Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-0-4177-1		

4. Title and Subtitle Recycled Asphalt Pavement and Crushed Concrete Backfill: State-of-the-Art Review and Material Characterization	5. Date 31 October 2001
7. Author(s) Ellen M. Rathje, Alan F. Rauch, Kevin J. Folliard, David Trejo, Dallas Little, Chirayus Viyanant, Moses Ogalla, Michael Esfeller	6. Performing Organization Code 8. Performing Organization Report No.
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650	10. Work Unit No. (TRAIS) 11. Contract or Grant No. 0-4177
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080	13. Type of Report and Period Covered Research Report, 1 Sept 2000 – 1 Sept 2001 14. Sponsoring Agency Code

15. Supplementary Notes

Project conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration, and the Texas Department of Transportation.

16. Abstract

This report describes research results from the first year of a three-year study focused on the use of recycled asphalt pavement (RAP) and crushed concrete (CC) as backfill for mechanically stabilized earth (MSE) walls. The design procedures for MSE walls are reviewed. The current Texas Department of Transportation (TxDOT) and Federal Highway Administration (FHWA) backfill specifications for MSE walls are described and the critical backfill properties for MSE walls are identified. The geotechnical engineering and durability issues related to using RAP and CC as backfill for MSE walls are discussed. Based on the information collected, the most critical geotechnical issues are likely to be the hydraulic conductivity of CC and the creep potential of RAP. The most critical durability issue most likely is the corrosion of metallic reinforcement. The current uses of RAP and CC in transportation-related projects are described, and the performance of projects incorporating RAP and CC is generally reported as good. Characterization of RAP and CC samples from throughout the state of Texas was carried out and indicated that RAP meets the current backfill specifications for gradation, Atterberg limits, pH, and resistivity. However, the CC samples did not meet the current backfill specifications for pH and resistivity. Bulk samples of RAP and CC, as well as a conventional fill material (CFM), were obtained for use in future testing. Compaction testing of these materials indicated that the dry density of each material is not very sensitive to water content.

17. Key Words	18. Distribution Statement
Recycled asphalt, crushed concrete, MSE walls	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	116	

RECYCLED ASPHALT PAVEMENT AND CRUSHED CONCRETE BACKFILL: STATE-OF-THE-ART REVIEW AND MATERIAL CHARACTERIZATION

by

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Research Report Number 0-4177-1

Research Project 0-4177

Use of Recycled Asphalt Pavement and
Crushed Concrete as Backfill for
Mechanically Stabilized Earth Retaining Walls

Conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

in cooperation with the

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

by the

CENTER FOR TRANSPORTATION RESEARCH

Bureau of Engineering Research

THE UNIVERSITY OF TEXAS AT AUSTIN

October 2001

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ACKNOWLEDGMENTS

The support and assistance of the personnel at the Texas Department of Transportation are gratefully acknowledged. These personnel include the project coordinator Mark McClelland (BRG), the project director Marcus Galvan (BRG), and the project monitoring committee, which includes John Delphia (BRY), Harold Albers (CST), and Mario Garza (CRP).

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

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Chapter 1: Introduction and Overview

1.1 RESEARCH BACKGROUND AND OBJECTIVE

A large number of mechanically stabilized earth (MSE) walls have been constructed over the past twenty years. It is believed that MSE walls have been constructed in every state in the U.S. with the majority in Texas, Georgia, Florida, Pennsylvania, New York, and California (Elias and Christopher 1996). The major advantages of MSE walls compared to other retaining structures are their flexibility, aesthetic appeal, ease of installation, and low cost (Morris and Delphia 1999). To ensure long-term integrity of the wall, conventional backfills consisting predominantly of granular soils have been recommended and used in the past. This limitation on material type can significantly increase the cost of construction on some projects due to the cost of transporting select material to the construction site.

Texas is one of the largest MSE wall builders in the nation. However, many parts of Texas do not have backfill materials that meet the current material specifications required by the Texas Department of Transportation (TxDOT) for MSE walls. In these cases, transporting select material from other parts of the state is necessary. However, this leads to an increased cost of construction that makes an MSE wall less economically attractive. One solution is to use recycled materials, such as recycled asphalt pavement (RAP) and crushed concrete (CC) acquired from the local area, as an alternative backfill. This not only minimizes the cost of material transportation but also alleviates the cost of disposing of excess materials.

TxDOT Project 0-4177 will evaluate the potential use of RAP and CC as a backfill material in MSE walls. Typical geotechnical tests, reinforcement pullout tests, and corrosion and degradation tests will be performed. The project will consist of an extensive laboratory investigation that will fully characterize RAP and CC, and will evaluate the effect of these materials on the durability of geosynthetic and metallic reinforcements typically used in MSE walls. The results from this investigation will be used to develop appropriate modifications to the materials specifications, laboratory test methods, and other related design and construction issues as needed to permit the use of these materials as backfill for MSE walls.

This report consists of seven chapters. After the introduction and overview presented in Chapter 1, Chapter 2 reviews the design of MSE walls. Chapter 3 discusses the engineering issues related to using RAP and CC as backfill in MSE walls, and Chapter 4 presents the literature regarding recent uses of RAP and CC in transportation-related applications. Chapter 5 describes the results from a survey of RAP and CC producers in Texas and presents results from the characterization of RAP and CC samples from throughout Texas. The identification of sources of RAP, CC, and reinforcement for the remainder of the project is also discussed in Chapter 5. Chapter 6 presents the results from the initial material characterization of bulk RAP and CC samples obtained for use throughout the duration of the project. Additionally, a conventional backfill material was obtained and its characterization is included in Chapter 6. A summary and conclusions are presented in Chapter 7.

1.2 DEFINITION AND DESCRIPTION OF MSE WALLS

An MSE wall is defined as a vertical or near vertical earth retaining structure consisting of three major components: a facing panel, earth reinforcement, and reinforced backfill (Figure 1.1.). Such walls are similar to a reinforced earth system, but with the addition of facing components for aesthetic purposes. An MSE wall functions through interaction between the soil and earth reinforcement. Once the vertical stress is introduced to the backfill, e.g., applied surcharge and soil self-weight, an inherent horizontal pressure is mobilized simultaneously from the stress transfer between soil particles (Figure 1.2). While the soil itself does not have tensile strength to resist such lateral pressure, buried reinforcement can provide efficient soil stabilization. The concept of soil reinforcement is based on the existence of a strong interaction between soil and an inclusion (Schlosser and Delage 1988). Consequently, the lateral tensile stress in the soil is transferred to this inclusion. The reinforced soil thus performs as a composite material that combines the best load-carrying features of both components (Morris and Delphia 1999).

The reinforcement reduces the lateral strain within the backfill through the shear resistance between the backfill soil and the reinforcement. As a consequence, the reinforced soil behaves as if an additional lateral confinement was applied, leading to an increase in strength in the reinforced zone. Based on field measurements, Schlosser (1990) showed that the tensile stress in the reinforcements drops off in the vicinity of the wall with the maximum value measured at a certain distance from the facing panels, indicating that not all of the tensile stress in the reinforcement was transferred to the facing panels. This indicates that the reinforced soil itself can stand vertically without the facing system. As the result, the facing panels are not designed for structural purposes but for aesthetic appearance and for preventing soil erosion between the reinforcements. The primary functions of the individual wall components are discussed below, along with a broad overview of the common types of each component in current use.

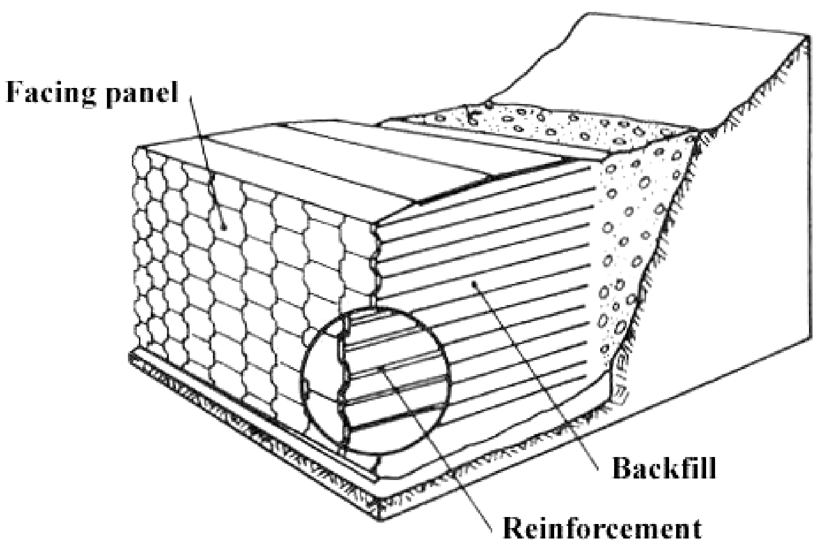


Figure 1.1 Schematic view of a reinforced earth wall (after Schlosser and Delage 1988)

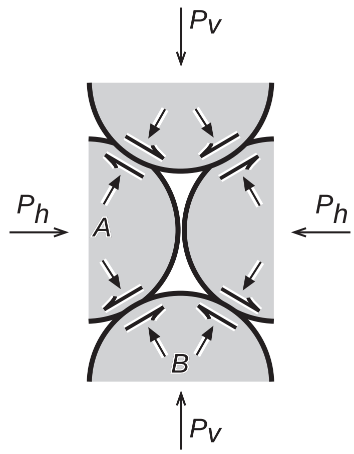


Figure 1.2 Stress transfer mechanism of soil particles when subjected to vertical loading (from Lambe and Whitman 1969)

Facing panel **–** Segmental precast concrete panels are most often used in MSE walls for their aesthetic appearance and ease of installation. Each panel is interlocked with adjacent panels to form a continuous and flexible wall facing. Vertically adjacent units are typically connected with shear pins. Additionally, as mentioned earlier, the facing panels are not designed for structural support purposes. Their primary functions are to prevent backfill erosion and, in some cases, to provide drainage paths through the wall. Geotextile strips are usually placed at the joints to prevent washout of the backfill between the adjacent panels.

Reinforcement **–** The primary function of the reinforcement is to strengthen the mechanical properties of the backfill. Currently, reinforcement is classified as either metallic (typically mild steel) or nonmetallic (generally polymer). Also, reinforcements can be categorized based on their extensibility. Inextensible reinforcement is reinforcement with its deformation at failure much less than the deformability of the soil, whereas extensible reinforcement has comparable deformation at failure to that of soil.

There are two types of stress transfer mechanisms between the reinforcement and soil. First, frictional resistance develops where there is a relative shear displacement, and corresponding shear stress, between the soil and reinforcement surface (Elias and Christopher 1996). Second, passive resistance develops through bearing type stresses on transverse reinforcement surfaces normal to the direction of relative movement (Elias and Christopher 1996). Figure 1.3 illustrates both shear mechanisms. The most commonly used reinforcement in Texas is galvanized ribbed steel strips, which have a combination of the two stress transfer mechanisms (Figure 1.4).

Backfill **–** The cost of the backfill material dominates the total cost of MSE wall construction. Elias and Christopher (1996) indicate that utilizing locally available soil can significantly reduce the total cost of construction on the order of 20 to 60 percent compared to conventional walls. The major function of the backfill is to provide the weight, compression resistance, and shearing strength to ensure the stability of the retaining wall (Morris and Delphia 1999). Also, in terms of physical properties, the select backfill should be a free-draining, high frictional strength material. More detail on backfill material and current specifications are discussed in Chapter 3.

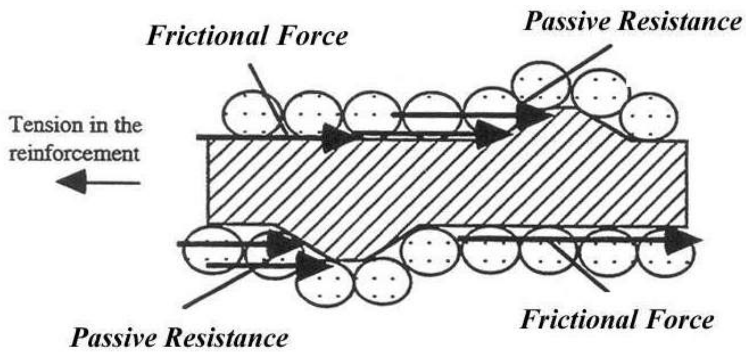


Figure 1.3 Stress transfer mechanisms mobilized along reinforcement (after Morris and Delphia 1999)

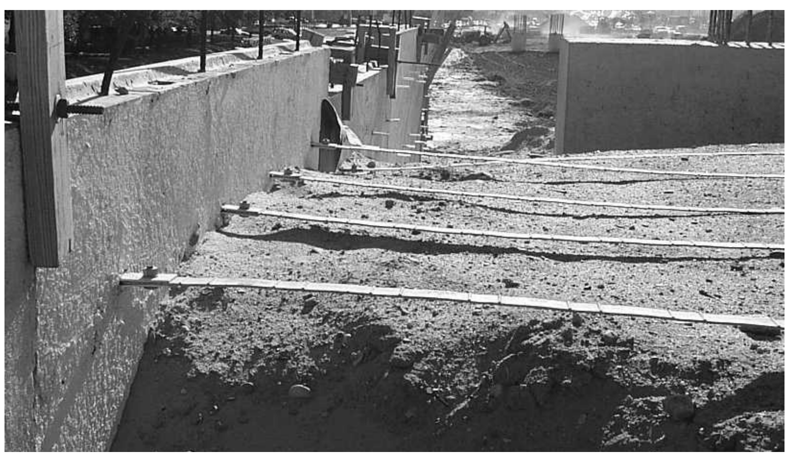


Figure 1.4 Typical ribbed steel strip reinforcement from MSE wall construction at IH-35 and US290 in Austin

1.3 DEFINITION AND DESCRIPTION OF RAP AND CC

RAP is removed and/or reprocessed pavement material containing asphalt and aggregates. Asphalt pavement is generally removed either by milling or full-depth removal. Milling involves removal of the pavement surface using a milling machine, which can remove up to 2 inches with a single pass. Full-depth removal is usually achieved with a pneumatic pavement breaker or a rhino horn on a bulldozer. The broken materials are transferred to a central facility for a series of recycling processes including crushing, screening, conveying, and stacking. Asphalt pavement can also be pulverized in place and incorporated into granular or stabilized base courses using a self-propelled pulverizing machine (FHWA 2000). In-place recycling eliminates the cost of transporting material to and from the processing facility.

CC is generated through the demolition of Portland cement concrete elements from roads, runways, and concrete structures. Crushed concrete is generally removed by a backhoe or payloader and loaded into dump trucks for removal from the site. In cases where crushed concrete is secured from demolished pavements, soil and small quantities of bituminous concrete can be expected in the excavated materials. Usually, reclaimed concrete materials are hauled to a central processing plant where crushing, screening, and ferrous metal recovery are performed before stockpiling. However, on-site recycling and processing can be performed with a mobile plant. At a central plant, reclaimed materials are subjected to primary and secondary crushers. The primary crusher breaks the reinforcing elements from the concrete debris and breaks down the rubble to 3 or 4 inches. Removal of reinforcing steel by an electromagnetic separator occurs while conveying the materials to the secondary crusher. The secondary crusher further breaks down the particle sizes to the desired gradation. Stockpiling of crushed concrete is usually done through the separation of coarse and fines particles to avoid inadvertent mixing of materials.

Chapter 2: Design of Mechanically Stabilized Earth (MSE) Walls

This chapter describes the design procedures for MSE walls based on FHWA construction guidelines (Elias and Christopher 1996), and discusses the critical backfill properties for adequate MSE wall design and performance. The FHWA guidelines are for walls with near-vertical faces and identical reinforcement length. The current design procedures consist of determining the geometric and reinforcement requirements that prevent internal and external failure of the MSE wall. The internal and external stability of the wall are evaluated using limit equilibrium methods of analysis. The following sections provide a broad overview of the design methodologies and backfill properties for MSE walls.

2.1 EXTERNAL STABILITY

The external stability of an MSE wall involves the geometry of the entire wall system. There are four potential failure mechanisms associated with external stability: (1) sliding, (2) overturning, (3) bearing capacity, and (4) deep-seated stability, as shown in Figure 2.1. A preliminary length of reinforcement is chosen as 0.7H or 2.5 m (whichever is

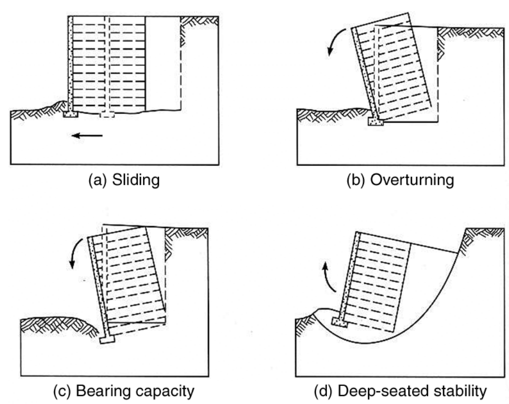


Figure 2.1 External stability mechanisms of failure (from Elias and Christopher 1996)

greater), where H is the height of the wall. After the preliminary dimensions of the entire wall system are chosen, external stability checks are performed. The external stability of each failure mode is represented in terms of a factor of safety (FS). The following subsections describe design procedures and stability checks with respect to the individual failure mechanisms.

2.1.1 Sliding Stability

The factor of safety against sliding ($FS_{sliding}$) is the ratio of the total horizontal resisting force divided by the total horizontal driving force acting on the wall. For the general case, the resisting force is the smaller of the shear resistance along the base of the wall or of a weak layer near the base of the wall. The driving force is the horizontal component of the active earth pressure from the retained soil behind the reinforced zone. Backfills with high internal friction angles will contribute to a higher resisting force at the base of the wall and thus save the cost of reinforcement. FHWA specifies that the factor of safety against sliding should be greater than 1.5. If the factor of safety is below 1.5, an increase in reinforcement length is required and sliding stability calculations are repeated. Such an increase in the reinforcement length will lead to a higher resisting force due to a larger area of sliding. The requirement for external stability against sliding generally governs the overall dimensions of the wall (Anderson et al. 1995).

2.1.2 Overturning Stability

In the FHWA design procedures, overturning stability of an MSE wall is determined with respect to the maximum permissible eccentricity of the resulting force. The resulting force (R) is the summation of the vertical forces acting on the reinforced fill. For instance, in Figure 2.2, the resulting force is attributed to the weight of fill itself (V_1+V_2) plus the vertical component of the earth pressure from the retained soil (F_T $\sin\beta$). The eccentricity (e) is computed by summing the moments of the mass of the reinforced soil section about the centerline of the mass and dividing by R. FHWA recommends a maximum eccentricity of $^{L}/_{6}$ for a soil foundation and $^{L}/_{4}$ for a rock foundation (L = reinforcement length). For general practices, the length of reinforcement will be increased if the eccentricity is larger than the value recommended in the FHWA manual. The flexibility of MSE walls should make the potential for overturning failure highly unlikely (Elias and Christopher 1996).

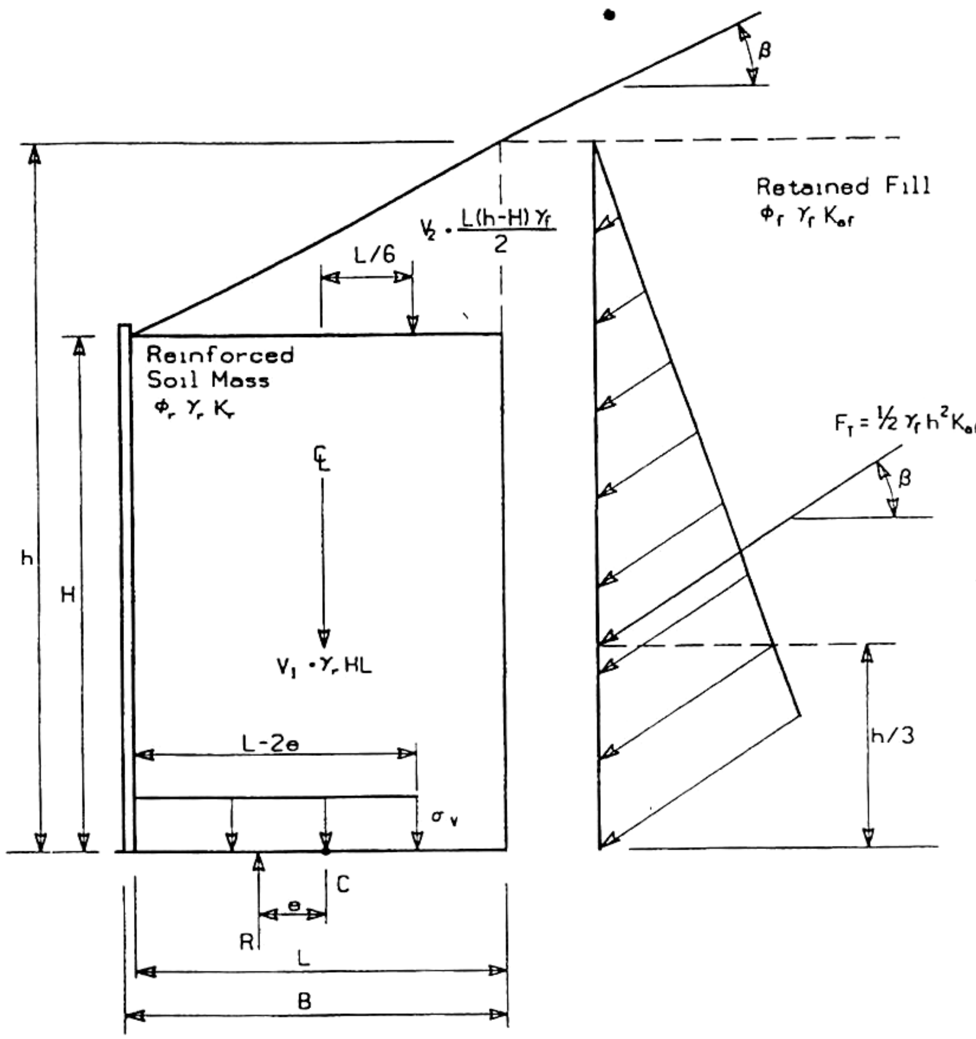


Figure 2.2 Detail of force diagram for eccentricity calculation (from Elias and Christopher 1996)

2.1.3 Bearing Capacity Failure

Bearing capacity failure is a major concern in the design of conventional retaining structures. Generally, two modes of bearing capacity failure are considered in MSE wall design: general shear failure and local shear failure. A factor of safety against general shear failure is defined by the ratio between the ultimate bearing capacity (q_{ult}), obtained from classical bearing capacity theory, and the vertical stress (σ_v) acting on the effective base area (L-2e) of the wall. FHWA specifies a minimum factor of safety of 2.5 for a bearing capacity failure. In a case where the wall is constructed on a soft soil foundation, the FHWA manual recommends that the stability against local shear failure can be neglected if the vertical stress from the backfill (γH) is less than three times the cohesion of the foundation soil (3c); otherwise, improvement of the foundation soil are required to improve its shear strength properties.

2.1.4 Deep-Seated Stability (Overall Stability)

Deep-seated stability can be critical for walls on steep slopes or on soft foundation soils. The overall stability is determined using rotational or wedge stability analyses, which can be performed using classical slope stability methods. When computing stability, the entire MSE wall system is considered as a rigid body and only failure surfaces completely outside the reinforced soil mass are considered. A minimum factor of safety of 1.3 is recommended for this case. Increased reinforcement length or ground improvement techniques are required if the preliminary design cannot satisfy deep-seated stability.

2.2 INTERNAL STABILITY

Evaluation of internal stability involves the interaction between the reinforcing elements and the backfill (Anderson et al. 1995). The mechanism of stress transfer depends on the type of wall system, extensible or inextensible. In design, two internal failure modes are taken into account. These failure modes are: tension failure and pullout failure (Figure 2.3). Preliminary evaluations regarding the force transfer mechanism must be determined prior to the internal stability check. These include: (1) determination of the maximum developed tensile forces and their locations along a locus of critical slip surfaces and (2) evaluation of tension resistance and pullout capacity by geotechnical laboratory tests. The following subsections describe the general procedure to check these internal failure modes.

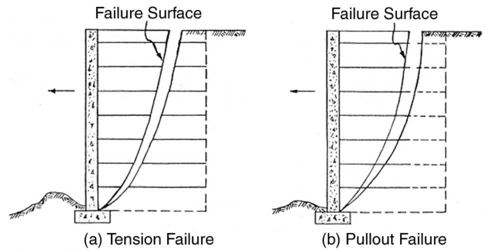
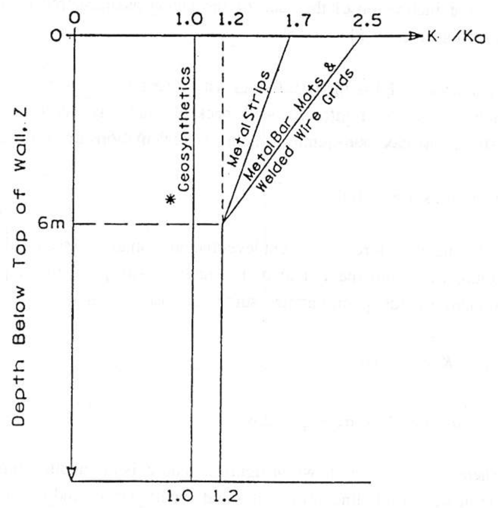


Figure 2.3 (a) Internal tension failure and (b) internal pullout failure of MSE wall (from Anderson et al. 1995)

2.2.1 Tension Failure

Tension failures occur when the tensile force in the reinforcement becomes so large that the reinforcement elongates excessively or breaks (Elias and Christopher 1996). The check for tension failure involves determining the maximum developed tensile force (T_{max}) and comparing it with the allowable tension (T_a) of the reinforcement. The allowable tension (T_a) is generally provided from either the manufacturer's specification or from laboratory testing. The maximum developed tensile force in each layer is obtained by multiplying the lateral earth pressure coefficient (K) by the vertical stress at that depth and the contributing area for each reinforcing element. Previous studies have indicated that the maximum tensile force is related to the type of reinforcement, as indicated in Figure 2.4. This graphical figure was developed through back analysis of the lateral stress ratio (K) from available field data where stresses in the reinforcements have been measured and normalized as a function of an active earth pressure coefficient (K_a) (Elias and Christopher 1996). Near the top of the wall, metallic reinforcement develops forces greater than predicted by the active earth pressure coefficient (K_a), and larger values of K should be used. For geosynthetic reinforcement, K_a is the appropriate earth pressure coefficient. After the value of T_{max} is calculated, a factor of safety against tension failure is determined by comparing it with the T_a value.

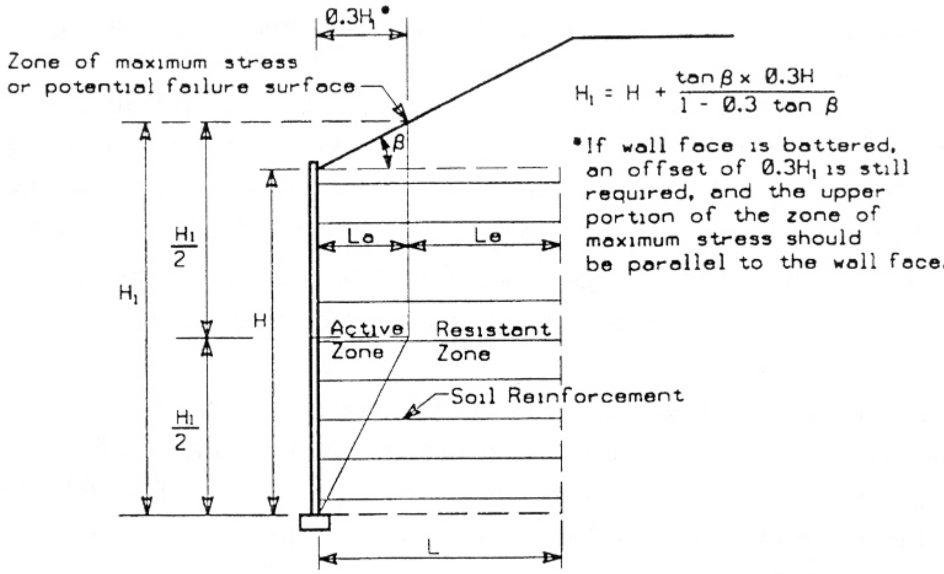


* Does not include polymer strip reinforcement

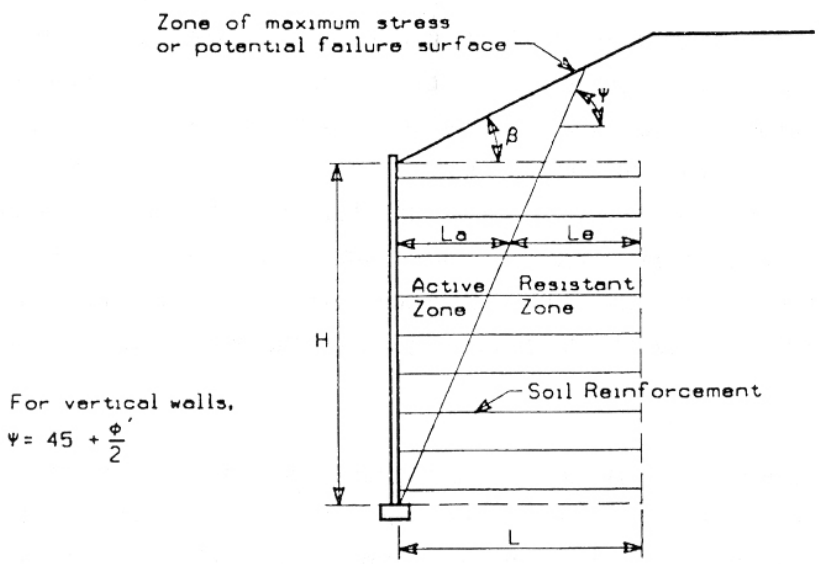
Figure 2.4 Variation of stress ratio with depth in a MSE wall (from Elias and Christopher 1996)

2.2.2 Pullout Failure

Unlike the other stability checks, pullout stability does not involve a comparison between driving forces and resisting forces through a factor of safety calculation. Calculations for this failure mode involve evaluating the length of reinforcement in the resistance zone (L_e) beyond the potential internal failure surface. This resistance zone develops the resisting force against pullout of the reinforcement. Earlier practice assumed that the pullout resistance was developed behind the Coulomb failure plane. However, field measurements and theoretical analyses have shown that the potential failure surface is coincident with the maximum tensile forces (T_{max}) in the reinforcements (Anderson et al. 1995). Also, the locations of the maximum tensile forces are mainly dependent on the extensibility of the reinforcement. Figure 2.5 illustrates the location of the potential failure surface for both inextensible and extensible reinforcement. The L_e value for each reinforcement layer is calculated by dividing the maximum tensile force developed at that layer (T_{max}) by the allowable pullout capacity obtained from the laboratory. Finally, a recommended factor of safety of 1.5 is applied to the calculated L_e value.



(a)



(b)

Figure 2.5 Location of potential failure surface for (a) inextensible reinforcement and (b) extensible reinforcement (from Elias and Christopher 1996)

2.3 CRITICAL BACKFILL PROPERTIES

The backfill is the key element to achieve satisfactory MSE wall performance. Based on the stability analyses presented earlier, an ideal backfill material will exhibit high drained shear strength parameters (c' and ϕ') and have good drainage properties. To avoid excessive surface deformations, the backfill should also exhibit low compressibility over time. Using high-quality materials will lead to minimal reinforcement length, which lowers the total cost of an MSE wall. Well-draining backfill prevents water from accumulating behind the wall and increasing the lateral pressure on the facing system. These properties are major factors when selecting a backfill material. However, to ensure proper long-term performance of the wall, requirements on other engineering-related properties of the backfill must be considered. Other critical characteristics of backfill materials are discussed below.

Hydraulic conductivity **–** Backfill materials for all types of retaining walls must be free draining so that water pressures do not build up behind the wall. Specifications typically limit the percentage of fines in the backfill. For MSE walls, backfill materials that are not free draining also increase the corrosion potential of metallic reinforcements. Therefore, a backfill with high water absorption potential, such as clay or silt, is generally not considered acceptable for MSE wall backfill.

Shear strength parameters **–** The backfill material should exhibit high shear strength to ensure stability within the backfill alone and to achieve an adequate interaction with the reinforcement (Morris and Delphia 1999). The forces developed in MSE wall reinforcement are related to the horizontal earth pressures acting on the wall at different depths, as shown in Figure 2.4. This horizontal earth pressure is calculated using the shear strength parameters of the backfill. For the general case, when a free draining material is used, effective stress shear strength parameters (c' and ϕ') will be required to calculate the wall stability. On the other hand, if the backfill has low permeability and the dissipation of pore water pressure would take a long time to complete, undrained shear strength parameters for the backfill must also be considered in the design process.

Interface friction **–** The interface friction angle between the backfill and reinforcement is needed for the design of the reinforcement length. This parameter is generally acquired from pullout capacity tests. According to recent research on interface friction, a well-graded material with high angularity tends to give higher values of interface friction angle. Also, moisture content and percent fines can affect the pullout resistance of the reinforcement.

Compaction characteristics **–** During construction, the backfill material must be well compacted to ensure adequate shear strength, adequate interface friction, and minimal compression. If the compaction method is the same, differences in the maximum dry density of two different materials are generally attributed to the particle shape, grain size distribution, and water content during compaction. Materials with low angularity and a wide range of grain sizes will tend to exhibit higher maximum dry density values. Materials with low water contents will have internal capillary stresses that resist the densification of the material, resulting in lower dry densities (Morris and Delphia 1999). Particle breakdown during compaction is another key factor to backfill drainage properties. The additional fines from this breakdown mechanism may lower the hydraulic conductivity and change the shear strength properties of the backfill.

Compressibility of compacted material **–** In general, the presence of fines in the backfill material indicates the potential for long-term settlement. Also, differential settlement may occur when the backfill is not compacted uniformly throughout the wall area. These settlements can create problems with the performance of pavements on the backfill surface, for example. This potential settlement may also cause significant damage to the reinforcement and the facing system. When such settlements occur, the reinforcements are forced downward, creating an undesirable vertical stress on the facing system.

In addition, when a granular backfill is compacted at a low dry density and water content, it can undergo significant settlement upon wetting. This deformation mechanism is called collapse. A collapsible soil of this type may withstand relatively large applied vertical stress with small settlement while at a low water content, but exhibit considerable settlement after wetting with no additional increase in vertical stress (ASTM D5333). Therefore, the collapse potential of the backfill must be studied.

Time-dependent effects (creep behavior) **–** Creep behavior is defined as "an irrecoverable time-dependent deviatoric deformation that results from long-term application of a deviatoric stress" (ASTM D5520). For MSE walls, creep deformation is believed to interfere with the development of forces in the reinforcement and could lead to a wall failure through excessive deformation or collapse. In some materials, creep behavior is temperature dependent and may be enhanced at higher temperatures. For long-term stability, the backfill material should not be susceptible to creep. For a given material, a creep testing program is used to define the relationship between the creep strength and such factors as time to failure, steady-state or minimum creep rate, strain at failure, and temperature. These relationships assist engineers in selecting the appropriate material properties for a given loading condition.

Corrosivity – Corrosion is a major concern for MSE walls incorporating metallic reinforcement. Accelerated or unanticipated corrosion of the reinforcements could cause sudden and catastrophic failure of MSE structures, generally along a potential failure line of maximum tensile stresses in the reinforcements (Elias 1996), as shown in Figure 2.5. The backfill should not contain highly deleterious materials that would attack the reinforcement or cause distress to the material itself. FHWA has used resistivity and pH values as indicators to reflect the corrosion potential of backfill material.

Resistivity is a measurement of the difficulty an electric current has in flowing through a material. As a result, a low resistivity value is indicative of high potential for corrosion. Furthermore, highly acidic (low pH) or highly alkaline (high pH) material is corrosive because the presence of an electrolyte causes a voltage difference between metal surfaces, which induces a current. Therefore, an ideal backfill that will resist corrosion should have high resistivity and exhibit a pH value in the permitted range. The resistivity value is also influenced by the presence of soluble salts in the material. High concentrations of soluble salts will affect the electrochemical reaction at the metal surface and decrease the resistivity of the material. The type of ion is also important to the corrosion process. Two major chemical ions that have been identified with high corrosivity potential are chlorides and sulfates (Rabeler 1989). Specifications have limited the presence of these two chemicals in the backfill to be less than 100 ppm and 200 ppm for chlorides and sulfates, respectively.

2.4 SUMMARY

This chapter describes the design procedures for MSE walls recommended by FHWA (Elias and Christopher 1996). These design procedures evaluate the internal and external stability of MSE walls using limit equilibrium methods. The critical backfill properties that affect MSE wall stability were identified as hydraulic conductivity, shear strength, interface friction between the backfill and reinforcement, compaction characteristics, compressibility, creep, and corrosivity. These critical backfill properties will be evaluated over the course of this three-year study.

Chapter 3: Engineering Issues Related to the Use of RAP and CC as Backfill for MSE Walls

This chapter presents current specifications for key geotechnical properties of backfill materials as required by TxDOT and FHWA design guidelines, and describes the engineering issues related to using RAP and CC as backfill in MSE walls. Specific issues include backfill properties (e.g., hydraulic conductivity, shear strength, creep), backfill–reinforcement interactions (e.g., pullout resistance), and reinforcement durability. All of the specifications presented here were initially evaluated for traditional backfill materials.

3.1 GEOTECHNICAL ISSUES

This section presents the specifications for MSE wall backfill materials and describes the issues related to using RAP and CC as backfill in MSE walls.

3.1.1 Gradation

An important geotechnical aspect commonly used to classify different backfill materials is particle gradation. Gradation is an important factor that affects backfill performance including stability, drainage, and frost susceptibility. Because specifications for recycled materials have not been standardized, the gradation requirement for traditional backfill proposed by TxDOT and FHWA can be used as an initial guideline. The backfill should be free from organic and deleterious materials with the gradation determined in accordance with sieve test methods Tex-110-E or AASHTO T-27. TxDOT and FHWA have relatively similar requirements on backfill gradation, with the major difference being the maximum allowable particle size. Both specifications permit up to 15% fines (i.e., material passing the No. 200 sieve).

TxDOT has categorized backfill materials into two types, Type A and Type B. The major difference between Type A and Type B is the maximum allowable size. Particles as large as 3 in. are allowed for Type A and 6 in. for Type B. FHWA has only one backfill specification, with a maximum size of 4 in. Table 3.1 compares the gradation requirements in the TxDOT and FHWA specifications. In cases where nonmetallic or epoxy-coated metallic reinforcement are used, both FHWA and TxDOT limit the maximum particle size of the backfill to 0.75 in. (19 mm) to ensure minimal abrasion of the reinforcement.

The recycling process and the size of aggregates in the virgin asphalt and concrete control the particle size distribution of RAP and CC. The maximum particle size can be as large as 3 in. according to data from several regions; however, most TxDOT districts have reported a maximum particle size of 1-½ in. More data on the typical gradation of RAP and CC materials in Texas are given in Chapter 5.

Table 3.1 TxDOT and FHWA particle gradation specifications for MSE wall backfill

Classification		Sieve Size	% passing
TxDOT	Type A	3 in.	100
		# 40	0 - 60
		# 200	0 - 15
	Type B	6 in.	100
		3 in.	75 - 100
		# 200	0 - 15
FHWA		4 in.	100
		# 40	0 - 60
		# 200	0 - 15

3.1.2 Laboratory Compaction

It is important to evaluate the compaction characteristics of RAP and CC because these materials may not yield moisture–dry density curves similar to those for traditional backfill. Also, because potential crushing of the grains during compaction is a major concern, sieve analysis before and after compaction is required to evaluate the possible increase in fines content. The following sections describe the compaction tests currently used by FHWA and TxDOT. Table 3.2 summarizes the compaction test methods used by TxDOT and FHWA. The compaction methods specified by TxDOT and FHWA are described in the following sections.

The TxDOT specifications express the required compaction of backfill materials for retaining structures in terms of percent relative compaction in accordance with test method

Tex-114-E. The specifications stipulate that the backfill be compacted at 95% relative compaction based on Tex-114-E in the top 3 ft of the fill, while 90% is required for the underlying layers. However, the Tex-114-E test method limits the maximum particle size to $^{7}/_{8}$ in. (22 mm), which is significantly smaller than the maximum particle size allowed in the backfill and significantly smaller than the maximum particle size in typical RAP and CC material. Accordingly, Tex-114-E recommends test method Tex-113-E be used for materials with particles larger than $^{7}/_{8}$ in. because Tex-113-E allows maximum particle sizes up to 1-¾ in.

Morris and Delphia (1999) recently studied the backfill specifications for MSE walls. They recommend using a vibrating hammer in accordance with British Standard 1377 (1990) as a standard laboratory compaction test, because test method Tex-113-E tends to give low values of maximum dry density, especially for coarse materials. However, the subsection on "Materials Difficult to Compact" in Tex-113-E proposes a higher compaction effort with 100 blows per layer to achieve a higher dry density. In a recent study at The University of Texas at Austin (Marx 2001), it was indicated that this modification in Tex-113-E gave the highest dry density among all compaction tests.

In general, the FHWA specification requires that the backfill be compacted to at least 95% of the maximum dry density and within ± 2% of optimum moisture as determined by methods C or D of AASHTO T-99. This compaction method is similar to the standard Proctor compaction test (5.5-lb hammer compacting 3 equal layers of material with 12-in. drop height). Test methods C and D differ in the diameter of the compaction mold, but the compaction energy is the same. The allowable maximum particle size is ¾ in., which is smaller than the expected maximum particle size in RAP and CC. For such cases, the AASHTO specifications recommend a scalping procedure that maintains the same percentage of coarse particles as in the virgin material. Particles with a size larger than 19 mm (¾ in.) are discarded prior to compaction. Particles passing the 50-mm (2-in.) sieve but retained on the 19-mm (¾-in.) sieve are replaced with an equal mass of material passing the 19-mm (¾-in.) sieve but retained on the 4.75-mm sieve (sieve No. 4).

Table 3.2. Compaction test methods used by TxDOT and FHWA.

Compaction Test Method		*Use for Materials*	*Mold Dimensions*		*Compaction Hammer*			*# Layers*	*Blows per Layer*	*Compaction Energy (ft-lb/ft³)*
			Diameter (in.)	*Height (in.)*	*Weight (lb)*	*Drop (in.)*	*Face Shape*			
Tex-113-E		100% passing 1 ¾" sieve materials with particles > 7/8" base materials treated subgrade/embankment materials	6	8	10	18	sector	4	50	22,900
		"materials difficult to compact" 100% passing 1 ¾" sieve materials with particles > 7/8" base materials treated subgrade/embankment materials	6	8	10	18	twin sector, on neoprene pad	8	100	91,700
Tex-114-E	I	≤ 20% retained on ¼" sieve 100% passing 3/8" sieve untreated subgrade/embankment material	4	6	5.5	12	sector	4	25	12,600
	II	> 20% retained on ¼" sieve 100% passing 7/8" sieve untreated subgrade/embankment material	6	8	5.5	12	sector	4	75	12,600
AASHTO T 99-97	A	100% passing No. 4 sieve	4	4.584	5.5	12	circular or sector	3	25	12,400
	B	100% passing No. 4 sieve	6						56	12,300
	C	100% passing ¾" sieve	4						25	12,400
	D	100% passing ¾" sieve	6						56	12,300
AASHTO T 180-97	A	100% passing No. 4 sieve	4	4.584	10	18	circular or sector	5	25	56,200
	B	100% passing No. 4 sieve	6						56	56,000
	C	100% passing ¾" sieve	4						25	56,200
	D	100% passing ¾" sieve	6						56	56,000
ASTM D 698-00	A	≤ 20% retained on No. 4 sieve	4	4.584	5.5	12	circular	3	25	12,400
	B	> 20% retained on No. 4 sieve and ≤ 20% retained on 3/8" sieve	4				circular		25	12,400
	C	> 20% retained on 3/8" sieve and < 30% retained on ¾ inch sieve	6				circular or sector		56	12,300
ASTM D 1557-00	A	≤ 20% retained on No. 4 sieve	4	4.584	10	18	circular	5	25	56,200
	B	> 20% retained on No. 4 sieve and ≤ 20% retained on 3/8" sieve	4				circular		25	56,200
	C	> 20% retained on 3/8" sieve and < 30% retained on ¾" sieve	6				circular or sector		56	56,000

3.1.3 Field Compaction

The FHWA manual indicates that field compaction of RAP and CC can be accomplished with similar methods and equipment as used with conventional backfill materials. It is reported that granular materials containing RAP appear to compact better if incorporated with some water (Senior et al. 1994). Compaction of CC usually requires additional water to facilitate particle arrangement. Also, due to the high angularity of CC materials, equipment with higher compaction energies is often required to achieve the specified level of relative compaction. Additionally, when compacting gravel-size particles, caution is needed to ensure that no large zones of poorly compacted material are formed within the fill that could contribute to subsequent long-term differential settlement.

3.1.4 Hydraulic Conductivity

Backfill materials must be free draining to ensure that pore water pressures do not develop behind the retaining structure. The rate of corrosion of metallic reinforcement, which is the typical reinforcement used in Texas, is also primarily dependent on the moisture content in the backfill. Cohesive particles present in the backfill impede the dissipation of pore water pressures behind the retaining structure, resulting in large forces on the wall and a higher rate of reinforcement corrosion. Therefore, under all of these primary considerations, cohesive material is undesirable as a backfill material. As a result, the best backfill for an MSE wall is a cohesionless material with little or no plastic fines.

In general, the hydraulic conductivity of an MSE wall backfill is not explicitly measured because the specification on material gradation generally results in a high hydraulic conductivity. TxDOT and FHWA both permit up to 15% fines (passing the No. 200 sieve) in the backfill. This ensures that the select material is free draining. However, it is still unknown how water will interact with RAP and CC backfill. Also, crushing of coarse-size particles can be anticipated during compaction, which can potentially lead to a significant decrease in hydraulic conductivity with the increase in fines. For crushed concrete, potential water absorption is expected to be higher than in conventional backfill material due to the presence of mortar and debris. Moreover, unhydrated cement in CC may react with seepage water to cause a significant reduction in the hydraulic conductivity of the backfill. Therefore, the hydraulic conductivity may vary with time after compaction is completed.

3.1.5 Settlement

Generally, settlement in an MSE wall backfill is related to the quality of the backfill material used, uniformity of the material, and the uniformity of compaction in the reinforced zone. Also, the more fines the material contains, the larger the tendency for settlement in the backfill. Because significant fines are not permitted in the backfill material, settlement is mainly a function of the compaction method and uniformity of the backfill. The compressibility characteristics of RAP and CC may not be the same as for conventional backfill. Creep behavior is potentially significant in RAP, which involves movement with time at constant stress level. According to reported data on the particle distribution of RAP, the percentage of fines is usually low (less than 1%). This indicates that RAP may not be susceptible to creep. For material with substantial fines, one-dimensional consolidation tests can be performed (in accordance with ASTM D2435) to evaluate the compressibility. This test provides the material properties that are used to estimate primary and secondary consolidation deformation. Moreover, measurement of the collapse potential (ASTM D5333) should also be conducted to evaluate the potential for collapse upon wetting. Collapsing materials can lead to the differential settlement behind the wall.

As for compaction uniformity, settlement of the backfill is primarily expected in the area right behind the facing panels, where only small compaction equipment is allowed. The settlement in this area is generally attributed to low density in the backfill. Hence, to minimize such settlement problems, TxDOT and FHWA require that regular, periodic field inspection of backfill compaction be performed throughout the construction period.

3.1.6 Shear Strength

The shear strength parameters of the backfill are critical properties in the design phase because they govern the stability of the wall. FHWA and TxDOT specifications on material gradation and compaction should already yield high friction-angle backfill. Laboratory tests on RAP and CC have shown high internal friction angles with little or no cohesion observed (Petrarca and Galdiero 1984). Shear strength of RAP should be comparable to a similarly graded natural aggregate, whereas the shear strength of CC was reported to be similar to that of crushed limestone aggregates. Accordingly, RAP and CC should have adequate shearing resistance for the backfill system. Because TxDOT and FHWA do not specify the same tests for measuring shear strength, the shear strength tests specified by each agency are described below.

TxDOT does not specify a minimum internal friction angle for backfill material. However, the specifications require that the shear strength of the material be measured according to test method Tex-117-E, "Triaxial compression tests for disturbed soils and base materials." This test measures the shearing resistance, water absorption, and potential expansion of the soil. Each specimen is subjected to an absorption measurement prior to a consolidated-drained triaxial compression test. At the end of the test, shear strength parameters are reported in terms of cohesion and internal friction angle, along with absorption and expansion characteristics of the materials.

A problem regarding test setup arises because geotechnical testing practice requires that triaxial specimens have a diameter at least six times greater than the largest particle size. As RAP and CC often contain particles larger than 1 in., conventional testing equipment will be too small to accurately measure the strength of these materials. For example, tests on 2-in. diameter specimens would require scalping out particles larger than $^{1}/_{3}$ in. during sample preparation. However, screening out large particles may yield a higher measured shear strength because the large particles tend to be more fractured and thus weaker.

Unlike the TxDOT specification, the FHWA design manual specifies a minimum friction angle of 34° for backfill materials. The FHWA design manual indicates that the friction angle should be measured by the standard direct shear test (AASHTO T-236) and that only the material finer than the No. 10 sieve should be tested. The test specimen is compacted at 95% of AASHTO T-99 (method C or D) and sheared in a consolidated-drained condition at different normal pressures. For a conventional backfill, the design manual specifies that this test is not required if the backfill material contains more than 80% by weight of particles larger than ¾ in.

3.1.7 Creep Characteristics

One common assumption made when designing MSE walls with traditional, cohesionless backfill is that creep is a concern only for the reinforcement, not the soil backfill. However, RAP is possibly susceptible to creep behavior due to the viscosity of the asphaltic content in the material. It is possible that excessive creep deformations will occur in RAP backfill or at the RAP-reinforcement interface under sustained loads below failure. Such creep behavior in RAP is likely to be temperature dependent, with higher severity expected at higher temperatures, because the asphalt stiffness is temperature dependent.

Because TxDOT and FHWA do not anticipate creep behavior from the backfill material itself, they do not include any standard creep testing procedure in their specifications or design manuals. However, the creep potential of the backfill can be studied by conducting a classical creep test with a direct shear setup, where typical stress levels are applied to different specimens. Because creep behavior is temperature dependent, the creep testing program should also be arranged to run under different temperature conditions.

One more concern regarding creep characteristics is the creep pullout behavior. This type of creep mechanism is usually associated with creep deformations in polymeric reinforcing elements. Sawicki (1999) indicated that the creep of an MSE wall tends to take place in the active zone (Figure 2.5), where the soil is in the plastic state and the reinforcement is viscoelastic. For a conventional backfill, this plastic flow of the soil in the active zone is controlled by viscoelastic deformations of the reinforcement. However, in a case of reinforced RAP backfill, the RAP itself is believed to be a creep-susceptible material. Thus, it is possible that excessive deformation may occur due to creep of both the RAP and the reinforcement.

3.1.8 MSE Wall Reinforcement

The addition of horizontal reinforcements to the backfill soil produces a composite material, like reinforced concrete, which combines the best load-carrying features of both components. The characteristics for reinforcement to be used in backfill materials should include the following: 1) high tensile strength; 2) a failure mode that is not brittle; 3) a high resistance to creep; 4) a moderate amount of flexibility; 5) be economical; 6) high durability;

and 7) should develop a high shear resistance at the interface (Morris and Delphia 1999). Reinforcements for MSE walls are typically of two different types: strips and grids. These are commonly made of metal and polymers. Figure 3.1 illustrates the different types and materials used for reinforcement.

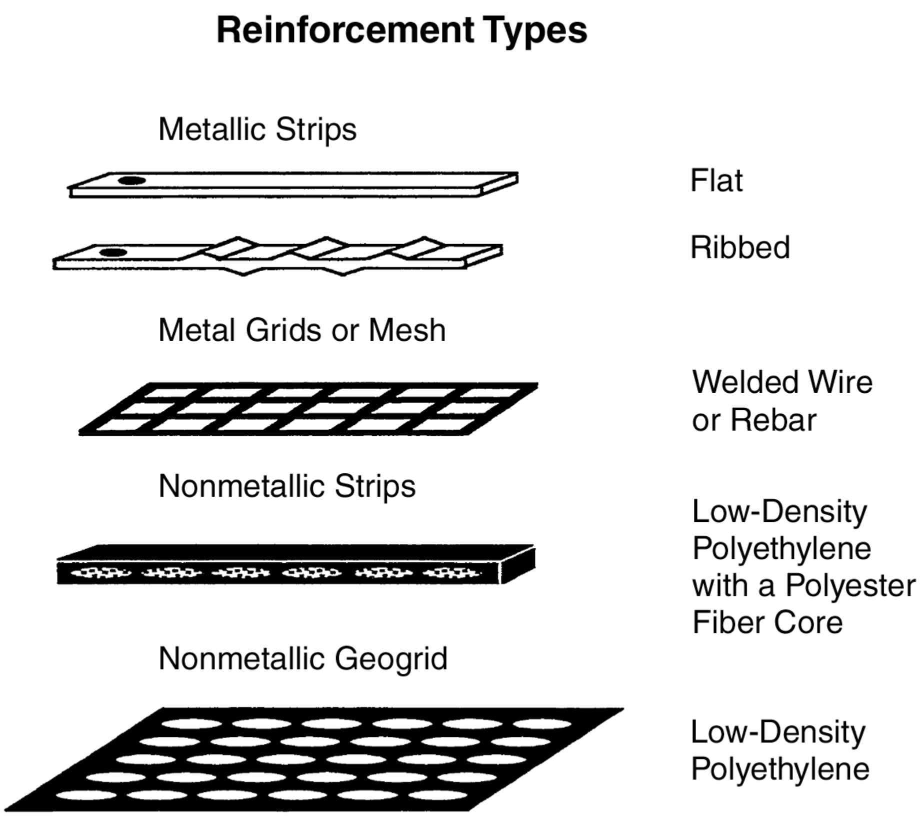


Figure 3.1 Reinforcement types (Morris and Delphia 1999)

The following is a list of the common types of reinforcements used in MSE walls (Morris and Delphia 1999):

- Galvanized ribbed and non-ribbed steel strips: These are usually 0.16 in. thick and 2 in. wide. They may have epoxy coating to reduce corrosion effects.
- Rectangular grid steel bars: The mesh is usually 24 in. x 12 in. W11 or W20 plain steel bars. 3/8 in. diameter plain steel bars on a 24 in. x 12 in. grid are also used.

- Welded wire mesh: These come in different sizes, such as 2 in. x 6 in. grid of W4.5 x W3.5, W7 x W3.5, W9.5 x W4 and W12xW5 in 8 ft wide mats. They can also be 6 in. x 24 in. mesh of W9.5 x W20.
- Non-metallic polymeric grid mat: These grids are made from high-density polyethylene or polypropylene.
- Paraweb: The 5.3 in. wide Paraweb have been used in MSE walls. They are made from high-tenacity polyester fibers and polyethylene.

3.1.9 Pullout Capacity

As mentioned previously in Chapter 2, the pullout capacity of the reinforcing element is one of the major internal stability considerations. The pullout resistance of the reinforcing element is generally provided by two mechanisms; interface friction and passive resistance. These two mechanisms contribute to pullout capacity, but one may dominate depending on the type of reinforcement. Interface friction is mobilized between soil and the horizontal surface area of the reinforcement, whereas the passive resistance is attributed to a bearing stress mechanism between the soil and transverse reinforcing components. For RAP and CC backfill, pullout testing with various reinforcement types should be performed. This information is needed to accurately determine the pullout resistance for the internal stability check in MSE wall design.

Because an ASTM standard for pullout testing is currently under development, FHWA proposed that the measurement of pullout capacity conform to GRI GG-5 (Geogrid Pullout) and GRI GT-6 (Geotextile Pullout), using the controlled strain rate method for short-term testing. For long-term pullout capacity, the constant stress (creep) method can be used. In addition, this long-term pullout test is essential for RAP, especially when subjected to loading at a relatively high temperature.

3.1.10 Summary of MSE Wall Material Specifications

The backfill soil is the key element in the satisfactory performance of an MSE wall. Table 3.3 below shows the backfill specifications required by TxDOT and FHWA. Both the TxDOT and FHWA design manuals recommend the use of cohesionless material to ensure high internal friction angle and free draining characteristics. The specifications for particle size distribution ensure that appropriate strength and free draining characteristics are achieved. Both TxDOT and FHWA permit up to 15% fines in the backfill. The required compaction methods, method Tex-114-E by TxDOT and method AASHTO T-99 by FHWA, have nearly the same compaction energy (Table 3.2). TxDOT does not have a requirement for minimum internal friction angle, while FHWA specifies a minimum (section 3.1.6). Finally, two parameters, pH and resistivity, are used to indicate the corrosion potential of the backfill. Both specifications have almost the same range of tolerable pH and resistivity values. pH and resistivity will be discussed further in the next section on durability.

Table 3.3 TxDOT and FHWA MSE wall backfill specifications

Requirement	TxDOT (Type A)	TxDOT (Type B)*	FHWA
1. Gradation			
Maximum size	3 in.	6 in.	4 in.
Percent passing sieve 3 in.	-	75 - 100	-
Percent passing sieve No. 40	0 - 60	-	0 - 60
Percent passing sieve No. 200	0 - 15	0 - 15	0 - 15
2. Plasticity Index (PI)	-	-	< 6
3. Compaction			
Dry Density	95% (Tex-114-E)		95% (AASHTO T-99)
Moisture content	$\pm$ 2% of W_{opt}		within 2% dry of W_{opt}
4. pH	5.5 – 10		5 – 10
5. Resistivity (ohm-cm)	> 3000		> 3000

Remark: * Type B backfill that does not meet the sieve No. 200 requirement may be used if:

- Less than 25% passes sieve No. 200
- PI $\leq$ 6
- At 95% dry density (Tex-114-E) and W_{opt}, $\phi \geq 34°$ (Tex-117-E)

3.2 DURABILITY ISSUES

When considering the use of CC and RAP as backfill for MSE walls, the long-term durability of the structures must be evaluated. Potential durability problems may affect the backfill itself (e.g., chemical attack on CC), or more likely, may affect the reinforcement placed in the backfill (e.g., corrosion of metallic reinforcement or degradation of synthetic reinforcement). This section will discuss some of the more important durability considerations. Potential durability problems related strictly to CC and RAP will be discussed first, followed by issues related to degradation and corrosion of MSE reinforcement.

3.2.1 Recycled Asphalt Pavements

The durability of RAP in MSE walls does not appear to be as much of a concern as with crushed concrete. Asphalt pavements are not generally attacked chemically; therefore, when crushed, they are generally free from damaging chemical compounds. Pavements subjected to de-icing salts may contain some chlorides, but not as much as in concrete (due to lower permeability and higher aggregate contents). Asphalt pavements contain about 95% aggregates, and thus RAP will be mainly composed of aggregates. Thus, the long-term durability of RAP used in MSE walls will be affected by the type and nature of aggregates present in the original pavements.

Because a small layer of asphalt will cover the aggregates contained in RAP, the properties of the original asphalt will have some effect on performance. Asphalt cements are subject to aging in pavement applications, in which the asphalt cement oxidizes, converting oils to resins and resins to asphaltenes (FHWA 1998). These conversions lead to a higher viscosity of the asphalt cement and may affect the engineering properties of RAP in MSE walls. The time-dependent changes in asphalt properties, which are accelerated in high temperature conditions, should be considered when evaluating the use of RAP in MSE walls. The creep of RAP, perhaps a very important issue, is discussed elsewhere in this report. Creep will be affected by the aging of asphalt, as well as by exposure to hot weather.

Another potential issue with RAP may be the use of RAP containing aggregates that have exhibited “stripping” or loss of asphalt adhesion. If this process continues to occur when RAP is used as a backfill, it is possible that the strength of RAP or the mechanical interaction between RAP and reinforcement will change.

3.2.2 Crushed Concrete

Crushed concrete will in most cases be a durable, sound material. However, if the original concrete contained harmful amounts of reactive aggregates (resulting in alkali-silica reaction) or sulfates (resulting in chemical sulfate attack), then the crushed product may still be subject to the same deterioration mechanism. Both alkali-silica reaction (ASR) and sulfate attack have been observed in concrete structures in Texas, so it is anticipated that structures that have exhibited damage may ultimately be removed and recycled. These deterioration mechanisms ultimately involve deleterious expansion, and such expansion could prove to be damaging in MSE applications. Water is a key instigator of these problems, and backfill with good drainage conditions may be sufficient to mitigate any potential problems. Because only limited work (if any) has been done in assessing the durability of CC when used as a backfill for MSE walls, research is needed to assess the potential issues related to using CC that previously showed poor durability in its originally intended use.

When concrete containing high concentrations of chlorides is recycled, the resulting CC will still contain appreciable amounts of chlorides. The presence of these salts may have a serious effect on metallic reinforcement corrosion, described later in this section.

3.2.3 Durability of Reinforcements in MSE Walls

Both polymeric and metallic reinforcements are used in MSE wall applications. The potential for durability problems will depend on the specific type of reinforcement, as well as the specific type of backfill. A comprehensive study was completed by Elias (1996) on the corrosion/degradation of reinforcements in MSE wall applications. However, that study focused almost exclusively on aggressive soils and not on recycled materials, such as RAP and CC. Thus, research is needed to study the durability of various reinforcement types in RAP and CC. Perhaps the largest concern with using CC is the potential for metallic reinforcement corrosion, and as such, a significant portion of this study will focus on this critical aspect.

When considering the potential degradation or corrosion of reinforcements in MSE walls using RAP or CC as backfill, all of the most commonly used reinforcements should be considered. In addition, the most important variables concerning the backfill material should be investigated, especially the effects of pH, moisture, and impurities.

3.2.4 Degradation of polymeric reinforcement

When polymeric reinforcement is used in MSE wall applications, the long-term performance is estimated based on the time-dependent loss in tensile strength of the reinforcement. It has been found that tensile capacity will be reduced in MSE wall applications due to creep, installation damage, and durability problems (Elias 1996). The response of reinforcement to these parameters will be a function of polymer type, exposure condition (e.g., backfill properties), and applied load. The majority of polymeric reinforcements used in MSE walls are composed of polypropylene (83%), with the next two most common polymers being polyester (14%) and polyethylene (2%) (Elias 1996).

The most common causes of polymeric reinforcement degradation include (Elias 1996):

1. Oxidation of polypropylene and polyethylene
2. Hydrolysis of polyester
3. Stress cracking of polyethylene
4. UV degradation
5. Biological degradation
6. General chemical dissolution

In practice, polymers are rarely affected by only one of the above mechanisms. Rather, a combination of the degradation mechanisms tends to occur. Fortunately, the polymers used in soil reinforcement are typically processed to minimize long-term degradation. For example, polymers often contain antioxidants (to minimize oxidation), stress-crack resistant materials, and UV stabilizers. Nevertheless, the long-term degradation of important polymer properties must be considered when using polymeric reinforcement for MSE walls, especially when new types of backfill, such as RAP and CC, are used.

Based on extensive research, the inherent durability of various polymeric reinforcements in different types of soils has been catalogued. Table 3.4 shows the anticipated resistance of various polymers to specific soil environments. Note that recycled materials, such as RAP and CC, have not been studied. The use of polymeric reinforcement in these new materials should be investigated.

Table 3.4 Anticipated resistance of polymers to specific soil environments (FHWA 2000; ACPA 1993; Petrarca and Galdiero 1984)

Soil Environment	Polymer Type		
	Polyester (PET)	Polyethylene (HDPE)	Polypropylene
Acid sulfate soils	No effect	Questionable Exposure tests required	Questionable Exposure tests required
Organic soils	No effect	No effect	No effect
Saline soils (pH<9)	No effect	No effect	No effect
Calcareous soils	No effect	No effect	No effect
Modified soils (lime, cement treated)	Questionable Exposure tests required	No effect	No effect
Sodic soils (pH>9)	Questionable Exposure tests required	No effect	No effect
Soils with transition metals	No effect	Questionable Exposure tests required	Questionable Exposure tests required

To test the durability of polymeric reinforcement in the laboratory, various techniques are available. To accelerate the tests, higher temperatures and aggressive environments can be used. It is important to generate data that can be used in existing design methods and that can be used to predict the service life of polymeric reinforcements in MSE walls using RAP and CC as backfill.

3.2.5 Corrosion of Metallic Reinforcement Embedded in Soil

As mentioned in Chapter 2, backfill characteristics play a large role in the corrosion of metallic reinforcement or grids. A number of factors can influence the corrosion rate of embedded metal in soils. Some of the factors associated with the soil environment that have been reported to affect corrosion of metal elements embedded in soil include:

- Resistivity
- Differential environment
- Water hardness
- Soluble salts
- Redox potential
- Texture
- Moisture content
- Dissolved oxygen
- Organic content
- Differential environment

A brief description of the influence of these factors and how they influence the corrosion of steel embedded in soils is presented next.

Resistivity – Resistivity and the pH are the most commonly used methods for estimating corrosivity. Even so, because of the synergistic effects of other factors on corrosion, to better estimate corrosion activity of metals embedded in soils or engineered backfill, these soils and engineered backfills must be well characterized. Soil resistivity is often reported as the best indicator of a corrosive soil environment. Resistivity is a measurement of how difficult it is for an electric current to flow through a material and is expressed in units of ohm-cm. Soil resistivity indicates the capability of the soil, as an electrolyte, to carry corrosion currents. As a result, a low resistivity value is indicative of high potential for corrosion, and conversely a high resistivity value indicates a lower potential for a corrosive environment.

While resistivity is recognized as a key parameter for measuring corrosion potential in soils, there is considerable variability in the criteria for resistivity as a measurement of corrosivity. This variability in resistivity limits is illustrated by the different criteria adopted by different countries as shown in Table 3.5. The current limits imposed by TxDOT and FHWA require a resistivity greater than 3,000 ohm-cm (Table 3.3). An ideal backfill material is believed to have higher resistivity values and exhibit a pH value in the permitted range, although debate continues on the reliability for determining soil corrosivity using soil resistivity values.

Table 3.5 Test criteria from different countries for galvanized steel reinforcement (Elias 1990)

Property	U.S. FHWA	France	United Kingdom	Germany
Resistivity (ohm-cm)	>3,000	>1,000 dry, >3,000 wet	>5,000	>3,000
pH	>5 & <10	>5 & <10	>6 & <10	>5 & <9
Chloride Content (ppm)	<200	<200 dry, <100 wet	<500	<50
Sulfate Content (ppm)	<1,000	<500 dry, <1,000 wet	<500	<500
Sulfide Content (ppm)	---	<300 dry, <100 wet	---	---
Organic Content (ppm)	---	100 ppm	---	---
Biochemical Oxygen Need	---	Minimal	---	---
Redox Potential (+ mV)	---	---	200-400	100-200

Resistivity is related to several other factors. The measured resistivity is influenced by the presence of soluble salts and moisture content. High concentrations of soluble salts will decrease the resistivity of the material and affect the electrochemical reactions at the metal surface.

Caution should be used when using only resistivity to assess corrosion potential. Poor correlation between soil resistivity and pH measurements with observed corrosion rates has been documented (Escalante 1989). For example, a low redox potential can indicate a microbial-induced highly corrosive environment in what would otherwise be a mildly corrosive environment based on soil resistivity data. Even so, soil resistivity and pH are the most commonly used parameters for predicting soil corrosivity.

pH **–** It is well known that the pH of a solution can influence the corrosion of metal. Whitecavage (1990) showed a direct relationship between corrosion rate (K) and pH values for various metals, as shown in Figure 3.2. For iron (Fe), Figure 3.2 shows that high pH values (greater than 10) reduce corrosion rates. Yet, most criteria limit the pH to a value below approximately 10 for MSE wall reinforcement. Thus, for plain steel, higher pH values can provide more protection against corrosion, and limiting pH may not be desirable. For zinc, Figure 3.2 depicts the corrosion rate increasing rather sharply as the pH shifts from the near neutral pH value. Current pH criteria tend to limit the pH such that the corrosion activity of zinc is minimized. Because zinc is a sacrificial anode, the zinc on galvanized steel will eventually corrode away, leaving bare steel. Steel in a high pH solution exhibits low corrosion rates. Developing materials to optimize the service-life is essential, and as such, currently imposed pH limits need further investigation.

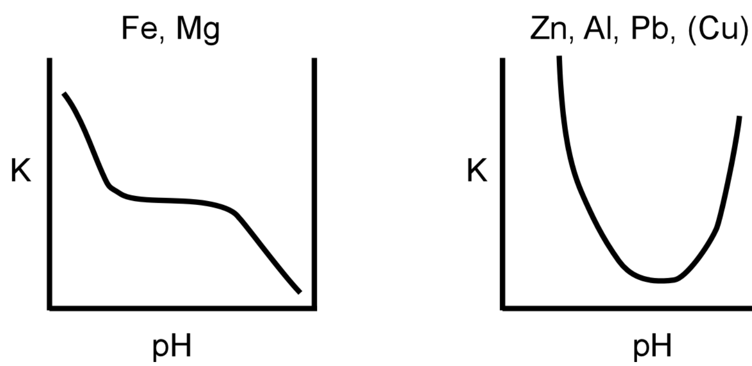


Figure 3.2 Influence of pH on the corrosion rates of various metals (Whitecavage 1990)

Water hardness **–** High water hardness (i.e., high concentrations of calcium carbonate) tends to decrease corrosion rates of steel (Moore and Hallmark 1987). Overall, this factor is believed to have a low impact on the corrosion performance of steel in soils or engineered backfill.

Soluble salts **–** As the soluble salt content increases, soil resistivity generally decreases. The presence of soluble salts decreases the resistivity of the soil and affects the electrochemical reaction at the metal surface. While resistivity measurements provide a measure of soluble salt concentration, the type of ion that reduces the resistivity is important. Certain ions have been associated with accelerated rates of corrosion. Particular ions that have been identified with high corrosivity include chlorides and sulfates (Rabeler 1989). The presence of sulfides can be an indicator of sulfate-reducing bacteria (Bushman and Mehalick 1989). The suggested limits for the presence of these two ions in the backfill is 100 ppm and 200 ppm for chlorides and sulfates, respectively (Elias 1990). As already noted in Table 3.5, various limits are used throughout the world.

Redox potential **–** The oxidation-reduction (redox) potential can provide information on the critical type of corrosion mechanism, such as anaerobic bacterial corrosion. Elias (1990) reported that a low value of redox potential could indicate susceptibility to microbial attack, while a high value indicates the presence of oxygen-supported corrosion. Anaerobic bacterial corrosion, as indicated by a low redox potential, has provided a plausible explanation for corrosion problems in soils that would otherwise be considered mildly corrosive based on soil resistivity. While redox potential measurements can be readily performed, the resulting data can be highly scattered, as the measurements are very sensitive to local soil variability and disturbance. Consequently, correlations to corrosion rates are of limited value. Limited research work has been performed in backfill applications on redox potential, especially when recycled materials are used as the backfill material.

Gradation **–** A fine-grained soil will have a low hydraulic conductivity, which will raise the moisture content of the soil and increase the possibility of stagnation. Stagnated conditions promote microbiological activity that can significantly affect corrosion rate. Coarse-grained soil typically possesses a higher hydraulic conductivity, thus providing better drainage and less aggressive conditions. Soil gradation also influences the air-water permeability of the soil. Therefore, soil gradation may be considered an indirect measure of aeration in soil, with coarse-grained sandy soils having higher air-water permeability capable of providing good aeration and fine-grained clayey soils having poor aeration. Good aeration may allow an increase in the initial corrosion rate; however, once corrosion products are formed in the soil, the corrosion products may form a protective barrier, resulting in lower corrosion activity. Localized corrosion, which is usually observed in an unaerated soil (although this is also a function of ion content), is significantly more damaging than a uniform corrosion process. Case histories (Camitz and Vinka 1989, Escalante 1989, Miller et al. 1981) appear to consistently indicate more severe corrosion problems in fine-grained clayey soils than in coarse-grained sandy soils, although in backfill applications various results have been documented.

Moisture content **–** Corrosion will not occur on metals in dry soil. Moisture is required for corrosion to occur. Soil resistivity decreases with increasing moisture content from its dry state, followed by increasing in resistivity as soil moisture is further increased. Maximum corrosion rates often occur at intermediate moisture contents, corresponding to saturation levels of about 65% (Briaud et al. 1998). At low moisture contents, there is insufficient water to support the corrosion process. At higher moisture contents, oxygen is excluded from the metal surface and corrosion rates are low. An example of this trend is given by Camitz and Vinka (1989), who report higher rates of corrosion in steels above the groundwater table, which they attribute to the availability of oxygen to support the corrosion reactions. Miller et al. (1981) reported high corrosivity values in soils with high moisture contents and soils below the groundwater table. Some of this observed behavior might have been due to corrosion mechanisms resulting from specific environmental conditions, such as microbial corrosion (Miller et al. 1981, Escalante 1989). An increase in soil moisture results in a lower presence of oxygen. The oxygen that is needed for the corrosion process must then be transferred by diffusion through the soil water. Some research indicates that below approximately 20% moisture content, the rate of oxygen transfer between the air and soil water can be quite high, and higher corrosion rates will occur (King 1977). Soil with a moisture content of 20% or more will probably suffer from uniform corrosion (assuming no chloride or sulfate ions are present), while those below 20% are more likely to endure pitting corrosion (King 1977). The moisture content providing the maximum corrosion rate varies depending on the soil characteristics.

A number of parameters are possible for describing soil moisture. Moisture content is defined as the ratio, measured on a mass basis, of free water to solid material. Due to the significance of oxygen in corrosion, a moisture measure more relevant to corrosion studies may be the degree of saturation, defined as the percentage of void (non-solid) volume occupied by water.

Dissolved oxygen **–** Metal components can be embedded in essentially undisturbed soil, disturbed soil, or recycled backfill materials (also disturbed). Studies by Romanoff (1957) indicate that corrosion is more severe in disturbed soils. This observation is supported by a number of investigators. Fischer and Bue (1981) reported that piles in undisturbed Norwegian sediments experienced very little corrosion, in spite of low soil resistivity. Escalante (1989) postulated that the diffusion of oxygen in undisturbed soils, particularly in undisturbed soils beneath the groundwater table, is sufficiently low that the corrosion process is effectively stifled. This effect tends to override the effects of the usual indicators of corrosivity (resistivity, pH, etc.). Because oxygen availability is critical to general corrosion, measurement of dissolved oxygen in the pore water could provide a meaningful parameter relevant to corrosivity.

Organic content **–** Backfill materials should not contain large amounts of deleterious materials that could attack the reinforcement or cause some distresses to the material itself. This could be a significant issue with recycled materials and care must be taken to limit the organics in these materials.

Differential environment **–** For the corrosion process to occur, an electrolyte must be present. A corroding metal must be in an environment in which surrounding elements can act as the electrolyte. A differential environment or electrolyte can affect the corrosion rate because inhomogeneities in the electrolyte can cause potential differences on a metal surface. Examples of potential inhomogeneities reported by Escalante (1989) are differences in aeration, temperature, chemical composition, and dissimilar rates of flow.

Many situations can lead to the creation of a differential environment in a backfill. Facing panels, which are important components in MSE walls, are exposed to heat and radiation from the sun and can experience frequent temperature changes. The soil behind an MSE wall is not subject to these frequent temperature changes. The temperature difference between the facing panels and the soil behind them could cause potential differences between the different parts of the reinforcement, which could result in elevated corrosion activity. In addition, soil directly behind the facing panels normally is compacted less than the soil further behind the facing panels. The select backfill behind the wall may be different from the existing soil or other fill further away from the wall; therefore a difference in potential could arise that may accelerate corrosion. The difference between the two soils may not be a concern if the soil reinforcement does not extend beyond the select backfill, which is typically the case. Drainage from nearby roadways and natural soil water movement could carry salts into the backfill, creating a chemical difference at the steel surface. Potential differences created by differential environments in an MSE wall backfill are difficult to measure, and little is known about the magnitude of their contribution to the corrosion of MSE reinforcement; consequently, further investigation is required.

3.2.6 Predicting Corrosion Rates

As described earlier, many variables affect the corrosion activity of metals underground. Corrosion activity in soils is dependent on parameters related to the metal and the soil environment. Soil environmental factors that can influence the corrosion rate of metals were described above. Metal parameters that can alter the corrosion performance include alloying elements, processing techniques, and dielectric coatings placed on the surface. These parameters can alter the corrosion mechanism, thus altering the rate of corrosion. Therefore, a description of general corrosion mechanisms common to underground metal structures are presented, followed by a discussion of current models used to predict the corrosion service life of metals.

***Corrosion mechanisms* –** A constant, regular removal rate of metal from the overall surface of a metal is defined as **uniform corrosion**. This type of corrosion is the most common and costly corrosion phenomenon and is often associated with atmospheric corrosion, but also occurs in underground structures. For uniform corrosion to occur, the metal must be metallurgically and compositionally uniform, and the exposure conditions must be such that all surfaces are exposed to the same uniform environment. In underground environments, uniform corrosion is typically not as common as other corrosion mechanisms due to variations in water levels, non-uniformity of soils and/or backfill materials, varying oxygen contents, and other factors that result in other corrosion mechanisms. This corrosion process is relatively predictable, and correlation between the calculated and actual service life can be determined.

Dissimilar alloys coupled in the presence of a corrosive electrolyte can result in the preferential corrosion of one of the alloys. When this coupling occurs, **differential, or galvanic, corrosion** can occur. Metals, alloys, and microstructural phases have unique corrosion potentials when immersed in corrosive electrolytes. When any two different metals or alloys are connected while immersed in a corrosive electrolyte, the metal or alloy with the more active (more negative) corrosion potential, E_{corr}, loses excess electrons to the less active (more positive) metal or alloy. In backfill applications, this mechanism of corrosion is common where different material types are present in the same general area. Galvanic corrosion generally results in uniform corrosion of the active metallic surface.

To extend the life of the reinforcement in MSE walls, common practice is to galvanize the reinforcement. This galvanization acts as a sacrificial anode and the process is similar to galvanic corrosion. If the thickness of the galvanization (often zinc based) and the general corrosion rate is known, the length of extended service-life from galvanizing the steel can be predicted.

In addition to uniform corrosion processes, localized corrosion processes are common in underground applications. Localized corrosion results in accelerated local attack on the metal surface and is often referred to as **pitting corrosion**. These localized areas of attack often appear to be quite small, but can severely undercut the metal and result in significant cross-section loss. Unlike uniform corrosion, pitting corrosion is very unpredictable. The rate of this process is variable and depends on the migration of deleterious substances moving into and out of the corrosion pit. Localized corrosion mechanisms are typical when the electrolyte contains chlorides, sulfates, or other salts, and is a common mechanism of deterioration for backfill applications when these ions are present.

***Predicting corrosion service life* –** Several attempts have been made at estimating the corrosion potential of soil by using parameters from the soil environment to predict the service life of underground structures. The American Water Works Association (AWWA) developed a system to determine whether or not protective action against corrosion should be taken. The system is simply the sum of weighted numbers called "points" that correlate to measures of resistivity, pH, redox potential, sulfides, and moisture levels of soil corrosivity. If these soil characteristics are known, points can be assigned for different characteristics, as shown in Table 3.6. If the sum of the points is greater than ten, the AWWA (and FHWA) suggests that protective coatings be used. One potential disadvantage of the system is that points are not added for the presence of chloride ions. The reason for the omission is the assumption that if chlorides are present, they will cause a decrease in soil resistivity, which is included in the AWWA (and FHWA) rating method. However, the corrosion effect due to the chlorides may be more aggressive than the relative decrease in resistivity that occurs.

Thus, the potential for corrosion may be underestimated when using the AWWA system if chlorides are present. The points system may also be biased because the number of points assigned for a pH of 2.0-4.0 is the same as the number assigned for a pH greater than 8.5. When Figure 3.2 is examined for the pH influence on the corrosion rate of iron for the pH ranges of 2.0-4.0 and 8.5-14, one finds that the magnitudes of the expected corrosion rates differ substantially.

Table 3.6 AWWA Rating - Standard C105-72 (Palmer 1989)

Soil Characteristics		Points
	RESISTIVITY - OHM-CM	
	(based on single probe at pipe depth or water-saturated Miller soil box)	
	<700	10
700 to	1000	8
1000 to	2000	5
1200 to	1500	2
1500 to	2000	1
	>2000	0
pH		
0.0 to	2.0	5
2.0 to	4.0	3
4.0 to	6.5	0
6.5 to	7.5	0
7.5 to	8.5	0
	>8.5	3
	REDOX POTENTIAL	
	> + 100 mV	0
+50 to	+100 mV	3.5
0 to	+ 50 mV	4
Negative		5
	SULFIDES	
Positive		3.5
Trace		2
Negative		0
	MOISTURE	
Poor drainage, continuously wet		2
Fair drainage, generally moist		1
Good drainage, generally dry		0

Statistical analysis methods have also been used to predict service life. One such method developed by Bushman and Mehalick (1989) is used to predict mean time to corrosion failure (MTCF). This study found that considerable variance occurs in the measurement of corrosion-inducing variables, and that the study of a single variable to predict MTCF would not be sufficient. To deal with this problem, a multiple regression analysis model was developed in which the MTCF is impacted by each independent variable multiplied by a coefficient representing the relative contribution of the variable to MTCF. The general form of the multiple regression analysis model developed by Bushman and Mehalick (1989) is:

$$Y = B_o + B_1 X_1 + B_2 X_2 + \ldots B_k X_k + e \tag{1}$$

where,

Y = the dependent variable (for example, MTCF in years for each tested cast iron water pipeline location),

$X_{1,2,\ldots,k}$ = each independent variable that impacts the MTCF (for example, soil resistivity, moisture content, etc.),

$B_{1,2,\ldots,k}$ = coefficient developed for each independent variable based on the relative contribution of each variable on the MTCF,

B_o = constant, and

e = random error possessing a normal probability distribution and having a mean equal to zero with a constant variance.

The difficulty in determining the coefficients for this method have limited its introduction into practice.

AASHTO and the California Department of Transportation (DOT) currently have design guidelines for evaluating the service-life for galvanized steel in soil applications. The AASHTO model requires a 75-year design life for permanent structures and provides parameters specifically for MSE structures. The California DOT method estimates the service-life of 18-gage steel with 2 ounces of zinc coating per square foot for culvert applications.

The AASHTO design method is illustrated here. For MSE wall applications meeting the following criteria:

- resistivity greater than 3,000 ohm-cm,
- 5 < pH < 10,
- organic content less than 1%,
- chloride content less than 100 ppm, and
- sulfate content less than 200 ppm

AASHTO specifies that the maximum mass presumed to be lost is:

- 15 μm/yr for the corrosion of the zinc coated during the first two years,
- 4 μm/yr until the zinc coating is depleted, and then
- 12 μm/yr for the remaining life of the structure.

Using these values, the diameter of the steel after 75 years of service can be determined, and the capacity of corroded reinforcement can be determined for the MSE wall system. If the capacity of the system after 75 years is greater than the design requirements, the proposed system is allowed. It should be noted that no reduction in pullout strength is used in the design procedure.

The California design method is illustrated here. California DOT Test Method 643 determines the time of maintenance-free service for galvanized steel culverts in soils with pH values less than 7.3 using the following equation:

$$\text{Years} = 13.79 \cdot \left[\log_{10} R - \log_{10}\left(2160 - 2490 \cdot \log_{10}(\text{pH})\right)\right] \tag{2}$$

where R is the minimum resistivity. For pH values greater than 7.3 the following equation is used:

$$\text{Years} = 1.47 \cdot R^{0.41} \tag{3}$$

In discussions with transportation personnel, mixed reviews on the applicability of the design guidelines have been expressed. The research team currently believes that other parameters should be addressed in order to better estimate the corrosion performance of these systems.

3.3 SUMMARY

Proper backfill specifications are critical to the acceptable performance of MSE walls. This chapter discussed current TxDOT and FHWA specifications for MSE wall backfill materials. The geotechnical issues related to using RAP and CC as backfill are listed below.

1. The maximum particle size in RAP and CC, as compared with current backfill gradation specifications.
2. The moisture-density compaction characteristics for RAP and CC, as compared with traditional granular backfill.
3. Further crushing of RAP and CC during field compaction.
4. The hydraulic conductivity of RAP and CC, as compared with traditional granular backfill.
5. The shear strength of RAP and CC, as compared with traditional granular backfill.
6. The potential for creep in RAP and at the RAP-reinforcement interface.

The most critical geotechnical characteristics affecting the performance of MSE walls most likely are the hydraulic conductivity of CC and the creep potential of RAP. The hydraulic conductivity of CC may be smaller than conventional backfill due to the presence of unhydrated cement that hydrates during the compaction process and reduces the pore space. The lower hydraulic conductivity could result in an accumulation of water behind MSE walls constructed with CC. Further, the additional moisture accumulating in the CC backfill may accelerate corrosion of metallic reinforcement. Creep is mainly a concern for RAP, where the viscosity of the bitumen may cause excessive deformations in MSE walls. Both the creep characteristics of the RAP itself and the reinforcement-RAP interface are a concern and will be tested as part of this research study.

The geotechnical testing encompassed in the remainder of this three-year study will focus on the issues outlined above, with particular emphasis on the hydraulic conductivity of CC and the creep potential in RAP. To fully characterize RAP and CC as backfill materials for MSE walls, various other geotechnical tests will be performed, including shear strength, compaction, and field density tests.

The durability of the backfill materials and reinforcement are essential to the satisfactory long-term performance of MSE walls. The durability of the RAP material itself appears to be a non-issue, while the durability of CC will be a function of the source concrete. If the original concrete experienced alkali-silica reaction or sulfate attack, the resulting CC backfill may have problems related to excessive expansion upon wetting. Research will be performed to study this phenomenon.

The durability of the MSE wall reinforcement is a major concern. Polymeric reinforcement is typically processed to minimize long-term degradation, but the effects of recycled materials, such as RAP and CC, on the degradation of polymeric reinforcement has not been studied. This interaction will be studied as part of this research project. The potential corrosion of metallic reinforcement is also a significant concern. The characteristics of the soil and the metallic reinforcement both affect the potential for corrosion. This research project will include corrosion experiments to define parameters that should be considered when estimating the corrosion performance of metallic reinforcement in MSE walls with RAP or CC backfill.

Chapter 4: Current Uses of RAP and CC

Over the past several years, there has been an increasing use of RAP and CC in highway construction applications throughout the United States. However, there has been less interest in utilizing RAP and CC as a backfill material compared with other applications. It appears that RAP and CC are most frequently used as an aggregate substitute for roadway construction, and good performance has been reported. This chapter discusses previous uses of RAP and CC in highway-related construction, along with recent research studies on both materials.

4.1 Current Geotechnical Uses of RAP

The recycling of asphalt pavements is not a new concept in the U.S. With the increase in the price of asphalt during the oil crisis of the early 1970s, the recycling of asphalt pavements became a feasible way of lowering highway construction costs (Ahmad 1991, 1992). It has been estimated that as much as 33 million metric tons, about 80% of the excess asphalt presently generated, is being used either as a portion of recycled hot mix asphalt, in cold mixes, or as aggregate in granular or stabilized base materials (FHWA 2000). RAP has been used in many highway construction applications, including as an aggregate substitute and asphalt supplement in recycled asphalt paving granular base or subbase, a stabilized base aggregate, or as backfill. Based on TxDOT Report 1272 (Estakhri and Button 1992), TxDOT has successfully used untreated RAP in highway applications. These applications include paving driveway and country road approaches, paving mailbox and litter barrel turnouts, and repairing pavement edges.

According to current TxDOT data (TxDOT 1999b), more than 90% of RAP construction projects in Texas used RAP for paving purposes. The other projects used RAP as the backfill material for embankment construction. More detail on these embankment projects is shown in Table 4.1. The long-term performance of these embankments has been satisfactory, with no collapse or noticeable distress observed.

Because RAP has typically been used as a paving material in highway construction, the majority of past research projects on RAP focused on its potential as a paving material. Experimental programs typically focused on the mechanical properties used in pavement design, such as resilient modulus, modulus of elasticity, and fatigue characteristics. Experimental data from past research have indicated that RAP offers comparable performance, in terms of a paving material, to virgin aggregates (Kennedy et al. 1977; Maher et al. 1997). Also, research has indicated that recycled pavements offer the same durability as pavements constructed with 100% virgin aggregates (TxDOT 1999b).

Table 4.1 TxDOT projects using RAP as backfill (after TxDOT 1999b)

District	Location	Installed	TxDOT Specification	Result	Comments
Austin	Travis County	1995	NA	Unknown	
Beaumont	Liberty	1987	NA	Good	
Beaumont	Jasper	1987	NA	Excellent	
Bryan	SH-21 at Brazos River	1996	Material # 132	Excellent	Mixed soils with sized RAP
El Paso	El Paso	1993	Material # 132	Unknown	Used as a stabilizer for shoulder surface

The most recent research study on RAP sponsored by TxDOT is Project 1348 completed by Saeed et al. (1995, 1996). These studies focused on the potential use of waste and recycled materials in roadbase construction. Eight recycled materials were evaluated in this study including RAP, reclaimed Portland cement concrete pavement (RPCP), iron blast furnace slag, steel slag, coal ashes, building rubble, glass, and rubber tires. Each recycled material was evaluated separately and four evaluation criteria were used: technical, economic, societal, and environmental. For each of these criteria, a different method for assigning a score was devised. The scores for each criterion were combined in a normalizing equation to indicate the potential use of the material in terms of the "waste recycled material utilization potential" (WRMUP). The study rated RAP as the most suitable material for roadbase (Table 4.2), mostly due to its high availability in most local TxDOT districts. The report indicates that about 90% (19 out of 21) of the responding districts have stockpiles of old asphaltic concrete and further concludes that only RAP and RPCP are stockpiled in sufficient volume by TxDOT to make their use economically attractive. More detail regarding how the scores were assigned to each material can be found in TxDOT Report 1348 (Saeed et al. 1995, 1996).

Table 4.2 Categorization of Waste Recycled Materials as Roadbase Construction Aggregate (after Saeed et al. 1996)

Material	WRMUP (%)	Remarks
Reclaimed Asphalt Concrete	68.60	Best materials
Electric Arc Furnace Steel Slag	60.40	Suitable material
Standard crushed limestone roadbase	58.85	For comparison only
Reclaimed Portland cement concrete	50.20	Marginal
Fly ash	45.20	Unsuitable as aggregate in roadbase construction
Bottom ash	45.20	
Pond ash	45.00	

Note: WRMUP = Waste Recycled Material Utilization Potential

In accounting for the creep effect in RAP, Ayoub (1983) studied the long-term behavior of softening agents on cold process recycled asphalt pavement. A series of nondestructive creep experiments was set up to evaluate such long-term behavior. At the end of the experimental program, the initial and long-term behavior of recycled mixtures were reported and compared with other virgin specimens that had different properties. According to this study, no significant difference was found between the creep behavior of virgin and recycled mixtures. The test results showed that creep decreased rapidly at early ages because of hardening of emulsified asphalt, then increased slightly due to the effects of the softening agent. Temperature had a major effect on the creep of the recycled mixture. Higher temperatures (140°F) resulted in more creep in the recycled mixture than in the virgin mixture, but no significant difference was observed at a lower temperature (75°F).

FHWA (2000) indicates that at least five states (Connecticut, California, Illinois, Louisiana, and Tennessee) have used RAP directly as a backfill material, while some other states have used RAP as an additive in embankment construction. The performance of RAP in these applications was generally considered as satisfactory to good (FHWA 2000). When used as an embankment or fill material, the undersize portion of RAP (smaller than 2 in.) was sometimes blended with soil and/or finely graded aggregate. RAP with larger particles was usually used as an embankment base. The required construction procedures for a RAP embankment (i.e., material storage, field compaction, quality control, design considerations) are generally the same as the procedures used for conventional embankments. However, FHWA (2000) describes a few specific recommendations regarding construction procedures for RAP embankments.

1. Random sampling and testing of the RAP stockpile must be performed because various sources of RAP may be different.
2. Additional attention must take place during compaction to ensure that no poorly compacted zones are created in the fill, which could lead to long-term differential settlement.
3. Some jurisdictions may require a minimum separation distance between water sources and fill materials containing RAP to avoid submersion of RAP in water, because water leaching from RAP may be a potential environmental concern.

4.2 CURRENT GEOTECHNICAL USES OF CC

Crushed concrete (CC) has been used successfully in highway construction since the 1940s. Laboratory research on recycled concrete was first carried out in Europe and the USSR shortly after World War II (Halm 1980). The considerable amount of CC produced by bombing and shelling during the war was used in rebuilding urban areas. In the U.S., the majority of CC was generated through the demolition of Portland cement concrete elements in roads and buildings. The major application of CC in the U.S. is as an aggregate substitute in pavement construction. This practice has become so common that CC aggregate is considered by many agencies as conventional aggregate. Using CC as a backfill material has apparently gained the lowest interest compared to other applications. However, it is reportedly one of the first waste materials considered for backfill applications (FHWA 2000).

Several research studies have evaluated the potential uses of CC. Significant attention focused on the suitability of CC as aggregate for structural concrete in buildings or paving structures. As a result, the scope of the laboratory tests in most of the recent research has focused on the material characteristics of CC for such applications. Previous research indicated that concrete made with CC aggregate had comparable performance to concrete made with virgin aggregate (Cuttell et al. 1997, Barksdale et al. 1992, ACPA 1993). A summary of CC research described in the literature is given below.

Physical properties **–** FHWA (2000) indicates that CC is more angular in shape, has lower specific gravity, and has higher water absorption than comparatively sized virgin aggregate. The specific gravity of CC ranges between 2.0 and 2.5, while the water absorption varies from 2 to 8%, depending on the size of the CC particles (Table 4.3). The low value of specific gravity is attributed to the addition of mortar from the original concrete structure. As seen in Table 4.3, fine CC particles have lower specific gravity than coarse CC particles because more mortar is found in the finer part of the material. Higher water absorption is expected because mortar is more absorbent than natural aggregate (ACPA 1993, Rashwan and Abourizk 1997, O'Mohany 1997, Mack and Solberg 1993).

Table 4.3 Physical properties of crushed concrete material (from ACPA 1993)

Property	Coarse Particles	Fine Particles
Specific gravity	2.2 to 2.5	2.0 to 2.3
Water absorption (%)	2 to 6	4 to 8

Mechanical properties **–** It has been reported that for pavement construction, concrete made with CC aggregate generally exhibits lower compressive and flexural strengths than concrete mixed with natural aggregate (ACPA 1993; Malhotra 1978). Malhotra (1978) reports compressive strengths up to 30% lower and flexural strengths up to 20% lower. Further minor compressive strength reduction will likely occur when the recycled-aggregate mix also contains recycled fines, because a significant portion of recycled fines is mortar from the concrete. ACPA (1993) further indicated that the majority of strength loss is attributed to material smaller than 0.08 in. (2 mm).

Other researchers have focused on the shear strength of unbound CC for geotechnical purposes. O'Mahony and Milligan (1991) indicated that although CC had lower dry density, the shear strength of CC was as high as that of limestone (Figure 4.1). The researchers interpret the experimental results to conclude that vertical stress has little influence on the friction angle of the CC over the range of stress applied (Figure 4.2). However, Figure 4.2 seems to show a slight decrease in friction angle with vertical stress, as commonly observed for granular materials.

Chemical properties **–** The pH of a CC-water mixture often exceeds 11. The high alkalinity of CC can cause corrosion of aluminum or galvanized steel pipes that are in direct contact with CC and in the presence of moisture (FHWA 2000). Moreover, CC may be contaminated with chloride ions, due to the application of deicing salts, or with sulfates, due to contact with sulfate-rich soils. The presence of sulfate is also linked to CC obtained from buildings, which is likely to contain calcium sulfates from plaster or gypsum wallboard (Buck 1973). Chloride ions are associated with the corrosion of steel, while sulfate reactions lead to expansive disintegration of cement paste (FHWA 2000). However, ACPA (1993) indicates that the quantity of chloride typically found in old concrete pavement is below the critical threshold values of 0.03 to 0.09%. When aluminum is present within the CC, such as a conduit pipe surrounded by CC backfill, the high pH of the CC can cause accelerated corrosion and formation of expansion products and hydrogen gas (Barksdale et al. 1992).

The first TxDOT project that used CC as aggregate in new pavement was in the Houston district. There was no virgin aggregate used in this project, meaning that both coarse and fine aggregates were from recycled concrete. The important findings concerning CC performance from this project are listed below (TxDOT 1999a).

1. There was no distress found in the pavement section utilizing 100% recycled coarse and fine aggregate.
2. The large amount of old mortar in recycled concrete did not appear to have an adverse effect on the new pavement.
3. Moisture in the CC played a major role in producing consistent and workable concrete.
4. The use of both recycled coarse and fine aggregate reduced the modulus of elasticity of the pavement significantly.
5. Recycled coarse aggregate has a much higher thermal coefficient than virgin aggregate, due to the attached old mortar.

TxDOT has reported three projects that used CC as backfill material. Table 4.4 provides information on these three projects.

As mentioned earlier, the major concern when using CC as a backfill material in MSE walls is the potential corrosion of metallic reinforcements. This assumption is drawn from the hypothesis that the high pH of a crushed concrete-water mixture will increase the rate of steel corrosion. Popova et al. (1998) studied the corrosive behavior of crushed concrete for potential use as a backfill material in MSE walls. For a galvanized steel rod embedded in fill material, the rates of corrosion at the beginning of the test were the same for both crushed concrete and granular soil fill (approximately 0.02 mm/year). However, the rate of corrosion increased with time for the CC material (0.075 mm/year at 400 days), while it decreased for the case of granular soil fill (0.005 mm/year at 400 days).

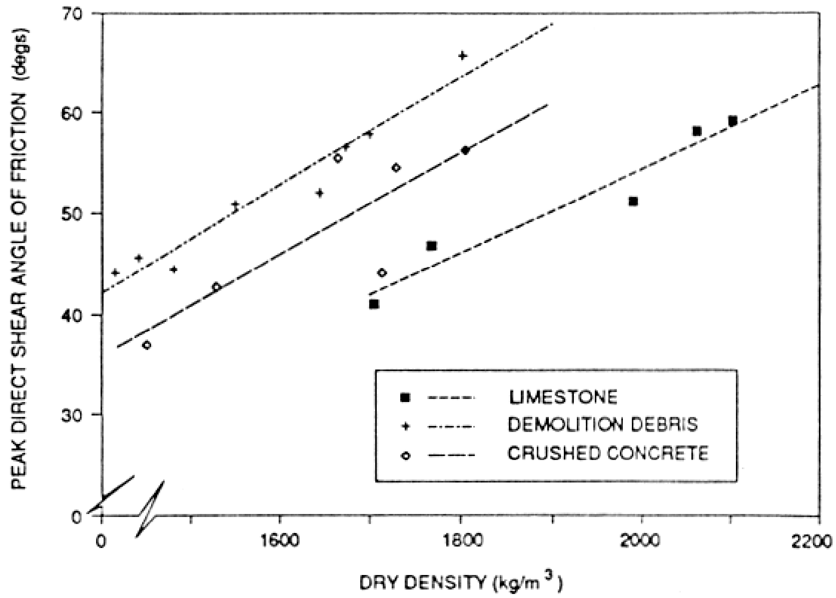


Figure 4.1 Influence of dry density on peak direct shear angle of friction (from O'Mahony and Milligan 1991)

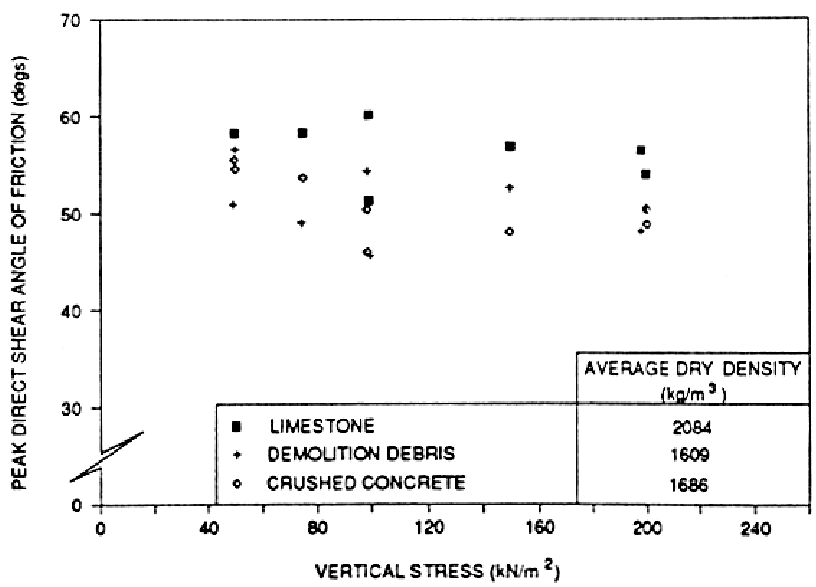


Figure 4.2 Influence of vertical stress on peak direct shear angle of friction (from O'Mahony and Milligan 1991)

Table 4.4 TxDOT projects using CC as backfill material (after TxDOT 1999a)

TxDOT District	Location	Results	Installed	Specification	Comments
Corpus Christi	Various	Excellent	1977	132	Used for embankment and outfall erosion protection
Lufkin	District wide	Excellent	1982	--	--
Beaumont	SH 82, SH 87	Good	1994	None	Used for embankment to control erosion on intercoastal waterway.

Popova et al. (1998) also studied the potential use of cement as a soil stabilization agent and found that the corrosion rates of crushed concrete and cement-stabilized granular soil fill are almost identical when both contain the same cement content. Further, this study indicated that the rate of corrosion of a galvanized steel rod in a cement-stabilized CC mixture were comparable to, and possibly slightly better than, a sample embedded in a cement-stabilized granular fill. The average values of the rate of corrosion in cement stabilized CC were in the range of 0.005 mm/year to 0.02 mm/year, which are in the range of the commonly accepted target of 0.01 mm/year, given a design life of around 100 years (Popova et al. 1998).

Another concern for CC backfill is precipitation of calcium carbonate contained in the leachate from CC. The calcium carbonate precipitates, called tufa, are formed through a series of chemical reactions. This problem was brought to light because significant CC fines have been observed to clog up filter fabric wrapped around subsurface drains in CC pavement subbases (Barksdale et al. 1992; Mack et al. 1993). Gupta et al. (1993) conducted a research study on different pavement aggregates and suggested that free lime (CaO) is responsible for producing tufa. Gupta et al. (1994) proposed a series of chemical reactions that leads to the formation of tufa, as shown below.

1. Free lime reacts with rainwater and forms calcium hydroxide [$Ca(OH)_2$]

$$CaO + H_2O \rightarrow Ca(OH)_2$$

2. Carbon dioxide (CO_2) from the atmosphere and autmobile exhaust reacts with rainwater, forming carbonic acid (H_2CO_3).

$$CO_2 + H_2O \rightarrow H_2CO_3$$

3. Carbonic acid reacts with calcium hydroxide forming calcium bicarbonate [$Ca(HCO_3)_2$].

$$2H_2CO_3 + Ca(OH)_2 \leftrightarrow Ca(HCO_3)_2 + H_2O$$

4. At the drainage outlet, water from this enriched solution of calcium bicarbonate evaporates because of warm temperatures, and the carbon dioxide escapes into the atmosphere. This condition leads to the precipitation of calcium carbonate ($CaCO_3$) and the formation of tufa.

$$Ca(HCO_3)_2 \leftrightarrow CaCO_3 (\downarrow) + H_2O (\uparrow) + CO_2 (\uparrow)$$

In the last chemical reaction, warm temperatures in the summer months increase the rate of deposition of tufa, whereas cold temperatures in the winter months cause the CO_2 to remain in solution. The preceding chemical reactions clearly indicate that the concentration of free lime, water, carbon dioxide, temperature, and humidity are the major parameters that control tufa precipitation. According to the research findings, CC containing both calcium hydroxide [$Ca(OH)_2$] and calcium bicarbonate [$Ca(HCO_3)_2$] can produce tufa. Therefore, the presence of free lime or calcium hydroxide in the cement paste of concrete can result in tufa precipitation (Gupta et al. 1993).

To control tufa precipitation, washing of CC aggregates is required by some agencies to remove the dust that typically contains free lime and calcium hydroxide. FHWA recommends using suitable CC that does not contain significant quantities of unhydrated cement or free lime for embankment or fill applications. Also, leachate testing may be required to obtain the tufa precipitate potential of CC for MSE wall applications.

4.3 SUMMARY

RAP and CC have been used increasingly in transportation-related projects. RAP has been used mostly as a paving material in highway construction, and the TxDOT projects that have used RAP in new pavements have exhibited good performance. RAP was rated highly by Saeed et al. (1996) as a potential material for roadbase construction, and other states have used RAP in embankment construction with favorable results. However, some researchers have reported excessive creep deformations in RAP, particularly at elevated temperatures.

CC has been used extensively as an aggregate substitute in pavement construction and structural concrete. In these applications, CC appears to have performed satisfactorily. However, reductions in strength and stiffness of new concrete constructed with CC aggregate have been reported. TxDOT has used CC as fill for embankment construction and has reported satisfactory performance. When used as a pavement subbase, CC has not performed as well. Several researchers have reported that filter fabric surrounding subsurface drains has clogged due to the precipitation of calcium carbonate from CC. Clogging of filter fabric is a significant concern for MSE walls because these walls often rely on drainage through filter fabric placed across joints between facing panels. The precipitation of calcium carbonate will be studied as part of this research project.

Chapter 5: Characterization of Materials Used in MSE Walls in Texas

5.1 INTRODUCTION AND SCOPE

This chapter gives information about the work done for characterization of MSE wall materials in Texas and the procedure for selection of material sources for further research. The approaches used for source selection and material testing are outlined in the following paragraphs.

There are many RAP and CC producers in Texas; therefore, it is important to consider different plants and locations in the characterization of these materials. Based on the surveys sent to commercial producers and TxDOT districts, major recycled RAP and CC producers were identified (e.g., Corpus Christi TxDOT district, Big City Crushed Concrete in Dallas, Southern Crushed Concrete in Houston). These major producers of RAP and CC were asked to periodically sample their product over a period of two months for characterization at the University of Texas and Texas A&M University. Characterization tests included gradation, Atterberg limits, specific gravity, pH, and resistivity. These are typical tests required by TxDOT and FHWA specifications for MSE wall backfill. The characterization of the various RAP and CC samples were used to evaluate the variation in properties between producers and over time.

After the initial characterization study was complete, a screening procedure was used to choose a single RAP producer and a single CC producer to provide bulk material for the remainder of the research project. Additionally, a producer of a conventional fill material was chosen to provide bulk material for comparison with the RAP and CC.

5.2 SURVEY RESULTS

5.2.1 Tables of Survey Results

An investigation was performed to identify sources of RAP and CC recycled materials. TxDOT was helpful in providing additional information on TxDOT districts and commercial producers producing and utilizing these materials. A list of districts and companies involved in the production of RAP and CC was prepared and surveys were sent to each source. Tables 5.1 through 5.4 summarize the information collected from the surveys of TxDOT districts and commercial producers.

Table 5.1 TxDOT District Survey Results

TxDOT District	Product	Production (Tons/Annum)	Tons/Annum Next 5 yrs	Stockpile	Sources of Raw Materials
Abilene	RAP	33,184	35,000	YES	NA
Amarillo	RAP	100,000	100,000	YES	Pavements
Atlanta	RAP	25,000	100,000???	YES	???
Austin	RAP	10,000	10,000	???	NA
Childress	NA	35,000	40,000	YES	Pavements
Corpus Christi	RAP	59K-118K	59K-118K	YES	NA
Dallas	RAP/CC	???	???	YES	Pavements/
					Commercial
Fort Worth	RAP/CC	50,000:10,000	50,000:10,000	YES	Pavements
Houston	NA	Not known	Not known	YES	Pavements
Laredo	NA	NA	NA	NA	NA
Lubbock	RAP	10,000	10,000	NO	NA
Lufkin	RAP	12,000	12,000	YES	NA
Odessa	RAP	???	???	YES	NA
Paris	RAP	No estimate	No estimate	???	Pavements
Tyler	RAP	100,000	100,000	YES	NA
Waco	RAP	100,000	100,000	YES	NA
Wichita Falls	RAP	20,000	20,000	YES	NA
Yoakum	RAP	15,000	15,000	YES	NA

Table 5.2 TxDOT District Survey Results

TxDOT District	Applications for Products	Maximum Size (in.)	Processing Recycled Material	Physical/ Chemical Tests	Research Cooperation
Abilene	Pvmt, edge repair, recycle into a mix	2	Milling in place	NO	???
Amarillo	Pavement	3	Milling in place	Asphalt Content, Gradation	???
Atlanta	Geotechnical	3	Milling in place	AC, Pen Test, DSR	???
Austin	Pavement	???	Milling in place	NO	NO
Childress	Pavement	2	Crushing, Milling in place, Screening	NO	YES
C. Christi	Pavement	1	Milling in place	NO	YES
Dallas	RAP-pavement, CC-flexible base	2 RAP, 1.75 CC	Crushing, Milling in place, Screening	TEX 528C, 502C, 211F,117E	YES
Fort Worth	Pavement	2	Crushing, Milling in place, Screening	Gradation, Decant	YES
Houston	Stab base, Conc., Asphalt mix	2 RAP, 1.75 CC	Milling in place	PI, LL, Gradation	YES
Laredo	Pavement	1	Milling in place	NO	YES
Lubbock	Pavement	1	Milling in place	NO	YES
Lufkin	Pavement, Backfill	2	Milling in place	YES	YES
Odessa	Backfill pvmt. edges and slopes	2	Milling in place	Phy. prop. of asphalt in RAP	???
Paris	Backfill pvmt. edge & private drives	???	Milling in place	NO	NO
Tyler	Pavement, Geotechnical	1	Milling in place	???	YES
Waco	Pavement/ Backfill pvmt. edges	2	Milling in place	Gradation	YES
Wichita Falls	Base material	???	Milling in place	NO	???
Yoakum	Pavement, Backfill	2.5	Milling in place	NO	???

Table 5.3 Commercial Producer Survey Results

Company	Location	Product	Production (Tons/Annum)	Tons/Annum Next 5 yrs	Stockpile	Sources of Raw Materials
Amarillo Road Co.	Amarillo	RAP/CC	???	30,000	NO	Pavements 95%, Old structures 5%
Arbuckle Materials Inc.	Edmond, OK	???	???	???	???	???
Archer Western Cont.	Arlington	RAP	???	50,000	NO	Pavements
Big City Crushed Concrete, Inc.	Dallas	RAP/CC	1,000,000	1,000,000	YES	Pavements, Bldgs/Structs, Ready Mix Conc.
Cherokee Bridge & Road, Inc.	Junction	NA	NA	NA	NA	NA
Foremost Paving Inc.	Weslaco	RAP	50,000	50,000	YES	Pavements
Frontera Materials Inc.	???	RAP/CC	20,000	10,000	YES	90% Pavement,5% Buildings, 5% Pipes
Fuller & Sons	Amarillo	???	???	???	???	???
Holms Construction Co., Inc.	Amarillo	RAP/CC	100,000	100,000	YES	Pavements, Concrete paving and Sidewalks
J.H. Strain & Sons, Inc.	TYE	???	250,000	100-200K	NO	???
Jobe Concrete Products	El Paso	RAP/CC	200,000	Over 200,000	YES	Pavements
Southern Crushed Concrete	Houston	RAP/CC	2,000,000	Over 2,000,000	YES	Pavements/Buildings
Stringtown Materials L.P.	Oklahoma	NA	NA	NA	NA	NA
Valero Refining	Houston	CC	50	100	NO	Pavements, Unit Bases
Williams Brothers	Houston	???	???	???	???	???
Zack Burkett Co.	Jacksboro	NA	NA	NA	NA	NA

Table 5.4 Commercial Producer Survey Results

Company	Applications for Products	Maximum Size (in)	Processing Recycled Material	Physical/ Chemical Tests	Research Cooperation
Amarillo Road Co.	Pavement	2	Crushing/Milling in place	Sieve analysis only	YES
Arbuckle Materials Inc.	???	???	???	???	???
Archer Western Cont.	Pavement	1.5	Milling in place	NO	YES
Big City Crushed Concrete, Inc.	Pavement/Geotechnical	3**	Crushing/Screening	PH & Resistivity	YES
Cherokee Bridge & Road, Inc.	NA	NA	NA	NA	YES
Foremost Paving Inc.	Pavement	2	Milling in place	NO	YES
Frontera Materials Inc.	Structural/Pavement	2	Crushing/Milling in place/	Triaxle, PI,	YES
			Screening	Asphalt Gradation	
Fuller & Sons	???	???	???	???	???
Holms Construction Co., Inc.	Pavement	3	Crushing/Milling in place &	NO	YES
			Screening		
J.H. Strain & Sons, Inc.	Pavement	2	Milling in place	NO	NO
Jobe Concrete Products	Pavements	2	Crushing	Gradation,PI, LL	YES
Southern Crushed Concrete	Structures/Pavement/	TxDot #247	Crushing/Milling in place	YES	YES
	Geotechnical				
Stringtown Materials L.P.	NA	NA	NA	NA	NA
Valero Refining	Structures/Pavement	???	Dig it up and send to	YES	NO
			Southern C.C		
Williams Brothers	???	???	???	???	???
Zack Burkett Co.	NA	NA	NA	NA	NA

5.2.2 Survey Description

For each commercial producer and district, information was requested about the type of recycled material produced and annual production capacity. The survey also requested information about the estimated annual production over the next five years and the existence of stockpiles at their locations. This information would be indicative of the direction the commercial producers and districts are taking in the production and utilization of recycled materials.

The sources of the recycled materials and the typical uses for these products were also requested, as well as the maximum particle size of the recycled material produced. The maximum particle size was important because TxDOT specifies a maximum particle size for backfill materials used in MSE walls. Other information requested included: the methods used for processing recycled materials, physical and chemical tests typically performed, and whether the commercial producer and districts would be willing to participate in the research project.

5.2.3 Key Findings from the Survey

The survey results provided important information regarding RAP and CC production across the state of Texas. The reported annual production of RAP from TxDOT districts is shown in Figure 5.1. Generally, most TxDOT districts produce RAP. Only a few districts produce both RAP and CC (Table 5.1). The maximum annual production of RAP from the districts surveyed was about 100,000 tons. Four districts produce this amount of RAP (i.e., Amarillo, Corpus Christi, Tyler, and Waco). The Corpus Christi district produces between 59,000 and 118,000 tons of RAP each year (Table 5.1). The Corpus Christi district recycles numerous asphalt pavements per year and the uses of the RAP produced has been a major concern for the district. The districts that responded to the survey had an average annual RAP production of about 46,000 tons.

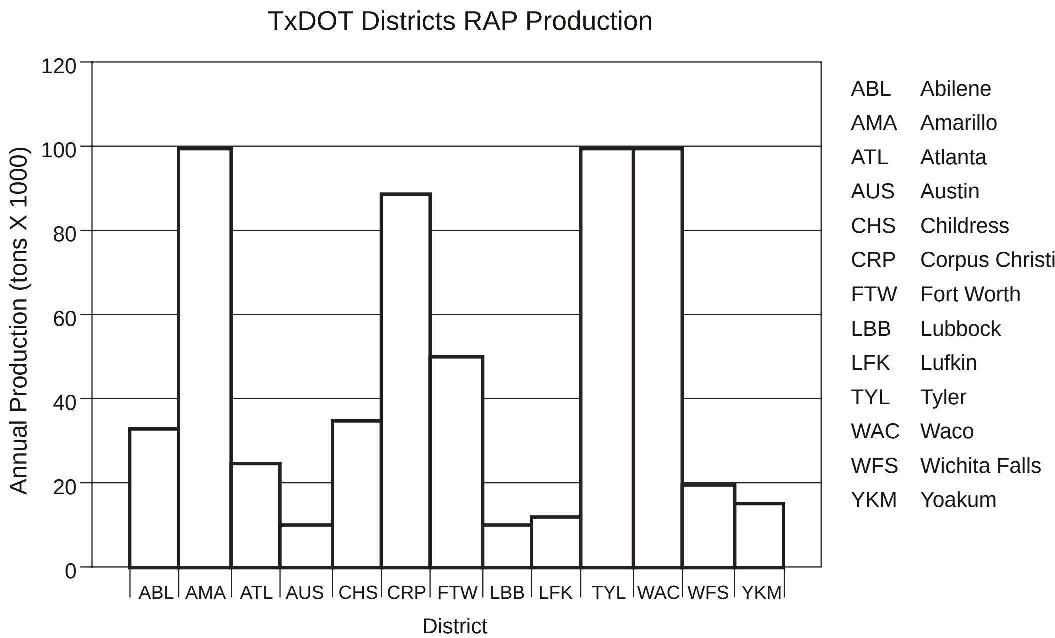


Figure 5.1 TxDOT district RAP production

The survey responses from commercial producers varied somewhat from the responses from TxDOT districts. The commercial producer tended to produce both RAP and CC. The reported annual production of RAP and CC from commercial producers is shown in Figure 5.2. Southern Crushed Concrete was the biggest producer of both RAP and CC, producing about 2,000,000 tons/annum. Big City Crushed Concrete produces about 1,000,000 tons/annum (Figure 5.2).

The results from the survey indicate that the main uses of RAP and CC are pavement construction and fill for embankments. A final observation from the survey is that the majority of districts and commercial producers responded “yes” when asked about their interest in research cooperation. This positive response is an indication of the potential for the use of these recycled materials in the state of Texas.

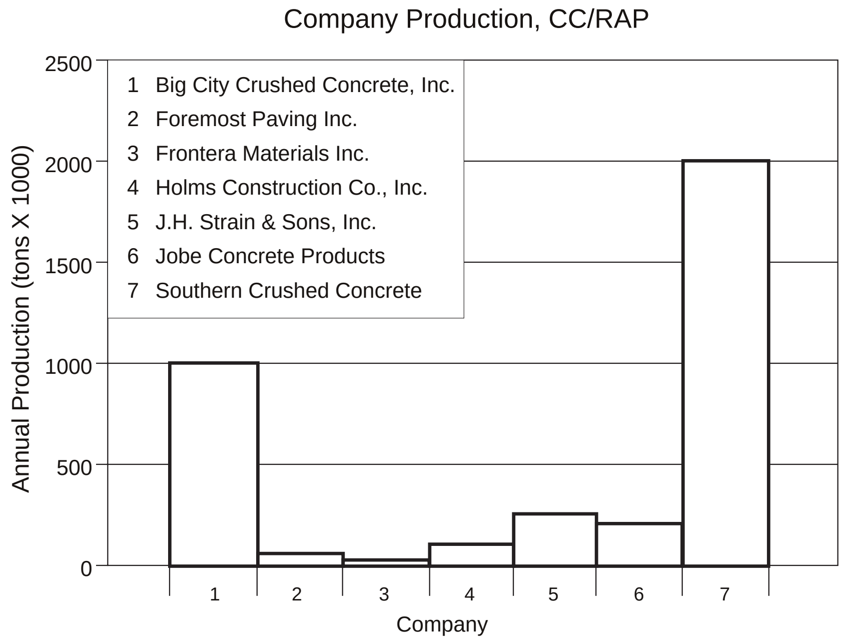


Figure 5.2 Commercial Producer RAP and CC Production

5.2.4 Use of Survey Results to Select Sources for Initial Characterization Tests

All commercial producers and districts that did not respond to the survey were assumed to have no significant interest in the research project at this moment. Each source was contacted to ensure that all surveys reached their intended destination, to avoid excluding sources that did not receive the survey. Sixteen commercial producers responded by completing and returning the surveys. Eighteen responses were received from the twenty-five TxDOT districts surveyed.

The annual production capacity was one of the main considerations for source selection for the initial characterization tests. Big City Crushed Concrete (BC) and Southern Crushed Concrete (SCC) each produced over 1,000,000 tones of RAP and CC per year. These two companies also showed significant interest in the research and were willing to send samples for testing. Therefore, BC and SCC were selected as a source of recycled material for the initial characterization tests. SCC supplied both RAP and CC samples, while BC only supplied CC samples.

The choice of TxDOT district to be used in the project was heavily influenced by the need expressed for RAP research by the Corpus Christi district. Corpus Christi has numerous asphalt pavements that are recycled annually. The volume of this production is expected to grow with time. Given this background, RAP from Corpus Christi was selected for initial characterization tests. Samples from four separate stockpiles within the district were used in the initial characterization study.

The question about research cooperation was also used to identify sources to be considered for the research project. Consequently, any company or district that said “NO” to this question was excluded from the research.

5.3 LABORATORY TEST RESULTS

5.3.1 Introduction

Corpus Christi, BC, and SCC were asked to periodically sample their products over a period of two months and ship these samples to the University of Texas and Texas A&M University for testing. Several tests were performed to characterize the RAP and CC samples received from Corpus Christi, BC, and SCC. Characterization tests included gradation, Atterberg limits, specific gravity, pH, and resistivity. The purpose of the testing was to determine the variation in properties and characteristics of material sampled from different producers and at different times. The following sections provide detailed information regarding the results from these tests.

5.3.2 Grain Size Distribution

The grain size distribution of RAP and CC samples were evaluated in accordance with TxDOT test method Tex-401-A. Grain size distribution curves were plotted and compared with the gradation requirements for MSE walls, as indicated in the TxDOT Standard Specifications for Construction of Highways and Bridges Item 423.2.

Tables 5.5 through 5.9 list the results from sieve tests on RAP samples from SCC and Corpus Christi. The gradation curves for each sample are shown in Figure 5.3 and are compared with the TxDOT specifications for MSE walls. All RAP samples fell within the specified envelope for MSE walls. It was also observed that all of the RAP samples tested contained less than 1% fines (i.e., material passing the #200 sieve), which is much smaller than the allowable maximum of 15%. The RAP from Corpus Christi had more large particles than the RAP from SCC. However, the RAP from SCC may have been scalped during processing to remove large particles. For each source, the gradation curves were similar over different sampling dates.

Table 5.10 lists the results from sieve tests on CC samples from BC and SCC. The gradation curves for each sample are shown in Figure 5.4 and are compared with the TxDOT specifications for MSE walls. Again, all of the CC samples tested were observed to fall within the specified envelope for MSE walls. The gradation curves from each source are very similar, and gradation curves from different dates are also similar. It was also observed that almost all of the CC samples tested contained less than 1% fines (i.e. material passing the #200 sieve), which is much smaller than the allowable maximum of 15%.

Table 5.5 Gradation of RAP from SCC

		RAP Percent Passing		
Sieve No.	Sieve Size (mm)	SCC RAP (12/07/00)	SCC RAP (12/11/00)	SCC RAP (12/18/00)
6	150.00	100.0	100.0	100.0
3	75.00	100.0	100.0	100.0
1 1/2	37.50	100.0	100.0	100.0
1	25.00	89.3	87.2	75.5
3/4	18.75	70.2	59.4	54.0
1/2	12.50	54.3	39.9	35.9
3/8	9.38	46.6	31.6	28.6
4	4.75	32.2	18.1	18.4
8	2.36	22.9	12.5	13.1
16	1.18	16.1	9.1	9.6
30	0.60	10.8	6.5	6.6
40	0.43	8.1	5.0	4.9
50	0.30	5.8	3.6	3.3
100	0.15	2.4	1.7	1.6
200	0.08	0.8	0.5	0.5

Table 5.6 Gradation of RAP from Corpus Christi

		RAP Percent Passing			
Sieve No.	Sieve Size (mm)	C. Christi 1-4 (11/15-16/00)	C. Christi 4-4 (11/30/00)	C. Christi 4-4 (12/13/00)	C. Christi 1-4 (01/09/01)
6	150.00	100.0	100.0	100.0	100.0
3	75.00	100.0	100.0	100.0	100.0
1 1/2	37.50	91.9	91.2	87.9	87.3
1	25.00	81.2	74.2	81.2	82.4
3/4	18.75	74.5	68.2	75.3	77.2
1/2	12.50	67.0	59.5	69.4	70.9
3/8	9.38	59.8	50.9	61.2	64.6
4	4.75	36.3	27.3	37.9	42.8
8	2.36	22.7	14.6	23.9	27.1
16	1.18	15.1	8.8	16.8	18.3
30	0.60	10.1	5.5	11.8	12.9
40	0.43	7.6	4.3	5.9	10.1
50	0.30	4.4	3.0	2.6	6.6
100	0.15	1.3	1.0	0.4	1.9
200	0.08	0.2	0.2	0.0	0.4
Sampling Location: US-77 & IH-37 int, E side of N. US-77					

Table 5.7 Gradation of RAP from Corpus Christi

		RAP Percent Passing			
Sieve No.	Sieve Size (mm)	C. Christi 2-4 (11/15-16/00)	C. Christi 1-4 (11/30/00)	C. Christi 2-4 (12/13/00)	C. Christi 3-4 (01/09/01)
6	150.00	100.0	100.0	100.0	100.0
3	75.00	100.0	100.0	100.0	100.0
1 1/2	37.50	100.0	100.0	93.9	95.9
1	25.00	98.4	95.4	87.5	85.2
3/4	18.75	90.7	86.8	76.9	76.0
1/2	12.50	79.2	71.1	63.9	64.3
3/8	9.38	68.8	58.4	53.8	52.2
4	4.75	44.0	33.3	31.1	30.5
8	2.36	31.4	21.5	18.8	19.8
16	1.18	21.9	15.3	12.3	13.5
30	0.60	12.4	9.9	7.6	8.0
40	0.43	7.1	6.7	5.4	5.0
50	0.30	3.7	4.1	3.2	2.9
100	0.15	0.6	1.0	0.7	0.7
200	0.08	0.1	0.2	0.1	0.1
Sampling Location: US-77 & SH-239 int, W. side of N. US-77, stockpile N. of SH-239					

Table 5.8 Gradation of RAP from Corpus Christi

		RAP Percent Passing			
Sieve No.	Sieve Size (mm)	C.Christi 3-4 (11/15-16/00)	C.Christi 2-4 (11/30/00)	C.Christi 3-4 (12/13/00)	C.Christi 4-4 (01/09/01)
6	150.00	100.0	100.0	100.0	100.0
3	75.00	100.0	100.0	100.0	84.9
1 1/2	37.50	75.4	88.2	87.0	80.6
1	25.00	72.2	79.4	78.6	73.9
3/4	18.75	64.4	73.3	70.5	67.8
1/2	12.50	56.3	65.5	61.1	60.2
3/8	9.38	48.8	57.2	53.0	53.3
4	4.75	27.9	33.6	31.7	31.8
8	2.36	16.7	20.7	17.9	19.9
16	1.18	11.6	14.8	11.1	14.0
30	0.60	8.5	11.0	7.0	10.6
40	0.43	7.0	8.0	5.3	8.7
50	0.30	4.9	5.4	3.7	6.3
100	0.15	1.7	1.6	1.3	2.1
200	0.08	0.4	0.1	0.2	0.5
Sampling Location SH-286 near line P					

Table 5.9 Gradation of RAP from Corpus Christi

		RAP Percent Passing			
Sieve No.	Sieve Size (mm)	C.Christi 4-4 (11/15-16/00)	C.Christi 3-4 (11/30/00)	C.Christi 1-4 (12/13/00)	C.Christi 2-4 (01/09/01)
6	150.00	100.0	100.0	100.0	100.0
3	75.00	100.0	100.0	100.0	100.0
1 1/2	37.50	100.0	96.2	91.5	95.3
1	25.00	99.1	92.8	88.1	89.2
3/4	18.75	93.4	90.7	83.4	84.0
1/2	12.50	84.1	84.3	77.3	76.5
3/8	9.38	72.9	73.9	68.3	66.6
4	4.75	37.8	41.4	38.2	36.7
8	2.36	18.8	22.4	20.6	20.5
16	1.18	10.1	13.4	12.8	12.5
30	0.60	5.1	8.1	8.2	8.0
40	0.43	3.3	6.0	6.2	6.0
50	0.30	1.8	4.1	4.2	4.1
100	0.15	0.5	1.5	1.3	1.5
200	0.08	0.1	0.3	0.2	0.3
Sampling Location: US-77 near rest area, N.of Aransas River Bridge E side of N.US-77					

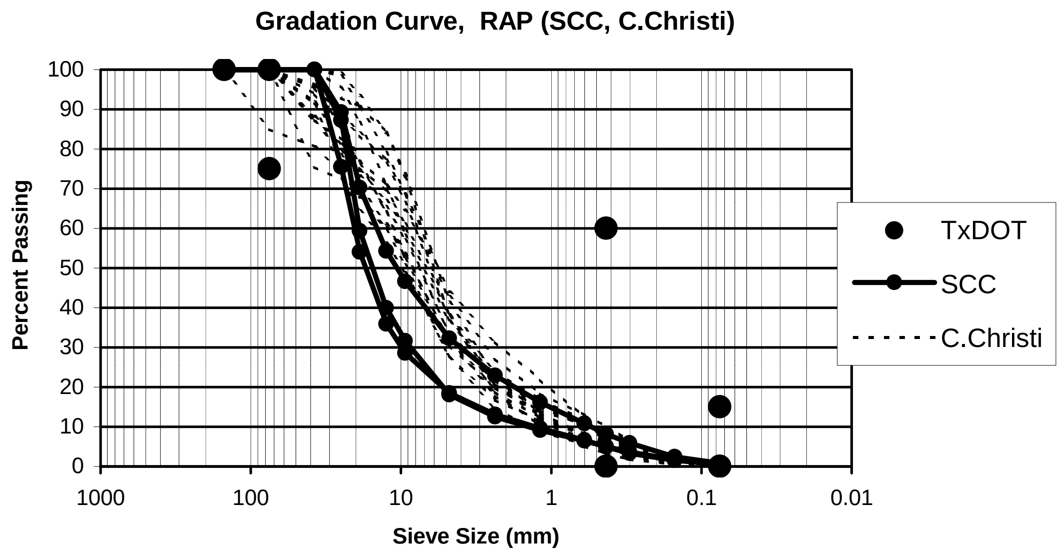


Figure 5.3 Gradation curves for RAP samples from SCC and Corpus Christi

Table 5.10 Gradation of CC from BC and SCC

					Crushed Concrete Percent Passing			
Sieve No.	Sieve Size (mm)	BC CC (11/27/00)	BC CC 01/12/01	BC CC 01/19/01	BC CC 01/26/01	SCC CC (12/07/00)	SCC CC (12/11/00)	SCC CC (12/18/00)
6	150.00	100.0	100.0	100.0	100.0	100.0	100.0	100.0
3	75.00	100.0	100.0	100.0	100.0	100.0	100.0	100.0
1 1/2	37.50	100.0	100.0	100.0	99.0	100.0	100.0	100.0
1	25.00	90.0	89.2	83.6	84.2	89.3	87.2	75.5
3/4	18.75	77.1	76.1	67.3	73.2	70.2	59.4	54.0
1/2	12.50	66.2	63.9	50.6	55.6	54.3	39.9	35.9
3/8	9.38	58.3	53.8	41.0	45.3	46.6	31.6	28.6
4	4.75	41.4	36.8	26.3	27.7	32.2	18.1	18.4
8	2.36	30.5	26.2	18.5	18.9	22.9	12.5	13.1
16	1.18	23.0	19.0	13.5	13.5	16.1	9.1	9.6
30	0.60	16.7	13.3	9.6	9.4	10.8	6.5	6.6
40	0.43	13.4	9.9	7.2	7.2	8.1	5.0	4.9
50	0.30	8.6	6.2	4.6	4.7	5.8	3.6	3.3
100	0.15	3.7	2.2	1.7	1.7	2.4	1.7	1.6
200	0.08	1.2	0.7	0.5	0.5	0.8	0.5	0.5

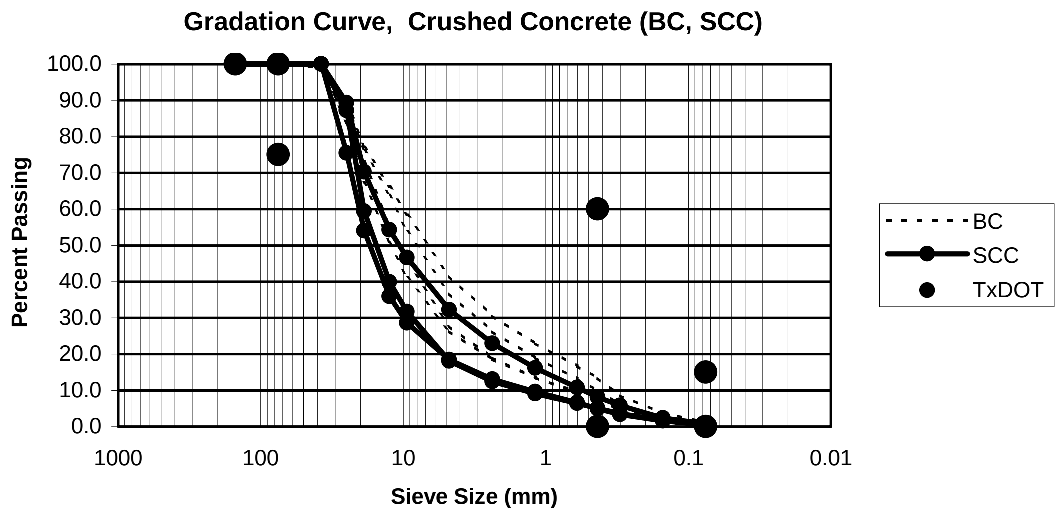


Figure 5.4 Gradation curves for CC samples from BC and SCC

5.3.3 Atterberg Limits

The purpose of the test was to measure the liquid limit (LL) and plastic limit (PL) of the RAP and CC samples. These results are used to calculate the plasticity index (PI, where PI = LL – PL) of the samples. TxDOT specifies a PI of less than or equal to 6 for backfill materials for MSE walls. The tests were performed in accordance with TxDOT method Tex-104-E, which specifies testing only the fraction of the material passing the No. 40 sieve. The LL was determined for each sample, but the PL could not be determined for any sample because the material was non-plastic. Therefore, the PI could not be calculated for these materials. Because RAP and CC are non-plastic, they meet the specification regarding PI.

Table 5.11 provides a summary of the LL test results. The measured values of LL were averaged for each source, as well as across the entire data set for RAP and CC. The RAP has an average LL of 23, with a standard deviation of 3. Similarly, CC has an average LL of 31, with a standard deviation of 4. The results indicate that the materials exhibit similar characteristics.

Table 5.11 Summary of Atterberg Limits

Summary of LL		Std. Dev.
BC CC	32	1
SCC RAP	18	0
SCC CC	29	6
C.Christi-A	23	1
C.Christi-B	22	4
C.Christi-C	22	3
C.Christi-D	26	3
CC	31	4
RAP	23	3
Nb. All samples non-plastic (NP)		

5.3.4 Specific Gravity

The specific gravity of the RAP and CC samples was measured using test method ASTM D854. Table 5.12 provides a summary of the measured values of Gs. The measured values of Gs were averaged for each source, as well as across the entire data set for RAP and CC. The average Gs for RAP was measured as 2.28 with a standard deviation of 0.07. Similarly, the Gs for CC was found to be 2.62, with a standard deviation of 0.04. The results indicate similarity between the samples from different sources over the testing period.

Table 5.12 Summary of Specific Gravity

Summary of SG		Std. Dev.
BC CC	**2.64**	**0.02**
SCC RAP	**2.40**	**0.06**
SCC CC	**2.59**	**0.04**
C. Christi-A	**2.26**	**0.07**
C. Christi-B	**2.29**	**0.03**
C. Christi-C	**2.23**	**0.05**
C. Christi-D	**2.24**	**0.03**
CC	2.62	0.04
RAP	2.28	0.07

5.3.5 pH

ASTM Designation 4972-95a (Standard Test Method for pH of Soils) was used to measure the pH of the CC and RAP from the various suppliers. This method determines the solubility of soil minerals and ion mobility in the soil. Note that another test method for measuring pH in soil, ASTM G51-95 (Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing), also exists. ASTM D4972-95a was chosen by the research team for testing because California DOT Test 643 uses a similar procedure to determine the pH in soil. The objective is to use the measured pH in the service-life equation formulated by the California DOT. ASTM G51-95 measures pH without adding water to the soil, whereas water is added to create a soil-water slurry for the measurement of pH using ASTM 4972-95a.

Results from the pH testing, summarized in Table 5.13, indicate a pH of 12.4 for crushed concrete from BC and SCC, 8.0 for RAP from Corpus Christi, and 8.4 for RAP from SCC. The pH level of the RAP meets the current TxDOT and FHWA backfill specification for MSE walls (Table 3.3). The CC samples obtained from BC and SCC do not meet the current pH specification set by TxDOT and FHWA for MSE walls. However, it is expected that the high pH will be protective for uncracked steel products.

Table 5.13 Summary of pH test results

Abbreviation	pH	Std. Dev.	Sample Description
BC CC	12.34	0.79	Big City Crushed Concrete, CC
SCC CC	12.40	0.10	Southern Crushed Concrete, CC
SCC RAP	8.41	0.21	Southern Crushed Concrete, RAP
C. Christi-A	8.09	0.09	TxDOT Corpus Christi District, Site A, RAP
C. Christi-B	8.07	0.18	TxDOT Corpus Christi District, Site B, RAP
C. Christi-C	8.09	0.15	TxDOT Corpus Christi District, Site C, RAP
C. Christi-D	8.09	0.19	TxDOT Corpus Christi District, Site D, RAP

5.3.6 Resistivity

Measurements of the minimum soil resistivity of each of the RAP and CC samples collected was made according to the procedure outlined in California DOT Test 643. The samples were each placed in a soil box for determining the resistivity. The soil box test setup is similar to that of Figure 2 found in ASTM Designation G57-95a.

Minimum resistivity measurements of the RAP and CC are summarized in Table 5.14. Minimum resistivity values of 760 ohm-cm were determined for the CC supplied by BC and SCC, with values in the range of 2,640-4,840 ohm-cm for RAP. All of the RAP samples from the TxDOT Corpus Christi sites met the current MSE wall TxDOT and FHWA backfill specification for resistivity with the exception of Site B. In addition to not meeting the current MSE TxDOT and FHWA backfill specification for pH, the CC samples obtained from BC and SCC also do not meet the requirements for resistivity. It should be noted that further testing is needed because it has been well established that the corrosion rates for uncoated steel decrease with increasing pH. It is believed that the current pH limitation may be more applicable for galvanized steel.

Table 5.14 Summary of resistivity test results

Abbreviation	Resistivity ohms-cm	Std. Dev.	Sample Description
BC CC	760	50	Big City Crushed Concrete, CC
SCC CC	760	500	Southern Crushed Concrete, CC
SCC RAP	3750	1380	Southern Crushed Concrete, RAP
C. Christi-A	3160	800	TxDOT Corpus Christi District, Site A, RAP
C. Christi-B	2640	970	TxDOT Corpus Christi District, Site B, RAP
C. Christi-C	3780	1470	TxDOT Corpus Christi District, Site C, RAP
C. Christi-D	4830	1650	TxDOT Corpus Christi District, Site D, RAP

5.3.7 Predicting Corrosion Performance

Based on resistivity and pH measurements of the CC and RAP samples obtained from Big City Crushed Concrete (BC) in Dallas, Southern Crushed Concrete (SCC) in Houston, and the TxDOT District in Corpus Christi, the research team determined the potential corrosivity values of the materials. The California DOT Test Method 643 was used to determine these values. Figure 5.5 shows results from the assigned points from ASTM A 674-95 (Standard Practice for Polyethylene Encasement for Ductile Iron Pipe for Water or Other Liquids). Figures 5.6 and 5.7 show the predicted service life values for the CC and RAP, respectively.

Both methods indicate that the CC may be corrosive and that the RAP is most likely not as corrosive. It should be noted that these estimates are for galvanized steel samples and that these rates are specifically related to the performance of zinc materials in these environments (pH and resistivities). Comparison of Figures 5.6 and 5.7 indicates that galvanized steel in RAP could have approximately twice the predicted service life of galvanized steel in CC. Because these results are specifically for zinc and zinc-based products, the applicability of these results to uncoated steel (i.e., no galvanization), or to galvanized steel where the galvanized coating has been removed as a result of active corrosion, may not be applicable. Actual material corrosion performance testing is needed to validate or invalidate these predictions.

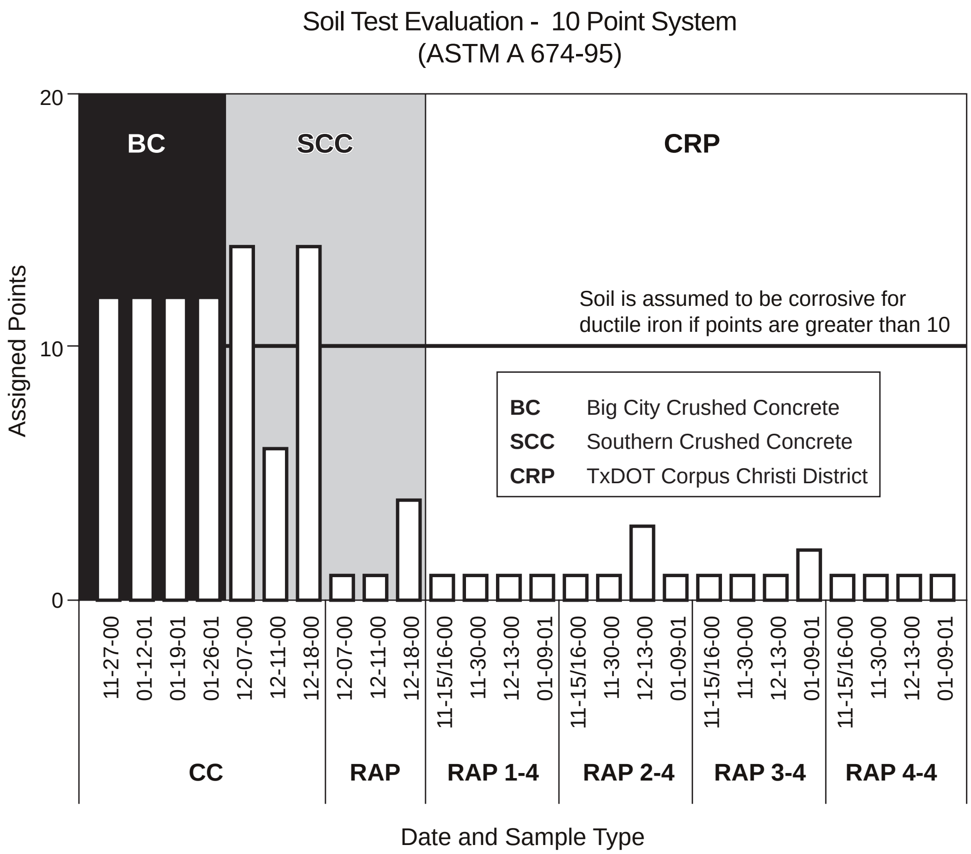


Figure 5.5 Calculated ranking for CC and RAP

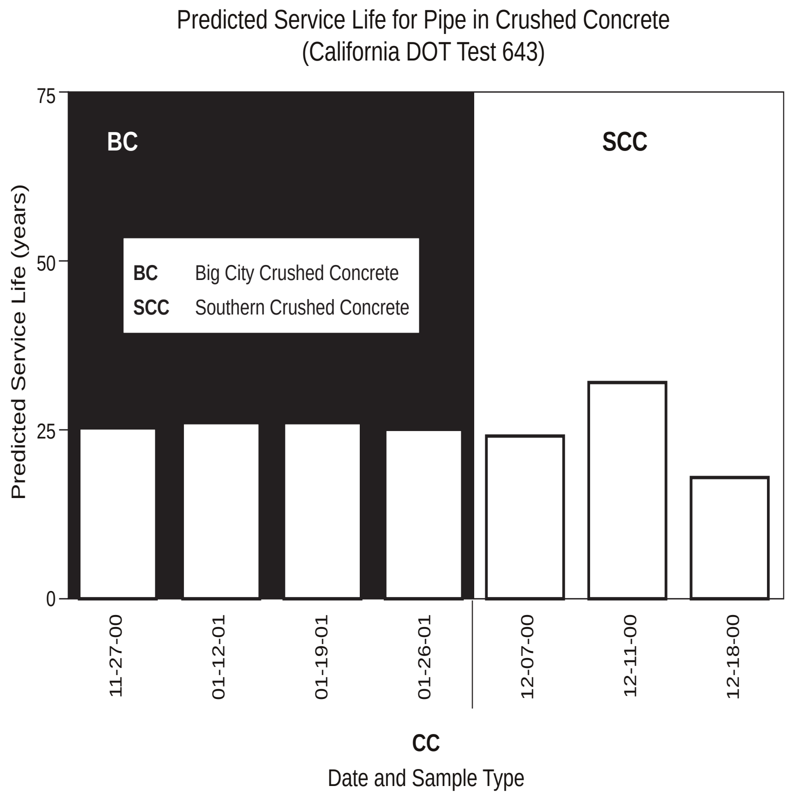


Figure 5.6 Predicted service life for pipe in CC materials

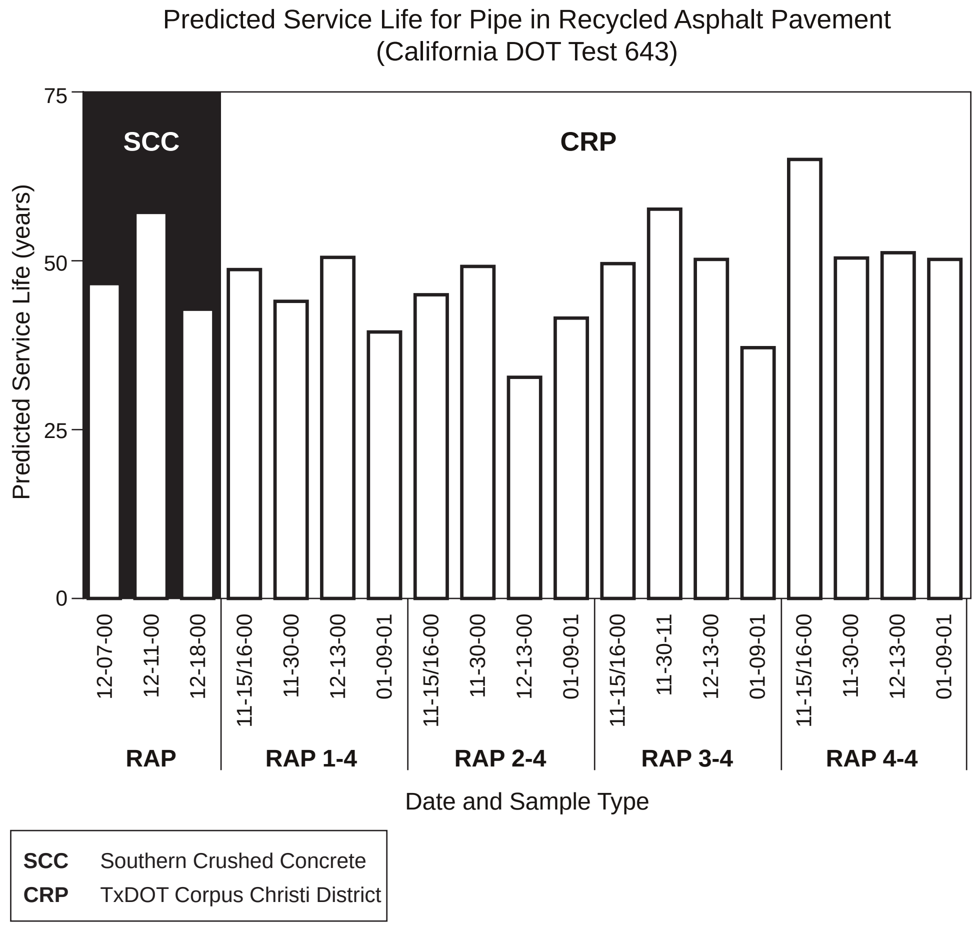


Figure 5.7 Predicted service life for pipe in RAP materials

5.4 SCREENING PROGRAM FOR RAP AND CC

It was important to select one source of RAP and one source of CC to be used as a representative recycled material for further research and investigation. Additionally, a producer of conventional fill material used in MSE walls was identified to provide material for comparison. The initial characterization tests indicated that the recycled materials from different sources had similar characteristics. Three important aspects were considered in the screening program for selection of the RAP and CC sources: (1) annual production from the survey results, (2) initial characterization test results, and (3) research cooperation from the districts and commercial producers.

Corpus Christi district was selected as the source of RAP for further research to establish performance in MSE walls. The district is one of the leading producers of RAP, producing 59,000 to 118,000 tons/annum. The initial characterization test results indicate that the gradation, plasticity, pH, and resistivity of RAP samples meet the TxDOT specifications for MSE walls. Additionally, Corpus Christi expressed a great need for research on the uses of RAP because of the growing amounts of RAP in the district.

Big City Crushed Concrete was selected as the source of CC for further research. The company has shown great willingness and cooperation in providing information and samples for the research. The survey results clearly indicate that Big City is a major producer of both CC and RAP in Texas, with an annual production of 1,000,000 tons/annum. Similarly, the results from the characterization tests also show that the gradation and plasticity of CC samples meet the TxDOT specifications for MSE walls. However, the CC samples from all suppliers did not meet the current TxDOT specifications for pH and resistivity. It should be noted that these limits are specifically for galvanized steels. Corrosion activity could be significantly reduced for ungalvanized steel products in a high pH environment, such as that provided by crushed concrete pore water. Further investigations are required to establish limits for galvanized and plain steel for various conditions.

After the initial characterization of RAP and CC, it was decided that a conventional fill material (CFM) should be selected for comparison with the recycled materials. Texas Crushed Stone, a local supplier located in Georgetown, Texas, was chosen as an economical supplier of CFM. This material is produced by crushing quarried limestone.

5.5 REINFORCEMENT

Information was gathered from TxDOT districts and companies regarding the types and configurations of reinforcement commonly used in MSE walls in Texas. The type of reinforcement used is specified by the wall manufacturer in the shop drawings. A listing of the types of reinforcement often specified is given in Morris and Delphia (1999). This information is summarized in Table 5.15. There are two main types of reinforcement for MSE walls: strips and grids. Either type may be made of metals or polymers, so it is important to study the performance and durability of both metallic and polymeric reinforcements in MSE walls.

The Corpus Christi TxDOT district indicated that galvanized metallic flat strap (strip) reinforcement is commonly used in the district. A welded grid system has also been used in the past (mail correspondence March 5, 2001). Given that RAP from the district has been selected for further investigation, we propose to use these galvanized metallic strips in future geotechnical and durability testing. Based on the information gathered in Table 5.13, the Reinforced Earth Company will be used as the provider of the galvanized metallic strip reinforcement.

The Tensar Corporation is a company that provides reinforcement to TxDOT. Tensar provided information regarding the different types of polymeric reinforcement used in MSE walls. Earlier telephone and mail correspondence show the willingness of the company to participate in the research project. We therefore propose to use Tensar as a source of polymeric reinforcement to be used in future geotechnical and durability testing.

Table 5.15 Types of MSE wall reinforcement (Morris and Delphia 1999)

Company Name	Reinforcement Detail
The Reinforced Earth Company	Galvanized ribbed and non-ribbed steel strips, 0.16 in thick; 2 in wide.
VSL Corporation	Rectangular grid plain steel bars (W11 or W20), 24 in x 6 in grid
The Hilfiker Company	Welded wire mesh, 2 in x 2 in grid Welded wire mesh, 6 in x 24 in grid
The Tensar Corporation	Polymeric grid mat
DOT California	Rectangular grid plain steel bars 3/8 in diameter,
DOT Georgia	Rectangular grid plain steel bars 3/8 in diameter,
Soil Structures Int. Ltd.	Paraweb 135mm x (5.3 in), high tenacity polyester

5.6 *RECOMMENDATIONS*

The recommendations for RAP, CC, CFM, and reinforcement to be used for further research are based on discussions presented in earlier sections of this chapter.

5.6.1 Recycled Asphalt Pavement

Corpus Christi district will be used as the source of RAP for further geotechnical testing and durability performance evaluation in the project. The initial characterization of RAP samples indicated that the gradation, plasticity, pH, and resistivity of RAP meet the current TxDOT specifications for MSE walls.

5.6.2 Crushed Concrete

Big City Crushed Concrete in Dallas has been identified as the source of CC for further geotechnical and durability testing in the project. The initial characterization of CC samples indicated that the gradation and plasticity of CC meet the current TxDOT specifications for MSE walls, but the pH and resistivity do not.

5.6.3 Conventional Fill Material

Texas Crushed Stone in Georgetown, Texas, has been identified as the source of the CFM for further geotechnical and durability testing in the project. Initial characterization of the CFM, which consists of crushed limestone, has not been carried out yet.

5.6.4 Reinforcement

The Reinforced Earth Company will be used as the supplier of galvanized ribbed steel strip reinforcement for the project. The Tensar Corporation will be used as the supplier of polymeric grid mat reinforcement for the project.

Chapter 6: Basic Material Characteristics of Bulk Samples of RAP, CC, and CFM

This chapter presents the results from tests performed to evaluate the basic geotechnical characteristics of the bulk samples of crushed concrete (CC), recycled asphalt pavement (RAP), and a conventional fill material (CFM) obtained from the suppliers indicated in Chapter 5. The tests performed include material gradation, specific gravity, and compaction.

Bulk samples of CC and RAP were obtained in May 2001 and a bulk sample of the CFM was acquired in August 2001. Approximately 20 tons of RAP and CC were received from the suppliers indicated in Chapter 5, while 10 tons of the CFM were obtained. The bulk material is stockpiled at the Pickle Research Center in Austin, Texas. These stockpiles will be the source of material for the entire experimental program.

6.1 Grain Size Distribution

Grain size distribution was evaluated through sieve tests conducted in accordance with the ASTM D422 test method. Sieves ranging in size from 75 mm (3 in.) to 0.075 mm (0.0029 in.) were used. Sieve tests were performed on test samples taken from the stockpiled material at the Pickle Research Center.

6.1.1 Crushed Concrete (CC)

Crushed concrete was provided by Big City Crushed Concrete, a crushed concrete supplier located in Dallas, Texas. This company was selected as our crushed concrete supplier because they are one of the major crushed concrete suppliers in Texas and have been involved in a number of projects involving crushed concrete material (Chapter 5).

Figure 6.1 illustrates the grain size distribution of CC samples taken from four locations in the stockpile. Figure 6.1 indicates that the grain size distribution of the CC is relatively uniform over the stockpile. Less than 5% of the material is larger than 40 mm (1.57 in.), and there are no particles larger than 75 mm (3 in.). Approximately 10% of the material passes the No. 40 sieve (0.425 mm), but no fines pass the No. 200 sieve (0.075 mm). The USCS classification of this material is poorly graded gravel (GP). Based on the sieve analysis, the bulk CC satisfies both the TxDOT and FHWA gradation specifications for MSE walls.

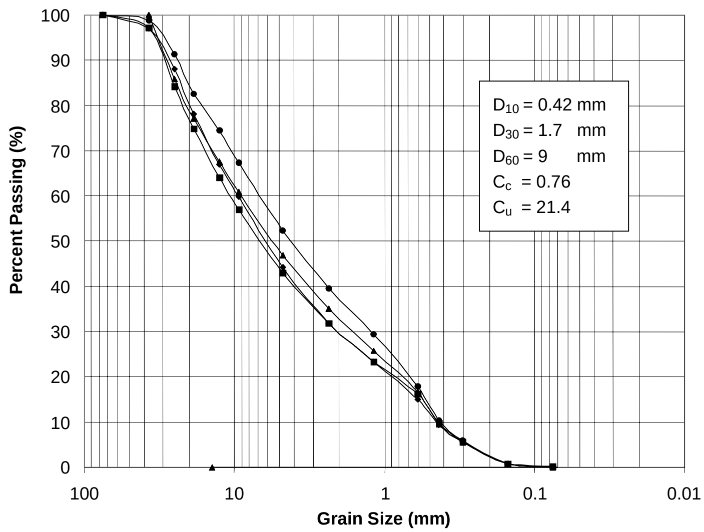


Figure 6.1 Grain size distribution of CC

6.1.2 Recycled Asphalt Pavement (RAP)

RAP was supplied by the Corpus Christi TxDOT district. The Corpus Christi district was chosen as the RAP supplier because of their large stockpiles of RAP.

Figure 6.2 shows the grain size distribution of RAP taken from four locations in the stockpile at the Pickle Research Center. Figure 6.2 shows that the grain size distribution of the RAP is very uniform over the stockpile. Similar to CC, less than 5% of the material is larger than 40 mm (1.57 in.), and no particles are larger than 75 mm (3 in.). Only 2% of the material passes the No. 40 sieve (0.425 mm) and there are no fines passing the No. 200 sieve (0.075 mm). The USCS classification of this material is well graded gravel (GW). Based on the sieve analysis, RAP is also an appropriate material for MSE walls based on its gradation.

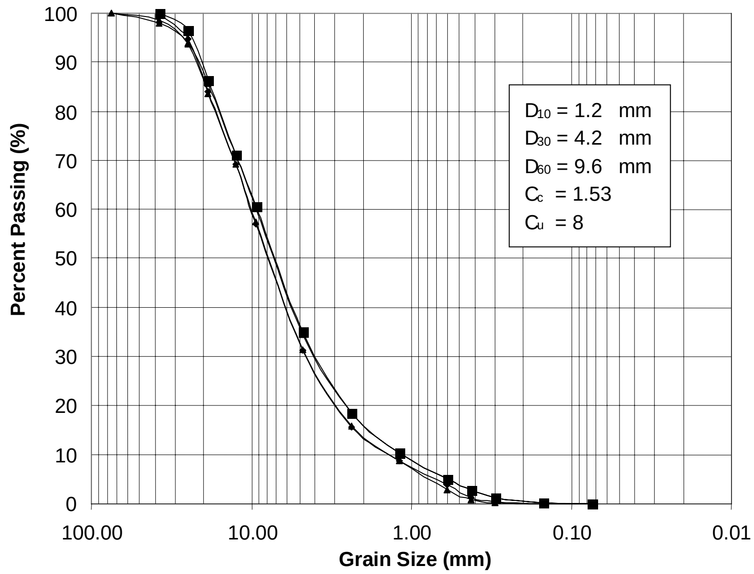


Figure 6.2 Grain size distribution of RAP

6.1.3 Conventional Fill Material (CFM)

The conventional fill material (CFM) was provided by Texas Crushed Stone, a local supplier located in Georgetown, Texas. This conventional fill material consists of crushed limestone, and is similar to materials that have been used as backfill for a number of MSE walls in Texas.

The results from four sieve tests are shown in Figure 6.3. The results in Figure 6.3 show that the grain size distribution of the CFM is very uniform over the stockpile. Figure 6.3 shows less than 5% of the CFM is larger than 40 mm (1.57 in.), and no particles are larger than 75 mm (3 in.). However, 28% of the CFM material passes the No. 40 sieve (0.425 mm) and 10% passes the No. 200 sieve (0.075 mm), indicating significantly more fines than the CC and RAP materials. Nevertheless, the CFM still meets the TxDOT backfill specification for MSE walls. The USCS classification for this material is poorly graded gravel (GP).

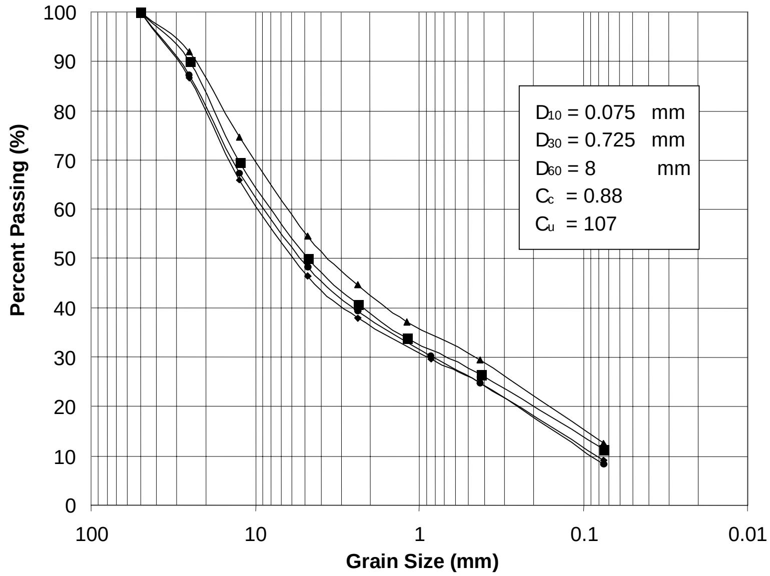


Figure 6.3 Grain size distribution of the CFM

6.1.4 Proposed Target Gradation

Because of the different gradations of the stockpiled materials, a single reference gradation was proposed (Figure 6.4). The material for test specimens for subsequent tests will be mixed to match this reference gradation. Using a single reference gradation will eliminate the effect of grain size distribution on future test results, thereby allowing tests to concentrate on the effects of the composition of the different materials. Figure 6.4 shows that the proposed reference gradation limits the maximum particle size to 50 mm (2 in.). The reference gradation also limits the material passing the No. 40 sieve to 7% and allows no fines passing the No. 200 sieve.

Compared with the measured gradations of the test materials, the proposed reference gradation is similar to the RAP gradation. However, the reference gradation is somewhat different than those for the CC and CFM. The CC measured gradation deviates from the reference gradation in the particle size range of 0.4 to 10 mm. The CFM gradation indicates more smaller particles than the reference gradation. Moreover, CFM contains approximately 10% fines (smaller than the No. 200 sieve), significantly more than the other materials and the reference gradation. When constructing CFM test specimens, the fines will be discarded to conform with the proposed reference gradation.

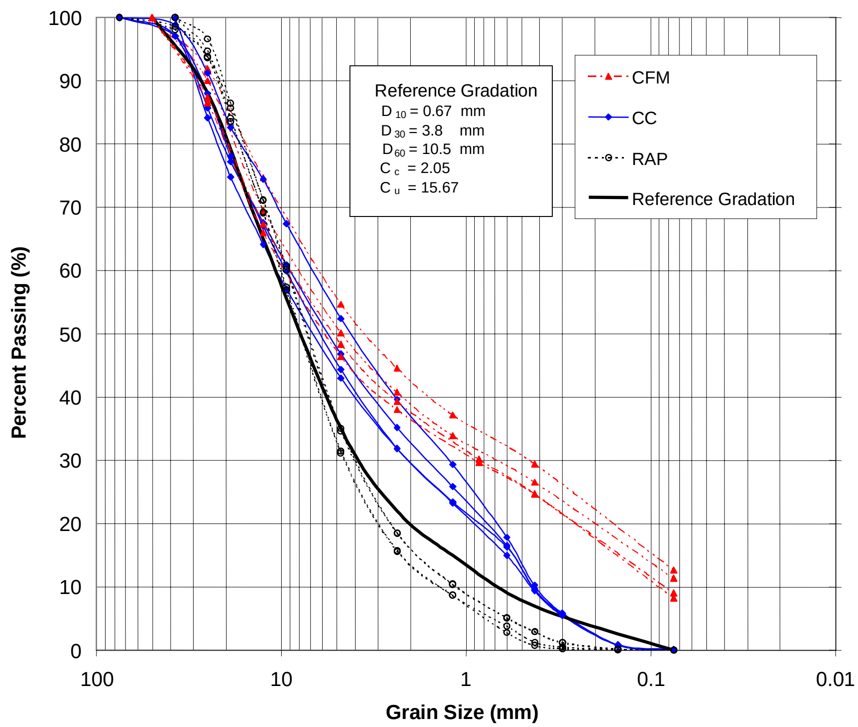


Figure 6.4 Proposed reference gradation for all testing materials

6.2 SPECIFIC GRAVITY

Specific gravity (Gs) is defined as the ratio of the mass of a volume of solid particles to the mass of an equal volume of water. The specific gravity of particles larger than the No. 4 sieve (4.75 mm) was measured using test method ASTM C127, and the specific gravity of particles smaller than the No. 4 sieve (4.75 mm) was measured using test method ASTM D854.

The ASTM C127 standard involves measuring the weight of a sample of dry material in air and measuring the weight of the same material in the fully water-submerged condition. The resulting parameter is called the "apparent specific gravity" because it represents the specific gravity of the impermeable part of solid particles and does not account for any entrapped air voids. The apparent specific gravity is calculated as follows:

$$\text{Apparent Specific Gravity} = G_s = \frac{A}{(A-C)} = \frac{V\gamma_{dry}}{V\gamma_w} = \frac{VG_s\gamma_w}{V\gamma_w} \qquad (1)$$

where: A = weight of oven-dry test sample in air

C = weight of saturated test sample in water

A – C = $V\gamma_w$ = weight of equal volume of water

For particles smaller than the No. 4 sieve, the ASTM D854 test method is used. In this method, the specific gravity is still calculated as the ratio of the mass in air of a given volume of soil to the mass of an equal volume of water. However, the mass of the equal volume of water is measured by considering the mass of a pycnometer flask filled completely with water and the same pycnometer flask filled with soil and water. The specific gravity is calculated with the following equation:

$$G_s = \frac{M_s}{M_s + M_{pw} - M_{pws}} \qquad (2)$$

where: M_s = mass of oven-dried soil

M_{pw} = mass of pycnometer filled with water

M_{pws} = mass of pycnometer filled with water and soil

To attain a proper specific gravity that represents the entire material, a weighted average of the specific gravities measured by ASTM C127 and ASTM D854 is calculated. This weighted average is calculated by weighing each specific gravity by the percent of large and small particles, as shown in Equation 3.

$$(G_s)_{avg} = \frac{1}{\frac{P_1}{100G_1} + \frac{P_2}{100G_2}} \qquad (3)$$

where; P_n = percentage by weight of each size fraction

G_n = appropriate specific gravity of each size fraction

Table 6.1 shows the specific gravity of all three materials obtained from this approach. Table 6.1 indicates that the large and small particles have very similar values of specific gravity. Additionally, Table 6.1 shows that CC and CFM have similar values of specific gravity. However, the specific gravity of RAP is significantly smaller. The lower value of specific gravity for RAP is most likely due to the bitumen coating around particles that creates a larger impermeable volume of solids and, thus, results in a smaller specific gravity.

Table 6.1. Specific gravity of test materials

Material	Specific Gravity		
	> No. 4 sieve*	< No.4 sieve**	$(G_s)_{avg}$
CC	2.62	2.62	2.62
RAP	2.36	2.28	2.33
CFM	2.64	2.69	2.66

Note: * Apparent Specific Gravity of Coarse Materials from ASTM C127
** Specific Gravity of Fine Materials from ASTM D854

6.3 COMPACTION CHARACTERISTICS

Compaction characteristics and moisture-density relationships for each material were measured in accordance with the Tex-113-E test method. In this test, soil is compacted in a compaction mold (6-in. diameter by 8-in. high) using a 10 lb hammer with a sector face dropped from a height of 18 in. Compaction is performed in 4 layers, with 50 blows of the hammer applied to each layer. With this compaction procedure, the sample receives a compaction energy of 22,900 ft-lb/ft^3. After the material is compacted in the mold, the sample is weighed, extruded, and dried in the oven. The total weight and total volume are used to calculate total unit weight, and water content is calculated from the weight of the oven-dried material. The total unit weight and water content are used to calculate dry unit weight. The calculated dry unit weight and water content represent a single data point on the dry density–water content relationship. Compacting material at various water contents is required to obtain the complete compaction curve.

A compaction curve for each material was established using Tex-113-E. For each material, stockpile samples were screened and mixed to match the reference gradation before compaction testing. The compaction curves are shown in Figures 6.5 to 6.7. The zero air voids curve, representing 100% saturation, for the specific gravity of each material is shown on each plot. Each compaction curve is discussed below.

The compaction curve in Figure 6.5 for CC shows that the dry density increases as water content increases from 0 to 12%. However, the dry density remains constant for water contents greater than 12%. CC specimens could not be constructed at water contents greater than about 14% because excess water drained out of the compaction mold. The CC compaction curve does not exhibit the distinct peak that is typical for soil. However, compaction curves for clean gravel soils typically do not exhibit a peak because their dry density is not significantly sensitive to water content (Lambe and Whitman 1979). Based on the compaction curve in Figure 6.5, a water content of 10% and the corresponding dry unit weight of 119 pcf are recommended for future testing (e.g., shear strength testing and corrosion testing). A water content of 10% corresponds to approximately 70% saturation. This value of water content was chosen because it represents a value that is easy to handle in the laboratory, while still yielding a high dry density.

The compaction curve for RAP (Figure 6.6) shows a slight peak in dry density at 3% water content. However, the dry density values at larger water contents are not significantly smaller than the peak, and at 7% water content the dry density starts to increase again. Similar to CC, specimens at larger water contents could not be formed because of loss of water through the base of the compaction mold. The largest value of dry density occurs at a smaller water content for RAP because of the influence of the bitumen coating on the particles. Based on this compaction curve, a water content of 3% and a corresponding dry unit weight of 117 pcf are recommended for future testing. This water content corresponds to about 30% saturation. Although this saturation level is lower than for CC, it was necessary to choose a lower water content because the more pervious RAP samples were difficult to construct at moderate to large water contents.

The compaction curve for the CFM (Figure 6.7) is similar to the compaction curve for CC. The dry density increases with increasing water content until a water content of about 11% is reached. All of the compaction tests at 11% water content resulted in dry densities that fell close to the zero air voids curve, indicating 100% saturation. Based on this compaction curve, a water content of 10% and a corresponding dry unit weight of

125 pcf are recommended for future testing. This water content corresponds to about 80% saturation.

Table 6.2 summarizes the recommended water contents and expected dry densities for compacting specimens for future testing.

Table 6.2. Recommended specimen water content and dry density

Material	Water Content (%)	Dry Density (lb/ft^3)
CC	10	119
RAP	3	117
CFM	10	125

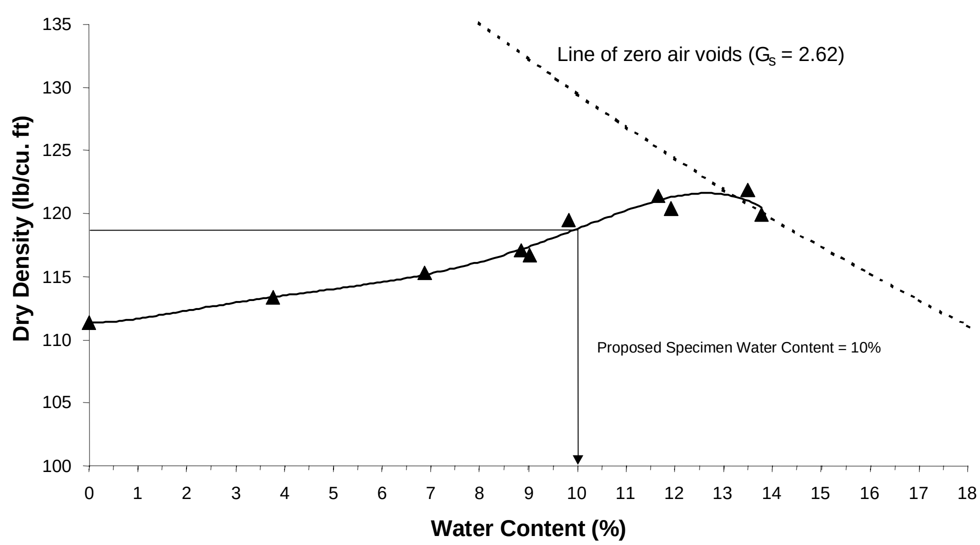


Figure 6.5 Compaction curve for CC based on Tex-113-E test method

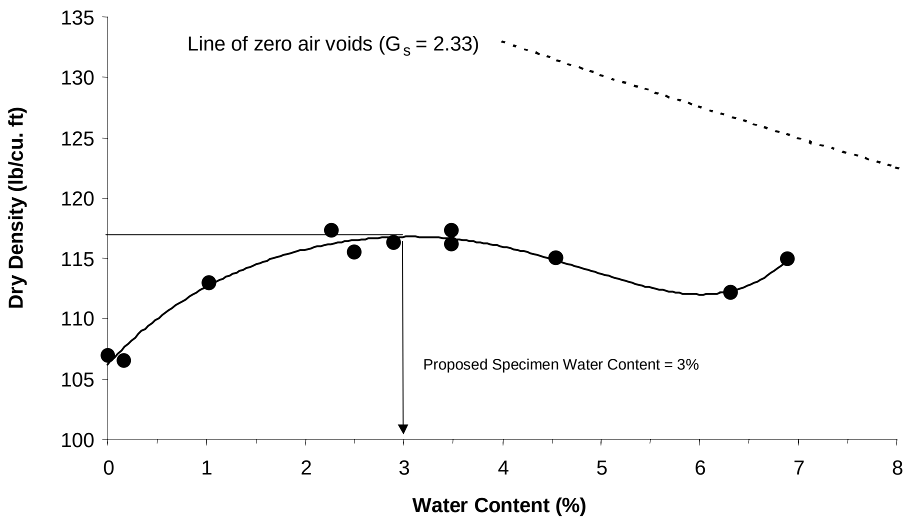


Figure 6.6 Compaction curve for RAP based on Tex-113-E test method

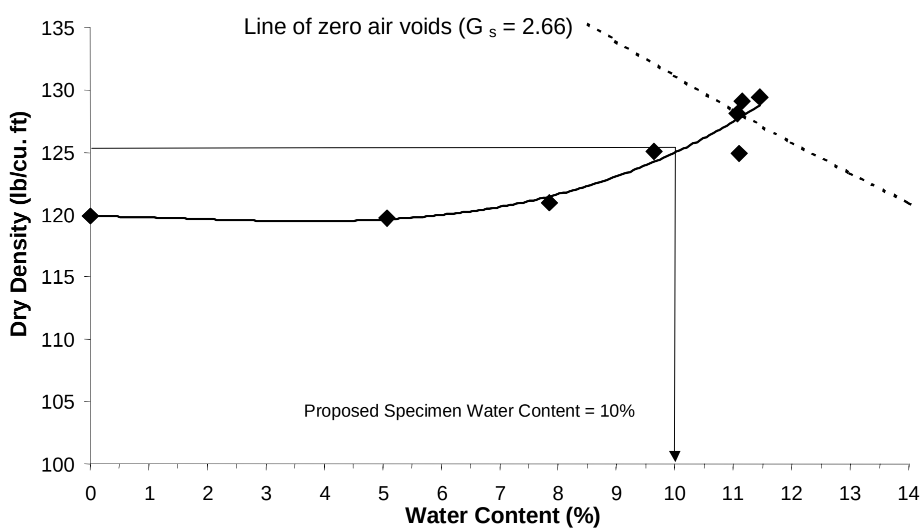


Figure 6.7 Compaction curve for CFM based on the Tex-113-E test method

6.4 SAMPLE PREPARATION PROCEDURE

This section describes the method of sample preparation for subsequent testing. When compacting test specimens, it is important to apply the same compactive effort as in the Tex-113-E compaction test (22,900 ft-lb/ft^3). The compaction energy applied to a volume of soil is calculated as follows:

$$\text{Compaction Energy} = \frac{(hammer\ \ weight)(drop\ \ height)(\#layers)(\#blows/layer)}{(total\ \ volume\ \ of\ \ \ specimen)} \quad (4)$$

For the Tex-113-E compaction method, equation (4) is calculated using:

Hammer weight = 10 lb

Drop height = 18 in. = 1.5 ft

Number of layers = 4

Number of blows per layer = 50

Total volume of specimen = 0.13 ft^3 (6-in. diameter, 8-in. height)

These values result in a compaction energy of 22,900 ft-lb/ft^3.

Equation (4) indicates that compaction energy is expressed in terms of applied impact energy per unit volume of sample. If specimens of different volume are desired, the impact energy must be modified to achieve the same level of impact energy per unit volume. For example, a 4 in. diameter triaxial test specimen that is 8 in. high will require fewer hammer impacts. The total volume of the 4-in. diameter triaxial specimen is 0.058 ft^3, which will require an impact energy of 1,328 ft-lb to achieve a compaction energy of 22,900 ft-lb/ft^3. If the specimen is compacted in 8 1-in. layers with a 10 lb hammer dropped from 1.5 ft, 11 blows per layer are necessary to achieve the correct impact energy. This same approach can be used to develop compaction procedures for preparing samples for future testing in this project.

6.5 SUMMARY

This chapter discusses the basic geotechnical characteristics of the bulk samples of CC, RAP, and CFM, including gradation, specific gravity, and compaction. The CFM was included in the testing program to compare with CC and RAP.

Section 6.1 discusses the grain size distribution of the test materials in the actual stockpiles. The CC and RAP have similar gradations, but the CFM has a significant amount of fines passing the No. 200 sieve. A reference gradation is proposed for future testing to eliminate the effect of grain size distribution on test results. All future test specimens will be formed by screening the test materials and re-mixing them to match the reference gradation.

Section 6.2 discusses the specific gravity of each test material. The CC and CFM have specific gravities between 2.60 and 2.65, values common for soil. The specific gravity of RAP is about 2.30, which is lower than typical for soil. The smaller value of specific gravity is due to the bitumen coating on the aggregate particles.

The compaction characteristics of each material are described in Section 6.3. Results from the compaction tests indicate that the dry density of these materials is not very sensitive to water content. The recommended values of water content and dry density for subsequent test specimens are specified in Section 6.3. In Section 6.4, a procedure for developing compaction methods for the construction of different-sized test specimens with the same compaction energy as test method Tex-113-E is described. This procedure will be used to compact test specimens for all future testing.

Chapter 7: Summary and Conclusions

7.1 SUMMARY

MSE walls are common highway structures in Texas and throughout the United States. These earth retaining structures are made up of facing panels, reinforcement elements, and high quality granular backfill. In areas where high quality granular backfill is not available, recycled materials such as RAP and CC may provide cost-effective alternatives.

This research report focuses on a state-of-the-art review of issues related to the use of RAP and CC as backfill in MSE walls, and the initial material characterization of RAP and CC. The state-of-the-art review includes discussions regarding the design of MSE walls (Chapter 2), current TxDOT and FHWA backfill specifications for MSE walls (Chapter 3), issues related to the application of current backfill specifications to RAP and CC (Chapter 3), and current transportation-related uses of RAP and CC (Chapter 4). Additionally, basic material characterization of RAP and CC samples from throughout the state was performed (Chapter 5). This characterization was used to select suppliers of bulk RAP and CC samples. A supplier of a conventional fill material (CFM) was also chosen. Bulk samples of RAP, CC, and CFM will be used in the future testing that will take place as part of this research project. Finally, the basic material characteristics (i.e., gradation, specific gravity, and compaction characteristics) of the bulk RAP, CC, and CFM samples were evaluated (Chapter 6).

7.2 STATE-OF-THE-ART REVIEW

The state-of-the-art review identified the important backfill properties for MSE walls and described the engineering issues related to using RAP or CC as backfill for MSE walls. The critical backfill properties were identified as hydraulic conductivity, shear strength, interface friction between the backfill and reinforcement, compaction characteristics, compressibility, creep potential, and corrosivity.

There are several specific geotechnical issues related to using RAP and CC as backfill for MSE walls. These issues include:

1. The maximum particle size of RAP and CC.
2. The shape of the moisture-density compaction curve for RAP and CC.
3. Crushing of RAP and CC during compaction.
4. The hydraulic conductivity of RAP and CC.
5. Formation of precipitates in drainage structures from CC.
6. The shear strength of RAP and CC.
7. The potential for creep in RAP and at the RAP-reinforcement interface.

Based on the results from this state-of-the-art review, the most critical geotechnical issues are likely to be the hydraulic conductivity of CC and the creep potential of RAP.

The durability of the MSE wall reinforcement is a major concern. Polymeric reinforcement is typically processed to minimize long-term degradation, but the effects of RAP and CC on the degradation of polymeric reinforcement has not been studied. The potential corrosion of metallic reinforcement is also a significant concern. The characteristics of the soil and the metallic reinforcement both affect the potential for corrosion. However, the commonly used corrosivity parameters, pH and resistivity, may not be adequate in assessing the corrosion potential of RAP and CC backfill materials.

7.3 MATERIAL CHARACTERIZATION

Material characterization encompassed testing of samples from different suppliers throughout Texas and then choosing three suppliers of bulk material for further characterization. For the initial characterization, major producers of RAP and CC were asked to periodically sample their product over a two-month period. The gradation, Atterberg limits, specific gravity, pH, and resistivity were measured for each sample. The test results from the various suppliers were remarkably similar, indicating that RAP and CC materials are similar around the state. Therefore, future testing will take place on RAP and CC samples from a single supplier and the results should be applicable to RAP and CC throughout the state. The initial material characterization indicated that all of the RAP samples met the current TxDOT specifications for gradation, Atterberg limits, pH, and resistivity. However, the CC samples did not meet the specifications for pH and resistivity.

Bulk samples of RAP, CC, and a conventional fill material (CFM) were obtained for use in testing throughout the remainder of this three-year project. Basic characterization of these samples included gradation, specific gravity, and compaction characteristics. The gradations of the three materials all meet the current TxDOT specifications for MSE wall backfill, but the gradations are not similar in the small particle size range. Therefore, a reference gradation was proposed, and the material used to construct test specimens in subsequent tests will be mixed to match this reference gradation. The reference gradation will eliminate the effect of grain size distribution on future test results. The specific gravity of the CC and CFM were in the range of 2.60 to 2.65, but the specific gravity of RAP was significantly smaller (G_s = 2.33). Results from the compaction tests indicate that the dry density of RAP, CC, and CFM is not significantly sensitive to water content. Based on the compaction curves, values of water content were recommended for compacting future testing with a specified compaction energy.

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