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SOIL DYNAMICS, DEEP STABILIZATION, AND SPECIAL GEOTECHNICAL CONSTRUCTION

DESIGN MANUAL 7.3

DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND
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ABSTRACT

Design guidance is presented for use by experienced engineers. The contents include: dynamic and seismic aspects of geotechnical analysis; deep stabilization and grouting; and special geotechnical construction.

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FOREWORD

This design manual for Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction is one of a series that has been developed from an extensive re-evaluation of the relevant portions of Soil Mechanics, Foundations, and Earth Structures, NAVFAC DM-7 of March 1971, from surveys of available new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command, other Government agencies, and private industries. This manual includes a modernization of the former criteria and the maximum use of national professional society, association and institute codes. Deviations from these criteria should not be made without the prior approval of the Naval Facilities Engineering Command Headquarters (NAVFAC HQ).

Design cannot remain static any more than can the naval functions it serves, or the technologies it uses. Accordingly, this design manual, Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction, NAVFAC DM-7.3, along with the companion manuals, Soil Mechanics, NAVFAC DM-7.1 and Foundations and Earth Structures, NAVFAC DM-7.2, cancel and supersede Soil Mechanics, Foundations, and Earth Structures, NAVFAC DM-7 of March 1971 in its entirety, and all changes issued.

This publication is certified as an official publication of the Naval Facilities Engineering Command and has been reviewed and approved in accordance with the SECNAVINST 5600.16.

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SOIL MECHANICS AND FOUNDATION DESIGN MANUALS

DM No.	Title
7.1	SOIL MECHANICS
7.2	FOUNDATIONS AND EARTH STRUCTURES
7.3	SOIL DYNAMICS, DEEP STABILIZATION AND SPECIAL GEOTECHNICAL CONSTRUCTION

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CHAPTER 1. DYNAMIC AND SEISMIC ASPECTS

Section 1. INTRODUCTION

1. SCOPE. This chapter is concerned with geotechnical problems associated with vibratory loads and seismic forces. Dynamic response of foundations and structures depends on the magnitude, frequency, direction, and location of the dynamic loads or ground motions; the geometry of the soil-foundation contact system; and the dynamic properties of the supporting soils and structures. Dynamic ground motions considered in this chapter are those generated from the following sources:

a. Machine Foundations. Machine foundations on which the operation of machinery causes vibratory motions in the foundations and soils. These loads are generally assumed to persist during the design life of the structure (Type c and/or d, Figure 1).

b. Earthquake Ground Motions. Earthquake ground motions which cause dynamic loads in the foundations and structures. Earthquake ground motions are transient and may or may not occur several times during the design life of the structure (Type a, Figure 1).

c. Impact Loading. Impact loading generated transient type motions such as those generated by pile driving and blasting (Type b, Figure 1). Criteria for blast loadings on structures is covered in NAVFAC DM-2. Empirical estimates for ground motion velocity due to blasting is covered in DM-7.2, Chapter 1.

2. RELATED CRITERIA. Additional criteria relating to dynamic problems appear in the following sources:

Subject	Source
Blast Loading on Structures	NAVFAC P-397
Seismic Design for Buildings	NAVFAC P-355

Section 2. MACHINE FOUNDATIONS

1. ANALYSIS OF FOUNDATION VIBRATION

a. Characteristics of Vertical Oscillations. Ordinarily, vibrations are produced by vertically or horizontally oscillating loads of two types: (1) the force produced depends on the angular velocity of movement of the unbalanced mass, such as those from rotating machinery; (2) the force is independent of frequency of oscillator, such as those for periodic impact vibration produced by hammers. See Figure 2.

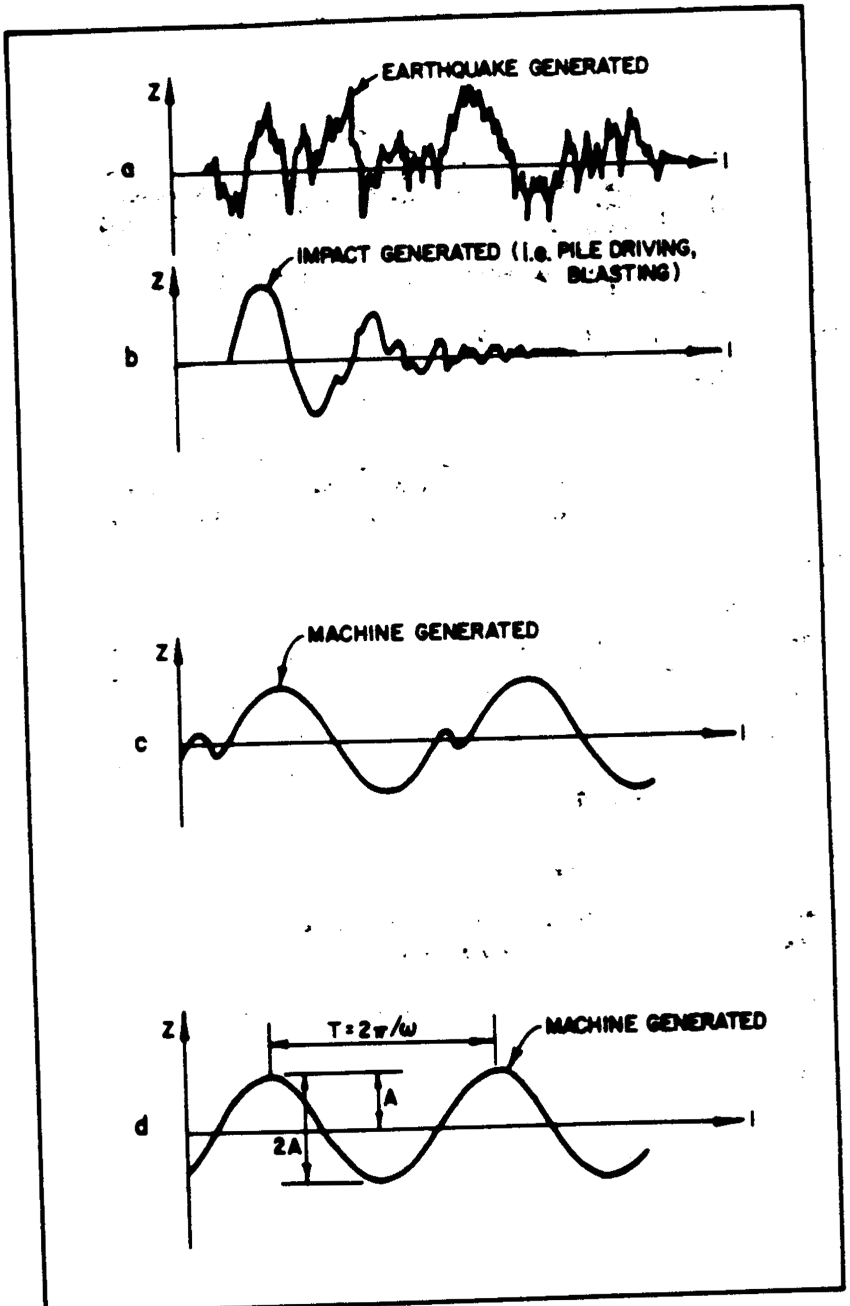
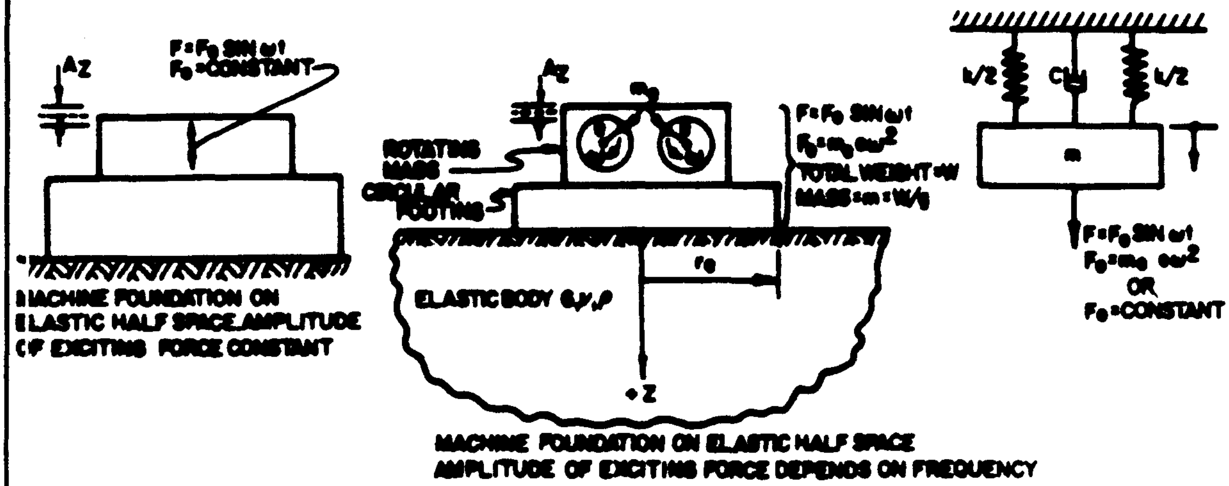


FIGURE 1
Wave Forms of Vibrations from Various Sources



DEFINITIONS:

ν = POISSON'S RATIO

m = MASS OF FOUNDATION AND MACHINE

ρ = FOUNDATION MASS DENSITY = γ/g

r_0 = EFFECTIVE RADIUS

= $\sqrt{\frac{BL}{\pi}}$ FOR VERTICAL OR HORIZ. TRANSLATION

= $\sqrt[4]{\frac{BL^3}{3\pi}}$ FOR ROCKING

= $\sqrt[4]{\frac{BL(B^2+L^2)}{6\pi}}$ FOR TORSION

B = WIDTH OF FOUNDATION (ALONG AXIS OF ROTATION FOR CASE OF ROCKING)

L = LENGTH OF FOUNDATION (IN PLANE OF ROTATION OR ROCKING)

I_ψ = MASS MOMENT OF INERTIA AROUND AXIS OF ROTATION FOR ROCKING

I_θ = MASS MOMENT OF INERTIA AROUND AXIS OF ROTATION FOR TORSION

G = DYNAMIC SHEAR MODULUS

ω = FREQUENCY OF FORCED VIBRATION (RADIANS/SEC.)

FIGURE 2

Frequency Dependent and Constant Amplitude Exciting Forces

b. Analysis of Foundation Vibration. Analyze foundation vibration as follows:

(1) Simplify the actual foundation geometry and soil properties into a single degree of freedom system, involving a spring constant K and damping ratio D. Compute spring constants K and damping ratio D for anticipated modes of vibration. See Figure 3 (Reference 1, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), by the Departments of the Army and Air Force).

(2) Specify the type of exciting force. For a constant amplitude exciting force the motion is expressed by:

$$F = F_o \sin \omega t$$

where:

ω = operating frequency (rad/sec) = $2\pi f$

f = operating frequency (cycle/sec)

F_o = amplitude of exciting force (constant)

F = exciting force

t = time

The exciting force F may depend on the frequency ω and the eccentric mass. In this case:

$$F_o = m_e e\omega^2$$

where:

m_e = eccentric mass

e = eccentric radius from center of rotation to center of gravity

(3) Compute the undamped natural frequency, f_n, in cycles/second or ω_n in rad/second

$$f_n = \frac{1}{2\pi}\sqrt{\frac{K}{m}}$$

$$\omega_n = \sqrt{\frac{K}{m}}$$

where:

K = K_z for vertical mode, K_x for horizontal mode, K_ψ for rocking mode and K_θ for torsional mode

m = mass of foundation and equipment for vertical and horizontal modes

I_ψ = mass moment of inertia around axis of rotation in rocking modes

I_θ = mass moment of inertia around axis of rotation in torsional modes.

MODE OF VIBRATION	MASS (OR INERTIA) RATIO	DAMPING COEFFICIENT	DAMPING RATIO $D = \frac{C}{\sqrt{Km}}$	SPRING CONSTANT K
VERTICAL	$B_Z = \frac{(1-\nu)}{4} \frac{m}{\rho r_o^3}$	$C_Z = \frac{3.4 r_o^2}{1-\nu} \sqrt{\rho G}$	$D_Z = \frac{0.425}{\sqrt{B_Z}}$	$K_Z = \frac{4 G r_o}{1-\nu}$
SLIDING (HORIZONTAL)	$B_X = \frac{(7-8\nu)}{32(1-\nu)} \frac{m}{\rho r_o^3}$	$C_X = \frac{4.6 r_o^2}{2-\nu} \sqrt{\rho G}$	$D_X = \frac{0.288}{\sqrt{B_X}}$	$K_X = \frac{32(1-\nu)}{7-8\nu} G r_o$
ROCKING	$B_\psi = \frac{3(1-\nu)}{8} \frac{I_\psi}{\rho r_o^5}$	$C_\psi = \frac{0.8 r_o^4 \sqrt{\rho G}}{(1-\nu)(1+B_\psi)}$	$D_\psi = \frac{0.15}{(1+B_\psi)\sqrt{B_\psi}}$	$K_\psi = \frac{8 G r_o^3}{3(1-\nu)}$
TORSIONAL	$B_\theta = \frac{I_\theta}{\rho r_o^5}$	$C_\theta = \frac{4 \sqrt{B_\theta \rho G}}{1+2 B_\theta}$	$D_\theta = \frac{0.50}{1+2B_\theta}$	$K_\theta = \frac{16}{3} G r_o^3$

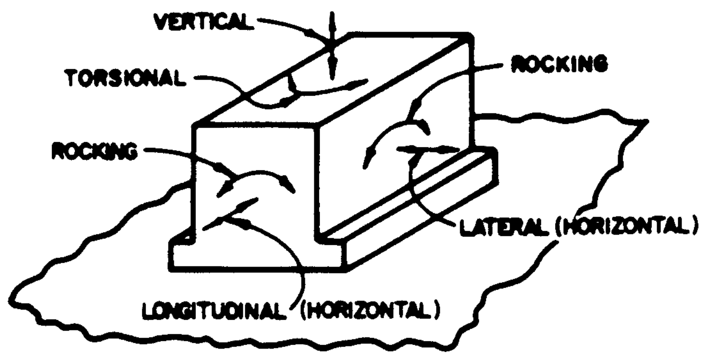


FIGURE 3
Modes of Vibration

thus

for vertical mode $f_n = \frac{1}{2\pi}\sqrt{\frac{K_z}{m}}$

for horizontal mode $f_n = \frac{1}{2\pi}\sqrt{\frac{K_x}{m}}$

for torsional (yawing) mode $f_n = \frac{1}{2\pi}\sqrt{\frac{K_\theta}{I_\theta}}$

for rocking mode $f_n = \frac{1}{2\pi}\sqrt{\frac{K_\psi}{I_\psi}}$

(4) Compute the mass ratio B and damping ratio D for modes analyzed using the formulas in Figure 3.

(5) Calculate Static Displacement Amplitude, A_s

$$A_s = \frac{F_o}{K}$$

(6) Compute the ratio f/f_n (same as ω/ω_n).

(7) Calculate magnification factor $M = \frac{A_{max}}{A_s}$ from Figure 4.

(8) Calculate maximum amplitude $A_{max} = M \cdot A_s$.

(9) If amplitudes are not acceptable, modify design and repeat Steps 3 through 8.

(10) See Figure 5 for example calculations illustrating the calculation of vertical amplitude, horizontal amplitude alone and rocking amplitude alone.

c. Dynamic Soil Properties. Guidance on dynamic soil properties and their determination is given in DM-7.1, Chapters 2 and 3.

2. DESIGN TO AVOID RESONANCE. Settlements from vibratory loads are accentuated if imposed vibrations are resonant with the natural frequency of the foundation soil system. Both the amplitude of foundation motion and the unbalanced exciting force are increased at resonance, and even compact cohesionless soils will be densified to some degree with accompanying settlement. Avoidance of resonance is particularly important in cohesionless materials, but should be considered for all soils. Analyze foundations for vibrating machinery to avoid unacceptable amplitude by methods given previously. In order to avoid resonance, the following guidelines may be considered for initial design to be verified by the previous methods.

a. High-Speed Machinery. For machinery with operating speeds exceeding about 1,000 rpm, provide a foundation with natural frequency no higher than one-half of the operating value, as follows:

(1) Decrease natural frequency by increasing the foundation block weight, analyze vibration in accordance with the methods discussed previously.

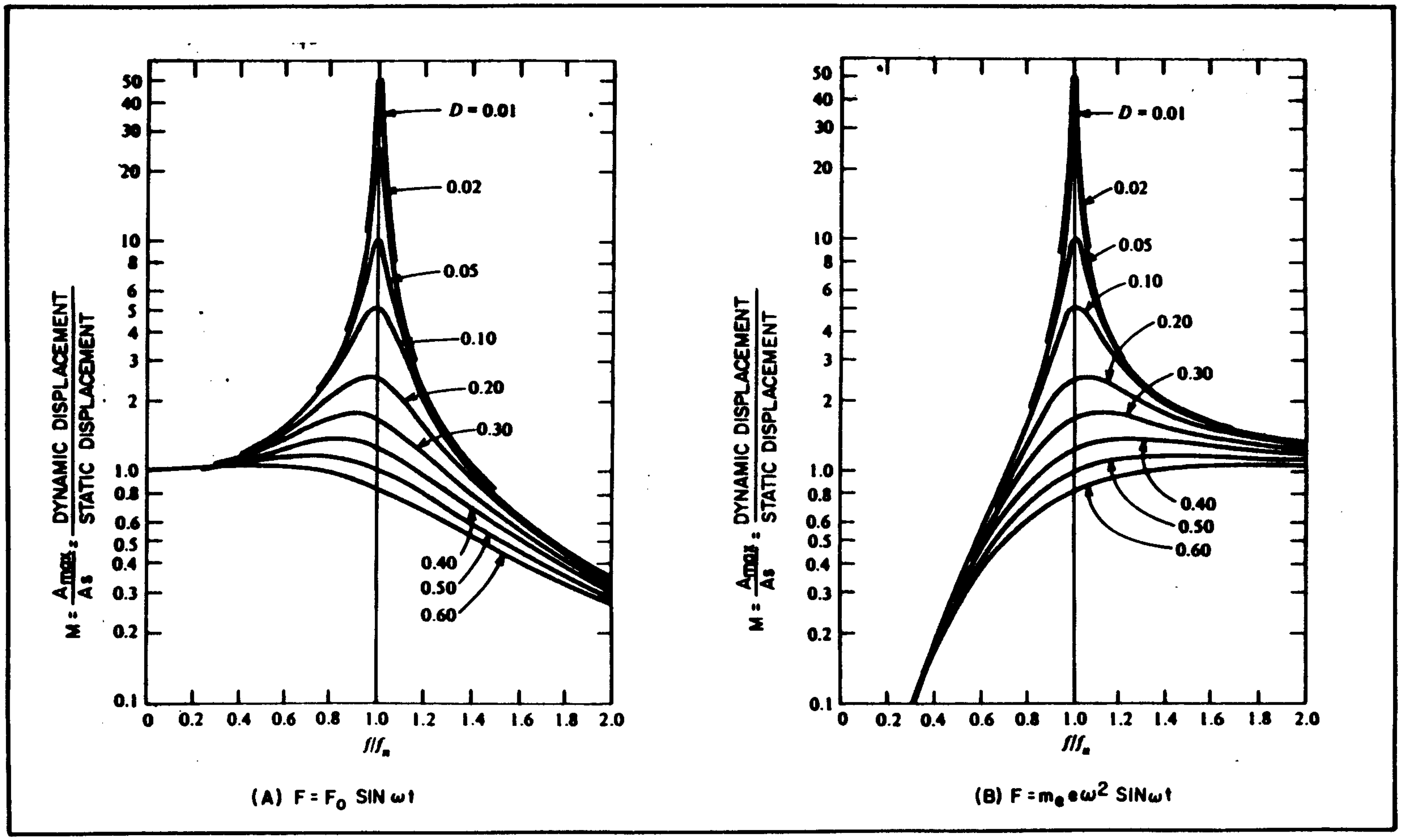
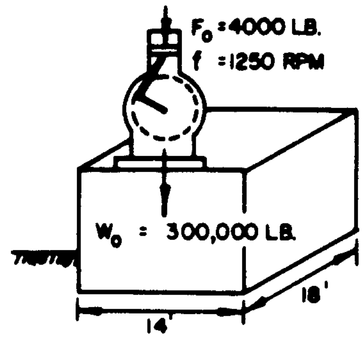


FIGURE 4
Response Curves for a Single-Degree-of-Freedom System with a Viscous Damping

A. VIBRATION IN VERTICAL MODE



EQUIPMENT DATA

GIVEN A HIGH SPEED GENERATOR WITH A FREQUENCY DEPENDENT AMPLITUDE $F_0 = 4000$ LB.
W_0 = WEIGHT OF VIBRATING EQUIPMENT AND FOUNDATION BLOCK = 300,000 LB.

OPERATING FREQUENCY $f = 1250$ RPM = 20.83 CPS
$\omega = 20.83 \times 2\pi = 131$ RAD./SEC.

SOIL PROPERTIES

TOTAL UNIT WEIGHT $\gamma_T = 120$ PCF
POISSON'S RATIO $\nu = 0.35$
SHEAR MODULUS G = 6700 PSI

EQUIVALENT RADIUS $r_0 = \sqrt{\frac{BL}{\pi}} = 8.95$ FT.

SPRING CONSTANT

$$K_z = \frac{4 G r_0}{1-\nu} = \frac{4(6700)(8.95)(12)}{1-0.35}$$

$$= 44.28 \times 10^5 \text{ LB/IN} = 531.36 \times 10^5 \text{ LB/FT}$$

MASS RATIO

$$B_z = \frac{(1-\nu)(m)}{4 \rho r_0^3}$$

$$\rho = \frac{\gamma_T}{32.2} = 3.73 \frac{\text{LB-SEC}^2}{\text{FT}^4} \text{ MASS DENSITY}$$

$$m = \frac{W}{g} = \frac{300,000}{32.2} = 9316.8 \frac{\text{LB-SEC}^2}{\text{FT}}$$

$$B_z = \frac{(1-0.35)(9316.8)}{4(3.73)(8.95)^3} = 0.57$$

DAMPING RATIO

$D_z = 0.425/\sqrt{B_z} = 0.56$

A_s STATIC AMPLITUDE $= F_0/K_z =$
$= 4000/44.28 \times 10^5$
$= 0.9 \times 10^{-3}$ INCH

NATURAL FREQ. $\omega_n = \sqrt{\frac{K}{m}} = \sqrt{\frac{531.36 \times 10^5}{9316.8}} = 75.5$ RAD/SEC

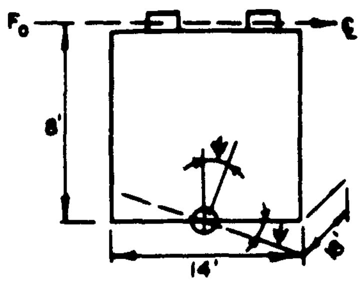
DYNAMIC AMPLITUDE

$\frac{\omega}{\omega_n} = \frac{131}{75.5} = 1.74$, THEN FROM FIGURE 4-B AND FOR $D = 0.56$
$M \cong 1.1$

$A_{MAX} = (A_z)(M) = 0.001$ INCH. = MAXIMUM DYNAMIC AMPLITUDE

FIGURE 5
Example Calculation of Vertical, Horizontal and Rocking Motions

B. EXAMPLE CALCULATION FOR HORIZONTAL TRANSLATION AND ROCKING



$F = 300 \ SIN \omega t$
$f = 1250$ RPM
$I_\psi = 4 \times 10^5$ FT.-LB./SEC.^{2}

EQUIPMENT DATA

ASSUME CONSTANT AMPLITUDE $F_0 = 300$ LB
WEIGHT OF FOUNDATION AND MACHINERY = 400,000 LB.
MASS MOMENT OF INERTIA I_ψ AROUND AXIS OF ROTATION = 4×10^5 FT-LB/SEC^2
OPERATING FREQUENCY $f = 350$ RPM = 5.8 CPS
$\omega = 5.8\,(2\pi) = 36.4$ RAD/SEC

SOIL PROPERTIES

TOTAL UNIT WEIGHT $\gamma_T = 120$ PCF
POISSON'S RATIO $\nu = 0.35$
SHEAR MODULUS $G = 6700$ PSI

HORIZONTAL TRANSLATION ONLY

EQUIVALENT RADIUS $r_0 = \sqrt{\frac{BL}{\pi}} = \sqrt{\frac{18 \times 14}{\pi}} = 8.95$ FT

$$K_x = \frac{32(1-\nu)\,G r_0}{7-8\nu}$$

$$= \frac{32(1-0.35)}{7-2.8}\,6700\,(12)(8.95) = 35.64 \times 10^5 \text{ LB/IN}$$

$$B_x = \frac{7-8\nu}{32(1-\nu)}\,\frac{m}{\rho r_0^3}$$

$$m = \frac{400{,}000}{32.2} = 12.42 \times 10^3 \ \frac{\text{LB-SEC}^2}{\text{FT}}$$

$$\rho = \frac{120}{32.2} = 3.73 \ \frac{\text{LB}\cdot\text{SEC}^2}{\text{FT}^4}$$

$$B_x = 0.94$$

$$D_x = \frac{0.288}{\sqrt{B_x}} = \frac{0.288}{\sqrt{0.94}} = 0.3$$

$$A_s = \frac{300}{K_x} = 0.000084 \text{ INCH (STATIC DISPLACEMENT)}$$

$$\omega_n = \sqrt{\frac{K}{m}} = \sqrt{\frac{35.64 \times 10^5 \times 12}{12.42 \times 10^3}} = 58.7 \text{ RAD/SEC}$$

$$\frac{\omega}{\omega_n} = \frac{36.4}{58.7} = 0.62$$

FROM FIGURE 4-A, $M \approx 1.4$
MAXIMUM HORIZONTAL MOVEMENT (A_{MAX}) = $1.4 \times 0.000084 = 0.00012$ INCHES

FIGURE 5 (continued)
Example Calculation of Vertical, Horizontal and Rocking Motions

ROCKING ALONE

$$r_0 = \sqrt[4]{\frac{BL^3}{3\pi}} = \sqrt[4]{\frac{18 \times 14^3}{3\pi}} = 8.5 \text{ FT}$$

$$K_\psi = \frac{8G\,r_0^3}{3\,(1-\nu)} = \frac{8\,(6700)(8.5 \times 12)^3}{3\,(1-\nu)} = 2.9 \times 10^{10} \text{ IN-LB/RAD}$$

$$B_\psi = \frac{3(1-\nu)}{8}\,\frac{I_\psi}{\rho\, r_0^5} = \frac{3(1-.35)}{8}\;\frac{(4 \times 10^5)}{3.73(8.5)^5}$$

$$= 0.59$$

$$D_\psi = \frac{0.15}{(1+B_\psi)\sqrt{B_\psi}} = 0.12$$

$$\omega_n = \sqrt{\frac{K_\psi}{I_\psi}} = \sqrt{\frac{2.9 \times 10^{10}}{12 \times 4 \times 10^5}} = 77.7 \text{ RAD/SEC}$$

$$F_0 = 300 \times 8 \times 12 = 28{,}800 \text{ IN-LB}$$

STATIC ROTATION $A_\psi = \frac{F_0}{K_\psi} = \frac{28{,}800}{2.9 \times 10^{10}} = 1 \times 10^{-6}$ RADIANS

$$\frac{\omega}{\omega_n} = \frac{131}{77.7} = 1.68$$

FROM FIGURE 4-A, $M \approx 0.4$

MAXIMUM ROCKING MOVEMENT

$$(A_\psi)_{MAX} = 0.4 \times 10^{-6} \text{ RAD}$$

HORIZONTAL MOTION AT MACHINE CENTER LINE $= 0.4 \times 10^{-6} \times 8 \times 12 = 0.038 \times 10^{-3}$ INCH

NOTE:

ABOVE ANALYSIS IS APPROXIMATE SINCE HORIZONTAL AND ROCKING MODES ARE COUPLED. SEE TEXT FOR GUIDANCE ON DETAILED ANALYSIS. A LOWER BOUND ESTIMATE OF FIRST MODE FREQUENCY MAY BE CALCULATED BASED ON NATURAL FREQUENCIES ω_n FOR ROCKING MODE ALONE, AND HORIZONTAL TRANSLATION MODE ALONE (SEE TEXT).

FIGURE 5 (continued)
Example Calculation of Vertical, Horizontal and Rocking Motions

(2) During starting and stopping, the machine will operate briefly at resonant frequency of the foundation. Compute probable amplitude at both resonant and operating frequencies, and compare them with allowable values to determine if the foundation arrangement must be altered.

b. Low-speed Machinery. For machinery operating at a speed less than about 300 rpm, provide a foundation with a natural frequency at least twice the operating speed, by one of the following:

(1) For spread foundations, increase the natural frequency by increasing base area or reducing total static weight.

(2) Increase modulus of shear rigidity of the foundation soil by compaction or other means of stabilization. See DM-7.2, Chapter 2.

(3) Consider the use of piles to provide the required foundation stiffness. See example in Figure 6 (Reference 2, Foundation Vibrations, by Richart).

c. Coupled Vibrations. Vibrations are coupled when their modes are not independent but influence one another. A mode of vibration is a characteristic pattern assumed by the system in which the motion of each particle is simple harmonic with the same frequency. In most practical problems, the vertical and torsional mode can be assumed to be uncoupled (i.e., independent of each other). However, coupling effects between the horizontal and rocking modes can be significant depending on the distance between the center of gravity of the footing and the base of the footing. The analysis for this case is complicated and time consuming.

A lower bound estimate of the first mode, f_o, of coupled rocking and horizontal vibration can be obtained from

$$\frac{1}{f_o^2} = \frac{1}{f_x^2} + \frac{1}{f_\psi^2}$$

f_x and f_ψ are the undamped natural frequencies in the horizontal and rocking mode respectively. For further guidance see Reference 3, Vibrations of Soils and Foundations, by Richart, et al. and Reference 4, Coupled Horizontal and Rocking Vibrations of Embedded Footings, by Beredugi and Novak.

d. Effect of Embedment. Stiffness and damping are generally increased with embedment. However, analytical results (especially for damping) are sensitive to the conditions of the backfill (properties, contact with the footings, etc.). For footings embedded in a uniform soil with a Poisson's ratio of 0.4, the modified stiffness parameters are approximated as follows (Reference 5, Stiffness and Damping Coefficients of Foundations, by Roesset):

$$(K_z)_d \approx K_z\left(1 + 0.4\frac{d}{r_o}\right)$$
$$(K_x)_d \approx K_x\left(1 + 0.8\frac{d}{r_o}\right)$$
$$(K_\psi)_d \approx K_\psi\left[1 + 0.6\frac{d}{r_o} + 0.3\left(\frac{d}{r_o}\right)^3\right]$$
$$(K_\theta)_d \approx K_\theta\left(1 + 2.4\frac{d}{r_o}\right)$$

$(K_z)_d$, $(K_x)_d$, $(K_\psi)_d$ and $(K_\theta)_d$ are spring constants for depth of embedment d.

Increases in damping also occur with embedment d, but the results are believed to be more sensitive to condition of backfill. For footings embedded in a uniform soil, the approximate modifications for damping coefficient C (in Figure 3) are:

$$(C_z)_d \approx C_z \left(1 + 1.2 \frac{d}{r_o}\right)$$

$$(C_\theta)_d \approx r_o^4 \sqrt{\rho G} \left(0.7 + 5.4 \frac{d}{r_o}\right)$$

when $(C_z)_d$ and $(C_\theta)_d$ are the damping coefficients in vertical and torsion modes for embedments d. The expression for rocking and sliding are complicated, see Reference 4 for further guidance.

e. Proximity of a Rigid Layer. A relatively thin layer of soil over rigid bedrock may cause serious magnification of the vertical amplitude of vibration. In general, the spring constants increase with decreasing thickness of soil while damping coefficents decrease sharply for the vertical modes and to a lesser extent for horizontal and rocking modes. Use the following approximate relation for adjusting stiffness and damping to account for presence of a rigid layer (from Reference 6, Soils Structure Interaction by Richart and Reference 7, Dynamic Stiffness of Circular Foundations by Kausel and Roesset):

$$(K_z)_L = K_z\left(1 + \frac{r_o}{H}\right), \quad \frac{r_o}{H} < \frac{1}{2}$$

$$(K_x)_L = K\left(1 + \frac{1}{2}\frac{r_o}{H}\right), \quad \frac{r_o}{H} < \frac{1}{2}$$

$$(K_\psi)_L = K\left(1 + \frac{1}{6}\frac{r_o}{H}\right), \quad \frac{r_o}{H} < \frac{1}{2}$$

where $(K_z)_L$, $(K_x)_L$, $(K_\psi)_L$ are stiffness parameters in case a rigid layer exists at depth H below a footing with radius r_o.

The damping ratio parameters D are reduced by the presence of a rigid layer at depth H. The modified damping coefficient (D_z) is 1.0 D_z for $H/r_o = \infty$, and approximately 0.31 D_z, 0.16 D_z, 0.09 D_z and 0.044 D_z for H/r_o = 4, 3, 2 and 1 respectively (see Reference 6).

f. Vibration For Pile Supported Machine Foundation. For piles bearing on rigid rock with negligible side friction, use Figure 6 for establishing the natural frequency of the pile soil system. Tip deflection and lateral stiffness can have significant effect on natural frequency of the pile soil system. (See Reference 8, Response of Piles to Vibratory Loads by Oweis). A detailed analysis of the pile problem is complex and requires the use of the computer for the lengthly calculations. Reference 9, Impedence Functions of Piles in Layered Media, by Novak and Aboul-Ella, presents solutions for simple but practical cases for stiffness and damping coefficients. Alternatively, and for important installations, such coefficients can be evaluated from field pile load tests.

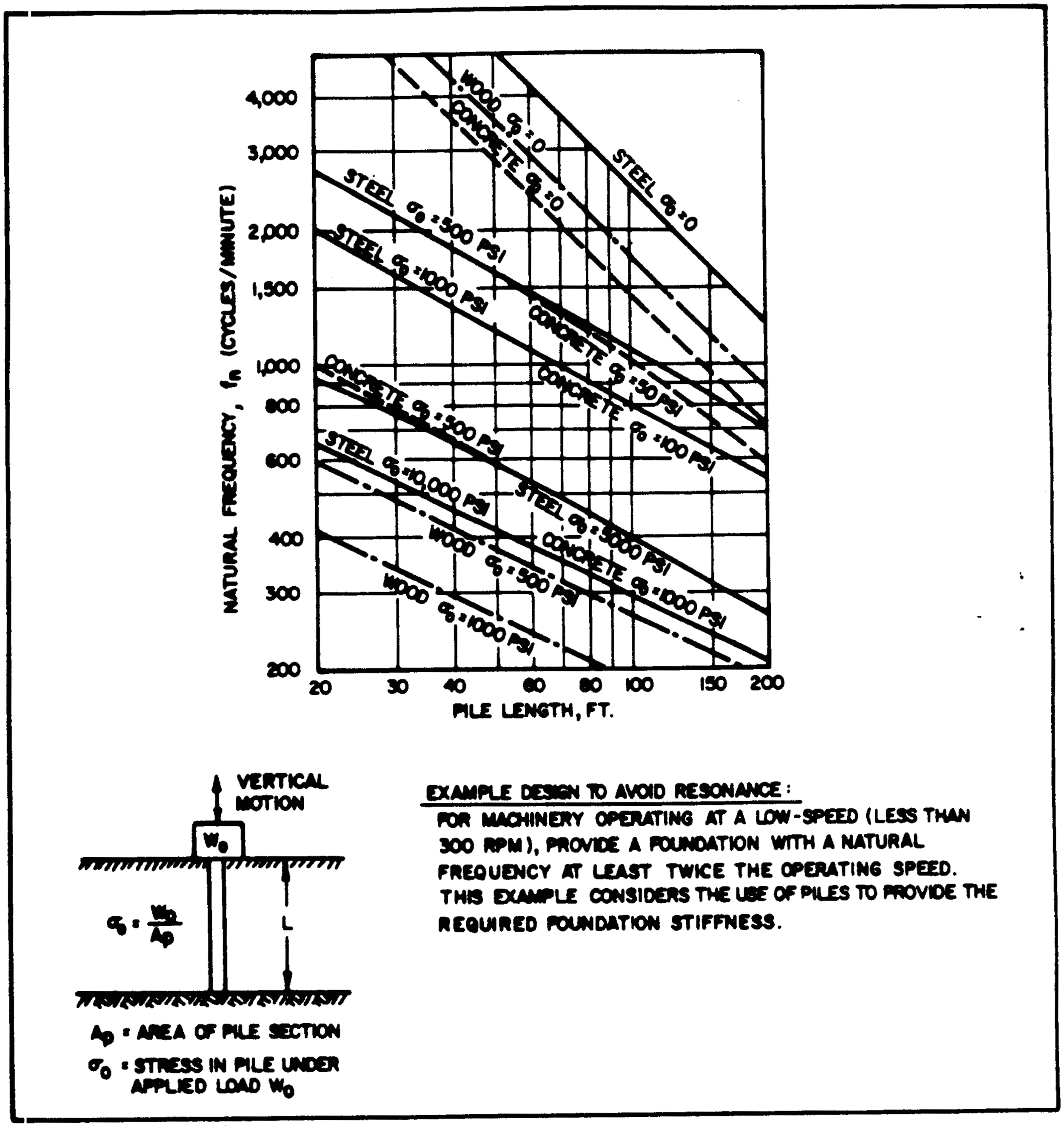


FIGURE 6
Natural Undamped Frequency of Point Bearing Piles on Rigid Rock

3. BEARING CAPACITY AND SETTLEMENTS. Vibration tends to densify loose non-plastic soils, causing settlement. The greatest effect occurs in loose, coarse-grained sands and gravels. These materials must be stabilized by compaction or other means to support spread foundations for vibrating equipment; see methods of DM-7.2, Chapter 2. Shock or vibrations near a foundation on loose, saturated nonplastic silt, or silty fine sands, may produce a quick condition and partial loss of bearing capacity. In these cases, bearing intensities should be less than those normally used for static loads. For severe vibration conditions, reduce the bearing pressures to one-half allowable static values.

In most applications, a relative density of 70% to 75% in the foundation soil is satisfactory to preclude significant compaction settlement beneath the vibratory equipment. However, for heavy machinery, larger relative densities may be required. The following procedure may be used to evaluate the order of magnitude of compaction settlement under operating machinery.

The critical acceleration of machine foundations, $(a)_{crit}$, above which compaction is likely to occur may be estimated based on

$$(a)_{crit} = \frac{-\ln\left(1-\frac{(Dr)_o}{100}\right)}{\beta}$$

where: $(a)_{crit}$ = critical acceleration expressed in g's

$(Dr)_o$ = initial (in situ) relative density at zero acceleration expressed in percent

β = coefficient of vibratory compaction, a parameter depending on moisture content; varies from about 0.8 for dry sand down to 0.2 for low moisture contents (about 5%). It increases to a maximum value of about 0.88 at about 18% moisture content. Thereafter, it decreases.

(See Reference 10, Dynamics of Bases and Foundations, by Barkan.)

When compaction occurs as a result of vibrations there will be an increase in relative density ΔD_r, and for a sand layer with a thickness H, the settlement would be ΔH. The strain $\Delta H/H$ can be expressed in terms of ΔD_r as:

$$\frac{\Delta H}{H} = 0.0025 \left(\frac{\Delta D_r\%}{100}\right) \gamma_{do}$$

where: γ_{do} is the initial dry density of the sand layer (lb/cu ft).

The above equation is based on the range of maximum and minimum dry densities for sands reported in Reference 11, Field Testing of Soils, by Burmister.

The change in relative density ΔD_r due to vibrations is defined as

$$\Delta D_r = (Dr)_f - (Dr)_o$$

where: $(D_r)_o$ = initial in situ relative density which may be estimated from the standard penetration resistance (see DM-7.1, Chapter 2).

$(D_r)_f$ = final relative density, which may be conservatively estimated based on

$$(D_r)_f = 100\left\{ 1-e^{-\beta\left[(a_i)_{crit}+a_i\right]} \right\} \text{for } a_i > (a_i)_{crit}$$

$$(D_r)_f = (D_r)_o \text{ for } a_i \leq (a_i)_{crit}$$

a_i = acceleration expressed in g's

The above equation is based on the work reported in Reference 10.

In the above equation $(a_i)_{crit}$ and (a_i) are the critical acceleration and acceleration produced by equipment in each layer i.

The acceleration a_i produced by equipment may be approximated using the following:

$$a_i = a_o \sqrt{\frac{r_o}{d}} \text{ for } d > r_o$$

$$a_i = a_o \quad \text{for } d = r_o$$

where: a_o = acceleration of vibrations in g's at foundation level

d = distance from base of foundation to mid point of soil layer

r_o = equivalent radius of foundation

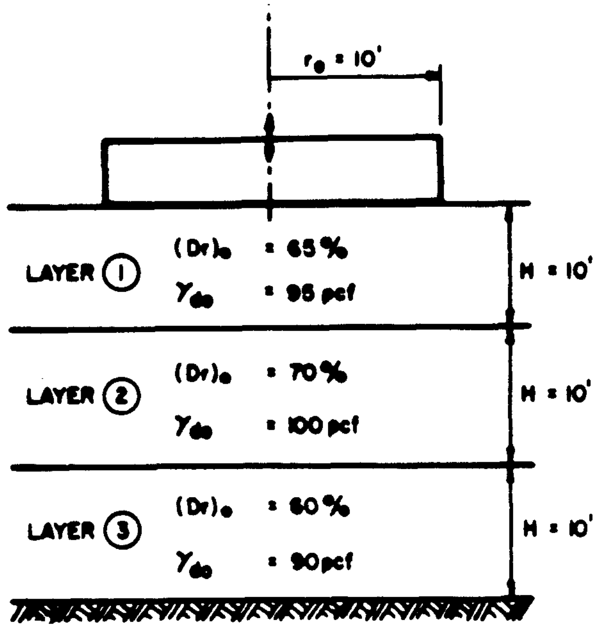
If maximum displacement, A_{max}, and frequency of vibration, ω (rad/sec), are known at base of foundation then:

$$a_o = (\omega)^2 A_{max}$$

An example illustrating the use of the above principles is shown in Figure 7.

4. VIBRATION TRANSMISSION, ISOLATION, AND MONITORING.

a. Vibration Transmission. Transmission of vibrations from outside a structure or from machinery within the structure may be annoying to occupants and damaging to the structure, or may interfere with the operation of sensitive instruments. See Figure 8 for the effect of vibration amplitude and frequency. Tolerable vibration amplitude decreases as frequency increases. For methods of reducing amplitude of vibrations transmitted into a structure or away from a vibrating source, see the following paragraphs. For approximate estimates of vibration amplitude transmitted away from the source use the following relationship:



GIVEN: SOIL PROFILE AS SHOWN: FOOTING WITH A RADIUS OF 10 FEET SUBJECTED TO A VIBRATORY LOAD CAUSING A PEAK DYNAMIC DISPLACEMENT Amax = 0.007 IN.. OPERATING FREQUENCY f = 2500 RPM. MOISTURE CONTENT ABOUT 16%, USE β = 0.88

$$\omega = \frac{2500}{60} \times 2\pi = 261.8 \text{ RAD/SEC}$$

$$a_o = \frac{(261.8)^2 \times 0.007}{12 \times 32.2} = 1.24\,g$$

LAYER 1:

d = DEPTH TO MID LAYER = 5' < r_o = 10'

USE $a_i = a_o = 1.24g$

CRITICAL ACCELERATION $$(a)_{crit} = \frac{-\ln\left(1 - \frac{(Dr)_o}{100}\right)}{\beta}$$

$(a)_{crit}$ = 1.19 g

a_i = 1.24 g > a_{crit}

$$(D_r)_f = 100\left[1 - e^{-\beta\left[(a_i)_{crit} + a_i\right]}\right] = 88\%$$

ΔDr = 88% − 65% = 23%

$$\frac{\Delta H}{H} = 0.0025\left(\frac{\Delta Dr}{100}\right)(\gamma_{do})$$

$= 0.0025 \times 0.23 \times 95$

$= 0.055$

$\Delta H = 0.055 \times 10 = 0.55$ FT. = 6.6 IN.

LAYER 2:

d (MID LAYER) = 15 FEET

$$a_i = a_o\sqrt{\frac{r_o}{d}} = 1.24\sqrt{\frac{10}{15}} = 1.01\,g$$

$$(a_i)_{crit} = \frac{-\ln(1 - 0.7)}{0.88} = 1.37\,g$$

a_i = 1.01 g < a_{crit}

$(Dr)_f = (Dr)_o$

NO SIGNIFICANT COMPACTION SETTLEMENT IS LIKELY

LAYER 3:

d = 25'

$$a_i = 1.24\sqrt{\frac{10}{25}} = 0.78\,g$$

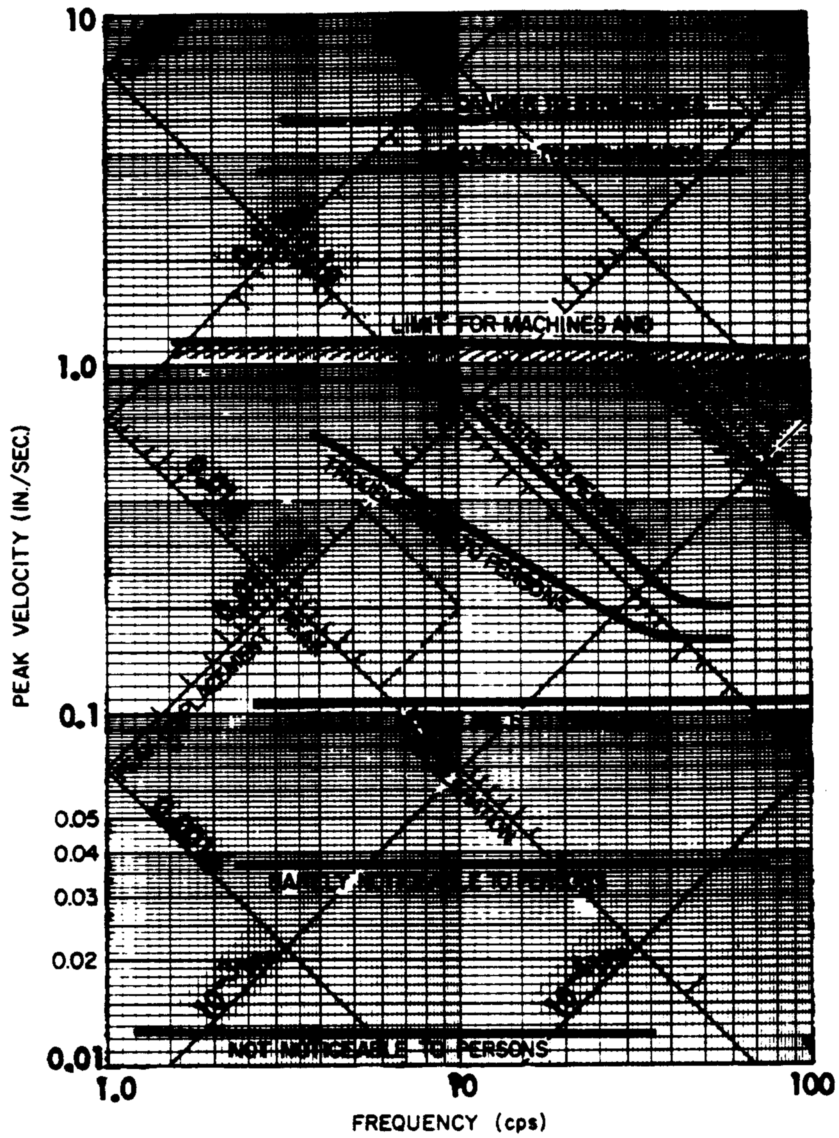
$$(a_i)_{crit} = \frac{-\ln(1 - 0.6)}{0.88} = 1.04$$

$a_i < (a_i)_{crit}$

NO SIGNIFICANT COMPACTION SETTLEMENT IS LIKELY

ANTICIPATED COMPACTION SETTLEMENT = 6.6 IN.. INCREASE RELATIVE DENSITY OF TOP LAYER TO 70% OR GREATER.

FIGURE 7
Example Calculation for Vibrations Induced Compaction Settlement Under Operating Machinery



GIVEN VELOCITY = 0.2 INCH/SEC.
FREQUENCY = 10 CPS
THEN FROM GRAPH, DISPLACEMENT = 0.003 INCHES
ACCELERATION = 0.03g
MOTION IS EASY NOTICEABLE OR TROUBLESOME TO PERSONS

FIGURE 8
Allowable Amplitude of Vertical Vibrations

$$A_2 = A_1 \sqrt{\frac{r_1}{r_2}} \; e^{-\alpha(r_2-r_1)}$$

where: A_1 = computed or measured amplitude at distance r_1 from vibration source

A_2 = amplitudes at distance r_2, $r_2 \geq r_1$

α = coefficient of attenuation depending on soil properties and frequency. Use Table 1.

b. Vibration and Shock Isolation.

(1) General Methods. For general methods of isolating vibrating equipment or insulating a structure from vibration transmission, see Table 2. These methods include physical separation of the vibrating unit from the structure, or interposition of an isolator between the vibrating equipment and foundation or between the structure foundation and an outside vibration source. Vibration isolating mediums include resilient materials such as metal springs, or pads of rubber, cork, felt, or lead and asbestos in combination.

(2) Other Methods. Additional methods available include the installation of open or slurry-filled trenches, sheet pile walls or concrete walls. These techniques have been applied with mixed results. Analytical results suggest that for trenches to be effective, the depth of the trench should be 0.67λ or larger, where λ is wave length for Rayleigh wave and is approximately equal to $\frac{v_s}{\omega}$; when λ is the frequency of vibration in radian/sec, v_s is the shear wave velocity of the soil. Concrete core walls may have isolating efficiency depending on the thickness, length and rigidity. (See Reference 12, Isolation of Vibrations by Concrete Core Walls, by Haupt).

c. Vibration Monitoring. Control of ground vibrations is necessary to ensure the acceptable level of vibration amplitudes for structural safety are not exceeded. The sources of vibrations which may affect nearby structures are those generated by blasting, pile driving or machinery. Acceptable vibration amplitudes are usually selected based on conditions of the structure, sensitivity of equipment within the structure, or human tolerance. See DM-7.2, Chapter 1 for selection of blasting criteria in terms of peak particle velocity to avoid damage to structure.

For structures which may be affected by nearby sources of vibrations (e.g., blasting, pile driving, etc.) seismographs are usually installed at one or more floors to ensure the site vibration limits are not exceeded. A seismograph usually consists of one or more transducers which are either embedded, attached or resting on the vibrating structure, element, or soil, connected by a cable to the recording unit. The recording medium may be an oscilloscope or a magnetic tape. The actual details of installation depend on the type of equipment, nature of vibration surface, and expected amplitudes of motion. For further guidance see Reference 13, Some Current Methods in Vibration Measurements, by Pretlove, and Reference 14, Measurement of Blast Induced Ground Vibrations and Seismograph Calibration, by Stagg and Englor. Specifications of available commercial seismographs are given in Reference 14.

TABLE 1
Attenuation Coefficients for Earth Materials

Materials		α* (1/ft) @ 50 Hertz**
Sand	Loose, fine	0.06
	Dense, fine	0.02
Clay	Silty (loess)	0.06
	Dense, dry	0.003
Rock	Weathered volcanic	0.02
	Competent marble	0.00004

* α is a function of frequency. For other frequencies, f, compute $\alpha_f = (f/50) \times \alpha_{50}$

** Hertz - one cycle per second.

TABLE 2
Vibration and Shock Isolation

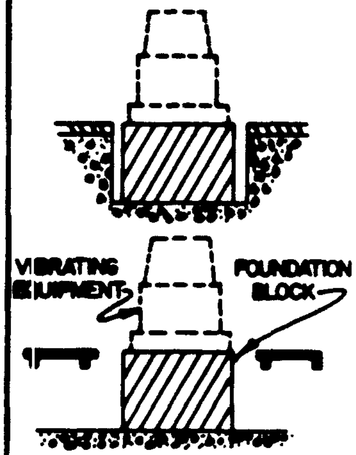
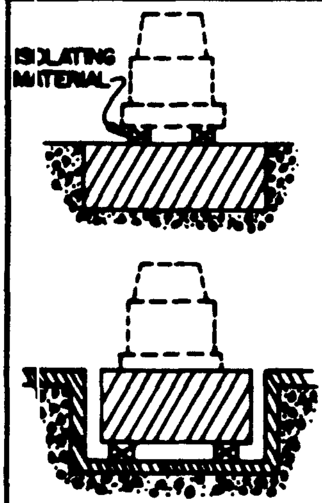
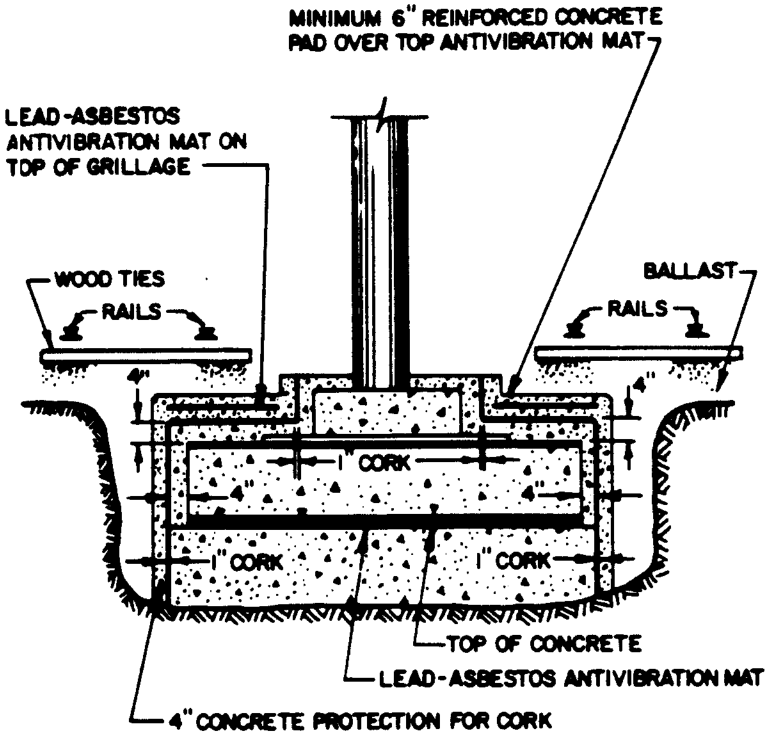
METHOD OF ISOLATION		APPLICATIONS
	PHYSICAL SEPARATION OF VIBRATING EQUIPMENT FROM THE STRUCTURE. EQUIPMENT IS INSTALLED ON A CONCRETE BLOCK SET IN OR ON THE GROUND NOT IN CONTACT WITH THE SURROUNDING FLOOR OR STRUCTURE.	THIS IS THE CHEAPEST AND SIMPLEST METHOD OF ISOLATING VIBRATING EQUIPMENT BUT THE LEAST EFFECTIVE. FREQUENTLY USED FOR MOUNTING MACHINE TOOLS OR SIMILAR EQUIPMENT WHERE MODERATE VIBRATION TRANSMITTED TO THE STRUCTURE IS TOLERABLE AND NO SENSITIVE INSTRUMENTS ARE INVOLVED.
	VIBRATING EQUIPMENT IS MOUNTED WITH OR WITHOUT THE BASE BLOCK, SUPPORTED OR SURROUNDED ON ISOLATING MATERIAL. ISOLATORS CONSIST OF METAL SPRINGS, RUBBER, CORK OR FELT BLOCKS, PADS OR MATS, OR OTHER RESILIENT MATERIAL. ISOLATOR MAY BE PLACED DIRECTLY BENEATH THE EQUIPMENT OR UNDER A BASE BLOCK THAT PROVIDES ADDITIONAL INERTIA MASS. IN SOME CASES, ISOLATORS ARE LOCATED AT THE CENTER OF GRAVITY OF THE COMBINED MASS OF EQUIPMENT PLUS BASE BLOCK. IN OTHER CASES THE EQUIPMENT IS PLACED ON A BLOCK SUSPENDED AS A PENDULUM THAT IS SUPPORTED ON THE ISOLATING MATERIAL.	ISOLATORS ARE UTILIZED FOR A VARIETY OF VIBRATING EQUIPMENT. RUBBER ISOLATORS ARE FREQUENTLY EMPLOYED FOR ENGINES AND COMPRESSORS. HEAVY HAMMERS AND PRESSES MAY BE PLACED IN PITS LINED BY ISOLATING MATERIAL OR SUPPORTED ON SPRINGS. SENSITIVE INSTRUMENTS FREQUENTLY ARE MOUNTED ON ISOLATING MATERIALS.

TABLE 2 (continued)
Vibration and Shock Isolation

METHOD OF ISOLATION	APPLICATIONS
 FOR INSULATION OF STRUCTURAL FRAME AGAINST VIBRATIONS TRANSMITTED FROM THE OUTSIDE, INSULATING PADS MAY BE INCORPORATED IN THE FOOTINGS. ONE SUITABLE METHOD UTILIZES FABRICATED PADS OF LEAD AND ASBESTOS. IN SPECIAL CIRCUMSTANCES IT MAY BE PRACTICAL TO ISOLATE AN ENTIRE BUILDING BY SURROUNDING IT BY A DITCH. THIS SHOULD BE AT LEAST 12 FEET DEEP WITHOUT CROSS BRACING BETWEEN SIDES. SHOULD CROSS BRACING BE NECESSARY IT SHOULD BE ARRANGED WITH VIBRATION ISOLATING MATERIAL INCORPORATED IN THE BRACING.	LEAD-ASBESTOS PADS HAVE BEEN UTILIZED FOR HEAVY INDIVIDUAL COLUMN FOOTINGS TO REDUCE THE VIBRATION FROM RAILROAD NEARBY. THE PADS ARE EFFECTIVE BECAUSE VIBRATIONS RESIST BEING TRANSMITTED THROUGH DISSIMILAR MATERIALS IN CONTACT.

Section 3. SEISMIC ASPECTS

1. DESIGN EARTHQUAKE

a. Design Parameters. In evaluating the soil behavior under earthquake motion, it is necessary to know the magnitude or intensity of the earthquake, as well as soil strength in terms of peak acceleration. The most reliable method for accomplishing this is to find another site similar in geologic and seismic setting where ground motion was measured during a design level magnitude earthquake. However, this will usually not be possible, and estimates of ground motion based on correlations and geologic and seismologic evidence for the specific site become necessary (see Reference 15, State-of-the-Art for Assessing Earthquake Hazards in the United States, by Slemmons).

b. Site Specific Studies. In areas where faults are reasonably mapped and studied, site specific investigations can assure that such faults are not trending towards the site and that the facility is not on an active fault. Studies may involve trenching and mapping, geophysical measurements, and other investigation techniques (see Reference 15). The extent of the area to be investigated depends on geology and the type and use of the structure. In some localities, state, or local building codes establish minimum setback distances from active faults. The minimum distance from a fault shall be 300 feet and for important structures their distances should be increased appropriately.

In seismically active areas where faults are not well mapped, site specific investigations and regional investigations may also be required. Other hazards to be considered by a site investigation include the potential for liquefaction, sliding, lurching, and flooding.

Site studies are being made for Naval activities located in seismic zones 3 and 4. These studies plus the soil data for the project will usually be adequate to assess the seismic hazard. Individual studies have been made for existing hospitals and drydocks located in seismic zones 3 and 4. A site study may occasionally be warranted for a very important structure to be located in seismic zone 2, if the mission is sensitive to earthquake damage. A critical structure (where earthquake damage could create a life endangering, secondary hazard) require special consideration in all earthquake zones.

c. Earthquake Magnitude. Ground motion parameters have been correlated with magnitude and distance by several investigators. The correlation in Figure 9 (Reference 16, Acceleration in Rock for Earthquakes in the Western United States by Schnabel and Seed) is based on ground motion records from western United States and is believed more applicable to small and moderate earthquakes (magnitudes 5.5 and 6.5) for rock and statistically applicable for stiff sites (e.g., where overburden is of stiff clays and dense sands less than 150 feet thick). For other site conditions, motion may occur as illustrated in Figure 10 (Reference 17, Relationship Between Maximum Acceleration, Maximum Velocity, Distance from Source and Local Site Conditions for Moderately Strong Earthquakes, by Seed, et al.).

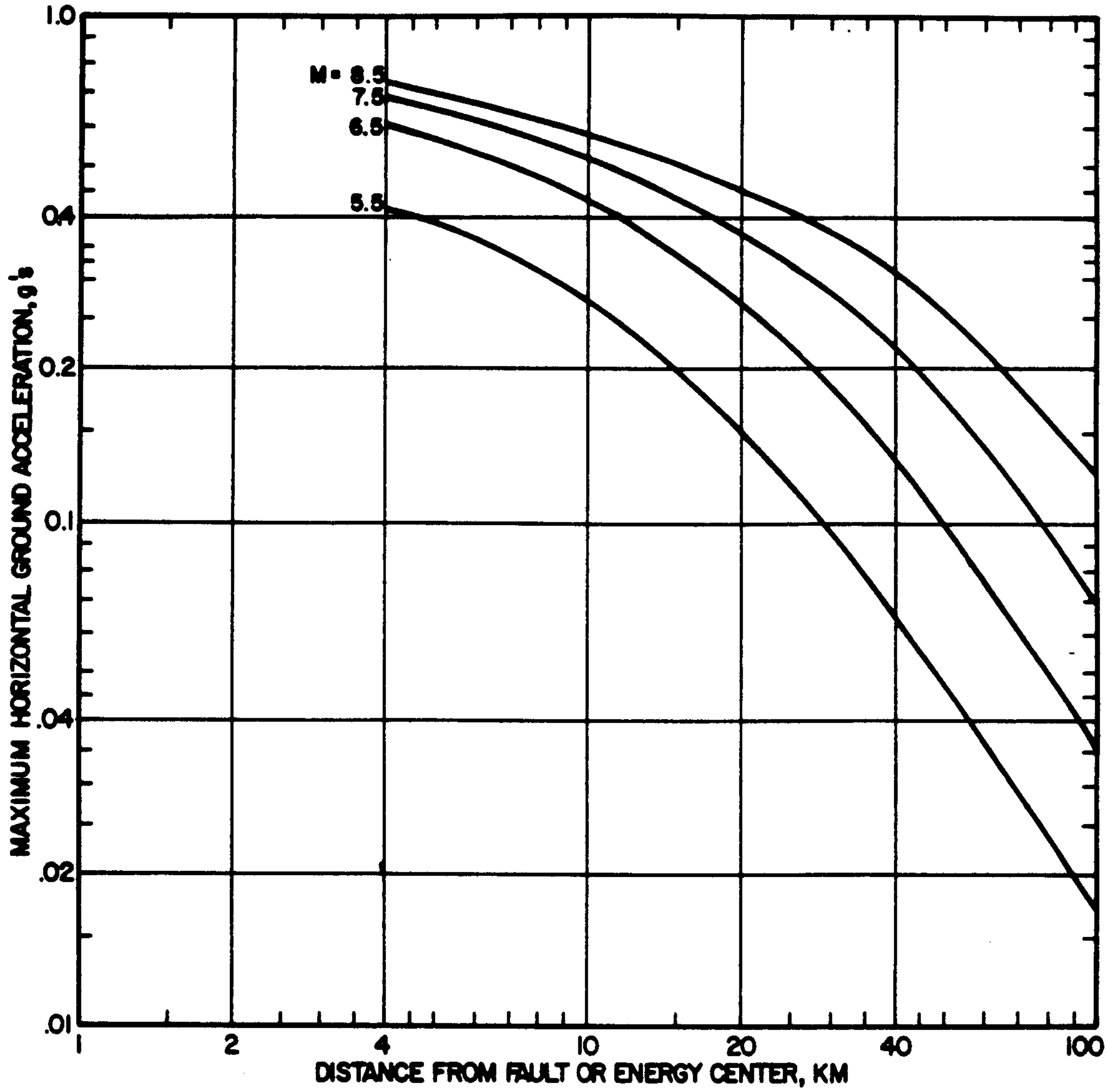


FIGURE 9
Example of Attenuation Relationships in Rock

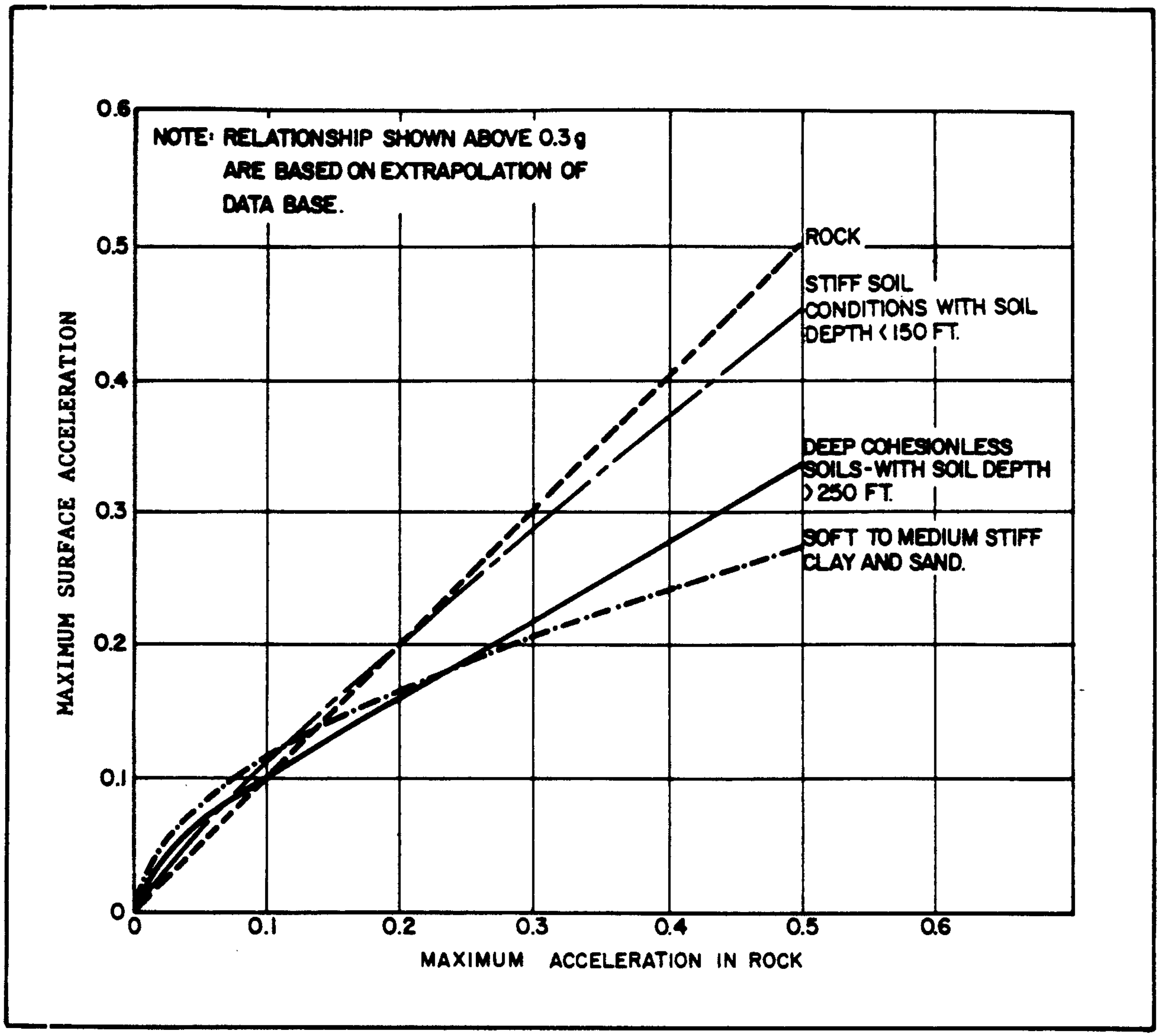


FIGURE 10
Approximate Relationship for Maximum Acceleration in
Various Soil Conditions Knowing Maximum Acceleration in Rock

Magnitude may not be the parameter controlling ground motion in the near field and large variations of acceleration for the same magnitude may be expected (see Reference 18, Fallacies in Current Ground Motion Predictions, by Bolt). The user should also be aware of new relationships appearing in the literature as more data become available. See for example Reference 19, Peak Acceleration, Velocity and Displacement for Strong Motion Records, by Boure, et al., and Reference 20, Attenuation of Strong Horizontal Ground Acceleration in the Western United States and Their Relationship to Local Magnitude, by Espinosa.

d. Intensity. In areas where active faults are not delineated, the strength of the design earthquake is usually estimated on the basis of the Modified Mercalli (MM) Intensity scale. The MM scale is a number based on mostly subjective description of the effects of earthquakes on structures and people. The MM Intensity scale has been correlated with peak acceleration by several investigators, as illustrated in Figure 11. See Reference 18, Reference 21, Earthquake Intensity and Related Ground Motion, by Neumann, Reference 22, On the Correlation of Seismic Intensity Scales with Peaks of Recorded Strong Ground Motion by Trifunac and Brady, and Reference 23, Correlation of Peak Ground Acceleration Amplitude with Seismic Intensity and Other Physical Parameters, by Murphy and O'Brien.

e. Peak Horizontal Ground Acceleration. NAVFAC has conducted seismic investigations of activities located in seismic zones 3 and 4. The seismic investigations include a site seismicity study. Where such studies have been completed, they shall be used to determine the peak horizontal ground acceleration. Where a site seismicity study has not yet been completed, it may be warranted in connection with the design and construction of an important new facility. Consult NAVFAC for the status of site seismicity investigations. In connection with soil related calculations, the peak horizontal ground acceleration for seismic zone 2 may be taken as 0.17g and for zone 1 as 0.1g. Locations of seismic zones 1 through 4 are given in NAVFAC P-355.

f. Magnitude and Intensity Relationships. For purposes of engineering analysis it may be necessary to convert the maximum MM intensity to magnitude. The most commonly used formula is that in Reference 24, Seismicity of the Earth, by Gutenberg and Richter.

$$M = 1 + 2/3\ I_{MM}$$

The above formula was derived to fit a limited data base primarily composed of western United States earthquakes. It does not account for the difference in geologic structures or for depth of earthquakes which may be important in the magnitude - intensity relationship. For other relationships see Reference 25, State-of-the-Art for Assessing Earthquake Hazards in the United States - Report 13, by Yegian.

g. Reduction of Foundation Vulnerability to Seismic Loads. In cases where potential for soil failure is not a factor, foundation ties, and special pile requirements can be incorporated into the design to reduce the vulnerability to seismic loads. Details on these are given in NAVFAC P-355 and in Reference 24. In cases where there is a likelihood for soil failure (e.g., liquefaction), consider employing one or a combination of the stabilization techniques covered in Chapter 2 and in DM-7.2, Chapter 2.

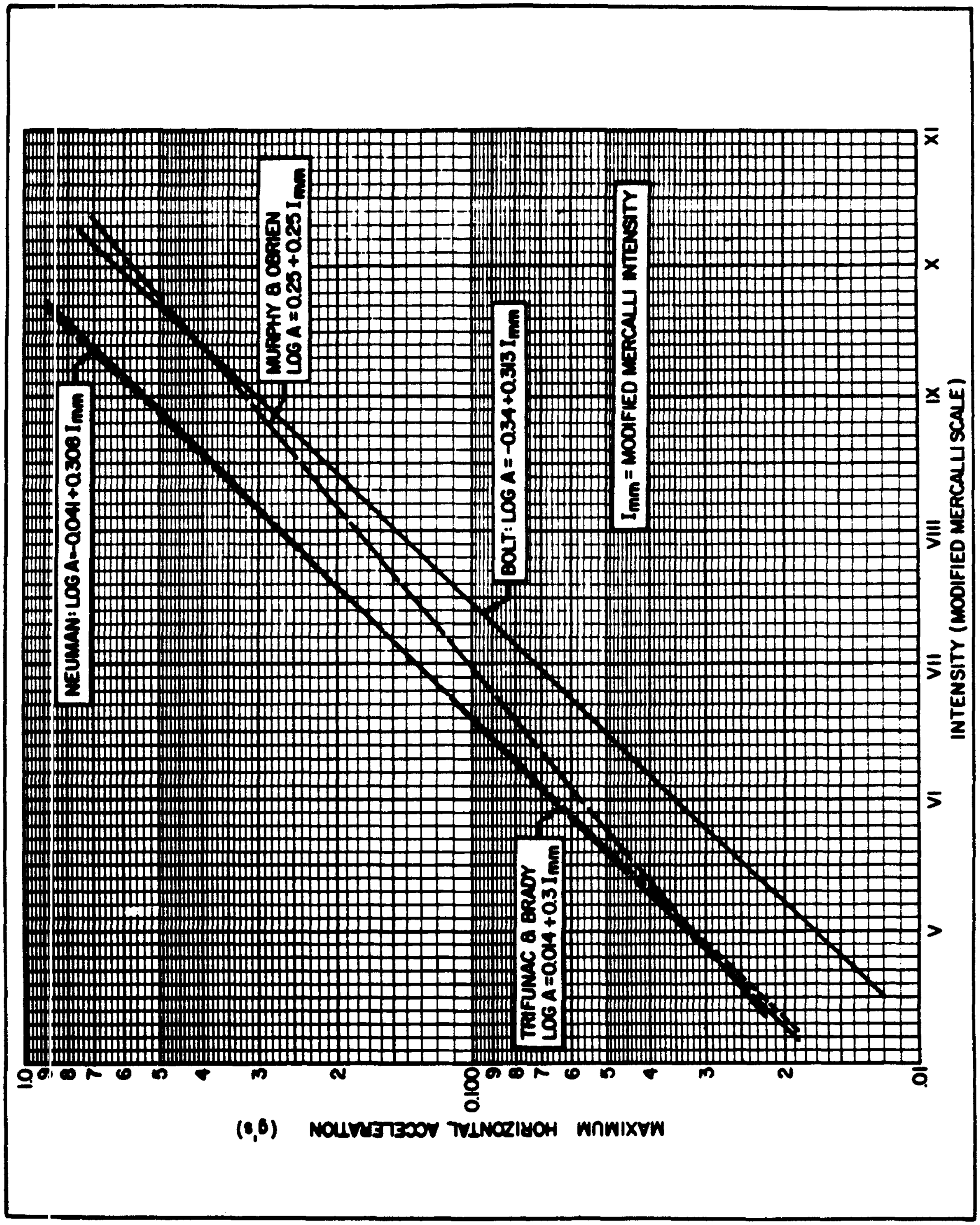


FIGURE 11
Approximate Relationships Between Maximum Acceleration and
Modified Mercalli Intensity

2. SEISMIC LOADS ON STRUCTURES. Earthquake effects from ground shaking primarily depend on the ground motion and lateral resistance of the structure. Basic criteria for structures is given in NAVFAC P-355. In a few cases, it may be appropriate to investigate soil-structure interaction.

a. Foundation Loads. The soil pressures resisting combined static and seismic loads can usually exceed the normal allowable pressure for static loads by 1/3. However, as explained in NAVFAC P-355, the various types of soils react differently to shorter term seismic loading, and any increase over normal allowable static loading is to be confirmed by soil analysis. In many cases, soil analysis is desirable where foundation soils consist of loose sands and highly sensitive clays. In addition to static stresses existing prior to earthquake motion, random dynamic stresses are exerted on the foundation soils. The shear strength of some saturated sensitive clays may be reduced under dynamic stresses, and loose to medium dense saturated granular soils may experience a substantial reduction in volume and strength during an earthquake. Special consideration should be given to the potential loss of bearing capacity or settlement of foundations on loose granular soil or highly sensitive clay.

b. Wall Loads. See DM-7.2, Chapter 3 for analysis of wall pressures to account for earthquake loading. Allowable stresses in walls or retaining structures are increased for transient shocks per NAVFAC DM-2 series.

3. LIQUEFACTION POTENTIAL. The reported damage to light buildings on soft or loose soils has not been caused by seismic building loads but by differential settlement of the surface caused by ground shaking combined with the natural variability of subsoils. Considerable damage of this sort may also occur to buildings founded on fills. In seismically active regions, every effort should be made to compact any fills used for structural foundation support. In saturated loose to medium compact granular soils seismic shocks may produce unacceptable shear strains. In such cases the high shearing deformations and decreased shear strength is the consequence of the progressive buildup of high pore pressure generated by seismic shaking and seismic building loads. With no or limited drainage, cyclic shear stresses can produce a progressive buildup of pore water pressures significantly reducing the effective stress which controls the strength. For practical purposes, the effective stress after several cycles of shear straining may ultimately be reduced to zero with total liquefaction. The progressive weakening leading to liquefaction is called cyclic mobility.

a. Factors Affecting Liquefaction. Character of ground motion, soil type, and in situ stress conditions are the three primary factors controlling the development of cyclic mobility or liquefaction.

The character of ground motion (acceleration and frequency content) controls the development of shear strains causing liquefaction. For the same acceleration, higher magnitude earthquakes are more damaging because of the higher number of applications of cyclic strain.

Relatively free draining soils such as GW, GP are much less likely to liquefy than SW, SP or SM. Dense granular soils are less likely to liquefy than looser soils. Granular soils under higher initial effective confining pressures (e.g., lower water table beneath surface, deeper soils, larger past pressure) are less likely to liquefy. Case histories indicate that the liquefaction has occurred within a depth of 50 feet or less.

b. Evaluation of Liquefaction Potential. With the present state of knowledge the prediction of liquefaction is an approximation. Two basic approaches are used:

(1) Empirical methods based on evaluation of liquefaction case histories, and in situ strength characteristics such as measured by the Standard Penetration resistance N, as outlined below:

(a) Compute the cyclic stress ratio, R_1, developed in the field during design earthquake:

$$R_1 = \frac{\tau_{av}}{\sigma_o'} = 0.65\ a_{max} \frac{\sigma_o}{\sigma_o'} r_d$$

where:

τ_{av} = average cyclic shear stress produced by design ground motion

σ_o' = initial static effective overburden stress on sand layer under consideration

σ_o = total overburden stress on sand layer under consideration

a_{max} = peak surface acceleration in g's

r_d = a stress reduction factor varying from a value of 1 at ground surface to a value of 0.9 at a depth of about 30 feet.

(b) Knowing the value of standard penetration resistance N, correct N for overburden using Figure 12. Note that N is sensitive to type of equipment used for the standard penetration test, and other factors (see DM-7.1, Chapter 2).

$$N_1 = C_N \cdot N$$

where C_N is a correction factor based on the effective overburden stress.

(c) Knowing magnitude M, and N_1, estimate cyclic stress ratio R_f required to cause liquefaction from Figure 13.

(d) Calculate factor of safety against liquefaction F_s for each layer, to obtain an appropriate factor of safety compatible with the type of structure.

$$F_s = \frac{R_f}{R_1}$$

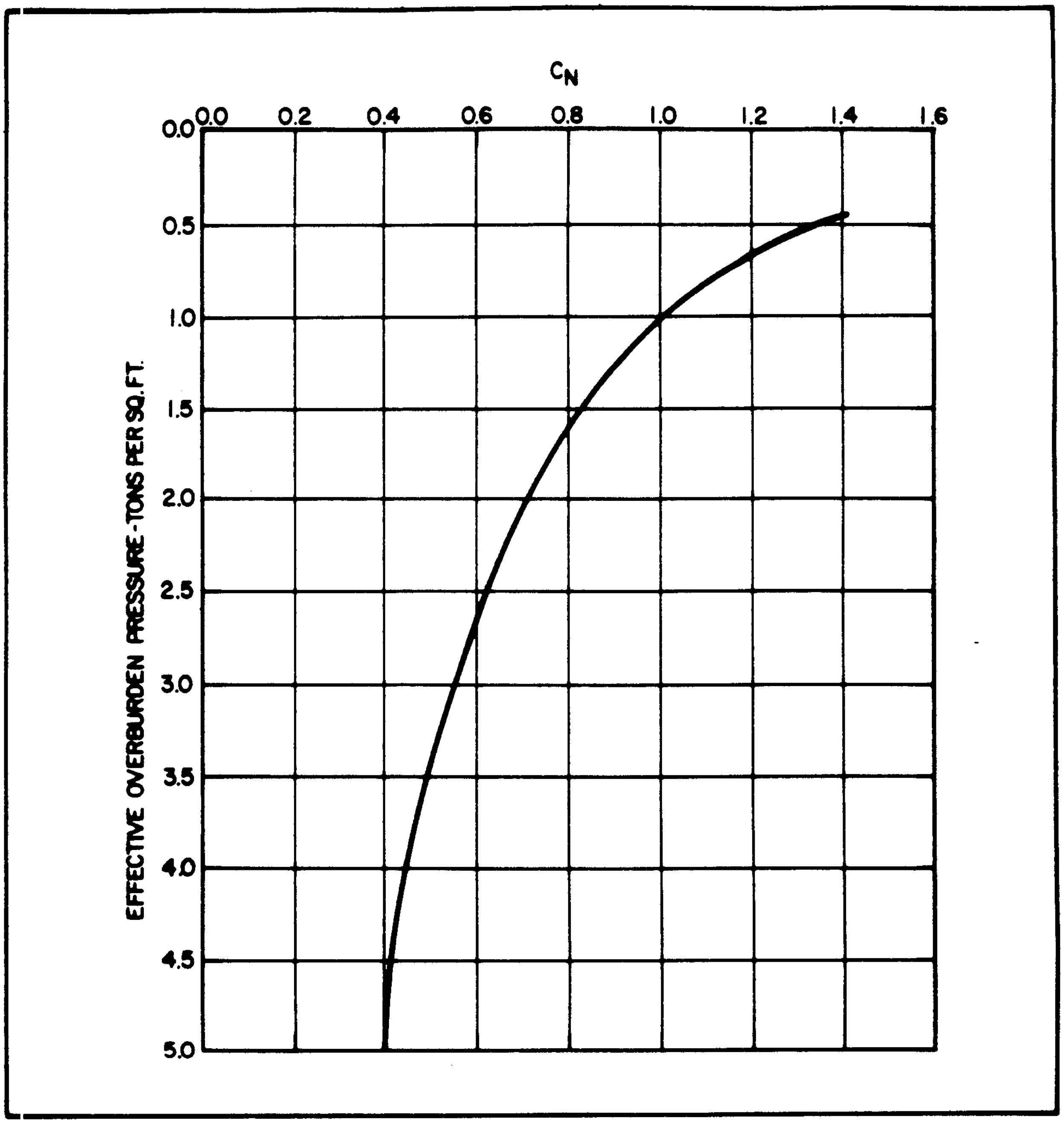


FIGURE 12
Correlation Between C_N and Effective Overburden Pressure

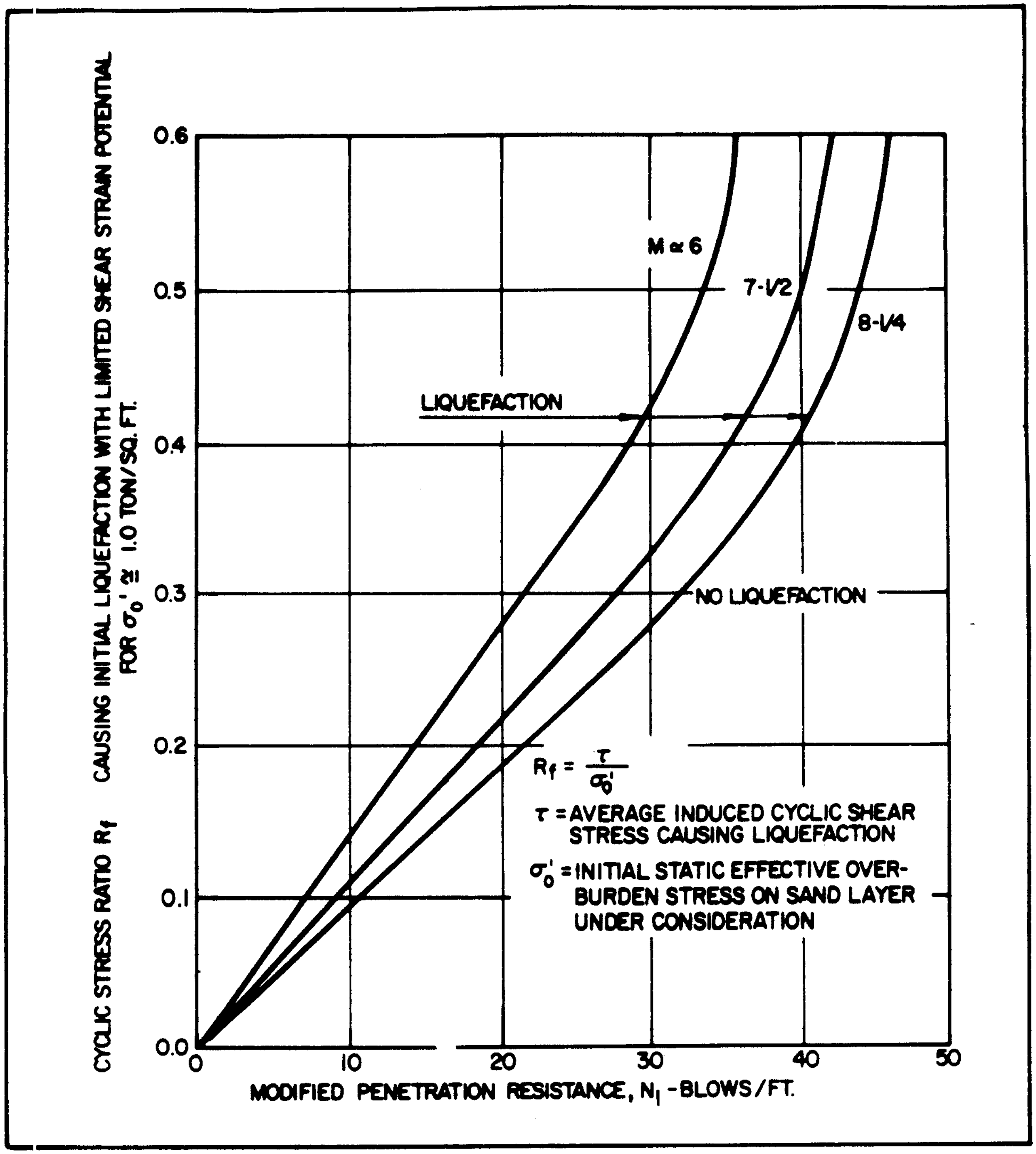


FIGURE 13
Correlation Between Field Liquefaction Behavior of Sands for Level Ground Conditions and Modified Penetration Resistance

Consult NAVFAC HQ for selection of appropriate factors of safety for design of critical structures. Use of the above procedure may be considered satisfactory for sand deposits up to 40 feet. For depths greater than 40 feet, it is recommended that this procedure be supplemented by method (2) below.

(2) Procedures Based on Laboratory Tests and Site Response Analysis. These procedures evaluate cyclic stress conditions likely to develop in the soil under a selected design earthquake and compare these stresses with those observed to cause liquefaction of representative samples in the laboratory (i.e. ratio R_f). Laboratory test results should be corrected for the difference between laboratory and field conditions. For further guidance see Reference 26, Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes, by Seed.

c. Slopes. Relatively few massive slope failures have occurred during earthquakes. Many superficial (shallow) slides have been induced by seismic loads. The performance of earth slopes or embankments subjected to strong ground shaking is best measured in terms of deformation, see Reference 27, Simplified Procedures for Estimating Dam and Embankment Earthquake Induced Deformations, by Makdisi and Seed. Saturated loose to medium dense cohesionless soils are subject to liquefaction and these soils deserve special consideration in design. Extra-sensitive clays also require special treatment.

(1) Pseudostatic Design. See DM-7.1, Chapter 7 for procedure. Pseudostatic design, including a lateral force acting through the center of the sliding mass continues to be used in practice today. Acceptable factors of safety against sliding generally range from 1.0 to 1.5 according to different codes and regulations. The most important and most difficult question in this type of analysis deals with the shear resistance of the soil. In many cases, the dynamic shear resistance of the soil is assumed equal to the static shear strength, prior to the earthquake. This would not be a conservative assumption for saturated loose to medium dense cohesionless soils. In cases of high embankments where failure may cause major damage and/or loss of life, the psuedostatic design should be verified by detailed dynamic analysis (see Reference 28, Analysis of Slides in the San Fernando Dams During the Earthquake of February 9, 1971, by Seed).

(2) Strain potential design (see Reference 28). The axial strain which occurs in triaxial compression tests during undrained cyclic shear has also been used for analysis and design, especially for earth dams. The strain potential is only a measure of field performance and is not assumed to well represent permanent deformations. A two-dimensional finite element model is normally used to calculate seismic stress histories. These stresses are then simulated, as well as possible, using existing cyclic shear equipment in the laboratory. The performance of the laboratory specimens (including liquefaction) is then assumed to be a measure of the performance of the field construction. Corrections may be applied to correct laboratory results to better represent field conditions.

4. SLOPE STABILITY. Well compacted cohesionless embankments or reasonably flat slopes in insensitive clay, which are safe under static conditions are unlikely to fail under moderate seismic shocks (up to 0.15 g or 0.20 g acceleration). Embankment slopes made up of insensitive cohesive soils founded on

cohesive soil or rock can withstand higher seismic shocks. For earthen embankments in seismic regions, provide internal drainage and select core material best resistant to cracking. In regions where embankments are made u of saturated cohesionless soil, the likelihood for liquefaction should be evaluated using detailed dynamic analysis (see Reference 28). Slope materials vulnerable to earthquake shocks are:

(a) Very steep slopes of weak, fractured, and brittle rocks or unsaturated loess are vulnerable to transient shocks due to opening of tension cracks.

(b) Loose, saturated sand may be liquefied by shocks with sudden collapse of structure and flow slides.

(c) Similar effects are possible in sensitive cohesive soils with natural moisture exceeding the liquid limit.

(d) Dry cohesionless material on a slope at the angle of repose will respond to seismic shock by shallow sloughing and slight flattening of the slope.

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CHAPTER 2. DEEP STABILIZATION AND GROUTING

Section 1. INTRODUCTION

1. SCOPE. Materials, procedures, and applicability of methods for stabilizing soil or rock are described in this chapter. Methods include densification, drainage, changing soil properties at depth by grouting, injection, dynamic consolidation, surcharging and freezing.

2. RELATED CRITERIA. For detailed criteria concerning stabilization for specific purposes, see the following sources:

Subject	Source
Methods of Decreasing or Accelerating Settlements	DM-7.1, Chapter 5
Reservoir Impermeabilization	DM-7.1, Chapter 6
Slope Stabilization	DM-7.1, Chapter 7
Stabilization by Drainage	DM-7.1, Chapter 6
Densification by Surface Compaction	DM-7.2, Chapter 2
Stabilization by Reinforcement	DM-7.2, Chapter 3
Stabilization for Roads	DM-5.4

3. APPLICATIONS. If the soil conditions at the site are poor for the proposed structure, in place treatment can be used to improve properties such as increasing shear strength, increasing resistance to dynamic loading, decreasing expected settlement, and decreasing seepage loss. Improvements may be necessary for foundations of embankments and structures, or where unsuitable soils, such as collapsing soils, waste fills of dredged materials, or mine tailings, are encountered. These methods can also be used to stabilize slopes and sides of excavations. The selection of a particular method of stabilization or grouting is dictated by the soil or rock properties, intended purpose, and economics. The range of particle sizes for which some methods of stabilization are appropriate is shown in Figure 1 (Reference 1, Improving Soil Conditions by Surface and Subsurface Treatment Methods - Overview, by Mitchell).

Section 2. DEEP STABILIZATION

1. PROCEDURES. Several methods are available for improving the properties of soils at depth. The choice of method depends upon the type of soil to be improved (sand, clay, etc), type of structure to be built, area and depth of treatment required, material available for use in the treatment, effect of treatment on the environment and adjacent structures, time available, and the cost.

2. DENSITY CONTROL. The relative increase in soil density at depth due to any of the treatments can be approximated by correlation with Cone Penetration Tests, Standard Penetration Tests, pressuremeter and other in situ probes (see DM-7.1, Chapter 2). Tests must be performed before and after soil treatment.

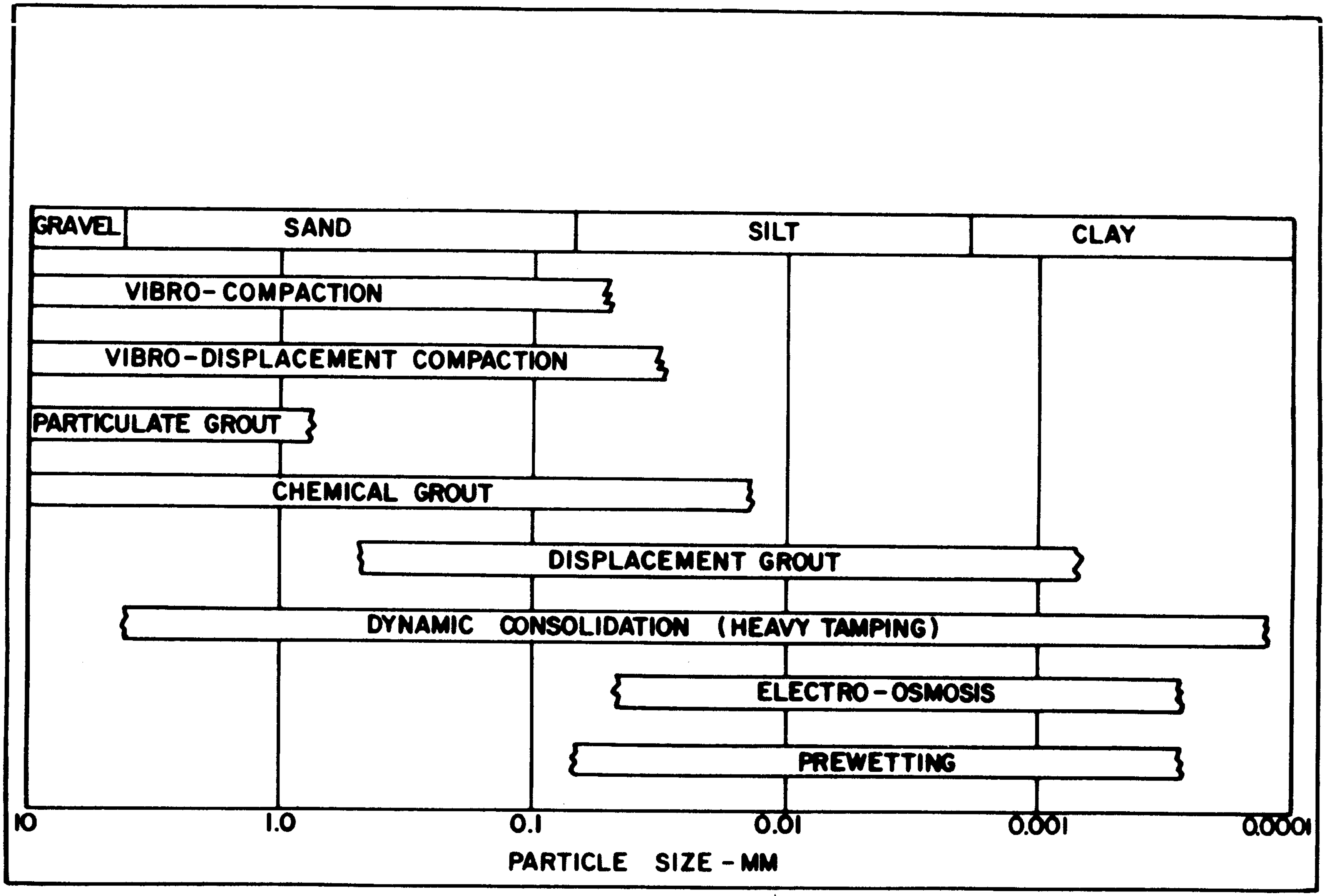


FIGURE 1
Grain Size Ranges Considered for Different Stabilization Methods

3. VIBRO-DENSIFICATION. Stabilization by densifying in-place is used primarily for granular soils where excess pore water may drain rapidly. It is effective when the relative density is less than 70%. At higher densities, compaction may not be needed and may even be difficult to achieve. By proper treatment, the density of the soil in place can be increased considerably to a sufficient depth so that most types of structures can be supported safely without undergoing unexpected settlements. Table 1 summarizes the procedures and applicability of the most commonly used in-place densification methods. Figure 2 (Reference 1) indicates the range of grain size distribution for soils amenable to vibro-densification. Effectiveness is greatly reduced in partly saturated soils in which 20% or more of the material passes a No. 200 sieve.

a. Vibrating Probe (Terraprobe). A 30-inch O.D., open-ended pipe pile with 3/8 inch wall thickness is suspended from a vibratory pile driver operating at 25 Hertz (Hz). Use a probe length 10 to 15 feet greater than the soil depth to be stabilized. Vibrations of 3/8 to 1 inch amplitude occur in a vertical mode. Space probes at 3 to 10 feet intervals. After sinkage to the desired depth, hold the probe for 30 to 60 seconds before extraction in 1.5 to 3 minutes. Backfilling is not required. Effective treatment depths range from 12 to 60 feet. Areas in the range of 450 to 700 square yards may be treated per machine per 8-hour shift. Establish test sections about 30 to 60 feet on a side to evaluate effectiveness and required probe spacing. Consider a square pattern with a fifth probe at the center of each square. Saturated soil conditions are necessary. Underlying soft clay layers may dampen vibrations.

b. Vibrodisplacement Compaction. The methods in this group are similar to those described in the preceding section. The vibrations are supplemented by active displacement of the soil and, in the case of vibroflotation and compaction piles, supplemented by backfilling the zones from which the soil has been displaced.

(1) Compaction Piles. Partly saturated or freely-draining soils can be effectively densified and strengthened. Drive displacement piles at 3 to 6 feet centers. Use either an impact hammer or a vibratory driver. Introduce sand or other backfill material in lifts with each lift compacted concurrently with withdrawal of the pipe pile. The resulting compacted column expands laterally below the pipe tip.

(2) Heavy Tamping. Drop a heavy weight (10-40 tons or more) from a height of 50 to 130 feet at points spaced 15 to 30 feet apart over the area to be densified. Apply a total energy of 2 to 3 blows per square yard. In saturated granular soils the impact energy will cause liquefaction followed by settlement as the water drains. Radial fissures that form around the impact points will facilitate drainage. The method may be used to treat soils both above and below the water table. In granular soils, the depth to which densification is significant is controlled mainly by the energy per drop. Use the following relationship to estimate effective depth of compaction:

TABLE 1
Stabilization in Depth by Vibro-Densification

Method	Procedure Used	Applications and Limitations	Modification of Soil Properties
Vibrating Probe	Open ended pipe 30 in. dia. and 3/8" wall thickness is inserted in the ground by a vibratory pile driver operating at 25 Hz. Probes are spaced 3 to 10 feet and are held in place from 1/2 to 1 minute after reaching the desired depth. No sand need be fed around the pile. A surcharge of sand placed on the area compensates for the settlement due to densification.	Sand must be completely saturated and clean. Difficult to penetrate stiff overlying layers. Soft lower layers may dampen vibrations.	Rela ive density may be increased t 55 - 90 percent. Maximum dept of treatment in responsive soil 65 ft. Allowable bearing pressures for structures 3 TSF and greater.
Compaction Piles	Short displacement piles or mandrel with temporary plug are driven. During withdrawl of mandrel, the hole is backfilled with soil or soil cement mixture.	Applicable in loose, coarse-grained materials, or in special cases, in dry sandy silts and loess which are unsaturated and have numerous air voids, and in partially saturated clayey soils.	Radius of influence of compaction is about 5 times pile diameter. Relative density increased typically 30 percent in loose sand, 10 percent in medium compact sand. Good uniformity. Maximum depth of improvement about 65 ft. Allowable pressures in granular soils 3 TSF or greater, 1-2 TSF in other soils.

TABLE 1 (continued)
Stabilization in Depth by Vibro-Densification

Method	Procedure Used	Applications and Limitations	Modification of Soil Properties
Heavy Tamping	Heavy weights (typically 10-40 tons) are dropped repeatedly from height of 50 to 130 ft on points 15 to 30 ft apart. Tamper mass times the height of fall should be greater than the square of the thickness of the layer to be densified. A total energy of 2 to 3 blows per square yard is considered adequate.	Can be used both above and below the ground-water level. In granular soils high energy impact causes partial liquefaction. Low frequency vibrations are produced which make the use of this method less desirable in urban areas and near existing structures. Not a proven technique in saturated fine-grained soils.	Relative density may be increased to 70 to 90 percent. Relatively uniform increase in density. Maximum depth of improvement about 90 feet.
Vibroflotation	Large vibrating spud is jetted into the ground. During withdrawl, water jets directed downward from the spud combined with vibrating action compact material below point of spud while sand is fed around spud from surface. It may be used for densification of an entire area or under isolated footings.	Greatest effect in relatively uniform coarse-grained soils with less than about 20 percent passing the No. 200 sieve. In dirtier material with more fines, the excess water cannot be expelled to permit densification. Suitable above or below the ground water table.	Relative density increase to 70% or higher depending upon soil and spacing of vibrator. Improvement to a maximum depth of 100 ft. Uniform increase in relative density. Allowable bearing pressure 3 TSF or more depending upon the treatment received.

TABLE 1 (continued)
Stabilization in Depth by Vibro-Densification

Method	Procedure Used	Applications and Limitations	Modification of Soil Properties
Vibroflotation with Stone Columns	Holes are jetted into the soil using water or air, and backfilled with densely compacted coarse gravel.	Used in soft fine grained soils (clays and silts). Faster than preloading.	Increased allowable bearing capacity and reduced settlement. Maximum depth of improvement about 65 ft. The properties of soil are relatively unchanged.

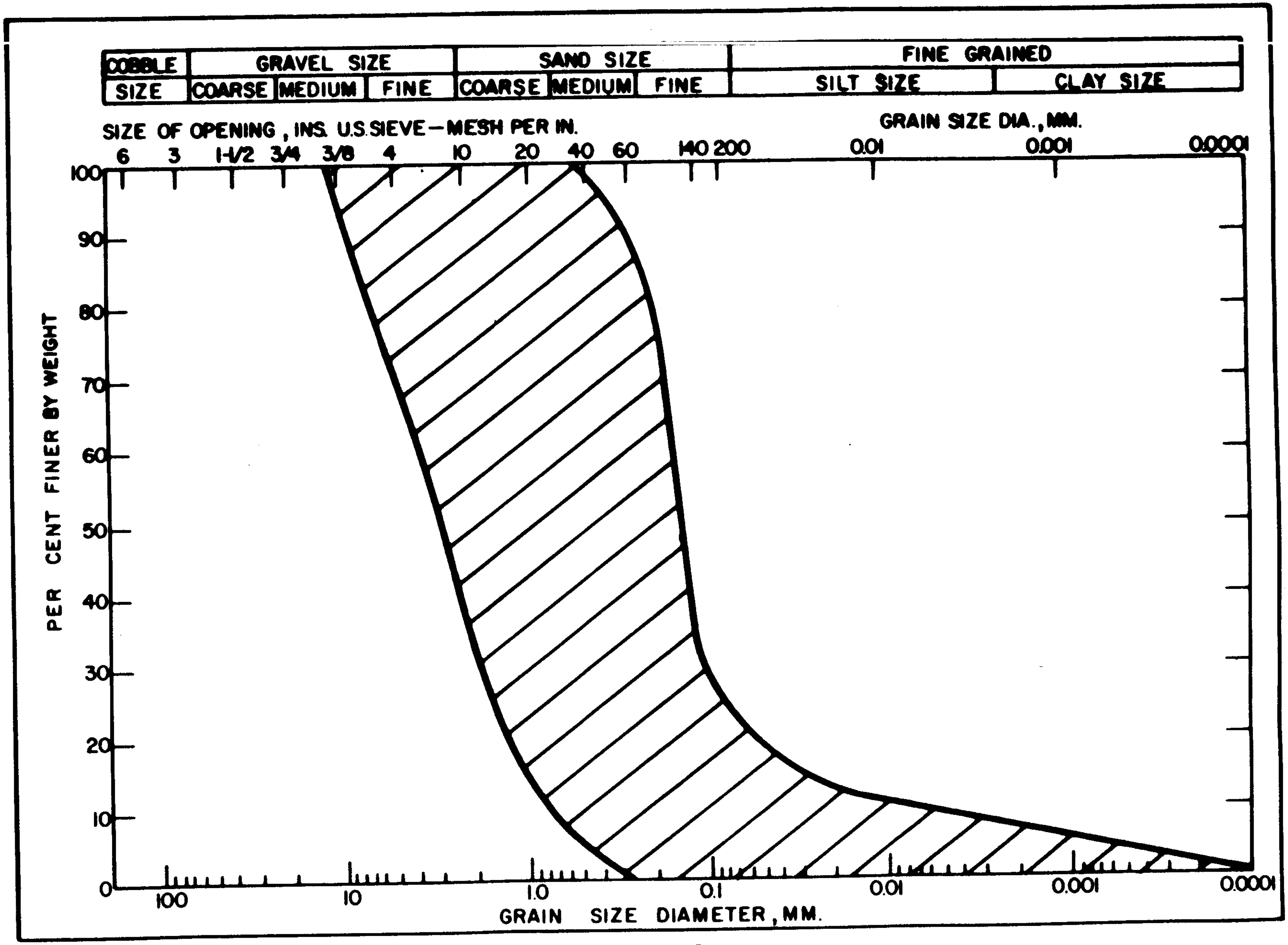


FIGURE 2
Grain Size Ranges Considered for Vibro-Densification

$$D = 1/2\ Wh$$

where:

D = depth of influence, in feet
W = falling weight in tons
h = height of drop in feet

Relative densities of 70 to 90 percent can be obtained. Bearing capacity increases of 200 to 400 percent are usual for sands. A minimum treatment area of 4 to 8 acres is necessary for economical use of the method. This method is presently considered experimental in saturated clays. Because of the high-amplitude, low-frequency vibrations (1 to 12Hz), maintain minimum distances from adjacent facilities as follows:

Piles or bridge abutment	15 - 20 feet
Liquid storage tanks	30 feet
Reinforced concrete building	50 feet
Dwellings	100 feet
Computers (not isolated)	300 feet

(3) Vibroflotation. Vibroflotation is used to densify granular soils. For the optimum soil gradation for densification see Figure 2. A crane-suspended cylindrical penetrator about 16 inches in diameter and 6 feet long, called a vibroflot, is attached to an adapter section containing lead wires and hoses. Electrically driven vibrators have RPM's in the order of 1,800 to 3,000. Hydraulically driven vibrators have variable frequencies. Total weight is generally about two tons. Power ranges between 30 and 134 Hp are available with corresponding centrifugal force ranging from 10 to 31 tons and peak-to-peak amplitude ranging from 3 to 10 inches. To sink the vibroflot to the desired treatment depth, a water jet at the tip is opened and acts in conjunction with the vibrations so that a hole can be advanced at a rate of 18 inches per minute. The bottom jet is then closed and the vibroflot is withdrawn at a rate of about one ft/min for 30 Hp vibroflots and approximately twice that rate for vibroflots over 100 Hp. Concurrently, a sand or gravel backfill is dumped in from the ground surface and densified. Backfill consumption is at a rate of about 0.5 to 1.5 cubic yards per minute. In partly saturated sands, water jets at the top of the vibroflot can be opened to facilitate liquefaction and densification of the surrounding ground. Most of the compaction takes place within the first 2 to 5 minutes at any elevation. See Figure 3 (Reference 1) for guidance on the relationship between vibration center spacing versus relative density. For guidance on the relationship between spacing and allowable bearing pressure with respect to settlement see Reference 1. Equilateral grid probe patterns are best for compacting large areas, while square and triangular patterns are used for compacting soils for isolated footings. See Table 2 (after Reference 1) as a guide for patterns and spacings required for an allowable pressure of 3 tsf under square footings using a 30 Hp unit.

(4) Vibro-Replacement (Stone Columns). The vibro-replacement method is a modification of the vibroflotation method for use in soft cohesive soils.

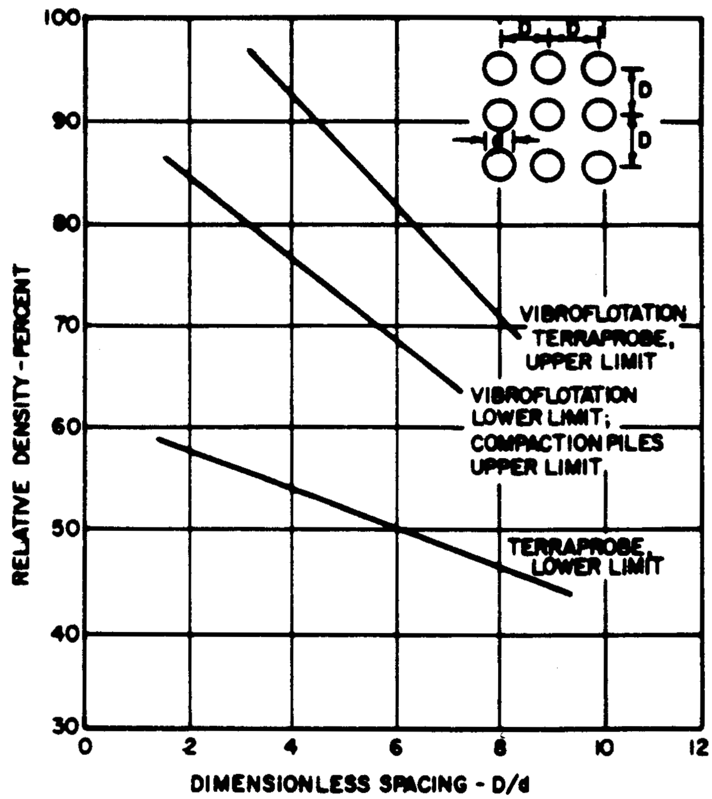


FIGURE 3
Relative Density vs. Probe Spacing for Soil Stabilization
by Three Methods

TABLE 2
Examples of Vibroflotation Patterns and
Spacings for Isolated Footings

Desired allowable bearing pressure = 3 TSF			
Square Footing (size - ft.)	Number of Vibroflotation Points	c-c Spacing (feet)	Pattern
<4	1	---	---
4.5 - 5.5	2	6	Line
6 - 7	3	7.5	Triangle
7.5 - 9.5	4	6	Square
10 - 11.5	5	7.5	Square plus one @ center

Use a vibroflot to make a cylindrical, vertical hole by jetting to the desired depth. Dump in 0.5 to 1 cubic yard of coarse granular backfill (well graded between 1/2 and 3 inches), let the vibroflot compact the gravel vertically and radially into the surrounding soft soil. Continue the process of back-filling and compaction by vibration until the densified stone column reaches the surface. The diameter of the resulting column will range from about 2 feet for stiffer clays (undrained shear strength greater than 0.5 tsf) to 3.5 feet for very soft clays (undrained shear strength less than 0.2 tsf). The host soil surrounding the dense gravel columns is relatively unaffected by the action of the vibrator. Space stone columns on 3 to 9 feet on centers, in square or triangular grid patterns under mat foundations. Cover the entire foundation area with a blanket of sand or gravel at least one foot thick to help distribute loads and to facilitate drainage. Calculate the allowable stress on a stone column q_a by:

$$q_a = \frac{25s_u}{F_s}$$

where: s_u = undrained shear strength of surrounding soil

F_s = factor of safety

A factor of safety of 3.0 is recommended.

(5) Vibroflotation and Vibro-Replacement. Where both cohesive and granular soils exist, vibro-compaction is combined with vibro-replacement using granular fill (1/2 to 2 inches). In addition to compaction of natural soil between probe positions, a stone column is also formed at points of penetration. Where layered sands and silts occur, such as estuarine deposits, this method is useful.

(6) Figure 3 (Reference 1) presents a comparison of various methods of vibrodensification in regard to relative density and probe spacing.

4. DRAINAGE. A soil mass can be stabilized by drainage, reducing the water content, or increasing effective stress. Methods include drawing down the water table, reducing excess pore water pressures built up under load, and drainage by electro-osmotic forces (see Table 3). Further guidance on drainage is given in DM-7.1, Chapter 6.

a. Gravity Drainage. Materials stabilized range down to silt sizes, but also include stratified sand-silt-clay, or clay and rock with water-bearing fractures, fissures, or lenses.

b. Reduction of Excess Pore Water Pressures. Surface load is applied at the ground surface in the form of an earth fill or water fill. This results in buildup of excess pore water pressure. Drainage of pore water is accelerated by vertical drains or sanded wellpoints. Alternately, a vacuum may be applied to the soil causing the atmospheric pressure to act as load. These methods are used in compressible, fine-grained soils including organic materials.

TABLE 3
Stabilization by Drainage

Method	Procedure Used	Applications and Limitations	Modification of Soil Properties
Stabilization by Wells	Groundwater level is drawn down by flow to deep wells, well points, or sumps while water is pumped from them.	Generally effective in materials with no more than about 25 percent smaller than 0.05 mm. In laminated or varved fine sandy silts or varved sand-silt-clay mixture, drawdown may be effective with as much as 50 percent of the average material smaller than 0.05 mm. Selection of method depends on arrangement of permeable strata, total depth of drawdown required, and character of excavation to be protected.	Increases rigidity and strength of material by increasing effective stresses acting in the soil. Prevents erosion and piping from breakout of seepage in the excavation. If sufficient time is available, drawdown will consolidate compressible silt-clay strata.
Reduction of Excess Pore water Pressures by Drainage	Vertical sand drains (see DM-7.1, Chapter 5) or sanded wellpoint holes under superposed load, or sanded wellpoint holes with vacuum seal are installed in compressible stratum. Pore pressures exceeding boundary pressures in the drain holes cause drainage of pore water.	Applicable to soft and compressible, unstable fine grained soils with high void ratio, including organic materials.	Accelerates drainage of pore water pressure by providing closely spaced drains at atmospheric or less than atmospheric pressure. This speeds consolidation and increase in shear strength.

c. Drainage by Electro-osmosis. Soils treated include silt and clay too fine to be drained by gravity with a coefficient of permeability in the range of $2x10^{-4}$ to $2x10^{-6}$fpm. Electro-osmosis develops tension in pore water, causing consolidation and gain in strength of compressible soils. Careful study of soil characteristics is required to evaluate suitability of electro-osmosis. Highly plastic clays require approximately ten times the energy input of nonplastic silts. Method is usually expensive. In many cases, other stabilization methods are more cost effective.

5. SPECIAL METHODS.

a. Inundation of Foundation Soils. This method is used primarily in two situations, as follows:

(1) In conjunction with pumping from wellpoints to densify loose, coarse-grained fills. Seepage directed downward towards wellpoints applies a consolidating force.

(2) For prewetting loessial silts and other collapsible soils and fine sands of low density and low natural moisture content. Inundation may result in a compression of about 4 to 8 percent of the original thickness of loose silts. Inundation of clayey loess may not be effective unless a surcharge is applied in conjunction with the wetting. The purpose of this treatment is to induce collapse, thus increasing the density of the foundation soil for support of a structure or for excavating (see DM-7.2, Chapter 1). Current methods of treatment of collapsible soil are described in Table 4 (Reference 2, Soil Improvement, History, Capability and Outlook, by the American Society of Civil Engineers).

b. Balancing Pressure of Compressed Air. Compressed air is applied to stabilize excavations for tunnels and deep vertical shafts. The following should be noted:

(1) The method is effective over a wide range of soil types but is most frequently applied to silts and clays near the liquid limit, or to fine sandy silts that are difficult to drain by gravity.

(2) In coarse sand and gravel, clay blanketing of open faces may be necessary to avoid air loss or blowouts.

(3) Generally, compressed air is not applied for hydrostatic heads exceeding 50 psi.

c. Freezing. Stabilization by freezing has been performed as a construction expedient in excavation and shaft sinking where compressed air is not practical, or where soils are too fine-grained to be drained by gravity or so pervious that the flow cannot be controlled. Methods include circulation of chilled calcium chloride brine in boreholes, expansion of carbon dioxide into a circuit of freezing pipes, or direct application of solid carbon dioxide and alcohol. Freezing is relatively costly and is generally utilized only where conditions are difficult for alternative procedures and are conducive to freezing. For further guidance see Reference 3, Lateral Support Systems and Underpinning, Vol. III, Construction Methods, by Goldberg, et al.

Table 4
Methods of Treating Collapsible Foundation Soils

Depth of Subsoil Treatment Desired	Foundation Treatment Method
0 to 5 feet	1. Moistening and compaction (conventional, extra-heavy, impact, or vibratory rollers)
5 to 30 feet	1. Overexcavation and recompaction (with or without stabilization by additives such as lime or cement) 2. Vibroflotation (free-draining soils) 3. Stone Columns (Vibro-replacement) 4. Displacement piles 5. Injection of silt or lime 6. Ponding or flooding (if no impervious layers exist)
Over 30 feet	1. Any of the above or combination of the above methods, where applicable 2. Ponding and infiltration wells

Section 3. STABILIZATION BY GROUTING

1. APPLICATION. Grouting is done to increase shear strength, to densify, to stiffen, or to decrease the permeability of soil or rock. It is often used as a remedial expedient in construction, to deal with unanticipated problems of flowing water, or loss of formation strength. The process is also gaining acceptance as a preconstruction procedure to eliminate problems that might otherwise occur during the construction phase.

The use of grouting, either as a planned part of construction procedure or as a remedial measure, is dependent upon its being cost effective compared to other alternatives. In general, grouting is most likely to be cost effective in treating zones of limited volume at substantial distances from an accessible location.

All grouting is done for one or more of the following reasons:

a. Impermeabilization and Water Cutoff. Reduction of seepage or larger flows into excavations is generally done with chemical grouts. Required pressures vary with formation and depth, and location with respect to the excavation. Grout curtains around and under dams are generally large operations, thoroughly preplanned. Three or more rows of grout holes may be used with center row serving the additional function of monitoring grouting results in the outer rows.

b. Strengthening Formations and Reducing Settlements. Strengthening is often done with chemicals, sometimes with cement grouts, if those will penetrate, and usually at low pressures to avoid fracturing. Reducing settlements is done with either chemicals or cement grouts. In some cases, fracturing may be deliberate to cause excursions of lenses or fingers of solid grout and thus densify the formation. Strengthening is often required under existing foundations where nearby cuts are being made, or where scour, erosion, or settlement has reduced the original bearing capacity of the soil. The purpose of the grouting is to increase the shear strength by increasing the cohesion component of strength. Grouting, when properly done around underground excavations, can reduce surface settlements.

c. Filling Voids. Generally done with cement based grouts. Sealing foundation strata near the surface is termed area or blanket (low pressure) grouting. Grouting between a man-made structure and the formation (e.g., tunnels) is termed contact grouting.

3. FIELD INVESTIGATION. It is essential to know the types of materials present before planning a grouting program. Field investigations may be required to determine the nature, scope, and cause of the problem for which grouting is being considered and to evaluate groutability of the formation.

Soil boring and sampling should be used to define the location of the strata or mass to be grouted. Samples should be classified for grain size and plasticity (for grouting purposes, field classification by a qualified soils engineer or geologist is generally adequate). Laboratory permeability tests are generally of limited additional value if grain size information is available.

Field permeability tests, particularly those which are packer isolated (see DM-7.1, Chapter 2), are highly desirable. A factor of major importance in such tests is that they must be continued until field equilibrium conditions are established.

The most meaningful test for determining formation groutability is a field pumping test. In such tests, a fluid with a viscosity similar to that of the proposed grout is pumped directly into the formation at rates, pressures, and volumes consistent with the proposed field work. These tests should be performed with the equipment that will be used for the actual grouting. A successful pumping test is virtually always indicative that the formation is groutable.

3. EVALUATING FORMATION GROUTABILITY. The economic feasibility of grouting depends to a large extent upon the rate at which the formation will accept grout. The following broad generalizations can often aid preliminary evaluations.

a. Grain Size. Granular deposits classified as finer than coarse sand or equivalent rock fissure openings cannot be grouted with suspended solids type of grouts. Granular deposits classified as finer than medium sand (or equivalent rock fissure opening or sandstone) cannot be grouted with the more viscous chemical grouts. Materials classified as clay cannot be grouted.

b. Permeability. Formations with permeabilities of 10^{-1} cm/sec and more will accept suspended solids grouts. Formations with permeabilities more than 10^{-3} cm/sec will accept the more viscous chemical grouts. Formations with permeabilities more than 10^{-4} cm/sec will accept the less viscous chemical grouts. Formations with permeabilities of 10^{-5} cm/sec and less are generally ungroutable.

c. Viscosity. Coarse sands and gravels will accept (in addition to suspended solids grouts) chemical grouts with viscosities up to 50 centipoises per second (cps). Medium sands will accept up to 15 cps. Fine sands will accept up to 5 cps. Silts will accept up to 2 cps.

d. Grouting Materials. Cement grouts are suitable for coarse sands and gravels. The Joosten process and most other high strength silicate formulations are suitable for medium sands. Medium viscosity (and low strength) silicate formulations, the aminoplasts and the phenoplasts are suitable for fine to medium sands. The polyacrylamides are suitable for fine sands and coarse silts. None of the grouts will penetrate clay.

4. MATERIALS, EQUIPMENT, AND PROCEDURES. Table 5 describes the materials, procedures and applicability of the most commonly used grouting methods.

a. Selection of Grouting Materials. The choice of a grouting material depends upon the size of the voids and the purpose of grouting. For filling large voids, or grouting formations of coarse sand or larger particles, or grouting rock fissures half a millimeter (0.02 inch) or larger, suspended particle grouts such as cement, bentonite and sanded cement may be used.

TABLE 5
Stabilization by Grouting

Process	Materials, mixtures and admixtures	Procedure	Applicability
Sand-Cement Grouting	Typical sand-cement ratio of loose volume varies from about 2:1 to 10:1. Addition of bentonite or fly ash reduces segregation and increases pumpability. Water-cement ratios from about 2:1 to 5:1 by volume.	Pumped in conventional grout system with slush pumps of long stroke having large valve openings. Generally mixing facilities are larger than for neat cement grouts. Volume of water required for pumpability varies with the sand-cement ratio roughly in the range of equal volumes of water and sand to a volume of water equal to 1/3 volume of sand.	Used for grouting large foundation voids, for mud jacking, or for contact grouting on the periphery of a structure or tunnel, or used without pressure or at low pressure for slush grouting to fill surface irregularities in rock foundation on which embankment is to be placed. Grout will penetrate gravels depending on gradation of sand in mix. D_{10} size of gravels generally about 3/4 in. For usual mixes, strength varies with water content from 100 to 700 psi.
Clay-Cement Grouting	Typical clay (CH) - Portland cement ratio of loose volumes varies from about 3:1 to 8:1. Water-clay ratios from about 3:1 to 10:1 by volume.	Setting time is delayed and grouting can be continuous or intermittent without danger of losing the hole. Before mixing, clay should be screened to remove erratic larger particles.	Penetrates sand of approximately the same grain size as does neat cement grout. Use for grouting comparatively large voids where clay admixture is included for economy. Strength of mix depends on water-cement ratio and averages about 100 psi for typical mix.

TABLE 5 (continued)
Stabilization by Grouting

Process	Materials, mixtures and admixtures	Procedure	Applicability
Cement Bentonite Grouting	Dispersed cement slurry with bentonite as a floculation agent to prevent sedimentation of cement particles. Water-cement-bentonite properties determined by strength requirements, water tightness requirements or pumpability.	A mixture of dry cement and bentonite powder is added to water or a pre-gel of bentonite is added to cement slurry.	Relatively low strength grout used to grout rock fissures and granular soil to reduce permeability and improve strength utilizing sleeve pipe multi-stage grouting. Nominal spacing of grout pipes about 4 feet. Cement-bentonite can also be used as permanent cut-off wall material to provide an impervious water barrier.
Portland Cement Grouting	Used with water-cement ratios generally between 1:1 and 4:1. Admixtures include: bentonite; silica gels; pozzolans to reduce bleeding and segregation; lignosulphonates to increase pumpability; set accelerators such as calcium chloride.	Pumped under low pressure for blanket grouting or under high pressure for deep cutoff grouting. Water-cement ratio and type and amount of admixtures are varied in the field to alter consistency of mix and penetration. For increased penetration scalped Type III cement finer than about 0.03 mm is used.	Utilized in coarse-grain soils with D_{10} size as small as about 1 mm. Penetrates loose sands with D_{10} of 0.8 mm or dense sands with 1.2 mm D_{10} size. Fissures can be grouted in the range of about 0.06 to 0.01 mm depending on pressures, water-cement ratio, and cement types. Not appropriate for grouting large voids with vigorous ground water movement.

TABLE 5 (continued)
Stabilization by Grouting

Process	Materials, mixtures and admixtures	Procedure	Applicability
Bituminous Emulsions	Bitumen particles with diameter between 0.001 and 0.005 mm are dispersed in water. Before injection, a substance such as an ester of formic acid is added which hydrolyzes to act as a coagulant.	Injected by grouting system. Speed of coagulation may be greatly influenced by chemical composition of soil or groundwater so that careful control is necessary to obtain desired penetration.	Utilized to decrease permeability in sands with D_{10} size as small as about 0.1 mm. Increase in strength is relatively insignificant. Currently very limited use in the U.S.
Single Solution Sodium Silicate Grouting	Sodium silicate with a setting agent such as sodium biocarbonate in water solution.	Sodium silicate and setting agent are premixed in proportions to obtain setting time in a range from a few minutes to several hours. Mixture is injected in driven pipes or pipes in boreholes.	Used to decrease permeability in sands with D_{10} size as small as about 0.008 mm. Compressive strength of grouted sand is very low. Not a permanent grout.
Single Solution or Two Solution Sodium Silicate Grouting	Sodium silicate with organic and inorganic accelerators and hardeners, sometimes with Portland cement.	May be premixed as a single solution or pumped by separate pumps and hoses to the grout pipe. A wide range of viscosities, strengths and setting times may be attained.	Used primarily for increasing strength of granular deposits. Will penetrate sand down to D_{10} size of 0.08 mm. Will give strengths up to 200 psi, and decrease permeability to 10^{-4} - 10^{-5} cm/sec.

TABLE 5 (continued)
Stabilization by Grouting

Process	Materials, mixtures and admixtures	Procedure	Applicability
Two-Stage Sodium Silicate Grouting	Two materials consist of sodium silicate and calcium chloride.	Two fluids are injected successively. Reaction between them is almost instantaneous and calcium silicate is precipitated in soil voids. Rapidity of the chemical action requires care in the injection to avoid premature contact of the chemicals.	Penetrates sands with D_{10} size as small as 0.08 mm. Permeability is reduced. Compressive strength of grouted sand ranges from about 500 to 1,000 psi.
Acrylamide	Generally used as 7 to 10 percent solution in water with catalyst controlling gel time such as ammonium persulfate.	Always pumped by separate pumps and hoses to the grout pipe. Lowest viscosity of all the chemical grouts. Gel times from several seconds to many hours may be used. Grout may be used to control the setting time of cement mixtures. Powder and solution are neurotoxic. Final gel is innocuous.	Penetrates silt and sand with D_{10} size as small as 0.013 mm. Applicable to grouting in moving groundwater because gelling time can be made very short. Compressive strength of grouted sand ranges from 50 to 100 psi.

TABLE 5 (continued)
Stabilization by Grouting

Process	Materials, mixtures and admixtures	Procedure	Applicability
Chrome Lignin	Various combinations of a lignosufate and a hexavalent chromium salt, usually in combination with an acid salt and other reagents. Available in preblended and proportioned formulations for ease of application.	Pumped through injection points either as a single solution or as two solutions blended in a mixing manifold at one point of injection. Setting time from one minute to several hours varied by adjusting water content. Strength inversely proportional to water content.	Used to decrease permeability in sands with D_{10} size as small as 0.08 mm. Compressive strength of grouted sand ranges from 20 to 50 psi. Limited use in the U.S. due to toxicity of dichromates.
Aminoplasts	Urea formaldehyde (or other formaldehydes) generally set by an acid or acid salt.	May be premixed as a single solution or pumped by separate pumps and hoses to grout pipe.	Penetration and strength similar to two-solution sodium silicates. Sets on the acid side, so is particularly useful in coal mines. Cannot be used after cement injections. Limited use in the U.S. due to health hazard.
Phenoplasts	Phenolic resins generally containing 150-cyanates, set by various salts.	Generally handled as two solutions. Formulations for low viscosity and high strength are available.	Penetration and strength ranges between acrylamide and two solution silicates. Limited use in U.S. due to health hazard.

Cement and bentonite grouts are used successfully in slightly finer formations. However, in fine and medium sands, as well as in sandstones and siltstones and fine rock fissures, these grouts will not penetrate, and chemical grouts must be used.

(1) Cement and Sanded Cement Grouts. Cement and sanded cement grouts, when injected in sufficient quantity, will add significantly to the strength of a fractured formation. They are considered permanent grouts. Bentonite, on the other hand, does not contribute significantly to the long term strength of a grouted formation. Small quantities of bentonite increase the settling time of the cement particles. Further, it is subject to removal by flowing water within the formation.

(2) Chemical Grouts. With the exception of sodium silicate with sodium bicarbonate solutions, chemical grouts are considered permanent. A wide range of viscosities and strengths are available. Table 6 lists some of the commercial products and general property ranges. For additional guidance see Reference 4, Chemical Grouts for Soils, Volume 1, Available Materials, by the Federal Highway Administration, and Reference 5, Chemical Grouting Technology, by Karol.

Except for the silicates, all chemical grouts are to some degree toxic and hazardous. Use in the field must be in compliance with common sense, and good practice in handling hazardous materials.

b. Grouting Equipment. Suspended solids grouts are almost always applied by a batch system, in which the ingredients are mixed with water in a tank and then pumped directly into the formation. In batching, the tank must be emptied before the grout begins to set. Since cement grouts have long setting times, batch systems are adequate. However, chemical grouts are often used at short gel times, and require at least two separate tanks each with its own pump. The components of the grout are pumped separately to the point where the grout enters the formation. Mixing occurs in the formation, permitting very short gel times.

Grouting of both cement grouts and chemical grouts is often done through open-ended grout pipes. More and more chemical grouting is now being done with special grout pipes (tube-a-manchete) which permit close control of the stratum being grouted at any given time, thus resulting in increased cost-effectiveness.

c. Pressures and Volumes. Grouting pressure must be carefully controlled. Safety considerations normally impose a grouting pressure limitation that in turn limits pumping volume. In theory, a liquid injected in a horizontal sheet between two strata can cause uplift if the liquid unit pressure exceeds the unit weight of overburden. In practice, a conservative rule of thumb is to limit grouting pressures to 3/4 to 1 psi for each foot of overburden depth. This is applicable to using large volumes of cement grouts in fractured rock under dams. For most other cases, higher grouting pressures are reasonable and safe. Such pressures vary with the formation and job, and whether fracturing may be permitted.

TABLE 6
Chemical Grouts

	Viscosity	Strength
SILICATES		
Joosten Process	high	high
Siroc	medium	medium-high
Silicate-Bicarbonate	medium	low
LIGNOSULFITES		
Terra Firma	medium	low
Blox-all	medium	low
PHENOPLASTS		
Terranier	medium	low
Geoseal	medium	low
AMINOPLASTS		
Herculox	medium	high
Cyanaloc	medium	high
POLYACRYLAMIDES		
AV-100	low	low
Rocagel BT	low	low
Nitto-SS	low	low

d. <u>Field Control</u>. Control of a field grouting operation is largely done on the basis of details of volume and pressure at each grouting location. Assuming adequate equipment, whose volume and pressure outputs are variable during pumping, the only regular field check which must be made is of the grout itself.

There may be no immediate way to measure the effectiveness of the grouting operation, which must then be inferred from the degree to which the planned volume of grout was placed in the proper location. Adequate, detailed field records are mandatory for such inference. Where the purpose of grouting is to reduce permeability, the effectiveness may be obvious if seepage or flows are reduced or shut off, or it may be possible to install piezometers in some instances.

REFERENCES

1. Mitchell, J.K., Improving Soil Conditions by Surface and Subsurface Treatment Methods- Overview, ASCE Metropolitan Section, Foundation and Soil Mechanics Group Seminar, New York, Oct. 25-68, 1978.

2. American Society of Civil Engineers, Soil Improvement, History, Capability and Outlook, Report by the Committee on Placement and Improvement of Soils of the Geotechnical Engineering Division of the ASCE, February 1978.

3. Goldberg, D.T., Jaworski, W.E., and Gordon, M.D., Lateral Support Systems and Underpinning, Volume III, Construction Methods, FHWA-RD-75-130, Federal Highway Administration, April, 1976.

4. Federal Highway Administration, Chemical Grouts for Soils, Volume I, Available Materials, Report No. FHWA-RD-77-50, Washington, D.C., June, 1977.

5. Karol, R.H., Chemical Grouting Technology, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 94, No. SM1, 1968.

CHAPTER 3. SPECIAL GEOTECHNICAL CONSTRUCTION

Section 1. INTRODUCTION

1. SCOPE. This chapter addresses problems related to specialized types of construction including tunneling, dredging, underpinning, and offshore platforms; problems related to special soil and rock types; and some special structures.

2. RELATED CRITERIA. Many of the criteria applicable to construction and structures referred to in this chapter are addressed in other chapters of this Manual and other publications.

Section 2. TUNNELING

1. CONTROLLING FACTORS. Factors which generally control design and construction of tunnels include:

(1) Overall geologic setting;
(2) Soil and/or rock material;
(3) Groundwater regime;
(4) Proximity and types of adjacent overlying structures;
(5) Consequences of ground loss (subsidence);
(6) Type of tunneling equipment;
(7) Rate of tunnel advancement.

2. LOADS ON TUNNELS. Refer to DM-7.1, Chapter 4 for criteria and procedures for determining design loads on tunnels. Design of tunnel linings (primary and/or permanent) is dependent on the type of lining and method of construction employed. See Reference 1, Tunneling in Soft Ground, Geotechnical Considerations, by Peck; Reference 2, State of the Art of Soft Ground Tunneling, by Peck, et al.; Reference 3, Deep Excavations and Tunneling in Soft Ground, by Peck; Reference 4, The Uncertain Equation Between Design and Construction in Soil Engineering for Excavations, Deep Foundations, and Tunnels, by Milligan; and Reference 5, Rock Tunneling with Steel Supports, by Proctor and White.

3. SOFT GROUND TUNNELING

a. Tunneling Methods. Tunneling methods can be broadly categorized as follows:

(1) Hand mined method,
(2) Shield method,
(3) Machine method.

Mechanized face excavating and spoil removal equipment is becoming more frequently used, usually with shield tunneling methods, to replace hand-labor. Many variations in construction techniques can be applied to each method. See Reference 6, Earth Tunneling with Steel Supports, by Proctor and White, and Reference 7, The Role of the Tunneling Machine, by Hamilton for guidance. The need for compressed air depends on soil and groundwater conditions.

b. Key Considerations

(1) Subsurface Conditions. Detailed subsurface investigations along proposed tunnel routes must be performed to define soil, rock, and groundwater conditions. These factors affect the tunnel design, methods and equipment required, and economics. Such investigations should attempt to define problem soil/groundwater areas so as to plan special procedures for treating problem soil or groundwater conditions prior to start of construction.

Procedures for treating soil and groundwater problems should consider:

(1) Dewatering
(2) Soil freezing
(3) Grouting
(4) Use of compressed air.

As unexpected problems in tunneling can occur, which no reasonable subsurface investigation could have anticipated, the engineer and contractor must be prepared to respond rapidly to such problems and implement appropriate solutions. Selection of the solution depends on the nature and extent of the problem condition, proximity of the tunnel to above structures or utilities, and the likelihood of encountering similar conditions elsewhere along the tunnel. (See References 4 and 6.)

(2) Loss of Ground-Subsidence. A likely consequence of tunneling is some loss of ground or subsidence of overlying soil. This is a result of one or more of the following:

(a) Deformations due to stress release which develop in the soil ahead of the tunnel as the supporting soil is excavated at the tunnel face.

(b) Radial deformations as the surrounding soil moves into the annular spaces between the excavated soil and the tunnel shield or that left by the tailpiece clearance.

(c) Deformations related to the quality of workmanship; such as the timeliness and adequacy of grouting and filling the annular spaces around the shield/tailpiece, control of pitching and yawing of the tunneling machine, etc.

(d) Deformations resulting from large ground movement at the tunnel face, usually due to unexpected changes in soil or groundwater conditions. See Reference 4 for guidance on surface settlement and volume of settlement trough above tunnel.

(3) Stability of Tunnel Face. Factors affecting face stability include:

(1) Shear strength and soil stress-strain relationship;
(2) Overburden pressure;
(3) Groundwater regime;
(4) Tunnel geometry and cross-section;
(5) Time dependent soil strength loss and delayed soil deformation;
(6) Construction techniques.

In cohesive soils, face stability is generally related to undrained shear strength (c) and overburden pressure (γZ), where γ is the soil unit weight and Z is depth. Typical limiting values, or the threshold of serious face in stability problems, is $\gamma Z/c \geq 6$ for open air tunnels, and for compressed air tunnels $(\gamma Z - P_a)/c \geq 6$, where P_a = air pressure. (See Reference 4.)

As the value c is time dependent, the rate of tunnel advance must be considered in determining strength values for calculating face stability. Face stability can be improved by the use of one or more of the techniques listed in paragraph 3.b.(1). For guidelines and case histories on tunnel face stability problems, see Reference 4.

4. ROCK TUNNELING.

a. Tunneling Methods. Methods employed depend on rock behavior and the geometry and size of the tunnel excavations. For details on some of the common tunneling methods for manually advanced tunnels, see Reference 5.

Mechanized tunneling methods are becoming more common as technology and equipment improve. Systems include fully mechanized tunnel boring machines and mechanical face excavating equipment to replace the standard drilling and blasting operations.

b. Key Considerations.

(1) Subsurface Conditions. Detailed geotechnical investigations should address:

(1) Geologic conditions along tunnel route especially strike and dip of geologic features such as faults, fractures, bedding, and in situ stresses,
(2) Classification and extent of defective rock intrusions,
(3) In situ stress state of the rock formations,
(4) Zones of soft or squeezing ground,
(5) Groundwater regime,
(6) Potential tunneling problems,
(7) Procedures for treating anticipated problems.

(2) Tunnel Hazards. Potential tunneling hazards peculiar to certain geologic formations, include the following:

(1) Limestone - cavities containing water and/or sand, crushed zones, possible presence of CO_2 or H_2S gas.

(2) Sandstone - crushed zones, possible presence of CO_2 or H_2S gas.

(3) Shale - presence of methane gas in coal bearing shale, high swelling pressures where layers of anhydrite are present, calcium sulfate (attacks concrete), and hydrogen sulfide in infiltrating water which flows across anhydrite layers.

(4) Schist - heavy squeezing and swelling pressures in chemically altered schists, possible large water inflow especially where fractured or folded.

(5) Extrusive Igneous - Unconsolidated strata of decomposed tuff and breccias, water from fault zones, possible harmful gases.

For additional information on rock tunneling practices, see Reference 3.

Section 3. DREDGING

1. INTRODUCTION. Dredging is normally used to either: (1) provide fill material for waterfront construction, or (2) deepen shipping lanes or harbors. Periodic dredging is necessary in many areas because of silting (maintenance dredging).

2. FIELD INVESTIGATION FOR DREDGE OPERATIONS. Both the area to be dredged and the disposal area must be thoroughly investigated by borings and hydrographic surveys. The spacing and depth of borings is dependent on the specific project requirements; borings spaced at 200 to 300 feet are typical.

3. ENGINEERING ASPECTS OF DREDGE OPERATIONS.

a. Material. The type of material available and its position must be considered when planning a dredging operation. Dredging of soft soils can cause environmental problems with turbidity, while dredging of blasted rock may cause difficulties with dredge production. Granular material may run (or flow) into a point of excavation allowing substantial excavation without moving the dredge. Excavation of stiff cohesive material will require nearly constant movement of the intake. The depth of the material as well as the type of material to be dredged will influence the selection of dredge type as well as the rate of production.

b. Dredging Equipment. Dredges are of two types: mechanical and hydraulic. Hydraulic dredges are most commonly used in the United States. Table 1 lists typical types of dredges and their most frequent applications.

TABLE 1
Dredge Types

Type		Suitable Material	Special Characteristics
HYDRAULIC	Pipe Line - Plain suction	Soft - loose soil	
	Pipe Line - Cutterhead	Soft soil to blasted rock	Cutterhead can be varied to suit characteristics of material.
	Pipe Line - Dust Pan	Soil only	Especially suitable for removing sand bars.
	Hopper	Soil only	Self-propelled; especially suitable for excavating channels while underway without anchor or mooring.
	Sidecasting	Soil only	Self-propelled; effective where litoral currents do not return dredged material.
MECHANICAL	Ripper	Soft soil to blasted rock	Mechanical dredges normally used in conjunction with barges, therefore smaller disposal area is required, less turbidity during excavation and filling, and less particle separation by size.
	Bucket	Soft soil to blasted rock	
	Ladder	Soft soil to blasted rock	

c. Length of Sluice Line. The location of the disposal area relative to the point of dredging will determine if it is practical to transport the material by sluicing through a pipeline. The length of the pipeline and required pumping can be theoretically established, but due to the leakage, special elbows, and pipe wear, the maximum length of pipeline may best be determined empirically from the past performance of dredge equipment. The maximum economic distance for hydraulic sluicing depends on many factors, including available equipment and type of material (i.e. fine-grained soils can be sluiced further than coarse-grained material with same input of energy); however, the limit for economical hydraulic sluicing is considered to be typically about five miles (Reference 8, Personal Communication with Great Lakes Dredge and Dock Company).

d. Disposal of "Deleterious" Dredge Material. Deleterious dredge material, derived from the maintenance dredging of shipping channels, is usually disposed of in diked settling basins. Design of the impoundment must consider the rate at which the soil particles will settle out of the pumped liquid and the depth of water required for this action. The following "rules of thumb" are often used to determine the size of the required disposal area. An example of the sizing of a disposal area by this method is shown in Figure 1. Further guidance can be found in Reference 9, Analysis of Dredging Projects, by Pearce.

(1) Sandy materials require 1-1/2 times the volume they occupied prior to dredging.

(2) Soft silts and other "maintenance" removed materials require about 3 times the volume they occupied prior to dredging.

(3) Approximately three to five feet for pooled water at the impoundments surface should be added to the above volume, and about two to three feet of dike free-board above the pool of water.

e. Dredging and Placing Material for use as a Structural Fill. See Table 2 (from Reference 10, Hydraulic Fills to Support Structural Loads, by Whitman) for description of the nature of hydraulically placed fills derived from different borrow sources. If the material is to be used as structural fill, the following guidelines should be considered:

(1) Borrow material being dredged hydraulically should generally have less than 15% fines (passing No. 200 sieve).

(2) Discharge of material within the fill area should be in a manner that prevents the fines from settling out in pockets or layers, i.e., ponding should be minimal with filling done in a manner that carries the fines to a waste area.

(3) The height of the mound at discharge should be restricted to minimize segregation, thus allowing the fill to be well graded. The steepness of the mound is controlled by the shear strength of the fill and the distance water can carry the soil particles before settling; for gravelly fills, slopes are typically 3 horizontal to 1 vertical. For further guidance, see Reference 9.

Example:

Find size of impoundment required for the disposal for 100,000 c.y. (in situ) of organic silt.

Silts require about 3 times the volume in situ for disposal.

$$\text{Volume of Soil After Dredging} = 3 \times 100{,}000 = 300{,}000 \text{ c.y.}$$

Area available for impoundment about 1,000 feet by 1,000 feet; allowing 50± feet on all sides for retaining dikes. Assume net area of 900 feet by 900 feet available for impoundment

$$\text{Depth of dredged soil impoundment (ft)} = \frac{300{,}000 \times 27 \text{ cf/cy}}{900 \times 900} = 10 \text{ ft}$$

Increase depth 5 feet to allow pooled surface water, plus an additional 3 feet of dike free board.

Required size of impoundment (interior):

900' by 900' in plan, 15' deep.

Adjustments in size could be made by considering exact cross sections of containing dikes.

FIGURE 1
Calculation of Required Size of Disposal Area for Dredged Soils

TABLE 2
General Characteristics of Hydraulically Placed Fills

Nature of Borrow Material	Characteristic of Fill Placed
Fairly clean sand (less than 15% passing No. 200 sieve)	Reasonably uniform fill of moderate density
Silty or clayey sand	Very heterogeneous fill of large void ratio
Stiff cohesive soil	Skeleton of clay balls, with matrix of sand and clay
Soft cohesive soil	Laminated normally consolidated or underconsolidated clay

4. IMPROVEMENT OF DREDGED MATERIAL DISPOSED AS WASTE FILL. The most common techniques for improvement and stabilization of fine-grained hydraulic fills include the use of a surcharge, gravity drainage, and dessication. Other potential techniques include chemical treatment, and electro-osmosis. Additional information may be found in Reference 11, Properties, Behavior, and Treatment of Waste Fills, by Bromwell .

Section 4. UNDERPINNING

1. PURPOSE. Underpinning is utilized to transfer a load carried on an existing foundation from its present bearing level to a new level at a lower depth. This operation may be necessary to prevent continuing settlement, to increase foundation load capacity, or to permit adjacent excavation without damage to existing structures. Underpinning elements may be either temporary or permanent. For detailed treatment of the subject see Reference 12, Lateral Support Systems and Underpinning, Volumes I, II, and III, Goldberg, et al., Reference 13, Foundation Construction, by Carson, and Reference 14, Underpinning, by White.

2. INVESTIGATIONS. Determine by exploration the materials through which underpinning must be carried and the final bearing stratum. Where settlement of existing structures has occurred, evaluate the subsurface conditions that are responsible.

3. PROCEDURE. Underpinning should be performed with a carefully planned sequence of operations. Several common methods of underpinning are illustrated in Figure 2 with a brief description of each.

a. Load Relief. Carefully examine the structure for indications of settlement or weakness that may be accentuated during underpinning. Before excavation, reduce load on existing wall or foundation as much as possible.

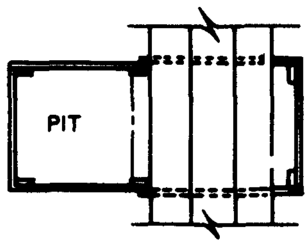
b. Excavation. Limit excavation to the minimum size necessary for underpinning in stages. Sheet and brace the excavation as necessary to prevent horizontal movement of surrounding ground. Provide for dewatering as necessary for the work to avoid piping or disturbance of bearing materials.

c. Temporary Support. Provide support of the structure over sections of the excavation by means of needles passing through, into, or under the existing structure, and support on cribs, grillages, posts, or piles. Load bearing surfaces must be kept in close contact by the use of wedges or jacks.

d. Underpinning Members. Commence underpinning construction as soon as practicable after excavation subgrade has been exposed. Underpinning may be formed of concrete walls, piers, and caissons, or bored piles, steel piles, or precast piles placed in sections.

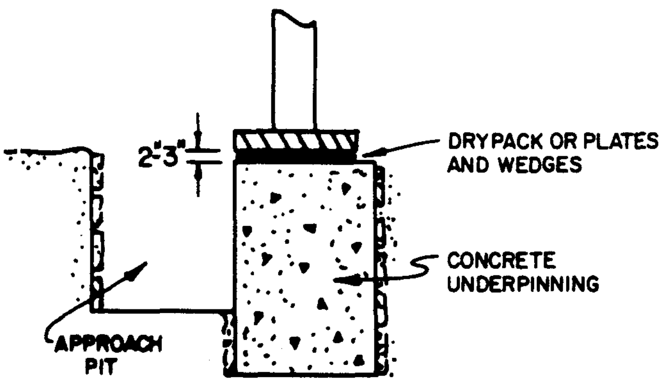
(1) Foundation. Before final underpinning in concrete, the lower sections of the underpinning should be allowed to complete their set. Final contact with structure is made by wedging between steel bearing plates or by dry-pack concrete.

PIT OR PIER UNDERPINNING



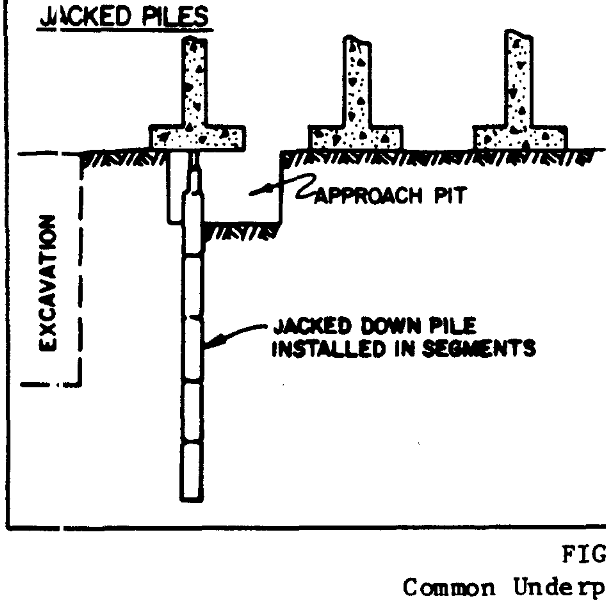
PLAN

Excavate approach pit under the footing, excavate the underpinning pit to adequate bearing material, fill the pit with concrete and transfer the foundation load.



ELEVATION

JACKED PILES

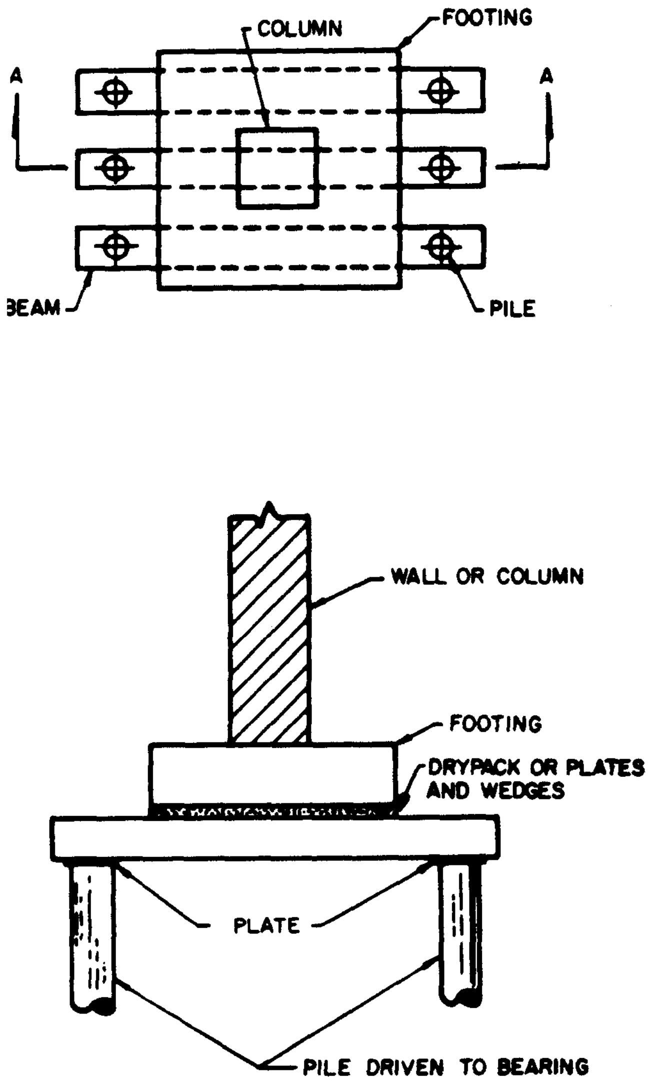


Excavate approach pit, hydraulically jack pipe segments to required depth or load resistance using foundation as reaction load, remove soil within pipe (if open-ended), fill pipe with concrete, pre-test to 150% of design load, transfer load to pile using wedging beam, plates and wedges, and remove jack. Frequently, wedging beam, plates and wedges are encased in concrete.

FIGURE 2
Common Underpinning Methods

PILES AND SUPPORT BEAMS

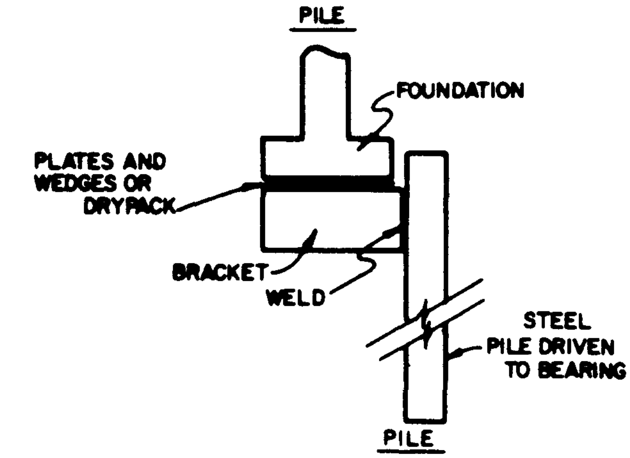
TYPICAL DETAIL



Piles may be driven or jacked, closed or open-ended. Excavate adjacent to footing to about the footing bottom, in sequence, excavate under the footing (may require sheeting) and install support beams, transfer the footing load using plates and wedges, drypack or jacks at the footing bottom and/or tops of piles.

FIGURE 2 (continued)
Common Underpinning Methods

BRACKET PILE OR BRACKET CAISSON



Support by driven steel H-pile or augered caisson. A steel or concrete bracket is welded or formed at the top of the support extending under the footing, load is transferred to the bracket using plates and wedges or dry-pack. Excavation loading of the support pile or caisson must be considered.

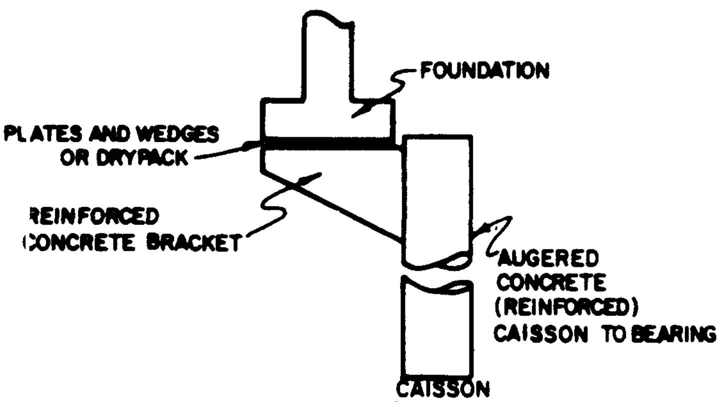
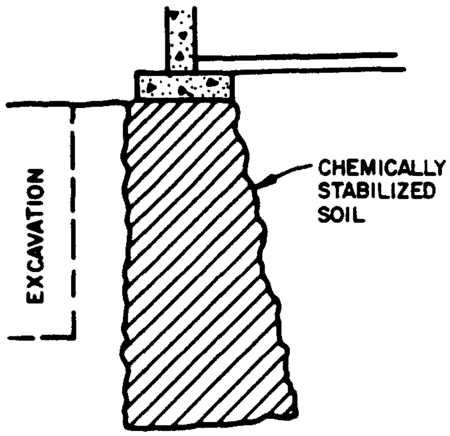


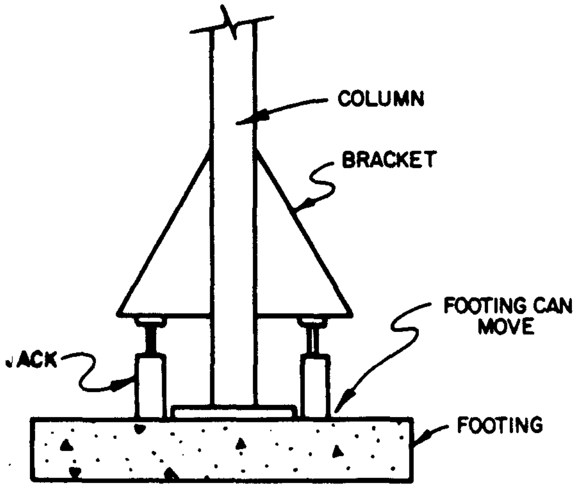
FIGURE 2 (continued)
Common Underpinning Methods

CHEMICAL, GROUT OR FREEZE STABILIZATION



The required soil mass is either chemically solidified, the voids in mass filled with pressurized grout (pervious soils), the mass densified by compaction grouting, or the mass frozen. For details on chemical grout and freeze stabilization see Chapter 1. Note: pressure grouting may also be used to raise or relevel structures or structural elements which have undergone settlement. Caution must be exercised in using grouting or freezing techniques to avoid undesired uplift pressures on walls within the zone of freezing influence.

COLUMN JACKING



Brackets are welded to column, anchor bolts are loosened or removed, jacks are installed to compensate for subsidence of foundation, shim installed between base plate and footing, jacks and brackets removed and anchor bolts reinstalled. Brackets can be left in place when repeated applications are anticipated. Frequently utilized where one-time subsidence due to adjacent or underlying construction (tunnels) is anticipated.

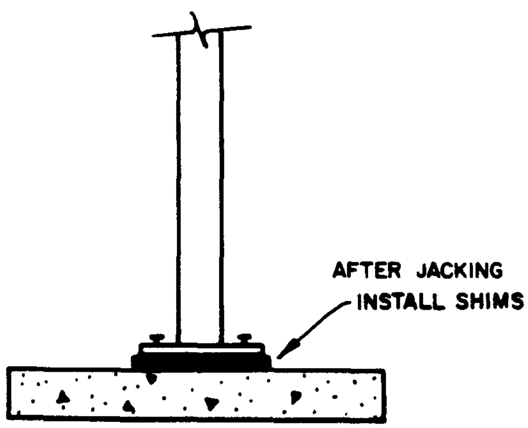
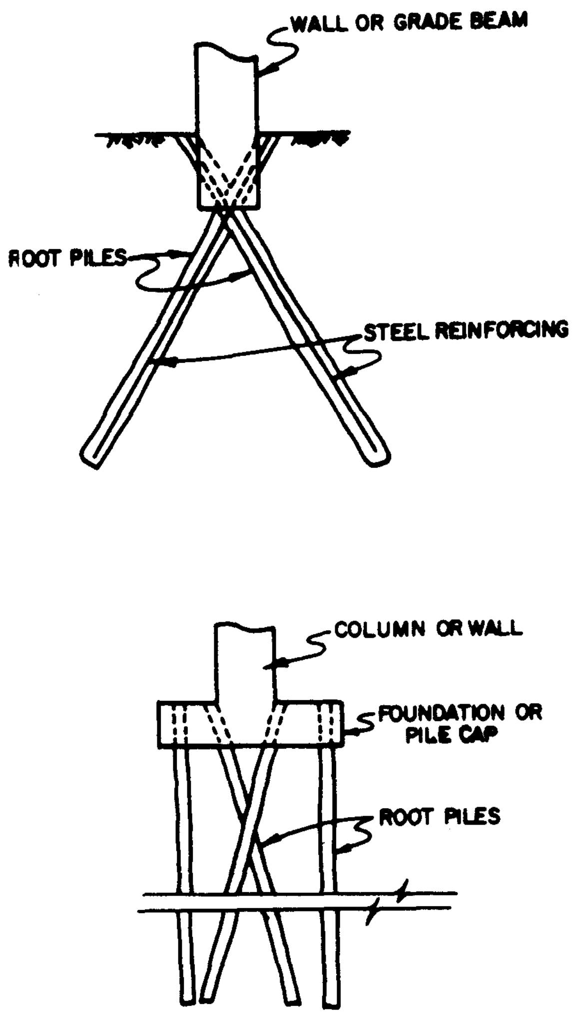


FIGURE 2 (continued)
Common Underpinning Methods

ROOT PILES



Generally installed by drilling through existing foundations. Drill holes can be cased or uncased depending on soil type. Pile holes are drilled to obtain required load capacity in friction and/or end bearing, reinforcing bars or cages installed, concreting with mortar is performed from the bottom of the hole (compaction of the mortar can be accomplished using compressed air blasts) as the casing (if any) is withdrawn. Load transfer is by bond shear between pile and foundation. Settlement due to elastic shortening of root piles must be considered.

FIGURE 2 (continued)
Common Underpinning Methods

(2) Installing Piles. Piles are installed in sections and jacked down against a reaction provided by the existing structure. In their final position, underpinning piles are generally pretested with jacking loads of 1.5 times the intended working load. Movement under pretest load should be negligible.

e. Other Techniques. Some less common methods of underpinning include chemical stabilization, grout stabilization (pressure/compaction), and soil freezing. Descriptions and details of these methods can be found in the previously cited references.

4. DESIGN. Design of underpinning elements must take into consideration the following:

(1) Loads on existing foundations - vertical and horizontal;
(2) Sensitivity of existing structures to total and differential settlement;
(3) Soil and/or geologic conditions;
(4) Groundwater conditions;
(5) Proximity and depth of planned adjacent excavations;
(6) Lateral loads on underpinning elements;
(7) Duration of time the underpinning is required.

Foundation loads for recent structures can generally be found in the design documents. For older structures, it may be necessary to analyze the structure to approximate these loads. Sensitivity to settlement should be evaluated with the assistance of a structural engineer, considering the type of construction and age of the existing structure. A detailed inventory of the structural condition, including existing cracks and other damage, should be made prior to the start of underpinning.

A detailed geotechnical investigation should be performed to evaluate subsurface conditions, the effects of planned excavations (if any), soil bearing values, appropriate depths to which underpinning must extend and lateral loads on underpinnings.

Actual design of underpinning elements should follow standard design procedures for pile or spread foundations subjected to the vertical and horizontal loadings anticipated. Stability and settlement of the structure and pressures on the braced excavation during construction may be evaluated. For underpinning piles the possibility of eventual removal of side friction restraint should be considered. Pretest loads must be increased to allow for side friction that may be removed with completion of adjacent work. See DM-7.2, Chapters 3, 4, and 5 for standard design procedures.

Section 5. OFFSHORE PLATFORM FOUNDATIONS

1. GENERAL. Many offshore facilities require either structural platforms supported by clusters of steel pipe piles, or gravity-type platforms supported by mats resting on the sea floors.

Piled template platforms have tubular substructure prefabricated on land and towed to the site. After placing on the sea floor, open end steel piles are driven through the columns.

Gravity structures are usually made of concrete. Construction begins in dry dock, and is completed in deep sheltered waters from where the structure is towed to the site and placed on the sea bottom by selective flooding. The mat foundations usually include perimeter skirts and dowels which penetrate the upper soils 10 to 12 feet.

2. SITE INVESTIGATION. The investigation program is similar to that used on land, except that the equipment used for obtaining samples is different. See DM-7.1, Chapter 2, and Reference 15, Underwater Sampling and Testing, by Noonary. Reliance is placed on in situ measurements such as vane shear tests and cone penetration tests, and geophysical testing. Geophysical measurements are correlated with boring data to better define the soil profile. See DM-7.1, Chapter 2 for detailed methods of offshore exploration.

Geophysical surveys should cover the entire area of interest with details at the area where the structure is to be built. These should include echo soundings for the bottom, and boomer-sparker profiling for detecting the depths to the various strata. Gravity structures require a large area survey, perhaps a half mile square, because the possibility for modifications after construction starts is very limited. Template structures usually require only adjustment of pile length if there is some error or change in location. The detailed exploration must be done from drilling ships, and is costly. It should include several deep borings, three or four, supplemented by cone penetration tests. Borings for pile supported structures should extend at least to a pile-group diameter below the anticipated tip elevation. In many cases, this will exceed 300 feet. Borings for gravity structures might include two borings to a depth of 1.5 times the base diameter with other borings and cone penetration tests to investigate the upper 100 feet of soil.

Obtain continuous samples of the materials to a depth of 40 feet or greater below mud line. Thereafter, sample at significant changes in strata, at approximately 10 feet intervals to 200 feet and approximately 25-foot intervals below 200 feet Perform standard penetration tests or equivalent on significant sand strata; retain and carefully package samples for laboratory test. Use vane shear tests for soft to stiff clays and also obtain "undisturbed" samples for laboratory testing. The cone penetrometer is extensively used for determining in situ soil strength characteristics. The quality of the "undisturbed" samples using nonproprietary samplers is much poorer from offshore samplers than from terrestrial samplers.

a. Scour. The potential for scour may be great where sand and silt comprise the sea floor, and perimeter skirts are usually provided on gravity structures. Some useful guidance is given in Reference 16, Scour at Bridge Waterways, by the Highway Research Board.

b. Seafloor Instability. Large movement of sea floor may result from wave action, earthquakes, etc. Detailed information such as sea bottom topography, rate of deposition, and gas content are required for the evaluation of areas suspected for sea floor movements. Large forces on foundation elements can result from sea floor movements. For further guidance, see Reference 17, Wave-Induced Slides in South Pass Block 70, Mississippi Delta, by Bea, et al.

3. SHALLOW FOUNDATIONS. The principles of design for shallow foundation are given in DM-7.1, Chapter 5, and DM-7.2, Chapter 4. Additional factors that must be considered for offshore structures include: prediction of skirt and dowel penetration at emplacement; resistance to overturning and sliding; instability due to scour; and pore water pressure build up due to construction procedures, cyclic loading, earthquakes; etc. For detailed design procedures see Reference 18, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms, by the API, and Reference 19, Design and Construction of Dry Docks, by Mazurkiewicz.

4. PILE FOUNDATIONS. See DM-7.2, Chapter 5 for design of deep foundations. The loads carried by piles supporting offshore structures are many times those on land; working loads on the order of 3,000 tons in compression, 1,000 tons in tension are quite normal. In addition, the piles must resist large lateral forces. The susceptibility of the foundations to corrosion should be considered, and appropriate precautions/compensation must be taken. In addition, see Reference 18 for methods of designing and installing piles.

Section 6. SPECIAL PROBLEM SOILS

1. SANITARY LANDFILLS.

a. Introduction. Sanitary landfills are becoming the major sites for solid waste disposal. The geotechnical engineer's role in solid waste disposal includes:

(1) Evaluation of physical and chemical material properties;

(2) Design and supervision during construction of disposal facilities;

(3) Monitoring of facilities during operation to ensure satisfactory performance; and

(4) Evaluation of potential land uses after completion of disposal operations.

b. Composition of Material. The engineering properties of sanitary landfill are largely influenced by the composition of the refuse. Reference 11 presents the results of numerous determinations of refuse composition.

c. Settlement Characteristics.

(1) Unit Weights. Table 3 (Reference 11) presents typical unit weights of municipal refuse.

(2) Subsidence of Refuse Fill Under Self-weight.

TABLE 3
Typical Unit Weights of Municipal Refuse

	Unit Weight (lbs./cu. ft.)	
	Total (γ_T)	Dry (γ_D)*
Household Trash Can	7	5
Delivery Truck	15	10
<u>Sanitary Landfill: Not Shredded</u>		
- poor compaction	20	15
- good compaction	40	28
- best compaction	60	42
Sanitary Landfill: Shredded	55	39
<u>High-Pressure Baling (3500 psi)</u>		
- during compaction	90	64
- after volume expansion	60	42
Complete Elimination of Voids	-	90

* Calculated for moisture content of 42% (dry weight basis)

(a) The following mechanisms can lead to surface subsidence:

(1) Movement of particles into large voids;

(2) Biological decomposition of organics;

(3) Chemical reactions, including oxidation and combustion;

(4) Dissolving of soluble substances by percolating groundwater or leachate;

(5) Change in deformation properties with time;

(6) Plastic flow or creep.

(b) The time-settlement relationship of subsidence under self weight is analagous to the secondary compression of soils after a short period of pseudo-primary (mechanical) settlement typically 1 to 4 months long. Measurements indicate a coefficient of secondary compression ranging from 0.1 to 0.4. Thus, settlement of the fill under its own weight after completion can be estimated by:

$$(\Delta H) = HC_\alpha \quad \mathrm{Log}\ \frac{t_2}{t_1}$$

where:

(ΔH) = settlement at time t_2 (length unit)

H = thickness of fill (length unit)

t_1 = time pseudo-primary (mechanical settlement) to occur after completion of fill

t_2 = time after completion of fill

C_α = coefficient of secondary compression (any mathematically compatible units acceptable)

(3) Subsidence of Refuse Fills Under External Loads.

(a) The time-settlement behavior of old refuse fills under an applied load is analagous to the behavior of peat. Primary settlements will likely occur as the load is applied. Secondary compression occurs over a long period of time and the amount of long-term settlement is determined by envirormental conditions (i.e. humid environment is more conducive to decomposition) as well as the composition of the refuse. Reported primary compression indexes (C_c/1+eo) ranged from 0.1 to 0.4 and the coefficient of secondary compression (C_α) from 0.02 to 0.07. These values are for fills which have undergone decomposition for some time prior to loading (10 to 15 years, typically). Higher compressibility is usually associated with high organic content and/or advanced degree of decomposition.

d. Construction Over Sanitary Landfills. Any foundation investigation for a structure being built over a sanitary landfill should include the evaluation of the following potential problems:

(1) Differential settlement of floor slabs, walls, and utilities;
(2) Irregular subsidence due to highly variable composition;
(3) Corrosion of concrete foundations and pipe utilities;
(4) Generation of methane gas (see DM-7.1, Chapter 2);
(5) Slope stability;
(6) Effect of construction on leachate control.

e. Methods of Treatment for Foundation Support.

(1) Control and compaction during placement. Compaction and shredding of refuse as it is being placed in the landfill will greatly increase its suitability for later use. The typical unit weights of municipal waste presented in Table 3 give an indication of the reduction of voids and volume by such treatment.

(2) Proofrolling of fills and replacement of soft pockets with compacted soil will reduce irregular settlements.

(3) Use of surcharge fills where refuse is thick.

(4) Deep foundations founded below the refuse fills. If piles are used provisions must be made for the corrosive environment and possible damage during driving, as well as re-sealing any holes created in leachate cutoffs.

(5) Grouting of refuse fills to stabilize voids.

(6) Use of flexible connections for utilities.

Further guidance on construction over sanitary landfills is given in Reference 20, Design and Construction of Covers for Solid Waste Landfills, by Lutton et al., and Reference 21, Development of Construction and Use Criteria for Sanitary Landfills, by the County of Los Angeles and Engineering-Science, Inc.

2. COLLAPSING SOILS.

a. General. The distinctive characteristics, geographic distribution and methods of identifying collapsing soils are given in DM-7.1, Chapter 1.

b. Foundation Difficulties. The problem of sudden settlements results from the loss of capillarity, cementation, or bonding as water comes in contact with soil. Wetting may result from landscaping, leakage through water pipes, drains, and reservoirs.

The conventional methods of sampling, where water is used for cleaning bore holes, are unsuitable for collapsing soils. For shallow depths trim specimens manually from test pits. For deep sampling, use air for cleaning bore holes and obtain undisturbed specimens using thin walled tubes.

c. Suitable Foundations. As in any other type of unsuitable foundation material, replace the collapsible soil with a noncollapsing soil or provide a deep foundation to bypass it. Alternate methods of either precollapsing by wetting or preventing water inflow are described in Reference 22, Classes of Problem Soils, by Murphy, and Reference 23, Soil Improvement, History, Capabilities, and Outlook, by the American Society of Civil Engineers.

3. SWELLING AND SHRINKING SOILS.

a. General. These soils have great potential for volume change with change in water content. Clay soils with high colloidal contents, such as montmorillonite, found in regions where high rainfalls are followed by long periods of little or no rainfall, exhibit high volume increases and decreases. The geographic distribution and methods of identification and classification are given in DM-7.1, Chapter 1. Laboratory test procedures for determining the swell potential are described in DM-7.1, Chapter 5.

b. Foundation Problems. Problems associated with swelling and shrinking soils are total and differential settlements or heave, excessive pressures on retaining structures, and cracking of embankments.

c. Suitable Foundations. Suitable foundations can be provided by removing and replacing the undesirable soil, isolating the structural element of foundation from the soil, designing a structure capable of resisting heave pressures, or preventing heave from occurring by prewetting. Prevention of water access can be accomplished by membranes and surficial grading.

Methods of estimating heave and procedures for treatment of heave are given in DM-7.1, Chapter 5. Estimating swell using the South African method is illustrated in Figure 3 (Reference 24, The Prediction of Heave from the Plasticity Index and Percentage of Clay Fraction, by Van der Merwe). This method is usually conservative. Swelling pressures are usually relieved with little displacement. It is advantageous to isolate the floor from the soil by using collapsible cardboard forms or leaving a similar void space. Further isolation is achieved by lubricating deep foundation shafts or installing them in pre-bored holes filled with vermiculite or bentonite. For backfill of retaining structures, swelling soils should not be used. See Table 4, (Reference 25, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), by the Departments of the Army and Air Force) for recommended foundation systems, Table 5 for methods of controlling heave, and Table 6 for remedial measures for existing foundations on swelling soils. For further guidance, see Reference 26, Foundations on Expansive Soils, by Chen.

d. Design Guidelines. See References 25 and 26 for guidance on design of foundation element in expansive soils. Mat foundations are usually appropriate if expansive soil extends to great depths that precludes economic use of drilled piers founded in a constant moisture zone. In cases where the potential heave is estimated at one inch or less, continuous wall footings and individual spread footings may be used in conjunction with a slab on grade. Ribbed mats (slab on grade with thickened edge and integral interior beams) may be used instead of continuous wall footings.

Procedure

1. Classify swell potential of soil using chart in DM-7.1, Chapter 1 (Very high, high medium or low).

2. Assign potential expansion (P.E.) as in./ft. of thickness based on

Swell Potential	Potential Expansion (P.E.) In./ft.
Very high	1
High	1/2
Medium	1/4
Low	0

3. Assume depth of lowest level of the groundwater table = 20 ft. Divide this thickness of 20 feet to several soil layers with variable swell potential. Assume thickness of individual layer = ΔD.

4. Calculate the factor $F = \log^{-1}(-\frac{D}{Z}) = \log^{-1}(-\frac{D}{20})$ for each soil layer. D is depth in feet to mid point of each layer.

5. Compute expansion for each individual layer $\Delta_e = (P.E.)(\Delta_D)(F)$

6. Compute total expansion $(\Delta H)_S$

$$(\Delta H)_s = \sum_{e=1}^{n} \Delta_e$$ where n is number of soil layer.

FIGURE 3
Estimating Swell Using the South African Method

EXAMPLE:

LAYER	THICKNESS (FT.) Δ_D	P.E.	D (FT.)	F	Δ_e (INCHES)
1	5	0	2.5	0.75	0
2	8	1	9	0.35	2.80
3	2	1/2	14	0.20	0.20
4	5	1	17.5	0.13	0.65
					$\Sigma = 3.65$

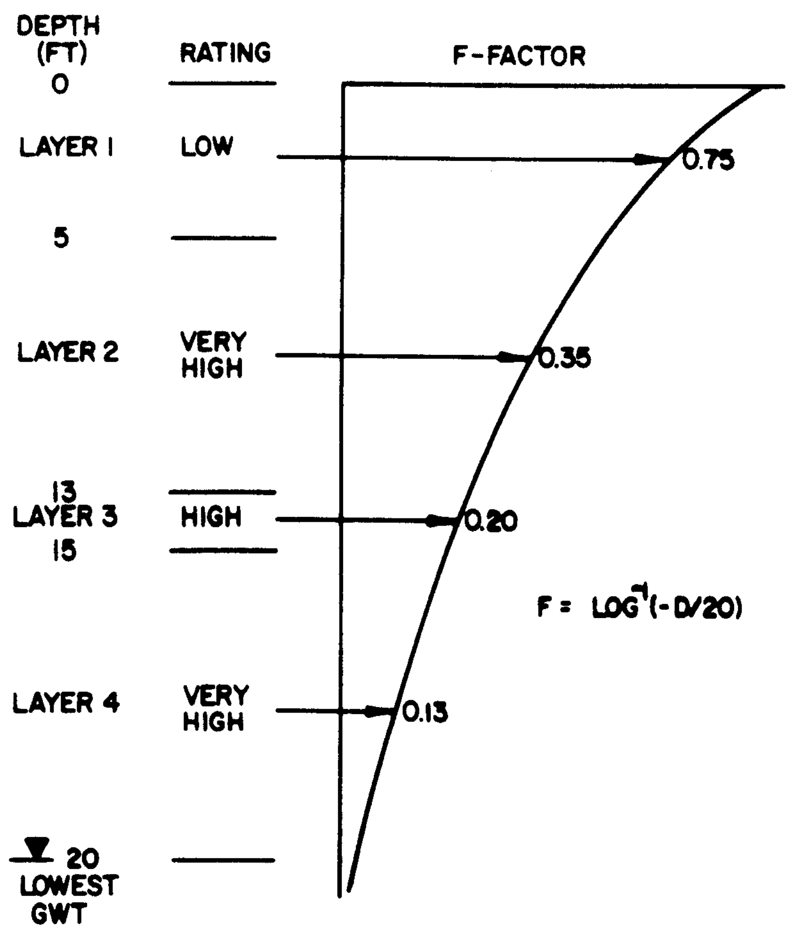


FIGURE 3 (continued)
Estimating Swell Using the South African Method

TABLE 4
Recommended Foundations for Expansive Soils

Predicted Differential Movement inches	Foundation System	Description	
<1/2	Shallow; standard	Continuous wall, individual spread footings.	
	Reinforced and stiffened waffle mat	Residences and lightly loaded structures; on-grade 4-inch reinforced concrete slab with stiffened berms; 0.5 percent reinforcing steel; 10 to 12 inch thick beams; external beams thickened and extra steel stirrups added to tolerate high edge forces. Dimensions adjusted to resist loading.	
		Beam Depth, in.	Beam Spacing, ft.
1/2 - 1	Light	18	15
1 - 2	Medium	24	12
2 - 4	Heavy	30	10
High swelling potential	Thick, reinforced mat	Large, heavy structures; thickness of more than 1 ft.	
	Grade beams on concrete piers	Suspended floors or on-grade first floor isolated from grade beams and walls; grade beams span between piers about 12 inches above ground level. Reinforced cast-in-place concrete piers contribute anchorage against uplift forces due to heave of soils surrounding the shaft; reinforcing should resist tensile forces applied to the shaft by friction in the active zone; tensile force may be assumed equal to the circumferential area times the difference between the average swelling pressure above and below the point under consideration; sleeve of bitum slip layers, roofing felt, PVC, or polyethylene may be applied around the shaft to reduce skin friction and uplift forces on the shaft and inhibit moisture migration down the concrete shaft.	

TABLE 5
Soil Stabilization Methods to Control Heave

Method	Remarks
Lime, cement, and fly ash	Two to five percent lime thoroughly mixed is the most successful chemical agent. In-place mixing by scarifying feasible up to 36-inch depth. Montmorillonites should be conditioned with lime if cement is also added. Fly ash may be added to improve strength.
Compaction control	Compact by kneading (sheepsfoot roller) to 90 to 95% standard - Proctor optimum density at water contents from 2 to 5% greater than optimum.
Moisture control	Horizontal plastic membranes of controversial value due to possible punctures and leaks. Catalytically blown asphalt membranes effective in minimizing penetration of moisture below membrane from surface. Vertical membranes may minimize horizontal moisture flows. Ground surface should slope slightly from structure. Add drains for downspouts and faucets and discharge away from foundation soil. Provide subdrains if perched water table or freeflow of subsurface water are problems. Provide watertight utility connections. Drains should not be installed in desiccated soils as moisture from drains will be drawn into soil.
Removal and replacement with nonexpansive backfill	Useful for replacing surface expansive soils to about 4- to 6-foot depths. Backfill should be impervious. Replacement soil may be in situ soil treated with lime or other chemical agent. Use compaction control; avoid low water contents and high densities.

TABLE 6
Remedial Measures for Existing Foundations on Swelling Soils

Structure Element Treated	Description
Superstructure	Free slabs from foundation by cutting along foundation walls; provide slip joints in interior walls and door frames; reinforce masonry and concrete block walls with horizontal and vertical tie bars or reinforced concrete beams; provide fanlights over doors extended to ceilings.
Spread footings and deep foundations	Decrease footing size; underpin with piers; mudjack; reconstruct void beneath grade beams; eliminate mushroom at top or adding shims; increase footing or pier spacing to concentrate loading and to reduce angular distortion from differential heave between adjacent footings and piers.
Continuous wall foundation	Provide voids beneath portions of wall foundation; post tension; reinforce with horizontal and vertical tie bars or reinforced concrete beams.
Reinforced and stiffened slab-on-ground	Mudjack; underpin with spread footings or piers to jack up the edge of slabs.

Deep foundations (e.g., drilled piers) should extend below the active zone of swelling (typically 20 feet). Drilled piers are belled to provide anchorage to resist uplift forces, and reinforcement is provided to carry uplift tensile force. Uplift forces can be minimized by using the smallest appropriate shaft diameter. In computing magnitude of uplift use an adhesive factor of 1.0 (i.e. $C_a = C$).

Section 7. SPECIAL GEOTECHNICAL ENGINEERING STRUCTURES

1. CONCRETE DIAPHRAGM WALLS.

a. Introduction. A diaphragm wall, or slurry wall, refers to a continuous concrete wall built below ground using a fluid-filled trench. The general construction method includes (see Figure 4): excavation of the trench with introduction of bentonite slurry as the digging proceeds; insertion of steel reinforcement; placement of the tremie concrete which displaces the slurry. Walls are typically 24 to 36 inches wide and constructed in 10 to 20 foot sections. Fluid pressure and soil arching are the primary factors providing trench stability. Construction of walls is generally done by the contractors specializing in this work, and the design is often based on experience and the construction of trial panel sections. Detailed guidance on diaphragm walls is given in Reference 12, Reference 27, Diaphragm Walls and Anchorages, published by the Institute of Civil Engineers and Reference 28, A Review of Diaphragm Walls, published by the Institute of Civil Engineers.

b. Application. Diaphragm walls can be used as retaining walls and/or as load bearing walls and for seepage control beneath or around water retaining structures and for stabilizing potentially unstable slopes.

c. Subsurface Conditions. Diaphragm walls have been constructed in virtually all types of soil. However, some subsurface conditions may induce premium costs making construction impractical. Table 7 (after Reference 12) presents some conditions which may present problems for construction of slurry walls.

d. Construction Procedures.

(1) Method of Construction. Trenching is the usual technique for constructing cast in place diaghragm walls. Excavation should proceed with minimum disturbance of the soil at the cutting face and slowly enough to permit the stabilization of the trench walls with the bentonite slurry (the rate of excavation often determined by construction of trial section). Excavation can be done by augers, clam shell or special trenching machines. Rotary and percussion tools are used for excavating rock or hard formations. Guide walls are built at the surface to align the trench, contain the slurry, and support reinforcing or precast elements if used. Alternate panels are excavated and concreted between stop tubes (see Figure 4). Another joint construction technique is to use a structural section (precast I-beam) to provide a joint capable of transferring shear and providing vertical reinforcing.

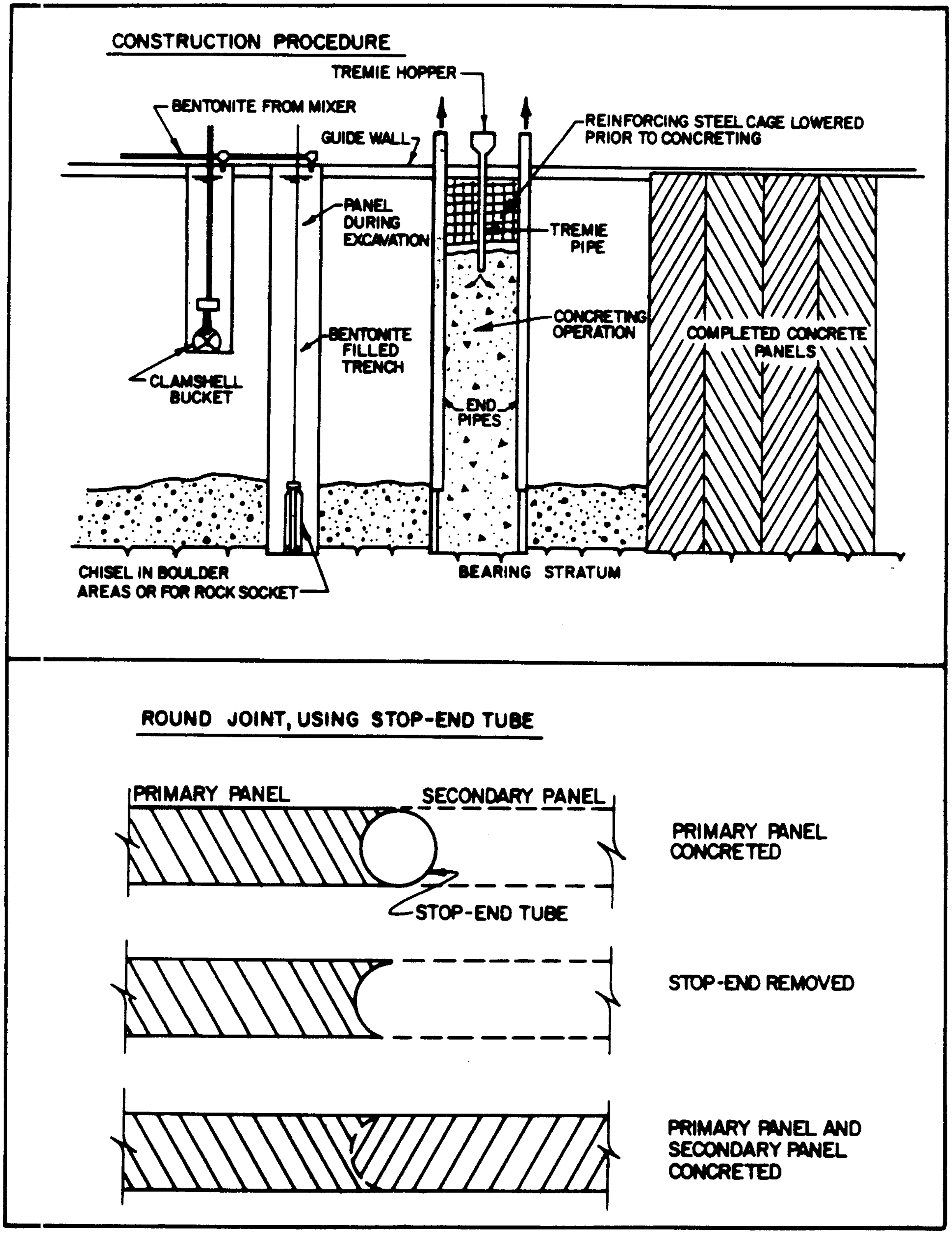


FIGURE 4
Diaphragm Wall: General Construction Method

TABLE 7
Construction of Diaphragm Walls in Difficult Subsoil Conditions

Soil Type	Potential Construction Difficulty
Highly Pervious Soils	Infiltration of groundwater within trench decreasing wall stability (can sometimes be solved by thickening bentonite and adding plugging agents).
Soft Clays	Wall stability may be a problem with clay having a shear strength less than 500 psf. (Test sections are recommended to determine panel length and construction procedures.)
Calcium Laden Soils	Calcium contamination from lime soils, gypsum, anhydrite may lead to flocculation of slurry and an ineffective mudcake on the trench wall.
Peat and Organic Soils	Peat may overbreak causing an irregular wall, in addition to stability problems caused by low shear strength. It may also float free and become embodied within concrete. Adverse pH of undecayed organic material, in some cases, makes construction impractical.
Residual Soils Containing Iron Oxides	Severe pH contamination causing adverse thickening of slurry.
Stiff Fissured Clays	Severe overbreaks and local collapses causing problems with wall stability.
Loose Silts	Local liquefaction, perhaps initiated by construction equipment, causing instability of trench walls.
Soil Containing Boulders	Difficulties in excavation will incur premium costs in addition to possible stability problems or irregularly shaped wall.
Saline Soils	Sea water acts as flocculating agent in slurry (by mixing slurry with fresh water, salt water contamination is typically limited to 5% to 10% and does not adversely affect construction) or use sodium bentonite.
Artesian Conditions	This will cause dilution of bentonite and trench stability problems. (Such condition must be suppressed by pumping or by overcoming with positive head of bentonite.)

The detailed method of construction is usually stated by the contractor at the time of bidding. Of primary significance in any technique is the need to avoid damage to panels already cast. Ridges and abrupt changes in the wall face are usually not acceptable. A typical specification calls for no more than 3/4 inch variation in 10 feet of profile. Exposed wall faces at the end of panels should be vertical (a tolerance of 1:80 can be accomplished with good technique).

(2) Materials.

(a) Bentonite. The bentonite slurry is normally mixed at a 4% to 6% concentration. The slurry forms a mudcake on the sides of the trench which aids stability. It must be dense enough to provide stability yet be fluid enough to allow circulation and concreting. The height of the bentonite within the trench is generally kept at least 4 feet above the groundwater level, to insure a positive fluid pressure on the walls of the excavation. Depending on project requirements the density, viscosity, shear strength, and pH of the bentonite slurry should be specified.

(3) Reinforcement and Concrete. Steel reinforcement of walls may include a rebar cage, wide flange sections, or a combination of both. The design of reinforcement of load bearing walls which require a substantial amount of steel must be carefully done to prevent the trapping of slurry and mud during concreting. Concrete is placed by one or more tremie pipes in each panel. Concrete typically is designed for a 7 to 8 inch slump with a water cement ratio less than 0.6. General practice limits the horizontal flow of concrete to less than 10 feet to prevent segregation.

(4) Other Typical Walls. The slurry excavation technique has been used for constructing walls of precast concrete panels, walls of preset steel soldier piles with interconnecting concrete walls, and walls of bored piles. Further information may be found in the references previously cited.

2. GROUND ANCHORS.

a. Introduction. There are two general categories of anchors:

(1) Grouted anchors when load is transferred from tendon to grout then from grout to soil. Load transfer is by either friction along a straight shaft or by bearing against an underream or both.

(2) Mechanical anchor where load is transferred to soil by an expanding bit or other means.

The basic components (Figure 5, from Reference 29, Tentative Recommendations for Prestressed Rock and Soil Anchors, by the Prestressed Concrete Institute) of a grouted ground anchor are: (a) the prestressing steel, which may be one or more wire cables or bars; the bond length of the steel is the grouted portion of the tendon which transmits force to the surrounding soil or rock; the stressing length of the tendon is the portion which is free to elongate during stressing, (b) the stressing anchorage, which permits the stressing and anchoring of the steel under load, and (c) the grout and vent pipes required for injecting the anchor grout. Secondary grouting of the stressing length is often done for corrosion protection.

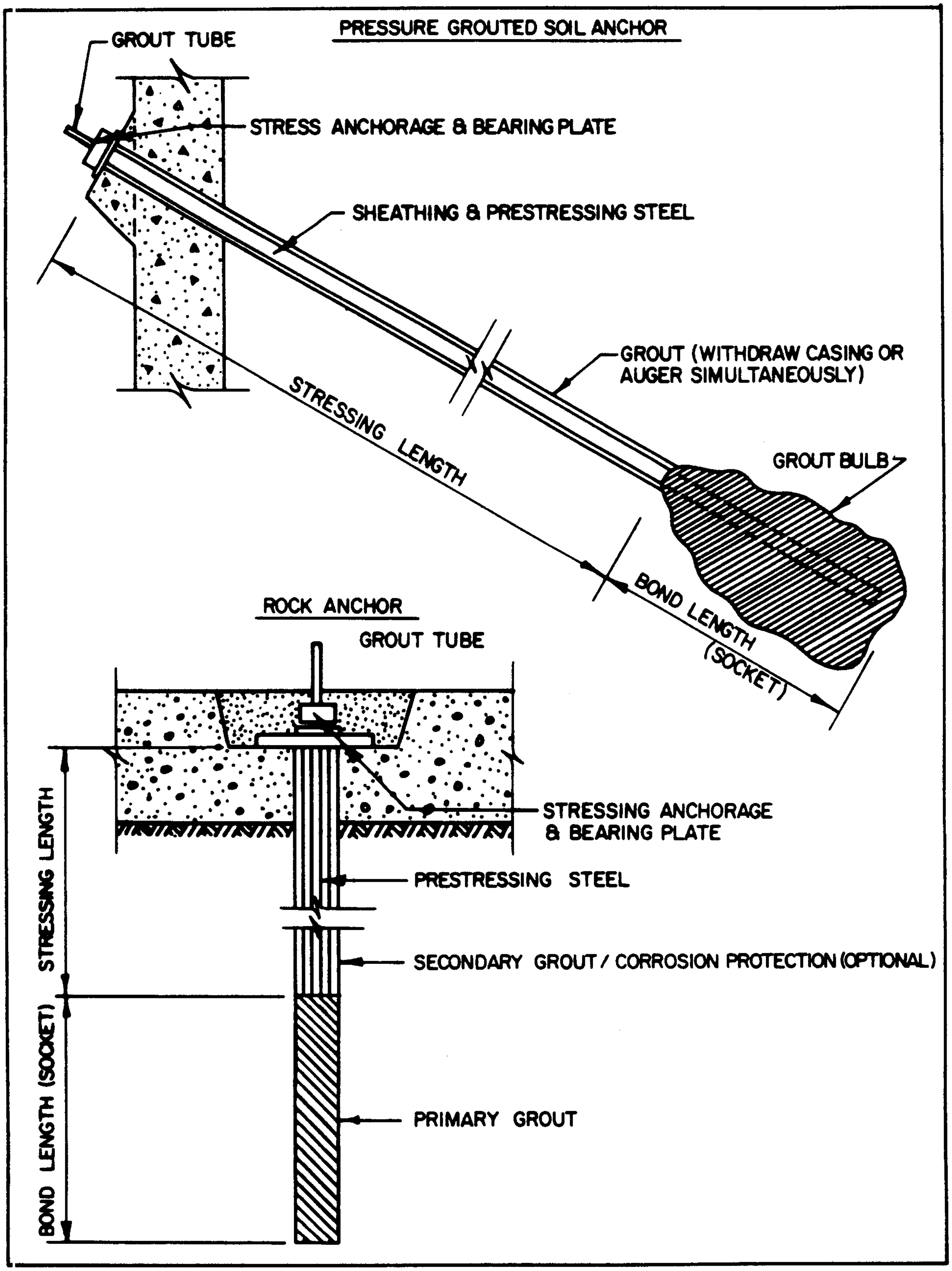


FIGURE 5
Basic Components of Ground Anchors

b. Soil Anchors.

(1) Introduction. Soil anchors can be installed in nearly all type of soil. Types of anchors and applicable soils are presented in Table 8 (Reference 12). Anchor capacity depends on various factors, including soil type and grout penetration. Estimate of anchor capacity should consider past experience, pull out testing of anchors, soils data and consequences of failure. In some cases, field testing of all anchors is necessary. Anchors in coarse sands and gravels have had working loads up to 80 tons (factor of safety, F_s = 1.5) where the fixed anchor has had about 40 feet of overburden on it. Anchors in medium sands, with the fixed anchor below 20 to 30 feet of overburden, have been installed with working loads up to 40 tons (F_s = 2). Anchors with working loads up to 60 tons (F_s = 3) have been installed in stiff clays. For further guidance, see Reference 30, Ground Anchors, by Jackson, et al.

(2) Design.

(a) Anchors in Granular Soils. The anchorage is formed by injection of grout under high pressure so that a grout bulb forms along the bond length of the anchor. Figure 6 (see Reference 31, Construction, Carrying Behavior and Creep Characteristics of Ground Anchors, by Ostermayer) presents a graph of anchor capacity versus bond length for granular soil types of various densities and may be used for pre-test estimate of bond length (free or stressing length of anchor is normally a minimum of 20 to 25 feet).

Because of the large number of variables affecting anchor performance, anchors are normally proof loaded to at least 115% to 125% of design load with selected anchors tested to higher loads and for long-term creep characteristics. Permanent anchors should be tested to 150% of design load. Guidelines for testing may be found in the references.

(b) Anchors in Cohesive Soils. Guidance is given in Figure 6 for pre-test estimating and pull out capacity of anchors in cohesive soils.

c. Rock Anchors.

(1) Introduction. Rock anchors have a wide variety of applications and may be installed in most rock types. Figure 5 shows the basic components of a rock anchor.

(2) Design. Anchor design must consider the following failure modes:

(a) Failure of Steel Tendon. Design stress within the steel is usually limited to 50 to 60% of the ultimate stress (50% for permanent installations).

(b) Failure of Grout-steel Bond. The bond capacity depends on the number and length of tendons, or steel bars (plain or deformed) and other factors. See Reference 32, Rock Anchors, State of the Art, by Littlejohn and Bruce for guidance.

TABLE 8
Types of Soil Anchors

	Diameter (inches)					
Method	Shaft Type	Bell Type	Gravity Concrete	Grout Pressure (psi) (1)	Suitable Soils for Anchorage	Load Transfer Mechanism
1. LOW PRESSURE Straight Shaft Friction (Solid stem auger)	12 - 24"	NA	A	NA	Very stiff to hard clays Dense cohesive sands	Friction
Straight Shaft Friction (Hollow stem auger)	6 - 18"	NA	NA	30 - 150	Very stiff to hard clay Dense cohesive sands Loose to dense sands	Friction
Underreamed Single Bell at Bottom	12 - 18"	30 - 42"	A	NA	Very stiff to hard cohesive soils Dense cohesive sands Soft rock	Friction and bearing
Underreamed Multi-bell	4 - 8"	8 - 24"	A	NA	Very stiff to hard cohesive soils Dense cohesive sands Soft Rock	Friction and bearing

TABLE 8 (continued)
Types of Soil Anchors

Method	Diameter (inches)		Gravity Concrete	Grout Pressure (psi) (1)	Suitable Soils for Anchorage	Load Transfer Mechanism
	Shaft Type	Bell Type				
2. HIGH PRESSURE - SMALL DIAMETER						
Non-regroutable (2)	3 - 8"	NA	NA	150	Hard clays Sands Sand-gravel formations Glacial till or hardpan	Friction or friction and bearing in permeable soils
Regroutable (3)	3 - 8"	NA	NA	200 - 500	Same soils as for non-regroutable anchors plus: a) Stiff to very stiff clay b) Varied and difficult soils	Friction and bearing

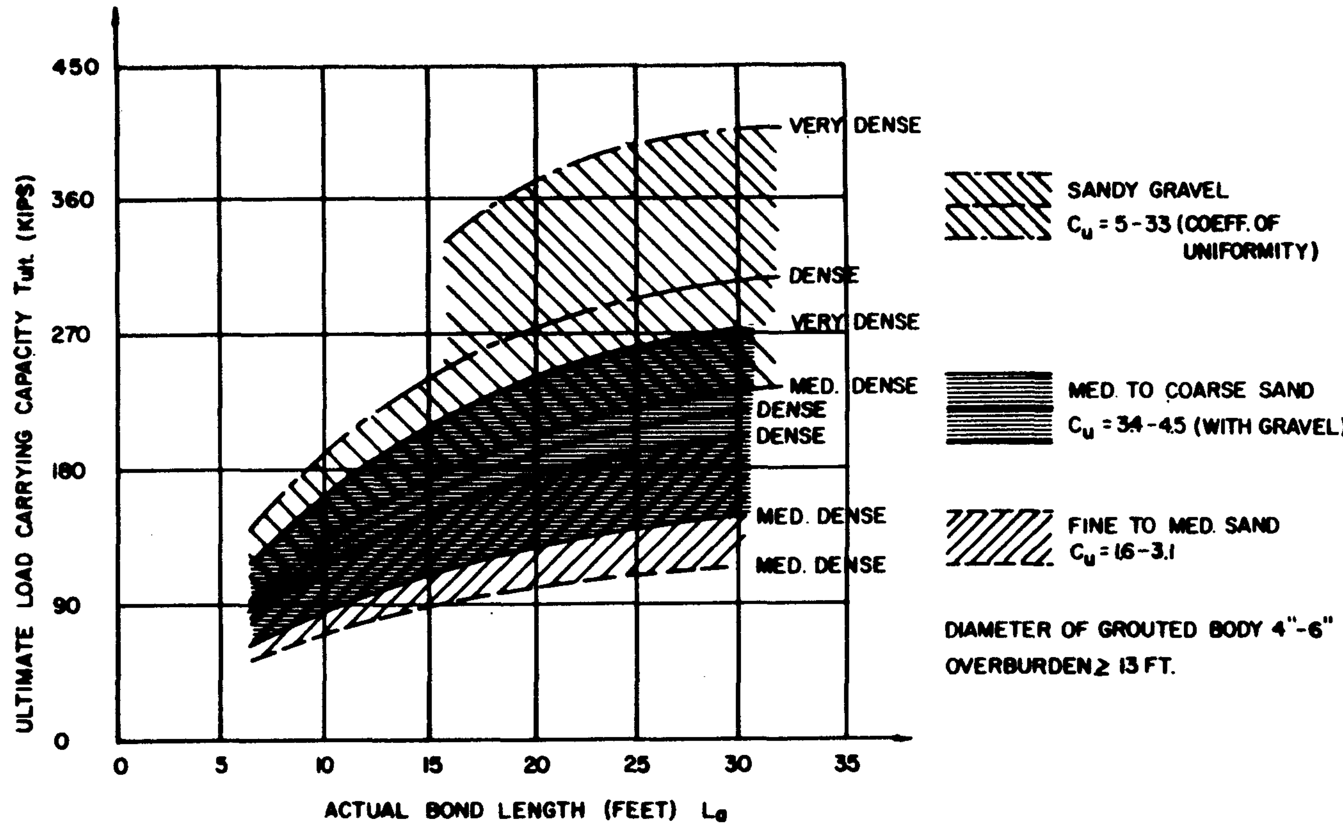
(1) Grout pressures are typical

(2) Friction from compacted zone having locked in stress. Mass penetration of grout in highly pervious sand/gravel forms "bulb" anchor."

(3) Local penetration of grout will form bulbs which act in bearing or increase effective diameter.

A - Applicable

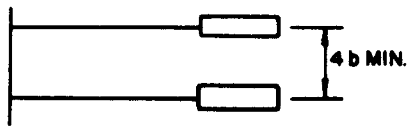
NA - Not applicable



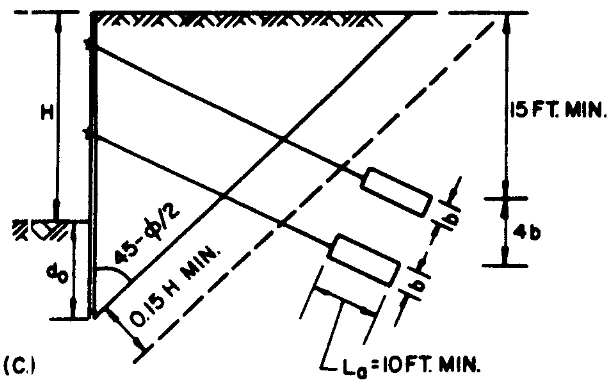
(A) EMPIRICAL ULTIMATE CAPACITY OF ANCHORS IN GRANULAR SOILS SHOWING INFLUENCE OF SOIL TYPE (COEFFICIENT OF UNIFORMITY), DENSITY AND BOND-TO-GROUND LENGTH.

FIGURE 6
Estimate of Anchor Capacity

(B.)



PLAN



(C.)

d_o = DEPTH OF PENETRATION REQUIRED FOR STABILITY (SEE DM 7.2, CHAPTER 3)

ESTIMATE b FROM

$$b = \sqrt{\frac{Vg}{0.78\ La}}$$

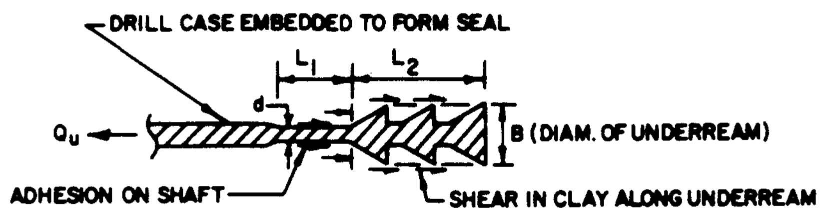
Vg = VOLUME OF GROUT IN LENGTH La
La = ACTUAL BOND LENGTH.

ANCHOR CAPACITY - GRANULAR SOIL

FOR A GIVEN La ESTIMATE ULTIMATE CAPACITY FROM FIGURE "A". THE ESTIMATED VALUE SHOULD NOT EXCEED

$$T_{ult} = 1.5\ (La \tan \phi)\ \pi b P_o$$

WHERE P_o = VERTICAL EFFECTIVE STRESS AT MID POINT OF La.



(D.)

ANCHOR CAPACITY - COHESIVE SOIL

$$Q_u = 0.75\pi B L_2 C + \frac{9\pi}{4}(B^2-d^2)\ C + \pi d L_1 C_a$$

WHERE C_a = ADHESION RELATION TO C (SEE FIGURE 2, DM 7.2, CHAPTER 5)

FIGURE 6 (continued)
Estimate of Anchor Capacity

(c) Failure of Grout-rock Bond. The bonding capacity between the rock and the grout may be determined from the following formula:

$$P_u = \pi \cdot d_s L_o \delta skin$$

where: P_u = load capacity of anchor

d_s = diameter of drilled shaft

L_o = length of grout-anchor bond

δ skin = grout-rock bond strength

Typical grout-rock stresses for various rock types are presented in Table 9 (after Reference 12).

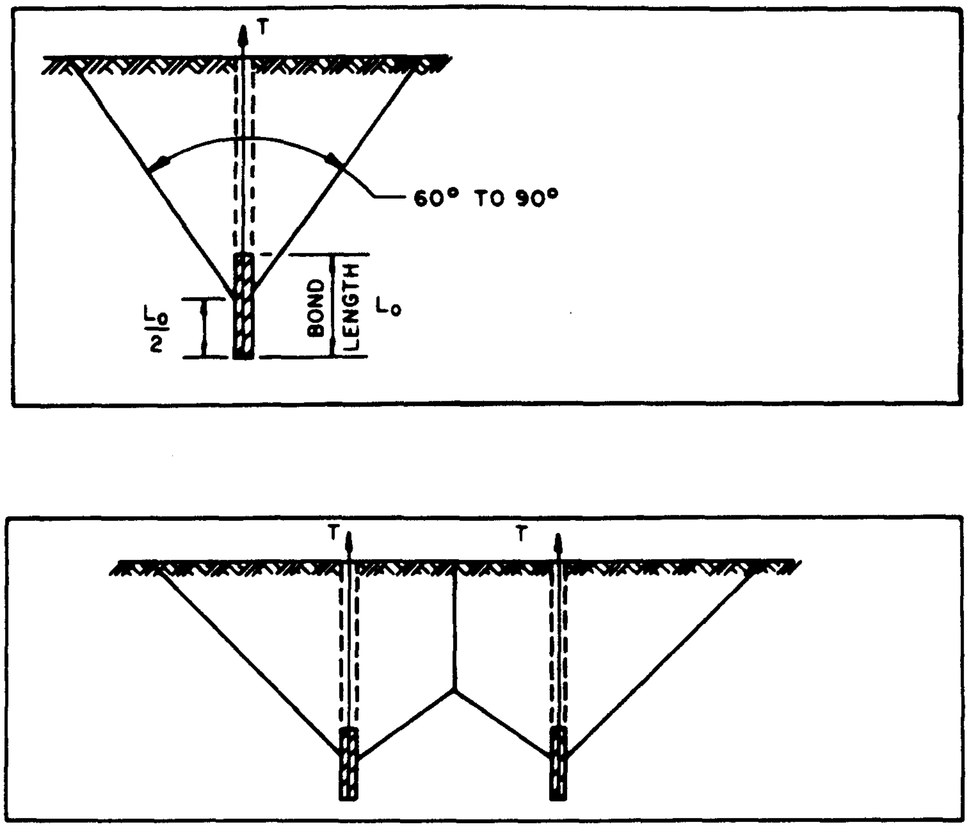
(d) Failure of Rock Mass. The criterion for failure in rock mass is based on the weight of rock contained within a cone emanating from the bonded zone. Figure 7 shows design criteria. Actual failure of anchor in this mode would be controlled by discontinuity patterns and weathering of the rock.

(3) Factor of Safety and Testing. Anchors in soil should be designed using a mininum factor of safety of 2.0; a higher factor of safety is used for permanent or critical structures. All production anchors should be proof loaded to 115% to 150% of the design load. Additional testing to higher capacities and to determine creep characteristics may be justified for permanent installations or where the design conditions warrant. Guidelines for testing are found in the references previously cited.

TABLE 9
Typical Values of Bond Stress for Selected Rock Types

Rock Type (Sound, Non-Decayed)	Ultimate Bond Stresses Between Rock and Anchor Plus (δ_{skin}), psi
Granite & Basalt	250 - 450
Limestone (competent)	300 - 400
Dolomitic Limestone	200 - 300
Soft Limestone	150 - 220
Slates and Hard Shales	120 - 200
Soft Shales	30 - 120
Sandstone	120 - 150
Chalk (variable properties)	30 - 150
Marl (stiff, friable, fissured)	25 - 36

Note: It is not generally recommended that design bond stresses exceed 200 psi even in the most competent rocks.



COMPUTE ALLOWABLE PULL-OUT CAPACITY

BASED ON $T_{all} = \dfrac{W_c + F_t}{3}$

W_c = WEIGHT OF CONE

F_t = SHEARING RESISTANCE ALONG THE SURFACE OF THE CONE

Note: This analysis is not applicable if blocky joints are predominant. In such case use the weight of joint block and available side shear which may be negligible depending upon the joint filling.

For shallow anchors in ordinary fractured rock, compute allowable pullout based on ultimate rock shear strength of 900 psf, cone of 60°, and ignore weight of rock in cone (see Figure 16, DM-7.2, Chapter 4 for example).

FIGURE 7
Pullout Capacity - Shallow Anchors in Rock

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APPENDIX A
Listing of Computer Programs

Topic	Program	Description	Availability
Dynamic and Seismic (Chapter 1)	PILAY	Computes dynamic stiffness and damping constants for piles embedded in a multi-layered viscoelastic medium. Pile deflections and internal forces are also provided.	SACDA - Faculty of Engineering Sciences, University of Western Ontario, London, Ontario, Canada N6A 5B9
	RIGDF	Computes dynamic responses of rigid footing to harmonic loads in all six degrees of freedom. Effect of embedment can be included.	"
	DYNA	A general program for dynamic response of a rigid footing supported by soil, piles or other means.	"
	FLUSH	A finite element program for seismic two-dimensional soil-structure interaction by the complex response method.	Professor J. Lysmer 435 Davis Hall University of California Berkeley, CA 94720 (available on NAVFAC CADLOG)

GLOSSARY

Electro-osmosis - A method of dewatering, typically used for fine-grained soils, in which an electric field is established in the soil mass to cause the movement by electro-osmotic forces of pore water to wellpoint cathodes.

Liquefaction - The sudden, large decrease of shear strength of a cohesionless soil caused by collapse of the soil structure, produced by shock or small shear strains, associated with sudden but temporary increase of pore water pressures.

Loess - A wind deposited silt with high porosity and low unit weight which is extremely susceptible to collapse of its granular structure upon wetting.

Vibrodensification - The densification or compaction of cohesionless soils by imparting wave energy to the soil mass so as to rearrange soil particles resulting in less voids in the overall mass.

SYMBOLS

Symbol	Designation
A	Cross-sectional area; also amplitude.
B,b	Width in general, or narrow dimension of a foundation unit.
C_a	Unit adhesion between soil and pile surface or surface of some other foundation material.
C_u	Coefficient of uniformity of grain size curve.
C_α	Coefficient of secondary compression.
c	Cohesion intercept for Mohr's envelope of shear strength based on total stresses.
c'	Cohesion intercept for Mohr's envelope of shear strength based on effective stresses.
D,d	Depth, diameter, or distance; also damping coefficient.
D_R	Relative density.
D_{15}, D_{60}, D_{85}	Grain size division of a soil sample, percent of dry weight smaller than this grain size is indicated by subscript.
E	Modulus of elasticity of structural material.
E_s	Modulus of elasticity or "modulus of deformation" of soil.
e	Void ratio.
F_s	Safety factor in stability or shear strength analysis.
f	Frequency.
G	Shear modulus.
H,h	In general, height or thickness.
I	Moment of inertia.
k	Coefficient of permeability in general.
ksf	Kips per sq ft pressure intensity.
ksi	Kips per sq in pressure intensity.
L,l	Length in general or longest dimension of foundation unit.
pcf	Density in pounds per cubic foot.
P_o	Existing effective overburden pressure acting at a specific height in the soil profile or on a soil sample.
p	Intensity of applied load.
q	Intensity of vertical load applied to foundation unit.
q_u	Unconfined compressive strength of soil sample.
R,r	Radius of pile, caisson, well, or other right circular cylinder.
S	Percent saturation of soil mass.
s	Shear strength of soil for a specific stress or condition in situ, used instead of strength parameters c and ϕ.
T	Thickness of soil stratum, or relative stiffness factor of soil and pile in analysis of laterally loaded piles.
tsf	Tons per sq ft pressure intensity.
t, t_1, t_2, t_n	Time intervals from start of loading to the points 1, 2, or n.
w	Moisture content of soil.
γ_D	Dry unit weight of soil.
γ_{SUB}, γ_B	Submerged (buoyant) unit weight of soil mass.
γ_T	Wet unit weight of oil above the groundwater table.
γ_W	Unit weight of water, varying from 62.4 pcf for fresh water to 64 pcf for sea water.
ϵ	Unit strain in general.

SYMBOLS

Symbol	Designation
$\delta, \delta_v, \delta_c$	Magnitude of settlement for various conditions.
ρ	Foundation mass density.
Φ	Angle of internal friction or "angle of shearing resistance," obtained from Mohr's failure envelope for shear strength.
σ_1	Total major principal stress.
σ_3	Total minor principal stress.
$\bar{\sigma}_1$	Effective major principal stress.
$\bar{\sigma}_3$	Effective minor principal stress.
$\sigma_x, \sigma_y, \sigma_z$	Normal stresses in coordinate directions.
ν	Poisson's Ratio.
τ	Intensity of shear stress.
τ_{max}	Intensity of maximum shear stress.

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