Research Article

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Nonlinear numerical simulation of dynamic response of pile site and pile foundation under earthquake

Abstract: To study the influence of the nonlinear connection of pile and soil on the dynamic response characteristics of the pile foundation, this article proposes to study the dynamic response of the bridge pile foundation to the slope by combining the centrifugal shaking table test and OPENSEES open source finite element program. This article introduces the pressure-dependent multiyield surface model based on confining pressure. Through the inverse calculation of the similarity ratio of the centrifuge model test, the OPENSEES two-dimensional nonlinear finite element model of the pile group in the slope section can be established. The centrifuge shaking table test is to input the preset seismic wave horizontally at the bottom of the model box. The form of seismic wave is El Centro wave verification of two-dimensional finite element model of the pile group in slope section under earthquake. The reliability of the model is verified by comparing the test and calculated values of dynamic response (residual horizontal displacement and final bending moment) of the pile body under five different peak acceleration seismic wave loading conditions. In the dynamic response experiment of slope pile foundation, in the embedded part below the bedrock surface, the residual horizontal displacement of each pile body is zero. Constrained by the cap beam and tie beam, the displacement of the free section of the pile group at these two positions is basically the same. Through comprehensive analysis, the displacement of P1 and P2 piles is basically the same. The calculated value of the final bending moment of P1 and P2 piles shows the same change trend as the test value, and the test value is slightly larger than the calculated value. The relative errors of the maximum final bending moment of P1 pile under each loading condition are 7.4, 7.8, 12.6, 3.9, and 9.6%, respectively, and the relative errors of P2 pile are 4.6, 3.6, 12.5, 13.6, and 11.5%, respectively. The analysis relative error is caused by the elastic element used in the calculation of the pile body, which is different from the mechanical behavior of the simulated pile body material in the test. Dynamic response of slope site according to the existing centrifuge test results can be seen that the deformation at the slope shoulder of slope site is the most obvious under the earthquake. The inclined interface of soft and hard rock and soil layer will aggravate the dynamic response of the overburden layer on the slope, weakening its ability of seismic energy consumption.

Keywords: earthquake, pile laying site, dynamic response of pile foundation, nonlinearity, numerical simulation

1 Introduction

Most of the reasons for the destructive nature of earthquakes are as follows: during an earthquake, a weak surface called a fault in the crust suddenly ruptures and dislocates, and this rupture and dislocation may sometimes directly lead to the Earth’s surface. Severe damage occurs directly when the ground ruptures through the foundations of houses, structures, or underground pipelines, but such damage can only threaten structures that happen to lie in the ground rupture zone. In recent years, the crustal movement is relatively active and earthquake disasters occur frequently. Earthquakes are still one of the main natural disasters suffered by mankind. Earthquake refers to the phenomenon that the long-term accumulated deformation of rocks in the Earth’s crust is transformed into kinetic energy in a very short time, releasing energy,
causing vibration and generating seismic waves [1]. Due to
the sudden occurrence of earthquake disasters, the damage
caused by earthquakes is often accompanied by serious
casualties, secondary disasters such as fires and barrier
lakes, as well as the corresponding social impact, in addi-
tion to the direct economic and property losses. Previous
earthquakes have shown that the causes of building damage
cau sed by earthquakes can be roughly divided into two
categories: the first is the damage caused by the vibration
of the structure caused by earthquakes; The second type is
the damage caused by the failure of foundation [2]. The
failure of the foundation and foundation under an earth-
quake is mostly caused by foundation liquefaction. The
liquefaction of the foundation sand caused by the earth-
quake will make the foundation of the structure lose its
bearing capacity and directly endanger the safety of the
building. The state of saturated loose sand will change
rapidly under the action of ground motion load and lose
its original shear strength and bearing capacity, resulting
in the destruction of ground and aboveground buildings,
which is the so-called liquefaction phenomenon [3]. For
many important structures, such as high-rise buildings,
bridges, and nuclear power facilities, the seismic design of
pile–soil structure system is a very important part. The
seismic response analysis of pile–soil structure interaction
system involves soil nonlinearity, motion interaction
between pile and soil, and dynamic interaction between
structure and soil, which has been studied at home and
abroad [4,5]. The dynamic damage of buildings is mainly
manifested in two forms: damage caused by insufficient
strength of the main structure and structural loss of integ-
re. The strength failure is mainly caused by insufficient
shear, bending, and compressive strength of structural
load-bearing members, such as wall cracks, cracking or
cracking of reinforced concrete members. Before and after
the strength failure of structural members, the structure
generally enters the elastoplastic deformation stage. At
this stage, under the action of strong vibration, the structure
will lose its integrity due to insufficient ductility, failure
of joint connections, and instability of main load-bearing mem-
bers, resulting in partial or whole structure collapse.

The type and properties of rock and soil have the
most significant influence on macroscopic intensity, and
it is also the most deeply studied factor. According to
the research, it can be investigated from three aspects: the
degree of softness and hardness of the rock, the thickness
of the soft soil, and the stratigraphic structure.

Generally speaking, under the same seismic force,
the earthquake damage on the rock is the lightest, which
is the hard soil, and the soft soil is the heaviest. The
influence of the thickness of the soft sediment on the
earthquake disaster is also obvious. The rock properties
and the thickness of the soft soil will have an impact on
the earthquake disaster, and the fundamental reason is
the effect of the characteristic period. Because the softer
and thicker the soil is, the longer its characteristic period
is. Therefore, high-rise buildings and wooden-framed
houses with longer natural vibration periods can cause
resonance and aggravate earthquake disasters. In addi-
tion, the longer the vibration time of the thick layer of soft
goods, will also make the earthquake disaster aggra-
vated. If the surface is distributed with saturated fine
sand, silt, and silt, the foundation will fail due to vibration
liquefaction and seismic subsidence. The stratigraphic
structure also has a great influence on earthquake disas-
ters. The general situation is that the structure with the
hard bottom and the soft top has the most severe earth-
quake damage, while the structure with the soft bottom
and the hard structure has less damage. When there is a
soft soil interlayer in the hard soil, the seismic energy can
be reduced. The stratigraphic structure also has a great
influence on earthquake disasters. The general situation
is that the structure with the hard bottom and the soft
structure has the most severe earthquake damage, while
the structure with the soft bottom and the hard structure
has less damage. When there is a soft soil interlayer in the
hard rock, the seismic energy can be reduced. Based on
the existing research, this article proposes a method that
combines the centrifugal shaking table experiment with
the open source finite element software OPENSEES to
study the dynamic response of the pile foundation of the
inclined bridge.

2 Literature review

Aiming at this research problem, Cao and Wei studied the
influence of horizontal and vertical seismic interaction on
the dynamic liquefaction site pile bridge based on OPE-
NSEES finite element seismic simulation platform [6].
Beer et al. studied the influence of vertical acceleration
on site liquefaction effect through the ICFEP finite ele-
ment program [7]. Zhang et al. considered the influence
of horizontal coupling and analyzed it through the finite
difference program FLAC [8]. Feng et al. believe that
under the cyclic load of ground motion, which leads to
the reduction of its effective stress, the reduction of soil
shear capacity or the complete loss of its own shear
bearing capacity, and the soil body is in a near-water-
like liquid state [9]. Bhaduri and Choudhury calculated
the seismic response of the pile–soil bridge structure
using two-dimensional (2D) and three-dimensional (3D)
finite elements, assuming that the motion between the pile and the soil is correlated and relatively non-slip [10]; Zhang et al. found that the failure of pile foundation is closely related to its slenderness ratio. Through earthquake damage investigation and centrifuge test, the failure mechanism of the pile foundation on the liquefied foundation is explored. The dynamic interaction and inertial interaction of piles on the liquefied foundation are divided into four stages, as shown in Figure 1 [11]. Chen et al. through laboratory model tests, the mechanical properties of passive piles are studied and the final Earth pressure acting on the piles is determined. The ultimate soil pressure of the pile body with two rows of piles and two rows of piles with a spacing of three times and five times the pile width, respectively, \((B = 20\, \text{mm})\) is lower than that of a single pile; for single piles and coupled passive piles, and the limit soil pressure along the pile body. The distribution is different [12]. Long et al. contact surface element is widely used in linear and nonlinear finite element calculation of pile–soil interaction because of its clear concept and convenient calculation [13]. Zhang et al. solved the problem of energy loss in pile–soil interaction by introducing the damping term into the Goodman element [14]. Zhang et al. studied the mechanical response characteristics and failure law of the support system and the foundation pit slope during the excavation process of the deep foundation pit and the surrounding loading of the foundation pit based on the model test of the micro-pile support in the deep foundation pit [15] Marinichev and Tkachev discussed the correlation between the support forms such as composite soil nails, cast-in-place piles-anchor rods, and the displacement of the foundation pit slope based on the field monitoring data [16]. Feng et al. took the foundation pit project at the entrance and exit of a station of Dalian Metro Line 1 as the research object and used FLAC 3D simulation technology to simulate the excavation process of unsupported slope and soil nail support. Nail support can effectively limit the plastic deformation, surface settlement, and soil displacement of each part of the soil after the original stress balance is destroyed, so as to ensure the stability of the foundation pit [17]. Gazali studied the deformation response of expansive soil foundation pit slope under different foundation pit supporting structures [18].

Based on the existing research, this article proposes a method that combines the centrifugal shaking table experiment with the open source finite element software OPENSEES to study the dynamic response of the pile foundation of the inclined bridge. The experimental results show that the numerical model can better reflect the measured law; with the increase in the El Centro wave ground acceleration amplitude, the maximum residual horizontal displacement increases nonlinearly. The maximum final bending moment of the pile body appears at the junction of bedrock and soil layer. The shoulder of the slope site is more prone to shear deformation than the top of the slope under earthquake, and the inclined interface of soft and hard rock and soil layer will weaken its ability of seismic energy consumption.

3 Method

3.1 Pile soil constitutive model

3.1.1 Multiple yield surface model

The multi-yield surface model can simulate the elastic-plastic state of soil under dynamic action, and effectively

![Figure 1: Schematic diagram of pile foundation failure loading time history in Bhattacharya and other liquefaction sites.](image-url)
reflect the stress response of soil under variable load conditions under complex stress paths and seismic loads. The pressure-dependent multi-yield (PDMY) surface model based on confining pressure integrated into OPENSEES software can effectively simulate the nonlinear deformation effect of cohesive soil under reciprocating load, reflecting the stress–strain cyclic hysteresis phenomenon. In the PDMY constitutive model, it is assumed that the elastic stage is linearly isotropic, while the plastic stage shows nonlinear anisotropy due to the stress path. The yield function formula is as follows:

\[ f = \frac{3}{2} (s - pa) : (s - pa) - m^2 p^2 = 0, \]  

(1)

where \( s = \sigma - p\delta; p = p' + p'_0; m = 6 \sin \phi/(3 - \sin \phi) \).

Here, \( s \) is the partial stress tensor; \( p' \) is the effective average stress; and \( p'_0 \) is a small normal number (generally 1.0 kPa), so as to ensure that the size of the yield surface is still limited when \( p' \) is 0; \( m \) is the size of yield surface; \( \phi \) is the internal friction angle; and \( \alpha \) is the motion partial tensor of the central coordinate of the yield surface in the principal stress subspace. The yield surface appears as a conical surface in the principal stress space, while a series of conical surfaces with common vertices form a hardening zone, and the outermost conical surface is defined as the failure surface.

### 3.1.2 Pile soil dynamic spring model

The interaction between pile and soil is simulated by a series of dynamic springs. These dynamic springs are expressed by inserting nonlinear uniaxial zero-length elements discretized at the same position of continuous soil and pile body, so as to consider the three-dimensional dynamic stress effect of pile foundation. There are three types of springs: \( p-y \) spring simulates the soil resistance of the soil around the pile along the vertical direction of the pile body; \( T-z \) spring simulates the vertical resistance caused by friction along the outer surface of the pile body; and \( Q-z \) spring simulates the bearing capacity of soil at the pile end.

### 3.2 Numerical calculation model of slope pile

Through the inverse calculation of the similarity ratio of the centrifuge model test, the OPENSEES two-dimensional finite element model of the pile group in the slope section can be established. The slope of the model is about 27°, which is divided into two layers of rock and soil: sand and bedrock. The thickness of the sand layer is 11.0 m, and the bedrock surface is parallel to the upper surface of the sand layer, with a thickness of 3.25–14.0 m. The pile length of the model pile group foundation is 27.5 m, and the pile spacing is 2.4 m. The sand is simulated by the PDMY model introduced above, and the bedrock and pile body are simulated by the elastic element model. Table 1 lists its parameters.

The lateral boundary of the model takes part of the soil as the free field element, and its thickness is much greater than that of the internal soil element. Then, the periodic boundary conditions are realized by the given displacement equivalent command. The bottom boundary is set to be fixed vertically, and LK viscous boundary is adopted horizontally to simulate the absorption of reflected waves from the bottom boundary by the underlying layer of an elastic body in semi-infinite space. The seismic wave is transmitted from the bottom node in the form of equivalent force \( P \), which is calculated by the recommended formula:

\[ P = \rho_E v_s A_E \ddot{u}, \]  

(2)

where \( \rho_E \) and \( v_s \) are the mass density and shear wave velocity of the underlying layer, respectively; \( A_E \) is the total cross-sectional area of the base; \( \ddot{u} \) is the time history of input seismic wave velocity. Generally speaking, the minimum grid size is required to ensure the smooth passage of the wave with the highest frequency of the input seismic wave. The lower limit of the cell grid size shall meet the following requirements:

\[ \Delta h \leq \frac{1}{8} \lambda_{\text{min}} = \frac{1}{8} \frac{v_s}{f_{\text{max}}}, \]  

(3)

where the height \( \Delta h \) of the quadrilateral element is selected according to the wavelength \( \lambda_{\text{min}} \) passing through the rock and soil mass. In order to correctly capture the propagation of an earthquake in the grid, there must be at least eight units with the shortest wavelength to prevent the relevant higher frequencies from being filtered out. From the spectrum characteristics after seismic wave processing earlier, it is

| Table 1: Material parameters of bedrock and pile foundation |
|-------------|-----------|-----------|-----------|
| Density (g/cm³) | Modulus of elasticity (GPa) | Poisson’s ratio | Diameter (m) |
| Bedrock | 2.2 | \(3 \times 10^7\) | 0.35 | — |
| Pile | 2.1 | \(2.85 \times 10^7\) | 0.35 | 0.9 |
determined that the maximum frequency content \( f_{\text{max}} \) in the selected seismic input is 20 Hz.

The centrifuge shaking table test is to input the preset seismic wave horizontally at the bottom of the model box. The form of seismic wave is the El Centro wave. According to the table accelerometer of the shaking table, the measured peak acceleration of seismic waves in fewer than five loading conditions is 0.1497, 0.2106, 0.3055, 0.4303, and 0.4809 g, respectively. Through one-time integration and baseline calibration, the corresponding seismic wave velocity time history data required in the numerical model can be obtained, and the equivalent effectiveness of input nodes under various numerical calculation conditions can be obtained through Eq. (2) [19].

The reliability of the model is verified by comparing the test and calculated values of dynamic response (residual horizontal displacement and final bending moment) of the pile body under five different peak acceleration seismic wave loading conditions. The test value can calculate the bending moment corresponding to the measuring point by applying the strain value \( \varepsilon \) measured on the pile body to the simple beam theory and satisfying the relationship (4) [20]. The variation relation \( m(z) \) of the bending moment along the pile depth can be obtained by the seventh-degree polynomial fitting 201 of the bending moment value of each measuring point. The displacement data of the pile top can be obtained by the quadratic integration of the acceleration data measured by the pile top accelerometer, and the overall displacement distribution of the pile body can be obtained by the integration [21]

\[
M(z) = EI \times \frac{d^2u}{dz^2} = EI \times \frac{\varepsilon}{h}.
\]

Here, \( z \) is the depth coordinate along the pile; \( h \) is the distance from the pile in meters to the neutral axis of the pile; and the flexural stiffness of the pile is \( EI \).

4 Results and analysis

4.1 Dynamic response of slope pile foundation

Figures 2 and 3 show the residual horizontal displacement of P1 and P2 piles under different peak acceleration seismic wave loading conditions. The residual horizontal displacement of each pile body is basically zero. Constrained by the cap beam and tie beam, the displacement of the free section of the pile group at these two positions is basically the same. Through comprehensive analysis, it can be concluded that the displacement of P1 and P2 piles is basically the same [22]. The maximum displacement of the pile body appears at the pile top. Compared with the peak acceleration of 0.1497 g, the horizontal displacement of the pile group top increases by 0.39, 1.16, 2.79, and 4.34 times, respectively, under other loading conditions, indicating that the horizontal displacement of the pile top increases nonlinearly under various loading conditions. In the sand layer, compared with the 0.1497 g seismic wave loading condition, the residual horizontal displacement of the pile body at the slope surface under other conditions increases by 1.39, 2.24, 4.28, and 5.89 times.
times, respectively. It shows that under the direct influence of sandy soil sliding under an earthquake, the change of residual displacement of pile body at the slope is greater than that of free section displacement. The influence of slope sliding layer on the residual displacement of pile body should be considered in design [23].

Figures 4 and 5 show the distribution of the final bending moment of the buried section of the pile body under different peak acceleration seismic wave loading conditions. It can be seen from the figure that the bending moment of P1 and P2 piles reaches the maximum value of the reverse bending moment in a small range on or below the slope surface. Then, with the decrease in the distance from the measuring point to the pile bottom, the positive bending moment begins to appear after reaching the zero point of the bending moment and then increases to the maximum value of the positive bending moment at the junction of bedrock and sandy soil layer. Finally, it decreases to zero at a certain depth below the bedrock surface, showing an asymmetric “s” shape distribution [24]. This indicates that the slope pile is the most unfavorable section at the interface between bedrock and soil layer, and pile P1 bears more bending moments than pile P2 at this position, which should be considered as the control section in the design.

The calculated value in Figures 2–5 can be obtained: The distribution of the calculated residual horizontal displacement of P1 and P2 piles is basically the same as the test value. There is little difference in the residual horizontal displacement of the buried section below the slope surface of each pile, but there is a great difference in the residual horizontal displacement of the free section. It is considered that the larger peak acceleration of input seismic waves leads to a larger error value of pile top displacement [25]. The calculated value of the final bending moment of P1 and P2 piles shows the same change trend as the test value, and the test value is slightly larger than the calculated value. The relative errors of the maximum final bending moment of P1 pile under each loading condition are 7.4, 7.8, 12.6, 3.9, and 9.6%, respectively, and the relative errors of P2 pile are 4.6, 3.6, 12.5, 13.6, and 11.5%, respectively. The relative error of analysis is because the elastic element is used in the calculation. Generally speaking, the test value and calculated value of pile dynamic response show the same change trend and law. Although there are some differences in value, the relative error between them is acceptable for using a numerical method to explore the deformation and mechanical characteristics of slope piles and the research on influencing factors [26].

4.2 Discussion

According to the existing centrifuge test results, it can be seen that the deformation at the slope shoulder of the slope site is the most obvious under the earthquake. In order to analyze its dynamic response changes in more detail, combined with the PDMY model used in the numerical model, the dynamic response of the slope site is compared and analyzed at the slope shoulder covered with sandy soil layer on the slope site and the slope top 7.0 m away from the slope shoulder [27]. It can be seen from Figures 6 and 7 that in the shallow soil near the
surface with a depth of 0.5 m (0–1.0 m), the shear stress shear strain hysteretic curve shows an obvious slip phenomenon. Compared with the weakened hysteretic circle at the slope top, there is basically no completed hysteretic circle at the slope shoulder, indicating that the shallow soil at the slope site, especially the soil at the slope, basically loses its seismic capacity under the action of earthquake. In the middle soil (1.0–5.0 m) near the depth of 4.5 m, the shear strain decreases greatly compared with the shallow soil, and the hysteretic curve at the top of the slope has an obvious symmetrical hysteretic circle, while the hysteretic curve at the slope shoulder still has a certain slip under the influence of the slope, and the symmetry of the hysteretic circle is not obvious [28]; In the deep soil near the depth of 10.5 m (>5.0 m), the shear strain at the top of the slope is further reduced and the hysteretic circle is fuller, indicating that it has good energy dissipation properties and is conducive to the seismic resistance of the structure. However, the shear strain at the slope shoulder at this depth does not decrease, but increases to a certain extent, indicating that the inclined interface of soft and soil layers will aggravate the dynamic response of the overlying soil layer on the slope and weaken its ability of seismic energy consumption [29–34].

The solution is to demarcate the possible ground rupture zone and not build in this zone. The difficulty encountered in dividing the ground rupture zone is how to determine the location of the seismogenic fault for future earthquakes. The location of these faults is in most cases completely unknown, or only very inaccurate judgments can be made. However, ground-breaking, although it can have serious consequences, is a very local phenomenon. In addition, ground fractures often occur in weak overburden layers, which are not as powerful as bedrock. Even though the structure can be damaged through foundation deformation and cracking, the degree is often relatively light. Because the investigation and research of the rupture surface of the seismogenic fault is quite arduous work and the cost is relatively large, the required cost and the actual effect that may be obtained should be comprehensively considered when dividing the dangerous section of the ground rupture. The geological structure and active fault data are analyzed and judged to make reasonable decisions.

The pile foundation displacement calculated by the elastic foundation beam method is compared with the pile foundation displacement calculated by the numerical analysis method. In the absence of field monitoring data, the two can corroborate each other.

5 Conclusion

In this article, the nonlinear numerical simulation of the dynamic response of pile site and pile foundation under earthquake is proposed. Through the inverse calculation of the similarity ratio of the centrifuge model test, the OPENSEES two-dimensional finite element model of the pile group in the slope section is established. The slope of the model is about 27°, which is divided into two layers of rock and soil: sand and bedrock. The sand is simulated by the PDMY model, and the parameters are taken based on the fitting formula of various sand parameters given in the OPENSEES manual. The maximum final bending
moment of the pile body appears at the junction of bedrock and soil layer. The shoulder of the slope site is more prone to shear deformation than the top of slope under earthquake, and the inclined interface of soft and hard rock and soil layer will weaken its ability of seismic energy consumption. Pile foundation is widely used. Compared with the weakened hysteretic circle at the slope top, there is basically no completed hysteretic circle at the slope shoulder, indicating that the shallow soil at the slope site, especially the soil at the slope, basically loses its seismic capacity under the action of an earthquake. Therefore, in order to effectively reduce the seismic damage of pile foundations caused by foundation liquefaction in the future, dynamic characteristics of pile foundation on the liquefied foundation are of great significance to the seismic design, disaster prevention, and reduction of pile group foundation on the liquefiable foundation.

Through the research and analysis of pile–soil interaction under heap load, some conclusions for reference are drawn. However, due to the lack of test and field measurement data, only qualitative analysis can be made. In addition, some simplifications were made when simulating working conditions, but in actual engineering, the interaction between piles and soil is more complicated and cannot reflect the actual situation of the engineering very accurately. Therefore, the pile–soil interaction of passive piles still needs further research and development. The main problems to be solved are as follows:

1) The properties of internal force, deformation, and pore water pressure of pile foundation and soil at different times under repeated stacking loads are not considered. There is a difference between the deformation and internal force of soil under repeated stacking loads and those under static loads. Under repeated stacking loads, the soil will have irrecoverable residual strains after each stacking load. The internal force and deformation of the pile body will change with the number of repeated stacking.

2) The rheology under long-term stacking load is not considered. The deformation of soil is affected by time, including creep, relaxation, flow, strain rate effect, and long-term strength effect of soil.

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