For both methods, the vibration amplitude is assumed to be constant out to the edge of the foundation.

Note that the vibration levels have attenuated to the ISO office criteria at 70 feet (21 m) from the center of the foundation using the SRSS method and at less than 60 feet (18 m) away using the hybrid method.

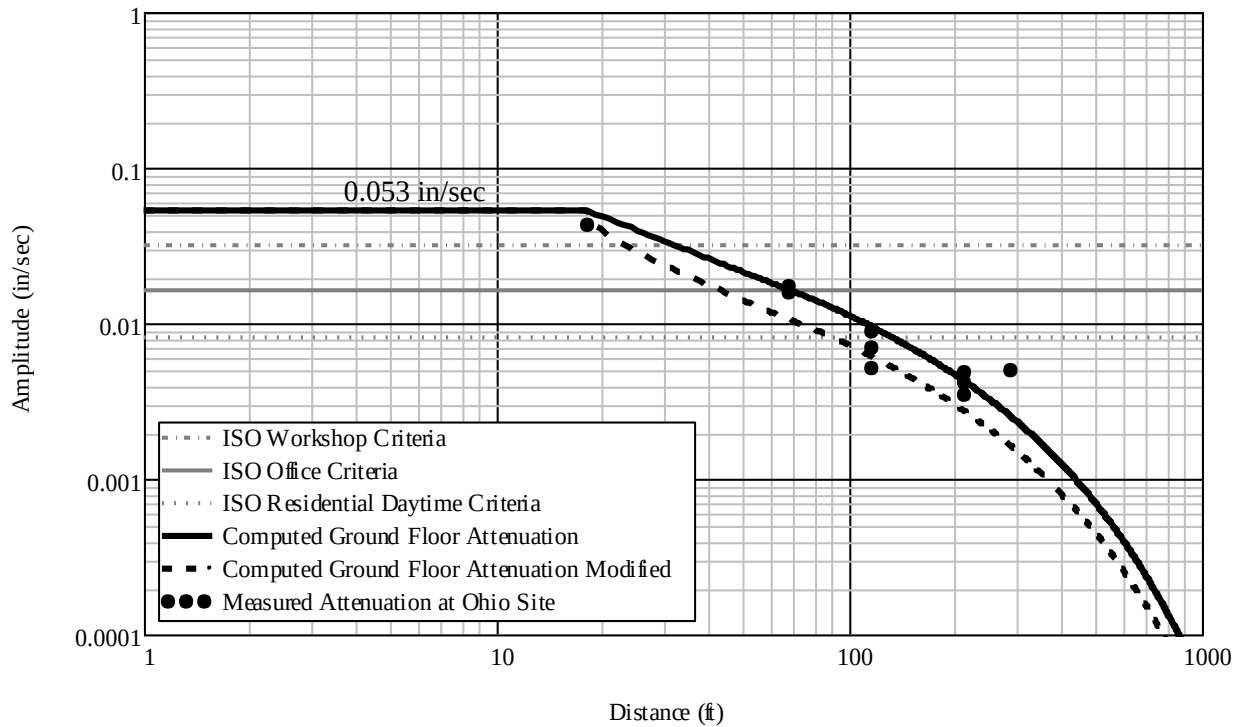


Figure 10—SRSS method for attenuation using RMS velocity and alpha at each 1/3-octave band frequency (1 in/sec = 25.4 mm/s)

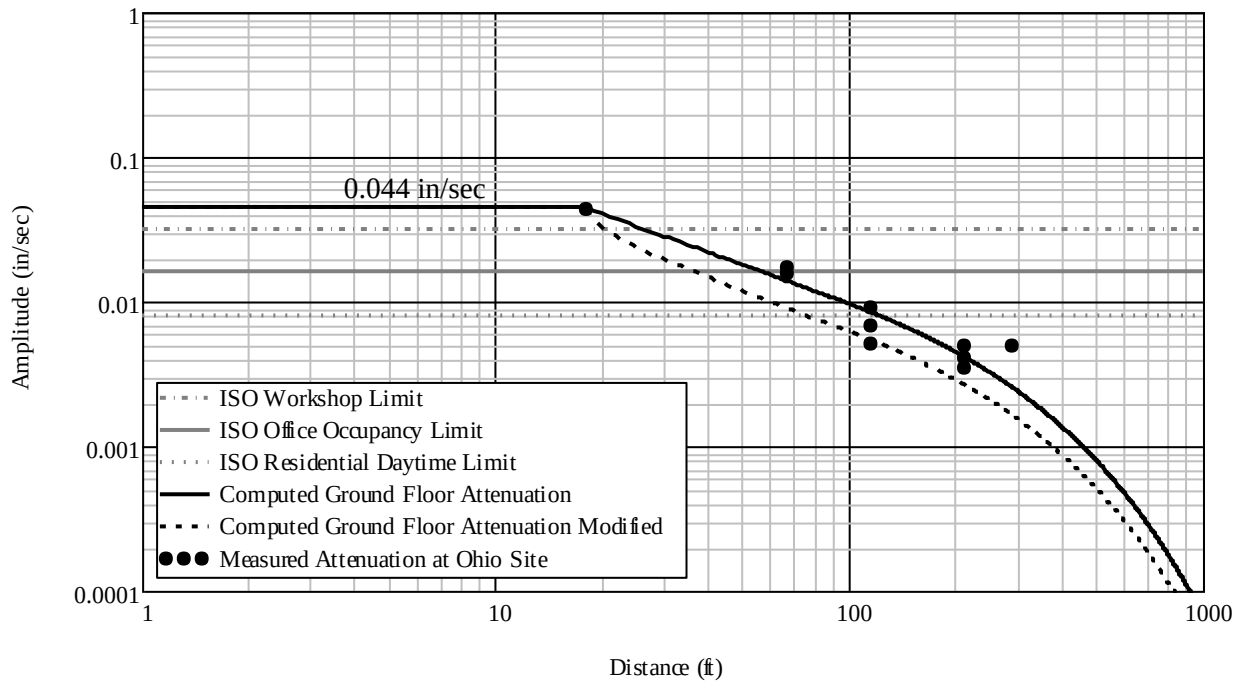


Figure 11—Hybrid method for attenuation using max velocity and alpha at frequency of max velocity (1 in/sec = 25.4 mm/s)

MODIFIED METHOD OF VIBRATION ATTENUATION

Macijauskas [17] has shown that the attenuation in the near field drops off more sharply starting at the edge of the foundation. His test data and FEA models show the attenuation in the near field follows an arcsine function of geometric damping as shown in Eq. 15.

$$V_{gm}(r) = V_{ref} \frac{2}{\pi} \arcsin\left(\frac{r_{ref}}{r}\right)^n \quad (15)$$

These curves are shown for comparison as “Computed Ground Floor Attenuation Modified” in Figures 10 and 11.

COMPARISON OF FIELD TESTS TO COMPUTATIONS

Floor vibration tests were conducted at the Ohio site along three radial lines at multiple distances away from the foundation. The building facility houses several other large machines and equipment which were not in operation during the tests. Table 7 shows a comparison of the measured data to the computed values using the SRSS method and the Hybrid method. Both methods show the Normal and the Modified case with the arcsine function of the square root of the reciprocal distance. The Modified method is seen to significantly under-predict the measured vibration, and thus should be used with caution until further experimental validation can be carried out.

There is a wider than expected variation in the measured values along the three radial lines, particularly at 115 ft (35 m) and 212 ft (65 m) away from the foundation. This may be due to differences in the soil strata and/or other footings and foundations. Underground piping, conduit and drainage systems could also act as a transmitter or barrier of the floor vibration.

Table 7—Comparison of Measured Versus Computed Attenuation

Locations			Measured Velocity		SRSS Method				Hybrid Method			
Distance		Line			Normal		Modified		Normal		Modified	
feet	(m)		in/sec	(m/s)	in/sec	(m/s)	in/sec	(m/s)	in/sec	(m/s)	in/sec	(m/s)
1	0.30	a	0.0430	0.00109	0.0530	0.00135	--	--	0.0444	0.00113	--	--
67	20.42	a	0.0155	0.00039	0.0164	0.00042	0.0106	0.00027	0.0138	0.00035	0.0089	0.00023
		b	0.0173	0.00044								
115	35.05	a	0.0052	0.00013	0.0095	0.00024	0.0062	0.00016	0.0085	0.00022	0.0054	0.00014
		b	0.0089	0.00023								
		c	0.0090	0.00023								
212	64.62	a	0.0034	0.00009	0.0042	0.00011	0.0027	0.00007	0.0041	0.00010	0.0027	0.00007
		b	0.0041	0.00010								
		c	0.0049	0.00012								
288	87.78	a	0.0050	0.00013	0.0024	0.00006	0.0016	0.00004	0.0026	0.00007	0.0017	0.00004

COMPARISON OF RESULTS WITH VARIATIONS IN STIFFNESS AND DAMPING

The vibration computed on the foundation was compared for variations in stiffness, damping and α -term to evaluate the sensitivity to error in field measurement and/or in calculation. Only the Normal cases are discussed for both the SRSS and Hybrid methods. Table 8 shows the sensitivity to changes in stiffness and/or possible error in establishing the effective shear modulus, with baseline values shown in bold type. There is nearly a direct proportion to the percent change in stiffness.

Table 8—Comparison of Results with Variation in Stiffness

Shear Modulus			SRSS Method			Hybrid Method		
psi	(MPa)	% Chg	in/sec	(m/sec)	% Chg	in/sec	(m/sec)	% Chg
12,829	84.730	-25%	0.0669	0.0017	26%	0.0588	0.0015	32%
17,105	117.935	--	0.0530	0.0013	--	0.0444	0.0011	--
21,381	147.417	25%	0.0441	0.0011	-17%	0.0354	0.0009	-20%

Table 9 shows the sensitivity to changes and/or possible error in computing the effective damping, with the baseline values shown in bold type. Vibration on the foundation in this specific case was more sensitive to stiffness than damping due to the peak dominating input force being somewhat lower in frequency than the natural frequency of the foundation, i.e., in the stiffness-dominated, flat region of the frequency response. However, in general, damping would be more of a controlling factor.

Table 9—Comparison of Results with Variation in Damping

Damping		SRSS Method			Hybrid Method		
%	% Chg	in/sec	(m/sec)	% Chg	in/sec	(m/sec)	% Chg
0.421	-25%	0.0622	0.0016	17%	0.0507	0.0013	14%
0.561	--	0.0530	0.0013	--	0.0444	0.0011	--
0.701	25%	0.0460	0.0012	-13%	0.0390	0.0010	-12%

Table 10 indicates less sensitivity to changes in the α -term than to changes in stiffness or damping. For example, at 115 ft (35 m) away, a 25% change in alpha results in a 2% to 5% change in the vibration amplitude and a 100% change results only 13% to 15%. At greater distances, the sensitivity to α is greater since the attenuation due to material damping becomes more dominant with increasing distance from the source. When the response at locations far from the source is of primary interest, a conservative (low) value of α should be used. In all cases, a good prediction of the response at the foundation itself is essential. Once this is obtained with confidence, a reasonable prediction of the response at a distance can be made using the methods described above.

Table 10—Comparison of Vibration Attenuation at 115 Feet with Variation in Alpha Term

Base α Alpha Term			SRSS Method			Hybrid Method		
ft ⁻¹	(m ⁻¹)	% Chg	in/sec	(m/sec)	% Chg	in/sec	(m/sec)	% Chg
0.0022	0.0072	-50%	0.0105	0.00028	11%	0.0090	0.00023	6%
0.0032	0.0105	-25%	0.0100	0.00025	5%	0.0087	0.00022	2%
0.0043	0.0141	--	0.0095	0.00024	--	0.0085	0.00022	--
0.0054	0.0178	25%	0.0091	0.00023	-4%	0.0082	0.00021	-4%
0.0086	0.0282	100%	0.0081	0.00021	-15%	0.0074	0.00019	-13%

CONCLUSIONS AND RECOMMENDATIONS

Engineering firms designing equipment foundations are often faced with the need to predict vibration amplitude at receivers who are both near and far away from the foundation. This paper shows a methodology for determining this information using a calculation template. Once all the elastic half-space equations, loading functions, dynamic response and attenuation equations are set up, results can be obtained quickly by entering soil information, foundation sizes and loading data.

The foundation damping values used in the analysis are of critical importance, and care must be exercised in determining these values. The widely published geometric damping ratios tend to over-estimate the translational damping for large, rigid foundations. The dynamic shear modulus is usually unknown and must be estimated from the information in the geotechnical survey or obtained from field testing. The dynamic shear modulus can be calculated directly if the shear wave velocity is known. Generally, soils are stratified in depth and an equivalent value needs to be determined. The method of springs in series is one approach to establish the G value. This is an area for more research since it is unclear if the weighting of influence coefficients for stress distribution versus depth is a more accurate way to determine the dynamic shear modulus.

Vibration amplitudes do not follow the same attenuation rates near the foundation as they do farther away, where ground vibrations are mainly dominated by Rayleigh waves. In the near field, geometric damping dominates the attenuation while both geometric and material damping account for vibration attenuation in the far field. The material damping is represented by the coefficient of attenuation term (α) and is specific to the site, soil type and frequency of the dynamic loading. When the vibrating source has a broad band frequency content, the method using square root of the sum of the squares (SRSS method) of the response at each one-third octave band frequency should be used to compute the α value. When the vibrating source has a dominant frequency, a hybrid method using the α -term at the dominant frequency can be used for computational efficiency.

A modified method of vibration attenuation in the near field using an arcsine function has been proposed but has not been evident in the field measurements of large equipment foundations thus far. More extensive measurements close to the foundation are required to demonstrate this effect. This is recommended as another area for future research.

In a sensitivity analysis of selected parameters, it was found that changes in stiffness from possible error in establishing the effective shear modulus result in nearly direct proportion in the results for velocity response. This is less true for changes to damping. There is lower sensitivity to changes in the α -term for attenuation in the far field than to changes in stiffness or damping. A large change in the α -term ($\pm 100\%$) results in a small change in the vibration in the far field.

The methodology discussed in this paper has been shown to be useful for computing vibration at the foundation and computing the decay in vibration amplitude with distance. With the growing number of people working and living close to facilities of all kinds that produce ground vibration, the accurate prediction of ground-borne vibration propagation will become increasingly important over the next few decades and promises to be a fruitful area for research.

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Innovative Retrofit of a Steam Turbine Generator Foundation to Mitigate Settlement

Hongchun Liu, Jaspal S. Saini, Gang Zhao, Sushil Chauhan, Namgyu Park, Mahi Galagoda, Steven Wu

Synopsis: This paper discusses an innovative retrofit that stabilized a Steam Turbine Generator (STG) pedestal foundation undergoing unexpected differential settlements during construction. The innovative solution involved driving steel H-piles around the STG foundation perimeter. A new concrete bracket (a.k.a. corbel) was added around the STG foundation perimeter to fully engage and integrate the H-piles with the existing pedestal foundation. The pile layout was established and optimized based on dynamic and static performance analyses of the modified foundation geometry using finite element (FE) software ANSYS, considering bounding pile and soil dynamic impedances. The frequency-dependent dynamic pile impedances were calculated using DYNA6. The continuous settlement monitoring of the STG foundation demonstrated that the retrofit effectively seized the ongoing settlements and stabilized the foundation enabling the subsequent machine installation.

Keywords: steam turbine generator foundation, foundation retrofit, foundation settlement, piles, dynamic analysis, frequency-dependent dynamic impedances, soil impedances, pile impedances.

Hongchun Liu, Senior Engineering Specialist, Bechtel Distinguished Technical Specialist, Bechtel Infrastructure & Power Corp.

ACI member **Jaspal S. Saini** is a senior structural engineering specialist at Bechtel Infrastructure & Power. He is a member of Joint ACI-ASME Committee 359 (Concrete Containments for Nuclear Reactors) and an associate member of ACI Committee 349 (Concrete Nuclear Structures) and Joint ACI-ASCE Committee 447 (Finite Element Analysis of Reinforced Concrete Structures). He is also a member of ASCE Working Group on Dynamic Analysis of Nuclear Structures. He is a licensed Professional Engineer in California, Virginia, and Arizona.

Gang Zhao is a senior structural engineer with Bechtel Nuclear, Security & Environmental Inc, with over 20 years of experience. He received his Master's Degree in civil-structural engineering from North Carolina State University. He is a licensed Professional Engineer in Virginia.

Sushil K. Chauhan is a senior structural engineer with Bechtel Oil Gas & Chemicals, with over 20 years of experience. He received his Master's Degree in structural engineering & geomechanics from Stanford University. He is a member of Joint ACI-ASCE Committee 550 (Precast Concrete Structures). He is a licensed Structural Engineer in California.

Park Namgyu, Senior Geotechnical Engineering Specialist, Bechtel Oil, Gas and Chemicals Corp.

Mahi Galagoda, Geotechnical Engineering Lead, Bechtel Oil, Gas and Chemicals Corp.

Steven Wu is the Civil and Structural Team Lead for Shell Pennsylvania Chemicals Project and a Principal Engineer in Shell Projects & Technology. He received his PhD in civil-structural engineering from the University of Illinois at Urbana-Champaign. He is a technical authority & subject matter expert at Shell and has more than 30 years of professional experience in delivering onshore and offshore capital projects.

INTRODUCTION

At an under-construction process plant, unexpected excessive differential settlements of the Steam Turbine Generator (STG) foundation were observed, based on the settlement data collected at the four corners of the basemat as part of the routine monitoring program, after the foundation had been constructed but prior to the installation of the machine.

In the original design, the pedestal foundation was supported on soil without piling. To arrest further settlements and stabilize the pedestal foundation, a structural retrofit scheme was required for all design loads with high degree of confidence and certainty.

The innovative solution involved driving steel H-piles (HP14×73) around the STG foundation perimeter and additional piles driven through the existing concrete foundation in the condenser area. New concrete bracket (a.k.a. corbel) was added around the STG foundation perimeter to fully engage and integrate the H-piles with the existing foundation. The pile layout was established and optimized based on dynamic and static performance analyses of the modified foundation geometry using finite element (FE) software ANSYS [5], considering bounding pile and soil impedances. The frequency-dependent dynamic pile impedances were calculated by using DYNA6 [6]. Pile embedment requirements and allowable lateral and axial capacities, with appropriate reduction factors as applicable, were determined by geotechnical evaluations. Reinforcement connecting the new bracket and existing STG foundation was designed to transfer all design loads from the existing STG foundation to the new bracket, then to the new H-piles supporting the bracket.

Constructability, including pile installation and utilization of mechanical rebar couplers, was extensively accounted for during the retrofit design phase to ensure smooth execution. Settlements and vibrations were monitored during pile driving to ascertain that there was no adverse impact on the existing foundation. Settlement data from continuous monitoring demonstrated that the retrofit scheme was effective and arrested further settlement and stabilized the foundation, allowing successful installation and alignment of the machine on the retrofitted foundation.

FOUNDATION DESCRIPTION

The subject STG foundation is a massive concrete structure consisting of one front pier and one concrete block, sitting on a mat foundation (basemat) supported by soil. The footprint of the pier and block has an overall dimension of 55 ft × 24 ft-8 in (16.76 m × 7.52 m), and their geometries are per the layout requirements of the machinery. In the original design, the mat foundation (basemat) footprint was also 55 ft × 24 ft-8 in (16.76 m × 7.52 m) and was 7 ft-7 in (2.31 m) thick. The top of basemat is 3 ft-7 in (1.09 m) above the surrounding area slab at elevation 100 ft (30.48 m). The pier supporting the steam turbine is 9 ft-6 in (2.90 m) above the basemat, whereas the block supporting the generator is 13 ft-7 in (4.14 m) above the basemat. The geometry of the STG foundation is shown in Figure 1.

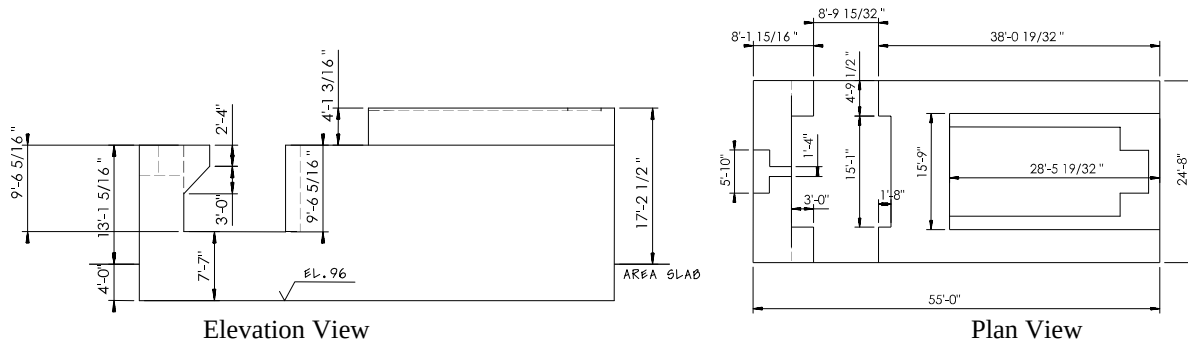


Figure 1—Geometry of STG Foundation (Original Design) [1 ft = 0.305 m; 1 in = 25.4 mm]

The condenser has an axial-exhaust configuration with respect to the STG. The condenser foundation is a 4-ft (1.22 m) thick mat with five concrete columns supporting the condenser (the four corner columns resist vertical loads, whereas the center column resists lateral loads). It has a footprint of 31 ft × 27 ft-9 in (9.45 m × 8.46 m).

In the original designs, the STG and the condenser foundations were separate structures with 1-inch (25.4 mm) gap. Both foundations were soil-supported. When unexpected differential settlements were observed at the STG foundation, extensive studies and site investigations were conducted, which concluded with the final mitigation actions as follows:

1. Installing piles around the STG perimeter (3 sides) to arrest the current settlements, to stop or minimize future settlement, and to stabilize the overall foundation.
2. Integrating the STG foundation with the condenser foundation to avoid possible differential settlement issues between the two foundations.
3. Installing piles underneath the four vertical-load-bearing columns in the condenser foundation to mitigate possible high moment and shear demands.

The ultimate goal of these mitigation steps was to ensure high certainty of outcome of the retrofit, so that installation and future operation of the equipment would not be affected.

Note that to accomplish step 1, a new concrete bracket (a.k.a. corbel) around the STG foundation (3 sides) was required to receive the piles, since new piles could not be installed underneath the already constructed massive STG foundation block. The bracket would have to be sufficiently integrated with the existing STG foundation. To accomplish steps 2 & 3, the already-constructed condenser foundation (the columns had not yet been poured) would have to be locally demolished and then re-constructed after installation of new piles and reinforcement. The STG and condenser foundations were integrated using mechanical splices.

The new geometry of the combined STG and condenser foundation is shown in Figure 2, with the outline of the original foundations shown in black and the new structural elements shown in red for clear distinction (*Red lines show the new HP14×73 H-piles, new concrete bracket around three sides of the STG foundation, and thickened portion of the condenser foundation underneath four concrete columns*).

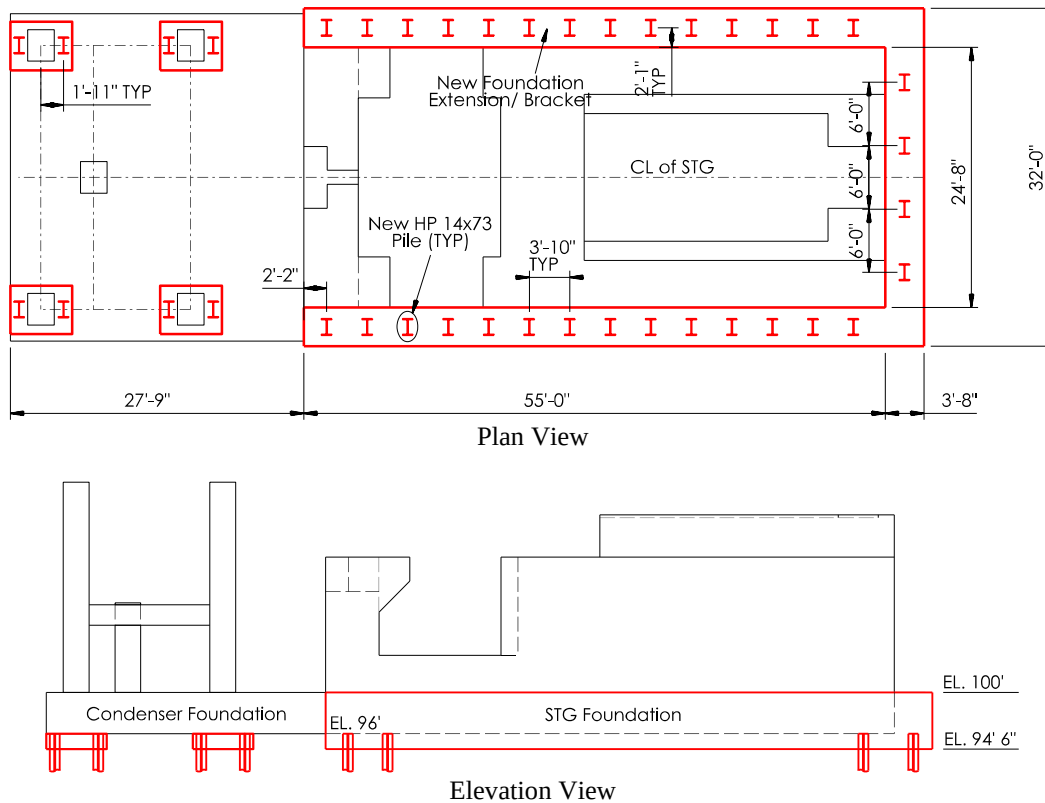


Figure 2—Combined STG and Condenser Foundation [1 ft = 0.305 m; 1 in = 25.4 mm]

FINITE ELEMENT MODEL

A finite element model (FEM) of the combined STG and condenser foundation was created using ANSYS (Figure 3). In this model, brackets were added to three sides of the STG basemat perimeter. The other side of the STG foundation was connected to the condenser foundation.

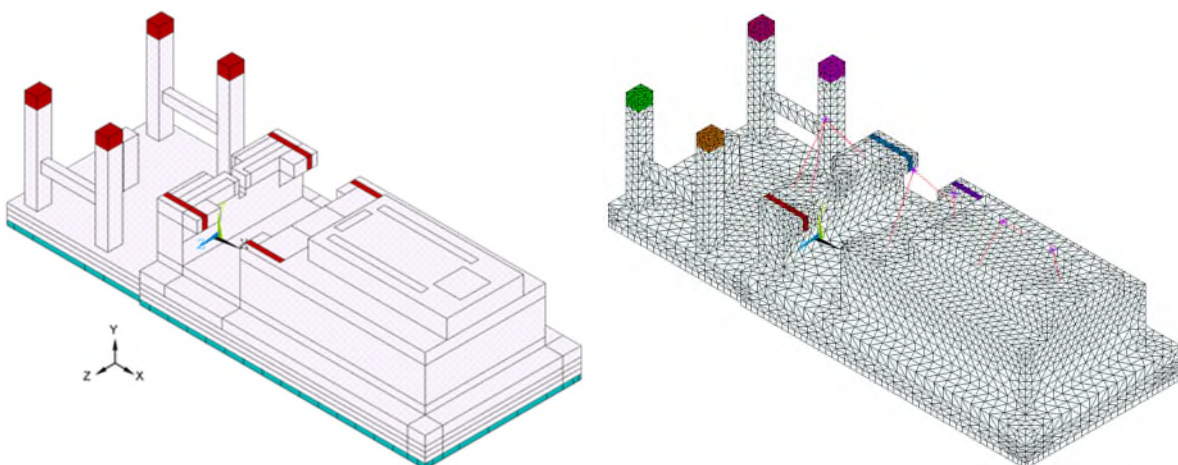


Figure 3—Finite Element Model of Combined Foundation (Left: Geometry Volumes, Right: Meshed Model)

8-node hexahedral solid elements (SOLID185) and 20-node brick/wedge/pyramid/tetrahedral solid elements (SOLID186) were used to model different components of the foundation structure. A nominal element size of 18" was used to mesh the solid volumes representing the foundation structure. At each node at bottom of the basemat, as well

as at each pile location, three spring-damper elements (COMBIN14) were used to model the subsurface impedances in two horizontal and one vertical directions. Rigid links and rigid areas were used to connect the bearings at the shaft elevation with the machine mounting plates at the machine-foundation interfaces. Mass elements (MASS21) were used to model the machine weights at each bearing location. The finite element model was oriented such that the global X, Y, and Z axes aligned with the longitudinal, vertical, and transverse directions, respectively.

The STG foundation was constructed using a concrete with design compressive strength of 4000 psi (27.58 MPa). For static analyses, a static modulus of elasticity of 3605 ksi (2.486E+04 MPa) was used for concrete, whereas for dynamic analyses, a dynamic modulus of elasticity of 4686 ksi (3.231E+04 MPa) was used to account for strain effects. A Poisson's ratio of 0.17 and a unit weight of 150 pcf (23.56 kN/m³) was used for the normal density concrete.

METHODOLOGY

Dynamic Impedances of Piles and Soil

The retrofit involved installing forty (40) HP14×73 steel piles, with a minimum soil embedment of 90-ft (27.43 m) per geotechnical requirements, to support the loads imposed on the combined foundation. In the as-built condition, majority of the weight of the previously constructed foundation had already been transferred by bearing to the underlying soil. After the piles were installed, the extended areas were built, and equipment was installed on the foundation. The loads from the equipment and the weight of the additional concrete would be transferred to the piles and soil in proportion to their stiffnesses. However, there was a large uncertainty associated with the prevailing soil conditions underneath the foundations at the site. Therefore, the retrofit design considered two extreme/bounding conditions to confirm the adequacy of the pile layout to meet the dynamic criteria provided by the machine vendor. These sets of analyses were:

- One set of analyses considered the structure to be a pile-supported foundation, wherein the contribution of soil in bearing was completely ignored and, thus, served as the worst scenario for the soil/pile impedance calculations.
- Second set of analyses considered the full contribution from both the piles and the underlying soil acting together and, thus, served as the best scenario for the soil/pile impedance calculations.

Within each set of the analyses mentioned above, lower-bound (LB) and upper-bound (UB) conditions were further considered to account for the variation of pile or soil properties. Therefore, the dynamic analyses were performed for the following four boundary conditions: (1) LB Piles-Only, (2) UB Piles-Only, (3) LB "Soil + Piles", and (4) UB "Soil + Piles". Figures 4 and 5 show the finite element model details for the "Piles-Only" and "Soil + Pile" boundary conditions, respectively.

The spring-damper elements associated with dynamic soil impedances of the underlying soil, which were distributed across all the nodes at the bottom of foundation, were included along with the pile spring-damper elements for the "Soil + Piles" cases, but were not incorporated in the "Piles-Only" cases.

For the "Soil + Piles" cases, the finite element model incorporated a modeling simplification wherein at the nodes corresponding to the pile locations, both the COMBIN14 elements representing the pile impedances and the COMBIN14 elements representing the dynamic soil impedances were included. Therefore, at the nodes for pile locations, two COMBIN14 elements aligned in each direction were modeled.

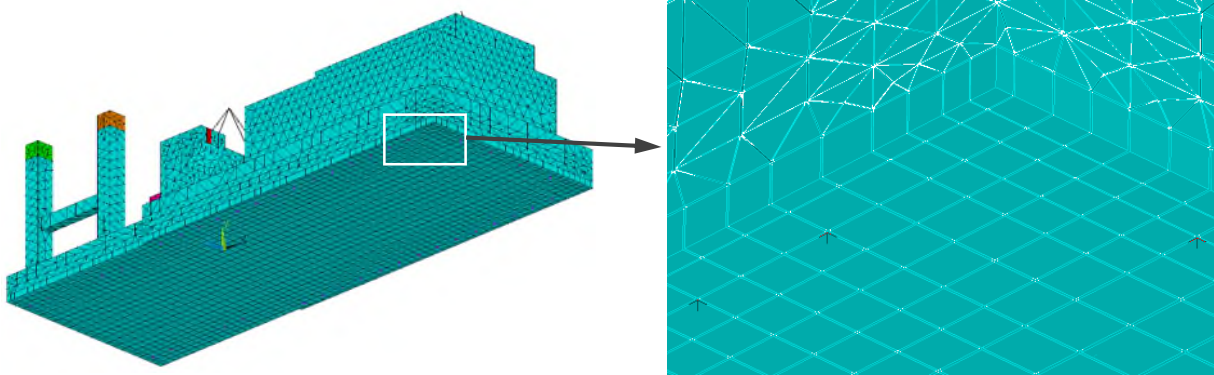


Figure 4—Piles (Spring-Damper Elements) in “Piles-Only” Cases
(Left: Overall Rendered View; Right: Enlarged View from Below)

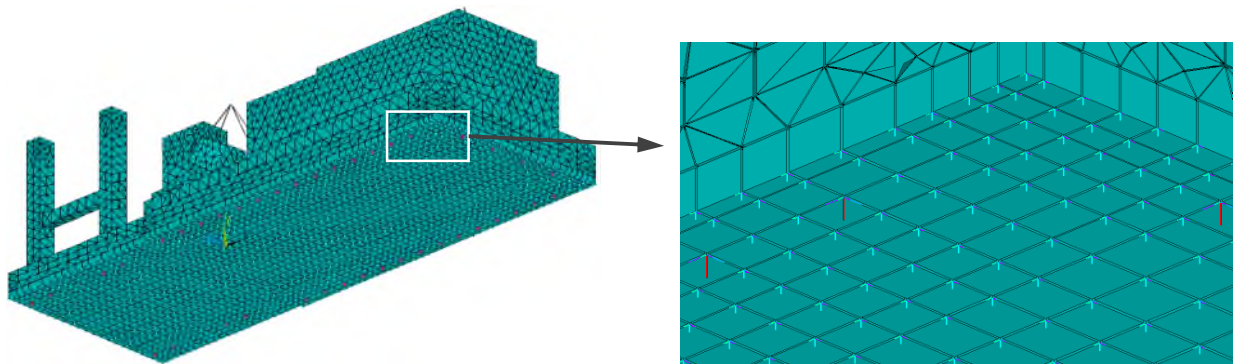


Figure 5—Piles and Soil Elements in “Soil + Pile” cases
(Left: Overall Rendered View; Right: Enlarged View from Below)

The dynamic soil impedance calculations for the combined STG and condenser foundation were performed following ACI 351.3R [1] and ASCE Manual-Of-Practice (MOP) #136 [2] recommendations. The frequency-dependent impedance functions (stiffness and damping) for the pile group were developed by geotechnical engineers using DYNA6.

The frequency-dependent dynamic pile impedances (stiffness and damping components in each direction) for the designated pile layout shown in Figure 2 were calculated up to approximately 73Hz using DYNA6 for lower bound and upper bound conditions. A review of the data indicated that the pile dynamic impedance variation over the 48-72 Hz frequency range was not significant (Figure 7). Therefore, the stiffness values for each pile in a particular direction were calculated by using the average pile group stiffness in that direction over the 48-72 Hz frequency range and distributing equally among the piles. Also, conservatively, the pile damping in a particular direction was taken as the minimum pile group damping in that direction over the 48-72 Hz frequency range and distributed equally among the piles.

Static Design Method

The “Piles-Only” case with the pile static stiffness was conservatively used for the static analyses to perform the foundation static deflection checks, pile geotechnical and structural capacity checks, and for the reinforced concrete strength design checks.

The solid element stress results from ANSYS were post-processed to determine the critical forces and moments at suitable section cut locations used for the reinforced concrete design. The design of the new bracket was performed in accordance with the original code of record, ACI 318-05 [3].

GEOTECHNICAL EVALUATION

The original site consisted of surficial fill and natural deposit of loose alluvium that transitioned into residual soils underlain by bedrock. The project site grading plan included various amounts of cut and fill to achieve the final site grade. At the STG location area, the fill thickness was approximately 11-ft (3.35 m). Fill had been derived from cut areas of the site and, in general, the characteristics of fill material are similar to those of native site soils. Site groundwater level is around 60-ft (18.29 m) below the final grade; however, shallow perched water had been observed during wet seasons.

It was believed that the fill quality, softening of fill material due to increased moisture and nearby construction activities, along with other possible soil deficiencies, had attributed to the observed excessive differential settlements of the STG foundation.

For the retrofit design, the geotechnical design soil profile and the engineering parameters were developed based on the soil boring data in the vicinity of the STG foundation. The assumed strength of the subgrade soil immediately below the foundation was reduced to account for potential disturbance during pile installation and to reflect poor fill quality.

As described previously, the piling solution was developed to retrofit the STG foundation. Steel H-piles (HP14×73) were selected because the H-piles had been previously tested and installed at the site for support of various structures. The pile length was selected to provide adequate pile capacity to support both vertical and lateral loads on the piles and to control settlement. The design philosophy was that 100% of the structure load would be transferred to the piles if underlying soils continue to settle, although unlikely to occur.

There was an additional design requirement that pile load under dead load must be limited to 50% of the allowable pile capacity, and hence the target pile allowable geotechnical compression capacity was established as 300 kips (1334 kN). With a typical SF of 2.0, the required ultimate geotechnical capacity was established as 600 kips (2669 kN). Based on the available geotechnical, static and dynamic pile load testing data at the site, and consideration of settlement, 90-ft (27.43 m) long HP14×73 piles were selected for the retrofit.

Static Spring Constants of Piles

It is typical industry practice to use equivalent springs and dashpots to replace soil and/or pile support in structural analysis of the superstructure. Equivalent springs need to reflect the expected performance of the support system during operational life.

The vertical static spring constant of piles was developed to bound the responses of the pile group under static load, which considered the elastic shortening of the pile as well as the potential settlement of the pile group (relatively short-term elastic settlement due to granular nature of the subsurface soils).

The lateral static spring constant of piles is a function of pile and soil stiffness and the pile head condition in terms of head rotation. Because of the rigidity of the existing STG concrete block, the piles were considered to behave as fixed-head (zero rotation with respect to pile cap). However, an allowance of 0.0003 radians of rotation was considered at the pile cap to satisfy the structural design requirements. LPILE [7] was used to obtain equivalent lateral spring constants for both strong and weak axes of the H-pile.

Dynamic Impedance of the Pile Group

Dynamic impedance functions of the pile group were developed using DYNA6. The dynamic properties in terms of shear wave velocity of soil layers were based on downhole seismic measurement data at the site and are presented in Figure 6. The dynamic impedance functions were defined for three shear-wave velocity (V_s) profiles; average, upper bound and lower bound profiles, in order to bound the response of the foundation system due to potential variability on soil properties. The upper bound and lower bound V_s profiles were selected as approximately ± 20 -25% from the average V_s profile. Note that due to the relatively large range of V_s variability, properties of the near surface soils were not adjusted to reflect poor fill quality.

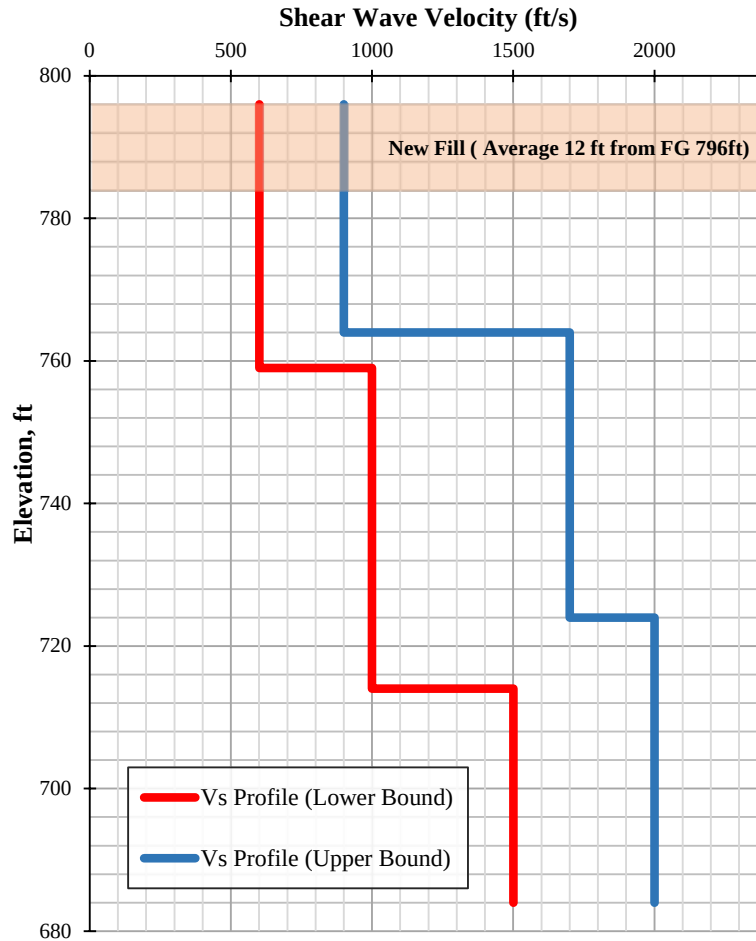
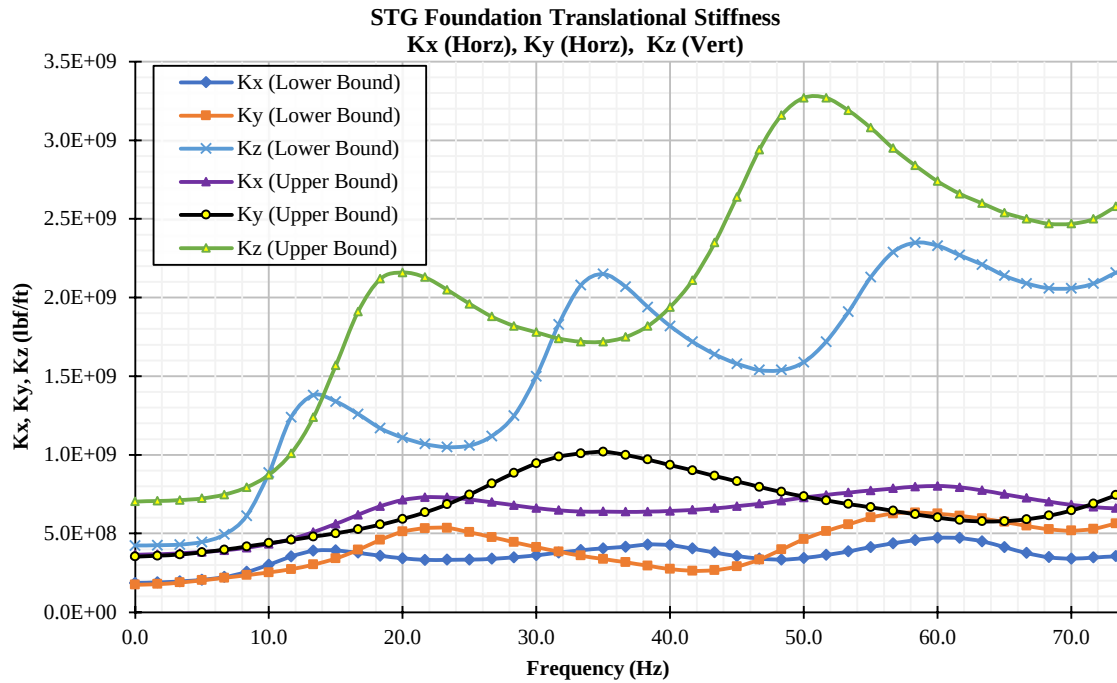
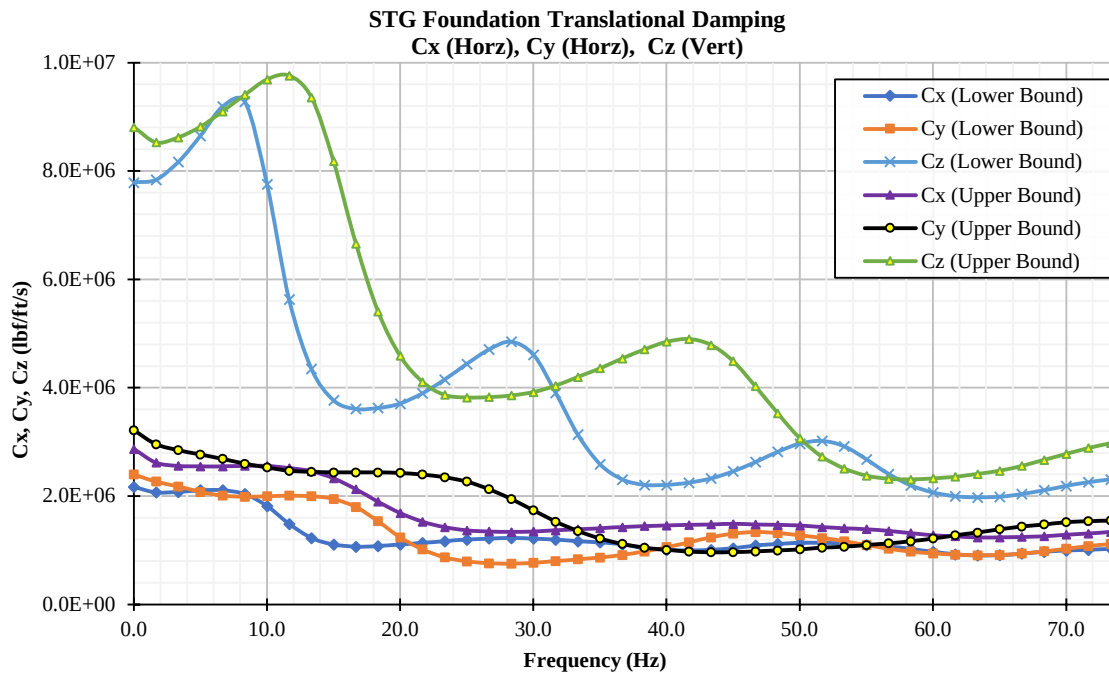


Figure 6—Shear-Wave Velocity Profile [1 ft = 0.305 m; 1 ft/s = 0.305 m/s]

The DYNA6 results representing the horizontal and vertical dynamic impedances for the pile group are presented for upper and lower bounds in Figure 7. The DYNA6 analysis followed a different axis convention from that used in the ANSYS analysis, with the Y and Z axes in DYNA6 corresponding to the transverse and vertical directions, respectively. The structural retrofit design of the STG foundation used these derived upper bound and lower bound responses.



(a) Stiffness



(b) Damping

Figure 7—Translational Dynamic Impedance Functions [1 lbf/ft = 14.594 N/m; 1 lbf/ft/s = 14.594 N/m/s]

LOADS AND LOAD COMBINATIONS

The same loads and load combinations, which had been considered in the original design of both foundations, were also used for the updated combined foundation. Two sets of load combinations were used: one for service level checks, and the other for strength level checks.

Among all applicable load combinations for strength design, those that included the machine rotor-blade-loss (RBL) accidental load would cause the worst loading demands on piles, as well as on the new bracket around the STG pedestal foundation. The machine vendor provided a very conservative equivalent static load for this rare-probability, accidental loading scenario. This equivalent static load, if directly used, however, would have resulted in a larger number of piles that were practically very difficult and expensive to install. Therefore, to provide a more reasonable pile layout, additional analyses were performed to reduce the RBL load for the pile capacity checks and for the reinforced concrete design of the bracket.

Towards this goal, two sets of analyses were performed as follows:

- In one set of analyses, the maximum RBL load was applied as an equivalent static load concurrently in both transverse and vertical directions at the applicable bearing location and a static analysis was performed. From this static analysis, the transverse and vertical displacements at the bearing location were extracted (u_{stat}).
- In the second set of analyses, the RBL load was applied as a harmonic load at the bearing location and a harmonic analysis was performed over the critical frequency range of 48-72 Hz. The corresponding dynamic transverse and vertical displacements at the bearing location were determined for all frequencies of interest (u_{dyn}).

The maximum ratio of the dynamic displacement to the static displacement (i.e., $u_{\text{dyn}}/u_{\text{stat}}$) over the entire frequency range was used as the reduction factor for the static RBL load. These analyses were separately performed for both transverse and vertical directions, and for both “Soil + Piles” and “Piles-Only” cases.

From the results of the analyses described above, the minimum reduction factor was used in each direction to reduce the RBL load for the pile capacity checks and for the reinforced concrete design of the bracket.

FOUNDATION PERFORMANCE EVALUATIONS

Dynamic analyses were performed for the “Soil + Piles” and “Piles-Only” cases to ensure that the dynamic performance criteria provided by the machine vendor were all met. These checks involved acceptability of the natural frequencies (modal analysis), vibration amplitudes in transverse and vertical directions (forced-vibration/harmonic analysis over a range of frequencies), and dynamic rigidity (harmonic analysis at operating frequency).

Foundation Natural Frequencies

Modal analyses were performed for all four cases to compute the natural vibration frequencies, and associated mode shapes of the combined STG and condenser foundation were carefully reviewed. As an example, Table 1 shows the results for the lower-bound “Piles-Only” case.

Table 1—Foundation Natural Frequencies for Lower Bound Piles-Only Case

Lower Bound Piles-Only Modal Analysis Result									
Mode	Frequency (Hz)	Translational Effective Mass to Total Mass Ratio (%)			Mode	Frequency (Hz)	Translational Effective Mass to Total Mass Ratio (%)		
		X(Long.)	Y(Vert.)	Z(Trans.)			X(Long.)	Y(Vert.)	Z(Trans.)
1	2.90	0.00%	0.00%	5.54%	26	28.72	0.03%	0.01%	0.01%
2	3.34	6.64%	0.00%	0.24%	27	29.03	0.00%	0.00%	0.00%
3	3.35	0.11%	0.00%	3.85%	28	29.20	0.00%	0.00%	0.00%
4	3.39	0.51%	0.00%	0.31%	29	33.24	0.00%	0.00%	0.00%
5	3.88	4.59%	0.00%	0.01%	30	34.10	0.00%	0.00%	0.00%
6	4.00	0.07%	0.00%	0.45%	31	36.65	0.00%	0.00%	0.00%
7	4.67	0.18%	0.00%	0.00%	32	37.94	0.00%	0.02%	0.00%
8	5.50	0.33%	0.00%	0.00%	33	41.53	0.00%	0.01%	0.00%
9	7.17	61.41%	0.10%	0.00%	34	42.19	0.00%	0.01%	0.00%
10	8.30	0.02%	0.00%	81.60%	35	43.84	0.00%	0.00%	0.00%
11	8.31	20.53%	0.03%	0.07%	36	45.60	0.00%	0.00%	0.00%
12	11.49	0.00%	0.00%	3.99%	37	47.61	0.00%	0.00%	0.00%
13	11.52	0.00%	0.05%	0.02%	38	49.08	0.00%	0.00%	0.00%
14	12.76	5.03%	0.72%	0.00%	39	51.57	0.00%	0.00%	0.00%
15	15.65	0.02%	91.21%	0.00%	40	51.85	0.00%	0.00%	0.00%
16	17.24	0.37%	3.71%	0.01%	41	57.93	0.00%	0.00%	0.00%
17	19.14	0.11%	2.68%	0.44%	42	59.60	0.00%	0.00%	0.00%
18	19.88	0.01%	0.74%	2.85%	43	60.59	0.00%	0.00%	0.00%
19	21.25	0.00%	0.19%	0.01%	44	63.22	0.00%	0.00%	0.00%
20	21.62	0.00%	0.27%	0.05%	45	68.21	0.00%	0.00%	0.00%
21	22.09	0.02%	0.01%	0.00%	46	69.84	0.00%	0.00%	0.00%
22	22.45	0.00%	0.04%	0.18%	47	70.66	0.00%	0.00%	0.00%
23	24.20	0.00%	0.04%	0.20%	48	74.41	0.00%	0.00%	0.00%
24	25.51	0.00%	0.00%	0.00%	49	78.04	0.00%	0.00%	0.00%
25	27.58	0.01%	0.00%	0.13%	50	79.98	0.00%	0.00%	0.00%
Sum							100.00%	99.85%	100.00%

Review of all four sets of results have confirmed that the fundamental frequencies of the foundation are at least $\pm 20\%$ apart from the turbine-generator rated frequency of 60 Hz. This indicates that in the specified frequency range, no significant structural response is expected and the potential for resonance would be negligible from a global behavior perspective. Also, the results in Table 1 indicate that for modes with frequencies within the critical frequency range of 48-72 Hz ($\pm 20\%$ range of the rated frequency), the cumulative mass participation factors are practically zero in all three directions. It is also noted that no fundamental frequencies are close to the machine critical speeds provided by the vendor.

Forced Vibration Analysis

Forced vibration responses of the combined foundation due to normal rotor unbalance loads, for all four cases, were calculated for the frequencies in the range 48-72 Hz. In particular, the vibration amplitudes at each bearing in both vertical and transfer directions were reviewed for each case.

For example, Figure 8 shows the calculated vibration amplitudes at all bearing locations in the transverse direction for the “Soil + Piles” case. The equivalent displacement-based dynamic criterion of 3.8 mm/s (0.15 in/s) RMS vibration velocity was stipulated by the vendor. The response curves have confirmed that the calculated vibration amplitudes are lower than the maximum allowable amplitudes.

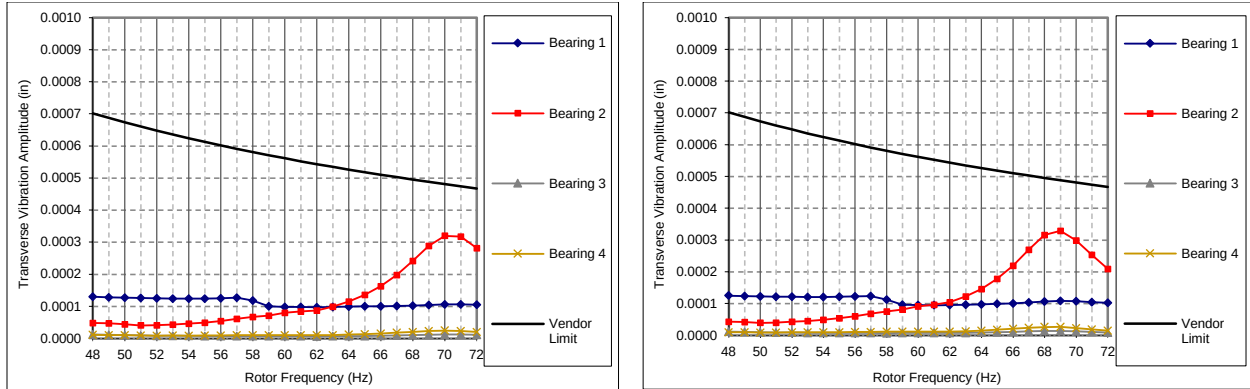


Figure 8—Vibration Amplitudes at Bearings in Transverse Direction for “Soil + Piles” Case
(Left: Lower-Bound; Right: Upper-Bound) [1 in = 25.4 mm]

Foundation Dynamic Rigidity

The dynamic rigidity of the combined STG foundation was checked by applying a unit rotating unbalance force at the operating frequency (60 Hz) at machine-foundation interface locations for all four cases. The resulting foundation stiffness values were reviewed. As shown in Table 2 below, the lowest dynamic stiffnesses in the transverse and vertical directions are $1.51\text{E}+07$ kN/m ($8.622\text{E}+04$ kip/in) and $2.36\text{E}+07$ kN/m ($1.348\text{E}+05$ kip/in), respectively, which are higher than the minimum dynamic stiffness requirement of $1.0\text{E}+07$ kN/m ($5.71\text{E}+04$ kip/in) required by the vendor. From the results, it is also worth noting that dynamic stiffnesses are functions of structural local natural frequencies, and thus it is not necessarily true that a statically stiffer boundary condition will result in a higher dynamic stiffness of a structural element.

Table 2—Foundation Dynamic Stiffness at Machine-Foundation Interfaces
(Left: “Soil + Piles” Case; Right: “Piles-Only” Case) [1 kN/m = 0.00571 kip/in]

Lower Bound Dynamic Stiffness Result (kN/m)			Upper Bound Dynamic Stiffness Result (kN/m)		
Locations	Y(Vert.)	Z(Trans.)	Locations	Y(Vert.)	Z(Trans.)
C1	4.87E+07	1.57E+07	C1	4.57E+07	1.51E+07
C2	3.68E+07	1.97E+07	C2	3.47E+07	1.94E+07
C3	2.79E+07	1.12E+08	C3	2.65E+07	1.21E+08
C4	3.66E+07	3.75E+07	C4	3.53E+07	3.47E+07
A1	2.41E+07	2.85E+07	A1	2.57E+07	3.61E+07
A2	4.49E+07	2.96E+07	A2	5.76E+07	3.79E+07
F	6.82E+07	1.98E+08	F	6.35E+07	2.60E+08
G1	2.00E+08	1.11E+08	G1	2.30E+08	1.03E+08
G2	1.38E+08	1.01E+08	G2	1.41E+08	9.51E+07
H	6.90E+07	3.74E+07	H	6.60E+07	4.50E+07
Min.	2.41E+07	1.57E+07	Min.	2.57E+07	1.51E+07

Lower Bound Dynamic Stiffness Result (kN/m)			Upper Bound Dynamic Stiffness Result (kN/m)		
Locations	Y(Vert.)	Z(Trans.)	Locations	Y(Vert.)	Z(Trans.)
C1	4.99E+07	1.59E+07	C1	4.81E+07	1.55E+07
C2	3.76E+07	1.99E+07	C2	3.62E+07	1.97E+07
C3	2.84E+07	1.13E+08	C3	2.75E+07	1.26E+08
C4	3.71E+07	3.80E+07	C4	3.63E+07	3.55E+07
A1	2.36E+07	2.64E+07	A1	2.46E+07	3.03E+07
A2	4.19E+07	2.75E+07	A2	4.91E+07	3.17E+07
F	7.07E+07	1.84E+08	F	6.67E+07	2.17E+08
G1	1.97E+08	1.11E+08	G1	2.08E+08	1.05E+08
G2	1.37E+08	1.02E+08	G2	1.37E+08	9.66E+07
H	7.03E+07	3.51E+07	H	6.65E+07	3.91E+07
Min.	2.36E+07	1.59E+07	Min.	2.46E+07	1.55E+07

Static Deflection Check

The static deflection checks for the foundation were performed. The goal was to limit the tilting (i.e. slope) in both machine axial and transverse directions as well as the absolute differential settlements at the top of pedestal for the normal torque load case. Results have confirmed that vendor’s limits have been all met.

Pile Capacity Check -Geotechnical

The suitability of the retrofit design of the combined foundation using piles was governed by the adequacy of the supporting piles to resist the applicable loads. All HP14×73 piles must be checked to ensure that they can resist the maximum demands for both service level and strength level load combinations. Since the “Piles-Only” case would lead to the maximum demands on the piles, the checks were based on the results for this condition only.

For the geotechnical capacity evaluation of the piles, the maximum demands from the service level load combinations were extracted from the ANSYS analysis. Since the combined foundation is a massive concrete block structure, the governing axial loads would be due to dead loads throughout the design life of the structure. Also, the project design

criteria stipulated that the pile capacity (geotechnical) under static loads shall not exceed 50% of the allowable, and under “static + dynamic” loads shall not exceed 75% of the allowable. Therefore, for a better understanding and realistic estimate of the design margins, the static analysis results were post-processed to extract the results for three sets of load combinations: (a) dead load only, (b) envelope from normal operation load combinations, and (c) envelope from accident load combinations, and appropriate checks were performed for each set accordingly.

The envelope pile reactions (vertical and lateral) were depicted as bubble-plots for each load combinations. Each bubble represents the pile location, and the bubble size is an indicator of the magnitude of pile load, with the magnitude (kips) shown within the bubble. All reactions were compared with the allowable limits to ensure that the limits are not exceeded. For example, Figure 9 shows the pile vertical reactions for the “Dead Load Only” case, for which the maximum pile axial compression is 144.7 kips (644 kN). Meanwhile, per the geotechnical evaluation, the pile has a maximum allowable axial compression of 300 kips (1334 kN). Considering the project requirement that the pile reactions shall not exceed 50% of the allowable, the allowable axial load is $0.5 \times 300 = 150$ kips (667 kN), which is larger than the maximum pile compression of 144.7 kips (644 kN).

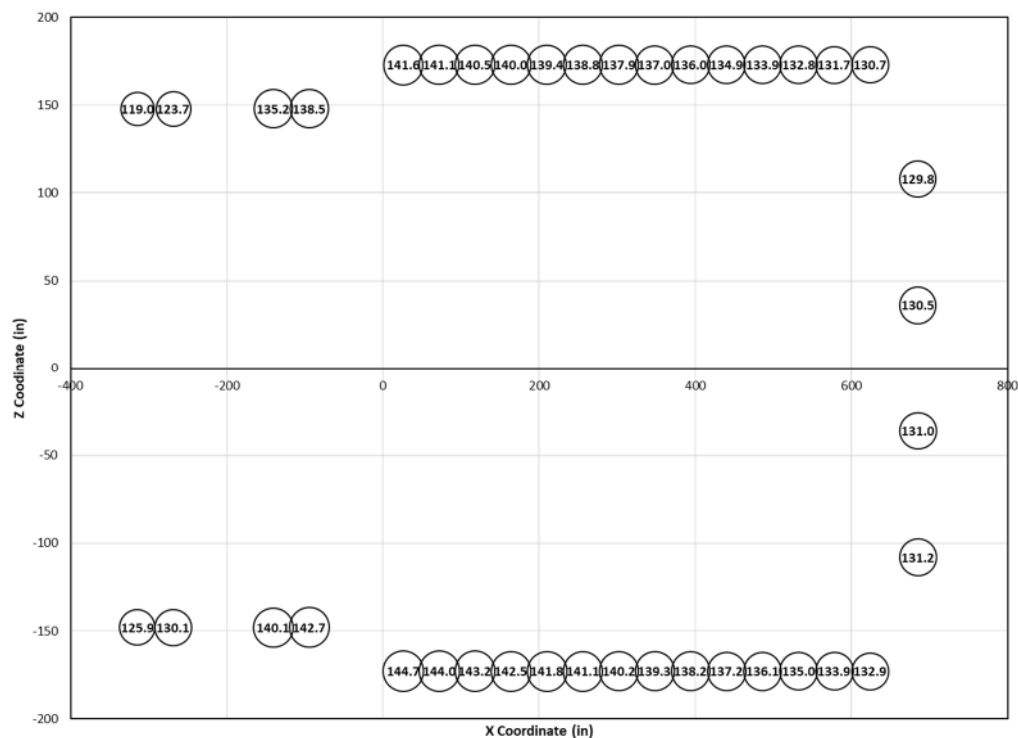


Figure 9—Enveloping Pile Axial Compression (kip) for Dead Load Only [1 kip = 4.448 kN; 1 in = 25.4 mm]

Pile capacity check-Structural

The structural adequacy of the pile cross-section was checked using the maximum moment and shear demands along the entire pile length from the LPILE analyses. The structural design checks were performed by conservatively considering the maximum reactions in all three directions occurring for the same pile, for each applicable normal operation and accidental load combinations.

A $\frac{1}{16}$ -in (1.59 mm) corrosion allowance was considered on all faces for the HP14×73 pile. Detailed checks following AISC 360-10 [4] have confirmed that the piles are structurally adequate to resist the maximum loading demands.

NEW BRACKET DESIGN

One unique aspect of this retrofit design for the combined foundation was to design the new bracket around the three sides of the STG foundation (See Figure 2). The new bracket provides the critical interface to allow the load transfer from the foundation to the piles and must be adequately designed through a robust mechanical transfer mechanism.

To achieve such goals, as well as to take account for efficient construction work, a cross section of 5 ft-6 in $\times$ 3 ft-8 in (1.68 m $\times$ 1.12 m) was chosen (Figure 10). The bracket was checked for pile punching shear, pile lateral shear (concrete breakout towards free surface), as well as resulting maximum moment, tension and shear at the bracket-pedestal interface for all applicable loads, following the code requirements of ACI 318-05.

To ensure high certainty of outcome of the retrofit, mechanical rebar couplers were chosen over epoxy rebar dowels to extend the existing primary #10 rebars into the new bracket (Figure 10), though this meant more construction effort was required.

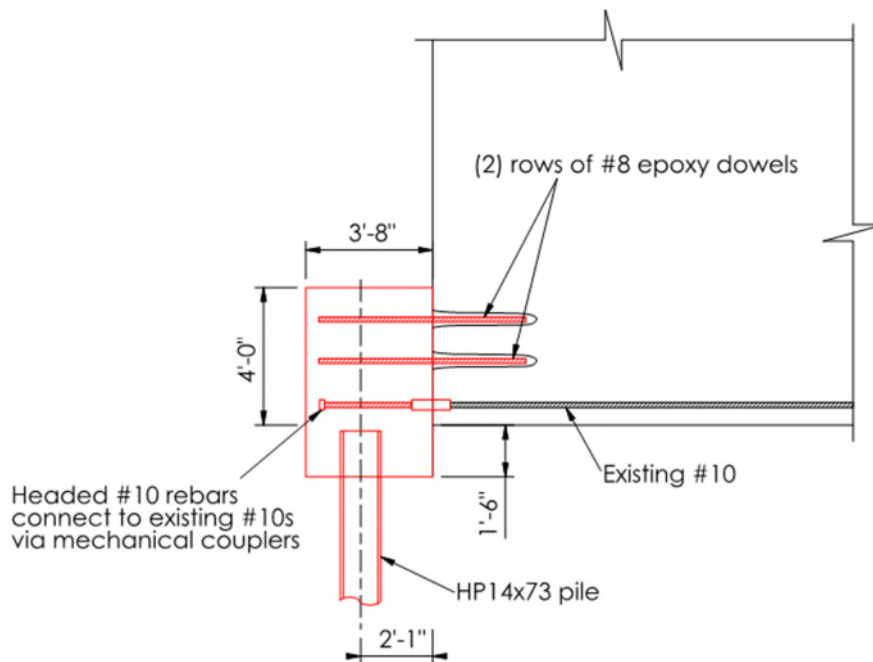


Figure 10—Bracket Reinforcement Design [1 ft = 0.305 m; 1 in = 25.4 mm]

CONSTRUCTION CONSIDERATIONS AND MONITORING SCHEME

The field implementation of STG foundation retrofit (Figure 11) involved several construction challenges, with the key challenge being to select an appropriate pile driver to minimize required demolition of existing area slab around the STG foundation and at the same time prevent any damage to the STG foundation during pile driving. H-piles had to be spliced due to long lengths and the pile driving had to be carefully sequenced to mitigate potential additional settlement due to the pile driving operations. Extensive settlement and vibration monitoring were conducted during pile driving operation and subsequent construction activities to ensure no damage to adjacent structures.

Hydro-demolition was used for surface preparation of STG foundation at concrete bracket interface. Careful planning was required to ensure adequate water drainage during the hydro-demolition process. Appropriate in-place testing of horizontal epoxy #8 rebar dowels was conducted to ensure the installation quality.

The 4 ft (1.22 m) thick condenser mat foundation was demolished using specialized cutting tools. Rebar adjustments had to be made in tight spaces just above the H-piles supporting four concrete columns.



(a) H-piles prior to driving



(b) H-piles driven to designed elevation



(c) Preparation for constructing the bracket



(d) Bracket concrete placed

Figure 11—Progress during Construction

CONCLUSIONS

An innovative retrofit scheme was developed and successfully implemented to arrest the unexpected settlements of a STG foundation under construction. Steel HP14×73 piles were effectively designed and installed around the already-constructed STG foundation. A new concrete bracket was added to receive the piles and to connect to the STG pedestal foundation. The STG foundation was also integrated with the condenser foundation. The new combined foundation was checked for various design requirements, such as pile capacities, dynamic performance, static deflections and strength designs.

All these mitigating measures have worked together to solve the complex settlement issue and effectively mitigated the overall cost and schedule impacts on the project. The settlement monitoring data demonstrated that since the completion of retrofit, the net change in total and differential settlement was negligible and only a very small fraction of the original settlements.

REFERENCES

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- [5] ANSYS Ver. 14.5, ANSYS Mechanical APDL
- [6] DYNA6 Ver. 6.1
- [7] LPILE Ver. 2018-10.006

Comparison of Several Methodologies Used for Designing Tabletop Foundations

Mukti Lal Das

Synopsis: The availability of high-speed computers at a reasonable price resulted in various sophisticated analysis and design methodologies for the elevated flexible pedestal (Tabletop) foundations replacing the rule-of-thumb processes based on static equivalent principle which used to produce either unsafe or overly conservative structures. A thorough study is undertaken with four typical models usually selected for static and dynamic analyses in the structural/mechanical engineering practice. It is found that all the models provide comparable results. However, each one has its own advantages and disadvantages. These will be highlighted in the Summary and Concluding Remarks of this paper.

Keywords: Rule-of-thumb, Eigenvalues, Rayleigh frequencies, Turbine force amplitude

ACI member **Mukti L. Das** is a Structural Engineering consultant located at North Andover, MA, USA. He has authored numerous articles and documents including a chapter of the book, “Proceedings of the First International Conference, Application of Artificial Intelligence in Engineering Problems,” Southampton University, UK [7]. He is the Chair of ACI Sub Committee 351.3 (Foundations for Dynamic Equipment) and member of ACI Committees 349 (Nuclear Structures) and 447 (Finite Element Analysis of Reinforced Concrete Structures). He is also a Fellow of ASCE (American Society of Civil Engineers). His field of research includes Finite Element Analysis and Dynamic Analysis related to Nuclear Power Plants. He is a Licensed Professional Engineer in the States of Massachusetts and Maine.

INTRODUCTION

The availability of the modern sophisticated digital computers at affordable prices resulted in the development of various modelling techniques for the analysis of tabletop foundations to better understand the dynamic characteristic of the structure due to the vibration of the machines that it supports. Those dynamic characteristics were ignored or accounted for by using simplified and conservative assumptions before this supercomputer era. One such rule-of-thumb based on simplified assumptions to analyze a structure supporting a centrifugally vibrating machine was used by the following static equivalent method:

1. Total vertical load plus 0.5 of the total full load acting in the vertical direction.
2. Total vertical load plus 0.3 of the total full load acting in the transverse direction.
3. Total vertical load plus 0.1 of the total full load acting in the longitudinal direction.

These three approximated load factors (0.5, 0.3, and 0.1) are acceptable when the provided ratio of machine frequency to the natural frequency in the specified direction is less than 1.0, greater than 1.0 and much greater than 1.0 respectively. These factors are derived for the machine frequency of 1,800 rpm as shown in [1].

At present, the above specified rule-of-thumb process based on static equivalent loading is not used. Instead, dynamic analyses based on various methodologies are used. In this paper, four Finite Element (FE) models based on frequently used methodologies are developed, compared with the study presented in [1], and a recommendation is made for the conclusion.

COMPONENTS OF TABLETOP FOUNDATIONS

A typical Tabletop Turbine/Generator foundation used in most power plants is shown in Figure 1 [2].

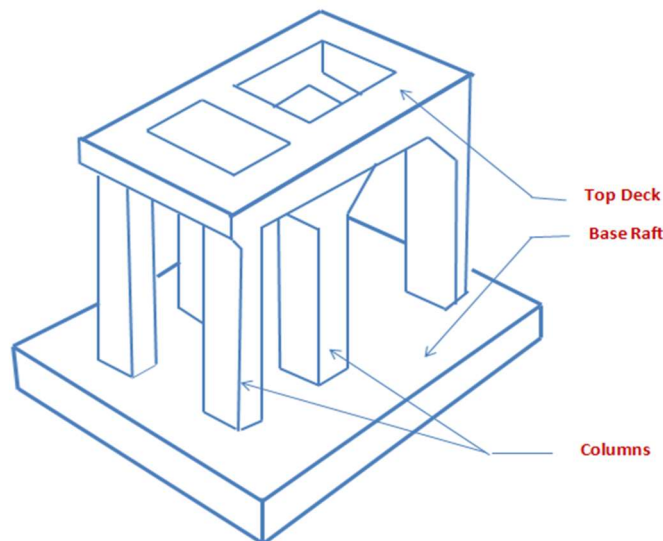


Figure 1—A Typical Tabletop

The Top Deck comprises transverse beams, longitudinal beams and/or a slab usually connecting these beams. If provided, the slab is modelled with Plate or Shell finite elements. Columns generally are made of concrete and modelled with Beam elements. The Base Raft is generally a thick concrete slab supported on soil or piles and is modelled generally with Plate or Shell finite elements. All the above components can also be modelled as 3D finite elements.

DISCUSSION ON COMPUTER INPUT MODEL BY ARYA, O'NEILL & PINCUS

Figures 2 & 3 show the Foundation and its dynamic analysis model used by Arya, O'Neill and Pincus in their first textbook [1]. The analysis was performed using the original STRUDL developed by the Civil Engineering Department of MIT in late 1960s.

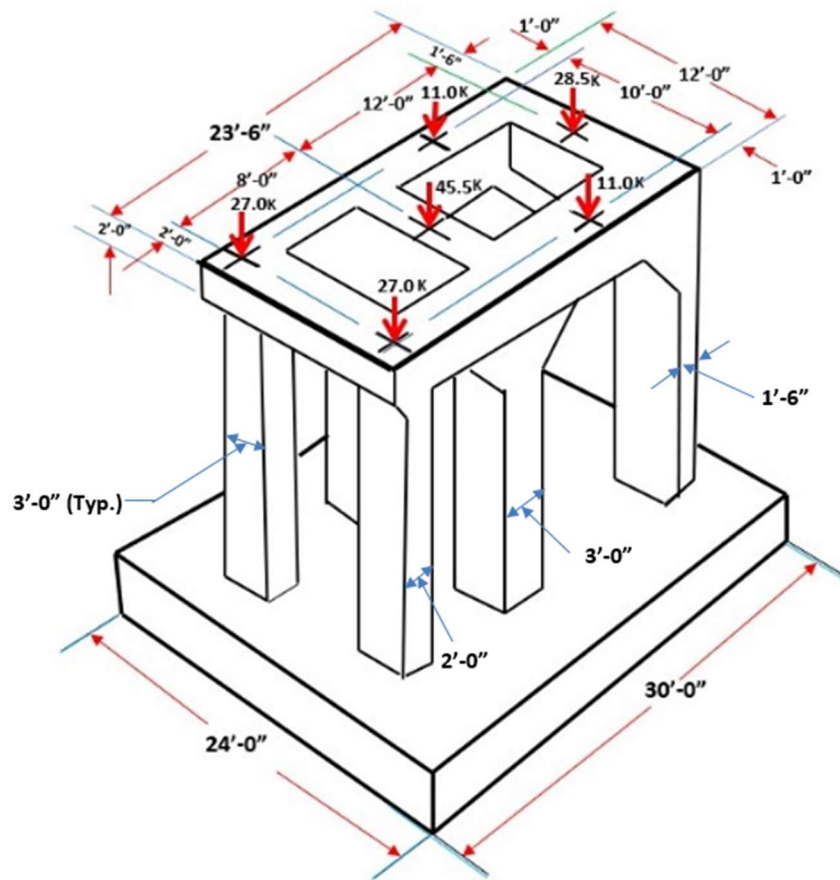


Figure 2—Tabletop Foundation Used in [1] (Including Machine Support Loads) [1 ft = .305 m, 1 in = 25.4 mm]

Figure 3 shows the computer model for the above tabletop foundation using lumped mass elements [1], [3]. The numbers on the top of each mass are the identification number of joints where the mass is lumped, while the numbers inside circles are the identification number of members closest to the circles. The constants k_{xi} , k_{yi} and k_{zi} represent corresponding soil springs in the X, Y and Z axes respectively. The component “i” represents the joint number at which the corresponding soil spring is connected. For instance, k_{zi} is the soil spring constant in the Z axis and connected to joint i.

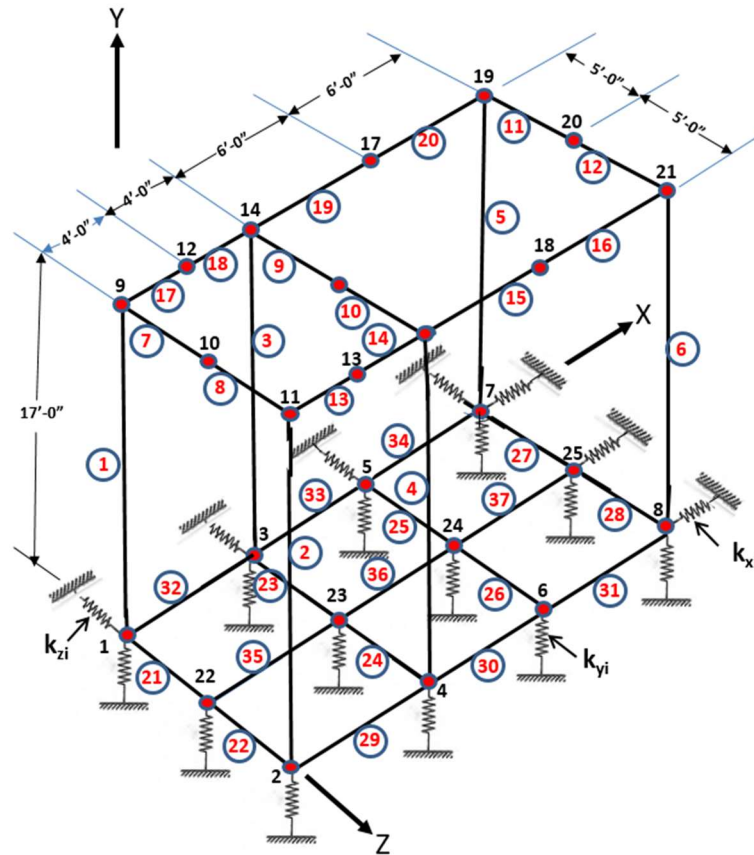


Figure 3—Computer Model Used in [1] (Referred to as Model 1 in This Paper) [1 ft = .305 m, 1 in = 25.4 mm]

All models (Model 1 to Model 4) are analyzed using STAAD [6]. Their results are compared with the one from [1]. Modelling parameters were obtained from [1] and same parameters were used for all four analyses (Model 1 through 4) here as follows:

1. Machine Parameters

Total machine weight = 150,000 lbs. [667.20 kN]

Turbine/Compressor speed = 6,949 rpm, $\omega = 727.7$ rad/sec

Turbine rotor weight = 159 lbs. [0.71 kN]

Amplitude of turbine force = $(W/g) e \omega^2$

Eccentricity = e (mil) = $\alpha \sqrt{\frac{12,000}{rpm}} < 1.0$ mil [0.0254 mm]

$\alpha = 0.5$ at installation time (Table 3-1 in [1])

= 1.0 after several years of operation

1 mil = 0.001 in [0.0254 mm]

Amplitude of Turbine force = $(W/g) e \omega^2 = [159/(32.2 \times 12)] \times (1.0 \sqrt{\frac{12,000}{6,949}} / 1000) \times (727.7)^2 = 286$ lbs. [1.27 kN]

Compressor rotor weight = 4,328 lbs. [19.25 kN]

Amplitude of compressor force = $(W/g) e \omega^2 = \left[\frac{4328}{32.2 \times 12} \right] \times \left(1.0 \sqrt{\frac{12,000}{6,949}} \frac{1}{1000} \right) \times (727.7)^2 = 7,794$ lbs. [34.67 kN]

2. Soil Parameters [1], [5]
 Shear Modulus, $G = 6,500$ psi [34.67 MPa]
 Poisson's ratio = 0.45
 Coefficient of subgrade reaction, $k_r = 120$ lbs./in³ [1.11 kg/m³]
 Soil density = 115 pcf [1842 kg/m³]
 Allowable bearing capacity = 2,000 psf [95.76 kN/m²]
 Predicted static settlement = 0.2 in. at 2000 psf [5.08 mm at 95.76 kN/m²]
3. Concrete Parameters
 Modulus of elasticity of concrete, $E = 3,122,000$ psi [1038.07 kN/m²]
 Poisson's ratio = 0.17

DISCUSSION ON COMPUTER INPUT MODEL 1 THROUGH MODEL 4

The tabletop foundation geometry and the supporting machines of the following 4 models are the same as that used in [1]. All models are also supported on the same type of soil as in this reference. Model 1 through Model 3 have identical Top Deck and Column configurations with different types of Base Rafts. Model 4 is completely different because it uses 3D Solid finite elements. The modelling differences of these are discussed as follows:

Model 1: The Top Deck, Columns and the Base Raft are modelled as a lumped mass system as in [1]. All members including the Base Raft slab are represented as a collection of one-dimensional beam elements as shown in Figure 4.

Model 2: As in Model 1, the Top Deck, Columns and the Base Raft are modelled as a lumped mass system as in [1]. All members including the Base Raft slab are represented as a collection of one-dimensional beam elements. Only difference is that each mass in the Base Raft is an average mass m where,

$$m = (m_1 + m_2 + m_3 + m_4 + m_5 + m_6 + m_7 + m_8 + m_{22} + m_{23} + m_{24} + m_{25})/12.$$

The masses, m_1 through m_8 and m_{22} through m_{25} are the same as in Model 1. The total soil spring constants on the slab is also divided into 12 equal spring constants for 12 joints (See Figure 5 for Model 2).

Model 3: The Top Deck and Columns are discretized as lump mass systems as in [1]. The Base Raft slab is modelled as 2D Plate Finite Elements. The masses and soil spring constants are lumped on each Finite Element joint by issuing two simple computer comments. By doing so we save labor.

Model 4: The Top Deck, Columns and the Base Raft slab are discretized as 3D Solid finite elements [4] and soil spring constants are lumped only on the Finite Element joints that are declared as support joints touching the soil by the software. Assigning springs constants this way saves some labor.

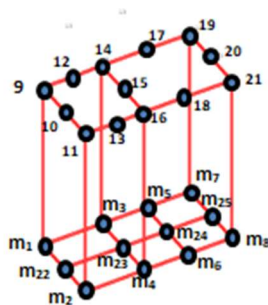


Figure 4—Model 1

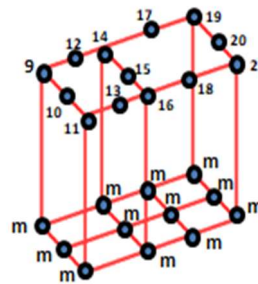


Figure 5—Model 2

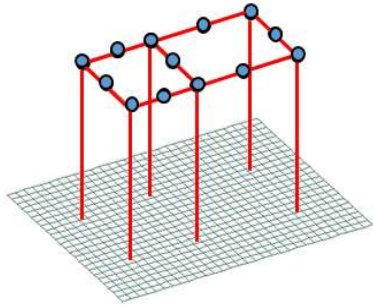


Figure 6—Model 3

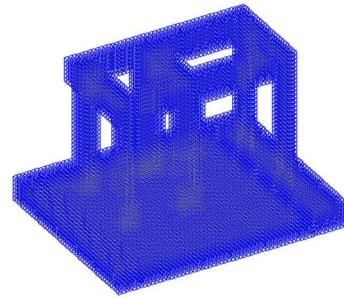


Figure 7—Model 4

STAAD INPUT FILE FOR MODEL 1

STAAD SPACE MODEL 1: STATIC & DYNAMIC OF A TABLE TOP

START JOB INFORMATION

Mukti L. Das, Engineer

Date: June 15, 2020

END JOB INFORMATION

*

INPUT WIDTH 79

UNIT FEET KIP

JOINT COORDINATES

1 0 0 0; 2 0 0 10; 3 8 0 0; 4 8 0 10; 5 14 0 0; 6 14 0 10; 7 20 0 0; 8 20 0 10;
9 0 17 0; 10 0 17 5; 11 0 17 10; 12 4 17 0; 13 4 17 10; 14 8 17 0; 15 8 17 5;
16 8 17 10; 17 14 17 0; 18 14 17 10; 19 20 17 0; 20 20 17 5; 21 20 17 10;
22 0 0 5; 23 8 0 5; 24 14 0 5; 25 20 0 5;

*

MEMBER INCIDENCES

1 1 9; 2 2 11; 3 3 14; 4 4 16; 5 7 19; 6 8 21; 7 9 10; 8 10 11; 9 14 15;
10 15 16; 11 19 20; 12 20 21; 13 11 13; 14 13 16; 15 16 18; 16 18 21; 17 9 12;
18 12 14; 19 14 17; 20 17 19; 21 1 22; 22 22 2; 23 3 23; 24 23 4; 25 5 24;
26 24 6; 27 7 25; 28 25 8; 29 2 4; 30 4 6; 31 6 8; 32 1 3; 33 3 5; 34 5 7;
35 22 23; 36 23 24; 37 24 25;

*

SUPPORTS

1 2 FIXED BUT MX MY MZ KFX 8649 KFY 11930 KFZ 8649
3 4 FIXED BUT MX MY MZ KFX 6727 KFY 9279 KFZ 6727
5 6 FIXED BUT MX MY MZ KFX 5766 KFY 7953 KFZ 5766
7 8 FIXED BUT MX MY MZ KFX 7688 KFY 10604 KFZ 7688
22 FIXED BUT MX MY MZ KFX 4552 KFY 6279 KFZ 4552
23 FIXED BUT MX MY MZ KFX 3541 KFY 4884 KFZ 3541
24 FIXED BUT MX MY MZ KFX 3035 KFY 4186 KFZ 3035
25 FIXED BUT MX MY MZ KFX 4046 KFY 5581 KFZ 4046

*

MEMBER PROPERTY AMERICAN

1 2 PRIS AX 6 IX 4.7 IY 4.5 IZ 2
3 4 PRIS AX 9 IX 11.4 IY 6.8 IZ 6.8
5 6 PRIS AX 4.5 IX 2.3 IY 3.8 IZ 0.8
7 8 PRIS AX 9 IX 11.1 IY 6.8 IZ 6.8

9 10 PRIS AX 6 IX 4.7 IY 2 IZ 4.5
 11 12 PRIS AX 4.5 IX 2.3 IY 0.8 IZ 3.8
 13 14 17 18 PRIS AX 6 IX 4.7 IY 2 IZ 4.5
 15 16 19 20 PRIS AX 7.5 IX 7.8 IY 3.91 IZ 5.6
 21 22 PRIS AX 22.5 IX 44.41 IY 1687.5 IZ 11.7
 23 24 PRIS AX 17.5 IX 34.54 IY 1312.5 IZ 9.1
 25 26 PRIS AX 15 IX 29.61 IY 1125 IZ 7.8
 27 28 PRIS AX 20 IX 39.48 IY 1500 IZ 10
 29 TO 34 PRIS AX 23.8 IX 46.23 IY 1140 IZ 12.4
 35 TO 37 PRIS AX 12.5 IX 24.33 IY 600 IZ 6.5

*

UNIT INCHES KIP
 DEFINE MATERIAL START
 ISOTROPIC MATERIAL1
 E 3122
 G 1334
 POISSON 0.17
 DENSITY 8.68e-005
 END DEFINE MATERIAL

*

CONSTANTS
 MATERIAL MATERIAL1 ALL

*

UNIT INCHES POUND
 DEFINE TIME HISTORY
 TYPE 1 FORCE
 FUNCTION SINE
 AMPLITUDE 143 RPM 6950 CYCLES 1000

*

TYPE 2 FORCE
 FUNCTION SINE
 AMPLITUDE 143 RPM 6950 PHASE 90 CYCLES 1000

*

TYPE 3 FORCE
 FUNCTION SINE
 AMPLITUDE 4040 RPM 6950 CYCLES 1000

*

TYPE 4 FORCE
 FUNCTION SINE
 AMPLITUDE 4040 RPM 6950 PHASE 90 CYCLES 1000

*

TYPE 5 FORCE
 FUNCTION SINE
 AMPLITUDE 3897 RPM 6950 CYCLES 1000

*

TYPE 6 FORCE
 FUNCTION SINE
 AMPLITUDE 3897 RPM 6950 PHASE 90 CYCLES 1000
 ARRIVAL TIME

0

DAMPING 0.1
 CUT OFF MODE SHAPE 20
 CUT OFF FREQUENCY 50

*

UNIT FEET KIP

LOAD 1 LOAD TYPE None TITLE FULL LOAD ACTING VERTICAL DIRECTION

JOINT LOAD

1 2 **FY** -38.4
 3 4 **FY** -34.4
 5 6 **FY** -21.4
 7 8 **FY** -33.2
 9 11 **FY** -38.5
 10 **FY** -6.8
 12 13 **FY** -3.6
 14 16 **FY** -16.9
 15 **FY** -50
 17 18 **FY** -17.8
 19 21 **FY** -9.8
 20 **FY** -31.9
 22 **FY** -16.9
 23 **FY** -13.1
 24 **FY** -11.3
 25 **FY** -15

LOAD 2 LOAD TYPE None TITLE FULL LOAD IN LONGITUDINAL DIRECTION

JOINT LOAD

1 2 **FX** 38.4
 3 4 **FX** 34.4
 5 6 **FX** 21.4
 7 8 **FX** 33.2
 9 11 **FX** 38.5
 10 **FX** 6.8
 12 13 **FX** 3.6
 14 16 **FX** 16.9
 15 **FX** 50
 17 18 **FX** 17.8
 19 21 **FX** 9.8
 20 **FX** 31.9
 22 **FX** 16.9
 23 **FX** 13.1
 24 **FX** 11.3
 25 **FX** 15

LOAD 3 LOADTYPE None TITLE FULL LOAD IN TRANSVERSE DIRECTION

JOINT LOAD

1 2 **FZ** 38.4
 3 4 **FZ** 34.4
 5 6 **FZ** 21.4
 7 8 **FZ** 33.2
 9 11 **FZ** 38.5
 10 **FZ** 6.8
 12 13 **FZ** 3.6
 14 16 **FZ** 16.9
 15 **FZ** 50
 17 18 **FZ** 17.8
 19 21 **FZ** 9.8
 20 **FZ** 31.9
 22 **FZ** 16.9
 23 **FZ** 13.1