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Bridge abutment sliding and its seismic response in liquefied areas

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ABSTRACT: In this paper, the problem of bridge abutment sliding in liquefaction areas is studied by theoretical and experimental methods. The abutment–soil system is treated as a plane–strain problem in the computation by considering the effect factors, i.e., weight of deck beams, liquefaction of saturated sand under the abutment and the different input accelerogram. A series of simulation tests on the shaking table were performed and analysed. The experiments successfully reappeared the sign of bridge abutment sliding during liquefaction on the quake site, which can fairly verify the mechanism of abutment sliding in liquefied areas.

1 INTRODUCTION

Bridges are important nods in lifeline system of a metropolitan areas. They must be able to carry limited traffice after an earthquake so that emergency vehicles may gain access to heavily damaged areas in order to remove victims and to provide disaster relief. Bridge must be brought back to full capacity in a short time after an earthquake so that the transportation network of the area will function smoothly and allow the economic system to recover and operate efficiently.

From the field investigation of bridge damages in many destructive earthquakes, the damages of a large number of bridges were mostly caused by the sliding of bridge abutments together with their backfills. In this case, the declining and breaking of bridge piers and the falling of deck beams are the secondary hazards caused by bridge abutment sliding. With concluding the damage features and the results of the field boring after an earthquake, we can see, as to the aseismic design of bridge abutment, the "Monoobe–Okabe Formula" adopted for half a century can not explain the fact of such damages of abutment mentioned above, especially that in the field of liquefaction. Hence, it is an urgent need to solve the problem of earthquake response of abutment–soil system with the methods of thorough theoretical computation and experimental research.

2 THEORETICAL COMPUTATION

2.1 Computation model

Bridge abutment and its attached construction form a complicated 3–dimensional system. Considering the practice of the computation, the finite element method is applied to analyse the seismic response of abutment–soil system in this paper, which is treated as a plane–strain problem in order to save computing time significantly. Most abutment were made of concrete or masonry which is supposed to be a linearly elastic material with viscous damping, while the soil is supposed to be a nonlinear material. For sand and clay, variation of shear modulus and damping ratio with shear strain are based on H. B. Seed's data[1]. Poisson's ratio of soil and abutment is taken as 0.4 and 0.1 respectively. The proposed method is reasonable in checking the stability and seismic response of abutment–soil system especially when any liquefied layer exists below the abutment.

The equation of earthquake motion of the system is given by

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+[K]\{x\}=-[M][I]\{\ddot{x}_g\} \qquad (1)$$

The damping of the system is incorporated using a Rayleigh–type damping matrix of the form

$$[C]=a[M]+b[K] \qquad (2)$$

We use Wilson–θ method to perform direct integration. For unconditional stability we employ $\theta=1.4$ and select a time step $t=0.01$ second. In the computation a liquefied layer of sand is supposed to be below the abutment as shown in Fig. 1. The shear modulus of this layer is selected from SHI's recommendation based on site and laboratory experiment[2]. That is

$$G_{liq}=0.0125G_{max} \qquad (3)$$

Table 1. Analysis results of case 1

W	0	5	10	20
D	0.629	0.640	0.644	0.654
ω	4.135	4.047	4.057	4.023
P_a	17.43	18.26	19.67	20.73
h	0.41H	0.44H	0.48H	0.51H
P_p	10.86	11.01	11.89	12.94

* note:
h is the height of the active earth pressure P_a from the bottom of the abutment. ω is the circle frequency of the system.

Table 2. Analysis results of case 2

A	Elcentro	QianAn(EW)	QianAn(NS)
D	0.593	0.289	0.276
ω	3.574	4.983	4.976
P_a	34.47	14.89	14.76
h	0.51H	0.54H	0.52H
P_p	8.54	3.86	3.92

* note:
A is acceleration input, h is the height of the active earth pressure P_a from the bottom of the abutment and ω is the circle frequency of the system.

In order to consider the influence of the pore water pressure, the effective stress analysis method is employed in the computing program. It is supposed that the shear modulus of sand decreases with the square root of the effective stress. That is

$$G = G_{max}(\sigma_1 / \sigma_0)^{\frac{1}{2}} \tag{4}$$

where, σ_0 is initial effective stress, $\sigma_1 = \sigma_0 - u$(u is pore water pressure). The details can be reffered from the author's previous work[3].

2.2 Case study and analysis

Case 1: Let the abutment height H = 8 meters, the weights of deck beam are taken as 0, 5, 10 and 20 ton / meter, respectively. Then, the calculating results as shown in Table 1 indicates that: (1) When a liquefied or soft layer is under the abutment, the earthquake–induced earth pressure increases with the increment of the weight of the deck beam. Also, the point of the dynamic earthquake pressure acted on the abutment goes up. Hence, the effect of the weight of deck beam must be taken into account in the aseismic design of bridge abutment. (2) In the range of 5–15 meters behind the abutment, the horizontal stress σ_x of the soil element changes irregularly during the accelerogram input from the bed rock. But from the time history of σ_x of the elements in the 2 or 3 soil columns between the range of 7–10 meters behind the abutment, we can see the maximum tensions σ_x appear nearly at the same time, which will cause one or two transverse vertical cracts on the surface of the backfill as described in the site investigation[4]. Combining the results of the investigation and experiment discussed below, we generally assume that the main transverse vertical cract usually appears in the range of 0.8–1.2H (H is the height of abutment) behind the abutment.

Case 2: Let H = 12 meters, the weight of the deck beam is assumed as 20 ton / meter, the earthquake accelerogram inputs are Elcentro, QianAn(EW) and QianAn(NS), respectively. From the analysis results as shown in Table 2, we can obtain: (1) Different accelerograms have different spectral characteristics, hence their effects on the response of abutment are different. (2) With increasing the height of the abutment, the active dynamic pressure P_a applied on the abutment increases, but the passive dynamic pressure P_p decreases. Therefore, the risk of abutment sliding is raised.

3 SHAKING TABLE TEST

As shown in Fig. 2, the model ground was made of saturated sand 40 cm deep and 135 cm long. A layer of coase sand was sticked to the bottom of the box to ptevent slippage between the model ground and the box. The model abutment was made of concrete. Furthermore, 5 cm thick foam cushions were placed at both side boundaries as radiational boundaries. The Elcentro earthquake accelerogram was applied as simulating seismic input, which intensity increased from 0.1 to 2.24 gravities gradually until the model ground liquefied and destructed.

3.1 Experimental phenomena

In the experiment, generally 2 or 3 transverse vertical cracts were found on the model ground surface of backfill, usually at the range of 0.8–1.2H(H is the height of the model abutment.); the abutment settled, slided and inclined backward; the surface of the model ground in the front side of the model abutment heaved with sand boiling; the boiling spot also reappeared generally. All these phenomena coincided with the sign of the quake site investigation[4].

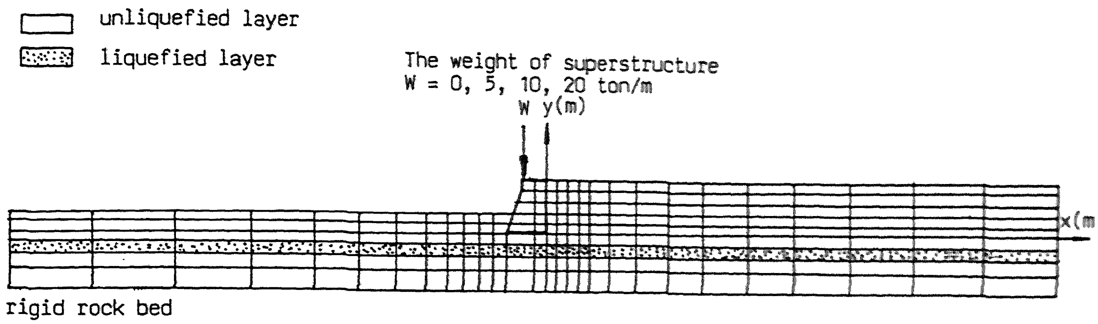

Fig. 1. Numerical model and its finite element subdivision

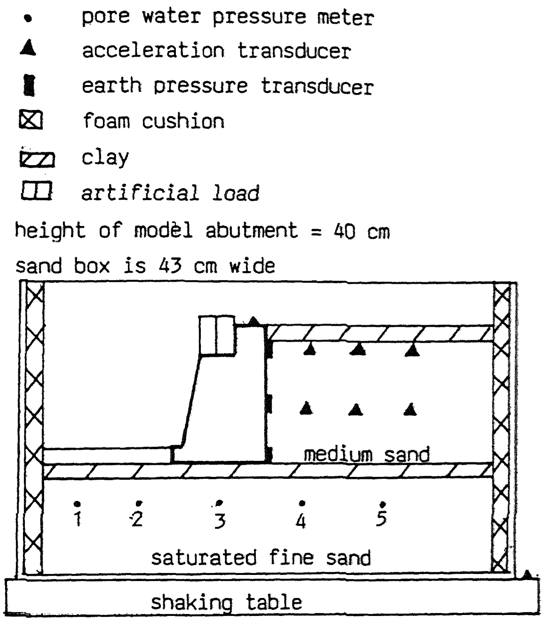

Fig. 2. Test model instruments

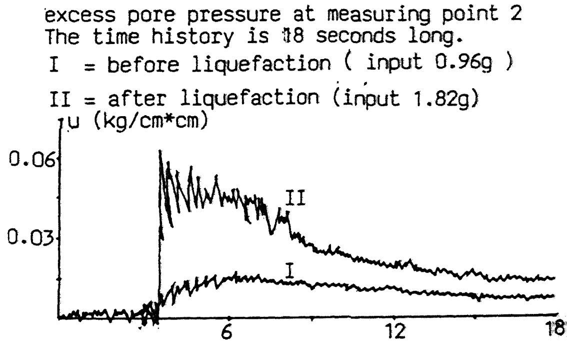

Fig. 3. Time history of excess pore pressure

3.2 Experimental data analysis

Before the model ground liquefied, as shown in Fig. 3, with the increment of the input intensity, the excess pore pressure incresed, but they dissipated slowly. After the model ground came to the stage of liquefaction, the excess pore pressure dissipated dramatically due to the sand boiling at the surface of the model ground in the front side of the abutment where the effective stress was lower than the other parts of the ground. From the analysis of the maximum excess pore pressures and their ratios shown in Figs. 4 and 5, we can acquire that the critical pore pressure ratio for the model ground to settle and slide dramatically are 0.2–0.4 beneath the model ground in the front side of abutment and 0.5–0.7 for the ground underlying the model abutment, which indicates that the abutment sliding is mostly caused by the ground liquefaction in the front side of the abutment that make the ground lose its bearing capacity and stability. As to the settlement of the model ground, before the liquefaction, the ratio of the accumulate settlement was 3%, and at the stage of liquefaction, the ratio of the accumulate settlement of the ground was nearly 7.5–8.7%. So the most important is that we should take appropriate measures to strengthen the ground in the front side of the abutment first[5].

For the acceleration response and their power spectra, when the input intensity was low, the acceleration response increased almost linearly and their nonlinear deformation was very small. However, continuing to input stronger seismic wave, the model specimen came to nonlinear stage and liquefaction gradually. Their acceleration amplified factor D degraded greatly and the peak values of their power spectra moved ahead, which indicated that the natural period of the site had been changed.

Finally, from the analysis of the earthquake–induced earth pressure acted on the model abutment, we can conclude that when artificial load was put on the top of the model abutment to simulate the weight of the superstructure of bridge, the distribution of the dynamic pressure was changed, also, the total pressure increased and its concentrated acting point

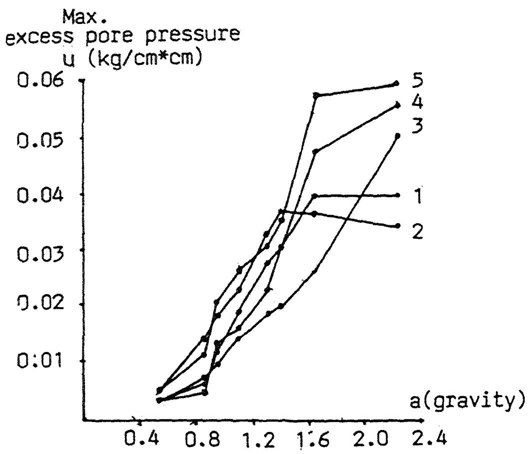

Fig. 4. Relationship between maximum excess pore pressure and input intensity

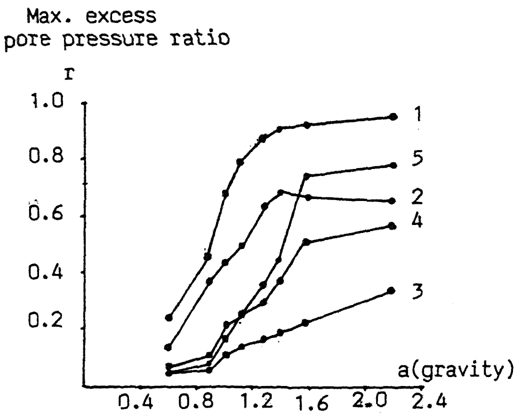

Fig. 5. Relationship between maximum excess pore pressure ratio and input intensity

moved up, which trend coincided with the numerical analysis.

4 CONCLUSIONS

Based on the above numerical and experimental analysis, it is concluded that bridge abutment sliding due to liquefaction is caused by the saturated sand firstly liquefied beneath the ground soil in the front side of the abutment which subsequently causes regional instability and make the ground soil lose its bearing capacity; the interaction between the superstructure and the abutment changes not only the distribution of the earthquake–induced earth pressure acted on the abutment but also its magnitude; in addition, the spectra of acceleration for the liquefied site are different from normal site spectra, which must be taken into account in the design and strengening of bridge abutments in liquefied areas.

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