SEISMIC BEHAVIOUR OF ACCESS FLOOR SYSTEM SUPPORTING TELECOMMUNICATIONS EQUIPMENT

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ABSTRACT

This paper describes a combined analysis-testing programme which was done as part of a major investigation into the seismic performance of a typical commercial access floor system used to support relatively heavy telecommunications equipment. The seismic floor motion is defined in terms of peak floor acceleration expected (in Canadian seismic zone 2) and a floor response spectrum; artificial time-histories enveloping that spectrum are used in the testing portion of the programme. The testing of one particular access floor/telecommunications frame configuration is described. The experimentally determined dynamic properties are used to calibrate a mathematical model of the system. The comparison of panel displacements and frame accelerations determined by analysis of the model and by time-history testing show reasonable agreement. The model is then used to develop panel displacement and frame acceleration response envelopes which can be used for performance evaluation and system design.

INTRODUCTION

The protection of the public and the capability of society to continue to function following a major earthquake requires that telecommunication facilities remain functional. Most such equipment is rather heavy and is mounted on an access floor system in order to facilitate operational flexibility, location of cabling, and air-conditioning. Typical access floors, while designed to carry substantial loads, are not designed specifically to resist lateral loads such as those induced by seismic ground motions. Such access floor systems, even when fastened directly to the supporting structural floor, are relatively flexible in the lateral direction. The large mass of the telecommunications equipment which is being supported produces high shears within the flexible access floor system, which can cause excessive lateral deformations in the system.

The purpose of this paper is to describe and discuss a combined analysis and testing programme done as part of a major investigation into the seismic performance of a typical commercial access floor system. The authors would like to thank Bell-Northern Research Limited, who were the sponsors of this investigation, for their co-operation and permission to publish these results.

This paper describes three distinct phases of the investigation: a) the determination and simulation of the seismic floor motion, b) the experimental testing of a typical access floor/telecommunications system configuration, and c) the mathematical modelling of the same configuration.

SEISMIC FLOOR MOTION

In terms of the appropriate seismic zone for the determination of the peak ground acceleration, an evaluation of the forecasted geographical locations of new telecommunications installations within Canada indicated that 92 percent of these would be in Canadian zone 2 or lower (seismic zones as defined in the 1980 edition of the National Building Code of Canada). It was therefore decided to conduct this investigation on the basis of the seismic risk associated with Canadian zone 2.

Canadian zone 2 is defined so that the accelerations having an annual probability of exceedance of 0.01 range from 0.03g to 0.06g. However this probability is deemed to be too high for adequate protection of telecommunications equipment. A more realistic annual probability level is 0.002, which corresponds approximately to a 10% probability of being exceeded within a 50-year period. It should be noted that this latter level of seismic risk is being proposed for seismic zoning in the 1985 edition of the National Building Code of Canada (1).

Accelerations having an annual probability of exceedance of 0.002 are approximately twice those having a probability of 0.01. Consequently it was decided to use a peak ground acceleration of twice the upper limit of Canadian zone 2, namely 0.12 g. However, since the most severe condition is not at ground level, but at some height within the building, it is necessary to determine the maximum floor acceleration corresponding to a ground acceleration of 0.12 g. Most equipment installations (80 to 90 percent) are on the ground or the second floor of buildings, many of these being extensions to taller buildings which may be up to nine storeys in height. It has been shown (2) that the amplification of ground acceleration is not dependent on the storey number alone, but also on the floor's nearness to the building's roof. Also, the largest acceleration amplifications occur in the shorter buildings, so that an installation near the top of a high-rise building would be subjected to smaller accelerations than an installation on the top floor of a low-rise building. Consequently, the most severe installation location within buildings would be the second floor of a two-storey building. A one, two or three storey extension of a taller building would be less severe since the motion amplification would be primarily governed by the tall building.

Reference 2 provides data based upon information obtained from the San Fernando earthquake of 1971 on ground-to-floor amplifications for various building types and geologic conditions. An amplification factor of 2.8 was used, which corresponds to the average-plus-one-standard-deviation amplification for the case of all buildings and all geologic conditions; this resulted in a peak floor acceleration of 0.34 g.

The most common method of specifying seismic floor motion for the purpose of dynamic analysis or testing is to specify a required response spectrum (RRS) for a particular level of equipment damping. Reference 2 also provides a set of upper-bound spectral shapes which were used to obtain the RRS (for 2 percent equipment damping) corresponding to a peak floor acceleration of 0.34 g. This RRS is shown in Fig. 1 and was used as the basis for all analysis and testing in this investigation.

For purposes of testing, it is necessary to create an artificial time-history (for use as input to the shaking table) which has a response spectrum which envelopes the RRS over the frequency range of the system. Two distinct artificial earthquake time-history records, denoted as TH1255 and TH1291 herein for ease of reference, were developed by superposition of a number of sine-beat motions of various amplitudes. The frequencies of these sine-beat motions range from 1 Hz to 32 Hz and are spaced 1/3-octave apart. Each earthquake record consisted of three identical segments each of 10-second duration; the response spectrum for TH1255 is included in Fig. 1.

The response spectrum of TH1255 falls below the RRS at frequencies below 1.9 Hz in order to improve the resolution of the shaking table by keeping the maximum displacement below one inch. This was deemed acceptable because the lowest frequencies of the system being tested are normally above 1.9 Hz.

DESCRIPTION OF ACCESS FLOOR SYSTEM AND TEST SET-UP

The access floor system used in this investigation was a typical commercial system based on a 2 ft. by 2 ft. square module. The system consists of an assembly of the following components:

a) pedestals of adjustable height which are used to support the access floor panels at the module corners; each pedestal has a square base plate which can be attached to the supporting structural floor,

b) stringers which interconnect the pedestals, providing continuity because each stringer is two or more modules in length, and

c) module-sized panels whose corners are bolted to the pedestal tops and whose edges rest on the stringers.

For the purpose of this investigation, it was decided to do the basic testing on a small system configuration consisting of 3 modules in each direction with extension of the results to larger systems by analysis. This configuration was selected because it enabled the assembly to be mounted on the McMaster shake table; it also enabled a two-unit telecommunication frame line-up to be mounted on the access floor system during testing. The telecommunication frame units (hereafter referred to as frames) were heavy line frames (800 lb. each) used in digital switching systems. By using these frames, which had very low natural frequencies and very large masses, it could be determined that other lighter frames would produce less critical response, both in the frames themselves and in the access floor system.

A variety of symmetrical and asymmetrical configurations were tested, with table motion both transverse and parallel to the longitudinal frame axis. Only one test configuration will be discussed in this paper, due to limitations of space, but it will enable the demonstration of how combined testing and analysis can be used for seismic performance evaluation.

The particular configuration discussed in this paper is shown diagramatically in Fig. 2, including the notation used for response parameters.

TESTING PROGRAMME

In order to determine access floor properties without the interaction of the flexible frames, Series I tests were done with rigid concrete masses mounted in place of the actual frames; tests done with the flexible frames were designated as Series II.

For each configuration (in both Series I and II), testing began by conducting a slow low-level sweep through the frequency range 1 to 32 Hz in order to identify the natural frequencies of the system. Following the sweep test, a number of single frequency sinusoidal tests with progressively increased excitation levels were applied at one or more of the identified natural frequencies. These tests enabled the evaluation of frequency shifts and the change in amplification with increased excitation levels. Typically, there were relatively small decreases in frequency and rather significant reductions in amplification (for both panel/table and frame top/table amplification) as excitation levels increased. After these single frequency tests, a second sweep was normally done in order to detect any changes in the test system. The two time-history records mentioned previously were then applied, separately, at progressively increased input levels. Finally, a third sweep test was run to detect any softening of the test system caused by the time-history tests.

The results of the tests will be presented and discussed in connection with analytical results in the next section.

SYSTEM MODELLING

The basic system model used in this investigation is described in Fig. 3, with the quantities used in this model defined as follows:

W_p = weight of panels and stringers (pedestals are not included since most of their weight is at the base which is not moving)

W_F = weight of frames

α = proportion of W_F which is assumed to be moving with the access floor panels

K_p = stiffness of access floor system

K_F = stiffness of frame, when assumed to act as a single degree of freedom system

Δ_p = lateral displacement of access floor system at panel level, relative to base

Δ_F = lateral displacement of top of frame, relative to access floor base.

Of the system properties, W_p and W_F are known at the outset and have the values: W_p = 223 lb. and W_F = 1600 lb. The parameters, K_p, K_F and α must be determined from test results.

First, concerning K_p, the measured frequencies of three different Series I configurations (i.e. with concrete weights only) resulted in an average dynamic stiffness of 640 lb./in. per pedestal (with a range from 611 to 671 lb./in.). The test system has 16 pedestals, yielding a value of K_p = 10240 lb./in.

The frame stiffness K_F is determined by knowing the basic frame frequency f_F (determined experimentally when mounted on a rigid base, and having a value of 2.3 Hz) using the following expression

$$K_F = 4\pi^2 f_F^2 (1-\alpha) \frac{W_F}{g} \qquad (1)$$

However, it is also necessary to know α, which can only be determined by an analysis of the complete model (Fig. 3). Such an analysis yields the two system frequencies f_1 and f_2 given by the following expressions

$$f^2_{1,2} = \frac{1}{4\pi^2} \{\frac{a+c}{2} \pm \sqrt{(\frac{a-c}{2})^2 + bc}\ \} \qquad (2)$$

in which $$a = \frac{(K_p+K_F)g}{W_p + \alpha W_F} \qquad (3)$$

$$b = \frac{K_F g}{W_p + \alpha W_F} \qquad (4)$$

$$c = \frac{K_F g}{(1-\alpha)W_F} \tag{5}$$

A useful parameter is the ratio of deformations, also known as the amplification ratio, which is given by

$$\left(\frac{\Delta_F - \Delta_p}{\Delta_p}\right)_{1,2} = \frac{a - 4\pi^2 f_{1,2}^2 - b}{b} \tag{6}$$

The computed values of f_1, f_2 and the associated amplification factors (given in Eq. 6) for different values of α are given in Table 1. This table also provides the experimental values for f_1 and its associated amplification factor; the second frequency was not evaluated experimentally. From this table it can be seen that the fundamental frequency f_1 is not sensitive to the parameter α, and consequently f_1 cannot be used to evaluate α. However the observed first mode frame/panel amplification ratio would suggest that a value of α of 0.5 would be appropriate. The use of this model to predict second mode properties is not recommended, since the model has not included more than one mode of basic frame vibration. Now that α has been determined, Eq. 1 can be used to evaluate K_F, yielding K_F = 432 lb./in.

The purpose of developing this model is to provide a means of estimating the maximum response of the system during seismic floor excitation. If the floor excitation is described in terms of a response spectrum, then the peak panel and frame responses are given by the response of the first mode (since the second mode contribution to displacement response is negligible in this case), yielding

$$\bar{\Delta}_p = \Gamma_1 S_{x1} \tag{7}$$

$$(\overline{\Delta_F - \Delta_p}) = \left(\frac{\Delta_F - \Delta_p}{\Delta_p}\right)_1 \bar{\Delta}_p \tag{8}$$

in which the first mode participation factor Γ_1 is given by

$$\Gamma_1 = \frac{W_p + \alpha W_F + (1-\alpha)W_F\,(\Delta_F/\Delta_p)_1}{W_p + \alpha W_F + (1-\alpha)W_F\,(\Delta_F/\Delta_p)_1^2} \tag{9}$$

and $\quad S_{x1}$ = displacement spectrum ordinate at frequency f_1 .

Applying Eqs. 7 to 9 to the results of the Series II test (f_1 = 2.3 Hz) and using α = 0.5 together with the experimentally observed first mode amplification ratio (Δ_F/Δ_p = 25) yields

$\Gamma_1 = 0.042$

$\overline{\Delta}_p = 0.042\ S_{x1}$

$\overline{\Delta_F - \Delta_p} = 1.01\ S_{x1}$

Frame accelerations are given by the following expressions

$$a_F = \Gamma_1 \left(\frac{\Delta_F}{\Delta_p}\right)_1 S_{a1} \tag{10}$$

in which the acceleration spectrum ordinate at frequency f_1 is given by

$$S_{a1} = 4\pi^2 f_1^2 S_{x1} \tag{11}$$

When these expressions are used to estimate the panel displacements arising during time-history excitation, the comparisons are tabulated in Table 2 for different levels of excitation. In the tabulated model results, S_{x1} has been obtained at the model frequency directly from the 2% damped RRS. Comparison with experiment is possible because the damping of the experimental system is in the neighbourhood of 2% of critical.

The panel displacements and frame accelerations predicted using the model are in reasonable agreement with experiment except for the last line in Table 2. At this high level of excitation, the system had actually softened to a natural frequency below 1.9 Hz yielding lower experimental values than predicted on the basis of the RRS.

This confirms that the model given in this section provides a valid basis for evaluating the fundamental frequency, the panel-frame amplification and peak response parameters during seismic floor excitation.

APPLICATION TO REALISTIC ACCESS FLOOR SYSTEMS

The purpose of developing the models in the previous section is to be able to evaluate the characteristics and performance of realistic access floor systems consisting of a large number of panels supporting a variety of frame line-ups. The major parameters affecting the system characteristics are:

a) The per pedestal stiffness K_p, which is constant at 640 lb./in.

b) The per pedestal weight $\bar{W}_p$, of the access floor system at 13.9 lb./ pedestal.

c) The per pedestal weight of the frames mounted on the access floor system, designated $\bar{W}_F$. This is a design parameter which is dependent on the weight of each frame and the spacing of the frames. The values of $\bar{W}_F$ used herein range from 100 to 400 lb./pedestal.

d) The basic frequency of each frame line-up, f_F. Each line-up will normally have slightly different frequencies but the range of such frequencies should not be large.

Using the same parameters as defined above, the maximum horizontal pedestal displacements for different values of f_F and $\bar{W}_F$ are shown in Fig. 4. These curves have been calculated using Eqs. 7 and 9, with S_{x1} determined from the RRS. On the basis of other test configurations not included in this paper, it is estimated that the maximum torsional effects due to a symmetry would increase Δ_p by no more than 25%. Therefore the curves in Fig. 4 can be considered as realistic upper limits of Δ_p inclusive of possible adverse torsional effects.

While it is beyond the scope of this paper to discuss the acceptability of the displacements shown in Fig. 4, it should be noted that they are not very sensitive to modest changes in the frame frequency, but increase in proportion to the overall frame load on the access floor system.

The maximum accelerations at the top of the frame are shown in Fig. 5. These accelerations are relatively independent of $\bar{W}_F$ and f_F, with a maximum of just under 3g. This is likely to be slightly conservative since increased damping at large excitation levels would decrease S_{a1} in Eq. 10.

CONCLUSIONS

The results of this investigation show that the experimentally determined dynamic properties of an access floor system can be used to calibrate a relatively simple mathematical model, enabling the maximum response during seismic floor motion to be estimated quite accurately. With the model validated in this manner, the results can be extrapolated to larger scale systems in order to determine response envelopes for performance evaluation and design purposes.

REFERENCES

1. Heidebrecht, A.C., Basham, P.W., Rainer, J.H. and Berry, M.J., "Engineering Applications of New Probabilistic Seismic Ground Motion Maps of Canada", submitted to the Canadian Journal of Civil Engineering.

2. Liu, S.C., Fagel, L.W. and Dougherty, M.R., "Earthquake-Induced In-Building Motion Criteria", Journal of the Structural Division, ASCE, Vol. 103, No. STI, January 1977, pp. 133-152.

Table 1

Calibration of Model to Determine α.

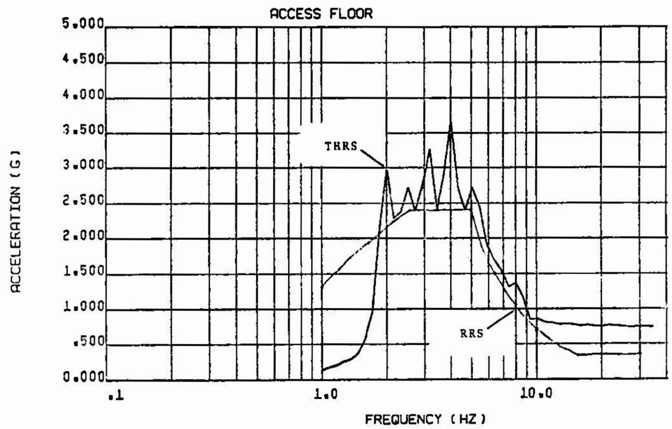
α	f_1 (Hz.)	f_2 (Hz.)	$\left(\frac{\Delta_F - \Delta_P}{\Delta_p}\right)_1$	$\left(\frac{\Delta_F - \Delta_P}{\Delta_p}\right)_2$
0.0	2.21	22.1	11.7	-1.01
0.17	2.22	14.8	13.9	-1.02
0.33	2.24	11.8	17.2	-1.04
0.50	2.25	10.1	22.8	-1.05
0.67	2.27	8.95	33.5	-1.07
Experimental	2.3	-	24	-

Table 2

Panel Displacement and Frame Acceleration Comparison, Longitudinal Frame Vibration

TABLE MOTION	AMPLITUDE (PERCENT OF RRS)*	S_{x1}	Model		Experimental	
			Δ_p (in.)	a_F/g	Δ_p (in.)	a_F/g
TH1291	4.3	0.21	0.009	0.11	0.009	0.16
TH1255	10	0.50	0.021	0.25	0.018	0.27
TH1291	43	2.1	0.09	1.08	0.082	0.95
TH1255	70	3.50	0.147	1.76	0.070	1.20

* Experimental values are based on percent of the full-scale time history which envelopes the RRS.



RRS = Required Response Spectrum

THRS - Time History Response Spectrum (TH1255)

Figure 1 Floor Response Spectra (2 percent damping)

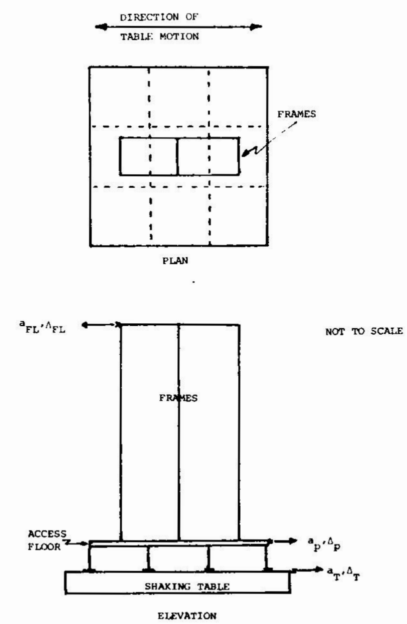


Figure 2 Test Configuration

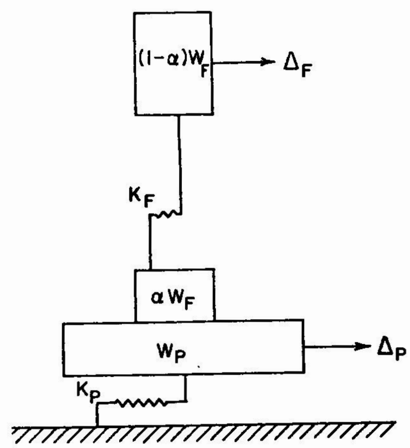


Figure 3 System Model

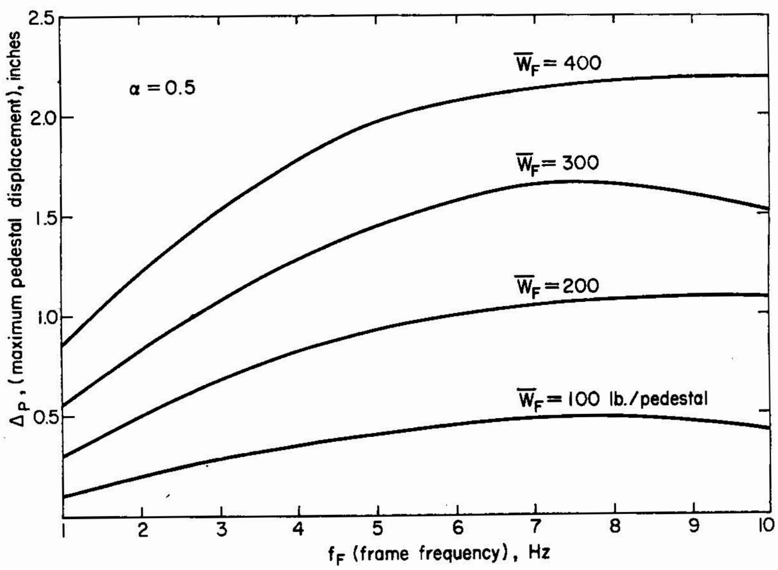


Figure 4 Maximum Pedestal Displacements

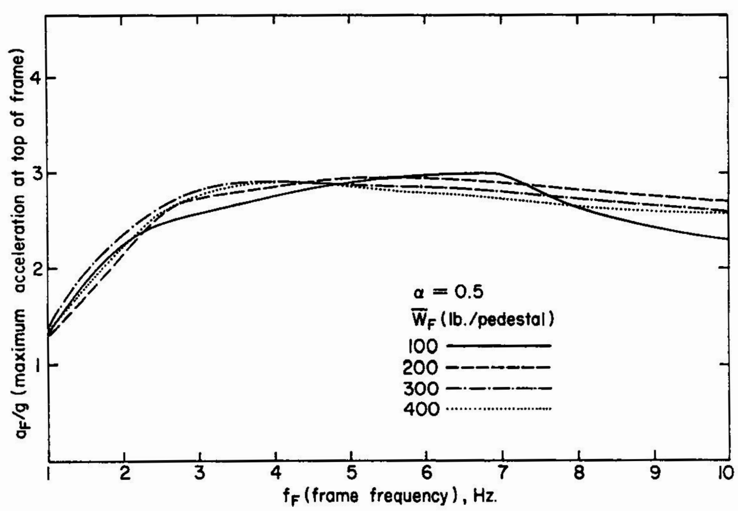


Figure 5 Maximum Frame Accelerations