Deformation and Stress Analysis of Pile-Supported Immersed Tunnels under Seismic Loads

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Abstract: The stress and deformation of pile-supported immersed tunnels under seismic loads is a critical issue in tunnel design. This paper utilizes ABAQUS (version 2020) finite element software to analyze the seismic load response of the sand compaction pile-immersed tunnel–seawater pressure (SIS) system, which is verified by a physical model. The study shows that the suppression effect of the seawater on the vertical frequency of the tunnel increases with depth. When the replacement rate of the piled foundation reaches 50%, the deformation of the tunnel “H-shaped” structures increases, which also changes the vertical frequency of the tunnel. However, the presence of the suppression effect causes resonance injury at the far end of the tunnel from the earthquake source, resulting in a shift of the peak stress point. It was also found that seawater pressure affects the resistance–deflection (p-y) at the tip of the pile more than at the end of the pile. The slenderness ratio (γ) of the pile affects the p-y value at the end of the pile more than at the tip of the pile. The connection between the piled foundation and the tunnel is most stable when γ is in the range of 9.25 to 15.

Keywords: sand compaction pile; immersed tunnel; frequency; slenderness ratio; replacement rate; stress and deformation

1. Introduction

Immersed tunneling is an engineering technology used to construct tunnels underwater. There have been numerous cases of the application of immersed tunnels for water resources development and energy transportation [1–3]. Horizontal and vertical loads acting on the tunnel structure by seismic waves cause deformation, stress concentration, and possible injury to the tunnel structure. It is important to analyze the fundamental frequency of the tunnel structure and the interaction with the frequency of seismic waves when analyzing the structural response of immersed tunnels [4,5], the nature of the undersea foundation geological layer, and the seismic wave load characteristics. Therefore, great attention should be paid to the dynamic response to earthquakes under tunnel foundation–seawater interaction [6,7].

When seismic waves are transmitted to the seabed and seawater, the effect of the presence of seawater on the propagation of seismic waves and energy absorption leads to a change in the vertical frequency of the structure, which attenuates the amplitude of seismic waves during propagation [8,9]. The ability of the seabed to enhance the seismic defenses of tunnels has been demonstrated when seismic SV waves are incident at different angles on the multilayered seabed [10], and the single-layered seabed [11]. The ambient temperature of seawater affects the tunnel stresses during earthquakes [12]. This shows the extent of the seismic response of immersed tunnel-piled foundations is related to seawater. The effect of seawater changing the vertical frequency of the tunnel on the stress and deformation needs to be further investigated.
Sand compaction pile (SCP) can effectively improve the rigidity and densification of the seabed and increase the bearing capacity and stability, which is well evidenced in existing studies [13–18]. Kiani et al. [19], Hossain et al. [20], and Heo et al. [21] showed the relationship between the SCP arrangement and the deformation of the foundation. Moreover, the piles can significantly improve the lateral bearing capacity of the foundation and effectively increase the foundation densification and the effective stress of the soil around the piles [22–25]. A large number of analysis methods show that the pile–soil interaction under dynamic loads is similar to that of Euler–Bernoulli beams and Pasternak foundations [26–31]. However, the relationship between the injury at the piled foundation–tunnel connection and the replacement rate and slenderness ratio of the piled foundation in seismic analysis is not clear.

In this paper, seismic effects are simulated by time history analysis within a sand compaction pile-immersed tunnel–seawater pressure (SIS) system. Numerical simulations (ABAQUS version 2020) were conducted to investigate pile-soil interaction, the connection between foundation piles and the tunnel, as well as the stress and deformation of the tunnel wall. Furthermore, the numerical simulations are validated by the established physical model experiment of the SIS system. The key influencing factors of seawater pressure, direction of the tunnel, replacement rate of the piled foundation, and slenderness ratio of the pile are parametrically analyzed. The optimum replacement rate and range of the slenderness ratio of the pile are determined. The positive and negative effects of seawater pressure on the seismic performance of the tunnel are evaluated. A more detailed basis for the seismic design of pile-supported immersed tube tunnels is provided.

2. Methodology
2.1. Physical Modelling Overview

The size of the physical model was 0.3 × 0.3 × 0.5 m to restore the change of pore pressure and deformation characteristics of the seabed. The physical experimental soil samples were soft clay mixtures with polystyrene (EPS) particles with a particle size of 0.3 mm under optimal water content conditions [32]. Based on the volume ratio, three types of proportioning were carried out: 1/9, 1/8, and 1/7. Figure 1a gradation curves show the results of sieve experiments of dry soil and EPS mixtures at three volume ratios, with the most uniform gradation curve at the 1/8 volume ratio. Based on Figure 1b,c, the 1/8 volume ratio mixture did not show multiple injury paths or penetrating injury paths and had the largest deformation modulus. The volume ratio of EPS and soft soil was determined to be 1/8.

![Figure 1](image-url)
2.1.1. Materials and Similarity Principle

To determine the sensitivity of different replacement rates and sand pile arrangement to the seismic effects on the seabed. The pipe diameters of 48 mm, 32 mm, and 20 mm casing were used to be divided into six layers of compaction. The dimensions of the experimental equipment and sensors are shown in Figure 2a. The model experiment scaling is shown in Table 1, and all parameters satisfy the similarity principle except time and frequency. The maximum pile spacing of the three arrangements of SCP are 0.1 m, 0.077 m, and 0.1 m, and the depth of burial is 0.3 m. The outer edge of the outermost pile is 0.1 m from the boundary of the foundation. The angles of the pile arrangements are 90°, 45°, and 60°, respectively, which are shown in Figure 2b.

Figure 1. Physical properties of EPS and clay with different volume ratios: (a) Grading curves, (b) Specimen injury paths, (c) Deformation modulus.

Figure 2. Cont.
Figure 2. (a) Experimental equipment and SCP installation process, (b) Model size (\(\theta\) is the angle of the pile arrangement).

Table 1. Dimensions of piled foundations in model experiments.

<table>
<thead>
<tr>
<th>Modeling Type</th>
<th>Component</th>
<th>Properties</th>
<th>Properties</th>
<th>M1</th>
<th>M2</th>
<th>M3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SCP</td>
<td>Outer diameter [m]</td>
<td>2.4</td>
<td>1.6</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Number × Length [m]</td>
<td>14 × 15</td>
<td>23 × 15</td>
<td>25 × 15</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foundation</td>
<td>Surface area [m²]</td>
<td>52.91 × 3 S (Pile spacing)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Deformation modulus (E_s) [kPa]</td>
<td>(E_s = E_{s}^*)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Physical model</td>
<td>SCP</td>
<td>Outer diameter [mm]</td>
<td>48</td>
<td>32</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Number × Length [m]</td>
<td>4 × 0.5</td>
<td>5 × 0.5</td>
<td>7 × 0.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foundation</td>
<td>Surface area [m²]</td>
<td>0.09</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Deformation modulus (E_{s}^*) [kPa]</td>
<td>(E_{s}^* = E_s)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: \(M_1\), \(M_2\), and \(M_3\) represent the three SCP arrangements.

2.1.2. Overburden and Seismic Waves

The extent of seismic influence on the SIS system is greater when SV waves are incident vertically in the supercritical angle case. The maximum vertical acceleration of the seismic wave increases at this time. The motion of the foundation surface is essentially the same regardless of the presence or absence of the overlying fluid. The influence of sea waves on the seismic response results is very small [11]. Therefore, the effect of seawater on seismic waves is not considered in this study, and seawater is equivalent to the overlying pressure.
It is assumed that the tunnel is in a straight line when it is subjected to upward movement by buoyancy, and the frictional $f_s$ resisting the uplift of the overburden and the tunnel is provided by the lateral earth pressure. The safety coefficient of the immersed tunnel $\omega = (W + f + G)/F_{\text{float}} \geq 1.1$ meets the design requirements, then the minimum thickness of the backfill layer is $h = f \left( F_{\text{float}}, G, W, f_s \right)$, where $W$ is the gravity in the overburden layer, $G$ is the gravity of the immersed tunnel, and $F_{\text{float}}$ is the buoyancy of the structure.

The physical model seismic waves were selected from PEER Ground Motion Database-NGA west2, and data from the Keslin Seismic Observatory in Greece were analyzed in the time history. The seismic intensity of 7.5, characteristic period of 0.40 s, damping ratio of 0.05, and peak acceleration of 0.05 g are shown in Figure 3.

![Time history curve of the seismic wave.](image)

**Figure 3.** Time history curve of the seismic wave.

### 2.2. Numerical Model

Figure 4 shows the schematic diagram of the numerical model. The model consists of subsea, immersed tunnel, SCP pile group, gravel cushion and backfill. The interaction between the structures in the SIS system due to the inertial effect is applied through foundation vibration and structural mass distribution [33,34]. The numerical modeling analysis process is divided into four phases. The first is consolidation by drainage for 60 days after installation of the SCP in the seabed. The immersed tunnel is again consolidated by drainage for 60 days after construction. Then seawater pressure is applied, at which time the seabed is in an undrained consolidation state. Finally, seismic loads and viscoelastic boundaries are imported into the numerical model for seismic analysis using Matlab. When the sand compaction pile (pile penetrating through multiple soil layers) interacts with the foundation, frictional and lateral earth pressure are set for the pile–soil contact surface. Different materials are set for different soil layers to analyze the interaction between different soil layers and piles. Due to the large thickness of seabed sedimentary soil, engineering generally uses floating sand compaction piles to improve the first layer of seabed sedimentary soil. The burial depth of the pile does not reach the holding soil layer, and the settlement is mainly concentrated in the soft soil layer. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero.
Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero. Therefore, the displacement of other soil layers below the soft soil layer is assumed to be zero.

Finite element analysis can only simulate finite space, and the seabed is surrounded by half-infinite space bodies Kou and Yang [33]. Seismic waves lack reflection and damping effects at the boundaries, which are inconsistent with the actual wave field in the half-space [39]. Therefore, viscoelastic boundaries are set up in addition to the constrained displacements for realizing the reflection and damping of seismic waves. The restraining effect of neighboring soil materials is simulated. This is described in detail in Section 2.2.3.

2.2.2. Material Properties

Parameters \( M, e_0, e_1, \lambda, K_r \), and \( \nu \) are common to the Cam–Clay model (MCC) for marine soil. In the MCC model, the slope of the critical line (CSL) in the \( p-q \) coordinate system is \( M = 6 \sin \varphi / (3 - \sin \varphi) \); the slope of the normal consolidation line (NCL) in the \( e-lnp \) coordinate system is \( \lambda = C_c / 2.3 \) and the slope of the swelling line in the \( e-lnp \) coordinate system \( K_r = (0.2-0.5) \lambda \); and the relationship between the soil void ratio and the consolidation ratio (OCR) at a given location is \( e_1 = e_0 + \lambda h p_0 + K_r ln(OCR) \) [40].
During consolidation, the void ratio of the seabed decreases as pore water is drained, leading to subsidence of the seabed and the marine structures above it. The hydrostatic pressure of the seabed foundation will change significantly under large deformations. Accordingly, in finite element calculations, the Darcy permeability coefficient \( k \) in the soil is related to the change in the void ratio \( e \) during soil deformation. In the calculation, the permeability of the seabed soil is \( k_i = C_f e^3 / (1 + e) \), where \( C_f = k_0 (1 + e_0) / e_0^3 \) is the empirical coefficient \[41\], and \( e_0 \) and \( k_0 \) are the initial void ratio and permeability, respectively.

SCP produces hardening behavior under the action of seismic waves, and the DP hardening model is coupled with a porous elastic material (Proelastic) to form a sand pile drainage channel used to reduce the exceeded porous water pressure in the subsea foundations. The immersed tunnel, gravel cushion, and backfill are practically designed with impermeable and rigid materials. Therefore, they are modeled with linear elastic (elastic) materials with the parameters given in Table 2.

### Table 2. Material constants for modeling.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Marine Clay</th>
<th>SCP</th>
<th>Immersed</th>
<th>Gravel Cushion</th>
<th>Backfill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Consolidation index, ( \lambda )</td>
<td>0.113</td>
<td>0.023</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Swelling index, ( Kr )</td>
<td>0.05058</td>
<td>0.0103</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Critical state parameter, ( M )</td>
<td>0.668</td>
<td>1.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio, ( \nu )</td>
<td>0.36</td>
<td>0.3</td>
<td>0.17</td>
<td>0.45</td>
<td>0.37</td>
</tr>
<tr>
<td>Density, ( \rho ) (g/m³)</td>
<td>1.851</td>
<td>1.765</td>
<td>2.45</td>
<td>0.9</td>
<td>2</td>
</tr>
<tr>
<td>Buoyant density, ( \rho_w ) (g/m³)</td>
<td>0.801</td>
<td>0.715</td>
<td>1.4</td>
<td>0.85</td>
<td>0.95</td>
</tr>
<tr>
<td>Cohesion, ( C ) (KPa)</td>
<td>7.4</td>
<td>0.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction angle ( \varphi ) (°)</td>
<td>15</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Young’s modulus, ( E ) (KPa)</td>
<td>2821.83</td>
<td>( E^* )</td>
<td>30,000</td>
<td>20,000</td>
<td>150,000</td>
</tr>
<tr>
<td>Permeability, ( k ) (m/s)</td>
<td>( 1.8 \times 10^{-6} )</td>
<td>( 3 \times 10^{-5} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coefficient of lateral pressure</td>
<td>0.577</td>
<td>0.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial void ratio, ( e_0 )</td>
<td>1.059</td>
<td>0.72</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Degree of saturation</td>
<td>1</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: \( E^* = 0.6 \sigma_m (\sigma_m / \sigma_a)^{0.5} \), where \( \sigma_m \) is the stress of atmospheric and \( \sigma_a \) is the current average principal stress in the soil unit under consideration \[42\].

#### 2.2.3. Viscoelastic Boundary and Seismic Loads

As shown in Figure 5, the viscoelastic boundary is actually a continuous distribution of springs and dampers around the perimeter and bottom of the model, except for the free top surface. The spring stiffness and damping coefficients of the nodes are the product of the linear stiffness and damping and the control area of the nodes. The mechanical parameters of the boundary springs and dampers are determined by the mechanical properties of the surrounding rock. The stiffness \( K \) and damping coefficient \( C \) of the spring are shown in Equations (1) and (2).

\[
K_{bT} = A_{bT} \frac{G_s}{R} \quad C_{bT} = A_{bT} \rho C_S
\]

\[
K_{bN} = A_{bN} \frac{G_s}{R} \quad C_{bT} = A_{bT} \rho C_S
\]

where \( G_s \) and \( \rho \) are the shear modulus and density of the medium, respectively; \( R \) is the distance from the wave source to the artificial boundary (straight distance from the boundary point of the finite element model to the source of the earthquake); \( A_b \) is the control area of the node \( b \) on the boundary; \( K_{bN} \) and \( C_{bT} \) are the correction coefficients for the normal and tangential directions, respectively; and \( C_S \) and \( C_P \) are the shear and longitudinal wave velocities of the medium and the Lamé constants \( \beta \) and \( \delta \), respectively \[40\].
Figure 5. Viscoelastic boundary and seismic wave reflection.

Neural networks and artificial intelligence effectively analyze the sensitivity of various factors in complex engineering [43–48]. The effect of seismic loads on piled foundations is assumed as the dynamic load transfer process in Euler–Bernoulli beams with Winkler foundations in Kiani et al. [49,50] and Selim and Liu [51]. In this study, the seismic wave input is realized by transforming the earthquake into a wave source problem and then acting on the boundary as a concentrated force. There is an equilibrium relationship between the equivalent force $F_b$, the normal stress $\sigma_n$ and the force $f(t)$ generated by the spring damper on any boundary point $b(x_b, y_b, z_b)$ of the model He et al. [52]. The conversion of seismic waves into equivalent seismic loads within the control area (Equation (3)).

$$F_b = \left( K_b v_b^n + C_b v_b^n + \sigma_b^n n \right) A_b$$  \hspace{1cm} (3)

where $K_b u_b^n$ is the additional stress generated by the spring unit due to the overcoming displacement; $C_b v_b^n$ is the additional stress generated by the damper due to the overcoming velocity; $\sigma_b^n n$ is the stress tensor generated by the free field vibration in the direction normal to the boundary; $u_b^n$ is the displacement vector of the incident wave; $v_b^n$ is the velocity vector of the incident wave; and $A_b$ is the area of the boundary node control.

3. Verification of Numerical Model

3.1. Convergence Analysis of Numerical Model

Equations (4) and (5) for a single pulse wave at a time step of 0.001 s, and a cutoff frequency of 50 Hz. Density of 2000 kg/m$^3$, Poisson’s ratio of 0.22, an elasticity modulus of $4.88 \times 10^9$ GPa and an amplitude of 1 m for the displacement of the incident SV wave are used.

$$\begin{align*}
  u^{ff}(t) &= \frac{1}{2} \left[ 1 - \cos(8\pi t) \right], \quad (0.75 \leq t \leq 1) \\
  u^{ff}(t) &= 0, \quad (0 \leq t < 0.75, 1 < t)
\end{align*}$$  \hspace{1cm} (4)

$$\begin{align*}
  v^{ff}(t) &= 12.5 \sin(8\pi t), \quad (0.75 \leq t \leq 1) \\
  v^{ff}(t) &= 0, \quad (0 \leq t < 0.75, 1 < t)
\end{align*}$$  \hspace{1cm} (5)

Figure 6 shows the normal velocity and displacement time history curves for the numerically modeled surface. It can be seen that the analytical and numerical solutions are consistent. The seismic wave transmitting to the top surface of the model will be reflected, and the velocity and displacement will increase because there is no surrounding constraint limitation on the top surface. The bottom surface produces two wave peaks due to the reflected wave, and the time gap is exactly one reflection period. Due to the presence of a viscoelastic boundary at the bottom surface, the reflected wave is absorbed, as is the case with the transfer of waves from the side of the model. Therefore, the viscoelastic boundary has better accuracy and can effectively represent infinite foundations.
Figure 6. Normal velocity and normal displacement time history curves on the surface of the numerical model: (a) bottom surface, (b) side surface.

3.2. Verification of the Results

The applicability of the numerical model was examined in terms of both settlement and acceleration. Figure 7a–c show the settlement curves of the piled foundation. It can be seen that the settlement produces a fluctuation process similar to the seismic frequency, but the settlement eventually stabilizes. The settlement fluctuation is due to the fact that the amplitude and propagation direction of the seismic wave in the numerical model will produce changes at different locations. This variation leads to an uneven response of the foundation, which results in fluctuations in the settlement curve. Whereas the whole physical model vibrates at the same acceleration, the foundation gradually densifies under this nondifferentiated vibration, which leads to a smooth settlement curve. The settlement results of the experiment can better describe the final value and trend of the numerical results. It can also be seen that under the same conditions, the settlement of the diagonal piled foundation (37% replacement rate) is 0.81 times that of the rectangular piled foundation (50% replacement rate). However, the settlement of the plum-piled foundation (62% replacement rate), which has a higher replacement rate, is rather smaller than that of...
the rectangular-piled foundation. This indicates that the piled foundation settlement is the most stabilized when the replacement rate reaches about 50%.

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Figure 7. (a–c) settlement of the piled foundation, (d) acceleration amplitude of the tunnel.

Figure 7d shows the magnitude acceleration of the tunnel in the numerical model and the acceleration reaction spectra in Ground Motion Database - NGA west2. It can be seen that the acceleration response spectra used in the physical model describe the numerical results trend better. It is worth noting that the numerical results show hysteresis. This is due to the fact that the transfer of seismic loads in the numerical model is assumed to be a wave source problem subjected to damping that is uniform in action. The actual foundation damping is inhomogeneous, and hence, deviation and reduction in the monitoring results can occur.

4. Parametric Analysis

4.1. Effects of the Seawater Depth

4.1.1. Peak Stresses in the Top and Bottom Plates of the Tunnel

The stress changes after an earthquake are very important for immersed tunnel design. Figure 8 shows the maximum principal stress distribution of the tunnel bottom plate (a) and top plate (b) under seismic loads. Under seawater pressure and seismic loads, the stress curves of the top plate and bottom plate are saddle-shaped. From Figure 8, it can be seen that stress concentration occurs on both sides of the top and bottom plates of the tunnel. With the increase in seawater depth, the maximum principal stresses of the tunnel bottom and top plates show an increasing trend of decreasing growth rate. This indicates that the seawater depth has a suppression effect on the vertical frequency of the tunnel [53].
With the increase in the replacement rate of the piled foundation, the stress peaks of the top and bottom plates are shifted toward the wave source direction, and the shift angle in the middle is about twice the shift angle of the left and right sides ($\theta_1 = 2\theta_2$), as shown in Figure 8a.

The maximum principal stresses in the top plate of the tunnel are always greater than those in the bottom plate. It can be seen that the effect of seismic loads on the bottom plate is greater than on the top plate. The difference between the two maximum principal stresses is due to the damping effect of the “H-shaped” structure of the tunnel. In addition, the maximum principal stresses in the top and bottom plates at a 50% replacement rate are smaller than the maximum principal stresses at 37% and 62% replacement rates.

### 4.1.2. Displacement of the “H-Shaped” Structures

Figure 9 shows that the horizontal displacements of the left vertical support and the right vertical support in the “H-shaped” structures of the tunnel are in the shape of “X”.

**Figure 8.** Maximum principal stresses of the tunnel: (a) bottom plate, (b) top plate.
The depth of seawater increases from $H = 23$ m to $H = 44.78$ m, and the reduction rate of left and right horizontal displacements is 13.63% and 36.84%, respectively; this is due to the fact that the seawater absorbs part of the seismic wave energy, and the suppression effect of seawater on the vertical frequency of the tunnel increases with the increase in the depth of the water, which can effectively increase the seismic performance of the structure [54,55].

![Figure 9. Horizontal displacement of the “H-shaped” structure.](image)

However, when the overall horizontal displacement of the “H-shaped” structures decreases, the difference in horizontal displacement between the left and right vertical supports tends to increase. The rate of increase in horizontal displacement in the lower part of the “H-shaped” structures is higher than that in the upper part (Figure 9 neutral axis rotated 7° counterclockwise). Seismic loading causes the tunnel to vibrate. This vibration amplifies the response of the tunnel to the loads and may result in injury to the weak areas of the structural design. The tunnel will dissipate the seismic energy through the deformation of the “H-shaped” structures and stress transfer in the walls to dampen itself and reduce the vibration amplitude. Secondly, this deformation and stress transfer will also change the fundamental frequency of the tunnel [56], preventing resonance injury and increasing seismic performance. This is consistent with the stress transfer mechanism of the top and bottom plates of immersed tunnels in Section 4.1.1.

### 4.2. Structural Response in Different Directions

#### 4.2.1. Settlement of Gravel Cushion

Figure 10a shows the settlement deformation at the bottom of the gravel cushion under the three SCP arrangements of the piled foundation. It can be seen that the settlement of the gravel cushion differs greatly at different seawater depths. The maximum settlement change rates of rectangular, diagonal, and plum blossom piles are $\eta_1 = 4.4$, $\eta_2 = 2.5$, and $\eta_3 = 14.5$, respectively. Seawater suppressed the tunnel significantly more at the end close to the seismic source than at the far end, and the maximum location of gravel cushion settlement is located at the far end. However, due to the damping effect of the foundation and the immersed tunnel, the far end of the cushion from the seismic source is subjected to smaller seismic loads relative to the close end, and the settlement difference is instead magnified. This is due to the seawater suppression that causes the vertical frequency of the overburden structure and the seismic frequency to resonate at the far end, resulting in additional seismic injury. The seismic time history response is sensitive to changes in the
fundamental frequency of the overburden structure. It is necessary to increase the restraint force of the pile foundation on the far end of the structure to offset the resonance injury.

![Settlement Diagram](image_url)

**Figure 10.** Settlement of the gravel cushion: (a) along the width direction, (b) along the length direction.

Figure 10b shows the settlement of the gravel cushion along the length direction. It can be seen that the settlement at the bottom of the cushion is symmetrically distributed. This is due to the fact that the immersed tunnel is similar to an infinitely long cavity in the longitudinal direction, and the neighboring structure produces a strong restraining effect. The neighboring constraint force can offset the longitudinal destructive force of the tunnel. Therefore, there is no gravel cushion in the settlement amplification phenomenon.

### 4.2.2. Vertical Strain of the Gravel Cushion

The strain in the gravel cushion after the earthquake can effectively reflect the superiority of the piled foundation design scheme. Figure 11 shows the vertical strain along the bottom of the gravel cushion along the length direction. When the seawater depth changes, the vertical strain curve is wavy at a 37% replacement rate, and the bottom of the gravel...
The tip of the pile settlement produces downward tensile stress, and the soil between the pile settlement is less than the tip of the pile settlement due to the friction of the piles and the soil arch effect. Therefore, a wave-like strain curve appears. The vertical strain of the gravel cushion tends to be consistent at a 50% replacement rate, and the strain is small. When the replacement rate reaches 62%, the bottom of the cushion is pierced, at which time obvious tensile strains appear at both ends of the gravel cushion.

The above data indicate that the seawater pressure has an effect on the lateral behavior of piled foundations. The effect of seawater pressure on the lateral behavior of the pile is through increasing the degree of soil densification and increasing the axial force to generate additional bending moment, as in Lu and Zhang [60]. It can be seen that the change in seawater depth affects the $p-y$ relationship of the pile. As the seawater depth $H$ increases, the characteristic lines of SCP under all three $\gamma$ are rotated clockwise, which coincides with the results of Lu et al. [57], who found that the $p-y$ curves of piles under compression conditions are steeper. At this time, the deflections at the tip of the pile varied more by 0.23 m, 0.23, and 0.2 m, respectively, while the difference in deflection variation at the end of the pile was not significant. The stiffness represented by the slope of the curve increases with depth, and similar results were obtained by Klinkvort and Hededal [58] and Zhu et al. [59] from centrifuge modeling tests of monopiles and jacketed piles. At $\gamma = 15$, the characteristic line is rotated clockwise by 45°, and the resistances at the tip of the pile are increased by 95 KPa, and that at the end of the pile is increased by 43 KPa. At $\gamma = 9.37$, the angle of rotation of the characteristic line is 11°, the resistances at the tip of the pile increase by 70 KPa, and the resistances at the end of the pile increase by 70 KPa. At $\gamma = 6.25$, the angle of rotation of the characteristic line is 10°, the resistances at the tip of the pile increase by 105 KPa, and the resistances at the end of the pile increase by 140 KPa.

The above data indicate that the seawater pressure has an effect on the lateral behavior of the piled foundations. The effect of seawater pressure on the lateral behavior of the pile foundation is through increasing the degree of soil densification and increasing the axial force to generate additional bending moment, as in Lu and Zhang [60]. It can be seen that the change in seawater depth affects the $p-y$ value at the tip of the pile more than the end of the pile, while the change of $\gamma$ affects the $p-y$ value at the end of the pile more than the tip.
of the pile. The combined consideration of seawater pressure and the slenderness ratio of the pile can prevent localized injury to the pile.

Figure 12. Cont.
The injury to the SIS system by seawater depth is significantly less than the direct effect of seismic transverse waves, indicating that the damping of seismic waves by the structure and the suppression frequency of seawater are favorable for seismic design. It is clearly observed that the equilibrium that occurs in the pile roof and pile end resistances when $\gamma$ from 15 to 9.35. It confirms the existence of an optimum slenderness ratio for the SCP-reinforced marine foundation in Section 3.

4.3.2. Soil Displacement around the Pile

The values of the slenderness ratio ($\gamma$) studied in this section are 15, 9.35, and 6.25, and the values of seawater depth variation are $H = 44.78$ m, $H = 32.12$ m, and $H = 23$ m, while other parameters are kept constant. Figure 13 shows the horizontal displacement of the foundation on the left and right sides of the center pile for the nine conditions of $\gamma$ and $H$. The immersed tunnel has a high density and large mass. Fixed contact is used between the sand compaction pile, gravel cushion, and tunnel in the numerical model, thus forming a top-heavy and bottom-light structure with SCP. Seismic injury mainly produces acceleration differences in the structure, which is then subjected to shear. The mass distribution of top-heavy and bottom-light will enlarge this acceleration difference, which makes the shear near the connection between the SCP and the tunnel larger, and thus, a larger displacement occurs at a depth of 2 m. The pile embedment depth below 2 m depth is large, and the soil densification degree is high enough to resist the lateral load, so the displacement below 2 m gradually decreases.
Figure 13. Cont.
In the case of the same seawater depth, when the embedment depth $L$ decreases from $15D$ to $6.25D$, the horizontal displacement difference between the two sides of the pile increases. The area of the difference extends from 7 m below the top of the pile to the end of the pile. It can be seen that when using SCP to strengthen the lateral deformation resistance of the foundation, it should be avoided that the embedding depth is too deep, leading to the destruction of the tip of the pile, and the embedding is too shallow, leading to the instability of the end of the pile.

In the case of the same embedding depth $L$, when the seawater depth $H$ increases from 23.12 m to 44.78 m, the horizontal displacements of the foundations on the left and right sides of the pile are shifted to the right, forming a transformation process from curved parabola to flat parabola. The piled foundation produces a horizontal displacement away from the direction of the earthquake source, which is related to the resonance of the structure away from the source end. At this time, the horizontal displacement of the foundation on both sides of the pile shifted to the right as a whole without any significant change in the difference. It is worth noting that the maximum depth of foundation horizontal displacement that exists below the end of the pile is within 1 m.

5. Conclusions

In this paper, the stresses and deformations of pile-supported immersed tunnels under seismic loads are investigated by using a numerical model developed in ABAQUS. The numerical model results are verified by the physical model experiment of the SIS system. The mechanical behaviors of the immersed tunnel and pile–soil interaction under different loads and SCP arrangement were parametrically analyzed. The conclusions are as follows:

1. Increased seawater pressure suppresses the vertical frequency of the tunnel. The suppression is enhanced by the deformation of the tunnel’s “H-shaped” structures and stress transfer in the tunnel wall when the pile replacement rate reaches 50%.

Figure 13. Horizontal displacements of foundations at 0.8 m, 1.6 m, and 2.4 m to the left and right of the center pile: (a) plum blossom piles, (b) diagonal piles, (c) rectangular piles.
(2) The vertical frequency at the far end of the immersed tunnel from the source is similar to the seismic excitation frequency along the width of the tunnel, resulting in resonance. The seismic injury at the distal end is amplified.

(3) The soil densification is higher at the end of the pile when the seawater pressure increases and the lateral earth pressure increases, resulting in a large change in the slope of the \( p-y \) curve at the tip of the pile. The increase in pile slenderness ratio (\( \gamma \)) will increase the axial force of the pile to produce an additional bending moment, resulting in large changes in the slope of the \( p-y \) curve at the end of the pile with larger axial force. When the slenderness ratio is in the range of 9.25 to 15, it can reduce the injury at the connection between piled foundations and tunnels.

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