Introduction to Excavation for Structures

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CONTENTS

The Figures, Tables and Symbols in this document are in some cases a little difficult to read, but they are the best available. **DO NOT PURCHASE THIS COURSE IF THE FIGURES, TABLES AND SYMBOLS ARE NOT ACCEPTABLE TO YOU.**

1. INTRODUCTION

This course is an introduction to the methods of evaluating the stability of shallow and deep excavations. There are two basic types of excavations:

- “open excavations” where stability is achieved by providing stable side slopes

- “braced excavations” where vertical or sloped sides are maintained with protective structural systems that can be restrained laterally by internal or external structural elements.

1.1 METHODOLOGY. In selecting and designing the excavation system, the primary controlling factors will include:

- soil type and soil strength parameters
- groundwater conditions
- slope protection
- side and bottom stability
- vertical and lateral movements of adjacent areas and effects on existing structures

2. OPEN CUTS

2.1 SLOPED CUTS. Depth and slope of an excavation, and groundwater conditions control the overall stability and movements of open excavations. In granular soils, instability usually does not extend significantly below the excavation, provided seepage forces are controlled. In rock, depths and slopes of excavation, particular joint patterns, in situ stresses, and groundwater conditions control stability. In cohesive soils, instability typically involves side slopes but may also include materials well below the base of the excavation. Instability below the base of excavation, often referred to as bottom heave, is affected by soil type and strength, depth of cut, side slope and/or berm geometry, groundwater conditions, and construction procedures. Methods may be used to evaluate the stability of open excavations in soils where behavior of such soils can be reasonably determined by field investigation, laboratory testing, and analysis. In certain geologic formations (stiff clays, shales, sensitive clays, clay tills, etc.) stability is controlled by construction procedures, side effects during and after excavation and inherent geologic planes of weaknesses. Table 1 presents a summary of the primary factors controlling excavation slopes in some problem soils. Table 2 summarizes measures that can be used for excavation protection for both conventional and problem soils.

SOIL TYPE	PRIMARY CONSIDERATIONS FOR SLOPE DESIGN
Stiff-fissured Clays and Shales	Field shear resistance may be less than suggested by laboratory tests. Slope failures may occur progressively and shear strengths reduced to residual values compatible with relatively large deformations. Some case histories suggest that the long-term performance is controlled by the residual friction angle which for some shales may be as low as 12 degrees. The most reliable design procedure would involve the use of local experience and recorded observations.
Loess and Other Collapsible Soils	Strong potential for collapse and erosion of relatively dry material upon wetting. Slopes in loess are frequently more stable when cut vertical to prevent infiltration. Benches at intervals can be used to reduce effective slope angles. Evaluate potential for collapse as described in UFC 3-220-10N.
Residual Soils	Significant local variations in properties can be expected depending on the weathering profile from parent rock. Guidance based on recorded observation provides prudent basis for design.
Sensitive Clays	Considerable loss of strength upon remolding generated by natural or man-made disturbance. Use analyses based on unconsolidated undrained tests or field vane tests.
Talus	Talus is characterized by loose aggregation of rock that accumulates at the foot of rock cliffs. Stable slopes are commonly between 1-1/4 to 1-3/4 horizontal to 1 vertical. Instability is associated with abundance of water, mostly when snow is melting.
Loose Sands	May settle under blasting vibration, or liquefy, settle and lose strength if saturated. Also prone to erosion and piping.

TABLE 1

Factors Controlling Stability of Sloped Cut in Some Problem Soils

Construction Activity	Objectives	Comments
Dewatering	To prevent boiling, softening, or heave in excavation bottom, reduce lateral pressures on sheeting, reduce seepage pressures on face of open cut, and eliminate piping of fines through sheeting.	Investigate soil compressibility and effect of dewatering on settlement of nearby structures; consider recharging or slurry wall cutoff. Examine for presence of lower aquifer and need to dewater. Install piezometer if needed. Consider effects of dewatering in cavity-laden limestone. Dewater in advance of excavation.
Excavation and Grading	Pipe trenching, basement excavation, site grading.	Analyze safe slopes or bracing requirement, effects of stress reduction on over-consolidated, soft or swelling soils and shales. Consider horizontal and vertical movements in adjacent areas due to excavation and effect on nearby structures. Keep equipment and stockpiles a safe distance from top of excavation.
Excavation Wall Construction	To support vertical excavation walls, to stabilize trenching in limited space.	Reduce earth movements and bracing stresses, where necessary, by installing lagging on front flange of soldier pile. Consider effect of vibrations due to driving sheet piles or soldier piles. Consider dewatering requirements as well as wall stability in calculating sheeting depth. Movement monitoring may be warranted.
Blasting	To remove or to facilitate the removal of rock in the excavation.	Consider effect of vibrations on settlement or damage to adjacent areas. Design and monitor or require the contractor to design and monitor blasting in critical areas; require a pre construction survey of nearby structures.
Anchor or Strut Installation, Wedging of Struts, Pre-stressing Ties	To obtain support system stiffness and interaction.	Major excavations require careful installation and monitoring, e.g., case anchor holes in collapsible soils; measure stress in ties and struts; wedging, etc.

TABLE 2
Factors Controlling Excavation Stability

2.2 VERTICAL CUTS. Many cuts in clays will stand with vertical slopes for a period of time before failure occurs. However, changes in the shear strength of the clay with time and stress release resulting from the excavation can lead to progressive deterioration in stability. This process can be rapid in stiff, highly fissured clays, but relatively slow in softer clays. For cuts in hard unweathered rock, stability is mostly controlled by strength along bedding planes, groundwater conditions, and other factors. Cuts in rock can stand vertical without bolting or anchoring depending on rock quality and joint pattern.

3. TRENCHING

3.1 SITE EXPLORATION. Individual trenching projects frequently extend over long distances. An exploration program should be performed to define the soil and groundwater conditions over the full extent of the project, so that the design of the shoring system can be adjusted to satisfy the varying site conditions.

3.2 TRENCH STABILITY. Principal factors influencing trench stability are the lateral earth pressures on the wall support system, bottom heave, and the pressure and erosive effects of infiltrating groundwater. External factors that influence trench stability include:

- **SURFACE SURCHARGE.** The application of any additional load between the edge of the excavation and the intersection of the ground surface with the possible failure plane must be considered in the stability analyses for the excavation.
- **EXTERNALLY IMPOSED EFFECTS.** The effects of vibrating machinery, blasting or other dynamic loads in the vicinity of the excavation must be considered. The effects of vibrations are cumulative over periods of time and can be particularly dangerous in brittle materials such as clayey sand or gravel.
- **GROUND WATER SEEPAGE.** Improperly dewatered trenches in granular soils can result in quick conditions and a complete loss of soil strength or bottom heave.
- **SURFACE WATER FLOW.** This can result in increased loads on the wall support system and reduction of the shear strength of the soil. Site drainage should be designed to divert water away from trenches.

3.3 SUPPORT SYSTEMS. Excavation support systems commonly used are as follows:

- **TRENCH SHIELD.** A rigid prefabricated steel unit used in lieu of shoring, which extends from the bottom of the excavation to within a few feet of the top of the cut. Pipes are laid within the shield which is pulled ahead as trenching proceeds. Typically, this system is useful in loose granular or soft cohesive soils where excavation depth does not exceed 3.5 m (12 ft). Special shields have been used to depths of 9 m (30 ft).

- **TRENCH TIMBER SHORING.** Table 3 illustrates the Occupational Safety and Health Act's minimum requirements for trench shoring. Braces and shoring of trench are carried along with the excavation. Braces and diagonal shores of timber should not be subjected to compressive stresses in excess of:

$$S = 1300 - 20\ L/D$$

where:

L = unsupported length (mm or inches)

D = least side of the timber (mm or inches)

S = allowable compressive stress in kilograms per square cm (pounds per square inch) of cross section

Maximum Ratio L/D = 50

(Note: L/D units need to be consistent)

- **SKELETON SHORING.** Used in soils where cave-ins are expected. Applicable to most soils to depth up to 9.1 m (20 ft). Structural components should be designed to safely withstand earth pressures.
- **CLOSE (TIGHT) SHEETING.** Used in granular or other running soils. Compared to skeleton shoring, it is applicable to greater depths.
- **BOX SHORING.** Applicable to trenching in any soil. Depth limited by structural strength and size of timber. Usually limited to 18.2 m (40 ft).
- **TELESCOPIC SHORING.** Used for excessively deep trenches.
- **STEEL SHEETING AND BRACING.** Steel sheeting and bracing can be used in lieu of timber shoring. Structural members should safely withstand water and lateral earth pressures. Steel sheeting with timber wales and struts has also been used.

Size and Spacing of Members												
Depth of Trench	Kind or Condition of Earth	Uprights		Stringers		Cross Braces[1]					Maximum Spacing	
						Width of Trench (feet)						
		Minimum Dimension	Maximum Spacing	Minimum Dimension	Maximum Spacing	Up to 3	4 to 6	7 to 9	10 to 12	13 to 15	Vertical	Horizontal
Feet		Inches	Feet	Inches	Feet	Inches	Inches	Inches.	Inches	Inches	Feet	Feet
5 to 10	Hard, compact	3 x 4 or 2 x 6	6	–	–	2 x 6	4 x 4	4 x 6	6 x 5	6 x 8	4	6
	Likely to crack	3 x 4 or 2 x 6	3	4 x 6	4	2 x 6	4 x 4	4 x 6	6 x 6	6 x 8	4	6
	Soft, sandy, or filled	3 x 4 or 2 x 6	Close Sheeting	4 x 6	4	4 x 4	4 x 6	6 x 6	6 x 8	8 x 8	4	6
	Hydrostatic Pressure	3 x 4 or 2 x 6	Close Sheeting	6 x 8	4	4 x 4	4 x 6	6 x 6	6 x 8	8 x 8	4	6
11 to 15	Hard	3 x 4 or 2 x 6	4	4 x 6	4	4 x 4	4 x 6	6 x 6	6 x 8	8 x 8	4	6
	Likely to crack	3 x 4 or 2 x 6	2	4 x 6	4	4 x 4	4 x 6	6 x 6	6 x 8	8 x 8		6
	Soft, sandy or Filled	3 x 4 or 2 x 6	Close Sheeting	4 x 6	4	4 x 6	6 x 6	6 x 8	8 x 8	8 x 10	4	6
	Hydrostatic Pressure	3 x 6	Close Sheeting	8 x 10	4	4 x 6	6 x 6	6 x 8	8 x 8	8 x 10	4	6
16 to 20	All kinds or Conditions	3 x 6	Close Sheeting	4 x 12	4	4 x 12	6 x 8	8 x 8	8 x 10	10 x 10	4	6
Over 20	All kinds or Conditions	3 x 6	Close Sheeting	6 x 8	4	4 x 12	8 x 8	8 x 10	10 x 10	10 x 12	4	

[1] Trench jacks may be used in lieu of, or in combination with, cross braces. Where desirable, steel sheet piling and bracing of equal strength may be substituted for wood.

Table 3

OSHA Requirements (minimum) for Trench Shoring

4. ROCK EXCAVATION

4.1 PRELIMINARY CONSIDERATIONS. The primary objective is to conduct work in such a manner that a stable excavation will be maintained and that rock outside the excavation prism will not be adversely disturbed. Rock excavation planning must be based on detailed geological data at the site. To the extent possible, structures to be constructed in rock should be oriented favorably with the geological setting. For example, tunnels should be aligned with axis perpendicular to the strike of faults or major fractures. Downslope dip of discontinuities into an open cut should be avoided. In general, factors that must be considered in planning, designing and constructing a rock excavation are as follows:

- Presence of strike, dip of faults, folds, fractures, and other discontinuities
- In situ stresses
- Groundwater conditions
- Nature of material filling joints
- Depth and slope of cut
- Stresses and direction of potential sliding; surfaces
- Dynamic loading, if any
- Design life of cut as compared to weathering or deterioration rate of rock face
- Rippability and/or the need for blasting
- Effect of excavation and/or blasting on adjacent structures

The influence of most of these factors on excavations in rock is similar to that of excavations in soil. General guidance to determine the need for underpinning excavation in rock is given in Figure 1. Zone A likely requires underpinning, Zone B is possible, and Zone C is unlikely.

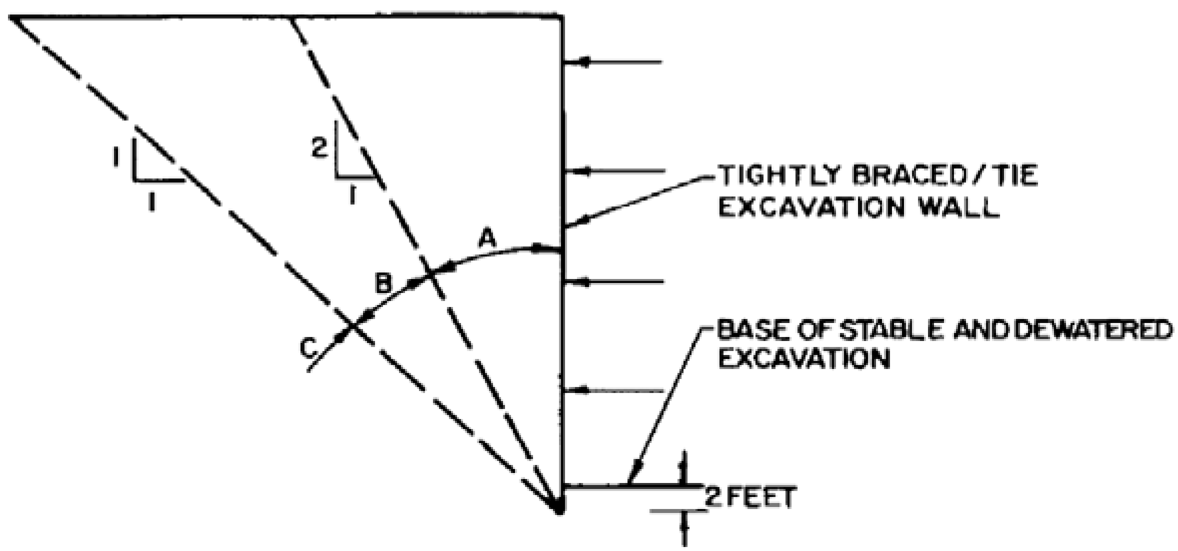


Figure 1
General Guidance for Underpinning

4.2 RIPPABILITY. Excavation ease or rippability can be assessed approximately from field observation in similar materials or by using seismic velocity, fracture spacing, or point load strength index. Figure 2 shows an example of charts for heavy-duty ripper performance (ripper mounted on tracked bulldozer) as related to seismic wave velocity. Charts similar to Figure 2 are available from various equipment manufacturers. Figure 2 is for guidance and restricted in applicability to large tractors heavier than 50 tons with engine horsepower greater than 350 Hp. Ripper performance is also related to configuration of ripper teeth, equipment condition and size, and fracture orientation. Another technique of relating physical properties of rock to excavation ease is shown in Figure 3, where fracture frequency (or spacing) is plotted against the point load strength index corrected to a reference diameter of 50 mm.

A third and useful technique is exploration trenching in which the depth of unrippable rock can be established by digging test trenches in rock using rippers (or other excavation equipment) anticipated to be used for the project. The size and shape of the area to be excavated is a significant factor in determining the need for blasting, or the equipment needed to remove the rock.

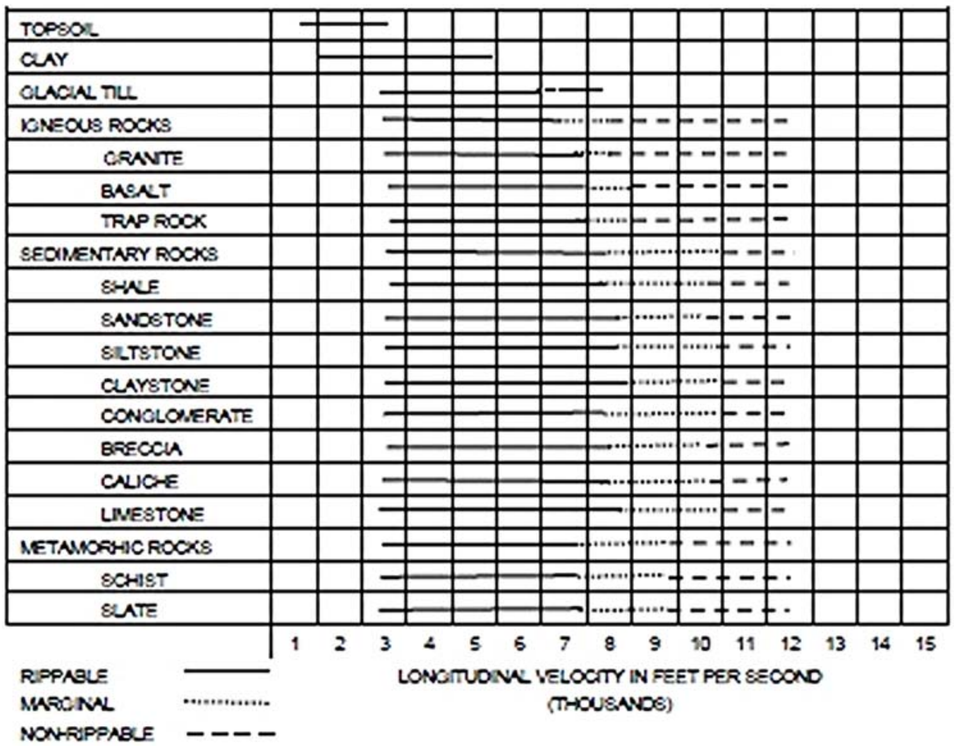


Figure 2

Rippability of Subsurface Materials Related to Longitudinal Seismic Velocity for a Heavy Duty Ripper (Tractor-Mounted)

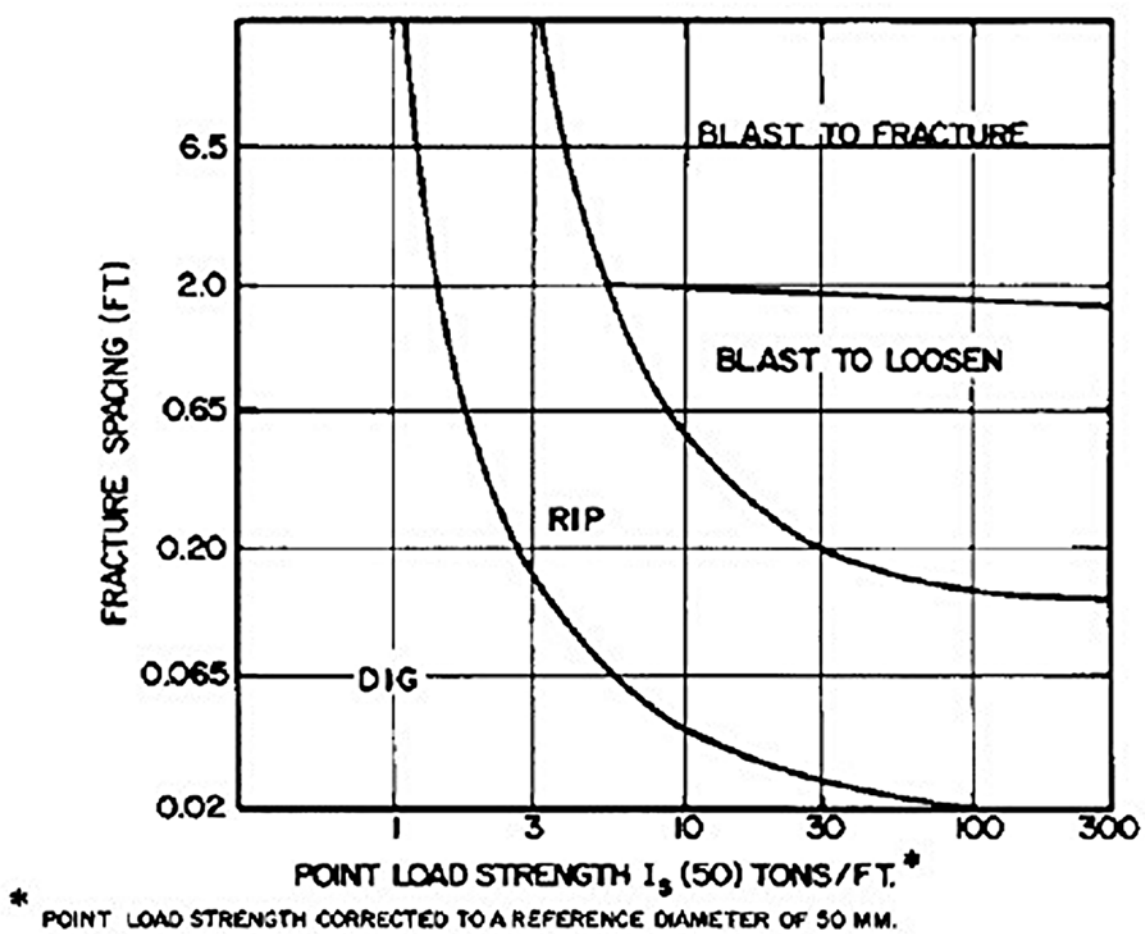


Figure 3

Suggested Guide for Ease of Excavation

4.3 BLASTING. Of major concern is the influence of the blasting on adjacent structures. The maximum particle velocity (the longitudinal velocity of a particle in the direction of the wave that is generated by the blast) is accepted as a criterion for evaluating the potential for structural damage induced by blasting vibration. The critical level of the particle velocity depends on the frequency characteristics of the structure, frequency of ground and rock motion, nature of the overburden, and capability of the structure to withstand dynamic stress. Figure 4 can be used for estimating the maximum particle velocity, which can then be used in Figure 5 to estimate potential damage to residential structures. Guidance for human response to blasting vibrations is given in Figure 6. Once it has been determined that blasting is required, a pre-blasting survey should be performed. At a minimum, this should include:

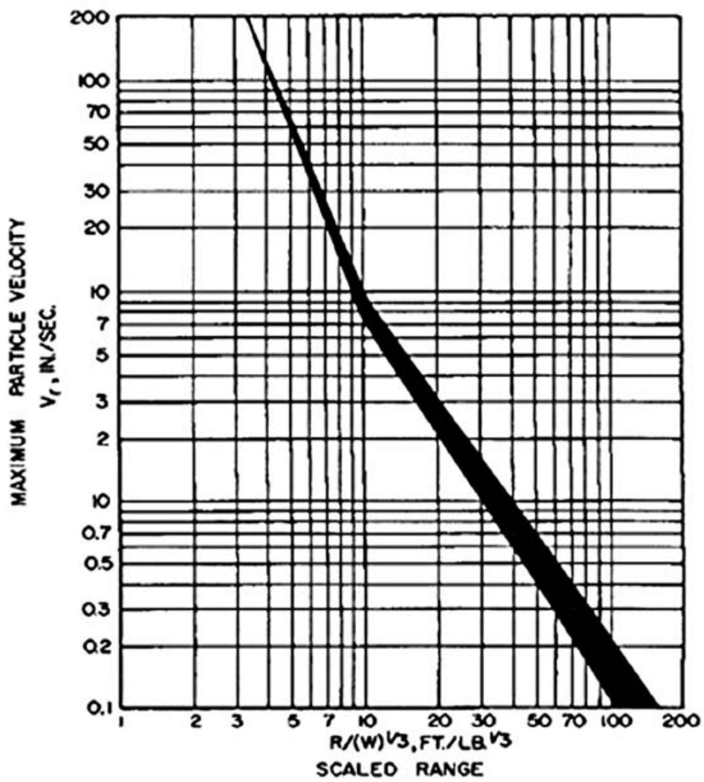
- Examination of the site
- Detailed examination and, perhaps, photographic records of adjacent structures
- Consideration of vibration monitoring as well as monitoring stations and schedules established
- Establishment of horizontal and vertical survey control points

During construction, detailed records of the following should be kept:

- Charge weight
- Location of blast point(s) and distance(s) from existing structures
- Delays
- Response as indicated by vibration monitoring (for safety, small charges should be used initially to establish a site-specific relationship between charge weight, distance, and response).

5. EXACAVATION STABILIZATION, MONITORING, AND SAFETY

5.1 STABILIZATION. During the planning and design stage, if analyses indicate potential slope instability, means for slope stabilization or retention should be considered. Occasionally, the complexity of a situation may dictate using very specialized stabilization methods. These may include grouting and injection, ground freezing, deep drainage and stabilization such as vacuum wells or electro-osmosis, and diaphragm walls.



Example: Weight of Explosive Charge: 8 Lbs. = W; Distance from Blast Point: 100 feet = R
$R/(W)^{1/3} = 50$; Peak $V_r = 0.5$ in/sec (from chart)

Figure 4

Cube Root Scaling Versus Maximum Particle Velocity

5.2 MONITORING. During excavation, potential bottom heave, lateral wall or slope movement, and settlement of areas behind the wall or slope should be inspected carefully and monitored if critical. Monitoring can be accomplished by conventional survey techniques, or by more sophisticated means such as heave points, settlement plates, extensometers or inclinometers, and a variety of other devices.

5.3 SAFETY. Detailed safety requirements vary from project to project. As a guide, safety requirements are specified by OSHA; see Public Law 91-596. A summary of the 1980 requirements follows:

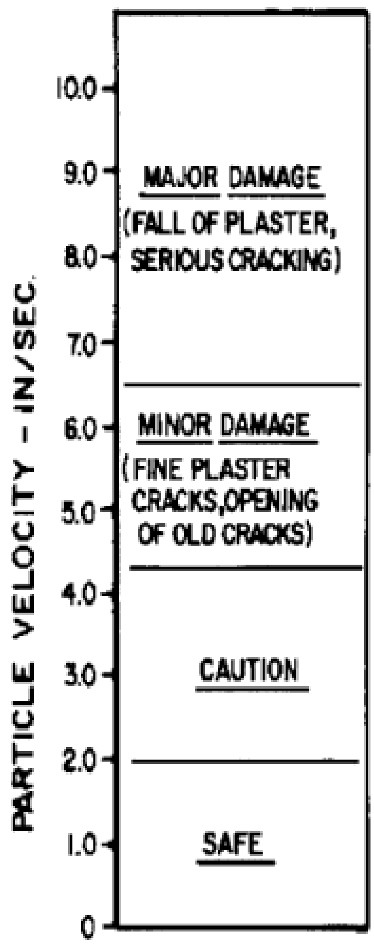


Figure 5

Guideline for Assessing Potential for Damage Induced by Blasting Vibration to Residential Structure Founded on Dense Soil or Rock

- Banks more than 1.2 m (4 ft) high shall be shored or sloped to the angle of repose where a danger of slides or cave-ins exists as a result of excavation.
- Sides of trenches in unstable or soft material, 1.2 m (4 ft) or more in depth, shall be shored, sheeted, braced, sloped, or otherwise supported by means of sufficient strength to protect the employee working within them.
- Sides of trenches in hard or compact soil, including embankments, shall be shored or otherwise supported when the trench is more than 1.2 m (4 ft) in depth and 2.4 m (8 ft) or more in length. In lieu of shoring, the sides of the trench above the 1.2 m (4

ft) level may be sloped to preclude collapse, but shall not be steeper than a 305 mm (1 ft) rise to each 152 mm (6 in) horizontal. When the outside diameter of a pipe is greater than 1.8 m (6 ft), a bench of 1.2 m (4 ft) minimum shall be provided at the toe of the sloped portion.

- Materials used for sheeting and sheet piling, bracing, shoring, and underpinning shall be in good serviceable condition. Timbers used shall be sound and free from large or loose knots, and shall be designed and installed so as to be effective to the bottom of the excavation.
- Additional precautions by way of shoring and bracing shall be taken to prevent slides or cave-ins:
    - When excavations or trenches are made in locations adjacent to backfilled excavations; or
    - Where excavations are subjected to vibrations from railroad or highway traffic, operation of machinery, or any other source.
- Employees entering bell-bottom pier holes shall be protected by the installation of a removable-type casing of sufficient strength to resist shifting of the surrounding earth. Such temporary protection shall be provided for the full depth of that part of each pier hole that is above the bell. A lifeline, suitable for instant rescue and securely fastened to the shafts, shall be provided. This lifeline shall be individually manned and separate from any line used to remove materials excavated from the bell footing.
- Minimum requirements for trench timbering shall be in accordance with tables herein.
- Where employees are required to be in trenches 3 ft deep or more, ladders shall be provided which extend from the floor of the trench excavation to at least 3 feet above the top of the excavation. They shall be located to provide means of exit without more than 25 ft of lateral travel.
- Bracing or shoring of trenches shall be carried along with the excavation.

- Cross braces or trench jacks shall be placed in true horizontal position, spaced vertically, and secured to prevent sliding, falling, or kickouts.
- Portable trench boxes or sliding trench shields may be used for the protection of employees only. Trench boxes or shields shall be designed, constructed, and maintained to meet acceptable engineering standards.
- Backfilling and removal of trench supports shall progress together from the bottom of the trench. Jacks or braces shall be released slowly, and in unstable soil, ropes shall be used to pull out the jacks or braces from above after employees have cleared the trench.

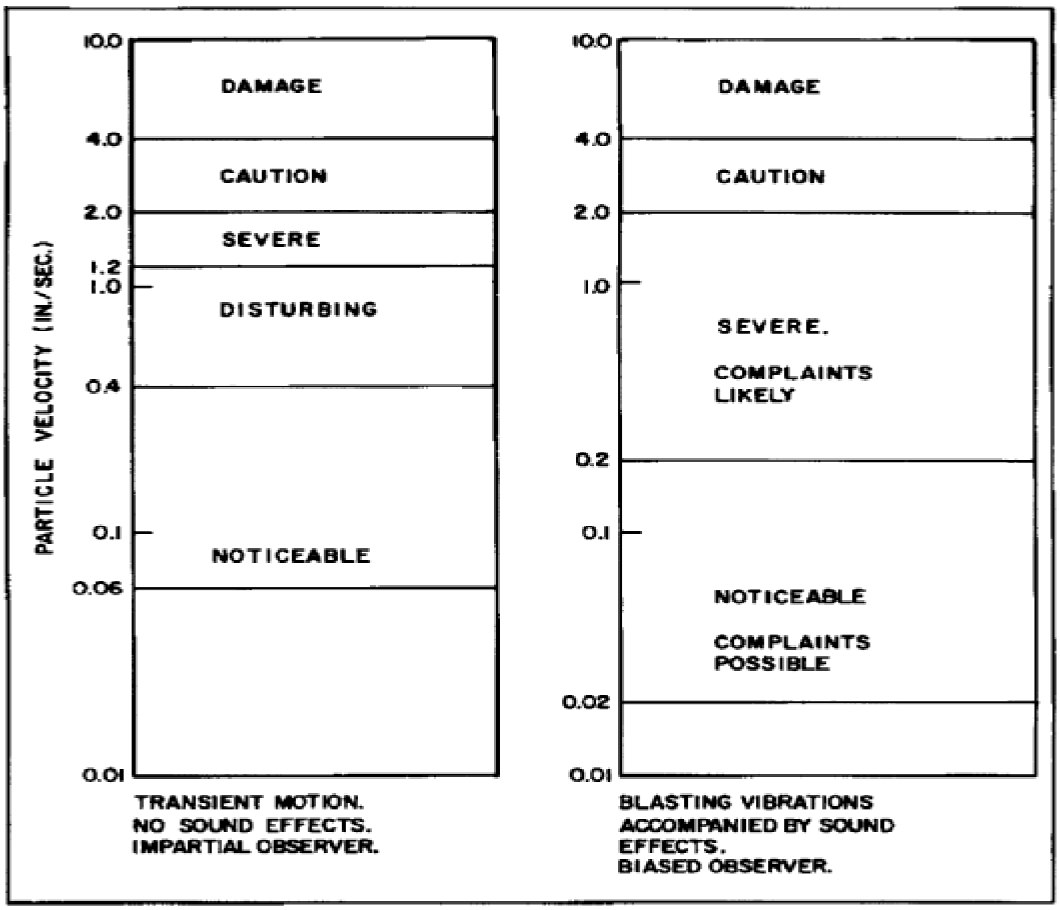


Figure 6

Guide for Predicting Human Response to Vibrations and Blasting Effects

6. EMBANKMENT CROSS-SECTION DESIGN

6.1 INFLUENCE OF MATERIAL TYPE. Table 4 lists some typical properties of compacted soils that may be used for preliminary analysis. For final analysis, engineering property tests are necessary. See Table 5 for relative desirability of various soil types in earth fill dams, canals, roadways and foundations. Although practically any non-organic insoluble soil may be incorporated in an embankment when modern compaction equipment and control standards are employed, the following soils may be difficult to use economically:

- Fine-grained soils may have insufficient shear strength or excessive compressibility.
- Clays of medium to high plasticity may expand if placed under low confining pressures and/or at low moisture contents.
- Plastic soils with high natural moisture are difficult to process for proper moisture for compaction
- Stratified soils may require extensive mixing of borrow.

6.2 EMBANKMENTS ON STABLE FOUNDATION. The side slopes of fills not subjected to seepage forces ordinarily vary between 1 on 1-1/2 and 1 on 3. The geometry of the slope and berms are governed by the requirements for erosion control and maintenance.

6.3 EMBANKMENTS ON WEAK FOUNDATIONS. Weak foundation soils may require partial or complete removal, flattening of embankment slopes, or densification.

6.4 EMBANKMENT SETTLEMENT. Settlement of an embankment is caused by foundation consolidation, consolidation of the embankment material itself, and secondary compression in the embankment after its completion.

- Significant excess pore pressures can develop during construction of fills exceeding about 80 ft in height or for lower fills of plastic materials placed wet of optimum moisture. Dissipation of these excess pore pressures after construction results in settlement. For earth dams and other high fills where settlement is critical, construction pore pressures should be monitored by recognized methods.

- Even for well-compacted embankments, secondary compression and shear strain can cause slight settlements after completion. Normally, this is only of significance in high embankments, and can amount to between 0.1 and 0.2 percent of fill height in three to four years or between 0.3 and 0.6 percent in 15 to 20 years. The larger values are for fine-grained plastic soils.

6.5 EARTH DAM EMBANKMENTS. Evaluate stability at three critical stages: the end of construction stage, steady state seepage stage, and rapid drawdown stage. Seismic forces must be included in the evaluation. Requirements for seepage cutoff and stability dictate design of cross section and utilization of borrow materials.

6.5.1 SEEPAGE CONTROL. Normally the earthwork of an earth dam is zoned with the least pervious fine-grained soils in the central zone and coarsest most stable material in the shell.

- Consider the practicability of a positive cutoff trench extending to the impervious strata beneath the embankment and into the abutments.
- For a properly designed and constructed zoned earth dam, there is little danger from seepage through the embankment. Drainage design is generally dictated by the necessity for intercepting seepage through the foundation or abutments. Downstream seepage conditions are more critical for homogeneous fills.

6.5.2 PIPING AND CRACKING. A great danger to earth dams, particularly those of zoned construction, is the threat of cracking and piping. Serious cracking may result from tension zones caused by differences in stress-strain properties of zoned material. Analyze the embankment section for potential tension zone development. Place an internal drainage layer immediately downstream of the core to control seepage from possible cracking, if foundation settlements are expected to be high.

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, gm/cm^3 (lbs/ft^3)	Range of Optimum Moisture, Percent	Typical Value of Compression		Typical Strength Characteristics				Typical Coefficient of Permeability cm/min (ft/min)	Range of CBR Values	Range of Subgrade Modulus k gm/cm^3 (lbs/ft^3)
				At 134 kPa (1.4 tsf)	At 345 kPa (3.6 tsf)	Cohesion (as Compacted) kPa (lb/ft^2)	Cohesion (Saturated) kPa (lb/ft^2)	Effective Stress Envelope Deg	Tan			
GW	Well graded, clean gravel – sand mixtures	2.002 – 2.162 (125 – 135)	11 – 8	0.3	0.6	0	0	>38	>0.79	9×10^{-2} (3×10^{-2})	40 – 80	8,300 – 13,800 (300 – 500)
GF	Poorly graded, clean gravel – sand mixtures	1.842 – 2.002 (115 – 125)	14 – 11	0.4	0.9	0	0	>37	>0.74	3.0 (10^{-1})	30 – 60	6,900 11,100 (250 – 400)
GM	Silty gravels, poorly graded gravel-sand-silt	1.922 – 2.162 (120 – 135)	12 – 8	0.5	1.1	---	---	>34	>0.67	3×10^{-4} ($>10^{-5}$)	20 – 60	2,800 – 11,100 (100 – 400)
GC	Clayey gravels, poorly graded gravel-sand-silt	1.842 – 2.082 (115 – 130)	14 – 9	0.7	1.6	---	---	>31	>0.60	3×10^{-6} ($>10^{-7}$)	20 – 40	2,800 – 8,300 (100 – 300)

Table 4

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, gm/cm^3 (lbs/ft^3)	Range of Optimum Moisture, Percent	Typical Value of Compression: At 134 kPa (1.4 tsf)	Typical Value of Compression: At 345 kPa (3.6 tsf)	Typical Strength Characteristics: Cohesion (as Compacted) kPa (lb/ft^2)	Typical Strength Characteristics: Cohesion (Saturated) kPa (lb/ft^2)	Typical Strength Characteristics: Effective Stress Envelope Deg	Typical Strength Characteristics: Tan	Typical Coefficient of Permeability cm/min (ft/min)	Range of CBR Values	Range of Subgrade Modulus k gm/cm^3 (lbs/ft^3)
ML	Well graded clean sands, gravelly sands	1.762 – 2.082 (110 – 130)	16 – 9	0.6	1.2	0	0	38	0.39	3×10^{-2} ($>10^{-3}$)	20 – 40	5,500 – 8,300 (200 – 300)
ML - CL	Poorly graded clean sands, sand gravel mix	1.602 – 1.922 (100 – 120)	21 – 12	1.0	2.2	65 (1350)	22 (460)	32	0.52	1.5×10^{-5} ($>5 \times 10^{-7}$)	***	***

Table 4 (continued)

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, gm/cm^3 (lbs/ft^3)	Range of Optimum Moisture, Percent	Typical Value of Compression		Typical Strength Characteristics				Typical Coefficient of Permeability cm/min (ft/min)	Range of CBR Values	Range of Subgrade Modulus k gm/cm^3 (lbs/ft^3)
				At 134 kPa (1.4 tsf)	At 345 kPa (3.6 tsf)	Cohesion (as Compacted) kPa (lb/ft^2)	Cohesion (Saturated) kPa (lb/ft^2)	Effective Stress Envelope Deg	Tan			
SW	Well graded clean sands, gravelly sands	1.762 – 2.082 (110 – 130)	16 – 9	0.6	1.2	0	0	38	0.79	3 x 10^{-2} (>10^{-3})	20 – 40	5,500 – 8,300 (200 – 300)
SP	Poorly graded clean sands, sand gravel mix	1.602 – 1.922 (100 – 120)	21 – 12	0.8	1.4	0	0	37	0.74	3 x 10^{-2} (>10^{-3})	10 – 40	5,500 – 8,300 (200 – 300)
SM	Silty soils, poorly graded sand-silt mix	1.762 – 2.002 (110 – 125)	16 – 11	0.8	1.6	50 (1050)	20 (420)	34	0.67	1.5 x 10^{-3} (>5 x > 10^{-5})	10 – 40	5,500 – 8,300 (200 – 300)
SM – SC	Sand-silt clay mix with slightly plastic fines	1.762 – 2.082 (110- 130)	15 – 11	0.8	1.4	50 (1050)	14 (300)	33	0.68	>6 x 10^{-5} (>2 x 10^{-6})	5-30	2,800 – 8,300 (100-300)

Table 4 (continued)

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, gm/cm^3 (lbs/ft^3)	Range of Optimum Moisture, Percent	Typical Value of Compression		Typical Strength Characteristics				Typical Coefficient of Permeability cm/min (ft/min)	Range of CBR Values	Range of Subgrade Modulus k gm/cm^3 (lbs/ft^3)
				At 134 kPa (1.4 tsf)	At 345 kPa (3.6 tsf)	Cohesion (as Compacted) kPa (lb/ft^2)	Cohesion (Saturated) kPa (lb/ft^2)	Effective Stress Envelope Deg	Tan			
SC	Clay like sands, poorly graded sand/ clay mix.	-2.002 (105-125)	19-11	1.1	2.2	74 (1550)	11 (230)	31	0.60	$>2 \times 10^{-7}$ ($>2 \times 10^{-7}$)	5-20	2,800 – 8,300 (100-300)
CL	Inorganic clays of low to medium plasticity	1.922 (95-120)	24-12	1.3	2.5	86 (1800)	13 (270)	28	0.54	$>3 \times 10^{-6}$ ($>10^{-7}$)	15 or less	1,400 – 5,500 (50-200)
OL	Organic silts and silt clays low plasticity	80-100	33-21	***	***	****	*****	***	***	***	5 or less	1,400 – 2,800 (50-100)

Table 4 (continued)

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, gm/cm³ (lbs/ft³)	Range of Optimum Moisture, Percent	Typical Value of Compression		Typical Strength Characteristics				Typical Coefficient of Permeability cm/min (ft/min)	Range of CBR Values	Range of Subgrade Modulus k gm/cm³ (lbs/ft³)
				At 134 kPa (1.4 tsf)	At 345 kPa (3.6 tsf)	Cohesion (as Compacted) kPa (lb/ft²)	Cohesion (Saturated) kPa (lb/ft²)	Effective Stress Envelope Deg	Tan			
MH	Inorganic clay silts/elastic silt	1.121 - 1.522 (70-95)	40-24	2.0	3.8	73 (1500)	20 (420)	25	0.47	$>1.5 \times 10^{-5}$ ($>5 \times 10^{-7}$)	10 or less	1,400 – 2,800 (50-100)
CH	Inorganic clays of high plasticity	1.201 – 1.682 (75-105)	36-19	2.6	3.9	342 (7150)	11 (230)	19	0.35	$>3 \times 10^{-6}$ ($>10^{-7}$)	15 or less	700 – 3,500 (50-150)
OH	Organic & silty clays	1.041 – 1.602 (65-100)	45-21	***	***	*****	*****	****	****	****	5 or less	350 – 2800 (25-100)

Notes:

1. All properties are for conditions of "Standard Proctor" maximum density, except volume of " k " and CBR which are for "Modified Proctor" maximum density.
2. Typical strength characteristics are for effective envelopes and are obtained USBR data.
3. Compression values are for vertical loading with complete lateral containment.
4. (>) indicates that typical property I greater than the value shown. Asterisks (*) indicate insufficient data available for an estimate.

Table 4 (continued)

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Relative Desirability for Various Uses									
		Rolled Earth Fill Dams			Canal Sections		Foundations		Roadways		
									Fills		
		Homogeneous Embankment	Core	Shell	Erosion Resistance	Compacted Earth Lining	Seepage Important	Seepage Not Important	Frost Heave Not Possible	Frost Heave Possible	Surfacing
GW	Well graded gravels, gravel-sand mixture, little or fines	-	-	1	1	-	-	1	1	1	3
GF	Poorly graded gravels, gravel-sand mixture, little or no fines	-	-	2	2	-	-	3	3	3	-
GM	Silty gravels, poorly graded gravel-sand-silt mixtures	2	4	-	4	4	1	4	4	9	5
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures	1	1	-	3	1	2	6	5	5	1
SW	Well graded sands, gravel like sands, little or no fines	-	-	3 if gravelly	6	-	-	2	2	2	4
SP	Poorly graded sands, gravel like sands, little or no fines	-	-	4 if gravelly	7 if gravelly	-	-	5	6	4	-
SM	Silty sands, poorly graded sand-silt mixtures	4	5	-	8 if gravelly	5 erosi on critic al	3	7	6	10	6
SC	Clay like sands, poorly graded sand-clay mixtures	3	2	-	5	2	4	8	7	6	2

Table 5

Relative Desirability of Soils as Compacted Fill

Group Symbol	Soil Type	Relative Desirability for Various Uses									
		Rolled Earth Fill Dams			Canal Sections		Foundations		Roadways		
									Fills		
		Homogeneous Embankment	Core	Shell	Erosion Resistance	Compacted Earth Lining	Seepage Important	Seepage Not Important	Frost Heave Not Possible	Frost Heave Possible	Sufacing
ML	Inorganic silts and very fine sands. Rock flower, silty or clayey fine sands with slight plasticity.	6	6	-	-	6 Erosion critical	6	9	10	11	–
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	5	3	-	9	3	5	10	9	7	7
OL	Organic silts and organic silts-clays of low plasticity	8	8	-	-	7 Erosion critical	7	11	11	12	-
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	9	9	-	-	-	8	12	12	13	-
CH	Inorganic clays of high plasticity, fat clays	7	7	-	10	8 volume change critical	9	13	13	8	-
OH	Organic clays of medium high plasticity	10	10	-	-	-	10	14	14	14	-

(-) indicates not appropriate for this type of use.

Table 5 (continued)

Relative Desirability of Soils as Compacted Fill

6.5.3 DISPERSIVE SOIL. Dispersive clays should not be used in dam embankments. Determine the dispersion potential using Table 6. A hole through a dispersive clay will increase in size as water flows through (due to the breakdown of the soil structure), whereas the size of a hole in a non-dispersive clay would remain essentially constant. Therefore, dams constructed with dispersive clays are extremely susceptible to piping.

*Percent Dispersion	Dispersive Tendency
Over 40	Highly Dispersive (do not use)
15 to 40	Moderately Dispersive
0 to 15	Resistant to Dispersion

*The ratio between the fraction finer than 0.005 mm in a soil-water suspension that has been subjected to a minimum of mechanical agitation, and the total fraction finer than 0.005 mm determined from a regular hydrometer test x 100.

Table 6

Clay Dispersion Potential

7. BORROW EXCAVATION

7.1 BORROW PIT EXPLORATION. Make exploratory investigations to determine the suitable sources of borrow material. Laboratory tests to determine the suitability of available materials include natural water contents, compaction characteristics, grain-size distribution, Atterberg limits, shear strength, and consolidation. Typical properties of compacted materials for use in preliminary analyses are given in Table 4. The susceptibility to frost action also should he considered in analyzing the potential behavior of fill material. The scope of laboratory testing on compacted samples depends on the size and cost of the structure, thickness and extent of the fill, and also strength and compressibility of underlying soils. Coarse-grained soils are preferred for fill; however, most fine-grained soils can be used advantageously if attention is given to drainage, compaction requirements, compaction moisture, and density control. The number and spacing of borings or test pits for borrow exploration must be sufficient to determine the approximate quantity and quality of construction materials within an economical haul distance from the project. For mass earthwork, initial exploration should be on a 61-meter (200-foot) grid. If variable conditions are found during the initial explorations, perform intermediate borings or test pits. Explorations should develop the following information:

- A reasonably accurate subsurface profile to the anticipated depth of excavation
- Engineering properties of each material considered for use
- Approximate volume of each material considered for use
- Water level
- Presence of salts, gypsums, or undesirable minerals
- Extent of organic or contaminated soils, if encountered

7.2 EXCAVATION METHODS. Consider the following when determining excavation methods:

- Design and efficiency of excavation equipment improves each year. Check various construction industry publications for specifications.
- Determine rippability of soil or rock by borings, geophysical exploration, and/or trial excavation.

7.3 UTILIZATION OF EXCAVATED MATERIALS. In the process of earthmoving, there may be a reduction of the volume ("shrinkage") because of waste and densification, or an increase of volume ("swell") in the case of rock or dense soils because the final density is less than its original density. Determine total borrow volume, VB, required for compacted fill as follows:

$$V_B = \left(\frac{\gamma_F}{\gamma_B} \cdot V_F\right) + \frac{W_L}{\gamma_B}$$

where: γ_F = dry unit weight of fill

γ_B = dry unit weight of borrow

V_F = required fill volume

W_L = weight lost in stripping, waste, oversize and transportation

The volume of borrow soil required should be increased according to the volume change indicated above. A "shrinkage" factor of 10 to 15 percent may be used for estimating purposes. Note that a large percentage of cobble size material will increase the waste, because sizes larger than 3 inches are generally excluded from compacted fill. Note the following for Rock Fill:

- Maximum expansion ("swell") from in-situ conditions occurs in dense, hard rock with fine fracture systems, which breaks into uniform sizes. Unit volume in a quarry will produce approximately 1.5 volumes in fill.
- Minimum expansion occurs in porous, friable rock that breaks into broadly graded sizes with numerous spalls and fines. Unit volume in quarry will produce approximately 1.1 volumes in fill.