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Supplement 1

EXPANDED DESCRIPTION OF SOIL PRESSURE ANALYSES

SUPPLEMENT NO. 1 TO

ADDITIONAL INVESTIGATIONS TO DETERMINE THE EFFECTS OF COMBINED VIBRATORY MOTIONS AND SURFACE RUPTURE OFFSET DUE TO AN EARTHQUAKE ON THE POSTULATED VERONA FAULT

prepared for

GENERAL ELECTRIC COMPANY
Vallecitos, California

27 June 1980

OFFICIAL SEAL
VIRGINIA C. CASQUEIRO
NOTARY PUBLIC - CALIFORNIA
ALAMEDA COUNTY
My comm. expires MAR 8, 1981

EDAC
ENGINEERING DECISION ANALYSIS COMPANY, INC.
480 CALIFORNIA AVE., SUITE 301
PALO ALTO, CALIF. 94306
BURNITZSTRASSE 34
6 FRANKFURT 70, W. GERMANY

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INTRODUCTION

This report presents an expanded discussion of the soil pressure analyses described on pages 2 and 3 of Reference 1, and has been prepared in response to a request received from the USNRC (Reference 2). The soil pressure analyses were performed to determine the physical load limits on the combined load case comprised of a ground acceleration vibratory motion and a surface rupture offset, the latter represented analytically as an "unsupported length" of the building (see Figure 1). Note that in Figure 1, and in subsequent figures, the plane of the offset has been shown as being vertical for illustration purposes only. Per Reference 5, the postulated dip angle varies from 10 to 45 degrees.

SOIL PROPERTIES

Properties of the foundation soils beneath the GETR reactor Building have been described previously in Reference 3 (Page 3-6, Table 3-1, and Figure 3-4). Table 3-1 and Figure 3-4 are reproduced in Appendix A of this report for easy reference. Based on these properties, an ultimate bearing capacity of 20ksf was developed as described in Appendix B.

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SOIL PRESSURE ANALYSES

A series of analyses of soil pressures under the Reactor Building was performed for different combinations of one component of horizontal ground acceleration and "unsupported length". These soil pressures are produced by the vertical weight of the structure and the overturning moment produced by horizontal seismic forces. Effects of other ground motion components are discussed near the end of this section. Soil ressures in the region of the edge of the supporting soil (Figure 1) we examined due to the application of vertical dead load and static lateral inertial forces. The inertial forces were determined from the previously performed linear dynamic analyses. To simplify the computational process, the foundation was assumed to be an equivalent rectangular plate (in plan). The procedures used to calculate the soil pressure distribution were the conventional methods for foundation design as described in Reference 4, relevant excerpts of which are reproduced in Appendix C.

In these analyses, "incipient local yielding" was defined as the loading combination which produces bearing pressure at the edge of the supporting soil equal to the ultimate bearing capacity (20 ksf). The soil pressure diagram for this situation is illustrated in Figure 2a for the example case of an unsupported length, Lc, of 13 ft. For this example, yielding in the soil begins at a horizontal ground acceleration of about 0.26g. The local mode of failure of the soil at the edge of the offset is shown schematically in Figure 3. In this case, rapid loading which produces a soil pressure slightly less than the ultimate bearing capacity will cause the soil to deform as shown. Analyses were also performed for several other cases of unsupported length, and the horizontal earthquake accelerations at which incipient yielding in the soil occurs were plotted versus unsupported lengths as the lower edge of the band in Figure 4. Thus, loading combinations of horizontal earthquake, gravity load, and unsupported length at which incipient local soil yielding will occur are shown graphically by the lower edge of the band in Figure 4. Load combinations greater than those at the lower edge of the band will induce additional soil yielding at the edge of the offset, which will result in the structure settling so as to be supported continuously or simply supported by the soil to the left of the offset zone in Figure 1.

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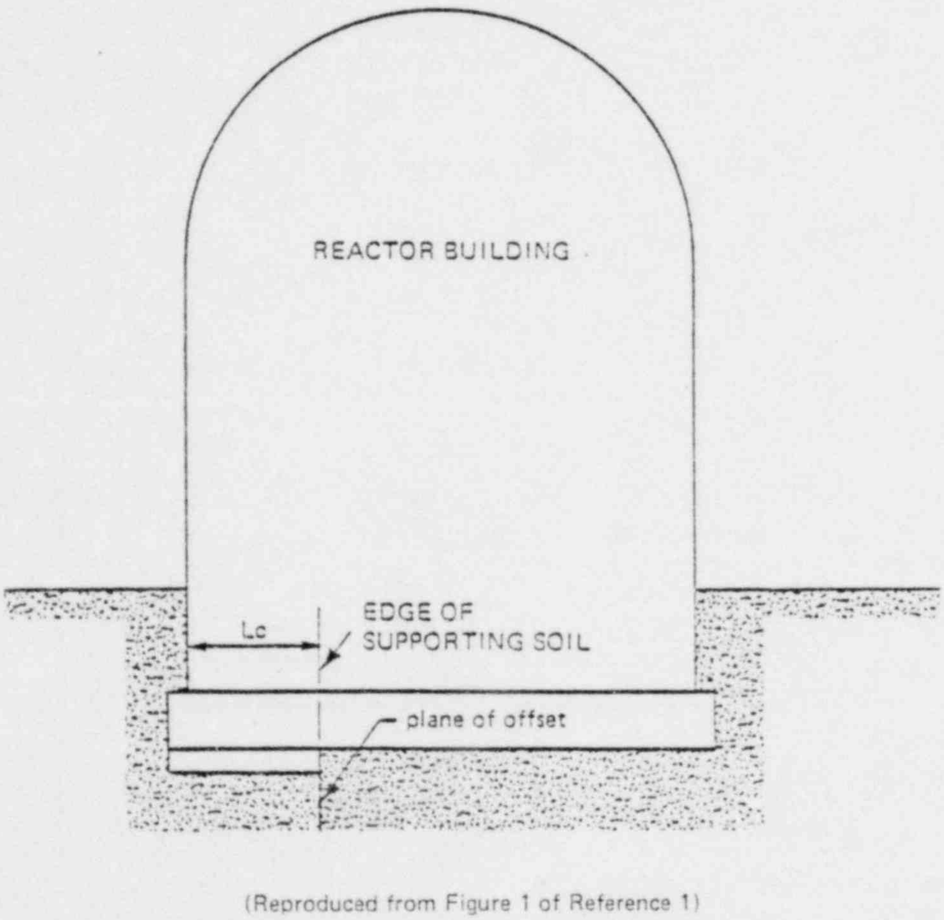
As the horizontal ground acceleration, and thus the over-turning moment caused by the lateral force, increases above the value at which incipient local yielding occurs, the region of local soil yielding will be represented by a pressure diagram as shown in Figure 2b. Finally, an upper limit on local soil pressures can be represented as shown in Figure 2c. For the selected example of 13 ft. unsuppported length in Figure 2c, the maximum ground acceleration is about 0.38g. For this case, the mode of deformation of the soil in the region of the edge of the offset is shown schematically in Figure 5. Rapid loading of the soil at pressures equal to the ultimate bearing capacity will induce movement of the soil as shown. Analyses were also performed for other cases of unsupported length, and the resulting horizontal accelerations at complete local soil yielding were plotted versus unsupported length as the upper edge of the band in Figure 4. The upper edge of the band in Figure 4 is a conservative estimate of the bound on complete local soil yielding in the region of the edge of the offset, at which point the structure will have completely settled down and be supported by the undisturbed soil to the left of the offset zone in Figure 1.

As discussed in the beginning of the text of this section, for simplicity in the calculations, the soil pressure analyses were performed for one component of horizontal ground acceleration. Subsequent analyses showed that inclusion of the vertical acceleration component will change the vertical amplitude of the band (ground acceleration) in Figure 4 by less than plus or minus five percent. In addition, inclusion of the second horizontal component will lower the band in Figure 4 by about 7 percent.

CONCLUSIONS

The soil pressure analyses described in this report demonstrated that there are physical limits on the combined loading of vibratory motion and "unsupported length", the latter of which is the selected analytical representation of the postulated surface rupture offset. Based on these soil pressure analyses, it was concluded that the structure will settle down for all load combinations above the band on Figure 4. Partial or complete settling down of the structure are conditions which can be easily tolerated without distress in either the soil or the structure. Only those load combinations which are below the band of Figure 4 actually need be considered in the structural evaluations.



(Reproduced from Figure 1 of Reference 1)

FIGURE 1. HYPOTHETICAL "UNSUPPORTED LENGTH," Lc

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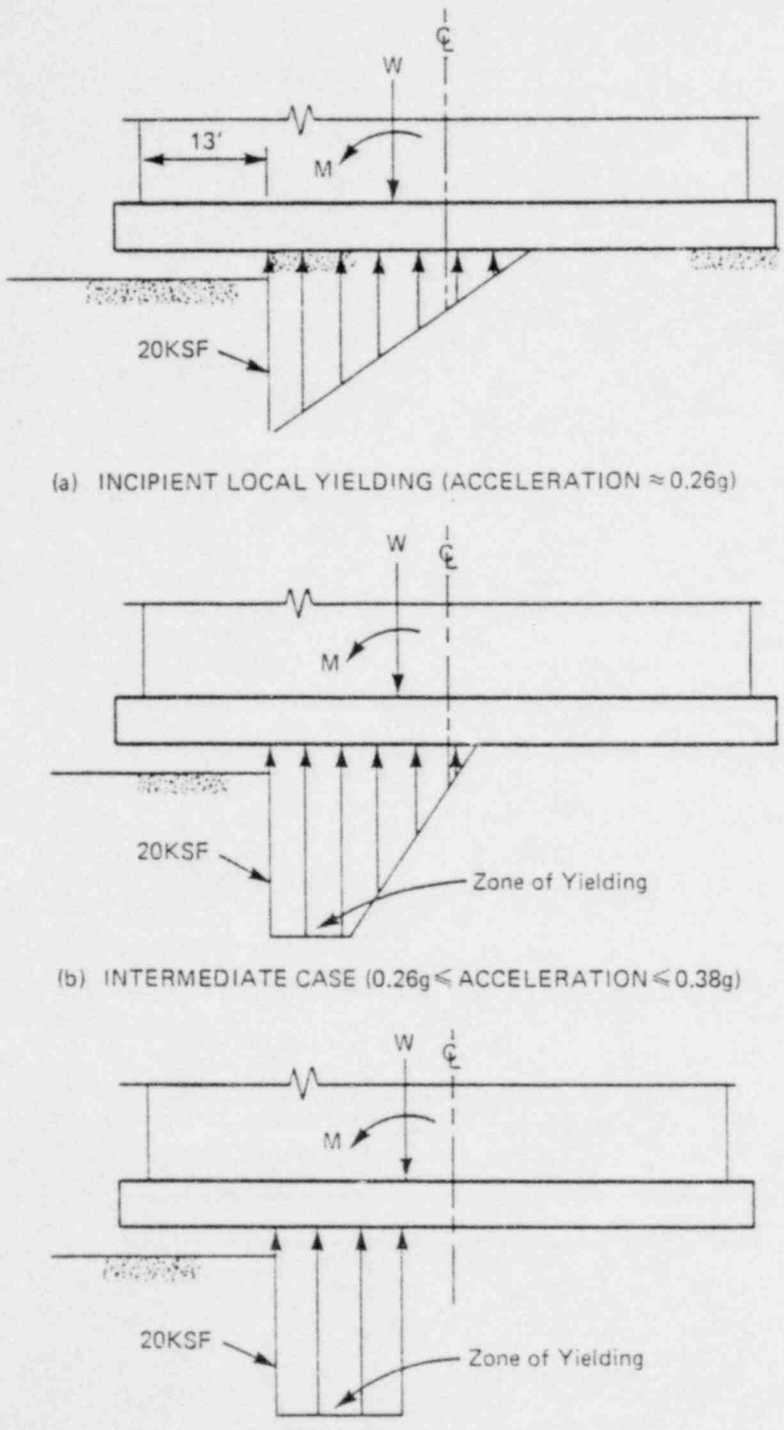

(a) INCIPIENT LOCAL YIELDING (ACCELERATION ≈ 0.26g)

(b) INTERMEDIATE CASE (0.26g ≤ ACCELERATION ≤ 0.38g)

(c) UPPER LIMIT ON LOCAL YIELDING (ACCELERATION ≈ 0.38g)

FIGURE 2. SOIL PRESSURE DISTRIBUTIONS (For Example Case of 13 ft Unsupported Length)

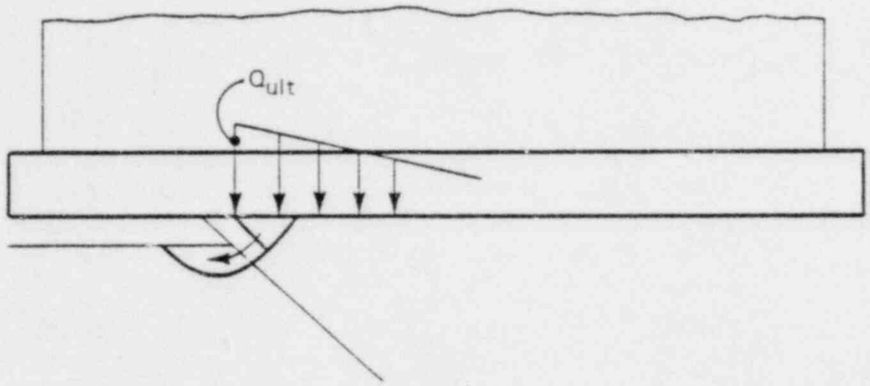

FIGURE 3. LOCAL MODE OF FAILURE OF SOIL AT EDGE OF OFFSET – INCIPIENT LOCAL YIELDING

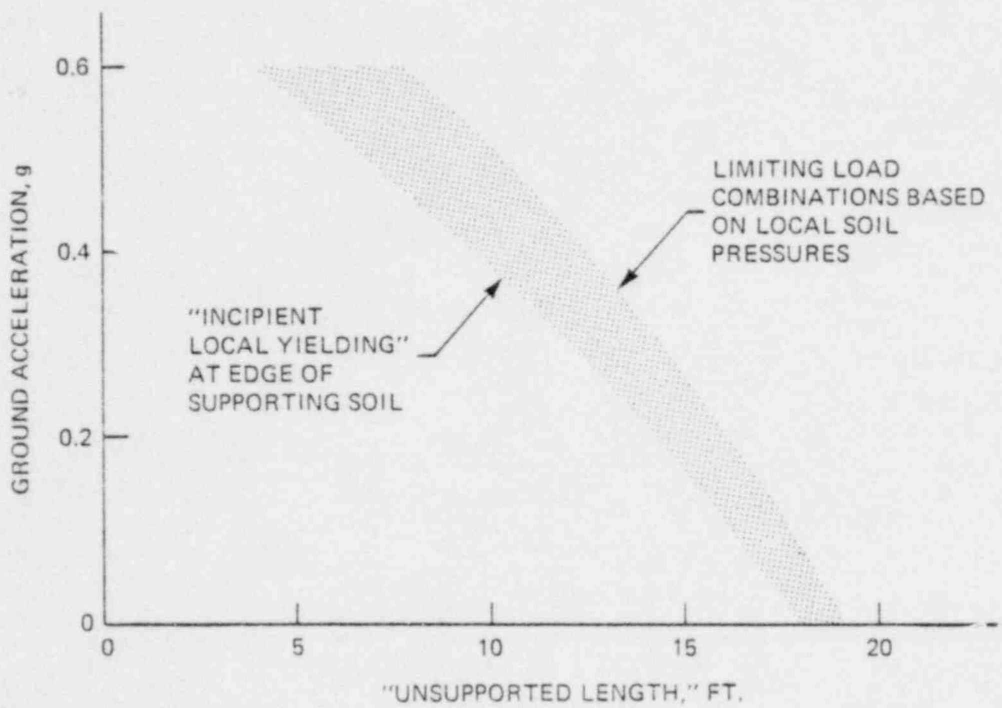

FIGURE 4. RESULTS OF SOIL PRESSURE ANALYSES

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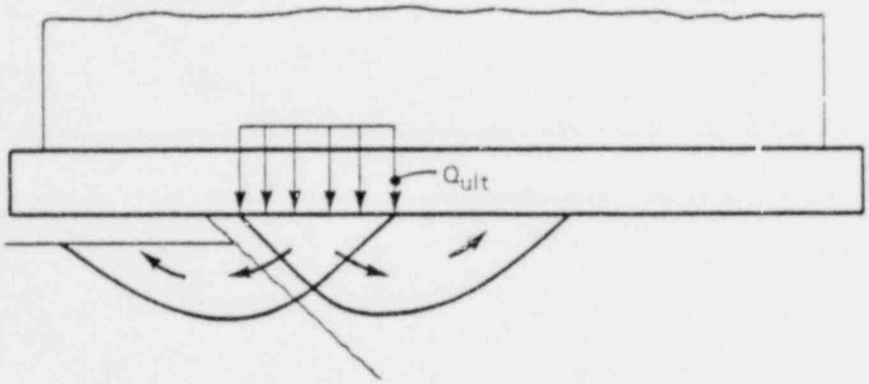

FIGURE 5. DEFORMATION OF FOUNDATION SOILS

REFERENCES

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REFERENCES

1. Engineering Decision Analysis Company, Inc., "Additional Investigations to Determine the Effects of Combined Vibratory Motions and Surface Rupture Offset Due to an Earthquake on the Postulated Verona Fault," EDAC-117-253.01, Revision 1, prepared for General Electric Company, 8 May 1980.

2. USNRC (R. A. Clark) Letter to General Electric Company (R. W. Darmitzel) 10 June 1980.

3. Engineering Decision Analysis Company, Inc., "Seismic Analysis of Reactor Building, General Electric Test Reactor - Phase 2," EDAC-117-217.03, prepared for General Electric Company, 1 June 1978.

4. Kramrisch, F., "Handbood of Concrete Engineering, Chapter 5 Footings," Ed. by M. Fintel, Van Nostrand Reinhold Co., 1974.

5. USNRC (D. G. Eisenhut) Letter to General Electric Company (R. W. Darmitzel) 23 May 1980.

APPENDIX A

PROPERTIES OF FOUNDATION SOILS

(Reproduced From Reference 4)

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TABLE 3-1

STRUCTURAL AND FOUNDATION MATERIAL PROPERTIES*

Structural Concrete Properties	Concrete Type 1 (Ordinary)	Concrete Type 2 (Magnetite)	Concrete Type 3 (Ferrophosphorus)
Unit weight, γ	150 lb/ft^3	225 lb/ft^3	280 lb/ft^3
Modulus of elasticity, E	3.83×10^6 psi	7.04×10^6 psi	9.78×10^6 psi
Compressive strength, f'_c	5,400 psi	3,400 psi	5,000 psi

Soil Properties Beneath Reactor Building*	Soil Type 1	Soil Type 2
Moisture content, w	13 percent	15 percent
Dry unit weight, γ_d	120 pcf	117 pcf
Average total unit weight, γ_T	135 pcf	135 pcf
Standard penetration resistance, N	50 - 100 blows/ft	50 - 100 blows/ft
Shear modulus, G	1.1×10^6 psf**	2.4×10^6 psf**
Shear velocity, V_s	500 fps**	750 fps**
Percent critical damping, λ	11 percent**	11 percent**

*Soil properties are averages based on Shannon and Wilson 1973 data (Ref. 10) and published correlations.

**At strain of 0.1 percent.

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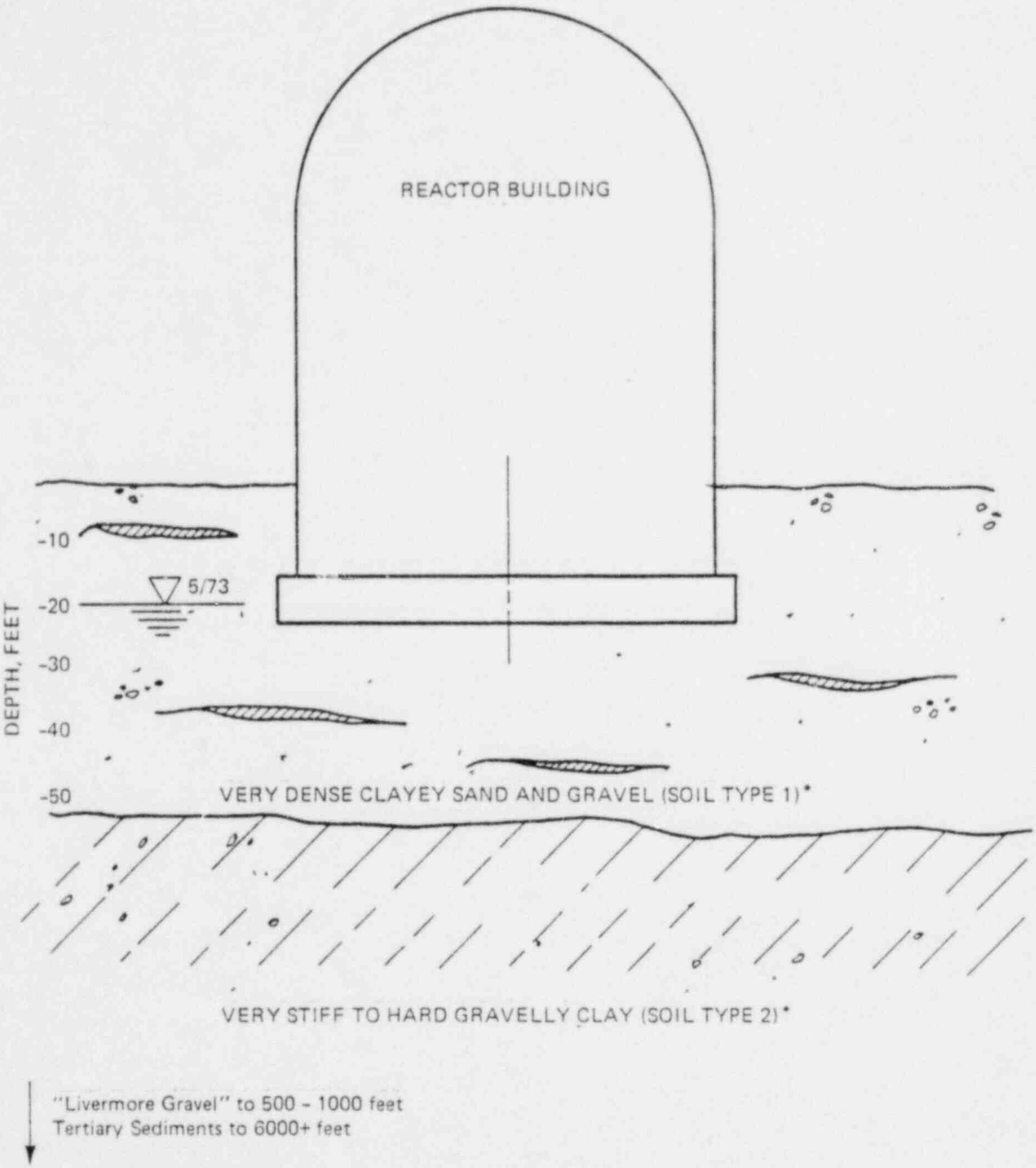

*See Table 3-1 for Soil Properties

FIGURE 3-4. REACTOR BUILDING FOUNDATION SOIL CONDITIONS

APPENDIX B

DISCUSSION OF FOUNDATION SOIL PROPERTIES

(Prepared by Richard L. Meehan, Earth Sciences Associates)

DISCUSSION OF FOUNDATION SOIL PROPERTIES

INTRODUCTION

Postulated simultaneous or near-simultaneous fault rupture and seismic ground shaking could cause relatively high local pressures on soils benheath the foundation slab of the GETR Reactor Building structure. Because the underlying soils are weaker than the concrete of the slab, yielding of the soils will occur when the pressures reach a certain limiting value. This limiting value then provides an upper bound on the pressures which may exist beneath the slab. If the structure imposes greater loads on the foundation, plastic yielding of the underlying soil will occur until the supporting soil area increases sufficiently so as to reduce the pressures to the limiting value. The point at which plastic yielding occurs depends upon the type of soil beneath the slab and the manner in which the load is applied (rapidly or slowly, locally or over a broad area).

SOIL CONDITIONS BENEATH GETR

Knowledge of soil characteristics beneath GETR comes from three sources:

1. General knowledge of the characteristics of Livermore Formation soils, known from recent trenches, borings, and geologic mapping in the general vicinity of GETR.

2. The following reports:

    a. Shannon and Wilson, Inc., 1973, Investigations of Foundation Conditions, G.E. Test Reactor. This investigation included two 70-ft. borings drilled near the reactor, various laboratory tests including triaxial strength tests, and an evaluation of bearing capacity under cyclic loadings.

b. Dames and Moore, 1960, Foundation Investigation Proposed Boiling Water Reactor, Vallecitos Atomic Laboratory. This report presents results of borings and tests for a different facility near the GETR site. Earth Sciences Associates (ESA) geologists believe that geological foundation conditions are similar at this other site, and that test data are generally applicable to the GETR foundation.

The 70-ft. diameter GETR foundation slab is founded about 20 ft. below grade, and rests on very dense clayey sand and gravel with the following trypical properties:

Water Content: 13 percent
Dry Density: 120 pcf
Standard Penetration N: 50-100 blows/ft.

Below a depth of about 50 ft., very stiff to hard gravelly clay is encountered. According to the Shannon and Wilson report, the water table is at or near foundation level, 20 ft. below ground surface.

LOADING CONDITIONS

The following sequence of loading conditions is postulated:

1. Approximately 1 meter of fault rupture occurs beneath the reactor. Most of this movement is postulated to occur in several seconds, with perhaps the last few centimeters extending over a period of minutes.

2. Nearly simultaneously, shaking of the ground occurs which causes vibration and cyclic loading of the reactor structure and foundation soils. These vibratory loadings are superimposed on the fault rupture condition.

This suggests that the peak loads of concern from a structural standpoint will be applied within a few seconds, rapidly enough so that pore pressures within the soil volume beneath the slab affected by high foundation loads -- a volume with dimensions of tens of feet -- will not dissipate by drainage. Thus, the soil loading will be in an undrained condition.

SOIL STRENGTH

The following strength parameters are from the Shannon and Wilson report:

Minimum strength:

$\bar{c}_{cu} = 1000$ psf

$\phi_{cu} = 16.5°$

Maximum strength:

$\bar{c}_{cu} = 1400$ psf

$\phi_{cu} = 31.5°$

Plots of the Dames and Moore data yield the following average results:

$\bar{c}_{cu} = 200$ psf

$\phi_{cu} = 22°$

The shear strength at a confining pressure of 4500 psf (the pressure on the soil beneath the slab before the fault or shaking loads occur) is as follows for the three sets of strength parameters:

S (S+W minimum): 2332 psf

S (S+W maximum): 4157 psf

S (D+M) : 2018 psf

ULTIMATE BEARING CAPACITY

Shannon and Wilson conclude that for earthquake-induced pressure concentrations beneath the foundation mat, the ultimate bearing capacity is controlled by the soils at shallow depth beneath the mat. Based on their minimum value of $C = 0.5$ psf and $\phi = 16.5$ degrees, they compute a bearing capacity from the Terzaghi formula of 20,000 psf. The loading condition they visualized is basically similar to the loading condition under consideration here.

Alternatively, ESA has considered the problem as one of rapid loading of soil, initially confined to a pressure of 4500 psf, whicn has a mean shear strength of 3300 psf, a value intermediate between Shannon and Wilson's minimum and maximum values.

For this approach, the Terzaghi bearing equation simplifies to:

$$Q_{ult} = sN_c$$

With $N_c = 6$, ultimate bearing capacity is 20,000 psf.

ESA therefore recommends use of the value of 20,000 psf as suitable for the loading and soil deformation condition shown on Figures 3 and 5 in the main body of this report.

APPENDIX C

EVALUATION OF BEARING PRESSURES AT FOOTING BASES

(Reproduction from Reference 5)

superstructure; he must assume the appropriate safety factor to arrive at the allowable bearing pressure; and finally, decide on the most economical type of foundation to be used. For this reason it is essential for a foundation engineer to possess a good knowledge of the problems that are involved in the design and behaviour of the superstructure, a certain familiarity with the basic principles of soil mechanics, and a good understanding of the interaction between both.

A detailed treatment of above topics does not fall within the scope of this handbook; however, a short discussion of the basic considerations affecting the evaluation and distribution of the bearing pressures under footing bases is given below.

5.2 EVALUATION OF BEARING PRESSURES AT FOOTING BASES

5.2.1 General Principles

The distribution of the bearing pressures under a concentrically loaded, infinitely stiff footing, with frictionless base, resting on an ideal, cohesionless or cohesive subsoil,[1,2] is generally known, and shown in Fig. 5-1. Under ordinary conditions few soils will exhibit such a behavior; no footing could be considered to be infinitely stiff. The distribution of the bearing pressure under somewhat flexible footings and ordinary soil conditions will be similar to those shown in Fig. 5-2; or it may assume any intermediate distribution. The assumption of a uniform bearing pressure over the entire base area of a concentrically loaded footing, as shown in Fig. 5-3, seems to be justified, therefore, for reasons of simplicity, and is common design practice. This assumption not only represents an average condition, but is usually on the safe side because most of the common soil types will produce bearing pressure distributions similar to that shown in Fig. 5-2a. The foundation designer, however, shall keep in mind that the assumption of a uniform bearing pressure distribution was primarily made for reasons of simplicity and may, in special cases, require adjustment.

Any footing that is held in static equilibrium solely by bearing pressures acting against its base has to satisfy the following basic requirements regardless of whether it is an isolated or a combined footing:

1. The resultant of all bearing pressures, acting against

Fig. 5-1 Bearing pressure distribution for a stiff footing with frictionless base on ideal soil. (a) On cohesionless soil (sand); and (b) on cohesive soil (clay).

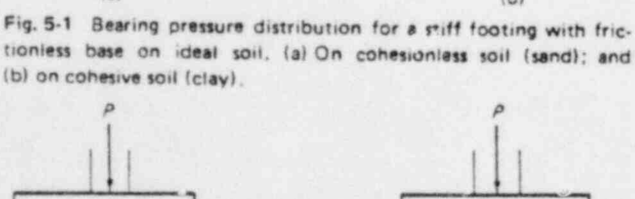

Fig. 5-2 Bearing pressure distribution for a flexible footing on ordinary soil. (a) On granular soil; and (b) on clayey soil.

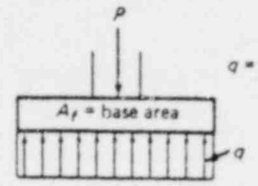

Fig. 5-3 Simplified bearing pressure distribution (commonly used).

the footing base (reaction), must be of equal intensity and opposite direction as the resultant of all loads and/or vertical effects due to moments and lateral forces, acting on the footing element (action).

2. The location where the resultant vector of the reaction intersects the footing base must coincide with the location where the resultant vector of the action is applied. Action and reaction are as defined under (1) above.

3. The maximum intensity of the bearing pressures under the most severe combination of service loads must be smaller than, or equal to, the maximum bearing pressure allowed for this kind of loading and type of soil, as determined by principles of soil mechanics.

4. The resultant vector of the least favorable combination of vertical loads, horizontal shears, and bending moments that may occur under service load conditions, including wind or earthquake, must intersect the footing base within a maximum eccentricity that will provide safety against overturning.

The method most commonly used for the design of footings and related elements for ordinary building construction, is the one where static equilibrium is obtained by bearing pressures against the footing base only. This method is also the standard method that has been included in the "Building Code Requirements for Reinforced Concrete" ACI 318-71.

For zero eccentricities, the bearing pressures will be uniformly distributed over the entire base area of the footing as shown in Fig. 5-3 and will have the intensity of $q = P/A_F$.

If the footing shall restrain the column base, i.e., if a bending moment has to be resisted by the subsoil alongside with a concentric load, or if the column load is applied outside of the centroid of the base area of the footing, the bearing pressure distribution will vary depending on the magnitude of the eccentricity and its relationship to the kern distance c_k. The kern distance can generally be evaluated as shown in Fig. 5-4.

When the eccentricity is equal to, or smaller than, the kern distance c_k, the extreme (maximum or minimum) bearing pressures q_{min}^{max} can be found by superposing the flexural bearing pressures over the axial bearing pressures, see Fig. 5-5a.

When the eccentricity becomes greater than the kern distance superposition cannot be applied anymore, because it would result in tensile stresses between soil and footing near the lifted edge of the base. Equilibrium can, however, be attained by resisting the load resultant by a bearing pressure resultant of equal magnitude and location. In this case the extreme bearing pressures at the edge of the base can be evaluated as shown in Fig. 5-5b. The maximum edge pressure q_{max} must, under all conditions, be smaller or equal than the maximum allowable soil pressure, q_a.

This condition applies until the excentricity, e, of the load, P, reaches the edge of the footing base. Any greater eccentricity will result in overturning. Such a condition, however, can only occur on rock or on very hard, stiff soils. For most practical cases, edge-yielding can make a footing

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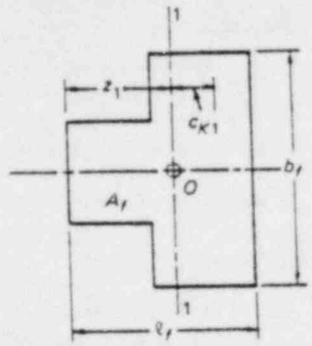

Fig. 5-4 Kern distance. (a) $c_{k1} = I_{F1}/A_{F1}z_1$; (b) I_{F1} = moment of inertia of footing base about neutral axis I-I; (c) z_1 = distance of extreme fiber at opposite side of desired kern distance; (d) for strips, $c_k = l_f/6$.

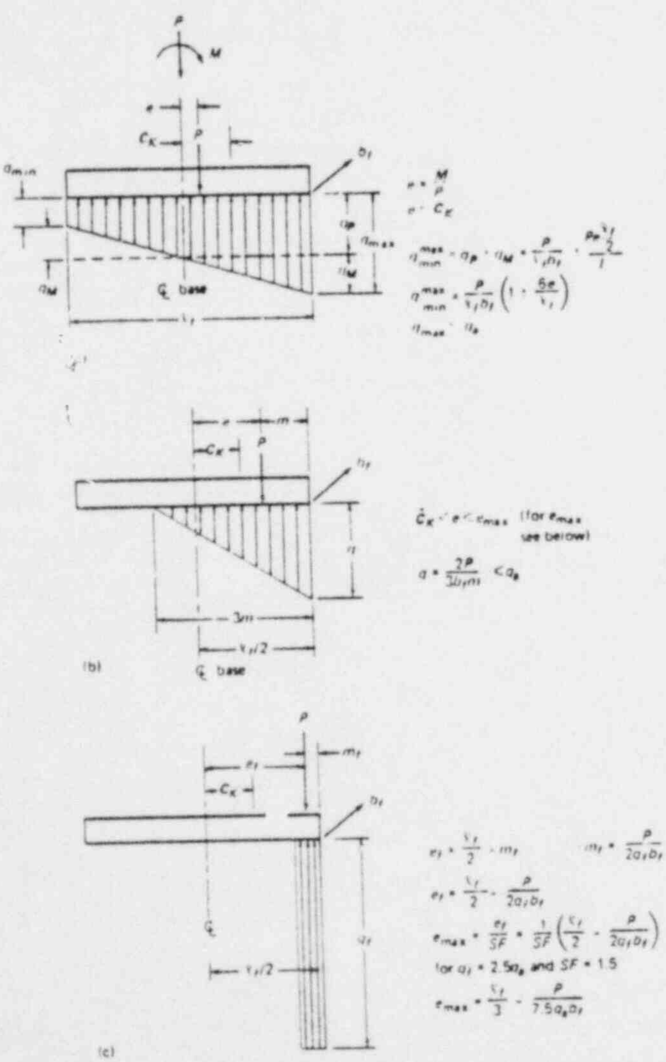

Fig. 5-5 Bearing pressure distribution under eccentric loading.

unusable and produce a condition that is equivalent to overturning. Edge-yielding will occur when the extreme bearing pressure at the pressed edge will cause failure in the bearing capacity of the subsoil. The eccentricity causing this condition will, therefore, limit the maximum useful eccentricity. Unless actual test results are available, the failure condition in the bearing capacity of the soil, q_f, can be assumed with about 2.5 times the allowable bearing capacity, q_a; the minimum safety factor against overturning is usually specified as 1.50; although somewhat greater safety factors are sometimes desirable. Introducing these requirements, we arrive in Fig. 5-5c at a maximum eccentricity, e_{max}, that can safely be utilized. How far the design engineer will take advantage of this condition will depend on his judgment of the soil and on the sensitivity of the superstructure to tolerate lateral tilting that may occur if a loading, causing such an eccentricity, is applied for a longer period.[5-3]

Moments occurring alongside with concentric loads, may be uniaxial or biaxial. If they occur in oblique directions it is most practical to have their influence divided into two perpendicular components, each of them parallel to the main axes of the footing mat, and superpose the resulting bearing pressures. Such conditions occur not only with isolated spread footings, but also with strip footings of limited length, as in the case of shear walls and similar.

Combined footings, (i.e., footings supporting more than one column load, such as exterior double-column footings), strip footings supporting spaced column loads, rafts, or mats can be designed as described above, as long as the entire foundation can be considered as infinitely stiff. In this case the resultant of all bearing pressures must be equal to the resultant of all loads, and its location must coincide with the eccentricity of the resultant. This approach is statically correct, but not necessarily close to the actual condition.

In certain cases it may be advisable to consider the footing as a beam on an elastic foundation and utilize the elasticity of the footing mat as well as that of the soil in the evaluation of the bearing pressures. The bearing pressures obtained by this method no longer follow a straight line distribution across the contact area. They show maximum accumulations immediately below, and in the vicinity of, concentrated loads and greatly reduced intensities between them, as shown in Fig. 5-6. Such a pressure distribution reduce the maximum design moments of a foundation considerably and is therefore in many cases quite economical. This method is, in addition, intriguing in its setup and appealing especially to the mathematically inclined engineer.[5-4]

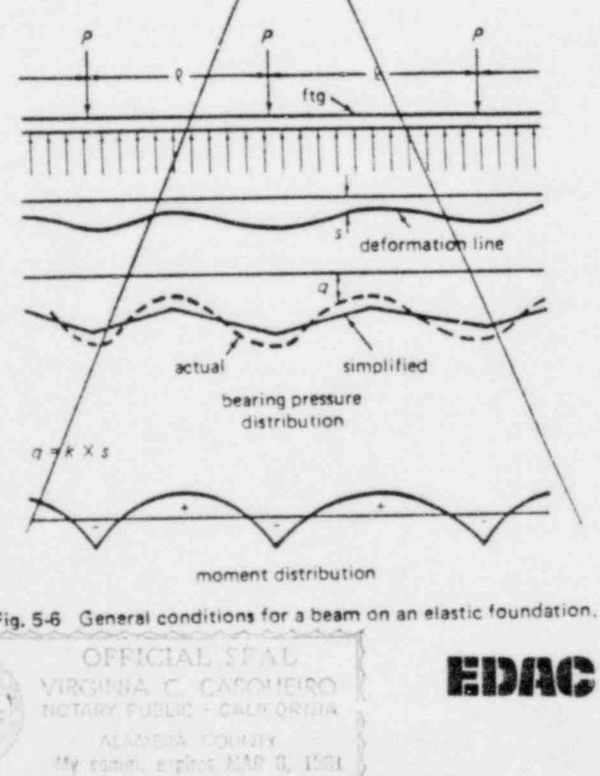

Fig. 5-6 General conditions for a beam on an elastic foundation.