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CONCRETE WALL ANCHORAGE FORCES IN INDUSTRIAL BUILDINGS WITH FLEXIBLE ROOF DIAPHRAGMS

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SUMMARY

Industrial buildings with flexible wood roof and heavy concrete walls have experienced extensive damage in the past. Linear dynamic analysis procedures are used to evaluate the amplification of earthquake motion in the flexible roof diaphragm and the demand for anchorage of the exterior concrete walls. Nonlinear dynamic analysis is also used to determine the extent of stress reduction due to nonlinearity of wall and roof elements. The results show that the present U.S. code requirements are, in certain cases, nonconservative and modification of code requirements for design of these elements is warranted.

INTRODUCTION

The application of tilt-up wall construction for industrial and commercial use in the U.S. has been extensive in the past forty years due to its speed of construction and economy. The seismic design criteria for these buildings has largely been based on static load tests and equivalent lateral load procedures. The 1971 San Fernando earthquake resulted in many cases of collapse of these structures. The seismic weakness in this type of construction was identified to be associated with wall anchorage and the need for positive roof ties. Recent U.S. codes require anchorage devices which are capable of transferring up to 30 percent of wall weight laterally into the roof diaphragm.

The objective of this study is to evaluate the adequacy of the code requirements in terms of wall anchorage forces. Both linear and nonlinear dynamic procedures are employed to determine wall forces for various ranges of dynamic properties of the roof structure and the supporting wall elements.

LINEAR ANALYSIS

A typical one-story tilt-up building as shown in Fig. 1 was used in this analysis. The building is 200 by 200 feet in plan dimension with 20-foot high and 6-inch thick concrete walls. The roof was assumed to have a seismic mass of 15 psf. The structure can be represented by the model shown in Fig. 1. The actual equivalent model used for the elastic dynamic analysis is shown in Fig. 2. The end walls parallel to the direction of earthquake motion are typically very stiff, compared to the stiffness of the roof diaphragm and thus their effect was not included in the model. Preliminary analysis with models that considered the two sidewalls as separate elements indicated that symmetric breathing modes do not significantly affect seismic response and thus, these walls can be lumpted together. The initial stiffness of the roof diaphragm was calculated by adding the shear deformation of plywood and nail slip. Bending deformation of the roof was ignored. The assumed roof nailing was based on Uniform Building Code requirements. The stiffness of the wall elements was based on uncracked (gross) concrete properties. The above assumptions resulted in a structure with a fundamental natural period of approximately one second and wall period of 0.2 seconds.

In order to evaluate the forces associated with other roof and wall configurations, the roof shear stiffness and wall bending stiffness were modified resulting in roof periods in the range of 0.25 to 2.0 seconds and wall period in the range of 0.1 to 0.60 seconds. This range of parameters covers most practical cases including masonry walls in highly stressed and cracked conditions.

The time-history motion employed was the recorded S69E component of the 1952 Taft earthquake modified by Fourier transform techniques to match a target average spectrum scaled to 0.40g ground acceleration. This level of ground acceleration is consistent with the requirements for Zone 4 of the Uniform Building Code. The response spectra for this motion is shown in Fig. 4. The SAP IV computer program was employed using damping of 5 and 10 percent, integration timestep of 0.01 second and earthquake duration of 15 seconds.

To simplify evaluation of the findings, the results are presented as equivalent lateral load coefficients. Roof shear force as a function of roof period was found to be consistent with the spectral ordinate of the input ground motion. Fig. 3 shows the distribution of shear force within the roof diaphragm. It can be seen that due to the effect of higher modes, the shear force differs considerably from the typically assumed linear variation. The deviation from linear variation is a function of structure period and is most pronounced for building period of 1 sec. which corresponds to a very common building configuration. It appears that for the purpose of design, it should be assumed that the maximum value of shear at the end of diaphragm remains constant within the end quarter span, as shown in Fig. 3.

The wall anchorage forces are shown in Fig. 4. It can be seen that the wall forces reduce with increasing roof period (following the general trend in spectral value) and with increasing wall period. It is also interesting to note that a relative increase in wall forces occurs when wall periods approaches roof period. This is due to the "tuning" effect of walls with roof diaphragm.

The most important finding of this analysis is that the wall forces can be as high as 90-100 percent of wall weight for stiff roof and wall elements. This corresponds to smaller buildings with uncracked walls. It also represents anchorage forces for long and narrow concrete diaphragms. It can also be concluded that increasing roof nailing and subsequent increase in roof stiffness (and strength) can increase the demand on wall anchorage forces.

The present Uniform Building Code for buildings in Zone 4 uses a coefficient of 0.3 for wall anchorage at working stress level. This is equivalent to about 0.5 at ultimate. Fig. 4 indicates that this force criteria does not provide sufficient strength for buildings with roof period of less than about 1 second. In these cases, a coefficient of about 0.5 at working stresses appears to be more appropriate, at least for building properties investigated here.

The location of wall elements along the diaphragm span with highest anchorage force changes with building period. For stiffer buildings, the largest loads appear to occur near the center of the building.

NONLINEAR ANALYSIS

In order to evaluate the change in anchorage forces due to nonlinearity in roof diaphragm and wall elements under high seismic loads, a series of nonlinear dynamic analyses were performed using the computer program DYNAS (a Dames & Moore modified version of DRAIN 2D). The equivalent model used is shown in Fig. 5. This model considers all aspects of the linear model described above, except the number of elements have been reduced to reduce cost of computer analysis.

The inelastic behavior of the roof elements was represented by a bi-linear approximation of plywood diaphragm test results reported by American Plywood Association. In this approach, the slope of the post-yield force deflection diagram is 0.25 times the initial slope. For the wall elements, an elastic perfectly plastic behavior was assumed. Full hysteretic loop (without pinching) was used for wall and roof elements. The coefficient of lateral load, corresponding to the yield point was chosen to be as low as 0.15 for the roof and 0.30 for the wall elements. Wall and roof periods were chosen such that tuning of roof and wall due to softening of the elements could be captured. Stiffness and mass proportional damping corresponding to 10 percent was employed. Other variables were the same as the linear analysis.

The resulting roof and wall forces are tabulated in Table I. The following observations are made.

1. Softening of elements due to nonlinearity did not cause "tuning" with other elements resulting in increased forces.

2. Reduction of roof shear forces due to nonlinearity did not cause substantial reduction in wall anchorage forces, i.e., reduction of forces is not proportional to ductility levels.

Although more detailed analysis is required to obtain definitive conclusions, it appears that nonlinearity and material yielding does not produce sufficient reduction in seismic forces to ensure protection against substantial damage and possible collapse. Consideration of pinching in hysteresis loop and P-W effect can further increase seismic demand on the structure.

Considering the typical nonductile connection of walls to roof diaphragm, it seems appropriate that analysis of structures with heavy concrete walls and flexible roof diaphragm should be based on elastic dynamic analysis with little or no reduction of forces as a consequence of nonlinearity.

CONCLUSIONS

The results of parametric linear and nonlinear dynamic analyses presented here indicate that the existing requirements for design of roof and wall anchorage forces are not sufficiently conservative for certain dynamic properties of structural elements. Preliminary suggestions are provided above for modifying the code requirements. More detailed parametric studies are needed to provide definitive recommnedations.

TABLE I

ROOF AND ANCHORAGE FORCES FROM NONLINEAR ANALYSIS

Period, Sec.		Yield Level		Roof	Wall	Ductility
Roof	Wall	Roof	Wall	Shear	Force	Demand
0.5	0.3	N	N	0.48	0.61	1.0
0.5	0.3	N	0.5	0.48	0.58	1.7
0.5	0.3	N	0.3	0.46	0.45	7.0
0.3	0.5	N	N	0.48	0.61	1.0
0.3	0.5	.35	N	0.42	0.58	1.7
0.3	0.5	.25	N	0.37	0.58	2.8
0.3	0.5	.15	N	0.33	0.54	5.8

N=No yielding; linear elastic performance

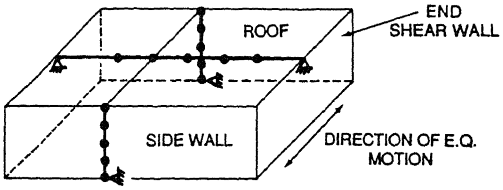

FIGURE 1 BUILDING STRUCTURE AND EQUIVALENT MODEL

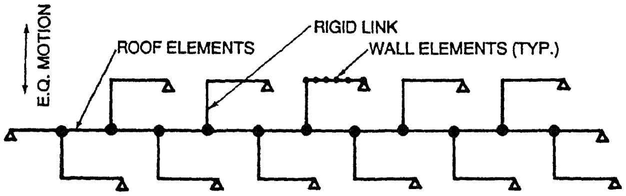

FIGURE 2 MODEL FOR ELASTIC ANALYSIS

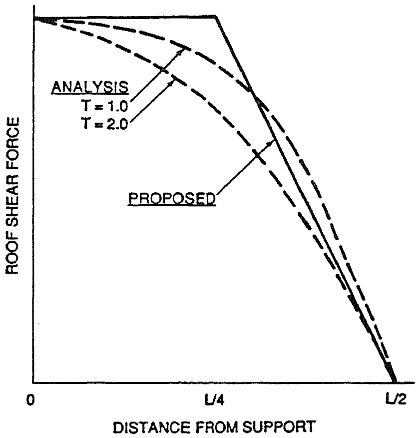

FIGURE 3 ROOF SHEAR DISTRIBUTION

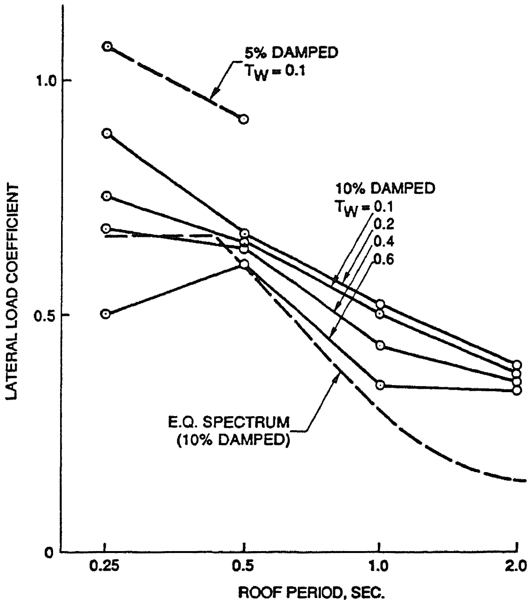

FIGURE 4 WALL ANCHORAGE FORCES

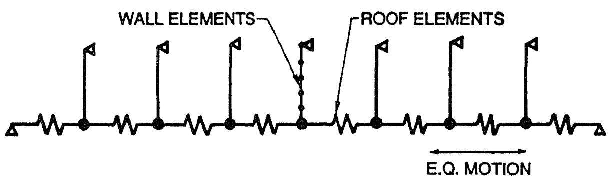

FIGURE 5 MODEL FOR NONLINEAR ANALYSIS