May 6th, 12:00 AM

Behavior of compressor foundation-Predictions and observations

Shamsher Prakash

Vijay Kumar Puri

Follow this and additional works at: http://scholarsmine.mst.edu/icchge

Recommended Citation
http://scholarsmine.mst.edu/icchge/1icchge/1icchge-theme6/2

This Article - Conference proceedings is brought to you for free and open access by the Geosciences and Geological and Petroleum Engineering at Scholars' Mine. It has been accepted for inclusion in International Conference on Case Histories in Geotechnical Engineering by an authorized administrator of Scholars' Mine. For more information, please contact weaverjr@mst.edu.
SYNOPSIS In a Compressor foundation undergoing excessive vibrations, its amplitudes at operating frequency and natural frequency in free vibrations were monitored. Also in-situ dynamic properties were determined to check design and predict its response. Since the soil constants are strain dependent, two sets of computations were done (1) from the known soil constants and permissible amplitudes and (2) from the known soil constants and the observed amplitudes. The soil constants were corrected for confining pressure and relative density of the non-cohesive soil also.

In both weightless spring theory (Barkan's Method) and elastic half space theory were used in predicting the response. A critical evaluation of these two design approaches has been made and necessity to monitor the performance of machine foundations is highlighted.

INTRODUCTION

A reciprocating compressor foundation was vibrating excessively. Its performance was monitored and in-situ soil properties were determined to check its design and compute its response.

Figure 1 shows a dimensional plan and section of the foundation. The pertinent machine and foundation data are as follows:

Operating speed 405 RPM
Weight of compressor and motor 11.0t
Horizontal unbalanced force = 0
t
Vertical unbalanced force Pz = 0.205 t
Horizontal moment Mxy = 0.185 t·m
Vertical moment Myz = 2.2 t·m

Permissible vibration amplitude (peak to peak) = 0.025 mm
Area of the foundation A = 7.103 m²
Weight of the foundation W = 49.79 t
Depth of foundation = 2.4 m

Subsequent to the monitoring of the foundation, performance, (Arya et al. 1978) and in-situ dynamic properties determination, the design of foundation by (1) the Barkan's approach and (2) elastic half-space approach for 2-cases have been discussed in this paper; one as for usual design stage as if the monitored performance of the machine is not known; two, after knowing the monitored performance. The two sets of computations are similar except that strains in the soil in two cases are different which affect the relevant soil properties considerably.

The computations by two methods and their comparison with the monitored performance throw light on the applicability of one method to the analysis of such problems better than the other. Remedial measures are described elsewhere (Arya et al. 1978).

OBSERVATIONS ON THE FOUNDATION

Amplitudes. Vertical and horizontal amplitudes of vibration were measured at a number of points on the foundation, with the pertinent data as follows:

Maximum amplitude of horizontal vibrations at top of block in y-direction 0.3156 and in z-direction = 0.1085 mm. Foundation was excited in free vibrations along x-directions and the natural frequency of free vibrations was observed to be 17.90 Hz. (Fig. 2)

In-situ Dynamic Properties. The dynamic properties of the soil used in the analysis of machine foundation may be determined by a number of laboratory or in-situ tests.

The most important parameters which affect these properties are (1) the mean effective confining pressure (2) the shear strain amplitude and (3) density in the soil. A good discussion on these corrections has been presented by (Nandakumaran and Puri 1977), Prakash and Puri (1977), Nandakumaran et al. (1977), Prakash and Puri (1981) and Prakash (1981) and Indian Standard Code (IS 5249 - 1977).

In-situ soil investigations consisted of (1) block vibration tests, (2) cyclic plate load tests and (3) standard penetration tests (Prakash et al. 1975). Figure 5 shows a typical borelog of the area. From the cyclic plate load test data, values of dynamic shear modulus "G" were computed. From the uncorrected standard penetration (N) values shear wave velocity "Vs" at a particular depth was determined from equation 1 (Ima 1977) and dynamic shear modulus "G" was computed from equation (2)

\[ \frac{V_s}{\sqrt{\rho G}} = 0.337 \]

in which \( \rho \) = mass density of soil. Values of "G" from different \( G \) tests were corrected for (1) effective confinement in each case computed for an effective overburden pressure of \( 1 \) kg/cm² using equation 3.

\[ \frac{G_1}{G_2} = \frac{G_1}{G_2} \]

where \( G_1 \) = shear modulus at an effective overburden pressure of \( \sigma_{v1} \)
and \( G_2 \) = shear modulus at an effective overburden pressure of \( \sigma_{v2} \)
The variation of $G$ with strains from these tests is shown in Fig. 4 curve A. Fig. 4 also shows a plot of $G_{\text{max}}$ vs. $\gamma_0$ obtained by dividing the ordinates of $G_{\text{max}}$ at this depth by the value of $G$ at the same depth using the data at 6.0 m depth.

(1) Value of $G_{\text{max}}$ at 2.4 m depth was computed using equation 1 and 2 and the $N$-value observed at that depth.

(2) The value of $G_{\text{max}}$ was corrected for effective overburden pressure and the value of $G$ at an effective overburden pressure of 1 kg/cm$^2$ computed using equation 3.

The Values of $G$ vs $\gamma_0$ (curve C) were obtained by multiplying $G_{\text{max}}$ with ordinates of curve B (Fig. 4). This plot was subsequently used to determine the values of $G$ at 2.4 m depth at the desired strain level for analysis of the foundation response.

The strain for two sets of computations (1) for design stage and (2) after monitoring performance, and the corresponding soil properties were picked up as follows:

1. Design Stage.

   Permissible amplitude in any mode = 0.0125 mm

   Average width of the foundation = 2104.5 mm

   Shear strain ($\gamma_s$) (Prakash = 0.0125 = 5.94x10$^{-6}$

   $G$ at $\gamma_0 = 5.94 \times 10^{-6}$ and $\bar{\sigma}_v$ = 1 kg/cm$^2$16885 kg/cm$^2$


   Measured amplitude in $y$ direction = 0.3156 mm

   and in $z$ direction = 0.1089 mm

   Shear strain $\gamma_0$ induced in the soil $G_{\text{max}}$ = 2104.5

   Value of $G_{\text{max}}$ corresponding to $\gamma_0 = 2.01 \times 10^{-4}$

   (Fig. 4) and effective confinement below the foundation

   $C_u = 415 \left(0.8381\right) 0.5$

   = 400 kg/cm$^2$

The corresponding value of $C_u$ from eqn. 5 is computed to be 5.130 kg/cm$^3$.

**PREDICTED RESPONSE OF THE FOUNDATION**

The methods commonly used for the analysis and design of foundations for machines are (1) Barkan’s approach and (2) Elastic half space approach. In the Barkan’s approach (Barkan, 1962) the foundation soil system is represented as a spring-mass system, the spring stiffness due to the soil and mass of the foundation and supported equipment only are considered and inertia of the soil and damping are neglected. In the elastic half space approach the vibrating footing is treated as resting on the surface of an elastic, semi-infinite, homogeneous, isotropic half space (Richart 1962). The elasticity of the soil and energy carried into the half space by waves travelling away from the vibrating footing (geometric damping) are thus accounted for and the response of such a system may be predicted using a mass-spring-dashpot model (Richart and Whitney 1967a, Richart, Hall and Woods 1970).

The dynamic response of the foundation was computed using both the above methods of analysis.

**Barkan’s Method**

(a) Vertical vibrations: - Natural frequency of vertical vibrations $\omega_{nz}$ is given by

$$ \omega_{nz} = \sqrt{\frac{C_u A}{m}} = \frac{\sqrt{K_z}}{m} $$

and amplitude of vertical vibration $\omega_{az}$ is given by

$$ \omega_{az} = \frac{P_z}{m(\omega_{nz} - \omega)} $$

in which $m$ = mass of the foundation and $K_z$ = stiffness of vertical soil spring.

(b) Simultaneous rocking and sliding: Limiting natural frequency of the foundation in sliding $\omega_{nx}$ is given by

$$ \omega_{nx} = \sqrt{\frac{C_x A}{m}} = \sqrt{\frac{K_x}{m}} $$

$C_x$ = Coefficient of elastic uniform shear = $1/2$ $C_u$ and limiting natural frequency in rocking $\omega_{nx}$ is given by

$$ \omega_{nx} = \sqrt{\frac{C_p L}{m^2 \omega_{nx}^2}} = \sqrt{\frac{K_x}{m^2 \omega_{nx}^2}} $$

$\omega_{nx}$ = operating frequency. The computed values of natural frequency of vertical vibrations and amplitudes of vibrations are listed in Table 1 line 1 and 7 respectively.
in which

\( C = \text{Coefficient of elastic non-uniform compression} = 2.C_u \)

\( I = \text{Moment of inertia of the foundation contact area about the axis of rotation and} \)

\( M_m = \text{Mass moment of inertia of the machine and foundation about the area of rotation.} \)

\[ M_m = M_m + m z^2 \quad (10) \]

in which \( M_m = \text{Mass moment of inertia of the system about an axis through its centre of gravity for the appropriate direction of vibration.} \)

The two natural frequencies of the system \( \omega_{n1} \) and \( \omega_{n2} \) due to combined rocking and sliding are obtained in terms of \( \omega_{nx} \) and \( \omega_{ny} \) using equation 11.

\[ \frac{\omega_h^2 + \omega_{nx}^2 + \omega_{ny}^2}{r} + \omega_{nx} \omega_{ny} = 0 \quad (11) \]

in which

\( r = \frac{M_m}{M_0} \)

The amplitudes of vibration due to combined rocking and sliding due to an exciting moment are given by equations 12 and 13 Horizontal displacement.

\[ A_x = \frac{C \cdot A \cdot z \cdot K_x}{\Delta (u^2)} \quad (12) \]

Rotation. \( A_\phi = \frac{C \cdot A \cdot n \cdot \omega^2}{\Delta (u^2)} \quad (13) \)

where \( \Delta (u^2) = m \cdot M_0 \left( \omega_{n1}^2 - \omega^2 \right) \left( \omega_{n2}^2 - \omega^2 \right) \quad (14) \)

Total horizontal displacement due to combined rocking and sliding \( A_x^2 \) is given by equation (15).

\[ A_x^2 = A_x + h \cdot A_\phi \quad (15) \]

where \( h = \text{height of block above the combined centre of gravity system.} \)

Total vertical displacement due to vertical vibrations and rocking is given by equation (16).

\[ A_z^2 = A_z + L \cdot A_\phi \quad (16) \]

where \( L = \text{distance of the point under consideration from the axis of rotation.} \)

The natural frequency and amplitude of motion in yawing were also computed and all the values computed for different modes of vibration for 2-values of the shear strain in the soil are listed in Table I.

ELASTIC HALF SPACE METHOD

(a) Natural Frequencies: The natural frequencies of the foundation may be computed using equations 6, 8, 9, and 11. The soil spring stiffness for different modes of vibration may be computed as follows (Richart and Whitman (1967), Richart, Hall and Woods (1970)).

\[ K_x = \frac{4Gr_0}{\pi} \quad (17) \]

\[ K_x = \frac{32(1-\nu)Gr_0}{7-8\nu} \quad (18) \]

\[ K_\phi = \frac{8Gr_0}{3(1-\nu)} \quad (19) \]

and \( K_\phi = \frac{15Gr_0}{3} \quad (20) \)

\( r_0 = \text{equivalent radius of the foundation and is given by eqns 21-23.} \)

For vertical vibrations or sliding

\[ r_0 = \sqrt{\frac{A}{\pi}} \quad (21) \]

For rocking

\[ r_0 = \sqrt{\frac{4L}{\pi}} \quad (22) \]

For yawing

\[ r_0 = \sqrt{\frac{4L}{\pi}} \quad (23) \]

Computed values of the natural frequency for different modes of vibrations are listed in Table I.

(b) Amplitudes of Vibration:

\[ A_z^2 = \frac{P_z}{S_n} \quad (24) \]

\[ K_z = \sqrt{\left(1-\frac{\omega_{n1}}{\omega_nz} \right)^2 + \left(2D_x \cdot \omega_{nx} \cdot \omega \right)^2} \quad (25) \]

where \( D_z = \text{Damping ratio} = \frac{0.425}{B_z} \quad (26) \)

\[ B_z = \text{Modified mass ratio} = \frac{1-\nu}{4 \cdot \frac{pr^3}{r_0^3}} \quad (26) \]

Amplitudes in Rocking and Sliding

Damped amplitudes in sliding and rocking due to the exciting moment \( M^2 \) are given by equation 27 and 28 respectively

\[ A_x = \frac{M_x^2}{K_x} \cdot \sqrt{(\omega_{nx})^2 + (2D_x \cdot \omega_{nx} \cdot \omega)^2} \quad (27) \]

\[ A_\phi = \frac{M_\phi^2}{K_\phi} \cdot \sqrt{(\omega_{n\phi}^2)^2 + (2D_\phi \cdot \omega_{n\phi} \cdot \omega)^2} \quad (28) \]

where

\[ \Delta(u^2) = \left[ \omega^{2} - \omega_{n\phi}^{2} \left( \frac{\omega_{n\phi}^{2} + \omega^{2}}{r} - \frac{4D_\phi \cdot \omega_{n\phi} \cdot \omega \cdot \omega_{nx} \cdot \omega_{ny}}{\sqrt{r}} \right) \right] \quad (29) \]
\( D_x = \text{Damping ratio in sliding} \)

and \( D_\phi = \text{Damping ratio in rocking} \)

\[
D_x = \frac{0.288}{\sqrt{B_x}} \tag{30}
\]

Where \( B_x = \text{modified mass ratio} = \frac{7 - 8\omega_0^2}{32(1-\nu)^2} \)  

\[
D_\phi = \frac{0.15}{(1+\phi/\sqrt{B_\phi}} \tag{31}
\]

In which \( B_\phi = \text{inertia ratio} = \frac{3(1-\omega_0^2)}{8} \cdot \frac{1}{\text{proportion}} \)

Total horizontal and vertical displacements may then be obtained using equations (15) and (16) respectively.

Amplitude in yawing may then similarly be computed (Line 12 Table 1)

**FREE VIBRATIONS**

The values natural frequencies in \( x \)-direction for a shear strain of \( 1 \times 10^{-6} \) (free vibration condition) are as follows:

**Barkan's method**

Elastic Half Space

<table>
<thead>
<tr>
<th>Frequency</th>
<th>Observed</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{x1} (Hz) )</td>
<td>14.75</td>
</tr>
<tr>
<td>( f_{x2} (Hz) )</td>
<td>36.77</td>
</tr>
</tbody>
</table>

**DISCUSSION AND CONCLUSIONS**

1. The computed amplitude \( A_x^* \) (line 10, cols. 3 and 4 for case I (\( \omega_0 = 5.94 \times 10^{-5} \)) is 0.1998 mm and 0.1091 mm respectively by Barkan's method and elastic half space approach against permissible amplitude of 0.0125 mm. Therefore, the foundation design needed revision. This highlights the necessity of proper design using realistic soil parameters in ensuring satisfactory performance of machine foundations.

2. The computed values of lower natural frequency \( f_{x1}^* \) in combined rocking and sliding (Line 5, cols. 5 & 6) for case II (\( \omega_0 = 2.01 \times 10^{-4} \)) are 5.93 and 7.03 Hz respectively, (frequency ratios of 0.88 and 1.09) respectively by Barkan's and elastic half space approach. These are too close to the operating frequency of 6.75 Hz. Hence large amplitudes should be anticipated, which actually have been observed to occur.

3. Vertical natural frequencies (line 1, cols. 3 & 4 and 5 & 6) show a remarkable agreement with each other. However, the natural frequencies in sliding and rocking (lines 4 & 5) differ from 1% to 31% from each other. The lowest natural frequency in horizontal free vibrations (\( \omega_0 = 10^{-6} \)) as computed by the Barkan's and elastic half space approach is 14.75 and 13.01 Hz. The percent error with respect to the smaller natural frequency is 16% and 26% respectively. Thus the computed natural frequency by Barkan's approach is closer to the observed frequency and the error by the elastic half space approach is large. No other published data is available on actual phototypes for comparison.

4. Amplitudes of vibration in the vertical direction \( A_y^* \) by Barkan's and elastic half space approach are 0.135 mm and 0.3031 respectively against the measured value of 0.1089 mm. Similarly in the case of horizontal vibrations the values of \( A_x^* \) are 0.4542 and 0.785 respectively against the measured value of 0.3156 mm.

The amplitudes computed using elastic half space model take into account the geometric damping. Even then these are higher than the undamped amplitudes by Barkan's approach and also much higher than the observed amplitudes. The observed amplitudes represent the overall effect of geometrical as well as material damping. Translational modes have a much higher geometrical damping associated with them compared to material damping but in rotational modes material damping may be significant since geometric damping is usually small. Therefore predicted amplitudes using half space model with geometric damping alone may be expected to be somewhat higher than observed amplitudes. In the present case the difference is much larger than what may be explained by inclusion of material damping in the system also. It was observed earlier (Richart and Whitman (1967b), that the half space approach generally do not agree with observed amplitudes and may be higher or lower than observed amplitudes and in some cases the difference may be as large as 100%. However, in this case, Barkan's approach predicts the amplitudes which are reasonably closer to observed amplitudes as compared with elastic half space approach. However a single set of data does not warrant a general conclusion.

5. There is an urgent necessity to monitor data on performance of machine foundations so that it may be possible to establish conclusively the superiority of one approach over the other in design of machine foundations. Such a data will be meaningful only if sufficient information on dynamic soil properties is also obtained.

**REFERENCES**


<table>
<thead>
<tr>
<th>TABLE I. Computed Natural Frequencies and Amplitudes</th>
</tr>
</thead>
<tbody>
<tr>
<td>S.No.</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>6</td>
</tr>
<tr>
<td>7</td>
</tr>
<tr>
<td>8</td>
</tr>
<tr>
<td>9</td>
</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>11</td>
</tr>
<tr>
<td>12</td>
</tr>
</tbody>
</table>


Fig. 2 Typical Free Vibration Reco.

Fig. 1 Foundation Details

Fig. 3 Typical Borelog

Fig. 4 G vs \( \gamma_0 \) and \( \frac{G}{G_{\text{max}}} \) vs \( \gamma_0 \)