(119) NONLINEAR STRUCTURAL RESPONSE OF A DYWIDAG BRIDGE PIER

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SUMMARY

This research was initiated after a couple of bridge piers failed due to an earthquake in Miyazaki. The main objective was to carryout an experimental evaluation of such failures. A Dywidag bridge pier model was constructed and tested. The results showed a strong nonlinear response and a softening of the hard spring under increased periodic loading. Analytical interpretations became necessary, because of the inability to evaluate the dynamic material properties experimentally, and a need for the explanation of the nonlinear failure phenomenon. The loss of ductility and reserve capacity with an increase in acceleration is verified by the experiment and the analytical model. This renders the pier inserviceable, though the structure did not collapse. The procedure used for system identification is equivalent linearization, but it is being carried forward to accomodate a nonlinear programming optimization model.

INTRODUCTION

Earthquakes are known to cause severe structural damage and it is generally accepted that the design of a structure which would allow an elastic response would be economically unrealistic. A more realistic design would permit yielding, resulting in energy absorption or dissipation; and because of this the structure is better able to withstand severe earthquake excitations. To predict this response, a mathematical model must incorporate some of the nonlinear parameters. This makes the construction of the mathematical model more difficult than, for the linear model.

Laboratory tests, using either a shaking table response or an harmonic oscillator response, enable the evaluation of the physical properties of the model. The measured inelastic response quantities (time history of accelerations, displacements and strains) can then be compared with the computed response quantities, for mathematical models subjected to equal excitations.

If the correlation between the measured and computed response are within acceptable limits the model is considered adequate, otherwise it must be modified. The specimen chosen to solve the problem of the bridge piers reflect our attitude toward such research. We feel strongly that complicated topics should begin with simple models to obtain the insight into the actual problem.

TESTS

The primary requirements for the test structure were essentially a single degree-of-

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freedom model (SDOF) and that it exhibit a very simple hysteretic energy absorbing behaviour. A Dywidag T-pier model made of reinforced concrete was constructed (Fig. 1) for an oscillator response evaluation. The model is $2.80m$ high, top to bottom, with a circular footing of $1.60m$ diameter and $0.55m$ depth; the top cantilevers were $2.00m$ either side and tapered. Rapid hardening cement concrete with a 3 day compressive strength of 234 kg/cm^2 was used in the construction. A total of 12 strain gauges $S1$ to $S12$ were attached to the main reinforcement at the base of the column for direct strain measurement.

The Dywidag test specimen was subjected to nine increasing periodic loadings and frequency response (accelerations, displacements and strains) was measured at the first resonant frequency and at 0.1 Hz intervals. After the testing was completed frequency reponse curves were drawn (Fig. 2) for the test loadings. These frequency response curves show a continuous increase in the displacement and acceleration response uptill periodic loading P6 and thereafter a reduction in the maximum displacement and acceleration response. The response curves indicate a strong hard spring effect and a progressive softening of this spring with increments in the amplitude of the cyclic loading.

We can conclude that the displacement response increased till the ultimate design capacity of the specimen was reached. The softening of the hard spring is due to crack propagation in concrete and increases with increase in micro cracks till P6 is reached. The succeeding response curves after P6 show an irregular shape and a reduced acceleration and displacement response. This could be possibly due to the cracks closing on one side of the column. The pinched or irregular response shape is because of the separation of concrete and steel and a possible effect of other modes becoming predominant.

METHODS AND COMPARISONS

The Dywidag T-pier was first modeled as a SDOF system with the girder mass and 0.234 of the pier mass lumped at the top of the pier spring (see Fig. 3). The strains and stresses from the SDOF model were on the lower side showing flexibility for higher periodic loads. Therefore a multi degree-of-freedom (MDOF) model became necessary to calculate the stresses and strains more accurately and a possible insight into the crack pattern. The 16 element FEM model (Fig. 3) was applied with an excitation equal to the actual periodic loading acceleration. An examination of the mode shapes (Fig. 4) shows the first two modes to be bending modes and the third one as a twisting mode, at a frequency close to 20 Hz.

The first mode at the experimental natural frequency resulted in a new value for the modulus of elasticity which was used in the FEM analysis. This stiffness, the first natural frequency and the acceleration excitation produced the needed dynamic stresses. The dynamic strains were then easy to calculate and compared with the direct strain measurements.

The FEM models were classified into three types (i) a constant variation of the modulus of elasticity, E along the pier (ii) a variation of E along the two bottom elements only (iii) a variation of E along the base element only. The first model's results were close to the experimental results for strains and actual crack patterns. Whereas the other two models resulted in much higher strains indicating the possibility of excessive crushing of concrete near the hinge.

The values of pier stiffness were directly calculated from the measured first resonant frequency and the mass. Then the maximum bending moment produced by the inertial force was calculated, and so were the dynamic bending stress and strain. This was then compared with the experimental strains.

The variation in the modulus of elasticity was also compared for the three types of the FEM models and the SDOF model (see Fig. 5). The SDOF model, and the FEM models (type 2 and 3) are unable to predict a realistic value. In case of SDOF model the value for E is on the lower side with high predicted strains. The other two FEM types also overestimate the strains, due to an apparent crushing of concrete. Case 1 of the FEM gives realistic values of strains, stresses and modulus of elasticity. A large crack opening at the P7 load reduces the displacement response and the strains. The structure at this stage, had essentially failed, although a collapse did not occur. In all cases, serviceability and not a total collapse is a definite code requirement, and excessive ductility and cracking could be a failure state. These models were applied with the oscillator load and the resulting accelerations calculated. Although the load varied between 29.0 kg. and 162.0 kg. it produced accelerations between 329.0 gal and 1600.0 gal for P1 to P6 respectively.

The damping ratio for the SDOF was calculated from the amplification factors and compared with the measured value of the damping ratio ζ from the half-power method. The first mode was also used in a reevaluation of the damping ratio, in Case 1 of the FEM for each harmonic load and a displacement equal to the measured displacement. The Figure 6 shows the damping ratios from three different methods with the highest values from the experiment and the analytical procedure predicting close damping ratio ζ values. This strengthens our conclusion of increased micro cracking, and an increased damping from P1 to P4. The damping ratio for P6 dips suddenly due to a crack opening and a possible start of the separation of concrete from the reinforcement, but an increased damping ratio is indicated prior to the structural failure.

CONCLUSIONS

The inclusion of material nonlinearity cannot be over looked in a dynamic response of ductile structures. The Dywidag pier acted as a hard spring for each load step earlier in the loading history and then gradually softened with increased loading. This seemed typical of reinforced concrete, the softening action being due to cracking of concrete, with the steel following a Ramberg-Osgood stiffness model. A comparison of strains reveal a smeared (uniform) crack propagation mode of the pier column only and this is confirmed from the experimental test. The frequency response calculations of a test structure could be used in designing economical and durable earthquake resistant structures.

REFERENCES

[1] W. D. Iwan, "A Distributed-Element Model For Hysteresis And Its Steady-State Response," Transactions of the ASME, Dec. 1966.

[2] H. Tsutsumi, et. al "A Consideration On Earthquake Resistance Of Bridge Piers With Various Types Of Foundations," 9WCEE, Tokyo, 1988.

[3] H. D. McNiven and V. C. Matzen, "A Mathematical Model To Predict The Inelastic Response Of A Steel Frame," Earthquake Engineering and Structural Dynamics, Vol. 6, 203-219 (1978).

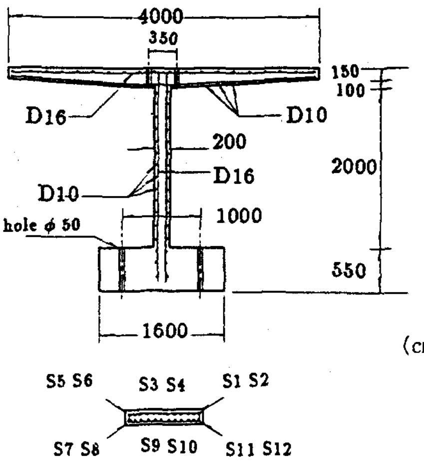
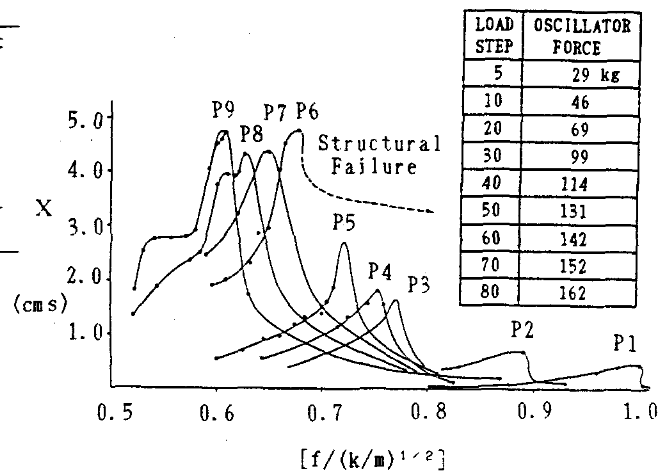


Fig. 1 Experimental Model



LOAD STEP	OSCILLATOR FORCE
5	29 kg
10	46
20	69
30	99
40	114
50	131
60	142
70	152
80	162

Fig. 2 Response Curves

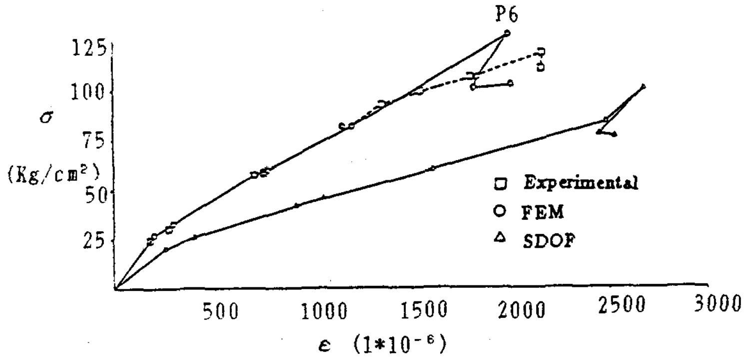


Fig. 5 Stress Vs Strain

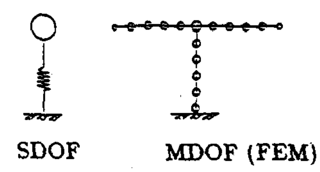


Fig. 3 Mathematical Models

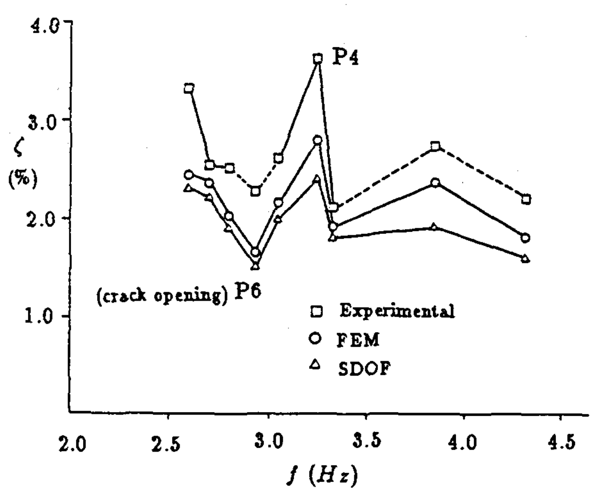


Fig. 6 Damping Ratio Vs Freqnency

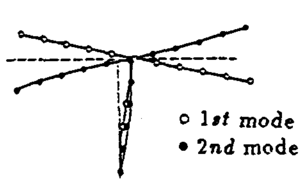


Fig. 4 Mode Shapes