A STUDY OF EARTH LOADINGS ON FLOODWAY RETAINING STRUCTURES IN THE 1971 SAN FERNANDO VALLEY EARTHQUAKE

by

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INTRODUCTION

The floodway system in the Greater Los Angeles, California area involves over 160 km of channel and serves to transport the runoff from the winter rains to the Pacific Ocean. The floodway structures consist of open U-shaped channels with the wall tops set flush to the ground surface and completely buried culverts; both types of sections were significantly damaged in localized regions during the 1971 San Fernando Valley Earthquake. For the investigation described in this paper, the behavior of the open channel structures was studied. The behavior of the underground structures has been previously described by Hradilek (1972).

The location of the study region, the floodways and the earthquake energy center and contours of estimated maximum accelerations are shown in Figure 1. Estimated maximum accelerations in the region range from 0.65g to 0.2g; the closest floodway is located only 10 km from the energy center. The well known San Fernando Dam which failed during the earthquake is located near several of the floodways (see Fig. 1).

The major cause of damage to the open channel floodways was excessive earth loads exerted on the channel walls by the wall backfills. Because the behavior of the walls was well documented and field conditions were well defined, the seismically induced earth loads can be accurately calculated. The principal effort of this paper is to use this information to: (1) evaluate the conventional approach towards determining seismically induced earth loadings; and, (2) investigate the reserve of strength built into earth retaining structures by conventional structural design procedures. Previous work of the type undertaken herein has had to rely on purely analytical or model studies. The San Fernando Valley floodways offer a case history which, for the first time, allows study of the earth loading problem under actual earthquake conditions.

DESCRIPTION OF THE STRUCTURES AND SUBSOIL CONDITIONS

The floodway channels included in the study were designed by the U.S. Corps of Engineers, Los Angeles District, in 1961 and constructed in 1963. A typical section of the open channel floodway is shown in Figure 2; it is essentially a U-frame structure with the tops of the walls set flush to the ground. The height of the walls varies from 1.8 to 7.9 m with channel widths varying from 2.3 to 17.4 m. The channels were

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constructed in sections 9 to 18 m long with construction joints between sections and no transverse reinforcing across the joints. A processed, free draining backfill was compacted within a wedge behind the walls (see Fig. 2). No water was in the floodways at the time of the earthquake.

The foundation soils in the study region consist typically of a lightly cemented to uncemented dense sand overlying a moderately cemented sandstone-conglomerate known as the Saugus Formation. The thickness of the overlying sands range from essentially zero near the base of the San Gabriel Mountains to 200 m at the Southern extremity of the study area (see Fig. 1).

Generally, the water table is located more than 50 feet below the ground surface, although in local areas near reservoirs it may be higher. In none of the investigated cases of floodway performance did the ground water table play a role.

DESIGN OF THE OPEN CHANNEL FLOOWDAY WALLS

The walls of the open channel floodways were designed as cantilevers loaded by a conventional Rankine triangular earth pressure diagram; no earth pressure loads were applied to account for seismic effects. The soil was assumed to produce an equivalent fluid pressure of $4.8 kN/m^3$. Allowable stresses in the concrete and steel were $20.7 MN/m^2$ and $138 MN/m^2$ respectively. Because the reinforcing bars were made of steel with a minimum tensile strength of $276 MN/m^2$, the use of $138 MN/m^2$ in design provided an inherent reserve of strength in the floodway walls against seismic loads not considered explicitly in design.

PERFORMANCE OF THE FLOODWAY STRUCTURES

Damages to the floodway system in the study area occurred in regions subjected to high accelerations or fault movements. Locations of the damaged sections relative to the estimated peak accelerations and observed faulting are depicted in Figure 1. Fault movements in the channels were easily detectable from relative offsets in the soil and structures and changes in invert elevation measured prior to and after the earthquake. The effects of the fault crossings were very localized.

The open floodways were primarily damaged by increased earth pressures exerted by the backfill behind the cantilever walls. Approximately two kilometers of the walls had to be replaced or repaired after the earthquake because of the damages. The mode of failure was remarkably uniform and consisted of an inward tilting of the walls towards the channel with the center of rotation at the floor-wall connection (see Fig. 2). A few wall sections toppled into the channel as shown in Figure 3. However, most damaged sections tilted just enough to seriously crack the wall-slab connection and yield the reinforcing steel.

The relationship between the wall damage and the ground acceleration is demonstrated in Figure 4, a plot of meters of damaged wall vs. peak acceleration. No damage to the walls occurred until accelerations of about 0.5 g were reached, a surprisingly large value of acceleration in view of the fact that no seismic loadings were explicitly considered in design. The key to this behavior is to be found in the factors of safety used in the design for static loadings and the duration of the peak accelerations as described in the following segments of the paper.

ANALYSIS METHODS

In order to calculate earth loadings on the walls, psuedo-static analyses were performed for damaged and undamaged sections of the open floodway. The commonly used Mononabe-Okabe procedure (3,4) was employed to obtain the magnitude of the additional earth loading due to a specified horizontal ground acceleration. In the calculations, the backfull friction angle was assumed to be 35° and the wall-soil friction angle was taken as 17°. Vertical soil accelerations were found to have only a small influence on calculated results and were generally neglected. In calculating the moment of the earth loading about the slab-wall connection, the static earth loading was placed at two-thirds the wall height from the ground surface while the dynamic earth loading was placed at one-third the wall height from the ground surface. In addition to the psuedo-static analyses, shear wave propagation studies of different typical soil profiles were performed allowing evaluation of the duration of the maximum surface accelerations.

RESULTS OF ANALYSES

The psuedo-static analyses were used to calculate moments about the slab-wall connection as induced by the increased earth loading due to the seismic accelerations. For five typical floodway designs the horizontal accelerations required to produce a moment equal to the moment capacity of the wall-slab connection were determined. The results of these calculations are summarized in Figure 5, a plot of acceleration required to cause failure (calculated moment = moment capacity) vs. the assumed value of yield stress for the reinforcing steel. Three values of yield stress were assumed, $138 MN/m^2$, as allowed in static design, $276 MN/m^2$, the minimum specified strength, and $345 MN/m^2$, an upper bound to the likely steel strength. As expected, the calculated acceleration to cause failure increases directly with the assumed yield stress. For reasonable values of the yield stress ($276 - 345 MN/m^2$), the analyses suggest that accelerations on the order of 0.35 to 0.5g would be necessary to cause failure. Failures actually occurred at peak accelerations of 0.5g and above (see Fig. 4), values somewhat higher than those indicated from the analyses. However, the peak acceleration does not reflect the time duration of the acceleration, an important variable. In fact, it has been previously suggested that the design acceleration for the Mononabe-Okabe procedure be taken as only 85% of the peak value (5) in order to approach a value of acceleration likely to last long enough to cause movement. This suggestion appears to be borne out by the results of these analyses, except that the Mononabe-Okabe acceleration required to produce the wall failures is only 70% of the peak values.

The one-dimensional shear wave analyses also fully substantiate the viewpoint that the peak acceleration is not a meaningful design parameter. The results of these analyses showed that while the peak acceleration was only a short spike, the acceleration at 70% of the peak occurred for over 0.5 second for all soil profiles studied.

The built-in reserve of strength of the walls produced by the conservative approach taken in the design for static loads is illustrated by the fact it is calculated that at least 0.35g is required to cause damage even though no seismic loadings were explicitly considered in design. The strength reserve comes from the fact that only one half the actual steel yield strength was used in static design of the floodway and a factor of safety (minimum 1.3) was applied in addition to the strength reduction. Clearly it was the hidden strength reserve of the floodways which kept damages to relatively low levels.

CONCLUSIONS

The following conclusions are drawn from the study:

1. Conventional factors of safety used in design of retaining structures for static loadings provide a substantial strength reserve to resist seismic loadings. Peak accelerations of up to 0.5g were sustained by the floodways with no damage even though no seismic loads were explicitly considered in design.

2. The Mononabe-Okabe procedure for calculating earth loadings on retaining walls due to accelerations yields results which are consistent with the floodway performance, if the resultant dynamic load is placed at one-third the wall height from the ground surface and the design acceleration is taken as 70% of the peak value.

ACKNOWLEDGMENTS

The Los Angeles District, U.S. Corps of Engineers provided much of the data for this study. Mr. Jack Bird of the Corps of Engineers gave the authors encouragement and support.

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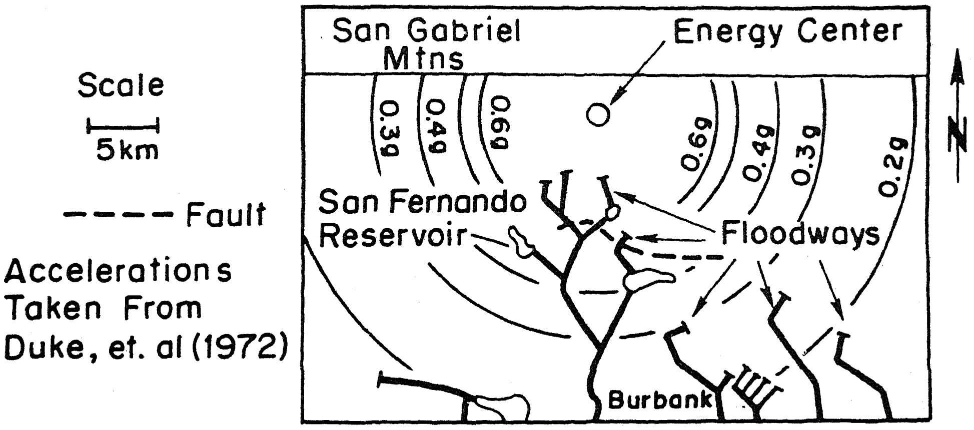

FIGURE 1. LOCATION OF FLOODWAYS AND CONTOURS OF PEAK ACCELERATIONS FROM SAN FERNANDO EARTHQUAKE.

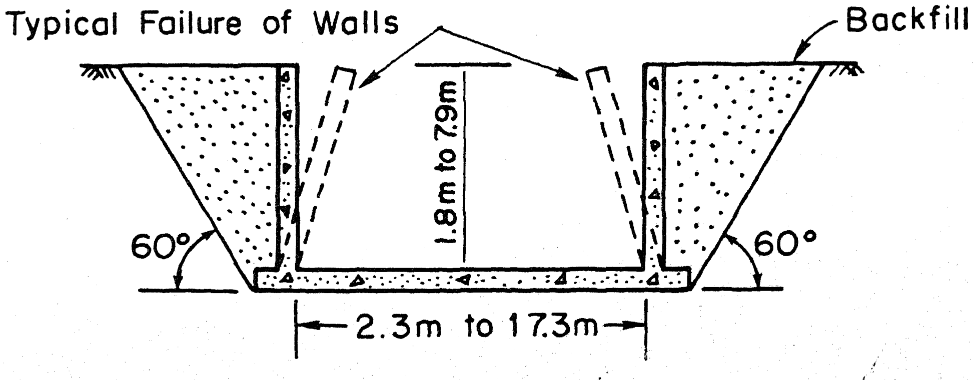

FIGURE 2. SECTION THROUGH OPEN CHANNEL FLOODWAY AND TYPICAL MODE OF FAILURE DUE TO EARTHQUAKE SHAKING.

FIGURE 3. PHOTOGRAPH OF TOPPLED OPEN CHANNEL WALL.

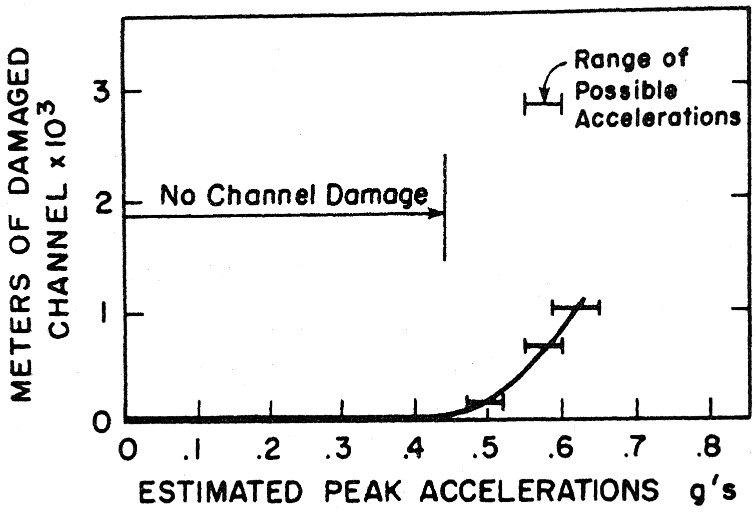

FIGURE 4. RELATIONSHIP BETWEEN CHANNEL DAMAGE AND PEAK ACCELERATIONS.

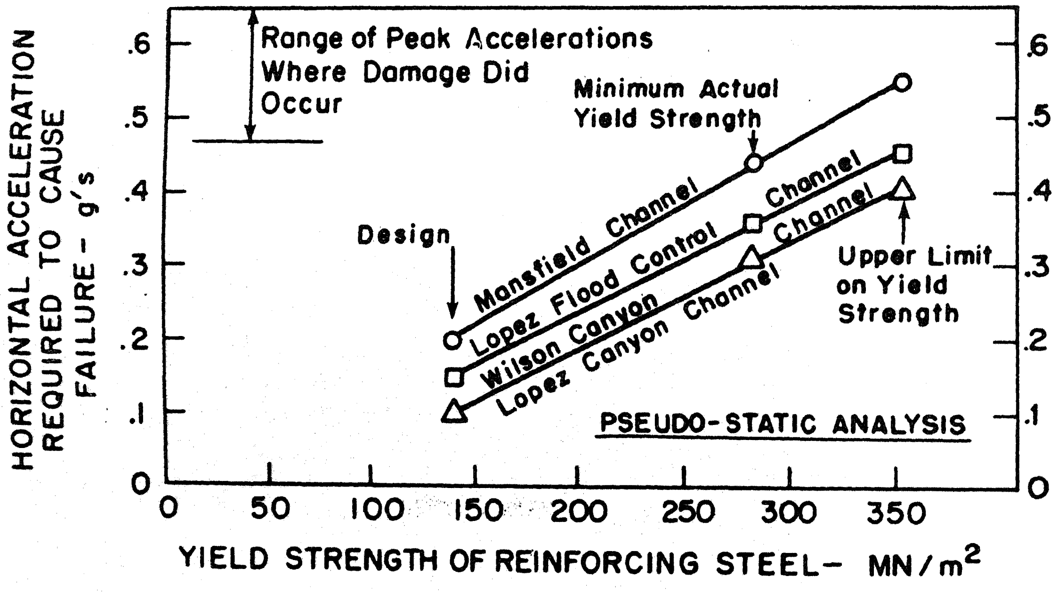

FIGURE 5. HORIZONTAL ACCELERATION REQUIRED TO CAUSE WALL TILTING FOR TYPICAL CHANNELS, PSUEDO-STATIC ANALYSIS.