SIMULATION ANALYSES OF VIBRATION TESTS ON PILE-GROUP EFFECTS USING BLAST-INDUCED GROUND MOTIONS

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ABSTRACT

Extensive vibration tests have been performed on pile-supported structures at a large-scale mining site to promote better understanding of the dynamic behavior of pile-supported structures, especially pile-group effects.

Two test structures were constructed in an excavated pit. One structure was supported on 25 tubular steel piles and the other on 4. The test pit was backfilled with sand of an appropriate grain size distribution to ensure good compaction. Ground motions induced by large-scale blasting operations were used as excitation forces for the tests.

The 3D Finite Element Method (3D FEM) and a Genetic Algorithm (GA) were employed to identify the shear wave velocities and damping factors of the compacted sand, especially of the surface layer. A beam-interaction spring model was employed to simulate the test results of the piles and the pile-supported structures. The superstructure and pile foundation were modeled by a one-stick model comprising lumped masses and beam elements. The pile foundations were modeled just as they were, with lumped masses and beam elements to simulate the test results showing that, for the 25-pile structure, piles at different locations showed different responses.

It was confirmed that the analysis methods employed were very useful for evaluating the nonlinear behavior of the soil-pile-structure system, even under severe ground motions.

Keywords: dynamic behavior of pile-supported structures, pile-group effect, nonlinear behavior, soil-pile-structure interaction model
1. INTRODUCTION

Vibration tests on pile-supported structures were conducted at a large-scale mining site in the USA. Two test structures were constructed in an excavated 4m-deep pit. The superstructures were exactly the same. One was supported on 25 tubular steel piles and the other on 4. The test pit was backfilled with sand of an appropriate grain size distribution to ensure good compaction, especially between the 25 piles. Ground motions induced by the large-scale blasting operations were used as excitation forces for the vibration tests. The tests were performed six times with different levels of input motions (Hijikata 2005).

The purpose of this analysis study was to determine if existing analysis methods could simulate vibration test results of pile-supported structures from the following viewpoints.

- (1) Pile-group effects of pile-supported structure
- (2) Vertical responses of pile-supported structure
- (3) Non-linear response of pile

Several response analyses were performed to simulate the vibration test results of pile-supported structures, even during severe ground motions.

First of all, the 3D FEM and the GA were employed to identify shear wave velocities and damping factors of the compacted sand especially at the surface layer, since the surface layer were considered to greatly influence the responses of the pile-supported structures. The same method was also used to obtain input motions to the test structures in the following response analyses.

Secondly, response analyses using a beam-interaction spring model were performed to simulate the test results for the piles and the pile-supported structures. The superstructure and pile foundation were modeled by a one-stick model comprising lumped masses and beam elements. The soil-pile interaction was modeled by nonlinear lateral interaction springs.

Finally, the pile foundations were modeled just as they were with lumped masses and beam elements to simulate the test results that, for the 25-pile structure, piles at different locations showed different responses. The soil-pile interactions were modeled with full matrix springs that were evaluated from the Thin Layered Element Method (TLEM). The strong nonlinearity of the sand around the piles was also modeled by interaction springs between the lumped mass and the surrounding free field.

2. VIBRATION TEST USING BLAST-INDUCED GROUND MOTIONS

The vibration test using ground motions induced by mining blasts is shown schematically in Figure 1. Vibration tests on pile-supported structures were conducted at Black Thunder Mine of Arch Coal, Inc. Black Thunder Mine is the largest coal mine in North America and is located in northeast Wyoming, USA.
The objectives of the vibration test were to investigate (1) pile-group effects of pile-supported structures, (2) vertical responses of pile-supported structures, (3) non-linear responses of piles themselves and (4) applicability of response analysis methods.

Figure 2 outlines the sand test pit and the test structures showing the locations of accelerometers in the test pit and the adjacent free field. Figure 3 shows the details of the 25-pile structure. Two test structures were constructed in an excavated 4m-deep pit. The test structures were exactly the same; one was supported on 25 tubular steel piles and the other on 4. The test pit was backfilled with sand with a grain size distribution suitable to ensure good compaction, especially between the 25 piles.

Accelerations were measured at the structures, in the test pit and in the adjacent free field. Pile strains were measured to evaluate the horizontal and vertical distributions of axial forces and bending moments in the piles. There were 47 channels for the accelerometers and 108 for the strain gauges.

The shear wave velocities of the test pit were measured after each test by geophones located in the test pit. Dynamic modal tests of the pile-supported structures were performed before and after the vibration tests to detect changes of the natural frequencies of the soil-pile-structure systems. The sampling frequency was 500 Hz.

The vibration test results are summarized in Table 1. In this study, simulation analyses were conducted for the test results of Test-1 (medium input level) and Test-2 (large input level). Test-0 was used to evaluate the initial properties of the soil-structure interaction system with smaller input motions.

![Figure 2 Test Pit and Test Structures](image)

![Figure 3 Details of the 25-pile structure](image)

### Table 1 Summary of Vibration Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Distance* (m)</th>
<th>Max. Acc. at Ground Surface (cm/s²)</th>
<th>Max. Acc. at Top Slab (cm/s²)</th>
<th>Max. Strain of Piles (micro strain)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Free Field</td>
<td>Test Bed</td>
<td>25-Pile Structure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EW NS UD</td>
<td>EW NS UD</td>
<td>EW NS UD</td>
</tr>
<tr>
<td>0</td>
<td>770</td>
<td>17 25 17</td>
<td>44 61 21</td>
<td>22 41</td>
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<tr>
<td>1</td>
<td>250</td>
<td>439 422 279</td>
<td>304 267 284</td>
<td>573 586 198</td>
</tr>
<tr>
<td>2</td>
<td>160</td>
<td>794 567 408</td>
<td>432 534 314</td>
<td>1244 748 246</td>
</tr>
<tr>
<td>3</td>
<td>240</td>
<td>349 382 283</td>
<td>222 366 296</td>
<td>355 541 225</td>
</tr>
<tr>
<td>4</td>
<td>190</td>
<td>410 607 357</td>
<td>272 401 337</td>
<td>939 689 223</td>
</tr>
<tr>
<td>5</td>
<td>1230</td>
<td>57 100 33</td>
<td>44 107 39</td>
<td>136 318 28</td>
</tr>
<tr>
<td>6</td>
<td>110</td>
<td>1683 1496 4065</td>
<td>1066 1356 3122</td>
<td>1449 1857 1955</td>
</tr>
</tbody>
</table>

* Distance between Test Site and the Center of Blast Area
3. SIMULATION ANALYSIS OF SOIL RESPONSE

3.1 System Identification of Soil Properties

Since the surface layer were loosened due to drying up and large ground motions, and since the surface layer was considered to greatly influence the responses of the pile-supported structures, the 3D FEM and the GA were employed to identify the shear wave velocities and the damping factors of the compacted sand, especially of the surface layer.

The responses of the test pit were measured at three levels: GL 0m, -2m and -4m. In the system identification analyses, it was assumed that the shear waves propagate vertically upward. A three-layer model was constructed on the basis of underground confinement pressure and the boundaries were at GL-0.4m and GL-0.8m. The shear wave velocities of the test pit measured by geophones located in the test pit were used as initial values for system identification. The system identification analyses were performed repeatedly until good agreement was obtained between the transfer factions evaluated from the measurement records and from the analyses.

The transfer functions evaluated from the acceleration records, initial values of identification analysis and optimal values obtained from identification analyses are compared in Figure 4. The transfer function evaluated from the acceleration records shows very good agreement with that obtained from the identification analyses. The 1st natural frequency of the test pit was around 15Hz (natural period: 0.07 seconds).

The optimal values for the shear wave velocities and the damping factors of the test pit are shown in Figure 5. The optimal values obtained from Test-1 corresponded to properties obtained by geophones, which confirm that the test pit was in a linear state. As the input motion levels increased, the shear wave velocities decreased and the damping factors increased. This was due to the non-linearity of the compacted sand in the test pit. The initial stiffness of the test pit decreased to 0.7 during Test-2.

![Figure 4: Comparison of Transfer Function](image)

![Figure 5: Optimal Values for Shear Wave Velocities and Damping Factors](image)
3.2 Simulation Analyses of Soil Response

The response analyses of the test pit were performed using a 3D FEM model. The strain-dependent properties of the sand were considered by multi-springs incorporated in the H-D model. The test pit and the supporting layer above GL-4.6m were modeled. The accelerations in three directions recorded at GL-4.0m were used as simultaneous input motions to the model. The acceleration records with different levels of input motions, such as Test-1 and Test-2, were used in the following analyses.

The nonlinear properties of the sand and soil were determined from the dynamic soil test results. The test results are shown in Figure 6 together with the G/Go-\(\gamma\) and h-\(\gamma\) relationships employed in the response analyses. The G/Go-\(\gamma\) relationships were normalized by the shear strain corresponding to G/Go=0.5. A density of 1.86g/cm\(^3\) was used for all layers of the analysis model based on the measurement results.

The distributions of the maximum responses for the EW and NS directions in the test pit are compared in Figure 7 for Test-1 and Test-2. The response displacements were greater in the EW direction than in the NS direction for both tests. There is good agreement between the test results and the analysis results. Although it is not reported in this paper, the 2D FEM analyses were conducted to confirm that the shape of the excavation had very little influence on the response of the test pit.

![Figure 6 G/Go-\(\gamma\) and h-\(\gamma\) Relationships](image)

![Figure 7 Distributions of Maximum Responses of Test Pit](image)
Figure 8 compares the acceleration records at the surface with the accelerations obtained from the response analyses for three directions. The durations of the horizontal and vertical accelerations were two to three seconds and the maximum accelerations were greater in the horizontal direction than in the vertical direction. The timings of the maximum horizontal and vertical accelerations were different.

Figure 9 compares the response spectra of Test-2 at the surface together with the test pit bottom for the EW and vertical directions. A peak is observed at 0.1s for the EW direction, and this peak is shifted to the longer direction due to non-linearity of the test pit compared with the dominant period of 0.07 seconds of Test-0, as can be seen in Figure 5. A peak is also observed at 0.1 seconds for the vertical direction. The amplification is smaller in the vertical direction than in the EW direction.

The shear stress and strain relationships in the EW direction obtained from the response analyses with simultaneous three-directional input are shown in Figure 10 at GL-2m for Test-2. The maximum shear strain was 0.02% and this reduced the shear stiffness ratio G/Go to around 0.7, which corresponds to the optimal values obtained by the system identification analyses, as shown in Figure 5.
4. SIMULATION ANALYSIS OF PILES AND SUPERSTRUCTURE USING BEAM-INTERACTION SPRING MODEL

4.1 Analysis Model

A dynamic response analysis using a beam-interaction spring model was performed to simulate the test results for the piles and the pile-supported structures. The superstructure and pile foundation were modeled by a one-stick model comprising lumped masses and beam elements, as shown in Figure 11 (Miyamoto 1995). The rocking spring of pile foundation was attached to the bottom level of the base mat. The soil-pile interaction was modeled by lateral interaction springs in consideration of pile-group effect. The dynamic response analyses of the soil-pile-structure system were conducted in the time domain with input motions obtained from response analyses of the compacted sand using 3D FEM.

The non-linear properties of the lateral interaction springs were evaluated step by step in accordance with the relative displacement between soil and piles. The lateral interaction springs were comprised of lateral soil springs \([k_a]\) and shear springs \([k_b]\). The lateral soil springs and the shear springs were constructed by condensing a full matrix spring of the pile foundation evaluated by the 3D TLEM.

The rocking spring of the pile foundation was evaluated from the soil stiffness at the pile tip and the axial stiffness of the piles without considering friction around the piles. Viscous damping was assumed for the pile foundation and the superstructure and the damping value was set at 1% for the 1st natural frequency of the soil-pile-structure system. In this analysis, ground motions with different amplitudes such as Test-1 and Test-2 were used as input motions. The nonlinearity of piles was not considered.

![Figure 10 Comparison of Shear Stress and Strain Relationships in EW direction](image)

![Figure 11 Beam-Interaction Spring Model](image)
4.2 Superstructure Response

Figure 12 compares the analytical and measured accelerations of the top slab and the base mat for Test-1 and Test-2. The accelerations were greater in the 25-pile structure than in the 4-pile structure. There was good agreement for Test-1. At the top slab, the analytical acceleration is faster in phase than after 3 seconds for Test-2. This is because the non-linearity caused by separation between piles and surrounding soil was not considered in the response analyses, and some space was created between piles and surrounding soil due to the severe vibration of the superstructure.

![Graphs showing comparison of accelerations](image)

Figure 12 Comparison of Accelerations obtained from Measurement and Analyses

The response spectra at the top slab are shown in Figure 13 for Test-1 and Test-2. Peaks are observed at 0.15 seconds for the 25-pile structure and 0.26 seconds for the 4-pile structure for Test-1, which was caused by the difference in soil springs due to the number of piles. Peaks were observed at 0.18 seconds for the 25-pile structure and at 0.30 seconds for the 4-pile structure for Test-2. The natural periods became longer in Test-2 compared with those in Test-1 because of the nonlinearity of the soil surrounding the piles. For both test structures, peaks were observed at 0.1 seconds. These peaks were created by amplifying the dominant components found in the input motions. The response analyses considering friction between piles and surrounding soil were also performed. It was confirmed that friction had very little effect on the responses of the superstructure.

4.3 Response of Pile Foundation

The distributions of pile bending moments obtained from the response analyses are compared with vibration test results for Test-1 and Test-2 in Figure 14. They can be divided into two parts. One was caused by inertial forces from the superstructure and is referred to as the inertial component. The other was induced by ground motions and is referred to as the kinematic component.
The maximum bending moment always occurred at pile tops for both test structures, and a large bending moment can also be found at GL-2m. There was good agreement between the analysis and test results. For the 25-pile structure in Test-1, the inertial component was much greater than the kinematic component in the upper half of the piles. The ratio of the kinematic component became greater in the lower half of the piles. For the 25-pile structure in Test-2, the influence of the inertial component became greater than that of Test-1 because the non-linearity of the sand increased with increasing input motions. For the 4-pile structure in Test-1, the inertial component was dominant over the whole length of the piles. As can be seen from these results, the balance between the inertial and kinematic components in the depth direction depended on the number of piles, input levels and so on.

The bending moments at the pile top and at GL-1.6m are shown in waveforms in Figure 15 for Test-1 and Test-2. The timings of the maximum bending moments at the pile top differed between the 25-pile structure and the 4-pile structure, since there were differences in

![Figure 13](response_spectra.png)

*Figure 13  Response Spectra at Top Slab for Test-1 and Test-2*

![Figure 14](moment_distribution.png)

*Figure 14  Comparison of Distributions of Bending Moment of Piles*
the responses of the superstructure. For both tests, the waveforms of the bending moments at the pile top and at GL-1.6m showed opposite phases and maximum values at the same timings. The analysis results showed good agreement with the test results for the pile top as well as at GL-1.6m.

![Figure 15 Bending Moments in Waveforms for Test-1 and Test-2.](image)

5. SIMULATION ANALYSIS OF PILES AND SUPERSTRUCTURE USING THIN LAYERED ELEMENT METHOD

5.1 Analysis Model

The analysis model employed is shown in Figure 16 (Iwamoto 2003). The pile foundations were modeled just as they were, with lumped masses and beam elements to simulate the test results showing that, for the 25-pile structure, piles at different locations showed different responses.

The soil-pile interactions were evaluated by a combination model comprising full matrix springs incorporating the interaction between piles and scholar springs, considering soil non-linearity. The full matrix springs were evaluated by the TLEM with equivalent linear sand properties. The viscous damping corresponding to the 1st natural period of the soil-pile-structure interaction system was used for the response analyses. This section describes the simulation analyses conducted for Test-1 only. The dynamic response analyses
of the soil-pile-structure system were conducted in time domain with simultaneous three-directional input motions obtained from the response analyses of the test pit using 3D FEM.

5.2 Response of 25-pile Structure

The acceleration records obtained at the top slab of the 25-pile structure are compared with analysis results in Figure 17. The responses in the EW and NS directions gave different phase shifts because the responses of the superstructure and the frequency properties of the input motions showed some differences in both directions. The analysis results show a good agreement with the test results.

The axial forces of piles of A2, A3 and A4 of the 25-pile structure are shown in waveforms in Figure 18. The axial forces of pile A3 showed the same phase as the vertical responses of the superstructure, as shown in Figure 17, which means that the horizontal responses of the superstructure had very little influence on the axial responses of pile A3. The axial forces in pile A2 were generated mainly by the rocking motions of the superstructure in the EW direction, because the phase of the axial forces corresponded to the EW responses of the superstructure. Pile A4 showed the highest axial forces because it was subjected to vertical motions and rocking motions in the NS and EW directions of the superstructure.

The horizontal distributions of the maximum values at the pile tops are shown in Figure 19 for the bending moment in the EW direction and in Figure 20 for the axial forces. The bending moments were greater in the outside piles than in the inside piles because of the pile-group effects. The maximum bending moments were 2.0kNm for the center pile and 3.4kNm for the corner piles and the latter were almost as twice the former. Since the axial forces were generated mainly by the rocking motions, the axial forces were small in the center pile and greater in the corner piles. The analysis results show good agreement with the test results.

![Figure 17](image1.png)  ![Figure 18](image2.png)
6. INVESTIGATION OF PILE-GROUP EFFECT OF 25-PILE STRUCTURE

The shear forces caused by the superstructure of the 25-pile structure and the relative displacements between the pile foundation and the surrounding soil are shown in waveforms in Figure 21 for Test-1 and Test-2. The shear forces took a maximum value at 3.0 seconds for Test-1 and at 2.4 seconds for Test-2, and at the same timing, the relative displacements also took large values.

The relationships between the shear forces $Q$ and the relative displacements $\delta$ were investigated for piles of A2, A3 and A4. The shear forces were evaluated from differences in pile bending moments. The relative displacements are the same as the above. The hysteresis loops between $Q$ and $\delta$ are shown in Figure 22 for the piles of A2, A3 and A4 for Test-1 and Test-2. The slope angles and the loop areas became greater from the center pile to the corner pile. The differences can be found in the positive and negative regions of the relative displacements in the hysteresis loops. This is because there were differences in soil-pile interaction when the pile moved forward and backward. This phenomenon was conspicuous for corner pile A4. These investigation results show that it is important to design pile-group foundations considering the fact that soil stiffnesses vary with pile location.

The corner pile was subject to soil nonlinearity more than the center pile. For Test-1, the ratio of the soil stiffness of the center pile to that of the corner pile was about 3.0 at 2.2 seconds, and reduced to 2.7 as the soil nonlinearity increased.

For Test-2, the ratio was about 3.0 at 2.1 seconds, which was similar to that found in Test-1. Then, the ratio reduced to 2.1 at 2.4 seconds when the shear forces took their maximum values, and further reduced to 1.8 at 3.7 seconds when the soil nonlinearity
increased, which resulted in the fading away of differences in soil stiffness with different locations. This means that the pile-group effects disappear gradually as the non-linear region of the soil around the piles increased outward.

![Shear Force vs Displacement](image)

**Figure 22  Relationship between Shear Forces and Relative Displacements (EW Direction)**

**7. CONCLUSIONS**

Simulation analyses were performed for the vibration test results of pile-supported structures. Concluding remarks are as follows.

1. The soil response displacements were greater in the EW direction than in the NS direction. The maximum shear strain of Test-2, large input level, was 0.02% and this reduced the shear stiffness ratio G/Go to around 0.7. There is good agreement between the test results and the analysis results.

2. A peak of response spectra of soil responses is observed at 0.1 seconds for the EW direction in Test-2 and this peak is shifted to the longer direction due to non-linearity of the test pit compared with the dominant period of 0.07 seconds of Test-0, small input level.

3. The maximum accelerations of the superstructures were greater in the 25-pile structure than in the 4-pile structure. Simulation results using beam interaction spring model are good agreement with the test results for the structure responses.

4. The natural periods of soil-pile-structure system became longer in Test-2 compared with those in Test-1, medium input level, because of the nonlinearity of the soil surrounding the piles.

5. The maximum bending moment always occurred at pile top for both test structures, and a
large bending moment can also be found at GL-2m. There was good agreement between
the analysis and test results.

(6) The occurrence time of the maximum bending moments at the pile top differed between the
25-pile and the 4-pile structure, since there were differences in the responses of the
superstructure. The waveforms of the bending moments at the pile top and at GL-1.6m
showed opposite phases and maximum values at the same timings.

(7) The bending moments of the 25-pile foundation were greater in the outside piles than in the
inside piles because of the pile-group effects. Since the axial forces were generated
mainly by the rocking motions of the superstructure, the axial forces were greater in the
corner piles than in the center piles.

(8) The soil spring hysteresis varied with the pile location, and the stiffness of the soil spring
became greater from the center pile to the corner pile due to the pile-group effect. The
pile-group effects disappear gradually as the non-linear region of the soil around the piles
increased outward.

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