DYNAMIC BEHAVIOR OF TALL BUILDINGS WITH CLADDING

by

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SYNOPSIS

An analytical study of a tall steel frame, with cladding action, has been reported. Structural partitions, performing as shear panels, increase the lateral stiffness of the frame. The dynamic behavior is presented for two cases; the "balanced design," where panels are of stiffness comparable to the frame, and the "overstiff design," where the panel stiffness is much greater. Certain restrictions for seismically active areas are immediately apparent.

INTRODUCTION

Designers of tall steel planar frames often find their designs to be governed by the lateral stiffness, requiring the addition of frame steel to an otherwise safe design. A research program at Cambridge University[1] has considered the use of structural partitions to perform this stiffening function. The design procedure and static behavior are presented elsewhere,[2] while the dynamic studies are now described.

One of the sample structures, a 30 storey 2 bay frame, is the subject of the dynamic analysis. In the "balanced design" the partition panels are rather flexible, increasing the overall stiffness about 30%, and just achieving the allowable drift index of 0.002. Studies of light gauge corrugated sheeting[1,3] indicate that such low panel stiffnesses may be realized. A straightforward shear panel, however, would have a stiffness several times that of the frame, and produces an "overstiff design" comparable to a braced frame.

Dynamic Analysis

A computer program using numerical integration and sparse matrix techniques was prepared. The "balanced design" was analyzed with elasto-plastic girders, and elastic-brittle panels. The panels were considered to fail when a deflection 3.0 times allowable was imposed on them, amounting to 21.9 mm in a 3.65 meter storey height. In this case all member axial deformations have been ignored, and the item of interest is the sequence of panel failures.

The "overstiff structure" was reduced to an equivalent one-bay frame, to decrease the number of unknowns, and permit the introduction of column axial displacements. Purely elastic behavior is considered in this case. The higher natural frequency of the stiffer system required a smaller integration step.

The forcing function is a sinusoidal ground motion having a period

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[I] Assistant Professor of Architecture and Civil Engineering, Carnegie-Mellon University, Pittsburgh, Pennsylvania, formerly Research Student, University of Cambridge of 1.64 seconds, and a maximum acceleration of 7.6% of gravity. This motion excites the 30 storey frame largely in the second mode, which is reasonable.

Results

Figure 1 shows the roof level displacement for the 30 storey "balanced design." Only the first six seconds of response are shown, although the analysis continued. The failure of a panel is indicated by a dot with the storey number. The upper four storeys required no stiffening, so panel number 26 is the topmost, panels in storeys 19 through 26 are seen to have failed. This is indicative of the whipping in tall buildings; displacement profiles (not shown) illustrate this further.

Note that a "balanced design" is by its very nature tapered in stiffness. It has been established that this taper induces the whipping action.[4]

The upper 20 storeys of the structure were then modelled as a complete building, with the response shown in figure 2. There are no panels above storey 16, and panels 13 through 16 fail in this instance. Again, panels in the upper third of the structure have failed.

When the upper 10 storeys were modelled as a structure, the results (figure 3) are surprising. This structure, much stiffer than the others because of its lesser height, has all its panels fail. Furthermore, the failure occurs within the very first cycle of response. The short structure has been excited in its first mode, whereas the taller ones respond rather in the second mode; moreover, an excitation of short duration is damaging. This fact has been noted by Housner.[5] When ground motions of similar duration were applied to the 30 storey structure, few panels were disturbed, and little inelastic action developed.

Figure 4 illustrates the response of the "overstiff design." The growth of the panel force envelope is shown, increasing with time. The first observation is the speed with which the panel forces develop; in the "balanced design" roof deflection does not occur until one second has passed, while in the "overstiff design" the force envelope is almost completed by that time. Obviously, the shortest excitation would still induce large panel forces.

The significance of panel forces is the axial load they induce in the columns. From the bay geometry (3.65 meters by 9.1 meters) the induced forces are 40% of the panel force. A simple observation shows that only a few panels need be at 1000 K.N to produce column loads in excess of the squash load. This dangerous behavior cannot, of course, be tolerated.

Conclusions

In "balanced designs" the upper storey panels will require large deformation capacity, because of the whipping effect. The designer may choose to place panels only in the lower two-thirds of the structure, where less sway occurs. Most of the steel savings arise from the use of panels in the lower storeys, so this is not a serious drawback. The odd behavior of the 10 storey structure is not of great consequence, as structural partitions would hardly be prescribed for such a low building.

The behavior of the "overstiff design" should be of great concern to the designer. The high axial forces induced must be avoided. The designer should reflect for a moment on the suitability of this design even in "safe" regions; earthquakes are not the only dynamic loads. The response of such a stiff structure to very short disturbances may make other loadings critical.

Acknowledgmments

This work was performed at the University of Cambridge Engineering Laboratory. The thesis supervisor was Professor J. Heyman, who contributed considerable advice to the study. His guidance is greatly appreciated.

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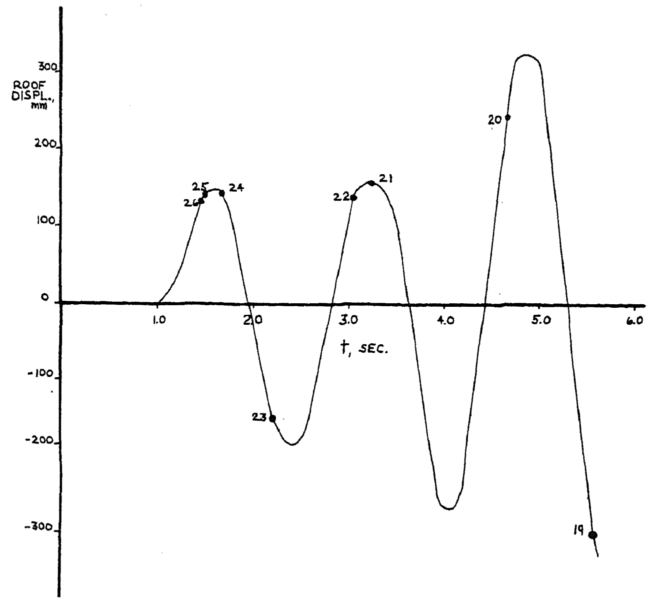

FIGURE 1. PANEL BEHAVIOR,
30 STOREY STRUCTURE

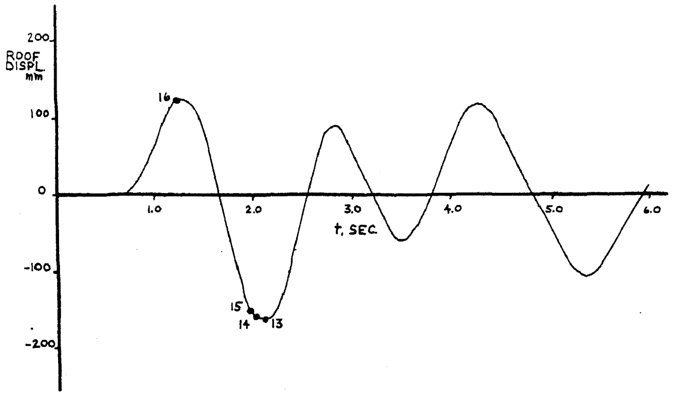

FIGURE 2. PANEL BEHAVIOR,
20 STOREY STRUCTURE

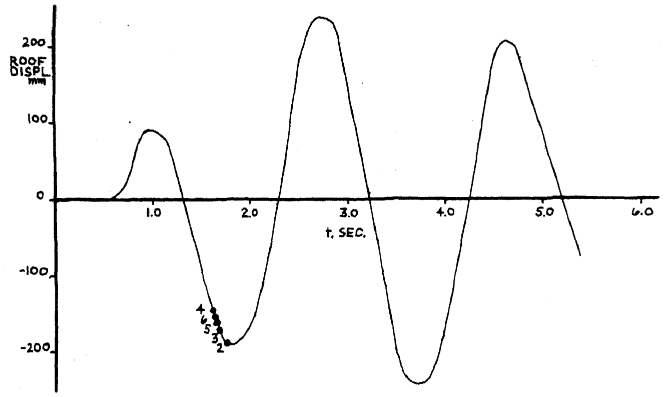

FIGURE 3. PANEL BEHAVIOR,
10 STOREY STRUCTURE

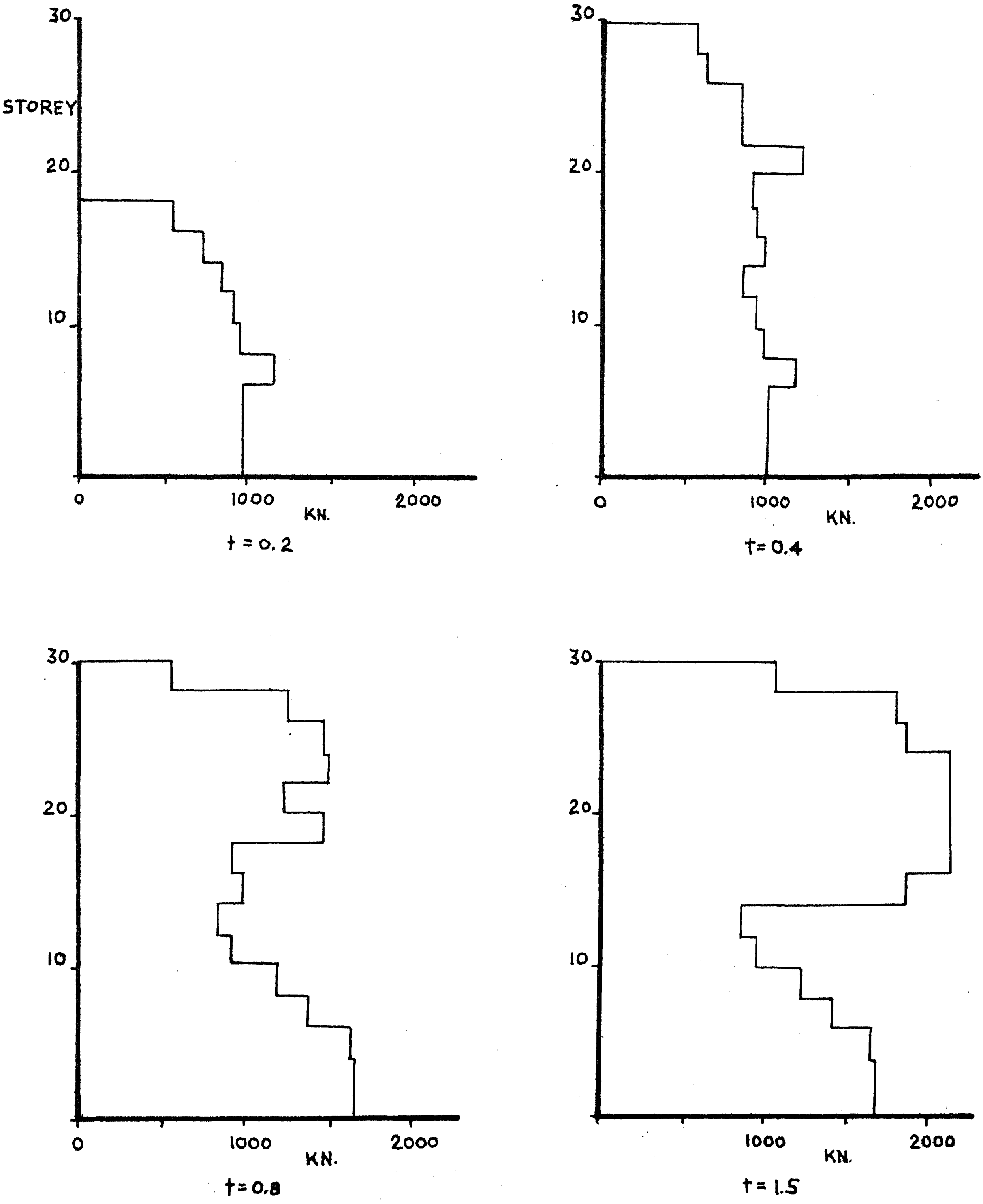

FIGURE 4. PANEL SHEAR FORCE ENVELOPE, OVERSTIFF STRUCTURE