13 Mar 1991, 1:30 pm - 3:30 pm

Response of Frame Foundations to Vertical Vibrations

P. J. Moore
University of Melbourne, Australia

T. P. Tan
University of Melbourne, Australia

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. 11.24

Response of Frame Foundations to Vertical Vibrations

.J. Moore
eader in Civil Engineering, University of Melbourne, Australia

T.P. Tan
Former Graduate Student, University of Melbourne, Australia

SYNOPSIS: A series of vertical vibration tests was carried out on small 1m span steel portal frames of different stiffnesses and supported on two isolated rigid circular footings on a compacted sand deposit. Dynamic loading was applied to the centre of the frame and the response was monitored by means of six accelerometers attached at various locations. The resonant frequency and the maximum amplitude of vibration were observed to be dependent on the frame stiffness and two theoretical models, the combined method and the dynamic deformation method, were used to compare calculated and observed results.

The combined method did not yield a good prediction of the natural frequencies of the frames tested, the theoretical values being significantly less that those observed. Much better but not close, agreement between calculated and observed values was obtained for natural frequency and vibration amplitude with the dynamic deformation method.

INTRODUCTION

Frame foundations are widely used for the support of machines, such as turbines, which have a considerable amount of auxiliary equipment and interconnecting pipework, ducting, etc. Traditionally the frame was made of relatively massive concrete members, but increasingly the use of more slender steel members is being explored with the expectation that a more economical design will result. In the design of these steel frames, doubts arise regarding the applicability of the semi-empirical approaches that have developed for the design of more massive concrete frames. These approaches to design have been described by Barkan (1962) and Major (1980). The experimental program described in this paper was devised in order to gain an improved understanding of the dynamic behaviour of relatively slender frames.

THEORETICAL EVALUATIONS OF NATURAL FREQUENCY AND VIBRATION AMPLITUDE

Natural frequencies of frame foundations are affected by many factors such as component dimensions and elastic moduli, foundation soil rigidity and degrees of freedom, but it is not normally practicable in routine design to assess the combined effects of these factors on the natural frequency of the foundation system. A more convenient approach is to determine the natural vibration frequencies on the basis of either the frame elements alone or the elasticity of the base under the foundation. Major (1980), in the so called combined method, suggests that the natural frequency should be calculated by considering the stiffnesses of the frame elements and the foundation material in the one calculation. For a single degree of freedom analysis, the natural frequency (f_n) may be calculated from

$$f_n = 0.5/\left(\delta_1+\delta_2+\delta_3+\delta_4\right)^{\frac{1}{2}} \quad \text{hertz} \qquad (1)$$

where
δ_1 = deflection due to bending (m)
δ_2 = deflection due to shear (m)
δ_3 = deflection due to compression of columns (m)
δ_4 = compression of the ground under the total weight of the frame and foundation.

For single bay frames with fixed columns these deflections have been given by Major as

$$\delta_1 = \frac{Pl^3}{96E\ I_b}\left(\frac{2k+1}{k+2}\right) + \frac{Ql^3}{384E\ I_b}\left(\frac{5k+2}{k+2}\right) \qquad (2)$$

$$\delta_2 = \frac{3l}{5\ E\ A_b}\left(P + \frac{Q}{2}\right) \qquad (3)$$

$$\delta_3 = \frac{h}{E\ A_c}\left(N + \frac{P+Q}{2}\right) \qquad (4)$$

$$\delta_4 = \left(\frac{P+Q+2N+Q_A}{BL\ C_z}\right) \qquad (5)$$

where
E = Young's modulus (MPa)
A_b = cross-sectional area of beam (m^2)
A_c = cross-sectional area of column (m^2)
I_b = second moment of area of beam (m^4)
I_c = second moment of area of column (m^4)
P = weight of any machine concentrated at centre of beam (MN)
Q = weight of beam and distributed load on it (MN)
h = frame height (m)
l = frame span (m)

N = load transmitted from any longitudinal frame plus weight of upper third of column (MN)
k = frame constant = $(I_b H/I_c l)$
Q_A = Combined weight of the lower two thirds of the two columns and that of the slab (MN)
B,L = dimensions of foundation slab (m)
C_z = Barkan's coefficient of elastic uniform compression (MN/m^3)

For the evaluation of vibration amplitude, Barkan's (1962) so called amplitude method may be used. As described in detail by Major (1980) the weights acting on the beam and columns are evaluated by means of empirical expressions. The frame is treated as a two degree of freedom mass-spring system, the amplitude being determined from the solution of the appropriate differential equations.

An improved and less empirical method of analysis for the evaluation of resonant frequency and vibration amplitude is Kohoutek's dynamic deformation method (1985). This is based on the development of a method of dynamic analysis of frames by Kolousek (1973). This development was a modification of the static slope deflection method for beams in which moments and shears were expressed in terms of frequency functions. Kohoutek modified Kolousek's solution by using exponential instead of hyperbolic functions in the general solution. Both Kolousek and Kohoutek have provided tabulations of all the frequency functions that are required for the solutions of problems involving dynamic loading of structures.

For a given number of degrees of freedom the equilibrium equations for a single bay frame in free vibration may be written in matrix form as follows

$$\mathbf{A}\,\mathbf{U} = 0 \qquad (6)$$

where $\mathbf{A}$ is the stiffness matrix, the elements of which are expressed in terms of frequency functions; and $\mathbf{U}$ is the deformation vector. Natural frequencies are determined by evaluating the frequencies at which the frame stiffness is zero. This corresponds to finding the roots of the determinant of the stiffness matrix. That is

$$\mathbf{A} = 0 \qquad (7)$$

The maximum vibration amplitude at the midspan of the cross beam of the frame may be calculated following substitution of the natural frequency into the equilibrium equations as described in detail by Kolousek (1973).

EXPERIMENTAL ARRANGEMENT

Vibration measurements were carried out on frames placed on a prepared bed of dry sand. The uniform sand (1mm - 3mm) was prepared by pluvial compaction in a steel tank 2.8m x 1.8m x 1.2m high. Pluvial compaction involved raining the sand from an overhead hopper through a diffuser system on to the rising surface of the prepared sand (see Jacobson (1976)). The physical properties of the sand are summarised in Table 1.

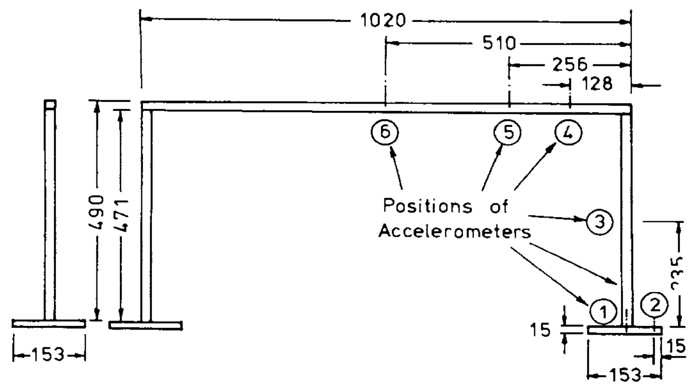

Fig. 1 Steel Frame Dimensions in mm.

TABLE 1: SAND PROPERTIES

Average density	1.74 t/m^3
Average grain size	1.5mm
Friction angle	42°
Youngs Modulus	110MPa
Maximum density	1.78 t/m^3
Minimum density	1.69 t/m^3

Tests were carried out with five steel frames manufactured from square section members. These frames were made with the same span and height (Fig. 1). Section properties of the frames are given in Table 2. The frames were bolted to rigid 300mm diameter footings and positioned on the sand surface.

TABLE 2: SECTION PROPERTIES OF FRAMES

Frame	Cross Sectional Area (mm^2)	Second Moment of Area (I) (cm^4)	Mass per Unit Length (kg/m)
A	144	0.17	1.13
B	196	0.32	1.54
C	256	0.55	2.01
D	361	1.09	2.83
E	625	3.26	4.91

Frame and footing vibrations were monitored by means of six B & K accelerometers, Types 4381 and 4370, with a frequency range from 0.2Hz to 6kHz. The positions of the accelerometers are shown in Fig. 1. The signals from the accelerometers were passed through a switch box then to a B & K Charge Amplifier Type 2635. The frame and footing responses were recorded via a Scientific Atlanta SD375 Spectrum Analyser II. The layout of the instrumentation is illustrated in Fig. 2. The vibration was generated by means of a Ling Dynamics Systems Model 409 Shaker which was mounted on a trunnion support. The trunnion was attached to a stiff overhead beam as shown in Fig. 2. The shaker was driven by a signal generator via a power amplifier. The signal generator used was a Hewlett Packard Oscillator Model 200CD. The dynamic load, which was applied to the centre of the frame cross beam, was monitored by means of an Interface Load Cell Model SSM-AS-100, with a capacity of 0.45kN.

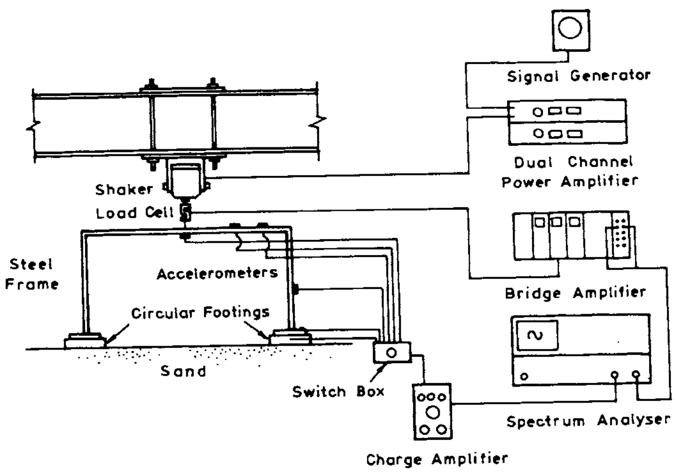

Fig.2 Layout of Instrumentation

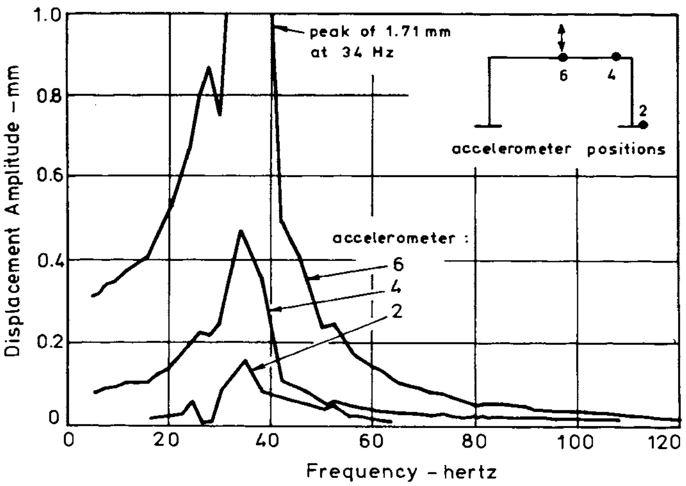

Fig.3 Typical Response Curves - Frame A

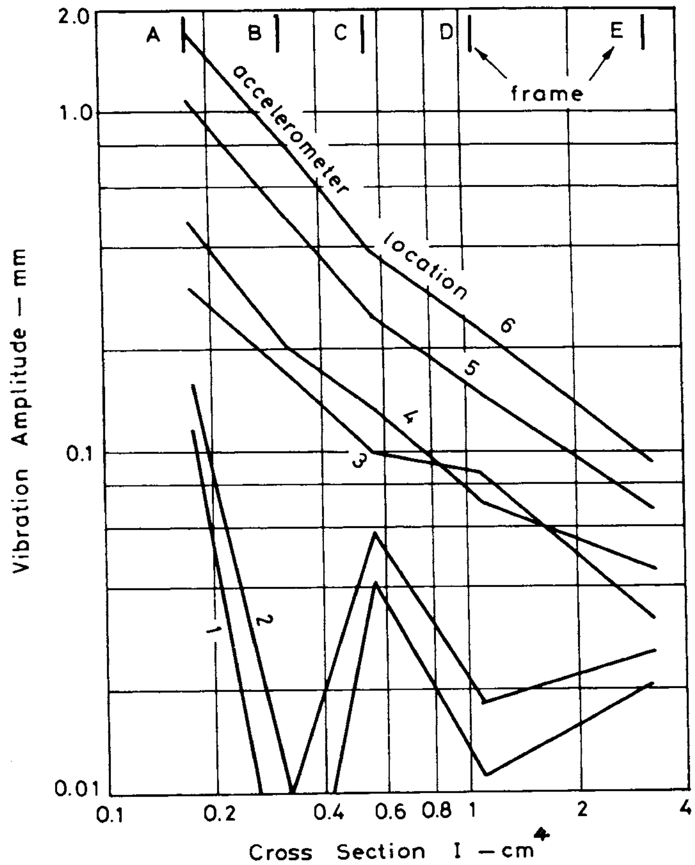

Fig.4 Observed Amplitudes at Resonance

OBSERVED FRAME RESPONSE

The vibrations were observed by means of the six accelerometers over the test frequency range from 5Hz to 130Hz. As would be expected the maximum amplitudes were observed on accelerometer No. 6. At resonance, the vibration amplitudes peaked at all accelerometer positions. Accelerometer No. 3 monitored horizontal vibrations, the other five monitoring vertical vibrations. Accelerometers 1 and 2 monitored the footing response, the observations confirming that the footing experienced both vertical and rocking modes of vibration. Typical response curves are shown in Fig. 3. The observed amplitudes of vibration at resonance on all accelerometers and for all frames tested are summarised in Fig. 4. This shows, as would be expected, that the vibration amplitudes for locations on the frames above the footings, decrease as the frame becomes stiffer. The observed resonant frequencies at the centre of the crossbeam are plotted in Fig. 5 for all frames. This confirms the anticipated trend, namely that the resonant frequency increases as the frame stiffness increases.

COMPARISONS WITH CALCULATED FRAME RESPONSE

The natural frequencies calculated by means of the combined method compare quite unfavorably with the measured values. As shown in Fig. 5 the calculated values significantly underestimate those observed for all frames tested. These calculated natural frequencies were not affected significantly by the response of the footings. The natural frequencies of the footings for the frames tested were quite high and outside the range of observed frequencies in the tests.

As previously mentioned in relation to the dynamic deformation method, the frame stiffness must be calculated for a range of frequencies in order to identify the natural frequencies. For this calculation the frame stiffness is the determinant of matrix **A** and natural frequencies correspond to the points where the frame stiffness passes through zero. This is illustrated for a three degree of freedom analysis for frame A in Fig. 6, where the first four natural frequencies may be identified. The first natural frequency for each frame has been plotted in Fig. 5. While providing better agreement with observations compared with the combined method, the dynamic deformation method does not yield calculated natural frequencies in close agreement with observed values, with the exception of frame C.

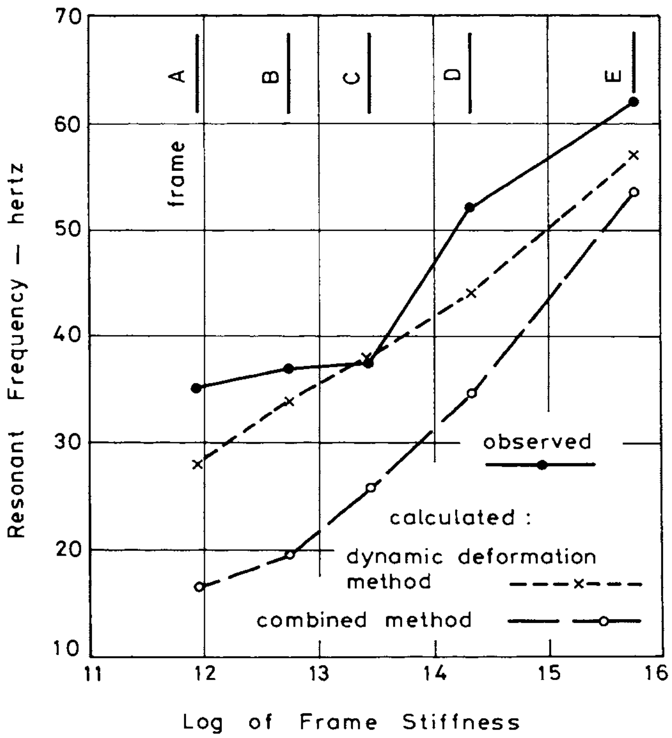

Fig.5 Observed and Calculated Resonant Frequencies

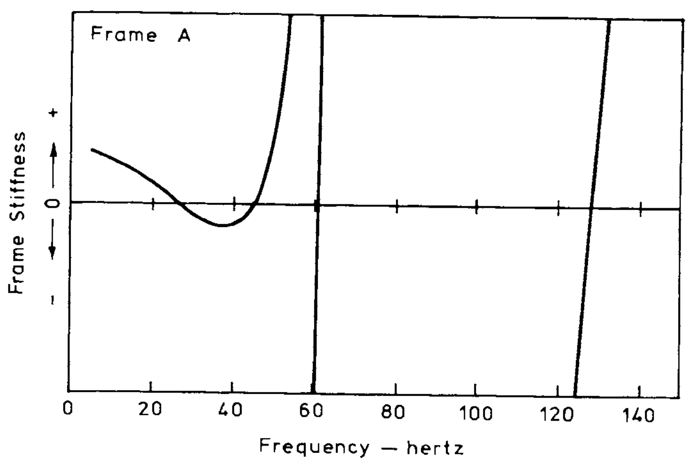

Fig.6 Effect of Frequency on Frame Stiffness

Regarding the calculation of maximum vibration amplitudes at the centre of the cross beam, Fig. 7 shows that the amplitude method yields amplitudes in reasonable agreement with those observed for all frames except frame A. Slightly better agreement is obtained with amplitudes calculated by the dynamic deformation method, although the observed vibration amplitude for frame A still exceeds the calculated value by about 40%.

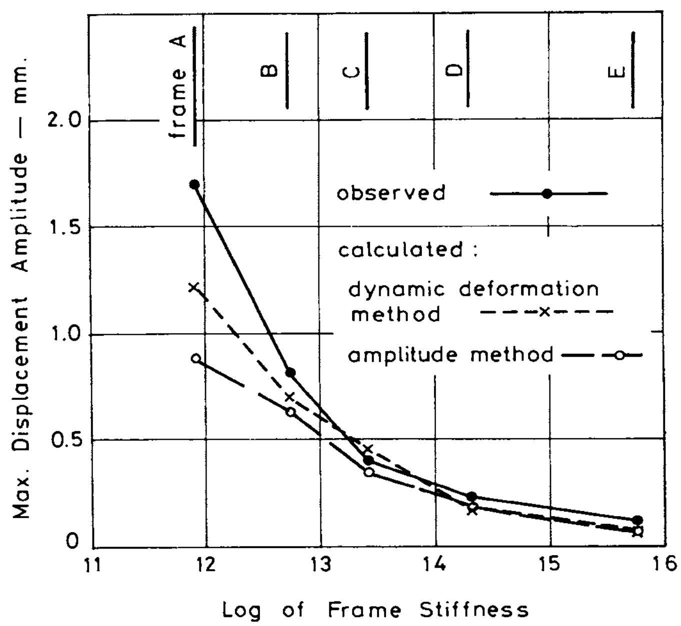

Fig.7 Observed and Calculated Vibration Amplitudes

CONCLUSIONS

The tests on the five frames in which vertical dynamic loading was applied to the centre of the cross beam, confirmed that the resonant frequency increased and the maximum vibration amplitude decreased as the stiffness of the frame increased. The combined method of calculation yielded resonant frequency values that were much too low. With the exception of the most flexible frame, a reasonable level of agreement was found between observed maximum vibration amplitude, and those calculated by the amplitude method and the dynamic deformation method.

REFERENCES

BARKAN, D.D. (1962), Dynamics of Bases and Foundations, McGraw-Hill Book Company, New York.

JACOBSON, M. (1976), On the Pluvial Compaction of Sand, Institute of Civil Engineers, Report No. 9, Aalborg University.

KOHOUTEK, R. (1985), "Analysis of Beams and Frames", Chapter 4 in Analysis and Design of Foundations for Vibrations edited by P.J. Moore, Balkama Publishers, Rotterdam.

KOLOUSEK, V. (1973), Dynamics in Engineering Structures, Butterworths, London.

MAJOR, A. (1980), Dynamics in Civil Engineering, Vol 3, Akademiai Kiado, Budapest.