SOME SPECIAL PROBLEMS IN THE DESIGN OF DEEP FOUNDATIONS

by

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SYNOPSIS

Some of the problems involved in the design of structures using deep foundations are discussed from the standpoint of the design engineer. It is noted that most of our design criteria for structures to resist seismic forces include no parameters for variations in foundation soils or have only crude empirical parameters. A specific example of a large structure founded on asphalt sands in the La Brea Tar Pit area of Los Angeles is given. Another hypothetical project in the Bay Mud area near San Francisco is discussed. The necessity for further research in this field is noted.

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I am sure that almost every engineer designing structures to resist seismic forces frequently encounters conditions that he feels are not properly covered by existing accepted design criteria. In such cases he must rely on his own judgment and "feel" and his design becomes an art rather than a science.

How does the average engineer go about designing a structure for seismic forces? There is usually a building code of some type which incorporates seismic design criteria. He checks the code as a starter. He may feel that the code is a reasonably good one which, if followed, will provide reasonable safety for his structure. Buildings designed under the code may have responded satisfactorily to real earthquakes, but the engineer knows that in some cases this has not occurred and there must be a reason. He has read as much literature on the subject as time will allow, but there are many records and papers that he has not had time to read. He probably knows that no one can write a code that will safely and equitably cover all the conditions that the imagination of owners, architects, and even engineers can produce. In earthquake engineering, he may feel that we have either oversimplified our criteria or have made these criteria unnecessarily complex. Some of us feel that all the parameters that affect the response of buildings are not included in our present criteria and that therefore the designs resulting from the use of code criteria may be either too conservative or not conservative enough. One of these missing parameters in most codes is the effect of foundation soils. Where a soil parameter is used, it is usually highly empirical. This paper deals with one small portion of this problem.

We are told by the soils experts that, given the physical properties of the foundation soils down to, say, bedrock, the response of a column of soil can be computed as for other structural materials. This sounds very simple, but immediately we begin to complicate the problem by the necessity to consider such limitations as:

(1) What is the size and shape of a column of soil under a structure?

(2) What is the effect of boundary conditions on this column of soil?

(3) What is the effect of other adjacent structures?

(4) What are the effects of water conditions on the soil foundation response.

(5) What are the problems involved with liquefaction caused by vibration?

We have noted the results of variations in building structure response caused by variations in quality and depth of underlying foundation soils. We have noted the damaging effects of liquefaction of soils due to earthquake vibration in Japan and in Alaska. We have seen how localized deep alluvial soil deposits affect the ground motion and building response in most of our well-recorded earthquakes, particularly in the recent earthquake in Caracas, Venezuela (June 29, 1967).

Each one of us has probably tried to generalize these foundation problems, but no one so far has been able to put an authoritative quantitative factor for these soils effects into our working criteria. We may say to ourselves that here we have a tall flexible building on deep alluvium and therefore we should be conservative in our earthquake resistant design or that here we have a one-story stiff structure on deep alluvium soils and therefore we do not need to be conservative. Or we can say that we will follow the code strictly and, if anything serious happens and we are sued, we can point to a building code as being the measure of accepted good practice. It is unfortunately true, probably, that this latter point of view is most frequently followed.

So much for generalization. Other papers will cover some of these items in more detail. In this paper I will try to describe some problems involved with structures using deep foundations through varying types of soils with specific reference to a couple of actual projects. But first, let us look at a few major but rather generalized problems by asking ourselves a few questions.

(1) Using the criteria set up in our seismic codes, where is the base of the structure? Where do the earthquake forces get into and out of the structure? About what level does a structure vibrate as a structural unit?

In California, our code authorities generally accept the adjoining ground surface as the "base". But is this realistic? Recent forced vibrations of the new Library Building at the California Institute of Technology indicate that this structure vibrated about the basement. The backfill around this basement is said to have been well compacted. What would have happened if this structure had had two, three, or four basements? What would have happened had the banks been shored and the basement walls poured or gunited directly against the bank? Had the forced vibrations of the Cal Tech Library Building been of much greater magnitude, would this building have still vibrated about the base? Assuming this building did vibrate about the basement level, does this mean that the horizontal earthquake forces were applied to the structure at basement level? I recommend reading "Vibration Test of a Multi-story Building" by Kuroiwa. This is a Cal Tech publication.

(2) If a telephone pole, embedded in the ground so far, resists overturning by a horizontal couple in the ground, does a small plan size structure with deep basement or basements act similarly? Do we have, in such cases, both a horizontal and a vertical couple resisting overturning? If so, how can we evaluate the relative magnitude of such couples quantitatively? I can see a lot of variables in this one question involved with the quality of the surrounding soil. But if we have flag pole or telephone pole action in a multi-story structure, we will be introducing horizontal shears that may be of high magnitude and this is not now covered in our seismic code criteria.

(3) Where a structure is supported by deep piles or caissons, how much should we worry about the relative horizontal displacement of the top of pile or caisson to the bottom of pile or caisson?

This question may be divided into at least two cases. Case I might be where the surface soils are of very poor quality all the way down to the supporting bedrock or firm soils. Case II might be where there is some depth of compacted fill or firm soil at the surface with relatively poor soils between this crust and the deep underlying firm soils.

Now suppose that we ask our soils experts how much relative movement we may get between surface and base of pile or caisson. Suppose also that we get an answer to this. It would appear then that we should design our piles or caissons to accommodate this displacement with end restraint conditions dependent on whether we have Case I or Case II. With small flexible piles this may involve no great problem except recognition that the piles must resist bending and shear and that there may be a vertical load eccentricity. With large diameter caissons this might create a problem. Or, is it possible that with a certain degree of fluidity the soils might flow around piles or caissons like water around a bridge pier? If so, how can we evaluate this phenomenon in our design? I might ask if anyone has ever determined whether caissons or piles have been broken below grade due to this type of action. How well can the structural engineer depend on the soils engineer to evaluate accurately the relative displacement of surface soils to the base of deep piles or caissons for a specific case? Perhaps this will be answered by other papers at this Conference, but to me it appears that the structural engineer will need to exercise a good deal of judgment in most cases and that he should probably pray that his assumptions are somewhere in the ball park.

Our office has a project under design at the time this paper is written which involves some of the problems discussed above. It is a project in the area of the La Brea Tar Pits in Los Angeles. There is a tower some 33 stories high. There are three basements below the plaza level. The area at and below the plaza level is much greater than the tower area. A cross-section through this structure is shown on Plate 1. As shown on this cross-section, the surrounding soils have a reasonably good top crust some 30 feet thick of clay and silt. Below this mantle is some 40 odd feet of asphaltic sand. This asphaltic sand is a stiff viscous material. A deep hole drilled today will partially close tomorrow. To drive steel piles through this material with heavy equipment takes over 400 blows per foot unless sub-drilling is permitted. In ordinary soils this would be called refusal and the piles stopped. But because of the slow liquid movement of this material with time, we did not feel that founding friction piles in this material would be definitely safe and founded the piles in the stiff clays down below and counted only the frictional resistance in these lower clays. We allowed subdrilling down to the stiff clay level. Extensive soil investigation was made. A long narrow pit was dug down to lowest basement level. The pit was a block long so as to work in a ramp for a pile driver to drive test piles. Sides of the pit were excavated at various slopes to determine the slope at which they would stand, with and without sun. We were able to observe how heavy equipment could operate on the asphalt sands. The pile driver needed heavy timbering under it to stay afloat. Now, what did we do about some of the questions previously asked?

Where is the base? We noted the 30 feet of upper crust. We followed code in assuming the base at surface level but hedged this by carrying our moment-resisting tower frame down one additional story. We further hedged this by providing shear walls from plaza level down, overlapping the moment-resisting frame by a story. The steel tower frame is carried all the way down although most of the basement area outside the tower is concrete frame.

Are we worried about bending in the piles? In this case we feel that the asphalt sands are so slow moving as compared to probable ground vibration that there will probably be very little lateral displacement between the upper clay silt material and the lower stiff clays. We hope this is true, but we are insisting on some moment capacity in the piles.

How did we consider overturning to be resisted? The tower structure overturning is resisted by vertical couples as determined by assuming the base at surface level. The horizontal shears are assumed to be co-planer at base level, but the added shear walls were felt to provide a factor of safety here in case this assumption is not correct.

Plates II and III illustrate a condition involving a similar basic problem but greatly different in quality of soils. This is a situation that could happen in the San Francisco Bay Area. Plate II involves a case where a short stiff structure is to be constructed on about 20 feet of fill over soft bay mud varying in depth from say 10 feet to 55 feet. Below this is a hard, silty clay. It is proposed to drive piles through the bay mud and into the hard, silty clay to support the structure. This bay mud is a great deal more fluid than the asphaltic sands of the La Brea Tar Pits area. Also, there is a crust of fill on top that has some compaction. Piles here will be restrained to some degree top and bottom.

The question arises as to the relative movement of the ground surface and the hard silty clay strata. Can this be accurately computed? Let us suppose that the bay mud peters out on one side and the hard clay rises to the surface. Does this condition restrict the relative movement of surface fill in one direction but not in the other? Can we estimate this quantitatively with any degree of accuracy? If we can estimate this movement we can compute the bending moments and shears in the piles. To complicate the problem, suppose that the bay mud runs on for miles or even runs into the ocean on one side. During a severe earthquake, would this boil out into the sea? Plate IV illustrates such a possible condition.

Plate III illustrates a condition where the bay mud is confined on both sides as in a valley. The surface width of the valley may vary from a quarter mile to five miles. Does this make a difference in relative movement of upper fill and lower clay? Can we evaluate this today? Now what does this condition do to the response of our structure? Our experience would indicate that the movement of the surface fill would be of a greater amplitude than the hard, silty clay with increased period of ground motion. Would it be conservative to assume that this would have a favorable effect on short, stiff structures but an adverse effect on tall flexible structures? Can we evaluate this quantitatively?

Now let us suppose that there were no fill crust on top of the bay mud and we supported a structure on piles with a reinforced concrete, self-supporting floor. We now have piles cantilevering more or less up from the hard, silty clay. The structure may move relative to the bay mud and we may have a really flexible basement story.

Model Testing.

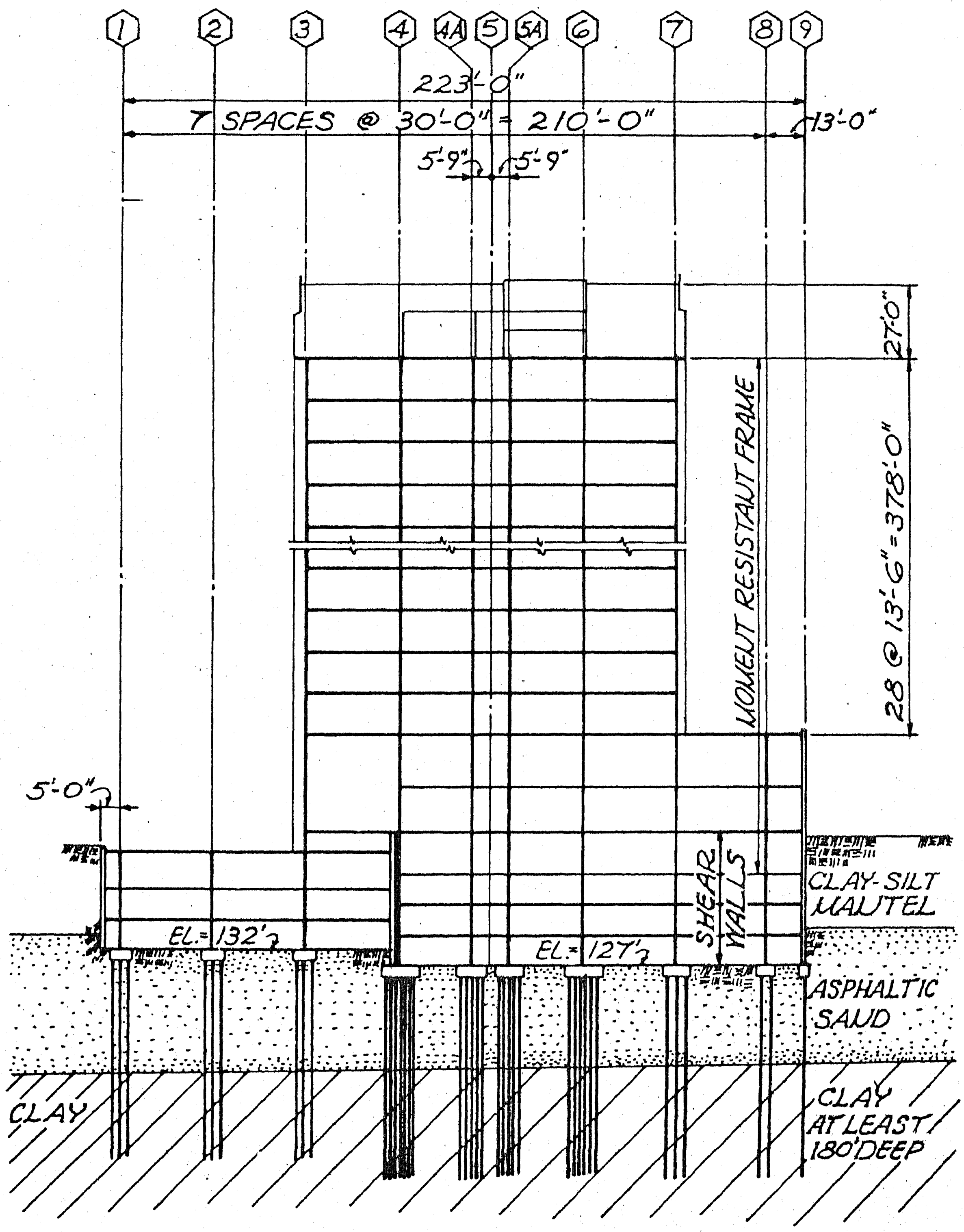
It is possible that area soil conditions might be modeled at some reduced scale and the model placed under forced vibration. Whether soils could be scaled down in size and still retain physical properties that could reasonably simulate actual conditions is questionable, but might be worthy of further study. Large scale models would, of course, be expensive. The proposed Large Scale Earthquake Simulator Facility at the University of California might be specially adapted to soil structure response. In the Feasibility Study[i] for this facility I find a comment as follows:

> "Experimental Program on Soil Structures and Foundations
>
> A shaking table could be employed to investigate the effects of earthquakes on soil structures and foundations. In particular, it would be of direct benefit to research in the following fields:
>
> 1. Soil Properties--Analytical research on the effects of strong earthquakes on soil structures has been limited by a lack of knowledge about stress-strain relationships for both cohesive and non-cohesive soils.
>
> 2. Soil Liquefaction--The phenomenon of soil liquefaction caused major damage in Niigata, Japan, during the 1964 earthquake.
>
> 3. Stability of Embankments--The damage caused by landslides during the 1964 Alaskan earthquake dramatically emphasized the need for research in the stability of both natural and man-made embankments during earthquakes.
>
> 4. Soil-Structure Interaction--The effects of soil-structure interaction during earthquakes is a field of research currently receiving much attention.
>
> 5. Response of Soil Deposits--More information is required on the response of non-homogeneous soil deposits to earthquake motion."

I also have heard that John Blume is contemplating the use of scale model shaking devices to assist in the analysis of soils effects on specific projects. Such soils or foundation shaking tests might give results which would assist in mathematical solutions also.

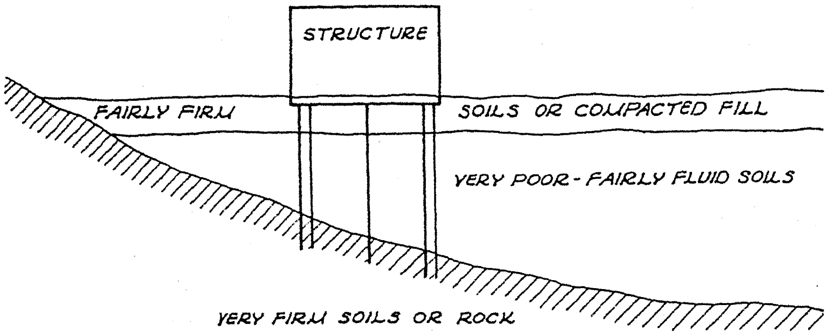
As a final comment, it would appear that much further research in the field of the effect of deep foundations on the response of structures subjected to earthquakes is vitally needed.

(i) "Feasibility Study, Large-Scale Earthquake Simulator Facility" by J. Penzien, J. G. Bouwkamp, R. W. Clough, and Dixon Rea. Report No. EERC-67-1, September 1967, College of Engineering, University of California, Berkeley, California

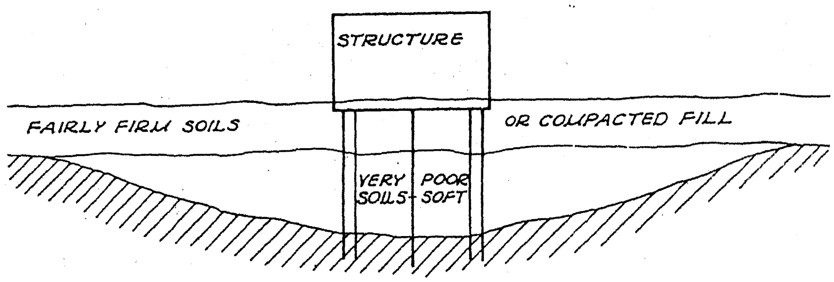


CROSS-SECTION THROUGH TOWER BUILDING

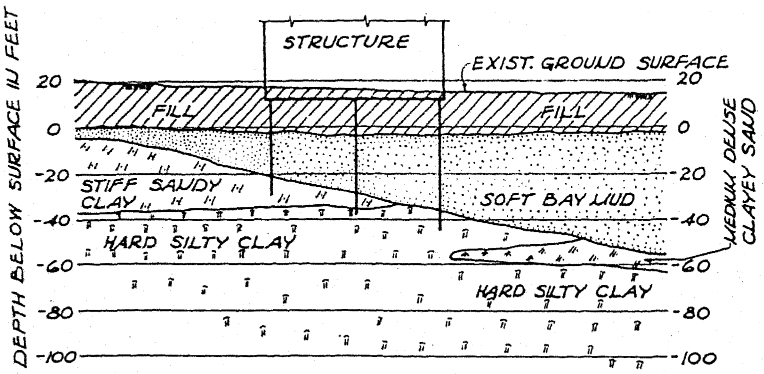
PLATE I



POOR MATERIAL
UNCONFINED LATERALLY
PLATE II



POOR MATERIAL
CONFINED LATERALLY
PLATE III



CROSS-SECTION OF HYPOTHETICAL
CASE, UNCONFINED ONE SIDE
PLATE IV