Proceedings of Master Theses Presentation in September 2015

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International Graduate Program on Civil and Environmental Engineering

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Behavior of geogrid reinforced sand was investigated through experimental studies. Triangular and biaxial geogrids were used in this study. Influences of geogrid stiffness and aperture shape, soil density and footing shape on reinforcement behavior were studied from monotonic loading tests. Surface deformation was also monitored during the tests. Test results showed that geogrid stiffness was more obvious at large stress levels. Higher confinement effect was observed in triangular geogrid reinforcement. Reinforcement behavior was significantly affected by soil density. However, footing shape influence was noticed only after geogrid bending. Uniform surface deformation was observed in triangular geogrid reinforcement. From cyclic loading tests, performance dependency on footing orientation was known, and almost same behavior was observed between 45° rectangular footing orientation and circular footing.

1. INTRODUCTION

Behavior of geogrid reinforced soil was studied by many researchers over decades. However, there is limited information for triangular geogrid. In the present study, reinforcement behavior of triangular geogrid was studied under different conditions, together with different biaxial geogrids. Experimental program included monotonic and cyclic loading tests.

2. MATERIAL PROPERTIES

(1) Sample soil

Nikko silica sand No.(5) was used for model soil preparation. This sand has the following properties: \( \rho_s = 2.675 \text{ g/cm}^3 \), \( D_{50} = 0.448 \text{ mm} \), \( C_u = 1.98 \), \( C_c = 0.943 \), \( \rho_{d_{\text{max}}} = 1.628 \text{ g/cm}^3 \), \( \rho_{d_{\text{min}}} = 1.338 \text{ g/cm}^3 \), \( \epsilon_{\text{min}} = 0.643 \) and \( \epsilon_{\text{max}} = 1.0 \). This sand can be regarded as poorly graded sand (SP) according to USCS. The internal friction angle of sand (\( \phi \)) was 33.5°, determined by direct shear test.

(2) Geogrids

Four types of geogrids were used in the present study; including triangular geogrid (TX160), and biaxial geogrids (SS1, SS2 & SS35). They are products of Tensar. Their physical properties are shown in Table 1.

<table>
<thead>
<tr>
<th>Geogrid</th>
<th>TX160</th>
<th>SS1</th>
<th>SS2</th>
<th>SS35</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>M</td>
<td>C</td>
<td>X</td>
<td>M</td>
</tr>
<tr>
<td>( T ), kN/m</td>
<td>10</td>
<td>12</td>
<td>22</td>
<td>19</td>
</tr>
<tr>
<td>( a \times b ), mm</td>
<td>40</td>
<td>40</td>
<td>28 x 40</td>
<td>28 x 40</td>
</tr>
<tr>
<td>( t ), mm</td>
<td>3.1</td>
<td>2.8</td>
<td>3.9</td>
<td>4.2</td>
</tr>
<tr>
<td>( w ), g/cm³</td>
<td>245</td>
<td>215</td>
<td>340</td>
<td>620</td>
</tr>
</tbody>
</table>

M, C & X: machine, cross machine & diagonal directions
\( T \): tensile strength, \( a \times b \): aperture dimension, \( t \): junction thickness, \( w \): unit weight

3. EXPERIMENTAL PROGRAM

Sand was uniformly prepared in square tank (1 x 1 x 0.8 m) by using about 100 kg of sand for each lift. The average dry density (\( \rho_d \)) was 1.53 g/cm³ (\( D_r = 70% \)) in all tests, except in three tests in which loose sand (\( \rho_d = 1.45 \text{ g/cm}^3 \), \( D_r = 43% \)) was prepared. Circular footing (\( D = \Phi 17.5\text{cm} \)) and rectangular footing (\( B = 8.75 \text{ cm}, L = 27.5 \text{ cm} \)) were used in the study.

In monotonic loading tests, load was applied in steps, maintaining until settlement increase was less than 0.02 mm. In cyclic load tests, load fluctuation was controlled by function generator, ranging from 0 to 100 kPa (1 cycle/min), and applied up to 100 cycles. In this test, rectangular footing orientation was taken in account in 0°, 45° and 90° directions.

4. TEST RESULTS

![Fig.1 Load vs. settlement behavior for triangular and biaxial geogrid reinforcements](image)

(1) Effects of stiffness and aperture shape

As shown in Fig.1, initial portions of load-settlement curves of SS1 (CD), SS2 (CD) and SS35 (CD) are almost
same until 4 – 5 mm settlement, after which significant influence of stiffness was noticed, showing different rates of settlement. The calculated bearing capacity ratios ($BCR = q_{reinforced}/q_{unreinforced}$) are 1.19, 2.12 and 3.04 for SS1, SS2 and SS35 reinforcements. Hence, larger bearing capacity was obtained with stiffer biaxial geogrids.

For TX160 (CD), it performed almost same as SS35 until 15 – 16 mm settlement ($s/D$: 8 – 9 %), despite its lowest tensile stiffness. Its $BCR$ is 2.37, which is larger than that of SS1 and SS2. After $s/D = 9$ %, load-settlement curve showed a rapid decline. Hence, interlocking effect of triangular shape was obvious until $s/D = 9$ %. After that, tensile stiffness might significantly affect the reinforcement behavior. In addition, pull-out resistance between geogrid and sand might also be decreased with continuous bending, and this would result in the rapid decline of performance.

(2) Influences of soil density and footing shape

TX160 and SS35 geogrids were used to investigate the effects of soil density and footing shape, shown in Fig.1. For no reinforcement, the influence of footing shape is higher than that of soil density. The estimated ultimate bearing capacities ($q_u$) are 101.62 kPa, 97.31 kPa and 48.42 kPa for No geogrid (CD), No geogrid (CL) and No geogrid (RD). In contrast, significant effect of soil density was noticed in reinforced sand tests. The values of $q_u$ are 240.70 kPa and 189.88 kPa for TX160 (CD) and TX160 (CL). With SS35 geogrid, $q_u$ values are 308.51 kPa and 136.23 kPa for SS35 (CD) and SS35 (CL). Hence, influence of soil density is higher in biaxial geogrid reinforcement. On the other hand, footing shape effect is negligible for geogrid reinforcements until 200 kPa footing pressure, after which the curves with rectangular footing show rapid declines. This may be due to higher stress concentration of rectangular footing, and this consequently results in dramatic decrease of reinforcement behavior after geogrid bending.

(3) Surface deformation characteristic

Surface deformation was monitored at six locations (two in each direction: 0°, 45° & 90°). In all tests, obvious bulging was observed only after 10% $s/D$ ratio. Uniform surface deformation was observed in no reinforcement and TX160 reinforcement. After testing, it was found that this geogrid was radially deformed, resulting uniform radial surface deformation. However, uneven surface deformation was observed in biaxial geogrid reinforcement, from which highest amount of bulging was recorded in 45° direction. This deformation behavior corresponds to the bending patterns of biaxial geogrid. Consequently, large amount of soil particles might have been allowed to move in this direction, resulting non-uniform radial surface deformation. Radial surface deformation characteristics are illustrated in Fig.2.

(4) Cyclic load tests

For SS35 geogrid, settlement variation with 45° footing orientation (45SS35R) was almost same as that with circular footing (SS35C), and highest performance was observed in this orientation. Although initial settlements were same with 0° (0SS35R) and 90° (90SS35R) footing orientations, performance with 90° orientation is higher than that with 0°, when load cycle increases. This might be due to thicker ribs in 90° direction, thus, resulting higher interlocking effect. For TX160 geogrid, behavior of 45° rectangular footing orientation (45TX160R) is almost same as that of circular footing (TX160C) at all load cycles. However, highest performance was noticed with 90° orientation (90TX160R), while lowest with 0° orientation (0TX160R). Therefore, highest interlocking effect was achieved in 90° orientation. Settlement variations of all cases are presented in Fig.3.

5. CONCLUSION

Monotonic and cyclic load tests were performed to investigate the behavior of geogrid reinforcement. For geogrid stiffness, this influence has been observed at large footing settlements. Higher ultimate bearing capacity was achieved with stiffer biaxial geogrid. In contrast, performance TX160 was higher than SS1 and SS2, and almost same as SS35 until $s/D = 8 – 9$ %. For soil density, this effect is significant for both geogrid reinforcements. However, it is less for no reinforcement. On the other hand, footing shape effect is obvious only after geogrid bending. The reverse behavior was observed in no reinforcement. For surface deformation, uniform radial deformation was observed in triangular geogrid reinforcement, while uneven surface deformation was found in biaxial geogrid reinforcements. These deformations coincide with the bending patterns of geogrids. In cyclic load tests, performance with circular footing was almost same as that with 45° rectangular footing orientation for both TX160 and SS35 geogrids. Additionally, performance dependency on footing orientation was noticed for both geogrids, corresponding to the effective interlocking of geogrids.
The effects of soil nonlinearity upon seismic response of soil-pile-structure system are investigated through shaking table experiments with scaled model of soil-pile-structure system and analytical computation employing superposition of kinematic and inertial interaction effects approach. Experimentally evaluated effective foundation input motion and pile head impedance are utilized in the analytical computation. The experimental responses at superstructure and footing show that the natural frequency and amplification of motion decrease with increasing amplitude of ground excitation which is comparable to anticipated trend. Discrepancies have been seen between experimental and analytical response amplification of superstructure particularly around the resonant frequency of superstructure and soil. Analytical resonant frequency of superstructure shifts toward higher frequency range while the resonant frequency of soil and footing shifts toward lower frequency range, as compared to experimental results. Disparity between site nonlinearity in soil induced by ground excitation and local nonlinearity induced by pile head loading reveals the necessity to investigate the effect of site nonlinearity in soil in estimating pile head impedance rather than considering the effect of local nonlinearity in soil only.

1. INTRODUCTION

Soil-pile-structure interaction (SPSI) analysis possess great importance in studying the seismic behavior of pile supporting structures as the responses of soil-pile and structure are interdependent. Several numerical and classical analytical methods have been introduced and utilized for performing such kind of SPSI analysis. Most of the methods are limited to encompass only the linear behavior of soil. Moreover, discrepancies observed between the numerical and analytical solution encourage the verification of results through experimental investigation and also lead to development of more simplified three consecutive step method for SPSI analysis considering superposition of kinematic interaction and inertial interaction effects. This research work is intended to make experimental and analytical investigation to understand the effect of soil nonlinearity upon seismic response of soil-pile-structure system and for verification of superposition of kinematic and inertial interaction effects approach.

2. METHODOLOGY

The effective foundation input motion and total response of the superstructure and footing are estimated by shaking table experiments with scaled model of soil-pile-structure system for harmonic loadings of amplitude (0.5 m/s², 1.0 m/s², 2.0 m/s², 3.0 m/s², 4.0 m/s², and 5.0 m/s²) and frequency range of 6 Hz-35 Hz to encompass typical structural periods. The analytical response is computed utilizing three consecutive step method described by Gazetas as follows: 1) Kinematic interaction effects: Determine the foundation input motion in response to base excitation in the absence of superstructure inertia. 2) Foundation stiffness: This is frequency dependent dynamic impedances (springs and dashpots), which express dynamic force-displacement ratios at the pile head or pile group cap. 3) Inertial interaction effects: Analysis of the dynamic response of the superstructure supported on the springs and dashpots of step 2 and subjected to the foundation input motion of step 1 at its base. Experimentally estimated effective foundation input motion and foundation stiffness reported by Goit are utilized as input in the single-sway model of soil-structure-foundation system for analytical computation. Euler-Lagrange equation of dissipation system is used in deriving the formula to compute analytical response.

3. EXPERIMENTAL COMPONENTS

An one degree of freedom shaking table of size 1800 x 1800 square millimeters and of capacity 5 (t-G) in full load was used for the experiments. Nine solid acrylic piles with diameter of 40 mm and length of 900 mm each, capped by a solid acrylic pile-cap of dimensions 280 mm x 280 mm x 100 mm was used to form a 3 x 3 pile group. Two different forms of superstructure with same total stiffness of 470.4 KN/m was used: 1) moving part weight = 50.3 kg, natural frequency \( f_{11} = 15.1 \text{ Hz} \) and 2) moving part weight = 23.02 kg, natural frequency \( f_{11} = 21 \text{ Hz} \). Cohesionless dry Gifu sand was employed to form soil mass. A laminar shear box of inner dimension 1200 mm x 800 mm x 1000 mm is used.
to mount the soil-pile-structure model on shaking table as shown in Figure 1.

4. RESULTS

Experimental total response of superstructure and footing shows that the natural frequency and amplification of motion decrease with increasing ground excitation. Experimental and analytical amplification of response at footing shows reasonable agreement except for one case. For superstructure responses, discrepancies have been seen between experimental and analytical amplification ratio particularly around the resonant frequency of superstructure and soil as shown in Figure 2. Analytical resonant frequency of superstructure is shifted towards higher frequency range while the resonant frequency of soil and footing is shifted towards lower frequency range, as compared to experimental results. Soil displacement field at resonant frequency of the superstructure and at resonant frequency of soil & footing shows that the effect of site nonlinearity in soil induced by ground excitation is greater than the local nonlinearity in soil induced by pile head loading as presented in Figure 3, which basically reveals the possibility of occurrence of an lower impedance than that used for analytical computation estimated by pile head loading considering local nonlinearity of soil only.

5. CONCLUSIONS

The superposition of kinematic and inertial interaction effects is a reasonable approximation. However, for the dominant mode of vibrations nonlinearity in soil induced by kinematic effects and inertial effects possess great importance to understand the discrepancies observed between experimental and analytical result. Estimating pile head impedance considering site nonlinearity in soil might eventually diminish the mismatch between analytical and experimental resonant frequency of superstructure. And the discrepancies between analytical and experimental resonant frequency of soil & footing suggests that for this part of computation it is worth to investigate the simultaneous effect of local and site nonlinearity in soil on pile head impedance.

REFERENCES

Development of a Smart device system for the identification and interactive visualization of structural response

13ME154 Sandhya Nepal
Supervisor Prof. Hideji Kawakami

Modern smartphones are equipped with variety of sensors for measuring the physical phenomena around their surroundings. Due to the significant computational power and large memory resources, smartphones can be used in various applications. One of such fields are for measuring seismic activity; seismic sensors for collecting, analyzing and visualizing the measured data in more and more advanced way. This research is focused on the measurement of structural properties of the civil engineering structures in an economical and effective way. An Android application for collecting the oscillation data from the structures, analyzing these events and representing these results visually has been developed. This system measures the vibration of the structure (i.e., acceleration time history) in all three directions, and calculates the natural frequency, velocity time history, displacement time history, Fourier spectrum and response spectrum of the structure. Thus, the creation of this system can enable every citizen to monitor their building effectively and without any economic burdens.

1. INTRODUCTION

Structural health monitoring (SHM), where different kinds of sensors are installed in a structure to make the structure able to sense its outer environmental loads and respond to these loads, in order to boost its performance and survivability. Nowadays, SHM becomes more and more important in people’s lives, and it has become a common to arrange the sensors to monitor the health of some major structures so that preventive action can be taken well in advance. However, the implementation of SHM technique in civil engineering has economic burdens associated with instruments and monitoring. Indeed, SHM is only used on some large-scale structures and has to be accomplished with expensive monitoring system by professional staff, leading to less application in small structure. It has become a serious matter to make the system popular in small structures and thus play a more important role in people’s daily lives when facing natural disasters (earthquake, hurricane, or floods).

Modern Smartphones are highly integrated devices that have significant computational power, large memory resources and are equipped with a variety of sensors in a very low price as compared to those devices. The sensor of the modern smartphones are based in the technology of very small devices called MEMS (Microelectromechanical systems). Thus, the sensor in the mobile device is also similar to the relevant sensor and can also be used for the similar function as other conventional types of sensor\(^1\). The possibilities of the smartphone to serve as the effective and economic seismic sensor due to its capabilities and features motivated this study.

2. SYSTEM OVERVIEW

First, an open source project ‘Earthquake Reading’ was selected which detect the motion event and modified to the system ‘Earthquake Analysis’ which detect and record the motion event (acceleration time history), stored recorded event in internal memory, processed it for different results and finally sending recorded event to server. Figure 1 shows the overview of the system.

![Figure 1 An overview of an earthquake analysis application processes](image)

The Earthquake analysis application has four components: sensing loop, sending loop, processing loop and visualizing loop. While the sensing loop interact with the main application and in-built sensors, the sending loop handles local storage, transmission of recorded events, and stored all the recorded events, the processing loop handles analysis part of the system and the visualizing loop handles the graphical representation of all the results.

(1) Description of the system

An android system is designed which,
If the system sense the vibration or jolt, it detect the motion and starts recording the event. This recorded event will stored in the internal memory and this can also be stored in the server via email. Further, all the recorded events are processed and analyzed by different methods such as Newmark-beta, Runge-kutta method, FFT, and omega arithmetic to obtain time histories, Fourier spectrums and response spectrums.

3. RESULTS AND DISCUSSIONS

(1) Identification of structure

If the system/application is used to collect the vibration data of the structure caused by natural forces such as earthquakes, then these data can be analyzed to identify the structural performance and properties like dominant period, acceleration, velocity, and displacement experienced by the structure. The period/frequency of the structure may also be changed due to earthquake. If the structure’s period approaching to that of the ground it experiences resonance, which may damage an already weakened structure. Hence, the condition of the structure can be determined by the change in dominant frequency (period of the structure).

Further, the developed system helps to determine maximum acceleration, velocity, and displacement experienced by the structure, caused by natural forces.

Moreover, this system helps to determine performance point using the capacity spectrum method. In this example, capacity curve of four-story reinforced concrete frame building was used and is combined with an acceleration displacement response spectrum (ADRS) of recorded earthquake and design spectrum (figure 3).

Considering the ductility of the structure, the intersection point of the design spectrum and capacity spectrum gives the performance point, i.e., spectral displacement can be evaluated as shown in figure 3. For this particular structure, performance point was determined at ductility factor $u = 1.69$ and spectral displacement was found to be $0.103m$. And, while comparing ADRS curve and capacity curve, ADRS curve intersects the capacity curve in the elastic portion, hence this particular structure is safe for the recorded earthquake.

Further, we can also compare design spectrum and ADRS curve, to determine the condition of the structure during earthquake. From above figure 3, it can be seen that, ADRS curve lies below the design spectrum. Thus, the structure constructed using this design spectrum is safe for this particular earthquake.

(2) Determining features of an earthquake

If the system/application is used to collect the vibration data of earthquakes, then these data can be analyzed to identify the severity and properties like natural frequency, acceleration, velocity, and displacement of the earthquake.

From the acceleration time history, we can calculate peak horizontal acceleration (PHA) which is an important parameter of any design. Further, velocity is also an important parameter for damage detection. Peak horizontal velocity (PHV) is better that PHA for intermediate frequencies as velocity is less sensitive to higher frequency. The frequency component of an earthquake history is often described using Fourier spectra and response spectrum. From the designed system, we can determine the natural frequency of any earthquake easily, which is an important parameter of damage detection.

Moreover, response spectrum is used to provide the most descriptive representation of the influence of a given earthquake on a structure. In the designed system, response spectrum of SDOF was determined from the recorded motion. Using the response spectrum, the peak response of the buildings to earthquakes can be assessed and their natural frequency/time period can be determined.

4. CONCLUSIONS AND FUTURE WORKS

- Smart device’s sensor can be used as a seismic sensor economically and effectively.
- Many vibration data in all three directions can be collected easily at low price. Thus, the creation of this system can enable every citizen to monitor their building effectively and economically.
- This research only includes development of the system, so experiments are needed to verify the result before using it practically.

REFERENCES