EARTHQUAKE RESPONSE ANALYSIS OF THE HIGASHI-KOBE BRIDGE

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SUMMARY

This paper presents results of observation and analysis of the response of one of the longest cable-stayed bridges in the world to the Hyogoken-nambu (Kobe) Earthquake of January 17, 1995. It is determined that interaction of the foundations of the bridge towers with the supporting soil plays a decisive role in the overall structural behavior. The key factor governing the changes of the soil properties at this site is pore water pressure buildup, which results in liquefaction of the saturated surface soil layers under large dynamic loads. Models of the soil and structure are created and initially validated by accurately simulating the system response to a small earthquake. Soil parameters reflecting the pore-water pressure buildup in the strong earthquake are determined by an advanced nonlinear effective stress analysis, combining the Ramberg-Osgood model of stress-strain dependence with a pore pressure model based on shear work concept. They are utilized to investigate and simulate the interaction of the foundation and the supporting soil using the program SASSI with the flexible volume substructuring approach. The results show a good agreement with the observations and have useful implications to the scientific and engineering practice.

KEYWORDS: Dynamic soil-structure interaction; Hyogoken-nambu Earthquake; cable-stayed bridge; pore water pressure buildup; liquefaction analysis

1. INTRODUCTION

The Higashi-Kobe Bridge in Kobe City, Japan, is one of the longest cable-stayed bridges in the world. It is a part of one of the most important transportation arteries in Japan-The Osaka Bay Route, which is an 80-kilometer expressway stretching from the western end of Kobe to the southern end of Osaka (Figure 1). The Higashi-Kobe Bridge spans the

Higashi-Kobe Channel connecting two reclaimed land areas: Uozaki-hamamachi on the west and Fukae-hamamachi on the east (Figure 1). The Higashi-Kobe Channel is 500m wide and has a 455m seaway where large ferries frequently pass and maneuver to dock at the nearby Ohgi Ferry Terminal.

At 5:46 a.m. on January 17, 1995, Kobe was struck by the most devastating earthquake[1)] since the Great Kanto Earthquake of 1923. Its epicenter was located at 34° 36"N and 135° 00"E. The JMA magnitude of the earthquake was 7.2, its surface wave magnitude M_S was 7.2 and the moment magnitude M_W was 6.9. The focal depth was 14 km. This paper presents results of observation and analysis of the behavior of the Higashi-Kobe Bridge during the Hyogoken-nambu (Kobe) Earthquake.

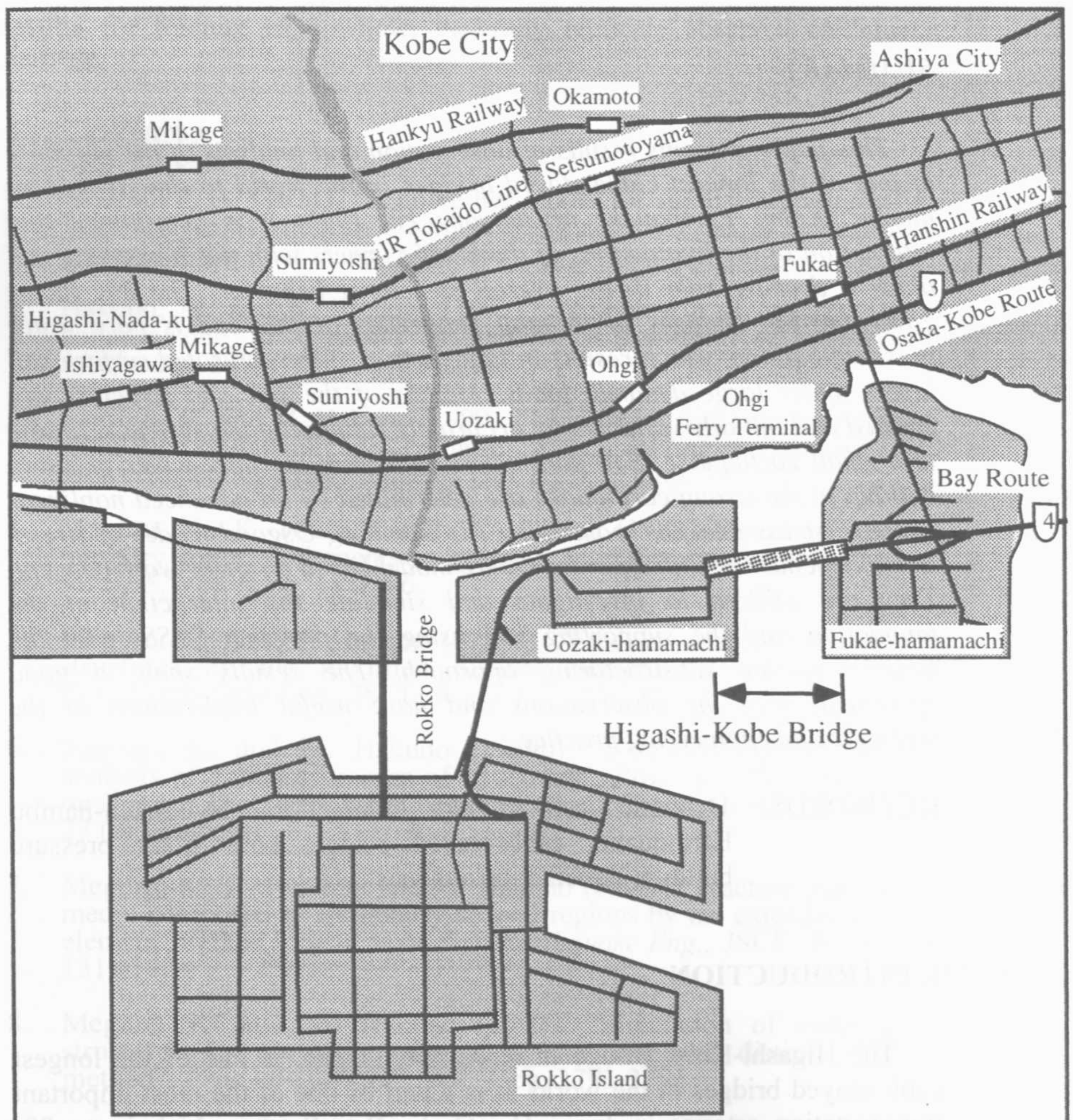

Figure 1. Location of the Higashi-Kobe Cable-Stayed Bridge

As will be shown in the following discussion, a simulation of the bridge response needs to take into account the soil-structure interaction. Engineering experience has shown that in addition to the well-developed theoretical basis of soil-structure interaction, it is necessary to validate the approaches in comparison with observed data and correct the numerical models to consider discontinuities and nonlinearities. One of the most important practical issues associated with interaction analysis is the degradation of the supporting soil due to large dynamic excitations. In general, three factors[2)] are considered most influential to the soil stiffness degradation: nonlinear stress-strain dependence of the soil material, separation of soil from the structure and pore-water pressure buildup. The relative importance of each of these factors varies with each analyzed case and usually determines the methodology of analysis.

The present authors have already published results of case studies in which the most influential factor has been the separation of soil from the structure[3)], and in which the decisive role has been played by soil nonlinearity[4)]. Of particular interest in the present case is that the key factor to soil-stiffness degradation is the pore-water pressure buildup. Publicized data of actual soil and structural response including this complicated phenomenon are not widely available.

The pronounced nonlinear effects accompanying pore-water pressure buildup and liquefaction are time-dependent. Nonlinear dynamic solutions in the time domain only for soil or for structural response are readily available and their practical application is feasible. However, when the interaction of soil and structure is addressed, it is important to consider also the frequency dependence of the soil parameters. For this reason, and to decrease the computational effort and data storage requirements, most of the commercially available programs such as SASSI[5)] operate in the frequency domain with time-independent parameters. In this way, it appears that the most practical way to analyze soil-structure interaction would be to determine equivalent time-independent parameters on the basis of a more rigorous solution and input them into a typical interaction analysis program. Obviously, separate sets of equivalent parameters are necessary to capture the system behavior in the small and large strain ranges. This approach was adopted in the present study.

2. CHARACTERISTICS OF THE HIGASHI-KOBE BRIDGE

The Higashi-Kobe Bridge has been designed[6)] by the Hanshin Expressway Corporation. The consultant services of Sogo Engineering Inc., Osaka, have been subcontracted to perform seismic analysis. Figure 2 presents a scheme of the eastern half of the bridge, which has three spans and a total length of 885 m. The center span measures 485 meters and the side spans are 200m each. The bridge piers at the east side are named P24, P25 and P26. Accelerometers are placed on three location on the tower at P24. They are designated by T1, T2 and T3 in Figure 2. The soil response during earthquakes is recorded by accelerometers buried in the ground at depths 34 (G1 in Figure 2) and 1.5 meters (G2 in Figure 2) at a 50 m distance from the foundation at P24. An accelerometer is also installed at the bottom of the caisson (K1 in Figure 2).

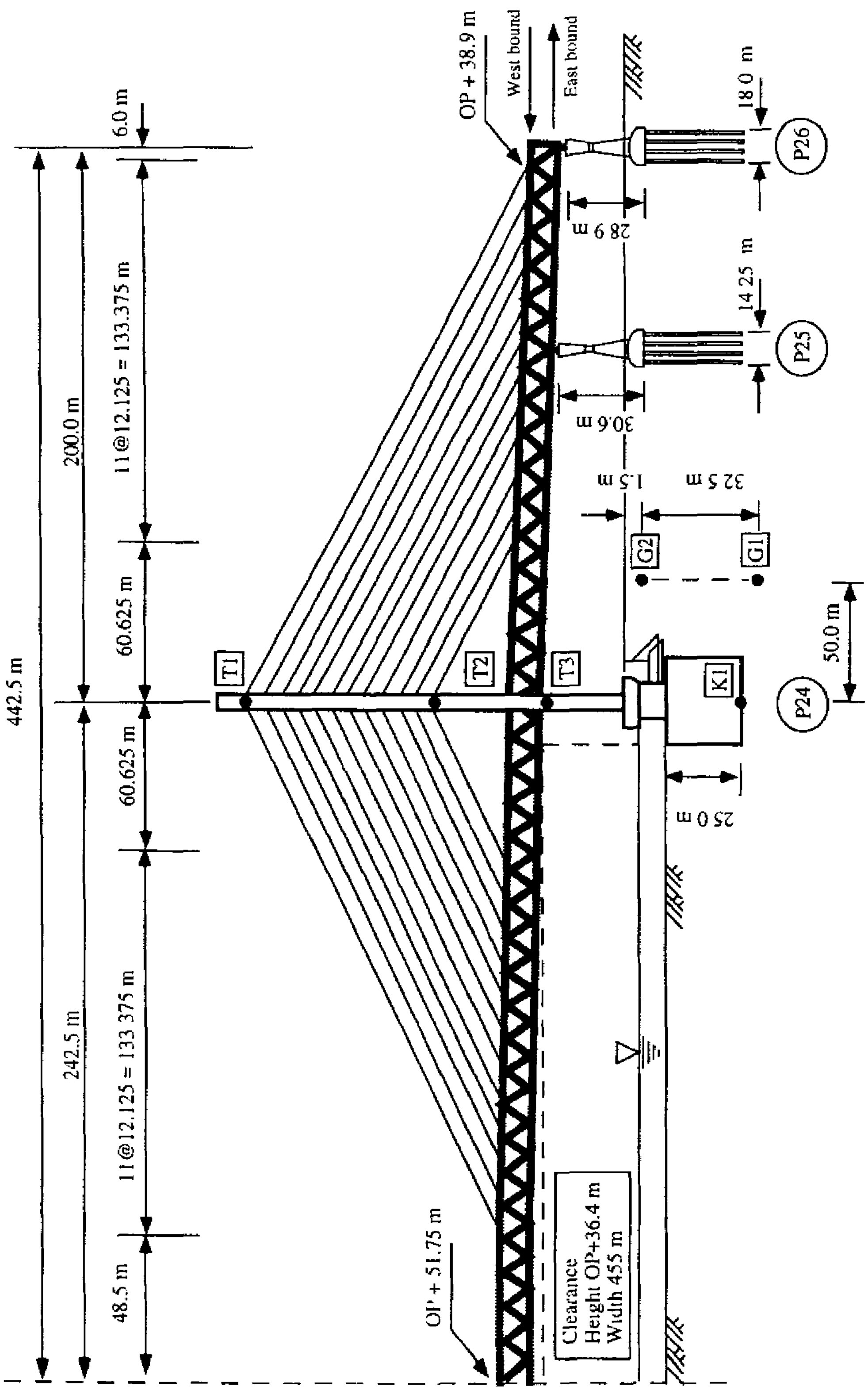

Figure 2. Scheme of the bridge and locations of the seismometers

The specifications of the bridge and its components are summarized in Table 1. The steel deck is an orthotropic slab structure with deck plates of thickness 12mm, reinforced with longitudinal ribs at 70 centimeter intervals and transverse ribs at 3 m intervals. Its cross-section is shown in Figure 3.

TABLE 1. SPECIFICATIONS OF THE HIGASHI-KOBE BRIDGE

Type	Three-span continuous steel cable-stayed bridge
Road category	Group 2, Class 1 (Japan Road Association Specifications[7])
Design velocity	80 km/h
Roadway	2 decks × 3 lanes
Length	200 + 485 + 200 = 885 m
Width	13.5 m × 2 decks
Tower	H-shaped tower (146.5 m)
Main girder	Warren truss (height 9 m)
Cables	Harp pattern multi cable (12 cables in a plane)
Substructure	Caisson foundation (for towers) Pile foundation(for piers)
Weight of the superstructure	Main girder 141, 000 kN Towers 79, 000 kN Cables 13, 000 kN Piers 17, 000 kN Others 24, 000 kN Total: 274, 000 kN
Specifications of the caissons	Weight of the steel shells 9500 × 2 = 19, 000 kN Volume of the concrete 15, 300 × 2 = 30, 600 m^3 Weight of the reinforcing bars 13, 000 × 2 = 26, 000 kN

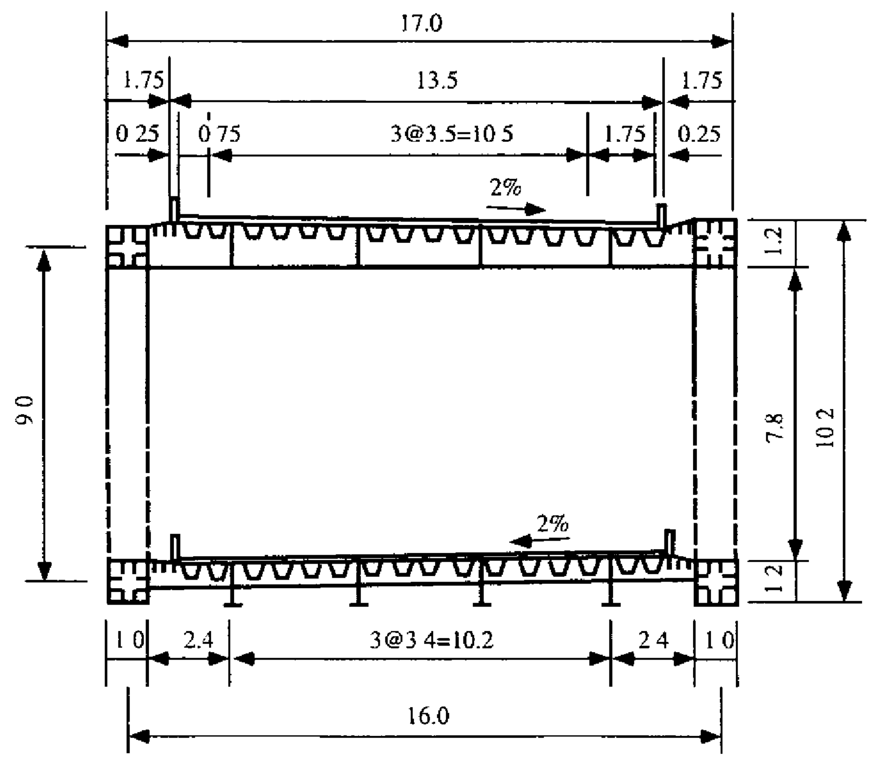

Figure 3. Cross-section of the main girder (dimensions in m)

An outstanding feature of the Higashi-Kobè Bridge is that the main girder can move longitudinally on all supports with the corresponding displacements restricted mainly by the cables. This provides an unusually long for such structures predominant period of sway mode oscillation-approximately 4.4 seconds. The largest values of design earthquake response spectra, prescribed by the standard code or formulated specifically for other large bridges in Japan, fall typically in the range from 0.2 to 2.5s[7]. Consequently, placing the natural period of the Higashi-Kobe Bridge outside this range contributes to seismic safety and enables economical and elegant design of the bridge towers. Wind shoes installed on the towers and the piers are designed in such a way, that the longitudinal motion of the main girder is not inhibited by the friction between the shoe components when the bridge is subjected to diagonal forces. Eye-bar pendulum supports are used for the side span piers to reduce the center span deflection by live loads.

The number of the cables (96 in total) is sufficiently large to account for low tension in each of them and this allows the use of small cable anchors, which are encased in the truss chord members. Each cable consists of 241 to 301 wires of diameter 7mm. They are clad in polyethylene tubes for protection from corrosion. Since wind and rain induced vibration have been reported on other bridges with polyethylene-covered cables, an unique measure has been taken to alleviate this problem for the Higashi-Kobe Bridge. After numerous experiments in wind tunnels it has been established that attaching protuberances parallel to the cable axis would control the vibrations by directing the rain drops in rivulets down the cable cover.

The bridge towers are H-shaped and have heights of 146.5 m. The cross-section of each tower column measures 3.5×6.5 meters at the base and 3.5× 4.5 meters at the top. The horizontal distance between the two columns is 24 meters. A scheme of the tower at bridge pier P24 is presented in Figure 4. The columns are constructed using hollow steel elements with rectangular cross-section, which are joined by welding. High-strength bolt joints are used at five locations per column to correct uneven shrinkage due to welding.

The tower foundations are pneumatic caissons of size 35 (W) × 32(L) × 26.5 (H) meters. A vertical and two horizontal cross-sections of the foundation at P24 are shown in Figure 5. The inside of each caisson is divided into 6 rows of 6 cells with partition walls. The foundations have been constructed by the floating caisson method. The steel shells of the caisson have been connected with the reinforcing bars and used as mold for the concrete foundation. The secondary piers at the side spans are founded on piles.

Numerous tests have been performed by the Hanshin Expressway Corporation to evaluate the properties of the soil around the foundations. Figure 6 shows a model of the soil profile at P24, which reflects the results of the comprehensive geotechnical investigations. The soil structure is very complicated and consists of interlacing layers of gravel, sand and clay with substantially varying characteristics.

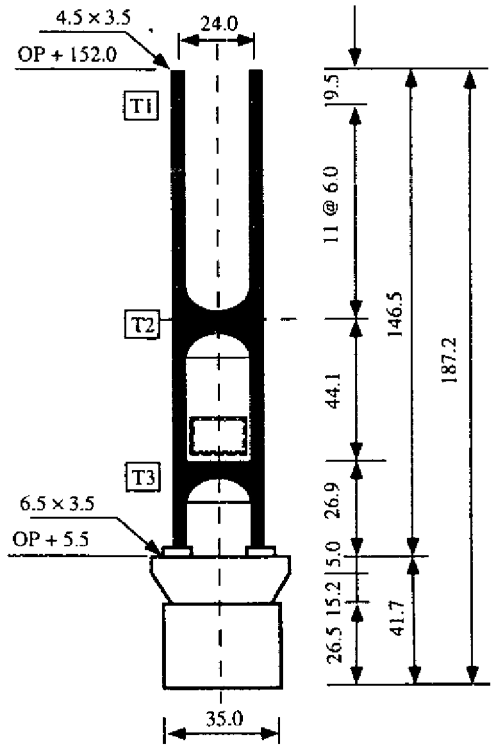

Figure 4. Scheme of the tower at P24 (dimensions in m)

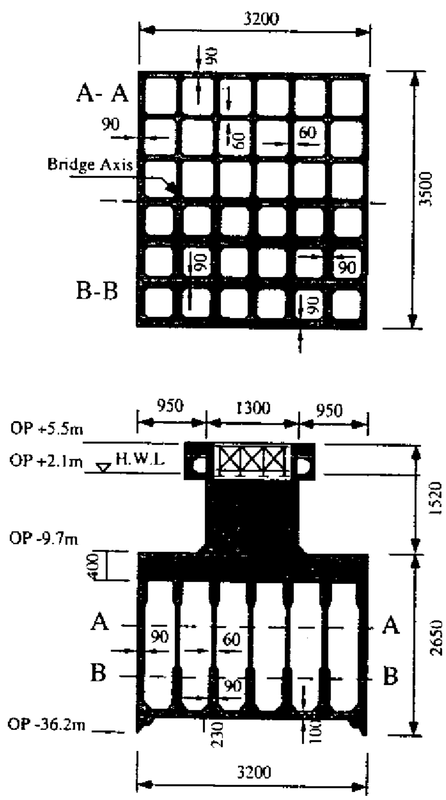

Figure 5. Horizontal and vertical cross-sections of the foundation at P24 (dimensions in cm)

Ordinate from OP (m)	Soil Type	Layer thickness (m)	SPT N value	Unit weight γ (kN/m^3)	Poisson ratio ν	Cohesion c (kN/m^2)	Internal friction angle ϕ (degrees)	Shear modulus G (kN/m^2)	Shear wave velocity (m/s)	Grain size $D50$ (mm)	Fine contents (%)
5.31	1	7.00	10	18.0	0.48	0.0	36.0	47020	160.0	2.20	9.0
-1.69	1	6.60	15	18.0	0.44	0.0	36.0	154470	290.0	2.20	9.0
-8.29	1	10.20	20	19.0	0.49	0.0	32.0	154470	290.0	2.70	5.0
-18.49	1	3.15	17	19.5	0.48	0.0	31.0	198530	320.0	1.50	10.0
-21.64	3	3.70	43	19.5	0.47	0.0	41.0	272400	370.0	2.50	12.0
-25.34	2	3.75	29	19.5	0.49	23.2	3.0	114610	240.0	0.03	80.0
-29.09	3	2.55	50	19.5	0.48	0.0	42.0	230020	340.0	2.10	13.0
-31.64	2	2.10	18	19.5	0.49	14.4	3.0	87750	210.0	0.10	57.0
-33.74	3	3.65	50	19.5	0.49	5.0	40.0	334480	410.0	0.79	12.0
-37.39											

Soil types : 1 Alluvial sands ; 2 Diluvial clay ; 3 Diluvial sand

Figure 6. Soil model at P24