Pipe Interaction with the Backfill Envelope

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FOREWORD

The project involved a study of the effects of pipe installation methods on pipe performance. Both laboratory and full-scale field tests were conducted. Pipes used in the tests were donated by Contech Construction Products, CSR/New England, Hancor, Inc., and Plexco/Spirolite, Inc. These pipes are representative of those widely used in practice for drainage applications. The results of the study, including a review of present practices, were used to develop recommendations for improving installation practices. This work is important to pipe design because proper design has to consider the effects of the installation process.

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16. Abstract

This report summarizes a study of installation practices for buried (culvert) pipes. Current practice was reviewed through a literature search and a survey of users, manufacturers, and other involved in the use of buried pipes.

Typical backfill materials were characterized through standard and variable effort compaction tests, CBR tests, penetration tests and one-dimensional compression tests. Standard classification systems were compared and standard groups of backfill materials were evaluated. The soil properties that were used to develop the AASHTO SIDD designs are proposed for use as standard properties for application to the installation of all types of pipes. The Constrained Modulus, M_s, is proposed as the standard measure of soil stiffness replacing the empirical Modulus of Soil Reaction E'.

Laboratory soil box tests and full-scale field tests were conducted to investigate soil behavior during installation. Variables include pipe type (concrete, steel, plastic) and size (600 & 1,500 mm {36 & 60 in.}), in-situ soil condition, trench width, backfill type, compactive effort, haunching effort, and bedding condition. The tests showed that all of the backfill-related test variables have a significant effect on pipe behavior. Tests with controlled low-strength materials showed this to be an excellent type of backfill. Computer modelling demonstrated that the finite element analysis can effectively model installation effects as well as effects of the fill over the pipe. The elastic solution for behavior of buried pipe, developed by Burns and Richard, shows promise as a basis for a simplified design method.

Recommendations for future practice include the use of soft bedding under the bottom of the pipe and of uncompacted fill over the top. Selection of trench width must consider the ability to place and compact backfill in the haunch zone and at the sides of the pipe. Hand tampers, sized differently for different backfills, were shown to be useful for providing haunching effort. It was shown that relatively small changes in backfill density can have significant effects on backfill stiffness.

The project shows that pipe performance is controlled by installation practices. Implementation of a design process that realistically assesses how a project will be built, and construction that understands and implements the design, is imperative for successful long-term culvert performance.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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CHAPTER 1
INTRODUCTION

1.1 Background

The long-term behavior of buried culverts and other gravity flow pipes is significantly affected by installation practice. While designers often think of the design process as design of a pipe, they are in fact designing a "pipe-soil system" where structural performance depends on structural characteristics of both the pipe and the soil. Rarely, with products in use today, can any rigid or flexible pipe carry all superimposed loads without depending in some way on the surrounding soil for support. Bedding must be uniform to prevent point loads, and lateral soil pressure at the sides of the pipe must be of sufficient magnitude to restrain deformation. Even the loads imposed on a buried pipe are related to the practices used at the time of construction. Thus, designing a buried pipe requires the simultaneous design of the surrounding backfill. Further, if the backfill conditions are important in the design phase, then, it becomes incumbent upon the designer and builder to see that the backfill assumptions made in design are implemented in the field during construction. This is the pipe-soil system design process.

Installation standards for buried pipe have not been thoroughly reviewed from a geotechnical perspective for many years, and some current installation standards use terminology that is outdated and unsuitable for current construction contracts. Also, many industries have proposed their own design and installation standards, suggesting that different types of pipes are fundamentally different and require separate treatment. This is a situation which creates confusion for both designers and installers. Present practice in these two areas needs to be reviewed for updating where necessary and for making standards as uniform as possible across all types of pipes.

A great deal of effort has been expended by the pipe industry and others on the development of mathematical models that describe pipe-soil interaction; however, most of this work has been on the properties of soil after compaction and does not evaluate the soil and pipe behavior that result from the application of compaction forces. Information is needed to correct this deficiency.

The overall goal of this research is to develop a fundamental understanding of the interactions between a buried pipe, the backfill soil around it, and the in situ soil in which the pipe/backfill system is installed. This improved understanding can in turn be used to develop more reliable and economical pipe installation and design methodologies based on improving the control of installation procedures during construction. Development of improved tools for use by designers in assessing the potential performance of installations is also a goal.

1.2 Objectives

The overall objective of the research was to investigate the fundamental interactions that take place during the process of excavating a trench, preparing the subgrade, installing the pipe, and then placing and compacting backfill around it. The materials and procedures used in this part of a pipe installation will strongly influence pipe performance as the balance of the fill is placed above the top of the pipe. An improved understanding of these fundamentals will aid designers in developing technically better and more economical specifications.

Specific objectives of this research were to:

1. Examine current pipe installation practices;
2. Evaluate the implications of current pipe installation practices on pipe performance and assess the potential benefit of new techniques;
3. Define bedding alternatives for buried pipe installations and their effect on pipe performance;
4. Develop improved compaction specifications relating compacted soil density to soil stiffness; and
5. Develop improved procedures for including installation effects in the design of buried pipe.

1.3 Scope

This research investigated the interactions that take place during soil placement around buried pipe and the soil properties that result from the installation process. This included:

1. Gathering information through literature review and survey of individuals and organizations involved in current projects;

2. Characterizing backfill materials in terms of desired soil properties for good pipe support;

3. Conducting laboratory tests to study significant installation parameters in a controlled environment;

4. Conducting full-scale field tests to evaluate findings from the literature review and laboratory tests and to investigate pipe installation techniques; and

5. Completing analyses and evaluations of field results and synthesis of findings into improved guidelines for design and installation of buried pipe.

1.4 Contents

Chapter 2 presents a review of the state of the art of current pipe installation practice among users and manufacturers, and where appropriate, a review of the design practice that is pertinent to installation. Chapter 3 describes the tests conducted on backfill soils, compares soil models, and proposes a set of design soil moduli based on the constrained (one-dimensional) modulus as a substitute for historical values based on the modulus of soil reaction. Chapter 4 presents the procedures and results of the laboratory and field tests conducted as part of this project to document installation behavior. Chapter 5 presents analysis of the field data with an idealized closed form elasticity solution for buried pipe and with finite element modeling of the actual test conditions. Chapter 6 presents a discussion of several key issues that are touched on in multiple chapters of this report. Finally, conclusions are drawn in chapter 7.

CHAPTER 2
STATE OF THE ART

This chapter presents the current state of the art of pipe installation practice based on a review of the literature, a limited survey of current users and specifiers, and review of current installation standards.

The technical literature related to buried pipe and culverts was collected by Selig, et. al., in preparation for the NSF Pipeline Workshop, held at the University of Massachusetts in 1987. This was compiled in an extensive document called "Bibliography on Buried Pipelines." The information provided in the bibliography will only be repeated as is pertinent to this study.

While the intent of the proposed research was to study installation practices, it is impossible to study the subject without also addressing pipe design practice because the two areas are so closely related. Pipe designers make implicit assumptions about installation materials and procedures to assess the pipe strength required for a given project. For example, in the case of rigid pipe design, the selection of a bedding factor involves an assumption of the lateral soil pressures applied to the pipe after installation. Thus, design issues are addressed as required to evaluate installation practice.

Terminology used in this report is defined in Fig. 2.1. Definitions of important terms follow:

Bedding is the soil on which the pipe is placed. The bedding may be in situ soil, but, in areas where naturally occurring soils are variable, it is preferred to use placed soil.

Embedment zone backfill includes all backfill that is in contact with the pipe.

Foundation is the soil which supports the embedment zone backfill. It must provide a firm stable surface and may be in situ soil or placed backfill. It may also serve as the bedding.

Haunch zone is the region of the backfill above the bedding and directly below the springline of the pipe. It is a region where hand placement and compaction methods are normally required for the backfill.

Initial backfill is the material placed at the sides and immediately over the pipe after it is installed on the bedding.

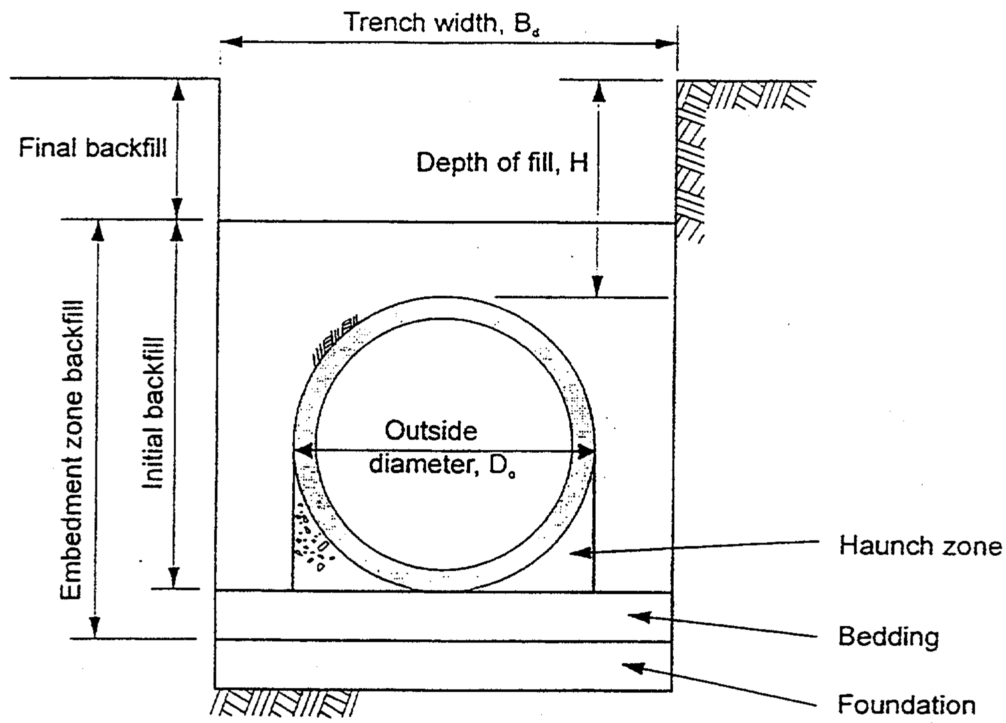


Figure 2.1 Standard Trench Terminology

Rigid Versus Flexible Pipe – This report uses the descriptive terms "rigid" and "flexible" to describe two general classes of pipes. These terms have traditionally been used to differentiate between a pipe with high flexural stiffness (rigid pipe) that carries load primarily through internal moments, and a pipe with low flexural stiffness (flexible pipe) carrying load through internal hoop thrust forces. Flexible pipe develop higher lateral soil pressures at the sides than do rigid pipe. The flexural stiffness of a pipe is described by the parameter EI/R^3, where E is the modulus of elasticity of the pipe material, I is the moment of inertia of the pipe wall, and R is the centroidal radius of the pipe. Concrete and clay pipes are examples of a rigid pipe, with values of EI/R^3 on the order of 7 MPa to 70 MPa (1,000 psi to 10,000 psi), while corrugated metal and plastic pipes are examples of a flexible pipe with EI/R^3 values on the order of 15 kPa to 700 kPa (2 psi to 100 psi). There are two problems with this classification system: (1) the actual response of a system is a function of the relative stiffness of the pipe and soil rather than just the pipe stiffness; and (2) there are no true boundaries to the flexural stiffnesses covered by the classifications, rather there is a transition region where both types of behavior contribute to the overall pipe response. These issues will be discussed further in later chapters.

2.1 Current Design and Installation Practice

The state of the art of current installation practice was evaluated by a survey of users, represented by the States and organizations that sponsored the project, public standards such as American Water Works Association (AWWA), American Society of Civil Engineers (ASCE), and American Society for Testing and Materials (ASTM), and the recommended practices of pipe producers.

2.1.1 General

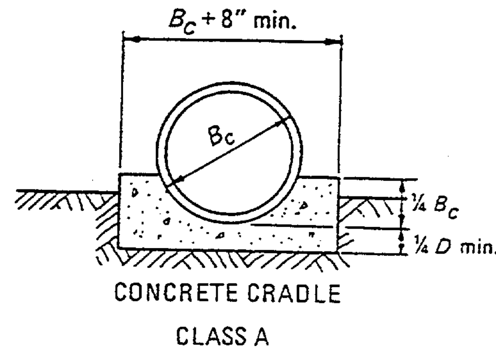
Rigid Pipe - The most commonly used installation specifications for rigid pipes are derived from the work of Marston, Spangler, and others during the first half of the twentieth century (1913, 1917, 1920, 1926, 1930, 1932, 1933, 1950, 1953). Bedding conditions presented in current references such as the ASCE *Manual of Practice No. 37*, (ASCE, 1970), and the American Concrete Pipe Association's (ACPA) *Concrete Pipe Design Manual* (ACPA 1987a), and *Concrete Pipe Handbook* (ACPA 1987b) continue to present installation details based on this early work, (Fig. 2.2). These details are outdated in that they include such vague terms for soils as "granular material," "backfill," "fine granular fill," and even "soil." The compaction requirements in these beddings are also vague, using terminology such as "densely compacted," "carefully compacted," "lightly compacted," "compacted," and "loose." This terminology of backfill materials and compaction levels are difficult to interpret in modern construction contracts.

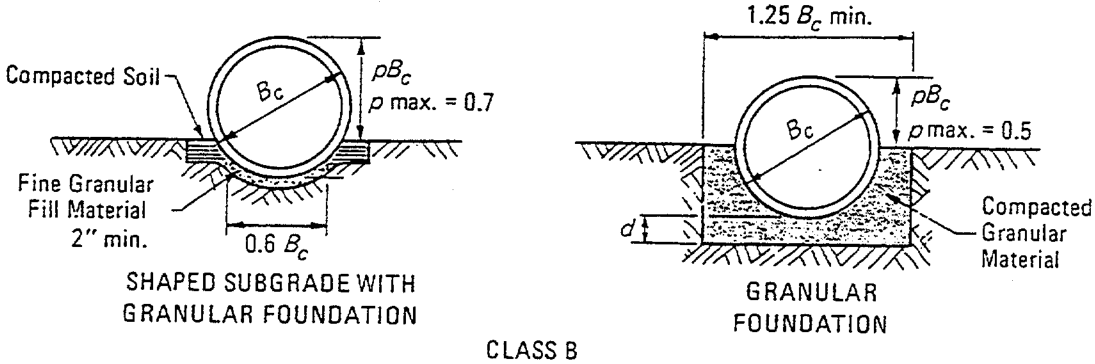
Heger (1988) proposed new "standard" installations for concrete pipes in the embankment condition, based on parametric studies with the finite element computer program SPIDA. These are called SIDD for _S_tandard _I_nstallation _D_irect _D_esign. The SIDD installations have been adopted in ASCE Standard 15-93, "Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations," (ASCE, 1994),

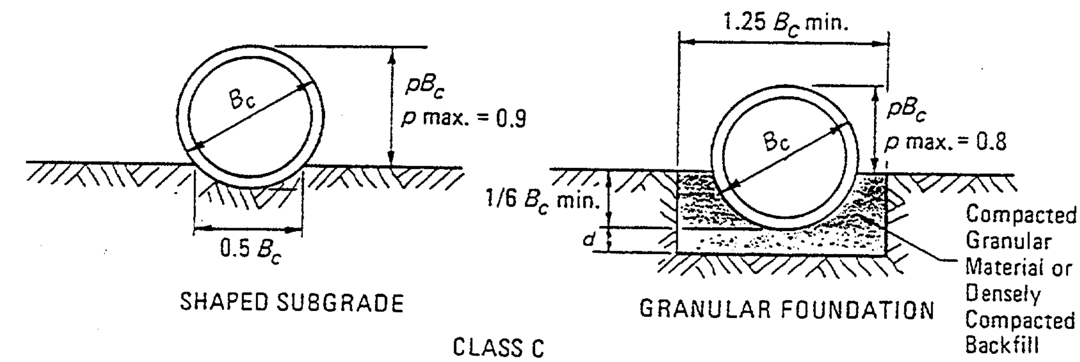
Notes:

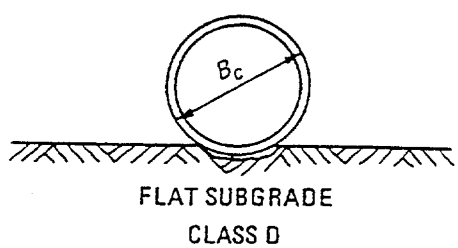
For Class B and C beddings, subgrades should be excavated or over excavated, if necessary, so a uniform foundation free of protruding rocks may be provided.

Special care may be necessary with Class A or other unyielding foundations to cushion pipe from shock when blasting can be anticipated in the area.









Depth of Bedding
Material Below Pipe

D	d (min.)
27" & smaller	3"
30" to 60"	4"
66" & larger	6"

Legend

B_c = outside diameter
H = backfill cover above top of pipe
D = inside diameter
d = depth of bedding material below pipe

Figure 2.2 Traditional Beddings for Rigid Pipe (ACPA 1987a)

"AASHTO Standard Specifications for Highway Bridges," 16th edition (AASHTO, 1996, hereafter called the Standard Specifications), and the AASHTO LRFD Bridge Design Specifications (AASHTO, 1994, hereafter called the LRFD Specifications). This approach is embodied in the Heger pressure distribution, Fig. 2.3, which shows significant variations in the pressure at the pipe-soil interface, particularly in the lower 180 degrees. Table 2.1 provides coefficients that describe the specific distributions for four standard installations. A Type 1 installation is constructed with coarse-grained, well compacted materials, a Type 4 installation is constructed with little control of backfill type or compaction, and Types 2 and 3 installations represent intermediate quality. Specific backfill and material requirements for each type of installation are presented in Fig. 2.4 and Table 2.2. Features of this approach are:

- Soil types and compaction levels are defined in accordance with accepted soil classification systems (AASHTO M 145 and ASTM D 2487), which are easily cited in contracts.
- The area of reduced pressure in the lower haunch zone acknowledges that, even with substantial effort during installation, it is unlikely that installers will achieve the same level of soil compaction as at the sides and bottom of the pipe.
- As the quality of backfill and the compaction level decrease, the invert pressure increases (note the relative values of the coefficients A1 and A2 which define the relative portion of the total load in each zone) and the lateral pressure decreases (note the coefficients A4, A5, and A6).

Liedberg (1991) has published detailed test results that evaluate the Heger work and concluded that the work is valid for embankment installations. Heger's findings should be applicable to pipes in trench installations as well, but the presence of trench walls and the influence of preexisting soils will also influence the selection of appropriate bedding conditions. In spite of the limited verification, the ASCE Standard 15-93 has incorporated the results of Heger's research and extended it to the trench condition. The trench installation is more complex than the embankment case because of the less predictable influence of the preexisting soils, the increased presence of groundwater problems, and the restricted space in which to work. ASCE does require that trench installations be designed for the embankment load condition that is conservative.

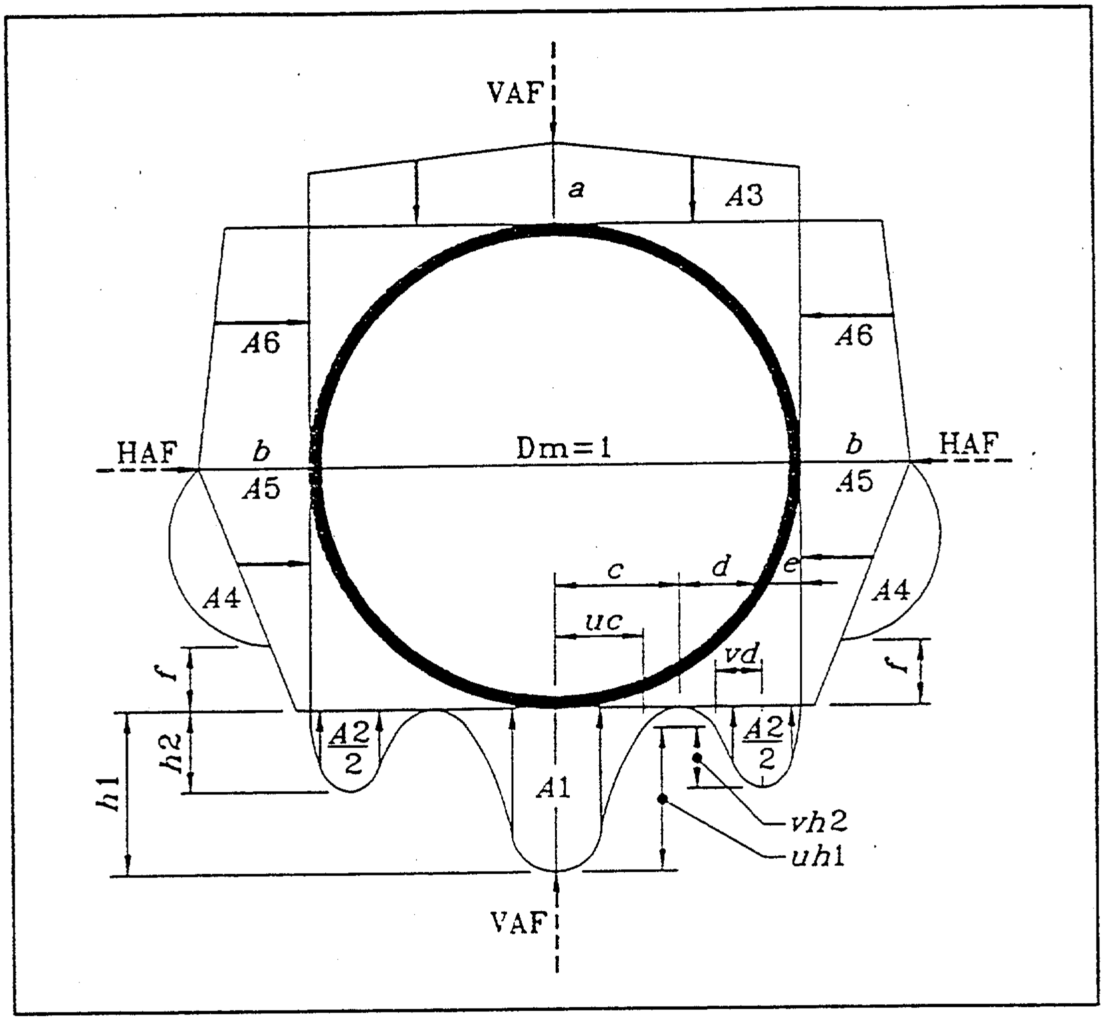


Figure 2.3 Heger Pressure Distribution for SIDD Installations (Heger 1988)

Table 2.1

Design Coefficients for Heger Pressure Distribution (Heger 1988)

Installation Type	VAF	HAF	A1	A2	A3	A4	A5	A6	a	b	c	e	f	u	v
1	1.35	0.45	0.62	0.73	1.35	0.19	0.08	0.18	1.40	0.40	0.18	0.08	0.05	0.80	0.80
2	1.40	0.40	0.85	0.55	1.40	0.15	0.08	0.17	1.45	0.40	0.19	0.10	0.05	0.82	0.70
3	1.40	0.37	1.05	0.35	1.40	0.10	0.10	0.17	1.45	0.36	0.20	0.12	0.05	0.85	0.60
4	1.45	0.30	1.45	0.00	1.45	0.00	0.11	0.19	1.45	0.30	0.25	0.00	-----	0.90	-----

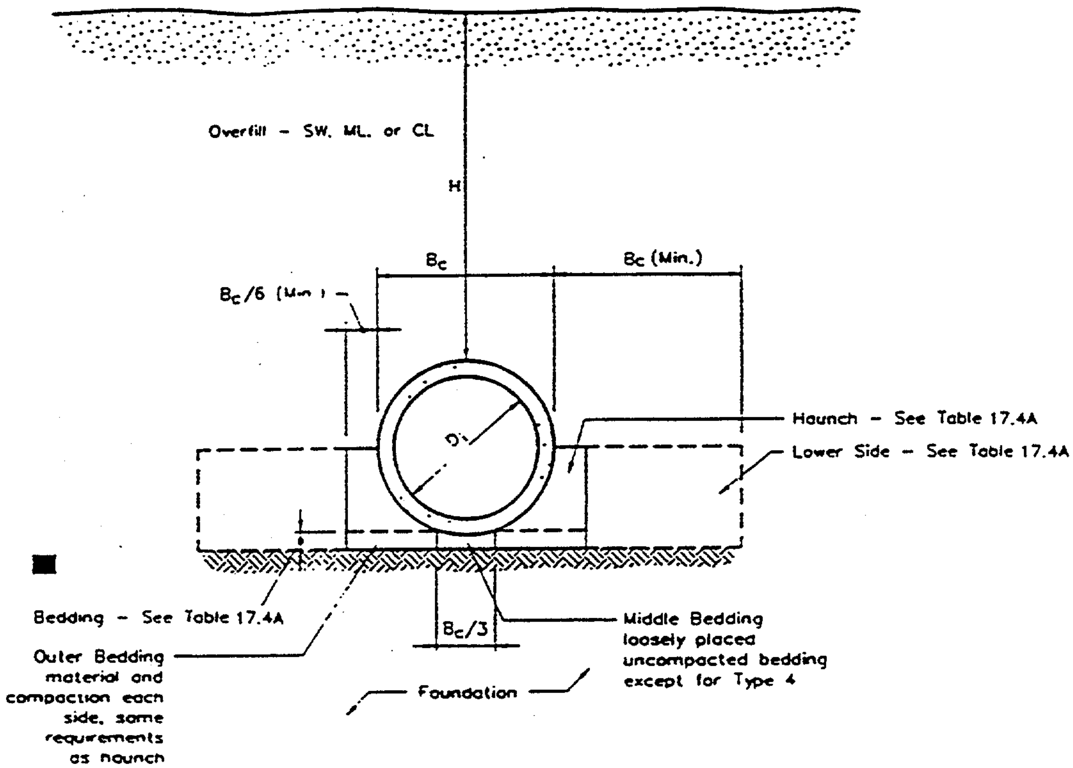


Figure 2.4 SIDD Type Embankment Installation

Table 2.2

SIDD Requirements for Embankment Installations

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	$B_c/24''$ (600 mm) minimum, not less than 3" (75 mm). If rock foundation, use $B_c/12''$ (300 mm) minimum, not less than 6" (150 mm).	95% SW	90% SW, 95% ML, or 100% CL
Type 2 (See Note 3.)	$B_c/24''$ (600 mm) minimum, not less than 3" (75 mm). If rock foundation, use $B_c/12''$ (300 mm) minimum, not less than 6" (150 mm).	90% SW or 95% ML	85% SW, 90% ML, or 95% CL
Type 3 (See Note 3.)	$B_c/24''$ (600 mm) minimum, not less than 3" (75 mm). If rock foundation, use $B_c/12''$ (300 mm) minimum, not less than 6" (150 mm).	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, or 95% CL
Type 4	No bedding required, except if rock foundation, use $B_c/12''$ (300 mm) minimum, not less than 6" (150 mm).	No compaction required, except if CL, use 85% CL	No compaction required, except if CL, use 85% CL

The SIDD method divides backfill soils into three general categories that use the designations SW, ML and CL. The category names are the Unified Soil Classification System (USCS) classifications (ASTM D 2487) of three soils characterized by Selig (1988) and used in the development of the SIDD standard installations. Table 2.3 (AASHTO, 1996) suggests a grouping of all other USCS soil classifications and AASHTO (AASHTO M 145) soil classifications into the three categories. The particular soils were selected as having strength and stiffness properties on the lower end of other soils in the same classification, thus they should be conservative in design.

Loads on rigid pipe in the SIDD system are computed using the Vertical Arching Factor, or VAF. The VAF is the ratio of the total load on the pipe, taken as the springline thrust, to the weight of the soil prism load. The soil prism load is the weight of the soil directly over the pipe. The soil prism load, total load, and VAF are defined in equation form as:

$$W_{sp} = \gamma_s D_o (H + 0.11 D_o) \ , \qquad (2.1)$$

$$W_p = 2 T_{sl} \ , \qquad (2.2)$$

and

$$VAF = \frac{W_p}{W_{sp}} \ , \qquad (2.3)$$

where

W_{sp}	=	weight of soil prism over pipe, kN/m, lb/ft,
γ_s	=	unit weight of soil, kN/m^3, lb/ft^3,
D_o	=	outside diameter of pipe, m, ft,
H	=	depth of fill over top of pipe, m, ft,
W_p	=	total load on pipe, m, ft,
T_{sl}	=	thrust force at springline in pipe wall, kN/m, lb/ft, and
VAF	=	vertical arching factor.

Suggested vertical arching factors for reinforced concrete pipes installed in embankment conditions vary from 1.35 to 1.45 (see table 2.1).

Table 2.3
Equivalent USCS and AASHTO Soil Classifications for SIDD Soil Designations (ASCE 1994)

	Representative Soil Types		Percent Compaction	
SIDD Soil	USCS	AASHTO	Standard Proctor	Modified Proctor
Gravelly Sand (SW)	SW, SP GW, GP	A1, A3	100	95
			95	90
			90	85
			85	80
			80	75
			61	59
Sandy Silt (ML)	GM, SM, ML Also GC, SC with less than 20% passing No. 200 sieve	A2, A4	100	95
			95	90
			90	85
			85	80
			80	75
			49	46
Silty Clay (CL)	CL, MH, GC, SC	A5, A6	100	90
			95	85
			90	80
			85	75
			80	70
			45	40
	CH	A7	100	90
			95	85
			90	80
			45	40

Flexible Pipe – Historically, installation trench details for flexible pipe were less detailed than those for rigid pipe. For example, ASCE Manual No. 37 (ASCE 1970) contains no suggested trench details for flexible pipe. In recent years, installation standards for flexible pipe in general and plastic pipe in particular have become far more detailed and provide excellent guidance for the installation process and for evaluating the potential support that can be derived from soil (see ASTM D 2321 and D 3839).

Flexible pipe design theories continue to rely on the work of Spangler (1941), Watkins and Spangler (1958), and White and Layer (1960). Spangler developed the Iowa formula for calculating pipe deflection under earth load, which uses the modulus of soil reaction, E', as the principal soil parameter. This formula is:

$$\Delta x = \frac{D_l K W}{EI/R^3 + 0.061 E'}, \qquad (2.4)$$

where

Δx	=	change in horizontal diameter, m, in.,
D_l	=	deflection lag factor,
K	=	bedding factor
W	=	load on pipe, MN/m, lb/in.,
E	=	modulus of elasticity of pipe material, MPa, psi,
I	=	moment of inertia of pipe wall, mm^4/mm, $in.^4/in.$,
R	=	centroidal radius of pipe, mm, in., and
E'	=	modulus of soil reaction, MPa, psi.

While E' has been used successfully, it is not a true soil property and efforts to characterize it (Krizek, et al. 1971) have been unsuccessful. Howard (1977, 1996, see section 2.3) showed that E' is a function of soil density and soil type and provided a table of values that have come into common usage; however, these values are back calculated from field deflection measurements and undoubtedly represent the effects of installation practices as well as soil behavior and pipe properties. Hartley and Duncan (1987) used the close relationship between the one-dimensional modulus, M_s, and E' to show that soil stiffness varies with depth. The one-dimensional modulus represents the soil stiffness under uniaxial strain conditions. It is related to Young's modulus of elasticity, E_s, and Poisson's ratio, ν_s, through the relationship:

$$M_s = \frac{E_s(1-\nu_s)}{(1+\nu_s)(1-2\nu_s)} \quad . \tag{2.5}$$

The Iowa formula also uses a bedding factor that is a function of the radial angle at the bottom of the pipe over which a uniform soil pressure is applied to represent the soil reaction. The bedding factor changes from 0.083 for 180 degree bedding to 0.110 for 0 degree bedding, thus, using the Iowa formula, a change from a high bedding angle to a small bedding angle could increase the calculated deflection by about 33 percent.

White and Layer introduced the ring compression theory which assumes that the load carried by a pipe is equal to the soil prism load (VAF = 1.0). This load assumption is widely used for flexible pipe design.

Design and installation standards for flexible pipe generally divide soil types into four or five general groups. ASTM D 2321 describes five soil "Classes." Class I is manufactured coarse graded material, Class II is gravel or sandy soil with less than 12 percent fines, Class III is gravel or sandy soil with 12 percent to 50 percent fines, and Classes IV and V are silts and clays, and organic soils, respectively. Classes I to III are considered good pipe backfills; some Class IV soils are acceptable as backfill under some conditions. The Howard E' table, noted above, classifies soils into four groups based on field data on pipe performance. Soil properties are discussed in more detail in section 2.2.

2.1.2 State and Federal Practice

Each State develops its own pipe design and installation standards based on local practice and conditions. Most States develop their own standards by adapting the general design guidelines contained in AASHTO Standards, historically the Standard Specifications. AASHTO has recently developed a load and resistance factor design method that is incorporated in the LRFD Specifications. Not all States use these specifications as yet; however, the culvert provisions are not substantially different. The following sections present the practice of individual States and the overall AASHTO specifications.

2.1.2.1 Departments of Transportation

Current practice among State Departments of Transportation was evaluated by surveying the practices of the project sponsors. This included 10 geographically diverse States and the Eastern Federal Lands of the Federal Highway Administration. Each organization was sent a questionnaire that inquired as to types of pipe used in highway practice, design methods and standards, backfill materials, methods of installation, and standards for controlling the quality of installations.

Design Practice – Questionnaire responses show that all but one respondent design rigid pipe by indirect design methods (determination of an equivalent three-edge bearing load). Some sponsors reference AASHTO and some reference ACPA literature. Pennsylvania has recently adopted the new SIDD direct design method for concrete pipes, and has developed fill height tables based on this method. California allows direct design (design based on an assumed pressure distribution) as well as indirect design for concrete pipes.

All respondents use AASHTO Sec. 12 for design of corrugated metal pipe. Three respondents include deflection checks for metal pipes even though not required by current AASHTO Specifications.

Seven respondents design plastic pipes by AASHTO Sec. 18, and four respondents limit plastic pipes to depths of fill between 3.5 m and 4.5 m (11 ft and 15 ft).

Other aspects of design practice from the questionnaire include:

- Eight use negative projecting installations but some do so only for reasons of ease of construction, rather than control of load on the culvert;
- Six use the induced trench method but one reports problems with this method; and
- Seven use the modulus of soil reaction, E', as a measure of soil stiffness:
  - Two use the Howard table of E' with values from 0.35 MPa to 21 MPa (50 psi to 3,000 psi) depending on the soil type and compaction level); and
  - Five use one or two values of E', varying between 7.2 MPa and 11.7 MPa (1,050 psi and 1,700 psi); however, three of these five are seeking better methods of determining soil stiffness.

Backfill – All respondents use "granular" backfill, however, the definitions of granular material vary. Materials that are allowed include large particle size, open graded aggregates (AASHTO No. 3), and some with fines content up to 15 percent. Names include select granular fill, granular backfill, gravel borrow, and select material. Some sponsors have separate gradations for select and granular materials. Four sponsors allow installation with fine-grained materials for some products or some situations. One sponsor allows select material to have up to 60 percent silt content.

Other information related to backfill materials used by the questionnaire respondents include:

- Three sponsors allow backfill with native material under some conditions;
- Compaction requirements generally vary from 90 to 100 percent of AASHTO T-99;
- Eight of eleven respondents use controlled low strength materials (CLSM), also called flowable fill, under some conditions;

- Some sponsors specify minimum trench widths as low as the outside diameter plus 150 mm (6 in.). Most sponsors specify maximum trench widths (generally O.D. plus 0.9 m (3 ft)) or three times the outside diameter. Some distinguish between flexible and rigid pipes and some have trench dimensions dependent on the diameter;
- Ten of eleven require or recommend inspection during backfilling;
- Two of eleven require mandrel tests after backfill of flexible pipe;
- Eight of eleven require compaction testing; and
- Two of eleven have specifications concerning groundwater control.

The most common need, based on the respondents' perception of current practice, was a better method to determine E'. Other issues include need for improved flexible pipe design procedures and better treatment of materials outside of the trench.

Of less overall importance but still desired by some respondents were:

- Refinement of the induced trench installation;
- Improved backfill procedures to achieve good support without developing excessive lateral pressures;
- Specifications that allow use of lower quality materials; and
- Better quality joints.

2.1.2.2 AASHTO

AASHTO Standards have been written around three product types: corrugated metal, concrete, and thermoplastic. The AASHTO standards for corrugated metal and reinforced concrete were largely developed by industry trade organizations and then adopted by AASHTO, while the standards for thermoplastic pipes were developed based on the metal pipe standards, presumably on the assumption that thermoplastic and corrugated metal pipes were both flexible conduits and behaved in the same fashion. The construction specifications for AASHTO set forth the installation requirements; however, many installation criteria are selected based on decisions made during the design process, thus, both the design and installation practices must be examined.

Corrugated Metal Pipe Design and Installation - AASHTO design methods for corrugated metal pipe consider hoop compression stresses, for yield and buckling analysis, and the flexibility coefficient, defined as:

$$FF = \frac{R^2}{EI} , \tag{2.6}$$

where

FF	=	flexibility coefficient, m/kN, in./lb,
R	=	centroidal pipe radius, mm, in.,
E	=	pipe modulus of elasticity, MPa, psi, and
I	=	pipe wall moment of inertia, mm^4/mm, $in.^4/in.$.

The flexibility coefficient is a flexural stiffness criterion that is intended to assure sufficient stiffness for the pipe to withstand handling and installation forces. The classical formula for a ring under diametrally opposed line loads (the parallel plate test) is:

$$\frac{F}{\Delta y} = \frac{EI}{0.149 R^3} , \tag{2.7}$$

where

F	=	line load, kN/m, lb/in., and
Δy	=	change in vertical diameter, mm, in..

By rearranging it to the form:

$$\frac{R^2}{EI} = \frac{\Delta y / R}{0.149 F} = FF , \tag{2.8}$$

it can be seen that the flexibility factor is proportional to the percent deflection ($\Delta y/R$) resulting from a unit line load (F), while the pipe stiffness ($F/\Delta y$) used to characterize thermoplastic pipe is the absolute deflection resulting from a line load. Limiting values for the flexibility coefficient have been set empirically based on experience.

Of current AASHTO criteria for metal culvert design, only the buckling equation considers soil stiffness. In the past, corrugated metal pipes were designed for deflection using the Iowa formula and the modulus of soil reaction, E'. This calculation was dropped from the specifications on the basis that if a pipe is properly installed it will not deflect more than the allowable value.

Reinforced Concrete Pipe Design and Installation - Traditional beddings for reinforced concrete pipes were noted above. These bedding conditions are associated with "bedding factors" that relate the load on the actual pipe to a load in a three-edge bearing test that will produce the same bending moment at the pipe invert. The pipe is then designed to resist the three-edge bearing load. This is called indirect design and is the predominant method of concrete pipe design. Alternatively, pipes can be analyzed and designed for the in-ground forces. This is direct design. It is used in some parts of the United States and is the preferred method of design for special conditions such as high fills.

The SIDD installations were actually developed as a direct design method; however, because of a long history of experience and confidence in indirect methods, bedding factors were developed for these installations and have been incorporated into AASHTO specifications. SIDD installations specify soil types in terms of AASHTO and ASTM soil classifications and compaction in terms of a percent of maximum Proctor density. Haunching effort is required for Installation Types 1 to 3. No special fill or compaction is required above the springline, except as required for support of surface pavement or other structures.

Thermoplastic Pipe Design and Installation - AASHTO developed a thermoplastic pipe design procedure on the assumption that thermoplastic pipes were flexible conduits and could be designed in the same manner as corrugated metal pipes. Issues pertinent to thermoplastic pipe design include:

- Design for total tensile strain, which is not considered for metal pipe, is required because not all thermoplastic pipes are ductile; and
- Design is currently based on the soil prism load, which is a common assumption for flexible pipe; however, Hashash and Selig (1990) have shown that loads on corrugated polyethylene pipes can be significantly less than the soil prism load.

2.1.3 Other Installation Practice

Different industries and specific pipe manufacturers have taken different approaches to the design of buried pipe installations. General practice of the corrugated metal, concrete, and thermoplastic pipe industries was explored above. Other industry practices of interest include:

Clay Pipe – Installation practice of the clay pipe industry is defined in ASTM C 12 Standard Practice for Installing Vitrified Clay Pipe. This standard focuses on support of the invert and haunch zones, as do standards for concrete pipes. The standard proposes beddings classified as B, C, D, crushed stone encasement, and controlled density fill (herein this material will be called CLSM for Controlled Low Strength Material).

The B, C, and D beddings are very much like the traditional reinforced concrete beddings, and use somewhat vague terminology such as "carefully placed material" and select material. A bedding using crushed stone encasement, suggesting a backfill material with angular particles, is shown to provide better support to a pipe with simply "gravel" backfill, such as a GW soil. This is consistent with the Howard table of E' values of soil stiffness for flexible pipe. The standard is the only one for pipe installation that currently provides a bedding detail for CLSM, as shown in Fig. 2.5. The detail shows the pipe laid on crushed stone bedding. This is a relatively simple installation from the point of labor, but allows the invert to have a potentially harder support point than the haunches which is undesirable. If the pipe is backfilled prior to the CLSM curing than the pipe could develop a line load at the invert and become overstressed. The standard also calls for a CLSM 28 day compressive strength of 700 to 2100 kPa (100 to 300 psi). This is high if the CLSM is to be considered excavatable. See Section 2.2.4 for additional discussion of CLSM.

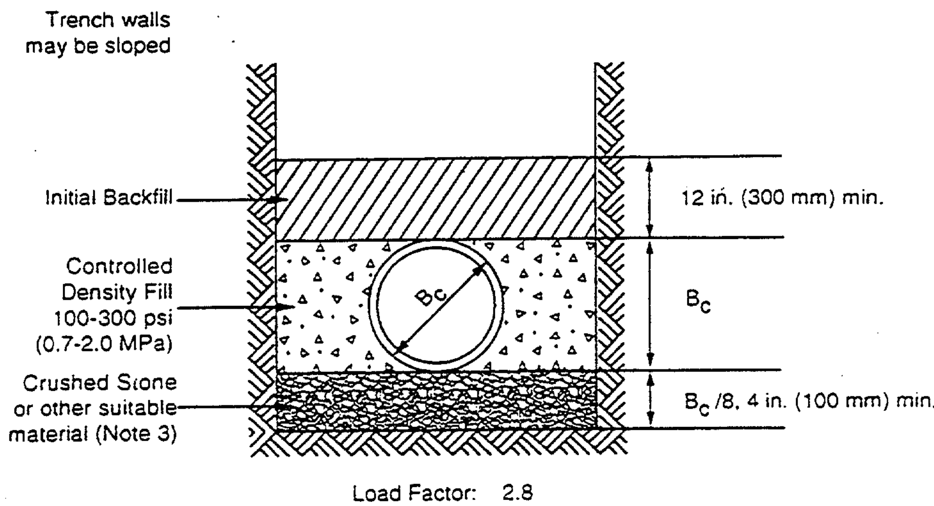


Figure 2.5 Bedding Detail for Clay Pipe with CLSM Backfill (ASTM C 12)

Fiberglass Pipe – Glass fiber reinforced plastic pipe, historically called GRP or FRP but now called simply fiberglass pipe in the United States, can be customized by changing the relative quantity of glass, resin, and, in some cases, sand filler. This allows the industry to produce a wide range of pipe stiffness which in turn allows a broader approach to installation, allowing several trench configurations and backfill conditions. This is documented in part in AWWA Manual M45 (AWWA 1996). One manufacturer's suggested installation details based on pipe stiffness and depth of fill are shown in table 2.4 and fig. 2.6. Fiberglass pipe is more strain sensitive than thermoplastic pipe, thus, more effort has been invested in the prediction of strains in this type of pipe and the design methods are more thorough than is traditionally the case for thermoplastic pipes. The design and installation procedures should be of interest to culvert designers, even if not specifically using fiberglass pipe.

Table 2.4
Installation Requirements for Hobas Fiberglass Pipe

NATIVE SOIL[2,4]	COVER DEPTH (ft.)	EMBEDMENT CONDITION[3] 1	2	3	4
	10 & <				SN[6]72
Rock	10 to 15	SN[6] 36			
Stiff to V. Hard Cohesive	15 to 20			SN[6] 46	
Compact to V. Dense Granular	20 to 25		SN[6] 46		
(Blows/ft.[4] > 8)	25 to 30	SN[6] 46			
	30 to 40		SN[6] 72	ALTERNATE INSTALLATION[7]	
	40 to 50				
	10 & <	SN[6] 36			SN[6] 72
Medium Cohesive	10 to 15		SN[6] 46	SN[6] 46	
Loose Granular	15 to 20	SN[6] 46			
(Blows/ft.[4] 4 to 8)	20 to 25		SN[6] 72	ALTERNATE INSTALLATION[7]	
	25 to 30				
	10 & <	SN[6] 36 to 46		SN[6] 72	
Soft Cohesive	10 to 15	SN[6] 72			
Very Loose Granular	15 to 20				
(Blows/ft.[4] 2 to 4)	over 20			ALTERNATE INSTALLATION[7]	

[1] Assuming minimum trench width per Figure 2.
[2] Blow counts should be representative of weakest condition
[3] Defined in Figure 3. If a cement stabilized sand pipe zone surround is utilized, use column 1 in the highest soils category.
[4] Standard penetration test per ASTM D1586
[5] For v. soft or v.v. loose soils with blow counts less than 2 use alternate installation per section 13, ¶ A8.
[6] SN is nominal stiffness in psi.
[7] Alternate installation per section 13, ¶ A8.

1 ft = 0.305 mm

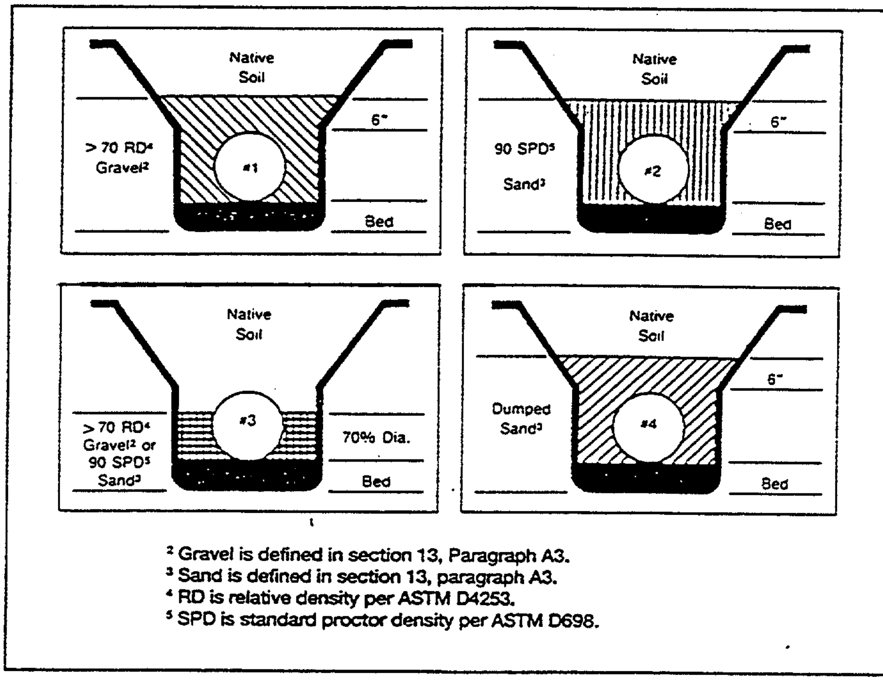


Figure 2.6 Trench Cross-Sections for Hobas Fiberglass Pipe

2.2 Classification and Characterization of Backfill Soils

Backfill materials are usually characterized in terms of gradation and density, and, in the case of fine-grained materials, Atterberg limits. The results of these standard tests are used to estimate a number of mechanical properties used in design. The most important property needed in the design of buried culverts is the soil stiffness; however, it is rare for specifications to require tests specifically for soil stiffness. Engineers often rely on simple empirical relations, such as gradation and density, to establish the soil stiffness. In the field, the importance of the soil stiffness often gets lost in the concern to meet a specification construction requirement for density or gradation. This section reviews standard practices for characterizing soils used as pipe backfill.

2.2.1 Classification Systems

The first step in engineering with soils is typically to characterize the material based on grain size and Atterberg limits (AASHTO M 145, T 88, T 89, and T 90, and the corresponding ASTM D 422, D 2487, D 2488, and D 4318). These tests and classification systems delineate some of the most basic differences among soil types, i.e., particle size and plasticity.

While the AASHTO and ASTM tests listed above for determining grain size and Atterberg limits are equivalent, the soil classification systems based on those test results are not. The AASHTO soil classification system (M 145) was developed for soils to be used as subgrades in road construction, while the ASTM system (D 2487, also called the Unified Soil Classification System or USCS) was developed for broader engineering purposes. Both systems rely on the percentage of material passing a No. 200 sieve (0.075 mm particle size) as the delineation between coarse-grained soils and fine-grained soils; however, each system considers a different percentages as critical. Behavior of coarse-grained soils is best described by particle size while behavior of fine-grained soils is best described by the liquid limit and plasticity index. The quantity of material passing the No. 200 sieve is called the percent fines.

The AASHTO classification system is shown in table 2.5. A soil is classified by using the table from left to right. The first group from the left to fit the soil is the correct AASHTO classification. In addition, the AASHTO system uses a group index based on the plasticity index and liquid limit. The group index is not often used in specifying pipe backfills and is not discussed further here. The AASHTO system classifies any soil with more than 35-percent fines a silt-clay material and any soil with less than 35-percent fines a granular material.

The ASTM classification system is shown in tables 2.6 and 2.7 for coarse and fine grained soils, respectively. A given soil is classified based on the grain size distribution, plasticity index, and liquid limit. The ASTM system classifies any soil with more than 50-percent fines as a fine-grained soil and any soil with less than 50-percent fines as a coarse-grained soil. Coarse-grained soils are characterized based on the coefficient of uniformity, C_u, and the coefficient of curvature, C_c. These coefficients are used to determine if a soil is uniformly or gap graded. Backfill soils are often specified in terms of the two letter group symbol (e.g., SW), however, much more information is available if the group name is used.

Table 2.5
AASHTO Soil Classification System (AASHTO M 145)

General Classification	Granular Materials (35% or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35% Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max.	—	—	—	—	—	—	—	—	—	—
0.425 mm (No. 40)	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
0.075 mm (No. 200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit	—		—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General Ratings as Subgrade	Excellent to Good							Fair to poor			

* Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

Table 2.6
ASTM Classification System for Coarse Grained Soils (ASTM D 2487)

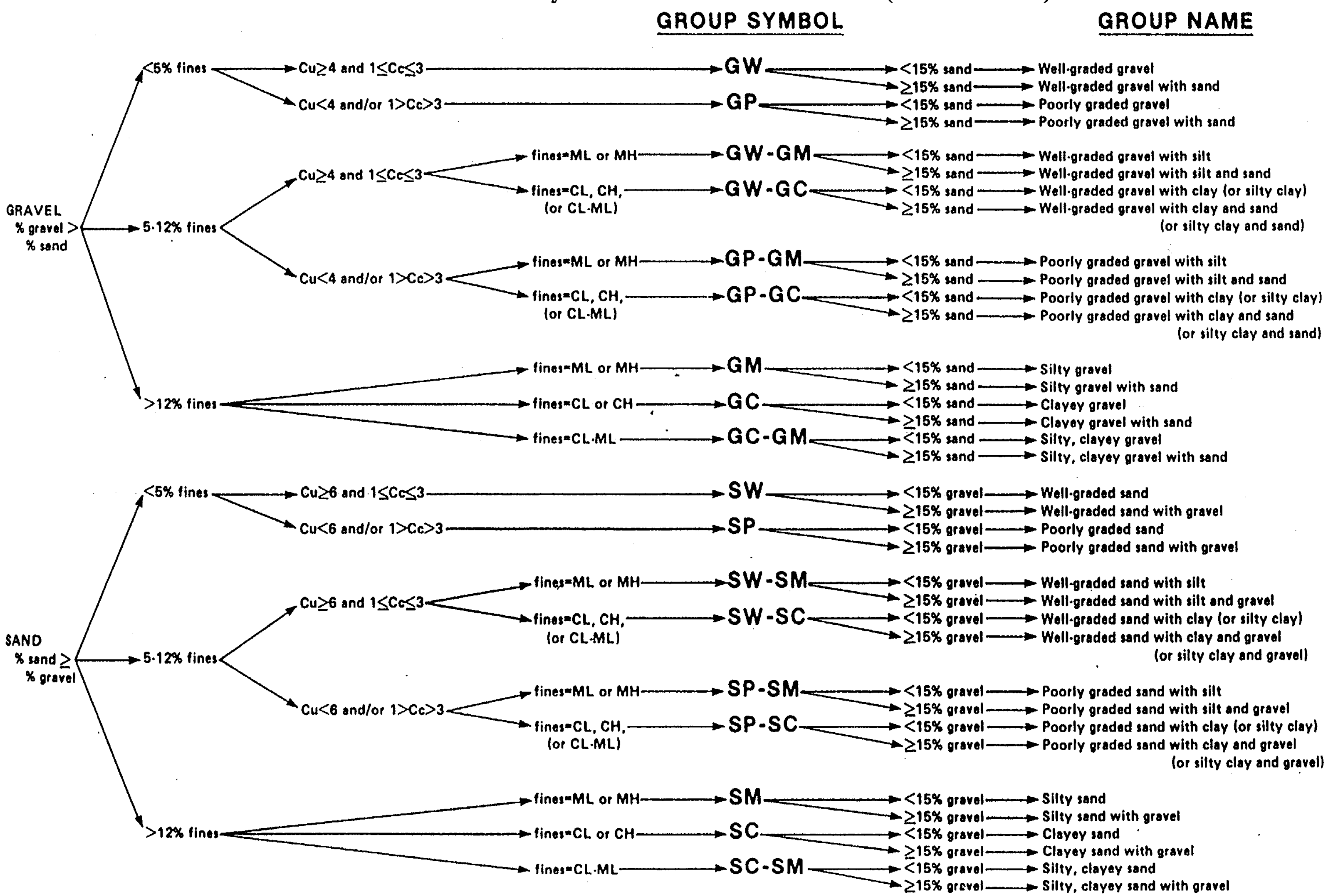
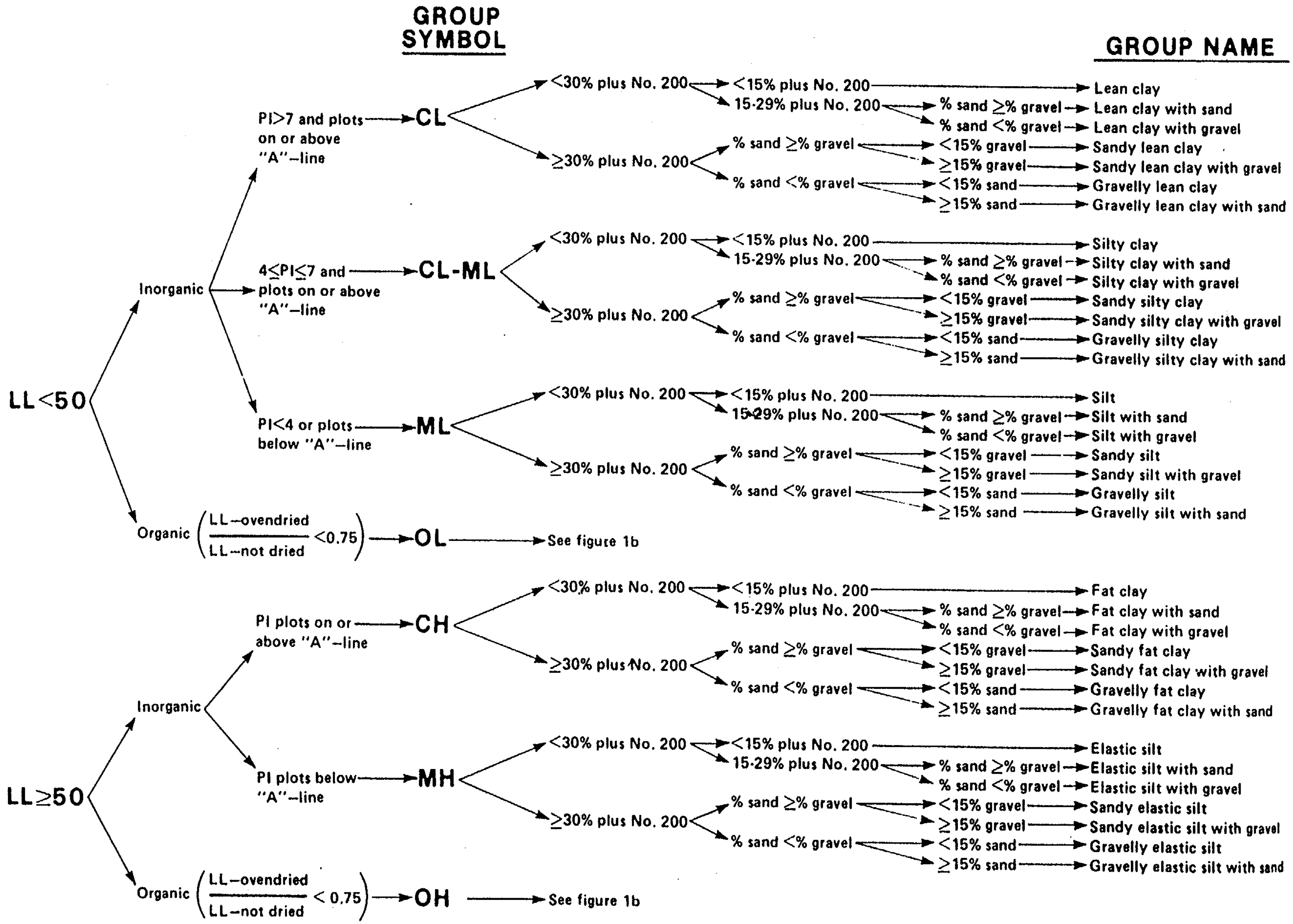


Table 2.7

ASTM Classification System for Fine Grained Soils (ASTM D 2487)

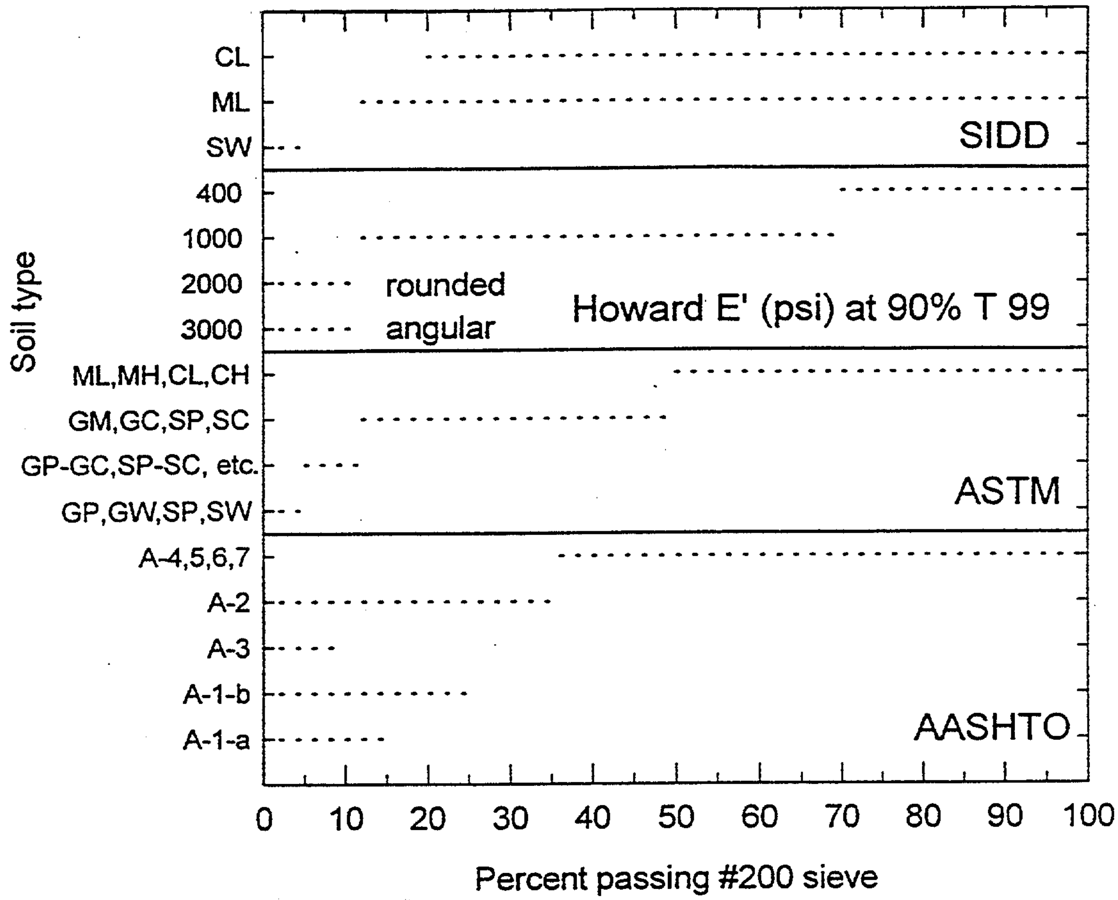


As noted above, a principal criterion for classification of soils is the quantity of fines. Fig. 2.7 compares the AASHTO and ASTM classification systems with the previously discussed soil groups made for structural purposes as assigned by Howard and SIDD based on the fines content. Observations based on this figure include:

- In the ASTM system, fines content is definitive as a first step in classification, i.e., a given soil with certain percentage of fines can only be classified into certain groups. The system uses fines content of 5, 12, and 50 percent as the principle limits; additional limits are available if the group names are used.
- The AASHTO system allows soils with limited fines to fall into one of several classifications as a function of other criteria, and depends on using table 2.5 from left to right to make the necessary distinctions.
- The Howard soil groups correspond closely to ASTM, except that an additional dividing point based on soils with more or less than 30 percent coarse-grained material is introduced, and the aforementioned grouping based on angularity.
- The SIDD soil groups use fines to distinguish between the SW group and both the ML and CL groups; however, for soils with more than 20 percent fines, Atterberg limits are used to distinguish among soils in the ML and CL groups.
- The SIDD soil groups do not specifically call out to which group soils with 5 to 12 percent fines should be assigned.
- The SIDD system puts all A-2 soils into the ML group. The A-2 soil classification group is very broad. It would be more consistent with assignment of ASTM soils if the A-2-6 and A-2-7 soils are reclassified in the CL group.

Review of the data on which the SIDD soil groupings were developed (Selig, 1988) shows that the soil used as the model for the "ML" classification had more than 30 percent coarse-grained material and that the soil used as the model for the "CL" soil classification had less than 30 percent coarse-grained material. This means that they would also fall into separate classification groups according to the E' soil table. The two systems should be reviewed to see if the criteria of silt versus clay, as used in SIDD, or the 30 percent coarse-grained material criteria used for E' is more appropriate as a backfill classification system.

Fig. 2.8 compares the AASHTO and ASTM systems based on plasticity as determined by the Atterberg limits. The figure shows that, while there are differences in details, the two systems generally have similar boundaries to distinguish between different types of behavior.



Notes:

SIDD soil groups:

CL includes A-5, A-6, CL, MH, GC, SC

ML includes GM, SM, ML, (and GC and SC if less than 20% fines), A-2, A-4

SW includes A-1, A-3, GW, SW, GP, SP

Howard soil groups:

E' = 400 includes CL, ML with less than 30% coarse particles

E' = 1000 includes CL, ML with more than 30 % coarse particles, and GM, GC, SM, SC

E' = 2000 inlcudes GW, GP, SW, SP and dual symbol groups GW-GC,etc.

E' = 3000 includes angular processed materials

Figure 2.7 Soil Classifications Based on Fines Content Compared to Howard Soil Stiffnesses and SIDD Soil Types

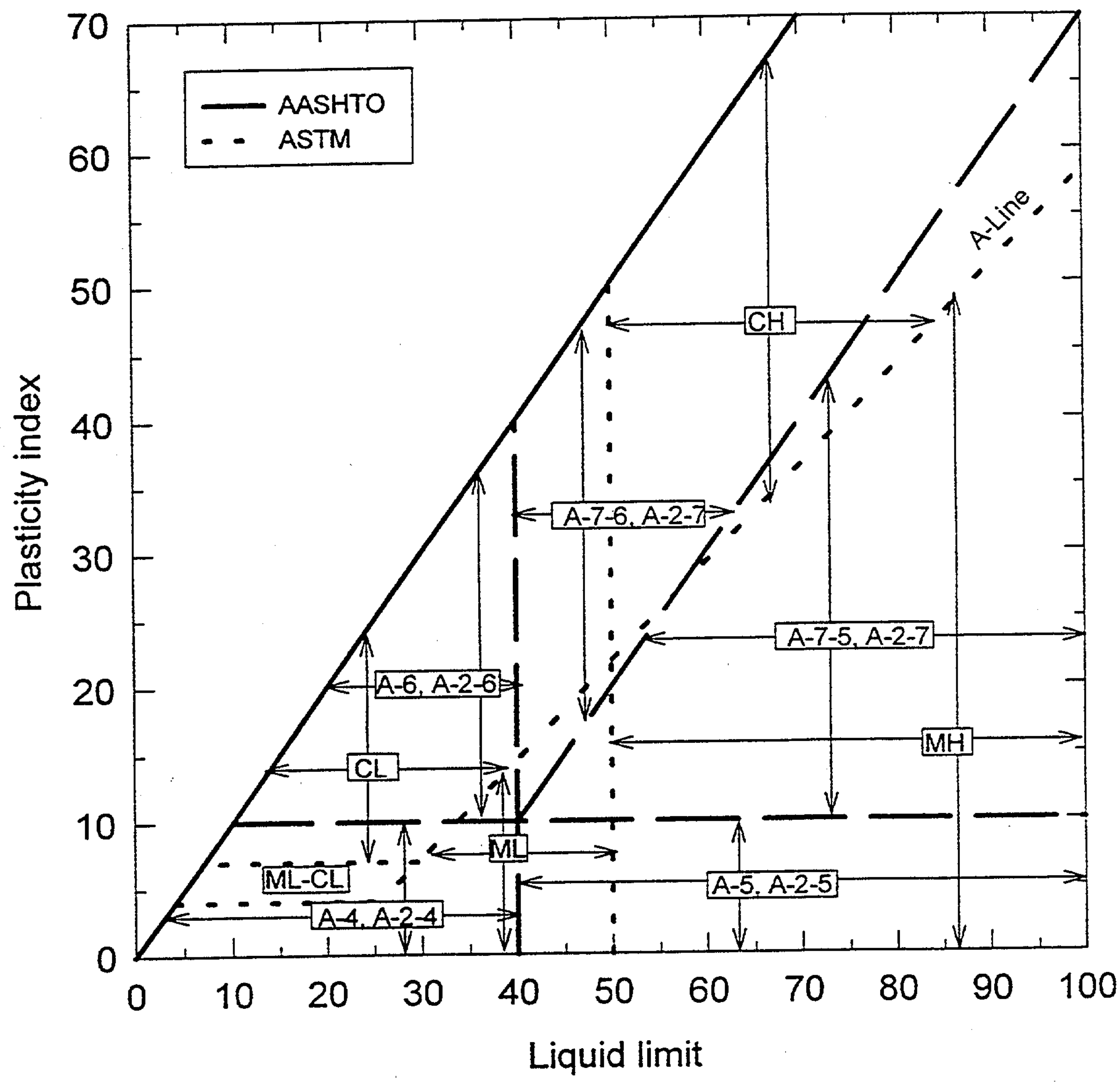


Figure 2.8 Comparison of Plasticity Charts for AASHTO and ASTM Classification

2.2.2 Compaction and Compactibility

Soils that are to be placed and compacted as part of engineered fills, such as pipe backfill, are also tested for moisture-density relationships due to compaction energy (AASHTO T 99 and T 180, and the equivalent ASTM D 698 and D 1557, called the standard and modified Proctor tests, respectively, herein). The density achieved during compaction of some coarse-grained soils with limited fines content (less than about 5 percent) is insensitive to moisture content. These soils are characterized using the relative density tests (ASTM D 4253 and D 4254).

A soil that achieves good stiffness characteristics with minimal compactive effort is said to be readily compactible. This generally applies to coarse grained materials such as A-1, A-2 and A-3 in the AASHTO system and GW, GP, SW, and SP in the USCS system. As grain size decreases and fines content increases the compactive effort required to achieve adequate stiffness increases and the maximum stiffness that can be achieved with compaction decreases. Selig (1988) demonstrated this in tests where the compactive effort was varied from 0 to 100 percent of the energy required by the standard Proctor test. McGrath (1990) developed this concept further to demonstrate the energy required to achieve a given level of soil stiffness (E') with various types of soil. Achieving an E' of 1000 psi with CL soil requires more than seven times the compactive energy of achieving the same E' with SW soil. This subject is explored more thoroughly in chapter 3.

2.2.3 Stiffness and Strength

Methods of modeling soil behavior for design of buried pipe vary from very simple procedures that assume linear, elastic soil behavior and do not consider strength, to very sophisticated models that consider true non-linear, stress-dependent soil behavior and strength parameters.

An example of a simple soil model is the above mentioned table of values for the modulus of soil reaction, E' (table 2.8), developed by Howard (1977) for use with the Iowa formula (Spangler, 1941). Howard's table divides soils into four principal groups and assigns values of E' as a function of the soil group and the density, which is expressed as function of the maximum density determined in a reference test, such as AASHTO T 99. The table makes a distinction, not made in the ASTM or AASHTO classification systems,

Table 2.8

Howard Design Values for Modulus of Soil Reaction, E' (Howard, 1977)

Soil type-pipe bedding material (Unified Classification System)[1]	E' for degree of compaction of bedding (lb/in.[2])			
	Dumped	Slight <85% Proctor <40% relative density	Moderate 85-95% Proctor 40-70% relative density	High >95% Proctor >70% relative density
Fine-grained soils (LL>50)[2] Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer; otherwise use E'=0			
Fine-grained soils (LL<50) Soils with medium to no plasticity, CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1000
Fine-grained soils (LL<50) Soils with medium to no plasticity, CL, ML, ML-CL, with more than 25% coarse-grained particles *Coarse-grained soils with fines* GM, GC, SM, SC[3] contains more than 12% fines	100	400	1000	2000
Coarse-grained soils with little or no fines GW, GP, SW, SP[3] contains less than 12% fines	200	1000	2000	3000
Crushed Rock	1000	3000		
Accuracy in terms of percent deflection[4]	±2%	±2%	± 1	± 0.5%

[1] ASTM Designation D 2487, USBR Designation E-3.
[2] LL = liquid limit.
[3] Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC).
[4] For ± 1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Note:
- A. Values applicable only for fills less than 50 ft.
- B. Table does not include any safety factor.
- C. For use in predicting initial deflections only, appropriate deflection lag factor must be applied for long-term deflections.
- D. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values.
- E. Percent Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/ft^3 (ASTM D-698, AASHTO T-99, USBR Designation E-11).

1 MPa = 145 psi, 1 kN-m/m^3 = 20.9 ft-lb/ft^3

between "crushed rock" and other granular soils. This table is widely cited in the literature. Other variations of this table have been proposed. The Water Research Centre (WRc) in the United Kingdom published table 2.9 (DeRosa et al., 1988). This is similar to the Howard table but distinguishes uniform gravel from single size gravel. The single size gravel is seen to have a higher initial stiffness prior to compaction while the graded gravel is able to achieve a higher stiffness after compaction.

Table 2.9

Water Research Centre Values for Modulus of Soil Reaction (DeRosa. et al., 1988)

EMBEDMENT MATERIAL		MODULUS OF SOIL REACTION (MN/m^2)				
DESCRIPTION	CASAGRANDE GROUP SYMBOL	UNCOMPACTED	80% Mp	85% Mp	90% Mp	95% Mp
Gravel – single size	GU	5	7	7	10	14
Gravel – graded	GW	3	5	7	10	20
Sand and coarse grained soil with less than 12% fines	GP SW SP	1	3	5	7	14
Coarse grained soil with more than 12% fines Fine grained soil (LL <50%) with medium to no plasticity and containing more than 25% coarse grained particles	GM GC SM CL, ML, mixtures ML/CL and ML/MH	-	1	3	5	10
Fine grained soil (LL <50%) with medium to no plasticity and containing less than 25% coarse grained particles	CL, ML, mixtures ML/CL, CL/CH and ML/MH	-	-	1	3	7
All values valid for semi-rigid pipe design. Data applies to cover depths in the range 0.9 to 10.0m.						

Range of E' values recommended for flexible pipe design.

Note: 1 MN/m^2 = 145 psi

An example of a sophisticated soil model for use in buried pipe design is the hyperbolic model (Duncan et al., 1980), which is used in most finite element models for analysis of buried pipe. The hyperbolic model uses nine separate parameters to completely define the stress-strain behavior of soil, including both strength and stiffness parameters. The Duncan model used a power law rule to model the bulk modulus which represents the volumetric behavior of soil. Selig (1988) found a hyperbolic model for the bulk modulus could more accurately represent the volumetric behavior and presented a set of parameters that were used to develop the soil groupings for the SIDD installations. Selig (1990) later proposed an alternative set of properties for the hyperbolic bulk modulus model that he recommended for use with flexible pipe.

2.2.4 Controlled Low Strength Material

Controlled Low Strength Material, or CLSM, also known as flowable fill, is a special material manufactured to have good flow characteristics. Typical mix designs use cement sand, fly ash, and water; however, the cement content is on the order of 30 to 60 kg/m^3 (50 to 100 lbs/yd^3), extremely low relative to structural concrete mixes. The fly ash is the key ingredient to create the good flow characteristics. An alternative to fly ash is to use high quantities of air. Twenty to thirty percent air content, with reduced or no fly ash, has also been found to produce mixes with good flow characteristics (Grace, 1996). Applications of CLSM have been discussed by Howard (1996) and Brewer (1993).

CLSM gains strength and stiffness over time. McGrath and Hoopes (1997) published recommended hyperbolic soil model properties and design values of bedding factors and E' values at ages of 16 hours, 7 days, and 28 days for CLSM mixes with high air contents. The values were based on triaxial and one-dimensional compression testing, and finite element analysis. The mix designs used in that study are presented in table 2.10. The proposed soil properties are presented in tables 2.11, 2.12, and 2.13.

Table 2.10

CLSM Test Program Variables (McGrath and Hoopes, 1997)

Parameter	Conditions
CLSM Mix 1	cement: 59 kg/m^3, Type 1; sand: 1480 kg/m^3; air: 25-30%
CLSM Mix 2	cement: 30 kg/m^3, Type 1; fly ash: 150 kg/m^3; sand: 1480 kg/m^3; air: 27%
Age at test	16 hours, 7 days, 28 days
Triaxial confining stress	20, 40, and 60 kPa (3, 6, and 9 psi)

Table 2.11

Hyperbolic Soil Model Parameters for Air-Modified CLSM (McGrath and Hoopes, 1997)

Parameter Symbol	Value		
	16 hours	7 days	28 days
K	630	800	1000
n	0.8	0.75	0.65
R_f	0.86	0.6	0.55
C, kPa (psi)	0 (0)	28 (4)	42 (6)
ϕ, deg	38	38	38
$\Delta\phi$, deg.(Note 1)	0	0	0
B_i/Pa	19	40	450
ϵ_u	0.17	0.15	0.09

Notes
1. The term $\Delta\phi$ accounts for the non-linear Mohr-Coulomb failure envelope observed in many soils. The scope of the testing program was not sufficient to determine the shape of the envelope for CLSM, thus it is assumed to be linear by setting $\Delta\phi$=0.

Table 2.12

Rigid Pipe Bedding Factors for Air-Modified CLSM (McGrath and Hoopes, 1997)

Age	Installation Type	
	Trench	Embank.
16 hours	1.8	2.5 to 2.8
7 days	2	3.0 to 3.4
28 days	2.5	4.0 to 4.8

Table 2.13

Modulus of Soil Reaction Values for CLSM, MPa (psi)

Mix	Age		
	16 hours	7 days	28 days
Air-modified CLSM	7 (1,000)	14 (2,000)	21 (3,000)

2.3 Influence, Properties, and Modeling of Pre-existing Soil

For pipes installed in trenches, the stiffness and strength properties of the in situ soils that form the trench bottom and trench wall can influence the pipe behavior. Characterizing these materials has posed a significant problem for designers, as the variability of in situ soils is immense. In addition to the variability in particle size and plasticity described by the soil classification systems, natural soils have highly variable moisture contents, tend to change stiffness with age, and may range in stiffness from wet runny conditions to solid rock. Unlike backfill soils, which can be selected for a project, the designer must accept the natural soils as a part of the design. From a structural point of view, it is often desirable to use wide trenches to isolate a pipe from poor natural soils; however, the increase in excavation and backfill costs can be significant and the question of how wide a trench must be is important.

AWWA Manual M45, The Fiberglass Pipe Design Manual (1996) has attempted to provide guidance on soil stiffness for in situ soils based on the unconfined compressive strength and the standard penetration test (commonly called blow counts). Table 2.14 provides suggested modulus values ranging from 350 kPa to 138 MPa (50 to 20,000 psi).

Table 2.14

AWWA Manual M45 Values for Modulus of Soil Reaction of In Situ Soils

Native in Situ Soils*				
Granular		Cohesive		E'_n (psi)
Blows/ft †	Description	q_u(Tons/sf)	Description	
>0-1	very, very loose	>0-0.125	very, very soft	50
1-2	very loose	0.125-0.25	very soft	200
2-4		0.25-0.50	soft	700
4-8	loose	0.50-1.0	medium	1,500
8-15	slightly compact	1.0-2.0	stiff	3,000
15-30	compact	2.0-4.0	very stiff	5,000
30-50	dense	4.0-6.0	hard	10,000
>50	very dense	>6.0	very hard	20,000

* The modulus of soil reaction E'_n for rock is ≥ 50,000 psi.
† Standard penetration test per ASTM D1586.

For embankment installation $E'_b = E'_n = E'$

Note: 1 m = 3.28 ft, 1 kN/m² = 0.010 tons/sq. ft, 1 MPa = 145 psi

Evaluating in situ soils in simplified design methods generally requires that the soil stiffness at the side of a pipe be represented by a single modulus value, which is a result of the composite behavior of the trench backfill and the natural soil. Very little work has been done on this issue. Leonhardt (1979) used the layered elastic theory to develop a simplified method to compute an "effective" E' value based on the relative value of the stiffness of the in situ and backfill soils and the trench width, expressed as a ratio of the width to the outside diameter of the pipe. The expression is:

$$E'_{design} = \zeta E'_b \,, \tag{2.9}$$

where

E'_{design} = value of E' used in Iowa formula, MPa, psi,

ζ = Leonhardt factor, and

E'_b = value of E' for backfill.

The Leonhardt factor is computed as:

$$\zeta = \frac{1.662 + 0.639\left(\dfrac{B_d}{D_o} - 1\right)}{\left(\dfrac{B_d}{D_o} - 1\right) + \left[1.662 + 0.361\left(\dfrac{B_d}{D_o} - 1\right)\right]\dfrac{E'_b}{E'_n}} \,, \tag{2.10}$$

where

B_d = trench width, m, ft,

D_o = pipe outside diameter, m, ft, and

E'_n = value of E' for in situ material.

The Leonhardt approach is thought to be conservative. AWWA Manual M45 presents a table of slightly less conservative values.

In computer analyses, in situ soils are often treated as exhibiting linear elastic behavior. This usually produces acceptable accuracy, because the imposed stresses are often not greater than the previous maximum stress experienced by the soil mass and because the in situ soil is separated from the pipe by the trench backfill and therefore has less impact on the behavior. Designers should be aware of instances where these two conditions do not exist and may wish to investigate more sophisticated assumptions.

2.4 Pipe-Soil Interaction Software

A number of finite element method (FEM) computer programs have been written specifically for the analysis of buried pipe problems, among these are CANDE (Katona, 1976, and Musser et al. 1989), and SPIDA (Heger et al. 1985). These programs are considered representative of the types of features that are available in other programs.

CANDE was developed under contract from the Federal Highway Administration. It was originally written for main frame computers but has since been modified to run on personal computers (Musser et al. 1989). It considers all types of pipe materials, including both rigid and flexible pipes. Several elastic soil models are available, including linear elastic, overburden dependent, and hyperbolic. CANDE has three solution levels. Level 1 does not utilize finite elements. It is an implementation of the elastic plate solution developed by Burns and Richard (1964). Level 2 is a finite element solution with a predefined mesh. The automated mesh assumes symmetry about the centerline of the pipe and models only half of the structure using ten bending elements, each 15 degrees long. Level 3 is a fully user defined finite element solution. CANDE is publicly available.

The Burns and Richard solution has received a great deal of attention as a simplified design method that is based on a theoretically sound development and can address the entire range of pipe stiffnesses. It is a closed form solution for an elastic circular ring embedded in an infinite homogenous, elastic, isotropic medium. The theory describes the pipe in terms of the hoop (axial) stiffness:

$$PS_H = \frac{EA}{R}, \tag{2.11}$$

where

PS_H = Pipe hoop stiffness, MN/m^2, psi,
E = Pipe material modulus of elasticity, MPa, psi,
A = Pipe wall area per unit length, mm^2/mm, $in.^2/in.$, and
R = Centroidal radius of pipe, mm, in.

and the pipe bending stiffness, which is defined here in terms of standard U.S. practice as the stiffness in the parallel plate test:

$$PS_B = \frac{EI}{0.149\,R^3}, \tag{2.12}$$

where

PS_B = Pipe bending stiffness, MN/m/m, lbs/in./in., and

I = Moment of inertia of pipe wall, mm^4/mm, $in.^4/in.$.

The pipe stiffness are combined with the soil stiffness, using the constrained modulus, M_s, to define the overall pipe-soil system stiffnesses, which are the hoop stiffness parameter, S_H:

$$S_H = \frac{M_s R}{E A} , \tag{2.13}$$

and the bending stiffness parameter, S_B:

$$S_B = \frac{M_s R^3}{E I} . \tag{2.14}$$

These parameters are very useful in understanding behavior, as will be discussed in later sections.

SPIDA was developed jointly by Simpson Gumpertz & Heger Inc. and the University of Massachusetts under contract from the American Concrete Pipe Association. It assumes symmetry about the centerline of the pipe using 17 bending elements varying in arc length from 7.5 degrees near the crown and invert, to 10 degrees near the springline, to 15 degrees at 45 degrees from the crown and invert. SPIDA uses an automatic mesh generator that can define trench and embankment installations. For installations that fall within the limits of the mesh generator it is easier to use than CANDE, but it does not have an option with the versatility of CANDE Level 3. The soil options in SPIDA are linear elastic and hyperbolic. SPIDA is a proprietary program, owned by the ACPA.

CANDE and SPIDA both allow modeling soil behavior using the Duncan hyperbolic Young's modulus soil model with the Selig hyperbolic bulk modulus. This is an elastic model that incorporates non-linear behavior as a function of the soil strength parameters. Properties for use in this model have been developed from tests on previously compacted soil. It is an elastic model.

CHAPTER 3
CHARACTERIZATION OF BACKFILL MATERIALS

Current practice in characterizing backfill materials focuses on soil classification and compaction characteristics. This was discussed in chapter 2 but, also noted, was the fact that the properties of interest for pipe backfill are stiffness and strength. A program of characterizing backfill materials by both the classical tests and other tests that may be more revealing about stiffness and strength properties was undertaken to explore changes to practice that might allow a more direct correlation between the measured properties and the desired properties.

A second effort in correlating backfill properties is to relate the more sophisticated soil models used in finite element analysis of buried pipe to the simplified properties used in hand calculations. The hyperbolic models of Duncan (1980) and Selig (1988) are complicated and require significant testing to develop the data necessary to characterize a soil, while the modulus of soil resistance values of Howard (1977) are readily determined and applied but empirical in nature and have not been successfully correlated to true soil properties. The relationship between the modulus of soil reaction and the hyperbolic soil model is explored.

3.1 Materials Tested

A total of 12 processed backfill materials and naturally occurring soils were collected for testing (for simplicity they will all be called "soils" below). The soil gradations, classifications and common names by which they are sold are listed in Table 3.1. They are described as follows:

- Soils 1 to 3 are angular crushed stone with widely varying gradations. All three soils were crushed from the same material, a local deposit called trap rock with a specific gravity of 2.9.
- Soil 4 is a uniform rounded stone.
- Soils 5 and 8 are rounded and subrounded sands. Soil 5 is manufactured as fine concrete aggregate and Soil 8 for use on roads in winter.

Table 3.1
Soil Gradation Characteristics and ASTM and AASHTO Classifications

Soil No.	Common name	D_{60}	D_{30}	D_{10}	C_u	C_c	Gradation (% passing)				ASTM D 2487	AASHTO
							#4	#10	#40	#200		
1	gravel trap rock	9.10	7.50	5.80	1.57	1.07	2	<1	<1	<1	GP - poorly graded gravel	A-1-a
2	sand trap rock	1.05	0.34	0.09	11.67	1.22	100	85	35	8	SW-SM - well graded sand with silt	A-1-b
3	shoulder stone	4.80 (3.30)	1.60 (1.30)	0.20 (0.20)	24.00 (11.00)	2.67 (1.71)	59 (72)	35 (44)	13 (12)	3 (4)	SW - well graded sand with gravel	A-1-a
4	pea gravel	8.90	7.00	5.20	1.71	1.06	8	1	<1	<1	GP - poorly graded gravel	A-1-a
5	concrete sand	0.69	0.34	0.20	3.45	0.84	97	89	39	2	SP - poorly graded sand	A-1-b
6	rewash	0.10	0.07	0.06	1.72	0.90	100	100	100	23-33	SM - silty sand	A-2-4
7	glacial till	2.80	1.10	0.30	9.33	1.44	71	51	8	<1	SW - well graded sand with gravel	A-1-b
8	winter sand	0.92	0.47	0.26	3.54	0.92	94	82	25	2	SP - poorly graded sand	A-1-b
9	top clay									90	CL - lean clay	A-6
10	varved clay									93	CL - lean clay	A-6
11	red sandstone	1.30	0.55	0.27	4.81	0.86	92	75	21	2	SP - poorly graded sand	A-1-b
12	native sand	0.76	0.27	0.08	9.50	1.20	92	85	43	9	SW-SM well graded sand w/ silt	A-1-b

Note: Two sieve analyses were made for Soil Nos. 3 and 6. Both analyses are reported for Soil No. 3. For Soil No. 6 only the percent finer than the No. 200 sieve varied significantly and is reported.

CBR is not a good indicator of unit weight for these soils in the range of 90 to 100 percent of maximum standard Proctor density.

Moisture-density relations and moisture-penetration resistance relations for Soil Nos. 6 and 8 to 12 are shown in figs. 3.6 and 3.7. Fig. 3.7 suggests that a relationship exists between moisture content and penetration resistance, and also between density and penetration resistance for the soils with more than 7 percent fines (Nos. 6, 9, 10, and 12). The penetration resistance varies almost 100 percent as the density varies between 90 and 100 percent of maximum standard Proctor density. The results for the two sands without fines (Nos. 8 and 11) show no correlation.

Together, figs. 3.4 to 3.7 indicate that relationships between penetration resistance (or CBR) could be established for soils with more than a few percent fines; however, the data in fig. 3.7 also show a strong relationship to moisture content, which may be the dominant variable.

Normalized results of the variable compactive effort tests are shown for individual soils in fig. 3.8 and for all data in fig. 3.9. Where the moisture content does not vary, a relation between CBR and density is present, as both parameters show an increase for compaction energy up to 100 percent of standard Proctor effort. Only Soil No. 5 shows a clear trend of continued increase in density as the compactive energy further increases from the standard effort to the modified effort; however, the data shows scatter. None of the soils show an increase in CBR over the range of standard to modified range of compactive energy. This lack of increase for compactive energies greater than the standard effort could have been anticipated as all of the tests were conducted at optimum moisture content determined from the standard test. Had the test been conducted at a lower moisture content a trend of increasing density and CBR value may have been evident over this range.

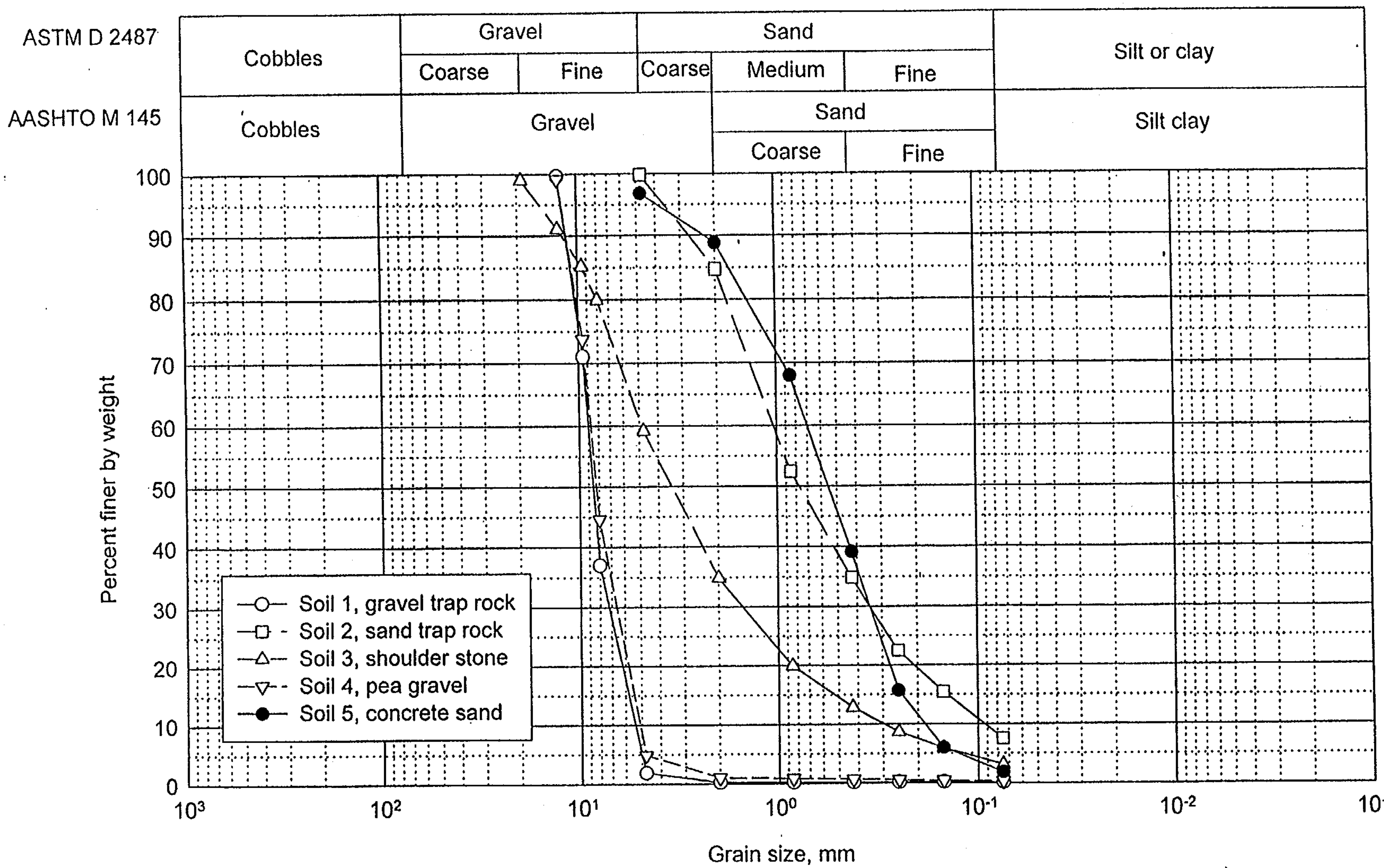


Figure 3.1 Grain Size Distribution for Soil Nos. 1 to 5

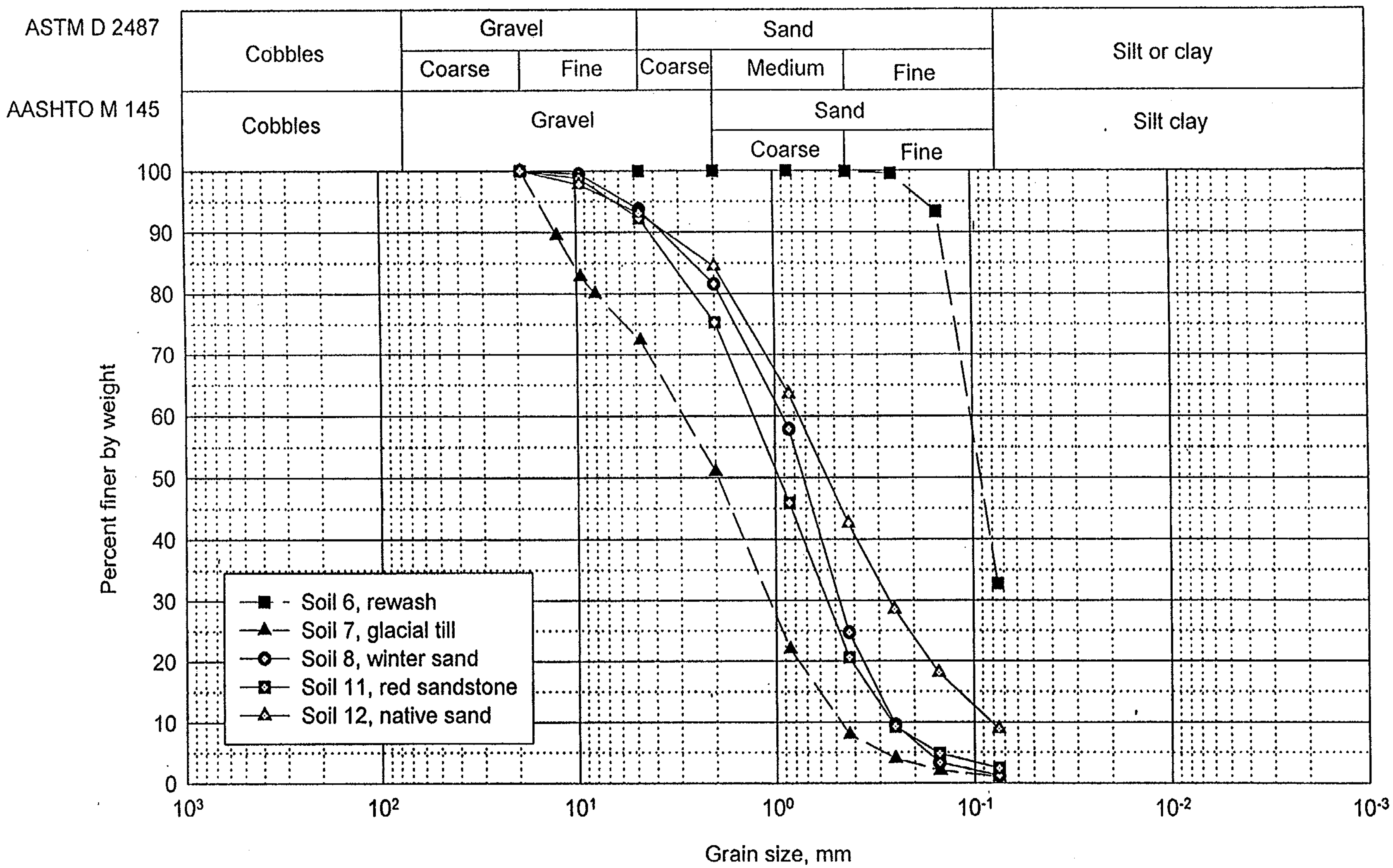


Figure 3.2 Grain Size Distribution for Soil Nos. 6 to 12

3.2 Characterization Tests

The tests for characterizing backfill materials included the traditional compaction tests as well as a number of tests that are not typically considered for pipe installation. These include the moisture-density relations using standard Proctor effort, California Bearing Ratio test, compaction tests conducted with variable effort, one-dimensional compression tests, and penetration tests. The CLSM material was tested for unconfined compression strength.

3.2.1 Compaction Characterization

Compaction characteristics of the test soils were determined in accordance with the standard Proctor test (AASHTO T 99, ASTM D 698). The Proctor tests were all conducted in 150 mm (6 in.) diameter molds suitable for conducting CBR tests (see section 3.3) after compaction. New soil was used for each test. Soils 1 to 6 were also characterized by relative density tests (ASTM D 4253 and D 4254). The maximum index density test was conducted on a cam driven vertically vibrating table using dry soil (Method 2A).

3.2.2 Variable Compactive Effort

After determination of maximum dry density and optimum water contents, compaction tests using variable levels of effort were conducted to determine the relationship of dry density to compactive effort. These tests were conducted on Soil Nos. 1 to 6. New soil was used for each test. All tests were conducted at near optimum water content as determined from the standard effort test and in 150 mm (6 in.) diameter molds with a mold volume of 0.0021 m^3 (0.075 ft^3). Compactive energy varied from none to the modified test energy, 2,700 kN-m/m^3 (56,000 ft-lb/ft^3), as summarized in table 3.3. CBR tests were conducted after completion of the compaction tests (See section 3.2.3).

Table 3.3
Parameters for Variable Compactive Effort Tests

Energy level	Weight (N)	Height of drop (m)	Blows per layer	Layers	Energy (kN-m/m^3)
Loose	0	0	0	1	0
0.25 * Std Proctor	24.5	0.305	14	3	150
0.50 * Std Proctor	24.5	0.305	28	3	300
0.75 * Std Proctor	24.5	0.305	42	3	440
1.00 * Std Proctor	24.5	0.305	56	3	590
2.19 * Std Proctor	44.8	0.457	27	5	1300
3.38 * Std Proctor	44.8	0.457	42	5	2000
4.58 * Std Proctor (Mod. Proctor)	44.8	0.457	56	5	2700

3.2.3 California Bearing Ratio

Soils 1 to 6 were tested by the California Bearing Ratio (CBR) test, AASHTO T 193 (ASTM D 1883). The test was conducted on specimens as compacted, without soaking, and with a 76.5 N (17.2 lb) surcharge (0.6 psi). The CBR was computed for a penetration depth of 5 mm (0.2 in.).

3.2.4 Penetration Tests

Soil Nos. 6 and 8 to 12 were also tested for penetration resistance in accordance with ASTM D 1558. The size of the penetrometer tip varied as a function of the density and soil type. The penetration force was read at a penetration depth of 50 mm (2 in.). The penetration test is similar to the CBR, except that the load is applied to a smaller bearing area with less control and there is no confining surcharge.

3.2.5 Results of Characterization Tests

Results of the standard Proctor compaction tests are given in table 3.4. The values are reported as unit weights (kN/m^3) rather than density (kg/m^3) to simplify calculation of loads and stress which are computed as force per unit length (kN/m) and force per unit area (kN/m^2), respectively. Table 3.4 also presents the results of the relative density tests in terms of the percentage of maximum standard Proctor density that was achieved and the loose density when soil was placed in the Proctor mold at optimum moisture content with no compaction. The data for Soils 1 to 6 is presented graphically in Fig. 3.3. This figure shows that the soils with less than 1 percent fines, whether dry or wet, are at 80 percent or more of maximum standard Proctor density when placed loose with no compactive effort. For the pea gravel in particular, which is uniformly graded and rounded, the soil is at 85 to 90 percent density when loosely placed. As the fines content increases, the loose density decreases. This demonstrates that, as the fines content increases, the loose density decreases which in turn increases the importance of applying proper compactive effort. Note also that the minimum relative density is not necessarily a lower bound for loosely placed soils. When moisture is added the soil can bulk, resulting in a lower density. In the case of Soil 6, the bulking is substantial, resulting in a loose density of about 55 percent of maximum standard Proctor density.

Table 3.4

Comparison of Relative Density and Standard Proctor Test Results

Soil No.	Common name	AASHTO T 99 max. unit weight, kN/m^3 (lb/ft^3)	AASHTO T 99 Optimum moisture (%)	Maximum relative density	Minimum relative density	Placed loose at optimum moisture
				% of maximum standard Proctor density		
1	gravel trap rock	16.6 (106)	2	97	81	83
2	sand trap rock	20.3(129)	12	96	75	58
3	shoulder stone	22.0(140)	9	94	70	71
4	pea gravel	16.9(108)	1	97	85	91
5	concrete sand	17.9(114)	10	107	86	70
6	rewash	15.0(96)	22 20	104	76	54
7	glacial till					
8	winter sand	17.6(112)	10			
9	top clay	17.1(109)	20			
10	varved clay	15.9(101)	22			
11	red sandstone	19.0(121)	12			
12	native sand	19.8(126)	9			

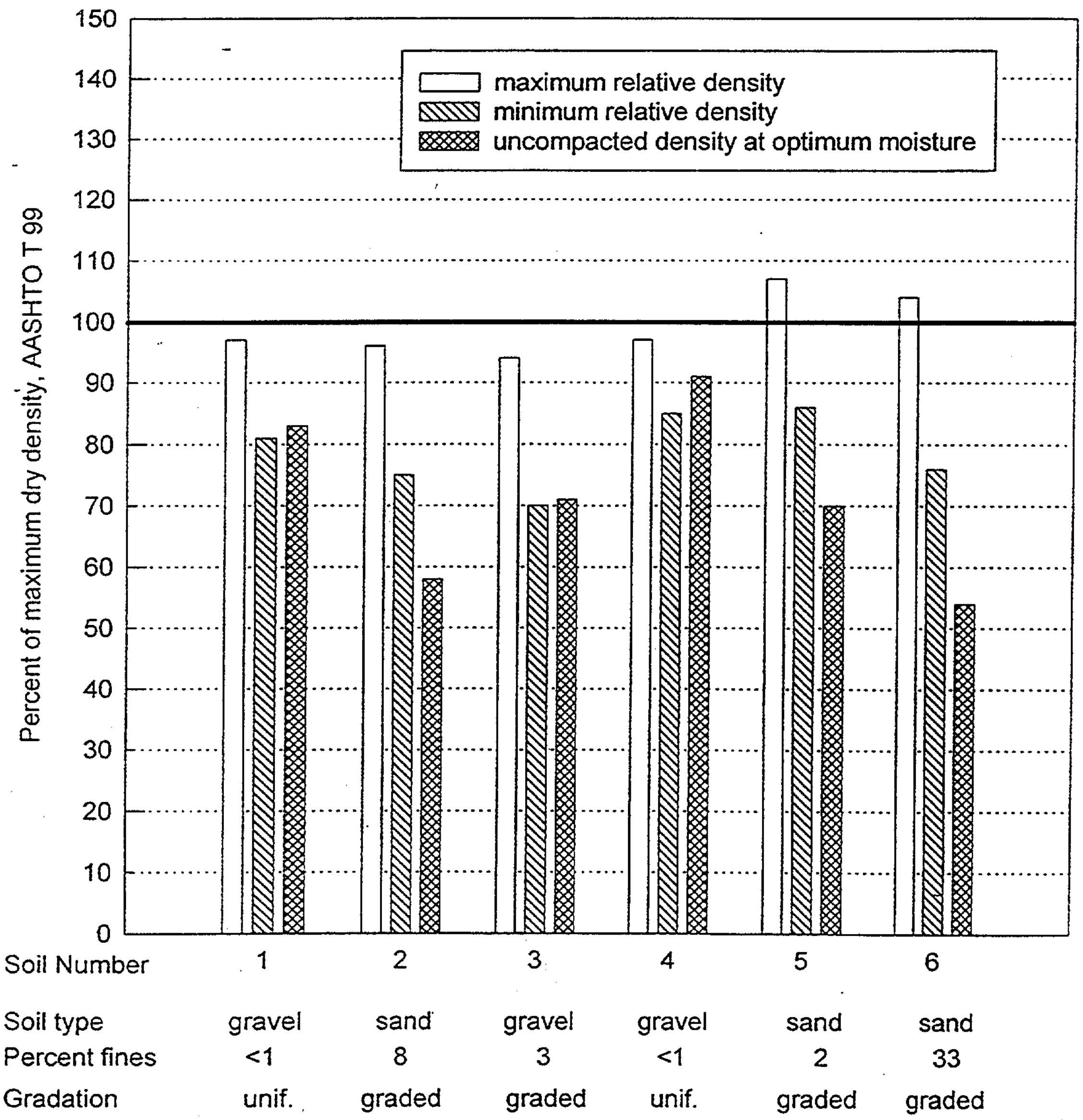


Figure 3.3 Loose and Compacted Density of Backfill Soils

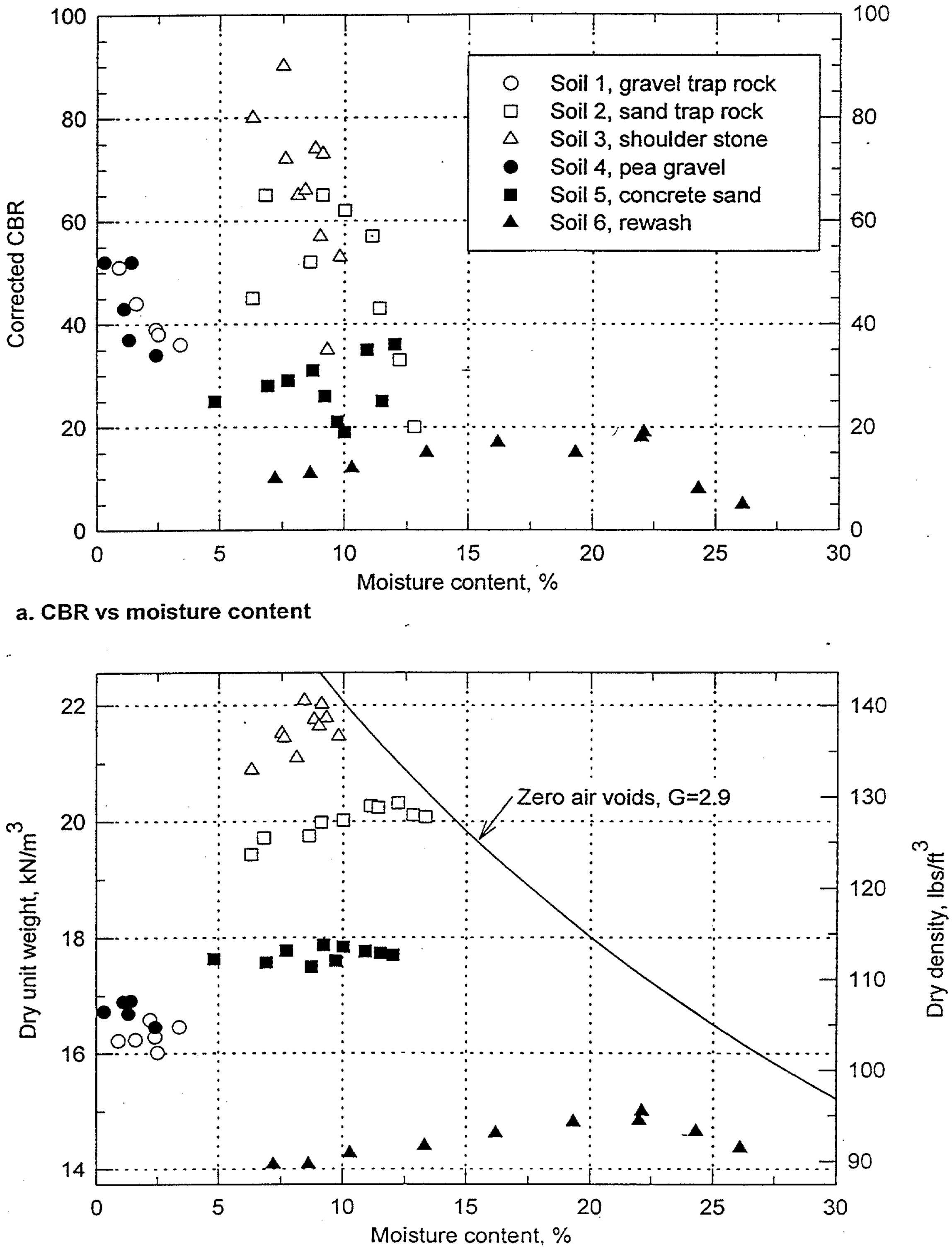
Moisture-density and moisture-CBR relations for Soil Nos. 1 to 6 are presented in fig. 3.4. Soil No. 6, with 30 percent fines, shows the classical moisture-density relation, while the other soils, with few fines, have a much less distinct, or no relationship between moisture content and unit weight (fig. 3.4b). The CBRs show a trend of increasing at a modest rate until the moisture content nears optimum and then dropping rapidly (fig 3.4a). Fig. 3.5 shows the same data but with the CBR on the x-axis, and all parameters normalized based on the value at 100 percent standard Proctor unit weight. The figure suggests that the

CBR is not a good indicator of unit weight for these soils in the range of 90 to 100 percent of maximum standard Proctor density.

Moisture-density relations and moisture-penetration resistance relations for Soil Nos. 6 and 8 to 12 are shown in figs. 3.6 and 3.7. Fig. 3.7 suggests that a relationship exists between moisture content and penetration resistance, and also between density and penetration resistance for the soils with more than 7 percent fines (Nos. 6, 9, 10, and 12). The penetration resistance varies almost 100 percent as the density varies between 90 and 100 percent of maximum standard Proctor density. The results for the two sands without fines (Nos. 8 and 11) show no correlation.

Together, figs. 3.4 to 3.7 indicate that relationships between penetration resistance (or CBR) could be established for soils with more than a few percent fines; however, the data in fig. 3.7 also show a strong relationship to moisture content, which may be the dominant variable.

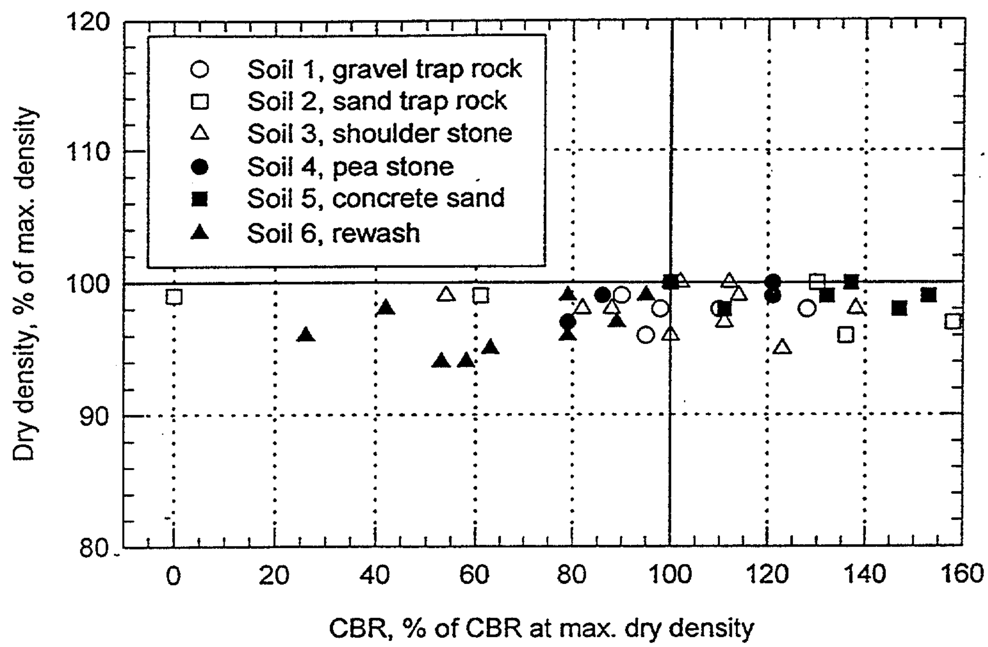
Normalized results of the variable compactive effort tests are shown for individual soils in fig. 3.8 and for all data in fig. 3.9. Where the moisture content does not vary, a relation between CBR and density is present, as both parameters show an increase for compaction energy up to 100 percent of standard Proctor effort. Only Soil No. 5 shows a clear trend of continued increase in density as the compactive energy further increases from the standard effort to the modified effort; however, the data shows scatter. None of the soils show an increase in CBR over the range of standard to modified range of compactive energy. This lack of increase for compactive energies greater than the standard effort could have been anticipated as all of the tests were conducted at optimum moisture content determined from the standard test. Had the test been conducted at a lower moisture content a trend of increasing density and CBR value may have been evident over this range.



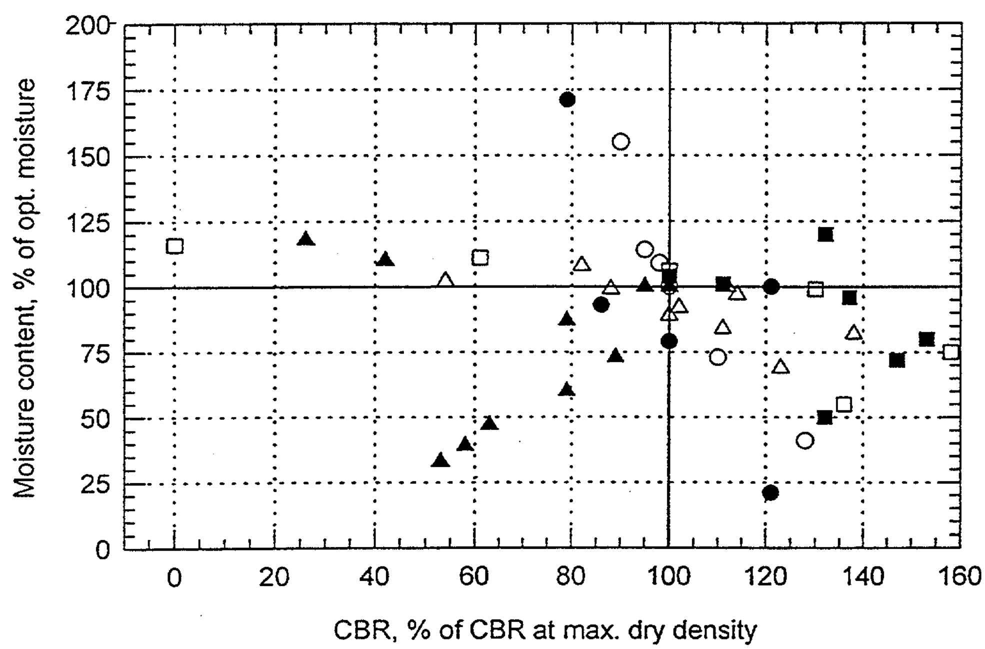
a. CBR vs moisture content

b. Dry density vs moisture content

Figure 3.4 Moisture Content Versus Standard Effort Unit Weight and CBR

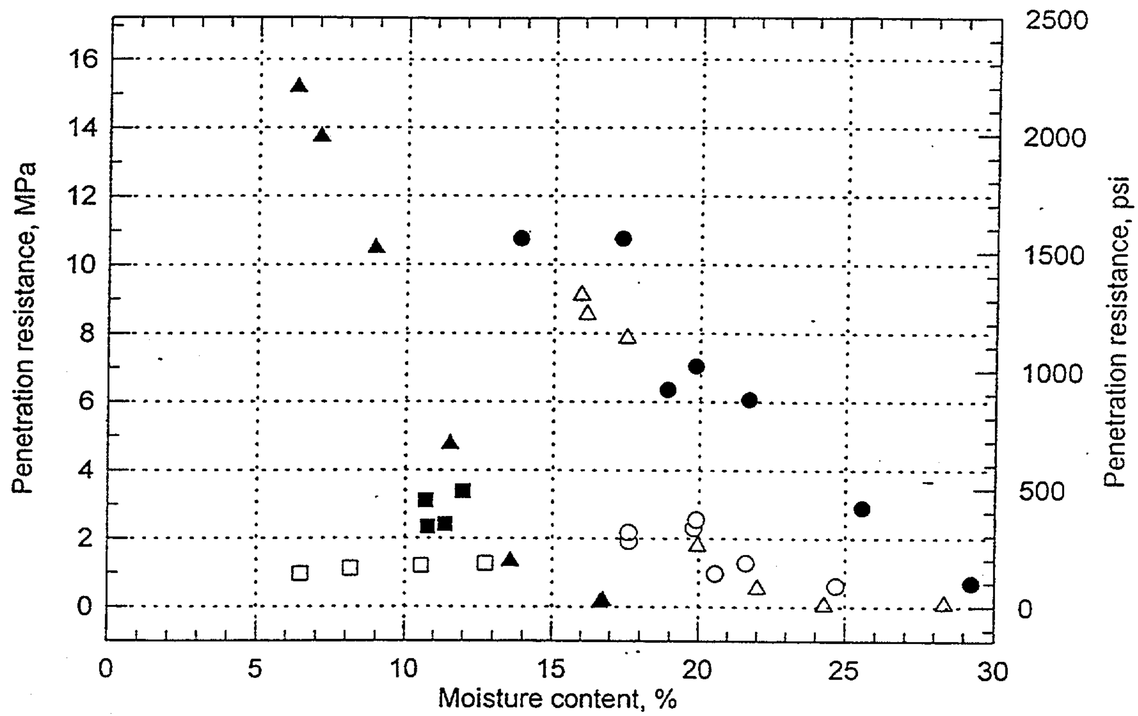


a. Dry density vs CBR

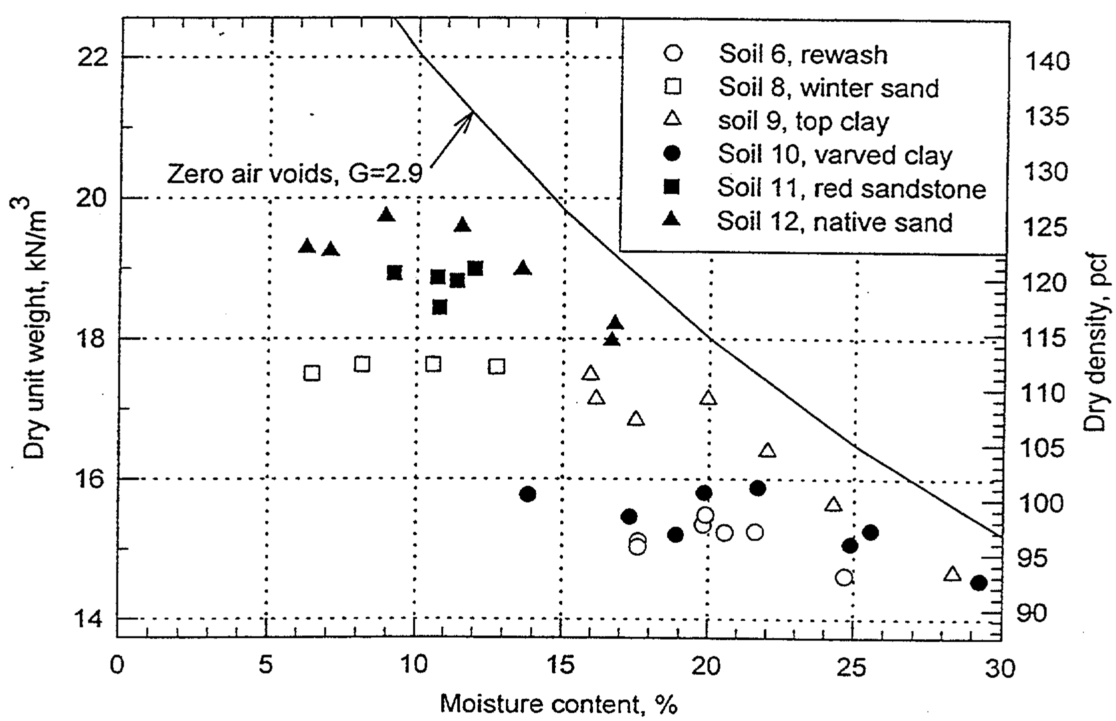


b. Moisture content vs CBR

Figure 3.5 CBR Versus Standard Effort Unit Weight and Moisture Content

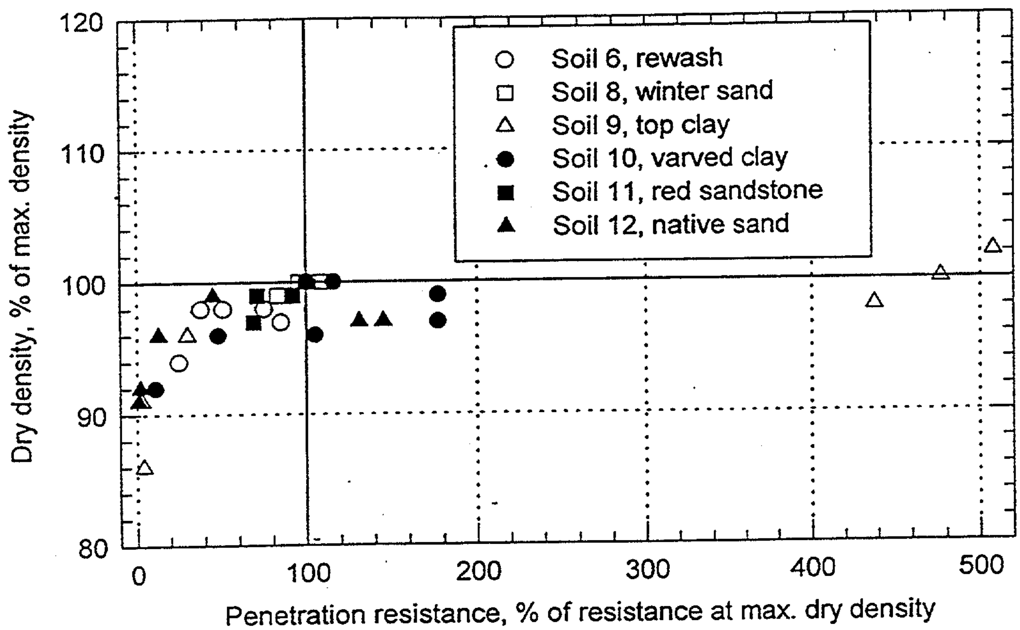


a. Proctor needle penetration resistance vs moisture content

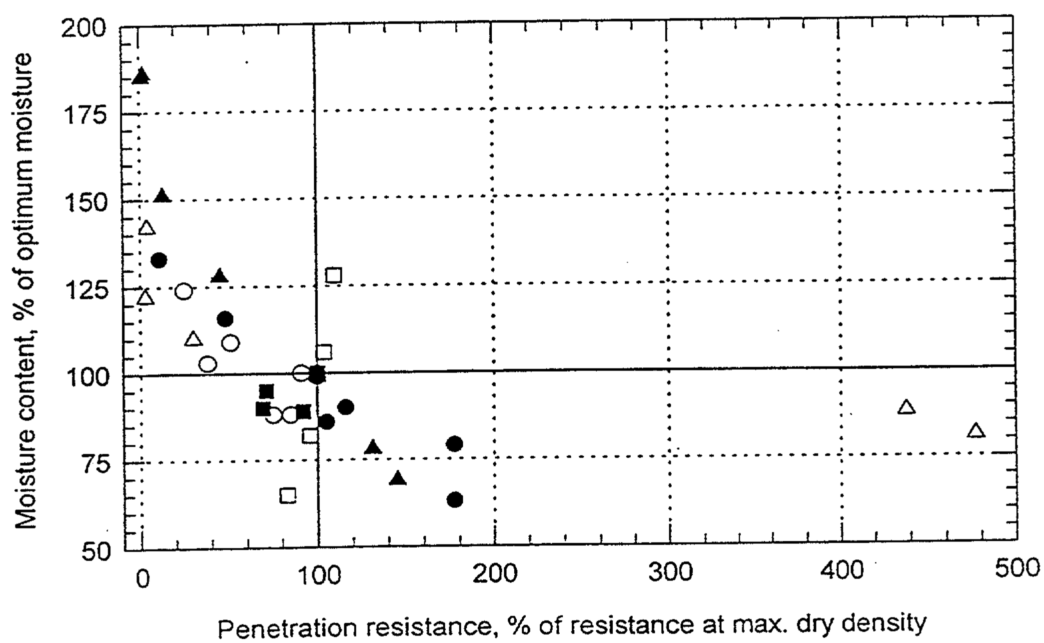


b. Dry density vs moisture content

Figure 3.6 Moisture Content Versus Standard Effort Unit Weight and Penetration Resistance



a. Proctor needle penetration resistance versus dry density



b. Proctor needle penetration resistance versus moisture content

Figure 3.7 Penetration Resistance Versus Moisture Content and Standard Effort Unit Weight

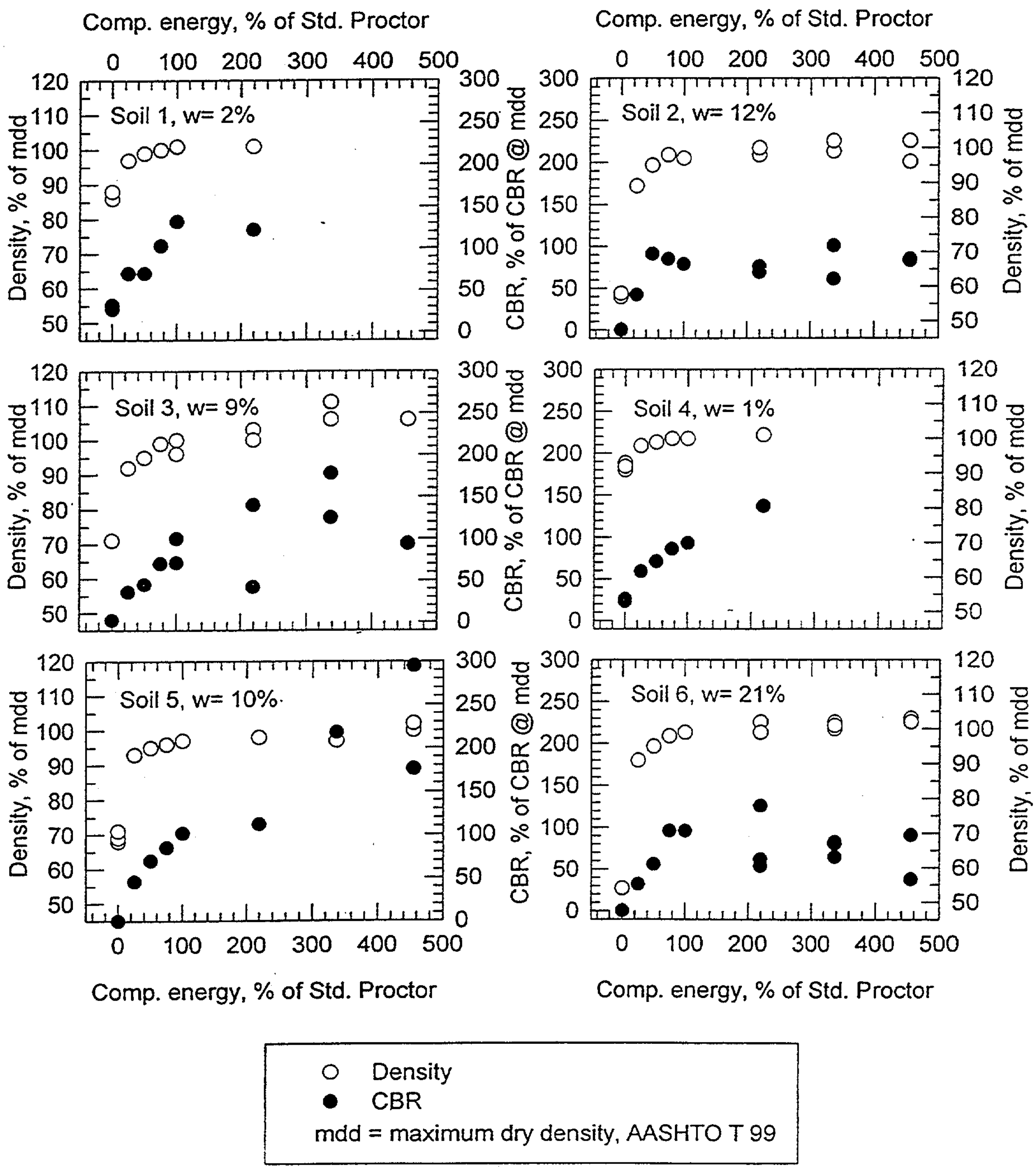


Figure 3.8 Variable Effort Compaction and CBR Conducted at Optimum Moisture Content

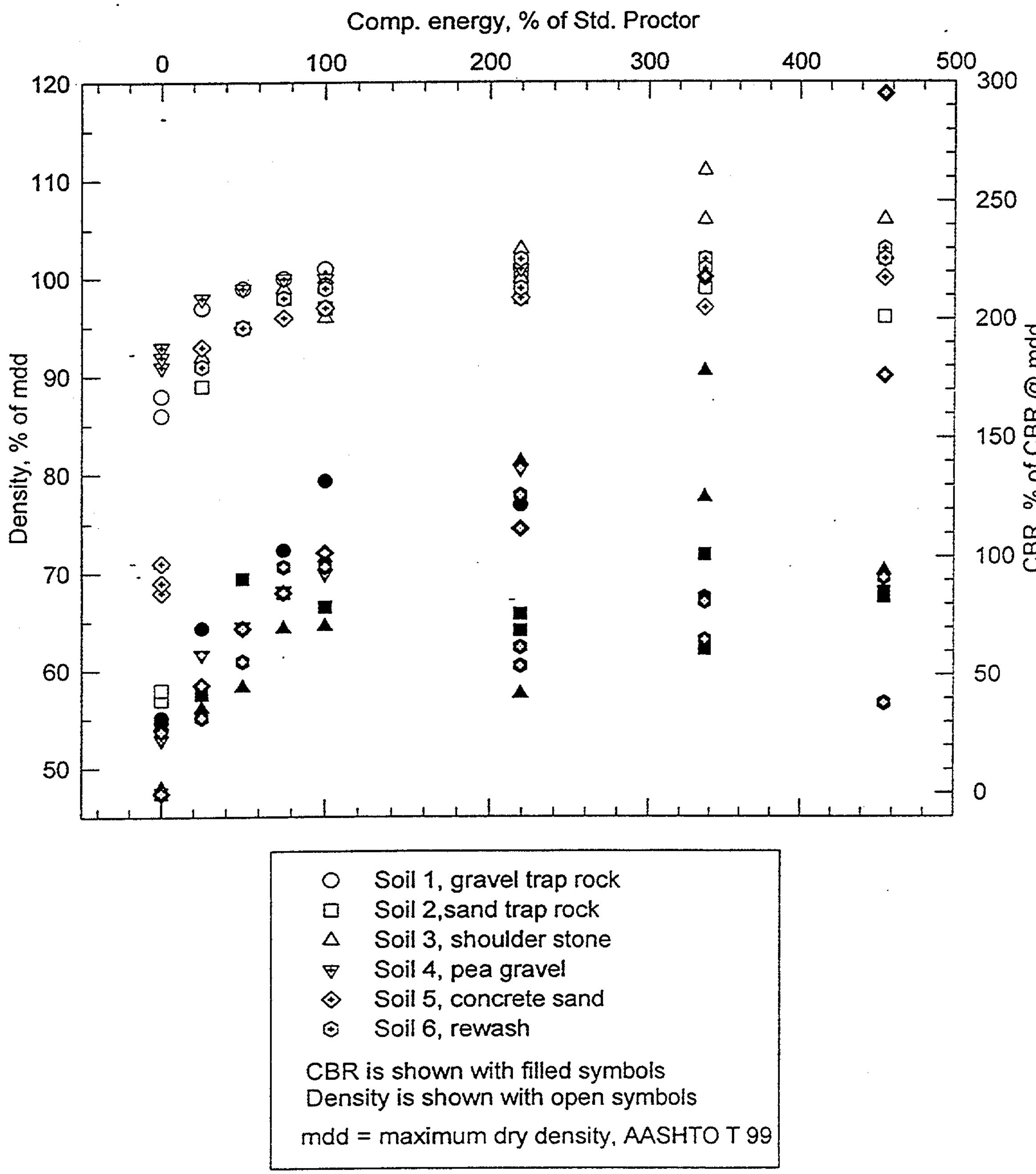


Figure 3.9 Normalized Variable Effort Compaction and CBR Test Results at Optimum Moisture Content

3.3 One-Dimensional Compression Tests

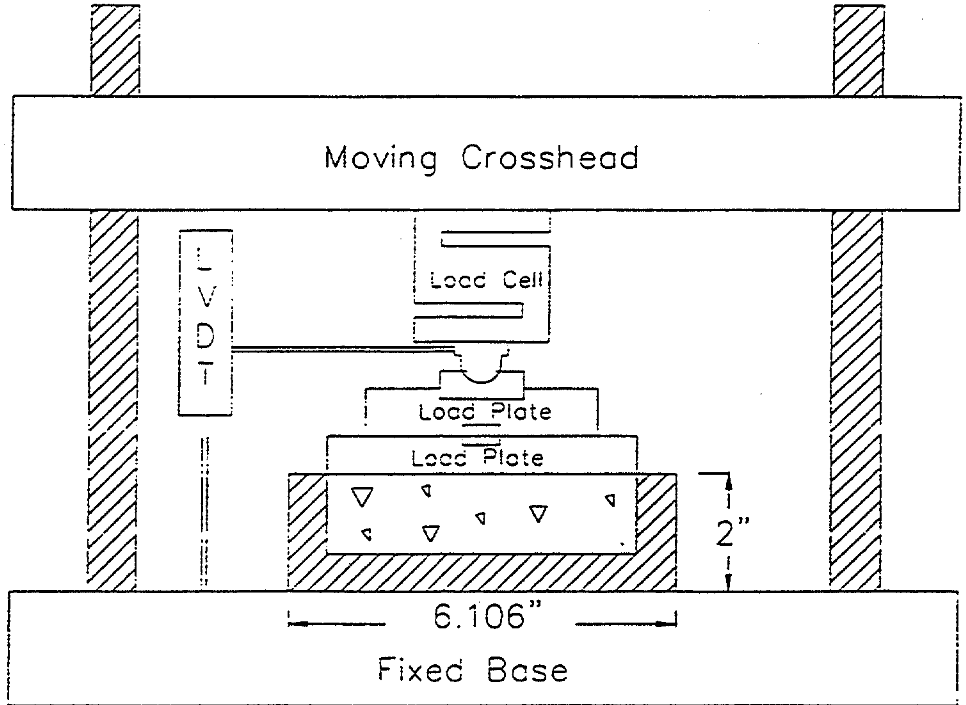
The variability of backfill materials and the lack of quality control on construction projects generally leads designers to accepting "standard" properties for soils, such as the hyperbolic properties of Duncan(1980) and Selig (1988, 1990) used in finite element analyses and the modulus of soil reaction values developed by Howard (1977). For some projects, however, it is desirable to conduct tests on actual backfill materials to determine the properties. The triaxial compression test is considered the most effective test to determine stiffness properties of soils; however, equipment for this test is not readily available to many pipe designers and the testing is relatively complex and time consuming. A relatively simple alternate to the triaxial test is the one-dimensional compression test which consists of compressing soil in a rigid mold that allows no lateral strain. This is essentially the oedometer test used for determining consolidation characteristics of clays.

The one-dimensional compression test is not typically used for coarse-grained soils because the standard mold is small relative to the particle sizes, because of edge effects at the soil-mold interface, and because of difficulty in leveling the sample surface and getting uniform contact with the loading plates. Even though these problems are known to exist, several of the backfill soils were evaluated with the one-dimensional compression test (Courtney, 1995, and Ramsay, 1994) and the results demonstrate important characteristics of backfill behavior.

3.3.1 Procedures

The test apparatus is shown in fig. 3.10. Tests were conducted in a 155 mm (6.11 in.) diameter mold with a height of 50.8 mm, (2 in.). All specimens were prepared at the optimum moisture content determined from the results of the standard Proctor test. Two methods of compaction of the compression test specimens were evaluated:

- Clay samples were compacted by static compression. This was accomplished in layers. The first layer of soil was placed in the mold and subjected to a static compression force in the compression testing machine until it reached the desired density. This was then repeated for the second layer of the specimen.



1 in. = 25.4 mm

Figure 3.10 Configuration of One-Dimensional Compression Test

- Coarse-grained soils were compacted by vibration. The full test amount of soil was placed in the test mold which was then secured to a vibrating table. The specimen was then vibrated at 60 hertz until the sample reached the desired density.

After preparation, samples were tested in a 53.3 kN (12,000 lb) capacity Tinius Olsen screw-drive compression machine. Load and strain were recorded at closely spaced intervals using an Artech 44.5 kN, (10,000 lb) load cell and a Hewlett Packard LVDT with a computerized data acquisition system. A test consisted of three load-unload cycles over a compression stress range from 0 to 1,000 kPa (0 to 145 psi).

Tests were conducted on the shoulder stone, rewash, winter sand, and top clay at several densities.

3.3.2 Results

All data was plotted by considering any load up to a stress level of 7 kPa (1 psi) as a seating effect. The stress and strain at this point on the raw data curves was subtracted from the remaining data prior to plotting. Stress-strain curves at a density of about 90 percent of maximum dry density are presented for each of the four soils tested in fig. 3.11, which shows the following:

- As the particle size decreases the total strain at 1,000 kPa (150 psi) increases. This demonstrates the relative stiffness of the soils.
- The high stiffness of the shoulder stone relative to the other soils is demonstrated by the high slope of the initial portion of the curve in the first load cycle.
- The slope of the curves for all three cycles of the coarse-grained soils are much higher than for the corresponding cycles of the clay. This also suggests the better performance of the coarse-grained materials.
- The stress-strain curve for the clay material shows a decrease in slope at about 4 percent strain. This "wave" is thought to be the result of the compaction method.

The stress-strain curves of the four soils in the lower stress region where pipes are typically installed are shown in fig. 3.12. This figure clearly shows the greater stiffness of the shoulder stone. The performance of the clay is much better than expected, showing a stress-strain curve similar to that of the winter sand and rewash. This is thought to be an effect of the differences in the compaction methods. The clay had been compacted using static compression, while the coarse grained soils were compacted using vibration. Thus, the stress-strain behavior of the sand represents a first load cycle while the clay is already on a second load cycle. The decrease in slope for the clay stress-strain curve at a strain level of about 3 percent supports this explanation.

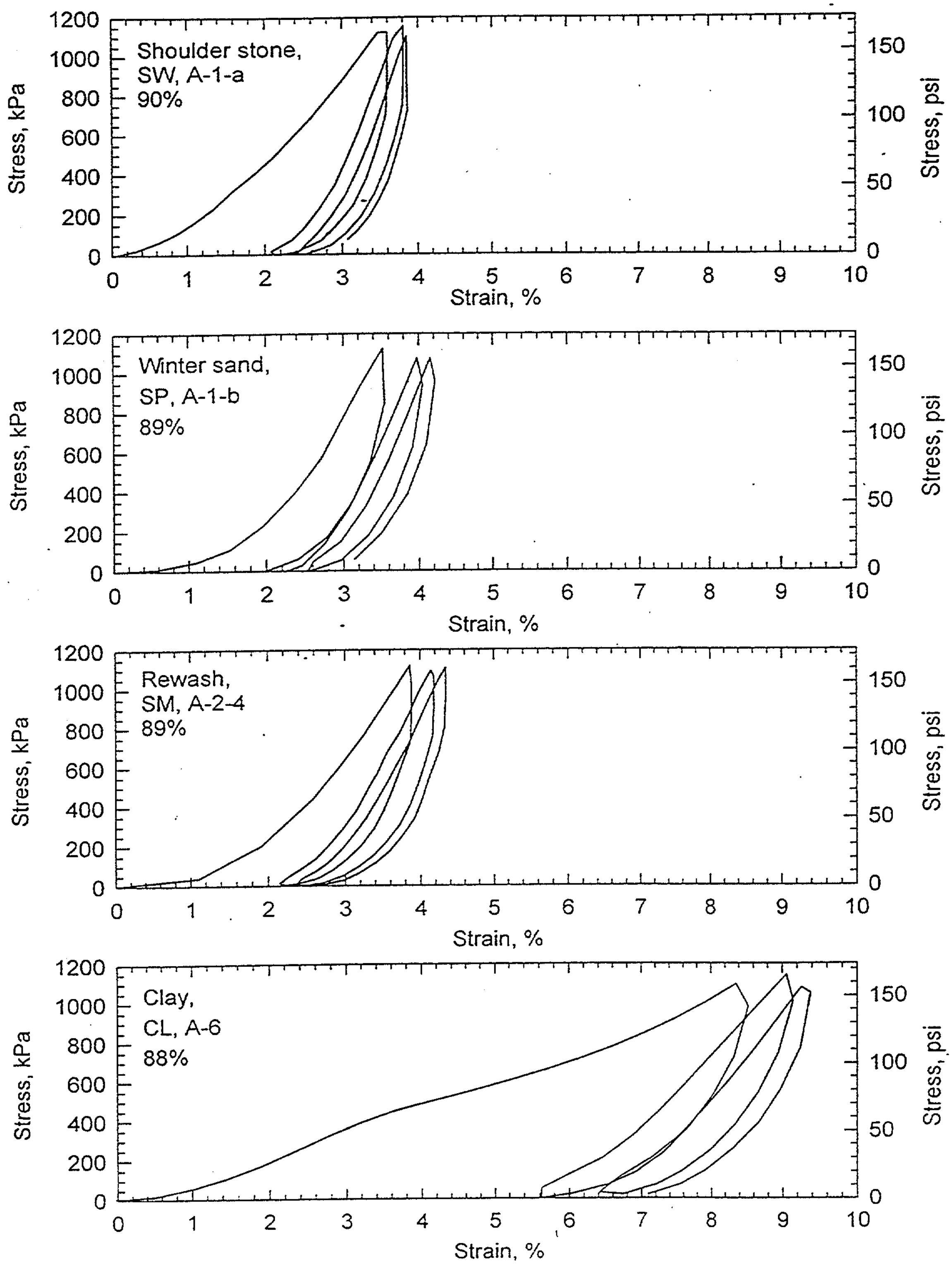


Figure 3.11 One-Dimensional Stress-Strain Curves at Approximately 90 Percent of Maximum Standard Proctor Density

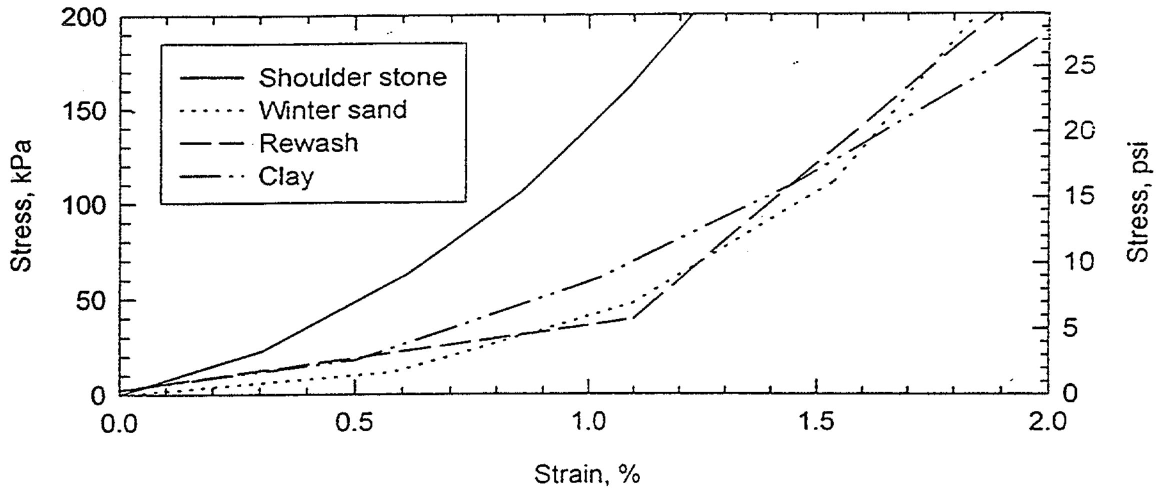


Figure 3.12 Stress-Strain Curves at Typical Stress Ranges, 90 Percent Density

Table 3.5 shows the secant constrained modulus, computed as the slope of the secant from the origin of the stress-strain curve to the "applied stress" level shown in the left hand column of the table. Modulus values are presented for several densities for each material. These values demonstrate the expected trends with changing density; however, the moduli are substantially lower than expected based on the predicted values from standardized soil properties, such as those used to develop the SIDD designs for reinforced concrete pipe, particularly those for the shoulder stone and winter sand. This will be discussed further in section 3.5.

Table 3.5
Constrained Modulus Values (MPa) from One-Dimensional Compression Tests

Applied stress (kPa)	Shoulder stone: Compaction level (% of maximum standard Proctor)				
	97%	90%	84%	75%	
7	7.3	5.6	3.3	1.9	
14	7.9	6.3	3.7	1.9	
34	9.3	8.2	4.9	2.1	
69	10.3	10.5	6.6	2.5	
138	12.6	13.8	9.3	3.1	
276	16.0	18.9	12.9	4.1	
413	18.7	21.7	14.9	5.0	
689	23.3	26.6	18.7	6.4	
1034	27.6	31.3	22.6	7.9	
Applied stress (kPa)	**Winter sand: Compaction level (% of maximum standard Proctor)**				
	94%	91%	89%	85%	63%
7	3.2	1.1	0.8	2.5	0.05
14	3.8	1.7	0.9	3.1	0.08
34	5.7	3.0	1.9	5.0	0.2
69	7.6	5.0	3.0	6.4	0.3
138	11.4	8.1	5.2	8.5	0.6
276	17.8	13.0	8.8	11.6	1.0
413	23.0	16.8	12.2	14.4	1.5
689	31.1	22.5	17.6	18.3	2.3
1034	38.8	28.2	23.8	22.0	3.3
Applied stress (kPa)	**Rewash: Compaction level (% of maximum standard Proctor)**				
	89%	84%	53%		
7	0.9	1.9	0.06		
14	1.6	2.1	0.09		
34	3.3	3.6	0.2		
69	5.1	5.9	0.3		
138	8.4	9.4	0.5		
276	13.0	13.7	0.9		
413	16.4	16.2	1.3		
689	22.2	19.2	2.1		
1034	27.8	22.0	3.0		

Table 3.5 (Cont.)
Constrained Modulus Values (MPa) from One-Dimensional Compression Tests

Applied stress (kPa)	Clay				
	Compaction level (% of maximum standard Proctor)				
	89%	**84%**	**53%**		
7	3.2	1.1	0.8		
14	3.8	1.7	1.0		
34	5.7	3.0	1.9		
69	7.6	5.0	3.0		
138	11.4	8.1	5.2		
276	17.8	13.0	8.8		
413	23.0	16.8	12.2		
689	31.1	22.5	17.6		
1034	38.8	28.2	23.8		

1 psi = 6.9 kPa, 1 psi = 0.0069 MPa

3.4 Correlation of Modulus of Soil Reaction with One-Dimensional Modulus

Most finite element analyses of pipes and culverts use soil models that represent the non-linear behavior of soils with reasonable accuracy. The hyperbolic model is used most in the United States. It models non-linear stress strain behavior and considers both strength and stiffness. Simplified pipe design has not progressed as far and still relies on the empirical modulus of soil reaction, E', as a measure of soil stiffness. The modulus of soil reaction is based on Spangler's Iowa formula and values are determined by back calculation from test results. As noted in chapter 2, the relationship between the modulus of soil reaction and true soil properties such as Young's modulus, E_s, or the constrained modulus, $M_{s,}$ has been investigated by a number of researchers. While not yet a consensus, there is a growing belief that the modulus of soil reaction can be related to the constrained modulus, which is reasonable since the soil around a pipe is generally well confined. The relationship between M_s, as expressed by the hyperbolic model, and E' was investigated and is reported here.

Two constants are required to define behavior of an elastic material. The hyperbolic model uses Young's modulus and the bulk modulus as the parameters. These parameters are both affected by the soil strength and state of stress. The basic equations for stress-vertical strain, and volumetric strain, as presented in Selig (1988), are:

$$(\sigma_1 - \sigma_3) = \frac{\epsilon_v}{\dfrac{1}{E_i} + \dfrac{\epsilon_v}{(\sigma_1 - \sigma_3)_u}} \quad , \tag{3.1}$$

where

σ_1	=	major principal stress, kPa, psi,
σ_3	=	minor principal stress, kPa, psi,
$(\sigma_1 - \sigma_3)$	=	deviator stress, kPa, psi,
ϵ_v	=	vertical strain, mm/mm, in./in.,
E_i	=	initial tangent Young's modulus, kPa, psi, and
$(\sigma_1 - \sigma_3)_u$	=	ultimate deviator stress, kPa, psi,

and

$$\sigma_m = \frac{B_i \, \epsilon_{vol}}{1 - \dfrac{\epsilon_{vol}}{\epsilon_u}} \quad , \tag{3.2}$$

where

σ_m	=	mean stress = $(\sigma_1 + 2\,\sigma_3)/3$, kPa, psi,	(3.3)
B_i	=	initial bulk modulus, kPa, psi,	
ϵ_{vol}	=	volumetric strain, and	
ϵ_u	=	ultimate volumetric strain.	

The one-dimensional compression test imposes the additional restriction that the volumetric strain is equal to the vertical strain because the lateral strains are zero:

$$\epsilon_{vol} = \epsilon_v \quad , \tag{3.4}$$

Substituting Eq. 3.4 into Eq. 3.2 yields:

$$\sigma_m = \frac{B_i \, \epsilon_v}{1 - \dfrac{\epsilon_v}{\epsilon_u}} \tag{3.5}$$

Eq. 3.3 can be rearranged to:

$$\sigma_3 = \frac{3\,\sigma_m - \sigma_1}{2} \quad , \tag{3.6}$$

substituted into Eq. 3.1, and simplified to:

$$\sigma_1 = \frac{0.667\,\epsilon_v}{\dfrac{1}{E_i} + \dfrac{\epsilon_v}{(\sigma_1 - \sigma_3)_u}} + \sigma_m \tag{3.7}$$

The initial Young's modulus, a function of the hyperbolic model soil parameters, K and n, and the confining stress, σ_3 is:

$$E_i = K\ P_a(\sigma_3/P_a)^n \tag{3.8}$$

Substituting Eq. 3.6 into Eq. 3.8 gives:

$$E_i = K\ P_a\left(\frac{3\,\sigma_m - \sigma_1}{2\,P_a}\right)^n \tag{3.9}$$

The ultimate deviator stress is a model parameter that is a function of the actual deviator stress at failure and the model parameter, R_f. In the hyperbolic model this is written as:

$$(\sigma_1 - \sigma_3)_u = \frac{(\sigma_1 - \sigma_3)_f}{R_f} \quad , \tag{3.10}$$

where the deviator stress at failure is a function of the soil friction angle, ϕ, the cohesion intercept, C, and the confining stress, σ_3, as follows:

$$(\sigma_1 - \sigma_3)_f = \frac{2\,C\,(\cos\phi) + 2\,\sigma_3\,(\sin\phi)}{1 - \sin\phi} \quad . \tag{3.11}$$

Substituting Eq. 3.6 into Eq. 3.11, and the result into Eq. 3.10 gives the expression:

$$(\sigma_1 - \sigma_3)_u = \frac{2\,C\,(\cos\phi) + 2\left(\dfrac{3\,\sigma_m - \sigma_1}{2}\right)\sin\phi}{(1 - \sin\phi)\,R_f} \quad . \tag{3.12}$$

Finally, the major principal stress, σ_1, can be expressed in terms of the vertical strain (which by definition of the one-dimensional compression test is the volumetric strain), by substituting Eqs. 3.12 and 3.9 into Eq. 3.7:

$$\sigma_1 = \frac{0.667\,\epsilon_v}{\dfrac{1}{K\;P_a\left(\dfrac{3\,\sigma_m - \sigma_1}{2P_a}\right)^n} + \dfrac{\epsilon_v}{\left(\dfrac{2\,C\,(\cos\phi) + 3\,\sigma_m \sin\phi - \sigma_1 \sin\phi}{(1 - \sin\phi)\,R_f}\right)}} + \sigma_m \quad . \tag{3.13}$$

This is the expression for the one-dimensional stress-strain curve and can be used to compute the constrained modulus, M_s.

The above solution is based on the assumption of a linear failure envelope (constant soil friction angle at all stress levels). To incorporate the effect of a curved failure envelope, the expression for ϕ may be corrected by introducing a stress sensitive model parameter, $\Delta\phi$, where:

$$\phi = \phi_o - \Delta\phi\ \log_{10}\,(\sigma_3/P_a) \ . \tag{3.14}$$

Substituting Eq. 3.6 into Eq. 3.14 gives:

$$\phi = \phi_o - \Delta\phi\ \log_{10}\left(\frac{3\;\sigma_m - \sigma_1}{2\;P_a}\right) \ . \tag{3.15}$$

Substituting Eq. 3.15 into Eq. 3.13 produces a complete equation that can be solved for the stress-strain curve under confined conditions. The complete expression is complex but is solved by publicly available mathematics software packages such as MathCad.

From the stress-strain curve the secant constrained modulus can be computed at various stress levels. The secant modulus is considered most appropriate for simplified design of buried pipe as it represents average soil behavior over the stress range of interest. Four sets of soil parameters were compared:

- Hyperbolic soil properties proposed by Selig (1988) were used to develop the SIDD design method for reinforced concrete pipe. They are referred to as the Selig/SIDD properties.

- Another set of hyperbolic soil properties proposed by Selig (1990) were developed based on research focused on flexible pipe. These properties have been incorporated into the finite element program CANDE and are the default values if the Selig soil model is selected within CANDE. These properties are referred to as the Selig/CANDE properties.

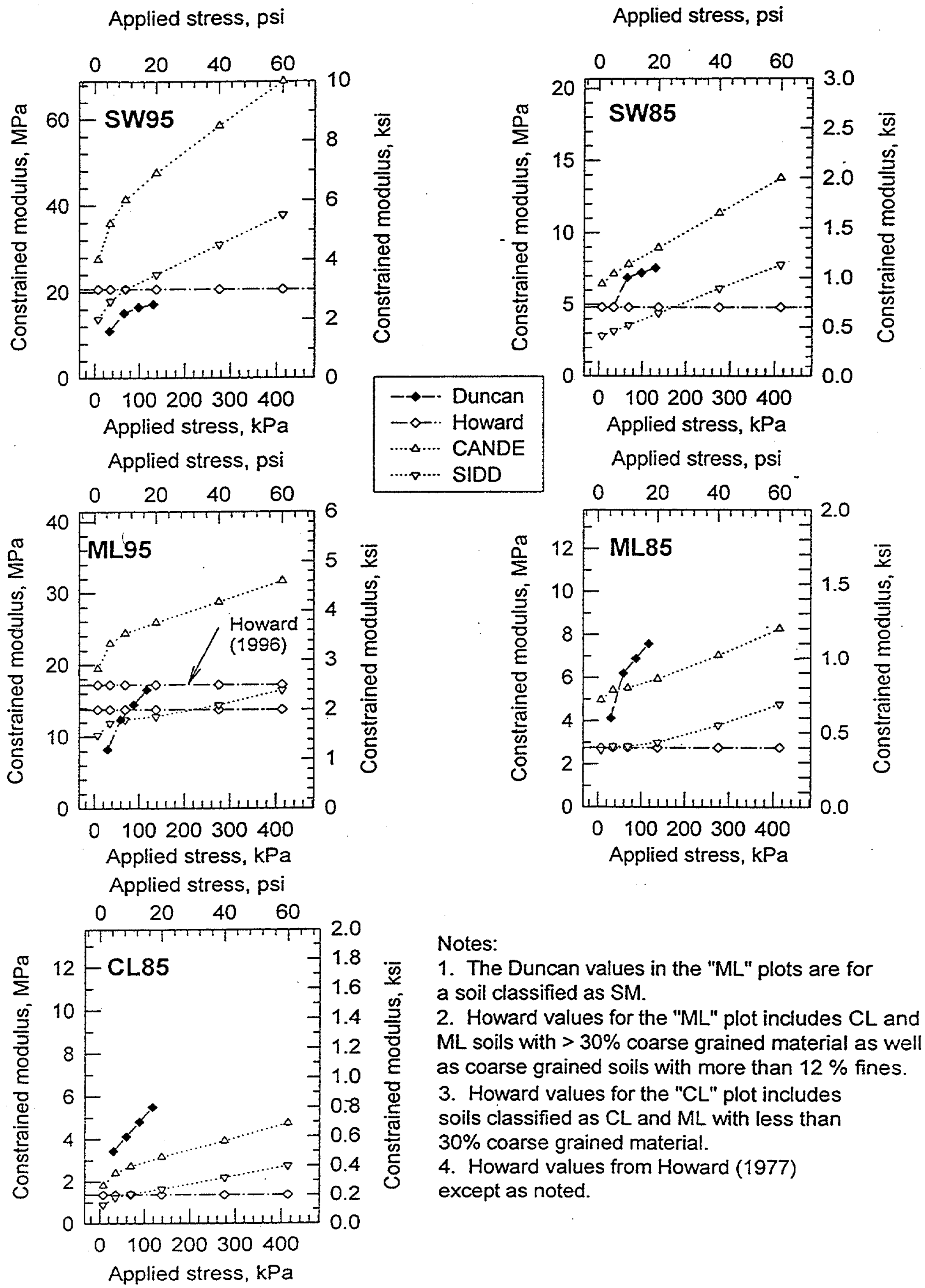
- E' values proposed by Duncan and Hartley (1987) were developed based on finite element analyses using hyperbolic soil properties previously proposed by Duncan et al. (1980). They are referred to as the Duncan properties.

- E' values proposed by Howard (1977) were developed based on back calculation, using the Iowa deflection formula, from measured deflections on a large number of projects. They are called the Howard properties.

The two sets of Selig soil properties include three general classifications of soil. Each general classification is given the name of the soil group which was actually tested, i.e., SW, ML, and CL. The two digit designation following the soil classification is the density as a percent of maximum standard Proctor density. A similar system is used to identify the Duncan Soil properties. Values of M_s and E', using the above four sets of data, are compared for different compaction levels in fig. 3.13, which indicates the following:

- The Selig/CANDE properties produce values of M_s that are consistently about twice the values produced by the Selig/SIDD properties.

- At stress levels less than about 70 kPa (10 psi) the Selig/SIDD properties are consistently similar to the values back calculated by Howard based on actual installations.



Notes:

1. The Duncan values in the "ML" plots are for a soil classified as SM.
2. Howard values for the "ML" plot includes CL and ML soils with > 30% coarse grained material as well as coarse grained soils with more than 12 % fines.
3. Howard values for the "CL" plot includes soils classified as CL and ML with less than 30% coarse grained material.
4. Howard values from Howard (1977) except as noted.

Figure 3.13 Comparison of Models for Secant Constrained Modulus

- The Duncan properties are somewhat erratic relative to all three of the other sets of properties.

The comparison in Fig. 3.13 suggests that for design purposes E' can be assumed equal to M_s and that the Selig/SIDD properties are roughly equivalent to the Howard values which represent a substantial amount of field data. This association further suggests that the same soil model could be used for simplified design of rigid and flexible pipe. This is a significant positive step in bringing together the currently diverse design methods used by different industries. Tabulated design values for M_S, computed from the Selig/SIDD properties at different stress levels are presented in table 3.6. These values can be used as a direct substitute for E' in design equations such as the Iowa formula.

The design values proposed in table 3.6 are compared with those determined by one-dimensional compression test and reported in table 3.5 and in fig. 3.14. This figure shows a poor match of properties from the two different sources. As noted previously, the problem is thought to be with the procedures used for the one dimensional testing, rather than the hyperbolic soil properties, which have had considerable successful use in design.

Table 3.6

Suggested Design Values for Constrained Soil Modulus, M_s

Stress level	**Soil type and Compaction Condition**		
	SW95	**SW90**	**SW85**
kPa (psi)	**MPa (psi)**	**MPa (psi)**	**MPa (psi)**
7 (1)	13.8 (2,000)	8.78 (1,275)	3.24 (470)
35 (5)	17.9 (2,600)	10.3 (1,500)	3.59 (520)
70 (10)	20.7 (3,000)	11.2 (1,625)	3.93 (570)
140 (20)	23.8 (3,450)	12.4 (1,800)	4.48 (650)
275 (40)	29.3 (4,250)	14.5 (2,100)	5.69 (825)
410 (60)	34.5 (5,000)	17.24 (2,500)	6.9 (1,000)

Table 3.6 (Cont.)
Suggested Design Values for Constrained Soil Modulus, M_s

Stress level	ML95	ML90	ML85
kPa(psi)	MPa (psi)	MPa (psi)	MPa (psi)
7 (1)	9.76 (1,415)	4.62 (670)	2.48 (360)
35 (5)	11.5 (1,670)	5.10 (740)	2.69 (390)
70 (10)	12.2 (1,770)	5.86 (750)	2.76 (400)
140 (20)	13.0 (1,880)	5.45 (790)	2.97 (430)
275 (40)	14.4 (2,090)	6.21 (900)	3.52 (510)
410 (60)	15.9 (2,300)	7.07 (1,025)	4.14 (600)
Stress level	**CL95**	**CL90**	**CL85**
kPa(psi)	**MPa (psi)**	**MPa (psi)**	**MPa (psi)**
7 (1)	3.68 (533)	1.76 (255)	0.90 (130)
35 (5)	4.31 (625)	2.21 (320)	1.21 (175)
70 (10)	4.76 (690)	2.45 (355)	1.38 (200)
140 (20)	5.10 (740)	2.72 (395)	1.59 (230)
275 (40)	5.62 (815)	3.07 (460)	1.97 (285)
410 (60)	6.17 (895)	3.62 (525)	2.38 (345)

3.5 CLSM Mix Design Study

A small scale study of CLSM mix designs was undertaken to investigate key elements of CLSM behavior and provide guidance in the selection of a mix design for the field studies reported in chapter 4. The study involved nine trial batches with different quantities of sand, fly ash, cement, and water. Testing was done for flowability and compressive strength. Materials were obtained from a nearby concrete batch plant.

The sand was fine aggregate for concrete batching per ASTM C 33. The component quantities for the nine trial mixtures are shown in table 3.7a. The quantities listed are for batch sizes of approximately 1 m^3; however, the actual batch sizes were much smaller.

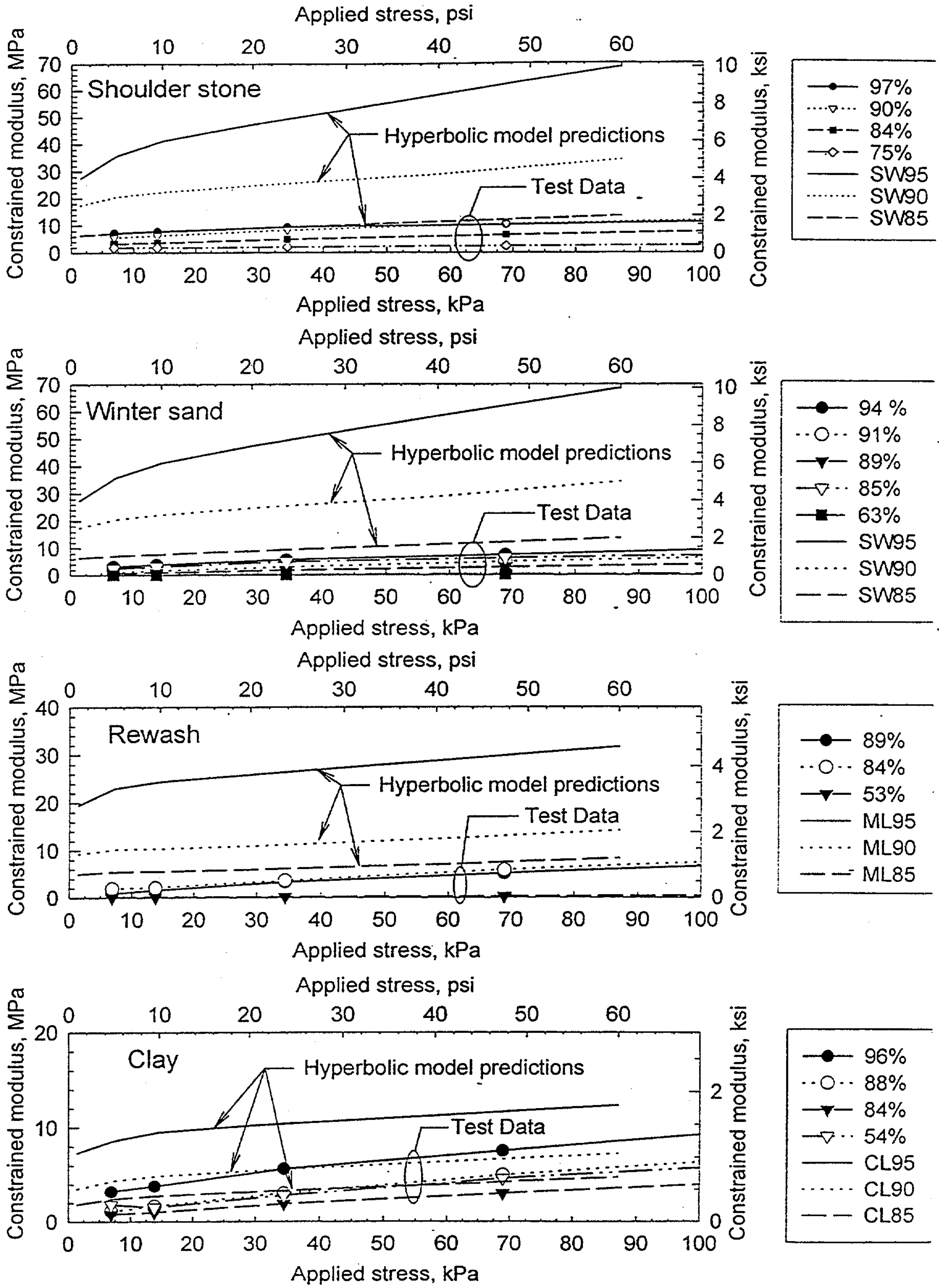


Figure 3.14 Comparison of Test Data for One-Dimensional Modulus with SIDD Soil Properties

Table 3.7

Mix Component Quantities and Strength Results

a) Mix Constituents (kg)

Material	Mix designation								
	Nom	A	B	C	D	E	F	X	Y
Cement	44	30	59	44	44	44	44	36	44
Fly Ash	296	148	296	222	296	296	296	148	148
Sand	1570	1570	1570	1570	1720	1570	1570	1570	1570
Water	296	296	296	296	296	237	355	296	296
w/c (1)	6.7	9.9	5.0	6.7	6.7	5.4	8.1	8.2	6.7
w/(c+fa) (1)	0.87	1.7	0.83	1.1	0.87	0.70	1.0	1.6	1.5

b) Test Results

7 Day compr. strength, kPa (psi)	1055 (153)	NT (2)	1410 (205)	515 (75)	825 (120)	1435 (209)	515 (75)	205 (30)	NT(2)
28 Day compr. strength, kPa (psi)	1890 (275)	350 (51)	2710 (393)	1645 (239)	1295 (188)	2900 (421)	1115 (162)	540 (79)	295 (43)
Segre-gation	None	Yes	Very little	Little	Little	Very little	Little	Yes	Yes
Spread, mm	380	No spread	250	280	220	No spread	315	–	No spread

Notes:
1. c = cement, w = water, fa = fly ash
2. Specimens A and Y were very fragile at an age of 7 days and broke up during the removal of the plastic molds and/or capping. NT = not tested.
3. ASTM Provisional Standard PS 28-95, Test Method for Flow Consistency of Controlled Low Strength Material
4. 6.89 kPa = 1 psi, 0.45 kg = 1 lbs, 25.4 mm = 1 in.

Specimen Preparation and Testing – Specimens were prepared in accordance with *ASTM Standard Test Method for Preparation and Testing of Soil-Cement Slurry Test Cylinders* (D 4832 - 88). The CLSM was mixed in a bowl with an egg beater type paddle for 2-3 minutes. Water was added to the mixer first, followed by sand, then cement, and finally fly ash. The addition of fly ash to the mix resulted in an enormous increase in flowability.

Flowability tests were conducted on all trial batches by placing a freshly mixed sample of CLSM in a 75 mm (3 in.) diameter by 150 mm (6 in.) high open ended tube, quickly lifting the tube vertically, and allowing the CLSM sample to flow into a circular mound. The circular sample spread was then measured. A minimum acceptable spread of 200 mm (8 in.) and no segregation of water were adopted acceptance criteria based on guide specifications of the Texas Aggregates and Concrete Association (TACA, 1989). These criteria have been adopted by other agencies as well.

The cylinders for compression testing were prepared and tested as follows:

1. The fresh mix was placed in three or four cylindrical plastic molds 100 mm diameter and 200 mm high (4 in. by 8 in.);
2. Specimens were allowed to set for 10 to 15 minutes, after which additional CLSM was added to displace bleed water and a lid was placed loosely on the filled mold;
3. Specimens were allowed to cure overnight in the laboratory and were then moved to a moist room;
4. Seven days after batching, two specimens of each mix were removed from the moist room, the plastic molds were stripped, and the test cylinders allowed to air dry for about 4 hours; and
5. The specimens were then capped with sulfur on both ends and tested in compression up to the ultimate strength.

Strength tests were conducted in the same fashion on the remaining test cylinders at an age of 28 days. In addition to monitoring load the cylinder strain was monitored with an LVDT for determination of modulus of elasticity.

Results – Compression and flowability test results are summarized in table 3.7b, along with observations of segregation. Findings include:

- Water to cement plus fly ash ratios greater than or equal to 1.5 produced the lowest compressive strengths. For example at an age of 7 days the strength of Specimen X was 205 kPa (30 psi) and Specimens Y and A broke up while being removed from the plastic molds. An inability to conduct compression tests does not mean that the mix is not suitable, only that the compression testing may not be an appropriate method of quality control.

- A 33 percent increase in cement content resulted in a 34 percent increase in the 7 day compressive strength and a 43 percent increase in the 28 day compressive strength (Specimens Nominal and B).

- A 25 percent decrease in the amount of the Class F fly ash resulted in about a 50 percent decrease in compressive strength (Specimens Nominal and C).

- A 10 percent increase in the amount of fine aggregate in the mix resulted in a 22 percent decrease in compressive strength (Specimens Nominal and D).

- A 20 percent reduction in the amount of water resulted in a 36 percent increase in compressive strength (w/c ratio of 0.87 for Specimen Nominal and 0.70 for Specimen E). Conversely, a 20 percent increase in the amount of water in the mix (w/c ratio of 0.87 for Specimen Nominal and 1.0 for Specimen F) resulted in about a 50 percent decrease in compressive strength when keeping the amount of cement and fly ash the same.

- Water segregated from the mixes with low amounts of fly ash as indicated by Specimens X, Y, and A. Specimen F which had more water than the others showed little water segregating from the mix. The remaining specimens, all of which had w/(c+fa) ratios of less than about 1.0, showed little or no segregation.

- Conversely, specimens with high amounts of fly ash (222 kg (488 lb) or greater) in the mix met minimum spread requirements of 200 mm (8 in.) except for Specimen E which fell over and which had the least amount of water. Specimens Y, X, and A having 148 kg (326 lb) of fly ash did not meet the 200 mm (8 in.) requirement.

The importance of fly ash in improving flowability, controlling water segregating from the mix, and increasing the compressive strength, is clearly indicated by these test results. Also, even though class F fly ash has no cementitious properties, an increase in compressive strength for increasing amounts of fly ash due to the pozzolanic reaction is clearly evident. The w/(c+fa) ratio (including the amount of fly ash) is a good indicator of expected material strength. Based on the results of this study, the mix design selected for the CLSM field test had 46 kg/m^3 (78 lb/ft^3) of cement and a water to cement plus fly ash ratio of 0.93. Additional details of the CLSM field test are provided in chapter 4.