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SEISMIC ANALYSIS AND DESIGN OF LONG SPAN CONTINUOUS BRIDGE WITH EMPHASIS ON SOIL-STRUCTURE INTERACTION

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SUMMARY

This paper presents the seismic design and analysis taken for bridges on a part of the Tomei Highway with the emphasis on the soil structure interaction, although the attention is paid to the 3-span continuous cable stayed Tomei Ashigara bridge. The items focussed are the soil spring and damping evaluation, the foundation input motion, and the method of interaction analysis. The structural safety is investigated against a possible "Tokai Earthquake".

INTRODUCTION

Aseismic design of bridge structures in Japan is regulated by the Specifications For Highway Bridges (**SHB**)−**V** (Ref.1). The **SHB−V** classifies site soil conditions into four groups and prepares the response spectra accordingly, which enables to adopt the so-called modified seismic coefficient method for the design work. This method takes account of the fundamental vibration period of the concerned structure for evaluating seismic force in an equivalent static way. However, it lacks in detailing the soil-structure interaction (SSI), just recommending a deliberate consideration in view of the state-of-the-art.

Since most of the bridge structures are supported by deeply embedded foundations in soils, the SSI effect is most significant factor to be considered in design work. Following the procedure in the **SHB−V**, we evaluate the Winkler type soil spring coefficients through a simple formula for insitu data, the N-value, for instance, from the standard penetration test. The state-of-the-art on this regard, however, has remarkably progressed with the development of the numerical analysis techniques. Among them, the finite element method is amenable straight-forwardly to the computer code implementation which gives stable solutions to engineers for practical problems. The dynamic analysis formulation which introduces effective reduction of degrees of freedom for the SSI system, is also well established (Ref.2). Updating the current practice is strongly desired.

This paper reports the procedures taken for the seismic design of six long-span bridges in the section between Gotenba and Ohi-Matsuda intersections of the Tomei Highway in Japan (Fig.1). The presentation is however addressed to the Tomei Ashigara bridge illustrated in Fig.2 among others (Ref.3). This area is anticipating a big "Tokai Earthquake" in the near future.

SEISMIC DESIGN BASED ON RESPONSE SPECTRUM

Fig.3 is the design flow for the bridges dealt with herein. From the previous case studies it was found that the modified seismic coefficient method, which assumes a uniform seismic force distribution along the structural height, yields excessively large internal forces at the pier feet while excessively small values at the superstructure. In order to give a rational design to structural sections, the dynamic response analysis with use of the response spectrum is employed.

Concerning the soil spring evaluation, some preliminary computations have shown that the static FEM solution gives a few or up to several times larger values than those form the **SHB−V** formula. The primary reason seems to lie in the deformation coefficient used in them; the former is based on the P- and S-wave test at site while the latter on a specific loading test. Referring to the comparison of the field vibration tests and the FEM analyses for several case studies along with the

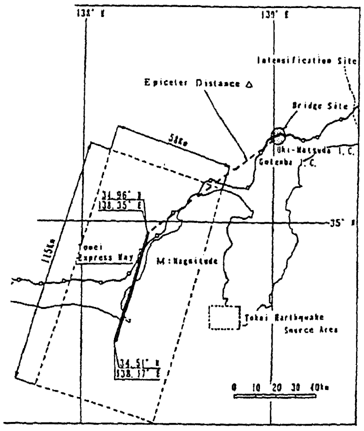


Fig.1 Bridge Site and Seismological Environment

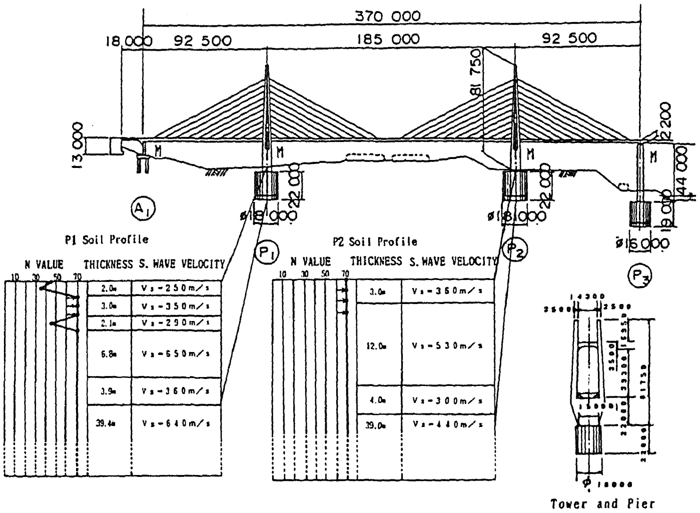


Fig.2 The Tomei Ashigara Bridge (Cable stayed bridge)

engineering judgement for uncertainties involved in determining the soil spring effect, we are forced to resort to take the band estimation across the FEM solution (standard value); as the upper bound the twice of the FEM solution and as the lower bound the smaller value of either a half of the FEM solution or the formula result from the N-value. Under this soil characteristic, the structural section should be designed against the most unconservative seismic force.

The state-of-the-practice for the seismic analysis is to adopt the discrete models for the whole structure, which derives the governing equation in the relative coordinates $\{U\}$ for a uniform base input motion $\ddot{U}_g$(acceleration), as

$$[\,M_{complete}\,]\{\ddot{U}\} + [\,C_{complete}\,]\,\{\dot{U}\} + [\,K_{complete}\,]\{\,U\} = -[\,M_{complete}\,]\{\gamma\}\ddot{U}_g \tag{1}$$

in which $[M_{complete}]$, $[C_{complete}]$ and $[K_{complete}]$ denote, respectively, the mass, damping and stiffness matrices; $\{\gamma\}$ stands for the vector whose elements consist of 1 or 0, to specify the horizontal input motion. For the classical normal modes analysis the damping effect assigned as in the inlet Table of Fig.3 is converted to the modal damping ratios on the strain energy proportional basis.

The dynamic design and response analysis are executed by use of the average response spectra in Fig. 4 which was derived from the analyses for the past 44 strong earthquake records observed in Japan (Ref.1). Although the soil condition at bridge site varies from the *1st* kind soil to the *2nd* kind in the classification of the **SHB−V** , herein the latter condition is used to make a unified design work for the bridges directly connected. The seismic coefficient at the site is multiplied to this response spectra to calculate the seismic load. Fig.5 gives the maximum response diagrams based on the rms modal superposition.

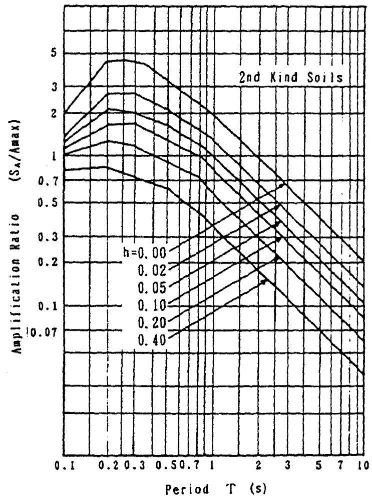
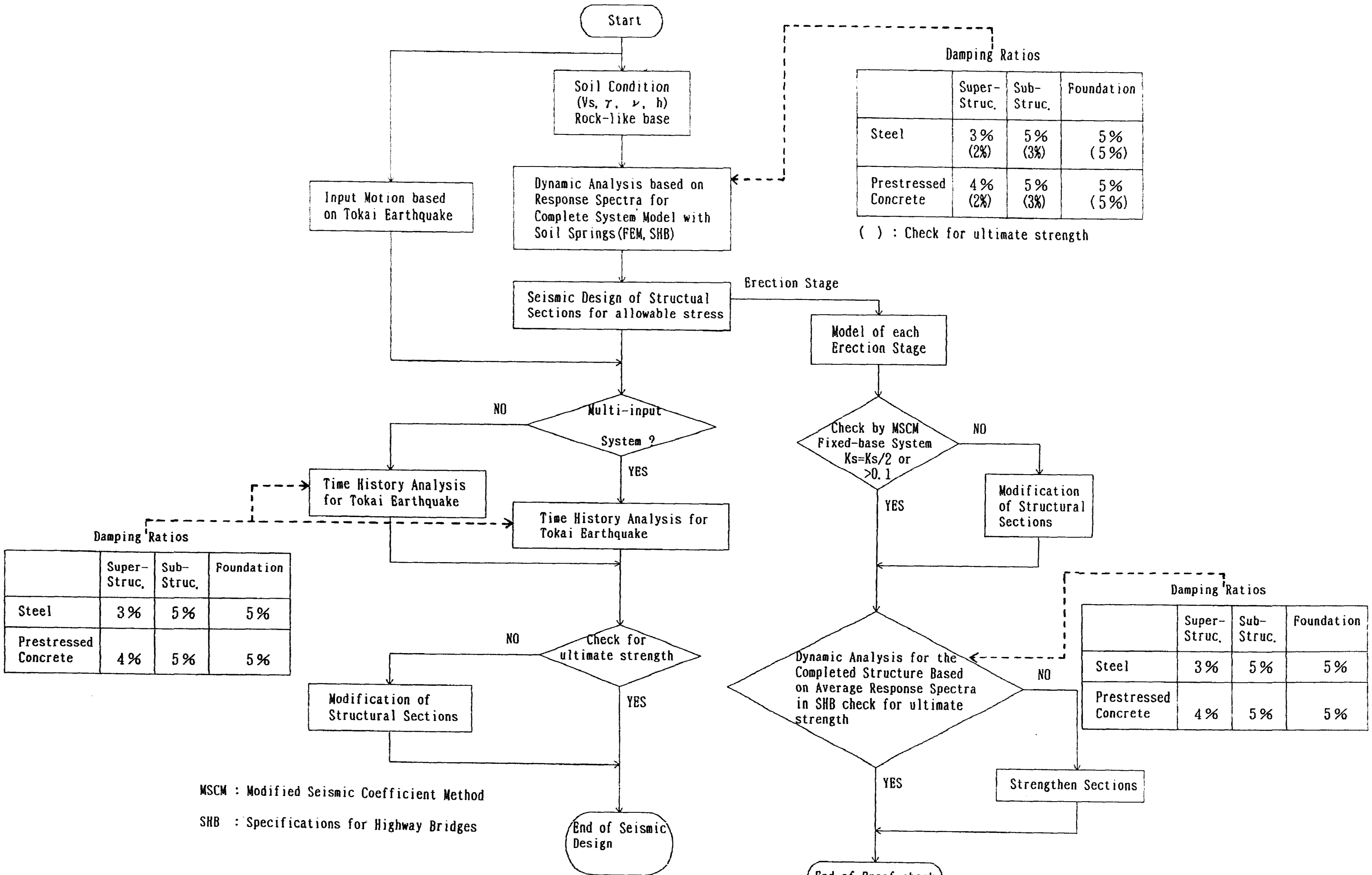


Fig.4 Average Response Spectra for Design Maximum Input Accel. $A_{max} = 200$ *gal*



Damping Ratios

	Super-Struc.	Sub-Struc.	Foundation
Steel	3% (2%)	5% (3%)	5% (5%)
Prestressed Concrete	4% (2%)	5% (3%)	5% (5%)

( ) : Check for ultimate strength

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MSCM : Modified Seismic Coefficient Method

SHB : Specifications for Highway Bridges

Fig.3 Flow for Design Work

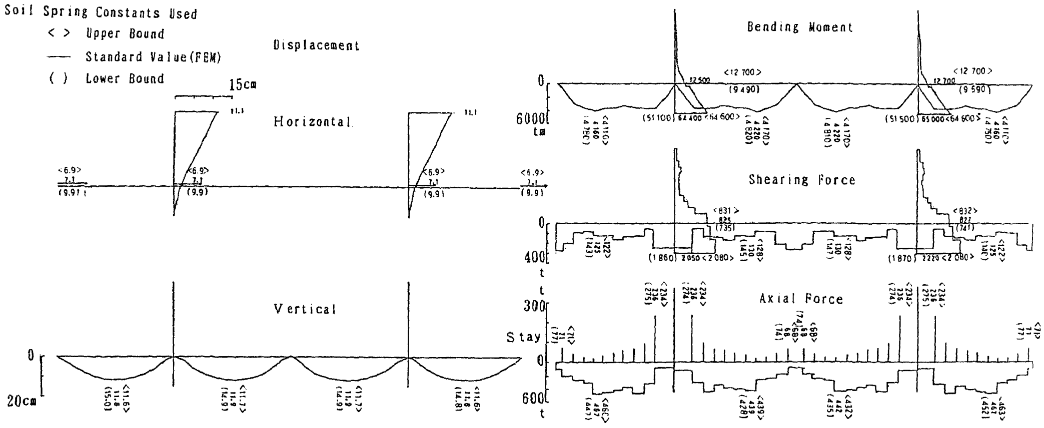


Fig.5 Maximum Response Diagrams

EARTHQUAKE SIMULATION AND TIME HISTORY ANALYSIS

Simulation of the rock-like base input motion is based on the seismological and geological inference in a probabilistic sense. The Public Works Research Center (**PWRC**) of Ministry of Construction proposes an attenuation formula regarding the response spectra to give a basic information on the input motion (Ref.4). Although the earthquake records of magnitudes more than 8 were not included in establishing the above formula, the extrapolation is made. Giving probable values to the parameters, we depict the response spectrum curve in Fig.6. As the soil condition, the average of the **I** and **II** kinds from the **PWRC** classification is comparable with the base assumption for the foregoing **SHB−V** response spectrum. For the simulation of ground motion as a time history, which can yield this response spectrum, first select an actual earthquake record of similar seismological and geological conditions from the past accelerograms. In this case, the EL CENTRO 1940, NS component was adopted. Then, adjust the record in the frequency domain to give a good fit to the above target spectrum. Fig.7(bottom figure) is thus obtained time history.

Time history response computation is carried out by the step-by-step integration from the complete system analysis (Eq.(1)) with use of the classical normal mode decomposition. The input motion in Fig.7(top figure), which is used for every foundation, is the filtered soil motion at the depth of foundation's gravity center, when the aforementioned base motion follows the one-dimensional shear wave propagation in the surface layered soils.

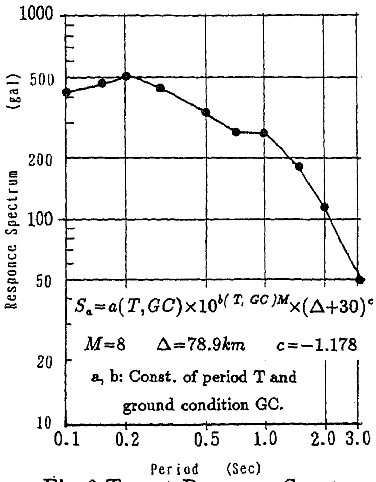


Fig.6 Target Response Spectrum
for the Tokai Earthquake

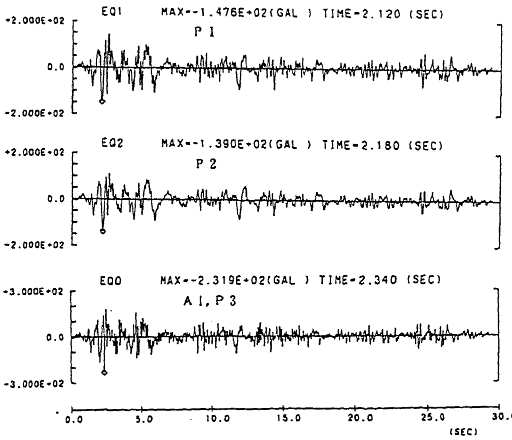


Fig.7 Simulation of Base Input Motion
and Foundation Input Motion
for Complete System Analysis

In view of the fact that the dynamic substructure method or the impedance method is rather a state-of-the-art, this approach is further adopted to check the result from the complete system analysis. The dynamic substructure formulation considers the coupling between the fixed base structural vibration modes and the frequency dependent soil impedance with introduction of the quasi-static displacement influence function on the structure when the foundation constraint is released. The effective approximate solution is attained by truncating the structural modes to superpose. Hence, the governing equation becomes

$$\begin{bmatrix} [K_{ss}] & [K_{bs}] \\ [K_{sb}] & ([K_{bb}]+[K^{*}_{subs}]) \end{bmatrix} \begin{Bmatrix} \{q\} \\ \{U_{subs}\} \end{Bmatrix} = \begin{Bmatrix} \{0\} \\ [K^{*}_{subs}]\{U^{*}_{subs}\} \end{Bmatrix} \tag{2}$$

in which $[K_{sb}]$'s stand for the partitioned dynamic stiffness matrices with the subscript "b" for soil-foundation interface and "s" for structure and they are obtained as

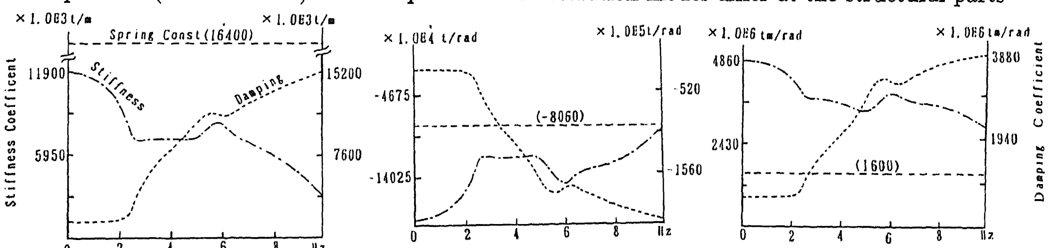
$$[K_{ss}] = -\omega^2 [I] + i\omega [\ \ 2\xi_i\, \omega_i\ \ ] + [\ \ \omega_i^2\ \ ]; \qquad [K_{sb}] = -\omega^2 [\phi]^T [M_{ss}] [\beta] = [K_{bs}]^T$$

$$[K_{bb}] = -\omega^2 \left([\beta]^T [M_{ss}] [\beta] + [M_{subs}] \right) + i\omega [\beta]^T [C_{ss}] [\beta] + [K_{sb}]^T [\beta]$$

in which ω_i's and ξ_i's are the modal frequencies and damping ratios, respectively, and $[\beta]$ is the modal matrix. Note that the soil spring and damping effect, which are calculated from the real and imaginary part of the complex soil impedance $[K^{*}]_{subs}$, respectively, are frequency dependent and also the foundation input motion $\{U^{*}\}_{subs}$ which is through the kinematic interaction with soils. Due to this nature, the coupled motion is first solved in the frequency domain and then transformed into the time domain. Note that the rotational component is also included besides the translational one in the input motion vector in contrast to the only horizontal input motion in Eq.(1).

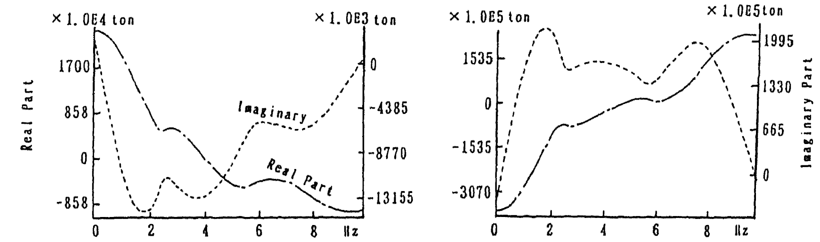
The soil-foundation system is analyzed based on the axisymmetric three-dimensional FEM model with the transmitting side boundary and an approximate halfspace base boundary set at deep enough (Ref.5), for the proper azimuthal orders. Fig.8 shows the impedance functions associated with the foundation's degree of freedom' for the structural behavior along the bridge axis. We observe that a significant variation of soil stiffness and the damping with the frequency. The constant spring coefficients (standard values) used for the design are also indicated by the dashed lines for comparison, which confirms the appropriateness of such procedure. Fig.9 depicts the effective foundation input forces. Some appreciable difference is noted to exist for the impedance functions and the effective input forces between foundations P1 and P2 (not shown here).

The vibration modes of the structure are noted that some are modified by the soil effect while others remain almost unchanged with no SSI. The frequency response functions from Eq.(2) gives a clear picture (omitted here) that the predominant vibration modes differ at the structural parts



(a)Horizontal (b)Coupling (c)Rotational

Fig.8 Soil Impedance Functions



(a)Horizontal (b)Rotational

Fig.9 Effective P1-Foundation Input Forces for Substructure Analysis addressed. This fact suggests that the modified seismic coefficient method in the **SHB−V** may no longer be appropriate for the design; rather, it claims that the dynamic response analysis should be used for the bridges with tall towers/piers.

Fig.10 indicates the maximum response diagrams. The dynamic substructure method takes the multiple input system analysis for the spacial variation of the foundation input motions due to the difference of soil condition and/or seismic wave propagation. The comparison with the results from the conventional complete system analysis reveals that the rigorous substructure solution results in slightly larger responses. This is mainly due to the rotational input motion in the substructure method. We must mention that the rotational input motion is discarded in the conventional complete system analysis. If we superpose this effect to the values in the conventional response computation due to the horizontal input motion, the greater responses tend to be attained than those by the substructure method. Since we observe that the time history response at some sections exceeds the design values, we confirmed the safety of the bridge from the ultimate strength analysis.

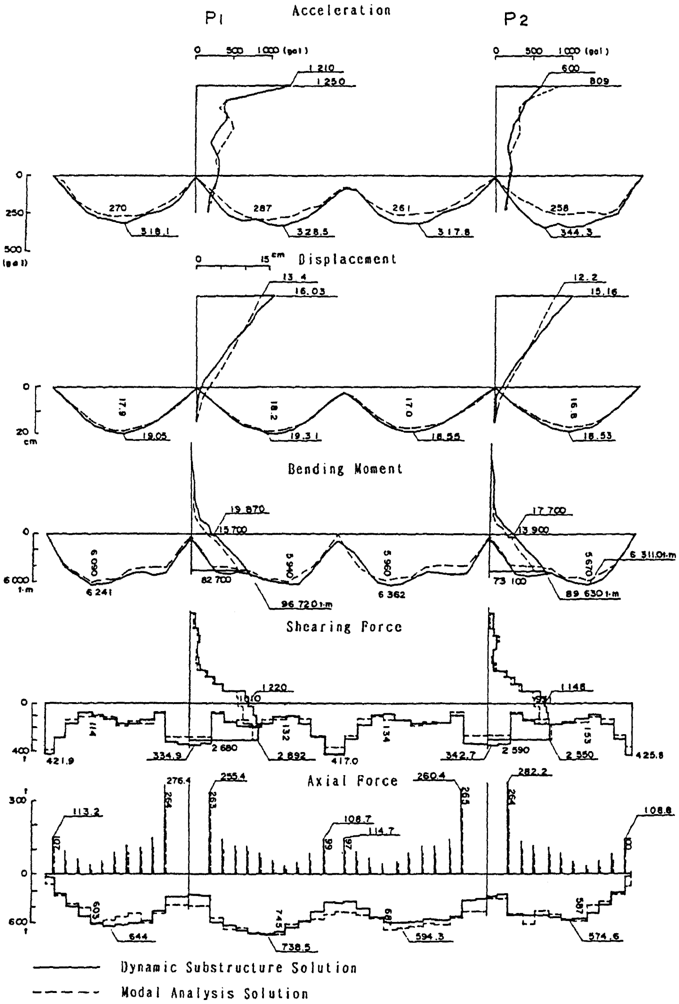


Fig.10 Maximum Response Diagrams

CONCLUSION

Through this study, we may conclude that for the rational seismic design of long-span bridges with tall towers/piers the dynamic response analysis is recommended. The spring constant evaluation proposed here at the design phase is substantiated by the succeeding dynamic substructure method for the detailed analysis. This method is effective, when the soil-structure interaction is concerned, for other foundations like group piles. The conventional complete system modeling and its modal method results in giving slightly smaller response values than those by the substructure technique, since the latter takes account of the rotational foundation input besides the horizontal input while the former the horizontal input only. The structural safety against a big "Tokai Earthquake" is confirmed from the ultimate strength analysis by a good margin.

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