Research Article

Shaking-Table Tests for Immersed Tunnels at Different Sites

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Immersed tunnels are typically built in areas subjected to ground motion. Therefore, an evaluation of the seismic performance of the soil-tunnel system is essential. A series of shaking-table tests was conducted to study the influences of the site soil and overlying water layer on the seismic responses of soil deposits and an immersed tunnel. Detailed information on the experiment setup is provided with special focus on the similitude relationship, fabrication of the model system, measurement setup, and loading procedures for a simulation of the seismic waves. Three groups of tests at different sites in dry sand, saturated sand, and saturated sand with an overlying water layer were carried out using the same seismic excitations. The seismic responses of the soil deposits and the dynamic responses of the tunnel model were obtained. The experiment results indicate that, when considering only horizontal earthquake excitations, soil liquefaction significantly influences the propagation of seismic waves and the dynamic responses of the tunnel, whereas the water layer has no obvious effects on the dynamic performance of the ground or tunnel. Furthermore, the acceleration responses of the tunnel elements were analyzed qualitatively, and the joints are deemed important elements in an antiseismic immersed tunnel design.

1. Introduction

Immersed tunnels typically serve as a vital infrastructure. They are mainly built at soft sites subject to seismic hazards. Since the development of the Detroit River tube tunnel (the first real immersed tunnel) in 1910, immersed tunnels have been developed in quick succession all over the world owing to their numerous advantages [1–3]. During the earthquake-resistance design process of an immersed tunnel, an evaluation of the seismic effects in terms of the structural acceleration, strain, and earthquake wave propagation in soft soil is essential to reveal the seismic performance of an immersed soil-tunnel system.

A number of papers on the design of immersed earthquake-resistant tunnels have been published. Ingerslev and Kiyomiya [4] provided guidelines on how to select and apply seismic loading to an immersed tunnel. Based on the elastic foundation beam method, Kiyomiya [5] proposed seismic design methods for use in Japan, as well as flexible joints for adoption in immersed tunnels. Anastasopoulos et al. [6] created a beam-spring system to analyze the nonlinear

dynamic response of a deep-water immersed tunnel located in an offshore location that often experiences large earthquakes. A discussion on the deformation mechanism of tunnel joints under longitudinal and lateral vibrations was presented. The author also suggested a suitable joint design, and the use of a relatively short segment to help avoid excessive joint tension and compression between two tunnel segments. Ding et al. [7] presented the dynamic behavior of an immersed tunnel located in Shanghai based on a three-dimensional simulation. Anastasopoulos et al. [8] investigated the seismic responses of a tunnel deeply immersed in a potential fault, which could experience seismic activity at the base rock beneath the tunnel and found that tunnel joint plays a significant role during the combination of a fault rupture and the subsequent strong ground motions. Peng et al. [9] conducted a contrastive analysis to study the influence of dynamic water pressure on an immersed tunnel under seismic excitation and found that the effects of the water differ based on the direction of the earthquake.

Along with theoretical and numerical studies, laboratory tests have also been conducted to study the seismic

performance of an immersed tunnel. Through a shaking-table test on a subaqueous tunnel, Okamoto and Tamura [10] created a soil-tunnel mathematical model which was composed of a concentrated mass system, a subaqueous tunnel, and springs. The authors also pointed out that the dynamic behaviors of the tunnel are caused by nonuniform deformations of the ground. Through centrifuge model testing on the George Massey tunnel, Yang et al. [11] demonstrated that ground motions could trigger soil liquefaction, causing upward and lateral movements of the tunnel. To validate the ground retrofit method in an immersed tunnel seismic design, the author also designed a model with a densification ground, and a second model with a gravel drainage foundation. Chen [12] studied the seismic mechanical characteristics of a semirigid joint based on a shaking-table test. However, most experiments described in the literature have focused mainly on immersed tunnels in either a dry soil site or a saturated soil site without a water layer on the surface. In reality, transient earthquake waves propagate differently between dry and saturated soil. Most immersed tunnels are built under rivers and ocean waters, and thus research on the influence of a water layer on the seismic response of an immersed tunnel needs to be carried out. However, a consideration of the soil properties and overlying water layer on the seismic responses of an immersed tunnel using a shaking-table test has yet to be reported.

Based on the Hong Kong-Zhuhai-Macau (HZM) linkage project, numerous shaking-table tests on a tunnel with three types of ground conditions were conducted. To investigate the effects of diverse sites on the dynamic responses of the soil and tunnel, the test conditions were divided into three groups: dry sand deposits (test 1), fully saturated sand deposits (test 2), and fully saturated sand deposits with a 200 mm deep overlying water layer (test 3). This is the first shaking-table test for an immersed tunnel with a comprehensive consideration of the different types of sites used, and the notable test details are introduced herein. In particular, the experiment results merit further study of immersed tunnels, as well as a comparative analysis of the three test conditions applied.

2. Research Background of HZM Linkage

HZM linkage [13], which serves as an efficient mode of coastal transport, has recently attracted the attention of the world. This megaproject will promote the development of tourism and economic trade among Hong Kong, Zhuhai, and Macau. As shown in Figure 1, the HZM linkage is composed of two large-scaled cable stayed bridges and an undersea tunnel (which is the core engineering aspect of the entire linkage project) linking two artificial islands. Thus far, tunnel engineers have completed the connection of 90% of the tubes (the HZM tunnel contains 33 tubes and over 200 joints). A normal tube usually contains eight segments prefabricated in order and assembled in a dockyard before immersion at the site. Each tunnel segment is 22.5 m in the longitudinal direction, 37.95 m in width, and 11.4 m in height, as shown in Figure 2.

FIGURE 1: Hong Kong-Zhuhai-Macau linkage.

FIGURE 2: Fabrication of HZM elements.

3. Experiment Setup

The shaking-table tests introduced in this paper were designed differently from previous experiments in the following ways: a relatively large model structure with a reduced scale of 1:30 is used, a fabricated rubber joint that could be easily implemented in the test was designed, both a dry sand site and liquefied sand ground were taken into account, and an overlying water level above the ground was considered.

All tests were carried out in the Key Laboratory of Earthquake Engineering and Engineering Vibration, China Earthquake Administration, Harbin, China. The shaking-table system was built in 1986, and its longitudinal and transverse dimensions are 5 m × 5 m. The maximum horizontal acceleration and vertical acceleration of the shaking table are 1.0 and 0.7 g, respectively, with a frequency of 0.5 to 40 Hz. The tests were conducted in a multifunctional laminar shear container [14] designed to avoid the boundary effects. The container is 3.5 m in length, 2.4 m in width, and 1.7 m in height, as shown in Figure 3. Using a shaking-table test and finite element analysis, Sun et al. (container inventor) [15, 16] demonstrated that the stiffness of the container is adjustable to eliminate the boundary effects.

3.1. Similitude Relation. During the design process of shaking-table tests, the issue of the similitude relation between the experimental model and the prototype is usually encountered first. Based on the Buckingham- π theory [17, 18], and a combined consideration of the working conditions of the shaking table and the size of the multifunctional soil container, the similitude relation assumed can be expressed as follows:

$$\sigma = f(L, E, \rho, d, a, t, \omega, u, V_s), \quad (1)$$

TABLE 1: Similitude relations and ratios of the model system.

Item	Symbol	Similitude relation	Tunnel	Soil
Geometry	L	L_r	1/30	1/4
Elasticity modulus	E	E_r	1/4	1/4
Mass density	ρ	ρ_r	5	1
Displacement	d	$d_r = L_r$	1/30	1/4
Acceleration	a	$a_r = E_r \rho_r^{-1} L_r^{-1}$	1.5	1
Time	t	$t_r = E_r^{-0.5} \rho_r^{0.5} L_r$	0.149	1/2
Frequency	ω	$\omega_r = t_r^{-1}$	6.711	2
Pore water pressure	u	$u = L_r a_r \rho_r$	/	1/4
Shear wave velocity	V	V_r	/	1/2
Stress	σ	$\sigma_r = E_r$	1/4	1/4

FIGURE 3: Soil container.

where σ is the stress, L is the model geometry, E is the elasticity modulus, ρ is the mass density, d is the displacement, a is the acceleration of the model system, t is the time, ω is the frequency, u is the pore water pressure, and V_s is the shear wave velocity. For a tunnel model, L , E , and ρ were selected as the fundamental quantities, whereas L , E , and V_s were selected as the fundamental quantities for the soil model. Finally, the remaining quantities in (1) can be derived from these fundamental quantities.

The similitude relations and ratios designed for the tests are listed in Table 1.

3.2. Experimental Model. The tunnel model was scaled from the HZM immersed tunnel. The tunnel model has three segmented elements and two flexible joints. The interior of each element is divided using two inner partition walls. For a consistent density of the prototype, the additional balance leads were arranged uniformly on the base floor of the model. Cubic concrete specimens maintained under standard conditions for 28 days were tested to obtain their mechanical parameters (the compressive strength was shown to be 5.4 Mpa).

FIGURE 4: Experiment tunnel model.

A practical tunnel joint is composed of reinforced concrete shear keys, a GINA water strip, and an OMEGA water strip. In reality, owing to the limitation of the experiment model size, the scaled shear keys are too small to bear static and dynamic loads. Considering the fact that a tunnel joint is not the focus of these tests, a simplified rubber joint that can not only link with the tunnel elements but also act as a flexible joint was designed based on the tunnel elements. The joints were produced in a factory before the test and could be connected to the tunnel segments using reserved bole holes. Figure 4 shows the runner joint and assembled tunnel.

To complete the experiment targets, the loading amount, as well as the designed soil depth and the buried depth of the tunnel, needed to remain the same during each test. Installation of the entire model was conducted as follows:

- The ground for test 1 was made up of unsaturated sand, and a hierarchical compaction method was adopted during the manufacturing process. The assembled tunnel model was placed in soil at the designed burial depth. The remaining space was then filled to the designed height.
- For test 2, all steps of test 1 were repeated completely, with a final step applied to fully saturate the sand deposit with water flowing from the embedded pipes.
- For test 3, all steps of test 2 were repeated in full, with water added over the ground to the designed height.

TABLE 2: Experiment conditions.

Test number	Soil depth (mm)	Water layer depth (mm)	Density of soil (g/cm^3)
Test 1	1200	0	1.51
Test 2	1200	0	2.03
Test 3	1200	200	2.03

FIGURE 5: Plane view of observation surfaces.

The three tests were carried out under experiment conditions listed in Table 2.

3.3. Measurement Setup. A large number of pore pressure sensors, accelerometers, and strain gauges were embedded in the soil and installed in the tunnel model. For convenience, *W* is short for the pore pressure sensor, *A* indicates the accelerometer, and *S* is the strain gauge. The observation surfaces and sensor arrangements were kept the same during the three tests.

As Figure 5 shows, two observation surfaces were designed for the tests, that is, observation surface I, which was perpendicular to element 2, and observation surface II, which was perpendicular to element 3. According to Figure 6, A4 and A14 were installed on the top slab of elements 2 and 3, respectively. A1, A2, A3, and A5, and W1 through W5, were embedded in the soil of observation surface I. S1 through S8 were installed on the interior surface of element 2. Different degrees of shaking were applied along the longitudinal direction (as shown in Figure 3) of the multifunctional laminar shear container (direction *Y* in Figures 6(a) and 6(b)).

3.4. Experimental Cases. The shaking table was used to apply a uniform excitation, and only horizontal shaking was considered because vertical shaking is not steady. Because the shaking system has been in use for a long period of time, there was a discrepancy between the input and output excitations. Thus, for this paper, the input excitations were analyzed by making them equivalent to the seismic acceleration histories recorded from accelerometer A0 (as shown in Figure 3), which was attached to the shaking table. Thus, the output excitations obtained from accelerometer A0 attached to the shaking table, which are herein called an MW wave and a KB wave, were regarded as input excitations. The loading excitation amplitudes ranged from 0.1 to 0.4 g, as listed in Table 3. The seismic waves obtained from cases 1 and 2 are illustrated in Figure 7, which shows that the predominant frequency of an MW wave is lower than that of a KB wave.

4. Experiment Results and Analysis

In this section, an analysis of the typical data obtained from the three series of tests is described. To provide a comprehensive understanding of the seismic responses of the soil and tunnel at diverse sites, the following analysis and a discussion of the experiment results focus mainly on the acceleration responses of the soil and tunnel, the dynamic strain responses of the segmented elements, and the excess pore pressure of the saturated soil.

4.1. Acceleration Response of Soil Deposit. For tests 1, 2, and 3, the acceleration amplification coefficient of the soil deposit was used to study the seismic behavior of the soil deposit during the three tests and estimate the effects of the site condition on the ground motion. Here, the acceleration amplification coefficient is defined as the ratio of the peak acceleration value of any deposits to the peak acceleration value of the countertop. For an MW wave, the results from the accelerometers A2, A3, and A5 were selected for a thorough analysis. Figure 8 shows the acceleration amplification coefficient of the soil during each of the tests. As shown in Figure 8(a), the acceleration amplification coefficients for test 1 range from 1.0 to 1.5. As expected, the smaller the burial depth used, the larger the acceleration amplification coefficient. The maximum coefficient occurs near the ground surface. However, as Figure 8(b) shows, the acceleration amplification coefficients for test 2 ranged from 0.97 to 1.4. Inputting MW-1, as a result of the slow growth of the pore water pressure, the soil sustains a high level of strength, and thus the acceleration is significantly amplified. However, the acceleration histories of the sand layer initially increase (which takes place at a location below the tunnel) and then decrease (which occurs at a location between the tunnel and face layer of the ground) under excitations MW-2 and MW-3. These plots are mainly due to the differential development of the pore water pressure in the soil below and over the tunnel. In other words, the soil over the tunnel tends to liquefy under a strong ground motion, and as a result of soil liquefaction, vibration energy dissipation is incurred. Furthermore, as shown in Figure 8(c), the acceleration response of the ground during test 3 indicates a similar trend with that of test 2.

The above results indicate that the soil properties have an exceeding influence on the earthquake wave propagation on site (the site amplification effect of test 1 differs significantly from that of tests 2 and 3). In addition, the results obtained from tests 2 and 3 indicate that an overlying water layer has no influence on the seismic wave propagation in a liquefied site.

FIGURE 6: Plane view of instrumentation used during the tests: (a) accelerometers and pore pressure transducers on observation surface I, (b) accelerometers and pore pressure transducers on observation surface II, and (c) strain gauges installed in element 2 (unit: mm).

4.2. Acceleration of Segmented Elements. To study the relative movement between two segmented tunnel elements, the resulting readings from accelerometers A4 and A14 were selected for analysis.

Figure 9 shows the acceleration results of elements 2 and 3 under loading case MW-3 for the three different test types. As the figure shows, differences can be observed between the two elements; that is, the maximum acceleration of element 2 under excitation MW-3 for tests 1, 2, and 3 is 0.484, 0.577, and 0.370 g, respectively, which are reductions of 11.1%, 7.5%, and 9.1% compared to element 3. This is mainly attributed to the existence of the flexible joints. Because of the constraints of the joint at both ends and the vibration absorption of the rubber joints, element 2 showed a smaller amount of motion than element 3. This observation indicates that the influence of the tunnel joint is prominent. When the relative movement is distinctly large, pullout cracking and shear failure may occur in the segmented tunnel elements. Similar findings on buried segmented pipelines have been previously reported [19–21].

It should be emphasized that this study mainly focuses on the influences of the soil parameters and overlying water layer on the entire soil-tunnel system rather than the tunnel joint itself. In reality, a real tunnel joint is composed of reinforced concrete shear keys, a GINA water stop, and an OMEGA water stop. Owing to a limitation of the reduced-scale model, reinforcement shear keys were unable to emerge, and thus a tunnel joint was simply used in the present paper instead of a rubber joint, which can connect the segmented tunnel elements together and act as a water stop used in a real tunnel joint. However, this simplified joint model seems unable to explain the dynamic response mechanism of a real tunnel joint, and thus further study on the mechanical behavior of a tunnel joint will be conducted during the next test.

4.3. Structural Dynamic Strain. Before the tests were conducted, strain gauges were attached to the nodes between the walls and top slab and to the nodes between the walls and floor slab. During the testing process, strain gauges S1, S6, and S8 broke down. Finally, the results (peak dynamic strain) obtained from the effective strain gauges are summarized in Table 4.

Table 4 shows that a greater excitation results in a greater peak dynamic strain. In contrast to test 2, it is clear that the dynamic strain at the same point during test 1 is smaller. This is due to the difference in sand density between tests 1 and 2, as shown in Table 2. In addition, the slurry induced through sand liquefaction is also an important factor. A comparison of tests 2 and 3 indicates that the dynamic strain obtained from survey points S2, S5, and S7 during test 2 is slightly smaller than that during test 3 as expected. Owing to the overlying water layer during test 3, this may increase the dynamic water pressure on the tunnel during the shaking process. Agreement still exists among all of the tests despite the above-mentioned differences. Under excitations MW-1 and MW-2, the locations with a relatively large peak dynamic strain are S2, S3, and S5, whereas locations with a relatively small peak dynamic strain are S4 and S7. Under excitation MW-3, the locations with a relatively large peak dynamic strain are S2 and S5, whereas the locations with a relatively small peak dynamic strain are S3, S4, and S7.

To analyze the increasing regularity of dynamic strain, the dynamic growth rate is defined through

$$\text{INCR} = \frac{(R_j - R_i)}{R_i}, \quad (2)$$

where R_i is the peak dynamic strain under excitation MW-1 and R_j is the peak dynamic strain under higher excitation (MW-2 and MW-3).

FIGURE 7: Excitations during testing: (a) MW wave obtained from loading case 1, (b) Fourier spectra of (a), (c) KB wave obtained from loading case 2, and (d) Fourier spectra of (c).

TABLE 3: Loading cases, where PGA indicates the peak ground acceleration (unit: g).

Item	Loading case	Loading direction	PGA (g)
1	MW-1	Y	0.1
2	KB-1	Y	0.1
3	MW-2	Y	0.2
4	KB-2	Y	0.2
5	MW-3	Y	0.4
6	KB-3	Y	0.4

Based on Table 3, with respect to the data under excitation MW-1, the dynamic strain growth curves under excitations MW-2 and MW-3 for all tests are illustrated in Figure 10. The larger the peak acceleration value is, the faster the dynamic strain growth rate that occurs. The soil properties differ in

terms of the structural dynamic strain, which are reflected in two ways. First, under excitation MW-1, the dynamic strain growth curve of test 1 is above that of the two other tests, and the inflection points of test 1 are also different from those of the two other tests. In particular, no significant differences in the dynamic strain growth between tests 2 and 3 were noted. Thus, it can be seen that the overlying water does not greatly influence the structural dynamic strain. Second, the spatial effect of the strain distribution and the influence of the acceleration amplitude on the dynamic strain growth can also be seen in Figure 10. For test 1, under excitation MW-2, the greatest growth rate occurs at S5, and the lowest growth rate occurs at S7, whereas under excitation MW-3, the greatest growth rate occurs at S2, and the lowest growth rate occurs at S3. However, the growth trends of all survey points during test 2 maintained pace with those during test 3 and illustrate that the fastest and slowest growing points are S7 and S2

FIGURE 8: Soil acceleration amplification coefficients of (a) test 1, (b) test 2, and (c) test 3.

under excitation MW-2 and S2 and S4 under excitation MW-3, respectively.

4.4. Effects of Overlying Water Layer on Pore Pressure Generation. In this section, the pore pressure data obtained from the series of shaking-table tests selected to study the influence of an overlying water layer on the liquefaction mechanism of a saturated sand deposit are described. It is worth noting that dry sand was used during test 1, and no pore pressure data were observed. Therefore, only the results of tests 2 and 3 were analyzed.

Because a pore fluid cannot escape quickly enough in a collapsing pore void, the pore pressure increases, whereas the frictional resistance of the soil decreases. The excess pore pressure ratio (EPPR) expressed in (3) can be used to

evaluate the soil liquefaction, where EPPR1.0 indicates full soil liquefaction.

$$\text{EPPR} = \frac{U}{\sigma}, \quad (3)$$

where U is excess pore pressure and σ is the initial effective stress.

Figure 11 shows the EPPR based on data obtained from the pore pressure transducers W2 through W5 (placed along the same vertical line) during tests 2 and 3 under excitations MW-3 and KB-3. Some similarities between the two tests were found. First, the larger the excitation, the faster the generation of the testing pore pressure, and the maximum EPPR takes place in the ground at W5. Second, when inputting the same amplitude with a different seismic wave, the EPPR of

FIGURE 9: Comparison of acceleration obtained from A4 and A14 during the testing; acceleration time histories of (a) test 1, (b) test 2, and (c) test 3.

TABLE 4: Peak dynamic strain values during the tests.

Item	Test 1			Test 2			Test 3		
	MW-1	MW-2	MW-3	MW-1	MW-2	MW-3	MW-1	MW-2	MW-3
S2	12	22	89	18	25	114	23	27	113
S3	14	26	56	19	32	67	17	28	50
S4	9	17	44	20	27	41	16	19	37
S5	18	41	101	33	51	97	39	60	112
S7	7	10	41	10	17	31	16	28	50

FIGURE 10: Comparison of INCR in tests under (a) excitation MW-2 and (b) excitation MW-3 as compared to excitation MW-1.

the sand deposit under excitation KB-3 is larger than that under excitation MW-3, which indicates that the excitation properties greatly influence the pore pressure generation. Lastly, the sand deposits from different burial depths differ greatly in terms of the EPPR. It is clear from Figure 11 that the EPPRs from W4 and W5 are greater than those from W2 and W3. The differences in EPPR between W5 and W2, W5 and W3, W4 and W2, and W4 and W3 under excitation MW-3 are 0.890, 0.881, 0.279, and 0.270, and 0.884, 0.882, 0.280, and 0.270 for tests 2 and 3, respectively. Furthermore, there are various consolidation stresses that occur in sand above and below a tunnel structure. The additional structural load significantly increases the consolidation stress of sand below the structure. In contrast to the sand layer over the structure, the movement of soil particles from the sand layer below the structure is slower. In this case, the degree of liquefaction of the sand layer below the structure is lower. This observation also agrees with results by Chen et al. (1998) and Men et al. (1998) [22, 23], who found through a series of small shaking-table tests that the pore pressure of an area suffering the greatest amount of pressure of a building is smaller than that of other areas.

However, a difference exists between the EPPR results of these two tests. Figures 11(a) and 11(c) describe the maximum EPPR that occurs during the growth phase at 11 and 20 s corresponding to tests 2 and 3 under excitation MW-3; the excess pore pressure dissipation phase then begins. Similar conclusions can also be obtained from Figures 11(b) and 11(d), in which the dissipation channels (influenced by water layer over the ground) during test 3 are shown to be rather occlusive compared to those during test 2.

5. Conclusions

A series of shaking-table tests were designed to investigate the dynamic performance of a segmented immersed

tunnel under seismic excitations. Detailed information on the experimental similitude relation design, model setup, measurement setup, and loading cases was introduced and would provide references for future researches in similar fields. Three tests classified based on the site conditions were then conducted. Finally, through these tests, important data were obtained, including not only the seismic responses of the soil, but also the acceleration responses and dynamic strain responses of the tunnel. The following conclusions were drawn based on an analysis of the experiment results:

- Whether the sand is saturated significantly influences the propagation of seismic waves because soil liquefaction usually absorbs the seismic energy. However, an overlying water layer has a slight influence during seismic wave propagation in a soil deposit composed of saturated sand.
- The effects of flexible joints on the relative movements among the segmented elements are obvious. The selection of appropriate tunnel joints is crucial during the design process of an immersed tunnel.
- In this study, a larger dynamic strain was shown to generally occur at the joints between the bottom plate and middle walls of the tunnel model. When inputting the same amount of excitation, the dynamic strain obtained from the tunnel model of test 2 was clearly larger than that of test 1, and simultaneously, the dynamic strain obtained from the tunnel during test 3 was nearly consistent with that of test 2. The effects of a liquefied site on the seismic performance of an immersed tunnel should be given special attention. However, the influence of an overlying water layer on the seismic responses of the tunnel can be neglected.

FIGURE 11: EPPR under (a) excitation MW-3 and (b) excitation KB-3 during test 2 and under (c) excitation MW-3 and (d) excitation KB-3 during test 3.

(d) The properties of the earthquake excitations influence the pore pressure of the soil layer. An overlying water layer makes no apparent difference on the excess pore pressure mechanism in addition to the small differential dissipation time of pore pressure. Thus, an overlying water layer can be overlooked when focusing on the general law of soil liquefaction.

Conflicts of Interest

The authors declare that they have no conflicts of interest regarding the publication of this paper.

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