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The Prediction of Liquefaction Damages to a Large Span Bridge on Karoon River, Iran

S. M. Mir Mohammad Hosseini
Amir Kabir University of Technology, Iran

THE PREDICTION OF LIQUEFACTION DAMAGES TO A LARGE SPAN BRIDGE ON KAROON RIVER

Dr. S. M. Mir Mohammad Hosseini
Associate prof. of Amirkabir University of Technology
Tehran - PO. Box 15875 - Fax No. 21-2574966 - IRAN.
E-mail:mirh53@hotmail.com

ABSTRACT

Several damages have been imposed to bridges due to liquefaction of the soil layer in the position of their foundations. However, there is not a specific world - wide provisions for design of bridges in these conditions. In this paper the results of evaluating liquefaction potential and the consequence damages which is likely to happen for a large span bridge (Shirin-Shahr Bridge) to be constructed on the Karoon river (one of the main river in the south-western part of Iran) are presented. Since the liquefaction potential of a site depends not only on the geotechnical characteristics, but on the intensity of the ground motions as well, a comprehensive seismic hazard analysis was carried out to estimate the appropriate PGA in the area. At the end some geotechnical comments are suggested for the aseismic design of the bridge foundation accordingly

INTRODUCTION

There has been an extensive progress in understanding the concepts and consequences of liquefaction of the ground during heavy earthquakes, since it was first recognized during Nigata earthquake in 1964. The flow failure and cyclic mobility are the two different phenomena which today are well defined and cover all behaviors of liquefiabel soils under earthquake loadings . Nevertheless, the influence of liquefied subsoils on the important and sensitive structures such as large bridges are not yet known and defined clearly. In Great Hanshin earthquake, 1995, several damages imposed to many bridges all designed to well resist against heavy earthquakes. In Northridge earthquake, 1994, several bridges collapsed due to liquefaction of the soil layers beneath their foundations.

Although the flow failure of the ground in the shallow layers may lead to a complete destruction of bridges rested on shallow foundations, the bridges on pile foundations may however, escape damaging in the same situation. Nevertheless, the degree of predictable damages for these type of bridge, the appropriate intensity of the ground shaking to be considered, and the precaution measures have to be taken into account for safe design of the deep foundations, are among the important subjects still need to be studied and investigated for a specific site. In the following sections, these problems for the site of interest are focused and described.

THE GENERAL SPECIFICATIONS OF THE BRIDGE SITE

The site of interest has located 35 kilometers far from city of Ahwaz , the capital of Khoozestan province (fig.F1) . The Khoozestan is one of the most important south-western province of Iran. The longitude and latitude of the site which is in the south-western of Ahwaz are $48°,22',21''$ and $31°,4',28''$ respectively. The site is surrounded by three areas namely: Farciat II, Mazban and Talle-Asvad . The site has been situated in the down stream of a few big rivers among which the Karoon has influenced its geological conditions dominantly. The site is far from the highly folded areas of Zagros chain of mountains. As a result the ground is nearly flat by a maximum slope of 2.7(%). Due to low level of the ground, the water has over-flown during the floods of Karoon River, which left sedimental deposits in place. According to local data, the maximum river flow at the position of the bridge has been recorded to about 3000 m^3/sec, which implies the potential of erosion in the area.

The Shirin-Shahr bridge which connects the two main areas of industrial centers on the two sides of the Karoon River, is a prestressed concrete boxbeams bridge with side cantilevers of 340 meters total length. The bridge has two side spans of 97 meters and one middle span of 146 meters. It has 15.5 meters width and is going to rest on pile foundations. The abutment foundations consist of 20 piles of 1.5 meters diameter and 38 meters length while the pier foundations have 26 piles of 1.5, meters diameter , extended to some 46 meters depth .

Fig. 1- The location of the site under consideration (Shirinshahr Bridge).

THE GEOLOGICAL AND GEOTECHNICAL CHARACTERISTICS OF THE SITE

According to existing data the site has been laid on the recent deposit having low resistance against flow erosion of the Karoon River. It has layers of nearly uniform and fine alluvium underneath. There are some sand lenses inside the layers which have been formed due to sedimentations in deferent ages. The thickness of so formed quaternary and recent alluviums, are estimated about 100 meters on the Aghajari formation and the whole seismic site effects of the area depend on the characteristics of this alluvium.

In order to obtain the geotechnical characteristics of the soil layers, two boreholes up to about 60 meters depths were drilled on both side of the bridge. Different samples were taken for laboratory tests and the SPT was carried out at each 1.5 meters intervals up to 20 meters depth after which the intervals were doubled (fig. 2).

According to logs of borings , the soil up to 20 meters depth consists of different layers of CL, MS, CH and SM with low penetration resistance which some of them are highly likely to liquefy in future earthquakes .

The layers in the east side of the river (bore hole No. 2) show an offset in comparison with the other side , and are considerably weaker , which may be attributed to more sedimentation activity in the past , as they are located near the bent of the river .

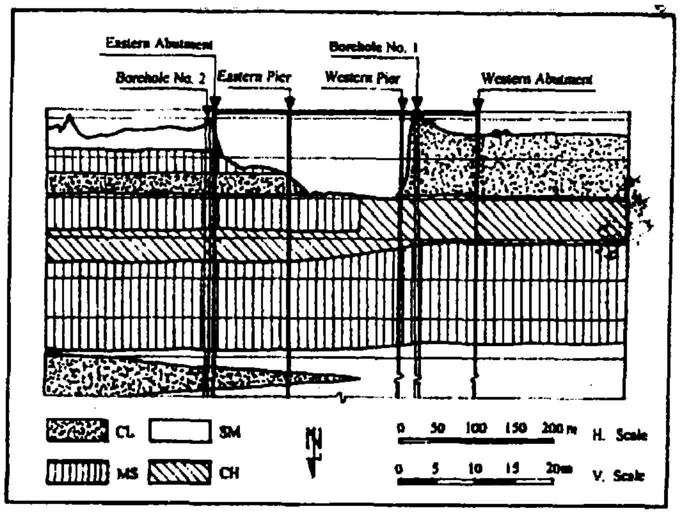

Fig. 2- The ground profile at the position of the bridge and location of two boreholes .

THE SEISMICITY OF THE SITE

Since a comprehensive and accurate seismic hazard analysis was not carried out in the past, in order to get appropriate information about seismicity of the site, an extensive seismic hazard analyses for the site specific were performed.

In this respect, all earthquake records (historical and instrumental) available in an area of 150 km radius around the site were collected. All active faults and seismic sources in this area were identified. The data were analysed , using both deterministic and probabilistic seismic hazard methods .

In these studies the seismic sources were modeled and the ground motion parameters were determined. The maximum magnitude for the main seismic source was estimated, and using the Gutenburg-Richter relationship the recurrence law was obtained. According to an appropriate attenuation relationship for the site to source distance, the intensity parameter was reduced and the seismic hazards for different probability in the site were calculated. The results of the analyses are shown in fig. 3 in form of the variation of peak ground acceleration (PGA) exceedance probability versus PGA, for a return period of 50 years (bridge effective lifetime) , and also annually . According to this graph, The PGA which is likely to exceed 10% in 50 years is equal to 0.226 g .

The period return of this earthquake would be 475 years. The PGA based on the deterministic seismic hazard analysis was also obtained, which is equal to 0.15 g. Since, the PGA obtained from PSHA is more realistic and considers all uncertainties in the seismic studies, it was used for liquefaction potential evaluation of the site.3

THE LIQUEFACTION POTENTIAL OF THE SITE

According to geotechnical characteristics of the site obtained from in situ and laboratory tests, and having the appropriate PGA obtained from seismic hazard analyses , the liquefaction potential of the site was evaluated , using different analytical

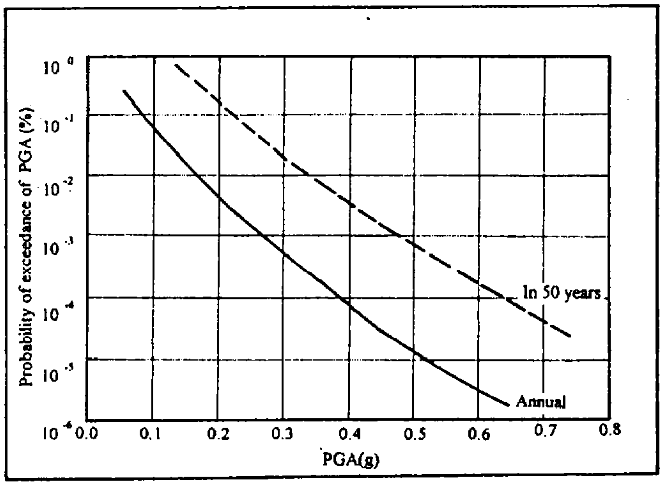

Fig. 3- Variations of exceedance probability of PGA versus PGA for the site .

methods . The methods used, were those based on the SPT results, namely " AASHTO code, Japanese Highway Bridge code (JRA , Japan Road Association , 1991), and corrected Seed and Idriss method (modified by TC4). Since each of the above methods are capable of giving relatively accurate results only in a certain type of soil deposit, during an extensive studies carried out by the author in microzonation of the south-eastern part of the city of Tehran (capital of Iran) financed by the IIEES and SMRM of Iran, a new method was developed to achieve the best results by a special combination of the above methods depending the type and texture of the soil deposit. In the current study this approach was used as the 4th one and called the suggestion method to offer the final results.

The whole procedure of liquefaction potential evaluation of the site for each side of the river and for pier and abutment foundations separately are compacted in three tables, a set of which are shown in tables 1 to 3. These tables are log of basic information, log of final information and methods of evaluation respectively. The two important factors; namely the liquefaction potential index (P_L) and the site liquefaction potential (I_{PL}), as well as the factor of safety against liquefaction (F_L) were also estimated. These factors for the site in positions of bridge piers and abutments in liquefiable zones are given in table 4.

THE PREDICTION OF PROBABLE DAMAGE TO THE BRIDGE DUE TO LIQUEFACTION

To evaluate the liquefaction damages which is likely to impose to the bridge, based on the Tokida, k. approach, and considering the liquefaction potential of the site and structural characteristics of the bridge, the following procedure was carried out:

- The Indices of p_1 and p_2 , for abutments and piers of the bridge were calculated according to equations (1) and (2) respectively:

$$P_1 = (I_C + I_J + I_F + I_{FS})\, I_{PL} \times I_{HB} \quad \text{(For abutment),} \qquad (1)$$

$$P_2 = (I_F + I_{FS})\, I_{PL} \quad \text{(For pier),} \qquad (2)$$

where ;

Ic = Effect of device against fall of a girder (0 , for the relevant bridge) .

I_J = Effect of girder - support condition on the top of pier/abutment (1, for pier and 0, for abutment of the relevant bridge).

I_F = Effect of foundation type (1, for pile foundation of the relevant bridge).

I_{FS} = Effect of bearing condition at the bottom of foundation (1 , in case of the relevant bridge)

I_{HB} = Effect by flow of abutment - backfill (1.2 for the relevant bridge due to more than 5 meters height of the backfill).

I_{PL} = Effect by liquefaction (site liquefaction potential: I_{PL}):

$0.2 \leq I_{PL} < 0.4$: 1.0

$0.4 \leq I_{PL}$: 1.5

- The obtained P_1 was compared with 6 (as the critical value).
- The obtained P_2 was compared with 4.5 (as the critical value).

The above mentioned limits are the critical values, below which the damages-imposed to the bridge were observed to be small and negligible. Since in case of the bridge under study, these indices were both found to be under the above limits, in spite of some liquefiable zones anticipated, the damages will not be considerable to make countermeasurements necessary. Nevertheless, for the sake of having more safety, it was suggested to reduce the bearing resistance of the liquifiable zones according to table 5 when the foundations piles are designed.

SUMMARY AND CONCLUSIONS

The liquefaction potential of a specific site, upon which a large span bridge is going to be constructed, was investigated. The bridge will connect two sides of the Karoon river at a point of about 35 km far from the city of Ahwaz (capital of Khoozestan province). It is a pre-stressed concrete boxbeams of 340 m length, deviding into two side spans of 97, and one middle span of 146 meters, resting on a group of deep piles. Prior to liquefaction studies a comprehensive seismic hazard analysis was carried out and the most appropriate PGA was estimated.

Two boreholes were drilled at the two sides of the river near the abutment of the bridge, and the SPT numbers of the soil layers, as well as other soil properties at different depths were measured. The liquefaction potential of the site at positions of abutments and piers were evaluated using three different methods, the results of which were combined by a special technique developed by the author.

Considering the liquefaction potential of the site and the structural characteristics of the bridge different indices were estimated to assess damages which is likely to happen to the bridge in case of earthquake loadings . In spite of some liquefiable zones observed in these conditions, no major damages were anticipated according to the calculated indices. Finally a geotechnical comment was made to design the pile foundations of the bridge more safely.

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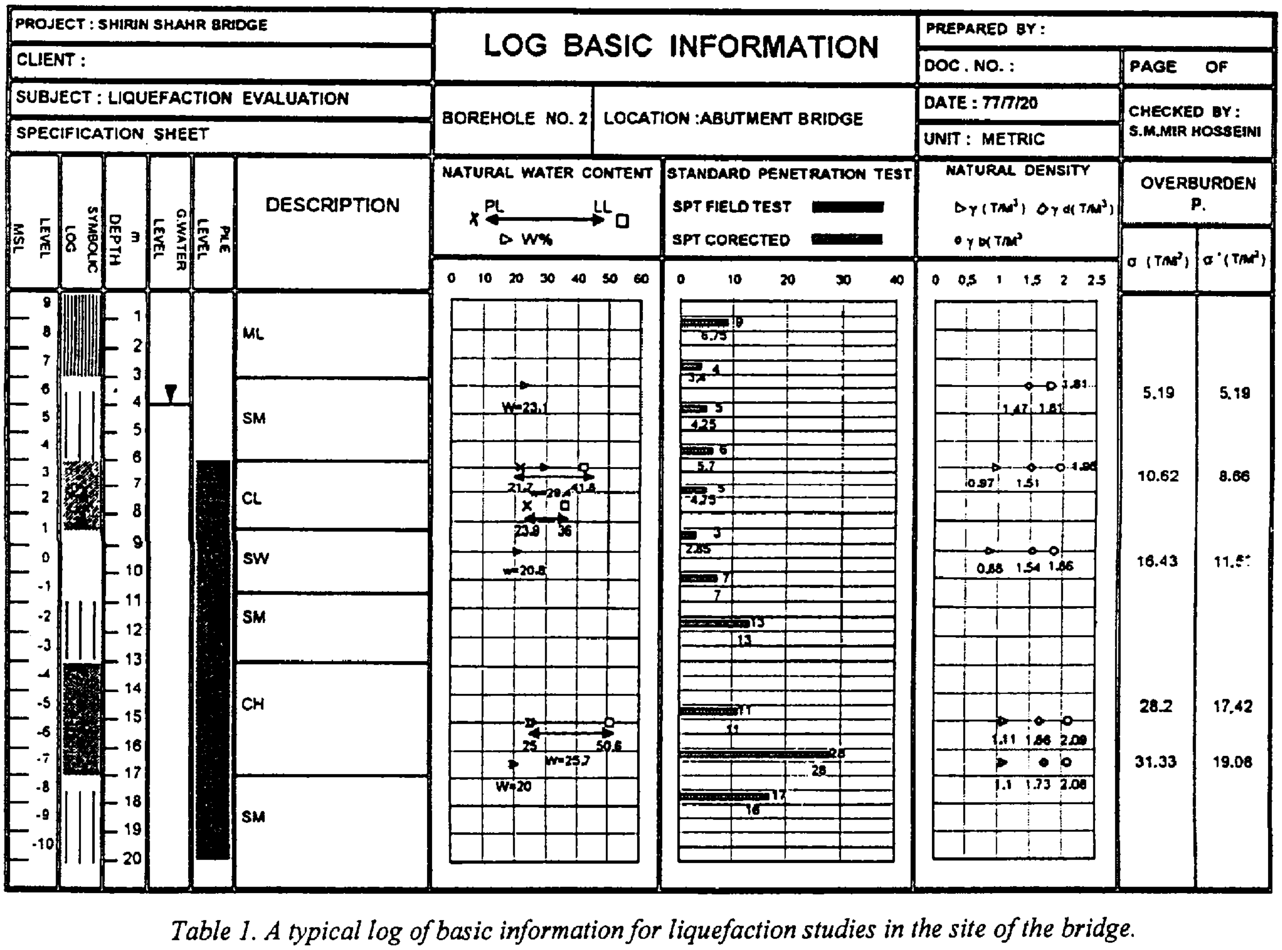

Table 1. A typical log of basic information for liquefaction studies in the site of the bridge.

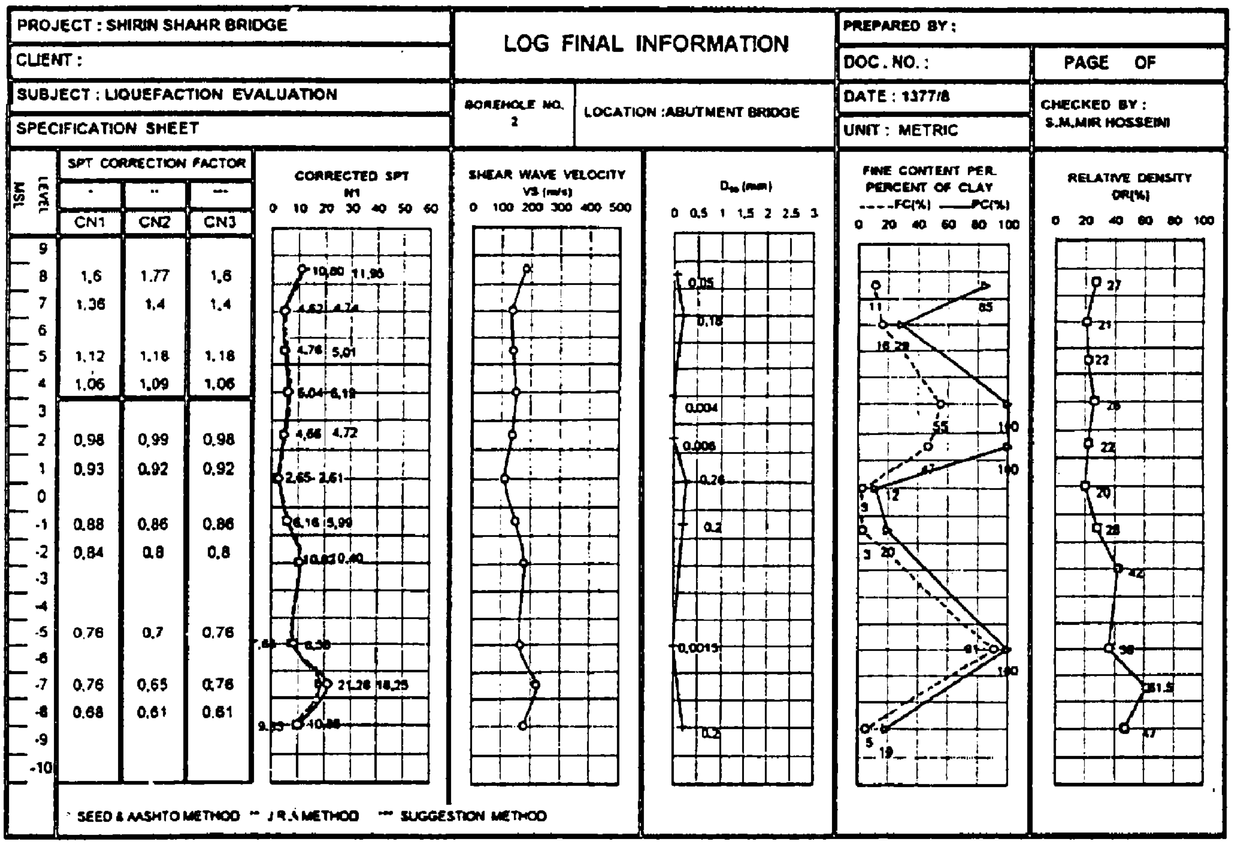

Table 2. A typical log of final information for liquefaction studies in the site of the bridge.

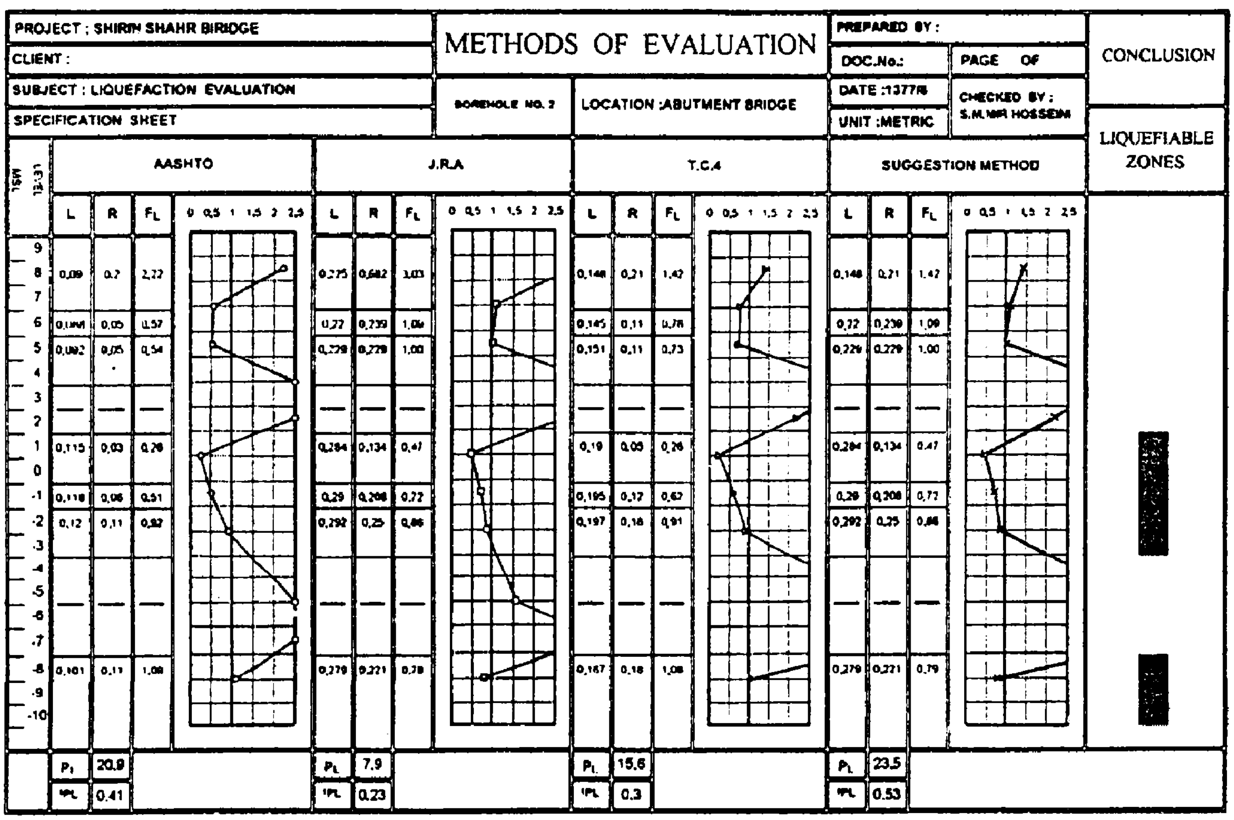

Table 3. A typical presentation for the liquefaction studies in the site of the bridge.

Depth (m)					
abutment		pier		P_l	I_{pl}
Left	Right	Left	right		
-	7-11	-	-	8.58	0.21
-	-	-	1-2	6.52	0.15
8-13	-	-	-	23.5	0.53
17-20	-	-	-		
-	-	2-7	-	14.5	0.36
-	-	11-20	-		

Table 4- The P_L and I_{PL} factors of the site in liquefiable zones .

Reduction Factors	Depth , Z. (m)	F_L
0	$0 \le z \le 10$	$F_L \le 0.6$
1/3	$10 < z \le 20$	
1/3	$0 \le z \le 10$	$0.6 < F_L \le 0.8$
2/3	$10 < z \le 20$	
2/3	$0 \le z \le 10$	$0.8 < F_L \le 1.0$
1	$10 < z \le 20$	

Table5 - Reduction factors for bearing capacity of the liquefiabe zones.